Appendix D

Design-Level Geotechnical Exploration

1265 MONTECITO AVENUE
MOUNTAIN VIEW, CALIFORNIA

DESIGN-LEVEL GEOTECHNICAL EXPLORATION

SUBMITTED TO
Ms. Kathy Robinson
Charities Housing
1400 Parkmoor Avenue, Suite 190
San Jose, CA 95126

PREPARED BY
ENGEO Incorporated

February 20, 2020

PROJECT NO.
16572.000.000

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Project No.
16572.000.000

February 20, 2020

Ms. Kathy Robinson
Director of Development
Charities Housing
1400 Parkmoor Avenue, Suite 190
San Jose, CA 95126

Subject: 1265 Montecito Avenue
Mountain View, California

DESIGN-LEVEL GEOTECHNICAL EXPLORATION

Dear Ms. Robinson:

With your authorization, we prepared this design-level geotechnical report for the subject project in Mountain View, California. This report summarizes findings from our geotechnical exploration and laboratory testing program, characterizes site conditions, and provides design-level recommendations and conclusions for earthwork activities, pavement sections, and foundation design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this design-level geotechnical report, as described in our proposal with a revised date of September 5, 2019, is to characterize and assess the geologic and geotechnical risk pertinent to the proposed development and provide design-level geotechnical recommendations for the proposed development in Mountain View, California. Our project scope of work included:

- Reviewing relevant background information, including available literature, geologic maps, and available geotechnical reports pertinent to the site.
- Performing field exploration at the subject site, consisting of advancing two soil borings, one hand-auger boring, and three cone penetration test (CPT) soundings, including one seismic CPT.
- Conduct laboratory testing on samples collected during field exploration.
- Evaluating geotechnical conditions and performing analyses of collected data.
- Preparing a geotechnical report to present the findings and conclusions of the above, and to provide recommendations for the proposed development.

This report was prepared for the exclusive use of Charities Housing and their consultants for design of this project. In the event that any changes are made in the character, design, or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended.

1.2 PROJECT LOCATION AND DESCRIPTION

The approximately 1.0-acre site is located west of the intersection at Montecito Avenue and North Shoreline Boulevard (Figure 1), in a mixed residential and commercial area of Mountain View (Figure 2). Currently, a one- to two-story commercial office building is situated centrally on the parcel, with paved parking on the west perimeter and landscaping on the eastern extent. The subject site is generally bounded by multi-family residences on the south and west perimeter, Montecito Avenue to the north, and North Shoreline Boulevard to the east.

1.3 PROPOSED DEVELOPMENT

In preparation of this report, we reviewed a conceptual plan developed by Studio-E Architects, dated October 28, 2019. After our review of the plan, we understand that proposed development includes a multi-family podium-style structure situated at-grade with paved parking and laneways. The configuration of the four-story structure will be situated at-grade along the northern and western perimeter of the property, in an “L” shaped structure. The proposed structure is comprised of three stories of residential apartments with a suspended courtyard, over at-grade parking. Proposed site parking include paved stalls and vehicle-lifts. Conceptual grading plans were not available for our review, but we anticipate grading will consist of minor cuts and fills (approximately 3 feet or less) to accommodate the development.

2.0 FINDINGS

2.1 SITE HISTORY

The subject site remained relatively undeveloped until the late 1970s, being used primarily for agricultural purposes. Through the 1960s, the region surrounding the site was incrementally developed with multi-family homes to the west and construction of North Shoreline Boulevard to the east. By 1980, the structure that currently occupies the site and paved parking can be seen, with Montecito Avenue to the north and multi-family apartments to the south. Between 1980 and 2016, the site has remained relatively unchanged.

2.2 SITE GEOLOGY

Regional mapping by Graymer (2000), as shown on Figure 3, indicates the site is underlain by Holocene basin deposits (Qhb) which generally include very fine silty clay to clay deposits. The site is mapped in proximity to Holocene-aged flood plain deposits (Qhfp) and levee deposits (Qhl) generally consisting of dense sandy to silty clay with lenses of coarser material (sand and gravel).

2.3 SEISMICITY

The project site is not located within a currently designated State of California Earthquake Fault Hazard Zone; Mountain View 7.5-Minute Quadrangle (2006) or a Santa Clara County Hazard Zone; (2015) Fault Rupture Hazard Zone, and no known active faults across the site (Figure 4). Numerous small earthquakes occur every year in the region, and large (>M7.0) earthquakes have been recorded and can be expected to occur in the future.

The region surrounding the proposed development contains numerous active earthquake faults. An active fault is defined by the State Mining and Geology Board as one that had surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997). We used the United States Geologic Survey (USGS) 2008 National Seismic Hazard Maps Fault Parameters to determine the distances of active faults located within 25 miles of the subject site. The nearest earthquake fault zoned as active by the USGS is the Monte Vista-Shannon fault, located approximately 4.5 miles to the southwest. Other active faults capable of producing significant ground shaking at the site are included in the following table.

TABLE 2.3-1: Active Faults Capable of Producing Significant Ground Shaking at the Site

FAULT NAME	DISTANCE FROM SITE (MILES)	DIRECTION FROM SITE	MOMENT MAGNITUDE (ELLSWORTH)
Monte Vista-Shannon	4.5	Southwest	6.5
San Andreas	7.1	Southwest	7.9
Hayward-Rodgers Creek	12.0	Northeast	7.3
Calaveras	15.2	Northeast	7.0
San Gregorio	18.9	Southwest	7.5

Site: Latitude = 37.401753; Longitude = -122.079882

Numerous small earthquakes occur every year in the nearby San Francisco Bay Region, and larger earthquakes have been recorded and can be expected to occur in the future.

2.4 FIELD EXPLORATION

To characterize subsurface conditions, our field exploration included advancing two soil borings, one hand-auger boring, and three cone penetrometer tests (CPT) including shear wave velocity measurements recorded in one CPT. We conducted our field exploration on January 16, 2020, and January 24, 2020. We performed the explorations in paved parking areas and landscape areas. Prior to conducting our field exploration, we were provided a preliminary site plan. Due to access restrictions from the structure currently occupying the site, our CPT and soil boring explorations are limited to the western portion of site. One hand-auger exploration was advanced in landscape areas near the north entrance of the building, where vehicle access was not permitted. We selected the exploration locations to characterize site conditions within a portion of the proposed building envelopes. The approximate locations of our explorations are presented in Figure 2. The location of our explorations are approximate and we estimated them by pacing from features; they should be considered accurate only to the degree implied by the method used. We estimated elevations using satellite imagery provided by Google Earth.

We used the field logs to develop the report logs in Appendix A. The logs depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.4.1 Soil Borings

A representative of our firm observed the drilling of two soil borings and logged the subsurface conditions at each location. We retained the services of a drilling crew operating a track-mounted drill rig and advanced borings using 8-inch-diameter mud-rotary methods. We advanced the borings to depths ranging from 31½ and 51½ feet below the existing ground surface (bgs). We permitted and backfilled the borings in accordance with the requirements of the Santa Clara Valley Water District.

We obtained soil samples at various intervals using standard penetration test (SPT) samplers with a 2-inch outside diameter (O.D. split-spoon sampler) and California Modified samplers with 2½-inch inside diameter (I.D.). We obtained the blow counts shown on our boring logs with an automatic trip, 140-pound hammer with a 30-inch free fall. We drove the sampler 18 inches and recorded the number of blows for each 6 inches of penetration. We have not converted the blow counts presented on the boring logs using any correction factors.

In addition to soil borings advanced by our drilling contractor, a representative of our firm advanced one hand-auger boring to a depth of 3 feet bgs. Our representative obtained one soil sample for index laboratory testing.

We provide additional information about specific subsurface conditions at each location in our exploration logs in Appendix A. The soil type, color, consistency, and visual classification provided in the logs are in general accordance with the Unified Soil Classification System.

2.4.2 Cone Penetration Tests

We retained the services of ConeTec to advance CPTs at three locations, to a maximum depth of approximately 100½ feet below ground surface (bgs) in general accordance with ASTM D-5778. We drilled one mud-rotary boring in proximity to 1-SCPT1 to allow direct comparison of the data (matched pair). Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). We performed shear wave velocity (Vs) measurements in Exploration 1-SCPT2. Pore pressure dissipation tests were also performed in all CPT soundings. We present the CPT logs in Appendix A, attached.

2.5 SURFACE CONDITIONS

The site is relatively flat with localized areas of change in elevation ranging from approximately 51 feet to 54 feet (WGS-84).

2.6 SUBSURFACE CONDITIONS

Our borings advanced in the parking lot encountered up to 12 inches of combined asphalt concrete pavement and aggregate base. Directly beneath the aggregate base, we encountered fat (high-plasticity) clay.

In general, this hard fat clay extends to depths of approximately 12 feet bgs, becoming stiff to very stiff between 10 and 12 feet bgs. Beneath the clay, we observed medium dense clayey sand and poorly graded sand with clay. We encountered sandy soil up to 21 feet bgs in Boring 1-B2, and up to 29 feet bgs in Boring 1-B1.

Beneath the sandy soil, we observed stiff to very stiff lean clay with varying sand content. This clay contains dense interbedded sand layers that we observed in Boring 1-B1 and 1-SCPT1 near depths of 33 to 38 feet bgs and 49 to 52 feet bgs. Beneath this sandy layer (below 52 feet bgs), we encountered clayey deposits with thick interbedded sandy layers that extended to the termination depth of 100½ feet bgs. The soil conditions encountered in our borings are consistent with geologic conditions in the mapped region and with other explorations conducted on site.

We include our boring logs and CPT results in Appendix A. The boring logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. The logs graphically depict the subsurface conditions encountered at the time of the exploration.

2.7 GROUNDWATER CONDITIONS

We performed pore pressure dissipation testing during CPT operations to estimate groundwater levels at the site. We did not encountered groundwater in either boring due to drilling method used. Results from CPT pore pressure dissipation show groundwater measurements recorded at depths ranging from 9 to 10 feet bgs.

Plate 1.2 of the Seismic Hazard Zone Report for the Mountain View Quadrangle (2006) shows historic groundwater at approximately 5 to 10 feet bgs in the vicinity of the site. Our review of available documents also includes publicly available groundwater data. Historical well data from the California State Resources Control Board Geotracker website indicate groundwater levels in the surrounding area range from approximately 5 to 9 feet bgs.

Based on the available information, we estimate groundwater varies between 5 to 10 feet bgs at the subject property. For purposes of our analyses and recommendations, we consider appropriate design groundwater at 5 feet bgs (approximately El. 46 feet, WGS-84). Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

2.8 LABORATORY TESTING

We performed the following laboratory tests on select samples recovered during boring operations.

TABLE 2.8-1: Laboratory Testing

SOIL TEST	TESTING METHOD	LOCATION OF RESULTS
Unit Weight and Moisture Content	ASTM D7263	Appendix A
Isotropic Unconsolidated Undrained Triaxial	ASTM D2850	Appendix C
Fines Content	ASTM D1140	Appendix C
Unconfined Compression	ASTM D2166	Appendix C
Redox	ASTM D1498	Appendix D
pH	ASTM D4972	Appendix D
Resistivity	ASTM G57	Appendix D
Sulfide	ASTM D4658M	Appendix D
Chloride	ASTM D4327	Appendix D
Sulfate	ASTM D4327	Appendix D

Some laboratory test results are included on the borelogs in Appendix A. Individual test results are presented in Appendix C.

In addition to the above-listed laboratory tests, a shallow soil sample collected from Boring 1-B1 was submitted to CERCO Analytical under a chain-of-custody for corrosivity testing. Results from CERCO Analytical are summarized in Section 3.6 and included in Appendix D.

3.0 DISCUSSION AND CONCLUSIONS

Based on the exploration and laboratory test results, the primary geotechnical concerns for the proposed site redevelopment are as follows.

- The presence of shallow groundwater.
- Relatively high seismicity.
- Expansive soil with potential for shrink/swell.

These and other issues such as corrosive soil are discussed below.

3.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections.

3.1.1 Ground Rupture

The site is not located within a State of California Earthquake Fault Hazard Zone and no known active faults cross the site. Therefore, it is our opinion that ground rupture is unlikely at the subject site.

3.1.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region, similar to those that have occurred in the past, could cause considerable ground shaking at the site. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements as a minimum.

Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally substantially smaller than the expected peak forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural, as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that well-designed and well-constructed structures will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.1.3 Liquefaction / Cyclic Softening

Review of the Seismic Hazard Zones Map for the Milpitas Quadrangle (CGS, 2006) indicates that the site is located within a mapped liquefaction zone (Figure 5).

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. Empirical evidence indicates that loose silty sand is also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective confining stress from the overlying soil, the sand may undergo deformation. If the sand undergoes virtually unlimited deformation without developing significant resistance, it is said to have liquefied, and if the sand consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur. In addition to liquefaction of sandy materials, clayey soil can also undergo “cyclic-softening” or strength loss as a result of cyclic loading.

We performed an analysis of liquefaction potential based on the CPT data using the software package CLiq (Version 2.1) developed by GeoLogismiki. The software incorporates the procedure introduced by the 1996 National Center for Earthquake Engineering Research (NCEER) workshop and the 1998 NCEER/National Science Foundation (NSF) workshop. The workshops are summarized by Youd et al. (2001) and updated by Robertson (2009). This software also incorporates the method introduced by Idriss and Boulanger (2008) and updated by Boulanger and Idriss (2014). For our analysis, we used a Peak Ground Acceleration (PGA_M) value of 0.73g with an earthquake magnitude of 7.90 and a groundwater depth of 5 feet. We performed the liquefaction assessment based on the methodology by Robertson (2009). To assess seismically induced settlement, we considered the methodology presented by Zhang et al. (2002).

Based on our analysis, we estimate that a maximum of 1½ inches of total liquefaction-induced settlement (approximately ¾-inch differential over 30-foot span) may occur during a maximum considered event (MCE) earthquake. We present the results of the liquefaction analysis in Appendix B with our estimation of post-earthquake settlements. The analysis sheets in

Appendix B summarize the CPT tip resistance, computed factor of safety, volumetric strain, and resulting settlement as a function of depth for each CPT.

3.1.4 Liquefaction-Induced Surface Rupture

In addition to the above analysis, we also evaluated the capping effect of any overlying non-liquefiable soil. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a sufficient force to break through the overlying soil and vent to the surface resulting in sand boils or fissures. This loss of soil through venting could potentially increase settlement by two times.

In 1985, Ishihara presented preliminary empirical criteria to assess the potential for ground surface disruption at liquefiable sites based on the relationship between thickness of liquefiable sediments and thickness of overlying non-liquefiable soil. A more recent study by Youd and Garris (1995) expanded on the work of Ishihara to include data from over 308 exploratory borings, 15 different earthquakes, and several ranges of recorded peak ground acceleration.

Based on the above studies and thickness of non-liquefiable material, it appears the surficial fat clay deposits provide a sufficient capping effect against surface rupture.

3.1.5 Dynamic Densification

Densification of loose granular soil above the water table can cause settlement of the ground surface due to earthquake-induced vibrations. As described in Section 2.6, the subsurface conditions generally consisted of clayey soil encountered above the assumed groundwater level. Based on observed conditions, we anticipate densification of the unsaturated soil to be negligible.

3.1.6 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone (possibly due to liquefaction) that causes the overlying soil mass to move toward a free face or down a gentle slope. Generally, effects of lateral spreading are most significant at the free face or the crest of a slope and diminish with distance from the slope.

Due to relatively flat to gentle sloping ground conditions observed in the surrounding project area, we consider the risk of lateral spreading at the project site to be very low.

3.2 EXISTING FILL

While construction debris and/or foreign object fragments were not specifically observed in our explorations, based on the current conditions, including a building and associated site improvements, it is likely that existing fill deposits are present at the site underlying existing pavement around buildings, and along utility trenches, landscape areas, and possibly buried structures.

Existing non-engineered fill could undergo vertical movement that is not easily characterized and could ultimately be inadequate to effectively support the proposed building loads. At the current time, we do not have records indicating the potential existing fill was engineered and monitored during placement consistent with standards consistent with the proposed project. If existing fill is encountered during construction, ENGEO should be notified to evaluate whether the existing fill is engineered. For budgeting purposes, we recommend considering the upper 12 inches of the site to be non-engineered. Non-engineered fill can undergo additional settlement under new fill or building loads. To reduce the risk of settlement, the existing fill should be removed and recompacted in accordance with compaction specifications in this report. The extent and quality of existing fill should be evaluated and mitigated during grading activities.

3.3 COMPRESSIBLE SOIL AND STATIC SETTLEMENT

Soil may be subject to settlement when loaded with a new structure or fill. As discussed above, we encountered up to 12 feet of stiff to hard clay across two exploration locations. Clayey soil samples were obtained and tested for strength, moisture content, and plasticity, as deemed appropriate.

Based on laboratory testing, the near-surface clay discussed in Section 2.6 may be slightly compressible when new loading is introduced. Provided our recommendations regarding fill placement are followed, we anticipate the majority of load-induced settlements will occur during construction, and post-construction settlement will be negligible for the proposed development. Design recommendations for fill placement are provided in Section 5.3.

3.4 EXPANSIVE SOIL

Highly expansive clay soil was encountered at the subject site. The lab results yielded plasticity indexes (PI) of 39 to 42 in the upper 6 feet, indicating the near-surface soil exhibits high shrink/swell potential.

Where encountered, expansive soil can shrink and swell as a result of moisture changes. This can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. We recommend special attention during construction in structural areas as well as areas with ridged surface improvements. It is imperative that exposed soil be kept moist prior to placement of concrete for foundation construction. It can be difficult to remoisturize clayey soil without excavation, moisture conditioning, and recompaction.

Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil, are a generally cost-effective measure to address the expansive potential of the foundation soil. We provide specific grading recommendations for compaction of the high-expansive clay soil at the site.

3.5 2019 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS

The 2019 CBC utilizes design criteria set forth in the 2016 ASCE 7-16 Standard. We used in-situ shear wave velocity measurements from our seismic CPT (1-SCPT1) to estimate the average shear wave velocity of the upper 100 feet of site soil; the average shear wave velocity in our measurement is approximately 782 feet per second, which classifies as a Site Class D soil. We provide the 2019 CBC seismic design parameters in Table 3.5-1 below, which includes design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered earthquake (MCER) spectral response acceleration parameters.

TABLE 3.5-1: 2019 CBC Seismic Design Parameters, Latitude: 37.401753**, Longitude:** -122.079882

PARAMETER	DESIGN VALUE
Site Class	D
Mapped MCE$_R$ spectral response accelerations for short periods, S_S (g)	1.62
Mapped MCE$_R$ spectral response accelerations for 1-second periods, S_1 (g)	0.60
Site Coefficient, F_A	1.00
Site Coefficient, F_V	Null*
MCE spectral response accelerations for short periods, S_{MS} (g)	1.62
MCE spectral response accelerations for 1-second periods, S_{M1} (g)	Null*
Design spectral response acceleration at short periods, S_{DS} (g)	1.08
Design spectral response acceleration at 1-second periods, S_{D1} (g)	Null*
Mapped MCE Geometric Mean Peak Ground Acceleration (g)	0.66
Site Coefficient, F_{PGA}	1.10
MCE Geometric Mean Peak Ground Acceleration, PGA_M (g)	0.73
Long period transition-period, T_L	12 sec

*Required site-specific ground motion hazard analysis per ASCE 7-16 Section 11.4.8

Considering the relatively low-rise multi-family residential development, we estimate the fundamental periods of the proposed structures to be less than 1.5T_s (where T_s is 0.63 second for this project based on tabulated values). Therefore, the structural engineer may consider exception of Section 11.4.8 of ASCE 7-16 as follows:

> "A ground motion hazard analysis is not required for structures… where, structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) of ASCE 7-16 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) of ASCE 7-16 for $1.5T_s < T \leq T_L$."

If the noted exception is not used, a ground motion hazard analysis can be provided upon request in a separate letter.

3.6 CORROSIVITY CONSIDERATIONS

A near-surface soil sample was collected and transported to CERCO Analytical, Inc. for laboratory testing. The sample was tested for redox potential, pH, resistivity, soluble sulfate, and chloride ion concentrations. An additional sample was collected at depth for sulfate testing. This sample was tested to evaluate concrete considerations for the subgrade parking garage.

The results are summarized below with the laboratory test results prepared by CERCO Analytical, Inc. included in Appendix D.

TABLE 3.6-1: Soil Corrosivity Test Results

SAMPLE NUMBER AND DEPTH (FEET)	REDOX POTENTIAL (MV)	PH	RESISTIVITY (OHM-CM)	SOLUBLE SULFATE* (MG/KG)	CHLORIDE ION* (MG/KG)	SULFIDE (MG/KG)
1-B1 @ 1½ - 2ft	270	8.05	2,200	300	ND (< 15)	ND (< 50)

*Results reported on a wet weight basis

Based on the resistivity measurements, the near-surface soil in the vicinity of 1-B1 in the south-central portion of the site is considered severely corrosive to buried metals. As such, all buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric steel or iron should be properly protected depending on the critical nature of the structure. A corrosion consultant should provide specific design recommendations on corrosion protection for important buried metallic lines.

The sulfate ion concentration was reported as non-detectable which is up to 300 mg/kg of water-soluble sulfate (SO_4). The 2019 California Building Code (CBC) references the American Concrete Institute Manual, ACI 318-14 (Chapter 19) for concrete requirements. ACI Table 19.3.1.1 for sulfate was summarized in the following Table 3.6-2, which presents the sulfate exposure category and classes.

