GEOTECHNICAL FEASIBILITY STUDY
PROPOSED GOODMAN LOGISTICS CENTER FULLERTON

2001 East Orangethorpe Avenue
Fullerton, California

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

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May 20, 2020

Goodman
18201 Von Karman Avenue, Suite 1170
Irvine, California 92612

Attention: Mr. Matthew McGuire

Project No.: **19G139-1R**

Subject: **Geotechnical Feasibility Study**
Proposed Goodman Logistics Center Fullerton
2001 East Orangethorpe Avenue
Fullerton, California

Dear Mr. McGuire:

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. It should be noted that this report is a revision to a previous report, dated August 13, 2019. This report supersedes the previous report.

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

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1.0 EXECUTIVE SUMMARY

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

It should be noted that this investigation was focused on determining the geotechnical feasibility of the proposed development. It was not intended to be a design level investigation. Future studies will be necessary to refine the preliminary design parameters that are presented within this current report.

Geotechnical Design Considerations

- Most of the borings encountered artificial fill soils directly beneath the pavements. The fill soils possess variable strengths and densities and are considered to represent undocumented fill materials. Some soils classified as possible fill were encountered beneath the fill soils.
- Native alluvium was encountered at all of the boring locations, beneath the artificial fill/ possible fill materials, where present, or at the ground surface. Some of the near surface soils extending to depths of 3 to 12± feet possess low relative densities and minor potentials for consolidation settlement.
- Remedial grading is considered necessary within the proposed building pad areas in order to remove the artificial fill soils in their entirety, any soils disturbed during demolition, and a portion of the near surface native alluvium in order to replace these materials as compacted structural fill.
- The demolition of the existing structures, pavements, above ground storage tanks (AST), and the stripping of the trees in the orchard area will result in extensive disturbance to the near surface soils. Any soil disturbed during demolition or stripping should also be overexcavated and recompacted as structural fill.
- Groundwater was not encountered at any of the boring locations which extended to depths of 15 to 30± feet.

Preliminary Site Preparation Recommendations

- Initial site stripping should include removal of any surficial vegetation. This should include any weeds, grasses, shrubs, and trees. These materials should be disposed of offsite.
- Demolition of the existing structures will be necessary in order to facilitate the proposed development at this site. Demolition should include all foundations, floor slabs, and any associated utilities. Any excavations associated with demolition should be backfilled with compacted fill soils. Debris resultant from demolition should be disposed of off-site.
- Remedial grading consisting of overexcavation to depths on the order of 3 to 5± feet below existing and proposed building pad grades should be anticipated. Below proposed foundation bearing grades, additional overexcavation to depths on the order of 2 to 3± feet below foundation bearing grades is expected to be necessary. Additional overexcavation may also be necessary in localized areas where loose native alluvial soils are exposed at the overexcavation bottoms. Loose native alluvial soils were encountered to depths of us to 12± feet at Boring No. B-1. Any soils classified as possible fill should be evaluated at the time of site grading to determine if they should be overexcavated.

- No significant overexcavation is expected to be necessary in the new pavement or flatwork areas, with the exception of soils disturbed during demolition or in localized zones of unsuitable existing fill or native alluvium.

Preliminary Foundation Design Parameters

- Spread footing foundations, supported in newly placed structural fill soils.
- Maximum, net allowable soil bearing pressure: 2,000 to 3,000 lbs/ft^2.
- The estimated allowable bearing pressures provided above should be refined during the design level geotechnical investigation, based on actual column loads and detailed settlement analyses.

Preliminary Building Floor Slab Recommendations

- Conventional Slabs-on-Grade, 5 to 6 inches thick
- The design of the floor slabs will depend on the results of the future geotechnical study.
- The actual thickness and reinforcement of the floor slabs should be determined by the structural engineer.

Preliminary Pavement Thickness Recommendations

ASPHALT PAVEMENTS (R = 30)					
Materials	**Thickness (inches)**				
	Automobile Parking (TI = 4.0)	Automobile Drive Lanes (TI = 5.0)	Truck Traffic		
			(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	6	8	10	11
Compacted Subgrade (90% minimum compaction)	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30)				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5½	6	7
Compacted Subgrade (95% Relative 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. 19P178, dated March 8, 2019. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory geotechnical 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 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 foundation, floor slab, and grading recommendations.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the street address of 2001 East Orangethorpe Avenue in Fullerton, California. The site is bounded to the north by Kimberly Avenue, to the west by South Acacia Avenue, to the south by East Orangethorpe Avenue, and to the east by South State College Boulevard. Excluded from the subject site is a rectangular area at the southeast corner of this bounded area with dimensions of approximately 425 feet by 450 feet. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The subject site consists of a several contiguous rectangular parcels totaling about 66± net acres in size. The site presently consists of an operational Kimberly-Clark Corporation facility. The site is currently developed with multiple buildings (approximately 1,210,720 ft² of building area), above ground storage tanks (AST), and equipment. The southwestern portion of the site is developed with a truck court, driveway, and a landscaped area. The south-central area is also developed with landscaped areas and an automobile parking lot. The north-central portion of the site is developed with a truck court, ASTs, a well, and a railroad spur that terminates on the northern side of the east portion of the main building. The eastern half of the site is developed with RV parking in the north, truck parking areas in the north and south, several storage buildings, an AST, and an orchard east of the storage buildings. This report also addresses an alternative site plan (discussed in the subsequent section of this report), which includes the approximately 0.7-acre site located south of proposed Building 3 and north of Orangethorpe Avenue. We understand that the Project Applicant has engaged in negotiations for the acquisition of this site.

Detailed topographic information was not available at the time of this report. However, based on topographic information provided by T&B planning, we understand that the site topography ranges from El. 184± feet above mean sea level (amsl) in the northeast area of the site to El. 174± feet amsl in the northwest portion of the site.

3.2 Proposed Development

A potential site plan for the proposed development, identified as Master Site Plan, Sheet DAB-A1.0, was provided to our office by the client. An alternative site plan, Sheet DAB-A-EIR was also provided. The preliminary conclusions and recommendations in this report apply to both site plans. Both of the schemes indicate that the proposed development will consist of four new industrial buildings. The Master Site Plan identifies the four new buildings as Buildings 1 through 4, from west to east. Based on this Scheme, Building 1 will possess an area of 342,695 ft² and will possess dock high doors on the east side of the building. Buildings 2 and 3 will each be constructed in a cross-dock configuration with dock high doors on the east and west sides of the buildings. Buildings 2 and 3 will possess footprint areas of 545,255 ft² and 495,290 ft², respectively. Building 4 will poses an area of 178,282 ft² and will be constructed with dock-high doors on the west side of the building. The Master Site Plan, Sheet DAB-A1.0, excludes the aforementioned 0.7- acre parcel located south of proposed Building 3.

The alternative site plan, Sheet DAB-A-EIR, is very similar to the Master Site Plan, except that it includes the aforementioned 0.7-acre parcel as a part of the proposed development area. For this scheme, Building 3 will possess an area of 543,152 ft^2.

Based on a conceptual grading plan prepared by Tait and Associates, the buildings will be surrounded by Portland cement concrete pavements in the truck court areas and in the automobile parking and drive areas. Limited areas of decorative concrete flatwork and landscape planter areas are also anticipated.

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

Detailed grading plans for the proposed development were not available at the time of this report. The conceptual grading plan provided by Tait and Associates indicates that proposed building pad grades will range between elevations of 181.92 and 184.92 feet mean sea level. However, the plan lacks detailed information about the existing site topography within the proposed building areas. Based on the assumed topography, cuts and fills of up to 6 to 8± feet are expected to be necessary to achieve the proposed site grades. No significant amounts of below grade construction, such as basements or crawl spaces, are expected to be included in the proposed development.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of nine (9) borings advanced to depths of 15 to 30± feet below existing site grades. All of the borings were logged during drilling by a member of our staff.

The borings were advanced with a conventional truck-mounted drill rig equipped with hollow-stem augers. Representative bulk and relatively 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 Plans, included as Plates 2A and 2B in Appendix A of this report. The Boring Logs, which illustrate the conditions encountered at the boring locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete pavements were encountered at the ground surface at all of the boring locations, except for Boring No. B-6 which was located in the orchard. The pavements at these boring locations consist of 3 to 4½± inches of asphaltic concrete with 0 to 7± inches of underlying aggregate base.

Artificial Fill

With the exception of Boring No. B-6, artificial fill soils were encountered beneath the pavements at all of the boring locations, extending to depths of 2 to 3± feet below the existing site grades. The fill soils generally consist of medium stiff to very stiff fine to coarse sandy clays and loose to medium dense clayey fine sands, fine sandy silts, and silty fine to coarse sands. The fill soils possess a mottled appearance with trace amounts of asphaltic concrete and brick fragments, resulting in their classification as artificial fill.

Some soils classified as possible fill were encountered below the artificial fills at Boring Nos. B-2 and B-8, extending to depths of 3 to 4½± feet. The possible fill soils generally consist of medium dense to very stiff clayey fine sands, fine sandy clays, and silty clays.

Alluvium

Native alluvium was encountered beneath the fill and possible fill soils at all of the boring locations, and at the ground surface at Boring No. B-6. Native alluvial soils extend to at least the maximum depth explored of 30± feet below existing site grades. The native alluvial soils within the upper 12 to 17± feet generally consist of loose to medium dense silty sands and fine to coarse sands with variable amounts of fine to coarse gravel, trace clay, occasional fine root fibers, and iron oxide staining. At greater depths the alluvial soils generally consist of medium dense fine sandy silts and silty fine sands, with occasional fine to coarse sand layers. Boring No. B-5 encountered a stiff silty clay layer from 14½ to 17± feet, and a medium dense fine sandy clay layer from 22 to 25± feet. Boring No. B-8 encountered interbedded lenses of clayey sands and sandy clays between depths of 7½ and 9± feet.

Groundwater

Free water was not encountered during the drilling of any of the borings. Based on the lack of any water within the borings and the moisture contents of the recovered soil samples, the static groundwater is considered to have existed at a depth in excess of 30± feet at the time of the subsurface exploration. As part of our research, we also reviewed recent groundwater data available within the vicinity of the site. The primary reference used to determine the groundwater depths in this area is the California Department of Water Resources website, http://wdl.water.ca.gov/waterdatalibrary/. The nearest monitoring well in this database is located in the north central part of the site. Water level readings within this monitoring well indicated a high groundwater level of 88± feet (June, 2010).

5.0 LABORATORY TESTING

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

Classification

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

Density and Moisture Content

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

Consolidation

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

Soluble Sulfates

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

Sample Identification	Soluble Sulfates (%)	ACI Classification
B-1 @ 0 to 5 feet	0.005	Not Applicable (S0)
B-9 @ 0 to 5 feet	0.039	Not Applicable (S0)

Corrosivity Testing

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

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)
B-1 @ 0 to 5 feet	3,320	8.3	2.3
B-9 @ 0 to 5 feet	1,040	7.8	67

Maximum Dry Density and Optimum Moisture Content

Representative bulk samples were tested for their maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Sheets C-9 and C-10 in Appendix C of this report. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Expansion Index

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

Sample Identification	Expansion Index	Expansion Potential
B-5 @ 0 to 5 feet	17	Very Low
B-7 @ 0 to 5 feet	15	Very Low

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our review, field exploration, laboratory testing, and geotechnical analysis, the proposed development, which will consist of a new industrial logistics center development, is considered feasible from a geotechnical standpoint. The recommendations contained in this report should be taken into the design, construction, and grading considerations. The recommendations are contingent upon all grading and foundation construction activities being monitored by the geotechnical engineer of record.

Based on the preliminary nature of this investigation, further geotechnical investigation will be required prior to construction of the proposed development. 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 2019 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

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

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

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.605
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0..566
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.605
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.981
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.070
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.654

It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the 2019 CBC. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2019 CBC using the value of S_1 obtained from the Seismic Design Maps Tool, assuming that a site-specific ground motion hazards analysis is not required for the proposed building at this site.

Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The Seismic Hazard Evaluation of the Anaheim Quadrangle and Open-File Report 97-08, prepared by the California Geological Survey (CGS) indicates that the subject site is not located within a designated liquefaction hazard zone. In addition, the subsurface conditions encountered at the boring locations are not considered to be conducive to liquefaction. Furthermore, the long-term groundwater table is considered to be present a depth in excess of 50± feet. Based on these considerations, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

Most of the borings encountered artificial fill soils beneath the existing pavements. The fill soils possess variable composition and variable densities. Based on the lack of documentation of the placement and compaction of the existing fill soil, these materials are considered to represent undocumented fill and are not considered suitable, in their present condition, to support the foundations and floor slabs of the proposed structures.

Some of the soils present directly beneath the fill soils were classified as possible fill because they possess a slightly disturbed appearance, but lack obvious indicators of artificial fill. All of the borings encountered native alluvium beneath the fill and possible fill soils, or at the ground surface. The native alluvium and possible fill soils possess variable densities and the results of laboratory testing indicate that some of these soils possess a moderate potential for consolidation settlement when loaded. In general, the native alluvial soils encountered within the upper 3 to 6± feet possess loose relative densities and Boring No. B-1 encountered loose alluvium extending to a depth of 12± feet. Loose, porous, soils were encountered within the upper 4± feet at Boring No. B-6. Based on these considerations, remedial grading will be necessary within the proposed building areas in order to remove the existing fill soils, and a portion of the near-surface alluvial soils and possible fill materials, and replace these soils as compacted structural fill.

Extensive demolition of the existing structures, ASTs, pavements, and other site improvements will be required to facilitate the proposed development. Demolition of the existing foundations and floor slab will cause extensive disturbance to the near surface soils. Stripping of trees from the orchard areas will also cause significant disturbance to the near surface soils. The recommended remedial grading should also remove any soils disturbed during demolition and site stripping and replace them as compacted structural fill.

Settlement

The undocumented fills soils and the near-surface alluvial soils possess variable densities and will be subject to consolidation settlement upon loading. Through remedial grading of the unsuitable fill and near-surface alluvium, it is considered feasible to reduce the projected settlements of the soils in the proposed building areas to within tolerable limits.

