

**GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL BUILDING**

5088 Edison Avenue
Chino, California
for
ProLogis



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

August 4, 2023

ProLogis
17777 Center Court Drive North, Suite 100
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SOUTHERN
CALIFORNIA
GEOTECHNICAL
A California Corporation

Attention: Mr. Laerte Fanucchi
Director, Development

Project No.: **23G162-1**

Subject: **Geotechnical Investigation**
Proposed Industrial Building
5088 Edison Avenue
Chino, California

Gentlemen:

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.

Joseph Lozano Leon
Staff Engineer

Robert G. Trazo, GE 2655
Principal Engineer



Distribution: (1) Addressee

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1.0 EXECUTIVE SUMMARY

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

Geotechnical Design Considerations

- The near-surface soils encountered at the boring locations from the previous and current studies consist of artificial fill soils and native alluvium. The artificial fill soils extend to depths of 1½ to 4½± feet below the ground surface.
- The fill soils and near-surface alluvial soils possess varying strengths. The existing fill soils are considered to represent undocumented fill.
- The near-surface native soils consist of loose to medium dense silty sands and sandy silts. Based on the results of laboratory testing, these soils possess variable densities. Neither the undocumented fill soils nor the near-surface native alluvium, in their present condition, are considered suitable to support the foundations loads of the new building.

Site Preparation Recommendations

- The site plan provided to our office indicates that the existing structures and pavements at the subject site will be demolished in order to facilitate the construction of the proposed development. Demolition should include all foundations, floor slabs, pavements, utilities and any other subsurface improvements that will not remain in place with the new development. Debris resultant from demolition should be disposed of off-site. Alternatively, existing asphalt and concrete materials may be crushed into miscellaneous base (CMB) and re-used at the site. Asphalt and concrete debris resultant from demolition may also be crushed to 2 to 4-inch particle size and used as a subgrade stabilization material. **Mixing concrete and asphalt debris with the on-site clayey soils is not recommended.**
- Remedial grading is recommended to be performed within the proposed building area in order to remove all of the undocumented fill soils in their entirety, the upper portion of the near-surface native alluvial soils, and any soils disturbed during the demolition process. The soils within the proposed building area should be overexcavated to a depth of 5 feet below existing grade and to a depth of at least 4 feet below proposed building pad subgrade elevations.
- The depth of overexcavation should also be sufficient to remove any existing fill soils. The proposed foundation influence zones should be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade.
- The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill placed below the foundation bearing grade, whichever is greater.
- After overexcavation has been completed, the subgrade soils should be evaluated by the geotechnical engineer to identify additional soils that should be overexcavated. The resulting subgrade 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. The previously excavated soils may then be replaced as compacted structural fill.

- The new pavement and flatwork subgrade soils are recommended to be scarified to a depth of 12± inches, moisture conditioned and recompactd 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.
- 2,500 lbs/ft² maximum allowable soil bearing pressure.
- Reinforcement consisting of at least four (4) No. 5 rebars (2 top and 2 bottom) in strip footings due to the presence of expansive soils. Additional reinforcement may be necessary for structural considerations.

Building Floor Slab Design Recommendations

- Conventional Slabs-on-Grade: minimum 6-inch thickness.
- Modulus of Subgrade Reaction: k = 100 psi/in.
- Minimum slab reinforcement: Reinforcement of the floor slab should consist of No. 3 bars at 18-inches on center in both directions due to the presence of expansive soils. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.

Pavement Design Recommendations

ASPHALT PAVEMENTS (R=30)					
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	6	8	10	11	13
Compacted Subgrade	12	12	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30)				
Materials	Thickness (inches)			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 23P299 dated June 22, 2023. The scope of services included the review of the previous geotechnical investigation performed within the overall project site, and 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. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The subject site is located on the north side of Edison Avenue, 615± feet east of Monte Vista Avenue in Chino, California. The site is also referenced by the street address 5088 Edison Avenue. The site is bounded to the north and west by railroad easements, to the south by Edison Avenue, and to the east by a commercial/industrial development. The general location of the site is illustrated on the Site Location Map, included as Plate 1 in Appendix A of this report.

The site consists of an irregular-shaped property, totaling 18.10± acres in size. The site is currently developed with four (4) commercial/industrial buildings. All of the buildings are single story structures of concrete tilt-up construction, presumably supported on conventional shallow foundation systems with concrete slab-on-grade floors. These buildings are utilized by RedBuilt to manufacture engineered wood products. The northern building has a footprint of 28,555± ft², the central building has a footprint of 29,560± ft², the southern building has a footprint of 36,290± ft², and the southern-most building has a footprint of 5,373± ft².

The northern portion of the site is currently being used for trailer parking. Ground surface cover consists of asphaltic concrete (AC) pavements, aggregate base, and limited areas of Portland cement concrete (PCC) pavements. AC pavements are located in the southern two-thirds of the site and were observed to be in fair to poor condition with areas of moderate to severe cracking. The aggregate base areas are located in the northern one-third of the site.

Detailed topographic information was not available at the time of this report. Based on elevations obtained from Google Earth, and visual observations made at the time of the subsurface investigation, the overall site topography slopes downward to the south-southwest at gradients of 1 to 1½± percent

3.2 Proposed Development

A conceptual site plan prepared by HPA, Inc., has been provided to our office by the client. Based on this plan, the subject site will be developed with a 394,230± ft² industrial building, located in the eastern area of the site. Dock-high doors will be constructed along a portion of the west side of the building. The proposed building is expected to be surrounded by AC pavements in the parking and drive areas, PCC pavements in the loading dock area, and concrete flatwork and landscaped planters throughout the site.

Detailed structural information has not been provided. We assume that the new building will be a single-story structure of tilt-up concrete construction, supported on a conventional shallow foundation system 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 basements or crawl spaces, are expected to be included in the proposed development. Based on the assumed topography, cuts and fills of up to 5 to 7± feet are expected to be necessary to achieve the proposed site grades. It should be noted that this estimate does not include any remedial grading, recommendations for which are presented in a subsequent section of this report.

3.3 Previous Study

Southern California Geotechnical, Inc. (SCG) previously performed a geotechnical investigation for the previously proposed development within the overall site. The previously proposed development consisted of a 394,230± ft² industrial building, located in the western area of the site. The results of this investigation are presented in the report referenced as follows:

Geotechnical Investigation, Proposed Commercial/Industrial Building, 5088 Edison Avenue, Chino, California, prepared by Southern California Geotechnical, Inc. (SCG), prepared for ProLogis, SCG Project No. 12G120-1, dated March 29, 2012.

As part of this investigation, eight (8) borings (identified as Boring Nos. B-1 through B-8) were advanced to depths of 5 to 50± feet below the previously existing site grades. The approximate locations of the previous borings are indicated on the Boring Location Plan, included as Plate 2 in Appendix A of this report.

Most of the borings were drilled through pavements that consisted of 3½ to 5½± inches of AC with no discernible aggregate base. The remaining borings were drilled through 2± inches of aggregate base material which was exposed at the ground surface. Artificial fill soils were encountered beneath the existing pavements and aggregate base at all of the boring locations, extending to depths of 1½ to 3± feet. The fill soils generally consisted of loose to medium dense, mottled, silty sands. Native alluvial soils were encountered beneath the fill soils at all of the boring locations. The alluvial soils consisted of loose to medium dense silty sands and sandy silts extending to depths of 8½ to 12± feet. Beneath these soils, and extending to depths of 17 to 27± feet, the alluvial soils consisted of soft to stiff silty clays. Between 27 and 37± feet, the alluvial soils generally consisted of sandy silts and clayey silts. Beneath these soils, and extending to the maximum depth explored of 50± feet, the alluvial soils generally consist of medium stiff silty clays and clayey silts.

Laboratory testing performed by SCG for the previous study included consolidation/collapse testing, maximum density/optimum moisture, soluble content, and expansion index testing.

The foundations were recommended to be designed for a maximum allowable soil bearing pressure of 2,500 lbs/ft². Excerpts from the previous study, including the boring logs, along with the results of laboratory testing, are included in Appendix F of this report.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this phase of the project consisted of two (2) borings (identified as Boring Nos. B-9 and B-10), advanced to a depth of 50± feet below the existing site grades. Both borings were logged during drilling by a member of our staff. As indicated in Section 3.3 of this report, a total of ten (10) borings were drilled for the overall project site. Both boring locations were cleared by a private geophysical testing company prior to drilling. It should be noted that the borings from the previous investigation were performed in 2012. However, based on recent visual observation and comparison of site photographs from 2012 and now, the surface grades are considered to be generally similar.

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

The approximate locations of the borings from this investigation 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

Boring Nos. B-1, B-4, B-5, B-6, and B-8 were drilled through pavements that consisted of 3½ to 5½± inches of AC with no discernible aggregate base. Boring Nos. B-2, B-3, B-7, and B-10 were drilled through 1 to 2± inches of aggregate base material which was exposed at the ground surface. Boring No. B-9 was drilled through asphalt consisting of 3½± inches of AC, underlain by 4½± inches of aggregate base.

Artificial Fill

Artificial fill soils were encountered beneath the existing pavements and aggregate base at all of the boring locations, extending to depths of 1½ to 4½± feet. The fill soils generally consist of

loose to medium dense silty sands and sandy silts with varying clay content, and occasional very stiff sandy clays. The fill soils possess a disturbed mottled appearance resulting in their classification as artificial fill.

Alluvium

Native alluvium was encountered beneath the artificial fill soils at all boring locations, extending to at least the maximum depth explored of 50± feet below existing site grades. The alluvial soils consist of loose to medium dense silty sands and sandy silts, with occasional medium dense sands and very stiff clayey silts, extending to depths of 8½ to 12± feet. Beneath these soils, and extending to depths of 17 to 27± feet, the alluvial soils generally consist of soft to stiff silty clays, clayey silts and sandy clays. At deeper depths and extending to at least the maximum depth explored of 50± feet, the alluvium generally consists of medium stiff to stiff sandy clays, silty clays, and clayey silts. Boring No. B-10 encountered a stratum of medium dense silty sands to sandy silts at depths of 32 to 37± feet.

Groundwater

Free water was encountered during drilling at Boring No. B-9 at a depth of 40± feet below the ground surface. Delayed groundwater level readings were taken at this boring 3 hours after the drilling was completed and the augers removed. These readings indicate that the groundwater was at a depth of 32± feet. Based on these observations and the moisture content of the recovered soil samples, the static groundwater table is considered to have been present at a depth as shallow as 32± feet below the existing grades at the time of the subsurface exploration.

As a part of our research, we reviewed available groundwater data in order to determine groundwater levels for the site. Recent water level data was obtained from the California Department of Water Resources website, <https://wdl.water.ca.gov/waterdatalibrary/>. One monitoring well is located as close as 2,770± feet south of the site. Water level readings within this monitoring well indicate a high groundwater level of 45± feet below the ground surface in April 2021.

Water level data was also obtained from the California State Water Resources Control Board, GeoTracker, website, <http://geotracker.waterboards.ca.gov/>. Several monitoring wells are located as close as 850± feet northwest of the site. Water level readings within these monitoring wells indicate a high groundwater level of 57± feet below the ground surface in June 2005.

5.0 LABORATORY TESTING

The soil samples recovered from the subsurface exploration were returned to our laboratory for further testing to determine selected physical and engineering properties of the soils. The tests are briefly discussed below. It should be noted that the test results are specific to the actual samples tested, and variations could be expected at other locations and depths. Test results from the previous study are denoted using the project number 12G120-1. The test results from the previous study are included in Appendix F.

Classification

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

Density and Moisture Content

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

Consolidation

Selected soil samples were tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-9 in Appendix C of this report. Additional consolidation test results performed during the previous study are included in Appendix F.

Maximum Dry Density and Optimum Moisture Content

A representative bulk sample has been tested for its maximum dry density and optimum moisture content. The results have been obtained using the Modified Proctor procedure, per ASTM D-1557 and are presented on Plate C-10 in Appendix C of this report. This test is generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional maximum dry density and optimum moisture content test results performed during the previous study are included in Appendix F. Additional testing of other soil types or soil mixes may be necessary at a later date.

Expansion Index

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

<u>Sample Identification</u>	<u>Expansion Index</u>	<u>Expansive Potential</u>
B-5 @ 0 to 5 feet (12G120-1)	10	Very Low
B-9 @ 1 to 5 feet	29	Low

Soluble Sulfates

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

<u>Sample Identification</u>	<u>Soluble Sulfates (%)</u>	<u>Severity</u>	<u>Class</u>
B-5 @ 0 to 5 feet (12G120-1)	0.005	Not Applicable	S0
B-9 @ 1 to 5 feet	0.005	Not Applicable	S0
B-10 @ 1 to 5 feet	0.005	Not Applicable	S0

Corrosivity Testing

Representative samples of the near-surface soils have been submitted to a subcontracted corrosion engineering laboratory for determination of electrical resistivity, pH, and chloride concentrations. The resistivity of the soils is a measure of their potential to attack buried metal improvements such as utility lines. The results of some of these tests are presented below.

<u>Sample Identification</u>	<u>Saturated Resistivity (ohm-cm)</u>	<u>pH</u>	<u>Chlorides (mg/kg)</u>	<u>Nitrates (mg/kg)</u>	<u>Sulfides (mg/kg)</u>	<u>Redox Potential (mV)</u>
B-9 @ 1 to 5 feet	2,211	8.1	9.4	95.3	0.6	138
B-10 @ 1 to 5 feet	3,015	8.2	11.8	89.3	0.4	139

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 conditions that differ from those stated in this report, or which may be detrimental for the development.

6.1 Seismic Design Considerations

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

Faulting and Seismicity

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

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

Seismic Design Parameters

The 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. Based on the adoption of the 2022 CBC on January 1, 2023, we expect that the proposed development will be designed in accordance with the 2022 CBC.

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 included as Plate E-1 in Appendix E of this report.

The 2022 CBC states that for Site Class D sites with a mapped S_1 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 S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in this Standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this Standard.”

The seismic design parameters presented in the table below were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2022 CBC. It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the 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 S_1 obtained from the Seismic Design Maps Tool. **The values of S_{M1} and S_{D1} 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, S_{M1} and S_{D1} 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	1.727
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.617
Site Class	---	D
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.727
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	1.049*
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	1.151
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.699*

*Note: These values must be increased by 50 percent if a site-specific ground motion hazard analysis has not been performed. However, this increase is not expected to affect the design of the structure type proposed for this site. This assumption should be confirmed by the project structural engineer. The values tabulated above do not include a 50-percent increase.

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 grain size 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). Clayey (cohesive) soils or soils which possess clay particles ($d < 0.005\text{mm}$) in excess of 20 percent (Seed and Idriss, 1982) are generally not considered to be susceptible to liquefaction, nor are those soils which are above the historic static groundwater table.

The California Geological Survey (CGS) has not yet conducted detailed seismic hazards mapping in the area of the subject site. The general liquefaction susceptibility of the site was attempted to be determined by research of the San Bernardino County Land Use Plan, General Plan, Geologic Hazard Overlays. No geologic hazard overlay was available for the Ontario Quadrangle nor the Prado Dam Quadrangle at the time of this report. The general plan update website indicates that if a geologic hazard map overlay does not exist, then there are no geologic hazards mapped by the state or county present in that community. Therefore, the subject site is not in a mapped geologic hazard zone. Furthermore, the native alluvium encountered beneath the localized ground water table, at a depth of $32 \pm$ feet at one of the boring locations, consist of medium stiff to stiff clayey soils. Based on the subsurface conditions encountered at the boring locations, liquefaction is not considered to be a design concern for this project.

6.2 Geotechnical Design Considerations

General

The near-surface soils encountered at the boring locations from the previous and current studies consist of artificial fill soils and native alluvium. The artificial fill soils extend to depths of $1\frac{1}{2}$ to $4\frac{1}{2} \pm$ feet below the ground surface. The fill soils and near-surface alluvial soils possess varying strengths. The existing fill soils are considered to represent undocumented fill. The near-surface native soils consist of loose to medium dense silty sands and sandy silts. Based on the results of laboratory testing, these soils possess variable densities. Neither the undocumented fill soils nor the near-surface native alluvium, in their present condition, are considered suitable to support the foundations loads of the new building. Therefore, remedial grading is considered warranted within the proposed building area in order to remove all of the undocumented fill soils in their entirety, the upper portion of the near-surface native alluvial soils, and any soils disturbed during the demolition process, and replace these materials as compacted structural fill soils.

Settlement

The proposed remedial grading will remove a portion of the loose, low strength, and potentially collapsible/compressible native alluvial soils, and all of the artificial fill materials, and replace these materials as compacted structural fill. The native soils that will remain in place below the

recommended depth of overexcavation will not be subject to significant stress increases from the foundations of the new structure. Therefore, following completion of the recommended grading, post-construction settlements are expected to be within tolerable limits.

Expansion

Laboratory testing performed on representative samples of the near surface soils indicates that these materials possess a very low to low expansion potential ($EI = 10$ to 29). Based on the presence of expansive soils at this site, care should be given to proper moisture conditioning of all building pad subgrade soils to a moisture content of 2 to 4 percent above the ASTM D-1557 optimum during site grading. In addition to adequately moisture conditioning the subgrade soils and fill soils during grading, special care must be taken to maintaining moisture content of these soils at 2 to 4 percent above the optimum moisture content. This will require the contractor to frequently moisture condition these soils throughout the grading process, unless grading occurs during a period of relatively wet weather. Civil and structural design considerations are presented in Section 6.4 of this report. 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.

Soluble Sulfates

The results of the soluble sulfate testing, discussed in Section 5.0 of this report, indicate soluble sulfate concentrations of 0.005 percent. These concentrations are considered to be negligible or "not applicable" with respect to the American Concrete Institute (ACI) Publication 318-05 Building Code Requirements for Structural Concrete and Commentary, Section 4.3. Therefore, specialized concrete mix designs are not considered to be necessary, with regard to sulfate protection purposes. It is, however, recommended that additional soluble sulfate testing be conducted at the completion of rough grading to verify the soluble sulfate concentrations of the soils which are present at pad grade within the building area.

Corrosion Potential

The results of laboratory testing indicate that the tested samples of the near-surface on-site soils possess saturated resistivities ranging from 2,211 to 3,015 ohm-cm, and pH values ranging from 8.1 to 8.2. The soils possess redox potentials of 138 and 139 mV and sulfide concentrations of 0.4 and 0.6 mg/kg. These test results have been evaluated in accordance with guidelines published by the Ductile Iron Pipe Research Association (DIPRA). The DIPRA guidelines consist of a point system by which characteristics of the soils are used to quantify the corrosivity characteristics of the site. Resistivity, pH, sulfide concentration, redox potential, and moisture content are the five factors that enter into the evaluation procedure. **Based on these factors, the on-site soils are considered to be mildly corrosive to ferrous pipes. Therefore, corrosion 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 ranging from 9.4 to 11.8 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 samples possess nitrate concentrations ranging from 89.3 to 95.3 mg/kg. **Based on the test results, the on-site soils are 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 is estimated to result in an average shrinkage of 4 to 14 percent. However, potential shrinkage for individual samples ranged locally between 1 and 19 percent. The potential shrinkage estimate is based on dry density testing performed on small-diameter samples taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.15 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 unavailable 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 and trench locations, and our understanding of the proposed development. We recommend that all grading activities be completed in accordance with the Grading Guide Specifications included as Appendix D of this report, unless superseded by site-specific recommendations presented below.

Site Stripping and Demolition

The proposed development will require demolition of the existing pavements and structures. Additionally, any existing improvements that will not remain in place for use with the new development should be removed in their entirety. This should include all utilities, and any other subsurface improvements associated with the existing pavements. 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 the on-site sands. Alternatively, existing asphalt and concrete materials may be crushed into miscellaneous base (CMB) and re-used at the site. Asphalt and concrete debris resultant from demolition may also be crushed to 2 to 4-inch particle size and used as a subgrade stabilization material. **Mixing concrete and asphalt debris with the on-site clayey soils is not recommended.**

Detailed structural information regarding the existing buildings has not been provided to our office. Therefore, the foundation systems supporting the existing buildings are generally unknown by SCG. We expect that the existing buildings are supported on conventional shallow foundations. The existing undocumented fill soils within the proposed building pad area should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils.

Initial site stripping should also include removal of any surficial vegetation from the unpaved areas of the site. This should include any weeds, grasses, shrubs, and trees. Root systems associated with the trees should be removed in their entirety, and the resultant excavations should be backfilled with compacted structural fill soils. Any organic materials should be removed and disposed of off-site, or in non-structural areas of the property. The actual extent of site stripping should be determined in the field by the geotechnical engineer, based on the organic content and stability of the materials encountered.

Treatment of Existing Soils: Building Pad

Remedial grading should be performed within the proposed building pad area in order to remove any soils disturbed during demolition, the existing undocumented fill soils, and the upper portion of the near-surface native alluvium. Based on conditions encountered at the boring locations from the previous and current investigations, we recommend that the existing soils within the proposed building area be overexcavated to a depth of at least 5 feet below existing grade and to a depth of at least 4 feet below proposed building pad subgrade elevations, whichever is greater. The depth of the overexcavation should also extend to a depth sufficient to remove all undocumented fill soils. The undocumented fills extend to depths of 1½ to 4½± feet at the boring locations. Additional overexcavation should be performed within the influence zones of the new foundations, to provide for a new layer of compacted structural fill extending to a depth of at least 3 feet below proposed bearing grades.

The overexcavation areas should extend at least 5 feet beyond the building and foundation perimeters, and to an extent equal to the depth of fill placed below the foundation bearing grade, whichever is greater. If the proposed structure incorporates any exterior columns (such as for a canopy or overhang) the area of overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the overexcavation areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade, as well as to support the foundation loads of the new structure. This evaluation should include proofrolling and probing to identify any soft, loose or otherwise unstable soils that must be removed. Some localized areas of deeper excavation may be required if 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, some zones of very moist soils may be encountered at or near the base of the recommended overexcavation. Stabilization of the exposed overexcavation subgrade soils may be necessary. 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, could 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. Asphalt and concrete debris resultant from demolition could be crushed to 2 to 4-inch particle size and 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 to 18± inches of subgrade material with cement to a concentration of 5 percent (by dry weight of soil).