TABLE 3.6-2: ACI Table 19.3.1.1: Exposure Categories and Classes

CATEGORY	SEVERITY	CLASS	CONDITION	
F Freezing and thawing	Not Applicable	F0	Concrete not exposed to freezing-and-thawing cycles	
	Moderate	F1	Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture	
	Severe	F2	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture	
	Very Severe	F3	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture and exposed to deicing chemicals	
			WATER- SOLUBLE SULFATE IN SOIL % BY WEIGHT*	**DISSOLVED SULFATE IN WATER MG/KG (PPM)****
S Sulfate	Not applicable	S0	$SO_4 < 0.10$	$SO_4 < 150$
	Moderate	S1	$0.10 \leq SO_4 < 0.20$	$150 \leq SO_4 \leq 1{,}500$ seawater
	Severe	S2	$0.20 \leq SO_4 \leq 2.00$	$1{,}500 \leq SO_4 \leq 10{,}000$
	Very severe	S3	$SO_4 > 2.00$	$SO_4 > 10{,}000$
			CONDITION	
P Requiring low permeability	Not applicable	P0	In contact with water where low permeability is not required.	
	Required	P1	In contact with water where low permeability is required.	
C Corrosion protection of reinforcement	Not applicable	C0	Concrete dry or protected from moisture	
	Moderate	C1	Concrete exposed to moisture but not to external sources of chlorides	
	Severe	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources	

* Percent sulfate by mass in soil determined by ASTM C1580
**Concentration of dissolved sulfates in water in ppm determined by ASTM D516 or ASTM D4130

Considering a ‘Not Applicable’ sulfate exposure, there is no requirement for cement type or water-cement ratio; however, a minimum concrete compressive strength of 2,500 psi is specified by the building code. For this sulfate range, we recommend Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

4.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design, or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to construction is important.

If we are not retained to perform the services described above, then we are not responsible for any party’s interpretation of our report and subsequent addenda, letters, and verbal discussions.

5.0 EARTHWORK RECOMMENDATIONS

As used in this report, relative compaction refers to the in-place dry unit weight of soil expressed as a percentage of the maximum dry unit weight of the same soil, as determined by the ASTM D1557 laboratory compaction test procedure, latest edition. Compacted soil is not acceptable if it is unstable; it should exhibit only minimal flexing or pumping, as observed by an ENGEO representative. The term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

5.1 DEMOLITION AND STRIPPING

Grading operations should be observed and tested by our qualified field representative. We should be notified a minimum of three days prior to grading in order to coordinate our schedule with the grading contractor.

Site development should commence with the removal of existing building and pavement as well as excavation and removal of buried structures, including utilities and foundations. All debris and soft compressible soil should be removed from any location to be graded, from areas to receive fill or structures, and from areas to serve as borrow. The depth of removal of such materials should be determined by our representative in the field at the time of grading.

Existing vegetation should be removed from areas to receive fill or improvements and those areas to serve for borrow. Tree roots should be removed down to a depth of at least 3 feet below existing grade. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations. All excavations from demolition below design grades should be cleaned to a firm undisturbed native soil surface determined by our representative. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. All backfill materials should be placed and compacted as engineered fill according to the recommendations in Section 5.4.

5.2 SELECTION OF MATERIALS

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soil, we anticipate the site soil is suitable for use as engineered fill. Unsuitable materials and debris, including trees with their roots and particles larger than 6 inches, should be removed from the project site.

Subject to approval by the Landscape Architect, organically contaminated soil may be stockpiled in approved areas located outside of the grading limits for future placement within landscape areas. Oversized soil or rock materials (those exceeding two-thirds of the lift thickness or 6 inches in dimension, whichever is less) should be removed from the fill and broken down to meet this requirement or otherwise off-hauled.

The Geotechnical Engineer should be informed when import materials are planned for the site. Import materials should be submitted to and approved by the Geotechnical Engineer prior to delivery at the site and should conform to the requirements provided in the Supplemental Recommendations in Appendix E.

5.3 STRUCTURAL BUILDING PAD TREATMENT

As described in Section 3.4, near-surface soil is highly expansive. To improve foundation performance for the planned residential structure, we recommend that the near-surface soil is comprised of uniform engineered fill. For a mat foundation system or footings with slab-on-grade foundation founded on native expansive soil (PI>20), we recommend the upper 24 inches of foundation subgrade to consist of engineered fill compacted in accordance with recommendations provided in Section 5.4.1. The engineered fill cap should extend at least 5 feet beyond the building footprint. Alternatively, the upper 18 inches of foundation subgrade can be over-excavated and replaced with low-expansive engineered fill. Import soil with a Plasticity Index less than 12 or chemically treated site soil is acceptable low-expansive fill. Based on our experience, chemical treatment of the highly expansive clay may require with 3 to 5 percent high calcium lime. The amount of lime required should be based upon an assumed 125 pounds per cubic foot (pcf) for the soil density.

5.4 FILL PLACEMENT

5.4.1 Placement and Compaction in Structural Areas

We should be present during all phases of grading operations to observe demolition, site preparation and grading operations. Areas to receive fill should be excavated to a firm undisturbed surface, scarified to a depth of 8 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. All fills should be placed in thin compacted lifts that do not exceed 12 inches or the depth of penetration of the compaction equipment used, whichever is less. Track rolling to compact faces of slopes is usually not sufficient; typically, slopes should be overfilled a minimum of 2 feet and cut back to design grades. We recommend the following compaction and moisture content requirements for the placement and compaction of engineered fills.

TABLE 5.4.1-1: Fill Compaction and Moisture Content Recommendations

MATERIALS	FILL LOCATION	MINIMUM RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (PERCENTAGE POINTS ABOVE OPTIMUM MOISTURE CONTENT)
Site Soil - Expansive (PI>20)	General Fill	87 to 92	3
Import Low Expansive Fill (PI<12) and Chemically Treated Site Soil	General Fill	90	2
Site Soil	Pavement Subgrade*	93	3
Import Material - Low-Expansive (PI<12)	Pavement Subgrade*	95	1
Class 2 Aggregate Base	Pavement Section	95	0

*Specifies requirements for upper 6-Inches of placed fill.

Compact the upper 6 inches of pavement subgrade to a minimum of 93 percent relative compaction for expansive subgrade conditions and a minimum of 95 percent relative compaction for low-expansive subgrade conditions. Compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to or slightly above the optimum moisture content prior to compaction.

5.4.2 Underground Utility Backfill

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe-bedding materials. Place and compact trench backfill in structural areas in accordance with Section 5.4.1. Where utility trenches cross perimeter building foundations, backfill with native clay soil for pipe bedding and backfill for a distance of 2 feet on each side of the foundation. This will help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the building. As an alternative, a sand cement slurry (minimum 28-day compressive strength of 500 psi) may be used in place of native clay soil. Jetting of backfill is not an acceptable means of compaction. We may allow thicker loose lift thicknesses based on acceptable density test results, where increased effort is applied to rocky fill or for the first lift of fill over pipe bedding.

5.4.3 Landscape Fill

Process, place, and compact fill in accordance with Section 5.4.1 except compact to at least 85 percent relative compaction (ASTM D1557).

5.5 DIFFERENTIAL FILL THICKNESS

Depending upon cuts associated with removal of undocumented fills and planned cuts and fills, differential fill thickness conditions may occur. For subexcavation activities that create a differential fill thickness across the building footprint, mitigation to achieve a similar fill thickness across the pad is beneficial for the performance of a shallow foundation system. We recommend that a maximum differential fill thickness of 10 feet is acceptable across a building footprint provided that it is a gradual transition. For a differential fill thickness exceeding 10 feet across a footprint, we recommend performing subexcavation activities to bring this vertical distance to within the 10-foot tolerance and that the material placed back as engineered fill. As a minimum, the subexcavation area should include the entire structure footprint plus 5 feet horizontal beyond the edges of the building footprint.

5.6 SLOPE GRADIENTS

In general, graded slopes should be no steeper than 2:1 (horizontal:vertical) and up to 8 feet high. All fill slopes should be adequately keyed into firm materials unaffected by shrinkage cracks. If a cut or cut-fill transition occurs within a graded slope, we recommend that it be overexcavated and reconstructed as an engineered fill slope.

5.7 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. We also recommend infiltration be restricted to prevent introducing collected runoff to subgrade with low permeability and to limit excessive sheet flow. As a minimum, we recommend the following:

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.

2. Consider the use of rear lot surface drainage collection systems to reduce overland surface drainage from back to front of lot.

3. Do not allow water to pond near foundations, pavements, or exterior flatwork.

6.0 FOUNDATION RECOMMENDATIONS

The main considerations in foundation design for this project is the potential for high shrink /swell behavior of highly expansive soil and appropriate foundation support for the relatively high loads. We developed foundation recommendations using data obtained from our exploration and engineering analysis.

Based on the near-surface soil conditions encountered during our exploration and the relatively high building loads associated with the proposed construction, shallow foundations are suitable for support of the planned structure. Due to presence of expansive soil, measures are recommended in subsequent sections to be implemented for the use of shallow footings. Suitable foundations for the building include shallow footings combined with floor slab-on-grade, underlain by an 18-Inch layer of low-expansive import engineered fill, or 18 inches of lime-treated fill materials. The building may also be supported on a steel-reinforced structural mat foundation, designed to accommodate movement of near-surface expansive soil. The building pad for structural mat should be overexcavated, moisture conditions, and recompacted as engineered fill per recommendations provided in Section 5.3.

6.1 SHALLOW FOOTINGS COMBINED WITH FLOOR SLAB-ON-GRADE

For the foundation of the proposed building, shallow footings combined with floor slab-on-grade are suitable. Building floor slab-on-grade should be underlain by an 18-inch-thick layer of non- to low-expansive engineered fill. Provide minimum footing dimensions as follows in the Table 6.1-1 below.

TABLE 6.1-1: Minimum Footing Dimensions

FOOTING TYPE	*MINIMUM DEPTH (INCHES)	MINIMUM WIDTH (INCHES)
Continuous	**30**	**12**
Isolated	**30**	**12**

* below lowest adjacent pad grade

Minimum footing depths shown above are taken from lowest adjacent pad grade. The cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent exterior grade. Design foundations recommended above for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for dead-plus-live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing.

If a two-pour system is used for footings and slab, the cold joint between the exterior footing and slab-on-grade should be located at least 4 inches above adjacent finish exterior grade. If this is not done, then we recommend the addition of a “waterstop” between a long joint to reduce moisture intrusion.

The Structural Engineer should design footing reinforcement to support the intended structural loads without excessive settlement. Reinforce continuous footings with top and bottom steel to provide structural continuity and to permit spanning of local irregularities. At a minimum, design continuous footings to structurally span a clear distance of 5 feet. Also, to help resist expansive soil movement, reinforce continuous footings with at least four No. 4 steel reinforcement bars, two top and two bottom.

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design:

- Passive Lateral Pressure: 250 pcf
- Coefficient of Friction: 0.30

The above allowable values include a factor of safety of 1.5. Increase the above values by one-third for the short-term effects of wind or seismic loading. Passive lateral pressure should not be used for footings on or above slopes.

6.2 STEEL REINFORCED MAT FOUNDATION

As a minimum, to address potential differential movement from expansive soil, we recommend the mat be designed to cantilever 5 feet at the perimeter and interior free span of 15 feet, provided the foundation subgrade consists of engineered fill.

The structural mat foundation should be designed to impose an average allowable bearing pressure of at most 1,500 pounds per square foot (psf) for dead-plus-live loads. Allowable bearing pressures of 2,000 psf can be used for concentrated line or column dead-plus-live loads. These values may be increased by one-third when considering transient loads, such as wind or seismic. If a spring constant is needed for design, a modulus of subgrade reaction (ks) of 100 pounds per square inch per inch of deflection (psi/in) may be used.

Resistance to lateral loads may be provided by frictional resistance between the foundation concrete and the subgrade soil and by passive earth pressure acting against the side of the foundation. A coefficient of friction of 0.30 can be used between concrete and the subgrade. Passive pressures can be taken as equivalent to the pressure developed by a fluid having a weight of 250 pounds per cubic foot (pcf).

6.2.1 Subgrade Treatment for Mat Foundations

The subgrade material under structural mat foundations should be uniform as discussed in Section 5.3. The pad subgrade should be moisture conditioned to the optimum moisture content provided in Table 5.4.1 for the representative soil type. The moisture-conditioned subgrade should be approved by the Geotechnical Engineer prior to placing the reinforcement or tendons and should not be allowed to dry prior to concrete placement.

For occupied rooms with floor coverings located on the ground floor, a tough, water vapor retarding membrane could be installed below the mat foundations to reduce moisture condensation under floor coverings. The vapor retarder should meet ASTM E 1745 Class A requirements for water vapor permeance, tensile strength, and puncture resistance. Vapor transmission through the mat foundations can also be reduced by using high strength concrete with a low water-cement ratio.

7.0 SLABS-ON-GRADE

Provided the expansive soil is mitigated as recommended in Section 5.3, the proposed building can incorporate interior slab-on-grade first floor.

7.1 INTERIOR CONCRETE FLOOR SLABS

Floor slab-on-grade underlain by a 18-inch-thick layer of non- to low-expansive engineered fill, or 18 inches of lime-treated fill materials. We recommend the following minimum design:

1. Provide a minimum concrete thickness of 5 inches.

2. Place minimum steel reinforcing of No. 3 rebar on 18-inch centers each way within the middle third of the slab to help control the width of shrinkage cracking that inherently occurs as concrete cures.

The structural engineer should provide final design thickness and additional reinforcement, as necessary, for the intended structural loads.

Water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Construct a moisture retarder system directly beneath the slab on-grade that consists of the following:
   a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs". The vapor retarder should be **underlain by**

   b. 4 inches of clean crushed rock. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve.

2. Use a concrete water-cement ratio for slabs-on-grade of no more than 0.50.

3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.

4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specified by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed on top of the vapor retarder membrane to assist in concrete curing.

8.0 RETAINING WALLS

Retaining walls may be supported on continuous footings designed in accordance with recommendations presented in Section 6.1, except the minimum embedment depth should be increased to 18 inches below lowest adjacent soil grade.

Design proposed retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, design walls restrained from movement at the top to resist an equivalent fluid pressure of 60 pounds per cubic foot (pcf). In addition, design restrained walls to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface.

Design unrestrained retaining walls with adequate drainage to resist an equivalent fluid pressure of 40 pcf plus one-third of any surcharge loads. Any retaining walls taller than 6 feet or that are within a 1:1 distance from the bottom of the footing of a structure, should be designed for seismic conditions in accordance with the 2016 CBC.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for both restrained and unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic. Construct a drainage system, as recommended below, to reduce hydrostatic forces behind the retaining wall.

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives:

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or

2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

- Place the rock drain directly behind the walls of the structure.

- Extend rock drains from the wall base to within 12 inches of the top of the wall.

- Place a minimum of 4-inch-diameter perforated pipe (glued joints and end caps) at the base of the wall, inside the rock drain and fabric, with perforations placed down.

- Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use. Backfill behind retaining walls should be placed and compacted in accordance with Section 5.4.1. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

9.0 PAVEMENT AND SECONDARY SLAB-ON-GRADE DESIGN

Preliminary pavement design is provided based on assumed Traffic Index and subgrade resistance values (R-value). The Traffic Index should be determined by the Civil Engineer or appropriate public agency. The sections provided below should be reviewed and revised, if applicable, based on R-value tests performed on samples of actual subgrade materials recovered at the time of grading.

9.1 FLEXIBLE PAVEMENTS

Using traffic indices for various pavement-loading requirements and an assumed R-value of 5, we developed the following preliminary pavement section recommendations using Topic 633 of the Caltrans Highway Design Manual, presented in the table below.

TABLE 9.1-1: Preliminary Flexible Pavement Design

TRAFFIC INDEX (TI)	AB (INCHES)	AC (INCHES)
5.0	10	3
6.0	12½	3½
7.0	15½	4

Notes: AC is asphalt concrete
AB is aggregate base Class 2 Material with minimum R = 78

The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies. Pavement construction and all materials should comply with the requirements of the Standard Specifications of the State of California Department of Transportation, Civil Engineer, and the appropriate public agency.

9.2 RIGID PAVEMENTS

We developed recommended rigid pavement sections according to the methodology presented in American Concrete Institute (ACI) report 330R-08, Guide for the Design and Construction of Concrete Parking Lots (2008), based on the assumed subgrade soil type. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 6 inches of Portland Cement concrete over 6 inches of Class 2 aggregate base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

9.3 PAVEMENT SUBGRADE PREPARATION

Pavement construction and all materials (hot mix asphalt and aggregate base) should comply with the requirements of the Standard Specifications of the State of California Division of Highways, City of San Jose requirements and the following minimum requirements.

- All pavement subgrades should be scarified to a depth of 12 inches below finished subgrade elevation, moisture conditioned, and compacted in accordance with the fill placement recommendations presented in Section 5.4.
- Subgrade soil should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted. Proof rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated, with suitable mitigation measures developed in coordination with the client, contractor, and Geotechnical Engineer.
- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted to at least 95 percent of the maximum dry density at a moisture content of at least optimum. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented after placement and compaction of the aggregate base. Yielding materials should be appropriately mitigated, with suitable

mitigation measures developed in coordination with the client, contractor, and Geotechnical Engineer.

- Adequate provisions must be made such that the subgrade soil and aggregate baserock materials are not allowed to become saturated.

- All concrete curbs separating pavement and irrigated landscaped areas, if applicable, should extend into the subgrade and below the bottom of adjacent aggregate baserock materials. An undercurb drain could also be considered to help collect and transport subsurface seepage.

9.4 SECONDARY SLAB-ON-GRADE CONSTRUCTION

This section provides guidelines for secondary slabs such as walkways, sidewalks, and steps. Secondary slabs-on-grade should be constructed structurally independent of adjacent foundation systems. This allows slab movement to occur with a minimum of foundation distress. Where secondary slab-on-grade construction is anticipated, a layer of low- to non-expansive fill should be used at near-saturated conditions, of a minimum of the upper 12 inches. If expansive material is selected, care must be exercised in attaining a minimum of 4 percentage points above optimum moisture of a minimum of the upper 12 inches of subgrade soil before concrete placement.

Secondary slabs-on-grade should be designed by the Structural Engineer specifically for their intended use and loading requirements. Cracking of conventional slabs should be expected as a result of concrete shrinkage. Slabs-on-grade may be reinforced for control of cracking, and frequent control joints should be provided to control the cracking. Reinforcement should be designed by the structural engineer. In our experience, welded wire mesh may not be sufficient to control slab cracking. Consider a thickness of 4 inches for secondary slabs-on-grade underlain by a 4-inch-thick layer of clean crushed rock or gravel.

10.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the proposed development and improvements discussed in Section 1.3. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than two years from the date of report issuance.

We strived to perform our professional services according to generally accepted principles and practices currently employed in the area; there is no warranty, either express implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based primarily upon field explorations and laboratory data discovered at the time of report preparation. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, then the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse; that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications, or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications, or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies, or other changes necessary to reflect changed field or other conditions.

