GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL BUILDING

17969 Railroad Street
City of Industry, California
for
Pacific Industrial

SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

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A California Corporation

June 13, 2023

Pacific Industrial
6272 East Pacific Coast Highway, Suite E
Long Beach, California 90803

Attention: Mr. Bo Prock
Acquisitions Manager

Project No.: **23G157-1**

Subject: **Geotechnical Investigation**
Proposed Industrial Building
17969 Railroad Street
Industry, California

Mr. Prock:

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.

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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

- Research of the map, Earthquake Zones of Required Investigation, La Habra Quadrangle, published by the California Geological Survey, indicates that the northern portion of the project site is located in a designated liquefaction hazard zone.
- 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 0.53± to 2.20± 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± inches.
- Based on the estimated magnitude of the differential settlements, the proposed structure may be supported on shallow foundations. Additional design considerations related to the potentially liquefiable soils are presented within of this report.
- The boring locations encountered artificial fill materials, extending from the ground surface to depths of 5½ to 6½± feet. The fill soils possess varying strengths and densities, and are considered to represent undocumented fill. These soils, in their present condition, are not considered suitable for support of the foundation loads of the new structure.
- The results of laboratory testing indicate that the near-surface soils possess a medium expansion potential.

Site Preparation Recommendations

- Demolition of the existing structures, including foundations, floor slabs, pavements, concrete flatwork, and subsurface improvements, which will not be utilized as part of the new development, will be required. Debris resulting from demolition activities should be disposed of off-site in accordance with local regulations. Alternatively, concrete and asphalt debris may be pulverized to a maximum 1-inch particle size, well mixed with on-site sandy soils, and incorporated into new structural fills or it may be crushed and made into crushed miscellaneous base (CMB), if desired. Mixing concrete and asphalt debris with clayey soils is not recommended.
- Initial site stripping should also include removal of surficial vegetation from the unpaved areas of the site. This should include weeds, grasses, shrubs, and trees. Root systems associated with the trees should be removed in their entirety.
- Remedial grading is recommended to be performed within the proposed building area to remove the undocumented fill soils in their entirety, the upper portion of the near-surface native alluvial soils, and soils disturbed during the demolition process. The soils within the proposed building area should be overexcavated to a depth of 6 feet below existing grade and to a depth of at least 4 feet below proposed building pad subgrade elevations, whichever is greater. The proposed foundation influence zones should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade.

- After the overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify additional soils that should be removed. The resulting subgrade should then be scarified to a depth of 12 inches and moisture conditioned (or air dried) to 2 to 4 percent above optimum. 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, thoroughly moisture conditioned and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density.

Foundation Design Recommendations
- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,000 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 the presence of potentially liquefiable soils and medium expansive soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab Design Recommendations
- Conventional Slab-on-Grade, 7 inches thick.
- Modulus of Subgrade Reaction: k = 80 psi/in.
- Minimum reinforcement of the floor slab should consist of No. 3 bars at 16-inches on center in both directions, due to the presence of potentially liquefiable and medium expansive soils. The actual floor slab reinforcement should be provided by the structural engineer, based on the imposed slab loading, geotechnical conditions and intended use.

Pavements

ASPHALT PAVEMENTS (R = 10)					
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	9	12	15	16	19
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 10)				
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. 23P200R, dated March 31, 2023. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the building foundations, building floor slab, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site-specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located at the northwest corner of Railroad Street and South Lawson Street in Industry, California. The site is bounded to the north and west by existing commercial/industrial buildings, to the south by Railroad Street, and to the east by South Lawson Street.

The site consists of a generally rectangular-shaped parcel, 9.81± acres in size. Based on observations made during our subsurface investigation, the site is presently developed with two (2) commercial/industrial buildings. The eastern building is approximately 65,000 ft² in size and the western building is approximately 20,000 ft² in size. The buildings are assumed to be single-story structures of CMU or metal frame construction supported on conventional shallow foundations with concrete slab-on-grade floors. The buildings are surrounded by asphaltic concrete parking, drive lanes and material storage. The northwestern area of the site is used as a storage yard. The ground surface cover in this area appears to consist of exposed soil with sparse to moderate native grass and weed growth with numerous trees.

3.2 Proposed Development

Based on a preliminary site plan prepared by RGA, the site will be developed with one (1) new industrial building, 213,500± ft² in size, located in the eastern area of the site. The building will be constructed with dock-high doors along a portion of the west building wall. Small areas of second floor mezzanine may be constructed in the northwest and southwest corners of the building. The new building will be surrounded by asphaltic concrete pavements in the automobile parking and drive areas, and Portland cement concrete pavements in the truck court and truck traffic areas. We expect the new development will also include areas of concrete flatwork and landscape planters.

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

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 4 to 5± feet are expected to be necessary to achieve the proposed site grades.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration for this project consisted of five (5) borings advanced to depths of 20 to 50± feet below the existing site grades. Two (2) of those borings were advanced to a depth of 50± feet as a part of the liquefaction analysis. Borings were logged during drilling by a member of our staff.

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

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

4.2 Geotechnical Conditions

Pavements

Asphaltic concrete pavements were encountered at the ground surface at Boring Nos. B-1 through B-4. The pavement sections consist of 2 to 3± inches of asphaltic concrete. Aggregate base was encountered beneath the asphaltic concrete at boring Nos. B-2 and B-4 at a thickness of 4± inches.

Artificial Fill

Artificial fill soils were encountered beneath the pavement at Boring Nos. B-1 through B-4, and at the ground surface at Boring No. B-5. The fill extended to depths of 5½ to 6½± feet below ground surface. The fill soils mostly consist of medium stiff to very stiff silty clays and clayey silts. The fill soils contained asphaltic concrete fragments and possess a disturbed and mottled appearance, resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered beneath the fill at each boring location, extending to at least the maximum depth explored of 50± feet below ground surface. The alluvial soils generally consist of stiff to very stiff clayey silts and medium dense to dense sands silty sands. The sandy layers were generally encountered at depths greater than 6± feet below ground surface. The alluvial soils generally possess trace to little iron oxide staining and calcareous veining.

Groundwater

Groundwater was encountered at the time of the subsurface exploration in Boring Nos. B-1 through B-4 at a depth ranging from 15 to 32± feet below existing grade.

As part of our research, we reviewed readily available groundwater data in order to evaluate regional groundwater depths. The primary reference used to evaluate the groundwater depths in the subject site area is the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. One monitoring well on record is located 2100± feet northeast of the site. Water level readings within this monitoring well indicate a high groundwater level of 15± feet below the ground surface in April 1993.

We also reviewed the CGS Open-File Report 97-17, the Seismic Hazard Zone Report for the La Habra 7.5-Minute Quadrangle, which indicates that the historic high groundwater level for the site is 20± feet below the ground surface.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to evaluate 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

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

Dry Density and Moisture Content

The density has been evaluated for selected relatively undisturbed ring samples. These densities were evaluated 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 evaluated 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 evaluate their consolidation potential, in general 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 evaluate their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-4 in Appendix C of this report.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample was tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil type or soil mixes may be necessary at a later date. The results of the testing are plotted on Plates C-5 in Appendix C of this report.

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 evaluate 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.

Expansion Index

The expansion potential of the on-site soils was evaluated in general accordance with ASTM D-4829 as required by the California Building Code. 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-2 @ 0 to 5 feet	53	Medium

Atterberg Limits

Atterberg Limits testing (ASTM D-4318) was performed on selected samples. This test is used to evaluate 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 expansion potential. Atterberg Limits are also useful in evaluating the susceptibility of soils to earthquake induced liquefaction and cyclic softening. The results of the Atterberg Limits testing are presented on the boring logs.

Soluble Sulfates

A representative sample of the near-surface soils was submitted to a subcontracted analytical laboratory for evaluation 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-2 @ 0 to 5 feet	0.0236	Not Applicable (S0)

Corrosivity Testing

A representative bulk sample of the near-surface soils were submitted to a subcontracted corrosion engineering laboratory to evaluate if the near-surface soils possess corrosive characteristics with respect to common construction materials. The corrosivity testing included an evaluation of the minimum electrical resistivity, pH, chloride and nitrate concentrations of the soils, as well as other tests. The results of some of these tests are presented below.

Sample Identification	**Saturated Resistivity (ohm-cm)**	**pH**	**Chlorides (mg/kg)**	**Nitrates (mg/kg)**
B-2 @ 0 to 5 feet	938	7.9	35.3	4.1

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Furthermore, SCG did not identify 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. Liquefaction is a potential geologic hazard for this site and is discussed below.

Seismic Design Parameters

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

The 2022 CBC Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool, a web-based software application available at the website www.seismicmaps.org. This software application calculates seismic design parameters in accordance with several building code reference documents, including ASCE 7-16, upon which the 2022 CBC is based. The application utilizes a database of risk-targeted maximum considered earthquake (MCE_R) site accelerations at 0.01-degree intervals for each of the code documents. The table below was created using data obtained from the application. The output generated from this program is attached to this letter.

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

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

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.815
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.640
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.815
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.088[1]
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.210
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.725[1]

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

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

Liquefaction

Liquefaction is the loss of strength in generally cohesionless, saturated soils when the pore-water pressure induced in the soil by a seismic event becomes equal to or exceeds the overburden pressure. The primary factors which influence the potential for liquefaction include groundwater table elevation, soil type and plasticity characteristics, relative density of the soil, initial confining pressure, and intensity and duration of ground shaking. The depth within which the occurrence of liquefaction may impact surface improvements is generally identified as the upper 50 feet below the existing ground surface. Liquefaction potential is greater in saturated, loose, poorly graded fine sands with a mean (d_{50}) grain size in the range of 0.075 to 0.2 mm (Seed and Idriss, 1971). Non-sensitive clayey (cohesive) soils which possess a plasticity index of at least 18 (Bray and Sancio, 2006) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table. Soils with a PI of 7 or greater will exhibit clay-like behavior, and may be susceptible to cyclic softening (Boulanger and Idriss, 2006).

The Earthquake Zones of Required Investigation, Baldwin Park Quadrangle map, published by the California Geological Survey (CGS), indicates that the subject site is located within a designated liquefaction hazard zone. Therefore, the scope of this investigation included a detailed liquefaction evaluation in order to evaluate the site-specific liquefaction potential.

The liquefaction analysis was conducted in general accordance with the requirements of Special Publication 117A (CDMG, 2008), and currently accepted practice (SCEC, 1997). The liquefaction potential of the subject site was evaluated using the empirical method developed by Boulanger and Idriss (Boulanger and Idriss, 2008). This method predicts the earthquake-induced liquefaction potential of the site based on a given design earthquake magnitude and peak ground acceleration at the subject site. This procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake moment magnitude). CRR is evaluated as a function of the corrected SPT N-value $(N_1)_{60\text{-}cs}$, adjusted for fines content. The factor of safety against liquefaction is defined as CRR/CSR. Based on Special Publication 117A, a factor of safety of at least 1.3 is required in order to demonstrate that a given soil stratum is non-liquefiable.

As part of the liquefaction evaluation, Boring No. B-1 and B-4 were extended to a depth of 50± feet. Four of the borings encountered groundwater ranging from a depth of 15 to 32 feet and based on the research discussed in Section 4.2 of this report, the historic high groundwater depth in a nearby monitoring well was 15 feet. Therefore, a groundwater depth of 15 feet below grade was used for this liquefaction evaluation.

The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis was performed for Boring Nos. B-1 and B-4. The liquefaction potential of the site was analyzed utilizing a PGA_M of 0.858 for a magnitude 6.72 seismic event.

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

Conclusions and Recommendations

Potentially liquefiable soils were encountered at both of the 50-foot-deep boring locations. The potentially liquefiable strata identified at Boring No. B-1 are present between depths of 42 to 47± feet. At Boring No. B-4, the potentially liquefiable soils are present between depths of 15 to 17± feet. The remaining soil strata encountered below the historic high groundwater table either possess factors of safety of greater than 1.3, or are considered non-liquefiable due to their cohesive characteristics. Settlement analyses were performed for the potentially liquefiable strata. The results of the settlement analyses indicate the following calculated total deformations:

- Boring No. B-2: 2.20 inches
- Boring No. B-3: 0.53 inches

Based on the results of the settlement analyses, total dynamic settlements due to liquefaction are expected to be on the order of ½ to 2½± inches. The resulting differential settlement is expected to be on the order of 1½± inches. The estimated differential settlement can be assumed to occur across a distance of 100 feet, indicating an angular distortion of approximately 0.002 inches per inch.

Based on our understanding of the proposed development, it is considered feasible to support the proposed structure on shallow foundations. Such a foundation system can be designed to resist the effects of the anticipated differential settlements, to the extent that the structure would not catastrophically fail. Designing the proposed structure to remain completely undamaged during a seismic event that could occur once every 2475 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 shallow foundation systems is considered to be the most economical means of supporting the proposed structure.

In order to support the proposed structure on shallow foundations (such as spread footings) the structural engineer should confirm that the structure would not catastrophically fail due to the predicted dynamic differential settlements. Utility connections to the structure 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 a shallow foundation system, as described in this report, is typical for buildings of this type, where they are underlain by the extent of liquefiable soils encountered at this site. The post-liquefaction damage that could occur within the building proposed for this site will also be typical of similar buildings in the vicinity of this project. However, if the owner evaluates that this level of potential damage is not acceptable, other geotechnical and structural options are available, including the use of ground improvement techniques or mat foundations.

6.2 Geotechnical Design Considerations

General

The near surface soils at this site consist of low to moderate strength fill and native alluvium. Most of the fill soils encountered during subsurface exploration are loose or soft to medium stiff and are considered to represent undocumented fill materials due to their disturbed appearance and the lack of documentation regarding the placement and compaction of these materials. In addition, the near surface alluvium possesses variable strengths and composition, and the results of laboratory testing indicate these near-surface alluvial soils possess a potential for consolidation and/or collapse. Based on these conditions, remedial grading is considered warranted within the proposed building area in order to remove the upper portion of the alluvium and the existing artificial fill soils, and to replace these soils as compacted structural fill.

