GEOTECHNICAL INVESTIGATION
5th & STERLING AVENUE

SEC 6th Street at Sterling Avenue
San Bernardino, California
for
Fifth & Sterling, LLC, a Delaware limited liability company

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

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May 26, 2023 (revised February 16, 2024)

Fifth & Sterling, LLC,
a Delaware Limited Liability Company
3501 Jamboree Road, Suite 230
Newport Beach, California 92660

Attention: David Drake
Executive Vice President

Project No.: **23G142-1R**

Subject: **Geotechnical Investigation**
Proposed Industrial Building
NEC 5th Street at Sterling Avenue
San Bernardino, California

Mr. Drake:

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

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Ricardo Frias, RCE 91772
Project Engineer

Gregory K. Mitchell, GE 2364
Principal Engineer

Distribution: (1) Addressee

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1.0 EXECUTIVE SUMMARY

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

Geotechnical Design Considerations

- The results of the liquefaction evaluation indicate total dynamic settlements ranging between 0 and 0.39± inches. The liquefaction-induced differential settlements are expected to be on the order of ¼± inch.
- Based on the estimated magnitude of the differential settlements, the proposed structure may be supported on shallow foundations.
- Artificial fill soils were encountered at all of the boring locations, extending from the ground surface to depths of 2 to 5½± feet. These soils, in their present condition, are not considered suitable for support of the foundation loads of the new structure.
- The near-surface alluvial soils possess varying strengths. These soils, in their present condition, are not considered suitable for support of the foundation loads of the new structures. The deeper alluvium generally possesses higher strengths and densities and more favorable consolidation/collapse characteristics.
- Based on the water level measurements performed after completion of drilling and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth of 37± feet below existing site grades at the time of the subsurface exploration.

Site Preparation

- Initial site preparation should include demolition of the remnants of the previous development including all foundations, floor slabs, utilities, septic systems, and any other subsurface improvements that will not remain in place for use with the new development. Stripping of the existing vegetation including grass, weed growth, trash, and furniture. These materials should be disposed of off-site. Concrete and asphalt debris may be crushed to a maximum 1-inch particle size, mixed well with the on-site soils, and incorporated into structural fills if desired. Alternatively, it may be feasible to process these materials into crushed miscellaneous base.
- Remedial grading is recommended to be performed within the proposed building pad area to remove the undocumented fill soils, which extend to depths of 2 to 5½± feet at all of the boring and trench locations, in their entirety. The building pad area should also be overexcavated to a depth of at least 4 feet below existing grade and to a depth of at least 3 feet below proposed pad grade, whichever is greater. Overexcavation within the foundation areas is recommended to extend to a depth of at least 3 feet below proposed foundation bearing grade.
- Deeper removals may be necessary in the areas of Boring Nos. B-5 and B-7 due to the presence of loose and compressible/collapsible soils extending to depths of 6½ to 8± feet below the existing site grades.
- After overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated.

The resulting soils should be scarified and thoroughly watered to achieve a moisture content of 0 to 4 percent above optimum moisture, to a depth of at least 24 inches. The overexcavation subgrade soils should then be recompacted and the excavated soils replaced as structural fill, compacted to 90 percent of the ASTM D-1557 maximum dry density.

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

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 3,000 lbs/ft^2 maximum allowable soil bearing pressure.
- Minimum reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade: minimum 6 inches thick.
- Modulus of Subgrade Reaction: k = 150 psi/in.
- Reinforcement is not considered necessary for geotechnical considerations.
- The actual floor slab reinforcement should be determined by the structural engineer, based on the imposed slab loading.

Pavement Design Recommendations

ASPHALT PAVEMENTS (R = 50)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	3	4	5	5	7
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 50)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 23P229, dated April 20, 2023. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site-specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the southeast corner of 6th Street and Sterling Avenue in San Bernardino, California. The site is bounded to the north by 6th Street, to the east by Armada Towing and an RV and trailer storage lot, to the south by 5th Street, and to the west by Sterling Avenue. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of an irregularly shaped parcel, 25.12± acres in size. Based on our subsurface investigation, the site is currently vacant and undeveloped except for the remnants of a concrete slab in the northeastern area of the site and associated foundations. The ground surface cover throughout the site generally consists of exposed soil with sparse native grass and weed growth, and areas of scattered debris including trash and furniture.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth and visual observations made at the time of the subsurface investigation, the site is relatively level with an overall site topography gently sloping downward to the west at a gradient less than 1 percent with an elevation differential of approximately 14 feet.

3.2 Proposed Development

Based on a conceptual site plan prepared by RGA, the site will be developed with one (1) new industrial building. The new building will be 537,618± ft² in size and will be located in the north-central area of the site. Dock-high doors will be constructed along the southern building wall. The building is expected to be surrounded by asphaltic concrete pavements in the parking and drive lanes, Portland cement concrete pavements in the loading dock areas, and limited areas of landscape planters.

Detailed structural information was not available at the time of this proposal. It is assumed that the new building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundations with concrete slab-on-grade floors. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 4 to 7 kips per linear foot, respectively.

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

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration for this project consisted of ten (10) borings advanced to depths of 5 to 50± feet. One (1) of the four 50-foot borings encountered refusal conditions at a shallower depth (32± feet) than proposed. Boring Nos. B-8 through B-10 were terminated at a depth of 5± feet below existing site grades. These borings did not encounter auger refusal conditions and are located within the area of the proposed parking lots. In addition to the borings, ten (10) trenches were excavated to depths of 8 to 10± feet below ground surface. The borings and trenches were logged during drilling and excavation by a member of our staff.

Hollow Stem Auger Borings

The borings were advanced with hollow-stem augers, by a conventional truck-mounted drilling rig. The trenches were advanced with a rubber-tire backhoe equipped with a 3-foot bucket. Representative bulk and relatively undisturbed soil samples were taken during drilling. Relatively undisturbed soil samples were taken with a split barrel "California Sampler" containing a series of one inch long, 2.416± inch diameter brass rings. This sampling method is described in ASTM Test Method D-3550. In-situ 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 and trenches are indicated on the Boring and Trench Location Plan, included as Plate 2 in Appendix A of this report. The Boring and Trench Logs, which illustrate the conditions encountered at the boring and trench locations, as well as the results of some of the laboratory testing, are included in Appendix B.

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface at all of the boring and trench locations, extending to depths of 2 to 5½± feet below the existing site grades. The fill soils generally consist of very loose to medium dense silty sands, sandy silts, and sands with varying amounts of silt and fine gravel. The fill soils possess a disturbed and mottled appearance resulting in the classification of artificial fill.

Alluvium

Native alluvial soils were encountered beneath the artificial fill soils at all of the boring and trench locations, extending to at least the maximum depth explored of 50± feet below existing site grades. The near surface alluvium generally consists of medium dense to very dense silty sands, sandy silts, and poorly- to well-graded sands with varying amounts of fine to coarse gravel, cobbles, and boulders, extending to depths of 12 to 25± feet below existing site grades. Deeper alluvial soils consist of dense to very dense silty sands, sandy silts and poorly-graded sands with varying amounts of fine to coarse gravel, cobbles, and boulders, extending to the maximum depth explored of 50± feet below the site grades. Boring Nos. B-5 and B-7 encountered loose poorly- to well-graded sands at depths of 4½ to 5½± feet. Boring No. B-3 encountered a layer of loose silty sands and medium dense well-graded sands at a depth of 22± feet.

Groundwater

Free water was encountered during the drilling at one (1) of the boring locations. Water was encountered at 37± feet below existing site grades at Boring No. B-3. Delayed groundwater level readings were taken at Boring No. B-3 approximately two hours after completion. Water was measured in this boring at a depth 37± feet. The remaining boreholes were dry at the completion of drilling. Very moist samples were also encountered at Boring No. B-1, at a depth of 42± feet and extending to the maximum depth explored of 50± feet. Based on the water level measurements and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth of 37± feet below existing site grades, at the time of the subsurface investigation.

A groundwater contour map titled, "Contour Map Showing Minimum Depth to Ground Water, San Bernardino Valley and Vicinity, 1973-1983," prepared by Carson and Matti in 1986 indicates that the minimum depth to groundwater at the site could be approximately 37 to 45 feet.

As a part of our research, we reviewed available groundwater data in order to determine groundwater levels for the site. Recent water level data was obtained from the California Department of Water Resources website, https://wdl.water.ca.gov/waterdatalibrary/. One monitoring well (Well No. 341072N1172350W001) is located approximately 1,675 feet southeast of the site. Water level readings within this monitoring well indicates a high groundwater level of 163± feet below the ground surface in April 2008.

Based on the available groundwater data, we used a conservative water level in our liquefaction analyses of 37 feet below the existing ground surface.

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.

Maximum Dry Density and Optimum Moisture Content

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

Soluble Sulfates

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

Sample Identification	**Soluble Sulfates (%)**	**Sulfate Classification**
B-1 @ 1 to 5 feet	0.002	Negligible (SO)
B-7 @ 1 to 5 feet	0.002	Negligible (SO)

Corrosivity Testing

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

Sample Identification	**Saturated Resistivity (ohm-cm)**	**pH**	**Chlorides (mg/kg)**	**Nitrates (mg/kg)**	**Sulfides (mg/kg)**	**Redox Potential (mV)**
B-1 @ 1 to 5 feet	9,380	7.4	7.1	22.1	0.8	150
B-7 @ 1 to 5 feet	7,370	6.9	24.7	61.7	0.7	153

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

The subject site is located in an area which is subject to strong ground motions due to earthquakes. The performance of a site specific seismic hazards analysis was beyond the scope of this investigation. However, numerous faults capable of producing significant ground motions are located near the subject site. Due to economic considerations, it is not generally considered reasonable to design a structure that is not susceptible to earthquake damage. Therefore, significant damage to structures may be unavoidable during large earthquakes. The proposed 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. 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. Liquefaction is a potential geologic hazard for this site and is discussed below.

Seismic Design Parameters

The 2022 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

The 2022 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 2022 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents. The table below was created using data obtained from the application. The output generated from this program is attached to this letter.

The 2022 CBC states that for Site Class D sites with a mapped S1 value greater than 0.2, a site-specific ground motion analysis may be required in accordance with Section 11.4.8 of ASCE 7-16. Supplement 3 to ASCE 7-16 modifies Section 11.4.8 of ASCE 7-16 and states that "a ground motion hazard analysis is not required where the value of the parameter SM1 determined by Eq. (11.4-2) is increased by 50% for all applications of SM1 in this Standard. The resulting value of the parameter SD1 determined by Eq. (11.4-4) shall be used for all applications of SD1 in this Standard."

The seismic design parameters presented in the table below were calculated using the site coefficients (Fa and Fv) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2022 CBC. It should be noted that the site coefficient Fv and the parameters SM1 and SD1 were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the ASCE 7-16 standard. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2022 CBC using the value of S1 obtained from the Seismic Design Maps Tool. **The values of SM1 and SD1 tabulated below** were evaluated using equations 11.4-2 and 11.4-4 of ASCE 7-16 (Equations 16-20 and 16-23, respectively, of the 2022 CBC) and **do not include a 50 percent increase.** As discussed above, if a ground motion hazard analysis has not been performed, SM1 and SD1 must be increased by 50 percent for all applications with respect to ASCE 7-16.

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	2.286
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.841
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2.286
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.430[1]
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.524
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.953[1]

[1]Note: These values must be increased by 50 percent if a site-specific ground motion hazard analysis has not been performed.

*The 2022 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site *coefficients* are to be determined in accordance with Section 11.4.7 of ASCE 7-16. However, Section 20.3.1 of ASCE 7-16 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structure is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site-specific seismic hazards analysis will be required and additional subsurface exploration will be necessary.

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 2022 CBC. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2022 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.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2022 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-16. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-16. The web-based software application SEAOC/OSHPD Seismic Design Maps Tool (described in the previous section) was used to determine PGA_M, which is 1.036g. A portion of the program output is included as Plate E-1 of this report. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated mean magnitude is 7.24, based on the peak ground acceleration and soil classification D for a return period greater than 2,500 years.

Liquefaction

Research of the San Bernardino County Land Use Plan, Geologic Hazard Overlays, San Bernardino South Quadrangle, FH30 C indicates that the subject site is located within a zone of liquefaction susceptibility. Therefore, the scope of this investigation included a detailed liquefaction evaluation in order to determine the site-specific liquefaction potential.

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 liquefaction analysis was conducted in accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008, 2014). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is determined as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content and/or the corrected CPT tip stress, $q_{c1N\text{-}cs}$. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable. Additionally, in accordance with Special Publication 117A, clayey soils which do not meet the criteria for liquefiable soils defined by Bray and Sancio (2006), loose soils with a plasticity index (PI) less than 12 and moisture content greater than 85 percent of the liquid limit, are considered to be insusceptible to liquefaction. Non-sensitive soils with a PI greater than 18 are also considered non-liquefiable.

The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis was performed for Boring Nos. B-1 through B-3. The liquefaction potential of the site was analyzed utilizing a PGA_M of 1.036g for a magnitude 7.24 seismic event.

The historic high groundwater depth was obtained from USGS Bulletin 1898, by Matti and Carson, 1991, which indicates high groundwater level ranging from 37 to 45± feet. We conservatively utilized a historic high groundwater table of 37 feet below grade to evaluate the liquefaction potential of the various layers encountered in the boring logs. Layers above this level were not considered in the liquefaction analysis. Soils in Boring No. B-1 at a depth of 42 to 50 feet were calculated to be potentially liquefiable.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are evaluated using the equation for volumetric strain due to post-cyclic reconsolidation (Yoshimine et. al, 2006). This procedure uses an empirical relationship between the induced cyclic shear strain and the corrected N-value to evaluate the expected volumetric strain of saturated sands subjected to earthquake shaking. This analysis is also documented on the spreadsheets included in Appendix F.

Conclusions and Recommendations

The results of the liquefaction analysis have identified a potentially liquefiable soil layer at Boring No. B-1. Soils which are located above the historic groundwater table or possess factors of safety of at least 1.3 are considered to be non-liquefiable. Settlement analyses was conducted for the potentially liquefiable layer. The total dynamic settlement for each boring location, based on the results of the dynamic settlement analyses (presented in Appendix F) are presented below:

- B-1: 0.39± inches
- B-2: 0 inches
- B-3: 0 inches

Based on these total settlements, differential settlements of up to¼± inch could be expected to occur during a liquefaction inducing seismic event. The estimated differential settlement could be assumed to occur across a distance of 50 feet, indicating a maximum angular distortion of less than 0.001 inches per inch. Based on this evaluation of potential settlement, no design considerations related to liquefaction are considered related to liquefaction are considered warranted for this site.

The use of a shallow foundation system, as described in this report, is typical for buildings of this type, where they are underlain by the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the building proposed for this site will also be typical of similar buildings in the vicinity of this project. However, if the owner determines that this level of potential damage is not acceptable, other geotechnical and structural options are available.

6.2 Geotechnical Design Considerations

General

The site is generally underlain by artificial fill soils, extending to depths of 2 to 5½± feet at all of the boring and trench locations. These soils possess variable densities, variable composition, and a disturbed, mottled appearance. Additionally, no documentation regarding the placement and compaction of these soils has been provided. The fill soils are therefore considered to be undocumented fill. The fill soils are underlain by native alluvium which possesses unfavorable consolidation/collapse characteristics to a depth of up to 6± feet below the existing site grades. Based on these conditions, the artificial fill materials and the near-surface alluvium, in their present condition, are not considered suitable for support of the foundations and floor slab of the new structure. Remedial grading will be necessary within the proposed building area to remove the artificial fill soils in their entirety as well as a portion of the near-surface alluvium, and to replace these soils as compacted structural fill.

Settlement

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

Soluble Sulfates

The results of the soluble sulfate testing indicated a sulfate concentration of approximately 0.002 percent for the selected sample of the near-surface soils. This concentration is considered to be "not applicable" (S0) with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Corrosion Potential

The results of laboratory testing indicate that representative samples of the on-site soils possess minimum resistivity values of 7,370 and 9,380 ohm-cm, and pH values of 6.9 and 7.4. These soils possess redox potentials of 150 and 153 mV and trace sulfide concentrations of about 0.1 parts per million. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity, pH, sulfide concentration, redox potential, and moisture content are the five factors that enter into the evaluation procedure. Based on these factors, the on-site soils are considered to be less corrosive to ferrous materials including iron pipes. Therefore, corrosion protection will likely not be required for cast iron or ductile iron pipes.

Low concentrations of chlorides (7.1 and 24.7 mg/kg) were detected in the samples submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested sample, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations of 22.1 to 61.7 mg/kg. **Based on these test results, the on-site soils are considered to be potentially corrosive to copper pipe with respect to their nitrate concentration.**

Since SCG does not practice in the area of corrosion engineering, we recommend that the client contact a corrosion engineer to provide a more thorough evaluation of these test results.

Shrinkage/Subsidence

Removal and recompaction of the existing fill soils and near-surface alluvium to an average 92 percent relative compaction is estimated to result in an average shrinkage of 5 to 15 percent. However, potential shrinkage for individual samples ranged between 1 and 18 percent. It should be noted that the shrinkage estimate is based on the results of dry density testing performed on small-diameter samples of the existing soils taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1 feet.

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

Grading and Foundation Plan Review

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

6.3 Site Grading Recommendations

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

Site Stripping and Demolition

Remnants of concrete slab and building foundations are present at the ground surface at the site. Initial site preparation should include the demolition of the existing slab and foundations. Site demolition should also include any utilities, septic systems, and any other subsurface improvements associated with the previous development of the site. Debris resultant from demolition should be disposed of off-site. Alternatively, concrete and asphalt debris may be crushed to a maximum 1-inch particle size, mixed with the on-site soils, and reused as compacted structural fill. It may also be feasible to process these materials into crushed miscellaneous base (CMB).

Initial site preparation should include stripping of any topsoil, vegetation, organic debris, and any scattered debris on the site. Based on conditions observed at the time of the subsurface exploration, this will include native grass, weed growth, trash, and furniture. These materials should be disposed of off-site. The actual extent of stripping should be determined in the field by a representative of the geotechnical engineer, based on the organic content and the stability of the encountered materials.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building area in order to remove the existing undocumented fill soils. Based on conditions encountered at the boring locations, excavation to depths of 2 to 5½± feet will be required to remove the existing fill soils. The existing soils within the proposed building area are also recommended to be overexcavated to a depth of at least 4 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevation, whichever is greater.

Where not encompassed within the general building pad overexcavations, additional overexcavation should be performed within the influence zones of the new foundations, to provide for a new layer of compacted structural fill extending to a depth of 3 feet below proposed bearing grade.

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

Slightly deeper areas of overexcavation will also be required in the area of Boring Nos. B-5 and B-7, where loose and potentially collapsible soils extend to depths of 6½ to 8± feet. Additional evaluation of the exposed overexcavation subgrade soils by the geotechnical engineer will be required in this area of the site to verify that the full extent of loose and potentially collapsible soils, as encountered at Boring Nos. B-5 and B-7, are removed.

