GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL BUILDING

15006-15100 Nelson Avenue
Industry, California
for
Nelson Avenue Owner LP

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

January 31, 2023

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Nelson Avenue Owner LP
19700 S. Vermont Avenue, Suite 101
Torrance, CA 90502

Attention: Ms. Montana Kanen
Analyst

Project No.: **21G265-1R**

Subject: **Geotechnical Investigation**
Proposed Industrial Building
15006 – 15100 Nelson Avenue
Industry, California

Ms. Kanen:

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

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

Respectfully Submitted,

SOUTHERN CALIFORNIA GEOTECHNICAL, INC.

Pablo Montes
Staff Engineer

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Principal Engineer

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1.0 EXECUTIVE SUMMARY

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

Geotechnical Design Considerations

- The results of the liquefaction evaluation indicate that some of the on-site soils are susceptible to liquefaction during a major seismic event. Based on the liquefaction evaluation, total dynamic settlements ranging from 0.6 to 3.3± inches could occur at the site during the design seismic event concurrent with historically high groundwater levels.
- Based on the predicted total settlements, the dynamic differential settlements are expected to be on the order of 1½ to 2± inches.
- Artificial fill soils were encountered at some of the boring locations, extending to depths of 2½ to 3± feet. The existing fill soils 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 artificial fill soils are underlain by native alluvial soils. Results of laboratory testing indicate that the native alluvium possesses favorable consolidation characteristics. However, it is anticipated that demolition of the existing structures and associated improvements will cause disturbance of the upper 3 to 5± feet of soil.

Site Preparation

- Demolition of the existing structures and pavements at the subject site will be necessary in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, pavements, utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of off-site. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well mixed with the sandy on-site soils, and incorporated into new structural fills or it may be crushed to create crushed miscellaneous base (CMB), if desired.
- Remedial grading is recommended to be performed within the proposed building area in order to remove the undocumented fill soils in their entirety, the upper portion of the near-surface native alluvial soils, and any soils disturbed during the demolition process. The soils within the proposed building area should be overexcavated to a depth of 3 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevation. The depth of overexcavation should also be sufficient to remove any existing fill soils.
- The proposed foundation influence zones should be overexcavated 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 and moisture conditioned to at least 2 to 4 percent above optimum moisture content. The overexcavation subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

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

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,500 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to minor amounts of liquefaction-induced settlement, and the presence of medium expansive soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab

- Conventional Slab-on-Grade, 6 inches thick.
- Modulus of Subgrade Reaction: k = 80 psi/in.
- Minimum slab reinforcement: Reinforcement of the floor slabs should consist of No. 3 bars at 18-inches on center in both directions due to the presence of potentially liquefiable soils and medium expansive soils. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

Pavements

ASPHALTIC CONCRETE PAVEMENTS (R=5)					
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	10	13	16	18	21
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R=5)				
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	7	8½
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. 21P363R, dated November 2, 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 on the southwest side of Nelson Avenue at Cadbrook Drive in City of Industry, California. The site is also referenced by the street addresses 15006, 15010 and 15100 Nelson Avenue. The site is bounded to the northeast by Nelson Avenue, to the northwest by an existing commercial/industrial building, to the southwest by a railroad easement, and to the southeast by existing commercial/industrial buildings. The general location of the site is illustrated on the Site Location Map, included as Plate 1 of this report.

The site consists of two irregular-shaped parcels, 8.87± acres in size. The site is presently developed with five (5) commercial/industrial buildings, ranging in size from 1,400 to 14,850± ft^2 in size. Additionally, the site is developed with one (1) canopy, 2,500± ft^2 in size. Pipe products are presently stored in the northwest area of the site. Semi-trucks are presently parked in the southwestern area of the site. A semi-truck driving school is in the southeastern area of the site. Ground surface consists of Portland cement concrete (PCC) pavements across the majority of the site, with asphaltic concrete pavements in northeastern area of the site. Landscape planters are present in the northwestern area of the site.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth and visual observations made at the time of the subsurface investigation, the overall site slopes downward to the southwest at gradients ranging from 0.5 to 1± percent.

3.2 Proposed Development

Based on a conceptual site plan provided to our office by the client, the site will be developed with one (1) new industrial building. The new building will be 142,730± ft^2 in size, and will be located in the southern area of the site. Dock-high doors will be constructed along a portion of the north building wall. The building will be surrounded by asphaltic concrete pavements in the parking and drive lanes, Portland cement concrete pavements in the loading dock area, and limited areas of concrete flatwork and landscape planters throughout.

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

No significant amounts of below grade construction, such as crawl spaces or new basements, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 2 to 3± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration for this project consisted of five (5) borings advanced to depths of 15 to 50± feet below the existing site grades. Three (3) of the borings were advanced to a depth of 50± feet as a part of the liquefaction evaluation. In addition to the borings, four (4) Cone Penetration Test (CPT) soundings were advanced to a depth of 50± feet at the site as part of the liquefaction analysis. All of the borings were logged during drilling by a member of our staff.

Hollow Stem Auger Borings

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

Cone Penetration Test (CPT) Soundings

The CPT soundings were performed by Kehoe Testing and Engineering (KTE) under the supervision of an SCG engineer. 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. Both of the CPT soundings were advanced to depths of 100± feet. 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 in the CPT report, 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 two reports prepared by KTE, included in Appendix F of this report.

4.2 Geotechnical Conditions

Pavement

Asphaltic concrete pavements were encountered at the ground surface of Boring No. B-4. The pavement section consists of 3± inches of Asphaltic concrete underlain by 2± inches of Aggregate base.

Portland cement concrete, 6± inches in thickness, was encountered at the ground surface of Boring Nos. B-1, B-2, B-3 and B-5.

Artificial Fill

Artificial fill soils were encountered beneath the pavements of Boring Nos. B-1, B-2 and B-5, extending to depths of 2½ to 3± feet below ground surface. The fill soils generally consist of medium stiff to very stiff silty clays and clayey silts. The fill soils possess a disturbed and mottled appearance, with some samples possessing a slight hydrocarbon odor, resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered beneath the pavements of Boring Nos. B-3 and B-4, and beneath the fill of the remaining borings, extending to at least the maximum depth explored of 50± feet below ground surface. The near-surface alluvial soils generally consist of loose to medium dense silty sands, clayey sands and sandy silts, and medium stiff to stiff silty clays, sandy clays and clayey silts, extending to depths of 12 to 22± feet. At greater depths the alluvium consists of medium dense to very dense fine to coarse sands, silty sands and sandy silts, and stiff to very stiff silty clays and sandy clays. The alluvium generally possesses trace to little iron oxide staining and calcareous nodules/veining.

Groundwater

Free water was not encountered during the drilling of any of the borings. Based on the lack of any water within the borings, and the moisture contents of the recovered soil samples, the static groundwater table is considered to have existed at a depth in excess of 50± feet at the time of the subsurface exploration.

As part of our research, we reviewed available groundwater data in order to determine the historic high groundwater level for the site. The primary reference used to determine the historic groundwater depths in this area is the California Geological Survey (CGS) Open File Report 98-13, for the Baldwin Park 7.5-Minute Quadrangle, which indicates that the historic high groundwater level for the site was about 10 feet below the ground surface.

Recent water level data was obtained from the California State Water Resources Control Board, GeoTracker, website, https://geotracker.waterboards.ca.gov/. One monitoring wells on record is located 30± feet north of the site. Water level readings within this monitoring wells indicate a high groundwater level of 66± feet below the ground surface in September 2016.

5.0 LABORATORY TESTING

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

Classification

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

Density and Moisture Content

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

Consolidation

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

Maximum Dry Density and Optimum Moisture Content

Two (2) representative bulks sample have been tested for their maximum dry densities and optimum moisture contents. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557, and are presented on Plates C-9 and C-10 in Appendix C of this report. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Expansion Index

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

Sample Identification	Expansion Index	Expansive Potential
B-1 @ 0 to 5 feet	75	Medium
B-3 @ 0 to 5 feet	57	Medium

Soluble Sulfates

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

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-1 @ 0 to 5 feet	0.010	Not Applicable (S0)
B-3 @ 0 to 5 feet	0.074	Not Applicable (S0)

Corrosivity Testing

Representative bulk samples of the near-surface soils were submitted to a subcontracted analytical laboratory for determination of electrical resistivity, pH, and chloride concentrations. The resistivity of the soils is a measure of their potential to attack buried metal improvements such as utility lines. The results of the resistivity and pH testing are presented below:

Sample Identification	Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)
B-1 @ 0 to 5 feet	1,400	7.4	8.1	17
B-3 @ 0 to 5 feet	560	8.3	122	34

Grain Size Analysis

Limited grain size analyses have been performed on several selected samples, in accordance with ASTM D-1140. These samples were washed over a #200 sieve to determine the percentage of fine-grained material in each sample, which is defined as the material which passes the #200 sieve. The weight of the portion of the sample retained on each screen is recorded and the percentage finer or coarser of the total weight is calculated. The results of these laboratory tests are shown on the attached boring logs.

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on one or more recovered soil samples. This test is used to determine the Liquid Limit and Plastic Limit of the soil. The Plasticity Index is the difference between the two limits. Plasticity Index is a general indicator of the expansive potential of the soil, with higher numbers indicating higher expansive potential. Soils with a PI greater than 25 are considered to have a high plasticity, and a high expansion potential. The results of the Atterberg Limits testing are presented on the test boring logs.

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

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

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

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.753
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.628
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.753
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.068
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.169
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.712

*The 2019 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site *coefficients* are to be determined in accordance with Section 11.4.7

of ASCE 7-16. However, Section 20.3.1 of ASCE 7-16 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structures is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structure has a fundamental period greater than 0.5 seconds, a site-specific seismic hazards analysis will be required and additional subsurface exploration will be necessary.

It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the 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 liquefaction evaluation, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2019 CBC. The peak ground acceleration (PGA_M) 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, based on ASCE 7-16 as the building code reference document. A portion of the program output is included as Plate E-1 in Appendix E of this report. As indicated on Plate E-1, the PGA_M for this site is 0.823g. 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 6.84, based on the peak ground acceleration and soil classification D.

Liquefaction

Research of the Earthquake Zones of Required Investigation Map for the Baldwin Park Quadrangle, published by the California Geological Survey (CGS) indicates that the site subject site is located within a liquefaction hazard zone. Based on this mapping, and the subsurface conditions encountered at the borings, the scope of this investigation included a detailed liquefaction evaluation in order to determine the site-specific liquefaction potential.

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

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

As part of the liquefaction evaluation, Boring Nos. B-1, B-3 and B-4, and CPT-1 through CPT-4, inclusive, were planned to be extended to a depth of 50 feet. Borings were also drilled in close proximity to the CPT locations to provide physical samples for further analysis and correlation with the CPT data. However, all of the CPTs were terminated at depths shallower than planned due to refusal on very dense soils. The soils below the refusal depths for the CPT soundings are considered non-liquefiable. The liquefaction potential for the on-site soils was evaluated the computer program CLiq V3.0.3.2, which was developed by Geologismiki, copyright 2007. The analysis method is based on Boulanger and Idriss, 2014. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.823g for a magnitude 6.84 seismic event. The potential settlements that could occur as a result of liquefaction for each of the potentially liquefiable layers was calculated. A copy of the program output is presented in Appendix G of this report. The historic high groundwater depth used in the liquefaction analysis was obtained from CGS Open File Report 98-13, the Seismic Hazard Evaluation of the Baldwin Park Quadrangle, which indicates that the minimum historic depth to groundwater at the site is approximately 10± feet below the ground surface in the area of the subject site.

Conclusions and Recommendations

The results of the liquefaction analysis have identified potentially liquefiable soils at all of the three (3) 50-foot boring locations, and at all of the four (4) CPT soundings performed at the site. Soils which are located above the historic groundwater table or possess factors of safety of at least 1.3 are considered non-liquefiable. Several clayey strata encountered between depths of 27 and 2± feet are considered to be non-liquefiable due to their cohesive characteristics, as interpreted by the CPT. Settlement analyses were conducted for each of the potentially liquefiable strata. The total dynamic settlement for each CPT location, based on the results of the dynamic settlement analyses (presented in Appendix G) are presented below:

- Boring No. B-1: 3.32± inch
- Boring No. B-3: 2.87± inch
- Boring No. B-4: 3.11± inch
- CPT-1: 0.60± inch
- CPT-2: 0.76± inch

- CPT-3: 2.19± inch
- CPT-4: 1.99± inch

The associated differential settlement is estimated to be on the order of 1.5 to 2± inches. The estimated differential settlement could be assumed to occur across a distance of 100 feet, indicating a maximum angular distortion of less than 0.002± inches per inch.

Based on our understanding of the proposed development, it is considered feasible to support the proposed structures on shallow foundations. Such a foundation system can be designed to resist the effects of the anticipated differential settlements, to the extent that the structures would not catastrophically fail. Designing the proposed structures to remain completely undamaged during a seismic event that could occur once every 2,500 years the code-specified return period used in the liquefaction analysis is not considered to be economically feasible. Based on this understanding, the use of a shallow foundation system is considered to be the most economical means of supporting the proposed structures.

In order to support the proposed structures on shallow foundations such as spread footings the structural engineer should verify that the structures would not catastrophically fail due to the predicted dynamic differential settlements. Any utility connections to the structures should be designed to withstand the estimated differential settlements. It should also be noted that minor to moderate repairs, including re-leveling, restoration of utility connections, repair of damaged drywall and stucco, etc., would likely be required after occurrence of the liquefaction-induced settlements.

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

6.2 Geotechnical Design Considerations

General

Undocumented fill soils were encountered at most of the boring locations, extending to depths of 2½ to 3± feet. These fill soils possess variable strengths and compositions, and no documentation concerning the placement or compaction of these soils is currently available. Based on these conditions, the undocumented fill soils are not considered suitable for support of the proposed structure, in their present condition. Beneath the fill soils, the borings encountered moderate to high strength native alluvium. Based on these conditions, remedial grading will be necessary within the proposed building area to remove the existing undocumented fill soils and a portion of the near surface native alluvial soils in order to replace these materials as compacted structural fill.

As discussed in the previous section of this report, potentially liquefiable soils were identified at this site. The presence of the recommended layer of newly placed compacted structural fill above these liquefiable soils will help to reduce any surface manifestations that could occur as a result of liquefaction. The foundation design recommendations presented in the subsequent sections of this report also contain recommendations to provide additional rigidity in order to reduce the potential effects of differential settlement that could occur as a result of liquefaction.

Settlement

The recommended remedial grading will remove the undocumented fill soils and a portion of the near-surface native alluvium from within the foundation influence zones, and replace these materials as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation will not be subject to significant load increases from the foundations of the new structure. Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structure are expected to be within tolerable limits.

Expansion

Laboratory testing performed on a sample of the near-surface soils indicates that some of the near-surface soils possess medium expansion potentials (EI = 57 and 75). Based on the presence of expansive soils at this site, care should be given to proper moisture conditioning of all building pad subgrade soils to a moisture content of 2 to 4 percent above the ASTM D-1557 optimum during site grading. In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintaining moisture content of these soils at 2 to 4 percent above the optimum moisture content. This will require the contractor to frequently moisture condition these soils throughout the grading process, unless grading occurs during a period of relatively wet weather.

Soluble Sulfates

The results of the laboratory testing indicate that the sulfate concentrations of the selected samples of the on-site soils correspond to Class S0 with respect to the American Concrete Institute (ACI) Publication 318-14 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building areas.

Corrosion Potential

The results of the electrical resistivity and pH testing indicate that tested samples of the on-site soils have saturated resistivities between 560 and 1,400 ohm-cm with pH values ranging between 7.4 and 8.3. 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. Relative soil moisture content as well as redox potential and sulfides are also included. Although redox potential and sulfide testing were 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**

considered to be corrosive to ductile iron pipes and other buried metallic improvements. Therefore, it is expected that polyethylene encasement will be required for iron pipes. If a more detailed evaluation is desired, we recommend that the client contact a corrosion engineer to provide a more thorough evaluation because SCG does not practice in the area of corrosion engineering.

Chloride concentrations between 8.1 and 122 mg/kg were detected in the samples submitted for corrosivity testing. The Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 ppm are considered to be corrosive. Based on the relatively low concentrations of significant chlorides in the tested samples, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested samples possess nitrate concentrations of 17 to 34 mg/kg. Based on these test results, the on-site soils are not corrosive to copper pipe.

It should be noted that SCG does not practice in the field of corrosion engineering. Therefore, the client may wish to contact a corrosion engineer to provide a more thorough evaluation

Shrinkage/Subsidence

Removal and recompaction of the near-surface fill and native soils is estimated to result in an average shrinkage of 5 to 12 percent. However, potential shrinkage for individual samples ranged locally between 3 and 18 percent. It should be noted that the potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.10 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

It is recommended that we be provided with copies of the grading and foundation plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 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 the existing structures and pavements will be necessary at this site to facilitate the proposed development. Demolition should include all foundations, floor slabs, pavements, septic systems, utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of off-site. Alternatively, concrete and asphalt debris may be pulverized to a maximum 2-inch particle size, well mixed with the sandy on-site soils, and incorporated into new structural fills or it may be crushed to create crushed miscellaneous base (CMB), if desired. We do not recommend that demolition debris be blended with clayey soils for use as fill. Alternatively, concrete and asphalt debris may also be crushed to a particle size of 2 to 4 inches and used as stabilization material for use at the base of the recommended overexcavation, as discussed in a Section 6.3 of this report.

Initial site preparation should also include stripping of any surficial vegetation and organic soils. Removal of any trees or shrubs should also include the associated root masses. All of these materials should be disposed of offsite. 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 encountered materials.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building pad area in order to remove the existing undocumented fill soils and a portion of the near-surface native alluvium. Additionally, the necessary demolition at this site will result in significant disturbance of the near surface soils. Any soils disturbed during demolition should also be overexcavated. At a minimum, we recommend that the existing soils with in the proposed building pad area be overexcavated to a depth of at least 3 feet below existing grade, and to a depth of at least 3 feet below proposed building pad grade, whichever is greater. Additional overexcavation should be performed within the foundation influence zones for the new structure. We recommend that the overexcavation extend to a depth of at least 3 feet below the proposed foundation bearing grades within the influence zones of the new foundations. The overexcavation should also extend to a sufficient depth to remove any undocumented fill soils and any soils disturbed during demolition. At the boring locations, the artificial fill soils extend to depths of 2½ to 3± feet.

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 overexcavations should also encompass these areas.