As discussed in a 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 the potential for 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 structural rigidity in order to reduce the potential effects of differential settlement that could occur as a result of liquefaction.

LA County Section 111 Statement

Based on the results of our geotechnical analysis, the proposed development will be safe with regard to landslides, settlement and/or slippage. In addition, the proposed development will not adversely affect the geologic stability of the adjacent properties. This finding is in accordance with Section 111 of the Los Angeles County Building Code

Settlement

The recommended remedial grading will remove the artificial fill soils and a portion of the variable strength near-surface native alluvium. The excavated soils will be replaced as compacted structural fill. The native soils that will remain in place below the recommended depth of overexcavation are not anticipated to be subject to significant load increases from the foundations of the new structures. Provided that the recommended remedial grading is completed, the post-construction static settlements of the proposed structure are expected to be within tolerable limits.

Soluble Sulfates

The results of the laboratory testing indicate that the concentration of soluble sulfates in the selected sample of the on-site soils corresponds 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 area.

Expansion

The majority of near-surface soils encountered at the boring locations consist of silty clays to clayey sands. Laboratory testing performed on representative samples of these materials indicate that they possess a medium expansion potential (EI = 53). Based on the presence of expansive soils, special care should be taken to properly moisture condition and maintain adequate moisture content within subgrade soils as well as newly placed fill soils. The foundation and floor slab design recommendations contained within this report are made in consideration of the expansion index test results. It is recommended that additional expansion index testing be conducted at the completion of rough grading to evaluate the expansion potential of the as-graded building pad.

Corrosion Potential

The results of the electrical resistivity and pH testing indicate that samples of the on-site soils have minimum resistivity value of 938 ohm-cm, and a pH value of 7.9. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity and pH are two of the five factors that enter into the evaluation procedure. Redox potential, relative soil moisture content and sulfides are also included. Although sulfide testing was not part of the scope of services for this project, we have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH and moisture content. **Based on these factors, and utilizing the DIPRA procedure, the on-site soils are considered to be highly corrosive to ductile iron pipe. Therefore, polyethylene protection is expected to be required for cast iron or ductile iron pipes.**

Based on American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary, reinforced concrete that is exposed to external sources of chlorides requires corrosion protection for the steel reinforcement contained within the concrete. ACI 318 defines concrete exposed to moisture and an external source of chlorides as "severe" or exposure category C2. ACI 318 does not clearly define a specific chloride concentration at which contact with the adjacent soil will constitute a "C2" or severe exposure. However, the Caltrans Memo to Designers 10-5, Protection of Reinforcement Against Corrosion Due to Chlorides, Acids and Sulfates, dated June 2010, indicates that soils possessing chloride concentrations greater than 500 mg/kg are considered to be corrosive to reinforced concrete. The results of the laboratory testing indicate chloride concentrations of 35 mg/kg. Although the soils contain some chlorides, we do not expect that the chloride concentrations of the tested soils are high enough to constitute a "severe" or C2 chloride exposure. Therefore, a chloride exposure category of C1 is considered appropriate for this site.

Nitrates present in soil can be corrosive to copper tubing at concentrations greater than 50 mg/kg. The tested sample possesses a nitrate concentration of 4 mg/kg. Based on this test result, the on-site soils are not considered to be corrosive to copper pipe.

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

Shrinkage/Subsidence

Removal and recompaction of the near-surface alluvium and fill soils is estimated to result in an average shrinkage of 5 to 15 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 evaluated using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

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

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

Grading and Foundation Plan Review

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

6.3 Site Grading Recommendations

The grading recommendations presented below are based on the subsurface conditions encountered at the boring locations and our understanding of the proposed development. We recommend that 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.

Demolition

The proposed development will require demolition of the existing pavements and structures. Additionally, existing improvements that will not remain in place for use with the new development should be removed in their entirety. This should include utilities, foundations, buried tanks and other existing subsurface improvements.

The existing pavements are not expected to be reused with the new development. Debris resultant from demolition should be disposed of off-site. Concrete and asphalt debris may be re-used as compacted fill, provided they are pulverized to a maximum particle size of less than 2 inches and mixed with sandy soils. Alternatively, existing asphalt and concrete materials may be crushed into miscellaneous base (CMB) and re-used at the site.

Existing trees, vegetation and organic materials within the landscape planters should be removed and disposed of offsite.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building area in order to remove the undocumented fill soils and a portion of the near-surface alluvial soils. The artificial fill soils extend to depths of 5½ to 6½± feet at the boring locations. Based on the conditions encountered at the boring locations, the existing soils within the proposed building area are recommended to be overexcavated to a depth of at least 6 feet below the existing grade and to a depth of at least 4 feet below the proposed building pad subgrade elevation, whichever is greater. Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade.

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

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

Based on the conditions encountered at the exploratory boring locations, there are a few locations where the existing soils are very moist soils at or near the base of the recommended overexcavation. Stabilization of the exposed overexcavation subgrade soils may be necessary if wet, pumping or unstable conditions are encountered. 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 or geotextile, will likely be necessary.** 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 RS580i, 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 be stabilized by treating the upper 12 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, thoroughly moisture conditioned, and recompacted. Overexcavation bottoms should be thoroughly moisture conditioned (or air dried) to achieve a moisture content of 2 to 4 percent above the optimum moisture content, extending to a depth of 12 inches below the overexcavation subgrade and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

<u>Treatment of Existing Soils: Retaining Walls and Site Walls</u>

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 3 feet below foundation bearing grade and replaced as compacted structural fill as discussed above for the proposed building pads. Undocumented fill soils encountered within these foundation areas should be removed in their entirety. The overexcavation areas should extend horizontally beyond the foundation perimeters to a distance equal to the depth of fill below the new foundations. These overexcavation recommendations also apply to erection pads for tilt-up concrete walls, since these pads are part of the foundation system.

The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning, and recompacting the upper 12 inches of exposed subgrade soils, as discussed for the building areas. The previously excavated soils may then be replaced as compacted structural fill. Please note that if the lateral and/or vertical extents of overexcavation are not achievable for the project retaining walls or site walls, then additional recommendations including, but not limited to reduced design bearing pressures may be required. Additionally, specialized grading techniques such as slot cutting or shoring may be required in order to facilitate construction.

<u>Treatment of Existing Soils: Parking and Drive Areas</u>

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

Subgrade preparation in the new parking and drive areas should initially consist of removal of soils disturbed during demolition operations. The geotechnical engineer should then evaluate the subgrade to identify 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 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.

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 existing undocumented fill soils and variable strength alluvium in the parking and drive 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.

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned (or air dried) to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of debris to the satisfaction of the geotechnical engineer and can be adequately moisture conditioned by wetting or drying, as needed.
- Grading and fill placement activities should be completed in accordance with the requirements of the 2022 CBC and the grading code of the City of Industry.
- 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

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, utility trench backfill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. As an alternative, a clean sand (minimum Sand Equivalent of 30) may be placed within trenches and compacted in place (jetting or flooding is not recommended). It is recommended that materials in excess of 3 inches in size not be used for utility trench backfill. Compacted trench backfill should conform to the requirements of the local grading code, and more restrictive requirements may be indicated by City of Industry. Utility trench backfills should be observed 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 mostly consist of medium stiff to stiff silty clays and fine sandy clays. However, some loose to medium dense silty fine to coarse sands and clayey fine sands were also encountered, which 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, the inclination of temporary slopes should not exceed 2h:1v for sands and 1.5h:1v for clays. 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. Excavation activities on this site should be conducted in accordance with Cal-OSHA regulations. Temporary excavation stability and safety are the responsibility of the contractor.

Moisture Sensitive Subgrade Soils

The near surface soils include appreciable silt and clay content that could become unstable if exposed to significant moisture infiltration or disturbance by construction traffic. If grading occurs during a period of relatively wet weather, an increase in subgrade instability should also be expected. The site should, therefore, be graded to reduce the potential for ponding of surface water and to prevent water from running into excavations.

As discussed in Section 6.3 of this report, unstable subgrade soils may be encountered at the base of the overexcavations within the proposed building area. 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. Due to the potential for subgrade instability, it is recommended that tracked vehicles be utilized for grading or construction activities that require traffic over the exposed subgrade soils.

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

Expansive Soils

The near surface soils have been evaluated to possess a medium expansion potential. Therefore, care should be given to proper moisture conditioning of building pad subgrade soils to a moisture content of 2 to 4 percent above the Modified Proctor optimum during site grading. Imported fill soils should have low expansion potential (EI < 50). **In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintain moisture content of these soils at 2 to 4 percent above the Modified Proctor optimum. 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**.

Due to the presence of expansive soils at this site, provisions should be made to reduce the potential for surface water to penetrate the soils immediately adjacent to the structure. 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 building. Where possible, it is recommended that landscaped planters not be located immediately adjacent to the building. 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 structures (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 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.
- 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 evaluated by the landscape architect or civil engineer, may also be appropriate.

Groundwater

Groundwater was encountered during the drilling of the borings for this investigation as shallow as 15± feet below grade. Based on the depth to groundwater, it is not expected to significantly impact the grading or foundation construction activities. However, deeper excavations at the site could encounter groundwater.

6.5 Foundation Design and Construction

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

Foundation Design Parameters

New continuous and square/rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,000 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) in strip footings, due to the presence of potentially liquefiable and 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 bottom of the floor slab.

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

The allowable bearing pressures presented above may be increased by 1/3 when considering short duration wind loads**. We do not recommend an increase in allowable soil bearing pressure for seismic loads because potentially liquefiable soils are present at depths as shallow as 15± feet below the existing site grades.** The minimum steel reinforcement recommended above is based on standard geotechnical practice where liquefiable and expansive soils are present. However, the recommendation for additional steel reinforcement is not intended to completely mitigate the potential differential settlements. 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 or for expansive soil design. The actual design of the foundations should be evaluated and provided by the structural engineer.

Foundation Construction

The foundation subgrade soils should be evaluated at the time of overexcavation, as discussed in Section 6.3 of this report. It is further recommended that the foundation subgrade soils be evaluated by the geotechnical engineer immediately prior to steel or concrete placement. Soils suitable for direct foundation support should consist of newly placed structural fill, compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Unsuitable materials, if encountered, 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 optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 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. 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: 0.25

When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill. The maximum allowable passive pressure is 2000 lbs/ft^2.

6.6 Floor Slab Design and Construction

Subgrades which will support new floor slab should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, and based on the design considerations presented in Section 6.1 of this report, the floor of the proposed structure may be constructed as a conventional slab-on-grade supported on newly placed structural fill, extending to a depth of at least 4 feet below proposed finished pad grade. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 7 inches

- Modulus of Subgrade Reaction: k = 80 psi/in.

- Minimum slab reinforcement: Minimum slab reinforcement: No. 3 bars at 16 inches on-center, in both directions, due to the presence of potentially liquefiable and medium expansive soils at the site. The actual floor slab reinforcement should be evaluated by the structural engineer, based on the imposed loading, and the liquefaction-induced settlements.

- Slab underlayment: If moisture sensitive floor coverings will be used or if moisture transmission through the slab is not acceptable then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire slab area where such moisture sensitive areas are expected. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with 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 or where moisture transmission through the slab is acceptable, 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 slabs should be completed by the structural engineer to provide adequate thickness and reinforcement. The steel reinforcement recommendations presented above are based on standard geotechnical practice, given the magnitude of predicted liquefaction-induced settlements, and the structure type proposed for the site. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements discussed in Section 6.1.

6.7 Retaining Wall Design and Construction

New retaining walls are expected to be necessary in the truck court and in the dock-high areas of the building. Additionally, although not indicated on the site plan, the proposed development may require some small retaining walls (less than 5± feet in height) to facilitate the new site grades.

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of on-site soils for retaining wall backfill. The near surface soils generally consist of silty clays, sandy silts, and silty sands. Based on their classifications, the silty sand and sandy silt materials are expected to possess a friction angle of at least 27 degrees when compacted to at least 90 percent of the ASTM-1557 maximum dry density. However, sufficient amounts of sandy soil may not be present on site at the anticipated depths of excavation, and based on the expansive potential and lack of adequate drainage properties of the site clays and silts, we do not recommend using silty or clayey soils as retaining wall backfill. Select grading to provide sandy material, or importing of sandy soil will likely be required for retaining wall backfill.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: On-Site Sandy Silts and Silty Sands
Internal Friction Angle (ϕ)		27°
Unit Weight		120 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	45 lbs/ft^3
	Active Condition (2h:1v backfill)	85 lbs/ft^3
	At-Rest Condition (level backfill)	66 lbs/ft^3

Regardless of the backfill type, the walls should be designed using a soil-footing coefficient of friction of 0.25 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 2022 CBC, retaining walls more than 6 feet in height must be designed for seismic lateral earth pressures. If walls 6 feet or more are required for this site, the geotechnical engineer should be contacted for supplementary seismic lateral earth pressure recommendations.

Retaining Wall Foundation Design

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

Backfill Material

Select on-site soils may be used to backfill the retaining walls if a sufficient amount of non-expansive, free draining sandy soil is encountered during grading. Select grading may be required to use on-site soils as retaining wall backfill. Backfill material placed within 3 feet of the back wall face should also have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

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

Retaining wall backfill should be placed and compacted under engineering observed conditions with the necessary layer thicknesses to allow an in-place density between 90 and 93 percent of the maximum dry density as evaluated 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 designed by the civil engineer to provide a drainage system that possesses adequate capacity and slope for its intended use.

