GEOTECHNICAL FEASIBILITY STUDY
BELL MOUNTAIN COMMERCE CENTER (BMCC) – PHASE II – BUILDING 4

Northwest Corner of Stoddard Wells Road and Grasshopper Road
Apple Valley, California
for
Covington Group, Inc.

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

December 4, 2025

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Covington Group, Inc.
14180 Dallas Parkway, Suite 730
Dallas, Texas 75254

Attention: Mr. Brandon Gallup
Acquisitions & Development

Project No.: **25G195-1**

Subject: **Geotechnical Feasibility Study**
Bell Mountain Commerce Center (BMCC) – Phase II – Building 4
Northwest Corner of Stoddard Wells Road and Grasshopper Road
Apple Valley, California

Mr. Gallup:

In accordance with your request, we have conducted a geotechnical feasibility study 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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TABLE OF CONTENTS

1.0 EXECUTIVE SUMMARY

Presented below is a brief summary of the conclusions and recommendations of this investigation. Since this summary is not all inclusive, it should be read in complete context with the entire report. It should be noted that this investigation was focused on determining the geotechnical feasibility of the proposed development. **This report is not a design-level investigation. Future studies will be necessary to confirm and refine the preliminary design parameters that are presented within this report**.

Preliminary Geotechnical Design Considerations

- Based on the mapping performed by the county of San Bernardino and the lack of a historic high ground water table within the upper 50± feet of the ground surface, liquefaction is not considered to be a design concern for this project.
- Native younger and older alluvium was encountered at the ground surface at all of the boring locations. The native alluvium possesses varying strengths and densities. The results of laboratory testing indicate that the younger alluvial soils within the upper 7 to 8± feet possess a potential for moderate to severe collapse when exposed to moisture infiltration as well as moderate consolidation when exposed to load increases in the range of those that will be exerted by the new foundations.

Preliminary Geotechnical Design Recommendations

- Initial site stripping should include removal of the surficial vegetation from the site. Stripping should include native grass, weeds, shrubs and trees. These materials should be properly disposed of off-site.
- Demolition of any improvements that will not remain in place for use with the new development will be required at this site. Debris resultant from demolition should be disposed of off-site.
- Preliminarily, the existing soils within the building pad area should be overexcavated to depths of 7 to 8 feet below existing grades, and to depths of 4 to 5 feet below proposed pad grades, whichever is greater. In addition, all of the younger alluvium within the proposed building area should be overexcavated in their entirety. The soils within the proposed foundation influence zones should be overexcavated to a depth of at least 4 to 5 feet below proposed foundation bearing grades.
- The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill placed below the foundation bearing grade, whichever is greater. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.
- Following completion of the overexcavation, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. Materials suitable to serve as the structural fill subgrade within the building area should consist of native soils which possess an in-situ density equal to at least 85 percent of the ASTM D-1557 maximum dry density. After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture treated 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 previously excavated soils may then be replaced as compacted structural fill.

- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Preliminary Foundation Design Recommendations

- Conventional shallow foundations, supported in newly placed compacted structural fill.
- 2,500 to 3,000 lbs/ft^2 maximum allowable soil bearing pressure.
- Minimum recommended reinforcement based on geotechnical conditions is expected to consist of two (2) to four (4) No. 5 rebars (1 to 2 top and 1 to 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Preliminary Floor Slab Design Recommendations

- Conventional slab-on-grade, minimum 6 to 7 inches thick.
- Modulus of Subgrade Reaction: k = 100 to 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.

Preliminary Pavement Design Recommendations

ASPHALT PAVEMENTS (R = 40)					
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	4	6	7	8	10
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 40)				
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	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 25P386, dated October 31, 2025. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to determine the geotechnical feasibility of the proposed development. This report also contains preliminary design criteria for building foundations, building floor slab, and parking lot pavements. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical feasibility study.

It should be noted that additional subsurface exploration, laboratory testing and engineering analysis will be necessary to provide a design-level geotechnical investigation with specific foundations, floor slab, and grading recommendations.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The site is located at the northwest corner of Stoddard Wells Road and Grasshopper Road in Apple Valley, California. The site is bounded to the north by Johnson Road, to the west by the future alignment of Wrangler Road, to the south by Stoddard Wells Road, and to the east by Grasshopper Road and a vacant lot. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The site consists of several contiguous parcels, totaling 97± acres in size. The site is vacant and undeveloped. Several natural drainages transect the site in the north-south direction. Ground surface cover consists of exposed soil with sparse to moderate native grass and weed growth.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth, and visual observations made at the time of the subsurface investigation, the overall site topography slopes downward to the south at a gradient of 2± percent. There is 60± feet of elevation differential across the site.

3.2 Proposed Development

A preliminary site plan has been provided to our office by the client. Based on this plan, the subject site is a part of Phase II of the BMCC proposed development. Phase II will consist of one (1) industrial building (identified as Building 4), 1,365,000± ft² in size, located in the central area of the site. Dock-high doors will be constructed along portions of the east and west building walls. The building is expected to be surrounded by asphaltic concrete pavements in the parking and drive lanes, Portland cement concrete pavements in the truck dock areas, concrete flatwork, and limited areas of landscape planters throughout.

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

Grading plans for the proposed development were not available at the time of this report. The proposed development is not expected to include any significant amounts of below-grade construction such as basements or crawl spaces. Based on the existing topography, cuts and fills of at least 10 to 15± feet are expected to be necessary to achieve the proposed building pad grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The preliminary subsurface exploration conducted for this project consisted of four (4) borings (identified as Boring Nos. B-1, through B-4) advanced to depths of 22 to 26± below the existing site grades. It should be noted that Boring Nos. B-1, B-3, and B-4 were terminated at shallower depths than planned after encountering refusal on very dense to hard native older alluvium. The borings were logged during drilling by a member 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

Younger Alluvium

Native younger alluvium was encountered at the ground surface at Boring Nos. B-3 and B-4, extending to depths of 3½ to 5½± feet below the existing site grades. The younger alluvium consists of medium dense to very dense silty sands with varying fine gravel content. Some of the younger alluvial soils are slightly porous.

Older Alluvium

Native older alluvium was encountered at the ground surface at Boring Nos. B-1 and B-2, and beneath the younger alluvium at Boring Nos. B-3 and B-4, extending to at least the maximum depth explored of 26± feet below the existing site grades. The older alluvium generally consists of very dense silty sands and clayey sands with varying fine gravel content, and hard sandy clays. The older alluvium generally possesses weak to moderate cementation. Some of the near-surface older alluvial soils are also slightly porous.

Groundwater

Free water was not encountered during the drilling of any of the borings. Based on the lack of encountered water and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth of 26± feet at the time of the subsurface exploration.

As a part of our research, we reviewed available groundwater data in order to determine groundwater levels for the site. Water level data was obtained from the California Department of Water Resources Water Data Library website, https://wdl.water.ca.gov/waterdatalibrary/. The nearest monitoring well on record (identified as State Well Number:c06N04W24J001S) is located 750± feet south of the project site center. Water level readings within this monitoring well indicate a high groundwater level of 153± feet below the ground surface in October 1950.

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

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

Density and Moisture Content

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

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch-high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-3 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative soil sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date. The result of this testing is plotted on Plate C-4 in Appendix C of this report.

Soluble Sulfates

A representative sample of the near-surface soils 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 (%)	Severity	Class
B-4 @ 1 to 5 feet	0.011	Not Applicable	S0

Corrosivity Testing

A representative sample of the near-surface soils was submitted to a subcontracted corrosion engineering laboratory for determination of electrical resistivity, pH, and chloride concentrations. The resistivity of the soils is a measure of their potential to attack buried metal improvements such as utility lines. The results of some of these tests are presented below.

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)	Sulfides (mg/kg)	Redox Potential (mV)
B-4 @ 1 to 5 feet	6,566	8.7	76.3	3.7	0.60	140

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing, and geotechnical analysis, the proposed development, which consists of one (1) industrial building, is considered feasible from a geotechnical standpoint. **Based on the preliminary nature of this investigation, further geotechnical investigation(s) will be required prior to construction of the proposed development.** The recommendations contained in this report should be taken into the design, construction, and grading considerations. 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 structure should, however, be designed to resist structural collapse and thereby provide reasonable protection from serious injury, catastrophic property damage and loss of life.

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. 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 2025 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.

The 2025 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 based on the project site class in accordance with several building code reference documents, including ASCE 7-22, upon which the 2025 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 table below was created using data obtained from the application. The output generated from this program is included as Plate E-1 in Appendix E of this report.

Chapter 20 of ASCE 7-22 indicates that the site soil shall be classified based on the average shear wave velocity parameter which is derived from the measured shear wave velocity profile from the ground surface to a depth of 100 feet. Furthermore, Section 20.3 of ASCE 7-22 indicates that where shear wave velocity data does not extend to depths of 50 feet, default site classes shall be used. Based on the preliminary subsurface exploration performed for this project, a Default Site Class is considered appropriate for this project. It is recommended that the future design-level geotechnical investigation includes a shear wave velocity to a depth of at least of 100 feet below the existing site grades to determine the actual Site Class for this project.

2025 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.150
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.390
Site Class	---	Default
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.510
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.010
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.010
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.670

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 grain size 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). Clayey (cohesive) soils or soils which possess clay particles (d<0.005mm) in excess of 20 percent (Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The California Geological Survey (CGS) has not yet conducted detailed seismic hazards mapping in the area of the subject site. The general liquefaction susceptibility of the site was determined by research of the San Bernardino County Land Use Plan, General Plan, Geologic Hazard Overlays.

Map of the EH31 C for the Apple Valley North 7.5-Minute Quadrangle indicates that the subject site is not located within an area of liquefaction susceptibility. Based on the mapping performed by the county of San Bernardino, the presence of dense to very dense soils, and the lack of a historic high ground water table within the upper 50± feet of the ground surface, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

Native younger and older alluvium was encountered at the ground surface at all of the boring locations. The native alluvium possesses varying strengths and densities. The results of laboratory testing indicate that the near-surface younger alluvial soils within the upper 7 to 8± feet possess a potential for moderate to severe collapse when exposed to moisture infiltration as well as moderate consolidation when exposed to load increases in the range of those that will be exerted by the new foundations. Therefore, remedial grading is considered warranted within the proposed building area in order to remove and replace the collapsible native alluvial soils as compacted structural fill.

We recommend that a supplemental geotechnical investigation be performed for the proposed development, in order to more completely characterize the subsurface conditions and confirm the suitability of the design recommendations provided in this report.

Settlement

The proposed remedial grading will remove the existing undocumented fill soils and a portion of the near-surface native alluvial soils from within the proposed building area, and replace these materials as compacted structural fill. The native soils that will remain in place beneath the recommended depth of overexcavation will not be subject to significant stress increases from the foundations of the new structure. Therefore, following completion of the recommended remedial grading, post-construction static settlements are expected to be within tolerable limits.

Expansion

The near-surface soils consist of silty sands with occasional clayey sands. These materials have been visually classified as non-expansive. Therefore, no design considerations related to expansive soils are considered warranted for this site. **It is recommended that additional expansion index testing be conducted during the subsequent design-level geotechnical investigation and at the completion of rough grading to verify the expansion potential of the as-graded building pad.**

Slope Stability

No evidence of landslides or deep-seated slope instability was noted during our investigation. However, loose granular soils on sloping ground surfaces could be prone to surficial failures.

Newly constructed fill slopes, comprised of properly compacted engineered fill, at inclinations of 2h:1v (horizontal to vertical) will possess adequate gross stability. Cut slopes excavated within the existing granular alluvial soils may be subject to surficial instability due to the lack of cohesion within these materials. Therefore, stability fills may be required within these areas. This condition may affect the proposed cut slopes at the site. The need for stability fills should be determined by SCG as part of the future design-level geotechnical investigation.

