Appendix H-3

Substation Geotechnical Report

Prepared for

Southern California Gas Company
555 West 5th Street
Los Angeles, California 90051

GEOTECHNICAL INVESTIGATION REPORT

SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

Prepared by

Geosyntec consultants

engineers | scientists | innovators

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Project Number: SC1339

January 22, 2024
(Revised April 5, 2024)

January 22, 2024
(Revised April 5, 2024)

Jessica Yuan, P.E.
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Southern California Gas Company | Gas Engineering
555 W 5th St.
Los Angeles, California 90013

Subject: Geotechnical Engineering Report
Honor Rancho Substation Option A
Santa Clarita, California

Dear Ms. Yuan:

Geosyntec Consultants (Geosyntec) is pleased to provide Southern California Gas Company (SCG) with the accompanying revised report presenting the results of our geotechnical investigation and recommendations for the proposed Substation Option A at SCG's Honor Rancho Facility in Santa Clarita, California. This revised report supersedes the previous geotechnical report dated January 22, 2024.

Our services were performed in general agreement with the Standard Services Agreement with SCG (Agreement No. CW10080), dated January 18, 2023.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us. We appreciate your business and look forward to our next project with you.

Sincerely,

Rehan Khan

Rehan Khan, P.E.(TX)
Project Engineer

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1. INTRODUCTION AND BACKGROUND INFORMATION

Southern California Gas Company's (SCG) Honor Rancho facility (Facility) is located in Santa Clarita, California, and situated to the north of Newhall Ranch Road (Figure 1). The Facility currently includes an existing gas compressor facility and other related gas storage facilities.

In 2021, Geosyntec had prepared a geotechnical engineering report (Geosyntec, 2021a) for a Compressor Modernization project at this site. That project included a substation in addition to other structures. In October 2022, SCG informed Geosyntec that it is considering a different location (referred to herein as Option A) for the proposed substation. This report presents the additional field investigations, testing, and engineering efforts that were performed for the Option A location.

The Option A location is about 1,600 ft to the east of the original substation location and located over a hillside consisting of bedrock outcrop. The construction of the project will mainly consist of cutting the hillside to create a pad for the substation. The substation structures are anticipated to be supported on shallow and deep foundations. Geosyntec performed additional field investigation for the Option A location and also utilized findings from Geosyntec (2021a) to develop recommendations for Option A locations as presented in this report.

1.1 Purpose and Scope of Investigation

The purpose of our services was to investigate subsurface conditions and provide geotechnical engineering recommendations for the design and construction of the project. The scope of the investigation is outlined in our contract agreement (Agreement No. CW10080) dated January 18, 2023, and includes field exploration, laboratory testing, engineering evaluation and analyses, and preparation of this geotechnical engineering report.

1.2 Geosyntec (2021a) Scope and Findings

As part of Geosyntec (2021a), we had performed the following field investigations:

- Six Cone Penetration Tests (CPTs) to depths that ranged from 11 to 39 feet below ground surface (ft bgs);
- Five hollow stem auger (HSA) borings that were advanced to depths of 6 to 31 ft bgs followed by infiltration testing inside two of the HSA borings;
- Four mud rotary (MR) borings advanced to depths of 16 to 56 ft bgs that were switched to rock coring upon encountering bedrock;
- OYO Suspension P-S logging inside one of the MR boring;
- P-wave seismic refraction survey; and

- Laboratory testing of soil and rock samples.

An executive summary of the findings from Geosyntec (2021a) that are relevant to our current study can be summarized as follows:

- Option A location is about 0.3 miles from the closest active fault (San Gabriel Fault).
- The subsurface soils at Option A location are anticipated to consist of Saugus formation. Saugus formation encountered generally consisted of interbedded, silty and clayey sandstones with gravels and cobbles, as well as sandy claystones. The Saugus formation indicated a general dip of 50 to 70 degrees to the southwest.
- P-wave seismic refraction survey performed at one location indicated that P-wave velocities in the Saugus formation within the upper 40 ft were in the range of 2,500 to 4,500 ft/sec. Based on this range, the Saugus formation is expected to be rippable within the upper 40 ft bgs.
- Groundwater was not observed during the field investigations that extended to 56 ft bgs. Therefore, shallow groundwater is not anticipated.
- Geologic/seismic hazards that may impact projects within the Facility include strong seismic shaking which can impact stability of structures and may cause landslides depending on the location of the proposed improvements.
- The samples of Saugus formation tested were not found to be corrosive.

2. GEOTECHNICAL FIELD EXPLORATION AND LABORATORY TESTING PROGRAM

2.1 Geotechnical Field Exploration

Geotechnical field exploration tasks performed for this study included the following:

- Geologic site reconnaissance;
- Hollow-stem auger drilling;
- Geotechnical laboratory testing;
- Analytical testing of soil cuttings; and
- Transportation and disposal of soil cuttings at an appropriate facility.

2.1.1 Pre-Field Activities

Geosyntec staff visited the site prior to fieldwork to perform geologic mapping of the vicinity of the proposed project location and to mark locations of exploratory borings.

Geosyntec then contacted Underground Service Alert (USA DigAlert) to coordinate clearance of the exploration locations with respect to below ground utilities. A site-specific health and safety Task Hazard Analyses (THA) was prepared in accordance with Geosyntec health and safety requirements. Prior to our fieldwork, Geosyntec and our drilling subcontractor attended site-specific environmental and safety training provided by SCG representatives.

Geosyntec also obtained a well/exploration hole permit from the County of Los Angeles, Environmental Health Division, Drinking Water Program (Permit No. SR0328919).

2.1.2 Exploratory Borings

Three HSA borings were advanced to depths ranging between 31.5 ft and 41.5 ft bgs on June 9, 2023. These borings were designated as HSA-3, HSA-4, and HSA-5. HSA-1 and HSA-2 designations were used for the two borings drilled as part of Geosyntec (2021a). The borings were advanced using a CME 75 truck-mounted drill rig equipped with 7-inch diameter hollow-stem augers. The approximate boring locations are shown on Figure 2.

At each boring location, the upper approximately 5 ft was hand-augered and the cuttings from hand-auguring were collected as bulk samples. Starting at approximately 5 ft bgs, soil samples were collected using a Standard Penetration Test (SPT) drive sampler or a 3-inch diameter split-spoon Modified California (Mod Cal) sampler driven with an automatic hammer (140-pound hammer falling approximately 30 inches). The hammer energy measurement for this hammer was performed by the drilling subcontractor on January 12, 2023, and the hammer energy transfer ratio was calculated to be 80.1 percent. The hammer calibration test program describing the instrumentation and procedure used for the energy measurement and the results are included in

Appendix A. The samples were collected at 2.5-foot intervals in the upper 15 ft, and at 5-foot intervals thereafter. Select samples from the borings were transported to the geotechnical laboratory for testing.

Descriptions and visual classification of the subsurface materials were logged in the field by a Geosyntec Engineer and reviewed by a California registered Geotechnical Engineer. Subsurface descriptions were based on the recovered soil samples and soil cuttings. The subsurface descriptions were developed in general accordance with ASTM International Test Procedure (ASTM) standard D2488. A summary of the exploratory borings is presented in Table 1, and the individual boring logs are presented in Appendix B. Sampling information and other pertinent field data and observations are included on the boring logs.

Exploration-derived soil cuttings were drummed in 55-gallon steel drums and stored on site pending analysis and disposal. One composite sample was collected for analytical testing and waste profiling. Based on the results of the analytical testing, the waste was classified as non-hazardous.

Once drilling and sampling was complete, the boreholes were tremie backfilled with cement-bentonite grout in general accordance with the County of Los Angeles drilling permit requirements.

2.2 Geotechnical Laboratory Testing

Selected soil samples from the borings were tested to evaluate the physical and engineering properties of the subsurface materials. The laboratory tests were performed in general accordance with the testing procedures of ASTM or other generally accepted test methods. A tabulated summary of the geotechnical laboratory test results is presented in Table 2 and Table 5, and test result data sheets are included in Appendix C.

3. SITE AND GEOLOGIC CONDITIONS

Our understanding of the site conditions has been developed based on a geologic site reconnaissance, the results of our field exploration and laboratory testing program and review of published geologic literature for the site.

3.1 Regional Geology

The Facility lies along the northeastern margin of the Ventura Basin within the Transverse Ranges geomorphic province, which extends approximately 320-miles (west to east) from Point Arguello and San Miguel Island to the Eagle and Pinto Mountains of the Mojave Desert. The Transverse Ranges is characterized by a series of east-west trending convergent deformational structural features in contrast to the predominant northwest-southeast structural trend of southern California. The trend of the Transverse Ranges is controlled by the effects of north-south compressive deformation attributed to the San Andreas fault system that has rotated and compressed the region to its current configuration. The compression has resulted in folding and reverse/thrust faulting with similar east to west trends, and regional uplift.

The Ventura Basin consists of an elongated sedimentary trough which extends from the Santa Barbara Channel on the west to the San Gabriel fault zone on the east. The axis of the Ventura Basin trends east-west reflecting the regional orientation of the Transverse Ranges, and generally coincides with the Santa Clara River Valley. This sedimentary basin contains a thick sequence of marine and non-marine sediments that have been uplifted and deformed by past tectonic forces.

The northeastern margin of the Ventura Basin is characterized by rugged, steep, hilly terrain and is dissected by numerous drainages that generally empty towards the Santa Clara River Valley to the south and the Castaic Valley to the west. Based on published surficial geologic maps (Dibblee and Ehrenspeck, 1996), the Facility is underlain by artificial fill, Quaternary-age alluvial and colluvial deposits, and the Pleistocene-age Saugus Formation. The regional surficial geology map is shown on Figure 3.

3.2 Seismic Setting

Faults in California are generally classified as “Holocene-active”, “Pre-Holocene”, and “Age-undetermined” faults. Division of these major groups are based on criteria by the California Geologic Survey (CGS, formerly known as California Division of Mines and Geology, CDMG) for the Alquist-Priolo Earthquake Fault Zoning Program (CGS, revised 2018). By definition, a “Holocene-active” fault is one that has had displacement within Holocene time (last 11,700 years). A “Pre-Holocene” fault has demonstrated displacement prior to the last 11,700 years. “Age-undetermined” faults have either not been studied or the study results were inconclusive for displacement ages.

The San Gabriel fault zone (SGFZ) is the closest major Holocene-active fault to the project area, with several mapped fault traces extending into or projecting towards the eastern portion of the Facility. The SGFZ comprises a complex group of predominantly northwest-southeast trending, right lateral strike-slip faults approximately 45-miles (72-km) long, which extend from near Frazier Mountain to the San Gabriel Mountains (Wills, Weldon, and Bryant, 2008). Recent studies indicate an estimated slip rate of 1.0 millimeters per year (mm/yr) along the fault (Wills, Weldon, and Bryant, 2008). According to the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) (Field et al., 2015) the SGFZ has a mean 30-year probability of an earthquake equal to or greater than 6.7 moment magnitude (**M**) (30-year **M**≥6.7 probability) of 0.36 percent (%), a mean 30-year **M**≥7.0 probability of 0.29%, a mean 30-year **M**≥7.5 probability of 0.23%, and a mean 30-year **M**≥8.0 probability of 0.01%.

The proposed Project site is situated approximately 0.6-miles to the southwest of the mapped Holocene-active trace of the SGFZ and approximately 1,000 ft west of Pre-Holocene splays of the fault according to the United States Geological Survey Quaternary Fault and Fold Database (USGS, 2006). Other Holocene-active faults in the vicinity of the project area include the Northridge blind thrust fault to the south, the San Cayetano fault to the southwest, and the San Fernando section of the Sierra Madre fault to the southeast (USGS, 2006). UCERF3 average 30-year participation probabilities for these faults are listed in Table 3.

The closest Pre-Holocene fault to the project includes mapped traces of the Holser Fault, which generally trends along the northern Santa Clara River Valley and extends through the southern portion of the Facility. The proposed Project site is situated approximately 1,500 ft to the north of the closest mapped Pre-Holocene trace of the Holser Fault (USGS, 2006).

Other Pre-Holocene faults closest to the project area include the Santa Felicia fault to the northwest and the Del Valle fault to the southwest. Regional Holocene-active faults in the vicinity of the project area include the San Andres fault zone to the northeast, and the Simi-Santa Rosa fault zone to the southwest (USGS, 2006). These faults and their respective distances from the Project site and UCERF3 participation probabilities are presented in Table 3, where available. The locations of regional faults and historical earthquake epicenters are shown on Figure 6.

These faults, their respective distances from the facility and design moment magnitudes are presented in Table 3. The locations of regional faults and historic earthquake epicenters are shown on Figure 4.

3.3 <u>Surface Conditions</u>

Generally, the Honor Rancho Facility lies within an area characterized by low hills with drainages that generally trend north-south ending at the Santa Clara River south of the site. Based on a review of historical aerial imagery (historicaerials.com, 2021) and topographic maps, the site was graded in the early 1950’s to support the oil and gas facility improvements. Roads were created around the site and various pads were constructed in the early 1970's to achieve the general layout that exists today.

The substation Option A location is located over the side slopes of a hill. Within the proposed substation footprint area, the maximum existing ground surface elevation difference (based on Google Earth) is about 50 ft in the east-west direction and about 25 ft in the north-south direction. The hill is mostly vegetated. An asphalt paved road was observed to the east of the hill and a gravel-surfaced yard was observed to the north of the hill.

3.4 Subsurface Conditions

Geosyntec's current subsurface explorations indicate that fill soils, alluvium soils, and Saugus Formation are anticipated to be present within the proposed substation footprint and its vicinity. Generalized local geology and locations of geologic cross section are presented in Figure 2. Geologic cross section is presented in Figure 5.

3.4.1 Fill

Fill was encountered to the north of the hill within the gravel-surfaced area. The thickness of the fill observed in the borings is approximately 11 to 12 ft. Fill soils observed primarily consist of loose to medium dense, slightly moist, silty and clayey fine-grained sands with gravels that were angular to sub-rounded. The fill soils exhibited lumps of clay of dissimilar color within the sand matrix.

The fill that was encountered during the investigation is considered undocumented in that the history of their placement is not known and compaction reports documenting that they were placed as engineered fill are not available.

3.4.2 Quaternary Young Alluvium (Q_{ya})

Based on geologic mapping (Dibblee, Jr. and Ehrenspeck, 1996) and observations during the explorations, Holocene-age alluvium underlies the valley areas of the Facility overlying the Saugus Formation. Alluvium was encountered beneath the fill soils to the north of the hills within the gravel-surfaced area down to approximate elevations of 1,101 ft MSL (Mean Seal Level) to 1,104 ft MSL. The alluvium observed generally consists of loose to medium dense, fine-grained sands and medium plasticity sandy clays.

3.4.3 Saugus Formation (Sandstone Unit) (Qss)

Based on the explorations as well as geologic mapping, Plio-Pleistocene age Saugus Formation underlies the proposed substation location. The Saugus Formation encountered during the explorations generally consists of interbedded, silty and clayey sandstones as well as low to medium plasticity claystone. The Saugus Formation was observed to be moderately to highly weathered and occasionally friable in the absence of fines. Based on geologic mapping the Saugus Formation indicated a general dip of 50 to 70 degrees to the southwest, correlating with the southern leg of an anticline with the axis located approximately parallel to the San Gabriel Fault, 1.5 miles north of the proposed location.

3.4.4 Groundwater

Groundwater was not encountered during the current or the previous explorations (Geosyntec, 2021a) at the site. Site specific data regarding recent groundwater levels at the site was not available.

Figure 6 is an excerpt of the historically highest depth to groundwater contour map from the CGS (1997) Seismic Hazard Zone Report for the Newhall 7.5 Minute Quadrangle. Information from this figure indicates that shallow groundwater levels were observed within Quaternary alluvium near the Santa Clara River approximately 1000 ft south of the Site at approximate elevation of 1,050 ft MSL. However, an interpretation specific to the vicinity of the site is not available.

Since the proposed substation footprint is directly on top of Saugus formation rock, groundwater is not anticipated to impact the site or pose associated geohazards if proper site drainage is designed, installed, and maintained per the recommendations of the project civil engineer.

4. GEOLOGIC HAZARDS

4.1 Surface Fault Rupture

Seismically induced surface fault rupture occurs as the result of differential movement across a fault. The potential for surface fault rupture is generally considered to be significant along "Holocene-active" faults and to a lesser degree along "pre-Holocene" faults (CGS, 1998b). A review of published geologic maps did not identify the presence of faults crossing or projecting towards the proposed Site. Therefore, the potential for surface fault rupture at the project site is considered to be low. Furthermore, the project site is not located within a delineated earthquake fault rupture hazard zone as defined by the California Geological Survey (CGS) (Bryant and Hart, 2007).

4.2 Strong Ground Shaking and Design Ground Motions

The Facility is situated within a seismically active region and will likely experience moderate to severe ground shaking in response to a large magnitude earthquake occurring on a local or more distant active fault during the expected lifespan of the proposed substation. As a result, seismically induced ground shaking in response to an earthquake occurring on a nearby active fault, such as the San Gabriel Fault, or a regional fault, such as the San Andreas fault zone, is considered to be a major geologic hazard affecting the project.

Seismic design parameters were developed in accordance with California Building Code (CBC, 2022) and ASCE 7-16. The risk category of the proposed facilities was assumed as IV per Table 1604.5 of the CBC (2022). Site Class was assessed using site specific shear wave velocity measurements performed as part of Geosyntec (2021a) using suspension logging and seismic CPTs. Shear wave velocity measured in the weathered bedrock was estimated to range in between 1,400 and 1,500 ft/sec which falls within the range of V_{S30} values for Site Class C (very dense soil and soft rock) according to Table 20.3-1 of ASCE 7-16 and the shear wave velocity measured in the intact bedrock was estimated about 2,600 ft/sec, which falls under Site Class B (rock). Therefore, we conservatively assumed site class C for this report.

The risk-targeted maximum considered earthquake (MCE_R) ground motion parameters S_S and S_1 were obtained for the Site using the ASCE 7 Hazard Tool (https://asce7hazardtool.online/). The output from the web tool is included in Appendix D. These mapped ground motion parameters were used to determine the MCE_R ground motion parameters adjusted for Site class effects, S_{MS} and S_{M1}, with appropriate site coefficients for Site Class C. The design ground motion parameters, S_{DS} and S_{D1}, were then determined as 2/3 of the site adjusted MCE_R ground motion parameters. The recommended seismic design parameters including the site adjusted Maximum Credible Earthquake Geometric Mean (MCE_G) peak ground acceleration (PGA_M) are summarized in Table 4. The design response spectrum developed for Geosyntec (2021a) and was utilized in our current evaluation.

4.3 Expansive Soils/Rocks

Based on the plasticity characteristics of the soils and rocks encountered, the site soils/rocks are generally considered to have negligible to low potential for expansion. Medium plasticity claystone, where observed, were deeper than 11 ft bgs. If during grading (cuts), the medium plasticity claystone is exposed, there may be moderate potential for expansion when the rock comes in contact with water followed by shrinkage and cracking upon drying. Geosyntec recommends 12 inches of overexcavation where claystone is encountered as discussed in Section 5.1.1.

4.4 Collapsible Soils/Rocks

No swell/collapse testing was performed as part of this study because the proposed substation is planned to be on top of Saugus formation which is not anticipated to be susceptible to collapse. However, collapse potential tests performed on fill and alluvium in other parts of this site, as part of Geosyntec (2021a) indicated significant collapse mechanism (collapse strain of up to 2 to 3 percent for loading conditions consistent with expected bearing pressures) of soils upon inundation.

4.5 Soil Liquefaction and Lateral Spreading

According to California Geological Survey's (CGS) Earthquake Zones of Required Investigation for Newhall Quadrangle (CGS, 1998a), the proposed Option A location of the substation does not fall within a liquefaction potential zone. The proposed substation location is underlain by Saugus formation bedrock which is not anticipated to be susceptible to liquefaction. In addition, absence of shallow groundwater at this site further makes liquefaction and consequent lateral spreading potential remote.

4.6 Seismic Settlements

Saugus formation bedrock is not anticipated to be subject to seismically induced settlements during ground shaking. However, if portion of the proposed substation foundation extends to the adjacent fill/native alluvium, then differential settlement is likely.

4.7 Flooding

The Federal Emergency Management Agency (FEMA) presents the flood hazard potential in the vicinity of the site as part of their Flood Insurance Rate Maps. FEMA Map No. 06037C0805F indicates that the Site is located in Zone D, which is defined as "area of undetermined flood hazard." (FEMA, 2008).

Seiches typically occur when enclosed bodies of water are seismically shaken to generate oscillations and waves resulting in overtopping. Damage resulting from oscillatory waves (seiches) at the nearby Castaic Lake and Castaic Lagoon is considered unlikely due to the high relief topography between the lake and the Site.

Based on our review of the FEMA mapping, the geologic and physiographic setting, distance to the ocean and other large water bodies, and the project elevations, the potential for flooding or inundation is considered low at the Site.

4.8 Landslide

According to CGS (1998a), the proposed Option A location of the substation falls within an earthquake-induced landslide potential zone. This means the site is located in an area where previous occurrence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements. The proposed substation location and the zones with landslide potential are shown on Figure 6. A slope stability analysis of proposed cut slopes was performed as part of study as discussed in Section 5.5

5. DESIGN RECOMMENDATIONS

The recommendations presented herein for the design of the proposed substation are based on desktop review of publicly available geotechnical information, results of our current and previous field investigations and laboratory testing, engineering and geologic evaluations, and professional judgment. In our opinion, Option A location is suitable for the construction of the proposed substation, provided the recommendations of this report are incorporated into design and construction.

5.1 Grading

The final grading plan at Option A location was not provided to us at the time of this report. For our evaluation, we assumed that the access to the substation will likely be from the gravel-surfaced vacant area to the north of the substation. Therefore, we assumed that the base of the substation will match the existing grades of the gravel-surfaced area (+1,120 ft MSL per Google Earth). Based on the elevation contours shown in Figure 3 – Exploration Plan, the maximum elevation of the hill within the footprint of the substation is approximately +1,170 ft MSL and the maximum elevation of the hill to the west of the Option A location is +1,200 ft MSL. Therefore, mass excavations to as much as 50 ft bgs in the Saugus formation may be required for the final grades. These excavations will require the construction of permanent cut slopes which may be as high as 80 ft as discussed further in Section 6.

Excavation should be performed in accordance with SCG requirements, the recommendations of this geotechnical report, applicable sections of the 2022 CBC, applicable Los Angeles County and City of Santa Clarita grading regulations, the current version of the Standard Specifications for Public Works Construction "Greenbook," as well as California Occupational Safety and Health Administration (Cal OSHA) safety requirements.

5.1.1 Site Preparation

No significant backfilling is anticipated to be required as part of grading. However, Geosyntec recommends overexcavating the subgrade under the shallow foundations and pavement for a minimum of 12 inches below the bottom of foundation or bottom of pavement section. Excavated area should be backfilled with imported engineered fill or processed onsite materials meeting requirements of Section 5.1.2. Prior to backfill, the bottom of the excavation in Saugus formation materials should be free of loose material, deleterious and organics matter and be observed by Geosyntec representative to confirm adequate preparation. Any identified unsuitable areas will need to be further overexcavated and backfilled. Backfill recommendations are provided in Sections 5.1.2 and 5.1.3.

5.1.2 Fill Materials

On-site or imported engineered fill soils should be free of perishable, organic, deleterious, or otherwise unsuitable materials, and have:

1. At least 40% material less than ¼-inch in size;
2. A maximum size in the largest dimension of 3 inches;
3. Less than 50% of fines content passing sieve No. 200;
4. A plasticity index of less than 15 and a liquid limit of less than 40; and
5. An expansion index (ASTM D4829) of less than 20.

Based on the field investigation, the onsite surficial fill, alluvial materials, and Saugus Formation sandstone generally meet this criteria.

Aggregate base, if used as fill, should meet the requirements specified for Class II aggregate base in Section 26 of the latest edition of the Caltrans Standard Specifications.

5.1.3 Fill Placement and Compaction

Fill soils should be moisture conditioned to a minimum of the optimum moisture content and compacted in layers that do not exceed 8-inch loose lifts for heavy equipment compaction and 4-inch loose lifts for hand-held equipment compaction. Each lift of fill should be compacted to a minimum of 95 percent relative compaction unless otherwise specified. Relative compaction is defined as the ratio (in percent) of the in-place dry density to the maximum dry density determined using the latest version of ASTM D1557 as the compaction standard. Modified Proctor Compaction Tests should be performed on the fill soils to determine the maximum dry density and optimum moisture content.

5.2 Surface Drainage

Surface drainage should be planned to prevent ponding and promote the drainage of surface water away from foundations, slabs, and edges of pavements, and towards suitable collection and discharge facilities. Paved areas should be sloped to drain water away from structures and flatwork at a minimum gradient of 1 percent, and unpaved areas should be finish graded with a minimum slope of 2 percent away from structures and pavements.

5.3 Foundations

It is our understanding that both shallow foundations and deep foundations are planned to support the proposed substation at Option A location. Foundation recommendations are provided in this section.

5.3.1 Shallow Foundations

5.3.1.1 Allowable Bearing Capacity

An allowable bearing capacity of 3,000 psf can be used for the shallow foundations with a minimum width of 2 ft and minimum embedment of 2 ft that bear on either Saugus formation material or on-site soils properly placed and compacted per recommendations provided in Section 5.1.2 and 5.1.3. For each additional foot of foundation width or foundation embedment, the allowable bearing capacity can be increased by 500 psf up to the maximum of 5,000 psf. The allowable bearing capacity can be increased by one-third for short term wind or earthquake loading conditions.

5.3.1.2 Settlement and Modulus of Subgrade Reaction

Shallow foundation total settlements under the allowable loads are expected to be less than 0.5 inch when bearing directly on Saugus Formation or on-site soils properly placed and compacted per recommendations provided in Section 5.1.2 and 5.1.3.

A unit modulus of subgrade reaction for a square foundation measuring one foot by one foot can be assumed as 150 pci. For larger loading areas, the modulus of subgrade reaction can be estimated by the following equation:

$$\mathrm{k} = k_1 \left(\frac{\mathrm{B}+1}{2\mathrm{B}} \right)^2 \left(\frac{1+0.5\frac{B}{L}}{1.5} \right)$$

where k_1 is coefficient of subgrade reaction in units of pounds per cubic inch (pci) of a square foundation measuring one foot by one foot, and B and L are width and length of the loaded portion of the mat in units of ft, respectively.

5.3.1.3 Resistance to Lateral Loading

Resistance to lateral loads may be provided by passive resistance along the outside face of the foundation and frictional resistance along the bottom. For allowable passive resistance, an equivalent fluid weight of 180 pcf can be used. If friction is used to resist lateral loads, an allowable friction coefficient of 0.35 between the subgrade and foundation concrete can be used.