6.8 Pavement Design Parameters

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

Pavement Subgrades

It is anticipated that the new pavements will be supported on the existing fill and/or native soils that have been scarified, moisture conditioned, and recompacted. These materials generally consist of silty clays and clayey silts. These materials are expected to exhibit poor pavement support characteristics, with estimated R-values of 10 to 20. Since R-value testing was not included in the scope of services for the current project, the subsequent pavement designs are based upon an assumed R-value of 10. 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 may be desirable to perform R-value testing after the completion of rough grading to verify the R-value of the as-graded parking subgrade.

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 evaluate 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. The traffic indices above allow for 1,000 automobiles per day.

ASPHALT PAVEMENTS (R = 10)					
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	9	12	15	16	19
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 evaluated 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 = 10)				
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. Reinforcement within the PCC pavements should be evaluated by the project structural engineer. The maximum joint spacing within 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 evaluate 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 confirm 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.

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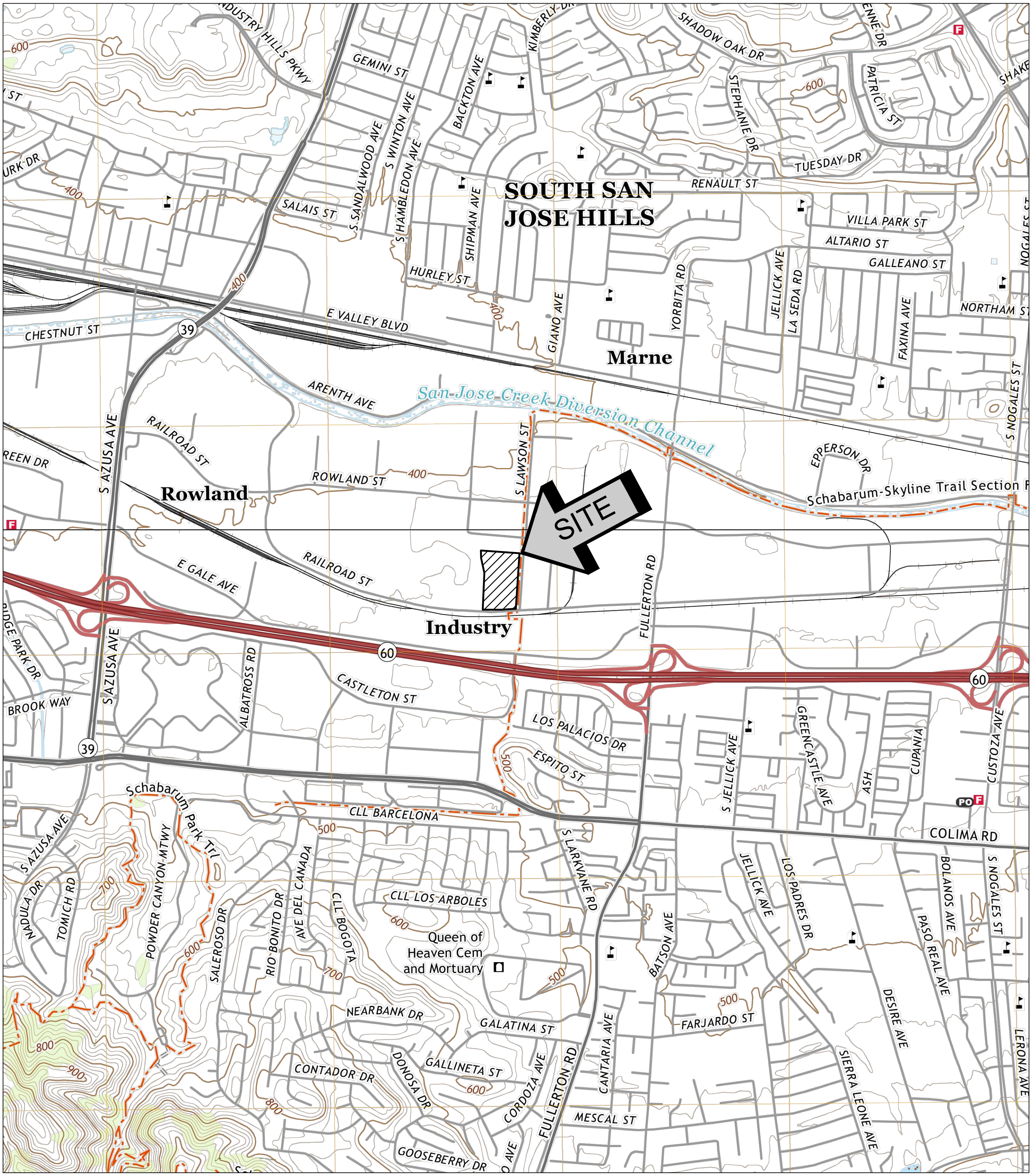
Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California," Committee formed 1997.

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

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

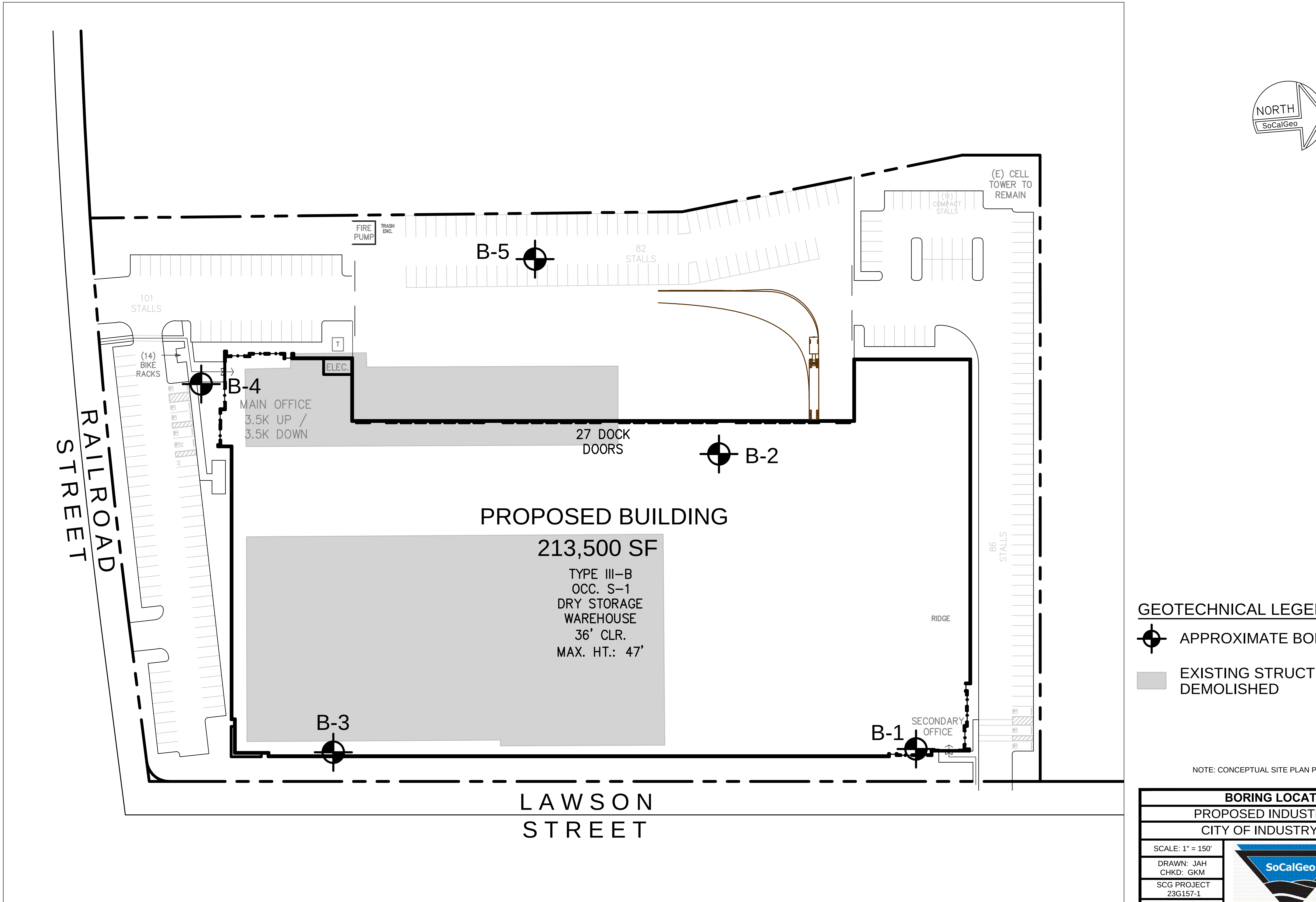
Youd, T. L. and Idriss, I. M. (Editors), "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Salt Lake City, UT, January 5-6 1996, NCEER Technical Report NCEER-97-0022, Buffalo, NY.

APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAPS OF THE 2021 BALDWIN PARK QUADRANGLE, LOS ANGELES COUNTY, AND THE 2022 LA HABRA QUADRANGLE, ORANGE COUNTY, CALIFORNIA.

SITE LOCATION MAP	
PROPOSED INDUSTRIAL BUILDING	
CITY OF INDUSTRY, CALIFORNIA	
SCALE: 1" = 2000'	SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAH CHKD: GKM	
SCG PROJECT 23G157-1	
PLATE 1	



GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATION

EXISTING STRUCTURES TO BE DEMOLISHED

NOTE: CONCEPTUAL SITE PLAN PREPARED BY RGA.

BORING LOCATION PLAN

PROPOSED INDUSTRIAL BUILDING

CITY OF INDUSTRY, CALIFORNIA

SCALE: 1" = 150'

DRAWN: JAH
CHKD: GKM

SCG PROJECT
23G157-1

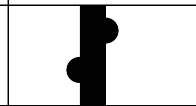
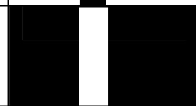
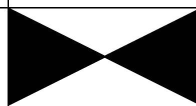
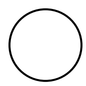
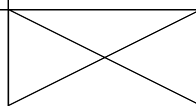
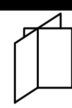
PLATE 2

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

DEPTH: Distance in feet below the ground surface.

SAMPLE: Sample Type as depicted above.

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

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

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

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

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

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

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

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

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

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS 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.: 23G157-1
PROJECT: Proposed Industrial Building
LOCATION: City of Industry, California

DRILLING DATE: 5/17/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: 15 feet
CAVE DEPTH: 22 feet
READING TAKEN: At Completion

DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	DESCRIPTION SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
				ASPHALT: 3±-inches Asphaltic Concrete with no discernible Aggregate Base							
	X	11	2.0	FILL: Dark Brown Silty Clay, trace to little fine Sand, trace AC fragments, trace medium to coarse sand, stiff-moist		14					
	X	8	1.0	@ 3½ feet, no AC, no medium to coarse Sand		14					
5											
	X	12	2.0	ALLUVIUM: Brown Clayey Silt, trace fine Sand, stiff to very stiff-moist to very moist		14					
	X	15	2.5			21					
10											
				Brown fine to medium Sand, trace coarse Sand, trace fine Gravel, medium dense-very moist to wet							
	X	22				9					
15				▽							
				Brown fine to coarse Sand, trace to little fine Gravel, little Silt, dense-wet							
	X	33				10					
20											
	X	31		@ 23½ feet, trace Clay		8					
25											
	X	20				13			10		
30				Dark Brown fine Sandy Clay, little Silt, little Iron Oxide staining, very stiff-wet		20			57		
	X	18				24	28	20	59		

TBL 23G157-1.GPJ SOCALGEO.GDT 6/13/23

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 23G157-1
PROJECT: Proposed Industrial Building
LOCATION: City of Industry, California

DRILLING DATE: 5/17/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: 15 feet
CAVE DEPTH: 22 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
					Dark Brown fine Sandy Clay, little Silt, little Iron Oxide staining, very stiff-wet							
					Brown Silty fine Sand, trace medium to coarse Sand, trace fine to coarse Gravel, trace Iron Oxide staining, dense-wet							
	X	41					17					
40												
					Brown Silty fine to medium Sand, little Silt, trace fine Gravel, loose to dense-wet							
	X	9					19			3		
45												
	X	32					18					
50												
					Boring Terminated at 50'							

TBL 23G157-1.GPJ SOCALGEO.GDT 6/13/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 23G157-1	DRILLING DATE: 5/17/23	WATER DEPTH: 20 feet
PROJECT: Proposed Industrial Building	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 22 feet
LOCATION: City of Industry, California	LOGGED BY: Michelle Krizek	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
					ASPHALT: 3±-inches Asphaltic Concrete with 4±-inches Aggregate Base							
		24	2.0		FILL: Dark Brown Silty Clay, little to some fine Sand, mottled, trace AC fragments, very stiff-very moist	103	19					EI = 53 @ 0-5 feet
		10	1.0		FILL: Gray Brown to Black Clayey Silt, little Iron Oxide staining, medium stiff-very moist	89	29					
5		11	2.5		FILL: Dark Brown Silty Clay, trace Iron Oxide staining, mottled, medium stiff-very moist	93	27					
		17	4.0		ALLUVIUM: Brown Silty Clay, stiff to very stiff-very moist	103	21					
		22	4.5		@ 9 feet, trace Calcareous veining	101	21					
10												
		14	1.5		@ 13½ feet, trace fine Gravel, little fine Sand		17					
15												
					Brown Silty fine Sand, medium dense-very moist to wet							
		16					23					
20												
					Brown Silty fine to medium Sand, trace coarse Sand, trace to little Clay, dense-very moist to wet							
		41					18					
25												
					Boring Terminated at 25'							

