GEOTECHNICAL INVESTIGATION
PROPOSED WAREHOUSE

SWC Tippecanoe Avenue and 9th Street
San Bernardino, California
for
Oakmont Industrial Group

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

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A California Corporation

August 17, 2021

Oakmont Industrial Group
3520 Piedmont Road, Suite 100
Atlanta, Georgia 30305

Attention: Mr. John C. Atwell
Senior Vice President

Project No.: **21G190-1**

Subject: **Geotechnical Investigation**
Proposed Warehouse
SWC Tippecanoe Avenue and 9t Street
San Bernardino, California

Mr. Atwell

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

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

Respectfully Submitted,

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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 site is located in a designated liquefaction hazard zone.
- Our site-specific liquefaction evaluation indicates that the on-site soils are not subject to liquefaction during the design seismic event. No design considerations related to liquefaction are considered warranted for this project.
- Artificial fill soils were encountered at of the boring locations, extending from the ground surface to depths of 2½ to 5½± feet. The fill soils possess varying strengths and densities, and are considered to represent undocumented fill. These soils, in their present condition, are not considered suitable for support of the foundation loads of the new structure.
- The fill soils are underlain by native alluvium which possesses varying strengths and densities. The results of laboratory testing indicate that the near-surface soils within the upper 3 to 4± feet possess unfavorable consolidation/collapse characteristics.
- Remedial grading will be necessary to remove the undocumented fill soils in their entirety and the upper portion of the near-surface native alluvial soils and replace these materials as compacted structural fill soils.

Site Preparation Recommendations

- Demolition of the remnants of the previous structures will be required in order to facilitate construction of the new building. Demolition should also include all floor-slabs, foundations, utilities and any other subsurface improvements that will not remain in place for use with the new development. Debris resultant from demolition should be disposed of offsite. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size and incorporated into new structural fills.
- Initial site preparation should include removal of all vegetation, including any organic topsoil. The surficial vegetation, and any organic soils should be properly disposed of off-site.
- Remedial grading is recommended within the proposed building pad area to remove the undocumented fill soils, which extend to depths of 2½ to 5½± feet at the boring locations, in their entirety. Additionally, the building pad area should also be overexcavated to a depth of at least 5 feet below existing grade and to a depth of at least 4 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.
- Following completion of the overexcavation, the exposed soils should be scarified to a depth of at least 12 inches, moisture conditioned to 0 to 4 percent above optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.
- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Building Foundation Recommendations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 3,000 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least two (2) No. 5 rebars (1 top and 1 bottom) in strip footings. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab Design Recommendations

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

Pavement Design Recommendations

ASPHALT PAVEMENTS (R=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. 21P213, dated April 8, 2021. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of this 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 southwest corner of Tippecanoe Avenue and 9th Street in San Bernardino, California. The site is bounded to the north by 9th Street, to the west by an existing commercial/industrial building, to the south by an existing commercial/industrial building, and to the east by Tippecanoe Avenue. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 in Appendix A of this report.

The subject site consists of an irregular-shaped parcel, 14.3± acres in size. Based on visiting the site and aerial photographs obtained from Google Earth, the site is currently vacant and undeveloped. A concrete pad remaining from the demolition of a single-family residence remains at the ground surface in the southeastern corner of the site. Along the southeastern boundary of the site, a group of medium to large-sized trees is also present. The ground surface cover in this area consists of exposed soil with sparse native grass and weed growth.

Detailed topographic information was not available at the time of this report. However, based on topographic information obtained from Google Earth, the site slopes gently downward to the west with an average gradient of about 1± percent. There is approximately 8± feet of elevation differential across the site.

3.2 Proposed Development

SCG was provided a site plan for the proposed development (Scheme 2). Based on the site plan prepared by HPA Architecture, the site will be developed with one warehouse building, approximately 287,270± ft² in size, located in the central-western area of the site. The building will be constructed with 39 dock-high doors along the eastern building wall. The building will be surrounded by asphaltic concrete pavements in the parking and drive lane areas, Portland cement concrete pavements in the loading dock areas. We expect the new development will also include areas of concrete flatwork and landscape planters.

Detailed structural information has not been provided. We assume that the new building will be a single-story structure of tilt-up concrete construction, typically supported on conventional shallow foundation systems with a concrete slab-on-grade floor. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 100 kips and 3 to 5 kips per linear foot, respectively.

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

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of five (5) borings advanced to depths of 20 to 50± feet below existing site grades. All of the borings were logged during drilling by a member of our staff. In addition to the borings, four (4) Cone Penetration Test (CPT) soundings (identified as CPT-1 through CPT-4) were advanced to depths of 50± feet as part of the liquefaction evaluation.

Hollow Stem Auger Borings

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

Cone Penetration Test (CPT) Soundings

The CPT soundings were performed by Kehoe Testing and Engineering (KTE) under the supervision of a member of our staff. The cone system used for this project was manufactured by Vertek. The CPT soundings were performed in general accordance with ASTM standards (D-5778). The cone penetrometers were pushed using 30-ton CPT rig. The cones used during the program recorded the cone resistance, sleeve friction, and dynamic core pressure at 2.5-centimeter depth intervals. Each CPT sounding was intended to extend to a depth of 50± feet as a part of the liquefaction evaluation. However, refusal conditions were encountered at depths of 21 to 31½± feet at all of the CPT locations. A more complete description of the CPT program as well as the results of the data interpretation are provided in the report prepared by KTE, enclosed in Appendix F of this report. The CPT soundings do not result in any recovered soil samples. However, correlations have been developed that utilize the cone resistance and the sleeve friction to estimate the soil type that is present at each 2.5-centimeter interval in the subsurface profile. These soil classifications are presented graphically on the CPT output forms enclosed in Appendix F.

The data generated by the cone penetrometer equipment has been reduced using CPeT-IT, V2.0, published by Geologismiki Geotechnical Software. The CPeT-IT program output as well as more details regarding the interpretation procedure are presented a report prepared by KTE, which is provided in Appendix F of this report.

General

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

4.2 Geotechnical Conditions

Artificial Fill

Artificial fill soils were encountered at the ground surface at all of the boring locations, extending to depths of 2½ to 5½± feet below existing site grades. The fill soils consist of medium dense to dense silty fine sands with variable medium to coarse sands and gravel content. The fill soils possess a disturbed appearance, with some of the borings containing AC and PCC fragments, resulting in the classification of artificial fill.

Additional soils classified as possible fill were encountered beneath the artificial fill soils at Boring No. B-4, extending to a depth of 4½± feet below existing site grades. These materials consist of medium dense fine to medium sands with variable silt, coarse sand and gravel content. These soils are similar in composition to the underlying alluvium, but possess a slightly disturbed appearance, resulting in their classification of possible fill.

Alluvium

Native alluvial soils were encountered beneath the artificial fill and possible fill soils at all boring locations, extending to at least the maximum depth explored of 50± feet below existing site grades. The near-surface alluvium, extending to depth of 12± feet, generally consists of loose to medium dense silty fine to coarse sands and gravelly fine to coarse sands. Occasional cobbles were also encountered at Boring Nos. B-3 and B-4. At greater depths, the alluvium generally consists of dense to very dense silty fine to coarse sands and gravelly fine to coarse sands.

Groundwater

Free water was not encountered during the drilling of any of the borings. Based on the moisture content of the recovered soil samples, groundwater levels exist at depths greater than the maximum explored depth of 50± feet below existing site grades. Research of historic high groundwater levels was performed as a part of the site-specific liquefaction evaluation. USGS Bulletin 1898 (Matti and Carson, 1991) indicates that the minimum historic depth to groundwater at the site is 8± feet below existing site grades. As part of our research, we reviewed readily available groundwater data in order to determine regional groundwater depths. The primary reference used to determine the groundwater depths in the subject site area is the California State Water Resources Control Board website, GeoTracker, https://geotracker.waterboards.ca.gov. The nearest monitoring well is located approximately 1± mile northeast of the site. Water level readings within this monitoring well indicates a high groundwater level of 52± feet below the ground surface in May 2010.

5.0 LABORATORY TESTING

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

Classification

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

Dry 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

A representative bulk sample has been tested to determine its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Sheet C-9 in Appendix C of this report. These tests are generally used for comparison with the 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

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

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-1 @ 0 to 5 feet	0.003	Not Applicable (S0)

Corrosivity Testing

One representative sample of the near-surface soils was submitted to a subcontracted corrosion engineering laboratory to identify potentially corrosive characteristics with respect to common construction materials. The corrosivity testing included a determination of the electrical resistivity, pH, 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)
B-1 @ 0 to 5 feet	11,600	7.9	10	22

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

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

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

Seismic Design Parameters

The 2019 California Building Code (CBC) provides procedures for earthquake resistant structural design that include considerations for on-site soil conditions, occupancy, and the configuration of the structure including the structural system and height. The seismic design parameters presented below are based on the soil profile and the proximity of known faults with respect to the subject site.

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

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

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	2.232
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.820
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	2.232
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.394
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.488
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.929

It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the 2019 CBC. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2019 CBC using the value of S_1

obtained from the Seismic Design Maps Tool, assuming that a site-specific ground motion hazards analysis is not required for the proposed buildings 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 2019 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 1.011. 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 7.29, based on the peak ground acceleration and soil classification D.

Liquefaction

Research of the San Bernardino County Land Use Plan, Geologic Hazard Overlays, San Bernardino S 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). 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. 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% 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.

As part of the liquefaction evaluation, Boring Nos. B-1 and B-4 were extended to depths of 50± feet. Based on the research discussed in Section 4.2 of this report, a conservative historic high groundwater depth of 8 feet was used for the liquefaction evaluation.

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 and B-4. The liquefaction potential of the site was analyzed utilizing a PGA_M of 1.011g for a magnitude 7.29 seismic event.

If liquefiable soils are identified, the potential settlements that could occur as a result of liquefaction are determined 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 determine the expected volumetric strain of saturated sands subjected to earthquake shaking. This analysis is also documented on the spreadsheets included in Appendix F.

In addition to the two borings, four (4) cone penetration tests (CPTs) were advanced to depths of 21 to 31½± feet below the existing site grades as part of the liquefaction evaluation. It should be noted that the CPTs were terminated shallower than their planned depth of 50 feet due to refusal on very dense granular soils.

Conclusions and Recommendations

The liquefaction analysis has identified no potentially liquefiable soils at Boring Nos. B-1 and B-4, nor at any of the CPT locations. The soils encountered below the historic high groundwater table either possess adequate factors of safety, or are considered non-liquefiable due to their cohesive characteristics and the results of the Atterberg limits testing with respect to the requirements of Special Publication 117A. Based on the results of the liquefaction analysis no design considerations related to liquefaction are considered warranted for this project.

6.2 Geotechnical Design Considerations

General

Artificial fill soils were encountered at the ground surface at all of the boring locations, extending to depths of 2½ to 5½± feet. These soils possess a mottled and disturbed appearance, with some samples possessing debris such as AC and PCC fragments. 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 a moderate potential for collapse when inundated with water. Therefore, remedial grading is considered warranted within the proposed building area in order to remove all of the undocumented fill soils in their entirety and the upper portion of the near-surface native alluvial soils, and replace these materials as compacted structural fill soils.

Settlement

The recommended remedial grading will remove the existing undocumented fill soils and a portion of the upper 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 large stress increases from the foundations of the new structure. Therefore, following completion of the recommended grading, post-construction settlements are expected to be within tolerable limits.

Expansion

The near-surface soils consist of sands, silty sands and gravelly sands with no appreciable clay content. These materials have been visually classified as non-expansive. Therefore, no design considerations related to expansive soils are considered warranted for this site.

Soluble Sulfates

The results of the soluble sulfate testing indicated that the selected sample of the near-surface soils possesses a sulfate concentration of approximately 0.003 percent. 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 the on-site soils possess a saturated resistivity value of 11,600 ohm-cm, and a pH value of 7.9. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity and pH are two of the five factors that enter into the evaluation procedure. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing was not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. Based on these factors, and utilizing the DIPRA procedure, the on-site soils are not considered to be corrosive to ductile iron pipe. Therefore, polyethylene encasement or some other appropriate method of protection may be required for iron pipes.

A relatively low concentration (10 mg/kg) of chlorides were detected in the samples submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested 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 sample possesses a nitrate concentration of 22 mg/kg. Based on this test result, the on-site soils are not considered to be corrosive to copper pipe.

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

Shrinkage/Subsidence

Removal and recompaction of the artificial fill and near-surface native soils is estimated to result in an average shrinkage of 6 to 14 percent. Shrinkage estimates for the individual samples range between 0 and 17 percent based on the results of density testing and the assumption that the on-site soils will be compacted to about 92 percent of the ASTM D-1557 maximum dry density. It should be noted that the shrinkage estimate is based on the results of dry density testing performed on small-diameter samples of the existing soils taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

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

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

No grading or foundation plans were available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary 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

Demolition of remnants of the previous structures, and any associated improvements will be necessary to facilitate the construction of the proposed development. Demolition of the existing structures should include all foundations, floor slabs, and any associated utilities. Any septic systems encountered during demolition and/or rough grading (if present) should be removed in their entirety. Any associated leach fields or other existing underground improvements should also be removed in their entirety. Debris resultant from demolition should be disposed of off-site in accordance with local regulations. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, and incorporated into new structural fills or it may be crushed and made into crushed miscellaneous base (CMB), if desired.

Initial site preparation should include stripping of any surficial vegetation from the site. This should include any weeds, grasses, and shrubs. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

Remedial grading will be necessary within the proposed building pad area to remove the existing undocumented fill soils and a portion of the variable strength native alluvium. The fill soils extend to depths of 2½ to 5½± feet at the boring locations.

In addition, the overexcavation is also recommended to extend to a depth of at least 5 feet below existing grade and 4 feet below proposed building pad subgrade elevation, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

The overexcavation areas should extend at least 5 feet beyond the building perimeters, 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 area of overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the building area should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. **Some localized areas of deeper excavation may be required if 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, moisture treated to 0 to 4 percent above the optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls and site walls should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pad. Any undocumented fill soils or disturbed native alluvium within any of these foundation areas should be removed in their entirety. Any erection pads used to construct the walls are considered to be part of the foundation system with respect to these remedial grading recommendations. 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 area. The previously excavated soils may then be replaced as compacted structural fill.

If the recommended remedial grading cannot be completed for screen walls located along property lines, such walls should be designed for a reduced allowable bearing pressure. The allowable bearing pressure will be determined based on the actual extent of remedial grading that can be accomplished.

Treatment of Existing Soils: Flatwork, Parking and Drive Areas

Based on economic considerations, removal and replacement of the variable strength existing fill and alluvial soils is not considered warranted within the proposed parking and drive areas. Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping and demolition operations.

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

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

Fill Placement

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

Imported Structural Fill

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

Utility Trench Backfill

In general, all utility trench backfill should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). 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 the 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 are predominately granular in nature. 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. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

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

Groundwater

The static groundwater table at this site is considered to exist at a depth of more than 50± 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 existing undocumented fill soils and the upper portion of the near-surface alluvium. 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, 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: Two (2) No. 5 rebars (1 top and 1 bottom).
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

The allowable bearing pressure presented above may be increased by one-third 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 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 the new floor slab 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 soils. These fill soils are expected to extend to a depth of at least 4 feet below finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.

- Minimum slab reinforcement: Reinforcement is not expected to be required for geotechnical conditions. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

- Modulus of subgrade reaction, k =150 psi/in

- Slab underlayment: If moisture sensitive floor coverings will be used the minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as a 15 mil Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

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

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

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

6.7 Retaining Wall Design and Construction

Small retaining walls are expected to be necessary in the truck dock areas and may also be required to facilitate the new site grades. The parameters recommended for use in the design of these walls are presented below.

Retaining Wall Design Parameters

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

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-site Sands and Silty Sands
Internal Friction Angle (ϕ)		30°
Unit Weight		127 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	42 lbs/ft^3
	Active Condition (2h:1v backfill)	68 lbs/ft^3
	At-Rest Condition (level backfill)	64 lbs/ft^3

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

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

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

Retaining Wall Foundation Design

The retaining wall foundations should be underlain by at least 3 feet of newly placed structural fill. Foundations to support new retaining walls should be designed in accordance with the general Foundation Design Parameters presented in a previous section of this report.

Seismic Lateral Earth Pressures

In accordance with the 2019 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.

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.