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

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of proposed retaining and non-retaining site walls should be overexcavated to a depth of at least 3 feet below foundation bearing grade and replaced as compacted structural fill. Any undocumented fill soils within any of these foundation areas should be removed in their entirety. The overexcavation areas should extend at least 3 feet beyond the foundation perimeters, and to an extent equal to the depth of fill below the new foundations. Please note that erection pads are considered to be part of the foundation system. These overexcavation recommendations apply to erection pads also. The overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning to within 2 to 4 percent above the optimum moisture content, and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If the full lateral extent of overexcavation is not achievable for the proposed walls, foundation elements must be redesigned using a lower bearing pressure. The geotechnical engineer of record should be contacted for recommendations pertaining to this type of condition.

Treatment of Existing Soils: Parking and Drive Areas

Based on economic considerations, overexcavation of the existing soils in the new parking and drive areas is not considered warranted from a geotechnical standpoint, with the exception of areas where lower strength, or unstable soils are identified by the geotechnical engineer during grading. Subgrade preparation in the new parking and drive areas should initially consist of removal of all soils disturbed during stripping operations.

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

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

Treatment of Existing Soils: Flatwork Areas

Subgrade preparation in the new flatwork areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above the optimum moisture content, and recompact to at least 90 percent of the ASTM D-1557 maximum dry density.

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

Fill Placement

- Fill soils should be placed in thin (6± inches), near-horizontal lifts, moisture conditioned to 2 to 4 percent above the optimum moisture content, and compacted.
- On-site soils may be used for fill provided they are cleaned of any debris to the satisfaction of the geotechnical engineer.

- It should be noted that the some of the encountered subsurface soils possess moisture contents above the anticipated optimum moisture content. Therefore, some drying of these materials will likely be required in order to achieve a moisture content suitable for recompaction.
- 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 Chino.
- 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 very low expansive ($EI < 20$), well graded soils possessing at least 10 percent fines (that portion of the sample passing the No. 200 sieve). Additional specifications for structural fill are presented in the Grading Guide Specifications, included as Appendix D.

Utility Trench Backfill

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

Utility trenches which parallel a footing, and extending below a 1h:1v (horizontal to vertical) 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.

Soils used to backfill voids around subsurface utility structures, such as manholes or vaults, should be placed as compacted structural fill. If it is not practical to place compacted fill in these areas, then such void spaces may be backfilled with lean concrete slurry. Uncompacted pea gravel or sand is not recommended for backfilling these voids since these materials have a potential to settle and thereby cause distress of pavements placed around these subterranean structures.

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of silty sands, sandy silt, with occasional silty clays, sandy clays and clayey silts. Some of these materials may be subject to caving within shallow excavations. Where caving occurs within shallow excavations, flattened excavation slopes may be sufficient to provide excavation stability. On a preliminary basis, temporary excavation slopes should be made no steeper than 1.5h:1v within clayey soils, and where sandier soils are encountered, temporary excavation slopes should be no steeper than 2h:1v. **The contractor should take all necessary precautions during grading and foundation construction to prevent damage to structures and improvements which are adjacent to the proposed development.** Deeper excavations may require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations.

Moisture Sensitive Subgrade Soils

The near-surface soils generally consist of moist to very moist silty sands, sandy silt, with occasional silty clays, sandy clays and clayey silts and will 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 prevent ponding of surface water and to prevent water from running into excavations.

As discussed in Section 6.3 of this report, unstable subgrade soils are likely to 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 only tracked vehicles be utilized for grading or construction activities that require traffic over the exposed subgrade soils.

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

Expansive Soils

The near-surface soils have been determined to possess a low expansion potential. Therefore, care should be given to proper moisture conditioning the subgrade soils to a moisture content of 2 to 4 percent above the Modified Proctor optimum during site grading. All imported fill soils should have low expansive ($EI < 50$) characteristics. **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 limit the potential for surface water to penetrate the soils immediately adjacent to the new 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 building 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 structure. 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 the proposed structure should be sloped at a minimum five percent gradient away from the structure (about three inches of fall in five feet), or the same area could be paved with a minimum surface gradient of one percent. Pavement is preferable.
- Decorative gravel ground cover tends to provide a reservoir for surface water and may hide areas of ponding or poor drainage. Decorative gravel is, therefore, not recommended and should not be utilized for landscaping unless equipped with a subsurface drainage system designed by a licensed landscape architect.
- Positive drainage devices, such as graded swales, paved ditches, and catch basins should be installed at appropriate locations within the area of proposed development.
- Concrete walks and flatwork should not obstruct the free flow of surface water to the appropriate drainage devices.
- Area drains should be recessed below grade to allow free flow of water into the drain. Concrete or brick flatwork joints should be sealed with mortar or flexible mastic.
- Gutter and downspout systems should be installed to capture all discharge from roof areas. Downspouts should discharge directly into a pipe or paved surface system to be conveyed off-site.
- Enclosed planters adjoining, or in close proximity to the proposed structure, should be sealed at the bottom and provided with subsurface collection systems and outlet pipes.
- Depressed planters should be raised with soil to promote runoff (minimum drainage gradient two percent or five percent, see above), and/or equipped with area drains to eliminate ponding.
- Drainage outfall locations should be selected to avoid erosion of slopes and/or properly armored to prevent erosion of graded surfaces. No drainage should be directed over or towards adjoining slopes.
- All drainage devices should be maintained on a regular basis, including frequent observations during the rainy season to keep the drains free of leaves, soil and other debris.
- Landscape irrigation should conform to the recommendations of the landscape architect and should be performed judiciously to preclude either soaking or excessive drying of the foundation soils. This should entail regular watering during the drier portions of the year and little or no irrigation during the rainy season. Automatic sprinkler systems should, therefore, be switched to manual operation during the rainy season. Good irrigation practice typically requires frequent application of limited quantities of water that are sufficient to sustain plant growth, but do not excessively wet the soils. Ponding and/or run-off of irrigation water are indications of excessive watering.

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

Groundwater

The static groundwater table is considered to have existed at a depth of 32± feet at the time of the subsurface exploration. Therefore, groundwater is not expected to impact the grading or foundation construction activities.

6.5 Foundation Design and Construction

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

Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,500 lbs/ft².
- Maximum, net allowable soil bearing pressure: 1,500 lbs/ft² if the full recommended lateral extent of remedial grading cannot be achieved, typically for new footings along the property lines.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Four (4) No. 5 rebars (2 top and 2 bottom).
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 18 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.
- It is recommended that the perimeter building foundations be continuous across all exterior doorways. Any flatwork adjacent to the exterior doors should be doweled into the perimeter foundations in a manner determined by the structural engineer.

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

Foundation Construction

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

Estimated Foundation Settlements

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

Lateral Load Resistance

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

- Passive Earth Pressure: 275 lbs/ft³
- Friction Coefficient: 0.28

These are allowable values, and include a factor of safety. When combining friction and passive resistance, the passive pressure component should be reduced by one-third. These values assume that footings will be poured directly against compacted structural fill soils. The maximum allowable passive pressure is 2,500 lbs/ft².

6.6 Floor Slab Design and Construction

Subgrades which will support the new floor slab should be prepared in accordance with the recommendations contained in the ***Site Grading Recommendations*** section of this report. Based on the anticipated grading which will occur at this site, the floor of the 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 finished pad grades. Based on geotechnical considerations, the floor slab may be designed as follows:

- Minimum slab thickness: 6 inches.
- Modulus of Subgrade Reaction: $k = 100$ psi/in.
- Minimum slab reinforcement: No. 3 bars at 18-inches on-center, in both directions, due to the expansive potential of the on-site soils. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading.
- Slab underlayment: If moisture sensitive floor coverings will be used then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab where such moisture sensitive floor coverings are anticipated. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as a 15-mil Stego® Wrap Vapor Barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.
- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.
- The floor slab should be structurally connected to the foundations as detailed by the structural engineer.

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

6.7 Trash Enclosure Design Parameters

The proposed development may include a trash enclosure. It is expected that the trash enclosure as well as the approach slab will be subjected to relatively heavy wheel loads, imposed by trash removal equipment.

The subgrade soils in the area of the trash enclosure and the approach slab should be prepared in accordance with the recommendations for the parking areas, presented in Section 6.3 of this report. As such, it is expected that the trash enclosure will be underlain by structural fill soils, extending to a depth of 1 foot below proposed subgrade elevation. Based on geotechnical

considerations, the following recommendations are provided for the design of the trash enclosure and the trash enclosure approach slab:

- The trash enclosure may consist of a 6-inch-thick concrete slab incorporating a perimeter footing or a turned down edge, extending to a depth of at least 12 inches below adjacent finished grade. If the trash enclosure will incorporate rigid walls such as masonry block or tilt-up concrete, the perimeter foundations should be designed in accordance with the recommendations previously presented in Section 6.5 of this report.
- Reinforcement within the trash enclosure slab should consist of at least No. 4 bars at 18-inches on-center, in both directions.
- The trash enclosure approach slab should be constructed of Portland cement concrete, at least 6 inches in thickness. Reinforcement within the approach slab should consist of at least No. 3 bars at 18-inches on-center, in both directions.
- The trash enclosure and approach slab subgrades should be moisture conditioned to 2 to 4 percent above the optimum moisture content to a depth of 12 inches. The trash enclosure slab and the approach slab should be structurally connected, to reduce the potential for differential movement between the two slabs.

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

6.8 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for sidewalks, patios, and other concrete flatwork, should be prepared in accordance with the recommendations contained in the ***Grading Recommendations*** section of this report. As noted previously, flatwork supported on the existing very low to low expansive soils will be subject to minor amounts of movement as the moisture content within the subgrade soils fluctuates. This movement may cause cracking or other distress within the flatwork. If additional protection against flatwork cracking is desired, consideration should be given to the placement of a 1 to 2-foot-thick layer of very low expansive structural fill beneath all flatwork sections. Assuming that the flatwork is supported on the existing soils, exterior slabs-on-grade may be designed as follows:

- Minimum slab thickness: 4½ inches due to the presence of expansive soils.
- Minimum slab reinforcement: No. 3 bars at 18 inches on center, in both directions, due to the presence of low to high expansive soils.
- The flatwork at building entry areas should be structurally connected to the perimeter foundation that is recommended to span across the door opening. This recommendation is designed to reduce the potential for differential movement at this joint.

- Moisture condition the flatwork subgrade soils to at least 2 to 4 percent above optimum moisture content, to a depth of at least 12 inches. Adequate moisture conditioning should be verified by the geotechnical engineer 24 hours prior to concrete placement.
- Proper concrete curing techniques should be utilized to reduce the potential for slab curling or the formation of excessive shrinkage cracks.
- Control joints should be provided at a maximum spacing of 8 feet on center in two directions for slabs and at 6 feet on center for sidewalks. Control joints are intended to direct cracking. Minor cracking of exterior concrete slabs on grade should be expected.
- Where flatwork is immediately adjacent to landscape planters, a thickened edge should be utilized. This edge should extend to a depth of at least 12 inches and incorporate longitudinal reinforcement consisting of at least two No. 4 bars.
- Expansion or felt joints should be used at the interface of exterior slabs on grade and any fixed structures to permit relative movement.

These recommendations are contingent upon additional expansion index testing being conducted at the completion of rough grading, to verify the actual expansion potential of the flatwork subgrade soils.

6.9 Retaining Wall Design and Construction

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

Retaining Wall Design Parameters

Based on the soil conditions encountered at the boring locations, the following parameters may be used in the design of new retaining walls for this site. We have provided parameters assuming the use of sandy silts for retaining wall backfill. **The potentially expansive sandy clays, silty clays and clayey silts should not be used for retaining wall backfill.** Based on their composition, the on-site sandy silts have been assigned a friction angle of 29 degrees when compacted to at least 90 percent of the ASTM D-1557 maximum dry-density.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type
		On-site Sandy Silts
Internal Friction Angle (ϕ)		29°
Unit Weight		125 lbs/ft ³
Equivalent Fluid Pressure:	Active Condition (level backfill)	44 lbs/ft ³
	Active Condition (2h:1v backfill)	72 lbs/ft ³
	At-Rest Condition (level backfill)	65 lbs/ft ³

The walls should be designed using a soil-footing coefficient of friction of 0.28 and an equivalent passive pressure of 275 lbs/ft³. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

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

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

Seismic Lateral Earth Pressures

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

Retaining Wall Foundation Design

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

Backfill Material

Retaining wall backfill soils should consist of imported select structural fill possessing an expansion index less than 20. Backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

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

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

Subsurface Drainage

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

- A weep hole drainage system typically consisting of a series of 2-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 10-foot on-center spacing. Alternatively, 4-inch diameter holes at an approximate 20-foot on-center spacing can be used for this type of drainage system. In addition, the weep holes should include a 2 cubic foot pocket of open graded gravel, surrounded by an approved geotextile fabric, at each weep hole location.
- A 4-inch diameter perforated pipe surrounded by 2 cubic feet of gravel per linear foot of drain placed behind the wall, above the retaining wall footing. The gravel layer should be wrapped in a suitable geotextile fabric to reduce the potential for migration of fines. The footing drain should be extended to daylight or tied into a storm drainage system. The actual design of this type of system should be determined by the civil engineer to verify that the drainage system possesses the adequate capacity and slope for its intended use.

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

6.10 Pavement Design Parameters

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

designs also assume a routine pavement maintenance program to obtain the anticipated 20-year pavement service life.

Pavement Subgrades

It is anticipated that the new pavements will be primarily supported on a layer of compacted structural fill, consisting of scarified, thoroughly moisture conditioned and recompacted existing soils. The near-surface soils generally consist of silty sands, sandy silt, with occasional silty clays, sandy clays and clayey silts. These soils are considered to possess good pavement support characteristics with estimated R-values of 30 to 45. The subsequent pavement design is based upon an R-value of 30. 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 observed and tested conditions. It is recommended that R-value testing be performed after completion of rough grading to verify that the pavement design recommendations presented herein are valid.

Asphaltic Concrete

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

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

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

ASPHALT PAVEMENTS (R=30)					
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	6	8	10	11	13
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 batch plant-reported maximum density. The aggregate base course may consist of crushed aggregate base (CAB) or crushed miscellaneous base (CMB), which is a recycled gravel, asphalt and concrete material. The gradation, R-Value, Sand Equivalent, and Percentage Wear of the CAB or CMB should comply with appropriate specifications contained in the current edition of the "Greenbook" Standard Specifications for Public Works Construction.

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 30)				
Materials	Thickness (inches)			
	Autos and Light Truck Traffic (TI = 6.0)	Truck Traffic		
		TI = 7.0	TI = 8.0	TI = 9.0
PCC	5	5½	6½	8
Compacted Subgrade (95% minimum compaction)	12	12	12	12

The concrete should have a 28-day compressive strength of at least 3,000 psi. 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

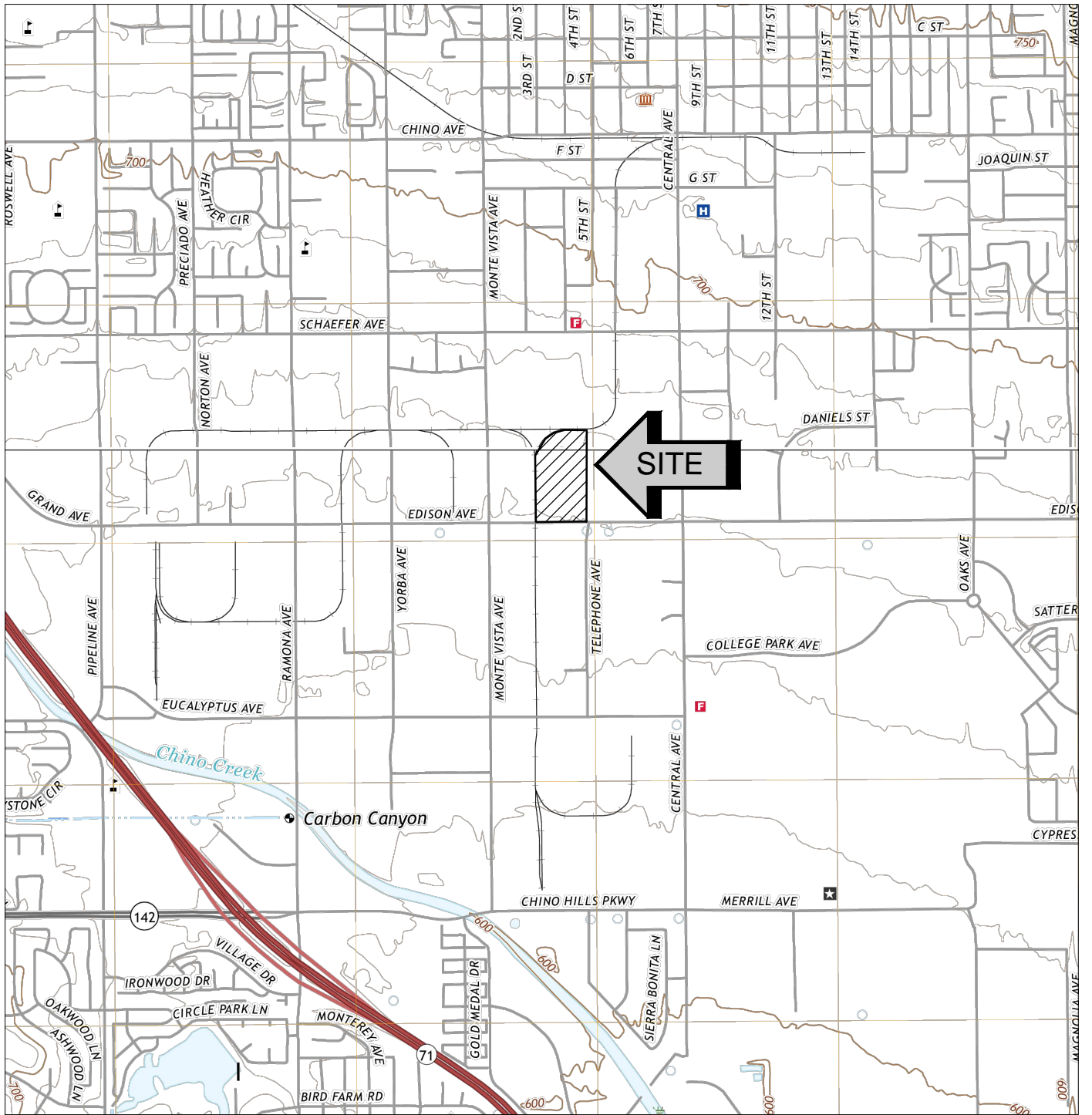
This report has been prepared as an instrument of service for use by the client, in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. This report may be provided to the contractor(s) and other 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 determine if the conditions alter the recommendations contained herein.

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

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

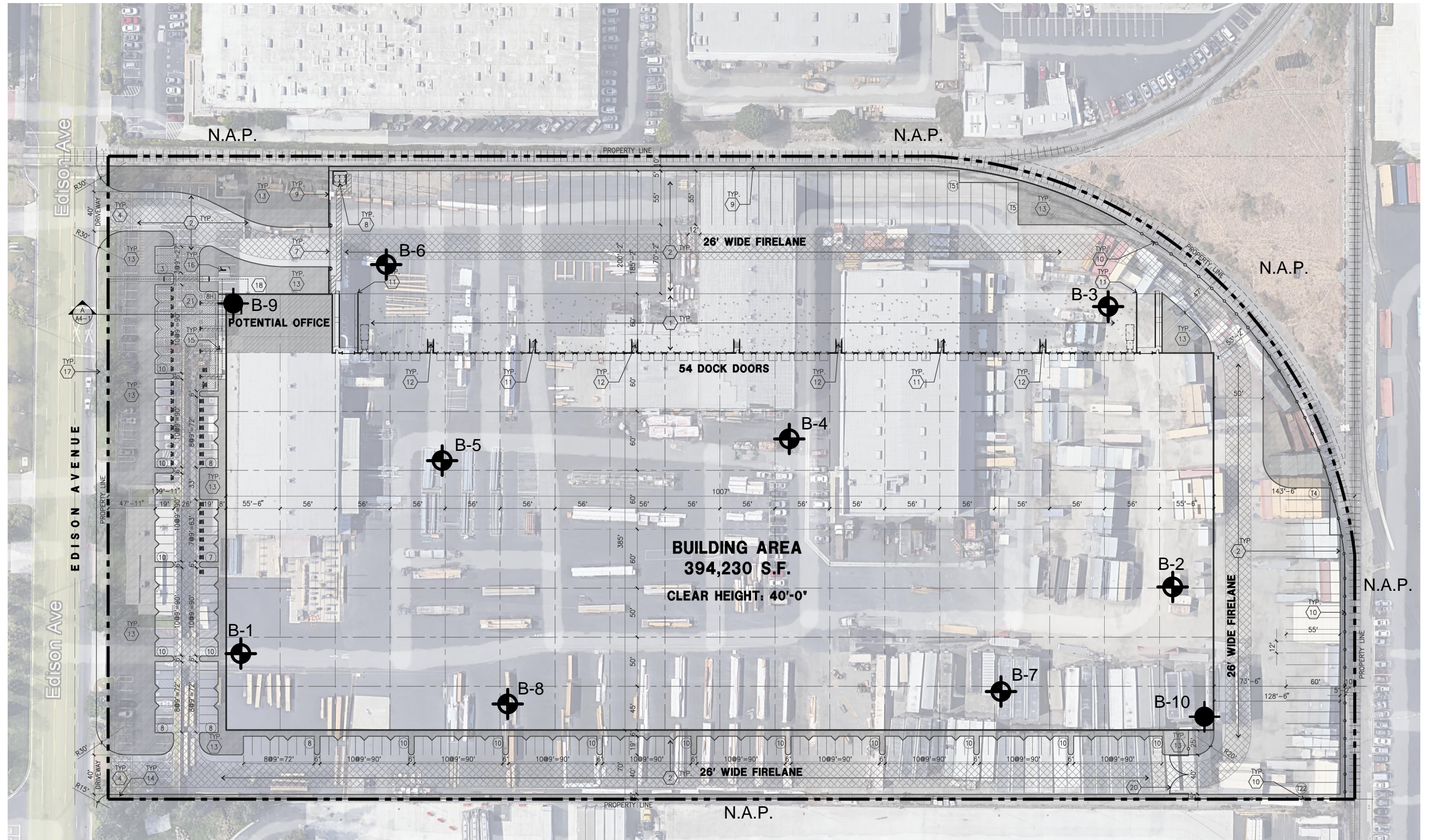
APPENDIX A



SOURCE: USGS TOPOGRAPHIC MAPS OF THE PRADO DAM QUADRANGLE AND THE ONTARIO QUADRANGLE, SAN BERNARDINO COUNTY, CALIFORNIA, 2022.