Expansion

The near surface soils generally consist of sands and silty sands and sandy silts. The results of expansion index testing indicate that these soils possess very low expansion potentials (EI = 15 and 17). Based on these test results, no design considerations related to expansive soils are considered warranted for this site. We recommend that additional expansion index testing be performed during the design level geotechnical investigation in order to more thoroughly characterize the expansive potential of the near-surface soils at the subject site.

Preliminary Shrinkage/Subsidence Estimates

Based on the results of the laboratory testing, removal and recompaction of the loose to medium dense near-surface soils, is estimated to result in an average shrinkage of 6 to 12 percent. It should be noted that this shrinkage estimate is based on the results of dry density testing performed on small-diameter samples of the existing soils taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

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

These estimates are based on previous experience in the area of the subject site 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. The shrinkage and subsidence estimates should be refined at the time of the design-level geotechnical investigation.

Grading and Foundation Plan Review

Only conceptual grading plans were available at the time of this report. Additionally, no foundation plans have been prepared for the proposed structures. It is therefore recommended that we be provided with copies of the grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Preliminary Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development, which consists of either a new building or a new truck parking lot. These recommendations are general in nature, and should be confirmed as part of the design level geotechnical investigation.

Site Stripping and Demolition

Initial site stripping should include removal of any surficial vegetation. This should include any weeds, grasses, and shrubs. Any trees that will not remain with the proposed development should also be removed from the site. Any tree root systems should be removed in their entirety. These materials should be disposed of off-site. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered. Any soils disturbed during the removal of tree root systems should be removed and recompacted as structural fill.

The proposed development will require demolition of the existing buildings, AST, pavements, and other improvements. Any existing improvements that will not remain in place for use with the new development should be removed in their entirety. This should include all foundations, floor slabs, utilities, and any other subsurface improvements associated with the existing structures. The existing pavements are not expected to be reused with the new development. Debris resultant from demolition should be disposed of offsite. Concrete and asphalt debris may be re-used within compacted fills, provided they are pulverized to a maximum particle size of less than 2 inches, and thoroughly mixed with the on-site soils. Alternatively, existing asphalt and concrete materials may be crushed into miscellaneous base (CMB) and re-used at the site

Treatment of Existing Soils: Building Pads

Remedial grading will be necessary within the proposed building pad areas to remove the artificial fill soils in their entirety, any soils disturbed during demolition of the existing structures, any soils disturbed during stripping of the trees in the orchard, and a portion of the near-surface native alluvium. The depth of overexcavation should be determined during the design level geotechnical investigation. On a preliminary basis, overexcavation to depths of 3 to 5± feet the below existing and proposed building pad grades should be anticipated. Overexcavation within the foundation areas will likely extend to depths of 2 to 3± feet below foundation bearing grades. Additional overexcavation may also be necessary in localized areas where loose native alluvial soils are exposed at the overexcavation bottoms. Loose native alluvial soils were encountered to depths of us to 12± feet at Boring No. B-1. Any possible fill soils remaining at overexcavation subgrades should be evaluated at the time of site grading to determine if these materials consist should be overexcavated. The discovery of any adverse geotechnical conditions encountered during the design level investigation could result in deeper recommended overexcavation depths.

Based on conditions encountered at the exploratory boring locations, some zones of moist to very moist clayey soils may be encountered at or near the base of the recommended overexcavation. Scarification and air drying of these materials may be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying additional overexcavation may be performed to replace these materials with drier, on-site granular soils.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, moisture treated to within 2 to 4 percent above the optimum moisture content, and recompacted. The previously excavated soils may then be replaced as compacted structural fill.

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 surface grade. Overexcavation will also be necessary in these areas to remove any existing fill soils and lower strength alluvium. The overexcavation depth should be expected to be on the order of 2 to 3± feet below proposed foundation bearing grade and to a depth sufficient to remove any undocumented fill or soils disturbed during demolition or site stripping.

Treatment of Existing Soils: Parking and Drive Areas

Based on economic considerations, overexcavation of the existing near-surface existing soils in the 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. Preliminarily, subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations.

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

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

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.

- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2019 CBC and the grading code of the city of Fullerton.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by the city of Fullerton. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of silty sands, fine sandy silts, and sands, as well as sandy clays and silty clays. Some of these materials will likely be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes should be made no steeper than 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

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

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

Groundwater

The static groundwater table at this site is considered to be present at a depth in excess of 30± feet. Therefore, groundwater is not expected to impact 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 buildings will be underlain by newly placed structural fill soils, extending to depths of at least 2 to 3 feet below foundation bearing grades. Based on this subsurface profile, the proposed structures 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: 2,000 to 3,000 lbs/ft^2.
- Minimum longitudinal steel reinforcement within strip footings: Two (2) to four (4) No. 5 rebars.

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 slabs and the passive earth pressure developed by footings below grade. The following friction and passive pressure may be used to resist lateral forces:

- Passive Earth Pressure: 275 - 325 lbs/ft^3
- Friction Coefficient: 0.28 to 0.30

6.6 Preliminary Floor Slab Design and Construction

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

- Minimum slab thickness: 5 to 6 inches.

- Modulus of Subgrade Reaction: k = 100 to 150 psi/in.

- Minimum slab reinforcement: Not required based on geotechnical considerations. Additional expansion index testing should be performed to confirm this recommendation at the time of the design level investigation. 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 which will incorporate such coverings. 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.

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

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

6.7 Preliminary Retaining Wall Design and Construction

Small retaining walls are expected to be necessary in the dock-high areas of the buildings 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.

Preliminary Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near-surface soils suitable for retaining wall backfill generally consist of silty fine sands, fine sandy silts and sands. Based on their classifications, these materials are expected to possess a friction angle of at least 29 degrees when compacted to 90 percent of the ASTM-1557 maximum dry density. The on-site sandy clays, silty clays, and clayey silts likely possess lower shear strength parameters and should not be used as retaining wall backfill.

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

PRELIMINARY RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type
		On-Site Sandy and Silty Soils
Internal Friction Angle (ϕ)		29°
Unit Weight		130 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	45 lbs/ft^3
	Active Condition (2h:1v backfill)	75 lbs/ft^3
	At-Rest Condition (level backfill)	70 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

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 2019 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.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 2 to 3 feet below the proposed bearing grade. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Backfill Material

On-site sands and silty sands may be used to backfill the retaining walls. However, all backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. 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.

Subsurface Drainage

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

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

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

6.8 Preliminary Pavement Design Parameters

Presented below are preliminary recommendations for pavements that may be required in the proposed development. Grading recommendations for these pavement areas should be developed during the design level geotechnical investigation.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of silty sands, sands, and sandy silts with silty clay, sandy clay, and clayey silt layers. Based on their classification, these materials are expected to possess fair pavement support characteristics, with R-values in the range of 30 to 40. Since R-value testing was not included in the scope of services for this feasibility study, the subsequent pavement design is based upon an assumed R-value of 30. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering-controlled conditions. It is recommended that R-value testing be performed during the design level geotechnical investigation, or at the completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

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

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

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

ASPHALT PAVEMENTS (R = 30)					
Materials	**Thickness (inches)**				
	Automobile Parking (TI = 4.0)	Automobile Drive Lanes (TI = 5.0)	Truck Traffic (TI = 6.0)	Truck Traffic (TI = 7.0)	Truck Traffic (TI = 8.0)
Asphalt Concrete	3	3	3½	4	5
Aggregate Base	3	6	8	10	11
Compacted Subgrade (90% minimum compaction)	12	12	12	12	12

The aggregate base course should be compacted to at least 95 percent of the ASTM D-1557 maximum dry density. The asphaltic concrete should be compacted to at least 95 percent of the Marshall maximum density, as determined by ASTM D-2726. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and

Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30)				
Materials	**Thickness (inches)**			
	Auto Parking & Drives (TI = 5.0)	Truck Traffic		
		(TI = 6.0)	(TI = 7.0)	(TI = 8.0)
PCC	5	5½	6	7
Compacted Subgrade (95% Relative 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.

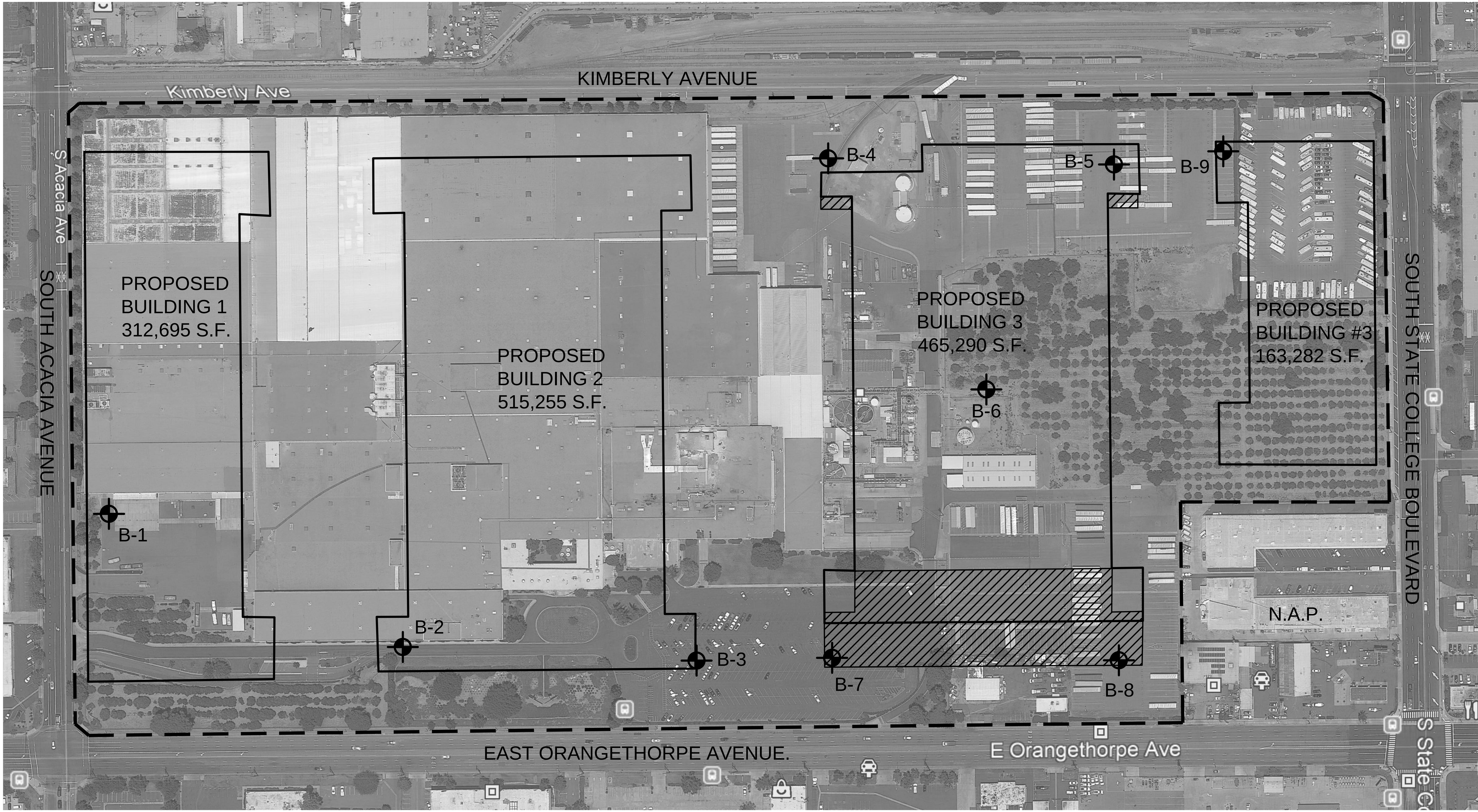
This report has been based on assumed or provided characteristics of the proposed development. It is recommended that the owner, client, architect, structural engineer, and civil engineer carefully review these assumptions to ensure that they are consistent with the characteristics of the proposed development. 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: ORANGE COUNTY
THOMAS GUIDE, 2013

SITE LOCATION MAP	
GOODMAN LOGISTICS CENTER FULLERTON	
FULLERTON, CALIFORNIA	
SCALE: 1" = 2400'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JH CHKD: RGT	
SCG PROJECT 19G139-1R	
PLATE 1	



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

OPTIONAL EXTENSION OF BUILDING 3

NORTH
SoCalGeo

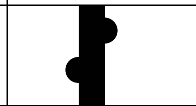
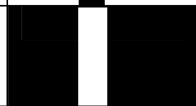
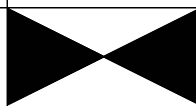
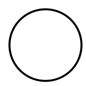
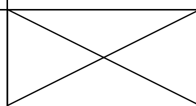
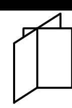
NOTE: AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH. CONCEPTUAL SITE PLAN PROVIDED BY THE CLIENT.