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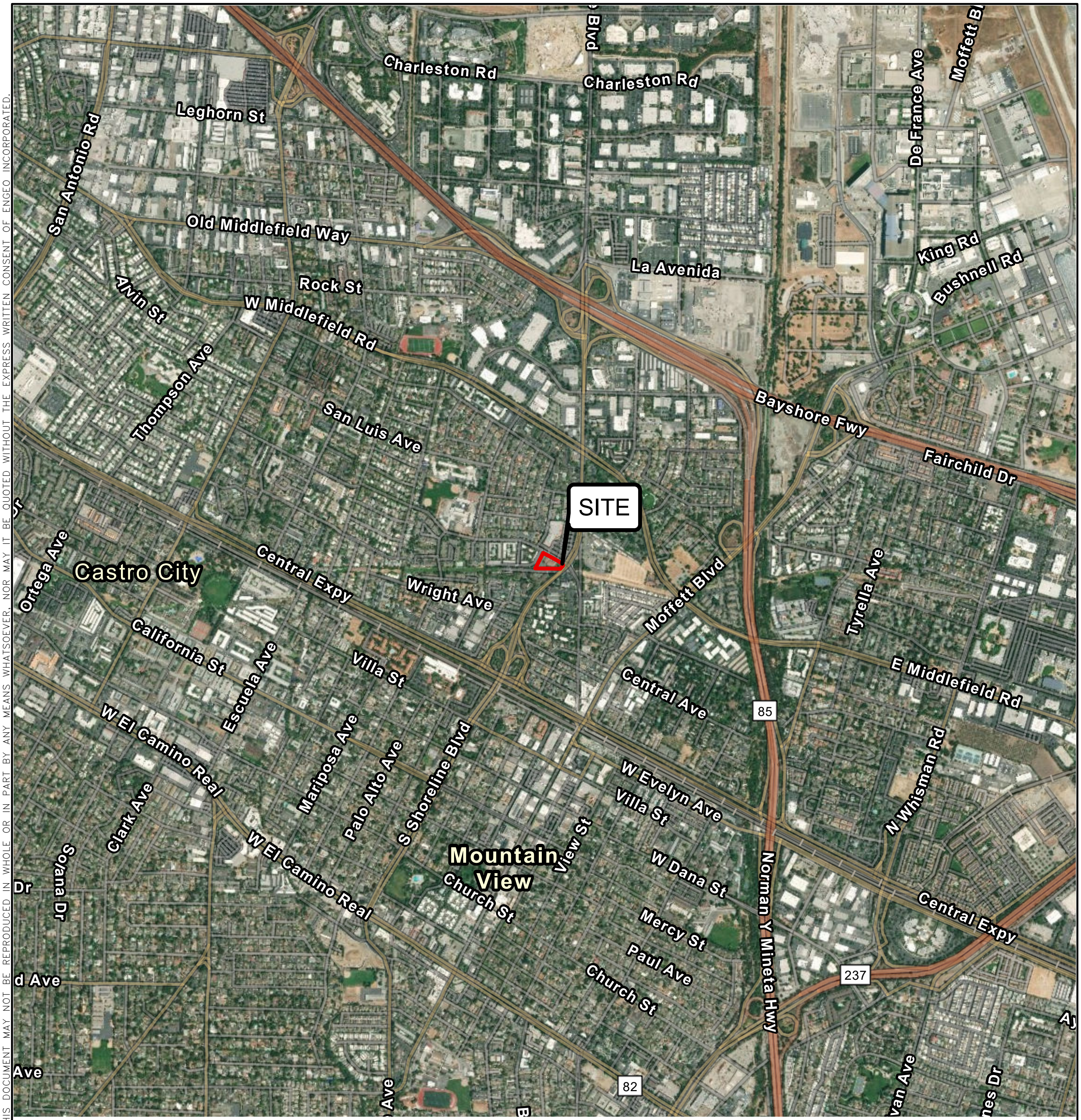
FIGURES



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BASEMAP SOURCE: ESRI MAPPING SERVICE 2017

ENGEO
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VICINITY MAP 1265 MONTECITO AVENUE MOUNTAIN VIEW, CALIFORNIA	PROJECT NO. : 16572.000.000		FIGURE NO.
	SCALE: AS SHOWN		1
	DRAWN BY: QRL	CHECKED BY: RHB	

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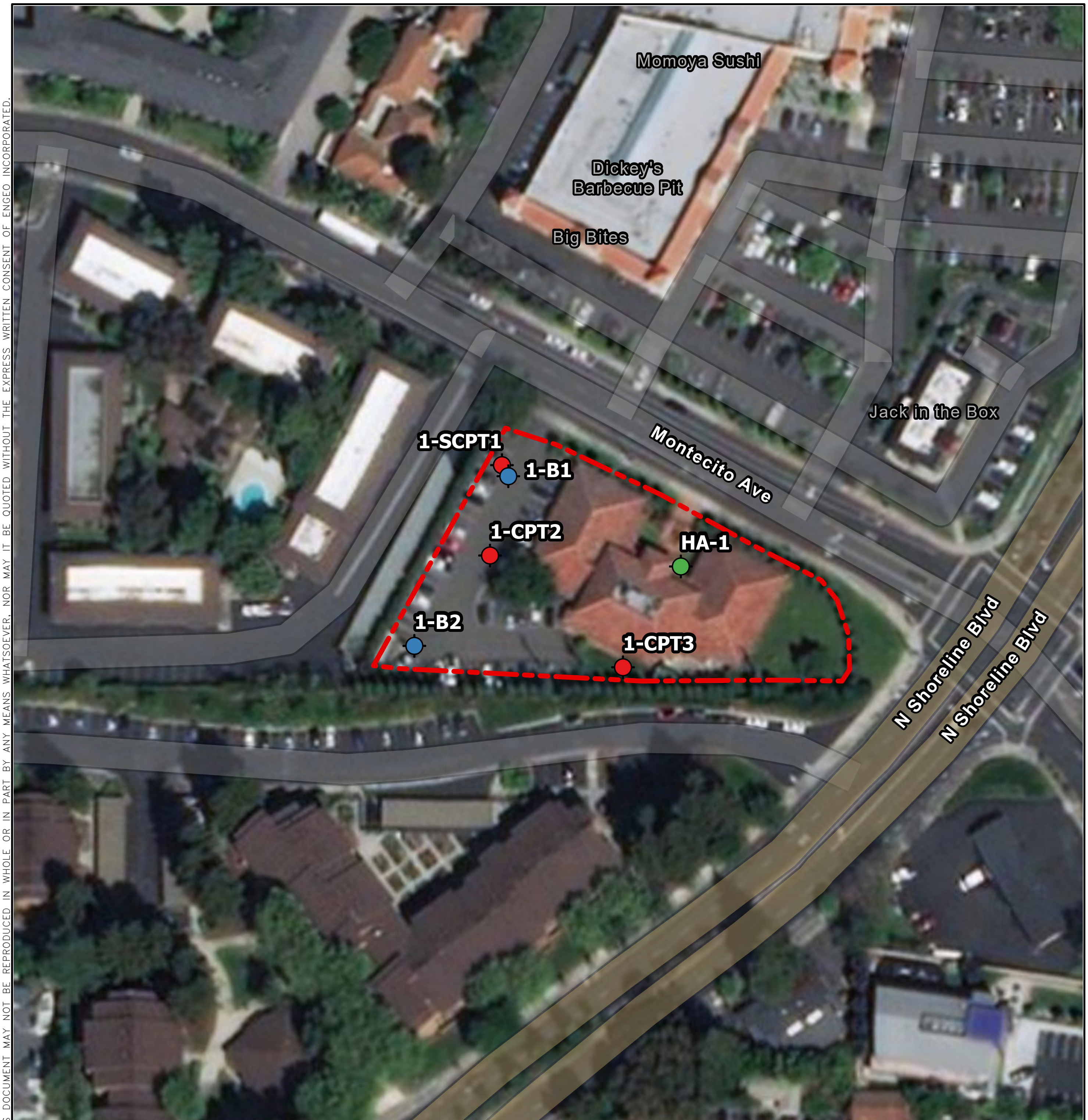
ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- PROJECT SITE
- CPT (ENGEO, 2020)
- BORING (ENGEO, 2020)
- HAND AUGER BORING (ENGEO, 2020)

BASEMAP SOURCE: ESRI MAPPING SERVICE 2017

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SITE PLAN
1265 MONTECITO AVENUE
MOUNTAIN VIEW, CALIFORNIA

PROJECT NO. : 16572.000.000		FIGURE NO. 2
SCALE: AS SHOWN		
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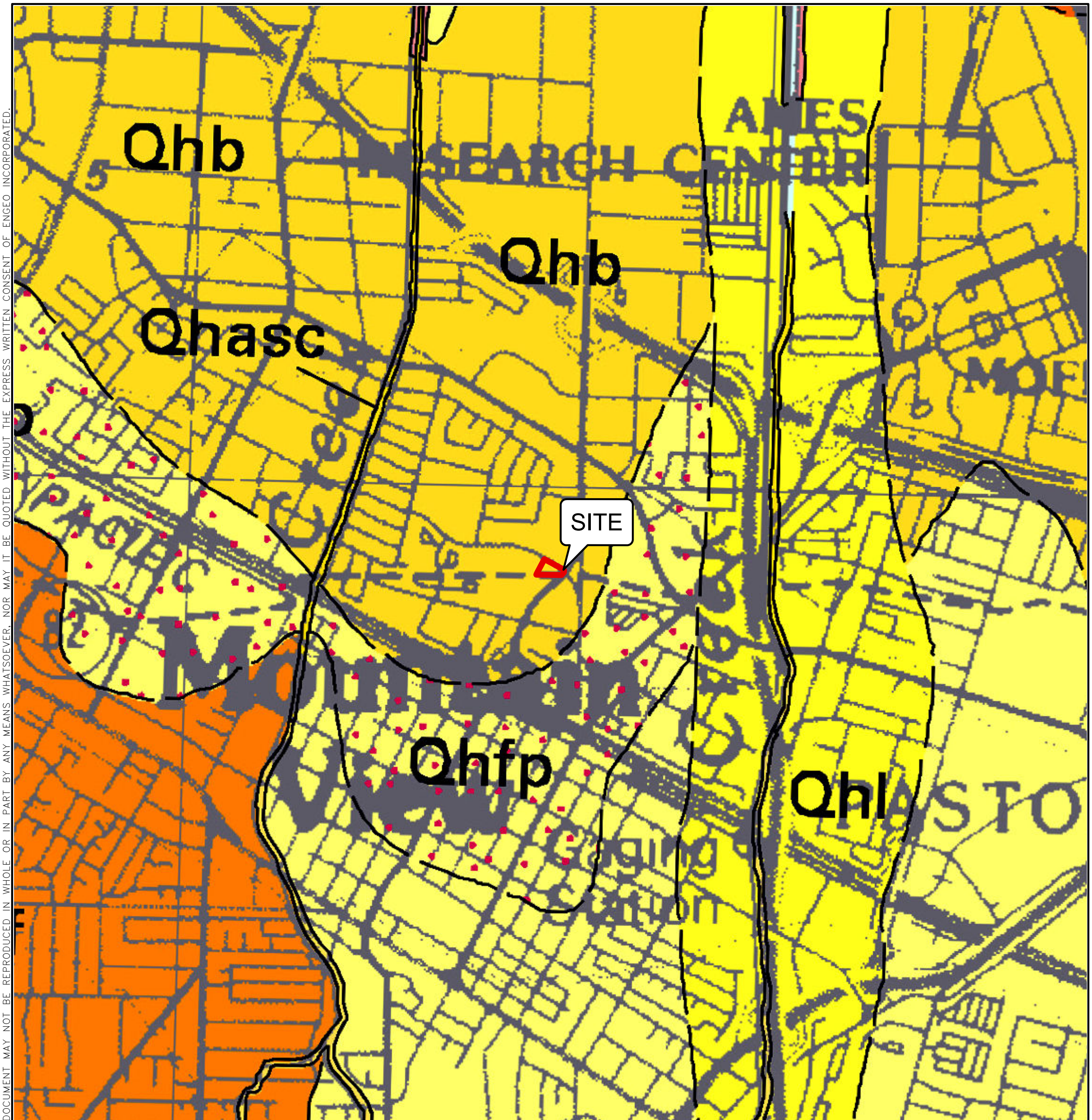

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qhasc	ARTIFICIAL STREAM CHANNELS (HISTORIC)
Qhb	BASIN DEPOSITS (HOLOCENE)
Qhfp	FLOOD-PLAIN DEPOSITS (HOLOCENE)
Qhl	NATURAL LEVEE DEPOSITS (HOLOCENE)
Qpaf	ALLUVIAL FAN AND FLUVIAL DEPOSITS (PLEISTOCENE)

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BASEMAP SOURCE: GRAYMER 2000

ENGEO Expect Excellence

REGIONAL GEOLOGIC MAP 1265 MONTECITO AVENUE MOUNTAIN VIEW, CALIFORNIA	PROJECT NO.: 16572.000.000 SCALE: AS SHOWN DRAWN BY: QRL CHECKED BY: RHB	FIGURE NO. 3

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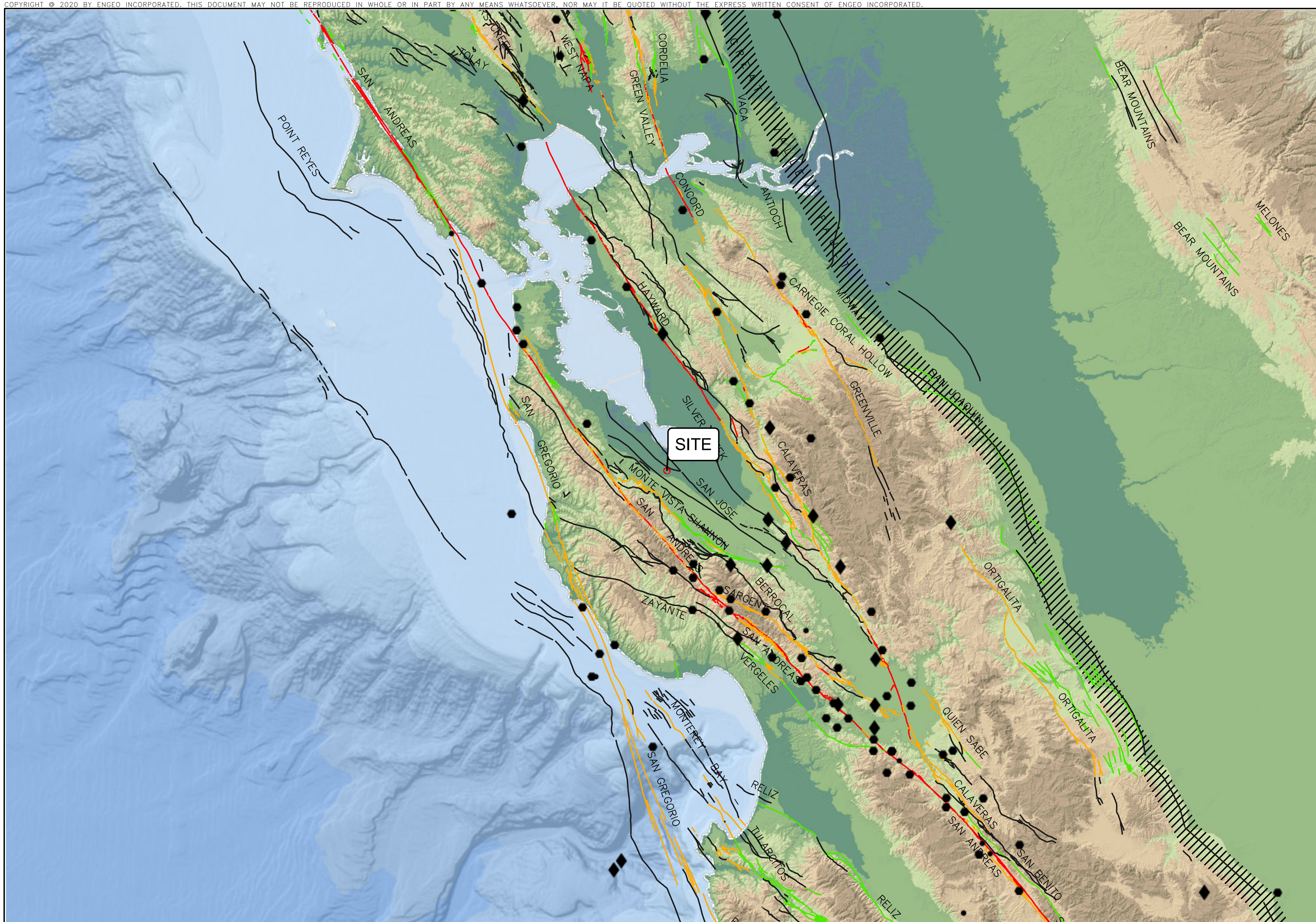

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

EARTHQUAKE

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6

USGS QUATERNARY FAULTS

- HISTORICAL
- LATEST QUATERNARY
- LATE QUATERNARY
- UNDIFFERENTIATED QUATERNARY

- HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE
ESRI, GARMIN, GEBCO, NOAA NGDC, AND OTHER CONTRIBUTORS
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2018
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)

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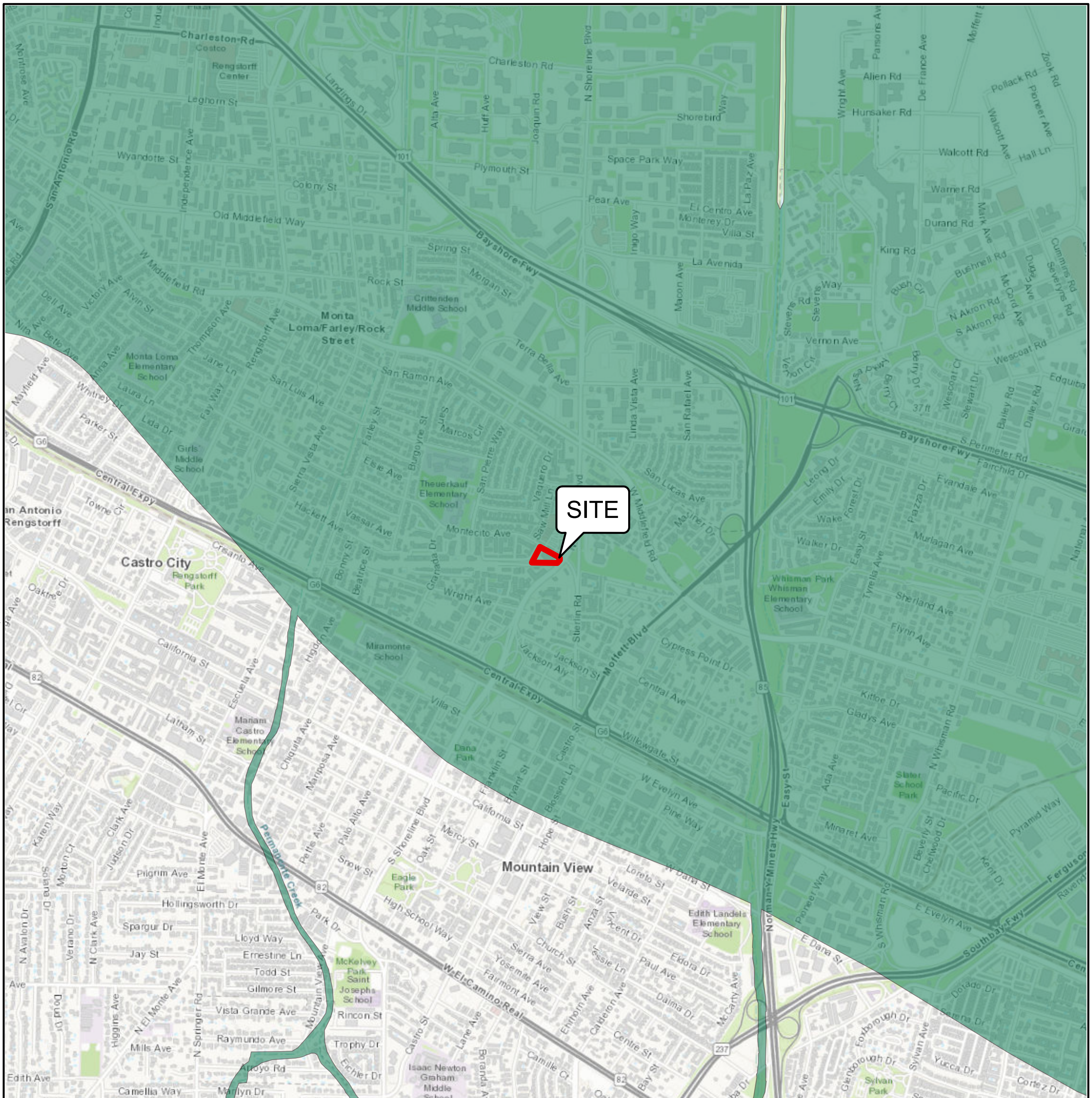
REGIONAL FAULTING AND SEISMICITY
1265 MONTECITO AVENUE
MOUNTAIN VIEW, CALIFORNIA

PROJECT NO. : 16572.000.000		FIGURE NO. 4
SCALE: AS SHOWN		
DRAWN BY: QRL	CHECKED BY: RHB	



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Liquefaction Zone
Areas where historical occurrence of liquefaction, or local geological,geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required

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BASEMAP SOURCE: ESRI MAPPING SERVICE
CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY

ENGEO Expect Excellence	SEISMIC HAZARDS ZONE MAP 1265 MONTECITO AVENUE MOUNTAIN VIEW, CALIFORNIA	PROJECT NO. : 16572.000.000 SCALE: AS SHOWN DRAWN BY: QRL CHECKED BY: RHB	FIGURE NO. 5

PATH: G:\DRAFTING\PROJECTS_16000 TO 17999\16572\16572000000\GEOTECH\GEX\1265 MONTECITO AVENUE.APRX
USER: QLIANG

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

BORING LOG KEY
BORING LOGS
CPT LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity CL - Inorganic clay with low to medium plasticity OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity CH - Fat clay with high plasticity OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAINE SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO
Expect Excellence

LOG OF BORING 1-B1

ENGEO Expect Excellence

LATITUDE: 37.402015 LONGITUDE: -122.079937

Geotechnical Exploration
1265 Montecito Avenue
Mountain View
16572.000.000

DATE DRILLED: 1/24/2020
HOLE DEPTH: 51.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (WGS-84): 51 ft.

LOGGED / REVIEWED BY: S. Brard /
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		4-Inches ASPHALT CONCRETE										
	50	8-Inches AGGREGATE BASE										
		FAT CLAY (CH), very dark gray to black, hard, moist, medium plasticity, fine-grained sand	17								4.0*	PP
		Becomes dark gray mottled with strong brown	22								4.0*	PP
5	45	Contains carbonates	33	68	26	42		21	101.5		4.0*	PP
			28								4.0*	PP
10	40	Becomes stiff to very stiff	21					33	88.8		1.72	UC
		SANDY LEAN CLAY (CL), grayish brown, very stiff, moist, medium plasticity, fine-grained sand	24									
15	35	LEAN CLAY WITH SAND (CL), grayish brown, soft, moist, medium plasticity, fine- to coarse-grained sand	2									
		Becomes grayish brown mottled with strong brown, very stiff	20									
		CLAYEY SAND (SC), grayish brown, medium dense, moist, fine- to coarse-grained sand										
20	30		29									
		POORLY GRADED SAND (SP), gray, medium dense, moist, fine- to coarse-grained sand	42				3					
25												

LOG OF BORING 1-B1

ENGEO Expect Excellence

LATITUDE: 37.402015 LONGITUDE: -122.079937

Geotechnical Exploration
1265 Montecito Avenue
Mountain View
16572.000.000

DATE DRILLED: 1/24/2020
HOLE DEPTH: 51.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (WGS-84): 51 ft.

LOGGED / REVIEWED BY: S. Brard /
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	25		CLAYEY SAND (SC), grayish brown, loose, moist, fine- to coarse-grained sand			15				27					
30	20		LEAN CLAY WITH SAND (CL), grayish brown, medium stiff, moist, fine- to medium grained sand			11					24	103.4	658		UU
35	15		POORLY GRADED SAND WITH CLAY (SP-SC), gray, dense, moist, fine- to coarse-grained sand, fine gravel			80				6					
40	10		LEAN CLAY WITH SAND (CL), gray to grayish brown, medium stiff, moist, fine- to coarse-grained sand			11								1.0*	PP
45	5		SANDY CLAY (CL), grayish brown, stiff, moist, fine- to coarse-grained sand, fine gravel			21								2.5*	PP
50			POORLY GRADED SAND WITH CLAY (SP-SC), gray, very dense, moist, fine- to coarse-grained sand, fine gravel												

LOG OF BORING 1-B1

ENGEO
Expect Excellence

LATITUDE: 37.402015 LONGITUDE: -122.079937

Geotechnical Exploration
1265 Montecito Avenue
Mountain View
16572.000.000

DATE DRILLED: 1/24/2020
HOLE DEPTH: 51.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (WGS-84): 51 ft.