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

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches, and thoroughly watered to raise the moisture content of the underlying soils to at least 0 to 4 percent above optimum moisture content, extending to a depth of at least 24 inches. The moisture conditioning of the overexcavation subgrade soils should be verified by the geotechnical engineer. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

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

Please note that if the lateral and/or vertical extents of overexcavation are not achievable for the project retaining walls or site walls (as may occur along property lines), then additional recommendations including, but not limited to reduced design bearing pressures may be required. Additionally, specialized grading techniques such as slot cutting or shoring may be required in order to facilitate construction.

Treatment of Existing Soils: Parking, Drive and Flatwork Areas

Based on economic considerations, overexcavation of the existing soils in the new parking, drive, and flatwork areas are not considered warranted, with the exception of areas where lower strength or unstable soils are identified by the geotechnical engineer during grading.

Subgrade preparation in the new parking, drive, and flatwork areas should initially consist of removal of all soils disturbed during stripping operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 0 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of undocumented fill soils and compressible/collapsible alluvial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

The grading recommendations presented above for the proposed parking, drive, and flatwork areas assume that the owner and/or developer can tolerate minor amounts of settlement within these areas. The grading recommendations presented above do not completely mitigate the extent of loose alluvium in the parking, drive, and flatwork areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking, drive, and flatwork areas should be overexcavated to a depth of 2 feet below proposed 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 0 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2022 CBC and the grading code of the City of San Bernardino.
- 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.

Selective Grading and Oversized Material Placement

Some of the native alluvial soils possess moderate cobble content. In general, these cobble-containing soils are located at depths of 4½ to 32± feet. It is expected that large scrapers (Caterpillar 657 or equivalent) will be adequate to move the cobble containing soils. Since the proposed grading will require excavation of cobble containing soils, it may be desirable to selectively grade the proposed building pad area. The presence of particles greater than 3 inches in diameter within the upper 1 to 3 feet of the building pad subgrade will impact the utility and foundation excavations. Depending on the depths of fills required within the proposed parking areas, it may be feasible to sort the on-site soils, placing the materials greater than 3 inches in diameter within the lower depths of the fills, and limiting the upper 1 to 3 feet of soils to materials less than 3 inches in size. Oversized materials could also be placed within the lower depths of the recommended overexcavations. In order to achieve this grading, it would likely be necessary to use rock buckets and/or rock sieves to separate the oversized materials from the remaining soil. Although such selective grading will facilitate further construction activities, it is not considered mandatory and a suitable subgrade could be achieved without such extensive sorting. However, in any case, it is recommended that all materials greater than 6 inches in size be excluded from the upper 1 foot of the surface of any compacted fills.

The placement of any oversized materials should be performed in accordance with the Grading Guide Specifications included in Appendix D of this report. If disposal of oversized materials is required, rock blankets or windrows should be used and such areas should be observed during construction and placement by a representative of the geotechnical engineer.

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 soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by City of San Bernardino. All utility trench backfills should be witnessed by the geotechnical engineer. The trench backfill soils should be compaction tested where possible; probed and visually evaluated elsewhere.

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

6.4 Construction Considerations

Excavation Considerations

The near surface soils generally consist of silty sands and sands. 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, the inclination of temporary slopes should not exceed 2h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Groundwater

The static groundwater table at this site is considered to exist at a depth of approximately 37± feet. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

Based on the preceding grading recommendations, it is assumed that the new building pad will be underlain by structural fill soils used to replace near-surface alluvial soils. These new structural fill soils are expected to extend to depths of at least 3 feet below proposed foundation bearing grade, underlain by 1± foot of additional soil that has been densified and moisture conditioned in place. Based on this subsurface profile, and based on the design considerations presented in Section 6.1 of this report, the proposed structure may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

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

- Minimum wall/column footing width: 14 inches/24 inches.

- Minimum longitudinal steel reinforcement within strip footings: Four (4) No. 5 rebars (2 top and 2 bottom).

- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.

- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on geotechnical considerations; additional reinforcement may be necessary for structural considerations. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Any unsuitable materials should be removed to a depth of suitable bearing compacted structural fill, with the resulting excavations backfilled with compacted fill soils. As an alternative, lean concrete slurry (500 to 1,500 psi) may be used to backfill such isolated overexcavations.

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

Estimated Foundation Settlements

Post-construction total and differential static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 50-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch.

Lateral Load Resistance

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

- Passive Earth Pressure: 300 lbs/ft^3
- Friction Coefficient: 0.30

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

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the new structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 4 feet below proposed finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

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

- Minimum slab reinforcement: Not required for geotechnical considerations. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab where such moisture sensitive floor coverings are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

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

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

The actual design of the floor slab should be completed by the structural engineer to verify adequate thickness and reinforcement. Additional rigidity may be necessary for structural considerations.

6.7 Exterior Flatwork Design and Construction

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

- Minimum slab thickness: 4½ inches.

- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.

- Moisture condition the slab subgrade soils to at least 0 to 4 percent above the optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 24 hours prior to concrete placement.

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

- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.

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

6.8 Retaining Wall Design and Construction

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

Retaining Wall Design Parameters

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

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

RETAINING WALL DESIGN PARAMETERS

<table>
<tr><th colspan="2" rowspan="2">Design Parameter</th><th>Soil Type</th></tr>
<tr><th>On-Site Sands and Silty Sands</th></tr>
<tr><td colspan="2">Internal Friction Angle (ϕ)</td><td>30°</td></tr>
<tr><td colspan="2">Unit Weight</td><td>130 lbs/ft^3</td></tr>
<tr><td rowspan="3">Equivalent Fluid Pressure:</td><td>Active Condition (level backfill)</td><td>44 lbs/ft^3</td></tr>
<tr><td>Active Condition (2h:1v backfill)</td><td>70 lbs/ft^3</td></tr>
<tr><td>At-Rest Condition (level backfill)</td><td>65 lbs/ft^3</td></tr>
</table>

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.30 and an equivalent passive pressure of 300 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

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

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

Seismic Lateral Earth Pressures

In accordance with the 2022 CBC, any retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

The retaining wall foundations should be supported within newly placed compacted structural fill, extending to a depth of at least 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 soils may be used to backfill the retaining walls. **However, all backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches.** The retaining wall backfill materials should be well graded.

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

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

Subsurface Drainage

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

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

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

6.9 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of silty sands and clayey sands. These soils are considered to possess good pavement support characteristics with estimated R-values of 50 to 70. The subsequent pavement design is based upon an R-value of 50. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

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

Traffic Index	No. of Heavy Trucks per Day
4.0	0
5.0	1
6.0	3
7.0	11
8.0	35
9.0	93

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

ASPHALT PAVEMENTS (R = 50)					
Materials	**Thickness (inches)**				
	Auto Parking and Auto Drive Lanes (TI = 4.0 to 5.0)	Truck Traffic			
		TI = 6.0	TI = 7.0	TI = 8.0	TI = 9.0
Asphalt Concrete	3	3½	4	5	5½
Aggregate Base	3	4	5	5	7
Compacted Subgrade	12	12	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 50)				
Materials	**Thickness (inches)**			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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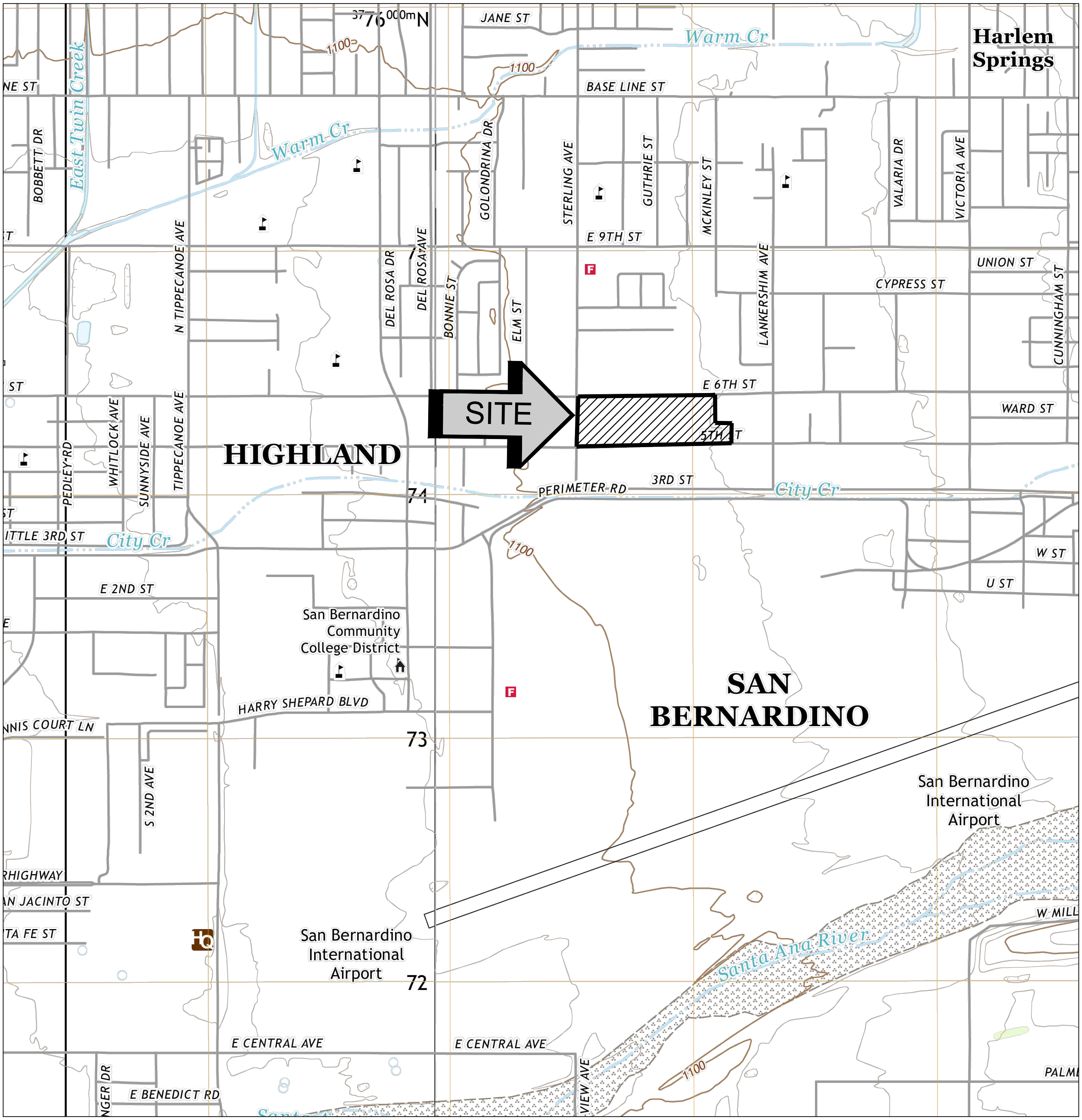
Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAP OF THE REDLANDS AND SAN BERNARDINO SOUTH QUADRANGLES, SAN BERNARDINO COUNTY, CALIFORNIA, 2021.

SITE LOCATION MAP

PROPOSED INDUSTRIAL BUILDING

SAN BERNARDINO, CALIFORNIA

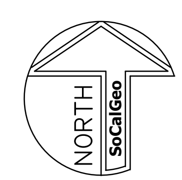
SCALE: 1" = 2000'	
DRAWN: MK CHKD: RF	
SCG PROJECT 23G142-1	
PLATE 1	

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- APPROXIMATE TRENCH LOCATION

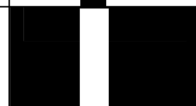
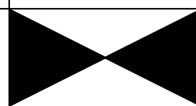
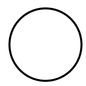
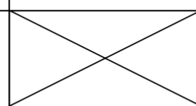
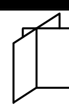


NOTE: SITE PLAN PREPARED BY RGA,

BORING AND TRENCH LOCATION PLAN	
PROPOSED INDUSTRIAL BUILDING	
SAN BERNARDINO, CALIFORNIA	
SCALE: 1" = 150'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: MK CHKD: RF	
SCG PROJECT 23G142-1	
PLATE 2	

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

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

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS 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.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 47 feet
READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)		SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					FILL: Dark Brown Silty fine Sand, little medium Sand, trace coarse Sand, loose to medium dense-damp to moist							
	X	10				102	8					
	X	15				123	7					
					ALLUVIUM: Brown Silty fine Sand, trace to little medium Sand, little iron oxide staining, medium dense-damp to moist							
5	X	15				103	13					
	X	20										No Sample Recovery
					Light Gray Brown fine to coarse Sand, little SIlt, trace to little fine to coarse Gravel, dense to very dense-dry							
	X	87/11"				116	1					
10												
	X	50			@ 13½ feet, occasional cobbles and boulders		1					
15												
	X	39					1					
20												
					Brown fine Sandy Silt, with 2-inch lense of Silty fine to medium Sand, medium dense-moist to very moist							
	X	21					16					
25												
					Gray Brown Silty fine Sand, trace medium Sand, trace fine Gravel, dense to very dense-damp to moist							
	X	63					6					
30												
	X	47					7					

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SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 47 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Gray Brown Silty fine Sand, trace medium Sand, trace fine Gravel, dense to very dense-damp to moist							
	X	33					12					
40												
					Gray fine Sandy Silt, trace medium Sand, medium dense-very moist							
	X	27					22					
45												
	X	24					23					
50												
					Boring Terminated at 50'							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 43 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
					FILL: Dark Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, very loose to loose-damp							
	X	2					6					
	X	8					6					
5												
					ALLUVIUM: Light Red Brown fine to coarse Sand, trace fine Gravel, trace Silt, medium dense-damp							
	X	14					3					
					Gray Brown fine to coarse Sand, little Silt, trace fine to coarse Gravel, dense-dry to damp							
	X	35					2					
10												
	X	42			@ 13½ feet, occasional cobbles		2					
15												
					Gray Brown Silty fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, medium dense-damp							
	X	25					4					
20												
					Brown fine Sandy Silt, litte iron oxide staining, occasional cobbles and boulders, very dense-very moist							
	X	57/7"					26					
25												
					Light Red Brown fine to medium Sand, little Silt, trace coarse Sand, dense to very dense-dry to damp							
	X	30					3					
30												
	X	58			@ 33½ feet, little coarse Sand, little fine to coarse Gravel		1					

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SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 43 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Red Brown fine to medium Sand, little Silt, trace coarse Sand, dense to very dense-dry to damp							
	X	67					1					
40												
					Gray Brown fine Sand, little Silt, with a 2-inch lense of Silt, little iron oxide staining, dense-moist to very moist							
	X	42					14					
45												
					Dark Gray Brown fine Sandy Silt, trace medium Sand, dense-moist to very moist							
	X	39					14					
50												
					Boring Terminated at 50'							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

BORING NO. B-3

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: 37 feet
CAVE DEPTH: 40 feet
READING TAKEN: 2 hrs. after drilling

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Dark Brown Silty fine Sand, little medium Sand, trace coarse sand, loose to medium dense-damp to moist							
		5					8					
		11					6					
5												
					ALLUVIUM: Light Red Brown fine to coarse Sand, trace Silt, little fine Gravel, occasional Cobbles, medium dense-damp							
		17					4					
		25			@ 8½ feet, little fine to coarse Gravel, dense		3					
10												
					Light Red Brown fine Sand, little medium to coarse Sand, trace Silt, trace fine Gravel, medium dense-damp							
		24					7					
15												
					Light Red Brown fine to medium Sand, trace coarse Sand, trace fine to coarse Gravel, trace Silt, dense-damp							
		31					3					
20												
					Dark Brown Silty fine Sand, trace Clay, loose-moist							
		7										No Sample Recovery
25												
		15				109	13					
					Dark Gray Brown fine to medium Sand, trace coarse Sand, medium dense to dense-damp to moist							
		39					4					
30												
					Gray Brown to Dark Gray Brown Silty fine Sand to fine Sandy Silt, medium dense-very moist							
		20					23					

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

TEST BORING LOG

BORING NO. B-3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 23G142-1	DRILLING DATE: 4/27/23	WATER DEPTH: 37 feet
PROJECT: Proposed Industrial Building	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 40 feet
LOCATION: San Bernardino, California	LOGGED BY: Joseph Lozano Leon	READING TAKEN: 2 hrs. after drilling

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Gray Brown to Dark Gray Brown Silty fine Sand to fine Sandy Silt, medium dense-very moist							
					Gray Brown fine Sandy Silt, dense-wet							
	X	31					23					
40												
					Light Gray Brown fine Sand, little Silt, dense-moist							
	X	39					8					
45												
					Light Brown fine Sand, trace medium Sand, with 2-inch lense of Dark Brown Silty fine to medium Sand, little iron oxide staining, medium dense-wet							
	X	29					15					
50					Boring Terminated at 50'							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 20 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
					FILL: Dark Brown Silty fine Sand, trace medium to coarse Sand, little fine root fibers, loose-damp							
		13				105	6					
					ALLUVIUM: Light Gray Brown fine to medium Sand, little coarse Sand, trace Silt, trace fine Gravel, medium dense-dry to damp							
		15				94	5					
5		20				105	2					
		28				105	2					
		40				124	2					
10					Gray Brown fine Sand, little medium Sand, little Silt, trace fine Gravel, medium dense-dry to damp							
					Gray Brown fine to medium Sand, trace Silt, trace coarse Sand, trace fine to coarse Gravel, occasional Cobbles and Boulders, dense to very dense-damp							
		50/3"					4					
15												
		82/10"			@ 18½ feet, little coarse Sand, little fine to coarse Gravel, abundant Cobbles, occasional Boulders		3					
20												
		42			@ 23½ feet, abundant Cobbles and Boulders		3					
25												
		50/5"					3					
30												
					Boring Terminated at 32' due to refusal							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: 10½
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	5			FILL: Dark Gray Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, loose-damp		7					
5	X	7			FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace Clay, loose-moist to very moist		15					
	X	9			ALLUVIUM: Light Red Brown fine Sand, trace medium Sand, trace fine Gravel, loose to dense-dry to damp		7					
10	X	20			@ 8½ feet, trace medium to coarse Sand, occasional Cobbles		3					
15	X	32			@ 13½ feet, little medium Sand		2					
					Boring Terminated at 15'							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-6

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: 7 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
					FILL: Dark Gray Brown Silty fine Sand, trace to little medium Sand, trace coarse Sand, medium dense-moist							
		17				116	8					
		15				114	10					
					ALLUVIUM: Light Red Brown Silty fine to medium Sand, trace to little coarse Sand, trace fine to coarse Gravel, occasional Cobbles, little iron oxide staining, medium dense-damp							
5		16				109	5					
					Light Brown fine to coarse Sand, trace fine to coarse Gravel, occasional Cobbles, little iron oxide staining, medium dense to very dense-damp							
		19				115	4					
		50/5"				111	4					
10												
		88/11"			@ 13½ feet, trace Silt		2					
15												
					Light Brown fine Sand, trace medium to coarse Sand, dense-dry to damp							
		42					2					
20												
					Boring Terminated at 20'							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-7

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: 7 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
		9			FILL: Dark Gray Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, loose-damp	106	4					
		10			FILL: Brown Silty fine Sand, trace to little medium Sand, trace coarse Sand, trace fine Gravel, loose-damp	100	6					
5		13			ALLUVIUM: Light Red Brown fine to medium Sand, trace Silt, trace coarse Sand, trace fine Gravel, loose to medium dense-damp	102	5					
		22				100	5					
10		32			Light Brown fine to coarse Sand, trace coarse Gravel, little fine Gravel, medium dense-dry to damp	107	2					
		60			Light Gray Brown Gravelly fine to coarse Sand, trace Silt, occasional Cobbles, very dense-dry to damp		2					
15					Boring Terminated at 15'							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-8

