GEOTECHNICAL INVESTIGATION
VETERANS MEMORIAL PARK

Faraday Avenue at Whitman Way
Carlsbad, California
for
RJM Design Group

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

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A California Corporation

August 7, 2020

RJM Design Group
31591 Camino Capistrano
San Juan Capistrano, California 92675

Attention: Mr. Eric Chastain, LLA, LEED AP
Principal Landscape Architect

Proposal No.: 19G109-2

Subject: Geotechnical Investigation
Proposed Veterans Memorial Park
Faraday Avenue at Whitman Way
Carlsbad, California

Dear Mr. Chastain:

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

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

Respectfully Submitted,

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1.0 EXECUTIVE SUMMARY

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

Geotechnical Design Considerations

- The extreme northern portion of the site is underlain by relatively deep alluvial soils. The remainder of the site is underlain by a relatively shallow layer of colluvium, which is underlain by Santiago formation sandstone and siltstone.
- Mapping performed by the County of San Diego indicates that the northern portion of the subject site is located within a liquefaction hazard zone. Therefore, one boring was extended to a depth of 50± feet below the existing site grades.
- The liquefaction analysis indicates a total settlement of 3.3± inches at Boring No. B-1. The liquefaction-induced differential settlements are conservatively estimated to be in the range of 1.6 to 2.2± inches. Assuming that these settlements occur across a distance of 50± feet, an angular distortion of 0.004± inches per inch would result.
- The proposed building areas are generally underlaying by medium expansive alluvial and colluvial soils that possess low to moderate strengths and a potential for hydrocollapse.
- Where new buildings are supported on shallow foundations and slabs-on-grade, remedial grading will be necessary to remove the upper portion of the near-surface native colluvial soils and replace these materials as compacted structural fill.
- Shallow drilled piers may be used to support shade structures, canopies, etc., to minimize remedial grading.

Site Preparation Recommendations

- Remedial grading is recommended to be performed within the proposed building areas in order to remove the upper portion of the existing low strength alluvial and colluvial soils. The soils present within the proposed building areas should be overexcavated to a depth of 5 feet below existing grade and to a depth of at least 3 feet below proposed building pad subgrade elevation. The northern restroom building should be overexcavated to a depth of at least 5 feet below proposed pad grade. The proposed foundation influence zones should also be overexcavated to a depth of at least 3 feet below proposed foundation bearing grade.
- After the recommended overexcavation has been completed, the resulting subgrade soils should be evaluated by the geotechnical engineer to identify any additional soils that should be overexcavated. The resulting subgrade should then be scarified to a depth of 12 inches, thoroughly moisture conditioned to 2 to 4 percent above optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.
- The subgrade soils within new flatwork, parking areas, bocce ball courts, playgrounds, etc., are recommended to 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.
- Due to the presence of medium expansive soils, consideration should be given to placing a 12 to 24-inch thick layer of very low expansive soils below the new flatwork.

Building Foundations

- Conventional shallow foundations, supported in newly placed compacted fill.
- 2,000 lbs/ft^2 maximum allowable soil bearing pressure.
- Reinforcement consisting of at least six (6) No. 5 rebars (3 top and 3 bottom) in strip footings, due to the liquefaction potential and expansive potential of the on-site soils. Additional reinforcement may be necessary for structural considerations.
- Structurally connect any isolated footings in both perpendicular directions within structures underlain by potentially liquefiable alluvium.

Pole Foundations

- Structures incorporating isolated poles, such as trellises, shade structures and light poles may be supported on shallow drilled pier foundations, supported in newly placed compacted fill.
- 3,000 lbs/ft^2 maximum allowable end-bearing pressure.
- Minimum embedment: 5 feet below adjacent grade; 2 feet below grade for fencing.

Building Floor Slabs

- Conventional Slab-on-Grade, 4½ inches thick.
- Minimum slab thickness 6 inches for buildings underlain by potentially liquefiable alluvial soils, such as the northern restroom building.
- Reinforcement consisting of No. 4 bars at 16-inches on center in both directions due to the expansion potential of the on-site soils. The actual floor slab reinforcement should be determined by the structural engineer.

Pavements

ASPHALT PAVEMENTS (R = 15)			
Materials	Thickness (inches)		
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Light Truck Traffic (TI = 6.0)
Asphalt Concrete	3	3	3½
Aggregate Base	6	9	11
Compacted Subgrade	12	12	12

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 15)			
Materials	Thickness (inches)		
	Automobile Parking and Drive Areas (TI = 4.0 to 5.0)	Truck Traffic Areas (TI =6.0)	Fire Lane (TI = 6.5)
PCC	5	5½	6
Compacted Subgrade (95% minimum compaction)	12	12	12

2.0 SCOPE OF SERVICES

The scope of services performed for this project was in accordance with our Proposal No. 18P372R, dated October 30, 2018. The scope of services included a visual site reconnaissance, subsurface exploration, field and laboratory testing, and geotechnical engineering analysis to provide criteria for preparing the design of the structure foundations, structure floor slabs, and parking lot pavements along with site preparation recommendations and construction considerations for the proposed development. Based on the location of the subject site, this investigation also included a site-specific liquefaction evaluation. The evaluation of the environmental aspects of this site was beyond the scope of services for this geotechnical investigation.

3.0 SITE AND PROJECT DESCRIPTION

3.1 Site Conditions

The overall site is located at the southeast corner of Whitman Way and Faraday Avenue in Carlsbad, California. The overall site is bounded to the west and south by Faraday Avenue and to the north by Whitman Way, vacant land, and existing single-family residential tracts. The general location of the site is illustrated on the Site Location Map, enclosed as Plate 1 of this report.

The subject site consists of the westernmost 48± acres of Veteran's Memorial Park. Based on information from the client, the eastern portion of Veteran's Memorial Park is an existing preserve. The park is currently unimproved with dirt trails that are utilized for hiking and/or biking. This area of the park consists of gently sloping terrain with groundcover comprised of heavy native grass, weeds, shrubs with areas of dense large trees.

Topographic information was obtained from a plan provided by the client. Based on this plan, topography within the proposed development area consists of rolling hills. Site grades range from elevation 222± feet mean sea level (msl) in the east-central area to 44± feet msl in the northwestern area of the site.

3.2 Proposed Development

Based on the site plan provided to our office, the subject site will be developed with active and passive amenities, open space areas, public art, trails, utilities, parking, playgrounds, a bocce ball court, restrooms, and maintenance facilities. It is also expected that the park will include lighting, trellis shade structures and fencing. The primary structures will include a restroom and catering support structure located in the northwestern region of the site, a Veterans Memorial in the central region of the site, and a second restroom building located in the southern region of the site.

Detailed structural information was not available at the time of this report. It is assumed that any new buildings at the subject site will be single story structures of wood frame or masonry block construction. Based on the assumed construction, maximum column and wall loads are expected to be on the order of 30 kips and 1 to 3 kips per linear foot, respectively. It is assumed that the proposed structures will be supported on shallow foundations and concrete slab on grade floors.

Grading plans for the proposed development were not available at the time of this report. The proposed development is not expected to include any significant amounts of below-grade construction such as basements or crawl spaces. Based on the existing site topography and assuming a relatively balanced site, cuts and fills of up to 5± feet are expected to be required.

4.0 SUBSURFACE EXPLORATION

4.1 Scope of Exploration/Sampling Methods

The subsurface exploration conducted for this project consisted of five (5) borings advanced to depths of 15 to 50± feet below the existing site grades. The 50-foot deep boring was advanced at the site as part of the liquefaction evaluation. All of the borings were logged during drilling by a member of our staff.

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

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

4.2 Geotechnical Conditions

Alluvium

Native alluvium was encountered at the ground surface at Boring No. B-1. The alluvium consists of medium dense clayey fine sands and stiff to very stiff fine sandy clays extending to a depth of 32± feet. Boring No. B-1 encountered medium dense clayey fine sands and silty fine sands extending from a depth of 32± feet to at least the maximum depth explored of 50± feet.

Colluvium

Native colluvium was encountered at the ground surface at Boring Nos. B-3 through B-5. The colluvium consists of medium dense silty fine sands and stiff fine sandy clays extending to depths of 2½ to 3½± feet.

Bedrock

Bedrock of the Santiago Formation was encountered at the ground surface at Boring No. B-2 and beneath the colluvium at Borings Nos. B-3 through B-5, extending to a depth of at least 15± feet. The bedrock consists of interbedded medium dense to very dense silty fine-grained sandstone and fine-grained sandy siltstone with very stiff to hard clayey siltstone. The bedrock was weakly cemented and friable with iron oxide staining throughout.

Groundwater

Groundwater was encountered during drilling at Boring No. B-1 at a depth of 43± feet. A delayed water level reading was taken after 4 hours. However, the boring caved to a depth of 37½± feet. Based on the depth of the water encountered during drilling, the moisture contents of the recovered soil samples, and the caving conditions, the depth to the static groundwater table is considered to have existed at a depth of approximately 43± feet below existing site grades, at the time of the subsurface investigation.

SCG reviewed the water level data was obtained from the California Department of Water Resources website, http://www.water.ca.gov/waterdatalibrary/. However, the nearest monitoring well on record is located approximately 5.5 miles north of the site. Therefore, this groundwater data would not represent the groundwater at the subject site. SCG also reviewed groundwater information provided on California State Water Resources Control Board (SWRCB) Geotracker website https://geotracker.waterboards.ca.gov/. The nearest groundwater monitoring well is located 1.7± miles to the southeast of the subject site. The water level readings within this monitoring well indicate a groundwater level of 3.9± feet below the ground surface. This well is approximately at an elevation of 198± msl. Another groundwater monitoring well is located 1.8± miles southeast of the subject site. The water level readings in this monitoring well indicate a groundwater level of 22± feet below the ground surface. This well is at an elevation of approximately 300± feet msl.

4.3 Geologic Conditions

The subject site is located within the Peninsular Ranges province. The Peninsular Ranges province consists of several northwesterly-trending ranges in the southwestern California. The province is truncated to the north by the east-west trending Transverse Ranges. Prior to the mid-Mesozoic, the region was covered by seas and thick marine sedimentary and volcanic sequences were deposited. The bedrock geology that dominates the elevated areas of the Peninsular Ranges consists of high-grade metamorphic rocks intruded by Mesozoic plutons. During the Cretaceous, extensive mountain building occurred during the emplacement of the southern California batholith. The Peninsular Ranges have been significantly disrupted by Tertiary and Quaternary strike-slip faulting along the Elsinore and San Jacinto faults. This tectonic activity has resulted in the present terrain.

The primary available reference applicable to the subject site is the Geologic Map of the Oceanside, San Luis Rey, and San Marcos 7.5′ Quadrangles, San Diego County, California, by Siang S. Tan and Michael P. Kennedy, 1996. A portion of this map indicating the location of the subject site is included as Plate 3 in Appendix A of this report.

This map indicates that the site is underlain by two geologic units. The first geologic unit is the Holocene-age alluvium and colluvium deposits (Map Symbol Qal) located in a minor portion of the northwestern area of the site. This unit is described as unconsolidated silt, clay, sand and gravel. The second geologic unit is the Tertiary-age Santiago Formation (Map Symbol Tsa) underlying the majority of the subject site. The Santiago Formation consists of light-colored, poorly-bedded, poorly-indurated, fine- to medium-grained sandstone interbedded with siltstone and claystone with localized coarse-grained sandstone and conglomerate. Bedding attitudes on this map indicate that the beds strike generally northeast-southwest, dipping 12 to 15 degrees downward to the northwest. A minor fault (shear joint) plane is depicted on this map near the western boundary of the site. The minor fault plane generally strike northeast-southwest, dipping 80 degrees to the northwest. Three questionable landslides are also mapped 700 to 1,200± feet south-southeast of the subject site. These questionable landslides are located within the Santiago Formation.

Based on the conditions encountered during drilling, the subsurface conditions are similar to the mapped geologic conditions. Holocene-age alluvial soils consisting of clayey sands and sandy clays were encountered in the northwestern area of the site at Boring No. B-1. Santiago Formation bedrock consisting of silty fine-grained sandstone, fine-grained sandy siltstone, and clayey siltstone were encountered below the shallow colluvial soils in the remaining areas of the site.

SCG previously conducted superficial geologic mapping at the subject site. The results of this mapping were presented in our Report No. 19G109-1, dated July 15, 2019. Excerpts from this report are presented below:

<u>Geologic Mapping</u>

As part of this investigation, SCG performed surficial geologic mapping at the subject site. It should be noted that no subsurface investigation was performed as part of this investigation. The geologic mapping was limited to surficial exposure and expression of geologic units and features. Due to the recent rainfall, the site was covered with heavy native grass and weed growth which obscured the near-surface alluvial soils. Bedrock was only exposed in six (6) limited areas throughout the site.

Based on the bedrock exposed at the outcrops at the subject site, it is our opinion that the site is underlain by massive silty fine-grained sandstone with localized interbedded fine-grained sandstone and conglomerate of the Santiago Formation (Map Symbol Tsa). The geologic conditions at the site are generally consistent with the mapped geologic conditions. The bedding attitudes in the Santiago Formation in south-central area of the site generally strike northwest-southeast and dip 15 degrees to the northeast and the bedding attitudes in the Santiago Formation in the eastern area of the site generally strike southeast-southwest and dip 18 to 19 degrees to the northwest. As noted previously, a minor fault (shear joint) plane is mapped near the western boundary of the subject site. However, based on surface observations at the time of the geologic mapping, no evidence of surface expression of faults (i.e. fault scarps, fault line scarps, or displacement in the near surface soils) is present on the subject site. Three questionable landslides are also mapped south-southeast of the subject site. Based on the surface observations at the time of the geologic mapping, no evidence of surface expression of landslides (i.e. head scarps, minor scarps, crown cracks, radial cracks, etc.) is present on the subject site.

Fault Rupture Hazard

Currently, there is no published Alquist-Priolo Earthquake Fault Zone Map for the San Luis Rey Quadrangle. Therefore, the CGS has not mapped any active or potentially active faults with potential surface fault rupture in the San Luis Rey Quadrangle.

The nearest fault zone is the Rose Canyon Fault Zone (RCFZ) located 5.8± miles west of the subject site. The RCFZ is a right-lateral strike-slip fault. The RCFZ has a total length of 30 km with a slip rate ranging from 1.1 to 5 mm/yr. The interval between surface ruptures ranges between 1,200 and 3,000 years (scec.org).

An active fault is defined by the State of California under the Alquist-Priolo Act of 1972 as a fault which has displaced earth materials during the Holocene Epoch (11,000± years). The Santiago Formation bedrock that underlies the subject site is indicated to be Middle Eocene-Tertiary age (45± million years). Therefore, even though this bedrock unit may exhibit indicators of faulting near the subject site, the faulting may be ancient and may not be active.

Based on the age of the bedrock and the lack of surface expression at the subject site, the minor fault mapped outside the western boundary of the subject site is considered to be inactive.

Landslide Hazard

Based on the surficial geologic mapping conducted by SCG, there is no evidence of surface expressions for landslides at the subject site. In addition, there were no mapped landslides at the subject site and there were no indicators of landslides during the aerial photograph review.

Conclusions and Recommendations

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. A minor fault (shear joint) plane is mapped on the western boundary of the subject site. This minor fault (shear joint) is not associated with any known active fault. Based on our observations of the ground surface at the time of the surficial geologic mapping and the historic aerial photograph review, the minor fault (shear joint) does not show surface expression (i.e. fault scarps or fault line scarps). This fault is not identified on the Alquist-Priolo Earthquake Fault Zone map. Therefore, the possibility of significant fault rupture on the site is considered to be low.

