GROUP DELTA

GEOTECHNICAL INVESTIGATION REPORT
SANTEE COMMUNITY CENTER
SANTEE, CALIFORNIA

Prepared for:

HMC Architects
8910 University Center Lane, #650
San Diego, CA 92122

Prepared by:

GROUP DELTA CONSULTANTS, INC
32 Mauchly, Suite B
Irvine, California 92618

Group Delta Project Number IR786
June 15, 2022

HMC Architects
8910 University Center Lane, #650
San Diego, CA 92122

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Attention: Kyle Peterson, AIA, LEED AP, DBIA
Managing Manager

Subject: Geotechnical Investigation Report
Santee Community Center
Santee, California

Dear Kyle:

Group Delta Consultants, Inc. (Group Delta) is pleased to submit this geotechnical investigation report for the proposed Santee Community Center Phase 1 project located in the City of Santee, California. This report and our associated geotechnical services were provided in general accordance with our consulting agreement with HMC Group (HMC), dated January 12, 2022.

We appreciate the opportunity to provided geotechnical services for this project. If you have any questions pertaining to this report, or if we can be of further service, please do not hesitate to contact us at (949) 450-2100.

Yours Sincerely,
Group Delta Consultants, Inc.

REGISTERED PROFESSIONAL ENGINEER
MICHAEL JAMES GIVENS
NO. 3145
Exp. 9/30/2023
GEOTECHNICAL
STATE OF CALIFORNIA

Michael Givens, PhD, PE, GE, PG
Associate Engineer

Giovani Valdivia
Staff Engineer

Distribution: Addressee (1 PDF file via email)

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

GEOTECHNICAL INVESTIGATION REPORT
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SANTEE, CALIFORNIA

1.0 INTRODUCTION

This report presents our geotechnical investigation and recommendations for design of the proposed Santee Community Center in Santee, California. The project site is located adjacent to the Cameron Family YMCA at 10123 Riverwalk Drive in the City of Santee. The project site is shown in Figure 1, Site Location Map.

1.1 Project Description

It is our understanding that the proposed building is part of phase one of three phases, consisting of the development of the primary Community Center. The proposed Community Center will have an approximate area of 12,500 square feet consisting of dedicated facilities for teens and seniors, lobby space, multi-purpose rooms, administrative offices, and storage. Other associated improvements include parking facilities and landscape architecture. A conceptual development plan is presented in Figure 2.

The finish floor elevation of the proposed Community Center is not available at the time of preparing this report and it has been assumed at grade (with no basement).

1.2 Scope of Work

The objective of this report was to provide geotechnical recommendations for design and construction for the proposed development. Our scope of work for the project includes the following task:

- Review of the available published geotechnical and geologic reports, maps, and subsurface data for the project site and surrounding area.
- Perform three geotechnical borings to evaluate the subsurface conditions.
- Perform laboratory testing to quantify physical and engineering properties of the subsurface soils.
- Evaluate geologic and seismic hazards including local seismicity, surface fault rupture, ground shaking, liquefaction, and other considered geologic hazards.
- Evaluate seismic design parameters in accordance with the 2022 California Building Code (CBC).
- Provide geotechnical recommendations for site development earthwork, including removal of unsuitable soils, excavations, placement of compacted fill/backfill, and reuse of excavated materials.
- Provide geotechnical recommendations for support of the proposed structures.

- Evaluate the corrosivity of the on-site soils.
- Provide pavement design recommendations.
- Prepare this report presenting the results of our investigation, conclusions, and recommendations.

1.3 Site Description

The project site is a 12,500 square feet rectangular shaped area east of Cameron Family YMCA, within the parking lot area. The project site is currently occupied by an asphalt concrete (AC) paved parking lot used for the YMCA. The site is relatively flat with an elevation of approximately 345 feet above mean sea level. The southern and eastern limits of the property has slopes that descend into the Woodglen Vista Creek.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 Field Exploration

The field exploration program was performed on February 17, 2022 and consisted of drilling three (3) hollow stem auger borings (B-1 through B-3) to a maximum depth of 50 feet below ground surface (bgs). The locations of these explorations are shown in Figure 3. A detailed explanation of the field exploration including boring logs is presented in Appendix A.

2.2 Laboratory Testing

Laboratory tests were performed on selected soil samples obtained during the field exploration to help characterize the subsurface materials and to evaluate their index and engineering properties. The performed tests are identified on the boring logs in Appendix A. A detailed description of the laboratory testing program including test results is presented in Appendix B. The laboratory testing program consisted of the following:

- Moisture content and dry density
- Grain size distribution and percent passing No. 200 sieve
- Atterberg Limits
- Direct Shear
- Expansion index test
- Corrosivity tests (pH, sulfates, chlorides and electrical resistivity)
- R-Value test

3.0 GEOLOGY AND SUBSURFACE CONDITIONS

3.1 Geology

The subject site is located within the Peninsular Ranges geomorphic province of southern California. The Peninsular Ranges are characterized by a series of northwest trending mountain ranges separated by valleys, with a coastal plain of subdued landforms. The mountain ranges are underlain primarily by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the southern California batholith, while the coastal plain is underlain by subsequently deposited marine and nonmarine sedimentary formations.

In general the project site is underlain at depth by early Cretaceous age undivided tonalite and granodiorite (Map Symbol – Kgr, referred to as Granitic Rock in this report) covered by Holocene young alluvial deposits (Map Symbol – Qya). The young alluvial deposits are associated with the San Diego River and generally consist of poorly to well graded sandy soils. Not presented in the geologic map, but identified during our field investigation was surficial fill material. A regional geologic map of the project site is illustrated in Figure 4 and the pertinent units are discussed below.

3.2 Fill

Undocumented fill is soil where there is no record of compaction testing and/or observation by a Geotechnical Engineer's representative. Undocumented fill depths ranged from 4 to 10 feet were encountered in our exploratory borings. As described in the boring logs, the fill generally consists of clayey sand (SC) with approximately 38 to 42 percent fines with trace to little gravel. Deeper fills could be present anywhere within the site and could locally extend deeper.

3.3 Alluvium

The Holocene-age young alluvial fan deposits (Map Symbol – Qya) are associated with the San Diego River. The alluvial deposits at the project site generally consist of loose to medium dense sands (SM, SP-SM, SW, SW-SM) to a depth of 28 feet below ground surface.

3.4 Granitic Rock

Early Cretaceous-age granitic rock (Map Symbol Kgr) comprised of tonalite and granodiorite is believed to underlie the entire site at depth. Decomposed granitic rock materials were encountered in our exploratory Boring B-1 at a depth of about 28 feet. As described in the boring logs, the granitic rock materials encountered are gray in color and highly weathered for the depth explored. Granitic rock weathered to well graded sand with silt or clay (SW-SC, SW-SM).

3.5 Groundwater

Groundwater was encountered at depths between 14.5 to 16.1 feet bgs (between 331.0 and 328.9 feet above MSL) during our field investigation. The State Water Resources Control Board website (GeoTracker, 2021) provides depth to groundwater data from 2002 to 2013 at the former RCP Block & Brick Inc. site located at 9631 N. Magnolia Avenue, about ¼-mile southeast of the site. The data indicates groundwater depths ranging from about 328.2 to 330.7 feet above MSL in the 8 monitoring wells installed at that site (SCS Engineers, 2013).

Groundwater levels may fluctuate over time due to changes in the water surface elevation and flow rate within the Woodglen Vista Creek, as well as variations in rainfall, irrigation and site drainage conditions.

4.0 SEISMICITY AND GEOLOGIC HAZARDS

Potential seismic hazards during an earthquake include ground rupture, strong ground shaking, seismic slope instability, liquefaction and dynamic settlement, and earthquake induced flooding due to tsunamis or dam failures. Potential geologic hazards include landslides, erosion, subsidence, volcanic eruptions, and poor soil conditions (compressible, collapsible or expansive soils). Each of the potential hazards is discussed in more detail below.

4.1 Ground Rupture

The project site is not located within an Alquist-Priolo Earthquake Fault Zone. The closest known active fault is the Mission Gorge located at a distance of approximately 10.4 km away from the project site as shown in Figure 5. Therefore, ground surface rupture due to active faulting is not considered a potential hazard at the project site.

4.2 Earthquake Ground Motions

Similar to most sites in southern California, the project site is susceptible to strong ground motions generated during earthquakes on nearby faults. The intensity of ground motion is dependent on the distance between the fault and the project site, the magnitude of the earthquake, and the subsurface soil conditions. These seismic hazards and their potential impact at the project site are discussed below.

Design ground motion parameters and response spectra were developed for the project site in accordance with the 2022 California Building Code (CBC) and the American Society of Civil Engineers (ASCE) 7-16 standard for essential facilities. Based on the underlying geology and subsurface exploration data the site classification for seismic design is Site Class D per Chapter 20 of ASCE 7-16. Mapped seismic design parameters for the project site using the USGS Seismic Design Maps web application are presented in Table 1.

Table 1: Mapped Seismic Design Parameters per CBC 2022 / ASCE 7-16

Latitude: 32.85076° Longitude: -116.97683°	
Site Class	D
Mapped MCE Spectral Response Acceleration at Short Period (S_S)	0.77
Mapped MCE Spectral Response Acceleration at Period of 1 Second (S_1)	0.283
Site Coefficient, F_a	1.192
Site Coefficient, F_v	2.034
Adjusted MCE Spectral Response Acceleration at Short Period (S_{MS})	0.918
Adjusted MCE Spectral Response Acceleration at Period of 1 Second (S_{M1})	0.576
Design Earthquake Spectral Response Acceleration at Short Period (S_{DS})	0.612[(1)]
Design Earthquake Spectral Response Acceleration at Period of 1 Second (S_{D1})	0.384[(2)]
Peak Ground Acceleration Adjusted for Site Class (PGA_M)	0.419

Notes
[(1)] For $T \leq 1.5\ T_s$, S_{DS} should be used only to obtain Cs using Equation 12.8-2.
[(2)]: If S_{D1} is used to obtain C_S with either equation 12.8-3 or 12.8-4 of ASCE 7-16, the value must be increased by a factor of 1.5. This may only be used for $T > 1.5\ T_S$.

4.3 Liquefaction and Seismically-Induced Settlement

Liquefaction involves the sudden loss in strength of a saturated, cohesionless soil (sand and non-plastic silts) caused by the build-up of pore water pressure during cyclic loadings, such as those produced by an earthquake. This increase in pore water pressure can temporarily transform the soil into a fluid mass, resulting in a vertical settlement, and can also cause lateral ground deformations. Typically, liquefaction occurs in areas where these three simultaneous conditions exist:

- Loose to medium dense cohesionless soils
- Groundwater within 50 feet of the surface
- Strong shaking, such as caused by an earthquake

The project site is mapped within a liquefaction zone identified by City of Santee in their General Plan, Geotechnical/Seismic Hazard Map, 2020 (Figure 6.). The project site is considered to have a moderate to high liquefaction potential.

Liquefaction triggering analyses was performed using simplified procedures recommended by NCEER (Youd and Idriss, 1997, 2001) for SPTs. The analyses uses a peak ground acceleration value for the 2,475-year return period earthquake (PGA_M) based on ASCE 7-16 of 0.42g and a moment magnitude of M_w 6.4 computed using the USGS based on the Dynamic U.S. 2014 (v4.2) deaggregation tool (https://earthquake.usgs.gov/hazards/interactive/). A design groundwater of 14.5 feet was used in the analyses.

Potential liquefiable layers were encountered at a depth between 14.5 and 28 feet bgs. The soils consist of loose to medium dense sandy soils. The potential for liquefaction to occur at the project site under the design earthquake is considered high. Liquefaction-induced settlement was calculated on the order of 4 inches. Dry seismic settlements above this depth are calculated at less than 0.1 inches. Differential settlement from the liquefaction induced settlements could be on the order of 2. The values reported are using the NCEER (Youd and Idriss, 1997, 2001) method for SPTs. Liquefaction triggering and liquefaction-induced settlement calculations are provided in Appendix C.

4.4 Landslides and Lateral Spreads

The project site is mapped as marginally susceptible to landslide based on City of Santee General Plan, Geotechnical/Seismic Hazard Map, 2020 (Figure 6). The project site is relatively flat with some embankments built up to the south and east near the Woodglen Vista Creek. Based on our review of an existing contour map the adjacent slopes are approximately 7 feet high and are sloped at approximate 4H:1V.

Two-dimensional limit equilibrium slope stability analyses were performed using the computer program SLIDE to assess the adjacent slopes for overall stability considering static, seismic and rapid drawdown conditions. Soil strength parameters for the analyses were selected based on our field and laboratory testing as summarized in Appendix A and B. The analyses were performed using Spencer's (1967) method of slices and the results are summarized in Table 2 and the calculations are presented in Appendix C.

Global stability under seismic loading conditions were conducted using a pseudo-static horizontal acceleration coefficient (k_h) equal to 2/3 x PGA = 0.279. Initial check for the seismic condition includes liquefied soil strengths evaluated according to Idriss and Boulanger (2008) to evaluate the potential for lateral spreading. A factor of safety greater than 1.1 for the aforementioned condition indicates a relatively small displacement and adequate stability, while a lower factor of safety requires a displacement analysis. The factor of safety was less than 1.1 and indicates a potential for lateral spreading. The horizontal displacements were calculated using a Newmark type simplified procedure recommended by Bray and Travasarou (2007). The estimated displacement is summarized in Table 2.

Table 2: Summary of Global Stability

Static Case	"Seismic Case – Pseudo-Static (Lateral Spreading)" with Reduced Liquefaction Strengths		
Factor of Safety	Factor of Safety (k_h = 2/3 PGA)	K_y	Lateral Spreading Displacement (in)
4.8	<1.1	0.14	8.6

4.5 Expansive Soils

The onsite near surface materials are generally clayey sands. A laboratory test was performed on one sample of the near surface materials that had a measured Expansion Index (EI) value of 68 as shown in Appendix B. This indicates a medium expansion potential (51<EI<90) based on American Society for Testing and Materials (ASTM) D4829 standard. Considering that moderately expansion soils are present at the project site, remedial grading should be performed to remove expansive materials or alternatively, foundations can be designed to resist these expansion pressures.

4.6 Flooding, Seiches and Tsunamis

The project site is located in a flood hazard zone X, areas with reduced flood risk due to levee, as established by the Federal Emergency Management Agency (FEMA) in Figure 7. The project site is located adjacent to Woodglen Vista Creek that is at a lower elevation. Consequently, the potential for flooding due to seiches or dam failures is considered to be low.

The project site is located at an elevation of about 340 feet above MSL and a distance of about 20 miles away from the coastal region and therefore, the potential for hazard associated with tsunami impact is negligible.

5.0 KEY GEOTECHNICAL FINDINGS

The proposed development is feasible from a geotechnical standpoint, provided that the recommendations presented in this report are implemented. A summary of key geotechnical findings is provided below.

- The upper 4 to 10 feet of the subsurface soils are undocumented fill that consists of predominantly clayey sand material that can have variable strengths. Remedial grading recommendations are provided in Section 7.3.1.2.
- Potentially liquefiable soils are present at the site between a depth of 14.5 to 28 feet bgs that could result in approximately 4 inches of liquefaction-induced settlement and a seismic differential settlement on the order of 2 inches. Measures such as post-tension slabs, mat foundations and/or ground improvement should be considered to mitigate the effects of the total and differential settlements. Lateral spreading is estimated to be on the order of 8.5-inches and shallow footings on ground improvement should be tie together.
- Groundwater was encountered at a depth of about 14.5 feet bgs. No basements are proposed, however, excavations extending near groundwater may encounter unstable bottoms. Excavations that extend within 5 feet or less of groundwater table should consider lighter construction equipment to avoid developing a “pumping” subgrade that would require remedial subgrade stabilization.

- Medium expansive soils were encountered in the upper 4 to 10 feet at the site. Foundations should be designed to resist the expansive soils as recommended in Section 6.2 or the upper 4 feet of soils from final grade should be removed and replaced with non-expansive material as recommended in Section 7.3.1.2.
- Laboratory tests results indicate that the onsite soils may be very corrosive to buried ferrous metals. A corrosion consultant may be contacted for specific corrosion control recommendations.
- Onsite infiltration is not feasible due to the presence of shallow groundwater and near surface soils that have an infiltration rate of less than 0.001 inches per hour as discussed in Section 6.1.

6.0 FOUNDATION RECOMMENDATIONS

6.1 General

City of Santee has commenced the predesign for the proposed Santee Community Center project. Community Center building will be part of phase one of a total of three phases. We understand that a two-story building is planned to be built at the project site. No basement levels are planned for this development.

Considering the subsurface soils are prone to liquefaction-induced settlements on the order of 4-inches and lateral spreading maybe on the order of 8.5-inches, it is recommended to use shallow foundations with grade beams or a reinforced mat foundation situated on ground improvement. Slabs on grade will need to be designed for the expansive nature of the onsite soils or slabs on grade will require removal and replacement of at least 4 feet below final grade to mitigate onsite expansive soils.

Pile foundations are not recommended for the building considering the need to account for the presence of sandy material below groundwater, presence of bedrock and the potential for lateral spreading. Drilled piles would most likely be needed for embedment into the bedrock and to accommodate the lateral spreading kinematic loads. Drilling would be prone to caving sands during construction and use of casings and/or “wet” method construction would be required. An experienced contractor specializing in these conditions would be required. It is most likely more cost effective to control overall settlement with ground improvement and there is a lower risk of constructability issues.

6.2 Community Center Structure

6.2.1 Post-Tensioned Slabs or Reinforced Mat Slab Foundations

6.2.1.1 Subgrade Reaction and Expansive Soil Design Parameters

The existing near surface soils at the project site consist of clayey sands. These materials generally have a medium expansion potential (51<EI<90). Site preparation and compaction requirements should follow recommendations provided in Section 7.3. A post-tensioned slab or mat thickness and reinforcement should be designed by the project structural engineer using parameters in Table 3 and considering liquefaction-induced .

Table 3. Post-Tensioned Slab Foundation Design Recommendations

Design Parameter		Value
Plasticity Index		20-30
Expansion Index		51-90
Percent Passing No. 200 Sieve		80
Thornthwaite Moisture Index		-20
Depth of Constant Soil Suction (feet)		9
Center Lift	Edge Moisture Variation Distance, e_m, (feet)	5.3
	Center Lift, y_m, (inches)	-0.5
Edge Lift	Edge Moisture Variation Distance, e_m, (feet)	2.7
	Edge Lift, y_m, (inches)	1.0

The modulus of subgrade reaction concept can be used in the design of mat foundations and slabs-on-grade. The modulus of subgrade reaction is not an intrinsic property of the soil since it also depends on the dimensions and stiffness of the slab and the stress level. The mat slab foundation should be designed for settlements and bending moments using a value of 150 pci for the normalized modulus of subgrade reaction coefficient K_{v1} (namely, corresponding to a 1-foot square bearing plate). Depending on the level of ground improvement, this value may be increased and should be coordinated with the ground improvement specialty contractor. To ensure rigidity of the foundation a subgrade reaction coefficient, K_v, should be used based on Terzaghi (1955) and is defined as:

$$K_v = K_{v1} * [(m + 0.5)/1.5m] * [(B+1)/2B]^2$$

where "B" is the width of the foundation measured in feet, and "m" is the ratio of length over width of a rectangular foundation. The flat concrete slab of the mat system should, at a minimum, have continuous two-way reinforcing at the top and the bottom and be designed by the project structural engineer.