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: 2 feet
READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL

FIELD RESULTS: DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS: DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Dark Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, loose-moist							
	X	8					11					
					ALLUVIUM: Brown fine to medium Sand, little Silt, trace coarse Sand, medium dense-damp							
	X	11					6					
5												
					Boring Terminated at 5'							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-9

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: 3 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
	X	3			FILL: Dark Brown Silty fine to medium Sand, trace coarse Sand, trace fine Gravel, loose-damp to moist		8					
5	X	7			ALLUVIUM: Dark Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, trace iron oxide staining, loose-damp		7					
					Boring Terminated at 5'							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

BORING NO. B-10

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California

DRILLING DATE: 4/27/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: Dry
CAVE DEPTH: 3½
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Dark Brown Silty fine Sand, trace medium to coarse Sand, loose-moist							
	X	6					9					
					ALLUVIUM: Light Red Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, trace Silt, medium dense-damp							
	X	10					3					
5												
					Boring Terminated at 5'							

TBL 23G142-1.GPJ SOCALGEO.GDT 5/26/23

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-1

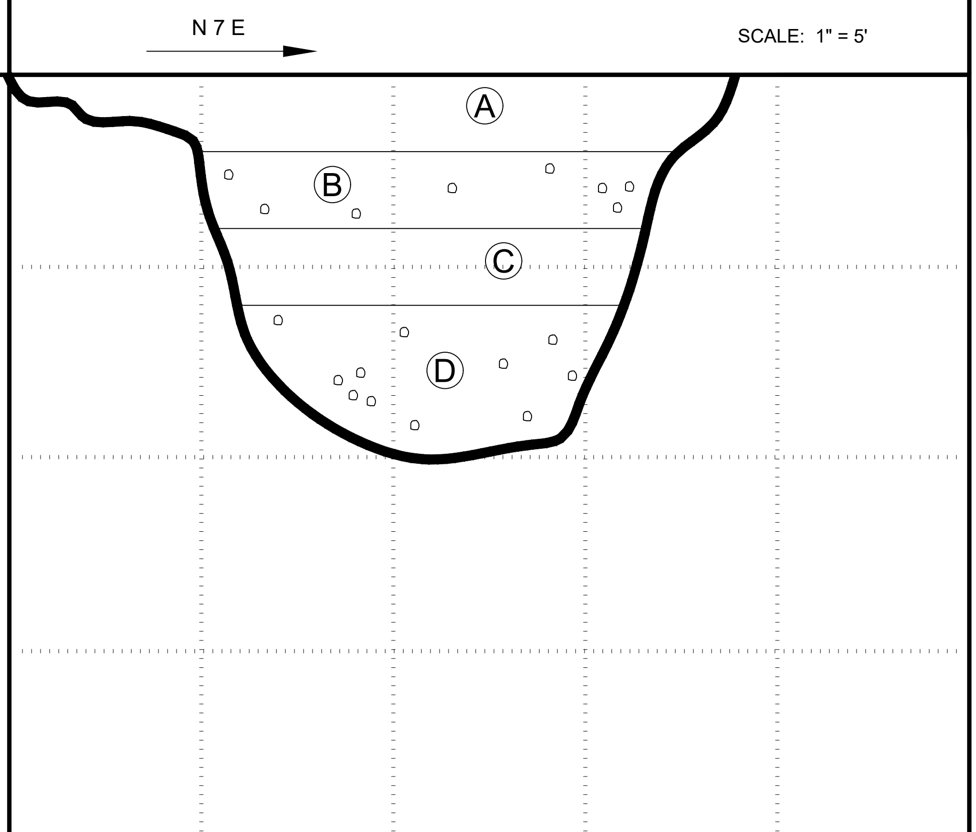
JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California
DATE: 4/28/2023

EQUIPMENT USED: Backhoe
LOGGED BY: Caleb Brackett
ORIENTATION: N 7 E
ELEVATION: ---

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: FILL: Dark Brown Silty fine Sand, medium dense-damp
	b		6	
	b		7	B: FILL: Dark Brown Silty fine Sand, trace fine Gravel, medium dense-damp
5	b		6	C: ALLUVIUM: Red Brown fine to medium Sand, medium dense-damp
	b		4	
	b		3	D: Red Brown fine to coarse Sand, trace fine to coarse Gravel, medium dense-damp
10				Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

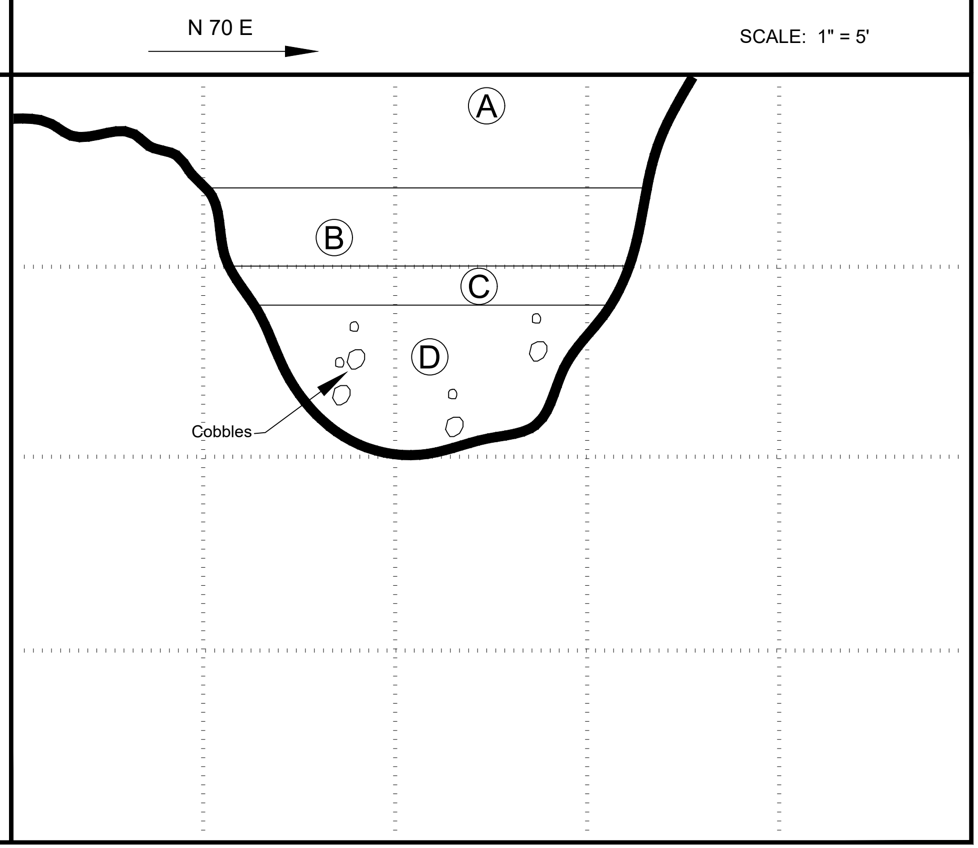
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-2

JOB NO.: 23G142-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Industrial Building	LOGGED BY: Caleb Brackett	SEEPAGE DEPTH: Dry
LOCATION: San Bernardino, California	ORIENTATION: N 70 E	READINGS TAKEN: At Completion
DATE: 4/28/2023	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		8	A: FILL: Dark Brown Silty fine Sand, loose-moist
	b		8	B: FILL: Dark Brown fine Sand, trace to little Silt, medium dense-damp to moist
5	b		8	C: ALLUVIUM: Brown fine Sand, trace Silt, medium dense-moist
	b		3	D: Red Brown fine to medium Sand, little coarse Sand, little fine to coarse Gravel, occasional Cobbles, medium dense-damp
	b		4	
10				Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-12

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-3

JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California
DATE: 4/28/2023

EQUIPMENT USED: Backhoe
LOGGED BY: Caleb Brackett
ORIENTATION: S 86 W
ELEVATION: ---

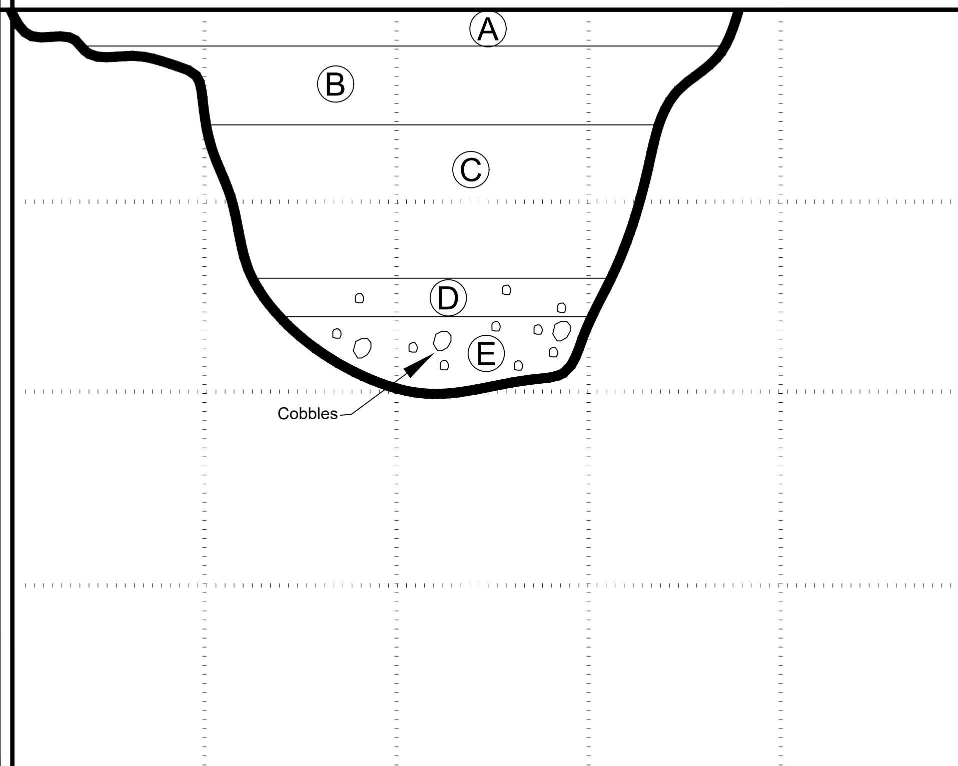
WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		8	A: FILL: Dark Brown fine Sandy Silt, loose-moist
	b		6	B: FILL: Brown Silty fine Sand, loose-damp
5	b		5	C: ALLUVIUM: Red Brown fine Sand, trace Silt, medium dense-damp
	b		3	D: Red Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense-damp E: Gray Brown Gravelly fine to medium Sand, occasional Cobbles, medium dense-damp
10	b		3	Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION

S 86 E

SCALE: 1" = 5'



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-13

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-4

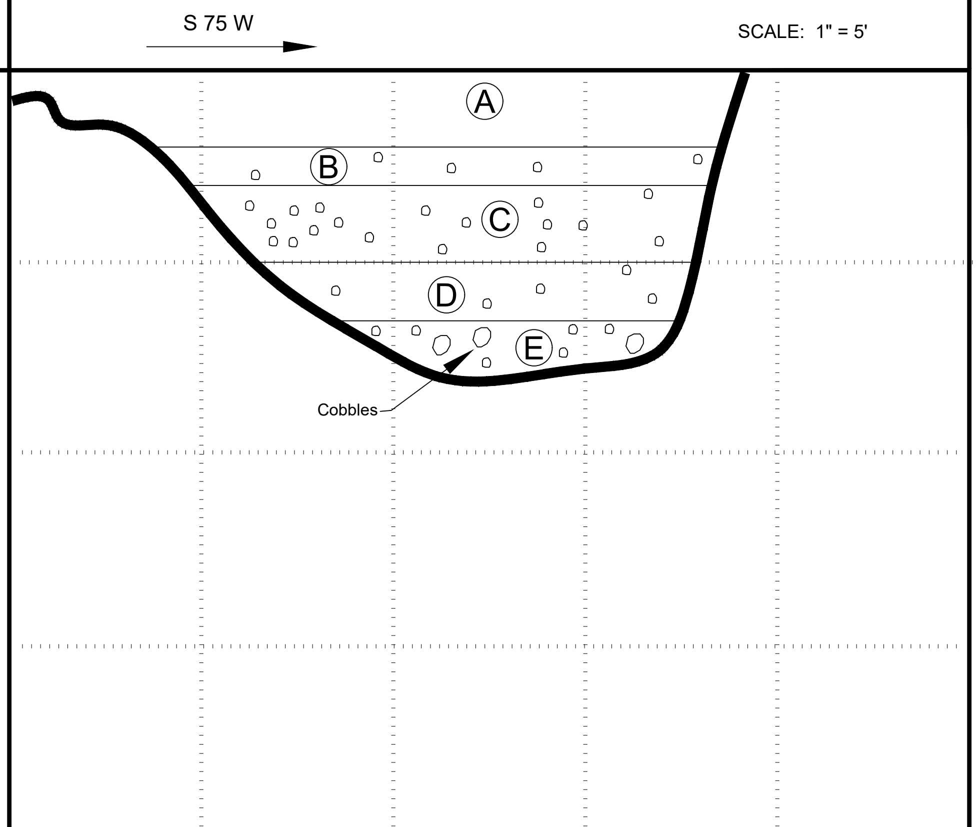
JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California
DATE: 4/28/2023

EQUIPMENT USED: Backhoe
LOGGED BY: Caleb Brackett
ORIENTATION: S 75 W
ELEVATION: ---

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: FILL: Dark Brown fine Sandy Silt, loose-damp
	b		7	
	b		4	B: FILL: Brown fine to coarse Sand, trace fine Gravel, loose-damp C: ALLUVIUM: Light Brown Gravelly fine to coarse Sand, medium dense-dry to damp
	b		2	
5				D: Dark Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense-damp E: Light Red Brown fine to medium Sand, little fine Gravel, occasional Cobbles, medium dense-damp
	b		4	
	b		5	
				Trench Terminated @ 8 feet due to caving
10				
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-14

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-5

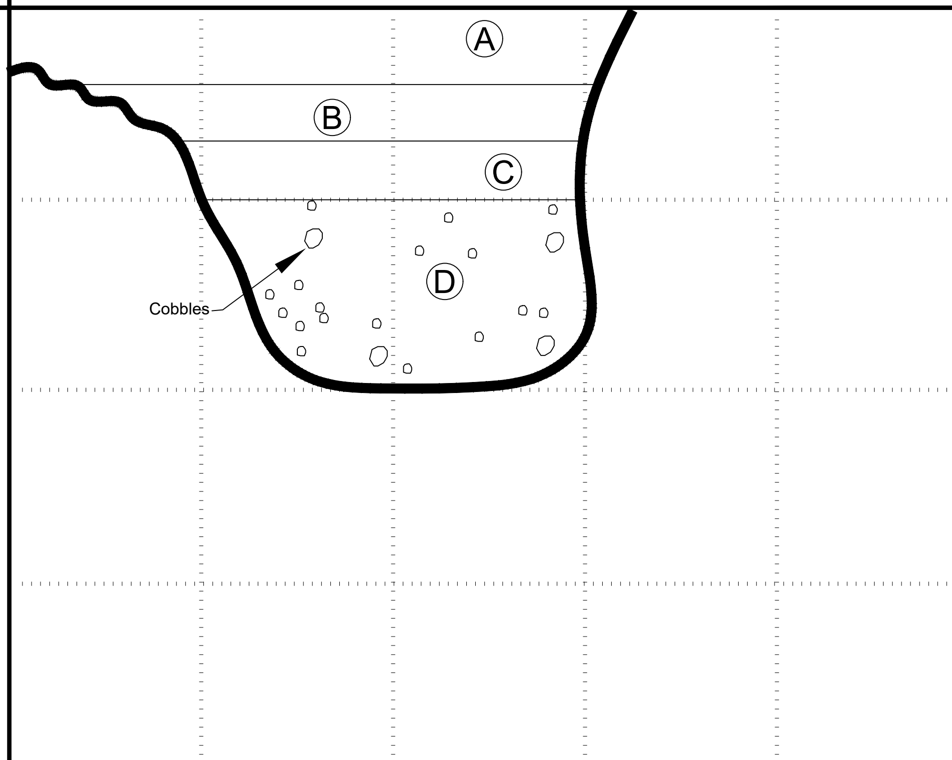
JOB NO.: 23G142-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Industrial Building	LOGGED BY: Caleb Brackett	SEEPAGE DEPTH: Dry
LOCATION: San Bernardino, California	ORIENTATION: N 90 W	READINGS TAKEN: At Completion
DATE: 4/28/2023	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: FILL: Dark Brown fine Sandy Silt, loose-moist
	b		8	B: FILL: Brown Silty fine Sand, medium dense-damp to moist
	b		7	
	b		5	C: ALLUVIUM: Red Brown fine to medium Sand, medium dense-damp
5				
	b		5	D: Gray Brown fine to coarse Sand, little fine to coarse Gravel, occasional Cobbles, medium dense-damp
10				Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION

N 90 W

SCALE: 1" = 5'



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-15

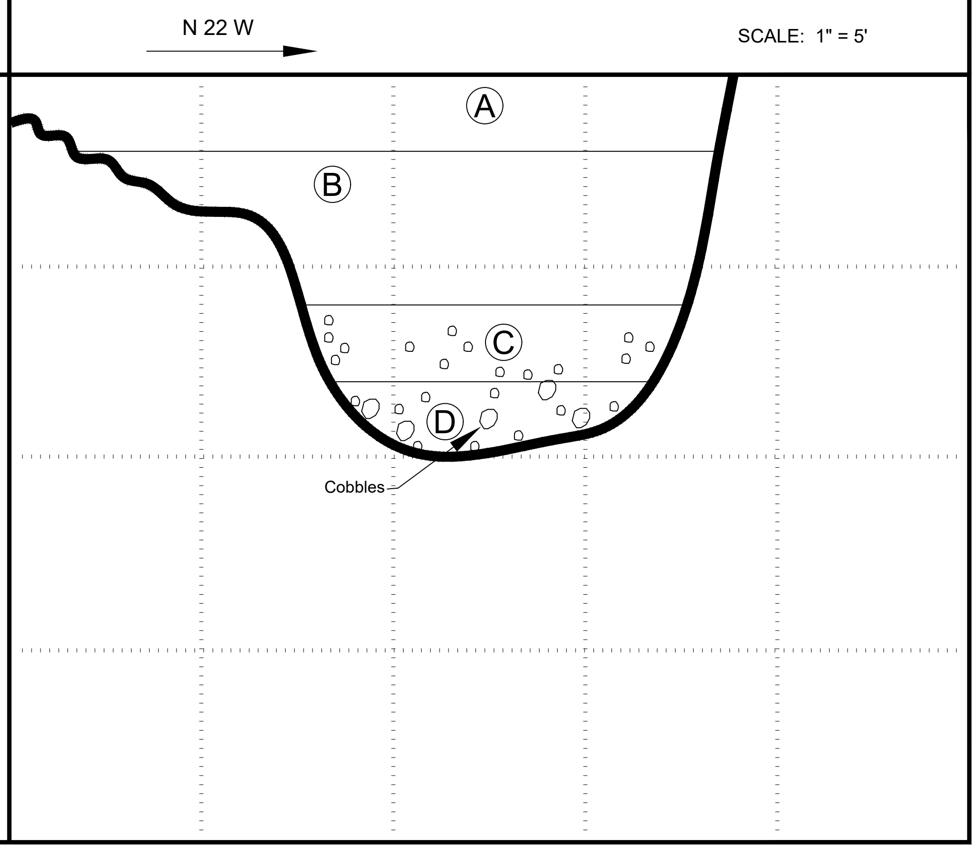
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-6