There are no mapped landslides on the subject site, no evidence for landslides observed during the surficial geologic mapping, and no evidence for landslides during the historical aerial photograph review. Therefore, the possibility of an active landslide at the subject site is considered low.

5.0 LABORATORY TESTING

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

Classification

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

Density and Moisture Content

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

Consolidation

Selected soil samples have been tested to determine their consolidation potential, in accordance with ASTM D-2435. The testing apparatus is designed to accept either natural or remolded samples in a one-inch high ring, approximately 2.416 inches in diameter. Each sample is then loaded incrementally in a geometric progression and the resulting deflection is recorded at selected time intervals. Porous stones are in contact with the top and bottom of the sample to permit the addition or release of pore water. The samples are typically inundated with water at an intermediate load to determine their potential for collapse or heave. The results of the consolidation testing are plotted on Plates C-1 through C-4 in Appendix C of this report.

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-5 in Appendix C of this report. These tests are generally used to compare the in-situ densities of undisturbed field samples, and for later compaction testing. Additional testing of other soil types or soil mixes may be necessary at a later date.

Soluble Sulfates

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

Sample Identification	Soluble Sulfates (%)	Sulfate Classification
B-1 @ 0 to 5 feet	0.013	Not Applicable (S0)

Corrosivity Testing

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

Sample Identification	Saturated Resistivity (ohm-cm)	pH	Chlorides (mg/kg)	Nitrates (mg/kg)
B-1 @ 0 to 5 feet	1,080	7.5	47	49

Expansion Index

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

Sample Identification	Expansion Index	Expansive Potential
B-1 @ 0 to 5 feet	68	Medium

6.0 CONCLUSIONS AND RECOMMENDATIONS

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

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

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

6.1 Seismic Design Considerations

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

Faulting and Seismicity

Research of available maps indicates that the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Therefore, the possibility of significant fault rupture on the site is considered to be low.

Seismic Design Parameters

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

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

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

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter		Value
Mapped Spectral Acceleration at 0.2 sec Period	S_S	1.004
Mapped Spectral Acceleration at 1.0 sec Period	S_1	0.365
Site Class	---	D*
Site Modified Spectral Acceleration at 0.2 sec Period	S_{MS}	1.205
Site Modified Spectral Acceleration at 1.0 sec Period	S_{M1}	0.706
Design Spectral Acceleration at 0.2 sec Period	S_{DS}	0.803
Design Spectral Acceleration at 1.0 sec Period	S_{D1}	0.471

*The 2019 CBC requires that Site Class F be assigned to any profile containing soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils. For Site Class F, the site *coefficients* are to be determined in accordance with Section 11.4.7 of ASCE 7-16. However, Section 20.3.1 of ASCE 7-16 indicates that for sites with structures having a fundamental period of vibration equal to or less than 0.5 seconds, the site coefficient factors (F_a and F_v) may be determined using the standard procedures. The seismic design parameters tabulated above were calculated using the site coefficient factors for Site Class D, assuming that the fundamental period of the structures is less than 0.5 seconds. However, the results of the liquefaction evaluation indicate that the subject site is underlain by potentially liquefiable soils. Therefore, if the proposed structures have a fundamental period greater than 0.5 seconds, a site-specific seismic hazards analysis will be required and additional subsurface exploration will be necessary. Additional subsurface exploration and laboratory testing should be performed at the time of the design-level geotechnical investigation to confirm that this is a Site Class D site.

It should be noted that the site coefficient F_v and the parameters S_{M1} and S_{D1} were not included in the SEAOC/OSHPD Seismic Design Maps Tool output for the 2019 CBC. We calculated these parameters-based on Table 1613.2.3(2) in Section 16.4.4 of the 2019 CBC using the value of S_1 obtained from the Seismic Design Maps Tool, assuming that a site-specific ground motion hazards analysis is not required for the proposed buildings at this site.

Ground Motion Parameters

For the purposes of the liquefaction analysis performed for this study, we utilized a site acceleration consistent with maximum considered earthquake ground motions, as required by the 2019 CBC. The peak ground acceleration (PGA) was determined in accordance with Section 11.8.3 of ASCE 7-16. The parameter PGA_M is the maximum considered earthquake geometric mean (MCE_G) PGA, multiplied by the appropriate site coefficient from Table 11.8-1 of ASCE 7-16. The web-based software application SEAOC/OSHPD Seismic Design Maps Tool (described in the previous section) was used to determine PGA_M, which is 0.527g. A portion of the program output is included as Plate E-1 of this report. An associated earthquake magnitude was obtained from the USGS Unified Hazard Tool, Interactive Deaggregation application available on the USGS website. The deaggregated modal magnitude is 6.67, based on the peak ground acceleration and soil classification D.

Liquefaction

The Liquefaction Hazard Map, published by San Diego County, indicates that the extreme northern portion of the subject site is located within a liquefaction hazard zone. The majority of the site is not located within a liquefaction hazard zone. Based on this mapping, and the subsurface conditions encountered at the borings, the scope of this investigation included a detailed liquefaction evaluation in order to determine the site-specific liquefaction potential.

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

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

The liquefaction analysis procedure is tabulated on the spreadsheet forms included in Appendix F of this report. The liquefaction analysis was performed for Boring No. B-1, which was advanced to a depth of 50± feet. The liquefaction potential was analyzed at the boring location utilizing a PGA_M of 0.527g related to a 6.67 magnitude seismic event. The liquefaction evaluation was performed using a historic high groundwater depth of 37.5 feet.

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

Conclusions and Recommendations

The liquefaction analysis has identified potentially liquefiable soils at Boring No. B-1. The liquefiable materials are present in a several layers between depths of 37.5 and 50± feet. Soils which are located above the historic high groundwater table (37.5 feet), or possess factors of safety in excess of 1.3, are considered non-liquefiable. Settlement analyses were conducted for each of the potentially liquefiable strata.

Based on the settlement analysis (also tabulated on the spreadsheets in Appendix F) a total dynamic (liquefaction induced) settlement of 3.3± inches could be expected at Boring No. B-1. The associated differential settlement is estimated to be on the order of 1.6 to 2.2± inches. The estimated differential settlement could be assumed to occur across a distance of 50 feet, indicating a maximum angular distortion of less than 0.004± inches per inch.

It should be noted that the potentially liquefiable alluvial soils are only located in the north and northwestern regions of the site. As shown on Plate 3, the Geologic Map, the majority of the site is underlain by Santiago formation bedrock which is non-liquefiable. Special design considerations related to liquefaction-induced settlements will only be required for shallow foundations located within the area of the site that is underlain by the alluvial soils.

Based on our understanding of the proposed development, it is considered feasible to support the proposed structure on shallow foundations. Such a foundation system can be designed to resist the effects of the anticipated differential settlements, to the extent that the structure would not catastrophically fail. Designing the proposed structure to remain completely undamaged during a seismic event that could occur once every 2475 years (the code-specified return period used in the liquefaction analysis) is not considered to be economically feasible. Based on this understanding, the use of a shallow foundation system is considered to be the most economical means of supporting the proposed structures.

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

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

6.2 Geotechnical Design Considerations

General

Most of the site is underlain by a thin layer of colluvium, which is underlain at relatively shallow depth by Santiago formation bedrock. The notable exception to this is the northwestern corner of the site, in the vicinity of our Boring No. B-1. In this area, the site is underlain by native alluvial soils extending to a depth of more than 50± feet.

The near surface alluvium and colluvium possesses moderate strengths. The results of laboratory testing indicate that the upper 3 to 4± feet of these soils possess a potential for moderate consolidation and hydrocollapse. Based on these conditions, it is recommended that remedial grading be performed within the areas of the proposed structures. For structures supported on isolated pole foundations, such as canapés or playgrounds, the foundations may consist of shallow drilled piers, extended through the existing alluvium/colluvium into the native Santiago formation bedrock or medium dense alluvium below.

As discussed in a previous section of this report, potentially liquefiable soils were identified at Boring No. B-1, drilled in the northwestern region of the site. Therefore, special grading and foundation design recommendations will apply to the proposed pavilion/restroom/catering support building. The presence of the recommended layer of newly placed compacted structural fill above these liquefiable soils will help to reduce any surface manifestations that could occur as a result of liquefaction. The foundation and floor slab design recommendations presented in the subsequent sections of this report also contain recommendations to provide additional rigidity in order to reduce the potential effects of differential settlement that could occur as a result of liquefaction.

Settlement

The proposed remedial grading will remove the existing compressible alluvium/colluvium from within the proposed building areas. Alternatively, the foundations may be extended through the surface of colluvial soils into the Santiago formation bedrock below. The underlying native materials possess relatively high strengths and will not be excessively compressible under the new foundation loads. Therefore, following completion of the recommended remedial grading, post-construction static settlements are expected to be within tolerable limits.

Expansion

Laboratory testing performed on a representative sample of the near-surface soils indicates that these materials are medium expansive (EI = 68). Based on the presence of expansive soils at this site, special design and construction considerations are warranted. All building pad and flatwork subgrade soils should be properly moisture conditioned and maintained at adequate moisture content throughout the construction process. Further recommendations concerning the expansive soils are presented in subsequent sections of this report. It is recommended that additional expansion index testing be conducted at the completion of rough grading to verify the expansion potential of the as-graded structure pads.

Soluble Sulfates

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

Corrosion Potential

The results of laboratory testing indicate that a representative sample of the on-site soils possesses a saturated resistivity value of 1,080 ohm-cm, and a pH value of 7.5. This test result has 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. Sulfides, and redox potential are factors that are also used in the evaluation procedure. We have evaluated the corrosivity characteristics of the on-site soils using resistivity, pH, and moisture content. Based on these factors, and utilizing the DIPRA procedure, the on-site soils are considered to be highly corrosive to ductile iron pipe. Therefore, polyethylene encasement or some other appropriate method of protection will be required for iron pipes. Since SCG does not practice in the area of corrosion engineering, the client may also wish to contact a corrosion engineer to provide a more thorough evaluation.

Only a low level (47 mg/kg) of chlorides was detected in the sample submitted for corrosivity testing. In general, soils possessing chloride concentrations in excess of 350 to 500 parts per million (ppm) are considered to be corrosive with respect to steel reinforcement within reinforced concrete. Based on the lack of any significant chlorides in the tested sample, the site is considered to have a C1 chloride exposure in accordance with the American Concrete Institute (ACI) Publication 318 Building Code Requirements for Structural Concrete and Commentary. Therefore, a specialized concrete mix design for reinforced concrete for protection against chloride exposure is not considered warranted.

Shrinkage/Subsidence

Based on the results of the laboratory testing, removal and recompaction of the loose near-surface alluvial/colluvial soils, extending to depths of 5 to 6± feet, is estimated to result in an average shrinkage of 10 to 16 percent. Shrinkage or bulking of less than 5 percent is expected to occur where Santiago formation bedrock materials are excavated and replaced as compacted fill. It should be noted that these shrinkage estimates are based on the results of dry density testing performed on small-diameter samples of the existing soils taken at the boring locations. If a more accurate and precise shrinkage estimate is desired, SCG can perform a shrinkage study involving several excavated test-pits where in-place densities are determined using in-situ testing methods instead of laboratory density testing on small-diameter samples. Please contact SCG for details and a cost estimate regarding a shrinkage study, if desired.

Minor ground subsidence is expected to occur in the soils below the zone of removal, due to settlement and machinery working. The subsidence is estimated to be 0.1± feet. This estimate may be used for grading in areas that are underlain by native alluvial or colluvial soils.

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

Grading and Foundation Plan Review

No detailed grading or foundation plans were available at the time of this report. It is therefore recommended that we be provided with copies of the preliminary plans, when they become available, for review with regard to the conclusions, recommendations, and assumptions contained within this report.

6.3 Site Grading Recommendations

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

Site Stripping and Demolition

Initial site preparation should include stripping of any existing surficial vegetation and organic materials. Based on conditions observed at the time of the field exploration, stripping of only minor native grass and weed growth may be necessary within the western half of the subject site. Any organic material should be disposed of off-site. Removal of any trees should include all of the associated root masses. The actual extent of site stripping should be determined by the geotechnical engineer at the time of grading, based on the organic content and the stability of the encountered materials.

Treatment of Existing Soils: Building Pads

It is recommended that remedial grading be performed within the proposed building areas to remove the collapsible/compressible near-surface native alluvium and colluvium. The proposed building areas should be overexcavated to a depth of at least 5 feet below existing grade, and to a depth of at least 3 feet below proposed pad grade. Buildings that are underlain by potentially liquefiable alluvium, such as the northern restroom building, should be overexcavated to a depth of 5 feet below proposed pad grade. These recommendations apply to the building pads associated with the restrooms and catering support structures. Any similar structures (potentially including the proposed veterans memorial) to be supported on conventional shallow foundations and a concrete slab on grade floor should also be prepared in this manner.

Within the influence zones of the new foundations, the overexcavation should extend to a depth of at least 3 feet below proposed foundation bearing grade. The overexcavation should extend at least 5 feet beyond the building perimeter, and to an extent equal to the depth of new fill below the foundation bearing grade. If the proposed structures incorporate any exterior columns (such as for a building canopy or overhang) the overexcavation should also encompass these areas.

Following completion of the overexcavation, the subgrade soils within the new building 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 structures. 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 loose, porous, or low density native soils are encountered at the base of the overexcavation.

Based on conditions encountered at the exploratory boring locations, some zones of moist to very moist soils will be encountered at or near the base of the recommended overexcavation. Stabilization of the exposed overexcavation subgrade soils is expected to 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.

After a suitable overexcavation subgrade has been achieved, the exposed soils should be scarified to a depth of 12 inches, and moisture conditioned to at least 2 to 4 percent above optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. The previously excavated soils may then be replaced as compacted structural fill.

Treatment of Existing Soils: New Pole Foundation Areas

Some of the new improvements including fencing, lighting, shade structures and trellises will require new foundations. As discussed in a subsequent section of this report, it is recommended that these improvements be supported on drilled pier foundations, extending through the surficial fill soils. Therefore, no significant remedial grading is considered warranted within these new foundation areas.

Treatment of Existing Soils: Bocce Ball Courts

Remedial grading should be performed within the proposed bocce court areas in order to remove the surficial alluvium/colluvium, and to create a more uniform subgrade condition. It is recommended that the existing soils within the area of the proposed bocce ball courts be overexcavated to a depth of 1 foot below proposed subgrade elevation. Following completion of the overexcavation, the subgrade soils within the field areas should be evaluated by the geotechnical engineer to verify their suitability to serve as the structural fill subgrade. 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 loose, porous, or low density fill soils are encountered at the base of the overexcavation.

Based on 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.

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 to 2 to 4 percent above optimum moisture content, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. It is recommended that the bocce court areas be raised to grade with imported select structural fill, possessing an expansion index no greater than 20.

Any remedial grading and site preparation activities within the area of the proposed bocce ball courts should also be in accordance with any relevant specifications of the synthetic turf/bocce court surface manufacturer.

Treatment of Existing Soils: Parking and Drive Areas

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

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

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 compressible native soils that may be present in the parking and drive areas. As such, settlement and associated pavement distress could occur. Typically, repair of such distressed areas involves significantly lower costs than completely mitigating these soils at the time of construction. If the owner cannot tolerate the risk of such settlements, the parking and drive areas should overexcavated to provide a new layer of structural fill at least 2 feet in thickness.

Treatment of Existing Soils: Retaining Walls and Site Walls

The existing soils within the areas of any proposed retaining walls (less than 5 feet of retained height) should be overexcavated to a depth of 3 feet below foundation bearing grade and replaced as compacted structural fill, as discussed above for the proposed building pads. The foundation subgrade soils within the areas of any proposed non-retaining site walls should also be overexcavated to a depth of 3 feet below proposed foundation bearing grade. For both types of walls, the overexcavation subgrade soils should be evaluated by the geotechnical engineer prior to scarifying, moisture conditioning and recompacting the upper 12 inches of exposed subgrade soils. The previously excavated soils may then be replaced as compacted structural fill.