6.2.1.2 Bearing Capacity and Settlement

An allowable average bearing pressure of 1,000 psf and a maximum allowable bearing of 1,500 psf for concentrated areas may be used for design if the remedial measure presented in Section 7.3.1.2 are followed. For planning purposes, an allowable bearing pressure of 2,000 psf can be anticipated if ground improvement is extended to the bottom of the foundation as discussed in Section 7.2. The final bearing capacity should be confirmed by the ground improvement specialty contractor. The allowable bearing pressure may be increased by one-third for short term wind or seismic loads. The expected total post-construction (static) settlement of a PT slab or mat foundation with the allowable bearing pressure is expected to be less than

1.5 inches of static settlement. The static differential settlement is expected to be less than ¾-inch over a distance of 40 feet. The foundation should be situated a minimum of 18 inches below the lowest adjacent soil grade.

The total static-plus-seismic settlement under PGA_M seismic settlement analysis will be on the order of 4 to 5-inches and a differential of 2 inches. The dry seismic are negligible and the large settlements are associated with the liquefaction induced settlements as discussed in Section 4.3. Mat foundations shall be designed to accommodate the expected vertical differential settlements per ASCE 7-16 Section 12.13.1.

6.2.1.3 Lateral Resistance

Resistance to lateral loads can be provided by friction developed between the bottom of footings and the supporting soil, and by the passive soil pressure developed on the face of the footing. An allowable passive resist of 300 pounds per cubic foot (pcf) and a coefficient of friction of 0.35 may be used for lateral sliding resistance of footings. Both values include a factor of safety of at least 1.5 and both passive and sliding resistance may be used in combination without reduction. The allowable lateral resistance may be increased by one-third for short term wind or seismic loads.

6.2.2 Shallow Foundations with Grade Beams and Ground Improvement

The Community Center can be supported by shallow foundations if they are horizontally tied with grade beams, founded over ground improvement as discussed in Section 7.2 and remedial grading is performed in accordance with 7.3.1.2. The seismic-induced permanent horizontal ground displacement exceeds 3 inches if the design earthquake is realized and ASCE 7-19 Section 12.13.9.2.1.1 shall be followed.

Final ground improvement layout shall be designed by a specialty contractor as discussed in Section 7.2. The ground improvement will be controlled by allowable settlements, that should not exceed 1.5 inches of total (static-plus-seismic) and should be evaluated by the specialty contractor as part of their submittal to support their plans. For planning purposes, spread footings situated on ground improvement as discussed in Section 7.2 is expected to accommodate an allowable dead-plus-live load pressure of at least 4,000 pounds per square foot (psf). The static differential settlement should be less than ½-inch over a distance of 40 feet. Ground improvement should be placed after remedial grading to remove and replace 4 feet of soil below the slab if the slab is not designed to accommodate expansive forces of the existing soils. Shallow footings should have a minimum dimension of 2 feet and shallow continuous footings should have a minimum width of 1.5 feet. Locate the bottoms of all footings at least 18 inches below the lowest adjacent grade.

Resistance to lateral loads recommendations from Section 6.2.1.3 apply for shallow footings.

6.3 Slope Setback

As per section 1808.7.2 of the 2022 CBC, which references Figure 1808.7.1 of the 2022 CBC, the distance between the face of the footing from the face of descending slopes should be at least the smaller of H/3 and 40 feet, where H is the height of the slope.

6.4 Soil Corrosion Potential

Near surface soils collected from Boring B-1 were tested to evaluate the corrosion potential of subsurface materials. The tests include pH, electrical resistivity, soluble chloride and soluble sulfate concentrations are summarized in Table 4 and attached at Appendix B.

Table 4: Corrosion Potential Test Results

Sample/Depth	pH	Resistivity [Ohm-cm]	Sulfate Content [ppm]	Chloride Content [ppm]
B-1 @ 1'- 5'	8.87	830	<100	<100

Based on pH, sulfate content and chloride content of the test sample, the near surface soils are considered not corrosive to concrete. The following correlation can generally be used between electrical resistivity and the corrosion potential of soils in contact with buried metals:

Electrical Resistivity (Ohm-Cm)	Corrosion Potential
Less than 1,000	Severe
1,000 to 2,000	Corrosive
2,000 to 10,000	Moderate
Greater than 10,000	Mild

On the basis of the laboratory testing, the onsite soils are very corrosive to buried metals. Typical corrosion control measures may be incorporated into the design, such as providing adequate concrete cover or protective coatings for steel reinforcement, and coating or providing sacrificial anodes as needed for buried metal pipes. Adequate cover for steel below grade shall follow ACI 318 guidelines. Group Delta does not practice corrosion engineering and our evaluation and recommendations are preliminary in nature. For further guidance and verification of our preliminary results, a corrosion specialist should be consulted.

6.5 Stormwater Infiltration

The City of Santee BMP Design Manual (2016) states that the depth to groundwater beneath the base of any infiltration BMP must be greater than 10 feet for infiltration BMPs to be allowed. The groundwater was encountered between 14.5 and 16.1 feet at the site during our investigation and would require the invert of any BMP to be situated in the upper 5 feet. The upper 5 feet of soil consists of clayey sands that are not suitable for infiltration. The hydraulic conductivity of the clayey sands was estimated to be less than 0.001 inches per hour utilizing the Hazen empirical equation that is a function of the grain size diameter corresponding to 10 percent passing as determined by ASTM D422. Consequently, the invert of the BMP system would need to extend within 10 feet of the measured groundwater and onsite infiltration is not considered feasible for the project.

6.6 Retaining Walls

Retaining walls should be placed on 2 feet of compacted fill that is prepared as discussed in Section 7.3. The continuous footings should have a minimum width of 18-inches and have a minimum embedment of at least 18-inches below the lowest adjacent grade. Footings with this minimum width and embedment may be designed for an allowable dead-plus-live pressure of 1,500 pounds per square foot (psf). The allowable bearing pressure may be increased by one-third when considering temporary loads associated with wind and seismic loading. Short-term static settlements for footings are expected to be less than 1-inch. Differential settlements will be less than ½-inch over a distance of 40-feet. Estimated liquefaction-induced settlements may be on the order of 4-inches if a design level earthquake is realized. Retaining walls typically can accommodate large displacements without collapse. However, if any retaining wall is considered a critical component to the structure, then ground improvement could be utilized to limit vertical displacements.

The magnitude of lateral earth pressure depends on wall movement. Cantilever retaining walls free to yield at the top at least 0.2 percent of the wall height may be designed for active pressure conditions. Active earth pressure for design may be taken as an equivalent fluid unit weight of 34 pcf for level backfill. The pressure does not include seepage forces or surcharge loads. Surcharge loads within a 1H:1V plane extending back and up from the base of the wall should be accounted for in design. For uniform areal surcharge loading the lateral pressure on the wall may be taken as a uniformly distributed pressure equal to 28 percent of the vertical pressure for active condition. Other surcharge loading conditions should be evaluated on a case-by-case basis.

The wall should be designed to resist an active pressure combined with a seismic increment of lateral earth pressure when the retained height H is 6-feet or greater. The combined active static and seismic lateral earth pressure were computed based on a horizontal acceleration coefficient k_h of 0.14g that is based on one third of PGA_M. The combined active static and seismic lateral earth pressure is equivalent to a fluid with a density of 53 pcf. Therefore, a seismic increment of 19 pcf may be used for design of seismic earth pressure.

Retaining wall backfill should be compacted to at least 90% relative compaction based on ASTM D1557. Backfill should not be placed until walls have achieved adequate strength. Heavy compaction equipment, which could cause distress to the walls, should not be used within 5-feet of the wall. We recommend that all retaining walls be backfilled with very low expansion granular soils (EI<20) and a Sand Equivalent (SE) of not less than 20. The shallow onsite material is considered expansive and import material will be required.

All walls should be constructed with a properly designed drainage system to prevent buildup of hydrostatic pressures behind the wall. This may consist of gravel and filter fabric or geocomposite panel drains discharging through weep holes or subdrains.

6.7 Ancillary Structures

Ancillary structures for the project are anticipated to be non-occupancy structures such as trash enclosures, site fence walls, etc. and recommendations provided in Section 6.6 may be used. In addition, associated pavement recommendations for trash enclosures are provided in Section 6.10.2.

6.8 Pole Type Foundation

Light poles may be supported on pole-type foundation. Pole-type foundation may be designed per Section 1807.3, Embedded Posts and Poles, of the CBC 2022. Equation 18-1 may be used for a non-constrained condition, and Equations 18-2 and 18-3 may be used for a constrained condition. S1 and S3, the allowed lateral soil bearing pressure may be taken as 200 psf. The allowable soil bearing pressure may be increased by one-third for short term wind or seismic loads.

6.9 Slab-on-Grade

6.9.1 Interior Slabs

Site preparation and compaction requirements should follow the recommendations provided in Section 7.3.1. Concrete slabs should have a minimum thickness of 4 inches and should have a minimum reinforcement with 6x6 W2.9/2.9 welded wire mesh or equivalent. All slab reinforcement should be properly supported to ensure the desired placement. In addition, final design of the foundation should be in accordance with ASCE 7-19 Section 12.13.9.2.1. The actual slab thickness and reinforcement should be designed by the project structural engineer. Building slabs should be either designed for the expansive soils or the upper 4 feet of soil should be removed and replaced with non-expansive material.

6.9.1.1 Moisture Protection

To reduce the potential for moisture transmission through slabs where moisture sensitive floor covering will be installed, we recommend that a vapor barrier be used. In accordance with ACI 302.2R-06, the material must comply with the requirements of ASTM E1745, “Standard Specification for Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs,” and have a permeance of less than 0.01 perms per ASTM E96. The installation of the moisture barrier should comply with ASTM E1643. Concerning whether to place two inches of sand over the retarder, reference is made to ACI 302.2R, Section 7.2, which states that the anticipated benefits and risks associated with the location of the vapor retarder should be reviewed on a case by case basis with all appropriate parties, considering anticipated project conditions and the potential effects of concrete curing, cracking, and curling.

6.9.2 Exterior Slabs

Exterior slabs and sidewalks should be at least 4-inches thick, and should be constructed on at least 2 feet of very low expansion (EI<20) compacted soil, as recommended in Section 7.3.1.

6.10 Pavement Design

6.10.1 Flexible Pavements

Asphalt concrete pavement design was conducted in general accordance with the Caltrans Highway Design Manual (Caltrans, 2020). In order to aid in pavement analyses, R-Value tests were conducted on shallow subgrade samples collected from Borings B-2 during the field investigation. The test result is provided in Appendix B and the test indicted an R-Value of 20. Table 5 provides recommendations for pavement sections considering alternative base sections for clayey subgrade and sandy subgrade conditions for several traffic index (TI) values considering a 20-year design.

Table 5: Preliminary Flexible Pavement Design Sections

Traffic Index (TI)	Asphalt Concrete (HMA) Thickness (inches)	Class II Aggregate Base Thickness (inches)
4.5	3.0	5.0
5	3.0	7.0
5.5	3.0	9.0
6	3.5	9.5
6.5	4.0	10.0

The asphalt concrete thickness can be divided into base and finish courses. The uppermost 1-foot of subgrade soil should be scarified immediately prior to constructing the new pavements, brought to about optimum moisture content and compacted to at least 95 percent relative compaction. All aggregate bases should also be compacted to at least 95 percent relative compaction. Aggregate base should conform to Section 200-2 of the Standard Specifications for Public Works Construction (*SSPWC)*. Asphalt concrete should conform to Section 203-6 of the *SSPWC* or Section 39 of the Caltrans Standard Specifications. We recommend that asphalt concrete be compacted to between 91 and 97 percent of the Rice density per ASTM D2041.

6.10.2 Rigid Concrete Pavements

Rigid concrete pavements may be desirable in certain areas where heavy equipment may induce large pavement loads, such as a fire access road or near trash bin storage locations. Portland Cement Concrete (PCC) pavement design was conducted in accordance with a simplified design procedure (Chapter 4) of the Portland Cement Association. The methodology is based on a 20-year design life. For design, it was assumed that aggregate interlock would be used for load transfer across control joints. The subgrade soils were assumed to provide “medium” subgrade support and the concrete modulus of rupture at 28-days was assumed to be 600 pounds per square inch (psi). Based on these assumptions, we recommend that the PCC pavement sections at the site consist of 6-inch of concrete placed over 6 inches of compact aggregate base.

Crack control joints should be constructed for all PCC slabs on a maximum spacing of 12-feet, each way.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Plan Review

Detail design and foundation and grading plans should be reviewed by Group Delta prior to beginning construction. Design loads and final structural configuration should be reevaluated to ensure conformance to the recommendations and findings of this report.

7.2 Ground Improvement

7.2.1 Depth Range of Ground Improvement

Ground improvement should be performed to mitigate the potential for liquefaction-induced settlements and differential movements from undocumented fill. The final design of the ground improvement should be based on permissible displacement allowed during a seismic event to avoid risk of collapse or other project damage goals. The minimum zone of improvement should extend from the bottom of footings to a few feet into the weathered bedrock layer below the lowest liquefiable layer. For planning purposes, the ground improvement may be assumed from El. 333 to El. 315 feet. Ground improvement is required to extend from El. 315 feet to the bottom of footings if shallow foundations are utilized as discussed in Section 6.2.2.

7.2.2 Methods of Ground Improvement

In general, methods to mitigate liquefaction include methods that mix soil and cement (soil mixing and jet grouting), grouts that permeate the soil pores (permeation grouting), and densification methods (compaction grouting, vibroflotation, or stone columns). Considering the readily accessible nature of the project site and the depths of needed improvements are between 13 feet and 30 feet below ground surface, a densification method is well suited for the project.

Compaction grouting would be the most effective method to densify the potentially liquefiable soils and limit liquefaction-induced settlements. Compaction grouting involves injecting grout into the ground in a grid pattern that displaces and compacts the surrounding soils. The grout columns have an added benefit in that they are rigid inclusions that provide additional resistance to further restrict lateral spreading. Pre- and post-grouting cone penetration tests (CPTs) can be performed to indicate the effectiveness of the improvement.

Stone columns are a more cost effective option that could be utilized for reducing liquefaction-induced settlements as well as providing adequate bearing to improve the undocumented fill at the site. Stone columns construction involves the introduction of rock material into the native material by downhole vibratory methods. Stone column construction is often referenced as vibro-replacement or vibro-displacement that can be a top or bottom feed process to install stone columns to the targeted depths. Alternative to vibration methods include rammed aggregate piers (RAP) that are installed by drilling and ramming lifts of well-graded aggregate to form the high-density columns.

A qualified soil improvement contractor should be selected and provide design of the mix proportions, depth, spacing, and size of the zone of treatment based on the target foundation design parameters and their design requirements for the selected ground improvement method. Quality control procedures for installation and verification of material strengths will be developed once the method of ground improvement has been selected.

7.2.3 Areal Extent of Ground Improvement

Compaction grouting and stone columns would both be placed in grid patterns for the entire area of the foundation and should extend at least five feet laterally from the foundation of the building.

7.3 Earthwork

Grading and earthwork should be conducted in general accordance with the applicable local grading ordinance and the requirements of the 2022 California Building Code. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on the conditions observed by our personnel during grading.

7.3.1 Site Preparation

7.3.1.1 Clearing and Grubbing

The area that will be developed should be cleared and grubbed of all improvements, and vegetation in general accordance with Section 300-1 of the Standard Specifications for Public Works Construction [SSPWC] (Green Book, 2018).

Existing subsurface utilities that are to be abandoned should be removed and the excavations backfilled and compacted as described in Section 7.3.2. Alternatively, abandoned utilities may be grouted with a two-sack sand-cement slurry under the observation of Group Delta. After clearing and grubbing the site, remedial grading should be performed in the building and other general improvement areas as recommended in the following sections.

7.3.1.2 Remedial Grading

Approximately 4 to 10 feet of undocumented fill as discussed in Section 3.4 was identified at the site that consists of predominantly clayey sand material that can have variable strength. The undocumented fill below the proposed building should be removed to at least 7 feet below the existing grade or at least 4 feet from the bottom of slab, whichever is deeper. The removal areas should extend laterally at least 5 feet beyond the edge of the footings in all directions. As an alternative, ground improvement may be utilized to improve the undocumented fill as discussed in Section 7.2. If the slab on grade is not designed for expansive soils of the existing soils, then removal and replacement of at least 4 feet below final grade is required. The replacement soils should meet the requirements noted in Section 7.3.3.

In general the exposed subgrade at the bottom of overexcavation should be proof rolled with loaded heavy equipment under Group Delta's observation to disclose any areas of deeper unsuitable soils. Areas of soft, loose, wet, pumping, or otherwise unsuitable soils should be further excavated or stabilized as recommended by Group Delta in the field. After proof-rolling the exposed subgrade should be scarified to a depth of 6 inches, brought to slightly above optimum moisture content, and compacted as described in Section 7.3.2. The excavation may then be backfilled from bottom of overexcavation to the planned finish subgrade with compacted fill.

7.3.2 Fill Compaction

All fill and backfill should be placed at slightly above optimum moisture content using equipment that is capable of producing a uniformly compacted product. In general, the minimum recommended relative compaction is 90 percent of the maximum dry density based on ASTM D1557. All fill placed within the proposed building areas and below foundations should be compacted to at least 95 percent relative compaction. Sufficient observation and testing should be performed by Group Delta so that an opinion can be rendered as to the compaction achieved. Rocks or concrete fragments greater than 4 inches in maximum dimension should not be used in structural fill.

7.3.3 Imported Fills

Imported fill sources, if any, should be observed and tested prior to hauling onto the site to evaluate the suitability for use. Imported fill materials should consist of granular soil with less than 35 percent passing the No. 200 sieve based on ASTM D 1140 and an EI less than 20 based on ASTM D4829. More stringent requirements may apply for soils to be used for specific purposes. Samples of the proposed import should be tested by Group Delta in order to evaluate the suitability of these soils for their proposed use. During grading operations, soil types may be encountered by the contractor that do not appear to conform to those discussed in this report. In that case, Group Delta should be notified in order to evaluate the suitability of these soils for their proposed use.

7.3.4 Temporary Excavations and Shoring

The contractor is responsible for excavation safety, and all excavations should comply with the current California and Federal Occupational Safety and Health Administration (OSHA) requirements (29 CFR-Part 1926, Subpart P), as applicable. For planning purposes OSHA Type C soils may be assumed for temporary excavations, which allows for temporary slopes up to 20 feet high at a gradient of 1.5:1 (horizontal: vertical). Unshored excavations should not extend below a 1:1 plane extending down from any improvements or foundations to be protected in place.

If sloping or benching is not practical due to space constraints, temporary shoring may be used. Vertical temporary excavations deeper than 5 feet should be shored. No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the excavation, unless the shoring is designed for surcharge loading. All shoring should comply with OSHA regulations and 29 CFR Part 1926 guidelines and be observed and deemed safe by the designated competent person on site. The designated competent person should observe all excavations to determine the safety prior to excavation. Shoring designs may utilize the following soil parameters:

- Unit Weight: 120 pcf
- Friction Angle: 32 degrees
- Active Coefficient (K_a): 0.307
- At-Rest Coefficient (K_0): 0.470

For design of cantilevered temporary shoring, where the surface of the backfill is level, it can be assumed that drained soils will exert a lateral pressure equal to that developed by a fluid with a density of 37 pcf. Surcharge loads from equipment or stockpiled material should be kept behind the top of the temporary excavations a horizontal distance equal to the depth of the excavation, or the shoring should be designed for the additional pressure. Foundation and traffic loads from adjacent areas should also be added to the lateral earth pressures.