TBL 23G157-1.GPJ SOCALGEO.GDT 6/13/23

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 23G157-1	DRILLING DATE: 5/17/23	WATER DEPTH: 22 feet
PROJECT: Proposed Industrial Building	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 20 feet
LOCATION: City of Industry, California	LOGGED BY: Michelle Krizek	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
					ASPHALT: 2±-inches Asphaltic Concrete with no discernible Aggregate Base							
		11	2.5		FILL: Dark Brown Silty Clay, trace AC fragments, trace fine Sand, very stiff-very moist	105	14					
		13	3.5		@ 3 feet, no AC fragments, stiff	106 115	20 17					
5		17	4.5			110	17					
		13			ALLUVIUM: Brown Silty fine Sand, trace Clay, trace medium Sand, loose to medium dense-damp to very moist	103	14					
		14			@ 9 feet, trace medium to coarse Sand, trace fine Gravel	110	8					
10												
		17					18					
15												
					Brown Silty Clay, little to some fine Sand, stiff-very moist to wet							
		14	1.5				22					
20												
		12	2.5		@ 23½ feet, trace to little fine Sand		18					
25												

Boring Terminated at 25'

TBL 23G157-1.GPJ SOCALGEO.GDT 6/13/23

TEST BORING LOG

JOB NO.: 23G157-1
PROJECT: Proposed Industrial Building
LOCATION: City of Industry, California

DRILLING DATE: 5/17/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: 32 feet
CAVE DEPTH: 47 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
			ASPHALT: 3±-inches Asphaltic Concrete with 4±-inches Aggregate Base							
	6		FILL: Dark Brown Clayey Silt, trace Iron Oxide staining, mottled, medium stiff-moist to very moist		16					
	7	2.5	FiLL: Dark Brown Silty Clay, little to some fine Sand, trace medium to coarse Sand, mottled, medium stiff-moist		13					
5	10	4.5	ALLUVIUM: Dark Brown Silty Clay, trace Calcareous veining, stiff to very stiff-moist		14					
	11	4.5			13					
10										
			Brown fine Sandy Silt, trace medium Sand, medium dense-very moist							
	11				16			58		
15										
			Brown fine Sandy Clay, little Silt, stiff to very stiff-very moist to wet							
	16	2.5			22	46	14	69		
20										
	15	3.0	@ 23½ feet, little Iron Oxide staining		21			60		
25										
	27	2.5			19					
30										
	24	3.0	@ 33½ feet, trace medium Sand		22					

TBL 23G157-1.GPJ SOCALGEO.GDT 6/13/23

BORING NO. B-4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 23G157-1
PROJECT: Proposed Industrial Building
LOCATION: City of Industry, California

DRILLING DATE: 5/17/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

WATER DEPTH: 32 feet
CAVE DEPTH: 47 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Brown fine Sandy Clay, little Silt, stiff to very stiff-very moist to wet							
					Gray Brown Clayey fine Sand to fine Sandy Clay, little Silt, very stiff-wet							
	X	17	4.0				18	51	15	46		
40												
	X	20	3.0		@ 43½ feet, trace Iron Oxide staining		17			56		
45												
					Gray Brown Clayey fine Sand, little Silt, trace Calcareous veining, little Iron Oxide staining, medium dense-wet							
	X	26					15	33	13	34		
50												

Boring Terminated at 50'

TBL 23G157-1.GPJ SOCALGEO.GDT 6/13/23

BORING NO. B-5

JOB NO.: 23G157-1
PROJECT: Proposed Industrial Building
LOCATION: City of Industry, California

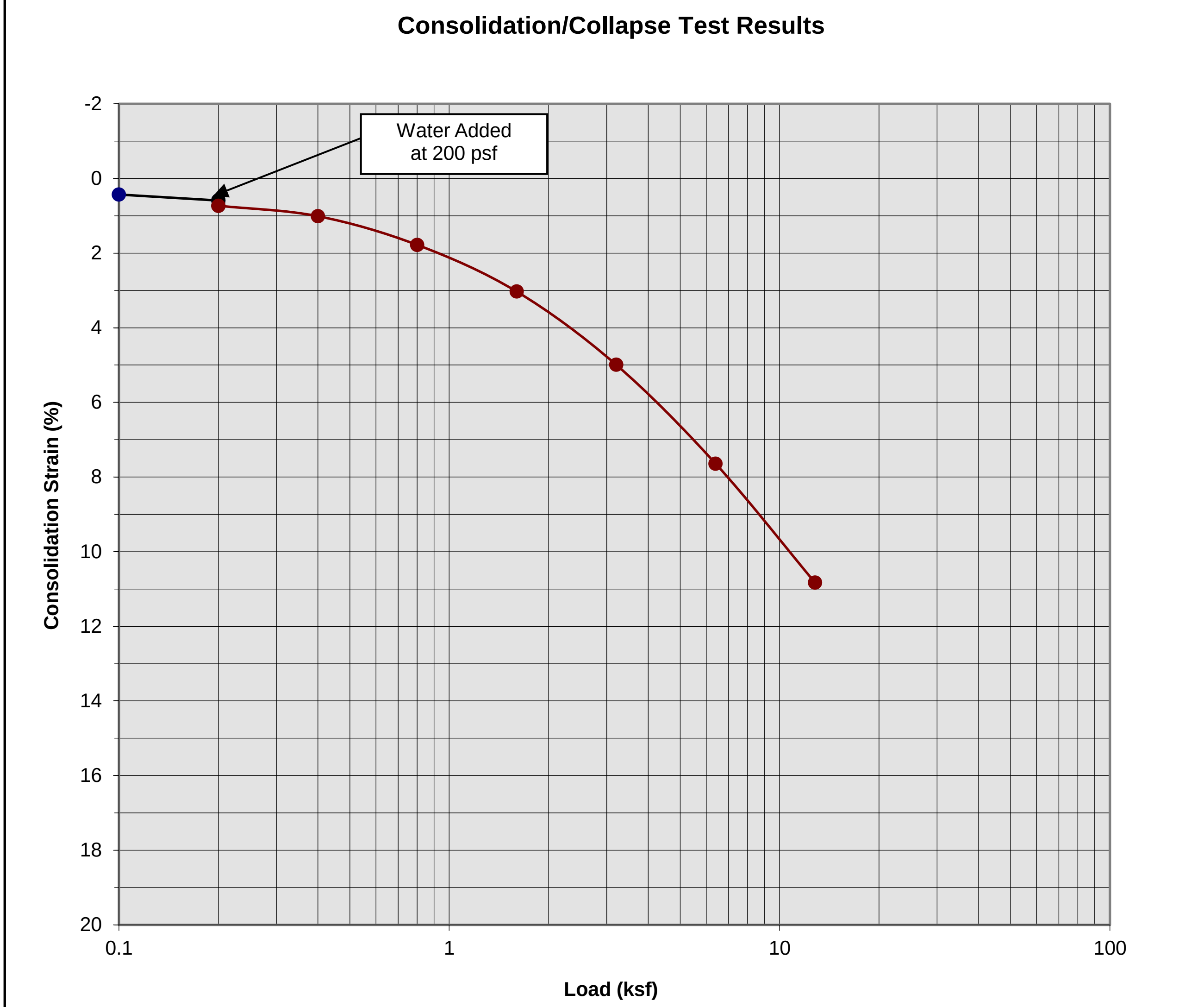
DRILLING DATE: 5/17/23
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Michelle Krizek

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

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: --- MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					FILL: Dark Brown Silty Clay, medium stiff to very stiff-very moist							
	X	6	4.0				17					
	X	5	1.0				20					
5												
					ALLUVIUM: Dark Brown Silty Clay, trace Iron Oxide staining, trace Calcareous veining, stiff to very stiff-very moist							
	X	11	4.5				20					
	X	16	3.0		@ 8½ feet, no Iron Oxide staining, trace fine root fibers		18					
10												
					Brown Silty fine Sand to fine Sandy Silt, medium dense-very moist							
	X	13					16					
15												
					Dark Brown fine Sandy Clay, stiff-moist							
	X	14	2.5				14					
20												
					Boring Terminated at 20'							

TBL 23G157-1.GPJ SOCALGEO.GDT 6/13/23

APPENDIX C



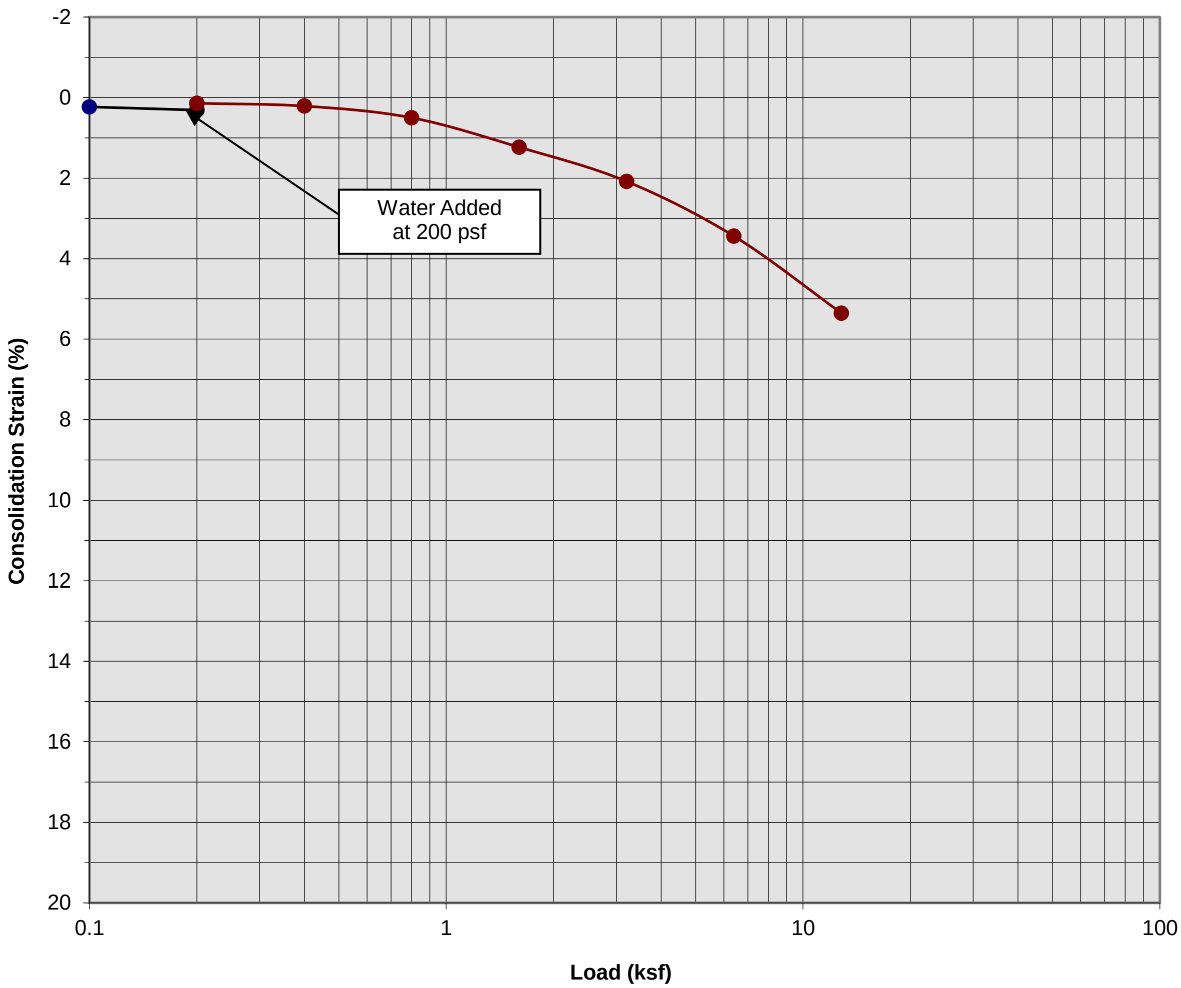
Classification: FILL: Dark Brown Silty Clay, trace fine Sand

Boring Number:	B-3	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	3.5 to 4	Initial Dry Density (pcf)	115.4
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	128.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.14

Proposed Industrial Building
City of Industry, California
Project No. 23G157-1
PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results



Classification: FILL: Dark Brown Silty Clay, trace fine sand

Boring Number:	B-3	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	5 to 6	Initial Dry Density (pcf)	110.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	116.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	-0.17

Proposed Industrial Building
City of Industry, California
Project No. 23G157-1
PLATE C- 2

