

A P P E N D I X E

PROPOSED HILTON GARDEN INN
GEOTECHNICAL EXPLORATION





PROPOSED HILTON GARDEN INN
111-137 EAST GISH ROAD
SAN JOSE, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Donovan Cole
Westlake Urban, Inc.
520 S. El Camino Real, 9th Floor
San Mateo, CA 94402

PREPARED BY
ENGEO Incorporated

April 19, 2017

PROJECT NO.
13855.000.000

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Mr. Donovan Cole
Westlake Urban, Inc.
520 S. El Camino Real, 9th Floor
San Mateo, CA 94402

Subject: Proposed Hilton Garden Inn
111-137 East Gish Road
San Jose, California

GEOTECHNICAL EXPLORATION

Dear Mr. Cole:

ENGEO prepared this geotechnical report for proposed Hilton Garden Inn project, located at 111-137 East Gish Road, San Jose, as outlined in our agreement dated March 3, 2017. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Based on the results of our exploration, it is our opinion from a geotechnical engineering viewpoint that the site is suitable for the proposed development, provided the conclusions and recommendations in this report are incorporated into the design and implemented in construction. The main geotechnical considerations for the planned development are: the presence of existing near-surface undocumented fill deposits susceptible to excessive settlement; the presence of near surface expansive soil that could adversely impact site improvements and shallow foundations; the presence of shallow groundwater that may be encountered in excavations and trenches; a risk of seismic-induced settlement; and potential compression and settlement induced by heavy building loads.

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


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

Sincerely,

ENGEO Incorporated



Seema Barua, EIT
sb/tpb/jf



Theodore P. Bayham, GE, CEG



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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

ENGEO prepared this geotechnical report for design of the proposed Hilton Garden Inn project in San Jose, California. We prepared this report as outlined in our agreement dated March 3, 2017. Westlake Urban, Inc. authorized ENGEO to conduct the following scope of services:

- Review of available literature and geologic maps for the study area.
- Subsurface field exploration consisting of three cone penetration test (CPT) probes and three soil borings.
- Soil laboratory testing of materials sampled during field exploration.
- Geotechnical data analysis.
- Report preparation summarizing our conclusions and recommendations for the proposed development.

For our use, we received the Conceptual Grading and Utility Plans prepared by BKF Engineers, dated March 10, 2017. We also received a preliminary architectural plan set prepared by Architects Orange, dated March 17, 2017.

This report was prepared for the exclusive use of Westlake Urban, Inc. and their consultants for design of this project. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION

Figure 1 displays a Site Vicinity Map. The approximately 2.2-acre L-shaped project site is located north of the intersection of Kerley Drive and East Gish Road. The site is surrounded by commercial and industrial development. Figure 2 shows site boundaries, proposed building and pavement areas, and our exploratory locations. The site is bordered by Kerley Drive to the west, East Gish Road to the south and commercial development to the north and east. The project site is currently occupied by a 2-story concrete building with various commercial tenants. The majority of the site consists of paved parking areas.

1.3 PROJECT DESCRIPTION

The plans indicate the proposed project consists of a 5-story hotel building having a podium structure, with the bottom level near existing ground surface level. It is anticipated that the first floor (podium) will consist of concrete with the four overlying levels consisting wood-frame construction. The balance of the site is planned to consist of site improvements such as paved parking, concrete flatwork, and landscaping.

2.0 FINDINGS

2.1 SITE BACKGROUND

We reviewed historical aerial photographs of the site dated 1939 through 2012 that were available online at <http://www.historicaerials.com>, that were provided by Environmental Data Resources (EDR), and through Google Earth. Based on a review of historical aerial photographs, the site historically supported orchards up to at least 1956. By 1963, the north portion of the site has been developed and appeared to consist of open parking lot areas. By 1974, the commercial building currently occupying the southern portion of the site is developed.

2.2 GEOLOGY

Regional geologic maps indicate the site lies within the broad, north-south trending, alluvial-filled Santa Clara Valley. As shown on Figure 3, regional geologic mapping by Dibblee (2007) characterizes the site as underlain by Holocene-age young alluvial valley deposits (Qya), consisting of alluvial fine-grained sand, silt, and clay.

2.3 SEISMICITY

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. Therefore, fault rupture through the site is not anticipated.

The site does lie within a seismically active region, as California has numerous faults that are considered active. Generally, a fault is considered active if it has ruptured within the Holocene epoch (11,700 years before the present). The following table summarizes the distances and estimated maximum magnitudes to mapped, active regional faults within approximately 50 miles using the USGS Spatial Query tool based on USGS 2008 National Seismic Hazard Maps. Refer to Figure 5 for the Regional Faulting and Seismicity Map.

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

FAULT NAME	DISTANCE FROM SITE (MILES)	DIRECTION FROM SITE	MAXIMUM MOMENT MAGNITUDE
Hayward-Rodgers Creek	7	Northeast	7.3
Calaveras	7.8	Northeast	7.0
Monte Vista-Shannon	8.5	Southwest	6.5
North San Andreas	12.6	Southwest	7.9

2.4 FIELD EXPLORATION

Our field exploration included drilling three borings and advancing four Cone Penetration Test (CPT) soundings at various locations on the site. We performed our field exploration on March 28, 2017. The location and elevations of our explorations are approximate and were estimated by pacing from features shown on the Site Plan, Figure 2. They should be considered accurate only to the degree implied by the method used.

2.4.1 Borings

We observed drilling of three borings at the locations shown on the Site Plan, Figure 2. An ENGEO engineer observed the drilling and logged the subsurface conditions at each location. We retained a truck-mounted drill rig and crew to advance the borings using 4-inch-diameter solid-flight auger and mud-rotary auger methods. The borings were advanced to depths ranging from 10 to 46½ feet below existing grade. We permitted and backfilled the borings in accordance with the requirements of Santa Clara Valley Water District.

We obtained soil samples at various intervals in the borings using standard penetration test (SPT) and Modified California driven samplers. The penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration. In addition, 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows to drive the last 1 foot of penetration; the blow counts have not been converted using any correction factors.

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

2.4.2 Cone Penetration Tests

We retained a CPT rig to push the cone penetrometer to a maximum depth of about 50 feet. The CPT has a 20-ton compression-type cone with a 15-square-centimeter (cm²) base area, an apex angle of 60 degrees, and a friction sleeve with a surface area of 225 cm². The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D-5778. Measurements include the tip resistance to penetration of the cone (Q_c), the resistance of the surface sleeve (F_s), and pore pressure (U) (Robertson and Campanella, 1988). CPT logs are presented in Appendix C.

2.5 SURFACE CONDITIONS

According to the Civil plans, site topography generally ranges from Elevation 49 feet (NAVD88) on the southern end of the site to approximately 52 feet in the northern portion of the site. We visit the site on March 21 and March 28, 2017, for our site reconnaissance and field exploration. At these times, the southern portion of the project site is currently occupied by a 2-story concrete building with various commercial tenants. The majority of the site consists of paved parking areas and landscape medians. Overhead powerlines traverse the northern and central portions of the site in a general east-west direction. An electrical line is shown going north-south, originating from the powerlines in the central portion of the site and connecting to the existing building. A storm drain crosses east west through the southeastern corner of the site.

2.6 SUBSURFACE CONDITIONS

Our borings were drilled within existing paved areas within the site. The borings generally encountered approximately 2 to 4 inches of asphaltic concrete overlying up to 5 inches of

aggregate base material. Below the pavement sections, the borings encountered a layer of existing “man-made” fill comprised of sandy silt; the existing fill varied in thickness and within our borings ranged from approximately 1 to 3½ feet.

Beneath the existing fill, borings generally encountered natural alluvial soil deposits consisting of stiff to very stiff, fat clays that extended to depths of approximately 37 to 40 feet below existing ground surface. At Boring 1-B1 at a depth of approximately 11½ feet, an interlayer (approximately 2 feet thick) of medium dense, poorly graded sand was encountered. At Boring 1-B1 at depths ranging between 40 and 46½ feet, we encountered a layer of dense to very dense, clayey gravel with sand. A layer of dense, silty sand was encountered in 1-B2 at approximately 38½ feet extending to the maximum depth drilled of 39 feet.

Also, at Borings 1-B1 and 1-B2, soft to medium stiff layer of fat clay approximately 2½ thick were encountered. The soft to medium stiff layer was also encountered at CPTs, 1-CPT2 and 1-CPT4. Selected samples obtained in the upper 20 feet were tested for Plasticity Index (PI), and yielded PI values of 32 and 36; the results indicate a high expansion potential for the near surface soils.

2.7 GROUNDWATER CONDITIONS

We observed groundwater in several of our subsurface explorations. We summarize our observations in the table below:

TABLE 2.7-1: Groundwater Observations

EXPLORATION LOCATION	APPROX. DEPTH TO GROUNDWATER (FEET)	CORROLATION TO APPROXIMATE ELEVATION (FEET)
1-B3	7	43
1-CPT1	9	43
1-CPT2	7 ½	45
1-CPT3	9 ½	43
1-CPT4	9 ½	41

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

2.8 LABORATORY TESTING

We performed laboratory tests on selected soil samples to evaluate their engineering properties. For this project, we performed moisture content, dry density, unconfined compression, undrained shear strength, plasticity index, gradation, hydrometer, and soil corrosion potential testing. Moisture contents and dry densities are recorded on the boring logs in Appendix A; other laboratory data is included in Appendix B.

3.0 CONCLUSIONS

Based on the results of our exploration, it is our opinion from a geotechnical engineering viewpoint that the site is suitable for the proposed development, provided the conclusions and recommendations in this report are incorporated into the design and implemented in construction.

The main geotechnical considerations for the planned development are: the presence of existing near-surface undocumented fill deposits susceptible to excessive settlement; the presence of near surface expansive soil that could adversely impact site improvements and shallow foundations; the presence of shallow groundwater that may be encountered in excavations and trenches; a risk of seismic-induced settlement; and potential compression and settlement induced by heavy building loads.

3.1 EXISTING UNDOCUMENTED FILL

A near surface layer of existing “man-made” undocumented fill is present extending to a depth of approximately 1 to 3½ feet. It underlies the existing pavement around buildings. It is likely that other existing fill deposits are present at the site, such as along utility trenches, landscape areas, and possibly buried structures. Undocumented existing fills may undergo excessive settlement, especially under new fill or building loads. To reduce the risk of settlement these existing fills may be removed and re-compacted in accordance with compaction specification in this report.

3.2 EXPANSIVE SOIL

We observed potentially expansive fat clays near the surface of the site in in all Borings 1-B1 through 1-B3. Our laboratory testing indicates that these soils exhibit high shrink/swell potential with variations in moisture content. Expansive soils change in volume with changes in moisture. They can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Building damage due to volume changes associated with expansive soils can be through proper compaction, selective grading or blending with non-expansive soil, and proper foundation design. Expansive soil mitigation recommendations are presented in a subsequent section.

3.3 SHALLOW GROUNDWATER

We provide the following information regarding depth to groundwater at the site and within the site vicinity.

- The measured groundwater level for CPT probes and at the time of drilling for borings ranged from depths of approximately 7 to 9½ feet.
- Plate 1.2 of the Seismic Hazard Zone Report for the San Jose West Quadrangle (2002) maps the historical groundwater in the site vicinity as roughly 10 feet below existing grade.

Fluctuations in groundwater levels occur seasonally and over a period of years because of variations in precipitation, temperature, irrigation, or other factors. A design groundwater level of between 7 to 9 feet below the existing ground surface has been selected for our analysis.

3.4 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, and ground lurching. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, soil

liquefaction, lateral spreading, landslides, tsunamis, flooding or seiches is considered low to negligible at the site.

3.4.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, it is our opinion that ground rupture is unlikely at the subject property.

3.4.2 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, structures should be designed using sound engineering judgment and the 2007 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).

3.4.3 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 in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Based on the site location, it is our opinion that the potential for ground lurching is low at the site.

3.4.4 Flooding

Based on site elevation and distance from water sources, flooding is not expected at the subject site; however, the Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if recommended.

3.4.5 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-grained sands below the groundwater table. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop and liquefaction of susceptible soils to occur. Historically, standard geotechnical

engineering practices for liquefaction assessment have included layers of loose to medium dense and saturated sandy deposits as being potentially liquefiable.

According to the State of California Seismic Hazard Zones Map for the San Jose West Quadrangle (2002), the site is located within an area mapped as being potentially susceptible to liquefaction (Figure 4). To assess liquefaction potential, we performed liquefaction analyses on the four CPT probes advanced at the site. The CPT logs provided a continuous soil profile using an empirically generated soil behavior type index, I_c . Through trial and error and calibration of the analyses, a cutoff value for the soil behavior type index, I_c , is selected.

In addition, we performed liquefaction analyses for the borings using SPT blow counts for saturated samples obtained from mud-rotary auger borings. Visual soil classification and laboratory testing of samples allowed for correlation between the CPTs and borings. We assigned a design groundwater level of 7 to 9 feet below the existing ground surface, a PGA of 0.50g (2% in 50 years probability of exceedance), and an earthquake magnitude of 7.1.

We evaluated potential post-liquefaction ground settlement at the site considering CPT conditions using the computer program CLiq, assuming an I_c cutoff of 2.6. Our analyses were based on guidelines provided in DMG Special Publication 117A (2008) and methods developed by Robertson (NCEER 2009), Moss et al. (2006), Idriss and Boulanger (2008) for liquefaction assessment for CPTs and Youd (2001), Seed (2003), and Idriss and Boulanger (2008) for the liquefaction assessment for borings. Our analyses are attached as Appendix D. Results of our liquefaction analysis indicated that soil in 1-CPT3 are unlikely to liquefy. Liquefaction assessments for the CPT probes and borings identified select thin (roughly 1-foot thick) lenses of potentially liquefiable sand and silt located between depths of 11 to 12 feet in 1-CPT1, 1-CPT2, and Boring 1-B1, and 37 and 42 feet in 1-CPT2 and 1-CPT3.

We also evaluated the capping effect of any overlying non-liquefiable soils. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a force sufficient to break through the overlying soil and vent to the surface resulting in sand boils or fissures. Based our analyses and review on guidelines provided by Ishihara (1985) and Youd and Garris (1995), the risk of secondary ground effects such as venting is considered low.

Our calculations indicated theoretical total liquefaction-induced volumetric strain settlement of ½ inch is estimated across the site. Recommendations to address this concern are provided in subsequent sections.

3.4.6 Lateral Spreading

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

The site is located approximately ½-mile west of Guadalupe River. Due to the lack of steep slopes or nearby open channels, it is our opinion that the potential for lateral spreading is low.

3.5 COMPRESSIBLE SOILS AND LOAD-INDUCED SETTLEMENT

Soils are subject to settlement when a new loading scenario is introduced by structures or fill. Based on review of boring and CPT logs, a soft to medium stiff layer of fat clay approximately 2½ feet thick were encountered. Soft to medium stiff clay deposits may undergo load-induced settlement in response to new loads. We performed load-induced settlement analyses for the planned hotel structures based on an assumed structural mat foundation type with an anticipated average uniform loading of 1,000 pounds per square foot (psf), and planned grades roughly matching existing grades. Based on the above conditions, we estimate that up to approximately 1-inch of total load-induced settlement may occur.

We also performed load-induced settlement analyses for the planned hotel structures based on an assumed isolated footing foundation type based on a 4-foot square footing, anticipated column dead plus live loads of up to 40 kips, allowable bearing capacity of 2,500 pounds per square foot (psf), and planned grades roughly matching existing grades. Based on the above conditions, we estimate that up to approximately ½-inch of total load-induced settlement may occur. Once building project plans and building loads are further defined, we can reassess our settlement analyses. If changes to the planned development result in changes to the above assumptions, we should also review and update our analysis as needed.

3.6 SOIL CORROSION POTENTIAL

As part of this study, we obtained a two representative soil sample and submitted to a qualified analytical lab for determination of pH, resistivity, sulfate, and chloride. The results are included in Appendix E and summarized in the table below.

TABLE 3.6-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
1-B1	1-2½	8.22	2,900	ND	27
1-B2	3-3½	NA	NA	NA	ND

NA – Not Analyzed
ND – None Detected

The 2016 CBC references the 2011 American Concrete Institute Manual, ACI 318-11, Chapter 4, Sections 4.2.1 for structural concrete requirements. ACI Table 4.2.1 provides the following exposure categories and classes, and concrete requirements in contact with soil based upon the exposure risk.

TABLE 3.6-2: ACI Table 4.2.1, Sulfate Exposure Categories and Classes

SULFATE EXPOSURE CATEGORY	EXPOSURE CLASS	WATER- SOLUBLE SULFATE IN SOIL % BY WEIGHT*	DISSOLVED SULFATE IN WATER MG/KG (PPM)**
Not applicable	S0	SO ₄ < 0.10	SO ₄ < 150
Moderate	S1	0.10 ≤ SO ₄ < 0.20	150 ≤ SO ₄ ≤ 1,500 seawater
Severe	S2	0.20 ≤ SO ₄ ≤ 2.00	1,500 ≤ SO ₄ ≤ 10,000
Very severe	S3	SO ₄ > 2.00	SO ₄ > 10,000

* Percent sulfate by mass in soil determined by ASTM C1580

** Concentration of dissolved sulfates in water in ppm determined by ASTM D516 or ASTM D4130

In accordance with the criteria presented in the above table, these soils are categorized as S0 sulfate exposure class. Cement type, water-cement ratio, and concrete strength, are not specified for these ranges.

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

Based on the resistivity measurements, the soils are considered moderately corrosive to buried metal piping. Values tested for chloride do not pose a significant impact to metals or concrete. Testing was not completed for all depths of potential embedment or across the entire site. If requested, we can provide additional testing and/or guidance regarding the exposure risk.

3.7 2016 CBC SEISMIC DESIGN PARAMETERS

The 2016 CBC utilizes design criteria set forth in the 2010 ASCE 7 Standard. Based on the subsurface conditions encountered, we characterized the site as Site Class D in accordance with the 2016 CBC. We provide the 2016 CBC seismic design parameters in Table 3.7-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3.7-1: 2016 CBC Seismic Design Parameters, Latitude: 37.36299 Longitude: -121.90791

PARAMETER	VALUE
Site Class	D
Mapped MCE _R Spectral Response Acceleration at Short Periods, S _S (g)	1.50
Mapped MCE _R Spectral Response Acceleration at 1-second Period, S ₁ (g)	0.60
Site Coefficient, F _A	1.00
Site Coefficient, F _V	1.50
MCE _R Spectral Response Acceleration at Short Periods, S _{MS} (g)	1.50
MCE _R Spectral Response Acceleration at 1-second Period, S _{M1} (g)	0.90
Design Spectral Response Acceleration at Short Periods, S _{DS} (g)	1.00
Design Spectral Response Acceleration at 1-second Period, S _{D1} (g)	0.60
Mapped MCE Geometric Mean (MCE _G) Peak Ground Acceleration, PGA (g)	0.50
Site Coefficient, F _{PGA}	1.00
MCE _G Peak Ground Acceleration adjusted for Site Class effects, PGA _M (g)	0.50
Long period transition-period, T _L	12 sec

4.0 FOUNDATION RECOMMENDATIONS

Based on the finding of our geotechnical exploration, shallow foundations are suitable for the support of the planned structures. Due to the presence of existing fills and expansive soils, measures are recommended in the following sections of this report to be implemented for the use of shallow foundations. Suitable foundations for the building include shallow footings

combined with floor slab on grade underlain by an 18-inch thick layer of non-to-low expansive engineered fill, or 18 inches of lime-treated fill materials. The building may also be supported on a shallow structural mat foundation (either a steel reinforced structural mat, or post-tensioned structural mat) designed to accommodate movement of near surface expansive soils, which may or may not be constructed directly upon properly moisture conditioned on-site expansive soils.

4.1 SHALLOW FOOTINGS COMBINED WITH FLOOR SLAB-ON-GRADE

For the foundation of the proposed building, shallow footings combined with floor slab-on-grade are suitable. Building floor slab on grade should be underlain by an 18-inch thick layer of non-to-low expansive engineered fill. Provide minimum footing dimensions as follows in the Table 4.1-1 below.

TABLE 4.1-1: Minimum Footing Dimensions

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

* below lowest adjacent pad grade

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

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

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

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

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

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

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

Provided our report recommendations are followed and given the proposed construction (Section 1.3), we estimate total settlements to be less than approximately 1-inch, and differential settlement to be less than ½ inch.

4.2 MAT FOUNDATION DESIGN

4.2.1 Steel Reinforced Mat Foundation

Mat foundation design should consider total and differential settlement. As a minimum, to address potential differential movement and add stiffness to the mat, we recommend the mat be designed to cantilever 6 feet at the perimeter, and interior free span of 20 feet. Also, we recommend that mat be designed to have a minimum thickness of 12 inches. The mat should be designed to accommodate total vertical settlement of 1½-inch and differential settlement of ¾-inch over similarly loaded elements, over a horizontal distance of 30 feet.

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

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

4.2.2 Post Tensioned Mat Foundation

The proposed building may be supported on post-tensioned mat foundations. Based upon the existing soil conditions, and using the 2004 (Third Edition) Post-Tensioned Institute (“Design of Post-Tensioned Slabs-on-Ground” manual to develop our soil parameters, we recommend the following soil criteria.

TABLE 4.2.2-1: Post-Tensioned Mat Design Recommendations

CONDITION	CENTER LIFT	EDGE LIFT
Edge Moisture Variation Distance, e_m (feet)	7.5	4.0
Differential Soil Movement, y_m (inches)	0.5	1.3

PT mats may be designed for an average allowable bearing pressure of up to 1,000 pounds per square foot (psf) for dead-plus-live loads with maximum localized bearing pressures of 1,500 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for wind or seismic loads.

We recommend that PT mats be approximately 12 inches thick or greater and have a thickened edge at least 2 inches greater than the mat thickness. Also, we recommend that mat be designed to have a minimum thickness of 12 inches. The mat should be designed to accommodate total vertical settlement of 1½-inch and differential settlement of ¾-inch over similarly loaded elements, over a horizontal distance of 30 feet.

4.2.3 Subgrade Treatment for Mat Foundations

The subgrade material under the structural mat foundations should be uniform as discussed in Section 6.7. The pad subgrade should be moisture conditioned to a moisture content of at least 4 percentage points above optimum. The subgrade should be thoroughly soaked and approved by the Geotechnical Engineer prior to placing the reinforcement or tendons and should not be allowed to dry prior to concrete placement.

A tough, water vapor retarding membrane should be installed below the mats to reduce moisture condensation under floor coverings. The vapor retarder should meet ASTM E 1745 – 97 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.

5.0 SLABS-ON-GRADE

Provided the expansive soil is mitigated as recommended in Section 6.7, the proposed building can incorporate interior slab-on-grade first floor.

5.1 INTERIOR CONCRETE FLOOR SLABS

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

1. Provide a minimum concrete thickness of 5 inches.
2. Place minimum steel reinforcing of No. 3 rebar on 18-inch centers each way within the middle third of the slab to help control the width of shrinkage cracking that inherently occurs as concrete cures.

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

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

1. Construct a moisture retarder system directly beneath the slab on-grade that consists of the following:
 - a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, "Standard Specification for Plastic Water Vapor Retarders

used in Contact with Soil or Granular Fill under Concrete Slabs”. The vapor retarder should be **underlain by**

- b. 4 inches of clean crushed rock. Crushed rock should have 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 Sieve.
2. Use a concrete water-cement ratio for slabs-on-grade of no more than 0.50.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.
4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specified by the structural engineer.

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

5.2 EXTERIOR CONCRETE FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum section of 6 inches of concrete over 4 inches of aggregate base. Compact the aggregate base to at least 90 percent relative compaction (ASTM D1557). Place rebar within the middle third of the slab to help control the width and offset of cracks. Exterior slabs should be constructed with thickened edges extending at least beneath the crushed rock or gravel into compacted soil to reduce water infiltration. Slabs should slope away from the buildings at a slope of at least 2 percent to prevent water from flowing toward the building. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

6.0 EARTHWORK RECOMMENDATIONS

The relative compaction and optimum moisture content of soil and aggregate base referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal flexing or pumping, as observed by an ENGEO representative. As used in this report, the term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry. We define “structural areas” in Section 5 of this report as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

6.1 General Site Clearing and Recycled Materials

Areas to be developed should be cleared of surface and subsurface deleterious materials, including existing building foundations, slabs, buried utility and irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots. Clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in Section 6.7. Retain ENGEO to observe and test backfilling. Following clearing, strip the site to remove surface organic materials. Strip organics from the ground surface to a depth of

at least 2 to 3 inches below the surface. Remove strippings from the site or, if considered suitable by the landscape architect and owner, use them in landscape fill.