SITE LOCATION MAP	
PROPOSED INDUSTRIAL BUILDING	
CHINO, CALIFORNIA	
SCALE: 1" = 2000'	
DRAWN: JJH	
CHKD: JLL	
SCG PROJECT 23G162-1	
PLATE 1	
SOUTHERN CALIFORNIA GEOTECHNICAL	



GEOTECHNICAL LEGEND

- APPROXIMATE BORING LOCATION
- PREVIOUS BORING LOCATION (SCG PROJECT NO. 12G120-1)


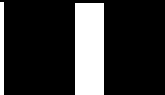



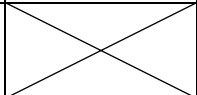
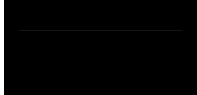



NOTE: OVERALL SITE PLAN PREPARED BY HPA, INC.

BORING LOCATION PLAN	
PROPOSED INDUSTRIAL BUILDING CHINO, CALIFORNIA	
SCALE: 1" = 100'	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JJH	
CHKD: JLL	
SCG PROJECT 23G162-1	
PLATE 2	

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

- DEPTH:** Distance in feet below the ground surface.
- SAMPLE:** Sample Type as depicted above.
- BLOW COUNT:** Number of blows required to advance the sampler 12 inches using a 140 lb hammer with a 30-inch drop. 50/3" indicates penetration refusal (>50 blows) at 3 inches. WH indicates that the weight of the hammer was sufficient to push the sampler 6 inches or more.
- POCKET PEN.:** Approximate shear strength of a cohesive soil sample as measured by pocket penetrometer.
- GRAPHIC LOG:** Graphic Soil Symbol as depicted on the following page.
- DRY DENSITY:** Dry density of an undisturbed or relatively undisturbed sample in lbs/ft³.
- MOISTURE CONTENT:** Moisture content of a soil sample, expressed as a percentage of the dry weight.
- LIQUID LIMIT:** The moisture content above which a soil behaves as a liquid.
- PLASTIC LIMIT:** The moisture content above which a soil behaves as a plastic.
- PASSING #200 SIEVE:** The percentage of the sample finer than the #200 standard sieve.
- UNCONFINED SHEAR:** The shear strength of a cohesive soil sample, as measured in the unconfined state.

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS



JOB NO.: 23G162-1 DRILLING DATE: 7/17/23 WATER DEPTH: 32 feet
 PROJECT: Proposed Industrial Building DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 14 feet
 LOCATION: Chino, California LOGGED BY: Michelle Krizek READING TAKEN: 3 Hrs. After Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: ----- MSL												
					3½± inches Asphaltic Concrete, 4½± inches Aggregate Base							
	X	24	3.5		FILL: Black fine Sandy Clay, little Silt, little medium to coarse Sand, little fine Gravel, mottled, very stiff-damp	109	9					EI = 29 @ 1 to 5 feet
	X	20			FILL: Dark Brown to Black fine Sandy Silt, little Clay, trace medium Sand, little Calcareous nodules/veining, medium dense-moist	94	22					
5	X	25	4.5		ALLUVIUM: Light Gray Brown Clayey Silt, little fine Sand, little Calcareous nodules/veining, very stiff-moist	104	18					
	X	22			Brown fine Sandy Silt, little Clay, trace to little medium Sand, medium dense-damp	107	11					
	X	16	1.5		Brown fine to medium Sand, little Silt, trace coarse Sand, medium dense-damp	108	7					
10	X				Dark Gray Brown Silty Clay, trace fine Sand, trace to little Iron Oxide staining, little Calcareous nodules/veining, medium stiff to stiff-very moist to wet	85	41					
					Light Gray Brown Clayey Silt, little fine Sand, trace Calcareous nodules/veining, medium stiff to stiff-very moist							
15	X	8	4.0			100	27					
	X	19	2.0		Brown Clayey fine Sand to fine Sandy Clay, little medium Sand, little Silt, medium dense to very stiff-very moist		23					
20	X											
	X	2	3.0		Light Gray Brown Silty Clay, trace medium Sand, little Calcareous nodules/veining, soft to medium stiff-very moist		26					
25	X	8	2.5			92	28					
	X	5	2.5		Gray fine Sandy Clay, little Silt, little medium Sand, trace coarse Sand, trace fine Gravel, trace to little Iron Oxide staining, trace to little Calcareous nodules/veining, medium stiff-very moist		23					

TBL_23G162-1.GPJ_SOCALGEO.GDT_8/3/23



JOB NO.: 23G162-1	DRILLING DATE: 7/17/23	WATER DEPTH: 32 feet
PROJECT: Proposed Industrial Building	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 14 feet
LOCATION: Chino, California	LOGGED BY: Michelle Krizek	READING TAKEN: 3 Hrs. After Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
(Continued)												
35		4	3.0	Gray fine Sandy Clay, little Silt, little medium Sand, trace coarse Sand, trace fine Gravel, trace to little Iron Oxide staining, trace to little Calcareous nodules/veining, medium stiff-very moist		33						
40		14	2.5	Light Gray Silty Clay, trace fine Sand, medium stiff-very moist to wet								
45		11	4.5	Gray Clayey Silt, trace fine Sand, trace Iron Oxide staining, little Calcareous nodules/veining, stiff-wet		27						
50		20	3.0	Light Gray Brown fine Sandy Clay, little Silt, trace to little Iron Oxide staining, medium stiff-wet	105	20						
				Gray Silty Clay, little fine Sand, stiff-wet	83	38						
Boring Terminated at 50'												

TBL_23G162-1.GPJ_SOCALGEO.GDT_8/3/23



JOB NO.: 23G162-1	DRILLING DATE: 7/17/23	WATER DEPTH: Dry
PROJECT: Proposed Industrial Building	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Chino, California	LOGGED BY: Michelle Krizek	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
SURFACE ELEVATION: ----- MSL												
				1± inch Aggregate Base								
	X	10		FILL: Dark Gray to Black fine Sandy Silt, little Clay, little medium to coarse Sand, little fine Gravel, loose-damp	98	9						
	X	28		ALLUVIUM: Brown Silty fine Sand, trace medium Sand, trace to little Iron Oxide staining, medium dense-damp to moist	114	8						
5	X	19			110	10						
	X	31		Light Gray fine to medium Sand, trace coarse Sand, little Silt, medium dense-damp	109	3						
10	X	9	3.0	Gray Clayey Silt, trace to little fine Sand, trace to little Iron Oxide staining, little Calcareous nodules/veining, medium stiff to stiff-moist to very moist	98	29						
15	X	17	2.0		99	26						
20	X	19		Brown fine Sand, little Silt, little medium Sand, trace to little Iron Oxide staining, medium dense-damp to moist		8						
25	X	6	3.0	Light Gray fine Sandy Clay, little Silt, little Calcareous nodules/veining, medium stiff to stiff-moist		22						
30	X	14	3.5	Gray Brown Silty Clay, little fine Sand, trace Iron Oxide staining, stiff-moist to very moist		27						

TBL_23G162-1.GPJ_SOCALGEO.GDT_8/3/23



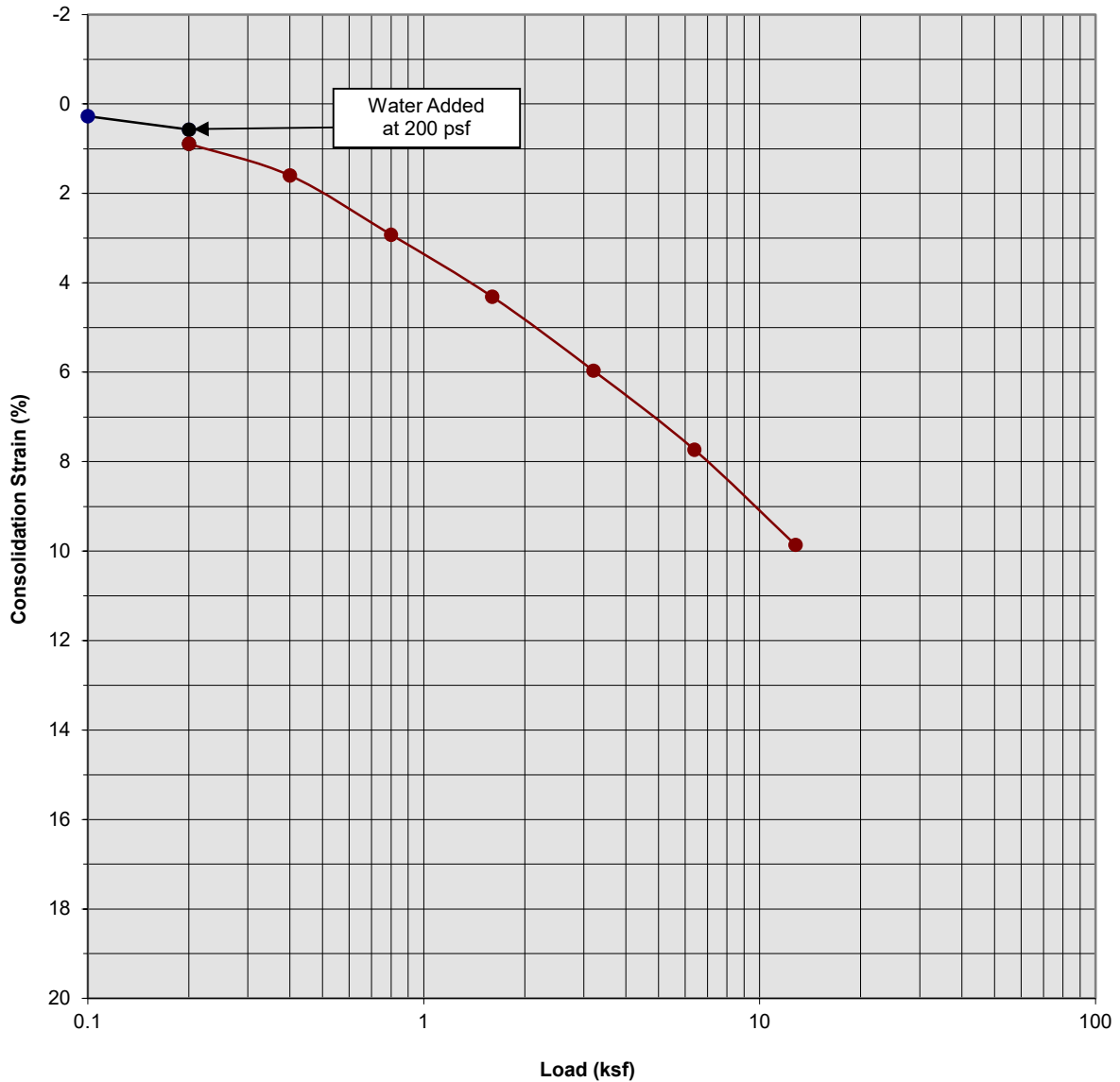
JOB NO.: 23G162-1	DRILLING DATE: 7/17/23	WATER DEPTH: Dry
PROJECT: Proposed Industrial Building	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 17 feet
LOCATION: Chino, California	LOGGED BY: Michelle Krizek	READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION (Continued)	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	
					Gray Brown Silty Clay, little fine Sand, trace Iron Oxide staining, stiff-moist to very moist							
35	X	24			Brown Silty fine Sand to fine Sandy Silt, trace medium to coarse Sand, trace Clay, trace to little Iron Oxide staining, medium dense-damp to moist		16					
40	X	12	2.0		Gray Silty Clay, trace fine Sand, trace to little Iron Oxide staining, trace to little Calcareous nodules/veining, stiff-moist to very moist							
45	X	10	4.0									
50	X	11	3.5									
					Boring Terminated at 50'							

TBL_23G162-1.GPJ_SOCALGEO.GDT_8/3/23

A P P E N D I X C

Consolidation/Collapse Test Results



Classification: Light Gray Brown Clayey Silt, little fine Sand

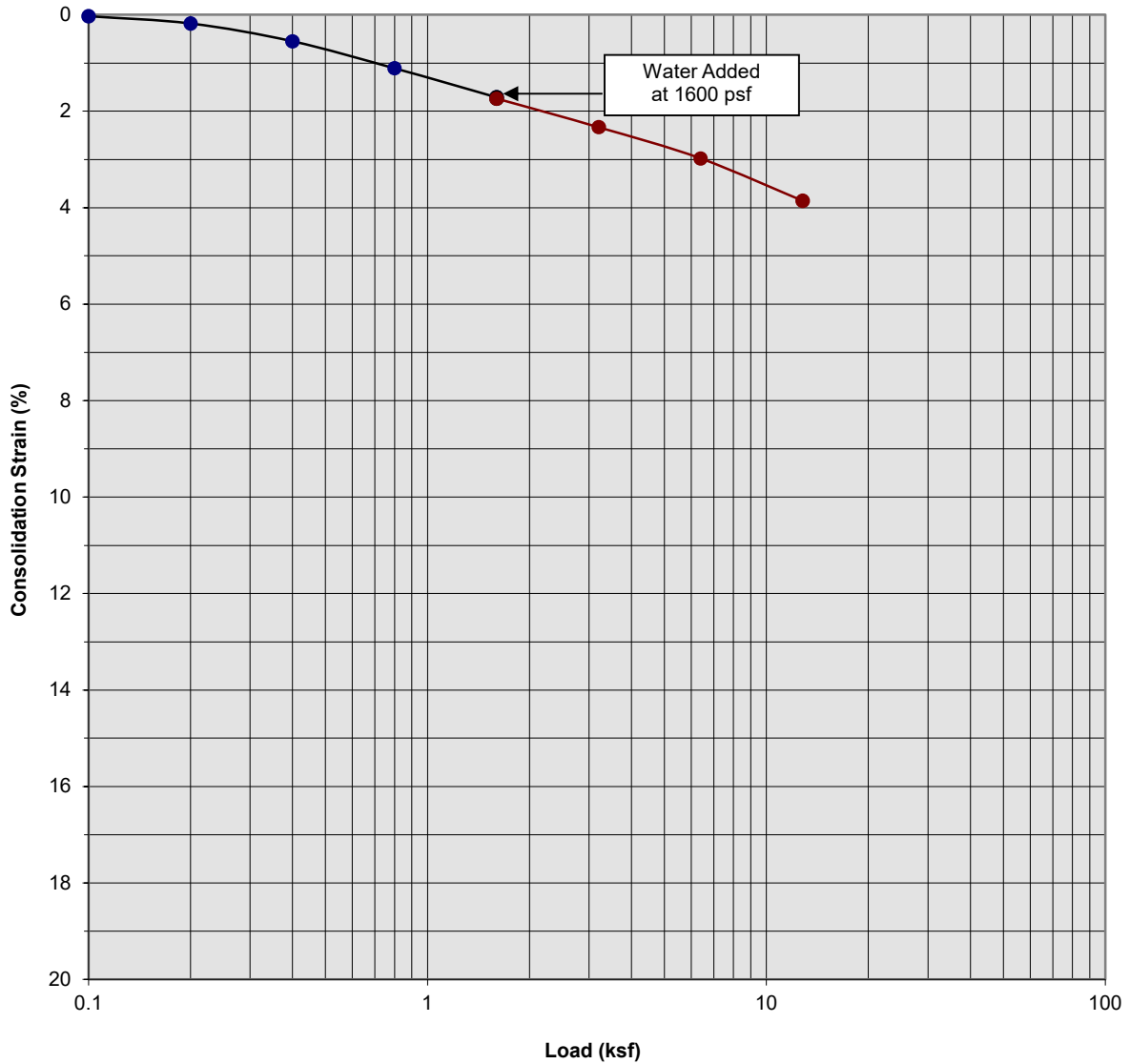
Boring Number:	B-9	Initial Moisture Content (%)	18
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	5 to 6	Initial Dry Density (pcf)	104.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.32

Proposed Industrial Building
 Chino, California
 Project No. 23G162-1
PLATE C- 1



**SOUTHERN
 CALIFORNIA
 GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown fine to medium Sand, little Silt, trace coarse Sand

Boring Number:	B-9	Initial Moisture Content (%)	7
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	7½ to 8	Initial Dry Density (pcf)	107.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.03

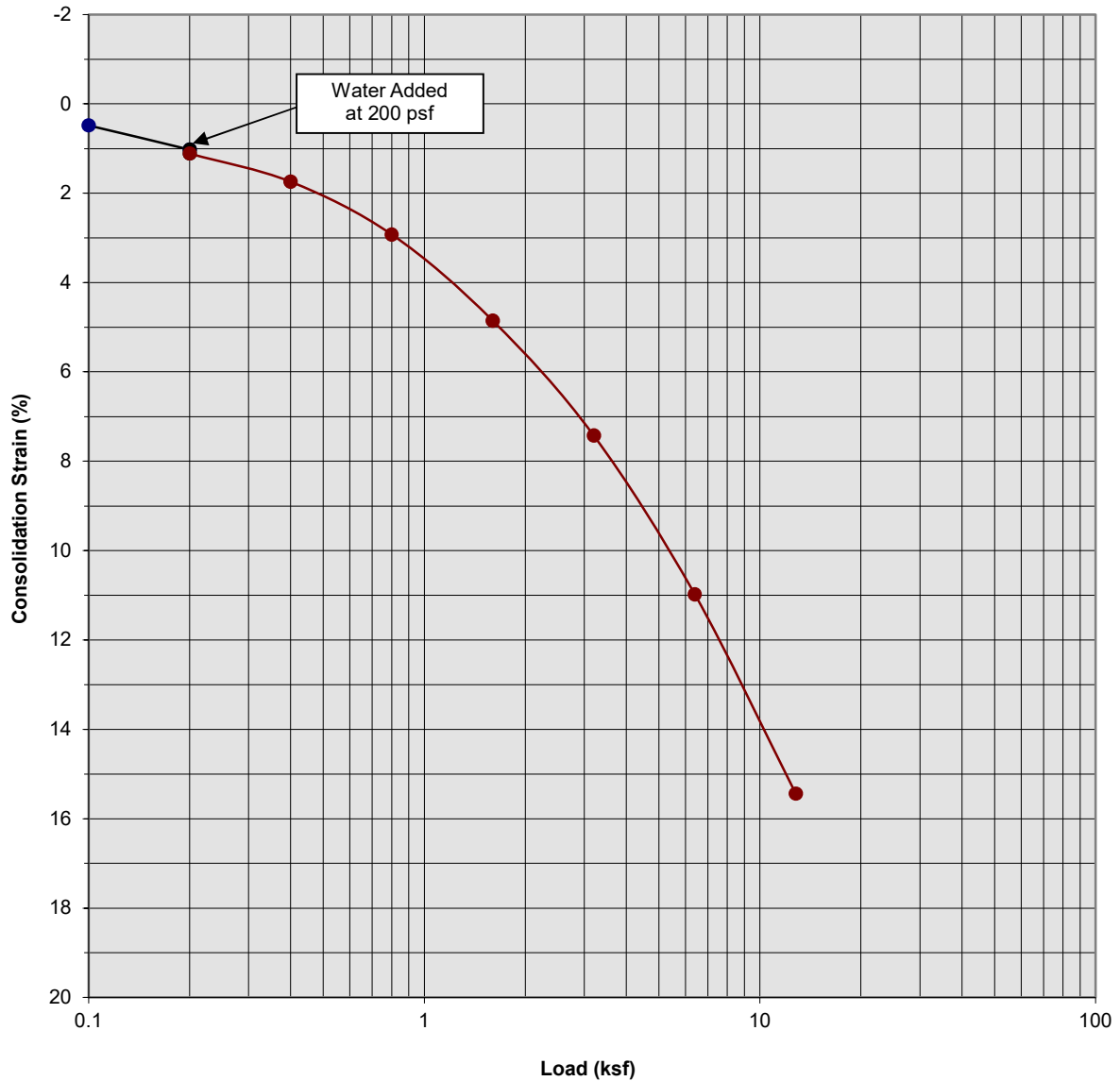
Proposed Industrial Building
Chino, California
Project No. 23G162-1

PLATE C- 2



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Dark Gray Brown Silty Clay, trace fine Sand

Boring Number:	B-9	Initial Moisture Content (%)	41
Sample Number:	---	Final Moisture Content (%)	32
Depth (ft)	9 to 10	Initial Dry Density (pcf)	84.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	100.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.09