For design of temporary rigid shoring such as braced shoring or trench shields in sandy soils, we recommend the use of a rectangular lateral pressure of 24H psf, where H is the height of the shoring in feet. In addition, 47 percent of any surcharge load should be included as a uniform rectangular loading on the shoring. For traffic loads no larger than highway trucks, the surcharge may be taken as a uniform lateral pressure of 100 psf. Other surcharge loads may be evaluated by Group Delta on a case-by-case basis.

Surface drainage should be controlled and prevented from running down the temporary excavations or down the face of the shoring. Ponding water should not be allowed within the excavation.

7.4 Utility Trenches

Excavations for utility trenches should be readily accomplished with conventional excavating equipment. All shoring and excavation should comply with current OSHA regulations and be observed by the designated competent person on site.

The bedding for any new sewer and water service pipelines should be a minimum of 4 inches thick and should consist of clean sand, No. 4 concrete aggregate or gravel, and should have a sand equivalent of not less than 30. The pipe zone material, which extends to a level 12 inches above the pipe should consist of sand and should have a sand equivalent of no less than 30, and a maximum rock size of 1 inch. All imported materials should be approved by the project geotechnical engineer before being brought on site.

Trench zone backfill extends from a level 12 inches above the pipe to the finished subgrade. In general, on-site excavated materials are suitable as backfill. Any boulders or cobbles larger than 3 inches in any dimensions, or any organics or other deleterious materials, should be removed before backfilling. We recommend that all backfill should be placed in lifts not exceeding six to eight inches in thickness and be compacted to at least 90% of relative compaction as determined by the ASTM D1557. Mechanical compaction will be required to accomplish compaction above the bedding along the entire pipeline alignments. Jetting or flooding of backfill should not be permitted.

In backfill areas, where mechanical compaction of soil backfill is impractical due to space constraints, 2-sack slurry (CLSM) may be substituted for compacted backfill.

7.5 Construction Observation and Testing

The Geotechnical Engineer should observe subgrade preparation, backfill and fill placement. Excavation bottom should be observed and approved by the Geotechnical Engineer prior to placement of concrete, steel, piping, or backfill materials. Sufficient in-place field density tests should be performed during fill placement to verify that the entire fill is placed in accordance with the recommendations provided in the geotechnical report and applicable codes.

8.0 LIMITATIONS

This investigation was performed per generally accepted Geotechnical Engineering principles and practice. The professional engineering work and judgments presented in this report meet the standard of care of our profession at this time. No other warranty, expressed or implied, is made. This report has been prepared for HMC Architects, the City of Santee, and its design consultants. It may not contain sufficient information for other parties or other purposes and should not be used for other projects or other purposes without review and approval by Group Delta.
The recommendations for this project, to a high degree, are dependent upon proper quality control of site grading, fill and backfill placement, and paving. The recommendations are made contingent on the opportunity for Group Delta to observe the earthwork operations. This firm

should be notified of any pertinent changes in the project, or if conditions are encountered in the field, which differs from those described herein. If parties other than Group Delta are engaged to provide such services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project and must either concur with the recommendations in this report or provide alternate recommendations.

9.0 REFERENCES

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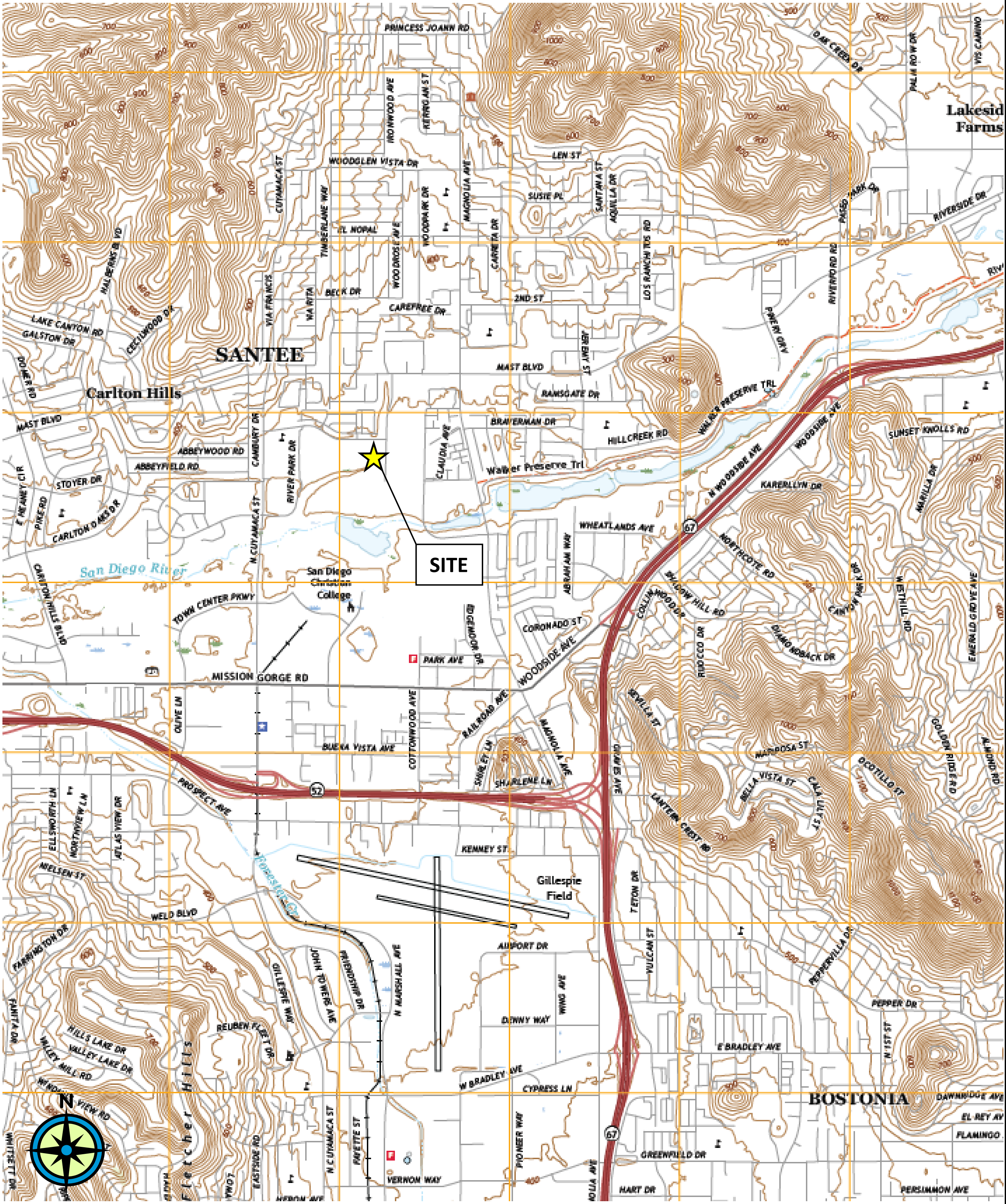
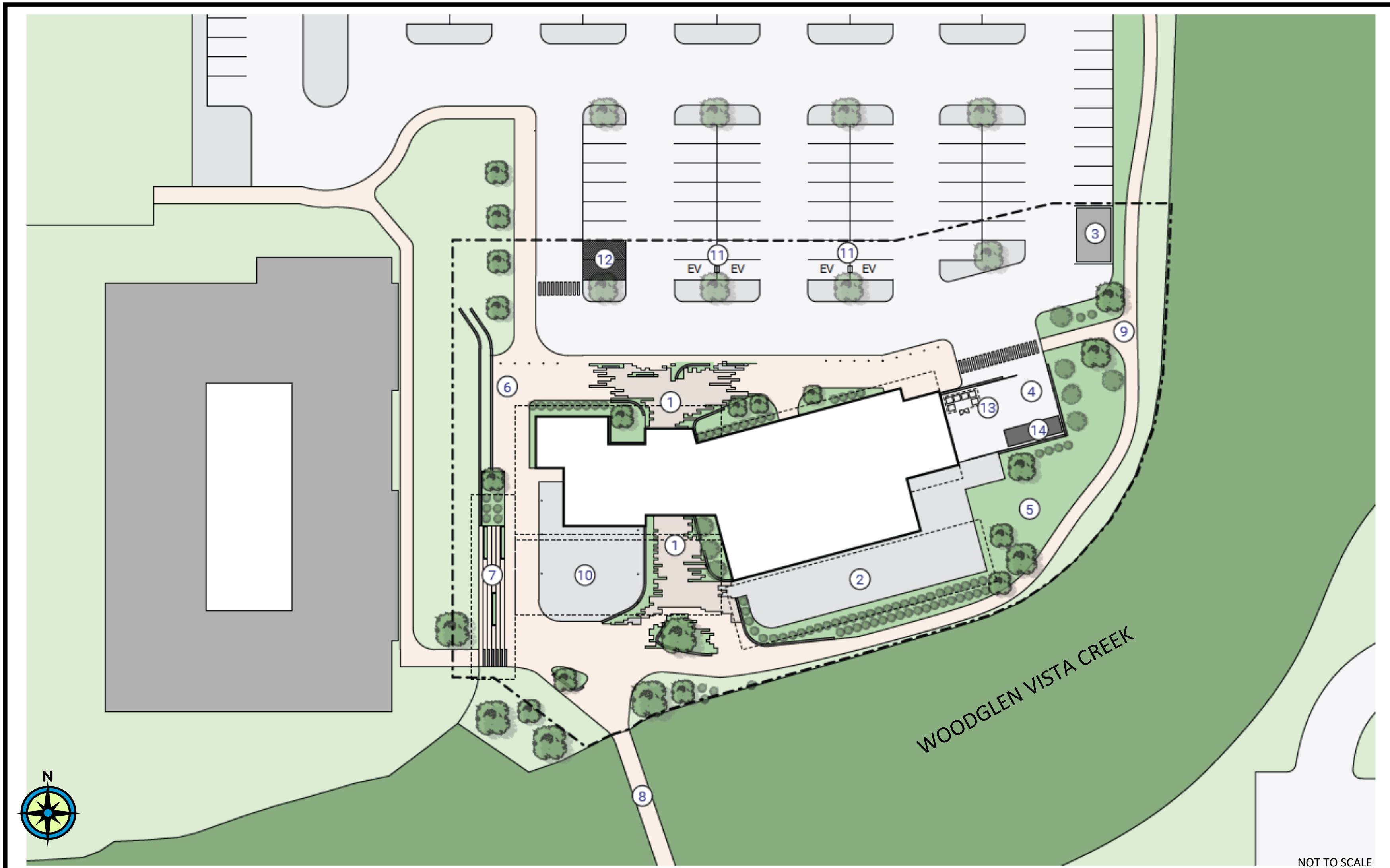
FIGURES



Reference: USGS, US Topo 7.5 Minute Map for El Cajon Quadrangle, CA 2021

GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	Figure Number: **1**
Project Name: Santee Community Center, Santee CA	Project Number: IR786
SITE LOCATION MAP	



Phase 1 Location (Community Center)

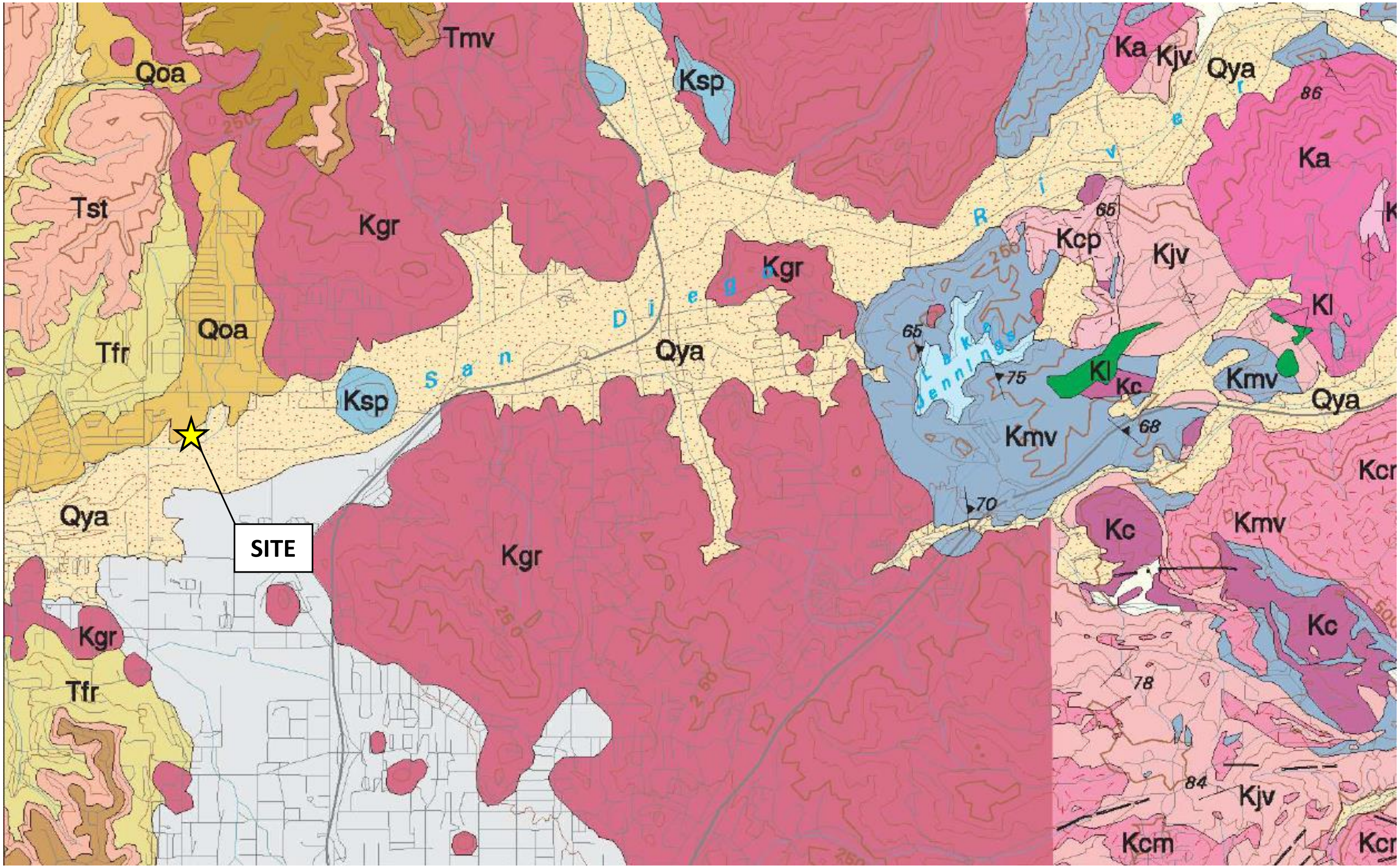
Note: Site Plan taken from HMC Architects 100% Schematic Design Package

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	Figure Number: 2
	Project Name: Santee Community Center Santee, CA	Project Number: IR789
	SITE CONCEPTUAL PLAN	

B-1

Approximate Boring Location

GROUP DELTA	**GROUP DELTA CONSULTANTS, INC.** ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	Figure Number: 3
	Project Name: Santee Community Center Santee, CA	Project Number: IR786
	EXPLORATION LOCATION PLAN	



NOT TO SCALE

Qya – Young alluvium (Holocene)

Qoa – Older alluvium (Holocene and Pleistocene)

Ksp – Santiago Peak Volcanics (Early Cretaceous)

Tfr – Friars Formation (Eocene)

Kgr – Granitoid rocks (Early Cretaceous)

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	Figure Number: 4
	Project Name: Santee Community Center Santee, CA	Project Number: IR789
	Regional Geologic Map	

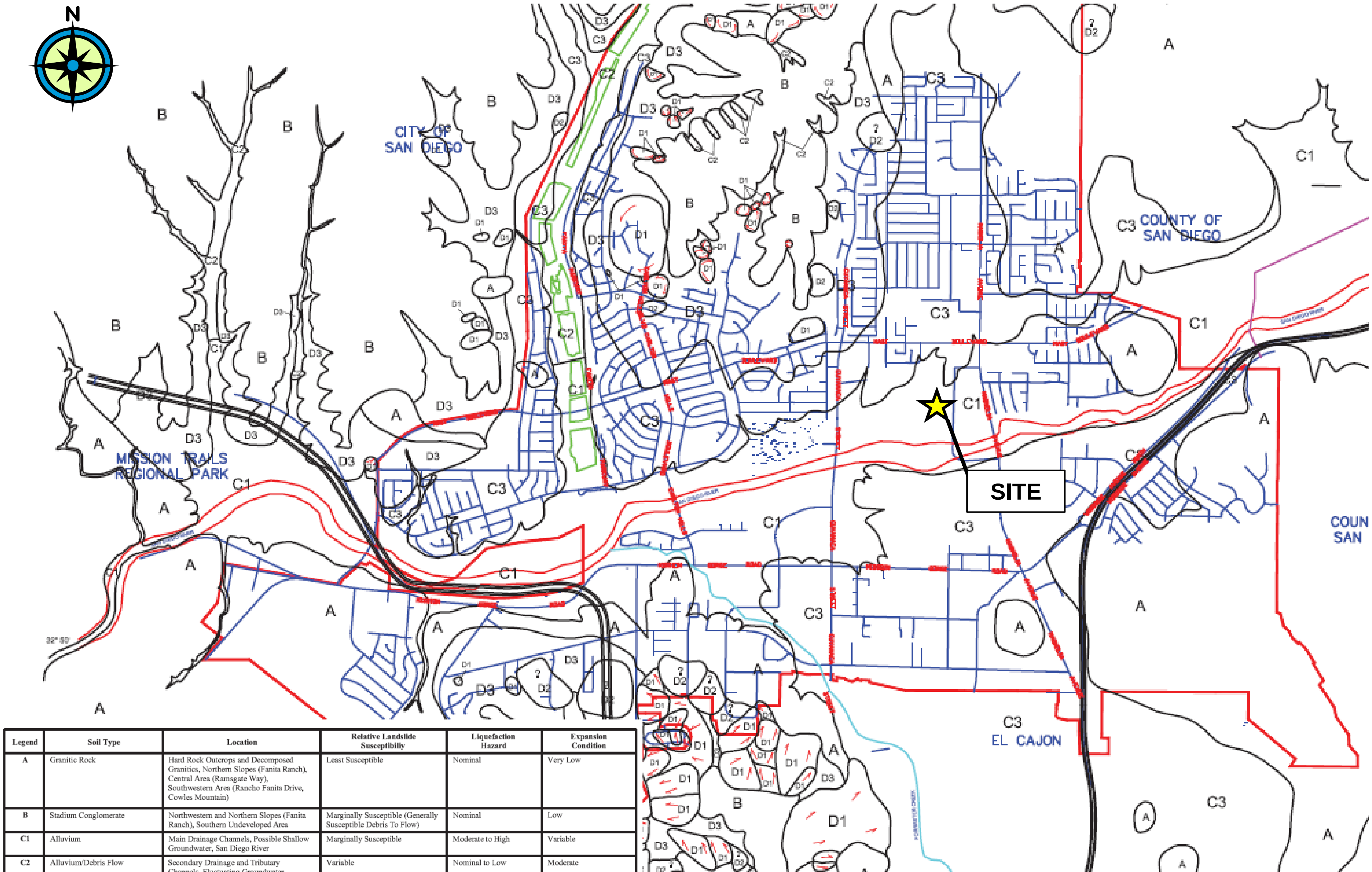
N

Reference: USGS Quaternary Faults, NSHM 2014 Fault Sources https://usgs.maps.arcgis.com/apps/webappviewer/index.html

NSHM 2014 Fault Sources

- Normal
- Strike Slip
- Thrust
- Unassigned

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	Figure Number: 5
	Project Name: Santee Community Center Santee, CA	Project Number: IR789
	REGIONAL FAULT MAP	