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

Based on conditions encountered at the exploratory boring locations, very moist soils will be encountered at or near the base of the recommended overexcavation. If grading is performed within a period of favorable weather, scarification and air drying of these materials may be sufficient to obtain a stable subgrade. However, if highly unstable soils are identified, and if the construction schedule does not allow for delays associated with drying, mechanical stabilization, usually consisting of coarse crushed stone and/or a geotextile, may be necessary in localized areas. In this event, the geotechnical engineer should be contacted for supplementary recommendations. Typically, an unstable subgrade can be stabilized using a suitable geotextile fabric, such as Mirafi 580I, HP 570 or HP 270, and/or a 12 to 18-inch thick layer of coarse (2 to 4-inch particle size) crushed stone. Crushed asphalt and concrete debris resultant from demolition could also be used as a subgrade stabilization material. Other options, including lime or cement treatment are also available. Typically an unstable subgrade may stabilized by treating the upper 12 to 18 inches of subgrade material with cement to a concentration of 5 to 6 percent (by dry weight of soil).

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of at least 12 inches and moisture conditioned or air dried to achieve a moisture content of 2 to 4 percent above optimum moisture content. The subgrade soils should then be recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The building pad area may then be raised to grade with previously excavated soils or imported, structural fill. All structural fill soils present within the proposed building area should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of 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. The overexcavation areas should extend at least 5 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Any erection pads for tilt-up concrete walls are considered to be part of the foundation system. Therefore, these overexcavation recommendations are applicable to erection pads. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to within 2 to 4 percent above the optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral recommended remedial grading cannot be completed for the proposed retaining walls and site walls located along property lines, the foundations for those walls should be designed using a reduced allowable bearing pressure. Furthermore, the contractor should take necessary precautions to protect the adjacent improvements during rough grading. Specialized grading techniques, such as A-B-C slot cuts, will likely be required during remedial grading. The geotechnical engineer of record should be contacted if additional recommendations, such as shoring design recommendations, are required during grading.

Treatment of Existing Soils: Parking Areas

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

Subgrade preparation in the new parking areas should initially consist of removal of all soils disturbed during stripping and/or 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 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of undocumented fill soils 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 the existing undocumented fill soils and compressible alluvium in the parking areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should be overexcavated to a depth of 2 feet below proposed pavement subgrade elevation, with the resulting soils replaced as compacted structural fill.

Treatment of Existing Soils: Flatwork Areas

Subgrade preparation in the new flatwork 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 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of existing undocumented fill and variable strength alluvial soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

Some movement and associated cracking of the flatwork materials should be expected, due to the presence of these expansive soils. If this movement and the associated cracking cannot be tolerated, consideration should be given to the use of an imported, non-expansive, granular fill material in order to reduce the potential for differential movements of lightly loaded slabs. Such select fill material could be placed within the upper 2± feet below the flatwork subgrade as compacted structural fill.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted. **Drying of the on-site soils will be required before placement and compaction as fill.**
- 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 CBC and the grading code of the city of Industry.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

All imported structural fill should consist of low expansive (EI < 50), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

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

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

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of moderate strength silty clays and sandy clays. Some of these materials may be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes should be made no steeper than 1½h:1v. Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Expansive Soils

Based on results of laboratory testing, the near-surface soils at this site possess medium expansion potentials. Due to the presence of expansive soils at this site, provisions should be made to limit the potential for surface water to penetrate the soils immediately adjacent to the structures. These provisions should include directing surface runoff into rain gutters and area drains, reducing the extent of landscaped areas around the structure, and sloping the ground surface away from the buildings. Where possible, it is recommended that landscaped planters not be located immediately adjacent to the buildings. If landscaped planters around the buildings are necessary, it is recommended that drought tolerant plants or a drip irrigation system be utilized, to minimize the potential for deep moisture penetration around the structures. Presented below is a list of additional soil moisture control recommendations that should be considered by the owner, developer, and civil engineer:

- Ponding and areas of low flow gradients in unpaved walkways, grass and planter areas should be avoided. In general, minimum drainage gradients of 2 percent should be maintained in unpaved areas.
- Bare soil within five feet of proposed structures should be sloped at a minimum five percent gradient away from the structure (about three inches of fall in five feet), or the same area could be paved with a minimum surface gradient of one percent. Pavement is preferable.
- Decorative gravel ground cover tends to provide a reservoir for surface water and may hide areas of ponding or poor drainage. Decorative gravel is, therefore, not recommended and should not be utilized for landscaping unless equipped with a subsurface drainage system designed by a licensed landscape architect.
- Positive drainage devices, such as graded swales, paved ditches, and catch basins should be installed at appropriate locations within the area of proposed development.
- Concrete walks and flatwork should not obstruct the free flow of surface water to the appropriate drainage devices.
- Area drains should be recessed below grade to allow free flow of water into the drain. Concrete or brick flatwork joints should be sealed with mortar or flexible mastic.
- Gutter and downspout systems should be installed to capture all discharge from roof areas. Downspouts should discharge directly into a pipe or paved surface system to be conveyed offsite.
- Enclosed planters adjoining, or in close proximity to proposed structures, should be sealed at the bottom and provided with subsurface collection systems and outlet pipes.
- Depressed planters should be raised with soil to promote runoff (minimum drainage gradient two percent or five percent, see above), and/or equipped with area drains to eliminate ponding.
- Drainage outfall locations should be selected to avoid erosion of slopes and/or properly armored to prevent erosion of graded surfaces. No drainage should be directed over or towards adjoining slopes.
- All drainage devices should be maintained on a regular basis, including frequent observations during the rainy season to keep the drains free of leaves, soil and other debris.
- Landscape irrigation should conform to the recommendations of the landscape architect and should be performed judiciously to preclude either soaking or excessive drying of the foundation soils. This should entail regular watering during the drier portions of the year and little or no irrigation during the rainy season. Automatic sprinkler systems should, therefore, be switched to manual operation during the rainy season. Good irrigation practice typically requires frequent application of limited quantities of water that are sufficient to sustain plant growth, but do not excessively wet the soils. Ponding and/or run-off of irrigation water are indications of excessive watering.

Other provisions, as determined by the landscape architect or civil engineer, may also be appropriate.

Moisture Sensitive Subgrade Soils

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

As discussed in Section 6.3 of this report, unstable subgrade soils will likely be encountered at the base of the overexcavation within the proposed building areas. The extent of unstable subgrade soils will to a large degree depend on methods used by the contractor to avoid adding additional moisture to these soils or disturbing soils which already possess high moisture contents. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. **If unstable subgrade conditions are encountered, it is recommended that only track-mounted vehicles be used for fill placement and compaction.**

Based on the moisture contents of the soils encountered at the boring locations, allowances should be made for costs and delays associated with drying the on-site soils or import of a less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad areas as well as the need for and or the thickness of the crushed stone stabilization layer, discussed in Section 6.3 of this report.

Groundwater

The static groundwater table is considered to exist at a depth greater than 50± feet below the existing site grades. 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 extending to depths of at least 3 feet below foundation bearing grades. Based on this subsurface profile, and based on the design considerations presented in Section 6.1 of this report, the proposed structure may be supported on conventional shallow foundations.

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

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

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

- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom), due to the potential for liquefaction-induced settlement, and the presence of medium expansive soils.

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

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

The allowable bearing pressures presented above may be increased by one-third when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements, as discussed in Section 6.1. 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 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 2 to 4 percent above the Modified Proctor (ASTM D-1557) optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively, under static conditions. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report.

These settlements are in addition to the liquefaction-induced settlements previously discussed in Section 6.1 of this report. However, the likelihood of these two settlements combining is considered remote. The static settlements are expected to occur in a relatively short period of time after the building loads being applied to the foundations, during and immediately subsequent to construction. It should be noted that the projected potential dynamic settlement is related to a major seismic event and a conservative historic high groundwater level.

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: 250 lbs/ft^3
- Friction Coefficient: 0.28

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 soils. The maximum allowable passive pressure is 2,500 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slabs should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, and based on the design considerations presented in Section 6.1 of this report, the floor of the proposed structure may be constructed as conventional slab-on-grade supported on newly-placed structural fill, extending to a depth of at least 3 feet below finished pad grade. Based on geotechnical considerations, the floor slabs may be designed as follows:

- Minimum slab thickness: 6 inches.

- Modulus of Subgrade Reaction: 80 psi/in.

- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to presence of potentially liquefiable and expansive soils. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading, and the potential liquefaction-induced settlements.

- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire slab area where such moisture sensitive floor coverings are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as 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 2 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 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for sidewalks, patios, and other concrete flatwork, should be prepared in accordance with the recommendations contained in the ***Grading Recommendations*** section of this report. As noted previously, flatwork supported on the existing medium expansive soils will be subject to minor to moderate amounts of movement as the moisture content within the subgrade soils fluctuates. **This movement may cause cracking or other distress within the flatwork.** If additional protection against flatwork cracking is desired, consideration should be given to the placement of a 2-foot-thick layer of very low expansive structural fill beneath all flatwork sections. Assuming that the flatwork is supported on the existing on-site soils, exterior slabs on grade may be designed as follows:

- Minimum slab thickness: 4½ inches.

- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions.

- The flatwork at building entry areas should be structurally connected to the perimeter foundation that spans across the door opening.

- Moisture condition the slab subgrade soils to at least 2 to 4 percent above optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 24 hours prior to concrete placement.

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

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

- Where flatwork is immediately adjacent to landscape planters, a thickened edge should be utilized. This edge should extend to a depth of at least 12 inches and incorporate longitudinal reinforcement consisting of at least two No. 4 bars.
- Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

6.8 Retaining Wall Design and Construction

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

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. The near-surface soils generally consist of silty clays, clayey silts, sandy clays, as well as silty sands, clayey sands, and sandy silts. The medium expansive silty clays, clayey silts and sandy clays should not be used as retaining wall backfill. Based on their composition, the on-site soils consisting of silty sands, clayey sands and sandy silts have been assigned a friction angle of 28 degrees. On-site silty clays and clayey silts are likely to possess higher expansion potentials and lower strengths and should be not be used as retaining wall backfill.

If desired, SCG can 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** Silty Sands, Clayey Sands, and Sandy Silts
Internal Friction Angle (ϕ)		28°
Unit Weight		125 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	45 lbs/ft^3
	Active Condition (2h:1v backfill)	79 lbs/ft^3
	At-Rest Condition (level backfill)	66 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 250 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

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

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

Seismic Lateral Earth Pressures

In accordance with the 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.

Retaining Wall Foundation Design

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

Backfill Material

On-site soils may be used to backfill the retaining walls, provided that they are low expansive (EI less than 50). All backfill material placed within 3 feet of the back-wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

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

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

Subsurface Drainage

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

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

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

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

6.9 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 clays and sandy clays. These soils are generally considered to possess poor pavement support characteristics with estimated R-values of 5 to 15. R-value testing was outside the scope of services. The subsequent pavement design is therefore based upon an assumed R-value of 5. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions. It is recommended that R-value testing be performed after completion of rough grading. Depending upon the results of the R-value testing, it may be feasible to use thinner pavement sections in some areas of the site.

Asphaltic Concrete

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

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

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

ASPHALTIC CONCRETE PAVEMENTS (R=5)					
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	10	13	16	18	21
Compacted Subgrade	12	12	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R=5)				
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	7	8½
Compacted Subgrade (95% minimum compaction)	12	12	12	12

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

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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.

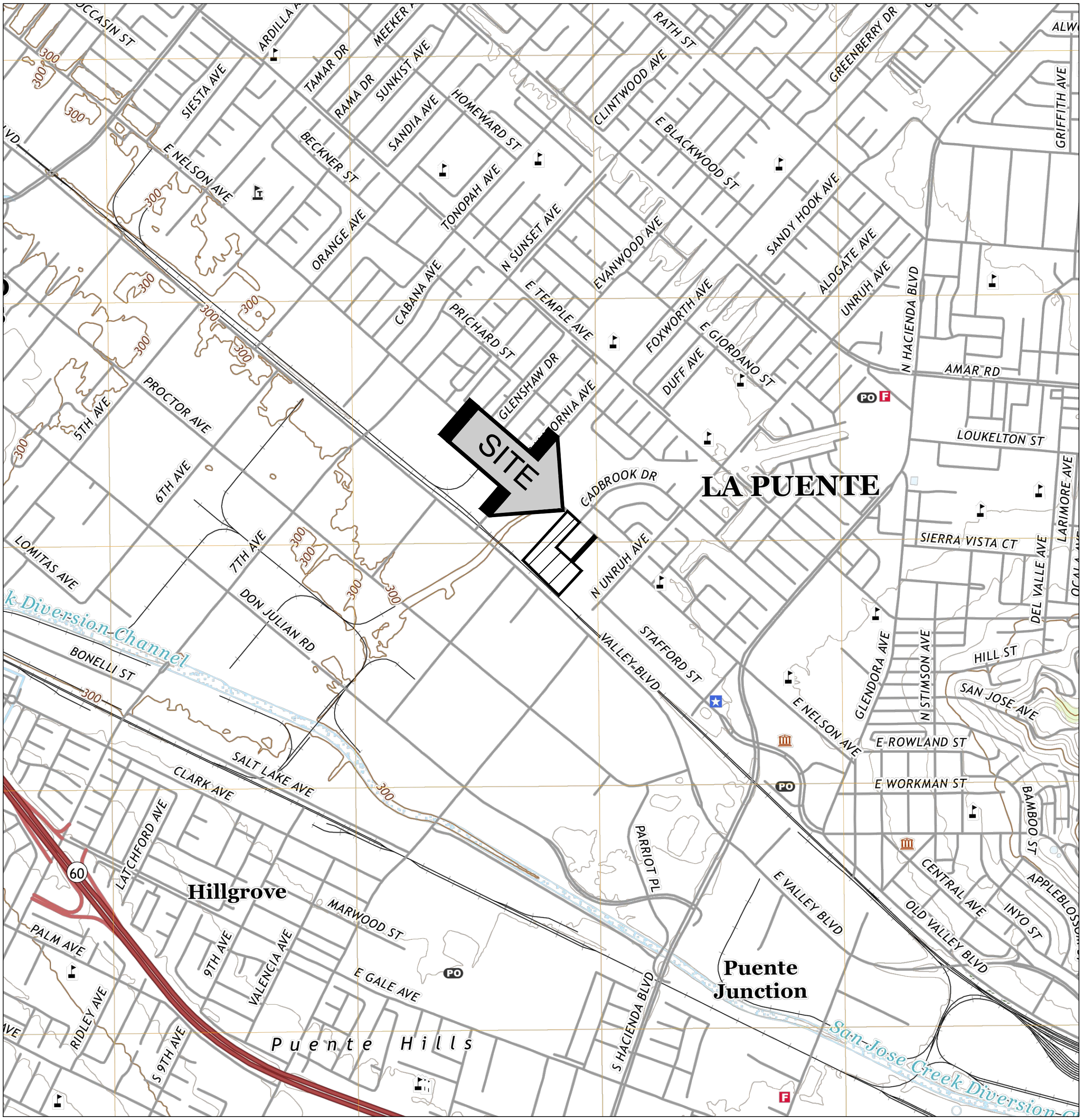
Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

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 MAP OF THE BALDWIN PARK, LOS ANGELES COUNTY, CALIFORNIA, 2018.

SITE LOCATION MAP

PROPOSED INDUSTRIAL DEVELOPMENT

CITY OF INDUSTRY, CALIFORNIA

SCALE: 1" = 2000'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: MD CHKD: RGT	
SCG PROJECT 21G265-1R	
PLATE 1	

GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

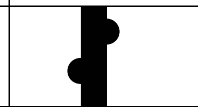
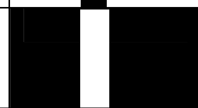
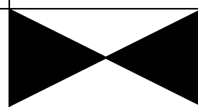
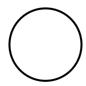
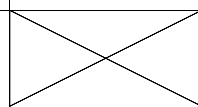
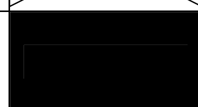
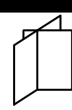
APPROXIMATE CPT LOCATION

NOTE: SITE PLAN PROVIDED BY THE CLIENT.
AIR PHOTO OBTAINED FROM GOOGLE EARTH.

BORING AND CPT LOCATION PLAN	
PROPOSED INDUSTRIAL BUILDING	
CITY OF INDUSTRY, CALIFORNIA	
SCALE: 1" = 100'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAH CHKD: RGT	
SCG PROJECT 21G265-1R	
PLATE 2	

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

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

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		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

BORING NO. B-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G265-1R
PROJECT: Proposed Industrial Building
LOCATION: Industry, California

DRILLING DATE: 11/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 42 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					6± inches Portland Cement Concrete; No Steel Reinforcement							
		24	4.5		FILL: Dark Brown Silty Clay, little fine Sand, trace Calcareous nodules and veining, very stiff-very moist	106	18					EI = 75 @ 0 to 5'
		15	4.5		ALLUVIUM: Brown fine Sandy Clay, little Silt, trace medium Sand, trace Calcareous nodules and veining, stiff to very stiff-very moist	107	17					
5		14	4.5			111	17					
		15	4..5			109	16					
					Gray Brown fine Sandy Silt, little Iron oxide staining, trace Calcareous nodules and veining, loose to medium dense-very moist							
		12				107	17			66		
10			4.5		Gray Clayey Silt, little Iron oxide staining, trace Calcareous nodules and veining, stiff to very stiff-very moist							
					Brown Clayey fine Sand to fine Sandy Clay, little medium to coarse Sand, little Iron oxide staining, trace Calcareous nodules and veining, medium dense to stiff-moist to very moist							
		11	4.0				14			48		
15												
		10	3.0				16			51		
20												
					Light Brown fine Sand, trace medium to coarse Sand, trace fine Gravel, dense-dry to damp							
		33					2					
25												
					Light Brown fine to coarse Sand, little fine to coarse Gravel, dense to very dense-damp							
		42					3					
30												
		52					3					

TBL 21G265-1R.GPJ SOCALGEO.GDT 1/31/23

BORING NO. B-1

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G265-1R
PROJECT: Proposed Industrial Building
LOCATION: Industry, California

DRILLING DATE: 11/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 42 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Brown fine to coarse Sand, little fine to coarse Gravel, dense to very dense-damp							
					Light Gray Brown Silty Clay, little Iron oxide staining, trace Calcareous nodules and veining, stiff-very moist							
	X	14	2.5				26	47	22	69		
40												
					Gray Brown fine Sandy Silt, little Iron oxide staining, medium dense to dense-very moist							
	X	26					17			47		
45												
	X	32					14					
50					Boring Terminated at 50'							

TBL 21G265-1R.GPJ SOCALGEO.GDT 1/31/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 21G265-1R
PROJECT: Proposed Industrial Building
LOCATION: Industry, California

DRILLING DATE: 11/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 14 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
					6± inches Portland Cement Concrete; No Steel Reinforcement							
	X	7	3.5		FILL: Dark Brown to Black Silty Clay to Clayey Silt, little fine Sand, medium stiff-very moist		20					
	X	11	4.5		ALLUVIUM: Brown fine Sandy Clay, little Silt, trace medium Sand, trace Iron oxide staining, trace Calcareous nodules and veining, stiff-very moist		20					
5												
	X	9			Brown Silty fine Sand to fine Sandy Silt, little Iron oxide staining, little Calcareous nodules and veining, loose to medium dense-moist to very moist		11					
	X	15					22					
10												
					Dark Brown to Black Silty Clay, little fine to medium Sand, stiff-very moist							
	X	10	3.0				20					
15												

