GEOTECHNICAL EXPLORATION

1210 CALIFORNIA CIRCLE
MILPITAS, CALIFORNIA

ENGEO

Expect Excellence

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Submitted to:
Mr. Erich Stiger
iStar Financial Incorporated
One Sansome Street, 30th Floor
San Francisco, CA 94104

Prepared by:
ENGEO Incorporated

August 15, 2014

Project No.
11185.000.000

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Project No.
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August 15, 2014

Mr. Erich Stiger
iStar Financial Incorporated
One Sansome Street, 30th Floor
San Francisco, CA 94104

Subject: 1210 California Circle
Milpitas, California

GEOTECHNICAL EXPLORATION

Dear Mr. Stiger:

As requested, we completed this geotechnical exploration for the proposed development in Milpitas, California. The accompanying report presents our field exploration and laboratory testing with our conclusions and recommendations regarding the proposed residential development at the site.

Our findings indicate that the site is suitable for the proposed development provided the recommendations and guidelines provided in this report are implemented during project planning, design, and construction. The main geologic/geotechnical issues at this site include: potential seismic-induced settlement, potential liquefaction-induced surface rupture (sand boils), risk of lateral deformation along Penitencia Creek, presence of soft compressible clays on the western portion of site, and undocumented existing fills, expansive near-surface soils, and presence of relatively shallow groundwater.

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We are pleased to have been of service to you on this project and are prepared to consult further with you and your design team as the project progresses. If you have any questions regarding the contents of this report, please do not hesitate to contact us.

Sincerely,

ENGEO Incorporated

Jeanine T. Ruffoni, EIT

Theodore P. Bayham, GE, CEG

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this geotechnical report, as described in our proposal dated June 9, 2014, is to provide design-level geotechnical recommendations associated with the proposed residential development of the site.

We performed the following services:

- Review of available literature and geologic maps for the study area.
- Subsurface exploration consisting of seven soil borings and six Cone Penetration Test (CPT) probes.
- Laboratory testing of materials sampled during the field exploration.
- Geotechnical data analyses.
- Report preparation summarizing our conclusions and recommendations for the proposed development.

We received the following document for our use:

- Kier & Wright; ALTA/ACSM Land Title Survey, 1210 California Circle, May 2001.

We prepared this report exclusively for iStar Financial Incorporated and their design team consultants. ENGEO should review any changes made in the character, design or layout of the development to modify the conclusions and recommendations contained in this report, as necessary. This document may not be reproduced, in whole or in part, by any means whatsoever, nor may it be quoted or excerpted without the express written consent of ENGEO.

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1.2 SITE LOCATION AND DESCRIPTION

The roughly 9.5-acre site is located at 1210 California Circle in Milpitas, California (Figure 1). The irregularly shaped site is identified by Parcels B 532-M-48 and 532-M-49.

The site is currently occupied by a one-story concrete tilt-up commercial building surrounded by asphalt-concrete paved parking. In addition, the site consists of perimeter landscaping with a paved walkway along the southern and western perimeters. The site is relatively flat and generally matches that of surrounding properties to the north. The western and southern landscaped areas of the site generally consist of 1:1 (horizontal:vertical) or flatter upslopes up to approximately 3 feet in height to California Circle. The site is bordered by California Circle to the west and south, existing industrial developments to the north, and the Lower Penitencia Creek to the east (Figure 2).

An existing earthen embankment/levee, with a walking trail forms the eastern boundary of the site, and is identified within a Santa Clara Valley Water District (SCVWD) easement. The crest of the levee is approximately 7 feet above the eastern portion of the site, with a slope gradient of roughly 2:1 or flatter. The southeastern end of the channel (approximately 160 linear feet) is concrete lined. The remaining portion of the channel bank is earthen and appears to be approximately 12 feet high with a slope gradient of approximately 2:1 or flatter.

During our site reconnaissance, the creek was flowing from south to north with approximately ½ foot of water. The earthen portions of the channel banks consisted of dry seasonal vegetation.

1.3 PROPOSED DEVELOPMENT

The Civil Engineering grading plans and topographic maps were not available at the time. However, we understand the site is under consideration for redevelopment into a townhome and single-family home project, totaling roughly 160 to 170 units and lots. Below-grade levels are not planned; however, a tuck-under townhome configuration is under consideration. We anticipate two- to three-story at-grade buildings of wood-framed construction resulting in light to moderately light building loads. We understand the site will likely receive at least 3 feet of fill owing to flood zoning.

1.4 SITE HISTORY AND ENVIRONMENTAL STUDIES

We reviewed individual aerial photographs of the site dated 1939, 1948, 1956, 1968, 1974, 1980, 1993, 1998, 2005, 2006, 2009, 2010, and 2012. Review of the aerial photographs indicates the site consisted of undeveloped land appearing to consist of marsh-like vegetation from at least 1939 to at least 1948. In the 1956 through 1979 aerial photographs, the site appears to have consisted of open agricultural land, and by 1980, the site appears to have been developed into a golf course. The site has been utilized as a commercial building with surrounding paved parking areas since 1987 to present day. The site may have received fill in order to achieve current grades from the prior use as a golf course, including significant amounts of fill to raise grades from former depressions such as sand traps.

Based on our discussions with SCVWD personnel, we understand the existing earthen embankment levee (identified as the west levee) located along the eastern bound of the site was constructed in the early 1980s and last improved in the late 1980s as a condition of the present-day development.

We performed a phase I environmental site assessment (ESA) for the site concurrent with this geotechnical study. During our ESA, we reviewed various City of Milpitas Building Department historic files of the site. The ESA is submitted under separate cover with pertinent items summarized below:

- In December 1980, Terratech Inc. (Terratech) performed a geotechnical report for the overall Milpitas Golf Course. The overall site conditions were generally described to consist of *earth mounds and a few water-filled lagoons.* Based on Terratech's grading experience of the

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adjacent southeastern residential development in which they reported a buried meander of Penitencia Creek was encountered during grading, Terratech indicated that old tributaries of the channel may be located on the golf course property. Furthermore, the subsurface conditions at the site were described to include near-surface compressible clays, which Terratech recommended should be preloaded or otherwise treated prior to building development. Terratech recommended an average surcharge of 3 feet of fill be placed and remain for approximately 3 months.

- In June 1986, Terratech provided geotechnical recommendations for development of the site for a commercial building, which was identified as the Dixon Landing Business Park – Building 5. Terratech recommended the building pad receive 3 feet of surcharge fill for approximately four to six weeks to compress the soft soils that underlie the site. Terratech provided foundation recommendations for continuous strip reinforced concrete footings with isolated interior footings and concrete slab-on-grade flooring.

- In the June 1986 report, Terratech provided reference to an undated Surcharge Plan prepared by Briggs Associates Architecture and Planning.

2.0 GEOLOGY AND SEISMICITY

2.1 REGIONAL AND SITE GEOLOGY

Based on regional geologic mapping by Helley and Graymer (1997, Figure 3), the soils underlying the subject site as Holocene-age alluvial floodplain deposits (Qhfp) generally consisting of sandy to silty clay. Similarly, mapping by Dibblee, et al. (2005) indicates the soils at the site are mapped as Holocene-age alluvial sediments consisting of gravel, sand and clay (Qa).

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2.2 FAULTING AND SEISMICITY

The site is not located within a State of California Earthquake Fault Hazard Zone (1982) and no known faults cross the site. Nearby active[1] common faults include the Hayward-Rodgers Creek fault, located approximately 3.1 miles east of the site; the Calaveras fault, located approximately 5.7 miles east of the site; the Monte Vista-Shannon fault, located approximately 12.9 miles southwest of the site; the San Andreas fault, located approximately 16.2 miles west of the site; and the Greenville fault, located approximately 20.0 miles east of the site. Figure 6 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region.

An unnamed extension of the Silver Creek fault, located approximately 1.4 miles west of the site, is listed on the USGS fault database (2010) as a quaternary aged fault (evidence of movement in the past 1.6 million years), and as such is considered potentially active.

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

Because of the presence of nearby active faults, the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (>M7) earthquakes have been recorded and can be expected to occur in the future. Ground motions are typically expressed as a fraction of the acceleration due to gravity (g). As described in the 2013 California Building Code (CBC), the calculated geometric mean peak ground acceleration for the project site is 0.68g (PGA_M).

3.0 FIELD EXPLORATION

The sections below summarize our field exploration activities and laboratory testing; as well as ground surface, subsurface, and groundwater conditions.

3.1 FIELD LOGGING

The field exploration for this study included drilling seven exploratory borings within the project site. The field exploration for the prior geotechnical feasibility assessment (unpublished, 2014) included advancing six cone penetration test (CPT) probe soundings within the site on May 30, 2014. Figure 2 presents the approximate locations of the exploratory borings and CPT probes obtained by taping or pacing from existing features. As a result, the mapped locations should be considered only as accurate as the methods used to determine them. We permitted and backfilled the explorations in accordance with the requirements of the Santa Clara Valley Water District.

3.1.1 Exploratory Borings

The exploratory borings were performed on July 7 and 8, 2014, and consisted of drilling seven borings to a maximum depth of approximately 50½ feet below existing grade at the approximate locations shown on Figure 2. The borings were performed using a track-mounted drill rig using 8-inch-diameter mud rotary and hollow-stem auger drilling methods.

The borings were logged in the field and soil samples were collected using either a 2½-inch inside diameter (I.D.) California-type split-spoon sampler fitted with 6-inch-long stainless steel liners, a 2-inch outside diameter (O.D.) Standard Penetration Test split-spoon sampler, or a hydraulically pushed 3-inch outside diameter (O.D.) Shelby Tube Sampler. The penetration of the drive samplers into the soil materials was recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring logs record blow count results as the actual number of blows required for the last one foot of penetration; no conversion factors have been applied. The samplers were driven with an automatic trip 140-pound hammer falling a distance of 30 inches. The field logs were then used to develop the report boring logs, which are presented in Appendix A.

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

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transitions may be gradual. Select samples recovered during drilling activities were tested to determine various soil characteristics as described in Section 3.2.

3.1.2 Cone Penetration Test Probes

Six CPT probes were advanced within the subject site on May 30, 2014, to approximately 50 feet below ground surface (bgs) and their locations are shown on Figure 2. The CPT locations were established by taping and visual sighting from existing features and should be considered accurately located only to the degree implied by the method used. In addition, the upper 5 feet of the CPTs were hand augered for utility clearance purposes.

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-3441. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and dynamic pore pressure (U) (Robertson and Campanella, 1988). The CPT data was provided by Gregg Drilling & Testing, Inc. The CPT logs and supporting empirical data are attached as Appendix C.

3.2 LABORATORY TESTING

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

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TABLE 3.2-1
Laboratory Testing

Soil Characteristic	Testing Method	Location of Results
Natural Unit Weight and Moisture Content	ASTM D2216	Appendix A
Atterberg Limits	ASTM D4318	Appendix B
Grain Size Distribution	ASTM D422	Appendix B
#200 Wash	ASTM D1140	Appendix B
Unconfined Compression	ASTM D2166	Appendix B
Constant Rate of Strain Consolidation	ASTM D4186	Appendix B
Laboratory Miniature Vane Shear	ASTM D4648	Appendix B
Corrosivity	ASTM D1498, D4327, D4658M, D4972, G57	Appendix E

3.3 SURFACE AND SUBSURFACE CONDITIONS

The borings and CPTs were advanced in asphalt-concrete paved parking areas of the site. The pavement section at the explored boring locations typically consisted of approximately 2 to 5 inches of asphaltic concrete overlying approximately 3 to 10 inches of aggregate base.

At the boring locations on the western and southern portion of the site (2-B1, 2-B4, 2-B5, and 2-B6), we encountered approximately 1 to 2½ feet of sandy lean clay to lean clay with trace rock fragments that is underlain by lean clay and silty sand (possibly manmade fill) extending to a depth of approximately 8 to 10 feet. Below this depth, we generally encountered soft to stiff fat clay to depths ranging from approximately 15 to 18 feet ranging, with an approximately 10-foot-thick soft layer of fat clay encountered in Boring 2-B1. At Borings 2-B1 and 2-B6, we encountered medium stiff to stiff lean clays from approximate depths of 16 feet to 24 feet. At Boring 2-B5, we encountered loose silty sand from approximately 15 to 22 feet below existing ground surface (bgs). Underlying this zone, we generally encountered stiff fat clay to an approximate depth of 30 feet. Below this depth in Boring 2-B1, we encountered soft to stiff to stiff layers of clayey silt to silty clay and sandy lean clay to an approximate depth of 41 feet. Underlying this zone, we encountered stiff sandy silt and a 1-foot lens of medium stiff clayey silt to approximately 46 feet bgs. Below this depth, we encountered stiff lean clay and fat clay to the maximum explored depth of 51½ feet.

At the boring locations along the eastern portion of the site (2-B2, 2-B3, and 2-B7), we encountered approximately 1 to 4 feet of lean clay with trace rock fragments and fine-grained sand underlain by medium dense silty sand and very stiff lean clay layers (possibly manmade fill) to a depth of approximately 6 feet. Below this depth, we generally encountered very stiff fat clay to an approximate depth of 20 feet. Underlying this fat clay layer, we encountered loose to medium dense layers of alternating silty sand and sand to an approximate depth of 34 feet, with density generally increasing with depth. Below this depth, very stiff lean clay and fat clay were encountered to at least the maximum explored depth of these borings at 43½ feet.

Review of the CPT data generally indicates similar conditions when compared to the boring logs.

Two samples of the near-surface lean clay in the upper 5 feet were tested for Plasticity Index (PI) and yielded values of 16 and 27, indicating a moderate to high expansion potential for the near-surface soils. We also encountered highly expansive fat clay (PI greater than 30 anticipated) in the upper 5 feet of some boring logs.

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3.4 GROUNDWATER

Groundwater was encountered at approximate depths ranging from 7 to 20 feet in the borings at the time of drilling. Groundwater was not measured in Boring 2-B3 due to the mud rotary drilling method. Furthermore, pore pressure dissipation tests performed at 1-CPT1, 1-CPT2, and 1-CPT6 indicate static groundwater levels of approximately 9½ feet, 6½ feet, and 8 feet bgs, respectively.

4.0 GEOLOGIC AND GEOTECHNICAL HAZARDS

From a geologic and geotechnical standpoint, the study area appears to be suitable for development of a residential neighborhood, provided the recommendations and guidelines provided in this report are implemented during project planning, design and construction.

The primary hazards and risks associated with known and potential geologic hazards common to the greater San Francisco Bay Region and related to the planned development are discussed in the following sections of this report. These potential concerns and other geotechnical issues relevant to the study area are discussed below.

4.1 SEISMIC HAZARDS

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

Based on topographic and lithologic data, risk from earthquake-induced regional subsidence or uplift, tsunamis, landslides and seiches is considered low at the site. Furthermore, the site is not mapped within a tsunami inundation area based on review of the Tsunami Inundation Map for Emergency Planning for the Milpitas Quadrangle prepared by the California Geological Survey (CGS, 2009).

4.1.1 Ground Rupture

As described above, the site is not located within a State of California Earthquake Fault Hazard Zone (1982) and no known active faults cross the site. Therefore, it is our opinion that the risk of ground rupture within the subject site is low.

4.1.2 Ground Shaking

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

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

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4.1.3 Liquefaction Susceptibility

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 sand below the groundwater table. Empirical evidence indicates that silts and low plasticity clays (fine-grained soils) are also potentially liquefiable, though this phenomenon is commonly referred to as cyclic softening. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective confining stress from the overlying soil, the soil may undergo deformation. If the soil consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur.

The site is mapped within a potentially liquefiable zone as identified by the State of California Seismic Hazard Zone Map (2004, Figure 4). In addition, the site is mapped in an area susceptible to liquefaction by the Santa Clara County Geologic Hazard Zones Map (2012, Figure 5).

4.1.3.1 Basis and Summary of Analyses

To assess liquefaction potential, we advanced six CPTs and drilled one boring using mud rotary drilling methods and six borings using hollow-stem auger drilling methods. Two of the CPTs and borings (1-CPT1 / 2-B1 and 1-CPT5 / 2-B6) were drilled roughly 5 to 10 feet apart to correlate CPT and boring data (matched pairs).

The CPT logs provided a continuous soil profile using an empirically generated soil behavior type index, Ic. Liquefaction analyses were performed on the CPT probes using the computer program CLiq. The program allows the user to include or exclude soil layers based on soil type. Through trial and error and calibration of the analyses, a cutoff value for the soil behavior type index, Ic, is selected.

Liquefaction analysis was performed for the mud rotary Boring 2-B3 while liquefaction analyses for the borings using hollow-stem auger methods were not performed since SPT blow counts for saturated samples from hollow-stem auger borings are unreliable. Visual soil classification and laboratory testing of samples from the matched-pair borings allowed for correlation between the CPTs and borings.

We assigned a design groundwater level of 5 feet below the existing ground surface based on review of Plate 1.2 of the Seismic Hazard Zone Report for the Milpitas Quadrangle (2001), a PGA of 0.68g based on the 2013 CBC PGA_M, and a moment magnitude (M_w) of 6.8. Our analyses were based on guidelines provided in DMG Special Publication 117A (2008) and methods developed by methods developed by (NCEER 1998) (2001), Moss et al. (2006, and Idriss and Boulanger (2008), Seed (2003), and Bray and Sancio (2006).

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4.1.3.2 Potentially Liquefiable Sand and Silty Sand Layers

As described in Section 3.3, we encountered loose to medium dense sand and silty sand layers at depths ranging from 5 to 34 feet at exploration locations. Thicknesses range from approximately 4 up to 12 feet, with the thicker zones generally encountered in exploration locations located along the eastern portion of the site and starting at an approximate depth of 20 feet. The analyses generally indicated that most loose to medium dense sand and silty sand layers encountered are potentially susceptible to liquefaction.

4.1.3.3 Potentially Liquefiable Silt and Clay Mixtures

To evaluate liquefaction potential of fine-grained soils, we collected and tested select samples of the encountered silt, lean clay, and fat clay deposits below the design groundwater level of 10 feet, most notably from the matched pair borings, 2-B1 and 2-B5. We utilized the criterion published by Bray and Sancio (2006) and Seed (2003) to evaluate the liquefaction susceptibility of these fine-grained soils based on liquid limit ratio (moisture content divided by liquid limit). This evaluation indicates the majority of the fat clay and lean clays encountered at depths throughout the explored borings are not potentially susceptible to liquefaction based on currently available methods. However, the tested lean clay with sand deposits encountered in Boring 2-B6 at approximately 20 feet bgs and in Boring 2-B1 at approximately 15 feet bgs are potentially susceptible to liquefaction while the underlying lean clay sampled at approximately 25 feet bgs in Boring 2-B6 is not susceptible. In addition to the select lean clay with sand deposits, the majority of soils identified as potentially susceptible to liquefaction consisted of transitional soils such as clayey silt, silty clay, sandy lean clay, and sandy silt deposits. The clayey silt encountered in Boring 2-B6 from approximately 6½ to 9 feet bgs is potentially susceptible to liquefaction based on currently available methods, in addition to the sandy silt layer encountered in Boring 2-B1 from approximately 41 to 45 feet bgs and the sandy lean clay lens encountered in Boring 2-B6 from approximately 32 to 33½ feet bgs.

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The matched pairs were used to allow us to correlate data collected from the borings to the results of the CPTs. The borings were compared directly with the I_c values that allowed for us to establish bounds for I_c to treat soil appropriately during liquefaction assessment. In pairing the boring log and the I_c vs. depth interpreted from the CPT results, we conclude that, for this site, soil with a calculated I_c greater than 2.6 is clay and does not appear to be liquefiable. When compared to the empirical correlations of the CPT data, generally the transitional fine-grained soils identified as potentially susceptible to liquefaction based on criterion published by Bray and Sancio (2006) and Seed (2003) and as described above, are generally identified with an I_c ranging generally from 2.5 to approximately 2.6.

We evaluated the liquefaction potential of the CPTs using the computer program CLiq Version 1.7 by Geologismiki Geotechnical Software. Our analysis was performed using methods published by Idriss and Boulanger (2008). We have included the CLiq output liquefaction analyses of our CPTs considering existing site grades in Appendix D.

Potentially liquefiable soils are commonly susceptible to earthquake-induced ground settlement and surface rupture (sand boils) as discussed in the following sections.

4.1.3.4 Liquefaction-Induced Ground Settlement

We evaluated potential post-liquefaction ground settlement at the site considering CPT and SPT boring conditions using methods outlined by Zhang et al. (2002) and Ishihara and Yoshimine (1992), respectively.

Based on our liquefaction analysis, when considering existing site conditions and up to 6 feet of fill, we compute volumetric strain settlements of 1.8 to 4.8 inches are possible under extreme seismic event, as summarized in the following table:

TABLE 4.1.3.4-1
Total Liquefaction-Induced Settlement Estimates

Exploration Location	Estimated Total Vertical Settlement (inches)
1-CPT1	3.4
1-CPT2	4.8
1-CPT3	4.1
1-CPT4	1.8
1-CPT5	2.9
1-CPT6	3.8
2-B3	2.8

Recommendations to address this concern are provided in subsequent sections.

4.1.3.5 Liquefaction-Induced Surface Rupture

In addition to the above liquefaction analysis, we also evaluated the capping effect of 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.

We based our analyses and review on guidelines provided by Ishihara (1985) and Youd and Garris (1995). Our assessment was performed considering both cases for existing conditions as well as up to 3 feet of fill. Based on our analyses and review, when considering existing conditions, it appears the project site does not have a thick enough cap of non-liquefiable soil to prevent venting of liquefiable soil. When considering 3 feet of fill, soil conditions at exploration locations 1-CPT1, 1-CPT4, 1-CPT5, and 2-B1 through 2-B5 indicate a risk for liquefaction-induced surface rupture or sand boils during a strong seismic event.

4.1.4 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.

Based on our site reconnaissance an existing earthen embankment levee with a walking trail along its crest forms the eastern boundary of the site and is identified as a Santa Clara Valley Water District (SCVWD) easement. The outboard slope of the embankment levee (creek side) consists of a generally uniform 1½:1 or flatter slope approximately 7 feet tall (field approximated). An approximately 160 lineal foot inboard portion of the Lower Penitencia Creek channel bank is concrete lined adjacent to the southeastern site area. The northern balance of the inboard channel bank appeared to consist of dry seasonal vegetation and comprised of an approximately uniform 1:1 to 1½:1 slope. During our site reconnaissance, the creek was observed to be low flowing, generally south to north.

The Lower Penitencia Creek channel bank is a free face that may be potentially susceptible to lateral spreading. We performed an assessment of lateral spreading potential for the site using guidelines developed by Youd et al. (2002). According to guidelines in this study, soils with corrected SPT blow counts, greater than 15 are generally considered not susceptible to lateral spreading. Based on review of empirically correlated blow counts for coarse-grained deposits in CPT Probes 1-CPT2 and 1-CPT6 performed adjacent to the creek channel, blow counts greater than 15 were generally logged. This review indicates that lateral spreading is unlikely to significantly impact the proposed development.

Based on the above, we believe the potential for lateral deformation occurring at proposed development locations is low provided the recommendations outlined in subsequent sections are followed. Minor ground cracking at or near the eastern site boundary may occur.

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4.2 SLOPE STABILITY

Detailed slope stability analyses of the Lower Penitencia Creek channel bank were performed to further assess potential lateral spreading/slope stability concerns, and evaluate proposed setback distances. We also assumed site grades will need to be raised due to flooding requirements.

4.2.1 Strength Parameters

To evaluate slope stability, soil strength parameters were selected for use in analyses based on subsurface conditions during field explorations and laboratory testing. We used undrained shear strengths from field exploration and laboratory testing for clays since we are modeling site performance during earthquake loading. Where our analyses indicate that soil layers are liquefiable; however, we used residual shear strengths (Seed and Harder, 1990) for stability modeling.

A summary of the strength parameters recommended for use in slope stability analyses is presented below. These are based on our experience with similar soil types on other projects, and review of subsurface data and testing onsite. We expect that actual factors of safety are somewhat higher than demonstrated in our analyses.

TABLE 4.2.1-1
Summary of Strength Parameters

Material	Strength Parameters	
	Friction Angle	Cohesion (psf)
Levee Fill	0	1,000
Engineered Fill	18	500
Lean Clay/Fat Clay	0	1500
Sand and Silty Sand (Seismic)	0	600

4.2.2 Slope Stability Analyses

Limit equilibrium slope stability analyses for both static and pseudo-static conditions were conducted. The purpose was to assess the proposed development and confirm appropriate factors of safety for structures and improvements.

We performed the analyses on cross-sections through the eastern portion of the site and considering up to 3 feet of engineered fill using the computer-aided program SLIDE©. The program analyzed circular and non-circular slip surfaces within a defined start and end section.

We performed the static and pseudo-static slope stability analyses in general accordance with guidelines provided in the California Geological Survey's SP 117A (2008). A seismic coefficient (k) of 0.26g was developed to model slope stability under seismic shaking conditions using the simplified methods of Blake et al. (2002). This seismic coefficient was developed using a PGA of 0.68g, a Magnitude of 6.8, a distance of less than 10 km, and a threshold displacement of 15 centimeters.

Using the above slope stability analysis methods, factors of safety of 1.5 and 1.0 for static and pseudo-static loading conditions, respectively, were targeted. Satisfactory results were achieved. We recommend slope stability be reassessed once final land plans are available.

4.2.3 Seismic Slope Deformation

To estimate seismically induced deformation of the creek bank slope, we performed a deformation analysis as recommended in the California Geological Survey's SP 117A using the simplified Newmark analysis method of Bray and Travasarou (2007). According to SP117A (Blake et al., 2002), seismic deformation for stability analyses achieving a minimum FS of 1.0 or above will generally be in the range of or less than 15 cm (6 inches).

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This estimate represents an indication that deformations may have minor impact (settlement and/or ground cracking) to portions of the site in close proximity to the Lower Penitencia Creek channel. The risk will diminish with increasing distance from the channel. It is important to note that developers of this approach consider the results of these analyses to be indices of expected seismic performance and not predictions of actual slope displacements.

4.3 LOWER PENITENCIA CREEK EMBANKMENT LEVEE

Detailed geologic assessment and stability and seepage analyses of the adjacent Santa Clara Valley Water District embankment levee were outside the scope of services for this study.

Based on a brief review of available documentation provided by the SCVWD, we understand the eastern levee (located east of the site, across the creek) of the adjacent Penitencia Creek was recertified in 2009. We understand the opportunity for levee accreditation of the west levee (abutting the eastern boundary of the site) is pending improvements of Upper Penitencia Creek. We also understand that the United States Army Corps of Engineers are in-progress on planning and design for the Upper Penitencia Creek.

For the purpose of levee recertification of the eastern levee (opposite side of creek channel), a geotechnical study was performed in July 2009 by AMEC Geomatrix, Inc. (AMEC). Based on our review of this study, AMEC concluded that earthquake-induced settlement is not expected to exceed more than 1 to 4 inches due to seismic settlement and portions of the west levee slope under seismic loading may deform "somewhat".

Based on our assessment of geologic hazards at the site, we anticipate similar concerns for the western levee adjacent to the subject site, as presented in the AMEC report for the eastern levee. Specifically, in our opinion there is potential for liquefaction-induced settlement and lateral spreading/slope deformation of the levee, including deformation away from or toward the site as a result of strong ground shaking.

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4.4 FLOODING

Based on a review of flood insurance rate mapping (Map No. 06085C0058J) prepared by the Federal Emergency Management Agency (FEMA, 2014), the project site is mapped within a special flood hazard area (SFHA) Zone AH which is subject to inundation by the 1 percent annual chance flood. A base flood elevation of 15 is indicated. Furthermore, FEMA describes Zone AH as flood depths of 1 to 3 feet (usually areas of ponding).

The project Civil Engineer should assess if the site is located above or below the 100-year flood elevation.

4.5 COMPRESSIBLE SOIL

Soils are subject to settlement when a new loading scenario is introduced by structures and fill. Consolidation settlement is highly dependent on the permeability of the deposit. Consequently, sandy soils will settle almost immediately, whereas clayey soils will settle much more slowly.

The western portion of the site (Figure 2) is underlain by soft, highly expansive fat clay up to approximately 10 feet thick at depths ranging from roughly 8 to 18 feet. We performed a constant rate of strain (CRS) consolidation test on one soil samples from Boring 2-B1 collected at a depth of 11 feet. The consolidation test result indicates that these clayey deposits are normally consolidated and highly compressible.

We performed settlement analyses for the proposed development based on the following.

- Placement of up to 3 feet of engineered fill to raise site grades.
- Assumed average bearing pressure of 400 pounds per square foot (psf) for proposed residential structures for dead-plus-live loads considering a post-tensioned mat foundation.

Based on the above, we estimate the western portion of the site may undergo total consolidation settlement of approximately 4 to 6 inches. Consolidation settlement is usually most significant within the first year after placement of new fill or addition of structural load. Based on laboratory test results, we anticipate approximately 50 percent of the estimated consolidation settlement to occur within the first 6 months to 1 year after placement of fill or structural loads. Settlement rates will then slow down and gradually achieve the estimated settlement. We estimate it could take 2 to 5 years to achieve 90 percent of the estimated consolidation settlement.

In our opinion, the above settlement estimates exceed a tolerable amount for the proposed development. A mitigation measure that has been successfully used for this type of development and site conditions is surcharging. Surcharge fill is placed to pre-consolidate the compressible deposits in areas subject to future increased loads.

If a surcharge program is planned to pre-consolidate soft deposits, then surcharge fill should be uniformly placed over areas where structures or raised grades are planned to adequately drive consolidation of the highly compressible soft clay. The surcharge fill area, grades and extent, should be determined by ENGEO, depending on surcharge period, building loads, and surcharge materials. Surcharge fill should remain in place for a period of time determined to be sufficient to allow the desired degree of consolidation to be achieved, so that the risk of settlement is sufficiently reduced for the planned development. We recommend foundation design consider up to 1 inch of residual static-settlement.

Depending on the construction schedule, implementation of a surcharge program may be accelerated by installing wick drains, which allow rapid pore pressure dissipation. Recommendations for the implementation of a surcharge program are provided in a subsequent section and more specific recommendations can be provided once land plans and time

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considerations are provided. Following surcharge removal, residual settlement of the soft clay is estimated to be small and structures supported on shallow foundations, such as post-tensioned mat foundations, should be designed to withstand the estimated post-construction differential settlement.

Where utilities are installed within the soft clay layer, the weight of the utility backfill will be heavier than the soft clayey deposits removed, resulting in undesirable potential settlement of the utility pipeline. For this case, the use of special "lightweight fill" should be considered to reduce additional loading on the compressible deposits. Recommendations for underground utilities are provided in a subsequent section.

4.6 EXPANSIVE SOILS

The near-surface lean clay soils display a moderate to high expansion potential with PI testing results ranging from 16 to 27. We also encountered highly expansive fat clay (PI greater than 30 anticipated) in the upper 5 feet of some boring logs. Expansive soils shrink and swell as a result of moisture changes. This can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Therefore, construction of improvements will need to consider the potential impacts of expansive soils.

Building damage due to volume changes associated with expansive soils can be reduced by supporting structures on structural mat foundations that are designed to accommodate shrinking and swelling subgrade soils.

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

Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil, and use of a mat foundation are common, generally cost-effective measures to address the expansive potential of the foundation soils.

4.7 EXISTING FILL

Based on our review historical aerial photographs, the site was previously used as a golf course before being developed into present-day conditions of a commercial building with surrounding paved parking areas. The site may have received fill in order to achieve current grades from the prior use as a golf course, including significant amounts of fill to raise grades from former depressions such as sand traps. The site has been utilized as a commercial building with surrounding paved parking areas since 1987 to present day.

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In general, existing fill is anticipated across the site as follows:

- The site may have received fill in order to achieve current grades from the prior use as a golf course, including significant amounts of fill to raise grades from former depressions such as sand traps and drainage areas.
- Beneath and adjacent to the existing building foundation.
- Within existing utility trench and landscape areas.

The presence of undocumented fill can lead to pavement subgrade instability and differential foundation movement due to the unknown density of the fill and due to differences in material properties for structures that span fill to native irregularities. Treatment of existing or undocumented fills typically includes removal and recompaction of soil deemed suitable for reuse.

4.8 SHALLOW GROUNDWATER

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

- Groundwater was encountered at approximate depths ranging from 7 to 20 feet in the borings at the time of drilling.
- Pore pressure dissipation tests performed at 1-CPT1, 1-CPT2, and 1-CPT6 indicate static groundwater levels of approximately 9½ feet, 6½ feet, and 8 feet bgs, respectively.
- Plate 1.2 of the Seismic Hazard Zone Report for the Milpitas Quadrangle (2001) maps the highest historical groundwater in the site vicinity as roughly 5 feet deep.

Fluctuations in groundwater levels should be expected during seasonal changes or over a period of years because of precipitation changes, perched zones, changes in drainage patterns, or irrigation. Based on our field exploration and review of data, a design groundwater depth of 5 feet bgs was utilized in our design and analyses.

4.9 2013 CBC SEISMIC DESIGN PARAMETERS

Based on the subsurface conditions and local seismic sources, the following 2013 California Building Code (CBC) seismic design parameters may be used for design.

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TABLE 4.9-1
ASCE 7-10 / 2013 CBC SEISMIC DESIGN PARAMETERS

Parameter	Design Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.76
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.70
Site Coefficient, F_A	1.0
Site Coefficient, F_V	1.5
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.76
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.05
Design spectral response acceleration at short periods, S_{DS} (g)	1.17
Design spectral response acceleration at 1-second periods, S_{D1} (g)	0.70
Mapped MCE Geometric Mean Peak Ground Acceleration (g)	0.68
Site Coefficient, F_{PGA}	1.0
MCE Geometric Mean Peak Ground Acceleration, PGA_M (g)	0.68

Latitude = 37.44744; Longitude = -121.91775

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Although the site could be classified as Site Class F, as described in ASCE 7-10 Section 20.3, structures having a fundamental period of vibration of 0.5 seconds or less may be classified in accordance with Table 20.3-1 and do not require a site-response analysis.

In addition, the site could be classified as Site Class E, considering the presence of approximately 10 feet of soft, highly expansive fat clay in some exploration locations. However, we anticipate surcharging these soils, resulting in increased undrained shear strengths of greater than 500 psf.

Therefore, we classified the site as Site Class D.

4.10 SOIL CORROSIVITY CONSIDERATIONS

Two soil samples collected from the near-surface clay at approximately 1.5 feet were transported under proper chain-of-custody to CERCO Analytical, Inc. for laboratory testing. The samples were tested for redox potential, pH, resistivity, chloride ion, and soluble sulfate concentrations. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes.

The results are summarized below with a detailed description of the laboratory results contained in the report prepared by CERCO Analytical, Inc. (Appendix E).

TABLE 4.10-1
Soil Corrosivity Test Results

Sample Number and Depth	Redox Potential (mV)	pH	Resistivity* (ohms-cm)	Sulfide* (mg/kg)	Soluble Sulfate* (mg/kg)	Chloride Ion* (mg/kg)
2-B5 @ 1.5’	190	8.65	1,200	< 50	37	150
2-B7 @ 1.5’	200	8.43	980	< 50	37	93

*Results reported on a wet weight basis

As indicated in the CERCO laboratory letter (Appendix E), because of the resistivity measurements, buried metal and steel should be protected against corrosion. A corrosion consultant should provide specific design recommendations on corrosion protection for any buried metallic lines.

According to the sulfate test results by CERCO, the sulfate ion concentrations resulted in 37 mg/kg of water-soluble sulfate (SO_4). The CBC references the 2008 American Concrete Institute Manual, ACI 318 (Chapter 4, Sections 4.2 and 4.3) for concrete requirements. ACI tables provide the following sulfate exposure categories and classes and concrete requirements in contact with soil based upon the exposure risk.

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TABLE 4.10-2
Sulfate Exposure Categories and Classes

Sulfate Exposure Category S	Exposure Class	Water- Soluble Sulfate in Soil % by Weight
Not Applicable	S0	$SO_4 < 0.10$
Moderate	S1	$0.10 \leq SO_4 < 0.20$
Severe	S2	$0.20 \leq SO_4 \leq 2.00$
Very Severe	S3	$SO_4 > 2.00$

TABLE 4.10-3
Requirements for Concrete by Exposure Class

Exposure Class	Max w/cm	Min f'c (psi)	Cement Type			Calcium Chloride Admixture
			ASTM C150	ASTM C595	ASTM C1157	
S0	N/A	2500	No Type restriction	No Type restriction	No Type restriction	No restriction
S1	0.5	4000	II†‡	IP(MS), IS(<70), (MS)	MS	No restriction
S2	0.45	4500	V‡	IP(HS), IS(<70), (HS)	HS	Not permitted
S3	0.45	4500	V + pozzolan or slag§	IP(HS) + pozzolan or slag or IS(<70) (HS) + pozzolan or slag§	HS + pozzolan or slag§	Not permitted

Notes: † For seawater exposure, other types of portland cements with tricalcium aluminate (C_3A) contents up to 10 percent are permitted if the w/cm does not exceed 0.40.
‡ Other available types of cement such as Type III or Type I are permitted in Exposure Classes S1 or S2 if the C_3A contents are less than 8 or 5 percent, respectively.
§ The amount of the specific source of the pozzolan or slag to be used shall not be less than the amount that has been determined by service record to improve sulfate resistance when used in concrete containing Type V cement. Alternatively, the amount of the specific source of the pozzolan or slag to be used shall not be less than the amount tested in accordance with ASTM C1012 and meeting the criteria in ACI 4.5.1.

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

Testing was not completed for all depths of potential embedment. Once more specifics of the proposed improvements are known, we can provide additional testing and/or guidance regarding the exposure risk for sulfates.

4.11 CONCLUSIONS

From a geologic and geotechnical standpoint, the study area appears to be suitable for the proposed residential development. The main geologic/geotechnical issues at this site include potential seismic-induced settlement, potential liquefaction-induced surface rupture (sand boils), risk of lateral deformation along Penitencia Creek, presence of soft compressible clays on the western portion of site, undocumented existing fills, expansive near-surface soils, and presence of relatively shallow groundwater.

5.0 RECOMMENDATIONS

The recommendations included in this report, along with other sound engineering practices, should be incorporated in the design and construction of the project.

5.1 SURCHARGE PROGRAM

To reduce post-construction consolidation settlements, a surcharge program to "pre-consolidate" the soft clay deposits prior to site development may be used. A surcharge program would involve the placement of temporary fills, uniformly blanketing the western portion of the site until the desired degree of consolidation in these areas has occurred, as determined by a site-specific settlement-monitoring program. Based on field exploration to date, the estimated minimum surcharge area is shown on Figure 2.

For planning purposes, we assumed up to 3 feet of engineered fill will be placed to raise site grades, and the proposed structures at the site will impose average floor loads of 400 pounds per square foot (psf) considering dead plus live loads. Based on the above description, we anticipate the following surcharge program:

- Surcharge height of at least 6 feet (assume average unit weight of surcharge material is 120 pounds per cubic foot (pcf)), above final grade elevations. We estimate it could take 2 to 5 years to achieve the target consolidation settlement.

- The surcharge program timing could be accelerated with the inclusion of wick drains. For planning purposes, we assume 4- to 6-foot spacing, triangular-pattern, to achieve desired pre-consolidation in a duration of approximately 4 to 6 months.

Variable surcharge heights and wick drain spacing combinations are possible and may be feasible depending on specifics of loads and project timelines. Additionally, if desired, the use of lightweight fill may be considered to reduce surcharge height. In addition, existing fill on the site could be excavated and used as temporary surcharge fill. ENGEO should be consulted to determine recommended combinations.

5.1.1 Surcharge Placement and Wick Drain Installation Procedure

Based on our experience, the optimum construction sequence to address the existing fill and compressible soil is as follows:

- Remove and replace existing fills as recommended in Section 5.6. Compact engineered fills in accordance with recommendations in Section 5.9.

- If planned, install vertical wick drains in designated surcharge areas. Wick drains should be placed in a triangular grid pattern. Spacing of the wick drains should be 4 to 6 feet on center. Wick drains should extend to below soft clay deposits to a depth of at least 20 feet.

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- Place the recommended thickness of additional engineered fill (including anticipated additional fill to address estimated settlement). Compact engineered fill in accordance with recommendations in Section 5.9.

- Place the recommended thickness of surcharge fill. Compact surcharge fill to at least 85 percent relative compaction.

5.1.2 Surcharge and Settlement Monitoring

We recommend that settlement-monitoring plates be installed prior to surcharge placement to monitor consolidation. We also recommend the installation of vibrating wire piezometers to monitor the pore water pressure dissipation. The number and location of the settlement monitoring plates and the vibrating wire piezometers should be determined by the Geotechnical Engineer when the surcharge staging has been determined. The settlement-monitoring plates should be surveyed to determine elevations at least twice monthly for the first 2 months and once monthly until the Geotechnical Engineer has determined that the desired degree of surcharge driven preconsolidation has been achieved. All readings of settlement should be tied to benchmarks established well beyond the zone of surcharge influence.

5.2 LOWER PENITENCIA CREEK BANK SETBACK

Based on the existing depth and configuration of Lower Penitencia Creek adjacent to the site and results of our stability analysis, it is our preliminary recommendation that a minimum setback distance of 40 feet from the top of the existing creek bank be used for structures. For other development related site improvements (i.e. pavements, sidewalks, and utilities) a minimum setback distance of 30 feet from the top of the existing creek bank is recommended. These recommended setbacks are suitable provided the other remedial grading recommendations provided in this report are incorporated into the final land plan and implemented during construction. Moreover, if structures or site improvements are planned within the recommended setbacks, additional remedial grading mitigation measures may be necessary (i.e. below-grade retention systems, shear keyway, etc.) that extend to adequate depths to provide adequate stability considering the adjacent creek. As part of the 40-scale grading plan review, the remedial grading and any other recommended measures may be outlined, as necessary.

5.3 GRADING

Site grading operations should meet the requirements of the Supplemental Recommendations in Appendix F and should be observed and tested by ENGEO's field representative. ENGEO should be notified a minimum of three days prior to grading in order to coordinate its schedule with the grading contractor.