If any walls retaining more than 5 feet of soil are proposed for this site, the geotechnical engineer should be contacted to provide supplementary remedial grading recommendations.

Treatment of Existing Soils: Flatwork

The proposed development is expected to include some areas of new Portland cement concrete flatwork. Based on conditions encountered at the boring locations, it is expected that these areas of flatwork will be underlain by moist to very moist medium expansive soils. The presence of these soils possesses a minor risk of heave and damage to new flatwork, which will be relatively lightly loaded. Based on economic considerations, flatwork is typically constructed immediately over medium expansive soils. However, if the owner desires protection against heaving of flatwork, a layer of very low expansive select structural fill could be placed below the flatwork areas. Typically, this layer of select fill is 1 to 2 feet in thickness.

Subgrade preparation in the new flatwork areas should initially consist of removal of all soils disturbed during stripping and demolition operations. The geotechnical engineer should then evaluate the subgrade to identify any areas of additional unsuitable soils. The subgrade soils should then be scarified to a depth of 12± inches, moisture conditioned to 2 to 4 percent above optimum, and recompacted to at least 90 percent of the ASTM D-1557 maximum dry density. Based on the presence of variable strength fill soils throughout the site, it is expected that some isolated areas of additional overexcavation may be required to remove zones of lower strength, unsuitable soils.

These flatwork subgrade preparation recommendations may also be used for areas of new exterior rubberized play surfacing and areas of new flagstone, subject to any applicable manufacturer's recommendations.

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. Some drying of the on-site soils will likely be necessary in order to achieve a moisture content suitable for compaction as structural fill. All fill should conform with the recommendations presented in the Grading Guide Specifications, included as Appendix D.
- All grading and fill placement activities should be completed in accordance with the requirements of the 2019 CBC and the grading code of the city of Carlsbad.
- All fill soils should be compacted to at least 90 percent of the ASTM D-1557 maximum dry density. Fill soils should be well mixed.
- Compaction tests should be performed periodically by the geotechnical engineer as random verification of compaction and moisture content. These tests are intended to aid the contractor. Since the tests are taken at discrete locations and depths, they may not be indicative of the entire fill and therefore should not relieve the contractor of his responsibility to meet the job specifications.

Imported Structural Fill

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

Utility Trench Backfill

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

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

6.4 Construction Considerations

Excavation Considerations

The near-surface soils generally consist of silty clays and sandy clays. These materials are not expected to be subject to significant caving within shallow excavations. If caving does occur within shallow excavations, flattened excavation slopes are expected to be sufficient to provide excavation stability. Deeper excavations will require some form of external stabilization such as shoring or bracing. Maintaining adequate moisture content within the near-surface soils will improve excavation stability. Temporary excavation slopes should be no steeper than 1.5h:1v. All excavation activities on this site should be conducted in accordance with Cal-OSHA regulations. Excavations into the Santiago formation bedrock can be sloped at 1h:1v.

We were able to extend borings 15 feet into the Santiago formation bedrock using a conventional hollow-stem auger drill rig. Therefore it is expected that conventional grading equipment will be adequate to excavate the bedrock materials at depths of up to 15± feet. However, due to the density of the bedrock, slower production rates should be expected. If excavations more than 15 feet into bedrock are required, additional studies may be necessary to evaluate the excavation characteristics of the deeper bedrock materials.

Expansive Soils

The near-surface soils at this site are potentially expansive. Therefore, care should be given to proper moisture conditioning of all on-site 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 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 structures. These provisions should include directing surface runoff into rain gutters and area drains, reducing the extent of landscaped areas around the structures, and sloping the ground surface away from the buildings. Where possible, it is recommended that landscaped planters not be located immediately adjacent to the buildings. If landscaped planters around the buildings are necessary, it is recommended that drought tolerant plants or a drip irrigation system be utilized, to minimize the potential for deep moisture penetration around the structures. Presented below is a list of additional soil moisture control recommendations that should be considered by the owner, developer, and civil engineer:

- Ponding and areas of low flow gradients in unpaved walkways, grass and planter areas should be avoided. In general, minimum drainage gradients of 2 percent should be maintained in unpaved areas.

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

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

Moisture Sensitive Subgrade Soils

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

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

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 less moisture sensitive fill material. Grading during wet or cool weather may also increase the depth of overexcavation in the pad areas as well as the need for and or the thickness of the crushed stone stabilization layer, discussed in Section 6.3 of this report.

Groundwater

The static groundwater table at this site is considered to exist at a depth in excess of 30± feet. Therefore, groundwater is not expected to impact grading or foundation construction activities. Perched water may be encountered within sandy seams in the bedrock or alluvium. It is expected that minor perched water can be removed with sump pumps.

6.5 Shallow Foundation Design and Construction – New Buildings

Based on the preceding grading recommendations, it is assumed that the new building pads will be underlain by new structural fill soils used to replace the existing compressible/collapsible native colluvial soil. These structural fill soils are expected to extend to a depth of at least 3 feet below proposed foundation bearing grade. Based on this subsurface profile, and the design considerations presented in Section 6.1 of this report, the proposed buildings may be supported on conventional shallow foundations.

Building Foundation Design Parameters

New square and rectangular footings may be designed as follows:

- Maximum, net allowable soil bearing pressure: 2,000 lbs/ft^2.
- Minimum wall/column footing width: 14 inches/24 inches.
- Minimum longitudinal steel reinforcement within strip footings: Six (6) No. 5 rebars (3 top and 3 bottom), due to the expansive potential of the encountered soils, as well as the potential for liquefaction-induced settlements.
- Within the northern restroom area, where potentially liquefiable soils exist, it is recommended that all isolated foundations be connected in both perpendicular directions to adjacent foundations. This is typically accomplished using grade beams, designed by the structural engineer.
- Minimum foundation embedment: 12 inches into suitable structural fill soils, and at least 24 inches below adjacent exterior grade. Interior column footings may be placed immediately beneath the floor slab.

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

The allowable bearing pressure presented above may be increased by 1/3 when considering short duration wind or seismic loads. The minimum steel reinforcement recommended above is based on standard geotechnical practice. Additional rigidity may be necessary for structural considerations, or to resist the effects of the liquefaction-induced differential settlements, as discussed in Section 6.1. The actual design of the foundations should be determined by the structural engineer.

Foundation Construction

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

The foundation subgrade soils should also be properly moisture conditioned to 2 to 4 percent above the Modified Proctor optimum, to a depth of at least 12 inches below bearing grade. Since it is typically not feasible to increase the moisture content of the floor slab and foundation subgrade soils once rough grading has been completed, care should be taken to maintain the moisture content of the building pad subgrade soils throughout the construction process.

Estimated Foundation Settlements

Post-construction total and differential static settlements of shallow foundations designed and constructed in accordance with the previously presented recommendations are estimated to be less than 1.0 and 0.5 inches, respectively. Differential movements are expected to occur over a 30-foot span, thereby resulting in an angular distortion of less than 0.002 inches per inch. These settlements do not include the liquefaction-induced settlements discussed in Section 6.1 of this report.

Lateral Load Resistance

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

- Passive Earth Pressure: 250 lbs/ft^3
- Friction Coefficient: 0.25

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

6.6 Drilled Pier Foundation Design and Construction

As discussed previously, the proposed improvements may include shade structures, permanent fencing, lighting and trellises. Such structures are typically supported on isolated pole foundations. It is recommended that these improvements be supported on drilled pier foundations to reduce the need for any remedial grading and to limit the effect of the medium expansive soils that exist at this site. These foundations may be designed as follows:

- Maximum, net allowable soil bearing pressure: 3,000 lbs/ft^2.
- Minimum pier diameter: 18 inches, 12 inches for fencing.
- Minimum pier embedment: 5 feet below adjacent exterior grade; 2 feet below adjacent exterior grade for fencing.

Non-structural fencing, such as chain link fence, is not required to be supported on drilled piers. The fence post foundations should extend to a depth of at least 18 inches.

The allowable bearing pressure presented above may be increased by 1/3 when considering short duration wind or seismic loads. The actual design of the foundations should be determined by the structural engineer.

Drilled Pier Construction

Minimum pier shaft diameters should be 18 inches (12 inches for fencing) to help eliminate arching of concrete and possible void formation within the piers. On-center pier spacing should be at least four (4) times the pier diameter at the bearing surface to eliminate an overlapping stress influence. At a minimum, a pier spacing equivalent to three (3) times the pier diameter could be utilized, with an associated 20 percent reduction in allowable capacities. Based on the conditions encountered at the boring locations, minor caving of the drilled pier excavations may occur. If caving or groundwater intrusion does occur during drilling, casing or liners will be required.

Prior to the placement of concrete, a clean-out bucket should be used to ensure that excess materials in the bottom of the pier have been sufficiently removed and that the dimensions of the pier are correct. Concrete should be place using a tremie pipe whenever the distance of fall is greater than five (5) feet. Concrete should be placed at about a 6-inch slump when long reinforcing steel is used. It is recommended that the pier construction be performed in accordance with American Concrete Institute documents (ACI 336, I-79 and ACI 336-3R-72, revised 1985). In the event that casing is required, a sufficient head of concrete (minimum of 5 feet) should be maintained in the casing as the casing is being removed to prevent the intrusion of caving soils in the pier.

It is recommended that the bearing materials at each drilled pier location be evaluated by the geotechnical engineer prior to placing steel or concrete.

Lateral Load Resistance

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

- Passive Earth Pressure: 250 lbs/ft^3
- Friction Coefficient: 0.25

These are allowable values, and include a factor of safety. The passive pressure may be increased by one-third for transient loads, but should not be doubled. Due to the presence of existing undocumented fill soils, the upper 12 inches of the soil subgrade should be neglected when determining the passive resistance. 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, low expansive, structural fill. The maximum allowable passive pressure is 2500 lbs/ft^2.

6.7 Floor Slab Design and Construction

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

- Minimum slab thickness: 4½ inches. Building floor slabs underlain by potentially liquefiable native alluvium (such as the restroom in the northwestern region of the site) should be at least 6 inches in thickness.

- Minimum slab reinforcement: No. 4 bars at 16-inches on-center, in both directions, due to presence of potentially liquefiable soils and expansive soils, at this site. The actual floor slab reinforcement should be determined by the structural engineer, based upon the imposed loading, and the potential liquefaction-induced settlements.

- The foundation/slab system may be designed using an effective plasticity index of 20.

- Slab underlayment: If moisture sensitive floor coverings will be used or if vapor transmission into the area above the building slab is problematic, then minimum slab underlayment should consist of a moisture vapor barrier constructed below the entire area of the proposed slab. The moisture vapor barrier should meet or exceed the Class A rating as defined by ASTM E 1745-97 and have a permeance rating less than 0.01 perms as described in ASTM E 96-95 and ASTM E 154-88. A polyolefin material such as 15-mil Stego

Wrap Vapor barrier or equivalent will meet these specifications. The moisture vapor barrier should be properly constructed in accordance with all applicable manufacturer specifications. Given that a rock free subgrade is anticipated and that a capillary break is not required, sand below the barrier is not required. The need for sand and/or the amount of sand above the moisture vapor barrier should be specified by the structural engineer or concrete contractor. The selection of sand above the barrier is not a geotechnical engineering issue and hence outside our purview. Where moisture sensitive floor coverings are not anticipated, the vapor barrier may be eliminated.

- Moisture condition the floor slab subgrade soils to 2 to 4 percent above the Modified Proctor optimum moisture content, to a depth of 12 inches. The moisture content of the floor slab subgrade soils should be verified by the geotechnical engineer within 24 hours prior to concrete placement.

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

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

6.8 Exterior Flatwork Design and Construction

Subgrades which will support new exterior slabs-on-grade for patios and sidewalks should be prepared in accordance with the recommendations contained in Section 6.3 of this report. Based on these recommendations, the exterior flatwork will be supported on existing native soils that have been scarified and moisture conditioned to a depth of 12 inches and recompacted to 90 percent of the ASTM D-1557 maximum dry density, or newly placed structural fill. The owner and/or developer should be aware that flatwork constructed over medium expansive soils may be subject to movements and minor distress due to heaving of the underlying expansive soils. If such movements are not acceptable, consideration should be given to the use of a low expansive layer of structural fill beneath the flatwork, as discussed in Section 6.3 of this report. Based on geotechnical considerations, exterior slabs on grade which are not subjected to any vehicular traffic may be designed as follows:

- Minimum slab thickness: 4½ inches

- Minimum slab reinforcement: No. 4 bars at 18 inches on center, in both directions.

- Moisture condition the flatwork subgrade soils to 2 to 4 percent above the optimum moisture content, to a depth of at least 12 inches.

- 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.

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

- Where the flatwork is adjacent to a landscape planter or another area with exposed soil, it should incorporate a turned-down edge. This turned-down edge should be at least 12 inches in depth and 6 inches in width. The turned-down edge should incorporate longitudinal steel reinforcement consisting of at least one No. 4 bar.

- Flatwork which is constructed immediately adjacent to the new structures should be dowelled into the perimeter foundations in a manner determined by the structural engineer.

- Some cracking of exterior flatwork at this site should be expected, due to the presence of expansive soils.

These flatwork design and construction recommendations may also be used for concrete slabs to be placed in areas of new exterior rubberized play surfacing and areas of new flagstone, subject to any applicable manufacturer's recommendations.

6.9 Retaining Wall Design and Construction

Although not indicated on the site plan, some small (less than 5 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. The on-site soils generally consist of potentially expansive alluvium or colluvium. These materials are not considered suitable for use as retaining wall backfill. Therefore, it is recommended that imported very low to non-expansive soils be used for retaining wall backfill. Typically, silty sands are used for this purpose. Such materials are expected to possess an internal angle of friction of at least 30 degrees when compacted to at least 90 percent of the ASTM D-1557 maximum dry density.

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

RETAINING WALL DESIGN PARAMETERS

Design Parameter		Soil Type: Imported Silty Sand
Internal Friction Angle (ϕ)		30°
Unit Weight		125 lbs/ft^3
Equivalent Fluid Pressure:	Active Condition (level backfill)	42 lbs/ft^3
	Active Condition (2h:1v backfill)	67 lbs/ft^3
	At-Rest Condition (level backfill)	63 lbs/ft^3

The walls should be designed using a soil-footing coefficient of friction of 0.25 and an equivalent passive pressure of 250 lbs/ft^3. The structural engineer should incorporate appropriate factors of safety in the design of the retaining walls.

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

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

Seismic Lateral Earth Pressures

In addition to the lateral earth pressures presented in the previous section, retaining walls which are more than 6 feet in height should be designed for a seismic lateral earth pressure, in accordance with the 2019 CBC. Based on the current site plan, it is not expected that any walls in excess of 6 feet in height will be required for this project. If any such walls are proposed, our office should be contacted for supplementary design recommendations.

Retaining Wall Foundation Design

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

Backfill Material

The on-site soils are not considered suitable for use as retaining wall backfill. All retaining wall backfill soils should consist of imported low expansive sands or silty sands. All backfill material placed within 3 feet of the back wall face should have a particle size no greater than 3 inches. The retaining wall backfill materials should be well graded.

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

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

Subsurface Drainage

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

- A weep hole drainage system typically consisting of a series of 4-inch diameter holes in the wall situated slightly above the ground surface elevation on the exposed side of the wall and at an approximate 8-foot on-center spacing. 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.

