Appendix G2

Geotechnical Investigation – Country Club and Resort Site

UPDATED
GEOTECHNICAL INVESTIGATION

CARLTON OAKS GOLF COURSE
COUNTRY CLUB AND RESORT SITE
(PA-3)
SANTEE, CALIFORNIA

PREPARED FOR:

CARLTON OAKS GOLF RESORT
SANTEE, CALIFORNIA

PREPARED BY:

GEOCON
INCORPORATED

GEOTECHNICAL ENVIRONMENTAL MATERIALS

FEBRUARY 4, 2022
REVISED JUNE 11, 2024
PROJECT NO. G2298-32-01

GEOCON
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GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. G2298-32-01
February 4, 2022
Revised June 11, 2024

Carlton Oaks Golf Resort
9200 Inwood Drive
Santee, California 92071

Attention: Mr. David Parks

Subject: UPDATED GEOTECHNICAL INVESTIGATION
CARLTON OAKS GOLF COURSE
COUNTRY CLUB AND RESORT SITE (PA-3)
SANTEE, CALIFORNIA

Dear Mr. Parks:

In accordance with the request of Summit Planning Group and Hunsaker & Associates, San Diego, Inc., we have prepared this updated geotechnical investigation for the subject project located within the Carlton Oaks Golf Course in Santee, California. The accompanying report presents the results of our study and our conclusions and recommendations regarding the geotechnical aspects of project development. This updated report was prepared to address revised grading plans, including off-site improvement areas, and to provide geotechnical design parameters in accordance with the 2022 California Building Code (2022 CBC).

The results of our study indicate that the site can be developed as planned, provided the recommendations of this report are followed. The primary geotechnical considerations during site development will be remedial grading of potentially compressible surficial deposits, settlement and/or liquefaction potential of young alluvium that will remain in-place below the groundwater, and settlement of existing utilities.

Should you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

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UPDATED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This updated report presents the results of a geotechnical study for the subject site located within a portion of the Carlton Oaks Golf Course property in Santee, California. (see *Vicinity Map*, Figure 1). The purpose of the study was to investigate the soil and geologic conditions at the sites, as well as evaluate geotechnical constraints, if any, that may impact areas of proposed development. This update report was prepared to address changes to the grading plans and to provide geotechnical design parameters in accordance with the 2022 CBC. In addition, we are addressing the proposed off-site improvements to Carlton Oaks Drive.

The scope of our study consisted of the following:

- Reviewing satellite imagery and readily available published and unpublished geologic literature.
- Reviewing grading plans prepared by Hunsaker and Associates, San Diego, Inc.
- Advancing twelve small-diameter borings within the development footprint to evaluate the underlying soil and geologic conditions (see Appendix A).
- Excavating twelve exploratory trenches using a rubber tire backhoe to evaluate the underlying soil and geologic conditions (see Appendix A).
- Performing laboratory tests on soil samples collected to evaluate their physical properties (see Appendix B).
- Performing two infiltration tests in select areas to be utilized during storm water management design and providing storm water management guidelines in accordance with the City of Santee Storm Water Standards Manual (see Appendix C).
- Performing a liquefaction evaluation (see Appendix D).
- Preparing this report presenting our exploratory information and our conclusions and recommendations regarding the geotechnical aspects of developing the site as presently proposed.

The approximate locations of the exploratory trenches, borings and infiltration tests are shown on the *Geologic Map*, Figure 3.

2. SITE AND PROJECT DESCRIPTION

We understand that the overall proposed project site (PA-1, PA-2 and PA-3) that will be developed is located on approximately 169 acres and would include the redesign of the existing Carlton Oaks Golf

Course and the following components: (1) redesign the golf course; (2) reconstruction of the clubhouse and pro shop, practice area, and learning center structure; (3) a hotel and associated cottages*;* (4) residential accessory uses consisting of two residential neighborhoods with open space areas *(reported under separate cover)*; and (5) related on-site infrastructure. Approximately 3.4 acres consist of areas outside of the project site that will be developed with improvements associated with the Project and are located either in the City of San Diego or Santee (Off-site improvement areas). The off-site improvement areas and the proposed project site (developed and undeveloped) make up the CEQA Study area, as shown on the Site Plan and Off-Site Improvement Area exhibit presented as Figure 2.

The project consists of a hotel, cottages, restaurant, cart barn/pro shop, tournament hall, and learning center site with associated parking lots and underground improvements that total approximately 8 acres of land located within the Carlton Oaks Golf Course in Santee, California. The existing golf course is bounded to the north by residential homes and open space, the south and west by golf course property, and the east by open space.

Topographically, the site exhibits relatively flat to gently sloping terrain with vegetation primarily consisting of maintained grass areas utilized for the golf course along with areas of heavy brush and dense vegetation and numerous mature trees scattered about the property. Man-made improvements consist of a groundkeeper maintenance yard, offices, maintenance buildings, and other hardscape improvements.

We understand that the proposed development includes grading to support a three-story hotel site, cottages, cart barn and pro shop, learning center, and associated improvements. Associated private roadways, parking lots, public and private underground utilities, and a practice area is planned. The main access to the site is from PA-2 via a bridge and access road. An emergency access road will be located off of Carlton Oaks Drive. Proposed off-site improvements consist of removing several parking stalls and constructing the emergency access roadway. As such, the removed parking stalls will be relocated to the northeast.

The recent revisions to the grading plans include:

- The country club and resort site (PA-3) footprint has been reduced and senior living facility and proposed condominiums have been eliminated.
- A practice area has been added to the northern portion of the resort site adjacent to the existing townhomes.
- The driving range has been removed and has been changed to a practice area.
- Cottages have been added west of the hotel.

- Addition of a vehicle crossing bridge from PA-2 (Residential North Site) to PA-3 (Residential Hotel Site) as the main access.
- Water quality basins are being replaced with Modular Wetland Systems.
- The former access through the adjacent condominiums (Vista del Verde) has been changed from a primary access to a secondary access point that will be used for emergency access only.

The grading plans indicate cuts and fills on the order of 10 and 15 feet, respectively, to create the building pads, parking areas and practice area. Grading will consist of raising the building pads approximately 10 feet to 15 feet and cuts of approximately 10 feet in the practice area.

Embankments up to approximately 10 feet thick are proposed over existing sewer and water utilities located in the parking lot and between the cart barn/pro shop and learning center. A discussion of the proposed grading and its potential impact to these improvements should occur between the appropriate parties as project development plans progress.

3. SOIL AND GEOLOGIC CONDITIONS

Based on a review of published geologic maps, and observations during our site reconnaissance and subsurface investigation, the site is underlain by two surficial soil units and one formational unit. The surficial units consist of previously placed artificial fill and Holocene-age young alluvial deposits. The formational unit consists of Eocene-age Friars Formation (see Regional Geologic Map, Figure 4). Each is discussed below in order of increasing age.

3.1 Artificial Fill (Qaf and Qaf_2)

Previously placed artificial fill consisting of golf course and roadway embankments were mapped across the site based on topographic interpretation. The fill was up to 7-feet-thick, and primarily consisted of loose to medium dense silty sand, with occasional abundant roots. The previously placed fill is not suitable for the support of proposed improvements or structural fill and will require remedial grading in the form of complete removal and recompaction where structural improvements are proposed. The golf course grass surface, along with other deleterious material, such as trees, heavy brush, concrete, trash, debris, etc., will require removal and exportation from the site.

3.2 Young Alluvium (Qya)

Young alluvial soils (Holocene-age) are present below the artificial fill on the site with a total thickness of approximately 12 to 27 feet. The base of this unit was not encountered in Boring No. 4. These deposits primarily consist of loose to medium dense silty, fine to coarse sands. The alluvial soils are slightly compressible when subjected to additional fill or structural loading and are potentially liquefiable.

3.3 Friars Formation (Tf)

The Middle Eocene-age Friars Formation was encountered beneath the alluvium and varies from 15 to 30 feet below the existing ground surface. This formation, where encountered, consists of very stiff to hard, pale green, sandy siltstone and dense, silty fine sandstone. We do not anticipate this unit will be encountered during grading of the site.

4. GROUNDWATER

Groundwater associated with the San Diego River and its tributaries was encountered in the exploratory borings and trenches from 3 to 10 feet below the existing ground surface. In addition, water is present at the surface in several ponds/lakes on the golf course. The groundwater will be an important factor in determining the depth of remedial grading of surficial deposits. In addition, groundwater should be considered when planning improvements that extend below these depths. The groundwater depths indicated on the *Geologic Map* are reflective of locations encountered during the time of our investigations and may vary seasonally. Wet alluvial removals will be encountered during grading operations, leading to difficult excavation and compaction conditions.

It is not uncommon for groundwater or seepage conditions to develop where none previously existed. Groundwater elevations are dependent on seasonal precipitation, irrigation, and land use, among other factors, and vary as a result. Proper surface drainage will be important to future performance of the project. Depending upon seasonal conditions at the time of grading, specialized equipment to excavate the surficial soils and drying or mixing with other onsite materials to reduce the moisture content prior to placement as compacted fill may be required.

5. GEOLOGIC HAZARDS

5.1 Faulting and Seismicity

Based on our reconnaissance, field investigation, and a review of published geologic maps and reports, the site is not located on any known “active,” “potentially active” or “inactive” fault traces as defined by the California Geological Survey (CGS). The CGS considers a fault seismically active when evidence suggests seismic activity within roughly the last 11,000 years.

According to the computer program *EZ-FRISK* (Version 7.65), 6 known active faults are located within a search radius of 50 miles from the property. The nearest known active faults are the Newport Inglewood and Rose Canyon Fault Zones, located approximately 12 miles west of the site and are the dominant sources of potential ground motion. Earthquakes that might occur on the Newport Inglewood or Rose Canyon Fault Zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. Table 5.1.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS2008, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships.

TABLE 5.1.1
DETERMINISTIC SEISMIC SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
Newport Inglewood	12	7.5	0.43	0.30	0.39
Rose Canyon	12	6.9	0.36	0.27	0.30
Coronado Bank	24	7.4	0.30	0.19	0.22
Palos Verdes Connected	24	7.7	0.33	0.20	0.26
Elsinore	30	7.85	0.31	0.19	0.24
Earthquake Valley	34	6.8	0.19	0.12	0.11

We used the computer program *EZ-FRISK* to perform a probabilistic seismic hazard analysis. The computer program *EZ-FRISK* operates under the assumption that the occurrence rate of earthquakes on each map-able Quaternary fault is proportional to the faults slip rate. The program accounts for fault rupture length as a function of earthquake magnitude, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS, Campbell-Bozorgnia (2008) NGA USGS, and Chiou-Youngs (2008) in the analysis. Table 5.1.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedance.

TABLE 5.1.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedance	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.43	0.35	0.41
5% in a 50 Year Period	0.32	0.27	0.30
10% in a 50 Year Period	0.25	0.21	0.22

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the California Building Code (CBC) and other currently adopted City of Santee codes.

5.2 Liquefaction, Seismically Induced Settlement and Lateral Spreading

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless and poorly graded sand, groundwater is encountered within 50 feet of the surface, and soil relative densities are less than about 70 percent. If the four previous criteria are met, a seismic event could result in a rapid pore water pressure increase from the earthquake-generated ground accelerations. We performed liquefaction analyses and the results indicate the potential is low.

The site is not located within a state-designated liquefaction hazard zone; however, the County of San Diego Hazard Mitigation Plan (2010) maps the site within a zone with liquefiable layers. The City of Santee Geotechnical/Seismic Hazard Study for The Safety Element of the Santee General Plan (2002) maps the site as having a "moderate to high" liquefaction hazard potential (See Reference No. 16). The current standard of practice, as outlined in the *Recommended Procedures for Implementation of DMG Special Publication 117A, Guidelines for Analyzing and Mitigating Liquefaction in California* requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structures. We explored to a maximum depth of approximately 32½ feet during our investigations; however, Friars Formation was encountered approximately 15 to 25 feet below the ground surface and we do not expect liquefaction to occur within the Friars Formation.

Exploratory borings excavated within the younger alluvium in the area of the hotel site and sheet-graded pad (formerly the assisted living site) revealed that this deposit is up to approximately 15 and 25 feet-thick, respectively, and is underlain by the Friars Formation. The water table is approximately 3 to 6 feet below the ground surface. The borings indicate the alluvium consists of loose to very dense, silty, fine to coarse sand. Laboratory testing indicates that this deposit has a relatively low compression potential. The grading plan indicates approximately 10 to 15 feet of fill is planned in the area of the sheet-graded pad (formerly assisted living site) and hotel site, respectively, where the younger alluvium will be left in place below the groundwater. Based on these factors, and considering the conditions required for liquefaction to occur, it is our opinion that the potential for liquefaction and seismically induced settlement occurring within the soils is considered to be "moderate" to "high".

We used the methods following the methodology of NCEER (2001 and 2008) to perform a liquefaction evaluation. We used a computed site acceleration (PGA_m) of 0.386g (based on ASCE 7-10) and a modal magnitude of 6.89 as evaluated from the NSHM 2014 Dynamic edition using a recurrence interval of 2,475 years (2% in 50 years) on the United States Geological Survey web site. We performed the liquefaction analysis using the data from the exploratory borings performed during our field investigation. The boring logs are presented in Appendix A, and the results of our liquefaction analyses are presented in Appendix D.

We used the blow counts for the liquefaction analysis based on the driven samplers in the field. In addition, we adjusted blow counts using a California sampler by two-thirds to obtain equivalent Standard Penetration Test (SPT) values. The blow counts were also adjusted for boring diameter, sampling method, rod length, overburden pressure, and energy delivered to the sampler corresponding to a driving-energy of 60 percent ($N_{1|60}$). We further adjusted the blow counts for estimated fines content and calculated a factor of safety. A site is considered to be susceptible to liquefaction when the computed factor of safety is less than 1.0. The results of our liquefaction analysis indicate factors of safety of approximately 0.2 to 3.6 within the alluvial soil below the groundwater table.

Our analyses indicate that liquefaction potential is moderate to high within the alluvial soil below the groundwater table in the areas of Borings B-1 through B-3 (sheet-graded pad-formerly assisted living site) and B-8 through B-11 (hotel site) for the levels of ground shaking assumed. We do not expect liquefaction to occur within the underlying Friars Formation due to the age and density of the formational material. Adverse impacts associated with liquefaction include ground rupture and/or sand boils, lateral spread, and settlement of the liquefiable layers.

Sand boils occur where liquefiable soil is extruded upward through the soil deposit to the ground surface. Providing an increase in overburden pressure and a compacted fill mat can mitigate surface manifestation. Research presented by Ishihara (1985) indicates that the presence of a non-liquefiable surface layer typically results in the effects of at-depth liquefaction from reaching the surface. Modifications to Ishihara's chart have been made to include higher ground accelerations (Ishihara's 1985 chart was based on a 0.2g ground acceleration) by Youd and Garris (1995). The liquefaction potential is considered moderate to high based on our analyses. Proposed fill placement above the liquefiable soil reduces the probability to experience surface manifestation. Based on review of the modified Ishihara chart, surface manifestation of liquefaction is likely beneath the hotel site, but unlikely beneath the assisted living site.

The liquefaction analyses (included in Appendix D) indicate that zones of the underlying alluvial soils to depths of approximately 25 feet below proposed on-grade structures could be prone to up to 5 inches of total liquefaction-induced settlement during PGA_M ground motion. Table 5.3 summarizes the zones of potentially liquefiable soils, the thickness of non-liquefiable soil, and the estimated dynamic settlement. Recommendations presented in this report are intended to reduce the effects of seismically-induced settlement on the proposed structures.

TABLE 5.3
ESTIMATED ZONES OF LIQUEFIABLE SOIL, THICKNESS OF NON-LIQUEFIABLE SOIL, AND ESTIMATED DYNAMIC SETTLEMENT

Boring No.	Location	Approximate Depth (ft) of Liquefiable Soil Zones	Thickness of Non-Liquefiable Soil Above Liquefiable Soil (ft)	Estimated Dynamic Settlement (in)
1	Former Assisted Living Site	NA	14.5	0.0
2		6-9	13.5	1.0
3		NA	14.5	0.0
8	Hotel Site	8-25	9	4.75
9		3-7	14	1.4
10		5-12	15	1.4
11		6-17	16	4.7

Lateral spreading occurs when liquefiable soil is in the immediate vicinity of a free face such as a slope. Factors controlling lateral displacement include earthquake magnitude, distance from the earthquake epicenter, thickness of liquefiable soil layer, grain size characteristics, fines content of the soil and soil density. Bartlett and Youd (1995) have concluded that lateral spreading is restricted to sediments with corrected SPT blow counts of 15 or less for earthquake magnitudes less than or equal to 8.0.

We analyzed the potential for lateral spreading using a conventional limit equilibrium slope stability analysis. We performed the slope stability analysis using residual undrained shear strength parameters (phi = 0) for the potentially liquefiable alluvial soils based on information provided in *Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction in California* and utilizing a published relationship between blow count and residual undrained shear strength presented by Seed and Harder (1990). Based on our analyses, a minimum factor of safety ranging between 1.1 and 1.6 was computed assuming residual undrained shear strength parameters for the liquefiable soils, therefore, it is our opinion there is a low potential for slope instability and lateral spreading impacts to the proposed buildings during a design level seismic event. The results of our analysis are shown graphically in Appendix D (see Figures D-15 through D-17).

5.3 Seiches and Tsunamis

Considering the project location in relation to the ocean and proposed grade elevation (305 to 325 feet above MSL), the site is not located within a tsunami inundation zone. Seiche-related phenomena are defined as being proximal to a lake, reservoir, or bay. The project is not located near a large body of water such as those; however, proximity to the San Diego River is discussed below.

5.4 Flooding from Dam Hazards

The City of Santee Geotechnical/Seismic Hazard Study for The Safety Element of the Santee General Plan (2002) identifies the site as being within the zone of inundation in the San Diego River Valley downstream of three major dams in San Diego County. These include the San Vicente Dam, the El Capitan Dam, and the Chet Harrit Dam (Lake Jennings). According to the Safety Element report, maps prepared in the 1970s indicate the site is located within the inundation limits considering complete failure of any one of the three dams. Information concerning the safety of these dams, which is reviewed annually by the California Department of Water Resources, Division of Dam Safety, may be obtained from that department.

5.5 Landslides

No evidence of landslide deposits was encountered at the site during the geotechnical investigation.

5.6 Static Settlement

Estimates of potential static settlement are generally based on the thickness of alluvium left-in-place, the thickness of additional fill to achieve finish grade and the compressibility characteristics of the alluvial materials. The rate of settlement is generally based on the compressibility characteristics of the alluvial materials and the drainage path thickness that would allow for pore water pressure dissipation.

The alluvial deposits beneath the senior living site were found to be slightly compressible when subjected to increased vertical stress. Laboratory consolidation tests were performed on samples of the alluvium to aid in evaluating the magnitude of settlement that could occur from the proposed fill and building loads presently planned. Based on the test results and analysis, it is estimated approximately 2.5 inches of static settlement could occur beneath the sites, and take at least several months without geotechnical mitigation. This settlement would delay construction of the project until primary consolidation is essentially complete.

It should be noted that the magnitude of the total settlement and the associated time rate of consolidation will not be uniform throughout the site due to the variable thickness and compressibility of the underlying alluvial materials. In addition, the variable thickness of proposed fill will affect the magnitude of settlement.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 No soil or geologic conditions were encountered that, in the opinion of Geocon Incorporated, would preclude the development of the property as proposed, provided the recommendations of this report are followed.

6.1.2 The site is underlain with artificial fill associated with golf course grading and off-site roadways. The artificial fill is underlain by saturated younger alluvium. Our study indicates that, in structural areas, all artificial fill (Qaf) and limited portions of young alluvial deposits above groundwater should be removed and recompacted as engineered fill. Removals should be performed to approximately 2 feet above the groundwater elevation at the time of grading. The estimated thickness of remedial grading, based on the water elevations at the time of our study, is shown on Figure 3.

6.1.3 Our study indicates that up to 2.5 inches of static settlement may occur after grading based on laboratory testing and the current development plan. As a consequence, construction of the proposed improvements, including underground utilities should be delayed until the primary consolidation of the younger alluvial deposits is essentially complete. We anticipate this time frame to be relatively short but settlement monitoring should be performed to verify when primary compression has occurred. The specific settlement monitoring procedure can be provided as development plans progress. If ground improvement techniques are used to mitigate the liquefaction potential, settlement monitoring would not be needed.

6.1.4 Embankments up to approximately 10-feet-thick are proposed over existing sewer and water utilities located along the east side of the assisted living site, central parking area, and north side of the entrance stormwater basin. A discussion of the proposed grading and its potential impact to these improvements should occur between the appropriate parties as project development plans progress.

6.1.5 The results of the liquefaction analyses indicate seismically-induced settlement up to 5 inches. Differential settlement is expected to be approximately half of the total estimated settlement across the building. The liquefaction analyses presented herein used available SPT blow counts and boring information. Consideration should be given to refining the liquefaction analyses provided herein using cone penetrometer (CPT) test results. The CPT based liquefaction analyses are considered superior to SPT-based calculations and the estimated dynamic settlement estimates could be reduced.

6.1.6 Planned structures and improvements should be designed considering the static and dynamic settlement estimates provided herein. For buildings and improvements that cannot accommodate the estimated settlement, ground improvement and/or deep foundations will be required.

6.1.7 The planned buildings can be supported by shallow or mat foundations on improved ground, or by deep foundations capable of transmitting foundation loads through the alluvium into the Friars Formation.

6.1.8 Proposed below grade improvements, such as underground utilities, should consider the groundwater elevation information contained in this study. Temporary and/or permanent design considerations may be necessary in the event that these improvements are located near or below the water table.

6.1.9 A proposed vehicle crossing and bridge between PA-2 and PA-3 is shown on the plans. The roadway and bridge abutments are expected to be supported on compacted fill placed above saturated younger alluvium. For preliminary design purposes, we have also provided drilled pier parameters for any bridge foundations extending beyond the younger alluvium and into the underlying formational materials.

6.2 Excavation and Soil Characteristics

6.2.1 Excavation of the surficial deposits should be possible with light to moderate effort using conventional heavy-duty equipment. Excavations into the Friars Formation are not anticipated. Hard concretionary fragments may be generated from this unit and require special handling.

6.2.2 The soils encountered in the field investigation are considered to be “non-expansive” (expansion index [EI] greater than 20) as defined by 2022 California Building Code (CBC) Section 1803.5.3. Table 6.2 presents soil classifications based on the expansion index. The soil materials observed on site are anticipated to have a “very low” to “medium” expansion potential (expansion index of 90 or less).

TABLE 6.2
EXPANSION CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	ASTM 4829 Expansion Classification	2022 CBC Expansion Classification
0 – 20	Very Low	Non-Expansive
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

6.3 Soluble Sulfate Exposure

6.3.1 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate. Results from the laboratory water-soluble sulfate content testing are presented in Table IV and indicate that the on-site materials at the locations tested possess a "Not Applicable" and "S0" sulfate exposure, or "Moderate" and "S1" sulfate exposure to concrete structures as defined by 2022 CBC Section 1904 and ACI 318. Table 6.3 presents a summary of concrete requirements set forth by 2022 CBC Section 1904 and ACI 318.

TABLE 6.3
REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Exposure Class		Water-Soluble Sulfate (SO_4) Percent by Weight	Cement Type (ASTM C 150)	Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
S0		$SO_4<0.10$	No Type Restriction	n/a	2,500
S1		$0.10\leq SO_4<0.20$	II	0.50	4,000
S2		$0.20\leq SO_4\leq 2.00$	V	0.45	4,500
S3	Option 1	$SO_4>2.00$	V+Pozzolan or Slag	0.45	4,500
	Option 2		V	0.40	5,000

[1] Maximum water to cement ratio limits do not apply to lightweight concrete

6.3.2 The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

6.3.3 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements that could be susceptible to corrosion are planned.

6.4 Mitigation of Liquefaction and Lateral Spreading

6.4.1 Mitigation of liquefiable soils will be necessary for settlement-sensitive structures. Ground improvement techniques, such as stone columns, soil mixing, compaction grouting, etc., should be considered to reduce the estimated settlements to tolerable ranges (typically 1 inch or less for conventional foundations). A design-build contractor specializing in ground improvement, such as Hayward Baker or Condon-Johnson, would review the available soil and geologic information presented herein and provide ground improvement to mitigate the estimated settlements to tolerable limits. Alternatively, planned settlement-sensitive structures could be supported on a deep foundation system.

6.5 Slopes

6.5.1 Generalized slope stability analyses were performed utilizing average drained direct shear strength parameters from the laboratory test results. These analyses indicate that the proposed 2:1 fill slopes, constructed of on-site materials, should have calculated factors of safety of at least 1.5 under static conditions for both deep-seated failure and shallow sloughing conditions to heights of at least 15 feet. Slope stability calculations for both deep-seated and surficial slope stability are presented on Figures 5 and 6, respectively.

6.5.2 The outer 15 feet of fill slopes, measure horizontal to the slope face, should be composed of properly compacted granular "soil" fill to reduce the potential for surface sloughing.

6.5.3 Fill slopes should be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet and should be track-walked at the completion of each slope such that the fill soils are uniformly compacted to at least 90 percent relative compaction to the face of the finished sloped. Alternatively, the fill slope may be over-built at least 3 feet and cut back to yield a properly compacted slope face.

6.5.4 All slopes should be landscaped with drought-tolerant vegetation, having variable root depths and requiring minimal landscape irrigation. In addition, all slopes should be drained and properly maintained to reduce erosion.

6.6 Grading

6.6.1 All grading should be performed in accordance with the attached *Recommended Grading Specifications* (Appendix E). Where the recommendations of this section conflict with Appendix E, the recommendations of this section take precedence. All earthwork should be observed and all fills tested for proper compaction by Geocon Incorporated.

6.6.2 Earthwork should be observed and compacted fill tested by representatives of Geocon Incorporated.

6.6.3 A pre-construction conference with a City of Santee representative, owner, contractor, civil engineer, and geotechnical engineer should be held at the site prior to the beginning of grading. Special soil handling requirements can be discussed at that time.

6.6.4 Site preparation should begin with the removal of all deleterious material and vegetation. There are areas of very thick brush, vegetation, and large trees in both sites. The depth of removal should be such that material to be used as fill are free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

6.6.5 Potentially compressible soils consisting of artificial fill and the upper portions of alluvium should be removed to approximately 2 feet above the groundwater table, or competent material, and properly compacted. Remedial grading in the driving range and practice area is not considered necessary unless structural improvements are proposed. The actual extent of unsuitable soil removals will be determined in the field during grading by the geotechnical engineer and/or engineering geologist. The estimated thickness of remedial grading is presented on Figure 3.

6.6.6 We understand that an emergency vehicle access road is planned that crosses known cultural resources. As a consequence, remedial grading to remove potentially compressible surficial soils is prohibited. In order to limit potential settlement beneath the roadway, stabilization measures, such as using geogrid reinforcement (such as Tensar TX-5 or equivalent), are recommended at the ground surface. The Project Civil Engineer has created an exhibit that shows the recommended stabilization measures using two rows of geogrid reinforcement.

6.6.7 After removal of unsuitable materials is performed, the site should then be brought to final subgrade elevations with structural fill compacted in layers. In general, soils native to the site are suitable for re-use as fill if free from vegetation, debris and other deleterious material. Layers of fill should be no thicker than will allow for adequate bonding and compaction. All fill, including backfill and scarified ground surfaces, should be compacted to at least 90 percent of maximum dry density at or above optimum moisture content, as determined in accordance with ASTM Test Procedure D1557. Fill materials below optimum moisture content will require additional moisture conditioning prior to placing additional fill.

6.6.8 Proposed off-site improvements to Carlton Oaks Drive consist of removing existing parking stalls and constructing an emergency vehicle access road. The removed stalls will be relocated to the northeast. Minor grading is anticipated once the existing surface improvements are removed. We understand a grasscrete pavement section is proposed. The subgrade soil and aggregate base beneath the pavement should be compacted to at least 95 percent of the applicable maximum dry density at slightly over optimum moisture content. In addition, a storm drain easement is shown. Utility trench backfill should be compacted to at least 90 percent of the applicable maximum dry density at slightly over optimum moisture content.

6.6.9 It is our understanding that imported soils will be required, and that this material may be generated during grading operations within other portions of the golf course. Import materials should consist of granular material with “very low” to “low” expansive (Expansion Index of 50 or less) potential. Prior to importing the material, samples from proposed export site should be obtained and subjected to laboratory testing to determine whether the material conforms to the recommended criteria. At least 5 working days should be allowed for laboratory testing of the soil prior to its importation. Import materials should be free of oversize rock and construction debris.

6.6.10 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations in order to maintain safety and maintain the stability of adjacent existing improvements.

6.7 Settlement Considerations

6.7.1 The alluvial deposits were found to be slightly compressible when subjected to increased vertical stress and will require remedial grading where practical. Specialized foundation design (deep foundations) or ground improvement will be necessary to reduce the potential adverse effects of settlement of these deposits. Section 5.6 provides a discussion on static settlement considerations.

6.8 Settlement Monitoring

6.8.1 The proposed structural areas underlain by additional fill and saturated alluvium should be monitored for settlement. In general, surface settlement plates should be installed at several locations within the development footprint and read periodically until primary consolidation has essentially ceased. Survey readings should be performed regularly following placement of the proposed fill. Specific details regarding the location and type of monitoring device as well as monitoring frequency, will be provided once the development plans have been finalized. However, weekly monitoring for approximately 2 to 3 months should be expected.

6.9 Seismic Design Criteria

6.9.1 Table 6.9.1 summarizes site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used the computer program *U.S. Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented herein are for the risk-targeted maximum considered earthquake (MCE_R). Sites designated as Site Class D, E and F may require additional analyses if requested by the project structural engineer and client.

TABLE 6.9.1
2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	0.783g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.287g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.187	Table 1613.2.3(1)
Site Coefficient, F_V	2.026	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	0.929g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.582g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.62g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.388g	Section 1613.2.4 (Eqn 16-39)

***Note:** Using the code-based values presented in this table, in lieu of a performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed by the project structural engineer. Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis should be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed.

6.9.2 Table 6.9.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 6.9.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.336g	Figure 22-9
Site Coefficient, F_{PGA}	1.264	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.425g	Section 11.8.3 (Eqn 11.8-1)

6.9.3 Conformance to the criteria in Tables 6.9.1 and 6.9.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.9.4 The project structural engineer and architect should evaluate the appropriate Risk Category and Seismic Design Category for the planned structures. The values presented herein assume a Risk Category of I and resulting in a Seismic Design Category D. Table 6.9.3 presents a summary of the risk categories in accordance with ASCE 7-16.

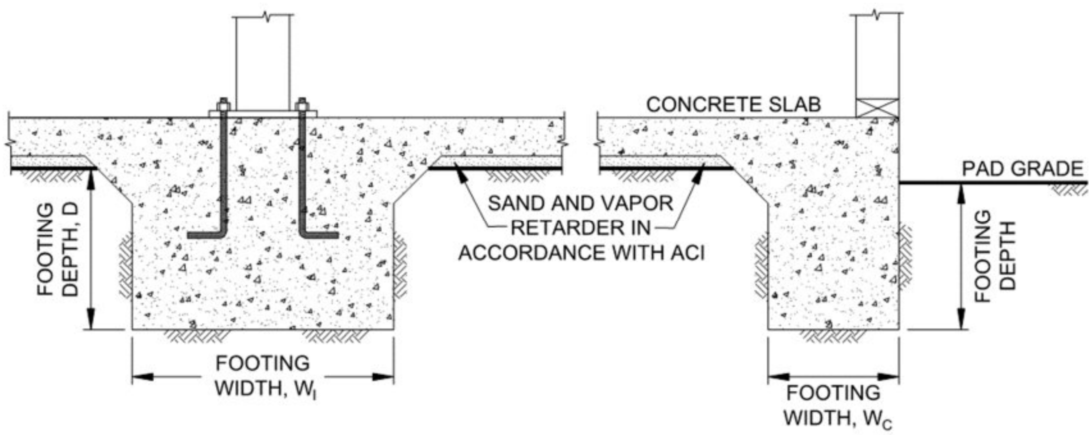
TABLE 6.9.3
ASCE 7-16 RISK CATEGORIES

Risk Category	Building Use	Examples
I	Low risk to Human Life at Failure	Barn, Storage Shelter
II	Nominal Risk to Human Life at Failure (Buildings Not Designated as I, III or IV)	Residential, Commercial and Industrial Buildings
III	Substantial Risk to Human Life at Failure	Theaters, Lecture Halls, Dining Halls, Schools, Prisons, Small Healthcare Facilities, Infrastructure Plants, Storage for Explosives/Toxins
IV	Essential Facilities	Hazardous Material Facilities, Hospitals, Fire and Rescue, Emergency Shelters, Police Stations, Power Stations, Aviation Control Facilities, National Defense, Water Storage

6.10 Foundations

6.10.1 If ground improvements are performed, such as stone columns, deep soil mixing, or compaction grouting, the proposed buildings can be supported on conventional shallow footings founded entirely in compacted fill. Foundations for the structures should consist of continuous strip footings and/or isolated spread footings. Continuous footings should be at least 12 inches wide and extend at least 24 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 2 feet and should extend at least 24 inches below lowest adjacent pad grade. Steel reinforcement for continuous footings should consist of at least four No. 5 steel reinforcing bars placed horizontally in the footings, two near the top and two near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer.