5.3.2 Deep Foundations

Based on information provided by Southern California Edison (SCE), Geosyntec understands that a deep foundation system consisting of drilled piers, with diameters ranging from 2 to 4 ft, will be required for some dead-end structures. This section provides a discussion of axial load carrying capacity for the proposed shafts in both compression and tension as well as the response of shafts to lateral loading.

5.3.2.1 Shaft Axial Load Analysis

Resistance to axial loads is provided by a combination of skin friction (F) along the sides of the shaft and end bearing (Q) at the bottom of shaft. An ultimate unit skin friction and unit end bearing of 1.5 (kips per square foot (ksf) and 20 ksf, respectively should be used for the Saugus formation material. Based on these unit resistance values, we estimated the resistance to axial loading from end bearing and skin friction for various shaft sizes. The axial load capacities for various shaft sizes are presented in Table 6.

The ultimate capacities provided in Table 6 should be reduced by an appropriate factor of safety (FS) to obtain the allowable values. The available allowable compressive drilled shaft capacity is equal to (Q+F * L)/FS. The allowable capacity in tension is equal to (F * L)/FS. L is the length of shaft embedded in Saugus formation. FS of 2 and 3 are recommended for compressive and tensile capacity, respectively. For short-term wind or earthquake loading conditions, FS can be reduced to 1.5 and 2 for compressive and tensile capacity, respectively.

Deep foundations settlement under allowable loads are not expected to exceed ¼ inch assuming the design and construction recommendations provided in this report are implemented.

Pile spacing should be kept at a minimum of 3 shaft diameters center-to-center for pile groups to limit the potential for reduction of axial capacity due to group effect.

Construction of drilled shafts within the Saugus formation is not expected to cause caving with open-hole drilling method. Therefore, casing or slurry is not anticipated to be required for the proposed construction within Saugus formation. The contractor’s installation procedure should include cleaning up the bottom of the hole from loose materials.

5.3.2.2 Shaft Lateral Load Analysis

Geosyntec performed lateral load soil-structure interaction analysis for the drilled shafts using the computer program LPILE 2019.11.06 (Ensoft Inc., 2019). The analyses were performed for a 2-, 3- and 4-ft diameter shaft for a free- and fixed-head condition when subject to ½-inch and one-inch lateral deflection at the shaft head. The shaft head was assumed at the ground surface. Analyses were performed by modeling the pile section with 50% of the full section stiffness to mimic the cracked section behavior. The soil parameters as used for input in LPILE are provided in Table 7. The analyses were performed for 12-ft long shafts to demonstrate “short-pile” lateral loading behavior. Table 8 provides the lateral loads at the shaft top for a short shaft with a length of 12 ft corresponding to 0.5-inch and 1-inch shaft top deflection, for various shaft diameters. Additionally, lateral load analyses were performed for longer shafts to demonstrate “long-pile” behavior to evaluate lateral capacities corresponding to 0.5-inch and 1-inch top deflection and the minimum length required for "long-pile" behavior, for various shaft diameters. The long-pile lateral load analyses results are presented in Table 9.

Pile spacing should be kept at a minimum of 8 shaft diameters center-to-center to avoid the potential reduction of lateral load carrying capacity of single pile due to group effect.

5.4 Corrosion Potential

A summary of the soil chemical laboratory testing results is presented in Table 5. Appendix C presents the soil chemical laboratory test results.

Based on the criteria established by the County of Los Angeles Public Works (LADPW, 2013), soils are considered corrosive when soluble sulfate concentrations in the soil are equal to or greater than 2,000 parts per million (ppm) (or milligram per kilogram (mg/kg)), or chloride concentrations in the soil are equal or greater than 500 ppm (or mg/kg), or the pH value of the soil is equal or less than 5.5, or the soil's minimum resistivity value is less than 1,000 ohm-centimeters. Soil chemical test results from one soil sample collected during the investigations indicate that the measured values are well outside the ranges typically considered harmful or deleterious to foundation elements.

In a review of American Concrete Institute (ACI) 318-19 (2022) Table 19.3.1.1, per the criteria established by California Building Code, the measured values of sulphate concentration indicate exposure class S0 which is assigned for conditions where the water-soluble sulfate concentration in contact with concrete is low and injurious sulfate attack is not a concern.

5.5 Stability of Permanent Cut Slopes

As discussed in Section 5.1, based on the existing ground elevation contours presented in Figure 2 and our assumed final ground elevation within the substation footprint, it appears that the tallest slope will be an east-facing slope and may be up to 80 ft in vertical height. In addition, there will likely be a north facing cut slope, but its height is expected to be significantly less than 80 ft.

LADPW (2013) requires slope stability analysis for all cut, fill, and natural slopes when the slope height exceeds 30 ft. Therefore, we evaluated the stability of a conceptual 80-ft tall cut slope ascending to the west from the proposed substation footprint. The details and the results of our evaluation is presented in this section. Once the final grading plan is available, Geosyntec should be provided a copy so we can evaluate if revisions to our limited slope stability evaluation is needed and issue a revised slope stability report as an additional service, if applicable.

5.5.1 Rock Strength Characterization

As discussed in Section 3.4.3, the Saugus Formation indicated a general dip of 50 to 70 degrees to the southwest. This bedding plane is a favorable orientation for the stability of the east facing slope. Therefore, we used isotropic shear strength parameters for the Saugus formation developed by the Generalized Hoek-Brown empirical failure criterion (Hoek, Carranza-Torres, Corkum, 2002). Geosyntec previously prepared a supplemental report (Geosyntec 2021b) to the Geosyntec (2021a) report presenting slope stability analyses for the proposed cut slopes for the Compressor Modernization project. The isotropic rock strength parameters were developed as part of the Geosyntec (2021b) report and a copy of the description of development of the parameters from that report is included in Appendix E herein. The table below summarizes the rock strength parameters and how they were evaluated.

Generalized Hoek-Brown Parameter	Design Value	Notes
Intact Rock Strength	21 ksf	Based on uniaxial compression test performed on rock cores sampled.
Geological Strength Index, GSI	65	Based on the observed surface condition (good to fair) and rock structure (rough, slightly to moderately weathered surfaces, altered surfaces, and blocky) of the thick to very thick beds observed during geologic site reconnaissance.
Material constant (for intact rock), m_i	10	This parameter is a function of the rock group. The design value of 10 is based on a 60:40 ratio of sandstone to claystone.
Disturbance Factor, D	0.7	This parameter varies from 0 for undisturbed in-situ rock masses to 1 for very disturbed rock masses. The design value of 0.7 is based on our observation that the rock masses at the site are weathered yet primarily intact on the surface.

Even though these parameters were developed for locations that are as far as 800 ft from our current Option A location, our geologic reconnaissance performed for the current study indicates that the rock surface condition, rock structures observed, and degree of weathering is very similar to what was assumed for Geosyntec (2021b). In addition, our boring logs HSA-3 through HSA-5 indicate that the Saugus formation consists primarily of sandstone and claystone which are close to the assumed 60:40 ratio as evaluated before. Therefore, we utilized the same isotropic rock shear strength parameters that are presented in the table above for slope stability evaluation at the Option A location.

5.5.2 Cross Sections Analyzed

Geosyntec performed stability evaluation of the conceptual 80-ft tall cut slope for two slope inclination configuration (i) 1 horizontal to 1 vertical (1H:1V) slope and (ii) 2H:1V slope, incorporating Los Angeles County Department of Public Works Grading Guidelines (LADPW, 2017). LADPW (2017) requires that drainage terraces at least 8 ft in width shall be established at no more than 30-foot vertical intervals on all slopes to control surface drainage and debris. Thus, our 80-ft tall slope was assumed to have two 8-ft wide horizontal drainage terraces, one at 30 ft above the toe of slope and the other at 60 ft above the toe of the slope.

5.5.3 Slope Stability Analysis and Seismic Deformation Evaluation Methods

For evaluating the slope stability of the configurations described above, Geosyntec used conventional two-dimensional limit equilibrium analysis method and calculated a factor of safety (FS) against sliding for static and seismic conditions. In particular, Geosyntec employed the Morgenstern and Price (1965) method, as implemented in SLIDE2 (Rocscience).

The results of the static analyses are presented in the form of critical (static) failure surfaces and the corresponding lowest calculated FS. For the seismic stability evaluation, a pseudostatic stability analysis was performed by applying a horizontal seismic coefficient, k_h, as an additional driving force and calculating the FS. Two separate pseudostatic analyses were performed for each cross section. The first analysis was performed by applying a k_h of 0.15 and calculating the FS as a screening analysis for deformations up to 3 ft per the LADPW (2013). The second analysis was performed following the method presented in Bray and Travasarou (2007) and provides an estimate of the anticipated permanent seismic deformations following the design seismic event. The input parameters to Bray and Travasarou (2007) method for each cross section are the k_h value providing an FS of 1.0 (i.e., yield acceleration), the natural period of the potential slip surface, and the design response spectral acceleration corresponding to 1.5 times the natural period of the potential slip surface. The design response spectrum presented in Section 4.2 was utilized in our evaluation of seismic deformation.

5.5.4 Acceptance Criteria

The slope stability acceptance criteria were developed primarily based on LADPW (2013) and are presented below.

Analysis Case	Criteria	Notes
Long-term static	FS ≥ 1.5	Per LADPW (2013)
Seismic	FS ≥ 1.1 with a k_h = 0.15	Per LADPW (2013)

FS = 1.1 for a horizontal seismic coefficient of 0.15 corresponds to seismic deformation of 36 inches. Therefore, if the above FS criteria are met, it implies that the proposed cut slope is anticipated to have deformations smaller than 36 inches.

5.5.5 Analysis Results

The calculated FS for the static and seismic conditions and the calculated permanent seismic deformations are summarized in Table 10. The graphical outputs of the Slope/W computer program are included in Appendix F. The FS criteria for static and seismic cases were met for both the 1H:1V and 2H:1V cut slope configurations analyzed. The calculated seismic deformation was 15 inches for the 1H:1V configuration and 3 inches for the 2H:1V configuration. If lower seismic deformation potential than 3 inches is desired, a flatter slope configuration than 2H:1V should be considered.

5.6 Pavement

Geosyntec understands that the paved roads inside the substation should be designed to support traffic load corresponding to a traffic index (TI) equal to 5.

The flexible pavement section should consist of asphalt concrete (as defined in Section 39 of the latest edition of the Caltrans Standard Specifications) over Class 2 aggregate base (as defined in Section 26 of the latest edition of the Caltrans Standard Specifications) over properly prepared subgrade as described in Section 5.1.1. Asphalt and aggregate base should be compacted to a minimum relative compaction of 95 percent.

The recommended pavement section for TI = 5 is 4-inch asphaltic concrete over 8 inches of Class II aggregate base.

Pavement section thicknesses were based on an R-value of 18 based on Geosyntec (2021a). The R-value test was performed on a soil sample collected from Geosyntec (2021a)'s boring HSA-2 from a depth of 0 to 5 feet below ground surface. This soil sample was classified as clayey sand fill. The subgrade under the proposed substation roads is anticipated to include Saugus formation siltstone/sandstone or engineered fill which may still consist of clayey sand fill soils. Therefore, the pavement section calculated using R-value of 18 is considered conservative yet plausible. The geotechnical consultant shall observe the prepared subgrade for the pavements to confirm the subgrade conditions are consistent with the design assumptions. Additional R-value testing may be required if varying soil conditions are encountered during construction.

Geosyntec recommends including subgrade enhancement geotextile or geogrid within the soil and aggregate base layer to reinforce the subgrade, distribute traffic loading and reduce the potential for cracking for flexible pavements. Non-woven geotextiles or geogrid used for subgrade enhancement shall conform to the requirement in Section 96 of the latest edition of the Caltrans Standard Specifications and Caltrans' Subgrade Enhancement Geosynthetic Design and Construction Guide (2013). If a geogrid layer is incorporated into the subgrade such that the R-value of the geogrid-enhanced subgrade is 25, then the thickness of Class II aggregate base may be reduced to 7 inches instead of 8.

The selection of the appropriate type of geotextile or geogrid shall be based on subgrade R-value and gradation of the subgrade and aggregate base materials, evaluated by the geotechnical consultant during construction. The subgrade preparation requirements would remain unchanged if a subgrade enhancement geotextile or geogrid is used.

6. CONSTRUCTION CONSIDERATIONS

6.1 Permanent Slope Cuts

Based on the conditions encountered in exploratory borings HSA-3 through HSA-5, the Saugus Formation materials are expected to be rippable with conventional excavation equipment. Geosyntec was able to penetrate the formation using the conventional auger drilling system without the need for rock coring bit. Additionally, a p-wave seismic refraction survey, to determine the rippability of the Saugus formation rock, was not part of our scope for the Option A location. However, this survey was performed for Geosyntec (2021a) at a location that is about 800 ft northwest from the Option A location and the geophysical report from Geosyntec (2021a) is presented in Appendix G of this report.

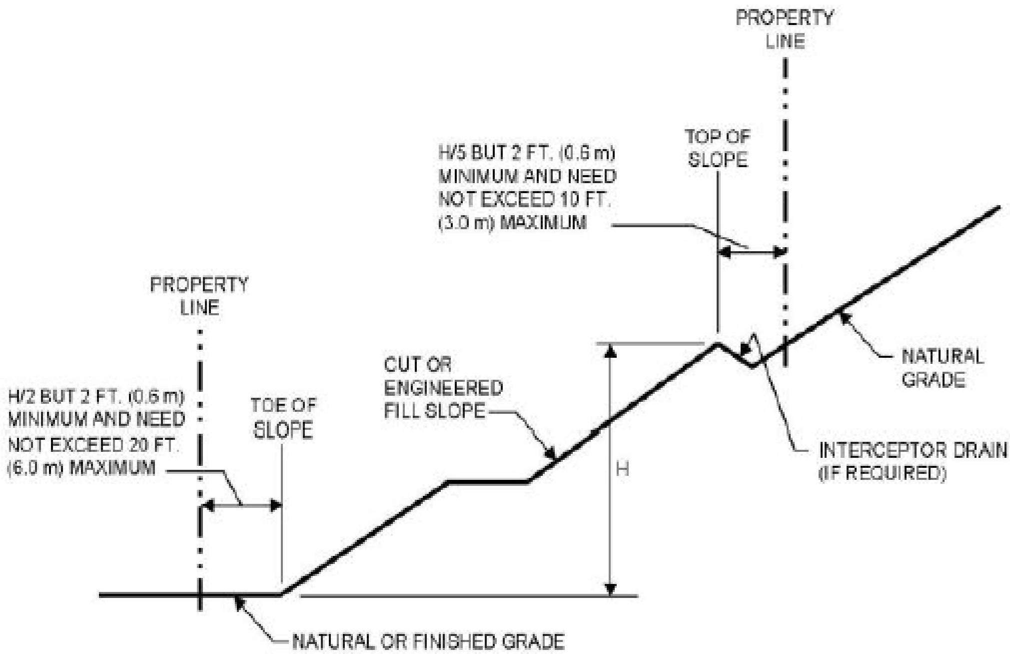
Appendix G shows that the Saugus bedrock portion of the subsurface exhibited p-wave velocities in the 2,500 to 4,500 ft/sec range to a depth of about 40 ft bgs. P-wave velocity profile below a depth of 40 ft bgs is not available. Based on Caterpillar Performance Handbook, 2018, 48th edition (Caterpillar, 2018), sandstone rock may broadly be characterized as rippable using a Caterpillar D8R ripper when p-wave velocities are less than 6,500 ft/sec, marginally rippable to 8,300 ft/sec, and non-rippable at p-wave velocities greater than 8,300 ft/sec. Thus, the Saugus formation within the upper 40 ft bgs generally appear to be rippable.

We understand SCG has extensive experience performing slope cuts in Saugus bedrock within the Honor Rancho site. However, if SCG requires input on the rippability of the rock specific to Option A location, we recommend performing a seismic refraction survey at this location, similar to what was done for Geosyntec (2021a). Geosyntec can provide this service, if needed.

Geosyntec recommends spraying the slopes with bonded fiber matrix (BFM) hydraulic mulch as part of regular maintenance of the cut slopes to reduce the potential of rain-based slope surface erosion. Additionally, drainage ditches should be incorporated along the 8-foot wide benches approximately at the slope mid-height.

Based on slope stability analysis, cut slopes as steep as 1H:1V may be stable based upon calculations, however Geosyntec recommends slopes flatter than 1.5H:1V be utilized for this project based on our experience with past slope performance and surface erosion at the Facility. A 1.5H:1V slope will experience significant surficial erosion and will require regular maintenance of surface drains and toe-of-slope areas. The need for surface erosion protection in the long term is also recommended.

Cut slopes should be set back from the perimeter of the substation in accordance with recommendations presented in LADPW (2017). Per LADPW (2017), setback dimensions should be measured perpendicular to the substation fence line and should be as shown in the image below.



Per this image, the substation fence line should be set back 20 ft from the toe of the 80-ft high slope. Greater set back may be considered by SCG if needed to incorporate drainage and access features.

6.2 Temporary Slopes

The design and excavation of temporary slopes and their maintenance during construction are the responsibility of the contractor. Based on the materials observed in the borings, the design of temporary slopes for planning purposes may assume Type B conditions. The contractor shall have a geotechnical or geological professional evaluate the soil conditions encountered during excavation, for any variation in soil conditions, to determine the appropriate permissible temporary slope inclinations and other measures required by Cal OSHA. Existing infrastructure within a 2:1 (H:V) line projected up from the toe of temporary slopes should be monitored during construction.

6.3 Construction Observation and Testing

Soil/rock deposits may vary in type, strength, and many other important properties between points of exploration, due to non-uniformity of the geologic formations or to man-made cut and fill operations, during construction at the site. To permit correlation between the investigation data, design, and the conditions encountered during construction, we recommend that the geotechnical engineer be retained to provide continuous observations of excavation operations and foundation construction. The project geotechnical engineer should review and approve all subgrade, excavation bottoms, and proposed import materials, if applicable, before their use.

7. LIMITATIONS

The geotechnical investigation for this project observed only a small portion of the pertinent subsurface conditions. The recommendations made herein are based on the assumption that soil/rock conditions do not deviate appreciably from those found during the current field investigation. This geotechnical investigation report has been prepared in accordance with current practices and the standard of care exercised by scientists and engineers performing similar tasks in this area. The conclusions contained in this report are based solely on the analysis of the conditions observed by Geosyntec personnel. We cannot make any assurances concerning the completeness of the data presented to us.

No warranty, expressed or implied, is made regarding the professional opinions expressed in this report. Site grading and earthwork, subgrade preparation under concrete slabs, and foundation excavations should be observed by a qualified engineer or geologist to verify that the site conditions are as anticipated. If actual conditions are found to differ from those described in the report, or if new information regarding the site conditions is obtained, Geosyntec should be notified and additional recommendations, if required, will be provided. Geosyntec is not liable for any use of the information contained in this report by persons other than SCG, or their subconsultants, or the use of information in this report for any purposes other than referenced in this report without the expressed, written consent of Geosyntec.

California, including Los Angeles County, is an area of high seismic risk. It is generally considered economically unfeasible to design structures to resist earthquake loadings without damage. Proposed structures designed in accordance with the recommendations presented in this report could experience limited distress/damage if subjected to strong earthquake shaking.

8. REFERENCES

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TABLES

Table 1
Summary of Exploratory Borings
Substation Option A
Honor Rancho Facility
Santa Clarita, California

Exploration Name	Exploration Type[1]	Surface Elevation (feet, NAVD88)[2,3]	Exploration Latitude (degrees)[4]	Exploration Longitude (degrees)[4]	Depth Advanced (feet bgs[5])	Date Advanced
HSA-3	HSA Boring	1103	34.444000	-118.583590	41.5	6/9/2023
HSA-4	HSA Boring	1123	34.444500	-118.583660	31.5	6/9/2023
HSA-5	HSA Boring	1125	34.444830	-118.583780	31.5	6/9/2023

Notes:

1. HSA = Hollow-stem Auger
2. NAVD = North American Vertical Datum
3. The surface elevation of the borings were obtained from site topographic map provided by Southern California Gas Company.
4. The latitude and longitude of the borings were estimated using Google Earth™ based on field measurement and are considered approximate.
5. bgs = below ground surface.

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Table 2
Summary of Geotechnical Laboratory Test Results
Substation Option A
Honor Rancho Facility
Santa Clarita, California

Sample Information				USCS Classification (3)	USCS Name	Percent Passing #200 Sieve	Atterberg Limits			Moisture-Density Tests			Other Tests (5)
						ASTM D1140	ASTM D4318			ASTM D2216 & D2937			
Boring ID	Sample ID	Sample Type (1)	Depth (ft bgs) (2)			Silt & Clay (% < #200)	Liquid Limit LL	Plastic Limit PL	Plasticity Index PI	Dry Density (pcf) (4)	Moisture Content (%)	Moist Unit Weight (pcf)	
HSA-3	S-1	Cal Mod	6-6.5	SM	Silty Sandstone					101.7	9.1%	111.0	
	S-3	Cal Mod	11-11.5	SC	Clayey Sandstone					109.0	7.7%	117.4	DS: c = 150 psf; ϕ = 28° (peak) c = 150 psf; φ = 27° (ult)
	S-5	Cal Mod	16-16.5	CL	Claystone with sand					105.7	8.2%	114.4	
	S-6	SPT	20-21.5	CL	Claystone with sand	82							
	S-7	Cal Mod	26-26.5	ML	Sandy Siltstone					110.1	6.6%	117.4	DS: c = 300 psf; ϕ = 29° (peak) c = 200 psf; φ = 27° (ult)
	S-9	Cal Mod	36-36.5	SC	Clayey Sandstone					117.4	9.1%	128.1	
HSA-4	S-2	Cal Mod	8.5-9	SC	Clayey Sand					120.1	10.3%	132.5	DS: c = 500 psf; φ = 27° (peak) c = 200 psf; φ = 30° (ult)
	S-3	SPT	10-11.5	CL	Sandy lean Clay	54							
	S-5	SPT	15-16.5	CL	Sandy lean Clay		30	16	14				
	S-6	Cal Mod	21-21.5	SM	Silty Sand					97.5	6.9%	104.2	
	S-8	Cal Mod	31-31.5	CL	Sandy Claystone					110.5	9.1%	120.6	
HSA-5	S-1	Cal Mod	6-6.5	SM	Silty Sand					103.4	5.7%	109.3	DS: c = 50 psf; φ = 32° (peak) c = 50 psf; φ = 33° (ult)
	S-2	SPT	7.5-9	SM	Silty Sand	30							
	S-3	Cal Mod	11-11.5	SM	Silty Sand					111.3	3.8%	115.5	
	S-5	Cal Mod	16-16.5	SM	Silty Sand					104.3	6%	110.6	
	S-7	Cal Mod	26-26.5	SM	Silty Sandstone					104.6	7%	111.9	

Notes:
1. Cal Mod = California Modified sampler; SPT = Standard Penetration Test Drive sample
2. bgs = Below Ground Surface
3. USCS = Unified Soil Classification System
4. pcf = pounds per cubic foot
5. DS = Direct Shear Test (ASTM D3080); c = cohesion; φ = friction; ult = ultimate; psf = pounds per square foot

Table 3
Nearby Faults
Substation Option A
Honor Rancho Facility
Santa Clarita, California

Fault Name	Distance and Direction from Site (1)	Mean 30-Year Participation Probability (%) (2)			
		M≥6.7	M≥7.0	M≥7.5	M≥8.0
Holser	0.3 mi (0.5 km) to south	0.42	0.40	0.26	<0.01
San Gabriel	0.6 mi (1.0 km) to northeast	0.36	0.29	0.23	0.01
Northridge	3.9 mi (6.3 km) to southwest	0.86	0.75	0.46	<0.01
Del Valle	5.3 mi (8.7 km) to southwest	0.68	0.56	0.30	<0.01
Santa Felicia (3)	5.2 mi (8.4 km) to northwest	NR	NR	NR	NR
Santa Susana	8.4 mi (13.5 km) to southwest	3.80	2.58	0.68	<0.01
Oak Ridge	8.7 mi (14.0 km) to southwest	2.80	2.77	1.10	<0.01
San Cayetano	10.2 mi (16.5 km) to west	2.16	2.06	0.98	<0.01
Northridge Hills	10.9 mi (17.5 km) to southwest	0.60	0.59	0.43	<0.01
Mission Hills	12.0 mi (19.4 km) to south	0.84	0.46	0.17	NR
Sierra Madre	12.3 mi (19.8 km) to southeast	1.06	0.75	0.38	0.01
Verdugo	16.8 mi (27.0 km) to southeast	0.41	0.40	0.26	<0.01
San Andreas	17.4 mi (28.0 km) to northeast	17.12	17.10	16.91	6.78

Notes:

1. Distances from site noted are the closest distance to the fault location according to the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) Fault Model 3.2 [Field et al., 2015], except for the Santa Felicia fault (see Note c). These distances may be different than the surface trace or inferred projection of the fault as measured from mapped traces in the USGS Quaternary Fault and Fold Database of the United States [USGS, 2006] and the Earthquake Zones of Required Investigation database [CGS, 2022].
2. As reported by UCERF3 Fault Model 3.2 [Field et al., 2015]. “NR” = Not Reported.
3. Distance as measured to the fault trace in the USGS Quaternary Fault and Fold Database of the United States [USGS, 2006]

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Table 4
Seismic Design Parameters
Substation Option A
Honor Rancho Facility
Santa Clarita, California

Seismic Hazard Parameter	**Value**
Approximate Site Latitude	34.444208 degrees
Approximate Site Longitude	-118.583851 degrees
Average Shear Wave Velocity of the top 100 ft (30 m), V_{S30} (estimated from Suspension Logging)	1650 ft/s to 2300 ft/sec
Risk Category	IV
Site Class	C
Mapped Short Period Spectral Response Acceleration, S_s	2.066 g
Mapped 1-second Spectral Response Acceleration, S_1	0.756 g
Short Period Site coefficient (at 0.2-s period), F_a	1.2
Long Period Site coefficient (at 1.0-s period), F_v	1.4
Site-modified Short Period Spectral Response Acceleration, S_{MS}	2.48 g
Site-modified 1-second Spectral Response Acceleration, S_{M1}	1.058 g
Design Short Period Spectral Response Acceleration, S_{DS}	1.653 g
Design 1-second Spectral Response Acceleration, S_{D1}	0.705 g
Mapped MCEG Peak Ground Acceleration, PGA	0.873 g
Site Coefficient, F_{PGA}	1.2
Site Class Adjusted MCEG Peak Ground Acceleration, PGA_M	1.048 g

Table 5
Summary of Soil Chemical Test Results
Substation Option A
Honor Rancho Facility
Santa Clarita, California

Boring ID	Sample ID	Depth (ft bgs)	USCS Classification	CTM 417	CTM 422	CTM 643	CTM 643
				Sulfates	Chlorides	Min. Resistivity	pH
				(ppm)	(ppm)	(Ohm-cm)	
HSA-3	B-1	1-5	SM	17	17	6,641	9.9

Notes:
ft bgs = feet below ground surface
CTM = California Test Method
ppm = parts per million

Table 6
Shaft Axial Load Capacities (Ultimate)
Substation Option A
Honor Rancho Facility
Santa Clarita, California

Shaft Diameter (inches)	End Bearing, Q (kips)	Skin Friction per foot socket, F (kips/ft)
24	62	9
36	141	14
48	251	19

Table 7
LPILE Soil Input Parameters
Substation Option A
Honor Rancho Facility
Santa Clarita, California

Depth (ft bgs)	Soil Model	Effective Unit Weight (pcf)	Cohesion (psf)	Strain Factor
0 to 55	Stiff clay without free water	135	4,000	0.004

Notes:
pcf = pounds per cubic feet
psf = pounds per square foot

Table 8
Lateral Load Capacities: Short Pile
Substation Option A
Honor Rancho Facility
Santa Clarita, California

Shaft Diameter (inches)	Head Fixity	Top Deflection (inch)	Lateral Load Capacity (kips)	Maximum Bending Moment (inch-kips)
24	Free-Head	0.5	75	2,200
		1	92	2,800
	Fixed-Head	0.5	136	5,900
		1	184	9,300
36	Free-Head	0.5	100	3,100
		1	120	3,700
	Fixed-Head	0.5	248	16,300
		1	336	24,800
48	Free-Head	0.5	124	3,600
		1	146	4,400
	Fixed-Head	0.5	352	26,500
		1	436	33,200

Notes:

The maximum shear within the shaft is at the shaft top and equal to the lateral load capacity provided in this Table.