If recycled pavement or construction materials are utilized at the site, we recommend full disclosure be provided. As a minimum, disclosed information should include the presence of asphaltic concrete and Portland cement concrete fill materials at the site. The reuse of the properly crushed asphalt concrete is considered suitable from a geotechnical standpoint, provided it meets the Selection of Materials recommendations in Section 6.2. 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 until used within street areas below subgrade. The asphaltic concrete and aggregate base should be thoroughly mixed and placed as engineered fill below street or parking lot subgrade elevations. Reuse of existing paving materials as engineered fill within future streets may increase the R-value of the subgrade soil, add a “green” recycling component to the project and also save costs to export and dispose of these materials. Reuse of these materials as part of the anticipated planned pavement section or placement within the future building pads is not recommended. Reuse of concrete materials from the existing buildings as a low-expansive engineered fill material may be considered. Reinforcing steel should be removed and the concrete materials should be reduced/broken down (not pulverized) to meet the following gradation requirements.

6.2 SELECTION OF MATERIALS

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils, we anticipate the site soils are suitable for use as engineered fill provided they are broken down to 6 inches or less in size. Other 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.

6.3 EXISTING UNDOCUMENTED FILL OVEREXCAVATION AND REMOVAL

Remove existing fill to competent native soil, as evaluated by ENGEO. The lateral extent and depth of fill is expected to vary. Consult the exploration logs in Appendix B for fill depths at specific locations. Following removal, place fill in accordance with subsequent sections.

6.4 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. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather.
2. Mixing with drier materials.
3. Mixing with a lime, lime-flyash, or cement product; or
4. Stabilizing with aggregate, geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated by ENGEO prior to implementation.

6.5 DIFFERENTIAL FILL THICKNESS

During removal of existing fills or encountered substructures, and depending upon the depth of excavation and planned cuts and fills, a differential fill thickness situation may occur that could adversely impact the performance of the hotel foundation systems. For subexcavation activities that create a differential fill thickness across an individual 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 in fill thickness of up to 5 feet is acceptable across a building footprint. For a differential fill thickness exceeding 5 feet across a footprint, we recommend performing subexcavation activities to bring this vertical distance to within the 5-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.

6.6 BUILDING PAD TREATMENT

As described in Section 3.2, near-surface potentially expansive clay layers will be encountered during grading. To improve foundation performance for the planned hotel structures, we recommend that the near-surface soils comprise uniform engineered fill. For a mat foundation system or footing with slab-on-grade foundation, we recommend that the proposed building be situated on a layer of engineered fill material that extends at least 2 feet below slab subgrade soil level and 5 feet beyond the building footprint. For a footing with slab-on-grade foundation, we recommend the upper 18 inches of engineered fill consist of a low-expansive import material (Plasticity Index less than 12) or soil that is chemically treated with 3 to 5 percent high calcium lime. The amount of lime required should be based upon an assumed 125 pounds per cubic foot (pcf) for the soil density.

6.7 FILL COMPACTION

6.7.1 Grading in Structural Areas

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

TABLE 6.7.1-1: Fill Compaction and Moisture Content Recommendations

MATERIALS	TEST PROCEDURES	REQUIRED RELATIVE COMPACTION (%)	REQUIRED MOISTURE CONTENT (PERCENTAGE POINTS ABOVE OPTIMUM MOISTURE CONTENT)
Expansive (PI>12)	ASTM D-1557	Not less than 87 to 92	Not less than 3
Low-expansive (PI<12)	ASTM D-1557	Not less than 90	Not less than 1

Compact the upper 6 inches of pavement subgrade to a minimum of 90 percent relative compaction for expansive subgrade conditions and minimum 95 percent relative compaction for low-expansive subgrade conditions. Compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to or slightly above the optimum moisture content prior to compaction.

6.7.2 Underground Utility Backfill

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe bedding materials. Place and compact trench backfill in structural areas in accordance with Section 6.7.1. Where utility trenches cross perimeter building foundations, backfill with native clay soil for pipe bedding and backfill for a distance of 2 feet on each side of the foundation. This will help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the building. As an alternative, a sand cement slurry (minimum 28-day compressive strength of 500 psi) may be used in place of native clay soil. Jetting of backfill is not an acceptable means of compaction. We may allow thicker loose lift thicknesses based on acceptable density test results, where increased effort is applied to rocky fill, or for the first lift of fill over pipe bedding.

6.7.3 Landscape Fill

Process, place and compact fill in accordance with Section 6.7.1 except compact to at least 85 percent relative compaction (ASTM D1557).

6.8 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. The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements.

6.9 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical. The latest California Building Code Section 1804.3 specifies minimum slopes of 5 percent away from foundations. Where development conditions restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following:

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.
2. Do not allow water to pond near foundations, pavements, or exterior flatwork.

7.0 RETAINING WALLS

Retaining walls may be supported on continuous footings designed in accordance with recommendations presented in Section 4.1, except the minimum embedment depth should be increased to 18 inches below lowest adjacent soil grade.

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

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

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

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

1. A minimum 12-inch-thick layer of Class 2 Permeable Filter Material (Caltrans Specification 68-2.02F) placed directly behind the wall, or
2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the $\frac{3}{4}$ -inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

- Place the rock drain directly behind the walls of the structure.
- Extend rock drains from the wall base to within 12 inches of the top of the wall.
- Place a minimum of 4-inch-diameter perforated pipe (glued joints and end caps) at the base of the wall, inside the rock drain and fabric, with perforations placed down.
- Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use. Backfill behind retaining walls should be placed and compacted in accordance with

Section 5.8.1. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

8.0 PAVEMENT DESIGN

8.1 FLEXIBLE PAVEMENTS

Based on our field exploration and laboratory testing, it is our opinion that an R-value of 5 is applicable for design. Using estimated traffic indices for various pavement loading requirements, we developed the following recommended pavement sections using Topic 633 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below.

TABLE 8.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX (TI)	SECTION	
	AC (INCHES)	AB (INCHES)
5	3	10
6	3 ½	13

The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies.

8.2 RIGID PAVEMENTS

Use concrete pavement sections to resist heavy loads and turning forces in areas such as fire lanes or trash enclosures. Final design of rigid pavement sections, and accompanying reinforcement, should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements:

- Use a minimum section of 6 inches of Portland Cement concrete over 12 inches of Caltrans Class 2 Aggregate Base.
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

8.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with Section 6.7.1. Aggregate Base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02a of the latest Caltrans Standard Specifications.

8.4 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that

are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers. If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, then the cutoff barrier may be eliminated.

9.0 EXCAVATION AND SHORING

All excavations, including utility trenches, should be properly excavated, and shored as applicable, to create a stable and safe condition. It is the responsibility of the Contractor to provide such stable, safe trench and construction slope conditions and to follow OSHA safety requirements. Since excavation procedures may be very 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.

10.0 STORMWATER INFILTRATION

If bioretention areas are implemented, we recommend that, when practical, they 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 site improvements can either:

1. Be constructed with structural side 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, one of the following options should be followed.

- We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.
- Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

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 reduce the exposure time such that the improvements are not detrimentally impacted.

10.1 LANDSCAPING CONSIDERATION

As the near-surface soils are highly expansive, we recommend greatly restricting the amount of surface water infiltration near structures, pavements, flatwork, and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements.
- Using low precipitation sprinkler heads.
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system.
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements.
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements.
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that these items be incorporated into the landscaping plans.

11.0 GROUND HEAT-EXCHANGE (GHX)

The site is considered to be suitable for using a Ground Heat-Exchange (GHX) system to achieve energy savings and to potentially eliminate the need for outdoor air conditioner units, if desired. For the thermal properties of the soil and groundwater conditions at the site, a GHX system would likely be well suited and could be implemented on select buildings, or integrated into a project-wide system with service laterals to each unit and an equipment vault to serve the development. As project planning progresses into architectural design, we can meet with you, your architect, and your MEP designer to further assess and develop GHX energy saving opportunities and efficiencies.

12.0 CONSTRUCTION MONITORING

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

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

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

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

13.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the Hilton Garden Inn project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks 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 report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify ENGEО immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, notify the proper regulatory officials immediately. This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEО. Such authorization is essential because it requires ENGEО 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 ENGEО's documents. Therefore, ENGEО must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEО's scope of services does not include

onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

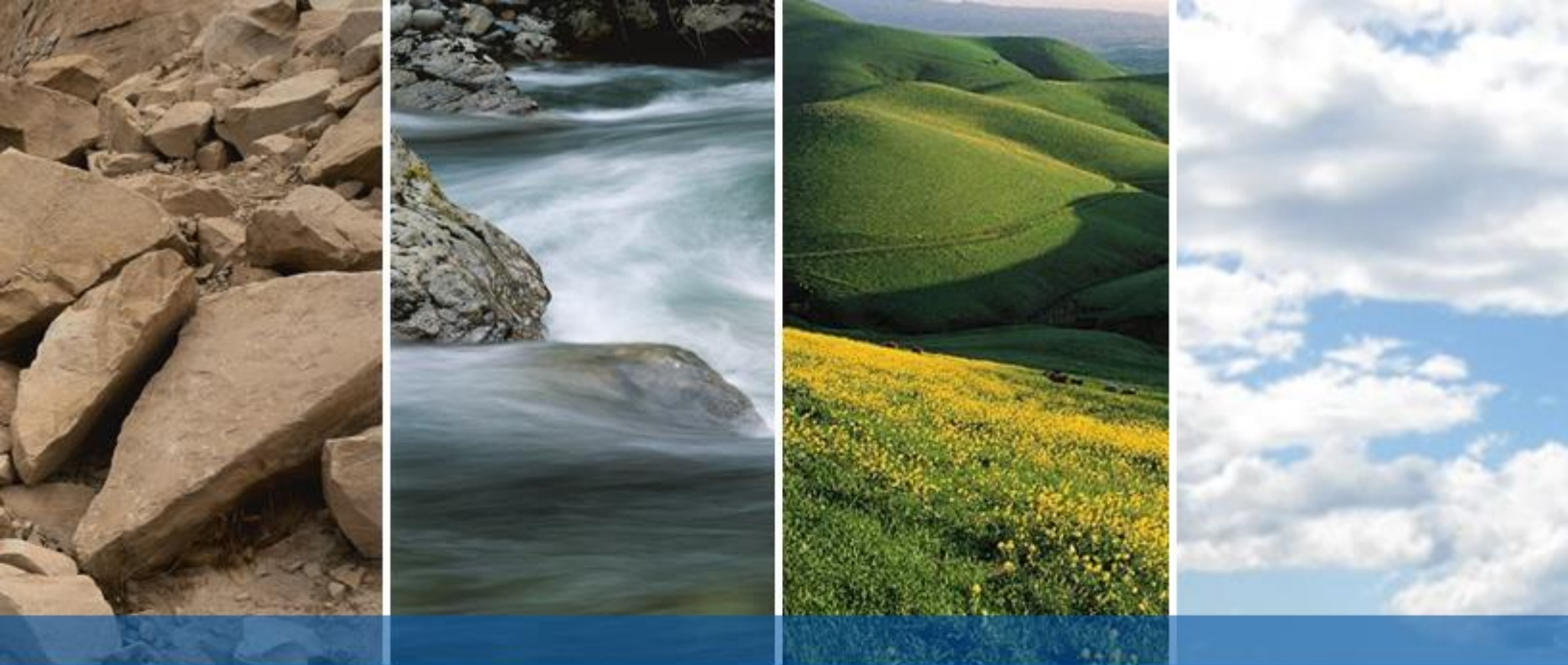
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FIGURES

FIGURE 1: Vicinity Map

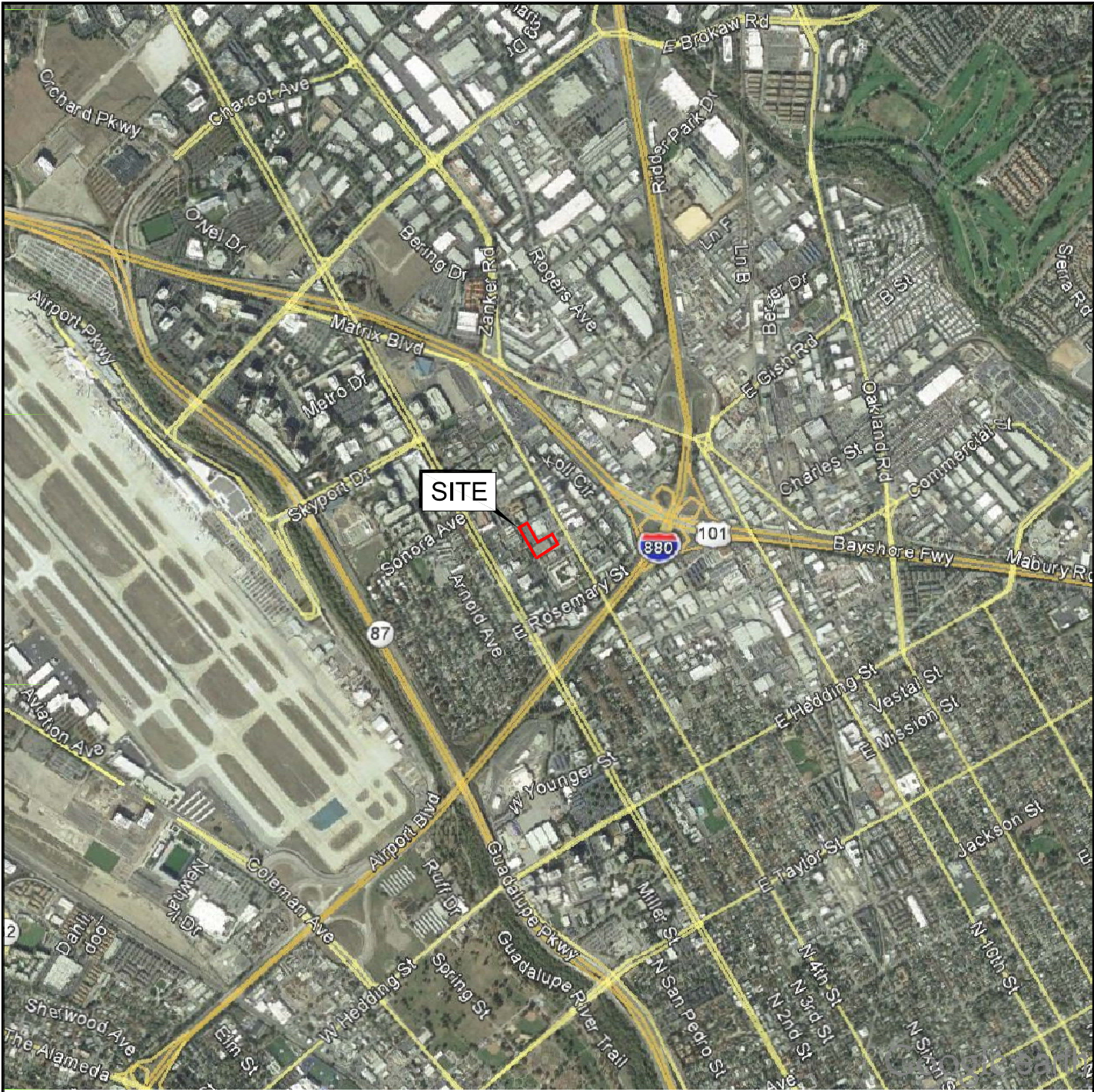
FIGURE 2: Site Plan

FIGURE 3: Regional Geologic Map

FIGURE 4: Seismic Hazard Zone Map

FIGURE 5: Regional Faulting and Seismicity Map

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BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

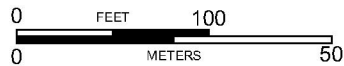
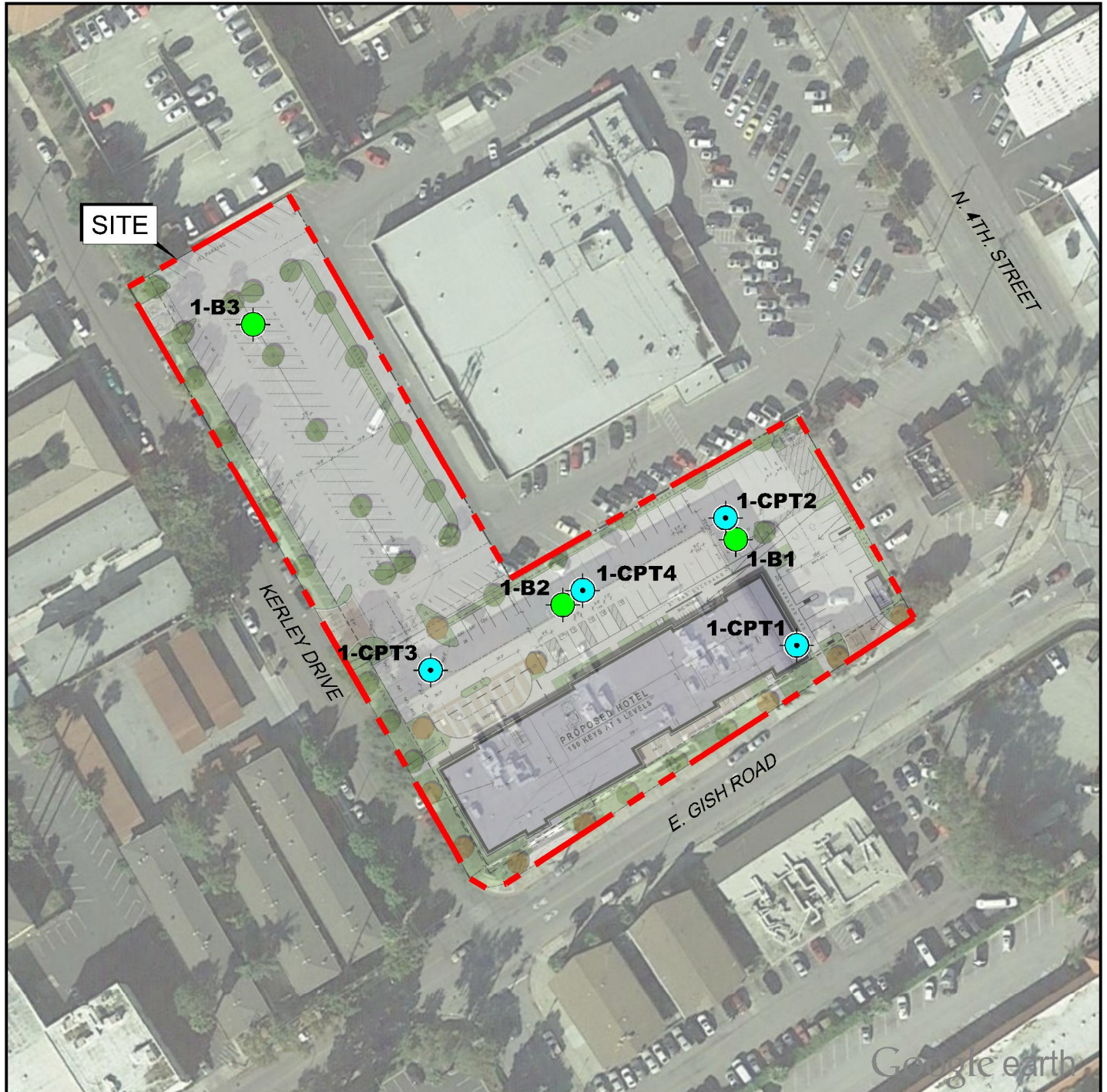


VICINITY MAP
 111-137 EAST GISH ROAD
 SAN JOSE, CALIFORNIA

PROJECT NO.: 13855.000.000	
SCALE: AS SHOWN	
DRAWN BY: GLJ	CHECKED BY: TPB

FIGURE NO.
1

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EXPLANATION

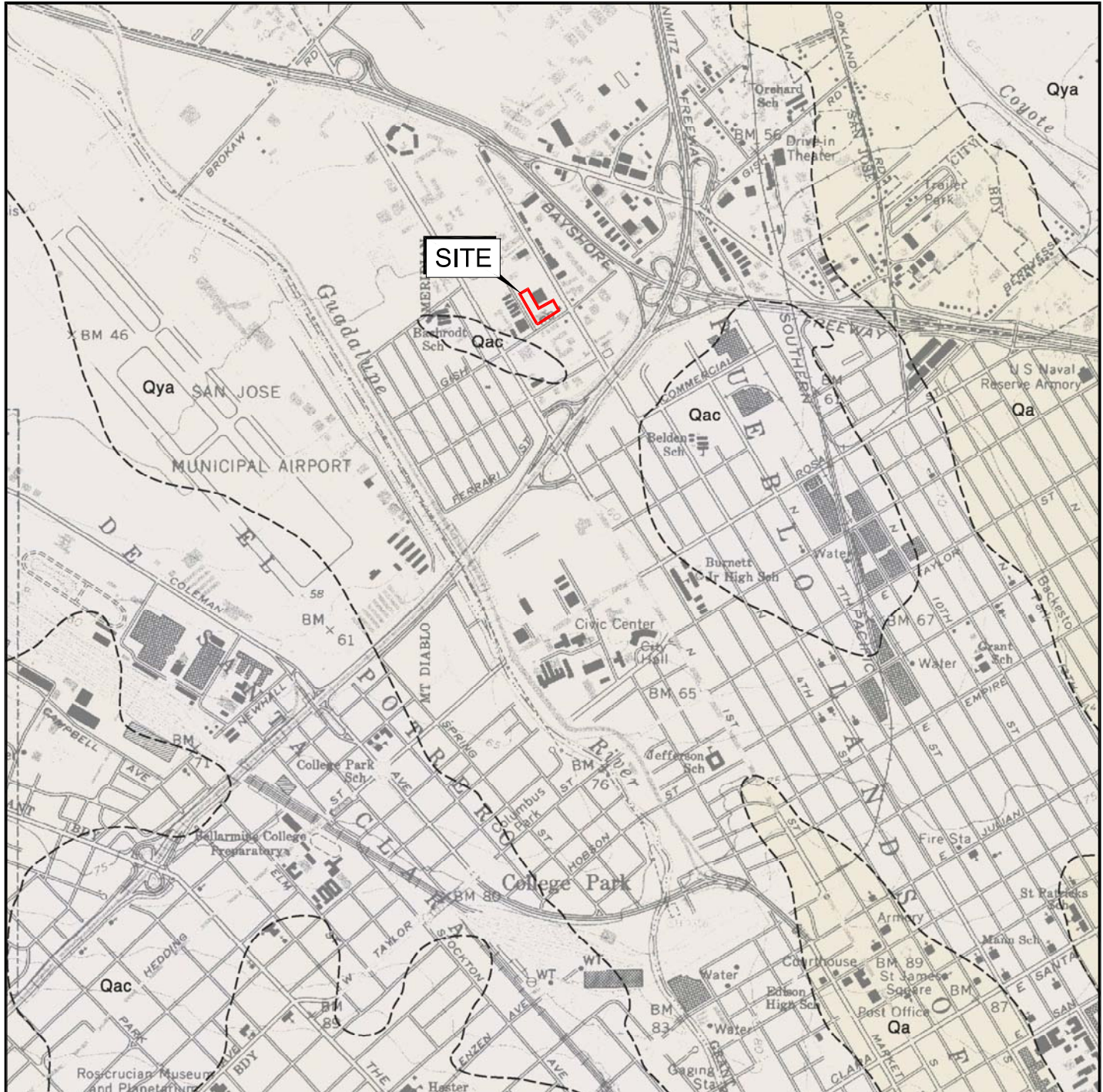
ALL LOCATIONS ARE APPROXIMATE

- 1-B3** BORING (ENGEO, 2017)
- 1-CPT4** CONE PENETRATION TEST (ENGEO, 2017)

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE, ARCHITECTS ORANGE, 2017

	SITE PLAN 111-137 EAST GISH ROAD SAN JOSE, CALIFORNIA	PROJECT NO.: 13855.000.000 SCALE: AS SHOWN DRAWN BY: GLJ	2
		CHECKED BY: TPB	
			FIGURE NO.