6.10.2 The foundations should be embedded in accordance with the recommendations herein and the Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent grade for both interior and exterior footings.



Wall/Column Footing Dimension Detail

6.10.3 The proposed structures can be supported on a shallow foundation system founded in the compacted fill. Table 6.10 provides a summary of the foundation design recommendations.

TABLE 6.10
SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Allowable Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf
Estimated Total Static Settlement	1 Inch
Estimated Differential Static Settlement	½ Inch in 40 Feet

6.10.4 The bearing capacity values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

6.10.5 The minimum reinforcement recommended above is based on soil characteristics only (Expansion Index of 90 or less) and is not intended to replace reinforcement required for structural considerations.

6.10.6 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

6.10.7 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

6.10.8 Foundation excavations should be observed by the geotechnical engineer (a representative of Geocon Incorporated) prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated and have been extended to appropriate bearing strata. If unanticipated soil conditions are encountered, foundation modifications may be required.

6.11 Concrete Slabs-on-Grade

6.11.1 Concrete slabs-on-grade for the structure should be at least 5 inches thick and reinforced with No. 3 steel reinforcing bars at 18 inches on center in both horizontal directions.

6.11.2 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements and installed in a manner that prevents puncture. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity controlled environment.

6.11.3 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

6.11.4 The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting vehicle, equipment and storage loads.

6.12 Post-Tensioned Foundation

6.12.1 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structures. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC10.5 as required by the 2022 California Building Code Section 1808.6.2. Although this procedure was developed for expansive soil conditions, we understand it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented on Table 6.12. The parameters presented in Table 6.12 are based on the guidelines presented in the PTI, DC10.5 design manual.

TABLE 6.12
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

Post-Tensioning Institute (PTI) DC10.5 Design Parameters	
Thornthwaite Index	-20
Equilibrium Suction	3.9
Edge Lift Moisture Variation Distance, e_M (Feet)	4.9
Edge Lift, y_M (Inches)	1.58
Center Lift Moisture Variation Distance, e_M (Feet)	9.0
Center Lift, y_M (Inches)	0.66

6.12.2 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

6.12.3 If the structural engineer proposes a post-tensioned foundation design method other than 2022 CBC (PTI, DC 10.5):

- The deflection criteria presented in Table 6.12 are still applicable.
- Interior stiffener beams should be used.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 18 inches. The embedment depths should be measured from the lowest adjacent pad grade.

6.12.4 Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift from tensioning, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

6.12.5 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless designed by the structural engineer.

6.12.6 Isolated footings outside of the slab area, if present, should have the minimum embedment depth and width recommended for conventional foundations. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams in both directions. In addition, consideration should be given to connecting patio slabs, which exceed 5 feet in width, to the building foundation to reduce the potential for future separation to occur.

6.12.7 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in any such concrete placement.

6.12.8 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations due to expansive soil. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.12.9 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute when establishing crack-control spacing. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

6.12.10 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

6.12.11 We should observe the foundation excavations prior to the placement of reinforcing steel to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. If unexpected soil conditions are encountered, foundation modifications may be required.

6.13 Driven Piles

6.13.1 If ground improvements to mitigate liquefaction are not performed or if building loads require footings that are too large, deep foundations are recommended. Because groundwater was encountered at about 4 to 6 feet below existing grades, drilled piers would require water- or slurry-displacement methods of construction. Therefore, driven pre-cast concrete piles (PCCP) or auger cast piles will likely be the most economical. Recommendations for the other types of piles such as driven steel H piles can be provided if required. For ease of evaluation, we have provided recommendations for PCCP driven piles. Slight modifications will be required if auger cast piles are used.

6.13.2 Piles will develop support by both friction and by end bearing in the Friars Formation at depth. Capacities are commonly limited to 70 tons for 12-inch square piles, 100 tons for 14-inch square piles, and 150 tons for 18-inch square piles due to structural and drivability concerns. Geocon will provide the tip elevations and lateral capacities based on the design service loads and the specific subsurface conditions below the structures. If different loadings or pile types are used, Geocon Incorporated should be contacted. Allowable uplift capacities can be taken as one half the allowable compressive capacities. The ultimate uplift capacity is equal to the allowable compressive capacity. The uplift capacity may also be limited by structural considerations and should be checked by the structural engineer. In general, pile lengths can be estimated for the loadings above at a penetration of 10 to 20 feet into the Friars Formation. Downdrag loads due to liquefaction will also be incorporated into the design.

6.13.3 If pile spacing is at least four times the maximum dimension of the pile, no reduction in axial capacity for group effects is considered necessary.

6.13.4 Pile settlement is expected to be on the order of ¼-inch for PCCP piles. Settlement should be essentially complete shortly after completion of the structure.

6.13.5 Pre-drilling should not be used. The geotechnical engineer (a representative of Geocon Incorporated) should observe pile driving and evaluate each pile on a case-by-case basis or a load test should be performed. It is recommended that a pile hammer that develops a minimum energy of 30,000 foot-pounds per blow be used. Each pile should be evaluated during driving to determine if adequate capacity has been attained. For practical purposes, the final set should equal or exceed that required for recommended allowable capacity based on dynamic equation or wave equation formulas.

6.14 Mat Foundations

6.14.1 As an alternative to deep foundations, the structures may be supported on a mat or raft foundation system. A raft or mat foundation consists of a thick, rigid concrete mat that allows the entire footprint of the structure to carry the structural loads. In addition, this type of foundation can tolerate greater differential settlements associated with liquefaction than conventional foundations. Liquefaction settlements of 0 to 5 inches are possible across the site.

6.14.2 We expect structural loads to impose a uniform bearing pressure of less than 300 pounds per square feet (psf). However, isolated column areas within the mat foundation could have bearing pressures exceeding 300 psf. The anticipated total and differential static settlements of mat imposing the above bearing pressures are estimated to be on the order of 1/2 inch and 1/4 inch, respectively.

6.14.3 The allowable bearing capacity can be taken as 2,000 psf following remedial grading beneath the building area. The modulus of subgrade reaction for design of the mat can be taken as 125 pounds per cubic inch (pci). This modulus should be modified using the conventional equation for mat dimensions.

6.14.4 Foundation excavations should be observed by the geotechnical engineer (a representative of Geocon Incorporated) prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

6.15 Proposed Bridge Foundations

6.15.1 We understand a bridge is proposed from PA-2 (North Site) to PA-3 (Hotel Site). We expect the abutment foundations to consist of isolated spread footings supported on compacted fill. Any bents, if needed, should be supported using drilled piers supported on Friars Formation beneath the younger alluvium.

6.15.2 The bridge abutments may be supported on a shallow foundation system founded in the compacted fill. Continuous footings should be at least 12 inches wide and extend 24 inches below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 2 feet and should also extend 24 inches below lowest adjacent pad grade. In addition, footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

6.15.3 Steel reinforcement for continuous footings should consist of at least four No. 5 steel reinforcing bars placed horizontally in the footings, two near the top and two near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer.

6.15.4 The recommendations herein are based on soil characteristics only (EI of 50 or less) and is not intended to replace reinforcement required for structural considerations.

6.15.5 The recommended allowable bearing capacity for foundations with minimum dimensions described herein and bearing in properly compacted fill is 2,000 pounds per square foot (psf). The values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

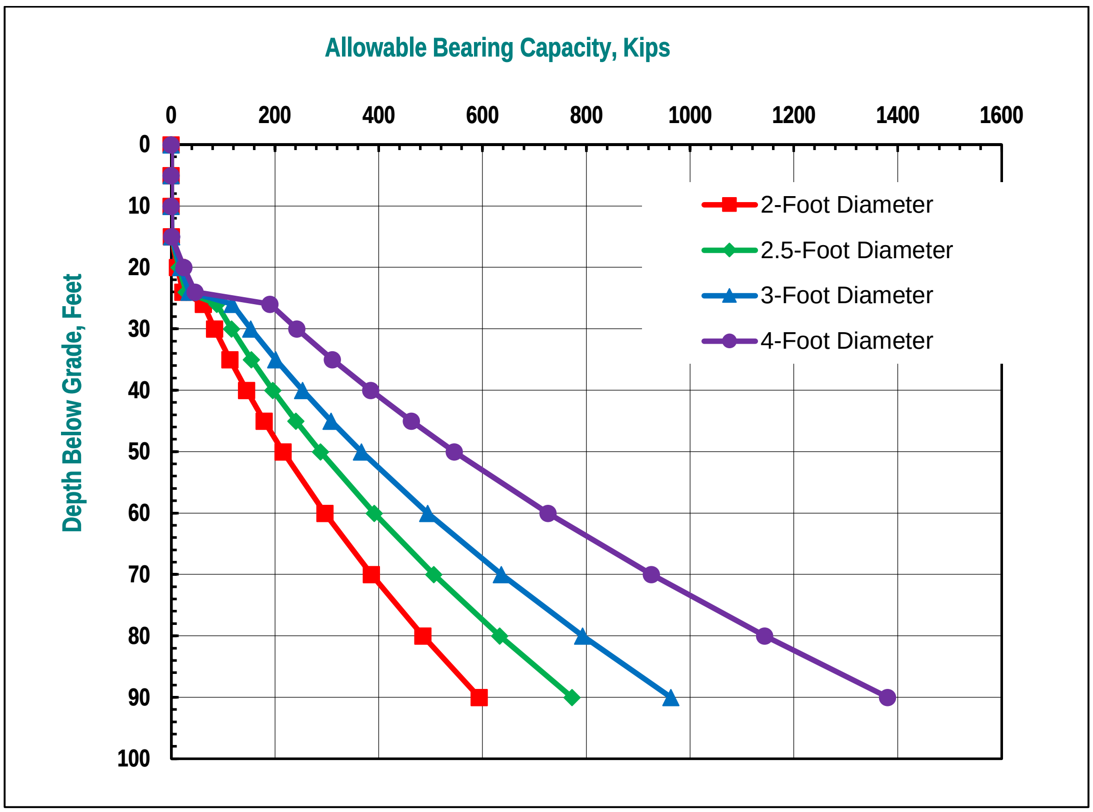
6.15.6 We estimate the total and differential settlements under the imposed allowable loads to be about 1 inch and ½ inch, respectively, based on a 5-foot-square footing. These settlement values are based on the underlying soil being densified as recommended herein.

6.15.7 We should observe the foundation excavations prior to the placement of reinforcing steel to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. If unexpected soil conditions are encountered, foundation modifications may be required.

6.16 Drilled Pier Recommendations

6.16.1 If needed, drilled piers should be used for foundation support for any bridge bents. In addition, drilled piers may be used to support the proposed buildings to mitigate liquefaction. The foundation recommendations herein assume that the piers will extend through the younger alluvium and into the Friars Formation. Groundwater and wet drilling techniques should be expected. The piers should be embedded at least 5 feet within the formational materials. For design purposes, a surficial soil thickness of 25 feet was used to compute the allowable bearing capacities shown below. Once actual foundation types and locations are determined, revised allowable capacities may be provided based on actual site conditions. Additional field exploration may be needed to refine the recommendations presented herein.

6.16.2 Piers can be designed to develop support by end bearing within the formational materials and skin friction within the formational materials and younger alluvium. The allowable bearing capacity can be determined by the chart presented below. These allowable values possess a factor of safety of 2 and 3 for skin friction and end bearing, respectively. Downdrag loads due to compressible younger alluvium has been incorporated in the design.



Allowable Bearing Capacity Chart

6.16.3 Piers can be designed to develop support by end bearing within the formational materials and skin friction within the formational materials and younger alluvium using the design parameters presented in Table 6.16.

TABLE 6.16
SUMMARY OF DRILLED PIER RECOMMENDATIONS

Parameter	Value
Minimum Pile Diameter	2 Feet
Minimum Pile Spacing	3 Times Pile Diameter
Minimum Foundation Embedment Depth	10 Feet
	5 Feet in Formational Materials
Allowable Bearing Capacity	Per Chart
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	½ Inch in 40 Feet

6.16.4 The design length of the drilled piers should be determined by the designer based on the elevation of the pile cap or grade beam and the elevation of the top of the formational materials obtained from the Geologic Map and Geologic Cross-Sections presented herein. It is difficult to evaluate the exact length of the proposed drilled piers due to the variable thickness of the younger alluvium; therefore, some variation should be expected during drilling operations.

6.16.5 If pier spacing is at least three times the maximum dimension of the pier, no reduction in axial capacity for group effects is considered necessary. If piles are spaced between 2 and 3 pile diameters (center to center), the single pile axial capacity should be reduced by 25 percent. Geocon Incorporated should be contacted to provide single-pile capacity if piers are spaced closer than 2 diameters.

6.16.6 The allowable downward capacity may be increased by one-third when considering transient wind or seismic loads.

6.16.7 The younger alluvial materials may contain gravel and cobble zones and could experience caving; therefore, the drilling contractor should expect wet and caving drilling conditions during excavations for the piers. Because a significant portion of the piers capacity will be developed by end bearing, the bottom of the borehole should be cleaned of loose cuttings prior to the placement of steel and concrete. Experience indicates that backspinning the auger does not remove loose material and a flat cleanout plate is necessary. We expect localized seepage may be encountered during the drilling operations and casing may be required to maintain the integrity of the pier excavation, particularly if seepage or sidewall instability is encountered. Concrete should be placed within the excavation as soon as possible after the auger/cleanout plate is withdrawn to reduce the potential for discontinuities or caving.

6.16.8 Pile settlement of production piers is expected to be on the order of ½ inch if the piers are loaded to their allowable capacities. Geocon should provide updated settlement estimates once the foundation plans are available. Settlements should be essentially complete shortly after completion of the building superstructure.

6.17 Concrete Flatwork

6.17.1 Exterior slabs not subjected to vehicular traffic should be a minimum of 4 inches thick and reinforced with 6 x 6-6/6 welded wire mesh or No. 3 steel reinforcing bars at 18 inches on center both directions. The reinforcement should be placed in the middle of the slab. Proper positioning is critical to future performance of the slab. The contractor should take extra measures to provide proper placement. Prior to construction of slabs, the upper 12 inches of subgrade soils should be moisture conditioned at or slightly above optimum moisture content and compacted to at least 90 percent of the laboratory maximum dry density per ASTM 1557.

6.17.2 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. A 4-inch-thick slab should have a maximum joint spacing of 10 feet. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented above prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be checked prior to placing concrete.

6.17.3 Even with the incorporation of the recommendations within this report, the exterior concrete flatwork has a likelihood of experiencing some settlement due to potentially compressible and liquefiable soil beneath grade; therefore, the welded wire mesh should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

6.17.4 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. Periotic maintenance such as slab replacement and/or grinding of elevated slab margins may be necessary due to the highly expansive soils. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

6.18 Retaining Walls

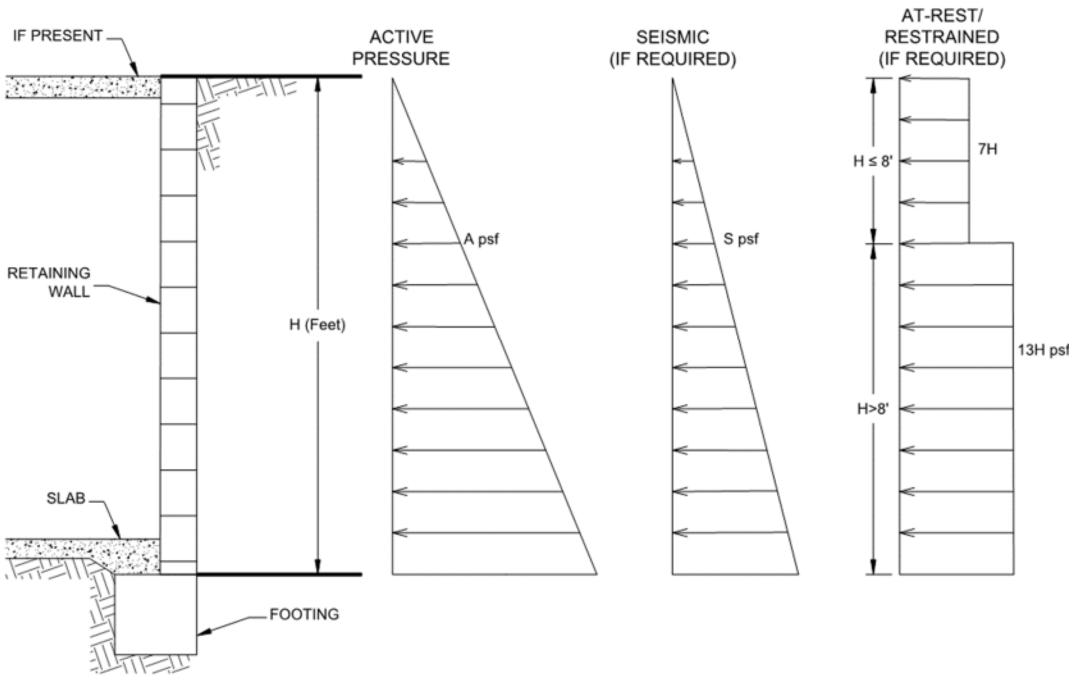
6.18.1 Retaining walls should be designed using the values presented in Table 6.18.1. Soil with an expansion index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

TABLE 6.18.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	35 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	50 pcf
Seismic Pressure, S	19H psf
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure (8+ Feet High)	12H psf
Expected Expansion Index for the Subject Property	EI $\leq$ 50

H equals the height of the retaining portion of the wall

6.18.2 The project retaining walls should be designed as shown in the Retaining Wall Loading Diagram.



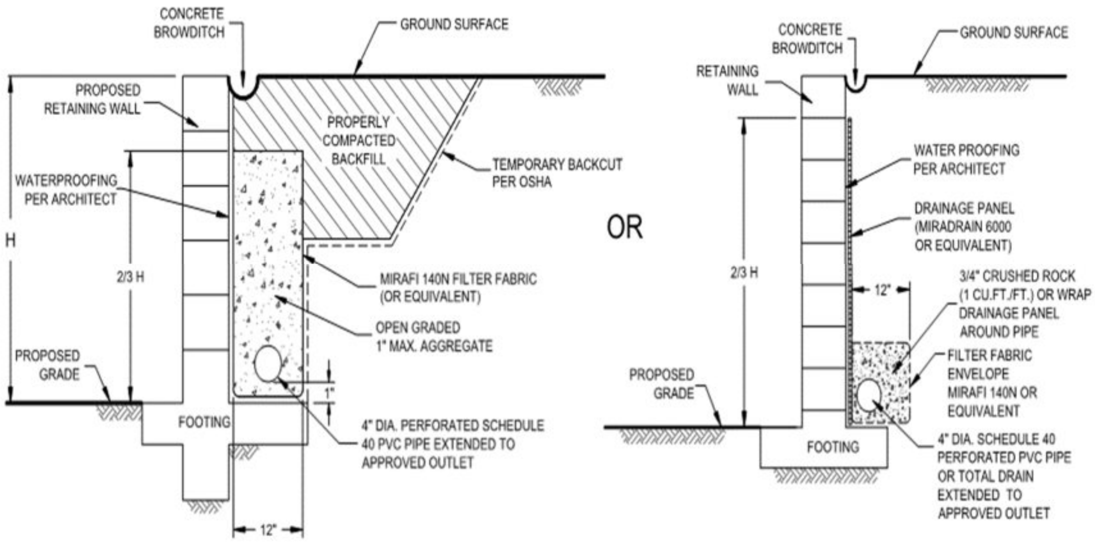
Retaining Wall Loading Diagram

6.18.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall) at the top of the wall. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure should be applied to the wall. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added.

6.18.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2022 CBC or Section 11.6 of ASCE 7-16. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2022 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

6.18.5 Retaining walls should be designed to ensure stability against overturning sliding, and excessive foundation pressure. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

6.18.6 Drainage openings through the base of the wall (weep holes) should not be used where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. The retaining wall should be properly drained as shown in the Typical Retaining Wall Drainage Detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

6.18.7 The retaining walls may be designed using either the active and restrained (at-rest) loading condition or the active and seismic loading condition as suggested by the structural engineer. Typically, it appears the design of the restrained condition for retaining wall loading may be adequate for the seismic design of the retaining walls. However, the active earth pressure combined with the seismic design load should be reviewed and also considered in the design of the retaining walls.

6.18.8 In general, wall foundations should be designed in accordance with Table 6.18.2. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

TABLE 6.18.2
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 inches
Minimum Steel Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	2,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf
Estimated Total Static Settlement*	1 inch
Estimated Differential Static Settlement*	½ inch in 40 Feet

6.18.9 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. In the event that other types of walls are planned, Geocon Incorporated should be consulted for additional recommendations.

6.18.10 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

6.18.11 Soil contemplated for use as retaining wall backfill, including import materials, should be identified in the field prior to backfill. At that time, Geocon Incorporated should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, on-site soil to be used as backfill may or may not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the on-site soil for use as wall backfill if standard wall designs will be used.

6.19 Lateral Loading

6.19.1 Table 6.19 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

TABLE 6.19
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	300 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

6.19.2 The passive and frictional resistant loads can be combined for design purposes. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

6.20 Preliminary Pavement Recommendations

6.20.1 We calculated the preliminary flexible pavement sections in general conformance with the *Caltrans Method of Flexible Pavement Design* (Highway Design Manual, Section 608.4) using an estimated Traffic Index (TI) of 4.5, 5.0, 6.0, and 7.0 for parking stalls, driveways, medium truck traffic areas, and heavy truck traffic areas, respectively. The project civil engineer and owner should review the pavement designations to determine appropriate locations for pavement thickness. The final pavement sections for the driveways and parking areas should be based on the R-Value of the subgrade soil encountered at final subgrade elevation. We have assumed an R-Value of 25 and 78 for the subgrade soil and base materials, respectively, for the purposes of this preliminary analysis. Table 6.20.1 presents the preliminary flexible pavement sections.

TABLE 6.20.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking stalls for automobiles and light-duty vehicles	4.5	25	3	5
Driveways for automobiles and light-duty vehicles	5.0	25	3	6.5
Medium truck traffic areas	6.0	25	3.5	8.5
Driveways for heavy truck traffic	7.0	25	4	11

6.20.2 Prior to placing base materials, the upper 12 inches of the subgrade soil should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D 1557. Similarly, the base material should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of at least 95 percent of the laboratory Hveem density in accordance with ASTM D 2726.

6.20.3 Base materials should conform to Section 26-1.02A of the *Standard Specifications for The State of California Department of Transportation (Caltrans)* with a ¾-inch maximum size aggregate. The asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Greenbook).*

6.20.4 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway entrance aprons and trash bin loading/storage areas. The concrete pad for trash truck areas should be large enough such that the truck wheels will be positioned on the concrete during loading. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 6.20.2.

TABLE 6.20.2
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of subgrade reaction, k	100 pci
Modulus of rupture for concrete, M_R	500 psi
Traffic Category, TC	A and C
Average daily truck traffic, ADTT	10 and 100

6.20.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 6.20.3.

TABLE 6.20.3
RIGID PAVEMENT RECOMMENDATIONS

Location	Portland Cement Concrete (inches)
Automobile Parking Areas (TC=A)	5.5
Heavy Truck and Fire Lane Areas (TC=C)	7.0

6.20.6 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,000 psi (pounds per square inch).

6.20.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., a 7-inch-thick slab would have a 9-inch-thick edge). Reinforcing steel should consist of No. 3 steel bars spaced at 24 inches on center, both directions.

6.20.8 To control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab. Crack-control joints should not exceed 30 times the slab thickness with a maximum spacing of 12.5 feet and 15 feet for the 5.5- and 7-inch-thick slabs, respectively, and should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be determined by the referenced ACI report.

6.20.9 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, cross-gutters, or sidewalk so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, consideration may be given to structurally connecting the concrete flatwork to the curbs to help reduce the potential for offsets between the curbs and the flatwork if differential settlement occurs.

6.21 Site Drainage and Moisture Protection

6.21.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2022 CBC 1804.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

6.21.2 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

6.21.3 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes can be used. In addition, where landscaping is planned adjacent to the pavement, construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material should be considered.

6.22 Slope Maintenance

6.22.1 Slopes that are steeper than 3:1 (horizontal:vertical) may, under conditions that are both difficult to prevent and predict, be susceptible to near-surface (surficial) slope instability. The instability is typically limited to the outer 3 feet of a portion of the slope and usually does not directly impact the improvements on the pad areas above or below the slope. The occurrence of surficial instability is more prevalent on fill slopes and is generally preceded by a period of heavy rainfall, excessive irrigation, or the migration of subsurface seepage. The disturbance and/or loosening of the surficial soils, as might result from root growth, soil expansion, or excavation for irrigation lines and slope planting, may also be a significant contributing factor to surficial instability. It is therefore recommended that, to the maximum extent practical: (a) disturbed/loosened surficial soils be either removed or properly recompacted, (b) irrigation systems be periodically inspected and maintained to eliminate leaks and excessive irrigation, and (c) surface drains on and adjacent to slopes be periodically maintained to preclude ponding or erosion. Although the incorporation of the above recommendations should reduce the potential for surficial slope instability, it will not eliminate the possibility and, therefore, it may be necessary to rebuild or repair a portion of the project's slopes in the future.

6.23 Grading and Foundation Plan Review

6.23.1 Geocon Incorporated should review the final grading plans and foundation plans for the project prior to final design submittal to evaluate whether additional analyses and/or recommendations are required.

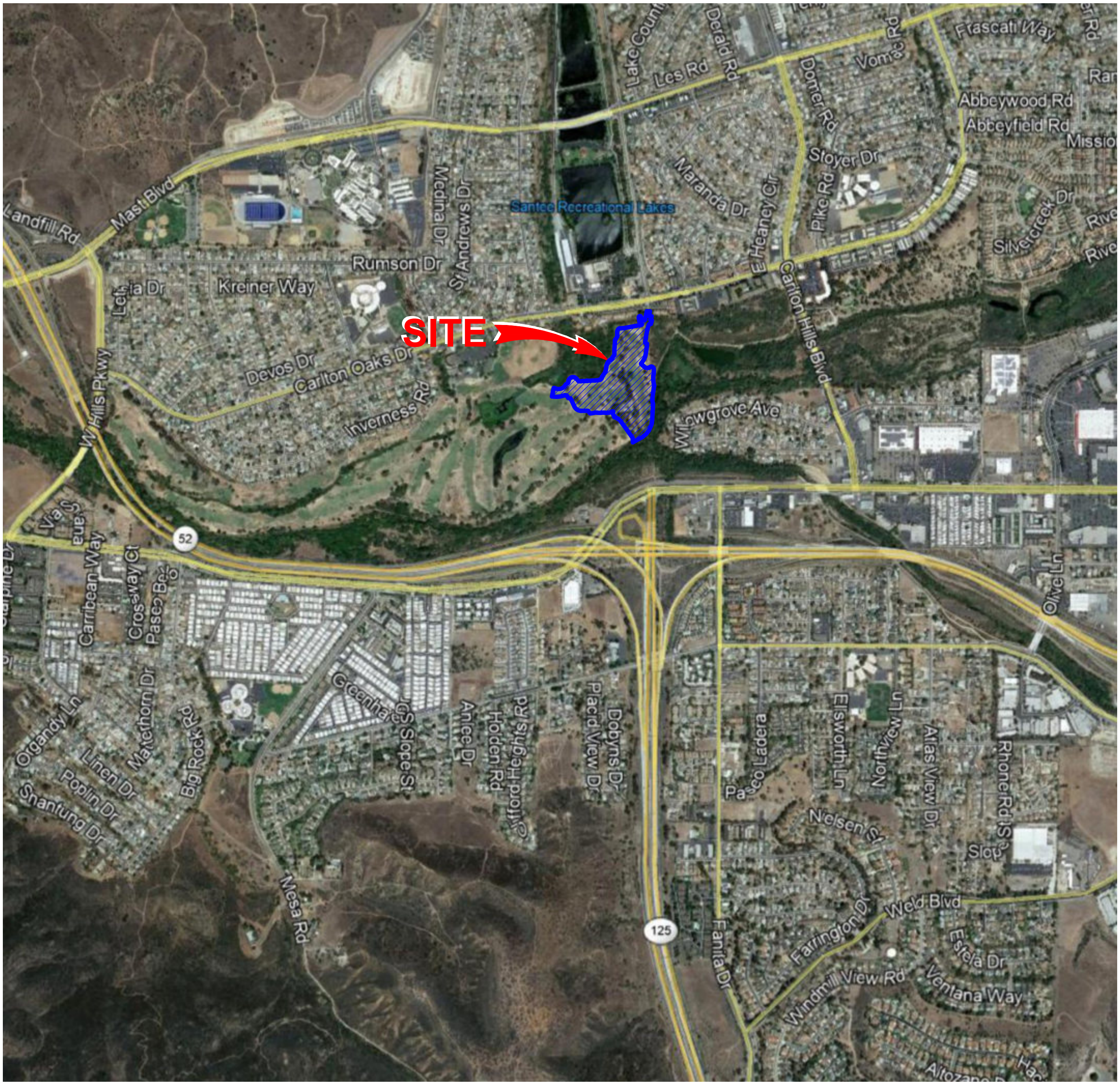
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

3. This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.



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NO SCALE

VICINITY MAP

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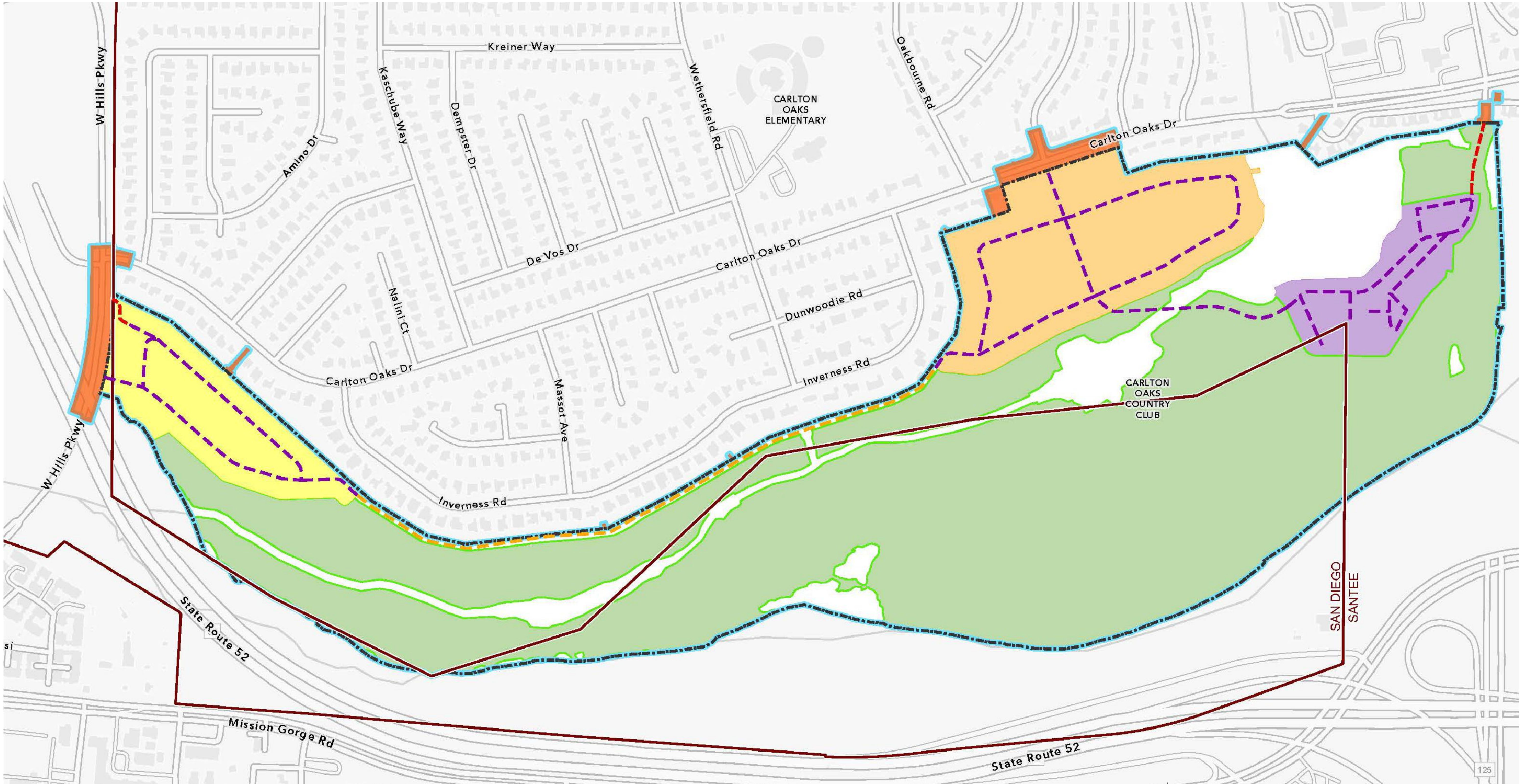
CARLTON OAKS GOLF COURSE
COUNTRY CLUB AND RESORT SITE (PA-3)
SANTEE, CALIFORNIA

RM / AML		DSK/GTYPD	DATE 06 - 11 - 2024	PROJECT NO. G2298 - 32 - 01	FIG. 1

Plotted:06/10/2024 2:38PM | By:ALVIN LADRILLONO | File Location:Y:\PROJECTS\G2298-32-01 (Golf Course - Hotel)\DETAILS\G2298-32-01 Vic Map.dwg