Table 9
Lateral Load Capacities: Long Pile
Substation Option A
Honor Rancho Facility
Santa Clarita, California

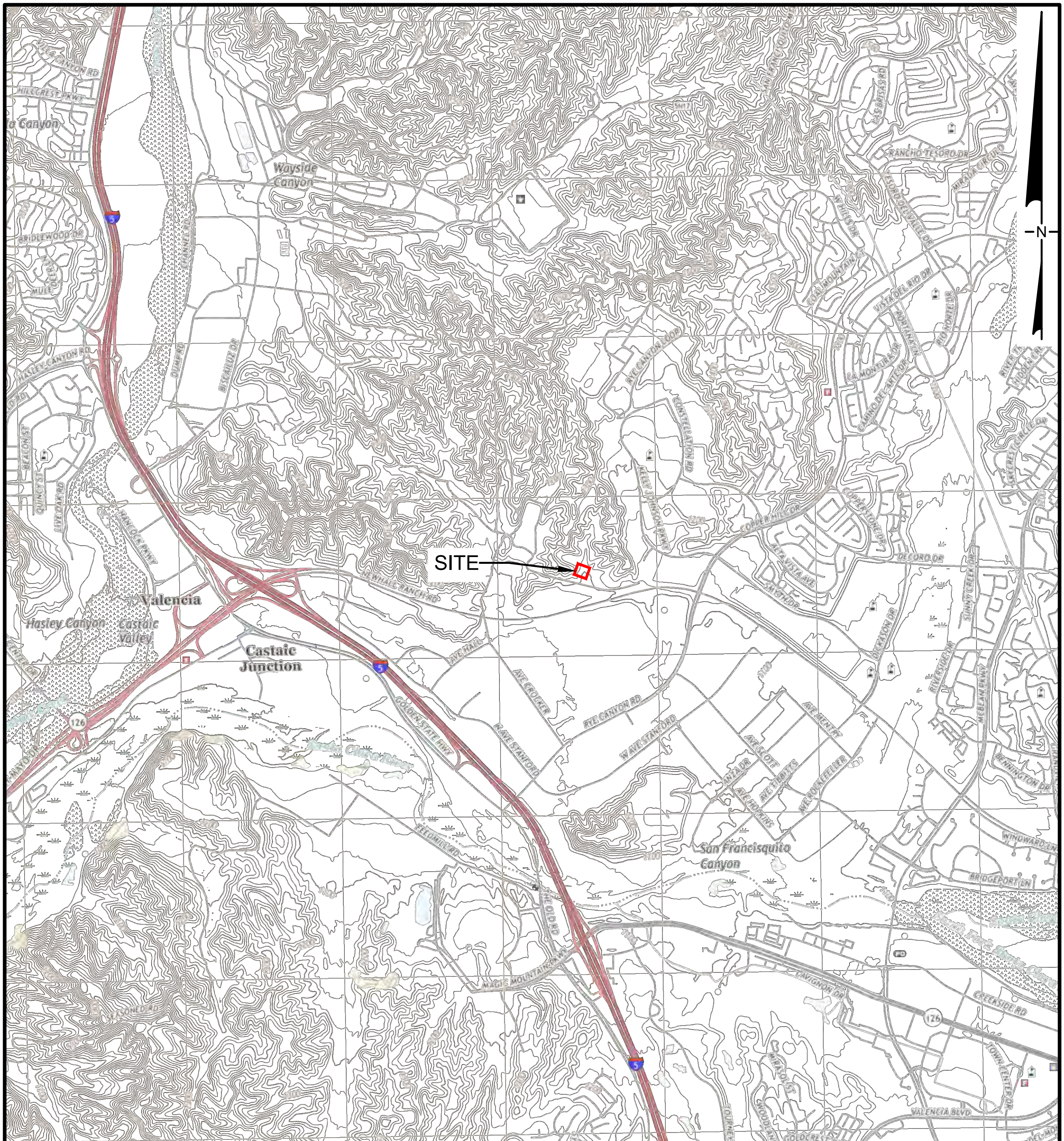
Shaft Diameter (inches)	Head Fixity	Top Deflection (inch)	Lateral Load Capacity (kips)	Minimum Shaft Length (ft)	Maximum Bending Moment (inch-kips)
24	Free-Head	0.5	80	16	2,500
		1	109	16	3,900
	Fixed-Head	0.5	155	17	6,500
		1	214	20	10,100
36	Free-Head	0.5	150	20	6,600
		1	207	22	10,400
	Fixed-Head	0.5	290	23	16,800
		1	403	26	26,500
48	Free-Head	0.5	235	24	13,100
		1	321	26	20,300
	Fixed-Head	0.5	455	28	33,300
		1	630	32	52,300

Notes:
The maximum shear within the shaft is at the shaft top and equal to the lateral load capacity provided in this Table.

Table 10
Summary of Slope Stability Analysis Results and Seismic Deformations
Substation Option A
Honor Rancho Facility
Santa Clarita, California

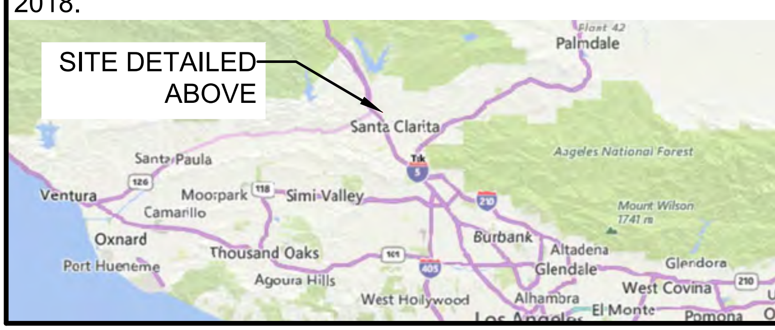
Slope Configuration	Analysis Case	FS (Calculated)	Seismic Deformation (inches)
1H:1V	Long-term Static	1.7	15
	Seismic (with k_h=0.15)	1.35	
2H:1V	Long-term Static	2.56	3
	Seismic (with k_h=0.15)	1.81	

FIGURES



SOURCE: TOPOGRAPHICAL CONTOUR MAP FOR THE 7.5-MINUTE NEWHALL QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA. U.S. GEOLOGICAL SURVEY, 2018.

0 3,000
SCALE IN FEET



SITE LOCATION MAP
SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

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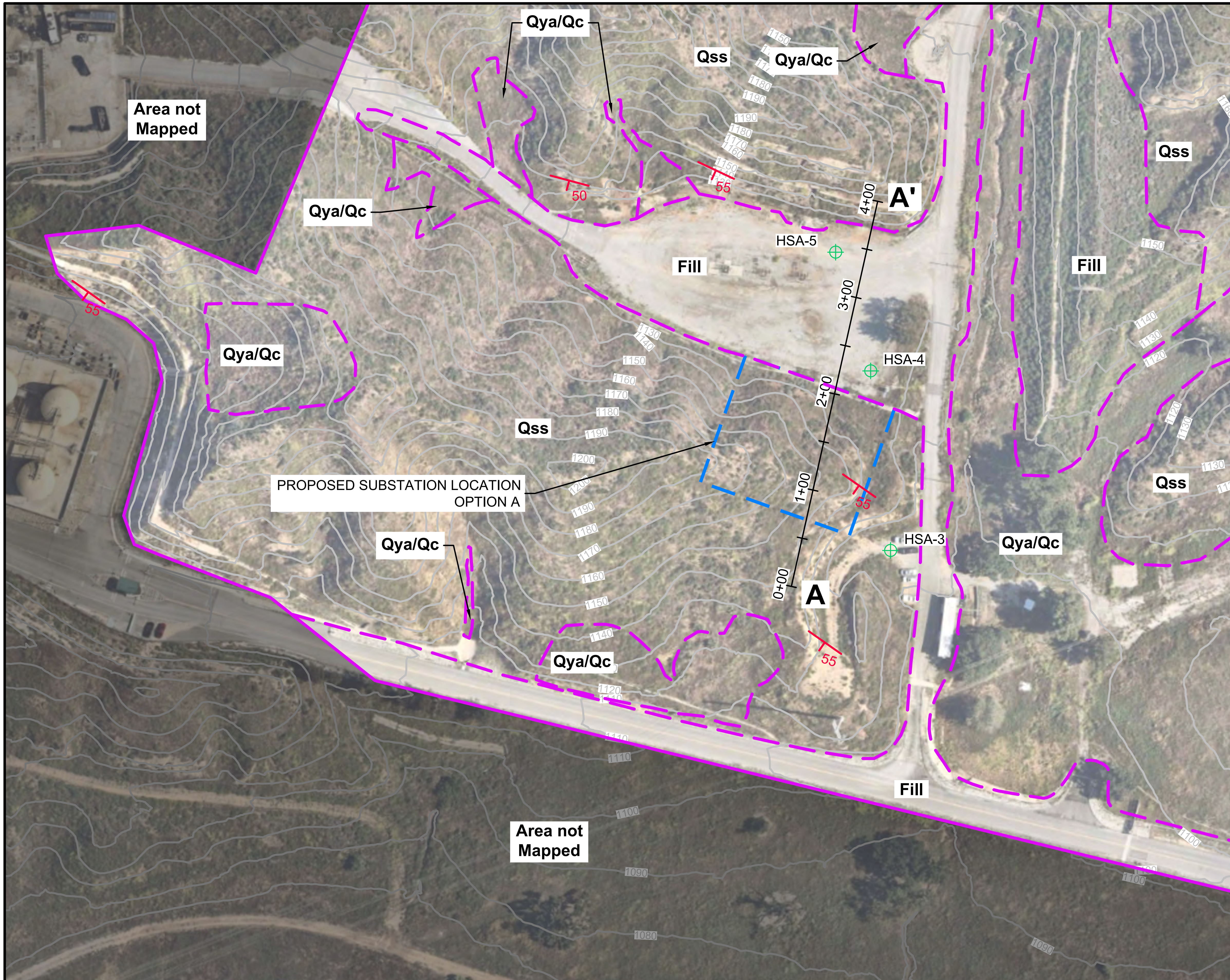
FIGURE 1

PROJECT NO: SC1339 | JULY 2023

N:\CACADD\SO CAL GAS COMPANY\SO CAL GAS COMPANY - SC1339\FIGURES\SC1339F001 - Last Saved By: KViswanathan on 7/17/23



LEGEND

1150	EXISTING GROUND MAJOR CONTOUR
HSA-5	HOLLOW STEM AUGER BORING (HSA)
A A'	CROSS SECTION (FIGURE 5)
Fill	ARTIFICIAL FILL
Qya/Qc	QUATERNARY YOUNG ALLUVIUM AND QUARTERNARY COLLUVIUM, UNDIVIDED
Qss	QUATERNARY SAUGUS FORMATION
	APPROXIMATE GEOLOGIC CONTACT
50	APPROXIMATE STRIKE AND DIP OF BEDROCK

EXPLORATION LOCATION / GEOLOGIC MAP
SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

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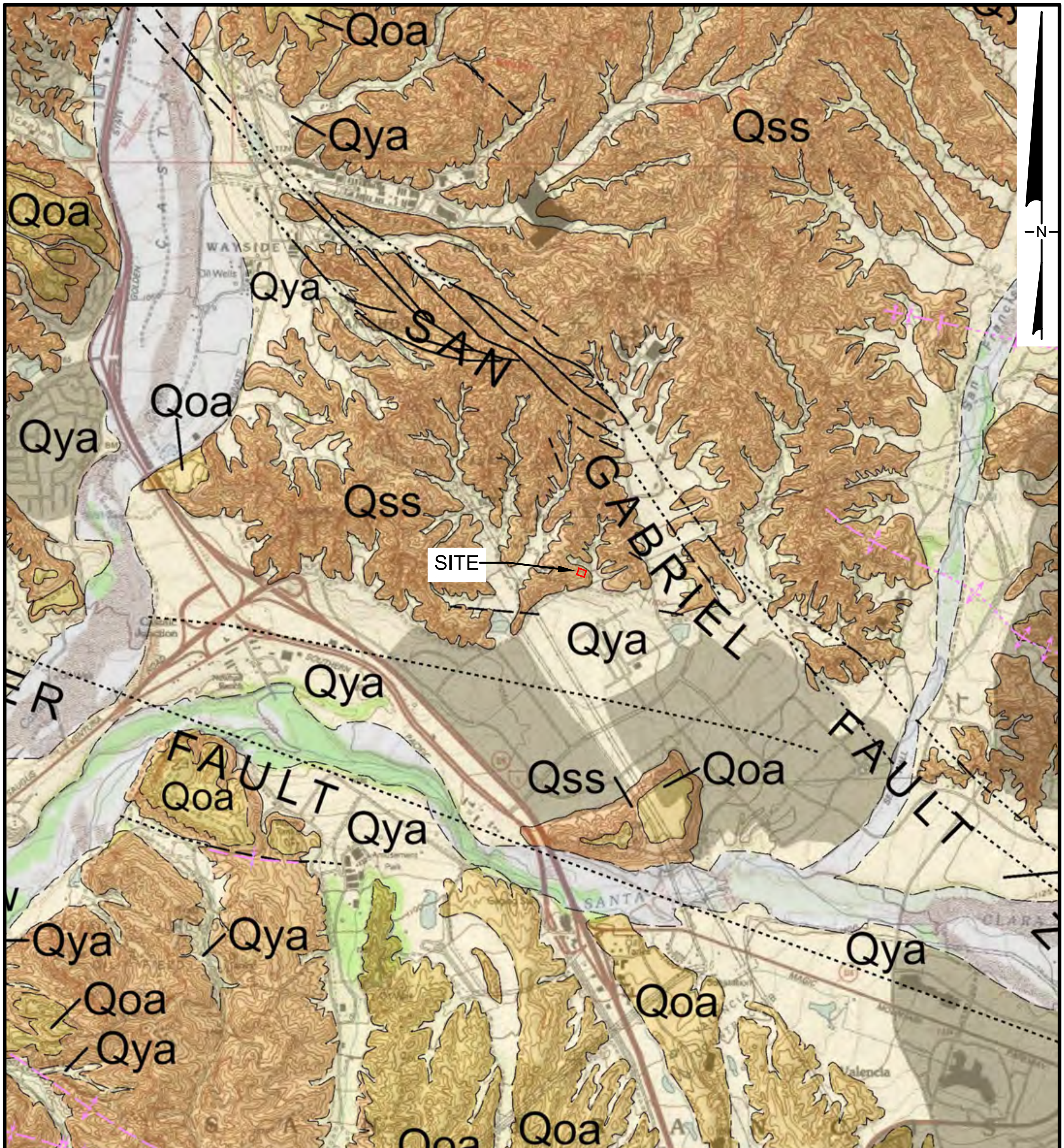
FIGURE 2

PROJECT NO: SC1339 | JULY 2023

N:\CADD\SISO CAL GAS COMPANY\SO CAL GAS COMPANY - SC1339\FIGURES\SC1339F002 - Last Saved by: KViswanathan on 7/17/23



SOURCE: SPECIAL REPORT 217, GEOLOGIC COMPILATION OF QUARTERNARY SURFICIAL DEPOSITS IN SOUTHERN CALIFORNIA. CALIFORNIA GEOLOGICAL SURVEY, 2012.

0 3,000
SCALE IN FEET

LEGEND

Qya – Young Alluvial Valley Deposit
Qoa – Older Alluvial Deposit
Qss – Sauqus Formation, Sandstone Unit
Qw – Active-wash Deposit

REGIONAL GEOLOGIC MAP
SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

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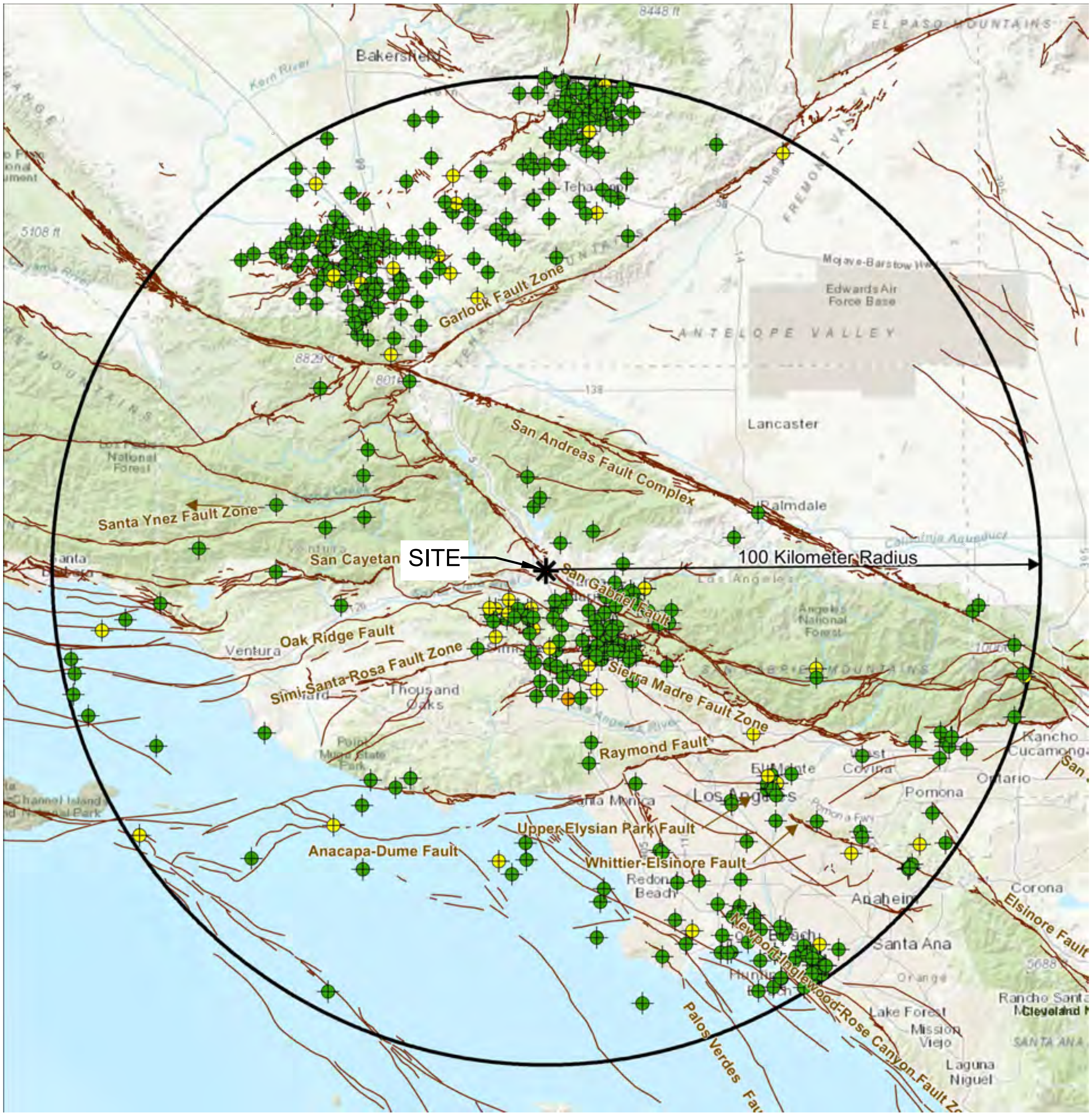
FIGURE 3

PROJECT NO: SC1339 | JULY 2023

N:\CACADD\S\SO CAL GAS COMPANY\SO CAL GAS COMPANY - SC1339\FIGURES\SC1339F003 - Last Saved By: KViswanathan on 7/15/23



SOURCE: SEISMIC HAZARD ZONE REPORT FOR THE NEWHALL 7.5-MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA. DEPARTMENT OF CONSERVATION, DIVISION OF MINES AND GEOLOGY, 1997.

0 20

SCALE IN MILES

LEGEND

Reported Earthquake Magnitudes 1932 - 2021

- 7.5
- 6.0 - 6.99
- 5.0 - 5.99
- 4.0 - 4.99
- Quarternary Fault

REGIONAL FAULT AND HISTORICAL EARTHQUAKE EPICENTER MAP
SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

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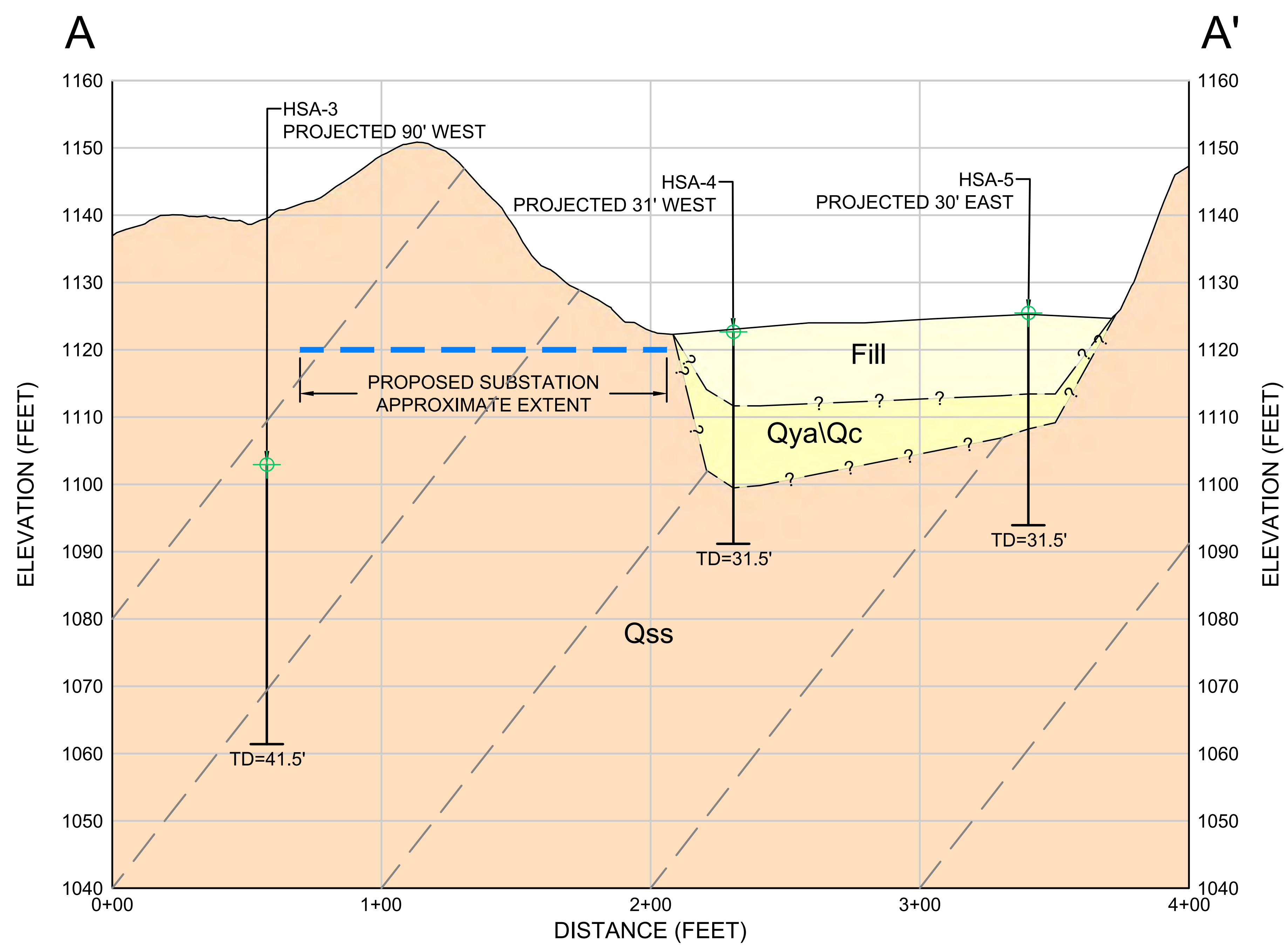
FIGURE 4

PROJECT NO: SC1339

JULY 2023

N:\CACADD\S\SO CAL GAS COMPANY\SO CAL GAS COMPANY - SC1339\FIGURES\SC1339F004 - Last Saved By: KViswanathan on 7/15/23



LEGEND

EXISTING GROUND MAJOR CONTOUR		Fill	UNDOCUMENTED FILL
HOLLOW-STEM AUGER BORING (HSA)		Qya/Qc	QUATERNARY YOUNG ALLUVIUM AND QUARTERNARY COLLUVIUM, UNDIVIDED
— ? — — — —	APPROXIMATE GEOLOGIC CONTACT (QUERIED WHERE UNCERTAIN)	Qss	QUARTERNARY SAUGUS FORMATION SANDSTONE WITH INTERBEDDED CLAYSTONE OF VARYING THICKNESS
APPROXIMATE DIP OF BEDDING (APPARENT DIP)			

NOTE: APPARENT DIP OF BEDDING APPEARS STEEPER THAN ACTUAL DUE TO VERTICAL EXAGGERATION.

