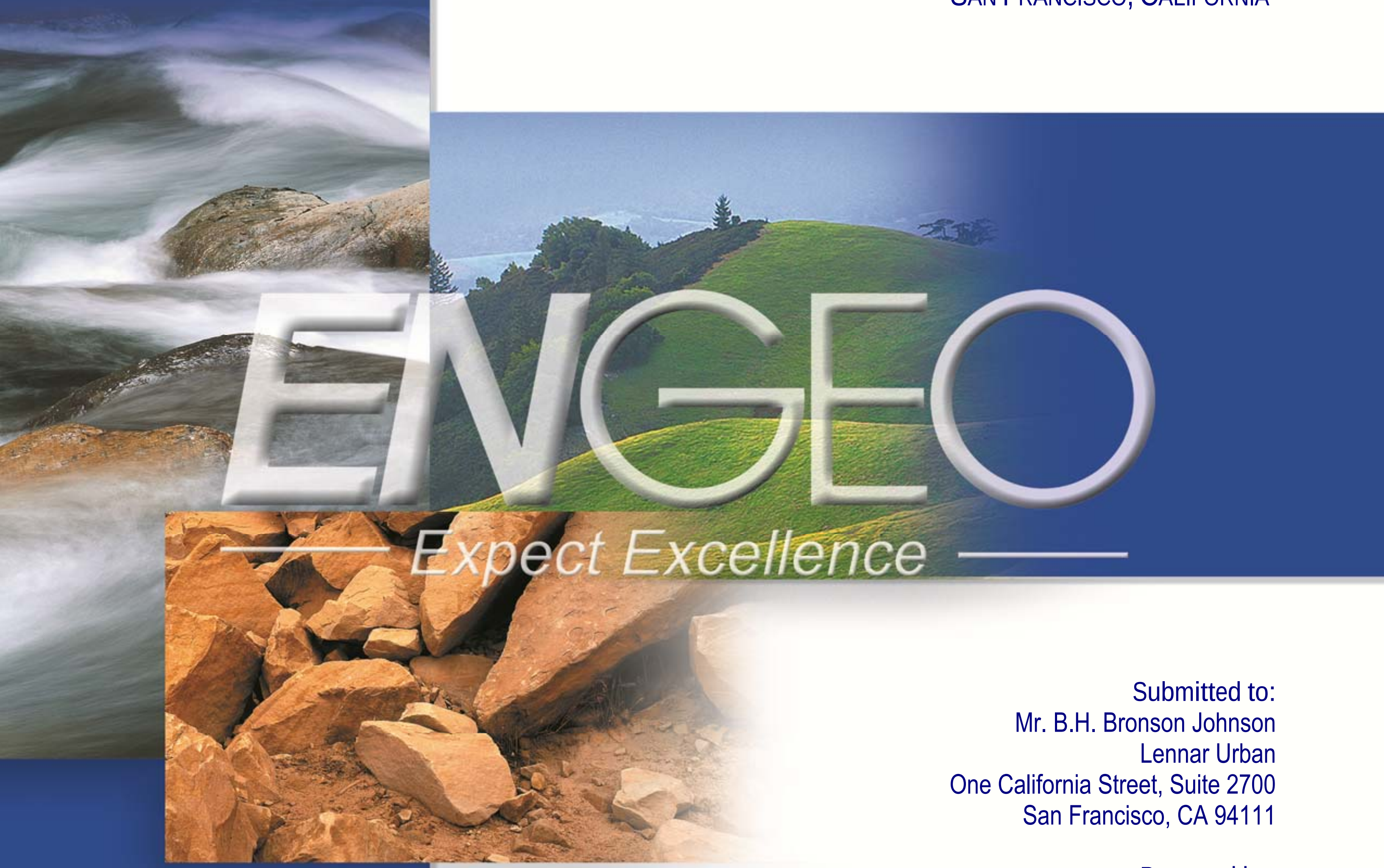
DESIGN GEOTECHNICAL EXPLORATION REPORT

ALICE GRIFFITH HOUSING REDEVELOPMENT INFRASTRUCTURE IMPROVEMENTS

SAN FRANCISCO, CALIFORNIA



Submitted to:
Mr. B.H. Bronson Johnson
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Prepared by:
ENGEO Incorporated

September 18, 2013

Project No:
8472.001.001

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September 18, 2013

Mr. B.H. Bronson Johnson
Lennar Urban
One California Street, Suite 2700
San Francisco, CA 94111

Subject: Alice Griffith Housing Redevelopment
Infrastructure Improvements
San Francisco, California

DESIGN GEOTECHNICAL EXPLORATION REPORT

Dear Mr. Johnson:

We prepared this design-level geotechnical exploration report for the Infrastructure Improvements at the Alice Griffith Housing Redevelopment project in San Francisco, California as outlined in our agreement dated October 15, 2012. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

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

Sincerely,

ENGEO Incorporated

REGISTERED PROFESSIONAL ENGINEER
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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this geotechnical report for design of the proposed infrastructure improvements at the proposed redevelopment of the Alice Griffith Housing site in the Candlestick Point neighborhood of San Francisco, California. This report addresses geotechnical aspects related to mass grading, utilities, roadways, streetscape and related secondary improvements. It should be noted that separate, subsequent reports will be provided for design of vertical improvements. Our approved scope of work included:

- Service Plan Development
- Subsurface Field Exploration
- Soil Laboratory Testing
- Data Analysis and Conclusions
- Report Preparation

For our use, we received an untitled preliminary grading plan for the development from BKF Engineers on July 24, 2013. BKF also provided us with a document titled “Grading and Storm Drain System Master Plan for the Candlestick Point Development,” dated August 22, 2013. We also received AutoCAD files showing additional grading information for the project. The plan shows grading sixteen pads as well as streets bordering the perimeters of the pads.

The assessments and recommendations contained in this report are in general compliance with the San Francisco Building Code, the Seismic Hazards Mapping Act and CGS Special Publication 117A “Guidelines for Evaluating and Mitigating Seismic Hazards in California.

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

1.2 PROJECT LOCATION

As shown in Figure 1, the site is located in the southeastern portion of the City of San Francisco in the Candlestick Point neighborhood. As shown in Figure 2, the project site covers the footprint of the existing Alice Griffith Public Housing development and portions of the parking lot to the north of the Candlestick Park stadium. The project is surrounded by Carroll Avenue on the north, Hawes Street on the west, Arelious Walker on the east, and existing housing along the northern side of Gilman Avenue on the south.