- Project Site
- Off-Site Improvements
- CEQA Study Area
- City of Santee/San Diego Municipal Boundary
- Residential North
- Residential West
- Golf Course Redesign
- Hotel and Clubhouse
- Emergency Vehicle Access Only
- Private Utility Maintenance Road
- Residential / Resort Circulation Roadway

GRAPHIC SCALE
0' 100' 200' 300' 400' 600' 800'
SCALE 1"=200' (On 36x24)

SITE PLAN AND OFF-SITE IMPROVEMENTS

CARLTON OAKS GOLF COURSE
COUNTRY CLUB AND RESORT SITE (PA-3)
SANTEE, CALIFORNIA

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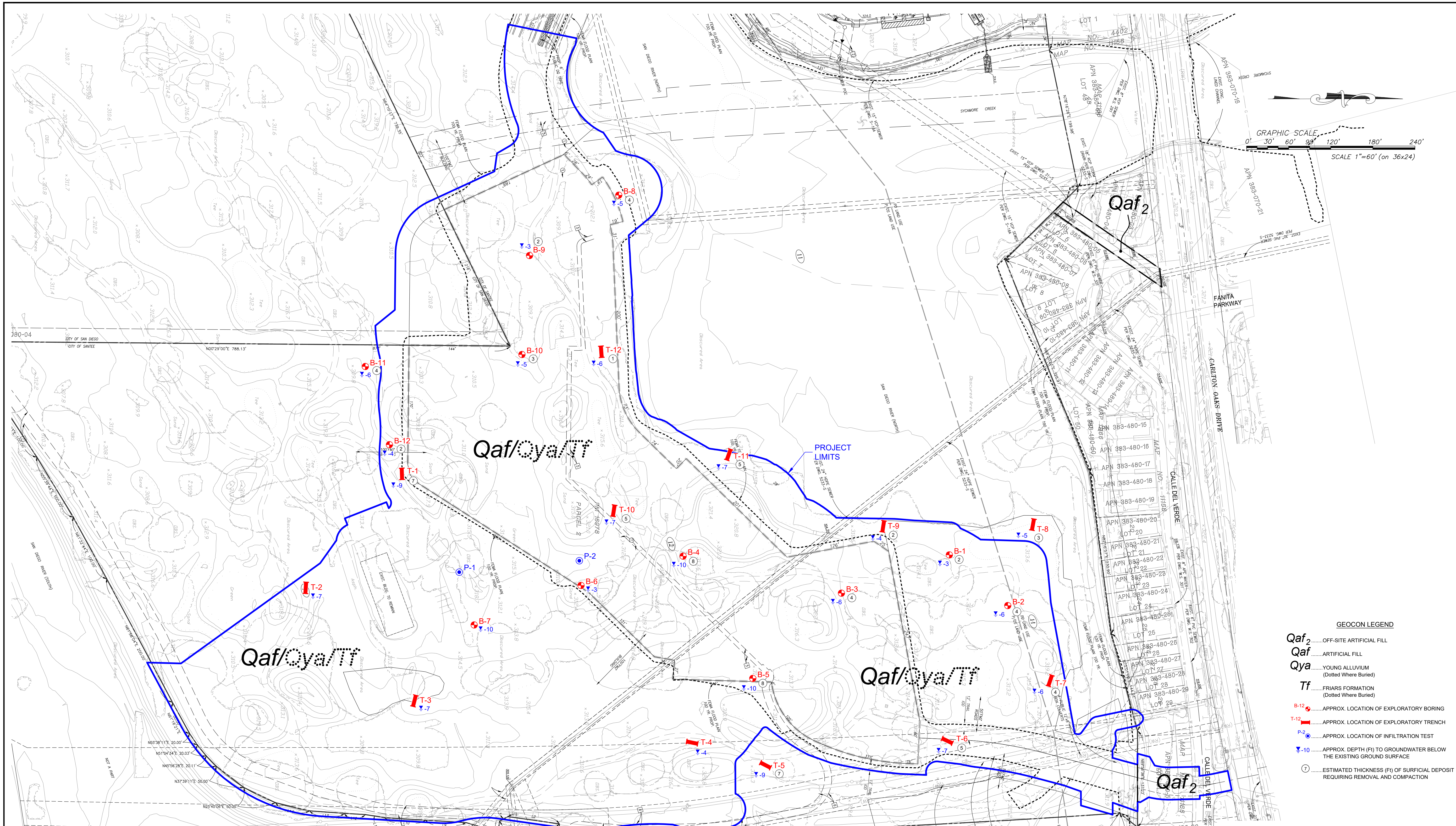
SCALE 1" = 200'	DATE 06 - 11 - 2024
PROJECT NO. G2298 - 32 - 01	FIGURE 2
SHEET 1 OF 1	



GEOLOGIC MAP

CARLTON OAKS GOLF COURSE
COUNTRY CLUB AND RESORT SITE (PA-3)
SANTEE, CALIFORNIA

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SCALE 1" = 60'	DATE 06 - 11 - 2024
PROJECT NO. G2298 - 32 - 01	FIGURE 3
SHEET 1 OF 1	

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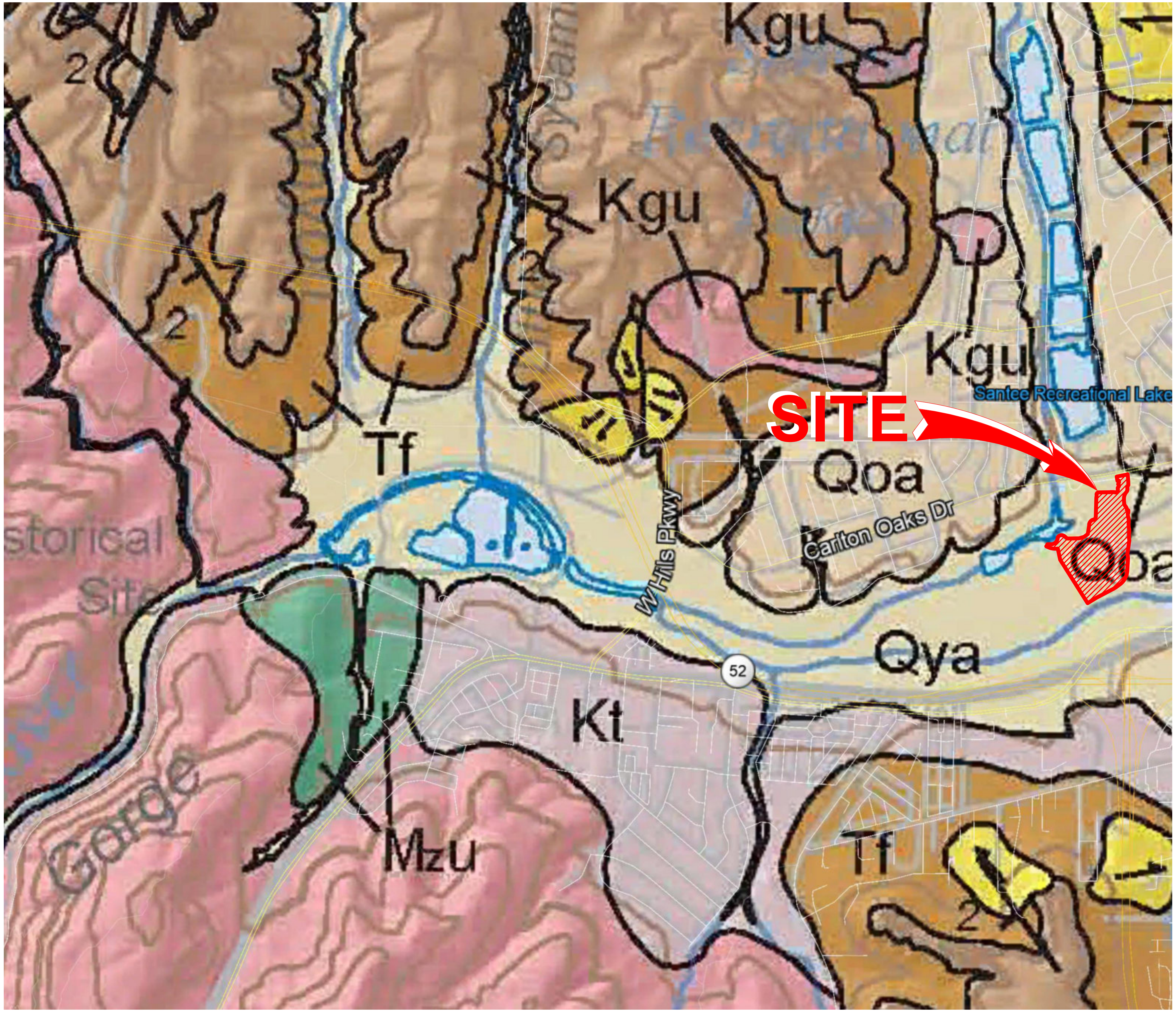

SOURCE: GEOLOGIC MAP OF THE SAN DIEGO 30'X60' QUADRANGLE, DATED 2008 AND EL CAJON 30'X60' QUADRANGLE, DATED 2004, CALIFORNIA PREPARED BY THE U.S. GEOLOGICAL SURVEY

CARLTON OAKS GOLF COURSE
COUNTRY CLUB AND RESORT SITE (PA-3)
SANTEE, CALIFORNIA

DESCRIPTION OF MAP UNITS

Qls	Landslide deposits, undivided (Holocene and Pleistocene)
Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Qoa	Old alluvial flood-plain deposits, undivided (late to middle Pleistocene)
Tst	Stadium Conglomerate (middle Eocene)
Tf	Friars Formation (middle Eocene)
Kgu	Granodiorite and tonalite, undivided (mid-Cretaceous)
Kt	Tonalite, undivided (mid-Cretaceous)
Mzu	Metamorphosed and unmetamorphosed volcanic and sedimentary rocks, undivided (Mesozoic)

REGIONAL GEOLOGIC MAP

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PROJECT NO. G2298 - 32 - 01
FIGURE 4
DATE 06 - 11 - 2024

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ASSUMED CONDITIONS :

SLOPE HEIGHT	H	= 15 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
TOTAL UNIT WEIGHT OF SOIL	γ_t	= 125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ	= 32 degrees
APPARENT COHESION	C	= 200 pounds per square foot
NO SEEPAGE FORCES		

ANALYSIS :

$\lambda_{c\phi}$	=	$\frac{\gamma_t H \tan\phi}{C}$	EQUATION (3-3), REFERENCE 1
FS	=	$\frac{N_{cf} C}{\gamma_t H}$	EQUATION (3-2), REFERENCE 1
$\lambda_{c\phi}$	=	5.9	CALCULATED USING EQ. (3-3)
Ncf	=	25	DETERMINED USING FIGURE 10, REFERENCE 2
FS	=	2.7	FACTOR OF SAFETY CALCULATED USING EQ. (3-2)

REFERENCES :

1......Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics, Series No. 46, 1954

2......Janbu, N., Discussion of J.M. Bell, Dimensionless Parameters for Homogeneous Earth Slopes, Journal of Soil Mechanics and Foundation Design, No. SM6, November 1967.

SLOPE STABILITY ANALYSIS - FILL SLOPES

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COUNTRY CLUB AND RESORT SITE (PA-3)
SANTEE, CALIFORNIA

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ASSUMED CONDITIONS :

SLOPE HEIGHT	H =	Infinite
DEPTH OF SATURATION	Z =	3 feet
SLOPE INCLINATION	2 : 1	(Horizontal : Vertical)
SLOPE ANGLE	i =	26.6 degrees
UNIT WEIGHT OF WATER	γ_w =	62.4 pounds per cubic foot
TOTAL UNIT WEIGHT OF SOIL	γ_t =	125 pounds per cubic foot
ANGLE OF INTERNAL FRICTION	ϕ =	32 degrees
APPARENT COHESION	C =	200 pounds per square foot

SLOPE SATURATED TO VERTICAL DEPTH Z BELOW SLOPE FACE

SEEPAGE FORCES PARALLEL TO SLOPE FACE

ANALYSIS :

$$FS = \frac{C + (\gamma_t - \gamma_w)\ Z \cos^2 i \tan \phi}{\gamma_t\ Z \sin i \cos i} = 1.95$$

REFERENCES :

1......Haefeli, R. *The Stability of Slopes Acted Upon by Parallel Seepage*, Proc. Second International Conference, SMFE, Rotterdam, 1948, 1, 57-62

2......Skempton, A. W., and F.A. Delory, *Stability of Natural Slopes in London Clay*, Proc. Fourth International Conference, SMFE, London, 1957, 2, 378-81

SURFICIAL SLOPE STABILITY ANALYSIS

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APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed in two phases in July 2018 and June 2019, and consisted of a visual site reconnaissance, drilling twelve small-diameter borings (Boring Nos. B-1 through B-12) and excavating twelve exploratory trenches (Trench Nos. T-1 through T-12). In addition, two infiltration tests (Infiltration Test Nos. P-1 and P-2) were performed within a proposed storm water management area at the location provided by SB&O, Inc. The approximate locations of the exploratory borings, test pits and infiltration tests are shown on the *Geologic Map*, Figure 3.

The exploratory borings were performed by Baja Exploration using a CME-95 drill rig to a maximum depth of 32.5 feet below existing grade. Samples were collected at 5-foot intervals using a 3-inch diameter California split-spoon sampler (CAL) or a 2-inch-diameter Standard Penetration Test (SPT) sampler, driven 12 and 18 inches, respectively into the undisturbed soil mass. An automatic trip hammer weighing 140 pounds and dropped 30 inches was used to drive the samplers.

The CAL sampler was equipped with 1-inch by 2⅜-inch, brass sampler rings to facilitate removal and testing. The soil collected within the SPT sampler was placed in plastic bags for testing. Blow counts were recorded for every 6 inches the sampler was driven and shown on the boring logs in terms of blows per foot. The values indicated on the boring logs are the sum of the last 12 inches of the sampler if driven 18 inches. These values are not to be taken as N-values, adjustments have not been applied. Logs of the borings depicting the soil and geologic conditions encountered and the depth at which samples were obtained are presented on Figures A-1 through A-12.

The exploratory trenches were excavated with a John Deere 310G backhoe, using a 24-inch-wide bucket. The soils encountered were visually examined, classified and logged. Logs of the trenches depicting the soil and geologic conditions encountered are presented on Figures A-13 through A-24.

The soils encountered in the excavations were visually classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual Manual Procedure D 2488).

BORING B 1

ELEV. (MSL.) **311'** DATE COMPLETED **07-17-2018**

EQUIPMENT **CME 95** BY: **D. GITHENS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, wet, dark gray, Silty SAND			
2				SM	**YOUNG ALLUVIUM (Qya)**			
	B1-1		▼		Medium dense, dark gray, Silty, fine to coarse SAND; low cohesion	29	109.1	19.9
4								
	B1-2				-Medium dense, dark gray Silty SAND	23	119.0	14.6
6								
8								
10	B1-3			SW	Medium dense, saturated, dark gray, fine to coarse SAND with little silt and gravel	19		
12								
14								
	B1-4			ML	**FRIARS FORMATION (Tf)** Very stiff, pale green, wet, fine Sandy SILTSTONE	18		
16								
18								
20	B1-5				-Stiff	35	84.7	35.7

BORING TERMINATED AT 21 FEET
Groundwater encountered at 3 feet
Backfilled with 7ft³ of bentonite chips

Figure A-1,
Log of Boring B 1, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 2

ELEV. (MSL.) 314' DATE COMPLETED 07-17-2018

EQUIPMENT CME 95 BY: D. GITHENS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ARTIFICIAL FILL (Qaf)** Loose, brown, moist, Silty, fine to medium SAND			
3	B2-1			SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, grey and reddish-brown, Silty, fine to coarse SAND	15	108.0	18.9
5	B2-2		▼			11	103.4	19.7
10	B2-3			SP	Medium dense, saturated, gray, poorly graded, medium grained SAND	13		
15	B2-4			ML	**FRIARS FORMATION (Tf)** Very stiff, saturated, pale green, fine Sandy SILTSTONE	27		
20	B2-5				-Stiff	23	82.6	39.5
25	B2-6				-Very stiff	22		

BORING TERMINATED AT 26 FEET
Groundwater encountered at 6 feet
Backfilled with 9ft³ of bentonite chips

Figure A-2,
Log of Boring B 2, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. G2298-32-01

BORING B 3

ELEV. (MSL.) 313' DATE COMPLETED 07-17-2018

EQUIPMENT CME 95 BY: D. GITHENS

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		SM	**ARTIFICIAL FILL (Qaf)** Loose, gray-brown, Silty, fine to medium SAND -Layer of mulch			
2						
	B3-1	SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, gray brown, Silty, fine to coarse SAND	14	108.6	16.9
4						
	B3-2			23	117.0	14.4
6			-Becomes saturated			
8						
10	B3-3		-Increase in silt content	15		
12						
14						
	B3-4	SW	Medium dense, saturated, gray-brown, fine to coarse SAND with gravel	20		
16						
18						
20	B3-5			30	129.4	11.6
22						
24						
	B3-6	ML	**FRIARS FORMATION (Tf)** Very stiff, saturated, pale green, fine, Sandy SILTSTONE	21		
26						
28						
30	B3-7			41	84.7	36.1

TERMINATED AT 31 FEET
Groundwater encountered at 6 feet
Backfilled with 11ft³ of bentonite chips

Figure A-3,
Log of Boring B 3, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL
... STANDARD PENETRATION TEST
... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE
... CHUNK SAMPLE
... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 4

ELEV. (MSL.) **314'** DATE COMPLETED **07-17-2018**

EQUIPMENT **CME 95** BY: **D. GITHENS**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ARTIFICIAL FILL (Qaf)** Loose, damp, brown/gray, Silty, fine to medium SAND			
2								
	B4-1					10	105.7	9.9
4								
	B4-2					12	103.5	9.1
6				SM	**YOUNG ALLUVIUM (Qya)** Loose, moist, gray brown, Silty, fine to medium SAND			
8								
10	B4-3		▼		-Becomes wet	5		
12								
14				SM	Loose, wet, gray, Silty, fine to coarse SAND			
16	B4-4					11		
18								
20	B4-5				-Medium dense	39	112.1	19.4
22								
24								
26	B4-6				-Some 2-inch size gravel	21		
28								
30	B4-7				-Very dense, no recovery	67/6"		
32	B4-8				-No recovery, cobble layer	66		

BORING TERMINATED AT 32.5 FEET
Groundwater encountered at 10 feet
Backfilled with 11ft³ of bentonite chips

Figure A-4,
Log of Boring B 4, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 5

ELEV. (MSL.) 315' DATE COMPLETED 07-17-2018

EQUIPMENT CME 95 BY: D. GITHENS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ARTIFICIAL FILL (Qaf)** Loose, moist, dark brown, Silty, fine to medium SAND			
3	B5-1					11	103.4	8.7
3				SM	**YOUNG ALLUVIUM (Qya)** Loose, moist, dark brown, Silty, fine- to medium-grained SAND			
5.5	B5-2					13	101.0	7.3
10	B5-3		▼	SP	-Becomes wet, poorly graded, medium grained SAND	2		
15	B5-4					10		
20	B5-5				-Medium dense	28	113.5	13.0
25	B5-6					13		
30	B5-7			ML	**FRIARS FORMATION (Tf)** Hard, pale green, saturated, fine, Sandy SILTSTONE	42		

TERMINATED BORING AT 31.5 FEET
Groundwater encountered at 10 feet
Backfilled with 11ft³ of bentonite chips

Figure A-5,
Log of Boring B 5, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 6

ELEV. (MSL.) **314'** DATE COMPLETED **07-18-2018**

EQUIPMENT **CME 95** BY: **D. GITHENS**

DEPTH IN FEET	SAMPLE NO.	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			SM	**ARTIFICIAL FILL (Qaf)** Loose, damp, brown, Silty, fine to medium SAND			
			SM	**YOUNG ALLUVIUM (Qya)** Loose, wet, tan brown to grayish Silty, fine to coarse SAND			
3	B6-1	▼			8	104.2	21.2
5.5	B6-2			-Becomes dark gray	7	105.4	20.4
10.5	B6-3				10		
15.5	B6-4			-With gravel	9		
20.5	B6-5				19	108.4	20.7
				-Medium dense			
21			ML	**FRIARS FORMATION (Tf)** Stiff, damp, pale green, fine Sandy SILTSTONE (Contact observed in sample)			
25.5	B6-6			-Becomes hard	76		
30.5	B6-7				50/6"	115.2	15.5

BORING TERMINATED AT 31 FEET
Groundwater encountered at 3 feet
Backfilled with 11ft³ of bentonite chips

Figure A-6,
Log of Boring B 6, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. G2298-32-01

BORING B 7

ELEV. (MSL.) 314' DATE COMPLETED 07-18-2018

EQUIPMENT CME 95 BY: D. GITHENS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ARTIFICIAL FILL (Qaf)** Loose, damp, brown, Silty, fine to medium SAND			
3	B7-1					28	107.6	18.5
3				SW	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, tan brown, fine to coarse SAND			
5.5	B7-2					18	104.8	4.8
10	B7-3		▼	SM	Very loose, gray, wet, Silty, fine to coarse SAND	2		
14				ML	Stiff, dark gray, fine, Sandy SILT			
15.5	B7-4					14		
18				SM	Medium dense, wet, dark gray, Silty, fine to coarse SAND			
20.5	B7-5					20	110.5	21.7
25	B7-6			ML	**FRIARS FORMATION (Tf)** Very stiff, pale green, saturated, fine, Sandy SILTSTONE	17		
30.5	B7-7					49	107.0	15.6

BORING TERMINATED AT 31 FEET
Groundwater encountered at 10 feet
Backfilled with 11ft³ of bentonite chips

Figure A-7,
Log of Boring B 7, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 8

ELEV. (MSL.) 317' DATE COMPLETED 06-10-2019

EQUIPMENT CME 95 BY: J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, moist, brown, Silty, fine to medium SAND			
2	B8-1							
4				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, wet, grayish brown, Silty, fine to medium SAND with clay -Groundwater encountered at 5 feet			
6	B8-2		▼			22	126.6	7.3
8								
10	B8-3			SM	Very loose, wet, dark grayish brown, Silty, fine to very coarse SAND with gravel	2		
12								
14								
16	B8-4				-Becomes medium dense with higher percentage gravel	20	120.4	14.2
18								
20	B8-5					13		
22								
24								
26	B8-6			ML	**FRIARS FORMATION (Tf)** Very stiff, wet, pale green, Sandy SILTSTONE	59	111.5	19.4
28								
30	B8-7					28		

BORING TERMINATED AT 31.5 FEET
Groundwater encountered at 5 feet
Backfilled with 11ft³ bentonite grout

Figure A-8,
Log of Boring B 8, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. G2298-32-01

BORING B 9

ELEV. (MSL.) 310' DATE COMPLETED 06-10-2019

EQUIPMENT CME 95 BY: J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B9-1			SM	**ARTIFICIAL FILL (Qaf)** Medium dense, moist, brown, Silty, fine to medium SAND with grass at surface			
2			▼	SM	**YOUNG ALLUVIUM (Qya)** Loose, wet, grayish brown, Silty, fine to coarse SAND -Groundwater encountered at 3 feet			
6	B9-2					5		
10	B9-3				-Becomes medium dense, wet, dark gray, silty, fine to very coarse sand	26	117.7	18.2
16	B9-4					16		
20	B9-5					43	121.9	13.7
20				ML	**FRIARS FORMATION (Tf)** Very stiff, wet, pale green, Sandy SILTSTONE			
26	B9-6					25		

BORING TERMINATED AT 26.5 FEET
Groundwater encountered at 3 feet
Backfilled with 9ft³ bentonite grout

Figure A-9,
Log of Boring B 9, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. G2298-32-01

BORING B 10

ELEV. (MSL.) 310' DATE COMPLETED 06-10-2019

EQUIPMENT CME 95 BY: J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SM	**ARTIFICIAL FILL (Qaf)** Loose, moist, brown, Silty, fine to medium SAND with grass at surface			
1				SM	**YOUNG ALLUVIUM (Qya)** Very loose, wet, dark gray, Silty, fine to very coarse SAND			
5	B10-1		▼		-Groundwater encountered at 5 feet	6	98.2	30.2
10	B10-2				-Becomes medium dense with gravel	11		
15	B10-3					26	125.3	13.1
20	B10-4				-Becomes very dense	56		
25	B10-5			SM	**FRIARS FORMATION (Tf)** Dense, wet, grayish green, Silty, fine SANDSTONE	49	108.0	20.9
30	B10-6					36		

BORING TERMINATED AT 31.5 FEET
Groundwater encountered at 5 feet
Backfilled with 11ft³ bentonite grout

Figure A-10,
Log of Boring B 10, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 11

ELEV. (MSL.) 310' DATE COMPLETED 06-10-2019

EQUIPMENT CME 95 BY: J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		SM	**ARTIFICIAL FILL (Qaf)** Loose, moist, brown, Silty, fine to medium SAND with grass at surface			
2	B11-1	SP	**YOUNG ALLUVIUM (Qya)** Very loose, wet, dark gray, poorly graded, medium grained SAND			
6	B11-2		-Groundwater encountered at 6 feet	2		
10	B11-3		-No recovery	4		
14		SM	Loose, wet, dark greenish black, Silty, fine to medium SAND			
16	B11-4			12	97.1	32.1
20	B11-5		-Becomes medium dense, wet, dark grayish brown, silty, fine to very coarse sand with gravel	16		
24		ML	**FRIARS FORMATION (Tf)** Stiff, wet, pale green, Sandy SILTSTONE			
26	B11-6			30	106.2	21.4
30			-Hard			

BORING TERMINATED AT 31.5 FEET
Groundwater encountered at 6 feet
Backfilled with 11ft³ bentonite grout

Figure A-11,
Log of Boring B 11, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS
... SAMPLING UNSUCCESSFUL ... STANDARD PENETRATION TEST ... DRIVE SAMPLE (UNDISTURBED)
... DISTURBED OR BAG SAMPLE ... CHUNK SAMPLE ... WATER TABLE OR SEEPAGE

BORING B 12

ELEV. (MSL.) **311'** DATE COMPLETED **06-10-2019**

EQUIPMENT **CME 95** BY: **J. PAGNILLO**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B12-1			SM	**ARTIFICIAL FILL (Qaf)** Loose, moist, brown, Silty, fine to medium SAND with grass at surface			
2				SM	**YOUNG ALLUVIUM (Qya)** Loose, wet, dark gray, Silty, fine to very coarse SAND			
4			▼		-Groundwater encountered at 4 feet			
6								
8								
10	B12-2					10		
12								
14								
16								
18								
20								
22								
24								
	B12-3				-Becomes medium dense; no recovery (Gravel)	44		
26	B12-4			ML	**FRIARS FORMATION (Tf)** Hard, wet, pale green, Sandy SILTSTONE	69		

BORING TERMINATED AT 27.5 FEET
Groundwater encountered at 4 feet
Backfilled with 9ft³ bentonite grout

Figure A-12,
Log of Boring B 12, Page 1 of 1

G2298-32-01.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJE(T NO. G2203-29-03

TRENCH T 1

ELE) . YMSL.C 315' DATE (OMPLETED 07-18-2018

E: UIPMENT 310G JD BACKHOE (W/ 24" BUCKET) B1% J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	MATERIAL DES(RIPTION	PENETRATION RESISTAN(E YBLOV S/FT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT YKC
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, moist, brown, Silty, fine to medium SAND			
2	T1-12							
4								
6								
				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist to wet, dark gray Silty, fine to coarse SAND			
8								
			▼		TRENCH TERMINATED AT 9 FEET Groundwater encountered at 9 feet			

Figure A-13,
Log of Trench T 1, Page 1 of 1

G2203-29-03.GPJ

SAMPLE S1MBOLS

- ... SAMPLING UNSU((ESSFUL
- ... STANDARD PENETRATION TEST
- ... DRI) E SAMPLE YUNDISTURBEDC
- ... DISTURBED OR BAG SAMPLE
- ... (HUN/ SAMPLE
- ... V ATER TABLE OR SEEPAGE

NOTE% THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

PROJE(T NO. G22Q89-29 03

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	TRENCH T 2 ELE) . YMSL.C 313' DATE (OMPLETED 07-18-2018 E: UIPMENT 310G JD BACKHOE (W/ 24" BUCKET) B1 % J. PAGNILLO	PENETRATION RESISTAN(E YBLOV S/FT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT YK C
					MATERIAL DES(RIPTION			
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, moist, brown to dark brown, Silty, fine to medium SAND with abundant roots			
2								
				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist to wet, dark gray, Silty, fine to medium SAND; cohesionless, side walls caving			
4	T2-1							
6								
			▼		TRENCH TERMINATED AT 7 FEET Groundwater at 7 feet			

Figure A-14,
Log of Trench T 2, Page 1 of 1

G22Q89-29 03.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE%THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

PROJE(T NO. G22Q39-29Q03

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	TRENCH T 3 ELE) . YMSL.C 314' DATE (OMPLETED 07-18-2018 E: UIPMENT 310G JD BACKHOE (W/ 24" BUCKET) B1% J. PAGNILLO	PENETRATION RESISTAN(E YBLOV SWFT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT Y%C
					MATERIAL DES(RIPTION			
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, moist, light brown, Silty, fine to medium SAND; roots			
	T3-1							
2								
4								
6				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, wet, dark gray, Silty, fine to coarse SAND			
			▼		TRENCH TERMINATED AT 7 FEET Groundwater at 7 feet			

Figure A-15,
Log of Trench T 3, Page 1 of 1

G22Q39-29Q03.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE% THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

PROJE(T NO. G22Q89-29 03

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	TRENCH T 4 ELE) . YMSL.C 312' DATE (OMPLETED 07-18-2018 E: UIPMENT 310G JD BACKHOE (W/ 24" BUCKET) B1% J. PAGNILLO	PENETRATION RESISTAN(E YBLOV SWT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT YK C
					MATERIAL DES(RIPTION			
0				SM	**ARTIFICIAL FILL (Qaf)** Loose, moist, dark brown, Silty, fine to medium SAND; roots			
2	T4-1			SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist to wet, brownish gray, Silty, fine to coarse SAND; cohesionless, caving			
4			▼		TRENCH TERMINATED AT 4 FEET Groundwater encountered at 4 feet			

Figure A-16,
Log of Trench T 4, Page 1 of 1

G22Q89-29 03.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE% THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

PROJE(T NO. G22Q89-2903

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	TRENCH T 5 ELE) . YMSL.C 316' DATE (OMPLETED 07-18-2018 E: UIPMENT 310G JD BACKHOE (W/ 24" BUCKET) B1% J. PAGNILLO	PENETRATION RESISTAN(E YBLOV S/FT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT Y%C
					MATERIAL DES(RIPTION			
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, dry to damp, light grayish brown, Silty, fine to medium SAND; roots			
2								
4								
6				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, damp to moist, grayish brown, Silty, fine to medium SAND; cohesionless, caving			
8				SW	-Becomes dark gray, fine to coarse sand below 8 feet			
			▼		TRENCH TERMINATED AT 9 FEET Groundwater encountered at 9 feet			

Figure A-17,
Log of Trench T 5, Page 1 of 1

G22Q89-2903.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE% THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

PROJE(T NO. G22Q89-29 03

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	TRENCH T 6 ELEV. (MSL.) 315' DATE COMPLETED 07-18-2018 EQUIPMENT 310G JD BACKHOE (W/ 24" BUCKET) BY: J. PAGNILLO	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, moist, brown, Silty, fine to medium SAND; roots			
2								
4				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, brownish gray, Silty, fine to medium SAND; cohesionless, caving			
6				SW	-Becomes dark gray, fine to coarse sand below 6 feet			
			▼		TRENCH TERMINATED AT 7 FEET Groundwater at 7 feet			

Figure A-18,
Log of Trench T 6, Page 1 of 1

G22Q89-29 03.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	**TRENCH T 7** ELE) . YMSL.C **315'** DATE (OMPLETED **07-18-2018** E: UIPMENT **310G JD BACKHOE (W/ 24" BUCKET)** B1% **J. PAGNILLO**	PENETRATION RESISTAN(E YBLOV S/FT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT Y%C
					MATERIAL DES(RIPTION			
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, moist, light grayish brown, Silty, fine to medium SAND; roots			
2				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, light brown, Silty, fine to medium SAND			
4	T7-1			SW	-Becomes fine to coarse sand below 4 feet; cohesionless, caving			
6			▼		TRENCH TERMINATED AT 6 FEET Groundwater at 6 feet			

Figure A-19,
Log of Trench T 7, Page 1 of 1

G22C89-2903.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE% THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

PROJE(T NO. G22C89-29-03

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	TRENCH T 8 ELE) . YMSL.C 313' DATE (OMPLETED 07-18-2018 E: UIPMENT 310G JD BACKHOE (W/ 24" BUCKET) B1% J. PAGNILLO	PENETRATION RESISTAN(E YBLOV S/FT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT YKC
					MATERIAL DES(RIPTION			
0				SM	**ARTIFICIAL FILL (Qaf)** Loose, moist, brown, Silty, fine to medium SAND; roots			
				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, grayish brown, Silty, fine to medium SAND; roots			
2								
				SW	-Becomes wet, dark gray, fine to coarse sand below 3 feet			
4								
			▼		TRENCH TERMINATED AT 5 FEET Groundwater encountered at 5 feet			

Figure A-20,
Log of Trench T 8, Page 1 of 1

G22C89-29-03.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE% THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	TRENCH T 9 ELE) . YMSL.C 312' DATE (OMPLETED 07-18-2018 E: UIPMENT 310G JD BACKHOE (W/ 24" BUCKET) B1 % J. PAGNILLO	PENETRATION RESISTAN(E YBLOV SWFT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT YK C
					MATERIAL DES(RIPTION			
0				SM	**ARTIFICIAL FILL (Qaf)** Loose, moist, brown, Silty, fine to medium SAND; roots			
				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, grayish brown, Silty, fine to medium SAND; roots			
2								
				SW	-Becomes wet, dark gray, fine to coarse sand below 3 feet			
4			▼		TRENCH TERMINATED AT 4 FEET Groundwater encountered at 4 feet			

Figure A-21,
Log of Trench T 9, Page 1 of 1

G22C89-2903.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE% THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

PROJE(T NO. G22Q89-29ŀ03

TRENCH T 10

ELE) . YMSL.C 314'   DATE (OMPLETED 07-18-2018

E: UIPMENT 310G JD BACKHOE (W/ 24" BUCKET)   B1% J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	MATERIAL DES(RIPTION	PENETRATION RESISTAN(E YBLOV S/FT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT Y%C
0	T10-1			SM	**ARTIFICIAL FILL (Qaf)** Medium-dense, moist, brown, Silty, fine to medium SAND; roots			
2								
4				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, grayish brown, Silty, fine to medium SAND; cohesionless, caving			
6			▼	SW	-Becomes wet, dark gray, fine to coarse sand below 6 feet			
					TRENCH TERMINATED AT 7 FEET Groundwater encountered at 7 feet			

Figure A-22,
Log of Trench T 10, Page 1 of 1

G22Q89-29ŀ03.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE%THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

PROJE(T NO. G2209-29-03

TRENCH T 11

ELE) . YMSL.C 314' DATE (OMPLETED 07-18-2018

E: UIPMENT 310G JD BACKHOE (W/ 24" BUCKET) B1% J. PAGNILLO

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	MATERIAL DES(RIPTION	PENETRATION RESISTAN(E YBLOV S/FT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT Y%C
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, damp, grayish brown, Silty, fine to medium SAND; roots			
2				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, grayish brown, Silty, fine to medium SAND; cohesionless, caving			
	T11-1							
4								
6				SW	-Becomes wet, dark gray, fine to coarse SAND			
			▼		TRENCH TERMINATED AT 7 FEET Groundwater encountered at 7 feet			

Figure A-23,
Log of Trench T 11, Page 1 of 1

G2209-29-03.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE% THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

PROJE(T NO. G2203-29-03

DEPTH IN FEET	SAMPLE NO.	LITHOLOG1	GROUNDV ATER	SOIL (LASS YUS(SC	**TRENCH T 12** ELE) . YMSL.C **313'** DATE (OMPLETED **07-18-2018** E: UIPMENT **310G JD BACKHOE (W/ 24" BUCKET)** B1% **J. PAGNILLO**	PENETRATION RESISTAN(E YBLOV S/FT.C	DR1 DENSIT1 YP.(.F.C	MOISTURE (ONTENT Y%C
					MATERIAL DES(RIPTION			
0				SM	**ARTIFICIAL FILL (Qaf)** Medium dense, damp, grayish brown, Silty, fine to medium SAND			
				SM	**YOUNG ALLUVIUM (Qya)** Medium dense, moist, grayish brown, Silty, fine to medium SAND; cohesionless, caving			
2								
4								
				SW	-Becomes wet, dark gray, fine to coarse SAND			
6			▼					
					TRENCH TERMINATED AT 6 FEET Groundwater encountered at 6 feet			

Figure A-24,
Log of Trench T 12, Page 1 of 1

G2203-29-03.GPJ

SAMPLE S1MBOLS			
	... SAMPLING UNSU((ESSFUL	... STANDARD PENETRATION TEST	... DRI) E SAMPLE YUNDISTURBEDC
	... DISTURBED OR BAG SAMPLE	... (HUN/ SAMPLE	... V ATER TABLE OR SEEPAGE

NOTE% THE LOG OF SUBSURFA(E (ONDITIONS SHOV N HEREON APPLIES ONL1 AT THE SPE(IFI(BORING OR TREN(H LO(ATION AND AT THE DATE INDI(ATED. IT IS NOT V ARRANTED TO BE REPRESENTATI) E OF SUBSURFA(E (ONDITIONS AT OTHER LO(ATIONS AND TIMES.

