STEVENS CREEK PROMENADE
SAN JOSE, CALIFORNIA

GEOTECHNICAL EXPLORATION

DRAFT

Submitted to
Mr. Perry Hariri
Fort Bay Stevens Creek, LLC
22 S. Santa Cruz Avenue, 2nd Floor
Los Gatos, CA 95030

Prepared by
ENGEO Incorporated

November 23, 2016

Project No.
13317.000.000

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Mr. Perry Hariri
Fort Bay Stevens Creek, LLC
22 S. Santa Cruz Avenue, 2nd Floor
Los Gatos, CA 95030

Subject: Stevens Creek Promenade
San Jose, California

GEOTECHNICAL EXPLORATION

Dear Mr. Hariri:

As requested, we completed this geotechnical exploration for the proposed Stevens Creek Promenade mixed-use retail and residential development in San Jose, California. The planned development will include several 6-story residential structures, a 6-story office building, a 6-level parking garage with several below grade levels, exterior courtyard, landscape, and related site improvements. The accompanying report presents our findings along with our conclusions and recommendations for the proposed development.

It is our opinion from a geotechnical standpoint that the site is suitable for the proposed development provided our recommendations are incorporated into the project plans and design during construction. The main geologic and geotechnical concerns at the site include presence of expansive soils; existing “man-made” fill associated with previous site development; presence of alluvial soil susceptible to settlements with high structural loads; seismic induced settlement potential; proper grading; and foundation support. Our recommendations to address these concerns are presented in the accompanying report.

We are pleased to have been of service to you on this project and are prepared to consult further with you and your design team as the project progresses.

Sincerely,

ENGEO Incorporated

Mathew D. Clark, EIT

Theodore P. Bayham, GE, CEG

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical report, as described in our proposal dated August 18, 2016, is to provide design-level geotechnical recommendations associated with the proposed mixed-use development at the site.

We performed the following services:

- Review of available literature, previous reports and geologic maps for the study area.
- Subsurface exploration consisting of five cone penetrometer test (CPT) probes and four soil borings.
- Laboratory testing of materials sampled during the field exploration.
- Geotechnical data analyses.
- Report of our conclusions and recommendations for the proposed development.

Our recommendations are based on plans by KTGY Architects dated August 10, 2016. We prepared this report exclusively for Fort Bay Stevens Creek, LLC and their design team consultants. ENGEO should review any changes made in the character, design or layout of the development to modify the conclusions and recommendations contained in this report, as necessary. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO.

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1.2 SITE LOCATION AND PROPOSED DEVELOPMENT

The approximately 10-acre, rectangular-shaped site is generally bounded by Stevens Creek Boulevard to the north, Albany Drive to the south, and car dealerships to the east and west. Lopina Way roughly divides the site (Figure 1) into halves. Currently, the property is occupied by one- to two-story retail and office structures, paved parking lots, and landscaped areas.

Based on plans and drawings by KTGY Architects (dated August 10, 2016), the proposed development will include two 6-story residential structures, a 6-story office building, a 6-level parking garage with several below grade levels, exterior courtyard, landscape and related site improvements.

1.3 AERIAL PHOTOGRAPH REVIEW

We reviewed historic aerial photographs of the site dated 1948 through 2016 available on Google Earth and www.historicaerials.com. Review of the photographs indicates that the site was used as agricultural land with at least two houses located onsite through at least 1968, with the construction of the current development substantially completed by 1980. Stevens Creek Boulevard was widened between 1960 and 1968. The current configuration of onsite structures appears to have changed little since 1980, with the most notable change in subsequent photos being the continuation of other development in the general site vicinity.

2.0 GEOLOGIC CONDITIONS

2.1 REGIONAL AND LOCAL GEOLOGY

The site is located within the Coast Ranges geomorphic province, dominated by a series of northwest-trending, fault-bounded mountain ranges and intervening alleviated valleys. As depicted on Figure 3, regional geologic maps by Wentworth et al (Preliminary Geologic Map of the San Jose Quadrangle, 1999), the property is underlain by Holocene-age alluvial fan deposits (Qhf2). Dibblee (2007) maps the site as underlain by "Qa1" surficial sediments geologic unit, consisting of alluvial fine-grained sand, silt, and gravel representing alluvial fan deposits at base of slopes and upper fan areas.

2.2 REGIONAL FAULTING

The site is not located within a State of California Earthquake Fault Hazard Zone for active faults, and no known faults cross the site. Nearby active[1], or potentially active faults include Monte Vista – Shannon Fault located approximately 4 miles southwest of the site; San Andreas Fault located approximately 7½ miles southwest of the site; and Hayward Fault located approximately 9½ miles northeast of the site.

Stanford Fault is located approximately ½ mile northeast of the site, and listed on the USGS fault database as a Quaternary-aged fault (evidence of movement in the past 1.6 million years), and therefore is considered potentially active.

Because of the presence of active faults in the Bay Area Region, it is considered seismically active. Numerous small earthquakes occur every year in the region, and large (>M7) earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region.

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3.0 FIELD EXPLORATION

The field exploration for the current study included advancing five cone penetration test (CPT) probe soundings within the site on September 21, 2016, and drilling four exploratory borings within the project site on September 23, 2016. Figure 2 presents the approximate locations of the exploratory borings and CPT probes obtained by taping or pacing from existing features. As a result, the mapped locations are only as accurate as the methods used to determine them.

Our five CPTs were advanced to depths ranging between approximately 30 feet to 65 feet below ground surface (bgs) as shown on Figure 2. The CPT equipment utilized a 20-ton compression-type cone with a 15 square centimeter (cm^2) base area, an apex angle of 60 degrees, and friction sleeve with surface area of 225 cm^2. The cone is pushed into the ground at a constant rate, and readings taken at approximately 5-cm intervals; the penetration rate is approximately 2 cm per second in accordance with revised (2002) ASTM standards (D-5778-95). Measurements include the tip resistance to penetration of the cone (Q_c), the resistance of the surface sleeve (F_s), and dynamic pore pressure (U). Groundwater was not encountered in any of the CPT probe holes. The CPT holes were backfilled with

[1] An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997).

cement-bentonite grout upon completion. The CPT logs and supporting empirical data are located in Appendix B.

The exploratory borings for our current study consisted of drilling four borings to depths ranging between approximately 16½ to 71½ feet bgs, see Figure 2. The borings were drilled using a truck-mounted rig using 4-inch-diameter solid-flight auger, and mud rotary drilling methods. We logged borings and soil samples were collected using either 2½-inch inside diameter (I.D.) California-type split-spoon sampler fitted with 6-inch-long brass liners or a 2-inch outside diameter (O.D.) Standard Penetration Test (SPT) split-spoon sampler. The penetration of the samplers into underlying materials was measured and recorded, which is number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs record blow count results as the actual number of blows required for the last 1 foot of penetration; no conversion factors have been applied. The samplers were driven with a 140-pound hammer falling a distance of 30 inches employing an automatic hammer system. The field logs were then used to develop the report boring logs, which are presented in Appendix A.

The boring logs depict subsurface conditions within the borings at the time of the exploration. Subsurface conditions at other locations may differ from conditions occurring at these boring locations, and the passage of time may result in altered subsurface conditions. In addition, stratification lines represent the approximate boundaries between soil types, and the transitions may be gradual.

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3.1 LABORATORY TESTING

We performed the following laboratory tests on select samples recovered during boring and CPT operations:

TABLE 3.1-1: Laboratory Testing

SOIL CHARACTERISTIC	TESTING METHOD	LOCATION OF RESULTS
Natural Unit Weight and Moisture Content	ASTM D7263	Appendix A
Moisture Content Only	ASTM D2216	Appendix A
Atterberg Limits	ASTM D4318	Appendix C
Specific Gravity	ASTM D6913	Appendix C
Sieve and Hydrometer	ASTM D422	Appendix C
#200 Wash	ASTM D1140	Appendix C
Unconfined Compression	ASTM D2166	Appendix C
Unconsolidated Undrained Triaxial Test	ASTM D2850	Appendix C
Water Soluble Sulfates in Soils	ASTM C1580	Appendix C
Corrosivity Testing (Redox, pH, Resistivity, Chloride, Sulfate)	ASTM D1498, D4972, G57, D4327	Appendix D

The laboratory test results are shown on the borelogs (Appendix A), with individual test results presented in Appendix C.

Corrosivity analysis was performed on a representative soil sample. The sample was delivered to CERCO Analytical, Inc. and tested according to above ASTM Test Methods for redox potential, pH, resistivity, sulfate, and chloride ion concentrations. The test results provide an

indication of soil corrosivity at the site, which may influence buried concrete structures and metal pipes or require protection from corrosion. A detailed description of the laboratory results is contained in the attached report prepared by CERCO Analytical, Inc. in Appendix D.

3.2 SUBSURFACE CONDITIONS

In general, our borings and CPTs encountered surface deposits of hard alluvial soils comprised of sandy and clayey soils within upper 25 feet bgs. This layer included interlayers of silts and fine gravels. Below the upper strata zone, the borings and CPTs encountered approximately 10 to 15 feet of medium dense to very dense, silty and clayey sands. At depths greater than 40 feet bgs, interlayered deposits of lean and fat clays, silty and clayey sands, and gravels. At 1-B1 and 1-CPT2, a zone of stiff soils was encountered from approximately 45 to 50 feet bgs.

Throughout the soil profile sampled during our field exploration, we noted soils were very stiff to hard or medium dense to very dense, representing a generally competent soil profile. The CPT probes indicated encountered a similar stratigraphic profile when compared to the exploratory borings.

Three samples from the near-surface and upper 15 feet below existing ground surface (bgs) of the sandy lean clay were tested for Plasticity Index (PI) and yielded PI values ranging from 12 to 17. The results indicate a low to moderate expansion potential for the site soils. Plasticity Index test results are included on the borelogs in Appendix A and are included in their entirety in Appendix C.

3.3 GROUNDWATER

Groundwater levels were not determined at any of our CPT probes, which extended to depths of approximately 65 feet bgs. Also, we did not encounter free groundwater in exploratory borings extending to approximately 21 feet bgs. The presence of free water during boring advancement could not be determined due to the use of mud rotary drilling methods. Regional groundwater maps report Historic high groundwater level is between about 30 and 40 feet bgs for the vicinity of the site (CGS, San Jose West Quadrangle, 2002). It should be recognized that fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, changes in drainage patterns, and irrigation.

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4.0 GEOLOGIC AND GEOTECHNICAL HAZARDS

4.1 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, ground lurching, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections. Based on topographic and lithologic data, risk from earthquake-induced regional subsidence/uplift, tsunamis, and seiches is considered low to negligible at the site.

4.1.1 Ground Rupture

The site is not located within a State of California Earthquake Fault Hazard Zone and no known faults cross the site (CDMG, 2002). Therefore, it is our opinion that the risk of ground rupture is low at the subject property.

4.1.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. 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 considered to be substantially smaller than the comparable 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 a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

4.1.3 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form. The potential for the formation of these cracks is considered greater at contacts between deep alluvium or fill and bedrock. Such an occurrence does not appear likely at this site and any offset or strain is expected to be minor.

4.1.4 Soil Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded fine sands below the groundwater table. Review of the Seismic Hazard Zones Map for the San Jose West Quadrangle (CDMG, 2002) indicates that the site is not located within an area prone to earthquake-induced liquefaction. To assess liquefaction potential, we performed liquefaction analyses on the five CPT probes advanced during the current field exploration. We assigned a design groundwater level of 35 feet below the existing ground surface, a PGA of 0.54g, and an M_w of 7.9.

Our CPT analyses were based on guidelines provided in DMG Special Publication 117A (2008) and methods developed by Youd et al. (NCEER 1998) (2001), Moss et al. (2006), and Idriss & Boulanger (2008), with post-liquefaction settlement according to Zhang et al (2002).Based on our analysis (Appendix D), it is our opinion that for the MCE-level earthquake, the site may experience up to ½ inch of seismic induced settlement due to vertical deformation strain associated with the sandy soil layer at approximately 38 to 41 feet bgs.

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We also evaluated the secondary deformation surface effects of seismic induced settlement, where the relationship of cap thickness of non-liquefiable soils is considered per reports and guidelines by Ishihara (1985), Youd and Garris (1995). Given the proposed final grades at the site, there is sufficient non-liquefiable soils for structures to preclude secondary ground deformation (e.g. surface rupture or sand boils) during a strong seismic event.

4.1.5 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 the lack of steep slopes or nearby open channels, our assessment is that the potential for lateral spreading at the site is considered low.

4.2 LOAD-INDUCED SETTLEMENT

Soils are subject to consolidation settlement when a new loading scenario is introduced by structures, earthwork, or equipment. The amount of consolidation settlement is dependent on the magnitude and duration of the applied load, the shape and size of the applied load area, and the depth, thickness, and stress history of the compressible soils. The time required for primary consolidation settlement to occur is highly dependent on the permeability of the deposit. Consequently, sandy soils will settle almost immediately, whereas clayey soils will settle much more slowly.

For the proposed project, we anticipate moderate to heavier building loads due to the proposed structure types and number of stories. At this time, we understand that one or two below-grade levels are being considered for some of the buildings onsite. Should any below-grade levels be incorporated into the building designs, we anticipate higher bearing pressures can be safely utilized due to the unloading effect of the removed soil compensating for the additional structural loading. We can provide bearing pressures specific to depths of excavation as supplemental recommendations at your request. For structures founded near the ground surface, our calculations indicate total immediate and consolidation settlement will be on the order of approximately 1 inch.

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4.3 EXISTING FILL

Although our borings did not encounter appreciable thicknesses of undocumented fills, it is possible that there may be deposits of existing undocumented fills associated with previous site activities based on prior agricultural use. Existing fills can be expected around the existing structures, as utility trench backfill, and/or any former underground storage tanks (USTs), if present. Existing fills could undergo vertical movement that is not easily characterized and could ultimately be inadequate to effectively support the proposed building loads. In general, undocumented fills should be excavated and replaced as engineered soil fill. The extent and quality of existing fills should be evaluated and mitigated during grading activities.

4.4 EXPANSIVE SOILS

The results of laboratory testing show Plasticity Indices (PI) ranging from 12 to 17, which is considered generally low expansion potential. Expansive soils may shrink and swell with fluctuation in moisture content, which may cause ground heaving and potentially crack slabs-on-grade, pavements, and structures founded on shallow foundations. Successful

construction on expansive soils requires special attention during grading. It is imperative to keep exposed soils moist by occasional sprinkling. If the soil is dry, it is extremely difficult to moisturize the soils (because of their clayey nature) without excavation, moisture conditioning, and recompaction.

Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil, and use of a mat foundation (either post-tensioned or conventionally reinforced) are common, generally cost-effective measures to address the expansive potential of the foundation soils. Based upon our initial findings, the effects of expansive soils are expected to pose a low impact when mitigated.

4.5 FLOODING

The project Civil Engineer should be consulted on the potential for localized flooding at the subject site and review the relationship of planned development to 100-year flood zones.

4.6 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS

Based on the subsurface conditions encountered and local seismic sources, the following 2013 CBC seismic design parameters should be used for design.

TABLE 4.6-1: 2013 CBC Seismic Information

PARAMETER	DESIGN VALUE
Site Class	D
Mapped MCE_R spectral response accelerations for short periods, S_S (g)	1.50
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	1.50
MCE spectral response accelerations for short periods, S_{MS} (g)	1.50
MCE spectral response accelerations for 1-second periods, S_{M1} (g)	0.90
Design spectral response acceleration at short periods, S_{DS} (g)	1.00
Design spectral response acceleration at 1-second periods, S_{D1} (g)	0.60
Mapped MCE Geometric Mean Peak Ground Acceleration (g)	0.54
Site Coefficient, F_{PGA}	1.00
MCE Geometric Mean Peak Ground Acceleration, PGA_M (g)	0.54
Long period transition-period, T_L	12 sec

MCE_R = Risk-Targeted Maximum Considered Earthquake
MCE = Maximum Considered Earthquake
Latitude: 37.32232; Longitude: -121.97862

4.7 CORROSIVITY CONSIDERATIONS

A soil sample was collected during our study and transported under proper chain-of-custody to CERCO Analytical, Inc. for laboratory testing. The samples were tested for redox potential, pH, resistivity, soluble sulfate, and chloride ion concentrations. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes. The results are summarized in the following table with a detailed description of the laboratory

results contained in the report prepared by CERCO Analytical, Inc. (Appendix C). In addition to the corrosivity testing suite performed by CERCO, two additional samples were tested for sulfate concentrations by ENGEO.

TABLE 4.7-1: Soil Corrosivity Test Results

SAMPLE NUMBER AND DEPTH	REDOX POTENTIAL (MV)	PH	RESISTIVITY* (OHM-CM)	SOLUBLE SULFATE* (MG/KG)	CHLORIDE ION* (MG/KG)
1-B1 @ 11'	-	-	-	N.D.	-
1-B2 @ 15½'-16½'	480	7.20	3,600	N.D.	N.D.
1-B4 @ 2½'	-	-	-	N.D.	-

* Results reported on a wet weight basis
N.D – None detected

Due to the resistivity measurements indicating a "moderately corrosive" potential, consideration should be given to protect buried metal and steel against corrosion. A corrosion consultant should provide specific design recommendations on corrosion protection for important buried metallic lines.

According to the sulfate test results from the three samples, the sulfate ion concentration resulted in N.D. for water-soluble sulfate (SO_4). The CBC references the American Concrete Institute Manual, ACI 318 (Chapter 4, Sections 4.2 and 4.3) for concrete requirements. ACI tables provide the following sulfate exposure categories and classes and concrete requirements in contact with soil based upon the exposure risk.

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TABLE 4.7-2: Sulfate Exposure Categories and Classes

SULFATE EXPOSURE CATEGORY S	EXPOSURE CLASS	WATER- SOLUBLE SULFATE IN SOIL % BY WEIGHT
Not Applicable	S0	$SO_4 < 0.10$
Moderate	S1	$0.10 \leq SO_4 < 0.20$
Severe	S2	$0.20 \leq SO_4 \leq 2.00$
Very Severe	S3	$SO_4 > 2.00$

TABLE 4.7-3: Requirements for Concrete by Exposure Class

EXPOSURE CLASS	MAX W/CM	MIN F'C (PSI)	CEMENT TYPE			CALCIUM CHLORIDE ADMIXTURE
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No Type restriction	No Type restriction	No Type restriction	No restriction
S1	0.5	4000	II†‡	IP(MS), IS(<70), (MS)	MS	No restriction
S2	0.45	4500	V‡	IP(HS), IS(<70), (HS)	HS	Not permitted

EXPOSURE CLASS	MAX W/CM	MIN F'C (PSI)	CEMENT TYPE			CALCIUM CHLORIDE ADMIXTURE
			ASTM C150	ASTM C595	ASTM C1157	
S3	0.45	4500	V + pozzolan or slag§	IP(HS) + pozzolan or slag or IS(<70) (HS) + pozzolan or slag§	HS + pozzolan or slag§	Not permitted

Notes: † For seawater exposure, other types of portland cements with tricalcium aluminate (C_3A) contents up to 10 percent are permitted if the w/cm does not exceed 0.40.

‡ Other available types of cement such as Type III or Type I are permitted in Exposure Classes S1 or S2 if the C_3A contents are less than 8 or 5 percent, respectively.

§ The amount of the specific source of the pozzolan or slag to be used shall not be less than the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slag to be used shall not be less than the amount tested in accordance with ASTM C1012 and meeting the criteria in ACI 4.5.1.

In accordance with the above criteria, the soil sample is classified in the S0 sulfate exposure class. The minimum concrete strength for this exposure class is specified by the CBC in the table above. As minimum requirements, we recommend that Type II cement be used in foundation concrete for structures at the project site and concrete should incorporate a maximum water cement ratio of 0.5 and a minimum compressive strength of 3,000 psi. It should be noted, however, that the structural engineering design requirements for concrete might result in more stringent concrete specifications.

4.8 CONCLUSIONS

It is our opinion from a geotechnical standpoint that the site is suitable for the proposed development, provided that our recommendations are incorporated into the project plans and designs, and implemented during construction. The main geologic and geotechnical concerns at the site include: presence of expansive soils; existing "man-made" fill associated with previous site development; presence of alluvial soil susceptible to settlements with high structural loads; seismic induced settlement potential; proper grading and foundation support. Our recommendations to address these concerns are presented in the following sections.

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5.0 RECOMMENDATIONS

5.1 GRADING

We anticipate that grading will consist of minor cuts and fills within the site to achieve finished grades. Grading operations should meet the requirements of the Supplemental Recommendations (Appendix E) and should be observed and tested by ENGEO's field representative. ENGEO should be notified a minimum of 3 days prior to grading in order to coordinate its schedule with the grading contractor.

5.1.1 Demolition and Stripping

Site demolition includes the removal of structures, foundations, and buried structures, including abandoned utilities and septic tanks and their leach fields, if any exist. Debris and soft compressible soils should be also removed from any location to be graded, from areas to receive fill or structures, or those areas to serve as borrow. The depth of removal of such materials should be determined by the Geotechnical Engineer in the field at the time of grading.

Existing surficial fill and disturbed soils may be found across the site, though they were not noted in the borings.

The existing vegetation should be removed from areas to receive fill or improvements, or 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 and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill. No loose or uncontrolled backfilling of depressions resulting from demolition or stripping is permitted. If planned for reuse, demolition recommendations for paved parking areas and existing buildings are provided in the subsequent sections.

For the existing asphalt pavements may be reused as engineered fill from a geotechnical standpoint. The material should be broken down, but not pulverized, to meet a 6 inch or less particle size and placed in a separate stockpile outside the limits of grading. The asphaltic concrete and aggregate base should be thoroughly mixed and placed as engineered fill. The reuse of concrete materials is suitable provided these are cleared of unsuitable debris and deleterious material.

If recycled pavements and construction materials are utilized at the site, we recommend full disclosure to future homeowners/tenants be provided. As a minimum, disclosed information should include the presence of asphaltic concrete and Portland cement concrete fill materials at the site.

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5.1.2 Selection of Materials

With the exception of construction debris (ie. wood, brick, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils, we anticipate the site soils are suitable for use as engineered fill. Unsuitable materials and debris, including trees with their root balls, 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.

5.2 EXISTING FILLS

Although our borings did not encounter appreciable thicknesses of undocumented fills, it is possible that there may be deposits of existing undocumented fills associated with previous site activities based on prior agricultural use. Existing fills can be expected around the existing structures, as utility trench backfill, and/or any former underground storage tanks (USTs), if present. Existing fills could undergo vertical movement that is not easily characterized and could ultimately be inadequate to effectively support the proposed building loads. In general, undocumented fills should be excavated and replaced as engineered soil fill. The extent and quality of existing fills should be evaluated and mitigated during grading activities.

5.3 DIFFERENTIAL FILL THICKNESS

Depending upon cuts associated with removal of undocumented fills, differential fill thickness conditions could possibly arise. 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 differential fill thickness of up to 10 feet is acceptable across a building footprint. 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 be replaced as engineered fill. As a minimum, the subexcavation area should include the entire structure footprint plus 5 feet beyond the edges of the building footprint.

5.4 FILL PLACEMENT

Once a suitable firm base is achieved, the exposed non-yielding native surface should be scarified to a depth of 10 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. All fills should be placed in thin lifts, with the lift thickness not to exceed 10 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following specifications applies to onsite clayey soils reused as engineered fill:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 2 percentage points above optimum moisture content.
Required Relative Compaction:	Not less than 90 percent.

The following specifications applies to non-expansive sandy soils, import fills, chemically treated soil used as engineered fill:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than optimum moisture.
Minimum Relative Compaction:	Not less than 90 percent.

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The following compaction control requirements should be applied to Class 2 Aggregate Base used in pavement construction:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than optimum moisture.
Minimum Relative Compaction:	Not less than 95 percent.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material.

5.5 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. In addition, wet soil conditions may be encountered near the bottom of excavations. Wet soil can make proper compaction difficult or impossible.

5.6 TEMPORARY EXCAVATIONS AND SHORING

The Contractor should be familiar with applicable local, state, and federal regulations, including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. It is the responsibility of the Contractor to provide stable, safe trench and construction slope conditions and to follow OSHA safety requirements. Since excavation procedures may be dangerous, it is also the responsibility of the Contractor to provide a trained "competent person" as defined by OSHA to supervise all excavation operations, ensure that all personnel are working in safe conditions and have thorough knowledge of OSHA excavation safety requirements.

We anticipate shoring will be utilized during excavation of below-grade level(s). Typical shoring including driven sheet piles or soldier piles with lagging may be appropriate for this project. Supplemental recommendations for shoring design will be provided if needed. The contractor should be responsible for the design and construction of all shoring systems, the monitoring and protection of adjacent improvements, and the safety of all workers within the excavation.

5.7 GRADED SLOPES

In general, graded slopes should be no steeper than 2:1 (horizontal:vertical). 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.

5.8 MONITORING AND TESTING

It is important that all site preparations for site grading be done under the observation of the Geotechnical Engineer's field representative. The Geotechnical Engineer's field representative should observe all graded area preparation, including demolition and stripping, following the recommendations contained herein and in the Supplemental Recommendations. The final grading and foundation plans should be submitted to the Geotechnical Engineer for review.

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5.9 FOUNDATION DESIGN

Shallow foundations are suitable for this site. Based on available shallow foundation systems one of the following may be used, stiff reinforced structural mats, post-tensioned slabs on ground, shallow continuous and isolated column footings combined with floor slab-on grade.

5.9.1 Structural Mat Foundations

We estimate that potential total settlement (seismic plus load induced settlements) at the site may range up to approximately 1½ inch. The foundation should be designed to accommodate the anticipated settlements. We recommend that structural mat foundations be designed for an edge-cantilever distance of 6 feet, and unsupported interior free span of 20 feet. In addition, the mat should be designed to be stiff and resist differential movement ½ inch over a 20-foot distance. An allowable bearing pressure of 2,000 psf (uniformly distributed over the mat footprint) can be considered for dead-plus-live loads. These values may be increased by one-third when considering total loads with wind or seismic loads.

5.9.2 Post-Tensioned Slabs on Ground

Similar to structural mat foundations, the foundation should be designed to accommodate anticipated settlements of up to approximately 1½ inch. In addition, the mat should be designed to be stiff and resist differential movement of ½ inch over a 20-foot distance. An allowable bearing pressure of 2,000 psf (uniformly distributed over the mat footprint) can be considered for dead-plus-live loads. These values may be increased by one-third when considering total loads with wind or seismic loads.

Based upon the existing soil conditions, and using the 2004 (Third Edition) Post-Tensioning Institute, “Design of Post-Tensioned Slabs-On-Ground” manual to develop our soil parameters, we recommend the following soil criteria.

TABLE 5.9.2-1: Post-Tension Mat Foundation Design Criteria

CONDITION	CENTER LIFT	EDGE LIFT
Edge Moisture Variation Distance, e_m (feet)	9.0	4.9
Differential Soil Movement, y_m (inches)	0.6	0.9

5.9.3 Shallow Continuous and Isolated Footings

The proposed buildings may be supported on shallow continuous and isolated footings. The footings should be a minimum of 18 inches wide, and the upper 24 inches of building pad subgrade should be comprised of low-expansive fill (Plasticity Index of 15 or less). Footing trenches should be cleared of loose soil prior to concrete placement. It is important that footing trenches not be allowed to desiccate prior to placing concrete.

Footings should be designed as follows:

Minimum depth of footing embedment: 24 inches below lowest adjacent soil subgrade elevation.

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Maximum allowable footing pressure:	3,500 psf dead plus live loads. Increase by one-third when considering total loads including wind or seismic loads.

The footings should be designed for total (load-induced plus seismic) settlement of up to 1½ inch and differential settlement of up to ½ inch over 20 feet. 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.

Resistance to lateral loads can be obtained from a combination of passive earth pressures against the face of the foundation and soil friction along the base of the foundation. To resist lateral loads, a passive pressure corresponding to an equivalent fluid weighing 300 pounds per cubic foot (pcf) and a base friction factor of 0.30 times net vertical dead load should be used. Lateral resisting pressures in the upper 1 foot should be neglected if the footings are not confined by slab or pavement.

Construction joints between the perimeter footings and floor slab-on-grade should be located at least 4 inches above adjacent exterior grade.

5.9.4 Foundation Subgrade Treatment

The subgrade material under structural mats, post-tensioned foundations, and slab-on-grade should be uniform. The upper 18 inches of pad subgrade for foundations should consist of engineered fill. The pad subgrade should be moisture conditioned to a moisture content of at least 2 percentage points above optimum. The subgrade should be thoroughly soaked and approved by the Geotechnical Engineer prior to placing reinforcement or tendons. The subgrade should not be allowed to dry prior to concrete placement.

A tough, water vapor retarding membrane should be installed below the mat foundations and floor slabs to reduce moisture condensation under floor coverings. The vapor retarder should meet ASTM E 1745 – 11 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.

5.9.5 Slabs on Grade

Interior concrete floor slabs may be used with footings provided the upper 2 feet of the pads consist of fill or native soil with relatively low expansion potential. 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. When buildings are constructed with concrete slab-on-grade, 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

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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 1745-97 "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 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 0.45.

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.

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5.10 BUILDING RETAINING WALLS

We understand that some buildings may include below-grade retaining walls (up to a maximum of 30 feet in height). We recommend that below-grade walls be designed as restrained walls. The walls may be designed for at-rest equivalent fluid weight of 60 pounds per cubic foot (pcf) which assumes behind wall drainage and level backfill conditions at the top of the wall. However, if walls are not drained these walls need to be resist hydro-static pressures increased to 100 pcf for level backfill conditions, and waterproofing of below grade portions will be necessary.

Also, buildings with retaining walls exceeding 12 feet or greater should be checked for seismic performance of the basement retaining walls be evaluated using an active equivalent fluid weight of 40 pcf, and a seismic increment of 20 pcf, in accordance with Lew, et al. (2010). This evaluation should be separate from the static design using at-rest earth pressures.

Appropriate surcharge loads from buildings, hardscape and vehicles should be incorporated when the surcharge loading is situated above a 1:1 (horizontal:vertical) line of projection extending up the rear base edge of the footing. We also recommend incorporating a nominal vertical surcharge load to allow moderate construction equipment to place wall backfill. A uniform, horizontal surcharge loading of 50 percent of the vertical surcharge load may be assumed to act over the upper 10 feet of the wall.

Below grade walls may be supported on shallow foundations in accordance with recommendations presented in Section 5.9 of this report. Alternatively, if walls are designed integral with structural mats the walls should impose a maximum uniform loads (dead plus live)

of 1,800 psf or less on the mat. This value may be increased by ⅓ when considering seismic and wind loading. Below grade retaining walls should be drained and waterproofed as discussed in the subsequent sections.

5.11 SITE RETAINING WALLS

This section is intended for retaining walls not integrated with buildings, such as outside of the buildings for grades separations or landscaping. Unrestrained drained retaining walls constructed on level ground and up to 6 feet in height may be designed using active equivalent fluid pressures as follows.

TABLE 5.11-1: Active Equivalent Fluid Pressures

BACKFILL SLOPE CONDITION (HORIZONTAL:VERTICAL)	ACTIVE PRESSURE (POUNDS PER CUBIC FOOT)
Level	40
3:1	60
2:1	70

Passive pressures acting on site retaining wall foundations may be assumed as 300 pounds per cubic foot (pcf) provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation and keyway, whichever is greater. The upper 1 foot of soil should be excluded from passive pressure computations. The friction factor for sliding resistance may be assumed as 0.30. It is recommended that retaining wall footings be designed using an allowable bearing pressure of 3,500 psf. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

5.12 RETAINING WALL DRAINAGE

In general, all walls retaining more than 2 feet of soil should be provided with drainage facilities to prevent the build-up of hydrostatic pressures behind the walls. Wall drainage may be provided using a 4-inch-diameter perforated pipe embedded in either free-draining gravel surrounded by synthetic filter fabric (minimum 6-ounce) or Class 2 permeable material (Part 2 of Supplemental Recommendations, Section 2.05B). The width of the drain blanket should be at least 12 inches, and the drain blanket should extend to about 1 foot below the finished grades. The upper 1 foot of wall backfill should consist of compacted site soils. Drainage should be collected into solid pipes and directed to an outlet approved by the Civil Engineer. Synthetic filter fabric should meet the minimum requirement listed in the Supplemental Recommendations and be preapproved by the Geotechnical Engineer prior to delivery.

Design details for draining any below grade retaining walls should be determined during the design process. A sump system may be needed for drainage unless the storm drain system will allow for gravity connection and outfall. 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 68-1.025) placed directly behind the wall, or

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2. A minimum 12-inch-thick layer of washed, crushed rock. 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 a depth of 5 feet below the existing ground surface to within 12 inches of the top of the wall.
- Place a minimum of 4-inch-diameter perforated pipe at the base of the drain material, inside the rock drain and fabric, with perforations placed down.
- Place pipe at a gradient of at least ½ percent to direct water away from the wall by gravity to a sump or 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 fill placement recommendations. Use light compaction equipment within 5 feet of the wall face. If moderate to heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement. Alternatively, the wall design can incorporate additional surcharge loading to allow moderate to heavy equipment.

5.13 EXTERIOR SLAB-ON-GRADE CONSTRUCTION

This section provides guidelines for exterior slabs such as driveways, patios, exterior walkways, steps, and sidewalks. As a minimum, we provide the following considerations and recommendations.

- Exterior slabs-on-grade should be constructed structurally independent of adjacent foundation systems. This allows slab movement to occur with a reduced potential for foundation distress.
- Cracking of slabs should be expected as a result of concrete shrinkage and the expansive soils at the site. Frequent control joints should be provided to control the cracking.
- More critical and/or heavily loaded slabs-on-grade (such as driveways and patios) should be reinforced for control of cracking. Such reinforcement should be designed by the Structural Engineer. In our experience, welded wire mesh may not be sufficient to control slab cracking. For less critical and/or lightly loaded slabs-on-grade (such as exterior walkways, including sidewalks), omission of the reinforcement may be considered with additional risk of cracking.
- Slabs should have a minimum thickness of 4 inches and should slope away from the buildings at a slope of at least 2 percent to prevent water from flowing toward the building.
- It is critical that uniformity in soil moisture conditioning be achieved in subgrade soils in accordance with recommendations provided above for engineered fill, and that subgrade soils are not allowed to dry out prior to slab construction.

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We recommend secondary slabs-on-grade be underlain by a 4-inch-thick layer of clean crushed rock or gravel. Turned down free edges extending at least 2 inches beneath the crushed rock or gravel into compacted soil may be considered adjacent to landscape areas to reduce water infiltration into subgrade soils. Waterproof barriers may also be considered.

5.14 PRELIMINARY PAVEMENT 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.

5.14.1 Flexible Pavement

Based on our field exploration, we estimate that site soil will have a resistance (R-value) value of 5. The following preliminary pavement sections have been determined based on an assumed R-value of 5 according to the method contained in the Highway Design Manual by CALTRANS.

TABLE 5.14.1-1: Preliminary Flexible Pavement Design

TRAFFIC INDEX (TI)	R-VALUE OF 5 (UNTREATED SUBGRADE)	
	AC (INCHES)	AB (INCHES)
5.0	3	10
6.0	3½	13
7.0	4	16

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

5.14.2 Rigid Pavements

We developed 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”. We used section 613.3 of the 2012 edition of the CalTrans Highway Design Manual to convert Traffic Indexes (TIs) to Average Daily Truck Traffic volumes (ADTTs) for use with the ACI method. The rigid pavement sections are presented as jointed plain concrete pavement (JPCP) over Class 2 Aggregate Base (AB). The sections required for the requested TIs over a soil subgrade with an R-value of 5 and a minimum 28-day concrete compressive strength of 3,500 psi are presented below.

TABLE 5.14.2-1: Preliminary Rigid Pavement Design

TI	ADTT	JPCP	CLASS 2 AB
5.0	5	6 inches	6 inches
6.0	25	6 ½ inches	6 inches
7.0	100	7 ½ inches	6 inches

Notes: PCC is Portland cement concrete
AB is aggregate base Class 2 Material with minimum R = 78

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5.14.3 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 10 inches below finished subgrade elevation and moisture conditioned and compacted per the fill placement specifications listed in section 5.4. Pavement subgrades should also be prepared in accordance with City of San Jose requirements.

- Subgrade soils 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.

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

- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted to at least 95 percent of 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.

- All concrete curbs separating pavement and irrigated landscaped areas 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.

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5.15 DRAINAGE

Perimeter grades should be positively sloped at all times to provide for rapid removal of surface water runoff away from the foundation systems and to prevent ponding of water under foundations or seepage toward the foundation systems at any time during or after construction. Ponded water may cause undesirable soil swell and loss of strength. As a minimum requirement, finished grades should have slopes of at least 5 percent within 10 feet from the exterior walls and at right angles to allow surface water to drain positively away from the structure. For paved areas, the slope gradient can be reduced to 2 percent.

All surface water should be collected and discharged into outlets approved by the Civil Engineer. Landscape mounds must not interfere with this requirement. All roof stormwater should be collected and directed to downspouts. Stormwater from roof downspouts should not be allowed to discharge directly onto the ground surface in close proximity to the foundation system, such as via spashblocks. Rather, stormwater from roof downspouts should be directed to a solid pipe that discharges into the street or to an outlet approved by the Civil Engineer. If this is not acceptable, we recommend downspouts discharge at least 5 feet away from

foundations. Alternatively, engineered stormwater systems can be developed under the guidance of ENGEO.

5.16 STORMWATER INFILTRATION

Due to the density and higher clay content of near-surface site soils, the site soils are expected to have low permeability for stormwater infiltration. Therefore, best management practices should assume that stormwater infiltration will be limited at the site unless an engineered system is designed.

If bioretention areas are implemented, we recommend that a subdrain or other storm drain system be incorporated to collect and convey water to an approved outlet, considering the low permeability of site soils. When practical, bioretention areas should be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural onsite or offsite improvements can either:

1. Be constructed with structural side walls (below-grade retaining walls) capable of withstanding the loads from the adjacent improvements, or

2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent site improvements include buildings greater than three stories, streets steeper than 3 percent, or design elements that will experience lateral loads (such as from impact or traffic patterns), additional design considerations may be required. If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

Given the nature of bioretention systems and possible proximity to improvements, we recommend ENGEO be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should minimize the exposure time such that the improvements are not detrimentally impacted.

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5.17 LANDSCAPING IRRIGATION

The geotechnical foundation design parameters contained in this report have considered the swelling potential of some of the site soils; however, it is important to recognize that swell in excess of that anticipated is possible under adverse drainage or irrigation conditions. Therefore, planted areas should be avoided immediately adjacent to the buildings. If planting adjacent to a structure is desired, the use of watertight planter boxes with controlled discharge or the use of plants that require very little moisture is recommended.

Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils within 3 feet from walls. Such ponding or saturation could result in undesirable soil swell, loss of compaction and consequent foundation and slab movements. Irrigation of landscaped areas should be strictly limited to that necessary to sustain vegetation. The Landscape Architect and prospective owners should be informed of the surface drainage and irrigation requirements included in this report.

5.18 UTILITIES

It is recommended that utility trench backfilling be done under the observation of a Geotechnical Engineer. Ideally, pipe zone backfill (i.e. material beneath and immediately surrounding the pipe) should consist of native material less than ¾ inch in maximum dimension compacted in accordance with recommendations provided above for engineered fill. Trench zone backfill (i.e. material placed between the pipe zone backfill and the ground surface) should also consist of native soil compacted in accordance with recommendations for engineered fill. Controlled density fill is also suitable for pipe zone and trench zone backfill.

If required by local agencies, where import material is used for pipe zone backfill, we recommend it consist of quarry fines, fine- to medium-grained sand, or a well-graded mixture of sand and gravel and that this material not be used within 2 feet of finish subgrades. This material should be compacted to at least 90 percent relative compaction at a moisture content of not less than optimum.

In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of soil into the relatively large void spaces present in this type of material and for movement of water or soil vapor along trenches backfilled with this type of material. If uniformly graded gravel is used, we recommend that it be encapsulated in 6-ounce filter fabric. Providing outlet locations into manholes or catch basins for water collected in granular trench backfill should also be considered.

All utility trenches entering the buildings and paved areas should be provided with an impervious seal where the trenches pass under or through the building perimeter or curb lines. The impervious plug should extend at least 3 feet to both sides of the crossing and should be placed below, around, and above the utility pipe such that it is entirely in contact with the trench walls and pipe. This is to prevent surface water percolation into the import sand or gravel pipe zone backfill under foundations and pavements where such water would remain trapped in a perched condition.

Care should be exercised where utility trenches are located beside foundation areas. Utility trenches constructed parallel to foundations should be located entirely above a plane extending

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down from the lower edge of the footing at an angle of 45 degrees. Utility companies and Landscape Architects should be made aware of this information.