1.3 PROJECT DESCRIPTION

At the time of this report writing, the development plans for the site include several pads that will be graded relatively flat with sloping edges surrounded by streets. The proposed streets are aligned in a grid pattern that travels northeast to southwest and northwest to southeast. An open space area will be formed in the median dividing the travel directions of future Egbert Avenue roadway. While pad and street grades have not been finalized at this time, it is our understanding that the site will generally be graded with approximately 20 feet of elevation difference. The low point of the site will be on Arelious Walker south of Fitzgerald Avenue where future grades will generally conform to existing grade of approximately Elevation 103.5 feet (Project Datum = City and County of San Francisco Datum + 100 feet). The highest point will be at the future interchange of Egbert Avenue and J Street at approximately Elevation 123.5 feet Cuts of up to 30 feet are anticipated near future J Street and Donner Avenue while fills of 5 feet or less are anticipated and are generally located in the southeastern portions of the site.

It is our understanding that portions of combined sanitary sewer/storm drain system will remain. The existing combined system is located along future H Street, where an existing 2.5 by 3.75 foot box enters the site from Gilman Avenue and ties into a 54-inch diameter pipe on the future Fitzgerald Avenue. This 54-inch diameter pipe ties into an existing pipe on Arelious Walker. The combined sewer on Arelious Walker is 66 inches in diameter between Gilman Avenue and future Fitzgerald Avenue where the pipeline transitions to an 81-inch diameter pipe and outfalls into Yosemite Slough near the intersection of Arelious Walker Drive and Carrol Avenue. According to as-built plans, majority of the existing pipe is supported on piles. However, as-built plans for the box system near H Street and Fitzgerald were not available for review and could possibly be supported by the underlying soil. The dimensions and depths of the piles are not shown on the plans provided to us. Based on conversations with representatives of BKF and Carlson, Barbee and Gibson (the civil engineering designers), the existing combined sewer lines along future H Street and Fitzgerald Avenue may be relocated with a new 54-inch pipeline constructed between H Street and Arelious Walker along Gilman Avenue reconnecting with the existing line at the intersection of Arelious Walker and Gilman Avenue.

It is planned to extend Arelious Walker Drive towards the northeast to cross Yosemite Slough. A bridge approach will be constructed at the intersection of Carroll Avenue and Arelious Walker Drive. Design recommendations for the extension are beyond the scope of this report.

1.4 EXISTING GEOTECHNICAL DATA

The site and its vicinity have been investigated in the past. Subsurface explorations performed previously are shown on Figure 2. The reports associated with these previous explorations are:

- Treadwell and Rollo – A Geotechnical Investigation Report was prepared dated March 3, 1998. The investigation included drilling 16 borings, advancing 18 cone penetration tests (CPT), and conducting five seismic refraction surveys between August 5 and September 16, 1997. Some of the borings and CPTs were performed in the parking lots to the south and east of the Alice Griffith site.

- Treadwell and Rollo – A Geotechnical Report was prepared dated June 2002 for the Bayview Hope Development at 950 Gilman Avenue. The investigation included four borings drilled to the north of the intersection of Gilman Avenue and Arelious Walker.

- Olivia Chen Consultants – "Geotechnical Investigation Report, Alice Griffith Portable Building, San Francisco, California," dated March 31, 2005. The subsurface exploration included two borings drilled within the footprint of the current Opportunity Center modular building.

- ENGEO 2011 – "Geotechnical Report, Hunters Point Shipyard Phase II Candlestick Point Redevelopment, San Francisco, California," dated November 10, 2011. The subsurface exploration included seven borings, 12 CPTs, ten surface samples, and two percolation tests performed on Candlestick Park. Several of the surface samples and one boring and one CPT were performed within the Alice Griffith site.

Several of the borings, CPTs and surface samplings were performed immediately adjacent to and within the project limits.

2.0 FINDINGS

2.1 GEOLOGY AND SEISMICITY

2.1.1 Geology

A published geologic map of the site and vicinity (Figure 3; Bonilla, 1971, 1998) indicates that the northwest portion of the subject site is underlain by Pleistocene slope debris and Cretaceous and Jurassic Franciscan Complex sandstone and shale. The southeast portion of the site is underlain by artificial fills of varying thickness overlying Young Bay Mud and alluvium. According to Bonilla (1998), artificial fills mapped at the southeastern portion of the site generally comprise clay, sand, silt, rock fragments, organic matter and man-made debris. The Pleistocene slope debris and ravine fills generally comprise unstratified to poorly stratified, silty to clayey sand or gravel (Bonilla, 1998). According to Bonilla (1998), the Franciscan Complex sandstone and shale are generally interbedded, variably weathered, weak to strong and range in color depending on degree of weathering from medium dark gray to yellowish orange.

2.1.1.1 Artificial fill

The most significant areas of artificial fill (Qaf) are located south of future G Street in the southeast portion of the site along Arelious Walker. The artificial fills generally thicken toward the southeast and are as thick as 30 feet in the vicinity of Arelious Walker and Gilman Avenue near the southern boundary of the project site (Figure 4). The northern portion of the site, in the vicinity of the existing Doublerock Street (near the intersection of future Egbert Avenue and future J Street), is located in an area of bedrock cuts with relatively minor fills that were likely placed during construction of the existing development (generally less than 5 feet thick).

Subsurface explorations generally indicate that the fills are highly variable and range from lean clay to a mixture of silts, sands and gravels, with scattered debris. Based on our subsurface data and the review of the previous subsurface information referenced, the coarse grained material within the fills varies in density from very loose to medium dense, and fine grained materials are typically stiff to very stiff. As shown in Figure 5, the artificial fill in the east portion of the project limit is mapped in a Liquefaction Seismic Hazard Zone by the State of California Geologic Survey. The USGS map is intended to be used for baseline studies since it is based on correlation between liquefaction potential and geologic units applied across a region mapped with a scale 1:24,000 to 1:200,000. This map is acknowledged to be limited and detailed liquefaction potential evaluation with geotechnical borings and site-specific studies by a licensed professional are necessary.

2.1.1.2 Young Bay Mud

The portion of the site located southeast of future G Street is beyond the former shoreline as mapped in 1903 and shown in aerial photographs from 1938. This area of the site is underlain by compressible Young Bay Mud as thick as15 feet beneath the fill. The Young Bay Mud thickness generally increases away from the former shoreline.