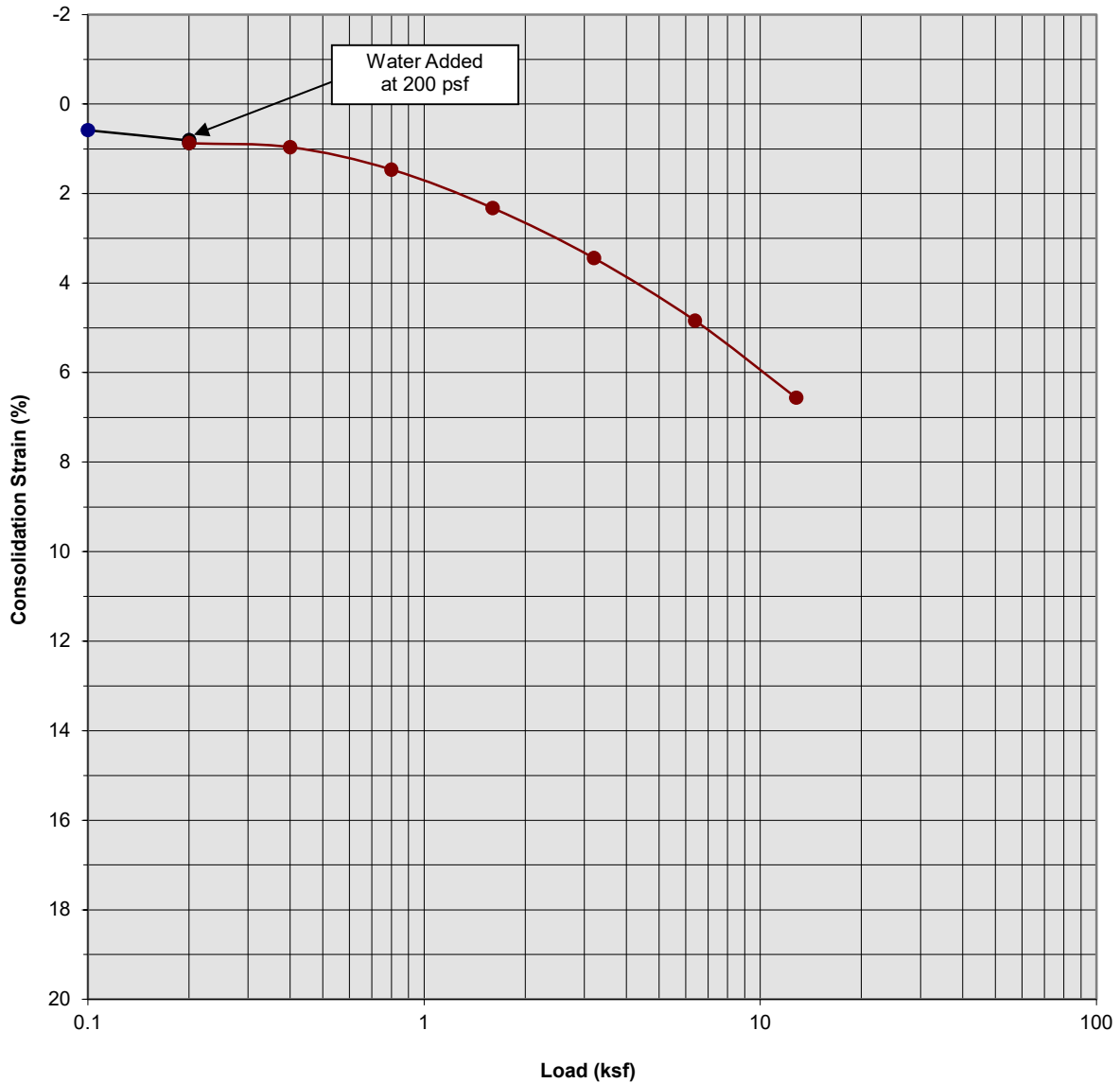
Proposed Industrial Building
Chino, California
Project No. 23G162-1

PLATE C- 3



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Light Gray Brown Clayey Silt, little fine Sand

Boring Number:	B-9	Initial Moisture Content (%)	27
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	14 to 15	Initial Dry Density (pcf)	99.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.06

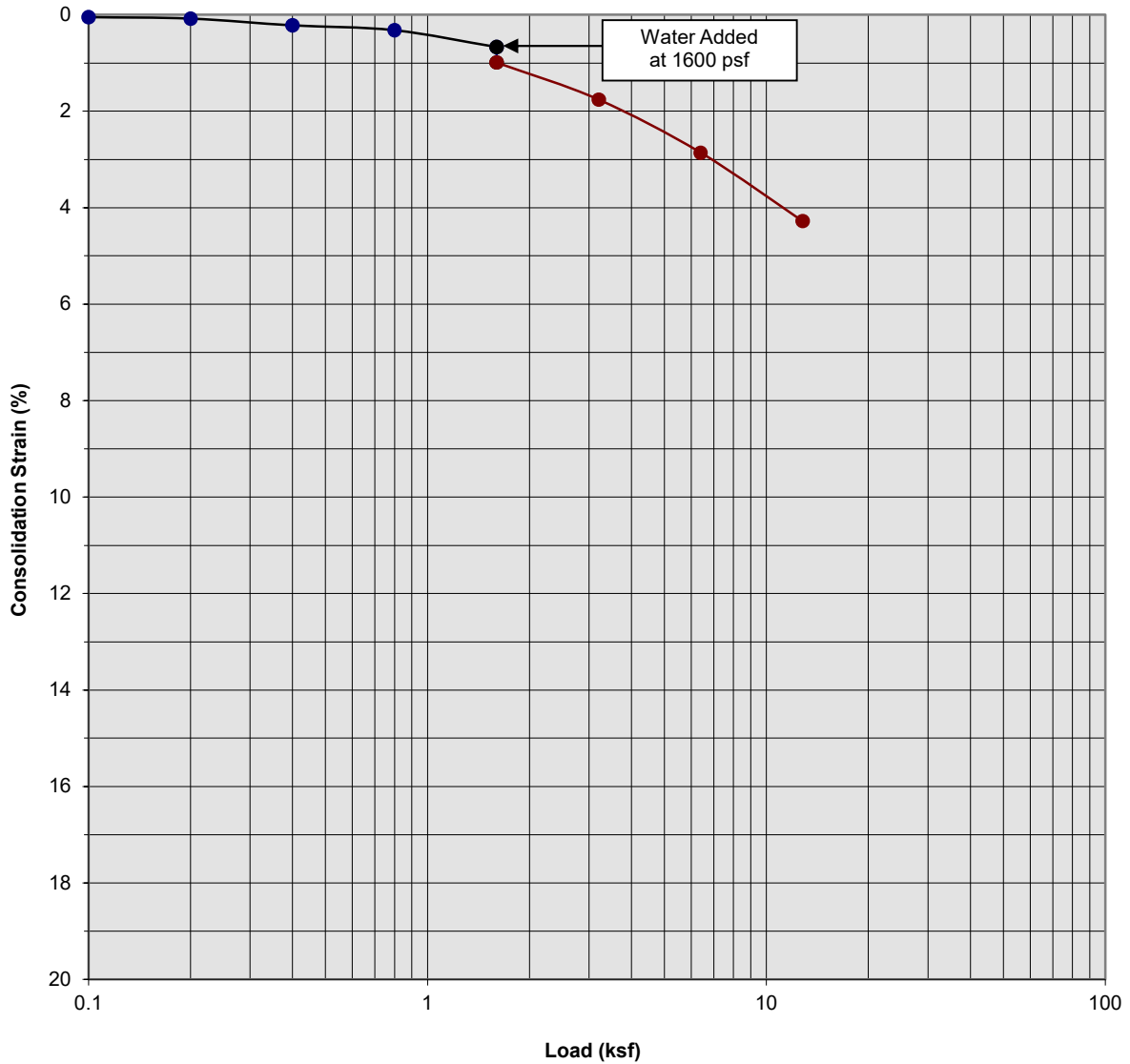
Proposed Industrial Building
Chino, California
Project No. 23G162-1

PLATE C- 4



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty fine Sand, trace medium Sand

Boring Number:	B-10	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	14
Depth (ft)	3 to 4	Initial Dry Density (pcf)	113.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	119.1
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.32

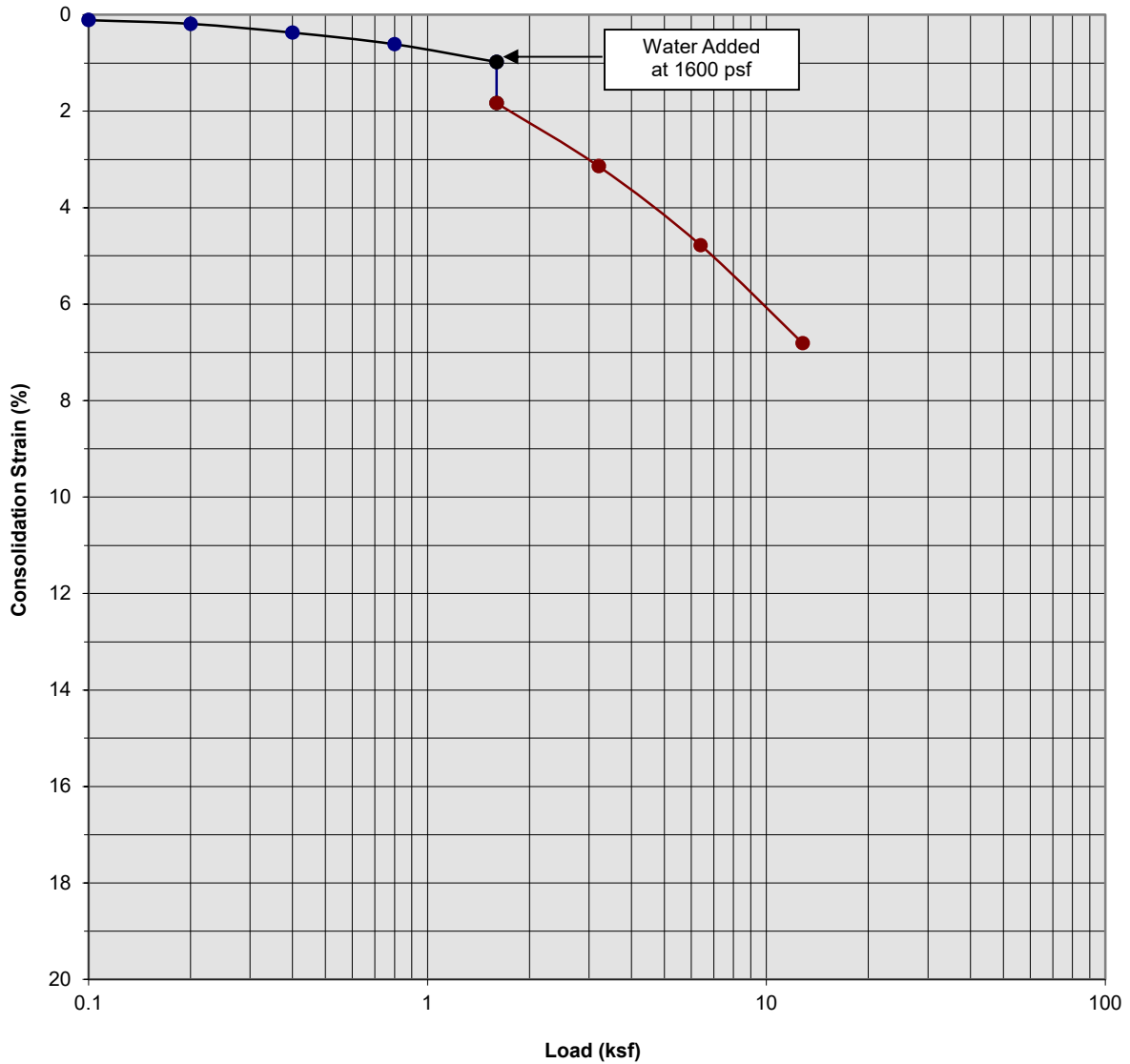
Proposed Industrial Building
 Chino, California
 Project No. 23G162-1

PLATE C- 5



**SOUTHERN
 CALIFORNIA
 GEOTECHNICAL**
A California Corporation

Consolidation/Collapse Test Results



Classification: Brown Silty fine Sand, trace medium Sand

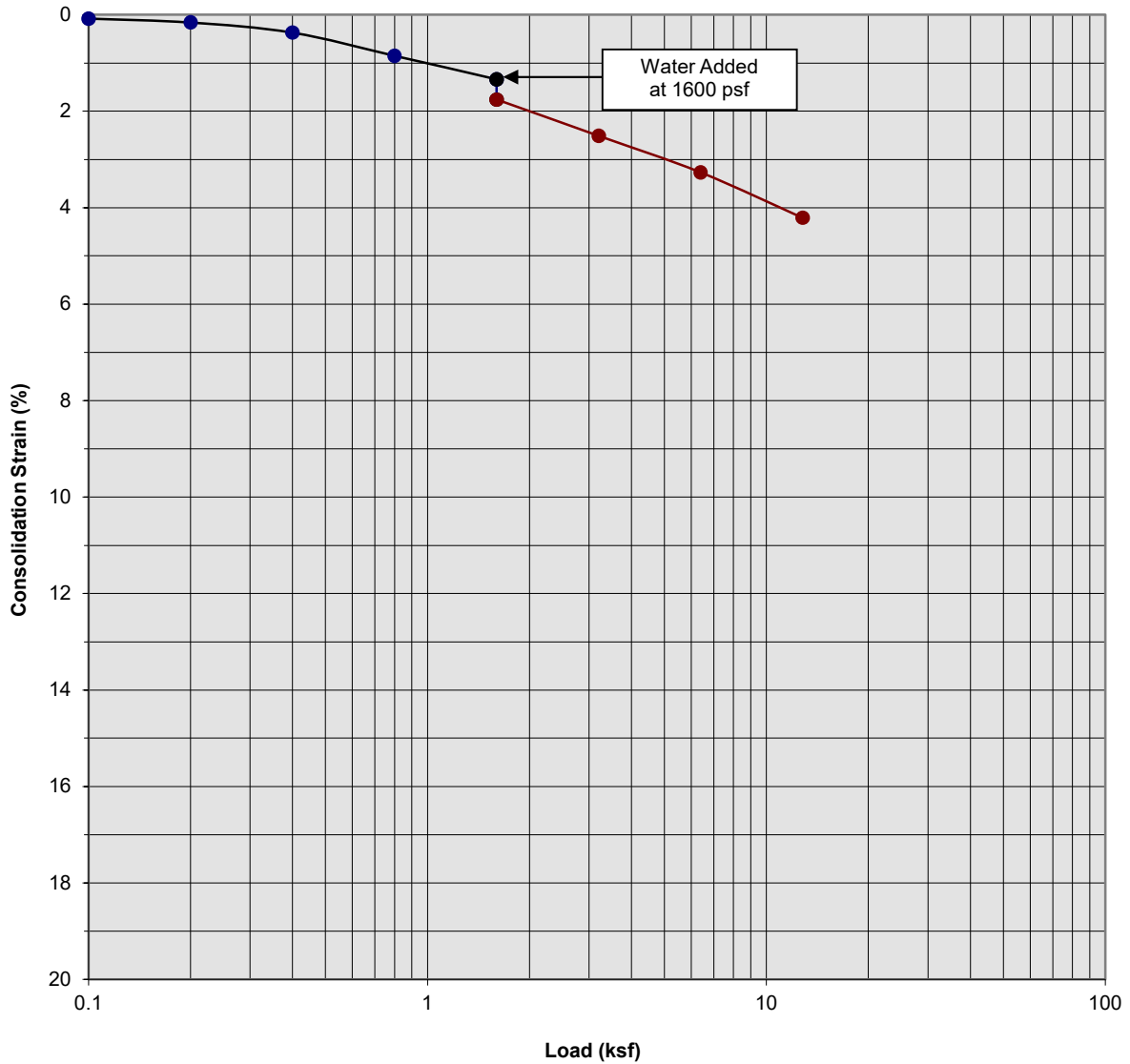
Boring Number:	B-10	Initial Moisture Content (%)	10
Sample Number:	---	Final Moisture Content (%)	16
Depth (ft)	5 to 6	Initial Dry Density (pcf)	110.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	118.3
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.85

Proposed Industrial Building
 Chino, California
 Project No. 23G162-1
PLATE C- 6



**SOUTHERN
 CALIFORNIA
 GEOTECHNICAL**
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Consolidation/Collapse Test Results



Classification: Light Gray fine to medium Sand, trace coarse Sand, little Silt

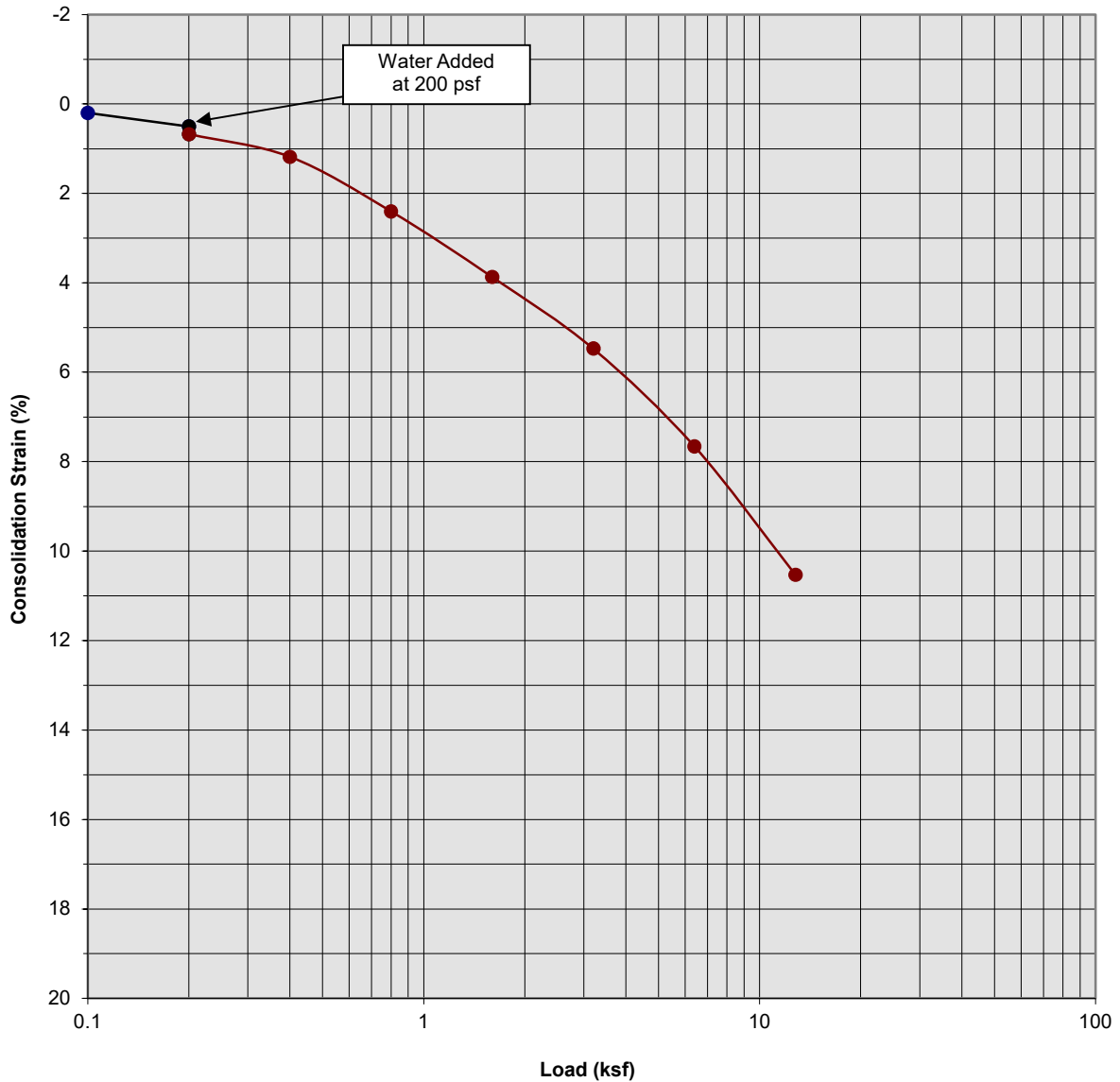
Boring Number:	B-10	Initial Moisture Content (%)	3
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	7 to 8	Initial Dry Density (pcf)	108.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	112.5
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.42

Proposed Industrial Building
 Chino, California
 Project No. 23G162-1
PLATE C- 7



**SOUTHERN
 CALIFORNIA
 GEOTECHNICAL**
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Consolidation/Collapse Test Results



Classification: Gray Clayey Silt, trace to little fine Sand

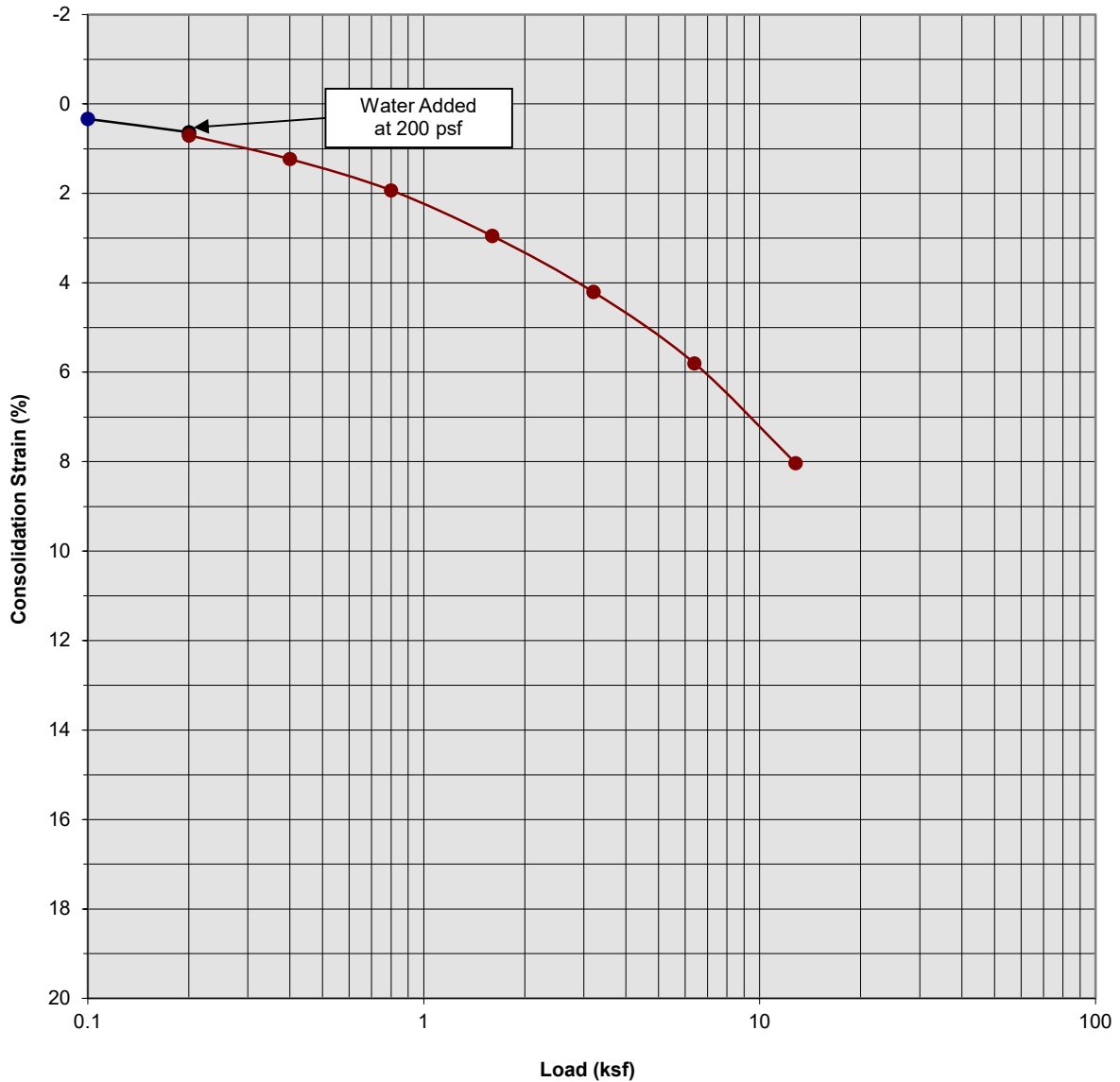
Boring Number:	B-10	Initial Moisture Content (%)	29
Sample Number:	---	Final Moisture Content (%)	26
Depth (ft)	9 to 10	Initial Dry Density (pcf)	97.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.17