BORING LOCATION PLAN
GOODMAN LOGISTICS CENTER FULLERTON
FULLERTON, CALIFORNIA

SCALE: 1" = 180'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JH CHKD: DN	
SCG PROJECT 19G139-1R	
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 GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 19G139
PROJECT: Proposed C/I Development
LOCATION: Fullerton, California

DRILLING DATE: 7/26/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					3± inches Asphaltic Concrete, 7± inches Aggregate Base							
	X	14			FILL: Dark Gray Brown fine Sandy Silt to Silty fine Sand, little Clay, loose-damp to moist	117	12					
	X	14	3.5		POSSIBLE FILL: Dark Brown Clayey fine Sand to fine Sandy Clay, medium dense/very stiff-very moist	103	19					
5	X	8			ALLUVIUM: Brown Silty fine Sand, loose-very moist	98	14					
	X	9			Light Gray Brown fine to medium Sand, loose-dry	97	2					
10	X	13				97	1					
					Light Gray fine Sand, trace Iron oxide staining, loose-dry							
					Light Gray fine to coarse Sand, trace fine Gravel, medium dense-dry							
15	X	37				103	2					
					Brown fine Sandy Silt, medium dense-moist							
20	X	17				104	12					
					Brown Silty fine Sand, medium dense-moist							
25	X	23				108	8					
					Boring Terminated at 25'							

TBL 19G139.GPJ SOCALGEO.GDT 8/9/19

BORING NO. B-2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 19G139
PROJECT: Proposed C/I Development
LOCATION: Fullerton, California

DRILLING DATE: 7/26/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
				3± inches Asphaltic Concrete, No discernable Aggregate Base							
	X	16	4.0	FILL: Dark Gray Brown fine Sandy Clay, slight hydrocarbon odor, stiff- very moist		17					
			3.0	POSSIBLE FILL: Brown fine Sandy Clay to Silty Clay, trace Iron oxide staining, very stiff-very moist		19					
	X	6		ALLUVIUM: Light Brown Silty fine Sand, loose-moist		11					
5											
	X	8		Light Gray Brown fine to medium Sand, trace fine Gravel, loose to medium dense-dry		1					
	X	11				1					
10											
	X	19				2					
15											
	X	24				1					
20				Boring Terminated at 20'							

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A California Corporation

BORING NO. B-3

JOB NO.: 19G139
PROJECT: Proposed C/I Development
LOCATION: Fullerton, California

DRILLING DATE: 7/26/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					3½± inches Asphaltic Concrete, No discernable Aggregate Base							
		14	1.0		FILL: Dark Brown Clayey fine Sand to fine Sandy Clay, some Sitly fine to coarse Sand, loose-very moist		16					@ 0-1' no recovery in sampler, grab sample from auger spoils
					ALLUVIUM: Light Gray Brown fine to coarse Sand, loose to medium dense-dry to damp							
		13				110	2					
5		19				107	3					
		24				104	3					
		34				103	3					
10												
					Gray to Gray Brown fine Sand, trace medium Sand, medium dense-damp to moist							
		19					6					
15												

Boring Terminated at 15'

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A California Corporation

BORING NO. B-4

JOB NO.: 19G139	DRILLING DATE: 7/26/19	WATER DEPTH: Dry
PROJECT: Proposed C/I Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 12 feet
LOCATION: Fullerton, 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
					4± inches Asphaltic Concrete, 6± inches Aggregate Base							
		22			FILL: Gray Brown Silty fine to coarse Sand, little fine to medium Gravel, trace Asphalitic concrete fragments, medium dense-moist to very moist	117	9					
		23			ALLUVIUM: Light Gray to Light Gray Brown, fine to coarse Sand, trace Iron oxide staining, medium dense-dry	108	2					
5		23					1					@ 5-6' Disturbed Sample
		18			@ 7' trace fine Gravel	107	2					
		26			Gray fine Sand, trace medium Sand, medium dense-dry	101	1					
10												
					Brown Silty fine Sand to fine Sandy Silt, trace Iron oxide staining, medium dense-very moist							
		14					15					
15					Light Gray Brown fine to coarse Sand, trace Clay nodules, trace fine Gravel, medium dense-moist							
		27					8					
20												
					Boring Terminated at 20'							

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SOUTHERN CALIFORNIA GEOTECHNICAL
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BORING NO. B-5

JOB NO.: 19G139	DRILLING DATE: 7/26/19	WATER DEPTH: Dry
PROJECT: Proposed C/I Development	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 16 feet
LOCATION: Fullerton, California	LOGGED BY: Jamie Hayward	READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					4± inches Asphaltic Concrete, 6± inches Aggregate Base							
	X	5	2.0		FILL: Brown fine to coarse Sandy Clay, trace fine Gravel, trace Asphaltic concrete fragments, soft-moist		11					EI = 17 @ 0-5'
	X	5			ALLUVIUM: Gray Brown Silty fine Sand, loose-moist		9					
5												
	X	10			Light Gray Brown fine to medium Sand, trace coarse Sand, trace Iron oxide staining, loose to medium dense-moist		4					
	X	15			@ 8½ feet trace fine to coarse Gravel, little coarse Sand		4					
10												
					Gray Brown Silty fine Sand, trace Clay, medium dense-moist							
	X	12					13					
15			2.5		Brown Silty Clay, trace to little fine Sand, trace Iron oxide staining, stiff-very moist		23					
					Light Brown fine Sand, trace to little Silt, medium dense-damp							
	X	15					7					
20												
					Gray Brown fine Sandy Clay, little Silt, trace Iron oxide staining, medium dense-very moist							
	X	17	2.75				17					
25												
					Boring Terminated at 25'							

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SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 19G139
PROJECT: Proposed C/I Development
LOCATION: Fullerton, California

DRILLING DATE: 7/26/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM: Gray Brown Silty fine Sand, trace Fine root fibers, slightly porous, loose-damp							
		15				96	3					
		10			@ 3 to 4 feet porous	97	3					
5					Red Brown to Dark Brown Silty fine to medium Sand, trace to little coarse Sand, trace to little Clay, medium dense-dry							
		21				106	2					
		19				103	1					
		19				106	2					
10					Light Gray fine to coarse Sand, medium dense-dry to damp							
		16					3					
15												

Boring Terminated at 15'

TBL 19G139.GPJ SOCALGEO.GDT 8/9/19

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 19G139
PROJECT: Proposed C/I Development
LOCATION: Fullerton, California

DRILLING DATE: 7/26/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					3± inches Asphaltic Concrete, 7± inches Aggregate Base							
	X	5	2.0		POSSIBLE FILL: Dark Gray Brown, fine Sandy Clay and Silty Clay, medium stiff-very moist		19					EI = 15 @ 0 to 5'
					Brown Silty fine to medium Sand, very loose to loose-moist							
	X	4			Light Gray fine to medium Sand, very loose to loose-moist		10 9					
5												
	X	16			Light Gray fine Sand, medium dense-damp		6					
	X	17			Light Gray fine to medium Sand, trace coarse Sand, medium dense-damp to moist		3					
10												
	X	13			@ 13½ feet trace fine Gravel		6					
15												
					Light Gray Brown fine Sand, trace to little Silt, medium dense-moist							
	X	16					9					
20												
					Light Gray Brown to Light Brown, fine to medium Sand, trace coarse Sand, medium dense-damp to moist							
	X	18					8					
25												
					Boring Terminated at 25'							

TBL 19G139.GPJ SOCALGEO.GDT 8/9/19

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 19G139
PROJECT: Proposed C/I Development
LOCATION: Fullerton, California

DRILLING DATE: 7/26/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

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

SURFACE ELEVATION: --- MSL

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
				3± inches Asphaltic Concrete, 6± inches Aggregate Base							
	X	27	4.5	FILL: Dark Gray Brown to Brown fine to medium Sandy Clay, trace fine to coarse Gravel, trace Asphaltic concrete fragments, mottled, very stiff-moist to very moist	118	13					
	X	16	4.5	POSSIBLE FILL: Brown Clayey fine Sand to fine Sandy Clay, medium dense/very stiff-moist to very moist	110	15					
5	X	20		ALLUVIUM: Light Brown fine to medium Sand, trace coarse Sand, medium dense-dry to damp	98	3					
	X	11		@ 7 to 7½ feet loose	107	1					
				Interbedded lenses of Gray Brown Clayey fine to medium Sand and Brown fine Sandy Clay, medium stiff-moist to very moist							
	X	21		Gray fine Sand, trace Iron oxide staining, medium dense-dry	97	2					
10				Light Gray to Light Gray Brown fine to coarse Sand, trace to little fine Gravel, medium dense-dry							
15	X	19				2					
	X	22				2					
20				Boring Terminated at 20'							

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SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

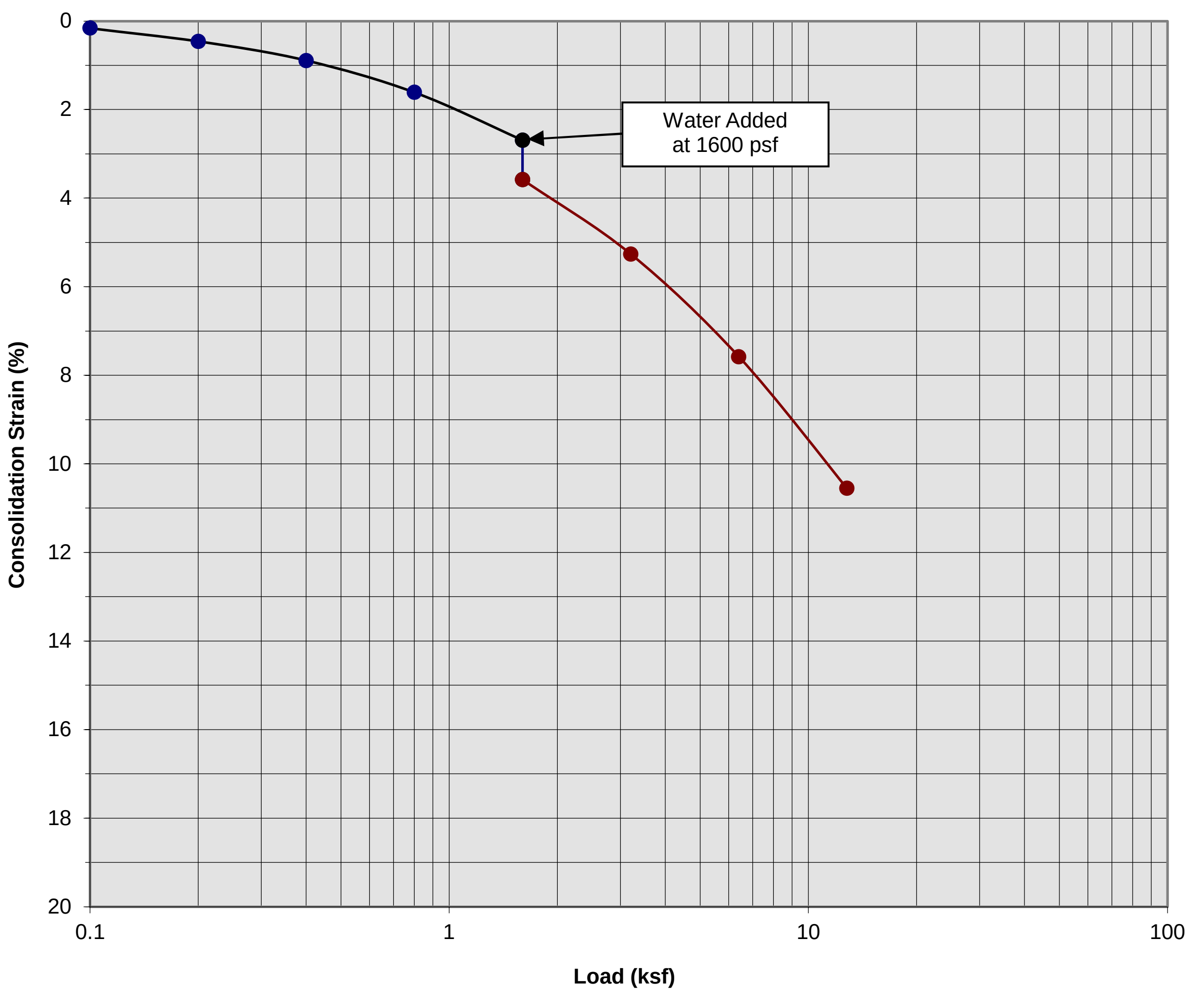
JOB NO.: 19G139
PROJECT: Proposed C/I Development
LOCATION: Fullerton, California
DRILLING DATE: 7/26/19
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward
WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					4± inches Asphaltic Concrete, 6± inches Aggregate Base							
	X	12	4.0		FILL: Red Brown to Brown fine Sandy Clay, some Brick fragments, very stiff-very moist	105	14					
					ALLUVIUM: Brown Silty fine Sand, loose-moist							
	X	9				98	10					
					Light Gray Brown fine to medium Sand, trace Iron oxide staining, loose to medium dense-dry							
5	X	17			@ 5 feet trace coarse Sand	90	3					
	X	17			Light Gray fine Sand, trace Iron oxide staining, medium dense-damp	92	1					
	X	26			Light Gray fine to coarse Sand, medium dense-dry	100	2					
10												
					Gray Brown Silty fine Sand, medium dense-moist							
	X	23				91	7					
15												
					Gray fine Sand, medium dense-damp							
	X	26				91	3					
20												
					Boring Terminated at 20'							