The Young Bay Mud is normally consolidated to slightly overconsolidated. Post-construction settlement as a result of consolidation of Young Bay Mud subjected to construction loading and new loads from fill or structures may have long-term detrimental effects on the planned infrastructure within the project area. Further discussion of the effects of this soft/compressible soil and possible mitigation measures are provided in this report.

2.1.1.3 Alluvial Soil

The Young Bay Mud is typically underlain by interbedded stiff clay, medium dense to dense sand, silty sand, and gravel layers. Based on subsurface explorations conducted, the alluvial soil extends to approximately 45 feet to 50 feet below the ground surface.

2.1.1.4 Slope Debris/Colluvial Soil

As described above, slope debris was deposited along the edges of the Franciscan bedrock within the area of the site to the east of the 1903 shoreline portion of the site. Our interpretation of the CPTs data advanced in this soil (2-CP-CPT3, 2-CP-CPT6, and 2-CP-CPT8) is the slope debris is comprised of interbedded layers of stiff clay and dense sand.

2.1.1.5 Old Bay Mud

Borings and CPTs that penetrated through the alluvial soil encountered stiff to hard clay locally referred to as Old Bay Mud or Yerba Buena Mud. The Old Bay Mud is similar in material composition but is much stiffer and significantly less compressible compared to the Young Bay Mud.

2.1.1.6 Bedrock

The site is underlain by Jurassic- and Cretaceous-age Franciscan bedrock that generally comprises interbedded graywacke and shale. The weak to very strong bedrock varies from yellowish brown to dark gray in color. Bedrock structure is somewhat chaotic with bedding, fractures and foliations in various directions. According to mapping by Bonilla (1998), the predominant trend of bedding is generally west-northwest striking and north dipping. Dips range from 35 to 45 degrees as shown on Figure 3. Boring CP-B7 encountered bedrock at a depth of 54 feet below ground surface while several of the CPTs also encountered bedrock ranging from 5 feet to 50 feet below ground surface across the site. The bedrock encountered in boring CP-B7 consisted of weak, highly weathered, greywacke. Areas of exposed bedrock outcrop on the slopes near Doublerock Street. A seismic refraction survey was also performed in this area and shows that bedrock is relatively shallow.

2.1.2 Seismicity

Numerous small earthquakes occur every year in the San Francisco Bay Region and larger earthquakes have been recorded and can be expected to occur in the future. Figure 6 shows the approximate locations of these faults and significant historic earthquakes recorded within the Greater Bay Area Region. The most common nearby active faults within 30 miles of the site and their estimated maximum earthquake magnitudes are provided in the following table based on United States Geologic Survey (USGS) 2008 National Seismic Hazard Maps. 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).

TABLE 2.1.2-1
Regional Faults

Fault Name	Approximate Distance (miles)	Direction from Site	Estimate of Maximum Magnitude (Ellsworth)
N. San Andreas	6.1	West	7.9
San Gregorio	10.7	West	7.5
Hayward-Rodgers Creek	12.0	East	7.3
Monte Vista-Shannon	20.9	Southeast	6.5
Calaveras	21.7	East	7.0
Mount Diablo Thrust	21.9	East	6.7

Fault Name	Approximate Distance (miles)	Direction from Site	Estimate of Maximum Magnitude (Ellsworth)
Green Valley	25.0	Northeast	6.8
Point Reyes	28.9	Northwest	6.9

Site: Latitude = 37.7136; Longitude = -122.3861

The United States Geologic Survey evaluated the Bay Area seismicity through a study by the Working Group on California Earthquake Probabilities (WGCEP, 2007). WGCEP estimated that there is a 21 percent probability that a moment magnitude (M_w) of 6.7 or greater earthquake will occur on the San Andreas fault within 30 years of the publish date (2007 – 2037). WGCEP estimated there is a 31 percent probability that a moment magnitude (M_w) of 6.7 or greater earthquake will occur on the Hayward Fault within the same time period. The aggregate probability of a similarly sized earthquake in the San Francisco Bay Area was estimated to be 63 percent in the study.

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

2.2 SURFACE CONDITIONS

During our field exploration, we performed a brief site reconnaissance and observed the following site features:

- The majority of the site is occupied by residential buildings, open spaces, and streets associated with the current Alice Griffith Housing Development.

- The housing development portion of the site is currently terraced with buildings that step down from approximately Elevation 148 feet along Doublerock Street down to approximately Elevation 103 feet at the current intersection of Fitzgerald Street and Griffith Street.

- The eastern side of the property is currently being used as parking lots for events held at Candlestick Park Stadium. The area is surfaced with raveled and cracked asphalt concrete in the northern parking lot and has a gravel surface in the southern portion.

- The site has an approximately 20 foot high slope between the residential buildings along Doublerock Street and Nichols Way. This slope is as steep as 2:1 (horizontal:vertical). A second slope, approximately 15 feet high is between the buildings on the south side of Nichols Street and the north side of Griffith Street; this slope is also as steep as 2:1.

Please refer to the Site Plan, Figure 2A, for more information on site features.

2.3 FIELD EXPLORATION

We performed supplemental subsurface exploration within the footprint of the future development, as shown on Figure 2A. Our supplemental exploration consisted of six CPTs, one boring, and two seismic refraction survey tests.

2.3.1 Field Exploration

Boring 2-CP-B3 was drilled concurrently with fieldwork performed at the existing Candlestick Park site on March 12, 2013. The boring was drilled using rotary-wash methods to a depth of approximately 54 feet. The boring was terminated in very stiff clay. Boring 2-CP-B2 was drilled on August 23, 2013 along with borings drilled in support of the first phase of the proposed vertical development. This boring was also drilled with rotary-wash methods and was terminated at a depth of 36 feet in weak rock. During drilling, samples were collected by driving either a 2.5-inch outside-diameter (OD) Standard Penetration Test (SPT) sampler or a 3-inch OD California-type split-spoon sampler fitted with 6-inch-long brass liners; relatively undisturbed samples were collected by pushing a 3-inch diameter, thin-walled “Shelby Tube” sampler. The SPT and California-type samplers were driven with a 140-pound automatic trip hammer falling a distance of 30 inches. The penetration of the sampler was field recorded as the number of blows needed to drive the sampler 18 inches in 6-inch increments. The boring log shows the number of blows required for the last one foot of penetration, and the blow counts have not been converted using any correction factors.