Boring Terminated at 15'

TBL 21G265-1R.GPJ SOCALGEO.GDT 1/31/23

TEST BORING LOG **PLATE B-2**

BORING NO. B-3

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G265-1R
PROJECT: Proposed Industrial Building
LOCATION: Industry, California

DRILLING DATE: 11/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 48 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					6± inches Portland Cement Concrete; No Steel Reinforcement							
			4.5		ALLUVIUM: Brown fine Sandy Clay, little Silt, little medium Sand, medium stiff to very stiff-very moist	103	18					EI = 57 @ 0 to 5'
		11	4.5			107	17					
5		13			Brown Silty fine Sand to fine Sand Silt, trace to little Clay, trace fine Gravel, trace Calcareous nodules and veining, loose-moist to very moist	105	10					
		13				113	12					
10		9				110	11			51		
					Gray Brown Silty fine Sand, trace medium to coarse Sand, trace fine Gravel, loose-very moist							
15		12				108	16			44		
					Light Brown to Brown fine to coarse Sand, little fine Gravel, trace Silt, medium dense to dense-dry to damp							
20		33					2			2		Disturbed Sample
25		33					2					
					Light Gray Brown Silty Clay, little fine Sand, little Iron oxide staining, some Calcareous nodules and veining, stiff-very moist							
30		12	4.5				21	46	18	74		
		11	3.0				28			87		

TBL 21G265-1R.GPJ SOCALGEO.GDT 1/31/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 21G265-1R
PROJECT: Proposed Industrial Building
LOCATION: Industry, California

DRILLING DATE: 11/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 48 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Gray Brown Silty Clay, little fine Sand, little Iron oxide staining, some Calcareous nodules and veining, stiff-very moist							
					Gray Brown fine Sandy Silt, trace Clay, little Iron oxide staining, some Calcareous nodules and veining, medium dense-very moist							
	X	21					17			50		
40												
	X	20					18			58		
45												
					Gray Brown fine Sand, trace to little Silt, little Iron oxide staining, dense-moist							
	X	30					8					
50												
					Boring Terminated at 50'							

TBL 21G265-1R.GPJ SOCALGEO.GDT 1/31/23

BORING NO. B-4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 21G265-1R
PROJECT: Proposed Industrial Building
LOCATION: Industry, California

DRILLING DATE: 11/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 32 feet
READING TAKEN: At Completion

SURFACE ELEVATION: MSL

DEPTH (FEET)	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
			3± inches Asphaltic concrete; 2± inches Aggregate Base							
	11	4.0	ALLUVIUM: Dark Gray Brown to Black Silty Clay, little fine Sand, trace Calcareous nodules and veining, medium stiff to very stiff-very moist	91	22					
	14	4.5		98	24					
5	12	4.5	Gray Brown fine Sandy Clay, little medium Sand, trace fine Gravel, trace Calcareous nodules and veining, little Iron oxide staining, medium stiff to very stiff-very moist	103	18					
	10		Gray Brown Silty fine Sand, trace Calcareous nodules and veining, little Iron oxide staining, loose-moist to very moist	114	13					
10	11			115	13			40		
	16	2.5	Gray Brown Clayey Sand to fine Sandy Clay, trace to little Silt, little Iron oxide staining, medium dense to stiff-very moist		17			50		
15										
20	30		Gray Brown fine to coarse Sand, trace to little Silt, little fine to coarse Gravel, dense-damp to moist		11					
25	38				3					
30	13	4.5	Gray Brown fine Sandy Clay, little Silt, trace Iron oxide staining, stiff-moist to very moist		17	25	14	56		
	38		Gray Brown fine Sand, little medium to coarse Sand, dense-damp		4					

TBL 21G265-1R.GPJ SOCALGEO.GDT 1/31/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-4

JOB NO.: 21G265-1R
PROJECT: Proposed Industrial Building
LOCATION: Industry, California

DRILLING DATE: 11/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Jamie Hayward

WATER DEPTH: Dry
CAVE DEPTH: 32 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Gray Brown fine Sand, little medium to coarse Sand, dense-damp							
					Gray Brown Silty fine Sand, little Iron oxide staining, medium dense-very moist							
	X	23					22			76		
40												
					Gray Brown fine Sandy Silt, little Calcareous nodules and veining, little Iron oxide staining, medium dense-very moist							
	X	26					20					
45												
					Gray Brown fine Sandy Clay, little Silt, stiff to very stiff-very moist							
	X	15	3.0				23			60		
50												
					Boring Terminated at 50'							

TBL 21G265-1R.GPJ SOCALGEO.GDT 1/31/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 21G265-1R
PROJECT: Proposed Industrial Building
LOCATION: Industry, California

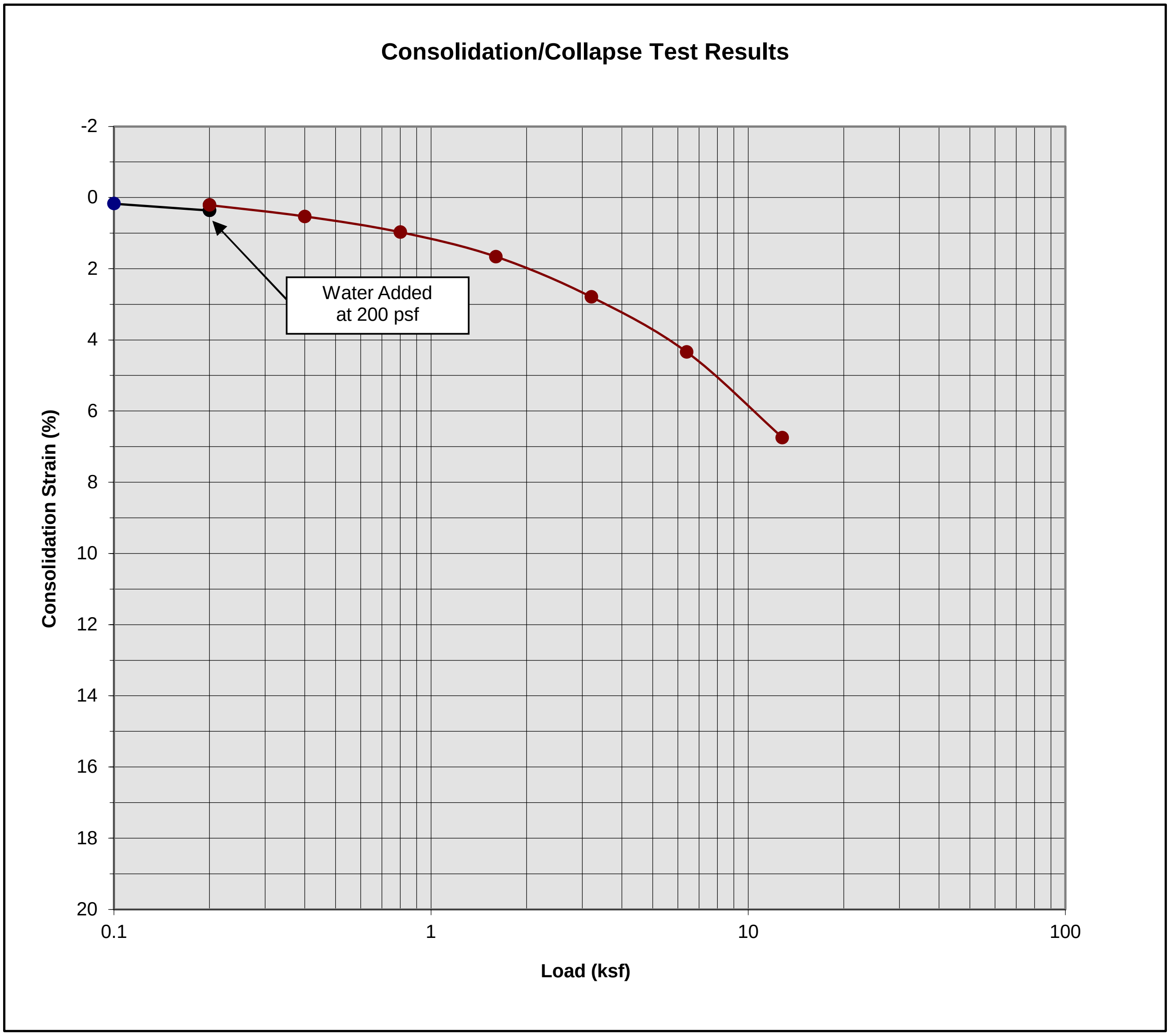
DRILLING DATE: 11/4/21
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Joseph Lozano Leon

WATER DEPTH: Dry
CAVE DEPTH: 14 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					6± inches Portland Cement Concrete; No Steel Reinforcement							
	X	9	4.5		FILL: Black Silty Clay, little fine to coarse Sand, slight hydrocarbon odor, stiff to very stiff-very moist		18					
	X	10	4.5		ALLUVIUM: Brown Silty Clay, trace medium Sand, trace Calcareous nodules and veining, stiff to very stiff-very moist		17					
5												
	X	11	3.5				18					
	X	10			Brown fine Sandy Silt, trace medium to coarse Sand, trace Clay, little Iron oxide staining, medium dense-very moist		15					
10												
	X	11					25					
15												
					Gray Brown Silty fine Sand, little to some Iron oxide staining, medium dense-moist to very moist							
	X	11					14					
20					Boring Terminated at 20'							

TBL 21G265-1R.GPJ SOCALGEO.GDT 1/31/23

APPENDIX C

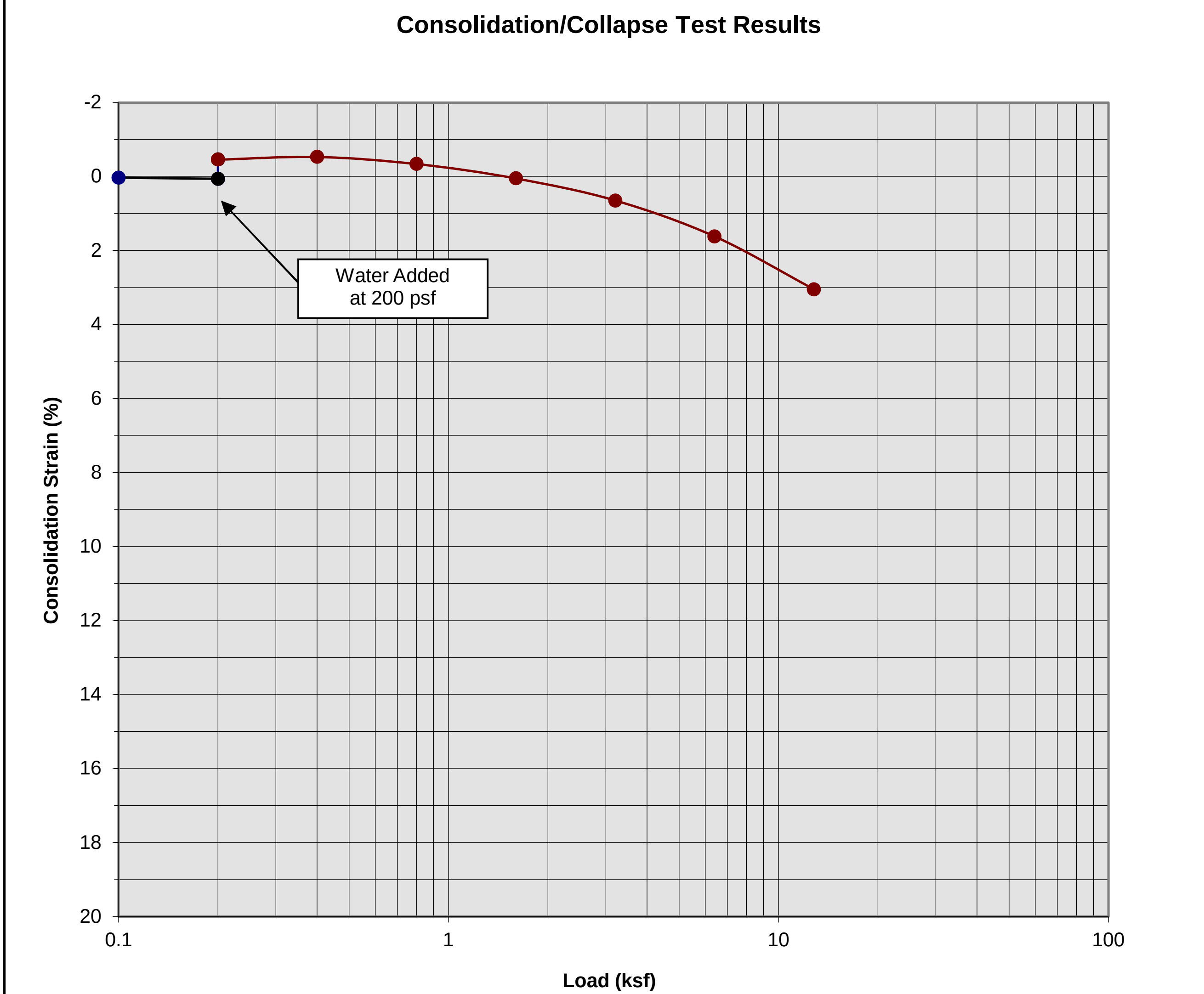


Classification: Brown fine Sandy Clay, little Silt, trace medium Sand

Boring Number:	B-1	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	3 to 4	Initial Dry Density (pcf)	107.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.15

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C- 1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

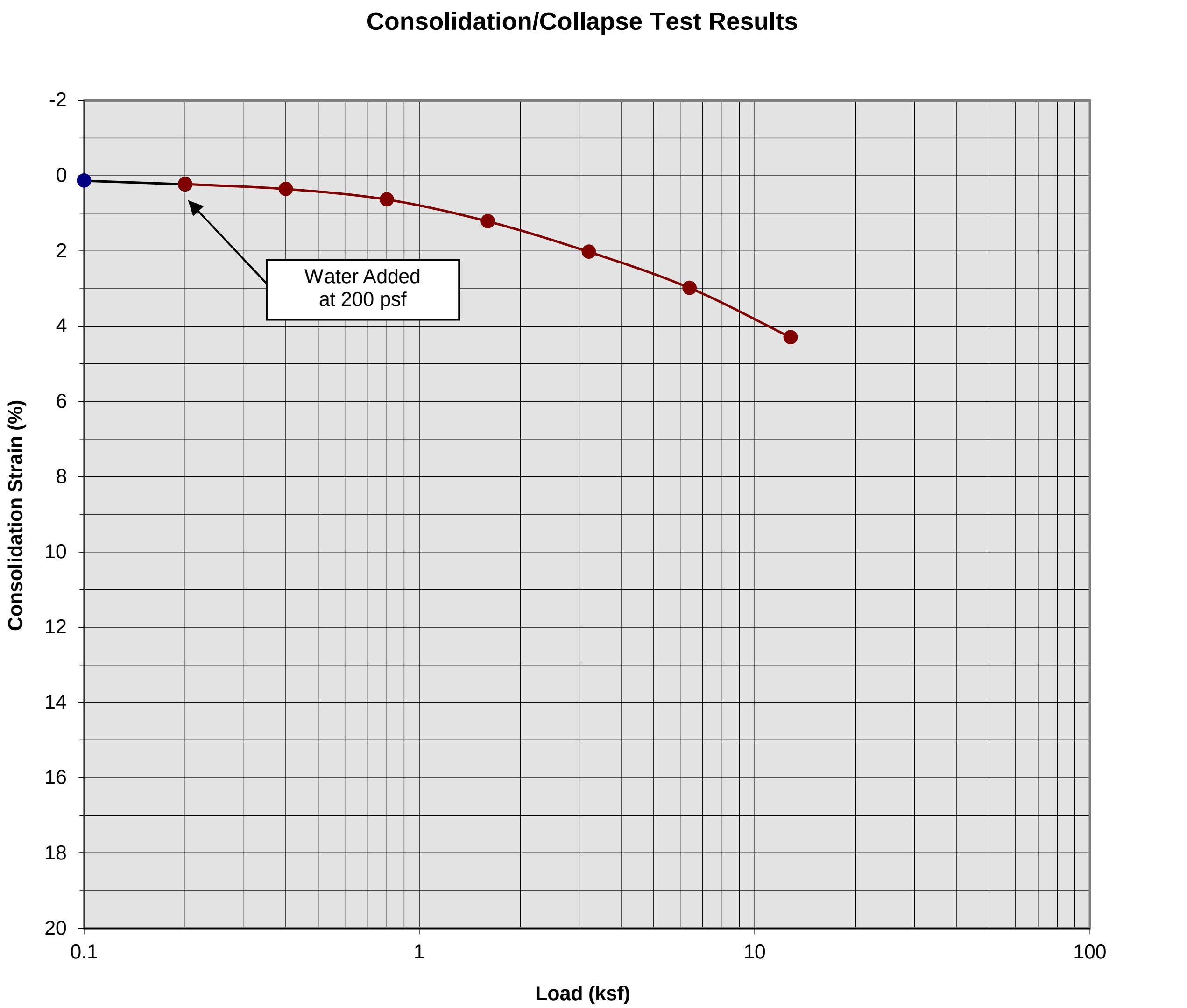


Classification: Brown fine Sandy Clay, little Silt, trace medium Sand

Boring Number:	B-1	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	5 to 6	Initial Dry Density (pcf)	111.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.52