GEOCON

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for in-place dry density and moisture content, maximum dry density and optimum moisture content, expansion index, shear strength, soluble sulfate content, consolidation and gradation characteristics. The results of our laboratory tests are summarized on Tables B-I through B-IV and Figures B-1 through B-13. The results of the in-place dry density and moisture content tests are presented on the boring logs.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
T1-1	Dark brown Silty, fine to coarse SAND with trace gravel	120.3	12.6
T3-1	Yellowish brown Silty, fine to coarse SAND, with trace gravel	125.4	9.9
T7-1	Yellowish brown Silty, fine to medium SAND	117.2	13.2
B11-1	Dark gray, Silty, fine to coarse SAND	124.1	11.9

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index
	Before Test	After Test		
T1-1	10.1	18.7	107.5	0
T3-1	8.7	15.6	113.1	0
T7-1	10.5	17.4	106.2	0
B11-1	8.8	17.1	112.9	1

TABLE B-III
SUMMARY OF LABORATORY DIRECT SHEAR TEST RESULTS

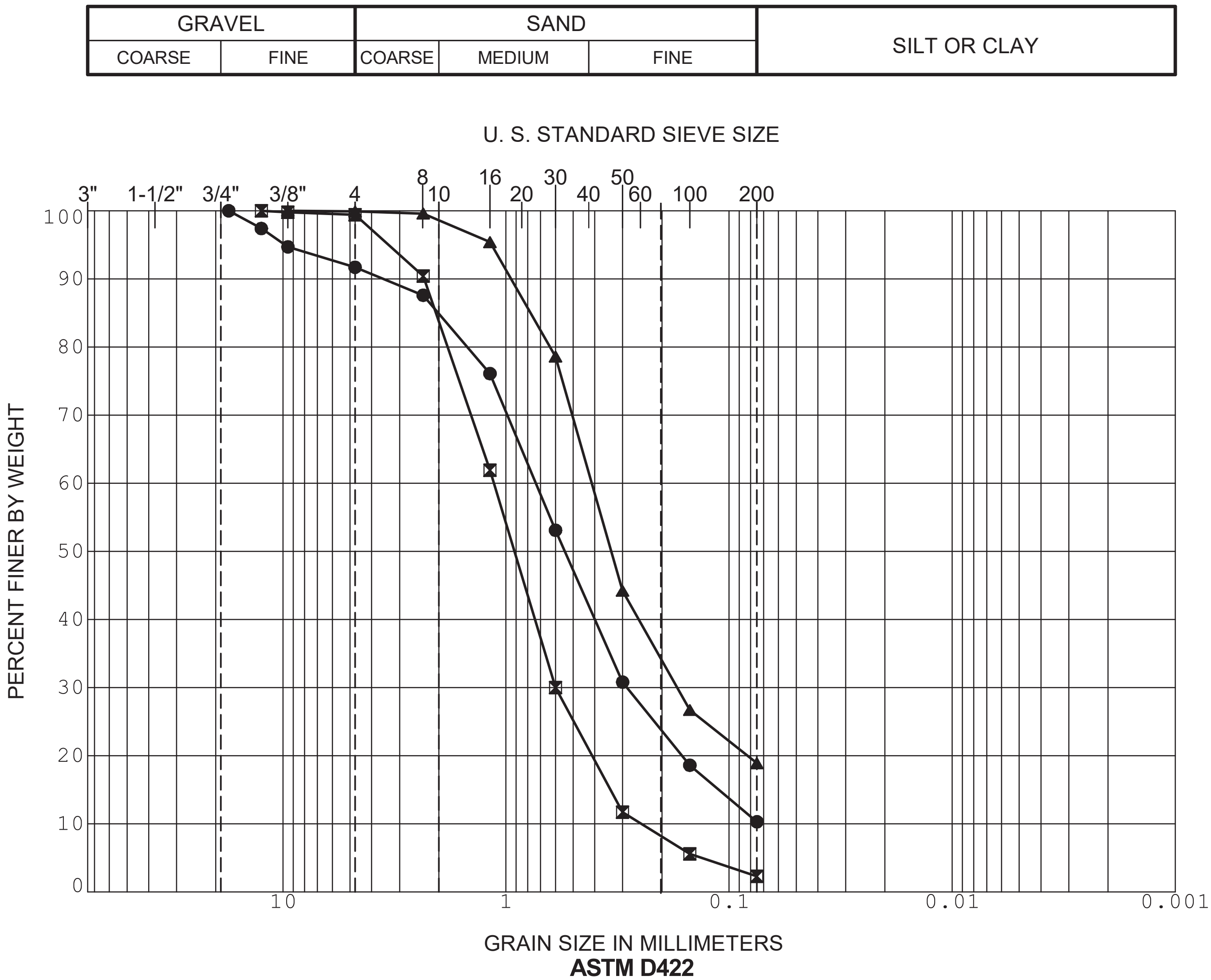
Sample No.*	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
T1-1*	108.8	19.2	640	24
T3-1*	113.5	14.7	355	32
T7-1*	105.2	17.8	290	32
B11-1*	116.9	6.5	165	44

*Samples remolded to approximately 90 percent of maximum dry density at near optimum moisture content.

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS

Sample No.	Water-Soluble Sulfate Content (%)	Exposure
T1-1	0.152	**Moderate**
T3-1	0.003	Not Applicable
T7-1	0.044	Not Applicable
B11-1	0.076	Not Applicable

PROJECT NO. G2298-32-01



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B1-3	10.0	SW - Fine to coarse SAND with little silt and gravel				
⊠	B2-3	10.0	SP - Medium grained SAND				
▲	B4-3	10.0	SM - Silty, fine to medium SAND				

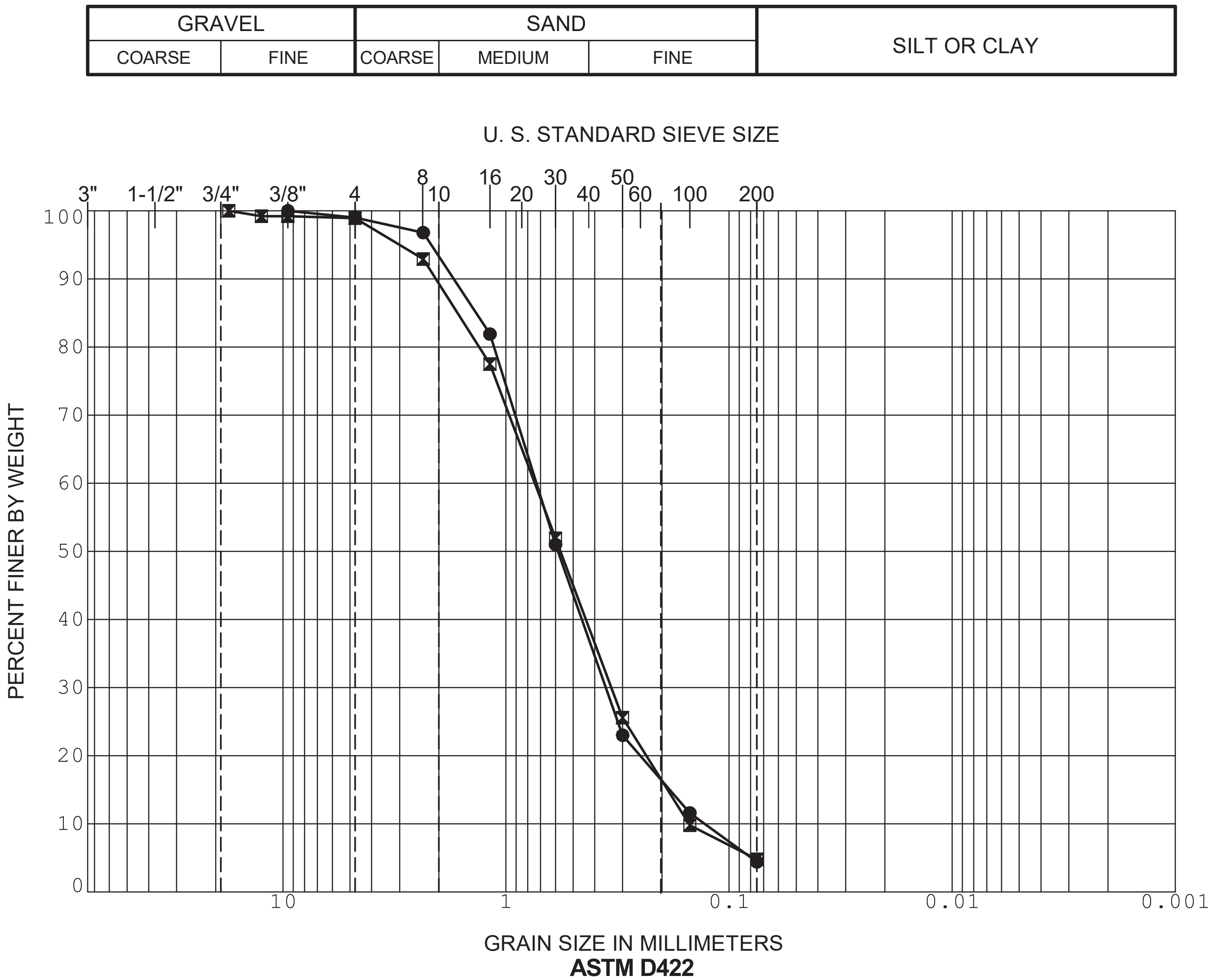
GRADATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

G2298-32-01.GPJ

Figure B-1

GEOCON

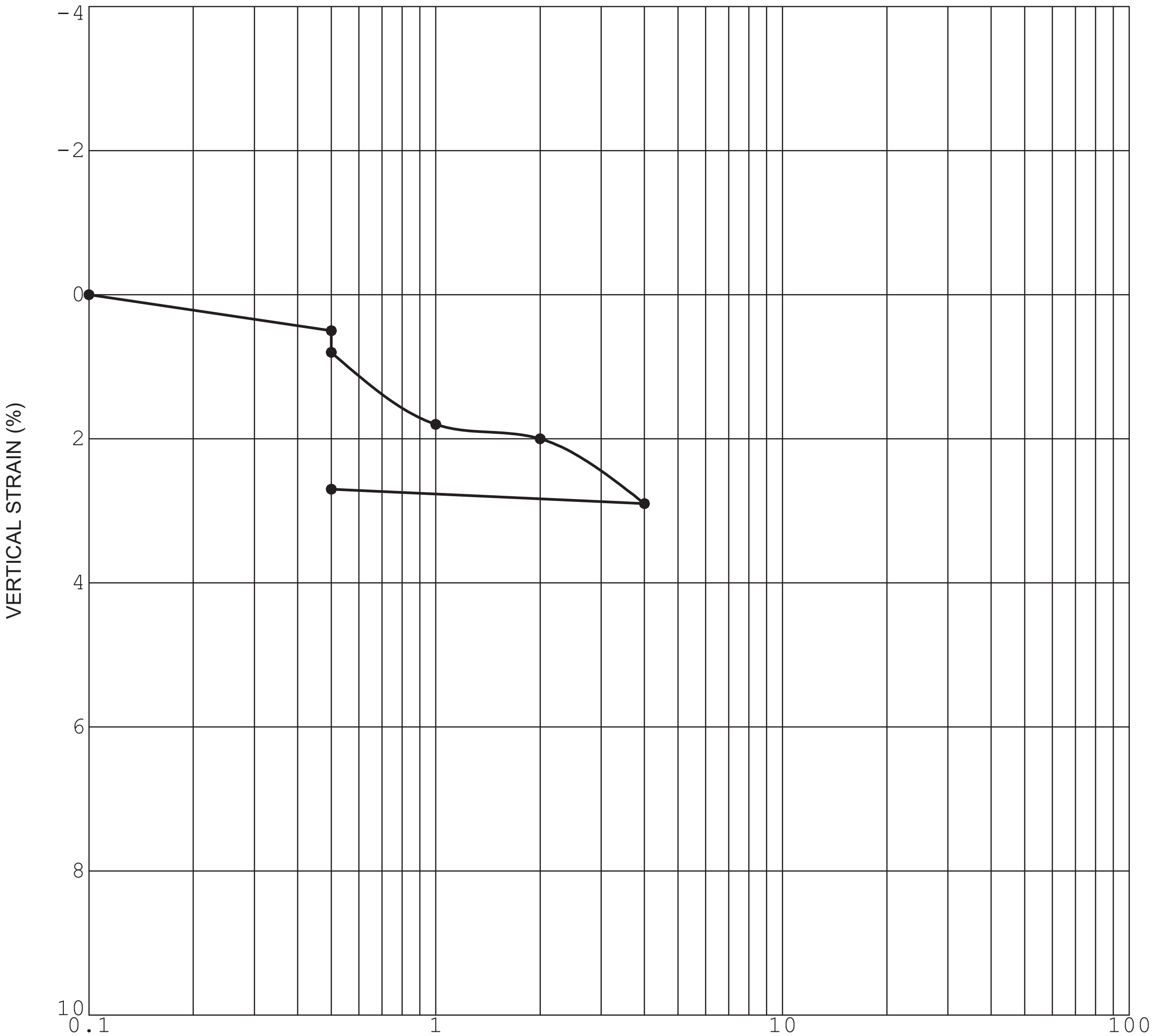


	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B5-4	15.0	SP - Poorly graded, medium grained SAND				
⊠	B11-2	5.0	SP - Poorly graded, medium grained SAND				
▲							

GRADATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

SAMPLE NO. B1-2



ASTM D2435

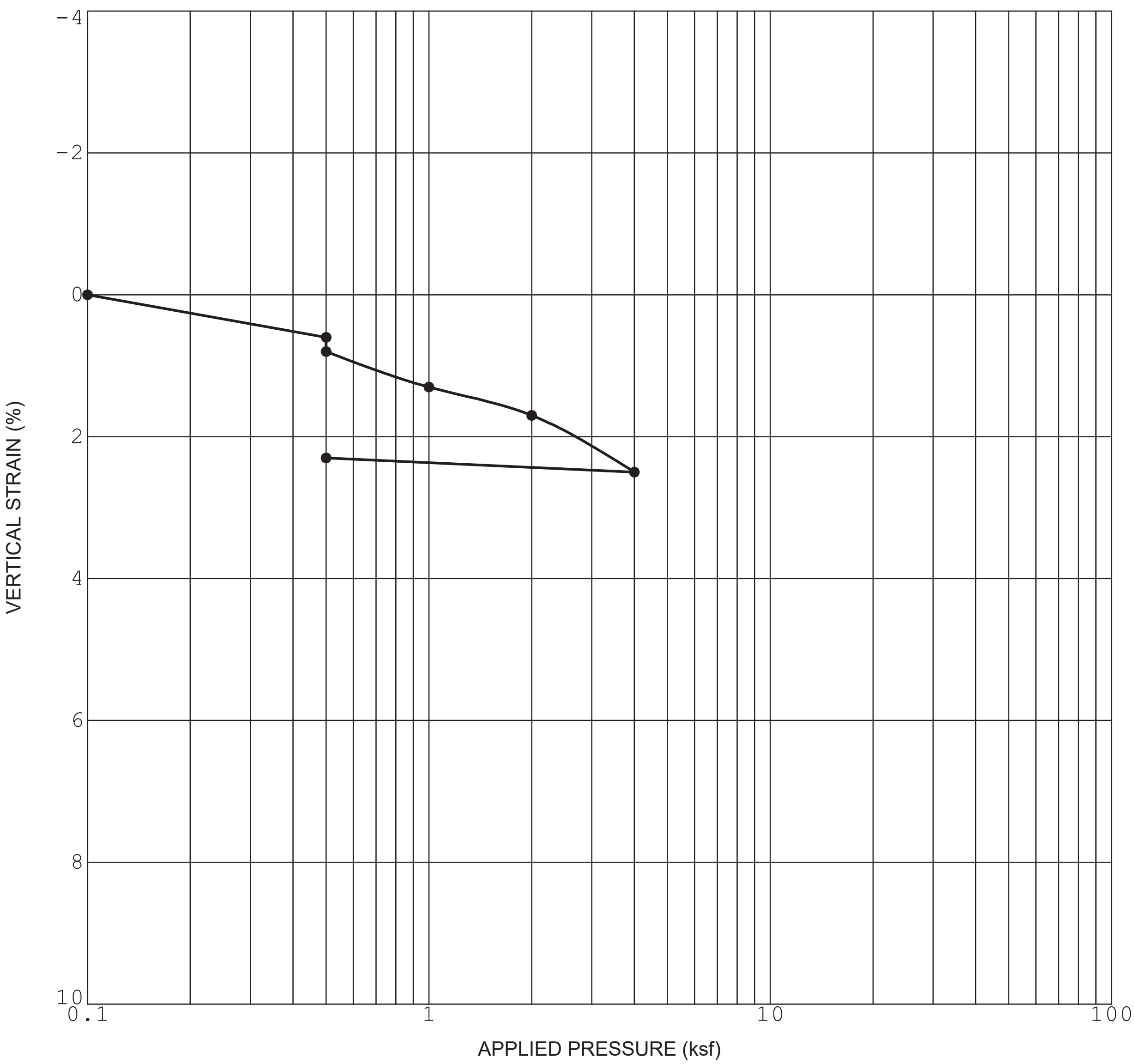
Initial Dry Density (pcf)	119.0	**Initial Saturation (%)**	98.1
Initial Water Content (%)	14.6	**Sample Saturated at (ksf)**	.5

CONSOLIDATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

PROJECT NO. G2298-32-01

SAMPLE NO. B2-2



ASTM D2435

Initial Dry Density (pcf)	103.4
Initial Water Content (%)	19.7

Initial Saturation (%)	86.5
Sample Saturated at (ksf)	.5

CONSOLIDATION CURVE

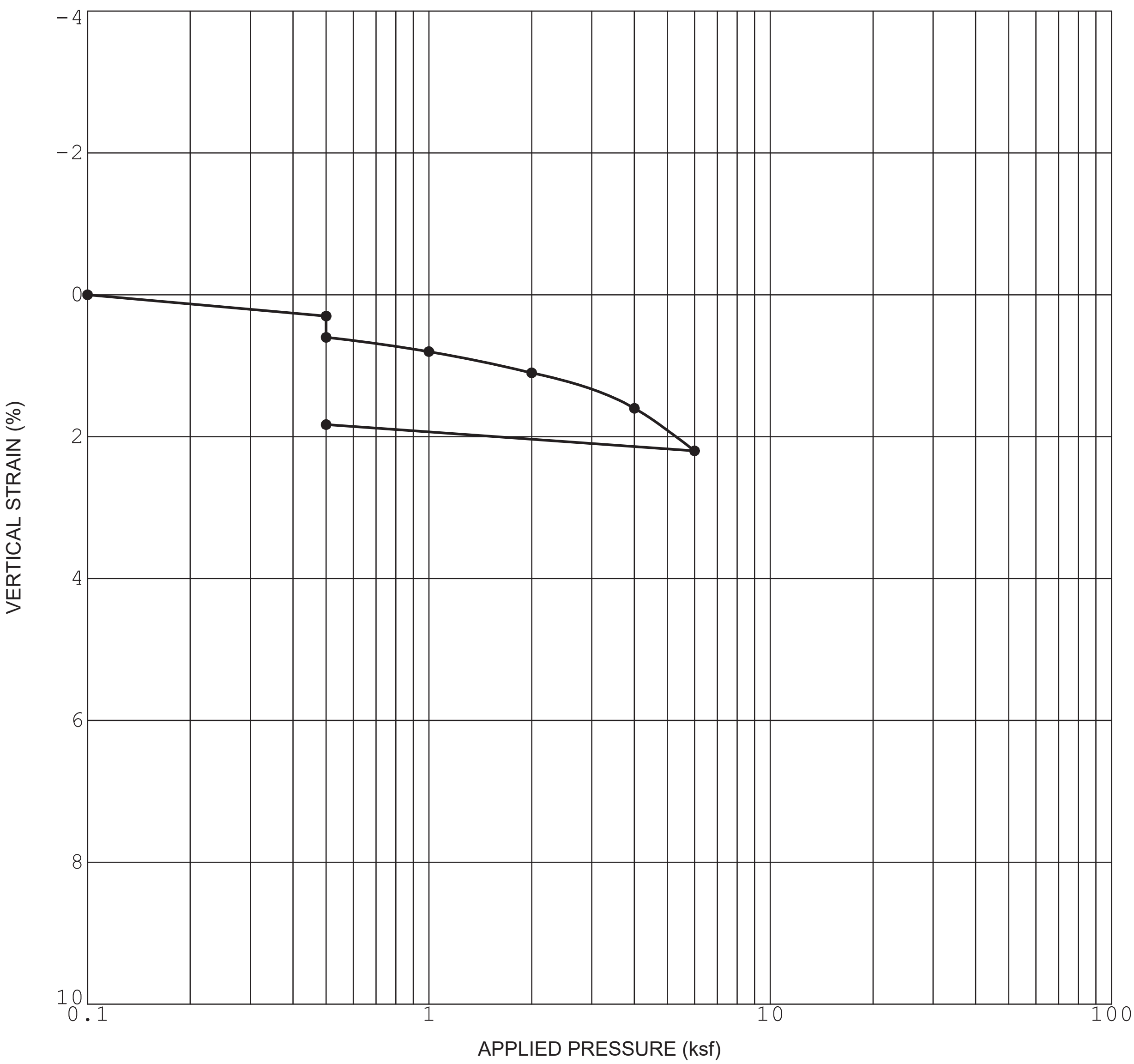
CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

G2298-32-01.GPJ

Figure B-4

GEOCON

SAMPLE NO. B3-5



ASTM D2435

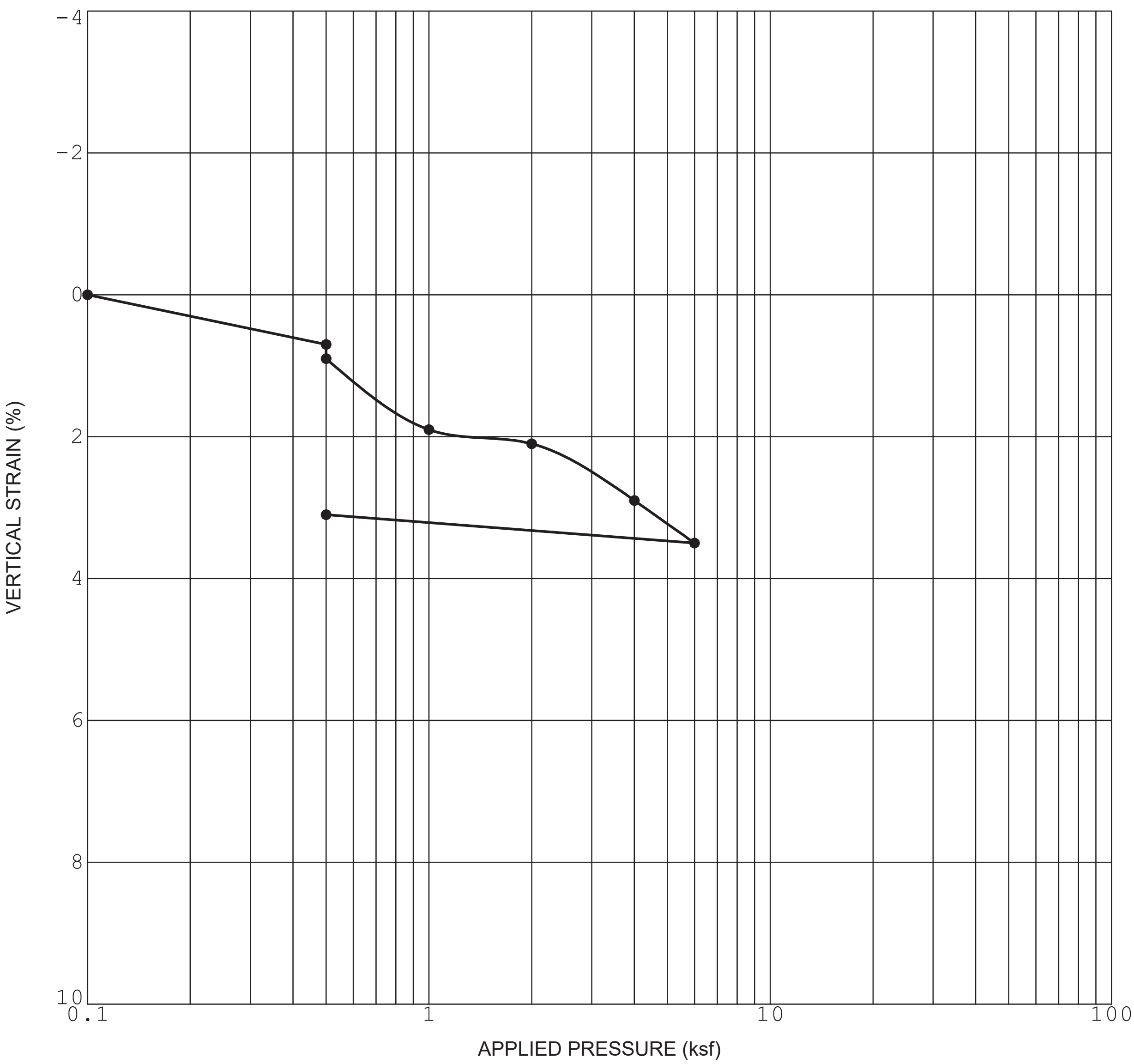
Initial Dry Density (pcf)	129.4	**Initial Saturation (%)**	100+
Initial Water Content (%)	11.6	**Sample Saturated at (ksf)**	.5

CONSOLIDATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

PROJECT NO. G2298-32-01

SAMPLE NO. B4-5



ASTM D2435

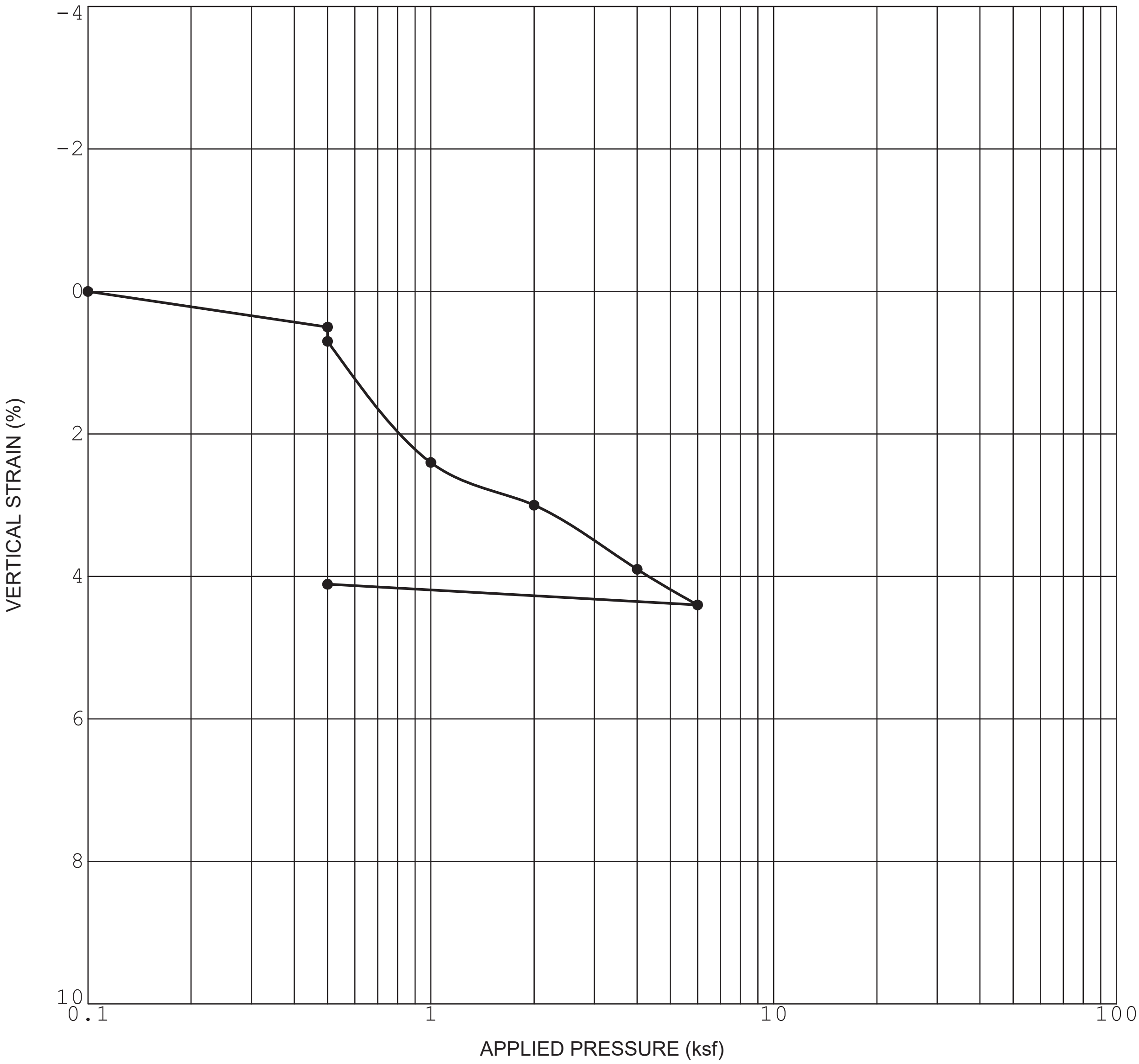
Initial Dry Density (pcf)	112.1
Initial Water Content (%)	19.4