5.4 DEMOLITION AND STRIPPING

Site development should commence with the removal of the existing building and utilities, vegetation, and surface and subsurface improvements. Tree rootballs should be removed down to a

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depth of at least 3 feet below finished grade. Following the demolition of existing improvements, site development should include removal of debris, loose soil, and soft compressible materials in any location to be graded. Any soft compressible soils should be removed from areas to receive fill or structures, or those areas to serve as borrow. Vegetation and debris should be separately stockpiled from soft compressible material and existing soil fill. The actual depths of removals should be determined by the Geotechnical Engineer's representative in the field.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill. No loose or uncontrolled backfilling of depressions resulting from demolition and stripping is permitted.

Demolition recommendations for paved parking areas and existing buildings are provided in the subsequent sections.

5.4.1 Paved Parking Areas

For the existing paved parking areas, if desired, reuse of the existing asphalt concrete and aggregate base within future parking lot areas may be considered from a geotechnical standpoint provided it meets the Selection of Materials recommendations in Section 5.5.

5.4.2 Existing Buildings

From a geotechnical standpoint, reuse of concrete materials from the existing building as a low-expansive engineered fill material may be considered.

All backfill materials should be placed and compacted as engineered fill according to the recommendations in a subsequent section.

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5.5 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.

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

5.6 EXISTING FILLS

Where encountered, existing fill, utility trench backfill, and existing foundation backfill are considered undocumented and should be subexcavated to expose underlying competent native soils that are approved by the Geotechnical Engineer. If in a fill area, the base of the excavations should be processed, moisture conditioned, as needed, and compacted in accordance with the recommendations for engineered fill.

5.7 DIFFERENTIAL FILL THICKNESS

Depending upon cuts associated with removal of foundations or undocumented fills, differential fill thickness conditions could possibly arise. For subexcavation activities that create a differential fill thickness across a building footprint, mitigation to achieve a similar fill thickness across the pad is beneficial for the performance of a shallow foundation system. We recommend that a differential fill thickness of up to 5 feet is acceptable across the building footprint.

5.8 BUILDING PAD TREATMENT

To improve foundation performance for the planned structures and to mitigate the risk of liquefaction-induced surface rupture (sand boils), we recommend near-surface soils comprise uniform fill of at least 3 feet and underlain by a woven geotextile fabric (Mirafi 500X or approved equivalent). Recommendations regarding placement depths of geotextile fabric should be readdressed once land plans are available and may be modified during grading activities depending on groundwater levels.

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5.9 FILL PLACEMENT

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

The following compaction control requirements should be applied to onsite expansive materials (PI>12):

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 4 percentage points above optimum moisture content.
Required Relative Compaction:	Not less than 90 percent.

The following compaction control requirements should be applied to imported low-expansive (PI<12) fill materials:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than optimum moisture for non-expansive fills.
Minimum Relative Compaction:	Not less than 92 percent.

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

5.10 GRADED SLOPES

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

5.11 MONITORING AND TESTING

The final grading and foundation plans should be submitted to the Geotechnical Engineer for review.

It is important that all site preparations for site grading be done under the observation of the Geotechnical Engineer's field representative. The Geotechnical Engineer's field representative should observe all graded area preparation, including demolition and stripping.

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5.12 FOUNDATION DESIGN

As previously discussed, in addition to static load-induced settlement, the site has risk of significant liquefaction-induced settlement, and moderate risk of liquefaction-induced surface rupture (sand boils) as a result of a large seismic event.

As described in Section 4.1.2, the planned structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural, as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that well-designed and well-constructed structures will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

The following sections provide foundation recommendations for a post-tensioned mat foundation in accordance with the above-described guidelines. In our opinion, there is a risk of structural

and non-structural damage to the planned structures as a result of a major earthquake when considering the recommendations provided below.

If the above-described conditions are not acceptable, or if the design recommendations provided below cannot be achieved by the structural engineer, alternative recommendations and foundation options may be considered to mitigate the liquefaction hazards and reduce the risk of damage during a major earthquake.

5.12.1 Static Foundation Design Considerations

The foundation design should consider 1-inch total load-induced settlement subsequent to completion of the surcharge operations. A differential value of ½ inch may be considered and should be assumed to act between adjacent column supports or over a 30-foot distance.

The effects of the expansive site soils are accounted for in the post-tensioned mat foundation design, Section 5.12.3

5.12.2 Seismic Foundation Design Considerations

As described in Section 4.1.3.4, we estimate 2 to 5 inches of total liquefaction-induced settlement may occur at the site as a result of a strong seismic event. We provide the following liquefaction-induced settlement estimates that should be considered in foundation design.

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TABLE 5.12.2-1
Total and Differential Liquefaction-Induced Settlement Estimates

Estimated Total Vertical Settlement (inches)	Estimated Differential Vertical Settlement (inches)
5	2½

The differential values should be assumed to act over a 30-foot distance.

Furthermore, localized liquefaction-induced surface rupture (sand boils) may result in a reduction in bearing capacity and foundation subgrade soil stiffness. To model this condition, we recommend assuming that the localized soil bearing capacity and stiffness are reduced to zero. This can be modeled by designing the mat foundation to withstand an edge cantilever distance of 6 feet and an interior span distance of 15 feet. Alternatively or in addition, consider placement of geogrid below building footprints.

5.12.3 Post-Tensioned Mat Foundation Design

Using the 2004 (Third Edition) Post-Tensioning Institute "Design of Post-Tensioned Slabs-On-Ground" manual and assuming proposed soil conditions roughly match existing

conditions (consisting of moderate to highly expansive materials), we recommend the following soil criteria.

TABLE 5.12.3-1
Post-Tension Design Criteria

Condition	Center Lift	Edge Lift
Edge Moisture Variation Distance, e_m (feet)	7.0	3.7
Differential Soil Movement, y_m (inches)	0.6	1.5

The foundation design criteria may be revisited, as needed, based on actual soil conditions at the building pad subgrade elevations.

Post-tensioned mats should be designed for an average allowable soil pressure of 1,200 pounds per square foot (psf) for dead-plus-live loads, with maximum localized bearing pressures of 1,500 psf for concentrated loads. Allowable bearing pressures can be increased by one-third when considering total loads, including wind or seismic loads.

A minimum mat thickness of 12 inches is recommended. The actual thickness of the mat should be determined by the project Structural Engineer based on structural calculations. The perimeter should be thickened by at least 2 inches. The structural mats may require stiffening to reduce differential movements due to liquefaction-induced movements.

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5.12.4 Subgrade Treatment for Structural Mat Foundation Design

The subgrade material under structural mat foundations should be uniform. The pad subgrade should be moisture conditioned to a moisture content of at least 2 percentage points above optimum. The subgrade should be thoroughly soaked and approved by the Geotechnical Engineer prior to placing the reinforcement or tendons. The subgrade should not be allowed to dry prior to concrete placement.

A tough, water vapor retarding membrane should be installed below the slab 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.

5.13 UTILITY DESIGN

Utility design should consider the following.

- Providing flexible connections for building utilities that allow for 2½ inches of vertical movement without breaking.

- Utilities on the project should be designed with either flexible materials or with flexible joints that allow the utility line to move at least 2½ inches over a distance of 30 feet without breaking.

Gravity utilities and streets should be as steep as practical to reduce the impacts of loss in slope due to settlement.

5.14 SECONDARY SLAB-ON-GRADE CONSTRUCTION

This section provides guidelines for secondary slabs such as exterior walkways, driveways, steps, approach ramps, and sidewalks.

Secondary slabs-on-grade should be constructed structurally independent of the foundation system. This allows slab movement to occur with a reduced potential for foundation distress. Secondary slabs-on-grade should be designed by the Structural Engineer specifically for their intended use and loading requirements. Cracking of conventional slabs should be expected as a result of concrete shrinkage. Slabs-on-grade should be reinforced for control of cracking, and frequent control joints should be provided to control the cracking. Such reinforcement should be designed by the Structural Engineer. In our experience, welded wire mesh may not be sufficient to control slab cracking.

Ideally, secondary slabs-on-grade should have a minimum thickness of 4 inches. A 4-inch-thick layer of clean crushed rock or gravel should be placed under slabs. Slabs should slope away from the buildings at a slope of at least 2 percent to prevent water from flowing toward the building. Turned down free edges extending at least beneath the crushed rock or gravel into compacted soil should be constructed to reduce water infiltration into subgrade soils. Waterproof barriers may also be considered.

Alternatively, and with some additional risk of cracking and/or heaving of secondary slabs, the layer of clean crushed rock or gravel beneath slabs and the turned down edges can be eliminated. If these recommendations are eliminated, it is critical that uniformity in soil moisture conditioning be achieved in subgrade soils and that subgrade soils are not allowed to dry out prior to slab construction.

5.15 RETAINING WALLS

Unrestrained drained retaining walls constructed on level ground and up to 10 feet in height may be designed using active equivalent fluid pressures as follows.

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TABLE 5.15-1
Active Equivalent Fluid Pressures

Backfill Slope Condition (horizontal:vertical)	Active Pressure (pounds per cubic foot)
Level	50
3:1	55
2:1	65

Restrained walls should be designed as drained retaining walls using an at-rest fluid pressure of 75 pcf for level backfill conditions.

Passive pressures acting on foundations may be assumed as 250 pounds per cubic foot (pcf) provided that the area in front of the retaining wall is level for a distance of at least 10 feet or three times the depth of foundation and keyway, whichever is greater. The upper 1 foot of soil should be excluded from passive pressure computations. The friction factor for sliding resistance may be assumed as 0.35. It is recommended that retaining wall footings be designed using an allowable bearing pressure of 2,000 pounds per square foot (psf). Wall footings should extend to a depth of at least 12 inches. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

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

All backfill should be placed in accordance with the recommendations provided above for engineered fill. Light equipment should be used during backfill compaction to reduce possible overstressing of the walls. The foundation details and structural calculations for retaining walls should be submitted for review.

5.16 PRELIMINARY PAVEMENT DESIGN

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

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5.16.1 Flexible Pavement

Based on soils encountered in our borings, we estimate that site soil will have a resistance value (R-value) of 5. The following preliminary pavement sections have been determined based on an assumed R-value of 5 and according to the method contained in Chapter 630 of Highway Design Manual by CALTRANS (revised May 7, 2012). The section below considers minimum pavement sections as specified by the City of Milpitas.

TABLE 5.16.1-1
Preliminary Flexible Pavement Design

Traffic Index (TI)	Pavement Design	
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5.0	4	8
6.0	5	10
7.0	5	14

Note: Class 2 aggregate base material with a minimum R-value of 78

These sections are for estimating purposes only. Actual sections to be used should be based on R-value tests performed on samples of actual subgrade materials recovered at the time of grading. Pavement construction and all materials should comply with the requirements of the Standard Specifications of the State of California Department of Transportation, Civil Engineer, and appropriate public agency.

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5.16.2 Rigid Pavement

We developed recommended pavement sections using the Portland Cement Association Thickness Design for Concrete Highway and Street Pavements manual (1995) based on the assumed subgrade soil type. We recommend the following minimum design sections for rigid pavements:

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

5.16.3 Pavement Subgrade Preparation

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

- All pavement subgrades should be scarified to a depth of 10 to 12 inches below finished subgrade elevation, moisture conditioned to at least 2 percentage points above optimum moisture content, and compacted to at least 95 percent relative compaction and in accordance with City of Milpitas requirements.

- Subgrade soils should be in a stable, non-pumping condition at the time aggregate baserock materials are placed and compacted. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented. Yielding materials should be appropriately mitigated with suitable mitigation measures developed in coordination with the client, contractor and Geotechnical Engineer.

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

- Aggregate baserock materials should meet current Caltrans specifications for Class 2 aggregate baserock and should be compacted to at least 95 percent of maximum dry density at a moisture content of at least optimum. Proof-rolling with a heavy wheel-loaded piece of construction equipment should be implemented after placement and compaction of the aggregate base. Yielding materials should be appropriately mitigated with suitable mitigation measures developed in coordination with the client, contractor and Geotechnical Engineer.

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

5.17 SHALLOW GROUNDWATER AND DEWATERING CONSIDERATIONS

If underground structures, utilities, or foundations extend approximately 5 feet or more below existing grades, shallow groundwater may impact the planned construction.

- The potential impacts to proposed foundations may be reduced by keeping pad subgrades near or above existing grades.

- Construction of underground utilities may require trench dewatering. In addition, saturated unstable trench bottoms may require overexcavation and placement of geotextile stabilization fabric prior to utility installation.

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- Assessment of dewatering and subsurface water migration and rates should be made during initial construction excavation procedures to determine level of groundwater control and dewatering, as necessary.

- Pavement subgrades may also become unstable in areas with high groundwater. Unstable pavement subgrades can be over-excavated and backfilled with aggregate base and fabric or treated with lime prior to paving.

5.18 SURFACE DRAINAGE

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

All surface water should be collected and discharged into outlets approved by the Civil Engineer. Landscape mounds must not interfere with this requirement.

All roof stormwater should be collected and directed to downspouts. Stormwater from roof downspouts should not be allowed to discharge directly onto the ground surface in close proximity to the foundation system, such as through the use of spashblocks. Rather, stormwater from roof downspouts should be directed to a solid pipe that discharges into the street or to an outlet approved by the Civil Engineer. If this is not acceptable, we recommend downspouts discharge at least 5 feet away from foundations. Alternatively, engineered stormwater systems can be developed under the guidance of ENGEO.

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5.19 STORMWATER INFILTRATION AND BIORETENTION AREAS

Based on fines content and density, the near-surface site soils are expected to have low to moderate permeability values to handle stormwater infiltration. However, infiltration will likely occur both vertically (as desired), and laterally (possibly as undesired seepage toward improvements). In our opinion, lateral seepage could lead to detrimental impacts (such as settlement) to adjacent improvements that are planned in close proximity to stormwater infiltration areas (such as bioretention areas), if not mitigated. Recommendations are provided below.

If bioretention areas are implemented, we recommend that, where practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. Where this is not practical, bioretention areas located within 5 feet of structural onsite or offsite improvements can either:

1. Be constructed with structural side walls 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).

In addition, one of the following options should be followed.

1. 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.

2. Alternatively, and with some moderate risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas/infiltration trenches be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

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

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

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

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

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5.20 REQUIREMENTS FOR LANDSCAPING IRRIGATION

Planted areas should be avoided immediately adjacent to the buildings. If planting adjacent to a structure is desired, the use of watertight planter boxes with controlled discharge or the use of plants that require very little moisture is recommended.

Sprinkler systems should not be installed where they may cause ponding or saturation of foundation soils within 5 feet from walls. Such ponding or saturation could result in undesirable loss of compaction and consequent foundation and slab movements. Irrigation of landscaped areas should be strictly limited to that necessary to sustain vegetation. The Landscape Architect and prospective owners should be informed of the surface drainage and irrigation requirements included in this report.

5.21 UTILITIES

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

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

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

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

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Care should be exercised where utility trenches are located beside foundation areas. Utility trenches constructed parallel to foundations should be located entirely above a plane extending down from the lower edge of the footing at an angle of 45 degrees. Utility companies and Landscape Architects should be made aware of this information.

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

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the 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 ENGEO 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.

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Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include on-site 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 Zones Map
Figure 5 – Liquefaction Hazard Zone Map
Figure 6 – Regional Faulting and Seismicity

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BASE MAP SOURCE: GOOGLE EARTH PRO

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VICINITY MAP

1210 CALIFORNIA CIRCLE

MILPITAS, CALIFORNIA

PROJECT NO.: 11185.000.000		FIGURE NO. 1
SCALE: AS SHOWN		
DRAWN BY: LL	CHECKED BY: TPB	

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ORIGINAL FIGURE PRINTED IN COLOR

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N

0 FEET 150

0 METERS 75

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-CPT6 — CONE PENETRATION TEST LOCATION

2-B7 — BORING LOCATION

ESTIMATED AREA OF PROPOSED SURCHARGE FILL PLACEMENT

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BASE MAP SOURCE: GOOGLE EARTH PRO

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SITE PLAN

1210 CALIFORNIA CIRCLE

MILPITAS, CALIFORNIA

PROJECT NO.: 11185.000.000		FIGURE NO. 2
SCALE: AS SHOWN		
DRAWN BY: LL	CHECKED BY: TPB	

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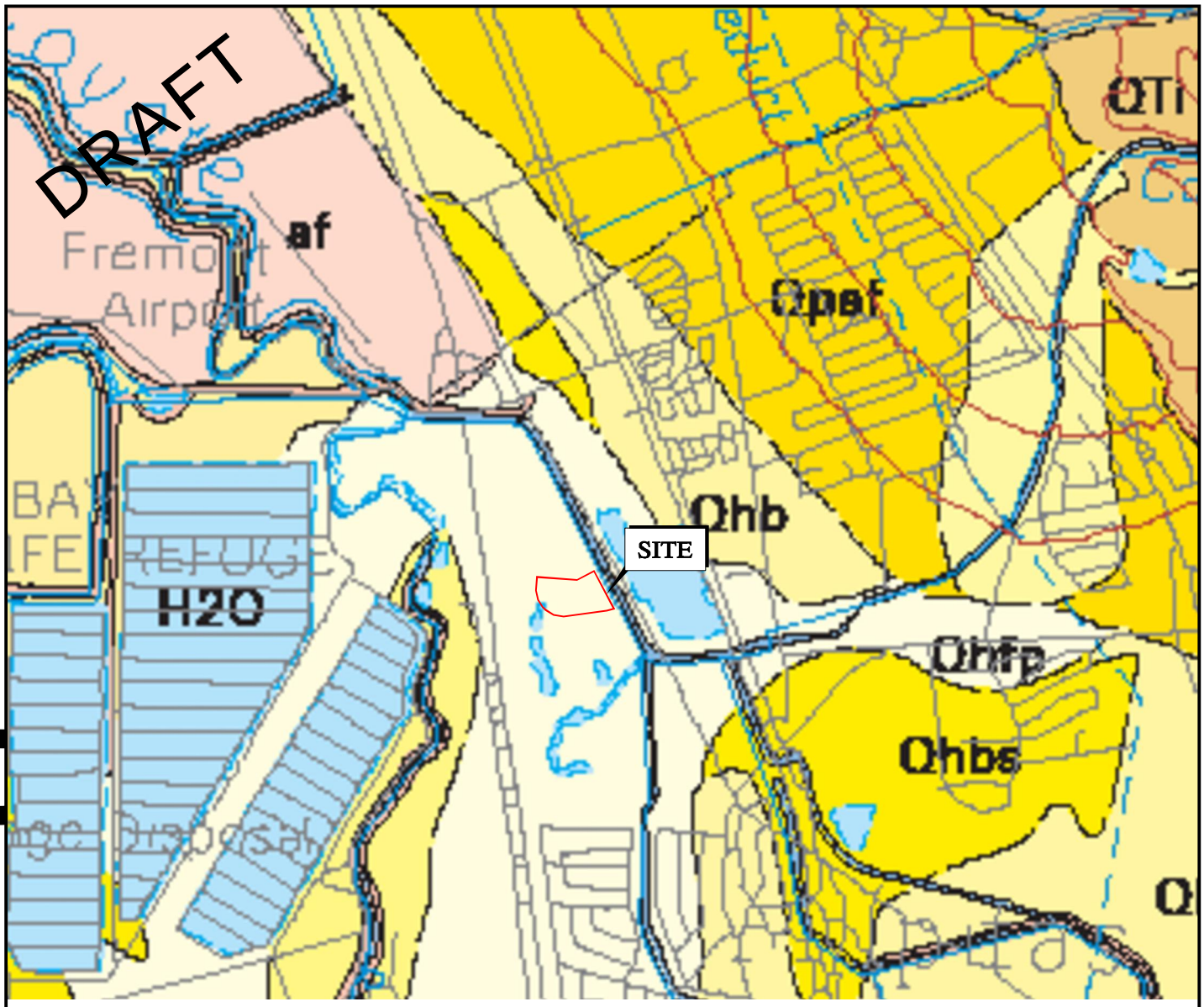

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0 FEET 2000

0 METERS 1000

EXPLANATION

af	ARTIFICIAL FILL
Qhb	BASIN DEPOSITS
Qhbs	SALT AFFECTED BASIN DEPOSITS (HOLOCENE)
Qhfp	FLOODPLAIN DEPOSITS (HOLOCENE)
Qpaf	ALLUVIAL FAN DEPOSITS (PLEISTOCENE)
QTi	IRVINGTON GRAVELS (PLEISTOCENE AND OR PLIOCENE)

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BASE MAP SOURCE: GRAYMER, 1997

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REGIONAL GEOLOGIC MAP

1210 CALIFORNIA CIRCLE

MILPITAS, CALIFORNIA

PROJECT NO.: 11185.000.000		FIGURE NO. 3
SCALE: AS SHOWN		
DRAWN BY: LL	CHECKED BY: TPB	

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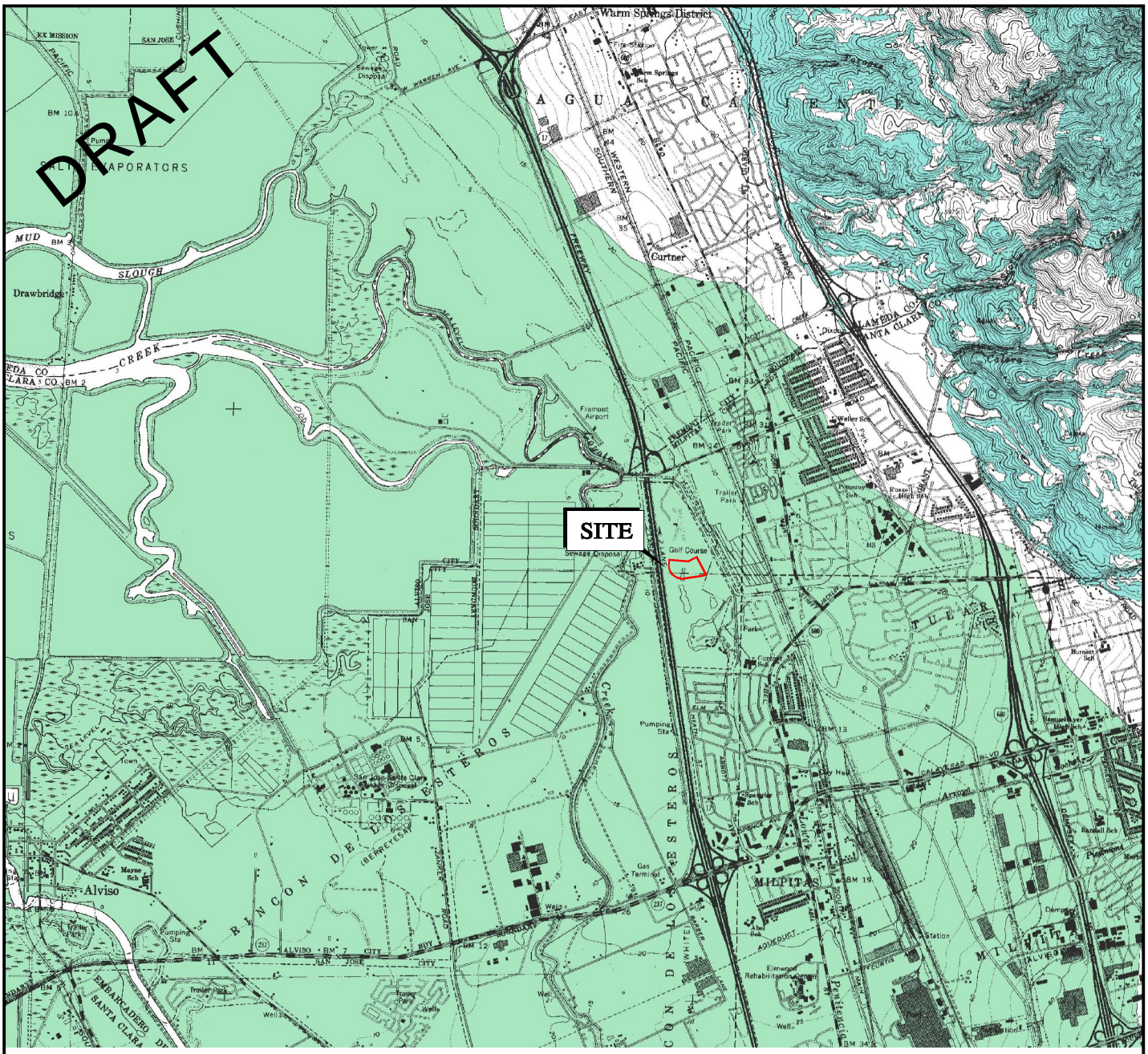

EXPLANATION

LIQUEFACTION

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

EARTHQUAKE-INDUCED LANDSLIDES

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

N

0 FEET 4000

0 METERS 2000

BASE MAP SOURCE: STATE OF CALIFORNIA SEISMIC HAZARD ZONES, 2004

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ENGEO Expect Excellence

SEISMIC HAZARD ZONES MAP

1210 CALIFORNIA CIRCLE

MILPITAS, CALIFORNIA

PROJECT NO.: 11185.000.000		FIGURE NO. 4
SCALE: AS SHOWN		
DRAWN BY: LL	CHECKED BY: TPB	

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Alameda County

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

PARCELS

COUNTY BOUNDARY

LIQUEFACTION HAZARD ZONES

BASE MAP SOURCE: SANTA CLARA COUNTY GEOLOGIC HAZARD ZONES, 2012

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N

0 FEET 3000

0 METERS 1500

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LIQUEFACTION HAZARD ZONE MAP

1210 CALIFORNIA CIRLE

MILPITAS, CALIFORNIA

PROJECT NO.: 11185.000.000	FIGURE NO. 5
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: TPB

ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\DRAFTING2_Dwg_10000 to 12999\11185\000\GEX\11185000000-GEX-5-LiqHazMap-0714.dwg Plot Date: 7-30-14 llee

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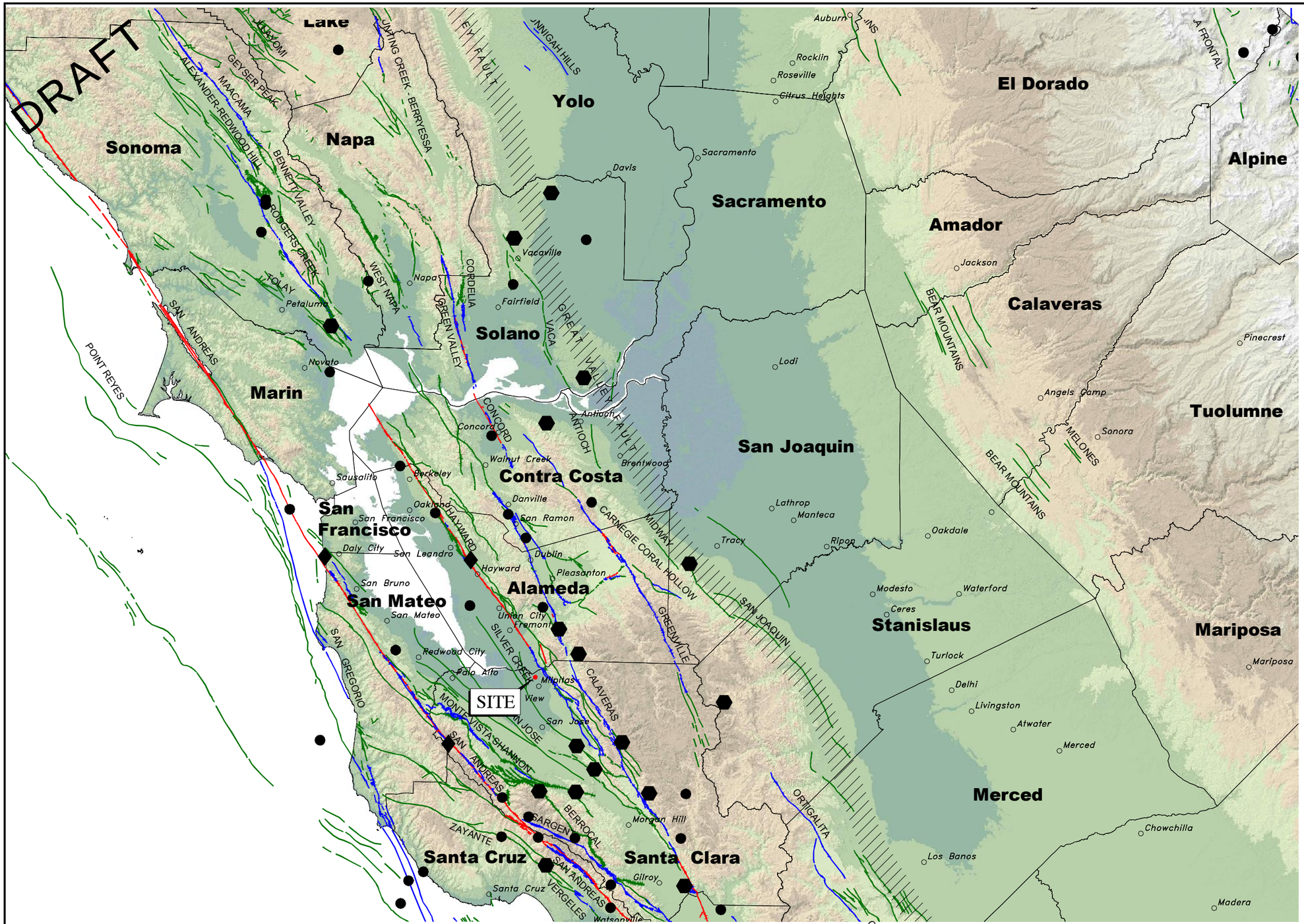

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BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

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REGIONAL FAULTING AND SEISMICITY
1210 CALIFORNIA CIRCLE
MILPITAS, CALIFORNIA

PROJECT NO.: 11185.000.000		FIGURE NO 6
SCALE: AS SHOWN		
DRAWN BY: LL	CHECKED BY: TPB	

G:\Drafting\DRAFTING2_Dwg_10000 to 12999\11185\000\GEX\11185000000-GEX-6-RegFaultMap--0714.dwg Plot Date: 7-30-14 llee

ORIGINAL FIGURE PRINTED IN COLOR

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APPENDIX A

Key to Boring Logs
Boring Logs
(ENGEO, 2014)

KEY TO BORING LOGS

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

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

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

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE					CLEAR SQUARE SIEVE OPENINGS			
200	40	10	4	3/4 "	3"	12"		
SILTS AND CLAYS	SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE			

RELATIVE DENSITY

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

CONSISTENCY

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

SAMPLER SYMBOLS

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

MOISTURE CONDITION

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

LINE TYPES

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

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

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

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

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LOG OF BORING 2-B1

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/7/2014
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: G. Cubbon / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3 inches asphaltic concrete over 10 inches aggregate base												
			SANDY LEAN CLAY (CL), brown, stiff, moist, fine-grained sand, trace rock fragments [FILL]											1.5	PP
						13								3.0*	PP
			LEAN CLAY WITH SAND (CL), gray brown, very stiff, moist, fine-grained sand [FILL?]												
	1														
			SILTY SAND (SM), brown, medium dense, moist, fine-grained sand [FILL?]												
5						12				23	16.5				
	2				▽										
			FAT CLAY (CH), gray and black, medium stiff, very moist, organic odor												
						2									
10	3		FAT CLAY (CH), gray and black, soft, very moist, visible plant matter, organic odor			150 psi							443		LVS
						150 psi									
						150 psi									
						150 psi	153	51	102		56.2				
						150 psi									
						150 psi									
	4														
15			FAT CLAY (CH), gray, soft, very moist, organic odor												
						3							400*		TV
	5														
	6														
20															

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LOG OF BORING 2-B1

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/7/2014
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: G. Cubbon / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), brown, stiff, very moist			11	39	20	19	96	24.5			1.75*	PP
25	7														
	8		FAT CLAY (CH), brown mottled with gray, stiff, very moist			19	56	22	34		28			1.75*	PP
30	9														
	10		CLAYEY SILT TO SILTY CLAY WITH SAND (CL-ML), brown mottled with orange, soft, wet, fine-grained sand			8	27	20	7	69	24.1				
			SANDY LEAN CLAY (CL), gray, stiff, very moist, fine-grained sand												
35	11														
	12														
40															

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LOG OF BORING 2-B1

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/7/2014
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: G. Cubbon / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SANDY LEAN CLAY (CL), gray, stiff, very moist, fine-grained sand			10					22.4			1.5*	PP
			SANDY SILT (ML), gray, stiff, wet, 40% fine-grained sand			11				59					
	13														
45			CLAYEY SILT WITH SAND (ML), gray, medium stiff, wet, fine-grained sand			10								1.5*	PP
	14		LEAN CLAY (CL), brown mottled with gray, stiff, very moist												
	15														
50			FAT CLAY (CH), brown, stiff, very moist			16								1.5*	PP
			Bottom of boring at approximately 51.5 feet below ground surface. Groundwater encountered at approximately 7 feet below ground surface during drilling.												

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LOG OF BORING 2-B2

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/8/2014
HOLE DEPTH: Approx. 26½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: J. Ruffoni / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3 inches asphaltic concrete over 9 inches aggregate base												
			LEAN CLAY (CL), black mottled with gray, stiff, moist, [FILL]			14					29.8	89.7		4.0*	PP
	1		LEAN CLAY (CL), olive brown, very stiff, moist, [FILL?]												
5			FAT CLAY (CH), dark reddish brown, very stiff, moist			16								2.5*	PP
	2		FAT CLAY (CH), dark reddish brown, stiff, moist			12					32.5	90.1		1.6	UC
10	3		LEAN CLAY (CL), light reddish brown mottled with orange, very stiff, moist			21								2.5*	PP
	4														
15			LEAN CLAY WITH SAND (CL), light reddish brown, medium stiff, wet, 26% fine-grained sand			7				74				1.0*	PP
	5														
20	6				▽										

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LOG OF BORING 2-B2

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/8/2014
HOLE DEPTH: Approx. 26½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: J. Ruffoni / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), light reddish brown, stiff, moist to wet			5								1.5*	PP
	7														
25			SANDY CLAYEY SILT (CL-ML), dark gray, medium stiff, wet, 30% fine-grained sand			3	25	21	4	70	27.7			1.0*	PP
	8		Bottom of boring at approximately 26.5 feet below ground surface. Groundwater encountered at approximately 20 feet below ground surface during drilling.												

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LOG OF BORING 2-B3

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/8/2014
HOLE DEPTH: Approx. 43½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: J. Ruffoni / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: HSA/Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3 inches asphaltic concrete over 8 inches aggregate base												
			LEAN CLAY (CL), dark gray, very stiff, moist, 11% fine-grained sand [FILL]			16	49	22	27	89	23.5			2.5*	PP
	1		LEAN CLAY (CL), dark olive brown, very stiff, moist, [FILL?]												
5			POORLY GRADED SAND (SP), dark yellowish brown, loose, moist, fine-grained sand [FILL?]			10								2.0*	PP
	2		FAT CLAY (CH), dark gray, very stiff, moist												
			FAT CLAY (CH), dark reddish brown, very stiff, moist			12					31.5	90.2		2.0*	PP
10	3		Drilling method switched from solid flight auger to mud rotary at approximately 9.5 feet below ground surface												
			FAT CLAY (CH), grayish brown, very stiff, moist			10					31.8	90.7	2,000*	2.0*	PP TV
	4														
15															
	5		FAT CLAY (CH), dark reddish brown mottled with orange, hard, moist, trace fine-grained sand			33							2,000*	4.5+*	PP TV
	6														
20															

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LOG OF BORING 2-B3

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/8/2014
HOLE DEPTH: Approx. 43½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: J. Ruffoni / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: HSA/Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), dark gray, very stiff, moist, trace fine-grained sand			27								2.0*	
	7		SILTY SAND (SM), dark brown, medium dense, moist, fine- to medium-grained sand, poorly graded, trace gravel												
25															
	8		WELL GRADED SAND (SW), dark gray, moist to wet, fine- to course-grained sand												
			SILTY SAND (SM), dark gray, moist to wet, fine-grained sand, 12% fines			11				13					
	9														
30			WELL GRADED SAND (SW), dark gray, wet, fine- to course-grained sand, minor fines content												
	10					31	NP	NP	NP	2					
			SILTY SAND (SM), dark gray, medium dense, wet, fine- to course-grained sand												
35															
	11		FAT CLAY (CH), reddish brown mottled with gray, very stiff, moist			11								2.0*	PP
	12														
40															

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LOG OF BORING 2-B3

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/8/2014
HOLE DEPTH: Approx. 43½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: J. Ruffoni / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: HSA/Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	13		LEAN CLAY (CL), dark brown, soft, wet			17							600*	0.5*	PP TV
			Bottom of boring at approximately 43.5 feet below ground surface. Groundwater not measured owing to drilling method.												

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LOG OF BORING 2-B4

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/7/2014
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: G. Cubbon / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3 inches asphaltic concrete over 9 inches of aggregate base												
			LEAN CLAY (CL), dark gray and black, stiff to very stiff, moist, organic odor [FILL]			13	32	16	16	99				2.0*	PP
	1														
			LEAN CLAY (CL), dark gray and black, very stiff, moist, organic odor [FILL?]			19								2.5*	PP
5															
	2		SILTY SAND (SM), light brown, medium dense, very moist, fine-grained sand [FILL?]			19									
					▽										
10	3		FAT CLAY (CH), gray and black, soft, very moist, organic odor			2					37.3	82.4	300*		TV
	4														
15			FAT CLAY (CH), gray and black, stiff, very moist, organic odor			5					40.2	79.6	600*		TV
	5		Bottom of boring at approximately 16.5 feet below ground surface. Groundwater encountered at approximately 9 feet below ground surface during drilling.												

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LOG OF BORING 2-B5

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/7/2014
HOLE DEPTH: Approx. 26½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: G. Cubbon / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			5-inches asphaltic concrete over 3 inches aggregate base												
			LEAN CLAY (CL), dark brown, medium stiff to stiff, moist, trace rock fragments and asphalt concrete fragments [FILL] Sulfate = 37 mg/kg			15					18.3	109		2.5*	PP
	1		LEAN CLAY (CL), dark gray and black, stiff, moist, organic odor [FILL?]												
5			LEAN CLAY (CL), brown, stiff, moist, 8% fine-grained sand			16				92				1.5*	PP
	2		LEAN CLAY (CL), brown, stiff, moist			5								1.5*	PP
10	3		No recovery			3									
			FAT CLAY (CH), gray and brown, medium stiff to stiff, very moist			2								1.0*	PP
	4														
15			SILTY SAND (SM), gray, loose, wet, fine- to medium-grained sand, 19% fines		▽	8				19					
	5														
20	6														

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LOG OF BORING 2-B5

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/7/2014
HOLE DEPTH: Approx. 26½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: G. Cubbon / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SILTY SAND (SM), gray, loose, wet, fine- to medium-grained sand, 19% fines			8									
	7		SILTY SAND (SM), gray, loose, wet, fine- to coarse-grained sand												
25			FAT CLAY (CH), brown, stiff, very moist			10								1.5*	PP
	8		Bottom of boring at approximately 26.5 feet below ground surface. Groundwater encountered at approximately 15 feet below ground surface during drilling.												

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LOG OF BORING 2-B6

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/8/2014
HOLE DEPTH: Approx. 41½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: J. Ruffoni / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			3 inches asphaltic concrete over 6 inches aggregate base												
			LEAN CLAY (CL), dark brown, moist, rock gragments [FILL]			12					25.1	98.3		2.0*	PP
1															
5			LEAN CLAY WITH SAND (CL), dark olive brown, stiff, moist, 22% fine-grained sand [FILL?]			7				78				1.5*	PP
2			CLAYEY SILT WITH SAND (ML), brown, medium stiff, moist, fine-grained sand												
			CLAYEY SILT (ML), brown, stiff, moist			3	32	23	9		33.9	76.8		0.5*	PP
10	3		LEAN CLAY (CL), dark olive brown, soft, moist to wet												
			FAT CLAY (CH), dark brownish gray, soft, moist			150 psi	68	27	41	98			488		LVS
	4					150 psi 150 psi 150 psi 150 psi 150 psi									
15					▽										
	5														
20	6														

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LOG OF BORING 2-B6

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/8/2014
HOLE DEPTH: Approx. 41½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: J. Ruffoni / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY WITH SAND (CL), dark reddish brown mottled with orange, medium stiff, wet, fine-grained sand			4	29	17	12	67	24.4		1,000*	1.0*	PP TV
	7														
25			LEAN CLAY (CL), reddish brown to olive brown, very stiff, moist			29					20.6	108.7		4.0*	PP
	8														
	9														
30			SANDY LEAN CLAY (CL), reddish brown, medium stiff, moist to wet, fine-grained sand			16	26	18	8	52	22.1			1.5*	PP
			SANDY LEAN CLAY TO CLAYEY SAND (CL-SC), reddish brown, very stiff to dense, moist, fine-grained sand			32	27	16	11	49	15.7			3.5*	PP
35	11														
			CLAYEY SAND (SC), dark reddish brown, medium dense, wet, fine-grained sand, 21% fines			8	26	14	12	21	26.4				
	12														
40															

DRAFT

LOG OF BORING 2-B6

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/8/2014
HOLE DEPTH: Approx. 41½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: J. Ruffoni / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), reddish brown to olive brown, very stiff, moist			10	35	18	17	91	22.7			2.0*	PP
			Bottom of boring at approximately 41.5 feet below ground surface. Groundwater encountered at approximately 15 feet below ground surface during drilling.												

DRAFT

LOG OF BORING 2-B7

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/7/2014
HOLE DEPTH: Approx. 36½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: G. Cubbon / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			2 inches asphaltic concrete over 4 inches aggregate base												
			LEAN CLAY (CL), dark black, medium stiff to stiff, moist, rock fragments and glass, organic odor [FILL] Sulfate = 37 mg/kg			13								1.0*	PP
5	1		SILTY SAND (SM), brown, medium dense, moist, fine-grained sand [FILL?]			24									
	2		FAT CLAY (CH), brown and gray, very stiff, moist			14					31.8	88.4		1.25*	PP
10	3		FAT CLAY (CH), brown, very stiff, moist			13								2.0*	PP
15	4		FAT CLAY (CH), dark reddish brown mottled with orange, very stiff, moist		▽	16								1.5*	PP
20	5														
	6														

DRAFT

LOG OF BORING 2-B7

ENGEO INCORPORATED

Geotechnical Exploration
1210 California Circle
Milpitas, California
11185.000.000

DATE DRILLED: 7/7/2014
HOLE DEPTH: Approx. 36½ ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (msl): Approx. 12 ft.