6.8 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of silty sands, well-graded sands, and gravelly sands. Based on their classification, these materials are expected to possess good to excellent pavement support characteristics, with R-values in the range of 50 to 60. Since R-value testing was not included in the scope of services for this project, the subsequent pavement design is based upon an assumed 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. An alternate pavement section has been provided for use in parking stall areas due to the anticipated lower traffic intensity in these areas. However, truck traffic must be excluded from areas where the thinner pavement section is used; otherwise premature pavement distress may occur. 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.

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 Portland cement 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. Reinforcing within all pavements should be designed by the 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. The actual joint spacing and reinforcing of the Portland cement concrete pavements should be determined by the structural engineer.

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

California Division of Mines and Geology (CDMG), "Guidelines for Evaluating and Mitigating Seismic Hazards in California," State of California, Department of Conservation, Division of Mines and Geology, Special Publication 117A, 2008.

Idriss, I. M. and Boulanger, R.W., "Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute, 2008.

National Research Council (NRC), "Liquefaction of Soils During Earthquakes," Committee on Earthquake Engineering, National Research Council, Washington D. C., Report No. CETS-EE-001, 1985.

Seed, H. B., and Idriss, I. M., "Simplified Procedure for Evaluating Soil Liquefaction Potential using field Performance Data," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, September 1971, pp. 1249-1273.

Sadigh, K., Chang, C. –Y., Egan, J. A., Makdisi. F., Youngs, R. R., "*Attenuation Relationships for Shallow Crustal Earthquakes Based on California Strong Motion Data"*, Seismological Research Letters, Seismological Society of America, Volume 68, Number 1, January/ February 1997, pp. 180-189.

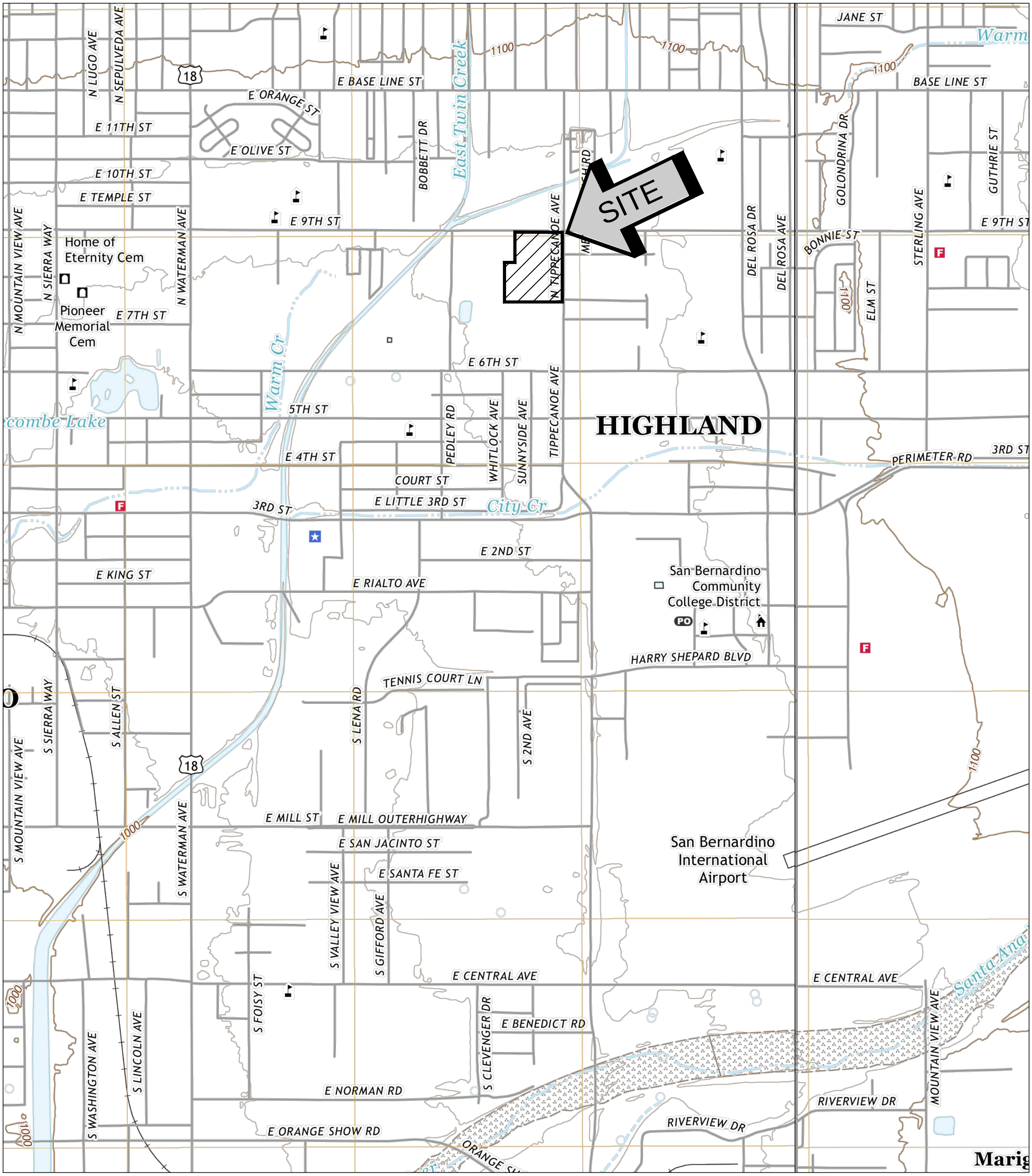
Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

Tokimatsu K., and Seed, H. B., "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of the Geotechnical Engineering Division, American society of Civil Engineers, Volume 113, No. 8, August 1987, pp. 861-878.

Tokimatsu, K. and Yoshimi, Y., "*Empirical Correlations of Soil Liquefaction Based on SPT N-value and Fines Content,"* Seismological Research Letters, Eastern Section Seismological Society Of America, Volume 63, Number 1, p. 73.

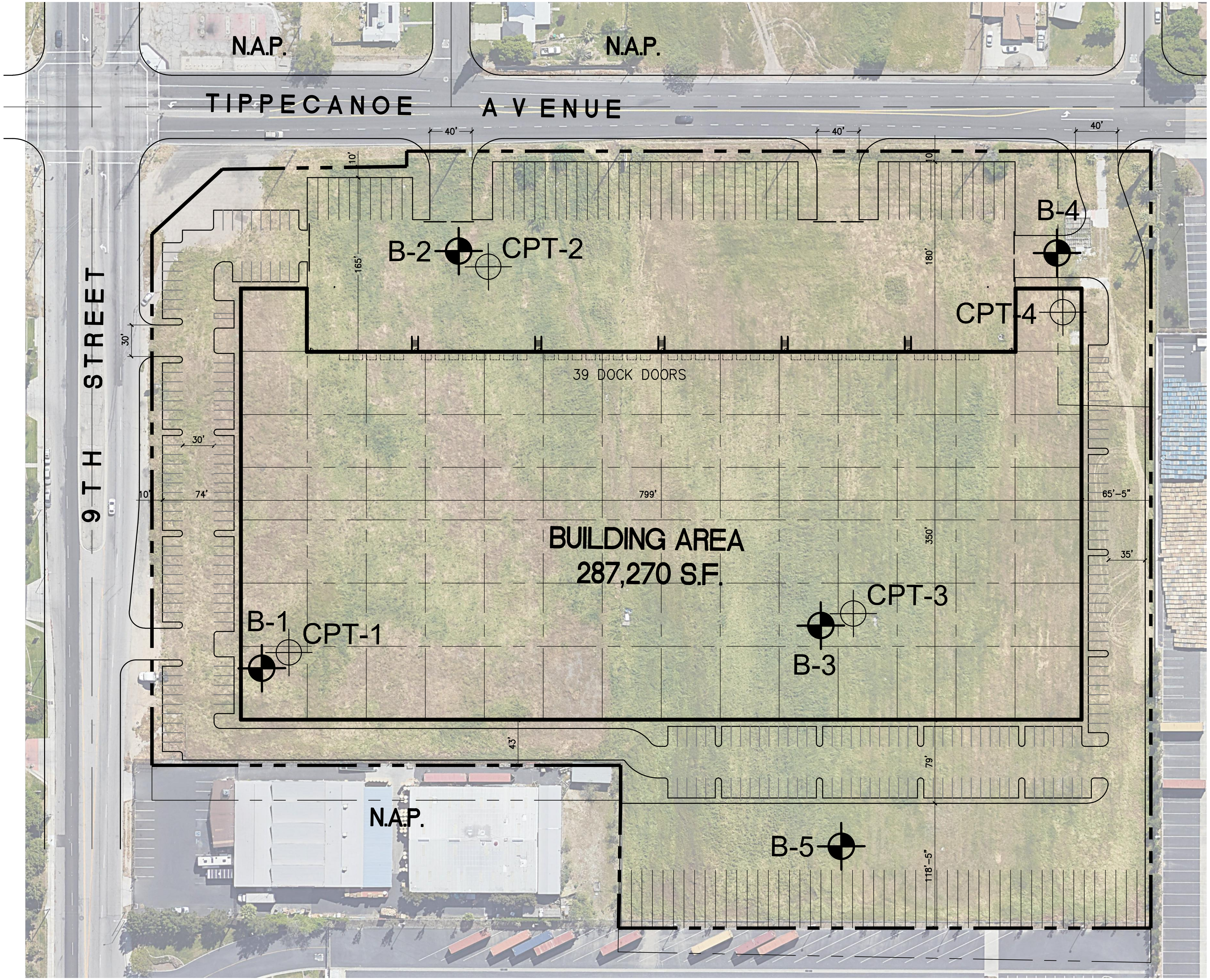
Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAPS OF THE SAN BERNARDINO SOUTH AND REDLANDS QUADRANGLES, SAN BERNANDINO COUNTY, CALIFORNIA, 2018.

SITE LOCATION MAP	
PROPOSED WAREHOUSE	
SAN BERNARDINO, CALIFORNIA	
SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JZ CHKD: RGT	
SCG PROJECT 21G190-1	
PLATE 1	



GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- APPROXIMATE CPT LOCATION

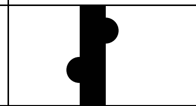
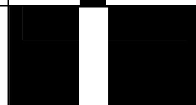
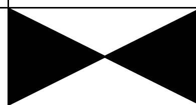
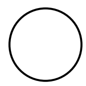
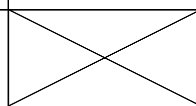
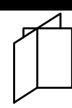
NOTE: SITE PLAN PROVIDED BY HPA.
AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH.

BORING AND CPT LOCATION PLAN
PROPOSED WAREHOUSE
SAN BERNARDINO, CALIFORNIA

SCALE: 1" = 90'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAZ CHKD: RGT	
SCG PROJECT 21G190-1	
PLATE 2	

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

BLOW COUNT: Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.

POCKET PEN.: Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.

GRAPHIC LOG: Graphic Soil Symbol as depicted on the following page.

DRY DENSITY: Dry density of an undisturbed or relatively undisturbed sample in lbs/ft^3.

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

LIQUID LIMIT: The moisture content above which a soil behaves as a liquid.

PLASTIC LIMIT: The moisture content above which a soil behaves as a plastic.

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

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 21G190-1
PROJECT: Proposed Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/15/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: ---
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
	X	16			FILL: Dark Brown Silty fine Sand, little medium to coarse Sand, trace AC fragments, trace fine root fibers, trace fine Gravel, medium dense-damp		6					
	X	21			ALLUVIUM: Brown Silty fine to coarse Sand, trace to little fine Gravel, trace fine root fibers, medium dense-damp		3					
5												
	X	24			Brown Silty fine Sand, trace fine to coarse Gravel, medium dense-damp		9					
	X	20			@ 8.5', little Iron Oxide staining		6			25		
10												
					Light Brown Silty fine to coarse Sand, trace fine to coarse Gravel, dense to very dense-dry to damp							
	X	63					2					
15												
	X	31			@ 18.5', 2" Silt nodules		4					
20												
	X	70					3					
25												
	X	26			@ 28.5', Brown, medium dense		4			8		
30												
					Gray Brown Gravelly fine to coarse Sand, little Silt, dense-dry to damp							
	X	37					2					

TBL 21G190-1.GPJ SOCALGEO.GDT 8/17/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 21G190-1
PROJECT: Proposed Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/15/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: ---
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
					Brown Gray Gravelly fine to coarse Sand, little Silt, dense-dry to damp							
					Brown to Dark Brown fine to coarse Sand, trace Silt, trace fine to coarse Gravel, very dense-damp							
	X	50/5"					3					
40												
	X	50/5"			@ 43.5', trace to little fine to coarse Gravel, 4" Silty fine Sand lense		5					
45												
	X	50/3"			@ 48.5', trace fine Gravel, very dense-damp		4					
50												
					Boring Terminated at 50'							

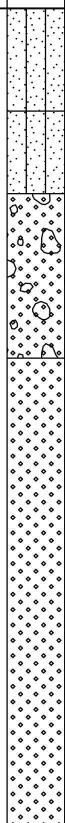
TBL 21G190-1.GPJ SOCALGEO.GDT 8/17/21

BORING NO. B-2

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G190-1	DRILLING DATE: 7/15/21	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 6 feet
LOCATION: San Bernardino, California	LOGGED BY: Ryan Bremer	READING TAKEN: At Completion

SURFACE ELEVATION: --- MSL



DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		17		FILL: Dark Brown Silty fine to medium Sand, trace PCC fragments, trace fine root fibers, little medium to coarse Sand, trace fine Gravel, medium dense-dry to damp	93	2					
		16		ALLUVIUM: Gray Silty fine Sand, trace Iron Oxide veining, trace medium Sand, medium dense-damp	104	3					
5		35		Light Brown Gravelly fine to coarse Sand, trace Silt, medium dense to very dense-dry		1					Disturbed Sample
		50/6"			110	1					
10		41		Gray fine to coarse Sand, trace Silt, medium dense to dense-dry to damp	107	1					
15		37		@ 13.5', Gray Brown		3					
20		40		@ 18.5', trace fine Gravel		4					

Boring Terminated at 20'

TBL 21G190-1.GPJ SOCALGEO.GDT 8/17/21

TEST BORING LOG **PLATE B-2**

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 21G190-1
PROJECT: Proposed Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/15/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

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
	X	34			FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace fine root fibers, little porosity, trace Iron Oxide staining, medium dense-damp	95	7					
	X	14			ALLUVIUM: Light Brown fine to medium Sand, trace coarse Sand, trace Silt, trace Iron Oxide staining, loose-dry to damp	98	2					
5	X	20			Light Brown Silty fine to medium Sand, some Iron Oxide staining, medium dense to dense-damp	98	5					
	X	48			@ 7', no recovery due to Cobbles							No Sample Recovery
10	X	42			@ 9', Gray, trace fine to coarse Gravel, trace Iron Oxide staining, little coarse Sand	106	3					
					Red Brown fine to coarse Sand, trace fine Gravel, trace Silt nodules, medium dense-damp to moist							
15	X	18					8					
20	X	23			@ 18.5', Brown, trace fine to coarse Gravel		2					
					Boring Terminated at 20'							

TBL 21G190-1.GPJ SOCALGEO.GDT 8/17/21

TEST BORING LOG

BORING NO. B-4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G190-1
PROJECT: Proposed Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/15/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: ---
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: Brown Silty fine to medium Sand, trace coarse Sand, little Iron Oxide staining, medium dense-damp							
		25				102	3					
		16			POSSIBLE FILL: Brown fine to medium Sand, trace Silt, trace coarse Sand, trace fine to coarse Gravel, trace Iron Oxide staining, medium dense-damp	102	3					
5		14			ALLUVIUM: Gray fine to coarse Sand, trace to little Silt, trace to little fine Gravel, loose-dry	97	1					
		14			@ 7', little fine Gravel	114	1					
		29			Red Brown Gravelly fine to coarse Sand, occasional Cobbles, trace Silt, little Iron Oxide staining, medium dense-dry	115	1					
10												
					Gray fine to coarse Sand, trace Silt, trace fine to coarse Gravel, dense to very dense-dry to damp							
		50					2					
15												
		48					2					
20												
		65			@ 23.5', Light Brown		3					
25												
		28			@ 28.5', trace fine Gravel, medium dense		2			5		
30												
		50/5"					3					

TBL 21G190-1.GPJ SOCALGEO.GDT 8/17/21

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G190-1	DRILLING DATE: 7/15/21	WATER DEPTH: Dry
PROJECT: Proposed Warehouse	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: San Bernardino, California	LOGGED BY: Ryan Bremer	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 fine to coarse Sand, trace Silt, trace fine to coarse Gravel, medium dense to very dense-dry to damp							
					Brown Silty fine to coarse Sand, trace fine Gravel, little Iron Oxide staining, dense to very dense-dry to damp							
	X	46					2					
40												
	X	50/4"			@ 43.5', trace to little fine to coarse Gravel		3					
45												
					Gray fine to coarse Sand, trace Silt, very dense-dry to damp							
	X	50/5"					2					
50					Boring Terminated at 50'							