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

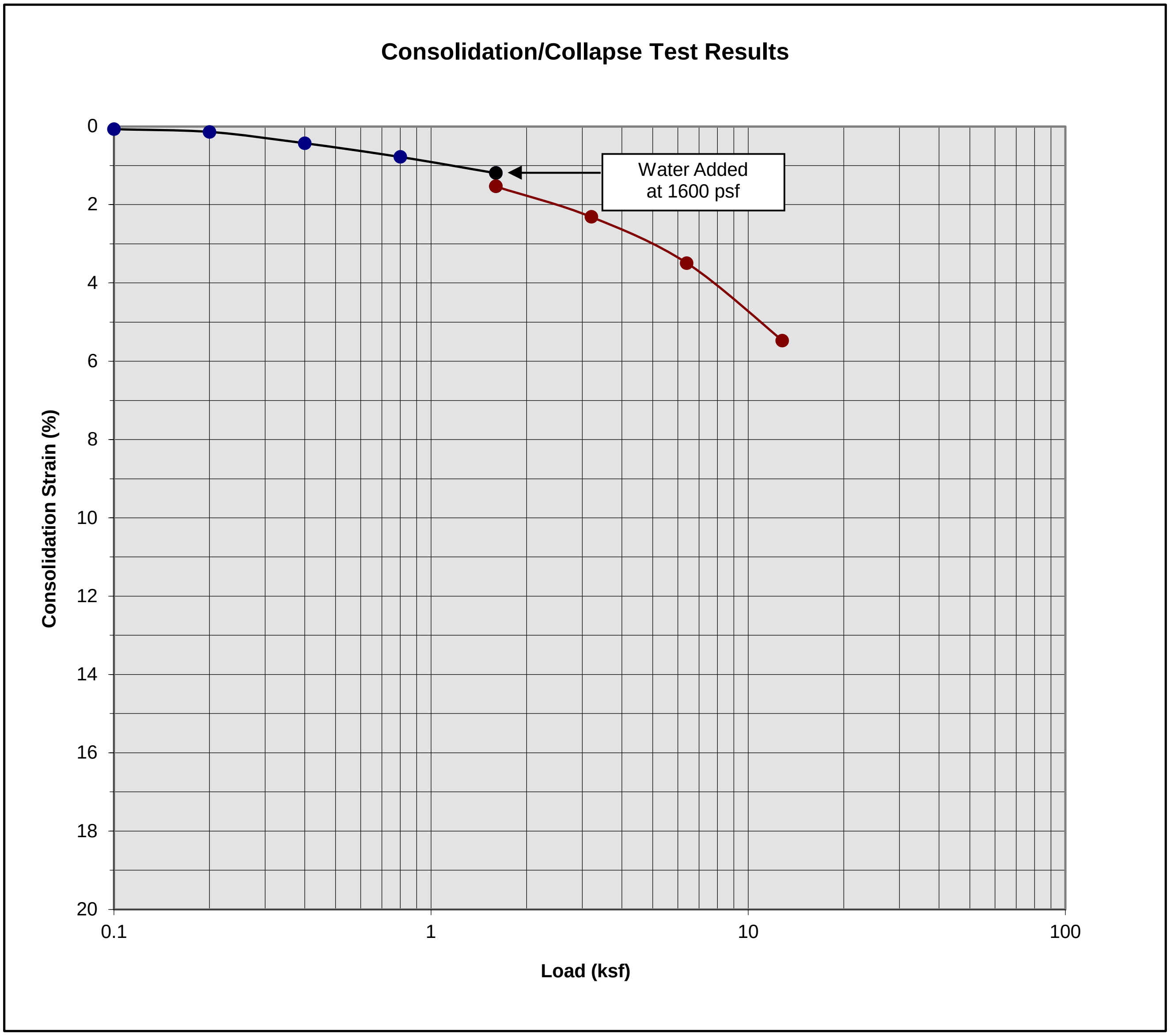


Classification: Brown fine Sandy Clay, little Silt, trace medium Sand

Boring Number:	B-1	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	12
Depth (ft)	7 to 8	Initial Dry Density (pcf)	109.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.01

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



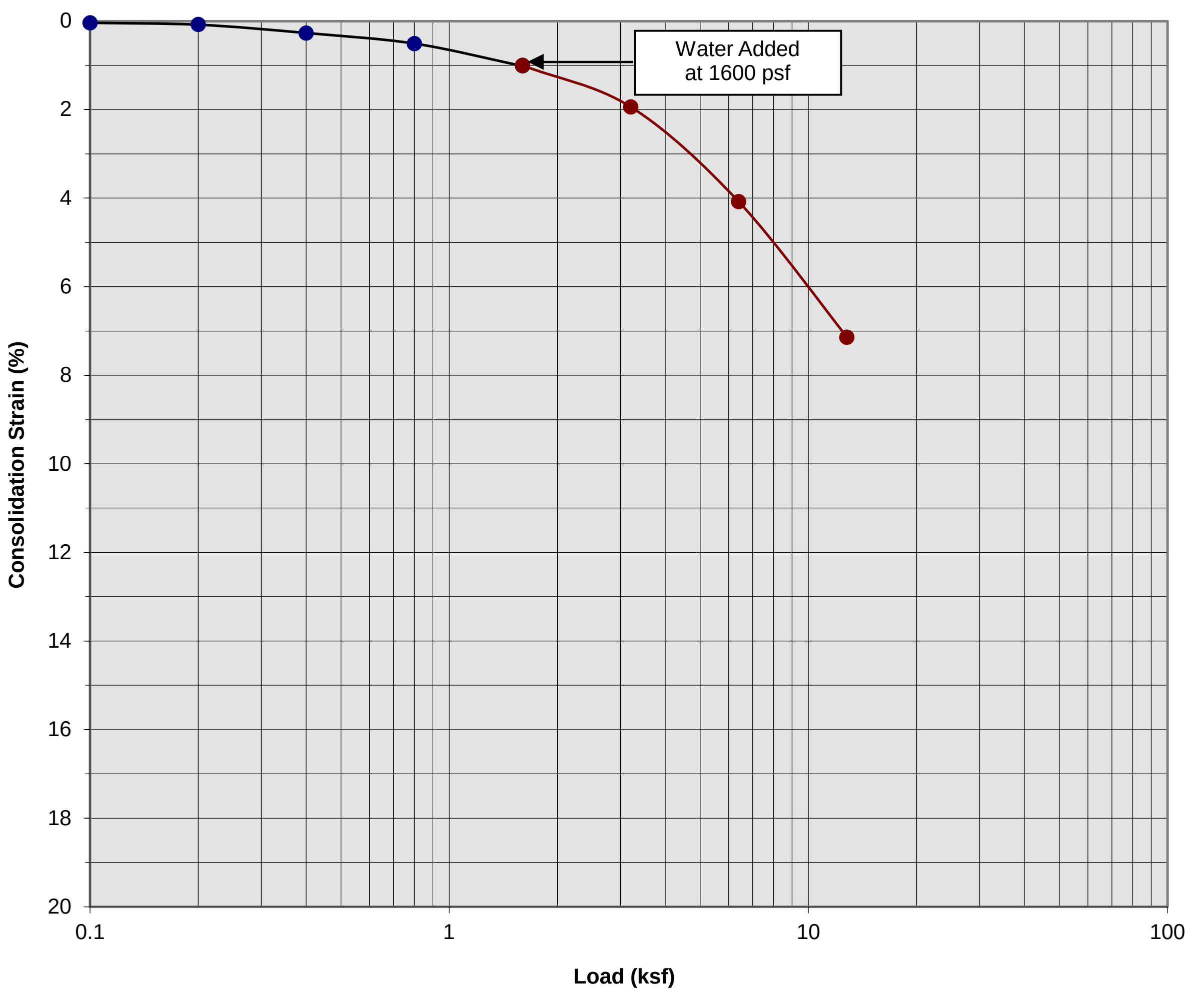
Classification: Gray Brown fine Sandy Silt

Boring Number:	B-1	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	9 to 10	Initial Dry Density (pcf)	107.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.34

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C- 4

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

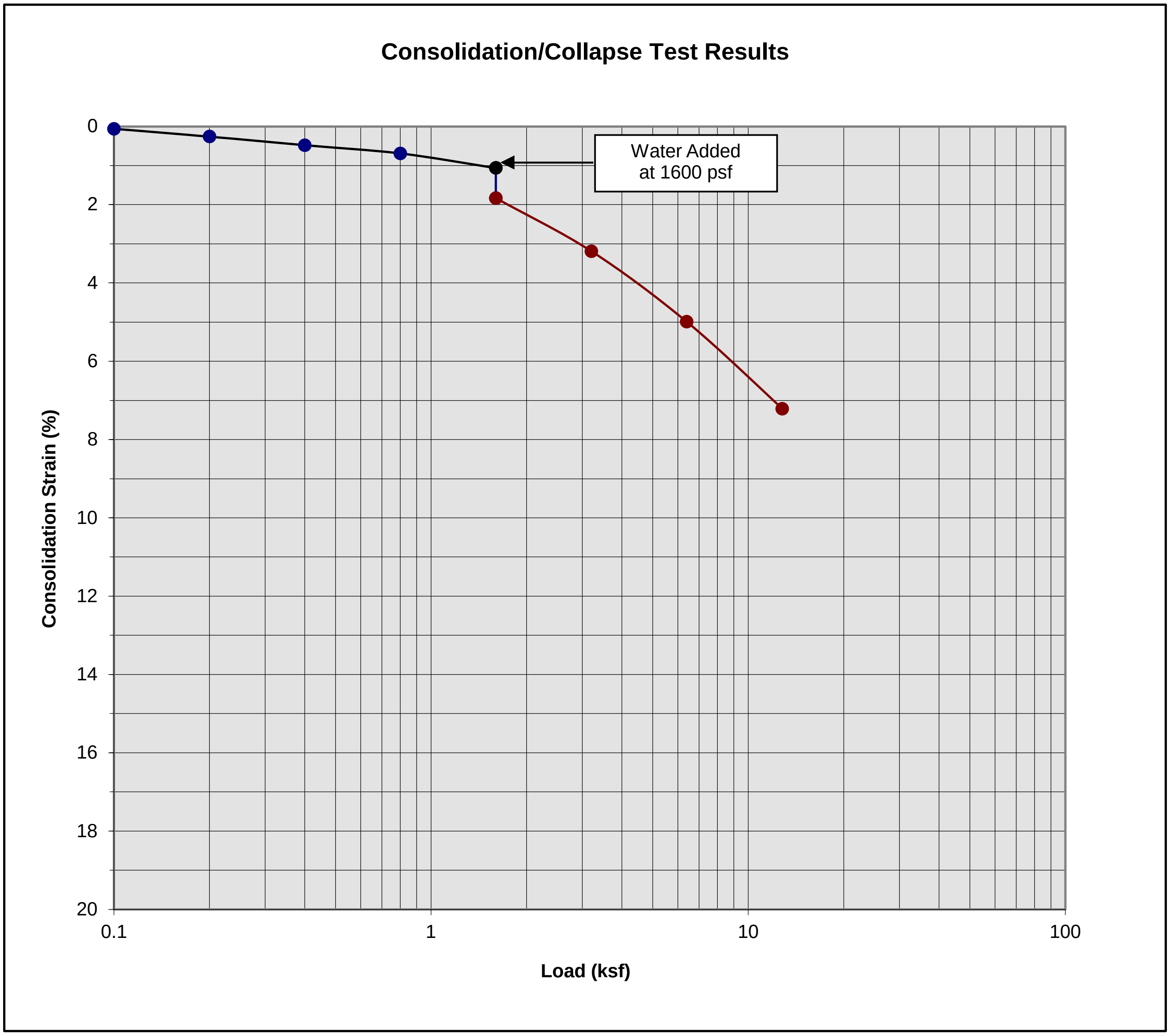


Classification: Brown fine Sandy Clay, little Silt, little medium Sand

Boring Number:	B-3	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	3 to 4	Initial Dry Density (pcf)	107.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.01

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C-5

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



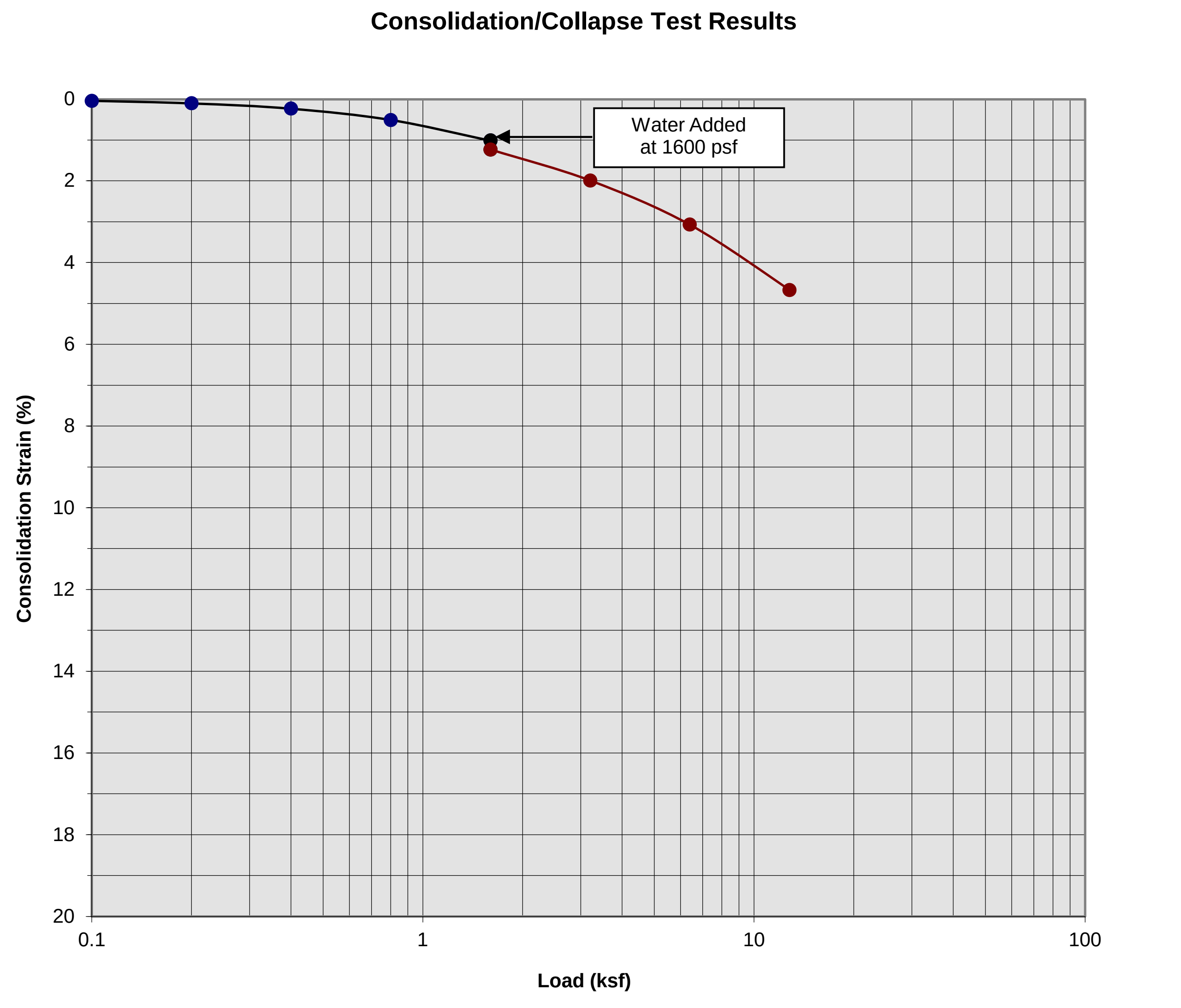
Classification: Brown Silty fine Sand to fine Sandy Silt, trace to little Clay, trace fine Gravel

Boring Number:	B-3	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	5 to 6	Initial Dry Density (pcf)	105.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.77

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

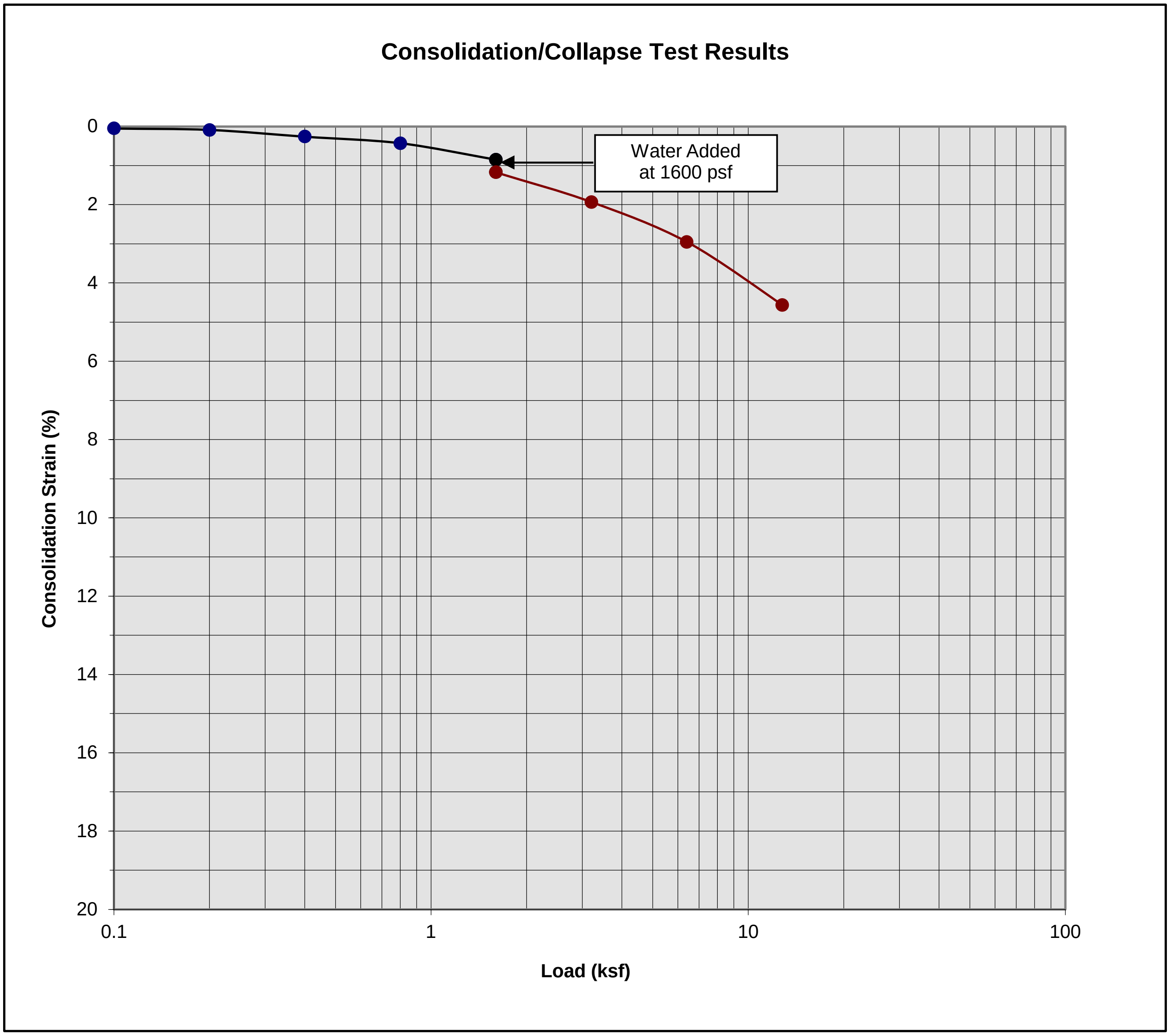


Classification: Brown Silty fine Sand to fine Sandy Silt, trace to little Clay, trace fine Gravel

Boring Number:	B-3	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	7 to 8	Initial Dry Density (pcf)	113.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.22