Soluble Sulfates

The results of the soluble sulfate testing, discussed in Section 5.0 of this report, indicate soluble sulfate concentrations of up to 0.011 percent. These concentrations are considered to be negligible or "not applicable" with respect to the American Concrete Institute (ACI) Code-318-19 Building Code for Structural Concrete – Code Requirements and Commentary, Chapter 19. 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 structure areas.

Corrosion Potential

The results of laboratory testing indicate that the tested sample of the near-surface soils possesses a saturated resistivity of 6,566 ohm-cm, and a pH value of 8.7. The soils possess a redox potential of 140 mV and a sulfide concentration of 0.60 mg/kg. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity, pH, sulfide concentration, redox potential, and moisture content are the five factors that enter into the evaluation procedure. Based on these factors, the on-site soils are considered to be mildly corrosive to ferrous pipes. Therefore, corrosion protection may 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 76.3 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 3.7 mg/kg. Based on the test results, 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. It is recommended that additional testing be conducted during the design-level geotechnical investigation.

Shrinkage/Subsidence

Removal and recompaction of the near-surface alluvial soils is estimated to result in an average shrinkage of 5 to 15 percent. However, potential shrinkage for individual samples ranged locally between 4 and 22 percent. The potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

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

These estimates are based on previous experience and the subsurface conditions encountered at the boring locations. The actual amount of subsidence is expected to be variable and will be dependent on the type of machinery used, repetitions of use, and dynamic effects, all of which are difficult to assess precisely.

Grading and Foundation Plan Review

Grading and foundation plans were unavailable at the time of this report. It is therefore recommended that we be provided with copies of the preliminary grading and foundation plans, if available, for the design-level geotechnical investigation.

6.3 Preliminary Site Grading Recommendations

The preliminary grading recommendations presented below are based on the design details that were available at the time of this report, and the subsurface conditions encountered at our boring locations. These recommendations are general and preliminary in nature, and should be confirmed as part of the future design-level geotechnical investigation.

Site Stripping and Demolition

Initial site stripping should include removal of the surficial vegetation from the site. Stripping should include native grass, weeds, shrubs and trees. Root systems associated with trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils. These materials should be properly 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.

Demolition of any improvements that will not remain in place for use with the new development will be required at this site. Debris resultant from demolition should be disposed of off-site. All applicable federal, state and local specifications and regulations should be followed in demolition, abandonment, and disposal of the resulting debris.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building area in order to remove the compressible/collapsible alluvial soils. Preliminarily, the existing soils within the building pad area should be overexcavated to depths of 7 to 8 feet below existing grades, and to depths of 4 to 5 feet below proposed pad grades, whichever is greater. In addition, all of the younger alluvium within the proposed building area should be overexcavated in their entirety. The soils within the proposed foundation influence zones should be overexcavated to a depth of at least 4 to 5 feet below proposed foundation bearing grades.

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

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

Materials suitable to serve as the structural fill subgrade within the building area should consist of native soils which possess an in-situ density equal to at least 85 percent of the ASTM D-1557 maximum dry density. After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture treated 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 previously excavated soils may then be replaced as compacted structural fill.

Deep Fill Areas

In order to reduce the settlement potential of the newly placed fill soils to acceptable levels and avoid excessive differential settlements, fill soils placed at depths greater than 10 feet below proposed building pad grades should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density.

Settlement of Deep Fill Soils

Additional consolidation may occur for fill soils placed at depths greater than 10 feet below proposed building pad grades. The primary settlement associated with these fill soils is expected to occur relatively quickly due to the generally granular nature of the on-site soils. Minor amounts of additional settlement may occur due to secondary consolidation effects. The extent of secondary consolidation is difficult to assess precisely, and will be reduced by the proposed mitigation measures recommended herein, but may be in the range of 0.2 to 0.4 percent of the fill thickness. Based on the expected differential fill thickness that will exist across the building footprints, the structural design will need to consider the distortions that could be caused by the secondary consolidation of the fill soils. Provided that the grading and foundation design recommendations presented in this report are implemented, these settlements are expected to be within the structural tolerances of the proposed building.

Treatment of Existing Soils: Cut and Fill Slopes

New cut and fill slopes may be required to stablish the proposed site grades. All slopes should be at an inclination not to exceed 2h:1v. A keyway should be excavated at the toe of new fill slopes which are not located in fill areas. The keyway should be at least 15 feet wide and 3 feet deep. The recommended width of the keyway is based on 1.5 times the width of typical grading equipment. If smaller equipment is utilized, a smaller keyway may be suitable, at the discretion of the geotechnical engineer. The base of the keyway should slope at least 1 foot downward into the slope. Following completion of the keyway cut, the subgrade soils should be evaluated by the geotechnical engineer to verify that the keyway is founded into competent materials. The resulting subgrade soils should then be scarified to a depth of 10 to 12 inches, moisture conditioned to 0 to 4 percent above optimum moisture content and recompacted. During construction of the new fill slope, the existing slope should be benched in accordance with the detail presented on Plate D-4. Benches less than 4 feet in height may be used at the discretion of the geotechnical engineer.

Stability fills for cut slopes will provide a more uniform appearance and allow landscaping on the slope. Should a stability fill for cut slope be necessary, the recommendations for the stability fill will be the same as the recommendations for the fill slopes, mentioned above.

Treatment of Existing Soils: Retaining Walls and Site Walls

Although not indicated on the site plan, it may be necessary to construct some small retaining walls or site walls at or near the existing ground surface. Overexcavation will also be necessary in these areas to remove any variable strength alluvium. The overexcavation depth should be expected to be on the order of 3 to 5 feet below proposed foundation bearing grade, and to depths of 3 to 5 feet below existing grade. Any undocumented fill soils or disturbed native alluvium within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend 3 to 5 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Any erection pads for tilt-up concrete walls are considered to be part of the foundation system. Therefore, these overexcavation recommendations are applicable to erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to within 0 to 4 percent above the optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

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

Treatment of Existing Soils: Flatwork, Parking and Drive Areas

Based on economic considerations, overexcavation of the existing low to moderate strength near-surface 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 0 to 4 percent above the optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength surficial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

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

Treatment of Existing Soils: Infiltration Systems

SCG performed a concurrent infiltration study at the subject site, referenced below:

> Results of Infiltration Testing, Bell Mountain Commerce Center (BMCC) – Phase II – Building 4, Northwest Corner of Stoddard Wells Road and Grasshopper Road, Apple Valley, California, prepared by SCG for Covington Group, Inc., SCG Project No. 25G195-2, dated December 3, 2025.

We understand that proposed infiltration system(s) will be included as part of the on-site improvements. Detailed infiltration recommendations will be provided in the concurrent infiltration testing results report. We recommend that scrapers and other rubber-tired heavy equipment not be operated at the bottom of the infiltration system(s), or at levels lower than 2± feet above the bottom of the infiltration system. Therefore, the bottom 2± feet of the infiltration system(s) should be excavated with non-rubber-tired equipment, such as excavators, to reduce compaction of the native soils at the bottom of the infiltration system(s).

If heavy equipment is operated within the bottom 2± feet of the infiltration system(s), excessive compaction of the native soils at the bottom of the infiltration system(s) will likely occur. In this case, mitigation measures will be required. Mitigation could include but not be limited to, deeper removals, dry wells, scarification, and possibly redesign of the infiltration system(s).

We recommend that a representative from the geotechnical engineer be on-site during the construction of the proposed infiltration system(s) to identify the soil classification at the bottom of the infiltration system(s) and assess if the native soils have been affected by construction equipment. The infiltration rate of the system will likely vary significantly if the composition or density of the soil located beneath the infiltration system(s) is not consistent with the tested soils.

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 2025 CBC and the grading code of the city of Apple Valley and/or the county of San Bernardino.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils placed at depths greater than 10 feet below proposed building pad grades should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of very low to non-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, utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Apple Valley and/or the county of San Bernardino. Utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

Any soils used to backfill voids around subsurface utility structures, such as manholes or vaults, should be placed as compacted structural fill. If it is not practical to place compacted fill in these areas, then such void spaces may be backfilled with lean concrete slurry. Uncompacted pea gravel or sand is not recommended for backfilling these voids since these materials have a potential to settle and thereby cause distress of pavements placed around these subterranean structures.

6.4 Preliminary Construction Considerations

Excavation Considerations

The near-surface soils generally consist of moderate strength silty sands and clayey sands. Some of these materials may be subject to minor to moderate caving within shallow excavations. Where caving does occur, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, the inclination of temporary slopes should not exceed 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

Based on their granular composition, the on-site soils are susceptible to erosion. The site should, therefore, be graded to prevent ponding of surface water and to prevent water from running into excavations.

Groundwater

The static groundwater table is considered to have existed at a depth in excess of 26± feet at the time of the subsurface exploration. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Preliminary Foundation Design Recommendations

Based on the preceding geotechnical design considerations and preliminary grading recommendations, it is assumed that the new building will be underlain by newly placed structural fill soils, extending to depths of at least 4 to 5 feet below foundation bearing grades. Based on this subsurface profile, the proposed structure may be supported on conventional shallow foundations.

The foundation design parameters presented below provide anticipated ranges for the allowable soil bearing pressures. These ranges should be refined during the subsequent design-level geotechnical investigation.

Preliminary Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 to 3,000 lbs/ft^2.

- Minimum longitudinal steel reinforcement within strip footings: Two (2) to four (4) No. 5 rebars (1 to 2 top and 1 to 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

General Foundation Design Recommendations

The allowable bearing pressures presented above may be increased by one-third when considering short duration wind or seismic loads. Additional reinforcement may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Estimated Foundation Settlements

Typically, foundations designed in accordance with the preliminary foundation design parameters presented above will experience total and differential static settlements of less than 1.0 and 0.5 inches, respectively. A detailed settlement analysis should be conducted as part of the design-level geotechnical investigation, once detailed foundation loading information is available.

Lateral Load Resistance

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

- Passive Earth Pressure: 275 to 350 lbs/ft^3
- Friction Coefficient: 0.28 to 0.35

6.6 Preliminary Floor Slab Design and Construction

Subgrades which will support the new floor slab should be prepared in accordance with the preliminary recommendations contained in the ***Preliminary Site Grading Recommendations*** section of this report with any additional recommendations provided in the design-level geotechnical report. Preliminarily, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 to 7 inches.

- Modulus of Subgrade Reaction: k = 100 to 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 floor slab coverings are anticipated. 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 design of the floor slab will depend on the results of a future design-level geotechnical study. The actual thickness and reinforcement of the floor slab should be determined by the structural engineer.

6.7 Preliminary Retaining Wall Design and Construction

Small retaining walls are expected to be necessary in the dock-high area of the building and may also be required to facilitate the new site grades. Preliminary design parameters recommended for use in the design of these walls are presented below. These recommendations should be refined during the design-level geotechnical investigation.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. The following parameters assume that only the on-site soils will be utilized for retaining wall backfill. The near-surface soils generally consist of silty sands and clayey sands. Based on their classification, the on-site soils are expected to possess a friction angle of at least 30 degrees when compacted to at least 90 percent of the ASTM D-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.

PRELIMINARY RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type
		On-site Silty Sands and Sandy Silts
Internal Friction Angle (ϕ)		30°
Unit Weight		135 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	45 lbs/ft^3
	Active Condition (2h:1v backfill)	73 lbs/ft^3
	At-Rest Condition (level backfill)	68 lbs/ft^3

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.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to depths of 3 to 5 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Preliminary Foundation Design Parameters presented in a previous section of this report.