TBL 19G139.GPJ SOCALGEO.GDT 8/9/19

APPENDIX C

Consolidation/Collapse Test Results



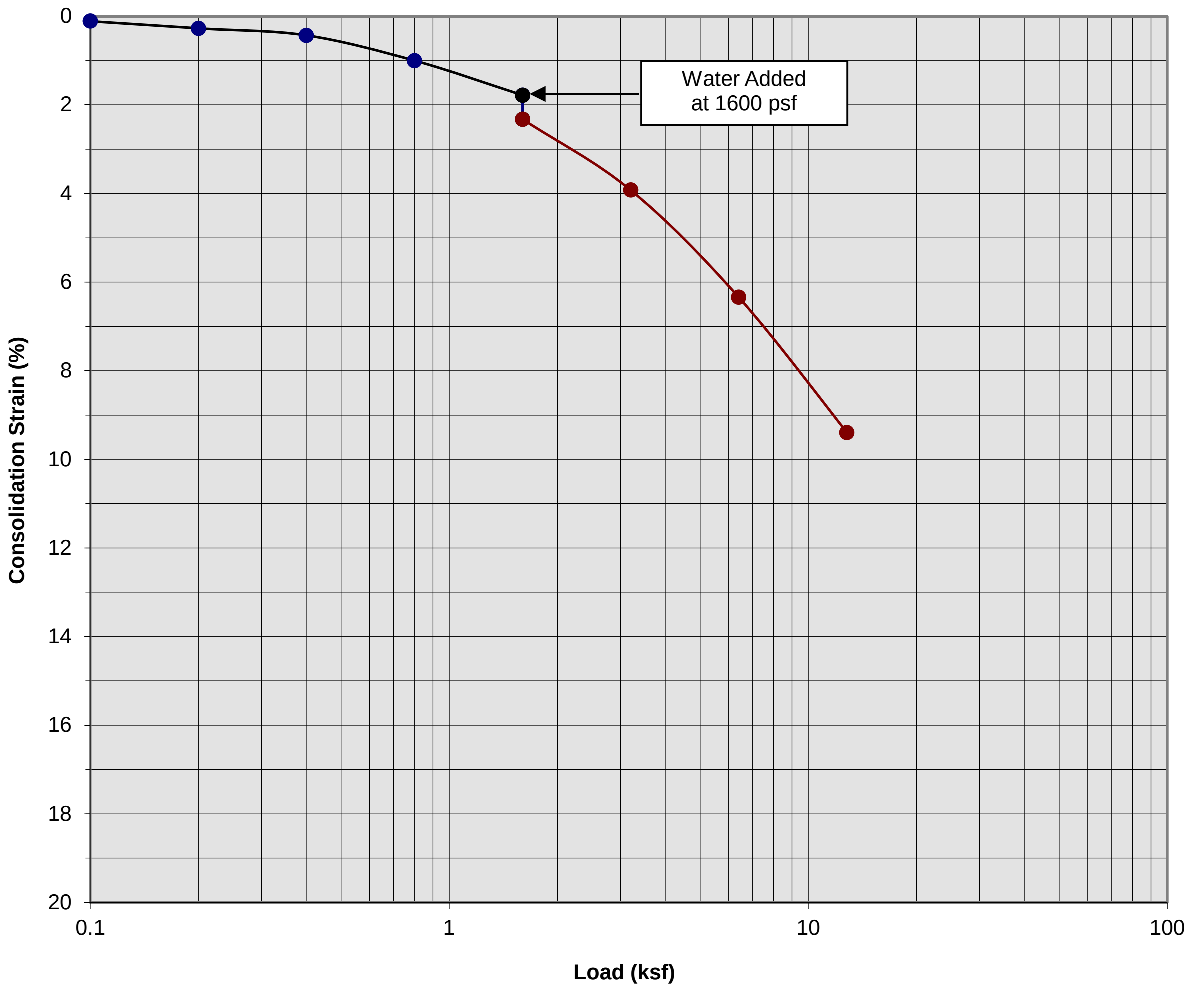
Classification: FILL: Dark Brown fine Clayey fine Sand to fine Sandy Clay

Boring Number:	B-1	Initial Moisture Content (%)	19
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	3 to 4	Initial Dry Density (pcf)	99.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.89

Proposed Goodman Logistics Center Fullerton
Fullerton, California
Project No. 19G139-1R
PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

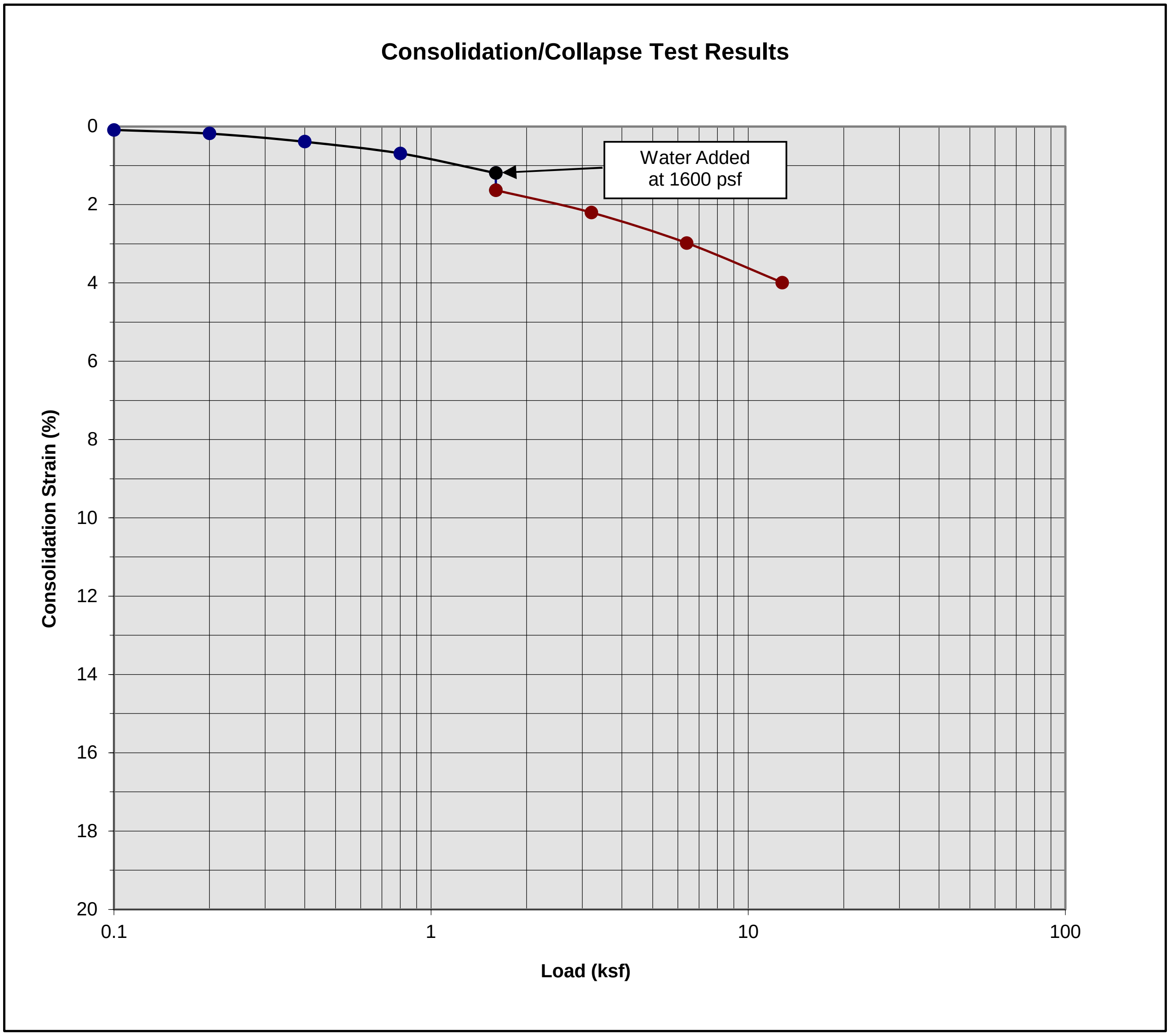


Classification: ALLUVIUM: Brown Silty fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	5 to 6	Initial Dry Density (pcf)	98.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.54

Proposed Goodman Logistics Center Fullerton
Fullerton, California
Project No. 19G139-1R
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

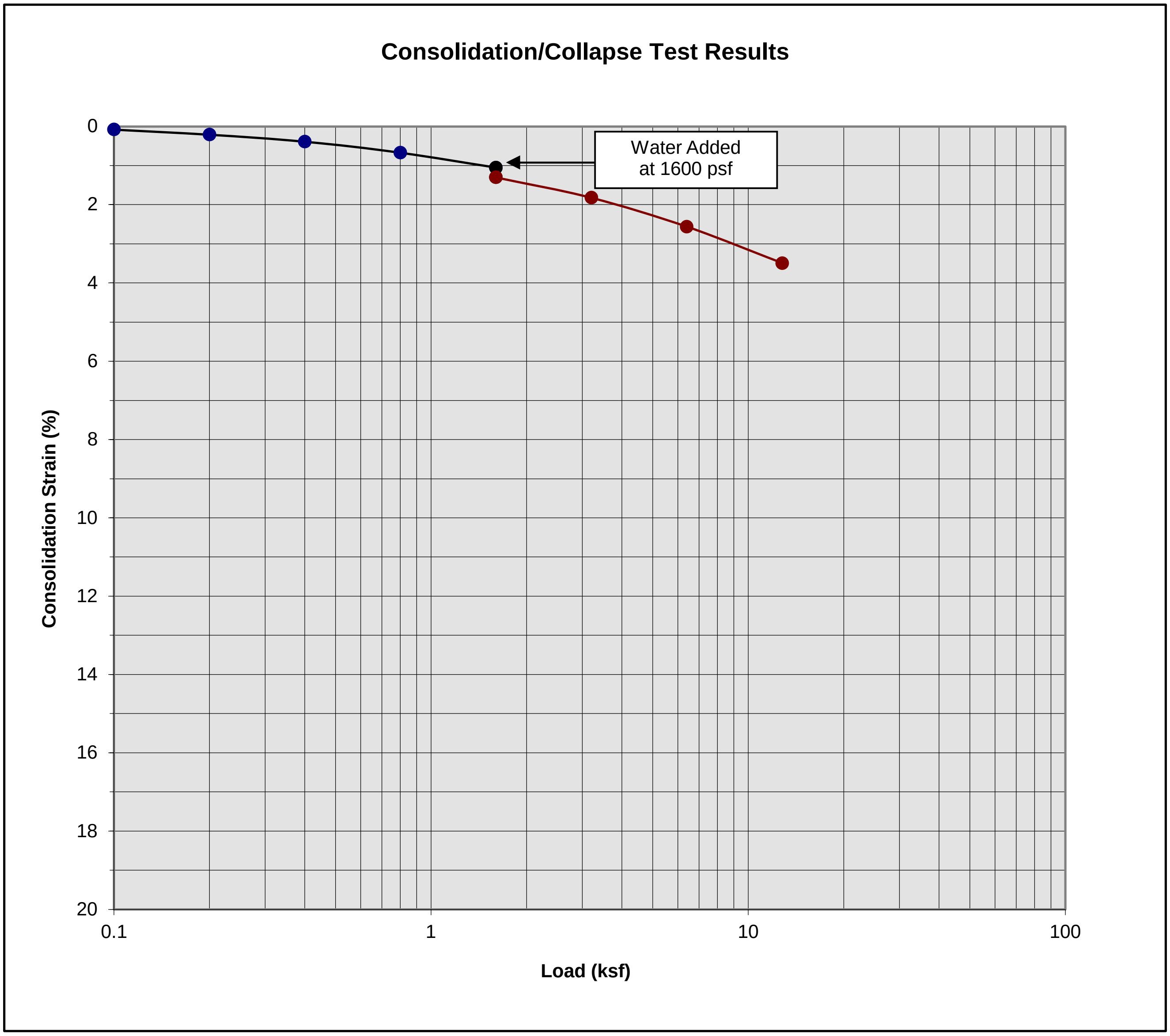


Classification: Light Gray Brown fine to medium Sand

Boring Number:	B-1	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	7 to 8	Initial Dry Density (pcf)	96.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	99.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.44

Proposed Goodman Logistics Center Fullerton
Fullerton, California
Project No. 19G139-1R
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation

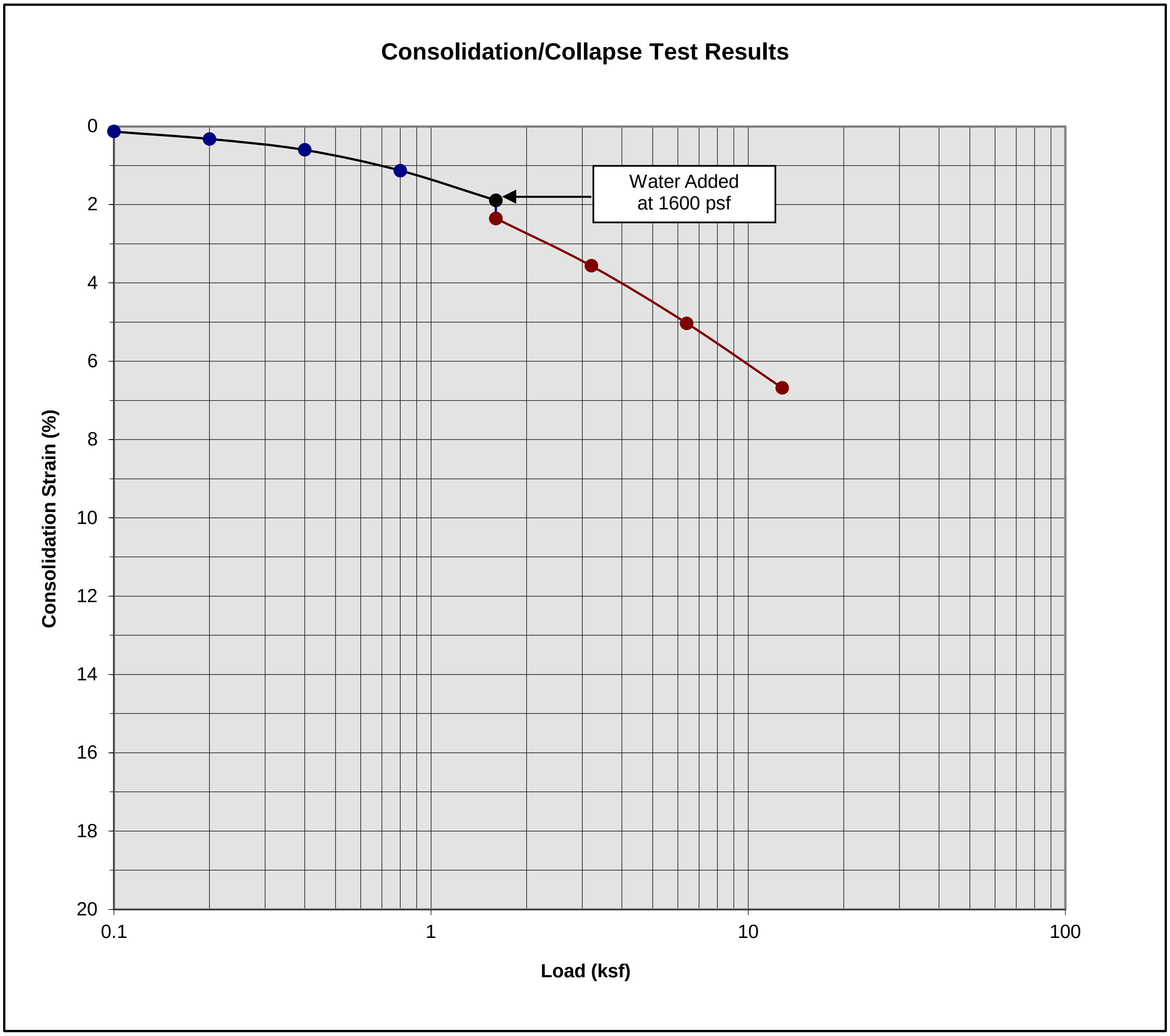


Classification: Light Gray fine Sand

Boring Number:	B-1	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	9 to 10	Initial Dry Density (pcf)	97.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	101.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.25