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL A California Corporation

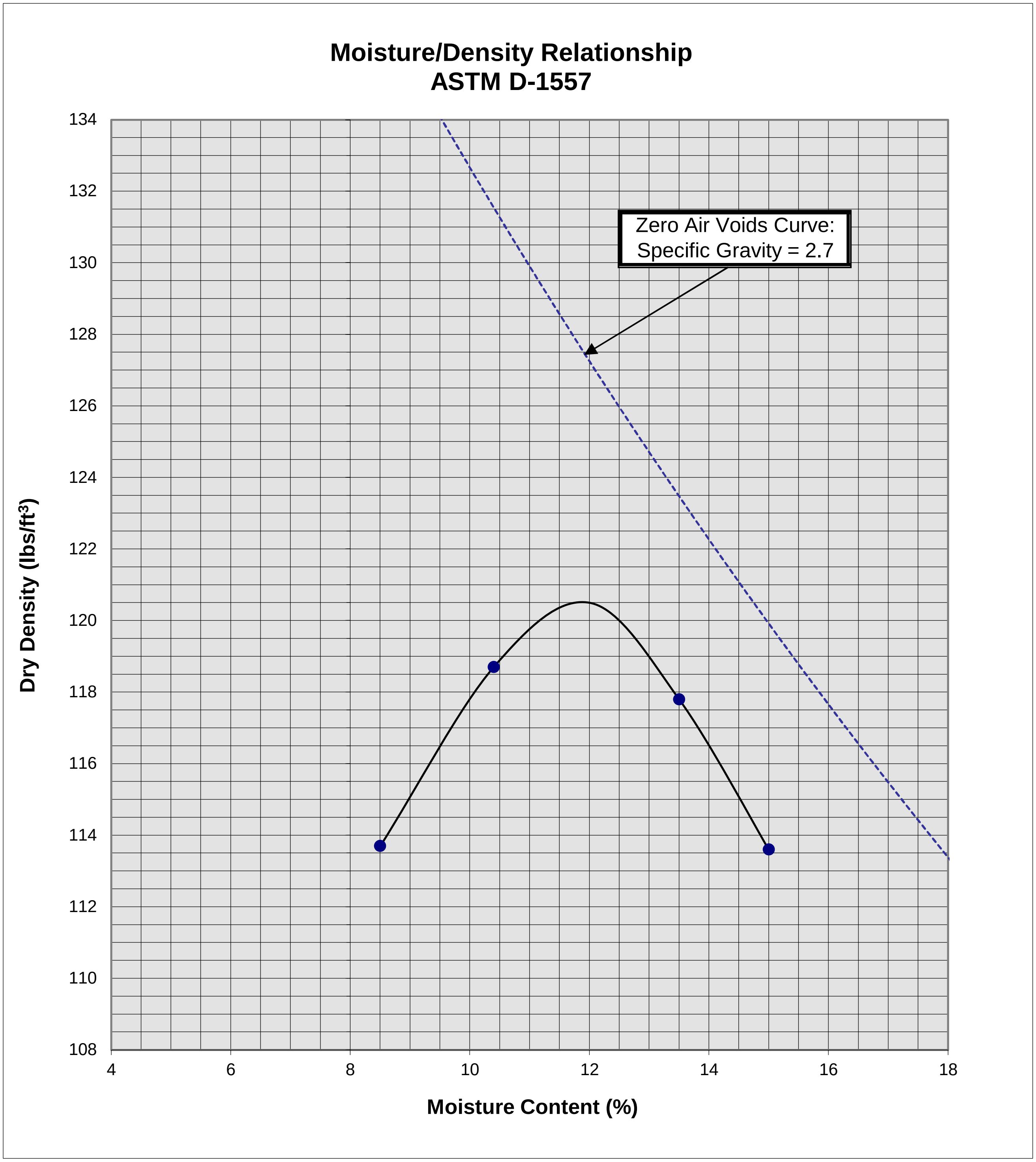


Classification: Brown Silty fine Sand to fine Sandy Silt, trace to little Clay, trace fine Gravel

Boring Number:	B-3	Initial Moisture Content (%)	11
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	9 to 10	Initial Dry Density (pcf)	110.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.32

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C-8

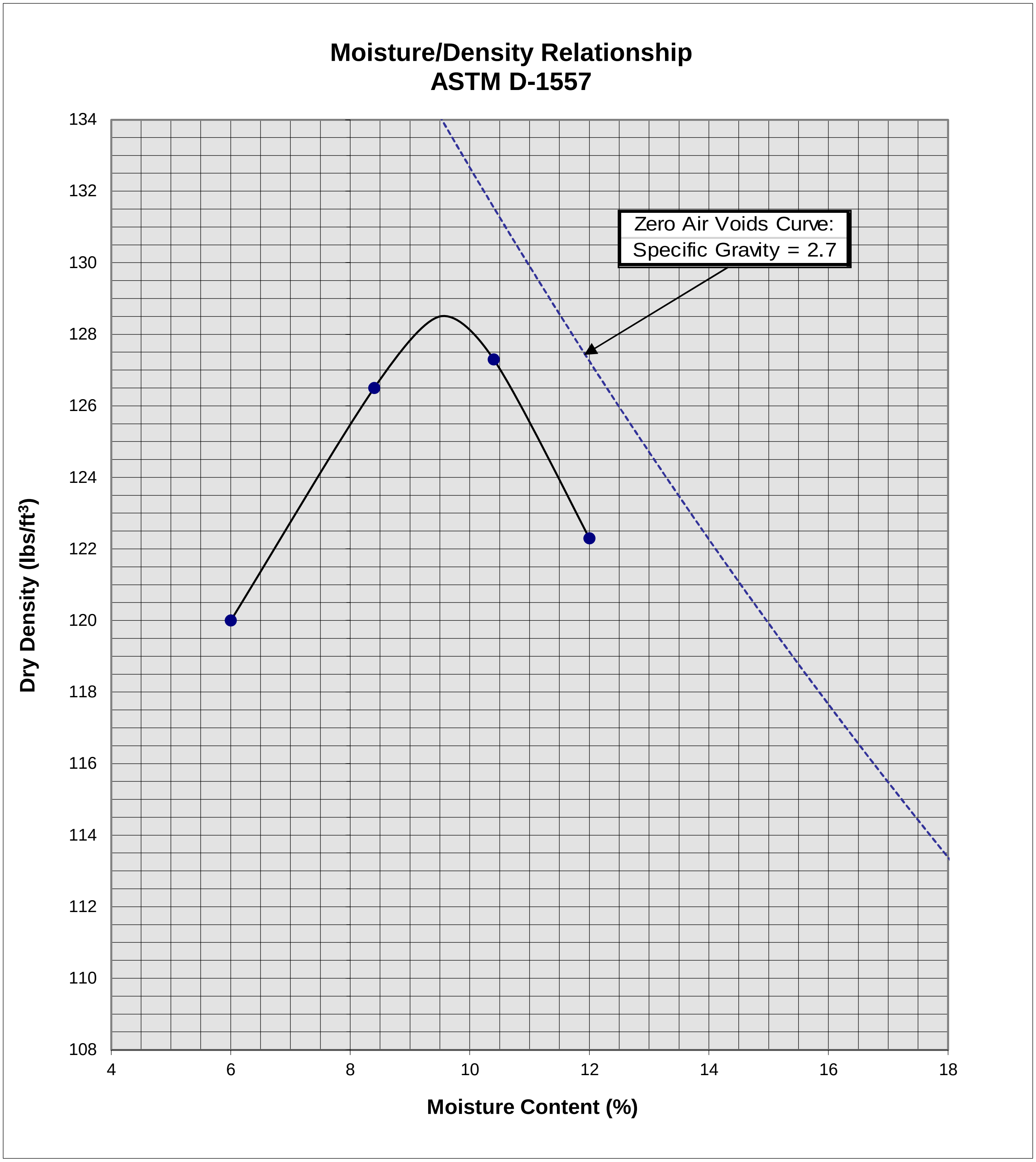
SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-1 @ 0-5 feet
Optimum Moisture (%)	12
Maximum Dry Density (pcf)	120.5
Soil Classification	Dark Brown Silty Clay, little fine Sand

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C-9

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-3 @ 0-5 feet
Optimum Moisture (%)	9.5
Maximum Dry Density (pcf)	128.5
Soil Classification	Brown fine Sandy Clay, little Silt, little medium Sand

Proposed Industrial Building
Industry, California
Project No. 21G265-1R
PLATE C-10

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

- The Earthwork Contractor is responsible for the satisfactory completion of all earthwork in accordance with the plans and geotechnical reports, and in accordance with city, county, and applicable building codes.
- The Geotechnical Engineer is the representative of the Owner/Builder for the purpose of implementing the report recommendations and guidelines. These duties are not intended to relieve the Earthwork Contractor of any responsibility to perform in a workman-like manner, nor is the Geotechnical Engineer to direct the grading equipment or personnel employed by the Contractor.
- The Earthwork Contractor is required to notify the Geotechnical Engineer of the anticipated work and schedule so that testing and inspections can be provided. If necessary, work may be stopped and redone if personnel have not been scheduled in advance.
- The Earthwork Contractor is required to have suitable and sufficient equipment on the job-site to process, moisture condition, mix and compact the amount of fill being placed to the approved compaction. In addition, suitable support equipment should be available to conform with recommendations and guidelines in this report.
- Canyon cleanouts, overexcavation areas, processed ground to receive fill, key excavations, subdrains and benches should be observed by the Geotechnical Engineer prior to placement of any fill. It is the Earthwork Contractor's responsibility to notify the Geotechnical Engineer of areas that are ready for inspection.
- Excavation, filling, and subgrade preparation should be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs, and seepage water encountered shall be pumped or drained to provide a suitable working surface. The Geotechnical Engineer must be informed of springs or water seepage encountered during grading or foundation construction for possible revision to the recommended construction procedures and/or installation of subdrains.

Site Preparation

- The Earthwork Contractor is responsible for all clearing, grubbing, stripping and site preparation for the project in accordance with the recommendations of the Geotechnical Engineer.
- If any materials or areas are encountered by the Earthwork Contractor which are suspected of having toxic or environmentally sensitive contamination, the Geotechnical Engineer and Owner/Builder should be notified immediately.

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

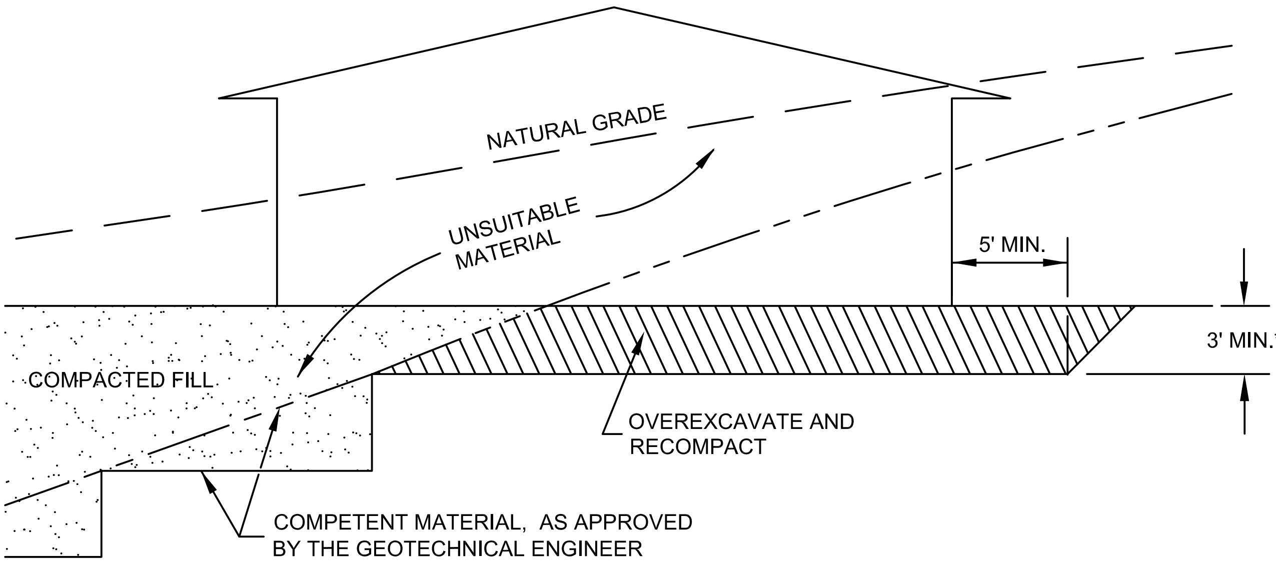
Subdrains

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

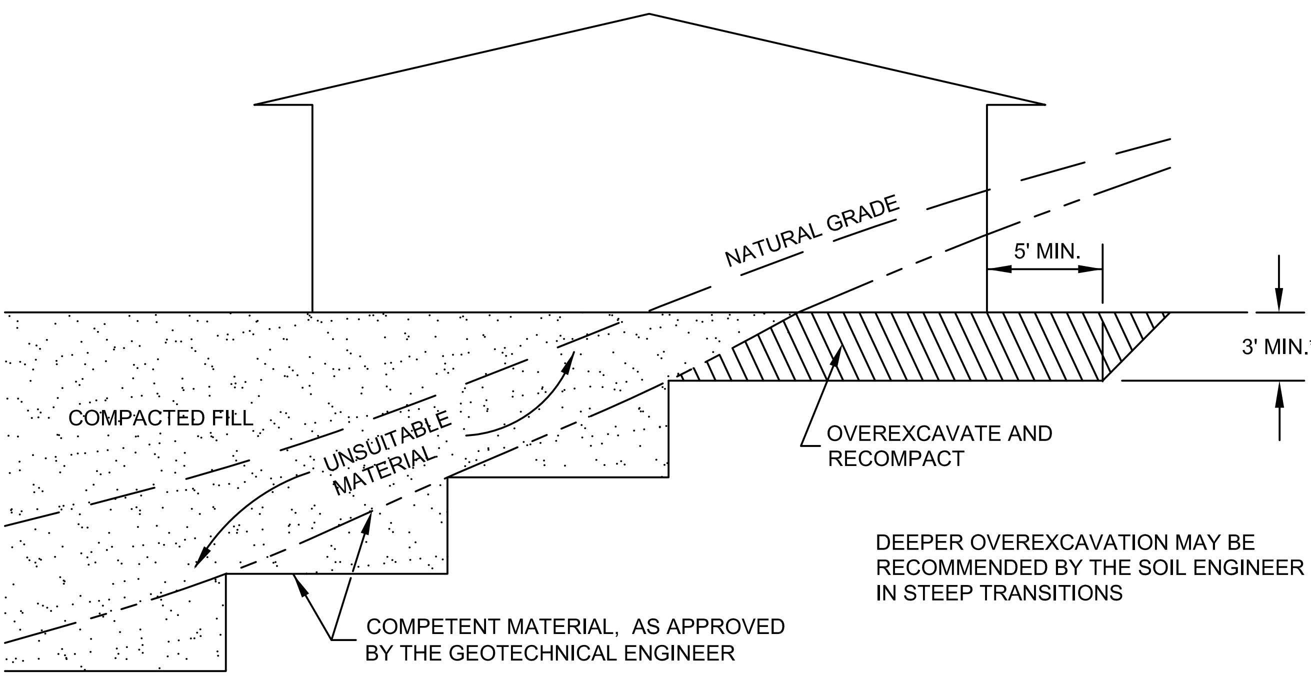
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



CUT/FILL LOT (TRANSITION)



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

TRANSITION LOT DETAIL

GRADING GUIDE SPECIFICATIONS

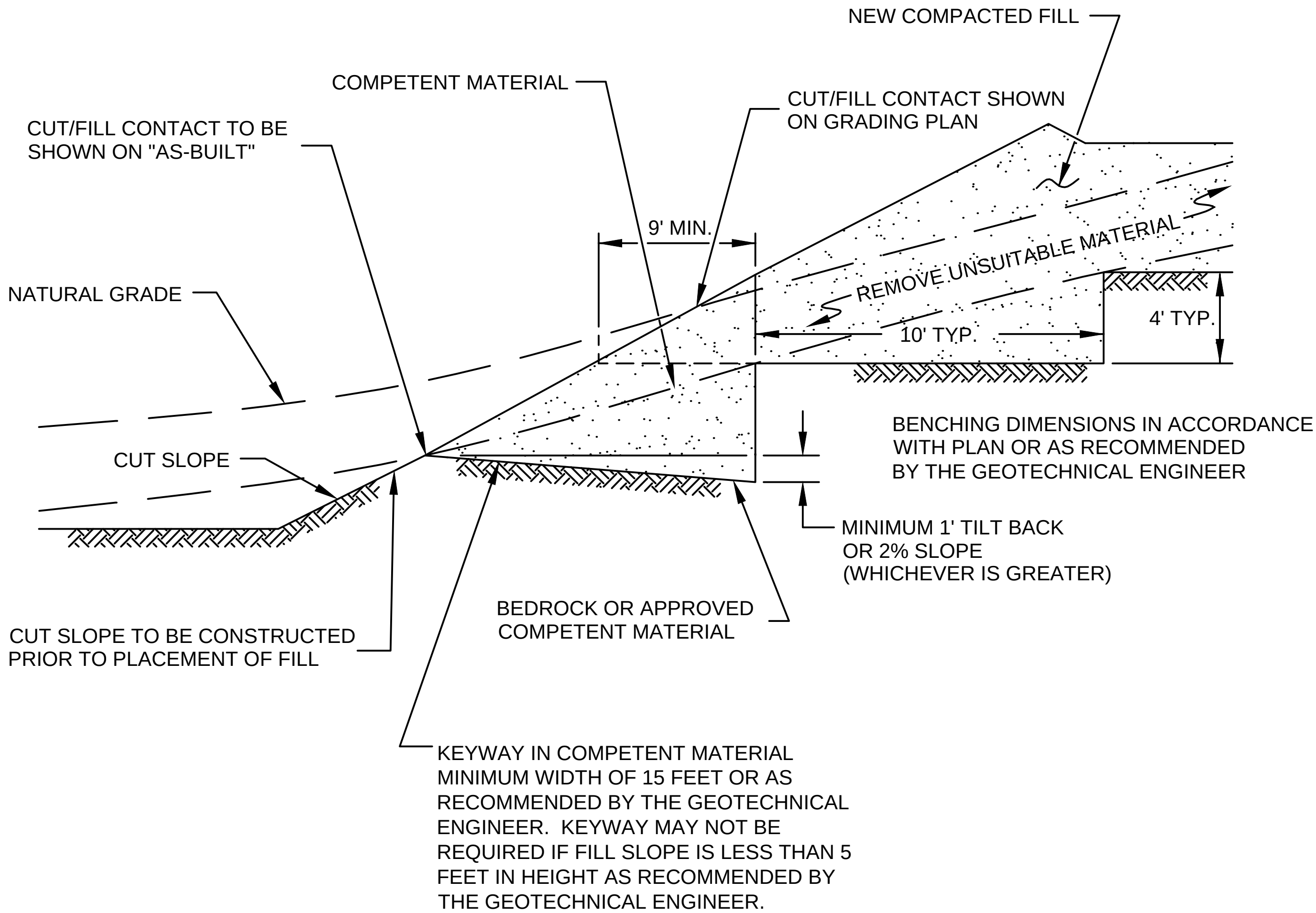
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