SoCalGeo
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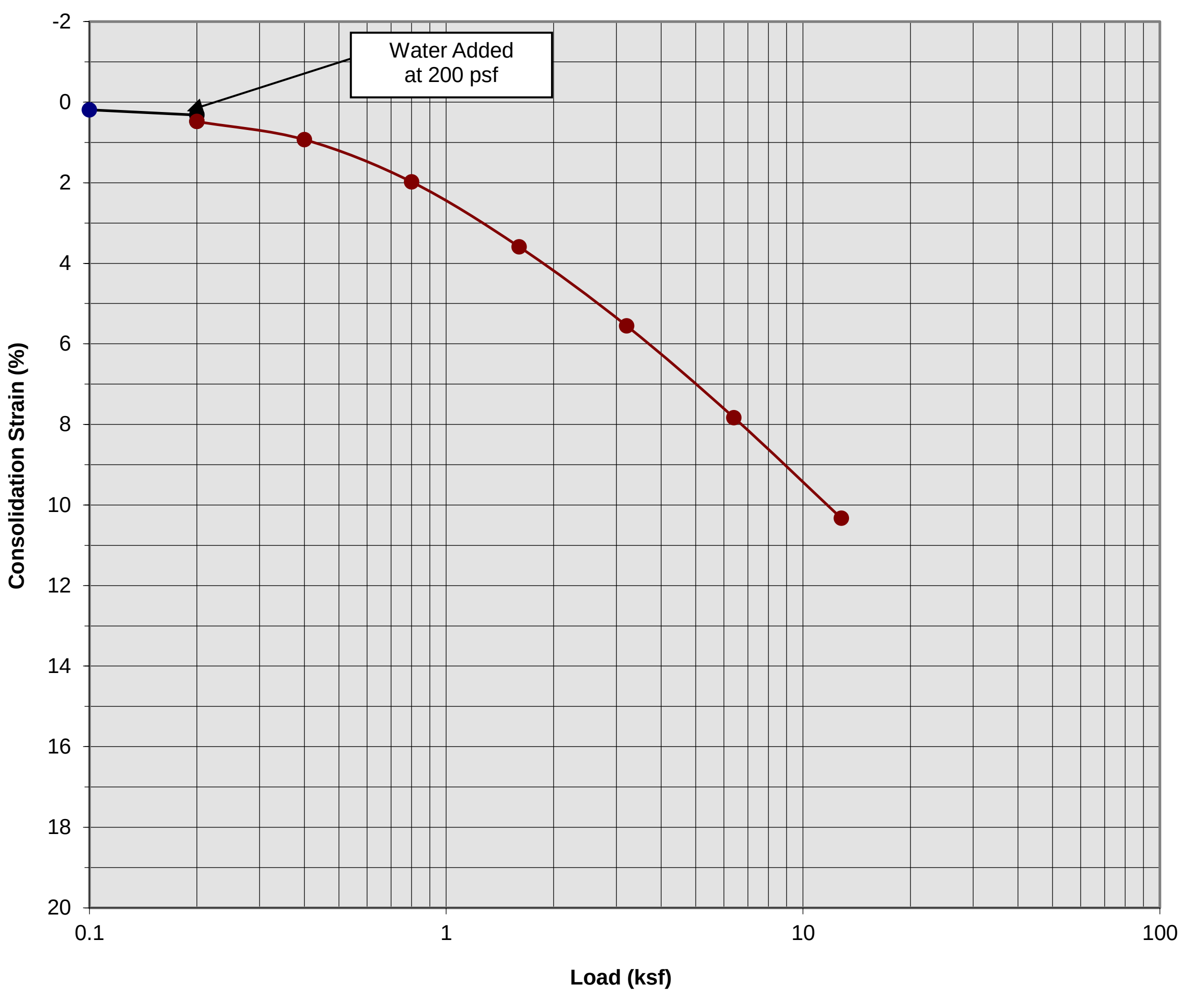
Consolidation/Collapse Test Results



Classification: Brown Silty fine Sand, trace medium Sand, trace Clay

Boring Number:	B-3	Initial Moisture Content (%)	14
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	7 to 8	Initial Dry Density (pcf)	103.2
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	115.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.16

Proposed Industrial Building
City of Industry, California
Project No. 23G157-1
PLATE C- 3

SoCalGeo
SOUTHERN
CALIFORNIA
GEOTECHNICAL
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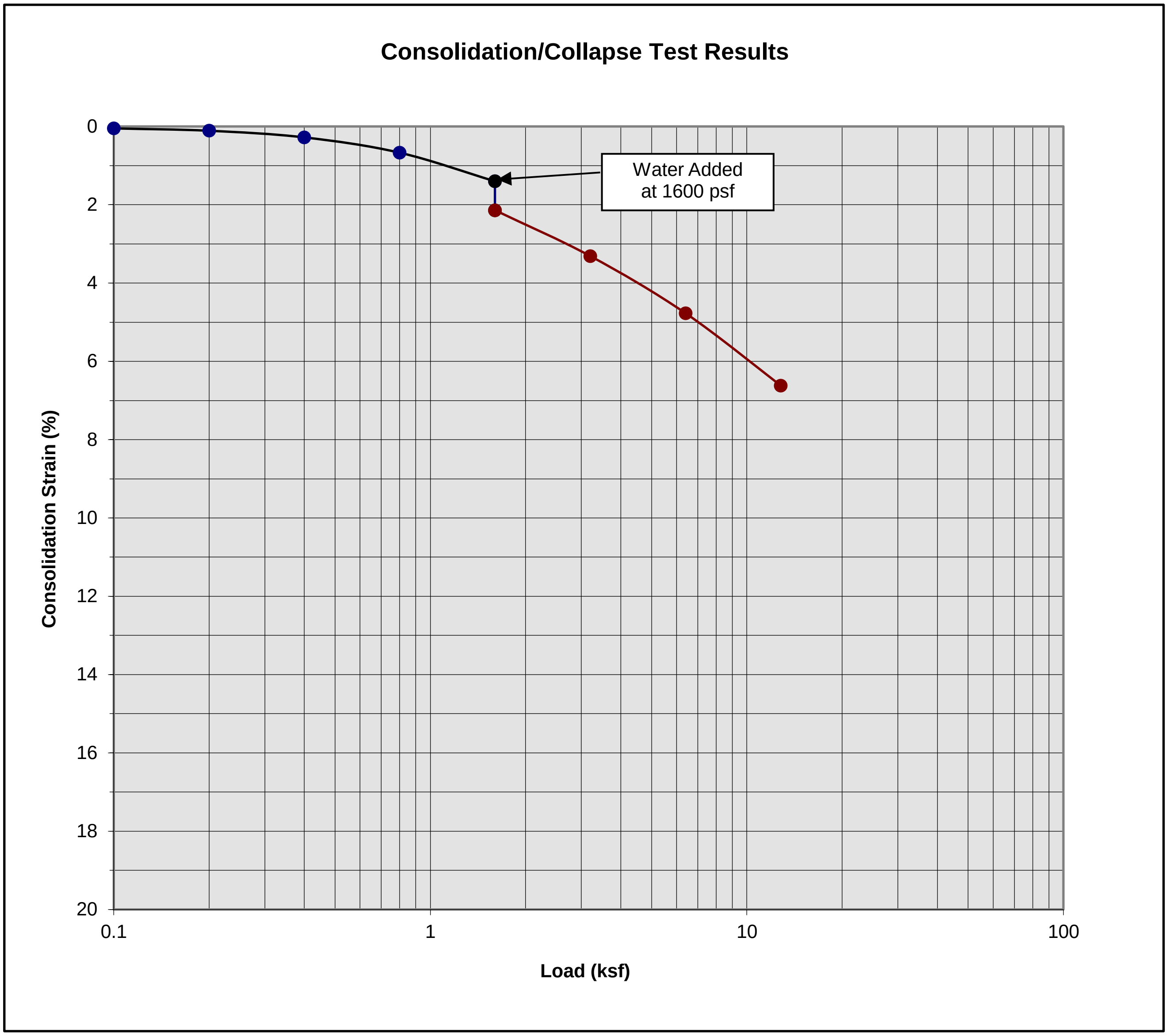

Classification: Brown Silty fine Sand, trace medium to coarse Sand, trace Clay

Boring Number:	B-3	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	9 to 10	Initial Dry Density (pcf)	110.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	117.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.74

Proposed Industrial Building
City of Industry, California
Project No. 23G157-1
PLATE C- 4

SoCalGeo
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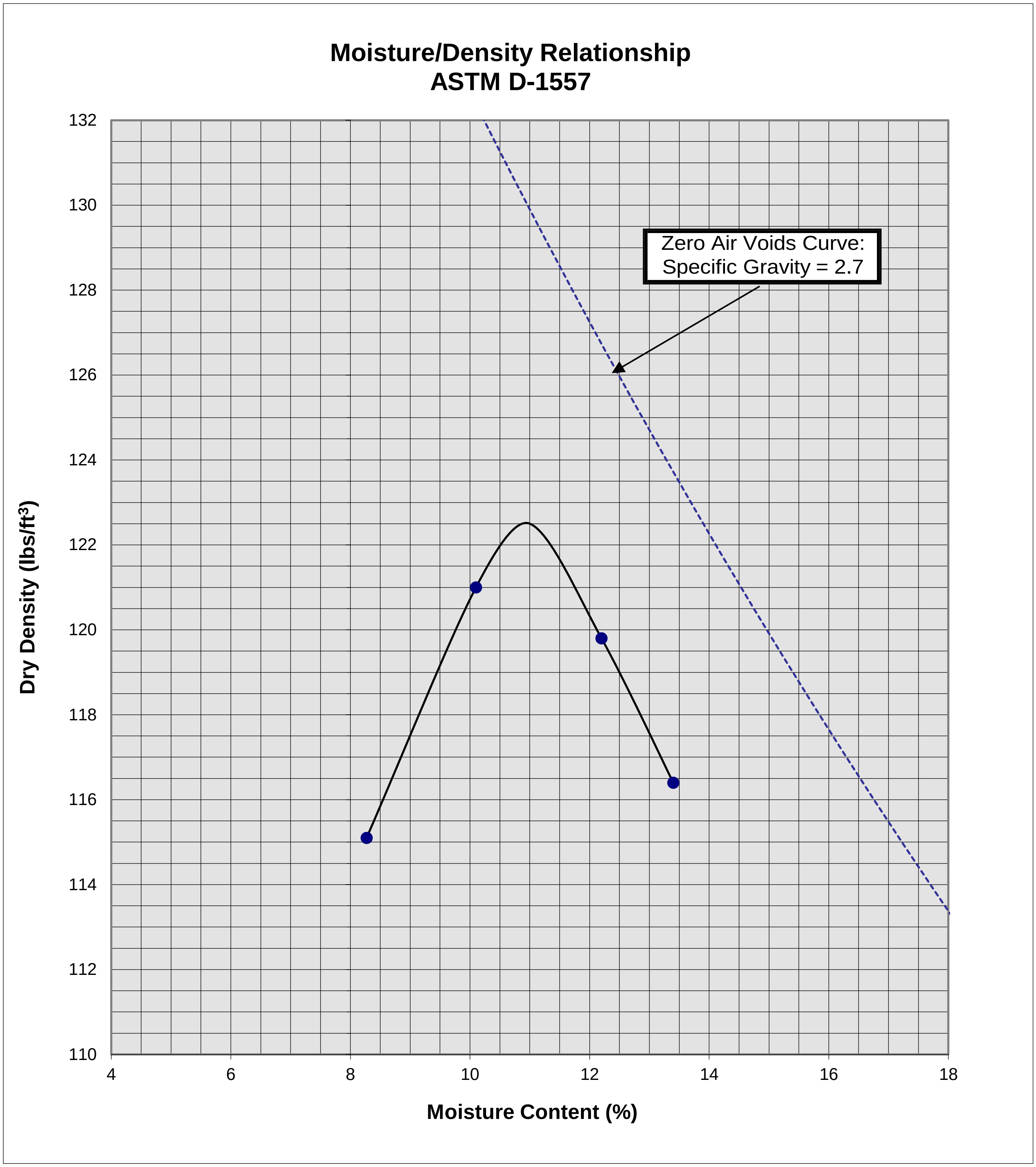

Soil ID Number	B-2 @ 0-5'
Optimum Moisture (%)	11
Maximum Dry Density (pcf)	122.5
Soil Classification	Dark Gray Brown Silty Clay, trace fine to medium Sand

Proposed Industrial Building
City of Industry, California
Project No. 23G157-1
PLATE C-5

SoCalGeo
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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.

Compacted Fills

- Soil materials imported to or excavated on the property may be utilized in the fill, provided each material has been determined to be suitable in the opinion of the Geotechnical Engineer. Unless otherwise approved by the Geotechnical Engineer, all fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as "contaminated," and shall be very low to non-expansive with a maximum expansion index (EI) of 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.

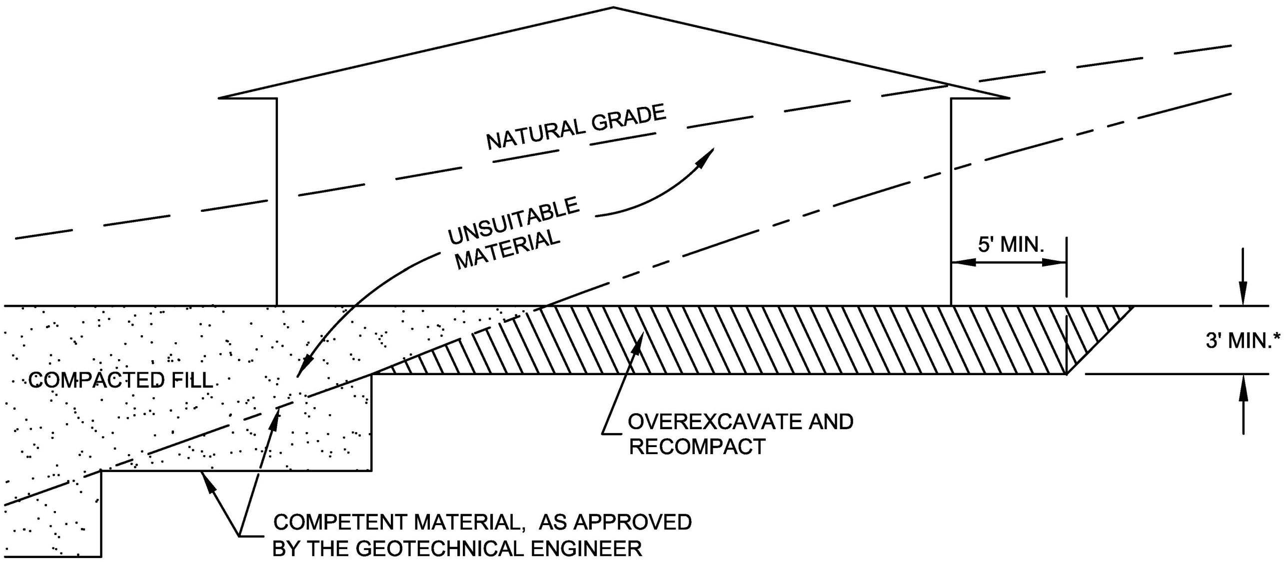
Subdrains

- 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.