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 supported on the existing soils that have been scarified, moisture conditioned, and recompacted. These materials generally consist of silty clays and sandy clays or newly placed engineered fill soils of similar composition. These materials are expected to exhibit poor pavement support characteristics, with estimated R-values of 15 to 25. Since R-value testing was not included in the scope of services for the current project, the subsequent pavement designs are based upon a conservatively assumed R-value of 15. Any fill material imported to the site should have support characteristics equal to or greater than that of the on-site soils and be placed and compacted under engineering controlled conditions.

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

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 = 15)			
Materials	Thickness (inches)		
	Auto Parking (TI = 4.0)	Auto Drive Lanes (TI = 5.0)	Light Truck Traffic (TI = 6.0)
Asphalt Concrete	3	3	3½
Aggregate Base	6	9	11
Compacted Subgrade	12	12	12

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

Portland Cement Concrete

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

PORTLAND CEMENT CONCRETE PAVEMENTS (R = 15)			
Materials	Thickness (inches)		
	Automobile Parking and Drive Areas (TI = 4.0 to 5.0)	Truck Traffic Areas (TI =6.0)	Fire Lane (TI = 6.5)
PCC	5	5½	6
Compacted Subgrade (95% minimum compaction)	12	12	12

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

7.0 GENERAL COMMENTS

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

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

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

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

8.0 REFERENCES

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APPENDIX A

SOURCE: USGS TOPOGRAPHIC MAP OF THE SAN LUIS REY QUADRANGLE, SAN DIEGO COUNTY, CALIFORNIA, 2018

SITE LOCATION MAP

VETERANS MEMORIAL PARK

CARLSBAD, CALIFORNIA

SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: RB CHKD: RGT	
SCG PROJECT 19G109-2	
PLATE 1	

GEOTECHNICAL LEGEND

APPROXIMATE BORING LOCATON

SANTIAGO FORMATION (Tsa) OUTCROP

15 STRIKE AND DIP

85 JOINTING

NOTE: SURFICIAL GEOLOGIC UNITS OBSCURED BY VEGETATION EXCEPT IN EXPOSED OUTCROP AREAS.

NOTE: BASE MAP PREPARED BY CLIENT

BORING LOCATION PLAN

VETERANS MEMORIAL PARK

CARLSBAD, CALIFORNIA

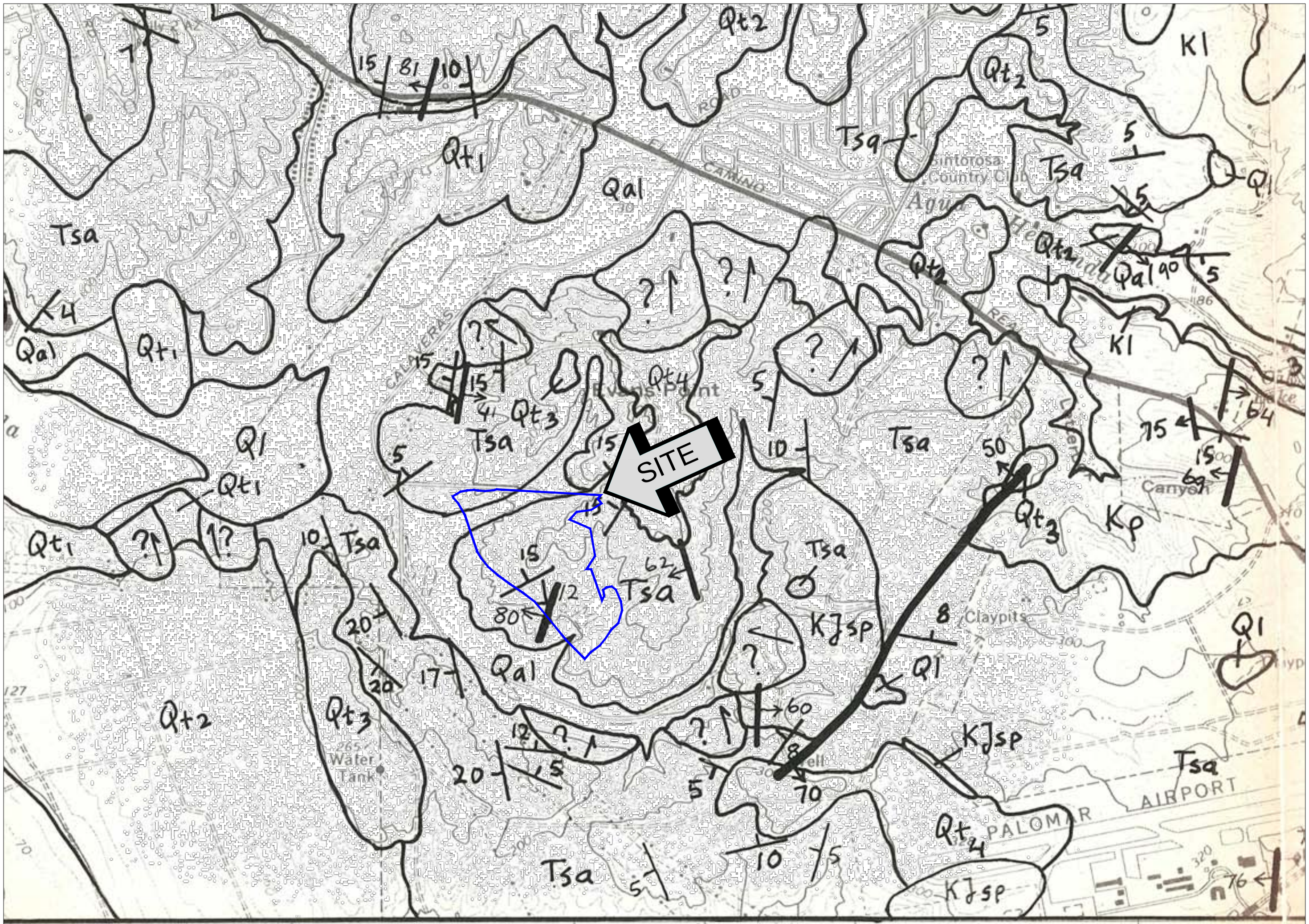
SCALE: 1" = 200'

DRAWN: DRK
CHKD: GKM

SCG PROJECT
19G109-2

PLATE 2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



MAP UNITS

Qal	Alluvium and colluvium Unconsolidated silt, clay, sand and gravel.
Qb	Beach deposits: unconsolidated sand.
Ql	Lake, reservoir and pond deposits; partly submerged, unconsolidated clay, silt, sand and gravel.
Ql/?	Landslide deposits (includes headscarp area). See further California Division of Mines and Geology Open-File Report 95-04.
Qt1-4	Terrace deposits; reddish brown, poorly bedded, poorly- to moderately-indurated sandstone, siltstone and conglomerate. Subscripts indicate relative level with 1 the lowest elevation (youngest age). The three lower levels have been correlated with the Bay Point Formation and the highest level with the Linda Vista Formation; see Kennedy (1975), Weber (1982), and Wilson (1972).
Tst	Stadium Conglomerate (Poway Group); poorly-bedded, poorly-to moderately-indurated, cobble conglomerate with coarse-grained sandstone matrix.
Tsa / Tt	Santiago Formation; light-colored, poorly-bedded, poorly-indurated, fine- to medium-grained sandstone interbedded with landslide-prone siltstone and claystone. Local coarse-grained sandstone and conglomerate. Renamed from Scripps Formation in the Encinitas (Tan, 1986) and Rancho Santa Fe (Tan, 1987) quadrangles. It interfingers with Torrey Sandstone. Torrey Sandstone (La Jolla Group); light-colored, massive and thick-bedded, well-indurated, medium-to coarse-grained arkosic sandstone. Resistant to landsliding. It interfingers with Santiago Formation.

MAP SYMBOLS

Contact (boundary) between map units. Most boundaries are not exposed and are inferred.

5 Strike and dip of inclined beds. Most bedding attitudes are estimated.

Horizontal beds.

82 U D Fault; dotted where concealed. Arrow and number indicate direction and amount of dip of exposed fault plane. U indicates upthrown side; D indicates downthrown side.

70 Strike, direction, and amount of dip of minor fault (shear joint) plane. Most fault displacements are less than 5 feet.

Landslide; arrows indicate general direction of movement. Both the headscarp area and debris deposit are included within the map symbol. Landslides are depicted prior to urban development. Only landslides larger than 300 feet across are shown on the map. For further information see California Division of Mines and Geology Open-File Report 95-04.

? Questionable landslide.

Site boundary

SOURCE: "GEOLOGIC MAP OF THE OCEANSIDE, SAN LUIS REY, AND SAN MARCOS 7.5' QUADRANGLES, SAN DIEGO COUNTIY, CALIFORNIA" TAN AND KENNEDY, 1996

GEOLOGIC MAP

VETERANS MEMORIAL PARK

CARLSBAD, CALIFORNIA

SCALE: 1" = 2000'	SOUTHERN CALIFORNIA GEOTECHNICAL
DRAWN: DRK CHKD: GKM	
SCG PROJECT 19G109-2	
PLATE 3	

APPENDIX B

BORING LOG LEGEND

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

COLUMN DESCRIPTIONS

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

SOIL CLASSIFICATION CHART

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 19G109-2
PROJECT: Veterans Park
LOCATION: Carlsbad, California

DRILLING DATE: 7/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 43 feet
CAVE DEPTH: 37½ feet
READING TAKEN: 4 Hours After Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					ALLUVIUM Gray Brown Clayey fine Sand to fine Sandy Clay, medium dense to stiff-moist							
	X	12					11					EI = 68 @ 0 to 5 feet
	X	14					12					
5												
					Gray Brown to Dark Gray Brown Silty fine Sand thinly interbedded with Clayey fine Sand, medium dense-moist to very moist							
	X	10					12					
					Gray Brown Clayey fine Sand, loose-very moist							
	X	8					14					
10												
					Gray Brown Clayey fine Sand to fine Sandy Clay, medium dense to very stiff-very moist							
	X	16					16	34	11	48		
15												
	X	16					20	44	18	53		
20												
					Light Gray Clayey fine Sand, medium dense-moist to very moist							
	X	10					14			36		
25												
	X	10					13			37		
30												
					Light Brown Silty fine Sand, trace Clay, occasional 1-inch fine Sandy Clay lenses, medium dense-moist to very moist							
	X	11					12			31		

TBL 19G109-2.GPJ SOCALGEO.GDT 8/7/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-1

JOB NO.: 19G109-2
PROJECT: Veterans Park
LOCATION: Carlsbad, California

DRILLING DATE: 7/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: 43 feet
CAVE DEPTH: 37½ feet
READING TAKEN: 4 Hours After Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION (Continued)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					Light Brown Silty fine Sand, trace Clay, occasional 1-inch fine Sandy Clay lenses, medium dense-moist to very moist							
					Gray Brown Clayey fine Sand, medium dense-moist to very moist							
	X	11					14			38		
40												
					Gray Brown Silty fine Sand, little Silt, medium dense-very moist to wet							
	X	11					16			27		Groundwater @ 43'
45												
	X	18					22			24		
50												
					Boring Terminated at 50'							

TBL 19G109-2.GPJ SOCALGEO.GDT 8/7/20

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-2

JOB NO.: 19G109-2
PROJECT: Veterans Park
LOCATION: Carlsbad, California

DRILLING DATE: 7/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
		62			SANTIAGO FORMATION (Tsa): Light Gray Silty fine to coarse-grained Sandstone, friable, weakly-cemented, trace Iron oxide staining, dense-moist	108	10					
		93			Light Gray to Light Brown thinly interbedded friable Silty fine-grained Sandstone and Clayey Siltstone, weakly-cemented, Iron oxide staining, very dense-moist to very moist	117	14					
5		50/4"				115	13					
		50/2"				107	12					
		50/2"			Light Gray fine-grained Sandstone, friable, weakly-cemented, dense-dry to damp	82	3					
10												
		50/5"			Light Gray Silty fine-grained Sandstone, friable, weakly-cemented, very dense-damp		8					
15					Boring Terminated at 15'							

TBL 19G109-2.GPJ SOCALGEO.GDT 8/7/20

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-3

JOB NO.: 19G109-2
PROJECT: Veterans Park
LOCATION: Carlsbad, California

DRILLING DATE: 7/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 13 feet
READING TAKEN: At Completion

FIELD RESULTS					DESCRIPTION	LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					COLLUVIUM (Qc): Dark Gray Brown Silty fine Sand, little Clay, medium dense-moist to very moist							
	X	26					12					
					SANTIAGO FORMATION (Tsa): Light Gray Silty fine-grained Sandstone, weakly-cemented, friable, medium dense-moist to very moist		12					
	X	50/4"			Light Gray Silty fine-grained Sandstone with thinly interbedded gray brown Clayey Siltstone, Iron oxide staining, friable, weakly-cemented, very dense-very moist		13					
5												
	X	50/5"			Light Gray Silty fine-grained Sandstone, Iron oxide staining, friable, weakly-cemented, very dense-damp to very moist		14					
	X	50/5"					12					
10												
	X	50/5"					7					
15												
					Boring Terminated at 15'							

TBL 19G109-2.GPJ SOCALGEO.GDT 8/7/20

BORING NO. B-4

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

JOB NO.: 19G109-2	DRILLING DATE: 7/16/20	WATER DEPTH: Dry
PROJECT: Veterans Park	DRILLING METHOD: Hollow Stem Auger	CAVE DEPTH: 11 feet
LOCATION: Carlsbad, California	LOGGED BY: Daryl Kas	READING TAKEN: At Completion

FIELD RESULTS						LABORATORY RESULTS						
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION SURFACE ELEVATION: MSL	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					COLLUVIUM (Qc): Light Brown Silty fine Sand, medium dense-very moist							
		18				94	18					
					Dark Gray Brown fine Sandy Clay, some Silt, mottled, stiff-moist							
		30				93	14					
					SANTIAGO FORMATION (Tsa): Gray Silty fine-grained Sandstone, friable, weakly-cemented, trace Iron oxide staining, medium dense-very moist							
5		40				109	15					
					Light Gray to Gray fine Sandy Siltstone interbedded with Clayey Siltstone, Iron oxide staining, weakly-cemented, friable, stiff-very moist to wet							
		31				107	16					
		37				110	22					
10												
					Light Gray to Gray Silty fine-grained Sandstone interbedded with fine Sandy Siltstone, weakly-cemented, friable, Iron oxide staining, medium dense-wet							
		27					20					
15					Boring Terminated at 15'							