Utility trenches in areas to be paved should be constructed in accordance with the City of San Jose requirements or approved alternatives. Compaction of backfill by jetting should not be allowed at this site. If there appears to be a conflict between the City or other Agency requirements and the recommendations contained in this report, this should be brought to the Owner's attention for resolution prior to submitting bids.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, contractors, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of preparation of ENGEO's documents of service. This document must not be subject to unauthorized reuse, that is, reuse 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, including, but not limited to claims arising from or resulting from the performance of such services by other persons or entities, and 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

FIGURE 1 - Vicinity Map
FIGURE 2 - Site Plan
FIGURE 3 - Regional Geologic Map
FIGURE 4 - Seismic Hazard Zone Map
FIGURE 5 - Regional Faulting and Seismicity

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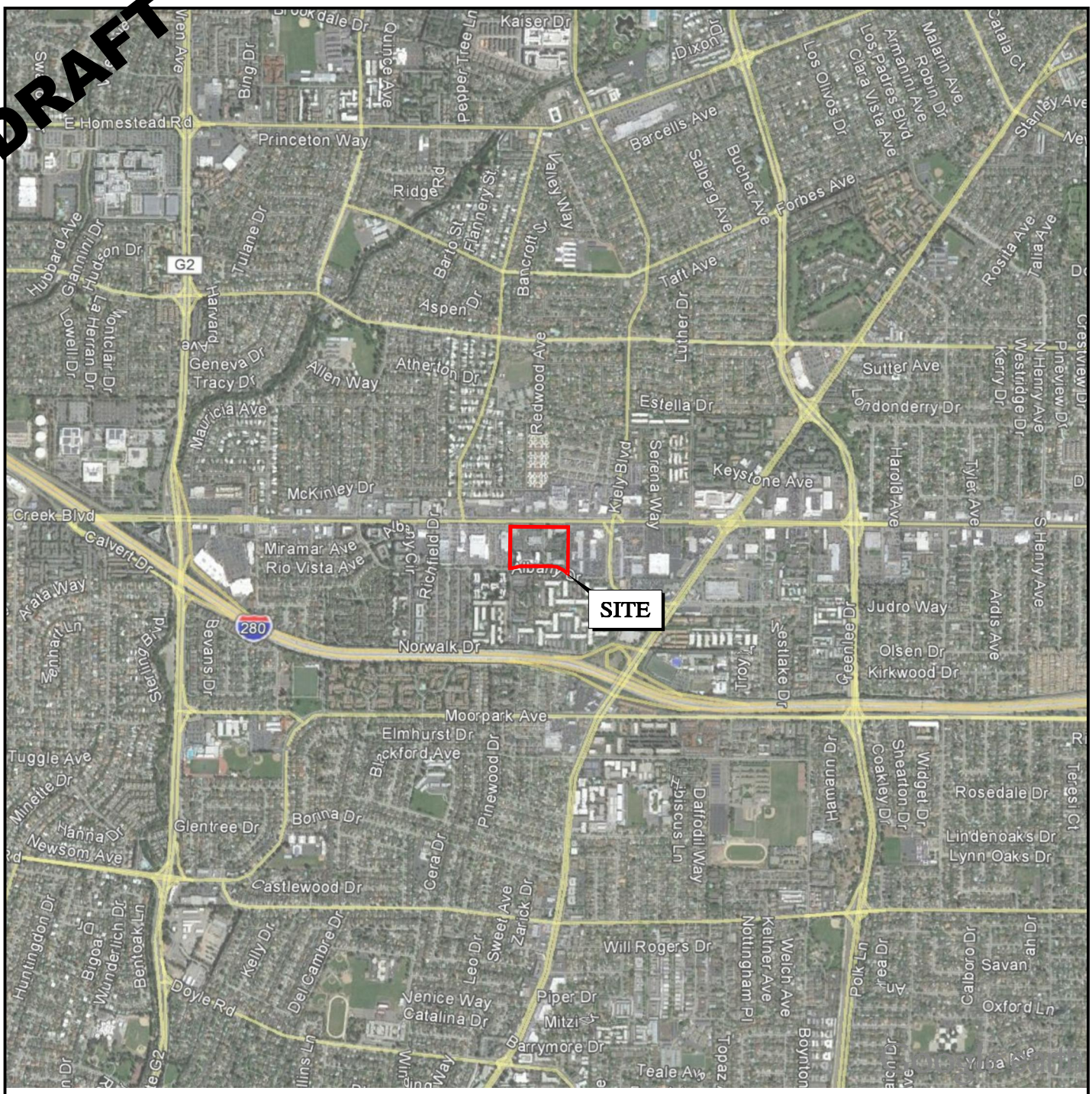

N

0 FEET 2000

0 METERS 1000

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BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

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VICINITY MAP STEVENS CREEK PROMENDADE SAN JOSE, CALIFORNIA	PROJECT NO.: 13317.000000	FIGURE NO.
	SCALE: AS SHOWN	1
	DRAWN BY: PC / CHECKED BY: TPB	

G:\Drafting\DRAFTING2_Dwg_13000 Plus\13317\000\GEX StevensCrk\13317000000-GEX-1-VicMap-StevensCrk-1016.dwg Plot Date: 11-21-16 pcrockett

ORIGINAL FIGURE PRINTED IN COLOR

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0 FEET 100
0 METERS 50

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-B4 BORING

1-CPT4 CONE PENETRATION TEST

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

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SITE PLAN
STEVENS CREEK PROMENADE
SAN JOSE, CALIFORNIA

PROJECT NO.: 13317.000.000		FIGURE NO. 2
SCALE: AS SHOWN		
DRAWN BY: PC	CHECKED BY: TPB	

ORIGINAL FIGURE PRINTED IN COLOR

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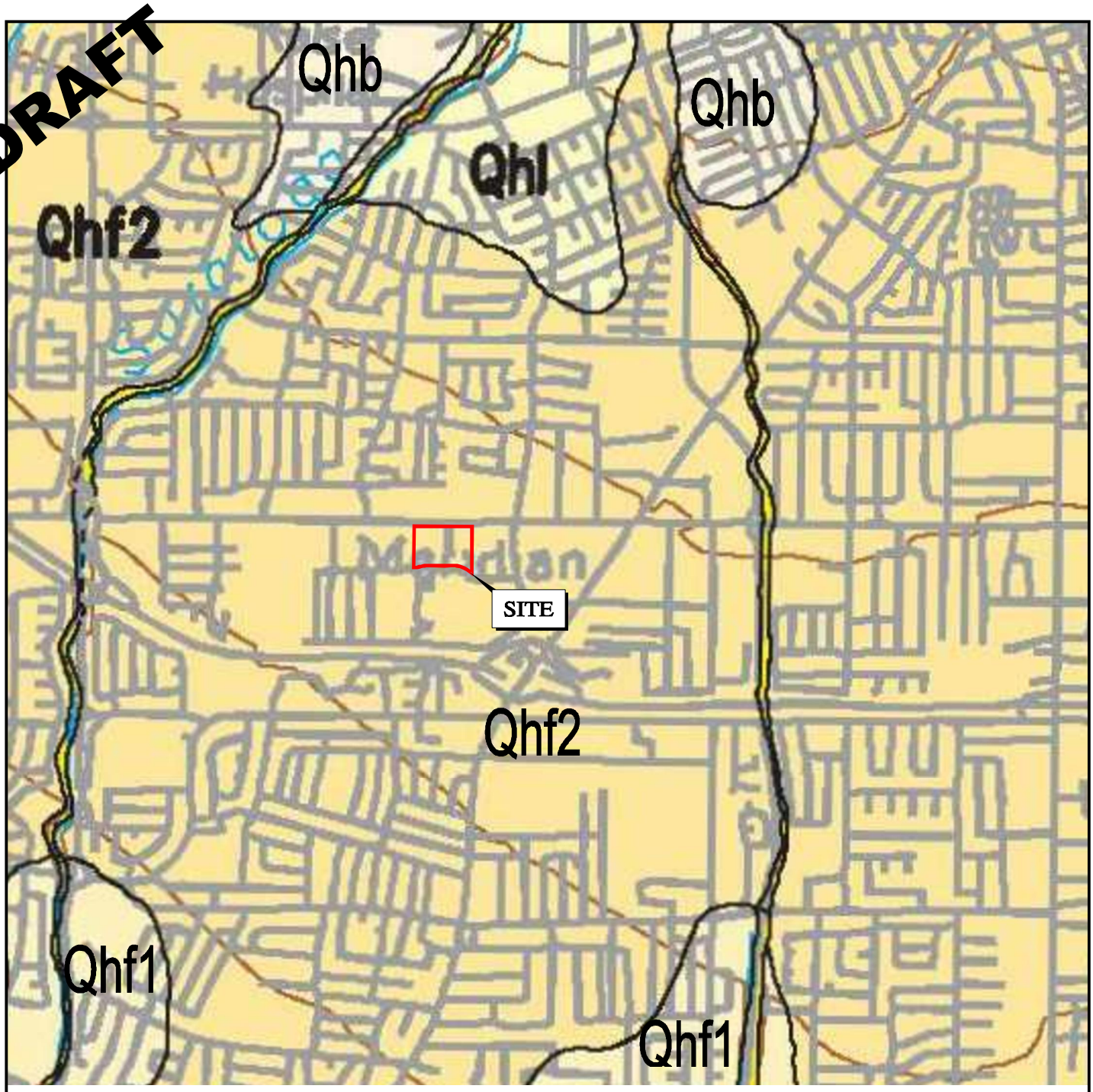

N

0 FEET 2000

0 METERS 1000

EXPLANATION

——— – – – –	GEOLOGIC CONTACT-DASHED WHERE APPROXIMATELY LOCATED
Qhb	BASIN DEPOSITS
Qhl	LEVEE DEPOSITS
Qhf1	ALLUVIAL FAN DEPOSITS (YOUNGER)
Qhf2	ALLUVIAL FAN DEPOSITS (OLDER)

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BASE MAP SOURCE: WENTWORTH, 1999

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REGIONAL GEOLOGIC MAP
STEVENS CREEK PROMENADE
SAN JOSE, CALIFORNIA

PROJECT NO.:	13317.000.000
SCALE:	AS SHOWN
DRAWN BY: PC	CHECKED BY: TPB

FIGURE NO. 3

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ORIGINAL FIGURE PRINTED IN COLOR

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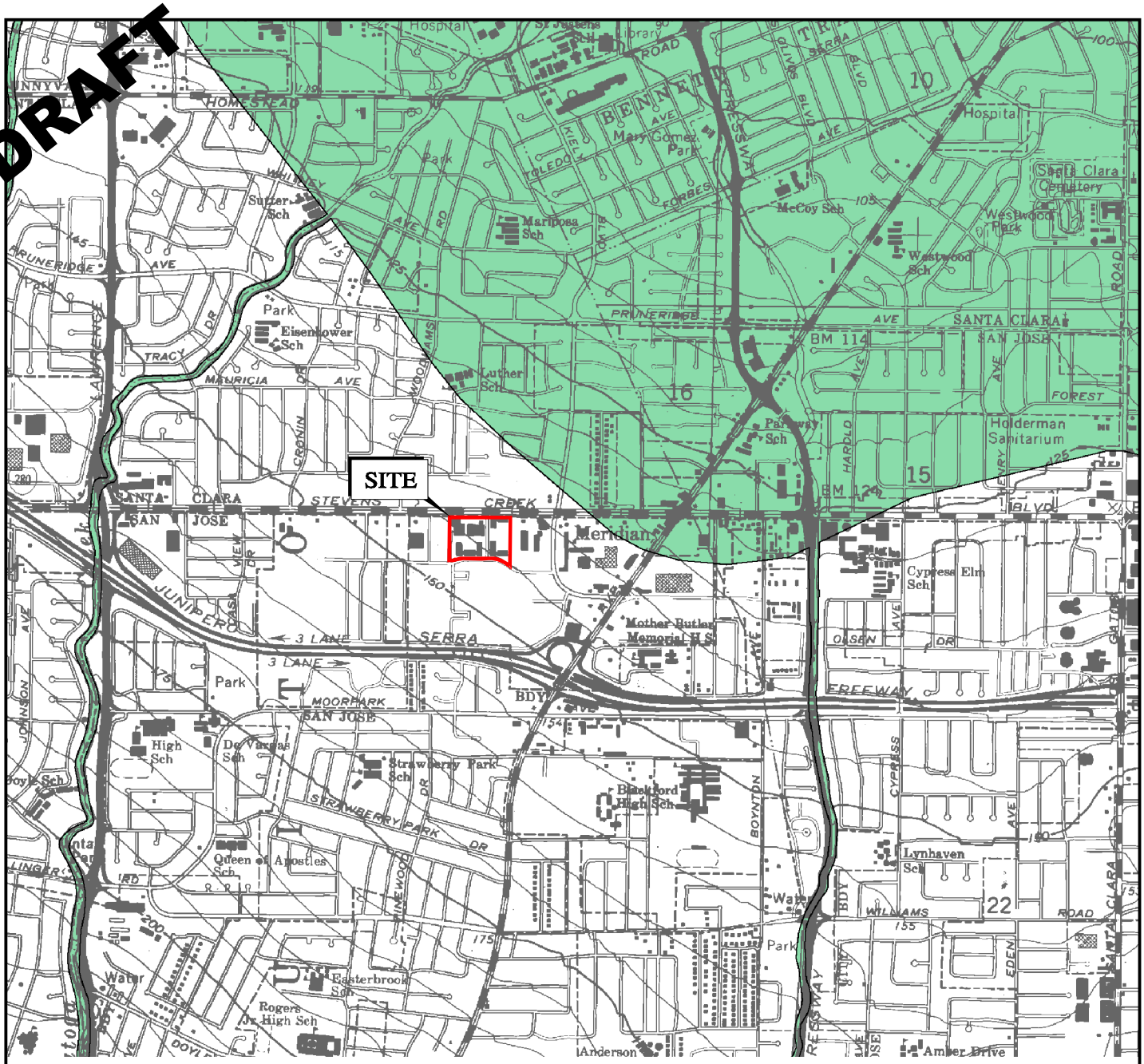

EXPLANATION

LIQUEFACTION

AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

EARTHQUAKE-INDUCED LANDSLIDES

AREAS WHERE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

0 FEET 2000
0 METERS 1000

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BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2006

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SEISMIC HAZARD ZONES MAP
STEVENS CREEK PROMENADE
SAN JOSE, CALIFORNIA

PROJECT NO.: 13317.000.000		FIGURE NO. 4
SCALE: AS SHOWN		
DRAWN BY: PC	CHECKED BY: TPB	

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ORIGINAL FIGURE PRINTED IN COLOR

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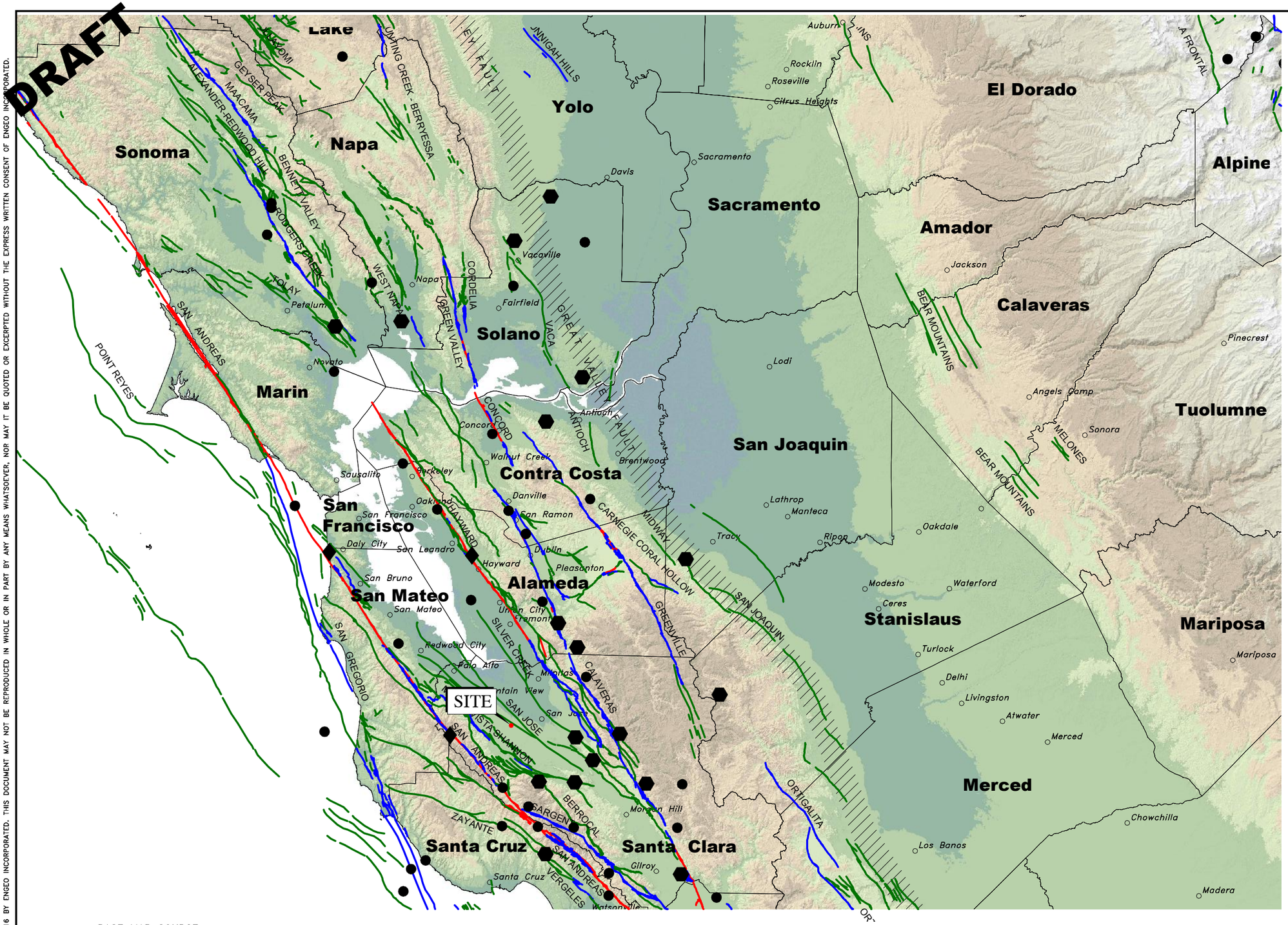

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

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REGIONAL FAULTING AND SEISMICITY
STEVENS CREEK PROMENADE
SAN JOSE, CALIFORNIA

PROJECT NO.: 13317.000.000		FIGURE NO
SCALE: AS SHOWN		5
DRAWN BY: PC	CHECKED BY: TPB	

ORIGINAL FIGURE PRINTED IN COLOR

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APPENDIX A

BORING LOG KEY
EXPLORATION 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.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE				CLEAR SQUARE SIEVE OPENINGS			
200	40	10	4	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
- 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

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LOG OF BORING 1-B1

ENGEO INCORPORATED

Geotechnical Exploration
Stevens Creek Promenade
San Jose, CA
13317.000.000

DATE DRILLED: 9/23/2016
HOLE DEPTH: Approx. 71½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (Approx.): 147 ft.

LOGGED / REVIEWED BY: Y. Zepeda / RB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SSA, Switch to Mud
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
			6-inches AC												
	145														
5		■	SANDY LEAN CLAY (CL), dark olive brown, hard, slightly moist, fine- to coarse-grained sand, 5 to 10% fine to coarse gravel			25				45				4.5+*	PP
	140														
10		■	LEAN CLAY (CL), olive brown, hard, slightly moist, low plasticity, <5% rounded fine gravel			21								4.5+*	PP
	135														
15		■	SANDY LEAN CLAY (CL), pale olive to olive brown, hard, slightly moist, fine- to coarse-grained sand			24				63				4.5+*	PP
	130														
20		■	SILTY SAND (SM), pale olive mottled with orange, medium dense, slightly moist, fine-grained sand			19				48	12.9				

LOG OF BORING 1-B1

ENGEO INCORPORATED

Geotechnical Exploration
Stevens Creek Promenade
San Jose, CA
13317.000.000

DATE DRILLED: 9/23/2016
HOLE DEPTH: Approx. 71½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (Approx.): 147 ft.

LOGGED / REVIEWED BY: Y. Zepeda / RB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SSA, Switch to Mud
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
	125		SANDY LEAN CLAY (CL), pale olive mottled with orange, very stiff, moist, fine-grained sand			25	32	20	12	59	17.7	110.7			
25						12					25.8				
	120														
30			SILTY SAND (SM), olive, medium dense to dense, moist			30					11.9				
	115														
35			CLAYEY SAND (SC), pale olive, very dense, moist, fine- to coarse-grained sand			68				14					
	110														
40															

SHEAR AND UNCONF STRENGTH W/ ELEV 13317 GINT LOGS.GPJ ENGEO INC.GDT 10/26/16

LOG OF BORING 1-B1

ENGEO INCORPORATED

Geotechnical Exploration
Stevens Creek Promenade
San Jose, CA
13317.000.000

DATE DRILLED: 9/23/2016
HOLE DEPTH: Approx. 71½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (Approx.): 147 ft.

LOGGED / REVIEWED BY: Y. Zepeda / RB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SSA, Switch to Mud
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 SILT (ML), dark yellowish brown with gray, hard, moist, fine- to coarse-grained sand			42				51	16.1	112.4			
	105														
45			FAT CLAY (CH), gray, stiff, moist			20					20.7	108.3		1.84	UC
	100														
50			SANDY LEAN CLAY (CL), gray with orange, soft, moist			43				56	20.1	108.1		0.30	UC
	95														
55						85									
			WELL GRADED SILTY SAND (SW-SM), dark yellowish brown to olive brown, very dense, moist												
	90														
60															

LOG OF BORING 1-B1

Geotechnical Exploration
Stevens Creek Promenade
San Jose, CA
13317.000.000

DATE DRILLED: 9/23/2016
HOLE DEPTH: Approx. 71½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (Approx.): 147 ft.

LOGGED / REVIEWED BY: Y. Zepeda / RB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SSA, Switch to Mud
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
			LEAN CLAY WITH SAND (CL), olive, stiff, moist Triaxial UU = 2518 psf			26					19.7	110.6	2518		UU
	85														
65			SILTY SAND (SM), dark reddish yellow with gray, very dense, moist			64				20	11.3				
	80														
70			CLAYEY GRAVEL WITH SAND (GC), dark yellowish brown, very dense, very moist, fine to coarse gravel, fine- to coarse-grained sand			83				14					
			Bottom of boring at approximately 71½ feet below ground surface. Groundwater not encountered due to drilling method.												

LOG OF BORING 1-B2

ENGEO INCORPORATED

Geotechnical Exploration
Stevens Creek Promenade
San Jose, CA
13317.000.000

DATE DRILLED: 9/23/2016
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (Approx.): 146 ft.

LOGGED / REVIEWED BY: Y. Zepeda / RB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
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
			3-inches AC over 3-inches AB												
	145														
5			SANDY LEAN CLAY (CL), dark brown, hard, slightly moist, 5 to 10% fine to coarse gravel			42				54				4.5+*	PP
	140														
10			Becomes brown.			30				50				4.5+*	PP
	135														
15						35								4.5+*	PP
	130														
			SILTY SAND WITH GRAVEL (SM), light brown to olive brown, dense, slightly moist, fine- to coarse-grained sand, fine gravel			32				36					
20															

LOG OF BORING 1-B2

Geotechnical Exploration
Stevens Creek Promenade
San Jose, CA
13317.000.000

DATE DRILLED: 9/23/2016
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (Approx.): 146 ft.

LOGGED / REVIEWED BY: Y. Zepeda / RB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
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
	125		SANDY LEAN CLAY (CL), pale olive mottled with orange, very stiff, slightly moist, fine- to coarse-grained sand			24					13.4	111.3		1.5	UC
			Bottom of boring at approximately 21½ feet below ground surface. Groundwater not encountered during drilling.												

LOG OF BORING 1-B3

ENGEO INCORPORATED

Geotechnical Exploration
Stevens Creek Promenade
San Jose, CA
13317.000.000

DATE DRILLED: 9/23/2016
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (Approx.): 143 ft.

LOGGED / REVIEWED BY: Y. Zepeda / RB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
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
			3-inches AC over 3-inches AB												
	140														
5			SANDY LEAN CLAY (CL), dark brown to olive brown, hard, slightly moist			24								4.5+*	PP
	135														
10						32								4.5+*	PP
	130					35	33	16	17		12.9	109.5		4.5+*	PP
15			Becomes stiff.			25				37				1.14	UC
	125														
20															

SHEAR AND UNCONF STRENGTH W/ ELEV 13317 GINT LOGS.GPJ ENGEO INC.GDT 10/26/16

LOG OF BORING 1-B3

Geotechnical Exploration
Stevens Creek Promenade
San Jose, CA
13317.000.000

DATE DRILLED: 9/23/2016
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (Approx.): 143 ft.

LOGGED / REVIEWED BY: Y. Zepeda / RB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
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
			Becomes very stiff.			26								2.75	UC
			Bottom of boring at approximately 21½ feet below ground surface. Groundwater not encountered during drilling.												

SHEAR AND UNCONF STRENGTH W/ ELEV 13317 GINT LOGS.GPJ ENGEO INC.GDT 10/26/16

LOG OF BORING 1-B4

ENGEO INCORPORATED

Geotechnical Exploration
Stevens Creek Promenade
San Jose, CA
13317.000.000

DATE DRILLED: 9/23/2016
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 4.5 in.
SURF ELEV (Approx.): 147 ft.

LOGGED / REVIEWED BY: Y. Zepeda / RB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
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
			3½-inches AC over 6-inches AB												
	145		SANDY LEAN CLAY (CL), very dark brown, hard, slightly moist, fine- to coarse-grained sand			32	28	16	12	58	9.1	103		4.5+*	PP
5			Becomes dark olive brown.			22								4.5+*	PP
	140														
10						38								4.84	UC
	135														
15						40								4.5+*	PP
			Bottom of boring at approximately 16½ feet below ground surface. Groundwater not encountered during drilling.												

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APPENDIX B

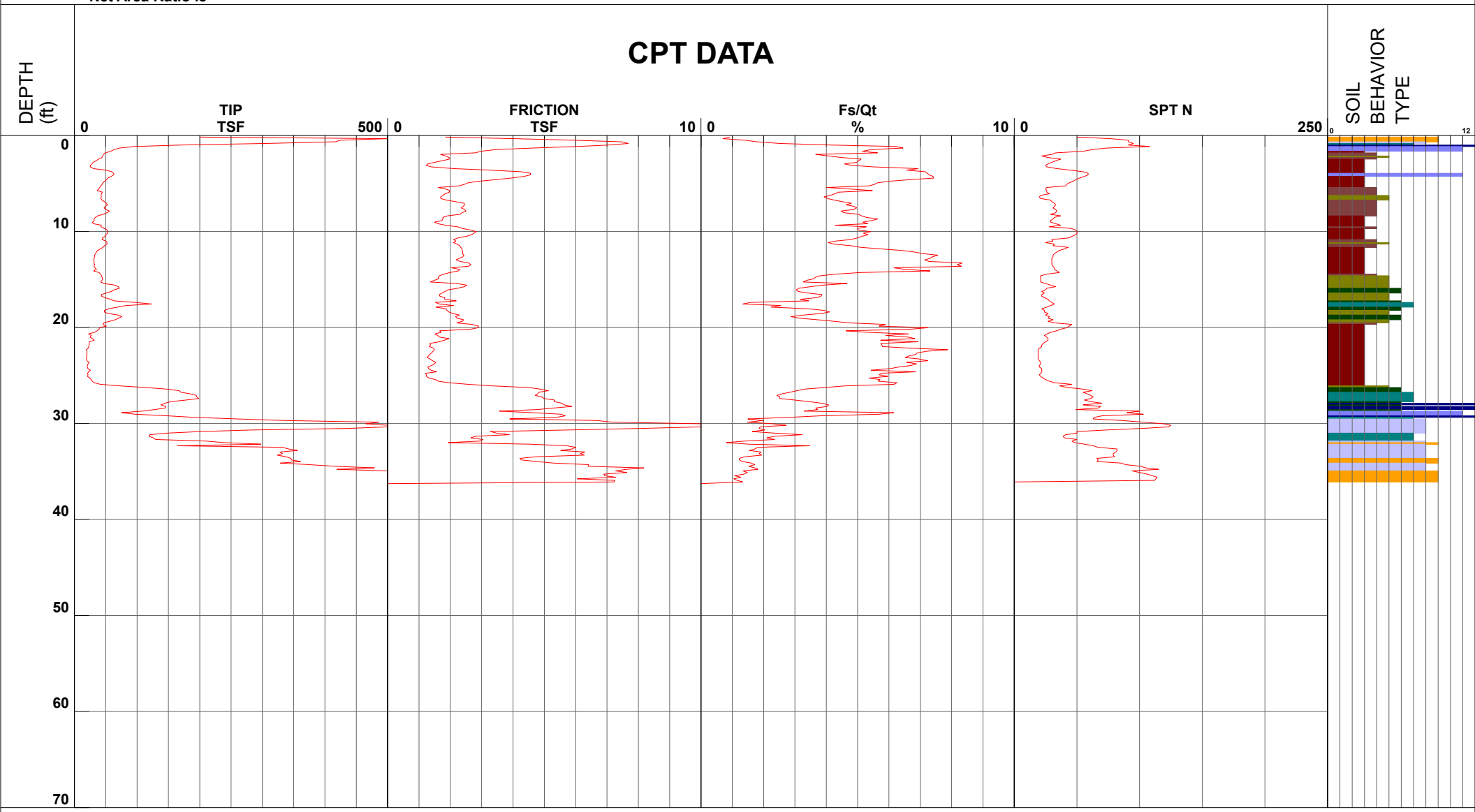
CONE PENETRATION TEST LOGS (MIDDLE EARTH GEO TESTING, 2016)

Engeo Inc

Project	Stevens Creek Promenade	Operator	JH-KK	Filename	SDF(103).cpt
Job Number	P2016.001.145	Cone Number	DDG1333	GPS	
Hole Number	1-CPT1	Date and Time	9/21/2016 2:55:01 PM	Maximum Depth	36.42 ft
EST GW Depth During Test		>36.42 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 10cm squared

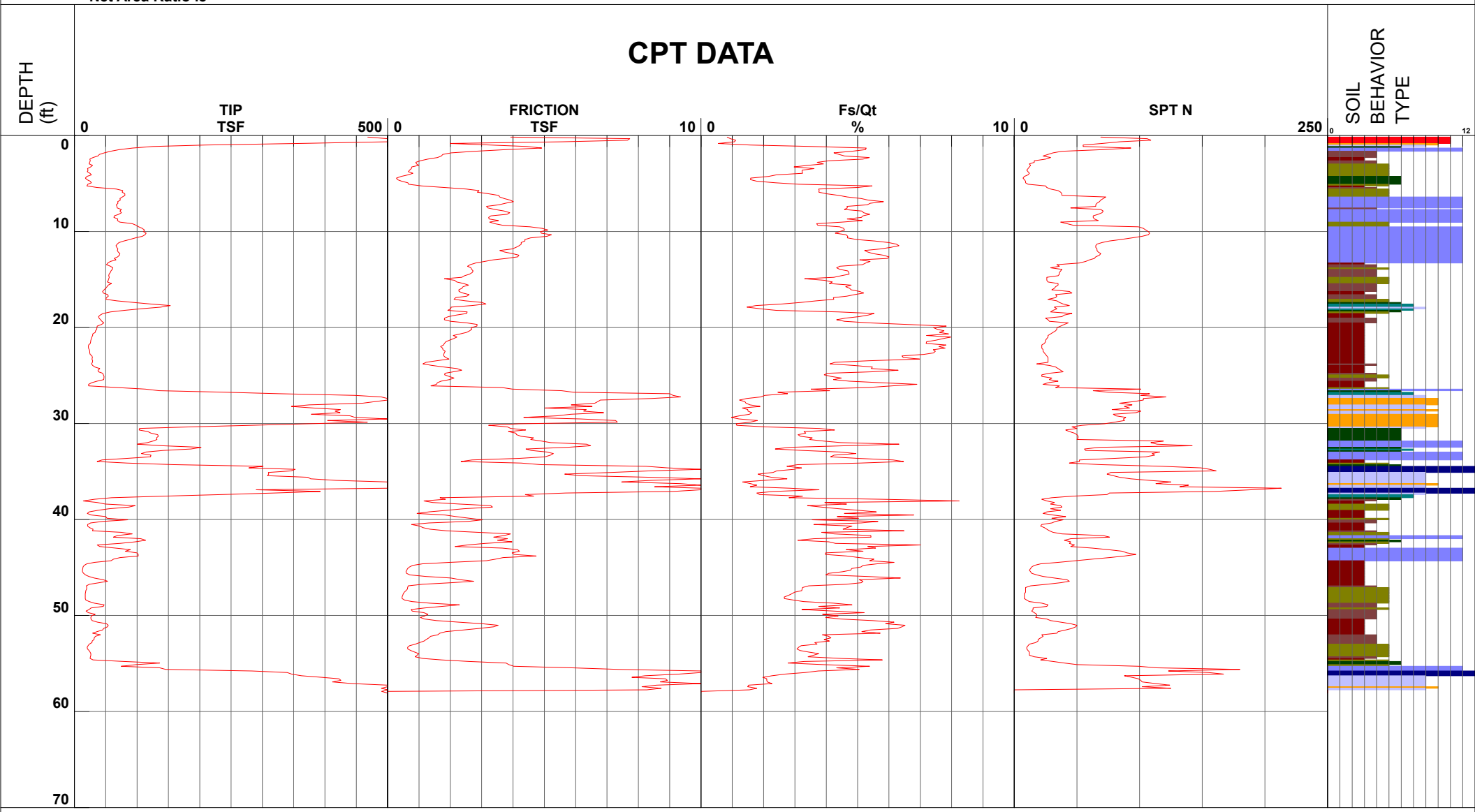
S*Soil behavior type and SPT based on data from UBC-1983

Engeo Inc

Project	Stevens Creek Promenade	Operator	JH-KK	Filename	SDF(105).cpt
Job Number	P2016.001.145	Cone Number	DDG1333	GPS	
Hole Number	1-CPT2	Date and Time	9/21/2016 4:35:42 PM	Maximum Depth	58.07 ft
EST GW Depth During Test		>58.07 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 10cm squared

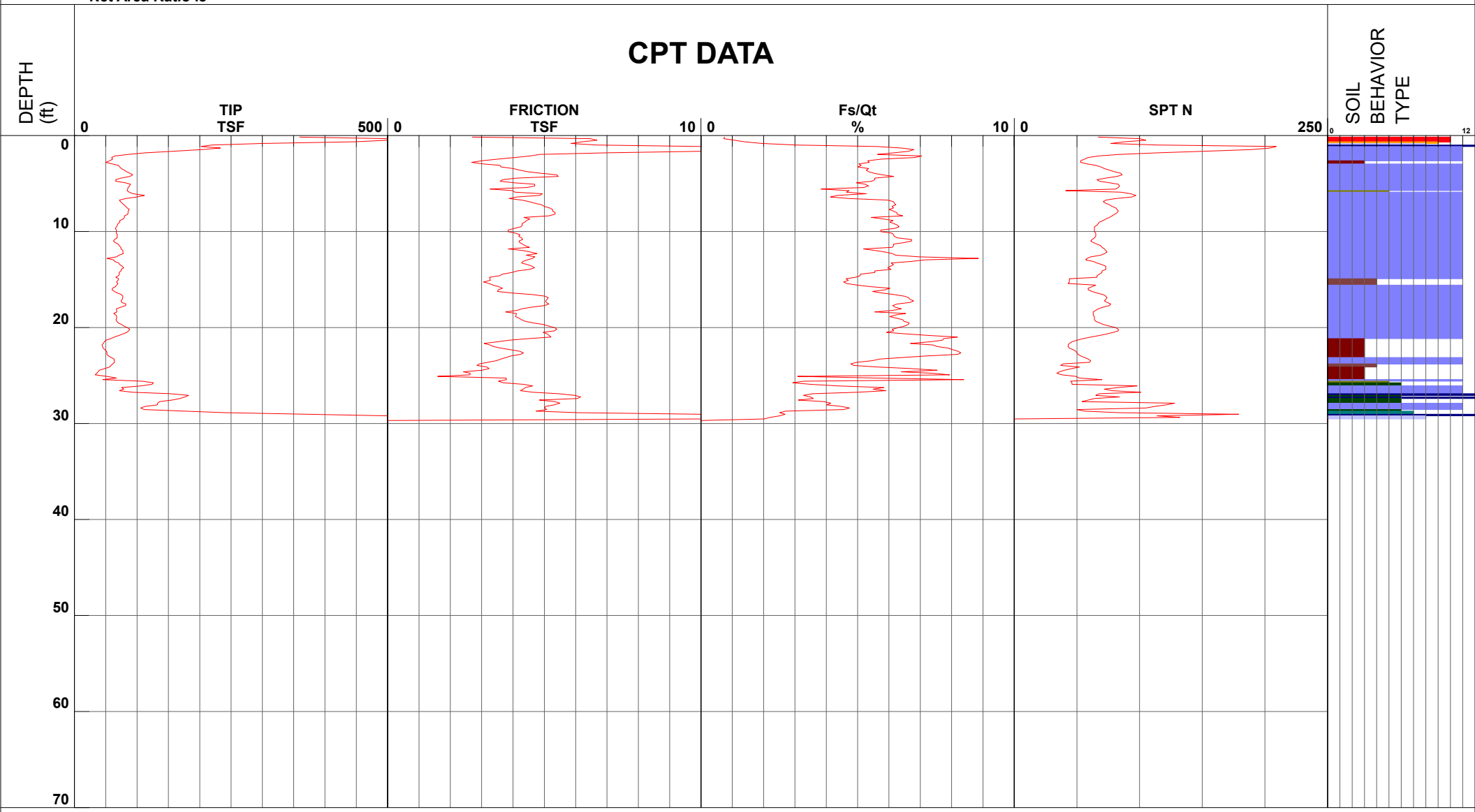
S*Soil behavior type and SPT based on data from UBC-1983

Engeo Inc

Project	Stevens Creek Promenade	Operator	JH-KK	Filename	SDF(104).cpt
Job Number	P2016.001.145	Cone Number	DDG1333	GPS	
Hole Number	1-CPT3	Date and Time	9/21/2016 3:51:09 PM	Maximum Depth	29.86 ft
EST GW Depth During Test		>29.86 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 10cm squared

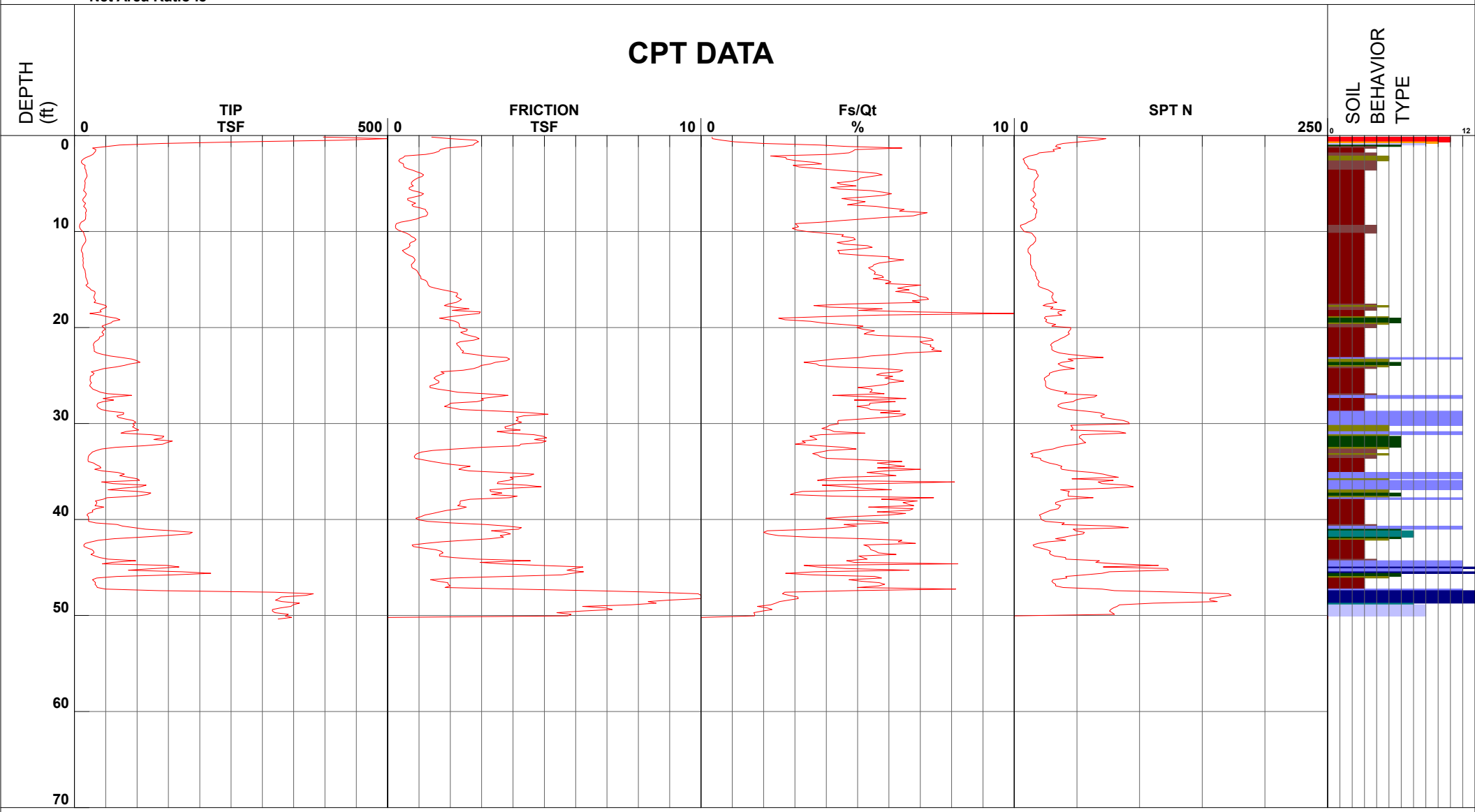
S*Soil behavior type and SPT based on data from UBC-1983

Engeo Inc

Project	Stevens Creek Promenade	Operator	JH-KK	Filename	SDF(101).cpt
Job Number	P2016.001.145	Cone Number	DDG1333	GPS	
Hole Number	1-CPT4	Date and Time	9/21/2016 12:28:47 PM	Maximum Depth	50.36 ft
EST GW Depth During Test		>50.36 ft			

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

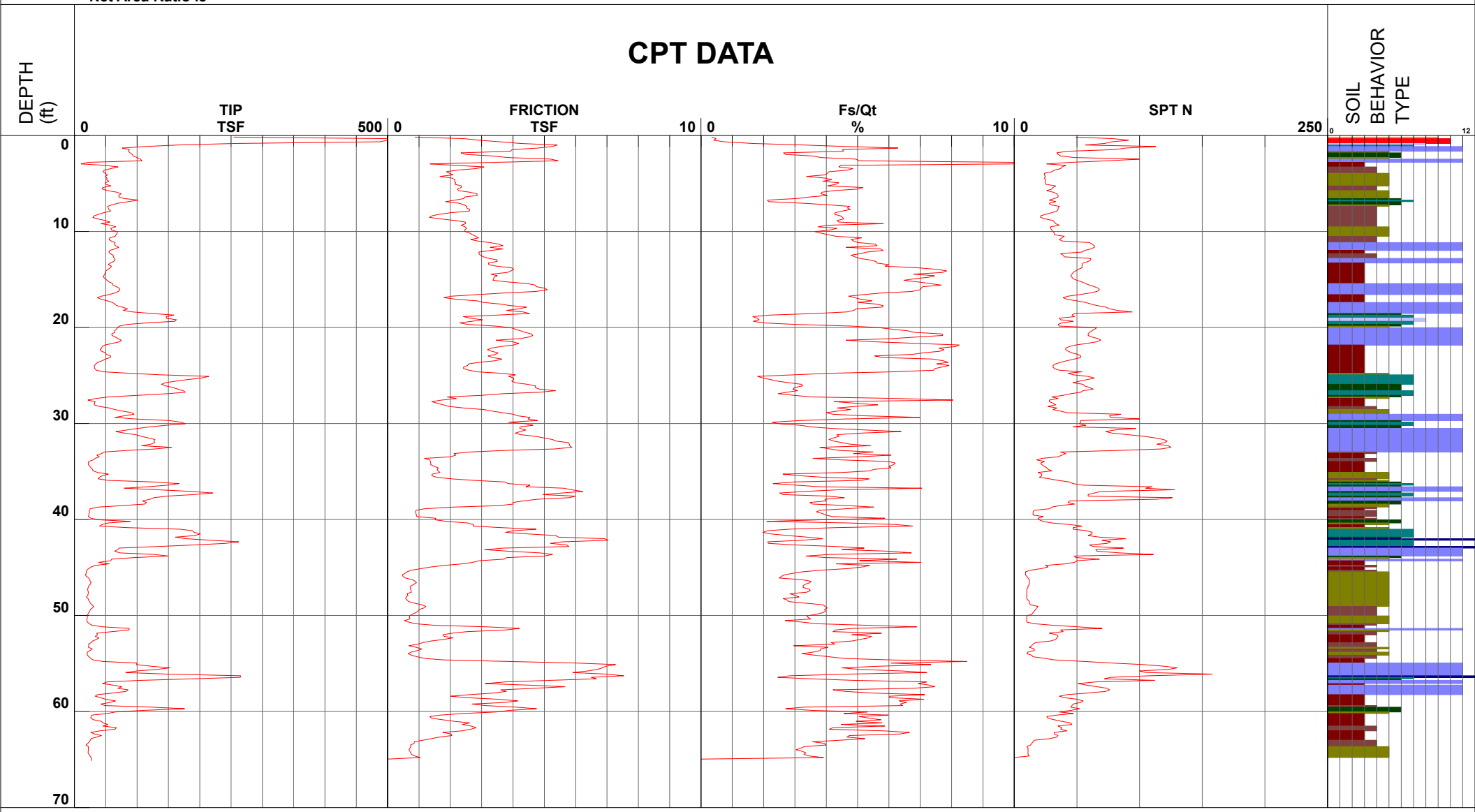
Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

Engeo Inc

Project	Stevens Creek Promenade	Operator	JH-KK	Filename	SDF(102).cpt
Job Number	P2016.001.145	Cone Number	DDG1333	GPS	
Hole Number	1-CPT5	Date and Time	9/21/2016 1:24:38 PM	Maximum Depth	65.12 ft
EST GW Depth During Test		>65.12 ft			

Net Area Ratio .8



1 - sensitive fine grained
2 - organic material
3 - clay
4 - silty clay to clay
5 - clayey silt to silty clay
6 - sandy silt to clayey silt
7 - silty sand to sandy silt
8 - sand to silty sand
9 - sand
10 - gravelly sand to sand
11 - very stiff fine grained (*)
12 - sand to clayey sand (*)

Cone Size 10cm squared

S*Soil behavior type and SPT based on data from UBC-1983

DRAFT

APPENDIX C

LABORATORY TEST RESULTS

MOISTURE-DENSITY DETERMINATION
ASTM D7263

BORING ID:	1-B1	1-B1	1-B1	1-B1	1-B1	1-B1	1-B3	1-B4
DEPTH (ft.):	19.5	23.5	25.5	30.5	41.0	66.0	13.5	6.0
%MOISTURE CONTENT:	12.9	17.7	25.8	11.9	16.1	11.3	12.9	9.1
DENSITY (lbs/ft^3):		110.7			112.4		109.5	103.0

Testing remarks: For moisture content only, ASTM D2216

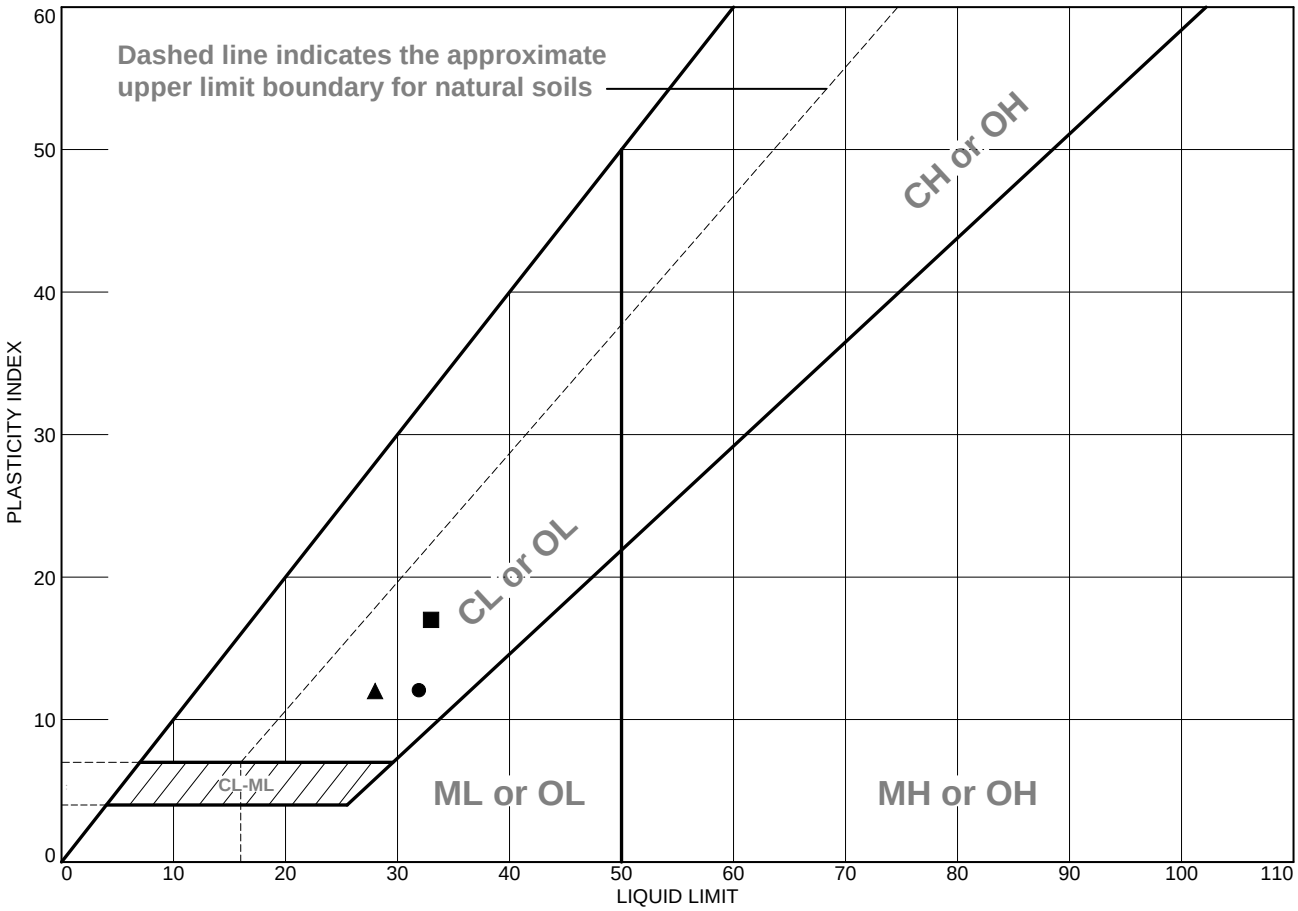
PROJECT NAME: 4300 Stevens Creek, San Jose
PROJECT NUMBER: 13317.000.000
CLIENT: Fort Bay Stevens Creek, LLC
PHASE NUMBER: 001

DATE: 10/06/16

ENGEO
Expect Excellence

Tested by: M. Quasem Reviewed by: G. Criste

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	32	20	12	93.6	59.0	CL
■	See exploration logs	33	16	17			
▲	See exploration logs	28	16	12			

Project No. 13317.000.000 **Client:** Fort Bay Stevens Creek, LLC

Project: 4300 Stevens Creek, San Jose

●**Depth:** 23.5 feet **Sample Number:** 1-B1 @ 23.5

■**Depth:** 13.5 feet **Sample Number:** 1-B3 @ 13.5

▲**Depth:** 6.0 feet **Sample Number:** 1-B4 @ 6

Remarks:

●PI: ASTM D4318, Wet method
GS: ASTM D6913
USCS: ASTM D2487

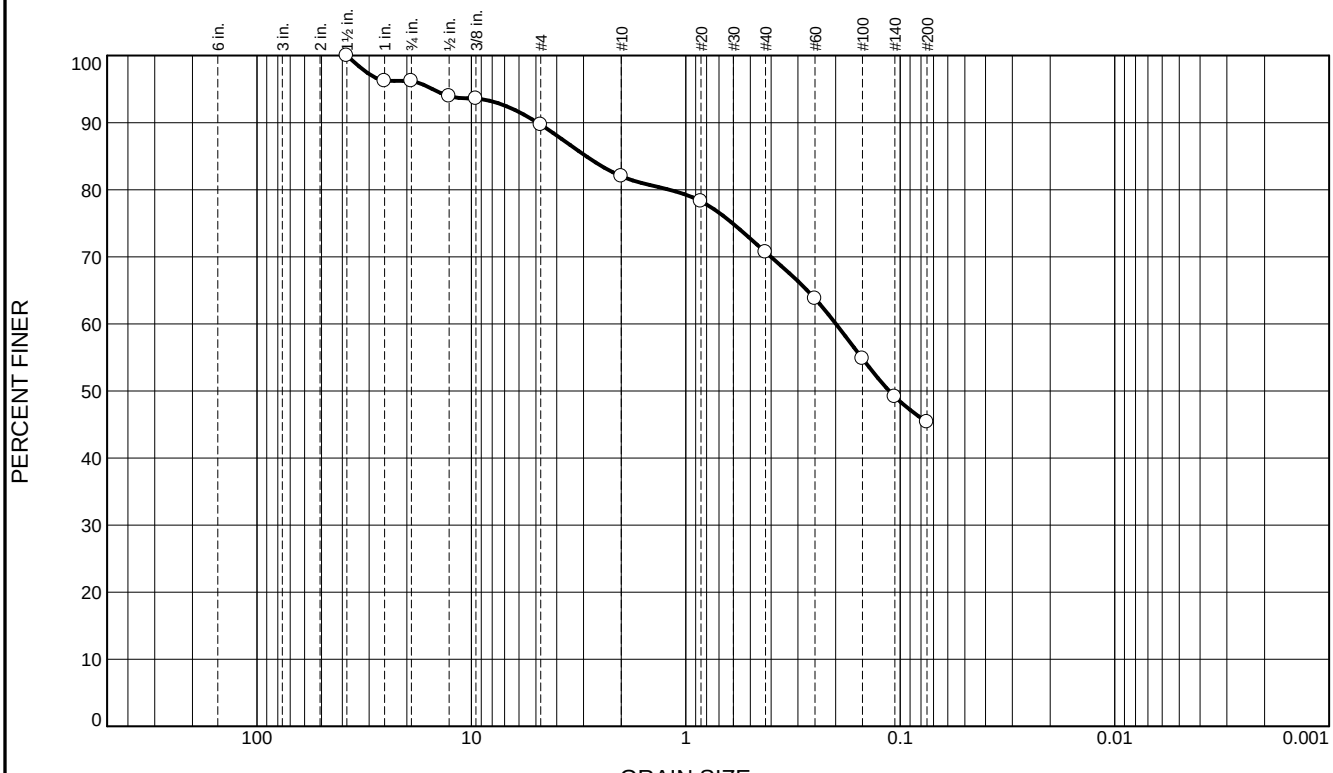
■ASTM D4318, Wet method

▲ASTM D4318, Wet method

ENGEO INCORPORATED

Tested By: M. Quasem **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.8	6.5	7.7	11.3	25.3	45.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2	100.0		
1	96.2		
3/4	96.2		
1/2	93.9		
3/8	93.6		
#4	89.7		
#10	82.0		
#20	78.3		
#40	70.7		
#60	63.8		
#100	54.8		
#140	49.1		
#200	45.4		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 4.9118 D_{85}= 2.9169 D_{60}= 0.1995
D_{50}= 0.1126 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Grain Size: ASTM D422

Sample Number: 1-B1 @ 6 **Depth:** 6.0 feet

Date: 10.10.16

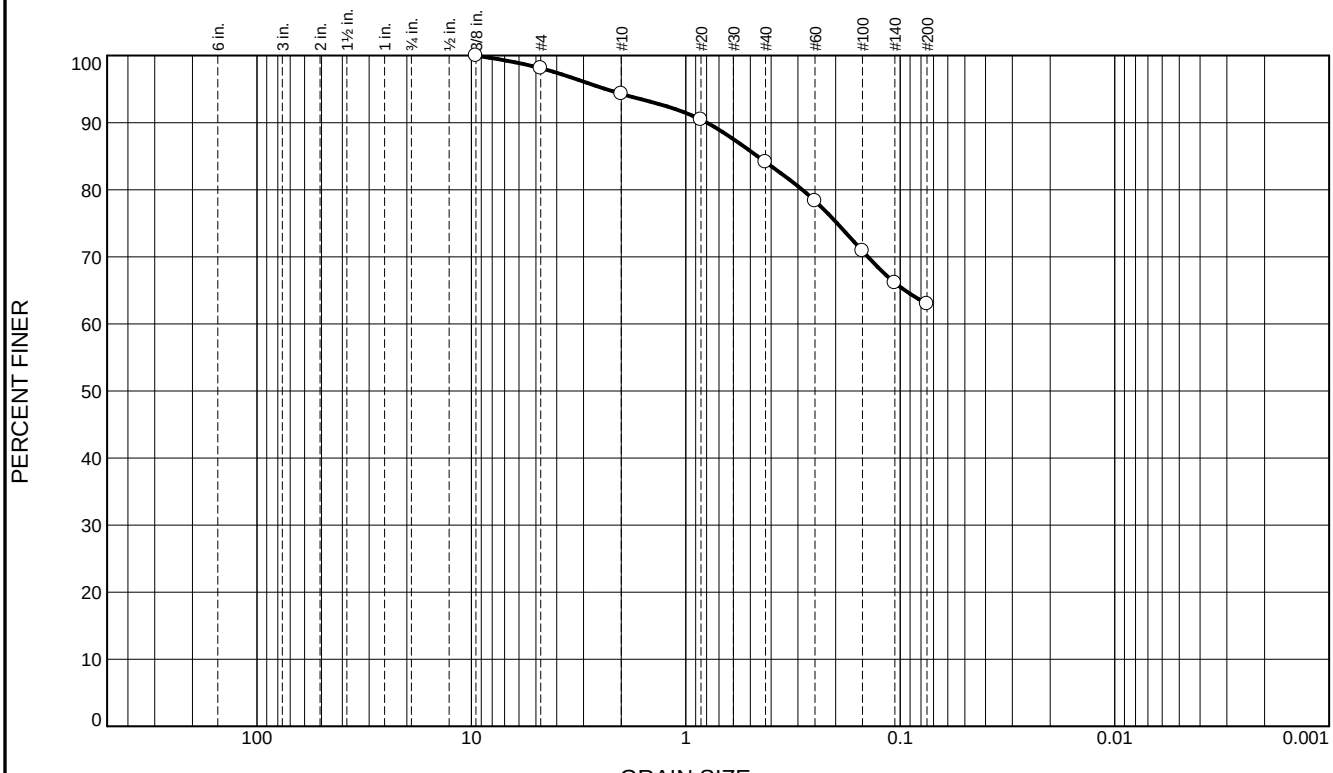
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.9	3.8	10.2	21.1	63.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100.0		
#4	98.1		
#10	94.3		
#20	90.5		
#40	84.1		
#60	78.3		
#100	70.9		
#140	66.1		
#200	63.0		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.7976 D_{85}= 0.4635 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Grain Size: ASTM D422

Sample Number: 1-B1 @ 16 **Depth:** 16.0 feet

Date: 10.10.16

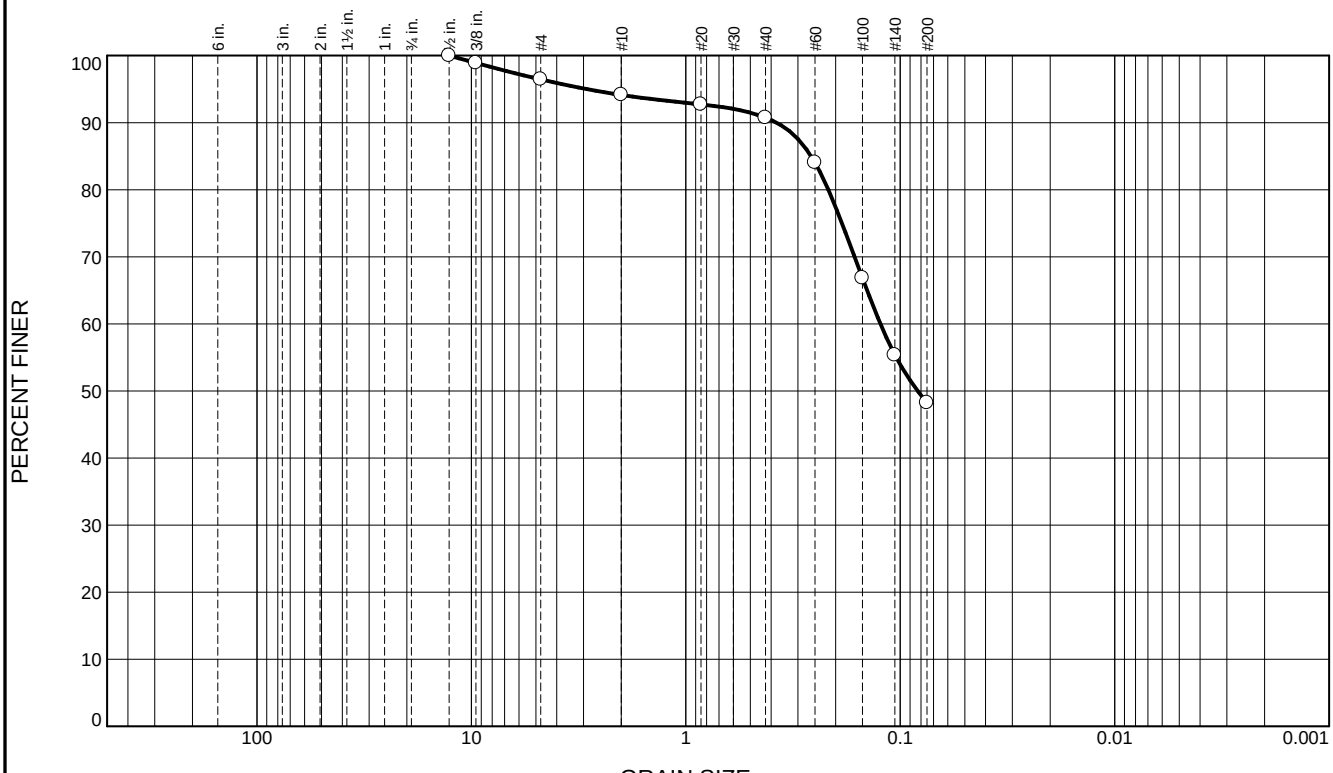
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	3.5	2.4	3.4	42.5	48.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2	100.0		
3/8	98.9		
#4	96.5		
#10	94.1		
#20	92.7		
#40	90.7		
#60	84.0		
#100	66.8		
#140	55.4		
#200	48.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3774 D_{85}= 0.2606 D_{60}= 0.1237
D_{50}= 0.0828 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Grain Size: ASTM D422

Sample Number: 1-B1 @ 19.5 **Depth:** 19.5 feet

Date: 10.10.16

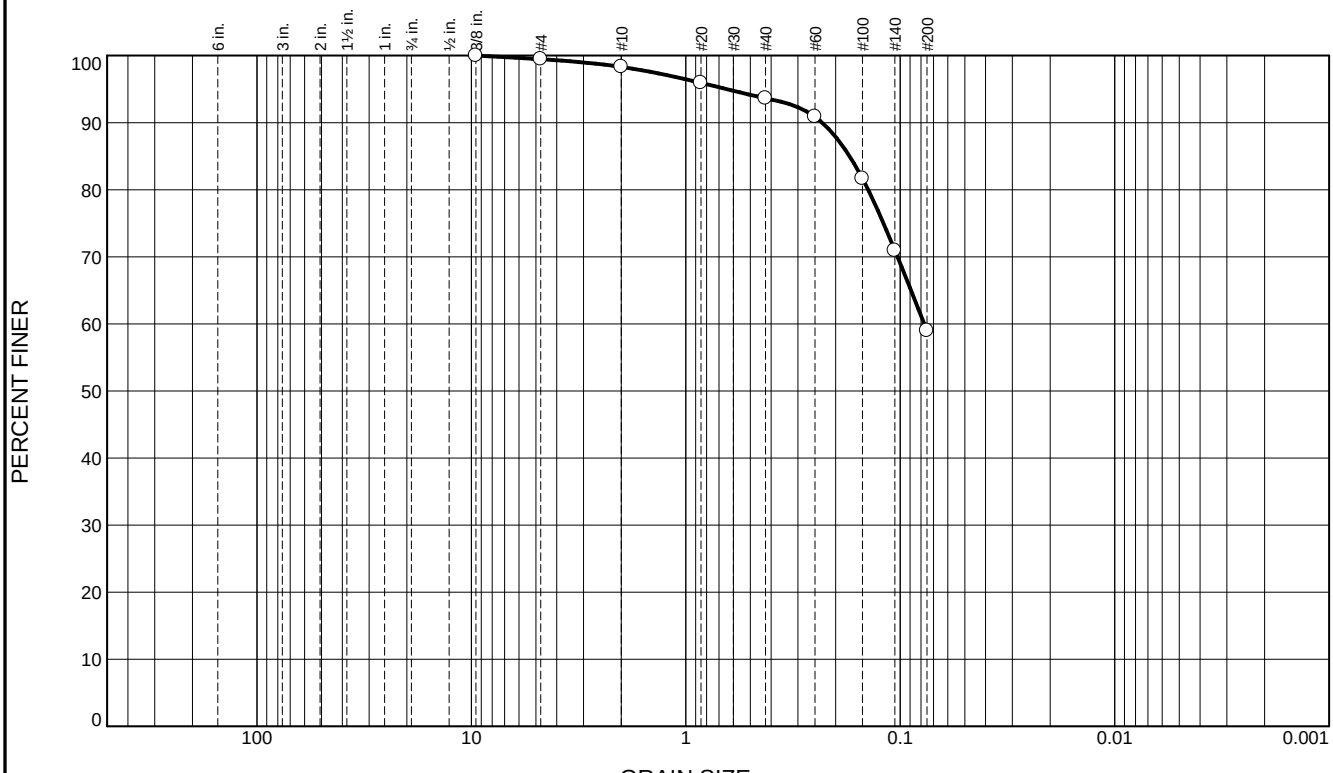
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.5	1.2	4.7	34.6	59.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100.0		
#4	99.5		
#10	98.3		
#20	95.9		
#40	93.6		
#60	90.9		
#100	81.7		
#140	70.9		
#200	59.0		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2313 D_{85}= 0.1722 D_{60}= 0.0772
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Grain Size: ASTM D422

Sample Number: 1-B1 @ 23.5 **Depth:** 23.5 feet

Date: 10.10.16

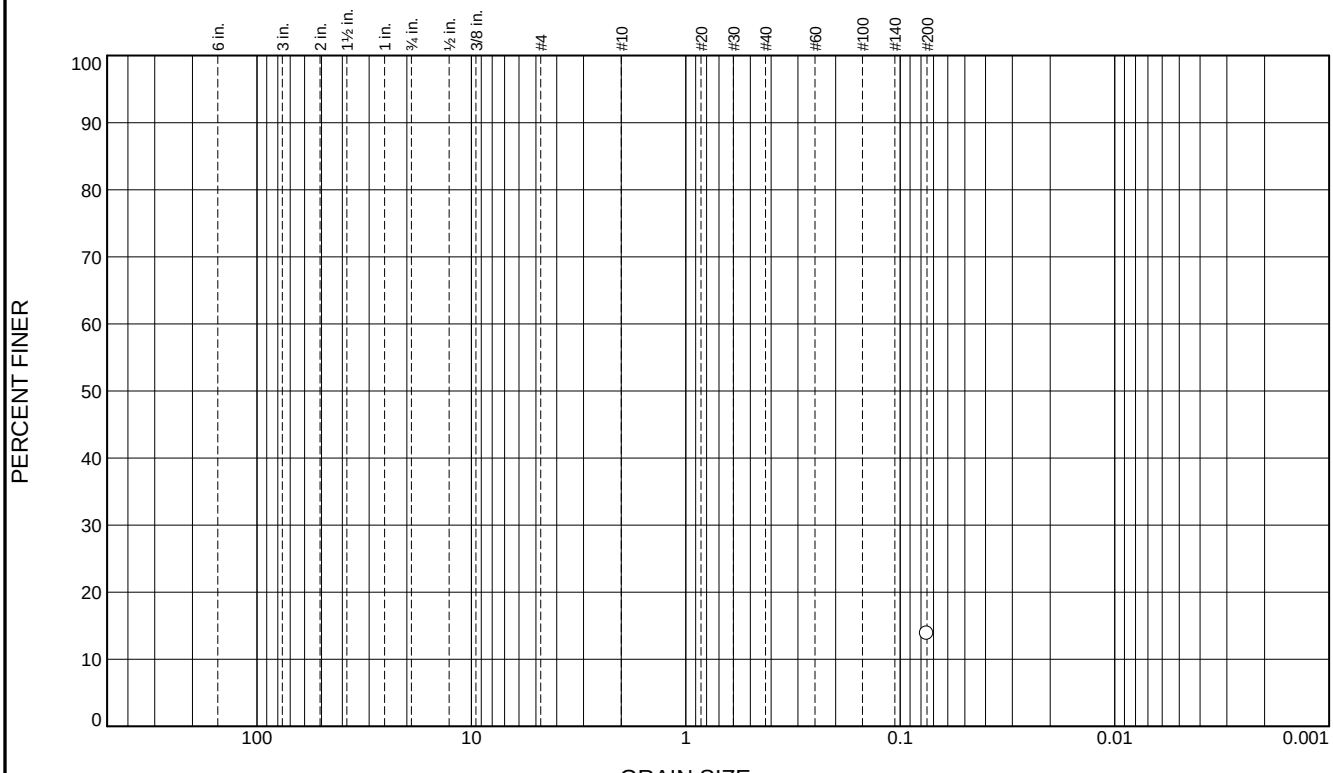
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						13.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	13.8		

* (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= AASHTO=

Remarks

Grain Size: ASTM D1140

Sample Number: 1-B1 @ 35.5 **Depth:** 35.5 feet

Date: 10.10.16

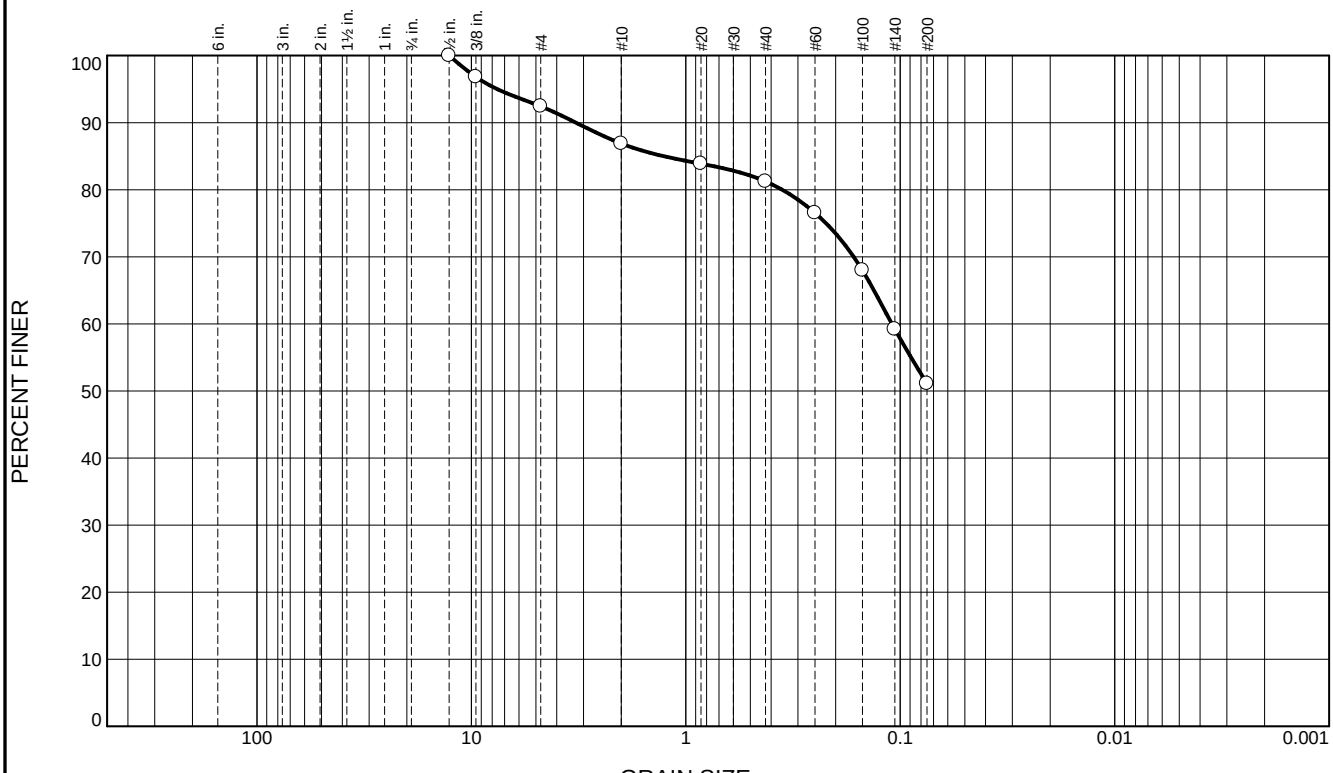
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	7.6	5.5	5.7	30.1	51.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2	100.0		
3/8	96.8		
#4	92.4		
#10	86.9		
#20	83.9		
#40	81.2		
#60	76.5		
#100	68.0		
#140	59.2		
#200	51.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.2573 D_{85}= 1.2688 D_{60}= 0.1094
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Grain Size: ASTM D422

Sample Number: 1-B1 @ 41 **Depth:** 41.0 feet

Date: 10.10.16

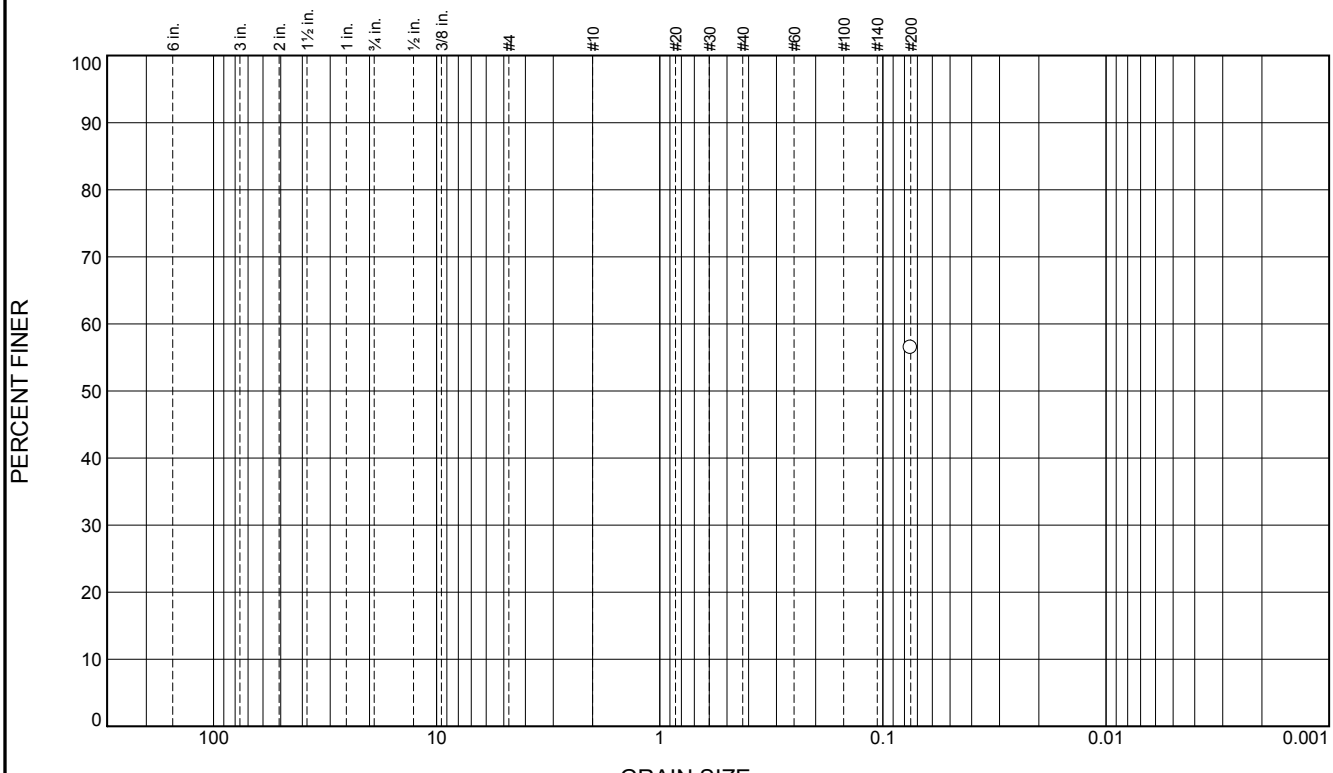
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						56.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	56.4		

* (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= AASHTO=

Remarks

ASTM D1140

Sample Number: 1-B1 @ 51 **Depth:** 51.0 feet

Date: 10/12/16

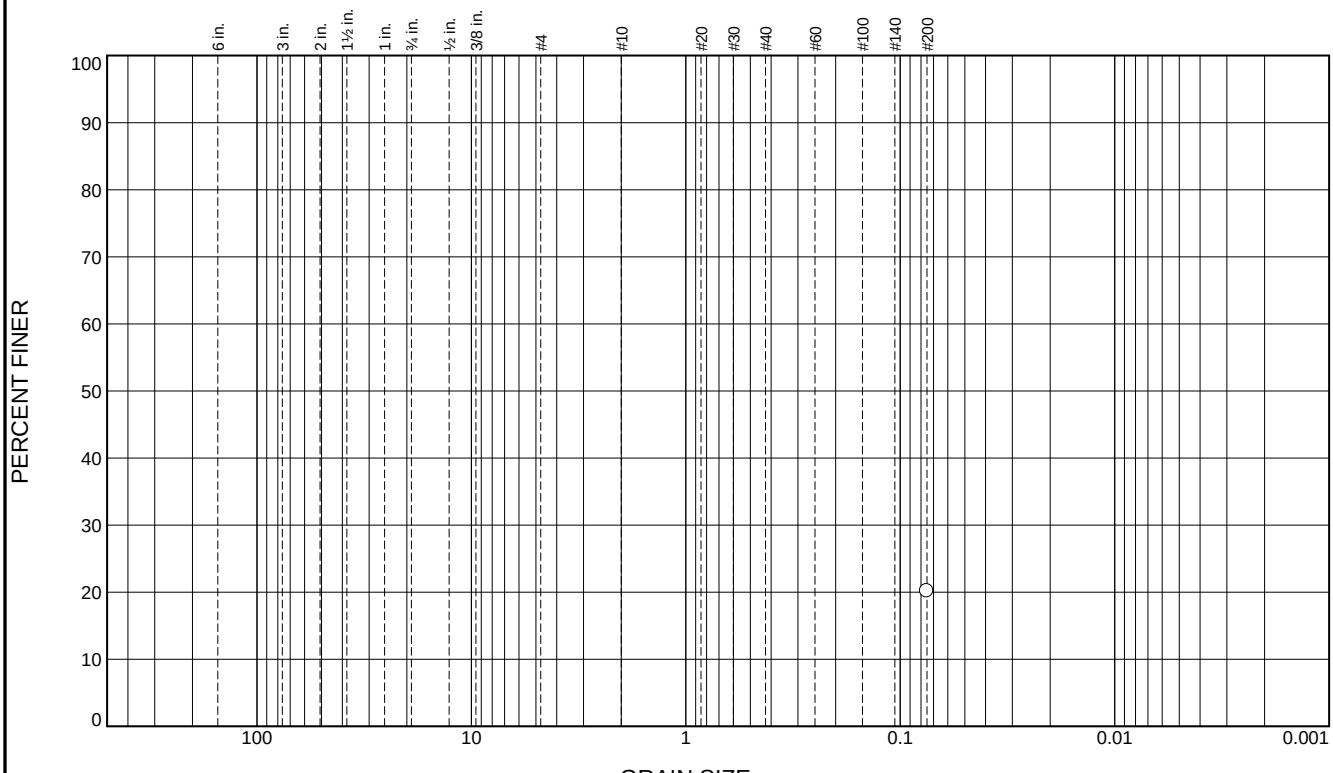
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: M. Quasem **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						20.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	20.1		

* (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= AASHTO=

Remarks

Grain Size: ASTM D1140

Sample Number: 1-B1 @ 66 **Depth:** 66.0 feet

Date: 10.10.16

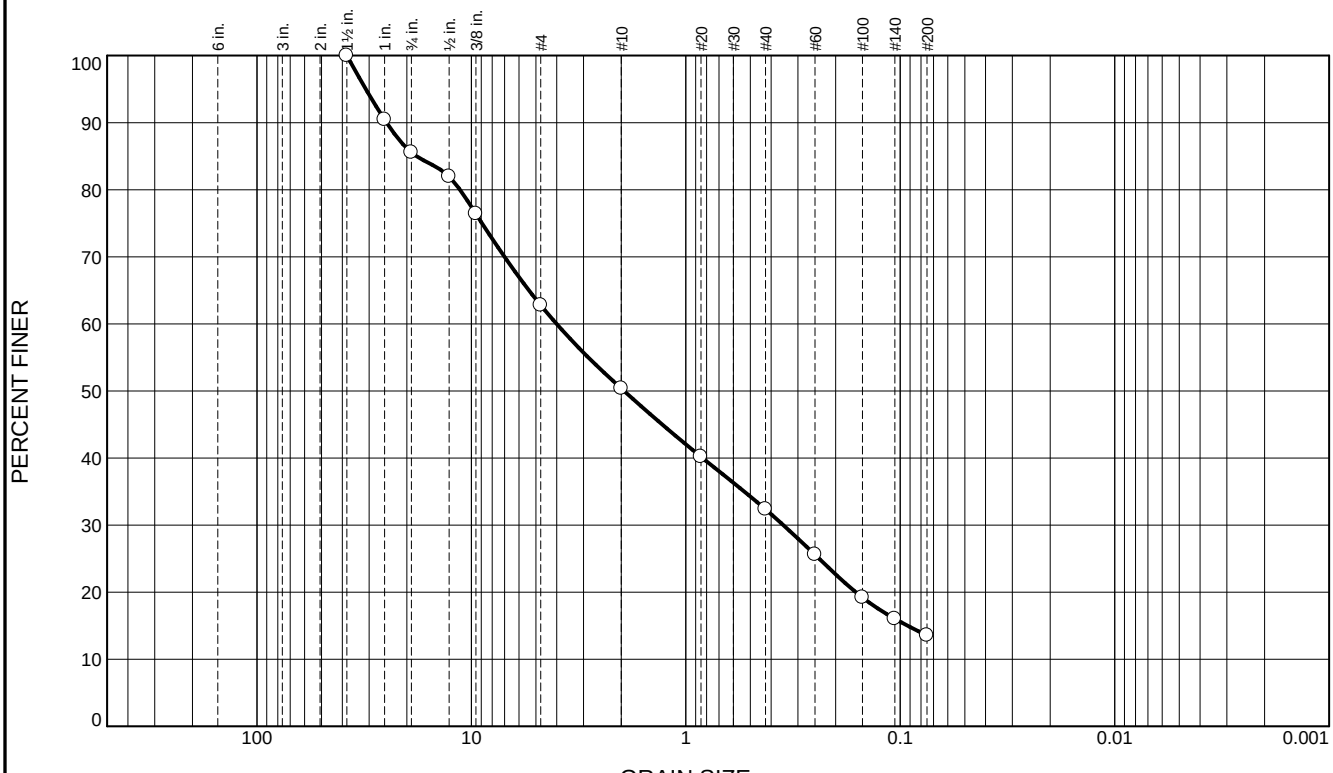
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	14.4	22.8	12.4	18.1	18.8	13.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2	100.0		
1	90.4		
3/4	85.6		
1/2	82.0		
3/8	76.4		
#4	62.8		
#10	50.4		
#20	40.2		
#40	32.3		
#60	25.6		
#100	19.2		
#140	16.0		
#200	13.5		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 24.8608 D_{85}= 18.0338 D_{60}= 4.0083
D_{50}= 1.9417 D_{30}= 0.3513 D_{15}= 0.0925
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Grain Size: ASTM D422

Sample Number: 1-B1 @ 70.5 **Depth:** 70.5 feet

Date: 10.10.16

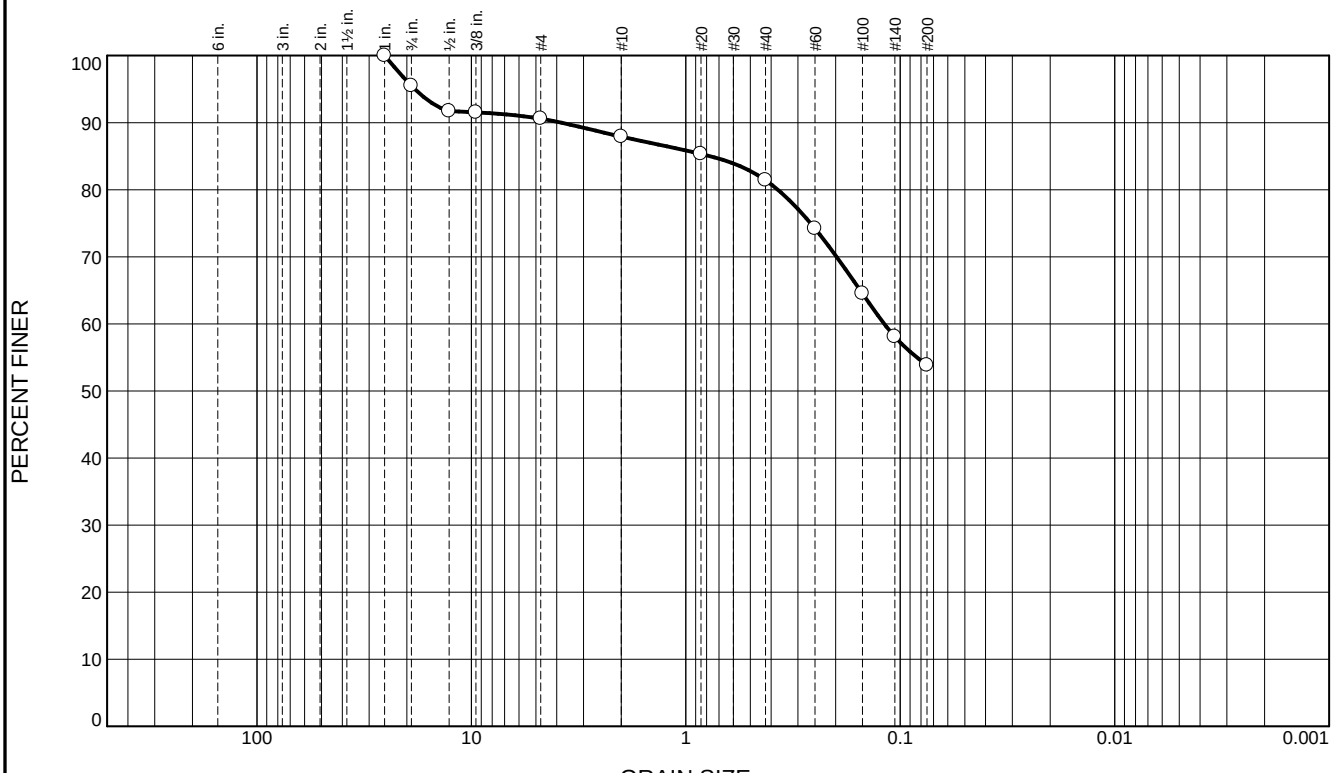
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	4.5	4.9	2.7	6.5	27.6	53.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	95.5		
1/2	91.7		
3/8	91.5		
#4	90.6		
#10	87.9		
#20	85.3		
#40	81.4		
#60	74.2		
#100	64.5		
#140	58.1		
#200	53.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 3.8192 D_{85}= 0.7710 D_{60}= 0.1188
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Grain Size: ASTM D422

Sample Number: 1-B2 @ 6 **Depth:** 6.0 feet

Date: 10.10.16

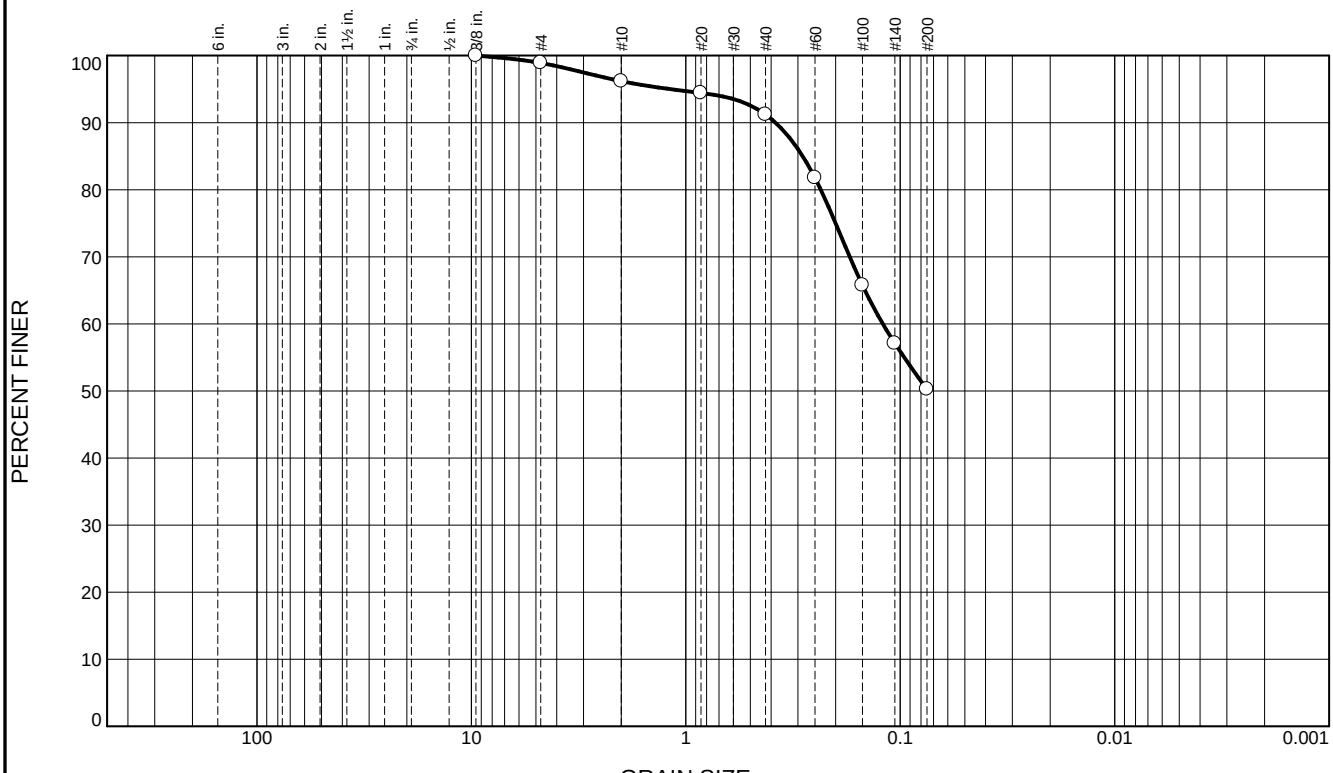
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.1	2.7	5.0	41.0	50.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8	100.0		
#4	98.9		
#10	96.2		
#20	94.4		
#40	91.2		
#60	81.8		
#100	65.7		
#140	57.1		
#200	50.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3819 D_{85}= 0.2852 D_{60}= 0.1206
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Grain Size: ASTM D422

Sample Number: 1-B2 @ 11 **Depth:** 11.0 feet

Date: 10.10.16

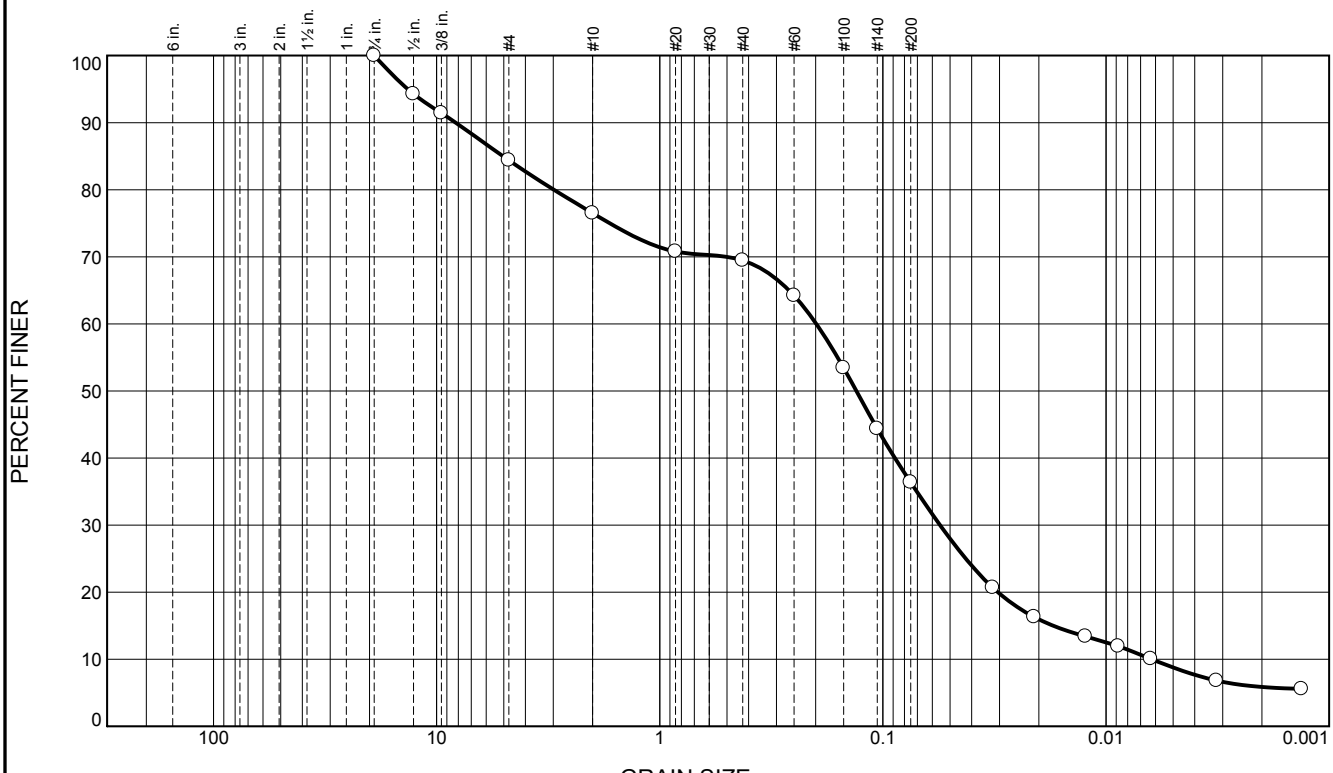
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	15.6	7.9	7.0	33.1	30.5	5.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	94.3		
3/8	91.4		
#4	84.4		
#10	76.5		
#20	70.8		
#40	69.5		
#60	64.2		
#100	53.4		
#140	44.4		
#200	36.4		
0.0322 mm.	20.7		
0.0210 mm.	16.3		
0.0124 mm.	13.4		
0.0088 mm.	11.9		
0.0063 mm.	10.0		
0.0032 mm.	6.8		
0.0013 mm.	5.6		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 8.2262 D_{85}= 5.0494 D_{60}= 0.1989
D_{50}= 0.1315 D_{30}= 0.0555 D_{15}= 0.0173
D_{10}= 0.0062 C_u= 31.88 C_c= 2.48

Classification

USCS= AASHTO=

Remarks

GS: ASTM D422; Specific gravity = 2.672, ASTM D854
Silt/clay division of 0.002mm was used

Sample Number: 1-B2 @ 18.5 **Depth:** 18.5 feet

Date: 10/11/16

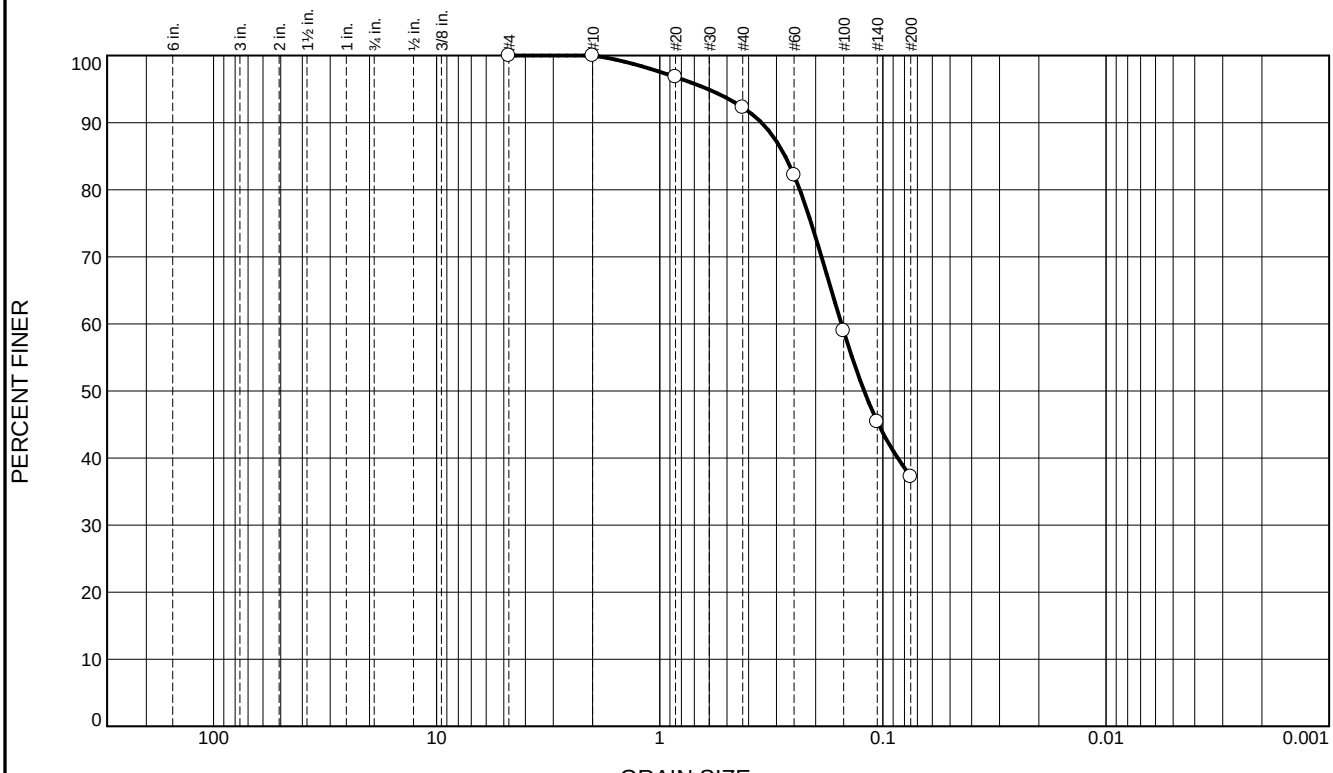
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: M. Quasem **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	7.7	55.1	37.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	96.8		
#40	92.3		
#60	82.2		
#100	58.9		
#140	45.4		
#200	37.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3499 D_{85}= 0.2739 D_{60}= 0.1534
D_{50}= 0.1213 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