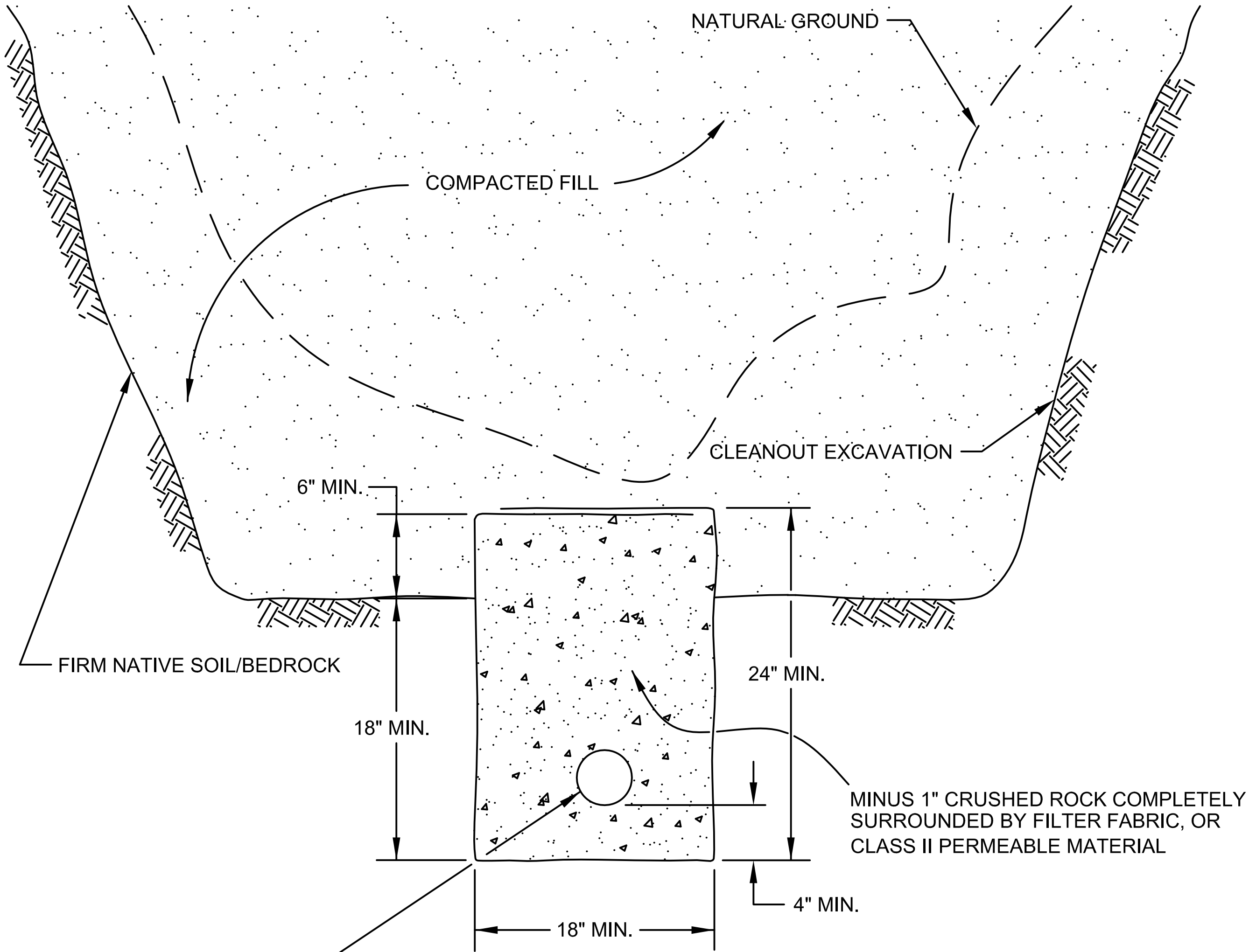
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

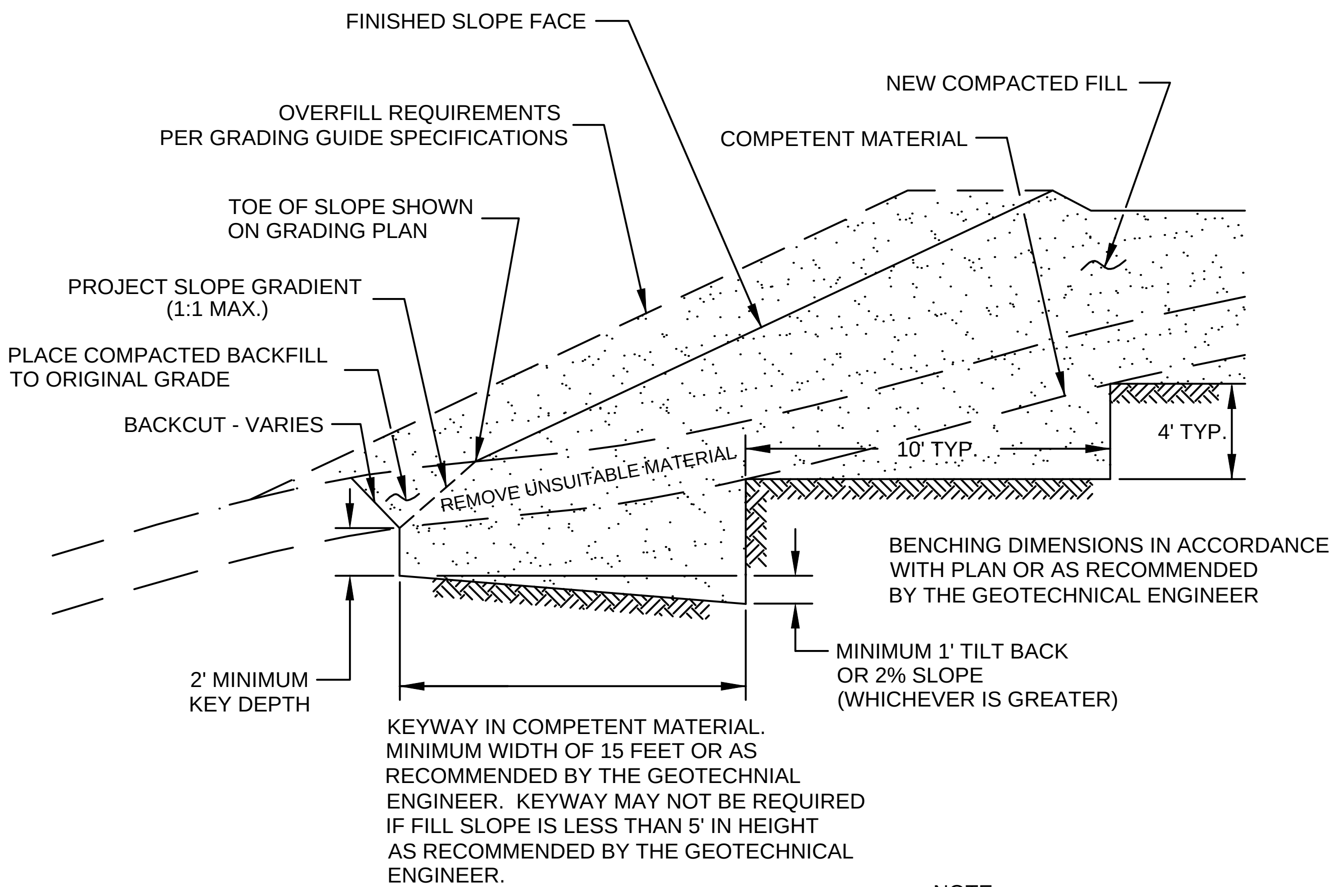
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

FILL ABOVE NATURAL SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

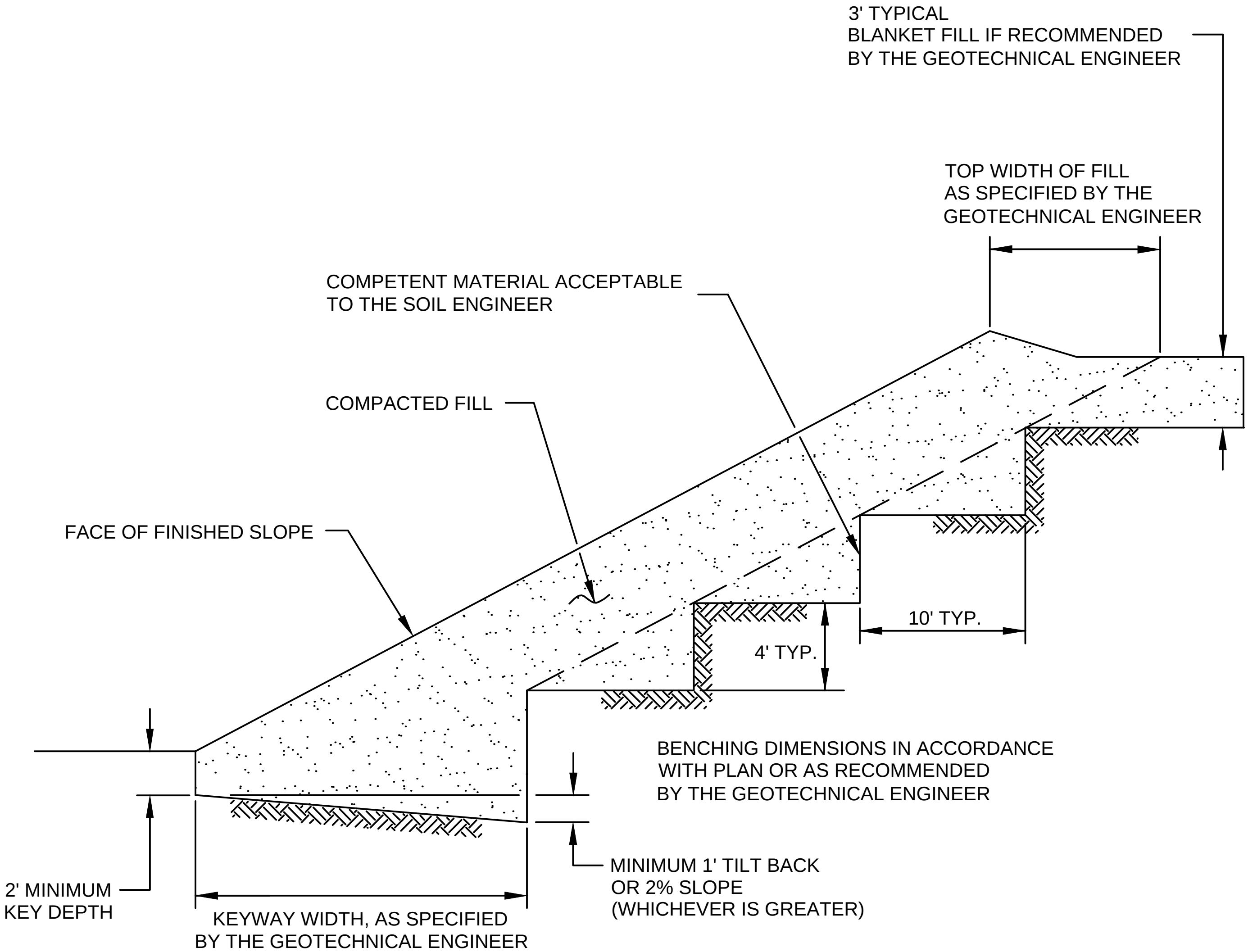
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN
CALIFORNIA
GEOTECHNICAL



STABILIZATION FILL DETAIL

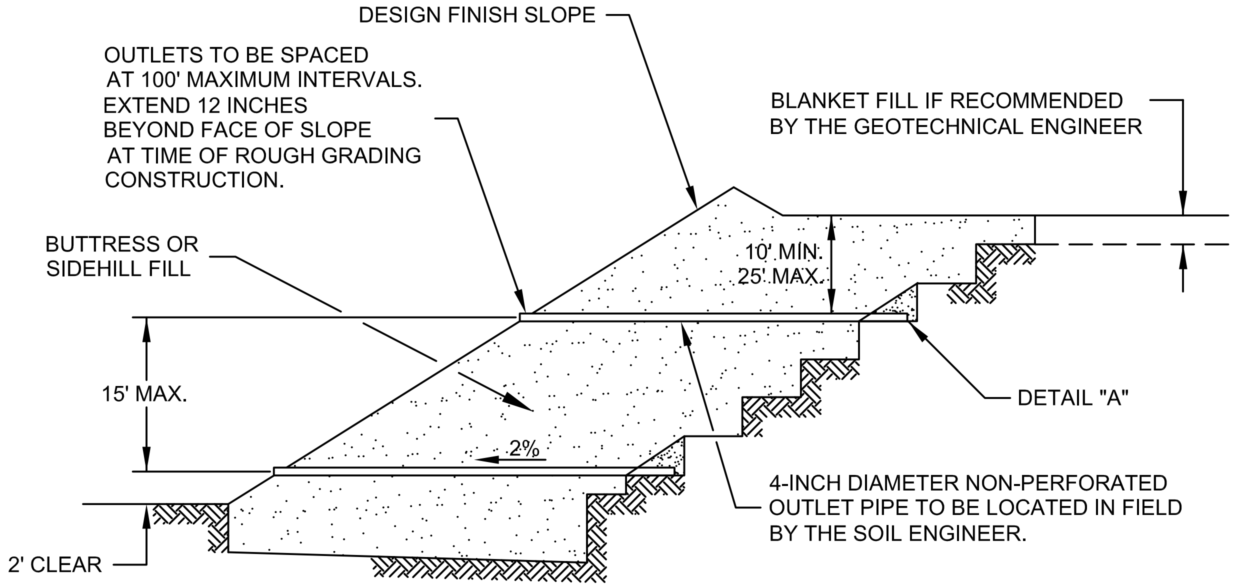
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

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

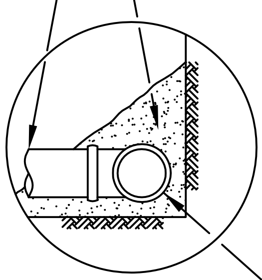
SAND EQUIVALENT = MINIMUM OF 50

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

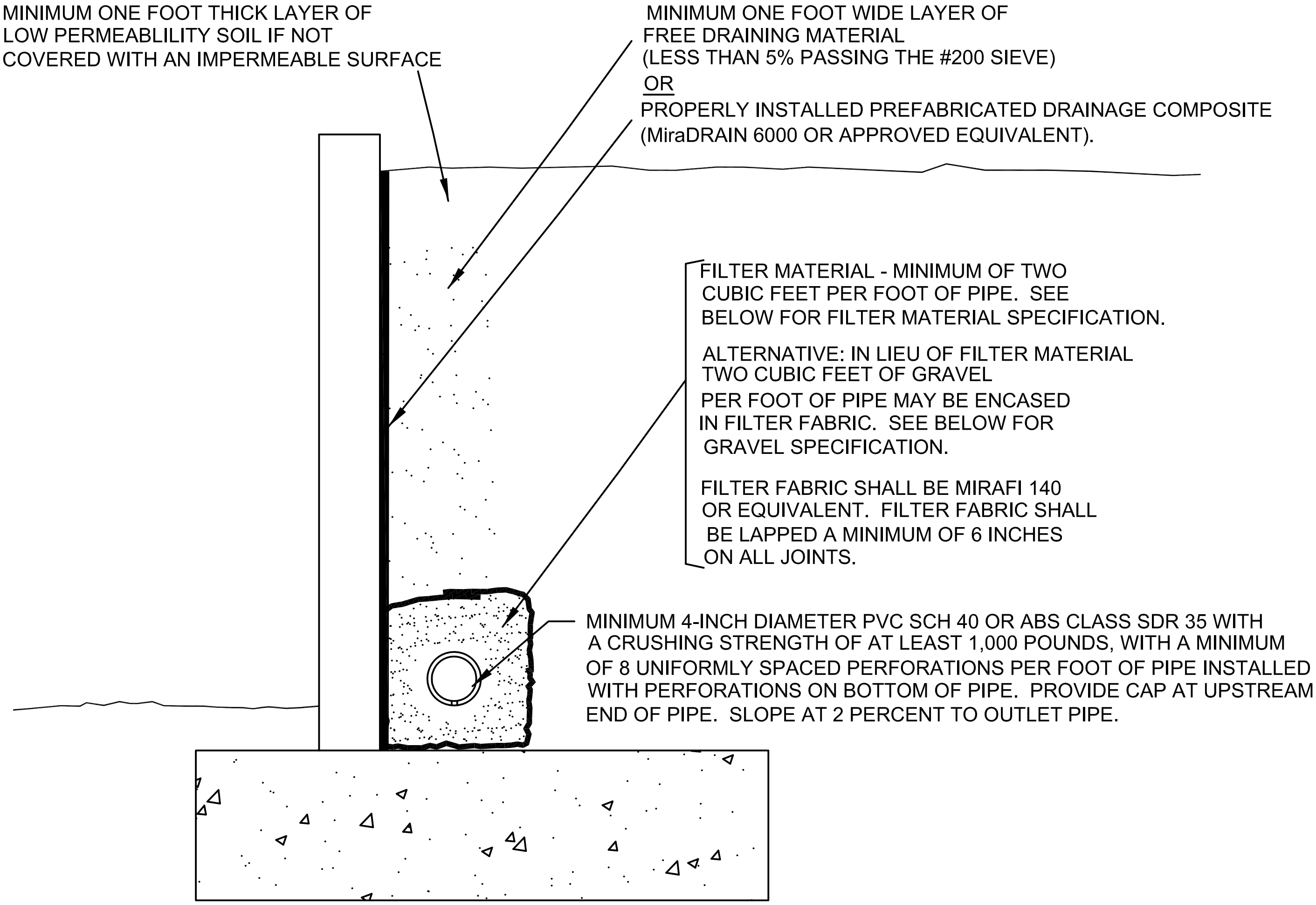
GRADING GUIDE SPECIFICATIONS

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

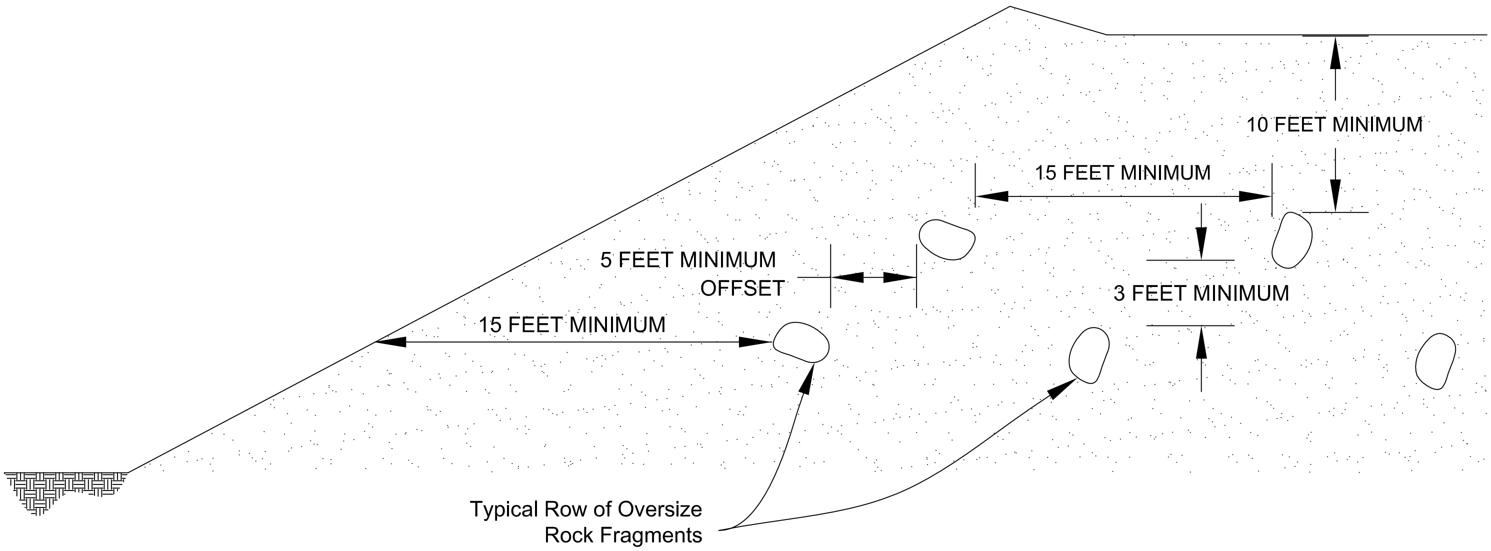
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