JOB NO.: 23G142-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Industrial Building	LOGGED BY: Caleb Brackett	SEEPAGE DEPTH: Dry
LOCATION: San Bernardino, California	ORIENTATION: N 22 W	READINGS TAKEN: At Completion
DATE: 4/28/2023	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: FILL: Dark Brown Silty fine Sand, medium dense-moist
	b		9	
				B: ALLUVIUM: Brown fine Sandy Silt, medium dense-very moist
5	b		20	
				C: Brown fine to coarse Sand, trace fine Gravel, medium dense-damp
	b		3	
				D: Brown fine to medium Sand, some fine to coarse Gravel, extensive Cobbles, medium dense to dense-damp
	b		3	
10				Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-16

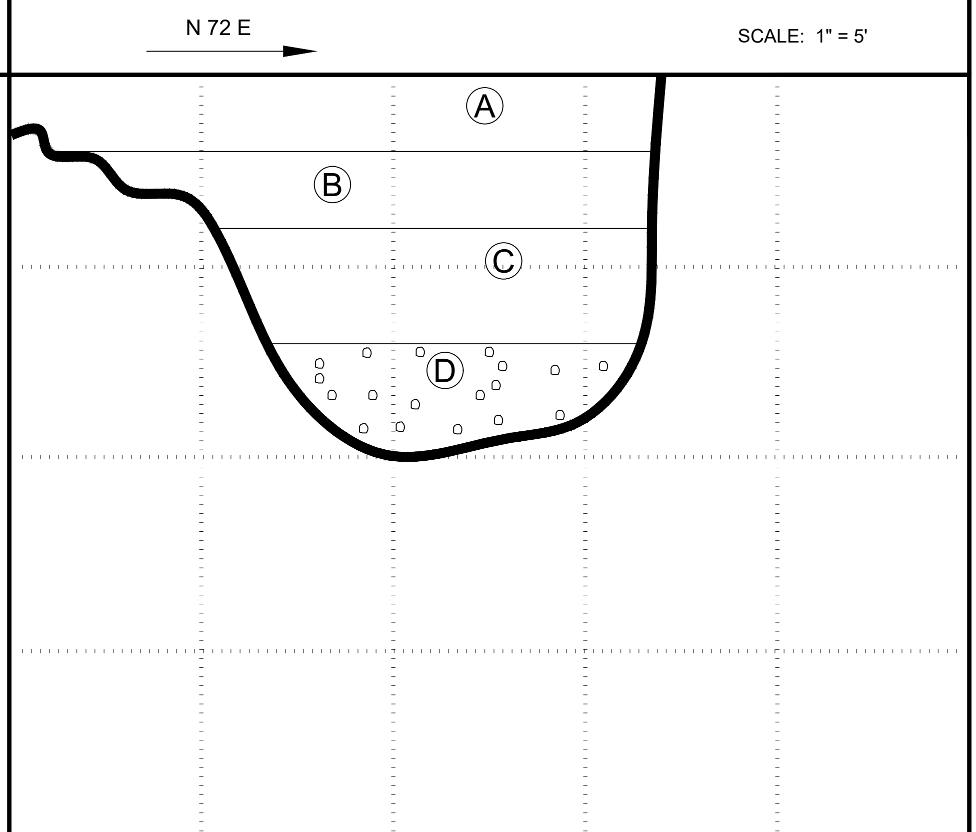
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-7

JOB NO.: 23G142-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Industrial Building	LOGGED BY: Caleb Brackett	SEEPAGE DEPTH: Dry
LOCATION: San Bernardino, California	ORIENTATION: N 72 E	READINGS TAKEN: At Completion
DATE: 4/28/2023	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
				A: FILL: Dark Brown Silty fine Sand, loose-damp
	b		6	
	b		7	B: FILL: Brown fine Sand, little Silt, loose-damp
				C: ALLUVIUM: Brown Silty fine Sand, medium dense-damp
5	b		6	
				D: Brown Silty fine Sand, trace fine Gravel, mottled, medium dense-damp
	b		3	
10				Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-17

SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO. T-8

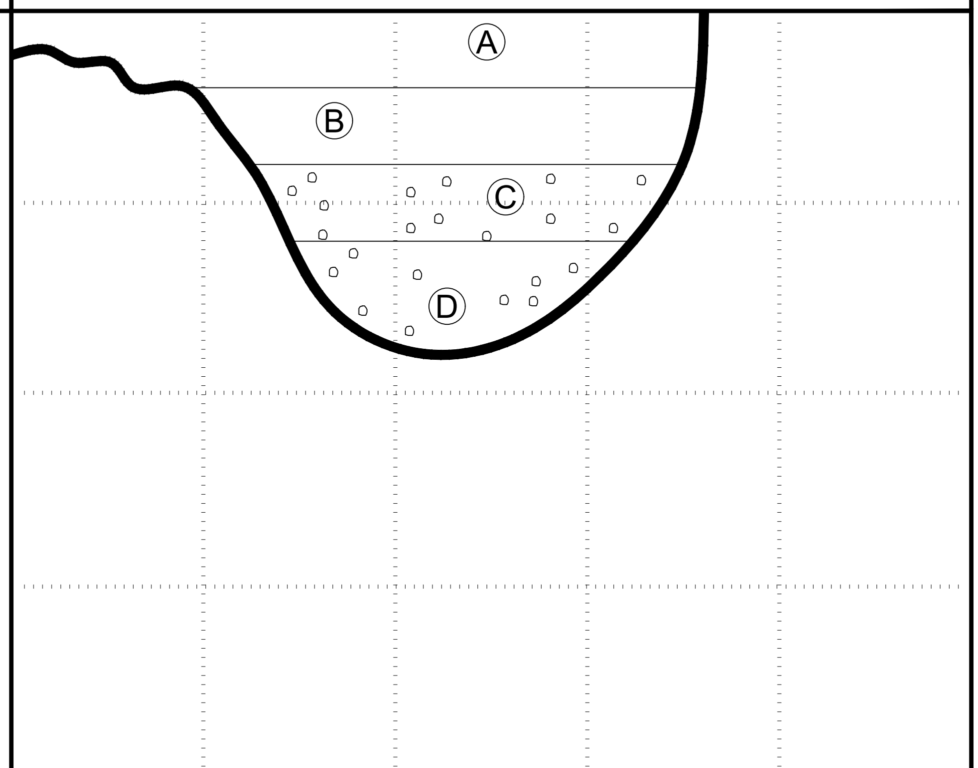
JOB NO.: 23G142-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Industrial Building	LOGGED BY: Caleb Brackett	SEEPAGE DEPTH: Dry
LOCATION: San Bernardino, California	ORIENTATION: N 38 E	READINGS TAKEN: At Completion
DATE: 4/28/2023	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		8	A: FILL: Dark Brown Silty fine Sand, loose-moist
	b		6	B: FILL: Brown fine Sand, little Silt, loose-damp
5	b		5	C: ALLUVIUM: Red Brown fine to medium Sand, trace fine Gravel, medium dense-damp
	b		4	D: Gray Brown, fine to coarse Sand, little fine to coarse Gravel, medium dense-damp
				Trench Terminated @ 9 feet
10				
15				

GRAPHIC REPRESENTATION

N 38 E

SCALE: 1" = 5'



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-18

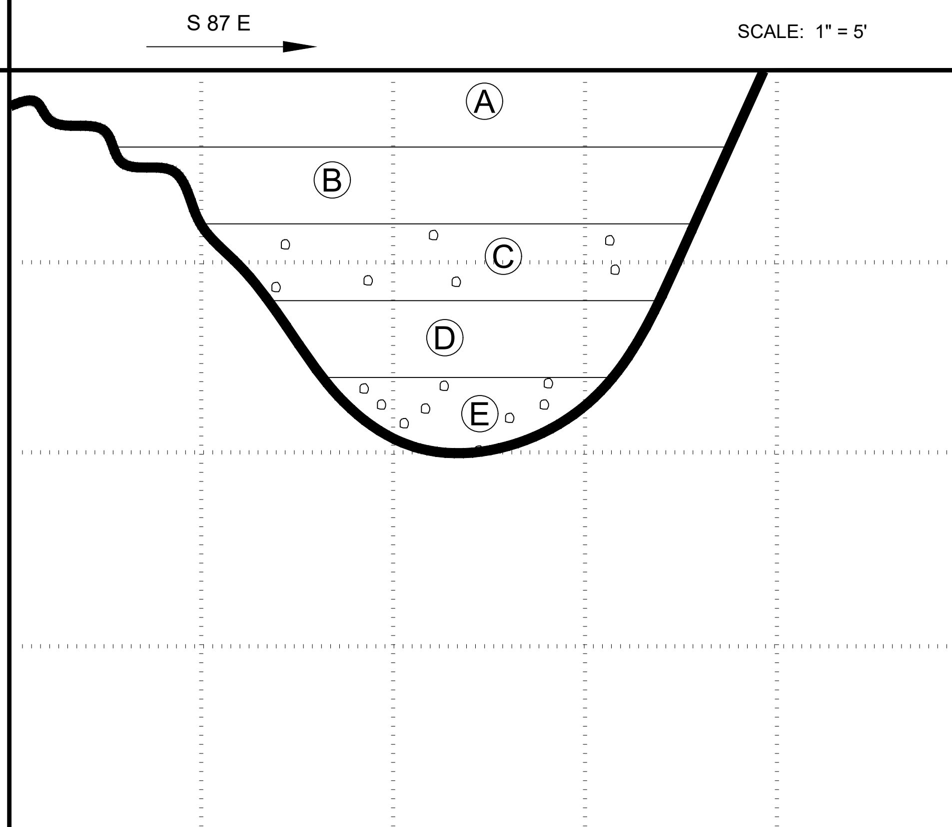
SOUTHERN CALIFORNIA GEOTECHNICAL

TRENCH NO.
T-9

JOB NO.: 23G142-1	EQUIPMENT USED: Backhoe	WATER DEPTH: Dry
PROJECT: Proposed Industrial Building	LOGGED BY: Caleb Brackett	SEEPAGE DEPTH: Dry
LOCATION: San Bernardino, California	ORIENTATION: S 87 E	READINGS TAKEN: At Completion
DATE: 4/28/2023	ELEVATION: ---	

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		9	A: FILL: Dark Brown Silty fine Sand, loose-moist
	b		12	B: FILL: Dark Brown fine Sand Silt, loose-moist
				C: ALLUVIUM: Brown fine Sand, trace Silt, trace fine Gravel, medium dense-damp
5	b		5	
				D: Light Red Brown fine to medium Sand, trace coarse Sand, medium dense-damp
	b		5	
				E: Light Red Brown fine to coarse Sand, little to some fine to coarse Gravel, medium dense-dry to damp
	b		2	
10				Trench Terminated @ 10 feet
15				

GRAPHIC REPRESENTATION



KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-19

SOUTHERN CALIFORNIA GEOTECHNICAL

**TRENCH NO.
T-10**

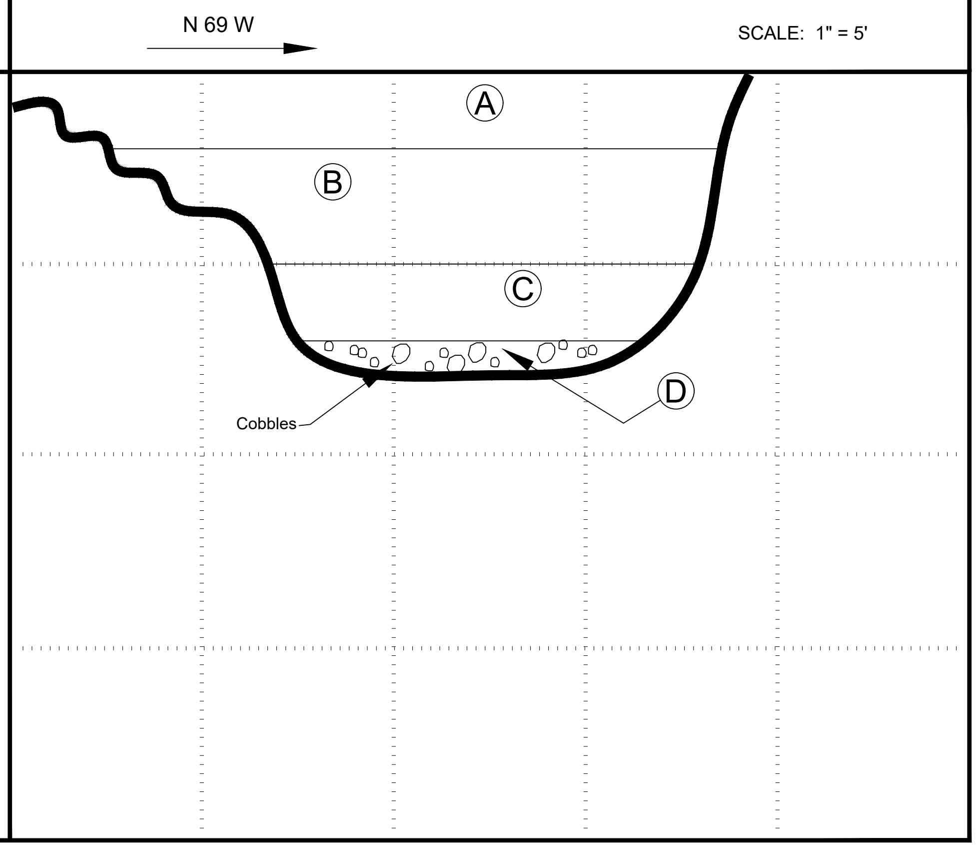
JOB NO.: 23G142-1
PROJECT: Proposed Industrial Building
LOCATION: San Bernardino, California
DATE: 4/28/2023

EQUIPMENT USED: Backhoe
LOGGED BY: Caleb Brackett
ORIENTATION: N 69 W
ELEVATION: ---

WATER DEPTH: Dry
SEEPAGE DEPTH: Dry
READINGS TAKEN: At Completion

DEPTH	SAMPLE	DRY DENSITY (PCF)	MOISTURE (%)	EARTH MATERIALS DESCRIPTION
	b		9	A: FILL: Dark Brown fine Sandy Silt, trace coarse Sand, loose-moist
	b		6	B: ALLUVIUM: Brown fine Sand, trace Silt, medium dense-damp
5				C: Brown fine to medium Sand, medium dense-damp
	b		4	D: Red Brown fine to coarse Sand, trace fine to coarse Gravel, extensive Cobbles, medium dense-damp
	b		4	
				Trench Terminated @ 8 feet due to caving
10				
15				

GRAPHIC REPRESENTATION



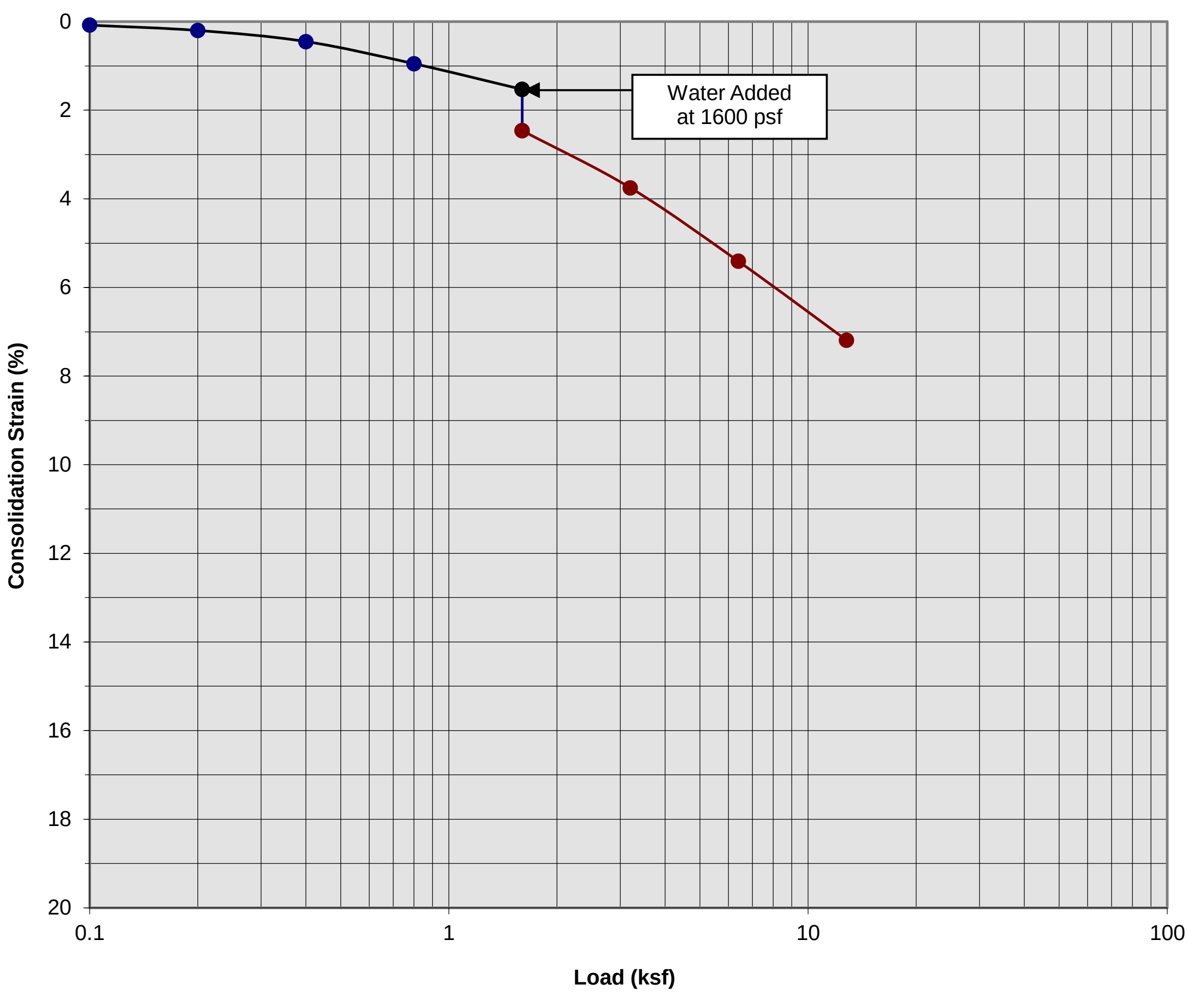
KEY TO SAMPLE TYPES:
B - BULK SAMPLE (DISTURBED)
R - RING SAMPLE 2-1/2" DIAMETER
(RELATIVELY UNDISTURBED)

TRENCH LOG

PLATE B-20

APPENDIX C

Consolidation/Collapse Test Results



Classification: FILL: Dark Gray Brown Silty fine Sand, trace to little medium Sand