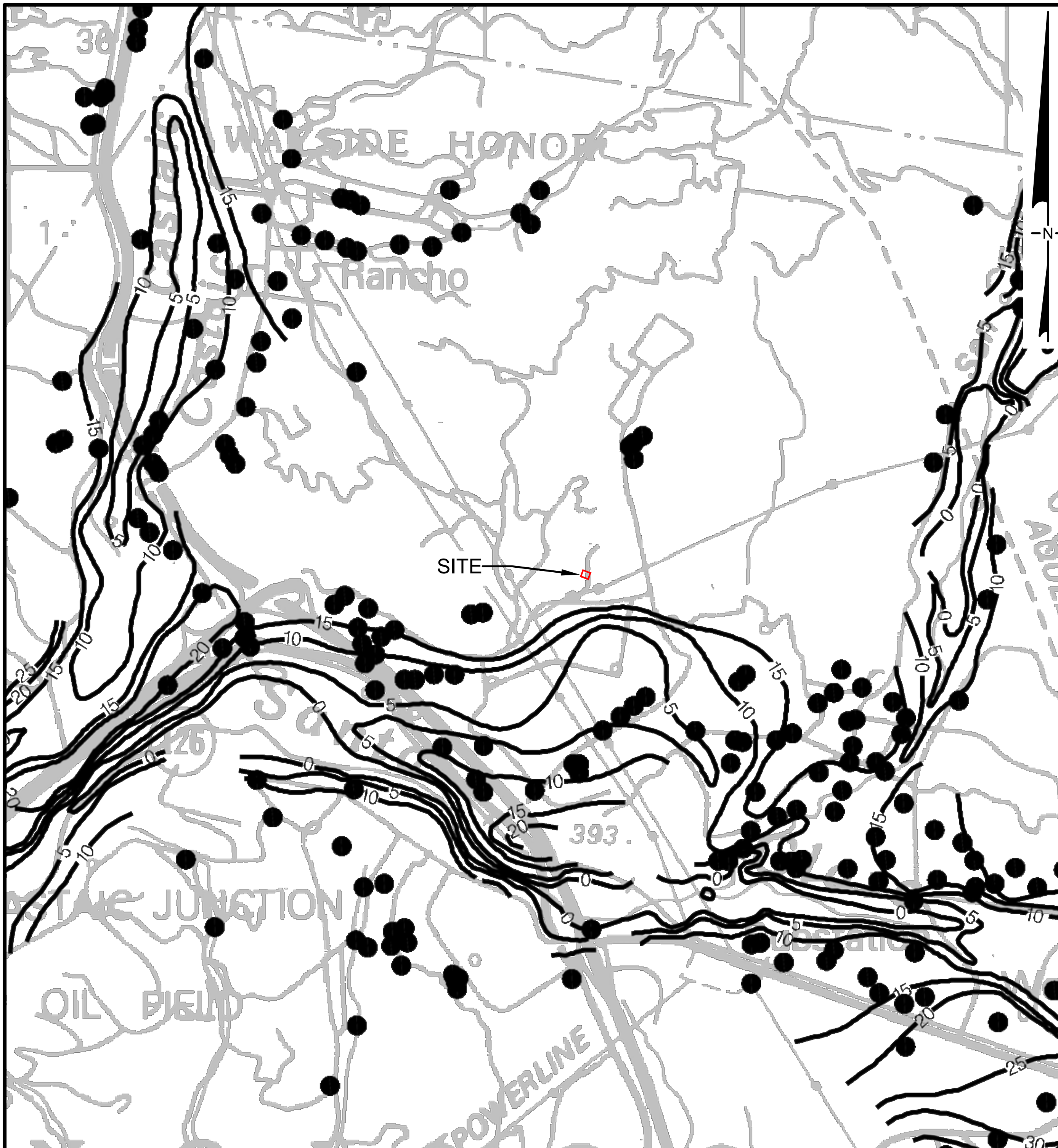
GEOLOGIC CROSS SECTION A-A'
SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

Geosyntec consultants

FIGURE 5

PROJECT NO: SC1339 | JULY 2023

N:\CADD\SISO CAL GAS COMPANY\SO CAL GAS COMPANY - SC1339\FIGURES\SC1339F002 - Last Saved by: KViswanathan on 7/17/23



SOURCE: SEISMIC HAZARD ZONE REPORT FOR THE NEWHALL 7.5-MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA. DEPARTMENT OF CONSERVATION, DIVISION OF MINES AND GEOLOGY, 1997.

0 3,000
SCALE IN FEET

LEGEND

● Borehole Site

30 Depth to ground water in feet

HISTORIC HIGH GROUNDWATER MAP
SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

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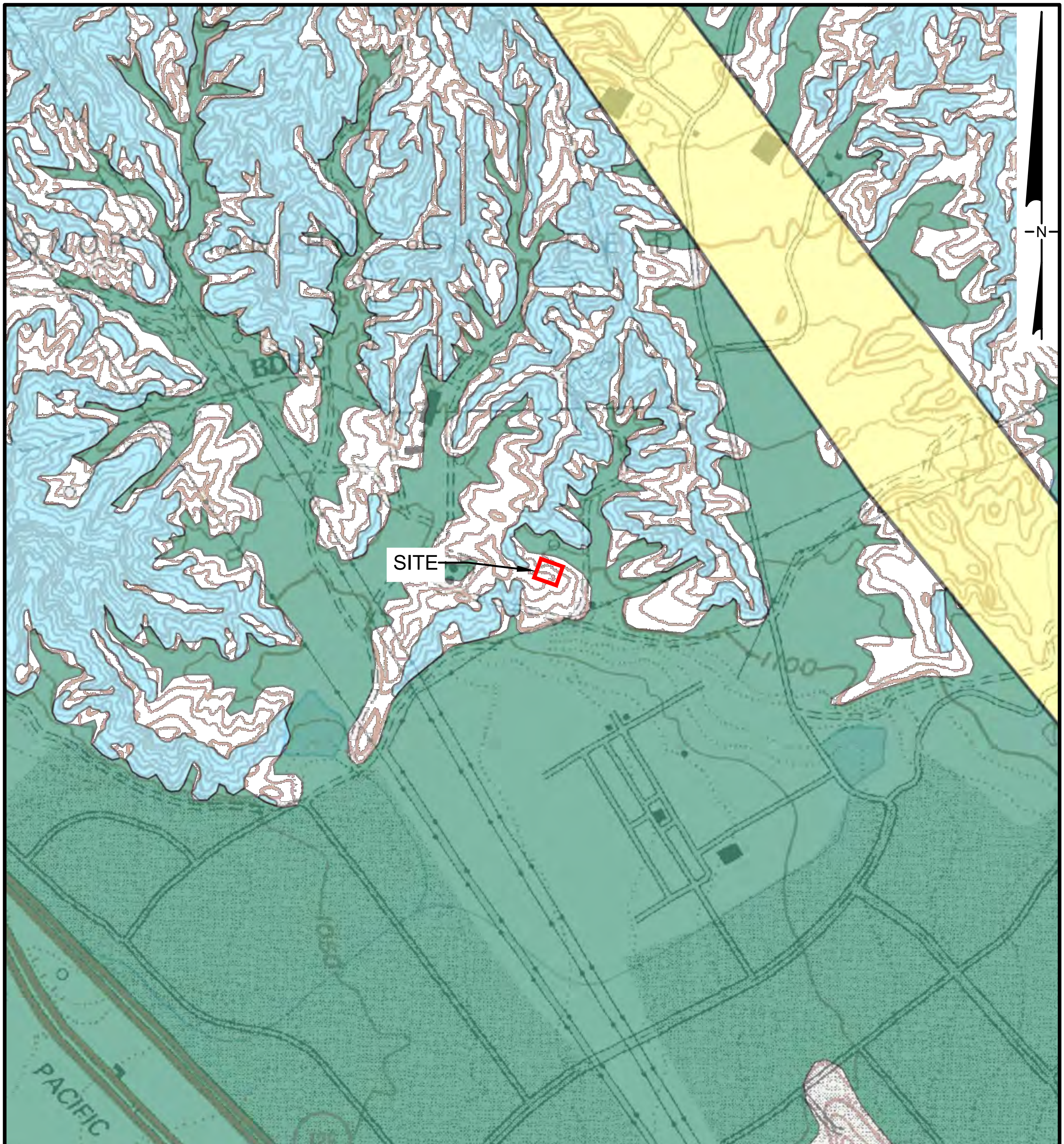
PROJECT NO: SC1339 | JULY 2023

FIGURE 6

N:\CACADD\S\SO CAL GAS COMPANY\SO CAL GAS COMPANY - SC1339\FIGURES\SC1339F005 - Last Saved By: KViswanathan on 7/15/23



SOURCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, NEWHALL QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA. CALIFORNIA GEOLOGICAL SURVEY, 1986.

0 1,000
SCALE IN FEET

LEGEND

EARTHQUAKE FAULT ZONE

LIQUEFACTION HAZARD ZONE

LANDSLIDE HAZARD ZONE

CGS LIQUEFACTION HAZARD MAP
SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

Geosyntec consultants

FIGURE 7

PROJECT NO: SC1339 | JULY 2023

N:\CACADD\S\SO CAL GAS COMPANY\SO CAL GAS COMPANY - SC1339\FIGURES\SC1339F006 - Last Saved By: KViswanathan on 7/15/23

APPENDIX A

Hammer Calibration Certifications

EARTHSPECTIVES
1920 E Warner Avenue, Suite 3-M
Santa Ana, California 92705
Phone: (949) 777-1270
Fax: (949) 777-1283

January 12, 2023

ABC Liovin Drilling Inc.
1180 East Burnett Street
Signal Hill, California 90755
Attention: Mr. Bill Borgo

Dear Bill:

SPT Hammer Energy Measurement
Drill Rigs # R-1, R-3, R-5, and R-9
ES Project No. 230102-365

INTRODUCTION

This letter report summarizes the results of EarthSpectives' (ES) SPT hammer energy measurements performed on January 12, 2023. It provides a description of the test program and the results.

SPT energy measurements were accomplished using a Pile Driving Analyzer (PDA-8G) system manufactured by Pile Dynamics, Inc. and was conducted in general accordance with ASTM 4945 and 6066 test standards. Results are summarized in Table 1, while more details regarding energy records are provided in Appendix A.

TESTING CONDITIONS

SPT hammer energy measurements were performed on drill rigs R-1 (CME85 Serial Number 325136), R-3 (CME75 No Serial Number), R-5 (CME85 Serial Number 276886), and R-9 (CME75 Serial Number 177367). All Testing was performed on Drill Rigs equipped with auto Trip hammers. Samplings were performed using NWJ drilling rod.

INSTRUMENTATION

SPT energy measurements were performed by placing a 2 ft instrumented section of drill rod at the top of the drill string between the hammer and the sampling rods. The instruments consist of two sets of accelerometers and strain transducers, mounted on opposite sides of the drill rod, with a view to evaluate normal and eccentric effects. The analyzer acquired and processed the signals during sampling, and provided real-time evaluations of the maximum SPT hammer transferred energy. The raw data were stored directly on a portable field computer for subsequent analysis in the office.

RESULTS

Results from SPT hammer energy measurements are summarized in Tables 1. It shows the Energy Transfer Ratio (ETR) for every sampling depth for the tested drill rig/hammer. ETR is the ratio of the measured maximum transferred energy to rated energy of the hammer which is the product of the weight of the hammer times the height of fall (140 lb x 30 inches = 4200 lb-in = 0.35 kip-ft).

Plots of the maximum transferred energy, energy transfer ratio, and blow rate is provided as function of depth in Appendix A. Table immediately following the plot also provides the minimum, maximum, and average values at every sampling depth. In general, average ETR value for the tested hammers R-1, R-3, R-5, and R-9 were 81%, 86%, 82%, and 80%, respectively, over all the sampling intervals as shown in Table 1.

TABLE 1 – SUMMARY OF SPT HAMMER ENERGY MEASUREMTS

Drill Rig Number Model, Serial Number	**AVERAGE SPT HAMMER EFFICIENCY (ENERGY TRANSFER RATIO)**				
	Data Set # 1	Data Set # 2	Data Set # 3	Data Set # 4	Average
Drill Rig R-1 CME85, 325136	79.6%	81.5%	81.2%	--	80.8%
Drill Rig R-3 CME75, No SN	86.8%	87.8%	82.3%	--	85.6%
Drill Rig R-5 CME85, 276886	82.0%	83.1%	79.6%	--	81.5%
Drill Rig R-9 CME75, 177367	74.5%	80.3%	81.7%	79.9%	80.1%

LIMITATIONS

Professional judgments represented in this report are based on evaluations of the technical information gathered, our understanding of the proposed construction, and our general experience in the geotechnical field. We do not guarantee the performance of the project in any respect, only that our engineering work and judgments are rendered while striving to meet the standard of care of our profession at this time.

CLOSURE

We hope the above information satisfies the project needs at this time. Please call if you have any question or need more information.

Sincerely submitted for EarthSpectives,

Hossein Rashidi

Hossein K. Rashidi, PhD, PE
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
HOSSEIN K RASHIDI
No. C 055140
Exp. 06–30–2024
CIVIL
STATE OF CALIFORNIA

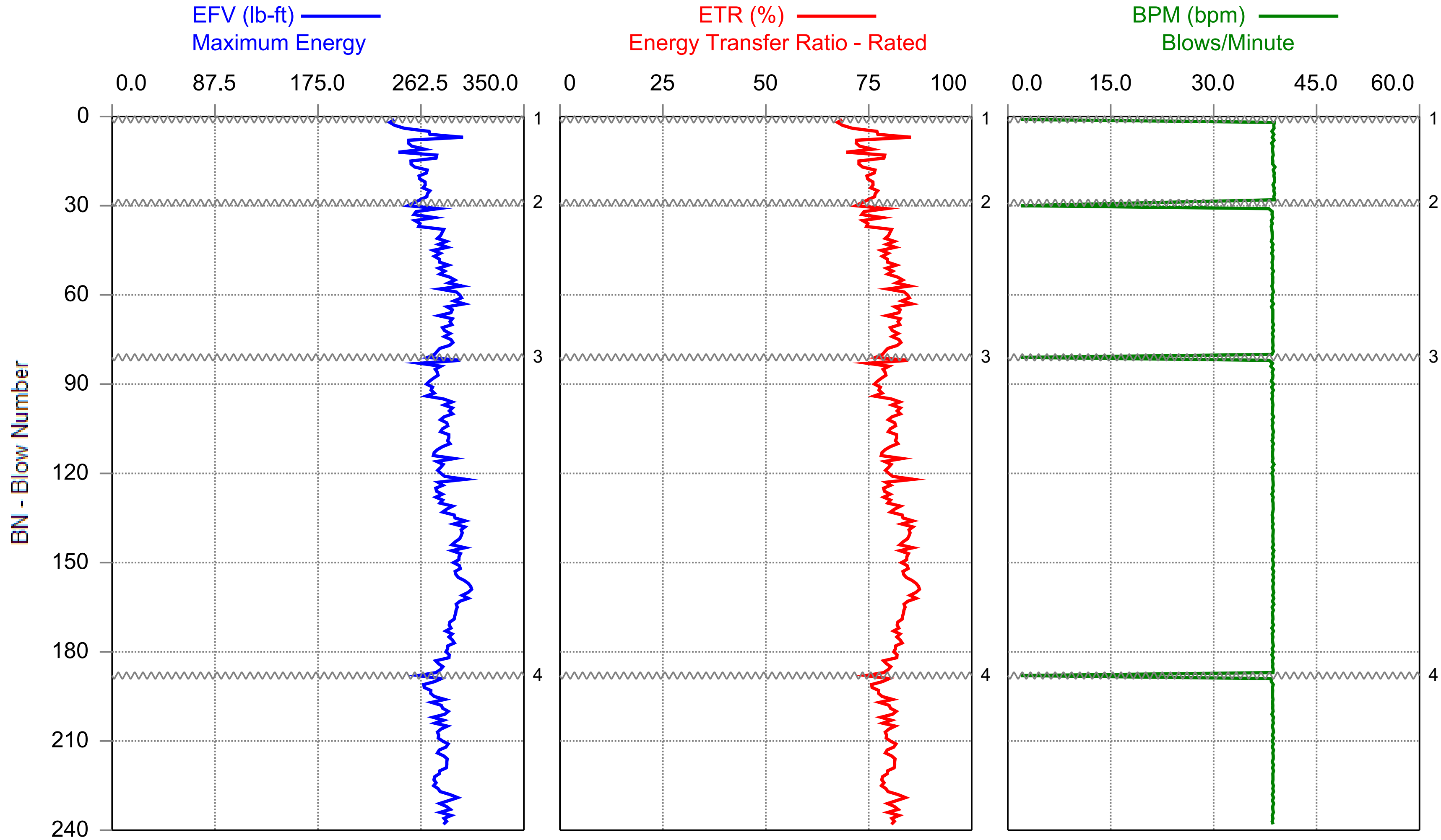
Appendix A
SPT Hammer Energy Data

GRL Engineers, Inc. - PDIPLOT2 Ver 2021.1.61.0 - Case Method & iCAP® Results

Printed: 12-January-2023 Test started: 11-January-2023

ABC Liovin Drilling - Rig R9

Rig: CME75, NWJ Rod, Hammer SN: 177367



1 - Sample #1
2 - Sample #2
3 - Sample #3
4 - Sample #4

Case Method & iCAP® Results

ABC Liovin Drilling - Rig R9 Rig: CME75, NWJ Rod, Hammer SN: 177367
OP: US Date: 11-January-2023

AR:	1.42 in²	SP:	0.492 k/ft³
LE:	13.00 ft	EM:	30,000 ksi
WS:	16,807.9 f/s	JC:	0.90

EFV: Maximum Energy ETR: Energy Transfer Ratio - Rated
BPM: Blows/Minute

BL#	TYPE	EFV lb-ft	BPM bpm	ETR (%)
30	AV29	260.7	36.2	74.5
	MAX	298.4	38.9	85.2
	MIN	236.1	1.9	67.4
81	AV51	281.1	37.8	80.3
	MAX	299.3	38.7	85.5
	MIN	256.6	1.9	73.3
188	AV107	285.8	38.3	81.7
	MAX	305.6	38.8	87.3
	MIN	260.1	1.9	74.3
238	AV50	279.5	38.6	79.9
	MAX	293.7	38.7	83.9
	MIN	264.9	38.4	75.7
Average		280.4	38.0	80.1
Maximum		305.6	38.9	87.3
Minimum		236.1	1.9	67.4

Total number of blows analyzed: 237

BL#	Sensors
1-238	F3: [NWJ 1] 216.0 (1.00); F4: [NWJ 2] 217.1 (1.00); A1: [K3611] 370.4 (1.00); A2: [K3734] 367.7 (1.00)

Time Summary

Drive	43 seconds	10:24 AM - 10:25 AM (1/11/2023) BN 1 - 29
Stop	22 minutes 31 seconds	10:25 AM - 10:47 AM
Drive	1 minute 17 seconds	10:47 AM - 10:49 AM BN 30 - 80
Stop	5 minutes 56 seconds	10:49 AM - 10:55 AM
Drive	2 minutes 44 seconds	10:55 AM - 10:57 AM BN 81 - 187
Stop	5 minutes 11 seconds	10:57 AM - 11:03 AM
Drive	1 minute 17 seconds	11:03 AM - 11:04 AM BN 188 - 238

Total time [00:39:43] = (Driving [00:06:03] + Stop [00:33:40])

APPENDIX B

Exploratory Boring Logs

Geosyntec consultants
3530 Hyland Ave
Suite 100
Costa Mesa, CA 92626
Tel: (714) 969-0800
Fax: (714) 969-0820

PROJECT Substation Option A, Honor Rancho Facility
PROJECT LOCATION Santa Clarita, California
PROJECT NUMBER SC1339

KEY SHEET - CLASSIFICATIONS AND SYMBOLS

GS FORM: KEY/SYMBOLS 01/04

EMPIRICAL CORRELATIONS WITH STANDARD PENETRATION RESISTANCE N60 VALUES *

	N60 VALUE * (BLOWS/FT)	CONSISTENCY	UNCONFINED COMPRESSIVE STRENGTH (TONS/SQ FT)
FINE GRAINED SOILS	0 - 2	VERY SOFT	<0.25
	3 - 4	SOFT	0.25 - 0.50
	5 - 8	FIRM	0.50 - 1.00
	9 - 15	STIFF	1.00 - 2.00
	16 - 30	VERY STIFF	2.00 - 4.00
	31 - 50	HARD	>4.00
	>50	VERY HARD	

	N60 VALUE * (BLOWS/FT)	RELATIVE DENSITY
COARSE GRAINED SOILS	0 - 4	VERY LOOSE
	5 - 10	LOOSE
	11 - 30	MEDIUM DENSE
	31 - 50	DENSE
	>50	VERY DENSE

* ASTM D 1586; NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 IN. O.D., 1.4 IN. I.D. SAMPLER ONE FOOT, CORRECTED FOR HAMMER EFFICIENCY.

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

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

NOTE: DUAL SYMBOLS USED FOR BORDERLINE CLASSIFICATIONS

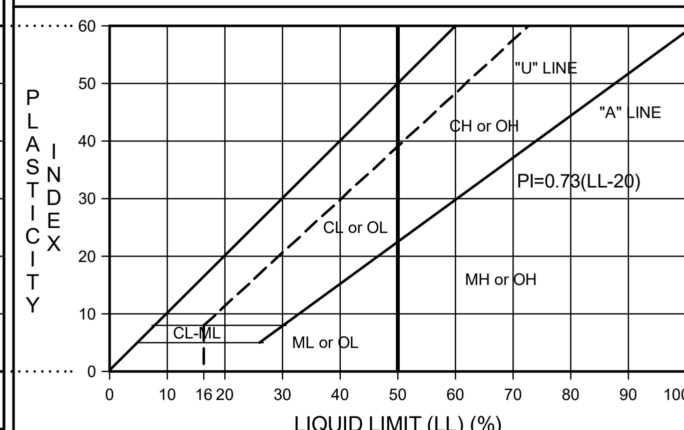
PARTICLE SIZE IDENTIFICATION

USCS (SOILS ONLY) *		SEDIMENTARY (ROCK ONLY)	
BOULDER	>300 mm	BOULDER	>256 mm
COBBLE	75 - 300 mm	COBBLE	64 - 256 mm
GRAVEL: COARSE	20 - 75 mm	PEBBLE	4 - 64 mm
GRAVEL: FINE	4.75 - 20 mm	GRANULE	2 - 4 mm
		SAND: V. COARSE	1 - 2 mm
SAND: COARSE	2 - 4.75 mm	SAND: COARSE	0.5 - 1 mm
SAND: MEDIUM	0.42 - 2 mm	SAND: MEDIUM	0.25 - 0.5 mm
SAND: FINE	0.074 - 0.42 mm	SAND: FINE	0.125 - 0.25 mm
		SAND: V. FINE	0.063 - 0.125 mm
SILT/CLAY	<0.074 mm	SILT	0.004 - 0.063 mm
		CLAY	<0.004 mm

* WELL GRADED - HAVING WIDE RANGE OF GRAIN SIZES AND APPRECIABLE AMOUNTS OF ALL INTERMEDIATE PARTICLE SIZES
* POORLY GRADED - PREDOMINANTLY ONE GRAIN SIZE, OR HAVING A RANGE OF SIZES WITH SOME INTERMEDIATE SIZES MISSING

PERCENTAGE OF PARTICLE TYPE IN DECREASING ORDER OF PARTICLE SIZE (GRAVEL,SAND,FINES), BASED ON VISUAL OBSERVATION

PLASTICITY CHART



OTHER MATERIAL SYMBOLS

- Conglomerate
- Sandstone
- Silty Sandstone
- Clayey Sandstone
- Sandy Siltstone
- Siltstone
- Claystone
- Clayey Siltstone/ Silty Claystone
- Sandy Claystone
- Granitic/Intrusive
- Volcanic/Extrusive
- Metamorphic
- Limestone
- Dolomite
- Glacial Till
- Landslide Debris
- Marker Bed
- Artificial Fill
- Refuse
- Concrete/Asphalt

WELL SYMBOLS

- CONCRETE
- GROUT
- BENTONITE SEAL
- TRANSITION SAND
- SAND PACK
- GRAVEL PACK
- NATIVE/SLUFF
- CENTRALIZER

SAMPLE TYPE AND OTHER SYMBOLS

- BULK SAMPLE
- STANDARD PENETRATION TEST
- MODIFIED CALIFORNIA SAMPLE
- CORE SAMPLE
- SHELBY TUBE
- DRIVE SAMPLE

- Water Level at Time Drilling, or as Shown
- Static Water Level
- Pump Inlet
- Loss of Drilling Fluid
- MSL: Mean Sea Level
- AGS: Above Ground Surface
- BGS: Below Ground Surface
- BTOC: Below Top of Casing
- HSA: Hollow Stem Auger

Geosyntec consultants

3530 Hyland Ave
Suite 100
Costa Mesa, CA 92626
Tel: (714) 969-0800
Fax: (714) 969-0820

GS FORM: GEOTECH2 01/04

BOREHOLE LOG

BORING HSA-3 **SHEET** 1 **OF** 1
START DRILL DATE 6/9/2023
FINISH DRILL DATE 6/9/2023
LOCATION Santa Clarita, California
PROJECT Substation Option A, Honor Rancho Facility
NUMBER SC1339

ELEVATION DATA:
GROUND SURF. 1103
DATUM WGS 1984

DESCRIPTION: 1) Soil Name (USCS) 2) Color 3) Moisture 4) Grain Size 5) Percentage [Gravel,Sand,Fines] 6) Density/Consistency 7) Plasticity 8) Other (Mineral Content, Discoloration, Odor, etc.)

COMMENTS: 1) Rig Behavior 2) Air Monitoring 3) Pocket Pen 4) Tor Vane

DEPTH (ft-bgs)	ELEVATION (ft)	DESCRIPTION	SAMPLE NO.	BLOWS PER 6"	N VALUE	RECOVERY (%)	PID READING (ppm)	TIME (00:00)	COMMENTS	DRY DENSITY (pcf)	MAX. DRY DENSITY (pcf)	PERCENT FINES (%)	PERCENT GRAVEL (%)	MOIST. CONTENT (%)	OPT. MOIST. CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0–1	1103–1102	**Saugus Formation (Qss):** Silty SANDSTONE (SM): tan; moist; fine to medium grained sand; [10,60,30]; low plasticity fines.	B-1						Hand-augered to 3 ft bgs. Difficulty augering around 2', had to use tunnel bar. Switched to HSA drilling at 3 ft.									
5	1098		S-1	48 50/3"		100	30.6			101.7				9.1				
6	1097	- clay pockets																
7.5	1095.5		S-2	8 50/5"		100	4.2											
9.5	1093.5	Clayey SANDSTONE (SC): brown; moist; fine-grained sand; [5,45,50]; low plasticity fines.	S-3	31 50/3"		100	15.0			109				7.7				
12	1091	CLAYSTONE with sand (CL): brown; moist; fine grained sand; [5,15,80]; low to medium plasticity.	S-4	19 50/4"		100												
15	1088		S-5	50/4"		100	21.9			105.7				8.2				
20	1083		S-6	34 50/2"		100	3.9					82.1						
23.5	1079.5	Sandy SILTSTONE (ML): brown; dry; fine-grained sand; [5,30,65]; low plasticity.	S-7	33 50/2"		100	8.7			110.1				6.6				
28.5	1074.5	CLAYSTONE with sand (CL): reddish brown; moist; [0,25,75]; medium plasticity.	S-8	16 38 50/3"		100	2.9											
33.5	1069.5	Clayey SANDSTONE (SC): gray; moist; fine-grained sand; [5,60,35]; low plasticity fines.	S-9	40 50/5"		100	18.8			117.4				9.1				
40	1063	becomes reddish brown; [0,75,25]; medium plasticity fines.	S-10	9 23 50/5"		100	3.4											
41.5	1061.5	Boring terminated at 41.5 ft bgs. After completion of drilling, borehole was backfilled with cement-bentonite grout with bentonite chips in the upper 6 inches.																