TBL 21G190-1.GPJ SOCALGEO.GDT 8/17/21

BORING NO. B-5

JOB NO.: 21G190-1
PROJECT: Proposed Warehouse
LOCATION: San Bernardino, California

DRILLING DATE: 7/15/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Ryan Bremer

WATER DEPTH: Dry
CAVE DEPTH: 7 feet
READING TAKEN: At Completion

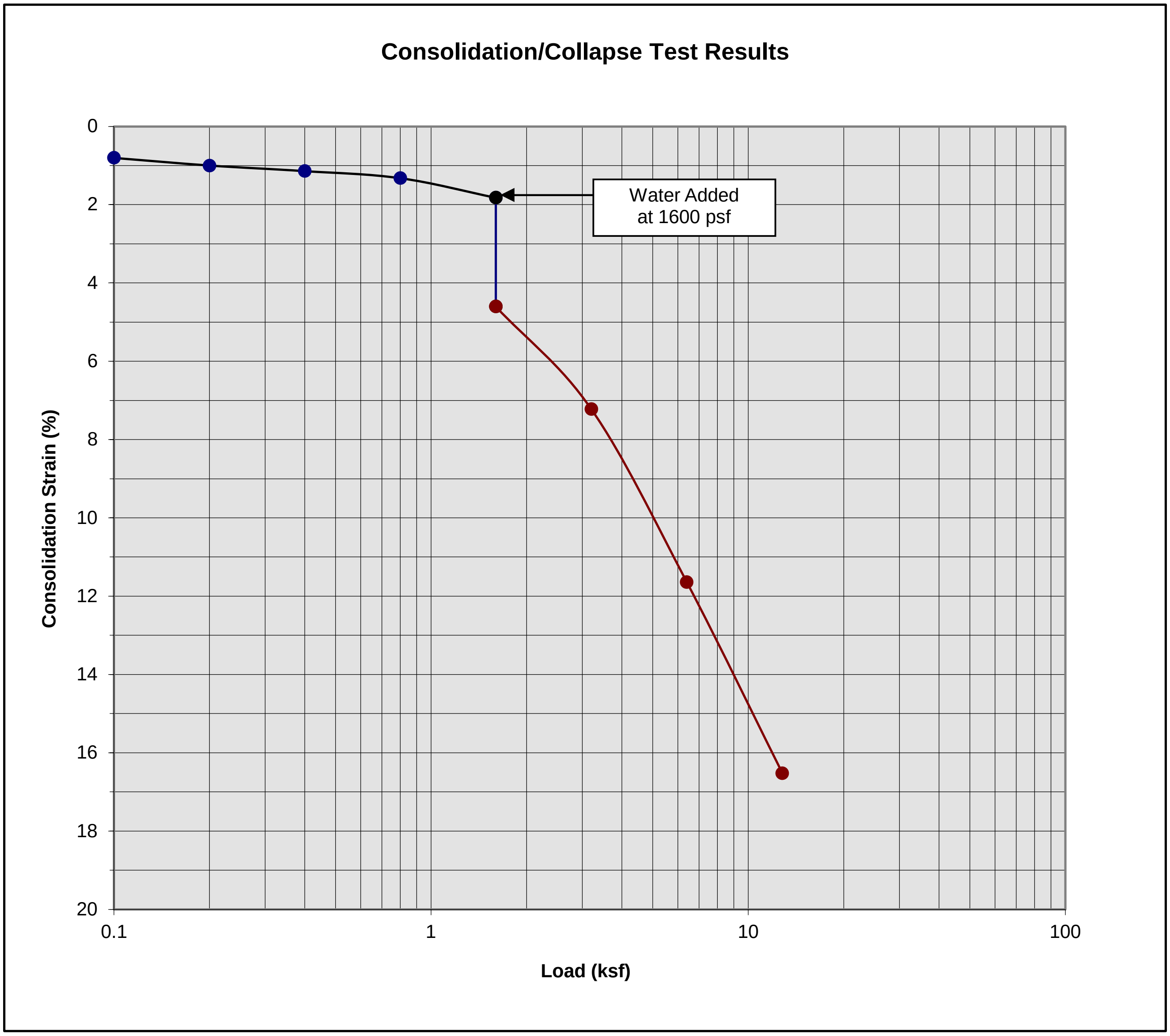
FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					FILL: Brown Silty fine Sand, trace medium to coarse Sand, trace Iron Oxide staining, medium dense to dense-damp							
	X	31					3					
	X	14			@ 3.5', trace fine root fibers		3					
5					ALLUVIUM: Brown fine to medium Sand, trace to little Silt, little Iron Oxide staining, loose to medium dense-damp							
	X	9					3					
	X	24					3					
10												
					Gray fine to coarse Sand, trace fine Gravel, trace Silt, medium dense-dry to damp							
	X	16					1					
15												
	X	23					2					
20												

Boring Terminated at 20'

TBL 21G190-1.GPJ SOCALGEO.GDT 8/17/21

TEST BORING LOG **PLATE B-5**

APPENDIX C



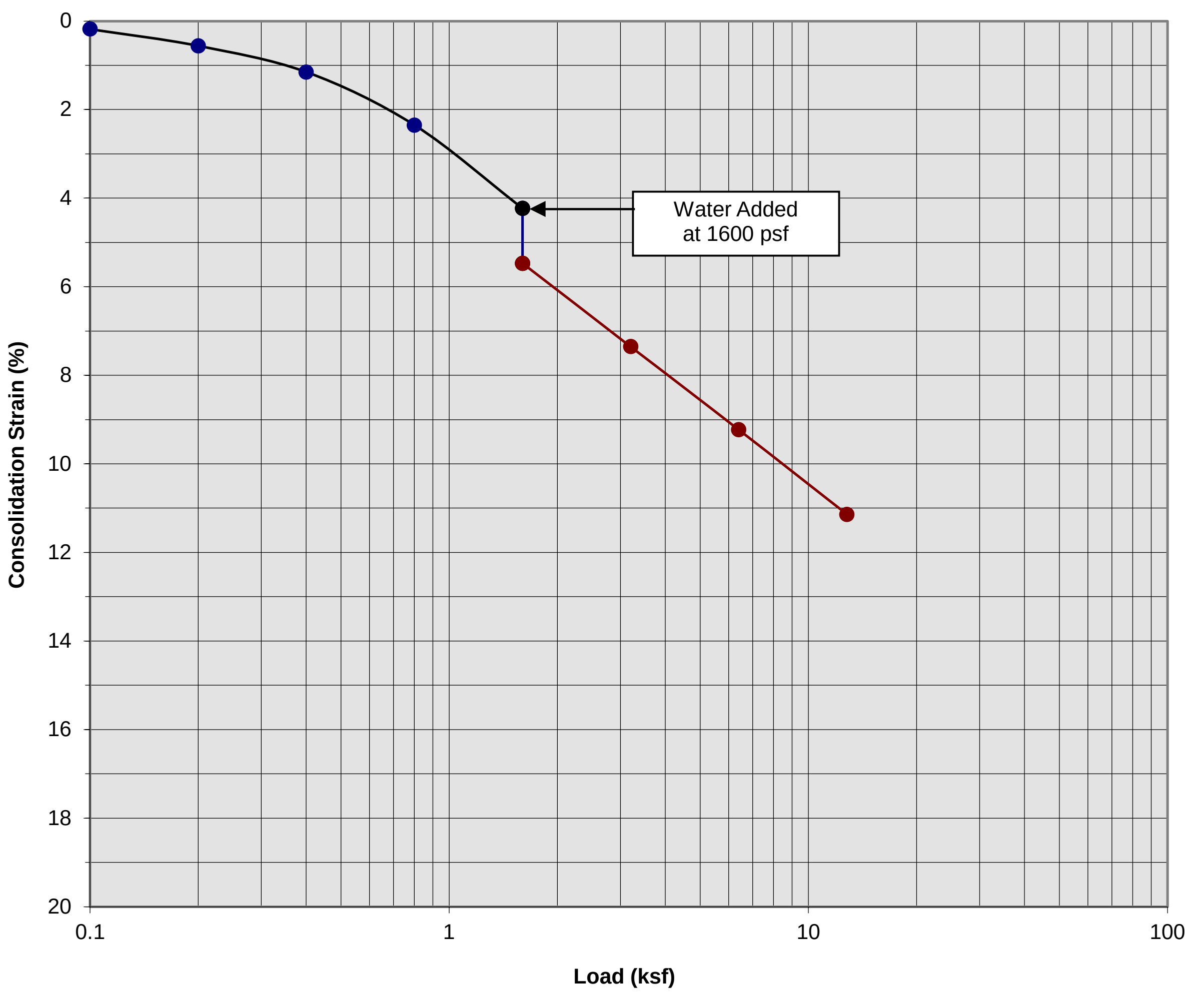
Classification: FILL: Silty fine Sand, trace medium to coarse Sand

Boring Number:	B-3	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	1 to 2	Initial Dry Density (pcf)	95.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	2.78

Proposed Warehouse
San Bernandino, CA
Project No. 21G190-1
PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: Fine to medium Sand, trace coarse Sand, trace Silt

Boring Number:	B-3	Initial Moisture Content (%)	2
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	3 to 4	Initial Dry Density (pcf)	98.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	110.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.24

Proposed Warehouse
San Bernandino, CA
Project No. 21G190-1
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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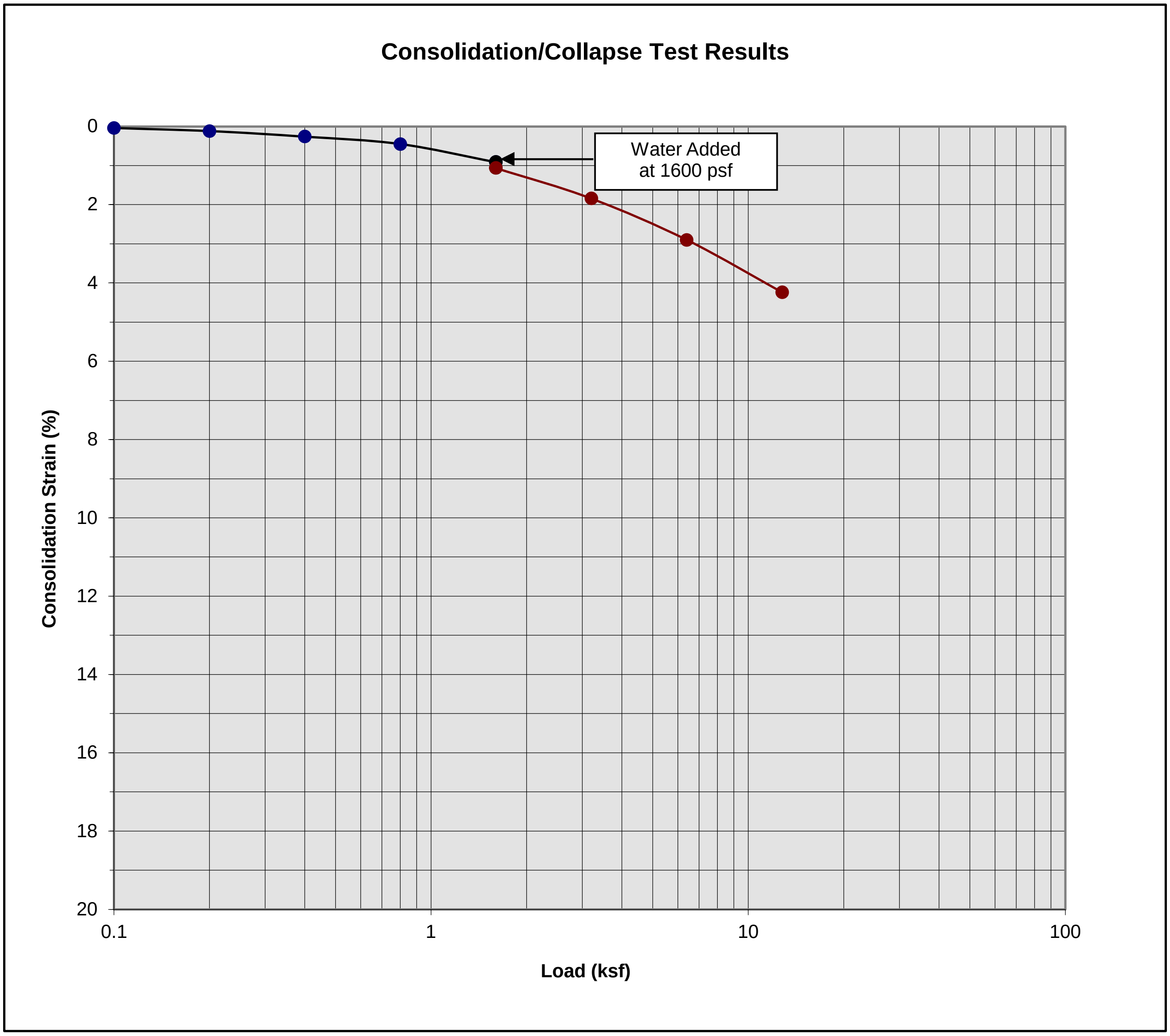

Classification: Silty fine to medium Sand

Boring Number:	B-3	Initial Moisture Content (%)	5
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	5 to 6	Initial Dry Density (pcf)	98.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	102.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.15