Seismic Lateral Earth Pressures

In addition to the lateral earth pressures presented in the previous section, retaining walls which are more than 6 feet in height should be designed for a seismic lateral earth pressure, in accordance with the 2025 CBC. Based on the current site plan, it is not expected that any walls in excess of 6 feet in height will be required for this project. If any such walls are proposed, our office should be contacted for supplementary design recommendations.

Backfill Material

On-site soils may be used to backfill the retaining walls, provided that they are very low expansive (EI < 20). All backfill material placed within 3 feet of the back wall-face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

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

All retaining wall backfill should be placed and compacted under engineering controlled conditions in the necessary layer thicknesses to ensure an in-place density between 90 and 93 percent of the maximum dry density as determined by the Modified Proctor test (ASTM D1557-91). Care should be taken to avoid over-compaction of the soils behind the retaining walls, and the use of heavy compaction equipment should be avoided.

Subsurface Drainage

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

- A weep hole drainage system typically consisting of a series of 2-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 10-foot on-center spacing. Alternatively, 4-inch diameter holes at an approximate 20-foot on-center spacing can be used for this type of drainage system. In addition, the weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.

- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system. The actual design of this type of system should be determined by the civil engineer to verify that the drainage system possesses the adequate capacity and slope for its intended use.

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

6.9 Preliminary Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of clayey sands and silty sands. These soils are generally considered to possess good to excellent pavement support characteristics, with R-values in the range of 40 to 60. The subsequent preliminary pavement design is therefore based upon an assumed R-value of 40. 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 during the design-level geotechnical investigation, or at the completion of rough grading to verify that the pavement design recommendations presented herein are valid.

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.

<table>
<tr><th colspan="6">ASPHALT PAVEMENTS (R=40)</th></tr>
<tr><th rowspan="3">Materials</th><th colspan="5">Thickness (inches)</th></tr>
<tr><td rowspan="2">Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)</td><td colspan="4">Truck Traffic</td></tr>
<tr><td>TI = 6.0</td><td>TI = 7.0</td><td>TI = 8.0</td><td>TI = 9.0</td></tr>
<tr><td>Asphalt Concrete</td><td>3</td><td>3½</td><td>4</td><td>5</td><td>5½</td></tr>
<tr><td>Aggregate Base</td><td>4</td><td>6</td><td>7</td><td>8</td><td>10</td></tr>
<tr><td>Compacted Subgrade</td><td>12</td><td>12</td><td>12</td><td>12</td><td>12</td></tr>
</table>

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 batch plant-reported maximum density. 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 = 40)				
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	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

7.0 GENERAL COMMENTS

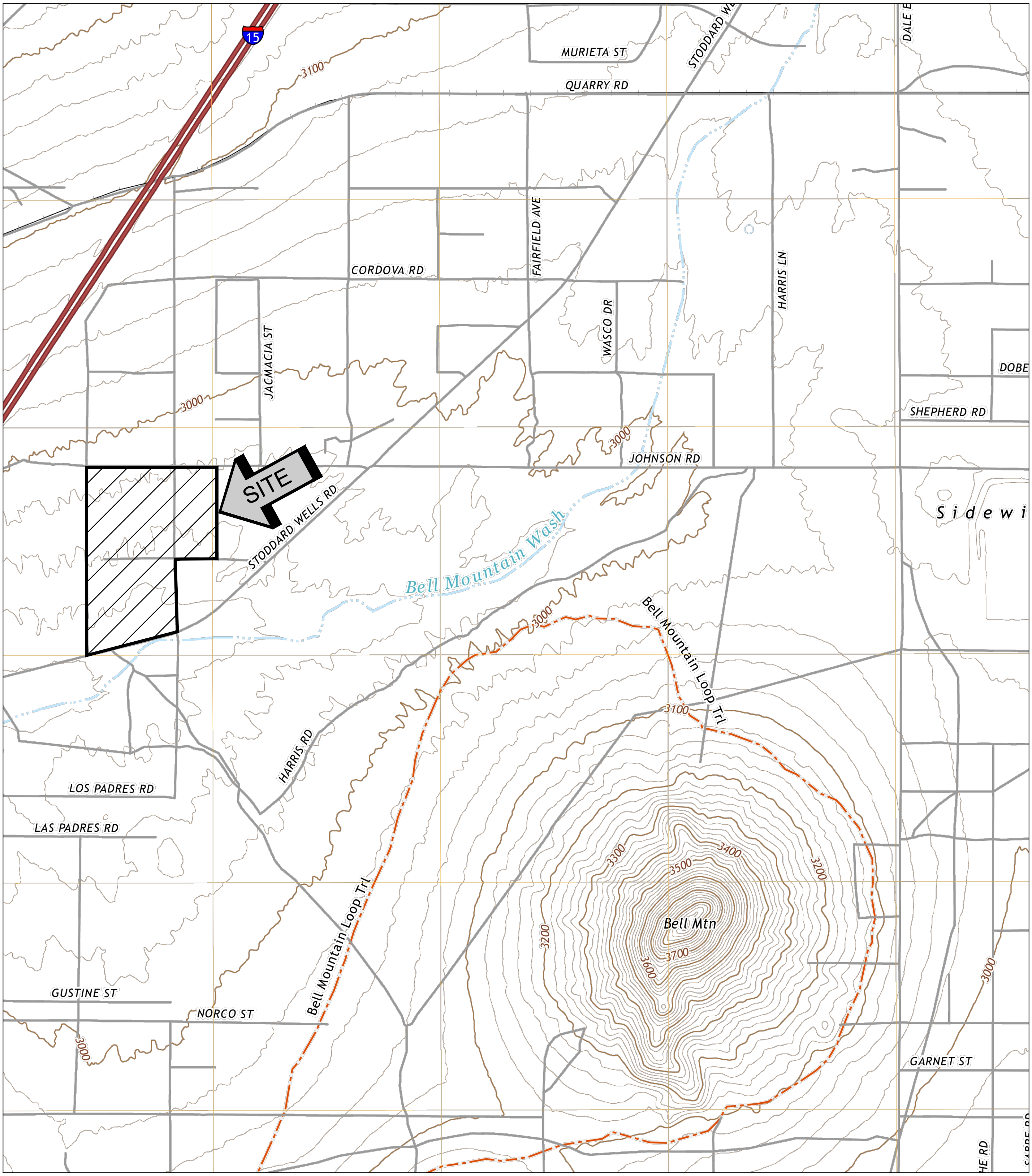
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other 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 boring locations and sample depths. If the conditions encountered during construction vary significantly from those detailed herein, we should be contacted immediately to determine if the conditions alter the recommendations contained herein.

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

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

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE APPLE VALLEY NORTH QUADRANGLE, SAN BERNARDINO COUNTY, CALIFORNIA, 2021.

SITE LOCATION MAP

BMCC - PHASE II - BUILDING 4

APPLE VALLEY, CALIFORNIA

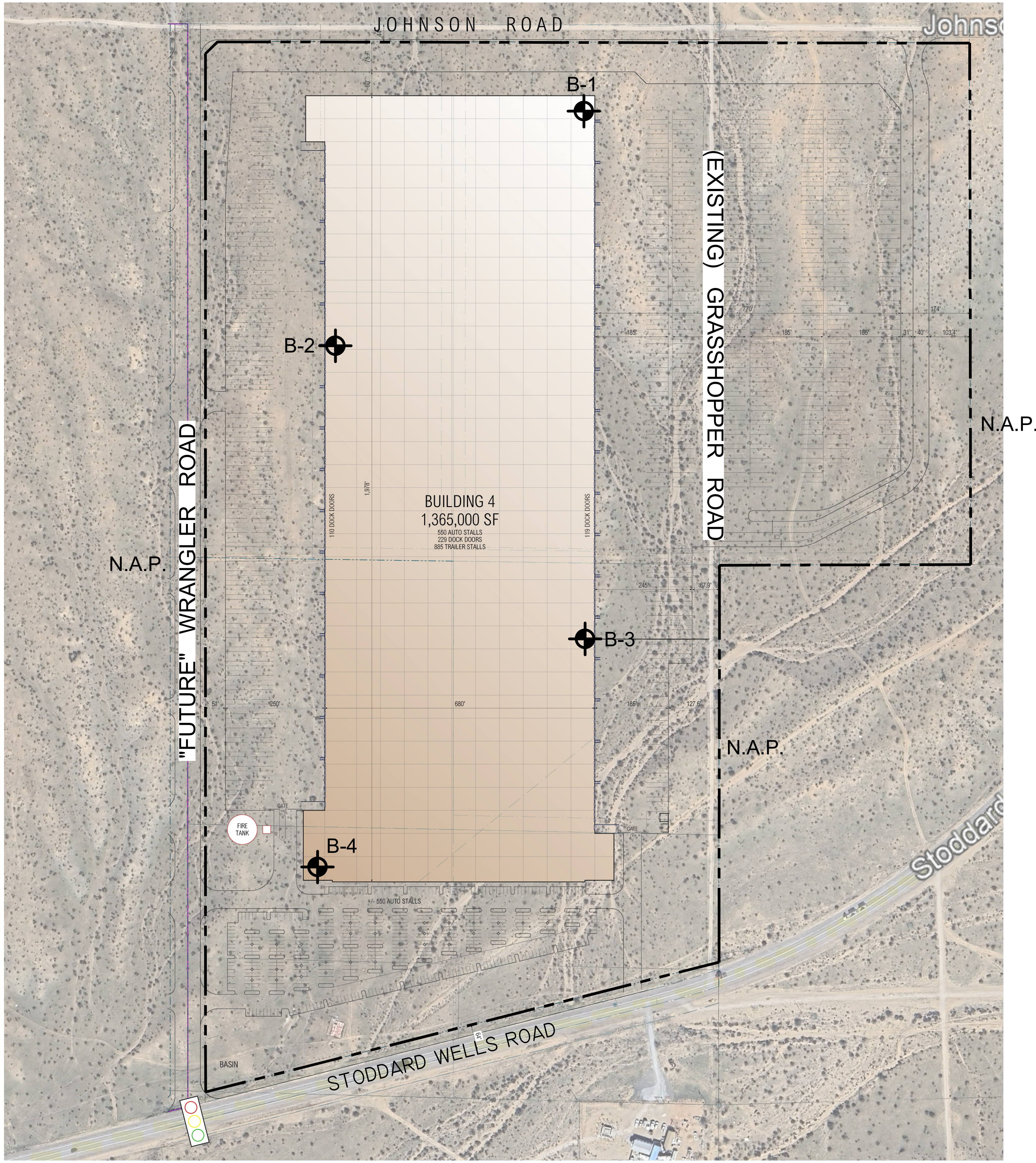
SCALE: 1" = 2,000'

DRAWN: JAH
CHKD: RGT

SCG PROJECT
25G195-1

PLATE 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

NOTE: SITE PLAN PROVIDED BY THE CLIENT.
AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH.