Legend	Soil Type	Location	Relative Landslide Susceptibility	Liquefaction Hazard	Expansion Condition
A	Granitic Rock	Hard Rock Outcrops and Decomposed Granitics, Northern Slopes (Fanita Ranch), Central Area (Ramsgate Way), Southwestern Area (Rancho Fanita Drive, Cowles Mountain)	Least Susceptible	Nominal	Very Low
B	Stadium Conglomerate	Northwestern and Northern Slopes (Fanita Ranch), Southern Undeveloped Area	Marginally Susceptible (Generally Susceptible Debris To Flow)	Nominal	Low
C1	Alluvium	Main Drainage Channels, Possible Shallow Groundwater, San Diego River	Marginally Susceptible	Moderate to High	Variable
C2	Alluvium/Debris Flow	Secondary Drainage and Tributary Channels, Fluctuating Groundwater	Variable	Nominal to Low	Moderate
C3	Terrace Deposits/ Older Alluvium	Gentle Slopes Western Area, Flanks of the San Diego River Valley (Carlton Oaks Drive), Central Area (Woodpark Drive)	Generally To Marginally susceptible (Where Underlain by Friars Formation)	Low to Moderate	Variable
D1	Landslides Confirmed	Sloping Southern Area (Route 125 and Fanita Drive, Fanita Ranch, Carlton Hills)	Most Susceptible	Nominal	Moderate to High
D2	Landslides Possible	Various Areas Throughout Friars Formation	Most Susceptible	Nominal	Moderate to High
D3	Friars Formation	Northern Slopes (Cuyamaca Street, Lake Canyon Road, Fanita Ranch) And Southern Slopes (Mesa Heights Road, Route 125)	Most Susceptible	Nominal	Moderate to High

REFERENCES:
City of Santee General Plan, Geotechnical/Seismic Hazard Map, Figure 8-3, 2020

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ENGINEERS AND GEOLOGISTS
32 MAUCHLY, SUITE B
IRVINE, CA 92618 (949) 450-2100

FIGURE NO.: **6**

PROJECT NAME:
Santee Community Center
Santee, CA

PROJECT NO.: IR786

LIQUEFACTION ZONE MAP

Flood Hazard Zones

- 1% Annual Chance Flood Hazard
- Regulatory Floodway
- Special Floodway
- Area of Undetermined Flood Hazard
- 0.2% Annual Chance Flood Hazard
- Future Conditions 1% Annual Chance Flood Hazard
- Area with Reduced Risk Due to Levee
- Area with Risk Due to Levee

GROUP DELTA	GROUP DELTA CONSULTANTS, INC. ENGINEERS AND GEOLOGISTS 32 MAUCHLY, SUITE B IRVINE, CA 92618 (949) 450-2100	FIGURE NO.: 7
	PROJECT NAME: Santee Community Center Santee, CA	PROJECT NO.: IR786
	FLOOD HAZARD ZONE MAP	

APPENDIX A
FIELD INVESTIGATION

APPENDIX A
FIELD INVESTIGATION

INTRODUCTION

The subsurface conditions at the project site were investigated by performing borings as described below. A summary of field explorations (Table A-1), boring record legend, key for soil classification, boring records, drive hammer energy calibrations, and other relevant information are presented in the attachments to this appendix. Specific details for each boring are presented in the title block of each boring record.

Prior to beginning the exploration program, access permission and drilling permits were obtained as necessary. Subsurface utility maps were reviewed prior to selecting locations for subsurface investigations. Underground Service Alert (USA) was notified, and each exploration location was cleared for underground utilities using geophysical techniques as needed. Approved traffic control plans were implemented where necessary during field activities. The exploration methods are described in the following sections.

SOIL DRILLING AND SAMPLING

Drilling, Logging, and Soil / Rock Classification

Borings were performed by Group Delta's drilling subcontractors under the continuous technical supervision of a Group Delta field engineer or geologist, who visually inspected the soil samples, maintained detailed records of the borings, measured groundwater levels, and visually / manually classified the soils in accordance with the ASTM D 2488 and the Unified Soil Classification System (USCS). Logging and classification was performed in general accordance with Caltrans "Soil and Rock Logging, Classification, and Presentation Manual (2010 Edition)". A Boring Record Legend, Key for Soil Classification, and boring records are presented in the attachments to this appendix.

Sampling

Bulk samples of soil cuttings were collected at selected depths and drive and/or push samples were collected at a typical interval of 5 feet from the borings (closer or wider sample spacing was employed where considered necessary or appropriate). The sampling was performed using Standard Penetration Test (SPT) samplers in accordance with ASTM D 1586, Ring-Lined "California" Split Barrel samplers in accordance with ASTM D 3550, and/or Thin-Walled "Shelby" Tube Samplers in accordance with ASTM D 1587.

Bulk samples were collected from auger cuttings and placed in plastic bags.

SPT drive samples were obtained using a 2-inch outside diameter and 1.375-inch inside diameter split-spoon sampler without lining. The soil recovered from the SPT sampling was sealed in plastic bags to preserve the natural moisture content.

Modified California ("MODCAL") drive samples were collected with a 3.0 inch outside diameter (OD) (unless indicated otherwise on the boring record) ring-lined split barrel sampler with a 2.42-inch inside diameter (ID) cutting shoe. The sampler barrel is lined with 18-inches of metal rings for sample collection and has an additional length of waste barrel. Stainless steel or brass liner rings for sample collection are 1-inch high, 2.42-inch inside diameter, and 2.5-inch outside diameter. California samples were removed from the sampler, retained in the metal rings, and placed in sealed plastic canisters to prevent loss of moisture.

At each sampling interval, the drive samplers were fitted onto sampling rod, lowered to the bottom of the boring, and driven 18 inches or to refusal (50 blows per 6 inches) with a 140-lb hammer free-falling a height of 30-inches (unless otherwise indicated on the boring record).

Shelby tube samplers (if used) were pushed to collect undisturbed samples of soft to stiff cohesive soils where encountered. The samples were secured within the tubes and the tubes were sealed with plastic caps and electric tape in the field to prevent moisture loss.

Compared to the SPT, the California sampler provides less disturbed samples. Shelby Tubes provide the least sample disturbance and are considered undisturbed.

Penetration Resistance

SPT blow counts adjusted to 60% hammer efficiency (N_{60}) are routinely used as an index of the relative density of coarse-grained soils, and are sometimes used (but less reliable) to estimate consistency of cohesive soils. For samples collected using non-SPT samplers, different hammer weight and drop height, and/or efficiency different than 60%, correction factors can be applied to estimate the equivalent SPT N60 value following the approach of Burmister (1948) as follows:

$$N^*_{60} = N_R * C_E * C_H * C_S$$

where

N^*_{60} = equivalent SPT N_{60}

N_R = Raw Field Blowcount (blows per foot)

C_E = Hammer Efficiency Correction = Er_i / 60%

C_H = Hammer Energy Correction = (W * H) / (140 lb * 30 in)

C_S = Sampler Size Correction = $[(2.0\text{ in})^2-(1.375\text{ in})^2]/[D_o^2-D_i^2]$

Er_i = hammer efficiency, %

W= actual drive hammer weight, lbs

H = actual drive hammer drop, inch

D_o, D_i = actual sampler outside and inside diameter, respectively, inches

Burmister's correction assumes that penetration resistance (blowcount) is inversely proportional to the hammer energy. For a hammer other than a 140# hammer with 30" drop the hammer energy correction is equal to the ratio of the theoretical hammer energy (weight times drop) to the theoretical SPT hammer energy, or C_H = (W * H) / (140 lb * 30 in).

Burmister's correction assumes that penetration resistance (blowcount) is proportional to the annular end area of the drive sampler. For example, California drive samplers with Do=3 inch and Di=2.42 inch the sampler size correction factor is the ratio of the annular area of an SPT split spoon to that of the California Sampler, or C_S =[2.02-1.3752]/[32-2.422] = 0.67.

To normalize the field SPT and California blowcounts to a hammer with 60% efficiency, an energy correction factor equal to Hammer Efficiency (%) / 60% was applied to the field blowcounts. Hammer efficiency was determined by Pile Driving Analyzer (PDA) measurement and/or by published correlations with the CME Automatic Hammer blow count rate (USBR, 1999). Hammer efficiency measurements are presented in the attachments to this appendix.

The correction factors applied to obtain N^*_{60} are shown in the "NOTES" section of the boring record title block. Corrected N^*_{60} are primarily used, with due engineering judgment, for qualitative assessment of in place density or consistency.

Relative Density and Consistency

Equivalent SPT N^*_{60} values were used as the basis for classifying relative density of granular/cohesionless soils. Wherever possible consistency classification of cohesive soils was based on undrained shear strength estimated in the field with a pocket penetrometer and/or Torvane or by testing in the laboratory. Where pocket penetrometer or other tests could not be performed, consistency of cohesive soils was estimated by correlations to Equivalent SPT N^*_{60}. The correlations for consistency and relative density are shown in the Boring Record Legend in the attachments to this appendix. Drive sample field blow counts, SPT N^*_{60} values, pocket penetrometer/Torvane readings, and corresponding density/consistency classifications are presented on the boring records.

Borehole Abandonment

At the completion of the drilling groundwater was measured (where possible) and the borings were abandoned by backfilling the borehole with as indicated on the records. Where necessary excess cuttings and drilling fluids were placed in 55-gallon drums, sampled, and tested for contaminants, temporarily stored at an approved location, and legally disposed of off-site. The surface was patched with cold mix asphalt concrete or quickset concrete, as necessary. Notes describing the borehole abandonment are presented in the title block of each boring record.

Sample Handling and Transport

Geotechnical samples were sealed to prevent moisture loss, packed in appropriate protective containers, and transported to the geotechnical laboratory for further examination and geotechnical testing.

Laboratory Testing

The soil samples were further examined and tested in the laboratory and classified in accordance with the Unified Soil Classification System following ASTM D 2487 and D 2488 (see the Key for Soil Classification in the attachments to this appendix). Field classifications presented on the records were modified where necessary on the basis of the laboratory test results. Descriptions of the laboratory tests performed and a summary of the results are presented in Appendix B.

LIST OF ATTACHMENTS

Tables

Summary of Field Explorations

Figures

Boring Record Legend

Key for Soil Classification

Boring Records

Hammer Efficiency Calibrations

Summary of Field Explorations

GROUP DELTA

Table A-1: Summary of Field Explorations

Exploration No.	Completion Date	Ground Surface Elevation (feet)	Total Depth (feet)	Groundwater	
				Depth (feet)	Elevation (feet)
B-1	2/17/22	345	50.0	16.1	328.9
B-2	2/17/22	346	21.5	15.9	330.1
B-3	2/17/22	345.5	21.5	14.5	331.0

Boring Record Legend, Key for Soil Classification, and Boring Records

SOIL IDENTIFICATION AND DESCRIPTION SEQUENCE

Sequence		Refer to Section: Field	Refer to Section: Lab	Required	Optional
1	Group Name	2.5.2	3.2.2	●	
2	Group Symbol	2.5.2	3.2.2	●	
	Description Components				
3	Consistency of Cohesive Soil	2.5.3	3.2.3	●	
4	Apparent Density of Cohesionless Soil	2.5.4		●	
5	Color	2.5.5		●	
6	Moisture	2.5.6		●	
7	Percent or Proportion of Soil	2.5.7	3.2.4	●	◍
	Particle Size	2.5.8	2.5.8	●	◍
	Particle Angularity	2.5.9			○
	Particle Shape	2.5.10			○
8	Plasticity (for fine-grained soil)	2.5.11	3.2.5		○
9	Dry Strength (for fine-grained soil)	2.5.12			○
10	Dilatency (for fine-grained soil)	2.5.13			○
11	Toughness (for fine-grained soil)	2.5.14			○
12	Structure	2.5.15			○
13	Cementation	2.5.16		●	
14	Percent of Cobbles and Boulders	2.5.17		●	
	Description of Cobbles and Boulders	2.5.18		●	
15	Consistency Field Test Result	2.5.3		●	
16	Additional Comments	2.5.19			○

Describe the soil using descriptive terms in the order shown

Minimum Required Sequence:

USCS Group Name (Group Symbol); Consistency or Density; Color; Moisture; Percent or Proportion of Soil; Particle Size; Plasticity (optional).

◍ = optional for non-Caltrans projects

Where applicable:

Cementation; % cobbles & boulders;
Description of cobbles & boulders;
Consistency field test result

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010)

HOLE IDENTIFICATION

Holes are identified using the following convention:

H-YY-NNN

Where:
H: Hole Type Code
YY: 2-digit year
NNN: 3-digit number (001-999)

Hole Type Code	Description
A	Auger boring (hollow or solid stem, bucket)
R	Rotary drilled boring (conventional)
RC	Rotary core (self-cased wire-line, continuously-sampled)
RW	Rotary core (self-cased wire-line, not continuously sampled)
P	Rotary percussion boring (Air)
HD	Hand driven (1-inch soil tube)
HA	Hand auger
D	Driven (dynamic cone penetrometer)
CPT	Cone Penetration Test
O	Other (note on LOTB)

Description Sequence Examples:

SANDY lean CLAY (CL); very stiff; yellowish brown; moist; mostly fines; some SAND, from fine to medium; few gravels; medium plasticity; PP=2.75.

Well-graded SAND with SILT and GRAVEL and COBBLES (SW-SM); dense; brown; moist; mostly SAND, from fine to coarse; some fine GRAVEL; few fines; weak cementation; 10% GRANITE COBBLES; 3 to 6 inches; hard; subrounded.

Clayey SAND (SC); medium dense, light brown; wet; mostly fine sand,; little fines; low plasticity.

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BORING RECORD LEGEND #1

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (CTM 216 - 06)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])

SAMPLER GRAPHIC SYMBOLS

- Standard Penetration Test (SPT)
- Standard California Sampler
- Modified California Sampler
- Shelby Tube
- Piston Sampler
- NX Rock Core
- HQ Rock Core
- Bulk Sample
- Other (see remarks)

DRILLING METHOD SYMBOLS

- Auger Drilling
- Rotary Drilling
- Dynamic Cone or Hand Driven
- Diamond Core

WATER LEVEL SYMBOLS

- First Water Level Reading (during drilling)
- Static Water Level Reading (after drilling, date)

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010)

DEFINITIONS FOR CHANGE IN MATERIAL

Term	Definition	Symbol
Material Change	Change in material is observed in the sample or core, and the location of change can be accurately measured.	———
Estimated Material Change	Change in material cannot be accurately located because either the change is gradational or because of limitations in the drilling/sampling methods used.	- - - - - - - - -
Soil/Rock Boundary	Material changes from soil characteristics to rock characteristics.	

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BORING RECORD LEGEND #2

CONSISTENCY OF COHESIVE SOILS

Descriptor	Shear Strength (tsf)	Pocket Penetrometer, PP Measurement (tsf)	Torvane, TV. Measurement (tsf)	Vane Shear, VS. Measurement (tsf)
Very Soft	< 0.12	< 0.25	< 0.12	< 0.12
Soft	0.12 - 0.25	0.25 - 0.50	0.12 - 0.25	0.12 - 0.25
Medium Stiff	0.25 - 0.50	0.50 - 1.0	0.25 - 0.50	0.25 - 0.50
Stiff	0.50 - 1.0	1.0 - 2.0	0.50 - 1.0	0.50 - 1.0
Very Stiff	1.0 - 2.0	2.0 - 4.0	1.0 - 2.0	1.0 - 2.0
Hard	> 2.0	> 4.0	> 2.0	> 2.0

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60} - Value (blows / foot)
Very Loose	0 - 5
Loose	5 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	> 50

MOISTURE

Descriptor	Criteria
Dry	No discernable moisture
Moist	Moisture present, but no free water
Wet	Visible free water

PERCENT OR PROPORTION OF SOILS

Descriptor	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

PARTICLE SIZE

Descriptor		Size (in)
Boulder		> 12
Cobble		3 - 12
Gravel	Coarse	3/4 - 3
	Fine	1/5 - 3/4
Sand	Coarse	1/16 - 1/5
	Medium	1/64 - 1/16
	Fine	1/300 - 1/64
Silt and Clay		< 1/300

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CONSISTENCY OF COHESIVE SOILS VS. N_{60}

Description	SPT N_{60} (blows / foot)
Very Soft	0 - 2
Soft	2 - 4
Medium Stiff	4 - 8
Stiff	8 - 15
Very Stiff	15 - 30
Hard	> 30

Ref: Peck, Hansen, and Thornburn, 1974, “Foundation Engineering”, Second Edition

Note: Only to be used (with caution) when pocket penetrometer or other data on undrained shear strength are unavailable. Not allowed by Caltrans Soil and Rock Logging and Classificaton Manual, 2010

CEMENTATION

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010), with the exception of consistency of cohesive soils vs. N_{60}.

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BORING RECORD LEGEND #3

ROCK GRAPHIC SYMBOLS

- IGNEOUS ROCK
- SEDIMENTARY ROCK
- METAMORPHIC ROCK

BEDDING SPACING

Descriptor	Thickness or Spacing
Massive	> 10 ft
Very thickly bedded	3 to 10 ft
Thickly bedded	1 to 3 ft
Moderately bedded	3-5/8 inches to 1 ft
Thinly bedded	1-1/4 to 3-5/8 inches
Very thinly bedded	3/8 inch to 1-1/4 inches
Laminated	< 3/8 inch

WEATHERING DESCRIPTORS FOR INTACT ROCK

	Diagnostic Features					
	Chemical Weathering-Discoloration-Oxidation		Mechanical Weathering and Grain Boundary Conditions	Texture and Solutioning		
Descriptor	**Body of Rock**	**Fracture Surfaces**		**Texture**	**Solutioning**	**General Characteristics**
Fresh	No discoloration, not oxidized	No discoloration or oxidation	No separation, intact (tight)	No change	No solutioning	Hammer rings when crystalline rocks are struck.
Slightly Weathered	Discoloration or oxidation is limited to surface of, or short distance from, fractures; some feldspar crystals are dull	Minor to complete discoloration or oxidation of most surfaces	No visible separation, intact (tight)	Preserved	Minor leaching of some soluble minerals may be noted	Hammer rings when crystalline rocks are struck. Body of rock not weakened.
Moderately Weathered	Discoloration or oxidation extends from fractures usually throughout; Fe-Mg minerals are "rusty"; feldspar crystals are "cloudy"	All fracture surfaces are discolored or oxidized	Partial separation of boundaries visible	Generally preserved	Soluble minerals may be mostly leached	Hammer does not ring when rock is struck. Body of rock is slightly weakened.
Intensely Weathered	Discoloration or oxidation throughout; all feldspars and Fe-Mg minerals are altered to clay to some extent; or chemical alteration produces in situ disaggregation (refer to grain boundary conditions)	All fracture surfaces are discolored or oxidized; surfaces are friable	Partial separation, rock is friable; in semi-arid conditions, granitics are disaggregated	Altered by chemical disintegration such as via hydration or argillation	Leaching of soluble minerals may be complete	Dull sound when struck with hammer; usually can be broken with moderate to heavy manual pressure or by light hammer blow without reference to planes of weakness such as incipient or hairline fractures or veinlets. Rock is significantly weakened.
Decomposed	Discolored of oxidized throughout, but resistant minerals such as quartz may be unaltered; all feldspars and Fe-Mg minerals are completely altered to clay		Complete separation of grain boundaries (disaggregated)	Resembles a soil; partial or complete remnant rock structure may be preserved; leaching of soluble minerals usually complete		Can be granulated by hand. Resistant minerals such as quartz may be present as "stringers" or "dikes".