Initial Saturation (%)	100+
Sample Saturated at (ksf)	.5

CONSOLIDATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

SAMPLE NO. B5-5



ASTM D2435

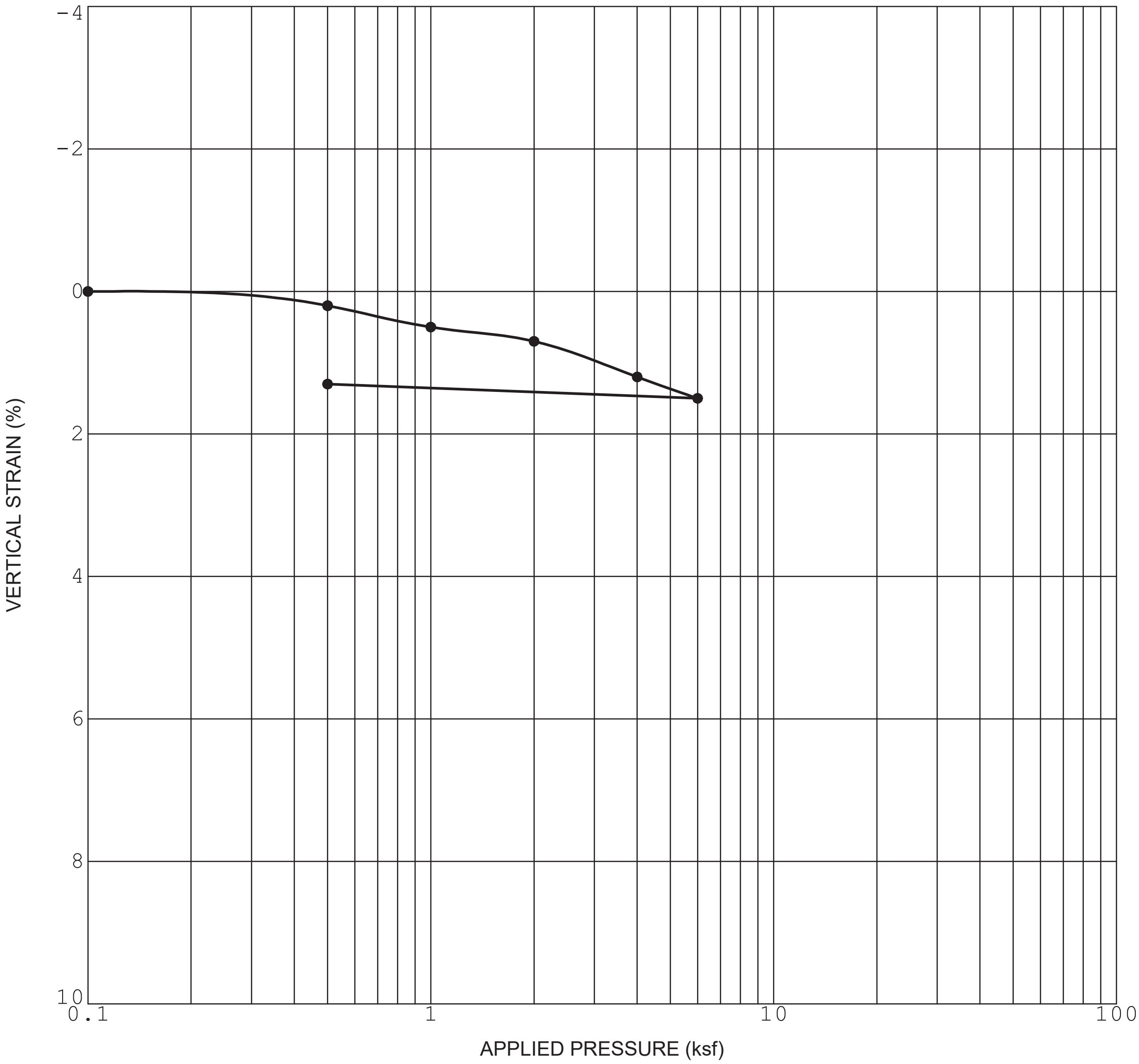
Initial Dry Density (pcf)	113.5
Initial Water Content (%)	13.0

Initial Saturation (%)	74.8
Sample Saturated at (ksf)	.5

CONSOLIDATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

SAMPLE NO. B7-5



ASTM D2435

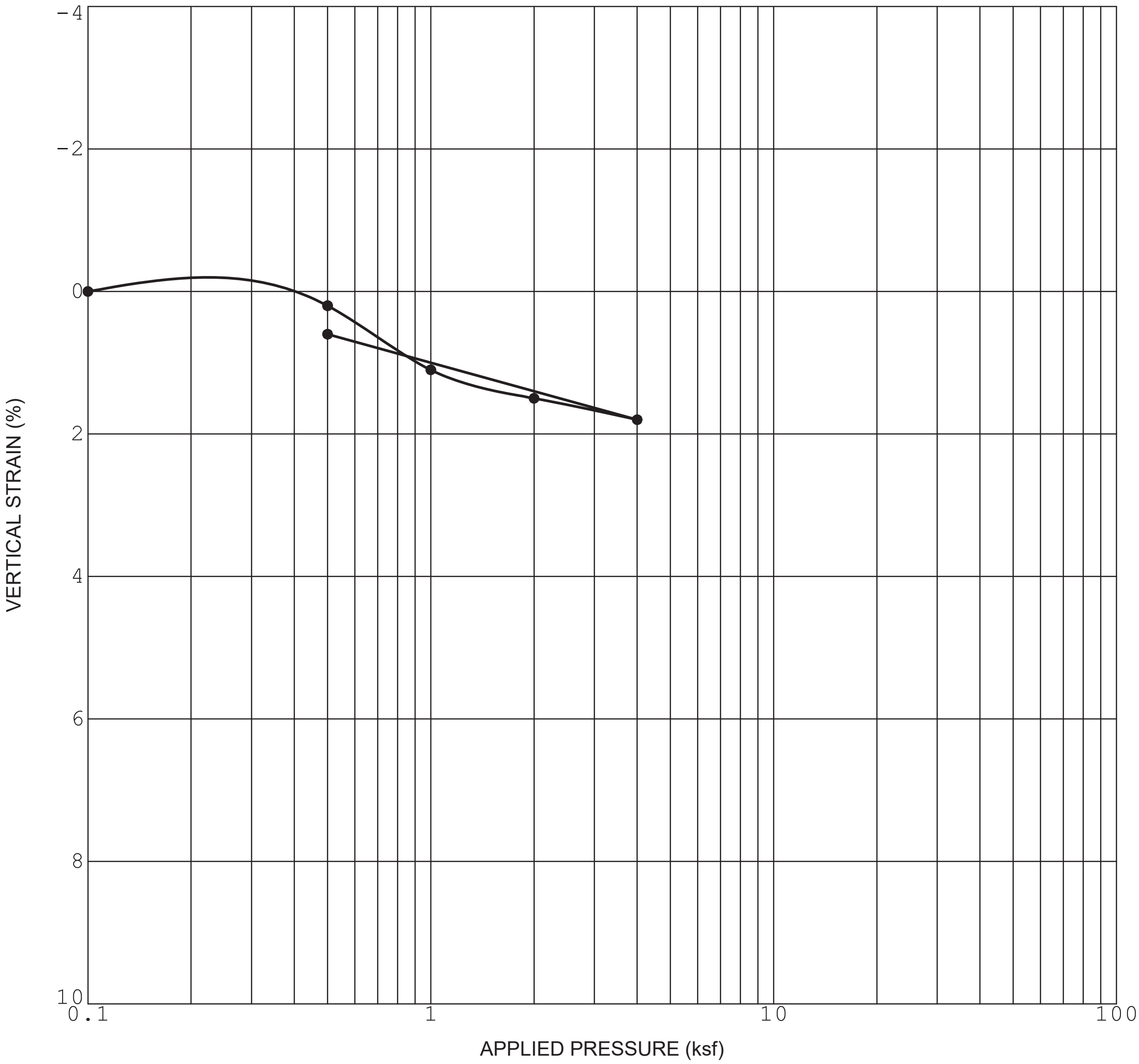
Initial Dry Density (pcf)	110.5
Initial Water Content (%)	21.7

Initial Saturation (%)	100+
Sample Saturated at (ksf)	.5

CONSOLIDATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

SAMPLE NO. B8-4



ASTM D2435

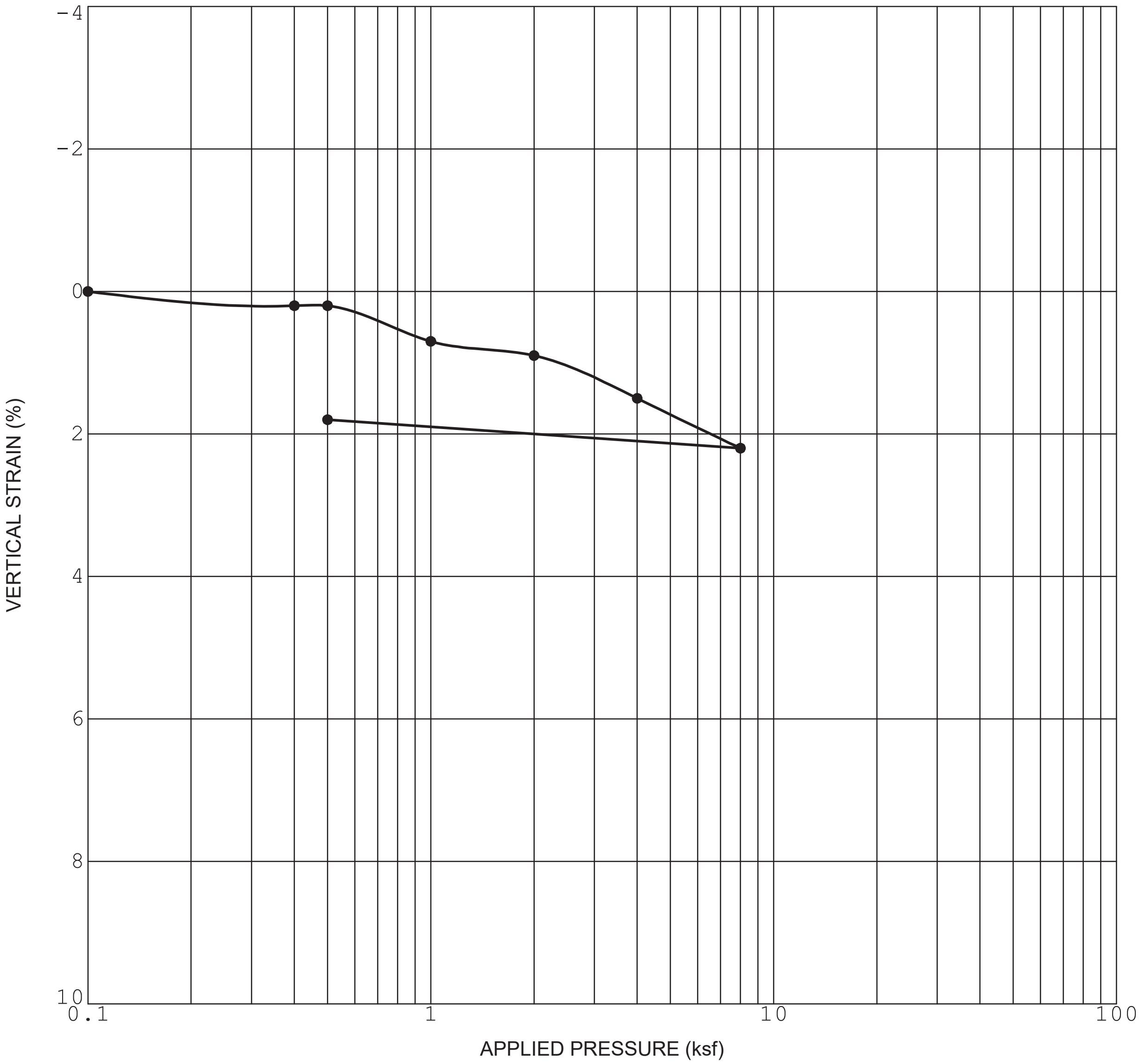
Initial Dry Density (pcf)	120.4	**Initial Saturation (%)**	99.8
Initial Water Content (%)	14.2	**Sample Saturated at (ksf)**	0.5

CONSOLIDATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

PROJECT NO. G2298-32-01

SAMPLE NO. B9-3



ASTM D2435

Initial Dry Density (pcf)	117.7
Initial Water Content (%)	18.2

Initial Saturation (%)	100
Sample Saturated at (ksf)	0.5

CONSOLIDATION CURVE

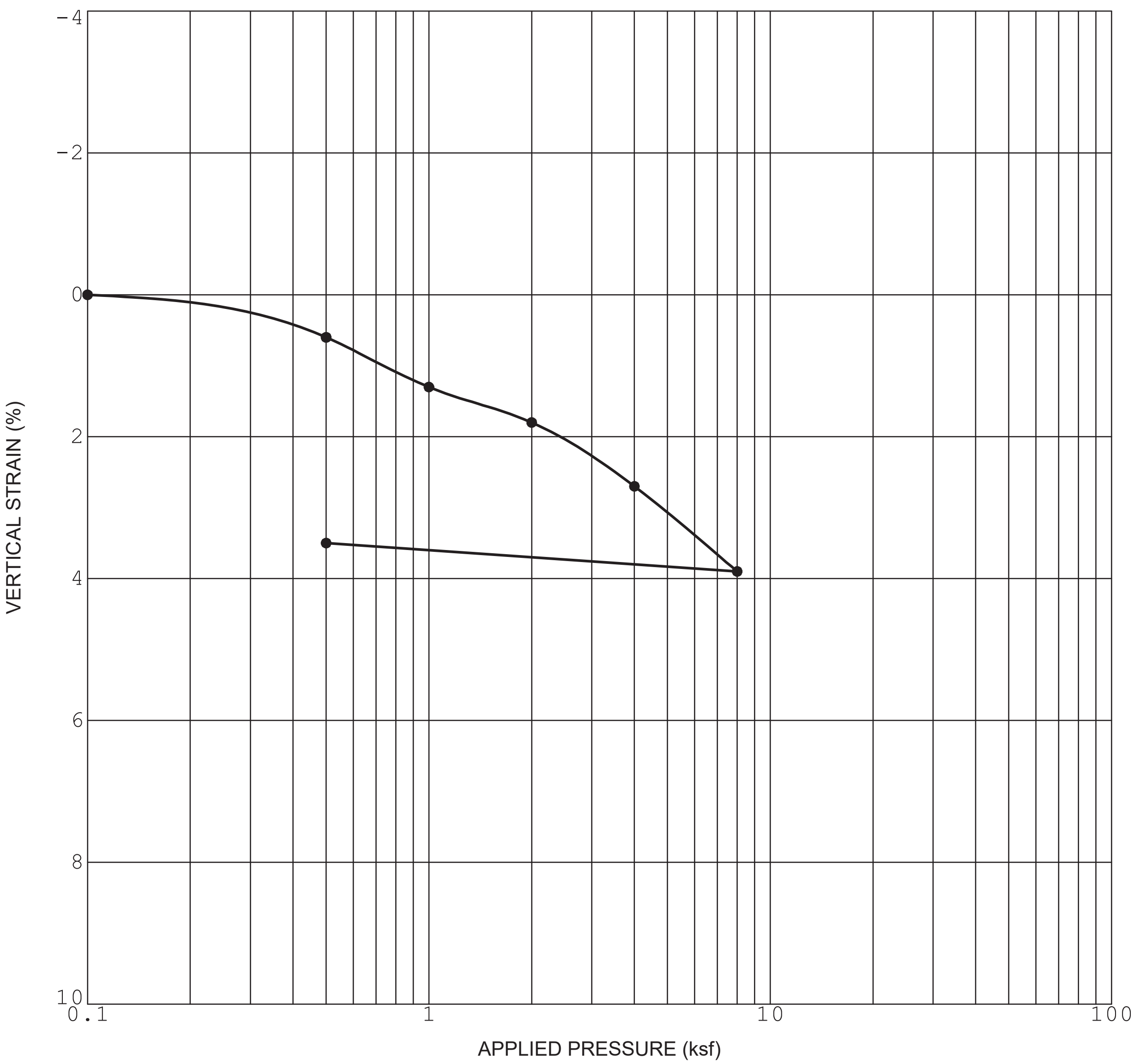
CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

G2298-32-01.GPJ

Figure B-10

GEOCON

SAMPLE NO. B10-1



ASTM D2435

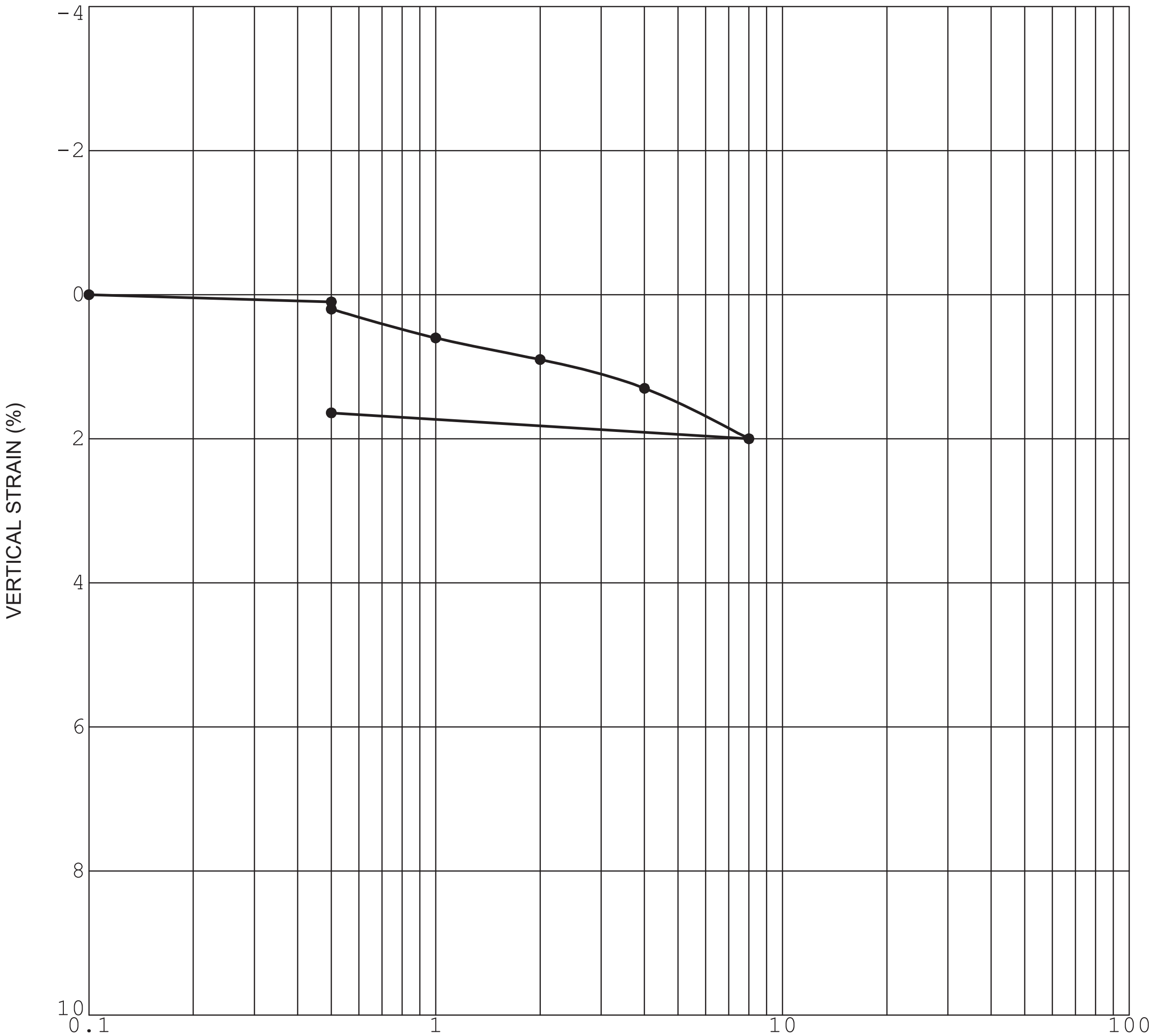
Initial Dry Density (pcf)	98.2	**Initial Saturation (%)**	100
Initial Water Content (%)	30.2	**Sample Saturated at (ksf)**	0.5

CONSOLIDATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

PROJECT NO. G2298-32-01

SAMPLE NO. B10-3



ASTM D2435

Initial Dry Density (pcf)	125.3	**Initial Saturation (%)**	100
Initial Water Content (%)	13.1	**Sample Saturated at (ksf)**	0.5

CONSOLIDATION CURVE

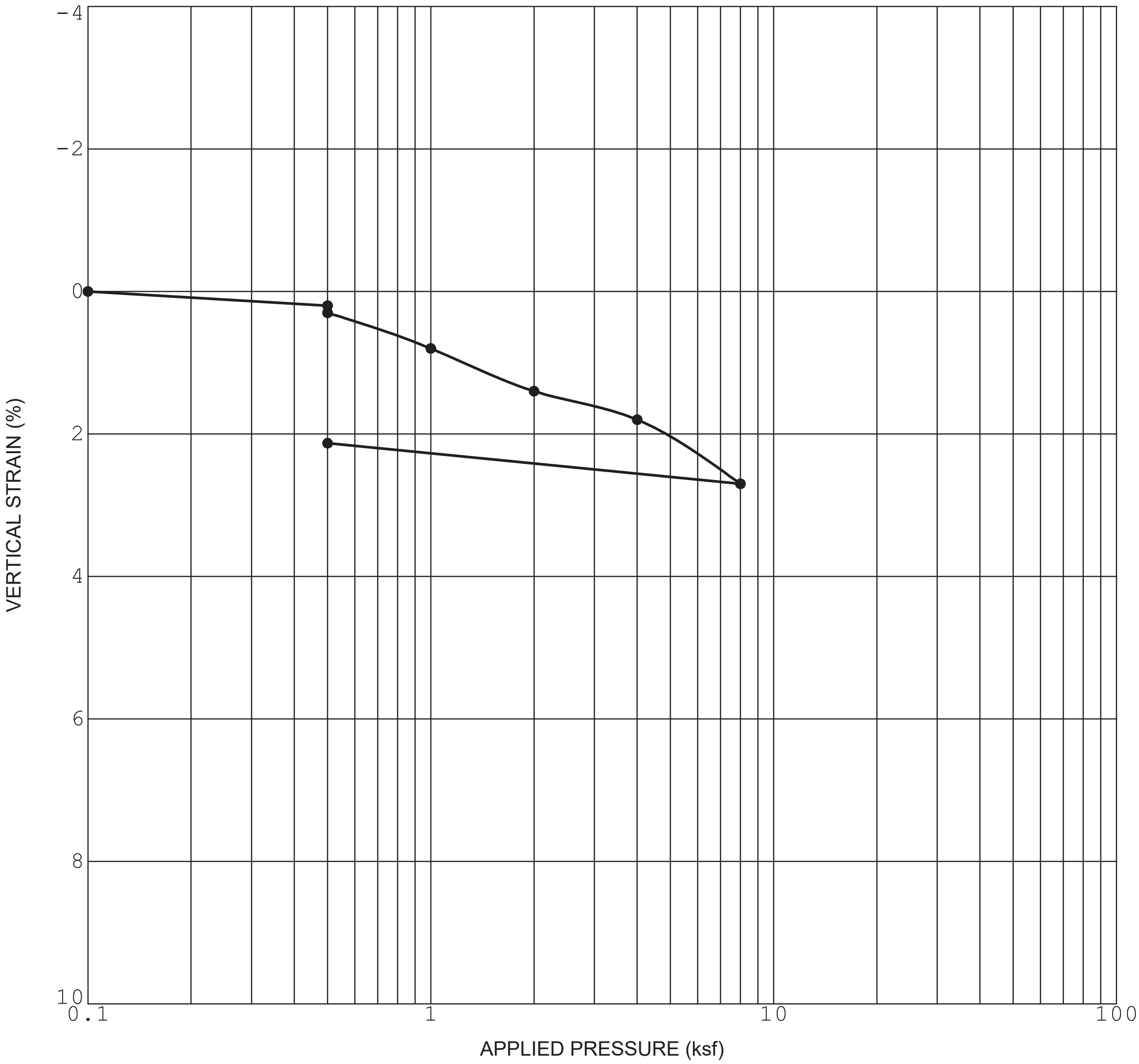
CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

G2298-32-01.GPJ

Figure B-12

GEOCON

SAMPLE NO. B11-4



ASTM D2435

Initial Dry Density (pcf)	97.1		**Initial Saturation (%)**	100
Initial Water Content (%)	32.1		**Sample Saturated at (ksf)**	0.5

CONSOLIDATION CURVE

CARLTON OAKS GOLF COURSE
HOTEL AND ASSISTED LIVING SITES
SANTEE, CALIFORNIA

APPENDIX C

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

FOR

CARLTON OAKS GOLF COURSE
COUNTRY CLUB AND RESORT SITE (PA-3)
SANTEE, CALIFORNIA

PROJECT NO. G2298-32-01

APPENDIX C

STORM WATER MANAGEMENT INVESTIGATION

We understand storm water management devices are being proposed in accordance with the *2016 City of Santee BMP Design Manual for Permanent Site Design, Storm Water Treatment and Hydromodification Management,* commonly referred to as the *Storm Water Standards* (SWS). If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to these devices. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeological study at the site. If infiltration of storm water runoff occurs, downstream properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other undesirable impacts as a result of water infiltration.

Hydrologic Soil Group

The United States Department of Agriculture (USDA), Natural Resources Conservation Services, possesses general information regarding the existing soil conditions for areas within the United States. The USDA website also provides the Hydrologic Soil Group. Table C-I presents the descriptions of the hydrologic soil groups. In addition, the USDA website also provides an estimated saturated hydraulic conductivity for the existing soil.

TABLE C-I
HYDROLOGIC SOIL GROUP DEFINITIONS

Soil Group	Soil Group Definition
A	Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.
B	Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.
C	Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.
D	Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high-water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

The Hotel and Assisted Living Sites are underlain by three units identified as Riverwash (Rm), Stony land (SvE), and Visalia gravelly sandy loam (VbB). The Riverwash (Rm), which encompasses approximately 99 percent of the property, is classified as Soil Group D. The Visalia gravelly sandy loam and Stony land (SvE) are classified as Soil Group A. Table C-III presents the information from the USDA website for the Hotel Site.

TABLE C-II
USDA WEB SOIL SURVEY – HYDROLOGIC SOIL GROUP

Map Unit Name	Map Unit Symbol	Approximate Percentage of Property	Hydrologic Soil Group	k_{SAT} of Most Limiting Layer (inches/hour)
Riverwash	Rm	99	D	5.95 – 19.98
Stony land	SvE	0	A	---
Visalia Gravelly Sandy Loam	VbB	1	A	1.98 – 5.95

In-Situ Testing

The infiltration rate, percolation rates and saturated hydraulic conductivity are different and have different meanings. Percolation rates tend to overestimate infiltration rates and saturated hydraulic conductivities by a factor of 10 or more. Table C-III describes the differences in the definitions.

TABLE C-III
SOIL PERMEABILITY DEFINITIONS

Term	Definition
Infiltration Rate	The observation of the flow of water through a material into the ground downward into a given soil structure under long term conditions. This is a function of layering of soil, density, pore space, discontinuities and initial moisture content.
Percolation Rate	The observation of the flow of water through a material into the ground downward and laterally into a given soil structure under long term conditions. This is a function of layering of soil, density, pore space, discontinuities and initial moisture content.
Saturated Hydraulic Conductivity (k_{SAT}, Permeability)	The volume of water that will move in a porous medium under a hydraulic gradient through a unit area. This is a function of density, structure, stratification, fines content and discontinuities. It is also a function of the properties of the liquid as well as of the porous medium.

The degree of soil compaction or in-situ density has a significant impact on soil permeability and infiltration. Based on our experience and other studies we performed, an increase in compaction results in a decrease in soil permeability.

We performed two Aardvark Permeameter Tests, P-1 and P-2, at locations shown on the attached *Geologic Map*, Figure 2. The test borings were 4 inches in diameter. The results of the tests provide parameters for the saturated hydraulic conductivity characteristics of onsite alluvial soil. Table C-IV presents the results of the estimated field saturated hydraulic conductivity and estimated infiltration rates obtained from the Aardvark Permeameter tests. The field sheets are also attached herein. We applied a feasibility factor of safety of 2 to the field results for use in preparation of Worksheet C.4-1. The results of the testing indicate adjusted soil infiltration rates of 1.2 and 2.6 inches per hour (iph) after applying a Factor of Safety of 2. Based on a discussion in the County of Riverside *Design Handbook for Low Impact Development Best Management Practices*, the infiltration rate should be considered equal to the saturated hydraulic conductivity rate.

TABLE C-IV
FIELD PERMEAMETER INFILTRATION TEST RESULTS

Test No.	Geologic Unit	Test Depth (feet)	Field-Saturated Hydraulic Conductivity, k_{sat} (inch/hour)	Worksheet[1] Saturated Hydraulic Conductivity, k_{sat} (inch/hour)
P-1	Qya	4.2	5.3	2.6
P-2	Qya	2.0	2.3	1.2

[1] Using a factor of safety of 2 for Worksheet C.4-1.

STORM WATER MANAGEMENT CONCLUSIONS

The *Geologic Map*, Figure 3, depicts the existing property, proposed development, the approximate lateral limits of the geologic units, the locations of the field excavations and the in-situ infiltration test locations.

Soil Types

Young Alluvium – Infiltration Tests P-1 and P-2 were performed in the young alluvium above the groundwater. The young alluvium consists of loose to very dense, silty, fine to coarse sand with varying amounts of gravel and cobble. Groundwater is expected to occur approximately 3 to 10 feet below existing grades. The infiltration rates obtained in the younger alluvial deposits above groundwater exhibit permeability characteristics that support either full or partial infiltration.

Infiltration Rates

The results of the infiltration rates (including the feasibility factor of safety of 2) ranged between 1.2 and 2.6 inches per hour. Therefore, based on the results of the infiltration testing, full and partial infiltration should be considered feasible.

Groundwater Elevations

Groundwater elevations at the proposed sites generally range between 304 ft (MSL) to 312 ft (MSL), or approximately 3 to 10 feet below existing grades. In accordance with the 2016 SWS, groundwater must be at least 10 feet below the bottom of the basin for infiltration BMP's to be allowed. Therefore, full and partial infiltration BMPs are considered infeasible based on the shallow groundwater elevations.

Soil or Groundwater Contamination

Although the proposed basins may be situated within 10 feet of groundwater, no soil or groundwater contamination is expected because the basins incorporate bio-filtration prior to infiltrating into the subsurface soils.

New or Existing Utilities

We expect that any on-site utilities would be removed prior to site development, if any. Full or partial infiltration near existing or proposed utilities should be avoided to prevent lateral water migration into the permeable trench backfill materials. The proposed basin is located outside the building pad limits adjacent to the golf course practice area/driving range.

Existing and Planned Structures

The property is a golf course with residential developments to the north and the San Diego River to the south. The existing residential developments in the area are at higher elevations than the proposed development or basins. No proposed structures are located in the vicinity of the proposed basin.

Slopes

The site is relatively flat to gently sloping and significant slopes do not exist adjacent to the site. An approximately 10-foot and 15-foot-high, 2:1 fill slope is shown in the vicinity of the assisted living and hotel sites, respectively.

Recommendations

Due to the shallow groundwater, full or partial infiltration is considered infeasible and liners and subdrains should be incorporated into the design and construction of the planned storm water devices. The liners should be impermeable (e.g., High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent Polyvinyl Chloride, PVC) to prevent water migration. The subdrains should be perforated within the liner area, installed at the base and above the liner, be at least 4 inches in diameter and consist of Schedule 40 PVC pipe. The subdrains outside of the liner should consist of solid pipe. Seams and penetrations of the liners should be properly waterproofed. The subdrains should be connected to a proper outlet. The devices should also be installed in accordance with the manufacturer's recommendations.

Storm Water Standard Worksheets

The SWS requests the geotechnical engineer complete the *Categorization of Infiltration Feasibility Condition* (Worksheet C.4-1 or I-8) worksheet information to help evaluate the potential for infiltration on the property. The attached Worksheet C.4-1 presents the completed information for the submittal process.

The regional storm water standards also have a worksheet (Worksheet D.5-1 or Form I-9) that helps the project civil engineer estimate the factor of safety based on several factors. Table C-V describes the suitability assessment input parameters related to the geotechnical engineering aspects for the factor of safety determination.