LOGGED / REVIEWED BY: S. Brard /
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	0		POORLY GRADED SAND WITH CLAY (SP-SC), gray, very dense, moist, fine- to coarse-grained sand, fine gravel			52									
			Boring terminated at 51.5 feet below ground surface (bgs) Groundwater not encountered due to drilling method.												

LOG - GEOTECHNICAL_SU+QU W/ ELEV 16572000000 BOREHOLES 1-28-2020.GPJ ENGEO INC.GDT 2/20/20

LOG OF BORING 1-B2

ENGEO Expect Excellence

LATITUDE: 37.401682 LONGITUDE: -122.080159

Geotechnical Exploration
1265 Montecito Avenue
Mountain View
16572.000.000

DATE DRILLED: 1/24/2020
HOLE DEPTH: 31.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (WGS-84): 53 ft.

LOGGED / REVIEWED BY: S. Brard /
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
		4-Inches ASPHALT CONCRETE										
		8-Inches AGGREGATE BASE										
		FAT CLAY (CH), very dark gray to black, hard, moist, medium plasticity, fine-grained sand	34	66	27	39					4.0*	PP
	50		39					22	78.3		4.0*	PP
5		Becomes very dark gray mottled with strong brown	51	68	26	42		21	101.5		4.0*	PP
	45		28					18	91.7		4.0*	PP
10		Becomes stiff	24									
	40	POORLY GRADED SAND WITH CLAY (SP-SC), gray, medium dense, moist, fine- to coarse-grained sand, fine gravel										
15			15					12				
	35	Becomes dense	48				9					
20			14				11					
	30	SANDY CLAY (CL), grayish brown, stiff, moist, medium plasticity, fine- to coarse-grained sand, fine gravel										
25												

LOG OF BORING 1-B2

ENGEO
Expect Excellence

LATITUDE: 37.401682 LONGITUDE: -122.080159

Geotechnical Exploration
1265 Montecito Avenue
Mountain View
16572.000.000

DATE DRILLED: 1/24/2020
HOLE DEPTH: 31.5 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (WGS-84): 53 ft.

LOGGED / REVIEWED BY: S. Brard /
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SANDY CLAY (CL), grayish brown, stiff, moist, medium plasticity, fine- to coarse-grained sand, fine gravel			9									
	25														
30			Becomes greenish gray, fine- to medium-grained sand			22									
			Boring terminated at 31.5 feet below ground surface (bgs) Groundwater not encountered due to drilling method.												

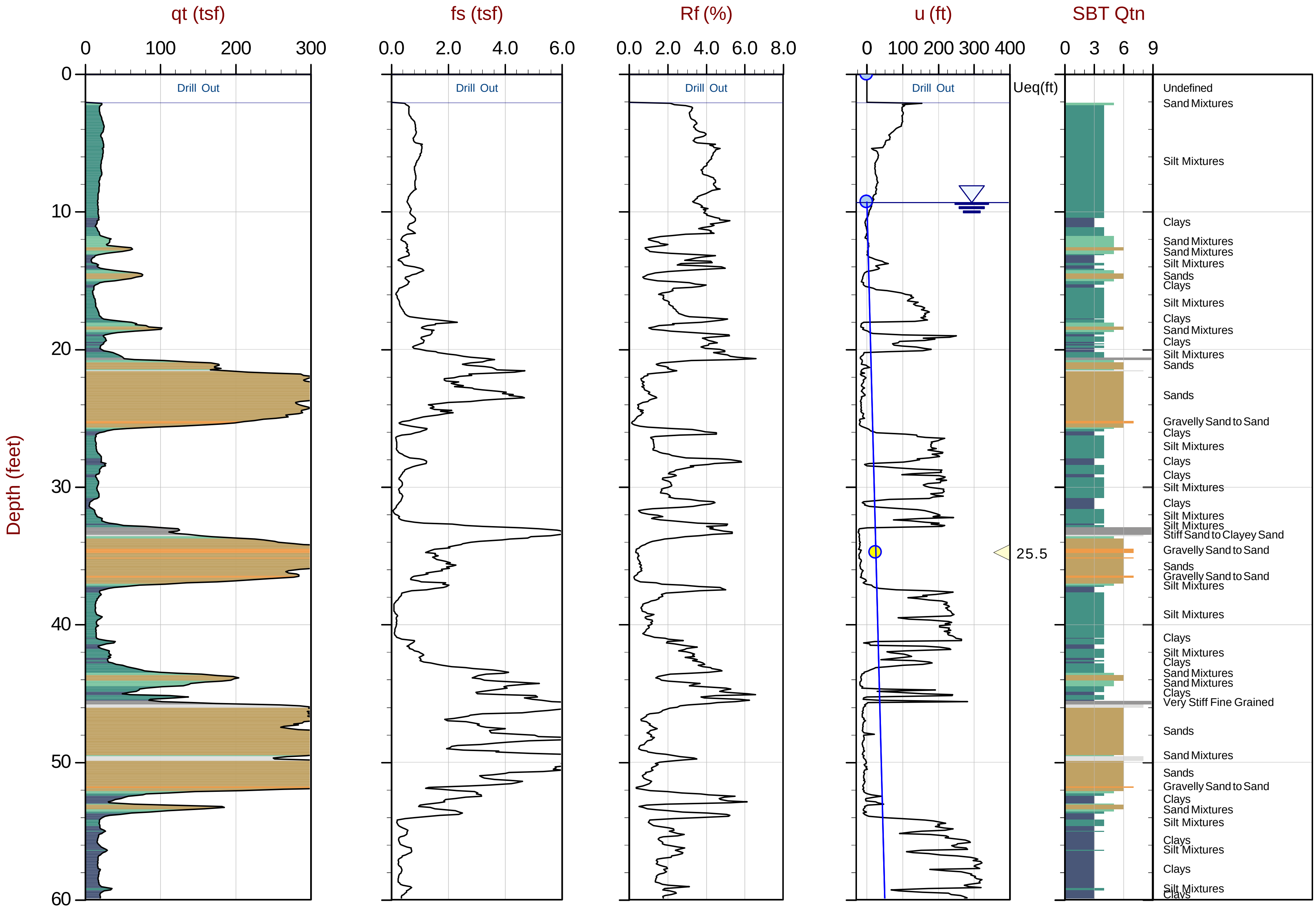
ENGEO Expect Excellence	HAND AUGER LOG (HA-1)	
1265 Montecito Avenue Mountain View, California 15188.000.000		Logged By: Stephen Brard Logged Date: January 24, 2020
Hand Auger Number	Depth (feet)	Description
TP-1 Lat: 37.401818° Long: -122.079402°	0 – 3	LEAN CLAY (CL), grayish brown, moist, moderate plasticity, fine-grained sand

CONETEC

ENGEO

Job No: 20-56-20396
Date: 2020-01-16 11:41
Site: 1265 Montecito Avenue

Sounding: 1-SCPT1
Cone: 483:T1500F15U500



Max Depth: 30.600 m / 100.39 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-20396_1SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139867m E: 581440m

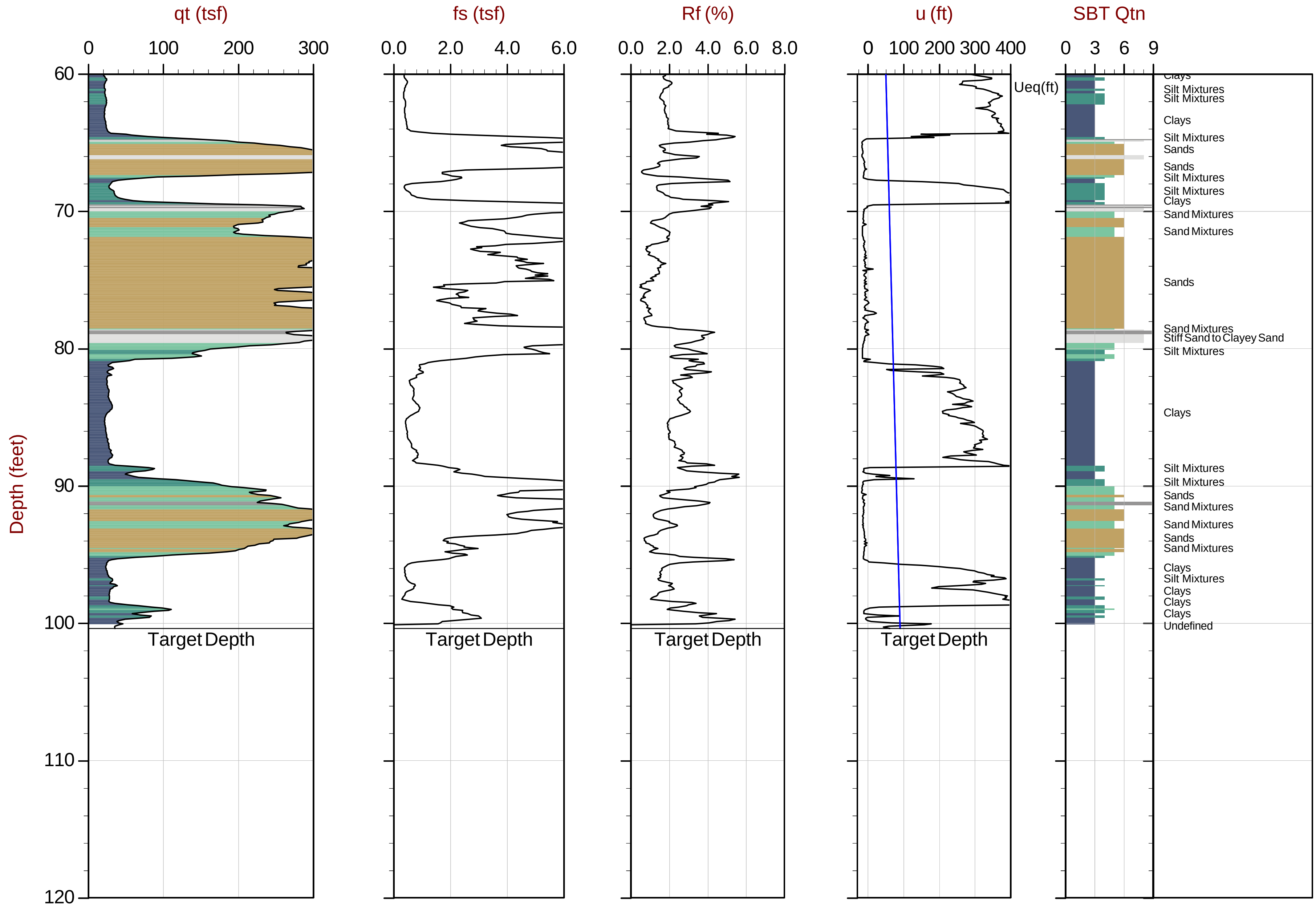
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

ENGEO

Job No: 20-56-20396
Date: 2020-01-16 11:41
Site: 1265 Montecito Avenue
Sounding: 1-SCPT1
Cone: 483:T1500F15U500



Max Depth: 30.600 m / 100.39 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-20396_1SP01.COR
Unit Wt: SBTQtn (PKR2009)

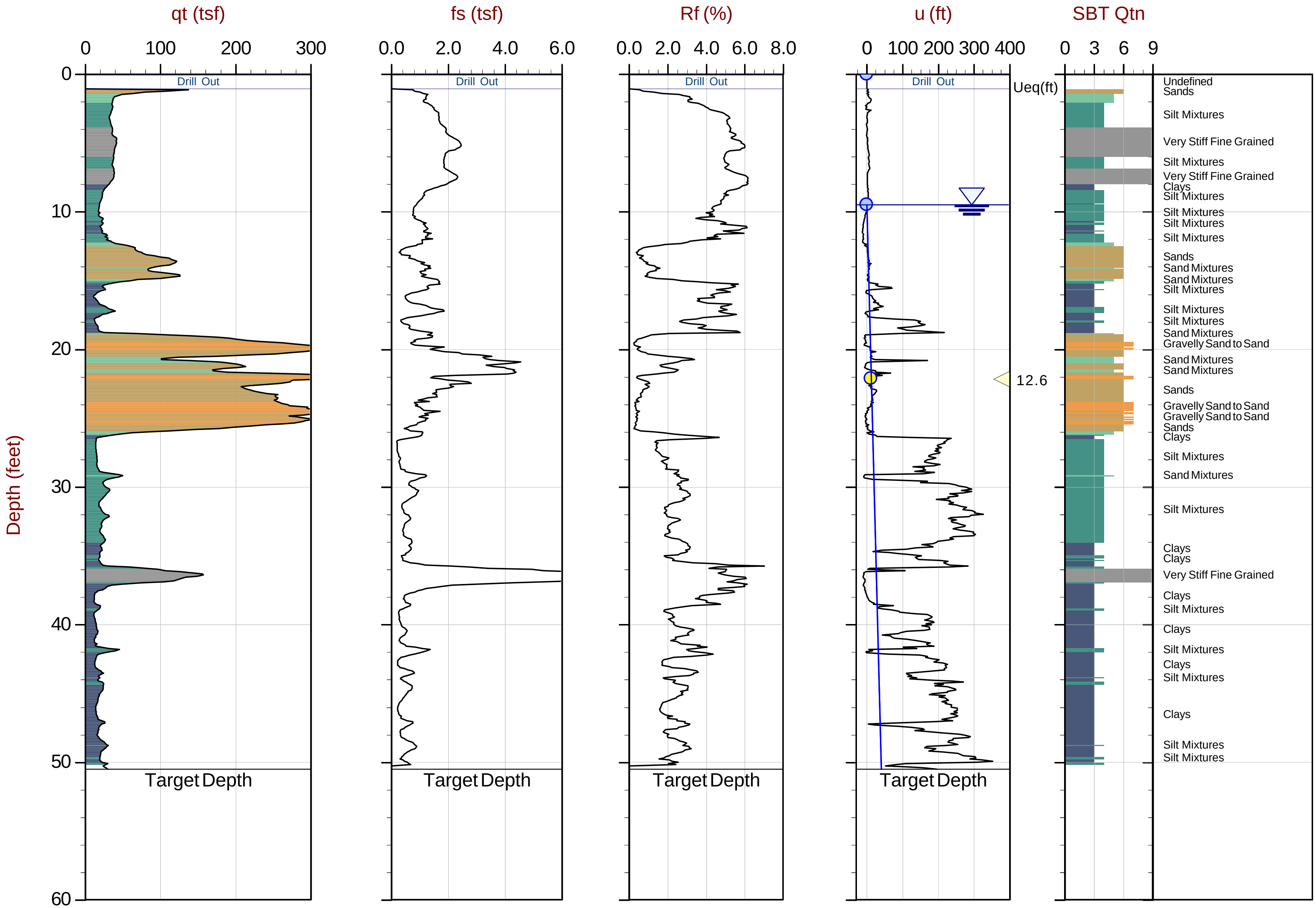
SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139867m E: 581440m

Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ConeTec | ENGEO

Job No: 20-56-20396
Date: 2020-01-16 10:43
Site: 1265 Montecito Avenue
Sounding: 1-CPT2
Cone: 483:T1500F15U500



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-20396_1CP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139857m E: 581431m

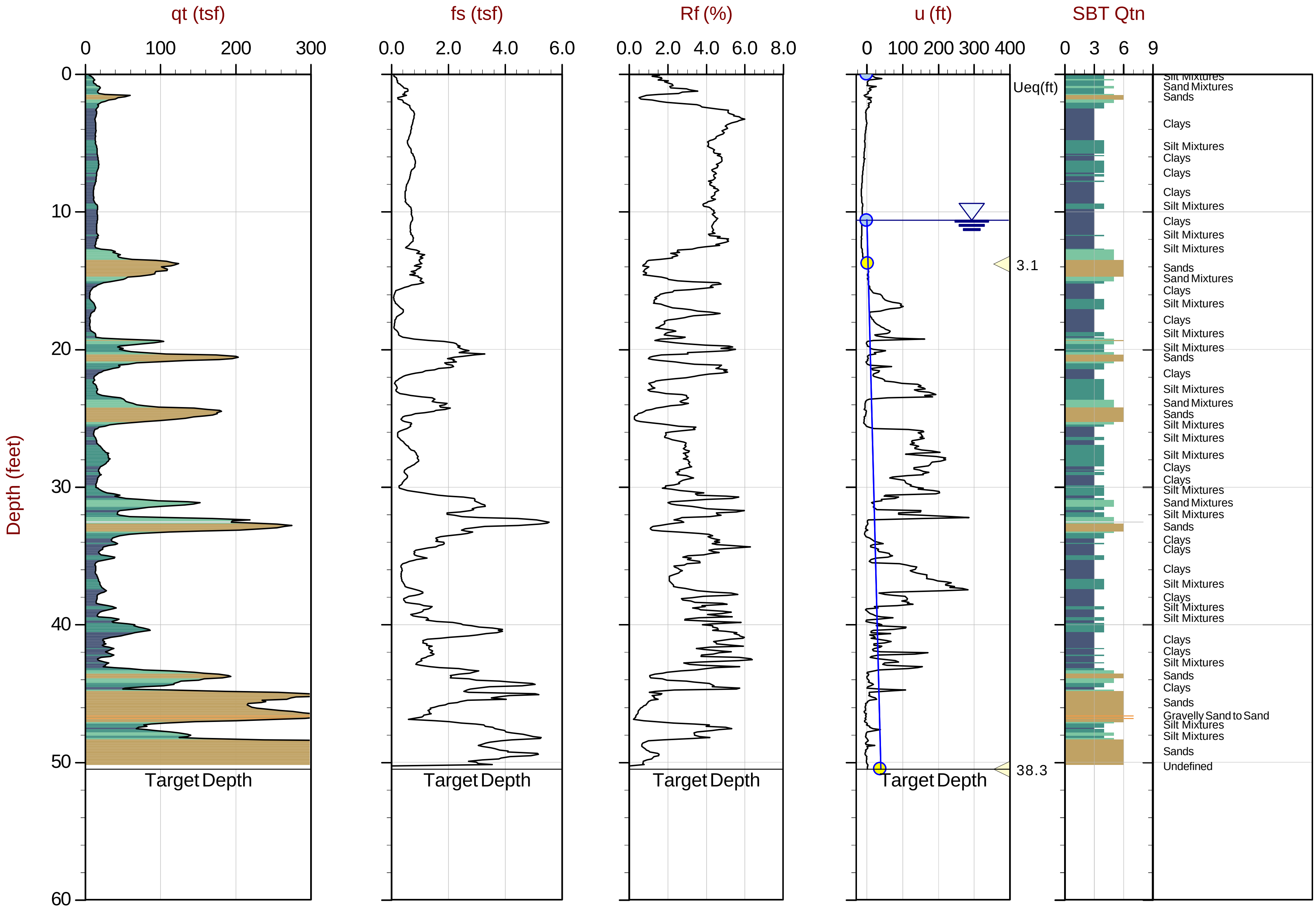
Equilibrium Pore Pressure (Ueq) | Assumed Ueq | Dissipation, Ueq achieved | Dissipation, Ueq not achieved | Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

ENGEO

Job No: 20-56-20396
Date: 2020-01-16 09:35
Site: 1265 Montecito Avenue

Sounding: 1-CPT3
Cone: 483:T1500F15U500



Max Depth: 15.400 m / 50.52 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 20-56-20396_1CP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: UTM 10N N: 4139825m E: 581459m

Equilibrium Pore Pressure (Ueq) — Assumed Ueq — Dissipation, Ueq achieved — Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

APPENDIX B

LIQUEFACTION ANALYSIS

ENGEO Incorporated
2010 Crow Canyon Place, Suite 250
San Ramon, CA
http://www.engeo.com