Sample Number: 1-B3 @ 16 **Depth:** 16.0 feet

Date: 10/12/16

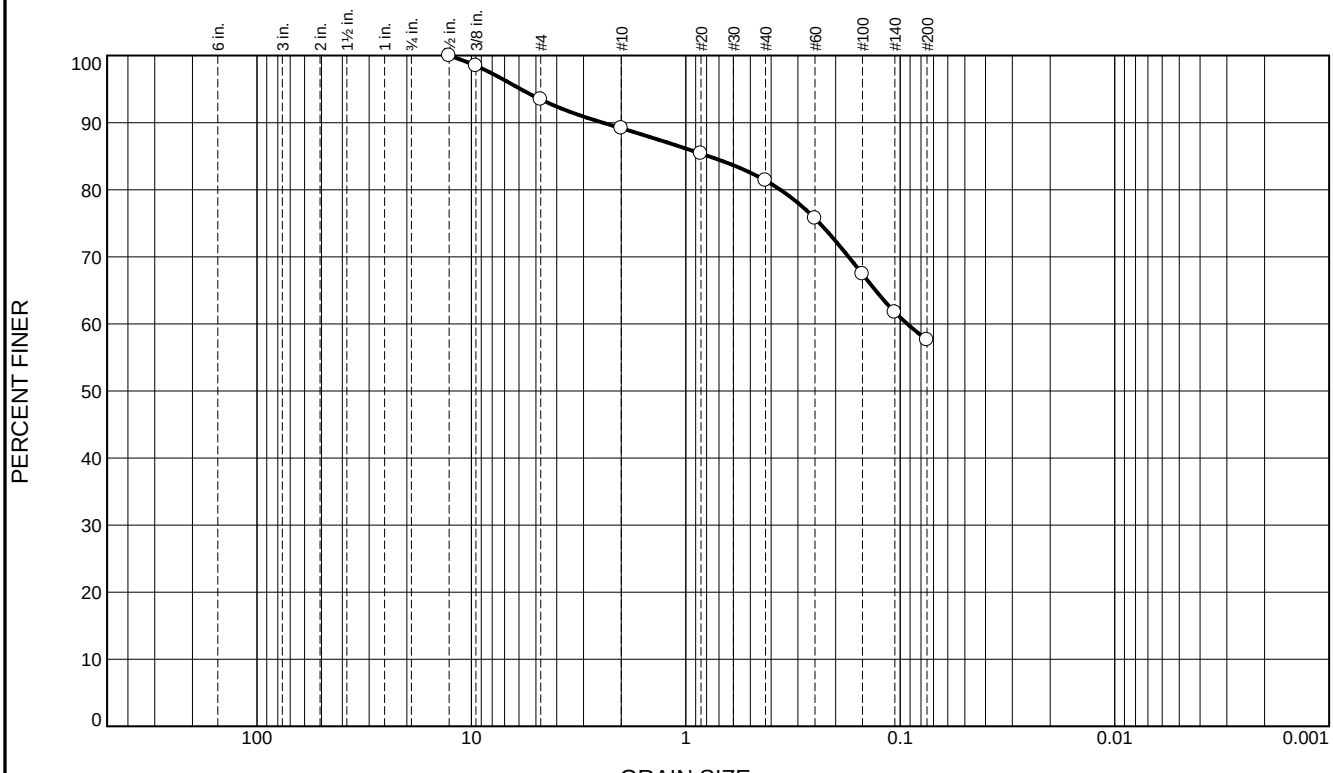
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: M. Quasem **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	6.5	4.3	7.8	23.8	57.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2	100.0		
3/8	98.5		
#4	93.5		
#10	89.2		
#20	85.4		
#40	81.4		
#60	75.7		
#100	67.4		
#140	61.7		
#200	57.6		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 2.4385 D_{85}= 0.7817 D_{60}= 0.0929
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Grain Size: ASTM D422

Sample Number: 1-B4 @ 3 **Depth:** 3.0 feet

Date: 10.10.16

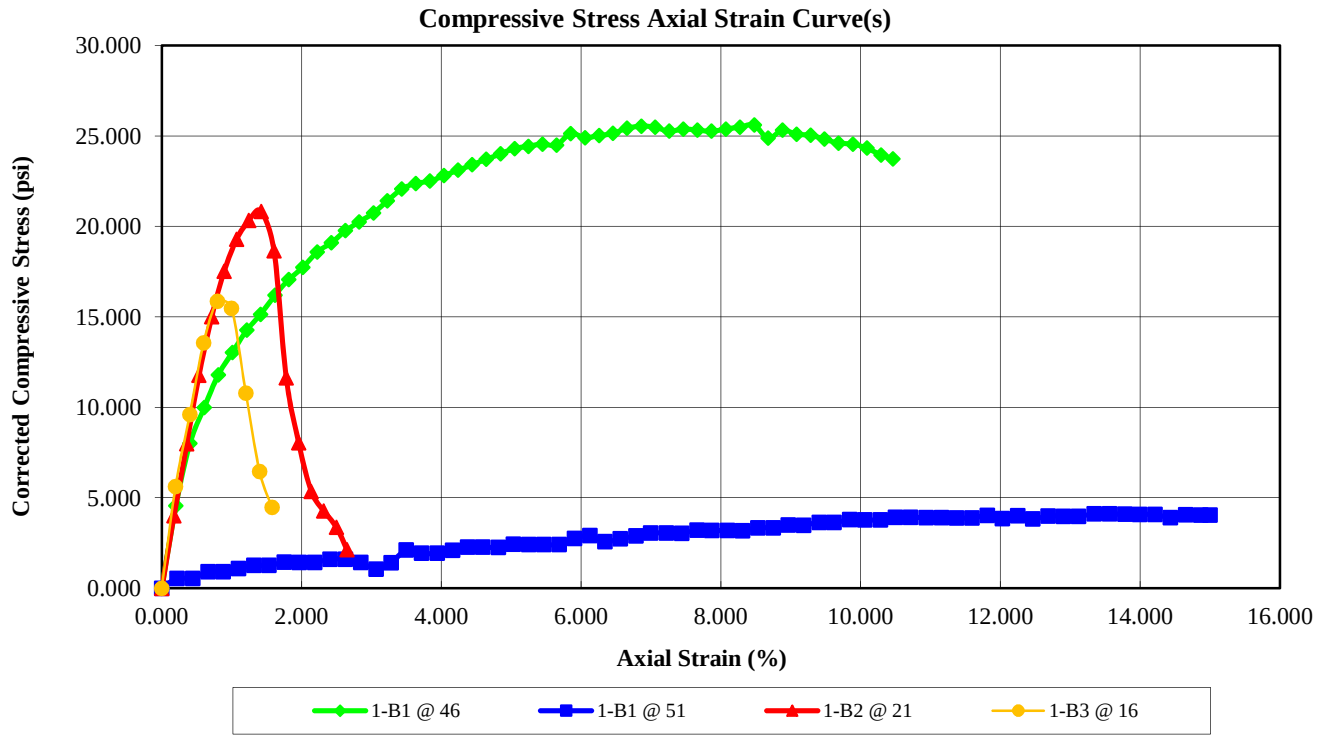
ENGEO INCORPORATED

Client: Fort Bay Stevens Creek, LLC
Project: 4300 Stevens Creek, San Jose

Project No: 13317.000.000

Tested By: T. Borde **Checked By:** D. Seibold

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B1 @ 46	1-B1 @ 51	1-B2 @ 21	1-B3 @ 16
Moisture Content (%)	20.7	20.1	13.4	8.6
Dry Density (pcf)	108.3	108.1	111.3	98.1
Saturation (%)	100.00	100.00	73.28	33.41
Void Ratio	0.53	0.53	0.49	0.69
Diameter (in)	2.396	2.397	2.400	2.403
Height (in)	4.998	4.611	5.649	5.029
Height-To-Diameter Ratio	2.086	1.924	2.354	2.093
TEST DATA				
Unconfined Compressive Strength (psf)	3686.868	592.719	3000.673	2285.122
Undrained Shear Strength (psf)	1843.434	296.359	1500.337	1142.561
Strain Rate (in./min.)	0.05	0.05	0.05	0.05
Specific Gravity	2.650	2.650	2.650	2.650
Strain at Failure (%)	8.48	13.34	1.42	0.80
Liquid Limit				
Plastic Limit				
Test Remarks				

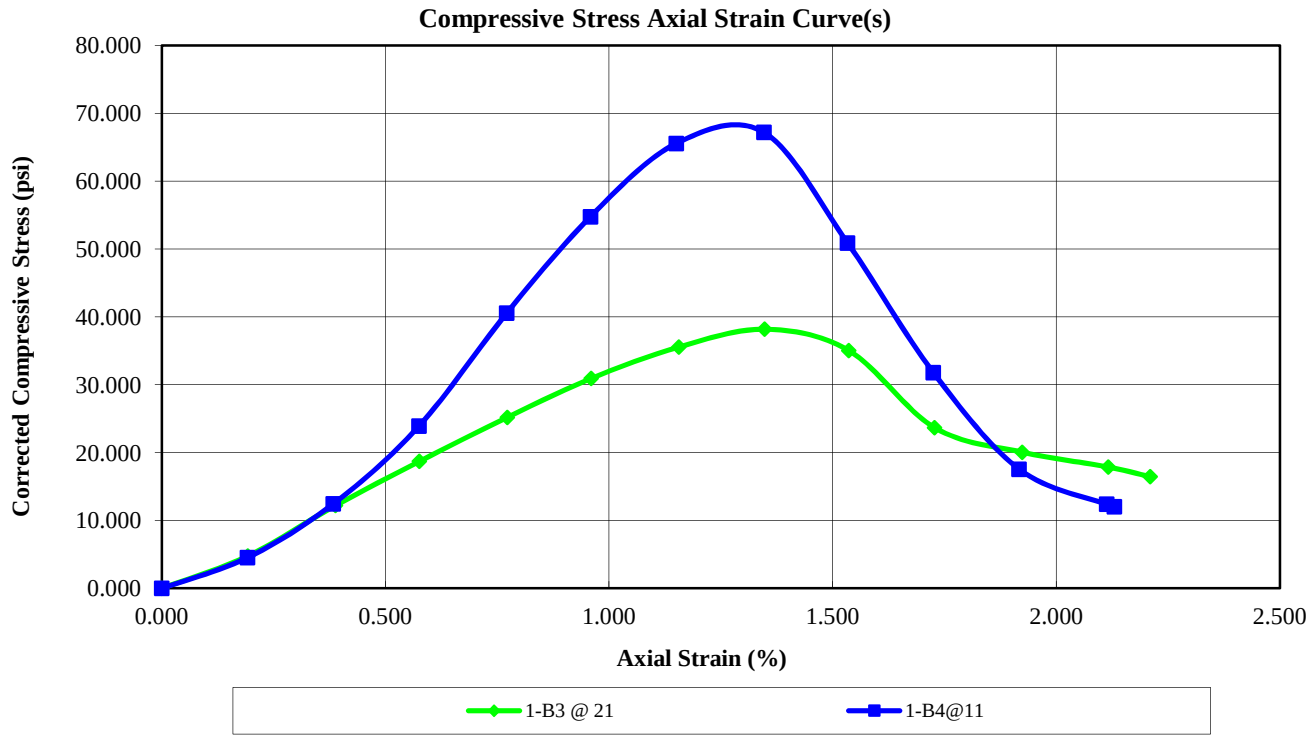
SPECIMEN	DESCRIPTION
1-B1 @ 46	See exploration logs
1-B1 @ 51	See exploration logs
1-B2 @ 21	See exploration logs
1-B3 @ 16	See exploration logs

PROJECT NAME: 4300 Stevens Creek, San Jose
PROJECT NO: 13317.000.000
CLIENT: Fort Bay Creek, LLC
LOCATION: San Jose, CA
PHASE NO: 001

Test Date: 10/10/16
Tested By: M. Quasem
Reviewed By: D. Seibold

ENGEO — Expect Excellence

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B3 @ 21	1-B4@11 ft
Moisture Content (%)	17.1	7.9
Dry Density (pcf)	105.6	113.5
Saturation (%)	79.78	45.58
Void Ratio	0.57	0.46
Diameter (in)	2.398	2.410
Height (in)	5.252	5.260
Height-To-Diameter Ratio	2.190	2.183
TEST DATA		
Unconfined Compressive Strength (psf)	5497.365	9679.064
Undrained Shear Strength (psf)	2748.683	4839.532
Strain Rate (in./min.)	0.05	0.05
Specific Gravity	2.650	2.650
Strain at Failure (%)	1.35	1.35
Liquid Limit		
Plastic Limit		
Test Remarks		

SPECIMEN	DESCRIPTION
1-B3 @ 21	See exploration logs
1-B4@11 ft	See Exploration Logs

PROJECT NAME: 4300 Stevens Creek, San Jose
PROJECT NO: 13317.000.000
CLIENT: Fort Bay Stevens Creek, LLC
LOCATION: San Jose, CA
PHASE NO: 001

Test Date: 10/10/16
Tested By: M. Quasem
Reviewed By: D. Seibold

ENGEO
Expect Excellence

EN GEO

Unconsolidated Undrained Triaxial Test (ASTM D2850)

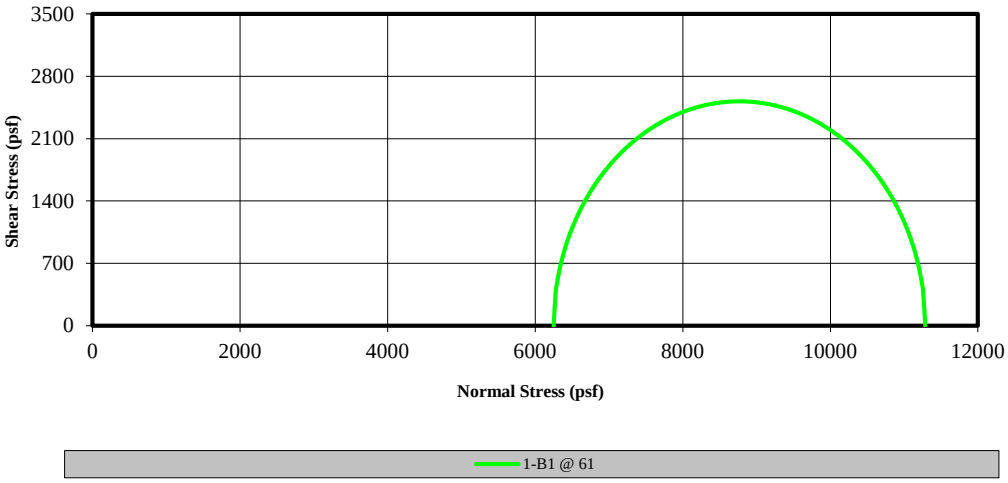
Date: 10/07/16

Checked By: G. Criste

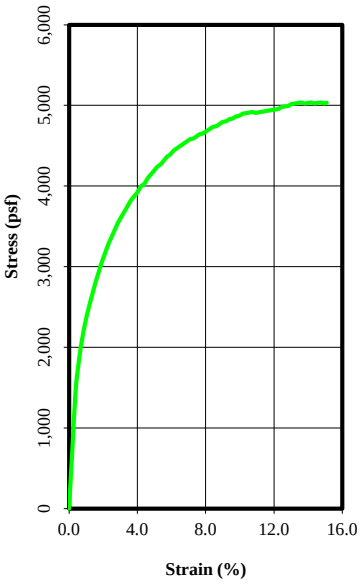
Date: 10/07/16

Tested By: M. Quasem

Mohr Circles



Stress-Strain Curve



Before Test	1-B1 @ 61	Specimen		
Water Content (%)	19.69			
Dry Density (pcf)	110.58			
Saturation (%)	100.00			
Void Ratio	0.52			
Diameter (in)	2.401			
Height (in)	5.258			
Liquid Limit	-			
Plastic Limit	-			
Specific Gravity (Assumed)	2.700			
Height-to-Diameter Ratio	2.190			
After Test	**1-B1 @ 61**			
Water Content (%)	19.69			
Saturation (%)	100.00			
Strain Rate (in/min)	0.05			
Peak Deviator Stress (psf)	5036.9			
Axial Strain @ Failure (%)	14.743			
	Cell Pressure			
Cell (psf)	6249.6			
Back (psf)	n/a			
	Principle Stresses at Failure			
σ1 (psf)	11286.5			
σ3 (psf)	6249.6			

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	0.0		2518.4			
Friction Angle Ø	0.00		n/a			

Project Information			
Project Name:	4300 Stevens Creek, San Jose		
Project Number:	13317.000.000	Job Number:	13317.000.000
Location:	San Jose, CA	Boring Number:	1-B1
Client:	Fort Bay Stevens Creek, LLC	Sample Number:	1-B1 @ 61
Description:	See exploration logs		

WATER SOLUBLE SULFATES IN SOILS

ASTM C1580

Sample number	Sample Location / ID	Matrix	Water Soluble Sulfate % by mass
1	1-B1 @ 11	soil	ND
2	1-B4 @ 2.5	soil	ND

Remarks: Results are reported to the nearest 0.01% by mass. Anything less than 0.005% will be reported as 'ND' for Not-Detectable.

PROJECT NAME: 4300 Stevens Creek, San Jose
PROJECT NUMBER: 13317.000.000
CLIENT: Fort Bay Stevens Creek, LLC
PHASE NUMBER: 001

DATE: 10/10/16

ENGEO
Expect Excellence

Tested by: M. Quasem

Reviewed by: G. Criste

Lab Address: 3420 Fostoria Way Suite E, San Ramon, CA 94583. Phone No. (925) 355-9047

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.: 13317.000.000
Client's Project Name: Stevens Creek Promenade
Date Sampled: 4-Oct-16
Date Received: 5-Oct-16
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 17-Oct-2016

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)*
1610047-001	1-B2 @ 16'	480	7.20	-	3,600	-	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	14-Oct-2016	14-Oct-2016	-	14-Oct-2016	-	14-Oct-2016	14-Oct-2016

* 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

1610047 11521

CHAIN OF CUSTODY RECORD

PROJECT NUMBER: 13317.000.000

PROJECT NAME: Stevens Creek Promenade

SAMPLED BY: (PRINT) Yanet Zepeda

PROJECT MANAGER: Jeanine Ruffoni / (925) 967-8740

ROUTING: E-MAIL JRuffoni@engeo.com Hard Copy NA

	SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE	Redox	pH	Sulfate	Resistivity	Chloride											REMARKS REQUIRED DETECTION LIMITS
001	1-B2 @ 16 ft	10/4/2016	-----	SOIL	1	Bag	NA	X	X	X	X	X											ASTM Test Methods

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
Cleanna Gallaty	10/5 2:14			10/5/16 1515	
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED FOR LABORATORY BY: (SIGNATURE)	DATE/TIME	REMARKS	
				STANDARD TAT	

ENGEO INCORPORATED

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SAN JOSE, CALIFORNIA 95119
(408) 574-4900 FAX (888) 279-2698
WWW.ENGEO.COM

DISTRIBUTION: ORIGINAL ACCOMPANIES SHIPMENT; COPY TO PROJECT FIELD FILES

DRAFT

APPENDIX D

LIQUEFACTION ANALYSIS

ENGEO Incorporated

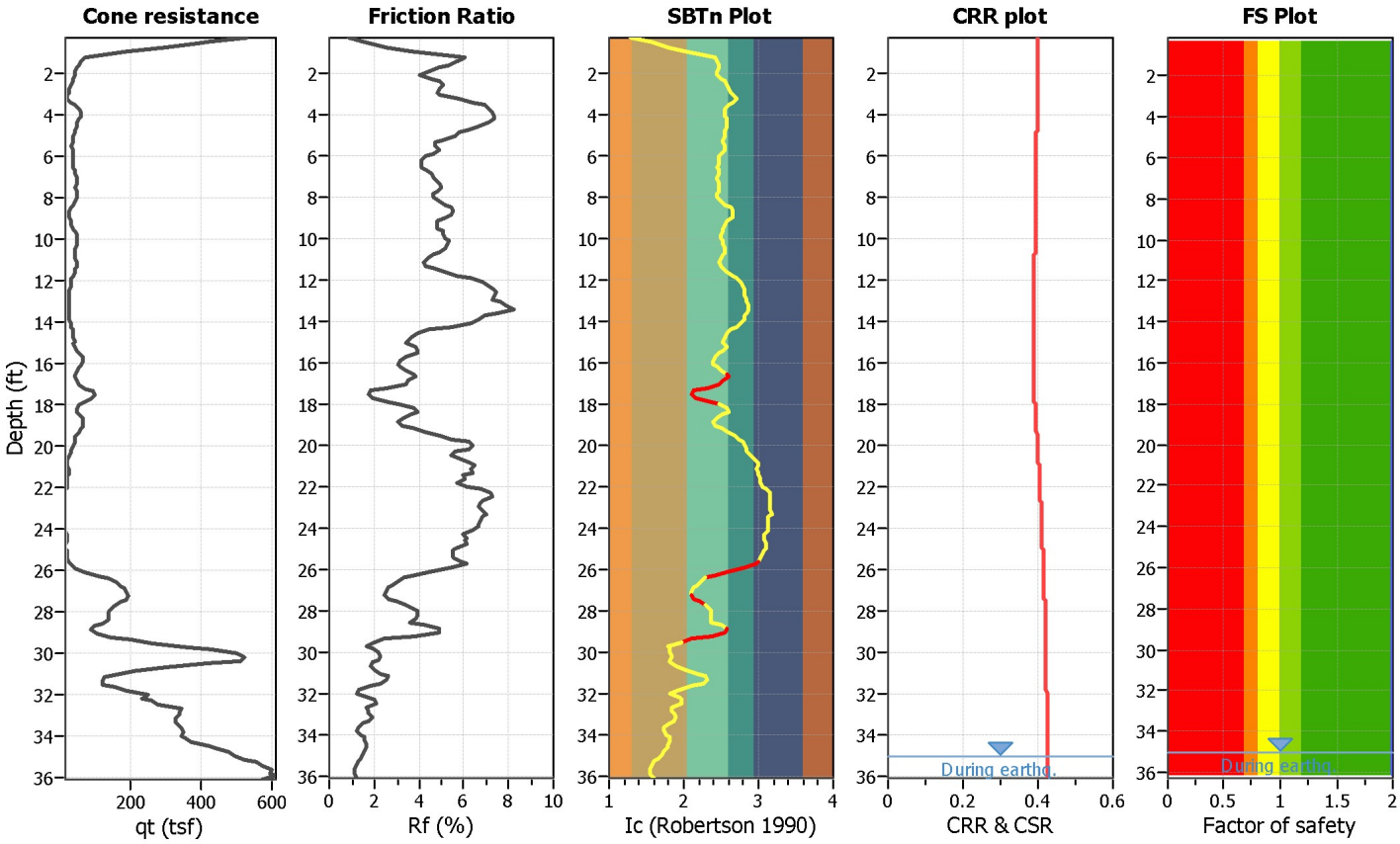
LIQUEFACTION ANALYSIS REPORT

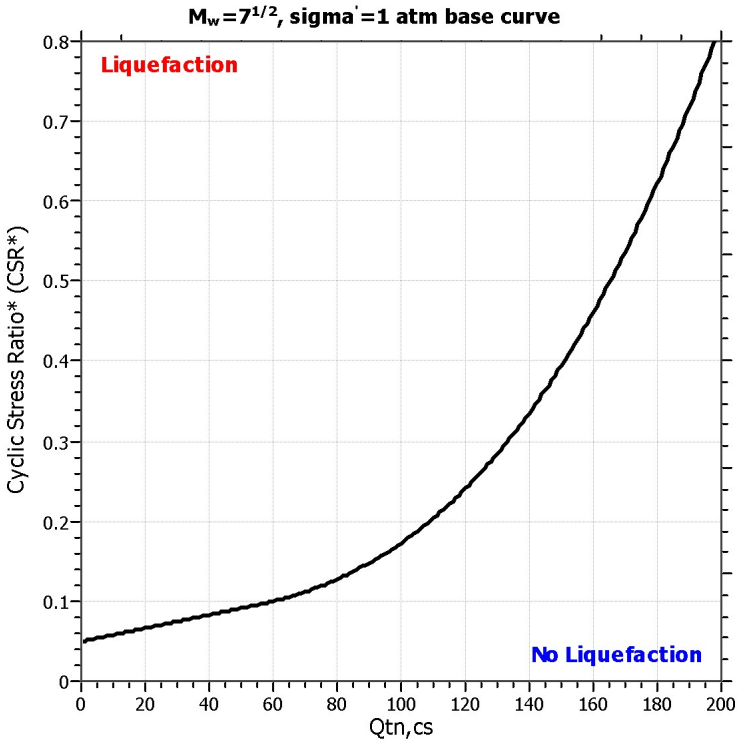
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

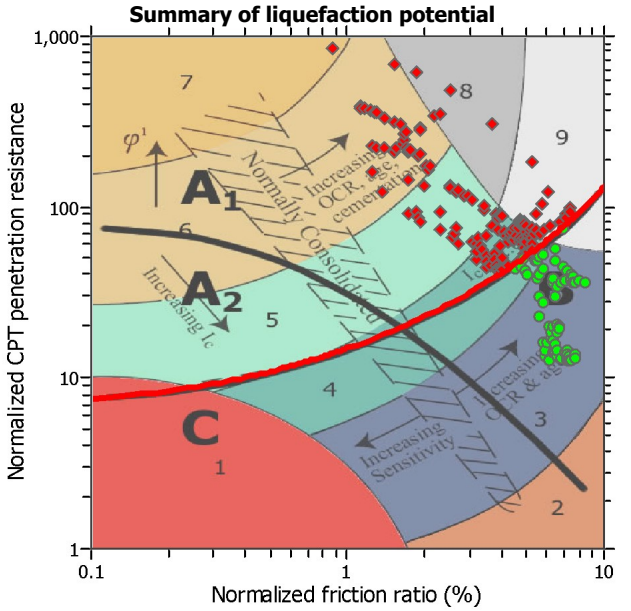
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

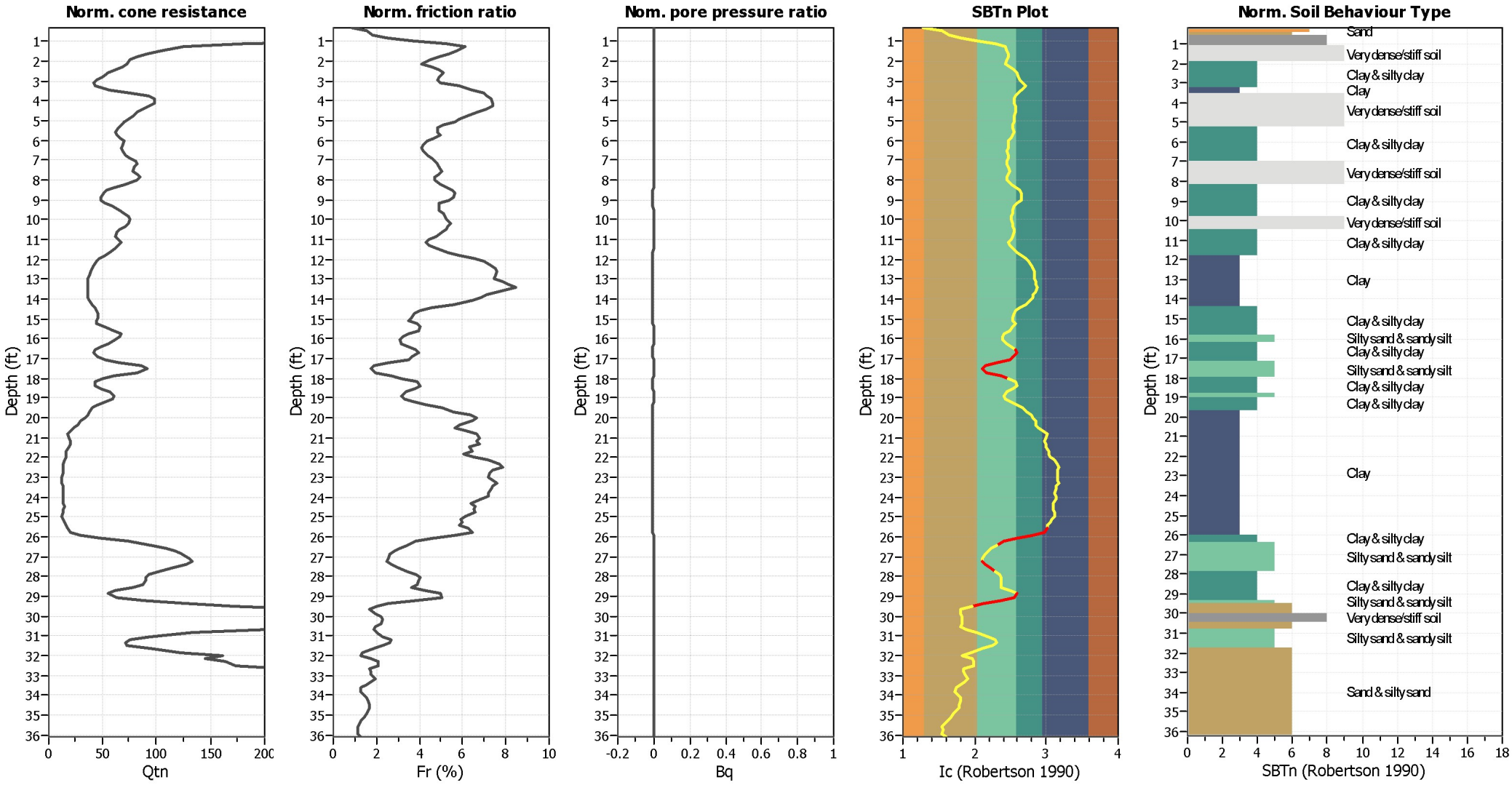






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



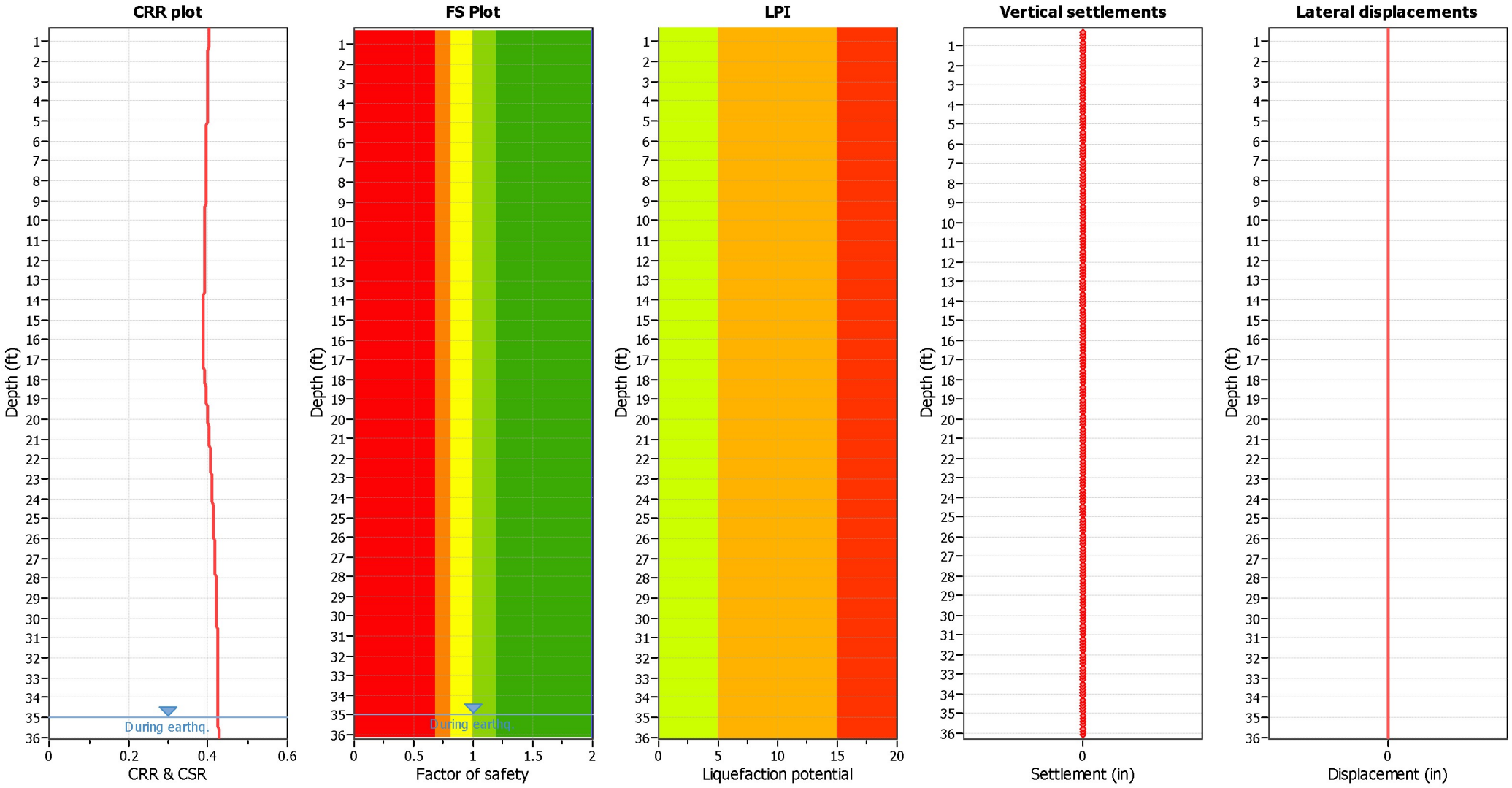
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

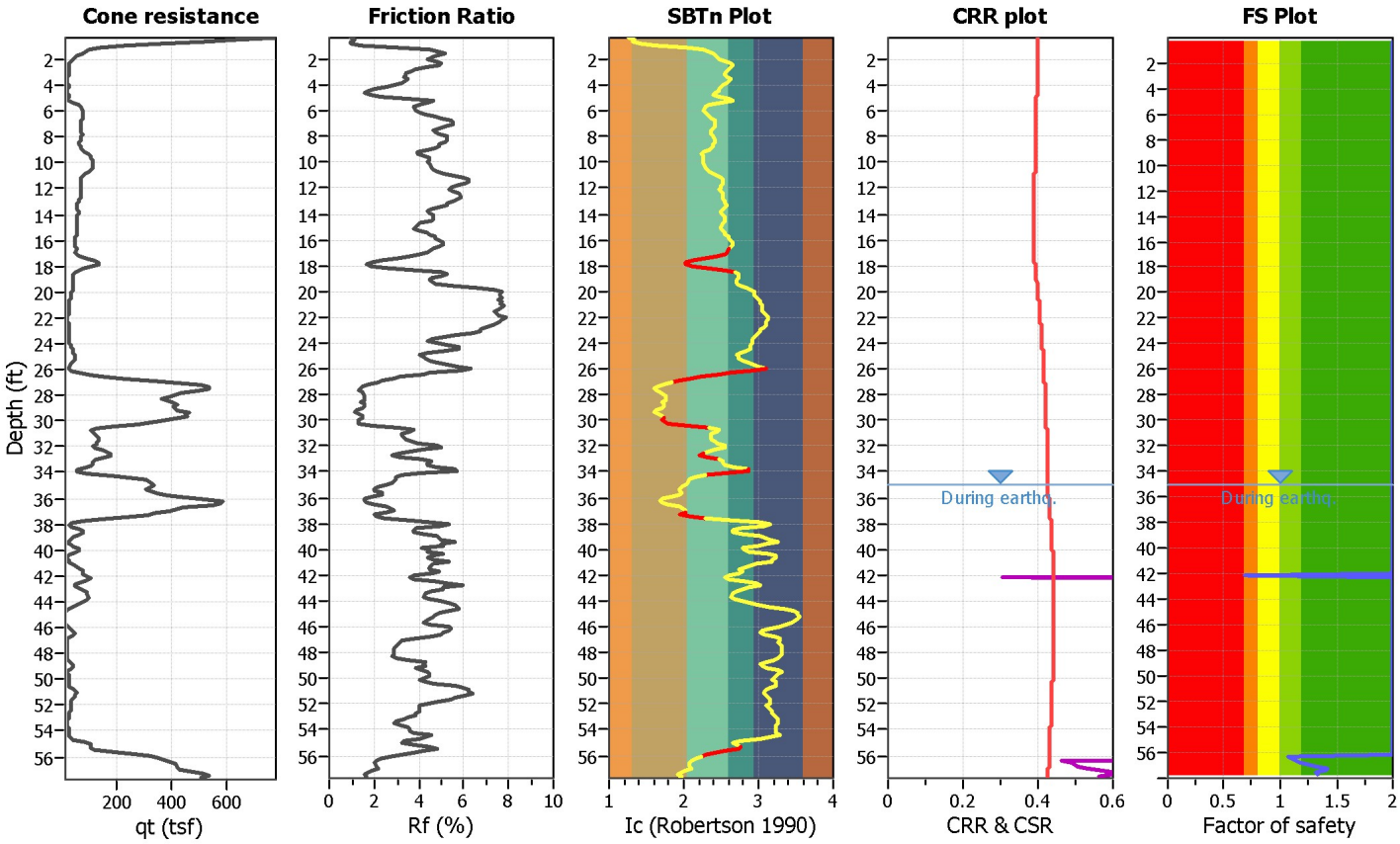
LIQUEFACTION ANALYSIS REPORT

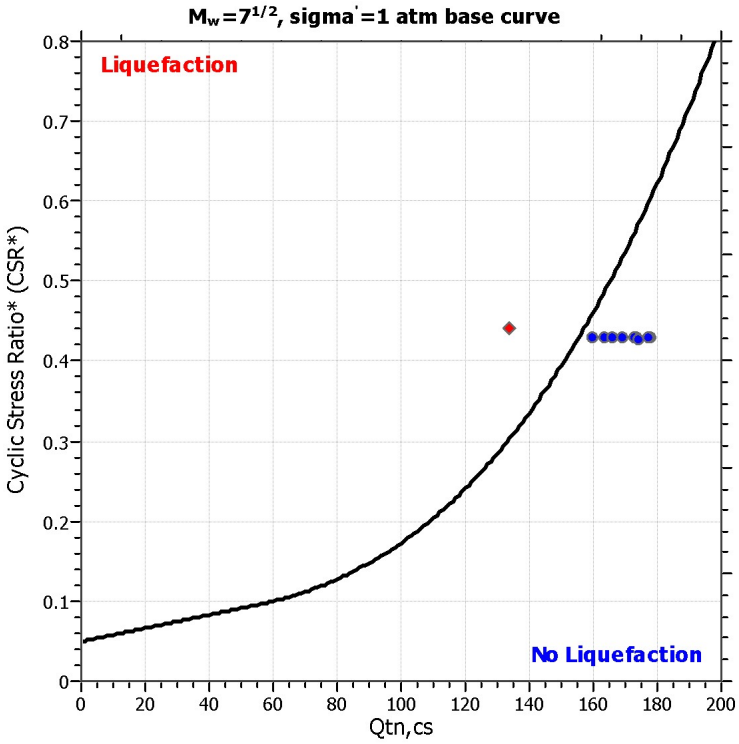
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

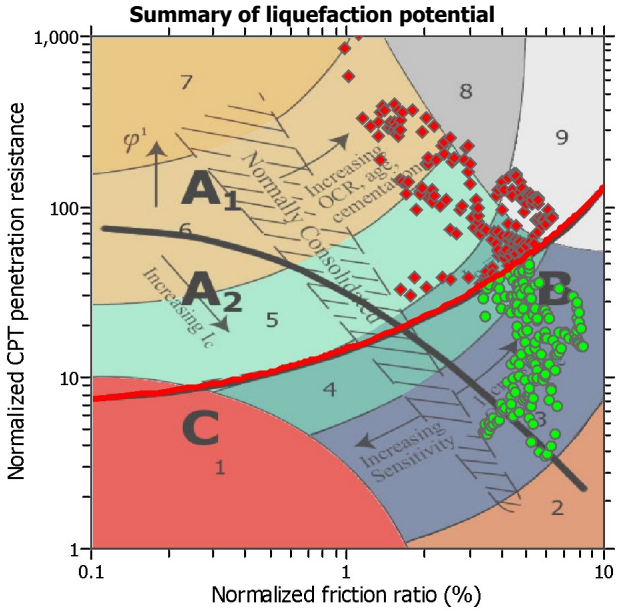
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

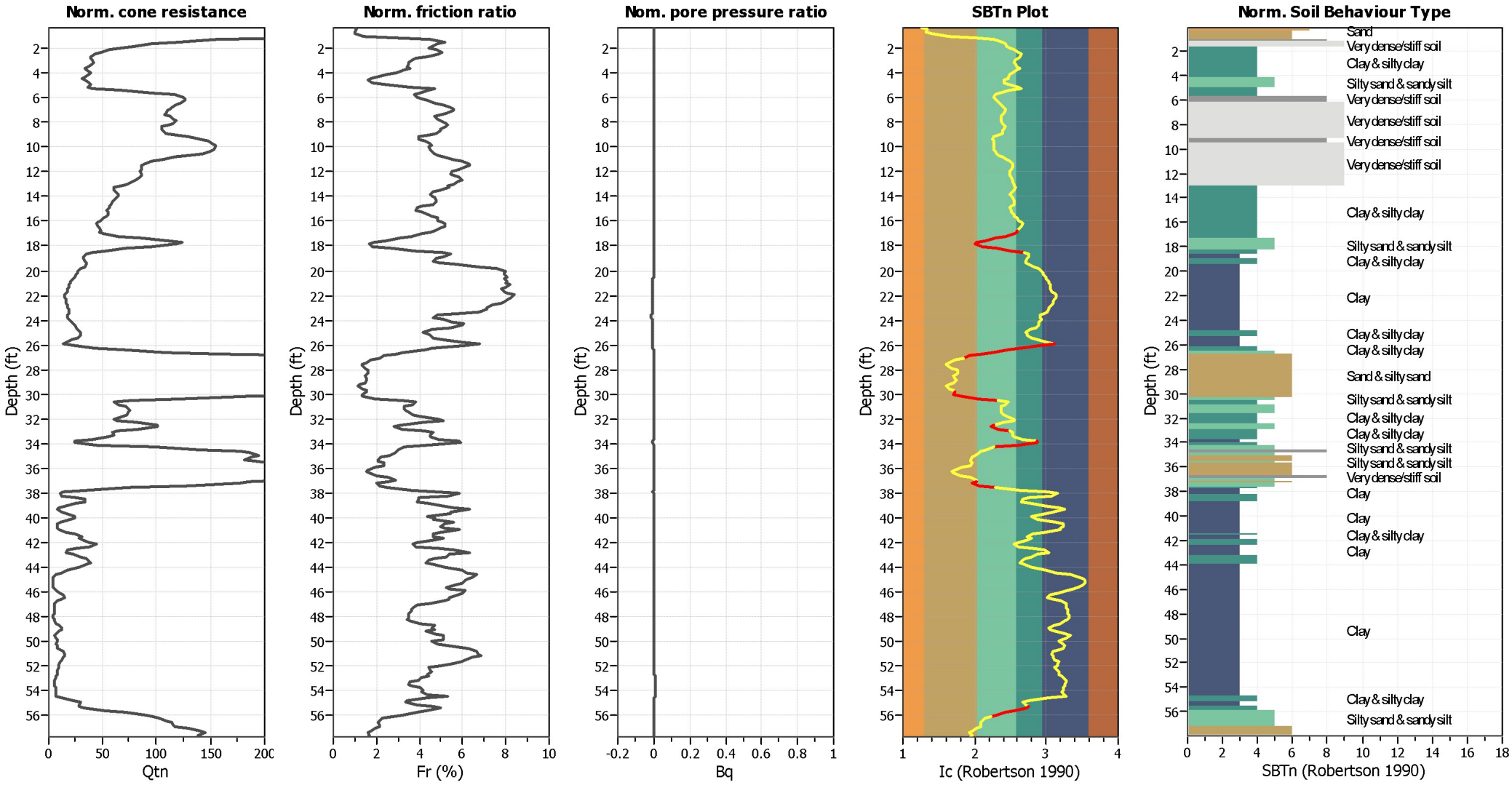






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



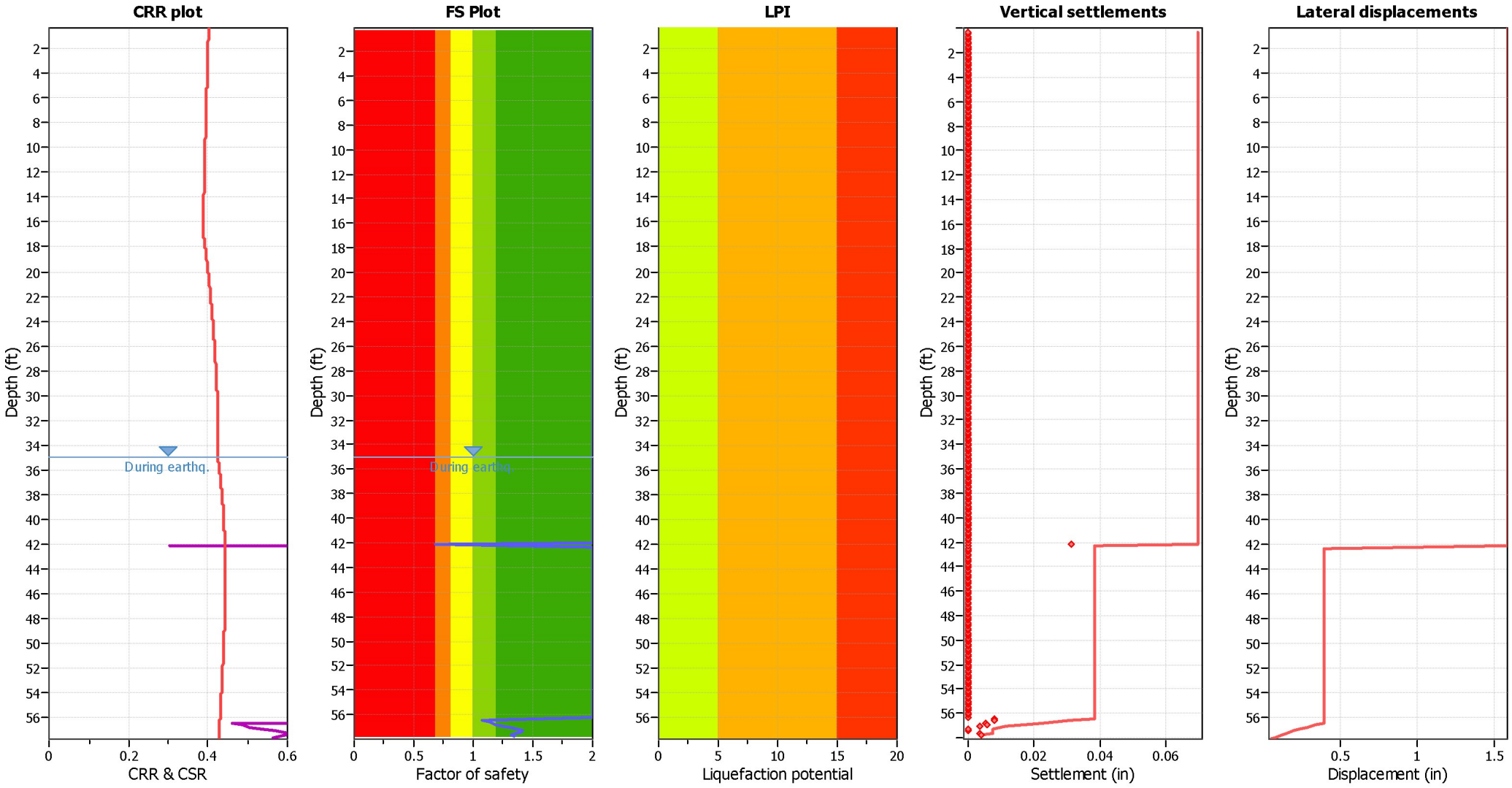
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