The borings were logged in the field by an ENGEO representative. The field log was then used to develop the report borelogs in Appendix A. The logs depict subsurface conditions encountered within the borings for the date of drilling; however, subsurface conditions may vary with time.

2.3.2 Cone Penetration Tests

Six CPT soundings were advanced at the site on May 28, 2013 to a maximum depth of approximately 80 feet below existing grade. Prior to advancing CPT soundings, the surface pavement materials were cored and the upper 5 feet of soil was excavated by hand auger. The CPTs are indicated as 2-CP-CPT1 through 7; 2-CP-CPT2 was not performed due to time constraints and will be supplemented by a boring during future exploration at the project.

The CPT equipment has a 20-ton compression-type cone with a 10-square-centimeter (cm^2) base area and a friction sleeve with a surface area of 150 cm^2. The cone, connected with a series of rods, is pushed into the ground at a constant rate of 2 cm per second. Cone readings are taken at approximately 2-cm intervals. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). The CPT data was provided by Gregg Drilling and is attached as Appendix A.

2.3.3 Seismic Refraction Survey Tests

The two seismic refraction survey tests, designated Line 2 and Line 3, were approximately 200 feet long and were performed along the slopes between Doublerock Street and Nichols Way; Line 1 was performed at the Candlestick Park site.

The seismic refraction line was performed by NorCal Geophysical Consultants, Inc.; the subconsultant's report is included in Appendix C.

2.4 LABORATORY TESTING

Select samples recovered during drilling activities were tested to determine the following soil characteristics:

TABLE 2.4-1
Laboratory Testing

Soil Characteristic	Testing Method	Location of Results
Natural Unit Weight and Moisture Content	ASTM D-2216	Appendix A
Grain Size Distribution	ASTM D 422	Appendix B
Incremental Consolidation	ASTM D-2435	Appendix B
Laboratory Mini Vane Shear	ASTM D-4648	Appendix B
Atterberg Limits	ASTM D-4318	Appendix B
Triaxial Compression – Undrained, Unconsolidated (TXUU)	ASTM D-2850	Appendix B

The laboratory test results are shown on the boring logs (Appendix A) with individual test results presented in Appendix B.

2.5 SUBSURFACE CONDITIONS

Based on information obtained to date, the subsurface conditions can be subdivided into two conditions. The conditions to the bay side and land side of the historic shoreline (Shown on Figure 2A) are dramatically different. The bedrock was observed at the ground surface or encountered at relatively shallow depths in our explorations performed to the west of the 1903 mapped shoreline. Where our CPTs were advanced within the mapped slope debris, the soil was generally relatively stiff clay and dense sand over bedrock that ranged in depth from 35 to 50 feet below existing grade. Our explorations to the east of the 1903 shoreline generally encountered variable thicknesses of fill over soft Young Bay Mud over dense alluvium and stiff Old Bay Mud. Boring CP-B7 was the only exploration in the vicinity of the project and on the east side of the historic shoreline that encountered bedrock within the depth explored; bedrock was at a depth of 54 feet below existing grade.

2.6 GROUNDWATER CONDITIONS

The groundwater level in boring CP-B7 conducted during the current scope was not observed due to the drilling methods employed.. Groundwater was encountered in some of the previous exploration borings within and adjacent to the project site. The following table summarizes reported groundwater elevations (The approximate locations of each boring are shown on Figures 2A and 2B).

TABLE 2.6-1
Groundwater Observations

Boring Location	Approximate Elevation of Groundwater (Feet)*
CP-B5	-8.5
DB-1	-5
DB-14	-5
DB-15	-10
DB-16	-10

**Elevation datum based on City and County of San Francisco (CCSF)*

It should be recognized that fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, sea level and other factors not evident at the time measurements were made.

Based on Historic Highest Ground Water Contours presented in the Seismic Hazard Zone Report for the City and County of San Francisco prepared by the California Geologic Survey (CGS), groundwater level at the site is recorded at approximately 10 feet below ground surface, which is equivalent to Elevation -5 feet (CCSF Datum) or approximately Elevation 95 feet (Project Datum). We recommend a groundwater level of Elevation 95 feet be used for design purposes.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications.

The primary geotechnical concerns that could affect development on the site are shallow rock excavatability, liquefaction potential of undocumented fill, and consolidation of Bay Mud.

3.1 SHALLOW ROCK EXCAVATION AND SUITABILITY

The following is provided for informational purposes. A grading contractor should perform their own assessment of appropriate equipment during the bid process.

The northern portion of the site, near the intersections of future J Street at Donner Avenue and Egbert Avenue and future H Street, is underlain by shallow Franciscan bedrock. This portion of the site is where the largest proposed cuts will be required with mass-grading cuts up to 35 feet below existing grade. While grading in the streets will not be as deep as within the pads, excavations for sanitary sewer trenches could be as deep as 10 feet below future road grade, which can extend to 35 feet below existing grade at the deepest. Based on our seismic refraction testing performed in the vicinity of these large excavations, the measured shear wave velocity of the upper 25 feet of bedrock is approximately 8,000 feet per second (fps) or less but increases to approximately 10,000 fps in Line 2 from 25 to 35 feet in depth. We anticipate that cuts to proposed grade in this area of the site may require a D11 bulldozer. Cuts deeper than 25 feet should be expected to be marginally rippable using a single tooth ripper shank, and non-rippable boulders and other resistant particles should be expected resulting in overexcavation and reconstruction of the cut slopes. However, localized massive hard rock should be expected, which may be difficult or impractical to rip in-place. It is also possible that limited areas may require hydraulic hammers or controlled blasting to perform the excavation; the use of controlled blasting would be subject to the City and County of San Francisco approval. If blasting is considered to be a viable option, consideration needs to be given to dust control and vibration monitoring. Alternative methods of excavation in hard rock include expansion grouting, hydraulic loading, controlled foam injection, and specialized mechanical fracturing methods.

If excavation of resistant rock results in oversize rock fragments beyond the size acceptable for engineered fill placement. It is likely that the oversize rock fragments will need to be processed with specialized equipment. Material that cannot be broken down to less than 12 inches in diameter may need to be removed from the site or selectively placed at the bottom of deeper fills on other portions of the project as approved by the Geotechnical Engineer; in general, the planned fill at this site is not deep enough to accept large diameter rocks; and in the areas where planned fill is thick enough to accept larger sized particles, the fill is within roadways and large particles placed in mass fill would likely cause issues with utility excavation.