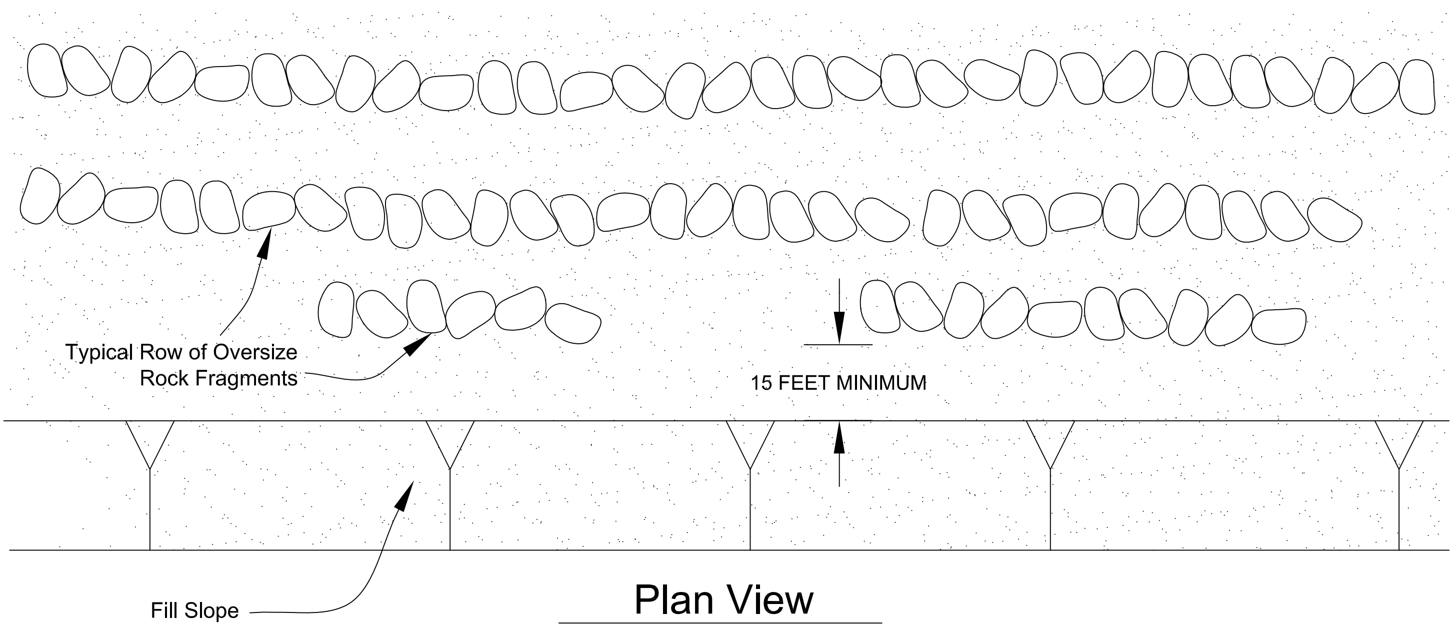
DRAWN: JAS
CHKD: GKM

PLATE D-7

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

Latitude, Longitude: 34.030591, -117.966049

Date	11/15/2021, 9:27:04 AM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.752	MCE_R ground motion. (for 0.2 second period)
S_1	0.628	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.102	Site-modified spectral acceleration value
S_{M1}	0.879	Site-modified spectral acceleration value
S_{DS}	1.402	Numeric seismic design value at 0.2 second SA
S_{D1}	0.586	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.4	Site amplification factor at 1.0 second
PGA	0.748	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.898	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.752	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.932	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.305	Factored deterministic acceleration value. (0.2 second)
S1RT	0.628	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.695	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.736	Factored deterministic acceleration value. (1.0 second)
PGAd	0.923	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.907	Mapped value of the risk coefficient at short periods
C_{R1}	0.903	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 INDUSTRIAL BUILDING

CITY OF INDUSTRY, CALIFORNIA

DRAWN: MD
CHKD: RGT

SCG PROJECT
21G265-1R

PLATE E-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

15010-15100 Nelson Avenue E.
City of Industry, CA
November 3, 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:

KTE

KEHOE TESTING & ENGINEERING

5415 Industrial Drive
Huntington Beach, CA 92649-1518
Office (714) 901-7270 / Fax (714) 901-7289
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 15010-15100 Nelson Avenue E. in City of Industry, California. The work was performed by Kehoe Testing & Engineering (KTE) on November 3, 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	50	
CPT-2	35	Refusal
CPT-3	50	
CPT-4	50	

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 with a cone net area ratio of 0.83. The following parameters were recorded 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

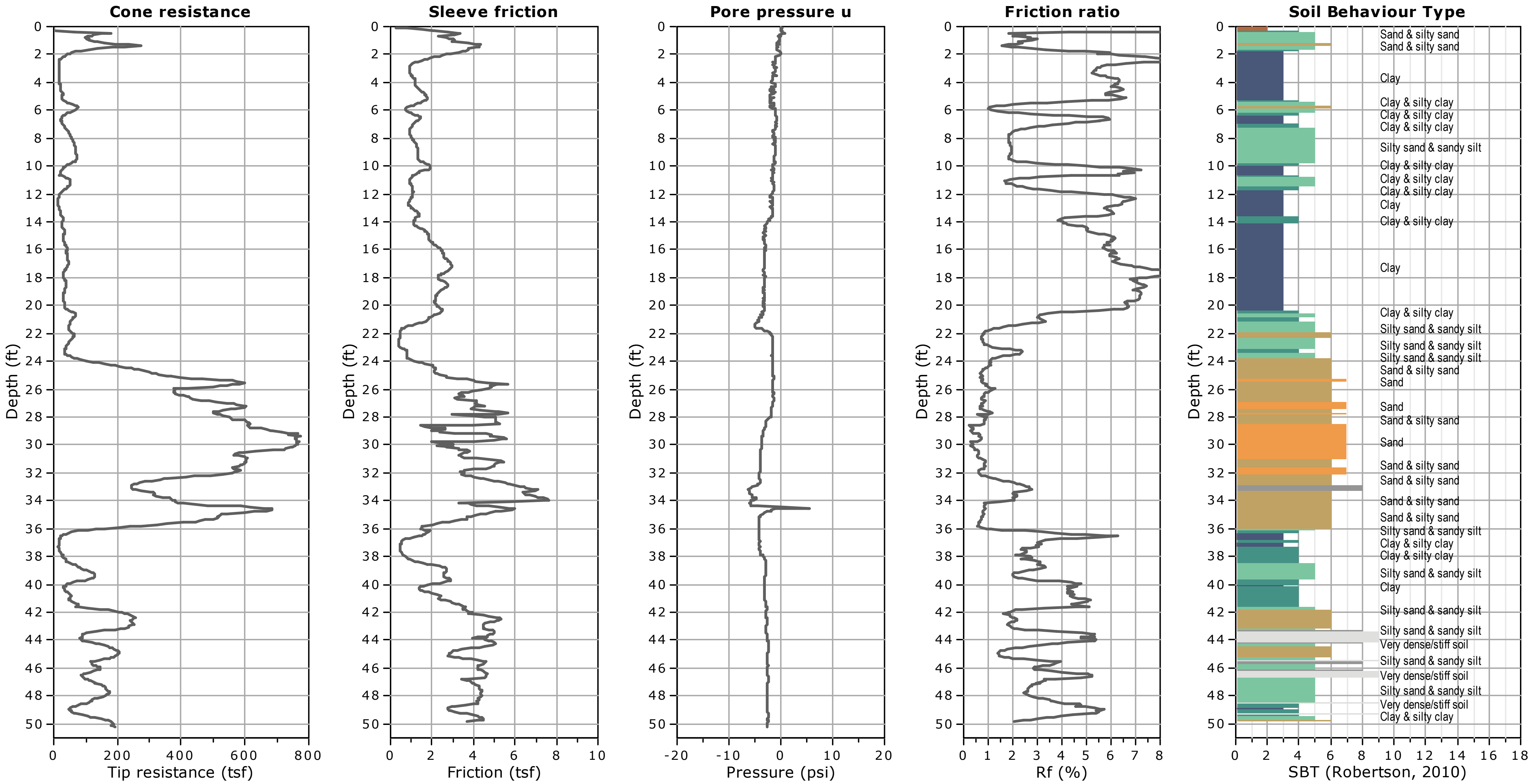
11/09/21-wt-3530

APPENDIX

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

Project: Southern California Geotechnical **CPT-1**

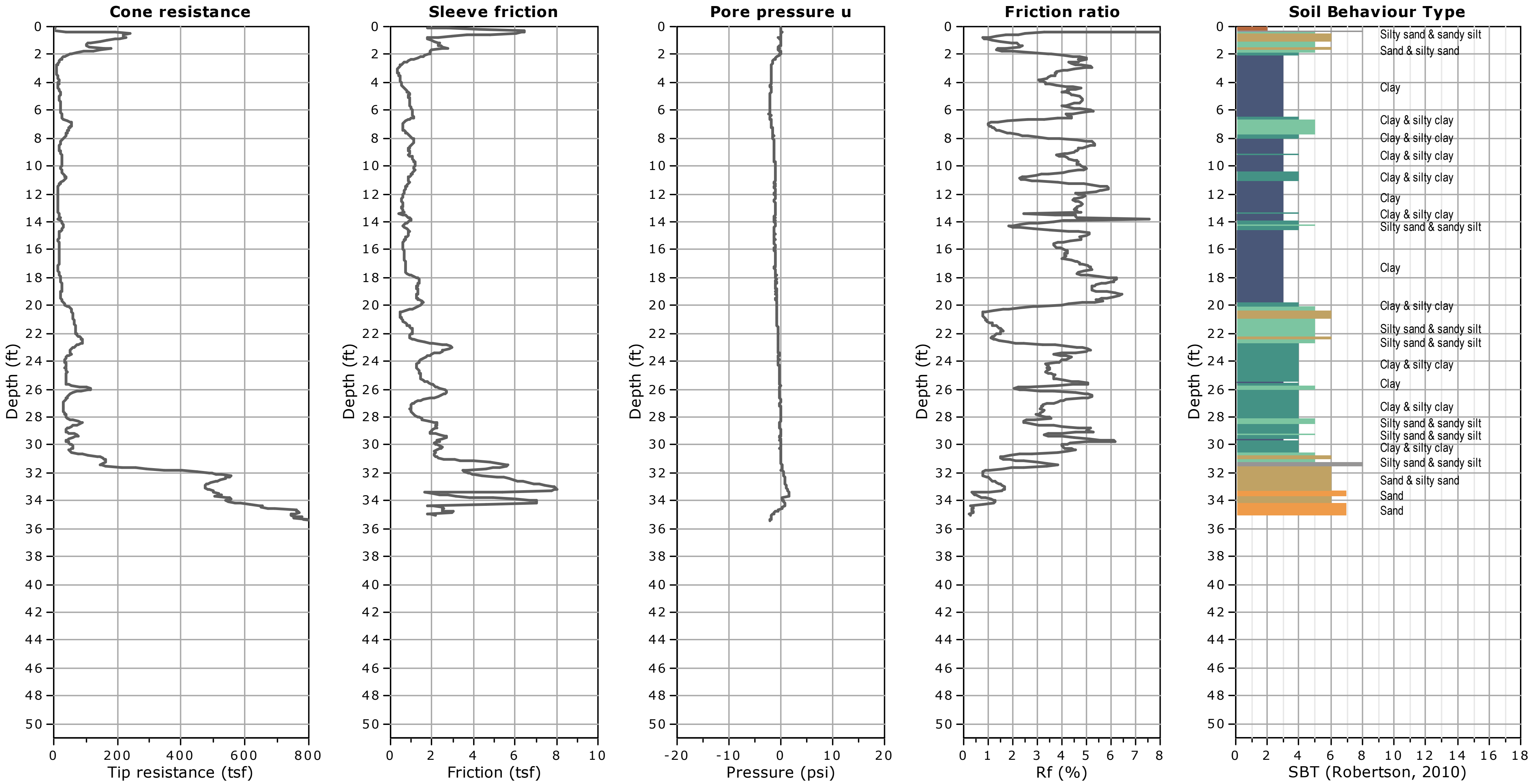
Location: 15010-15100 Nelson Avenue E, City of Industry, CA Total depth: 50.20 ft, Date: 11/3/2021



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

Project: **Southern California Geotechnical**
Location: **15010-15100 Nelson Avenue E, City of Industry, CA**

CPT-2
Total depth: 35.44 ft, Date: 11/3/2021



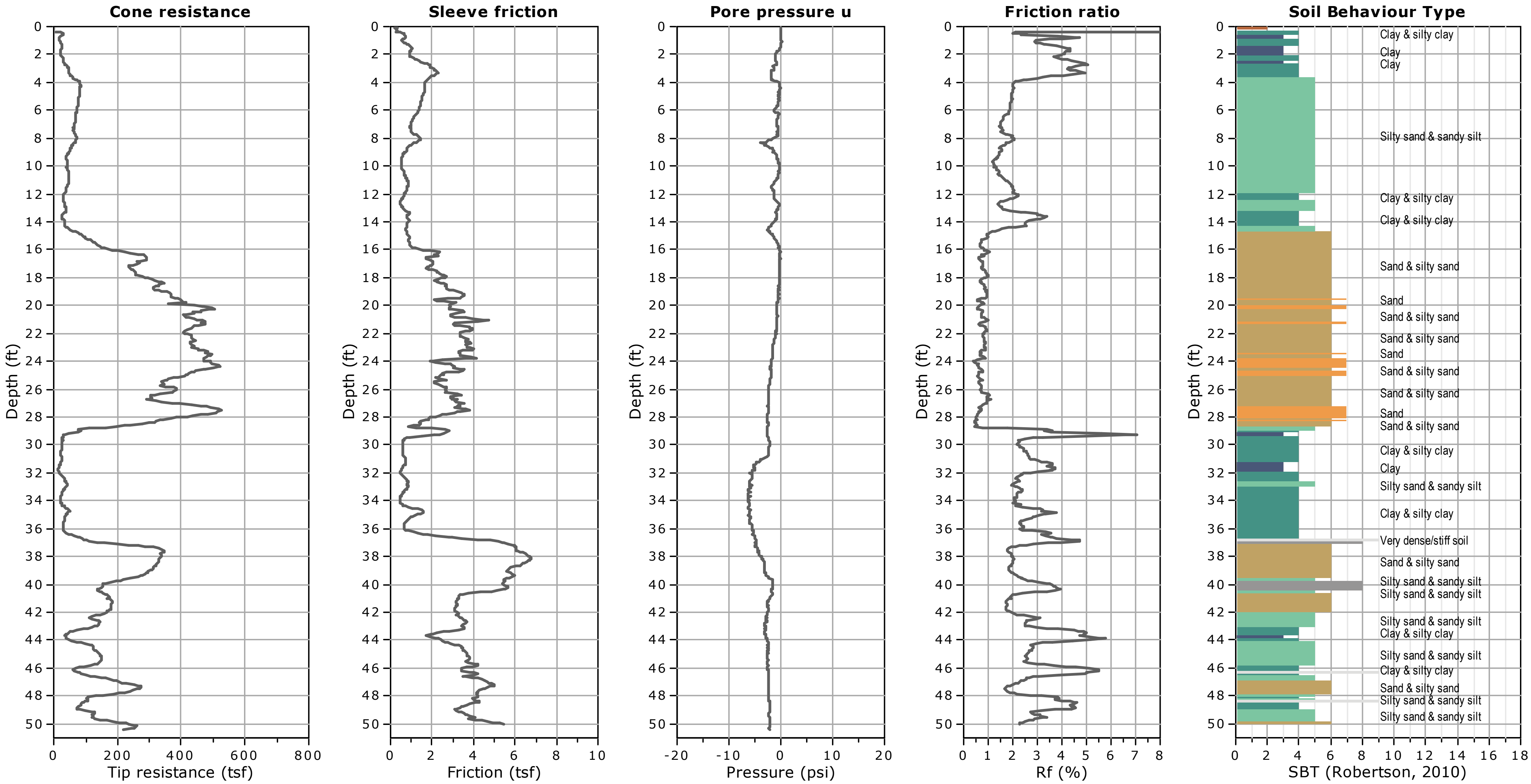
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Southern California Geotechnical