LOGGED / REVIEWED BY: G. Cubbon / TPB
DRILLING CONTRACTOR: Britton Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SILTY SAND (SM), gray dark brown, loose, wet, fine-grained sand, 34% fines			9				34					
	7		WELL GRADED SAND (SW), gray, medium dense, wet, fine- to coarse-grained sand			23									
25	8		POORLY GRADED SAND (SP), gray, medium dense, wet, fine- to medium-grained sand			25									
30	9		POORLY GRADED SAND (SP), gray, dense, wet, fine- to medium-grained sand			35									
	10														
35	11		LEAN CLAY (CL), brown and gray, very stiff, very moist			34								2.5*	PP
			Bottom of boring at approximately 36.5 feet below ground surface. Groundwater encountered at 15 feet below ground surface during drilling.												

DRAFT

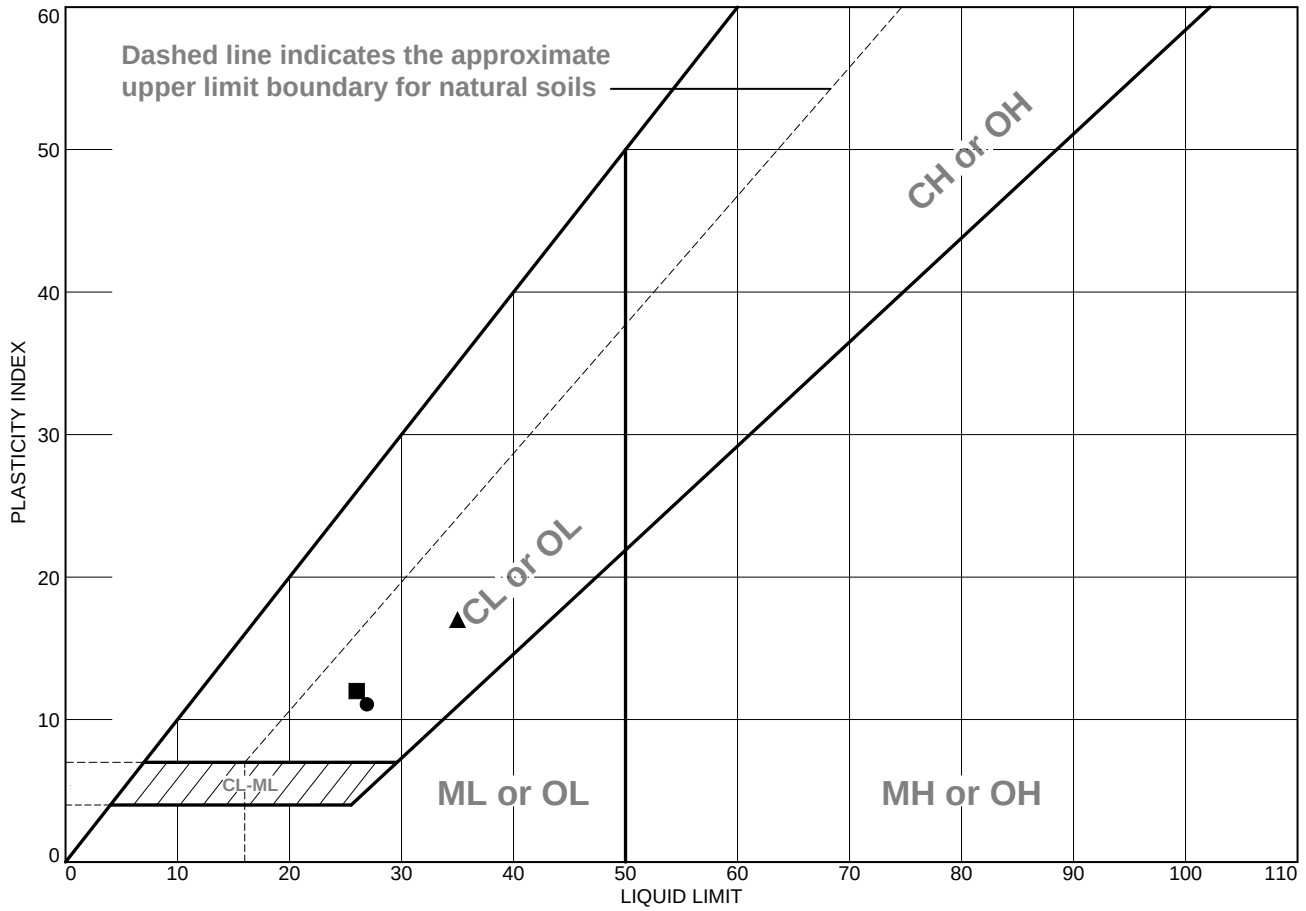
DRAFT

APPENDIX B

Laboratory Test Results

APPENDIX B

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See Exploration Logs	27	16	11		49.3	
■	See Exploration Logs	26	14	12		20.7	
▲	See Exploration Logs	35	18	17		90.7	

Project No. 11185.000.000 **Client:** iStar Financial

Project: 1210 California Circle, Milpitas, California

●**Depth:** 34 **Sample Number:** 2-B6 @ 34

■**Depth:** 38 **Sample Number:** 2-B6 @ 38

▲**Depth:** 40.5 **Sample Number:** 2-B6 @ 40.5

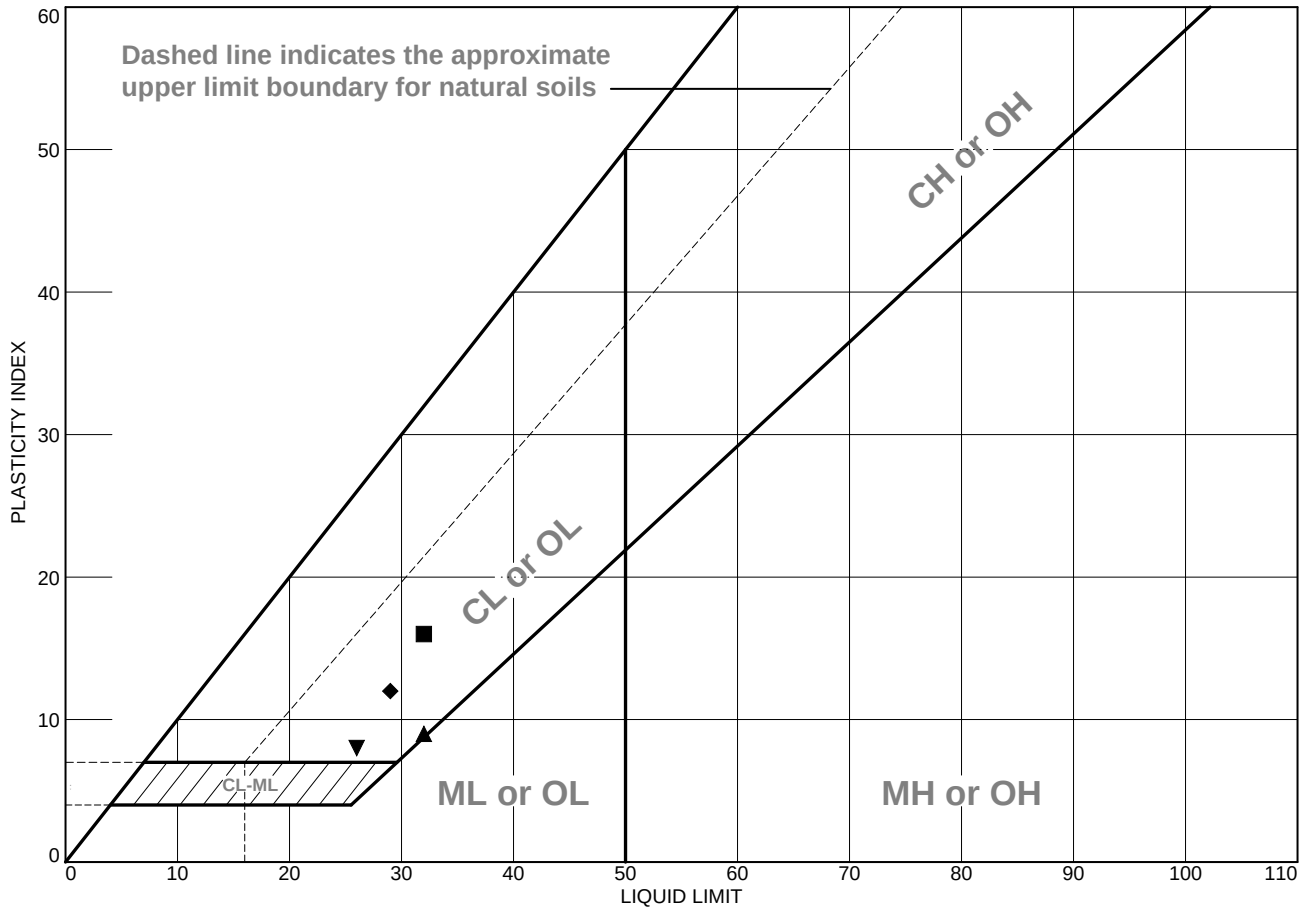
Remarks:

ENGEO INCORPORATED

Figure

Tested By: NS **Checked By:** KEL

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See Exploration Logs	NV	NP	NP			
■	See Exploration Logs	32	16	16			
▲	See Exploration Logs	32	23	9			
◆	See Exploration Logs	29	17	12		67.1	
▼	See Exploration Logs	26	18	8		52.1	

Project No. 11185.000.000 **Client:** iStar Financial

Project: 1210 California Circle, Milpitas, California

●**Depth:** 33 **Sample Number:** 2-B3 @ 33
■**Depth:** 2 **Sample Number:** 2-B4 @ 2
▲**Depth:** 9 **Sample Number:** 2-B6 @ 9
◆**Depth:** 21 **Sample Number:** 2-B6 @ 21
▼**Depth:** 32.5 **Sample Number:** 2-B6 @ 32.5

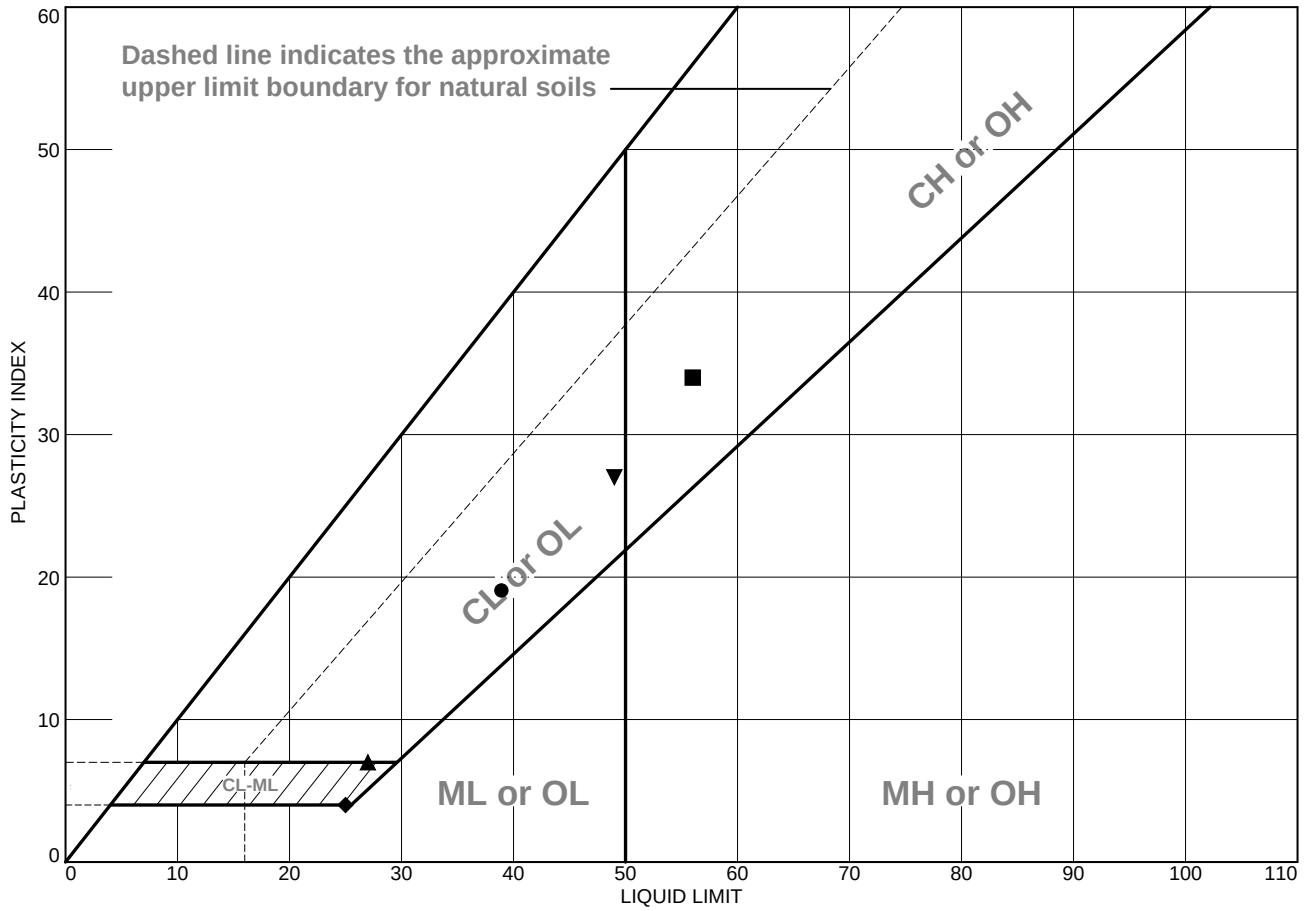
ENGEO INCORPORATED

Remarks:

Figure

Tested By: NS **Checked By:** KEL

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See Exploration Logs	39	20	19		95.5	
■	See Exploration Logs	56	22	34			
▲	See Exploration Logs	27	20	7		69.2	
◆	See Exploration Logs	25	21	4	99.8	70.0	CL-ML
▼	See Exploration Logs	49	22	27	99.5	88.6	CL

Project No. 11185.000.000 **Client:** iStar Financial

Project: 1210 California Circle, Milpitas, California

●**Depth:** 21 **Sample Number:** 2-B1 @ 21

■**Depth:** 28 **Sample Number:** 2-B1 @ 28

▲**Depth:** 33 **Sample Number:** 2-B1 @ 33

◆**Depth:** 25.5 **Sample Number:** 2-B2 @ 25.5

▼**Depth:** 2.5 **Sample Number:** 2-B3 @ 2.5

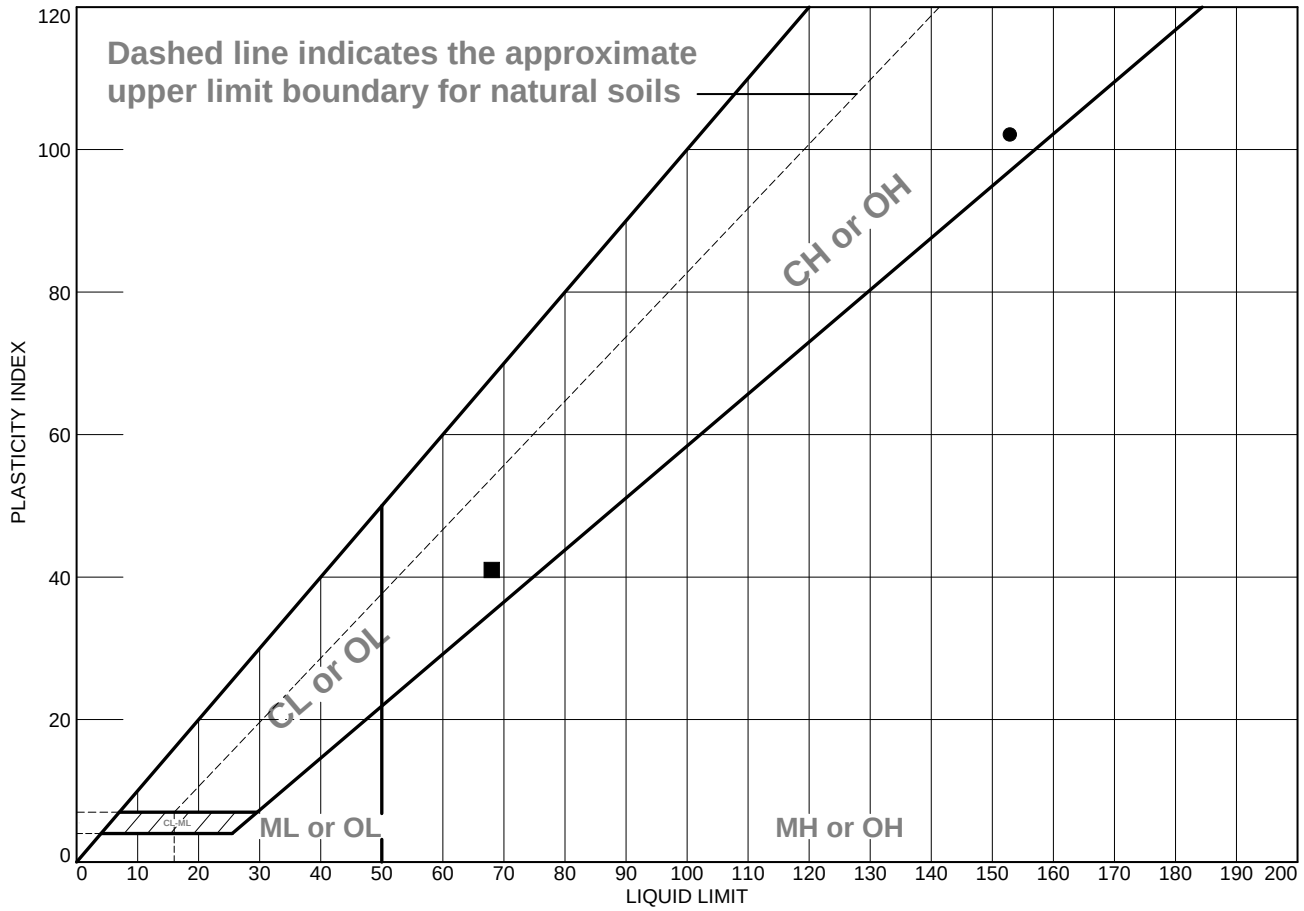
ENGEO INCORPORATED

Remarks:

Figure

Tested By: NS **Checked By:** KEL

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	153	51	102			
■	See exploration logs	68	27	41			

Project No. 11185.000.000 **Client:** iStar Financial Incorporated

Project: 1210 California Circle (iStar Financial)

●**Depth:** 10.5 feet **Sample Number:** 2-B1 @ 10.5

■**Depth:** 12.5 feet **Sample Number:** 2-B6 @ 12.5

Remarks:

●PI: ASTM D4318

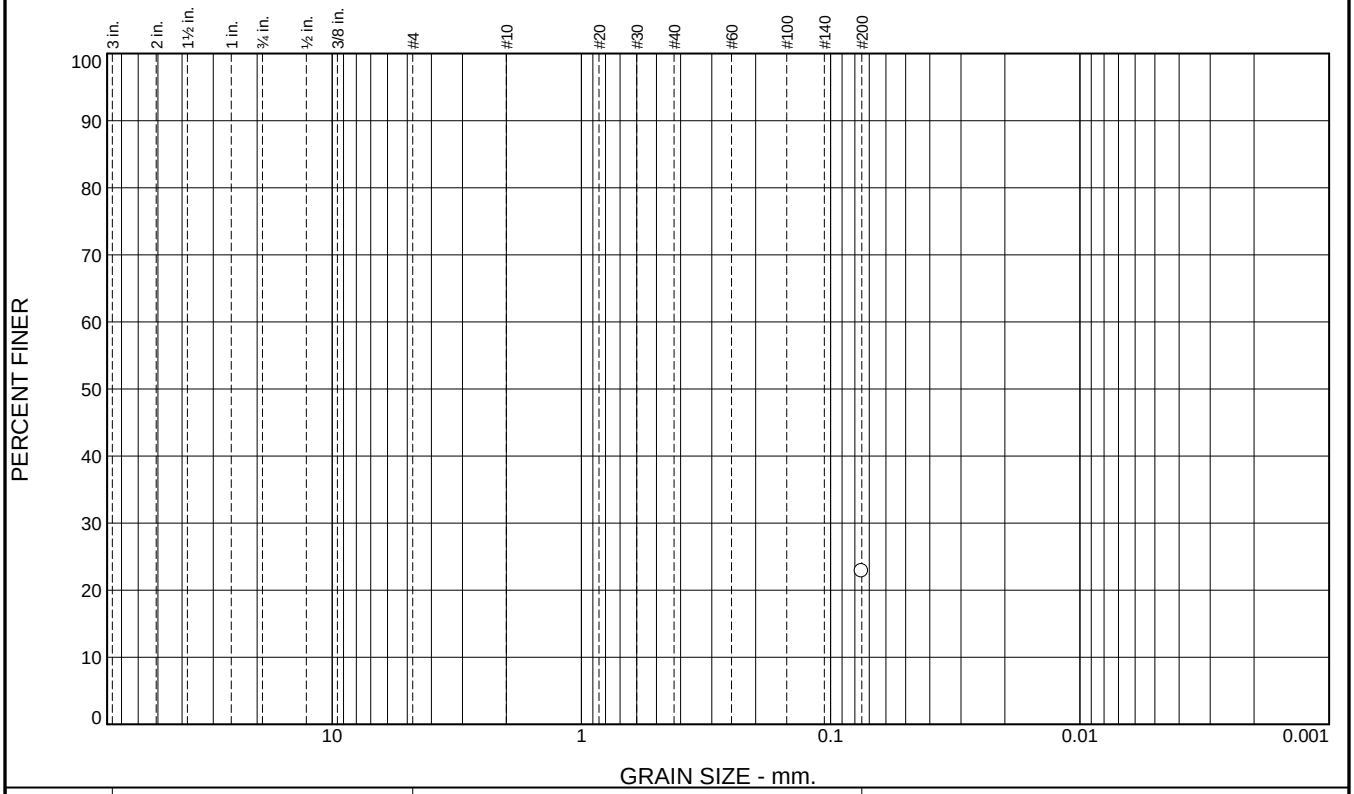
■PI: ASTM D4318

Grain Size: ASTM D1140

ENGEO INCORPORATED

Tested By: JL **Checked By:** TB

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						22.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	22.8		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B1 @ 5 **Depth:** 5

Date: 7-16-14

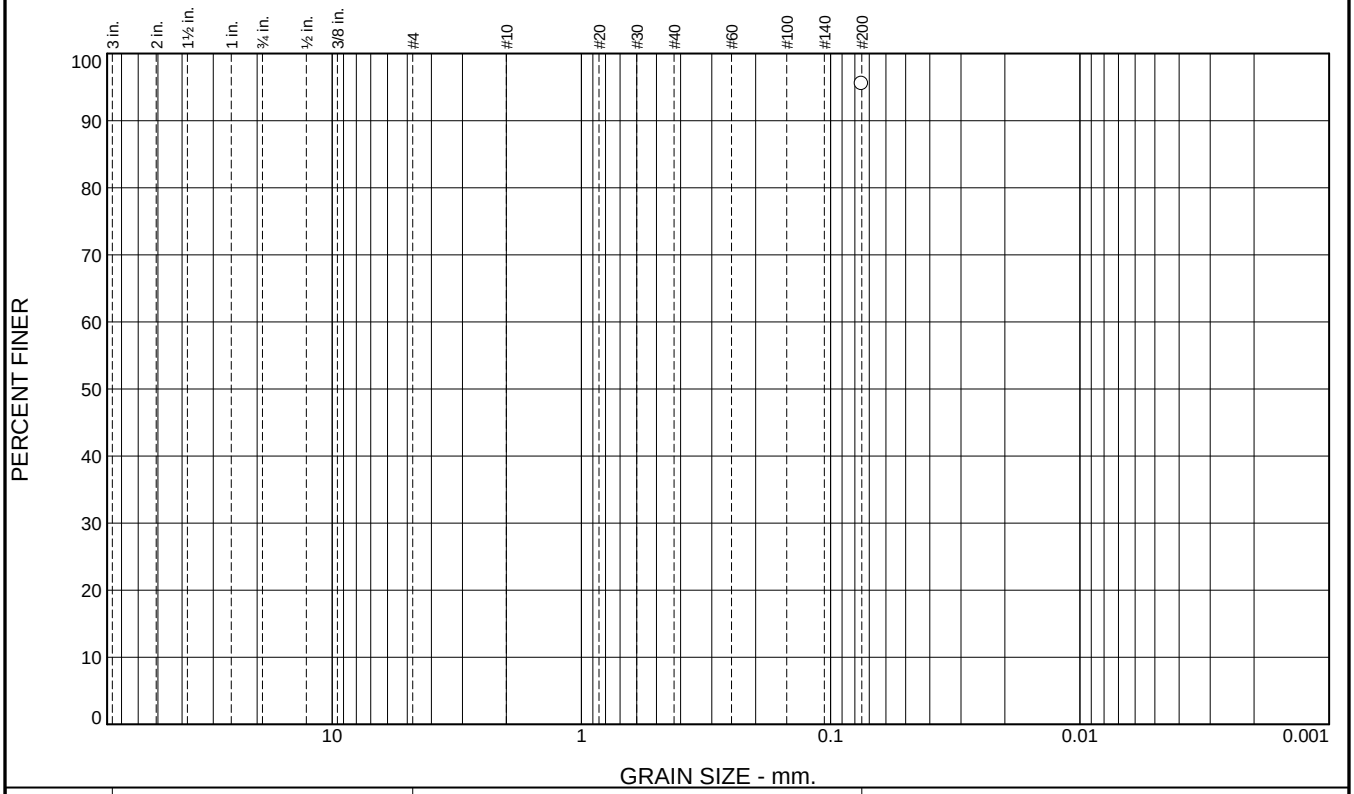
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						95.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	95.5		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= 20 LL= 39 PI= 19

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B1 @ 21 **Depth:** 21

Date: 7-16-14

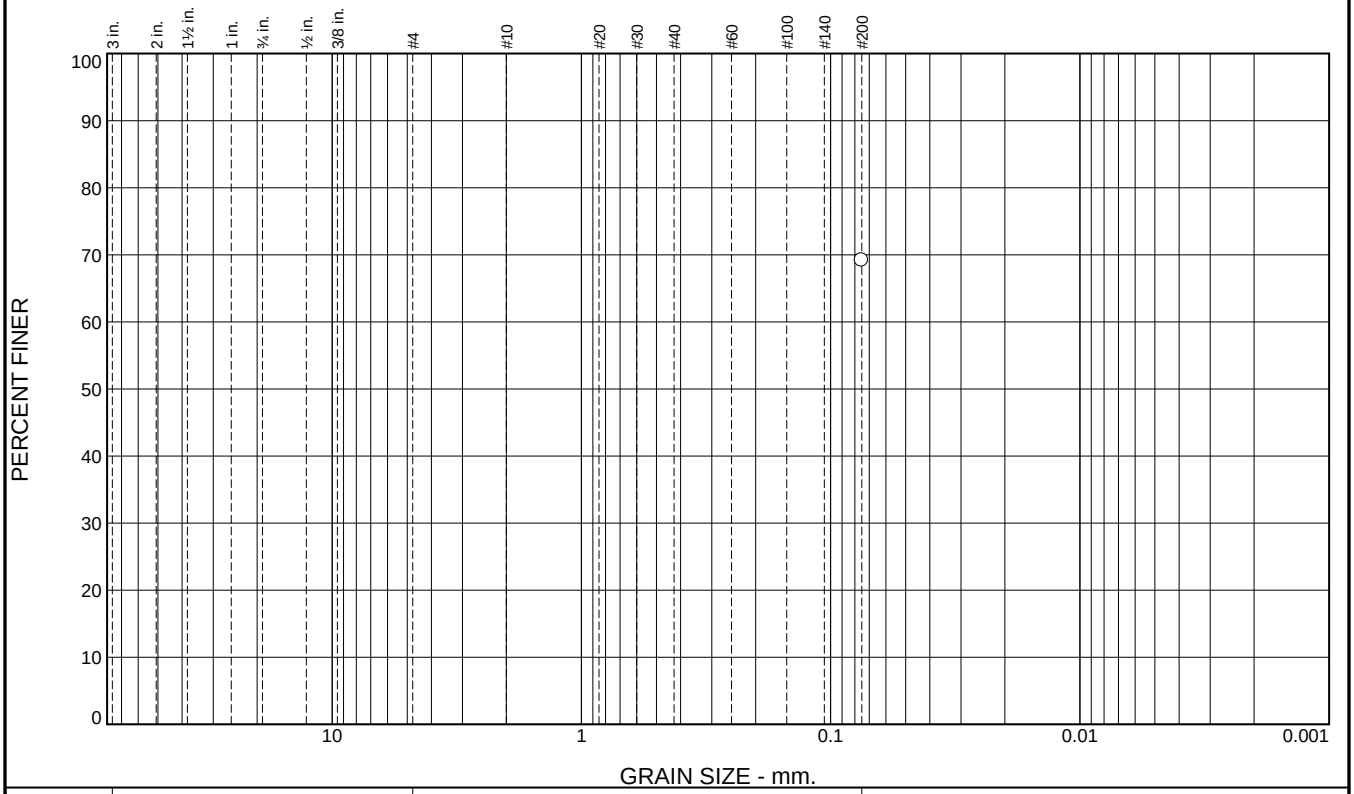
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						69.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	69.2		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= 20 LL= 27 PI= 7

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B1 @ 33 **Depth:** 33

Date: 7-16-14

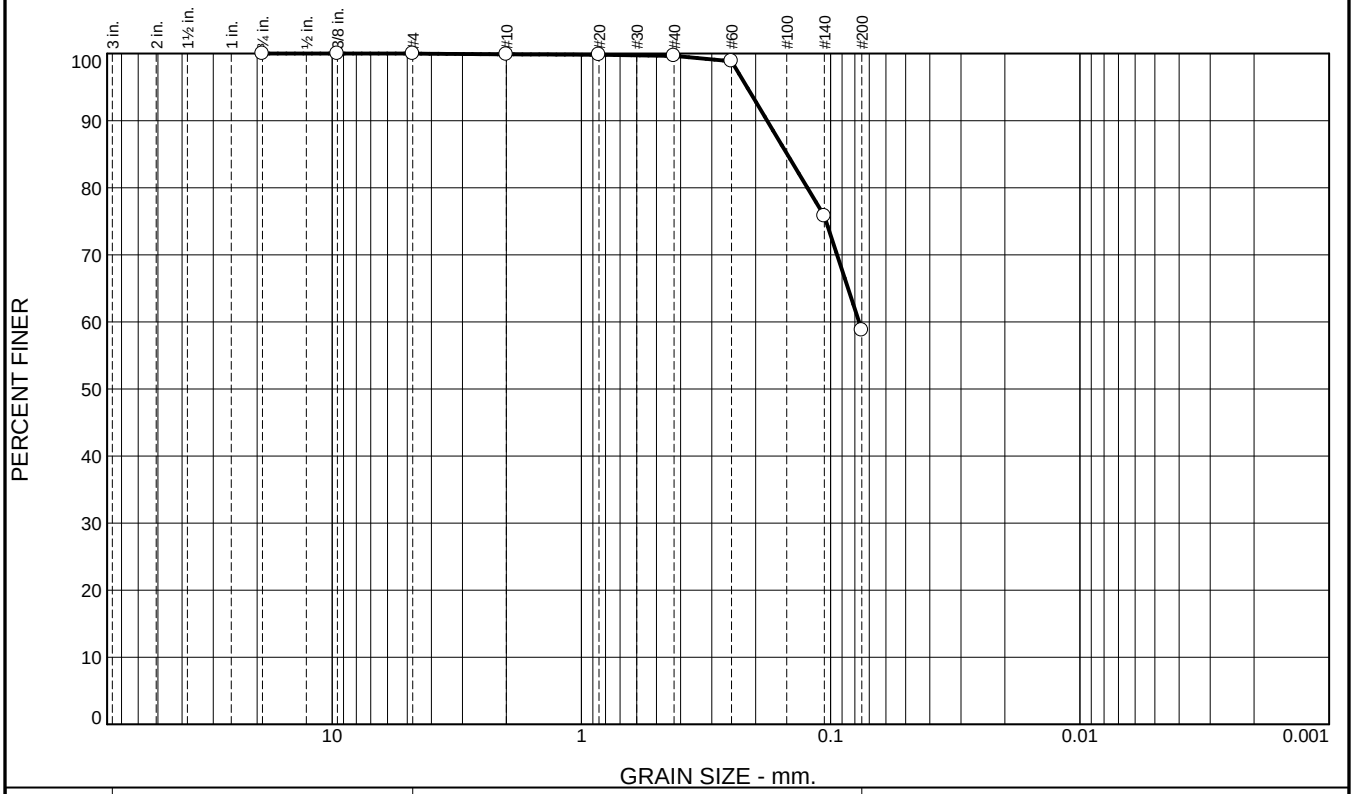
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.2	40.9	58.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	100.0		
#4	100.0		
#10	99.9		
#20	99.8		
#40	99.7		
#60	98.9		
#140	75.8		
#200	58.8		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1798 D_{85}= 0.1493 D_{60}= 0.0769
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B1 @ 41.5 **Depth:** 41.5

Date: 7-16-14

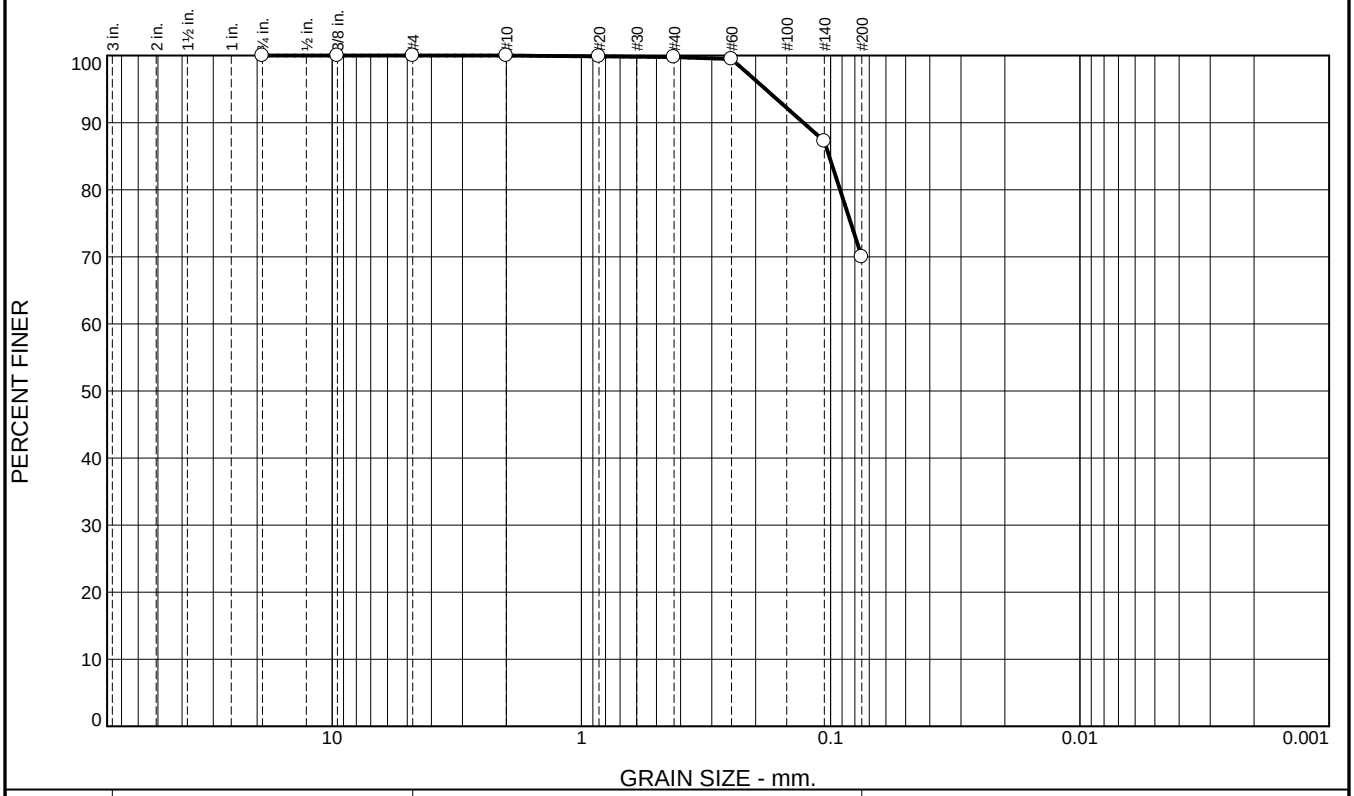
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.2	29.8	70.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	100.0		
#4	100.0		
#10	100.0		
#20	99.9		
#40	99.8		
#60	99.5		
#140	87.2		
#200	70.0		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= 21 LL= 25 PI= 4

Coefficients

D_{90}= 0.1286 D_{85}= 0.1013 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL-ML AASHTO= A-4(1)

Remarks

Sample Number: 2-B2 @ 25.5 **Depth:** 25.5

Date: 7-16-14

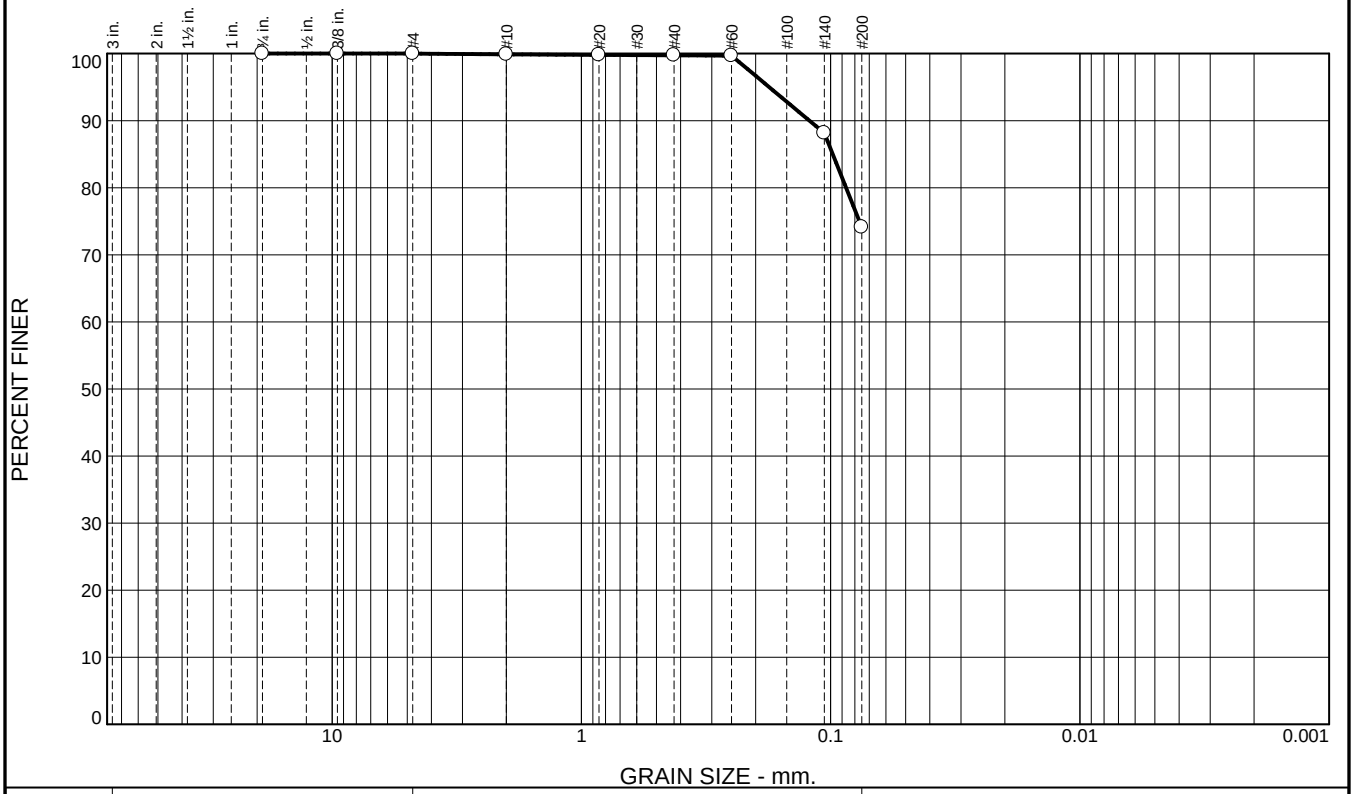
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.2	25.6	74.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	100.0		
#4	100.0		
#10	99.9		
#20	99.8		
#40	99.7		
#60	99.7		
#140	88.2		
#200	74.1		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.1216 D_{85}= 0.0981 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B2 @ 15.5 **Depth:** 15.5