Similarly, we anticipate that it will be possible to trench most of the bedrock using large excavator-type equipment. In the areas where utilities will be deepest below existing grade and in localized areas where lenses of massive hard rock are encountered, excavations will require laborious trenching efforts and may necessitate the use of excavators equipped with single-tooth ripping hooks or hydraulic hammers. Trenching of localized hard rock is likely to result in over-break of trench walls and oversized trench spoils. Depending on the phasing of construction, it may be preferable to overexcavate bedrock in areas of proposed trenching during grading when more effective and powerful equipment is available. In addition, consideration should be given to performing overexcavation within streets concurrently with utility construction to reduce the need for additional trenching and shoring typically associated with utility installation.

To assist in utility and foundation construction, we recommend that the upper 5 feet of the future pads where rock is encountered be excavated, processed and placed as engineered fill during site grading. Pads where overexcavation is recommended based on estimated depth to bedrock in relation to the grading shown on the current plans are shown on Figure 7. Once building layouts have been determined, the depth of overexcavation may need to be modified so that the differential thickness of fill across a building pad is no greater than 15 feet. This maximum differential fill thickness is important to limit differential settlement across a building foundation. While not part of the scope of the infrastructure planning, we recommend performing the remedial pad grading at the time of rough pad grading to improve efficiency and eliminate the risk of undermining infrastructure previously constructed.

3.2 EXISTING FILL

As previously mentioned, the portions of the site to the east of the 1903 shoreline are underlain by fill that is between 15 and 30 feet thick.. The explorations indicate that much of the fill is loose and includes wood, construction debris and other deleterious and over-sized material indicating it was not placed in an engineered manner. Non-engineered fills can undergo excessive settlement, especially under new fill or building loads. Additionally, based on our analyses, we estimate that a significant amount of this existing fill is subject to potential deformation due to seismic loading. Due to the depth of the fill, shallow groundwater, existing utilities, buildings and other improvements, we do not consider it feasible to remove non-engineered fill to develop the site. We recommend that the upper 5 feet of subgrade soil be excavated, processed to remove and oversized or deleterious material and recompacted as engineered fill to assist in providing a competent subgrade and enhance pavement performance.

The contractor should anticipate that oversized material may be encountered during underground construction. Trenches may also encounter areas where loose fill results in localized trench stability issues requiring sloping trench walls or using trench shields.

3.3 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, and ground lurching. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, soil liquefaction, lateral spreading, landslides, tsunamis, flooding or seiches is considered low to negligible at the site.

3.3.1 Ground Rupture

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

3.3.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the current California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.3.3 Liquefaction

We evaluated the susceptibility of the on-site fill material to liquefaction of sands, as well as the susceptibility of gravels and low plasticity fines to seismic loading based on methodology presented by Bray and Sancio (2006), Idriss and Boulanger, (2008) and Seed et al, (2003). The literature suggests that "transitional" material like silts and clays may be susceptible to seismic softening. As a result, we have divided our evaluation of liquefaction into soils exhibiting "sand-like" behavior and those exhibiting "clay-like" behavior. We have assessed the seismic susceptibility and deformation potential at the site based on material properties from laboratory testing and in-situ CPT data as discussed in the following sections. Our analyses of liquefaction potential were performed using a PGA of 0.59 and a M_w 7.9 earthquake; the PGA value is based on the 2013 California Building Code, which becomes effective on January 1, 2014, and the earthquake magnitude is associated with an earthquake on the San Andreas Fault.

3.3.3.1 Analyses of Clay-Like Material.

We have evaluated the susceptibility of fine-grained soil within the fill in the area east of the historic shoreline using the methods described in Bray and Sancio (2006) and criteria proposed by Idriss and Boulanger (2008). These methods evaluate the potential for cyclic softening based on the in-situ moisture content of the soil and properties determined during plasticity index testing. Based on this lab testing, the fine-grained soil that we sampled and tested within the fill in the project area is not expected to be susceptible to cyclic softening (a phenomenon similar to liquefaction where soil loses strength as a result of cyclic loading).

3.3.3.2 Analyses of Sand-Like Material

Our empirically-based analyses of the subsurface data collected from CPTs and borings are described in the following sections.

CPT Data

We evaluated liquefaction resistance and related settlement based on the CPT data collected for this report and from data collected for our 2011 report previously referenced. The analysis was performed in accordance with procedures developed by Robertson (2009) using the computer software Cliq. The software employs methodology discussed by Youd et al, (2001) Moss et. al. (2006), and Robertson (2009) to assess the liquefaction of sandy materials as well as cyclic softening of clay. The software uses methodology by Zhang (2004) to estimate post-liquefaction displacement.

To assess liquefaction hazard, we have calculated both the Factor of Safety and the Liquefaction Potential Index (LPI) for each CPT, as defined by Iwasaki (1982). LPI is a relative hazard index, calculated on a point-by-point basis using the factor of safety against liquefaction, as a function of depth. LPI has been correlated to observed damage in existing liquefaction case studies and is a more appropriate indicator of risk than factor of safety alone. The computed discrete factors of safety with depth, from which the LPI values for each CPT are derived, are summarized on the attached Cliq Output in Appendix E.

Deformation of the ground surface is a common result of liquefaction. Vertical settlement may result from densification of the deposit or volumetric loss from venting to the ground surface. We calculated potential liquefaction induced settlement estimates from the CPT data using the program CLiq. The estimates of potential liquefaction-induced settlement are included in Appendix E.

At the Candlestick Point site, our previous analyses indicated that potential volumetric settlement of the in-situ fill is highly variable. Based on the methodology described above, our calculations estimate settlement within the in-situ fill ranges from less than 1 inch to over 11 inches at the various explorations previously performed in the fill throughout Candlestick Point. Analysis of the CPTs performed for this scope of work and three nearby CPTs previously performed (CP-CPT6, -CPT9, and -CPT11) indicate settlement from liquefaction of 4½ inches or less assuming no mitigation is performed. This is in part because the thickness of fill in the project area is thinner than other areas of Candlestick Point.