Boring Number:	B-6	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	3 to 4	Initial Dry Density (pcf)	114.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	122.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.93

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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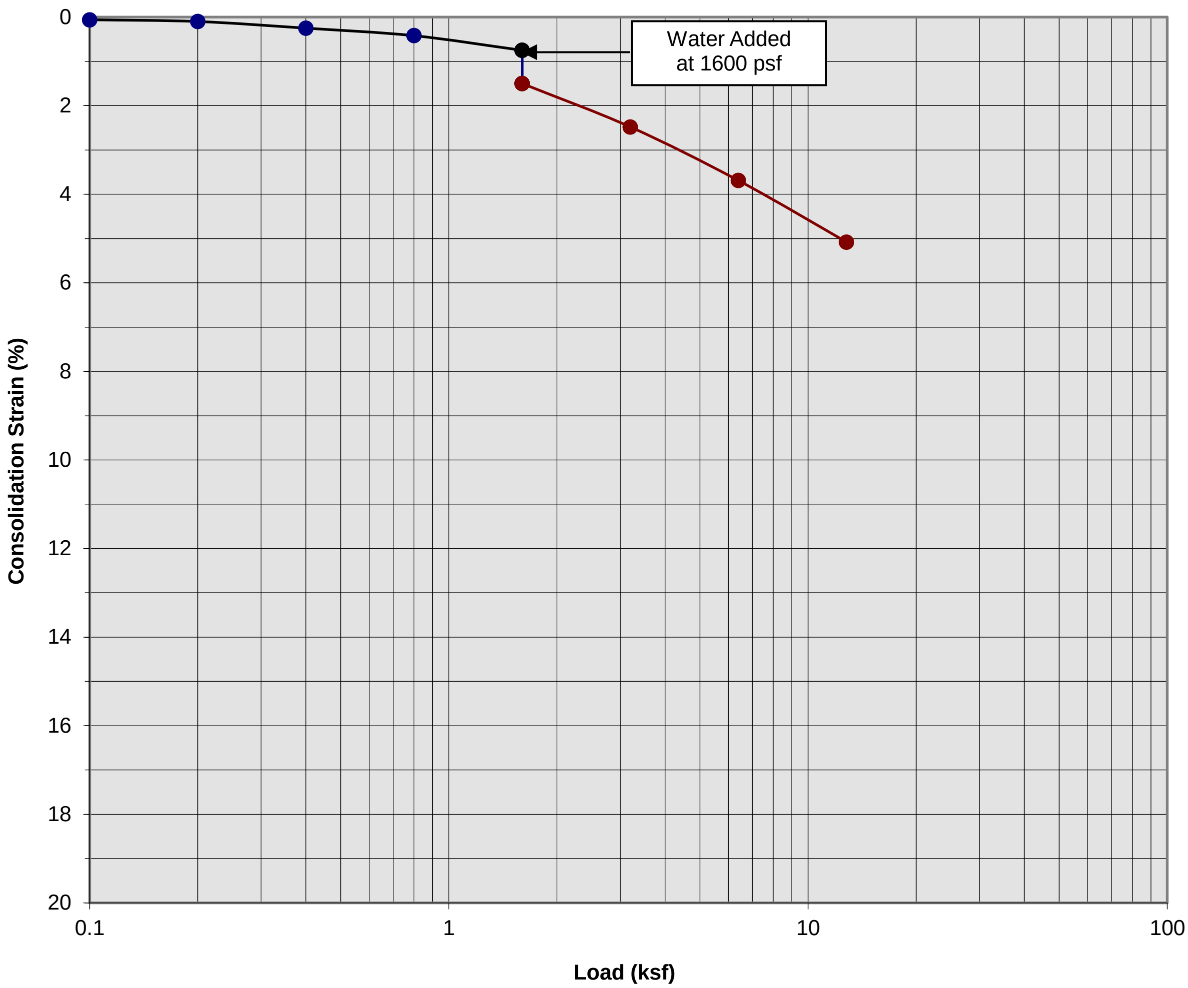
Consolidation/Collapse Test Results



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

Boring Number:	B-6	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	109.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.75

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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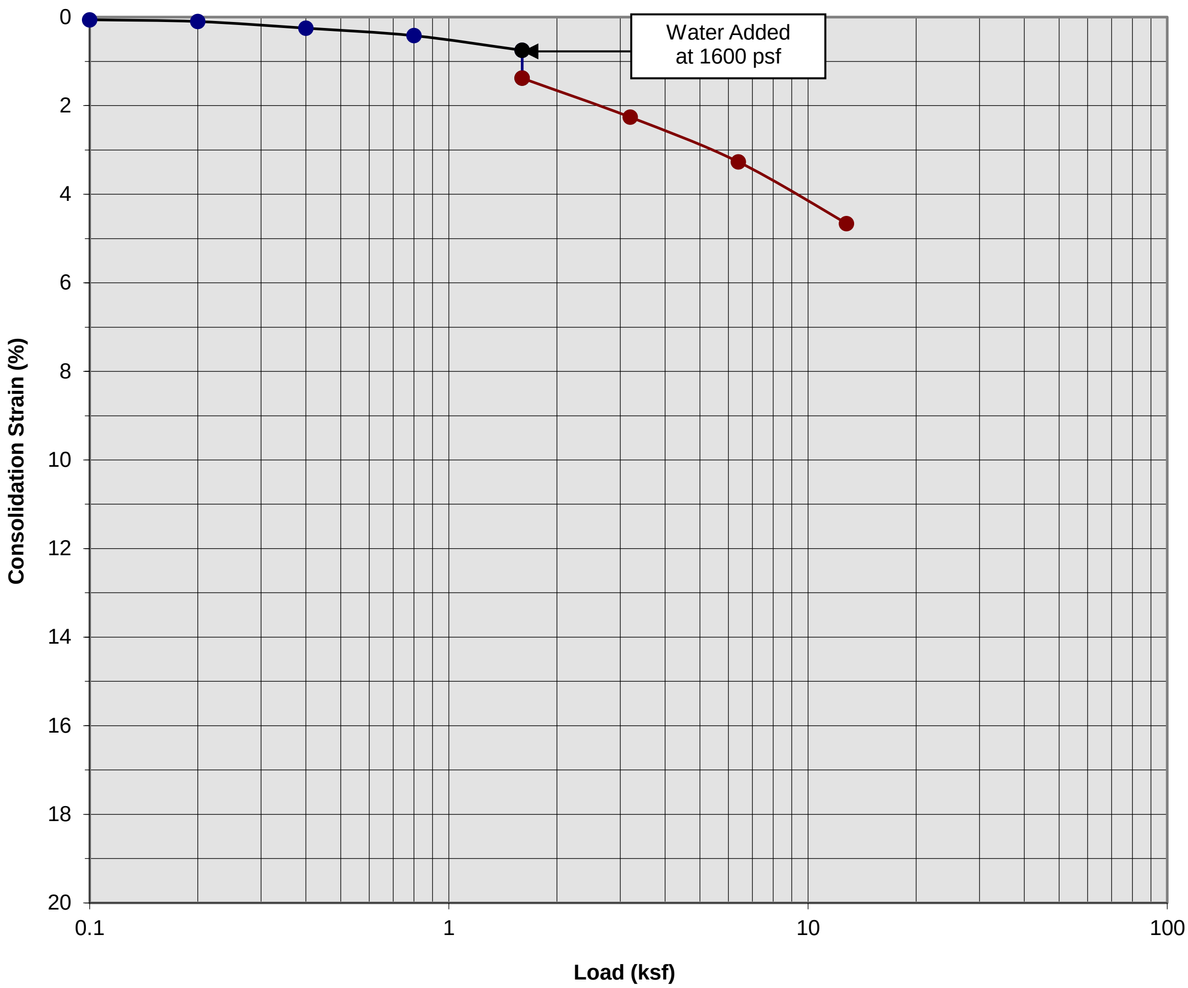
Consolidation/Collapse Test Results



Classification: Light Brown fine to coarse Sand

Boring Number:	B-6	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	7 to 8	Initial Dry Density (pcf)	115.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.63

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C- 3

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SOUTHERN CALIFORNIA GEOTECHNICAL
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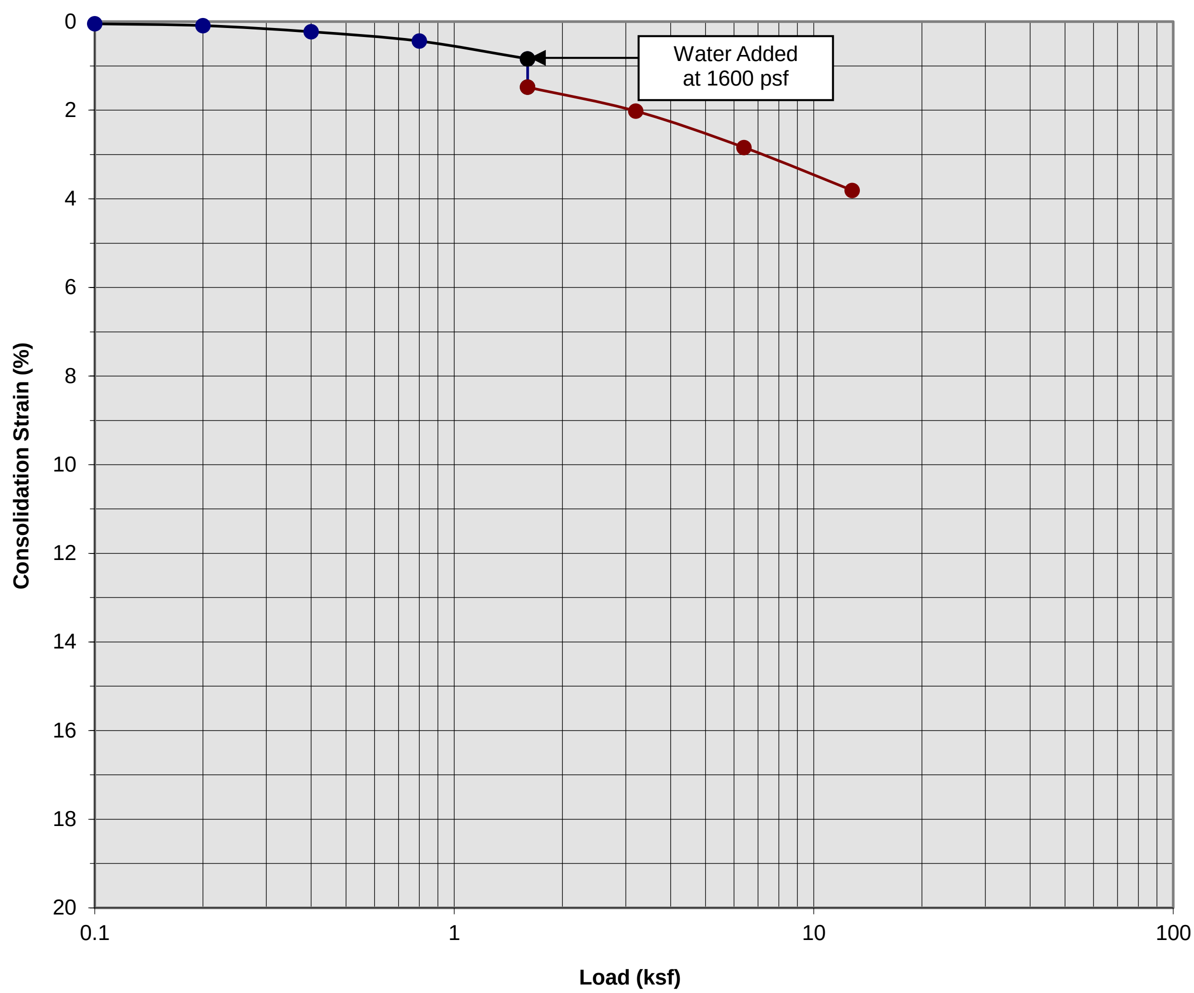
Consolidation/Collapse Test Results



Classification: Light Brown fine to coarse Sand

Boring Number:	B-6	Initial Moisture Content (%)	4
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	9 to 10	Initial Dry Density (pcf)	111.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	116.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.64

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C- 4

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SOUTHERN CALIFORNIA GEOTECHNICAL
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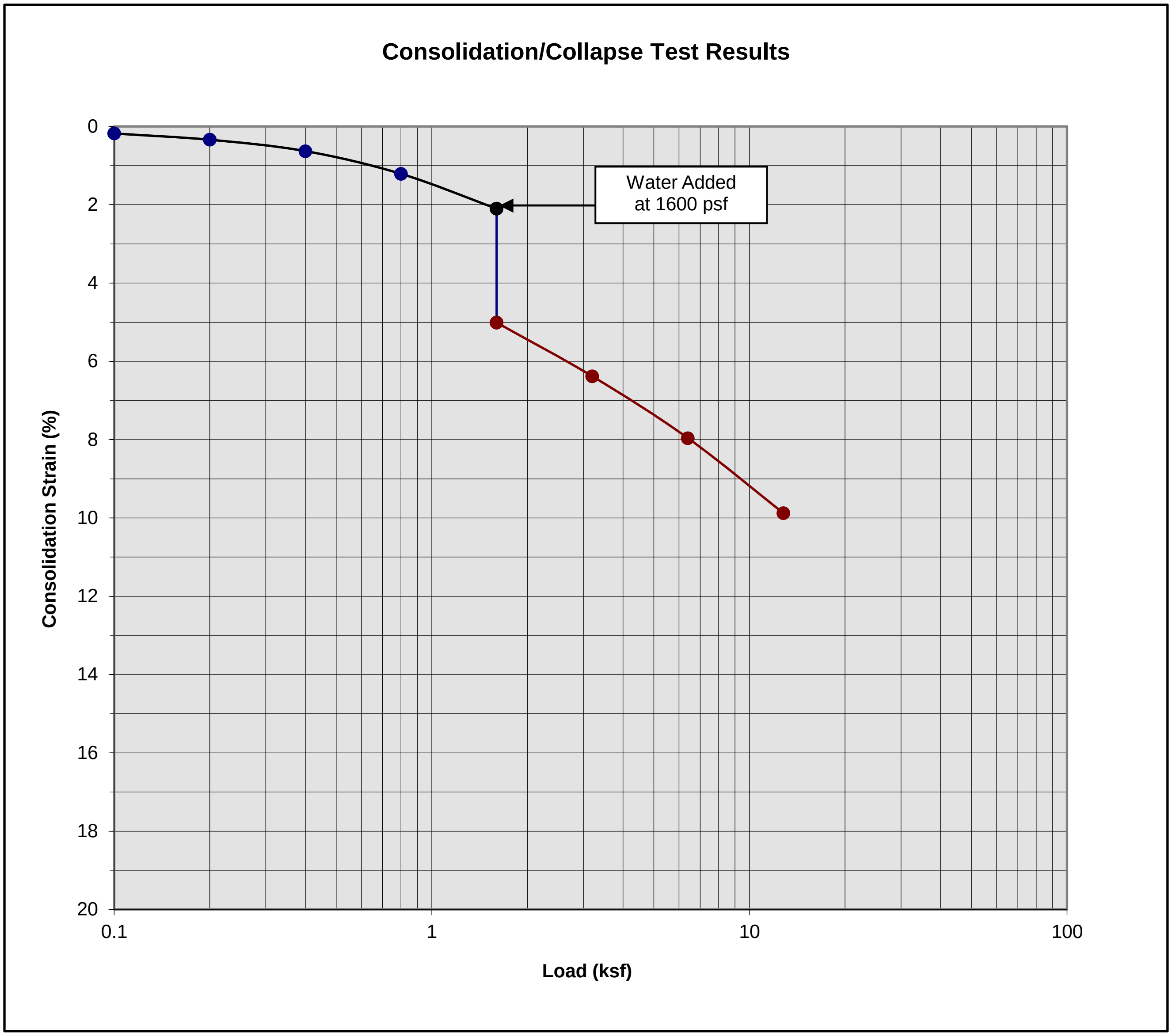

Classification: FILL: Brown Silty fine Sand, trace to little medium Sand

Boring Number:	B-7	Initial Moisture Content (%)	6
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	3 to 4	Initial Dry Density (pcf)	100.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.91