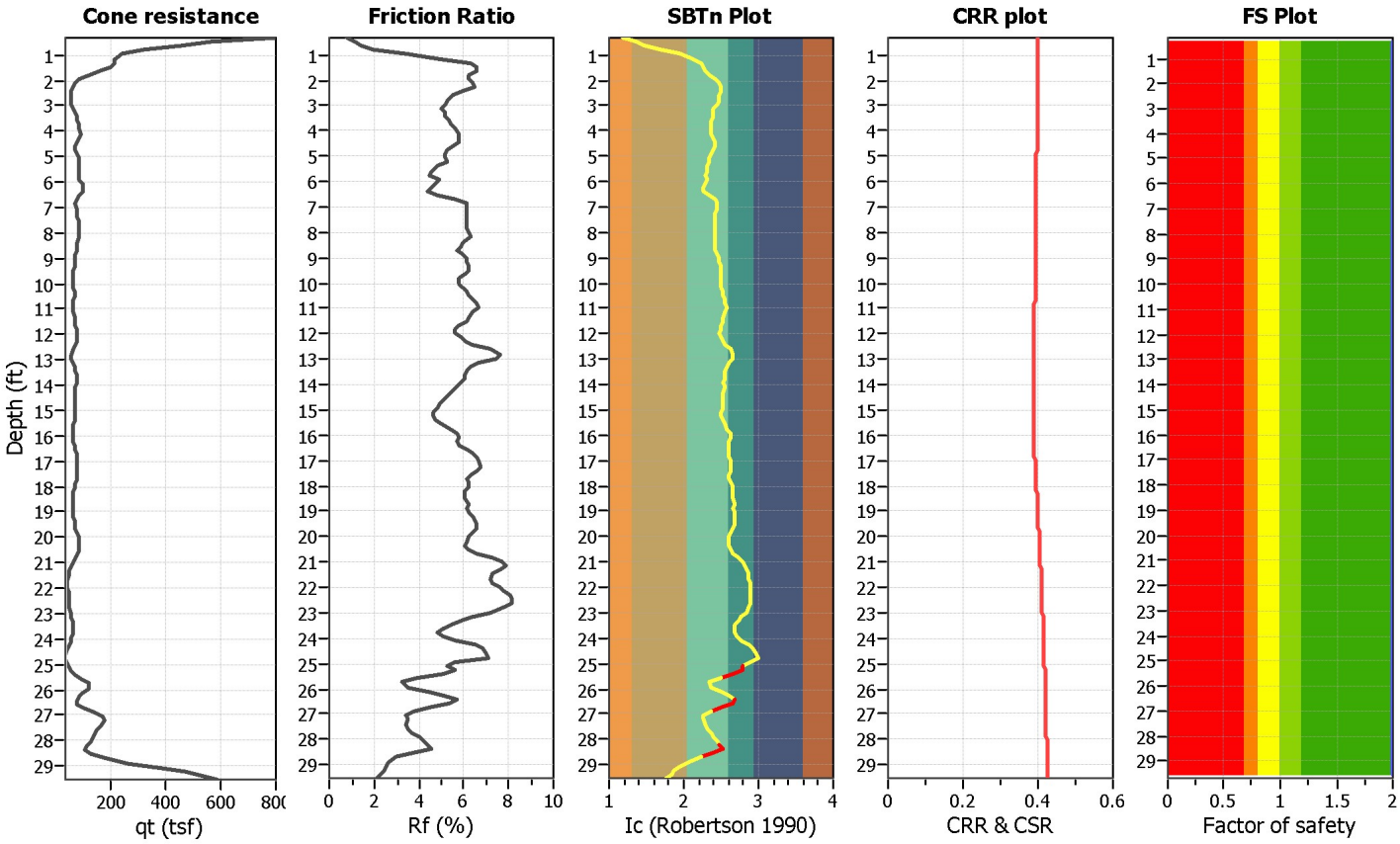
LIQUEFACTION ANALYSIS REPORT

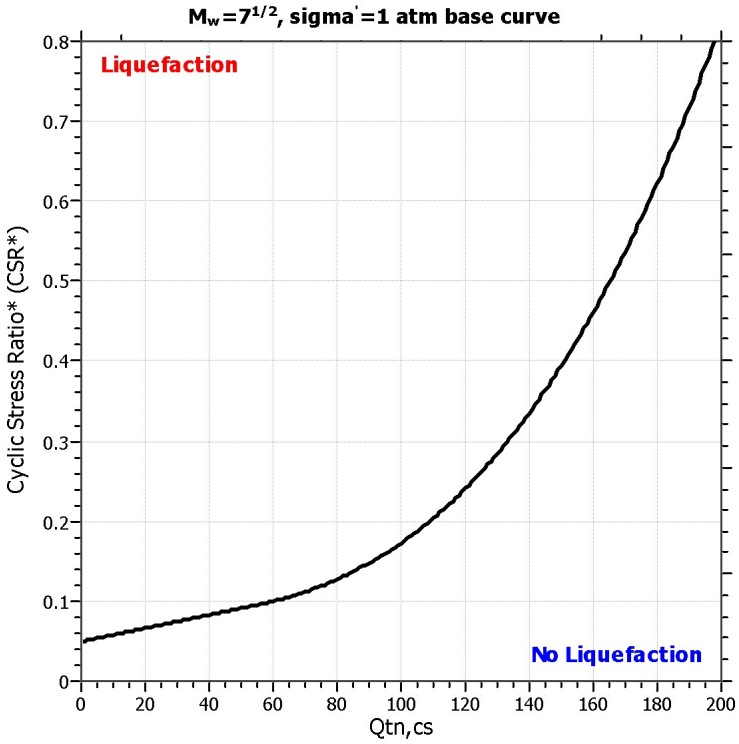
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

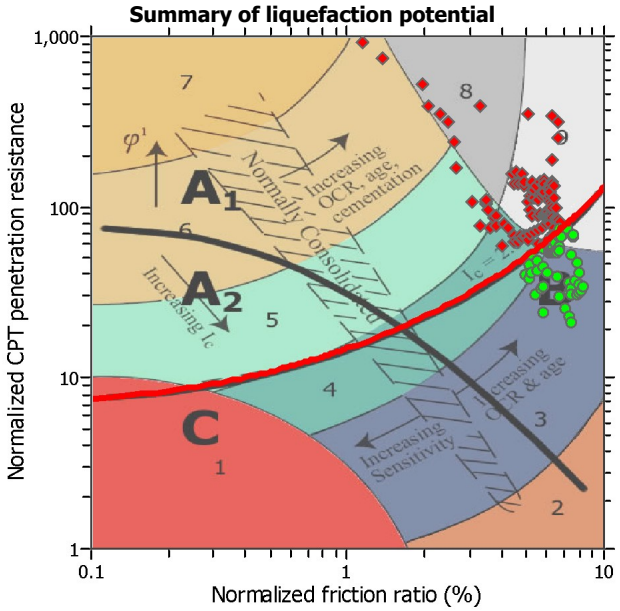
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

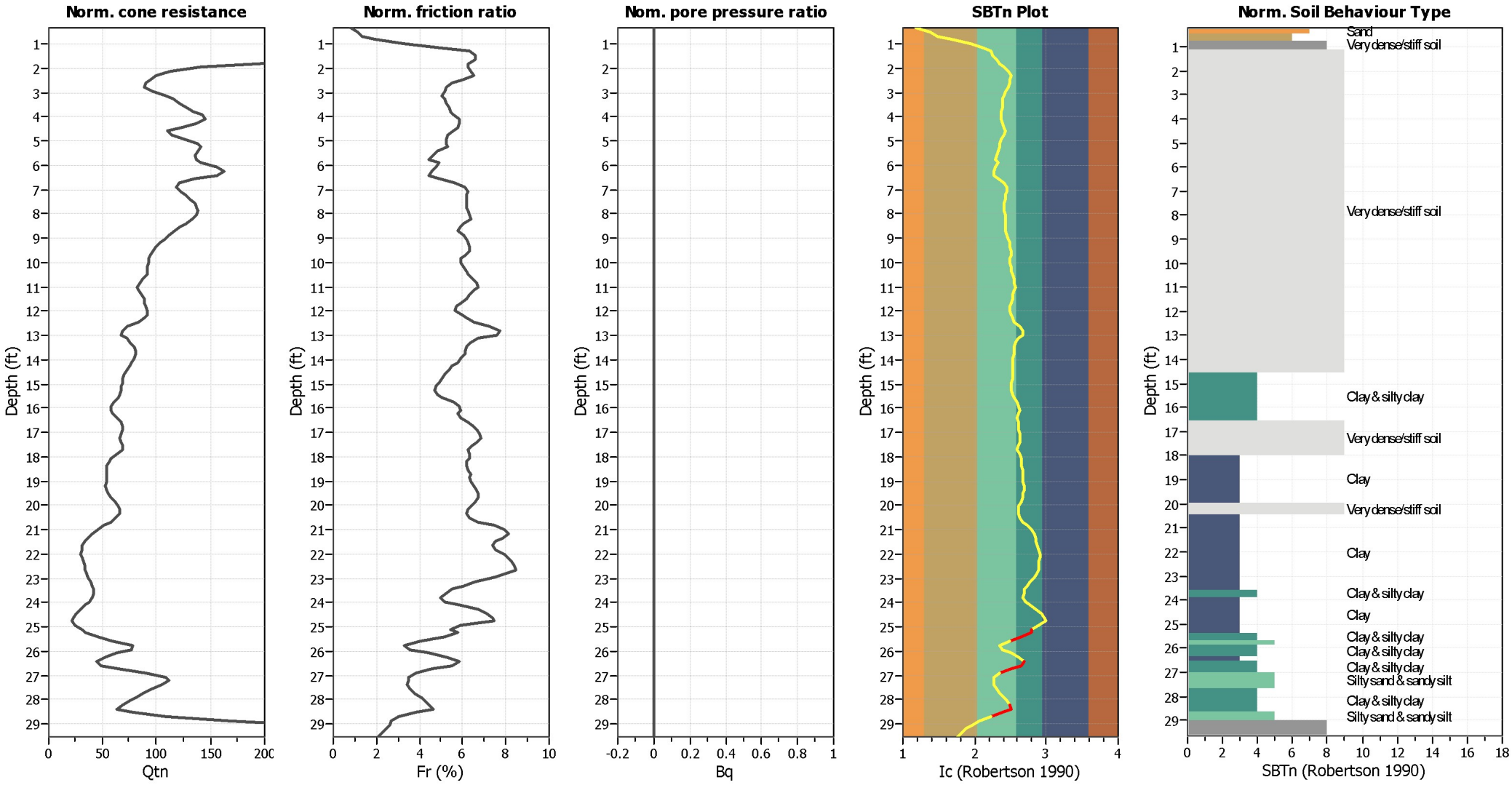






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



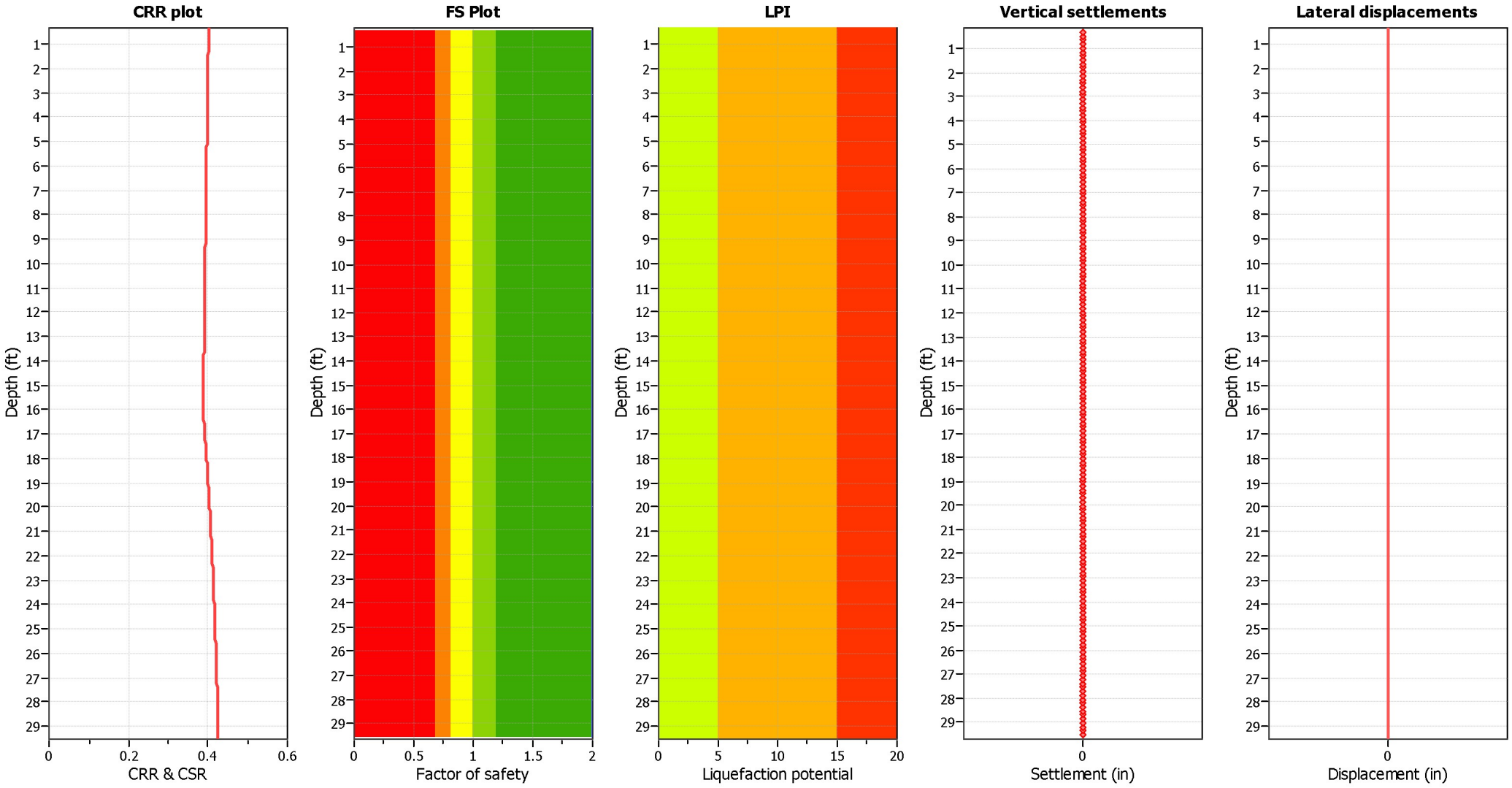
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

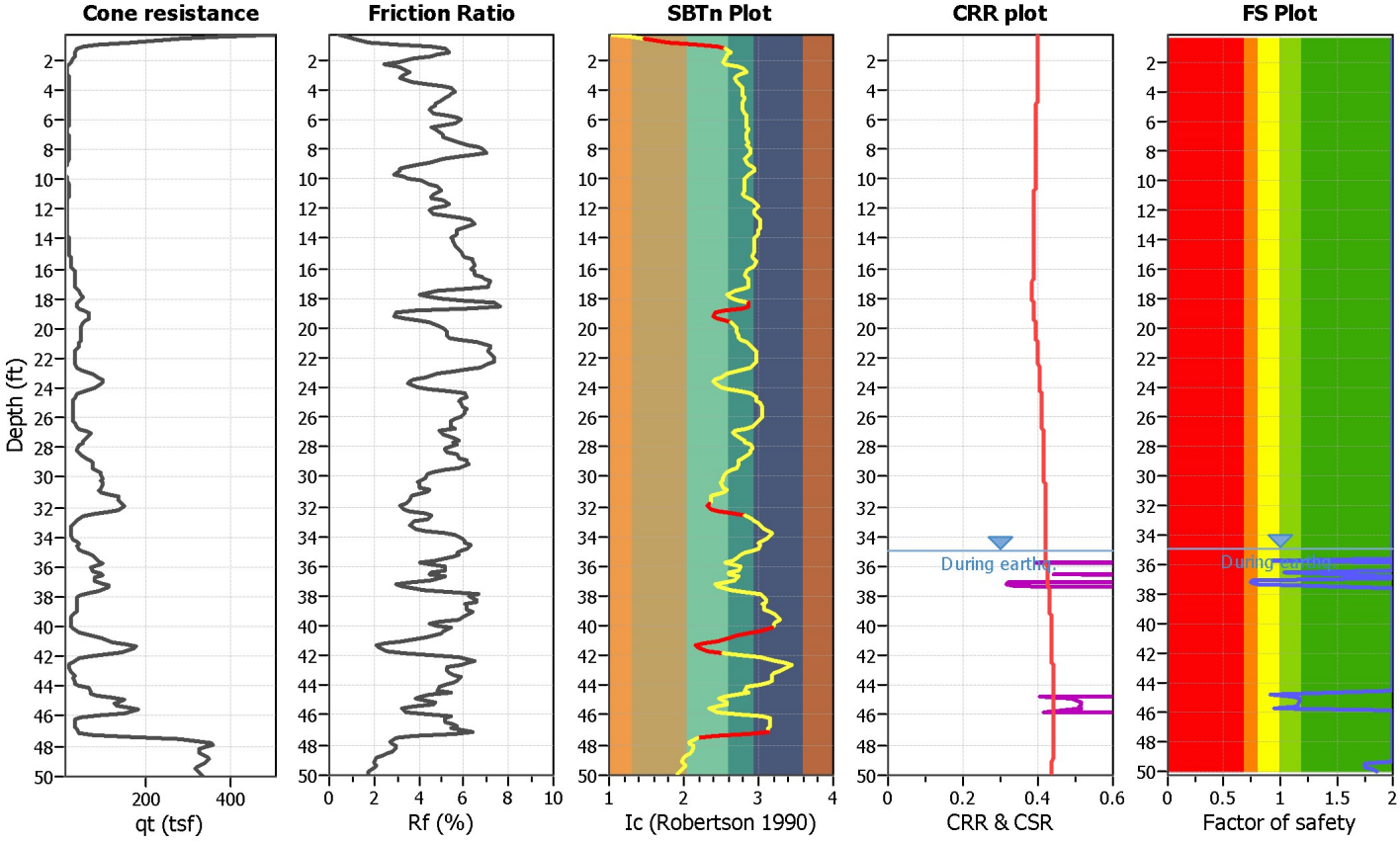
LIQUEFACTION ANALYSIS REPORT

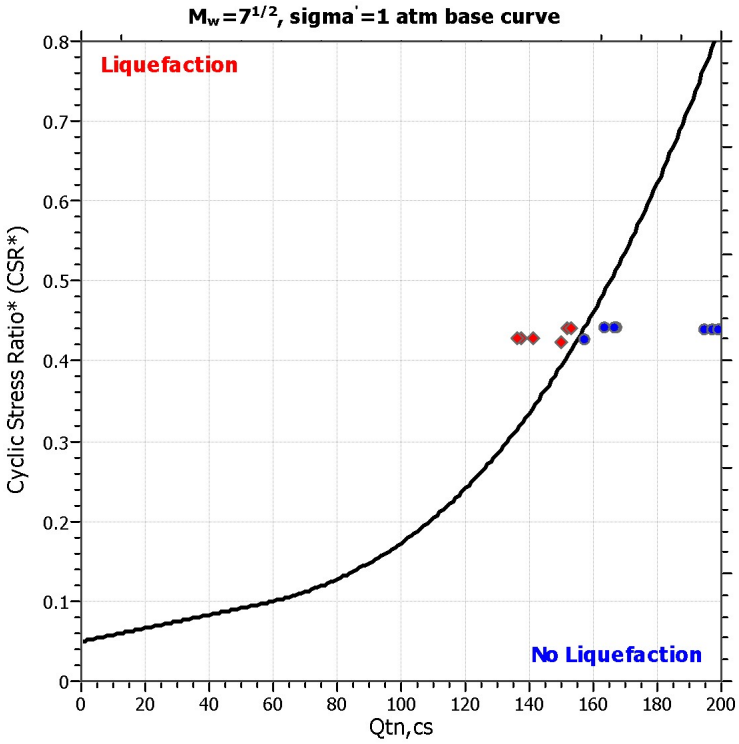
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

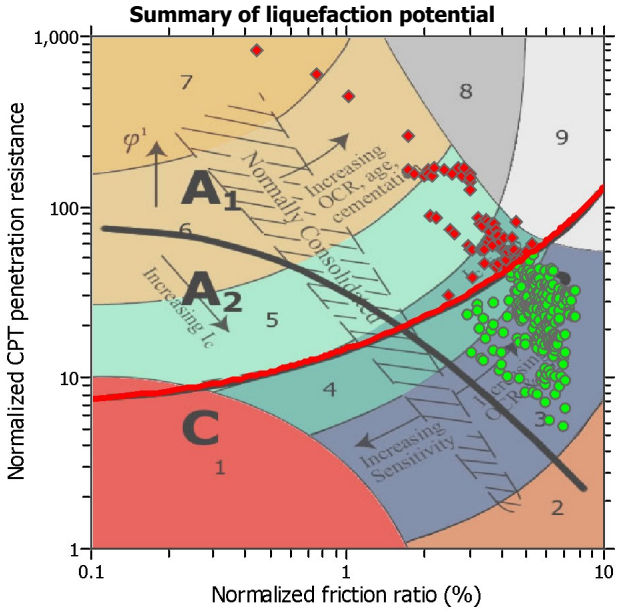
CPT file : 1-CPT4

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

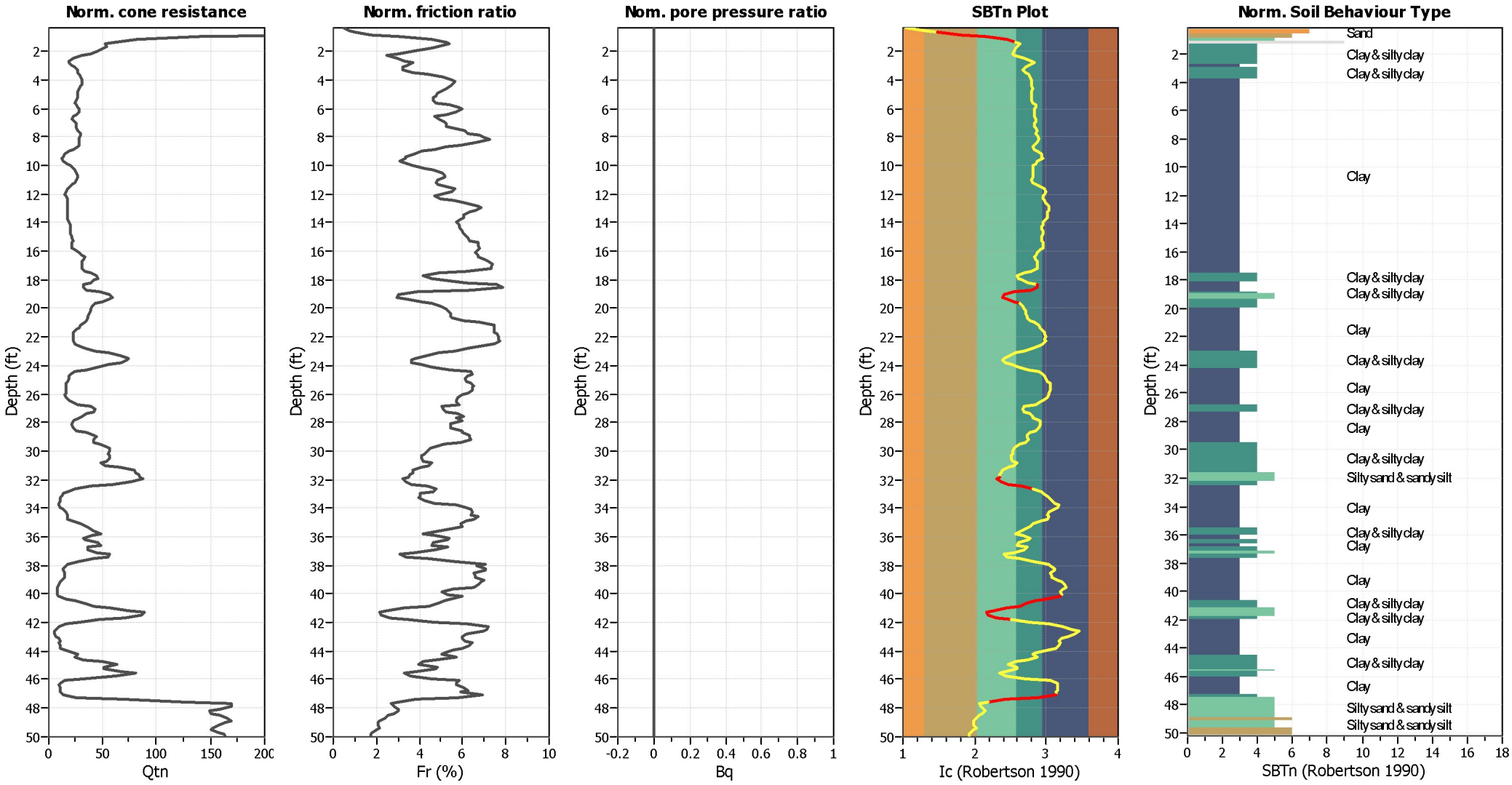






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



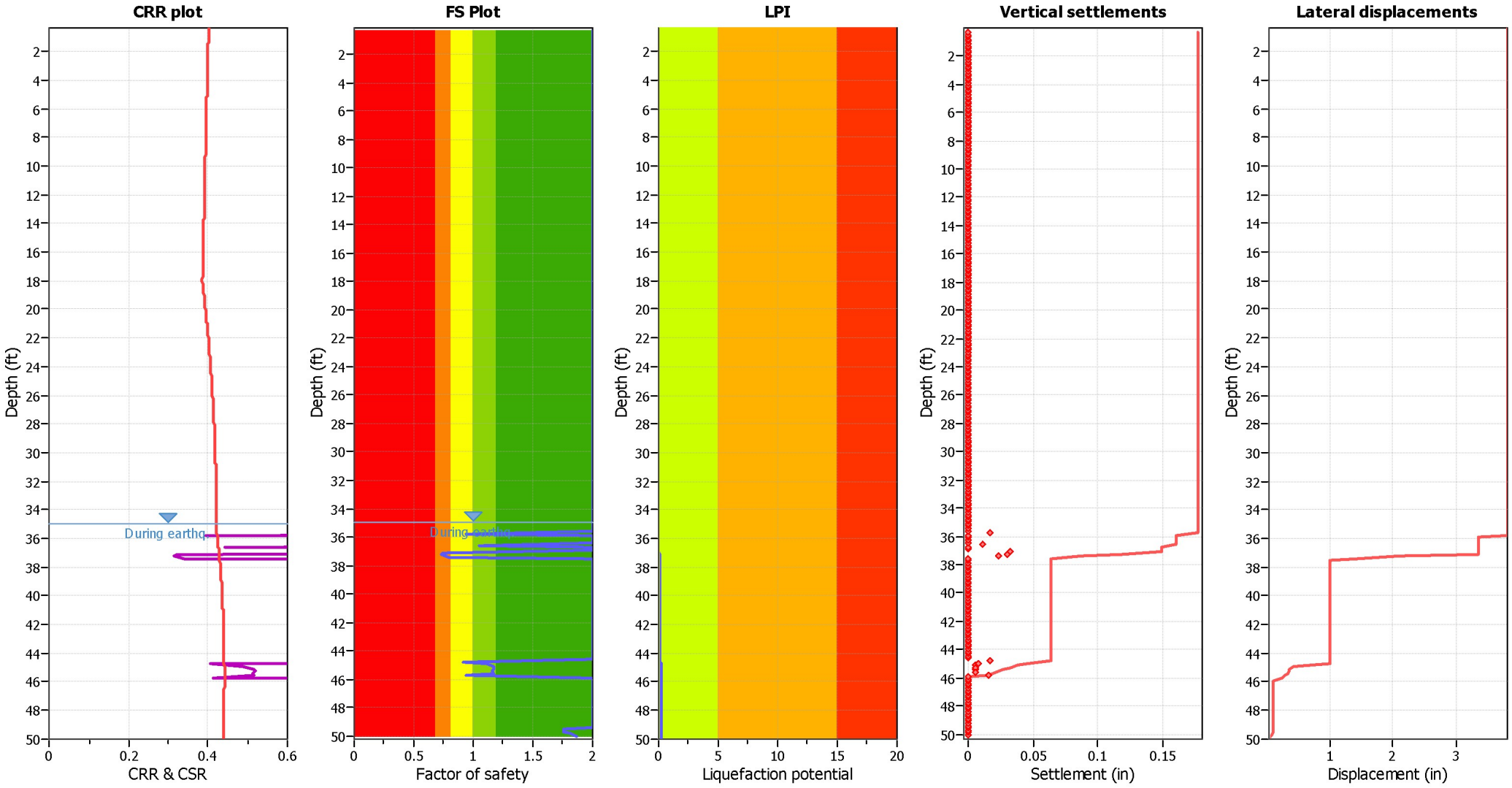
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

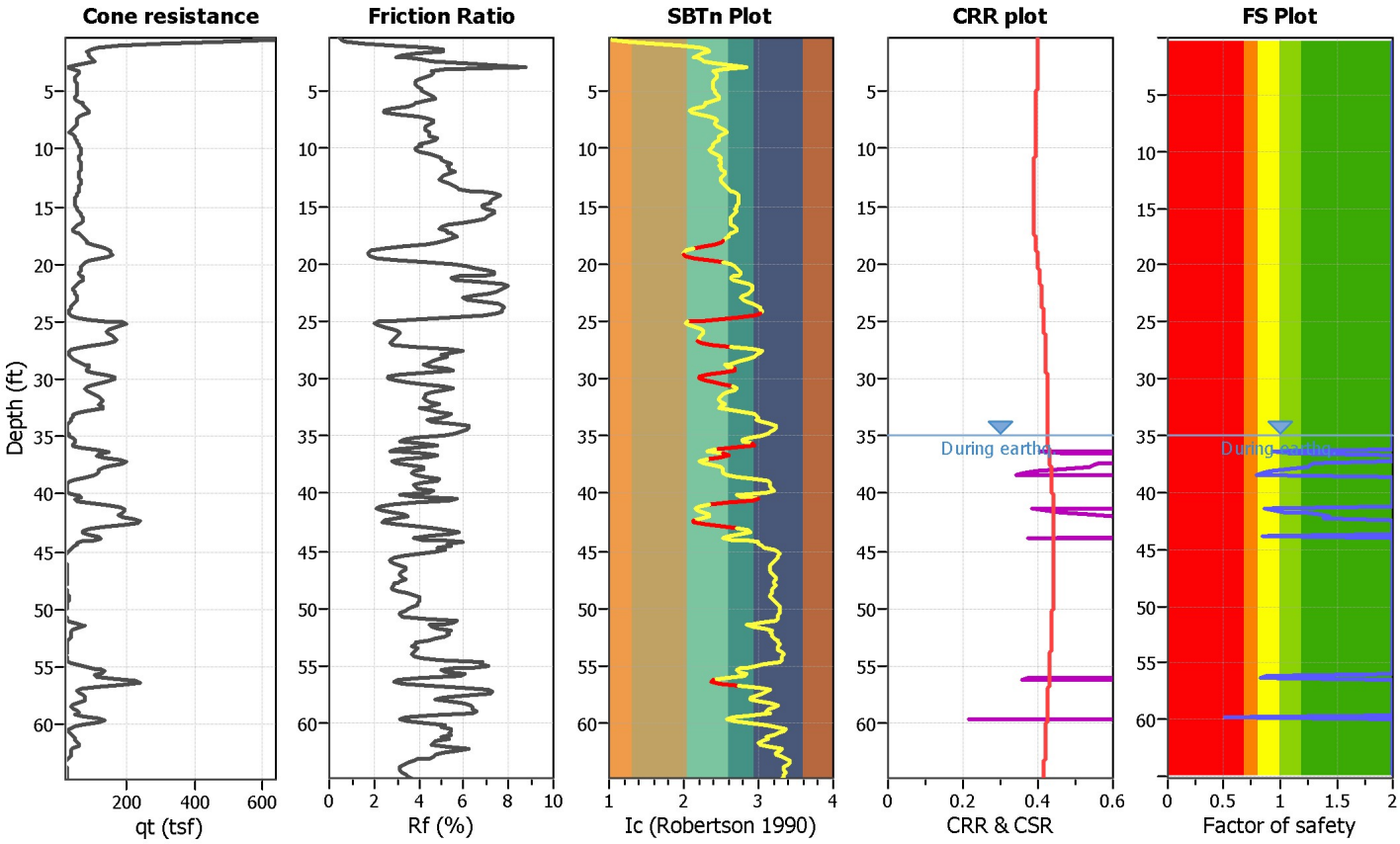
LIQUEFACTION ANALYSIS REPORT

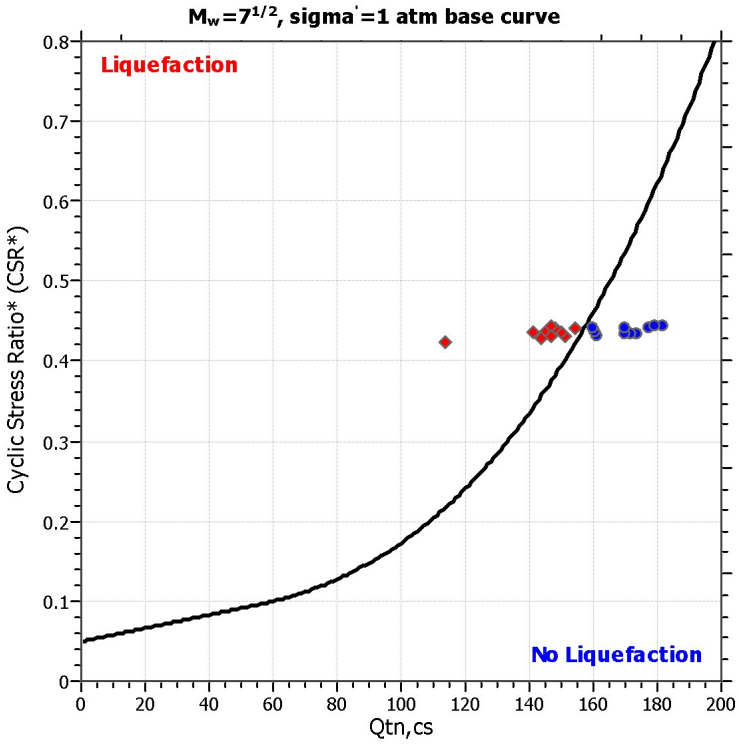
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

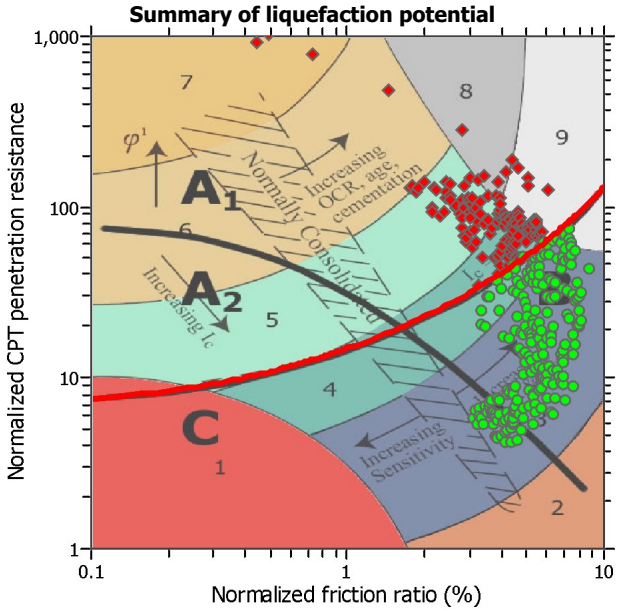
CPT file : 1-CPT5

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

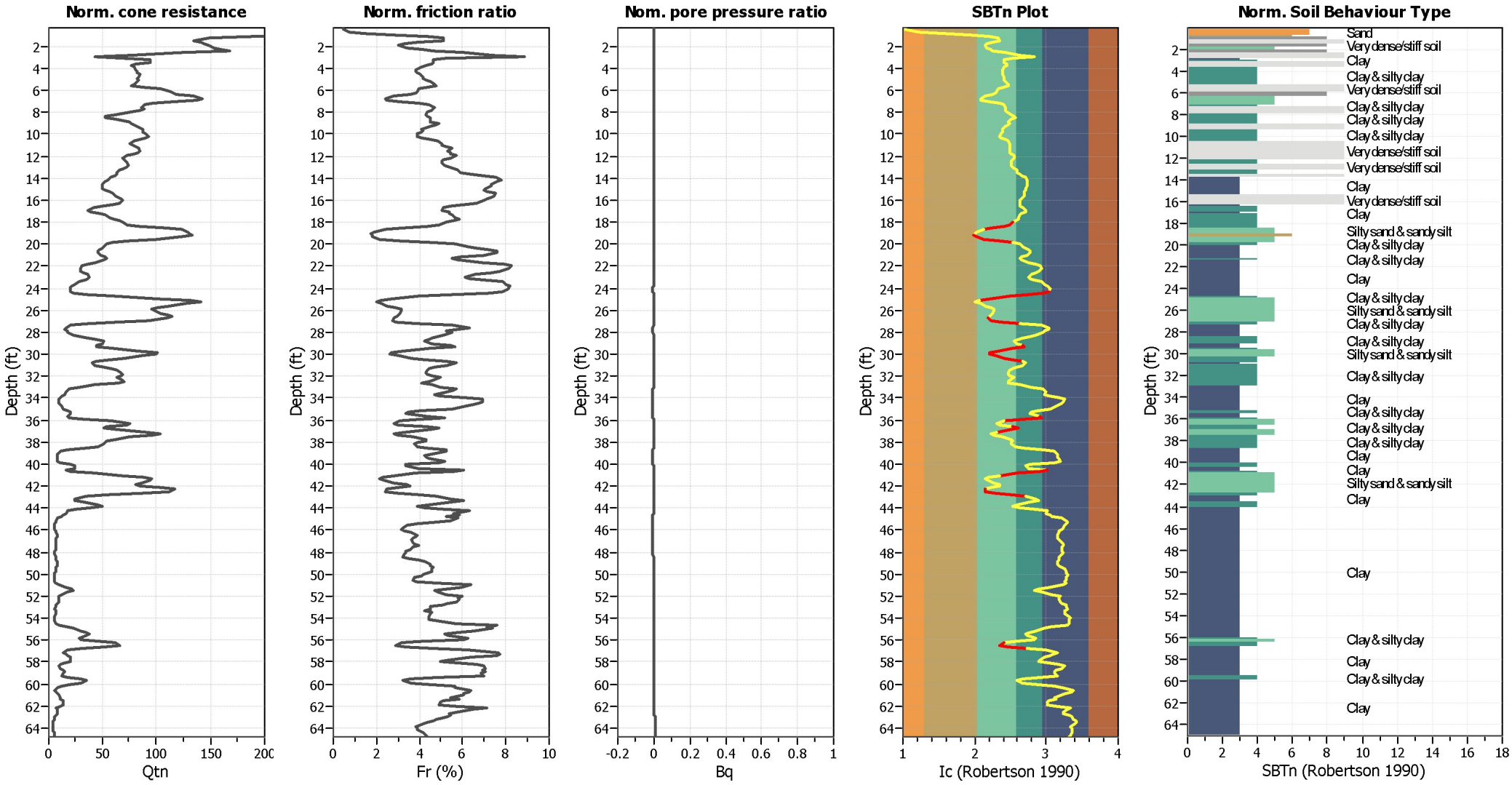






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



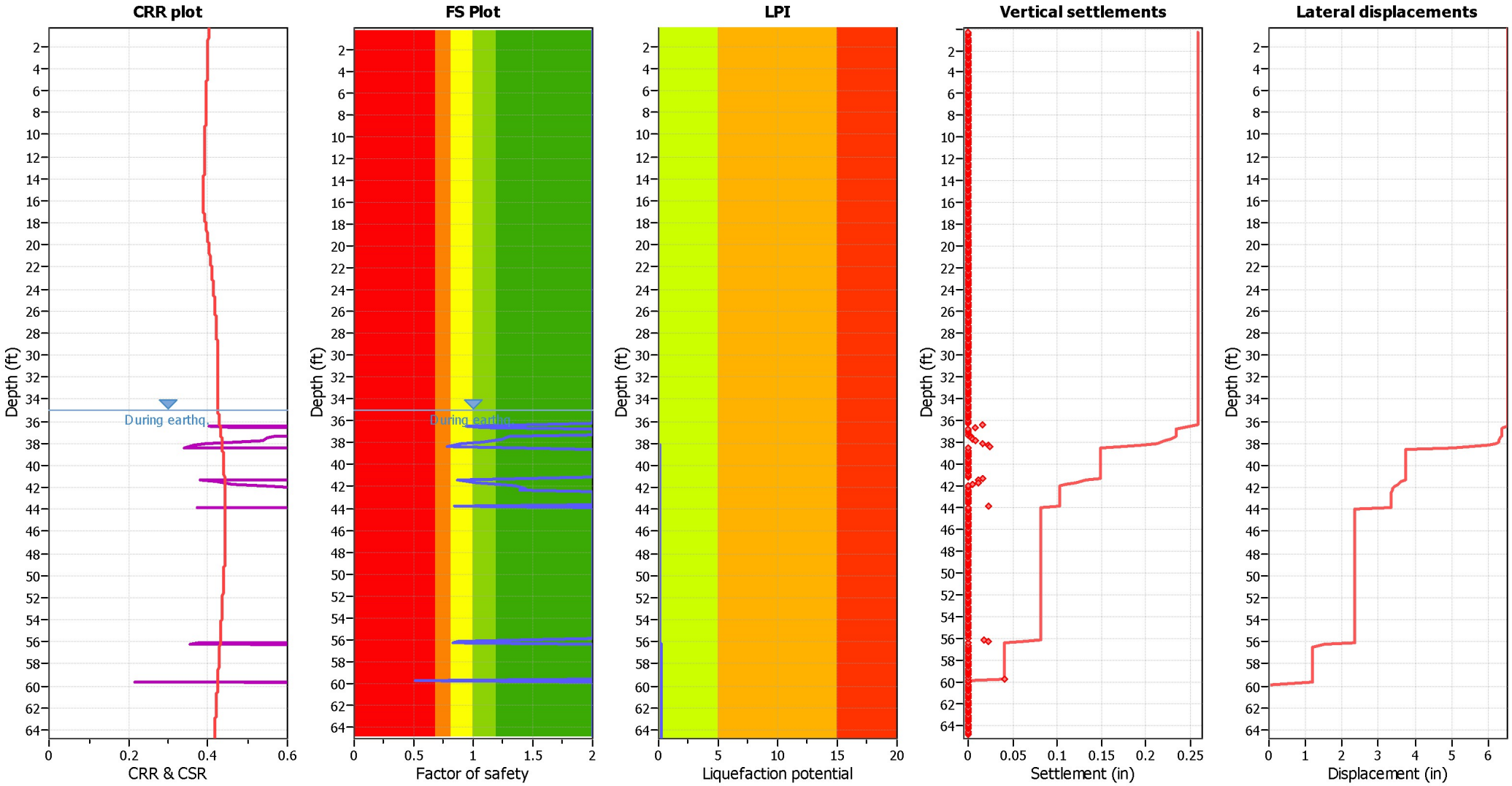
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

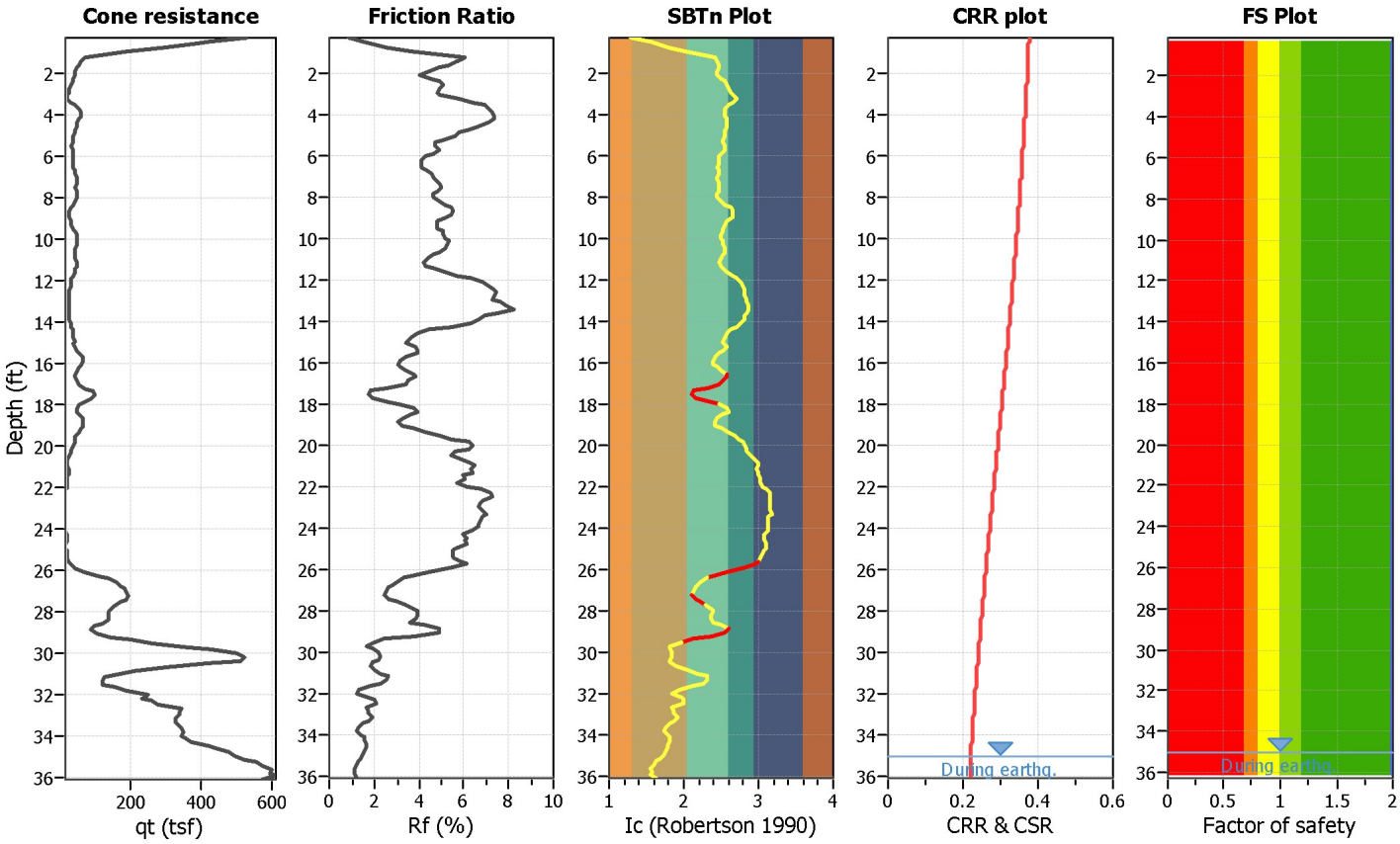
LIQUEFACTION ANALYSIS REPORT

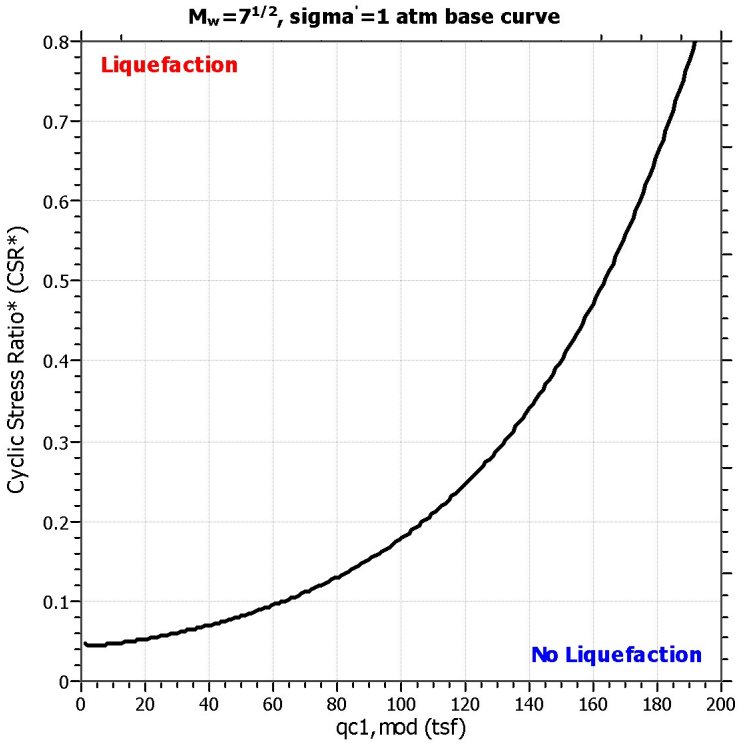
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