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EXPLANATION

- GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED
- ▲-▲- FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT

- Qa ALLUVIAL GRAVEL, SAND SILT, AND CLAY
- Qya ALLUVIAL SAND, FINE-GRAINED, SILT, AND GRAVEL
- Qac SILTY CLAY AND ORGANIC CLAY, FOSSILIFEROUS



BASE MAP SOURCE: DIBBLEE, 2007



REGIONAL GEOLOGY MAP
 111-137 EAST GISH ROAD
 SAN JOSE, CALIFORNIA

PROJECT NO.: 13855.000.000

SCALE: AS SHOWN

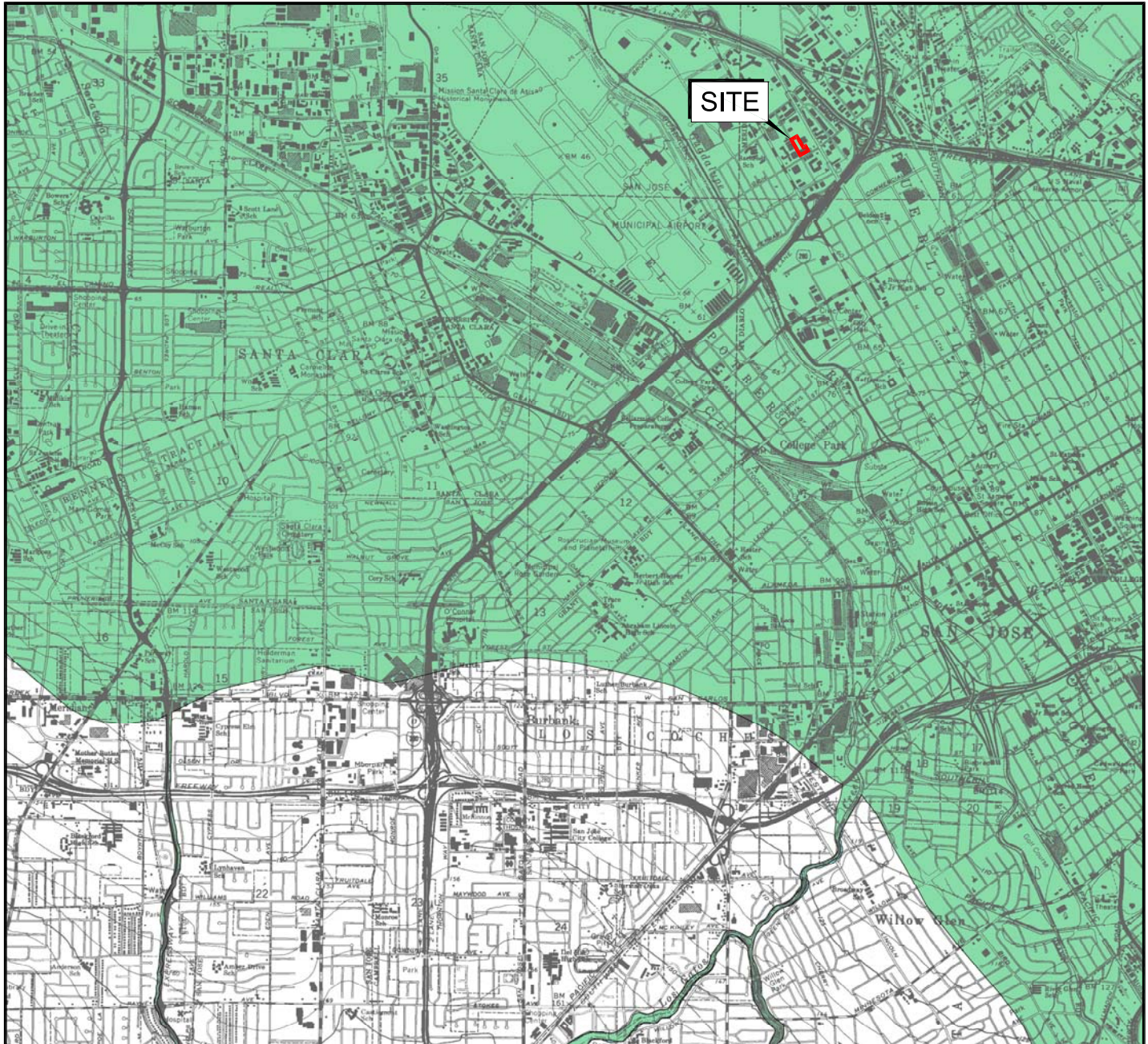
DRAWN BY: GLJ

CHECKED BY: TPB

FIGURE NO.

3

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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



BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2002

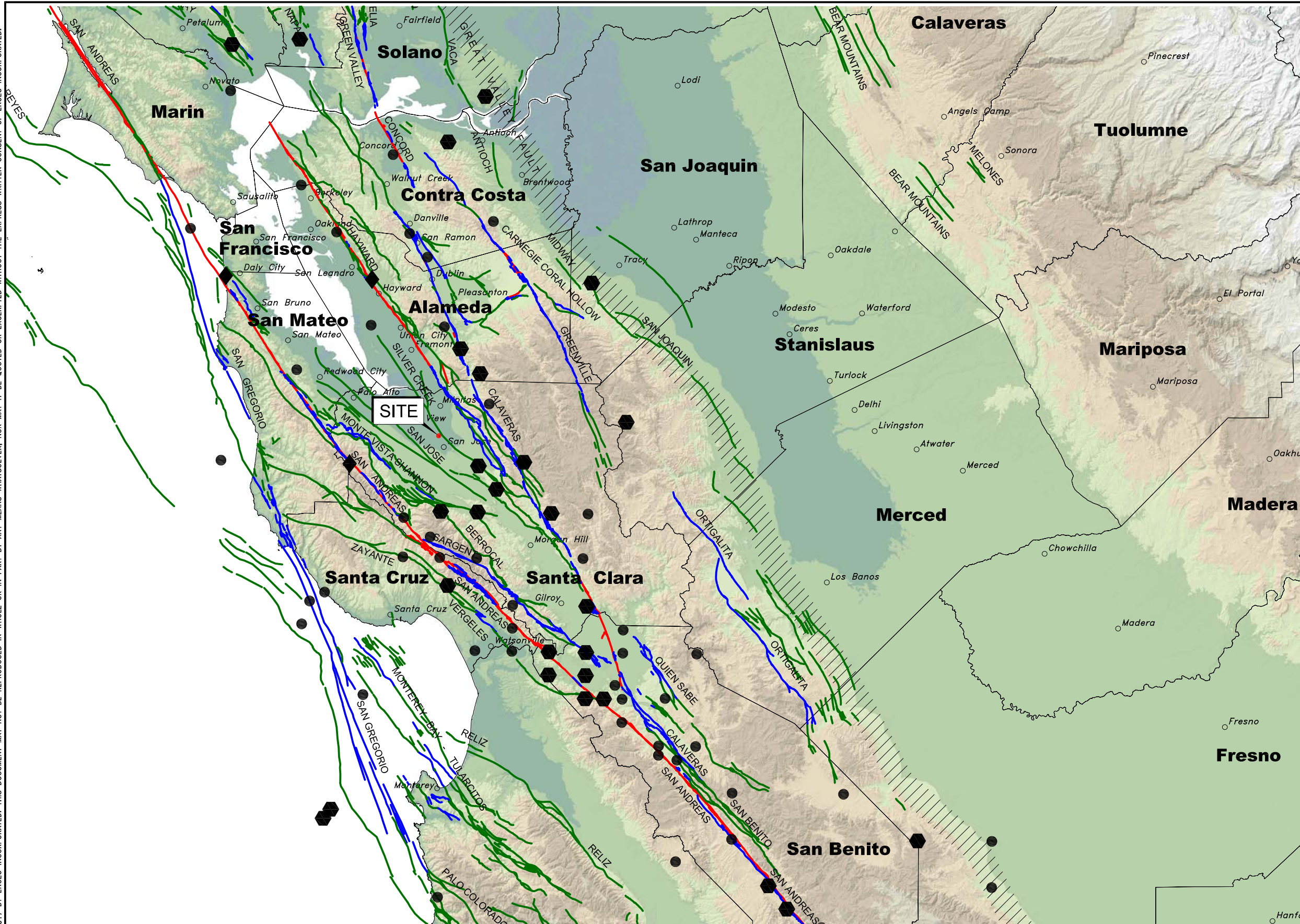


SEISMIC HAZARD ZONE MAP
 111-137 EAST GISH ROAD
 SAN JOSE, CALIFORNIA

PROJECT NO.: 13855.000.000	
SCALE: AS SHOWN	
DRAWN BY: GLJ	CHECKED BY: TPB

FIGURE NO.
4

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EXPLANATION

- ◆ MAGNITUDE 7+
- ⬡ MAGNITUDE 6-7
- MAGNITUDE 5-6
- HISTORIC FAULT
- HOLOCENE FAULT
- QUATERNARY FAULT
- ▨ HISTORIC BLIND THRUST FAULT ZONE

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)



REGIONAL FAULTING AND SEISMICITY
 111-137 EAST GISH ROAD
 SAN JOSE, CALIFORNIA

PROJECT NO.: 13855.000.000	FIGURE NO.
SCALE: AS SHOWN	5
DRAWN BY: GLJ	



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

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

CONSISTENCY

<u>SILTS AND CLAYS</u>	<u>STRENGTH*</u>
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



LOG OF BORING 1-B1

Geotechnical Exploration
 Hilton Garden Inn
 San Jose, California
 13855.000.000

DATE DRILLED: 3/28/2017
 HOLE DEPTH: Approx. 46½ ft.
 HOLE DIAMETER: 4.0 in.
 SURF ELEV (NAVD88): Approx. 52 ft.

LOGGED / REVIEWED BY: S. Barua / TB
 DRILLING CONTRACTOR: Pitcher Drilling
 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			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			2-inches AC over 4-inches AB										
50			SANDY SILT (ML), yellowish brown, slightly moist, [FILL] FAT CLAY (CH), dark brown mottled with olive, very stiff, moist, high plasticity			19						3.5*	
5			Becomes dark brown			20	59	23	36	98	29	88.7	2.5*
45			Becomes light gray mottled with reddish brown, stiff to very stiff			13							2.0*
10			Becomes light gray with yellowish brown, soft, wet			12					32.4	92.4	0.41
40			CLAYEY SAND (SC), dark reddish brown, medium dense, wet, medium-grained sand			10				23	27		
15			FAT CLAY (CH), dark gray mottled with reddish brown, stiff, wet, high plasticity			21	52	20	32		25.1	100	1.25*
35													
20			Becomes dark grayish brown			7					31.2	90.9	1.5*
30													
25													

LOG - GEOTECHNICAL W/ELEV. 13855_BORINGS_GINT.GPJ ENGEO INC.GDT 4/18/17



LOG OF BORING 1-B1

Geotechnical Exploration
Hilton Garden Inn
San Jose, California
13855.000.000

DATE DRILLED: 3/28/2017
HOLE DEPTH: Approx. 46½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 52 ft.

LOGGED / REVIEWED BY: S. Barua / TB
DRILLING CONTRACTOR: Pitcher Drilling
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			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
25			Becomes dark gray, stiff to very stiff			16							1.5*
30			Becomes medium stiff to stiff			15				48.6			0.75*
35			Becomes dark grayish green, stiff			19							1.5*
40			CLAYEY GRAVEL WITH SAND (GC), dark gray mottled with reddish brown, dense to very dense, wet, subrounded gravel, fine- to coarse-grained sand			51			13				
45			Becomes dark grayish brown, medium dense			17							
			End of boring at 46½ feet below grade. Groundwater not encountered due to drilling method.										

LOG - GEOTECHNICAL W/ELEV. 13855_BORINGS_GINT.GPJ ENGEO INC.GDT 4/18/17



LOG OF BORING 1-B2

Geotechnical Exploration
Hilton Garden Inn
San Jose, California
13855.000.000

DATE DRILLED: 3/28/2017
HOLE DEPTH: Approx. 39 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 50 ft.

LOGGED / REVIEWED BY: S. Barua / TB
DRILLING CONTRACTOR: Pitcher Drilling
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			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			4-inches of AC										
			SANDY SILT WITH GRAVEL (ML), dark yellowish brown, stiff to very stiff, moist, subrounded gravel, brick and concrete fragments [FILL]			16							2*
5	45		FAT CLAY (CH), dark brown mottled with reddish brown, stiff to very stiff, moist, high plasticity			20			87	32.1	86.7		2.5*
			Becomes dark yellowish brown mottled with reddish brown, stiff			13				38			1.5*
10	40		Becomes dark yellowish brown, medium stiff, wet			9				36.1	99.43		0.97
15	35		Becomes dark olive brown, very stiff			23							3.25*
20	30		Becomes grayish brown, stiff to very stiff			25							2.25*
25	25												

LOG - GEOTECHNICAL W/ELEV. 13855_BORINGS_GINT.GPJ ENGEO INC.GDT 4/18/17



LOG OF BORING 1-B2

Geotechnical Exploration
Hilton Garden Inn
San Jose, California
13855.000.000

DATE DRILLED: 3/28/2017
HOLE DEPTH: Approx. 39 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 50 ft.

LOGGED / REVIEWED BY: S. Barua / TB
DRILLING CONTRACTOR: Pitcher Drilling
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			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
28 - 32	22 - 26		Becomes dark gray, medium stiff to stiff			14				41.2	99.7	1.01	
32 - 36	16 - 20		Becomes dark gray mottled with blackish brown, stiff to very stiff			14						1*	
36 - 38	14 - 16		LEAN CLAY (CL), dark gray, stiff, wet, medium plasticity			40						1.25*	
38 - 39	13 - 14		SILTY SAND WITH GRAVEL (SM), dark gray and reddish brown, dense, wet, fine to coarse gravel			38			16				
			Bottom of boring at 39 feet below grade. No groundwater encountered due to drilling method.										



LOG OF BORING 1-B3

Geotechnical Exploration
 Hilton Garden Inn
 San Jose, California
 13855.000.000

DATE DRILLED: 3/28/2017
 HOLE DEPTH: Approx. 11½ ft.
 HOLE DIAMETER: 4.0 in.
 SURF ELEV (NAVD88): Approx. 50 ft.

LOGGED / REVIEWED BY: S. Barua / TB
 DRILLING CONTRACTOR: Pitcher Drilling
 DRILLING METHOD: SSA, Switch to Mud
 HAMMER TYPE: 140 lb. Auto Trip

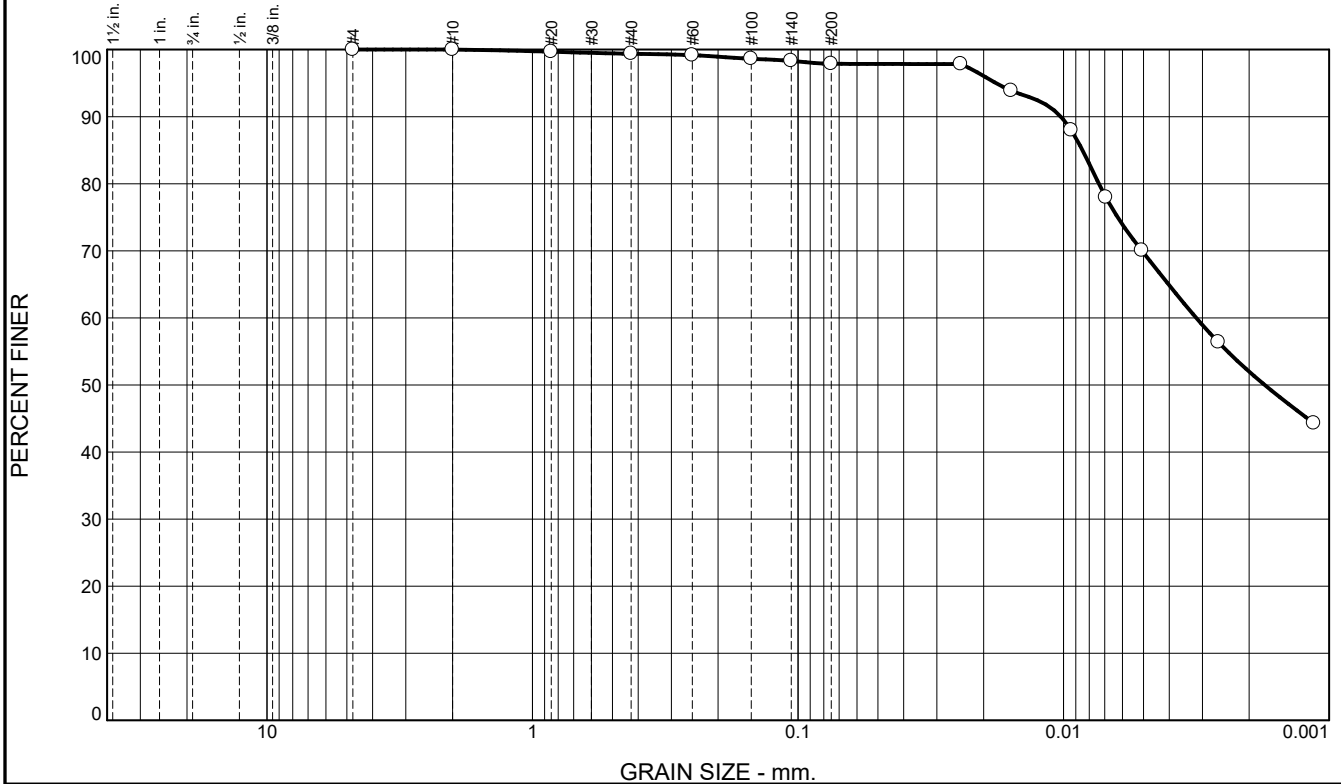
Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			3-inches AC over 5-inches AB										
			LEAN CLAY (CL), dark yellowish brown, very soft, moist			7							0*
			FAT CLAY (CH), olive brown, stiff to very stiff, moist			15							2*
5	45		Becomes reddish brown, very soft to soft, wet			8							0*
10	40		Becomes dark grayish brown, stiff			12							0.25*
			Bottom of boring at 11½ feet below grade. Groundwater encountered at 7 feet below grade during drilling.										



APPENDIX B

LABORATORY TEST DATA

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.6	1.5	45.9	52.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	100.0		
#20	99.7		
#40	99.4		
#60	99.2		
#100	98.6		
#140	98.3		
#200	97.9		
0.0244 mm.	97.8		
0.0157 mm.	93.9		
0.0093 mm.	88.0		
0.0069 mm.	78.0		
0.0051 mm.	70.0		
0.0026 mm.	56.4		
0.0011 mm.	44.3		

Soil Description

See exploration log

Atterberg Limits

PL= 23 LL= 59 PI= 36

Coefficients

D₉₀= 0.0102 D₈₅= 0.0085 D₆₀= 0.0032
 D₅₀= 0.0017 D₃₀= D₁₅=
 D₁₀= C_u= C_c=

Classification

USCS= CH AASHTO= A-7-6(40)

Remarks

ASTM D422
 ASTM D4318, wet method

* (no specification provided)

Sample Number: 1-B1 @ 5-5.5

Depth: 5-5.5 ft

Date: 4-5-17



Client: Westlake Urban
 Project: Hilton Garden Inn, San Jose

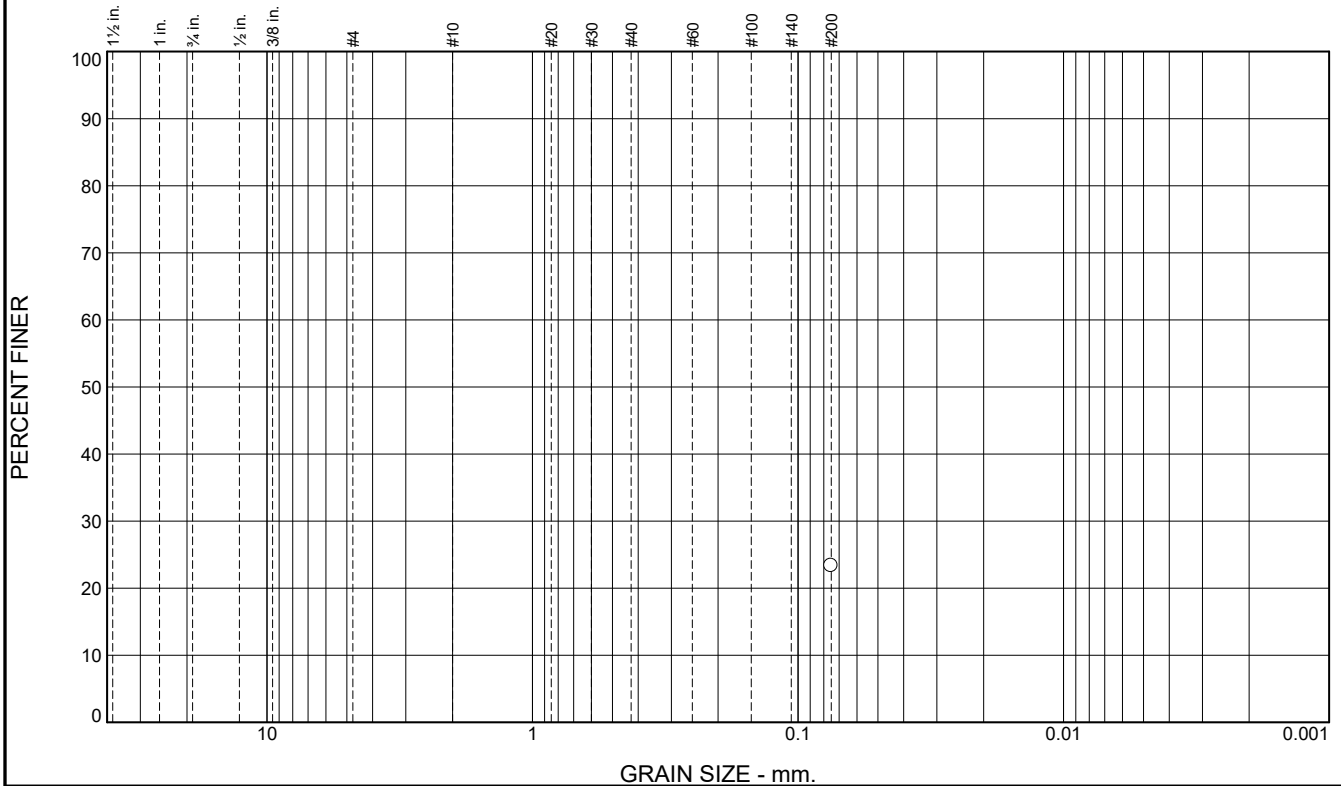
Project No: 13855.000.000

Figure

Tested By: I. McCauley

Checked By: K. Lecce

Particle Size Distribution Report



% Gravel		% Sand			% Fines	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
					23.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	23.4		

Soil Description

See exploration log

Atterberg Limits

PL= LL= PI=

Coefficients

D₉₀= D₈₅= D₆₀=
 D₅₀= D₃₀= D₁₅=
 D₁₀= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D1140

* (no specification provided)

Sample Number: 1-B1 @ 11.5-13

Depth: 11.5-13 ft

Date: 4-3-17



Client: Westlake Urban
Project: Hilton Garden Inn, San Jose

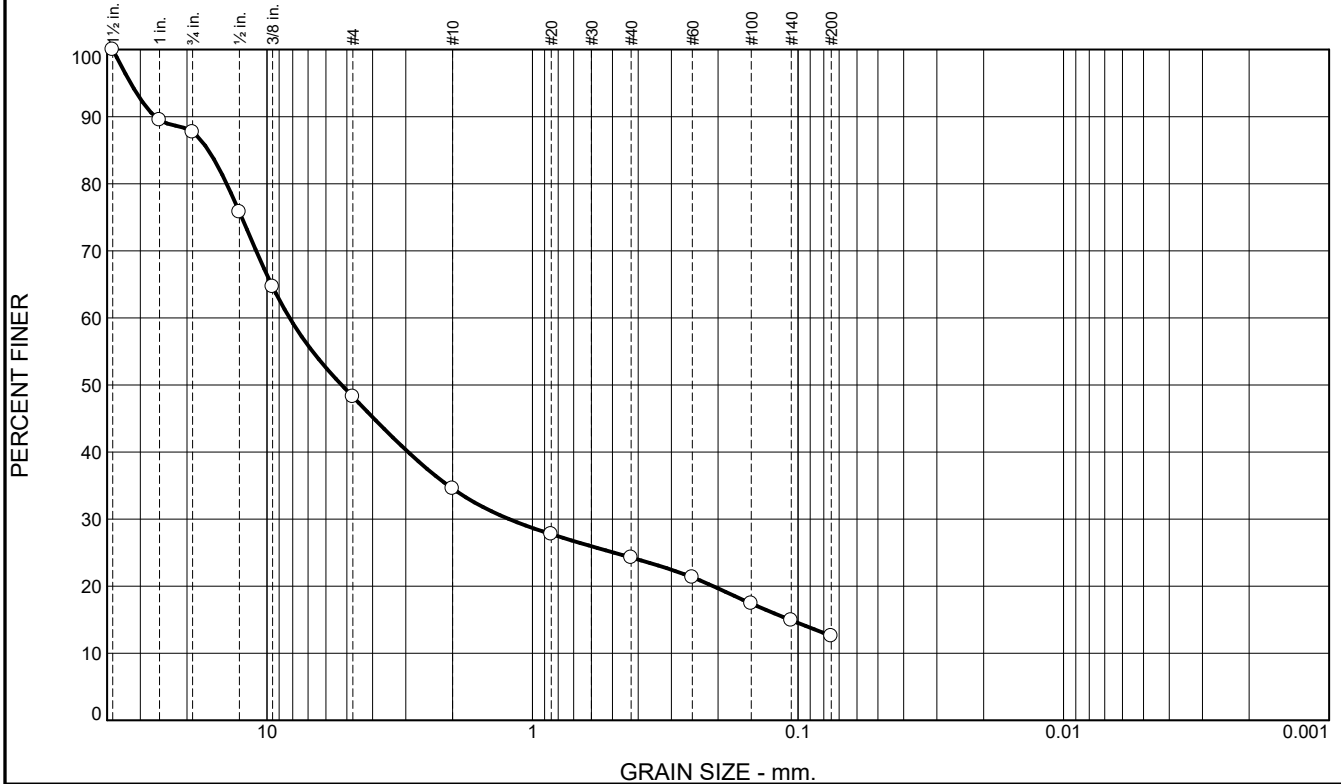
Project No: 13855.000.000

Figure

Tested By: K. Lecce

Checked By: I. McCauley

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	12.3	39.5	13.7	10.2	11.7	12.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	89.5		
3/4"	87.7		
1/2"	75.8		
3/8"	64.6		
#4	48.2		
#10	34.5		
#20	27.8		
#40	24.3		
#60	21.3		
#100	17.4		
#140	14.9		
#200	12.6		