Note: Combination descriptors (such as "slightly weathered to fresh") are used where equal distribution of both weathering characteristics is present over significant intervals or where characteristics present are "in between" the diagnostic feature. However, combination descriptors should not be used where significant identifiable zones can be delineated. Only two adjacent descriptors shall be combined. "Very intensely weathered" is the combination descriptor for "decomposed to intensely weathered".

RELATIVE STRENGTH OF INTACT ROCK

Descriptor	Uniaxial Compressive Strength (psi)
Extremely Strong	> 30,000
Very Strong	14,500 - 30,000
Strong	7,000 - 14,500
Medium Strong	3,500 - 7,000
Weak	700 - 3,500
Very Weak	150 - 700
Extremely Weak	< 150

ROCK HARDNESS

Descriptor	Criteria
Extremely Hard	Specimen cannot be scratched with pocket knife or sharp pick; can only be chipped with repeated heavy hammer blows
Very hard	Specimen cannot be scratched with pocket knife or sharp pick; breaks with repeated heavy hammer blows
Hard	Specimen can be scratched with pocket knife or sharp pick with heavy pressure; heavy hammer blows required to break specimen
Moderately Hard	Specimen can be scratched with pocket knife or sharp pick with light or moderate pressure; breaks with moderate hammer blows
Moderately Soft	Specimen can be grooved 1/6 in. with pocket knife or sharp pick with moderate or heavy pressure; breaks with light hammer blow or heavy hand pressure
Soft	Specimen can be grooved or gouged with pocket knife or sharp pick with light pressure, breaks with light to moderate hand pressure
Very Soft	Specimen can be readily indented, grooved, or gouged with fingernail, or carved with pocket knife; breaks with light hand pressure

CORE RECOVERY CALCULATION (%)

$$\frac{\Sigma \text{ Length of the recovered core pieces (in.)}}{\text{Total length of core run (in.)}} \times 100$$

RQD CALCULATION (%)

$$\frac{\Sigma \text{ Length of intact core pieces > 4 in.}}{\text{Total length of core run (in.)}} \times 100$$

FRACTURE DENSITY

Descriptor	Criteria
Unfractured	No fractures
Very Slightly Fractured	Lengths greater 3 ft
Slightly Fractured	Lengths from 1 to 3 ft, few lengths outside that range
Moderately Fractured	Lengths mostly in range of 4 in. to 1 ft, with most lengths about 8 in.
Intensely Fractured	Lengths average from 1 in. to 4 in. with scattered fragmented intervals with lengths less than 4 in.
Very Intensely Fractured	Mostly chips and fragments with few scattered short core lengths

Ref.: Caltrans Soil and Rock Logging Classification, and Presentation Manual (2010)

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BORING RECORD LEGEND #4

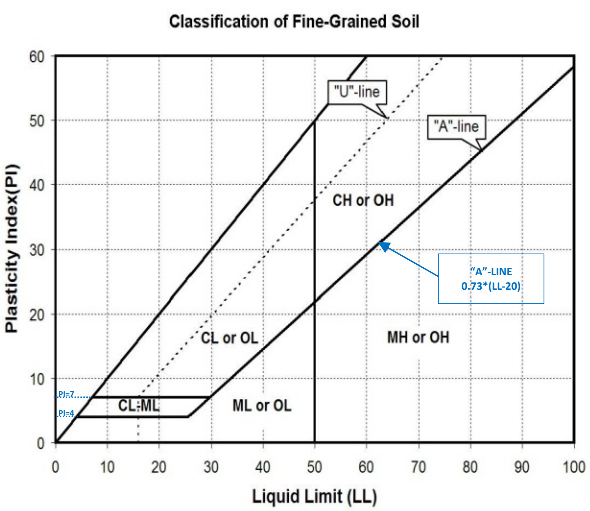
CLASSIFICATION OF INORGANIC FINE GRAINED SOILS (Soils with ≥50% finer than No. 200 Sieve)

GROUP SYMBOL				GROUP NAME
CL	<30% plus No. 200	<15% plus No. 200		Lean clay
		15-25% plus No. 200	%sand ≥ %gravel	Lean clay with sand
			%sand < %gravel	Lean clay with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy lean clay
			≥15% gravel	Sandy lean clay with gravel
		% sand < % gravel	<15% sand	Gravelly lean clay
			≥15% sand	Gravelly lean clay with sand
ML	<30% plus No. 200	<15% plus No. 200		Silt
		15-25% plus No. 200	%sand ≥ %gravel	Silt with sand
			%sand < %gravel	Silt with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy silt
			≥15% gravel	Sandy silt with gravel
		% sand < % gravel	<15% sand	Gravelly silt
			≥15% sand	Gravelly silt with sand
CH	<30% plus No. 200	<15% plus No. 200		Fat clay
		15-25% plus No. 200	%sand ≥ %gravel	Fat clay with sand
			%sand < %gravel	Fat clay with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy fat clay
			≥15% gravel	Sandy fat clay with gravel
		% sand < % gravel	<15% sand	Gravelly fat clay
			≥15% sand	Gravelly fat clay with sand
MH	<30% plus No. 200	<15% plus No. 200		Elastic silt
		15-25% plus No. 200	%sand ≥ %gravel	Elastic silt with sand
			%sand < %gravel	Elastic silt with gravel
	≥30% plus No. 200	% sand ≥ % gravel	<15% gravel	Sandy elastic silt
			≥15% gravel	Sandy elastic silt with gravel
		% sand < % gravel	<15% sand	Gravelly elastic silt
			≥15% sand	Gravelly elastic silt with sand

Reference:
ASTM D 2487 and 2488

Laboratory Classification of Clay and Silt

REFERENCE: Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010).



CL: LL<50; above A-Line.

CH: LL≥50; above A-Line.

ML: LL<50; below A-Line, or PI<4, or Non-Plastic

MH: LL≥50; below A-Line.

CL-ML: above A-Line and PI=4 to 7

CL/CH, ML/MH: at or near LL=50

ML/CL, MH/CH: at or near the A-Line

Field Identification of Clays and Silts

Group Symbol	Dry Strength	Dilatancy	Toughness	Plasticity
ML	None to low	Slow to rapid	Low or thread cannot be formed	Low to nonplastic
CL	Medium to high	None to slow	Medium	Medium
MH	Low to medium	None to slow	Low to medium	Low to medium
CH	High to very high	None	High	High

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KEY FOR SOIL CLASSIFICATION #1

CLASSIFICATION OF COARSE-GRAINED SOILS (Soils with <50% "fines" passing No. 200 Sieve)

				GROUP SYMBOL		GROUP NAME
GRAVEL % GRAVEL> % SAND	≤5%fines (<5% fines)	Well-graded		GW	<15% sand	Well-graded gravel
					≥15% sand	Well-graded gravel with sand
		Poorly graded		GP	<15% sand	Poorly graded gravel
					≥15% sand	Poorly graded gravel with sand
	10%fines (5-12% fines)	Well-graded	Fines=ML or MH	GW-GM	<15% sand	Well-graded gravel with silt
					≥15% sand	Well-graded gravel with silt and sand
			Fines=CL or CH	GW-GC	<15% sand	Well-graded gravel with clay
					≥15% sand	Well-graded gravel with clay and sand
		Poorly graded	Fines=ML or MH	GP-GM	<15% sand	Poorly graded gravel with silt
					≥15% sand	Poorly graded gravel with silt and sand
			Fines=CL or CH	GP-GC	<15% sand	Poorly graded gravel with clay
					≥15% sand	Poorly graded gravel with clay and sand
	≥15%fines (>12% fines)		Fines=ML or MH	GM	<15% sand	Silty gravel
					≥15% sand	Silty gravel with sand
			Fines=CL or CH	GC	<15% sand	Clayey gravel
					≥15% sand	Clayey gravel with sand
SAND % SAND≥ % GRAVEL	≤5%fines (<5% fines)	Well-graded		SW	<15% gravel	Well-graded sand
					≥15% gravel	Well-graded sand with gravel
		Poorly graded		SP	<15% gravel	Poorly-graded sand
					≥15% gravel	Poorly-graded sand with gravel
	10%fines (5-12% fines)	Well-graded	Fines=ML or MH	SW-SM	<15% gravel	Well-graded sand with silt
					≥15% gravel	Well-graded sand with silt and gravel
			Fines=CL or CH	SW-SC	<15% gravel	Well-graded sand with clay
					≥15% gravel	Well-graded sand with clay and gravel
		Poorly graded	Fines=ML or MH	SP-SM	<15% gravel	Poorly graded sand with silt
					≥15% gravel	Poorly graded sand with silt and gravel
			Fines=CL or CH	SP-SC	<15% gravel	Poorly graded sand with clay
					≥15% gravel	Poorly graded sand with clay and gravel
	≥15%fines (>12% fines)		Fines=ML or MH	SM	<15% gravel	Silty sand
					≥15% gravel	Silty sand with gravel
			Fines=CL or CH	SC	<15% gravel	Clayey sand
					≥15% gravel	Clayey sand with gravel

Reference:

ASTM D 2487 and 2488

Note: Values estimated to nearest 5% to be used for visual identification, values in parentheses to be used for classification when based on laboratory grain size data.

Granular Soil Gradation Parameters

Coefficient of Uniformity: $C_u = D_{60}/D_{10}$

Coefficient of Curvature: $Cc = D_{30}^2 / (D_{60} \times D_{10})$

D_{10} = 10% of soil is finer than this diameter

D_{30} = 30% of soil is finer than this diameter

D_{60} = 60% of soil is finer than this diameter

Group Symbol	Gradation or Plasticity Requirement
SW	$C_u > 6$ and $1 \leq C_c \leq 3$
GW	$C_u > 4$ and $1 \leq C_c \leq 3$
GP or SP	Clean gravel or sand not meeting requirement for SW or GW
SM or GM	Non-plastic fines or below A-Line or PI<4
SC or GC	Plastic fines or above A-Line and PI>7

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KEY FOR SOIL CLASSIFICATION #2

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Santee Community Center	IR786	**B-1**

SITE LOCATION	START	FINISH	SHEET NO.
Santee, CA	2/17/2022	2/17/2022	1 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Tri-County	DIEDRICH D120	Hollow Stem Auger	S. Narveson	M. Givens

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	85	8	50	345	⊻ 16.1 / *328.9* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
Bulk; SPT (1.4"); MC (2.4")	N_{60} =1.42 N_{SPT} =0.95N_{MC}	▼ 16.1 / *328.9*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
															ASPHALT: 4-inch
															AGGREGATE BASE: 6-inch
			B-1								43:16	-200 PI EI CR			CLAYEY SAND (SC): very dark grey (2.5YR 3/1); moist; little SAND; few GRAVEL; medium plasticity (FILL). (Fines = 42.5%)
5	340		R-2-2 R-2-1	3 5 6	11	10			13	95		DS			CLAYEY SAND (SC): loose to medium dense; dark grey (2.5YR 4/1); trace of GRAVEL; medium plasticity (ALLUVIUM).
10	335		S-3	2 3 7	10	14									SILTY SAND (SM): medium dense; dark grey (2.5YR 4/1); moist; fine grained SAND; some fines; non-plastic; micaceous.
15	330		R-4-2 R-4-1	3 6 7	13	12			20	95		-200			Well-graded SAND (SW): medium dense; greyish brown (2.5YR, 5/2); wet; fine to coarse grained SAND; trace fines and GRAVELS; slightly micaceous. (Fines = 4.7%)
20	325		S-5	1 2 4	6	9						-200			Loose; dark greyish brown (2.5YR 4/2); thinly bedded; iron oxide stainning. (Fines = 2.9%)
25	320		R-6-2 R-6-1	3 7 8	15	14			31	91		-200			Poorly-graded SAND with SILT (SP-SM): medium dense; dark greyish brown (2.5YR 4/2); wet; fine to medium grained SAND; trace of fine GRAVEL; slighlty micacenous. (Fines = 5%)
															GRANITIC ROCK: massive; grey (5YR, 6/1); highly weathered to decomposed; soft; Well-graded SAND with CLAY (SW-SC): very dense; wet; fine to coarse grained

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Santee Community Center	IR786	B-1

SITE LOCATION	START	FINISH	SHEET NO.
Santee, CA	2/17/2022	2/17/2022	2 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Tri-County	DIEDRICH D120	Hollow Stem Auger	S. Narveson	M. Givens

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	85	8	50	345	⊻ 16.1 / *328.9* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
Bulk; SPT (1.4"); MC (2.4")	N_{60} =1.42 N_{SPT} =0.95N_{MC}	▼ 16.1 / *328.9*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			S-7	50/3"	REF	REF									SAND; few to little fines; non-plastic to low plasticity.
35	310		S-8	60/3"	REF	REF									Iron oxide stainning
40	305		S-9	50/2"	REF	REF									Trace of iron oxide staining.
45	300		S-10	50/2"	REF	REF									
50	295		S-11	50/1"	REF	REF									Boring terminated at 50 feet below the existing ground surface (bgs). Groundwater was encountered at 16.1 feet bgs Backfilled with Cement and bentonite gel.
55	290														

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THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE

A-2 b

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Santee Community Center	IR786	B-2

SITE LOCATION	START	FINISH	SHEET NO.
Santee, CA	2/17/2022	2/17/2022	1 of 1

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Tri-County	DIEDRICH D120	Hollow Stem Auger	S. Narveson	M. Givens

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	85	8	21.5	346	∇ 15.9 / *330.1* DURING DRILLING; AFTER DRILLING ▼ 15.9 / *330.1*

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk; SPT (1.4"); MC (2.4")	N_{60} =1.42 N_{SPT} =0.95N_{MC}

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0	346														ASPHALT: 4-inch
															AGGREGATE BASE: 6-inch
	345														CLAYEY SAND (SC): dark gray (2.5YR 4/1); moist; little GRAVEL and cobble; medium plasticity (FILL).
			B-1									R			
5	340		S-2	18 8 8	16	23						-200			Poorly-graded SAND with SILT (SP-SM): medium dense; moist; fine to medium grained SAND; few fines (ALLUVIUM). (Fines = 9.5%)
10	335		R-3-2 R-3-1	14 23 27	50	48									No recovery.
15	330		S-4	4 4 2	6	9						-200			Well-graded SAND (SW): loose; grey (2.5YR 5/1); wet; fine to coarse grained SAND; trace of fines; trace of fine GRAVEL. (Fines = 3.1%)
20	325		R-5-2 R-5-1	12 9 13	22	21			37	90					SILTY SAND (SM): medium dense; very dark gray (2.5YR 3/1); wet; fine grained SAND; some fines; trace of GRAVEL; non-plastic; micaceous.
															Boring terminated at 21.5 feet below the existing ground surface (bgs). Groundwater was encountered at 15.9 feet bgs. Backfilled with Cement and Bentonite gel.
25	320														

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FIGURE

A-3

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Santee Community Center	IR786	**B-3**

SITE LOCATION	START	FINISH	SHEET NO.
Santee, CA	2/17/2022	2/17/2022	1 of 1

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
Tri-County	DIEDRICH D120	Hollow Stem Auger	S. Narveson	M. Givens

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
Hammer: 140 lbs., Drop: 30 in.	85	8	21.5	345.5	⊻ 14.5 / *331.0* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	AFTER DRILLING
Bulk; SPT (1.4"); MC (2.4")	N_{60} =1.42 N_{SPT} =0.95N_{MC}	▼ 14.5 / *331.0*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	SPT N^*_{60}	RECOVERY (%)	RQD (%)	MOISTURE (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (LL:PI)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
	345														ASPHALT: 3-inch
															AGGREGATE BASE: 6-inch
															CLAYEY SAND (SC): very dark grey (2.5YR 3/1); moist; little GRAVEL and cobble; medium plasticity (FILL).
			B-1												
5	340		R-2-2 R-2-1	2 11 11	22	21						PA			Medium dense; fine grained SAND; some fines; few GRAVEL. (Gravel = 7%, Sand = 55%, Fines = 38%)
10	335		S-3	9 5 5	10	14									Lean CLAY with SAND (CL): medium stiff to stiff; very dark (2.5YR 3/1); moist; little fine grained SAND; little GRAVEL and cobble; medium plasticity; PP = 1.0 tsf.
															SILTY SAND (SM): medium dense; dark gray (5YR 4/1); wet; fine grained SAND; some fines; non-plastic; micaceous (ALLUVIUM).
15	330		R-4-2 R-4-1	3 4 4	8	8			46	73		-200			SANDY SILT (ML): loose; dark grey (5YR 4/1); wet; some fine grained SAND; non-plastic; micaceous. (Fines = 50.5%)
20	325		S-5	2 5 7	12	17									Well-graded SAND (SW): medium dense; grayish brown (2.5YR 5/2); wet; fine to coarse grained SAND; trace of fines; trace of fine GRAVEL.
															Boring terminated at 21.5 feet below the existing ground surface (bgs). Groundwater was encountered at 15.9 feet bgs. Backfilled with Cement and Bentonite gel.
25	320														

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FIGURE
A-4

Hammer Efficiency Calibration

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For the Mobile Drill 77 at the calibration borehole, the average energy transfer ratio from individual sample depths ranged from 81-87%.

Table 5: Summary of SPT results

Sample Depth (ft)	Average ETR (%)	Average BPM (bpm)
5	81.0	51
10	82.6	52
15	87.2	51
20	85.4	51

For an overall transfer ratio of:

SPT Rig	Overall Transfer Efficiency	Hammer Operating Rate (BPM)
Drill Rig 01	89	50
Drill Rig 35	86	56
Drill Rig 78	85	50
Drill Rig 72	81	52
Drill Rig 77	84	51

We appreciate the opportunity to be of assistance to you. Please do not hesitate to contact us if you have any questions regarding this report, or if we may be of further service.

Respectfully,

GRL Engineers, Inc.

REGISTERED PROFESSIONAL ENGINEER
Camilo A Alvarez
No. 67938
Exp. 6/30/21
Civil
STATE OF CALIFORNIA

Camilo Alvarez, P.E.

Gabriela Wong, EIT

APPENDIX B - LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

B.1 General

The laboratory testing was performed using appropriate American Society for Testing and Materials (ASTM) and Caltrans Test Methods (CTM).

Modified California drive samples, Standard Penetration Test (SPT) drive samples and bulk samples collected during the field investigation were carefully sealed in the field to prevent moisture loss. The samples of earth materials were then transported to the laboratory for further examination and testing. Tests were performed on selected samples as an aid in classifying the earth materials and to evaluate their physical properties and engineering characteristics. Laboratory testing for this investigation included:

- Soil Classification: USCS (ASTM D 2487) and Visual Manual (ASTM D 2488);
- Moisture content (ASTM D 2216) and Dry Unit Weight (ASTM D 2937);
- Atterberg Limits (ASTM D 4318);
- Grain Size Distribution (ASTM D 422) & % Passing #200 Sieve (ASTM D 1140);
- Direct Shear (ASTM D 3080);
- Expansion Index (D 4829);
- Soil Corrosivity:
  - pH (CTM 643);
  - Water-Soluble Sulfate (ASTM D 516, CTM 417);
  - Water-Soluble Chloride(Ion-Specific Probe, CTM 422);
  - Minimum Electrical Resistivity (CTM 643);
- Resistance R-Value (CTM 301).

A summary of laboratory test results is presented in Table B-1. Brief descriptions of the laboratory testing program and test results are presented below.

B.2 Soil and Rock Classification

Earth materials recovered from subsurface explorations were classified in general accordance with Caltrans' "Soil and Rock Logging Classification Manual, 2010". The subsurface soils were classified visually / manually in the field in accordance with the Unified Soil Classification System (USCS) following ASTM D 2488; soil classifications were modified as necessary based on testing in the laboratory in accordance with ASTM D 2487. The details of the soil and rock classification systems and boring records presenting the classifications are presented in Appendix A.