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C- 5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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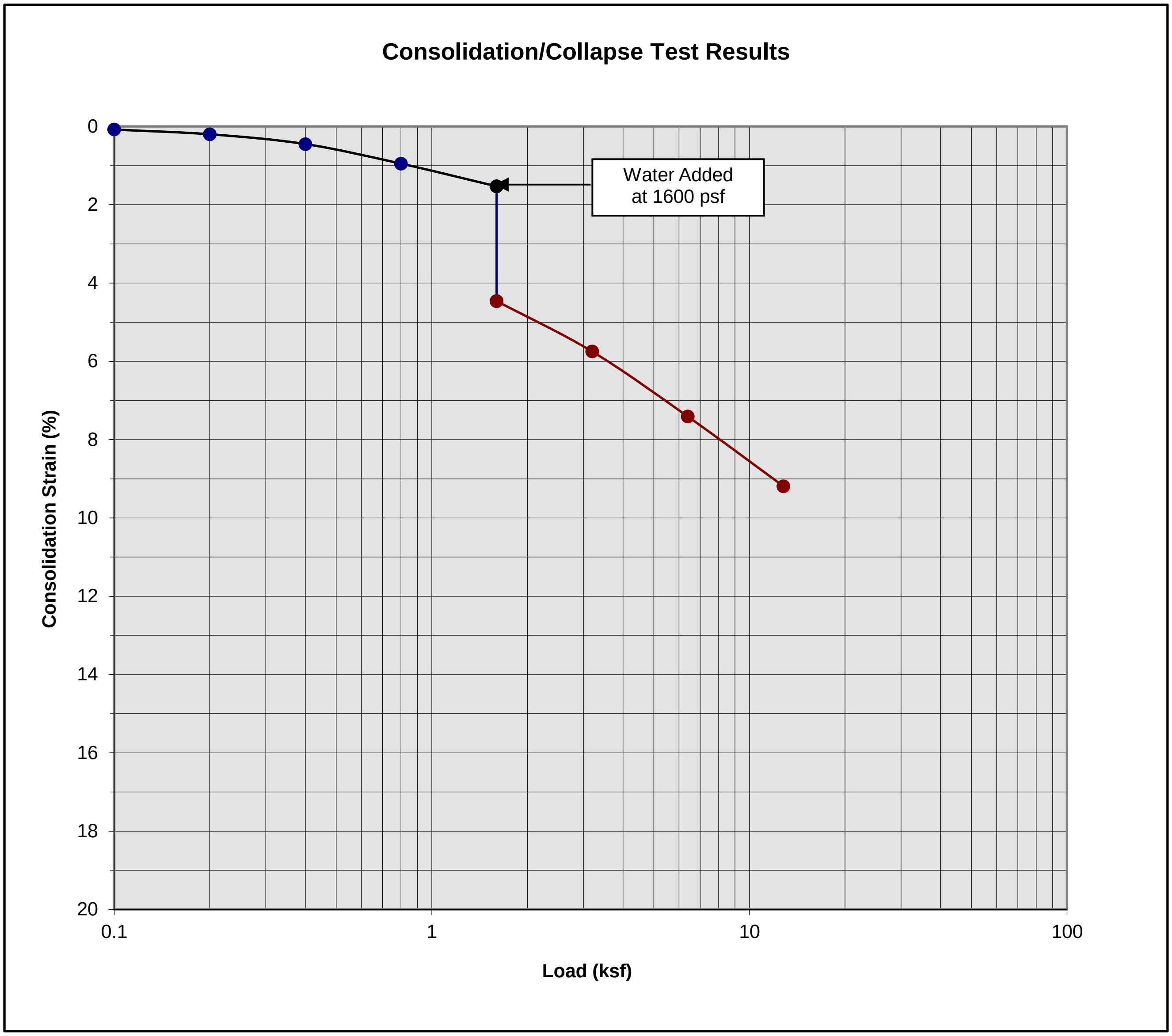
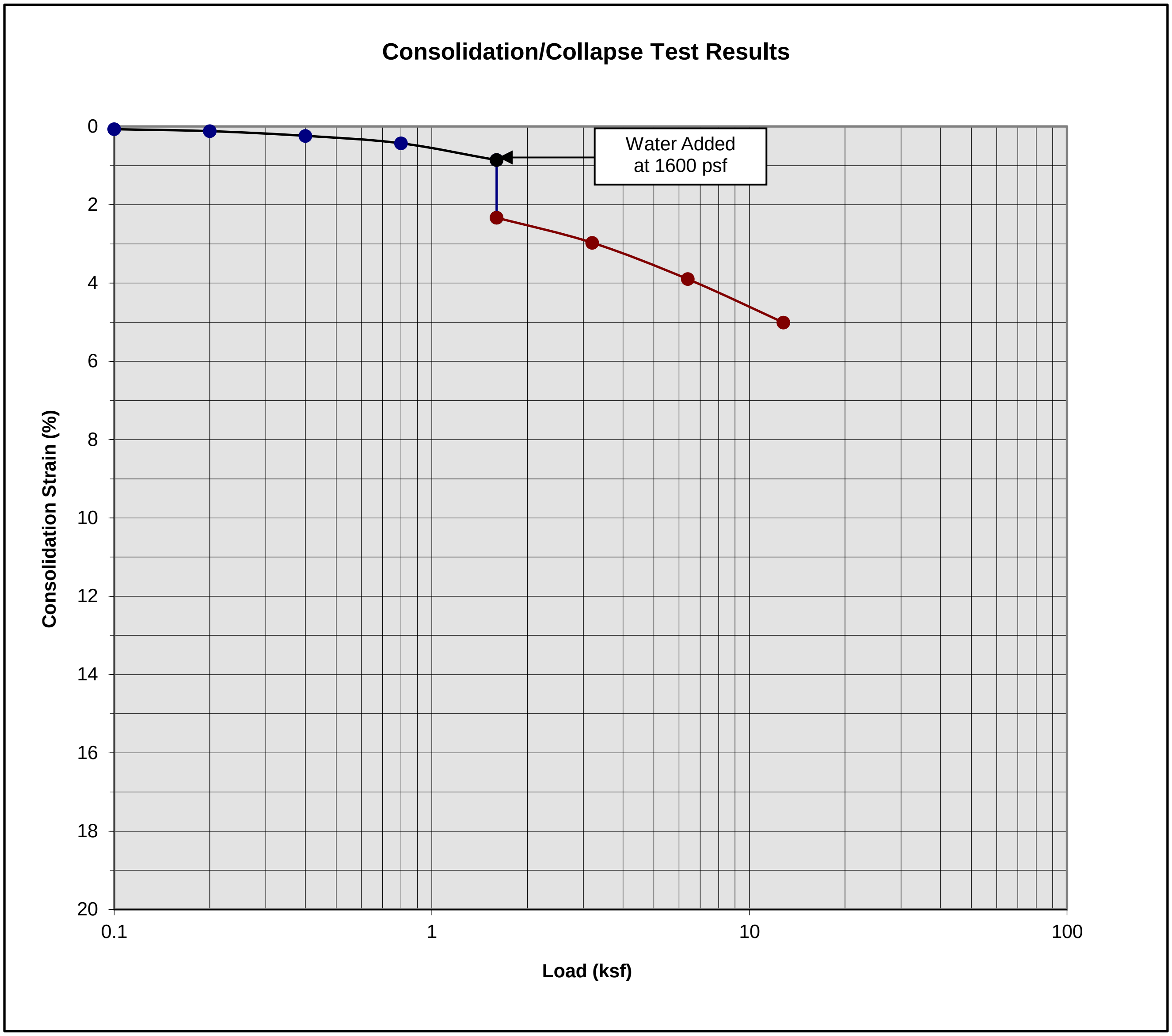


Classification: Light Red Brown fine to medium Sand, trace Silt, trace coarse Sand

Boring Number:	B-7	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	5 to 6	Initial Dry Density (pcf)	102.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.93

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C- 6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



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

Boring Number:	B-7	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	7 to 8	Initial Dry Density (pcf)	100.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	105.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.47

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C- 7

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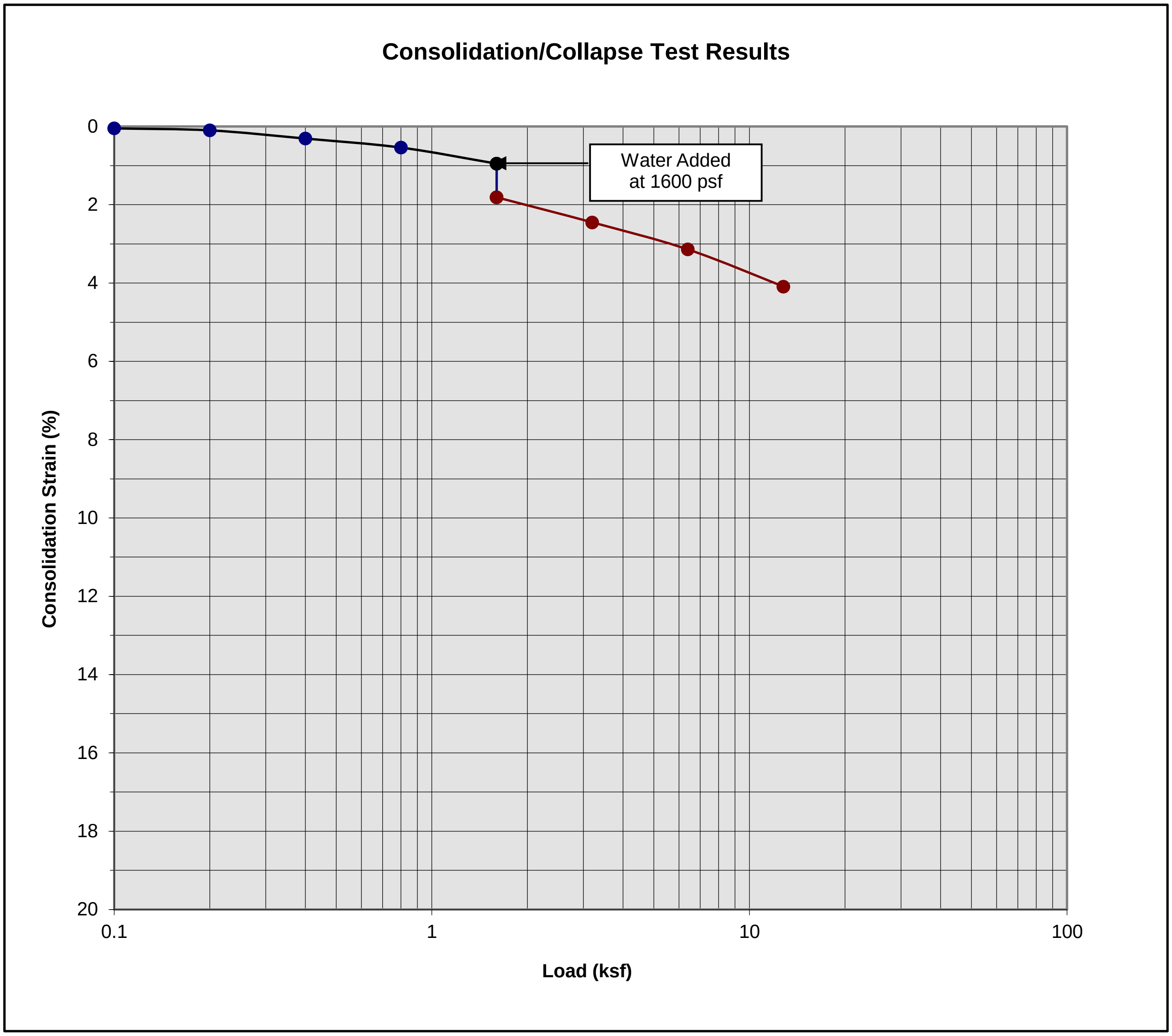
CALIFORNIA
GEOTECHNICAL
A California Corporation

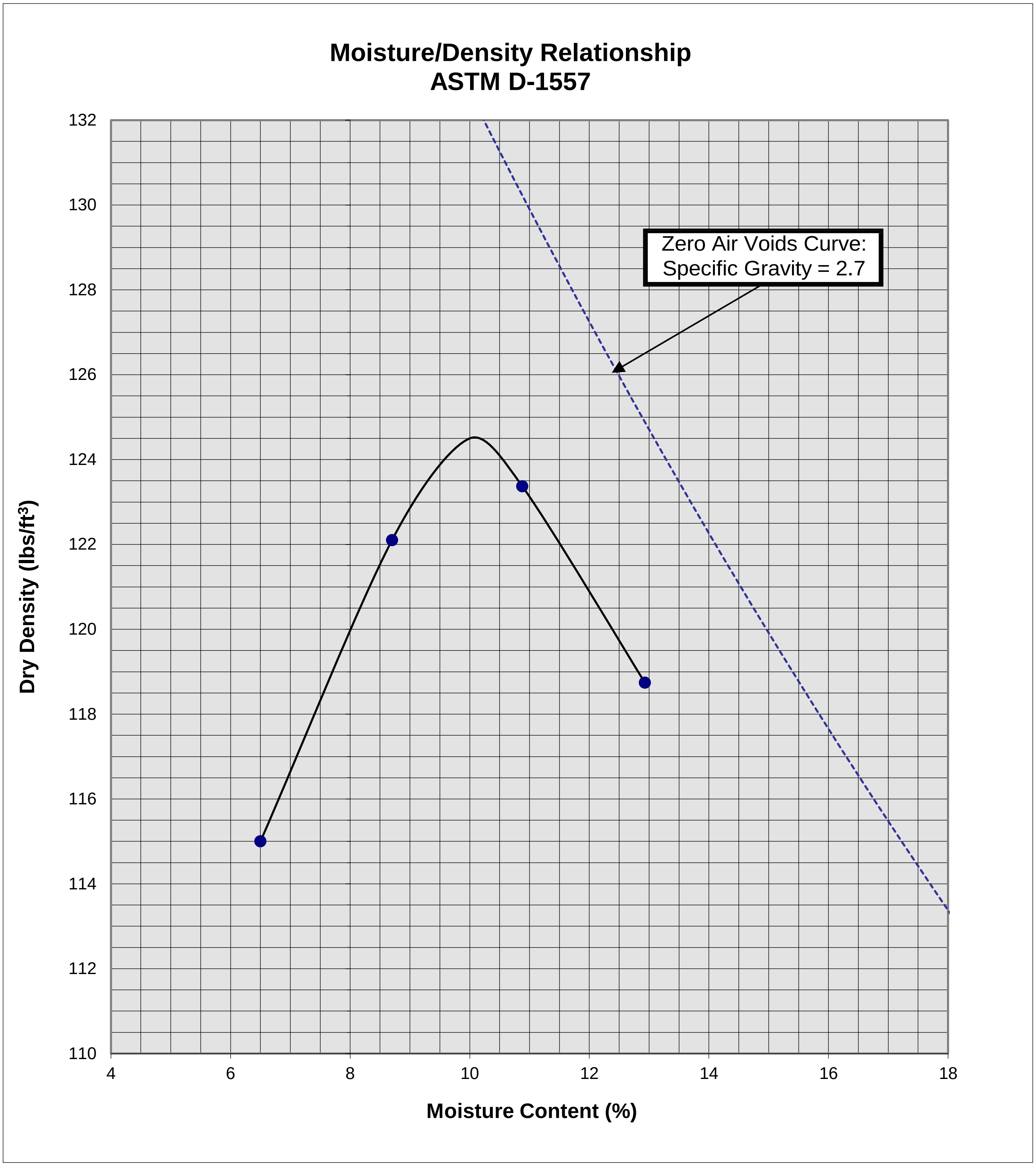


Classification: Light Brown fine to coarse Sand

Boring Number:	B-7	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	9 to 10	Initial Dry Density (pcf)	107.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	111.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.86

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C- 8

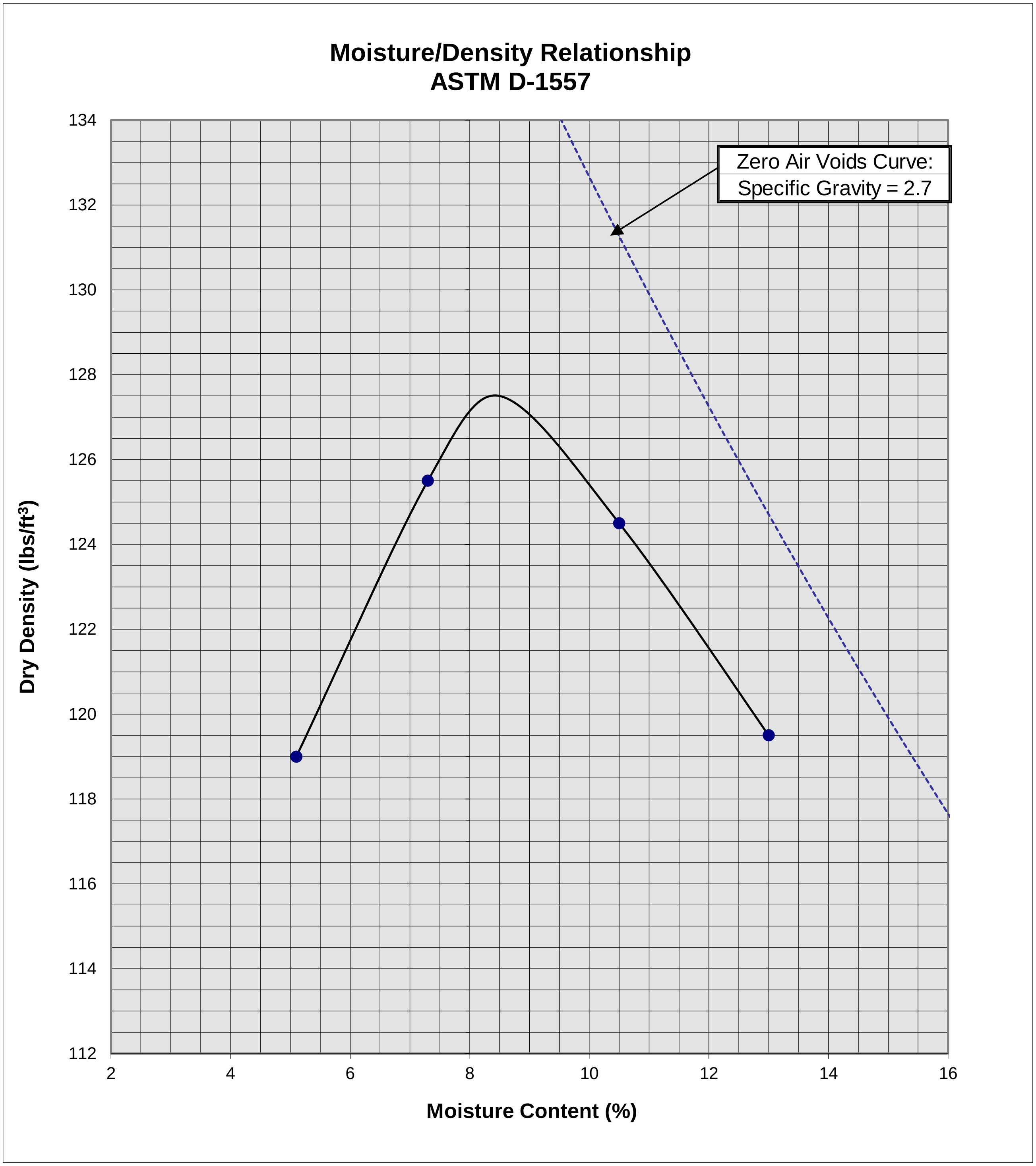
SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-1 @ 1-5'
Optimum Moisture (%)	10
Maximum Dry Density (pcf)	124.5
Soil Classification	Brown Silty fine Sand, little medium Sand, trace coarse Sand

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-7 @ 1-5'
Optimum Moisture (%)	8.5
Maximum Dry Density (pcf)	127.5
Soil Classification	Dark Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel

Proposed Industrial Building
San Bernardino, California
Project No. 23G142-1
PLATE C-10

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation

APPENDIX D

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.

Compacted Fills

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

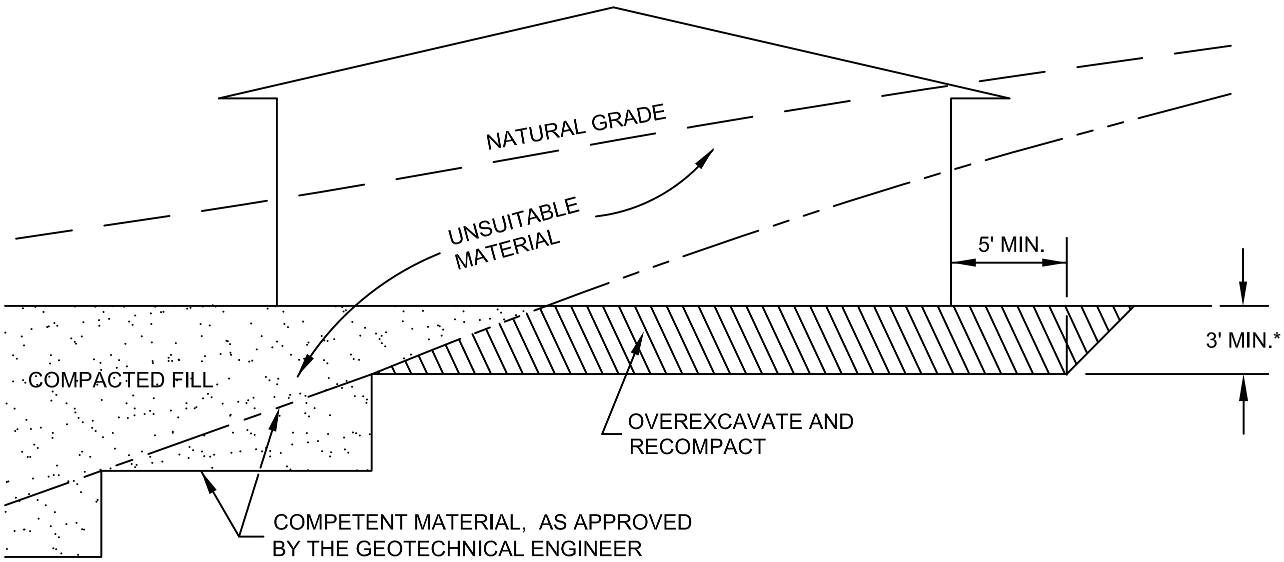
Subdrains

- 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.