Proposed Warehouse
San Bernandino, CA
Project No. 21G190-1
PLATE C- 3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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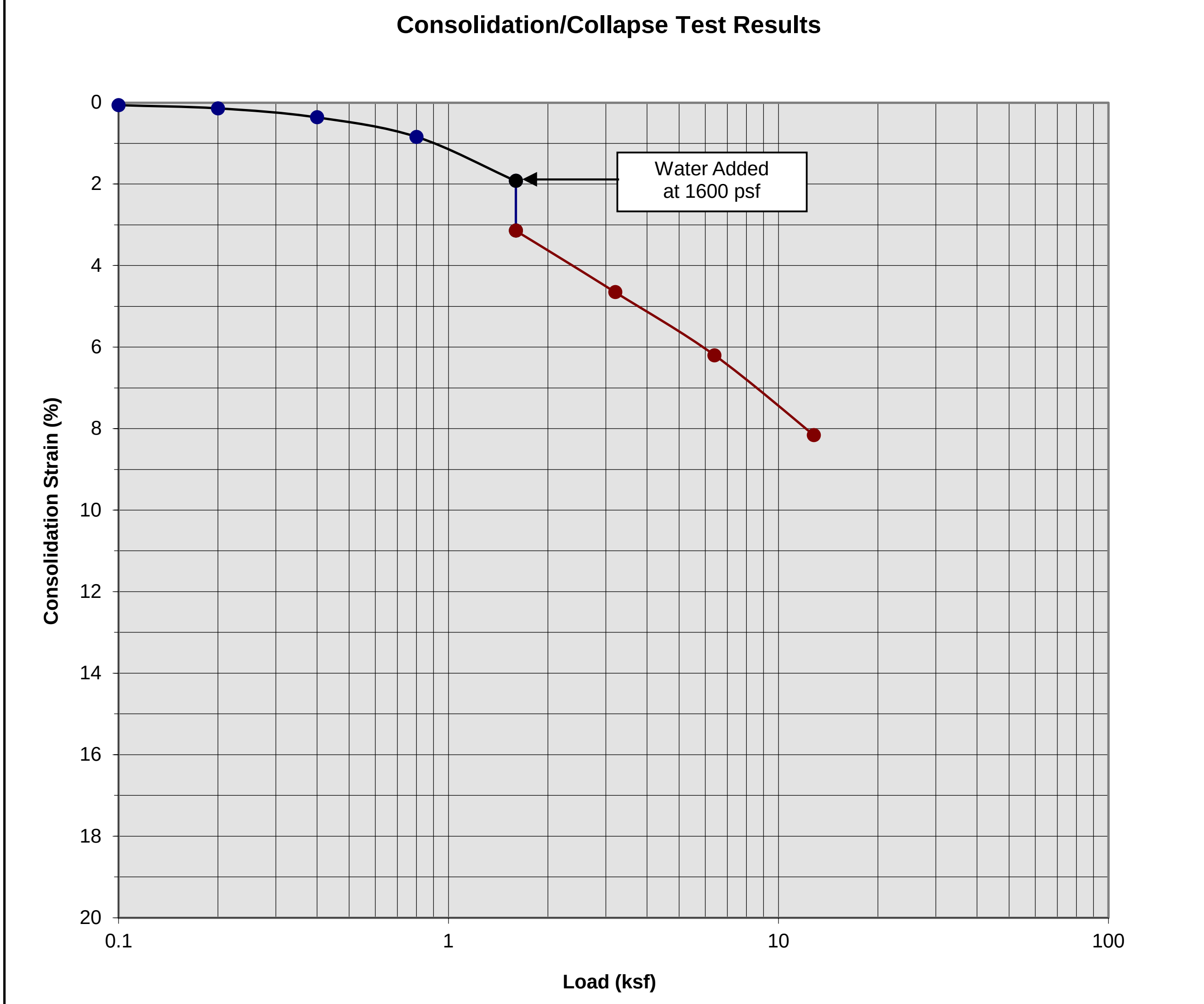
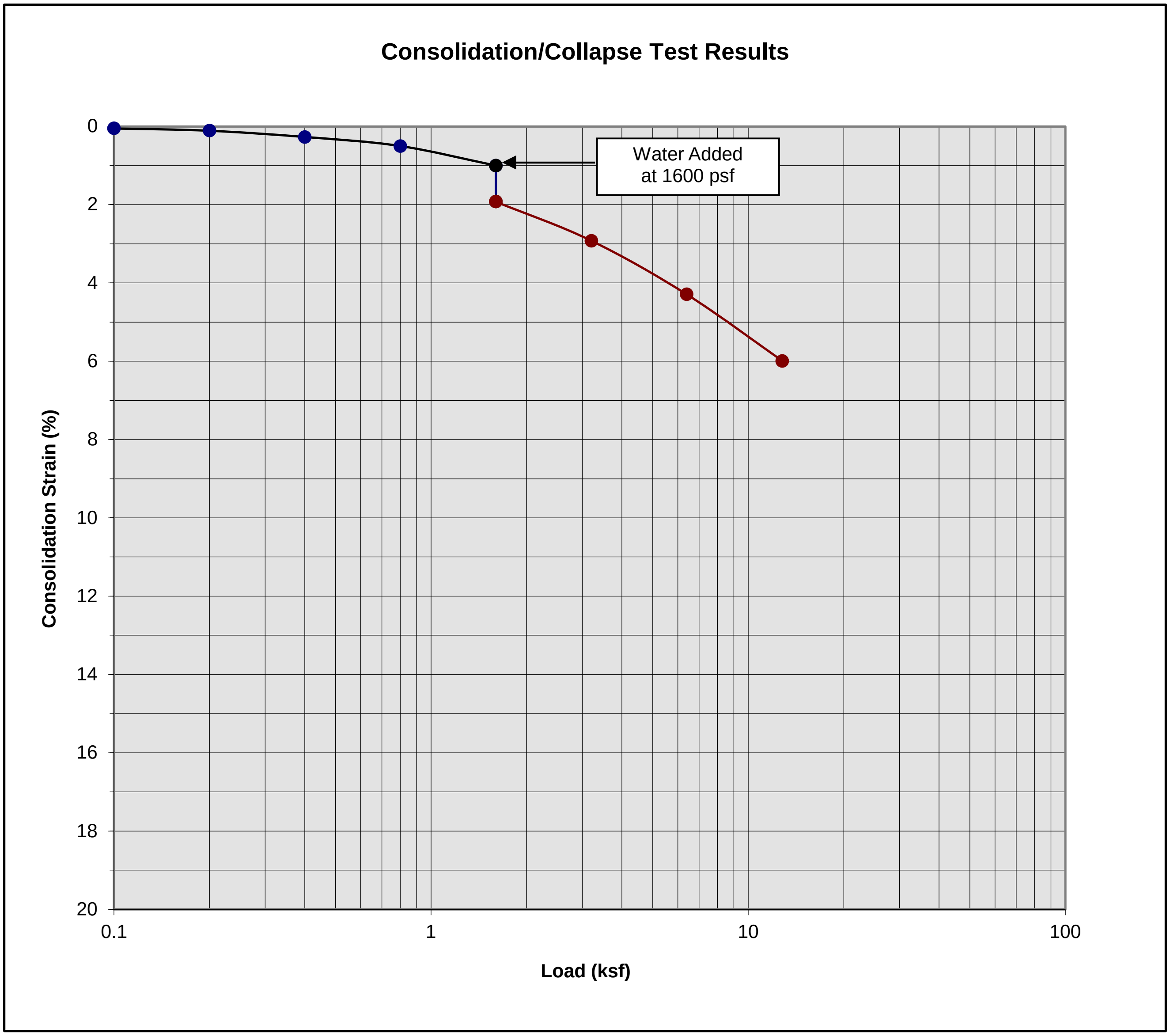


Classification: Silty fine to medium Sand, little coarse Sand, trace fine to coarse Gravel

Coarse Gravel

Boring Number:	B-3	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	9 to 10	Initial Dry Density (pcf)	106.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.22

Proposed Warehouse
San Bernandino, CA
Project No. 21G190-1
PLATE C- 4



Classification: POSSIBLE FILL: Fine to medium Sand, trace Silt, trace coarse Sand, fine to coarse Gravel

Boring Number:	B-4	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	3 to 4	Initial Dry Density (pcf)	102.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.92

Proposed Warehouse
San Bernandino, CA
Project No. 21G190-1
PLATE C- 5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
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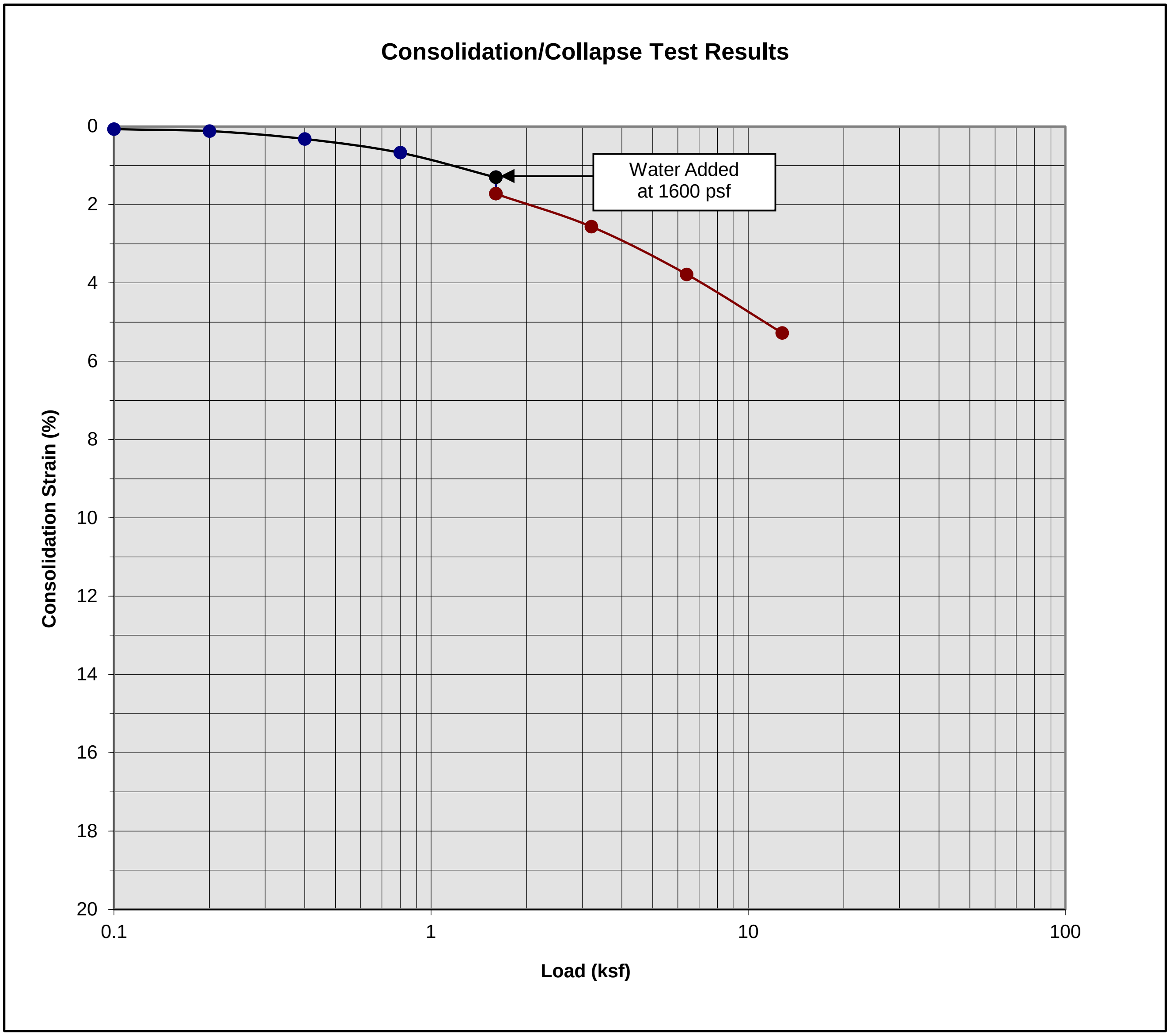

Classification: Fine to coarse Sand, trace to little Silt, trace to little fine Gravel

Boring Number:	B-4	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	23
Depth (ft)	5 to 6	Initial Dry Density (pcf)	97.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	102.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.42

Proposed Warehouse
San Bernandino, CA
Project No. 21G190-1
PLATE C- 6

SoCalGeo
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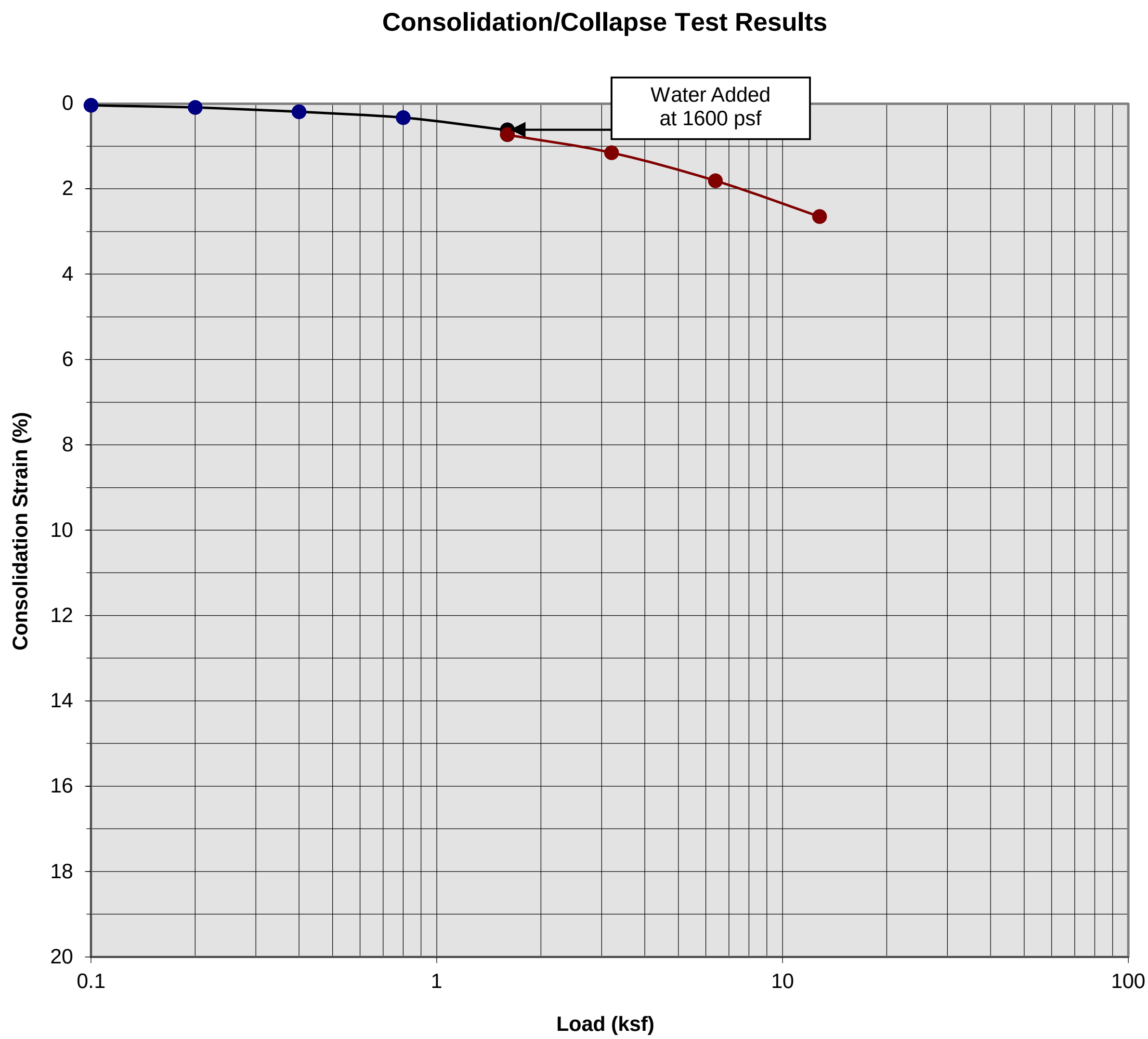
A California Corporation

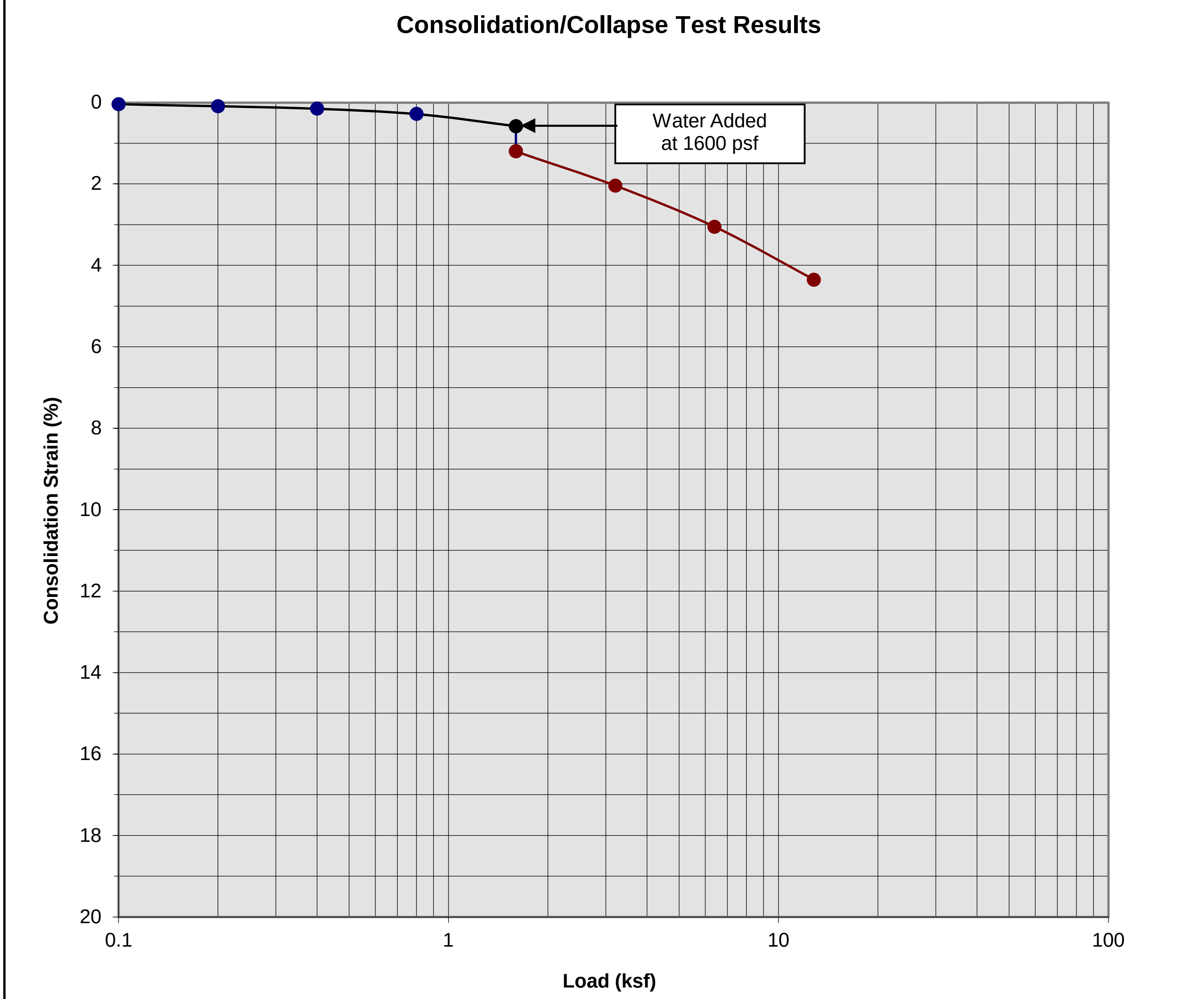


Classification: Fine to coarse Sand, trace to little Silt, trace to little fine Gravel