CONTRACTOR ABC Liovin Drilling
EQUIPMENT CME-75
DRILL MTHD Hollow Stem Auger
DIAMETER 7-inch
LOGGER R. Khan
REVIEWER B.Baturay, PhD, PE, GE

LAT.: 34.44400
LONG.: -118.58359
COORDINATE SYSTEM:

NOTES: No groundwater encountered. Hammer energy transfer ratio was 80%. Ground surface elevation is approximate and obtained from Google Earth.

SEE KEY SHEET FOR SYMBOLS AND ABBREVIATIONS

Geosyntec consultants
3530 Hyland Ave
Suite 100
Costa Mesa, CA 92626
Tel: (714) 969-0800
Fax: (714) 969-0820

GS FORM: GEOTECH2 01/04

BOREHOLE LOG

BORING HSA-4 **SHEET** 1 **OF** 1
START DRILL DATE 6/9/2023
FINISH DRILL DATE 6/9/2023
ELEVATION DATA: **GROUND SURF.** 1124 **DATUM** WGS 1984
LOCATION Santa Clarita, California
PROJECT Substation Option A, Honor Rancho Facility
NUMBER SC1339

DESCRIPTION: 1) Soil Name (USCS) 2) Color 3) Moisture 4) Grain Size 5) Percentage [Gravel,Sand,Fines] 6) Density/Consistency 7) Plasticity 8) Other (Mineral Content, Discoloration, Odor, etc.)

COMMENTS: 1) Rig Behavior 2) Air Monitoring 3) Pocket Pen 4) Tor Vane

DEPTH (ft-bgs)	ELEVATION (ft)	DESCRIPTION	SAMPLE NO.	BLOWS PER 6"	N VALUE	RECOVERY (%)	PID READING (ppm)	TIME (00:00)	COMMENTS	DRY DENSITY (pcf)	MAX. DRY DENSITY (pcf)	PERCENT FINES (%)	PERCENT GRAVEL (%)	MOIST. CONTENT (%)	OPT. MOIST. CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0–5	1124–1119	**Fill:** Silty SAND (SM): brown; moist; fine-grained sand; [15,60,25]; low plasticity fines; presence of clay pockets; up to 1" sub-rounded gravel.	B-1				0.0		Hand-augered to 5 ft bgs. The soils identified as fill on this log generally exhibited lumps of clay of dissimilar color within the sand matrix.									
5–9.5	1119–1114.5	Clayey SAND (SC): brown, moist; fine-grained sand; [10,60,30]; low plasticity fines, presence of clay pockets.	S-1	6, 7, 5	12	100	0.2											
			S-2	12, 12, 13		67	20.4			120.1				10.3				
9.5–11.5	1114.5–1112.5	Sandy Lean CLAY (CL): reddish brown; moist; fine-grained sand; [5,40,55]; low plasticity.	S-3	4, 4, 7	11	100	1.2					54.1						
11.5–15	1112.5–1109	**Quaternary Young Alluvium (Qya):** Sandy Lean CLAY (CL): reddish brown; moist; fine-grained sand; [5,30,65]; medium plasticity.	S-4	8, 15, 18		55	21.4											
15–19	1109–1105	becomes gray; [10,30,60]; presence of brown silt seams.	S-5	5, 4, 4	8	100	0.9									30	16	14
19–23.5	1105–1100.5	Silty SAND (SM): brown; moist; fine-grained sand; [10,60,30]; low plasticity fines;	S-6	6, 5, 7		100	45.6			97.5				6.9				
23.5–28.5	1100.5–1095.5	**Saugus Formation (Qss):** Clayey SANDSTONE (SC): reddish brown; dry; fine-grained sand; [5,60,35]; low plasticity fines.	S-7	18, 33, 50/5"		100	18.4											
28.5–31.5	1095.5–1092.5	Sandy CLAYSTONE (CL): reddish brown; dry; fine-grained sand; [5,30,65]; low plasticity.	S-8	8, 37, 50/4"		88	53.4			110.5				9.1				
		SILTSTONE (ML): brown; dry; [5,15,80]; low plasticity.																
		Boring terminated at 31.5 ft bgs. After completion of drilling, borehole was backfilled with cement-bentonite grout with bentonite chips in the upper 6 inches.																

CONTRACTOR ABC Liovin Drilling
EQUIPMENT CME-75
DRILL MTHD Hollow Stem Auger
DIAMETER 7-inch
LOGGER R. Khan
REVIEWER B.Baturay, PhD, PE, GE
LAT.: 34.44450
LONG.: -118.58366
COORDINATE SYSTEM:

NOTES: No groundwater encountered. Hammer energy transfer ratio was 80%. Ground surface elevation is approximate and obtained from Google Earth.

SEE KEY SHEET FOR SYMBOLS AND ABBREVIATIONS

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Costa Mesa, CA 92626
Tel: (714) 969-0800
Fax: (714) 969-0820

GS FORM: GEOTECH2 01/04

BOREHOLE LOG

BORING HSA-5 **SHEET** 1 **OF** 1
START DRILL DATE 6/9/2023
FINISH DRILL DATE 6/9/2023
LOCATION Santa Clarita, California
PROJECT Substation Option A, Honor Rancho Facility
NUMBER SC1339
ELEVATION DATA:
GROUND SURF. 1123
DATUM WGS 1984

DESCRIPTION: 1) Soil Name (USCS) 2) Color 3) Moisture 4) Grain Size 5) Percentage [Gravel,Sand,Fines] 6) Density/Consistency 7) Plasticity 8) Other (Mineral Content, Discoloration, Odor, etc.)

COMMENTS: 1) Rig Behavior 2) Air Monitoring 3) Pocket Pen 4) Tor Vane

DEPTH (ft-bgs)	ELEVATION (ft)	DESCRIPTION	SAMPLE NO.	BLOWS PER 6"	N VALUE	RECOVERY (%)	PID READING (ppm)	TIME (00:00)	COMMENTS	DRY DENSITY (pcf)	MAX. DRY DENSITY (pcf)	PERCENT FINES (%)	PERCENT GRAVEL (%)	MOIST. CONTENT (%)	OPT. MOIST. CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
0–1	1122–1121	**Fill:** Silty SAND (SM): tan; moist; fine-grained sand; [10,60,30]; low plasticity fines; presence of clay pockets; up to 1" sub-angular gravel.	B-1				0.0		Hand-augered to 5 ft bgs. The soils identified as fill on this log generally exhibited lumps of clay of dissimilar color within the sand matrix.									
5	1118		S-1	5, 3, 3		83	4.8			103.4				5.7				
7.5	1115	becomes brown	S-2	3, 3, 3	6	78	2.8					29.9						
10	1113		S-3	8, 7, 7		56	15.9			111.3				3.8				
12	1111	**Quaternary Young Alluvium (Qya):** Silty SAND (SM): tan; moist; fine-grained sand; [10,60,30]; low plasticity fines; presence of clay pockets; up to 1" sub-angular gravel.	S-4	4, 3, 3	6	67	3.2											
15	1108		S-5	3, 9, 9		78	21.6			104.3				6.0				
18	1105	**Saugus Formation (Qss):** Sandy CLAYSTONE (CL): light gray; moist; [10,40,50]; low to medium plasticity.	S-6	48, 26, 34	60	100	1.8											
23	1100	Silty SANDSTONE (SM): light gray and tan; moist; fine-grained sand; [10,60,30]; low plasticity fines.	S-7	14, 50/2"		100	134.5			104.6				7.0				
30	1093	becomes tan; medium grained sand; [5,70,25].	S-8	16, 50/3"		100	4.9											
31.5	1091	Boring terminated at 31.5 ft. bgs. After completion of drilling, borehole was backfilled with cement-bentonite grout with bentonite chips in the upper 6 inches.																

CONTRACTOR ABC Liovin Drilling
EQUIPMENT CME-75
DRILL MTHD Hollow Stem Auger
DIAMETER 7-inch
LOGGER R. Khan
REVIEWER B.Baturay, PhD, PE, GE
LAT.: 34.44483
LONG.: -118.58378
COORDINATE SYSTEM:

NOTES: No groundwater encountered. Hammer energy transfer ratio was 80%. Ground surface elevation is approximate and obtained from Google Earth.

SEE KEY SHEET FOR SYMBOLS AND ABBREVIATIONS

APPENDIX C

Laboratory Test Results

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: Geosyntec Consultants, Inc.
AP Lab No.: 23-0645
Project Name: Honor Rancho Substation Relocation
Test Date: 06/23/23
Project No.: SC1339

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
HSA-3	S-1	6-6.5	9.1	101.7
HSA-3	S-5	16-16.5	8.2	105.7
HSA-3	S-9	36-36.5	9.1	117.4

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: Geosyntec Consultants, Inc.
AP Lab No.: 23-0645
Project Name: Honor Rancho Substation Relocation
Test Date: 06/23/23
Project No.: SC1339

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
HSA-4	S-6	21-21.5	6.9	97.5
HSA-4	S-8	31-31.5	9.1	110.5

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MOISTURE AND DENSITY TEST RESULTS

ASTM D2216 and ASTM D7263 (Method B)

Client: Geosyntec Consultants, Inc.
Project Name: Honor Rancho Substation Relocation
Project No.: SC1339

AP Lab No.: 23-0645
Test Date: 06/23/23

Boring No.	Sample No.	Sample Depth (ft.)	Moisture Content (%)	Dry Density (pcf)
HSA-5	S-3	11-11.5	3.8	111.3
HSA-5	S-5	16-16.5	6.0	104.3
HSA-5	S-7	26-26.5	7.0	104.6

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PERCENT PASSING NO. 200 SIEVE

ASTM D1140

Client:	Geosyntec Consultants, Inc.	AP Lab No.:	23-0645
Project Name:	Honor Rancho Substation Relocation	Test Date:	06/23/23
Project Number:	SC1339		

Boring No.	Sample No.	Depth (ft)	Percent Fines (%)
HSA-3	S-6	20-21.5	82.1
HSA-4	S-3	10-11.5	54.1
HSA-5	S-2	7.5-9	29.9

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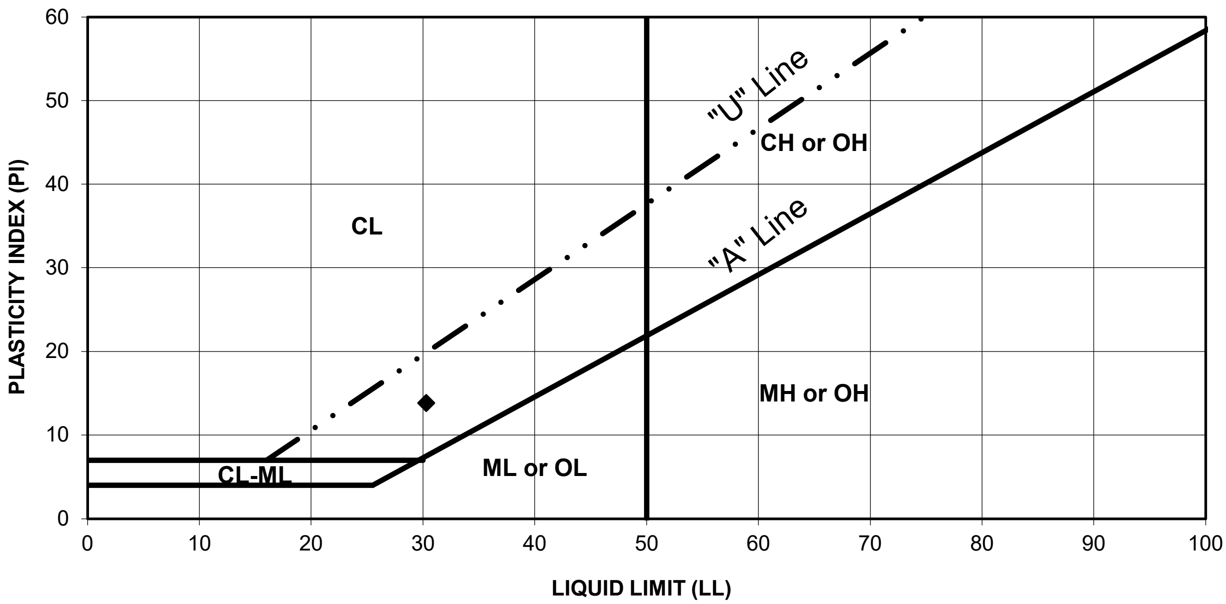
ATTERBERG LIMITS
ASTM D 4318

Client Name: Geosyntec Consultants, Inc. Tested By: LS Date: 06/26/23
Project Name: Honor Rancho Substation Relocation Computed By: NR Date: 06/27/23
Project No.: SC1339 Checked By: AP Date: 06/27/23



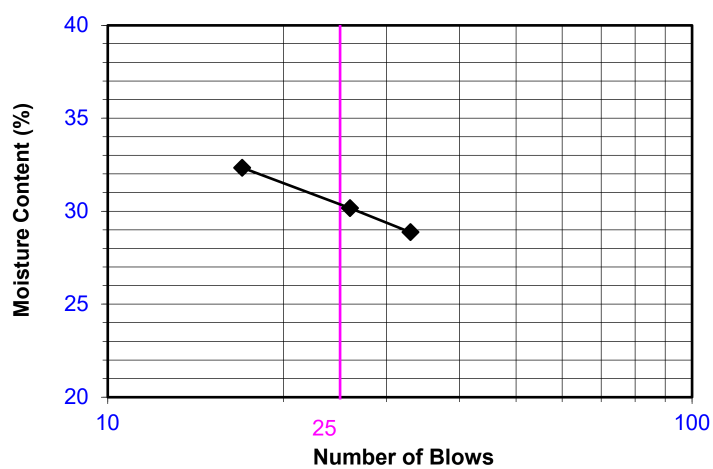
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	HSA-4	S-5	15-16.5	30	16	14	CL

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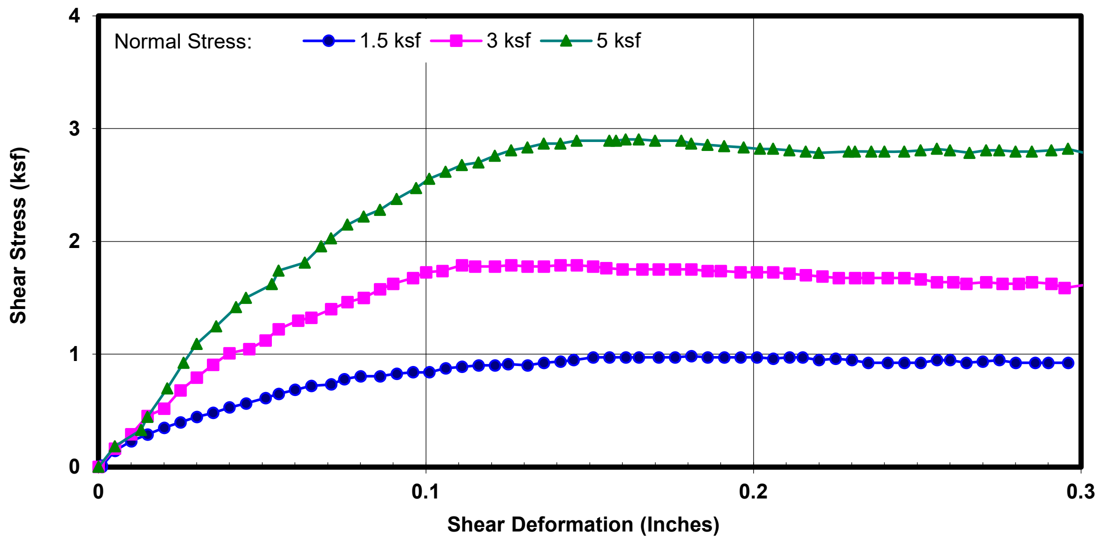
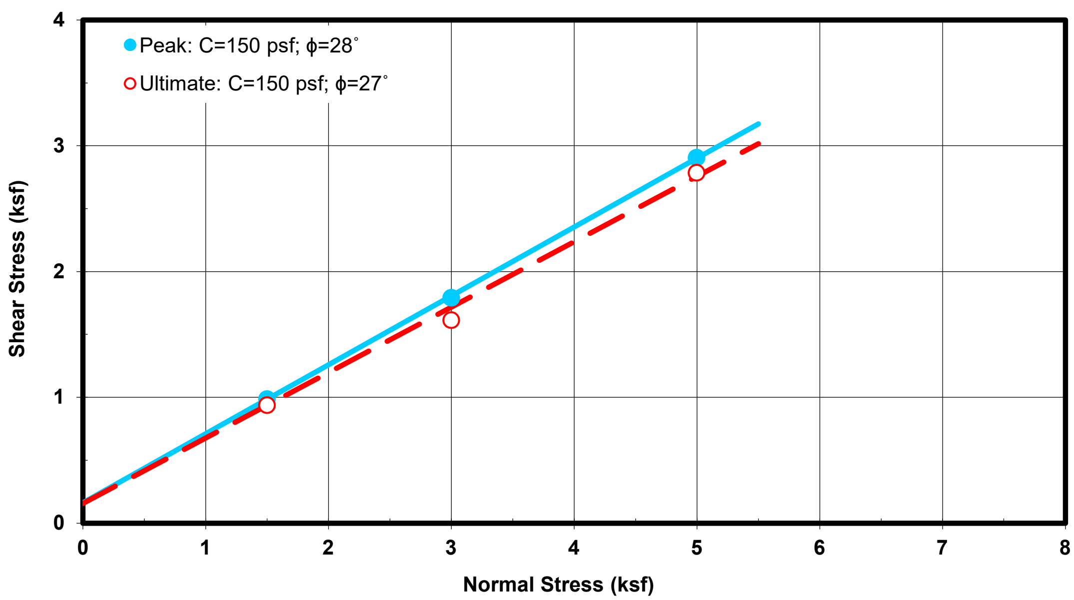
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Honor Rancho Substation Relocation	**Tested By:**	AP	**Date:**	06/23/23
Project No.:	SC1339	**Computed By:**	NR	**Date:**	06/27/23
Boring No.:	HSA-3	**Checked by:**	AP	**Date:**	06/27/23
Sample No.:	S-3	**Depth (ft):**	11-11.5		
Sample Type:	Mod. Cal.				
Soil Description:	Silty Sand				
Test Condition:	Inundated	**Shear Type:**	Regular		

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
117.4	109.0	7.7	18.3	38	91	1.5	0.984	0.936
						3	1.789	1.612
						5	2.904	2.784





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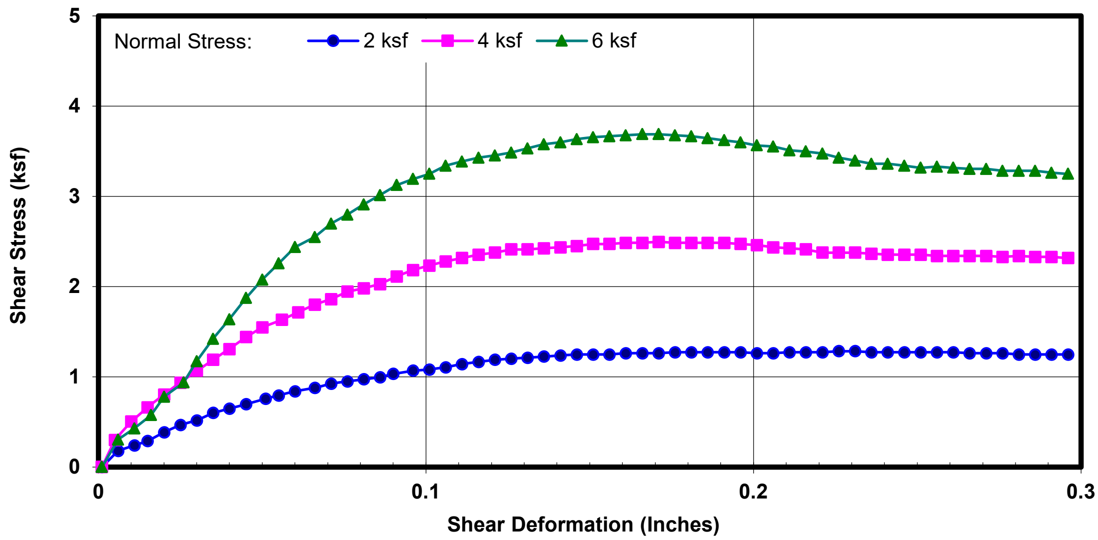
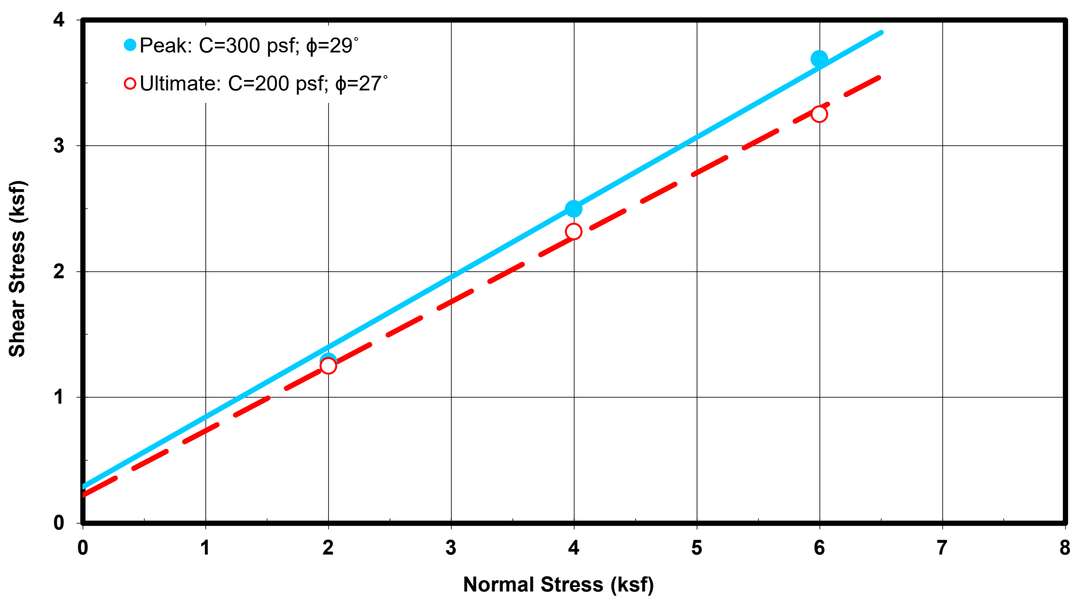
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Honor Rancho Substation Relocation	**Tested By:**	AP	**Date:**	06/23/23
Project No.:	SC1339	**Computed By:**	NR	**Date:**	06/27/23
Boring No.:	HSA-3	**Checked by:**	AP	**Date:**	06/27/23

Sample No.: S-7 **Depth (ft):** 26-26.5
Sample Type: Mod. Cal.
Soil Description: Clayey Sand
Test Condition: Inundated **Shear Type:** Regular

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
117.3	110.1	6.6	18.0	33	92	2	1.284	1.248
						4	2.496	2.316
						6	3.688	3.248





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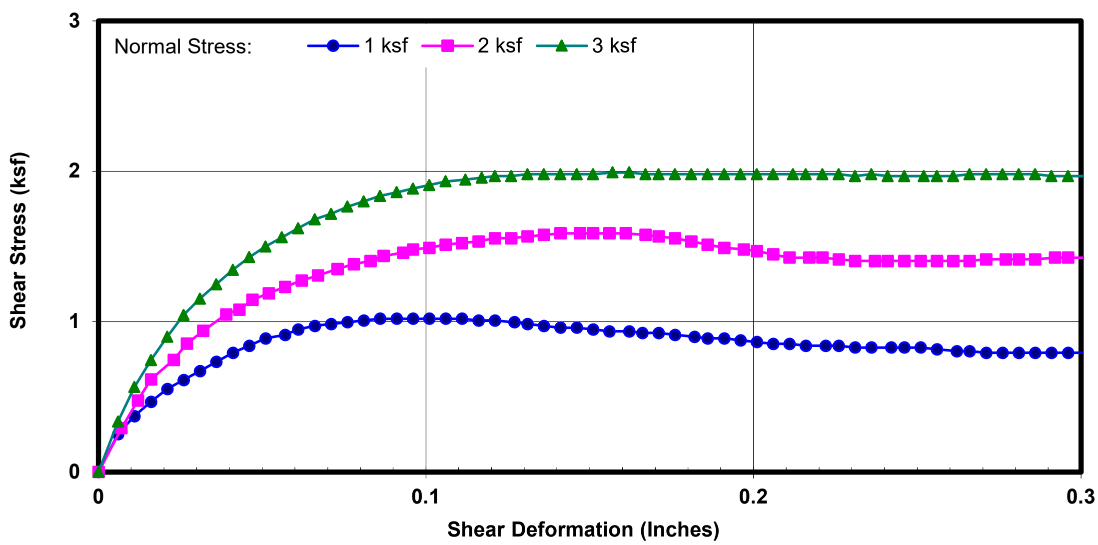
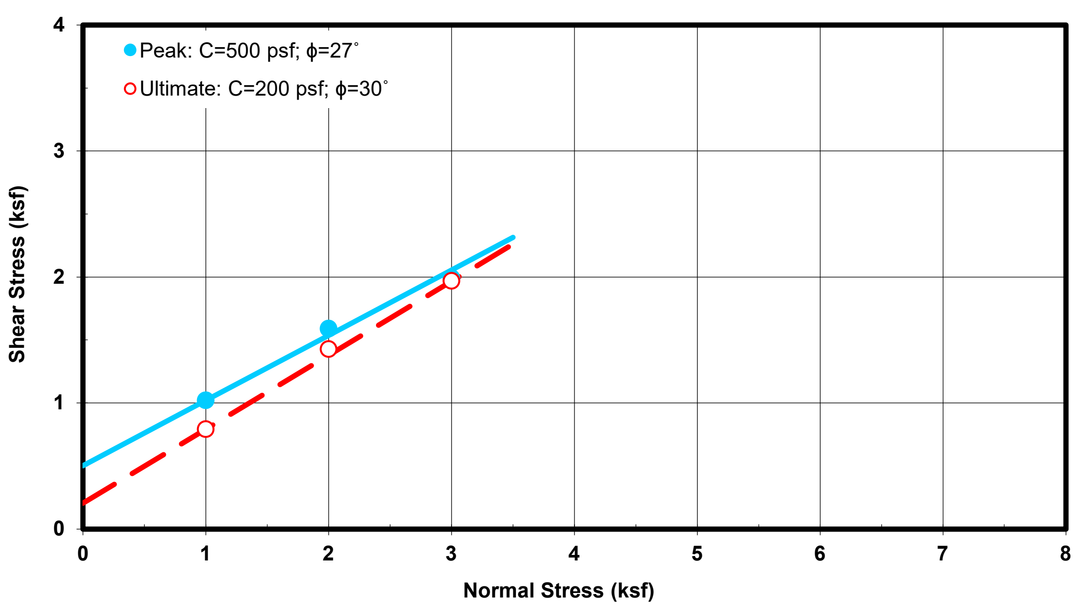
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Honor Rancho Substation Relocation	**Tested By:**	AP	**Date:**	06/23/23
Project No.:	SC1339	**Computed By:**	NR	**Date:**	06/27/23
Boring No.:	HSA-4	**Checked by:**	AP	**Date:**	06/27/23
Sample No.:	S-2	**Depth (ft):**	8.5-9		
Sample Type:	Mod. Cal.				
Soil Description:	Sandy Clay w/ traces of gravel				
Test Condition:	Inundated	**Shear Type:**	Regular		