Location: 15010-15100 Nelson Avenue E, City of Industry, CA

CPT-3

Total depth: 50.40 ft, Date: 11/3/2021



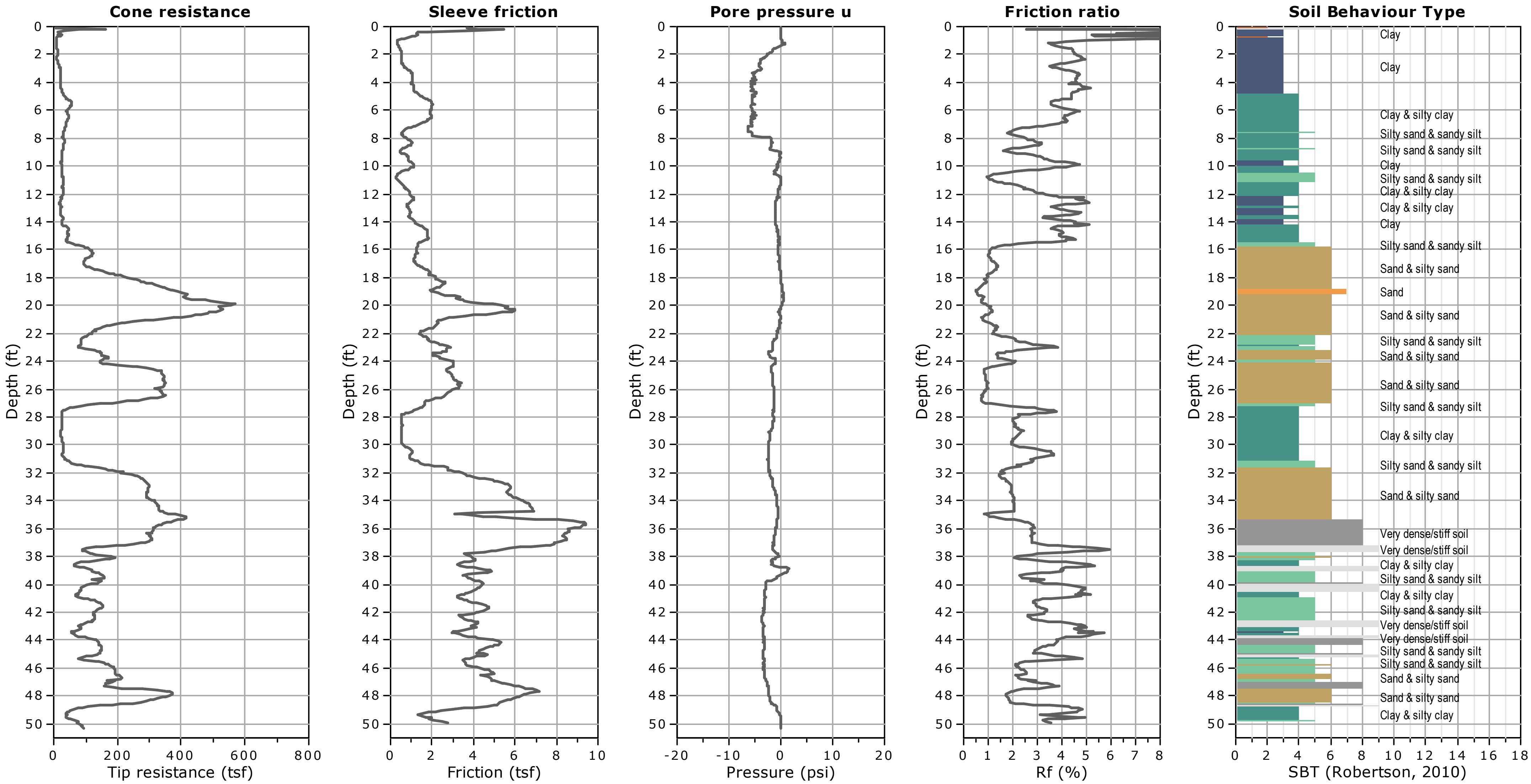
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Southern California Geotechnical

Location: 15010-15100 Nelson Avenue E, City of Industry, CA

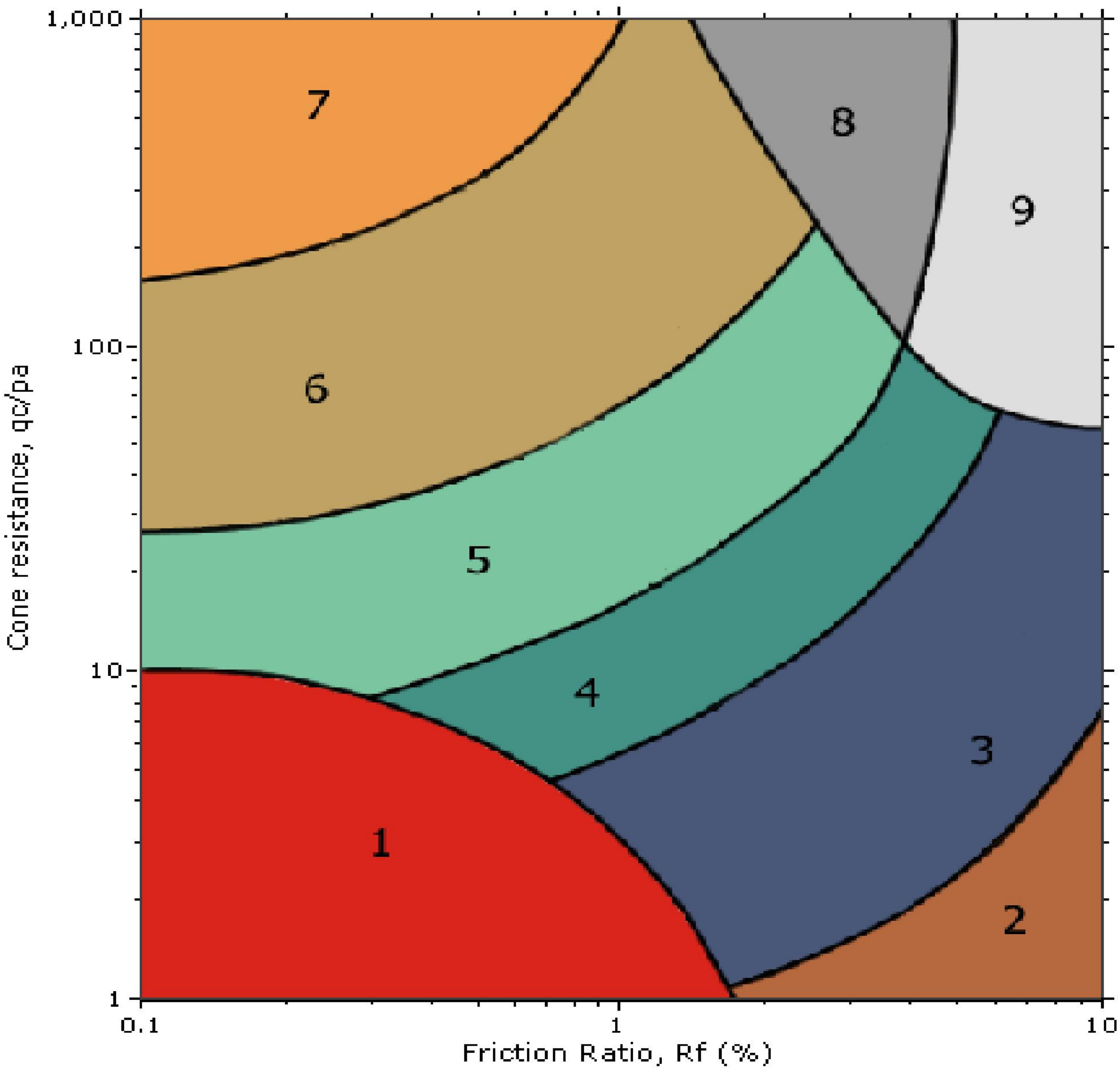
CPT-4

Total depth: 50.27 ft, Date: 11/3/2021



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

SBT plot



SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

APPENDIX G

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Building	MCE_G Design Acceleration	0.823 (g)
Project Location	Industry, CA	Design Magnitude	6.84
Project Number	21G265-1R	Historic High Depth to Groundwater	10 (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=6.84)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5	8	120		1.3	1.05	1.162	1.70	0.75	16.2	16.2	600	600	600	0.99	1.08	1.1	0.17	0.20	N/A	N/A	Above Water Table
9.5	10	12	11	8	120	66	1.3	1.05	1.114	1.25	0.75	11.4	17.0	1320	1258	1320	0.97	1.09	1.06	0.17	0.20	0.54	0.37	**Liquefiable**
14.5	12	17	14.5	11	120	48	1.3	1.05	1.161	1.09	0.85	16.1	21.7	1740	1459	1740	0.95	1.13	1.05	0.23	0.27	0.61	0.45	**Liquefiable**
19.5	17	22	19.5	10	120	51	1.3	1.05	1.142	0.96	0.95	14.2	19.8	2340	1747	2340	0.93	1.12	1.02	0.20	0.23	0.66	0.35	**Liquefiable**
24.5	22	27	24.5	33	120		1.3	1.05	1.3	0.93	0.95	51.5	51.5	2940	2035	2940	0.90	1.29	1.01	2.00	2.00	0.70	2.88	Nonliquefiable
29.5	27	32	29.5	42	120		1.3	1.05	1.3	0.92	0.95	65.1	65.1	3540	2323	3540	0.87	1.29	0.97	2.00	2.00	0.71	2.81	Nonliquefiable
34.5	32	37	34.5	52	120		1.3	1.05	1.3	0.96	1	88.6	88.6	4140	2611	4140	0.84	1.29	0.94	2.00	2.00	0.71	2.80	Nonliquefiable
39.5	37	42	39.5	14	120	69	1.3	1.05	1.156	0.71	1	15.6	21.2	4740	2899	4740	0.81	1.13	0.95	N/A	N/A	N/A	N/A	Non-Liq: PI>18
44.5	42	47	44.5	26	120	47	1.3	1.05	1.3	0.76	1	35.2	40.8	5340	3187	5340	0.78	1.29	0.88	2.00	2.00	0.70	2.85	Nonliquefiable
49.5	47	50	48.5	32	120		1.3	1.05	1.3	0.75	1	42.7	42.7	5820	3418	5820	0.76	1.29	0.86	2.00	2.00	0.69	2.89	Nonliquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Industrial Building
Project Location	Industry, CA
Project Number	21G265-1R
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)			
7	0	10	5	16.2	0.0	16.2	N/A	0.24	0.70	0.00	10.00		0.000		0.00	Above Water Table
9.5	10	12	11	11.4	5.6	17.0	0.37	0.22	0.67	0.22	2.00		0.026		0.63	**Liquefiable**
14.5	12	17	14.5	16.1	5.6	21.7	0.45	0.13	0.42	0.13	5.00		0.021		1.29	**Liquefiable**
19.5	17	22	19.5	14.2	5.6	19.8	0.35	0.16	0.53	0.16	5.00		0.023		1.40	**Liquefiable**
24.5	22	27	24.5	51.5	0.0	51.5	2.88	0.00	-1.71	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	65.1	0.0	65.1	2.81	0.00	-2.86	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	88.6	0.0	88.6	2.80	0.00	-4.99	0.00	5.00		0.000		0.00	Nonliquefiable
	37	42	39.5	15.6	5.6	21.2	N/A	0.14	0.45	0.00	5.00		0.000		0.00	Non-Liq: PI>18
	42	47	44.5	35.2	5.6	40.8	2.85	0.01	-0.86	0.00	5.00		0.000		0.00	Nonliquefiable
	47	50	48.5	42.7	0.0	42.7	2.89	0.00	-1.01	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				3.32	

Notes:

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

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

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

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

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

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

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

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

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Building	MCE_G Design Acceleration	0.823 (g)
Project Location	Industry, CA	Design Magnitude	6.84
Project Number	21G265-1R	Historic High Depth to Groundwater	10 (ft)
Engineer	PM	Depth to Groundwater at Time of Drilling	60 (ft)
		Borehole Diameter	6 (in)
Boring No.	B-3		

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.84)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5	6	120		1.3	1.05	1.117	1.70	0.75	11.7	11.7	600	600	600	0.99	1.05	1.1	0.13	0.15	N/A	N/A	Above Water Table
9.5	10	12	11	6	120	51	1.3	1.05	1.1	1.26	0.75	8.5	14.1	1320	1258	1320	0.97	1.07	1.06	0.15	0.17	0.54	0.31	**Liquefiable**
14.5	12	17	14.5	8	120	44	1.3	1.05	1.113	1.10	0.85	11.3	16.9	1740	1459	1740	0.95	1.09	1.04	0.17	0.20	0.61	0.32	**Liquefiable**
19.5	17	22	19.5	22	120	2	1.3	1.05	1.3	0.97	0.95	35.9	35.9	2340	1747	2340	0.93	1.29	1.05	1.35	1.82	0.66	2.74	Nonliquefiable
24.5	22	27	24.5	33	120		1.3	1.05	1.3	0.93	0.95	51.5	51.5	2940	2035	2940	0.90	1.29	1.01	2.00	2.00	0.70	2.88	Nonliquefiable
29.5	27	32	29.5	12	120	74	1.3	1.05	1.141	0.80	0.95	14.1	19.7	3540	2323	3540	0.87	1.11	0.99	N/A	N/A	N/A	N/A	Non-Liq: PI>18
34.5	32	37	34.5	11	120	87	1.3	1.05	1.124	0.73	1	12.4	17.9	4140	2611	4140	0.84	1.10	0.97	N/A	N/A	N/A	N/A	Non-Liq: PI>18
39.5	37	42	39.5	21	120	50	1.3	1.05	1.279	0.76	1	27.9	33.5	4740	2899	4740	0.81	1.29	0.92	0.83	0.98	0.71	1.38	Nonliquefiable
44.5	42	47	44.5	20	120	58	1.3	1.05	1.242	0.71	1	24.2	29.8	5340	3187	5340	0.78	1.23	0.92	0.47	0.53	0.70	0.76	**Liquefiable**
49.5	47	50	48.5	30	120		1.3	1.05	1.3	0.74	1	39.2	39.2	5820	3418	5820	0.76	1.29	0.86	2.00	2.00	0.69	2.89	Nonliquefiable

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Industrial Building		
Project Location	Industry, CA		
Project Number	21G265-1R		
Engineer	PM		

Boring No. B-3

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines 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)			
7	0	10	5	11.7	0.0	11.7	N/A	0.39	0.87	0.00	10.00		0.000		0.00	Above Water Table
9.5	10	12	11	8.5	5.6	14.1	0.31	0.30	0.79	0.30	2.00		0.030		0.72	**Liquefiable**
14.5	12	17	14.5	11.3	5.6	16.9	0.32	0.22	0.67	0.22	5.00		0.026		1.58	**Liquefiable**
19.5	17	22	19.5	35.9	0.0	35.9	2.74	0.02	-0.50	0.00	5.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	51.5	0.0	51.5	2.88	0.00	-1.71	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	14.1	5.6	19.7	N/A	0.16	0.53	0.00	5.00		0.000		0.00	Non-Liq: PI>18
34.5	32	37	34.5	12.4	5.5	17.9	N/A	0.20	0.62	0.00	5.00		0.000		0.00	Non-Liq: PI>18
	37	42	39.5	27.9	5.6	33.5	1.38	0.03	-0.33	0.00	5.00		0.000		0.00	Nonliquefiable
	42	47	44.5	24.2	5.6	29.8	0.76	0.05	-0.07	0.05	5.00		0.010		0.57	**Liquefiable**
	47	50	48.5	39.2	0.0	39.2	2.89	0.01	-0.74	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				2.87	

Notes:

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

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

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

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

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

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

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

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

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Building	MCE$_G$ Design Acceleration	0.823 (g)
Project Location	Industry, CA	Design Magnitude	6.84
Project Number	21G265-1R	Historic High Depth to Groundwater	10 (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=6.84)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	10	5	7	120		1.3	1.05	1.139	1.70	0.75	13.9	13.9	600	600	600	0.99	1.07	1.1	0.15	0.17	N/A	N/A	Above Water Table
9.5	10	12	11	7	120	40	1.3	1.05	1.1	1.26	0.75	9.9	15.5	1320	1258	1320	0.97	1.08	1.06	0.16	0.18	0.54	0.34	**Liquefiable**
14.5	12	17	14.5	16	120	50	1.3	1.05	1.249	1.07	0.85	24.9	30.5	1740	1459	1740	0.95	1.24	1.08	0.52	0.69	0.61	1.14	**Liquefiable**
19.5	17	22	19.5	30	120		1.3	1.05	1.3	0.98	0.95	49.3	49.3	2340	1747	2340	0.93	1.29	1.05	2.00	2.00	0.66	3.01	Nonliquefiable
24.5	22	27	24.5	38	120		1.3	1.05	1.3	0.94	0.95	60.2	60.2	2940	2035	2940	0.90	1.29	1.01	2.00	2.00	0.70	2.88	Nonliquefiable
29.5	27	32	29.5	13	120	56	1.3	1.05	1.156	0.80	0.95	15.6	21.2	3540	2323	3540	0.87	1.13	0.99	0.22	0.25	0.71	0.35	**Liquefiable**
34.5	32	37	34.5	38	120		1.3	1.05	1.3	0.88	1	59.2	59.2	4140	2611	4140	0.84	1.29	0.94	2.00	2.00	0.71	2.80	Nonliquefiable
39.5	37	42	39.5	23	120	76	1.3	1.05	1.3	0.78	1	31.6	37.2	4740	2899	4740	0.81	1.29	0.9	1.84	2.00	0.71	2.81	Nonliquefiable
44.5	42	47	44.5	26	120		1.3	1.05	1.3	0.73	1	33.7	33.7	5340	3187	5340	0.78	1.29	0.9	0.87	1.00	0.70	1.42	Nonliquefiable
49.5	47	50	48.5	15	120	60	1.3	1.05	1.152	0.64	1	15.2	20.8	5820	3418	5820	0.76	1.13	0.93	0.22	0.23	0.69	0.33	**Liquefiable**

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Proposed Industrial Building		
Project Location	Industry, CA		
Project Number	21G265-1R		
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)			
7	0	10	5	13.9	0.0	13.9	N/A	0.31	0.80	0.00	10.00		0.000		0.00	Above Water Table
9.5	10	12	11	9.9	5.6	15.5	0.34	0.26	0.73	0.26	2.00		0.028		0.67	**Liquefiable**
14.5	12	17	14.5	24.9	5.6	30.5	1.14	0.04	-0.12	0.03	5.00		0.005		0.31	**Liquefiable**
19.5	17	22	19.5	49.3	0.0	49.3	3.01	0.00	-1.54	0.00	5.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	60.2	0.0	60.2	2.88	0.00	-2.44	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	32	29.5	15.6	5.6	21.2	0.35	0.14	0.45	0.14	5.00		0.022		1.31	**Liquefiable**
34.5	32	37	34.5	59.2	0.0	59.2	2.80	0.00	-2.36	0.00	5.00		0.000		0.00	Nonliquefiable
	37	42	39.5	31.6	5.6	37.2	2.81	0.02	-0.60	0.00	5.00		0.000		0.00	Nonliquefiable
	42	47	44.5	33.7	0.0	33.7	1.42	0.03	-0.35	0.00	5.00		0.000		0.00	Nonliquefiable
	47	50	48.5	15.2	5.6	20.8	0.33	0.15	0.47	0.15	3.00		0.022		0.80	**Liquefiable**
											Total Deformation (in)				3.11	

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)