Proposed Industrial Building
 Chino, California
 Project No. 23G162-1
PLATE C- 8



**SOUTHERN
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Consolidation/Collapse Test Results



Classification: Gray Clayey Silt, trace to little fine Sand

Boring Number:	B-10	Initial Moisture Content (%)	26
Sample Number:	---	Final Moisture Content (%)	24
Depth (ft)	14 to 15	Initial Dry Density (pcf)	99.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	107.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.07

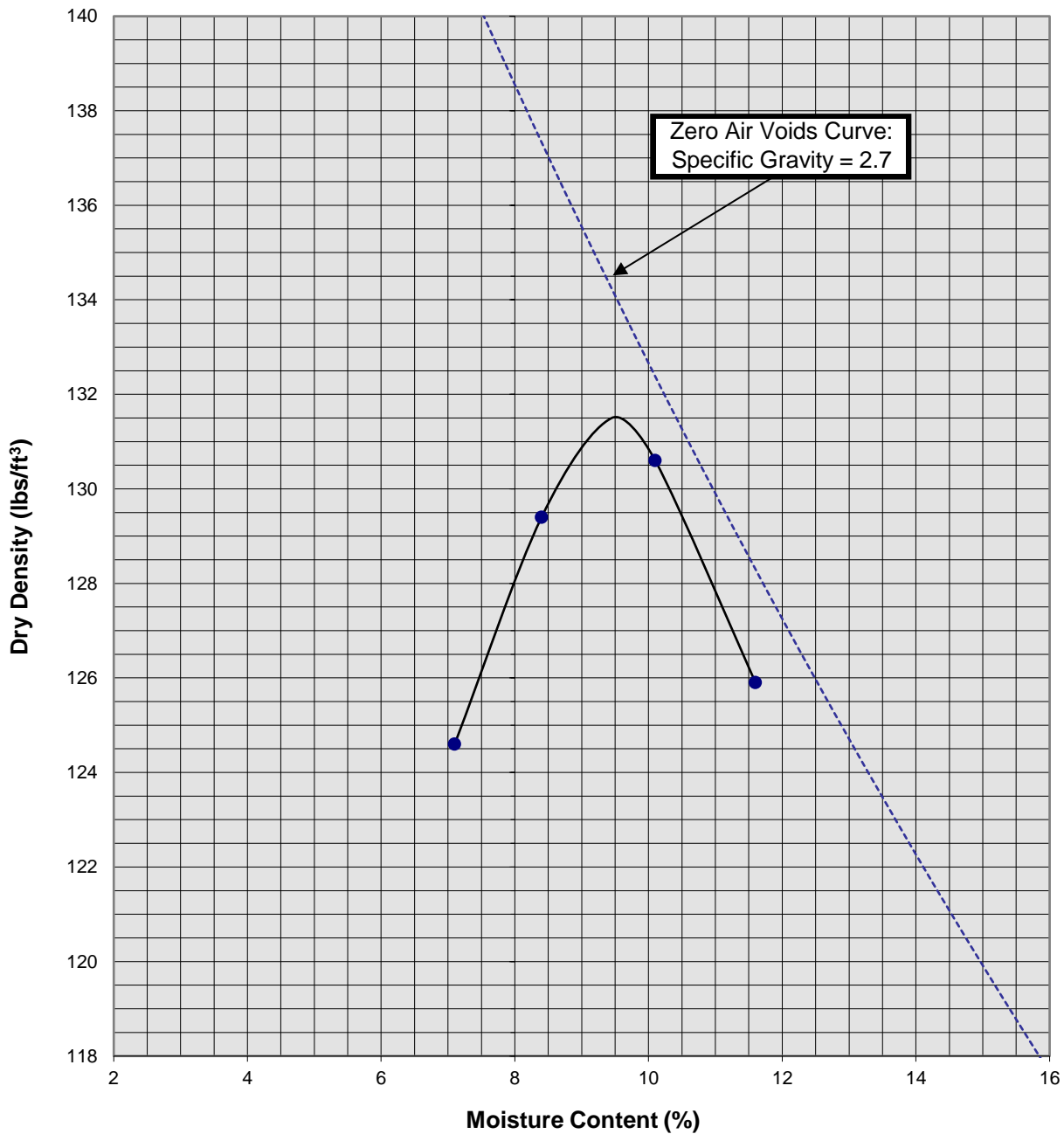
Proposed Industrial Building
 Chino, California
 Project No. 23G162-1

PLATE C- 9



**SOUTHERN
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Moisture/Density Relationship ASTM D-1557



Soil ID Number	B-10 @ 1-5'
Optimum Moisture (%)	9.5
Maximum Dry Density (pcf)	131.5
Soil Classification	Gray Brown Silty fine Sand, trace medium to coarse Sand, trace Clay

Proposed Industrial Building
Chino, California
Project No. 23G162-1
PLATE C- 10



SOUTHERN CALIFORNIA GEOTECHNICAL
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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

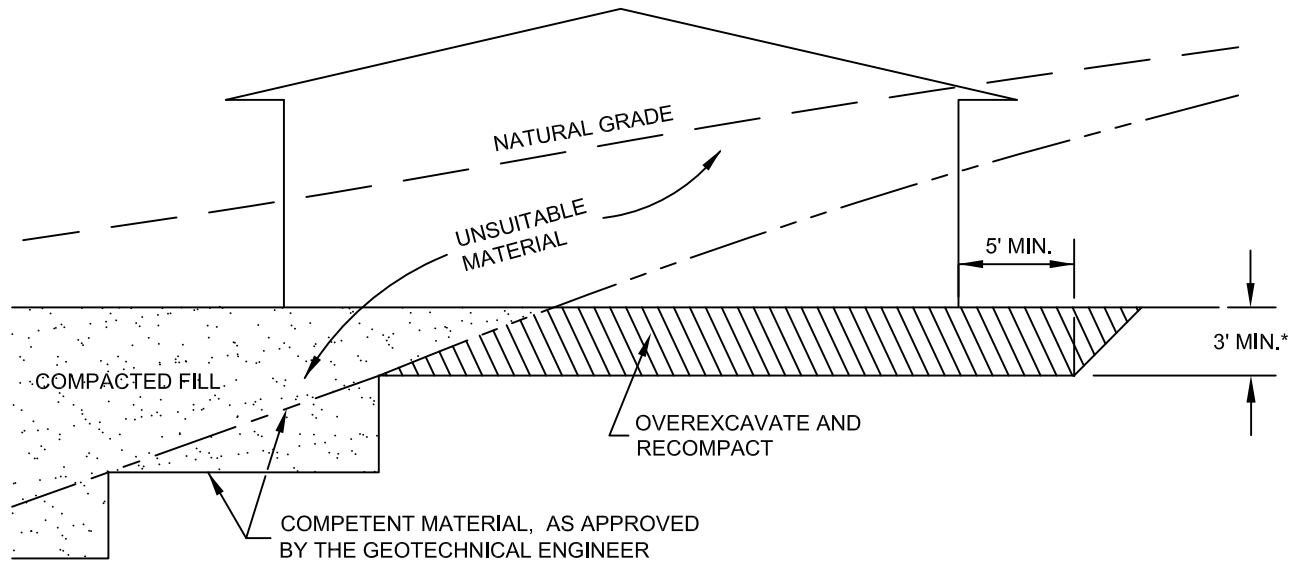
- All cut slopes should be inspected by the Geotechnical Engineer to determine the need for stabilization. The Earthwork Contractor should notify the Geotechnical Engineer when slope cutting is in progress at intervals of 10 vertical feet. Failure to notify may result in a delay in recommendations.
-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

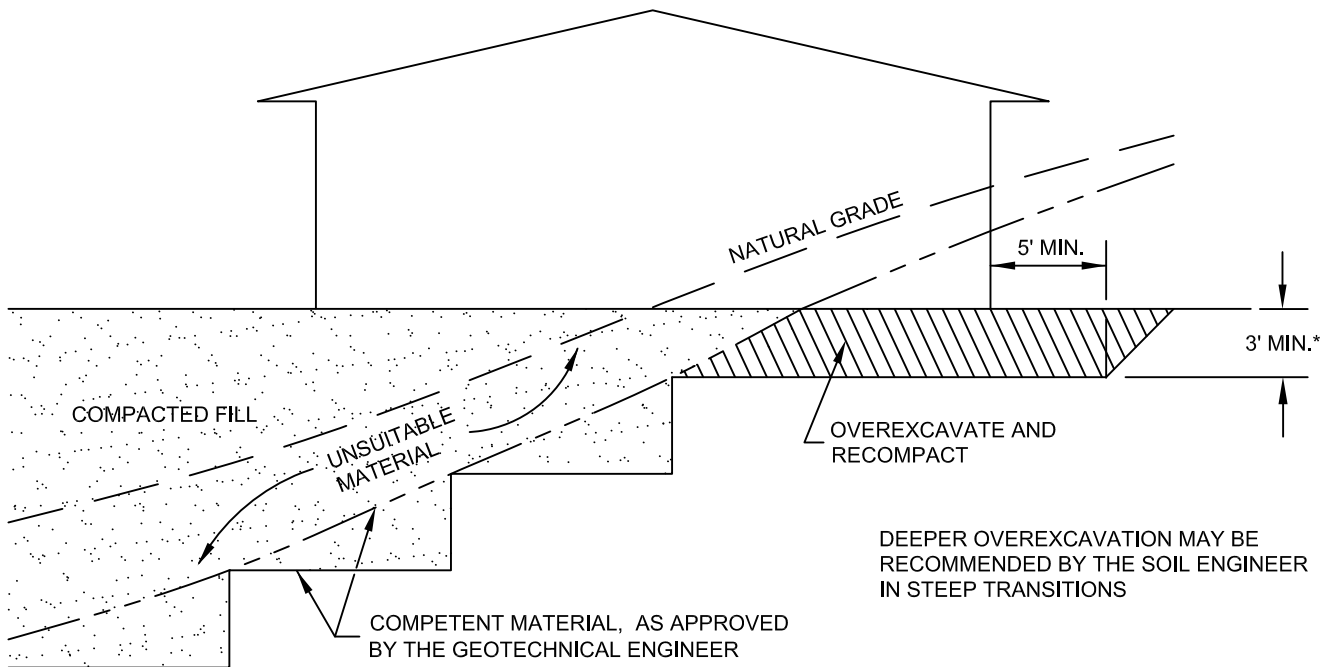
Subdrains

- 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 $\frac{3}{4}$ -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

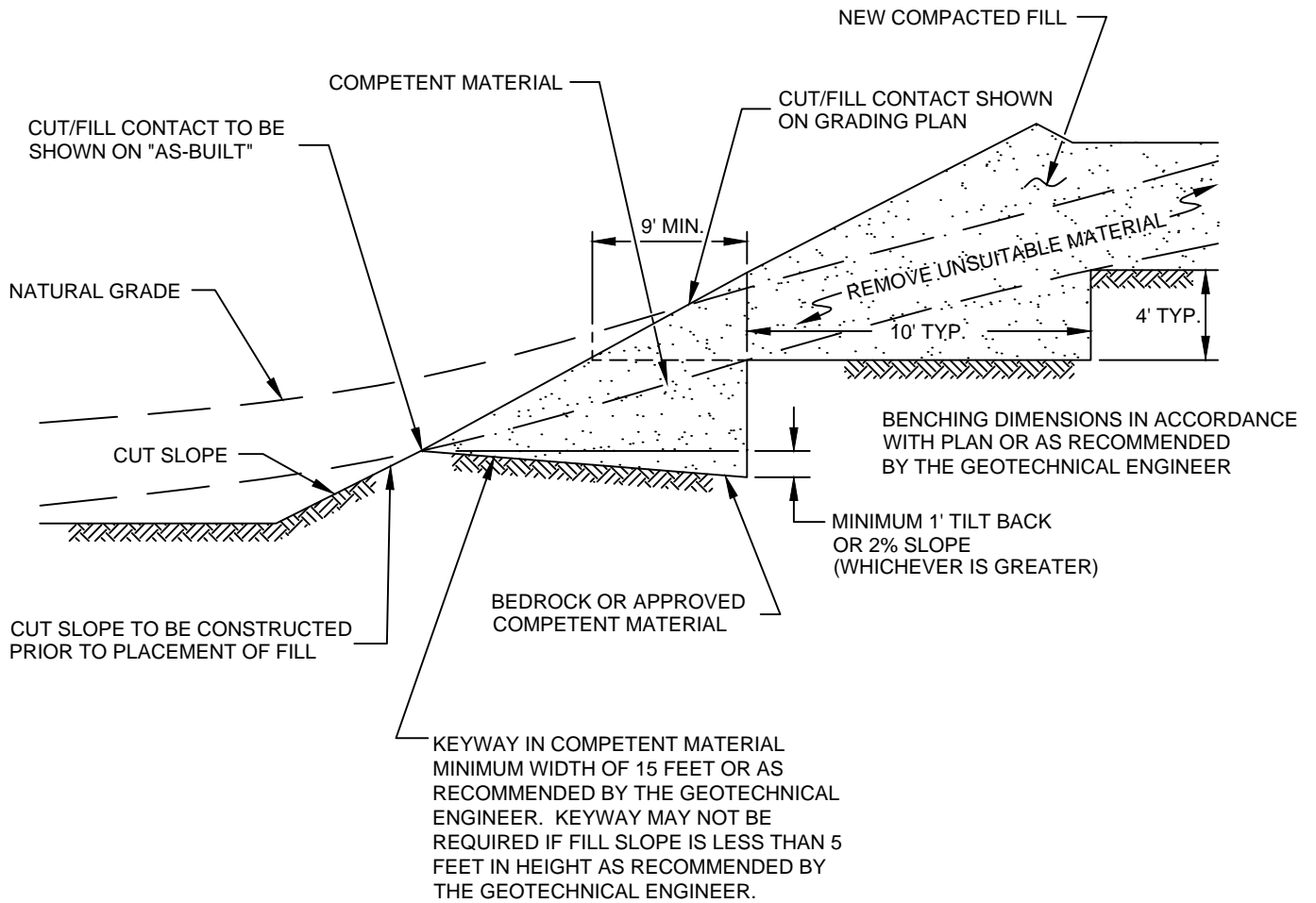


CUT/FILL LOT (TRANSITION)

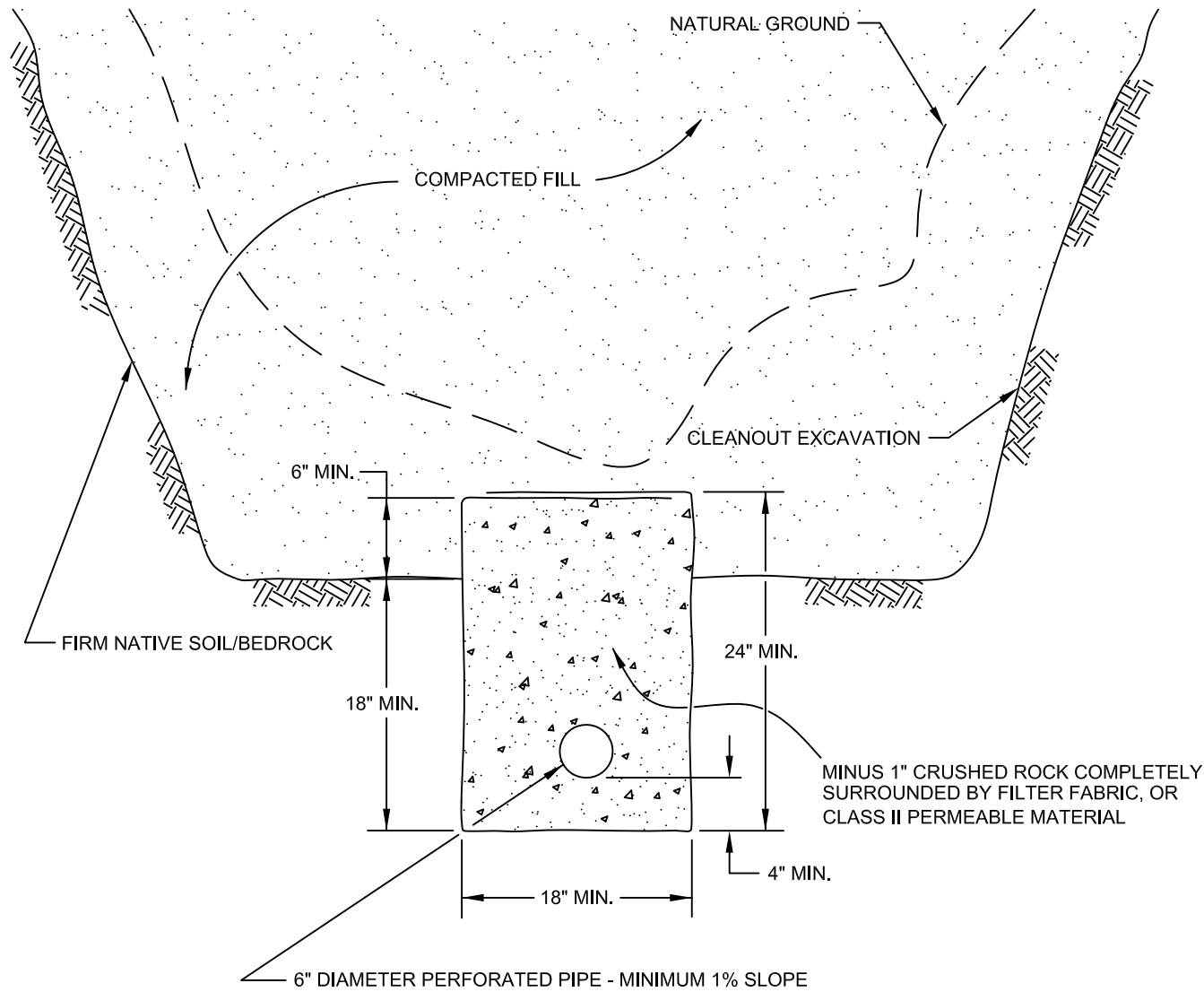


*SEE TEXT OF REPORT FOR SPECIFIC RECOMMENDATION.
ACTUAL DEPTH OF OVEREXCAVATION MAY BE GREATER.