Boring Data

We also performed an analysis of the potential liquefaction at the site based on boring 2-CP-B3 performed for this scope of work as well as borings CP-B5 and CP-B7 previously performed within or immediately adjacent to the project limits. We collected SPT blow counts and samples of the subsurface soil and used the laboratory test results and recorded blow counts to assess liquefaction using analytical methods published by Youd et al. (2001), Seed et al. (2003), and Idriss & Boulanger (2008). Recorded blow count resistances (N-value) were corrected for sampler and hammer type, overburden pressure, boring diameter, and fines content. We calculated potential settlement based on the results of our boring data using methods published by Ishihara and Yoshimine in 1992. The results of our liquefaction analyses performed on data from the borings are included in Appendix E. Each of these borings encountered soil within the fill that our analyses indicate could be liquefiable. Based on the thicknesses of potentially liquefiable soil in these three borings, we calculated settlement ranging from between 2 inches to 7 inches.

3.3.3.3 Shallow Soil Liquefaction

As discussed by Youd and Garris (1995), liquefiable soil that is not overlain by a sufficiently thick layer of soil that is not liquefiable is more prone to ground surface disruptions such as fissures and sand boils. The thickness of non-liquefiable soil necessary to reduce this risk is a function of the thickness of the liquefiable soil layer below. At this site, due to the chaotic nature of the fill, it is possible that in limited areas, the thickness of non-liquefiable soil will not be sufficient to eliminate ground surface disruptions. Without mitigation of liquefaction, we anticipate the potential for isolated areas of sand boils or ground cracking. These effects could result in limited areas of pavement buckling, utility breaks or settlement greater than the amounts discussed in Section 3.3.3.2 above.

3.3.3.4 Summary of Liquefaction Analyses

In summary, based on our interpretation of the existing data, we estimate that the areas underlain by artificial fill placed during reclamation of the site may experience from 2 to 7 inches of seismically induced settlement. The average settlement is anticipated to be 4 inches or less and the average differential settlement is anticipated to be approximately 2 inches over a distance of 50 feet. Additionally, it is possible that in limited areas, the existing fill could result in the development of sand boils or ground cracking. In general, the surface improvements can be expected to tolerate the average of total and differential settlement. The limited areas where excessive settlement or ground surface disruptions may occur will be difficult to predict.

Mitigation of the effects of liquefaction may be accomplished through:

- Removal and recompaction of the fill,

- In-situ densification through deep dynamic compaction or rapid impact compaction or similar technologies,

- Vibro-compaction, stone columns, rammed aggregate piers, or

- Deep soil mixing or grout mixing.

Due to shallow groundwater, the depth to the bottom of the fill and adjacent existing utilities and structures, complete removal of all potentially liquefiable material in all improvement areas is likely infeasible for this project. In-situ densification can be achieved by impact at the ground surface in order to densify the potentially liquefiable soil reducing the potential for liquefaction. These methods result in varying amounts of noise and vibration depending on the weight used and the frequency of the impact. A method such as rapid impact compaction (RIC) may be appropriate in some areas of planned improvements though, in areas where existing utilities will remain, RIC will be impractical within a horizontal distance of at least 10 feet from the utility or damage may occur. Other mitigation methods listed above are significantly more expensive and may not be a cost effective means of reducing liquefaction along the proposed roadway and utility construction though they could be performed adjacent and, in some cases, beneath existing utilities.

Due to the degree of anticipated settlement and difficulty in implementing the potential mitigation measures, it may be preferable to forego the above recommended liquefaction-specific mitigation measures within areas of planned improvements. The degree of anticipated settlement discussed above should not result in a significant reduction in performance of utilities and roadways after the design seismic event. However, performance could be enhanced by increasing gradient or oversizing gravity utilities, where possible, to account for differential settlements as much as 2 inches over 50 feet. Consideration should be given to providing flexible connections along utilities. If this approach is implemented, it is possible that larger amounts of settlement and ground surface disruption will only occur in isolated areas. Additionally, the recommended 5 foot removal and recompaction of fill in improvement areas will further reduce the risk of damage to surface improvements from liquefaction related deformation. In isolated areas where excessive settlement or ground surface disruptions occur, utility and roadway repair will be necessary after an earthquake large enough to cause liquefaction to occur.

3.3.4 Lateral Spreading

We previously identified a risk of lateral slope deformation along the nearby shoreline of Candlestick Point in our November 10, 2011 Geotechnical Report for the Hunters Point Phase II and Candlestick Point Redevelopment. This deformation is not expected to impact the project area though mitigation could impact utilities or roadways that are extended from this project to within 150 feet of the shoreline.

3.3.5 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Bay Area region, but based on the site location, it is our opinion that the offset is expected to be nominal.

3.4 CONSOLIDATION SETTLEMENT OF YOUNG BAY MUD

Based on our review of published maps and the existing information, the portion of the site east of the 1903 shoreline is underlain by natural soft, highly compressible Young Bay Mud deposits. Young Bay Mud deposits are of particular concern since these deposits are highly compressible and may be susceptible to significant settlement when subjected to additional loading, either through the placement of additional fill and/or additional structural loads. In addition, these deposits have low strength characteristics and may be problematic for underground construction due to their instability in temporary cuts and graded slopes. In general, these materials are not considered suitable for reuse as engineered fill and will necessitate mitigation as discussed in following sections of this report.

The estimated Elevation of the bottom of the Young Bay Mud is shown on Figure 4. In general, the Young Bay Mud within the project site is beneath at least 15 feet of fill and is typically between 10 and 15 feet in thickness. Based on the conceptual grading plan, the maximum amount of fill is planned near the intersection of the future Fitzgerald Avenue and H Street; approximately 4 feet of fill is planned in this area.

In general, consolidation settlement due to new fill placement over existing Young Bay Mud is anticipated to be approximately ½ foot or less with approximately 90 percent of the settlement occurring in the first year after placement. In order to reduce the effects of settlement, additional fill (surcharge) can be placed in areas to receive civil fill and removed once the settlement under the planned loading has been achieved. The time required for surcharging would be a function of the amount of surcharge used. Surcharging would be expected to take a year or less.

If time is not available in the construction schedule or where existing utilities would be damaged by surcharge and settlement, lightweight fill can be used to raise the site without adding new loads thus reducing settlement to nominal amounts. The most cost-effective lightweight fill currently available is cellular concrete. Typically, the unit weight for cellular concrete is around 30 pounds per square foot. For construction above the water table, where 4 feet of fill is planned over Young Bay Mud, the weight of the new fill could be compensated by removing approximately the upper 1½ feet of existing fill and constructing to subgrade using cellular concrete.