TABLE C-V
SUITABILITY ASSESSMENT RELATED CONSIDERATIONS FOR INFILTRATION FACILITY SAFETY FACTORS

Consideration	High Concern – 3 Points	Medium Concern – 2 Points	Low Concern – 1 Point
Assessment Methods	Use of soil survey maps or simple texture analysis to estimate short-term infiltration rates. Use of well permeameter or borehole methods without accompanying continuous boring log. Relatively sparse testing with direct infiltration methods	Use of well permeameter or borehole methods with accompanying continuous boring log. Direct measurement of infiltration area with localized infiltration measurement methods (e.g., Infiltrometer). Moderate spatial resolution	Direct measurement with localized (i.e., small-scale) infiltration testing methods at relatively high resolution or use of extensive test pit infiltration measurement methods.
Predominant Soil Texture	Silty and clayey soils with significant fines	Loamy soils	Granular to slightly loamy soils
Site Soil Variability	Highly variable soils indicated from site assessment or unknown variability	Soil boring/test pits indicate moderately homogenous soils	Soil boring/test pits indicate relatively homogenous soils
Depth to Groundwater/ Impervious Layer	<5 feet below facility bottom	5-15 feet below facility bottom	>15 feet below facility bottom

Based on our geotechnical investigation and the information in Table C-V, Table C-VI presents the estimated factor values for the evaluation of the factor of safety. This table only provides the suitability assessment safety factor (Part A) of the worksheet. The project civil engineer should evaluate the safety factor for design (Part B) and use the combined safety factor for the design infiltration rate.

TABLE C-VI
FACTOR OF SAFETY WORKSHEET DESIGN VALUES – PART A[1]

Suitability Assessment Factor Category	Assigned Weight (w)	Factor Value (v)	Product (p = w x v)
Assessment Methods	0.25	3	0.75
Predominant Soil Texture	0.25	1	0.25
Site Soil Variability	0.25	2	0.25
Depth to Groundwater/ Impervious Layer	0.25	3	0.75
Suitability Assessment Safety Factor, $S_A = \sum p$			2.00

[1] The project civil engineer should complete Worksheet D.5-1 or Form I-9 using the data on this table. Additional information is required to evaluate the design factor of safety.

Categorization of Infiltration Feasibility Condition	Worksheet C.4-1

Part 1 - Full Infiltration Feasibility Screening Criteria

Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
1	**Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

Based on results of permeability testing in two locations near the proposed basin, the unfactored infiltration rate was measured to be 2.3 inches/hour and 5.3 inches/hour using a constant head borehole permeameter. If applying a feasibility factor of safety of 2.0, the infiltration rates would be 1.2 iph and 2.6 iph, which are greater than the required threshold value of 0.5 iph. The Aardvark Permeameter test results are attached. In accordance with the Riverside County storm water procedures, which reference the United States Bureau of Reclamation Well Permeameter Method (USBR 7300), the saturated hydraulic conductivity is equal to the unfactored infiltration rate.

Criteria	Screening Question	Yes	No
2	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X

Provide basis:

The proposed basin is supported by potentially liquefiable soils with current groundwater elevations of approximately 3 to 10 feet below existing grades. Groundwater mounding could occur as a result of full infiltration, and an increase in groundwater elevation increases the potential for liquefaction and the adverse affects of liquefaction, such as differential settlement, loss of soil support, and lateral spreading.

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	**Yes**	**No**
3	**Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: Groundwater was measured to be approximately 3 to 10 feet below existing grades. The proposed basin bottom is the same as the existing ground surface. There is an increased risk to groundwater contamination due to shallow groundwater. In accordance with the City's guidelines, a minimum 10 foot separation between bottom of infiltration surface and top of groundwater should be met to avoid potential groundwater contamination issues.			
4	**Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We do not expect infiltration will cause water balance issues such as seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters.			
Part 1 Result*	If all answers to rows 1 - 4 are "**Yes**" a full infiltration design is potentially feasible. The feasibility screening category is **Full Infiltration** If any answer from row 1-4 is "**No**", infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a "full infiltration" design. Proceed to Part 2		No Full Infiltration

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by the City to substantiate findings.

Worksheet C.4-1 Page 3 of 4

<u>Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria</u>

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	**Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

Based on results of permeability testing in two locations near the proposed basin, the unfactored infiltration rate was measured to be 2.3 inches/hour and 5.3 inches/hour using a constant head borehole permeameter. If applying a feasibility factor of safety of 2.0, the infiltration rates would be 1.2 iph and 2.6 iph, which are greater than the required threshold value of 0.05 iph. The Aardvark Permeameter test results are attached. In accordance with the Riverside County storm water procedures, which reference the United States Bureau of Reclamation Well Permeameter Method (USBR 7300), the saturated hydraulic conductivity is equal to the unfactored infiltration rate.

Criteria	Screening Question	Yes	No
6	**Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	

Provide basis:

The proposed basin is situated in a golf course down-gradient from the proposed development. We do not expect slope instability, groundwater mounding, or an increase in liquefaction potential as a result of partial infiltration, provided a subdrain is incorporated into the design.

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	**Yes**	**No**
7	**Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?** The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		X
Provide basis: Groundwater was measured to be approximately 3 to 10 feet below existing grades. The proposed basin bottom is the same as the existing ground surface. There is an increased risk to groundwater contamination due to shallow groundwater. In accordance with the City's guidelines, a minimum 10 foot separation between bottom of infiltration surface and top of groundwater should be met to avoid potential groundwater contamination issues.			
8	**Can infiltration be allowed without violating downstream water rights**? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.	X	
Provide basis: We did not provide a study regarding water rights. However, these rights are not typical in the San Diego area.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is **Partial Infiltration.** If any answer from row 5-8 is no, then infiltration of any volume is considered to be **infeasible** within the drainage area. The feasibility screening category is **No Infiltration.**	No Infiltration	

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by the City to substantiate findings.

GEOCON

Aardvark Permeameter Data Analysis

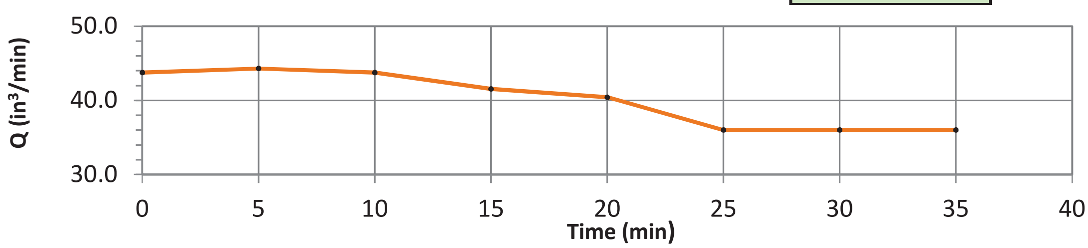
Project Name:	Carlton Oaks	Date:	6/10/2019
Project Number:	G2298-32-01	By:	DEG
Test Number:	P-1		

Borehole Diameter, d (in.):	4.00
Borehole Depth, **H** (in):	50.00
Distance Between Reservoir & Top of Borehole (in.)	30.00
Estimated Depth to Water Table, **S** (feet):	5.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Ref. EL (feet, MSL):	315.0
Bottom EL (feet, MSL):	310.8

Distance Between Reservoir and APM Float, **D** (in.):	71.75
Head Height Calculated, **h** (in.):	4.74
Head Height Measured, **h** (in.):	4.50
Distance Between Constant Head and Water Table, **L** (in.):	14.50

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	9.925	274.85	54.969
3	5.00	8.800	243.69	48.738
4	5.00	8.500	235.38	47.077
5	5.00	8.500	235.38	47.077
6	5.00	7.900	218.77	43.754
7	5.00	8.000	221.54	44.308
8	5.00	7.900	218.77	43.754
9	5.00	7.500	207.69	41.538
10	5.00	7.300	202.15	40.431
11	5.00	6.500	180.00	36.000
12	5.00	6.500	180.00	36.000
13	5.00	6.500	180.00	36.000
			Steady Flow Rate, Q (in^3/min):	36.000



Soil Matric Flux Potential, Φ_m

Φ_m= 0.85991 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 8.77E-02 in/min 5.263 in/hr

GEOCON

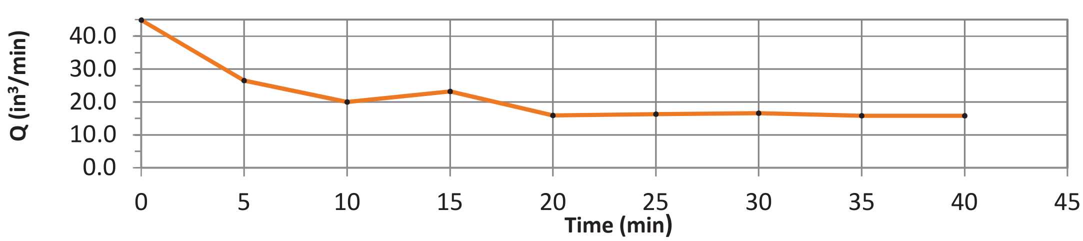
Aardvark Permeameter Data Analysis

Project Name:	Carlton Oaks	Date:	6/10/2019
Project Number:	G2298-32-01	By:	DEG
Test Number:	P-2	Ref. EL (feet, MSL):	312.0
		Bottom EL (feet, MSL):	310.0

Borehole Diameter, d (in.):	4.00
Borehole Depth, **H** (in):	24.00
Distance Between Reservoir & Top of Borehole (in.)	30.00
Estimated Depth to Water Table, **S** (feet):	3.00
Height APM Raised from Bottom (in.):	1.00
Pressure Reducer Used:	No

Distance Between Reservoir and APM Float, **D** (in.):	45.75
Head Height Calculated, **h** (in.):	4.65
Head Height Measured, **h** (in.):	4.50
Distance Between Constant Head and Water Table, **L** (in.):	16.50

Reading	Time Elapsed (min)	Water Weight Consumed (lbs)	Water Volume Consumed (in^3)	Q (in^3/min)
1	0.00	0.000	0.00	0.00
2	5.00	8.105	224.45	44.889
3	5.00	4.785	132.51	26.502
4	5.00	3.615	100.11	20.022
5	5.00	4.195	116.17	23.234
6	5.00	2.880	79.75	15.951
7	5.00	2.945	81.55	16.311
8	5.00	3.000	83.08	16.615
9	5.00	2.855	79.06	15.812
10	5.00	2.855	79.06	15.812
			Steady Flow Rate, Q (in^3/min):	15.812



Soil Matric Flux Potential, Φ_m

Φ_m= 0.3777 in^2/min

Field-Saturated Hydraulic Conductivity (Infiltration Rate)

K_{sat} = 3.85E-02 in/min 2.311 in/hr

United States Department of Agriculture

NRCS

Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for
San Diego County Area, California

June 14, 2019

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

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Contents

How Soil Surveys Are Made

Soil surveys are made to provide information about the soils and miscellaneous areas in a specific area. They include a description of the soils and miscellaneous areas and their location on the landscape and tables that show soil properties and limitations affecting various uses. Soil scientists observed the steepness, length, and shape of the slopes; the general pattern of drainage; the kinds of crops and native plants; and the kinds of bedrock. They observed and described many soil profiles. A soil profile is the sequence of natural layers, or horizons, in a soil. The profile extends from the surface down into the unconsolidated material in which the soil formed or from the surface down to bedrock. The unconsolidated material is devoid of roots and other living organisms and has not been changed by other biological activity.

Currently, soils are mapped according to the boundaries of major land resource areas (MLRAs). MLRAs are geographically associated land resource units that share common characteristics related to physiography, geology, climate, water resources, soils, biological resources, and land uses (USDA, 2006). Soil survey areas typically consist of parts of one or more MLRA.

The soils and miscellaneous areas in a survey area occur in an orderly pattern that is related to the geology, landforms, relief, climate, and natural vegetation of the area. Each kind of soil and miscellaneous area is associated with a particular kind of landform or with a segment of the landform. By observing the soils and miscellaneous areas in the survey area and relating their position to specific segments of the landform, a soil scientist develops a concept, or model, of how they were formed. Thus, during mapping, this model enables the soil scientist to predict with a considerable degree of accuracy the kind of soil or miscellaneous area at a specific location on the landscape.

Commonly, individual soils on the landscape merge into one another as their characteristics gradually change. To construct an accurate soil map, however, soil scientists must determine the boundaries between the soils. They can observe only a limited number of soil profiles. Nevertheless, these observations, supplemented by an understanding of the soil-vegetation-landscape relationship, are sufficient to verify predictions of the kinds of soil in an area and to determine the boundaries.

Soil scientists recorded the characteristics of the soil profiles that they studied. They noted soil color, texture, size and shape of soil aggregates, kind and amount of rock fragments, distribution of plant roots, reaction, and other features that enable them to identify soils. After describing the soils in the survey area and determining their properties, the soil scientists assigned the soils to taxonomic classes (units). Taxonomic classes are concepts. Each taxonomic class has a set of soil characteristics with precisely defined limits. The classes are used as a basis for comparison to classify soils systematically. Soil taxonomy, the system of taxonomic classification used in the United States, is based mainly on the kind and character of soil properties and the arrangement of horizons within the profile. After the soil scientists classified and named the soils in the survey area, they compared the individual soils with similar soils in the same taxonomic class in other areas so that they could confirm data and assemble additional data based on experience and research.

The objective of soil mapping is not to delineate pure map unit components; the objective is to separate the landscape into landforms or landform segments that have similar use and management requirements. Each map unit is defined by a unique combination of soil components and/or miscellaneous areas in predictable proportions. Some components may be highly contrasting to the other components of the map unit. The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The delineation of such landforms and landform segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, onsite investigation is needed to define and locate the soils and miscellaneous areas.

Soil scientists make many field observations in the process of producing a soil map. The frequency of observation is dependent upon several factors, including scale of mapping, intensity of mapping, design of map units, complexity of the landscape, and experience of the soil scientist. Observations are made to test and refine the soil-landscape model and predictions and to verify the classification of the soils at specific locations. Once the soil-landscape model is refined, a significantly smaller number of measurements of individual soil properties are made and recorded. These measurements may include field measurements, such as those for color, depth to bedrock, and texture, and laboratory measurements, such as those for content of sand, silt, clay, salt, and other components. Properties of each soil typically vary from one point to another across the landscape.

Observations for map unit components are aggregated to develop ranges of characteristics for the components. The aggregated values are presented. Direct measurements do not exist for every property presented for every map unit component. Values for some properties are estimated from combinations of other properties.

While a soil survey is in progress, samples of some of the soils in the area generally are collected for laboratory analyses and for engineering tests. Soil scientists interpret the data from these analyses and tests as well as the field-observed characteristics and the soil properties to determine the expected behavior of the soils under different uses. Interpretations for all of the soils are field tested through observation of the soils in different uses and under different levels of management. Some interpretations are modified to fit local conditions, and some new interpretations are developed to meet local needs. Data are assembled from other sources, such as research information, production records, and field experience of specialists. For example, data on crop yields under defined levels of management are assembled from farm records and from field or plot experiments on the same kinds of soil.

Predictions about soil behavior are based not only on soil properties but also on such variables as climate and biological activity. Soil conditions are predictable over long periods of time, but they are not predictable from year to year. For example, soil scientists can predict with a fairly high degree of accuracy that a given soil will have a high water table within certain depths in most years, but they cannot predict that a high water table will always be at a specific level in the soil on a specific date.

After soil scientists located and identified the significant natural bodies of soil in the survey area, they drew the boundaries of these bodies on aerial photographs and identified each as a specific map unit. Aerial photographs show trees, buildings, fields, roads, and rivers, all of which help in locating boundaries accurately.

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Soil Map

Map Scale: 1:2,970 if printed on A portrait (8.5" x 11") sheet.

Meters: 0, 40, 80, 160, 240

Feet: 0, 100, 200, 400, 600

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 11N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 13, Sep 12, 2018

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 7, 2014—Jan 4, 2015

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
Rm	Riverwash	19.5	99.3%
SvE	Stony land	0.0	0.0%
VbB	Visalia gravelly sandy loam, 2 to 5 percent slopes	0.1	0.7%
Totals for Area of Interest		**19.6**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

San Diego County Area, California

Rm—Riverwash

Map Unit Setting

National map unit symbol: hbg6
Elevation: 700 to 2,900 feet
Mean annual precipitation: 8 to 15 inches
Mean annual air temperature: 46 to 52 degrees F
Frost-free period: 110 to 180 days
Farmland classification: Not prime farmland

Map Unit Composition

Riverwash: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Riverwash

Setting

Landform: Drainageways
Parent material: Sandy, gravelly, or cobbly alluvium derived from mixed sources

Typical profile

H1 - 0 to 6 inches: gravelly coarse sand
H2 - 6 to 60 inches: stratified extremely gravelly coarse sand to gravelly sand

Properties and qualities

Slope: 0 to 4 percent
Natural drainage class: Excessively drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): High to very high (5.95 to 19.98 in/hr)
Depth to water table: About 60 to 72 inches
Frequency of flooding: Occasional
Available water storage in profile: Very low (about 1.9 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 8
Hydrologic Soil Group: D
Hydric soil rating: Yes

SvE—Stony land

Map Unit Setting

National map unit symbol: hbgv
Elevation: 650 to 4,000 feet
Mean annual precipitation: 8 to 15 inches
Mean annual air temperature: 45 to 52 degrees F
Frost-free period: 110 to 180 days
Farmland classification: Not prime farmland

Map Unit Composition

Stony land: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Stony Land

Setting

Landform: Mountains
Landform position (three-dimensional): Mountainflank
Down-slope shape: Convex
Across-slope shape: Linear
Parent material: Mixed colluvium

Typical profile

H1 - 0 to 60 inches: unweathered bedrock

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 8
Hydrologic Soil Group: A
Hydric soil rating: No

VbB—Visalia gravelly sandy loam, 2 to 5 percent slopes

Map Unit Setting

National map unit symbol: hbh6
Elevation: 0 to 1,500 feet
Mean annual precipitation: 15 inches
Mean annual air temperature: 61 degrees F
Frost-free period: 200 to 350 days
Farmland classification: Prime farmland if irrigated

Map Unit Composition

Visalia and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Visalia

Setting

Landform: Alluvial fans
Landform position (two-dimensional): Toeslope
Landform position (three-dimensional): Riser, flat
Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Alluvium derived from granite

Typical profile

H1 - 0 to 12 inches: gravelly sandy loam
H2 - 12 to 40 inches: gravelly sandy loam
H3 - 40 to 60 inches: gravelly loam

Properties and qualities

Slope: 2 to 5 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Runoff class: Very low
Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 5.95 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Moderate (about 6.4 inches)

Interpretive groups

Land capability classification (irrigated): 2e
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: A
Ecological site: LOAMY (1975) (R019XD029CA)
Hydric soil rating: No

Minor Components

Tujunga

Percent of map unit: 5 percent
Hydric soil rating: No

Greenfield

Percent of map unit: 5 percent
Hydric soil rating: No

Placentia

Percent of map unit: 5 percent
Hydric soil rating: No

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APPENDIX D

APPENDIX D

LIQUEFACTION ANALYSES

FOR

CARLTON OAKS GOLF COURSE
COUNTRY CLUB AND RESORT SITE (PA-3)
SANTEE, CALIFORNIA

PROJECT NO. G2298-32-01

GEOCON

Liquefaction Analysis Using SPT

References
1. *Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10*
2. *Seed, et al, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistant Framework, 2003.*

Project Name:	Carlton Oaks	Boring:	B-1
Project Number:	G2298-32-01		

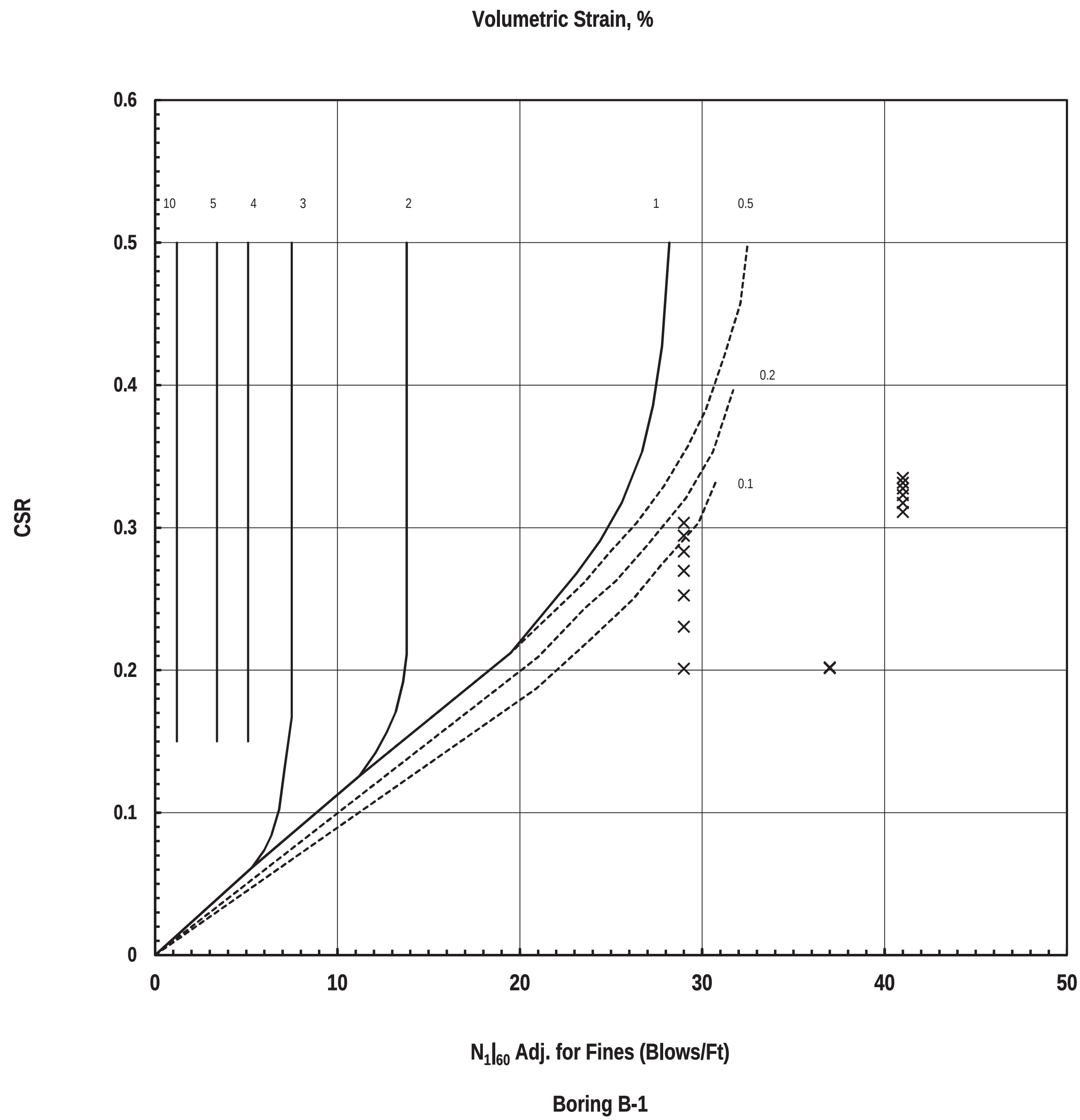
PGAm	0.386	Include Kσ (Y/N)	N
Modal Magnitude	6.80	Use NCEER CRR7.5 (1) or Rauch CRR7.5 (2)	1
Groundwater Depth, Ft	3.0	Minimum Factor of Safety for Liquefaction	1
Reference Pressure, p_a	1000		
Unit Weight of Water	62.4		
Soil Unit Weight, pcf	120		

			Enter for Fine-Grained Materials				Old	New						MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$					SP117 Graph	
Depth, ft	**$N_1\|_{60}$**	**Fines Content, FC (%)**	**Water Content, w_C (%)**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	**$N_1\|_{60}$, Adj. for Fines**	**$N_1\|_{60}$, Adj. for Fines**	**σ, psf**	**σ', psf**	**r_d**	**K_σ**	**NCEER $CRR_{7.5}$**	**RAUCH $CRR_{7.5}$**	**CSR M=7.5**	**Fines Liquefiable (Y/N)**	**Liquefaction Potential**	**Factor of Safety**	**Volumetric Strain, %**	**Settlement, in.**
1	37	5	15.0			0	37.0	37.0	120.0	120.0	1.00	1.00	0.800	0.800	0.195	Y	Above GWT	4.098	0	0
2	37	5	15.0			0	37.0	37.0	240.0	240.0	1.00	1.00	0.800	0.800	0.195	Y	Above GWT	4.107	0	0
3	29	5	15.0			0	29.0	29.0	360.0	360.0	0.99	1.00	0.386	0.410	0.194	Y	NL	1.986	0	0
4	29	5	15.0			0	29.0	29.0	480.0	417.6	0.99	1.00	0.386	0.410	0.223	Y	NL	1.732	0	0
5	29	5	15.0			0	29.0	29.0	600.0	475.2	0.99	1.00	0.386	0.410	0.244	Y	NL	1.581	0	0
6	29	5	15.0			0	29.0	29.0	720.0	532.8	0.99	1.00	0.386	0.410	0.261	Y	NL	1.480	0	0
7	29	5	15.0			0	29.0	29.0	840.0	590.4	0.99	1.00	0.386	0.410	0.274	Y	NL	1.409	0	0
8	29	5	15.0			0	29.0	29.0	960.0	648.0	0.98	1.00	0.386	0.410	0.285	Y	NL	1.356	0	0
9	29	5	15.0			0	29.0	29.0	1080.0	705.6	0.98	1.00	0.386	0.410	0.293	Y	NL	1.316	0	0
10	41	5	15.0			0	41.0	41.0	1200.0	763.2	0.98	1.00	0.800	0.800	0.301	Y	NL	2.661	0	0
11	41	5	15.0			0	41.0	41.0	1320.0	820.8	0.98	1.00	0.800	0.800	0.307	Y	NL	2.607	0	0
12	41	5	15.0			0	41.0	41.0	1440.0	878.4	0.97	1.00	0.800	0.800	0.312	Y	NL	2.563	0	0
13	41	5	15.0			0	41.0	41.0	1560.0	936.0	0.97	1.00	0.800	0.800	0.317	Y	NL	2.526	0	0
14	41	5	15.0			0	41.0	41.0	1680.0	993.6	0.97	1.00	0.800	0.800	0.321	Y	NL	2.496	0	0
15	41	5	15.0			0	41.0	41.0	1800.0	1051.2	0.97	1.00	0.800	0.800	0.324	Y	NL	2.470	0	0

Total Settlement, S_{LIQ} (in.) = 0

Total Liquefiable Layers = 15

FIGURE D-1



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LIQUEFACTION - VOLUMETRIC STRAIN

CARLTON OAKS GOLF COURSE
HOTEL AND ASSITED LIVING SITES
SANTEE, CALIFORNIA

TEM			07-17-2019	PROJECT NO. G2298-32-01	D-2

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Liquefaction Analysis Using SPT

References
1. *Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10*
2. *Seed, et al, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistant Framework, 2003.*

Project Name:	Carlton Oaks	Boring:	B-2
Project Number:	G2298-32-01		

PGAm	0.386
Modal Magnitude	6.80
Groundwater Depth, Ft	6.0
Reference Pressure, p_a	1000
Unit Weight of Water	62.4
Soil Unit Weight, pcf	120

Include Kσ (Y/N)	N
Use NCEER CRR7.5 (1) or Rauch CRR7.5 (2)	1
Minimum Factor of Safety for Liquefaction	1

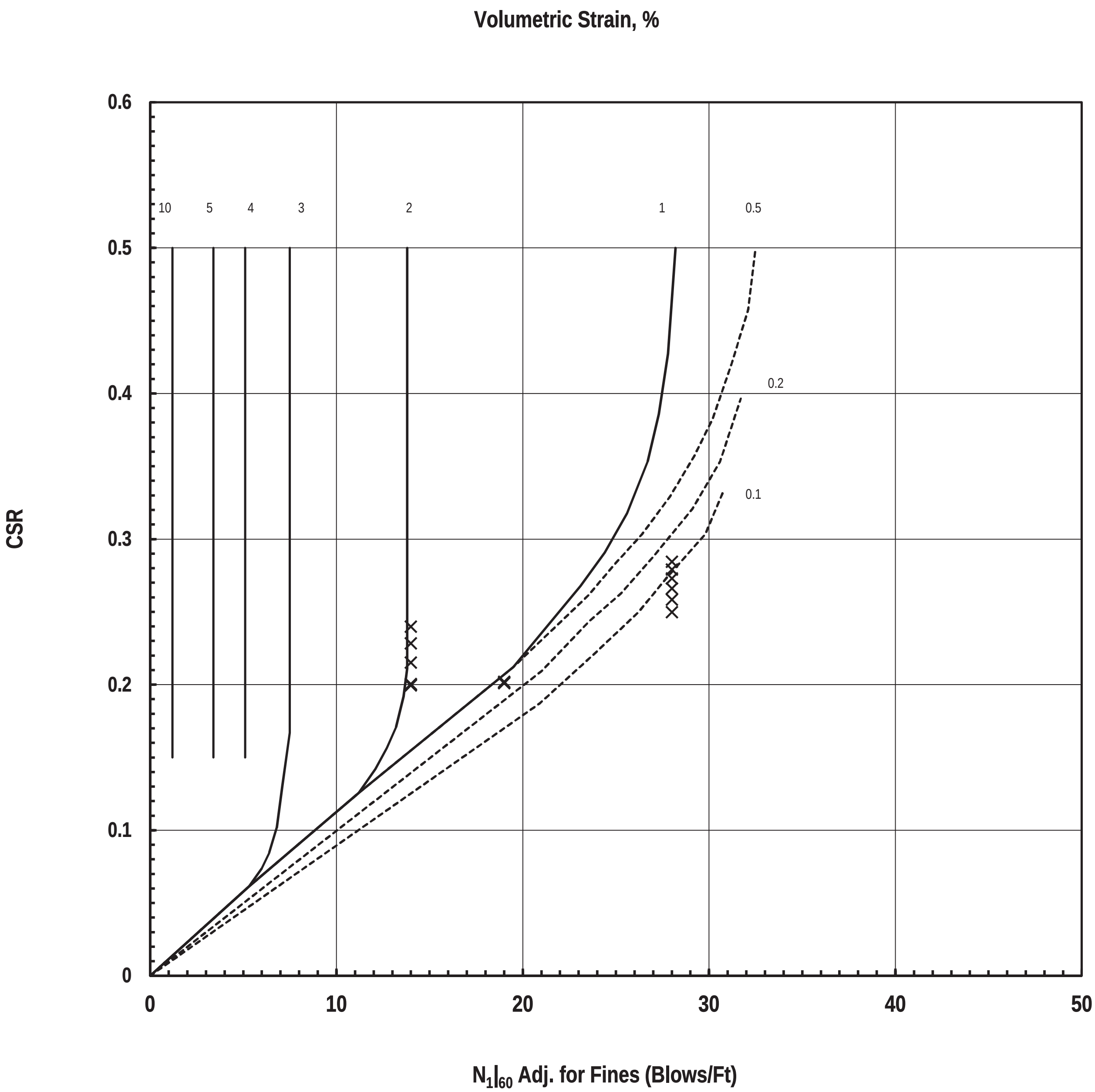
Enter for Fine-Grained Materials | Old | New | MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$ | SP117 Graph

Depth, ft	$N_1\|_{60}$	Fines Content, FC (%)	Water Content, w_C (%)	Liquid Limit	Plastic Limit	Plasticity Index	$N_1\|_{60}$, Adj. for Fines (Old)	$N_1\|_{60}$, Adj. for Fines (New)	σ, psf	σ', psf	r_d	K_σ	NCEER $CRR_{7.5}$	RAUCH $CRR_{7.5}$	CSR M=7.5	Fines Liquefiable (Y/N)	Liquefaction Potential	Factor of Safety	Volumetric Strain, %	Settlement, in.
1	19	5	15.0			0	19.0	19.0	120.0	120.0	1.00	1.00	0.207	0.203	0.195	Y	Above GWT	1.058	0	0
2	19	5	15.0			0	19.0	19.0	240.0	240.0	1.00	1.00	0.207	0.203	0.195	Y	Above GWT	1.061	0	0
3	19	5	15.0			0	19.0	19.0	360.0	360.0	0.99	1.00	0.207	0.203	0.194	Y	Above GWT	1.063	0	0
4	14	5	15.0			0	14.0	14.0	480.0	480.0	0.99	1.00	0.153	0.150	0.194	Y	Above GWT	0.788	0	0
5	14	5	15.0			0	14.0	14.0	600.0	600.0	0.99	1.00	0.153	0.150	0.193	Y	Above GWT	0.790	0	0
6	14	5	15.0			0	14.0	14.0	720.0	720.0	0.99	1.00	0.153	0.150	0.193	Y	LIQUEFIABLE	0.792	2	0.24
7	14	5	15.0			0	14.0	14.0	840.0	777.6	0.99	1.00	0.153	0.150	0.208	Y	LIQUEFIABLE	0.735	2	0.24
8	14	5	15.0			0	14.0	14.0	960.0	835.2	0.98	1.00	0.153	0.150	0.221	Y	LIQUEFIABLE	0.692	2	0.24
9	14	5	15.0			0	14.0	14.0	1080.0	892.8	0.98	1.00	0.153	0.150	0.232	Y	LIQUEFIABLE	0.659	2	0.24
10	28	5	15.0			0	28.0	28.0	1200.0	950.4	0.98	1.00	0.349	0.370	0.241	Y	NL	1.447	0	0
11	28	5	15.0			0	28.0	28.0	1320.0	1008.0	0.98	1.00	0.349	0.370	0.250	Y	NL	1.398	0	0
12	28	5	15.0			0	28.0	28.0	1440.0	1065.6	0.97	1.00	0.349	0.370	0.257	Y	NL	1.358	0	0
13	28	5	15.0			0	28.0	28.0	1560.0	1123.2	0.97	1.00	0.349	0.370	0.264	Y	NL	1.324	0	0
14	28	5	15.0			0	28.0	28.0	1680.0	1180.8	0.97	1.00	0.349	0.370	0.270	Y	NL	1.295	0	0
15	28	5	15.0			0	28.0	28.0	1800.0	1238.4	0.97	1.00	0.349	0.370	0.275	Y	NL	1.271	0	0