TBL 19G109-2.GPJ SOCALGEO.GDT 8/7/20

TEST BORING LOG

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

BORING NO. B-5

JOB NO.: 19G109-2
PROJECT: Veterans Park
LOCATION: Carlsbad, California

DRILLING DATE: 7/16/20
DRILLING METHOD: Hollow Stem Auger
LOGGED BY: Daryl Kas

WATER DEPTH: Dry
CAVE DEPTH: 12 feet
READING TAKEN: At Completion

SURFACE ELEVATION: MSL

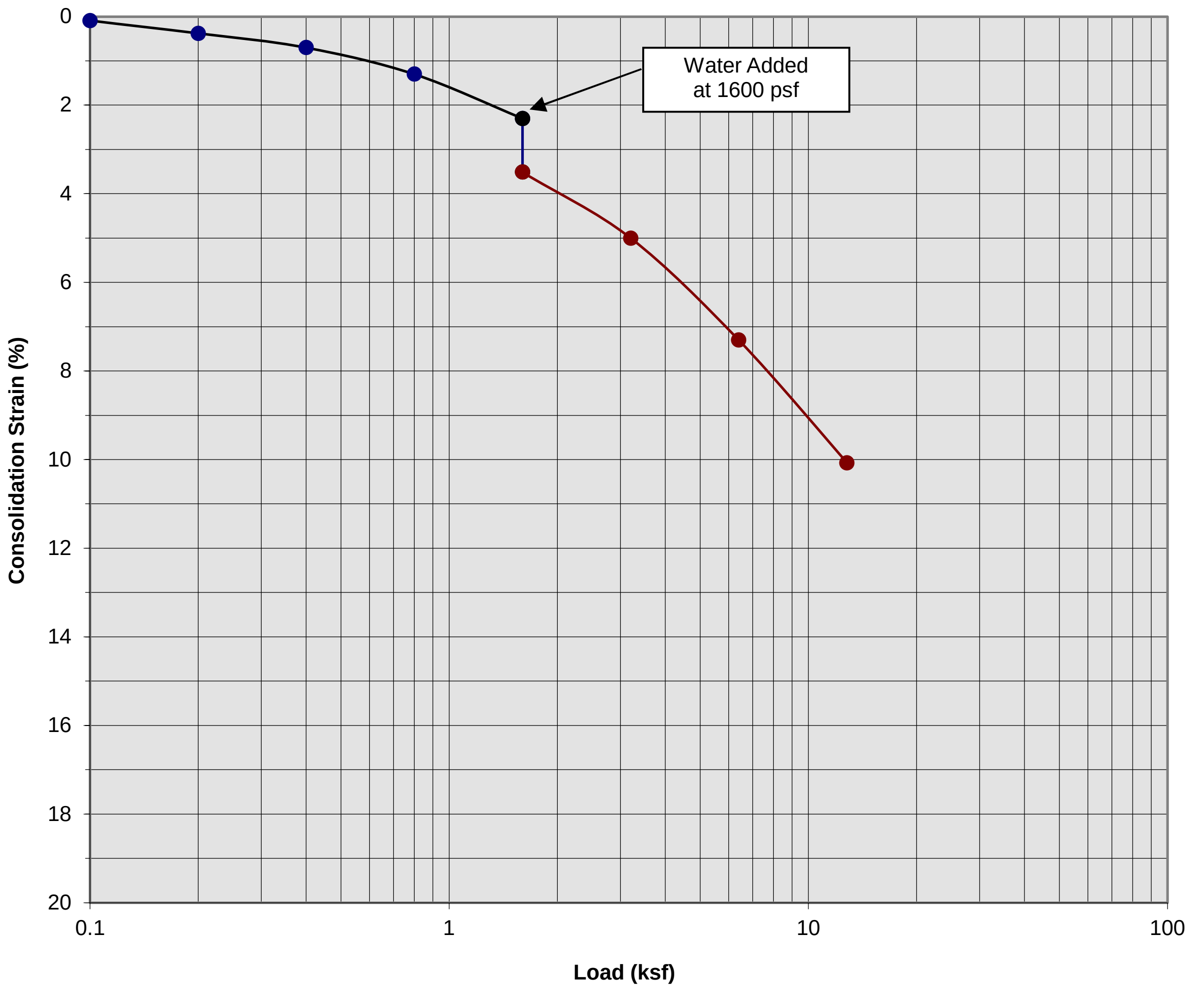
DEPTH (FEET)	SAMPLE	BLOW COUNT	POCKET PEN. (TSF)	GRAPHIC LOG	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PASSING #200 SIEVE (%)	ORGANIC CONTENT (%)	COMMENTS
					COLLUVIUM (Qc): Light Gray Brown Silty fine Sand, medium dense-damp							
		26			Dark Gray Brown fine Sandy Clay, very stiff- damp to moist	102	6					
		36				108	14					
					SANTIAGO FORMATION (Tsa): Gray Clayey Siltstone, Iron oxide staining, friable, weakly-cemented, dense to very stiff-very moist							
5		50			Light Gray interbedded Silty fine-grained Sandstone with Clayey Siltstone, Iron oxide staining, friable, weakly-cemented, dense to very stiff-very moist	101	17					
		65				109	18					
		66			Gray to Light Gray interbedded fine Sandy Siltstone and Clayey Siltstone, Iron oxide staining, friable, weakly-cemented, very stiff to very dense-very moist	110	18					
10												
		55					22					
15												

Boring Terminated at 15'

TBL 19G109-2.GPJ SOCALGEO.GDT 8/7/20

APPENDIX C

Consolidation/Collapse Test Results

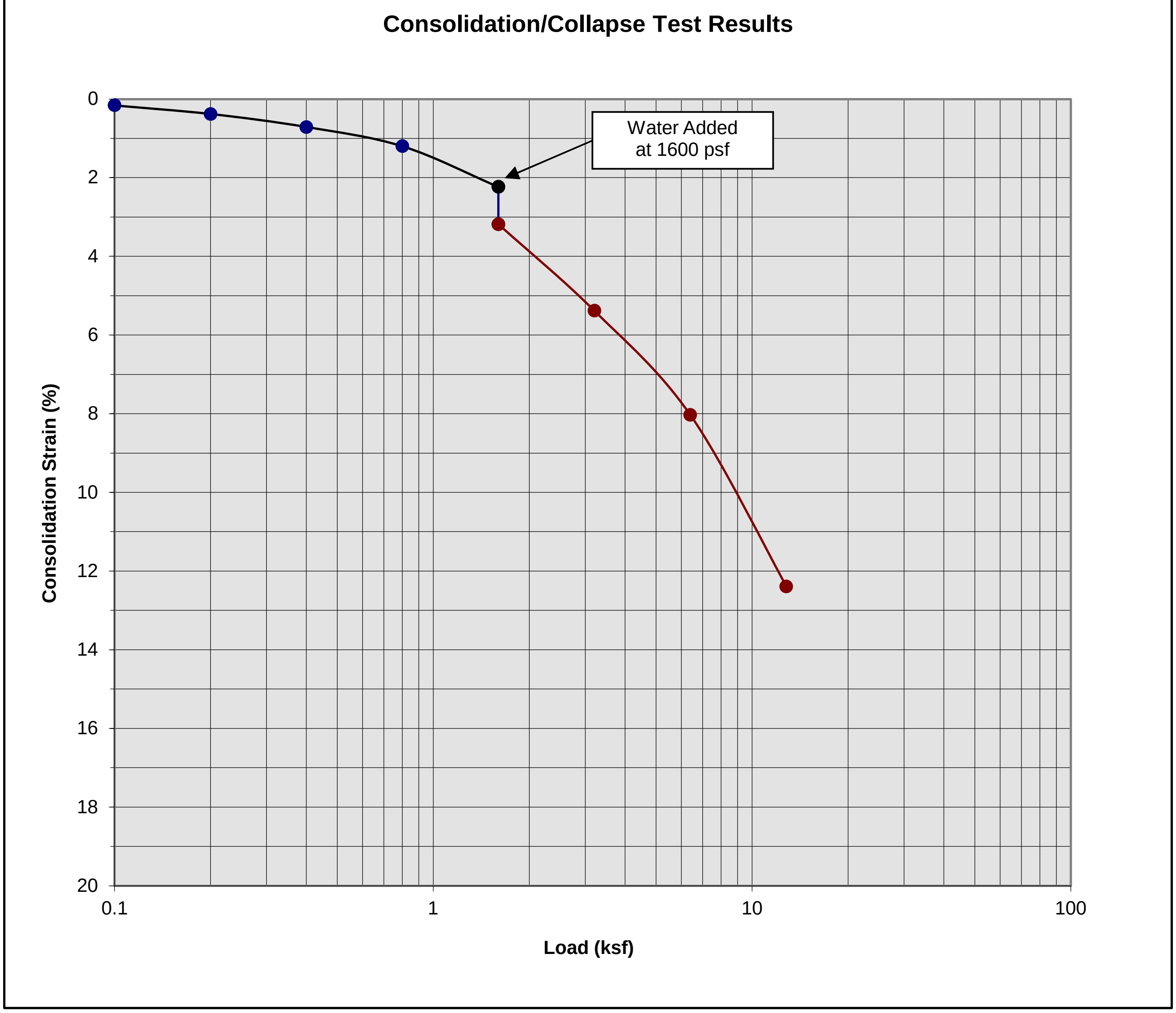


Classification: Colluvium: Light Brown Silty fine Sand

Boring Number:	B-4	Initial Moisture Content (%)	19
Sample Number:	---	Final Moisture Content (%)	24
Depth (ft)	1 to 2	Initial Dry Density (pcf)	93.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	103.7
Specimen Thickness (in)	1.0	Percent Collapse (%)	1.21

Veterans Park
Carlsbad, California
Project No. 19G109-2
PLATE C- 1

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



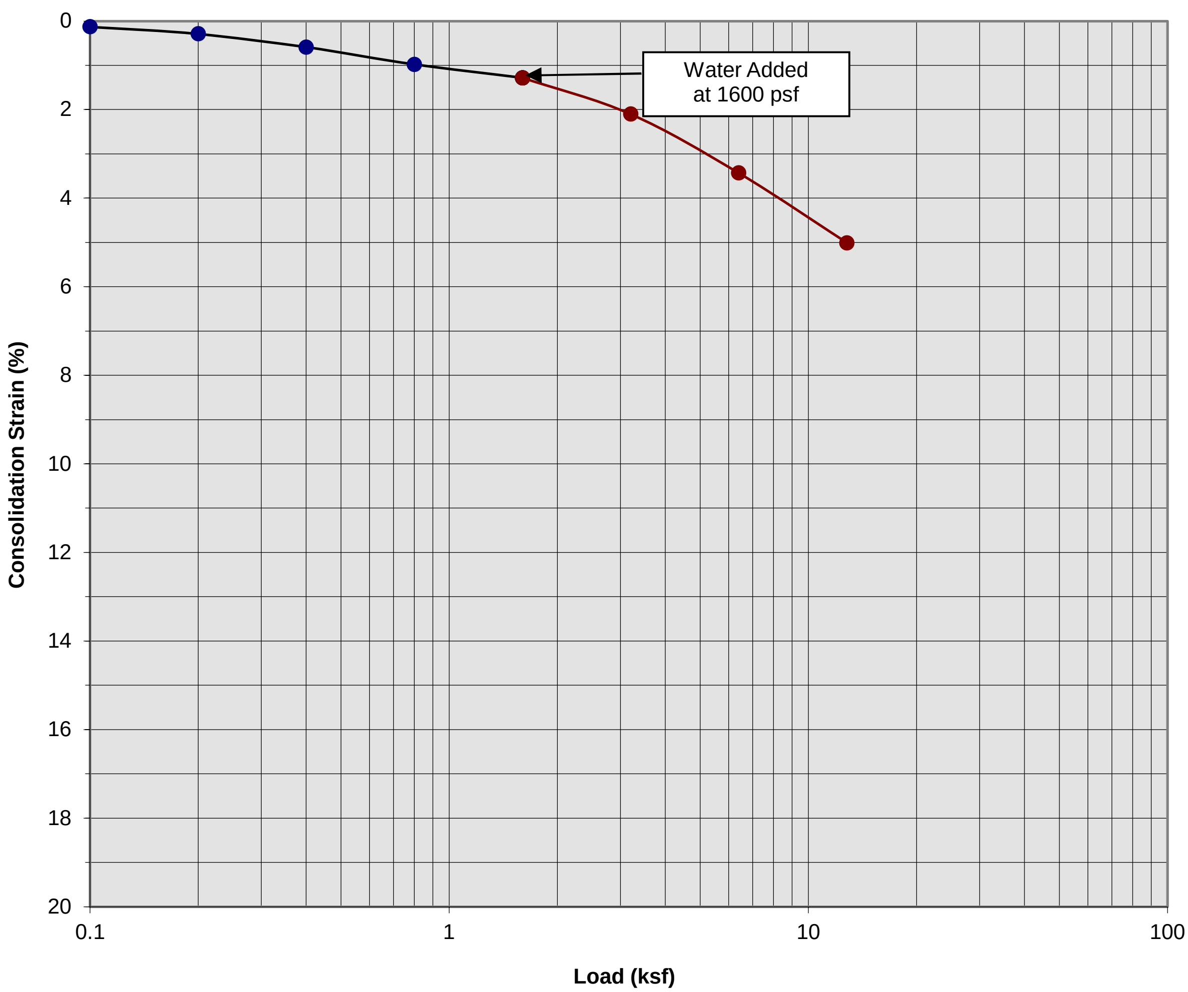
Classification: Colluvium: Dark Gray Brown fine Sandy Clay, some Silt

Boring Number:	B-4	Initial Moisture Content (%)	13
Sample Number:	---	Final Moisture Content (%)	20
Depth (ft)	3 to 4	Initial Dry Density (pcf)	92.8
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	105.0
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.95

Veterans Park
Carlsbad, California
Project No. 19G109-2
PLATE C- 2

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

Consolidation/Collapse Test Results

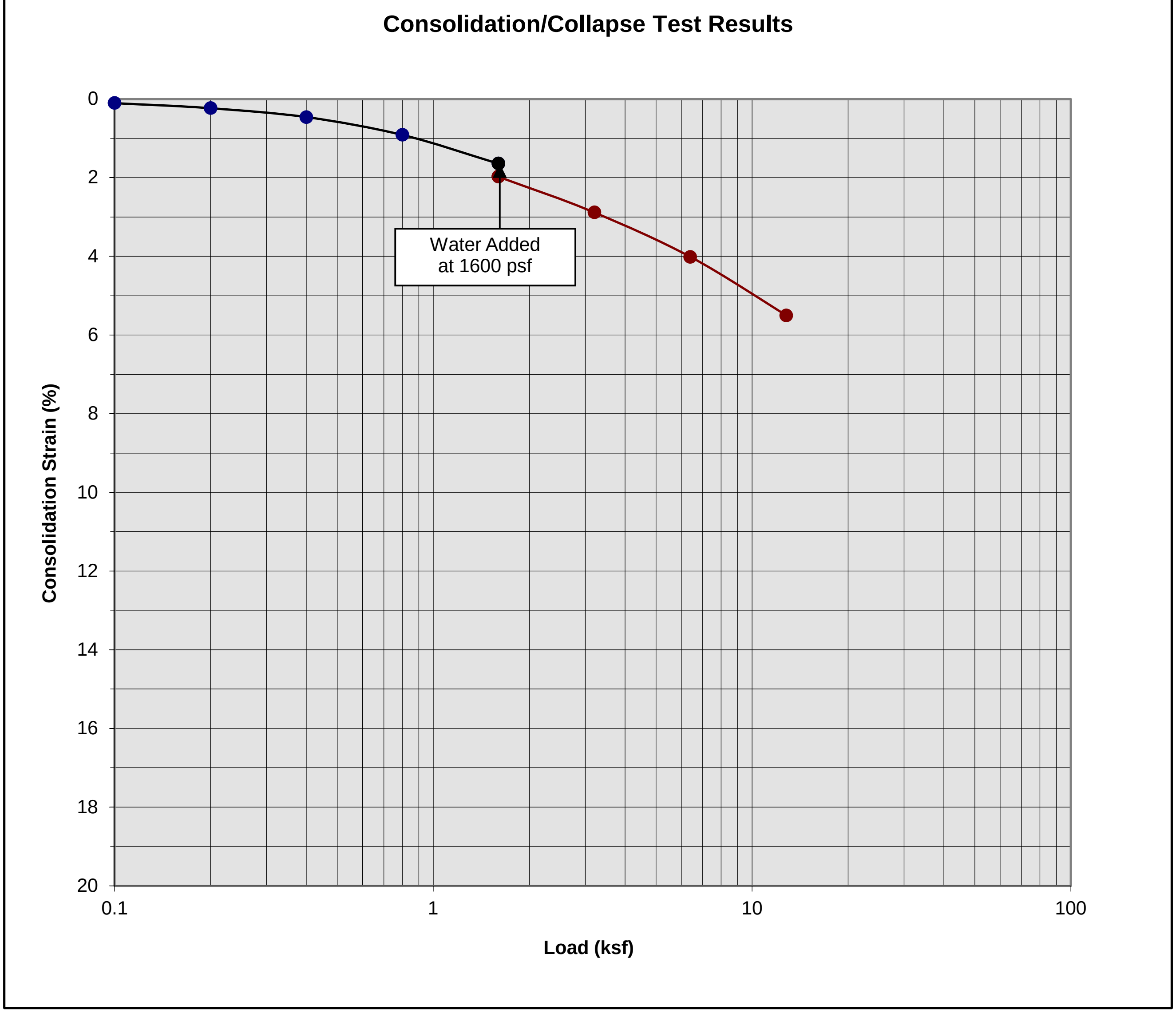


Classification: BEDROCK: Gray Silty fine grained Sandstone

Boring Number:	B-4	Initial Moisture Content (%)	15
Sample Number:	---	Final Moisture Content (%)	17
Depth (ft)	5 to 6	Initial Dry Density (pcf)	109.3
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	114.6
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.00