BORING LOCATION PLAN

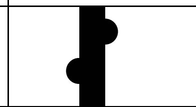
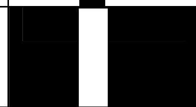
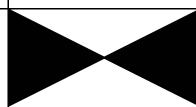
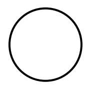
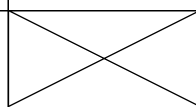
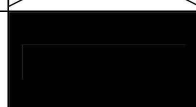
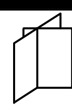
BMCC - PHASE II - BUILDING 4

APPLE VALLEY, CALIFORNIA

SCALE: 1" = 300'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: DK CHKD: RGT	
SCG PROJECT 25G195-1	
PLATE 2	

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 25G195-1	DRILLING DATE: 11/7/25	WATER DEPTH: Dry
PROJECT: BMCC - Phase II - Building 4	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 14 feet
LOCATION: Apple Valley, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					<u>OLDER ALLUVIUM:</u> Brown Silty fine to coarse Sand, little fine Gravel, dense to very dense-dry to moist							
		53				112	3					
		50/6"					6					@ 3 feet, Disturbed Sample
5		50/5"			@ 5 feet, slightly porous	112	6					
		50/5"				121	2					
		50/5"					3					@ 9 feet, Disturbed Sample
10												
		67/11"					5					
15												
					Brown fine to medium Sandy Clay, trace coarse Sand, weakly cemented, hard-damp							
		50/5"	3.5				8					
20												
		50/5"	4.5+				7					
25												
					Refusal at 26 feet due to very dense older alluvium							

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 25G195-1	DRILLING DATE: 11/7/25	WATER DEPTH: Dry
PROJECT: BMCC - Phase II - Building 4	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Apple Valley, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		45			OLDER ALLUVIUM: Brown Silty fine to coarse Sand, trace fine Gravel, trace to little Clay, slightly porous, weakly cemented, medium dense to very dense-dry to damp	111	3					
		50/6"				111	2					
5		50/6"				114	4					
		74/11"			@ 7 feet, little fine Gravel	122	2					
10		50/5"					4					@ 9 feet, Disturbed Sample
15		79/8"			@ 13½ feet, trace coarse Gravel		2					
20		90/11"					4					
25		50/4"	3.0		Brown Clayey fine to medium Sand to fine to medium Sandy Clay, trace coarse Sand, trace fine Gravel, weakly cemented, very dense/hard-damp		7					

Boring Terminated at 25 feet

TBL 25G195-1.GPJ SOCALGEO.GDT 12/4/25

TEST BORING LOG **PLATE B-2**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 25G195-1
PROJECT: BMCC - Phase II - Building 4
LOCATION: Apple Valley, California

DRILLING DATE: 11/7/25
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					YOUNGER ALLUVIUM: Brown Silty fine to coarse Sand, trace fine Gravel, medium dense-dry to damp							
	X	10					2					
	X	25					2					
5												
					OLDER ALLUVIUM: Brown Clayey fine to coarse Sand to fine to coarse Sandy Clay, slightly porous, moderately cemented, very dense/hard-damp							
	X	86/11"	4.5+				5					
					Brown Silty fine to coarse Sand, trace Clay, very dense-damp to moist							
	X	82/11"					6					
10												
	X	90/9"			@ 13½ feet, weakly cemented		6					
15												
	X	81			@ 18½ feet, moderately cemented		5					
20												
					Refusal at 22 feet due to very dense older alluvium							

TBL 25G195-1.GPJ SOCALGEO.GDT 12/4/25

SOUTHERN CALIFORNIA GEOTECHNICAL

A California Corporation

BORING NO. B-4

JOB NO.: 25G195-1
PROJECT: BMCC - Phase II - Building 4
LOCATION: Apple Valley, California

DRILLING DATE: 11/7/25
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

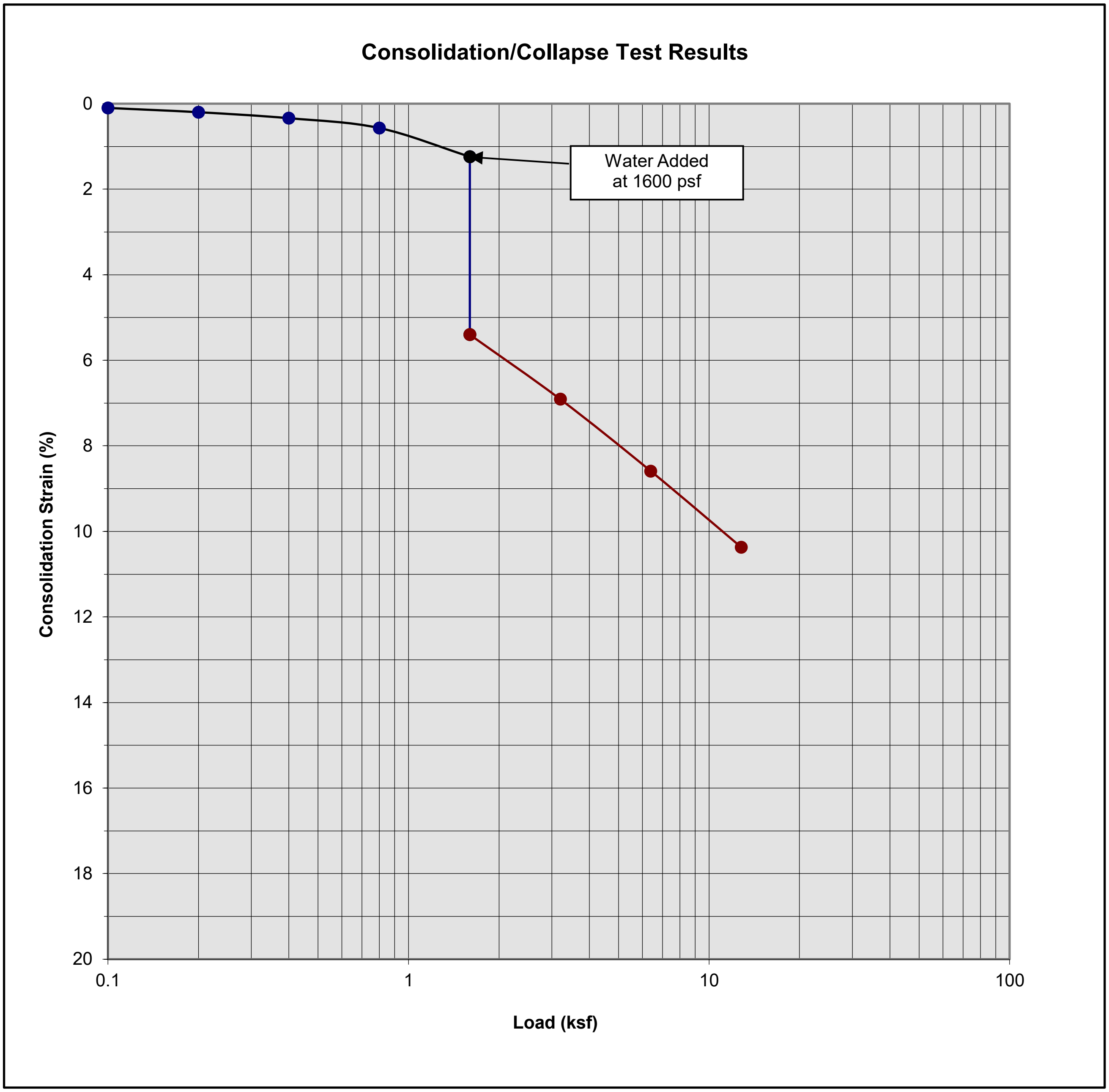
SURFACE ELEVATION: --- MSL

DEPTH (FEET)	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	36		YOUNGER ALLUVIUM: Brown Silty fine to coarse Sand, trace Clay, trace to little fine Gravel, slightly porous, medium dense to very dense-dry to damp	109	2					
	89/10"			112	3					
5	50/3"		OLDER ALLUVIUM: Brown Clayey fine to coarse Sand, little Silt, trace fine Gravel, slightly porous, weakly cemented, very dense-damp		3					@ 5 feet, Disturbed Sample
	87/10"		Brown Silty fine to coarse Sand, trace fine Gravel, slightly porous, very dense-damp to moist	114	5					
10	82/7"		Brown Clayey fine to coarse Sand, little Silt, very dense-damp		5					@ 9 feet, Disturbed Sample
15	79/11"	4.5+	Brown fine to coarse Sandy Clay, weakly to moderately cemented, hard-damp		4					
20	87/11	4.5+			6					
25	77/11"	4.5+	@ 23½ feet, moderately to strongly cemented		5					

Refusal at 25 feet due to very dense older alluvium

TBL 25G195-1.GPJ SOCALGEO.GDT 12/4/25

APPENDIX C

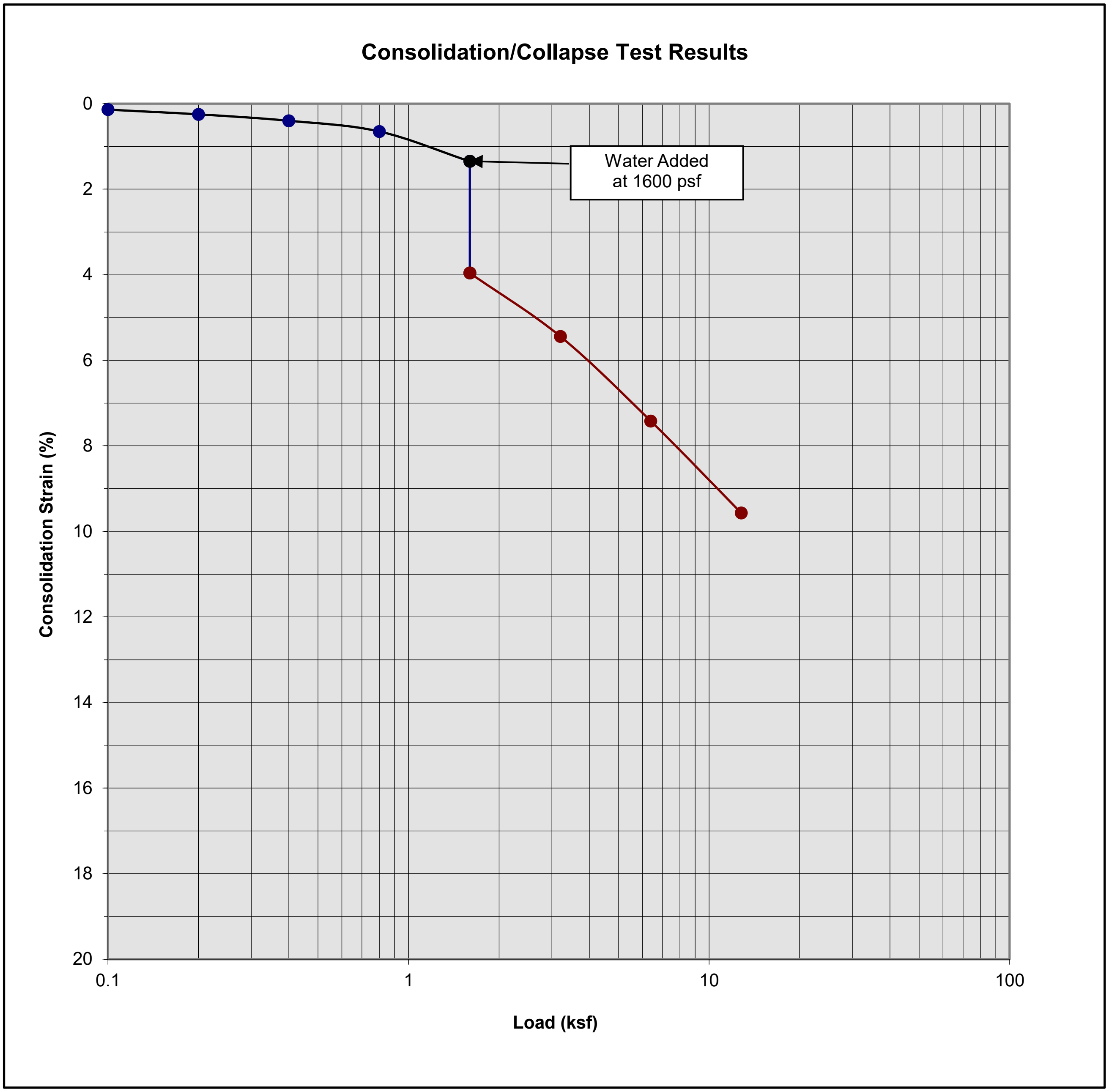


Classification: Brown Silty fine to coarse Sand, trace Clay

Boring Number:	B-4	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	1 to 2	Initial Dry Density (pcf)	109.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	122.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	4.16