Total Settlement, S_{LIQ} (in.) = 0.96

Total Liquefiable Layers = 15

FIGURE D-3



Boring B-2

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LIQUEFACTION - VOLUMETRIC STRAIN

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SANTEE, CALIFORNIA

TEM			07-17-2019	PROJECT NO. G2298-32-01	D-4

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Liquefaction Analysis Using SPT

References
1. *Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10*
2. *Seed, et al, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistant Framework, 2003.*

Project Name:	Carlton Oaks	Boring:	B-3
Project Number:	G2298-32-01		

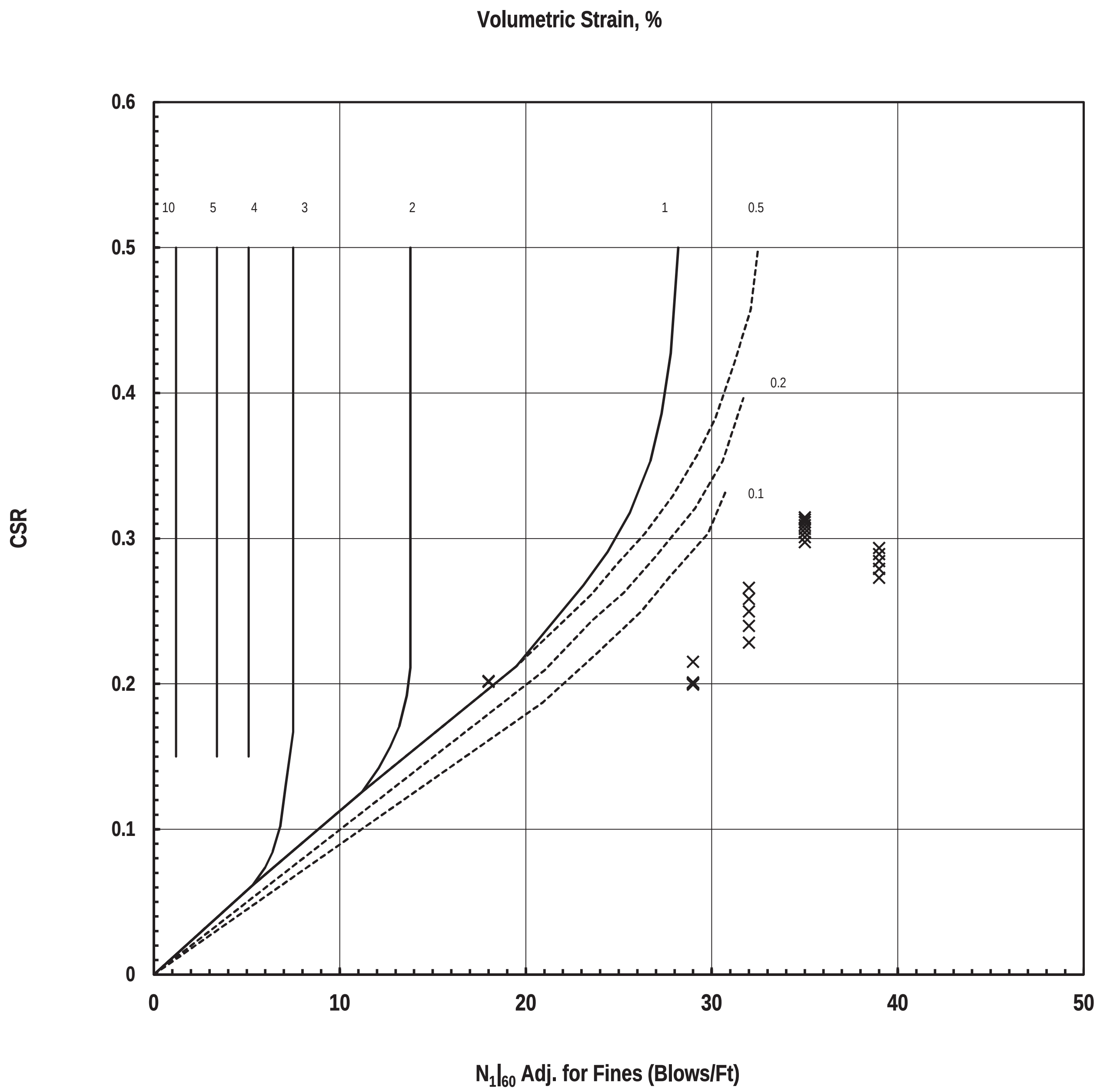
PGAm	0.386	Include Kσ (Y/N)	N
Modal Magnitude	6.80	Use NCEER CRR7.5 (1) or Rauch CRR7.5 (2)	1
Groundwater Depth, Ft	6.0	Minimum Factor of Safety for Liquefaction	1
Reference Pressure, p_a	1000		
Unit Weight of Water	62.4		
Soil Unit Weight, pcf	120		

			Enter for Fine-Grained Materials				Old	New							MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$				SP117 Graph	
Depth, ft	**$N_1\|_{60}$**	**Fines Content, FC (%)**	**Water Content, w_C (%)**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	**$N_1\|_{60}$, Adj. for Fines**	**$N_1\|_{60}$, Adj. for Fines**	**σ, psf**	**σ', psf**	**r_d**	**K_σ**	**NCEER $CRR_{7.5}$**	**RAUCH $CRR_{7.5}$**	**CSR M=7.5**	**Fines Liquefiable (Y/N)**	**Liquefaction Potential**	**Factor of Safety**	**Volumetric Strain, %**	**Settlement, in.**
1	18	5	15.0			0	18.0	18.0	120.0	120.0	1.00	1.00	0.196	0.192	0.195	Y	Above GWT	1.002	0	0
2	18	5	15.0			0	18.0	18.0	240.0	240.0	1.00	1.00	0.196	0.192	0.195	Y	Above GWT	1.004	0	0
3	29	5	15.0			0	29.0	29.0	360.0	360.0	0.99	1.00	0.386	0.410	0.194	Y	Above GWT	1.986	0	0
4	29	5	15.0			0	29.0	29.0	480.0	480.0	0.99	1.00	0.386	0.410	0.194	Y	Above GWT	1.991	0	0
5	29	5	15.0			0	29.0	29.0	600.0	600.0	0.99	1.00	0.386	0.410	0.193	Y	Above GWT	1.996	0	0
6	29	5	15.0			0	29.0	29.0	720.0	720.0	0.99	1.00	0.386	0.410	0.193	Y	NL	2.000	0	0
7	29	5	15.0			0	29.0	29.0	840.0	777.6	0.99	1.00	0.386	0.410	0.208	Y	NL	1.856	0	0
8	32	5	15.0			0	32.0	32.0	960.0	835.2	0.98	1.00	0.800	0.800	0.221	Y	NL	3.624	0	0
9	32	5	15.0			0	32.0	32.0	1080.0	892.8	0.98	1.00	0.800	0.800	0.232	Y	NL	3.451	0	0
10	32	5	15.0			0	32.0	32.0	1200.0	950.4	0.98	1.00	0.800	0.800	0.241	Y	NL	3.313	0	0
11	32	5	15.0			0	32.0	32.0	1320.0	1008.0	0.98	1.00	0.800	0.800	0.250	Y	NL	3.202	0	0
12	32	5	15.0			0	32.0	32.0	1440.0	1065.6	0.97	1.00	0.800	0.800	0.257	Y	NL	3.109	0	0
13	39	5	15.0			0	39.0	39.0	1560.0	1123.2	0.97	1.00	0.800	0.800	0.264	Y	NL	3.032	0	0
14	39	5	15.0			0	39.0	39.0	1680.0	1180.8	0.97	1.00	0.800	0.800	0.270	Y	NL	2.966	0	0
15	39	5	15.0			0	39.0	39.0	1800.0	1238.4	0.97	1.00	0.800	0.800	0.275	Y	NL	2.910	0	0
16	39	5	15.0			0	39.0	39.0	1920.0	1296.0	0.97	1.00	0.800	0.800	0.280	Y	NL	2.861	0	0
17	39	5	15.0			0	39.0	39.0	2040.0	1353.6	0.96	1.00	0.800	0.800	0.284	Y	NL	2.819	0	0
18	35	5	15.0			0	35.0	35.0	2160.0	1411.2	0.96	1.00	0.800	0.800	0.288	Y	NL	2.782	0	0
19	35	5	15.0			0	35.0	35.0	2280.0	1468.8	0.96	1.00	0.800	0.800	0.291	Y	NL	2.750	0	0
20	35	5	15.0			0	35.0	35.0	2400.0	1526.4	0.96	1.00	0.800	0.800	0.294	Y	NL	2.723	0	0
21	35	5	15.0			0	35.0	35.0	2520.0	1584.0	0.95	1.00	0.800	0.800	0.296	Y	NL	2.698	0	0
22	35	5	15.0			0	35.0	35.0	2640.0	1641.6	0.95	1.00	0.800	0.800	0.299	Y	NL	2.677	0	0
23	35	5	15.0			0	35.0	35.0	2760.0	1699.2	0.95	1.00	0.800	0.800	0.301	Y	NL	2.659	0	0
24	35	5	15.0			0	35.0	35.0	2880.0	1756.8	0.95	1.00	0.800	0.800	0.303	Y	NL	2.643	0	0
25	35	5	15.0			0	35.0	35.0	3000.0	1814.4	0.94	1.00	0.800	0.800	0.304	Y	NL	2.630	0	0

Total Settlement, S_{LIQ} (in.) = 0

Total Liquefiable Layers = 25

FIGURE D-5



Boring B-3

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TEM			07-17-2019	PROJECT NO. G2298-32-01	D-6

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Liquefaction Analysis Using SPT

References 1. *Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10*

2. *Seed, et al, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistant Framework, 2003.*

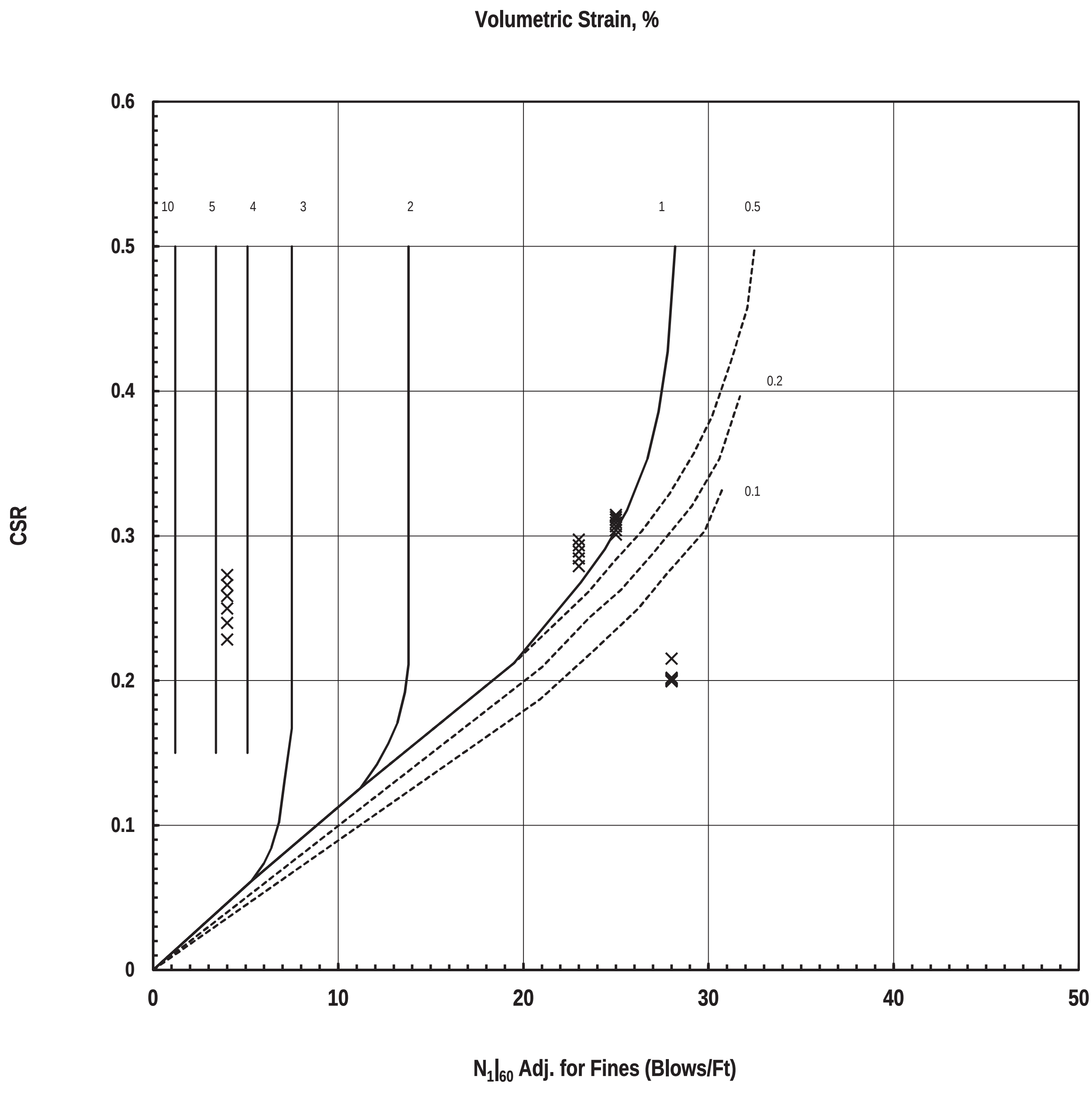
Project Name:	Carlton Oaks	Boring:	B-8
Project Number:	G2298-32-01		
PGAm	0.386	Include $K\sigma$ (Y/N)	N
Modal Magnitude	6.80	Use NCEER CRR7.5 (1) or Rauch CRR7.5 (2)	1
Groundwater Depth, Ft	6.0	Minimum Factor of Safety for Liquefaction	1
Reference Pressure, p_a	1000		
Unit Weight of Water	62.4		
Soil Unit Weight, pcf	120		

				Enter for Fine-Grained Materials			Old	New							MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$				SP117 Graph	
Depth, ft	$N_1\|_{60}$	Fines Content, FC (%)	Water Content, w_C (%)	Liquid Limit	Plastic Limit	Plasticity Index	$N_1\|_{60}$, Adj. for Fines	$N_1\|_{60}$, Adj. for Fines	σ, psf	σ', psf	r_d	K_σ	NCEER $CRR_{7.5}$	RAUCH $CRR_{7.5}$	CSR M=7.5	Fines Liquefiable (Y/N)	Liquefaction Potential	Factor of Safety	Volumetric Strain, %	Settlement, in.
1	28	5	15.0			0	28.0	28.0	120.0	120.0	1.00	1.00	0.349	0.370	0.195	Y	Above GWT	1.790	0	0
2	28	5	15.0			0	28.0	28.0	240.0	240.0	1.00	1.00	0.349	0.370	0.195	Y	Above GWT	1.794	0	0
3	28	5	15.0			0	28.0	28.0	360.0	360.0	0.99	1.00	0.349	0.370	0.194	Y	Above GWT	1.798	0	0
4	28	5	15.0			0	28.0	28.0	480.0	480.0	0.99	1.00	0.349	0.370	0.194	Y	Above GWT	1.802	0	0
5	28	5	15.0			0	28.0	28.0	600.0	600.0	0.99	1.00	0.349	0.370	0.193	Y	Above GWT	1.807	0	0
6	28	5	15.0			0	28.0	28.0	720.0	720.0	0.99	1.00	0.349	0.370	0.193	Y	NL	1.811	0	0
7	28	5	15.0			0	28.0	28.0	840.0	777.6	0.99	1.00	0.349	0.370	0.208	Y	NL	1.680	0	0
8	4	5	15.0			0	4.0	4.0	960.0	835.2	0.98	1.00	0.060	0.065	0.221	Y	LIQUEFIABLE	0.274	4.6	0.552
9	4	5	15.0			0	4.0	4.0	1080.0	892.8	0.98	1.00	0.060	0.065	0.232	Y	LIQUEFIABLE	0.261	4.6	0.552
10	4	5	15.0			0	4.0	4.0	1200.0	950.4	0.98	1.00	0.060	0.065	0.241	Y	LIQUEFIABLE	0.250	4.6	0.552
11	4	5	15.0			0	4.0	4.0	1320.0	1008.0	0.98	1.00	0.060	0.065	0.250	Y	LIQUEFIABLE	0.242	4.6	0.552
12	4	5	15.0			0	4.0	4.0	1440.0	1065.6	0.97	1.00	0.060	0.065	0.257	Y	LIQUEFIABLE	0.235	4.6	0.552
13	4	5	15.0			0	4.0	4.0	1560.0	1123.2	0.97	1.00	0.060	0.065	0.264	Y	LIQUEFIABLE	0.229	4.6	0.552
14	23	5	15.0			0	23.0	23.0	1680.0	1180.8	0.97	1.00	0.255	0.257	0.270	Y	LIQUEFIABLE	0.946	1	0.12
15	23	5	15.0			0	23.0	23.0	1800.0	1238.4	0.97	1.00	0.255	0.257	0.275	Y	LIQUEFIABLE	0.928	1	0.12
16	23	5	15.0			0	23.0	23.0	1920.0	1296.0	0.97	1.00	0.255	0.257	0.280	Y	LIQUEFIABLE	0.913	1	0.12
17	23	5	15.0			0	23.0	23.0	2040.0	1353.6	0.96	1.00	0.255	0.257	0.284	Y	LIQUEFIABLE	0.900	1	0.12
18	23	5	15.0			0	23.0	23.0	2160.0	1411.2	0.96	1.00	0.255	0.257	0.288	Y	LIQUEFIABLE	0.888	1	0.12
19	25	5	15.0			0	25.0	25.0	2280.0	1468.8	0.96	1.00	0.286	0.292	0.291	Y	LIQUEFIABLE	0.982	1	0.12
20	25	5	15.0			0	25.0	25.0	2400.0	1526.4	0.96	1.00	0.286	0.292	0.294	Y	LIQUEFIABLE	0.972	1	0.12
21	25	5	15.0			0	25.0	25.0	2520.0	1584.0	0.95	1.00	0.286	0.292	0.296	Y	LIQUEFIABLE	0.963	1	0.12
22	25	5	15.0			0	25.0	25.0	2640.0	1641.6	0.95	1.00	0.286	0.292	0.299	Y	LIQUEFIABLE	0.955	1	0.12
23	25	5	15.0			0	25.0	25.0	2760.0	1699.2	0.95	1.00	0.286	0.292	0.301	Y	LIQUEFIABLE	0.949	1	0.12
24	25	5	15.0			0	25.0	25.0	2880.0	1756.8	0.95	1.00	0.286	0.292	0.303	Y	LIQUEFIABLE	0.943	1	0.12
25	25	5	15.0			0	25.0	25.0	3000.0	1814.4	0.94	1.00	0.286	0.292	0.304	Y	LIQUEFIABLE	0.939	1	0.12

Total Settlement, S_{LIQ} (in.) = 4.75

Total Liquefiable Layers = 25

FIGURE D-7



Boring B-8

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07-17-2019	PROJECT NO. G2298-32-01	D-8

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Liquefaction Analysis Using SPT

References
1. *Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10*
2. *Seed, et al, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistant Framework, 2003.*

Project Name:	Carlton Oaks	Boring:	B-9
Project Number:	G2298-32-01		

PGAm	0.386
Modal Magnitude	6.80
Groundwater Depth, Ft	3.0
Reference Pressure, p_a	1000
Unit Weight of Water	62.4
Soil Unit Weight, pcf	120

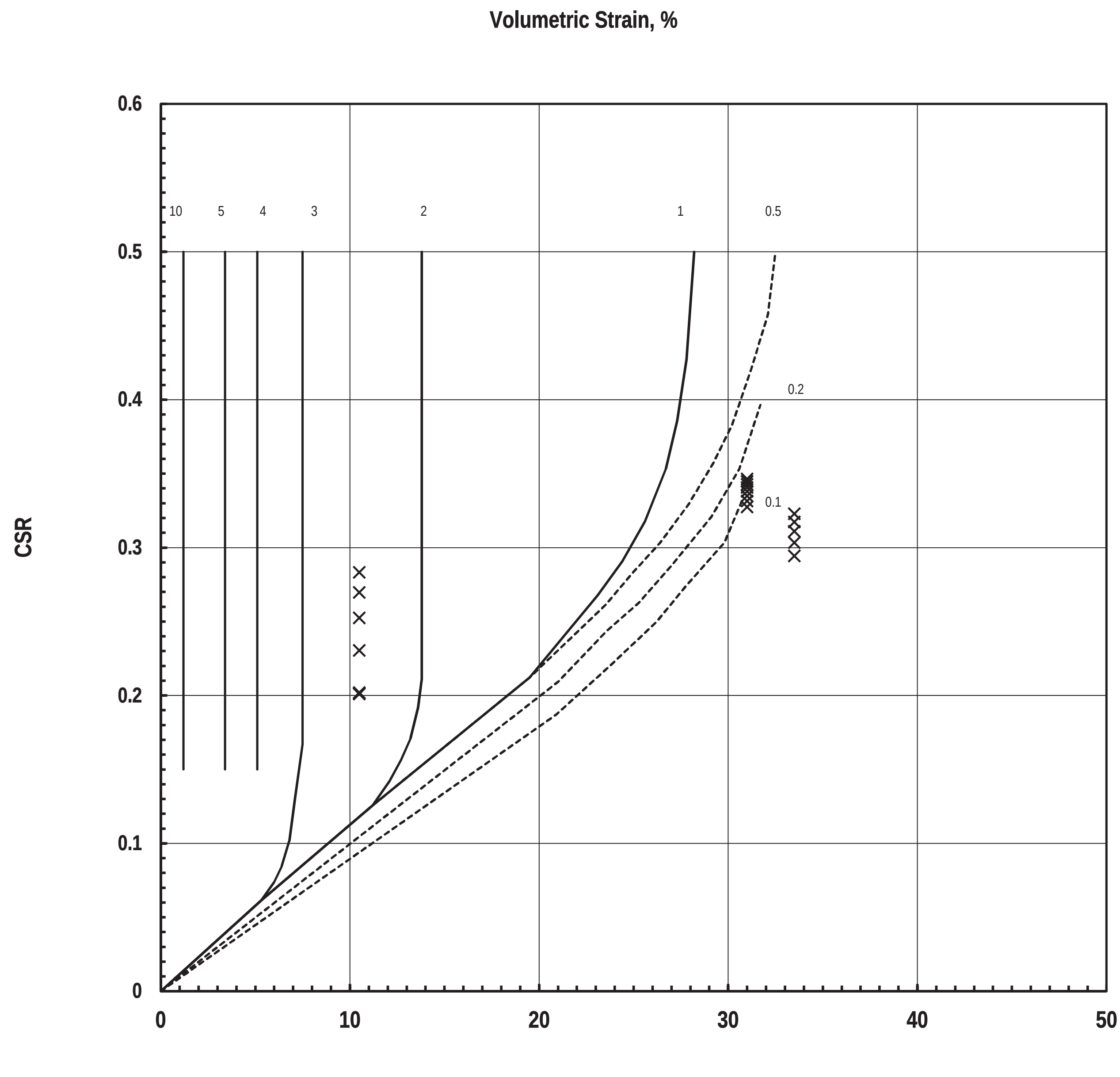
Include $K\sigma$ (Y/N)	N
Use NCEER CRR7.5 (1) or Rauch CRR7.5 (2)	1
Minimum Factor of Safety for Liquefaction	1

			Enter for Fine-Grained Materials				Old	New							MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$				SP117 Graph	
Depth, ft	**$N_1\|_{60}$**	**Fines Content, FC (%)**	**Water Content, w_C (%)**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	**$N_1\|_{60}$, Adj. for Fines**	**$N_1\|_{60}$, Adj. for Fines**	**σ, psf**	**σ', psf**	**r_d**	**K_σ**	**NCEER $CRR_{7.5}$**	**RAUCH $CRR_{7.5}$**	**CSR M=7.5**	**Fines Liquefiable (Y/N)**	**Liquefaction Potential**	**Factor of Safety**	**Volumetric Strain, %**	**Settlement, in.**
1	10.5	5	15.0			0	10.5	10.5	120.0	120.0	1.00	1.00	0.115	0.118	0.195	Y	Above GWT	0.589	0	0
2	10.5	5	15.0			0	10.5	10.5	240.0	240.0	1.00	1.00	0.115	0.118	0.195	Y	Above GWT	0.590	0	0
3	10.5	5	15.0			0	10.5	10.5	360.0	360.0	0.99	1.00	0.115	0.118	0.194	Y	LIQUEFIABLE	0.591	2.4	0.288
4	10.5	5	15.0			0	10.5	10.5	480.0	417.6	0.99	1.00	0.115	0.118	0.223	Y	LIQUEFIABLE	0.516	2.4	0.288
5	10.5	5	15.0			0	10.5	10.5	600.0	475.2	0.99	1.00	0.115	0.118	0.244	Y	LIQUEFIABLE	0.471	2.4	0.288
6	10.5	5	15.0			0	10.5	10.5	720.0	532.8	0.99	1.00	0.115	0.118	0.261	Y	LIQUEFIABLE	0.441	2.4	0.288
7	10.5	5	15.0			0	10.5	10.5	840.0	590.4	0.99	1.00	0.115	0.118	0.274	Y	LIQUEFIABLE	0.420	2.4	0.288
8	33.5	5	15.0			0	33.5	33.5	960.0	648.0	0.98	1.00	0.800	0.800	0.285	Y	NL	2.811		--
9	33.5	5	15.0			0	33.5	33.5	1080.0	705.6	0.98	1.00	0.800	0.800	0.293	Y	NL	2.727		--
10	33.5	5	15.0			0	33.5	33.5	1200.0	763.2	0.98	1.00	0.800	0.800	0.301	Y	NL	2.661		--
11	33.5	5	15.0			0	33.5	33.5	1320.0	820.8	0.98	1.00	0.800	0.800	0.307	Y	NL	2.607		--
12	33.5	5	15.0			0	33.5	33.5	1440.0	878.4	0.97	1.00	0.800	0.800	0.312	Y	NL	2.563		--
13	31	5	15.0			0	31.0	31.0	1560.0	936.0	0.97	1.00	0.800	0.800	0.317	Y	NL	2.526		--
14	31	5	15.0			0	31.0	31.0	1680.0	993.6	0.97	1.00	0.800	0.800	0.321	Y	NL	2.496		--
15	31	5	15.0			0	31.0	31.0	1800.0	1051.2	0.97	1.00	0.800	0.800	0.324	Y	NL	2.470		--
16	31	5	15.0			0	31.0	31.0	1920.0	1108.8	0.97	1.00	0.800	0.800	0.327	Y	NL	2.448		--
17	31	5	15.0			0	31.0	31.0	2040.0	1166.4	0.96	1.00	0.800	0.800	0.329	Y	NL	2.429		--
18	31	5	15.0			0	31.0	31.0	2160.0	1224.0	0.96	1.00	0.800	0.800	0.331	Y	NL	2.413		--
19	31	5	15.0			0	31.0	31.0	2280.0	1281.6	0.96	1.00	0.800	0.800	0.333	Y	NL	2.400		--
20	31	5	15.0			0	31.0	31.0	2400.0	1339.2	0.96	1.00	0.800	0.800	0.335	Y	NL	2.389		--

Total Settlement, S_{LIQ} (in.) = 1.44

Total Liquefiable Layers = 7

FIGURE D-9



Boring B-9

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07-17-2019	PROJECT NO. G2298-32-01	D-10

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Liquefaction Analysis Using SPT

References 1. *Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10*

2. *Seed, et al, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistant Framework, 2003.*

Project Name:	Carlton Oaks	Boring:	B-10
Project Number:	G2298-32-01		

PGAm	0.386
Modal Magnitude	6.80
Groundwater Depth, Ft	5.0
Reference Pressure, p_a	1000
Unit Weight of Water	62.4
Soil Unit Weight, pcf	120

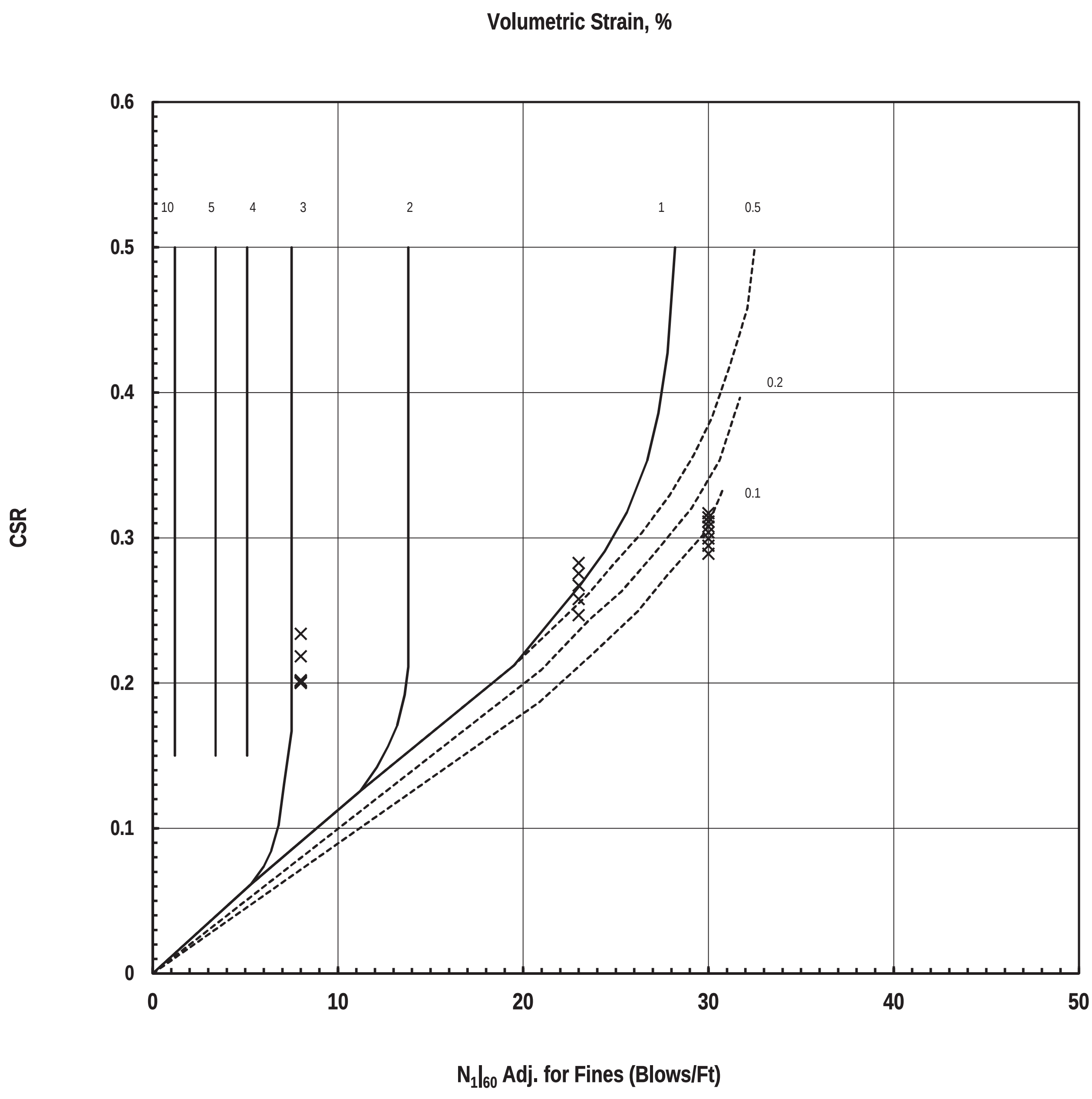
Include $K\sigma$ (Y/N)	N
Use NCEER CRR7.5 (1) or Rauch CRR7.5 (2)	1
Minimum Factor of Safety for Liquefaction	1.1