Proposed Goodman Logistics Center Fullerton
Fullerton, California
Project No. 19G139-1R
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

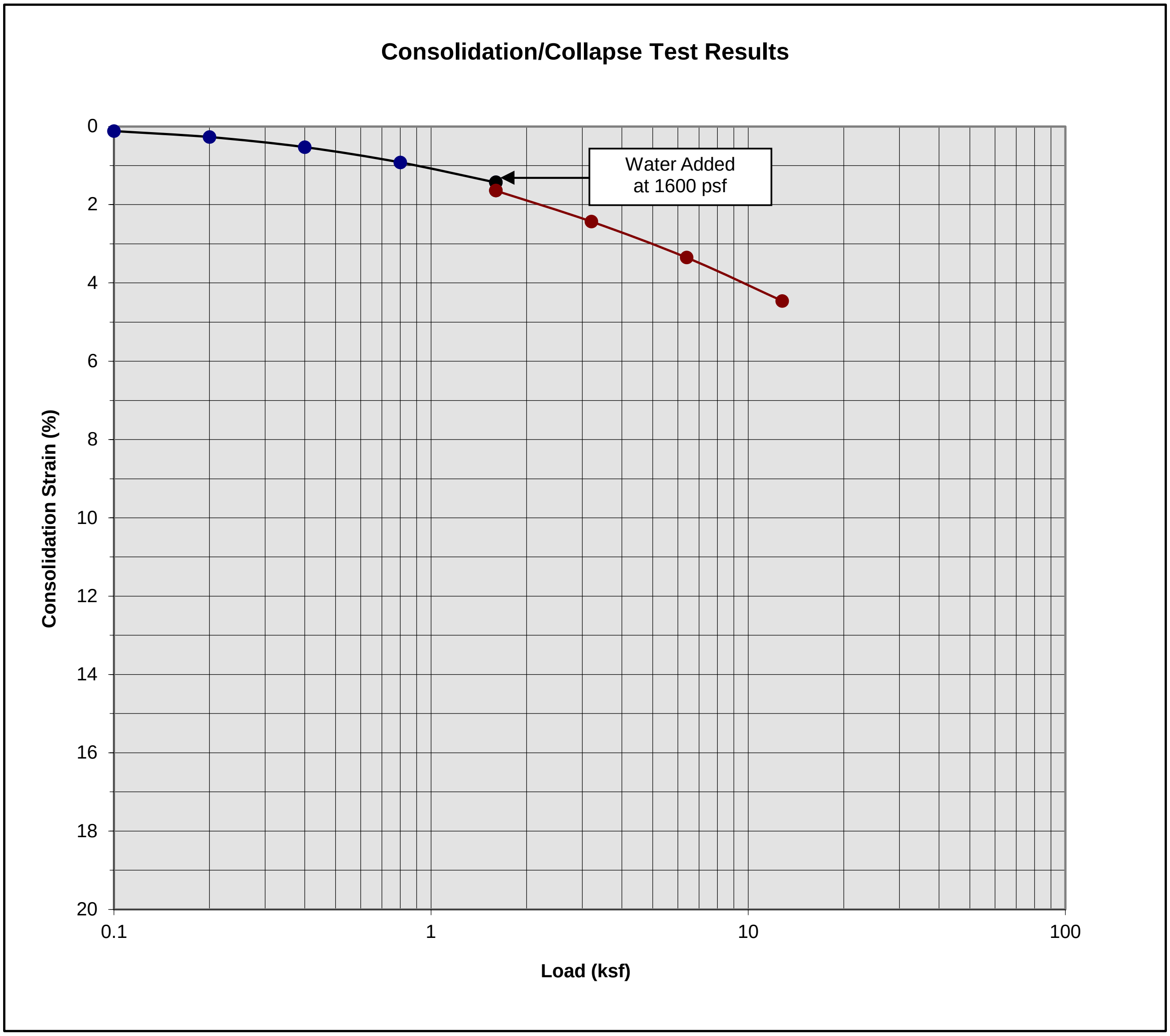


Classification: ALLUVIUM: Light Gray Brown fine to coarse Sand

Boring Number:	B-3	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	3 to 4	Initial Dry Density (pcf)	109.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	116.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.46

Proposed Goodman Logistics Center Fullerton
Fullerton, California
Project No. 19G139-1R
PLATE C- 5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



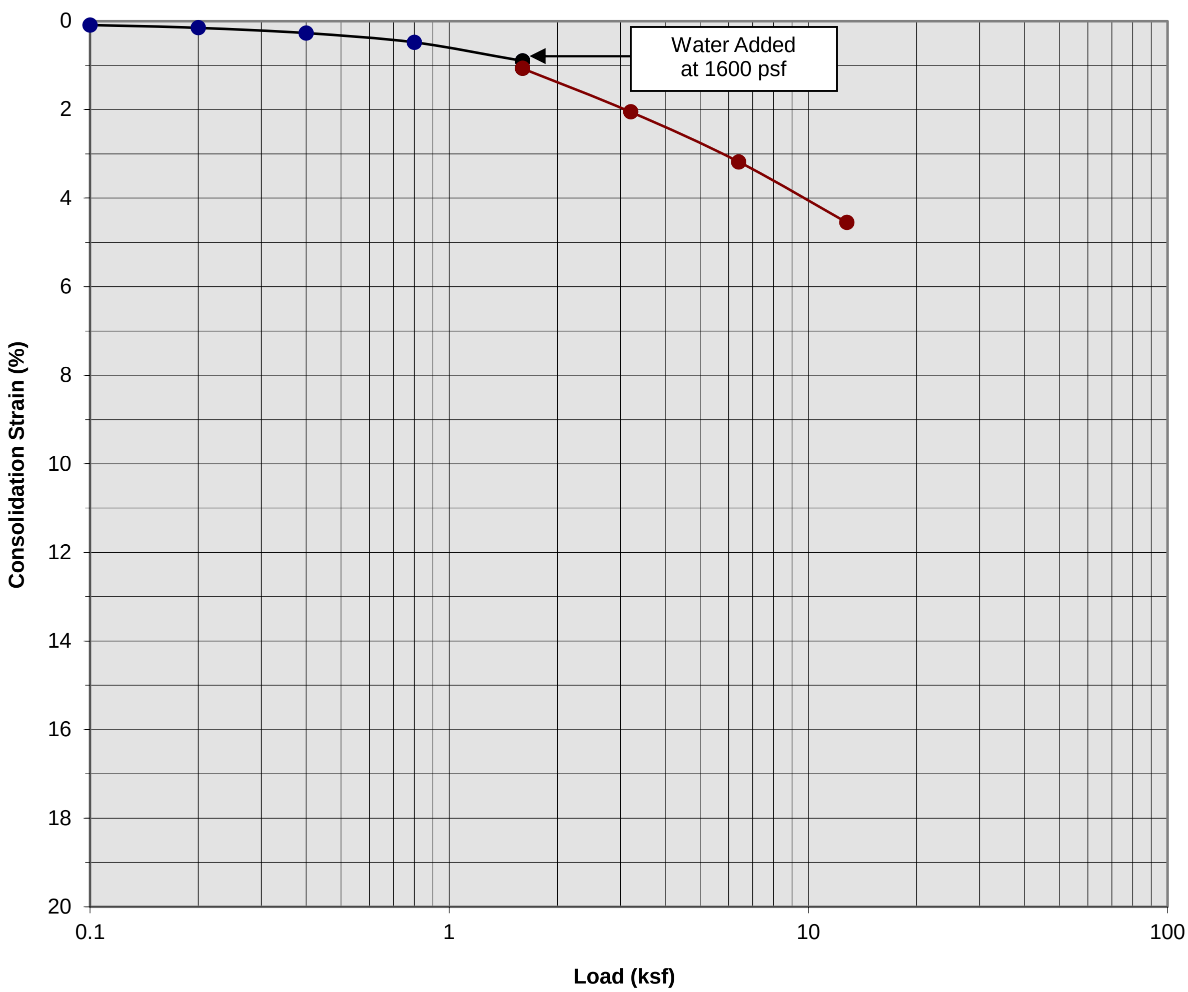
Classification: Light Gray Brown fine to coarse Sand

Boring Number:	B-3	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	13
Depth (ft)	5 to 6	Initial Dry Density (pcf)	107.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.21

Proposed Goodman Logistics Center Fullerton
Fullerton, California
Project No. 19G139-1R
PLATE C- 6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

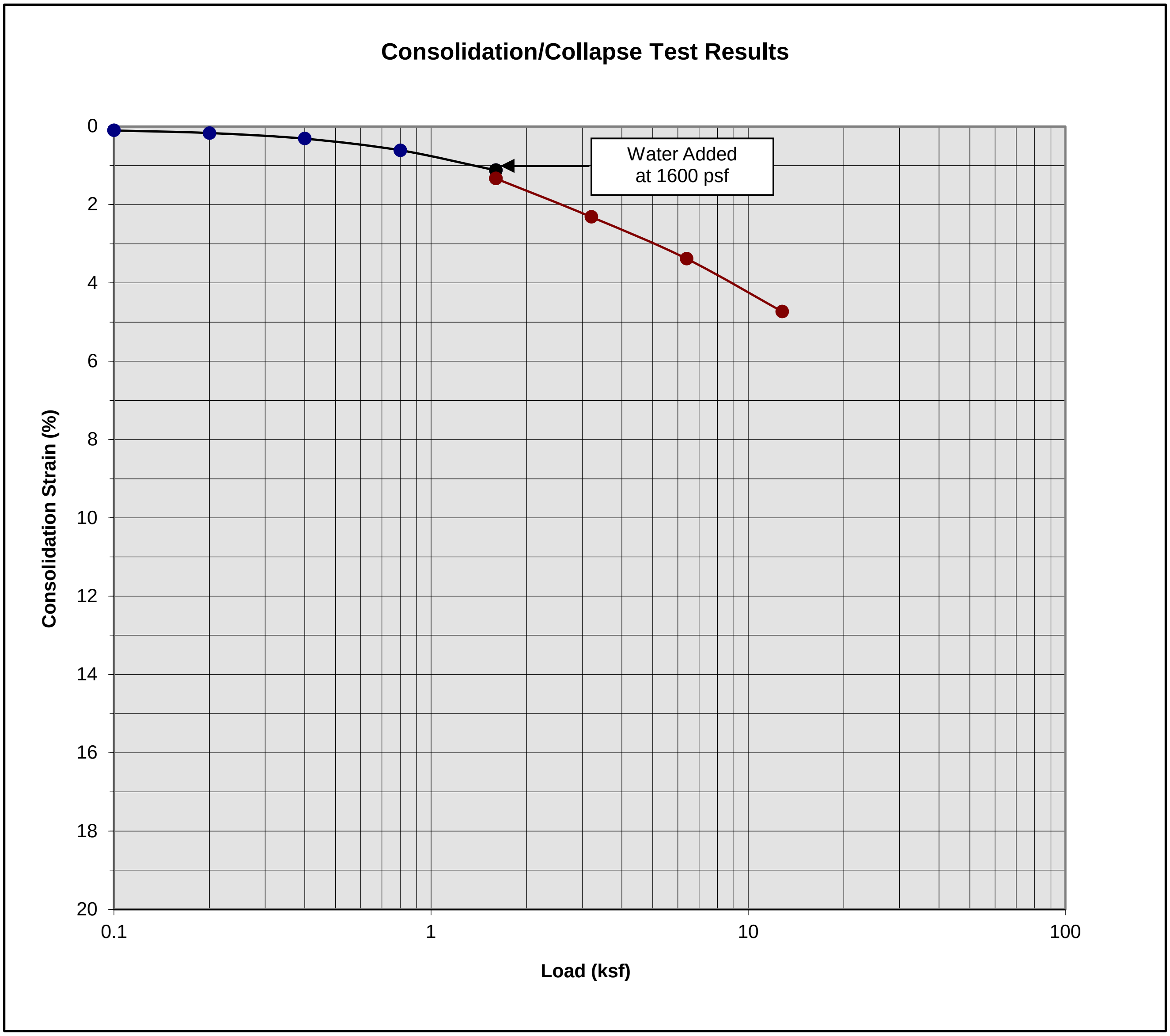


Classification: Light Gray Brown fine to coarse Sand

Boring Number:	B-3	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	7 to 8	Initial Dry Density (pcf)	103.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.17