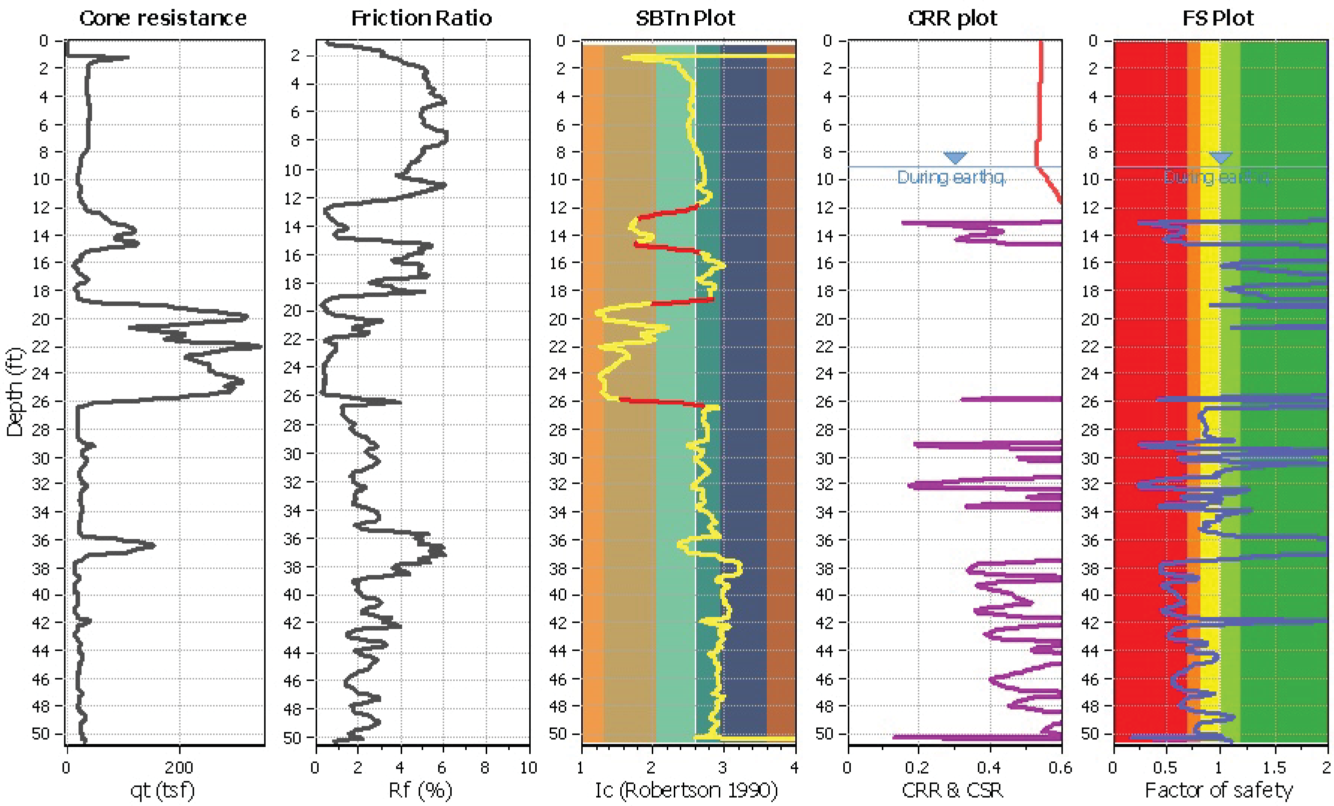
LIQUEFACTION ANALYSIS REPORT

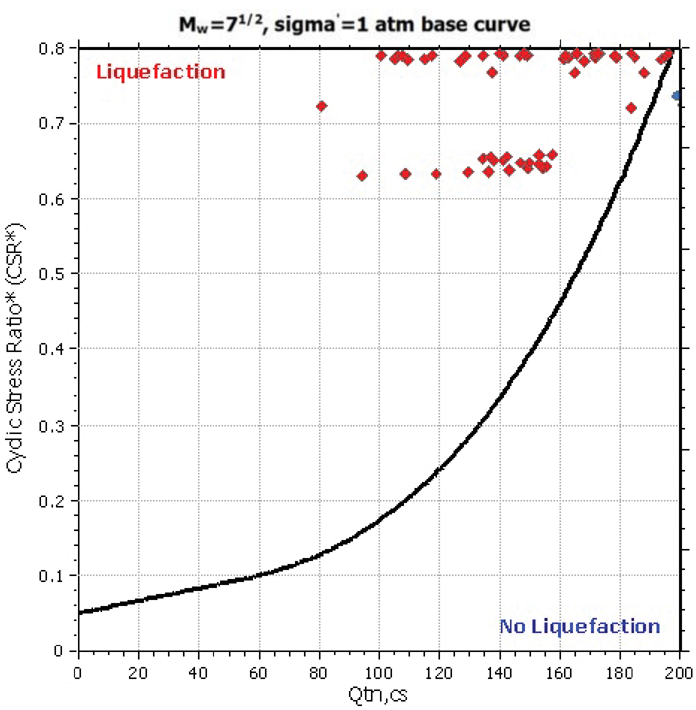
Project title : **Location :**

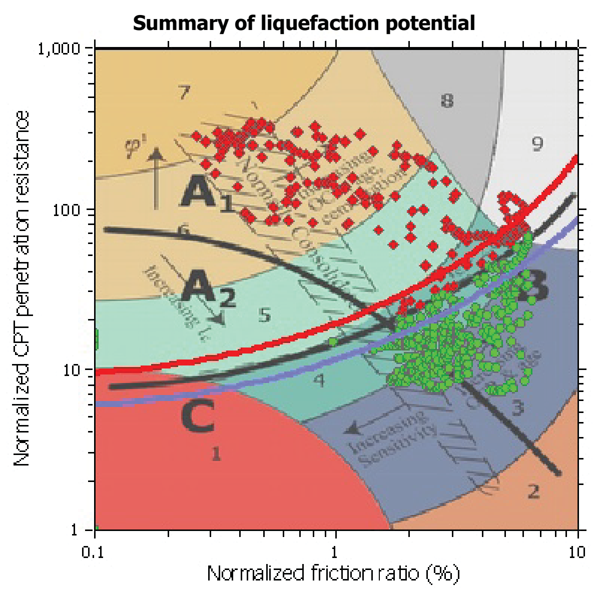
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	9.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	9.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	No		

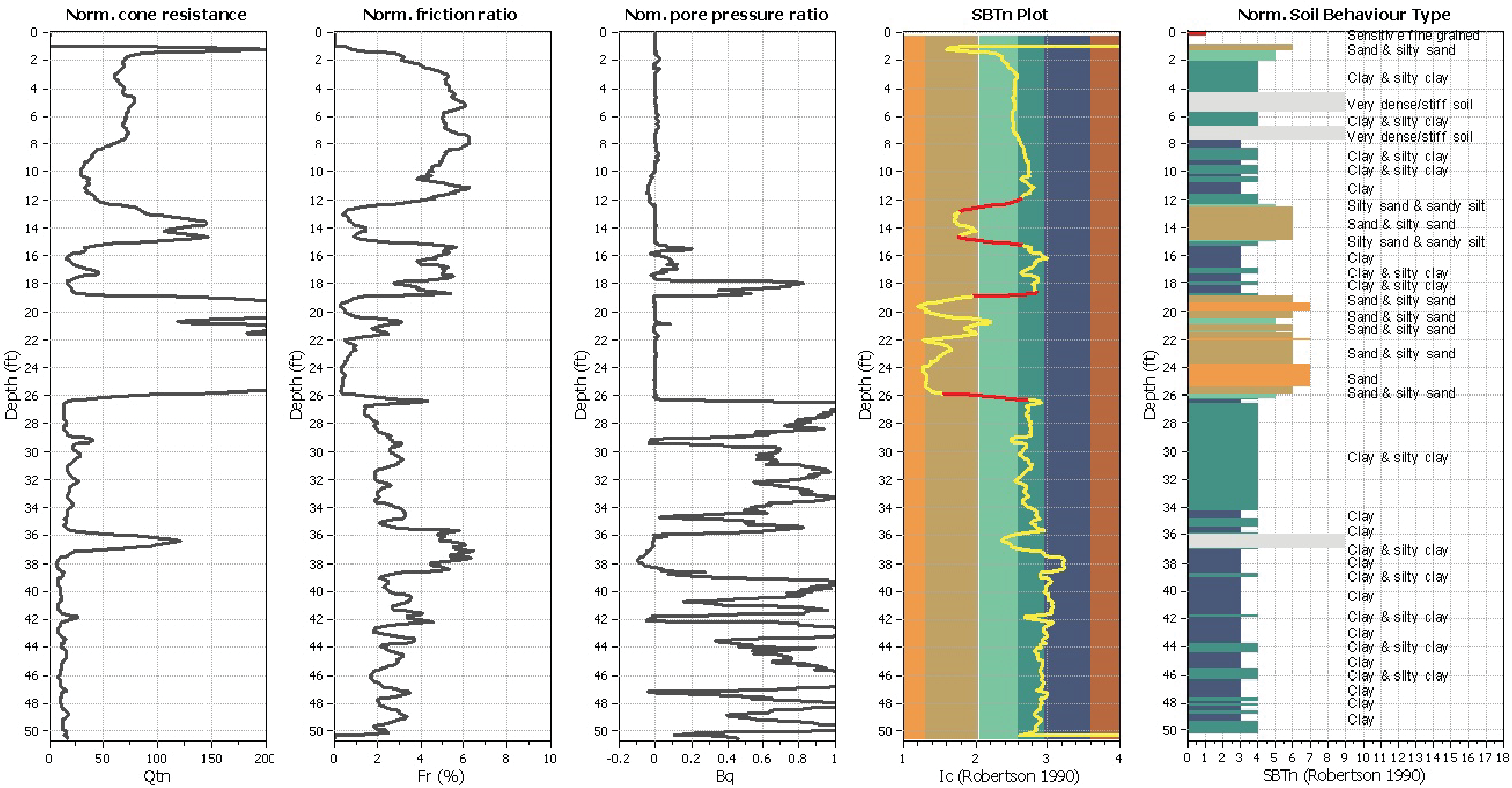






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



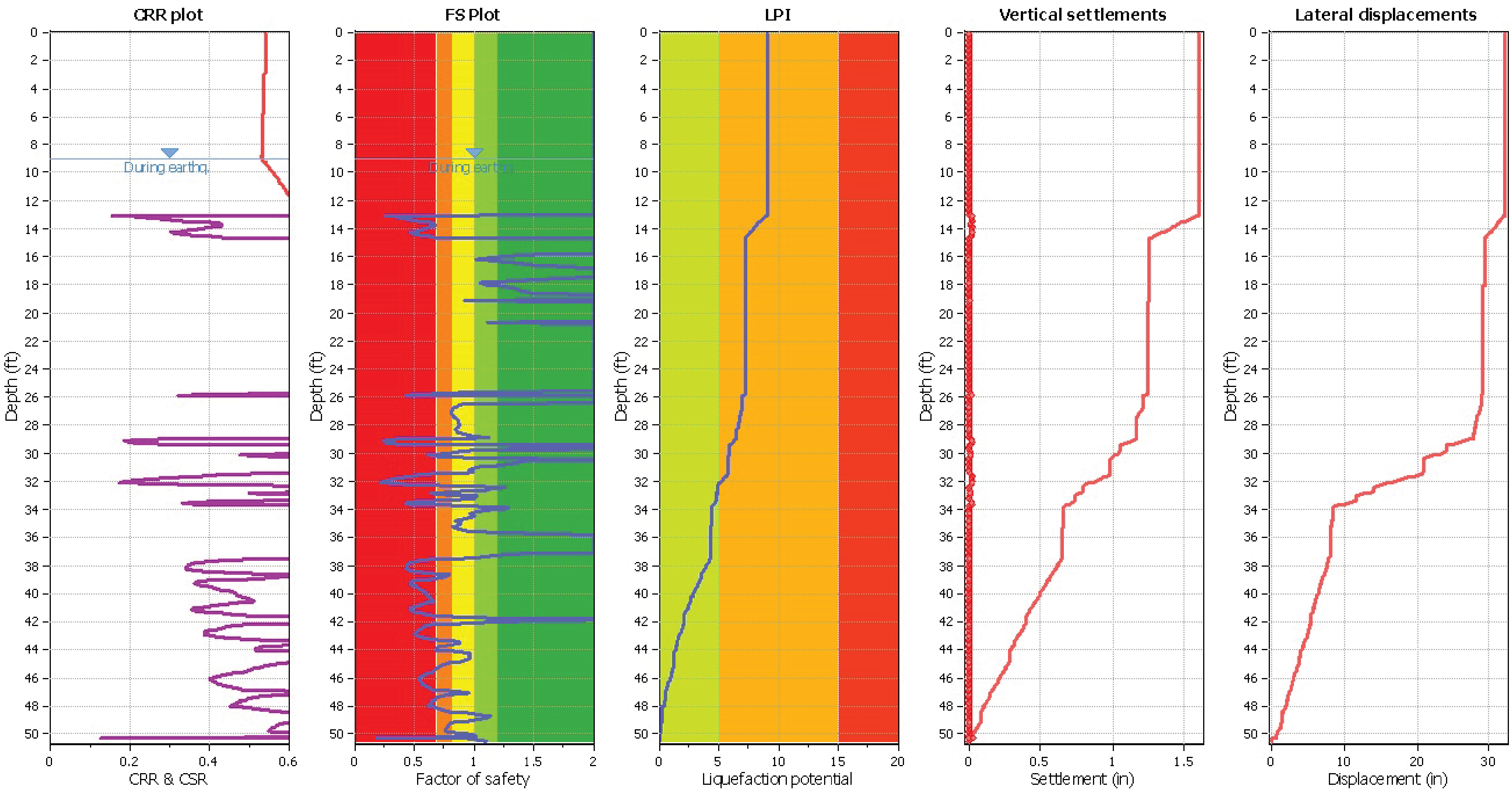
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
2010 Crow Canyon Place, Suite 250
San Ramon, CA
http://www.engeo.com

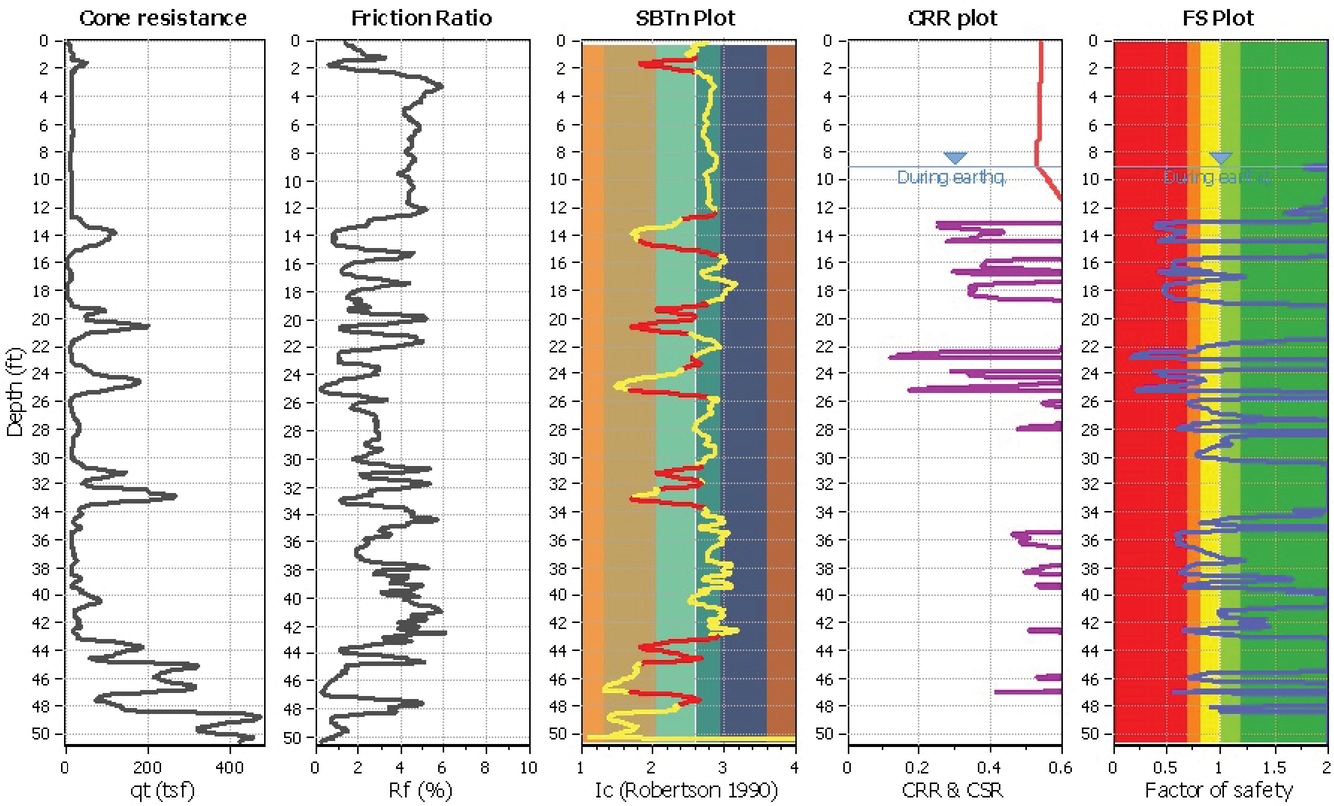
LIQUEFACTION ANALYSIS REPORT

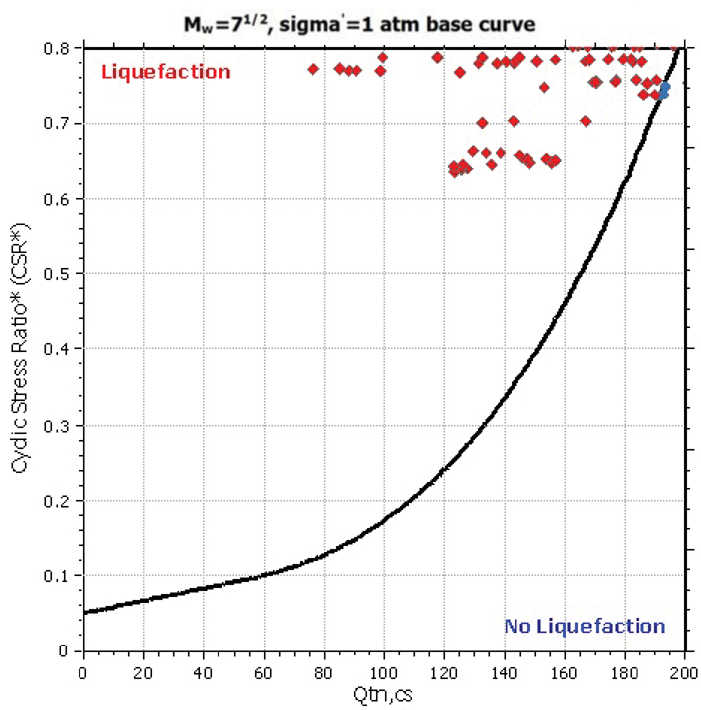
Project title : **Location :**

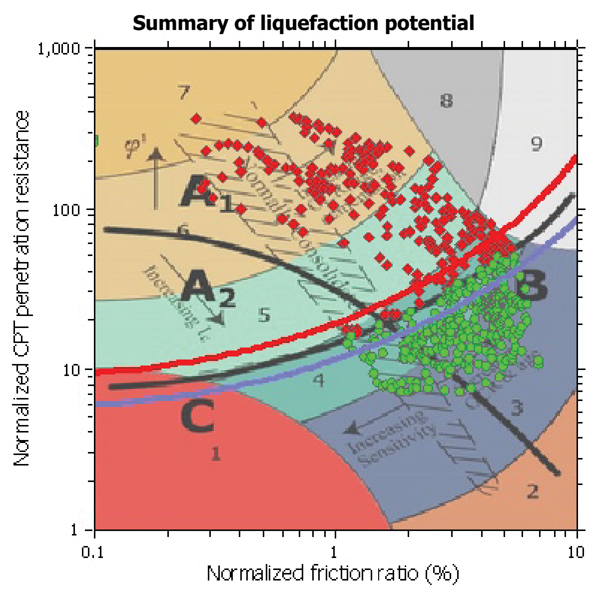
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	9.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	9.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	No		