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
132.5	120.1	10.3	14.1	69	94	1	1.020	0.792
						2	1.588	1.426
						3	1.992	1.968





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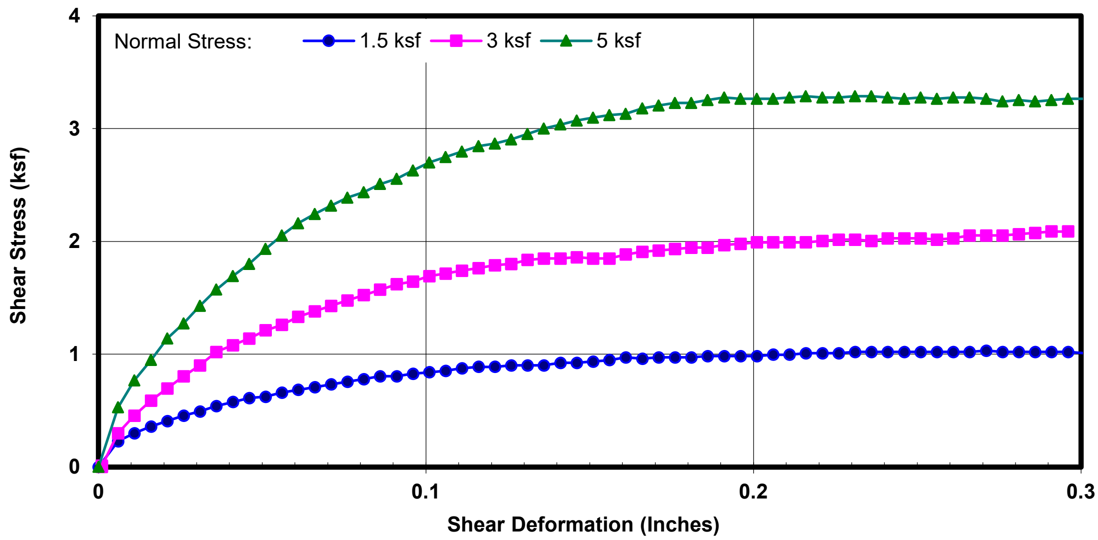
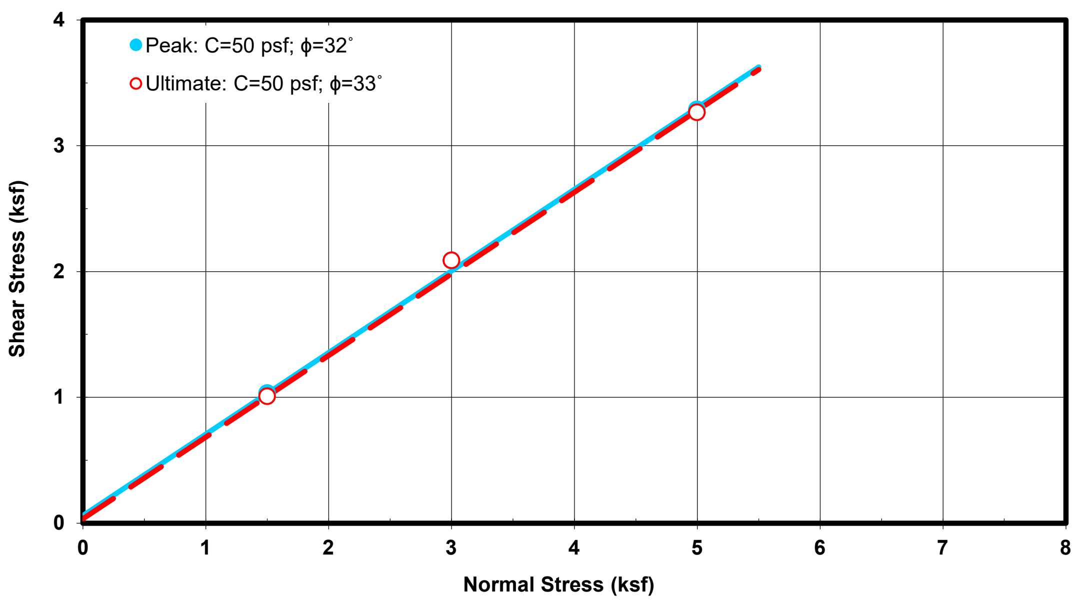
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Honor Rancho Substation Relocation	**Tested By:**	AP	**Date:**	06/23/23
Project No.:	SC1339	**Computed By:**	NR	**Date:**	06/27/23
Boring No.:	HSA-5	**Checked by:**	AP	**Date:**	06/27/23
Sample No.:	S-1	**Depth (ft):**	6-6.5		
Sample Type:	Mod. Cal.				
Soil Description:	Silty Sand				
Test Condition:	Inundated	**Shear Type:**	Regular		

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
109.3	103.4	5.7	20.9	24	90	1.5	1.032	1.008
						3	2.088	2.088
						5	3.288	3.264





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CORROSION TEST RESULTS

Client Name: Geosyntec Consultants, Inc.
Project Name: Honor Rancho Substation Relocation
Project No.: SC1339

AP Job No.: 23-0645
Date: 06/26/23

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
HSA-3	B-1	0-5	Silty Sand w/ gravel	6,641	9.9	17	17

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

APPENDIX D

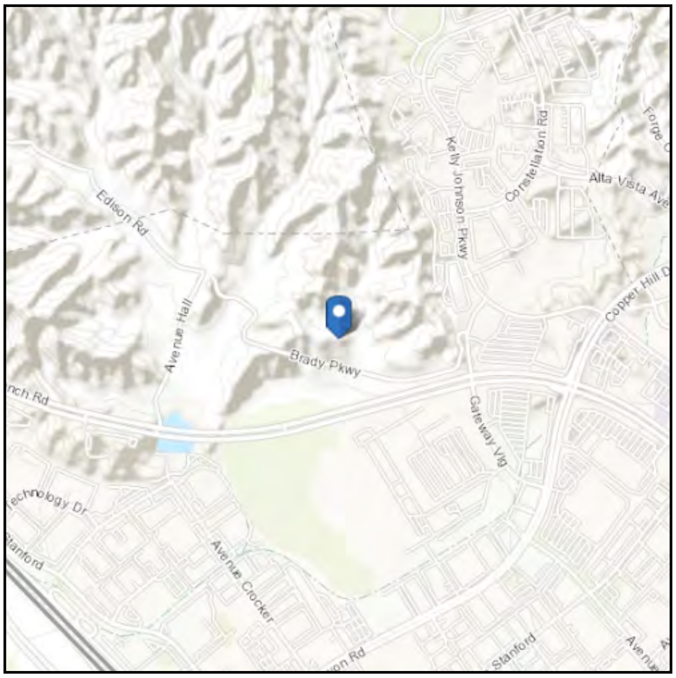
ASCE 7 Online Design Maps Tool Output

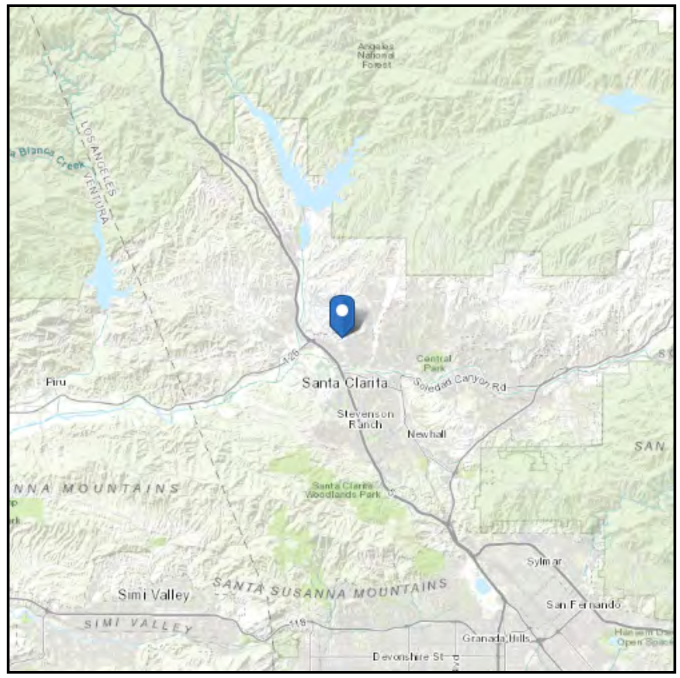
ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-16
Risk Category: IV
Soil Class: C - Very Dense Soil and Soft Rock

Latitude: 34.444208
Longitude: -118.583851
Elevation: 0 ft (NAVD 88)





Seismic

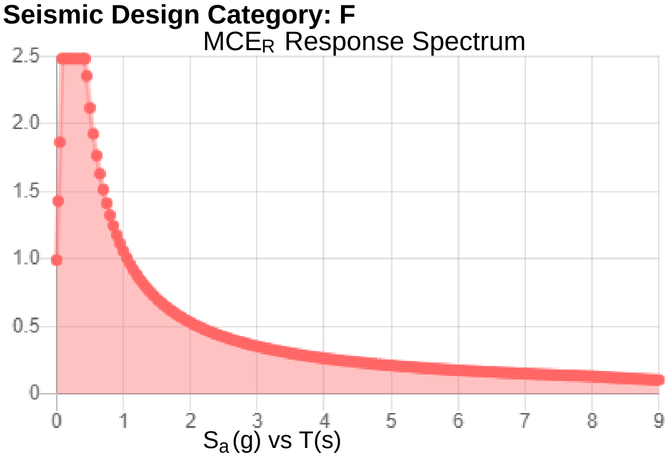
C - Very Dense Soil and Soft Rock

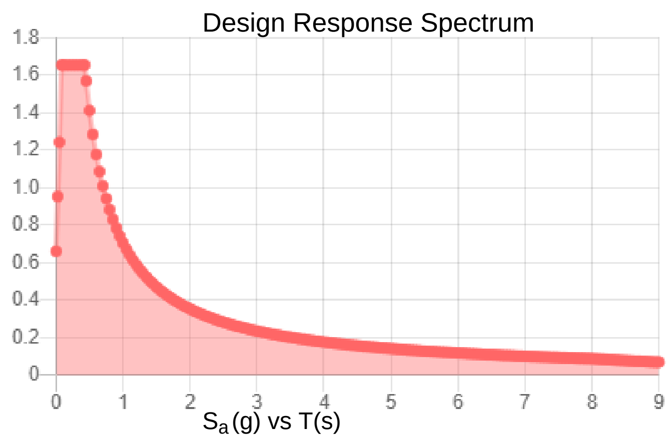
Site Soil Class:

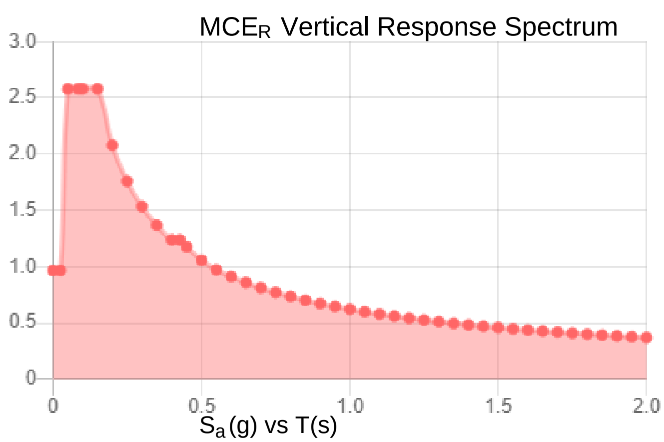
Results:

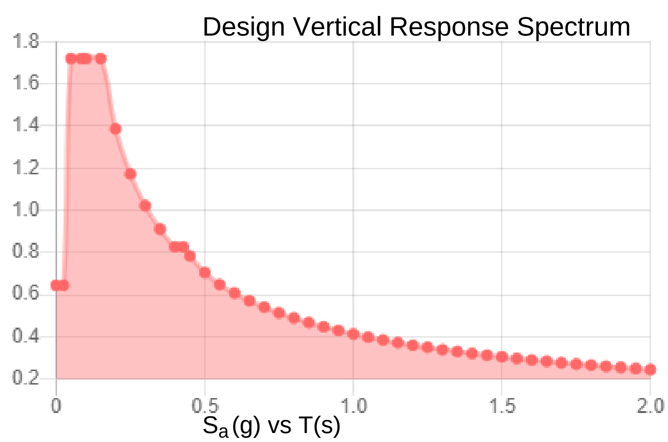
S_S :	2.066	S_{D1} :	0.705
S_1 :	0.756	T_L :	8
F_a :	1.2	PGA :	0.873
F_v :	1.4	PGA_M :	1.048
S_{MS} :	2.48	F_{PGA} :	1.2
S_{M1} :	1.058	I_e :	1.5
S_{DS} :	1.653	C_v :	1.3

Seismic Design Category: F









Data Accessed: **Wed Jul 12 2023**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE 7 standard.

In using this Tool, you expressly assume all risks associated with your use. Under no circumstances shall ASCE or its officers, directors, employees, members, affiliates, or agents be liable to you or any other person for any direct, indirect, special, incidental, or consequential damages arising from or related to your use of, or reliance on, the Tool or any information obtained therein. To the fullest extent permitted by law, you agree to release and hold harmless ASCE from any and all liability of any nature arising out of or resulting from any use of data provided by the ASCE 7 Hazard Tool.

APPENDIX E

Evaluation of Hoek-Brown Strength Parameters for Saugus Formation

Written by:	**O. Doygun**	Date:	**06/16/2021**	Reviewed by:	**Dennis Kilian**	Date:	**06/16/2021**
Client:	**SoCal Gas**	Project:	**Honor Rancho Compressor**	Project No.	**SC0766U**	Task No.:	**6**

EVALUATION OF MOHR COULOMB PARAMETERS FOR SAUGUS FORMATION

1. PURPOSE AND SCOPE

The purpose of this calculation package is to develop equivalent Mohr-Coulomb shear strength parameters (i.e., friction angle and cohesion) for the Saugus Formation at the Site. Hoek-Brown failure criterion (1980, 2002) was used along with the unconfined compression strength test results obtained from intact rock samples at the Site, observation of the continuous rock cores, and the site visit observations of exposed cut slopes.

2. BACKGROUND

The original Hoek-Brown failure criterion was developed in 1980 in the form of a dimensionless equation that could be scaled in relation to geological information and geological observations. In 2002, the entire Hoek-Brown criterion was re-examined and the relationships between the Mohr-Coulomb and the Hoek-Brown criteria were examined for slopes and a set of equations linking the two were presented (Hoek et al. 2002). The final relationships were derived by comparing hundreds of tunnel and slope stability analyses in which both the Hoek-Brown and the Mohr Coulomb criteria were used, and the best match was found by iteration. In the following, first the implemented equations and assumptions are introduced, which is followed by the presentation of the resulting equivalent Mohr Coulomb parameters (friction angle and cohesion)..

3. INPUT PARAMETERS

3.1 Rock Type

Based on the explorations, including continuous rock coring in multiple locations, as well as geologic mapping, Plio-Pleistocene age Saugus Formation underlies the slope area. The Saugus Formation encountered during the explorations generally consists predominantly of silty and clayey sandstones with gravels and cobbles with interbedded red and light brown sandy claystone. The approximate range observed in the unit was estimated at 60% sandstone to 40% claystone. The Saugus Formation was observed to be moderately to highly weathered and occasionally friable in the absence of fines. Topsoils, residual soils, and slopewash encountered overlying the Saugus formation are generally unconsolidated and remediated through grading and therefore, not considered for use in this analysis. Based on geologic mapping the Saugus Formation indicated a general dip of 50 to 70 degrees to the southwest, correlating with the southern leg of an anticline with the axis located approximately parallel to the San Gabriel Fault, 1.5 miles north of the site.

The unit observed was generally intact and massive with thick to very thick bedding. Well-developed jointing and fracturing of in-situ rock was minimal. Based on the results of the seismic refraction survey (Geosyntec, 2021), the Saugus Formation within the footprint of the proposed Compressor Facility is interpreted to be rippable with Primary compression wave (P-wave) velocities in the range of 2,500 to 4,500 ft/s within the depth of the investigation of about 40 ft. The maximum P-wave velocities in the Saugus Formation underlying the alluvium in the valley area were approximately 5,500 to 6,000 ft/s.

3.2 Intact uniaxial compressive strength

The unconfined compression test results for the rock samples from the Saugus formation are summarized in Table 1. Based on the evaluation of the results in Table 1 and averaging the results between the estimated sandstone and claystone ratio (60:40), an intact uniaxial strength of 21 ksf (1 MPa) was deemed representative for the Saugus formation. The chosen uniaxial compressive strength indicates that the Saugus formation on Site corresponds to a very weak formation based on Table 2, which is also in good agreement with our geological surveys on Site. Based on our review, the results from samples 3 and 6 were omitted because the samples may have been compromised and are not believed to be representative of the intact rock within the overall rock mass.

Table 1. Uniaxial Compressive Strength Test Results (Geosyntec, 2021)

Sample Information				**Description**	**Uniaxial Compressive Strength Test Results (ASTM D7012)**
Sample Number	**Sample ID**	**Sample Type (a)**	**Depth (ft bgs)**		
1	B-1@40-43	Rock Core	40.5-41	Clayey Sandstone	UCS = 14.08 ksf
2	B-1@43-46	Rock Core	44.1-44.5	Clayey Sandstone	UCS = 31.83 ksf
3	B-2@7.5-10	Rock Core	9-9.5	Silty Sandstone	UCS = 2.32 ksf
4	B-2@15-17.5	Rock Core	15.5-16	Silty Claystone	UCS = 7.04 ksf

5	B-2@22.5-25	Rock Core	24-24.5	Claystone	UCS = 6.14 ksf
6	B-2@27-30	Rock Core	27.5-28	Silty Sandstone	UCS = 0.89 ksf
7	B-3@6-11	Rock Core	7.5-8	Sandy Claystone	UCS = 5.33 ksf
8	B-4@1.5-6	Rock Core	5-5.5	Silty Sandstone	UCS = 47.60 ksf

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Table 2. Field estimates of unconfined compressive strength (Hoek, 2001)

Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field estimate of strength	Examples
R6	Extremely Strong	> 250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100 - 250	4 - 10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite, rhyolite, tuff
R4	Strong	50 - 100	2 - 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium strong	25 - 50	1 - 2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concete, phyllite, schist, siltstone
R2	Weak	5 - 25	**	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rocksalt,
R1	Very weak	1 - 5	**	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock, shale
R0	Extremely weak	0.25 - 1	**	Indented by thumbnail	Stiff fault gouge

* Grade according to Brown (1981).

** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.

3.3 Geological strength index (GSI)

The strength of a jointed rock mass depends on the properties of the intact rock pieces and also upon the freedom of these pieces to slide and rotate under different stress conditions. This freedom is controlled by the geometrical shape of the intact rock pieces as well as the condition of the surfaces separating the pieces. Angular rock pieces with clean, rough discontinuity surfaces will result in a much stronger rock mass than one which contains rounded particles surrounded by weathered and altered material. The Geological Strength Index (GSI), introduced by Hoek (1994) and Hoek, Kaiser and Bawden (1995) provides a system for estimating the reduction in rock mass strength for different geological conditions. This system is presented in Table 3, for blocky rock masses, and Table 4 for schistose metamorphic rocks.

Table 3. Characterization of a blocky rock masses based on particle interlocking and discontinuity condition (Hoek, 2001)

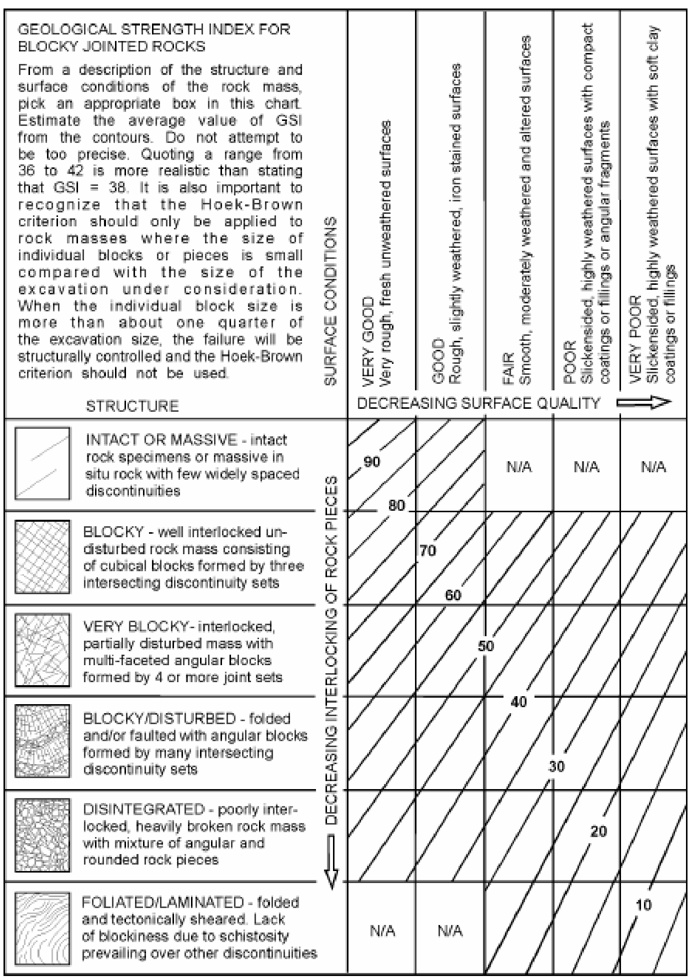

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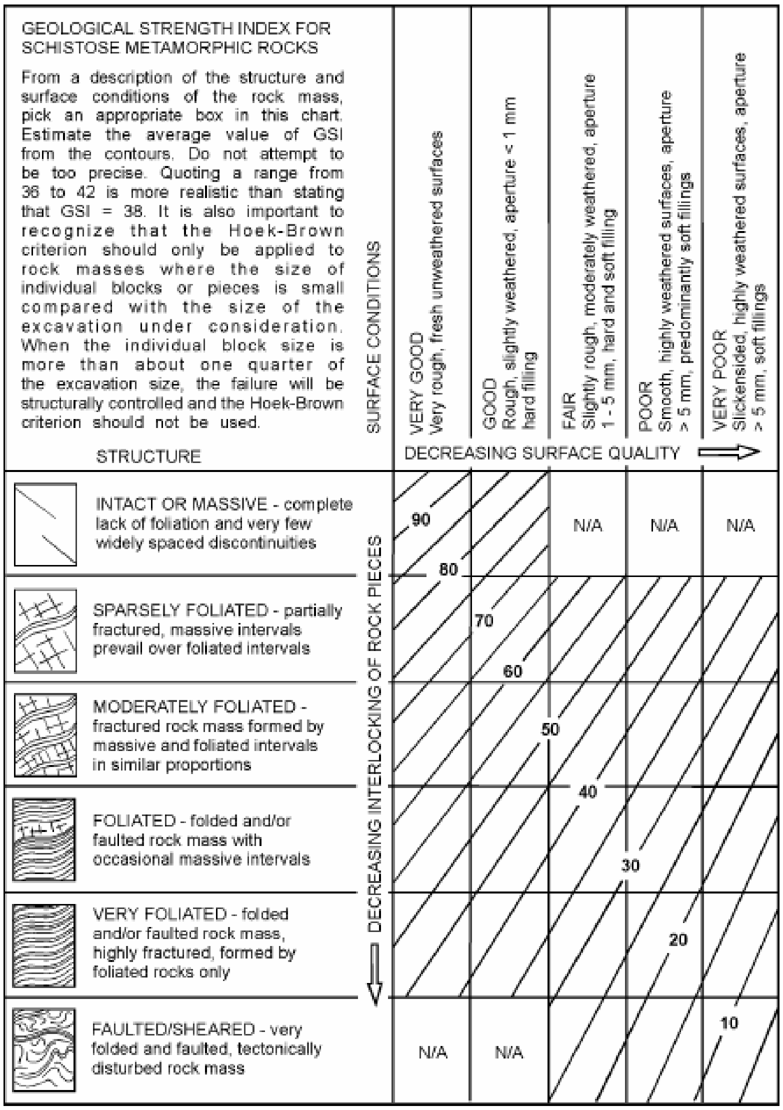
Table 4. Characterization of schistose metamorphic rock masses based on foliation and discontinuity condition (Hoek, 2001)



Based on the observed surface and structure conditions of the thick to very thick beds (generally greater than ten inches and as much as five feet thick) observed during Geosyntec field soil investigation visit on Site, a good to fair surface condition with a GSI value of 65 was deemed to

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be representative for the observed surface conditions and rock structure (Rough, slightly to moderately weathered surfaces, altered surfaces, and blocky).