			Enter for Fine-Grained Materials				Old	New							MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$				SP117 Graph	
Depth, ft	$N_1\|_{60}$	Fines Content, FC (%)	Water Content, w_C (%)	Liquid Limit	Plastic Limit	Plasticity Index	$N_1\|_{60}$, Adj. for Fines	$N_1\|_{60}$, Adj. for Fines	σ, psf	σ', psf	r_d	K_σ	NCEER $CRR_{7.5}$	RAUCH $CRR_{7.5}$	CSR M=7.5	Fines Liquefiable (Y/N)	Liquefaction Potential	Factor of Safety	Volumetric Strain, %	Settlement, in.
1	8	5	15.0			0	8.0	8.0	120.0	120.0	1.00	1.00	0.090	0.096	0.195	Y	Above GWT	0.459	0	0
2	8	5	15.0			0	8.0	8.0	240.0	240.0	1.00	1.00	0.090	0.096	0.195	Y	Above GWT	0.460	0	0
3	8	5	15.0			0	8.0	8.0	360.0	360.0	0.99	1.00	0.090	0.096	0.194	Y	Above GWT	0.461	0	0
4	8	5	15.0			0	8.0	8.0	480.0	480.0	0.99	1.00	0.090	0.096	0.194	Y	Above GWT	0.462	0	0
5	8	5	15.0			0	8.0	8.0	600.0	600.0	0.99	1.00	0.090	0.096	0.193	Y	LIQUEFIABLE	0.464	3	0.36
6	8	5	15.0			0	8.0	8.0	720.0	657.6	0.99	1.00	0.090	0.096	0.211	Y	LIQUEFIABLE	0.424	3	0.36
7	8	5	15.0			0	8.0	8.0	840.0	715.2	0.99	1.00	0.090	0.096	0.226	Y	LIQUEFIABLE	0.397	3	0.36
8	23	5	15.0			0	23.0	23.0	960.0	772.8	0.98	1.00	0.255	0.257	0.239	Y	LIQUEFIABLE	1.070	0.2	0.024
9	23	5	15.0			0	23.0	23.0	1080.0	830.4	0.98	1.00	0.255	0.257	0.249	Y	LIQUEFIABLE	1.024	0.2	0.024
10	23	5	15.0			0	23.0	23.0	1200.0	888.0	0.98	1.00	0.255	0.257	0.258	Y	LIQUEFIABLE	0.988	0.5	0.06
11	23	5	15.0			0	23.0	23.0	1320.0	945.6	0.98	1.00	0.255	0.257	0.266	Y	LIQUEFIABLE	0.958	0.5	0.06
12	23	5	15.0			0	23.0	23.0	1440.0	1003.2	0.97	1.00	0.255	0.257	0.273	Y	LIQUEFIABLE	0.934	1	0.12
13	30	5	15.0			0	30.0	30.0	1560.0	1060.8	0.97	1.00	0.498	0.468	0.279	Y	NL	1.782		--
14	30	5	15.0			0	30.0	30.0	1680.0	1118.4	0.97	1.00	0.498	0.468	0.285	Y	NL	1.748		--
15	30	5	15.0			0	30.0	30.0	1800.0	1176.0	0.97	1.00	0.498	0.468	0.290	Y	NL	1.720		--
16	30	5	15.0			0	30.0	30.0	1920.0	1233.6	0.97	1.00	0.498	0.468	0.294	Y	NL	1.695		--
17	30	5	15.0			0	30.0	30.0	2040.0	1291.2	0.96	1.00	0.498	0.468	0.298	Y	NL	1.674		--
18	30	5	15.0			0	30.0	30.0	2160.0	1348.8	0.96	1.00	0.498	0.468	0.301	Y	NL	1.655		--
19	30	5	15.0			0	30.0	30.0	2280.0	1406.4	0.96	1.00	0.498	0.468	0.304	Y	NL	1.639		--
20	30	5	15.0			0	30.0	30.0	2400.0	1464.0	0.96	1.00	0.498	0.468	0.306	Y	NL	1.625		--
21																				
22																				
23																				
24																				
25																				

Total Settlement, S_{LIQ} (in.) = 1.37

Total Liquefiable Layers = 12

FIGURE D-11



Boring B-10

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07-17-2019	PROJECT NO. G2298-32-01	D-12

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Liquefaction Analysis Using SPT

References 1. *Youd, et al, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Environmental Engineering, October, 2001, Vol. 127, No. 10*

2. *Seed, et al, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistant Framework, 2003.*

Project Name:	Carlton Oaks	Boring:	B-11
Project Number:	G2298-32-01		

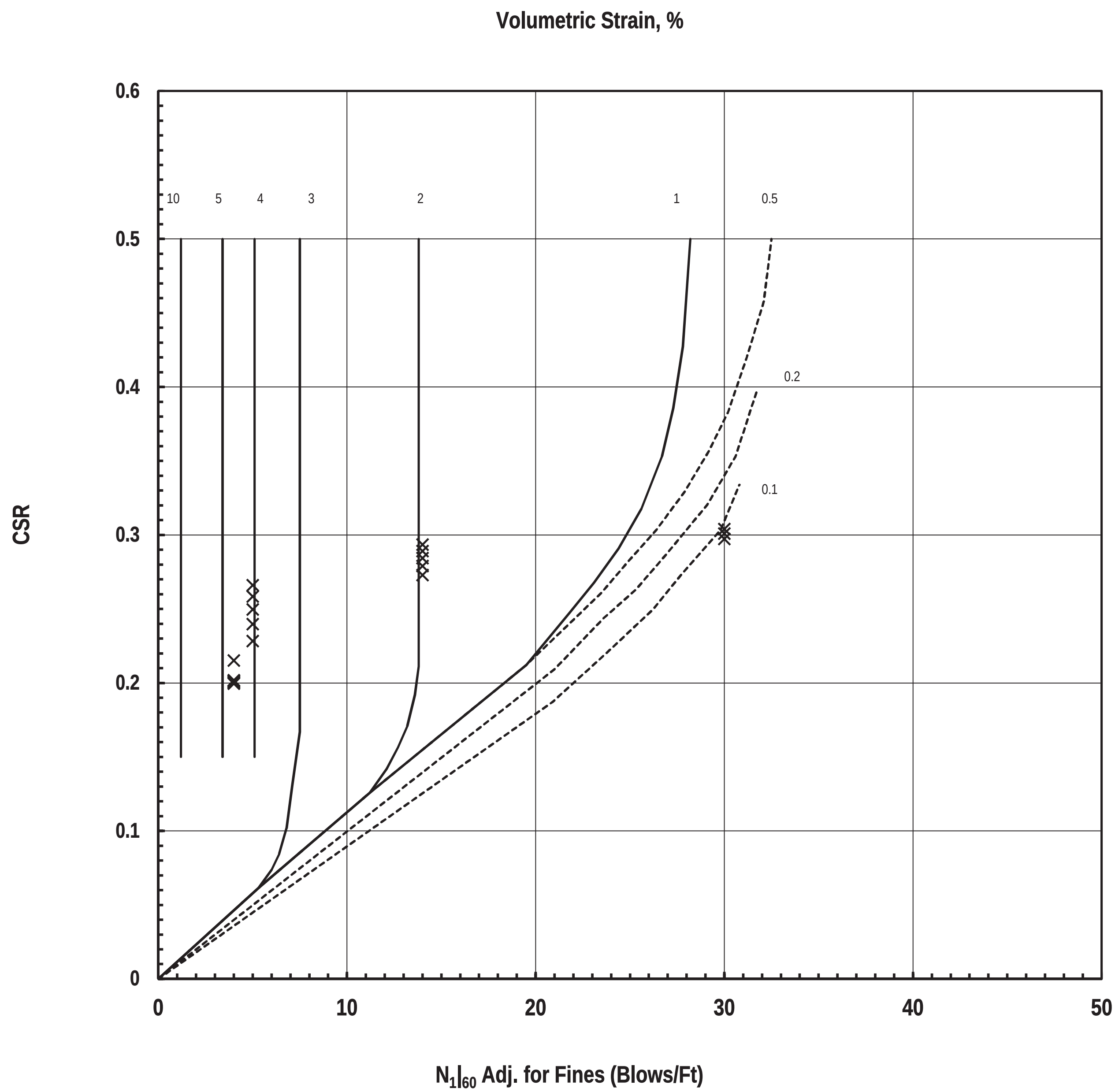
PGAm	0.386	Include Kσ (Y/N)	N
Modal Magnitude	6.89	Use NCEER CRR7.5 (1) or Rauch CRR7.5 (2)	1
Groundwater Depth, Ft	6.0	Minimum Factor of Safety for Liquefaction	1.1
Reference Pressure, p_a	1000		
Unit Weight of Water	62.4		
Soil Unit Weight, pcf	120		

			Enter for Fine-Grained Materials				Old	New						MWF Idriss(1997) = $(M)^{2.56}/10^{2.24}$					SP117 Graph	
Depth, ft	**$N_1\|_{60}$**	**Fines Content, FC (%)**	**Water Content, w_C (%)**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	**$N_1\|_{60}$, Adj. for Fines**	**$N_1\|_{60}$, Adj. for Fines**	**σ, psf**	**σ', psf**	**r_d**	**K_σ**	**NCEER $CRR_{7.5}$**	**RAUCH $CRR_{7.5}$**	**CSR M=7.5**	**Fines Liquefiable (Y/N)**	**Liquefaction Potential**	**Factor of Safety**	**Volumetric Strain, %**	**Settlement, in.**
1	4	5	15.0			0	4.0	4.0	120.0	120.0	1.00	1.00	0.060	0.065	0.202	Y	Above GWT	0.299	0	0
2	4	5	15.0			0	4.0	4.0	240.0	240.0	1.00	1.00	0.060	0.065	0.201	Y	Above GWT	0.300	0	0
3	4	5	15.0			0	4.0	4.0	360.0	360.0	0.99	1.00	0.060	0.065	0.201	Y	Above GWT	0.301	0	0
4	4	5	15.0			0	4.0	4.0	480.0	480.0	0.99	1.00	0.060	0.065	0.200	Y	Above GWT	0.301	0	0
5	4	5	15.0			0	4.0	4.0	600.0	600.0	0.99	1.00	0.060	0.065	0.200	Y	Above GWT	0.302	0	0
6	4	5	15.0			0	4.0	4.0	720.0	720.0	0.99	1.00	0.060	0.065	0.200	Y	LIQUEFIABLE	0.303	4.6	0.552
7	4	5	15.0			0	4.0	4.0	840.0	777.6	0.99	1.00	0.060	0.065	0.215	Y	LIQUEFIABLE	0.281	4.6	0.552
8	5	5	15.0			0	5.0	5.0	960.0	835.2	0.98	1.00	0.066	0.072	0.228	Y	LIQUEFIABLE	0.289	4	0.48
9	5	5	15.0			0	5.0	5.0	1080.0	892.8	0.98	1.00	0.066	0.072	0.240	Y	LIQUEFIABLE	0.275	4	0.48
10	5	5	15.0			0	5.0	5.0	1200.0	950.4	0.98	1.00	0.066	0.072	0.250	Y	LIQUEFIABLE	0.264	4	0.48
11	5	5	15.0			0	5.0	5.0	1320.0	1008.0	0.98	1.00	0.066	0.072	0.258	Y	LIQUEFIABLE	0.256	4	0.48
12	5	5	15.0			0	5.0	5.0	1440.0	1065.6	0.97	1.00	0.066	0.072	0.266	Y	LIQUEFIABLE	0.248	4	0.48
13	14	5	15.0			0	14.0	14.0	1560.0	1123.2	0.97	1.00	0.153	0.150	0.273	Y	LIQUEFIABLE	0.560	2	0.24
14	14	5	15.0			0	14.0	14.0	1680.0	1180.8	0.97	1.00	0.153	0.150	0.279	Y	LIQUEFIABLE	0.548	2	0.24
15	14	5	15.0			0	14.0	14.0	1800.0	1238.4	0.97	1.00	0.153	0.150	0.284	Y	LIQUEFIABLE	0.537	2	0.24
16	14	5	15.0			0	14.0	14.0	1920.0	1296.0	0.97	1.00	0.153	0.150	0.289	Y	LIQUEFIABLE	0.528	2	0.24
17	14	5	15.0			0	14.0	14.0	2040.0	1353.6	0.96	1.00	0.153	0.150	0.294	Y	LIQUEFIABLE	0.520	2	0.24
18	30	5	15.0			0	30.0	30.0	2160.0	1411.2	0.96	1.00	0.498	0.468	0.297	Y	NL	1.674	0	0
19	30	5	15.0			0	30.0	30.0	2280.0	1468.8	0.96	1.00	0.498	0.468	0.301	Y	NL	1.655	0	0
20	30	5	15.0			0	30.0	30.0	2400.0	1526.4	0.96	1.00	0.498	0.468	0.304	Y	NL	1.638	0	0
21																				
22																				
23																				
24																				
25																				

Total Settlement, S_{LIQ} (in.) = 4.70

Total Liquefiable Layers = 20

FIGURE D-13



Boring B-11

GEOCON
INCORPORATED
GEOTECHNICAL CONSULTANTS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121-2974
PHONE 858 558-6900 - FAX 858 558-6159

LIQUEFACTION - VOLUMETRIC STRAIN

CARLTON OAKS GOLF COURSE
HOTEL AND ASISTED LIVING SITES
SANTEE, CALIFORNIA

MEE			07-17-2019	PROJECT NO. G2298-32-01	D-14

Carlton Oaks Hotel and Assisted Living Sites
Project No. G2298-32-01
15 ft high Fill Slope above liquefiable soil
Name: AAc1 Layered B8.gsz
Date: 07/16/2019 Time: 08:21:20 AM

Proposed Condition
15 ft high fill slope above liquefiable soils
Using Boring B-8

Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)
	Friars Formation	130		500	35
	Qal - Alluvium - Residual Strength (N-Value=4)	120	140		
	Qal - Alluvium (Non-Liquefiable)	120		200	32
	Qcf - Compacted Fill	125		200	32

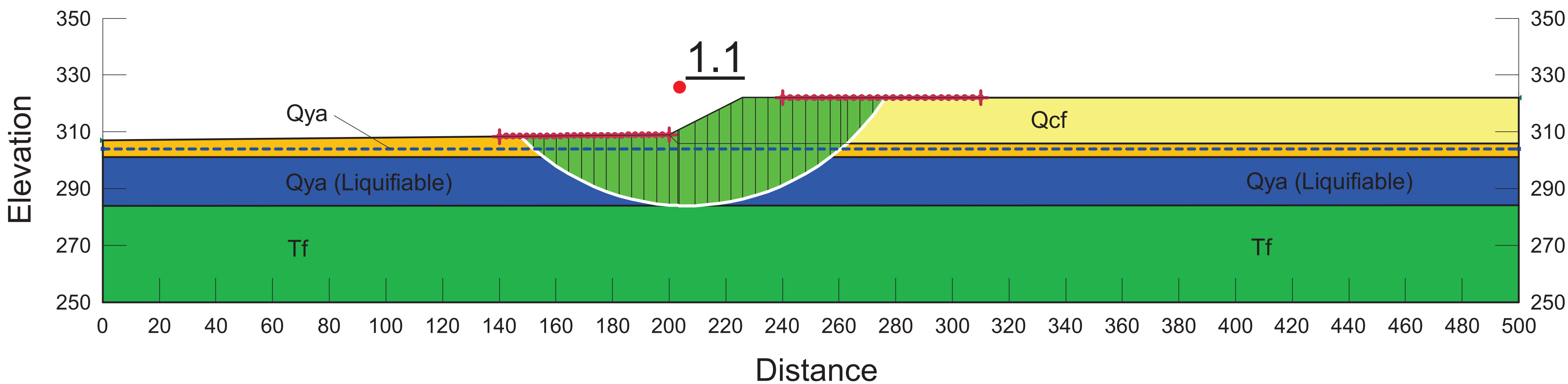


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Figure D-15

Carlton Oaks Hotel and Assisted Living Sites
Project No. G2298-32-01
15 ft high Fill Slope above liquefiable soil
Name: AAc1 Layered B9.gsz
Date: 07/16/2019 Time: 08:45:48 AM

Proposed Condition
15 ft high fill slope above liquefiable soils
Using Boring B-9

Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)
	Friars Formation	130		500	35
	Qcf - Compacted Fill	125		200	32
	Qya - Alluvium - Residual Strength (N-Value=10)	120	300		
	Qya - Alluvium (Non-Liquefiable)	120		200	32

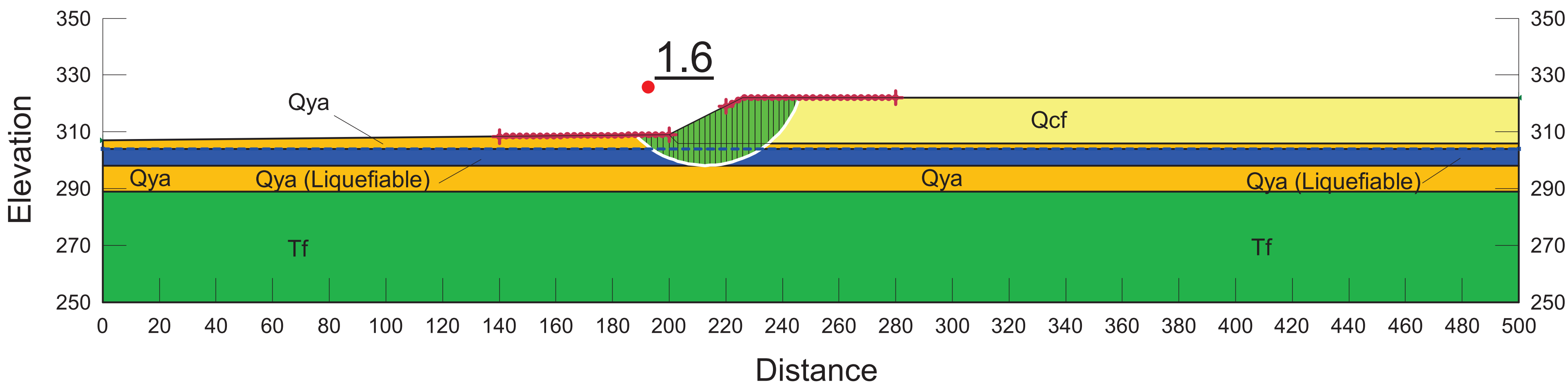

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Figure D-16

Carlton Oaks Hotel and Assisted Living Sites
Project No. G2298-32-01
15 ft high Fill Slope above liquefiable soil
Name: AAc1 Layered B11.gsz
Date: 07/16/2019 Time: 08:58:01 AM

Proposed Condition
15 ft high fill slope above liquefiable soils
Using Boring B-11

Static Analysis

Color	Name	Unit Weight (pcf)	Cohesion (psf)	Cohesion' (psf)	Phi' (°)
	Friars Formation	130		500	35
	Qcf - Compacted Fill	125		200	32
	Qya - Alluvium - Residual Strength (N-Value=14)	120	600		
	Qya - Alluvium - Residual Strength (N-Value=4)	120	140		
	Qya - Alluvium (Non-Liquefiable)	120		200	32

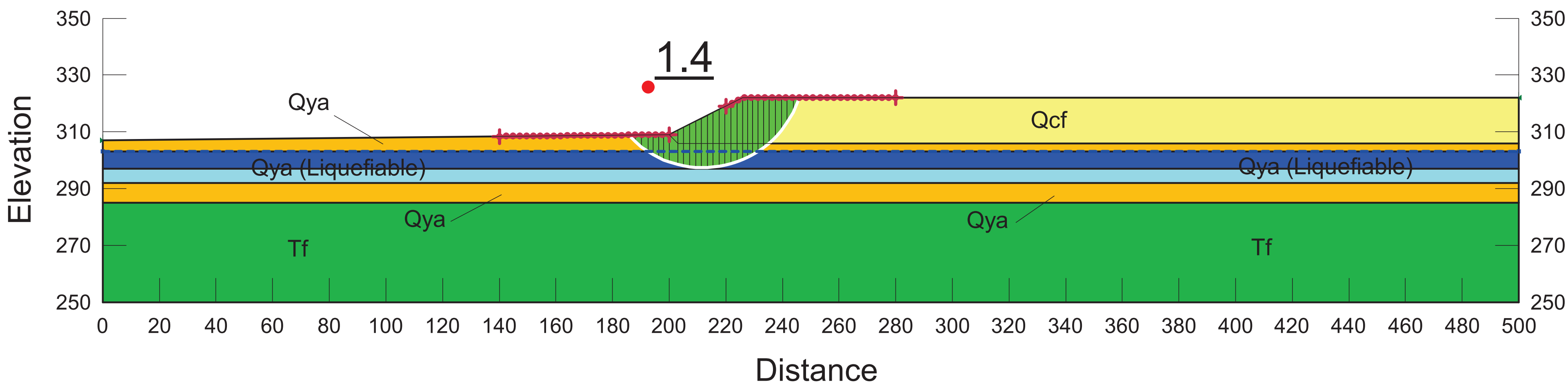

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Figure D-17

APPENDIX E

APPENDIX E

RECOMMENDED GRADING SPECIFICATIONS

FOR

CARLTON OAKS GOLF COURSE
COUNTRY CLUB AND RESORT SITE (PA-3)
SANTEE, CALIFORNIA

PROJECT NO. G2298-32-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, and/or adverse weather result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition.

4. CLEARING AND PREPARING AREAS TO BE FILLED

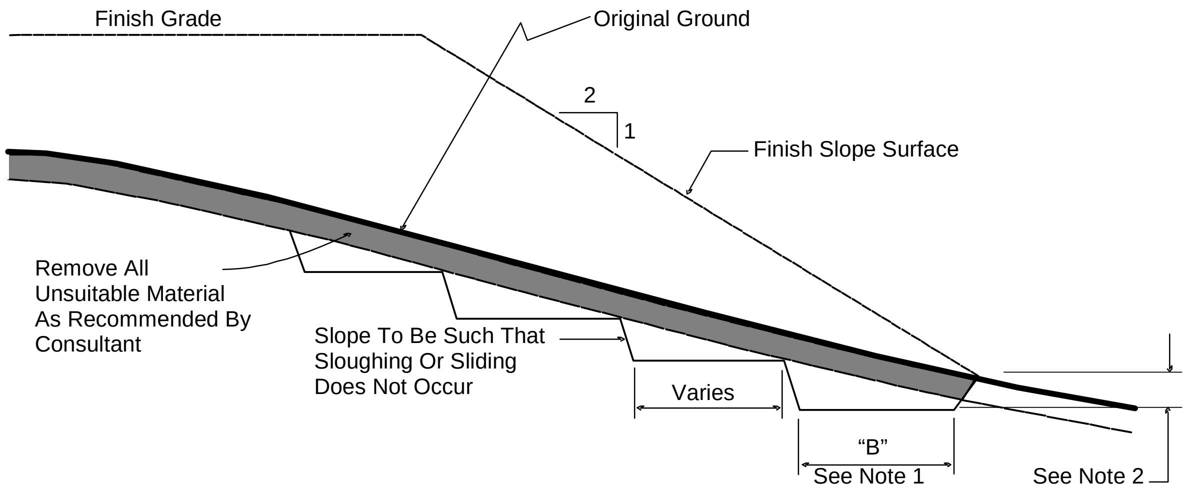
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



No Scale

DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of "passes" have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

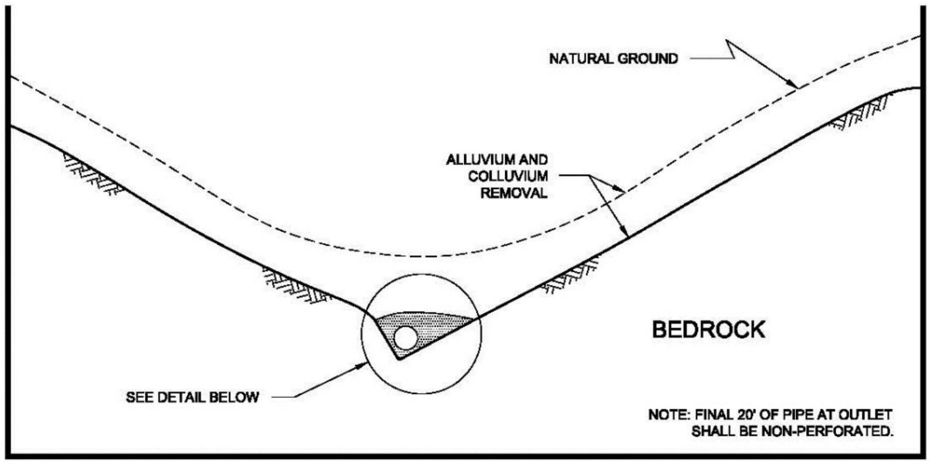
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

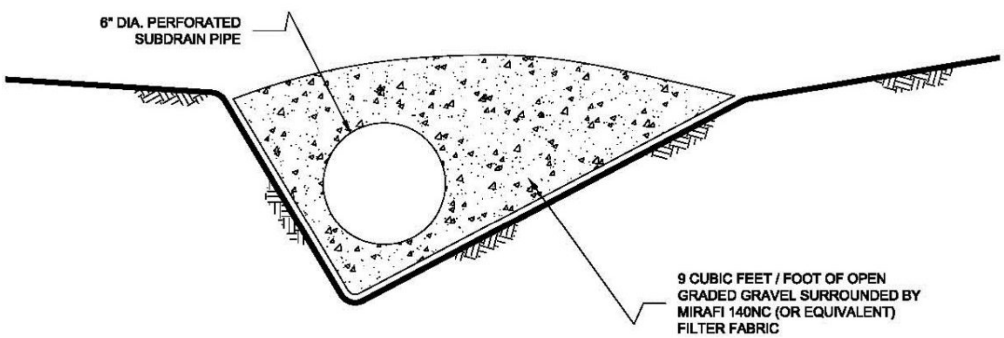
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL





NOTES:

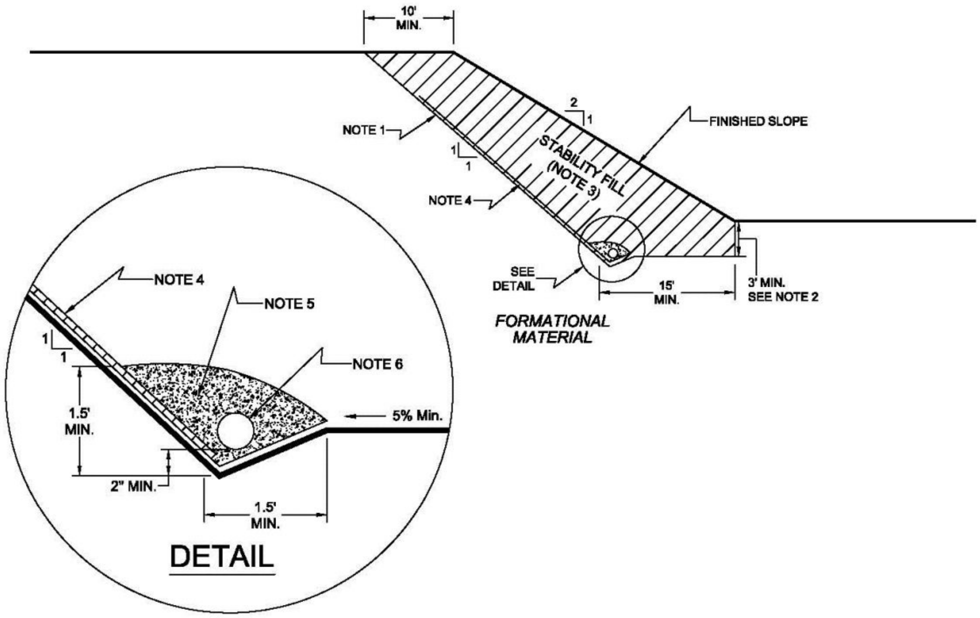
1......6-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2......6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

TYPICAL STABILITY FILL DETAIL



NOTES:

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

NO SCALE

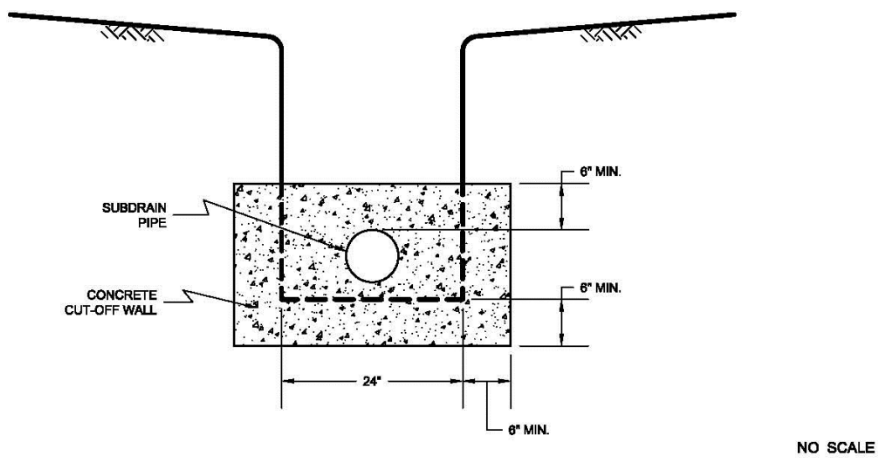
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

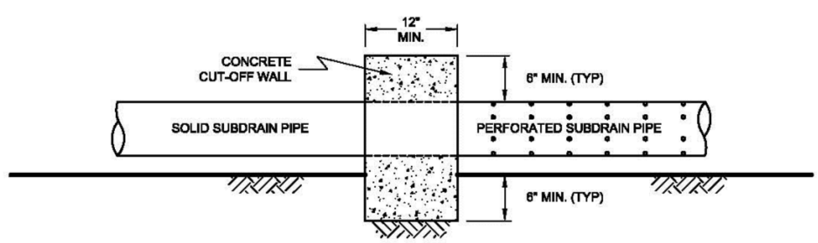
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



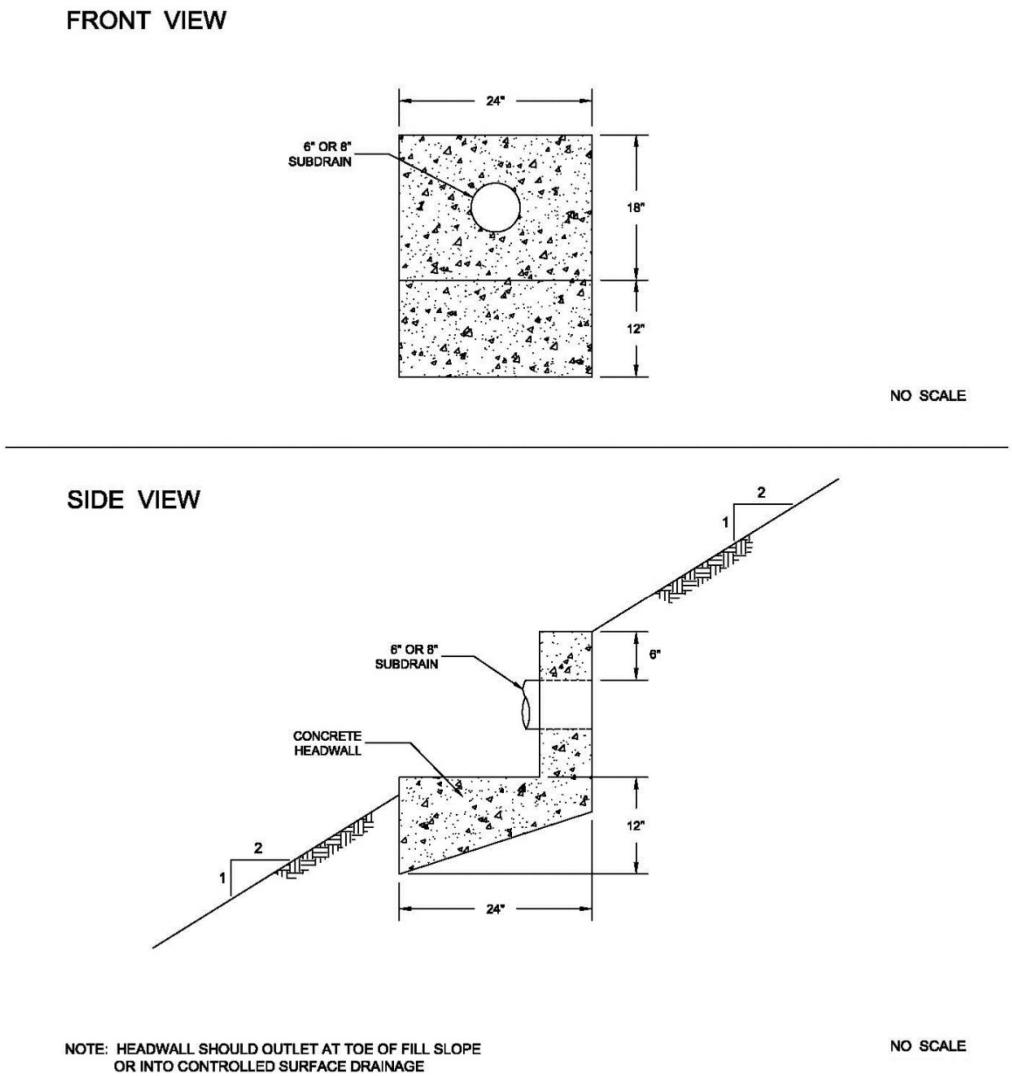
SIDE VIEW



NO SCALE

7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL



7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an "as-built" map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

8. OBSERVATION AND TESTING

8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 Soil and Soil-Rock Fills:

8.6.1.1 Field Density Test, ASTM D 1556, *Density of Soil In-Place By the Sand-Cone Method.*

8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop.*

8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

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14. Federal Emergency Management Agency, *Flood Insurance Rate Map, San Diego County, Map No. 06073C1634, Panel 1634 of 2375*, dated June 19, 1997.

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