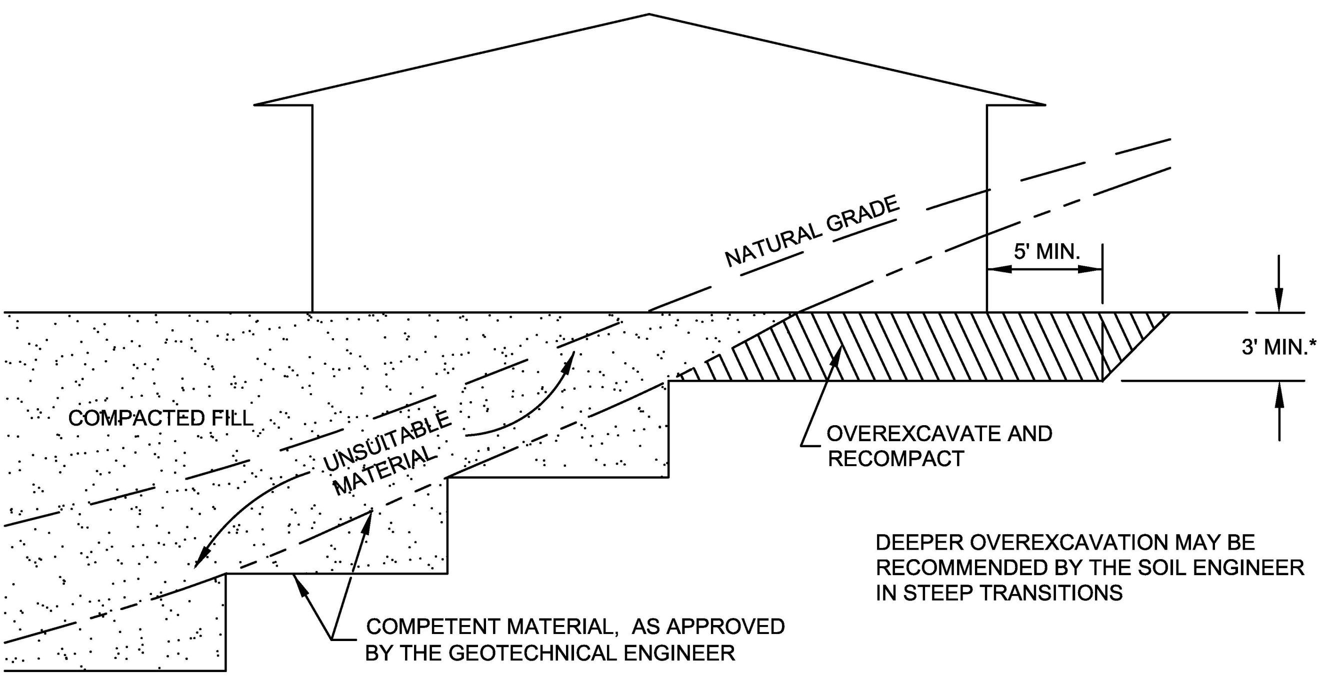
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

- 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

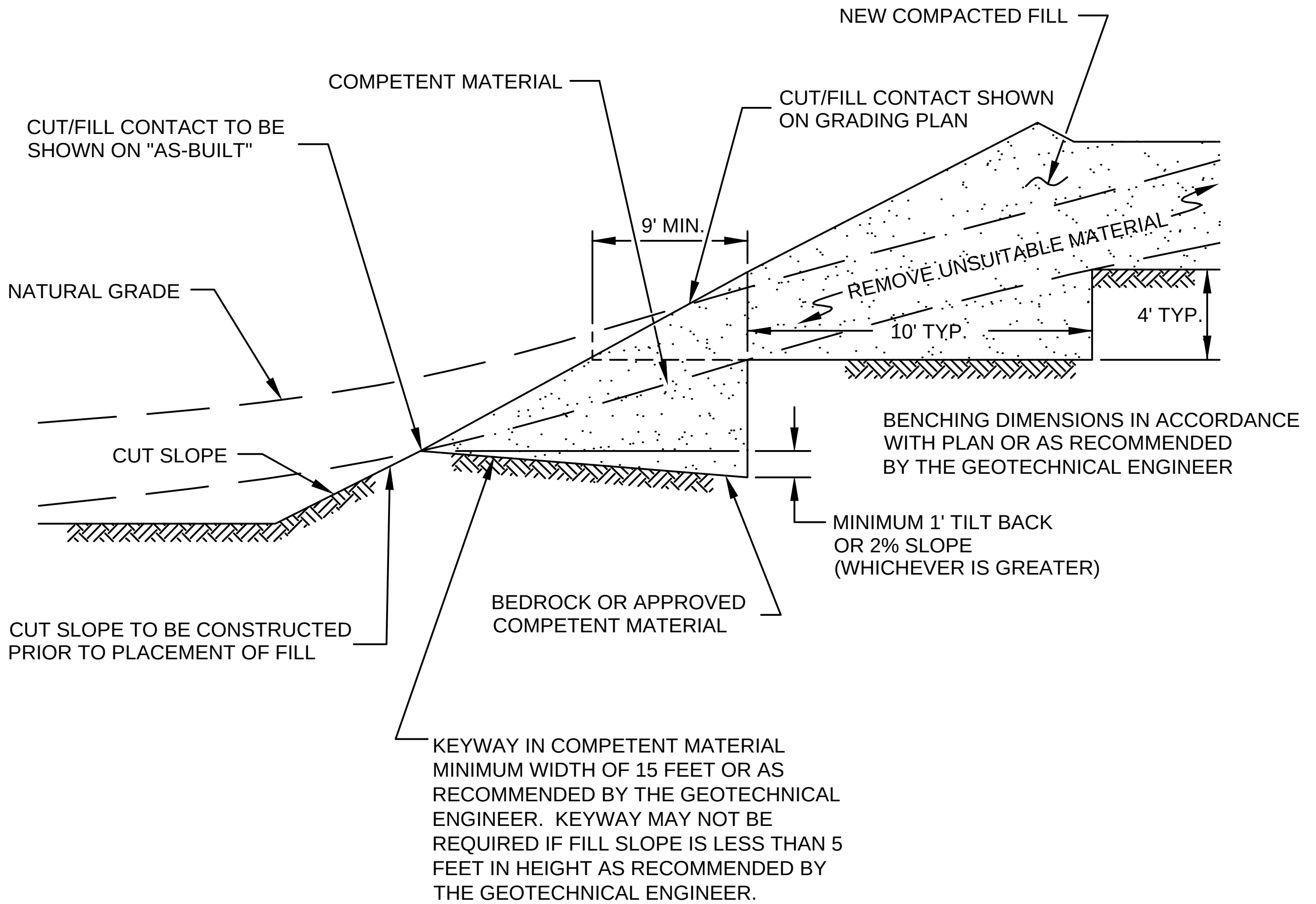
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-1