Date: 7-16-14

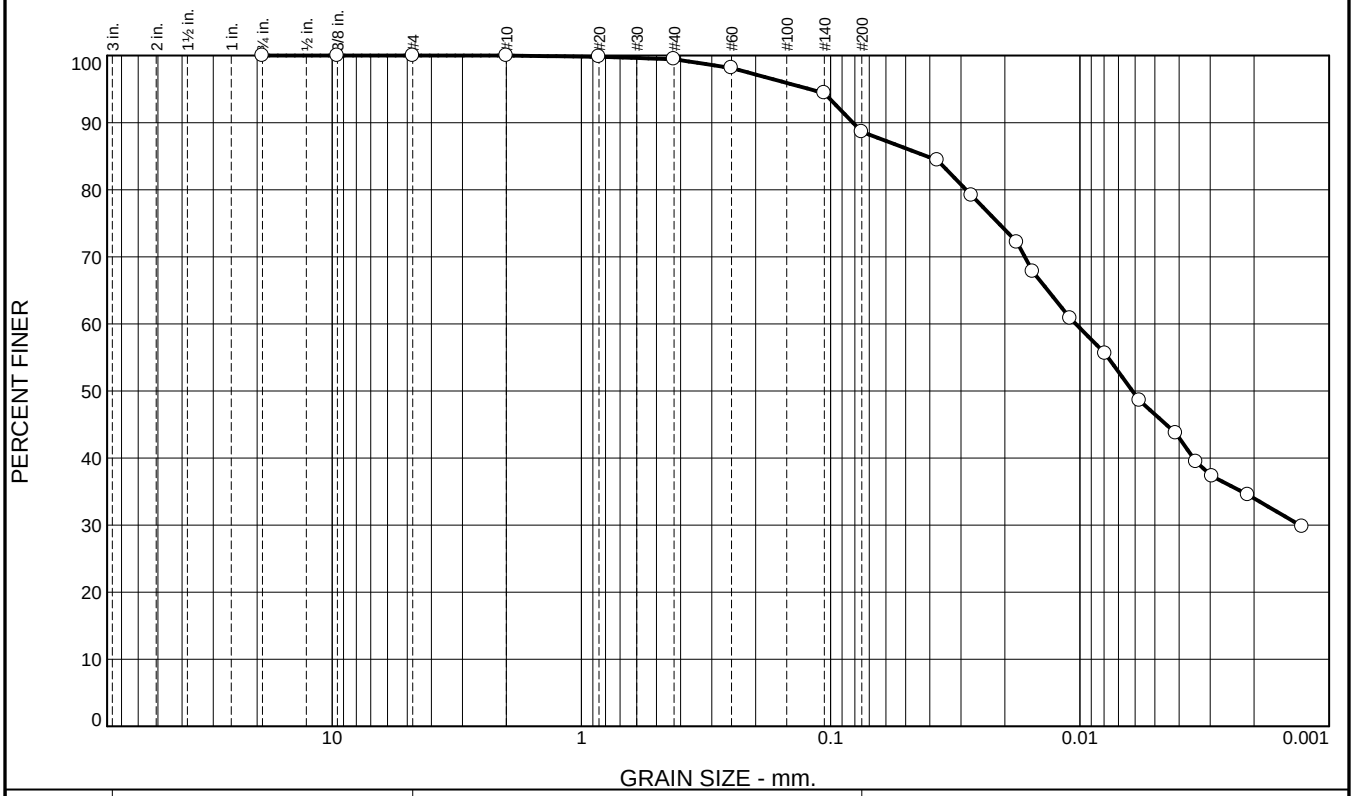
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.5	10.9	42.1	46.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	100.0		
#4	100.0		
#10	100.0		
#20	99.8		
#40	99.5		
#60	98.2		
#140	94.4		
#200	88.6		
0.0373 mm.	84.4		
0.0272 mm.	79.2		
0.0179 mm.	72.2		
0.0155 mm.	67.8		
0.0109 mm.	60.8		
0.0079 mm.	55.6		
0.0058 mm.	48.6		
0.0041 mm.	43.7		
0.0034 mm.	39.4		
0.0030 mm.	37.3		
0.0021 mm.	34.5		
0.0013 mm.	29.8		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= 22 LL= 49 PI= 27

Coefficients

D_{90}= 0.0814 D_{85}= 0.0412 D_{60}= 0.0104
D_{50}= 0.0062 D_{30}= 0.0013 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= CL AASHTO= A-7-6(26)

Remarks

Sample Number: 2-B3 @ 2.5 **Depth:** 2.5

Date: 7-16-14

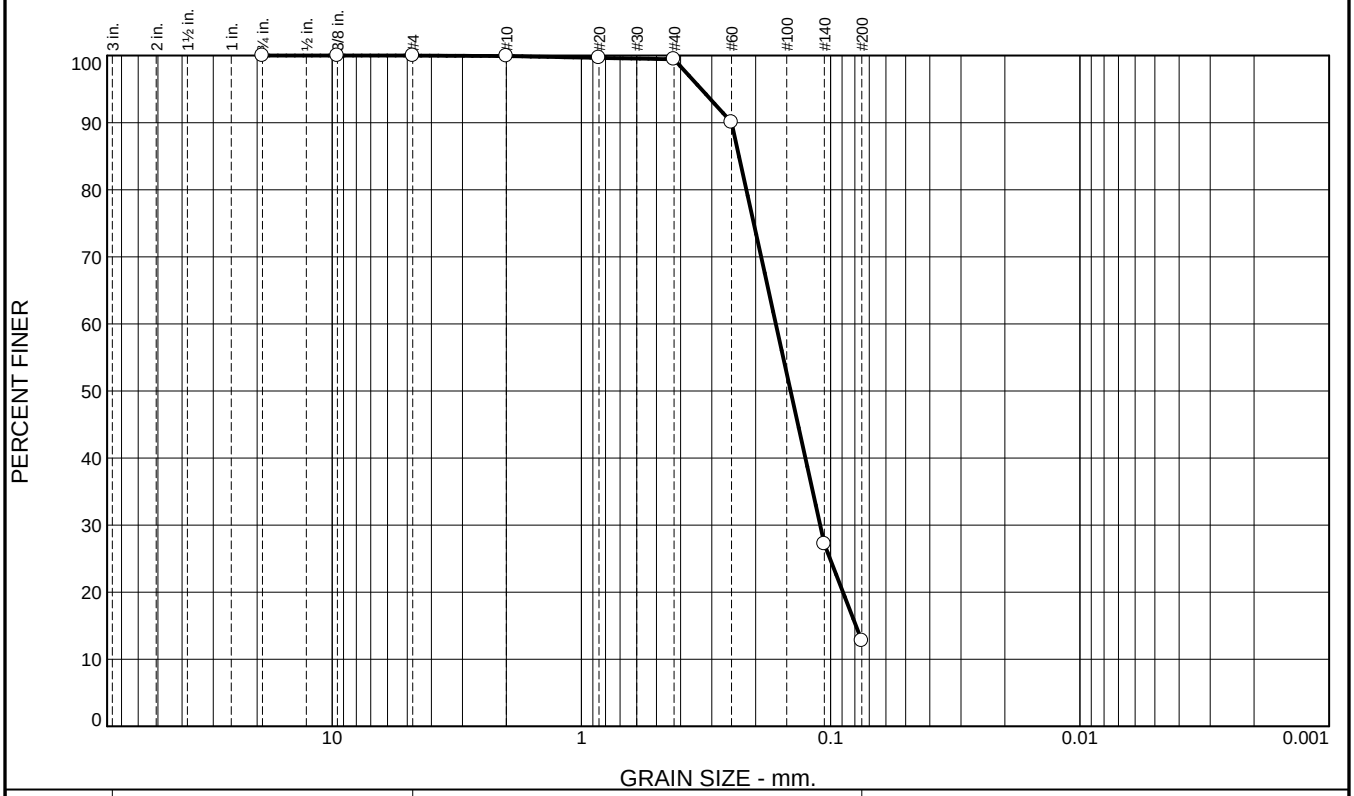
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.4	86.8	12.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	100.0		
#4	100.0		
#10	99.9		
#20	99.7		
#40	99.5		
#60	90.0		
#140	27.2		
#200	12.7		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2498 D_{85}= 0.2334 D_{60}= 0.1659
D_{50}= 0.1447 D_{30}= 0.1101 D_{15}= 0.0792
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B3 @ 27.5 **Depth:** 27.5

Date: 7-16-14

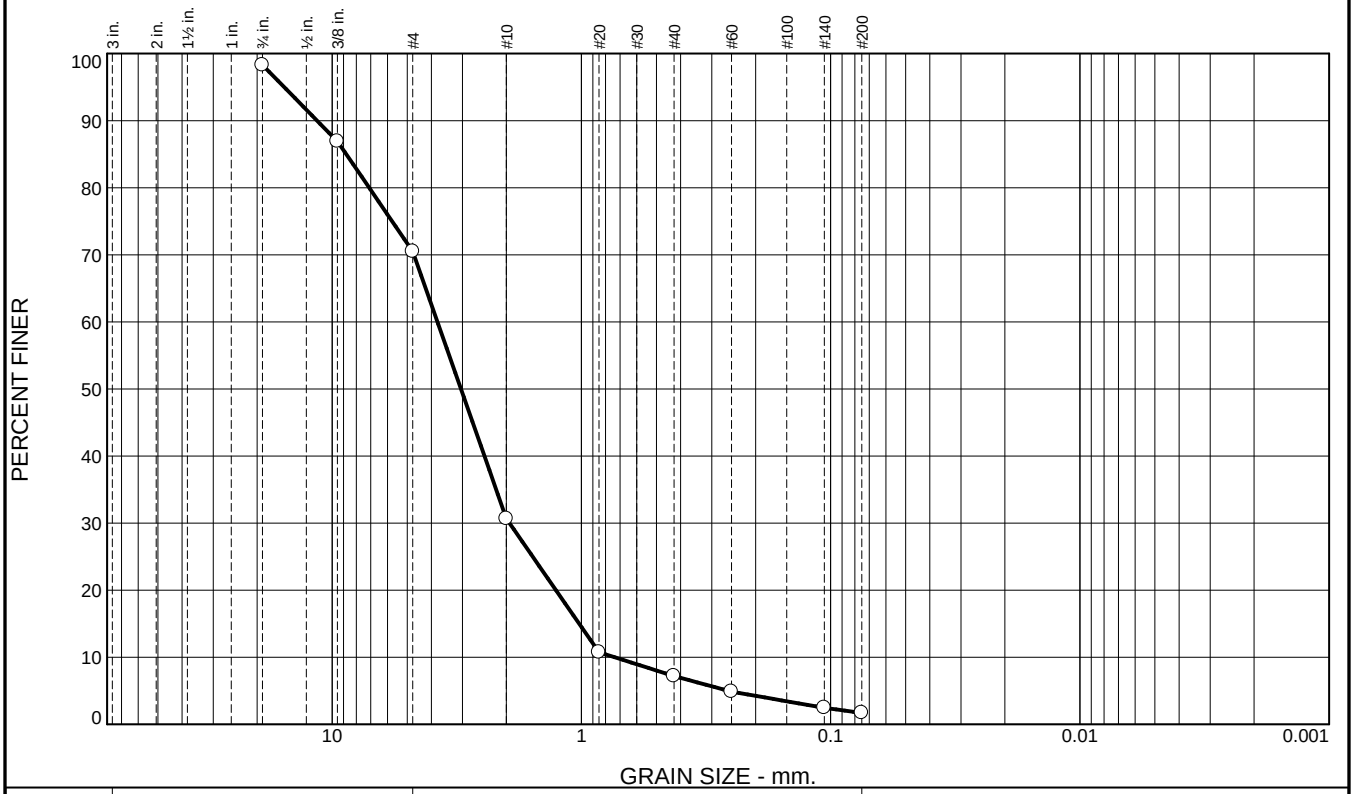
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
		27.8	39.8	23.5	5.5	1.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	98.3		
3/8"	86.9		
#4	70.5		
#10	30.7		
#20	10.7		
#40	7.2		
#60	4.9		
#140	2.5		
#200	1.7		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 11.4935 D_{85}= 8.7815 D_{60}= 3.7815
D_{50}= 3.0436 D_{30}= 1.9443 D_{15}= 1.0206
D_{10}= 0.7353 C_u= 5.14 C_c= 1.36

Classification

USCS= SP AASHTO=

Remarks

Sample Number: 2-B3 @ 32 **Depth:** 32

Date: 7-16-14

ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report

% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	0.1	0.9	70.5	28.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	100.0		
#4	100.0		
#10	100.0		
#20	100.0		
#40	99.9		
#60	99.9		
#140	99.7		
#200	99.0		
0.0371 mm.	83.8		
0.0281 mm.	71.7		
0.0192 mm.	56.1		
0.0166 mm.	50.9		
0.0118 mm.	42.2		
0.0085 mm.	35.3		
0.0061 mm.	31.0		
0.0044 mm.	26.9		
0.0036 mm.	25.3		
0.0031 mm.	23.2		
0.0022 mm.	20.5		
0.0013 mm.	17.5		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.0494 D_{85}= 0.0391 D_{60}= 0.0211
D_{50}= 0.0160 D_{30}= 0.0057 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B4 @ 1 **Depth:** 1

Date: 7-16-14

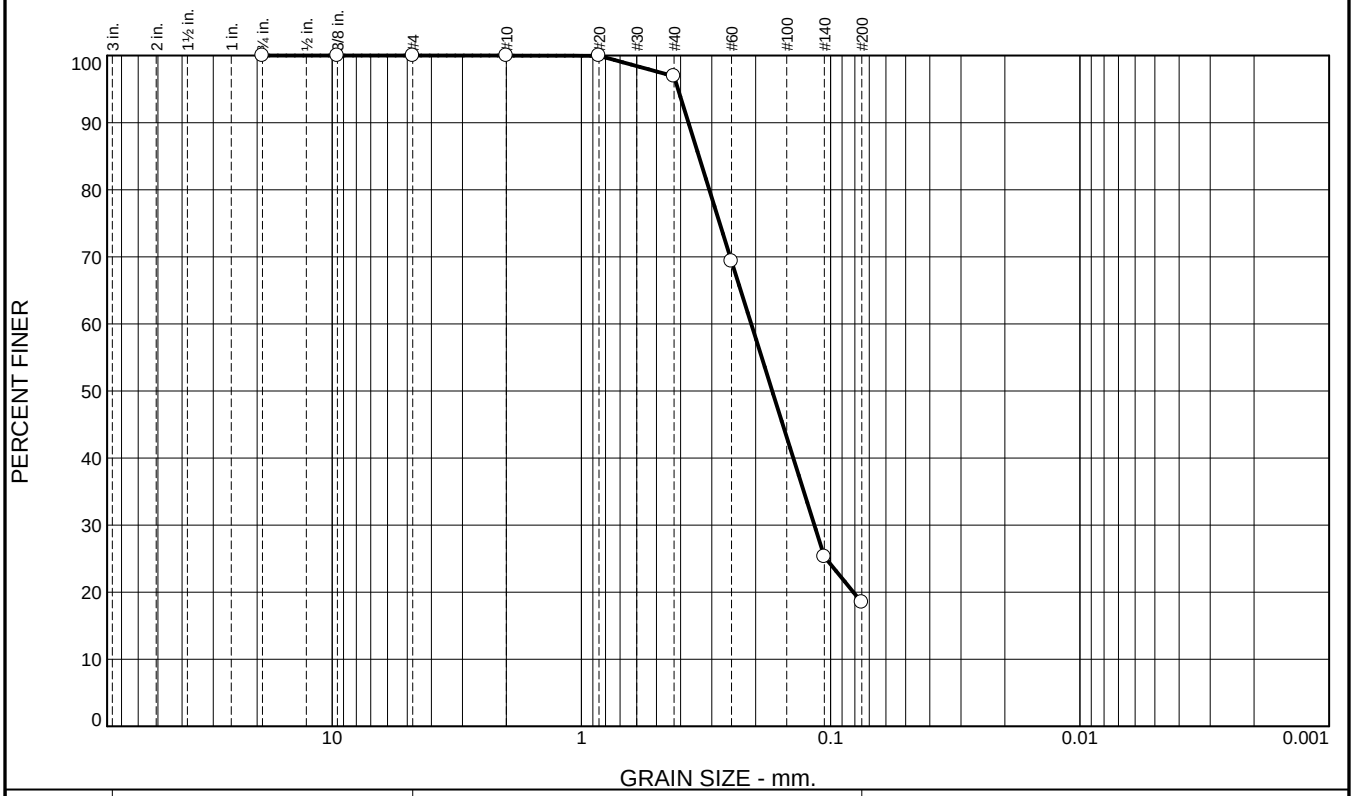
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	3.1	78.4	18.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	100.0		
#4	100.0		
#10	100.0		
#20	100.0		
#40	96.9		
#60	69.3		
#140	25.3		
#200	18.5		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.3721 D_{85}= 0.3379 D_{60}= 0.2084
D_{50}= 0.1715 D_{30}= 0.1162 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B5 @ 15 **Depth:** 15

Date: 7-16-14

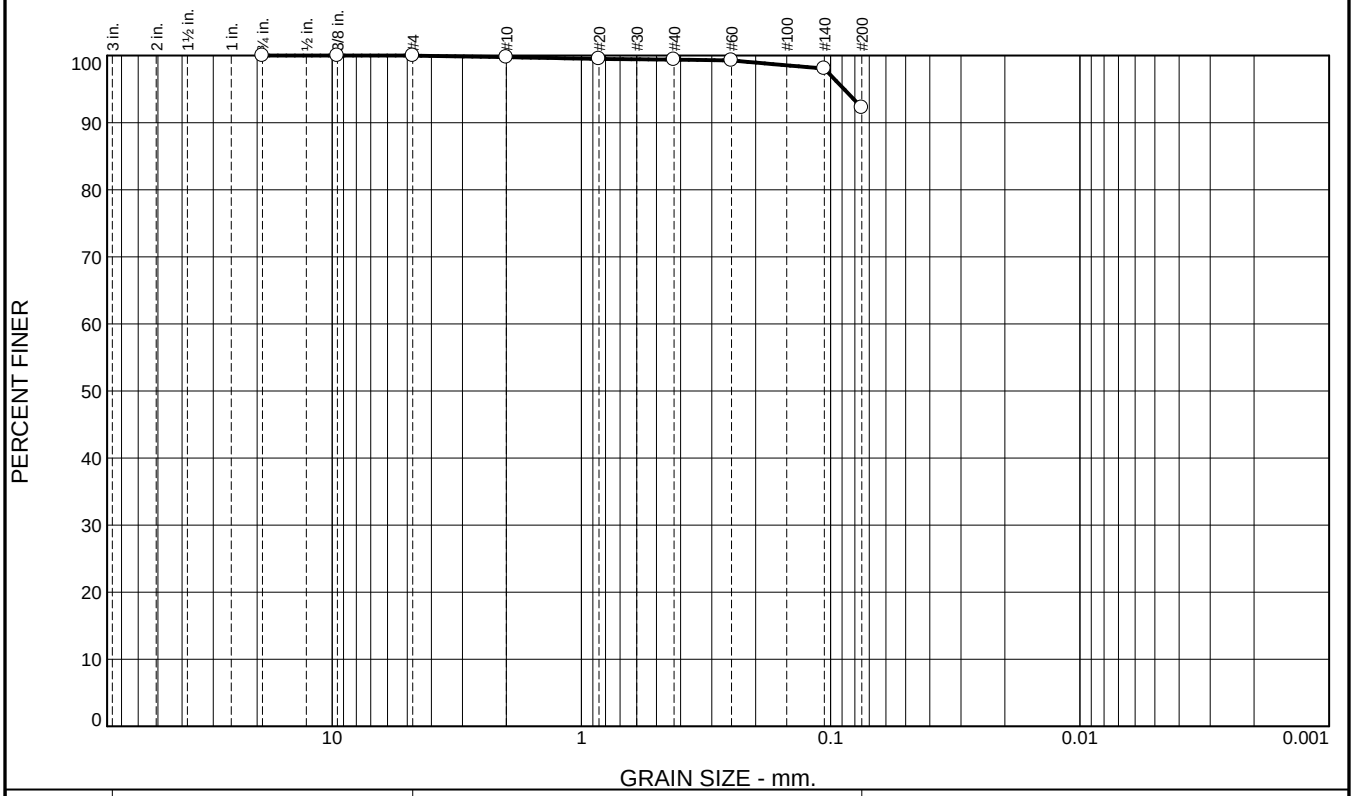
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	0.4	7.2	92.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	100.0		
#4	100.0		
#10	99.8		
#20	99.5		
#40	99.4		
#60	99.3		
#140	98.1		
#200	92.2		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B5 @ 5 **Depth:** 5

Date: 7-16-14

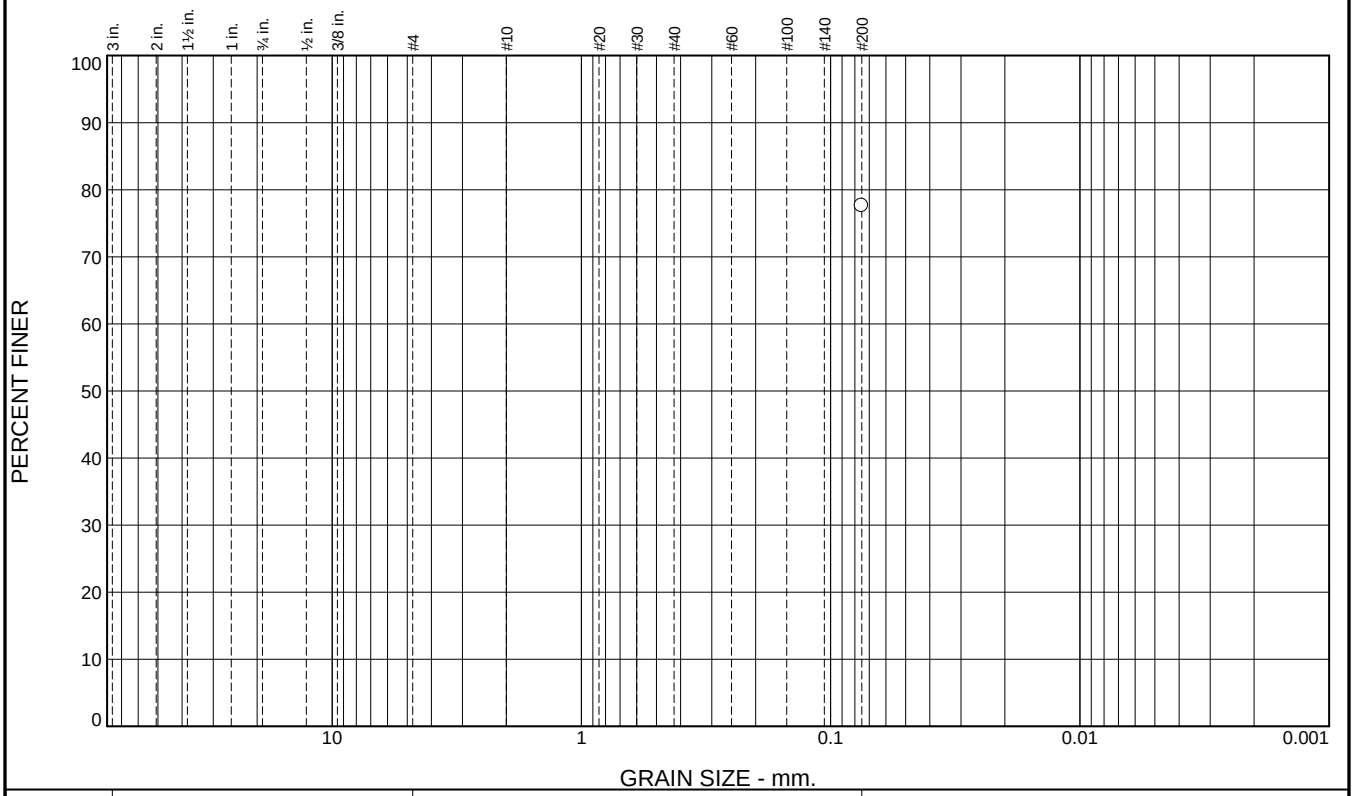
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						77.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	77.6		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B6 @ 5.5 **Depth:** 5.5

Date: 7-16-14

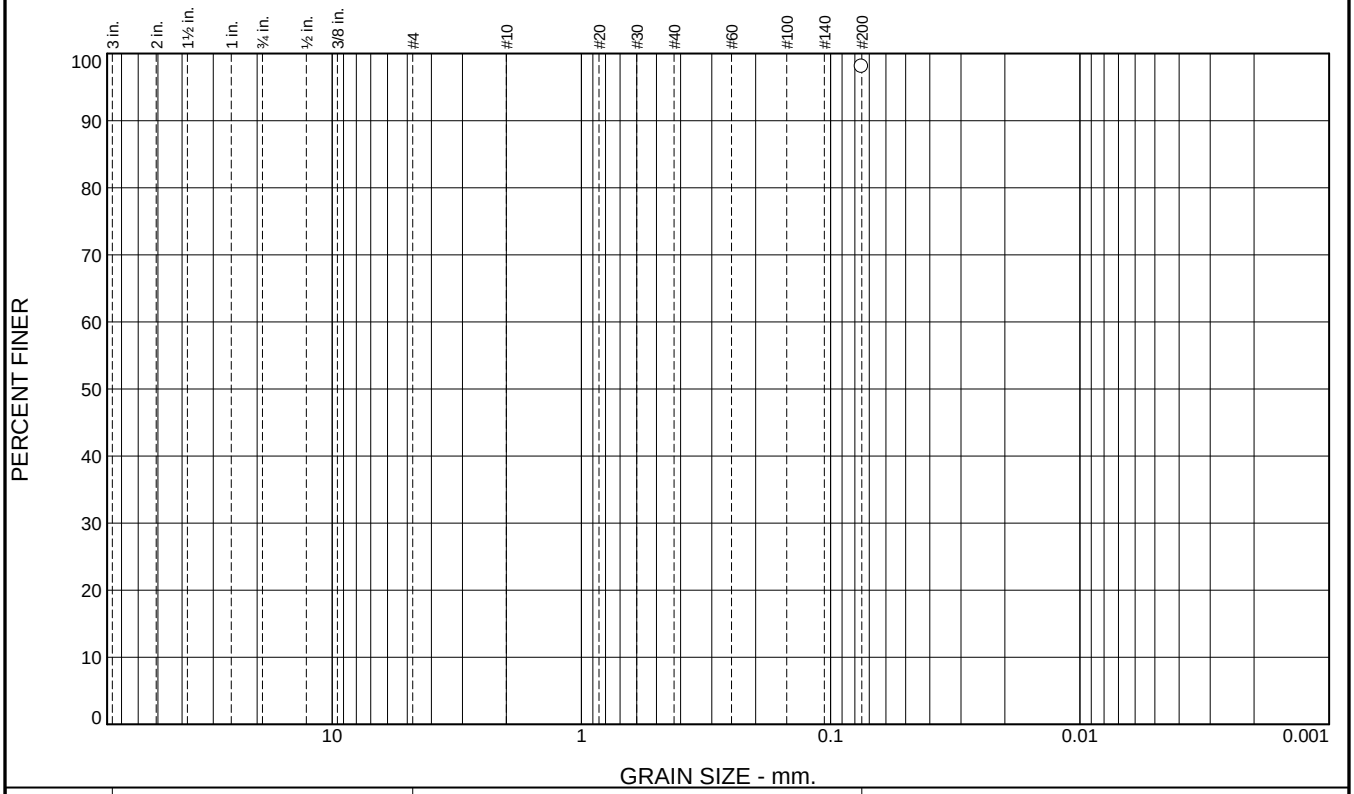
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						98.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	98.1		

* (no specification provided)

Soil Description

See Exploration logs

Atterberg Limits

PL= 27 LL= 68 PI= 41

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D422, PI: ASTM D4318

Sample Number: 2-B6 @ 12.5 feet **Depth:** 12.5 feet

Date: 07/15/14

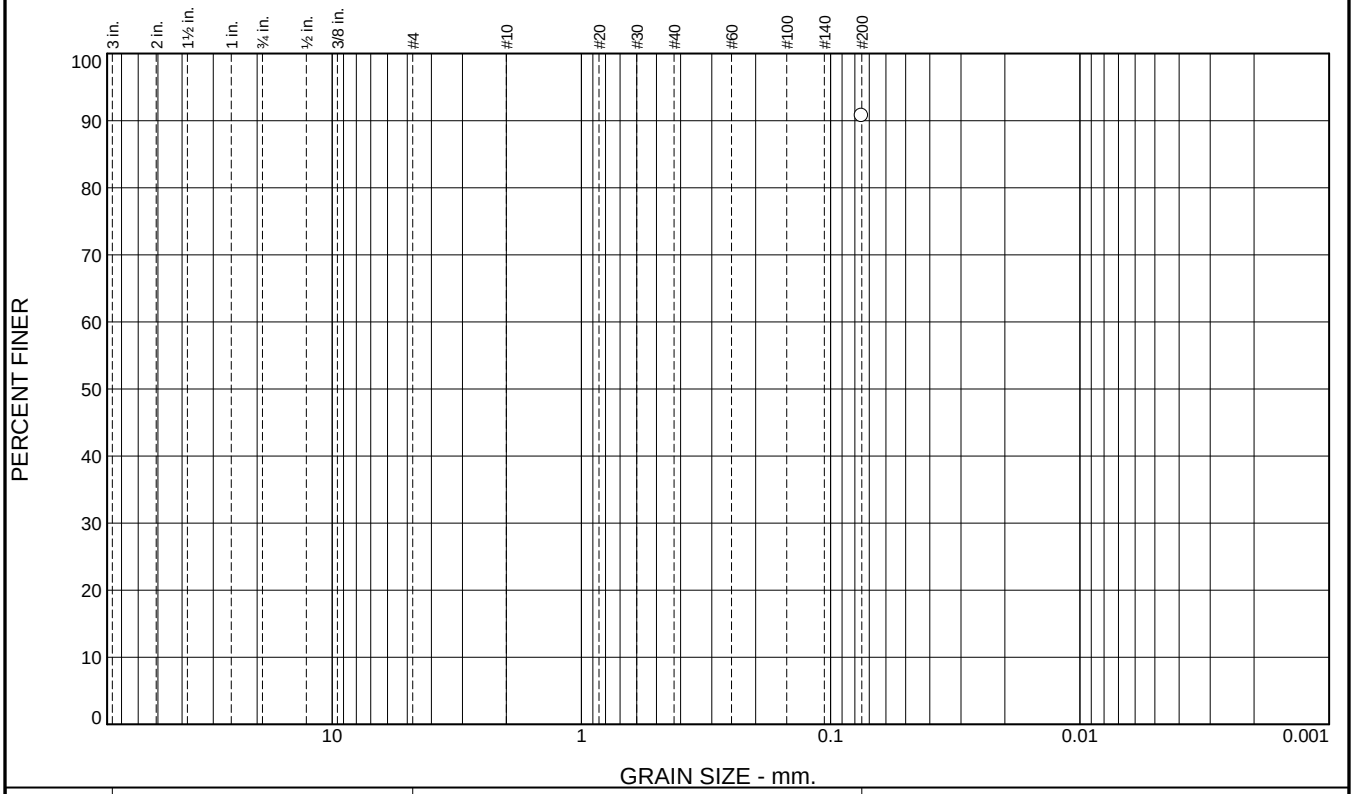
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002)

Tested By: JAL **Checked By:** GC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						90.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	90.7		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= 18 LL= 35 PI= 17

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B6 @ 40.5 **Depth:** 40.5

Date: 7-16-14

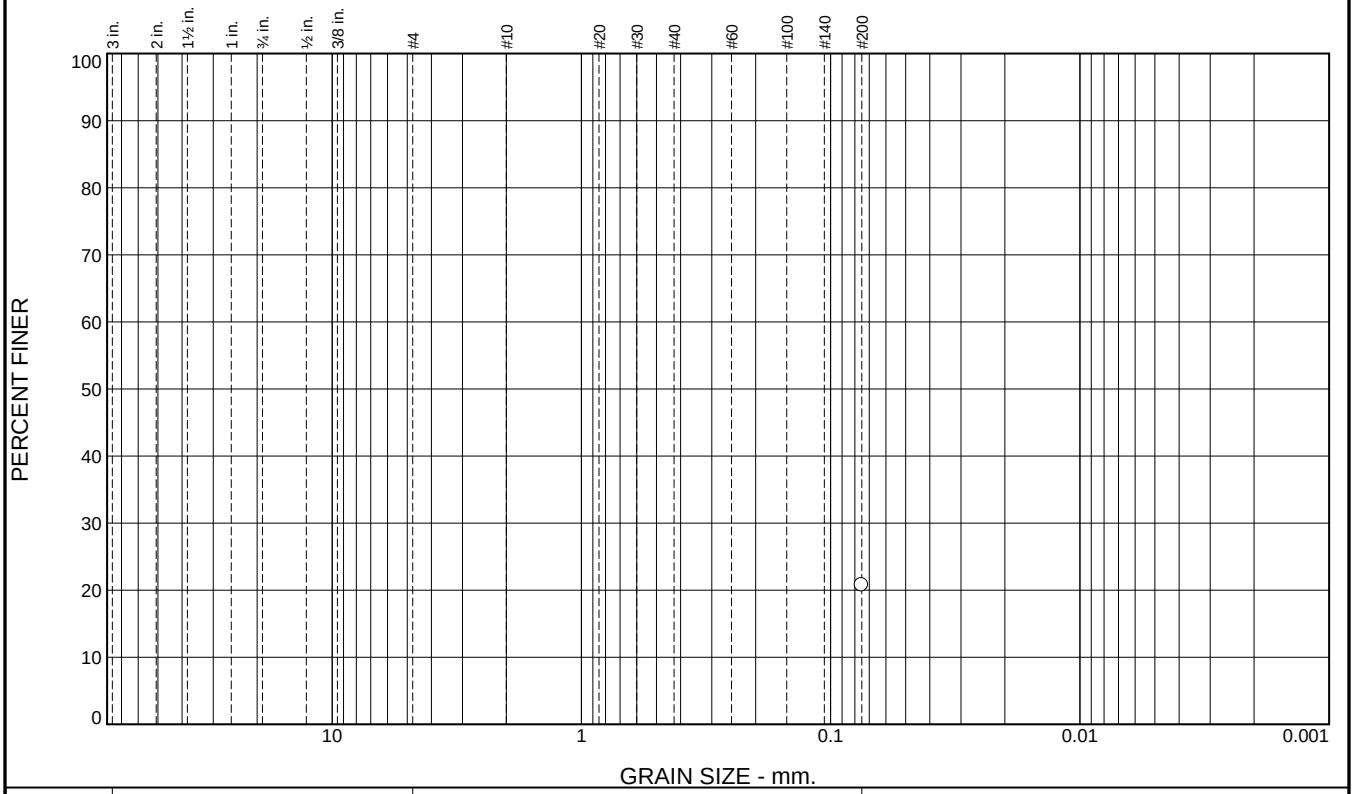
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						20.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	20.7		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= 14 LL= 26 PI= 12

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B6 @ 38 **Depth:** 38

Date: 7-16-14

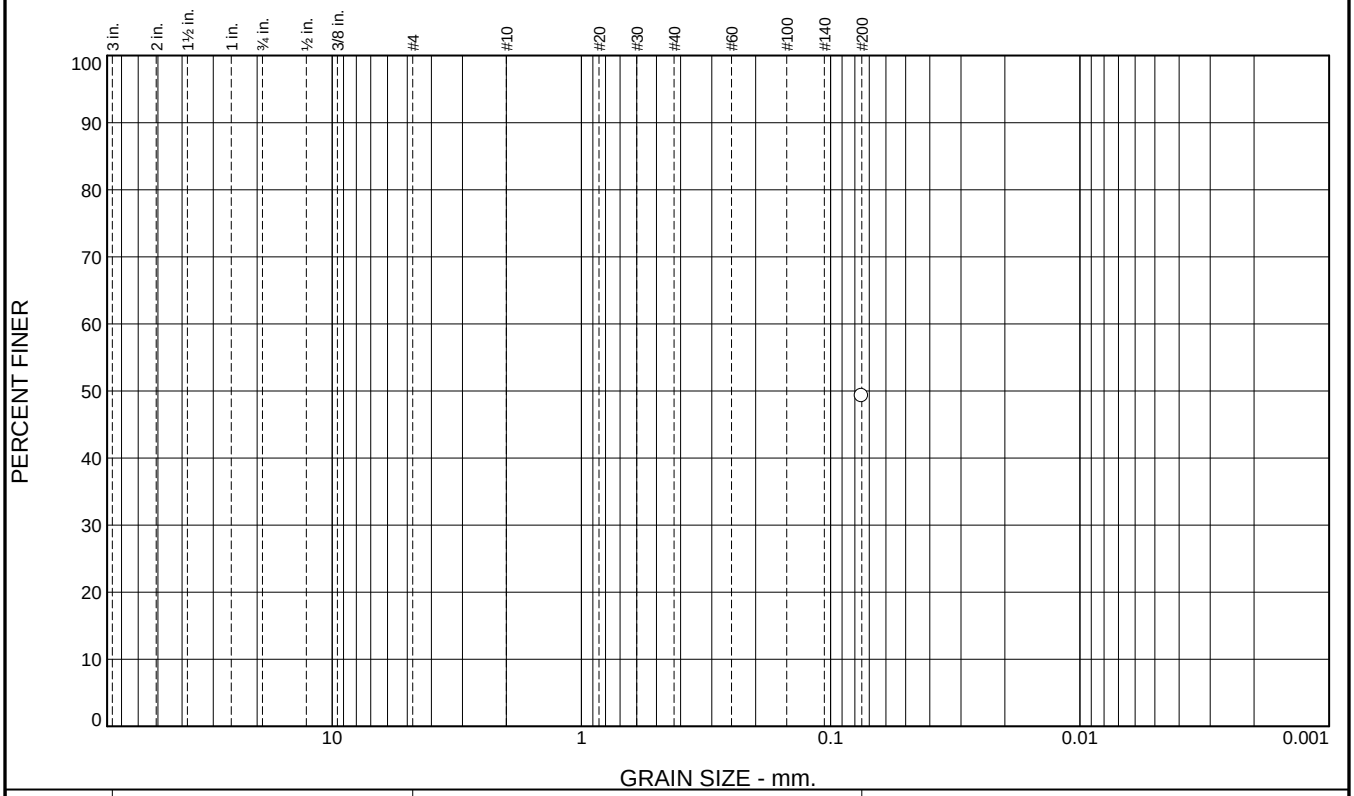
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						49.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	49.3		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= 16 LL= 27 PI= 11

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B6 @ 34 **Depth:** 34

Date: 7-16-14

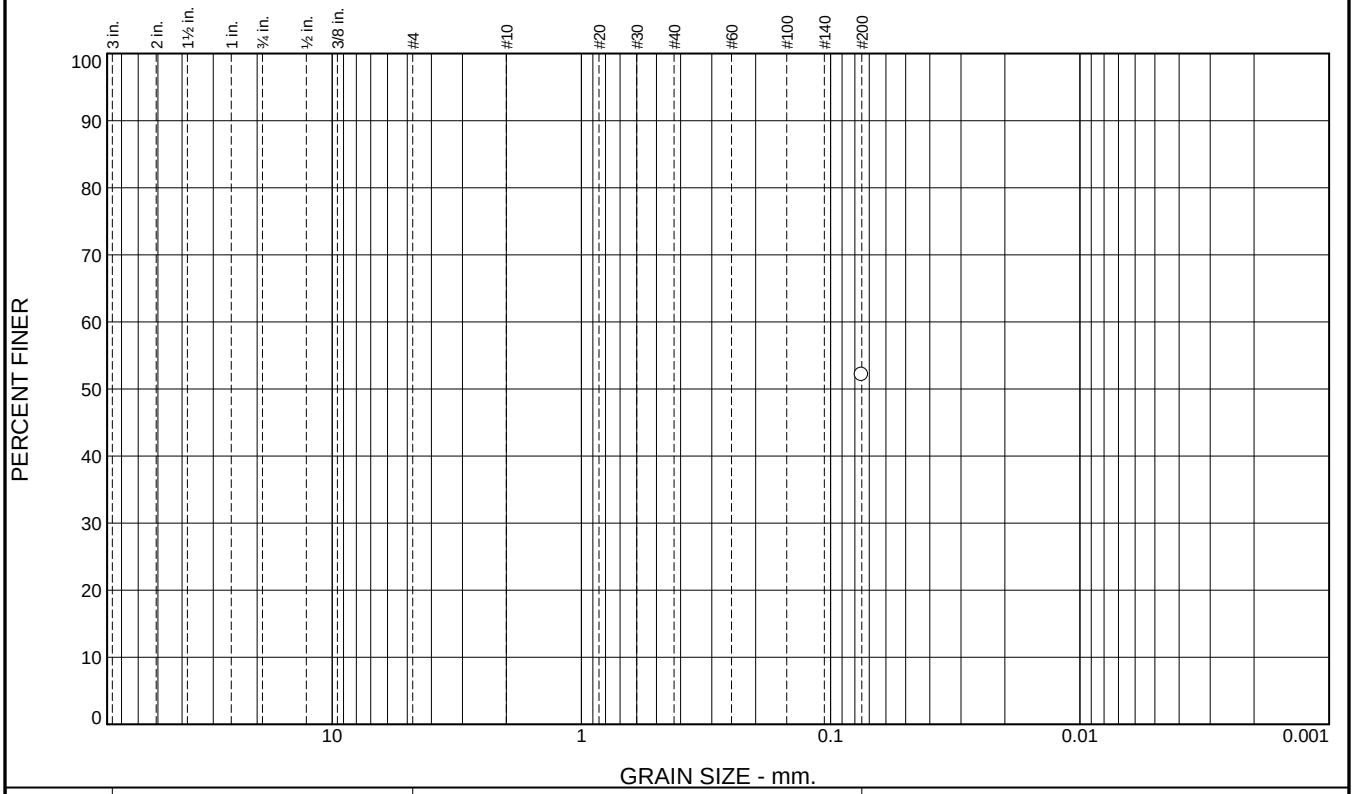
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						52.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	52.1		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= 18 LL= 26 PI= 8

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B6 @ 32.5 **Depth:** 32.5

Date: 7-16-14

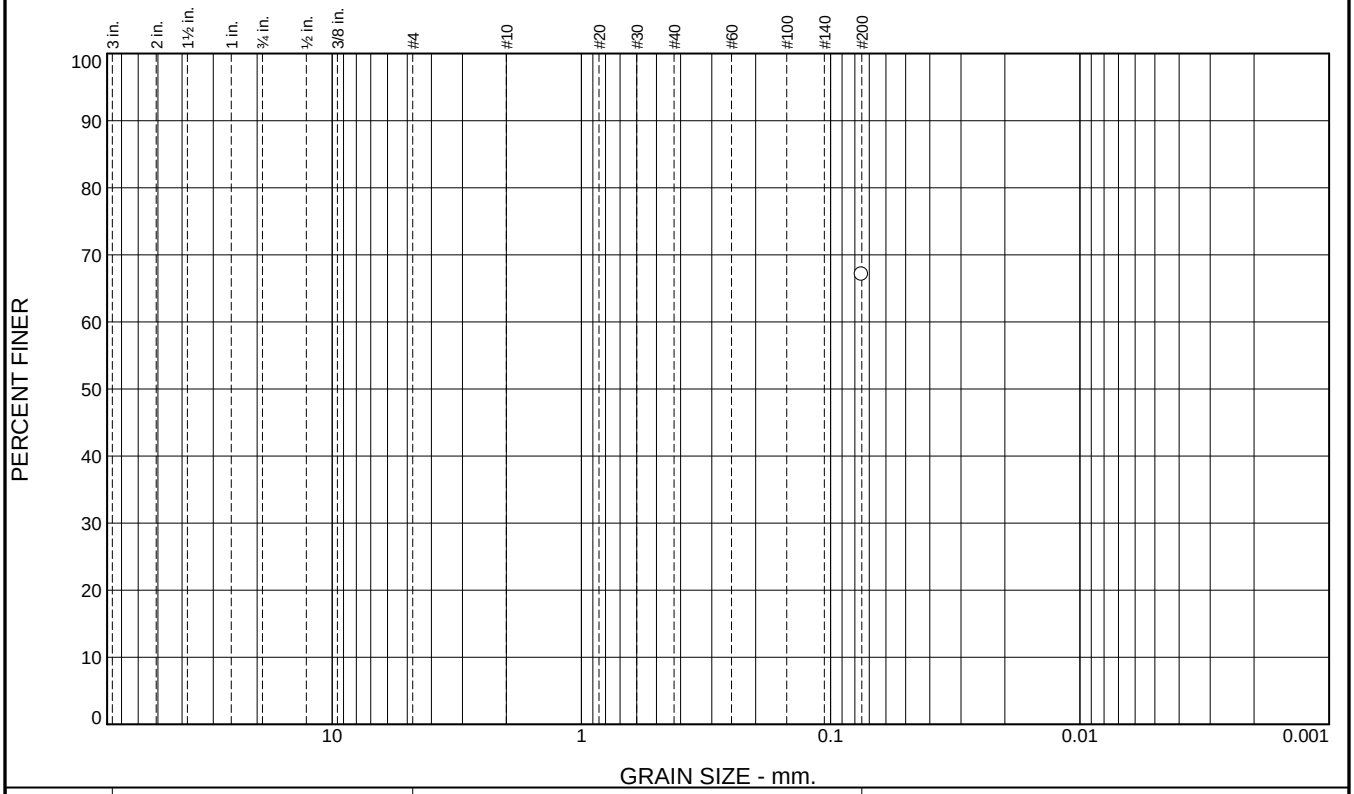
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						67.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	67.1		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= 17 LL= 29 PI= 12