B.3 Moisture Content and Dry Unit Weight

The in-situ moisture content of selected bulk, SPT and Ring samples was determined by oven drying in general accordance with ASTM D 2216. Selected California Ring samples were trimmed flush in the metal rings and wet weight was measured. After drying, the dry weight of each sample was measured, volume and weight of the metal containers was measured, and moisture content and dry density were calculated in general accordance with ASTM D 2216 and D 2937. Results of these tests are presented in Table B-1 and on the boring records in Appendix A.

B.4 Atterberg Limits

Characterization of the fine-grained fractions of soils was evaluated using the Atterberg Limits. This test includes Liquid Limit and Plastic Limit tests to determine the Plasticity Index in accordance with ASTM D 4318. Results of these tests are presented on the boring records in Appendix A, are summarized in Table B-1, and are attached at the end of this Appendix.

B.5 Grain Size Distribution and Percent Passing No. 200 Sieve:

Representative samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. The percentage of fines (soil passing No. 200 sieve) was determined for selected samples in accordance with ASTM D 1140. For selected samples the washed fraction retained on the No. 200 sieve was then screened on a No. 4 sieve, and the fraction retained on No. 4 was weighed to determine the percentage of gravel. For selected samples, the washed material retained on No. 200 sieve was shaken through a standard stack of sieves in accordance with ASTM D 422 to determine the grain size distribution. The relative proportion (or percentage) by dry weight of gravel (retained on No. 4 sieve), sand (passing No. 4 and retained on No. 200 sieve), and fines (passing No. 200 sieve) are listed on the boring records in Appendix A and summarized in Table B-1.

B.6 Direct Shear Test

To determine the drained shear strength parameters of the on-site soils, direct shear tests were performed on selected in situ samples in accordance with ASTM D 3080. After the initial weight and volume measurements were made, the sample was placed in the shear machine, and a selected normal load was applied. The sample was saturated or kept at field moisture (to model worst case field conditions), allowed to consolidate under the selected normal load, and then sheared to failure. Shear rate was selected to maintain drained conditions. Shear stress and vertical/horizontal sample deformations were monitored throughout the test. The process was repeated on additional samples of the same soil material at two additional normal loads. The test results are presented at the end of this appendix.

B.7 Expansion Index

The expansion potential of the site soils was estimated using the Expansion Index Test in accordance with ASTM D 4829. The results of this test are listed in Table B-1.

B.8 Soil Corrosivity

Tests were performed in order to determine corrosion potential of site soils on concrete and ferrous metals. Corrosivity testing included minimum electrical resistivity and soil pH (Caltrans method 643), water-soluble chlorides (Orion 170A+ Ion Probe or Caltrans Test Method 422), and water-soluble sulfates (ASTM D 516). The test results are summarized presented at the end of this appendix.

B.9 R-Value

Resistance "R" Value tests were performed by stabilometer method on selected bulk samples of the subgrade soils. The tests were conducted in general accordance with CTM 301. The test results are attached at the end of this appendix.

B.10 List of Attached Figures

The following tables and figures are attached and complete this appendix:

Tables

Summary of Laboratory Test Results

Figures

Atterberg Limits Test Results
Grain Size Analysis Test Results
Direct Shear Test Results
Corrosion Test Results
R-Value Test Results

Boring No.	Sample No.	Depth (ft)	Sample Type	Geologic Unit	USCS Group Symbol	SPT N*60 (blows/ft)	Undrained Shear Strength, Su (ksf)			Moisture Content (%)	Dry Unit Weight (pcf)	Total Unit Wt (pcf)	Atterberg Limits			Grain Size Distribution (%) by dry weight			Clay	Other Tests
							Pocket Pen.	Mini Vane	UU Test				LL	PL	PI	Gravel	Sand	Fines		
B-1	B-1	0.0	BULK		SC								43	16	27			42.5		-200, PI, EI, CR
B-1		5.0	MC		SC	10														DS
B-1	R-2-2	5.5			SC					13.0	95	107								
B-1	R-2-1	6.0			SC															
B-1	S-3	10.0	SPT		SM	14														
B-1		15.0	MC		SW	12												4.7		-200
B-1	R-4-2	15.5			SW															
B-1	R-4-1	16.0			SW					20.0	95	114								
B-1	S-5	20.0	SPT		SW	9												2.9		-200
B-1		25.0	MC		SP-SM	14												5.0		-200
B-1	R-6-2	25.5			SP-SM															
B-1	R-6-1	26.0			SP-SM					31.0	91	119								
B-1	S-7	30.0	SPT		SW-SC	REF														
B-1	S-8	35.0	SPT		SW-SC	REF														
B-1	S-9	40.0	SPT		SW-SC	REF														
B-1	S-10	45.0	SPT		SW-SC	REF														
B-1	S-11	50.0	SPT		SW-SC	REF														
B-2	B-1	0.8	BULK		SC															R
B-2	S-2	5.0	SPT		SP-SM	23												9.5		-200
B-2		10.0	MC		SP-SM	48														
B-2	R-3-2	10.5			SP-SM															
B-2	R-3-1	11.0			SP-SM															
B-2	S-4	15.0	SPT		SW	9												3.1		-200
B-2		20.0	MC		SM	21														
B-2	R-5-2	20.5			SM															
B-2	R-5-1	21.0			SM					37.0	90	123								
B-3	B-1	0.8	BULK		SC															
B-3		5.0	MC		SC	21										7	55	38		PA
B-3	R-2-2	5.5			SC															
B-3	R-2-1	6.0			SC															

GROUP DELTA

GROUP DELTA CONSULTANTS. INC.
32 Mauchly, Suite B
Irvine, California 92618
Voice: (949) 450-2100 Fax: (949) 450-2108
www.GroupDelta.com

TABLE B-1: Summary of Laboratory Results

Project: Santee Community Center

Location: Santee, CA

Number: IR786

Boring No.	Sample No.	Depth (ft)	Sample Type	Geologic Unit	USCS Group Symbol	SPT N*60 (blows/ft)	Undrained Shear Strength, Su (ksf)			Moisture Content (%)	Dry Unit Weight (pcf)	Total Unit Wt (pcf)	Atterberg Limits			Grain Size Distribution (%) by dry weight			Clay	Other Tests
							Pocket Pen.	Mini Vane	UU Test				LL	PL	PI	Gravel	Sand	Fines		
B-3	S-3	10.0	SPT		CL	14														
B-3		15.0	MC		ML	8												50.5		-200
B-3	R-4-2	15.5			ML															
B-3	R-4-1	16.0			ML					46.0	73	107								
B-3	S-5	20.0	SPT		SW	17														

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32 Mauchly, Suite B
Irvine, California 92618
Voice: (949) 450-2100 Fax: (949) 450-2108
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TABLE B-1: Summary of Laboratory Results

Project: Santee Community Center

Location: Santee, CA

Number: IR786

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ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CALIFORNIA 92126

STANDARD METHOD FOR ATTERBERG LIMITS
ASTM D4318

REVISION 0, DATED 1/31/15

Project Name: HMC - Santee
Project No. : IR786
Sample No.: B-1
Sample Location: 1-5'

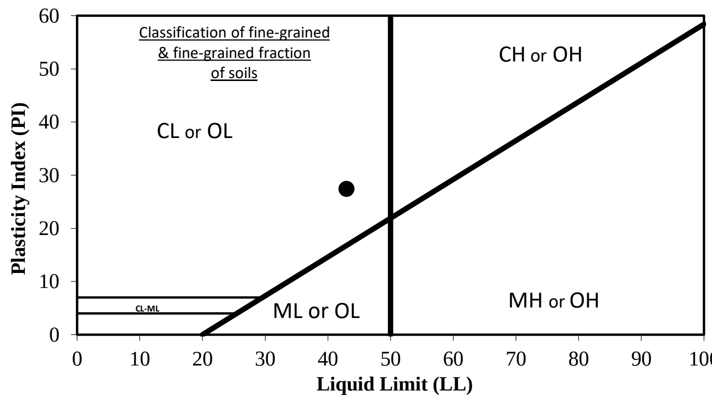
Tested By : J. Krehbiel **Date:** 03/09/22
Data Input By: J. Krehbiel **Date:** 03/10/22
Checked By: JLK **Date:** 03/10/22

	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			35	27	17	
Wet Wt. of Soil + Cont. (gm.)	17.98	17.71	28.66	27.53	28.33	
Dry Wt. of Soil + Cont. (gm.)	17.06	16.81	23.50	22.77	23.14	
Wt. of Container (gm.)	11.18	11.04	11.17	11.71	11.48	
Moisture Content (%) [Wn]	**15.65**	**15.60**	**41.85**	**43.04**	**44.51**	

LIQUID LIMIT	**43**
PLASTIC LIMIT	**16**
PLASTICITY INDEX	**27**

PI at "A" - Line = 0.73(LL-20) = **16.8**

One - Point Liquid Limit Calculation $LL=Wn(N/25)^{0.121}$



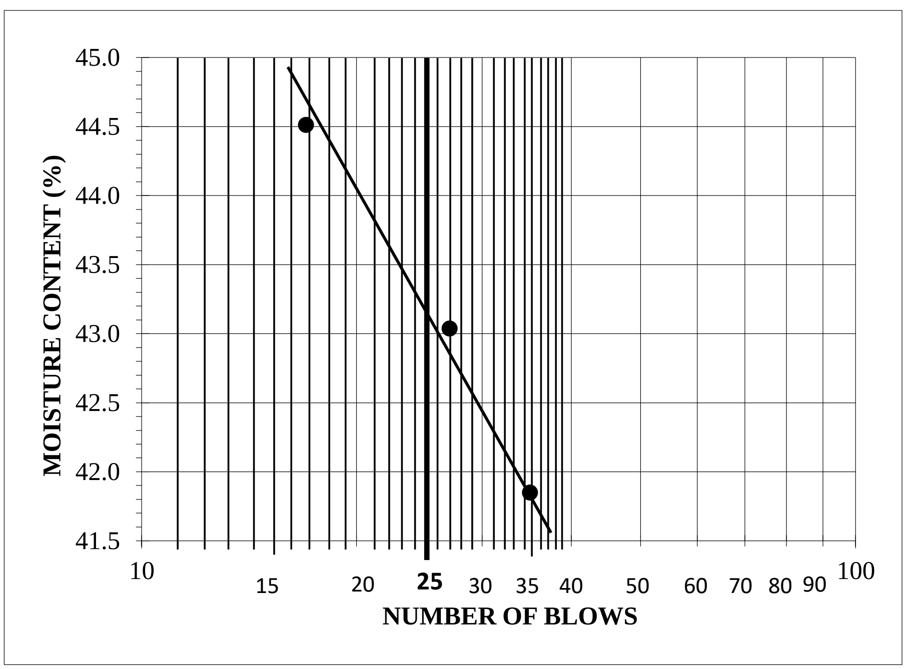
PROCEDURES USED

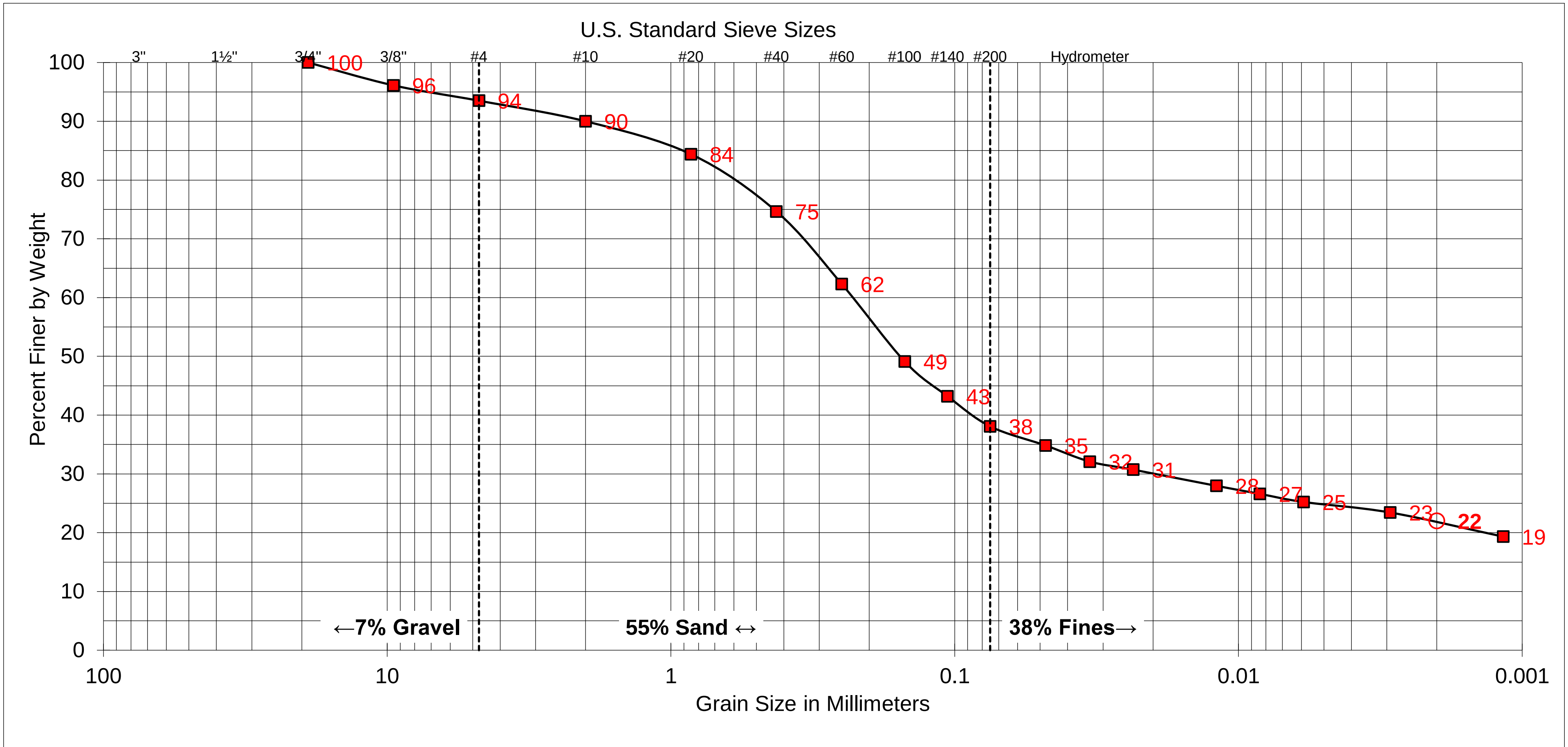
☐ Wet Preparation
Multipoint Wet Preparation

☒ Dry Preparation
Multipoint Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test





COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	
SAMPLE NUMBER:	B-3
SAMPLE DEPTH:	5'

UNIFIED SOIL CLASSIFICATION: SC

DESCRIPTION: CLAYEY SAND

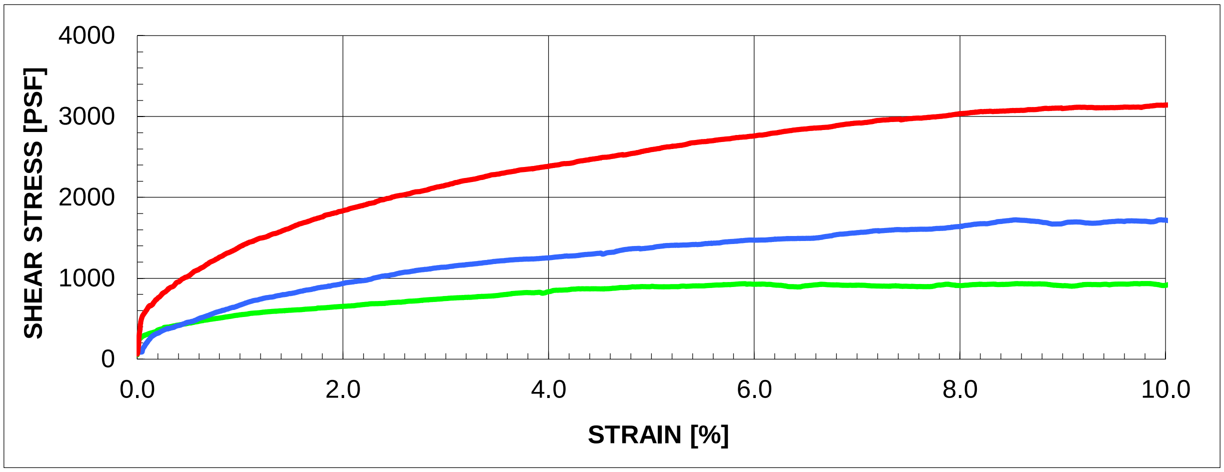
ATTERBERG LIMITS	
LIQUID LIMIT:	--
PLASTIC LIMIT:	--
PLASTICITY INDEX:	--

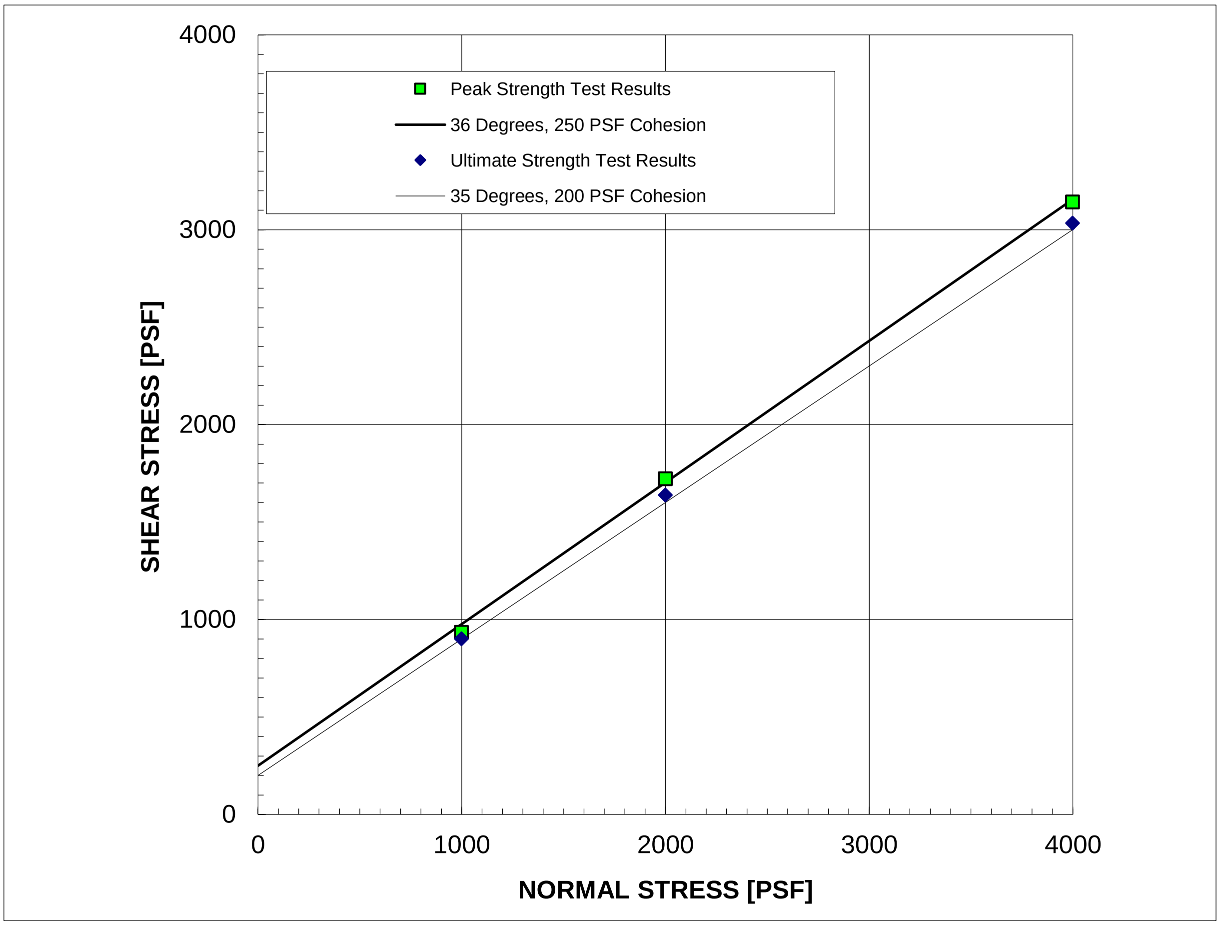
GROUP DELTA

SOIL CLASSIFICATION

Project No. IR786

FIGURE B-1.1





SAMPLE: B-1 @ 5'