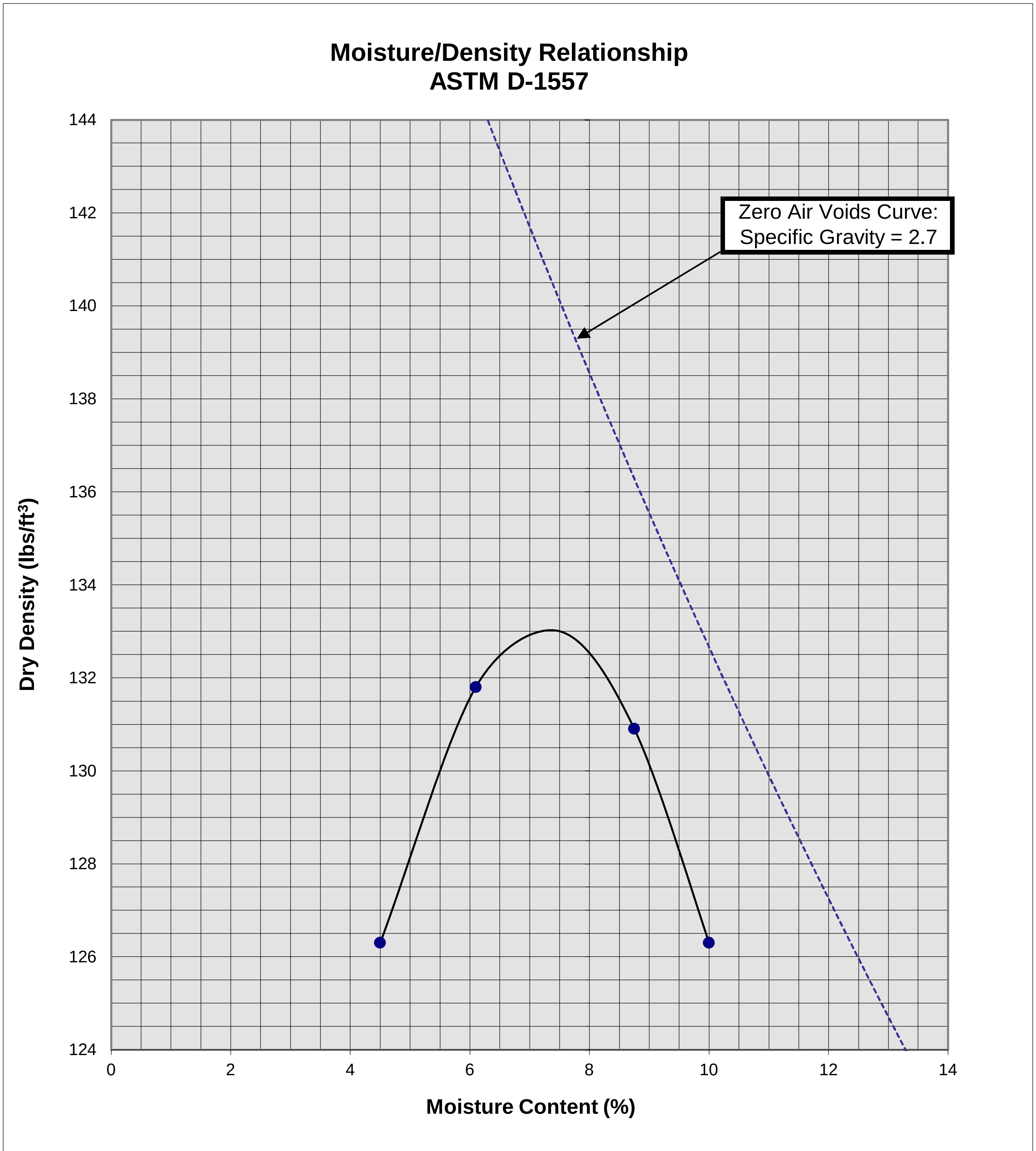
Proposed Goodman Logistics Center Fullerton
Fullerton, California
Project No. 19G139-1R
PLATE C- 7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Classification: Light Gray Brown fine to coarse Sand

Boring Number:	B-3	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	9 to 10	Initial Dry Density (pcf)	103.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	107.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.21



Soil ID Number	B-4 @ 0 to 5'
Optimum Moisture (%)	7.5
Maximum Dry Density (pcf)	133
Soil Classification	Light Gray Brown fine to coarse Sand, little Silt, little fine Gravel

Proposed Goodman Logistics Center Fullerton
Fullerton, California
Project No. 19G139-1R
PLATE C-9

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
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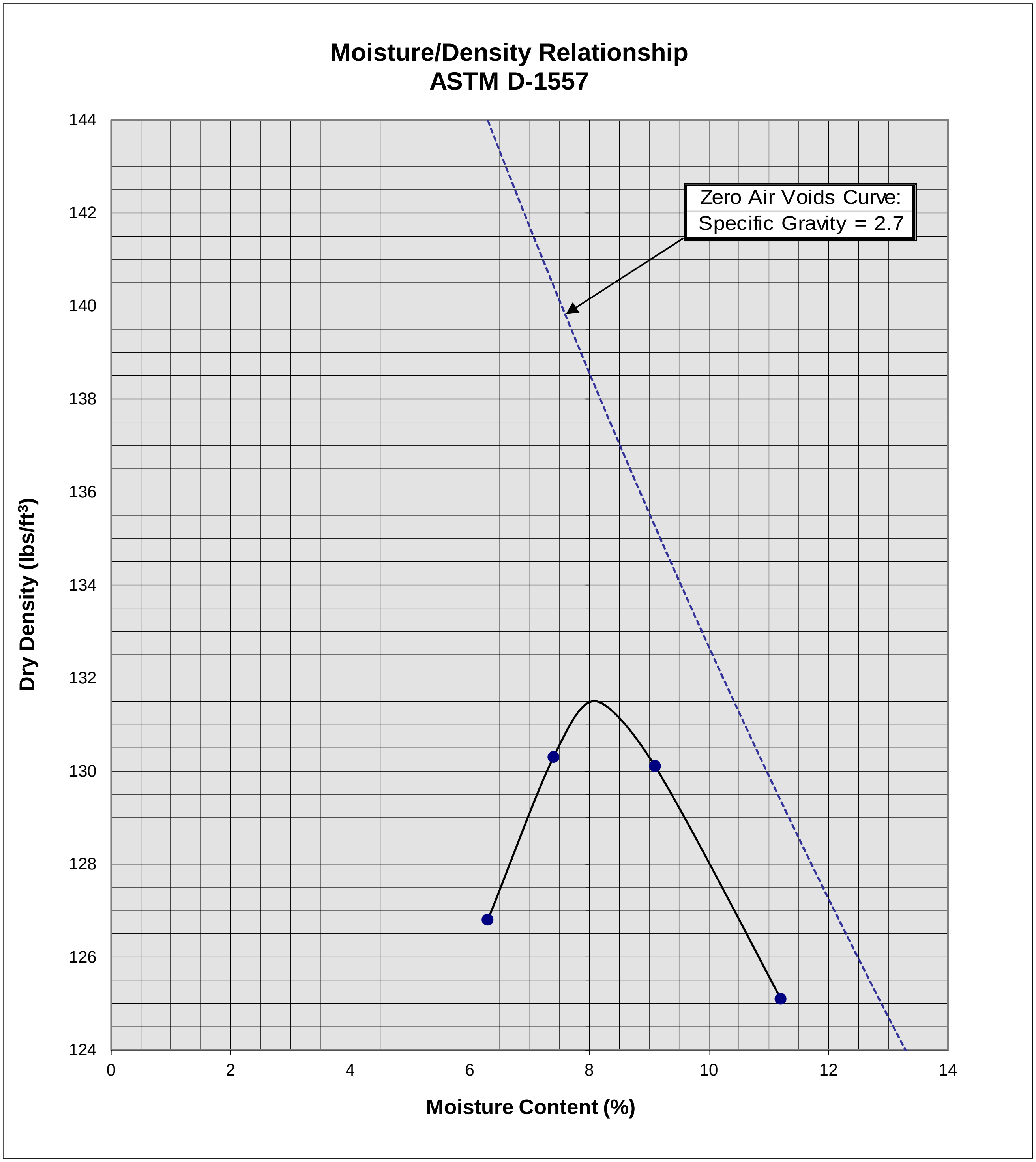


Soil ID Number	B-9 @ 0 to 5'
Optimum Moisture (%)	8
Maximum Dry Density (pcf)	131.5
Soil Classification	Gray Brown Silty fine to medium Sand, trace Brick fragments, little Clay

Proposed Goodman Logistics Center Fullerton
Fullerton, California
Project No. 19G139-1R
PLATE C-10

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

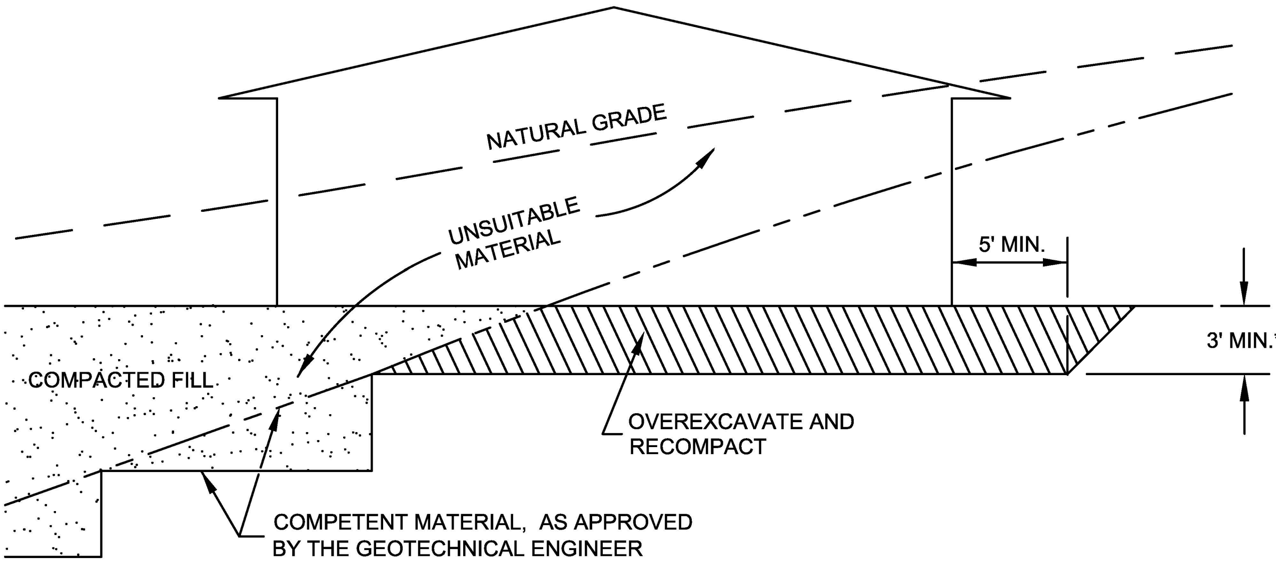
Subdrains

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

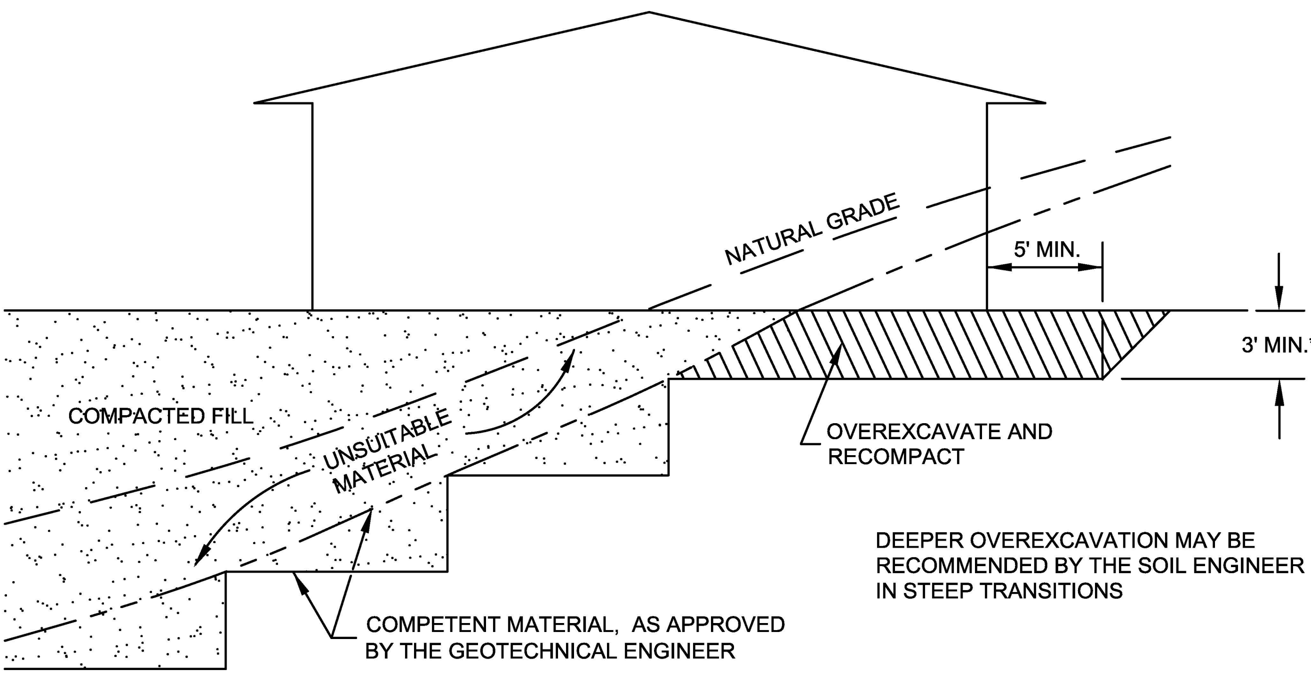
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



CUT/FILL LOT (TRANSITION)