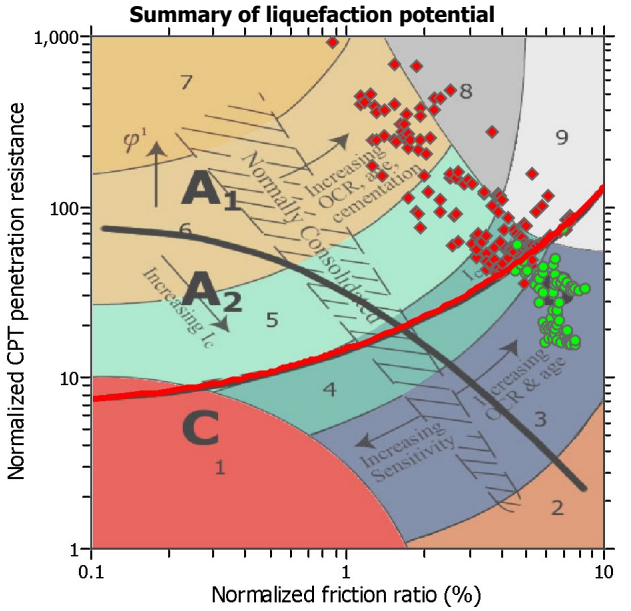
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	Moss et al. (2006)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	Moss et al. (2006)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

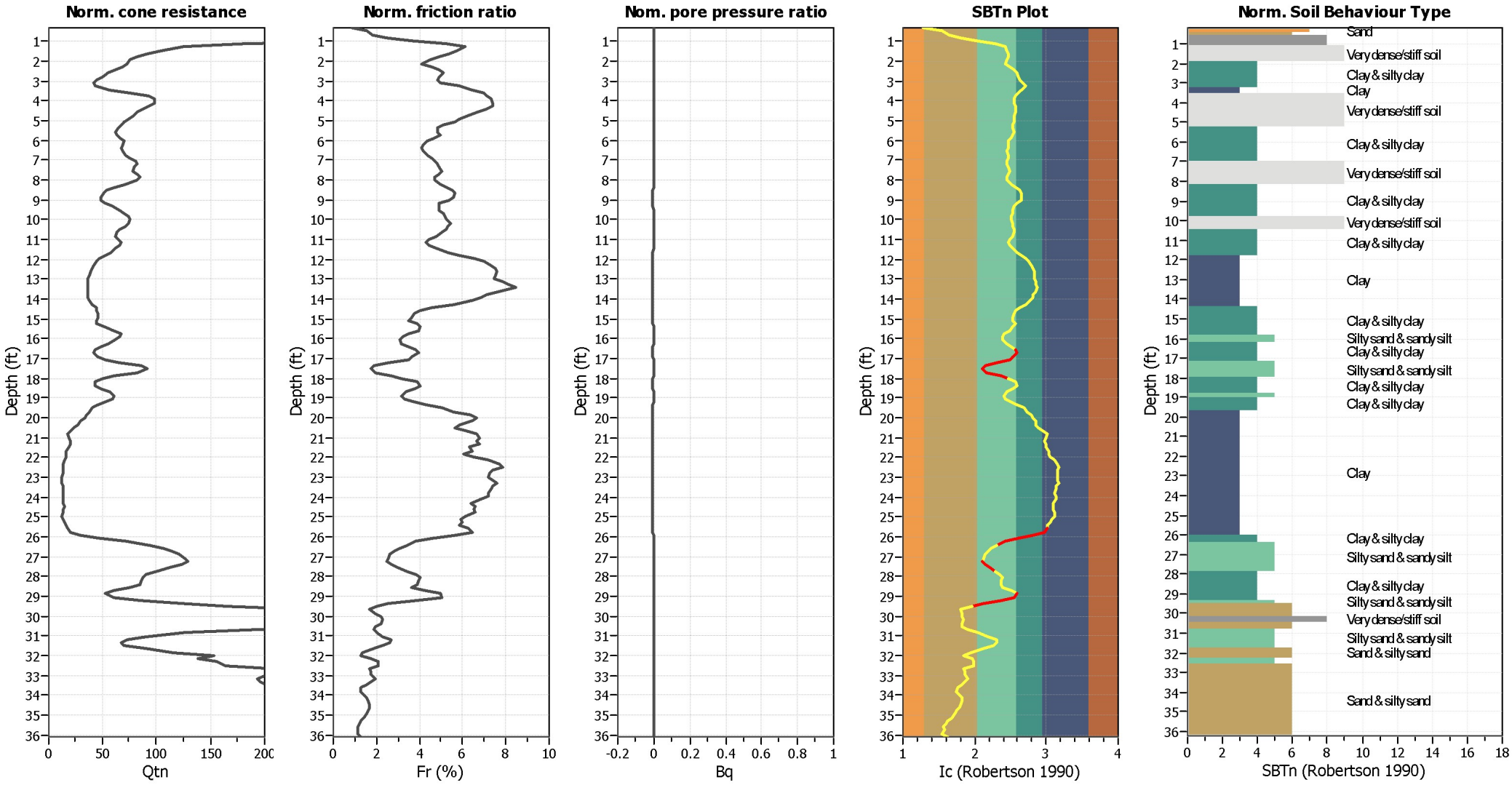






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



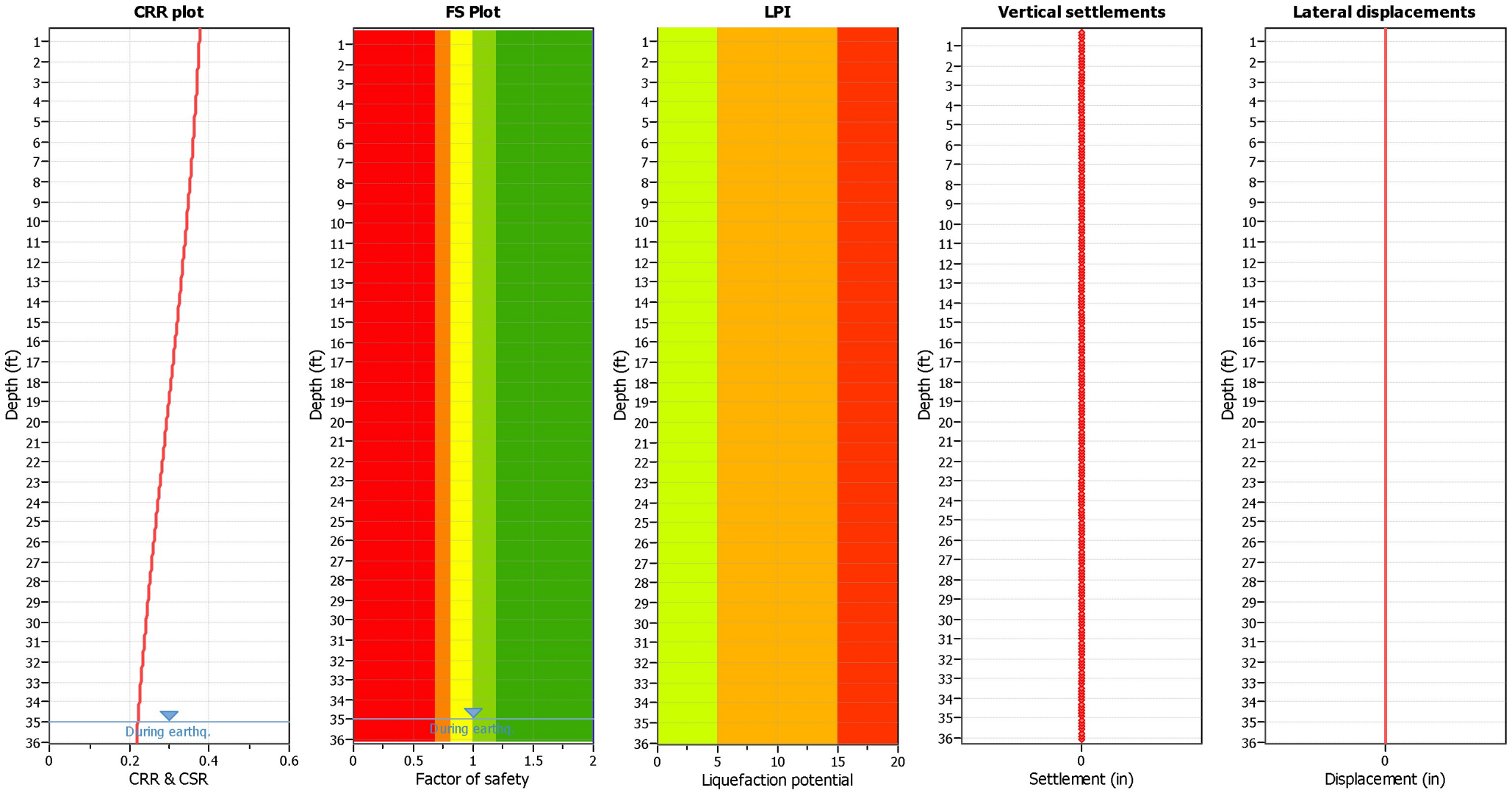
Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

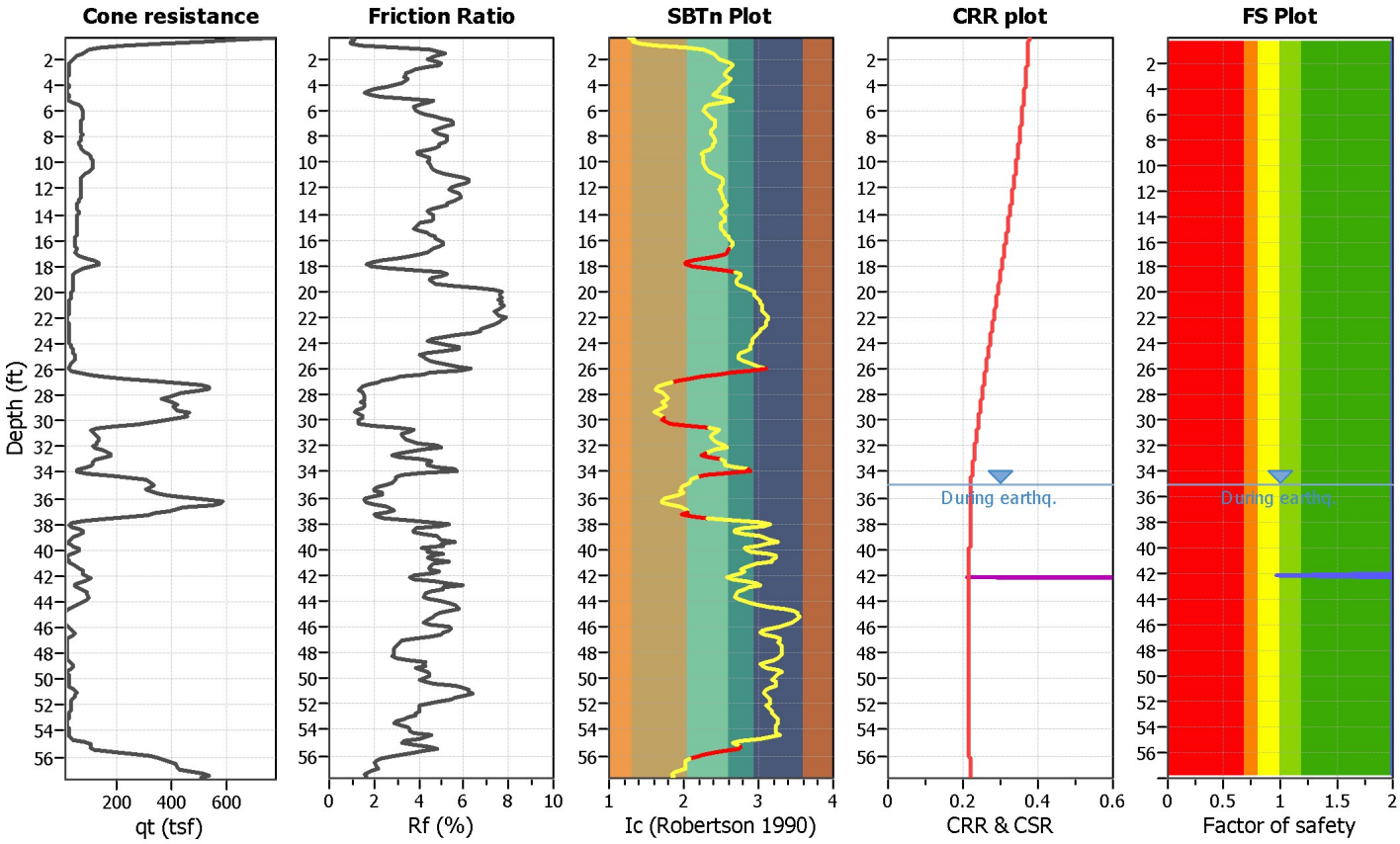
LIQUEFACTION ANALYSIS REPORT

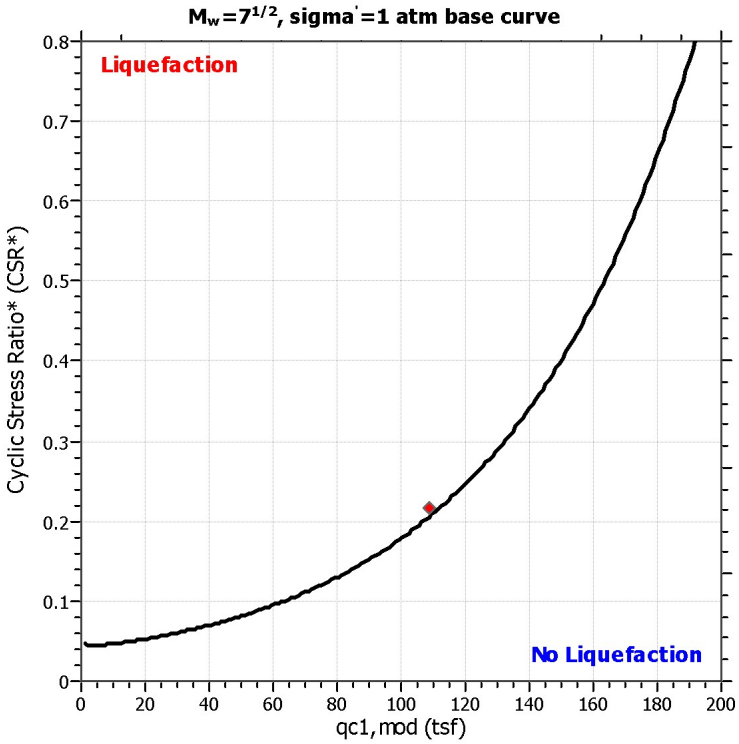
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

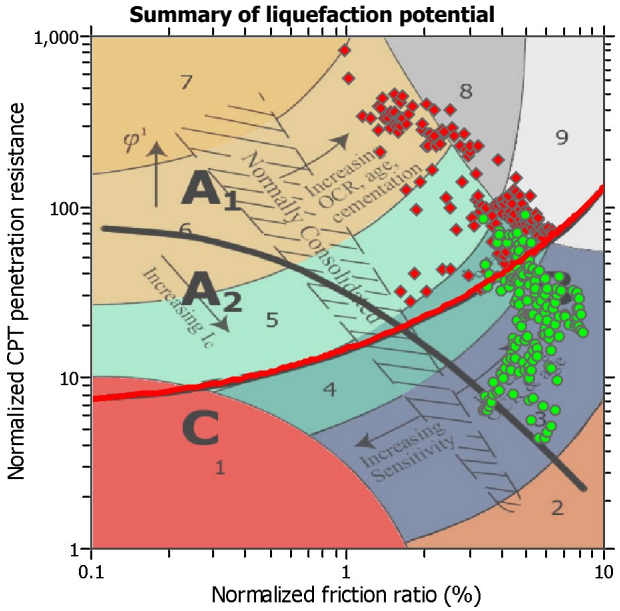
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	Moss et al. (2006)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	Moss et al. (2006)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

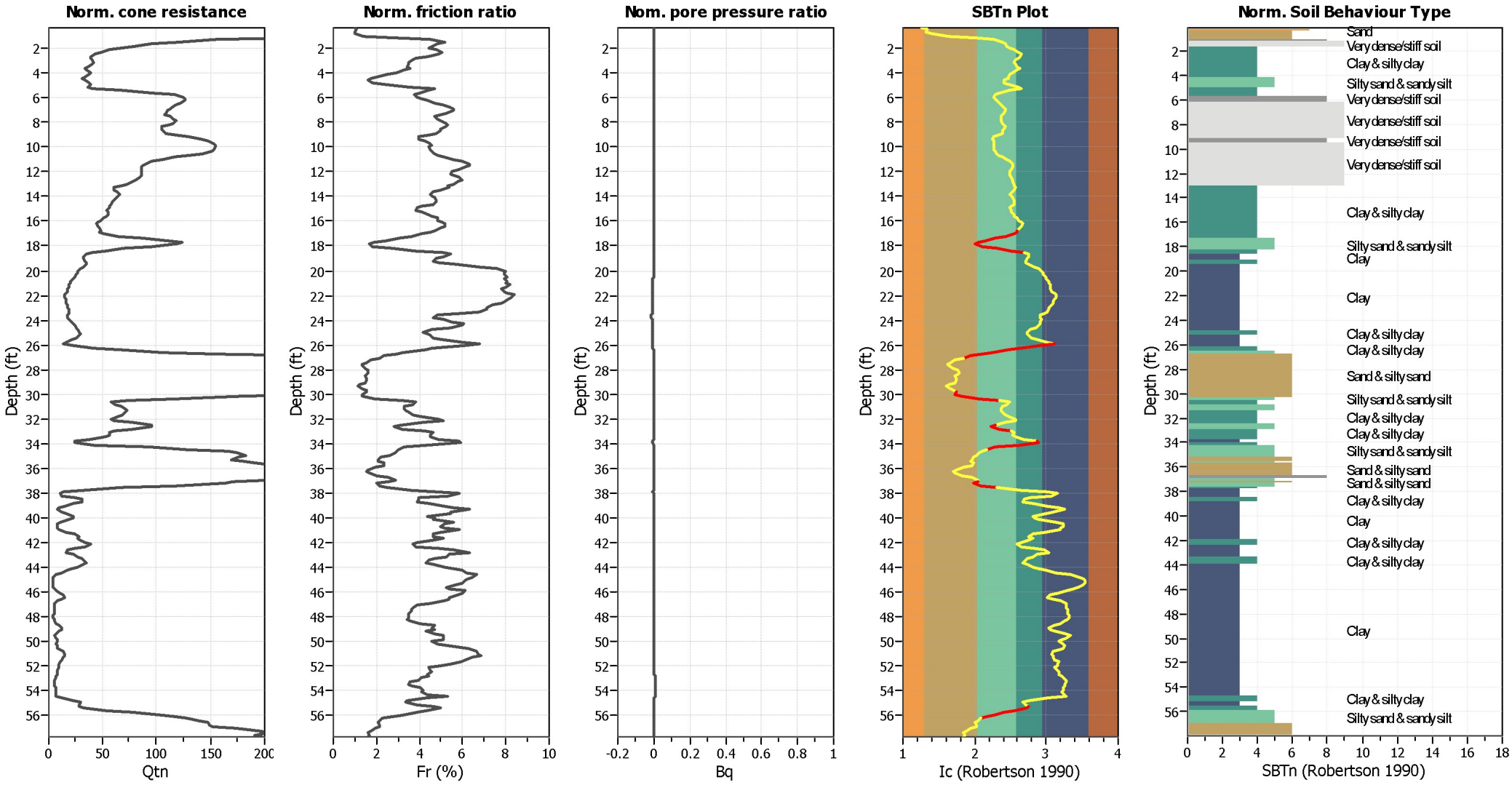






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



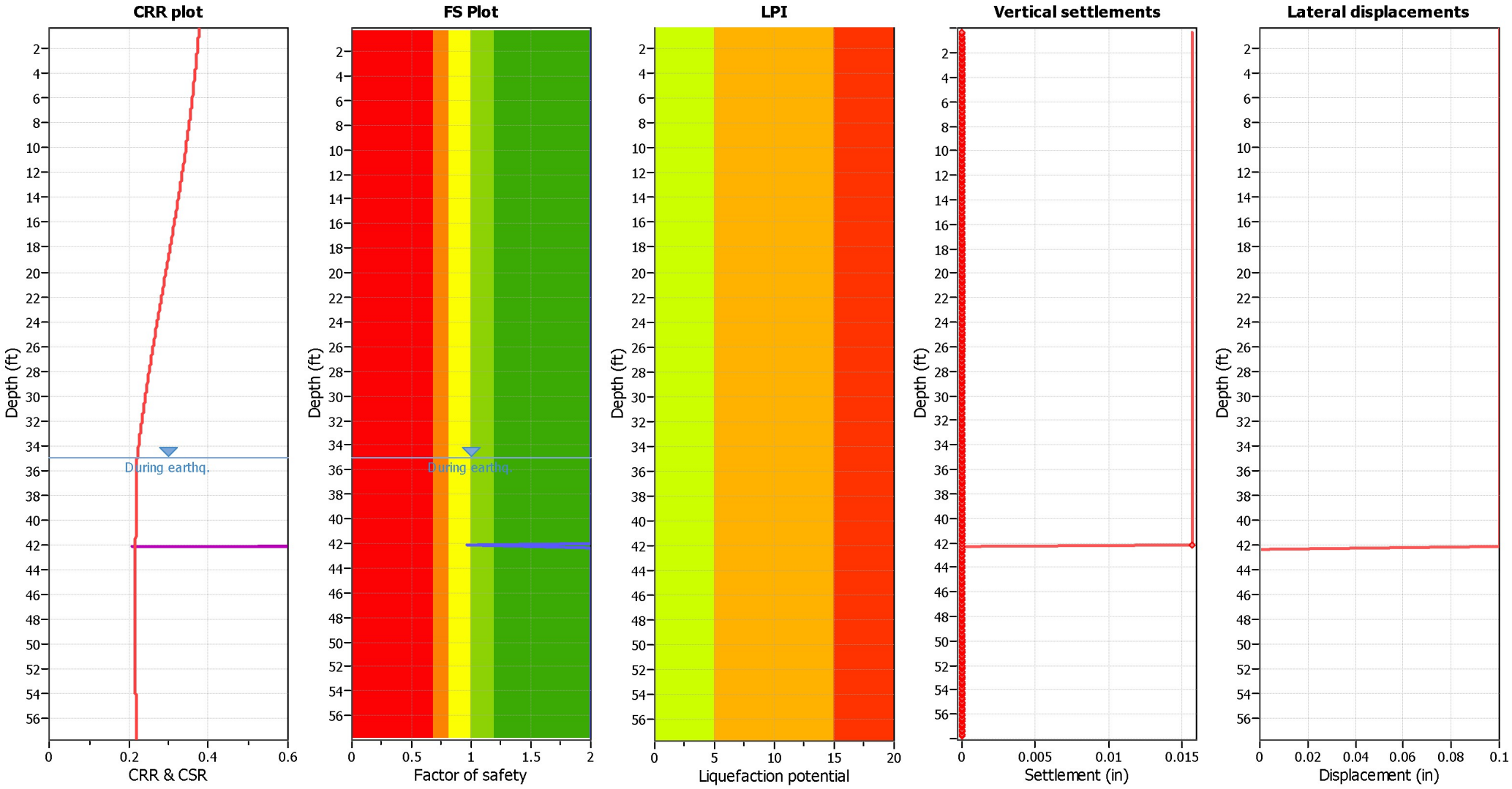
Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

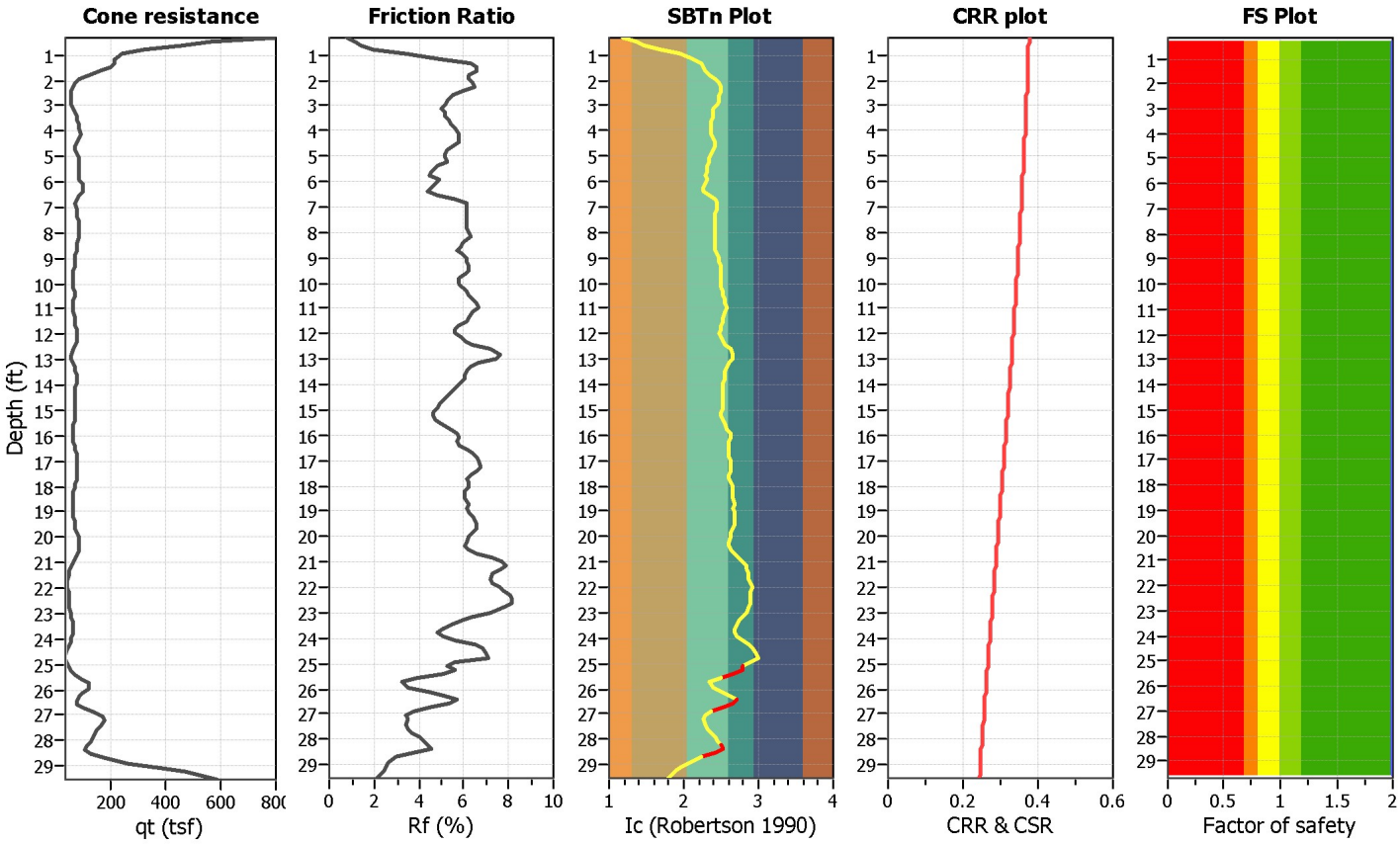
LIQUEFACTION ANALYSIS REPORT

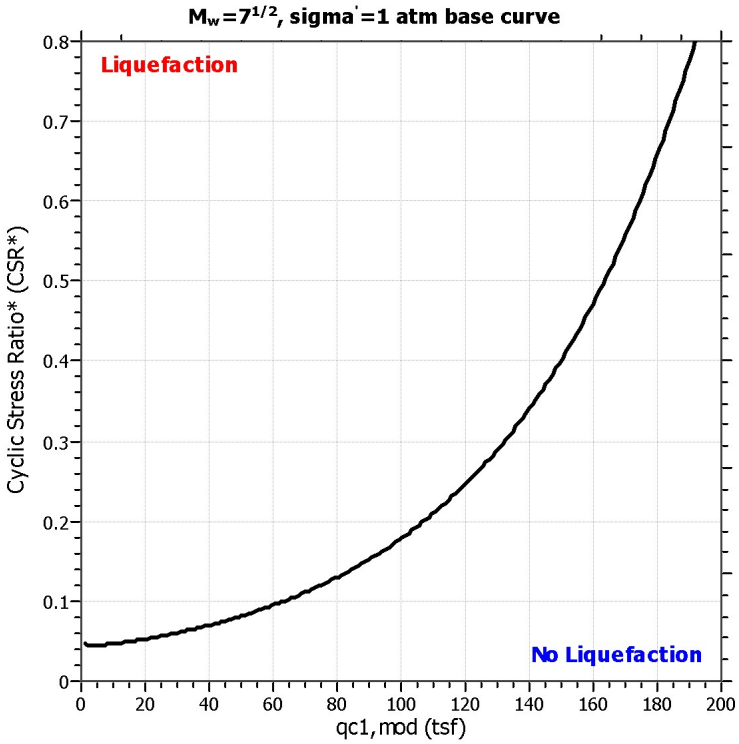
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

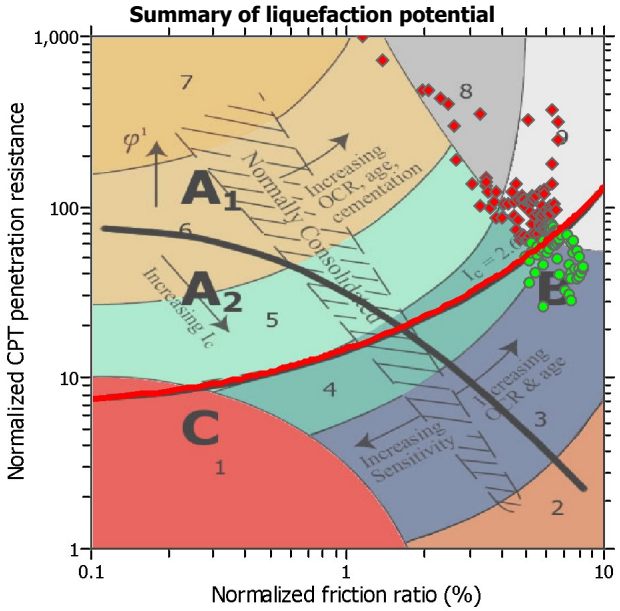
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	Moss et al. (2006)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	Moss et al. (2006)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

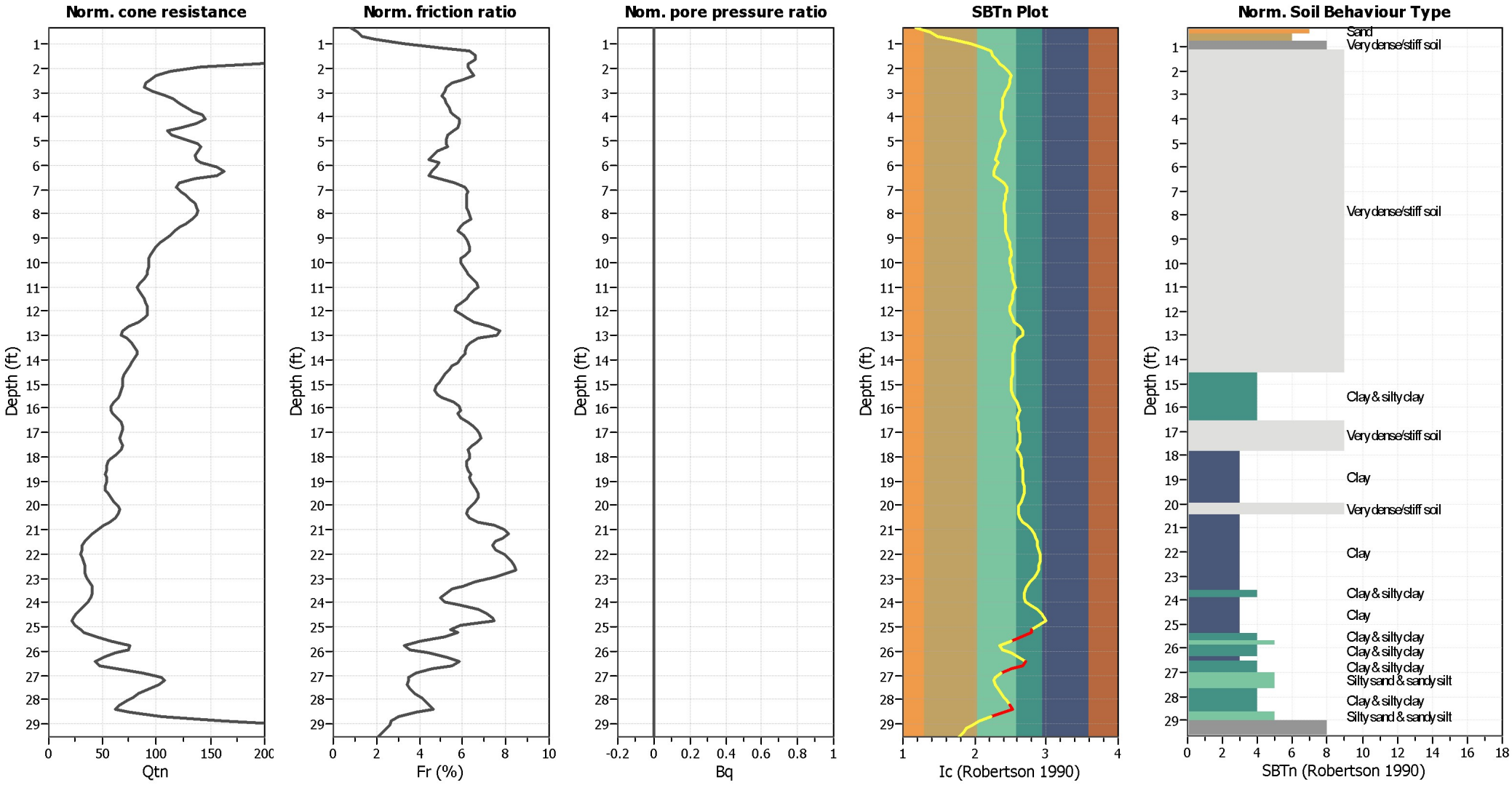






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



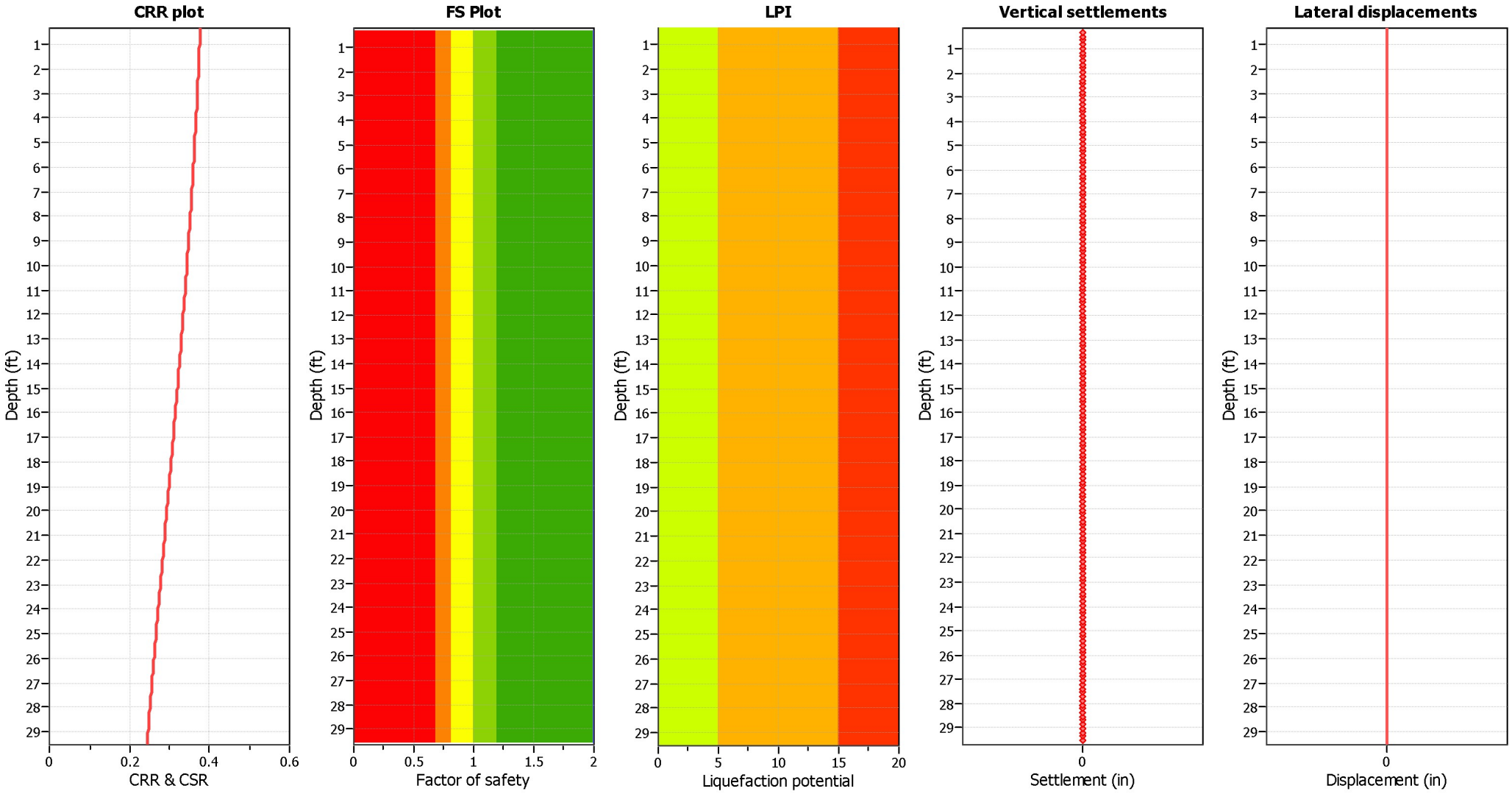
Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

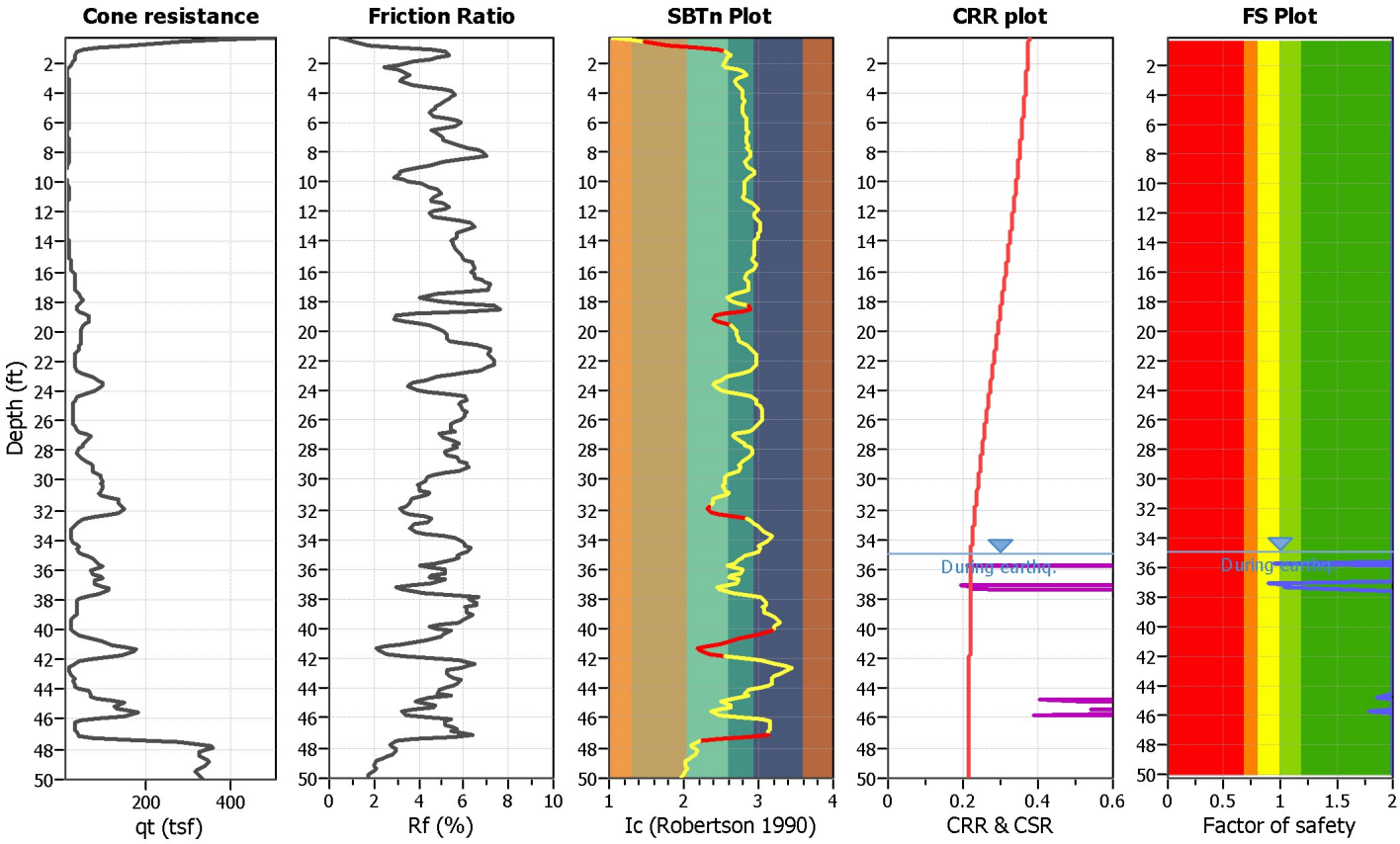
LIQUEFACTION ANALYSIS REPORT

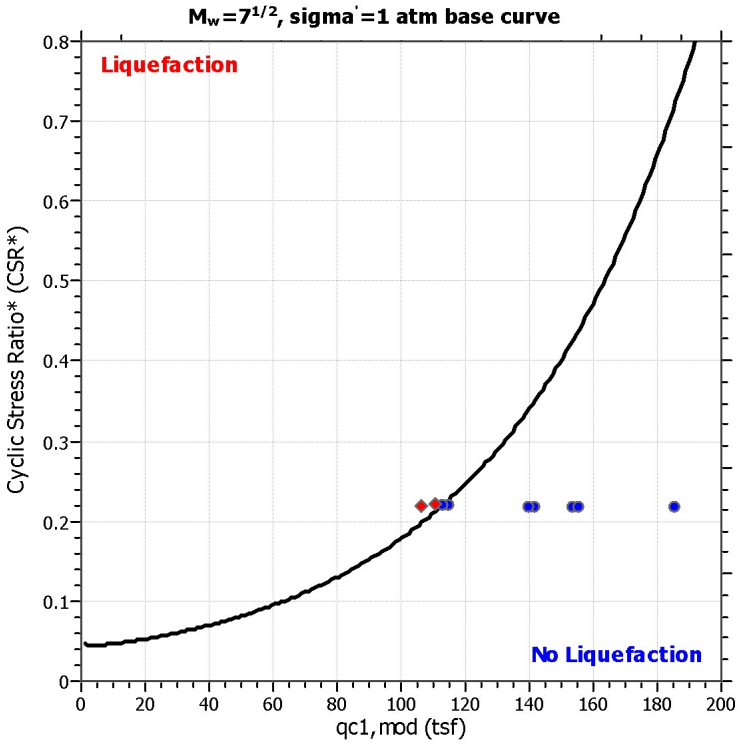
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

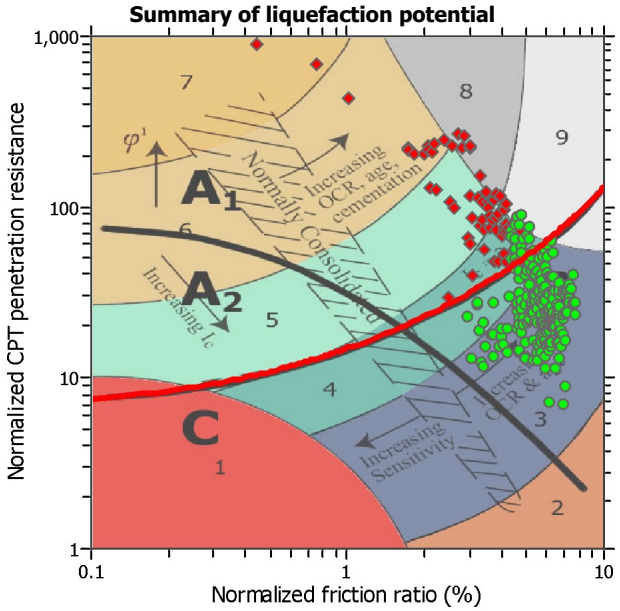
CPT file : 1-CPT4

Input parameters and analysis data

Analysis method:	Moss et al. (2006)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	Moss et al. (2006)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