Soil Description

See exploration log

Atterberg Limits

PL= LL= PI=

Coefficients

D₉₀= 26.3681 D₈₅= 16.6143 D₆₀= 8.2196
D₅₀= 5.2366 D₃₀= 1.2256 D₁₅= 0.1074
D₁₀= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D6913

* (no specification provided)

Sample Number: 1-B1 @ 40-41.5

Depth: 40-41.5 ft

Date: 4-3-17



Client: Westlake Urban
Project: Hilton Garden Inn, San Jose

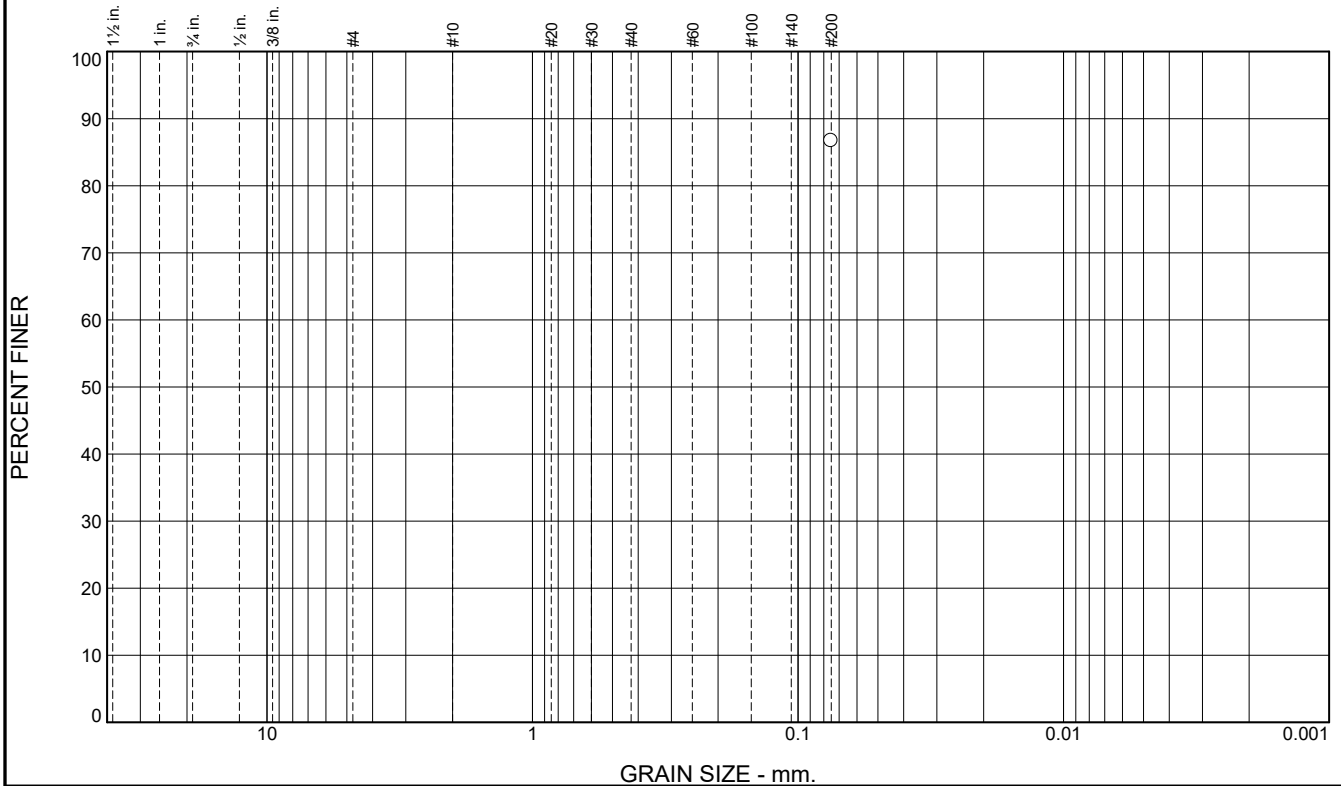
Project No: 13855.000.000

Figure

Tested By: K. Lecce

Checked By: I. McCauley

Particle Size Distribution Report



% Gravel		% Sand			% Fines	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
					86.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	86.7		

Soil Description

See exploration log

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₉₀= D₈₅= D₆₀=
 D₅₀= D₃₀= D₁₅=
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks

ASTM D1140

* (no specification provided)

Sample Number: 1-B2 @ 6-6.5

Depth: 6-6.5 ft

Date: 4-3-17



Client: Westlake Urban
Project: Hilton Garden Inn, San Jose

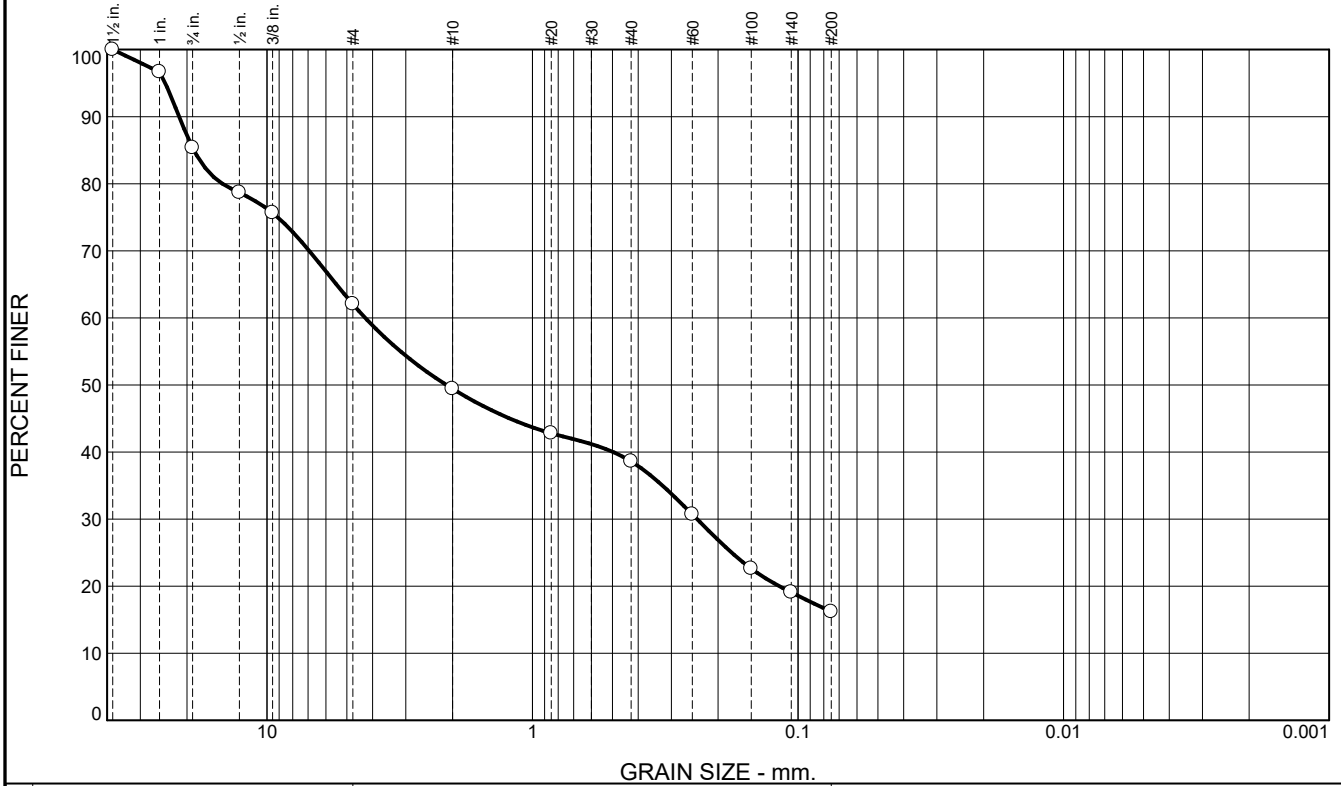
Project No: 13855.000.000

Figure

Tested By: K. Lecce

Checked By: I. McCauley

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	14.6	23.3	12.7	10.8	22.4	16.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2"	100.0		
1"	96.7		
3/4"	85.4		
1/2"	78.7		
3/8"	75.6		
#4	62.1		
#10	49.4		
#20	42.8		
#40	38.6		
#60	30.7		
#100	22.6		
#140	19.1		
#200	16.2		

Soil Description

See exploration log

Atterberg Limits
 PL= LL= PI=

Coefficients
 D₉₀= 21.4058 D₈₅= 18.8379 D₆₀= 4.2643
 D₅₀= 2.1091 D₃₀= 0.2400 D₁₅=
 D₁₀= C_u= C_c=

Classification
 USCS= AASHTO=

Remarks
 ASTM D6913

* (no specification provided)

Sample Number: 1-B2 @ 37.5-39

Depth: 37.5-39 ft

Date: 4-3-17



Client: Westlake Urban
 Project: Hilton Garden Inn, San Jose

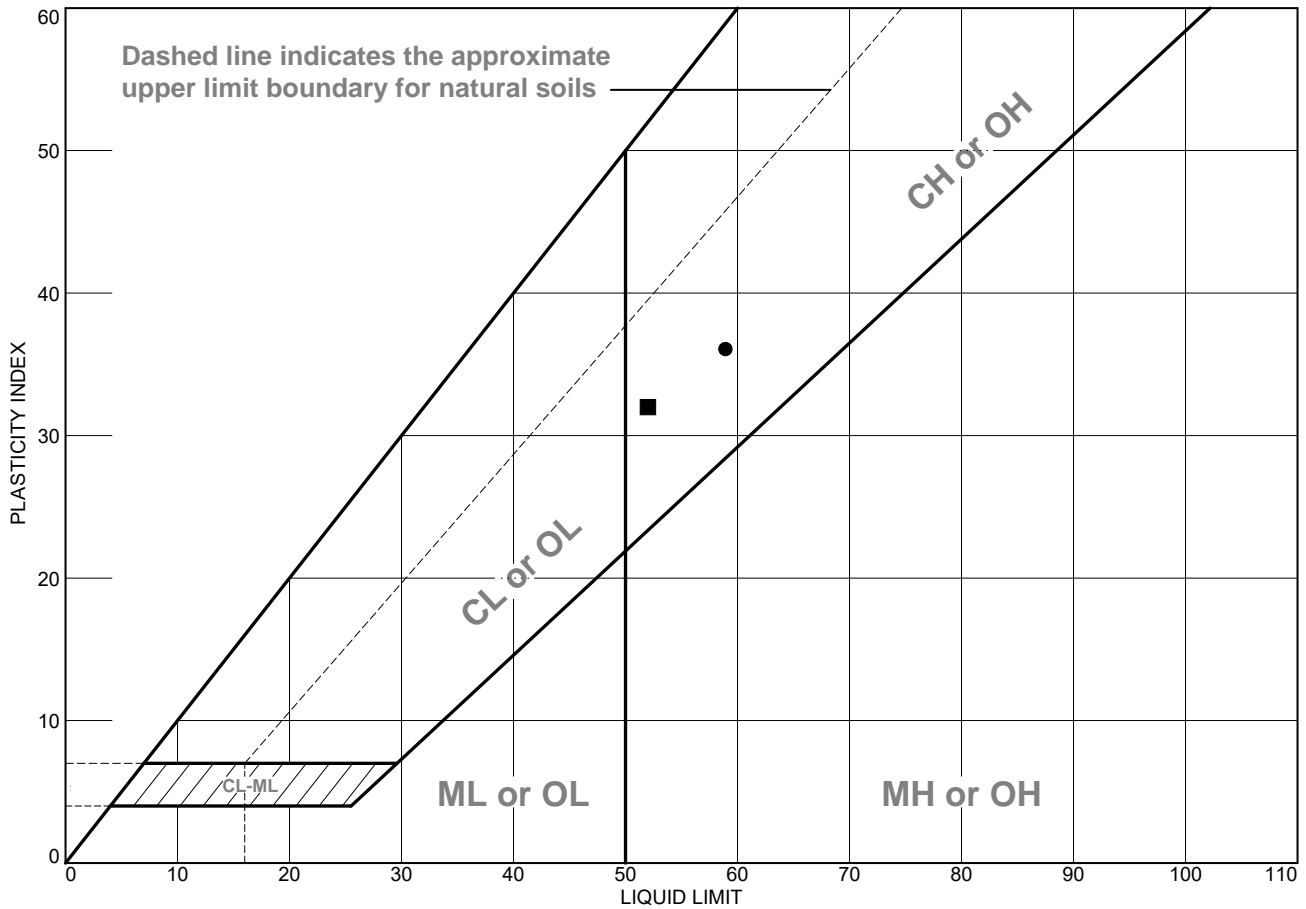
Project No: 13855.000.000

Figure

Tested By: K. Lecce

Checked By: I. McCauley

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration log	59	23	36	99.4	97.9	CH
■	See exploration log	52	20	32			

Project No. 13855.000.000 **Client:** Westlake Urban
Project: Hilton Garden Inn, San Jose
● Depth: 5-5.5 ft **Sample Number:** 1-B1 @ 5-5.5
■ Depth: 16-16.5 ft **Sample Number:** 1-B1 @ 16-16.5

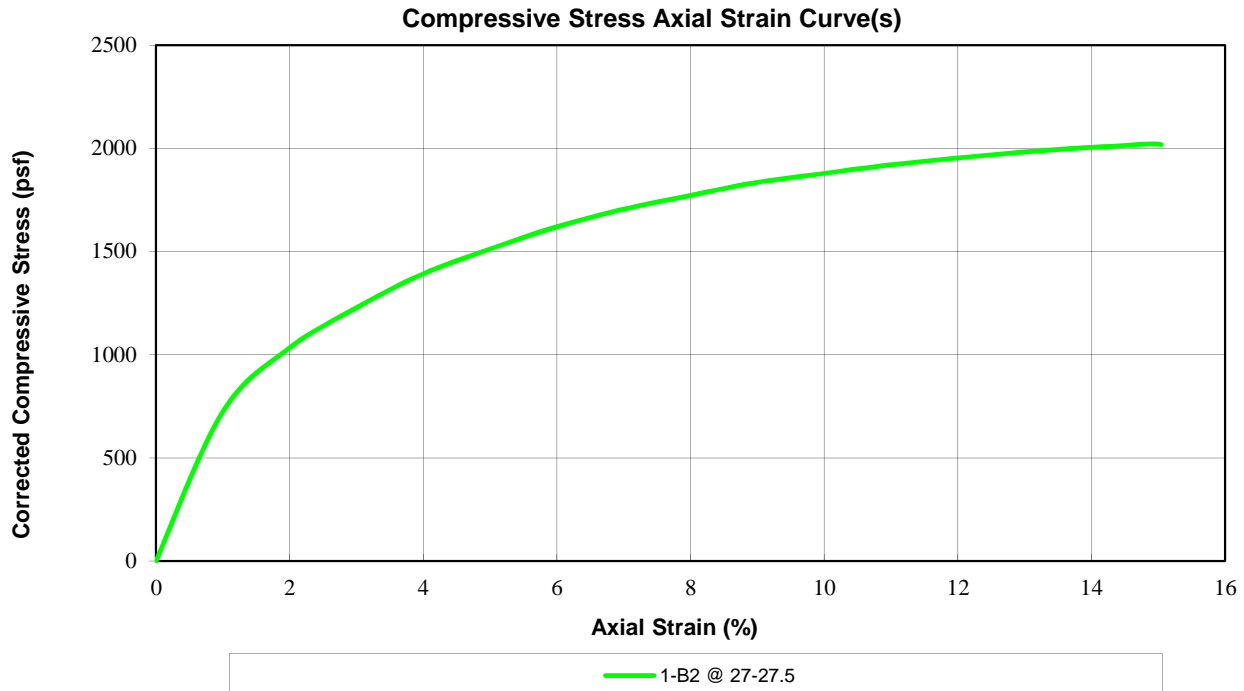
Remarks:
 ● ASTM D4318, wet method
 ASTM D422
 ■ ASTM D4318, wet method



Figure

Tested By: K. Lecce **Checked By:** I. McCauley

UNCONFINED COMPRESSION TEST REPORT (ASTM D2166)



SPECIMEN

BEFORE TEST 1-B2 @ 27-27.5

Moisture Content (%)	41.2
Dry Density (pcf)	81.2
Saturation (%)	99.7
Void Ratio	1.16
Diameter (in)	2.397
Height (in)	5.06
Height-To-Diameter Ratio	2.11

TEST DATA

Unconfined Compressive Strength (psf)	2022
Undrained Shear Strength (psf)	1011
Strain Rate (in./min.)	0.05
Specific Gravity (Assumed)	2.815
Strain at Failure (%)	14.89
Liquid Limit	
Plastic Limit	
Test Remarks	

SPECIMEN	DESCRIPTION
1-B2 @ 27-27.5	See exploration log

PROJECT NAME: Hilton Garden Inn, San Jose

Test Date: 4/3/2017

PROJECT NO: 13855.000.000

Tested By: K. Lecce

CLIENT: Westlake Urban

Reviewed By: I. McCauley

LOCATION: San Jose, CA

PHASE NO: 002



Isotropic Unconsolidated Undrained Triaxial Test

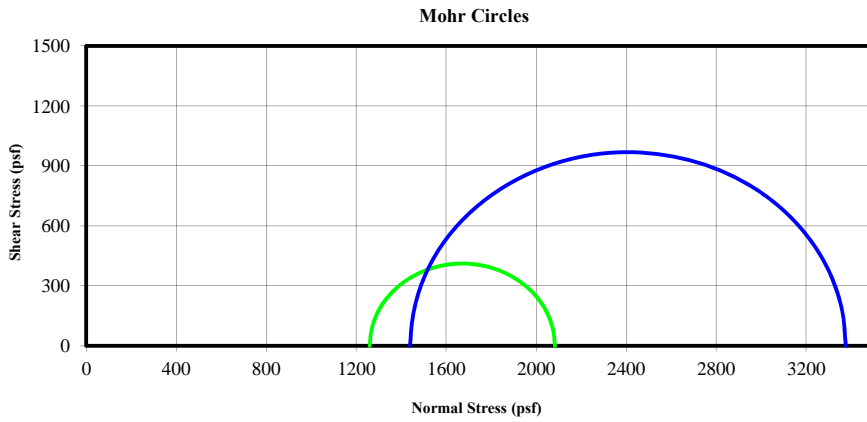
(ASTM D2850)

Date: 04/04/17

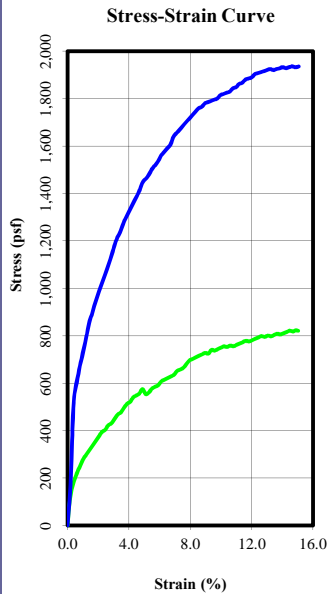
Checked By: I. McCauley

Date: 4/3/2017

Tested By: K. Lecce



— 1-B1 @ 10.5-11 — 1-B2 @ 12-12.5



Specimen			
Before Test	1-B1 @ 10.5-11	1-B2 @ 12-12.5	
Water Content (%)	32.39	36.13	
Dry Density (pcf)	92.42	85.87	
Saturation (%)	99.79	99.43	
Void Ratio	0.93	1.00	
Diameter (in)	2.401	2.395	
Height (in)	5.095	5.110	
Height-to-Diameter Ratio	2.122	2.134	
ASTM D4318 - Wet Method			
Liquid Limit	-	-	
Plastic Limit	-	-	
ASTM D854 - Measured			
Specific Gravity (Assumed)	2.850	2.750	
After Test			
Water Content (%)	32.39	36.13	
Saturation (%)	99.79	99.43	
Strain Rate (in/min)	0.05	0.05	
Peak Deviator Stress (psf)	823.3	1936.3	
Axial Strain @ Failure (%)	14.899	14.652	
Cell Pressure			
Cell (psf)	1260.0	1440.0	
Back (psf)	n/a	n/a	
Principle Stresses at Failure			
σ_1 (psf)	2083.3	3376.3	
σ_3 (psf)	1260.0	1440.0	
Corrected Peak Deviator Stress			

Mohr-Coulomb Parameters with a Non-zero Friction Angle ($\phi \neq 0$)		Cohesion at Failure with a Zero Friction Angle ($\phi = 0$)	
Cohesion, c (psf)	0.0	411.6	968.1
Friction Angle ϕ	0.00	n/a	n/a

Project Information	
Project Name:	Hilton Garden Inn, San Jose
Project Number:	13855.000.000
Project Location:	San Jose, CA
Client:	Westlake Urban
Description:	See exploration logs
Test Remarks:	



WATER SOLUBLE SULFATES IN SOILS
ASTM C1580

Sample number	Sample Location / ID	Matrix	Water Soluble Sulfate % by mass
1	1-B2 @ 3-3.5	soil	ND

Remarks: Results are reported to the nearest 100mg/kg. Anything less than 50mg/kg will be reported as 'ND' for Not-Detectable.