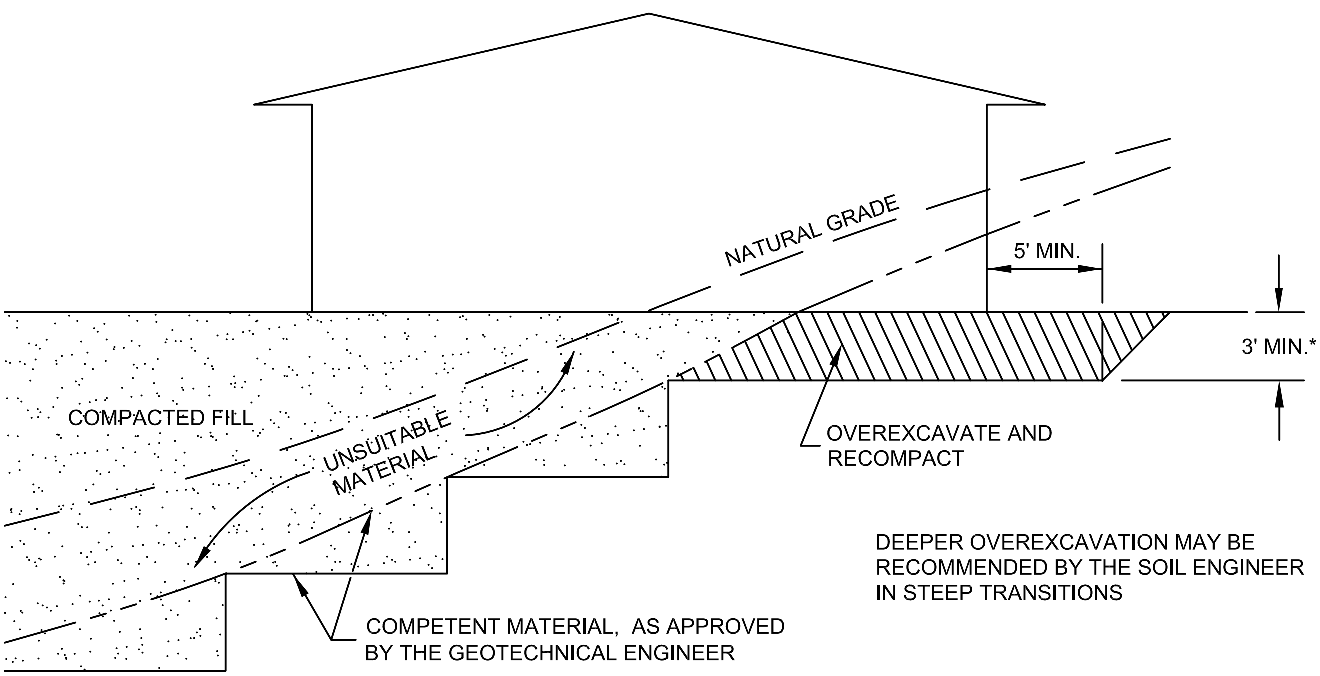
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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

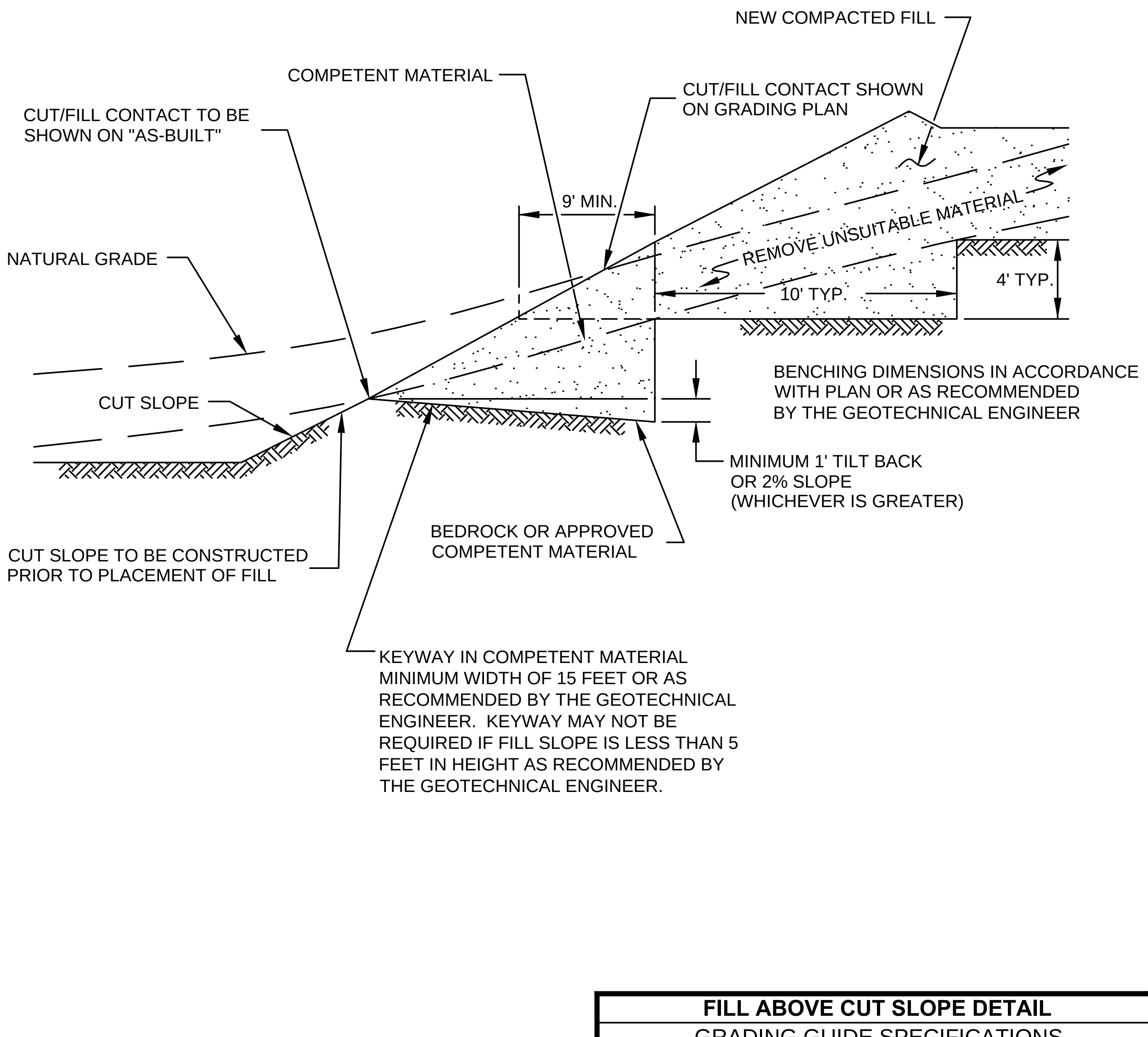
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

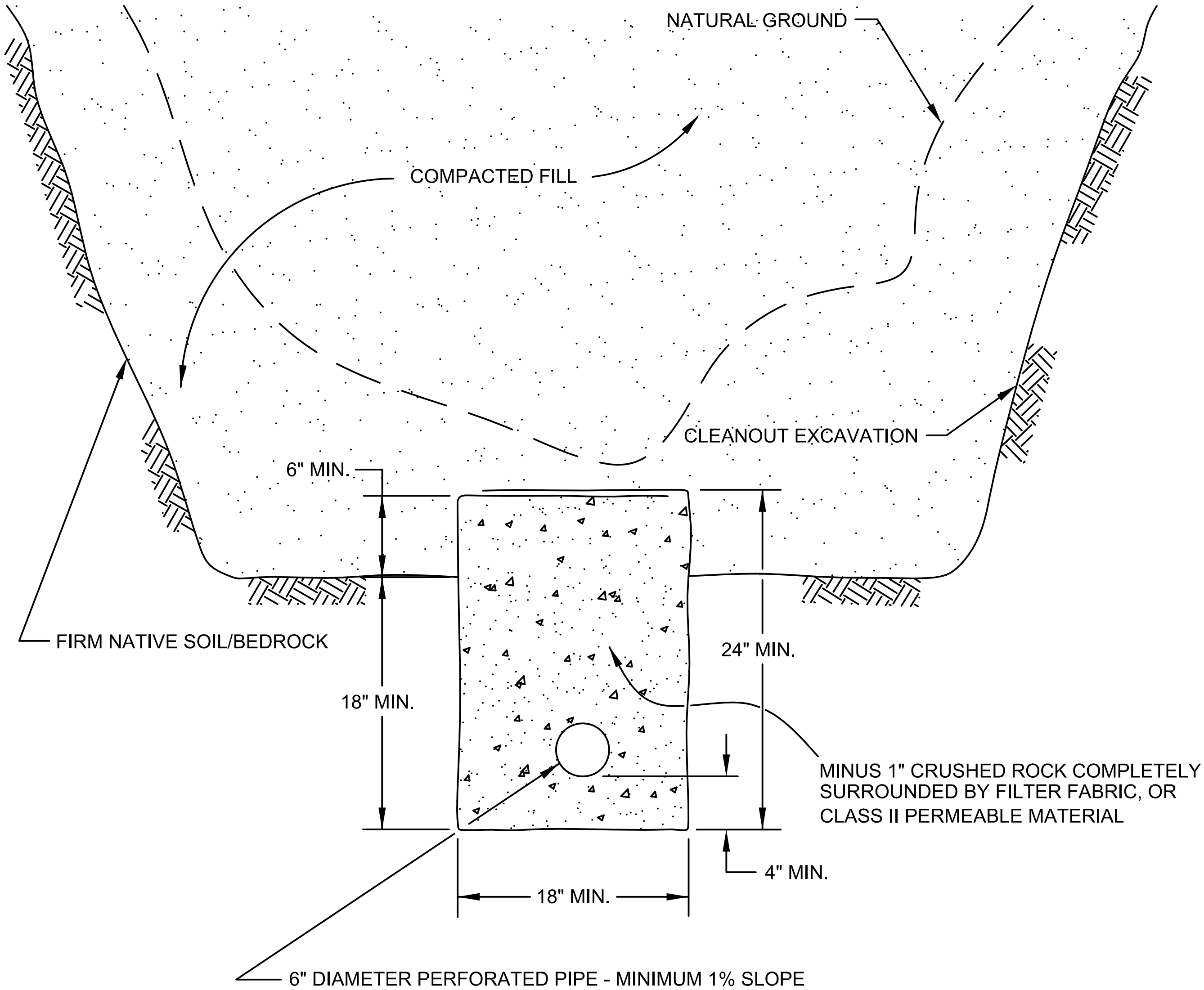
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

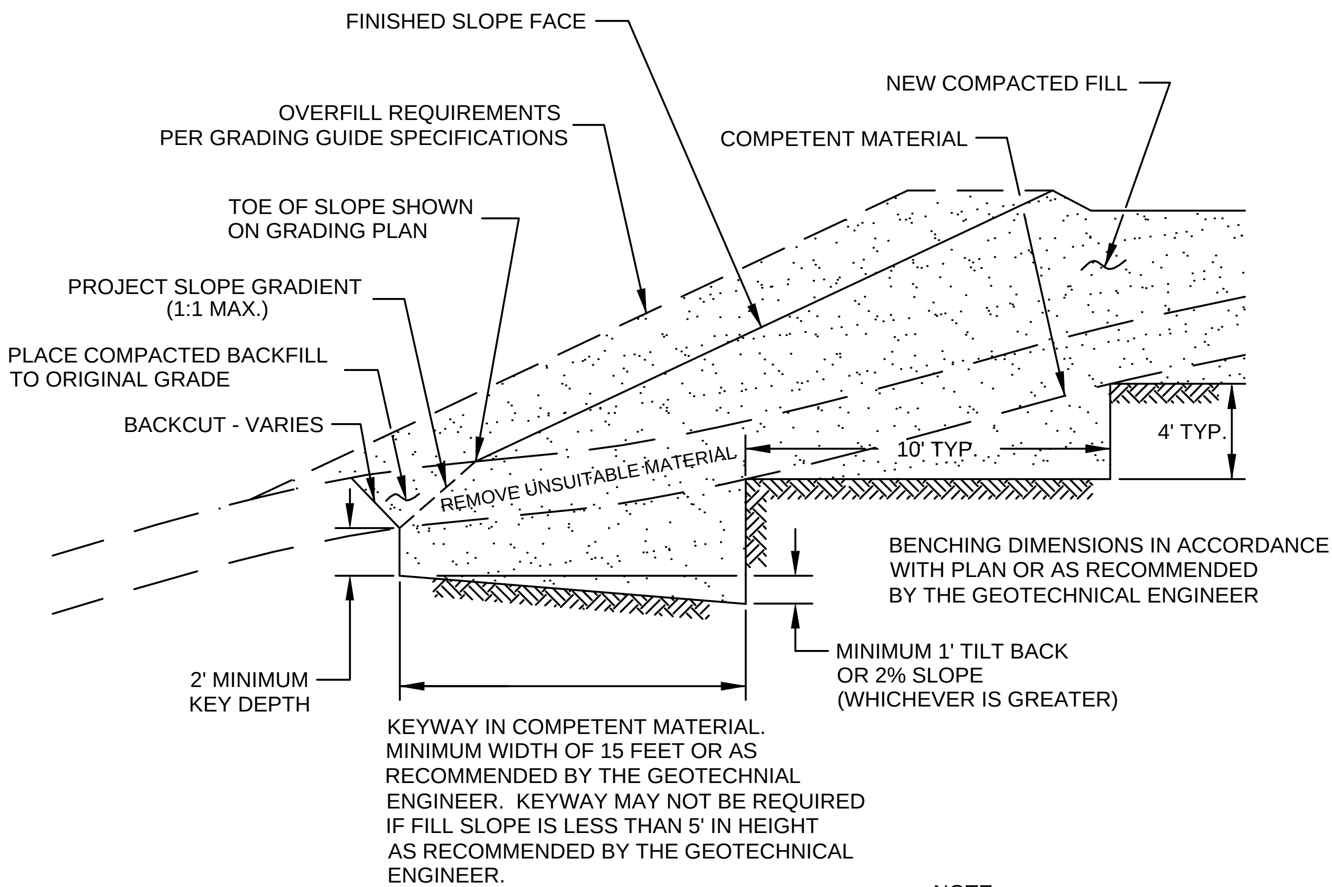
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

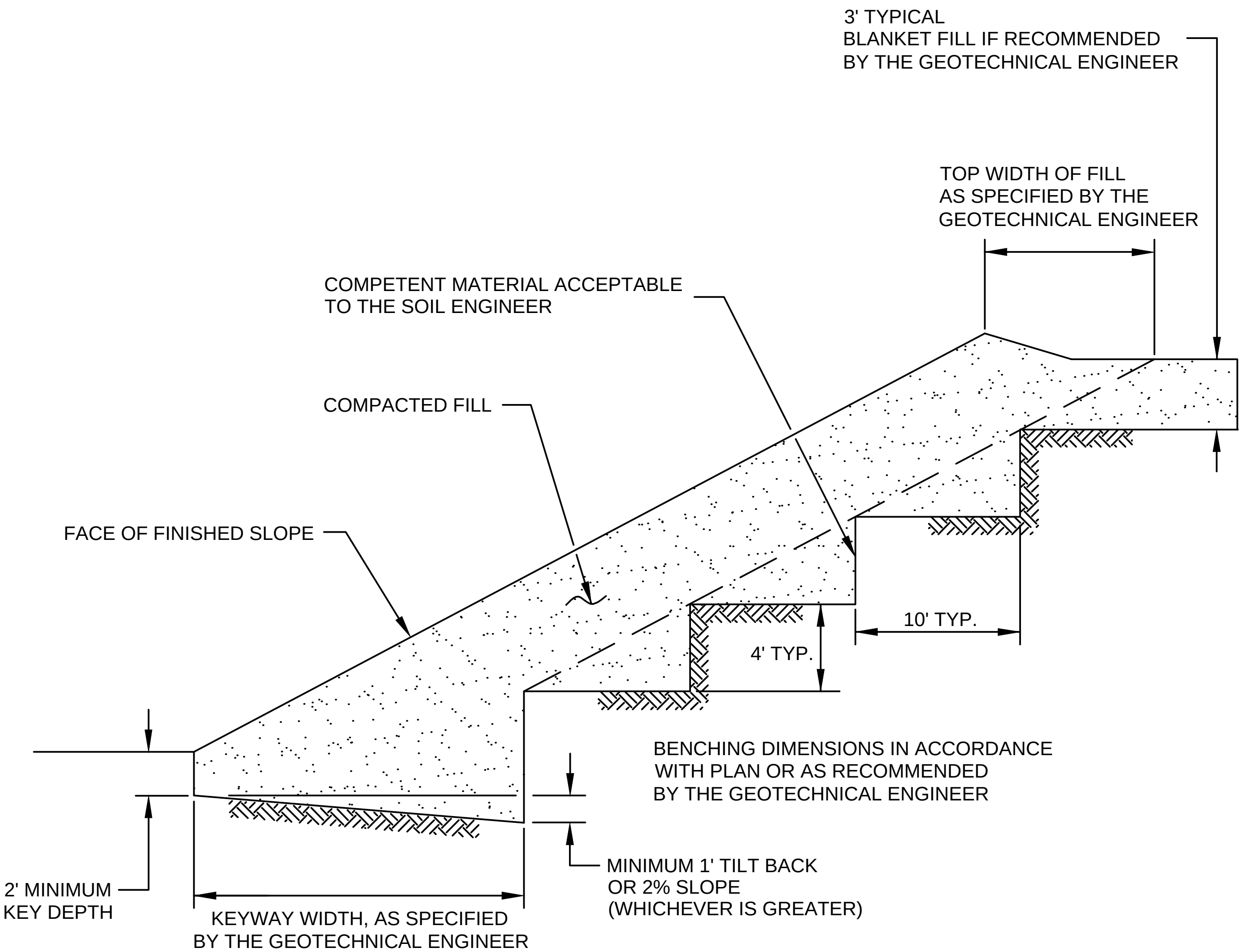
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