Coefficients

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

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B6 @ 21 **Depth:** 21

Date: 7-16-14

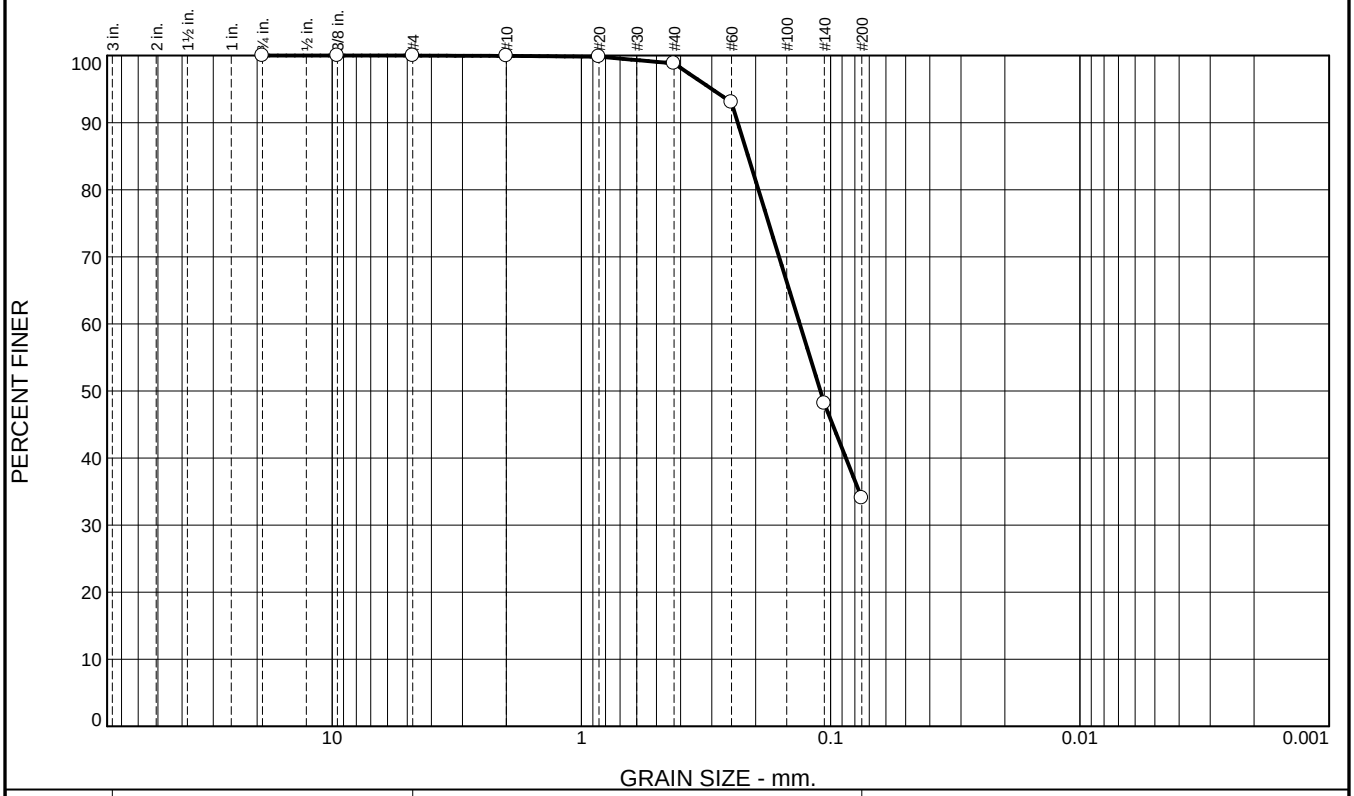
ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.2	64.8	34.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	100.0		
#4	100.0		
#10	100.0		
#20	99.8		
#40	98.8		
#60	93.0		
#140	48.1		
#200	34.0		

* (no specification provided)

Soil Description

See Exploration Logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.2360 D_{85}= 0.2145 D_{60}= 0.1330
D_{50}= 0.1099 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

Sample Number: 2-B7 @ 21 **Depth:** 21

Date: 7-16-14

ENGEO INCORPORATED

Client: iStar Financial
Project: 1210 California Circle, Milpitas, California

Project No: 11185.000.000 (002) **Figure**

Tested By: NS **Checked By:** KEL

ENGEO

Unconfined Compression Test Report (ASTM D2166)

Date 7/16/14

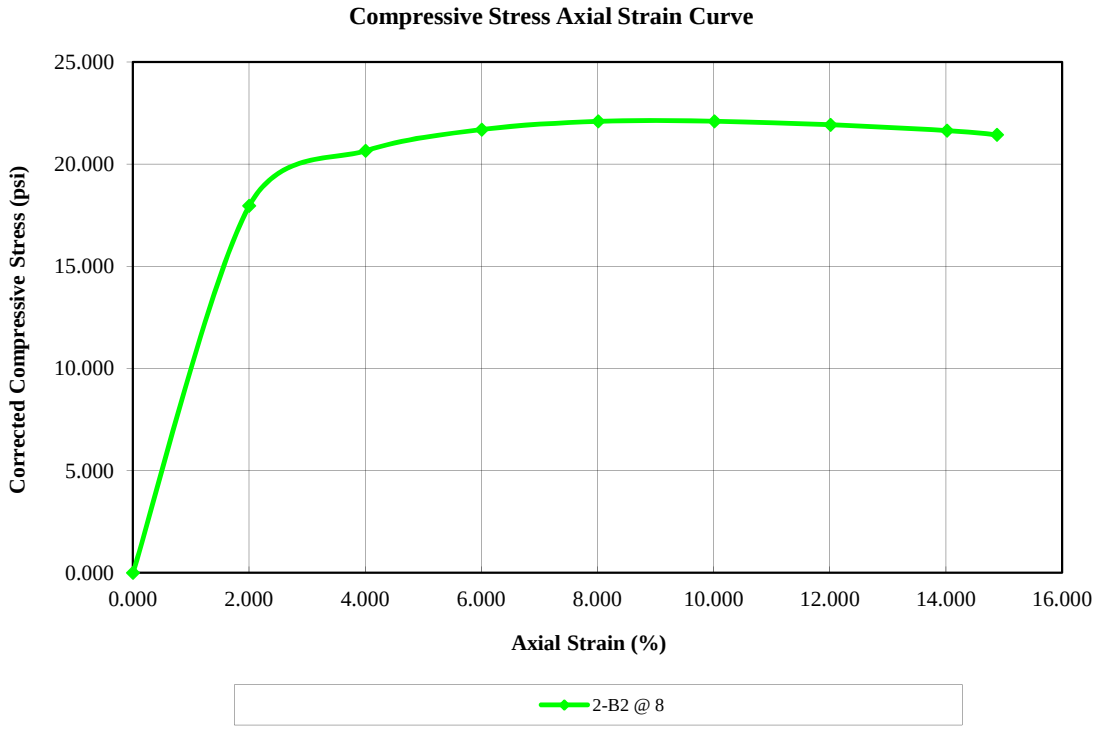
Checked By RWS

Date 7/16/14

Computed By KEL

Date 7/15/14

Tested By KEL



Before Test	Specimen			
	2-B2 @ 8			
Water Content (%)	32.54			
Dry Density (pcf)	90.100			
Saturation (%)	103.03			
Void Ratio	0.84			
Diameter (in)	2.403			
Height (in)	5.000			
Test Data	**2-B2 @ 8**			
Unconfined Strength (psi)	22.103			
Undrained Shear Strength (tsf)	0.796			
Undrained Shear Strength (psi)	11.051			
Rate of Strain (in/min)	0.050000			
Strain at Failure (%)	10.01			
Description	See Exploratory Boring Log			

Project Information		Specimen Description	
Project Num	11185.000.000 (002)	2-B2 @ 8	See Exploratory Boring Log
Project	210 California Circle (iStar Financia		
Sampling Date	7-8-14		
Sample #	2-B2 @ 8		
Client	iStar Financial Incorporated	**Test Variables**	
		Specific Gravity	2.65
		Liquid Limit:	
		Plastic Limit:	

Remarks

LABORATORY MINIATURE VANE SHEAR

ASTM D4648

APPARATUS USED: Wykeham Farrance, Model 27-WF1730/4

Sample #	Sample ID	Remold? (Y/N)	Test depth (ft)	Spring number	Shear strength (psf)
1	2-B1 @ 10.5	N	11.0-10.75	2	443
2	2-B6 @ 13.5	N	13.5-13.25	1	488

PROJECT NAME: **1210 California Circle (iStar Financial)** **DATE:** **07/11/14**

PROJECT NUMBER: **11185.000.000**

CLIENT: **iStar Financial Incorporated**

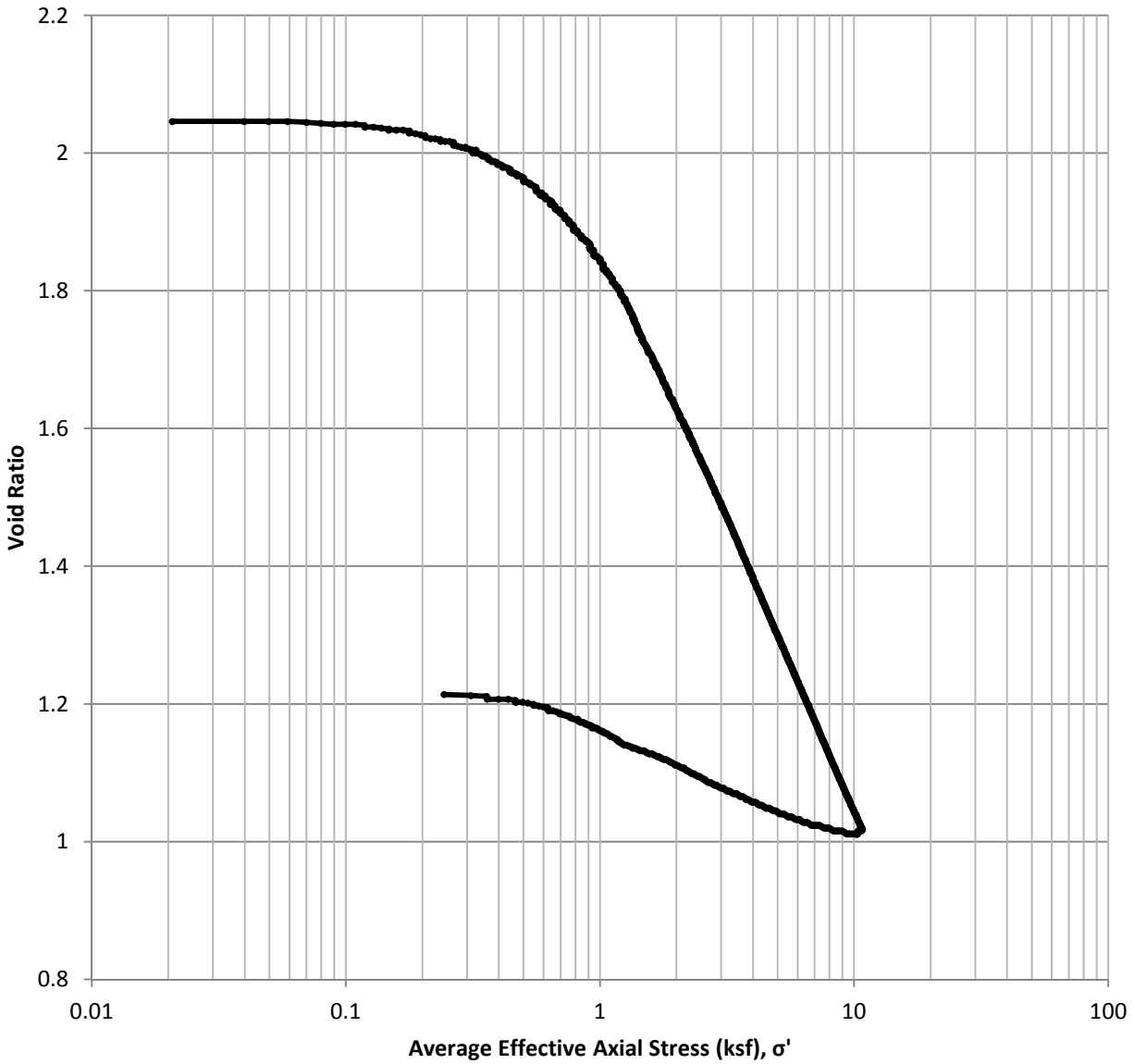
PHASE NUMBER: **002**

Tested by: JAL

Reviewed by: GC

ENGEO Incorporated 2010 Crow Canyon Place Suite 250 San Ramon, CA 94583
Laboratory address: 3420-E Fostoria Way, San Ramon, CA 94583

Constant Rate of Strain Consolidation
ASTM D4186



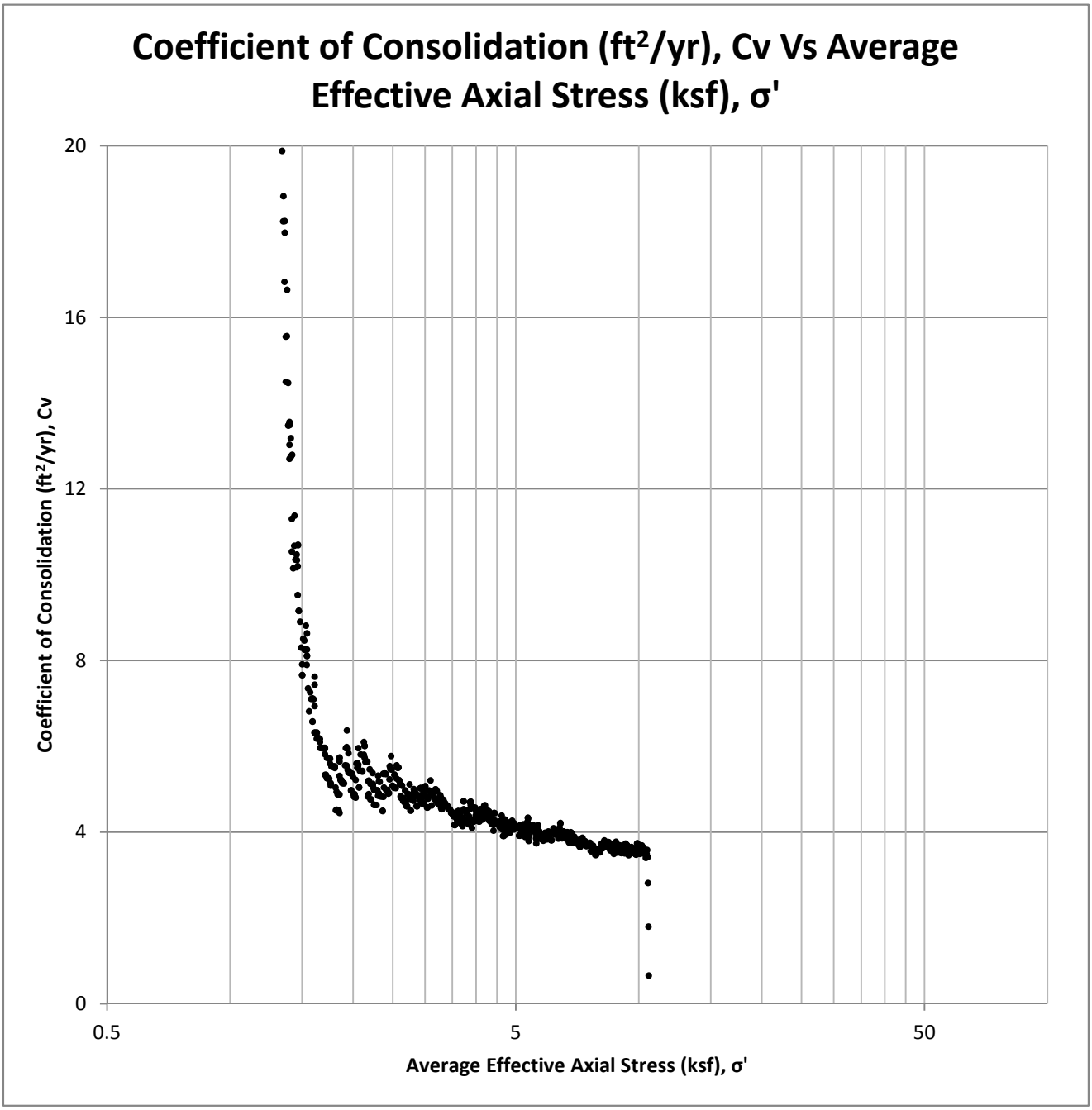
	Before	After
Moisture (%):	105.19%	57.07%
Dry Density (pcf):	46.81	63.00
Saturation (%):	115.83%	100.83%
Void Ratio:	2.1459	1.3374

Liquid Limits:		**Test Date:**	7/19/2014
Plastic Limits:			
Specific Gravity:	2.363	Measured	
Soil Description:	See exploration logs		
Soil Type:	Undisturbed		

Project Number:	11185.000.000	**Depth:**	11.0 ft.
Sample Number:	2-B1@11	**Boring #:**	B1
Project:	1210 California Circle (iStar Financial)		
Client:	iStar Financial		
Location:	Milpitas, California		
Tested By:	DWS		
Reviewed By:	JR		

ENGEO
Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



	Before	After			
Moisture (%):	105.19%	57.07%	**Liquid Limits:**		**Test Date:** 7/19/2014
Dry Density (pcf):	46.81	63.00	**Plastic Limits:**		
Saturation (%):	115.83%	100.83%	**Specific Gravity:**	2.363	Measured
Void Ratio:	2.1459	1.3374	**Soil Description:**	See exploration logs	
			Soil Type:	Undisturbed	

Project Number:	11185.000.000	**Depth:**	11.0 ft.
Sample Number:	2-B1@11	**Boring #:**	B1
Project:	1210 California Circle (iStar Financ	**Tested By:**	DWS
Client:	iStar Financial	**Reviewed By:**	JR
Location:	Milpitas, California		

ENGEO
Expect Excellence

APPENDIX C

Cone Penetration Test (CPT) Logs
(Gregg Drilling & Testing Inc., 2014)

DRAFT

June 2, 2014

Engeo
Attn: Jeanine Ruffoni

Subject: CPT Site Investigation
1210 California Circle
Milpitas, California
GREGG Project Number: 14-094MA

Dear Ms. Ruffoni:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	UVOST Laser Induced Fluorescence	(UVOST)	☐
5	Groundwater Sampling	(GWS)	☐
6	Soil Sampling	(SS)	☐
7	Vapor Sampling	(VS)	☐
8	Pressuremeter Testing	(PMT)	☐
9	Vane Shear Testing	(VST)	☐
10	Dilatometer Testing	(DMT)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
1-CPT1	5/30/14	50	-	-	40.8
1-CPT2	5/30/14	50	-	-	21.8
1-CPT3	5/30/14	50	-	-	-
1-CPT4	5/30/14	50	-	-	-
1-CPT5	5/30/14	50	-	-	33.1
1-CPT6	5/30/14	50	-	-	-

Bibliography

Lunne, T., Robertson, P.K. and Powell, J.J.M., "Cone Penetration Testing in Geotechnical Practice" E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., "Soil Classification using the Cone Penetration Test", Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

Mayne, P.W., "NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization", available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp. 791-803.

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Robertson, P.K., T. Lunne and J.J.M. Powell, "Geo-Environmental Application of Penetration Testing", Geotechnical Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

Campanella, R.G. and I. Weemees, "Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies", Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegger, "Reliability of Soil Gas Sampling and Characterization Techniques", International Site Characterization Conference - Atlanta, 1998.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, "Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT", 53rd Canadian Geotechnical Conference Montreal, QC October pp. 733-739, 2000.

Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, "Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers" BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

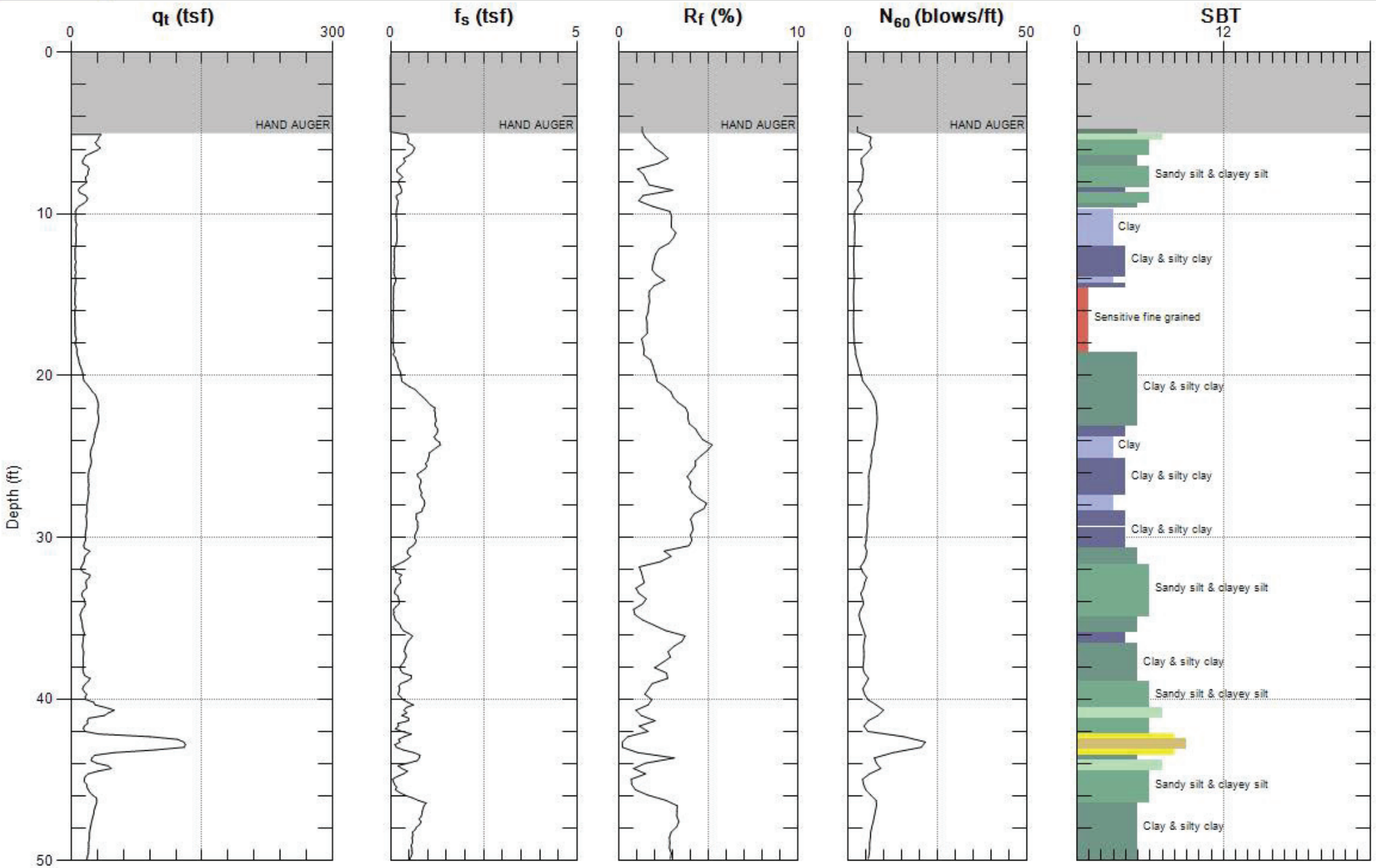
Copies of ASTM Standards are available through www.astm.org

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT1

Engineer: J.RUFFONI
Date: 5/30/2014 01:02



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

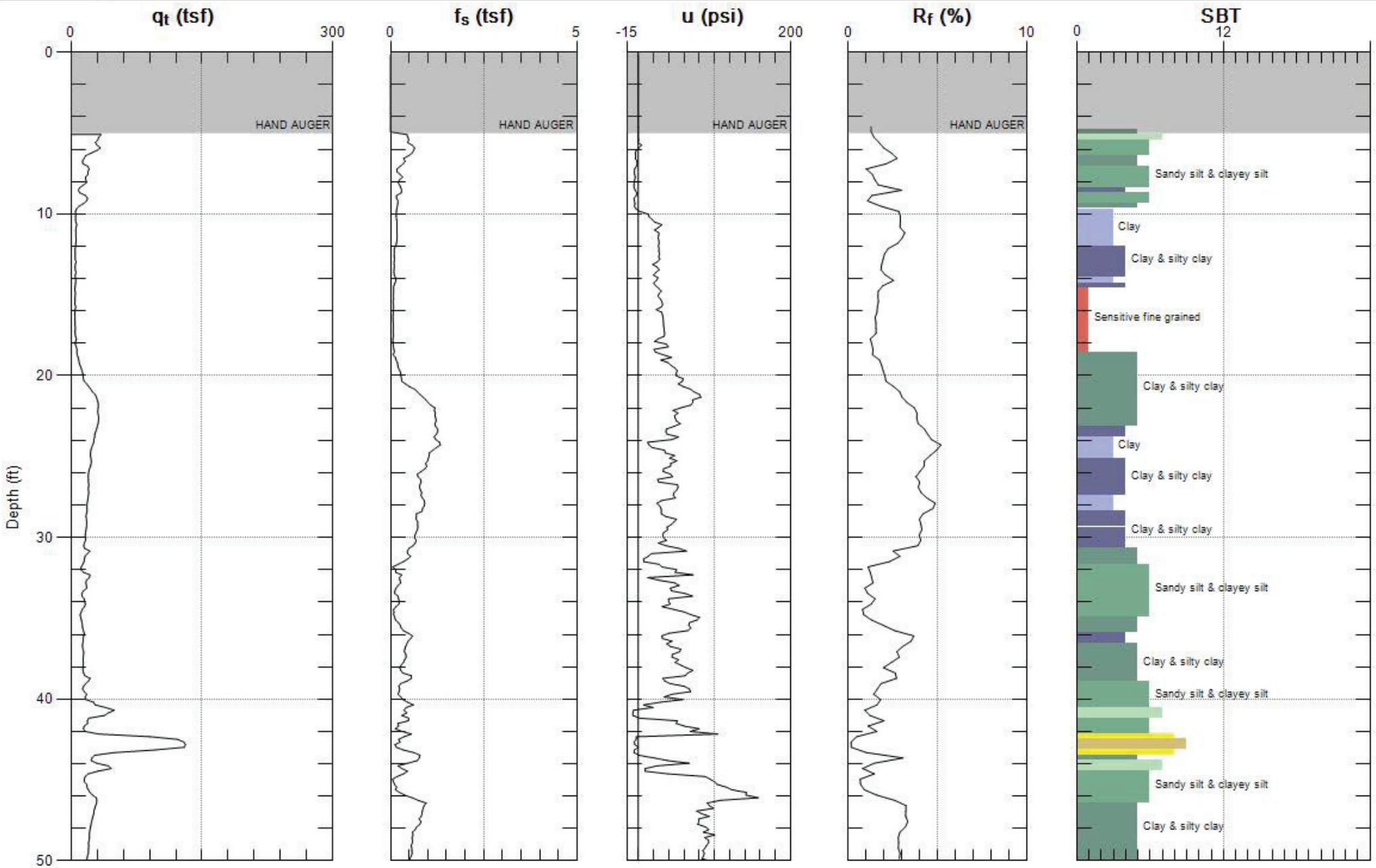
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT1

Engineer: J.RUFFONI
Date: 5/30/2014 01:02



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

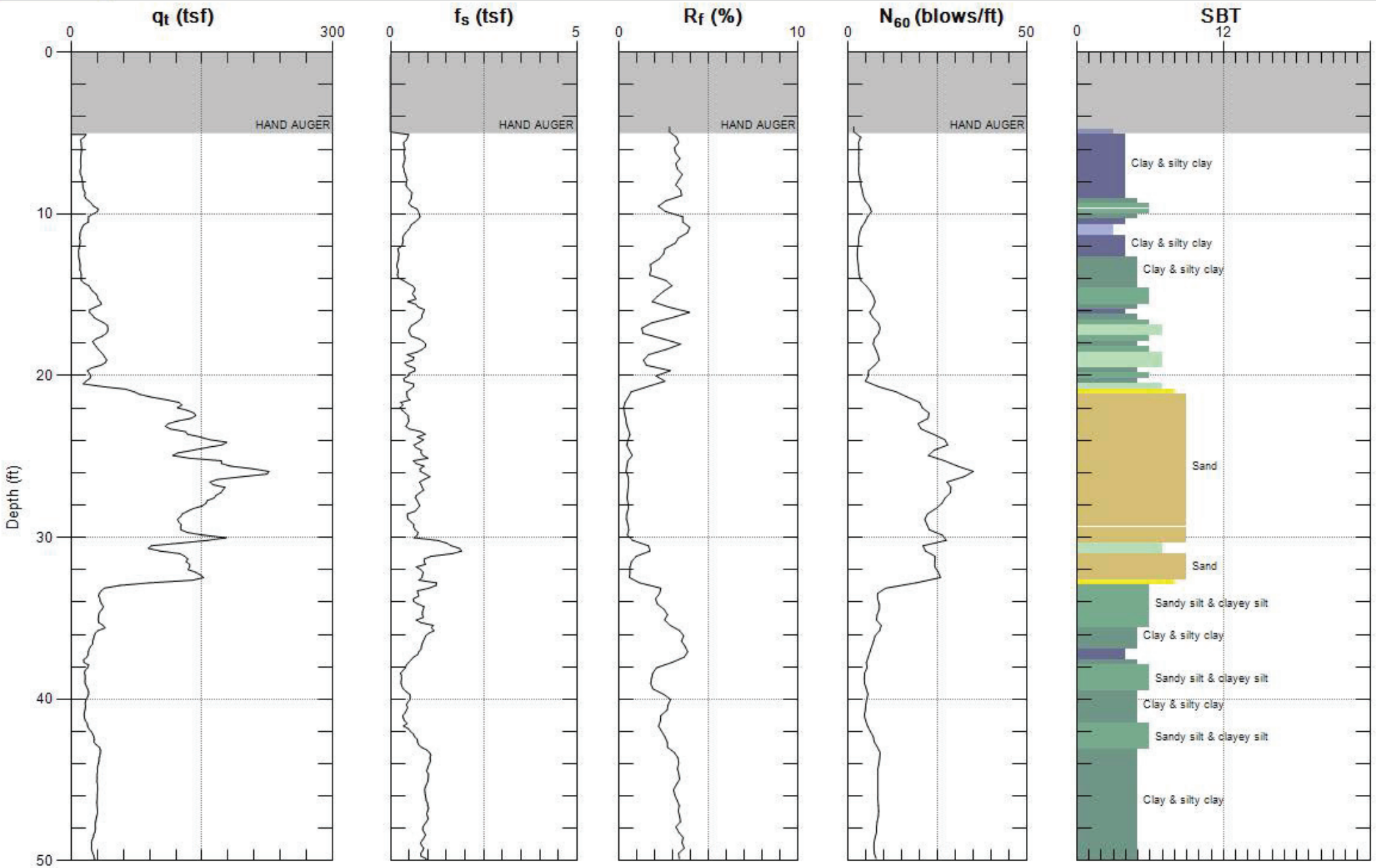
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT2

Engineer: J.RUFFONI
Date: 5/30/2014 10:31



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

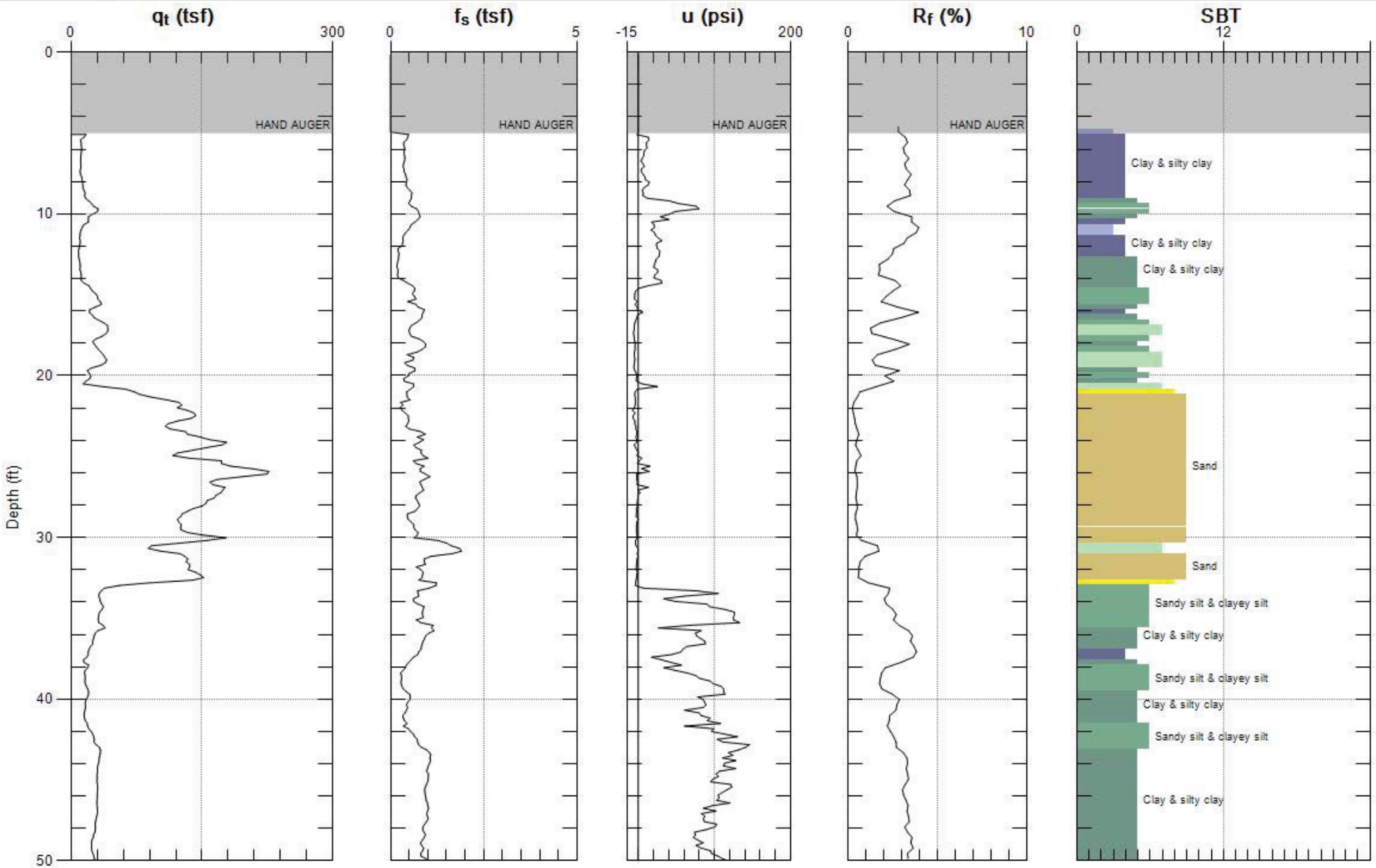
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT2

Engineer: J.RUFFONI
Date: 5/30/2014 10:31



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

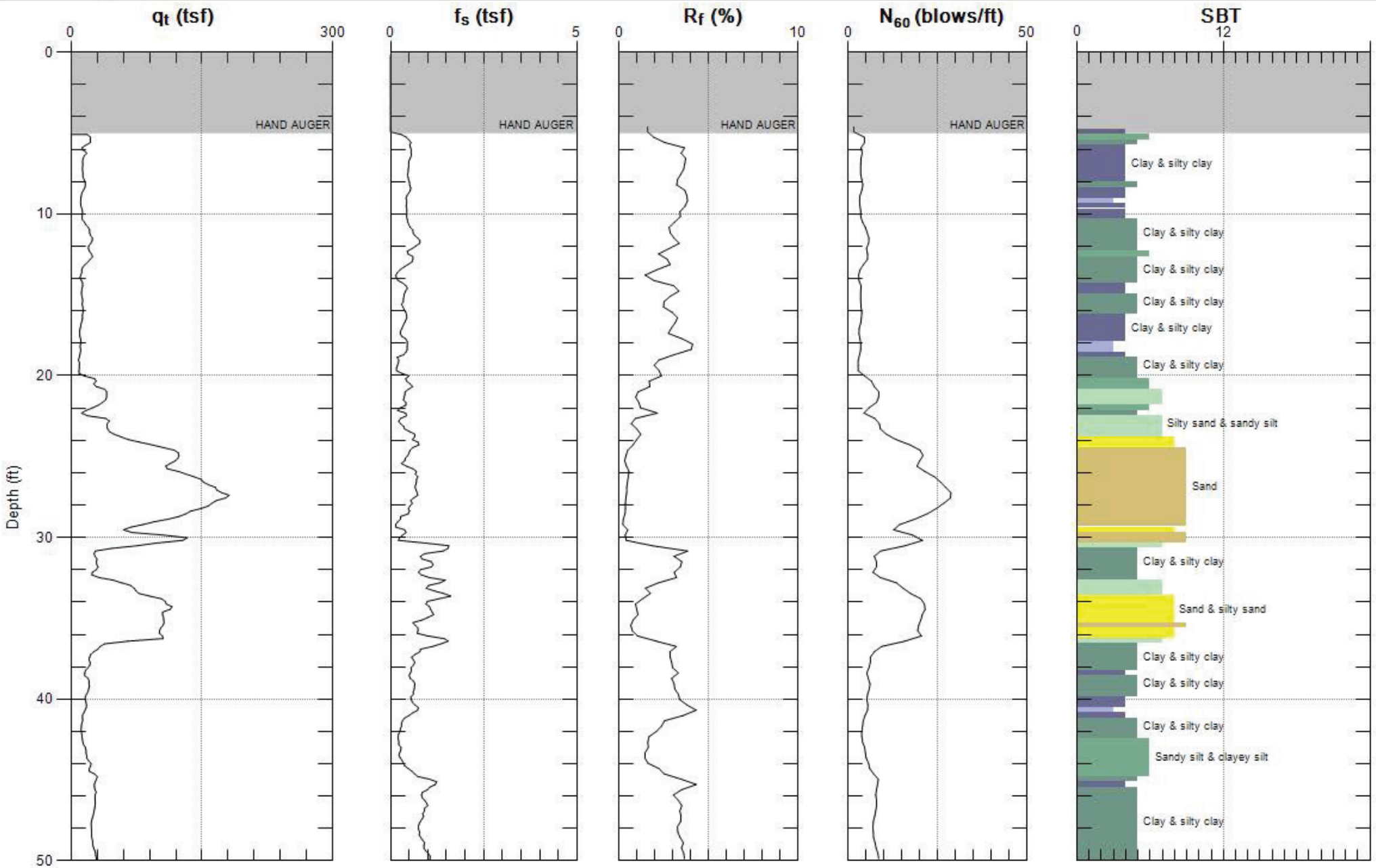
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT3