TRANSITION LOT DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-1	




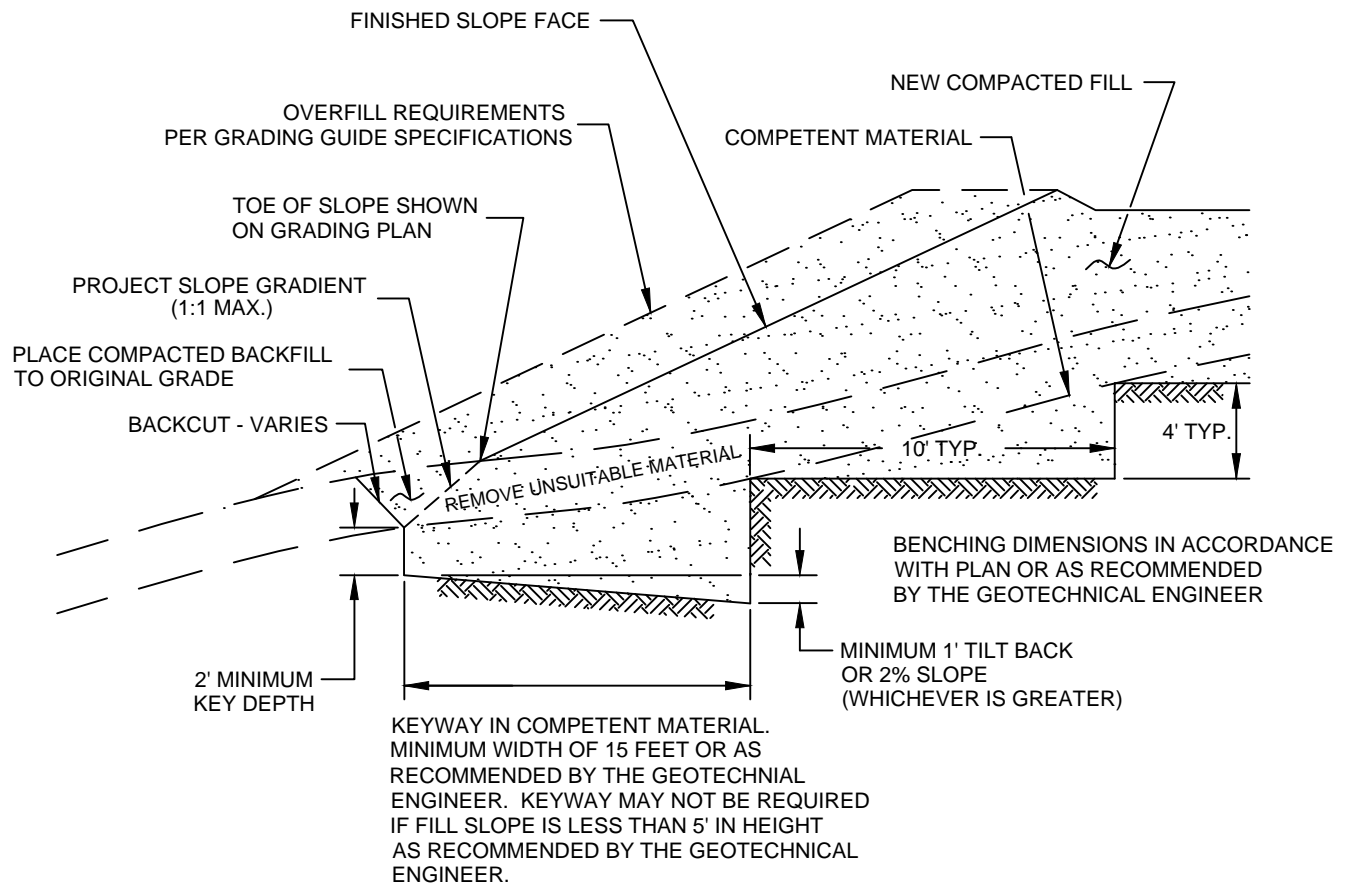
FILL ABOVE CUT SLOPE DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-2	




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

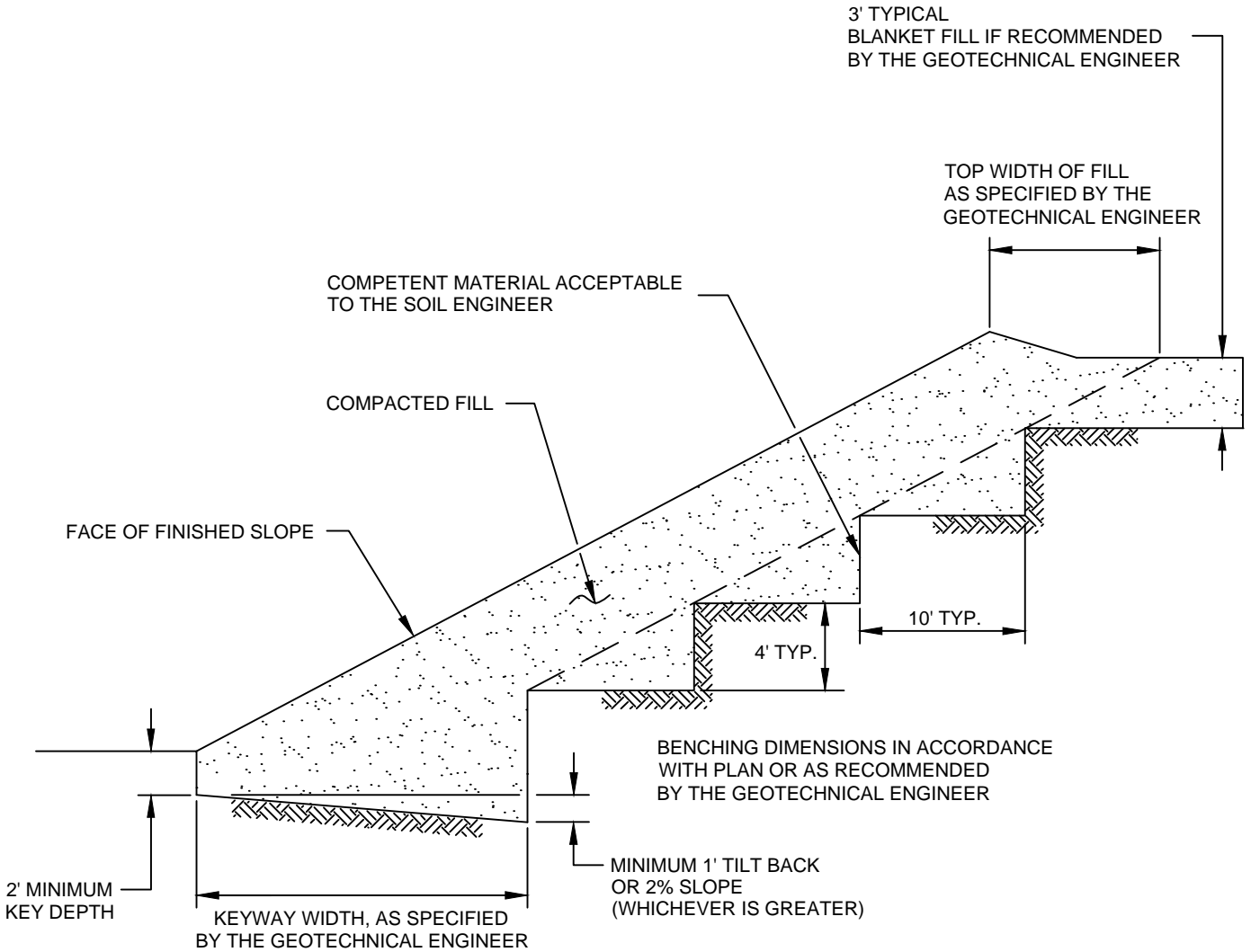
**SCHEMATIC ONLY
NOT TO SCALE**


CANYON SUBDRAIN DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-3	

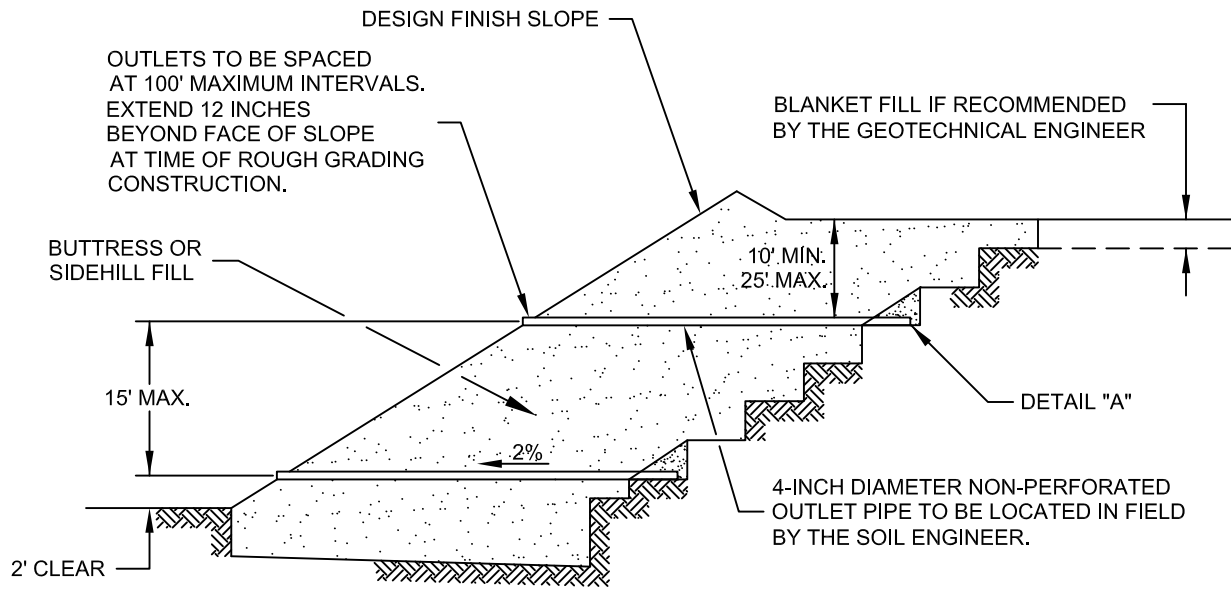


NOTE:
 BENCHING SHALL BE REQUIRED
 WHEN NATURAL SLOPES ARE
 EQUAL TO OR STEEPER THAN 5:1
 OR WHEN RECOMMENDED BY
 THE GEOTECHNICAL ENGINEER.

FILL ABOVE NATURAL SLOPE DETAIL	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-4	



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



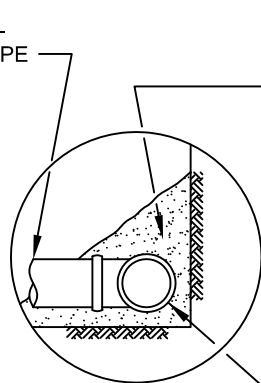
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

SIEVE SIZE	MAXIMUM PERCENTAGE PASSING
1 1/2"	100
NO. 4	50
NO. 200	8
SAND EQUIVALENT = MINIMUM OF 50	

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



DETAIL "A"

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


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

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

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

NOTES:

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

SLOPE FILL SUBDRAINS	
GRADING GUIDE SPECIFICATIONS	
NOT TO SCALE	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-6	

MINIMUM ONE FOOT THICK LAYER OF LOW PERMEABILITY SOIL IF NOT COVERED WITH AN IMPERMEABLE SURFACE

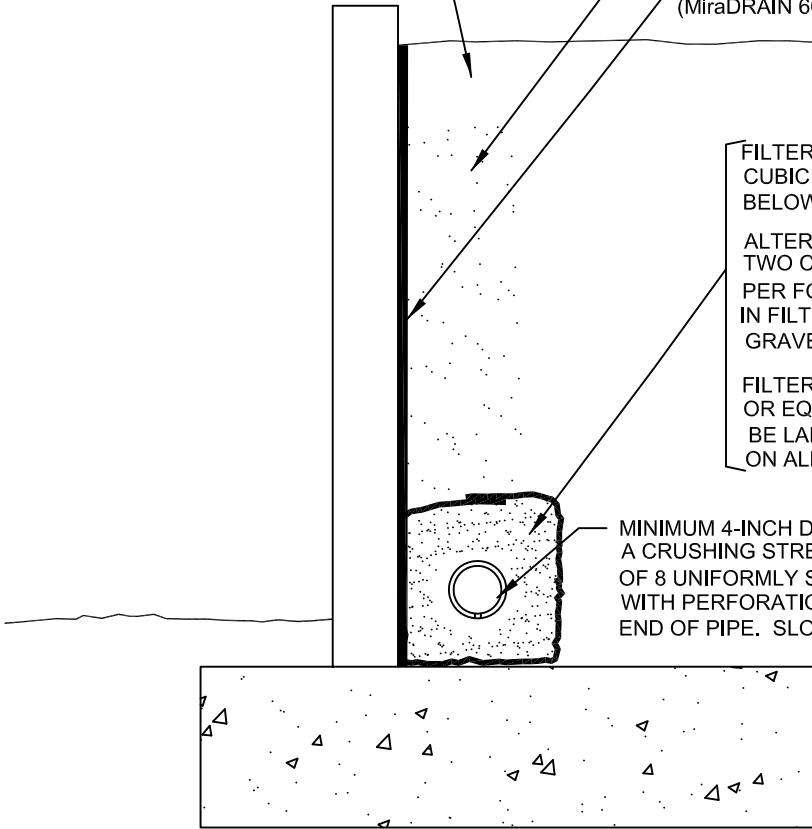
MINIMUM ONE FOOT WIDE LAYER OF FREE DRAINING MATERIAL (LESS THAN 5% PASSING THE #200 SIEVE) OR PROPERLY INSTALLED PREFABRICATED DRAINAGE COMPOSITE (MiraDRAIN 6000 OR APPROVED EQUIVALENT).

FILTER MATERIAL - MINIMUM OF TWO CUBIC FEET PER FOOT OF PIPE. SEE BELOW FOR FILTER MATERIAL SPECIFICATION.

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

FILTER FABRIC SHALL BE MIRAFAI 140 OR EQUIVALENT. FILTER FABRIC SHALL BE LAPPED A MINIMUM OF 6 INCHES ON ALL JOINTS.

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




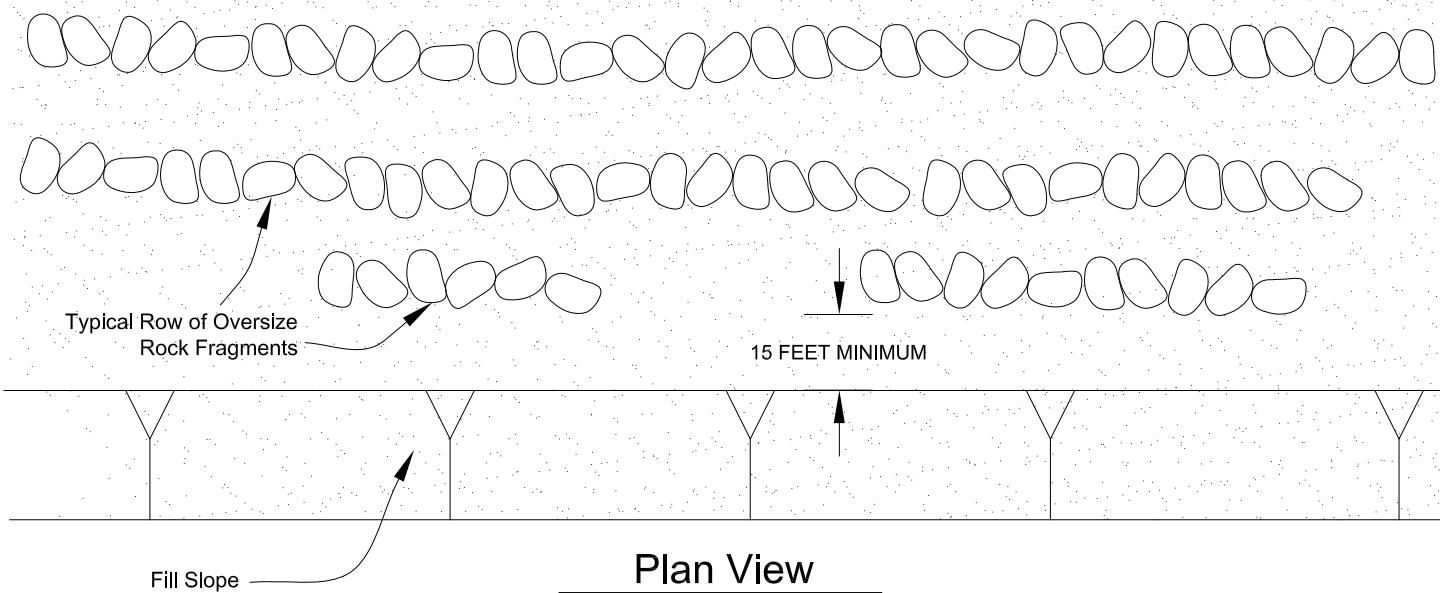
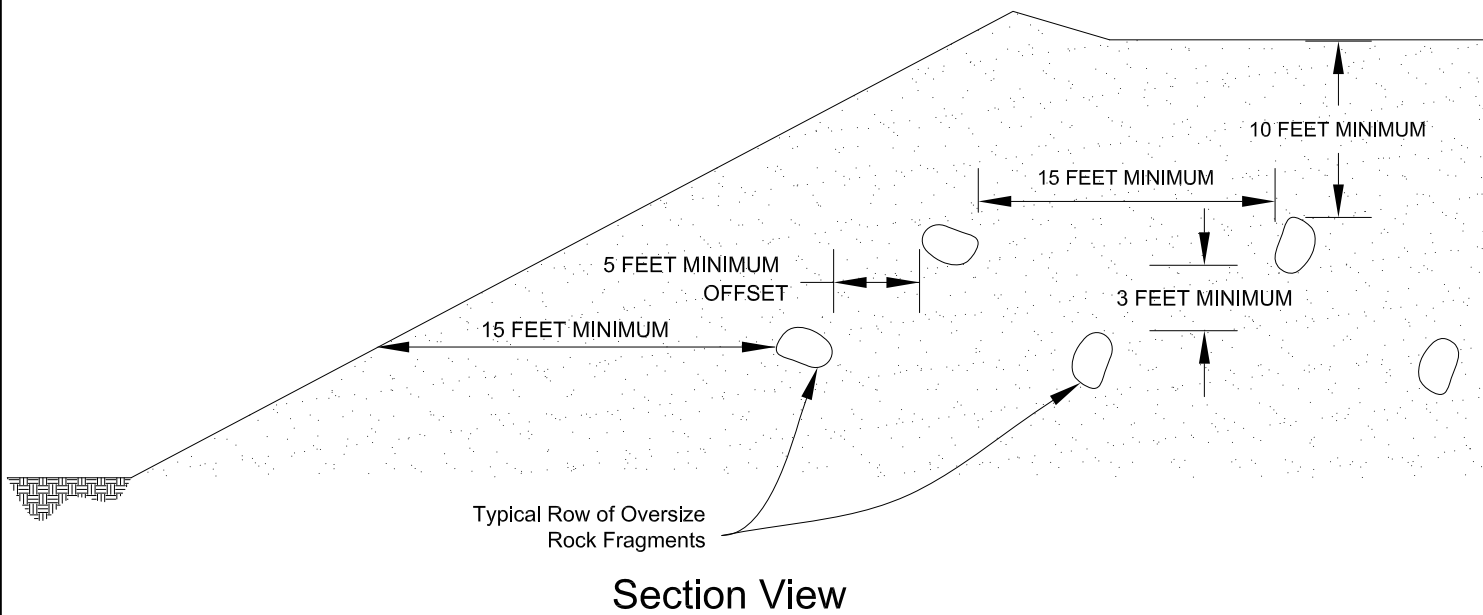
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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	 SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: JAS CHKD: GKM	
PLATE D-7	



**PLACEMENT OF OVERSIZED MATERIAL
GRADING GUIDE SPECIFICATIONS**

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8



**SOUTHERN
CALIFORNIA
GEOTECHNICAL**

APPENDIX E



Latitude, Longitude: 33.99879441, -117.69413558



Date	7/20/2023, 2:15:24 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Stiff Soil

Type	Value	Description
S_s	1.727	MCE_R ground motion. (for 0.2 second period)
S_1	0.617	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.727	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.151	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	0.73	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.803	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
$SsRT$	1.727	Probabilistic risk-targeted ground motion. (0.2 second)
$SsUH$	1.873	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.87	Factored deterministic acceleration value. (0.2 second)
$S1RT$	0.617	Probabilistic risk-targeted ground motion. (1.0 second)
$S1UH$	0.677	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
$S1D$	0.624	Factored deterministic acceleration value. (1.0 second)
PGA_d	0.767	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.73	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.922	Mapped value of the risk coefficient at short periods
C_{R1}	0.911	Mapped value of the risk coefficient at a period of 1 s
C_v	1.445	Vertical coefficient

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



SEISMIC DESIGN PARAMETERS - 2022 CBC	
PROPOSED INDUSTRIAL BUILDING	
CHINO, CALIFORNIA	
DRAWN: JLL CHKD: RGT SCG PROJECT 23G162-1 PLATE E-1	 SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX



JOB NO.: 12G120 DRILLING DATE: 3/13/12 WATER DEPTH: Dry
 PROJECT: Proposed C/I Bldg DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 47 feet
 LOCATION: Chino, California LOGGED BY: Eric Torres READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					SURFACE ELEVATION: --- MSL							
					4½ inches Asphaltic concrete, No discernible Aggregate base							
					FILL: Mottled Dark Gray Brown Silty fine Sand, medium dense-damp to moist	104	15					
					ALLUVIUM: Gray Brown Silty fine Sand to fine Sandy Silt, loose-moist	109	19					
5					Gray fine Sandy Silt, Iron oxide staining, medium dense-damp to moist	116	12					
					Gray Green Silty fine Sand, trace Iron oxide staining, trace Silt, medium dense-damp	111	9					
10					Light Gray fine Sand, medium dense-dry to damp	105	4					
15			1.25		Gray Brown Silty Clay trace Iron oxide staining, slightly porous, soft to stiff-very moist		27					
20			3.5			106	21					
25			1.0				38					
30			3.0		Light Gray Brown Clayey Silt, trace Iron oxide staining, medium stiff-very moist	95	28					
35					Brown fine Sandy Silt, trace Clay, trace Iron oxide staining, medium dense-moist		19					
					Light Gray Silty Clay, calcareous nodules, medium stiff-very moist	87	34					

TBL 12G120.GPJ SOCALGEO.GDT 3/29/12



JOB NO.: 12G120	DRILLING DATE: 3/13/12	WATER DEPTH: Dry
PROJECT: Proposed C/I Bldg	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 47 feet
LOCATION: Chino, California	LOGGED BY: Eric Torres	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION (Continued)	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
45	X	7	2.5		Light Gray Silty Clay, calcareous nodules, medium stiff-very moist		38					
50	X	53			Light Gray Brown Clayey Silt to Silty Clay, abundant Iron oxide staining, medium stiff-very moist							
					Gray Brown fine to coarse Sand, very dense-moist	92 117	35 12					
Boring Terminated at 50'												

TBL 12G120.GPJ_SOCALGEO.GDT 3/29/12



JOB NO.: 12G120 DRILLING DATE: 3/13/12 WATER DEPTH: Dry
 PROJECT: Proposed C/I Bldg DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 28 feet
 LOCATION: Chino, California LOGGED BY: Eric Torres READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					SURFACE ELEVATION: --- MSL							
					2 inches Aggregate base							
		16			FILL: Dark Gray Brown Silty fine Sand, mottled, medium dense-moist	102	19					
		15			ALLUVIUM: Gray Brown Silty fine Sand, slightly porous, loose to medium dense-damp to moist	110	11					
5		14			Gray Green Silty fine Sand to fine Sandy Silt, Iron oxide staining, slightly porous, loose-moist	113	15					
		18			Gray Green Silty fine Sand, trace Iron oxide staining, medium dense-damp to moist	112	5					
		9	2.0		Light Gray Brown fine Sand, trace Iron oxide staining, trace medium to coarse Sand, medium dense-moist	104	11					
10					Gray Brown Silty Clay, Iron oxide staining, porous, medium stiff-moist to very moist							
		6	2.5				33					
15												
		33			Gray Brown Silty fine Sand to fine Sandy Silt, abundant Iron oxide staining, medium dense-damp to moist	109	12					
		6	2.0		Light Gray Brown Silty Clay, trace Iron oxide staining, medium stiff-very moist		26					
25												
		9	2.0			107	23					
30												
					Boring Terminated at 30'							