Boring Number:	B-4	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	114.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.11

Proposed Warehouse
San Bernandino, CA
Project No. 21G190-1
PLATE C- 7

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

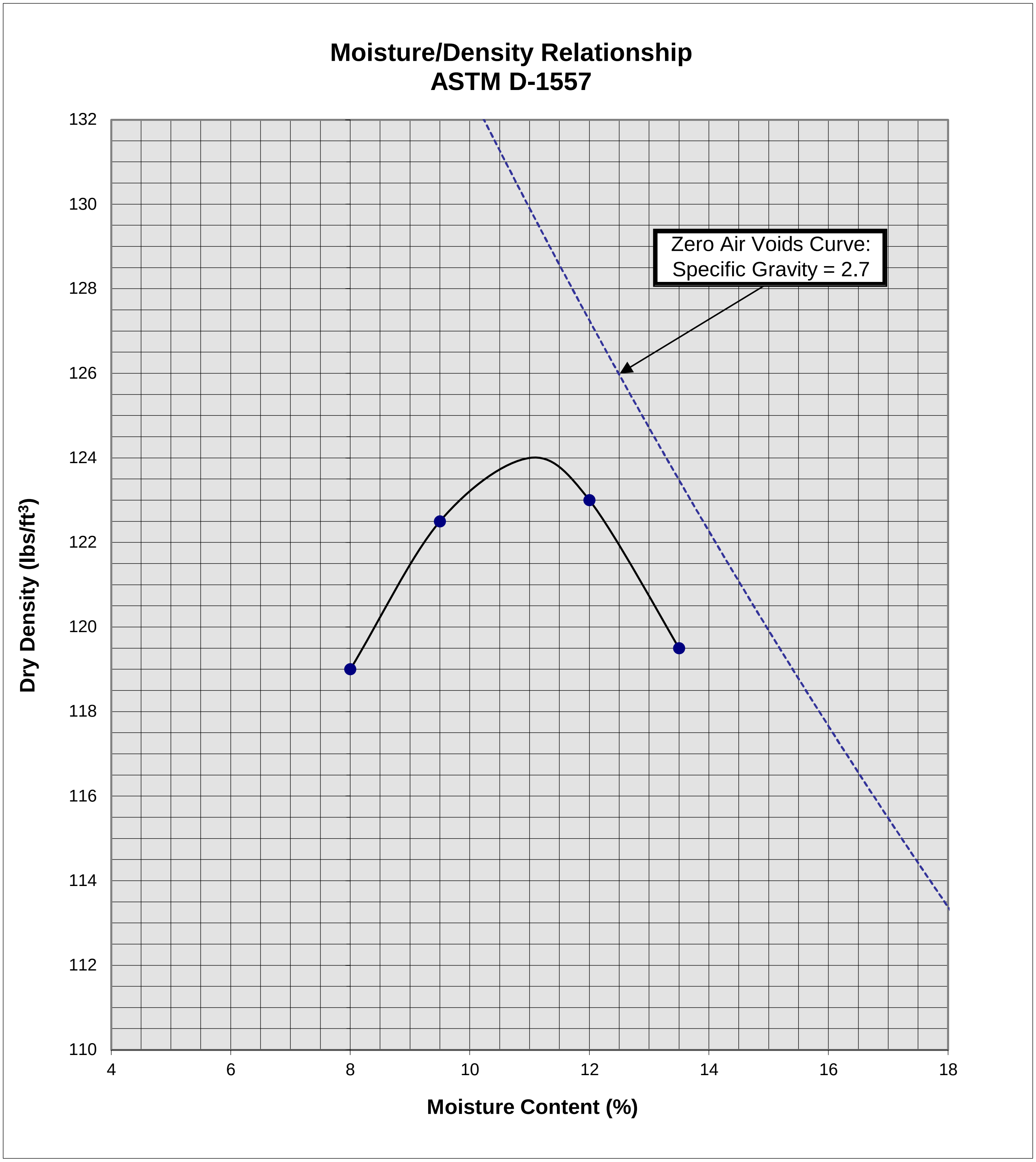


Classification: Gravelly fine to coarse Sand, occasional Cobbles, trace Silt

Boring Number:	B-4	Initial Moisture Content (%)	1
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	9 to 10	Initial Dry Density (pcf)	115.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.62

Proposed Warehouse
San Bernandino, CA
Project No. 21G190-1
PLATE C- 8

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-4 @ 0 to 5'
Optimum Moisture (%)	11
Maximum Dry Density (pcf)	124
Soil Classification	Brown Silty fine to medium Sand, trace coarse Sand

Proposed Warehouse
San Bernardino, California
Project No. 21G190-1
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

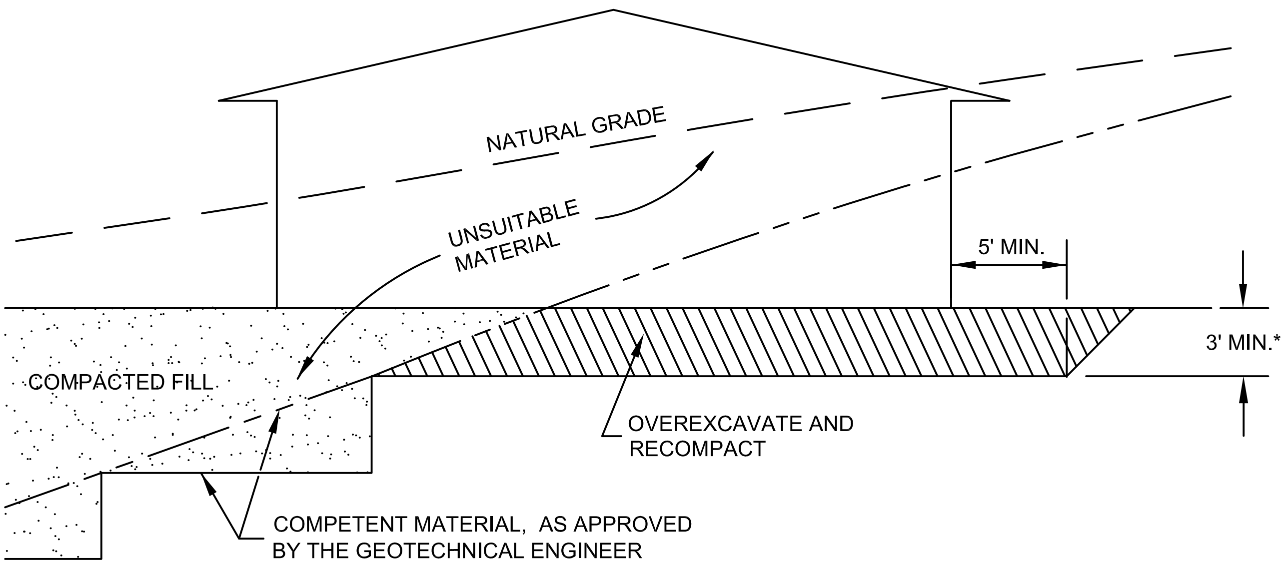
Subdrains

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

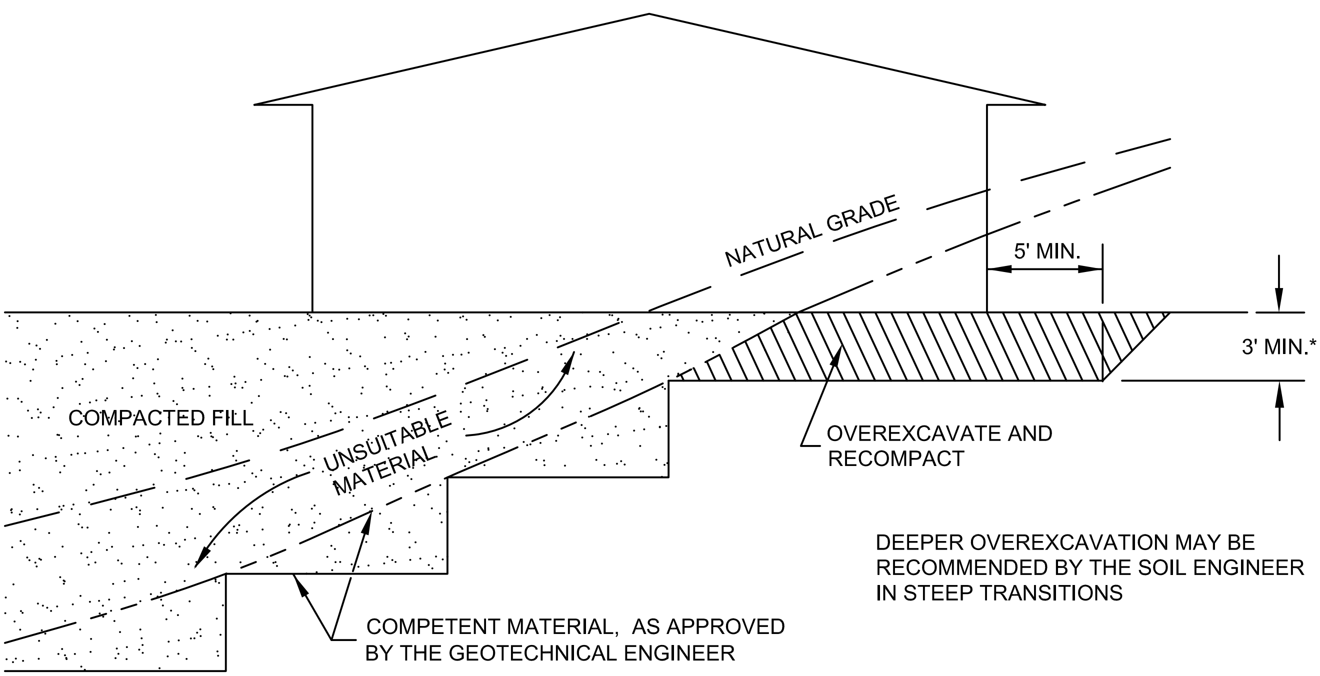
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



CUT/FILL LOT (TRANSITION)



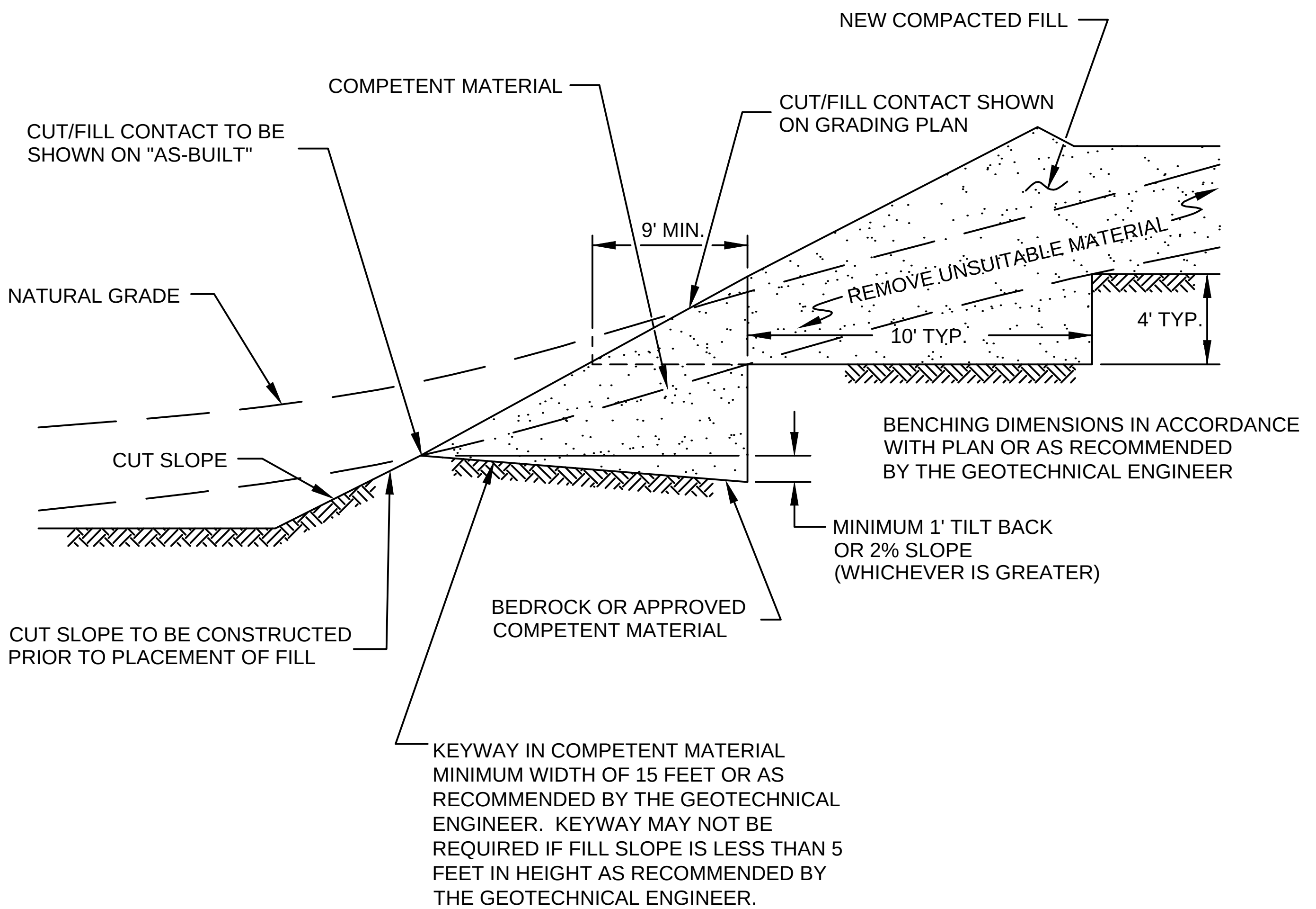
DEEPER OVEREXCAVATION MAY BE RECOMMENDED BY THE SOIL ENGINEER IN STEEP TRANSITIONS

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

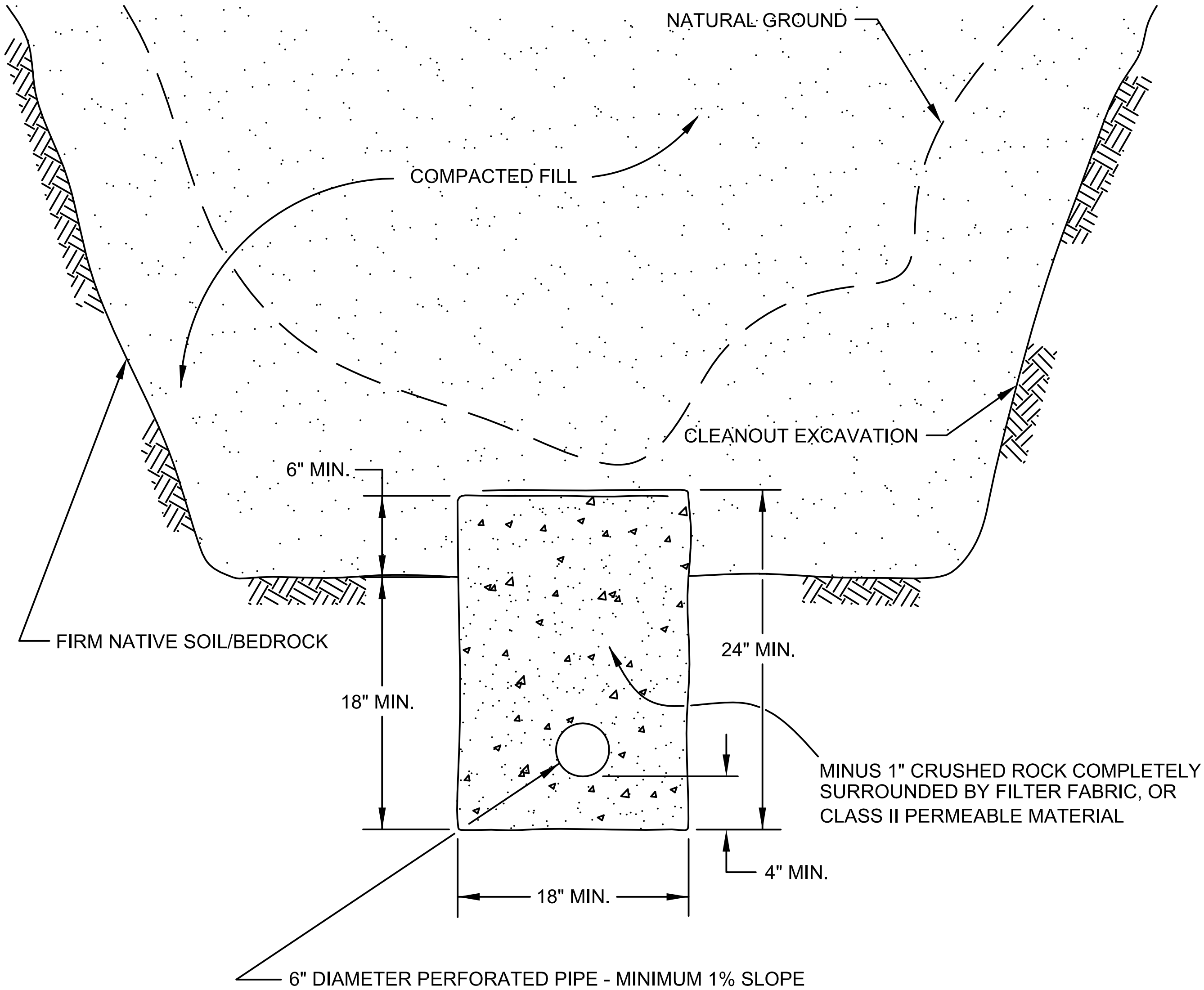
TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-1	