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



FILL ABOVE CUT SLOPE DETAIL

GRADING GUIDE SPECIFICATIONS

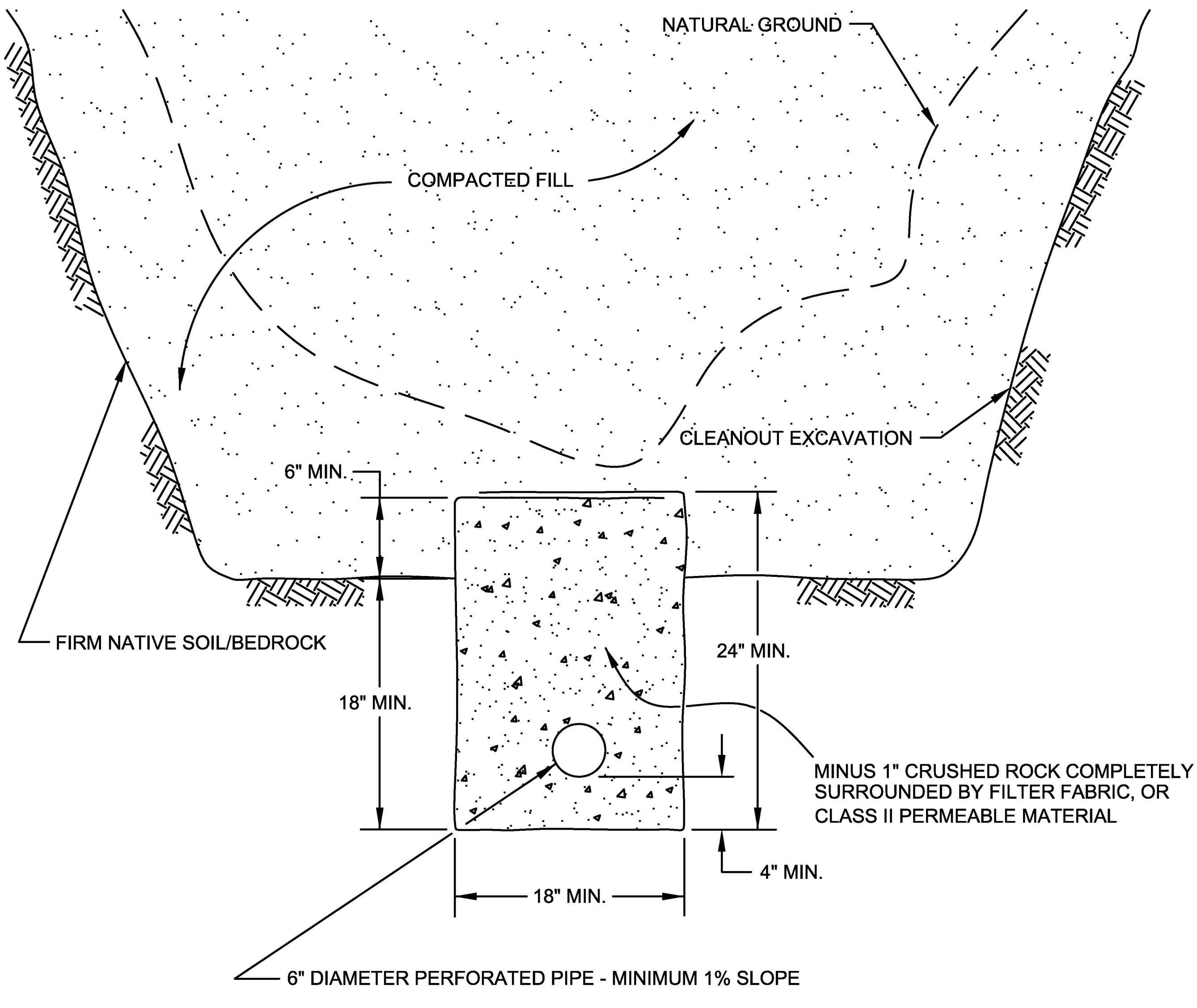
NOT TO SCALE

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

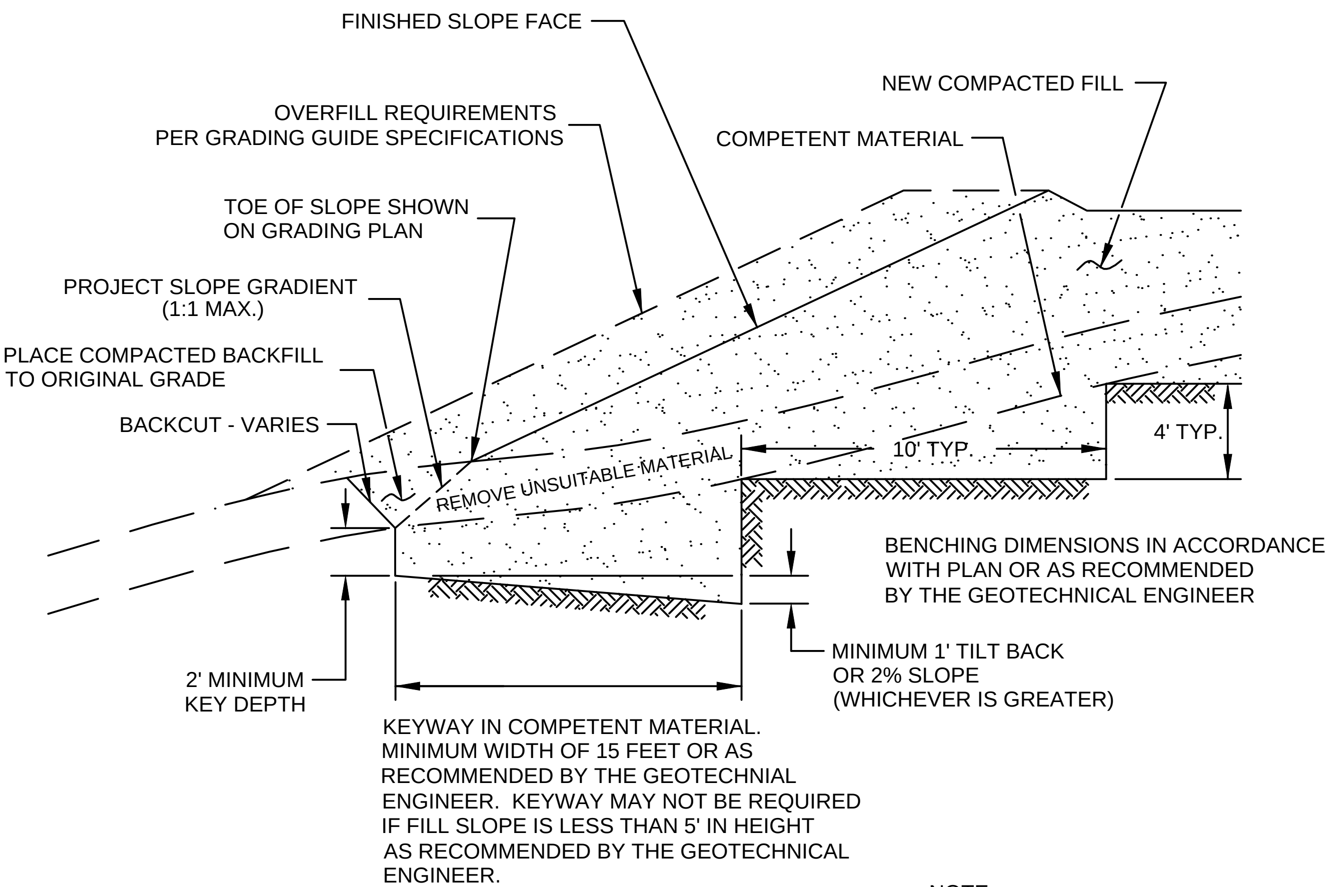
GRADING GUIDE SPECIFICATIONS

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

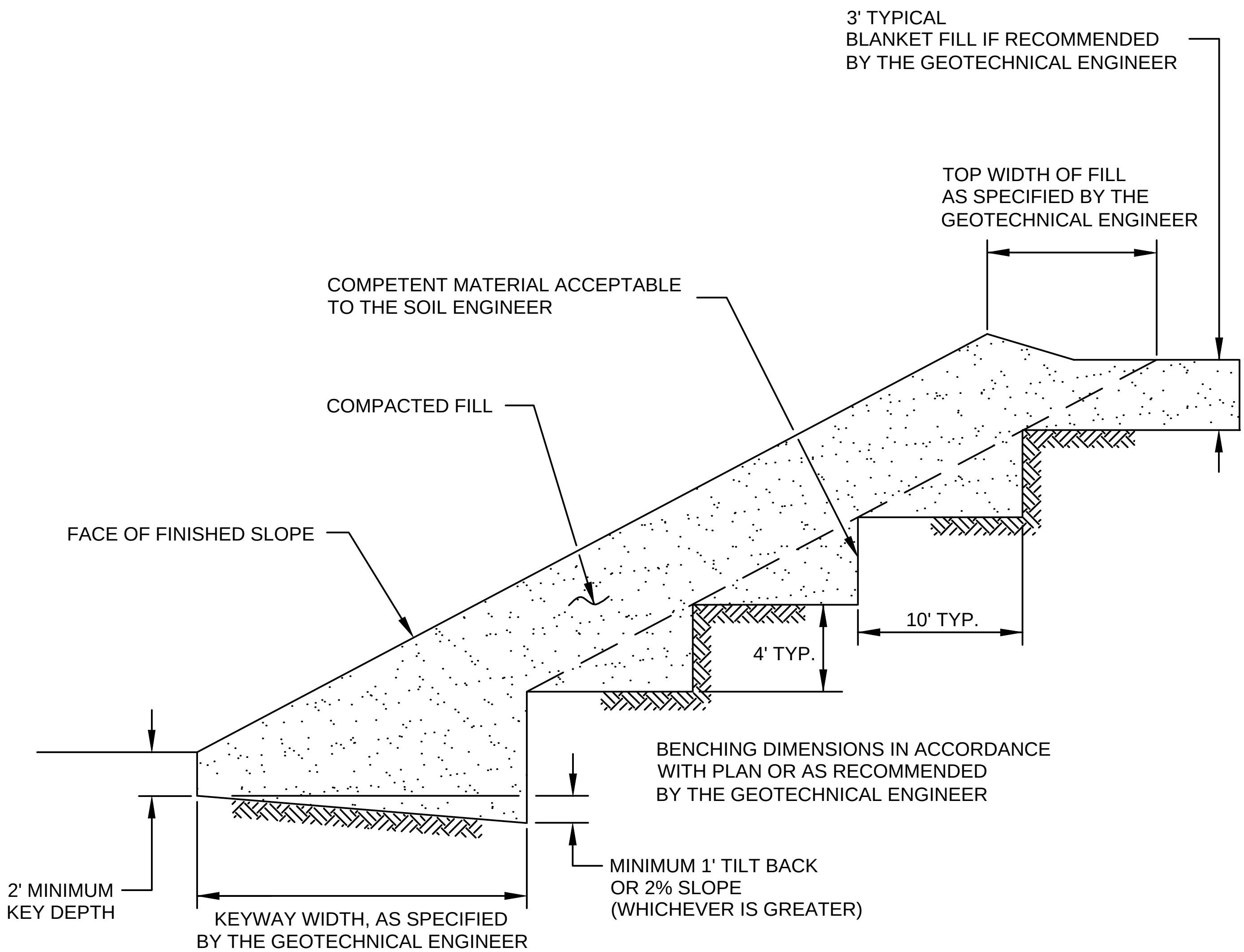
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

GRADING GUIDE SPECIFICATIONS

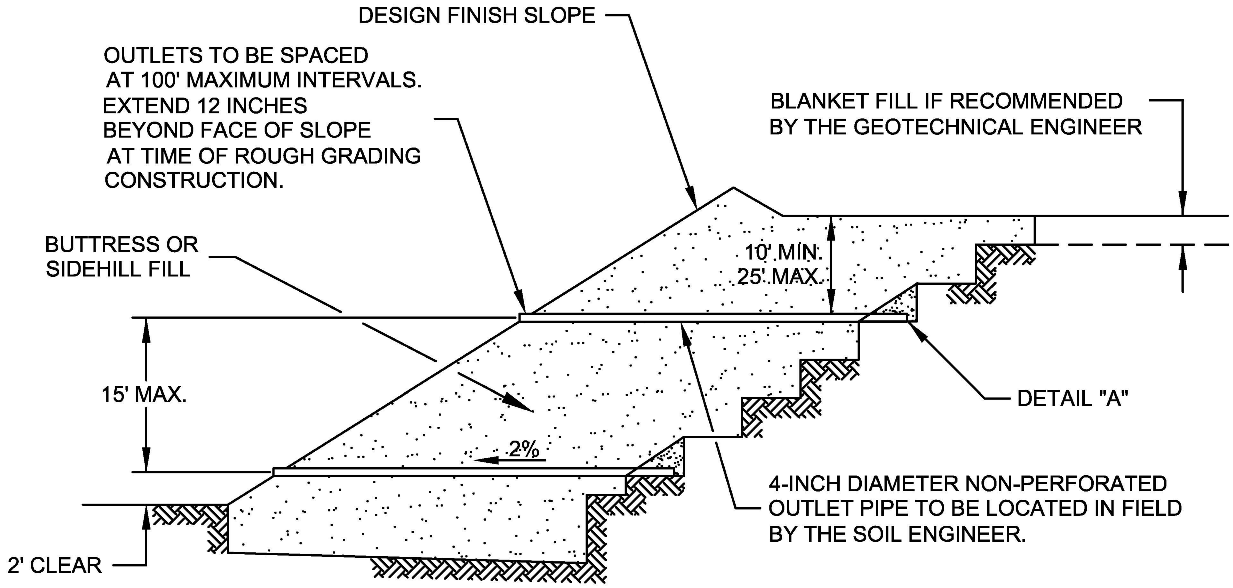
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



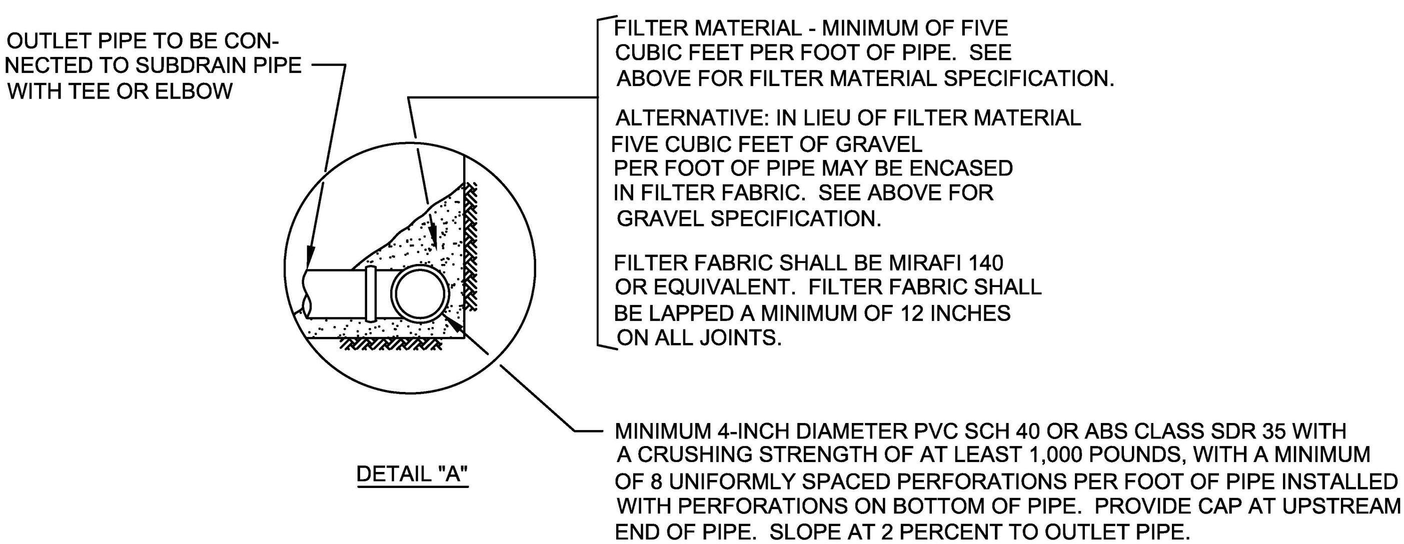
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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



DETAIL "A"

NOTES:

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

SLOPE FILL SUBDRAINS

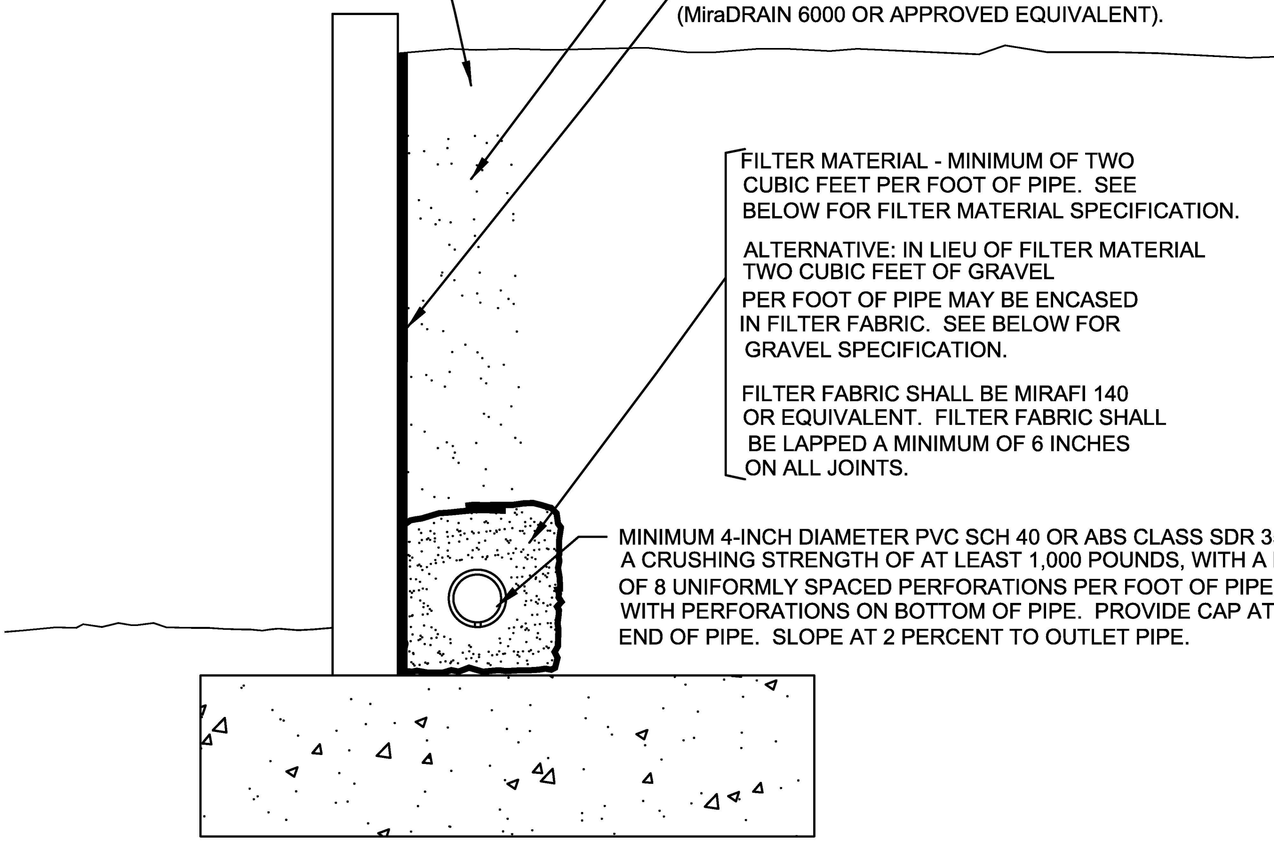
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