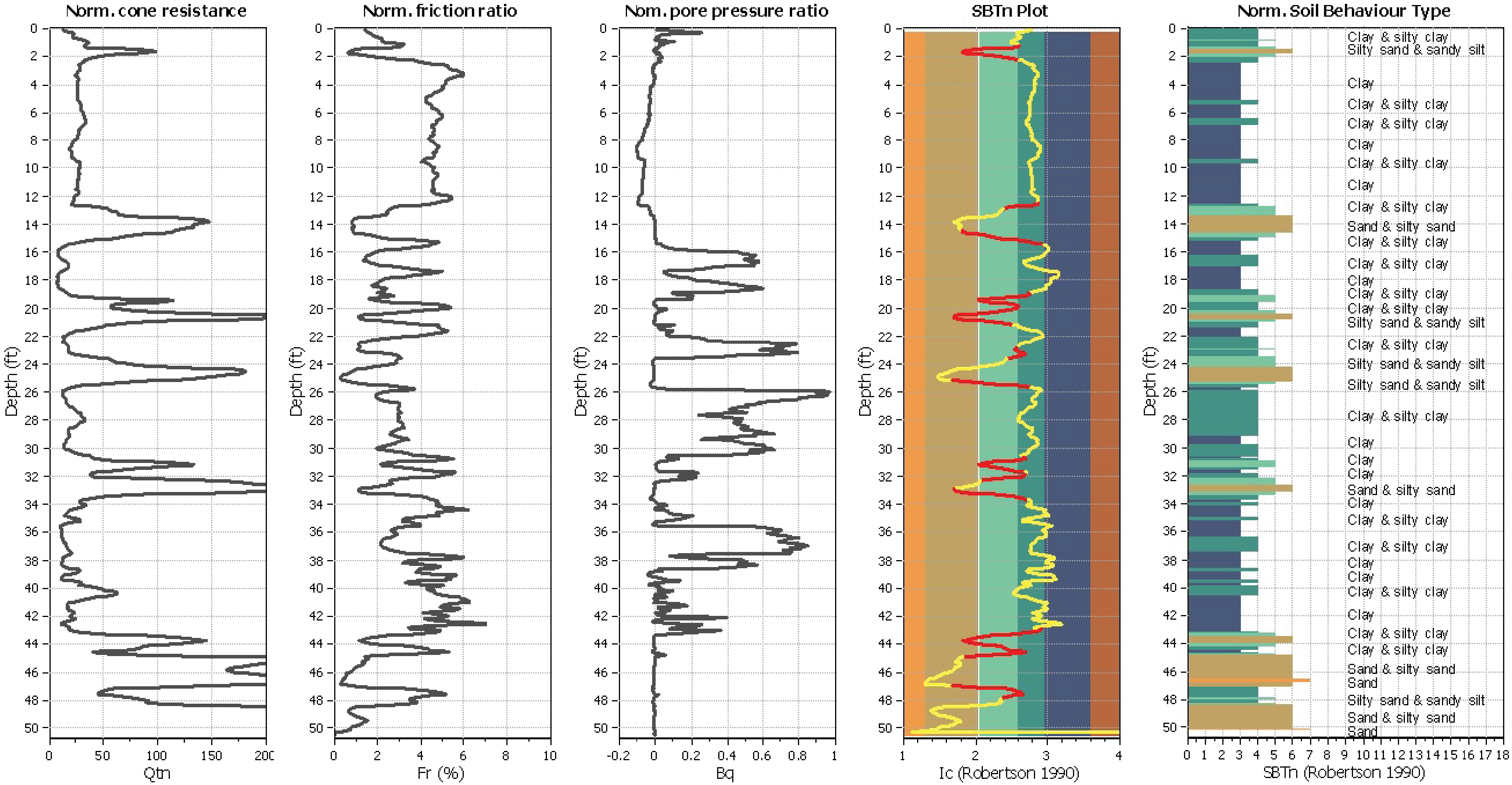






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



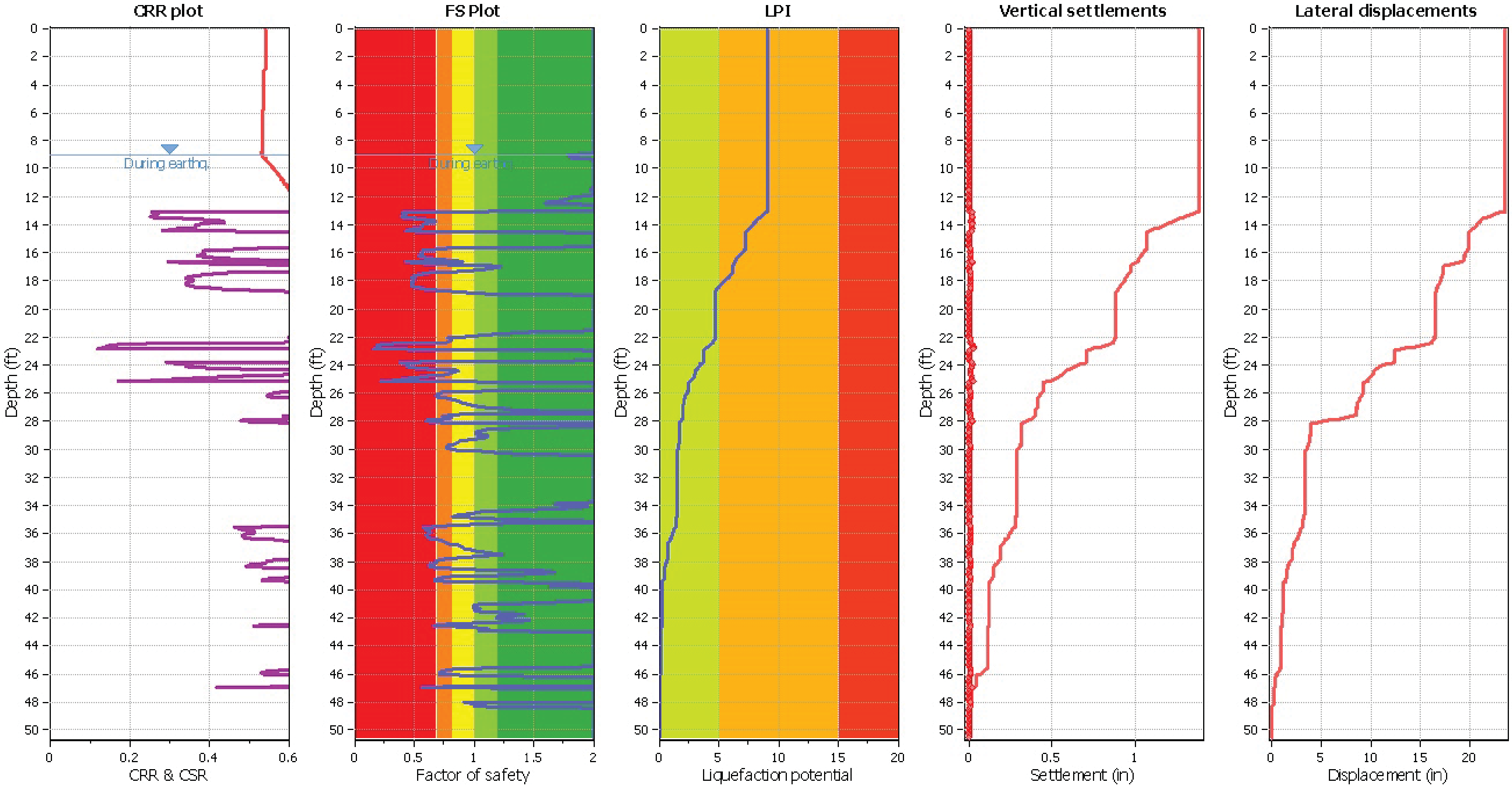
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO Incorporated
2010 Crow Canyon Place, Suite 250
San Ramon, CA
http://www.engeo.com

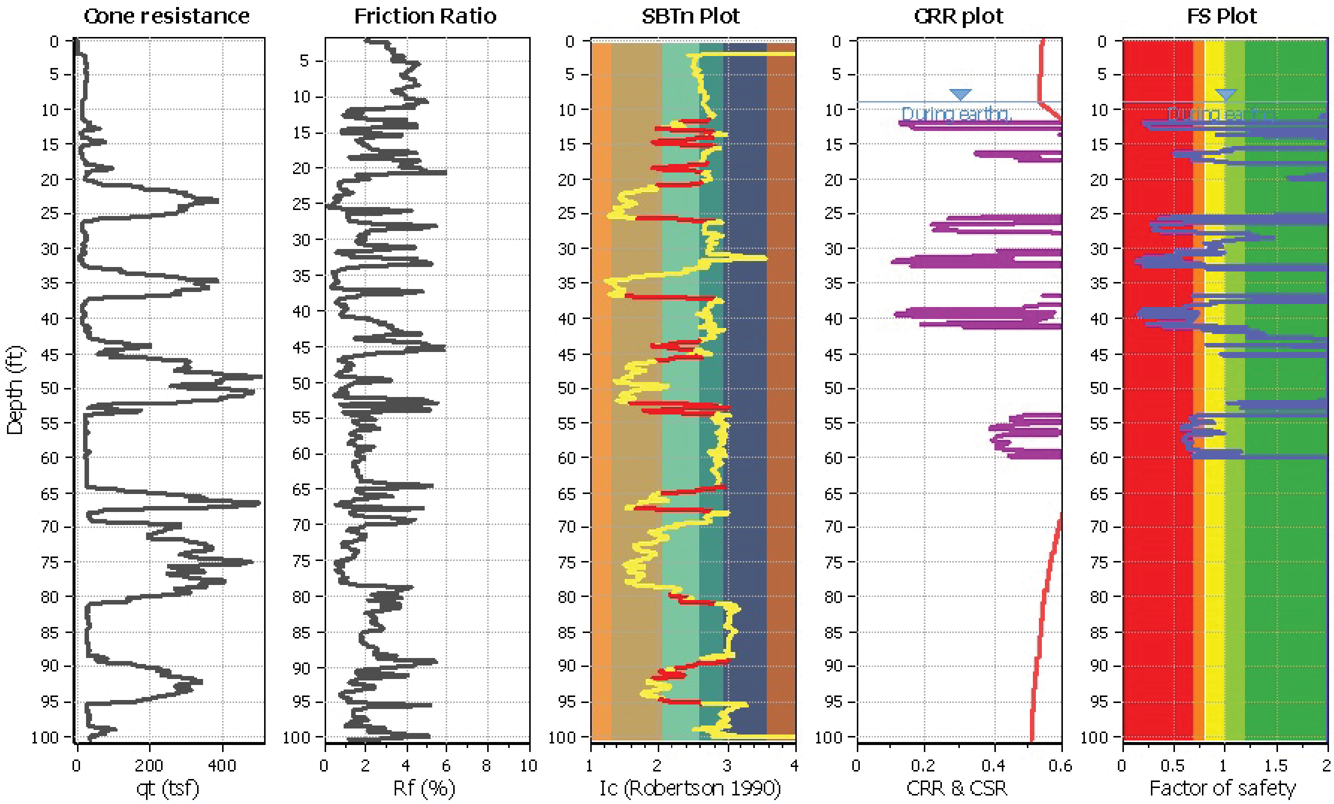
LIQUEFACTION ANALYSIS REPORT

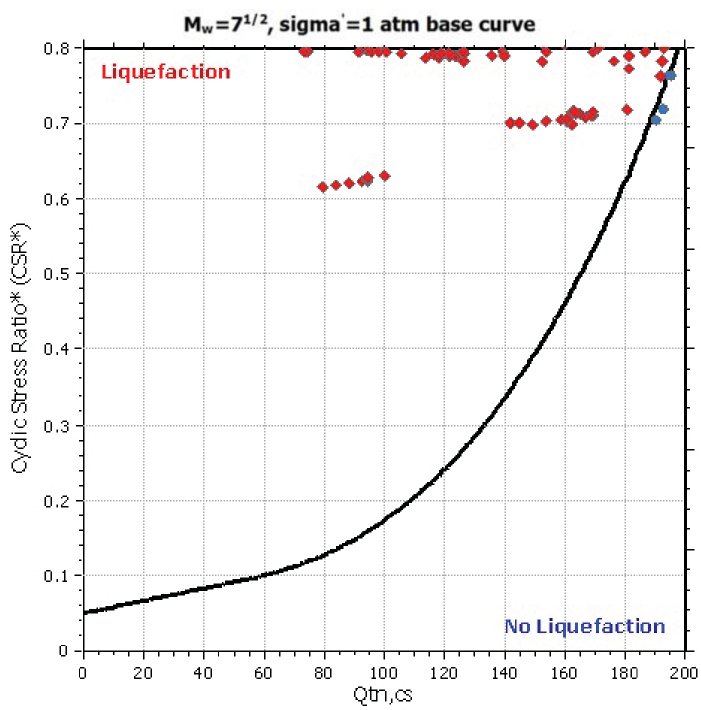
Project title : **Location :**

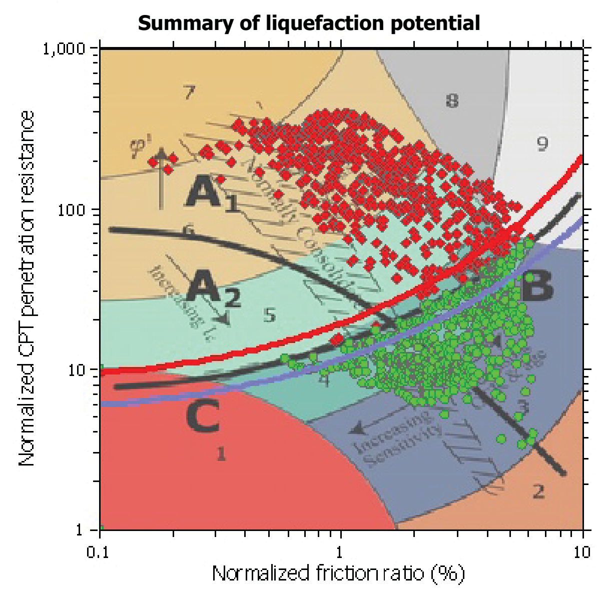
CPT file : 1-SCPT1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	9.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	9.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.73	Unit weight calculation:	Based on SBT	K_σ applied:	No		

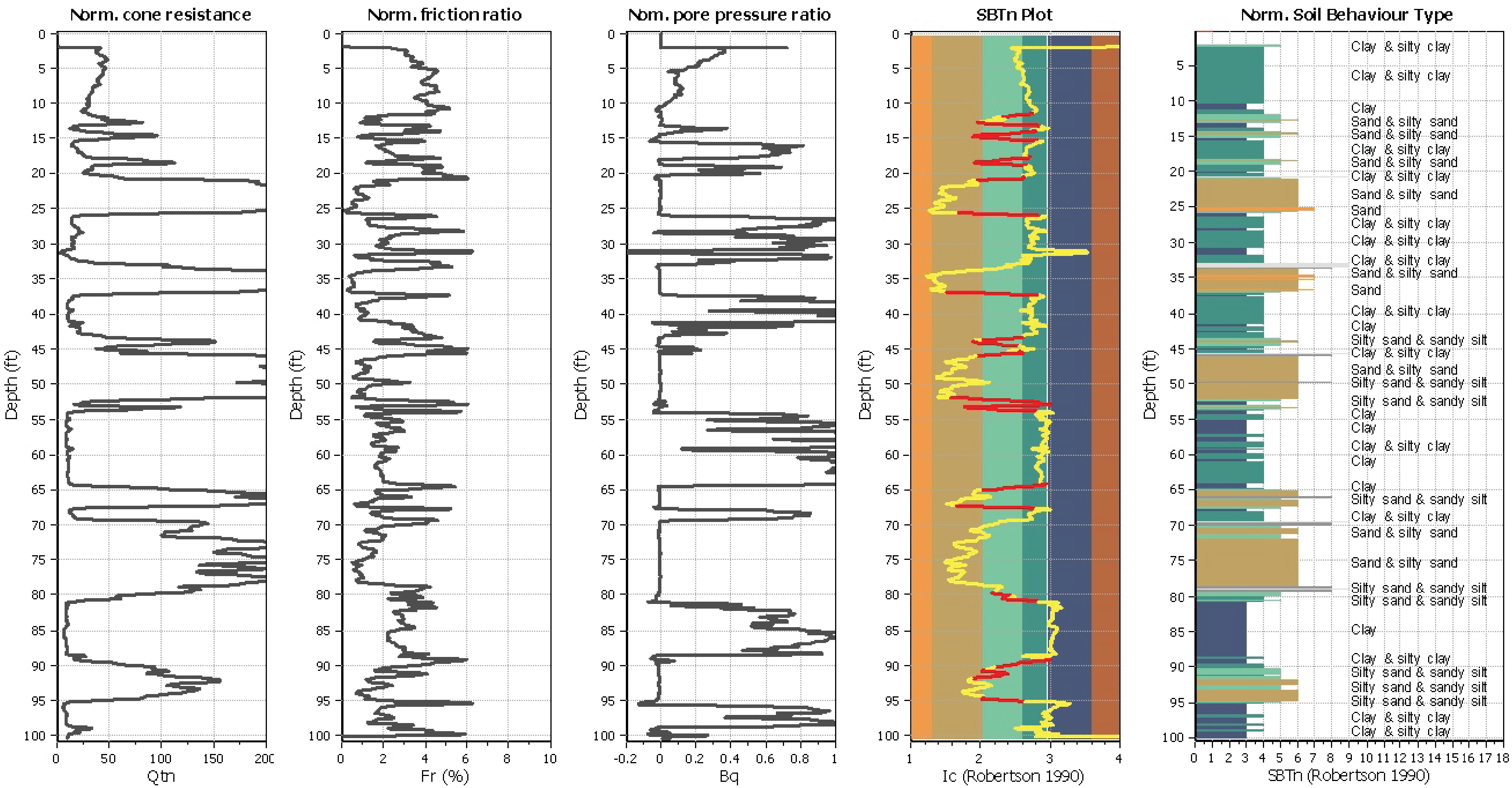






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normaliz



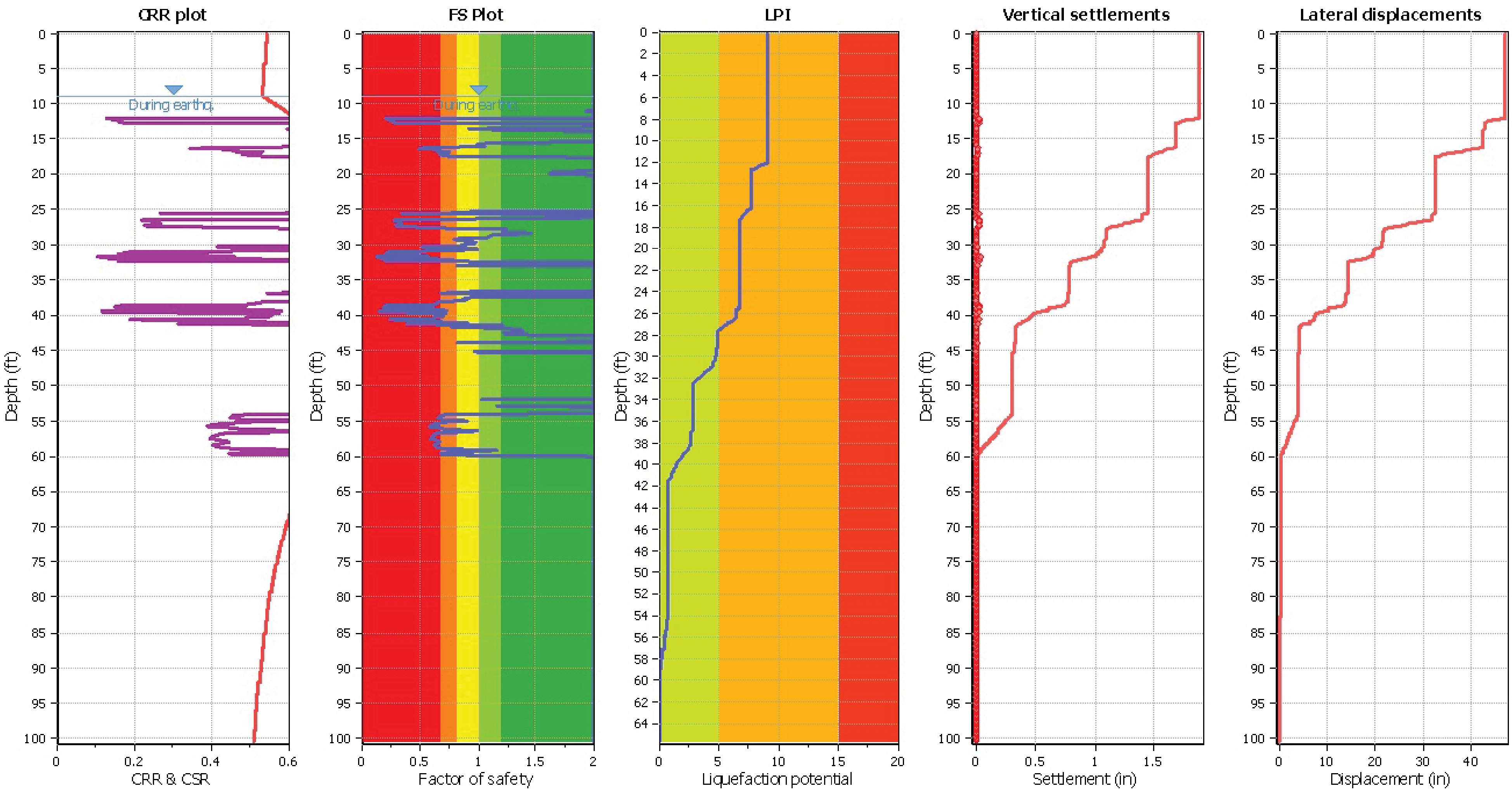
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.73	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

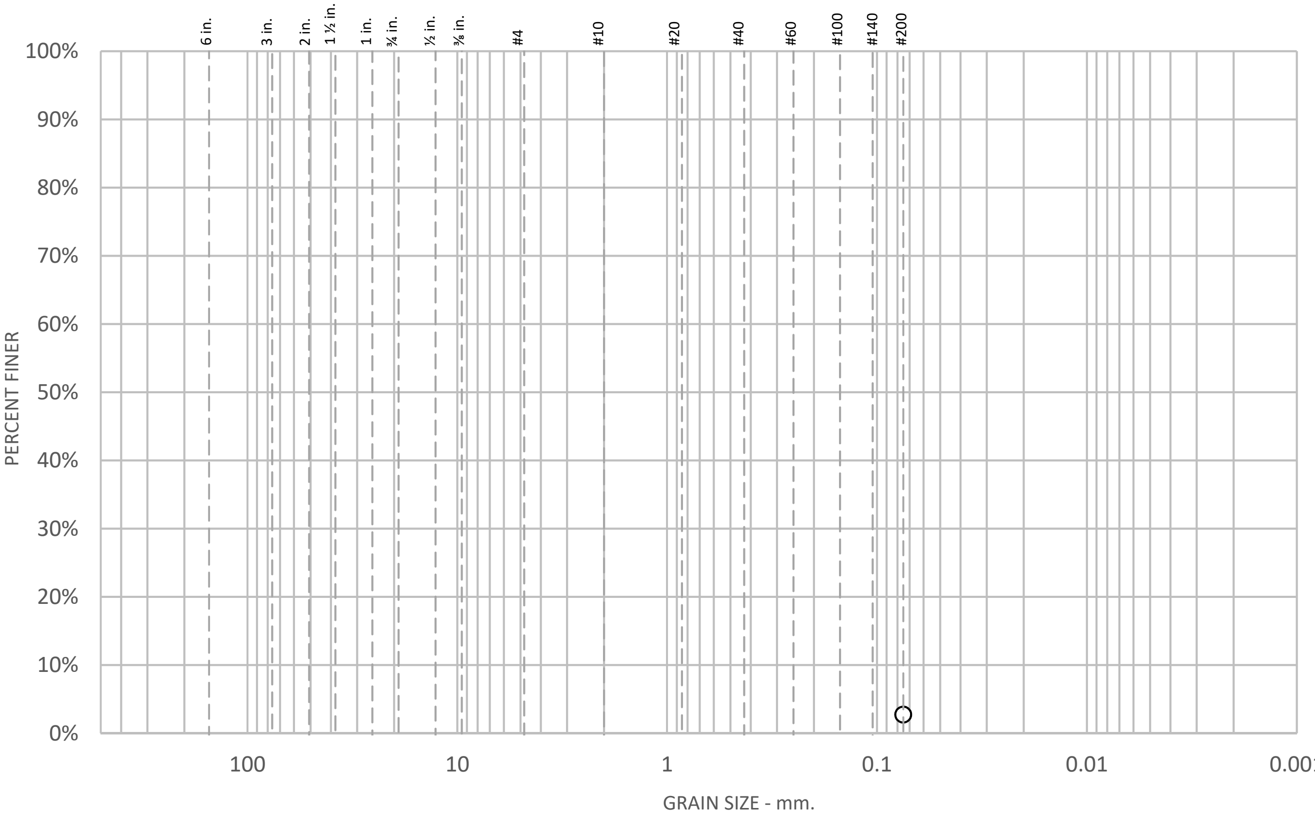
LPI color scheme

- Very high risk
- High risk
- Low risk

APPENDIX C

LABORATORY RESULTS

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method A
Soak time = 180 min
Dry sample weight = 468.4 g