BMCC - Phase II - Building 4
Apple Valley, California
Project No. 25G195-1
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

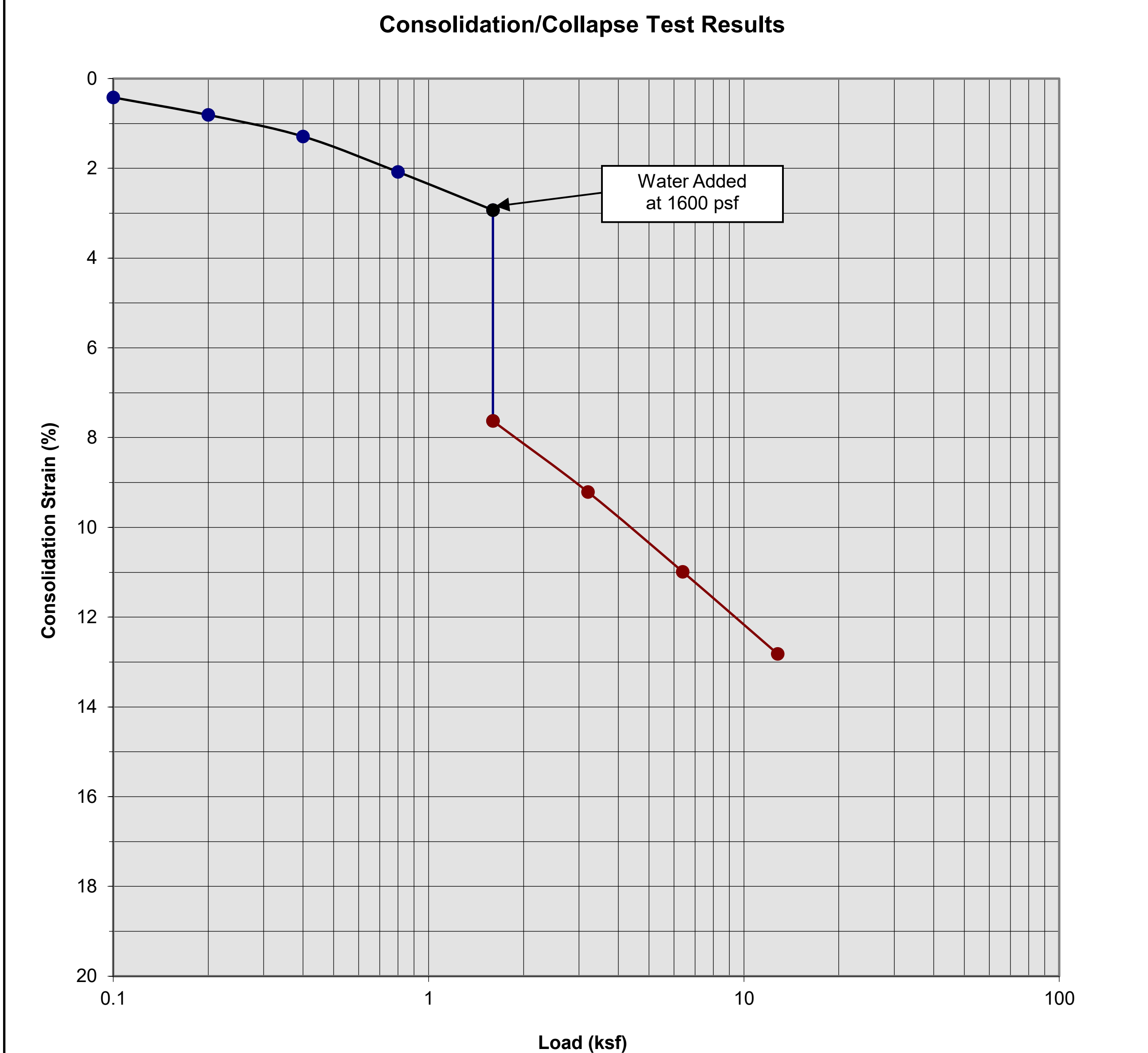


Classification: Brown Silty fine to coarse Sand, trace Clay

Boring Number:	B-4	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	3 to 3½	Initial Dry Density (pcf)	112.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	124.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.61

BMCC - Phase II - Building 4
Apple Valley, California
Project No. 25G195-1
PLATE C- 2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

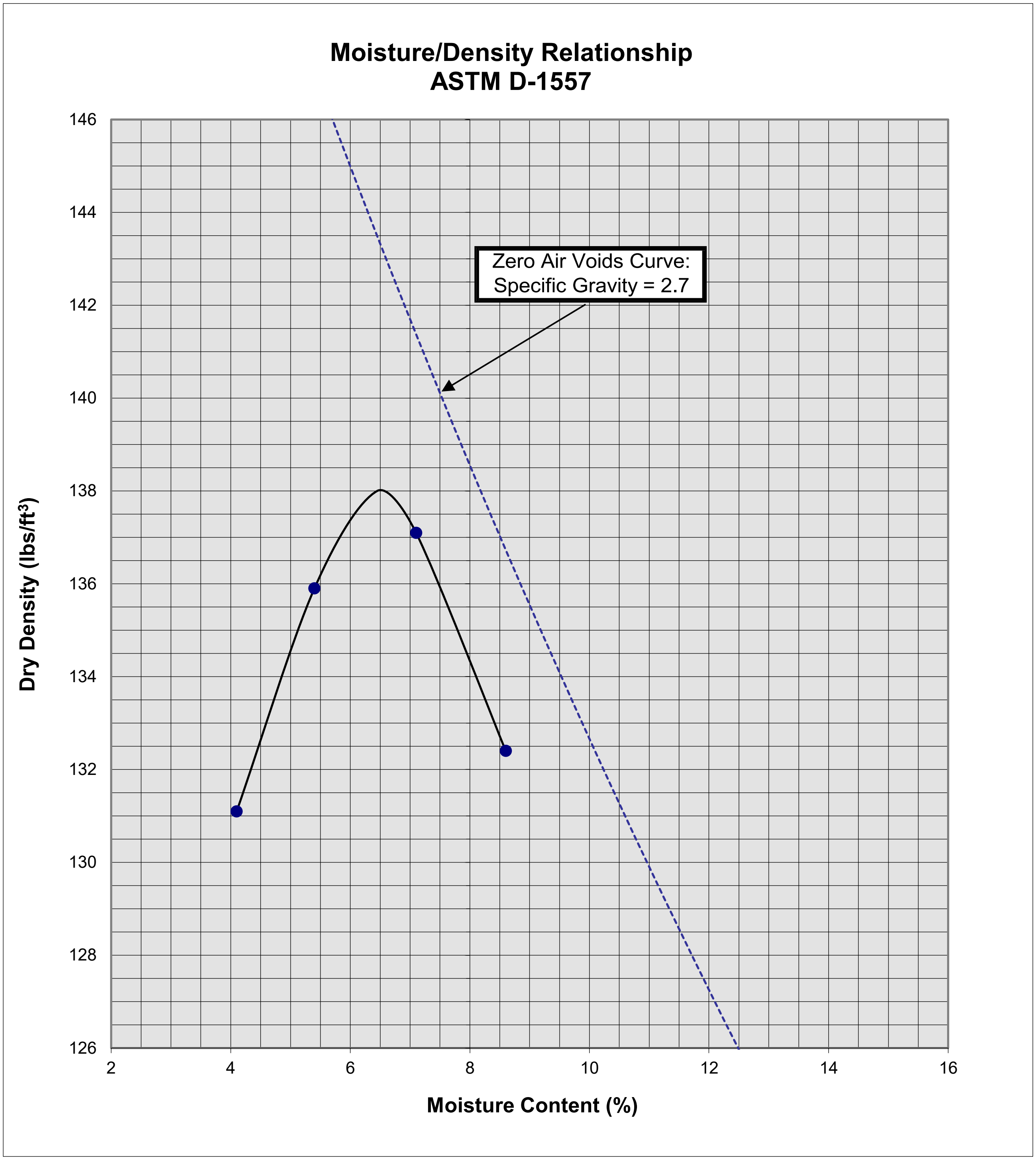


Classification: Brown Silty fine to coarse Sand, trace fine Gravel

Boring Number:	B-4	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	11
Depth (ft)	7 to 8	Initial Dry Density (pcf)	113.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	129.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	4.70

BMCC - Phase II - Building 4
Apple Valley, California
Project No. 25G195-1
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-4 @ 1-5'
Optimum Moisture (%)	6.5
Maximum Dry Density (pcf)	138
Soil Classification	Dark Brown Silty fine to coarse Sand, little to some Clay, trace to little fine Gravel

BMCC - Phase II - Building 4
Apple Valley, California
Project No. 25G195-1
PLATE C- 4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 20. The top 12 inches of the compacted fill should have a maximum particle size of 3 inches, and all underlying compacted fill material a maximum 6-inch particle size, except as noted below.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

- The foundation influence zone is defined as extending one foot horizontally from the outside edge of a footing, and proceeding downward at a 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.

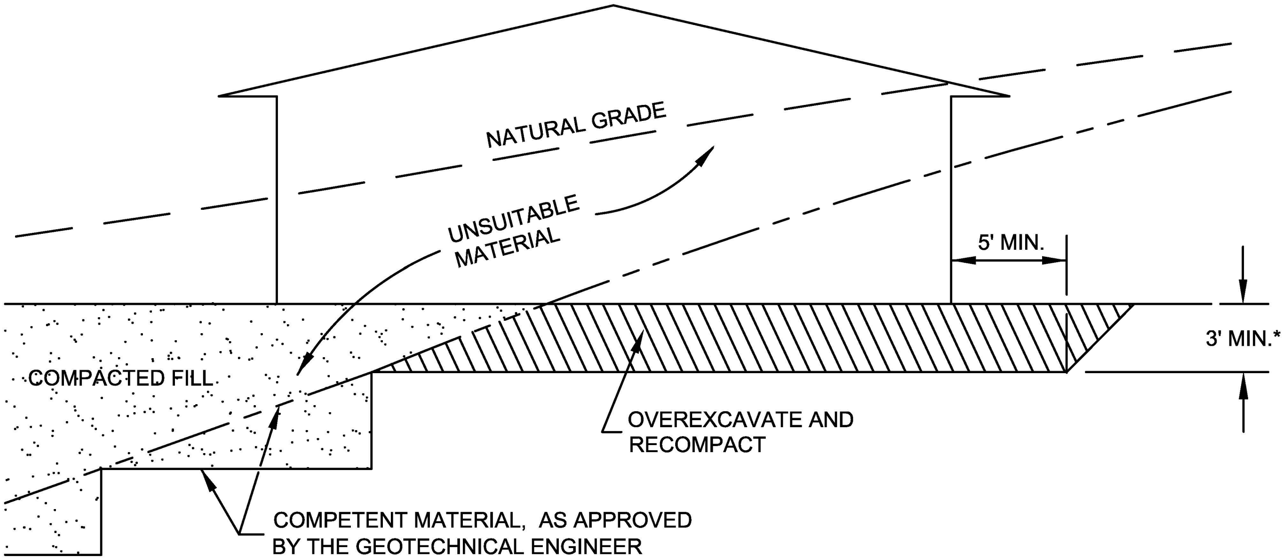
Subdrains

- 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.