FILL ABOVE CUT SLOPE DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-2	



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

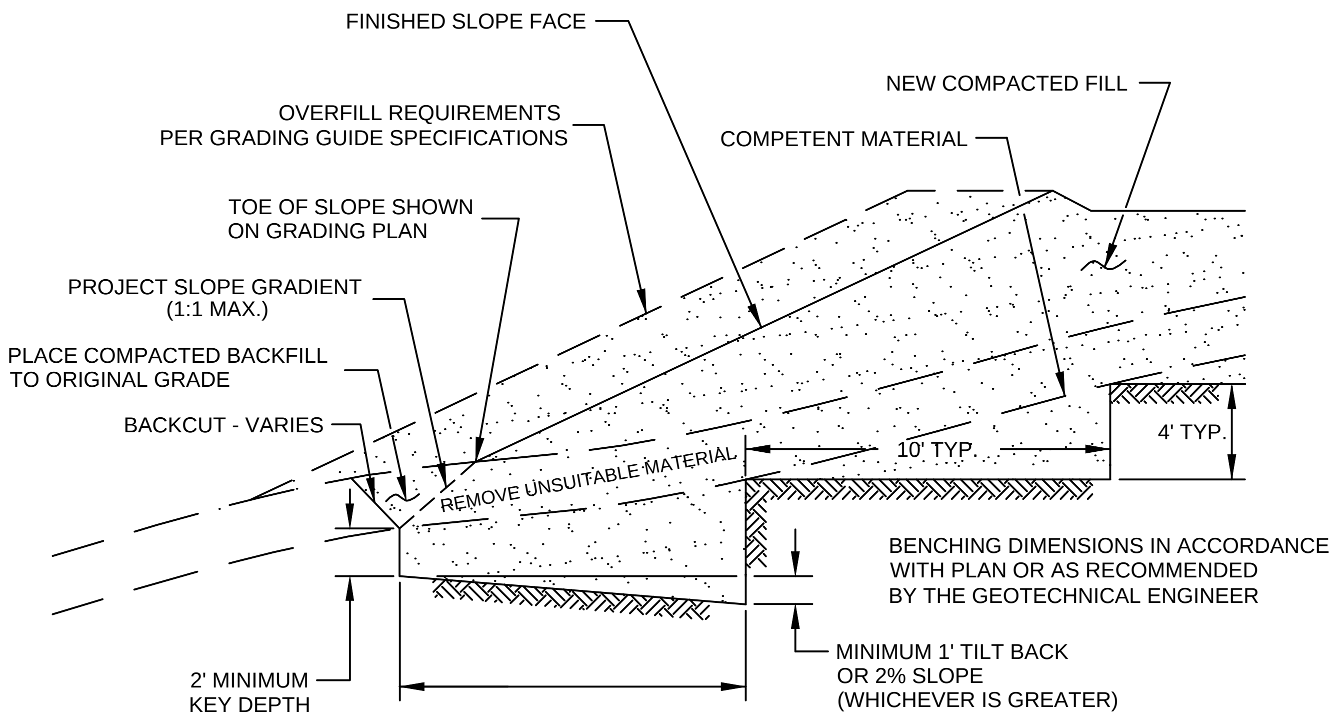
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



KEYWAY IN COMPETENT MATERIAL. MINIMUM WIDTH OF 15 FEET OR AS RECOMMENDED BY THE GEOTECHNIAL ENGINEER. KEYWAY MAY NOT BE REQUIRED IF FILL SLOPE IS LESS THAN 5' IN HEIGHT AS RECOMMENDED BY THE GEOTECHNICAL ENGINEER.

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

FILL ABOVE NATURAL SLOPE DETAIL

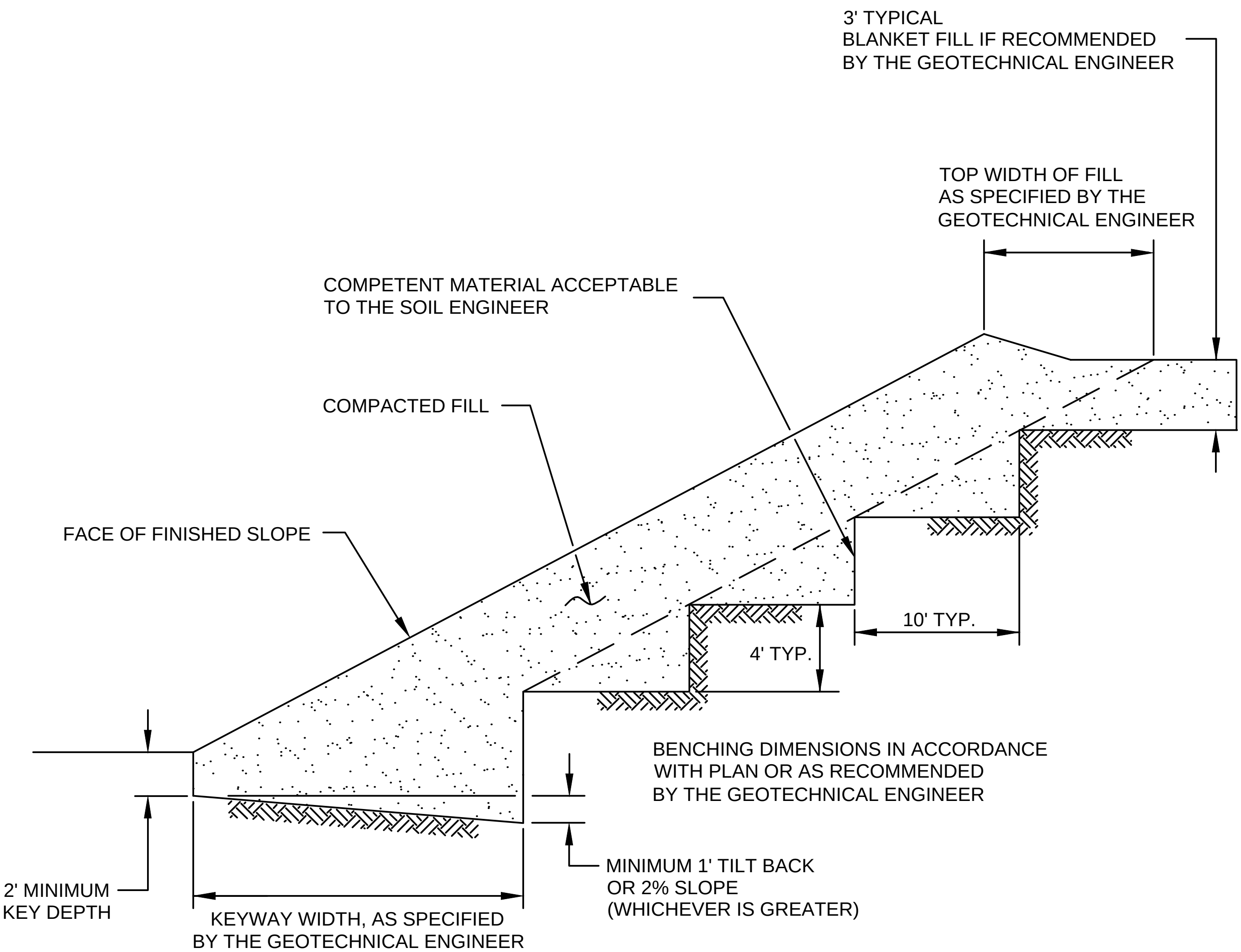
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

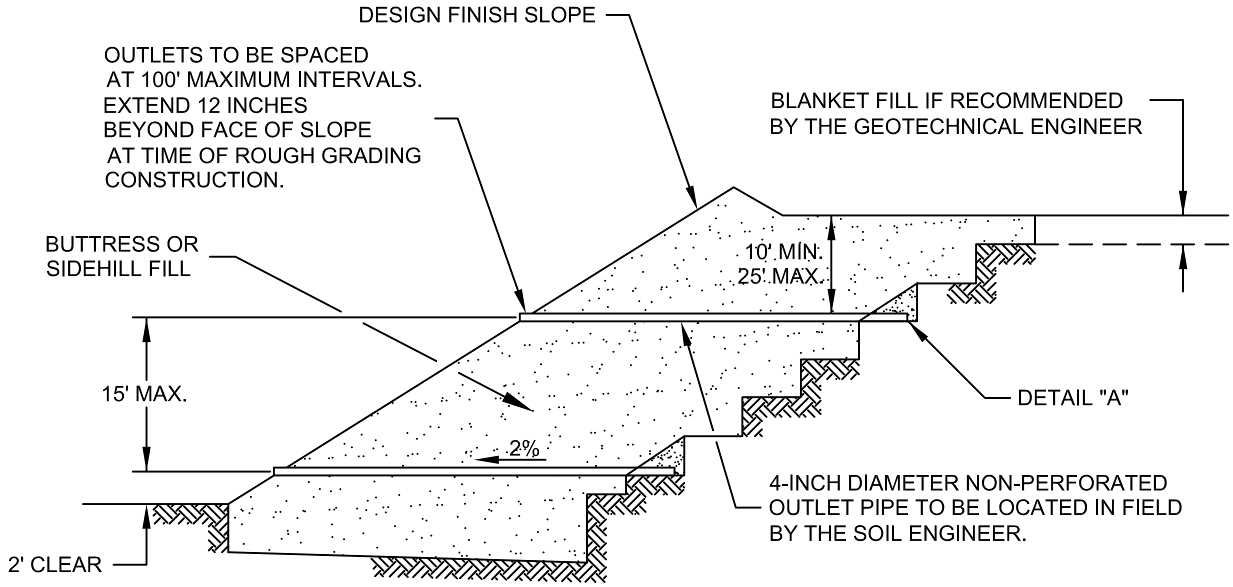
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

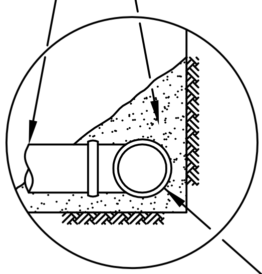
SIEVE SIZE	PERCENTAGE PASSING
1"	100
3/4"	90-100
3/8"	40-100
NO. 4	25-40
NO. 8	18-33
NO. 30	5-15
NO. 50	0-7
NO. 200	0-3

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

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



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

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

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

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

DETAIL "A"

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

SLOPE FILL SUBDRAINS

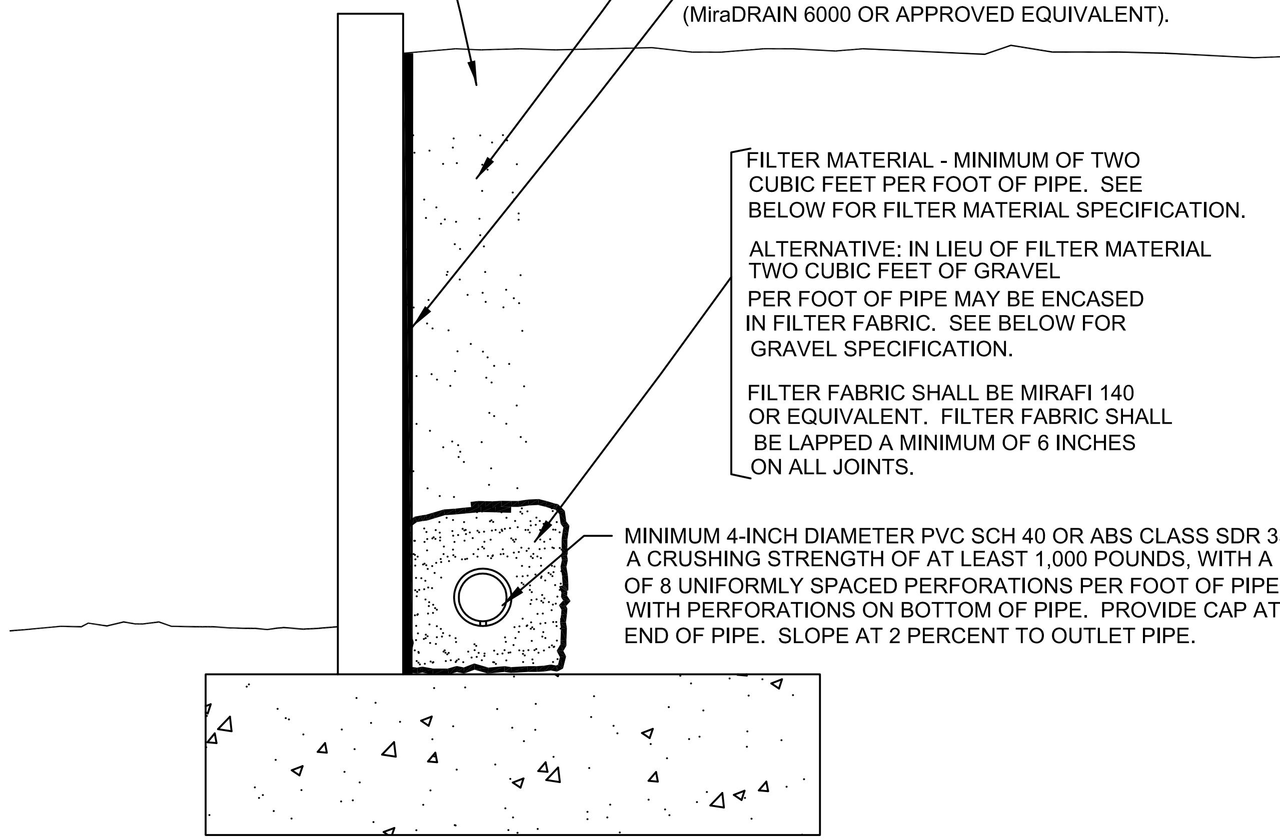
GRADING GUIDE SPECIFICATIONS

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

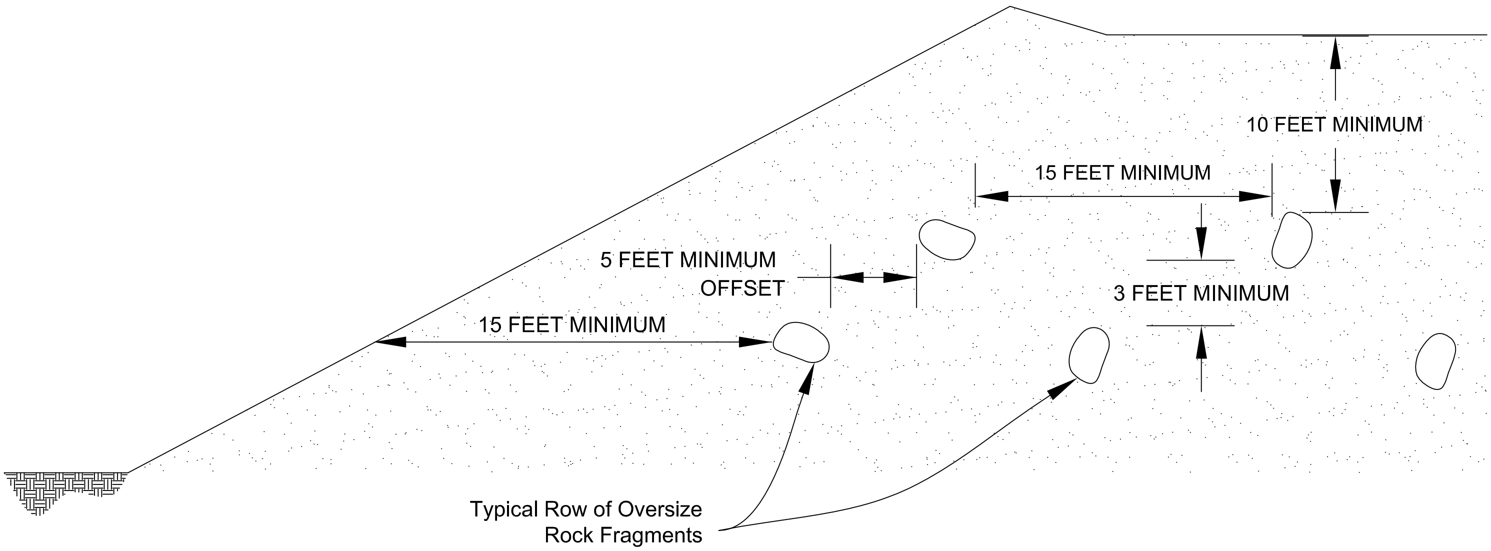
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