Sample Number: 1-B1 @ 23.0 feet

Client: Charities Housing
Project: 1265 Montecito PGEX
Project location: San Bruno, California

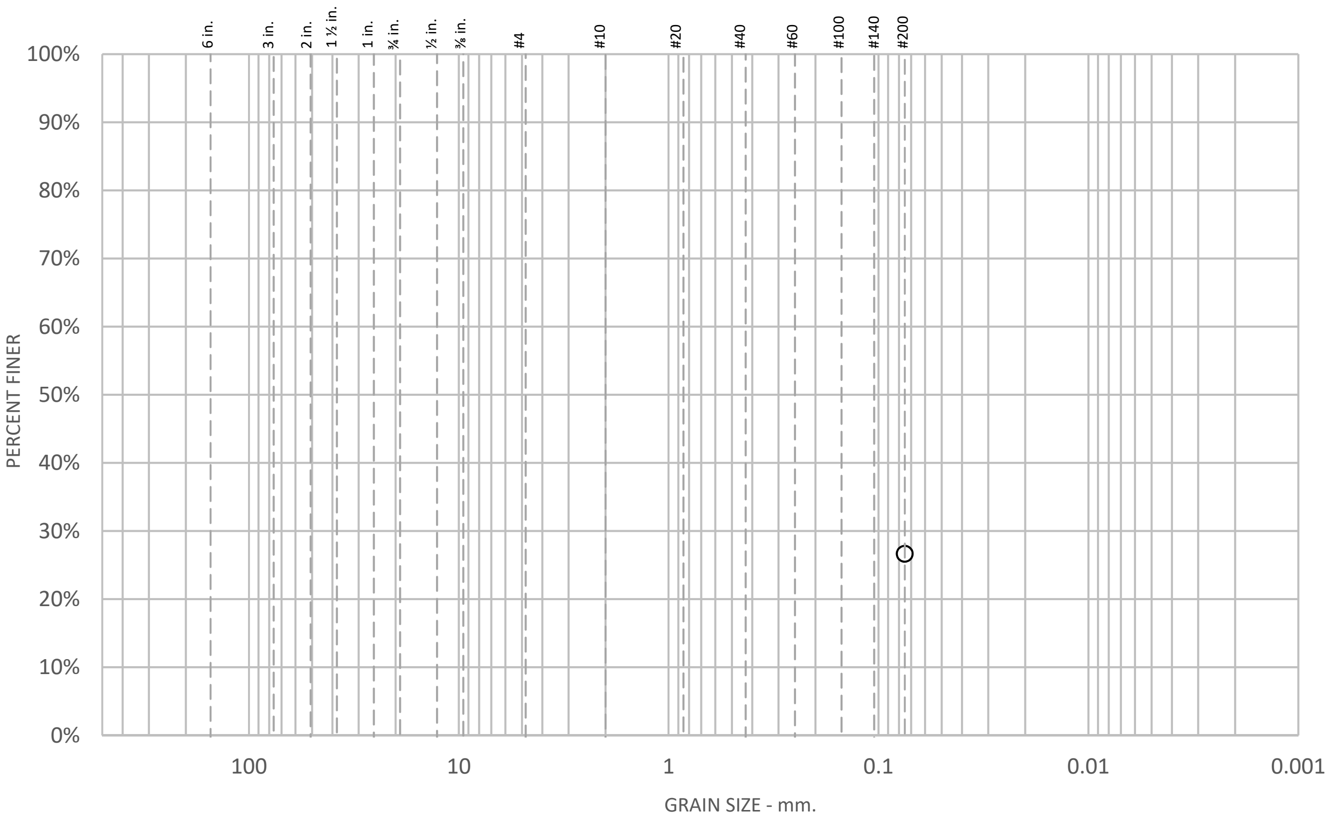
Project Number: 16572.000.000
Date: 1/30/2020

ENGEO Expect Excellence

Tested By: W. Miller **Checked By:** G. Criste

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						26.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	26.6		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method B
Soak time = 180 min
Dry sample weight = 429.7 g

Sample Number: 1-B1 @ 28.0-28.5 feet

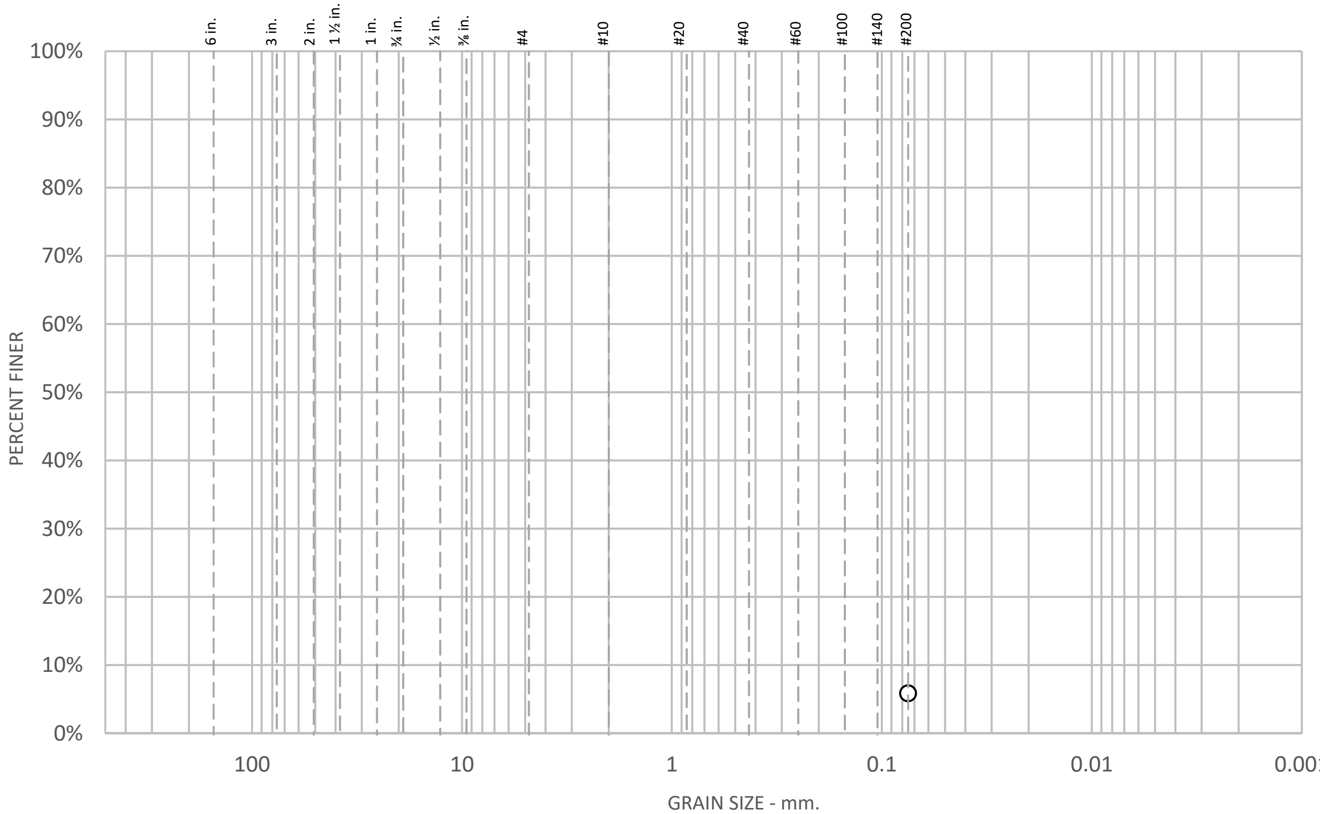
Client: Charities Housing
Project: 1265 Montecito PGEX
Project location: San Bruno, California
Project Number: 16572.000.000
Date: 1/30/2020

ENGEO
Expect Excellence

Tested By: W. Miller **Checked By:** G. Criste

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	6		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method A
Soak time = 180 min
Dry sample weight = 924.7 g

Sample Number: 1-B1 @ 36.0-36.5 feet

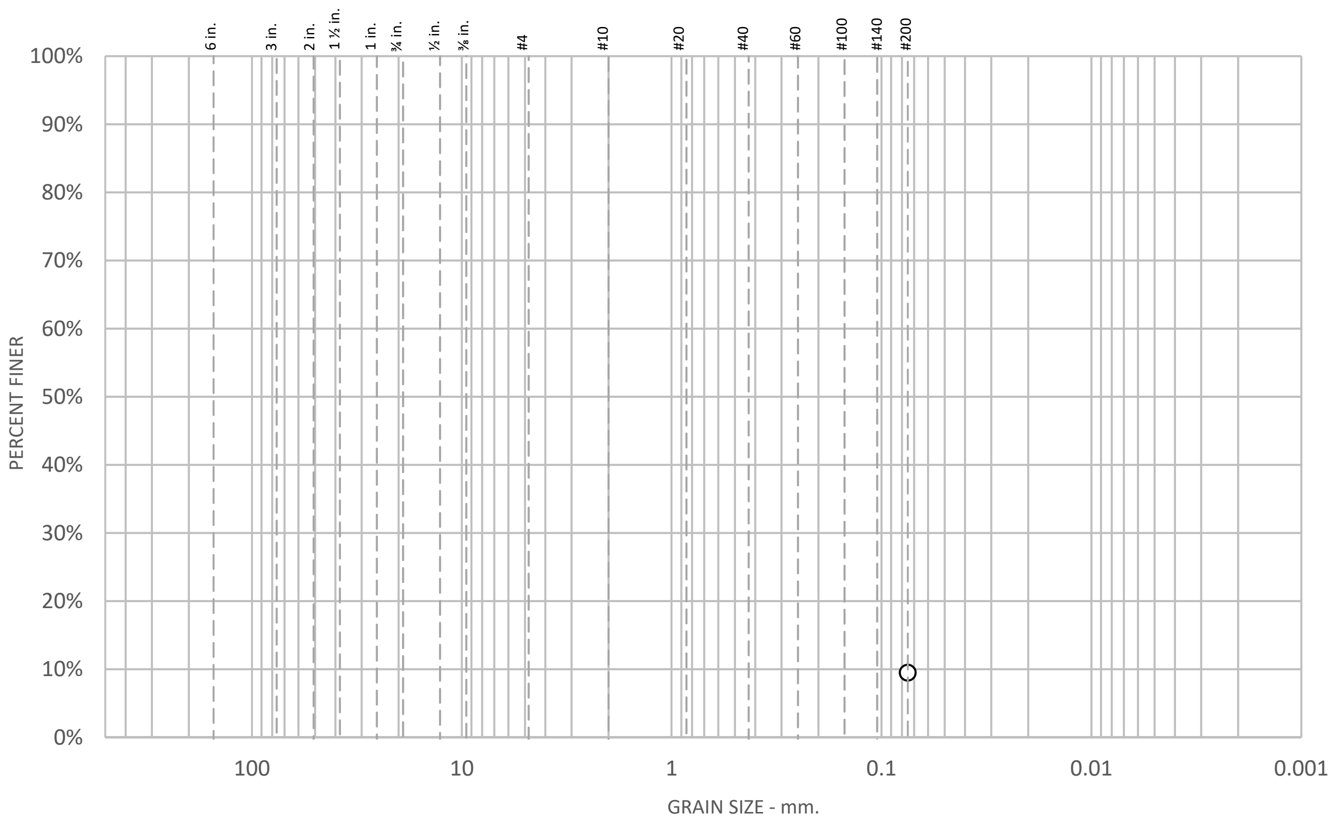
Client: Charities Housing **Project Number:** 16572.000.000
Project: 1265 Montecito PGEX **Date:** 1/30/2020
Project location: San Bruno, California

ENGEO
Expect Excellence

Tested By: W. Miller **Checked By:** G. Criste

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method A
Soak time = 180 min
Dry sample weight = 859.7 g

Sample Number: 1-B2 @ 19.0-19.5 feet

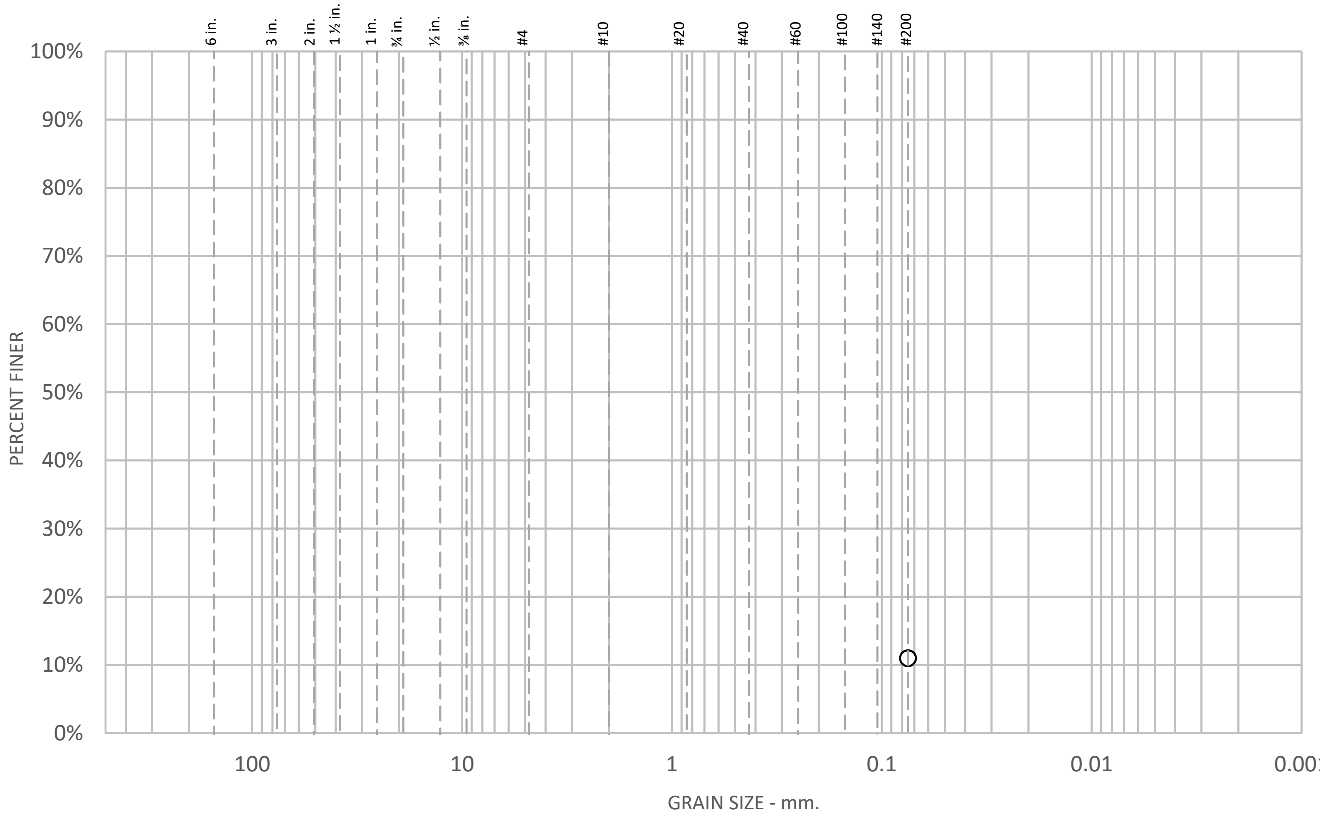
Client: Charities Housing **Project Number:** 16572.000.000
Project: 1265 Montecito PGEX **Date:** 1/30/2020
Project location: San Bruno, California

ENGEO Expect Excellence

Tested By: W. Miller **Checked By:** G. Criste

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						11	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	11		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL = LL = PI =

Coefficients

D_{90} = D_{85} = D_{60} =
D_{50} = D_{30} = D_{15} =
D_{10} = C_u = C_c =

Classification

USCS =

Remarks

ASTM D1140, Method A
Soak time = 180 min
Dry sample weight = 266.4 g

Sample Number: 1-B2 @ 21.0 feet

Client: Charities Housing
Project: 1265 Montecito PGEX
Project location: San Bruno, California
Project Number: 16572.000.000
Date: 1/30/2020

ENGEO Expect Excellence

Tested By: W. Miller **Checked By:** G. Criste

Test Location: 3420 Fostoria Way, Suite E, Danville, CA 94526

MOISTURE-DENSITY DETERMINATION
ASTM D7263

BORING ID:	**1-B1**	**1-B2**	**1-B2**	**1-B2**				
DEPTH (ft.):	**5.5-6**	**3.5-4**	**7.5-8**	**15**				
MOISTURE CONTENT (%):	**20.7**	**22.0**	**17.5**	**11.7**				
DRY DENSITY (lbs/ft^3):	**101.5**	**78.3**	**91.7**					

Testing remarks: For moisture content only, ASTM D2216

PROJECT NAME: **1265 Montecito PGEX**
PROJECT NUMBER: **16572.000.000**
CLIENT: **Charaties Housing**
PHASE NUMBER: **001**

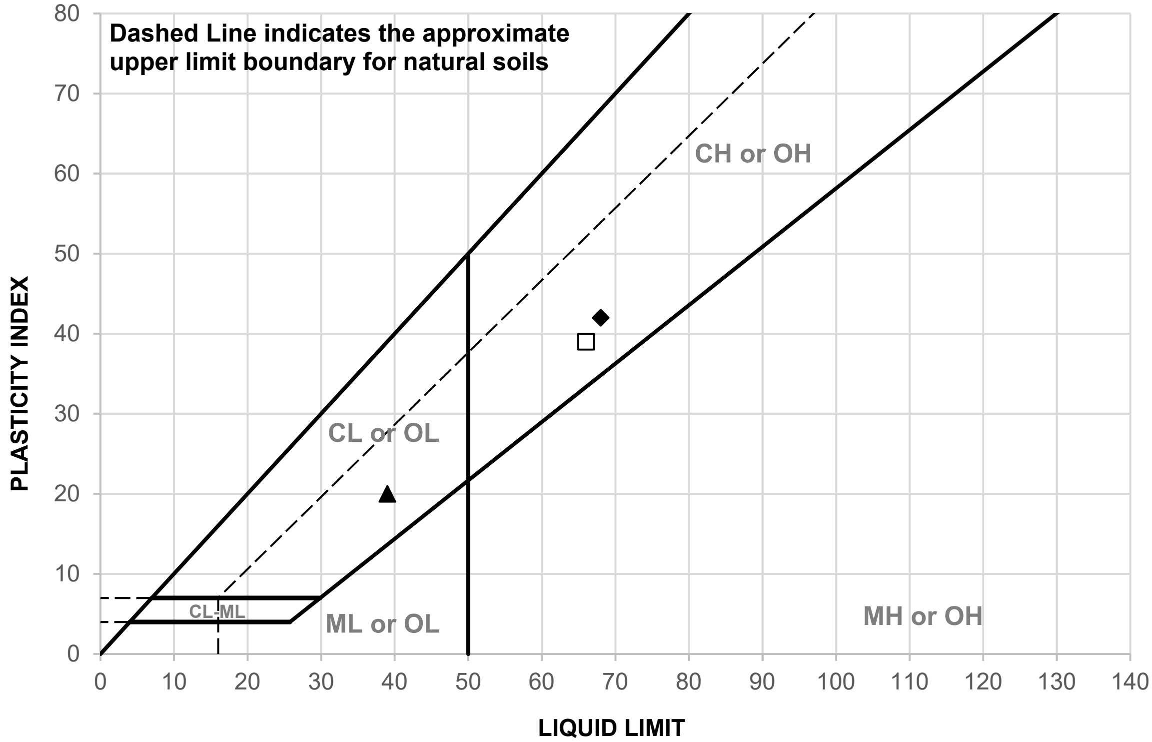
DATE: **01/29/20**

ENGEO
Expect Excellence

Tested by: W. Miller   Reviewed by: M. Quasem

Laboratory address: 3420 Fostoria Way, Suite E, Danville, CA 94526. Phone No. (925) 355-9047.