Veterans Park
Carlsbad, California
Project No. 19G109-2
PLATE C- 3

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

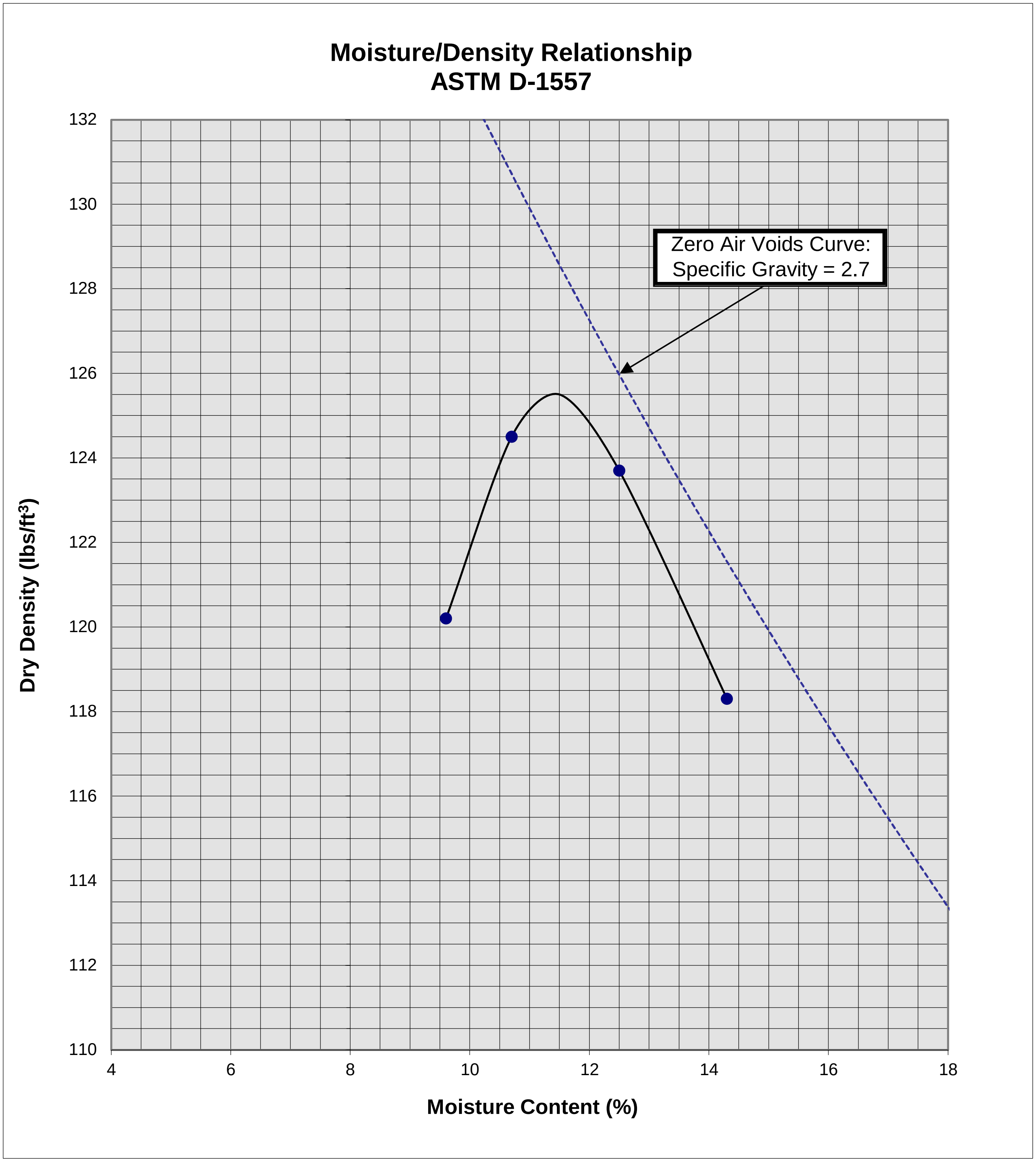


Classification: BEDROCK: Light Gray fine grained Sandy Siltstone

Boring Number:	B-4	Initial Moisture Content (%)	16
Sample Number:	---	Final Moisture Content (%)	18
Depth (ft)	7 to 8	Initial Dry Density (pcf)	106.5
Specimen Diameter (in)	2.4	Final Dry Density (pcf)	113.4
Specimen Thickness (in)	1.0	Percent Collapse (%)	0.33

Veterans Park
Carlsbad, California
Project No. 19G109-2
PLATE C- 4

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation



Soil ID Number	B-1 @ 0-5'
Optimum Moisture (%)	11.5
Maximum Dry Density (pcf)	125.5
Soil Classification	Gray Brown Clayey fine Sand to fine Sandy Clay

Veterans Park
Carlsbad, California
Project No. 19G109-2
PLATE C-5

SOUTHERN CALIFORNIA GEOTECHNICAL
A California Corporation

APPENDIX D

GRADING GUIDE SPECIFICATIONS

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

General

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

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

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

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

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

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

Site Preparation

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

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

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

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

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

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

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

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

<u>Compacted Fills</u>

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

Foundations

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

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

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

Fill Slopes

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

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

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

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

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

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

Cut Slopes

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

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

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

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

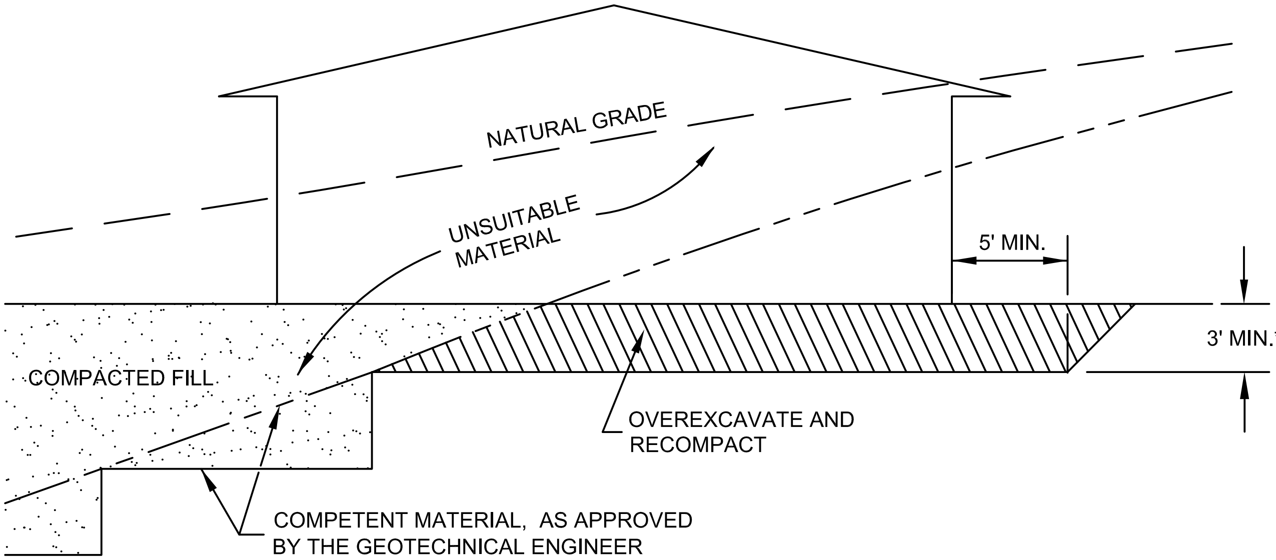
Subdrains

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

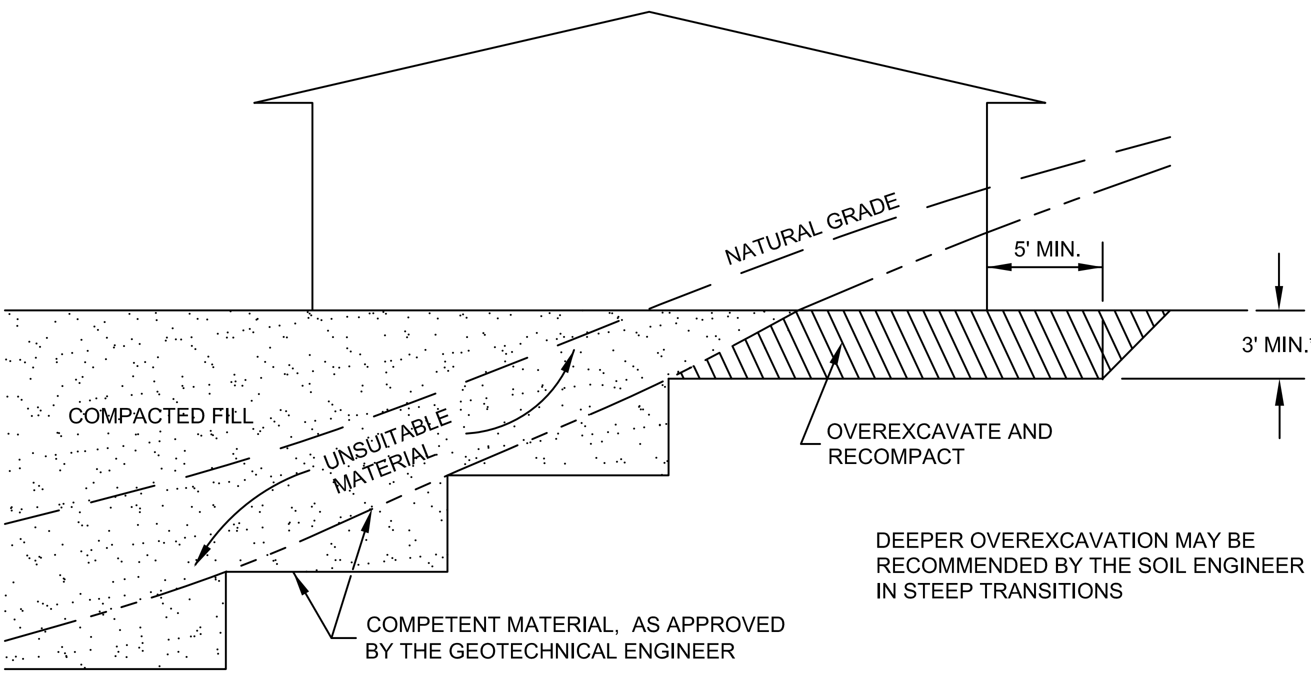
- Plastic pipe may be used for subdrains provided it is Schedule 40 or SDR 35 or equivalent. Pipe should be protected against breakage, typically by placement in a square-cut (backhoe) trench or as recommended by the manufacturer.

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

CUT LOT



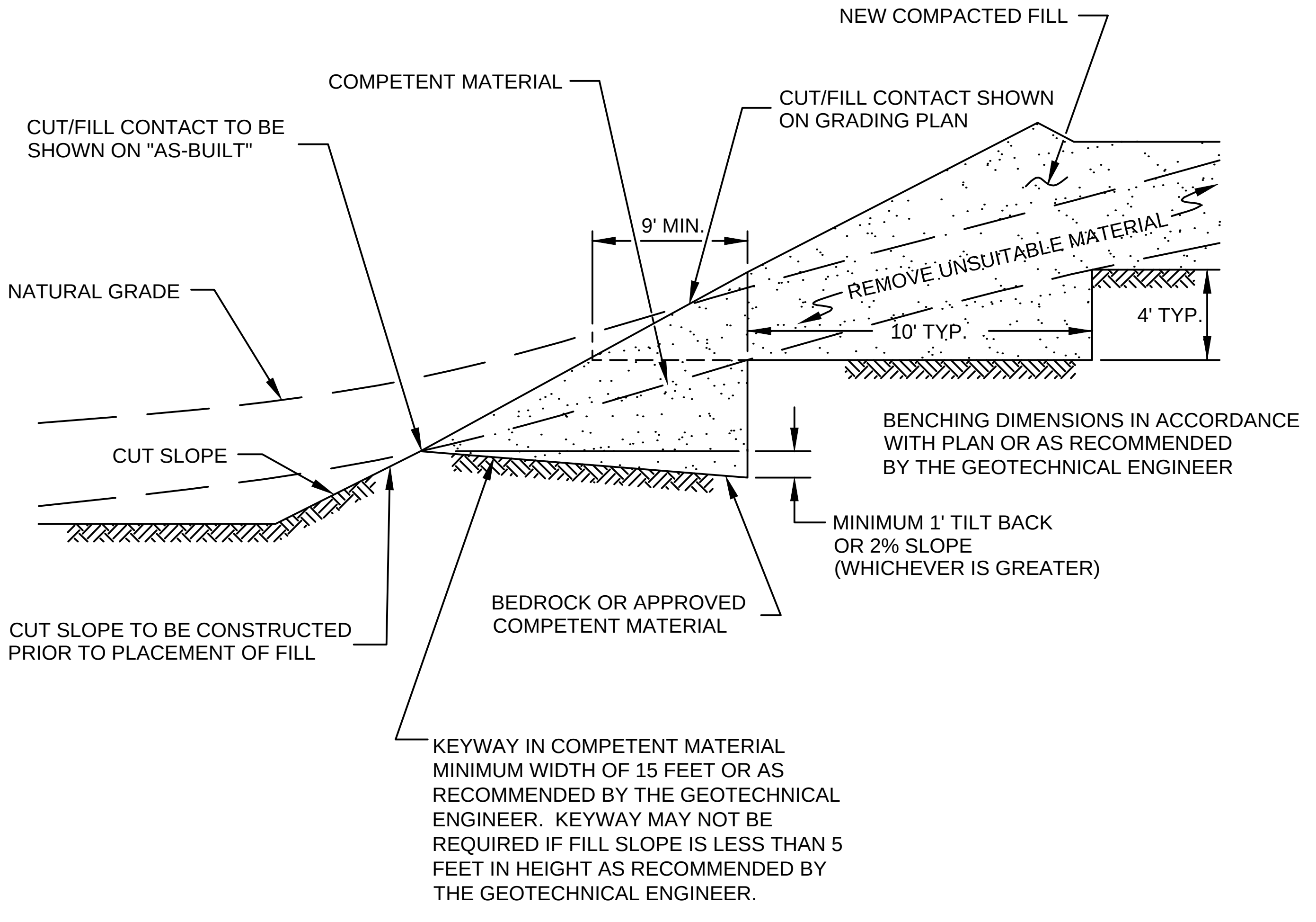
CUT/FILL LOT (TRANSITION)