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

TRANSITION LOT DETAIL

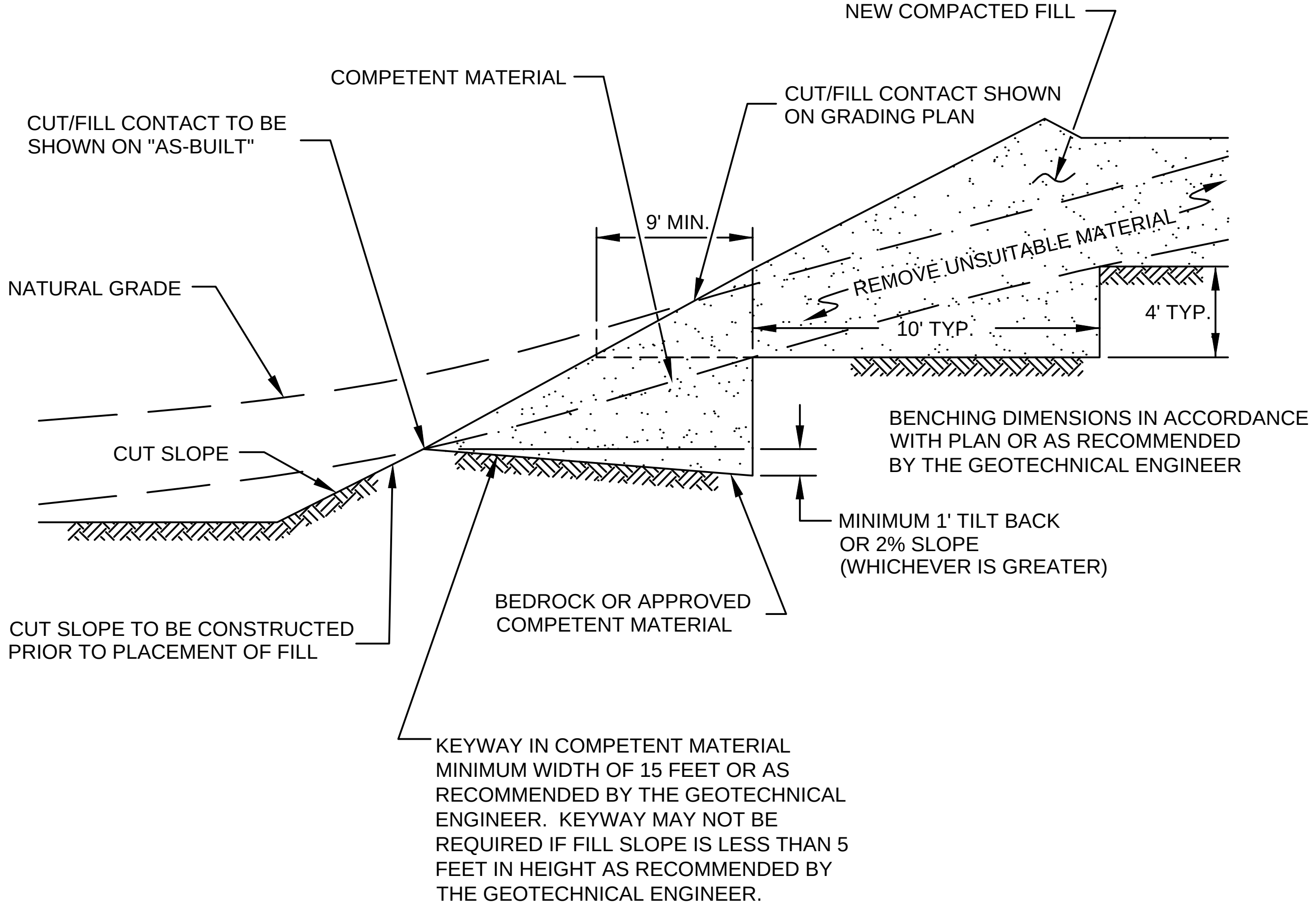
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

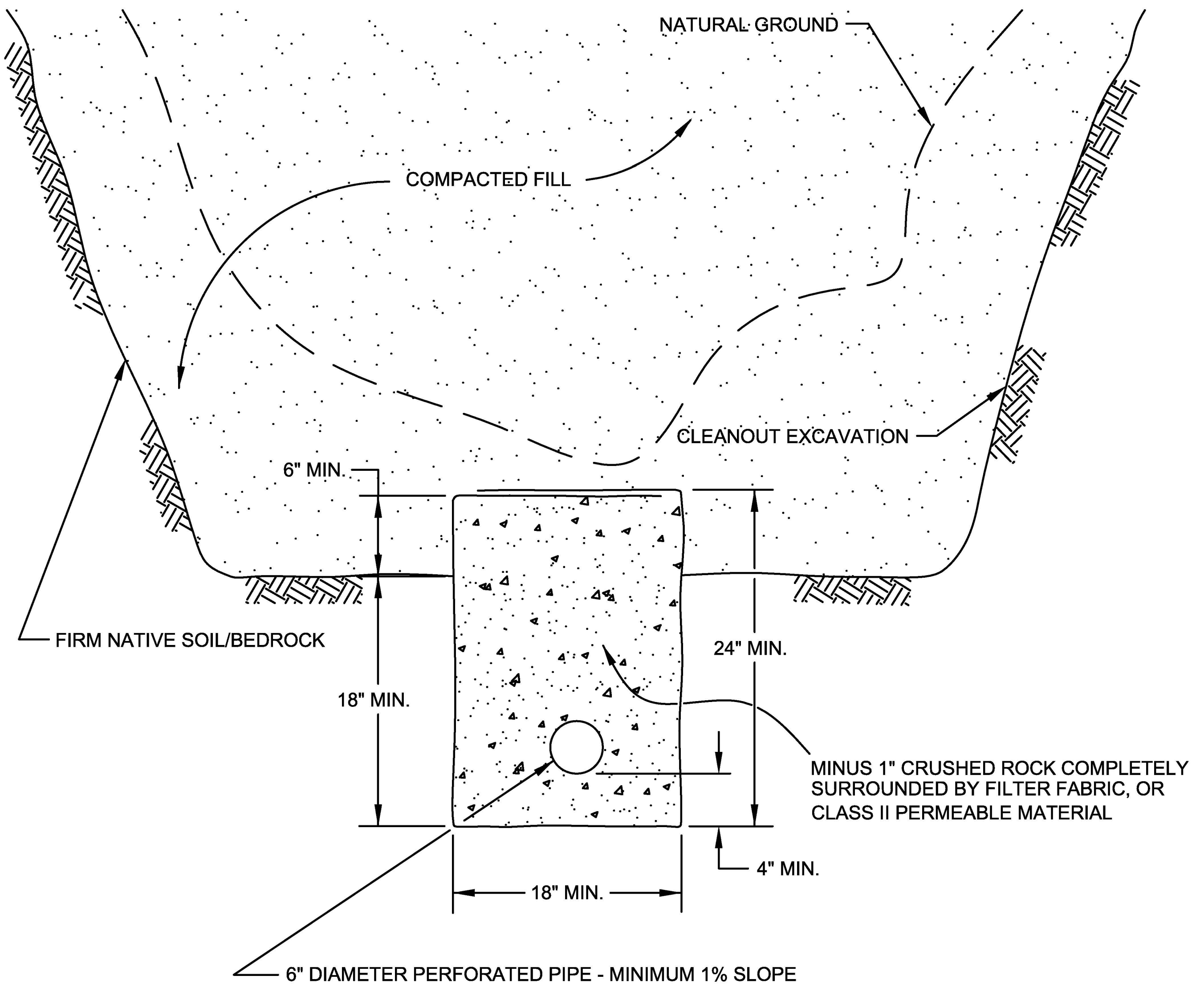
GRADING GUIDE SPECIFICATIONS

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

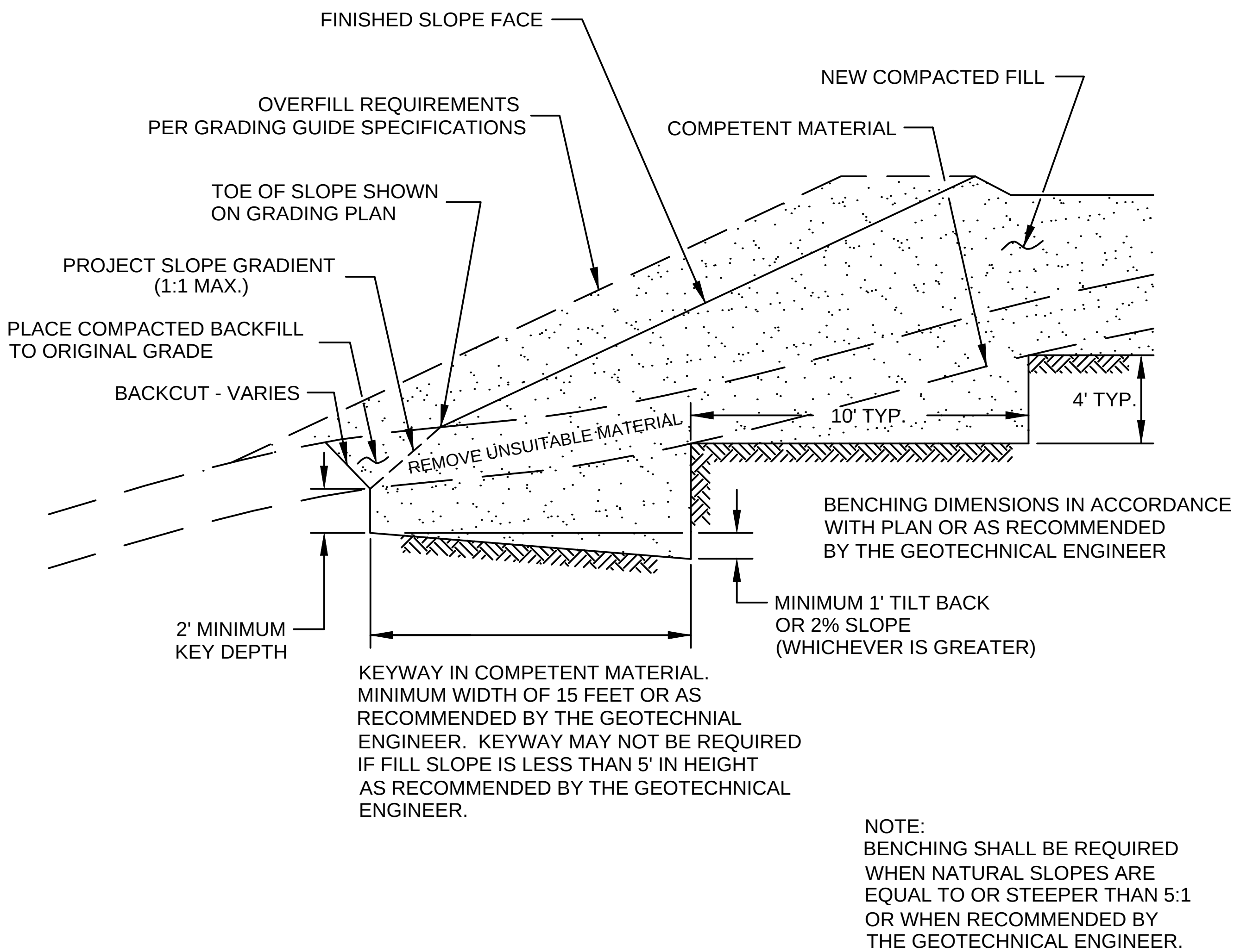
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

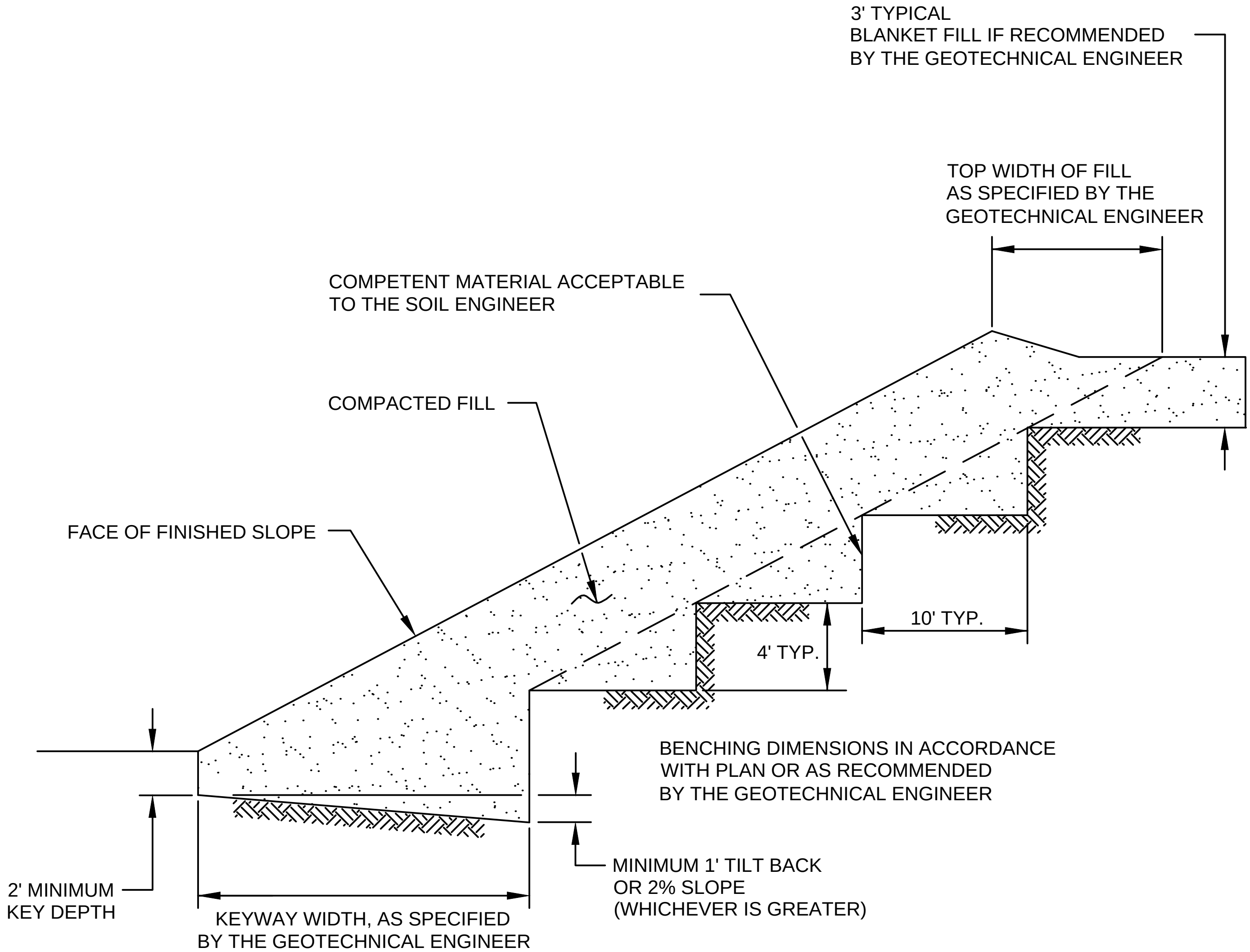
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