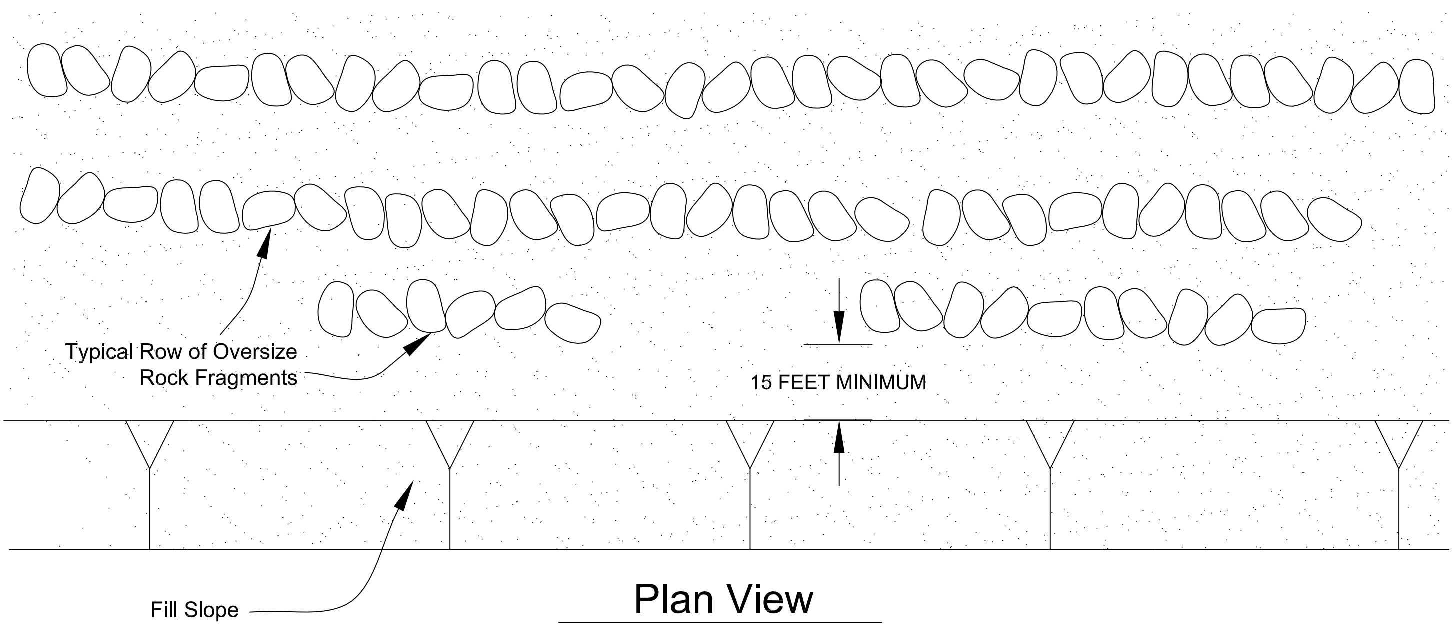
PLATE D-7

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

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

Latitude, Longitude: 34.114768, -117.261944

Date	7/19/2021, 3:40:35 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.232	MCE_R ground motion. (for 0.2 second period)
S_1	0.82	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.232	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.488	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	0.919	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	1.011	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	2.797	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	3.066	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.232	Factored deterministic acceleration value. (0.2 second)
S1RT	1.123	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	1.264	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.82	Factored deterministic acceleration value. (1.0 second)
PGAd	0.919	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.912	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

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

SEISMIC DESIGN PARAMETERS - 2019 CBC

PROPOSED WAREHOUSE

SAN BERNARDINO, CALIFORNIA

DRAWN: JAZ
CHKD: RGT

SCG PROJECT
21G190-1

PLATE E-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

9th Street & Tippecanoe Avenue
San Bernadino, Ca
July 22, 2021

Prepared for:

Mr. Daryl Kas
Southern California Geotechnical, Inc.
22885 E. Savi Ranch Parkway, Ste E
Yorba Linda, CA 92887
Office (714) 685-1115 / Fax (714) 685-1118

Prepared by:

KEHOE TESTING & ENGINEERING
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Huntington Beach, CA 92649-1518
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www.kehoetesting.com

TABLE OF CONTENTS

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at 9th Street & Tippecanoe Avenue in San Bernadino, California. The work was performed by Kehoe Testing & Engineering (KTE) on July 22, 2021. The scope of work was performed as directed by Southern California Geotechnical, Inc. personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at four locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	31	Refusal
CPT-2	21	Refusal
CPT-3	22	Refusal
CPT-4	26	Refusal

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, "Interpretation of Cone Penetration Test…", 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

Steven P. Kehoe
President

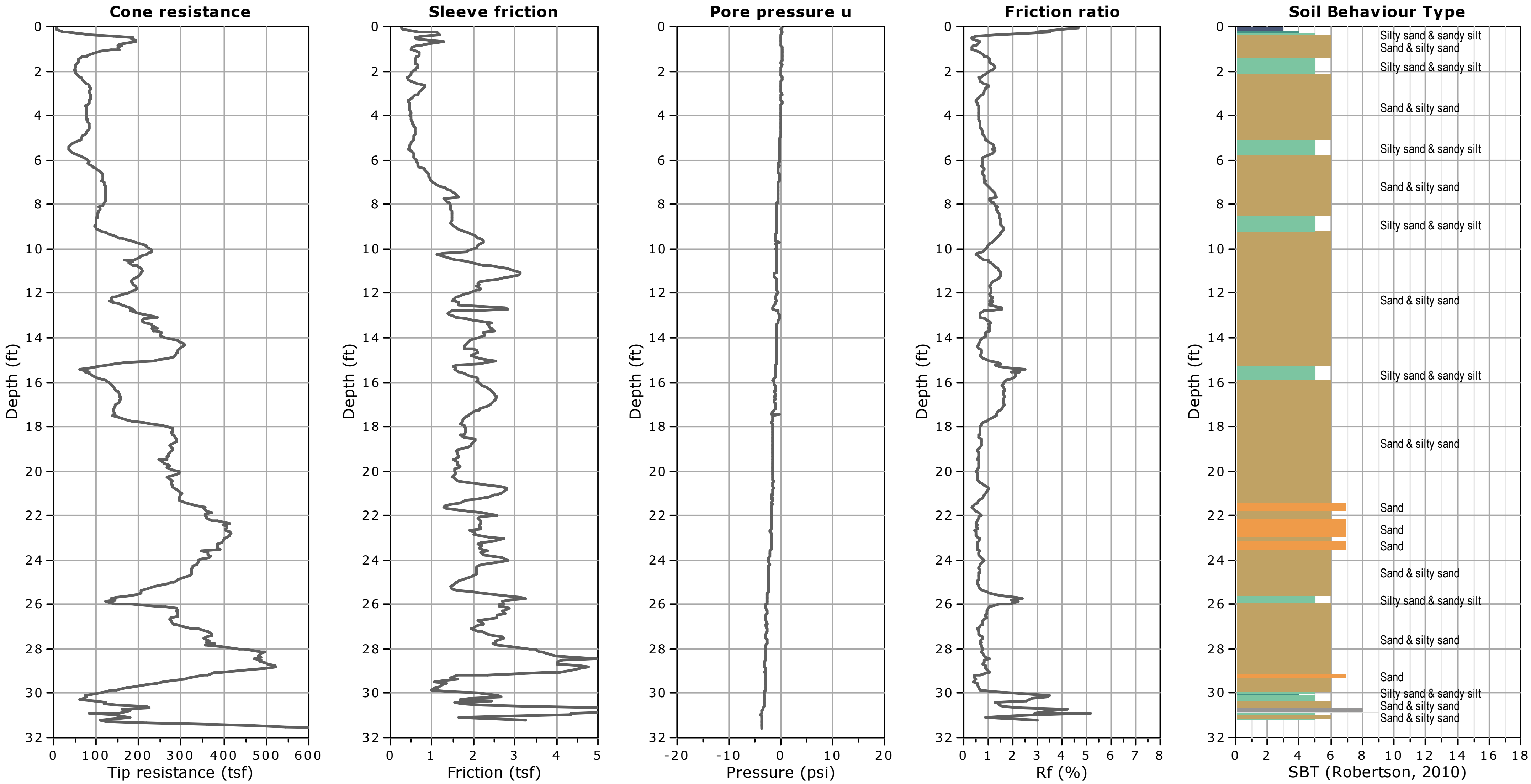
07/26/21-3052

APPENDIX

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Southern California Geotechnical**
Location: **9th St & Tippecanoe, San Bernadino, CA**

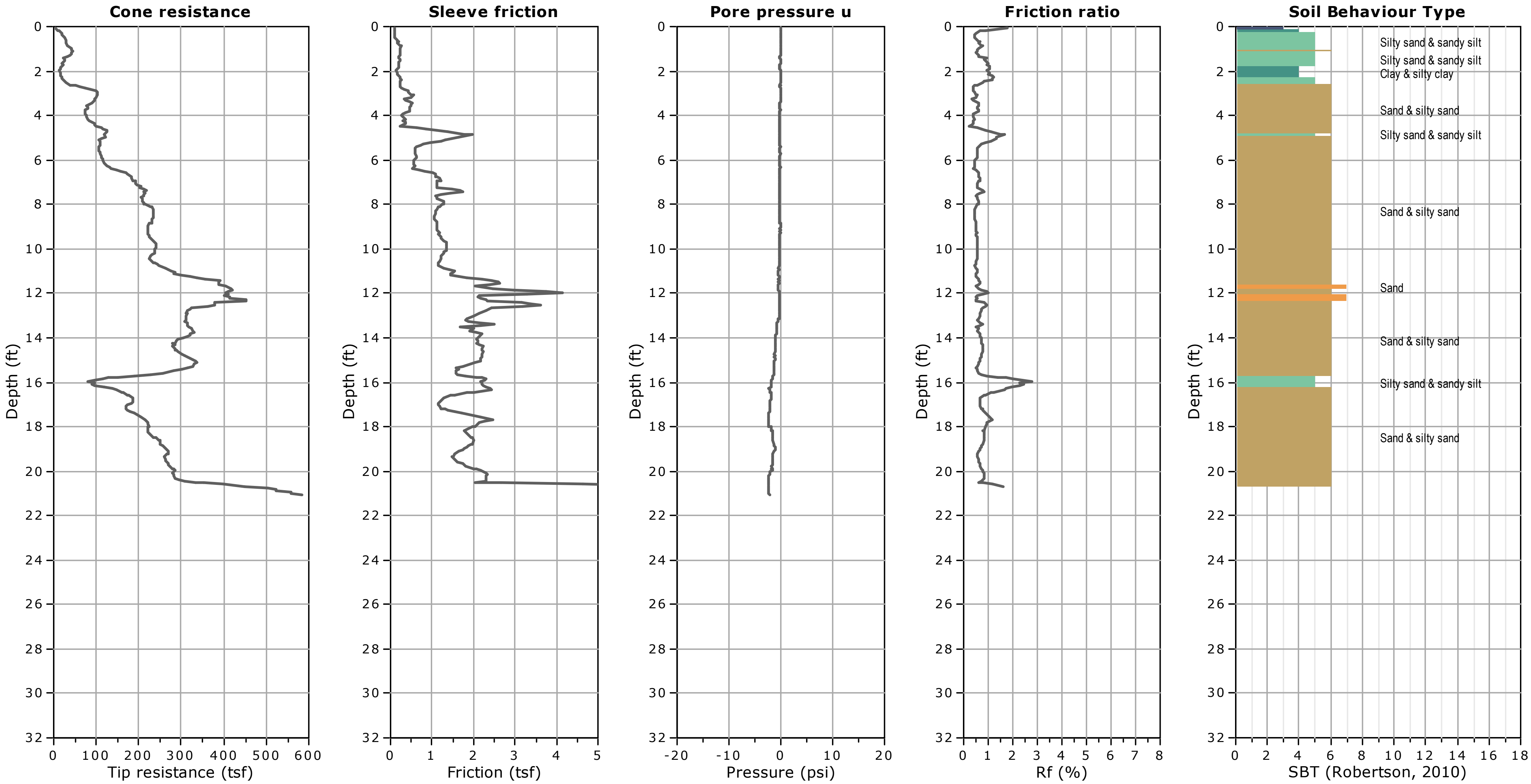
CPT-1
Total depth: 31.57 ft, Date: 7/22/2021



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Southern California Geotechnical**
Location: **9th St & Tippecanoe, San Bernadino, CA**

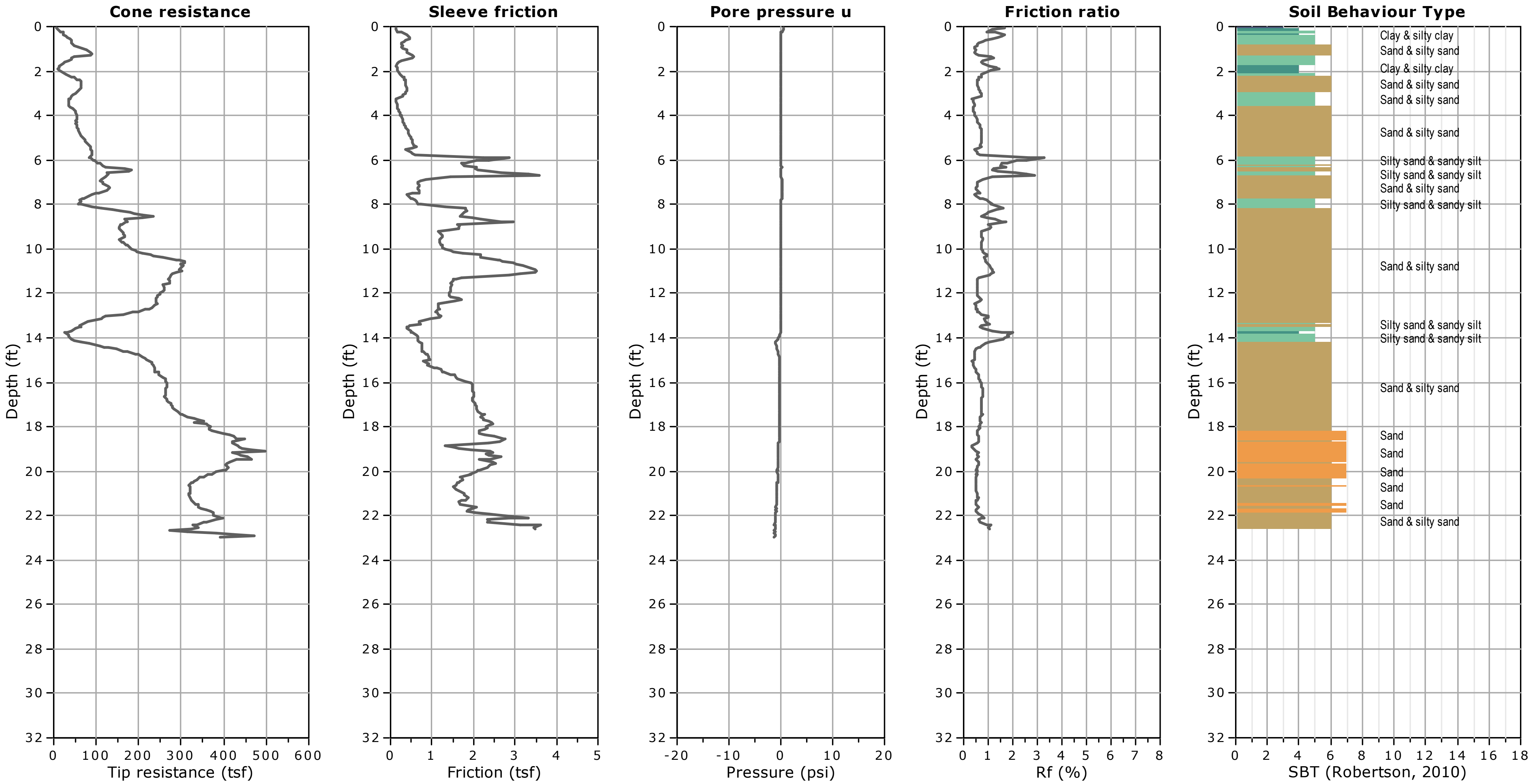
CPT-2
Total depth: 21.07 ft, Date: 7/22/2021