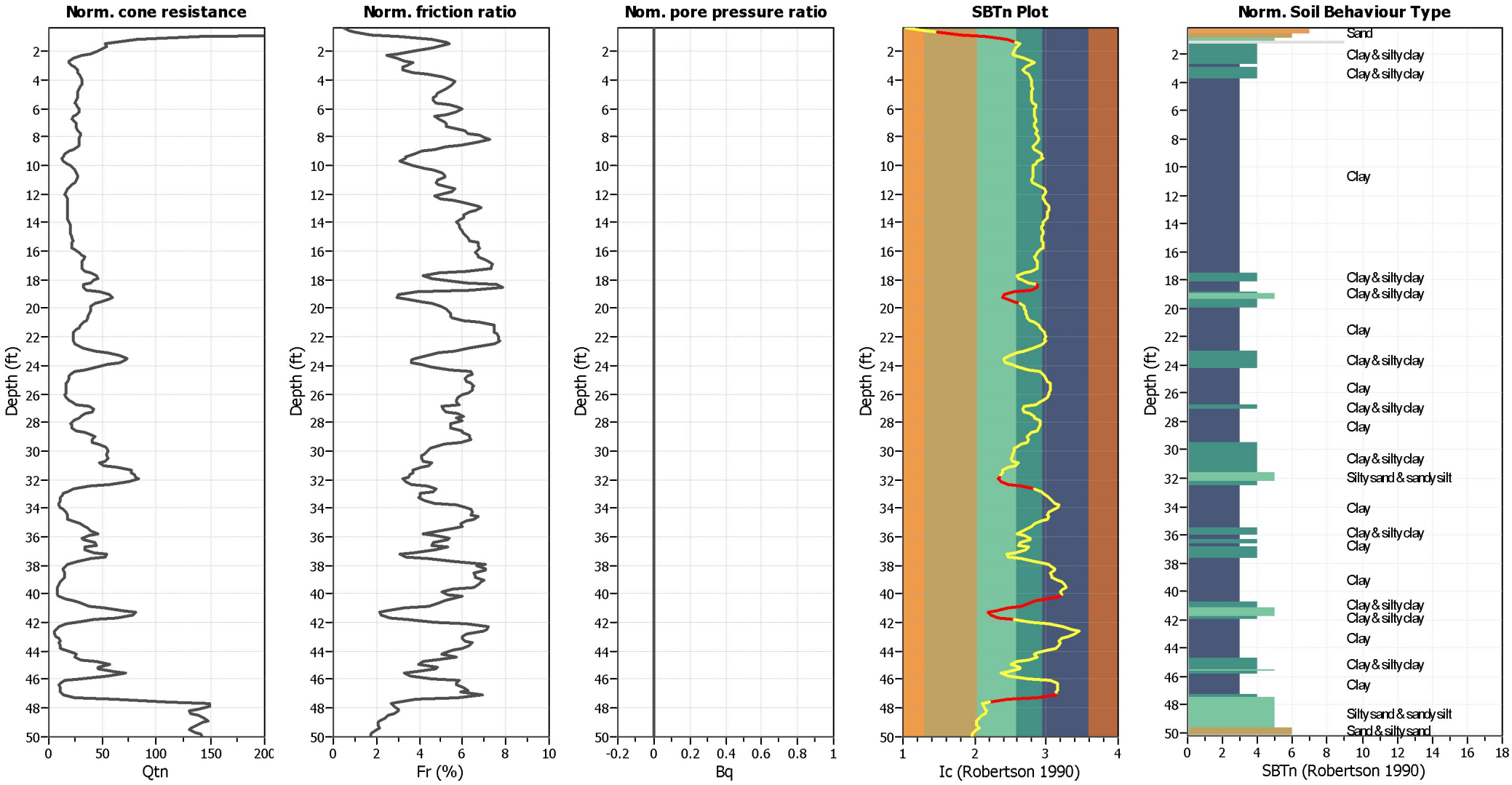






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



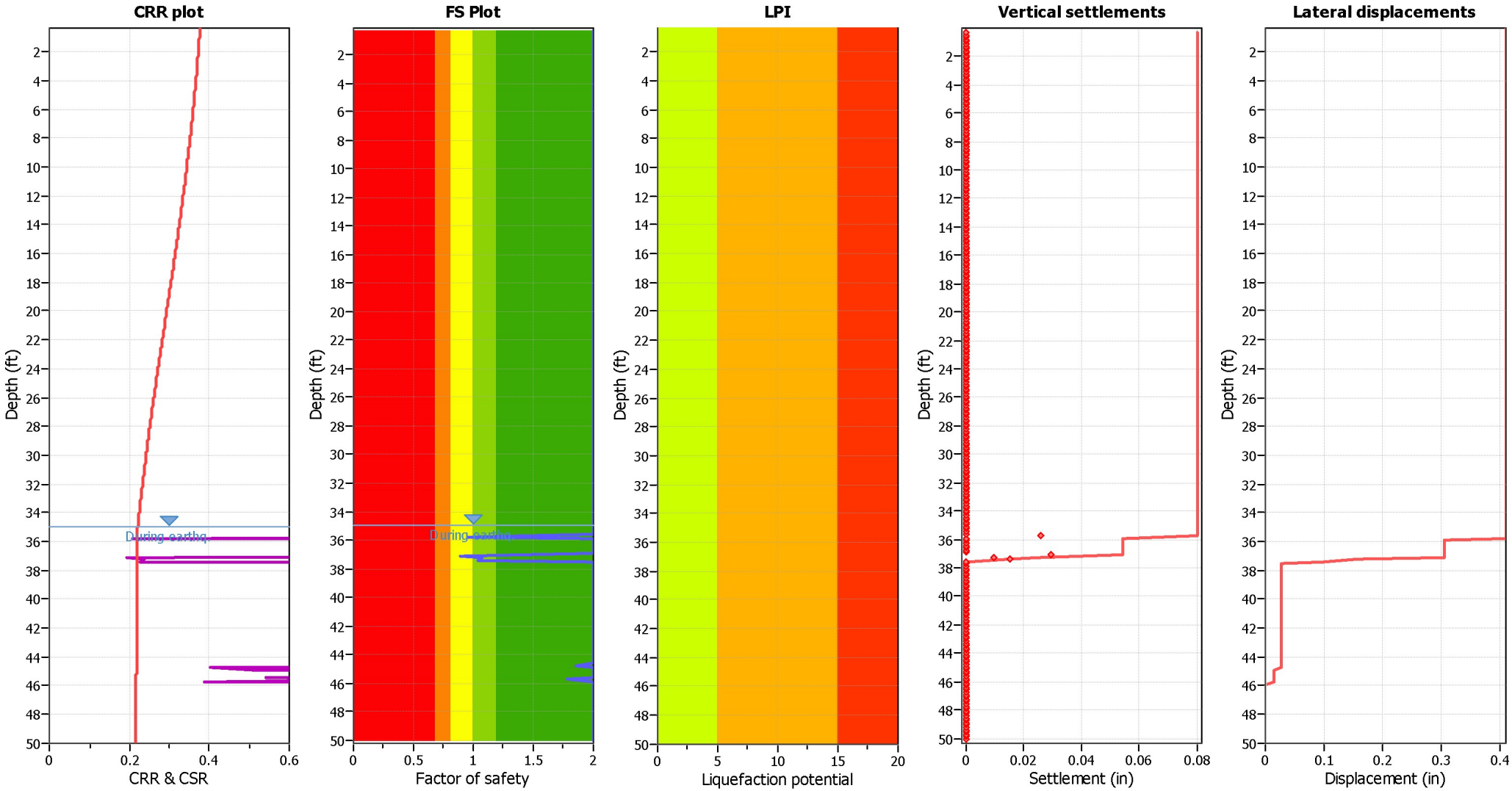
Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

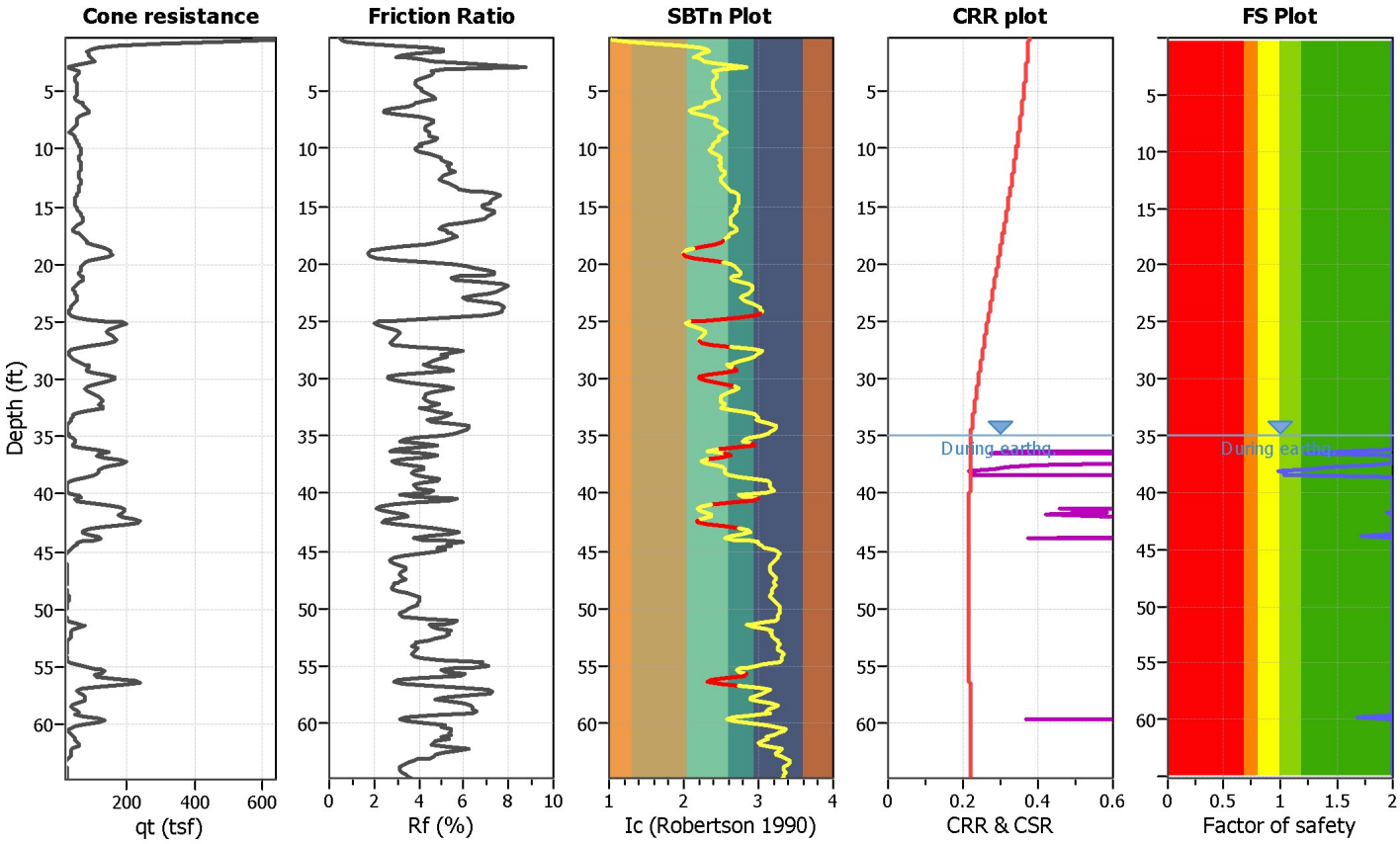
LIQUEFACTION ANALYSIS REPORT

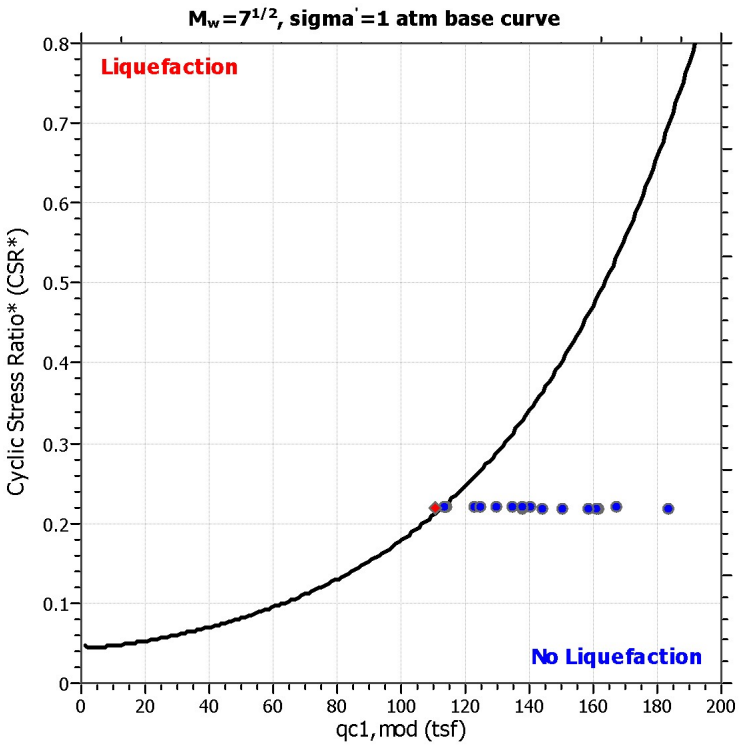
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

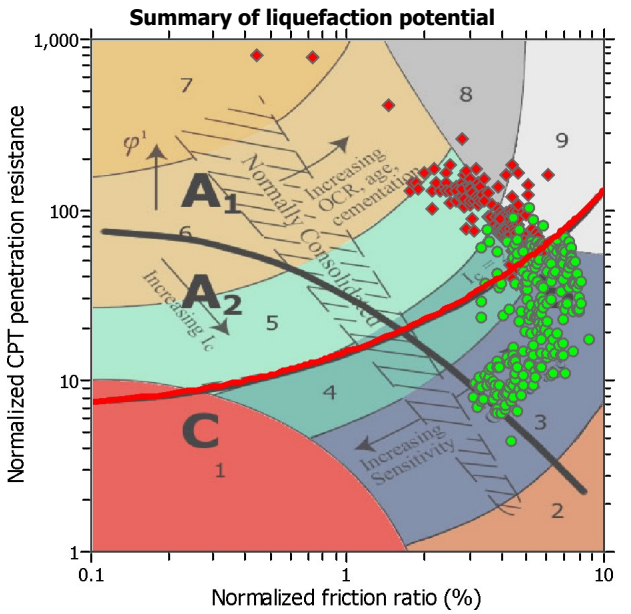
CPT file : 1-CPT5

Input parameters and analysis data

Analysis method:	Moss et al. (2006)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	Moss et al. (2006)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

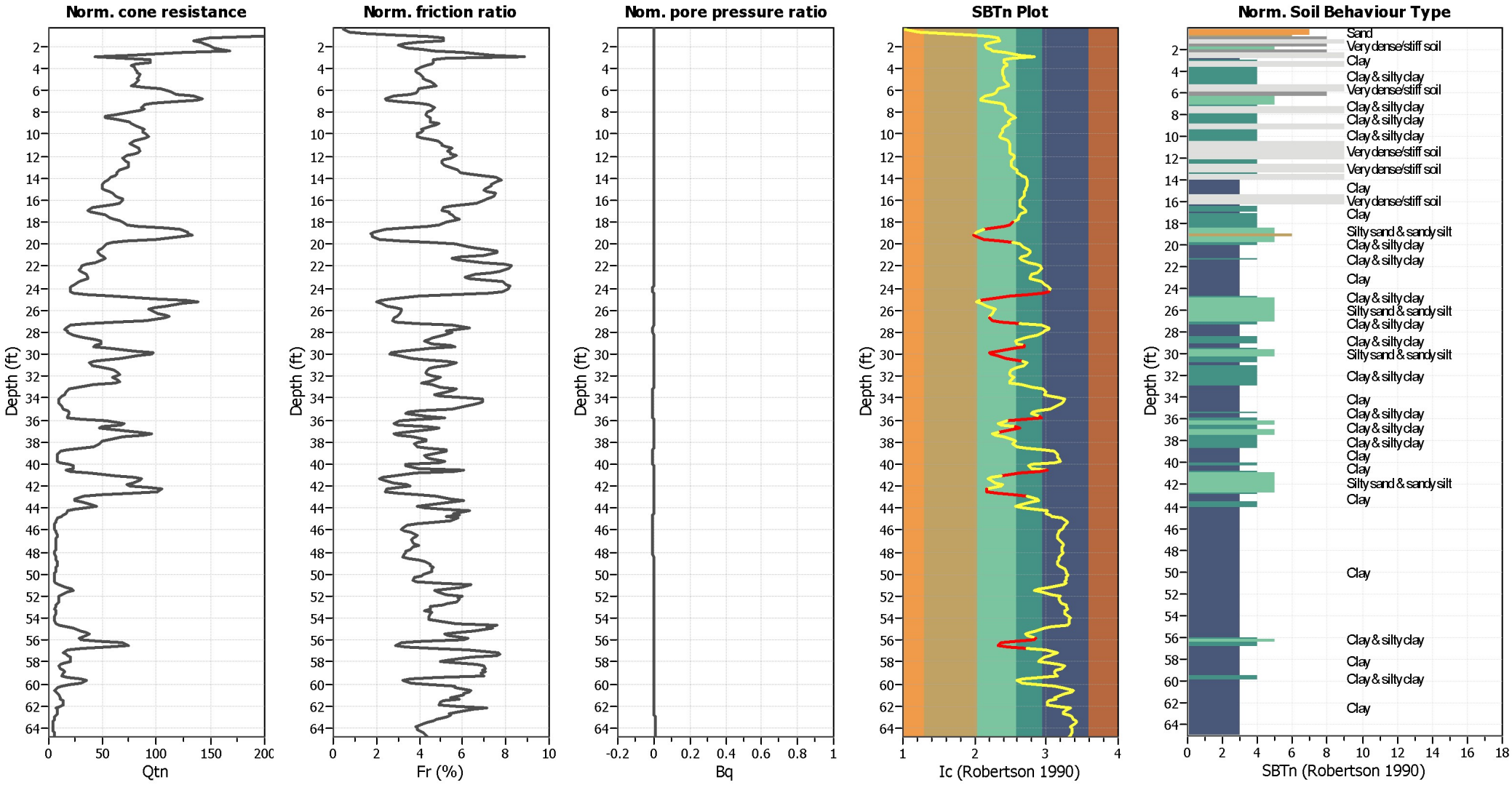






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



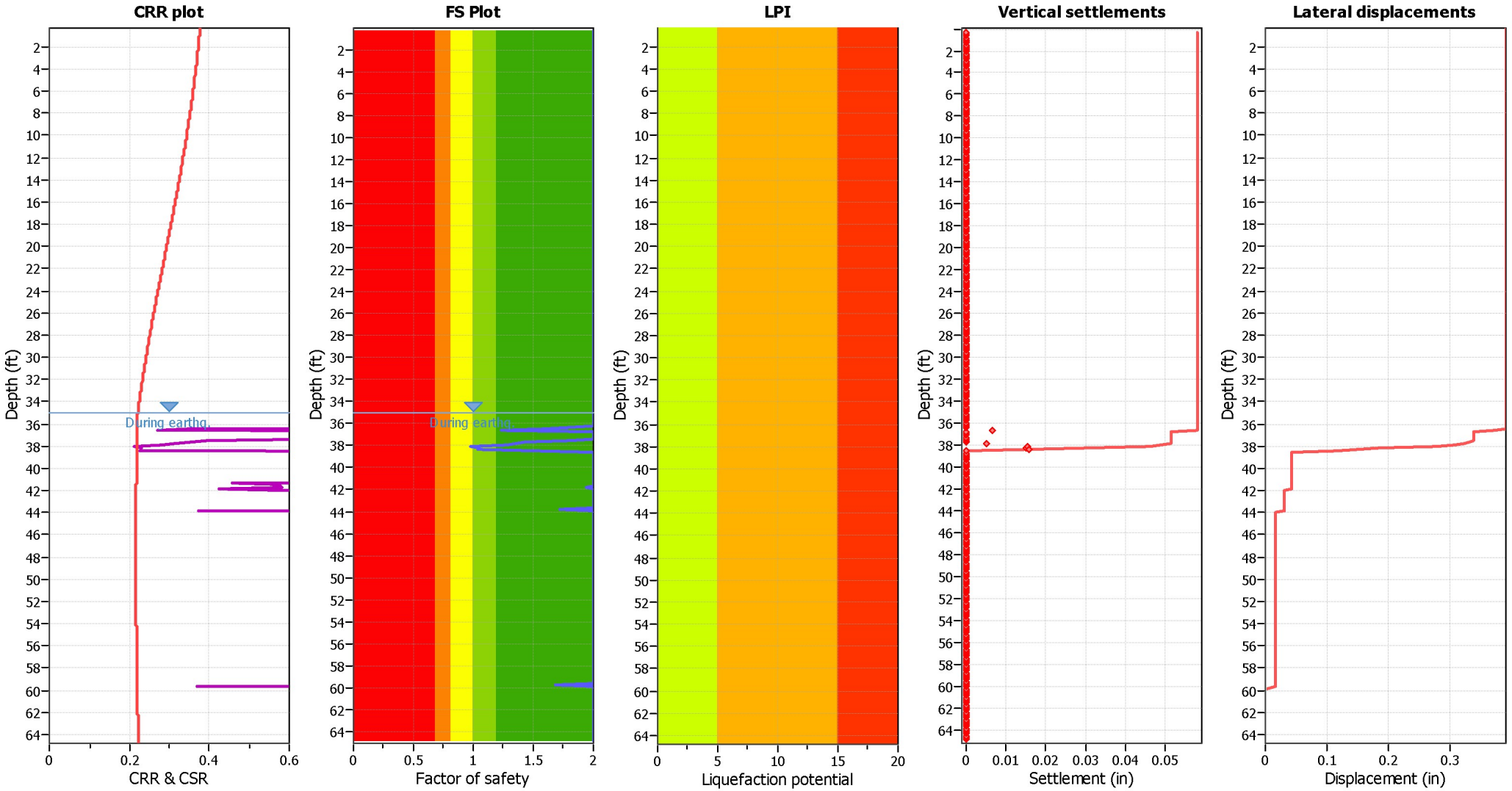
Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	Moss et al. (2006)	Depth to water table (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	Moss et al. (2006)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

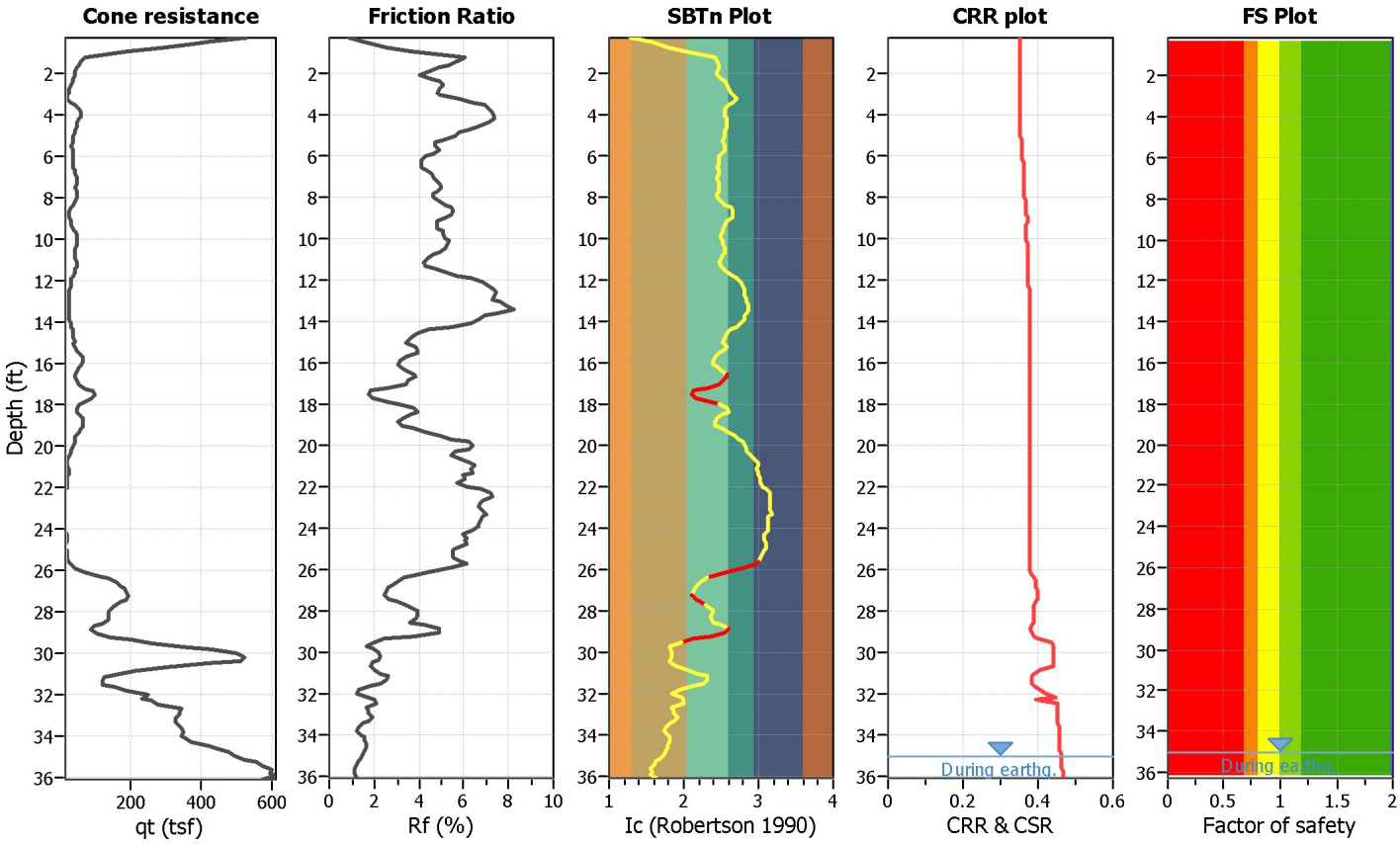
LIQUEFACTION ANALYSIS REPORT

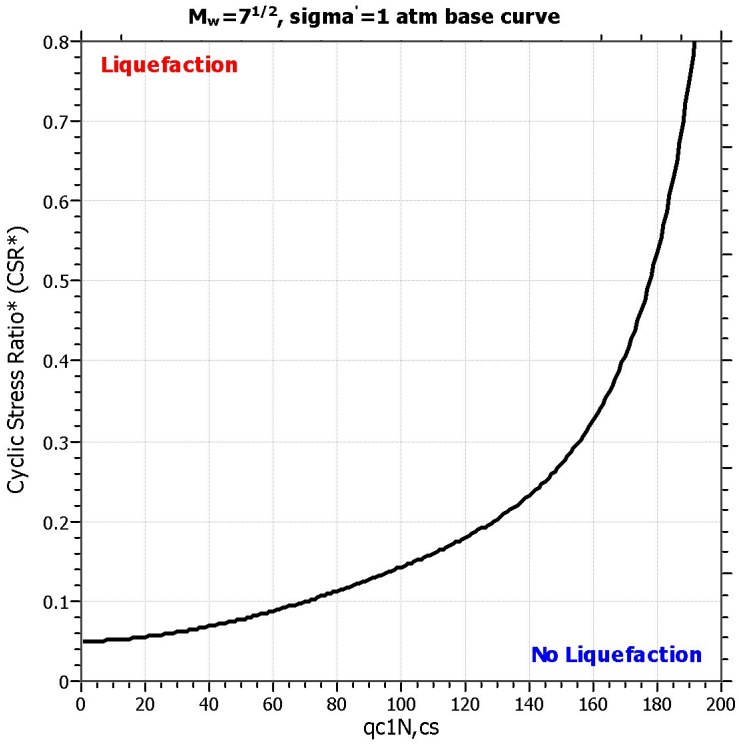
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

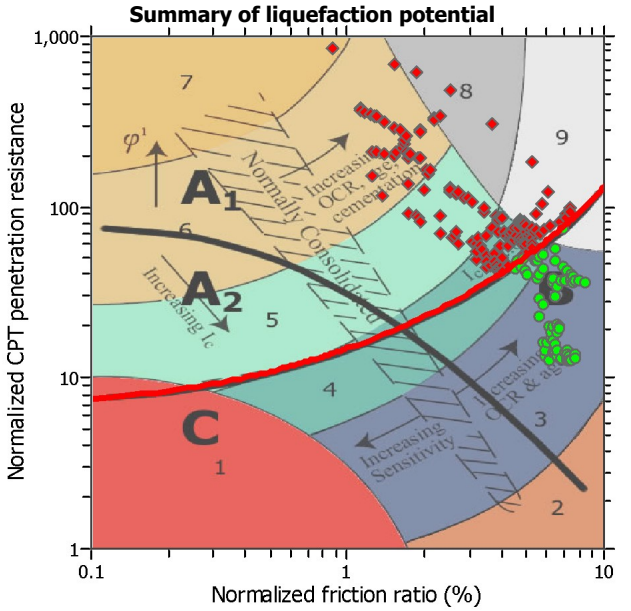
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

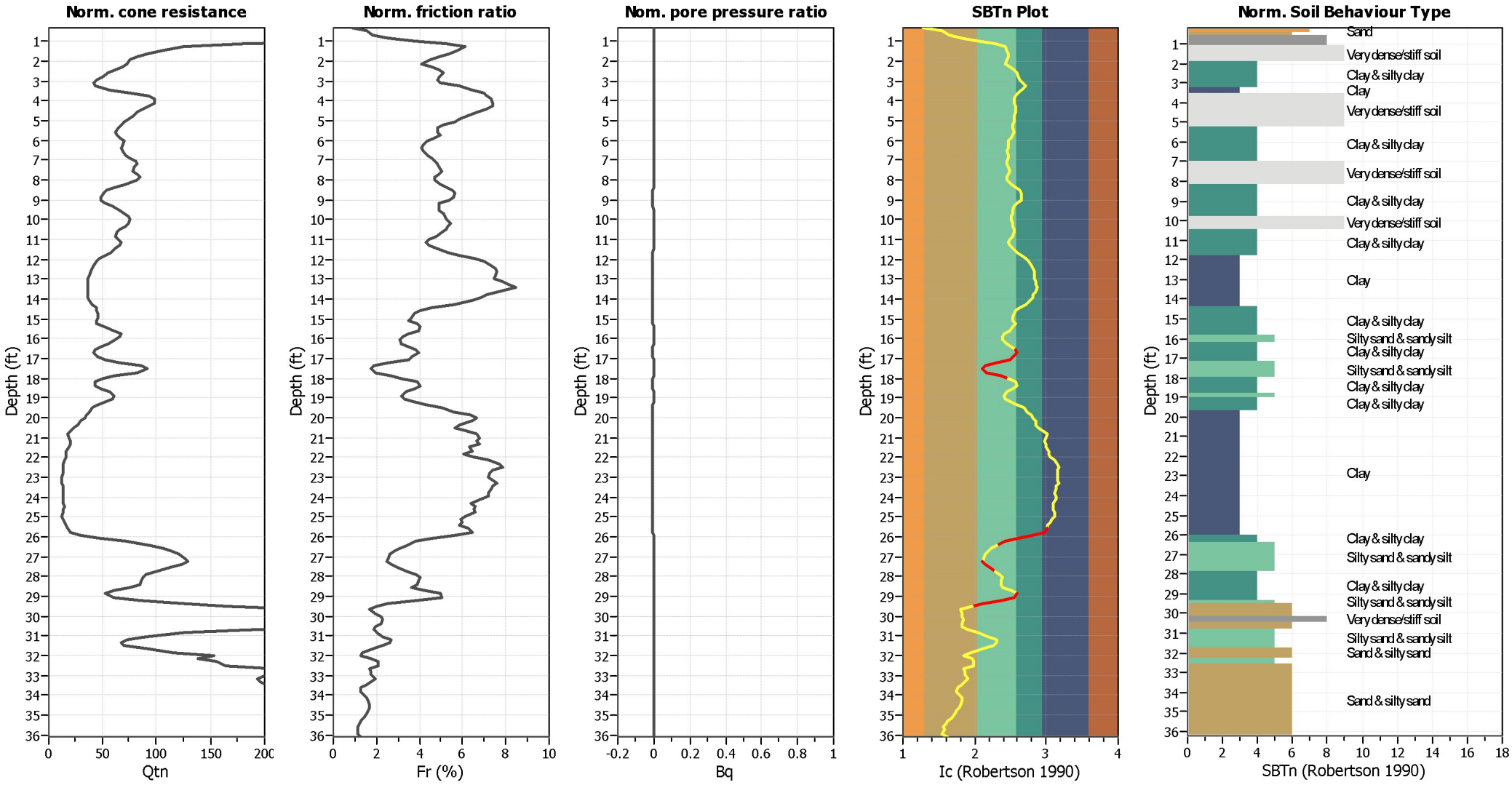






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



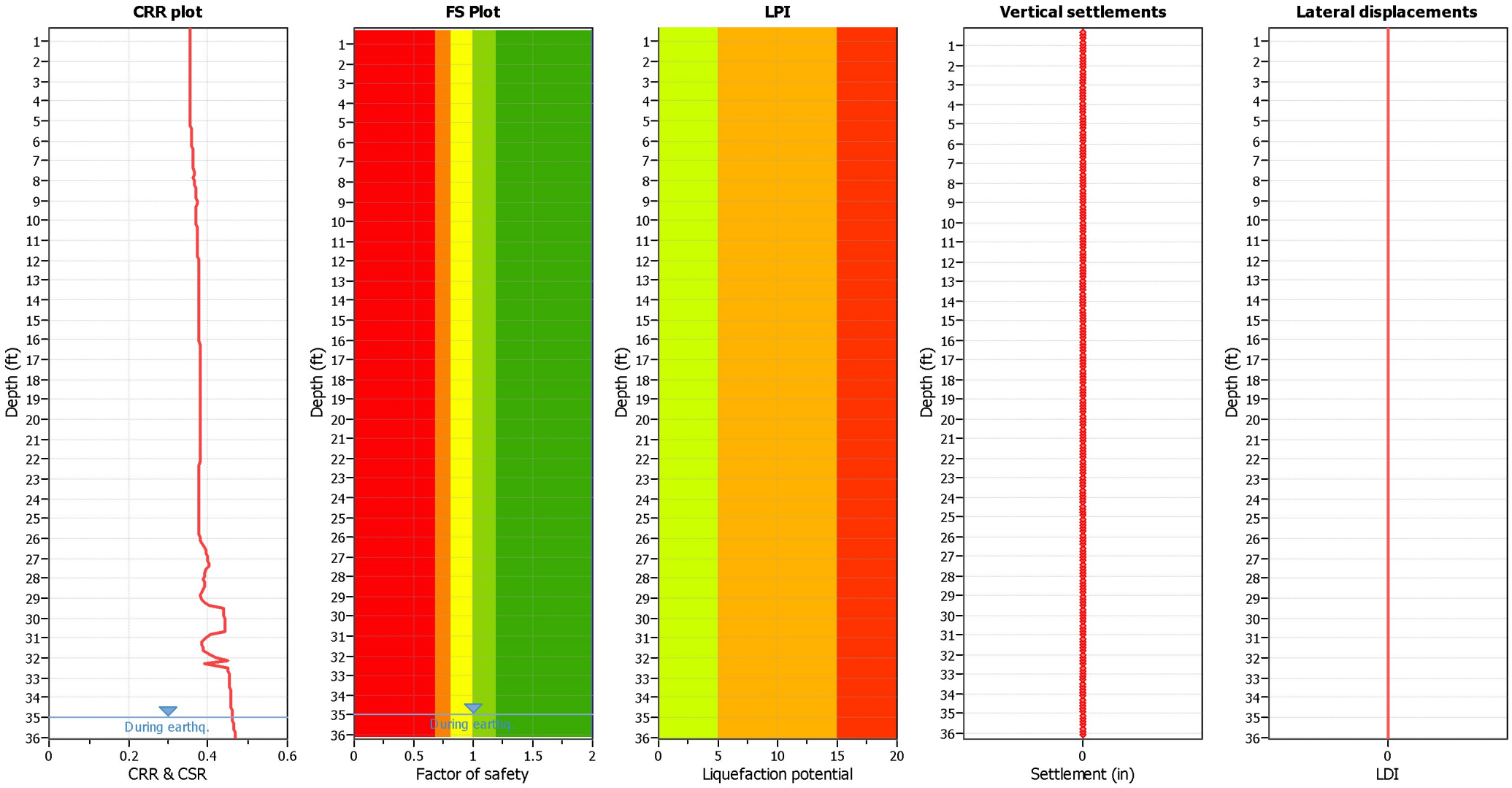
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

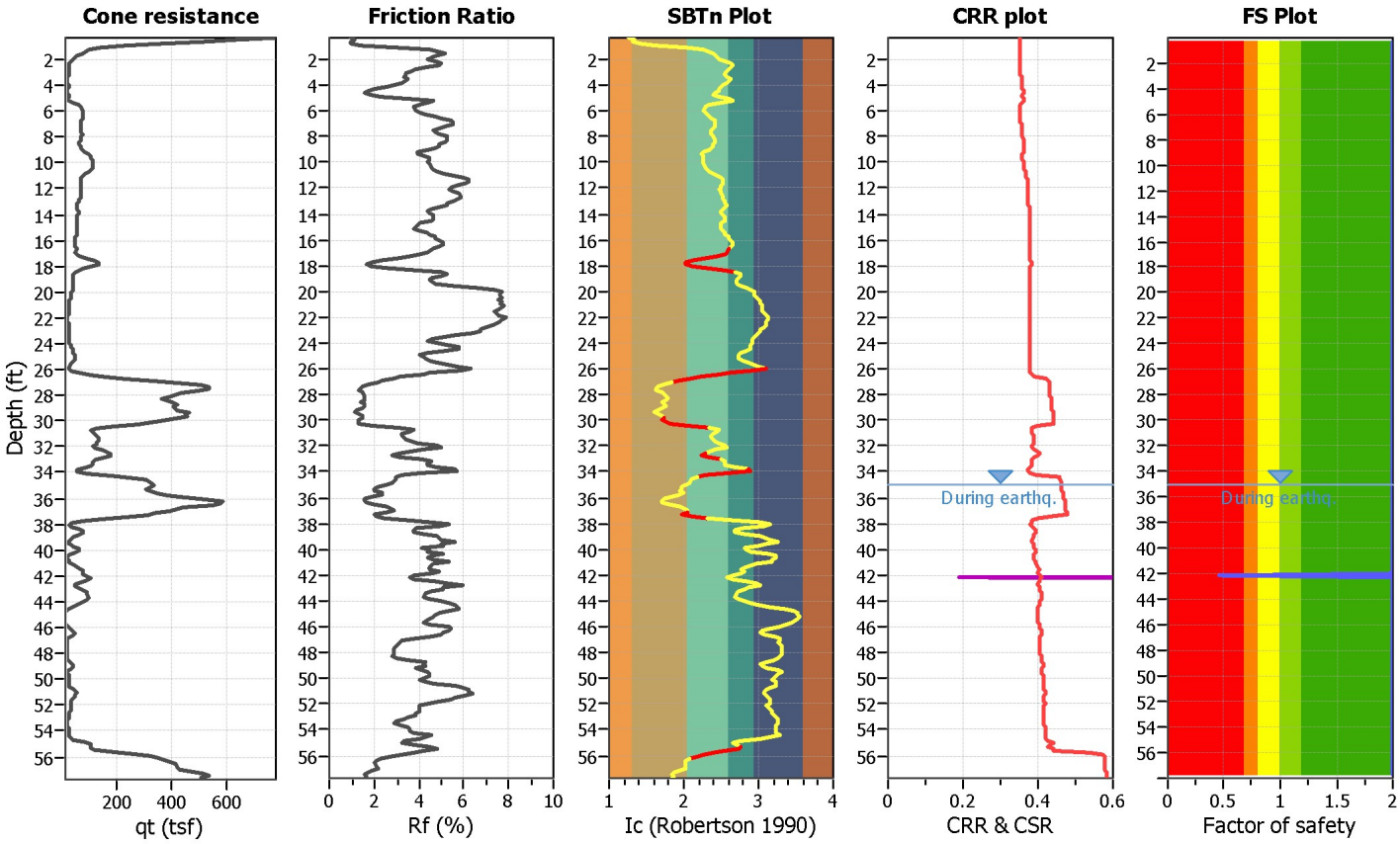
LIQUEFACTION ANALYSIS REPORT

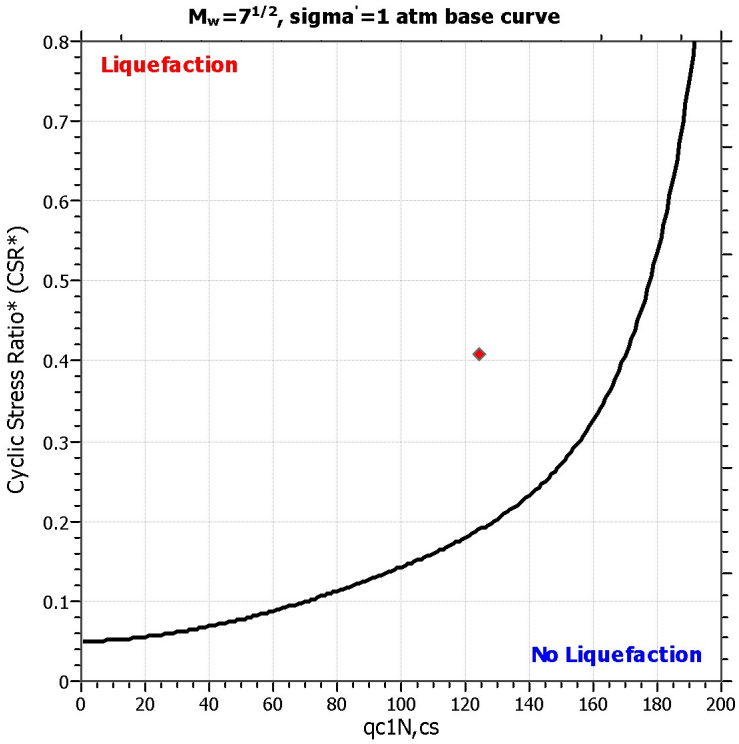
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

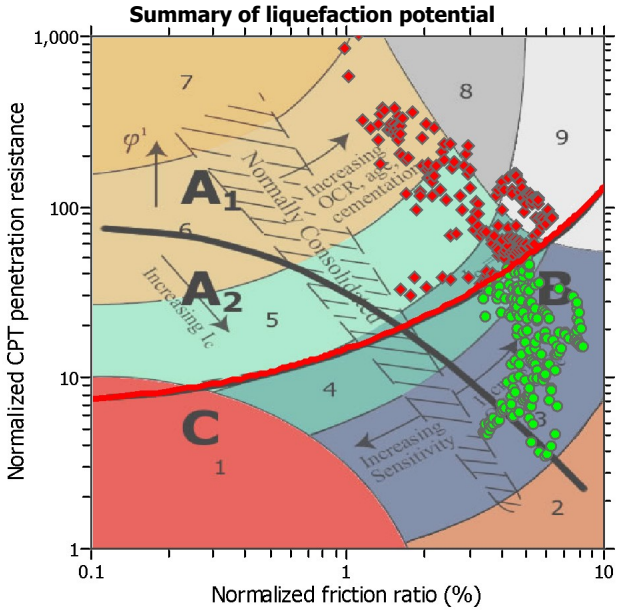
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

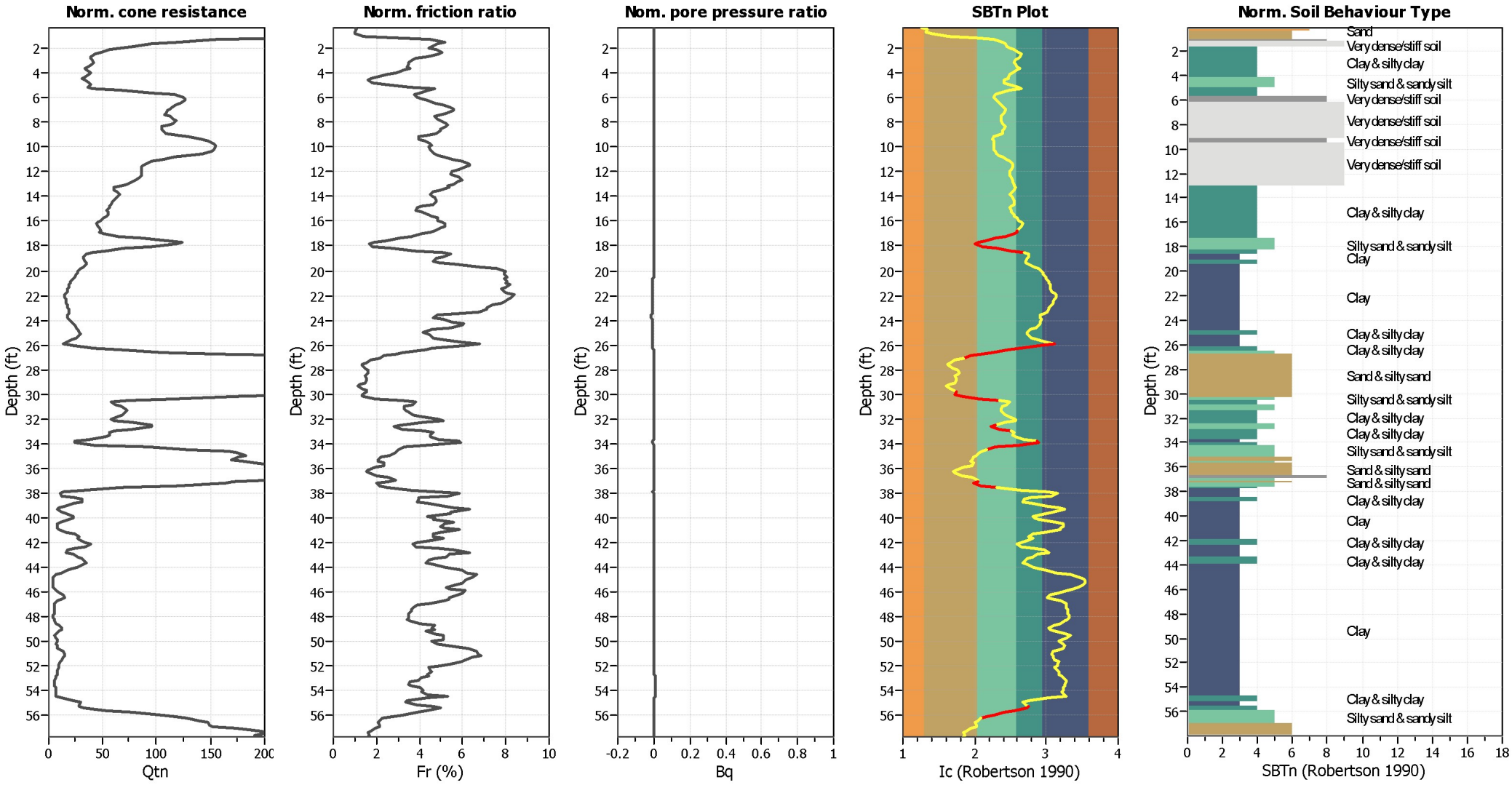






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



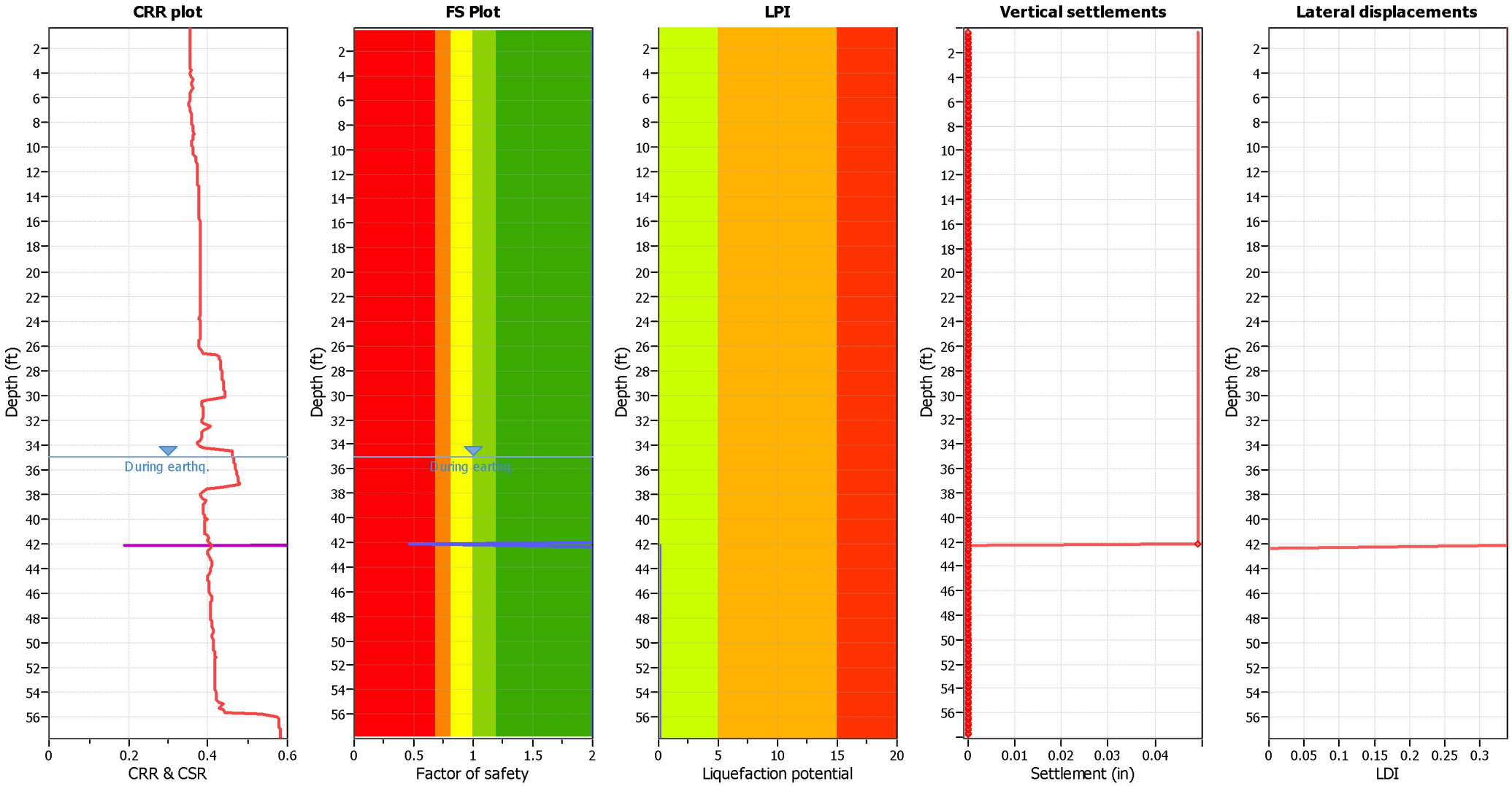
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