Description:
Yellowish brown clayey sand (SC)

	PEAK	ULTIMATE
ϕ'	36 ^{o}	35 ^{o}
C'	250 PSF	200 PSF

	IN-SITU	AS-TESTED
γ_d	92.3 PCF	92.3 PCF
w_c	15.0 %	23.0 %

STRAIN RATE: 0.0010 IN/MIN

(Sample was consolidated and drained)

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DIRECT SHEAR TEST RESULTS

Project No. IR786

FIGURE B-1.1

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ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CALIFORNIA 92126

STANDARD TEST METHOD FOR EXPANSION INDEX (ASTM D4829)

REV.1, DATED 1/31/15

PROJECT: HMC - Santee Community Center
SAMPLE NUMBER: B-1 @ 1-5'
PROJECT NO.: IR786
SAMPLE DESCRIPTION: Dk.yellowish brown clayey sand (SC)
TESTED BY: J. Krehbiel DATE: 3/10/2022
CHECKED BY: JLK SAMPLED BY: SRN
LOCATION ____
% COARSE: ____
Page____ of ____

MOISTURE CONTENT

	TRIAL NO.	NO. 1	NO. 2	NO. 3	
	WET SOIL WEIGHT	**269.6**			g
	DRY SOIL WEIGHT	**244.5**			g
A	MOISTURE (((WET - DRY) / DRY) X 100)	10.3%			%

RING PREPARATION

		NO. 1	NO. 2	NO. 3	
B	WET WEIGHT OF SOIL AND RING	**597.0**			g
C	RING WEIGHT	**200.8**			g
D	WET WEIGHT OF SOIL (B - C)	396.2			g
E	DRY WEIGHT OF SOIL (D / ((A /100)+1))	359.2			g
F	DRY DENSITY OF SOIL (E * 0.3016)	108.3			g
G	CALCULATE (2.7 * A * F)	3011.8			
H	CALCULATE (168.5 - F)	60.2			
J	SAMPLE SATURATION (G / H)	50.0%			%

DIAL READINGS

K	INITIAL SETUP READING	0.200	in
L	10 MINUTE DRY READING	0.200	in
M	24 HOUR WET READING	0.268	in
N	EXPANSION INDEX ((M - L) * 1000)	**68**	EI

Remarks (if any) ____

FINAL MOISTURE CONTENT

O	WET WEIGHT OF SOIL AND RING	**635.6**	g
P	DRY WEIGHT OF SOIL AND RING	**555.2**	g
Q	WEIGHT OF WATER (O - P)	80.4	g
R	DRY WEIGHT OF SOIL (P - C)	354.4	g
S	MOISTURE CONTENT ((Q/R)* 100)	22.7%	%

EXPANSION INDEX CORRECTION

T	CALCULATE (50 - J)	
U	CALCULATE ((65 + N) / (220 - J))	
V	CALCULATE (T * U)	
	CORRECTED EXPANSION INDEX (N - V)	

CORROSIVITY TEST RESULTS
(ASTM D516, CTM 643)

SAMPLE	pH	RESISTIVITY (OHM-CM)	SULFATE CONTENT (%)	CHLORIDE CONTENT (%)
B-1 @ 1 - 5'	*8.87*	*830*	*0.05*	*0.01*

CORROSIVITY PARAMETERS

SULFATE CONTENT (%)	SULFATE EXPOSURE	CEMENT TYPE
0.00 to 0.10	Negligible	--
0.10 to 0.20	Moderate	II, IP(MS), IS(MS)
0.20 to 2.00	Severe	V
Above 2.00	Very Severe	V plus pozzolan

SOIL RESISTIVITY (OHM-CM)	GENERAL DEGREE OF CORROSIVITY TO FERROUS METALS
0 to 1,000	Very Corrosive
1,000 to 2,000	Corrosive
2,000 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
Above 10,000	Slightly Corrosive

CHLORIDE (Cl) CONTENT (%)	GENERAL DEGREE OF CORROSIVITY TO METALS
0.00 to 0.03	Negligible
0.03 to 0.15	Corrosive
Above 0.15	Severely Corrosive

SAMPLE NO.: B-2 **SAMPLE DATE:** 2/17/22

SAMPLE LOCATION: 1' - 5' **TEST DATE:** 3/10/22

SAMPLE DESCRIPTION: Yellowish brown clayey sand (SC)

LABORATORY TEST DATA

	TEST SPECIMEN	**1**	**2**	**3**	**4**	**5**	
A	COMPACTOR PRESSURE	120	160	220			[PSI]
B	INITIAL MOISTURE	4.0	4.0	4.0			[%]
C	BATCH SOIL WEIGHT	1200	1200	1200			[G]
D	WATER ADDED	130	115	105			[ML]
E	WATER ADDED (D*(100+B)/C)	11.3	10.0	9.1			[%]
F	COMPACTION MOISTURE (B+E)	15.3	14.0	13.1			[%]
G	MOLD WEIGHT	2088.5	2103.2	2017.6			[G]
H	TOTAL BRIQUETTE WEIGHT	3187.0	3191.9	3133.6			[G]
I	NET BRIQUETTE WEIGHT (H-G)	1098.5	1088.7	1116.0			[G]
J	BRIQUETTE HEIGHT	2.50	2.45	2.47			[IN]
K	DRY DENSITY (30.3*I/((100+F)*J))	115.5	118.1	121.0			[PCF]
L	EXUDATION LOAD	2812	4802	6065			[LB]
M	EXUDATION PRESSURE (L/12.54)	224	383	484			[PSI]
N	STABILOMETER AT 1000 LBS	44	39	31			[PSI]
O	STABILOMETER AT 2000 LBS	117	104	84			[PSI]
P	DISPLACEMENT FOR 100 PSI	4.55	3.99	3.86			[Turns]
Q	R VALUE BY STABILOMETER	17	25	37			
R	CORRECTED R-VALUE (See Fig. 14)	17	25	37			
S	EXPANSION DIAL READING	0.0008	0.0030	0.0041			[IN]
T	EXPANSION PRESSURE (S*43,300)	35	130	178			[PSF]
U	COVER BY STABILOMETER	0.76	0.68	0.57			[FT]
V	COVER BY EXPANSION	0.27	1.00	1.37			[FT]

TRAFFIC INDEX:	4.5
GRAVEL FACTOR:	1.58
UNIT WEIGHT OF COVER [PCF]:	130
R-VALUE BY EXUDATION:	20
R-VALUE BY EXPANSION:	22
R-VALUE AT EQUILIBRIUM:	20

*Note: Gravel factor estimated from pavement section using CTM 301, Section C, Part b.

REV. 2, DATED 1/31/15

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SAN DIEGO, CALIFORNIA 92126

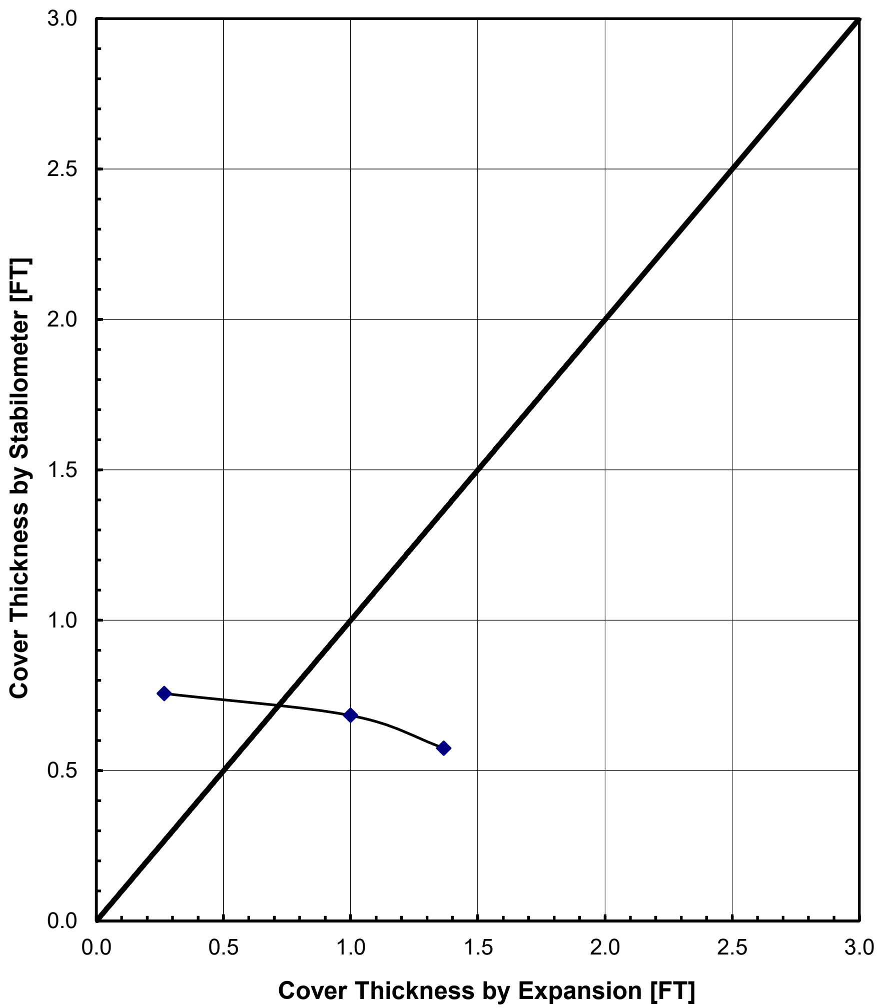
R-VALUE TEST RESULTS
CT301

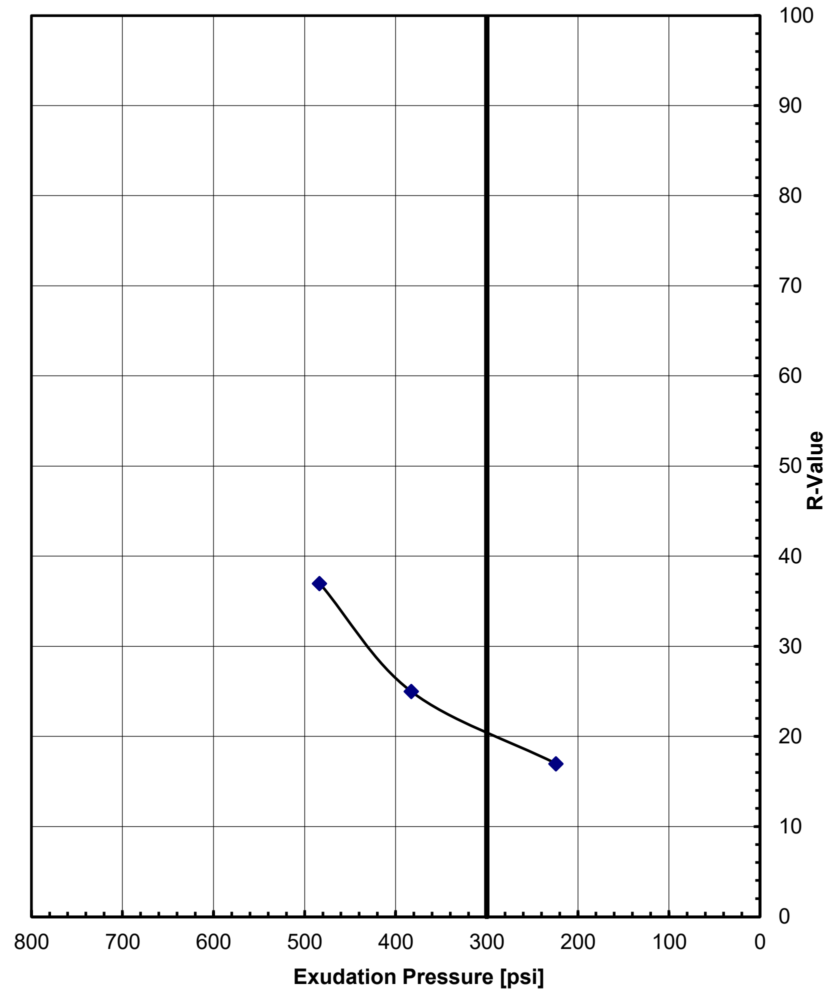
Project No. IR786

FIGURE 1.1

Sample: B-2, 1' - 5'

R-Value at Equilibrium: 20





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SAN DIEGO, CALIFORNIA 92126

COVER AND EXUDATION CHARTS

Project No. IR786

FIGURE 1.2

APPENDIX C - CALCULATIONS

Liquefaction Triggering Assessment and Settlement Calculation
Standard Penetration Tests

NCEER (2001)

Borehole No	B-1	
Ground Elevation (NAVD 88)	345.00	ft
Water Depth (Exploration)	16.10	ft
Water Depth (Design)	14.50	ft
Station		ft

A_{max}	0.42	g
M_w	6.40	
MSF	1.50	Triggering
MSF_{Vol}	0.78	Settlement

Energy Ratio	85	%
Settlement FS <=	1.0	
Finished Grade El.	345.00	ft
	User Input	

Borehole diameter (mm)	Correction C_B
115	1
150	1.05
200	1.15

El. Top of Bedrock (ft): 317

Settlement Calculation per Tokimatsu & Seed (1987)

From Tokimastu & Seed (1987)

Mw	Neq	Vol. Strain Ratio
8.50	26	1.25
7.50	15	1.00
6.75	10	0.85
6.00	5	0.60
5.25	2 - 3	0.40

Elevation	SPT Depth	SPT Corrected Depth	Thickness	Design Depth	Design Depth	Soil Parameters γ	Soil Parameters Soil Type	Soil Parameters FC	Soil Stress σ_{v0}	Soil Stress u	Soil Stress σ_{v0}'	Demand $\sigma_{v0}'_{design}$	Demand r_d	Bore Hole Diameter	Bore Hole Diameter	Bore Hole Sampler
ft	ft	ft	ft	ft	m	pcf		%	psf	psf	psf	psf		in	mm	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
340	5	5	8.5	5	1.5	115	SC	43	575	0	575	575	0.99	8.0	203	MC
333	10	12	6.5	12	3.7	120	SM	43	1,415	0	1,415	1,415	0.97	8.0	203	SPT
327	15	18	5.5	18	5.5	120	SW	5	2,135	119	2,016	1,917	0.96	8.0	203	MC
322	20	23	4.5	23	7.0	120	SW	3	2,735	431	2,304	2,205	0.95	8.0	203	SPT
318	25	27	3.0	27	8.2	120	SP-SM	5	3,215	680	2,535	2,435	0.93	8.0	203	MC
316	30	29	4.0	29	8.8	150	BR	10	3,515	805	2,710	2,610	0.93	8.0	203	SPT
310	35	35	5.5	35	10.7	150	BR	10	4,415	1,179	3,236	3,136	0.89	8.0	203	SPT
305	40	40	5.0	40	12.2	150	BR	10	5,165	1,491	3,674	3,574	0.85	8.0	203	SPT
300	45	45	5.0	45	13.7	150	BR	10	5,915	1,803	4,112	4,012	0.80	8.0	203	SPT
295	50	50	2.5	50	15.2	150	BR	10	6,665	2,115	4,550	4,450	0.75	8.0	203	SPT

Elevation	Blow Counts (N) Uncorrected	Blow Counts (N) Sampler Corrected	Blow Count Correction Factors C_E	C_B	C_R	C_S	Cyclic Resistance N_{60}	C_N	$(N_1)_{60}$	α	β	$(N_1)_{60,CS}$	$CRR_{7.5}$	K_σ	CRR	Demand CSR	Results FS
ft																	Above Gasp
1	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34
340	11	7	1.42	1.15	0.75	1.00	9	1.7	15	5.00	1.20	23	0.26	1.00	0.40	0.27	N.A.
333	10	10	1.42	1.15	0.80	1.00	13	1.2	16	5.00	1.20	24	0.28	1.00	0.41	0.27	N.A.
327	13	9	1.42	1.15	0.85	1.00	12	1.02	12	0.00	1.00	12	0.13	1.00	0.20	0.29	0.69
322	6	6	1.42	1.15	0.95	1.00	9	0.96	9	0.00	1.00	9	0.10	0.98	0.15	0.32	0.47
318	15	10	1.42	1.15	0.95	1.00	16	0.91	14	0.00	1.00	14	0.15	0.96	0.22	0.34	0.65
316	100	100	1.42	1.15	0.95	1.00	100	0.9	100	0.87	1.02	100	too dense	0.94	too dense	0.34	2.00
310	100	100	1.42	1.15	1.00	1.00	100	0.8	100	0.87	1.02	100	too dense	0.90	too dense	0.34	2.00
305	100	100	1.42	1.15	1.00	1.00	100	0.8	100	0.87	1.02	100	too dense	0.87	too dense	0.34	2.00
300	100	100	1.42	1.15	1.00	1.00	100	0.7	100	0.87	1.02	100	too dense	0.85	too dense	0.32	2.00
295	100	100	1.42	1.15	1.00	1.00	100	0.7	100	0.87	1.02	100	too dense	0.83	too dense	0.31	2.00

Elevation	Dry Sand ϕ'	σ_m'	G_{max}	γ_{eff} x	γ_{eff}			γ_{vol}	Saturated Sand $CSR_{M7.5}$	γ_{vol}	Settlement γ_{vol}	Layer	Cumulative	$F_{s,not adjusted}$ (No Minimum Elevation)
ft	degree	psf	ksf	(G_{eff}/G_{max})	%			%		%	%	in	in	
1	35	36	37	38	39	40	41	42	43	44	45	46	47	48
340	32	372	957	1.6E-04	0.05	-3.78	-3.07	0.04	0.18	-	0.04	0.04	3.74	N.A.
333	32	915	1,522	2.5E-04	0.07	-3.36	-2.94	0.05	0.18	-	0.05	0.04	3.70	N.A.
327	32	1,240	1,628	3.4E-04	-	-	-	-	0.19	2.19	2.19	1.44	3.66	0.69
322	32	1,426	1,565	4.5E-04	-	-	-	-	0.21	2.78	2.78	1.50	2.22	0.47
318	32	1,575	1,922	4.3E-04	-	-	-	-	0.22	2.00	2.00	0.72	0.72	0.65
316	32	1,688	3,808	2.3E-04	-	-	-	-	0.23	-	-	0.00	0.00	2.00
310	32	2,028	4,174	2.6E-04	-	-	-	-	0.23	-	-	0.00	0.00	2.00
305	32	2,311	4,456	2.7E-04	-	-	-	-	0.22	-	-	0.00	0.00	2.00
300	32	2,595	4,721	2.7E-04	-	-	-	-	0.22	-	-	0.00	0.00	2.00
295	32	2,878	4,972	2.8E-04	-	-	-	-	0.21	-	-	0.00	0.00	2.00