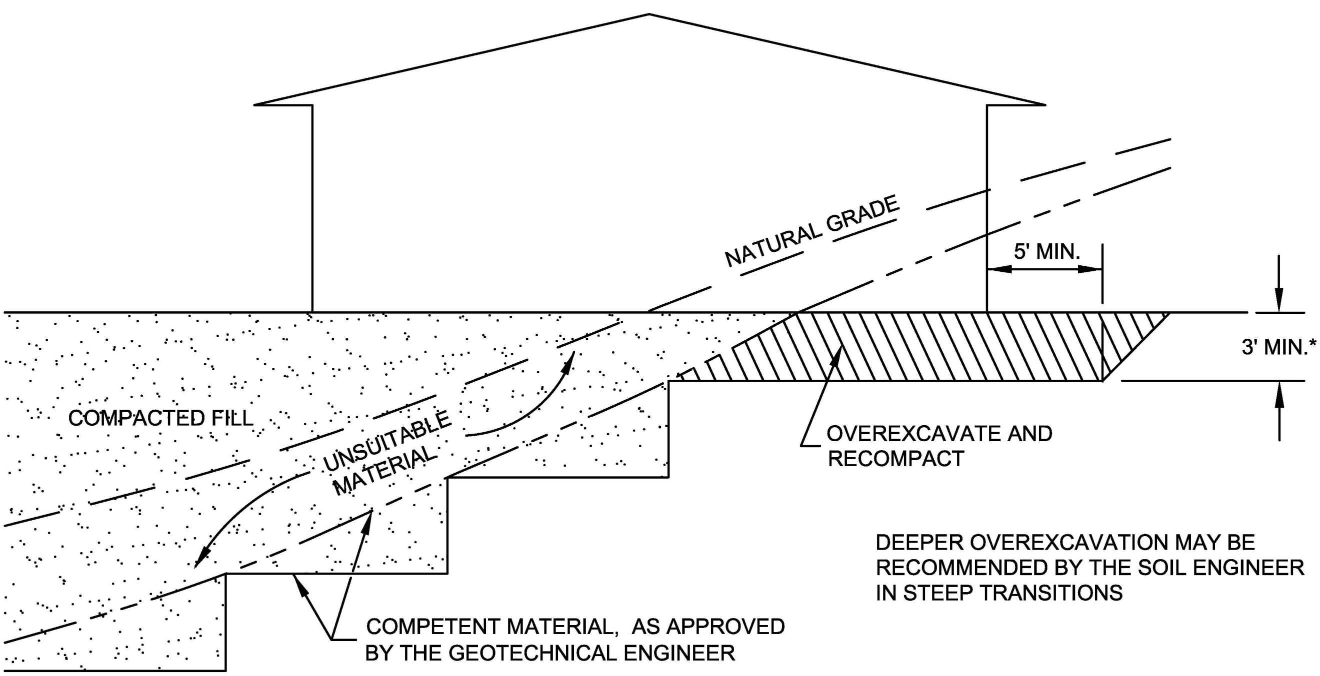
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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

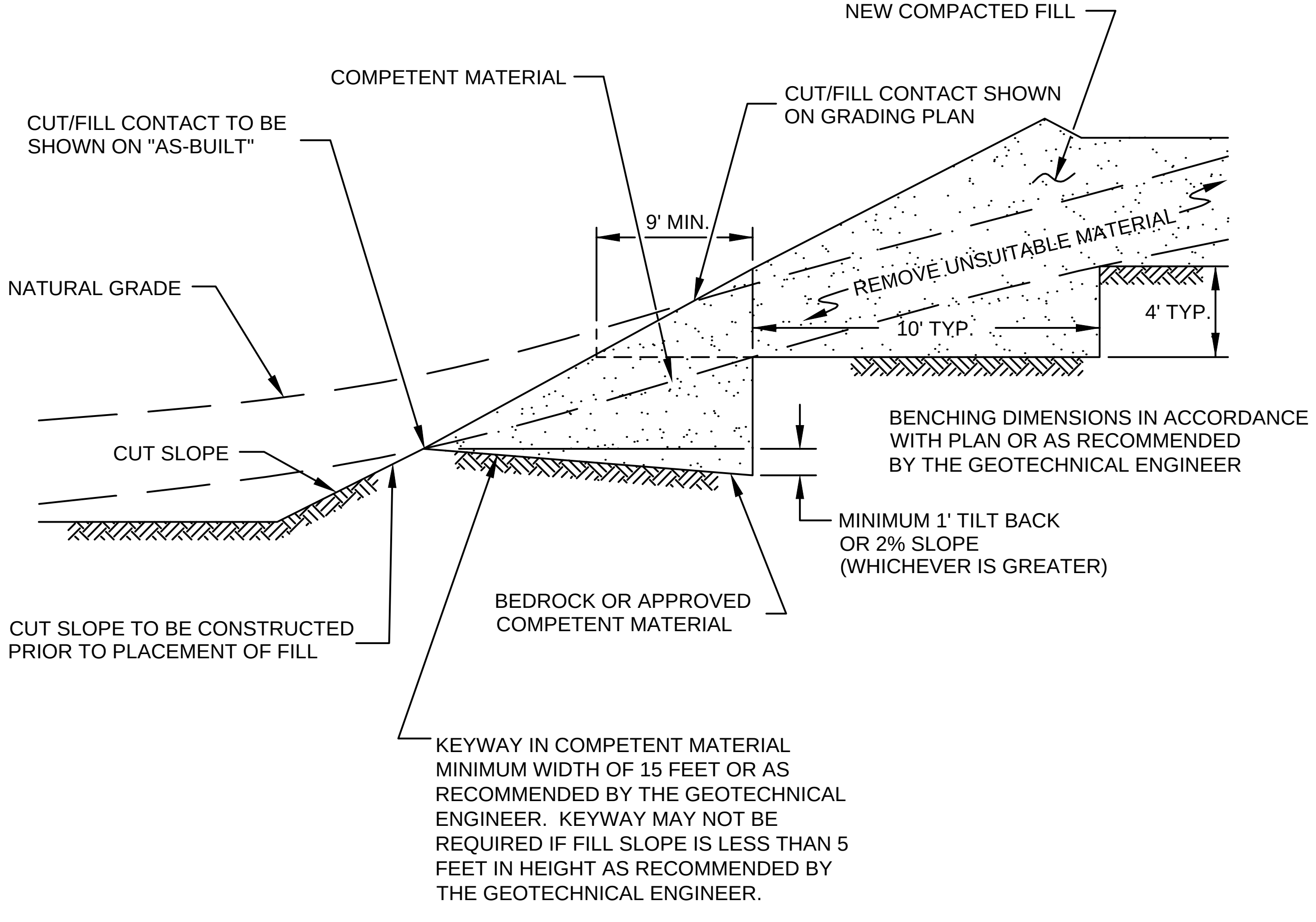
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