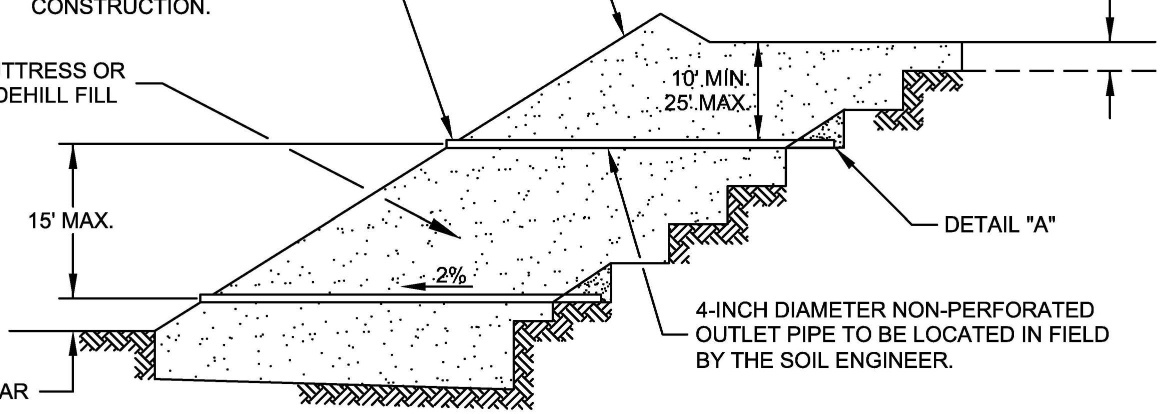
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



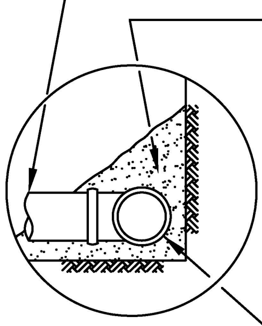
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

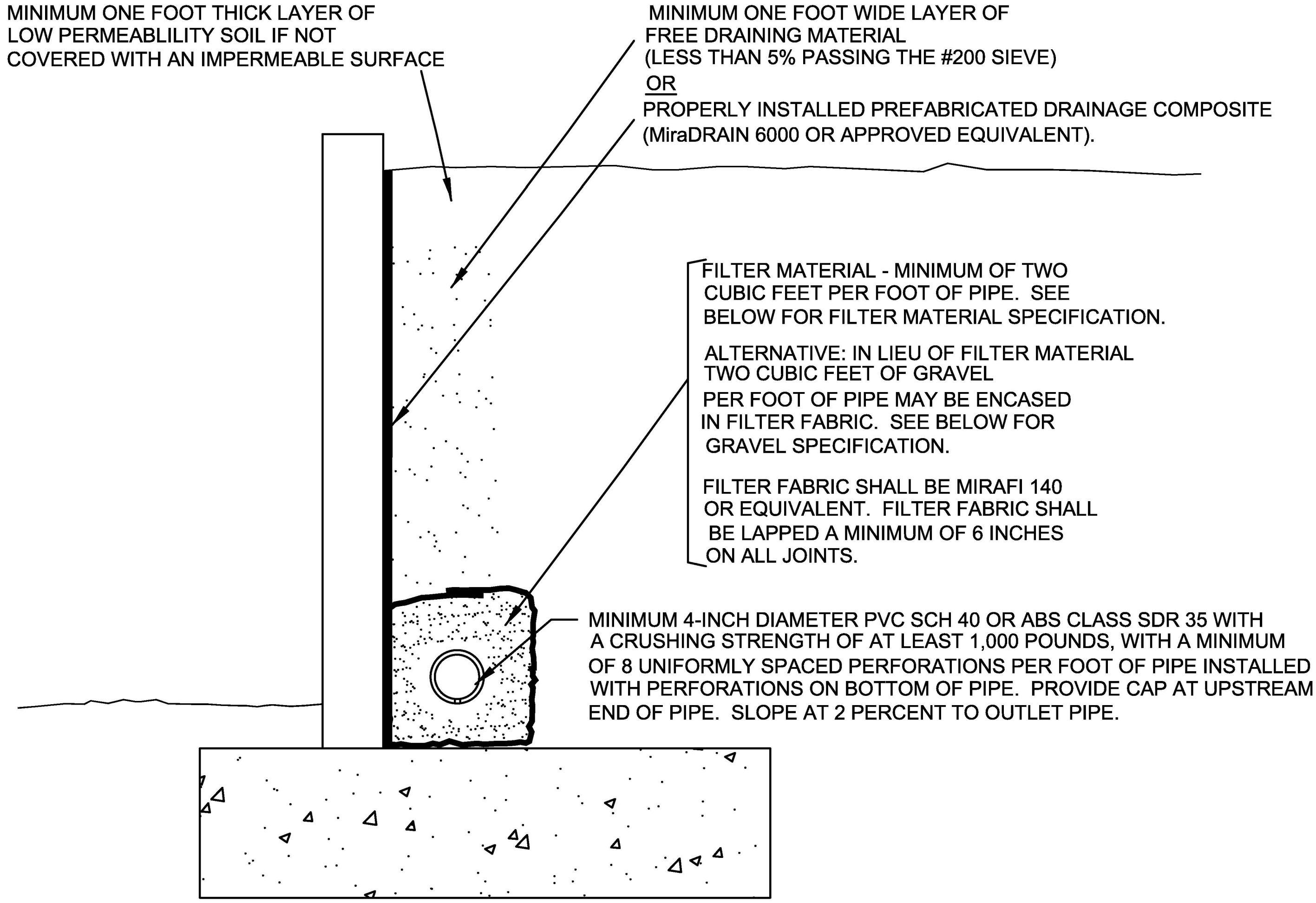
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

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

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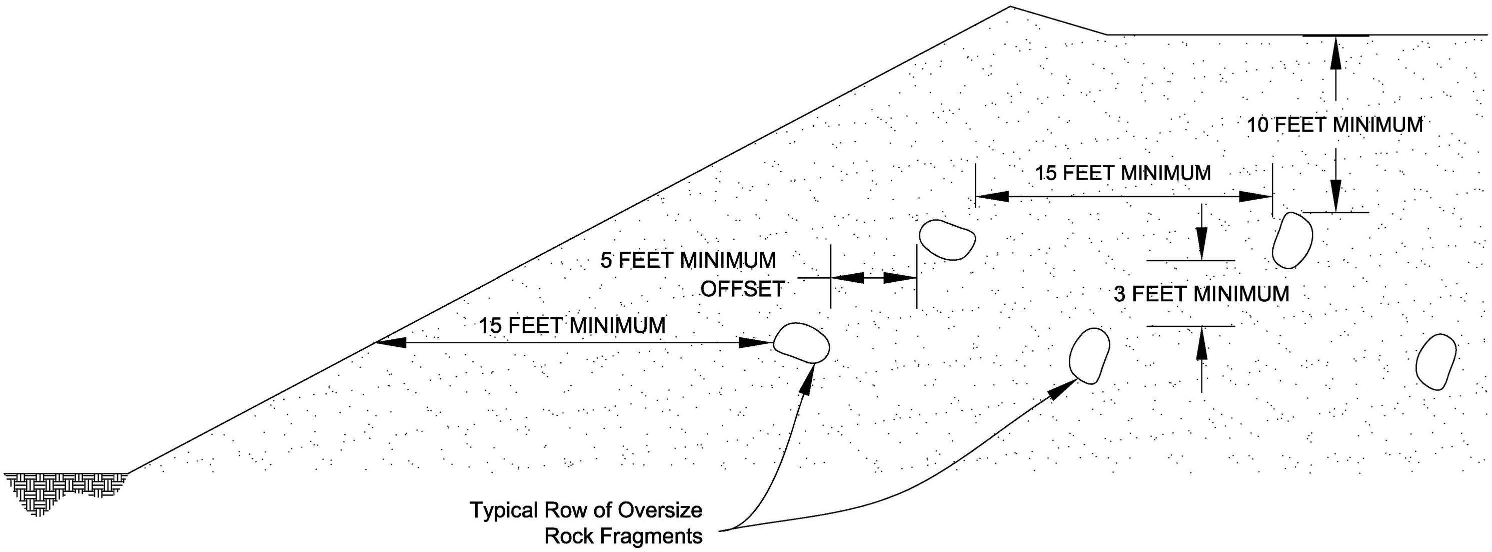
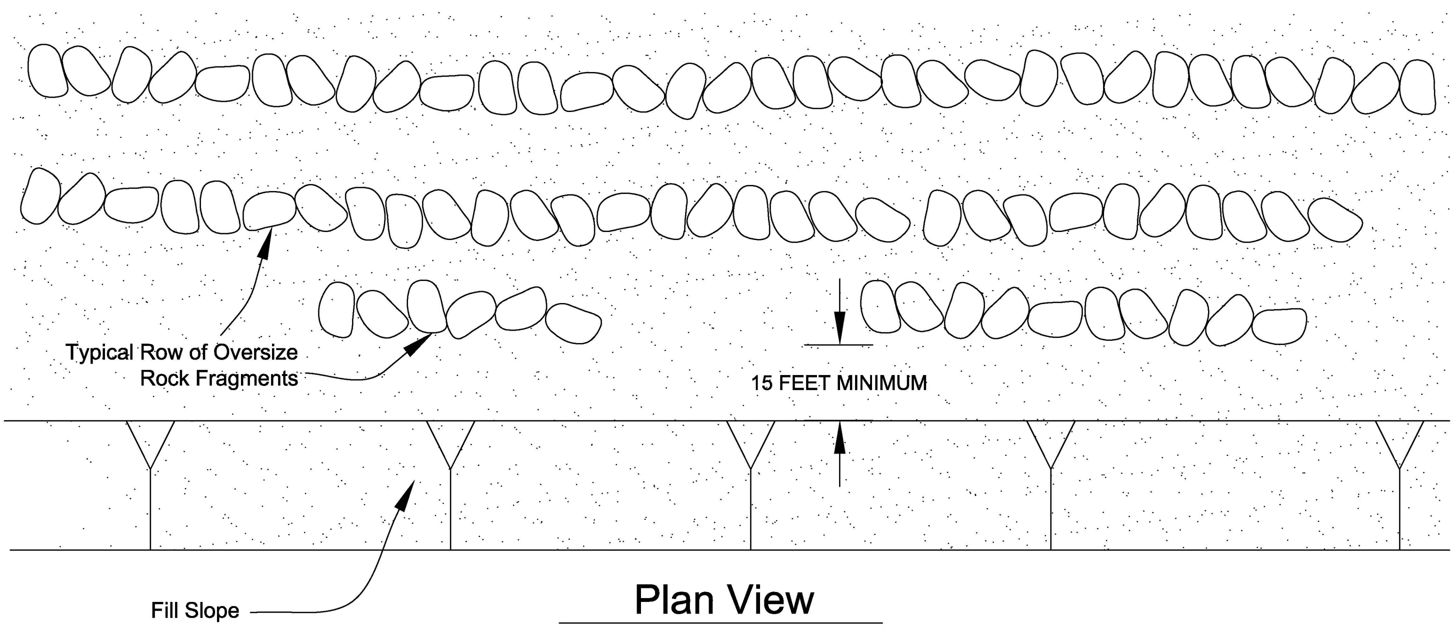
SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

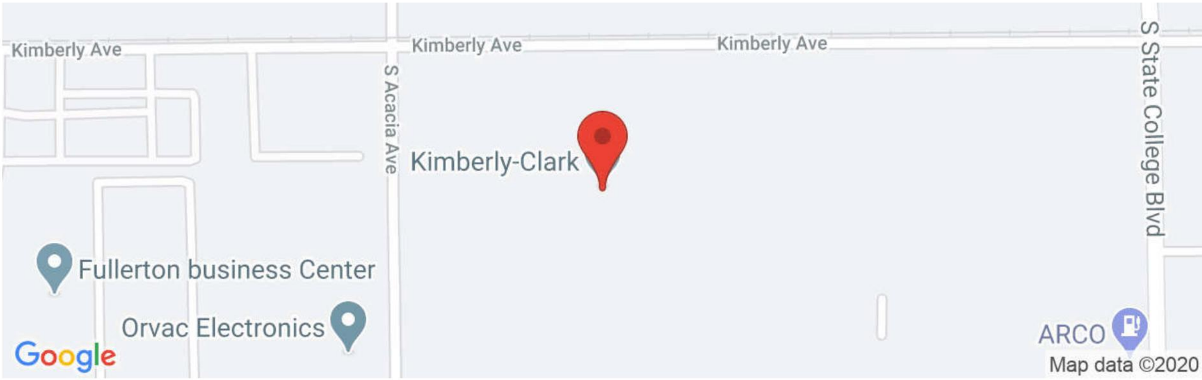
SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

OSHPD

Latitude, Longitude: 33.8617864, -117.8957217



Date	5/15/2020, 9:36:13 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.605	MCE_R ground motion. (for 0.2 second period)
S_1	0.566	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.605	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.07	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.685	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.754	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.605	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.762	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.275	Factored deterministic acceleration value. (0.2 second)
S1RT	0.566	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.621	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.756	Factored deterministic acceleration value. (1.0 second)
PGAd	0.924	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.911	Mapped value of the risk coefficient at short periods
C_{R1}	0.911	Mapped value of the risk coefficient at a period of 1 s

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

SEISMIC DESIGN PARAMETERS - 2019 CBC

GOODMAN LOGISTICS CENTER FULLERTON

FULLERTON, CALIFORNIA

DRAWN: JAH
CHKD: RGT

SCG PROJECT
19G139-1R

PLATE E-1

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SOUTHERN CALIFORNIA GEOTECHNICAL