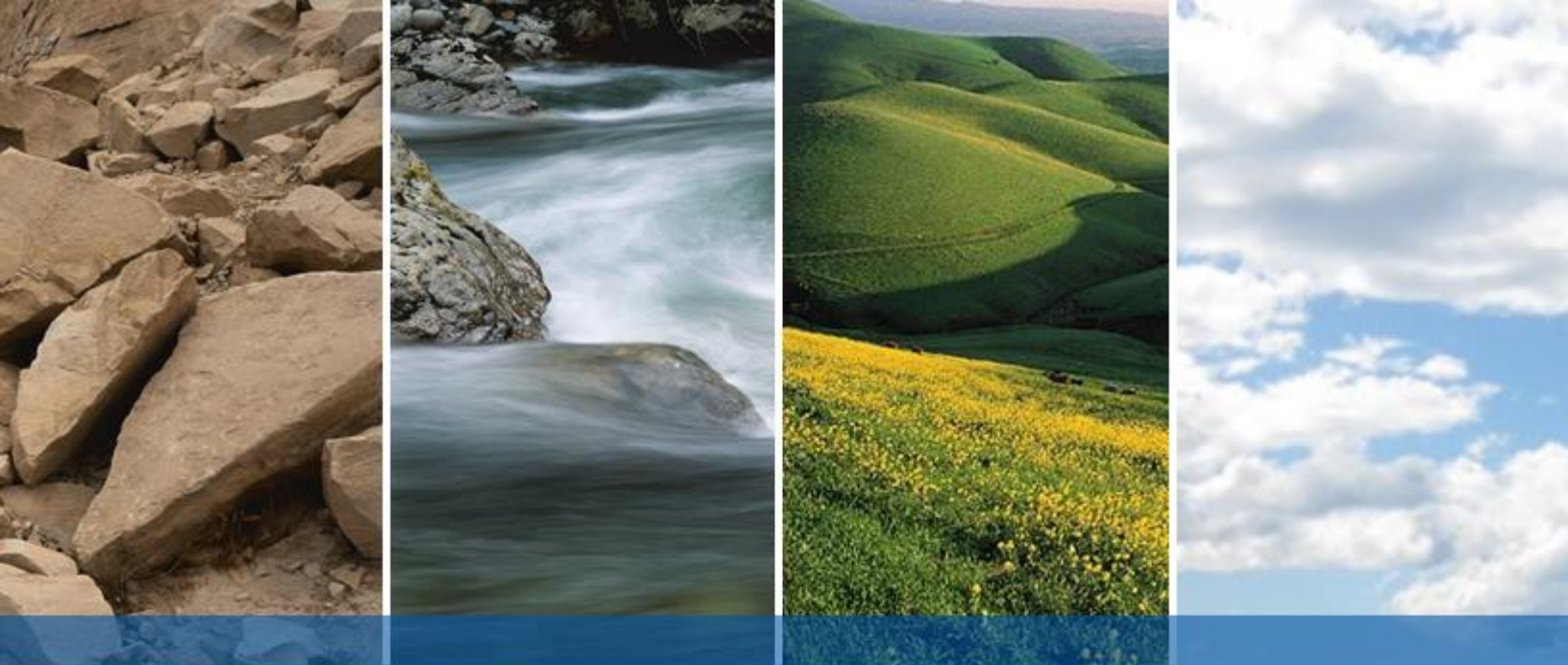
PROJECT NAME: Hilton Garden Inn, San Jose
PROJECT NUMBER: 13855.000.000
CLIENT: Westlake Urban
PHASE NUMBER: 002

DATE: 04/03/17



Tested by: K. Lecce

Reviewed by: I. McCauley



APPENDIX C

CONE PENETRATION TEST LOGS AND PORE PRESSURE DISSIPATION TESTS



Engeo Inc

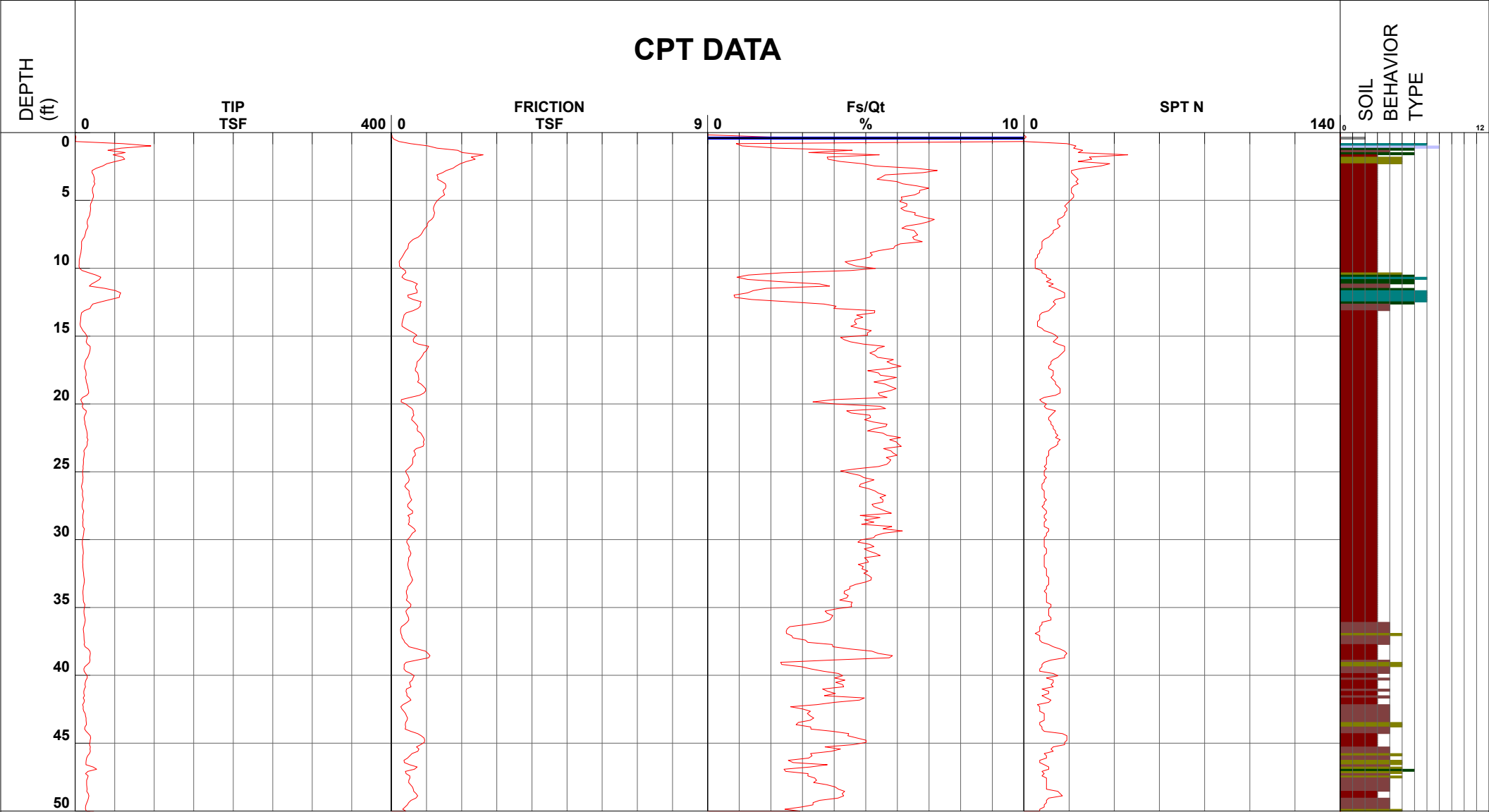
Project Hilton Garden Inn
 Job Number 13855.000.000
 Hole Number CPT-01
 EST GW Depth During Test

Operator RB KK
 Cone Number DDG1379
 Date and Time 3/24/2017 7:25:57 AM
 9.00 ft

Filename SDF(012).cpt
 GPS
 Maximum Depth 50.36 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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



Engeo Inc

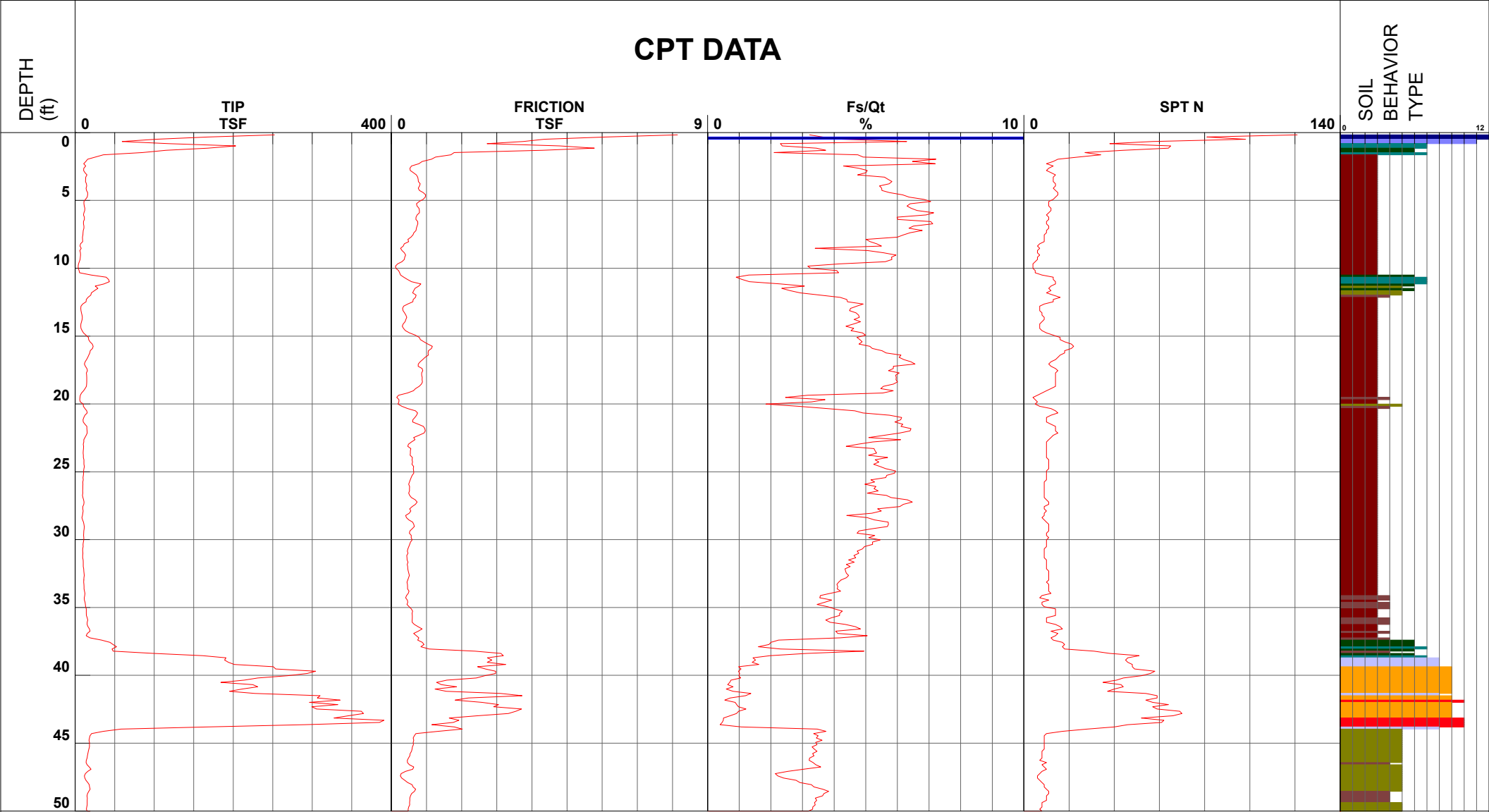
Project Hilton Garden Inn
 Job Number 13855.000.000
 Hole Number CPT-02
 EST GW Depth During Test

Operator RB KK
 Cone Number DDG1379
 Date and Time 3/24/2017 8:02:44 AM
 7.30 ft

Filename SDF(013).cpt
 GPS
 Maximum Depth 50.52 ft

Net Area Ratio .8

CPT DATA



SOIL BEHAVIOR TYPE

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

Cone Size 10cm squared

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



Engeo Inc

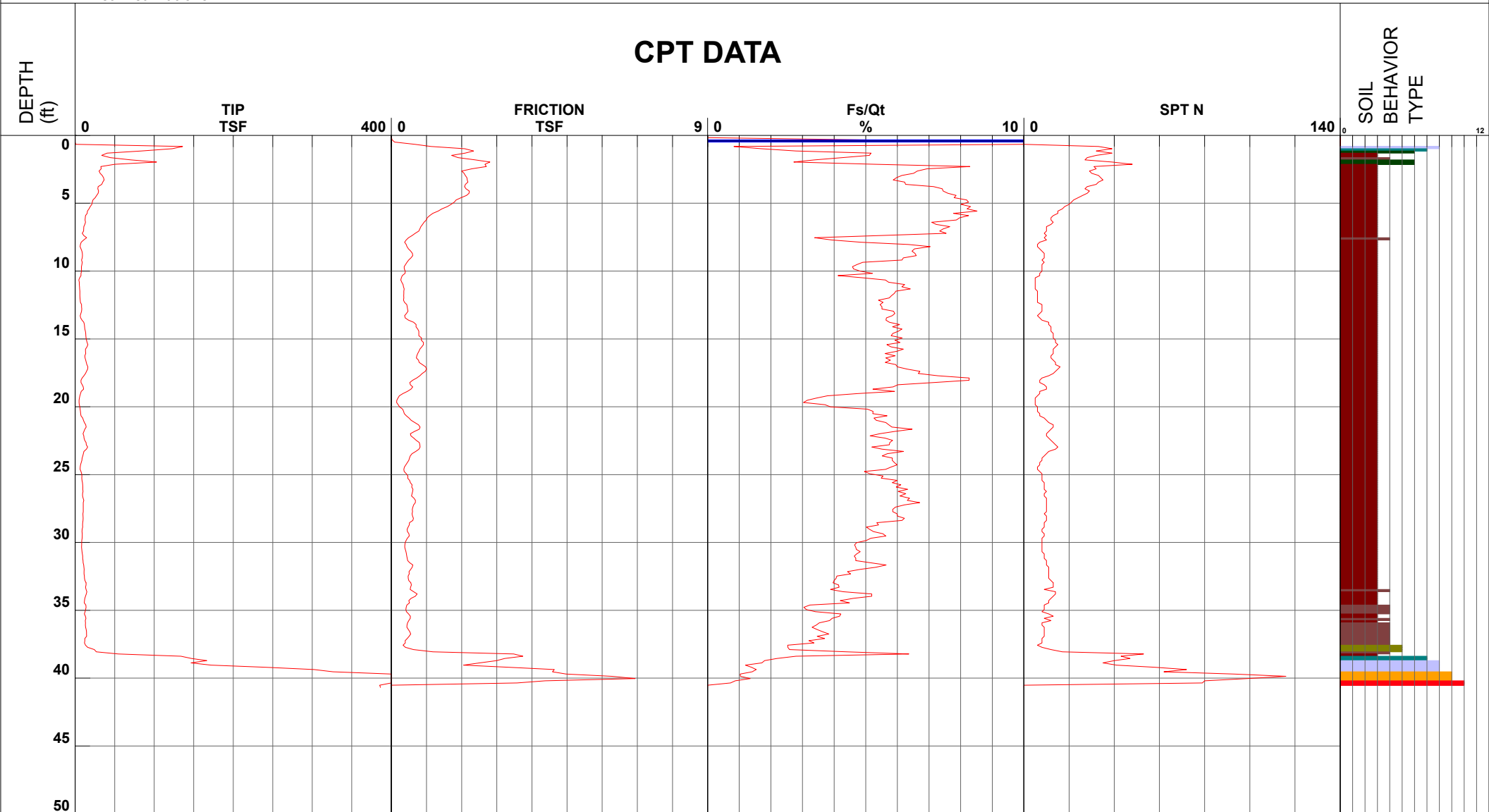
Project Hilton Garden Inn
 Job Number 13855.000.000
 Hole Number CPT-03
 EST GW Depth During Test

Operator RB KK
 Cone Number DDG1379
 Date and Time 3/24/2017 9:33:53 AM

Filename SDF(015).cpt
 GPS
 Maximum Depth 40.68 ft

Net Area Ratio .8

CPT DATA



SOIL
BEHAVIOR
TYPE

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

Cone Size 10cm squared

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



Engeo Inc

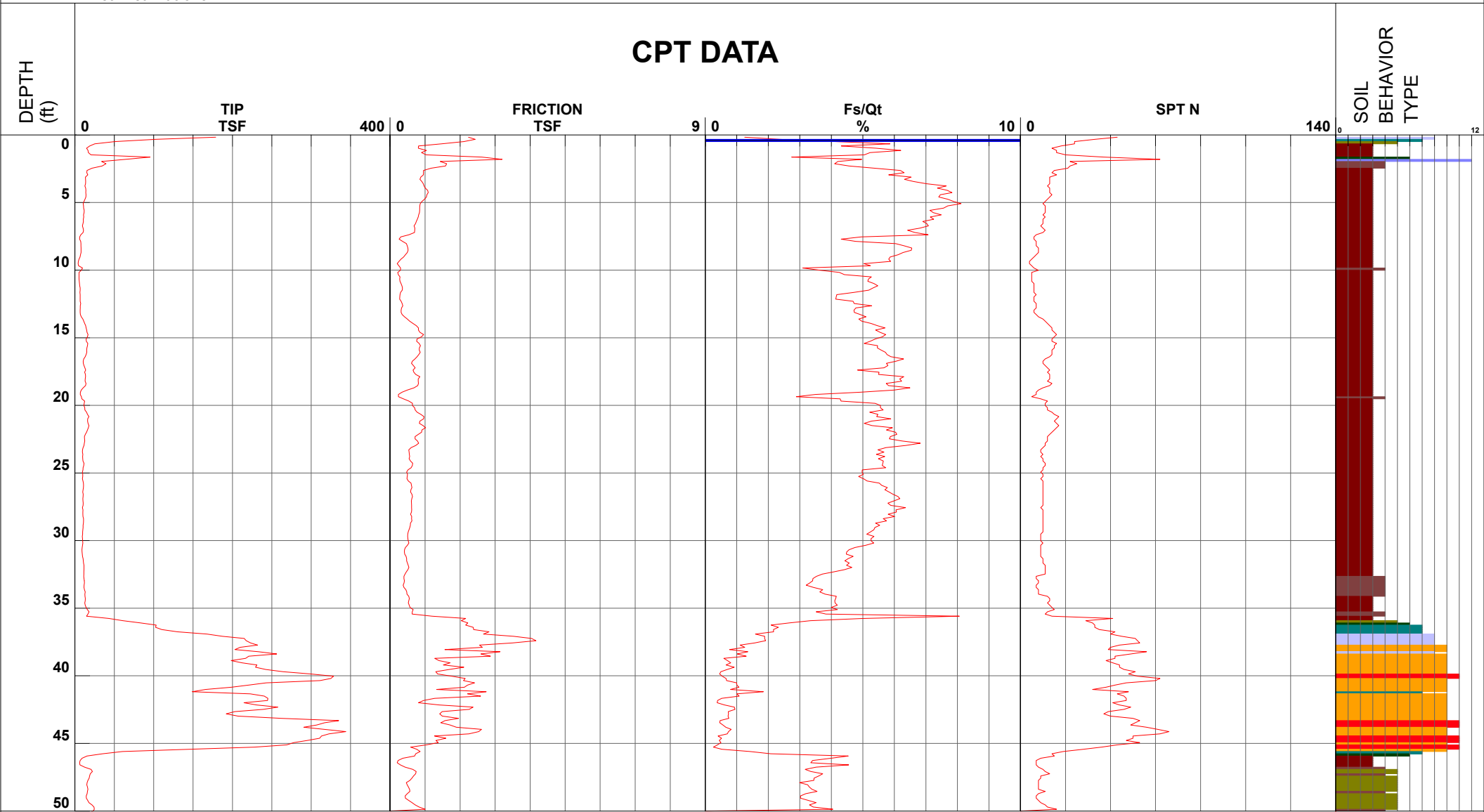
Project Hilton Garden Inn
 Job Number 13855.000.000
 Hole Number CPT-04
 EST GW Depth During Test

Operator RB KK
 Cone Number DDG1379
 Date and Time 3/24/2017 8:42:43 AM
 9.40 ft

Filename SDF(014).cpt
 GPS
 Maximum Depth 50.20 ft

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

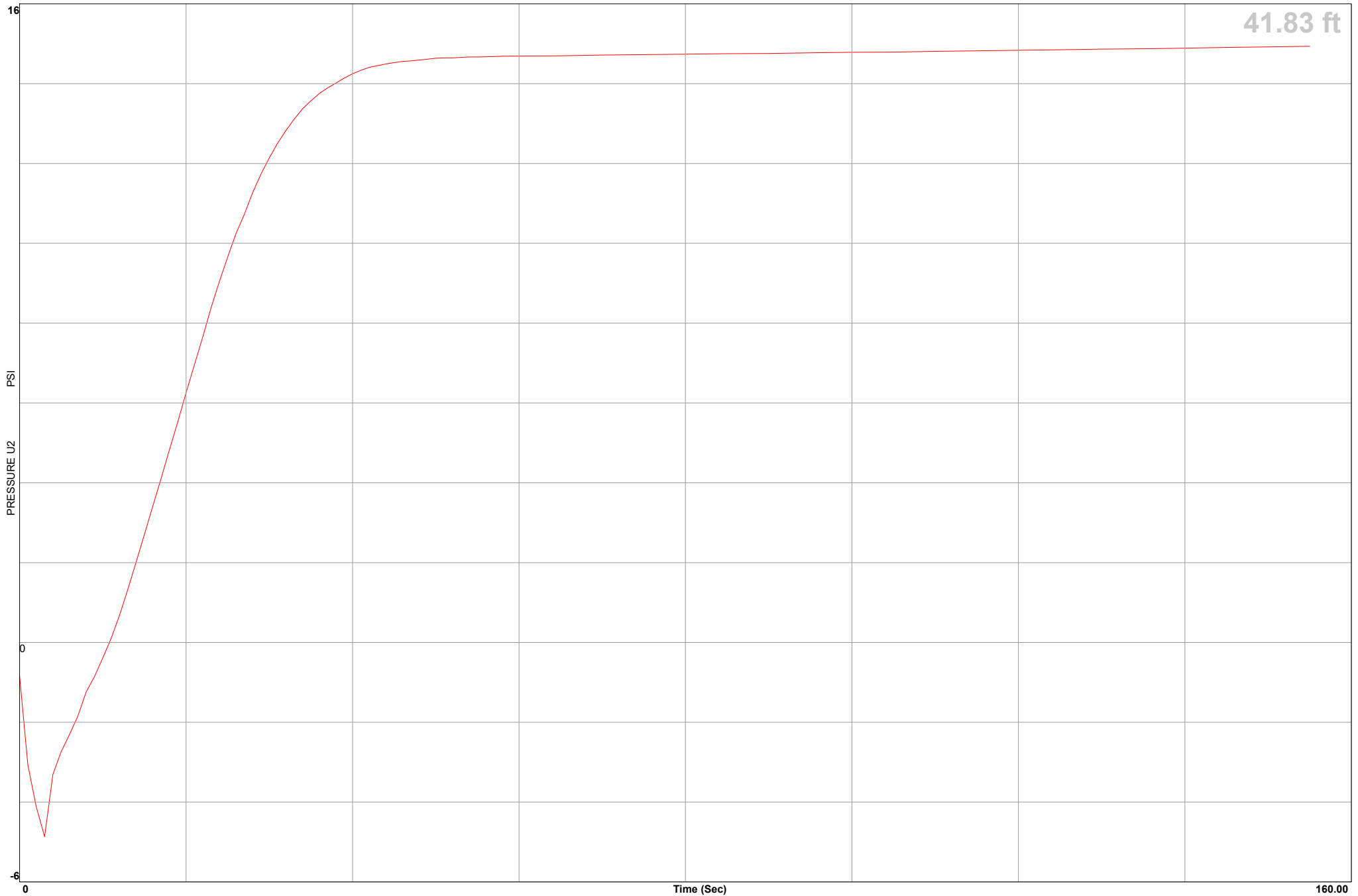


Engeo Inc

Location Hilton Garden Inn
Job Number 13855.000.000
Hole Number CPT-02
Equilized Pressure 14.9

Operator RB KK
Cone Number DDG1379
Date and Time 3/24/2017 8:02:44 AM
EST GW Depth During Test 7.3

GPS _____



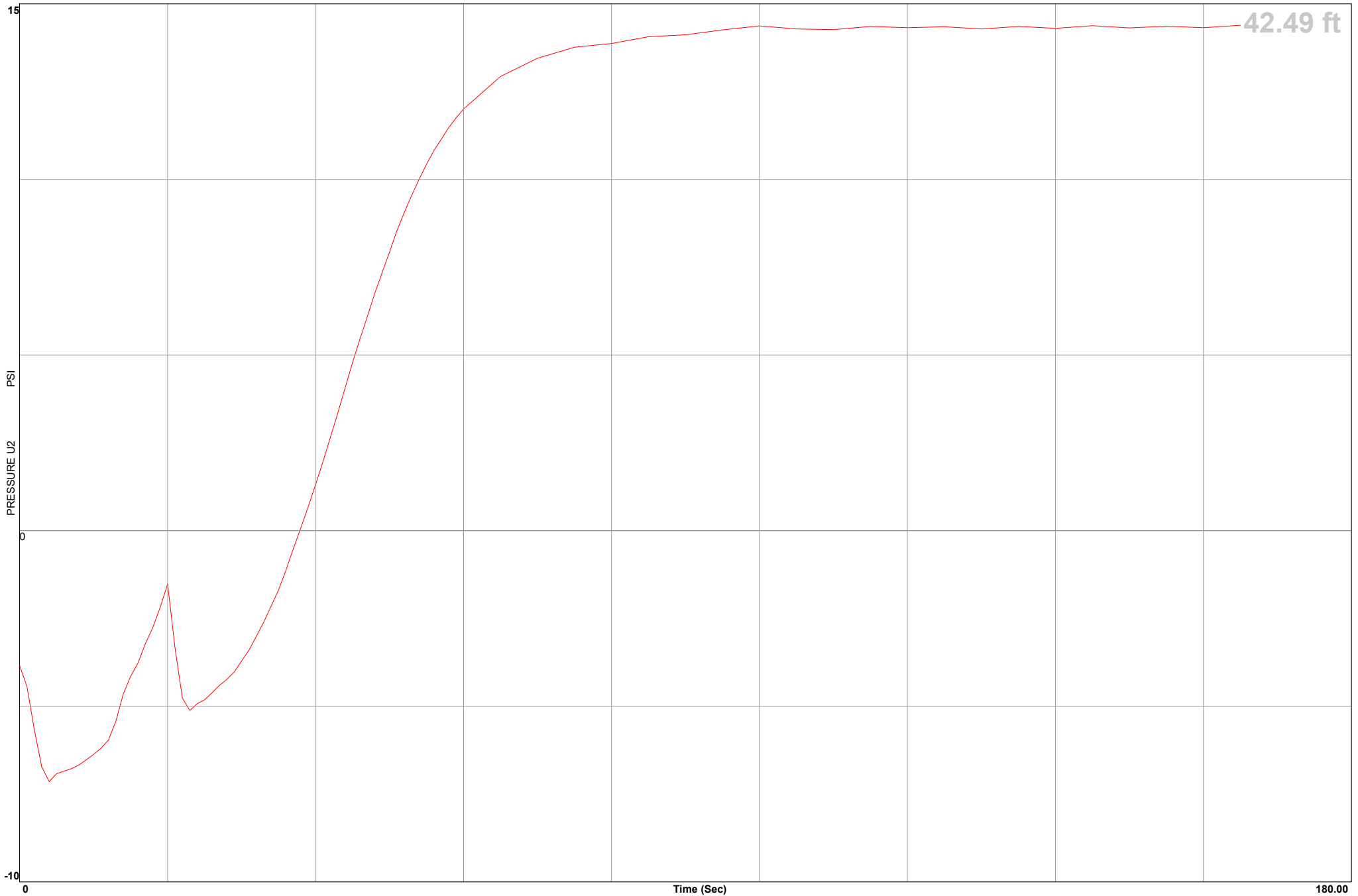


Engeo Inc

Location Hilton Garden Inn
Job Number 13855.000.000
Hole Number CPT-03
Equilized Pressure 14.3