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Southern California Geotechnical **CPT-3**

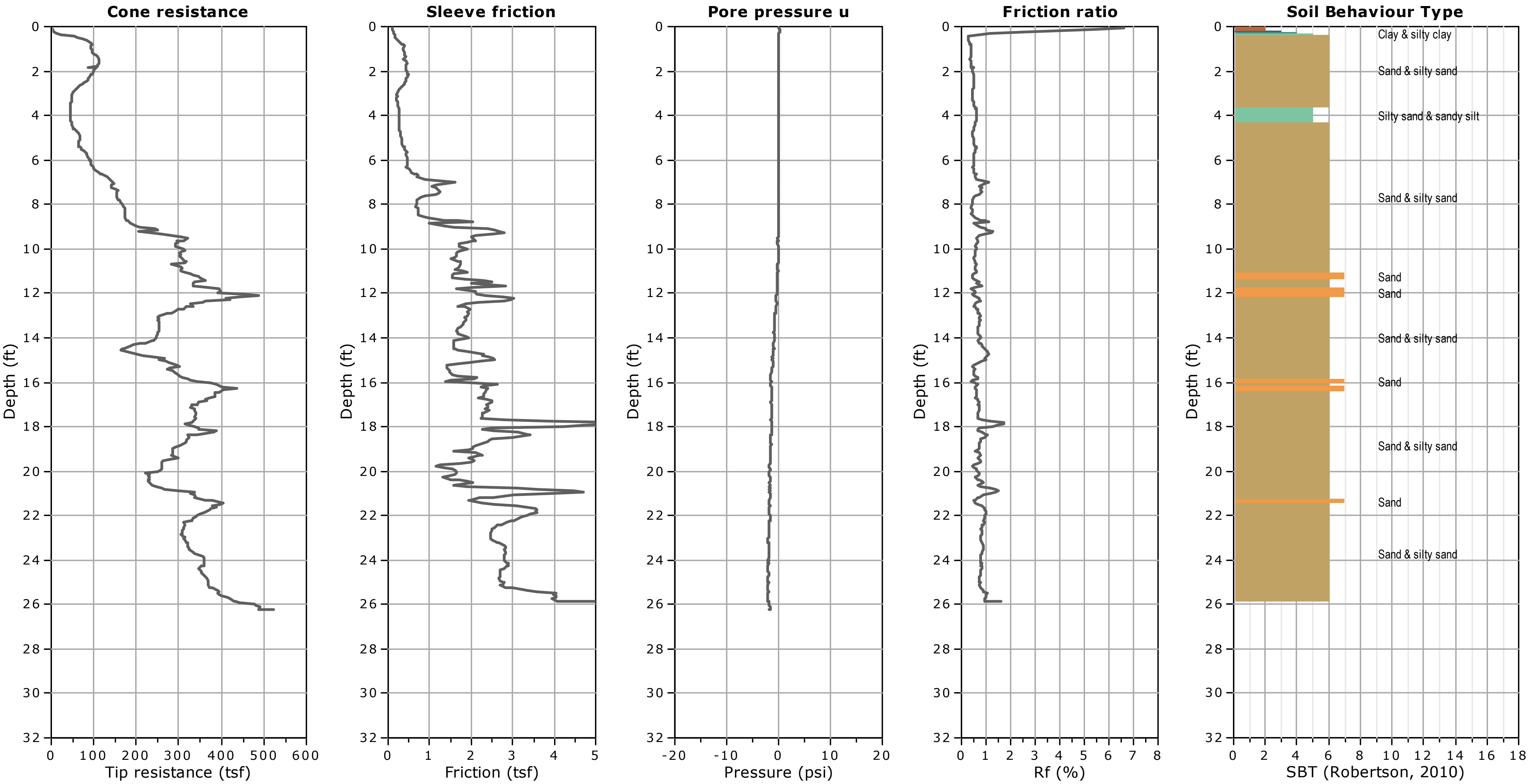
Location: 9th St & Tippecanoe, San Bernadino, CA Total depth: 22.98 ft, Date: 7/22/2021



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Southern California Geotechnical**
Location: **9th St & Tippecanoe, San Bernadino, CA**

CPT-4
Total depth: 26.25 ft, Date: 7/22/2021



Kehoe Testing & Engineering
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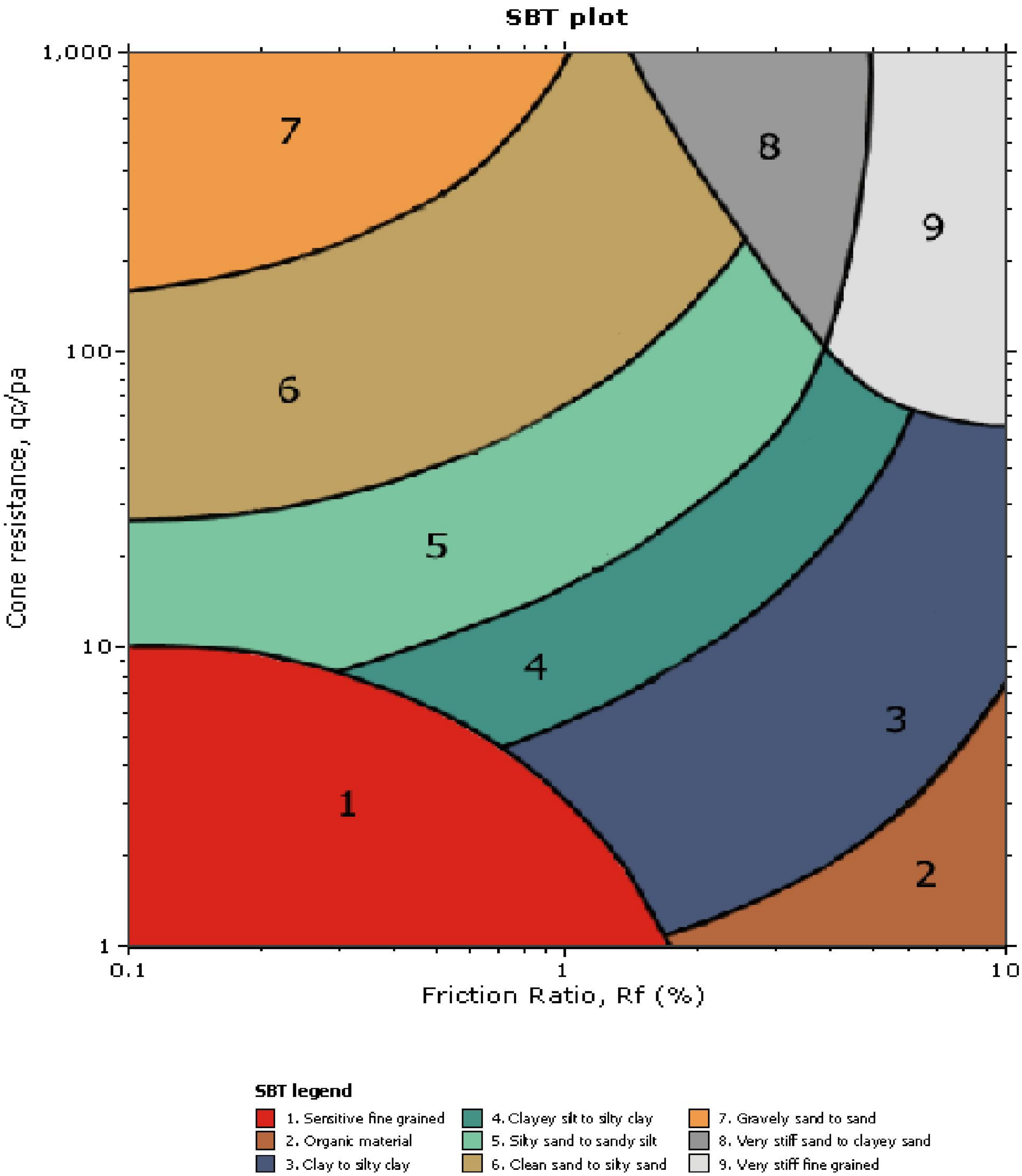
steve@kehoetesting.com
www.kehoetesting.com



APPENDIX G

LIQUEFACTION EVALUATION

Project Name	Proposed Warehouse	MCE$_G$ Design Acceleration	1.011 (g)
Project Location	San Bernardino, CA	Design Magnitude	7.29
Project Number	21G190-1	Historic High Depth to Groundwater	8 (ft)
Engineer	PM	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=7.29)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
9.5	0	8	4		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	480	480	480	1.00	1.01	1.08	0.06	0.07	N/A	N/A	Above Water Table
10.5	8	12	10	20	120	25	1.3	1.05	1.3	1.69	0.75	45.0	50.0	240	115	240	0.98	1.09	1.1	2.00	2.00	1.34	1.49	Nonliquefiable
14.5	12	17	14.5	63	120		1.3	1.05	1.3	1.01	0.85	95.6	95.6	1740	1334	1740	0.96	1.09	1.1	2.00	2.00	0.82	2.43	Nonliquefiable
19.5	17	22	19.5	31	120		1.3	1.05	1.3	0.98	0.95	51.0	51.0	2340	1622	2340	0.94	1.09	1.08	2.00	2.00	0.89	2.24	Nonliquefiable
24.5	22	27	24.5	70	120		1.3	1.05	1.3	1.02	0.95	120.3	120.3	2940	1910	2940	0.92	1.09	1.03	2.00	2.00	0.93	2.15	Nonliquefiable
29.5	27	32	29.5	26	120	8	1.3	1.05	1.3	0.85	0.95	37.3	37.7	3540	2198	3540	0.90	1.09	0.99	2.00	2.00	0.95	2.10	Nonliquefiable
34.5	32	37	34.5	37	120		1.3	1.05	1.3	0.87	1	57.3	57.3	4140	2486	4140	0.87	1.09	0.95	2.00	2.00	0.96	2.09	Nonliquefiable
39.5	37	42	39.5	50	120	3	1.3	1.05	1.3	0.93	1	82.8	82.8	4740	2774	4740	0.85	1.09	0.92	2.00	1.99	0.95	2.09	Nonliquefiable
39.5	42	47	44.5	50	120	5	1.3	1.05	1.3	0.92	1	81.6	81.6	5340	3062	5340	0.82	1.09	0.89	2.00	1.93	0.94	2.04	Nonliquefiable
4435	47	50	48.5	50	120	4	1.3	1.05	1.3	0.91	1	80.7	80.7	5820	3293	5820	0.80	1.09	0.87	2.00	1.88	0.93	2.01	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 Warehouse
Project Location	San Bernardino, CA
Project Number	21G190-1
Engineer	PM

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 cont	$(N_1)_{60\text{-CS}}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	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)			
9.5	0	8	4	0.0	0.0	0.0	N/A	0.50	0.95	0.00	8.00		0.000		0.00	Above Water Table
10.5	8	12	10	45.0	5.1	50.0	1.49	0.00	-1.59	0.00	4.00		0.000		0.00	Nonliquefiable
14.5	12	17	14.5	95.6	0.0	95.6	2.43	0.00	-5.65	0.00	5.00		0.000		0.00	Nonliquefiable
19.5	17	22	19.5	51.0	0.0	51.0	2.24	0.00	-1.67	0.00	5.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	120.3	0.0	120.3	2.15	0.00	-8.04	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	37.3	0.4	37.7	2.10	0.01	-0.63	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	57.3	0.0	57.3	2.09	0.00	-2.19	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	82.8	0.0	82.8	2.09	0.00	-4.46	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	81.6	0.0	81.6	2.04	0.00	-4.35	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	80.7	0.0	80.7	2.01	0.00	-4.26	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 Warehouse	MCE_G Design Acceleration	1.011 (g)
Project Location	San Bernardino, CA	Design Magnitude	7.29
Project Number	21G190-1	Historic High Depth to Groundwater	8 (ft)
Engineer	PM	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-4		

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=7.29)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
9.5	0	8	4		120		1.3	1.05	1.1	1.70	0.75	0.0	0.0	480	480	480	1.00	1.01	1.08	0.06	0.07	N/A	N/A	Above Water Table
10.5	8	12	10	19	120		1.3	1.05	1.3	1.70	0.75	43.0	43.0	240	115	240	0.98	1.09	1.1	2.00	2.00	1.34	1.49	Nonliquefiable
14.5	12	17	14.5	50	120		1.3	1.05	1.3	1.02	0.85	77.1	77.1	1740	1334	1740	0.96	1.09	1.1	2.00	2.00	0.82	2.43	Nonliquefiable
19.5	17	22	19.5	48	120		1.3	1.05	1.3	0.99	0.95	80.1	80.1	2340	1622	2340	0.94	1.09	1.08	2.00	2.00	0.89	2.24	Nonliquefiable
24.5	22	27	24.5	65	120		1.3	1.05	1.3	1.01	0.95	110.4	110.4	2940	1910	2940	0.92	1.09	1.03	2.00	2.00	0.93	2.15	Nonliquefiable
29.5	27	32	29.5	28	120	5	1.3	1.05	1.3	0.86	0.95	40.6	40.6	3540	2198	3540	0.90	1.09	0.99	2.00	2.00	0.95	2.10	Nonliquefiable
34.5	32	37	34.5	50	120		1.3	1.05	1.3	0.95	1	84.1	84.1	4140	2486	4140	0.87	1.09	0.95	2.00	2.00	0.96	2.09	Nonliquefiable
39.5	37	42	39.5	46	120		1.3	1.05	1.3	0.90	1	73.9	73.9	4740	2774	4740	0.85	1.09	0.92	2.00	1.99	0.95	2.09	Nonliquefiable
39.5	42	47	44.5	50	120		1.3	1.05	1.3	0.92	1	81.6	81.6	5340	3062	5340	0.82	1.09	0.89	2.00	1.93	0.94	2.04	Nonliquefiable
4435	47	50	48.5	50	120		1.3	1.05	1.3	0.91	1	80.7	80.7	5820	3293	5820	0.80	1.09	0.87	2.00	1.88	0.93	2.01	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 Warehouse
Project Location	San Bernardino, CA
Project Number	21G190-1
Engineer	PM

Boring No. B-4

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 cont	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter $F\alpha$	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)			
9.5	0	8	4	0.0	0.0	0.0	N/A	0.50	0.95	0.00	8.00		0.000		0.00	Above Water Table
10.5	8	12	10	43.0	0.0	43.0	1.49	0.00	-1.03	0.00	4.00		0.000		0.00	Nonliquefiable
14.5	12	17	14.5	77.1	0.0	77.1	2.43	0.00	-3.93	0.00	5.00		0.000		0.00	Nonliquefiable
19.5	17	22	19.5	80.1	0.0	80.1	2.24	0.00	-4.21	0.00	5.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	110.4	0.0	110.4	2.15	0.00	-7.07	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	40.6	0.0	40.6	2.10	0.01	-0.85	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	84.1	0.0	84.1	2.09	0.00	-4.57	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	73.9	0.0	73.9	2.09	0.00	-3.64	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	81.6	0.0	81.6	2.04	0.00	-4.34	0.00	5.00		0.000		0.00	Nonliquefiable
49.5	47	50	48.5	80.7	0.0	80.7	2.01	0.00	-4.26	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)

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(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)