TBL 12G120.GPJ_SOCALGEO.GDT 3/29/12



JOB NO.: 12G120 DRILLING DATE: 3/13/12 WATER DEPTH: Dry
 PROJECT: Proposed C/I Bldg DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 32 feet
 LOCATION: Chino, California LOGGED BY: Eric Torres READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					SURFACE ELEVATION: --- MSL							
					2 inches Aggregate base							
		17			FILL: Dark Gray Brown Silty fine Sand, mottled, medium dense-dry to damp	93	17					
		22			ALLUVIUM: Gray Brown Silty fine Sand, trace calcareous veining, medium dense-moist	105	14					
5		23			Light Gray Brown Silty fine Sand to fine Sandy Silt, trace calcareous veining, medium dense-moist	105	17					
		20			@ 7 feet, abundant Iron oxide staining	101	9					
10		16	3.75		Gray Green Clayey Silt, Iron oxide staining, medium stiff-moist	95	15					
		5	1.75		Gray Brown Silty Clay, trace Iron oxide staining, slightly porous, medium stiff-very moist	85	35					
		30			Light Gray Brown Gray fine Sandy Silt, trace to little Clay, abundant Iron oxide staining		26					
20					Gray Brown fine to coarse Sand, trace fine to coarse Gravel, trace Iron oxide staining, medium dense-damp		6					
					@ 22 to 28½ feet, dense-dry							
25		56					3					
		10	2.25		Gray Brown Silty Clay, stiff-moist to very moist		19					
30					Gray Brown fine to coarse Sand, fine to coarse Gravel, trace Silt, abundant Iron oxide staining, dense-damp to moist							
35		50				119	9					
					Boring Terminated at 35'							

TBL 12G120.GPJ_SOCALGEO.GDT 3/29/12



JOB NO.: 12G120 DRILLING DATE: 3/13/12 WATER DEPTH: Dry
 PROJECT: Proposed C/I Bldg DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 18 feet
 LOCATION: Chino, California LOGGED BY: Eric Torres READING TAKEN: At Completion

FIELD RESULTS					LABORATORY RESULTS						COMMENTS	
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)		UNCONFINED SHEAR (TSF)
SURFACE ELEVATION: --- MSL												
					4 inches Asphaltic concrete, No discernible Aggregate base							
					FILL: Dark Gray Brown Silty fine Sand, mottled, medium dense-very moist	88	29					
					ALLUVIUM: Gray Brown Silty fine Sand, trace calcareous veining and nodules, sightly porous, loose-moist	105	20					
5			4.5		Light Gray Brown Silty Clay, trace Iron oxide staining, trace calcareous nodules, medium stiff-moist	107	19					
					Dark Gray Green Silty fine Sand, trace Iron oxide staining, loose to medium dense-moist	114	13					
10			3.25		Gray Green fine Sand, trace Iron oxide staining, loose to medium dense-damp	106	5					
					Gray Brown Silty Clay, trace Iron oxide staining, medium stiff-moist							
15			3.0		@ 13½ feet, porous, stiff-very moist		28					
					Olive Brown Silty fine Sand, trace Clay abundant Iron oxide staining, medium dense-very moist		19					
20					Boring Terminated at 20'							

TBL 12G120.GPJ SOCALGEO.GDT 3/29/12



JOB NO.: 12G120 DRILLING DATE: 3/13/12 WATER DEPTH: Dry
 PROJECT: Proposed C/I Bldg DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 19 feet
 LOCATION: Chino, California LOGGED BY: Eric Torres READING TAKEN: At Completion

FIELD RESULTS				GRAPHIC LOG	DESCRIPTION	LABORATORY RESULTS					COMMENTS	
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)			DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)		UNCONFINED SHEAR (TSF)
SURFACE ELEVATION: --- MSL												
				3 1/2 inches Asphaltic concrete, No discernible Aggregate base								
				FILL: Dark Gray Brown to Black Silty fine Sand, mottled, loose-very moist		28						EI = 10 @ 0 to 5'
5				ALLUVIUM: Light Gray Brown Silty fine Sand to fine Sandy Silt, trace Clay, trace calcareous nodules, medium dense-moist to very moist		18						
				Gray Green Silty fine Sand, trace Iron oxide staining, loose-moist		14						
10				Gray Brown fine Sand, medium dense-damp		4						
15			1.5	Gray Brown Silty Clay, trace Iron oxide staining, medium stiff-very moist		28						
20				Light Gray Brown fine Sandy Silt, trace Iron oxide staining, loose-very moist		22						
			2.0	Gray Brown Gray Silty Clay, trace calcareous nodules, trace Iron oxide staining, stiff-very moist	105	19						
Boring Terminated at 21'												

TBL 12G120.GPJ_SOCALGEO.GDT 3/29/12



JOB NO.: 12G120 DRILLING DATE: 3/13/12 WATER DEPTH: Dry
 PROJECT: Proposed C/I Bldg DRILLING METHOD: Hollow Stem Auger CAVE DEPTH: 28 feet
 LOCATION: Chino, California LOGGED BY: Eric Torres READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
					SURFACE ELEVATION: --- MSL							
					5½ inches Asphaltic concrete, No discernible Aggregate base							
	16				FILL: Dark Gray Brown to Black Silty fine Sand, mottled, medium dense-very moist	89	29					
	9				ALLUVIUM: Light Gray Brown Silty fine Sand to fine Sandy Silt, porous, very loose to loose-very moist	89	27					
5	6					87	26					
	23				Gray Green fine Sandy Silt, abundant Iron oxide staining, medium dense-moist	111	17					
10	9		3.75		Gray Brown fine Sand, trace to little Silt, abundant Iron oxide staining,	112	8					
					Gray Brown Silty Clay, abundant Iron oxide staining, slightly porous, stiff-damp to moist							
					Gray Brown fine Sandy Silt, trace to little Clay, Iron oxide staining, stiff-very moist		26					
15	10											
					Gray Brown fine to coarse Sand, little fine to coarse Gravel, trace Iron oxide staining, dense-damp	119	5					
20	54											
					Light Gray Brown Silty Clay, trace calcareous nodules, trace Iron oxide staining, medium stiff-very moist		34					
25	5		1.0									
					Brown fine Sandy Clay to Clayey fine Sand, abundant Iron oxide staining, medium dense to stiff-very moist							
30	17		4.5			107	20					
					Boring Terminated at 30'							

TBL 12G120.GPJ_SOCALGEO.GDT 3/29/12



FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
JOB NO.: 12G120					DRILLING DATE: 3/13/12					WATER DEPTH: Dry		
PROJECT: Proposed C/I Bldg					DRILLING METHOD: Hollow Stem Auger					CAVE DEPTH: ---		
LOCATION: Chino, California					LOGGED BY: Eric Torres					READING TAKEN: At Completion		
SURFACE ELEVATION: --- MSL												
	X	12		[Symbol]	2 inches Aggregate base FILL: Dark Gray Brown Silty fine Sand, loose-moist		16					
	X	11		[Symbol]	ALLUVIUM: Light Gray Brown Silty fine Sand to fine Sandy Silt, trace calcareous nodules, loose-moist		17					
5					Boring Terminated at 5'							

TBL_12G120.GPJ_SOCALGEO.GDT 3/29/12

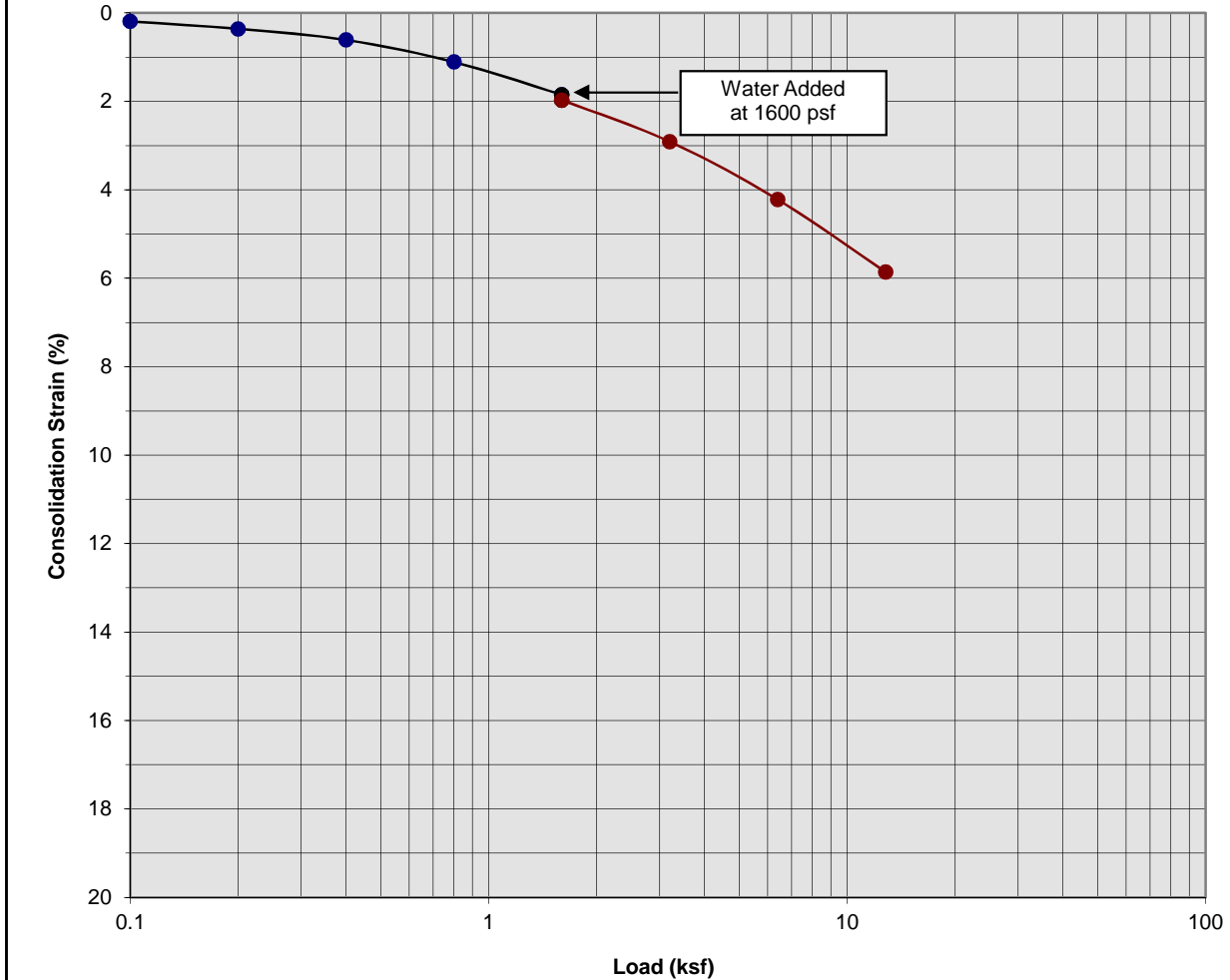


JOB NO.: 12G120	DRILLING DATE: 3/13/12	WATER DEPTH: Dry
PROJECT: Proposed C/I Bldg	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: ---
LOCATION: Chino, California	LOGGED BY: Eric Torres	READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						COMMENTS
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG		DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	UNCONFINED SHEAR (TSF)	
SURFACE ELEVATION: --- MSL												
	X	7		[Pattern]	4 inches Asphaltic concrete, no discernible Aggregate base		20					
	X	6		[Pattern]	<u>FILL</u> : Dark Gray Brown Silty fine Sand, loose-moist to very moist							
5				[Pattern]	<u>ALLUVIUM</u> : Light Gray Brown Silty fine Sand to fine Sandy Silt, trace Iron oxide staining, loose-moist		16					
					Boring Terminated at 5'							

TBL_12G120.GPJ_SOCALGEO.GDT 3/29/12

Consolidation/Collapse Test Results



Classification: Gray Brown Silty fine Sand to fine Sandy Silt

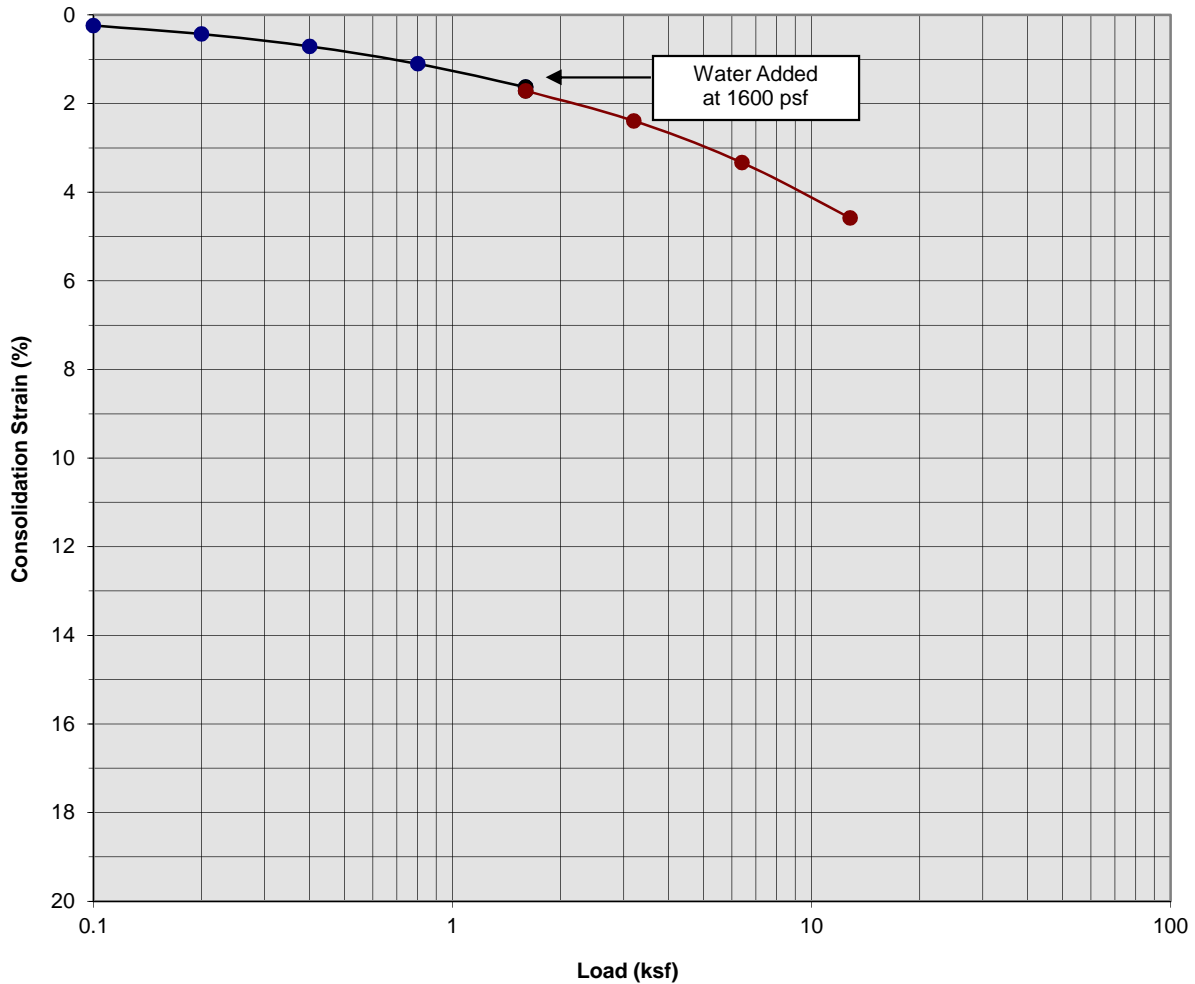
Boring Number:	B-1	Initial Moisture Content (%)	19
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	3 to 4	Initial Dry Density (pcf)	108.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.12

Proposed C/I Bldg
 Chino, California
 Project No. 12G120
PLATE C- 1



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Consolidation/Collapse Test Results



Classification: Gray Brown fine Sandy Silt

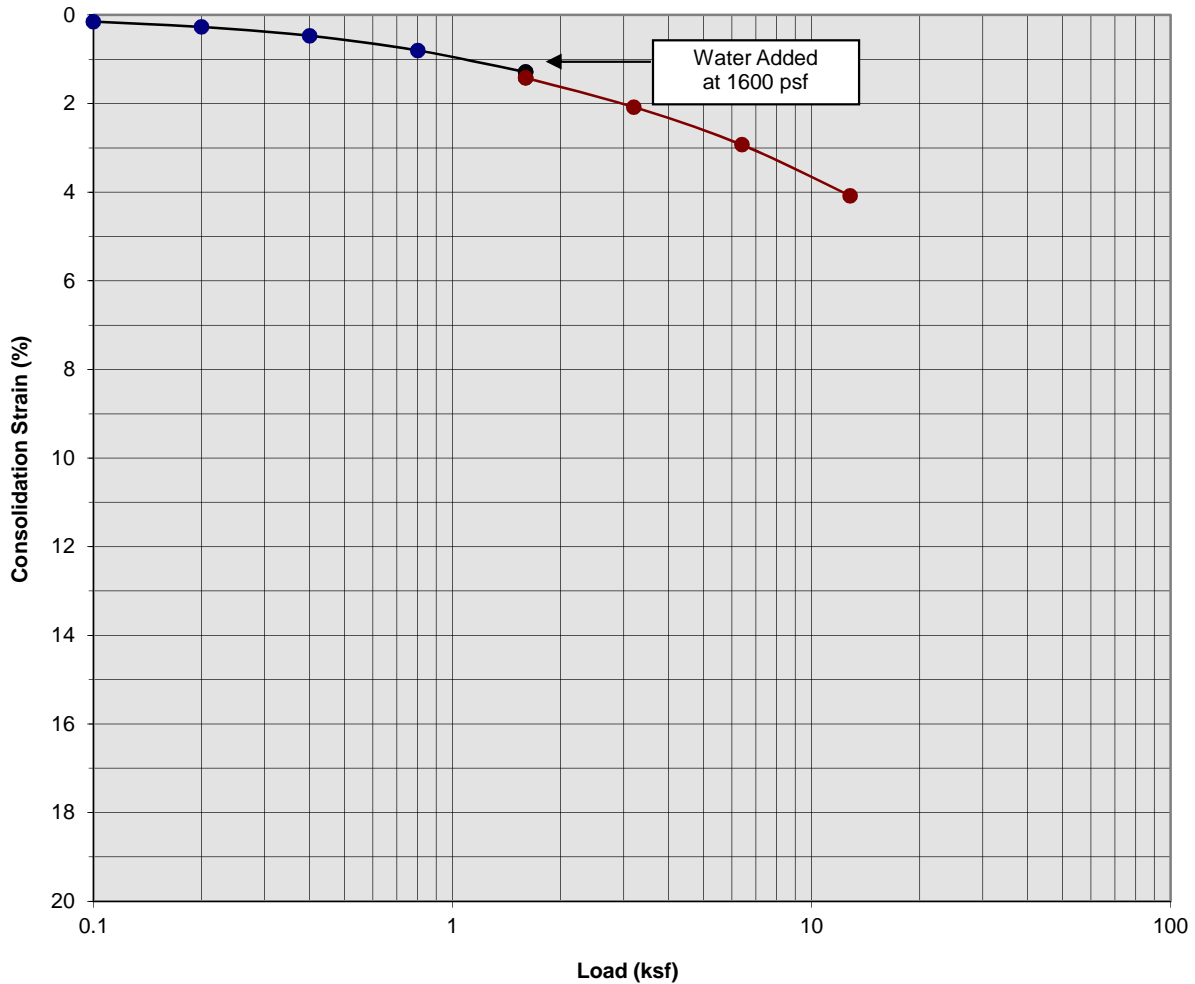
Boring Number:	B-1	Initial Moisture Content (%)	12
Sample Number:	---	Final Moisture Content (%)	15
Depth (ft)	5 to 6	Initial Dry Density (pcf)	116.1
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	120.8
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.08

Proposed C/I Bldg
Chino, California
Project No. 12G120
PLATE C- 2



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Consolidation/Collapse Test Results



Classification: Gray Green Silty fine Sand

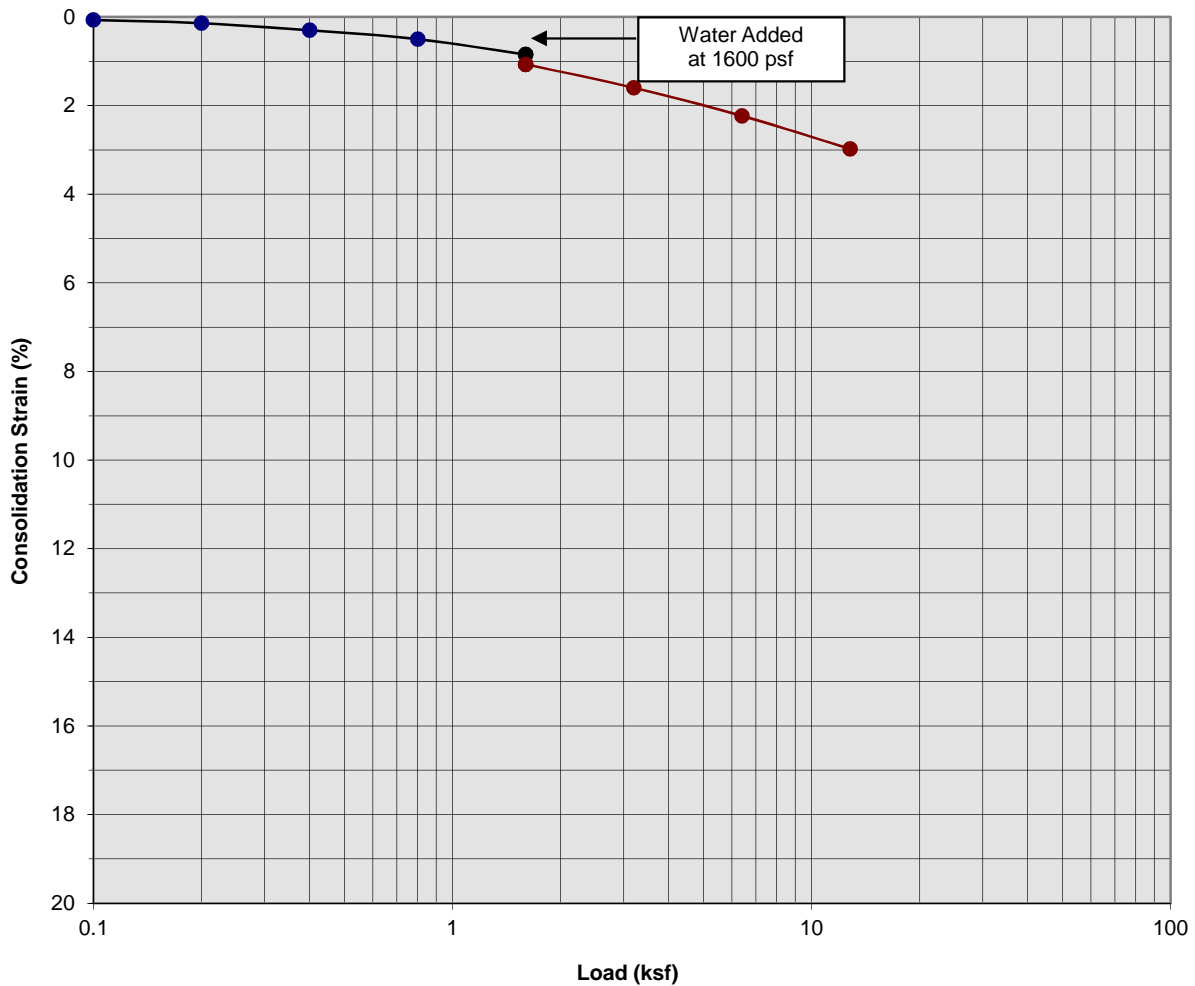
Boring Number:	B-1	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	7 to 8	Initial Dry Density (pcf)	106.0
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	109.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.13