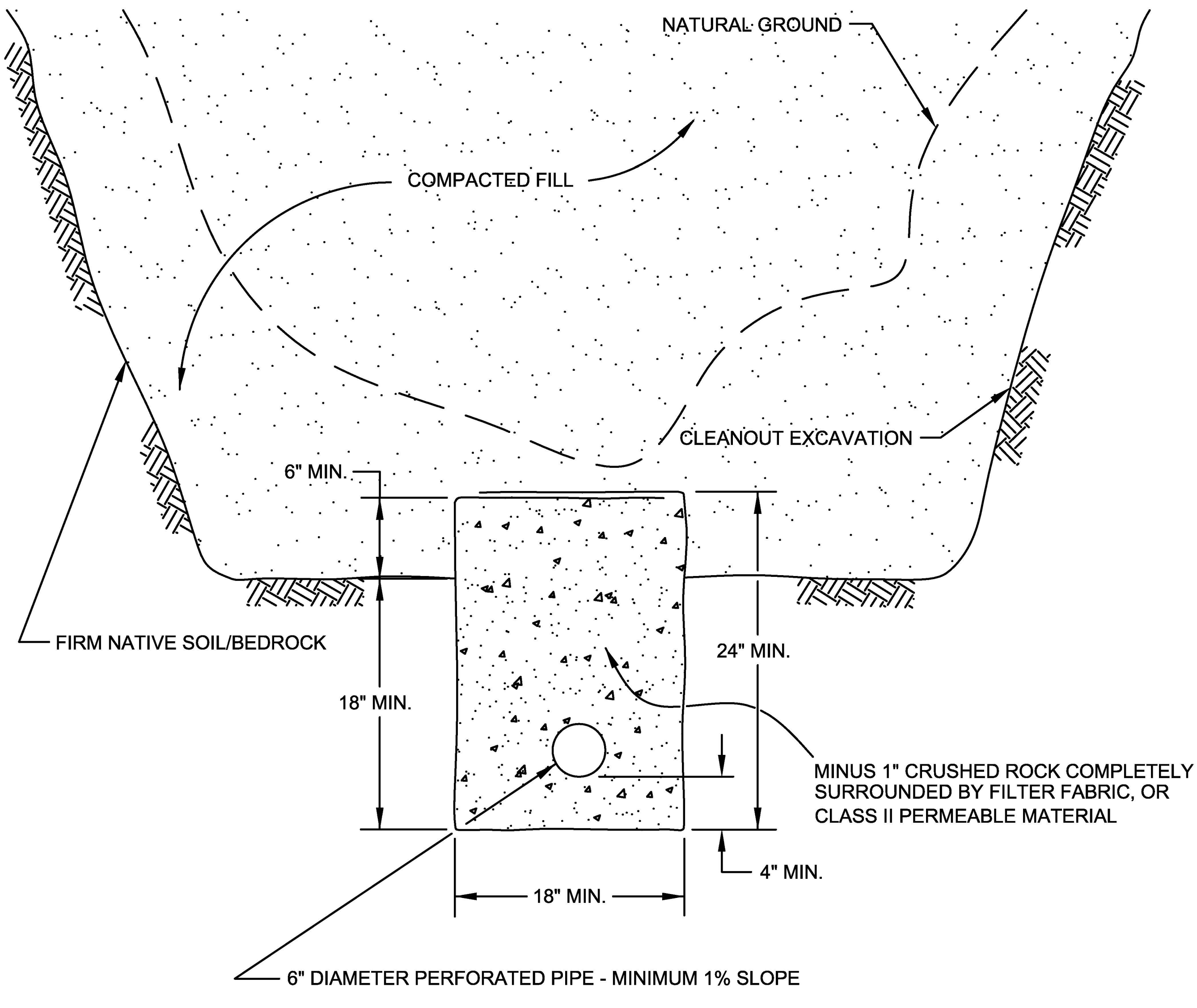
GRADING GUIDE SPECIFICATIONS

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

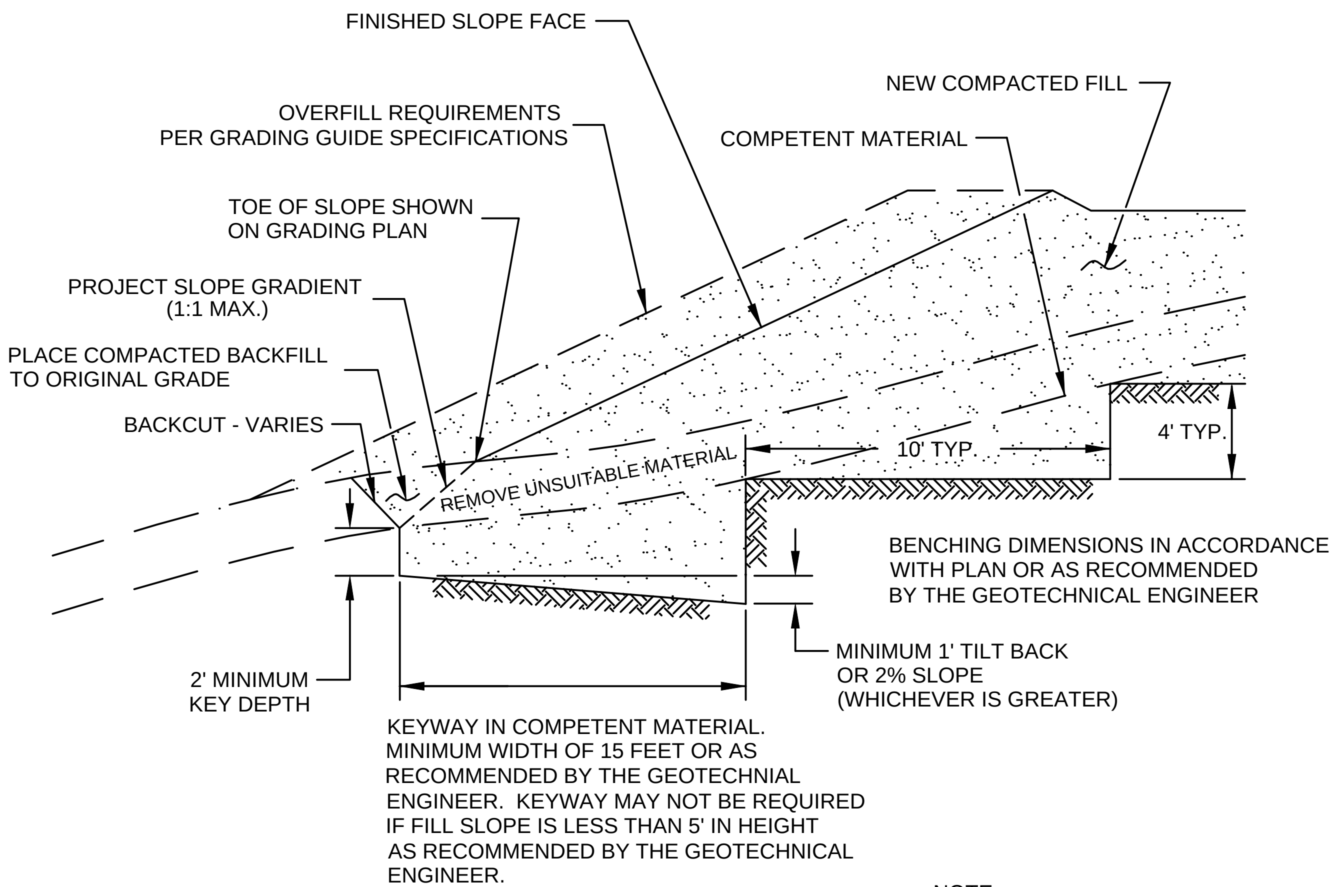
GRADING GUIDE SPECIFICATIONS

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

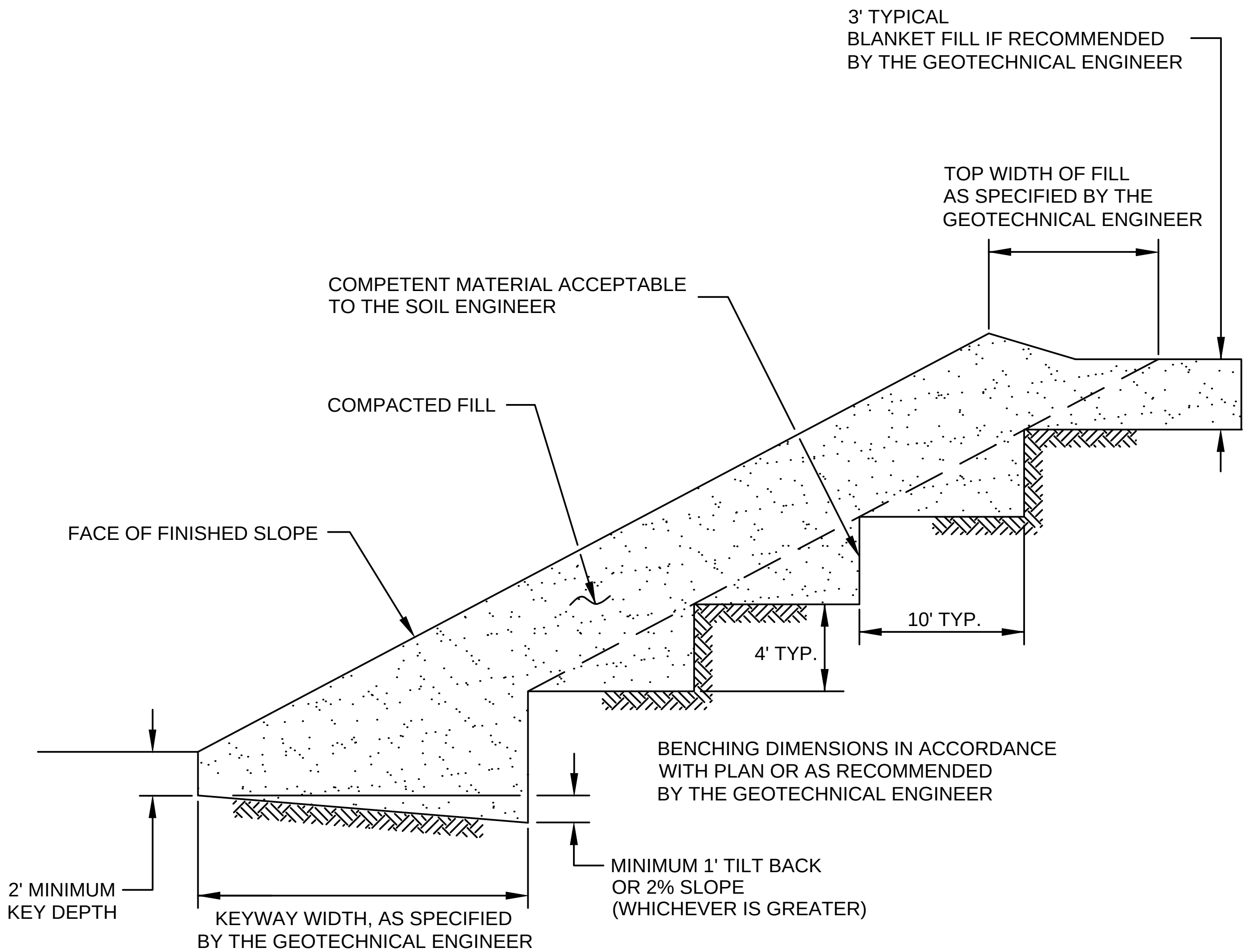
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

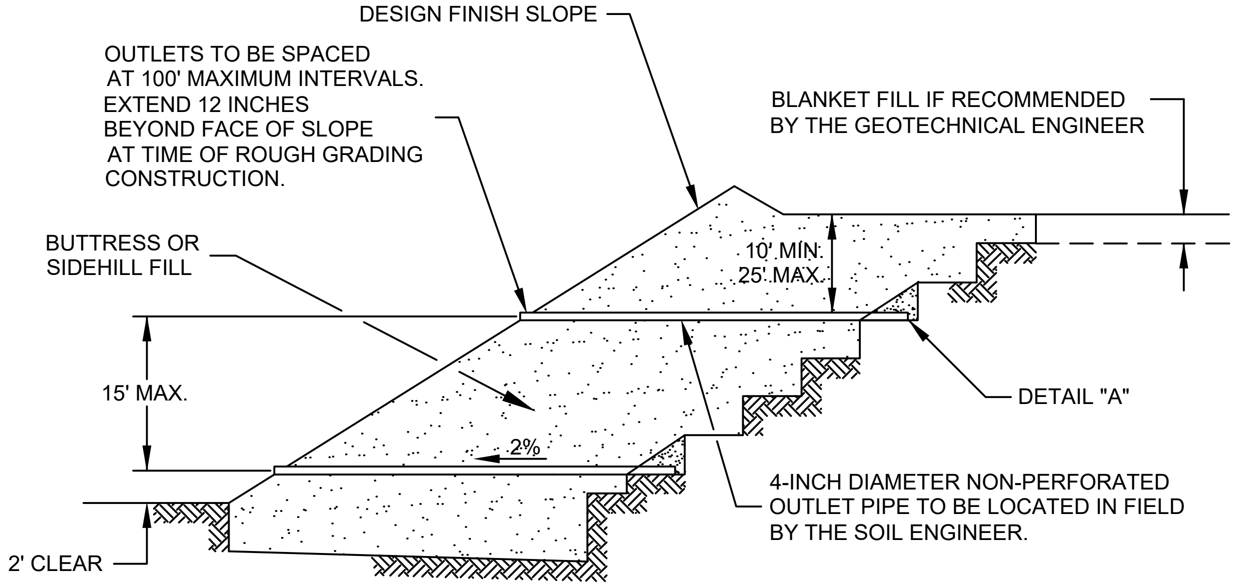
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



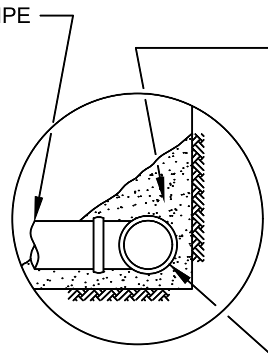
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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



DETAIL "A"

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

SLOPE FILL SUBDRAINS

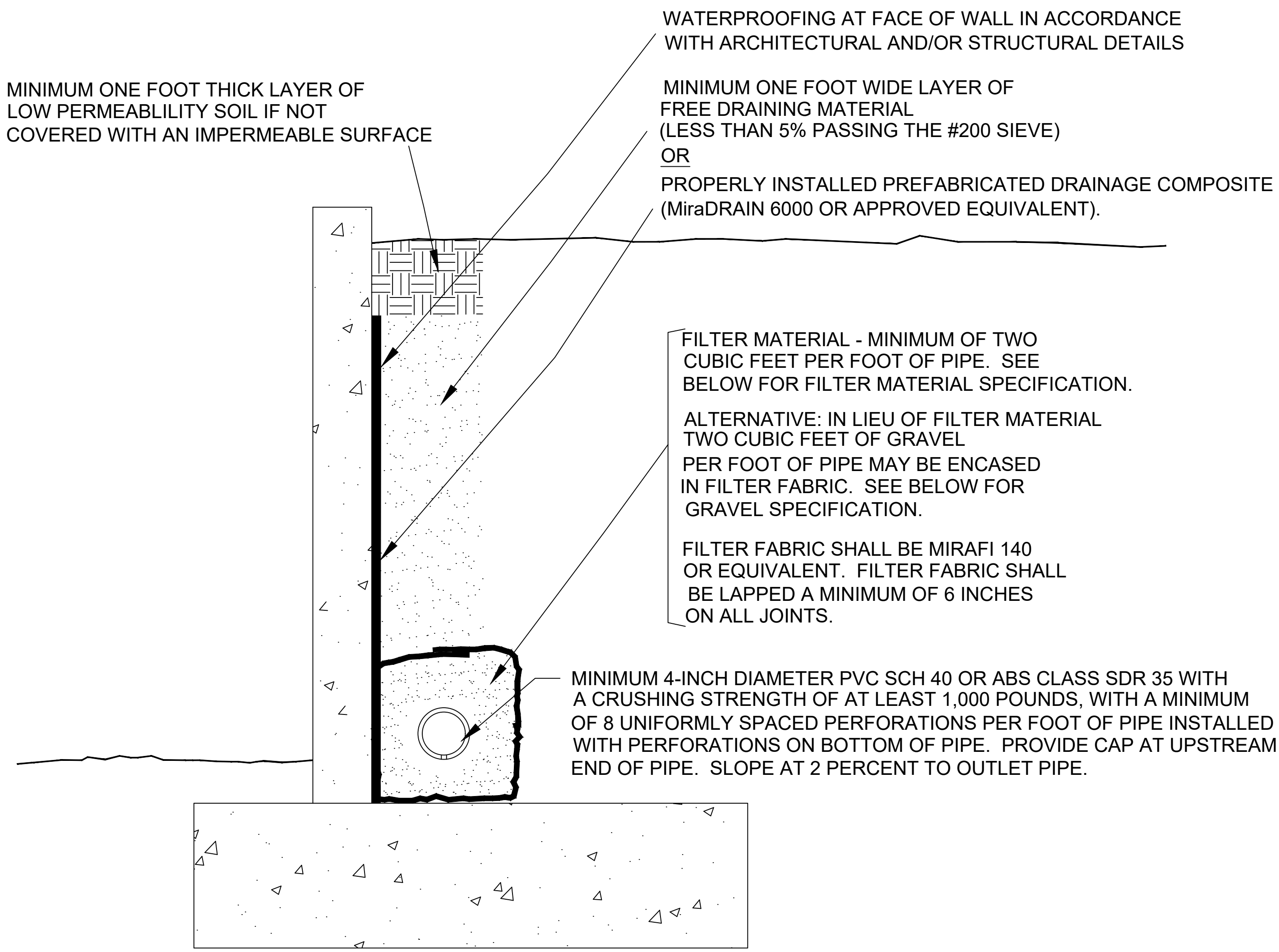
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