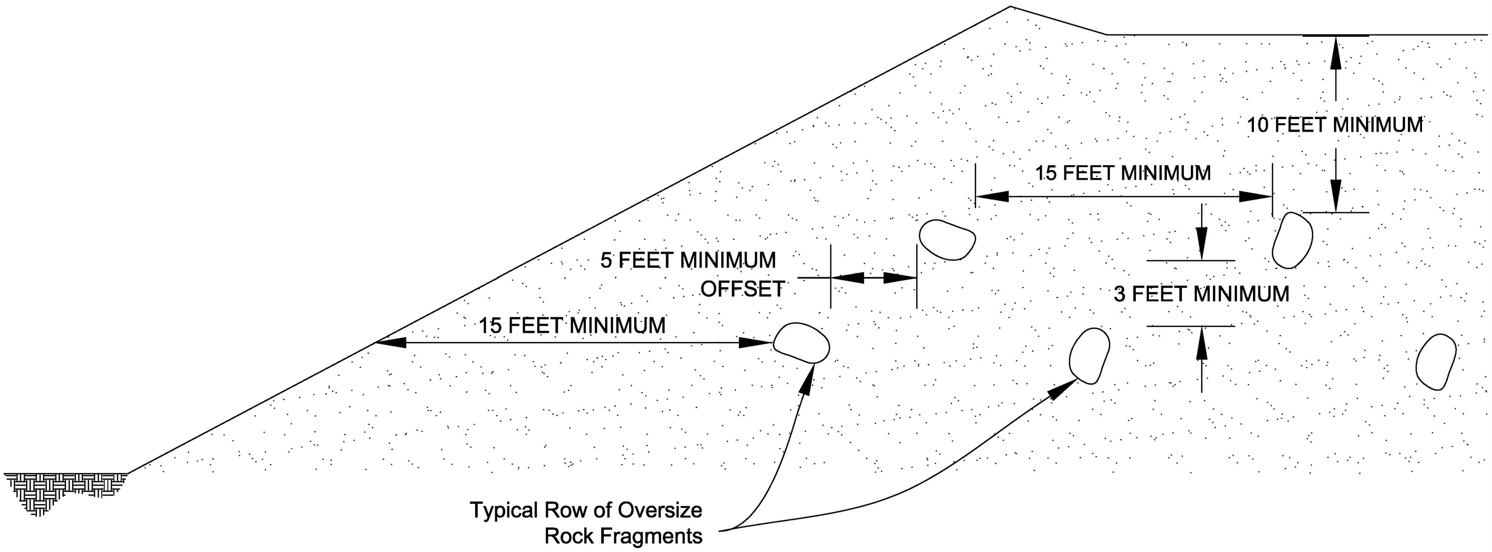
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

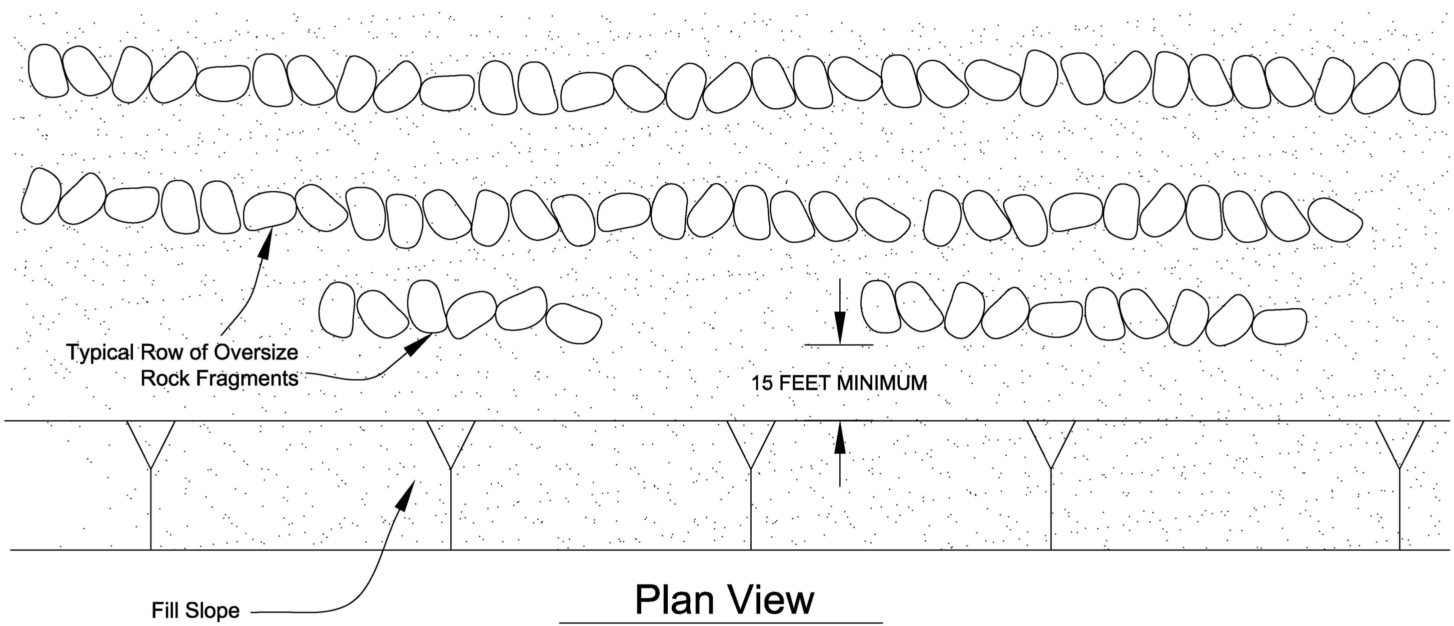
DRAWN: JAS
CHKD: GKM

PLATE D-7

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: PM CHKD: GKM	
PLATE D-8	

APPENDIX E

Latitude, Longitude: 33.997104, -117.912656

Date	6/8/2023, 3:53:12 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.815	MCE_R ground motion. (for 0.2 second period)
S_1	0.64	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.815	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.21	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.78	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.858	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.815	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.01	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.321	Factored deterministic acceleration value. (0.2 second)
S1RT	0.64	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.708	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.824	Factored deterministic acceleration value. (1.0 second)
PGAd	0.933	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.78	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.903	Mapped value of the risk coefficient at short periods

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

SEISMIC DESIGN PARAMETERS - 2022 CBC
PROPOSED INDUSTRIAL BUILDING
CITY OF INDUSTRY, CALIFORNIA

DRAWN: JAH
CHKD: GKM
SCG PROJECT
23G157-1
PLATE E-1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Proposed Industrial Building	Design PGA	0.858 (g)
Project Location	City of Industry, California	Design Magnitude	6.72
Project Number	23G157-1	Historic High Depth to Groundwater	15 (ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	15 (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=6.72)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
14.5	0	15	7.5	22	120		1.3	1.05	1.3	1.28	0.85	42.3	42.3	900	900	900	0.98	1.34	1.1	2.00	2.00	N/A	N/A	Above Water Table
14.5	15	17	16	22	120		1.3	1.05	1.3	1.04	0.85	34.6	34.6	1920	1858	1858	0.94	1.34	1.03	1.03	1.43	0.54	2.63	Nonliquefiable
19.5	17	22	19.5	33	120		1.3	1.05	1.3	1.01	0.95	55.9	55.9	2340	2059	2059	0.92	1.34	1.01	2.00	2.00	0.58	3.42	Nonliquefiable
24.5	22	27	24.5	31	120		1.3	1.05	1.3	0.98	0.95	51.0	51.0	2940	2347	2347	0.89	1.34	0.97	2.00	2.00	0.62	3.20	Nonliquefiable
29.5	27	29.5	28.25	20	120	10	1.3	1.05	1.3	0.94	0.95	31.6	32.7	3390	2563	2563	0.87	1.33	0.95	0.72	0.92	0.64	1.43	Nonliquefiable
29.5	29.5	32	30.75	20	120	57	1.3	1.05	1.3	0.92	0.95	31.2	36.8	3690	2707	2707	0.86	1.34	0.93	1.66	2.00	0.65	3.07	Nonliquefiable
34.5	32	37	34.5	18	120	59	1.3	1.05	1.283	0.90	1	28.3	33.9	4140	2923	2923	0.83	1.34	0.92	0.89	1.09	0.66	1.66	Nonliquefiable
39.5	37	42	39.5	41	120		1.3	1.05	1.3	0.94	1	68.4	68.4	4740	3211	3211	0.80	1.34	0.87	2.00	2.00	0.66	3.02	Nonliquefiable
44.5	42	47	44.5	9	120	3	1.3	1.05	1.103	0.76	1	10.3	10.3	5340	3499	3499	0.77	1.06	0.95	0.12	0.12	0.66	0.18	**Liquefiable**
49.5	47	50	48.5	32	120		1.3	1.05	1.3	0.87	1	49.5	49.5	5820	3730	3730	0.75	1.34	0.83	2.00	2.00	0.65	3.07	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	City of Industry, California
Project Number	23G157-1
Engineer	JLL

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 conte	$(N_1)_{60\text{-}CS}$	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)			
14.5	0	15	7.5	42.3	0.0	42.3	N/A	0.01	-0.98	0.00	15.00		0.000		0.00	Above Water Table
14.5	15	17	16	34.6	0.0	34.6	2.63	0.02	-0.41	0.00	2.00		0.000		0.00	Nonliquefiable
19.5	17	22	19.5	55.9	0.0	55.9	3.42	0.00	-2.08	0.00	5.00		0.000		0.00	Nonliquefiable
24.5	22	27	24.5	51.0	0.0	51.0	3.20	0.00	-1.67	0.00	5.00		0.000		0.00	Nonliquefiable
29.5	27	29.5	28.25	31.6	1.1	32.7	1.43	0.03	-0.27	0.00	2.50		0.000		0.00	Nonliquefiable
29.5	29.5	32	30.75	31.2	5.6	36.8	3.07	0.02	-0.56	0.00	2.50		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	28.3	5.6	33.9	1.66	0.03	-0.36	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	68.4	0.0	68.4	3.02	0.00	-3.15	0.00	5.00		0.000		0.00	Nonliquefiable
44.5	42	47	44.5	10.3	0.0	10.3	0.18	0.46	0.91	0.46	5.00		0.037		2.20	**Liquefiable**
47	49.5	50	48.5	49.5	0.0	49.5	3.07	0.00	-1.54	0.00	0.50		0.000		0.00	Nonliquefiable
											Total Deformation (in)				2.20	

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	Design PGA	0.858 (g)
Project Location	City of Industry, California	Design Magnitude	6.72
Project Number	23G157-1	Historic High Depth to Groundwater	15 (ft)
Engineer	JLL	Depth to Groundwater at Time of Drilling	32 (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=6.72)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
14.5	0	15	7.5	11	120		1.3	1.05	1.224	1.43	0.85	22.4	22.4	900	900	900	0.98	1.17	1.1	0.24	0.31	N/A	N/A	Above Water Table
14.5	15	17	16	11	120	58	1.3	1.05	1.154	1.04	0.85	15.4	21.0	1920	1858	1920	0.94	1.15	1.02	0.22	0.26	0.54	0.47	**Liquefiable**
19.5	17	22	19.5	16	120	69	1.3	1.05	1.25	0.96	0.95	25.0	30.6	2340	2059	2340	0.92	1.29	1	N/A	N/A	N/A	N/A	Non-Liq: PI>18
24.5	22	27	24.5	15	120	60	1.3	1.05	1.206	0.88	0.95	20.6	26.2	2940	2347	2940	0.89	1.22	0.98	N/A	N/A	N/A	N/A	Non-Liq: PI>18
29.5	27	32	29.5	27	120		1.3	1.05	1.3	0.85	0.95	38.9	38.9	3540	2635	3540	0.86	1.34	0.93	2.00	2.00	0.65	3.09	Nonliquefiable
34.5	32	37	34.5	24	120		1.3	1.05	1.3	0.81	1	34.5	34.5	4140	2923	3984	0.83	1.34	0.92	1.00	1.23	0.66	1.87	Nonliquefiable
39.5	37	42	39.5	17	120	46	1.3	1.05	1.215	0.76	1	21.5	27.1	4740	3211	4272	0.80	1.24	0.92	N/A	N/A	N/A	N/A	Non-Liq: PI>18
44.5	42	47	44.5	20	120	56	1.3	1.05	1.264	0.76	1	26.4	32.0	5340	3499	4560	0.77	1.32	0.89	N/A	N/A	N/A	N/A	Non-Liq: PI>18
49.5	47	50	48.5	26	120	34	1.3	1.05	1.3	0.79	1	36.5	42.0	5820	3730	4790	0.75	1.34	0.83	2.00	2.00	0.65	3.07	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	City of Industry, California
Project Number	23G157-1
Engineer	JLL

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 conte	$(N_1)_{60\text{-}CS}$	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)			
14.5	0	15	7.5	22.4	0.0	22.4	N/A	0.12	0.39	0.00	15.00		0.000		0.00	Above Water Table
14.5	15	17	16	15.4	5.6	21.0	0.47	0.14	0.47	0.14	2.00		0.022		0.53	**Liquefiable**
19.5	17	22	19.5	25.0	5.6	30.6	N/A	0.04	-0.13	0.00	5.00		0.000		0.00	Non-Liq: PI>18
24.5	22	27	24.5	20.6	5.6	26.2	N/A	0.08	0.16	0.00	5.00		0.000		0.00	Non-Liq: PI>18
29.5	27	32	29.5	38.9	0.0	38.9	3.09	0.01	-0.72	0.00	5.00		0.000		0.00	Nonliquefiable
34.5	32	37	34.5	34.5	0.0	34.5	1.87	0.02	-0.40	0.00	5.00		0.000		0.00	Nonliquefiable
39.5	37	42	39.5	21.5	5.6	27.1	N/A	0.07	0.10	0.00	5.00		0.000		0.00	Non-Liq: PI>18
44.5	42	47	44.5	26.4	5.6	32.0	N/A	0.04	-0.22	0.00	5.00		0.000		0.00	Non-Liq: PI>18
49.5	47	50	48.5	36.5	5.5	42.0	3.07	0.01	-0.96	0.00	3.00		0.000		0.00	Nonliquefiable
											Total Deformation (in)				0.53	

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)

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