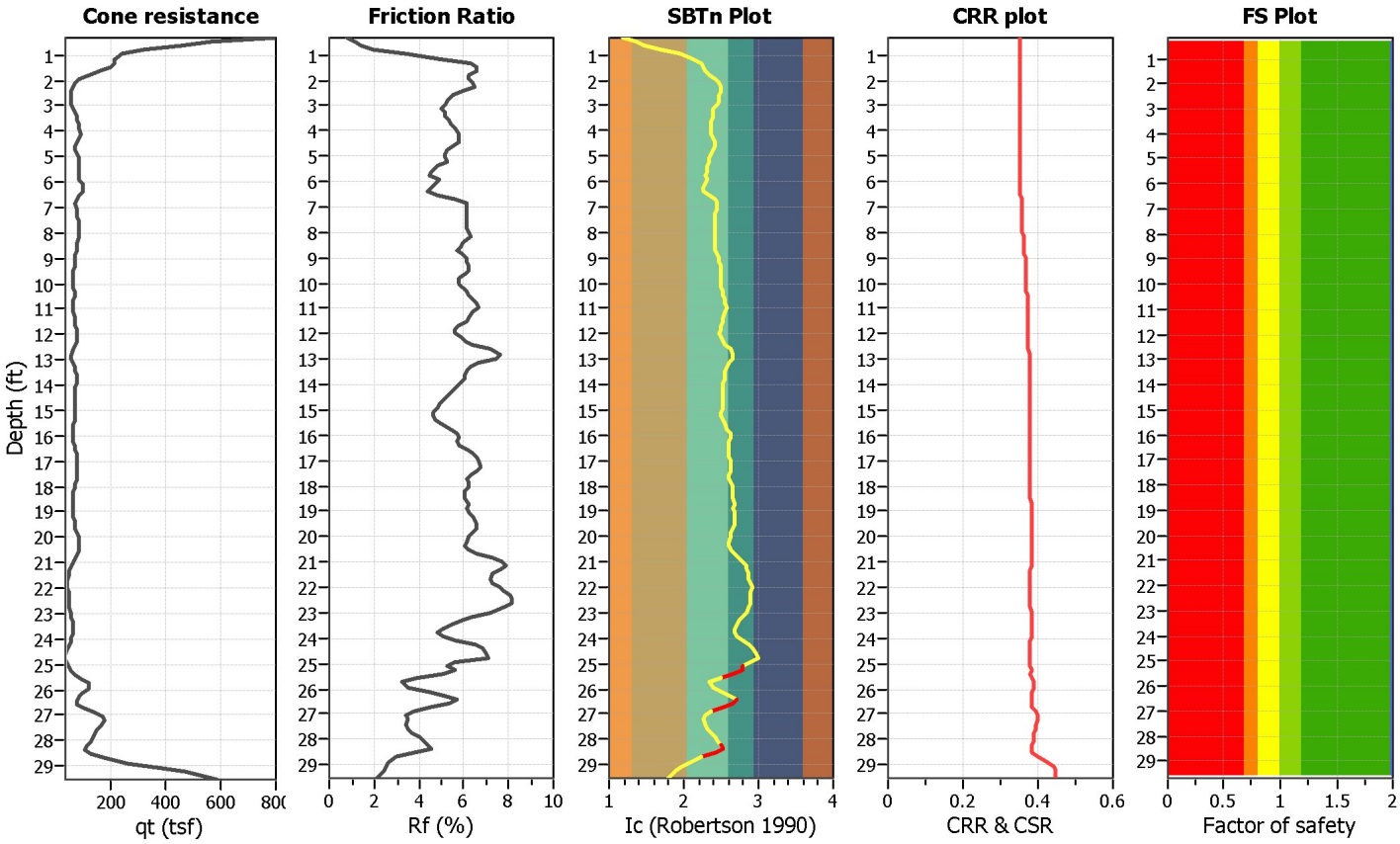
LIQUEFACTION ANALYSIS REPORT

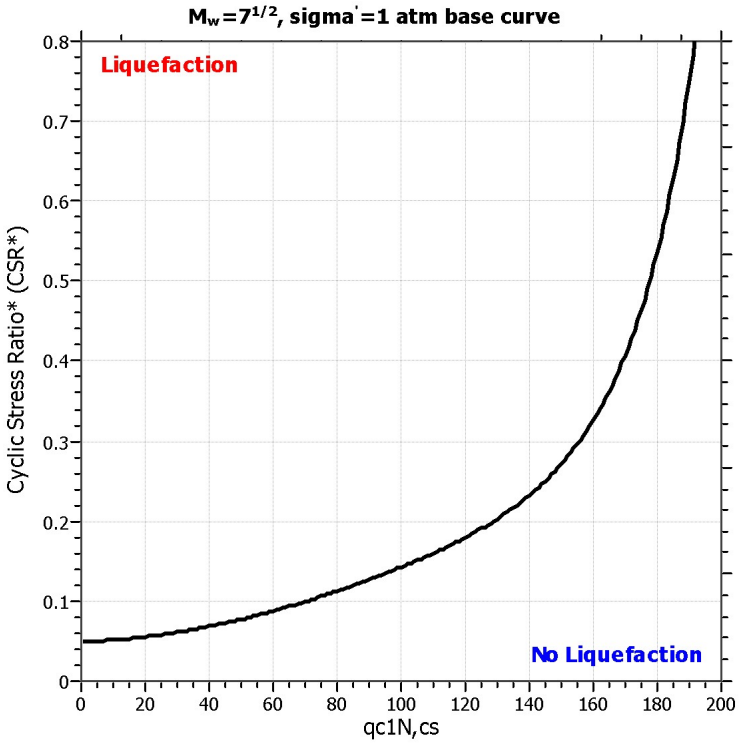
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

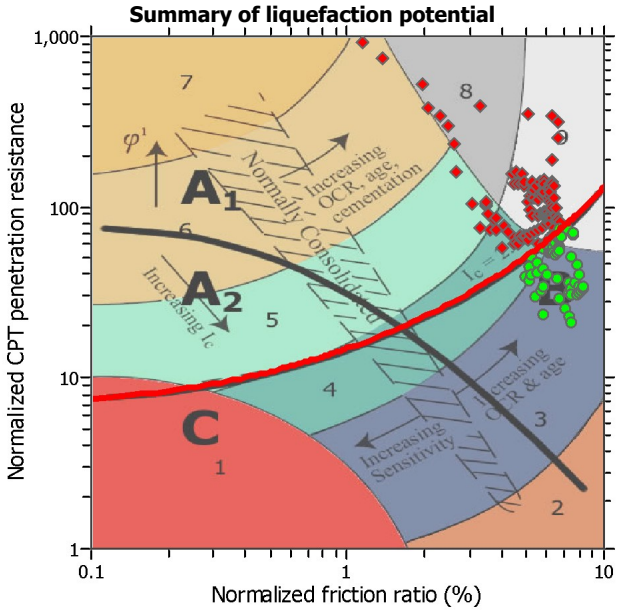
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

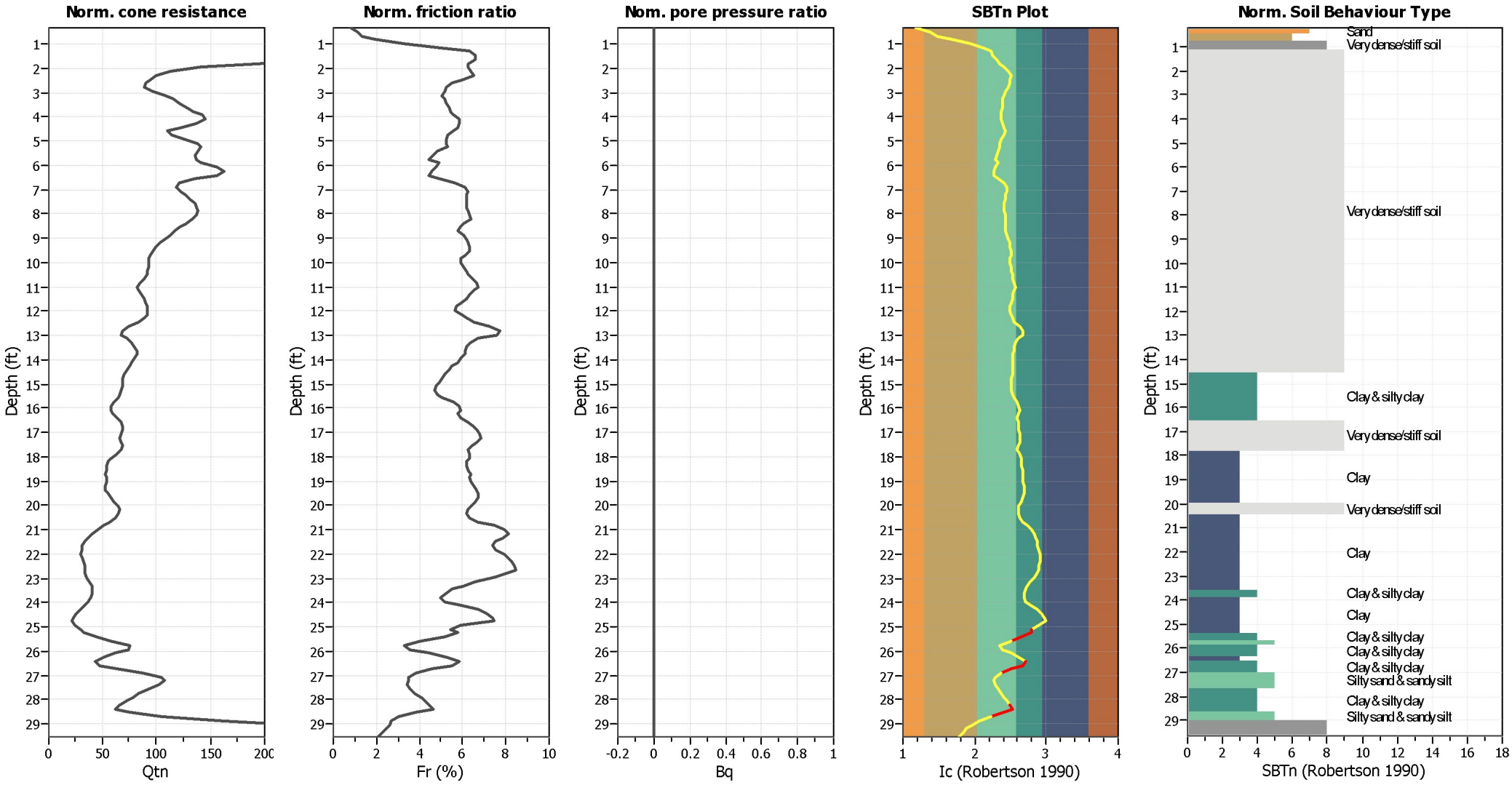






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots (normalized)



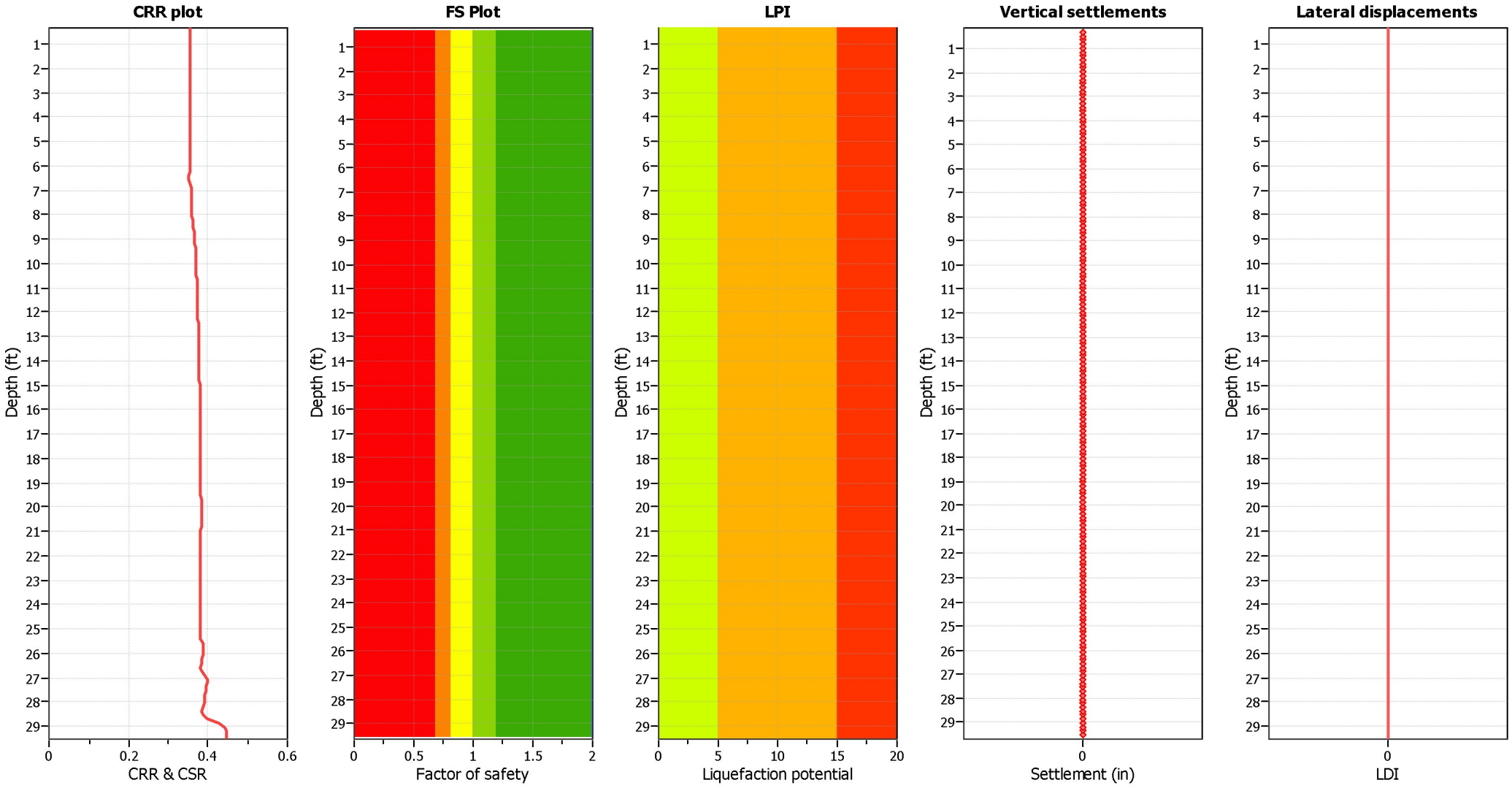
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	35.00 ft	Fill weight:	N/A
Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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 plots



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Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

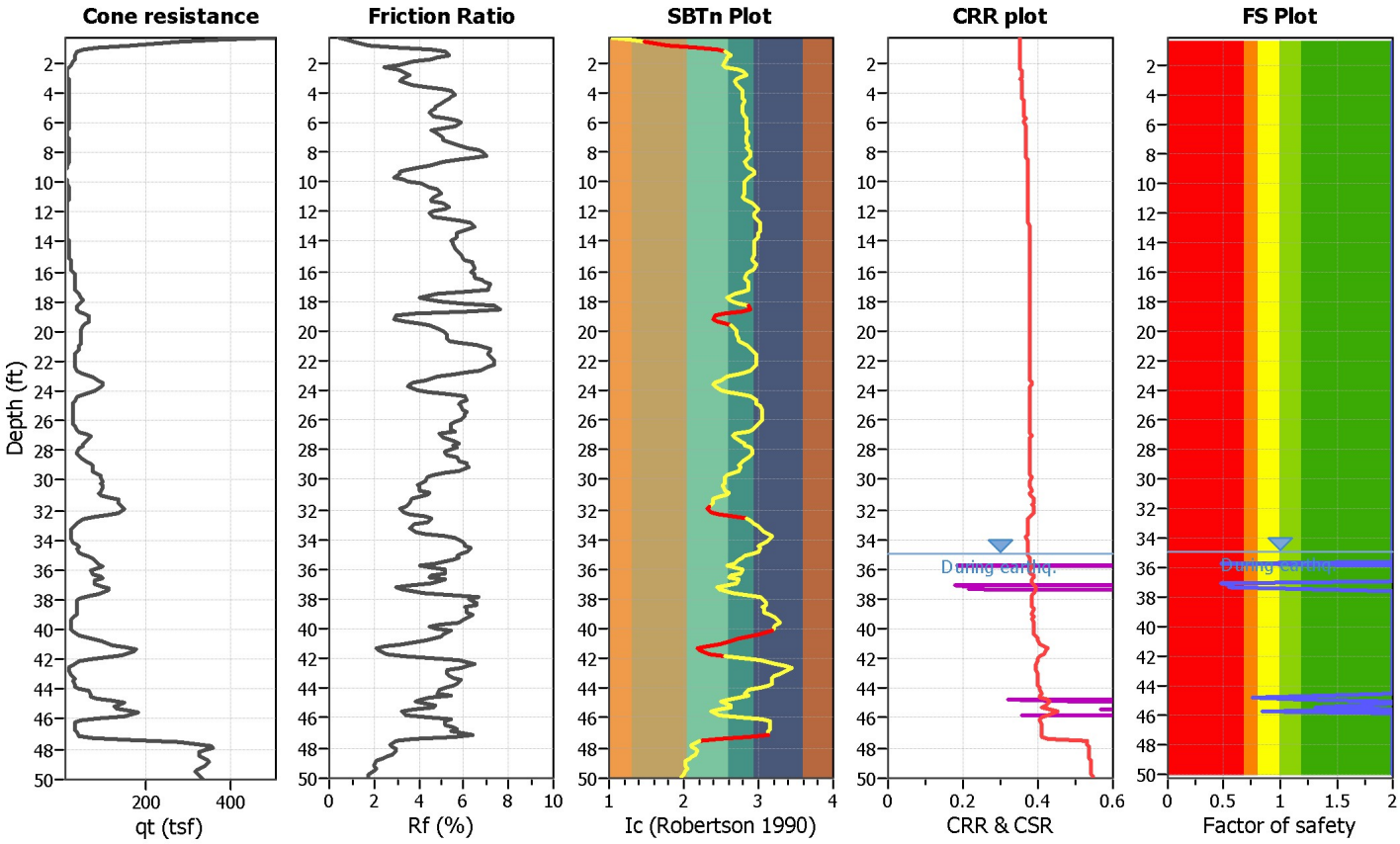
LIQUEFACTION ANALYSIS REPORT

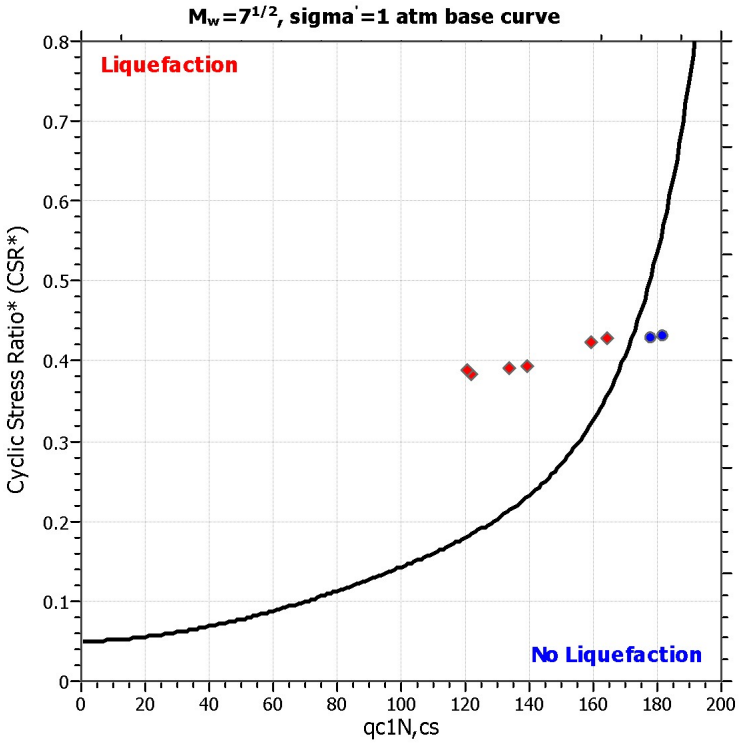
Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

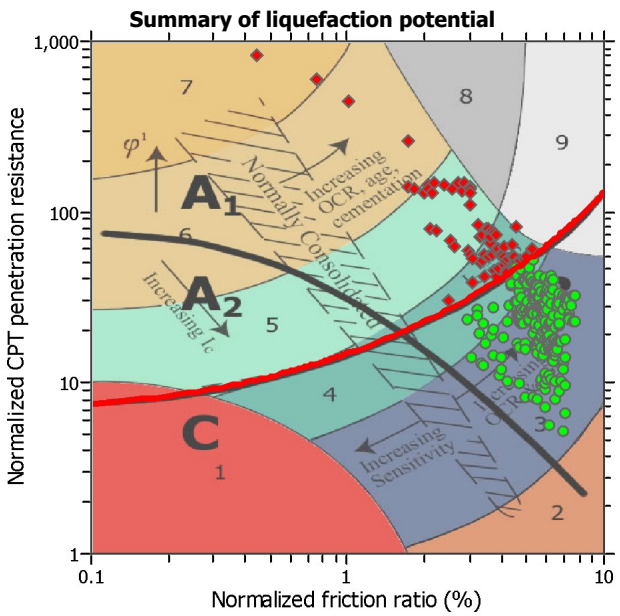
CPT file : 1-CPT4

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	R&W (1998)	G.W.T. (earthq.):	35.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







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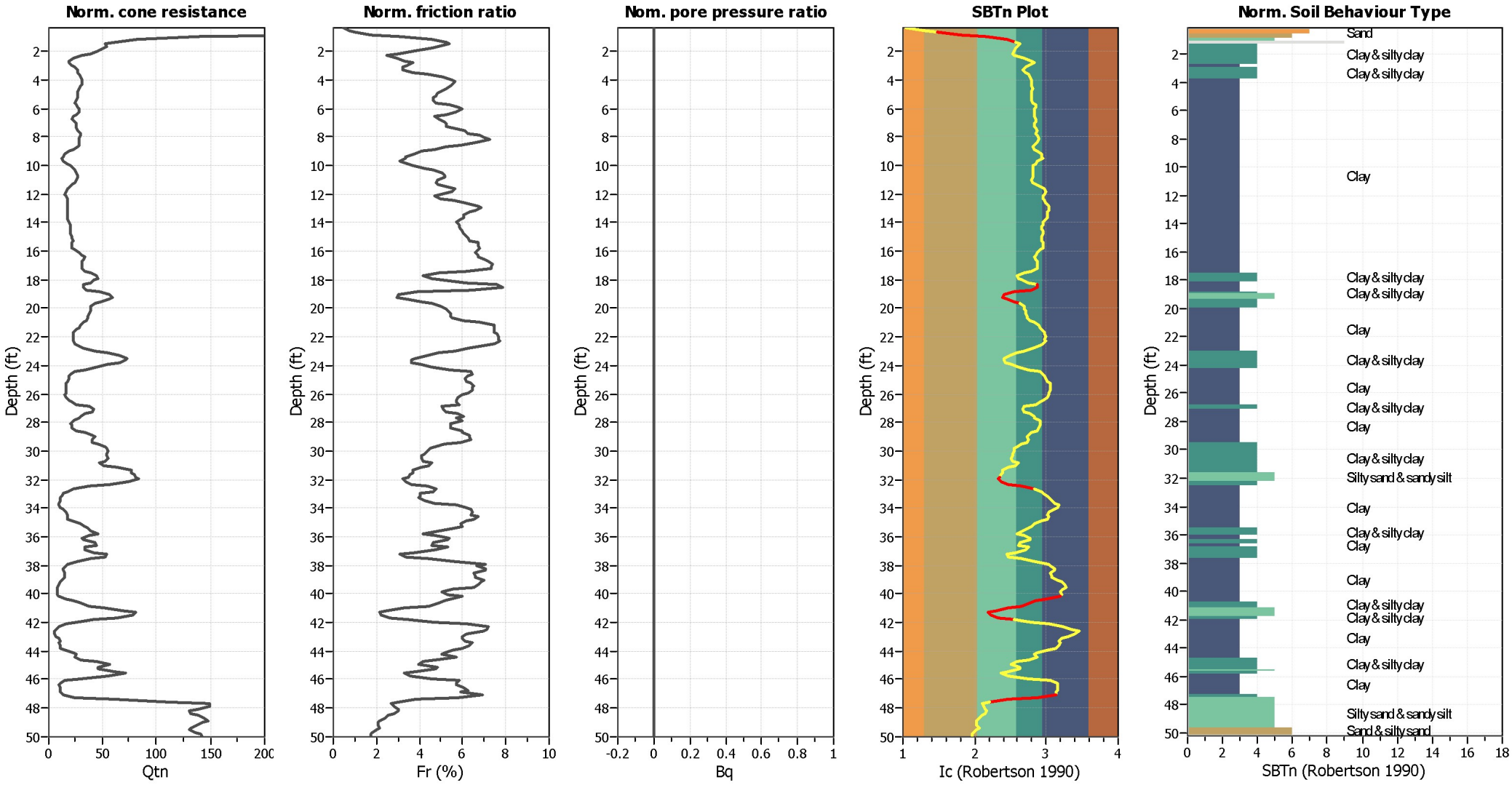
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



Input parameters and analysis data

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Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
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Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
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SBTn legend

1. Sensitive fine grained
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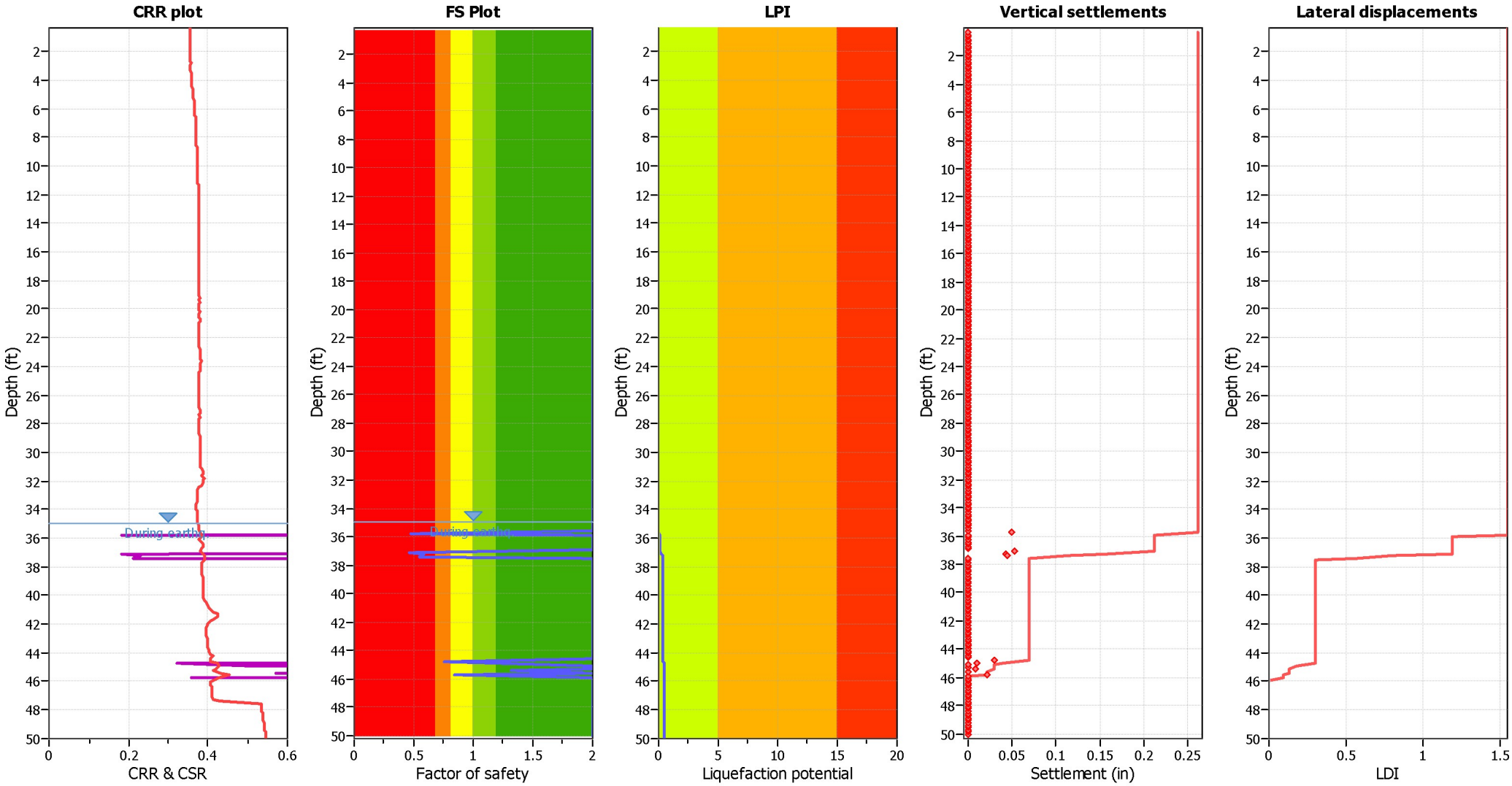
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots



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Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
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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

LIQUEFACTION ANALYSIS REPORT

Project title : Stevens Creek Promenade **Location : Stevens Creek Boulevard, San Jose**

CPT file : 1-CPT5

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	75.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
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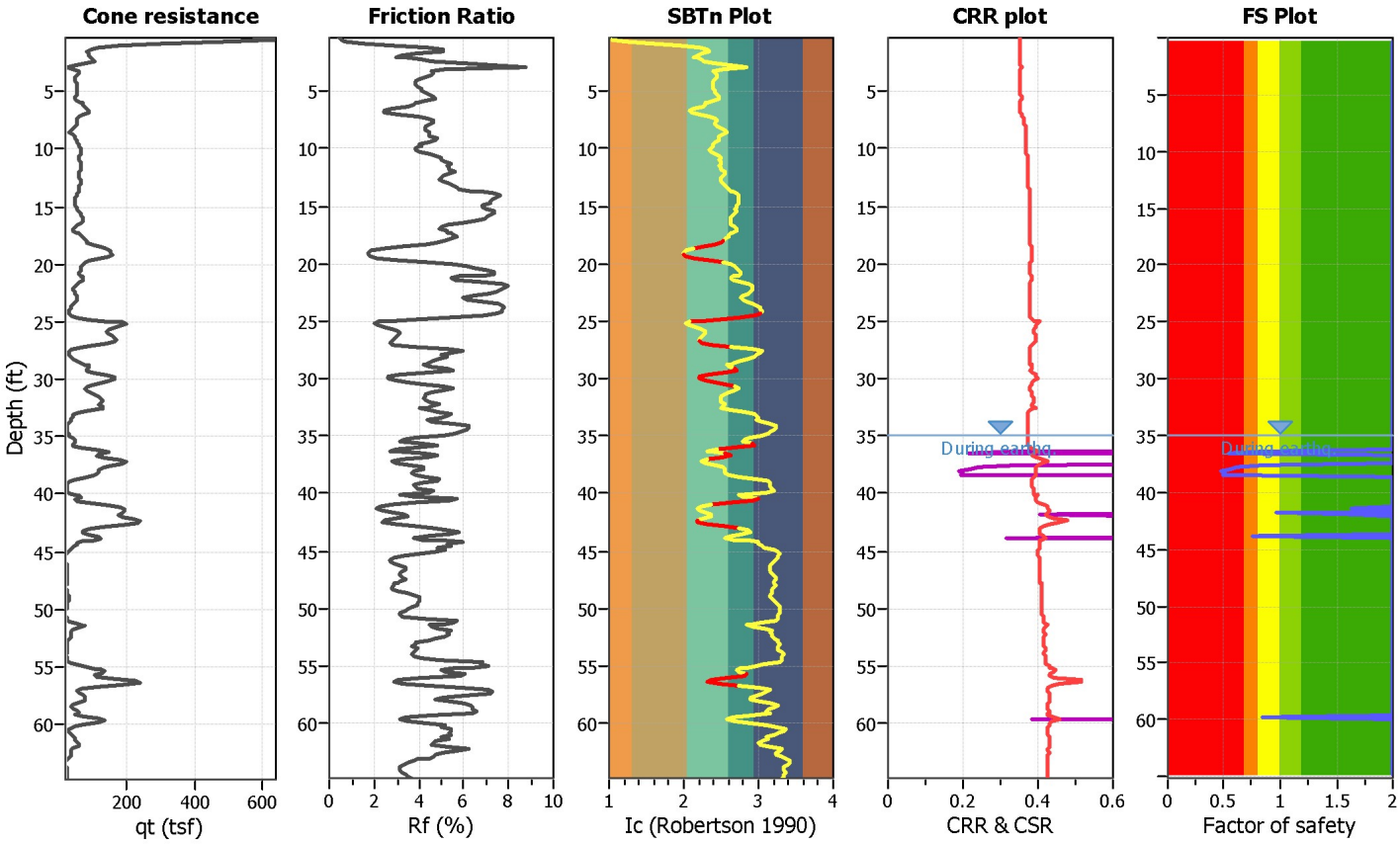
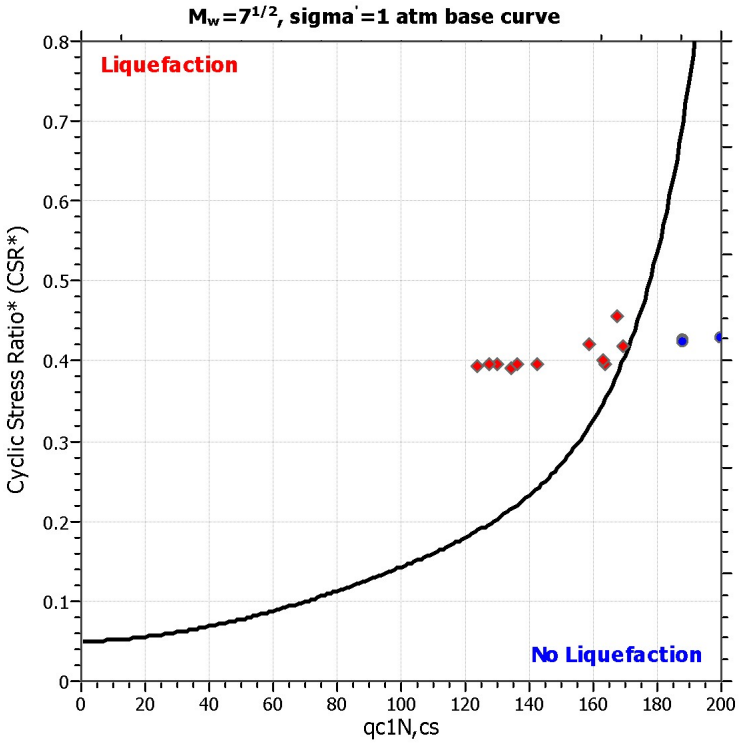
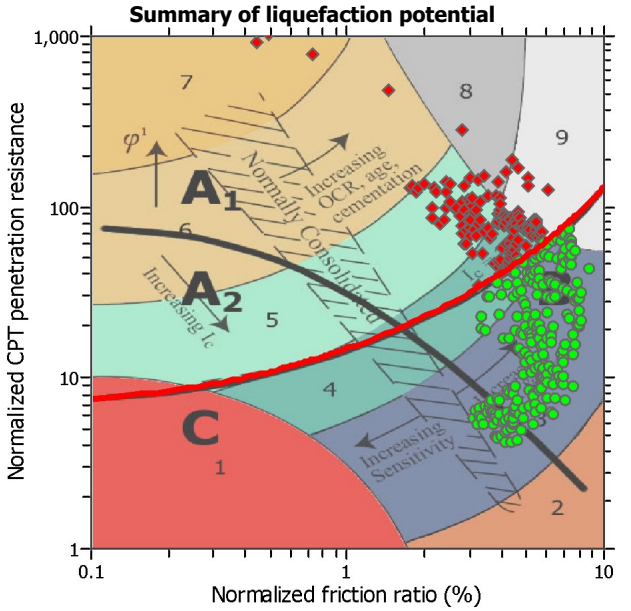
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.54	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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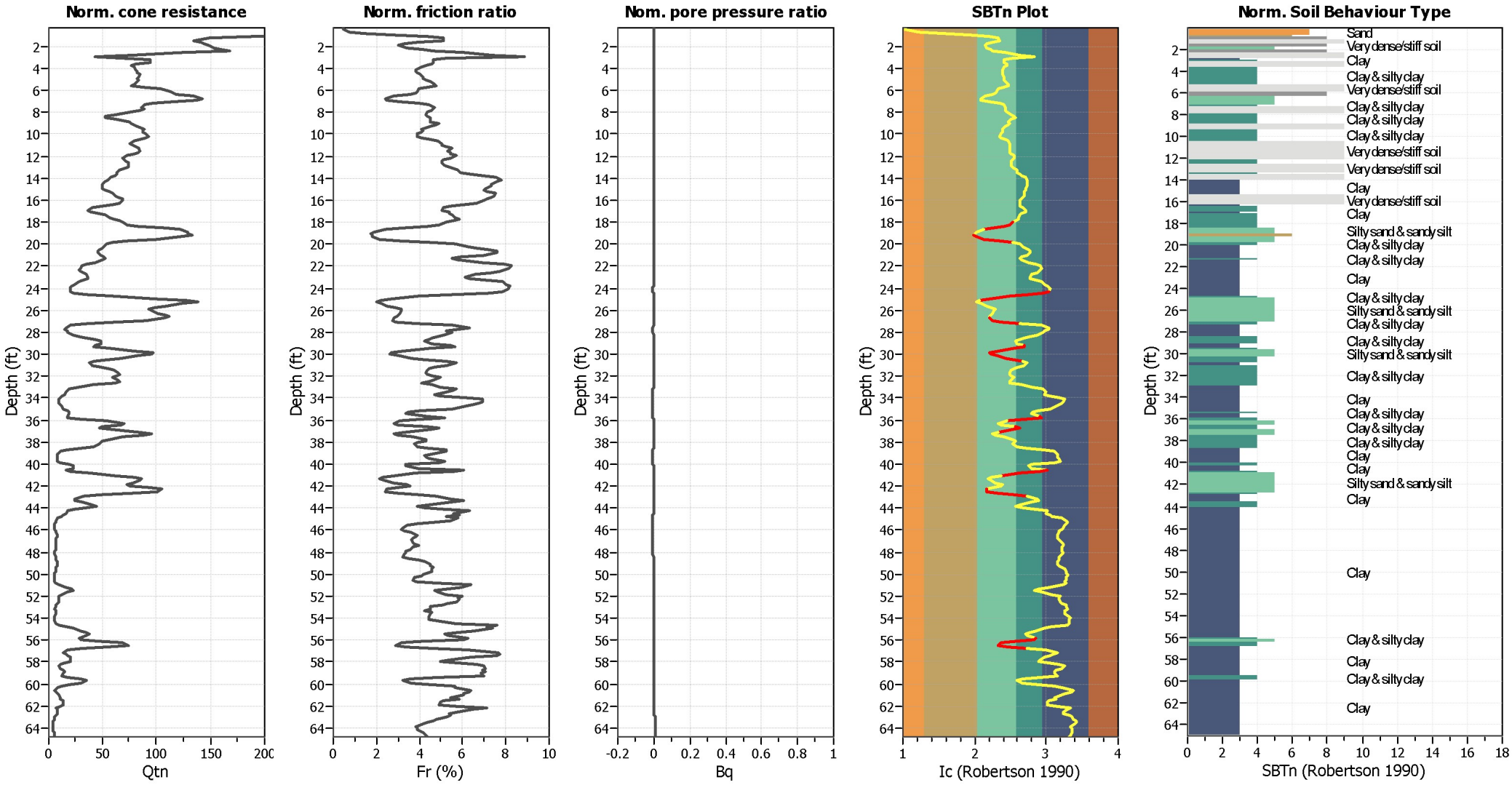
CPT basic interpretation plots (normalized)



Input parameters and analysis data

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Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
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Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.54	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

1. Sensitive fine grained
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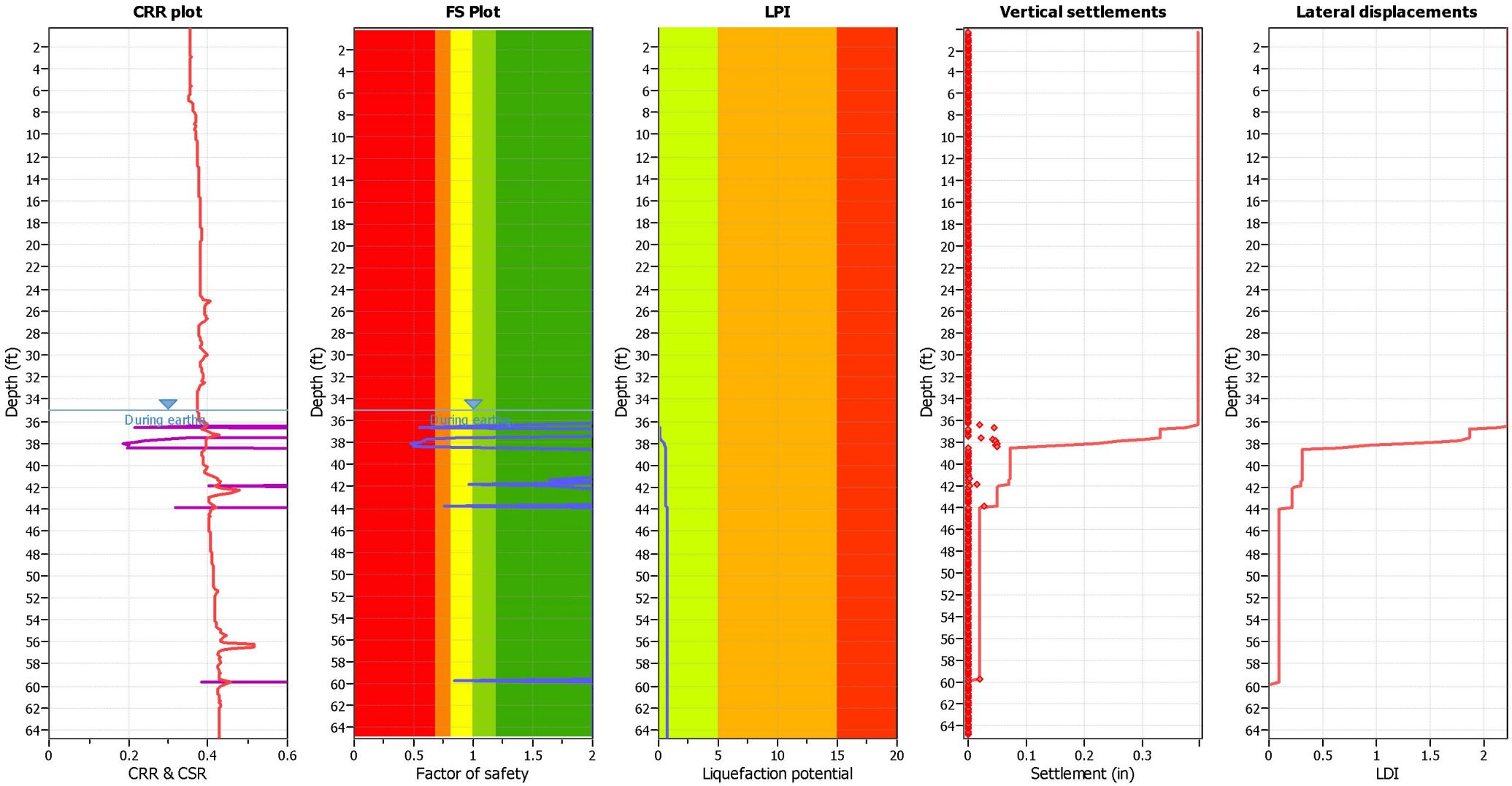
9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

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Fines correction method:	R&W (1998)	Average results interval:	3	Transition detect. applied:	Yes
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Depth to water table (insitu):	75.00 ft	Fill height:	N/A	Limit depth:	N/A

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

DRAFT

APPENDIX E

SUPPLEMENTAL RECOMMENDATIONS

SUPPLEMENTAL RECOMMENDATIONS

Prepared by
ENGEO Incorporated

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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 2 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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