Proposed C/I Bldg
Chino, California
Project No. 12G120
PLATE C- 3



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Consolidation/Collapse Test Results



Classification: Light Gray fine Sand

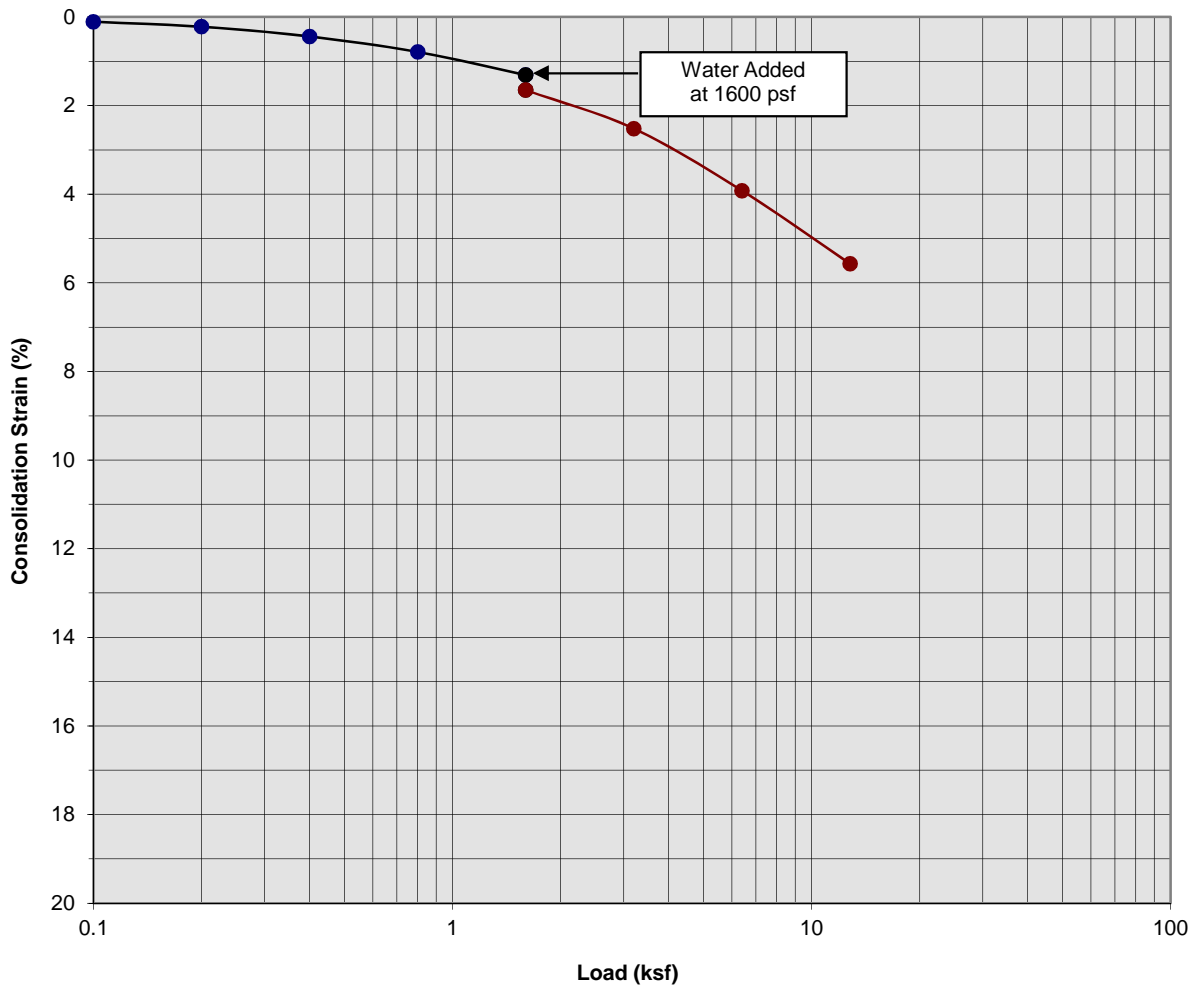
Boring Number:	B-1	Initial Moisture Content (%)	8
Sample Number:	---	Final Moisture Content (%)	19
Depth (ft)	9 to 10	Initial Dry Density (pcf)	107.6
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	108.2
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.22

Proposed C/I Bldg
Chino, California
Project No. 12G120
PLATE C- 4



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Consolidation/Collapse Test Results



Classification: Light Gray Brown Silty fine Sand to fine Sandy Silt

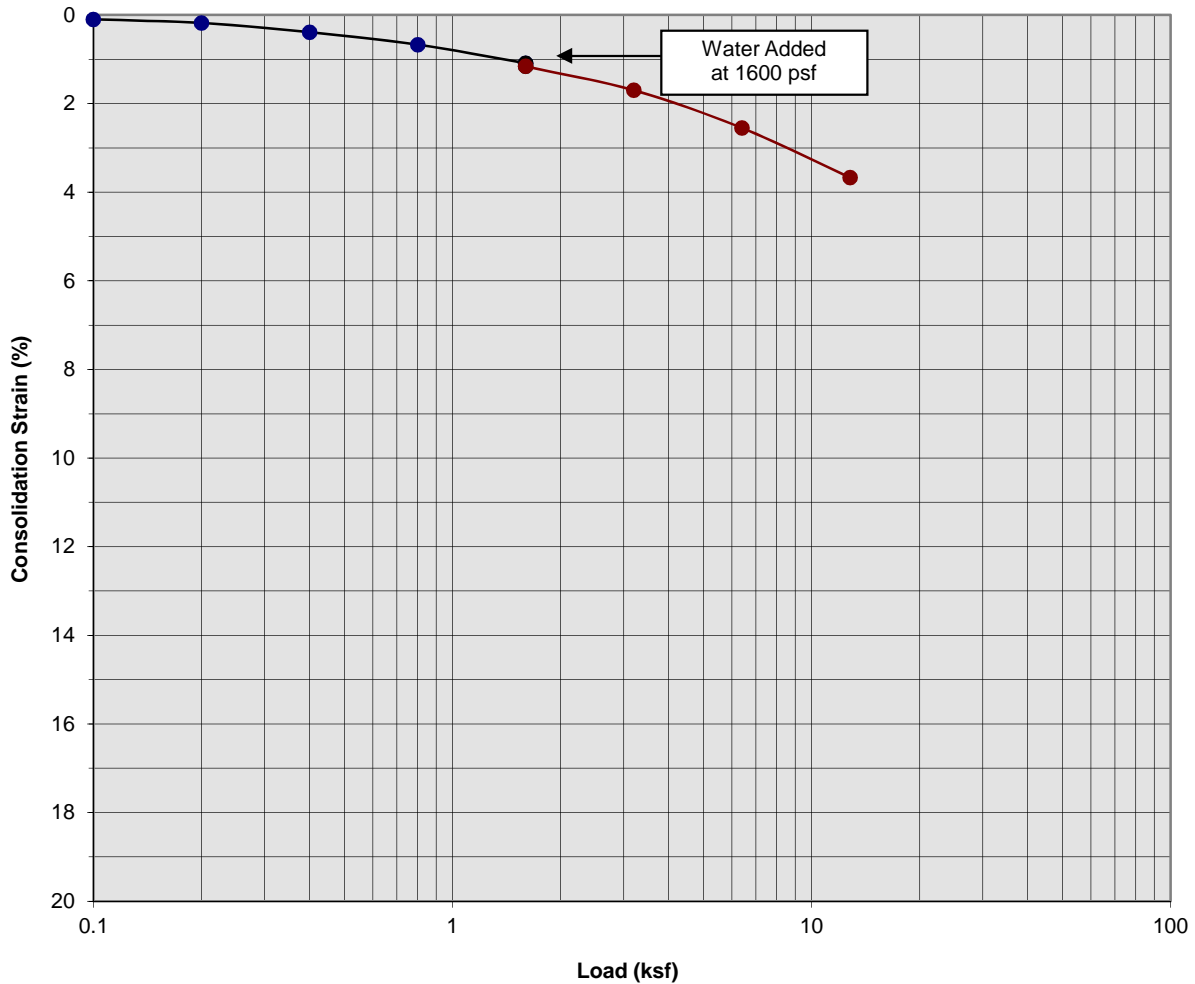
Boring Number:	B-3	Initial Moisture Content (%)	17
Sample Number:	---	Final Moisture Content (%)	21
Depth (ft)	5 to 6	Initial Dry Density (pcf)	103.7
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	110.9
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.34

Proposed C/I Bldg
Chino, California
Project No. 12G120
PLATE C- 5



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Consolidation/Collapse Test Results



Classification: Light Gray Brown Silty fine Sand to fine Sandy Silt

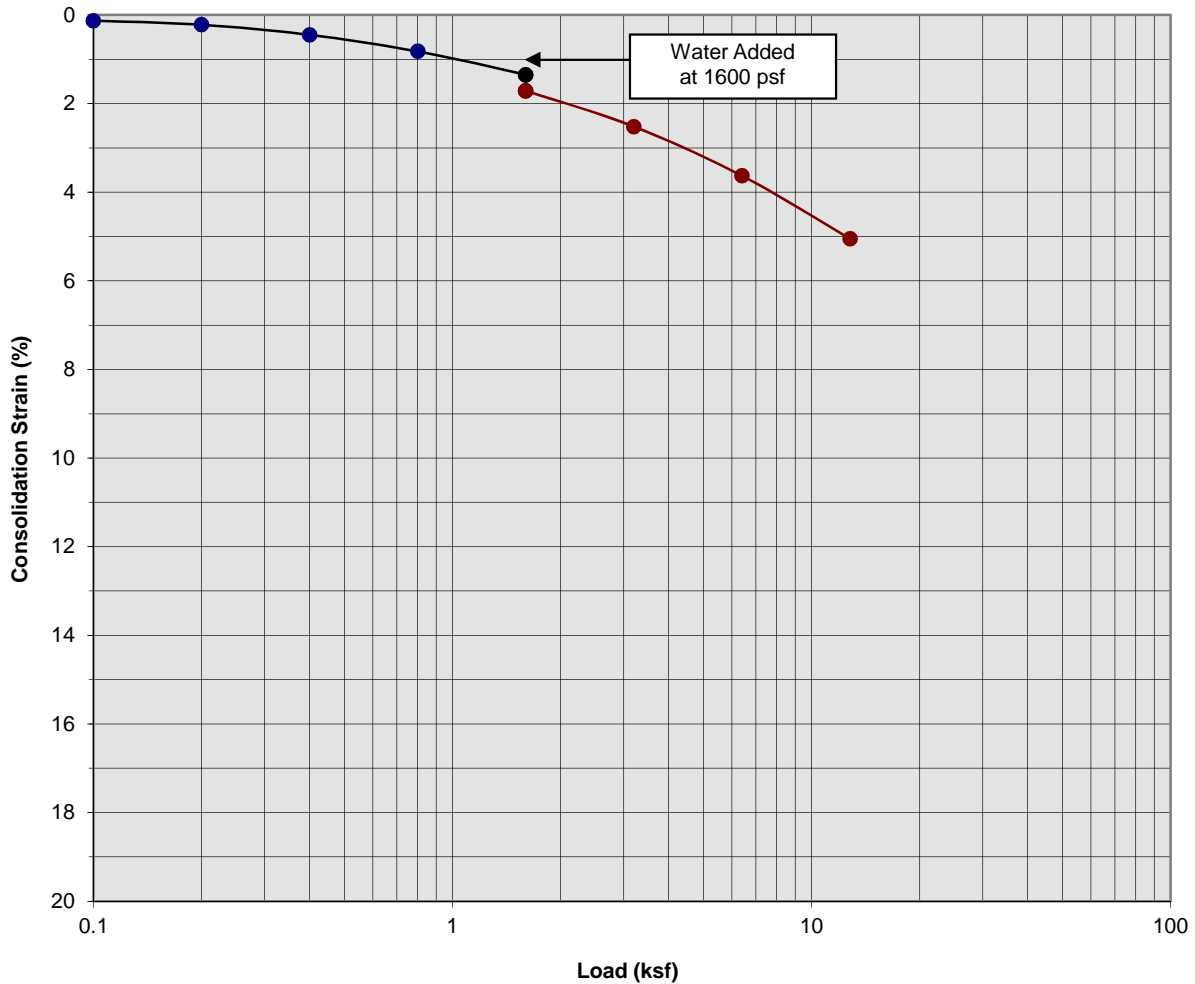
Boring Number:	B-3	Initial Moisture Content (%)	9
Sample Number:	---	Final Moisture Content (%)	25
Depth (ft)	7 to 8	Initial Dry Density (pcf)	105.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	106.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.07

Proposed C/I Bldg
Chino, California
Project No. 12G120
PLATE C- 6



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Consolidation/Collapse Test Results



Classification: Gray Green Clayey Silt

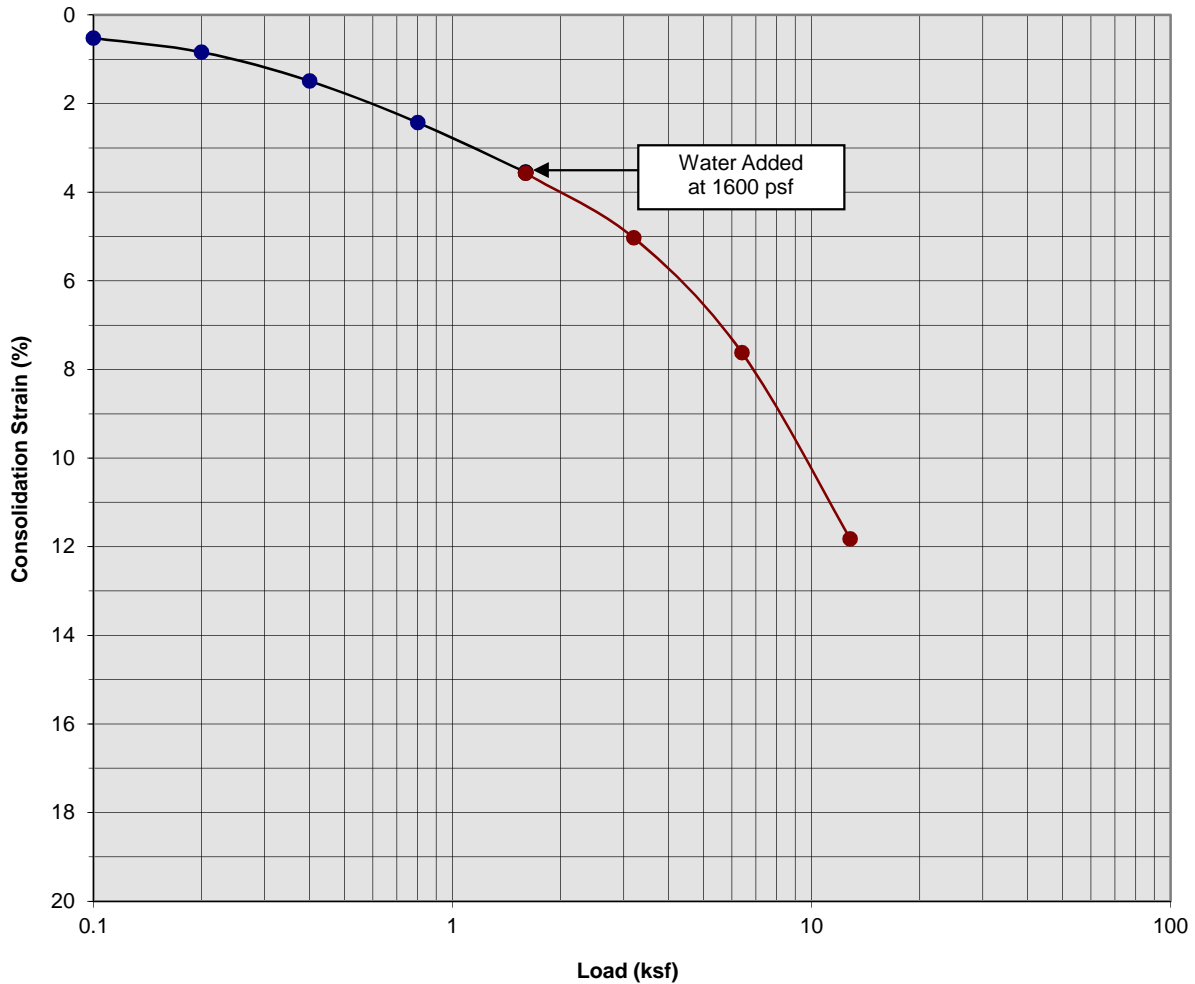
Boring Number:	B-3	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	22
Depth (ft)	9 to 10	Initial Dry Density (pcf)	92.9
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	105.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.36

Proposed C/I Bldg
 Chino, California
 Project No. 12G120
PLATE C- 7



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Consolidation/Collapse Test Results



Classification: Gray Brown Silty Clay

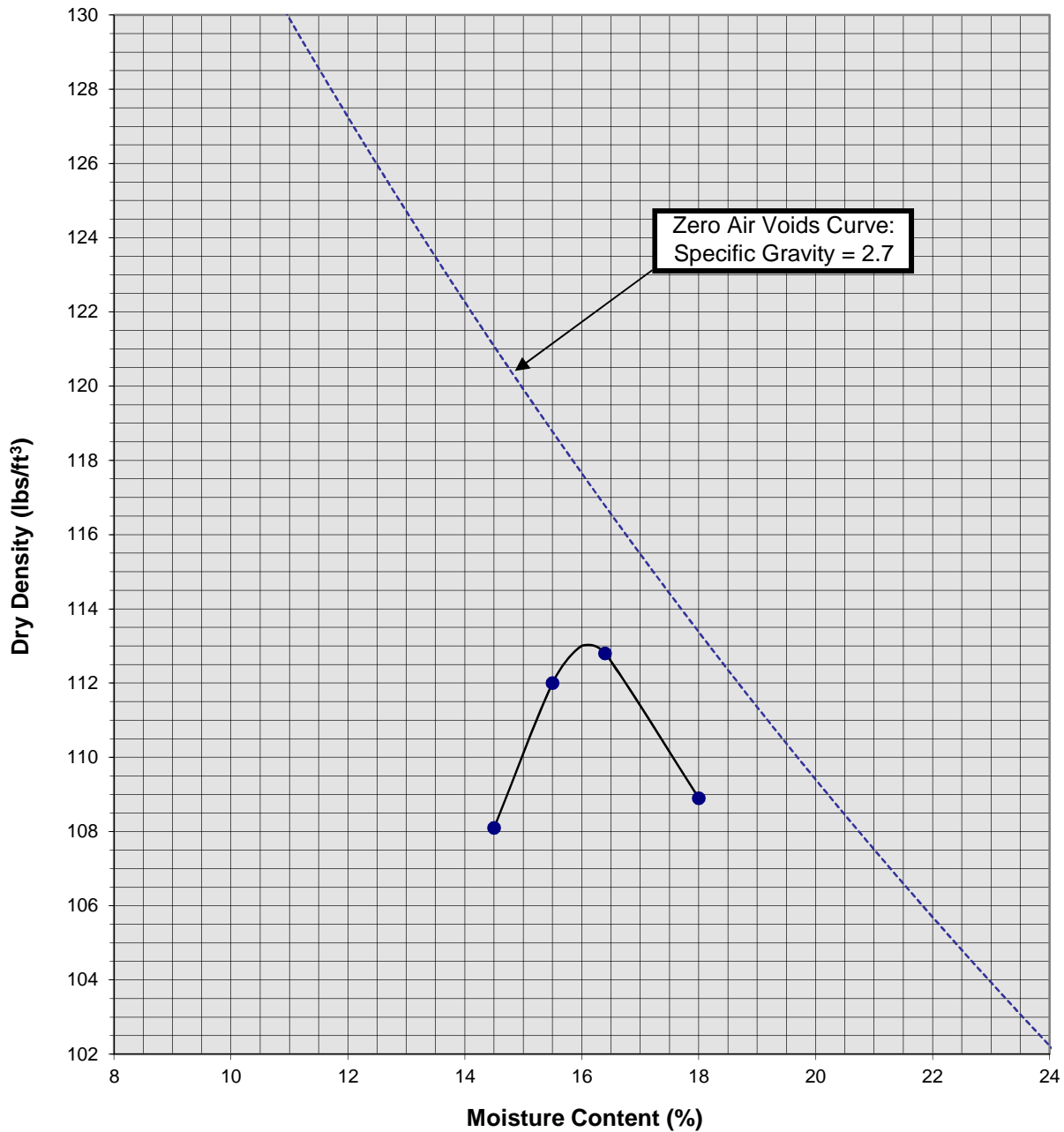
Boring Number:	B-3	Initial Moisture Content (%)	35
Sample Number:	---	Final Moisture Content (%)	34
Depth (ft)	14 to 15	Initial Dry Density (pcf)	85.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	96.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.02

Proposed C/I Bldg
 Chino, California
 Project No. 12G120
PLATE C- 8



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Moisture/Density Relationship ASTM D-1557



Soil ID Number		B-5 @ 0 to 5'
Optimum Moisture (%)		16
Maximum Dry Density (pcf)		113
Soil Classification	Dark Gray Brown Silty fine Sand to fine Sandy Silt	

Proposed C/I Bldg
Chino, California
Project No. 12G120
PLATE C-9



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