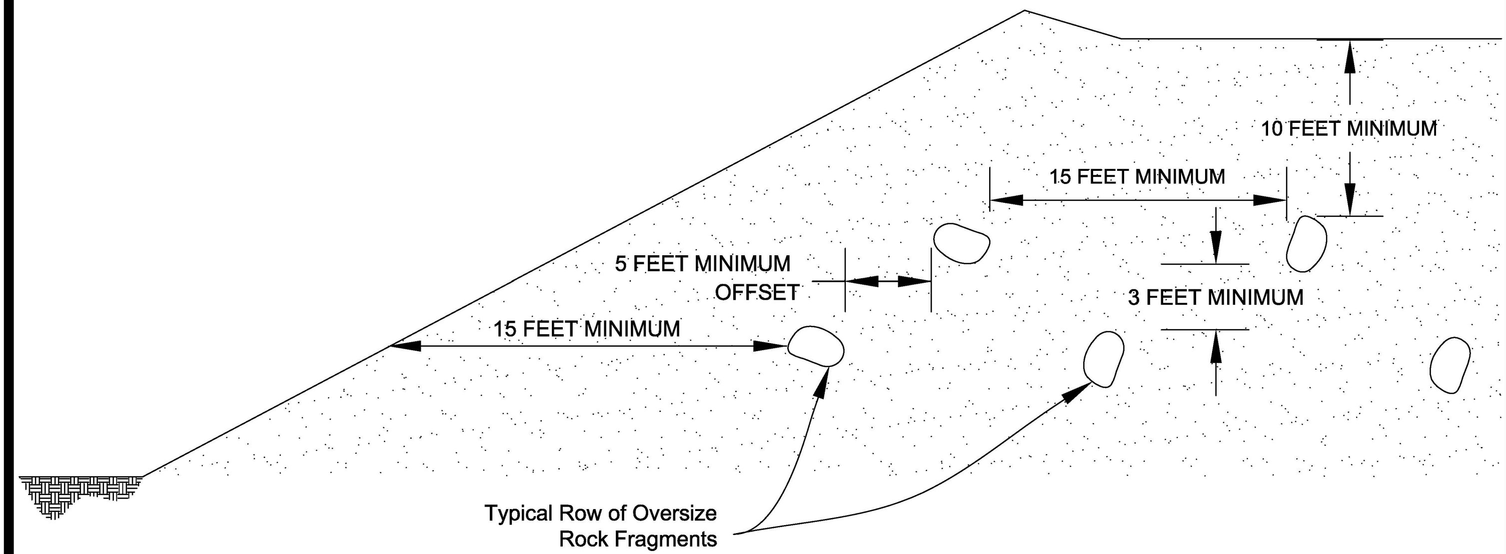
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

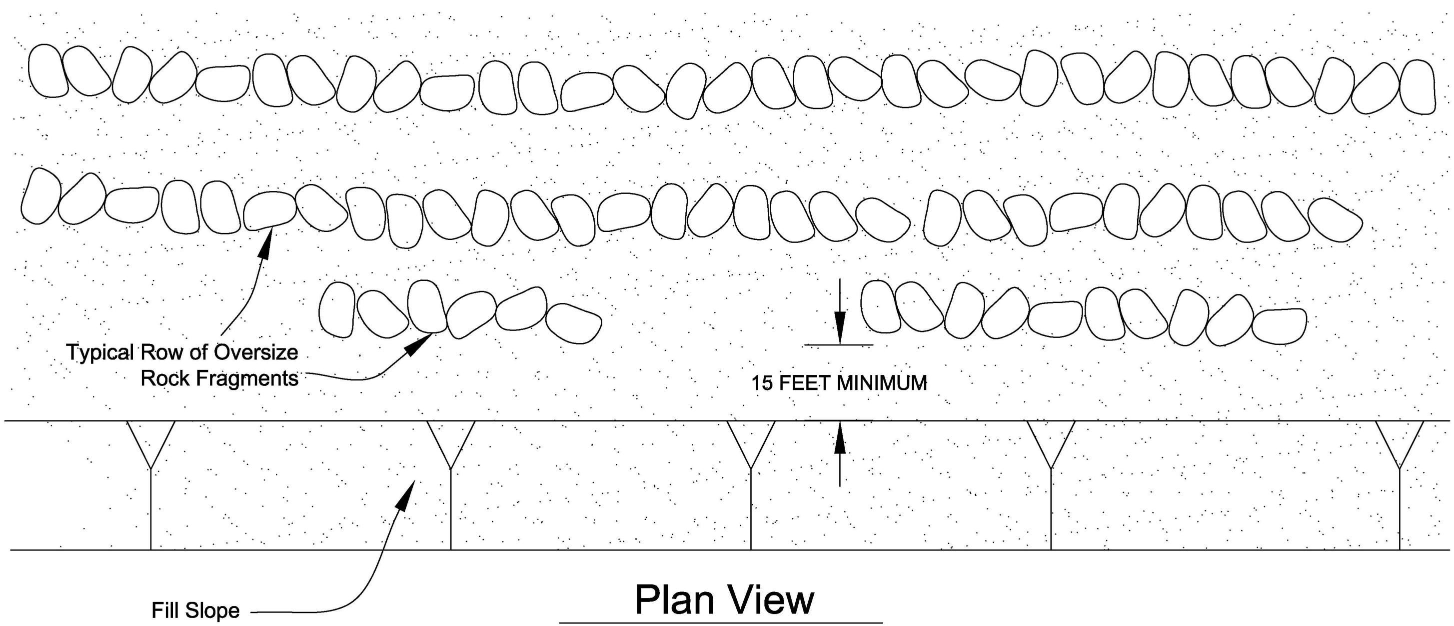
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

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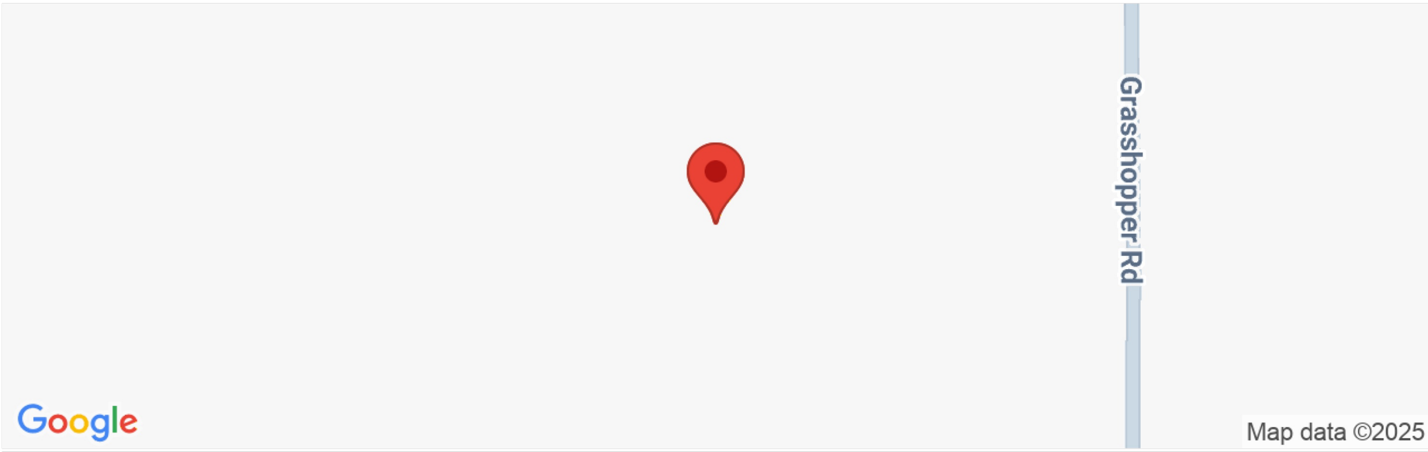
CALIFORNIA
GEOTECHNICAL

APPENDIX E

Latitude, Longitude: 34.597583, -117.243753



Date	12/2/2025, 4:27:44 PM
Design Code Reference Document	ASCE7-22
Risk Category	II
Site Class	Default

Type	Value	Description (Data)
S_S	1.15	The MCE_R spectral response acceleration at 0.2 seconds for Site Class BC, in units of g.
S_1	0.39	The MCE_R spectral response acceleration at 1 second for Site Class BC, in units of g.
S_{MS}	1.51	S_{MS} = 1.5 x S_{DS}, the Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration for short periods (of the two-period spectrum) and the user-specified Site Class.
S_{M1}	1.01	S_{M1} = 1.5 x S_{D1}, the MCE_R spectral response acceleration for 1 second (of the two-period spectrum) and the user-specified Site Class.
S_{DS}	1.01	The design spectral response acceleration for short periods (of the two-period spectrum) and the user-specified Site Class, in units of g.
S_{D1}	0.67	The design spectral response acceleration for 1 second (of the two-period spectrum) and the user-specified Site Class, in units of g

Type	Value	Description (Data Contd.)
SDC	D	Seismic design category
PGA_M	0.55	PGA_M, the Geometric-Mean Maximum Considered Earthquake (MCE_G) peak ground acceleration for the user-specified Site Class, in units of g
T_S	0.666	$T_S = S_{D1}/S_{DS}$, in seconds, for construction of the two-period design spectrum
T_0	0.133	$T_0 = 0.2 \times T_S$, in seconds, for construction of the two-period design response spectrum
T_L	12	T_L, the long-period transition period, in seconds, for construction of the two-period design response spectrum

Type	Value	Description (Underlying Data and Metadata)
PGA_{uh}	See underlying data for Site Classes C, CD, and D	Probabilistic uniform-hazard (2%-in-50-years), geometric-mean peak ground acceleration, in units of g.
PGA_{84th}	See underlying data for Site Classes C, CD, and D	Deterministic 84th-percentile, geometric-mean peak ground acceleration (without deterministic lower limit), in units of g.
V_{S30}	260	The shear-wave velocity used for the user-specified Site Class, in units of m/s
Spatial Interpolation Method	linearloglinear	Identifier for spatial interpolation method used to obtain values for location of interest from underlying gridded values: "linearloglinear" for bilinear of natural logarithm of values.
PGA_{dFloor}		Deterministic lower limit peak ground acceleration (PGA_G) for the user-specified Site Class, in units of g.
riskTargetedSpectrum		Probabilistic risk-targeted, maximum direction response spectrum (for 1%-in-50-years collapse risk)
eightyFourthSpectrum		Deterministic 84^{th}-percentile, maximum-direction response spectrum (without deterministic lower limit)

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

SEISMIC DESIGN PARAMETERS - 2025 CBC
BMCC - PHASE II - BUILDING 4
APPLE VALLEY, CALIFORNIA

DRAWN: JLL
CHKD: RGT
SCG PROJECT
25G195-1
PLATE E-1

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