Operator RB KK
Cone Number DDG1379
Date and Time 3/24/2017 9:33:53 AM
EST GW Depth During Test 9.4

GPS _____



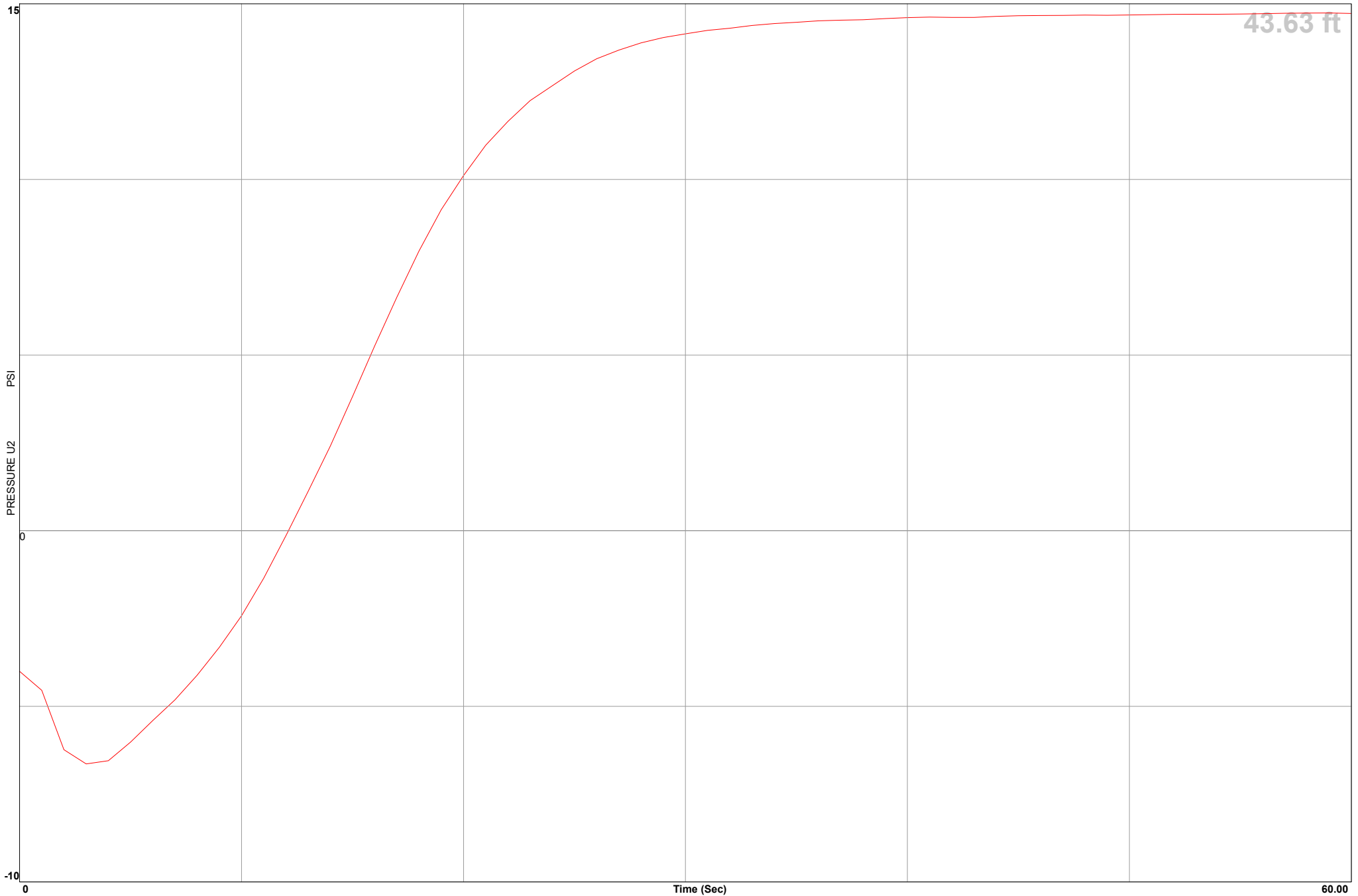


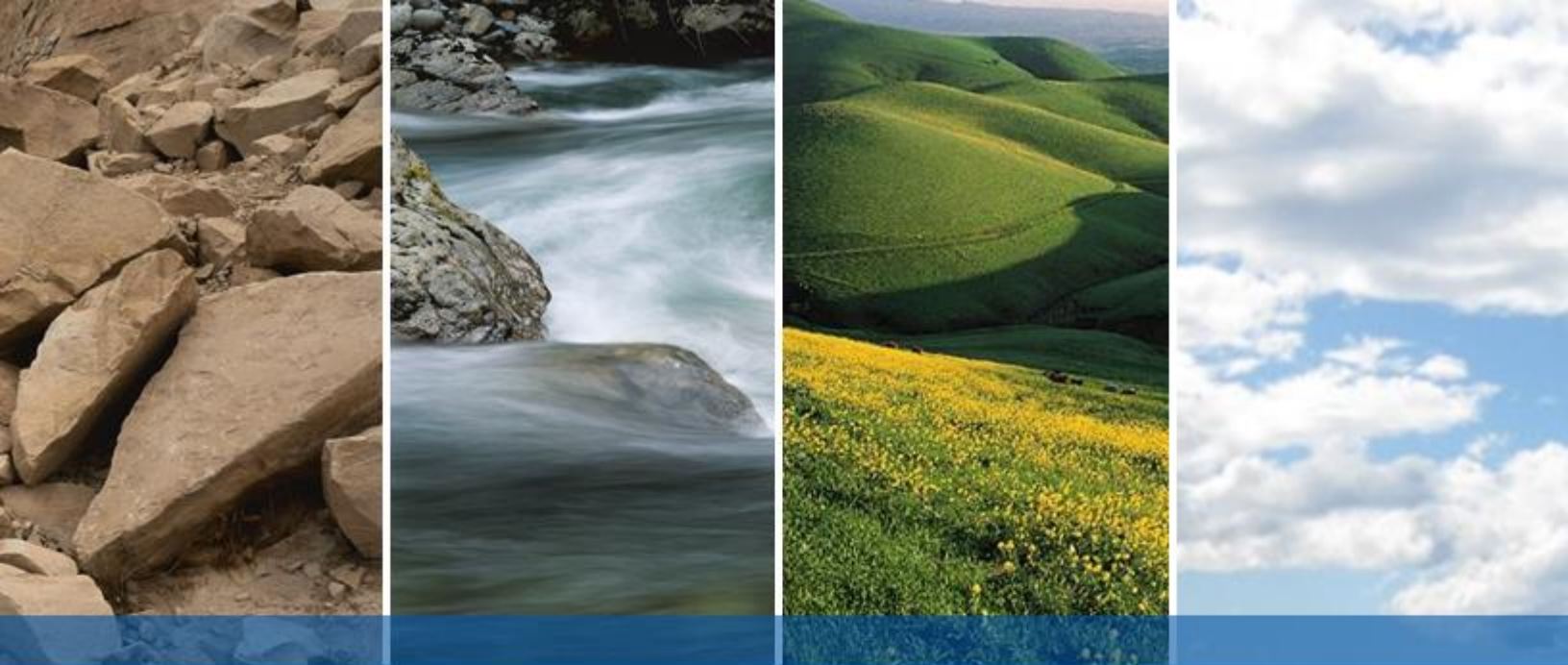
Engeo Inc

Location Hilton Garden Inn
Job Number 13855.000.000
Hole Number CPT-04
Equilized Pressure 14.7

Operator RB KK
Cone Number DDG1379
Date and Time 3/24/2017 8:42:43 AM
EST GW Depth During Test 9.5

GPS _____





APPENDIX D

LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS REPORT

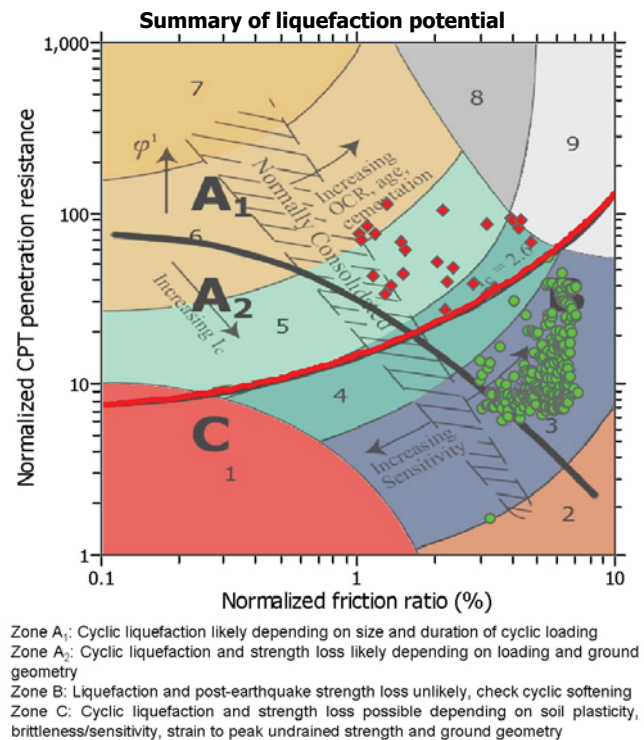
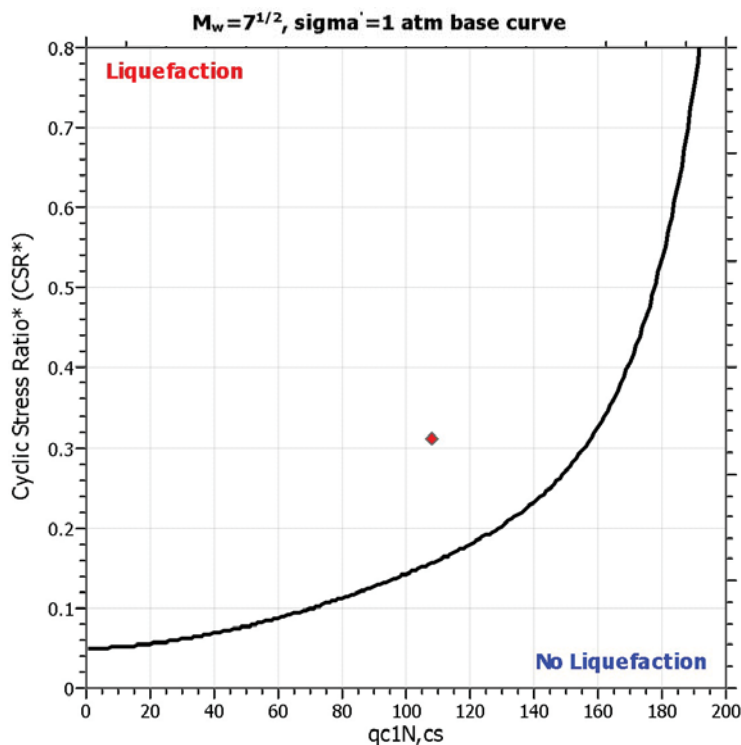
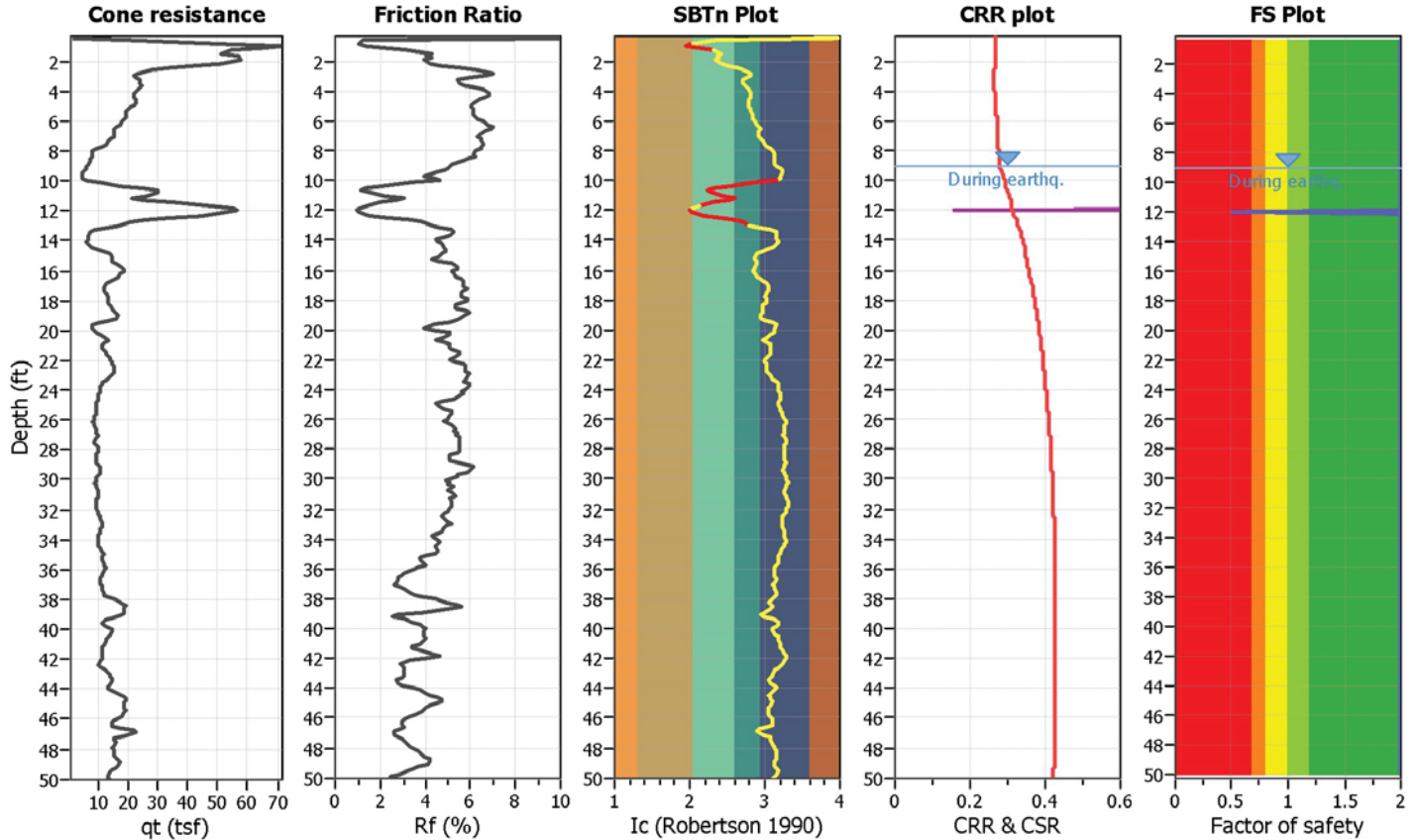
Project title :

Location :

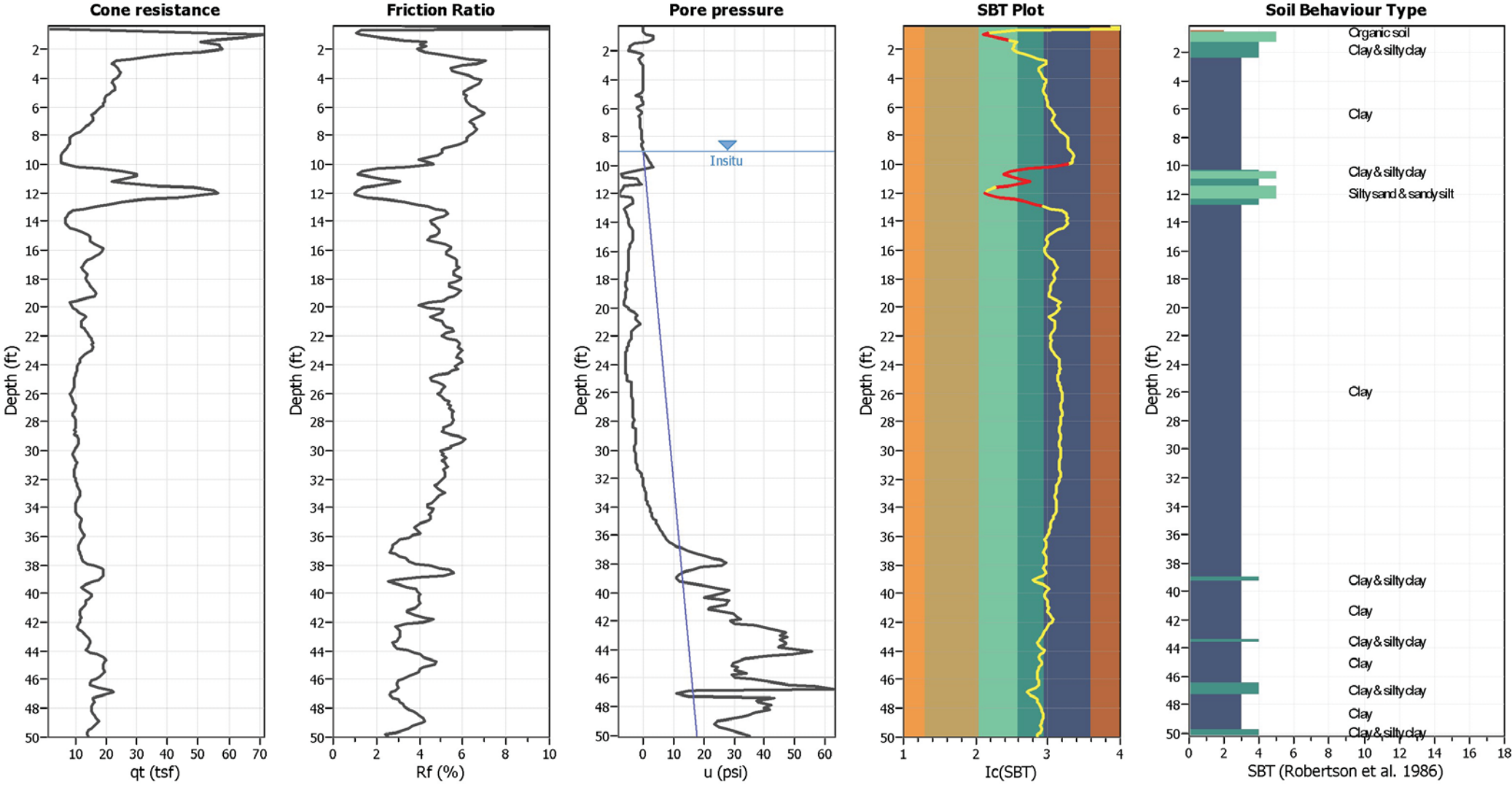
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	9.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	9.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based



CPT basic interpretation plots



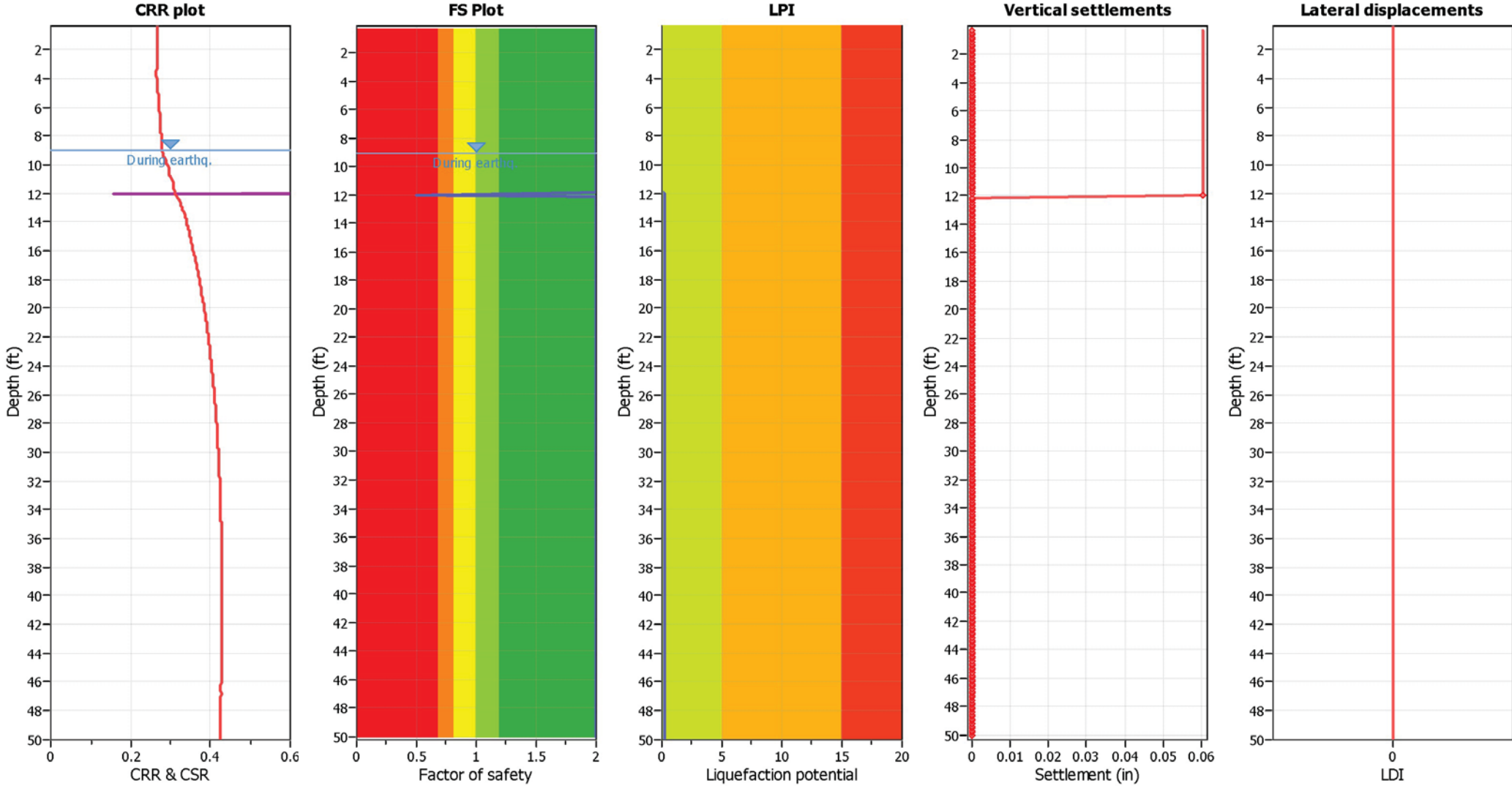
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	9.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	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 :	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: I&B (2008)
 Fines correction method: I&B (2008)
 Points to test: Based on Ic value
 Earthquake magnitude M_w : 7.10
 Peak ground acceleration: 0.50
 Depth to water table (insitu): 9.00 ft

Depth to GWT (erthq.): 9.00 ft
 Average results interval: 3
 Ic cut-off value: 2.60
 Unit weight calculation: Based on SBT
 Use fill: No
 Fill height: N/A

Fill weight: N/A
 Transition detect. applied: Yes
 K_{σ} applied: Yes
 Clay like behavior applied: Sands only
 Limit depth applied: No
 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