Engineer: J.RUFFONI
Date: 5/30/2014 11:44



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

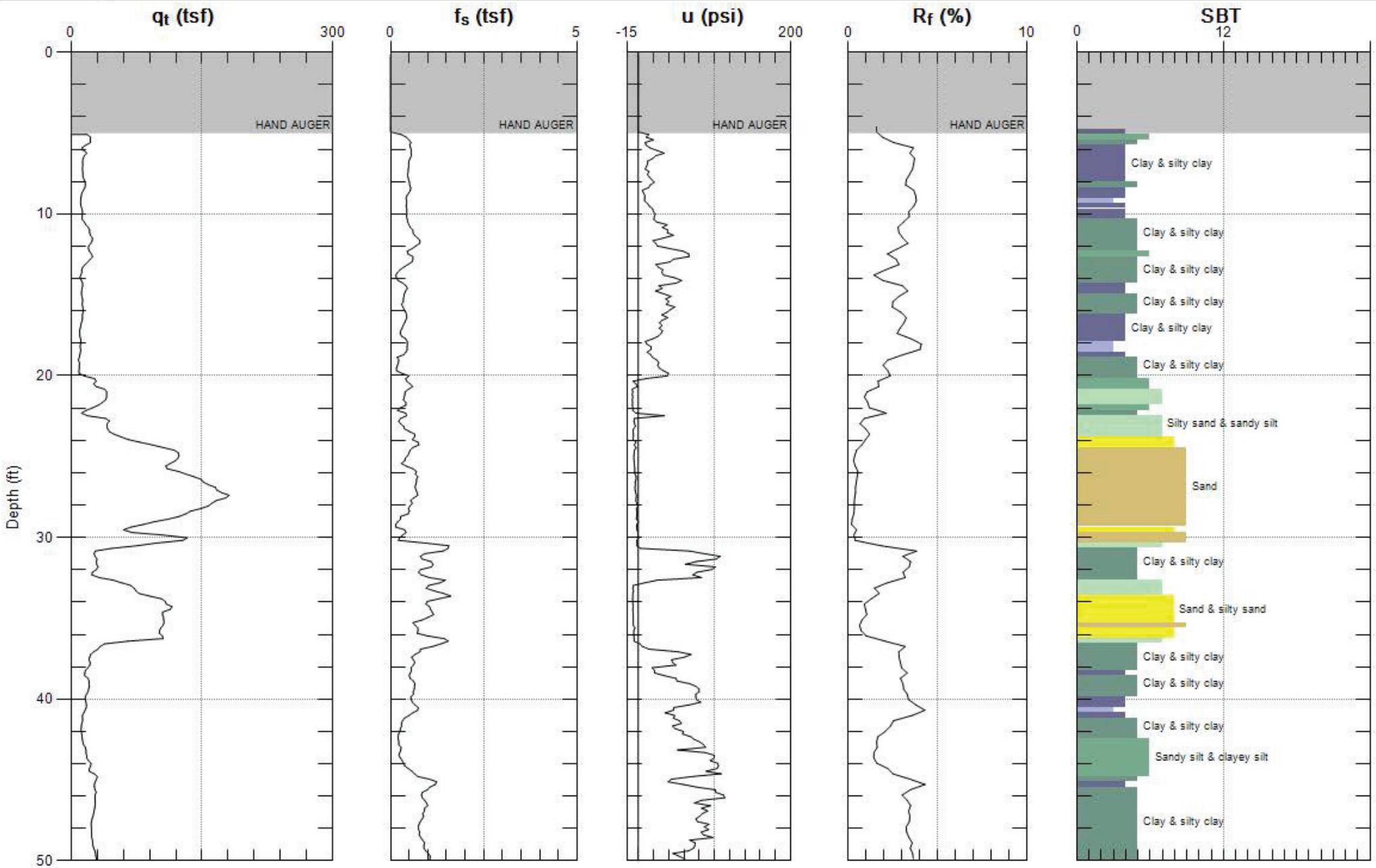
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT3

Engineer: J.RUFFONI
Date: 5/30/2014 11:44



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

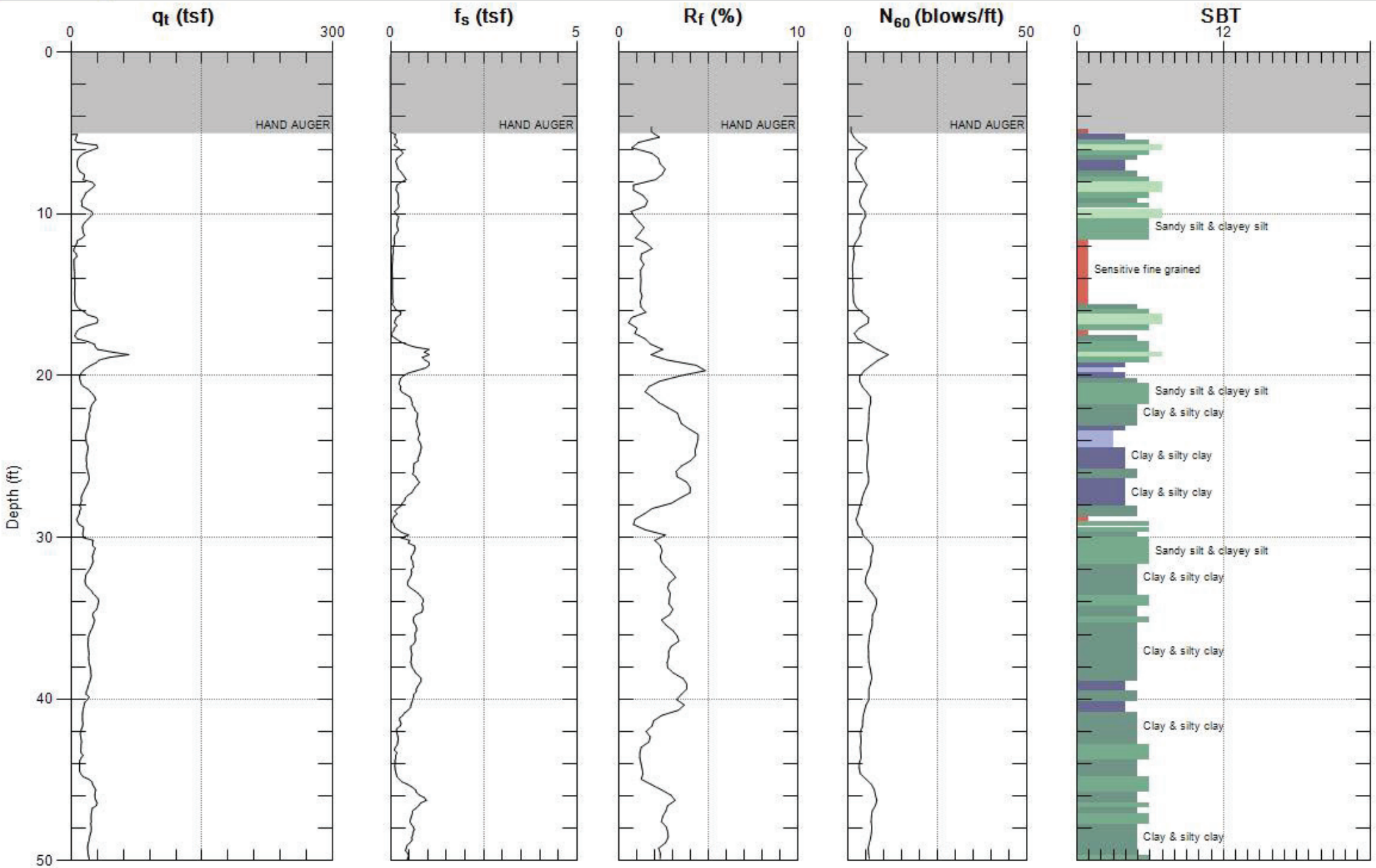
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT4

Engineer: J.RUFFONI
Date: 5/30/2014 02:06



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

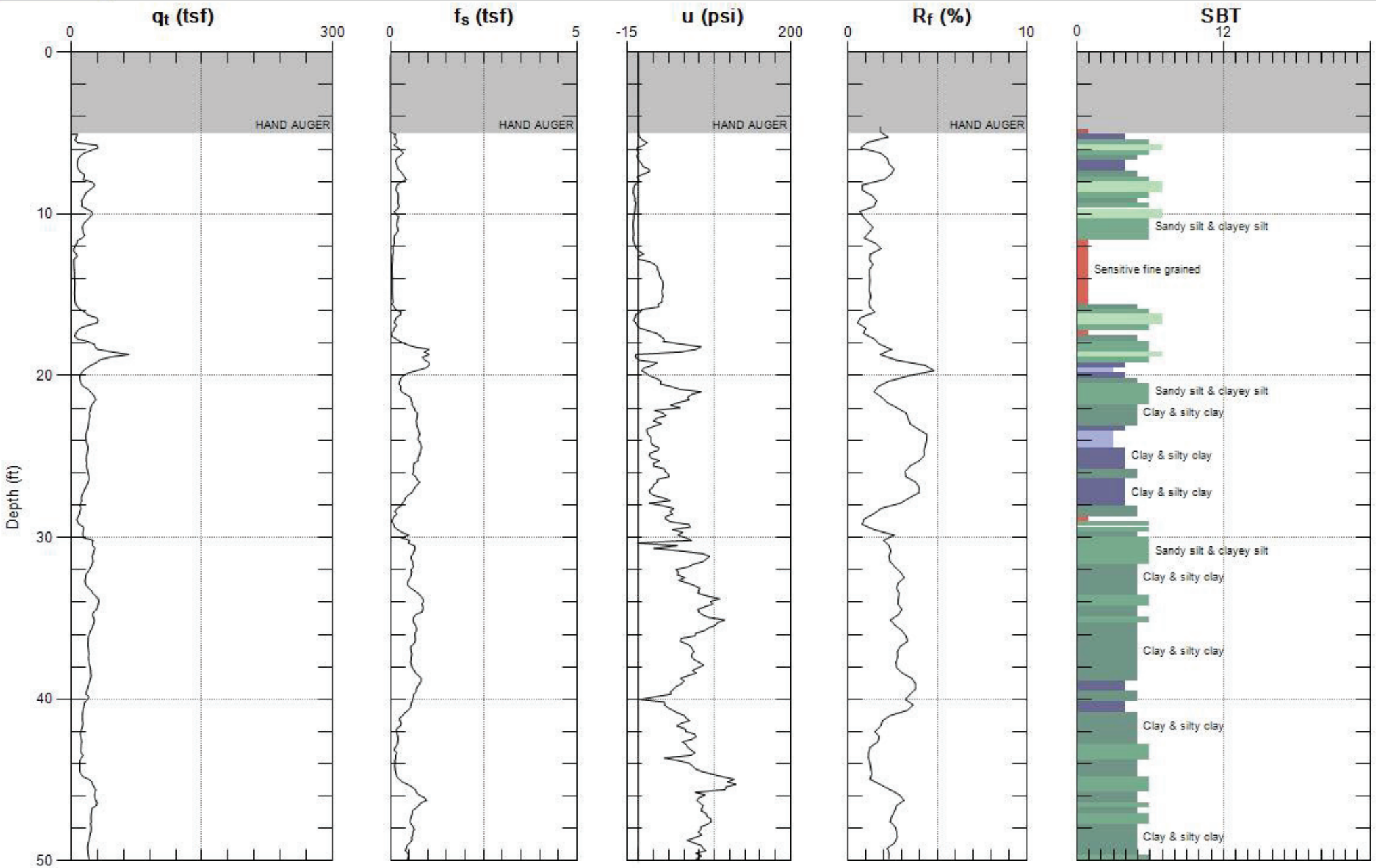
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT4

Engineer: J.RUFFONI
Date: 5/30/2014 02:06



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

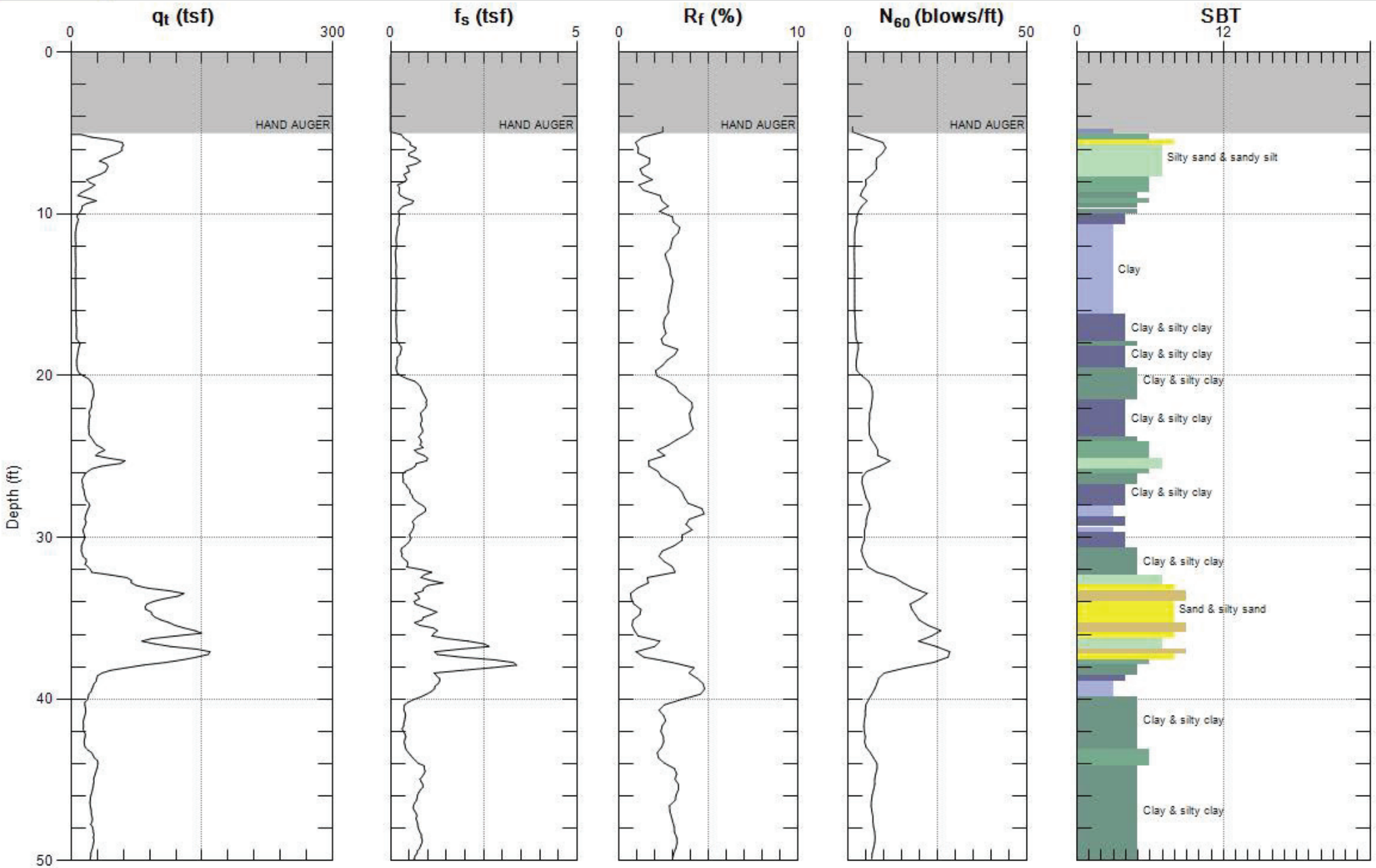
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT5

Engineer: J.RUFFONI
Date: 5/30/2014 08:27



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

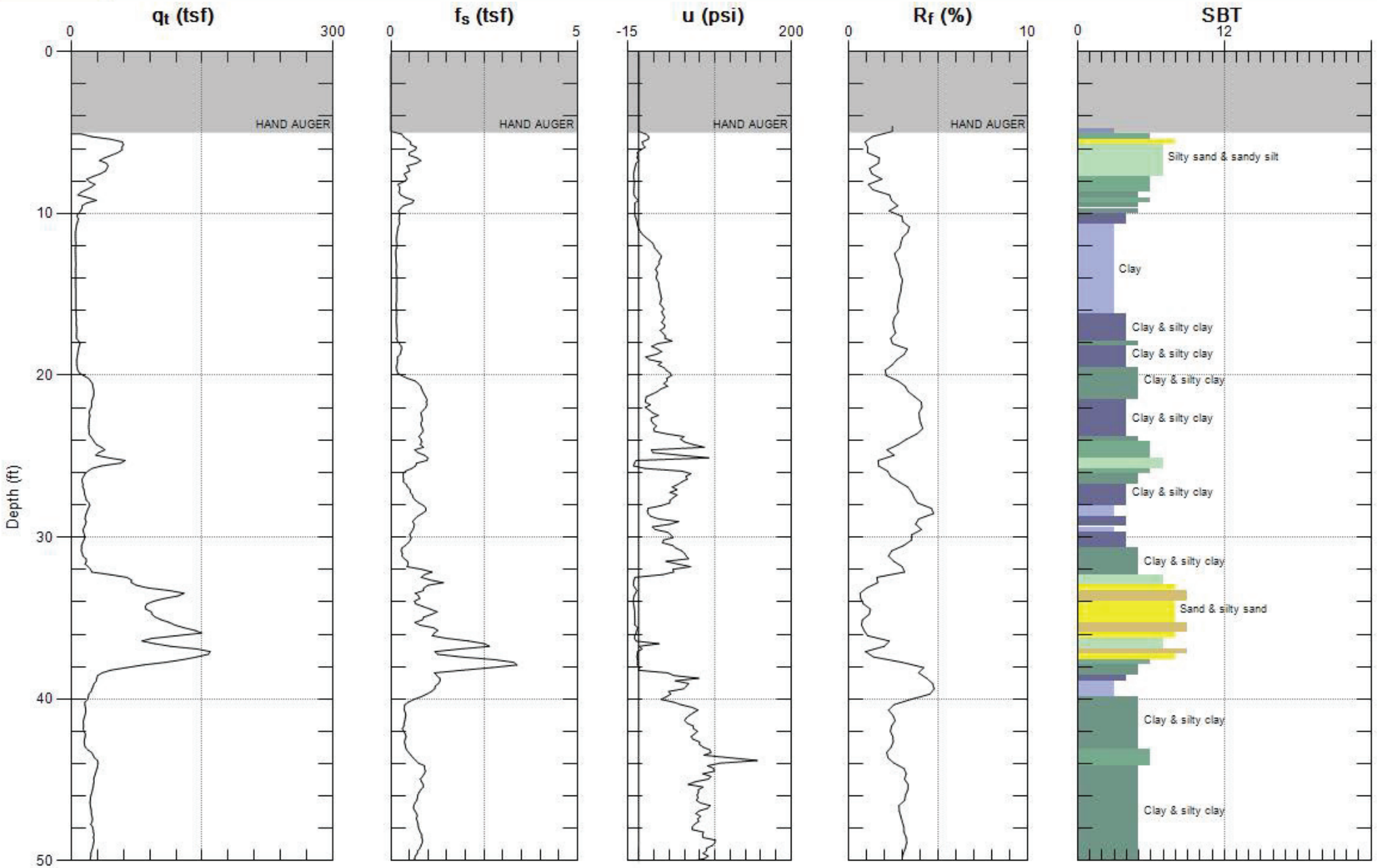
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT5

Engineer: J.RUFFONI
Date: 5/30/2014 08:27



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

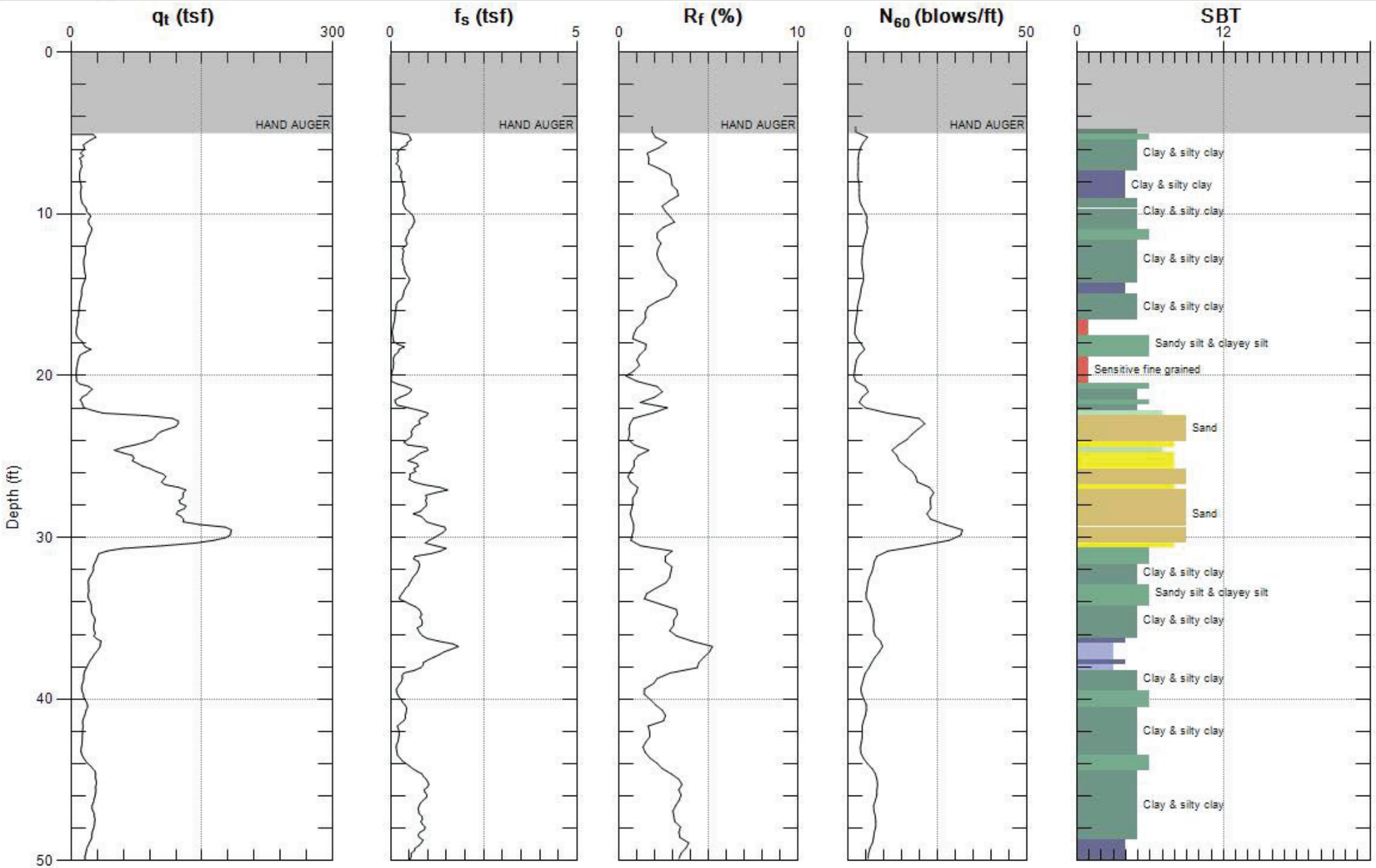
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT6

Engineer: J.RUFFONI
Date: 5/30/2014 09:35



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

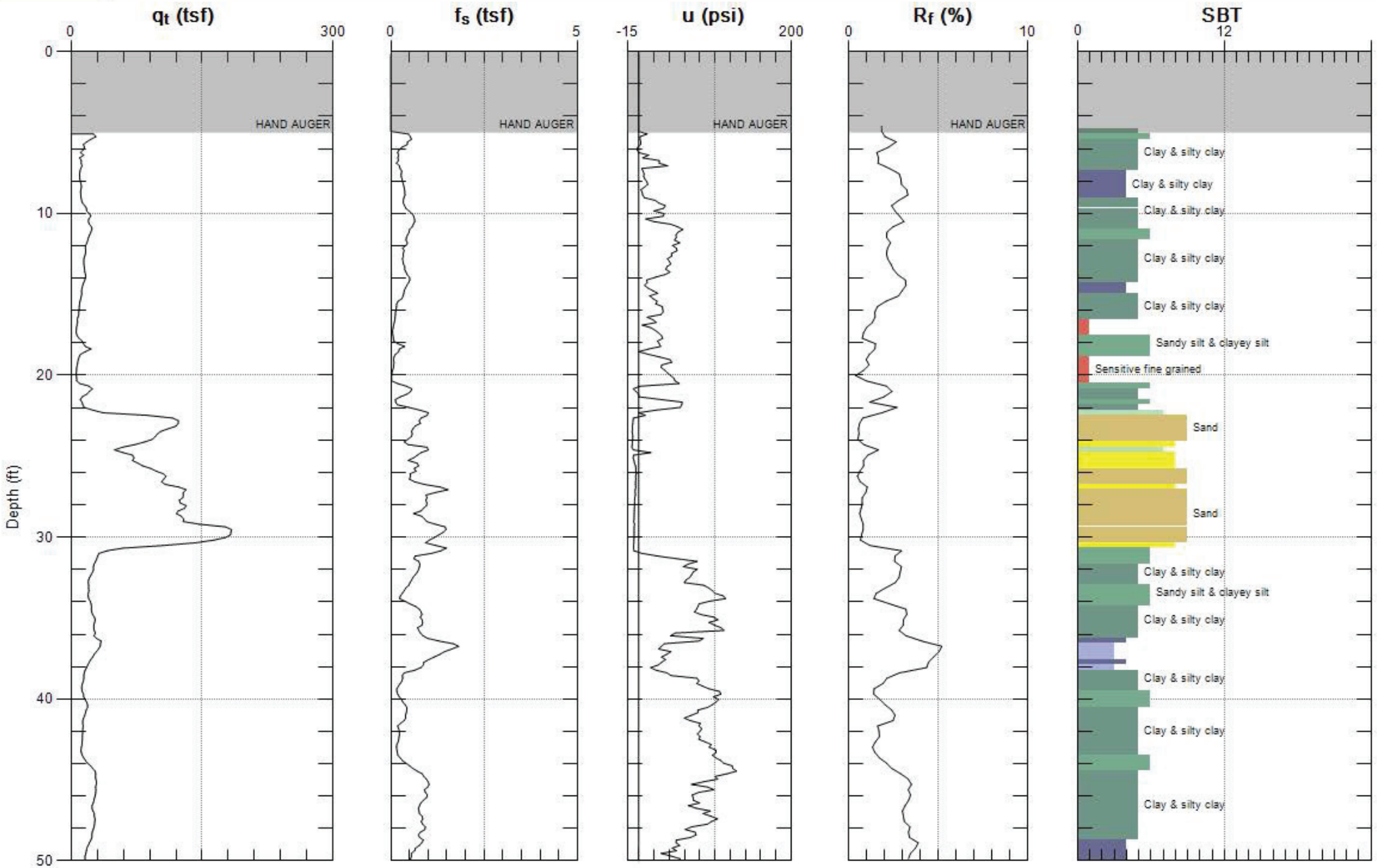
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: 1210 CALIFORNIA CIR.
Sounding: 1-CPT6

Engineer: J.RUFFONI
Date: 5/30/2014 09:35



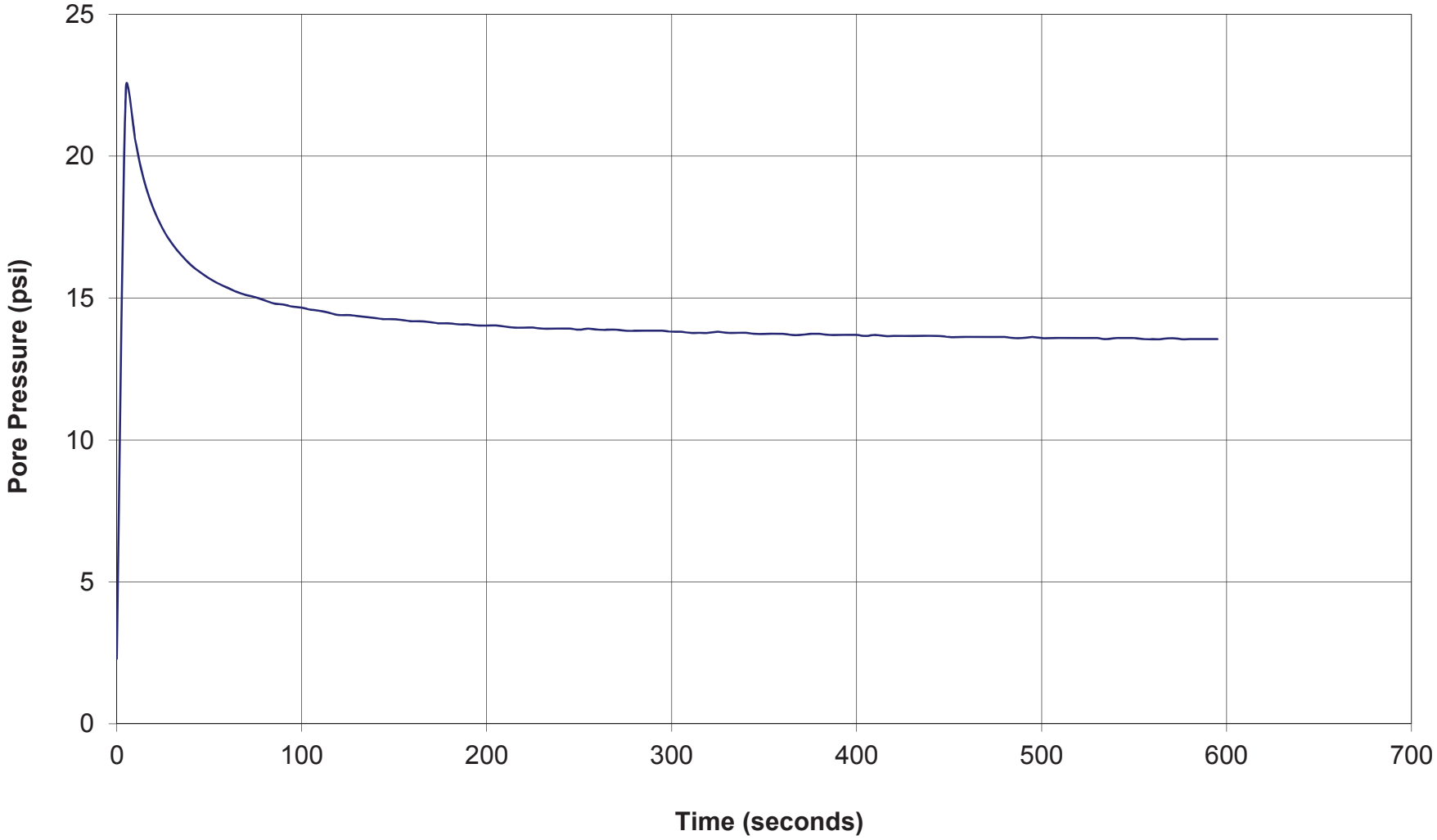
Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

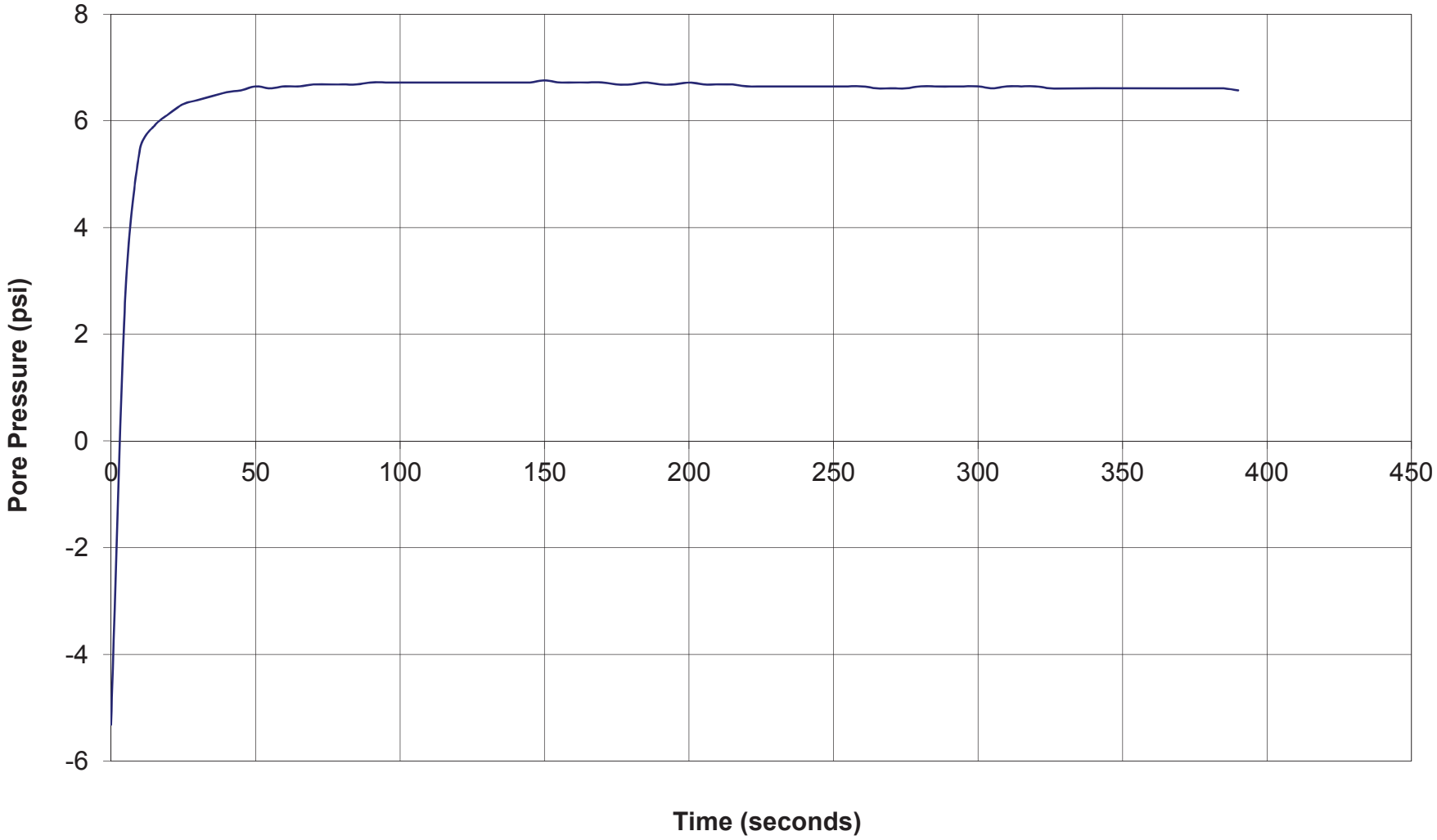
Sounding: 1-CPT1
Depth: 40.8463335
Site: 1210 CALIFORNIA
Engineer: J.RUFFONI



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

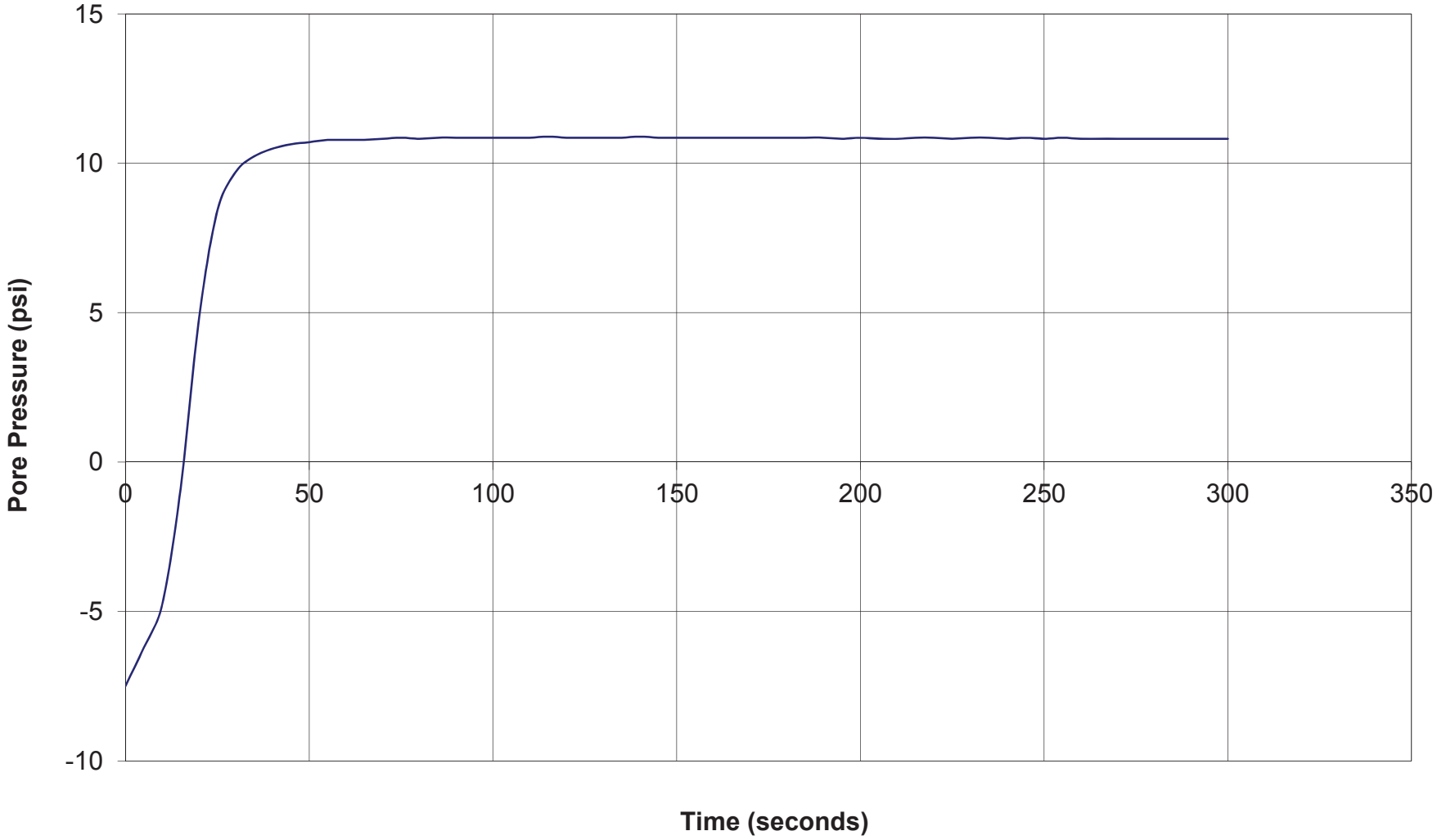
Sounding: 1-CPT2
Depth: 21.8175195
Site: 1210 CALIFORNIA
Engineer: J.RUFFONI



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: 1-CPT5
Depth: 33.136383
Site: 1210 CALIFORNIA
Engineer: J.RUFFONI



APPENDIX D

Liquefaction Analysis

DRAFT

APPENDIX D

ENGEO Incorporated
engeo.com

LIQUEFACTION ANALYSIS REPORT

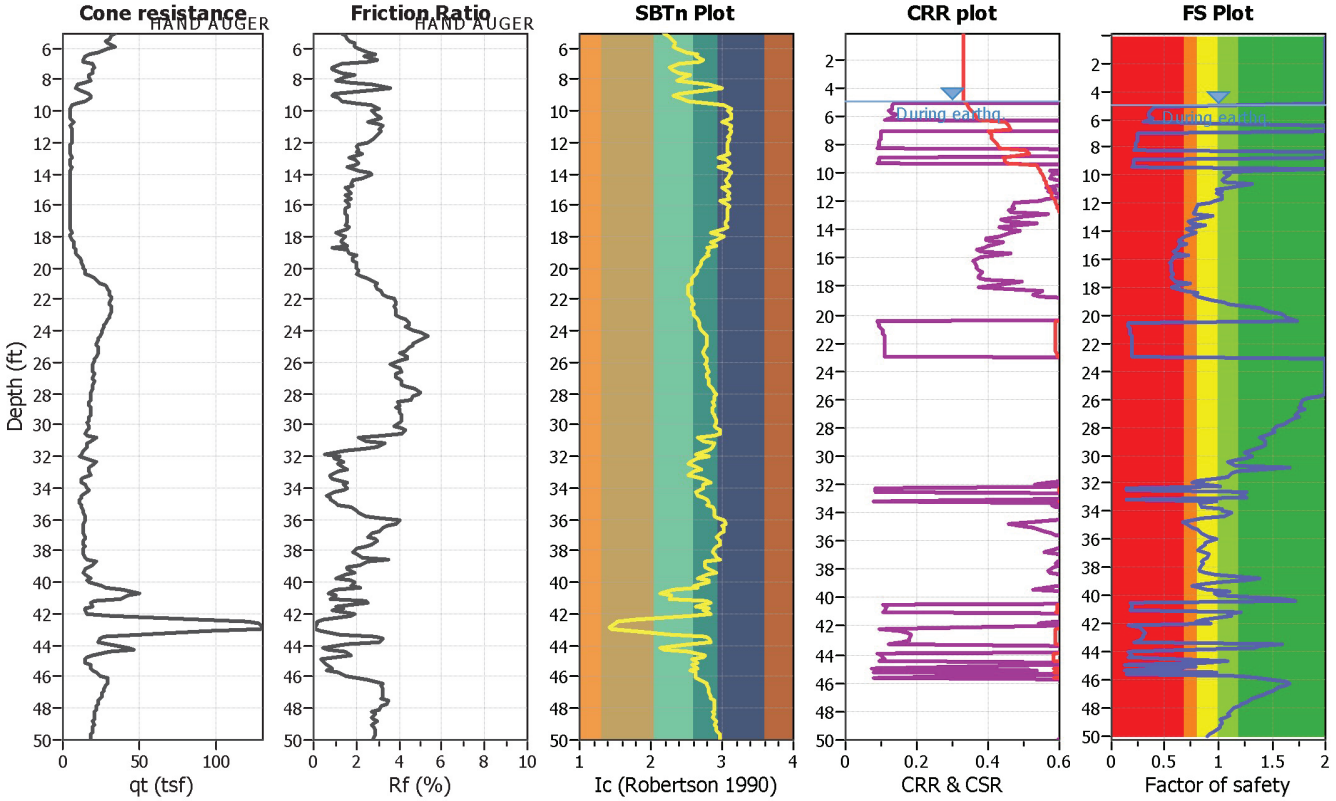
Project title : 11185.000.000

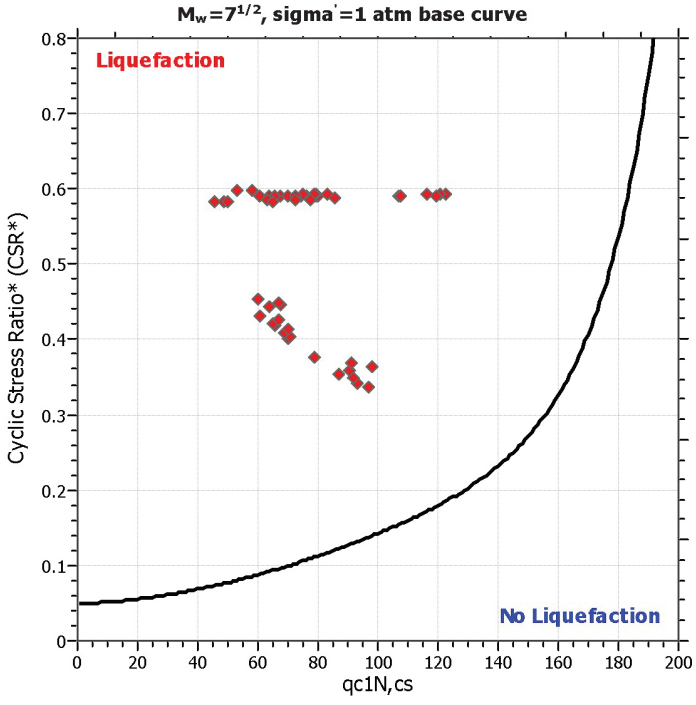
Location : 1210 California Circle, Milpitas, CA