3.4 Material Constants

The generalized Hoek-Brown failure criterion for rock masses is defined by the equation below:

$$\sigma_1' = \sigma_3' + \sigma_{ci} * (m_b * \frac{\sigma_3'}{\sigma_{ci}} + s)^a \quad (1)$$

where σ_1' and σ_3' are the major and minor effective principal stresses at failure;

m_b is a reduced value of the material constant m_i and is given by $m_b = m_i * \exp\,(\frac{GSI-100}{28-14D})$

D is disturbance factor (defined in Section 3.4)

σ_{ci} is the uniaxial compressive strength of the intact rock material;

m_i is the material constant for intact rock;

s and *a* are constants for the rock mass given by the following relationships:

$$s = \exp\,(\frac{GSI-100}{9-3D}) \quad (2)$$

$$a = \frac{1}{2} + \frac{1}{6} * (e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}) \quad (3)$$

The material constant, m_i can be estimated for the Saugus formation considering the 60 to 40 sandstone to claystone ratio as 10 based on Table 5.

3.5 Disturbance Factor (*D*)

Disturbance factor (*D*) depends on the degree of disturbance to which the rock mass has been subjected by blast damage, mechanical excavation and/or stress relaxation. It varies from 0 for undisturbed in-situ rock masses to 1 for very disturbed rock masses. Guidelines for the selection of *D* are presented in Table 6. Considering the fact that rock masses at Site are observed to be weathered yet primarily intact on the surface, the Saugus formation can be assumed to possess a *D* factor of 0.7 based on Geosyntec's observations on site and observed intact rock samples from the site.

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3.6 Rock mass deformation modulus (E_{rm}), Modulus Ratio (MR), Intact modulus (E_i)

Hoek and Diederichs (2006) re-examined existing empirical methods for estimating rock mass deformation modulus. In their analysis, they incorporated modulus ratio (MR), which is the ratio of rock mass deformation modulus to intact modulus (E_{rm}/E_i). Using the modulus ratio (MR), the intact modulus (E_i) can be estimated as:

$$E_i = MR * \sigma_{ci} \tag{4}$$

The modulus ratio (MR) in equation (4) can be assumed as MR=250 based on Table 7 for the encountered Saugus formation at Site, which results in an intact modulus value of E_i = 250 MPa based on the uniaxial compressive strength of the rock material (σ_{ci} = 1 MPa).

Based on the detailed analysis of Hoek and Diederichs (2006), rock deformation modulus (E_{rm}) can be estimated as:

$$E_{rm} = E_i * \left(0.02 + \frac{1-\frac{D}{2}}{1+e^{\left(\frac{60+15D-GSI}{11}\right)}}\right) \tag{5}$$

By considering a GSI value of 65 and D value of 0.7, the rock deformation modulus can be calculated as E_{rm}= 66.4 MPa.

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Table 5. Values of the constant m_i for intact rock, by rock group. Note that values in parenthesis are estimates. (Hoek, 2001)

Rock type	Class	Group	Texture: Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
		Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6		
	Foliated*		Gneiss 28 ± 5	Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	Obsidian (19 ± 3)
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	

* These values are for intact rock specimens tested normal to bedding or foliation. The value of m_i will be significantly different if failure occurs along a weakness plane.

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Table 6. Guidelines for estimating disturbance factor *D*. (Hoek et al, 2002)

Appearance of rock mass	Description of rock mass	Suggested value of *D*
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	$D = 0$
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	$D = 0$ $D = 0.5$ No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	$D = 0.8$
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

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Table 7. Guidelines for the selection of modulus ratio (MR) values in Equation (4) (Hoek, and Diederichs, 2006)

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
Sedimentary	Clastic		Conglomerates 300–400 Breccias 230–350	Sandstones 200–350	Siltstones 350–400 Greywackes 350	Claystones 200–300 Shales 150–250[a] Marls 150–200
	Non-clastic	Carbonates	Crystalline limestones 400–600	Sparitic limestones 600–800	Micritic Limestones 800–1000	Dolomites 350–500
		Evaporites		Gypsum (350)[b]	Anhydrite (350)[b]	
		Organic				Chalk 1000+
Metamorphic	Non-foliated		Marble 700–1000	Hornfels 400–700 Metasandstone 200–300	Quartzites 300–450	
	Slightly foliated		Migmatite 350–400	Amphibolites 400–500	Gneiss 300–750[a]	
	Foliated[a]			Schists 250–1100[a]	Phyllites/Mica Schist 300–800[a]	Slates 400–600[a]
Igneous	Plutonic	Light	Granite[c] 300–550 Granodiorite[c] 400–450	Diorite[c] 300–350		
		Dark	Gabbro 400–500 Norite 350–400	Dolerite 300–400		
	Hypabyssal		Porphyries (400)[b]		Diabase 300–350	Peridotite 250–300
	Volcanic	Lava		Rhyolite 300–500 Andesite 300–500	Dacite 350–450 Basalt 250–450	
		Pyroclastic	Agglomerate 400–600	Volcanic breccia (500)[b]	Tuff 200–400	

[a]Highly anisotropic rocks: the value of MR will be significantly different if normal strain and/or loading occurs parallel (high MR) or perpendicular (low MR) to a weakness plane. Uniaxial test loading direction should be equivalent to field application.

[b]No data available, estimated on the basis of geological logic.

[c]Felsic Granitoids: coarse grained or altered (high MR), fined grained (low MR).

3.7 Uniaxial compressive strength, tensile strength and rock mass (global) strength

The uniaxial compressive strength of in-situ rock mass is obtained by setting $\boldsymbol{\sigma_3'}$=0 in equation (1), giving:

$$\sigma_c = \sigma_{ci} * s^a \tag{6}$$

and, the tensile strength is:

$$\sigma_t = -\frac{s*\sigma_{ci}}{m_b} \tag{7}$$

The uniaxial compressive strength of the rock mass σ_c is given by equation (6). This strength is representative for failures that initiate at the boundary of an excavation when σ_c is exceeded by the stress induced on that boundary. The failure propagates from this initiation point into a biaxial stress field and it eventually stabilizes when the local strength, defined by equation (1), is higher than the induced stresses σ_1' and σ_3' .

Based on Hoek et al. (2002), it is useful to consider the overall behavior of a rock mass rather than the detailed failure propagation process described above. This leads to the concept of a global rock mass strength σ_{cm}', which can be estimated from the Mohr Coulomb relationship:

$$\sigma_{cm}' = \frac{2*c*cos\varphi}{1-sin\varphi} \tag{8}$$

with cohesion (c) and friction angle (φ) determined for the stress range $\sigma_t < \sigma_3 < \sigma_{ci}/4$, resulting in the rock mass (global) strength as:

$$\sigma_{cm}' = \sigma_{ci} * \frac{(m_b+4s-a(m_b-8s))*(\frac{m_b}{4}+s)^{(a-1)}}{2*(1+a)*(2+a)} \tag{9}$$

For the project boundary conditions, the resulting in-situ uniaxial compressive strength, tensile strength, and the rock mass (global) strength values are calculated as: σ_c = 0.08 MPa, σ_t = -0.004 MPa, σ_{cm}' = 0.17 MPa.

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3.8 Maximum confining Stress (σ_{3max}) and Mohr-Coulomb Criterion

The equivalent Mohr-Coulomb parameters (friction angle and cohesion) for a rock mass will be determined case-specifically for the relevant stress range. This is done by fitting an average linear relationship to the curve generated by solving equation (1) for a range of minor principal stress values defined by $\boldsymbol{\sigma_t < \sigma_3 < \sigma_{3max}}$, as illustrated in Figure 1.

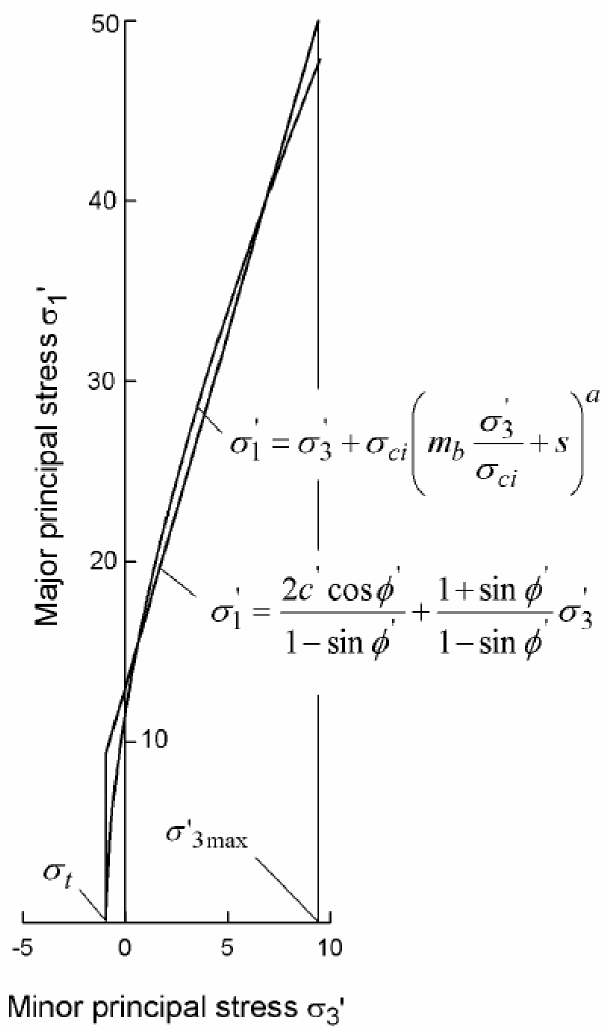

Figure 1. Relationships between major and minor principal stresses for Hoek-Brown and equivalent Mohr-Coulomb criteria (Hoek et al, 2002)

The fitting process involves balancing the areas above and below the Mohr-Coulomb plot. This results in the following equations for the equivalent angle of friction and cohesive strength of the in-situ rock mass:

$$\varphi = sin^{-1}\left[\frac{6am_b(s+m_b\sigma_{3n})^{(a-1)}}{2(1+a)(2+a)+6am_b(s+m_b\sigma_{3n})^{(a-1)}}\right] \quad (10)$$

$$c = \frac{\sigma_{ci}[(1+2a)s+(1-a)m_b\sigma_{3n}](s+m_b\sigma_{3n})^{(a-1)}}{(1+a)(2+a)\sqrt{1+\frac{(6am_b(s+m_b\sigma_{3n})^{(a-1)}}{(1+a)(2+a)}}} \quad (11)$$

where $\sigma_{3n} = \frac{\sigma_{3max}}{\sigma_{ci}}$

The maximum confining stress (σ_{3max}), is the upper limit of confining stress over which the relationship between the Hoek-Brown and Mohr-Coulomb criteria is considered and this has to be determined for each individual case. Based on Hoek et al. (2002), extensive studies for slopes, using Bishop's circular failure analysis for a wide range of slope geometries and rock mass properties, gave:

$$\frac{\sigma_{3max}}{\sigma'_{cm}} = 0.72 * (\frac{\sigma'_{cm}}{\gamma H})^{-0.91} \quad (12)$$

where H is the height of the slope.

4. RESULTS

Based on the previously described calculations (Equation 1 through 12), the equivalent Mohr-Coulomb strength parameters for the Saugus formation at Site are summarized in Table 8 for various slope heights considered in our slope stability evaluations. The graphical illustration of the equivalent Mohr-Coulomb soil strength model along with the Hoek-Brown rock model for the considered rock parameters is shown in Figure 2. Shear strength parameters used in slope stability analyses may either be based on equivalent Mohr-Coulomb parameters for corresponding equivalent slope height or the fully defined shear strength curve as a function of normal load as shown in the Hoek-Brown model solution in Figure 2 for the Saugus formation at the Site.

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Table 8. Equivalent Mohr Coulomb Parameters based on Hoek-Brown Model

Cross Section	Slope Height, H (ft)	phi (deg)	c (psf)
A, B	20	38	569
E	70	28	1107
C, D	100	26	1350

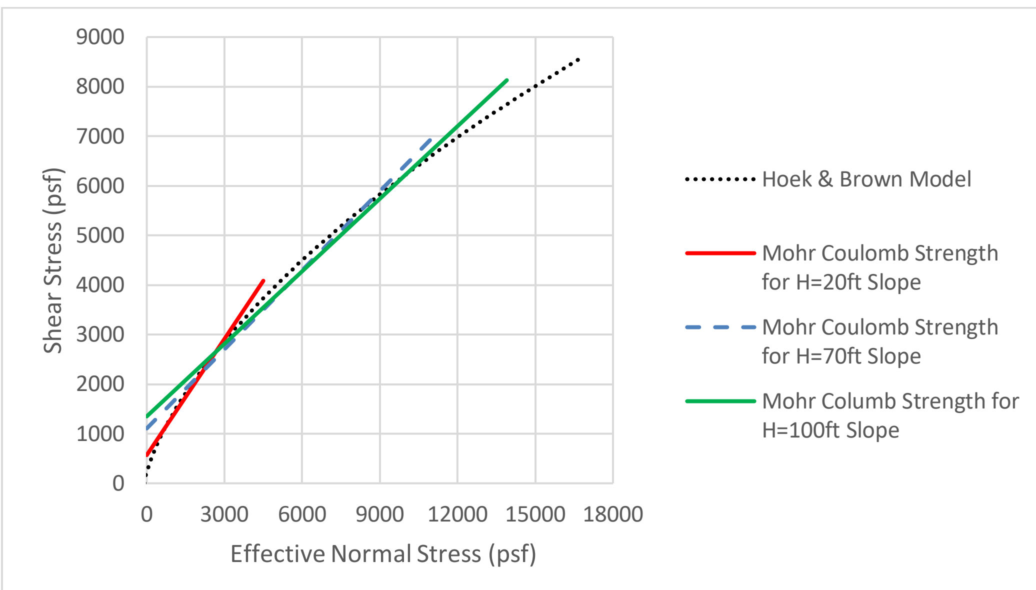
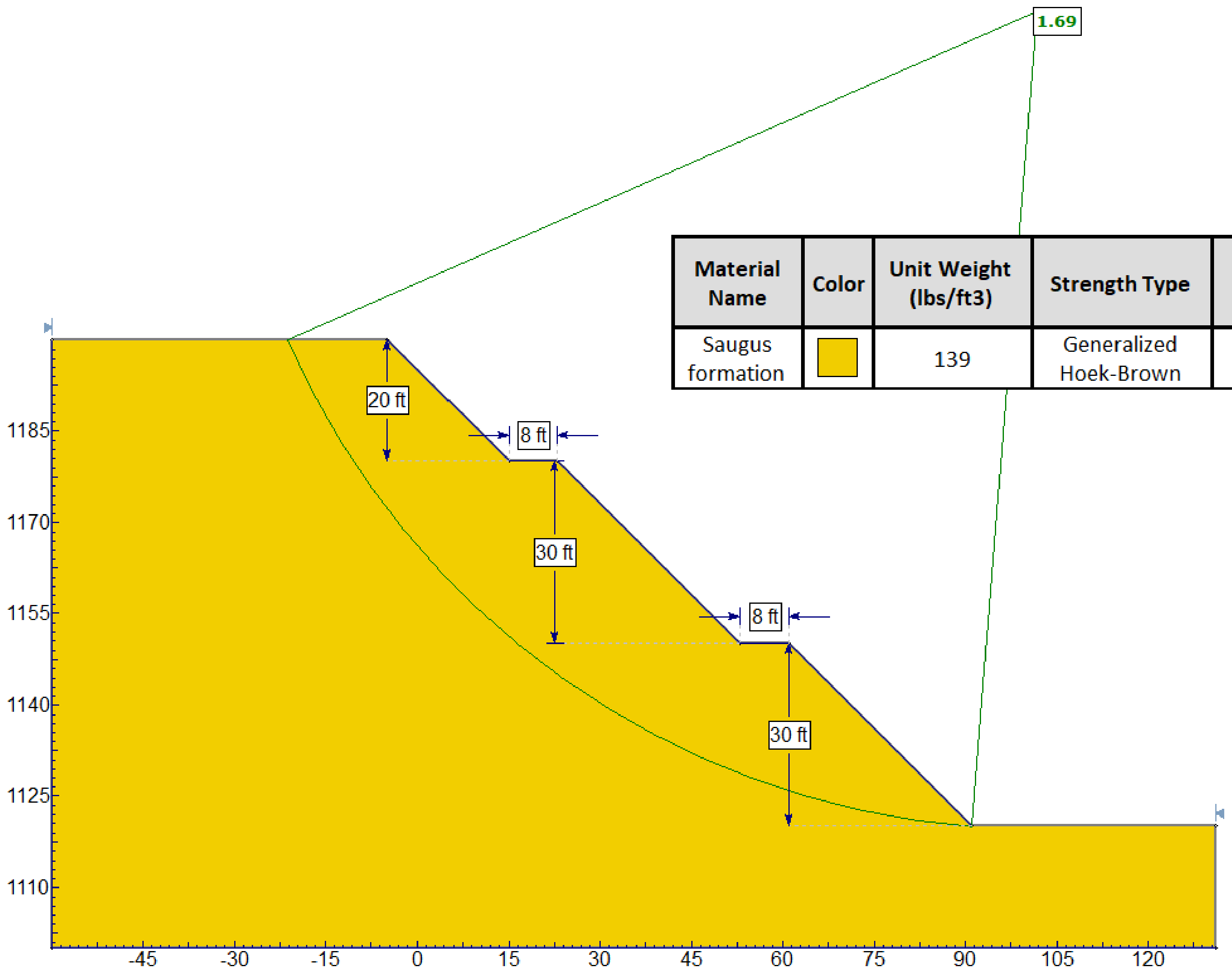


Figure 2. Equivalent Mohr Coulomb Strength based on Hoek-Brown Model

APPENDIX F

Slope Stability Analysis Output



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (intact) (psf)	GSI	mi	D
Saugus formation		139	Generalized Hoek-Brown	21000	65	10	0.7

STATIC 1:1 SLOPE
SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

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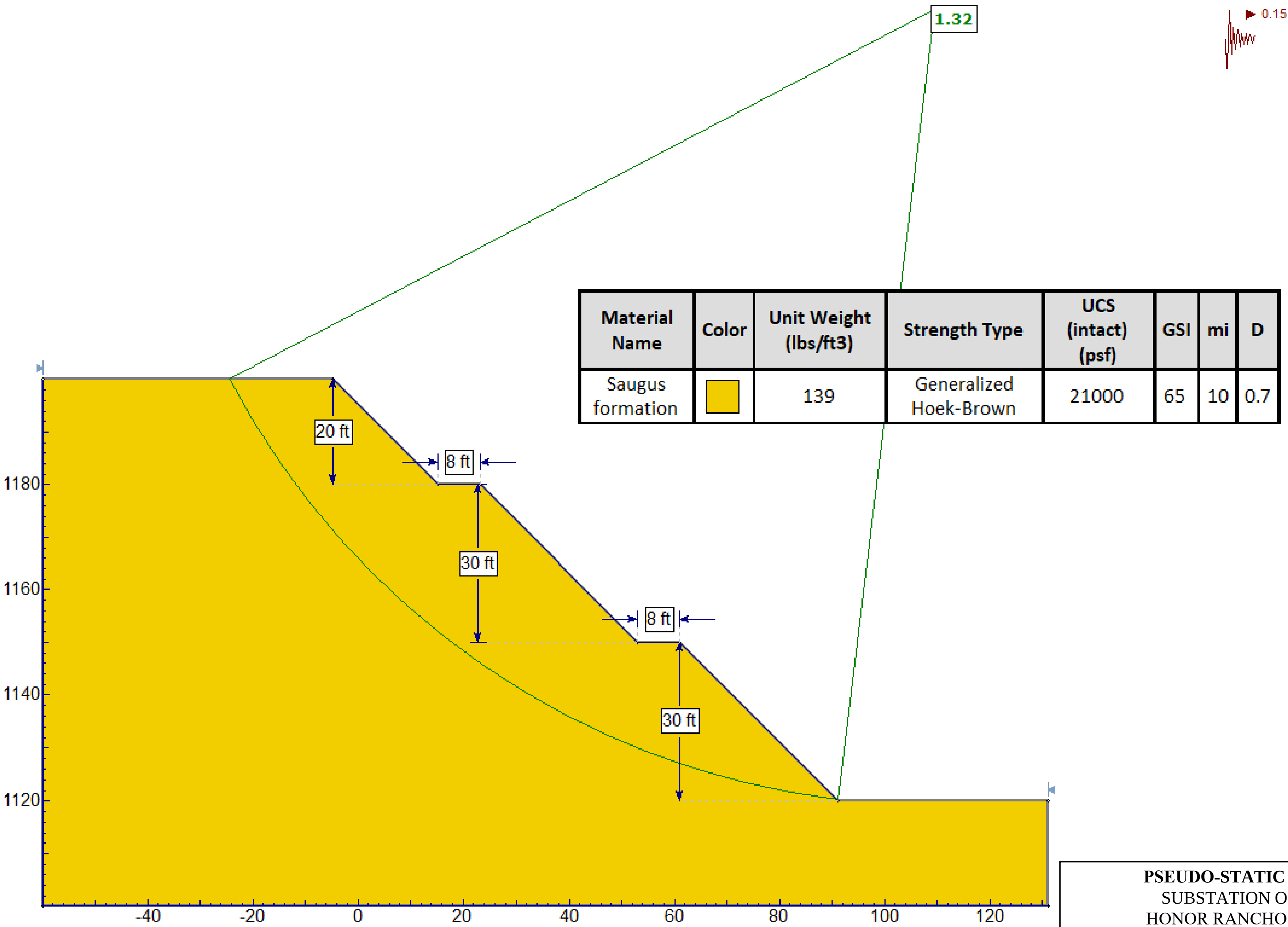
horizontal seismic coefficient (kh) = 0.15



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (intact) (psf)	GSI	mi	D
Saugus formation		139	Generalized Hoek-Brown	21000	65	10	0.7

PSEUDO-STATIC 1:1 SLOPE
SUBSTATION OPTION A
HONOR RANCHO FACILITY
SANTA CLARITA, CALIFORNIA

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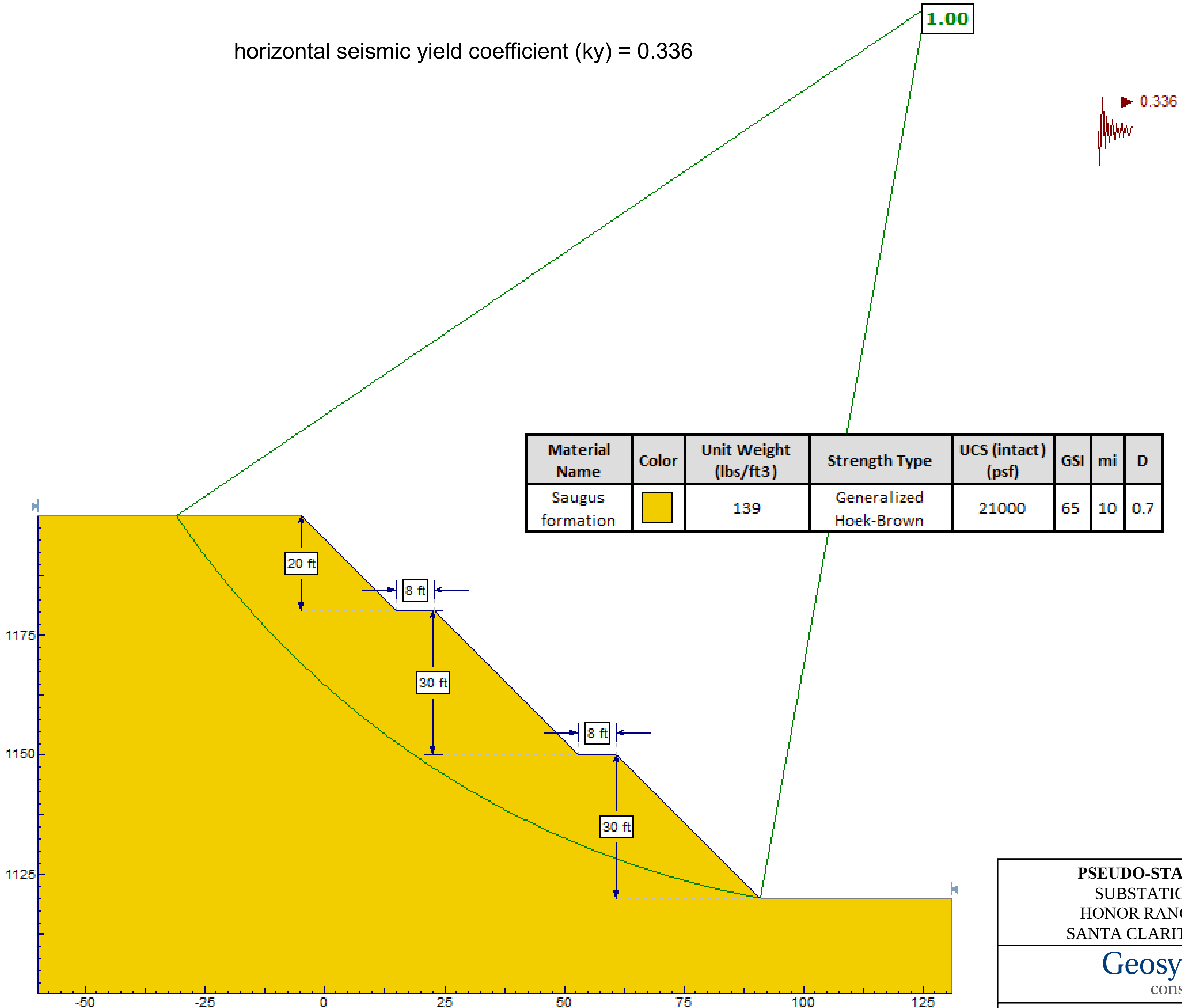
PROJECT NO. SC1339	JULY 2023

horizontal seismic yield coefficient (ky) = 0.336



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (intact) (psf)	GSI	mi	D
Saugus formation		139	Generalized Hoek-Brown	21000	65	10	0.7