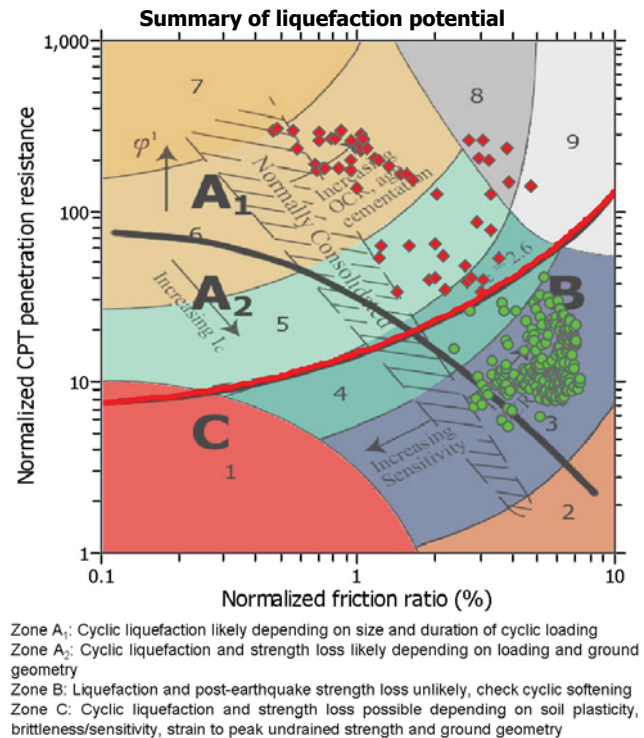
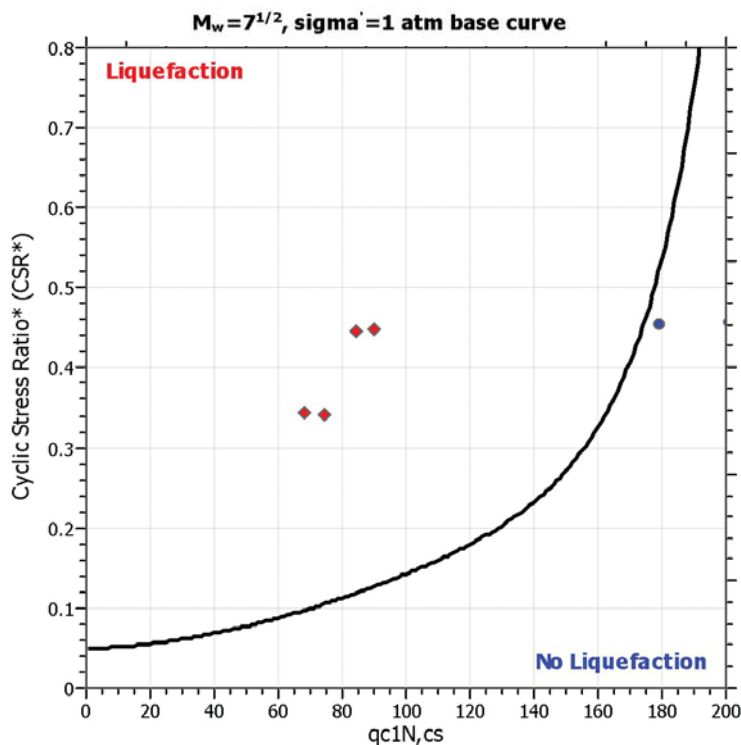
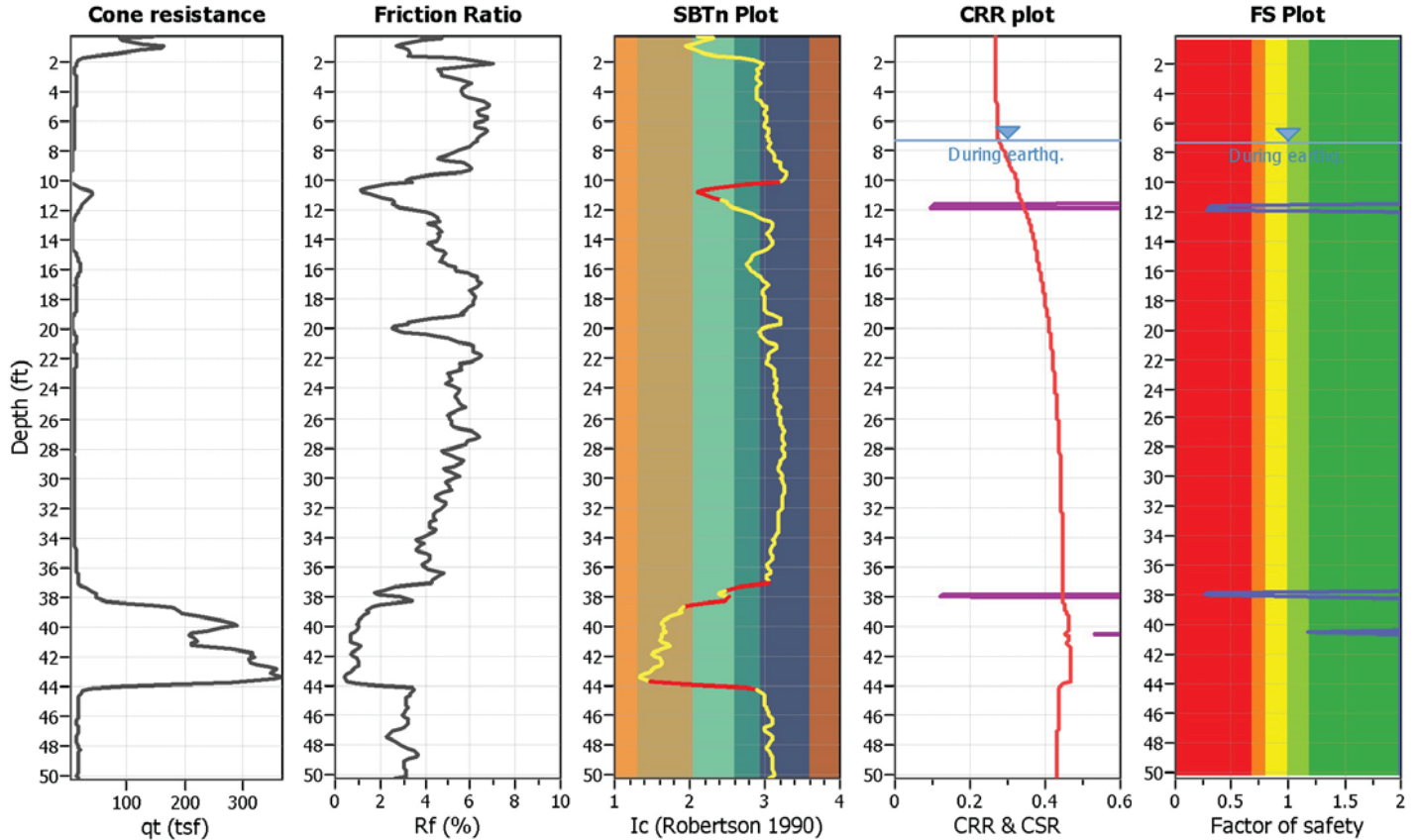
Project title :

Location :

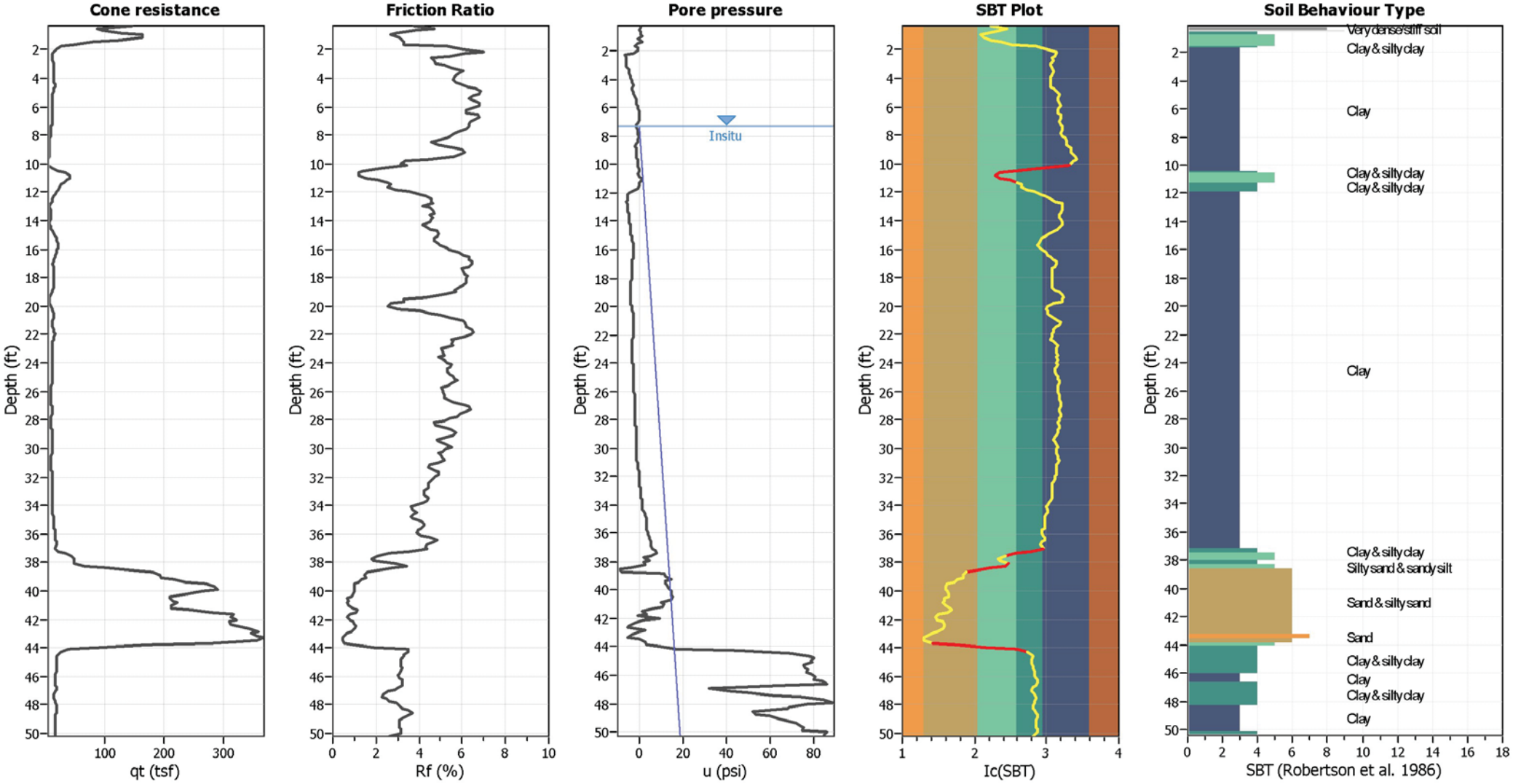
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	7.30 ft	Use fill:	No	Clay like behavior	
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	7.30 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based



CPT basic interpretation plots



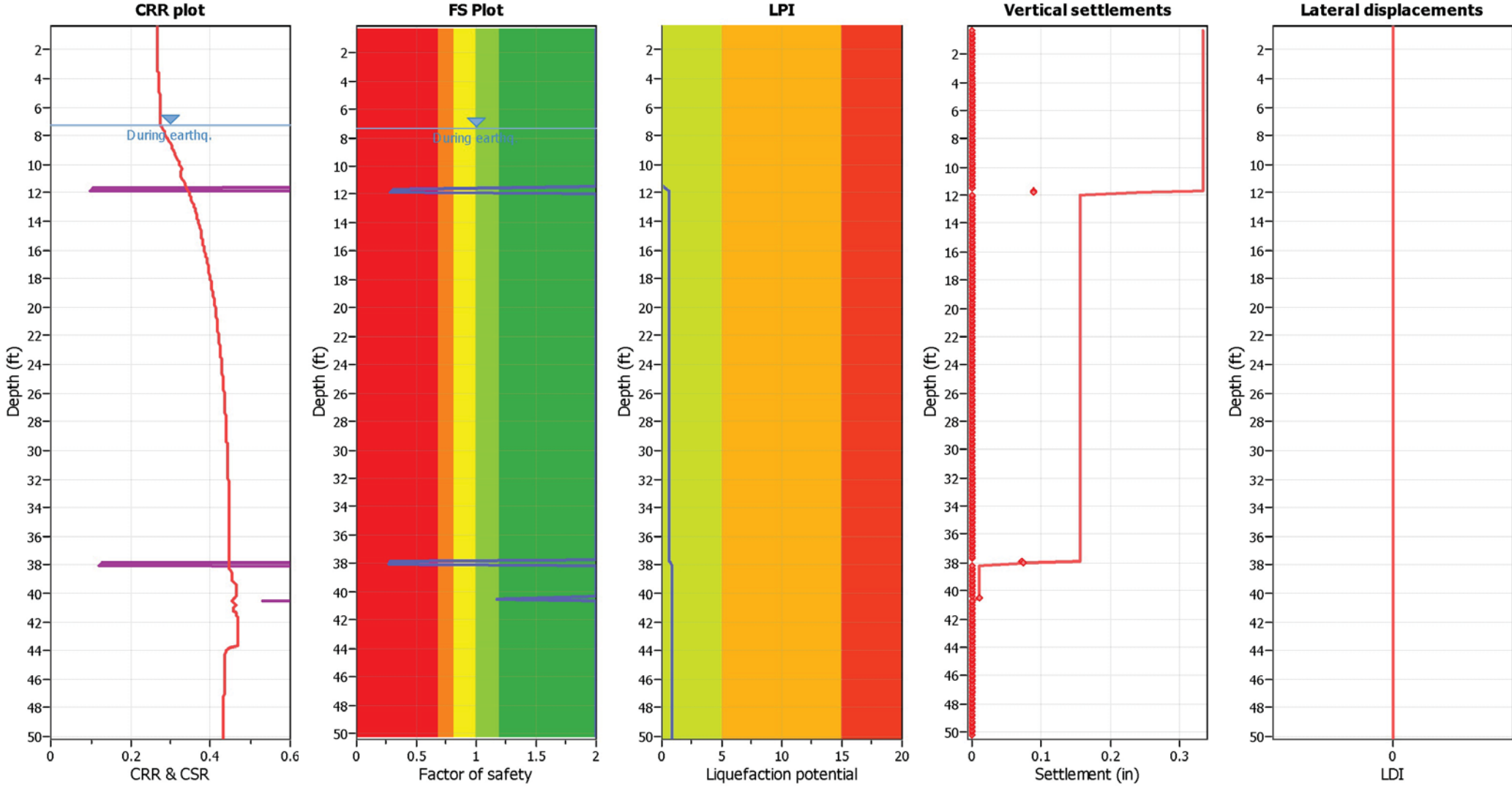
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	7.30 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{α} applied:	Yes
Earthquake magnitude M_w :	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.30 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: I&B (2008)
 Fines correction method: I&B (2008)
 Points to test: Based on Ic value
 Earthquake magnitude M_w : 7.10
 Peak ground acceleration: 0.50
 Depth to water table (insitu): 7.30 ft

Depth to GWT (erthq.): 7.30 ft
 Average results interval: 3
 Ic cut-off value: 2.60
 Unit weight calculation: Based on SBT
 Use fill: No
 Fill height: N/A

Fill weight: N/A
 Transition detect. applied: Yes
 K_{σ} applied: Yes
 Clay like behavior applied: Sands only
 Limit depth applied: No
 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

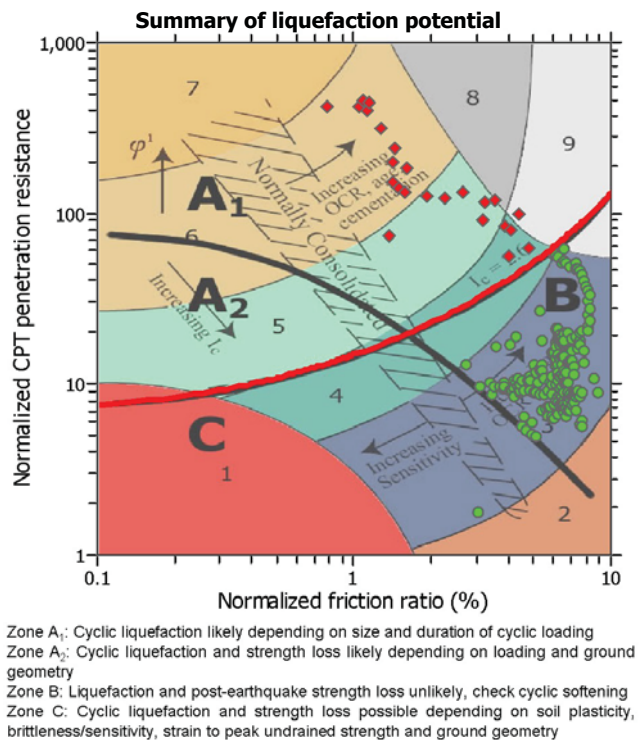
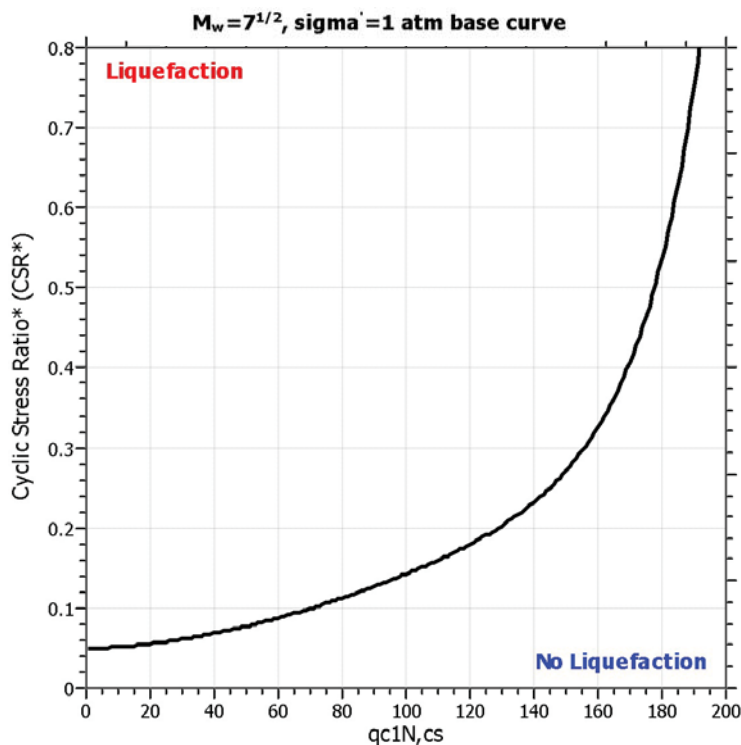
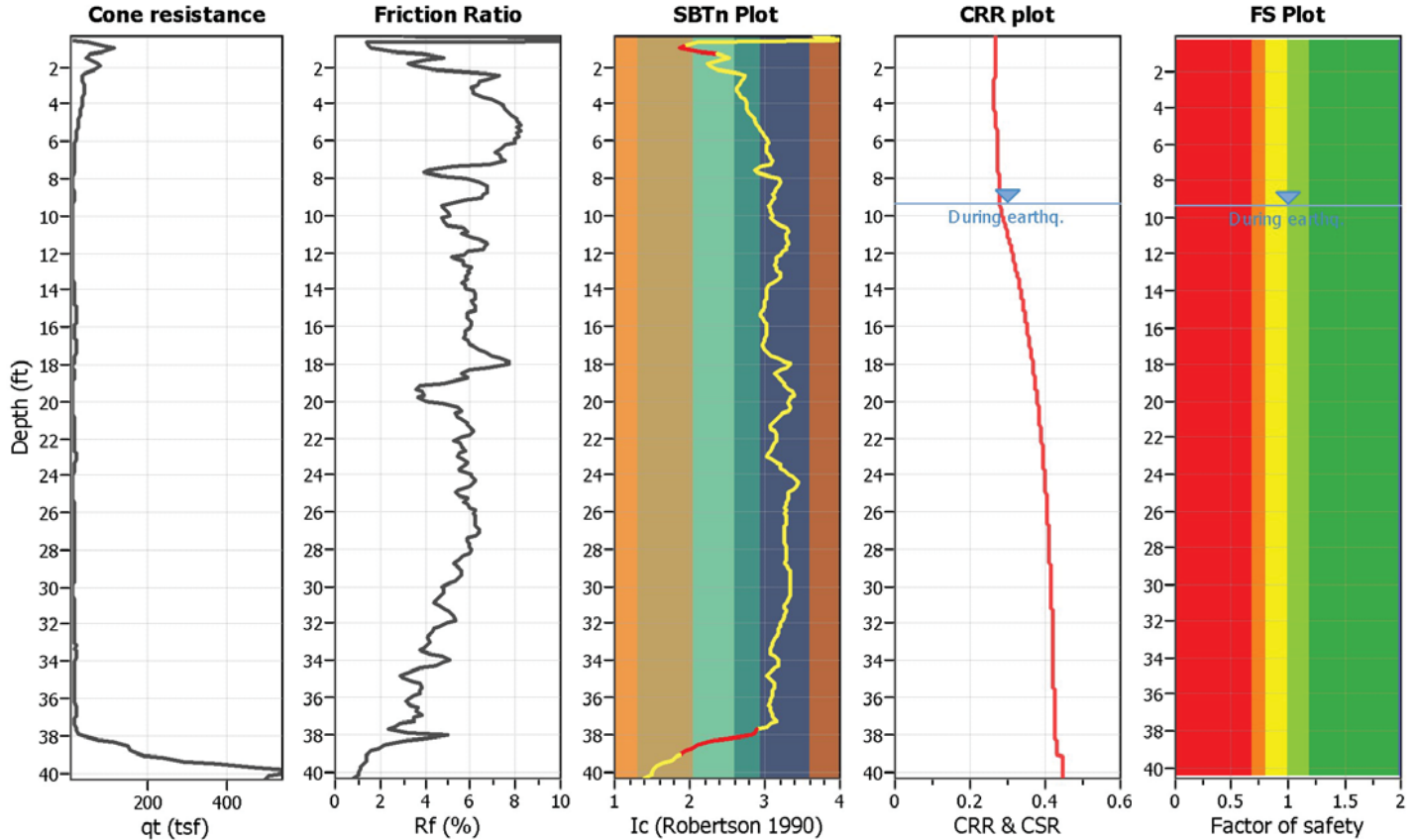
Project title :

Location :

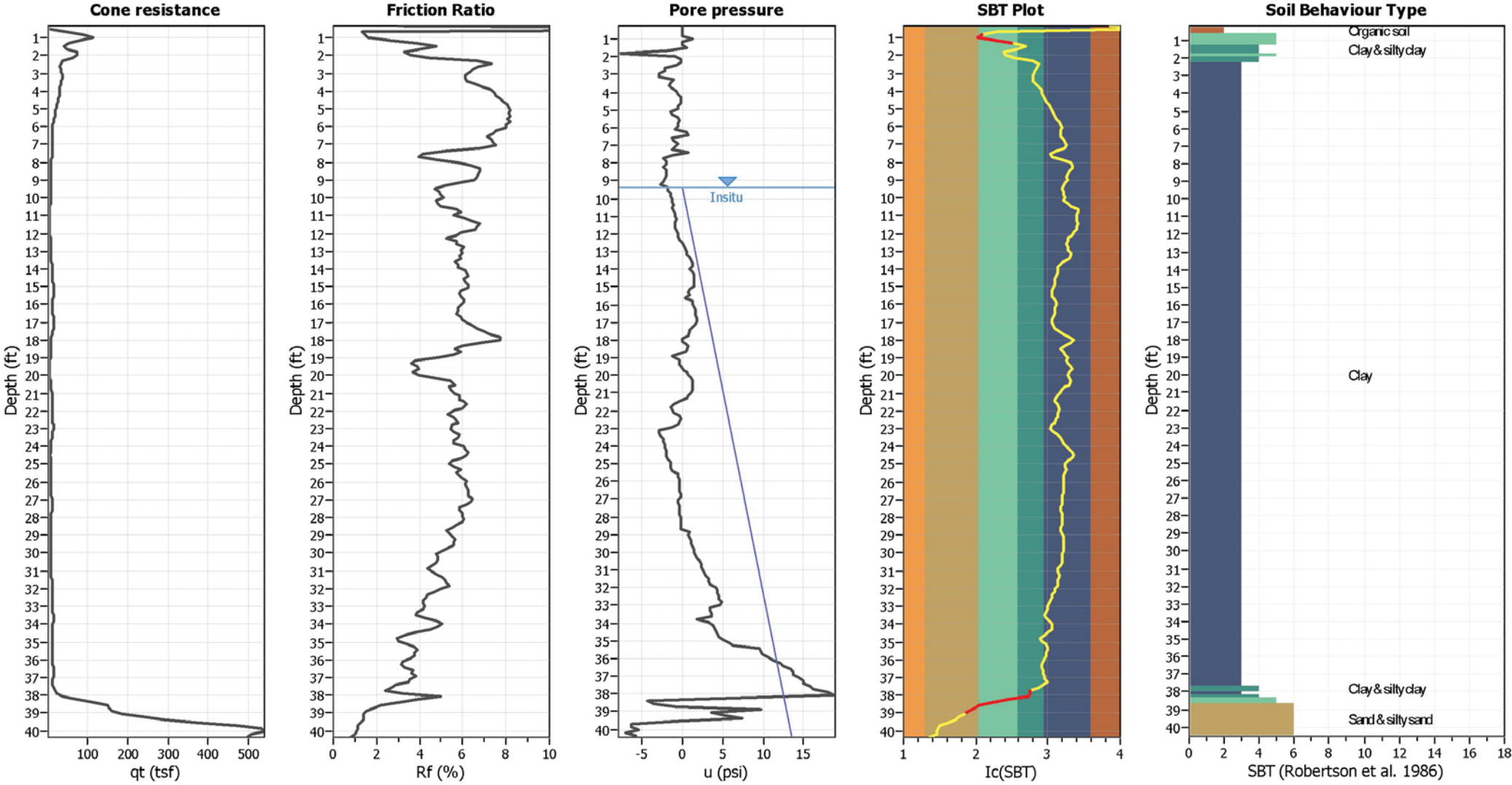
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	9.40 ft	Use fill:	No	Clay like behavior	
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	9.40 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based