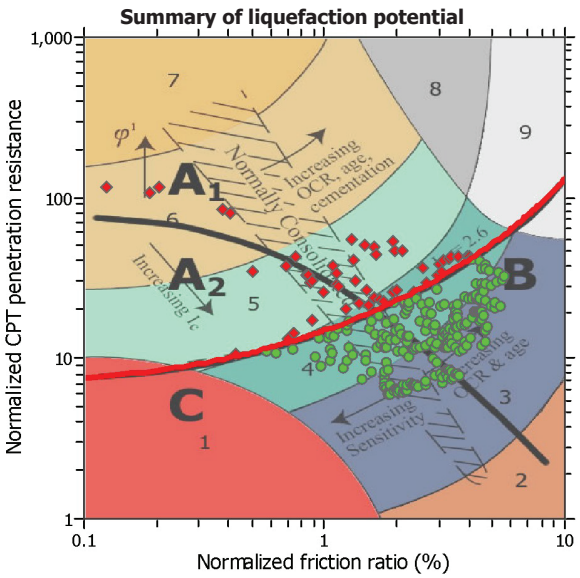
CPT file : 1-CPT1

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

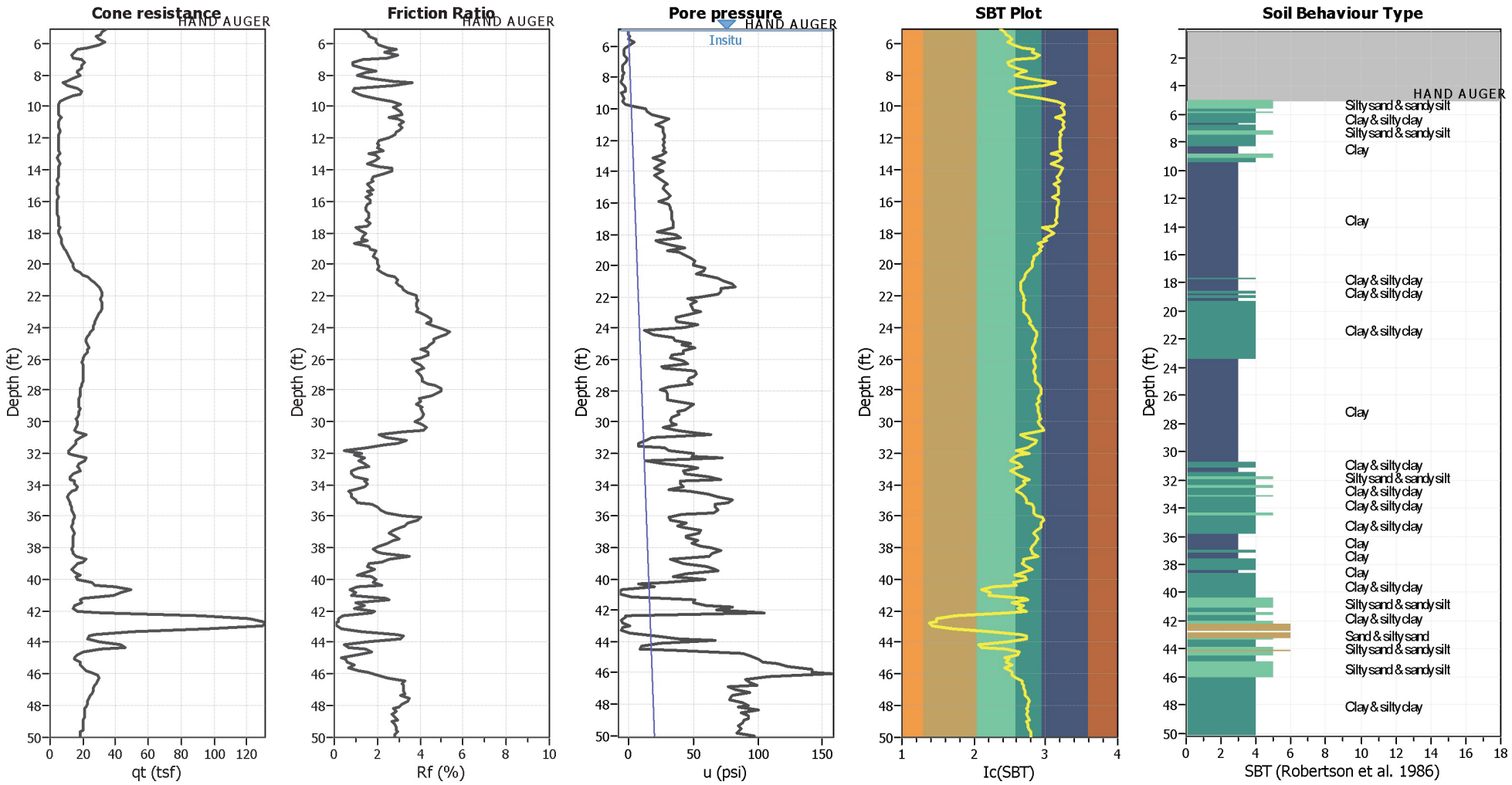






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



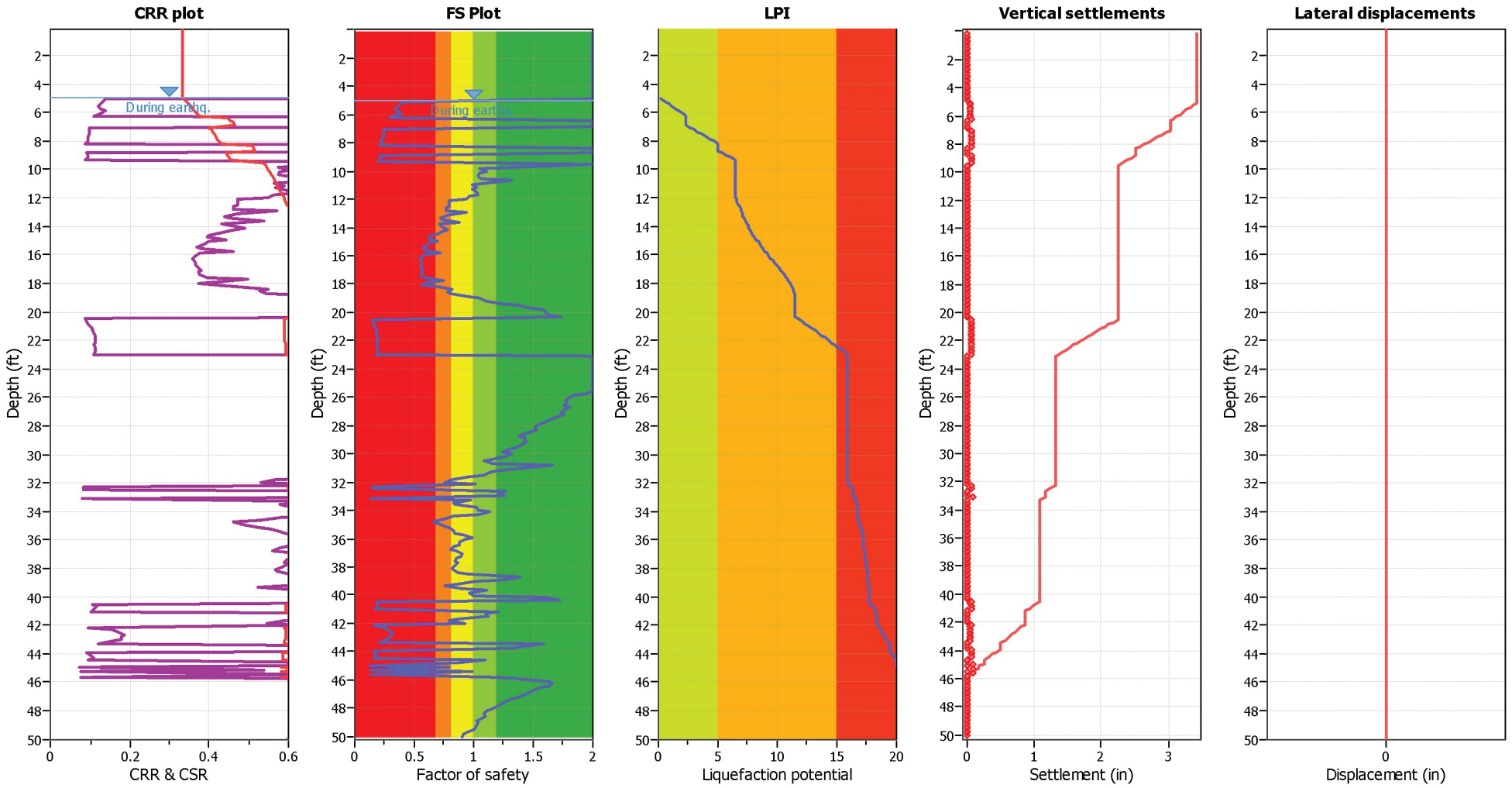
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	1	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:	6.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	1	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:	6.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

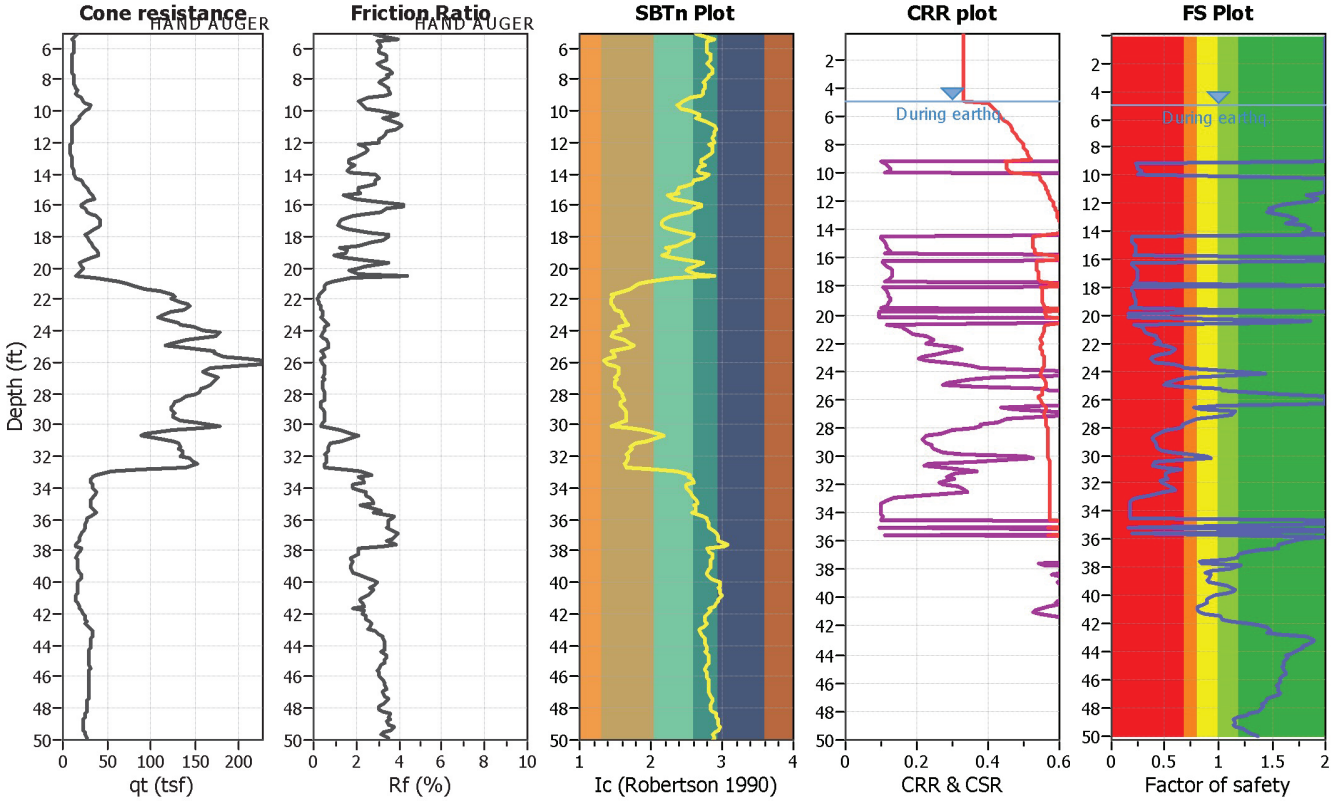
LIQUEFACTION ANALYSIS REPORT

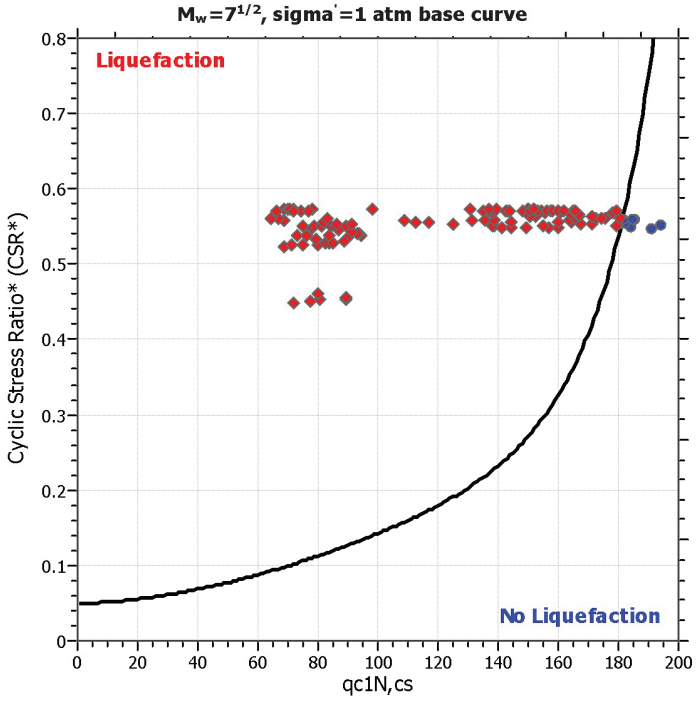
Project title : 11185.000.000 **Location : 1210 California Circle, Milpitas, CA**

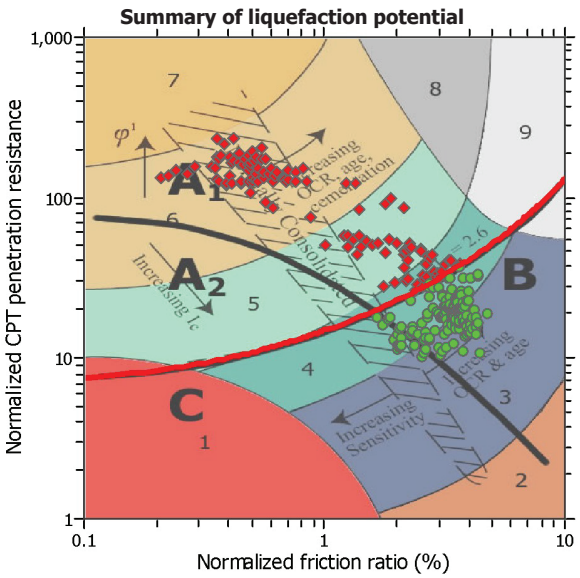
CPT file : 1-CPT2

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

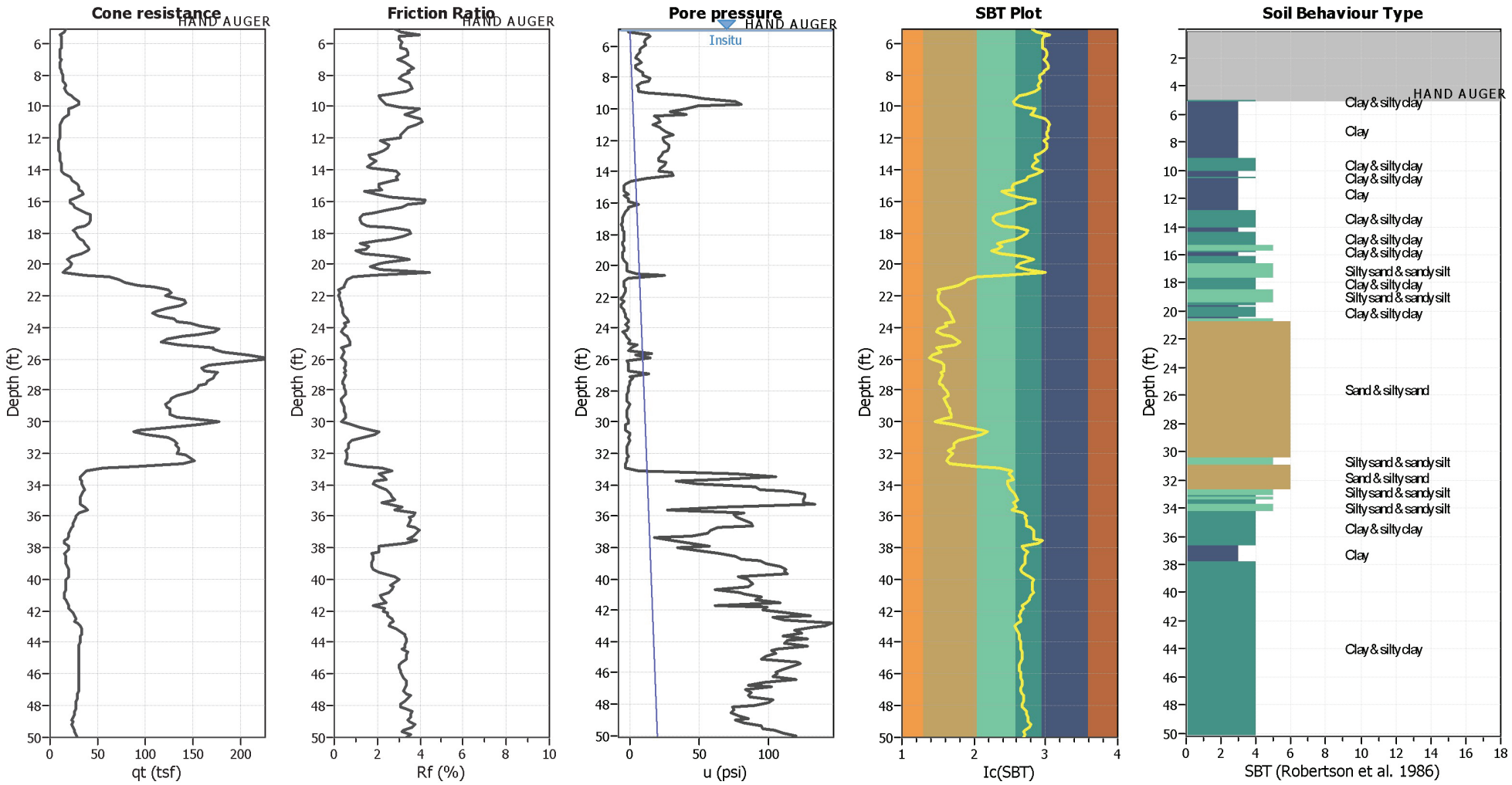






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



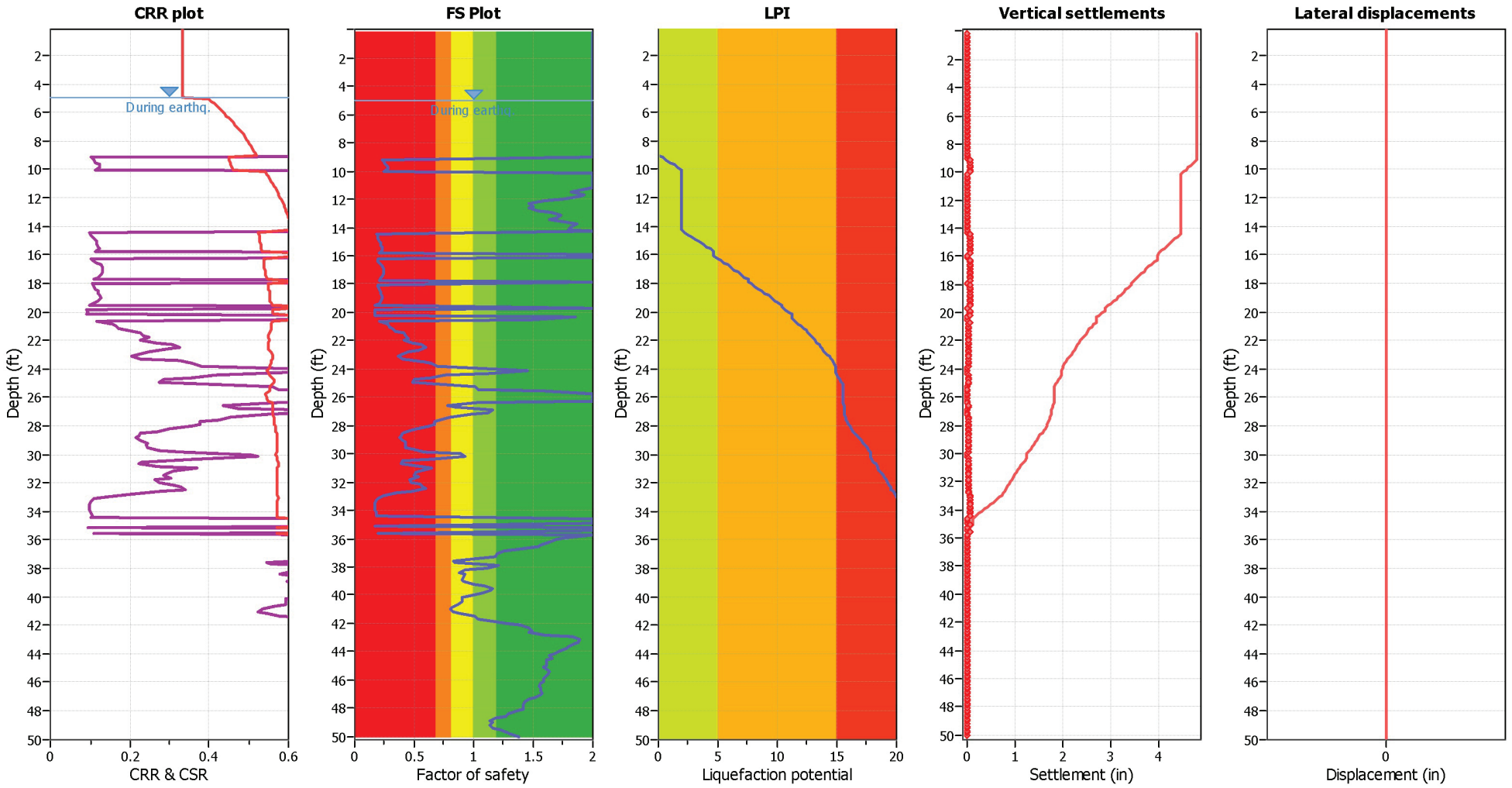
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	1	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:	6.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	1	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:	6.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

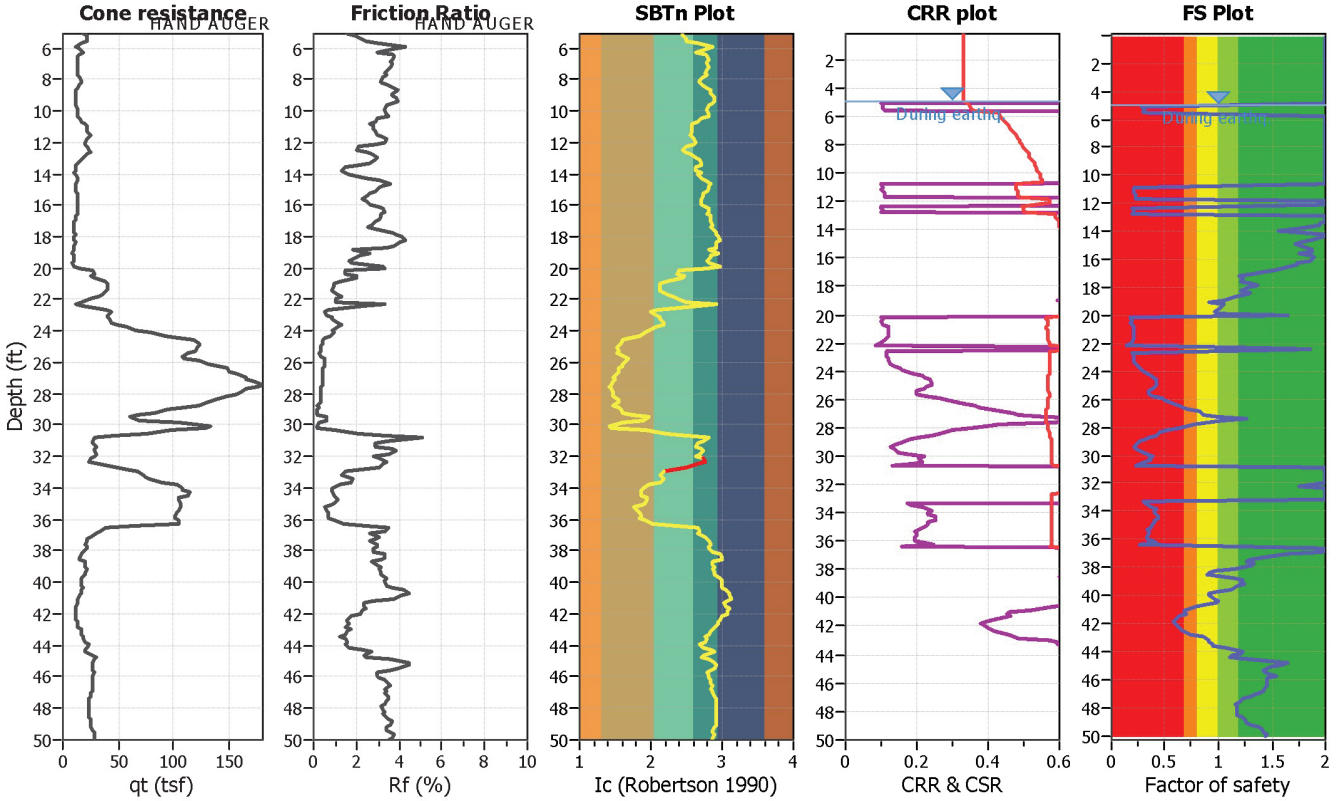
LIQUEFACTION ANALYSIS REPORT

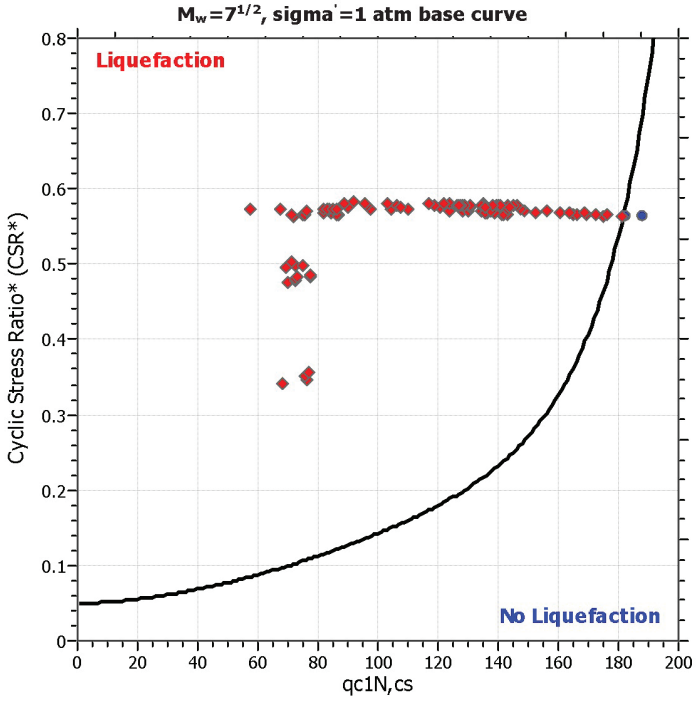
Project title : 11185.000.000 **Location : 1210 California Circle, Milpitas, CA**

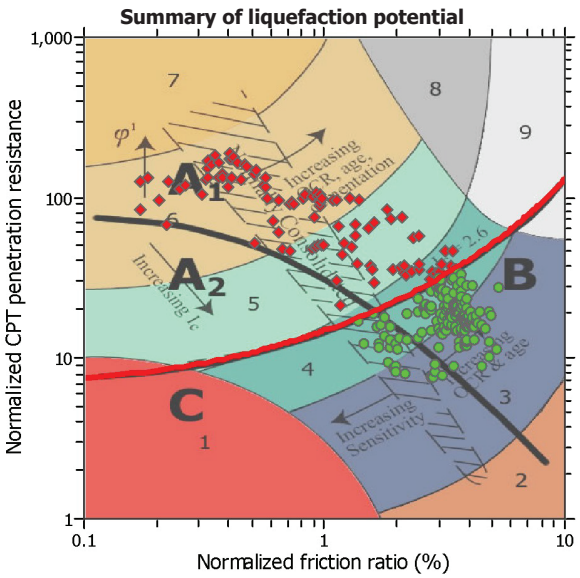
CPT file : 1-CPT3

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

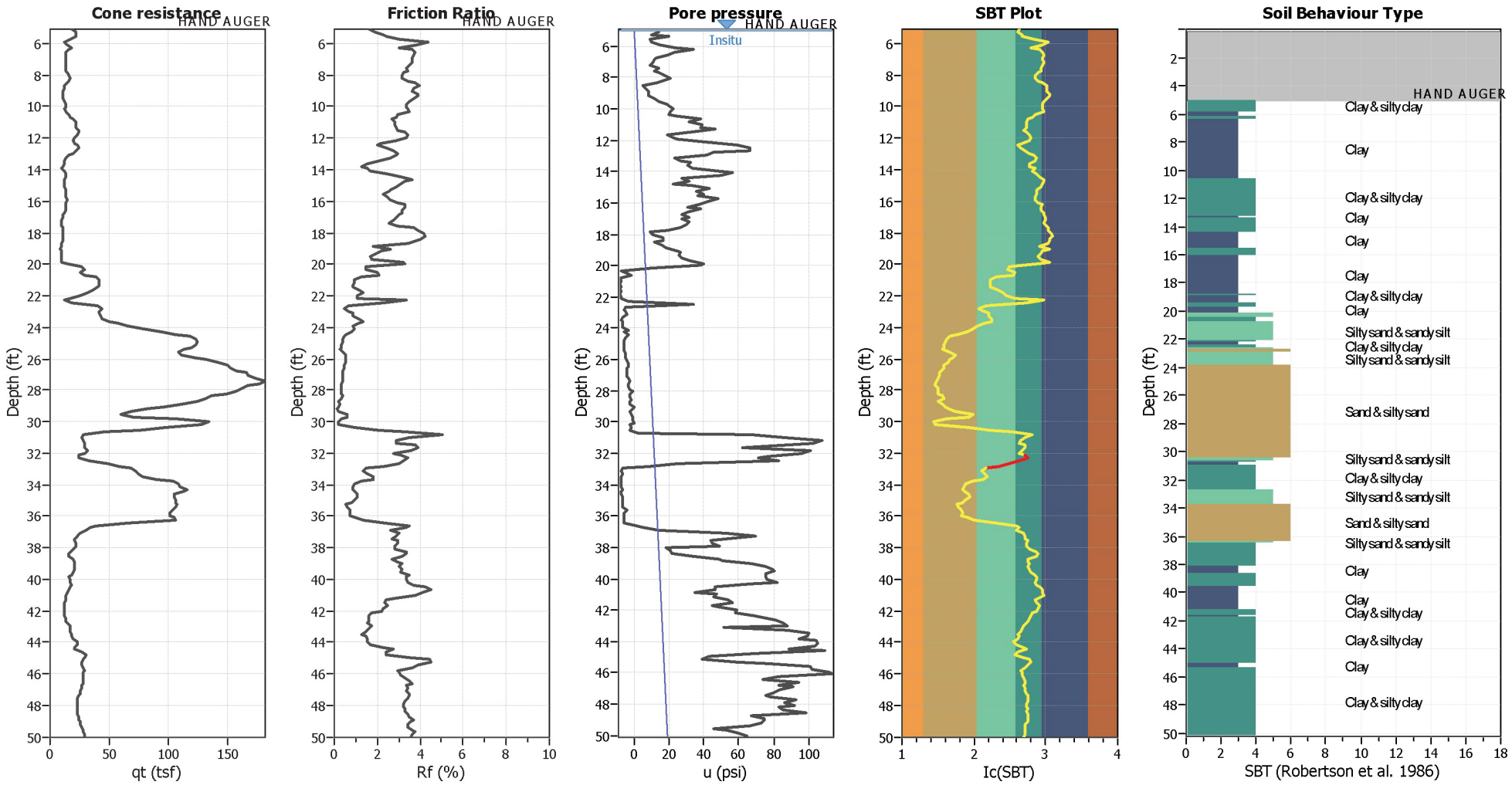






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



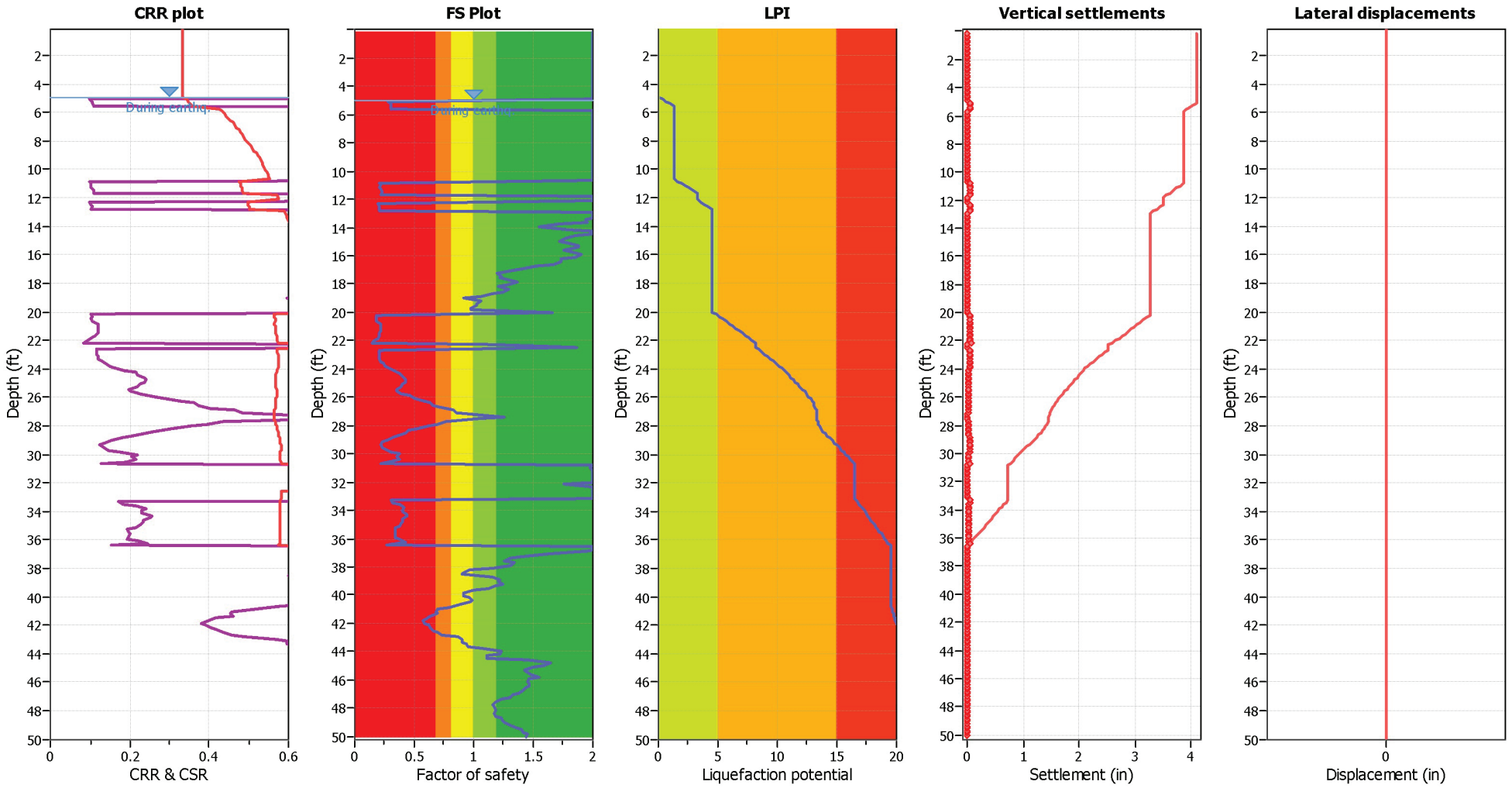
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	1	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:	6.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	1	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:	6.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

LIQUEFACTION ANALYSIS REPORT

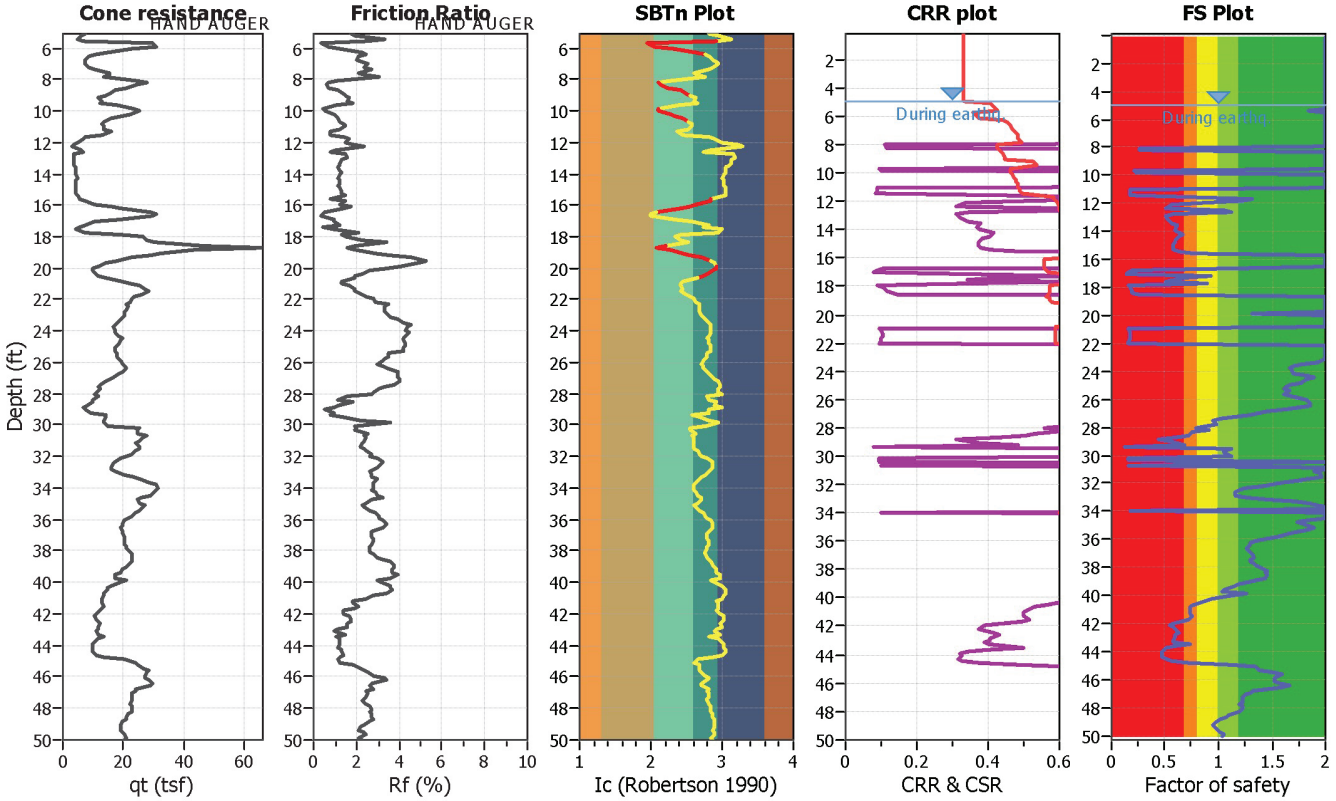
Project title : 11185.000.000

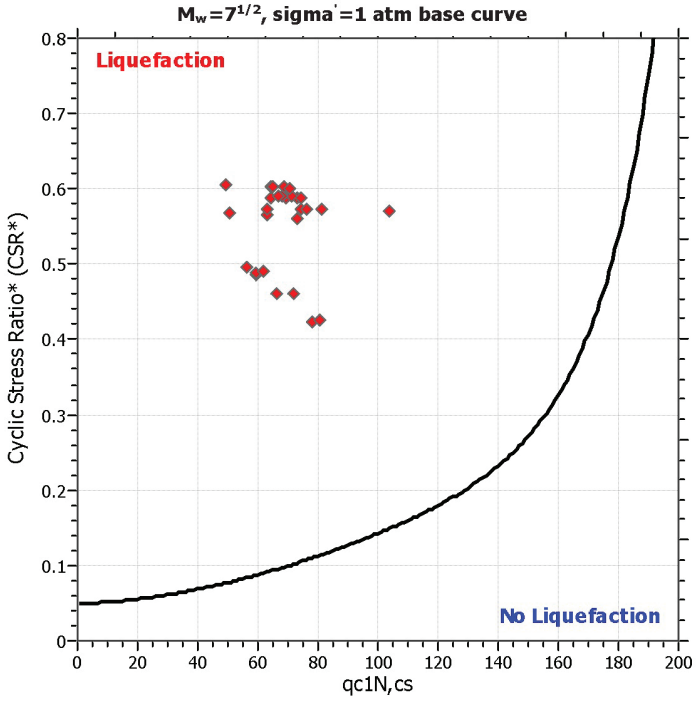
Location : 1210 California Circle, Milpitas, CA