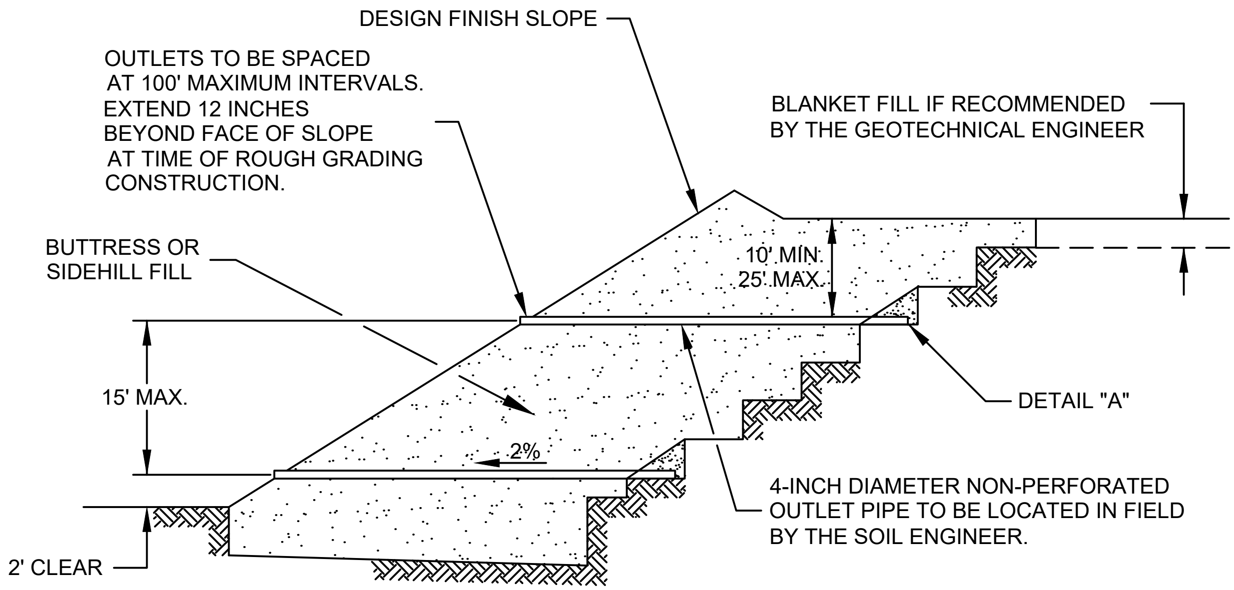
DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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



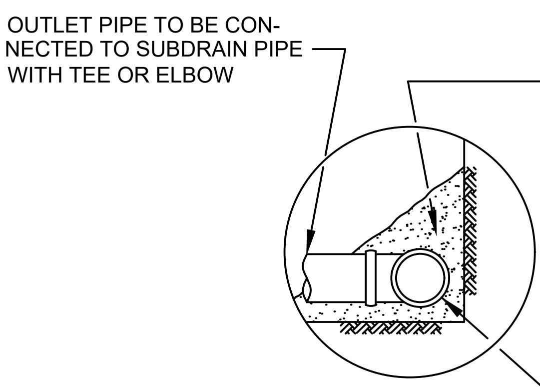
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50



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

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

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

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

DETAIL "A"

NOTES:

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

SLOPE FILL SUBDRAINS

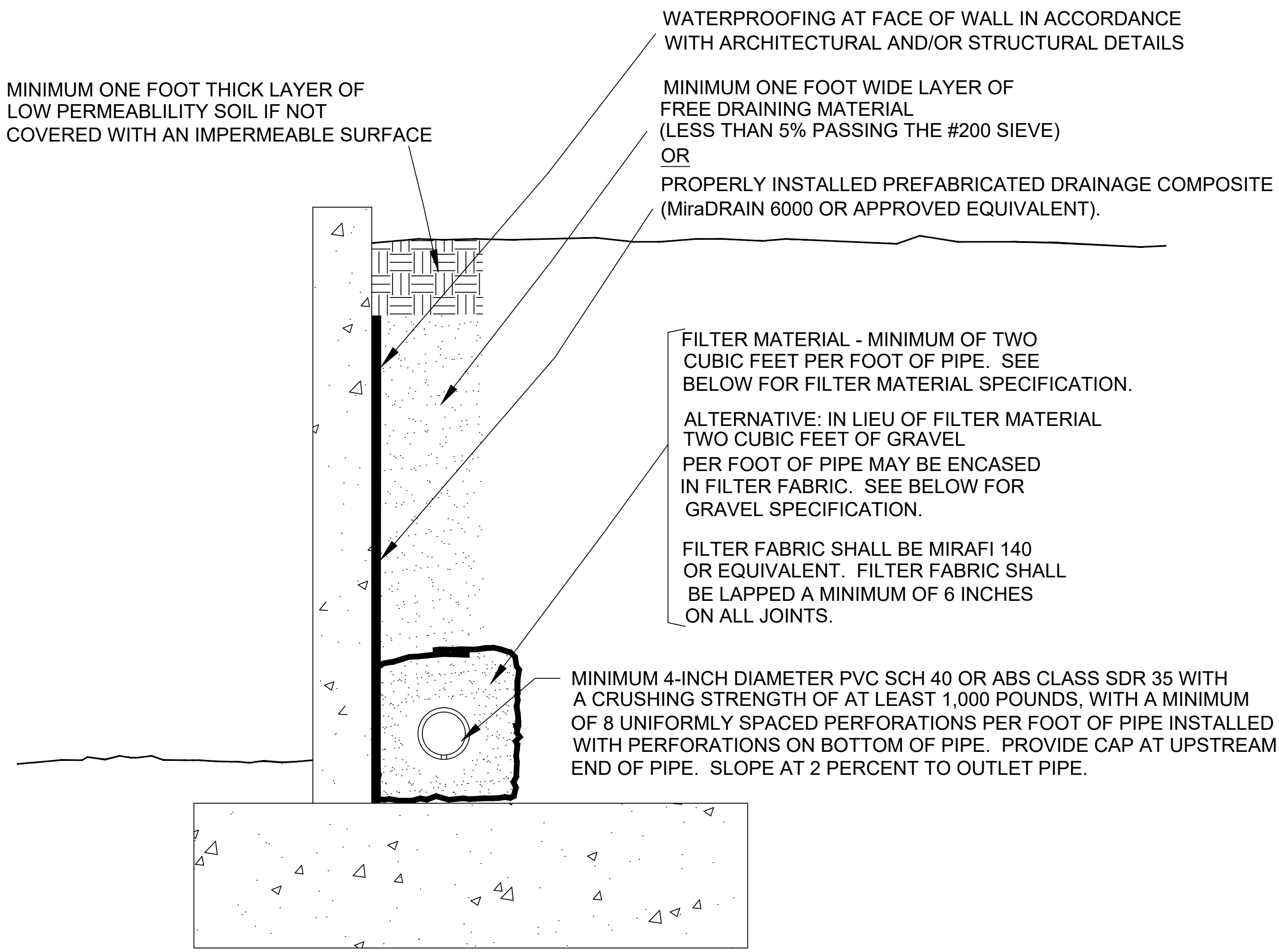
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

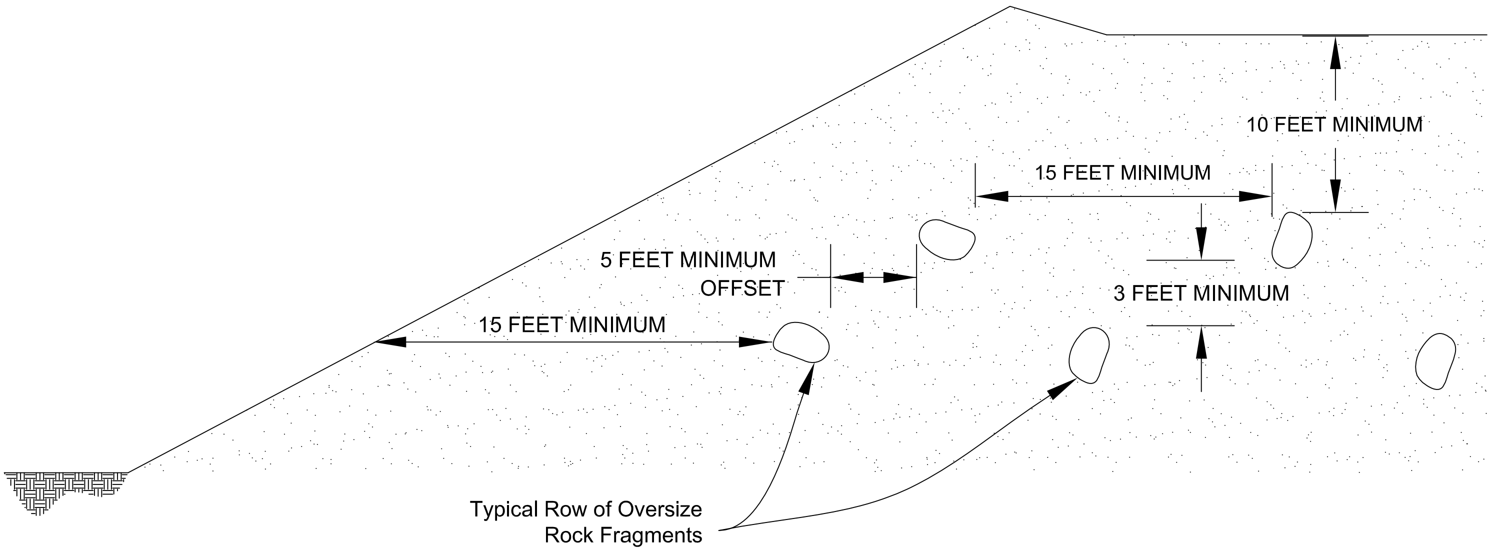
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

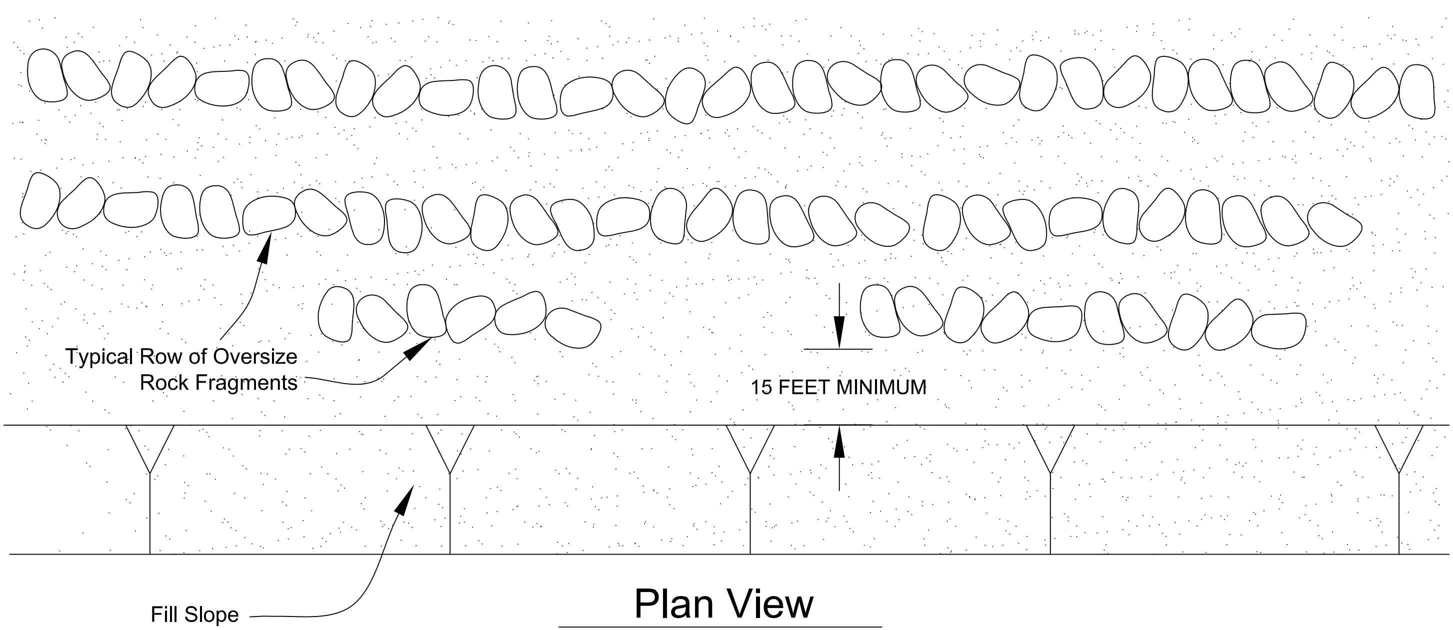
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

Latitude, Longitude: 34.10929205, -117.23972003

Date	5/9/2023, 2:28:44 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.286	MCE_R ground motion. (for 0.2 second period)
S_1	0.841	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.286	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.524	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	0.942	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	1.036	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.786	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	3.049	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.286	Factored deterministic acceleration value. (0.2 second)
S1RT	1.111	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.249	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.841	Factored deterministic acceleration value. (1.0 second)
PGAd	0.942	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	1.203	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.914	Mapped value of the risk coefficient at short periods
C_{R1}	0.889	Mapped value of the risk coefficient at a period of 1 s
C_V	1.5	Vertical coefficient

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

SEISMIC DESIGN PARAMETERS - 2022 CBC

PROPOSED INDUSTRIAL BUILDING

SAN BERNARDINO, CALIFORNIA

DRAWN: MK
CHKD: RF

SCG PROJECT
23G142-1

PLATE E-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Building	MCE_G Design Acceleration	1.036 (g)
Project Location	San Bernardino, California	Design Magnitude	7.24
Project Number	23G142-1	Historic High Depth to Groundwater	37 (ft)
Engineer	Ricardo Frias	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-1		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.24)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	37	18.5		120		1.3	1.05	1.1	0.96	0.75	0.0	0.0	2220	2220	2220	0.94	1.01	1	0.06	0.06	N/A	N/A	Above Water Table
39.5	37	42	39.5	33	120		1.3	1.05	1.3	0.82	1	47.7	47.7	4740	4584	4740	0.85	1.11	0.77	2.00	1.70	0.59	2.89	Nonliquefiable
44.5	42	47	44.5	27	120		1.3	1.05	1.3	0.74	1	35.4	35.4	5340	4872	5340	0.82	1.11	0.77	1.20	1.03	0.61	1.71	Nonliquefiable
49.5	47	50	48.5	24	120		1.3	1.05	1.291	0.69	1	29.1	29.1	5820	5102	5820	0.80	1.08	0.83	0.43	0.39	0.61	0.63	**Liquefiable**

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Industrial Building
Project Location	San Bernardino, California
Project Number	23G142-1
Engineer	Ricardo Frias

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	37	18.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	37.00		0.000		0.00	Above Water Table
39.5	37	42	39.5	47.7	0.0	47.7	2.89	0.00	-1.41	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	35.4	0.0	35.4	1.71	0.02	-0.46	0.01	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	29.1	0.0	29.1	0.63	0.05	-0.03	0.05	3.00		0.011		0.39	**Liquefiable**
											Total Deformation (in)				0.39	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Building	MCE_G Design Acceleration	1.036 (g)
Project Location	San Bernardino, California	Design Magnitude	7.24
Project Number	23G142-1	Historic High Depth to Groundwater	37 (ft)
Engineer	Ricardo Frias	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-2		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.24)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	37	18.5		120		1.3	1.05	1.1	0.96	0.75	0.0	0.0	2220	2220	2220	0.94	1.01	1	0.06	0.06	N/A	N/A	Above Water Table
39.5	37	42	39.5	67	120		1.3	1.05	1.3	1.07	1	127.0	127.0	4740	4584	4740	0.85	1.11	0.77	2.00	1.70	0.59	2.89	Nonliquefiable
44.5	42	47	44.5	42	120		1.3	1.05	1.3	0.85	1	63.6	63.6	5340	4872	5340	0.82	1.11	0.75	2.00	1.66	0.61	2.75	Nonliquefiable
49.5	47	50	48.5	39	120		1.3	1.05	1.3	0.81	1	56.0	56.0	5820	5102	5820	0.80	1.11	0.74	2.00	1.63	0.61	2.66	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}
(2) Borehole Diameter Correction (Skempton, 1986)
(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)
(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)
(5) Rod Length Correction for Samples <10 m in depth
(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden
(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)
(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)
(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)
(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)
(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)
(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)
(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Industrial Building
Project Location	San Bernardino, California
Project Number	23G142-1
Engineer	Ricardo Frias

Boring No. B-2

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60-CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	37	18.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	37.00		0.000		0.00	Above Water Table
39.5	37	42	39.5	127.0	0.0	127.0	2.89	0.00	-8.70	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	63.6	0.0	63.6	2.75	0.00	-2.73	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	56.0	0.0	56.0	2.66	0.00	-2.08	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.00	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Building	MCE_G Design Acceleration	1.036 (g)
Project Location	San Bernardino, California	Design Magnitude	7.24
Project Number	23G142-1	Historic High Depth to Groundwater	37 (ft)
Engineer	Ricardo Frias	Depth to Groundwater at Time of Drilling	37 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-3		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	Ks	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=7.24)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	37	18.5		120		1.3	1.05	1.1	0.96	0.75	0.0	0.0	2220	2220	2220	0.94	1.01	1	0.06	0.06	N/A	N/A	Above Water Table
39.5	37	42	39.5	31	120		1.3	1.05	1.3	0.81	1	44.6	44.6	4740	4584	4584	0.85	1.11	0.77	2.00	1.70	0.59	2.89	Nonliquefiable
44.5	42	47	44.5	39	120		1.3	1.05	1.3	0.85	1	58.8	58.8	5340	4872	4872	0.82	1.11	0.75	2.00	1.66	0.61	2.75	Nonliquefiable
49.5	47	50	48.5	29	120		1.3	1.05	1.3	0.77	1	39.5	39.5	5820	5102	5102	0.80	1.11	0.74	2.00	1.63	0.61	2.66	Nonliquefiable

Notes:

(1) Energy Correction for N_{90} of automatic hammer to standard N_{60}

(2) Borehole Diameter Correction (Skempton, 1986)

(3) Correction for split-spoon sampler with room for liners, but liners are absent, (Seed et al., 1984, 2001)

(4) Overburden Correction, Caluclated by Eq. 39 (Boulanger and Idriss, 2008)

(5) Rod Length Correction for Samples <10 m in depth

(6) N-value corrected for energy, borehole diameter, sampler with absent liners, rod length, and overburden

(7) N-value corrected for fines content per Eqs. 75 and 76 (Boulanger and Idriss, 2008)

(8) Stress Reduction Coefficient calculated by Eq. 22 (Boulanger and Idriss, 2008)

(9) Magnitude Scaling Factor calculated by Eqns. A.8 & A.10 (Boulanger and Idriss, 2014)

(10) Overburden Correction Factor calcuated by Eq. 54 (Boulanger and Idriss, 2008)

(11) Calcuated by Eq. 70 (Boulanger and Idriss, 2008)

(12) Calcuated by Eq. 72 (Boulanger and Idriss, 2008)

(13) Calcuated by Eq. 25 (Boulanger and Idriss, 2008)

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Industrial Building
Project Location	San Bernardino, California
Project Number	23G142-1
Engineer	Ricardo Frias

Boring No. B-3

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines content	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	37	18.5	0.0	0.0	0.0	N/A	0.50	0.95	0.00	37.00		0.000		0.00	Above Water Table
39.5	37	42	39.5	44.6	0.0	44.6	2.89	0.00	-1.16	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	58.8	0.0	58.8	2.75	0.00	-2.32	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	39.5	0.0	39.5	2.66	0.01	-0.76	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.00	

Notes:

(1) $(N_1)_{60}$ calculated previously for the individual layer

(2) Correction for fines content per Equation 76 (Boulanger and Idriss, 2008)

(3) Corrected $(N_1)_{60}$ for fines content

(4) Factor of Safety against Liquefaction, calculated previously for the individual layer

(5) Calcuated by Eq. 86 (Boulanger and Idriss, 2008)

(6) Calcuated by Eq. 89 (Boulanger and Idriss, 2008)

(7) Calcuated by Eqs. 90, 91, and 92 (Boulanger and Idriss, 2008)

(8) Volumetric Strain Induced in a Liquefiable Layer, Calcuated by Eq. 96 (Boulanger and Idriss, 2008)
(Strain N/A if Factor of Safety against Liquefaction > 1.3)