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by Jonathan D. Bray and Thaleia Travasarou
Journal of Geotechnical and Geonvironmental Engineering, ASCE, V. 133(4), pp. 381-392, April 2007

SEE NOTES BELOW FOR GUIDANCE IN THE USE OF SPREADSHEET

Input Parameters			
Yield Coefficient (ky)	0.14		Based on pseudostatic analysis
Initial Fundamental Period (Ts)	0.16	seconds	1D: Ts=4H/Vs 2D: Ts=2.6H/Vs
Degraded Period (1.5Ts)	0.24	seconds	
Moment Magnitude (M_w)	6.4		
Spectral Acceleration (Sa(1.5Ts))	0.918	g	

Additional Input Parameters		
Probability of Exceedance #1 (P1)	84	%
Probability of Exceedance #2 (P2)	50	%
Probability of Exceedance #3 (P3)	16	%
Displacement Threshold (d_threshold)	30	cm

Intermediate Calculated Parameters			
Non-Zero Seismic Displacement Est (D)	21.83	cm	eq. (5) or (6)
Standard Deviation of Non-Zero Seismic D	0.66		

Results			
Probability of Negligible Displ. (P(D=0))	0.00		eq. (3)
D1	11.3	cm	calc. using eq. (7)
D2	21.8	cm	calc. using eq. (7)
D3	42.1	cm	calc. using eq. (7)
P(D>d_threshold)	0.32		eq. (7)

Notes

1. Values highlighted in blue are input parameters, and results are presented in the table with the yellow heading.
2. Probability of Exceedance is the desired probability of exceeding a particular displacement value.
3. Displacements D1, D2, and D3 correspond to P1, P2, and P3, respectively.
   (e.g., the probability of exceeding displacement D1 is P1)
4. The 16%, 50%, and 84% percentile displacement values at selected ky values are shown to the right.
5. Calculated seismic displacements are due to deviatoric deformation only (add in volumetrically induced movement).
6. ky may range between 0.01 and 0.5, Ts between 0 and 2 s, Sa between 0.002 and 2.7 g, M between 4.5 and 9
7. Rigid slope is assumed for Ts < 0.05 s
8. When a value for D is not calculated, D is < 1cm
9. ky may be estimated using the simplified equations shown below.
10. Examples of how Ts is estimated are shown below.
11. Vs = weighted avg. shear wave velocity for the sliding mass, e.g., for 2 layers, Vs = [(h1)(Vs1) + (h2)(Vs2)]/(h1 + h2)

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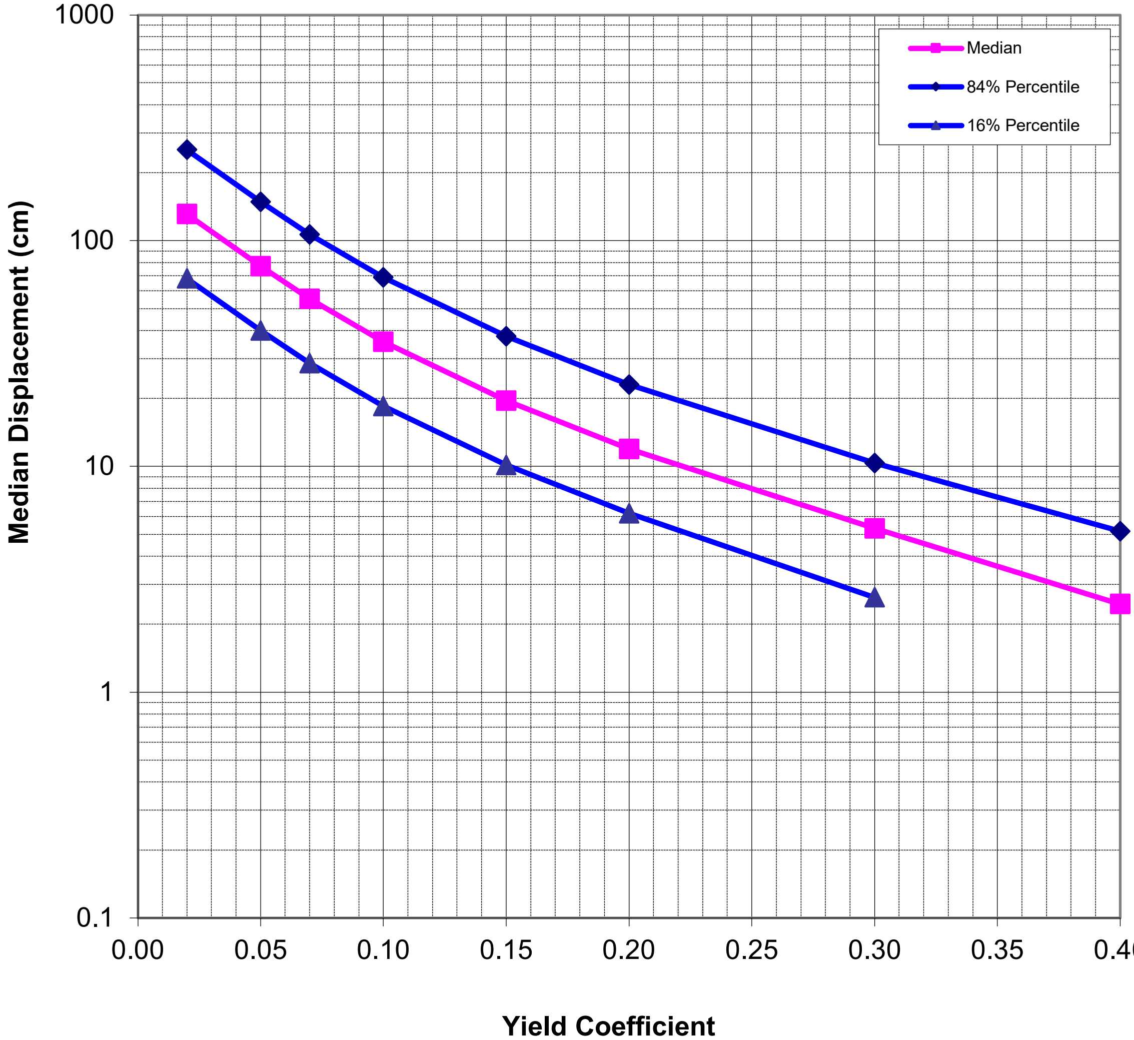
Dependence on ky

ky	P(D="0")	D (cm)	Dmedian (cm)	D-84% (cm)	D-16% (cm)
0.020	0.00	131.1	131.1	252.6	68.0
0.05	0.00	77.2	77.2	148.7	40.0
0.07	0.00	55.2	55.2	106.4	28.6
0.1	0.00	35.6	35.6	68.7	18.5
0.15	0.00	19.6	19.6	37.7	10.1
0.2	0.00	12.0	12.0	23.0	6.2
0.3	0.03	5.4	5.3	10.4	2.6
0.4	0.17	2.9	2.5	5.2	<1



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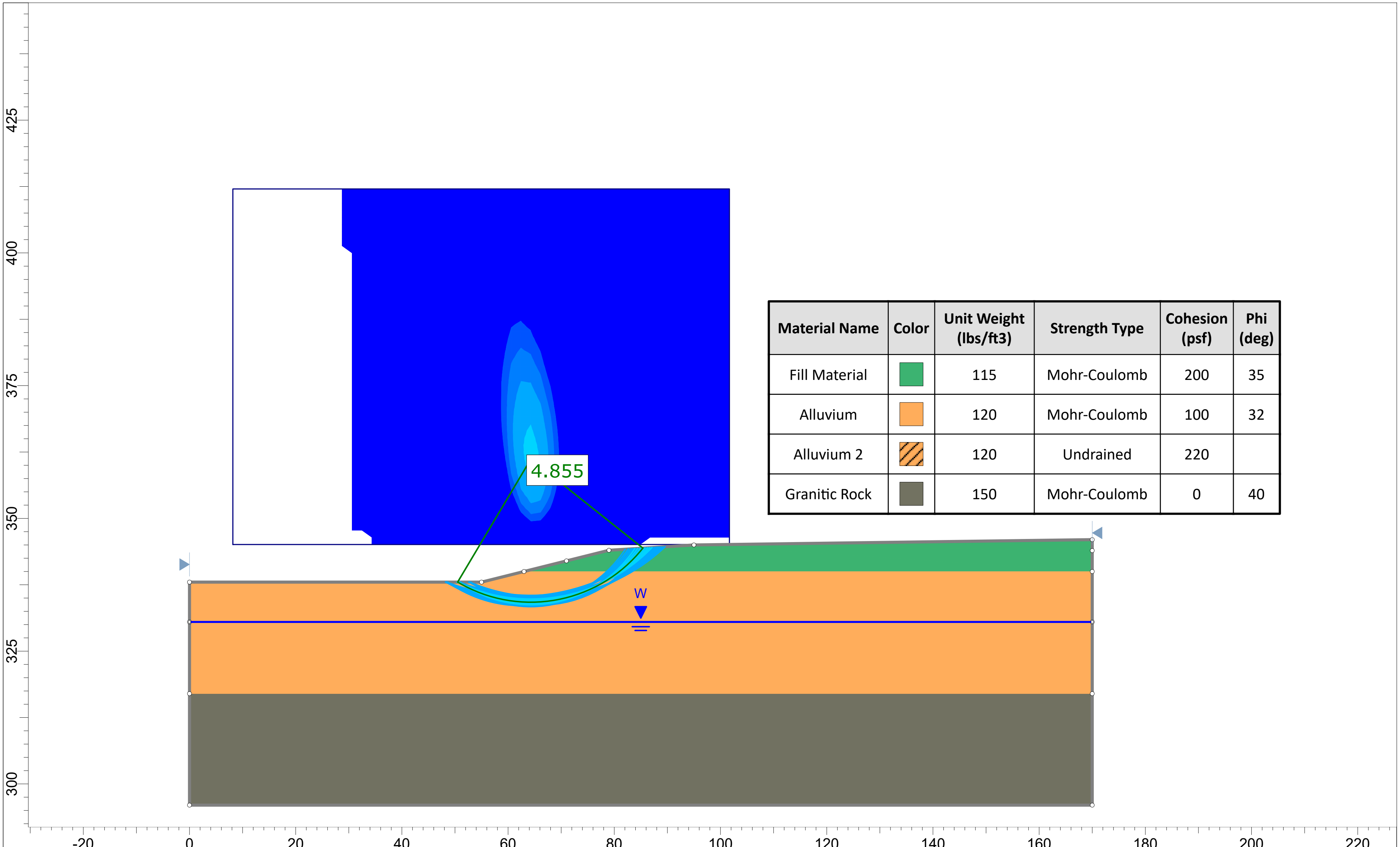

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill Material		115	Mohr-Coulomb	200	35
Alluvium		120	Mohr-Coulomb	100	32
Alluvium 2		120	Undrained	220	
Granitic Rock		150	Mohr-Coulomb	0	40

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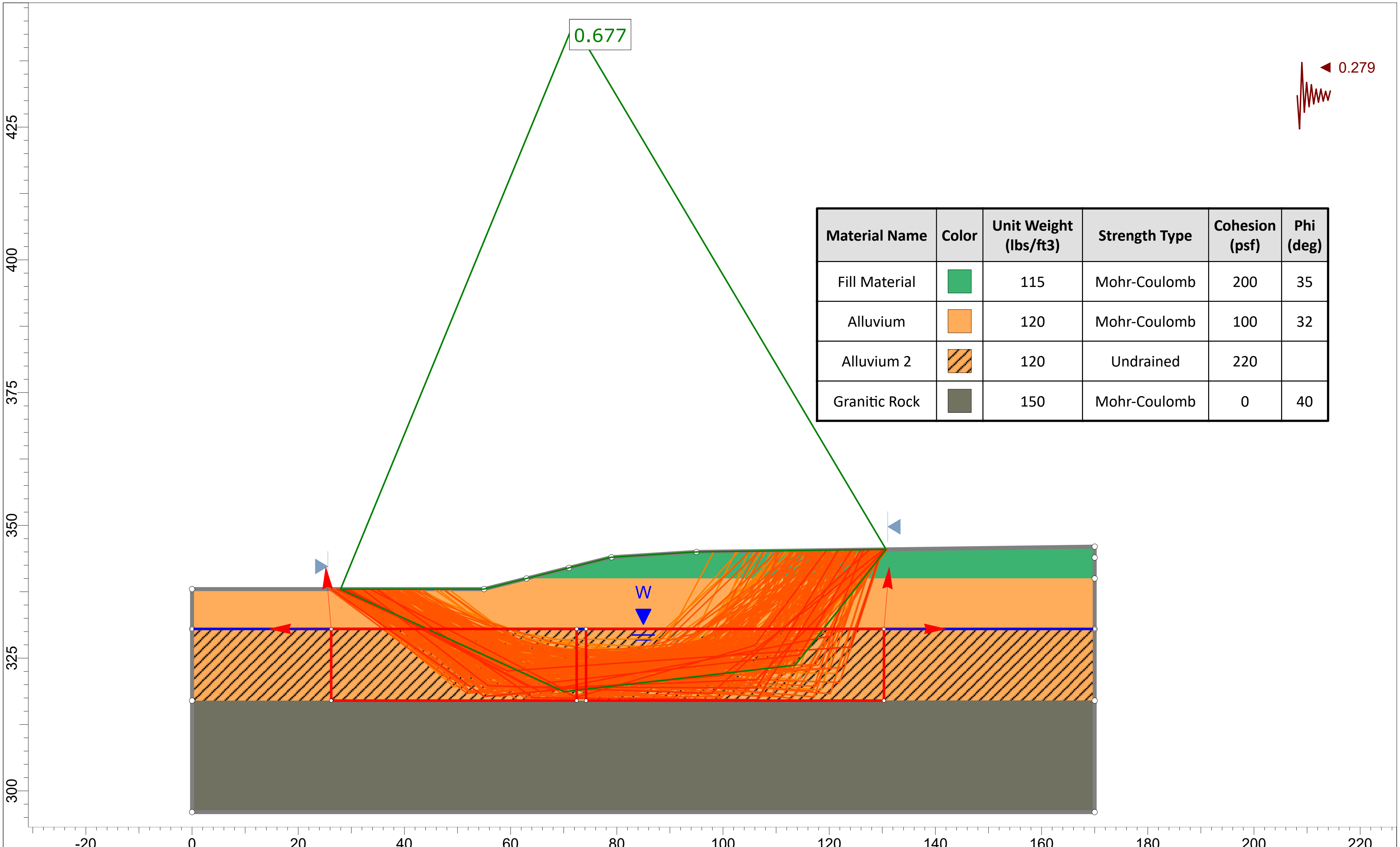
SLIDEINTERPRET 8.010

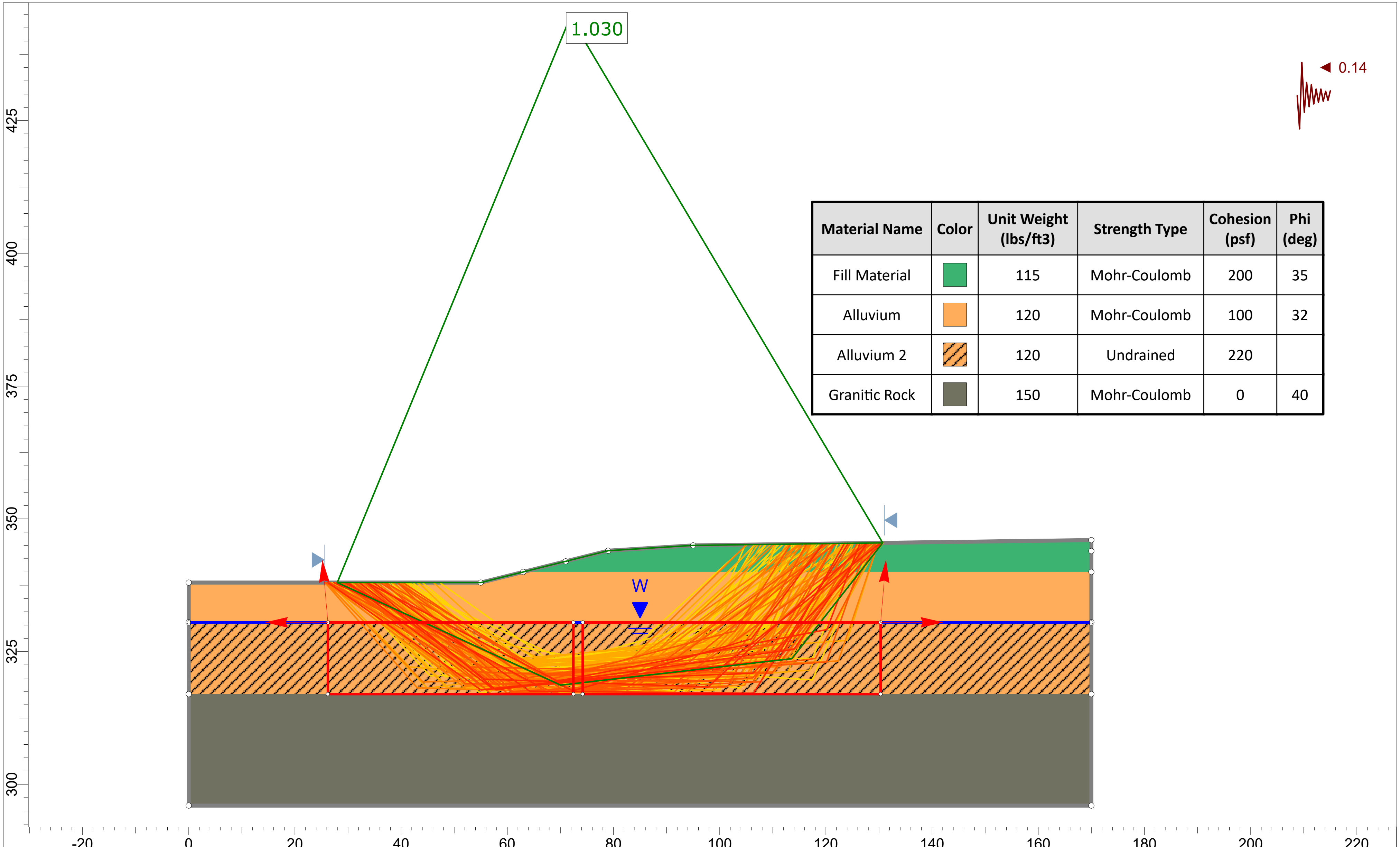
Project	IR786 Santee Community Center
Analysis Description	Static
Drawn By	GV
Scale	1:300
Company	Group Delta
Date	3/18/22
File Name	Existing.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill Material		115	Mohr-Coulomb	200	35
Alluvium		120	Mohr-Coulomb	100	32
Alluvium 2		120	Undrained	220	
Granitic Rock		150	Mohr-Coulomb	0	40

SLIDEINTERPRET 8.010

Project	IR786 Santee Community Center		
Analysis Description	Seismic		
Drawn By	GV	*Scale*	1:300
Company	Group Delta		
Date	3/18/22	*File Name*	Existing.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill Material		115	Mohr-Coulomb	200	35
Alluvium		120	Mohr-Coulomb	100	32
Alluvium 2		120	Undrained	220	
Granitic Rock		150	Mohr-Coulomb	0	40

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SLIDEINTERPRET 8.010

Project	IR786 Santee Community Center		
Analysis Description	Ky Run		
Drawn By	GV	*Scale* 1:300	*Company* Group Delta
Date	3/18/22	*File Name*	Existing.slmd