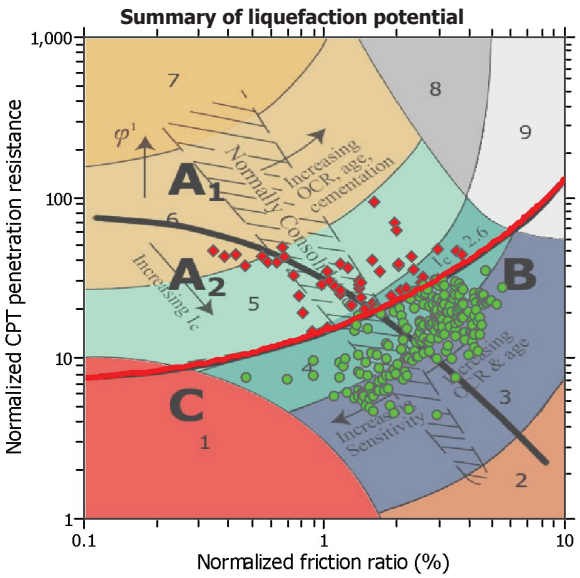
CPT file : 1-CPT4

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

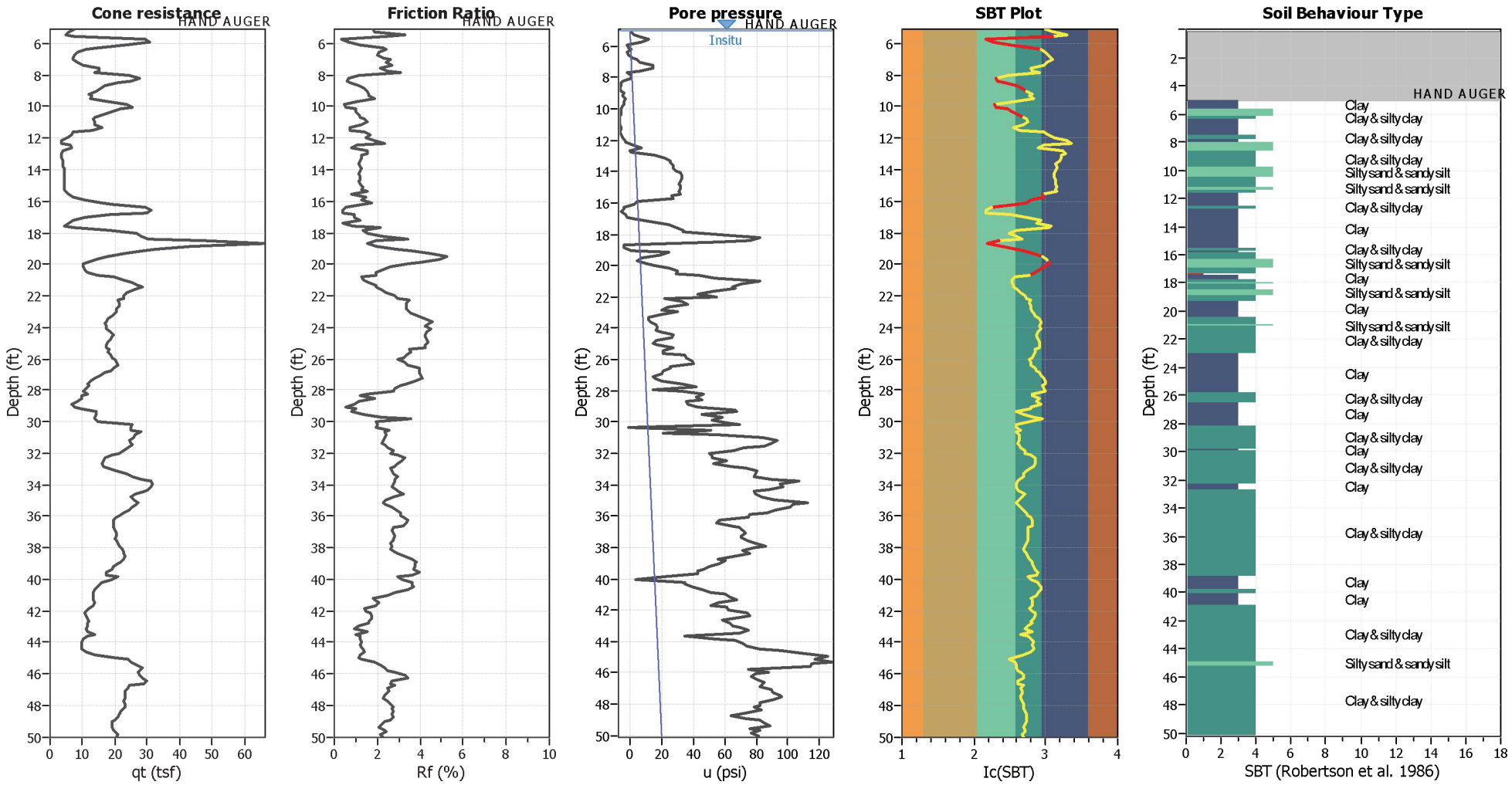






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



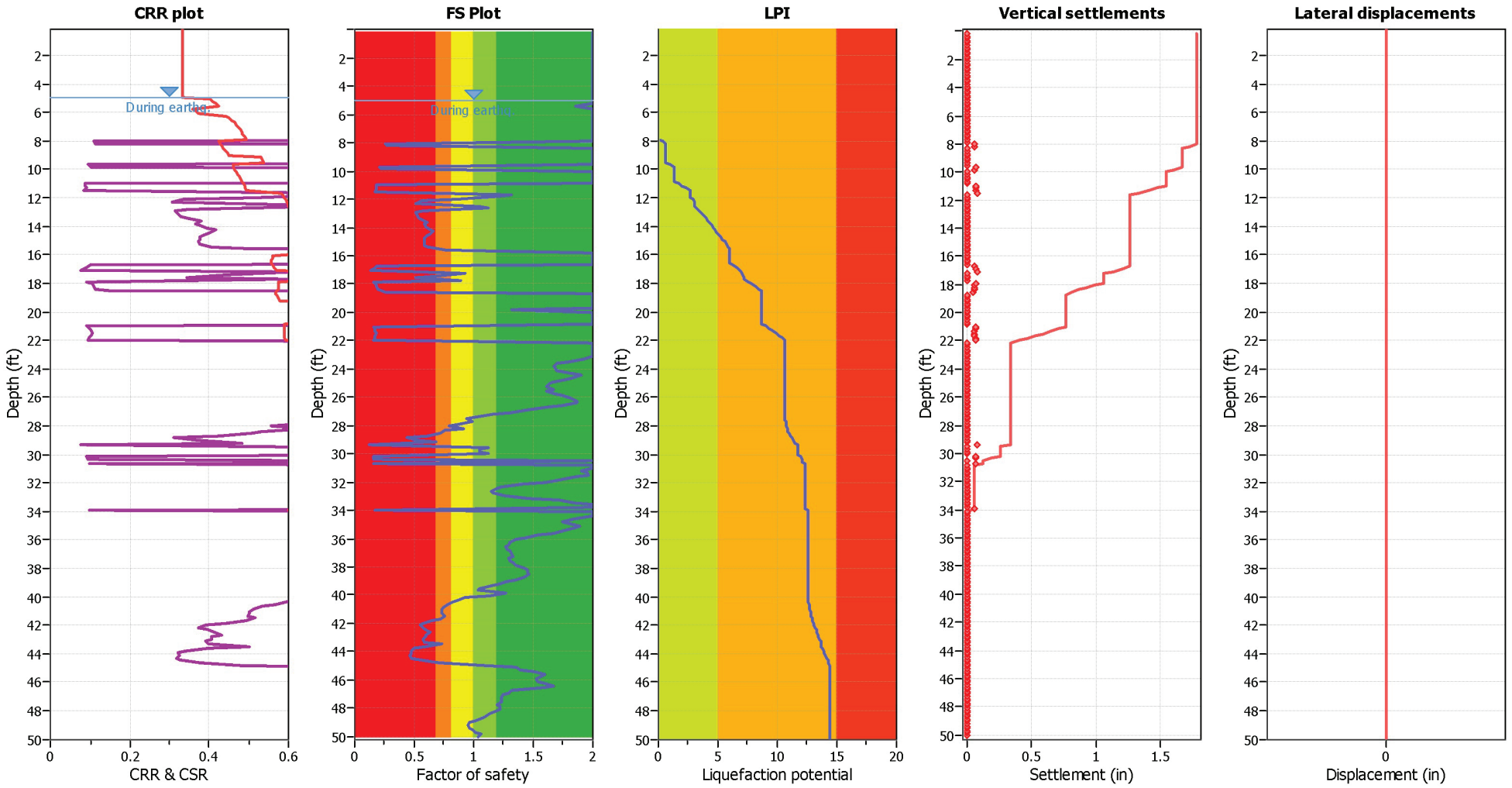
Input parameters and analysis data

Analysis method:	I&B (2008)	Depth to GWT (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	I&B (2008)	Average results interval:	1	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:	6.80	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



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Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

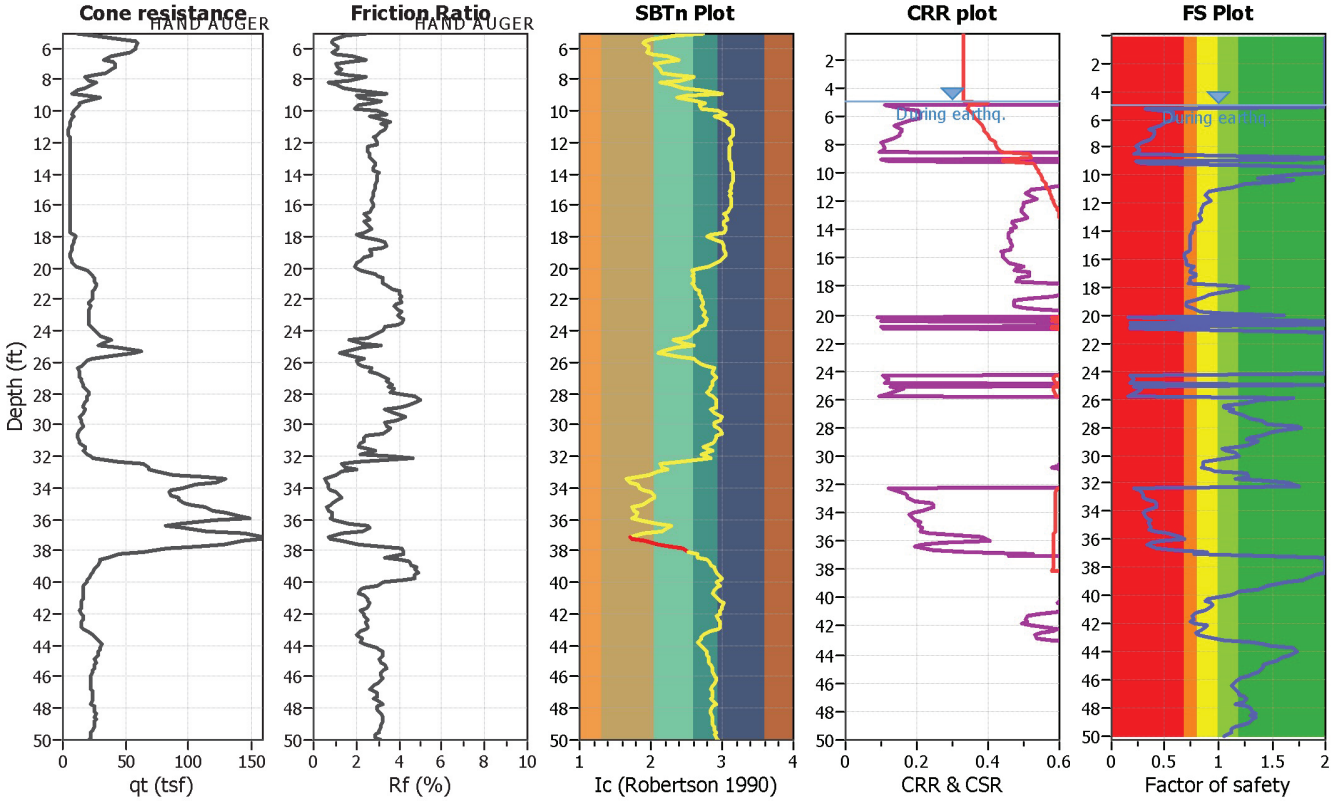
LIQUEFACTION ANALYSIS REPORT

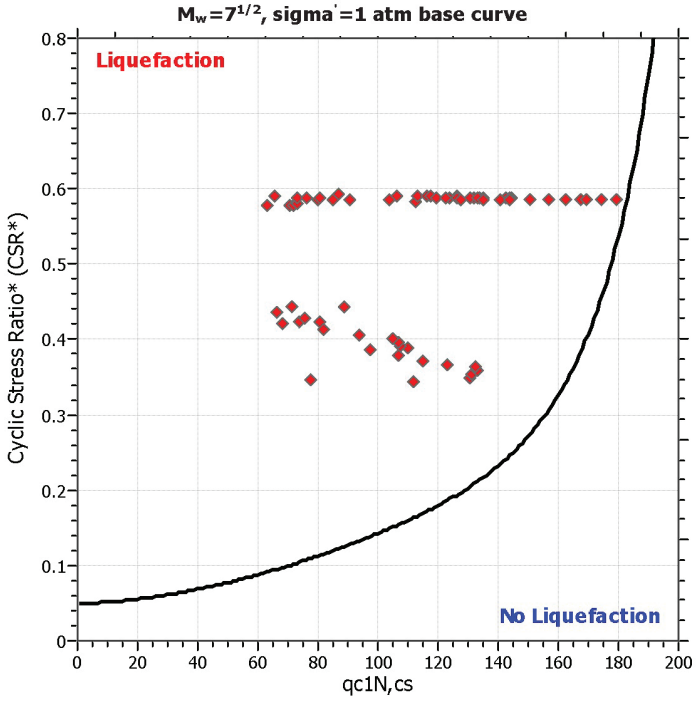
Project title : 11185.000.000 **Location : 1210 California Circle, Milpitas, CA**

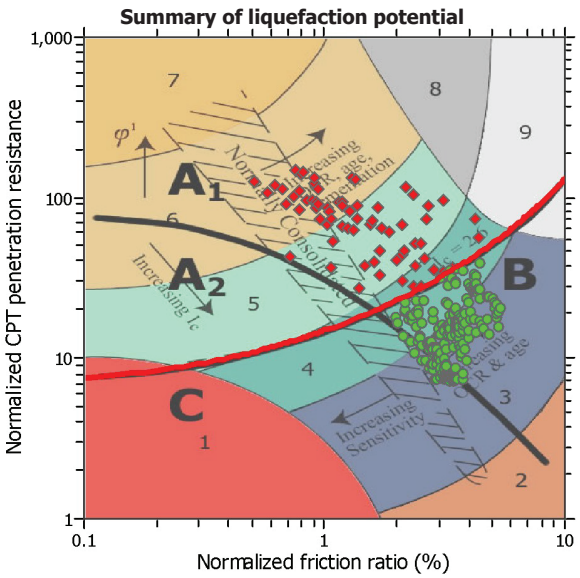
CPT file : 1-CPT5

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







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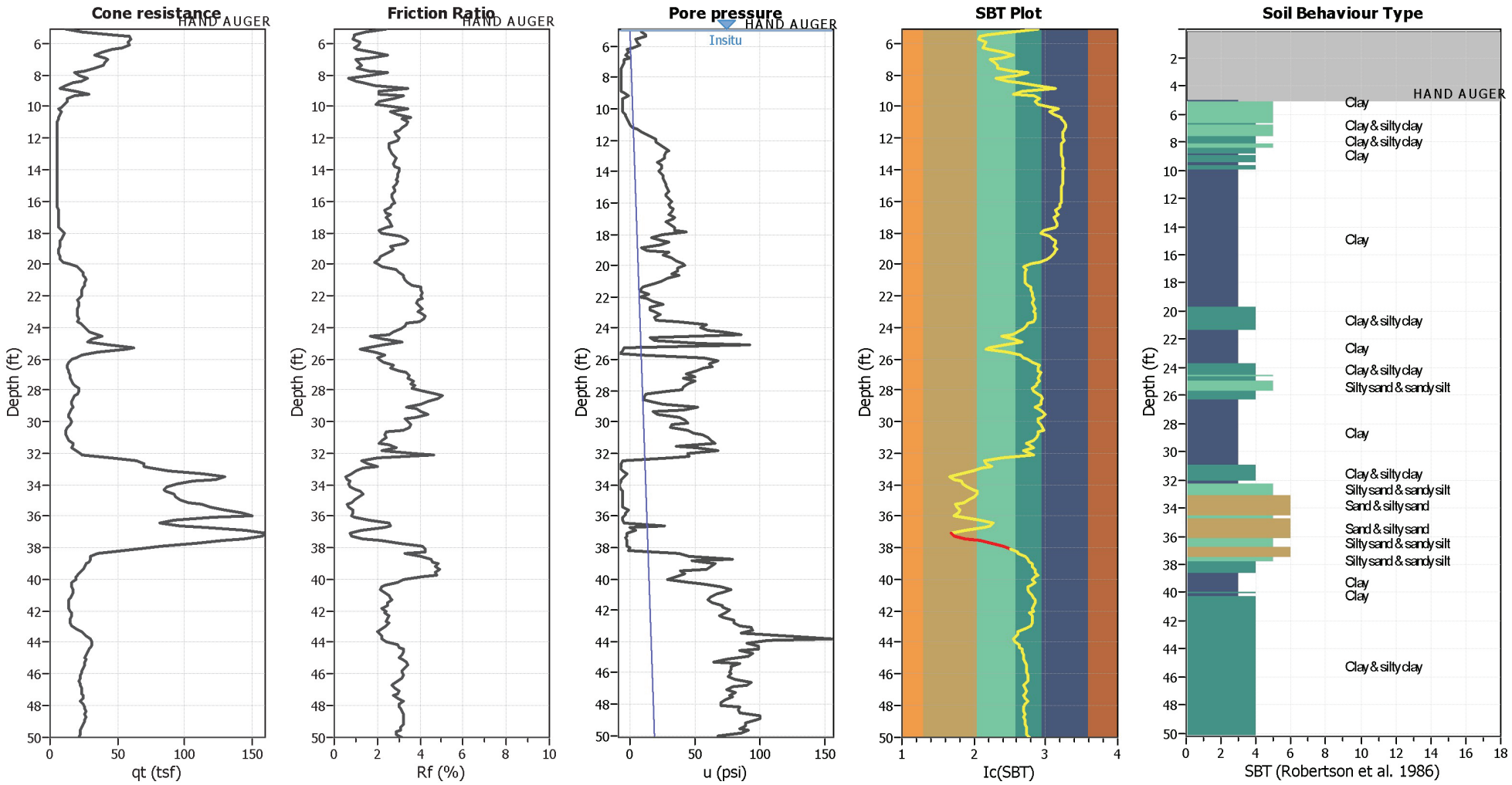
CPT basic interpretation plots



Input parameters and analysis data

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SBT legend

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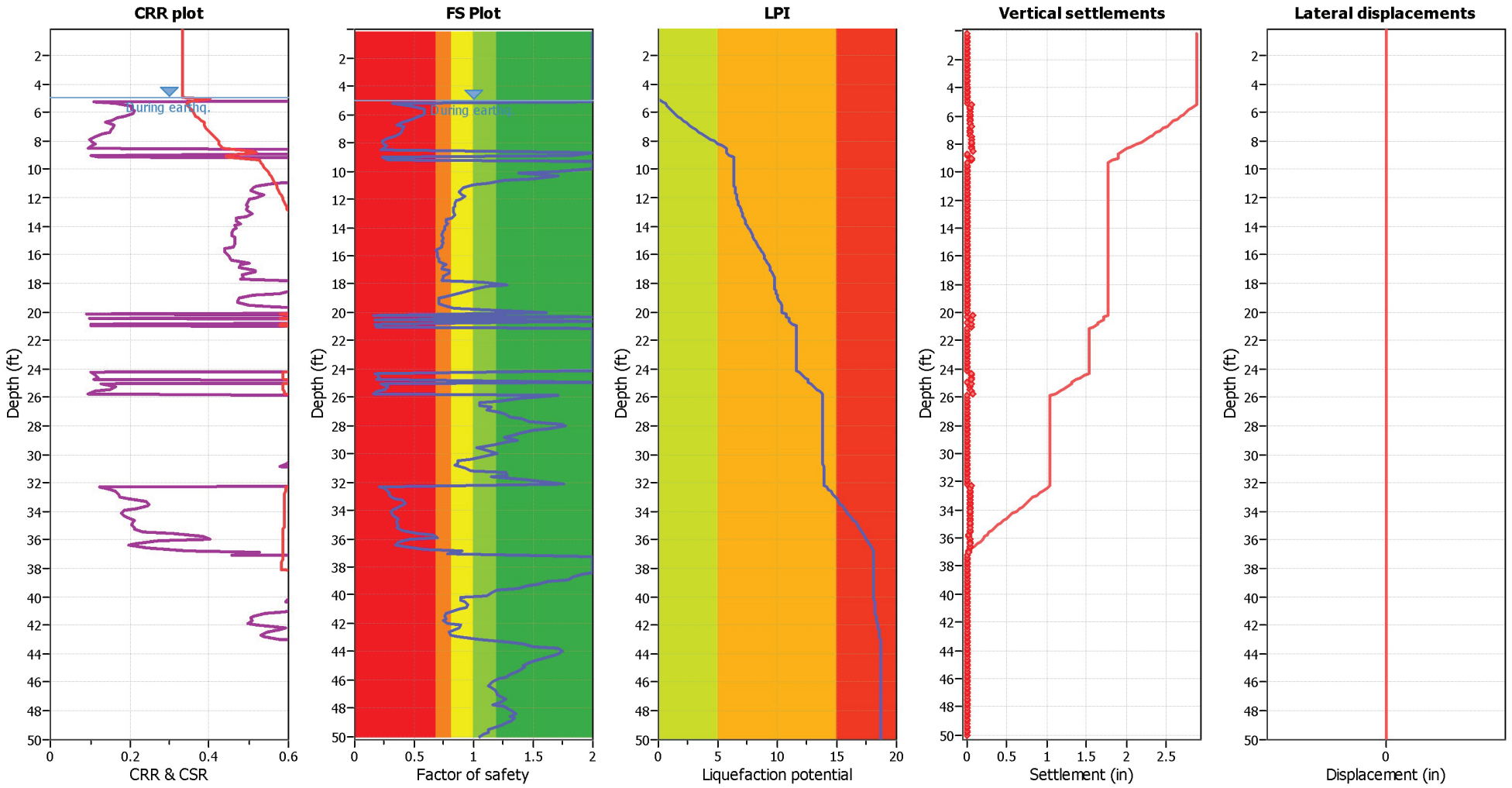
Liquefaction analysis overall plots



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LPI color scheme

- Very high risk
- High risk
- Low risk

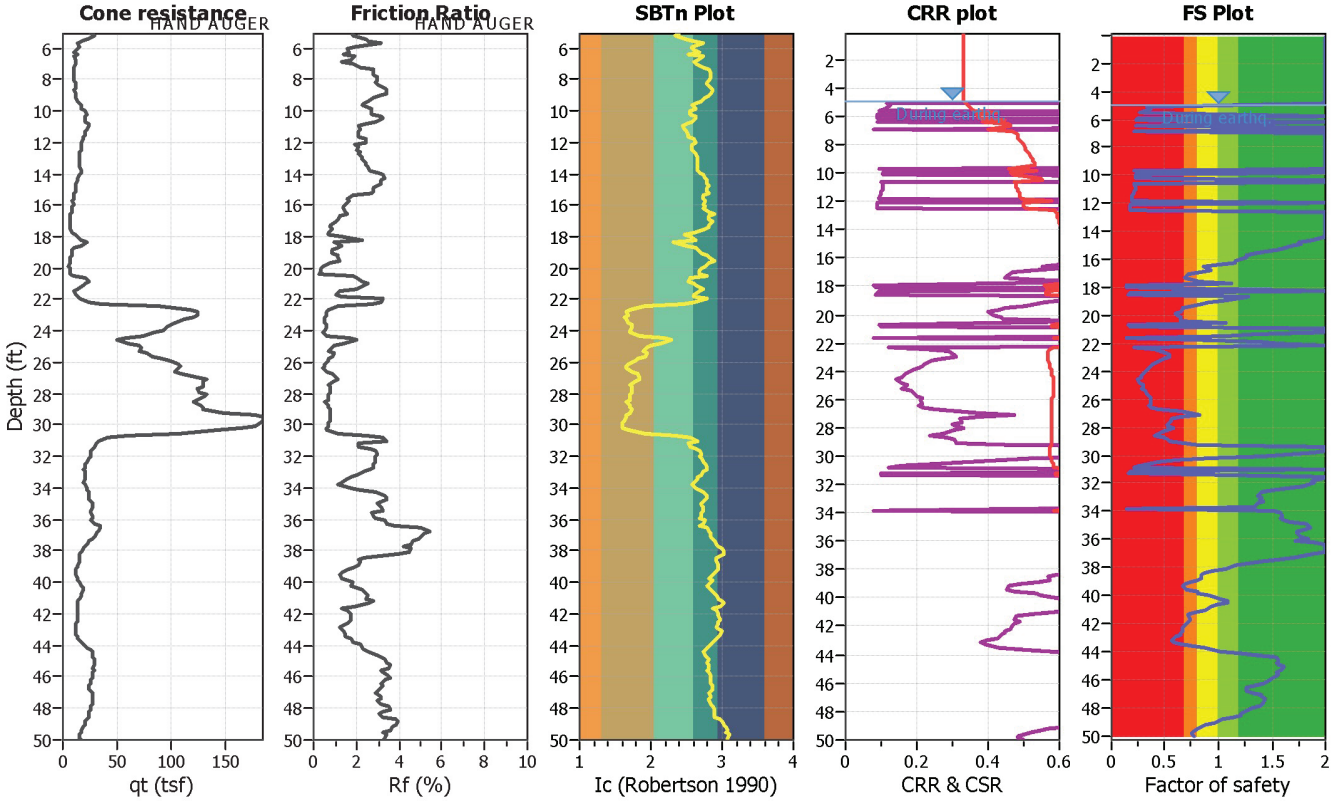
LIQUEFACTION ANALYSIS REPORT

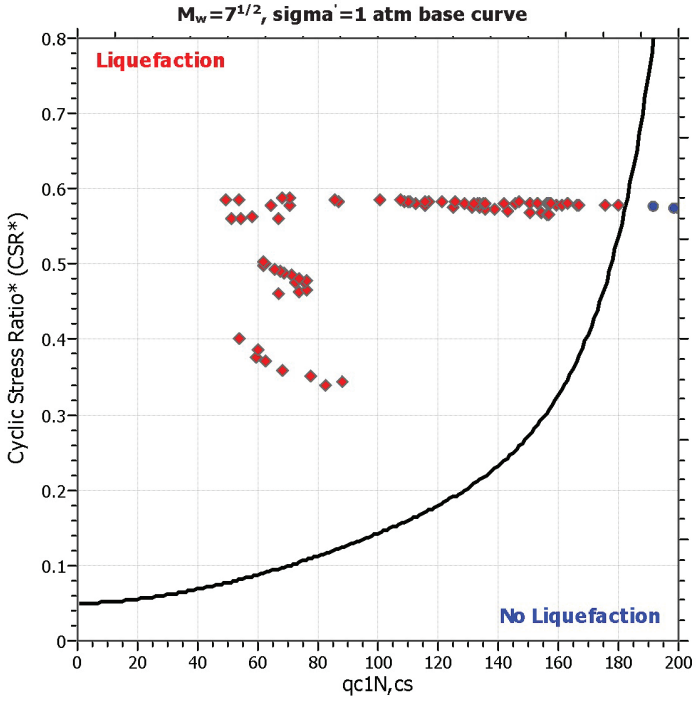
Project title : 11185.000.000 **Location : 1210 California Circle, Milpitas, CA**

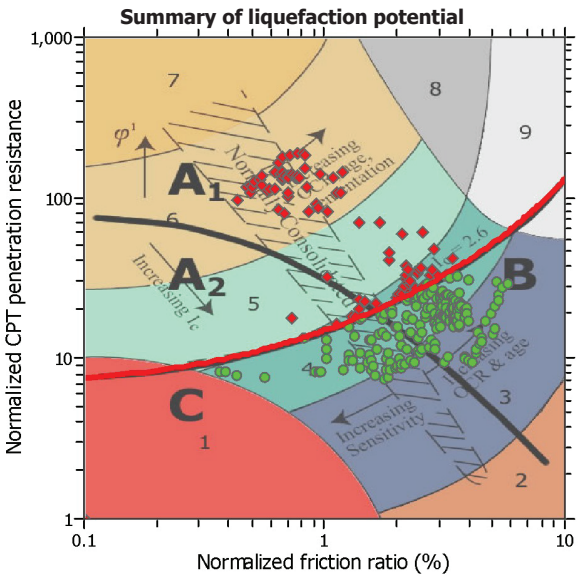
CPT file : 1-CPT6

Input parameters and analysis data

Analysis method:	I&B (2008)	G.W.T. (in-situ):	5.00 ft	Use fill:	No	Clay like behavior applied:	Sand & Clay
Fines correction method:	I&B (2008)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.80	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.68	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







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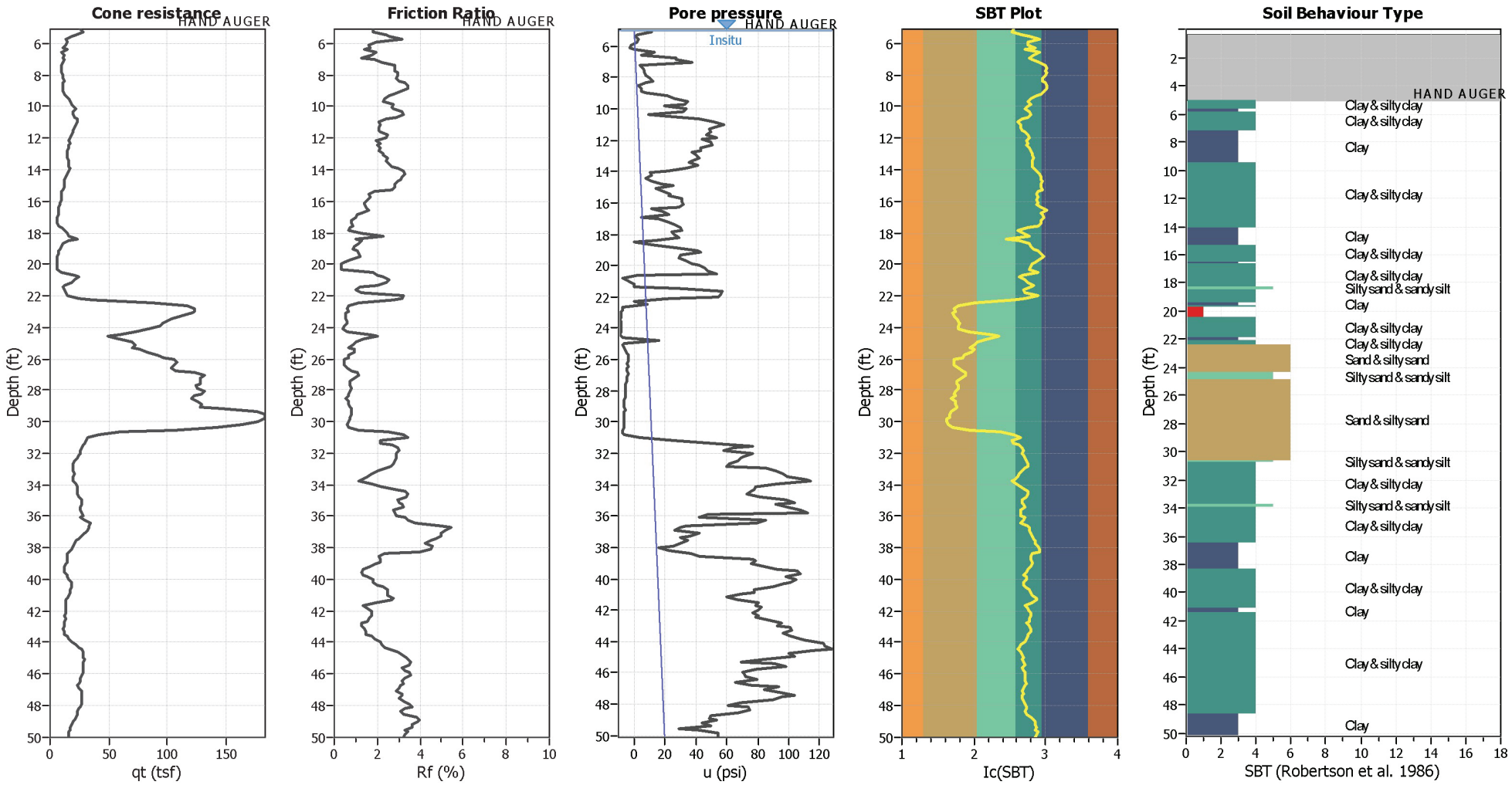

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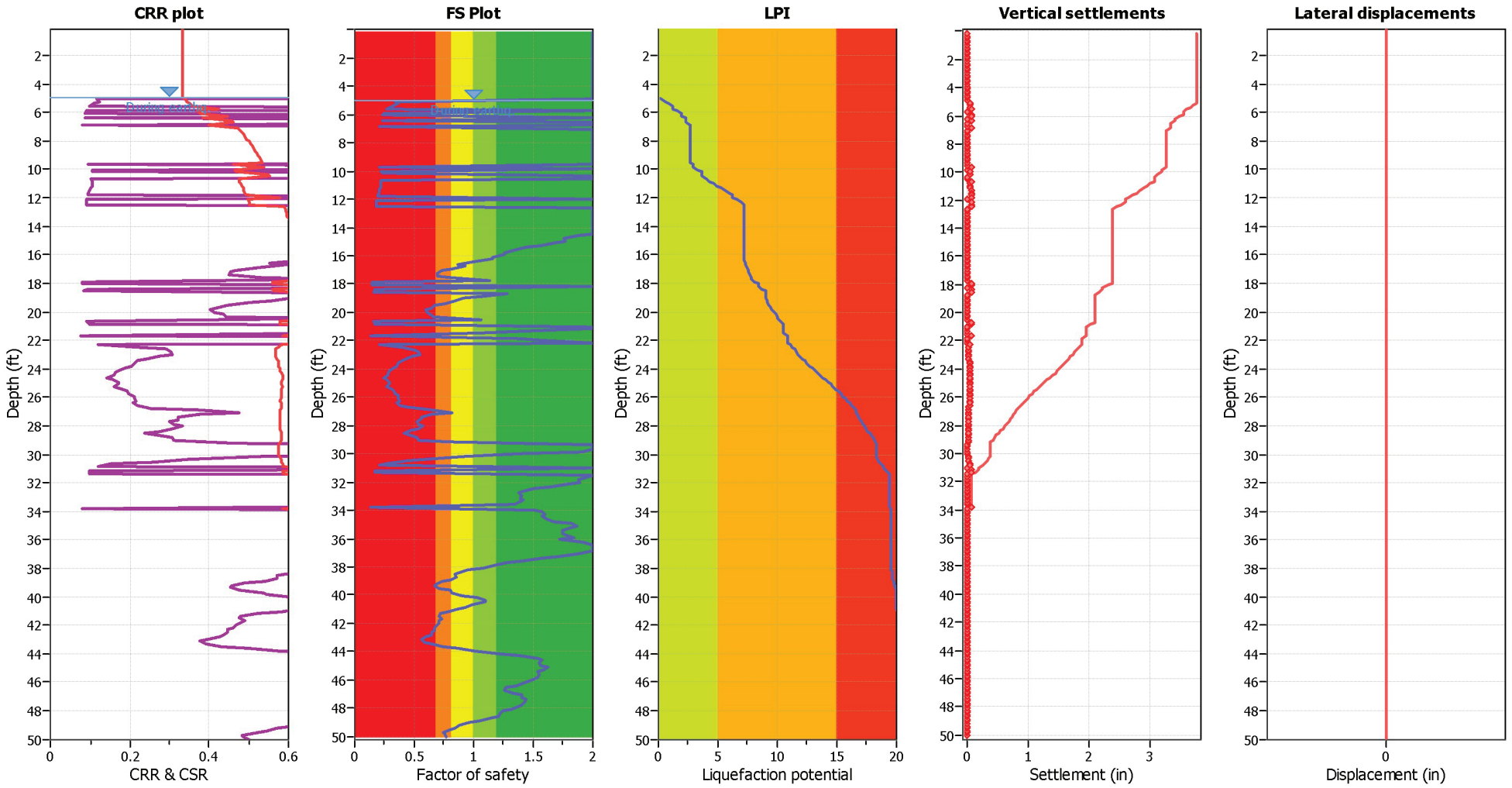
Liquefaction analysis overall plots



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LPI color scheme

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- High risk
- Low risk

DRAFT

APPENDIX E

Corrosivity Analysis
(CERCO Analytical, 2014)

California State Certified Laboratory No. 2153

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

18 July, 2014

Job No.1407078
Cust. No.11521

Ms. Jeanine Ruffoni
ENGEO Incorporated
6399 San Ignacio Avenue, Suite 150
San Jose, CA 95119

Subject: Project No.: 11105.000.000
Project Name: 1210 California Circle
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Ruffoni:

Pursuant to your request, Cerco Analytical has analyzed the soil samples submitted on July 10, 2014. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as "corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations ranged from 93 to 150 mg/kg and are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration for both samples is 37 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The sulfide ion concentrations reflect none detected with a detection limit of 50 mg/kg.

The pH of the soils ranged from 8.43 to 8.65, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials ranged from 190 to 200-mV. Samples No.001 and No.002 are indicative of potentially "moderately corrosive" soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please *call JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No. 2153

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

Client: ENGEO Incorporated
Client's Project No.: 11105.000.000
Client's Project Name: 1210 California Circle
Date Sampled: 8-Jul-14
Date Received: 10-Jul-14
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 18-Jul-2014

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)*
1407078-001	2-B5 @ 1.5'	190	8.65	-	1,200	N.D.	150	37
1407078-002	2-B7 @ 1.5'	200	8.43	-	980	N.D.	93	37

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	14-Jul-2014	14-Jul-2014	-	11-Jul-2014	15-Jul-2014	14-Jul-2014	14-Jul-2014

Cheryl McMillen
Laboratory Director

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

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

1409078 11521

CHAIN OF CUSTODY RECORD

PROJECT NUMBER: 11105.000.000	PROJECT NAME: 1210 CALIFORNIA CIRCLE
SAMPLED BY: (SIGNATURE/PRINT)	/J. RUFFONI
PROJECT MANAGER:	JEANINE RUFFONI
ROUTING: E-MAIL	JRUFFONI@ENGEO.COM Hard Copy

	SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE	Redox	pH	Sulfate	Resistivity	Chloride	Sulfide	REMARKS / REQUIRED DETECTION LIMITS
001	2-B5 @1.5'	7/8/14	-	SOIL	1	BAG	NA	X	X	X	X	X	X	ASTM w/ brief. evaluation
002	2-B7 @1.5'	7/8/14	-	↓	↓	↓	↓	X	X	X	X	X	X	

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
	7/10/14 11:55				
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	
				Std. TAT	

DISTRIBUTION: ORIGINAL ACCOMPANIES SHIPMENT; COPY TO PROJECT FIELD FILES

ENGEO INCORPORATED

6399 SAN IGNACIO AVENUE, SUITE 150
SAN JOSE, CALIFORNIA 95119
(408) 574-4900 FAX (888) 279-2698
WWW.ENGEO.COM

APPENDIX F

Supplemental Recommendations

DRAFT

APPENDIX F

SUPPLEMENTAL RECOMMENDATIONS

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1.1.2 CODES AND STANDARDS 1
1.1.3 TESTING AND OBSERVATION 1

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GENERAL INFORMATION

PREFACE

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

DEFINITIONS

Backfill	Soil, rock or soil-rock material used to fill excavations and trenches.
Drawings	Documents approved for construction which describe the work.
The Geotechnical Engineer	The project geotechnical engineering consulting firm, its employees, or its designated representatives.
Engineered Fill	Fill upon which the Geotechnical Engineer has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with geotechnical engineering recommendations.
Fill	Soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.
Imported Material	Soil and/or rock material which is brought to the site from offsite areas.
Onsite Material	Soil and/or rock material which is obtained from the site.
Optimum Moisture	Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.
Relative Compaction	The ratio, expressed as a percentage, of the in-place dry density of the fill or backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557.
Select Material	Onsite and/or imported material which is approved by the Geotechnical Engineer as a specific-purpose fill.

PART I - EARTHWORK

1.1 GENERAL

1.1.1 WORK COVERED

Supplemental recommendations for performing earthwork and grading. Activities include:

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

1.1.2 CODES AND STANDARDS

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

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

1.1.3 TESTING AND OBSERVATION

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

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

1.2 MATERIALS

1.2.1 STANDARD

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

1.2.2 ENGINEERED FILL AND BACKFILL

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

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

TABLE 1.2.2-1
Engineered Fill and Backfill Requirements

US Standard Sieve	Percentage Passing
3"	100
No. 4	35–100
No. 30	20–100

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

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

TABLE 1.2.2-2
Imported Fill Material Requirements

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

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

1.2.3 SUBDRAINS

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

1.2.3A Pipe

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

TABLE 1.2.3A-1
Perforated Pipe Requirements

Pipe Type	Standard	Typical Sizes (inches)	Pipe Stiffness (psi)
Pipe Stiffness above 200 psi (Below 50 feet of Finished Grade)			
ABS SDR 15.3		4 to 6	450
PVC Schedule 80	ASTM D1785	3 to 10	530
Pipe Stiffness between 100 psi and 150 psi (Between 15 and 50 feet of Finished Grade)			
ABS SDR 23.5	ASTM D2751	4 to 6	150
PVC SDR 23.5	ASTM D3034	4 to 6	153
PVC Schedule 40	ASTM D1785	3 to 10	135
ABS Schedule 40/DWV	ASTM D1527 & D2661	3 to 10	
Pipe Stiffness between 45 psi and 50 psi* (Between 0 to 15 feet of Finished Grade)			
PVC A-2000	ASTM F949	4 to 10	50
PVC SDR 35	ASTM D3034	4 to 8	46
ABS SDR 35	ASTM D2751	4 to 8	45
Corrugated PE	AASHTO M294 Type S	4 to 10	45

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

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

1.2.3B Outlets and Risers

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

1.2.3C Permeable Material

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

TABLE 1.2.3C-1
Class 2 Permeable Material Grading Requirements

Sieve sizes	Percentage passing
1"	100
3/4"	90 to 100
3/8"	40 to 100
No. 4	25 to 40
No. 8	18 to 33
No. 30	5 to 15
No. 50	0 to 7
No. 200	0 to 3

1.2.3D Filter Fabric

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

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

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

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

1.2.4 GEOCOMPOSITE DRAINAGE

Geocomposite drainage is a prefabricated material that includes filter fabric and plastic pipe. Filter fabric must be Class A. The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall

encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three-dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile.

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

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

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

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

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

PART II - GEOGRID SOIL REINFORCEMENT

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

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

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

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

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

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

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

PART III - GEOTEXTILE SOIL REINFORCEMENT

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

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

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

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

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

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

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

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

TABLE III-1

Geotextile Soil Reinforcements

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

PART IV - EROSION CONTROL MAT

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

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

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

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

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