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

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

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



FILL ABOVE CUT SLOPE DETAIL

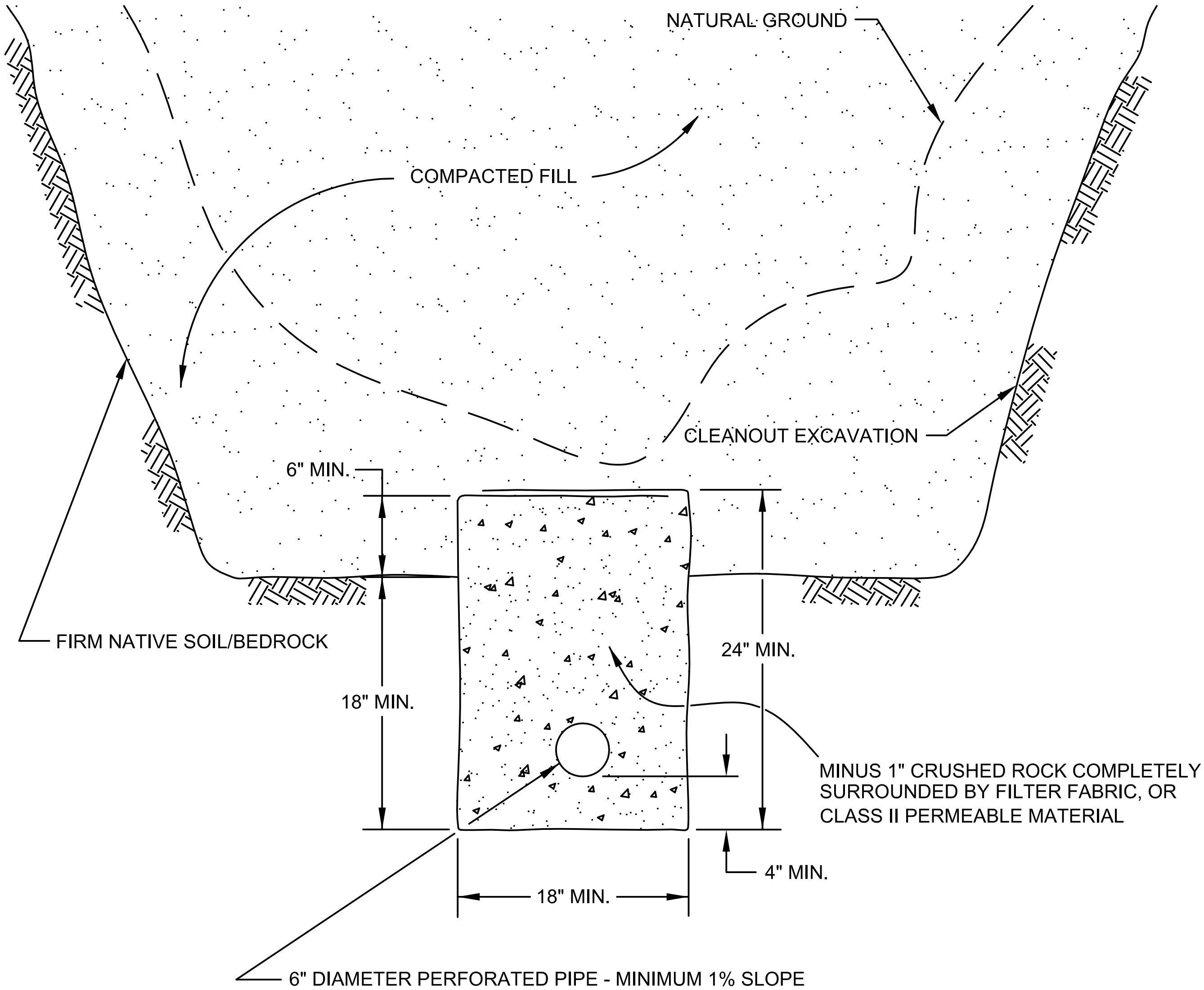
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-2

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

SCHEMATIC ONLY
NOT TO SCALE

CANYON SUBDRAIN DETAIL

GRADING GUIDE SPECIFICATIONS

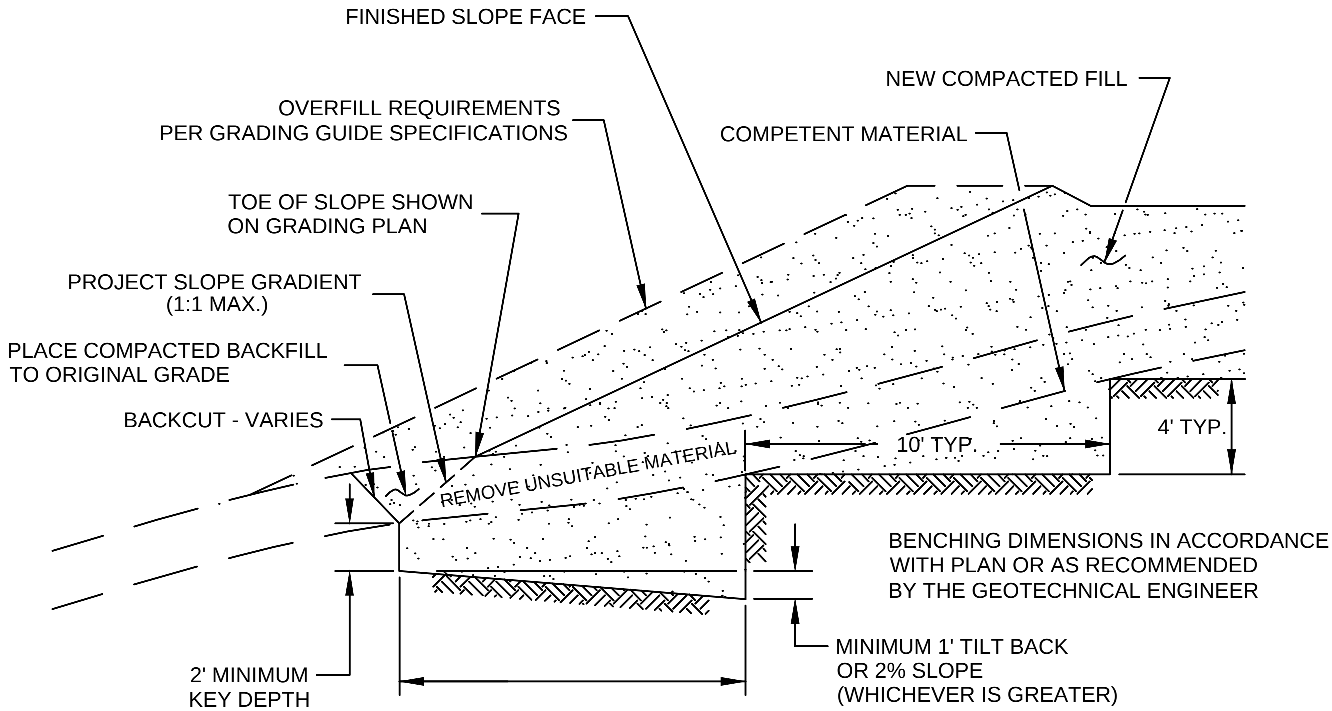
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-3

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

FILL ABOVE NATURAL SLOPE DETAIL

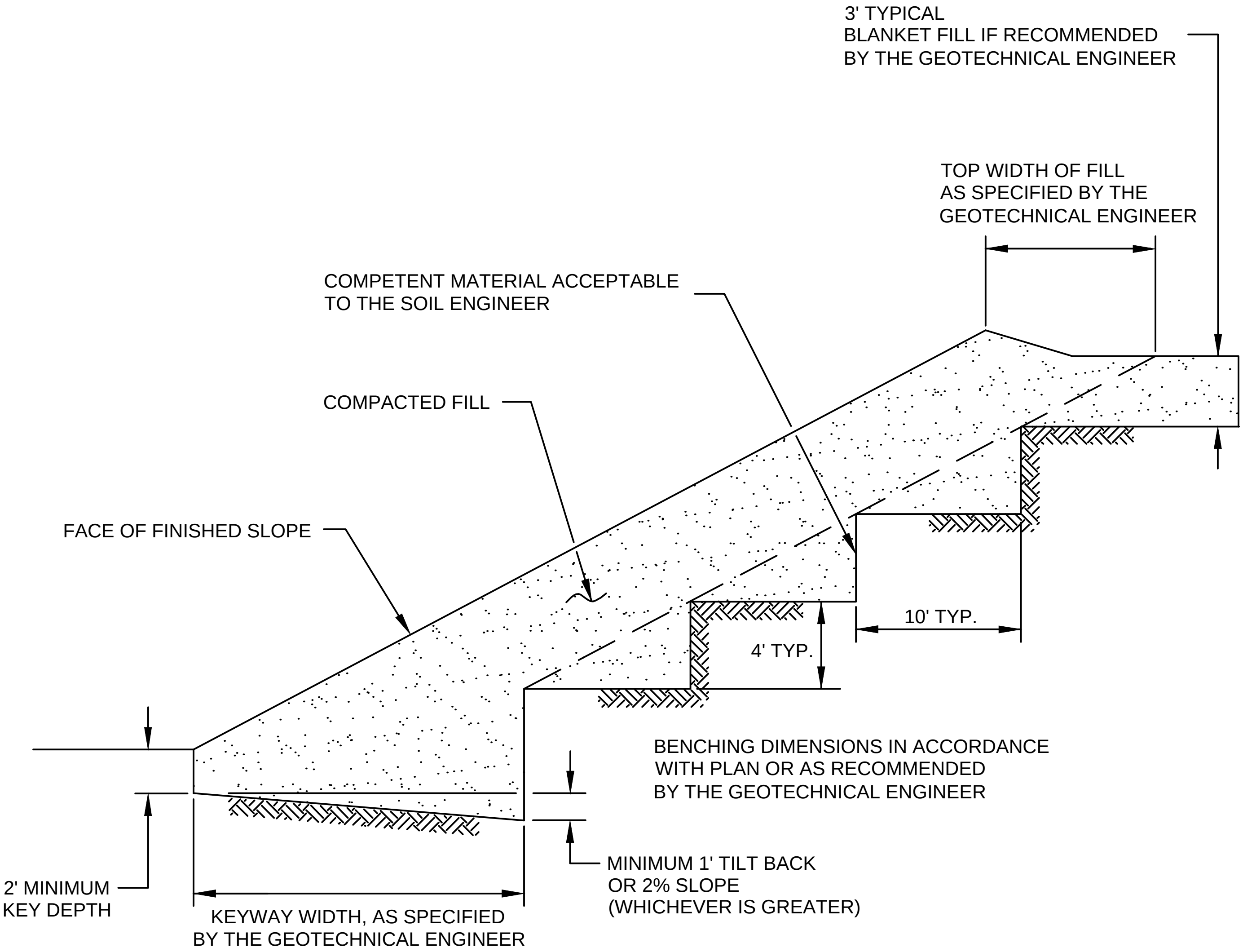
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-4

SOUTHERN CALIFORNIA GEOTECHNICAL



STABILIZATION FILL DETAIL

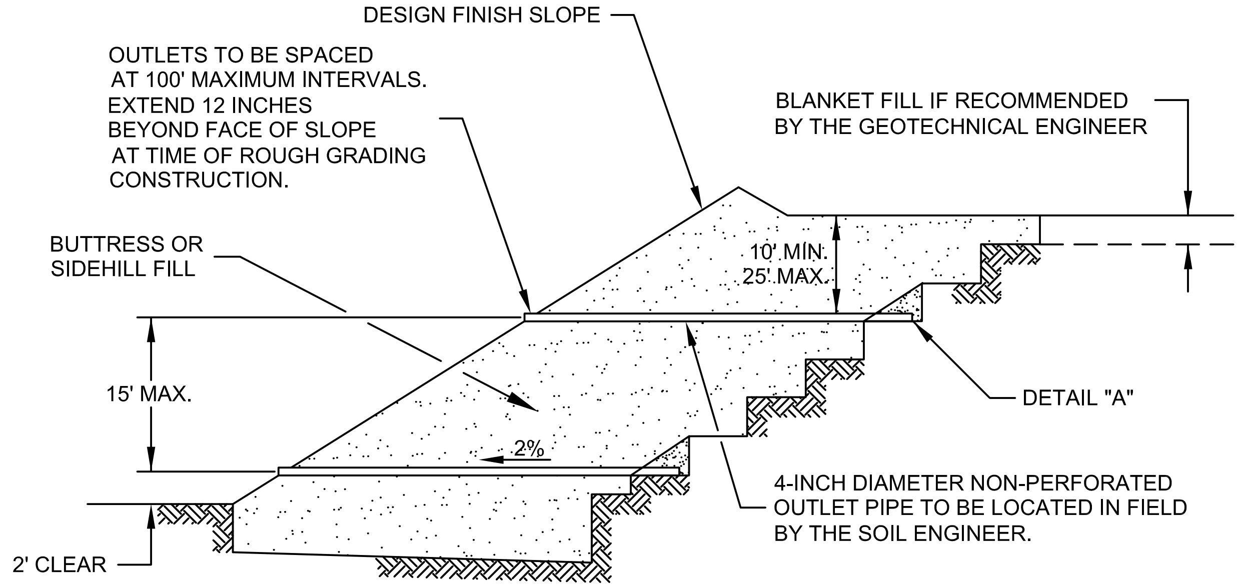
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-5

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



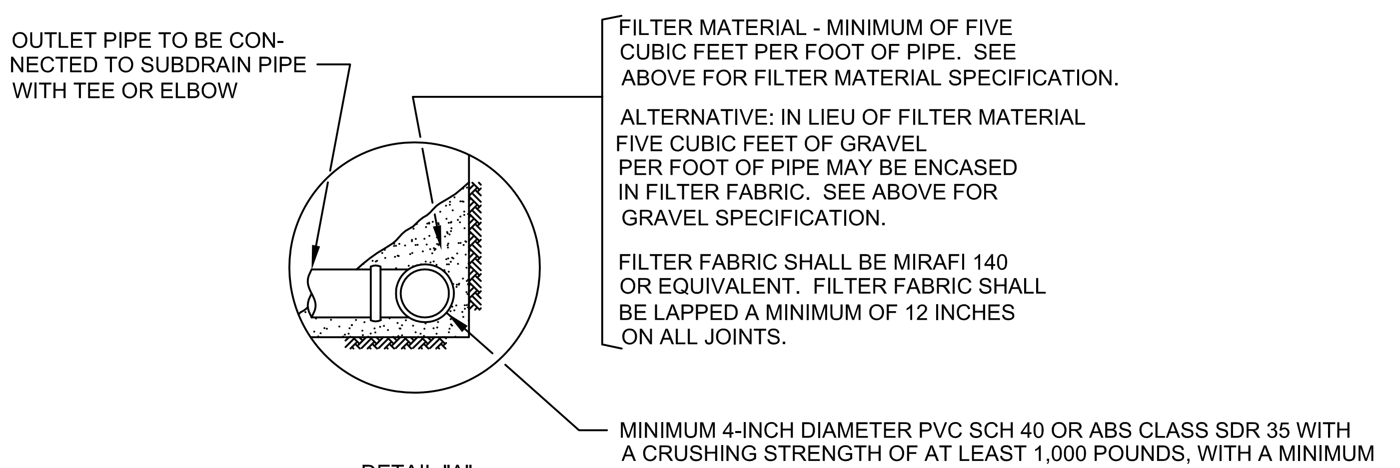
"FILTER MATERIAL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT: (CONFORMS TO EMA STD. PLAN 323)

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50



DETAIL "A"