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	HA-1	2 feet	See exploration logs	39	19	20
◆	1-B1	6-6.5 feet	See exploration logs	68	26	42
□	1-B2	2-2.5 feet	See exploration logs	66	27	39

	SAMPLE ID	TEST METHOD	REMARKS
▲	HA-1	PI: ASTM D4318, Wet Method	
◆	1-B1	PI: ASTM D4318, Wet Method	
□	1-B2	PI: ASTM D4318, Wet Method	

ENGEO
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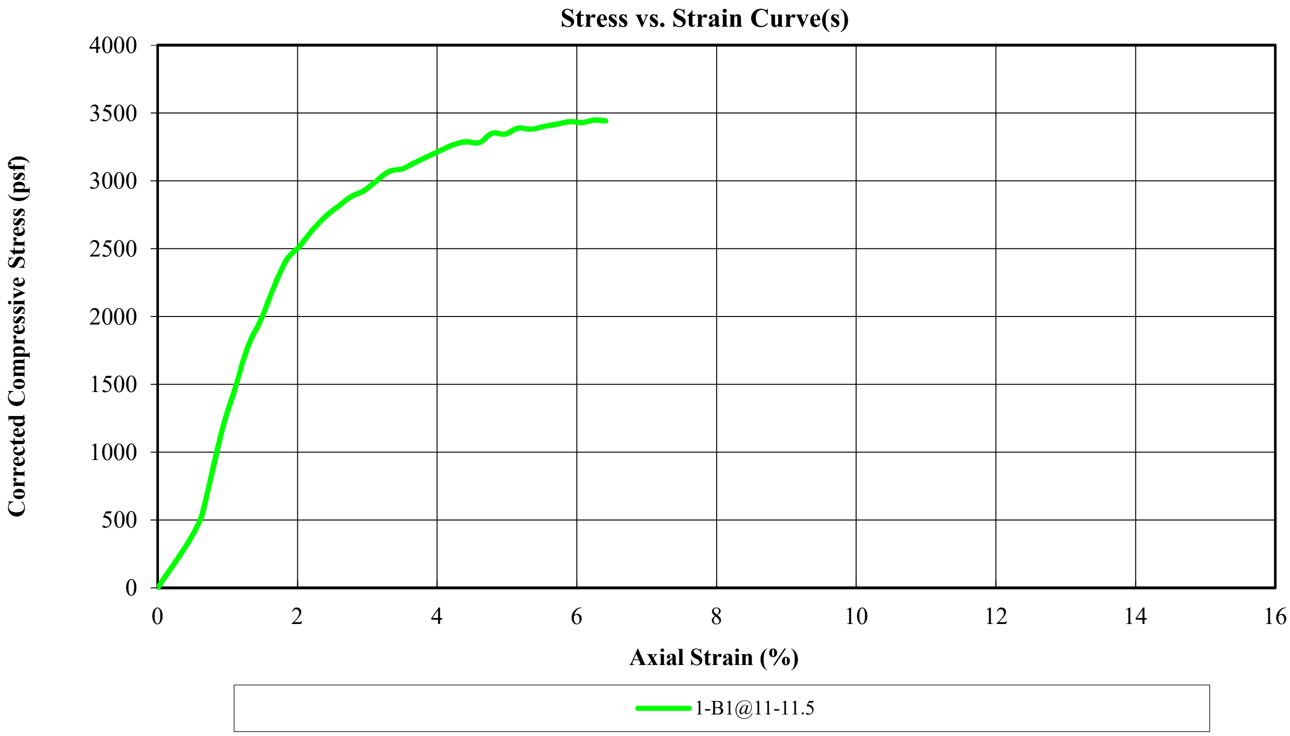
CLIENT: Charities Housing
PROJECT NAME: 1265 Montecito PGEX
PROJECT NO: 16572.000.000
PROJECT LOCATION: Mountain View, CA
REPORT DATE: 1/30/2020
TESTED BY: M. Quasem
REVIEWED BY: W. Miller

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B1@11-11.5
Moisture Content (%)	32.6
Dry Density (pcf)	88.8
Saturation (%)	99.8
Void Ratio	0.87
Diameter (in)	2.390
Height (in)	5.60
Height-To-Diameter Ratio	2.34
TEST DATA	
Unconfined Compressive Strength (psf)	3448
Undrained Shear Strength (psf)	1724
Strain Rate (in./min.)	0.05
Specific Gravity	2.655
Strain at Failure (%)	6.25
Liquid Limit	
Plastic Limit	
Test Remarks	

SPECIMEN	DESCRIPTION
1-B1@11-11.5	See exploration Logs

PROJECT NAME: 1265 Montecito PGEX
PROJECT NO: 16572.000.000
CLIENT: Charities Housing
LOCATION: San Bruno, California
PHASE NO: 002

Test Date: 01/29/2020
Tested By: W. Miller
Reviewed By: G. Criste

ENGEO Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

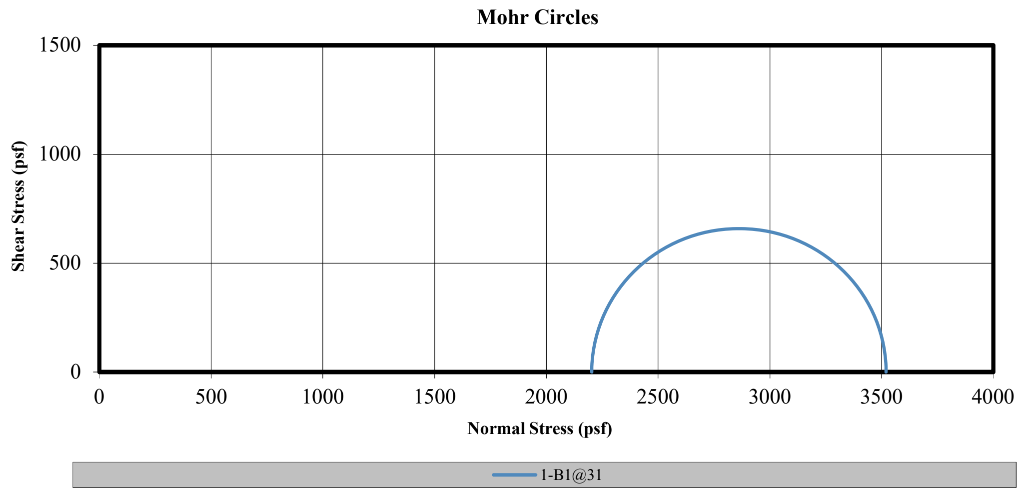
ASTM D2850

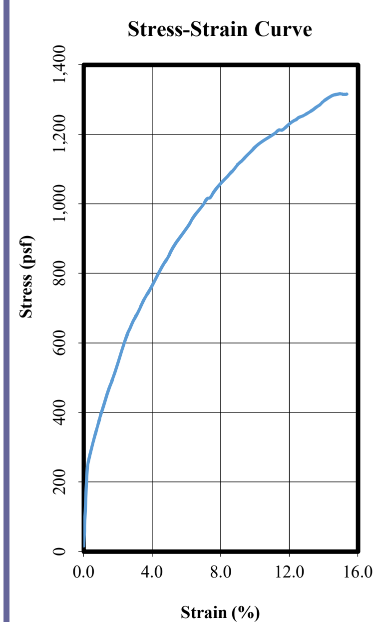
Date: 01/31/20

Checked By: G. Criste

Date: 1/30/2020

Tested By: M. Quasem





	Specimen			
Before Test	1-B1@31			
Water Content (%)	23.87			
Dry Density (pcf)	103.40			
Saturation (%)	99.44			
Void Ratio	0.66			
Diameter (in)	2.390			
Height (in)	5.010			
Height-to-Diameter Ratio	2.096			
ASTM D4318 - Wet Method				
Liquid Limit				
Plastic Limit				
ASTM D854 - Assumed				
Specific Gravity	2.750			
After Test	1-B1@31			
Water Content (%)	23.87			
Saturation (%)	99.44			
Strain Rate (in/min)	0.05			
Peak Deviator Stress (psf)	1316.9			
Axial Strain @ Failure (%)	14.972			
	Cell Pressure			
Cell (psf)	2203.2			
Back (psf)	n/a			
	Principle Stresses at Failure			
σ1 (psf)	3520.1			
σ3 (psf)	2203.2			
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	n/a		658.4			
Friction Angle Ø	n/a		n/a			

Project Information	
Project Name:	1265 Montecito PGEX
Project Number:	16572.000.000
Project Location:	San Bruno, CA
Client:	Charities Housing
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

ASTM D2850

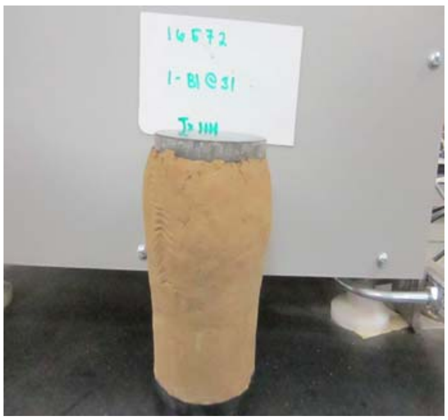
Date: 01/31/20

Checked By: G. Criste

Date: 1/30/2020

Tested By: M. Quasem

SPECIMEN PHOTOS

SAMPLE NUMBER: 1-B1@31	SAMPLE NUMBER:
	
SAMPLE NUMBER:	**SAMPLE NUMBER:**

Project Information	
Project Name:	1265 Montecito PGEX
Project Number:	16572.000.000
Project Location:	San Bruno, CA
Client:	Charities Housing
Description:	See exploration logs
Test Remarks:	

ENGEO
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APPENDIX D

CERCO ANALYTICAL RESULTS

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 **462 2771** Fax. 925 **462 2775**
www.cercoanalytical.com

Client:	ENGEO Incorporated
Client's Project No.:	16572.000.000
Client's Project Name:	1265 Montecito Avenue
Date Sampled:	24-Jan-20
Date Received:	29-Jan-20
Matrix:	Soil
Authorization:	Signed Chain of Custody

Date of Report: 12-Feb-2020

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2001197-001	1-B1 (1.5-2')	270	8.05	-	2,200	N.D.	N.D.	300

	Redox	pH	Conductivity	Resistivity	Sulfide	Chloride	Sulfate
Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	10-Feb-2020	10-Feb-2020	-	11-Feb-2020	31-Jan-2020	10-Feb-2020	10-Feb-2020

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX E

SUPPLEMENTAL RECOMMENDATIONS

SUPPLEMENTAL RECOMMENDATIONS

Prepared by
ENGEO Incorporated

ENGEO
Expect Excellence

TABLE OF CONTENTS

GENERAL INFORMATION

PREFACE

These supplemental recommendations are intended as a guide for earthwork and are in addition to any previous earthwork recommendations made by the Geotechnical Engineer. If there is a conflict between these supplemental recommendations and any previous recommendations, it should be immediately brought to the attention of ENGEO. Testing standards identified in this document shall be the most current revision (unless stated otherwise).

DEFINITIONS

BACKFILL	Soil, rock or soil-rock material used to fill excavations and trenches.
DRAWINGS	Documents approved for construction which describe the work.
THE GEOTECHNICAL ENGINEER	The project geotechnical engineering consulting firm, its employees, or its designated representatives.
ENGINEERED FILL	Fill upon which the Geotechnical Engineer has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with geotechnical engineering recommendations.
FILL	Soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.
IMPORTED MATERIAL	Soil and/or rock material which is brought to the site from offsite areas.
ONSITE MATERIAL	Soil and/or rock material which is obtained from the site.
OPTIMUM MOISTURE	Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.
RELATIVE COMPACTION	The ratio, expressed as a percentage, of the in-place dry density of the fill or backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557.
SELECT MATERIAL	Onsite and/or imported material which is approved by the Geotechnical Engineer as a specific-purpose fill.

PART I - EARTHWORK

1.0 GENERAL

1.1 WORK COVERED

Supplemental recommendations for performing earthwork and grading. Activities include:

- ✓ Site Preparation and Demolition
- ✓ Excavation
- ✓ Grading
- ✓ Backfill of Excavations and Trenches
- ✓ Engineered Fill Placement, Moisture Conditioning, and Compaction

1.2 CODES AND STANDARDS

The contractor should perform their work complying with applicable occupational safety and health standards, rules, regulations, and orders. The Occupational Safety and Health Standards (OSHA) Board is the only agency authorized in the State to adopt and enforce occupational safety and health standards (Labor Code § 142 et seq.). The owner, their representative and contractor are responsible for site safety; ENGEO representatives are not responsible for site safety.

Excavating, trenching, filling, backfilling, shoring and grading work should meet the minimum requirements of the applicable Building Code, and the standards and ordinances of state and local governing authorities.

1.3 TESTING AND OBSERVATION

Site preparation, cutting and shaping, excavating, filling, and backfilling should be carried out under the testing and observation of ENGEO. ENGEO shall be retained to perform appropriate field and laboratory tests to check compliance with the recommendations. Any fill or backfill that does not meet the supplemental recommendations shall be removed and/or reworked, until the supplemental recommendations are satisfied.

Tests for compaction shall be made in accordance with test procedures outlined in ASTM D-1557, as applicable, unless other testing methods are deemed appropriate by ENGEO. These and other tests shall be performed in accordance with accepted testing procedures, subject to the engineering discretion of ENGEO.

2.0 MATERIALS

2.1 STANDARD

Materials, tools, equipment, facilities, and services as required for performing the required excavating, trenching, filling and backfilling should be furnished by the Contractor.

2.2 ENGINEERED FILL AND BACKFILL

Material to be used for engineered fill and backfill should be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled.

Unless specified elsewhere by ENGEO, engineered fill and backfill shall be free of significant organics, or any other unsatisfactory material. In addition, engineered fill and backfill shall comply with the grading requirements shown in the following table:

TABLE 2.2-1: Engineered Fill and Backfill Requirements

US STANDARD SIEVE	PERCENTAGE PASSING
3"	100
No. 4	35–100
No. 30	20–100

Earth materials to be used as engineered fill and backfill shall be cleared of debris, rubble and deleterious matter. Rocks and aggregate exceeding the maximum allowable size shall be removed from the site. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

ENGEO shall be immediately notified if potential hazardous materials or suspect soils exhibiting staining or odor are encountered. Work activities shall be discontinued within the area of potentially hazardous materials. ENGEO shall be notified at least 72 hours prior to the start of filling and backfilling operations. Materials to be used for filling and backfilling shall be submitted to ENGEO no less than 10 days prior to intended delivery to the site. Unless specified elsewhere by ENGEO, where conditions require the importation of low expansive fill material, the material shall be an inert, low to non-expansive soil, or soil-rock material, free of organic matter and meeting the following requirements:

TABLE 2.2-2: Imported Fill Material Requirements

	SIEVE SIZE	PERCENT PASSING
GRADATION (ASTM D-421)	2-inch	100
	#200	15 - 70
PLASTICITY (ASTM D-4318)	Plasticity Index < 12	
ORGANIC CONTENT (ASTM D-2974)	Less than 3 percent	

A sample of the proposed import material should be submitted to ENGEO no less than 10 days prior to intended delivery to the site.

2.3 SUBDRAINS

A subdrain system is an underground network of piping used to remove water from areas that collect or retain surface water or subsurface water. Subsurface water is collected by allowing water into the pipe through perforations. Subdrain systems may drain and discharge to an appropriate outlet such as storm drain, natural swales or drainage, etc.. Details for subdrain systems may vary depending on many items, including but not limited to site conditions, soil types, subdrain spacing, depth of the pipe and pervious medium, as well as pipe diameter.

2.4 PIPE

Subdrain pipe shall conform with these supplemental recommendations unless specified elsewhere by ENGEO. Perforated pipe for various depths shall be manufactured in accordance with the following requirements:

TABLE 2.4-1: Perforated Pipe Requirements

PIPE TYPE	STANDARD	TYPICAL SIZES (INCHES)	PIPE STIFFNESS (PSI)
PIPE STIFFNESS ABOVE 200 PSI (BELOW 50 FEET OF FINISHED GRADE)			
ABS SDR 15.3		4 to 6	450
PVC Schedule 80	ASTM D1785	3 to 10	530
PIPE STIFFNESS BETWEEN 100 PSI AND 150 PSI (BETWEEN 15 AND 50 FEET OF FINISHED GRADE)			
ABS SDR 23.5	ASTM D2751	4 to 6	150
PVC SDR 23.5	ASTM D3034	4 to 6	153
PVC Schedule 40	ASTM D1785	3 to 10	135
ABS Schedule 40/DWV	ASTM D1527 & D2661	3 to 10	
PIPE STIFFNESS BETWEEN 45 PSI AND 50 PSI* (BETWEEN 0 TO 15 FEET OF FINISHED GRADE)			
PVC A-2000	ASTM F949	4 to 10	50
PVC SDR 35	ASTM D3034	4 to 8	46
ABS SDR 35	ASTM D2751	4 to 8	45
Corrugated PE	AASHTO M294 Type S	4 to 10	45

***Pipe with a stiffness less than 45 psi should not be used.**

Other pipes not listed in the table above shall be submitted for review by the Geotechnical Engineer not less 72 hours before proposed use.

2.5 OUTLETS AND RISERS

Subdrain outlets and risers must be fabricated from the same material as the subdrain pipe. Outlet and riser pipe and fittings must not be perforated. Covers must be fitted and bolted into the riser pipe or elbow. Covers must seat uniformly and not be subject to rocking.

2.6 PERMEABLE MATERIAL

Permeable material shall generally conform to Caltrans Standard Specification unless specified otherwise by ENGEO. Class 2 permeable material shall comply with the gradation requirements shown in the following table.

TABLE 2.6-1: Class 2 Permeable Material Grading Requirements

SIEVE SIZES	PERCENTAGE PASSING
1"	100
3/4"	90 to 100
3/8"	40 to 100
No. 4	25 to 40
No. 8	18 to 33
No. 30	5 to 15
No. 50	0 to 7
No. 200	0 to 3

2.7 FILTER FABRIC

Filter fabric shall meet the following Minimum Average Roll Values unless specified elsewhere by ENGEO.

Grab Strength (ASTM D-4632) .... 180 lbs
Mass per Unit Area (ASTM D-4751) ....6 oz/yd^2
Apparent Opening Size (ASTM D-4751) ....70-100 U.S. Std. Sieve
Flow Rate (ASTM D-4491)... 80 gal/min/ft^2
Puncture Strength (ASTM D-4833) .... 80 lbs

Areas to receive filter fabric must comply with the compaction and elevation tolerance specified for the material involved. Handle and place filter fabric under the manufacturer's instructions. Align and place filter fabric without wrinkles.

Overlap adjacent roll ends of filter fabric in accordance with manufacturer's recommendations. The preceding roll must overlap the following roll in the direction that the permeable material is being spread. Completely replace torn or punctured sections damaged during placement or repair by placing a piece of filter fabric that is large enough to cover the damaged area and comply with the overlap specified. Cover filter fabric with the thickness of overlying material shown within 72 hours of placing the fabric.

2.8 GEOCOMPOSITE DRAINAGE

Geocomposite drainage is a prefabricated material that includes filter fabric and plastic pipe. Filter fabric must be Class A. The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three-dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile.

A geotextile flap shall be provided along drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the core. The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes. If the fabric on the geocomposite drain is torn or punctured, replace the damaged section completely. The specific drainage composite material and supplier shall be preapproved by ENGEO.

The Contractor shall submit a manufacturer's certification that the geocomposite meets the design properties and respective index criteria measured in full accordance with applicable test methods. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor will supply design property test data from a laboratory approved by ENGEO, to support the certified values submitted.

Geocomposite material suppliers shall provide a qualified and experienced representative onsite to assist the Contractor and ENGEO at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications. The soil surface against which the geocomposite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. To prevent soil intrusion, exposed edges of the geocomposite drainage core edge must be covered.

Approved backfill shall be placed immediately over the geocomposite drain. Backfill operations should be performed to not damage the geotextile surface of the drain. Also during operations, avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than 7 days prior to backfilling.

PART II - GEOGRID SOIL REINFORCEMENT

Geogrid soil reinforcement (geogrid) shall be submitted to ENGEO and should be approved before use. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction to ultraviolet degradation and to chemical and biological degradation encountered in the soil being reinforced. The geogrids shall have an Allowable Tensile Strength (T_a) and Pullout Resistance, for the soil type(s) as specified on design plans.

The contractor shall submit a manufacturer's certification that the geogrids supplied meet plans and project specifications. The contractor shall check the geogrid upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geogrid will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geogrid damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geogrid material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed. The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil. Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least 6 inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided. During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed as shown on plans, and oriented correctly.

PART III - GEOTEXTILE SOIL REINFORCEMENT

The specific geotextile material and supplier shall be preapproved by ENGEO. The contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with specified test methods and standards.

The contractor shall check the geotextile upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geotextile shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geotextile will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geotextile damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geotextile material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project to assist the Contractor and ENGEO personnel at the start of construction. The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed, secured with staples, pins, or small piles of backfill, placed without wrinkles, and aligned with the primary strength direction perpendicular to slope contours. Cover geotextile reinforcement with backfill within the same work shift. Place at least 6 inches of backfill on the geotextile reinforcement before operating or driving equipment or vehicles over it, except those used under the conditions specified below for spreading backfill.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wraparound face system, as applicable.

The contractor may place only that amount of geotextile reinforcement required for immediately pending work to prevent undue damage. After a layer of geotextile reinforcement has been placed, the succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geotextile reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geotextile reinforcement and soil.

Geotextile reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geotextile reinforcement as slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geotextile reinforcement shall be placed directly on the compacted horizontal fill surface. Geotextile reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO.

Replace or repair any geotextile reinforcement damaged during construction. Grade and compact backfill to ensure the reinforcement remains taut. Geotextile soil reinforcement must be tested to the required design values using the following ASTM test methods.

TABLE III-1: Geotextile Soil Reinforcements

PROPERTY	TEST
Elongation at break, percent	ASTM D 4632
Grab breaking load, lb, 1-inch grip (min) in each direction	ASTM D 4632
Wide width tensile strength at 5 percent strain, lb/ft (min)	ASTM D 4595
Wide width tensile strength at ultimate strength, lb/ft (min)	ASTM D 4595
Tear strength, lb (min)	ASTM D 4533
Puncture strength, lb (min)	ASTM D 6241
Permittivity, sec^{-1} (min)	ASTM D 4491
Apparent opening size, inches (max)	ASTM D 4751
Ultraviolet resistance, percent (min) retained grab break load, 500 hours	ASTM D 4355

PART IV - EROSION CONTROL MAT

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels. The specific erosion control material and supplier shall be pre-approved by ENGEO.

The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. Jute mesh shall consist of processed natural jute yarns woven into a matrix, and netting shall consist of coconut fiber woven into a matrix. Erosion control blankets shall be made of processed natural fibers that are mechanically, structurally, or chemically bound together to form a continuous matrix that is surrounded by two natural nets.

The Contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 140°F, mud, dirt, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed by cutting out a section of the mat. The remaining ends should be overlapped and secured with ground anchors. Any erosion mat/blanket damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

Erosion control material suppliers shall provide a qualified and experienced representative onsite, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½ foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Anchors shall be as designated on the construction drawings, with a minimum of 12-inch length, and shall be spaced as designated on the construction drawings, with a maximum spacing of 4 feet.

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