PSEUDO-STATIC 1:1 SLOPE
SUBSTATION OPTION A
HONOR RANCHO FACILITY
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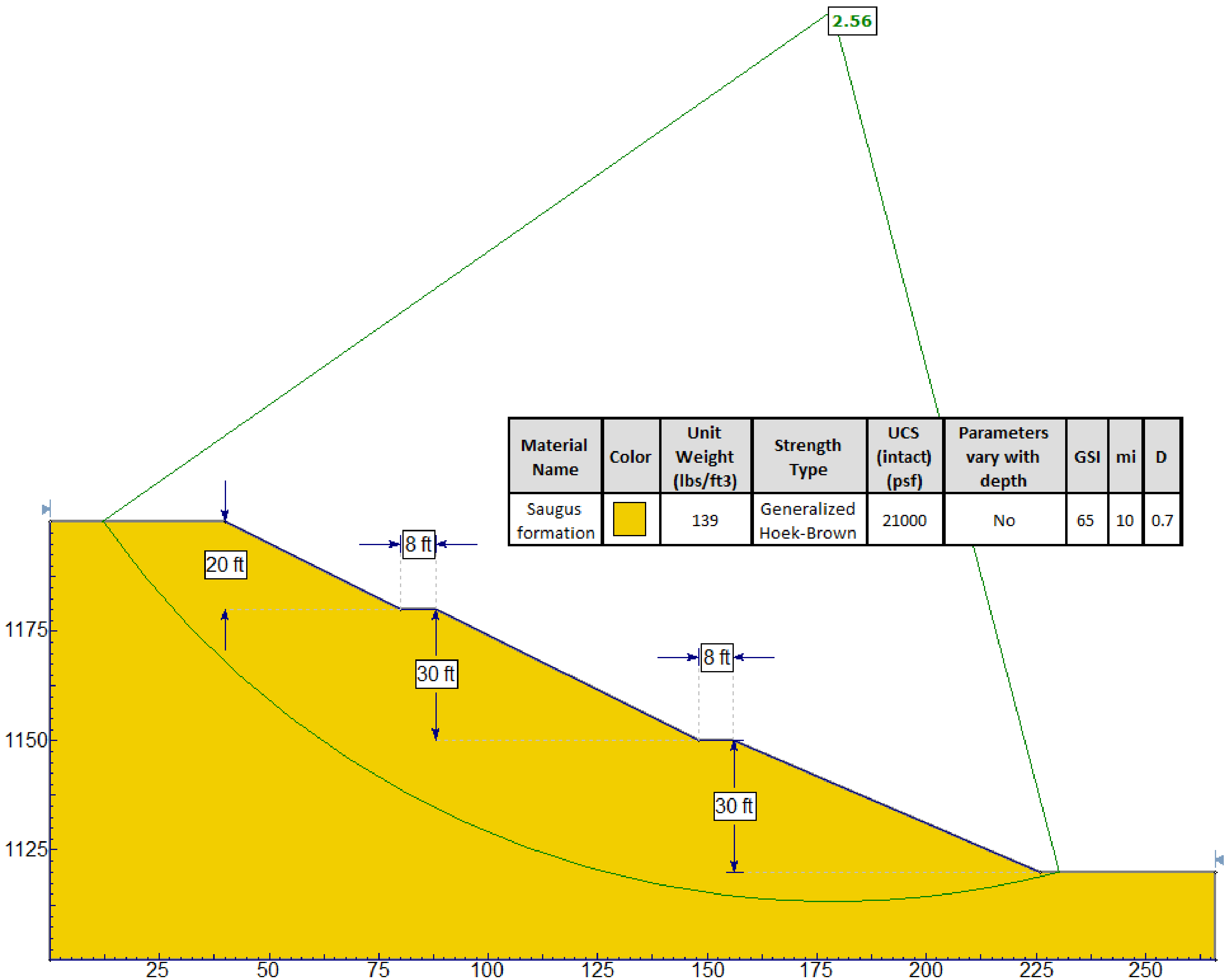

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (intact) (psf)	Parameters vary with depth	GSI	mi	D
Saugus formation		139	Generalized Hoek-Brown	21000	No	65	10	0.7

STATIC 2:1 SLOPE
SUBSTATION OPTION A
HONOR RANCHO FACILITY
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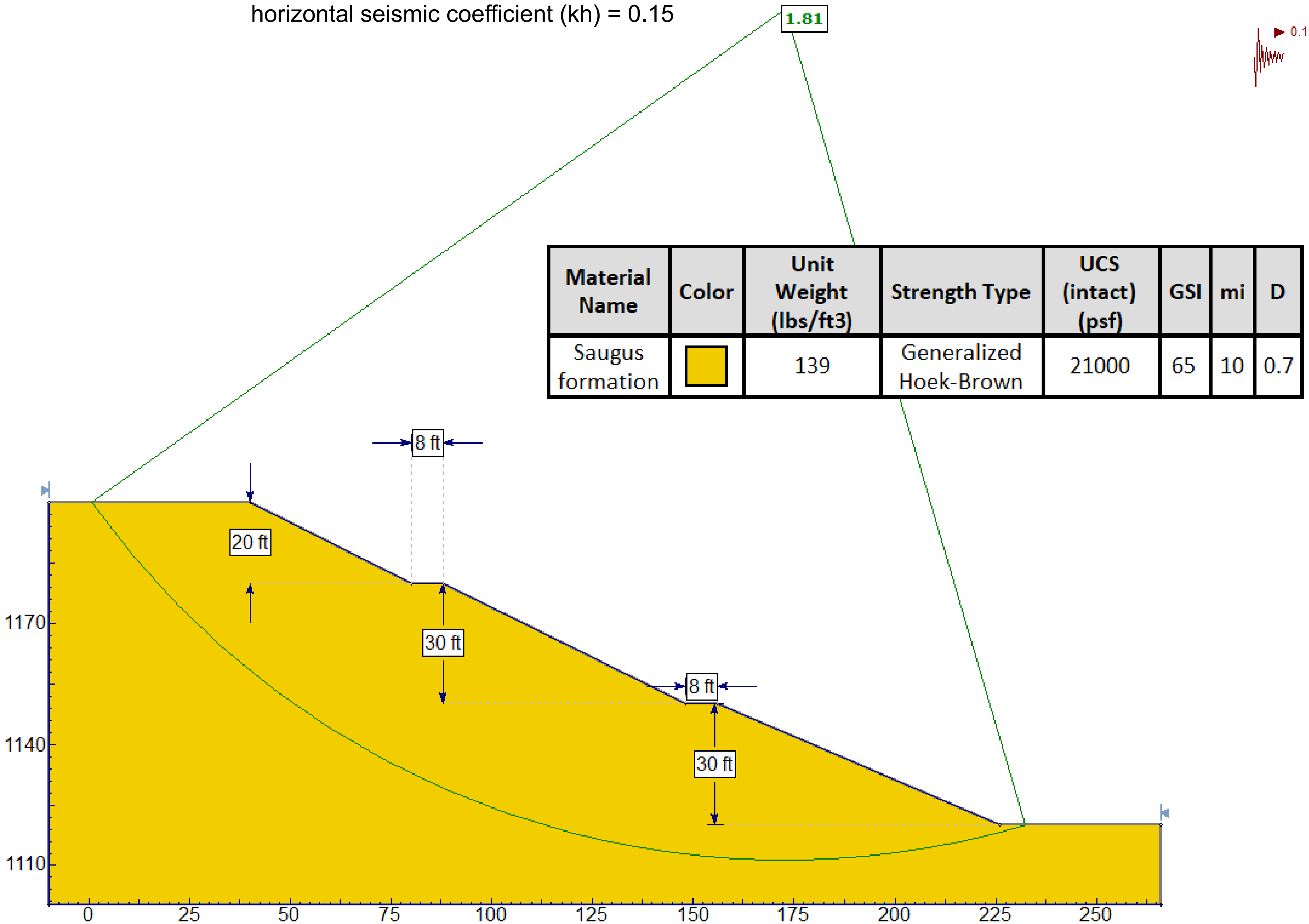

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (intact) (psf)	GSI	mi	D
Saugus formation		139	Generalized Hoek-Brown	21000	65	10	0.7

PSEUDO-STATIC 2:1 SLOPE
SUBSTATION OPTION A
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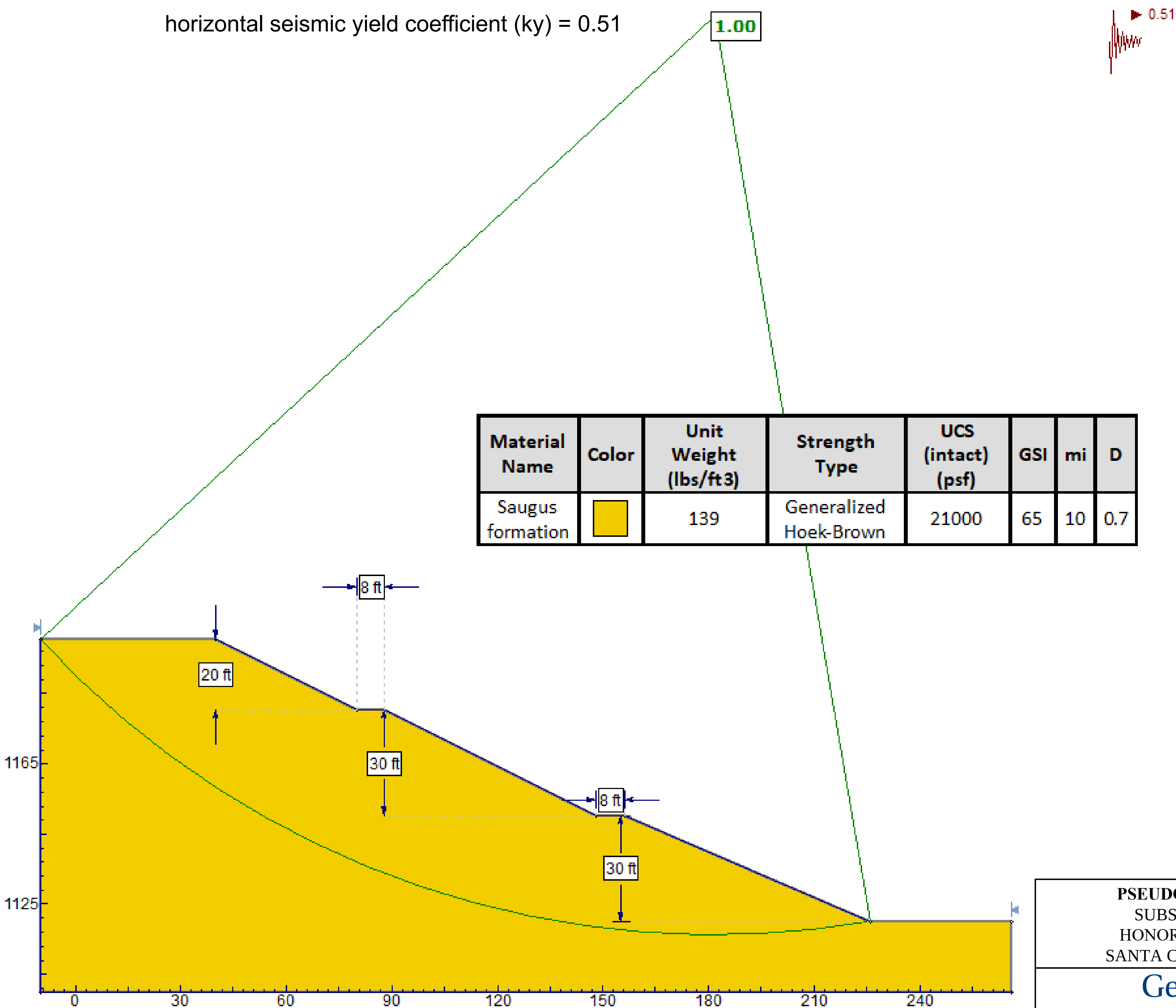

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (intact) (psf)	GSI	mi	D
Saugus formation		139	Generalized Hoek-Brown	21000	65	10	0.7

PSEUDO-STATIC 2:1 SLOPE
SUBSTATION OPTION A
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APPENDIX G

Seismic Refraction Survey Report, Geosyntec (2021a)

REPORT
SEISMIC REFRACTION SURVEY

28300 Brady Parkway
Santa Clarita, California

GEO*Vision* Project No. 21004

Prepared for

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January 22, 2021

Report 21004-01 Rev 0

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APPENDICES

1 INTRODUCTION

A P-wave seismic refraction survey was conducted at the property located at 28300 Brady Parkway, Santa Clarita, California on January 5, 2021. The purpose of this investigation was to determine the rippability of the sedimentary rock of the Saugus Formation. P-wave seismic refraction data was acquired along a single profile, designated as Line 1 (Figure 1).

The expected geology in this area consists of soil overlying the Saugus Formation, expected to be primarily comprised of sandstone. Depending on the bedding, degree of weathering, jointing, etc., sandstone rock may broadly be characterized as rippable using a Caterpillar D8R ripper to P-wave velocities of about 6,500 feet per second (ft/s), marginally-rippable to 8,300 ft/s, and non-rippable at P-wave velocity greater than 8,300 ft/s (Caterpillar, 2018). Using a Caterpillar D9R, rock is considered rippable to P-wave velocities of 7,300 ft/s, marginally-rippable to 9,600 ft/s, and non-rippable at P-wave velocity greater than 9,600 ft/s.

The following sections include a discussion of equipment and field procedures, methodology, data processing, and results of the geophysical survey.

P-wave Seismic Refraction Line

NOTES:
1. Coordinate System: California State Plane, NAD83, Zone V (0405), US Survey Feet
2. Base map source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

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Date:	1/7/2021
GV Project:	21004
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Drawn by:	T Rodriguez
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File Name:	21004-1.MXD

FIGURE 1
SITE MAP

SITE LOCATED AT
28300 BRADY PKWY
SANTA CLARITA, CALIFORNIA

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2 METHODOLOGY

Detailed discussions of the seismic refraction method can be found in Telford et al. (1990), Dobrin and Savit (1988), and Redpath (1973).

When conducting a seismic survey, acoustic energy is input to the subsurface by an energy source such as a sledgehammer impacting a metallic plate, weight drop, vibratory source, or explosive charge. The acoustic waves propagate into the subsurface at a velocity dependent upon the elastic properties of the material through which they travel. When the waves reach an interface where the density or velocity changes significantly, a portion of the energy is reflected to the surface and the remainder is transmitted into the lower layer. Where the velocity of the lower layer is higher than that of the upper layer, a portion of the energy is also critically refracted along the interface. Critically refracted waves travel along the interface at the velocity of the lower layer and continually refract energy back to the surface. Receivers (geophones) laid out in linear array on the surface record the incoming refracted and reflected waves. The seismic refraction method involves analysis of the travel times of the first energy to arrive at the geophones. These seismic first arrivals are from either the direct wave (at geophones close to the source) or critically refracted waves (at geophones further from the source).

Analysis of seismic refraction data depends upon the complexity of the subsurface velocity structure. If the subsurface target is planar in nature then the slope intercept method (Telford et al., 1990) can be used to model multiple horizontal or dipping planar layers. A minimum of one end shot is required to model horizontal layers and reverse end shots are required to model dipping planar layers. If the subsurface target is undulating (i.e. bedrock valley) then layer based analysis routines such as the generalized reciprocal method (Palmer, 1980 and 1981, Lankston and Lankston, 1986 and Lankston, 1990); reciprocal method (Hawkins, 1961) also referred to as the ABC method; Hales' method (Hales, 1958); delay time method (Wyrobek, 1956 and Gardner, 1967); time-term inversion (Scheidegger and Willmore, 1959); plus-minus method (Hagedoorn, 1959); and wavefront method (Rockwell, 1967) are preferred to model subsurface velocity structure. These methods generally require a minimum of 5 shot points per spread (end shots, off end shots and a center shot). If subsurface velocity structure is complex and cannot be adequately modeled using layer-based modeling techniques (e.g., complex weathering profile in bedrock, numerous lateral velocity variations), then Monte Carlo or tomographic inversion techniques (Zhang and Toksoz, 1998; Schuster and Quintus-Bosz, 1993) are required to model the seismic refraction data. These techniques require a high shot density; typically, every 1 to 6 stations/geophones. Generally, these techniques cannot effectively take advantage of off-end shots to extend depth of investigation, so longer profiles are required.

Errors in seismic refraction models can be caused by velocity inversions, hidden layers, or lateral velocity variations. At sites with steeply dipping or highly irregular bedrock surfaces, refractions from structures to the side of the line rather than from beneath the line may severely complicate modeling. A velocity inversion is a geologic layer with a lower seismic velocity than an overlying layer. Critical refraction does not occur along such a layer because velocity has to increase with depth for critical refraction to occur. This type of layer, therefore, cannot be recognized or modeled and depths to underlying layers would be overestimated.

A hidden layer is a layer with a velocity increase, but of sufficiently small thickness relative to the velocities of overlying and underlying layers, that refracted arrivals do not arrive at the geophones before those from the deeper, higher velocity layer. Because the seismic refraction method generally only involves the interpretation of first arrivals, a hidden layer cannot be recognized or modeled and depths to underlying layers would be underestimated. Saturated sediments overlying high velocity bedrock can be a hidden layer under many field conditions. Generally, saturated sediments generally have a much higher velocity than unsaturated sediments, typically in the 5,000 to 7,000 ft/s range and can occasionally be interpreted as a second arrival when the layer does not give rise to a first arrival.

A subsurface velocity structure that increases as a function of depth rather than as discrete layers will cause depths to subsurface refractors to be underestimated in a manner very similar to that of the hidden layer problem. Lateral velocity variations that are not adequately addressed in the seismic models also lead to depth errors. Tomographic imaging techniques can often resolve the complex velocity structures associated with hidden layers, velocity gradients, and lateral velocity variations. However, in the event of an abrupt increase in velocity at a geologic horizon, the velocity model generated using tomographic inversion routines will smooth the horizon with velocity possibly being underestimated at the interface and overestimated at depth.

3 EQUIPMENT AND FIELD PROCEDURES

Seismic refraction equipment used during this investigation consisted of a Geometrics Geode 24-channel signal enhancement seismograph, 10 Hz vertical geophones, seismic cables with 10-foot spaced connectors, piezo hammer switches, and a 20-lb sledgehammer and aluminum strike plate.

The seismic line consisted of 24 geophones spaced 6 feet apart for a total line length of 138 feet. Elevations along the refraction lines were surveyed using a Spectra SP60 GPS system with CenterPoint RTX real-time differential corrections. The location of the seismic line is presented in Table 1.

Sample photographs of seismic equipment is provided in Appendix A. Source locations included end shots at the end geophone, multiple off-end shot locations, and interior shot locations at every 4th geophone for a total of 11 shot points. A 20-lb sledgehammer was used as the energy source for all source locations. A hammer switch mounted on the aluminum plate was used to trigger the seismograph upon impact. The final seismic record at each shot point was the result of stacking 5 to 10 shots to increase the signal to noise ratio. All seismic records were stored on a laptop computer. Data acquisition parameters, file names, and other observations were recorded on a digital observers' log, which is retained in project files.

Table 1 Location of Seismic Line 1

Position (ft)	Northing (US ft)	Easting (US ft)	Elevation (ft)
0	1984999.6	6385146.3	1210.3
6	1985003.2	6385141.5	1210.3
12	1985006.3	6385136.4	1210.5
18	1985010.4	6385132.2	1210.2
24	1985013.7	6385127.2	1209.7
30	1985017.0	6385122.5	1209.0
36	1985020.4	6385117.8	1208.3
42	1985023.9	6385113.0	1206.8
48	1985027.5	6385108.5	1205.6
54	1985031.1	6385103.8	1205.0
60	1985034.7	6385098.9	1204.4
66	1985038.2	6385093.8	1204.8
72	1985041.5	6385089.1	1205.1
78	1985045.1	6385084.1	1205.7
84	1985048.4	6385079.3	1205.6
90	1985052.0	6385074.4	1205.9
96	1985055.6	6385069.7	1206.3
102	1985059.1	6385065.0	1206.7
108	1985062.7	6385060.2	1206.3
114	1985066.0	6385055.3	1205.7
120	1985069.3	6385050.3	1205.3
126	1985073.0	6385045.5	1204.8
132	1985076.2	6385040.5	1204.1
138	1985079.6	6385035.9	1202.5

Note: Coordinates in California State Plane Coordinate System, Zone 5, NAD83, US feet.

4 DATA REDUCTION AND MODELING

The first step in data processing consisted of picking the arrival time of the first energy (first-arrival) received at each geophone for each shot point. The first-arrivals on each seismic record are either a direct arrival from a compressional (P) wave traveling in the uppermost layer or a refracted arrival from a subsurface interface where there is a velocity increase. First-arrival times were selected using the manual picking routines in the SeisImager™ software suite (Geometrics, Inc.). These first-arrival times were saved in an ASCII file containing shot location, geophone locations, and associated first-arrival time. Errors in the first-arrival times were variable with error generally increasing with distance from the shot point.

Analysis of seismic refraction data depends upon the complexity of the subsurface velocity structure. Layer-based and tomographic inversion routines can be used to model the seismic data. Layer-based methods are better suited when subsurface units are arranged along distinct geologic boundaries, whereas tomographic methods may be better applied when gradational changes across geologic contacts. These different modeling schemes have their own strengths and weaknesses. Refraction tomography techniques are often able to resolve complex velocity structure (e.g. velocity gradients) that can be observed in bedrock weathering profiles. Layer-based modeling techniques such as GRM are not able to accurately model the velocity gradients that can be observed in weathered or transitional zones. However, tomographic modeling methods force a velocity gradient across apparent geologic units or vertical cross-section, smoothing the velocity ranges presented in the model.

Seismic refraction data were first modeled using a two or three-layer modeling algorithm to fit the major trends in the travel time data. This layer-based model was used as a starting model for preliminary analysis using the tomographic inversion routine in the SeisImager Plotrefa software package. Analysis was also conducted using the tomographic inversion routine with a smooth velocity gradient starting model, which was selected for site characterization.

The final tomographic velocity models for the seismic line were exported as ASCII files and imported into the Golden Software Surfer mapping system where the velocity model was gridded, contoured, and annotated for presentation.

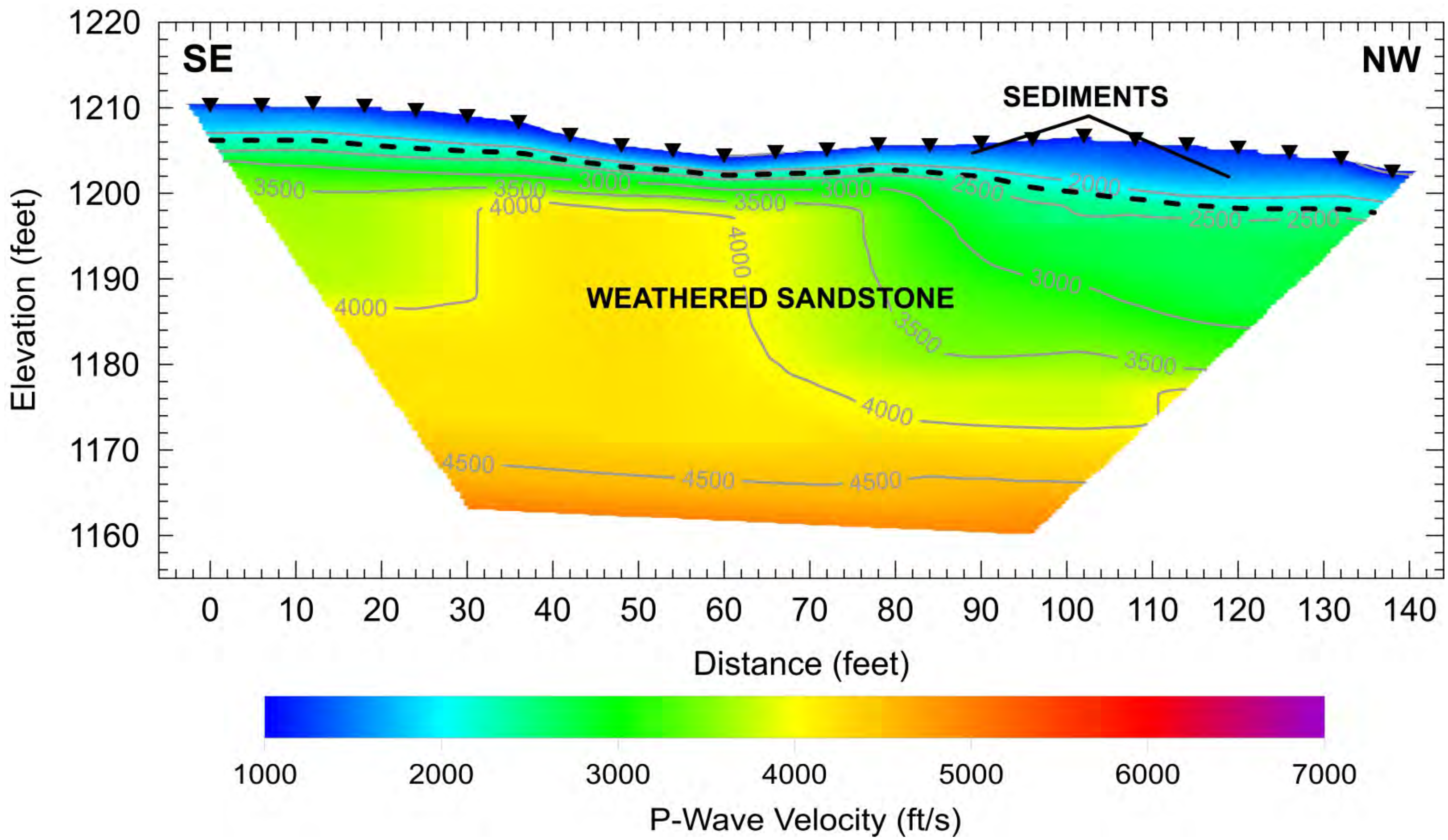
5 DISCUSSION OF RESULTS

The P-wave seismic refraction model for Line 1 is presented as Figure 2. In tomographic models, sharp layer contacts are not clearly defined and thus ranges of velocities are used to interpret possible rock conditions and competency. A color scheme with blue-cyan, green-orange, and red-purple indicating low, intermediate, and high P-wave velocities, respectively, and velocity contours at 500 ft/s intervals are used to display the seismic velocity model.

Tomographic inversion techniques will typically model a gradual increase in velocity with depth even if an abrupt velocity contact is present. Velocity gradients can, however, be very common in geologic environments comprised on weathered rock, such as the project site. In tomographic images, layer contacts are not clearly defined and thus, ranges of velocities are used to interpret possible rock conditions and competency.

For purpose of discussion, we assume that a Caterpillar D8R Ripper, or equivalent, will be used on site. Rock with P-wave velocity of less than approximately 6,500 ft/s should be rippable by a D8R assuming that the rock is sufficiently fractured. Rock with P-wave velocity of between about 6,500 and 8,300 ft/s should be marginally rippable by a D8R although it may be more cost effective to blast rather than rip rock in this velocity range. Rock with P-wave velocity greater than 8,300 ft/s is assumed to be non-rippable by a D8R.

Line 1 (Figure 2) has between about 2 and 8 ft of sediments or residual soil overlying weathered rock with P-wave velocity in the 2,500 to 4,500 ft/s range. Depth of investigation is about 40 ft and the sedimentary rock appears to be rippable to this depth



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Project No: 21004
Date: JAN 22, 2021
Drawn By: A MARTIN
Approved By:

R:\GVProjects\2021\21004 Geosyntec\Report\Figure 2.cdr

FIGURE 2
LINE 1 P-WAVE SEISMIC REFRACTION MODEL

28300 BRADY PARKWAY
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7 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO***Vision* California Professional Geophysicist.

Reviewed and approved by,

Antony J Martin

01/22/2021

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GEO*Vision* Geophysical Services

* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing, interpretation, and reporting. All original field data files, field notes, observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations, or ordinances