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

SLOPE FILL SUBDRAINS

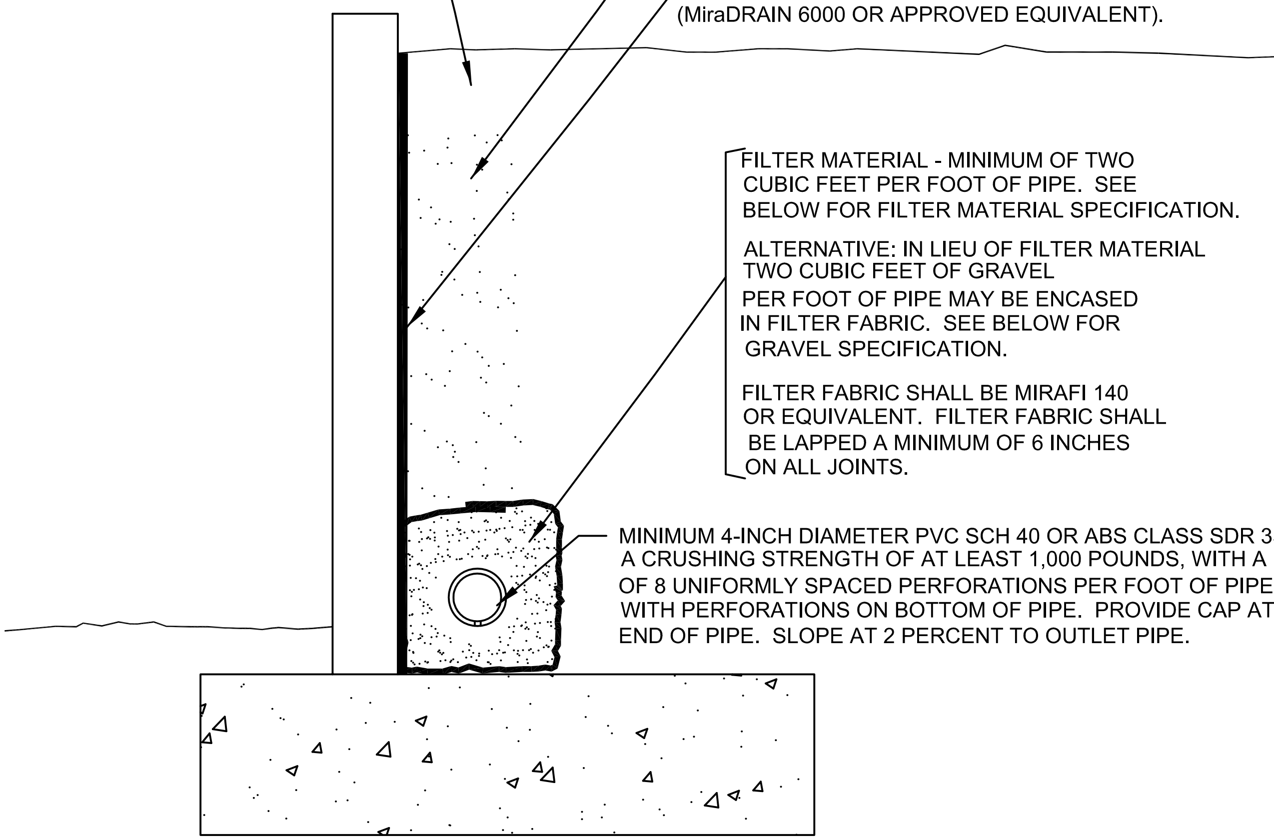
GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: JAS
CHKD: GKM

PLATE D-6

SoCalGeo
SOUTHERN CALIFORNIA GEOTECHNICAL



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

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

"GRAVEL" TO MEET FOLLOWING SPECIFICATION OR APPROVED EQUIVALENT:

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

SAND EQUIVALENT = MINIMUM OF 50

RETAINING WALL BACKDRAINS

GRADING GUIDE SPECIFICATIONS

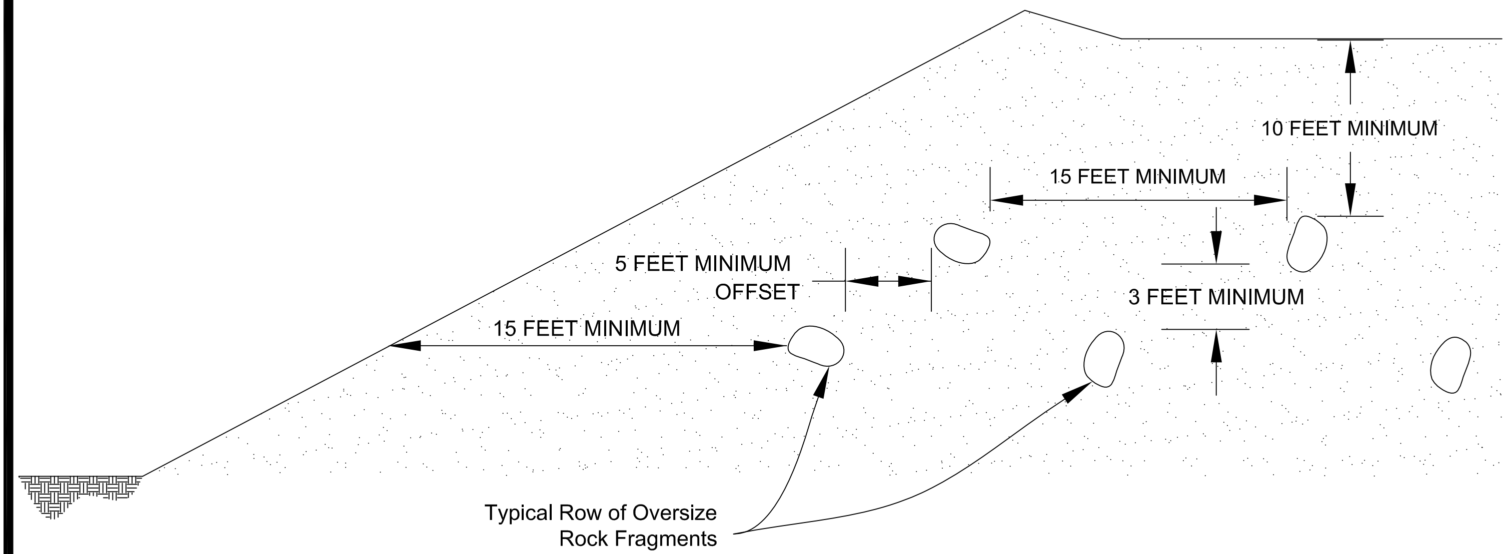
NOT TO SCALE

DRAWN: JAS
CHKD: GKM

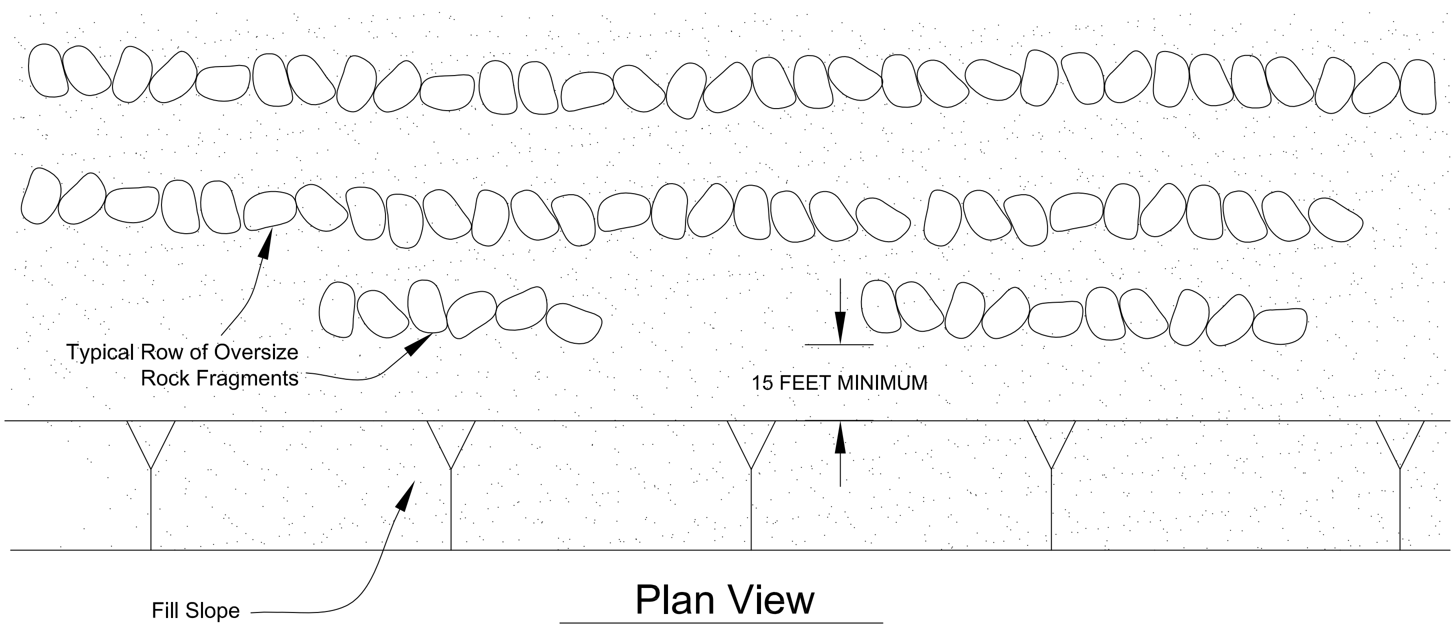
PLATE D-7

SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL



Section View



Plan View

PLACEMENT OF OVERSIZED MATERIAL

GRADING GUIDE SPECIFICATIONS

NOT TO SCALE

DRAWN: PM
CHKD: GKM

PLATE D-8

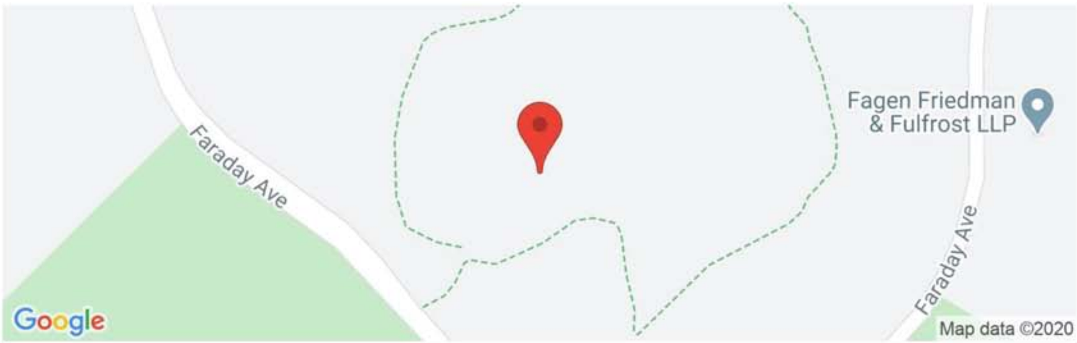
SoCalGeo

SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX E

OSHPD

Latitude, Longitude: 33.13589095, -117.30070677



Date	7/31/2020, 4:01:16 PM
Design Code Reference Document	ASCE7-16
Risk Category	III
Site Class	D - Default (See Section 11.4.3)

Type	Value	Description
S_S	1.004	MCE_R ground motion. (for 0.2 second period)
S_1	0.365	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.205	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.803	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	1.2	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.439	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.527	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.004	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.12	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.365	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.403	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.897	Mapped value of the risk coefficient at short periods
C_{R1}	0.908	Mapped value of the risk coefficient at a period of 1 s

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

SEISMIC DESIGN PARAMETERS - 2019 CBC

VETERANS MEMORIAL PARK

CARLSBAD, CALIFORNIA

DRAWN: RB
CHKD: RGT

SCG PROJECT
19G109-2

PLATE E-1

SoCalGeo SOUTHERN CALIFORNIA GEOTECHNICAL

APPENDIX F

LIQUEFACTION EVALUATION

Project Name	Veterans Memorial Park
Project Location	Carlsbad, CA
Project Number	19G109-2
Engineer	PM

MCE_G Design Acceleration	0.527	(g)
Design Magnitude	6.67	
Historic High Depth to Groundwater	37.5	(ft)
Depth to Groundwater at Time of Drilling	43	(ft)
Borehole Diameter	6	(in)

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	Uncorrected SPT N-Value	Unit Weight of Soil (pcf)	Fines Content (%)	Energy Correction	C_B	C_S	C_N	Rod Length Correction	$(N_1)_{60}$	$(N_1)_{60CS}$	Overburden Stress (σ_o) (psf)	Eff. Overburden Stress (Hist. Water) (σ_o') (psf)	Eff. Overburden Stress (Curr. Water) (σ_o') (psf)	Stress Reduction Coefficient (r_d)	MSF	KS	Cyclic Resistance Ratio (M=7.5)	Cyclic Resistance Ratio (M=6.67)	Cyclic Stress Ratio Induced by Design Earthquake	Factor of Safety	Comments
							(1)	(2)	(3)	(4)	(5)	(6)	(7)				(8)	(9)	(10)	(11)	(12)	(13)		
7	0	37.5	18.8		120		1.3	1.05	1.1	0.95	0.75	0.0	0.0	2250	2250	2250	0.92	1.03	1	0.06	0.06	N/A	N/A	Above Water Table
39.5	37.5	42	39.8	11	120	38	1.3	1.05	1.114	0.68	1	11.4	17.0	4770	4630	4770	0.80	1.12	0.91	0.17	0.18	0.28	0.62	**Liquefiable**
44.5	42	43	42.5	11	120	27	1.3	1.05	1.11	0.66	1	11.0	16.2	5100	4788	5100	0.78	1.11	0.9	0.17	0.17	0.29	0.59	**Liquefiable**
44.5	43	47	45	11	120	27	1.3	1.05	1.108	0.65	1	10.8	16.0	5400	4932	5275	0.77	1.11	0.9	0.16	0.16	0.29	0.57	**Liquefiable**
49.5	47	50	48.5	18	120	24	1.3	1.05	1.202	0.68	1	20.2	25.2	5820	5134	5477	0.74	1.22	0.85	0.30	0.31	0.29	1.07	**Liquefiable**

Notes:

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

(2) Borehole Diameter Correction (Skempton, 1986)

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

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

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

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

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

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

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

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

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

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

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

LIQUEFACTION INDUCED SETTLEMENTS

Project Name	Veterans Memorial Park
Project Location	Carlsbad, CA
Project Number	19G109-2
Engineer	PM

Boring No. B-1

Sample Depth (ft)	Depth to Top of Layer (ft)	Depth to Bottom of Layer (ft)	Depth to Midpoint (ft)	$(N_1)_{60}$	DN for fines cont	$(N_1)_{60\text{-}CS}$	Liquefaction Factor of Safety	Limiting Shear Strain γ_{min}	Parameter Fα	Maximum Shear Strain γ_{max}	Height of Layer		Vertical Reconsolidation Strain ε_v		Total Deformation of Layer (in)	Comments
				(1)	(2)	(3)	(4)	(5)	(6)	(7)			(8)			
7	0	37.5	18.8	0.0	0.0	0.0	N/A	0.50	0.95	0.00	37.50		0.000		0.00	Above Water Table
39.5	37.5	42	39.8	11.4	5.6	17.0	0.62	0.22	0.67	0.22	4.50		0.026		1.42	**Liquefiable**
44.5	42	43	42.5	11.0	5.2	16.2	0.59	0.24	0.70	0.24	1.00		0.027		0.33	**Liquefiable**
44.5	43	47	45	10.8	5.2	16.0	0.57	0.25	0.71	0.25	4.00		0.027		1.32	**Liquefiable**
49.5	47	50	48.5	20.2	5.0	25.2	1.07	0.09	0.22	0.03	3.00		0.007		0.26	**Liquefiable**
											Total Deformation (in)				3.32	

Notes:

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

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

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

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

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

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

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

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