Lightweight fill may also be a preferred means to raise grade along Arelious Walker and other streets with existing utilities that will remain in service. The lightweight fill can be used, in combination with existing fill removal, to raise the site grade without increasing the net effective stresses in the soil and minimize or eliminate settlement from new fill loads. Figure 7 shows areas where surcharging or lightweight fill would be appropriate.

Even with proper surcharging, some amount of long-term aerial settlement from secondary compression of the Young Bay Mud should be anticipated. The magnitude of this residual settlement will be dependent on the amount of fill placed, thickness of Young Bay Mud, and time allowed for surcharging. In general, this secondary settlement will be approximately 10 percent of the primary settlement (less than 1 inch).

Due to the anticipated loading condition and construction type of the planned structures at this project, we expect that buildings over fill and Young Bay Mud will be supported on deep foundations deriving support from deeper soil stratum with relatively less settlement compared to utilities. Utility connections to the buildings should have flexible connections to allow for the potential post-construction site settlement from compressible soil and liquefaction. These connections should allow for at least 6 inches of differential settlement between the site and building. Foundation design criteria will be provided in site specific design reports.

3.5 SLOPE CONSTRUCTION

In general, slopes are anticipated to be 15 feet or less based on the preliminary grading plans previously referenced. For slopes less than 15 feet in height, the maximum slope gradient should be 2:1 or flatter. We should be consulted to provide modified recommendations if permanent slopes higher than 15 feet are planned.

3.6 CORROSIVITY CONSIDERATIONS

An evaluation of possible corrosion impacts to study area improvements has not been conducted. We recommend that chemical tests be conducted on soils where the proposed utilities will be located and within building pads. The proximity to the Bay and corrosion considerations from the marine environment should be considered in the corrosion mitigation for the design of the project. It is common practice in the Bay Area to design improvements in Bay Mud for moderate sulfate conditions, consistent with seawater. As such, we recommend that you consider the subsurface conditions as exposure class S1, in accordance with the criteria presented in Table 19-A-4 of the 2007 CBC.

3.7 NATURALLY OCCURRING ASBESTOS

Some ultramafic rock, such as serpentinite and potentially greenstone, contains the fibrous mineral chrysotile, which is considered an asbestos mineral. Based on laboratory testing performed as part of our 2009 exploration (Appendix B), trace amounts of naturally occurring asbestos (NOA) were detected in soil and rock collected as surface samples at the project site. One of the four samples collected within the project limits encountered chrysolite. Based on our research, the fill east of the 1903 shoreline within the project site was predominantly derived from local bedrock and has the potential to contain NOA in varying amounts. Cuts into the native bedrock, likewise, could encounter NOA. Asbestos is considered hazardous when it becomes airborne. Additional testing from the current exploration is pending and will be issued as an addendum to this report when ready.

It is our opinion that the project will be required to follow the rules and regulations outlined in the Asbestos Airborne Toxic Control Measure (ATCM) for Construction, Grading, Quarrying and Surface Mining Operations established by the Bay Area Air Quality Management District (District) under California Code of Regulations, Title 17, Section 93015. The purpose of this regulation is to reduce public exposure to NOA from construction and mining activities that emit dust, which may contain NOA. The ATCM requires regulated operations engaged in road construction and maintenance activities, construction and grading operations, and quarrying and surface mining operations in areas where NOA is likely to be found, to employ the best available dust mitigation measures in order to reduce and control dust emissions.

As part of compliance with the ATCM, an Asbestos Dust Mitigation Plan (ADMP) should be prepared by a qualified representative for approval by the BAAQMD and for inclusion in the contract documents.

4.0 CONSTRUCTION MONITORING

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

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

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

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

5.0 RECOMMENDATIONS

5.1 EARTHWORK

The recommendations regarding relative compaction and optimum moisture content of soil referred to in this report are based on the most recent ASTM D1557 test method. Compacted soil is not acceptable if it is unstable. It should exhibit only minimal *flexing* or *pumping,* as determined by an ENGEO representative.

As used in this report, the term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry. We define “structural areas” as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, and pavement areas.

It is important that all construction activities be performed under the observation of the Geotechnical Engineer’s field representative, in accordance with the recommendations contained herein.

5.2 GENERAL SITE CLEARING

The contractor should clear areas to be developed of all surface and subsurface deleterious materials including existing building foundations, slabs, buried utility and irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots. The contractor should clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in this section. All backfill should be observed and tested by a representative of the Geotechnical Engineer. Foundations for the stadium will need to be completely removed and properly backfill. If deep foundations are present, the upper 10 feet of the foundation should be removed and the location surveyed for consideration in planning new building foundations.

Following clearing, strip the site to remove surface organic materials. Strip organics from the ground surface to a depth of at least 2 to 3 inches below the surface. Remove strippings from the site or, if considered suitable by the landscape architect and owner, use them in landscape fill.

If undocumented fill is encountered during site clearing, all existing fill should be removed to competent native soil, as determined by a representative of the Geotechnical Engineer.

5.3 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. As indicated above, a design groundwater at a Elevation of 95 feet (Project Datum) should be considered; overly wet conditions should be anticipated for excavations that extend below these elevations.

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

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

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

5.4 ACCEPTABLE FILL

Onsite soil is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 4 inches in maximum dimension.

Imported fill materials should be approved by the Geotechnical Engineer, meet the above requirements and have a plasticity index less than 12. Allow ENGEO to sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

5.5 FILL COMPACTION

5.5.1 Grading in Structural Areas

The contractor should perform subgrade compaction prior to fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 8 inches;

2. Moisture condition soil to at least 2 percentage points over the optimum moisture content; **and**

3. Compact the soil to at least 90 percent relative compaction. Compact the upper 6-inches of finish pavement subgrade to at least 95 percent relative compaction prior to aggregate base placement.

After the subgrade has been compacted, the contractor should place and compact acceptable fill (defined in Section 6.3) as follows:

1. Spread fill in loose lifts that do not exceed 8 inches;

2. Moisture condition lifts to at least 2 percentage points over the optimum moisture content; **and**

3. Compact fill to at least 90 percent relative compaction; compact the upper 6 inches of fill in pavement areas to at least 95 percent relative compaction prior to aggregate base placement.

The Contractor should compact the pavement Caltrans Class 2 Aggregate Base section to at least 95 percent relative compaction (ASTM D1557). The aggregate base should be moisture conditioned to or slightly above the optimum moisture content prior to compaction.