CPT basic interpretation plots



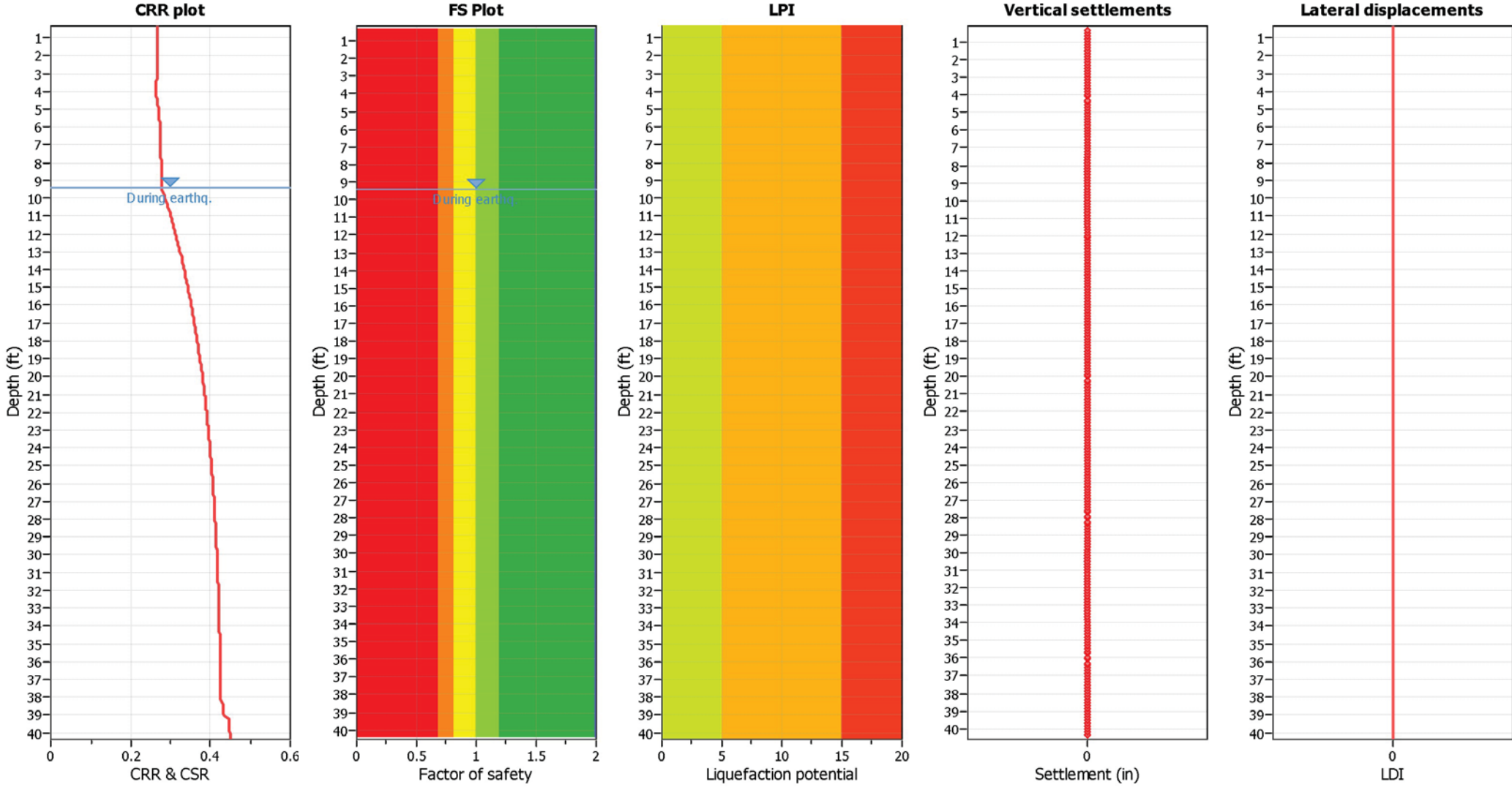
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	9.40 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.40 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: I&B (2008)
 Fines correction method: I&B (2008)
 Points to test: Based on Ic value
 Earthquake magnitude M_w : 7.10
 Peak ground acceleration: 0.50
 Depth to water table (insitu): 9.40 ft

Depth to GWT (erthq.): 9.40 ft
 Average results interval: 3
 Ic cut-off value: 2.60
 Unit weight calculation: Based on SBT
 Use fill: No
 Fill height: N/A

Fill weight: N/A
 Transition detect. applied: Yes
 K_{σ} applied: Yes
 Clay like behavior applied: Sands only
 Limit depth applied: No
 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

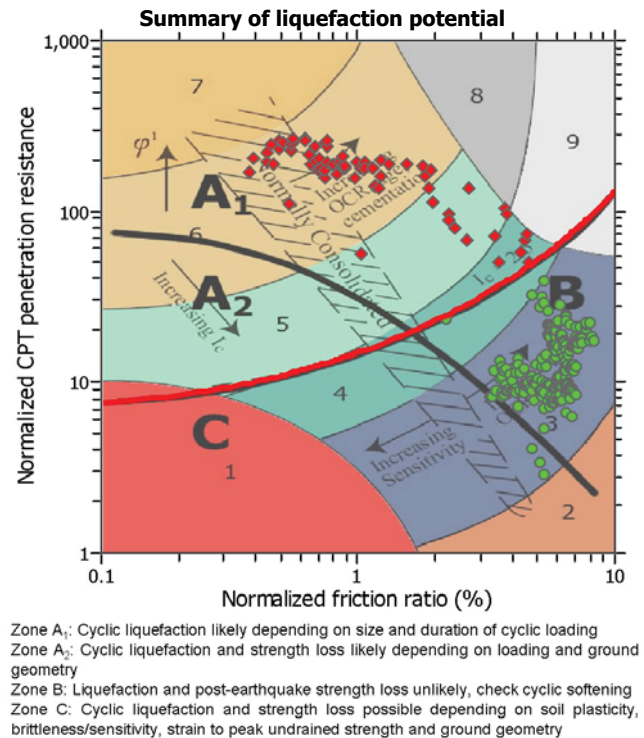
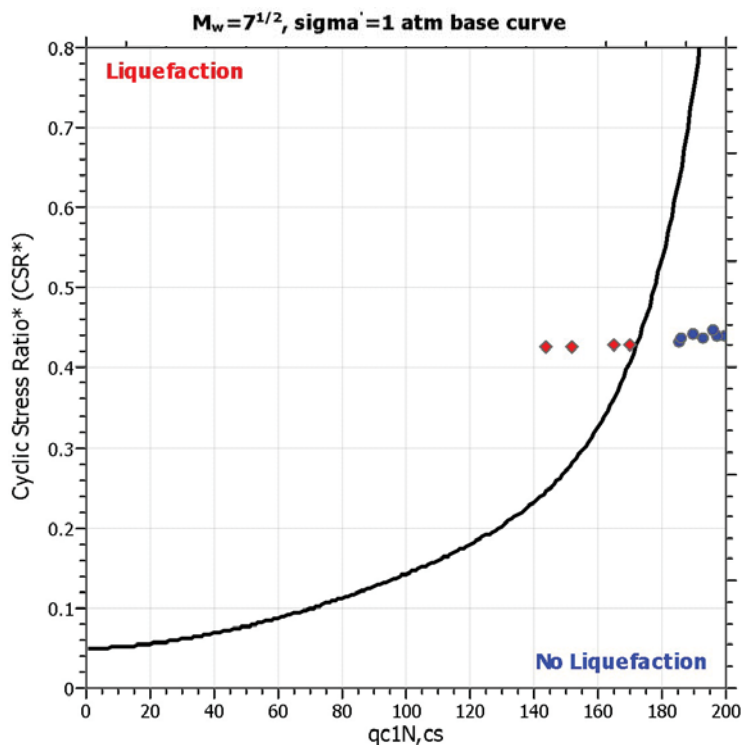
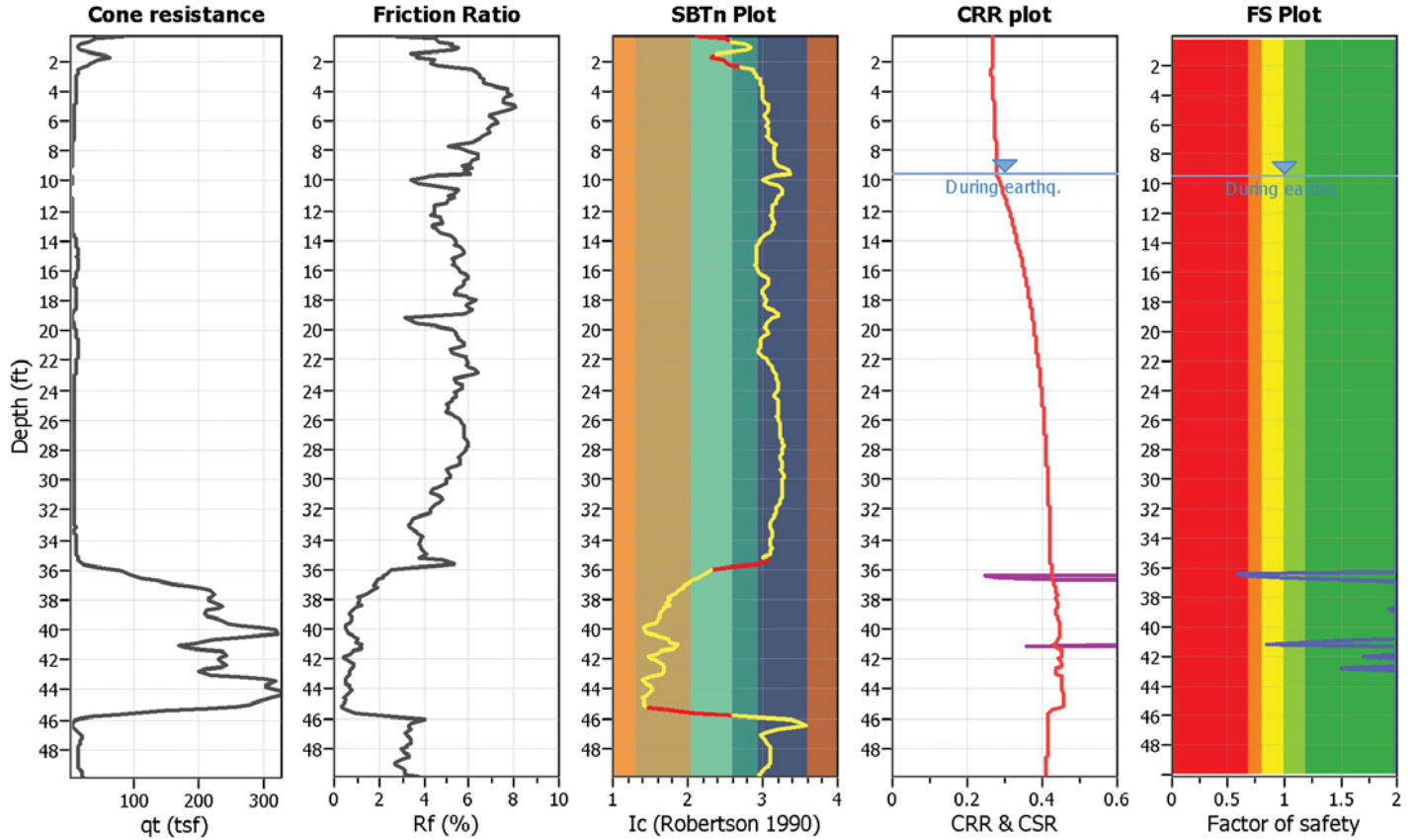
Project title :

Location :

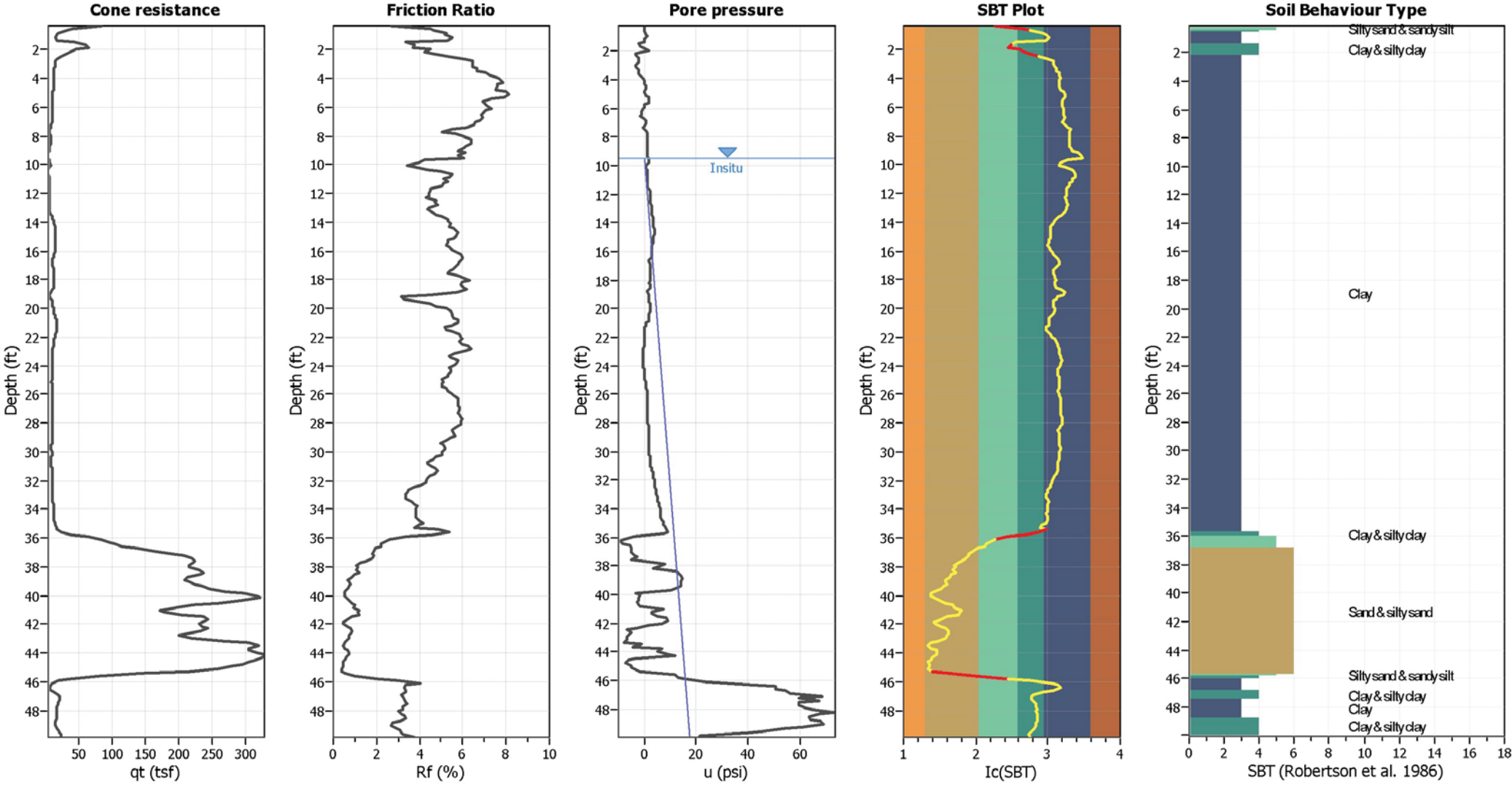
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	9.50 ft	Use fill:	No	Clay like behavior	
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	9.50 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based



CPT basic interpretation plots



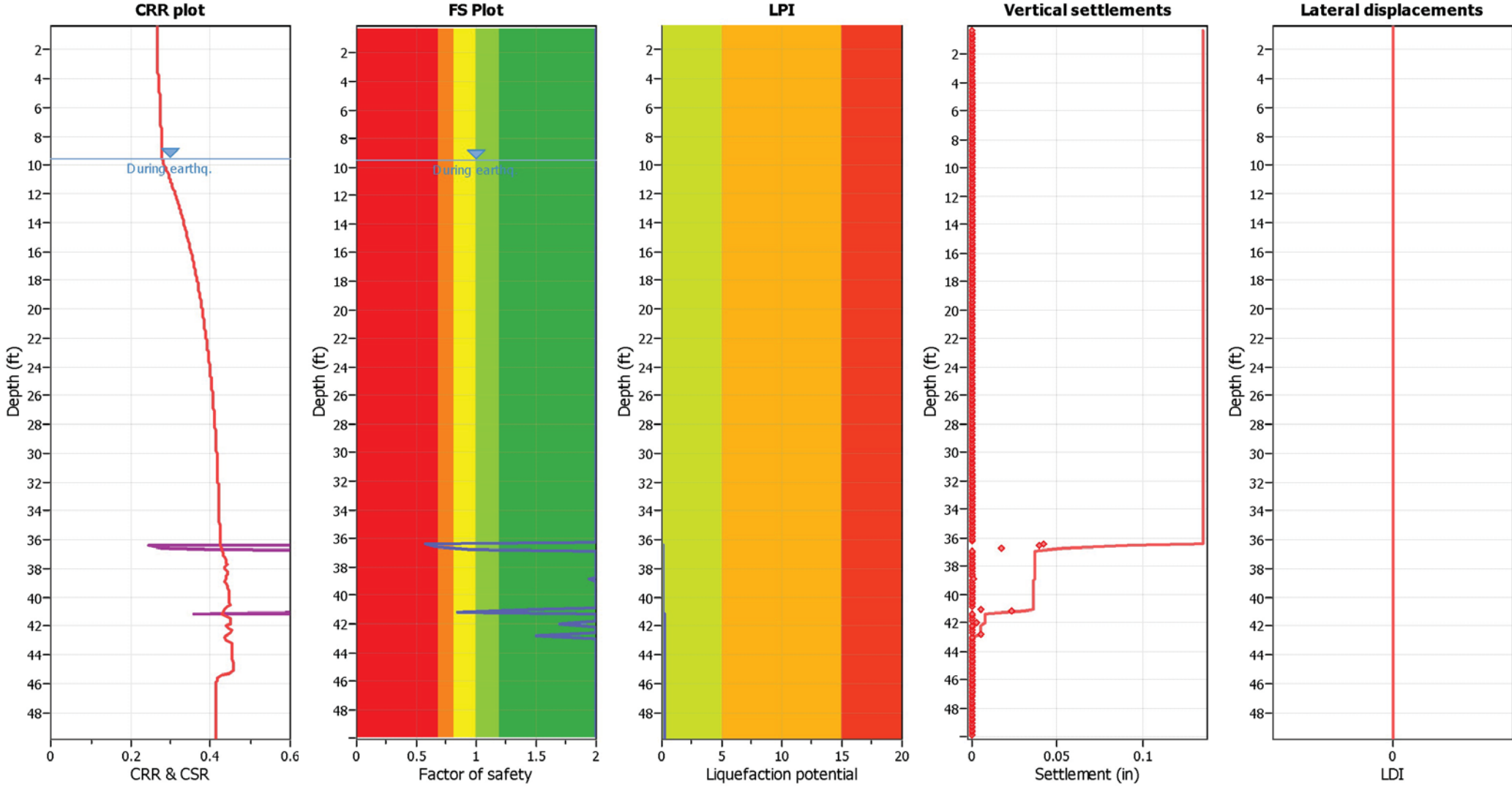
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{α} applied:	Yes
Earthquake magnitude M_w :	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: I&B (2008)
 Fines correction method: I&B (2008)
 Points to test: Based on Ic value
 Earthquake magnitude M_w : 7.10
 Peak ground acceleration: 0.50
 Depth to water table (insitu): 9.50 ft

Depth to GWT (earthq.): 9.50 ft
 Average results interval: 3
 Ic cut-off value: 2.60
 Unit weight calculation: Based on SBT
 Use fill: No
 Fill height: N/A

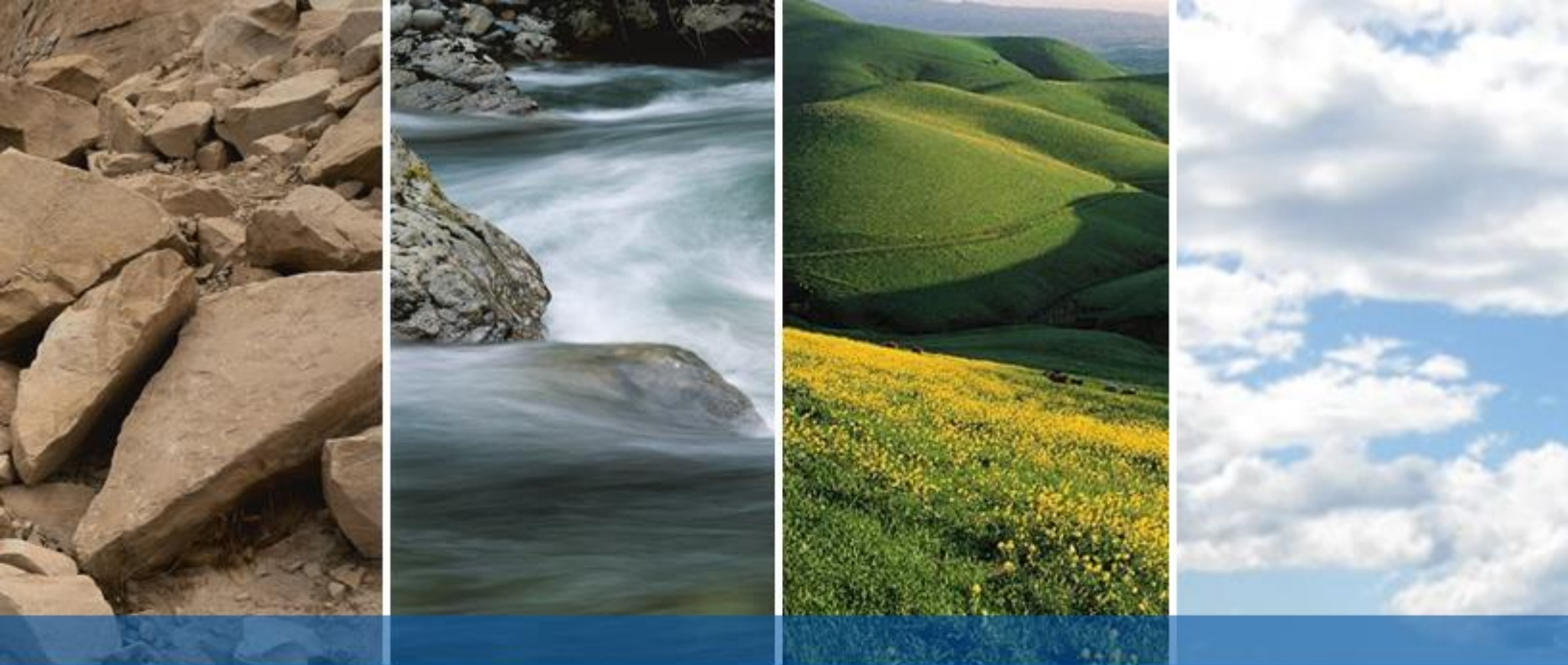
Fill weight: N/A
 Transition detect. applied: Yes
 K_{σ} applied: Yes
 Clay like behavior applied: Sands only
 Limit depth applied: No
 Limit depth: N/A

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



APPENDIX E

**CORROSIVITY ANALYSIS
(CERCO ANALYTICAL, INC.)**



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.: 13855.000.000
 Client's Project Name: Hilton Garden Inn
 Date Sampled: 28-Mar-17
 Date Received: 3-Apr-17
 Matrix: Soil
 Authorization: Signed Chain of Custody

Date of Report: 13-Apr-2017

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)*
1704003-001	1-B1 @ 1'-2.5'	370	8.22	-	2,900	N.D.	N.D.	27

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	6-Apr-2017	6-Apr-2017	-	6-Apr-2017	13-Apr-2017	6-Apr-2017	6-Apr-2017

Cheryl McMillen
 Cheryl McMillen
 Laboratory Director

* Results Reported on "As Received" Basis
 N.D. - None Detected

1704003

11521

CHAIN OF CUSTODY RECORD

PROJECT NUMBER: 13855.000.000				PROJECT NAME: Hilton Garden Inn			Redox	pH	Sulfate	Resistivity	Chloride	Sulfide	REMARKS REQUIRED DETECTION LIMITS
SAMPLED BY: (SIGNATURE/PRINT) Seema Barua													
PROJECT MANAGER: Jeanine Ruffoni													
ROUTING: E-MAIL <u>sbarua@engeo.com</u> Hard Copy <u>jruffoni@engeo.com</u>													
SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE							
1-B1 @ 1-2.5'	28-Mar-17	AM	Soil	1	Liner	None	x	x	x	x	x	x	ASTM Test Methods
RELINQUISHED BY: (SIGNATURE)			DATE/TIME		RECEIVED BY: (SIGNATURE)		RELINQUISHED BY: (SIGNATURE)		DATE/TIME		RECEIVED BY: (SIGNATURE)		REMARKS STANDARD 10 DAY TURNAROUND
[Signature]			4-3-17 2:30		[Signature]		[Signature]		4/3/17 1510		[Signature]		
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				

001



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 WWW.ENGEO.COM

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