5.5.2 Underground Utility Backfill

In structural areas, the contractor should place and compact trench backfill as follows:

1. Trench backfill should have a maximum particle size of 4 inches;

2. Moisture condition trench backfill to at least 2 percentage points above the optimum moisture content. Moisture condition backfill outside the trench;

3. Place fill in loose lifts not exceeding 12 inches; and

4. Compact fill to at least 90 percent relative compaction.

Jetting of backfill is not an acceptable means of compaction.

5.5.2.1 Excavation And Shoring Considerations

Due to the shallow groundwater table conditions and heterogeneity of the artificial fills on the east portion of the site, we anticipate shoring will be required for underground improvements construction. Temporary cut slopes may be considered for utilities proposed inland of the 1903 shoreline. The design of shoring systems and temporary cut slopes is the sole responsibility of the Contractor and/or the shoring designer. Excavations should be performed in conformance with applicable OSHA Excavation and Trench Safety Standards.

Where excavations extend below the groundwater, the use of braced, interlocked sheetpile shoring systems with toe embedment is suitable, in our opinion. Shoring systems should be designed by a qualified registered engineer. The pressure distribution for shoring design can be provided once improvement plans are available.

Excavated soils, construction materials or other items imposing a surcharge should be stockpiled at least 20 feet away from the edge of excavations to reduce potential adverse effect on slope or trench stability. We recommend that no vertical trench excavations be left open overnight without adequate shoring. Once shoring has been removed, the contractor should backfill the excavation to within 5 feet of the ground surface before the end of the day.

5.5.2.2 Temporary Dewatering

It is anticipated that groundwater will be encountered in excavations extending below Elevation 95 feet (Project Datum). Groundwater management and potential treatment prior to discharge will be required for the groundwater encountered.

The groundwater level at the trench locations should be maintained at a minimum of 2 feet below the bottom of the trenches for the duration of utility installation. The selection of equipment and method should be determined by the contractor. The dewatering system implemented should be selected so as to impose minimal impact on the groundwater level surrounding the proposed excavations. The dewatering system should be designed to prevent pumping soil fines with the discharge water. Uncontrolled dewatering could cause settlement of the general area.

Moist to saturated subgrade conditions should be anticipated at the bottom of the utility trench in areas underlain by fill and Bay Mud. The contractor may consider stabilizing the bottom of the utility trench with stabilization fabric such as Mirafi 600X or geogrid such as BX1200 or TX160 overlain by at least 18 inches of ¾- to 1½-inch crushed rock. Other approaches may be acceptable and ENGEO should be consulted if alternative approaches are desired.

5.5.3 Landscape Fill

In landscaping areas, the contractor should process, place and compact fill in accordance with Sections 5.5, except the compaction requirement is reduced to a minimum of 85 percent relative compaction.

6.0 FOUNDATION RECOMMENDATIONS

Depending on building types and foundation loads, it is likely that shallow foundations can be used to support buildings inland of the historic shoreline. The use of shallow foundations outside of the historic shoreline is likely infeasible due to the presence of Young Bay Mud and liquefiable soil since infrastructure will be constructed prior to building construction. Therefore, buildings outside the historic shoreline will likely be constructed on deep foundations such as driven piles. Site-specific foundation recommendations will be prepared once building layouts, types and loading have been determined and provided to us for review. We understand that prior to the issuance of building permits, the vertical developer will provide confirmation of HUD compliance to the City of San Francisco Department of Building Inspection for the replacement of the Alice Griffith Housing site.

7.0 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum section of 5 inches of concrete over 4 inches of aggregate base. The contractor should:

1. Compact the aggregate base to at least 90 percent relative compaction (ASTM D1557).

2. Thicken flatwork edges to at least 8 inches to help control moisture variations in the subgrade and place wire mesh or rebar within the middle third of the slab to help control the width and offset of cracks.

3. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

8.0 PAVEMENT DESIGN

8.1 PAVEMENTS

The City of San Francisco standard pavement design consists of a 2-inch-thick wearing course of HMA (hot mix asphalt concrete) constructed over a minimum 6-inch-thick Portland cement concrete (PCC) "base" underlain by a compacted soil subgrade. Due to the soil types underlaying this site, and potential settlement from soft soil compression and liquefaction and thermal shrinking and expanding of the PCC base, we anticipate that the City's standard pavement design will evidence cracks in the pavement surface early in the pavement design life. We recommend increasing the thickness of the HMA wearing course to 4 inches or increasing the HMA thickness to 3 inches and placing a "fabric" such as Owens-Corning-Trumbull's TruPave, or GlasGrid by Tensar in the middle of the HMA layer to retard surface cracking.

As an alternative to the City of San Francisco standard minimum pavement design provided above, we provide flexible pavement designs for various traffic indices using an R-value of 35 for the granular fill based on Section 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety). As discussed in our April 5, 2013, letter titled "Hunters Point Shipyard Phase II and Candlestick Point," we opine that a flexible pavement system may perform better than the City of San Francisco's standard pavement design in areas underlain by existing fill and Young Bay Mud.

TABLE 8.1-1
Recommended Asphalt Concrete Pavement Sections

	Section	
Traffic Index	**Asphalt Concrete (inches)**	**Class 2 Aggregate Base (inches)**
5	3	5
6	3.5	7
7	4	9

Notes: AC is asphalt concrete
AB is Class 2 aggregate base material with a minimum R-value of 78

The Traffic Index should be determined by the Civil Engineer or appropriate public agency. 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.

8.2 SUBGRADE COMPACTION

Subgrade soil within the upper 36 inches of finished roadway surface extending from back of curb to back of curb within in roadway areas should be compacted to at least 95 percent relative compaction prior to placement of PCC. Moisture condition subgrade soils to or slightly above the optimum moisture content prior to compaction.

8.3 AGGREGATE BASE COMPACTION (FLEXIBLE PAVEMENT ALTERNATIVE)

If Flexible Pavements are used, the contractor should compact the Caltrans Class 2 aggregate base to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to or slightly above the optimum moisture content prior to compaction. Aggregate Base should meet the requirements for ¾-inch maximum Caltrans Class 2 aggregate base per the latest Caltrans Standard Specifications.

If desired, pavement cutoff barriers should be considered where pavement areas lie downslope of any landscape areas that are to be irrigated, and should extend to a depth of at least 4 inches below the base of the PCC. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1 for the Alice Griffith Housing Redevelopment 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 us 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, then notify the proper regulatory officials immediately.

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

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to our documents. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If our 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.

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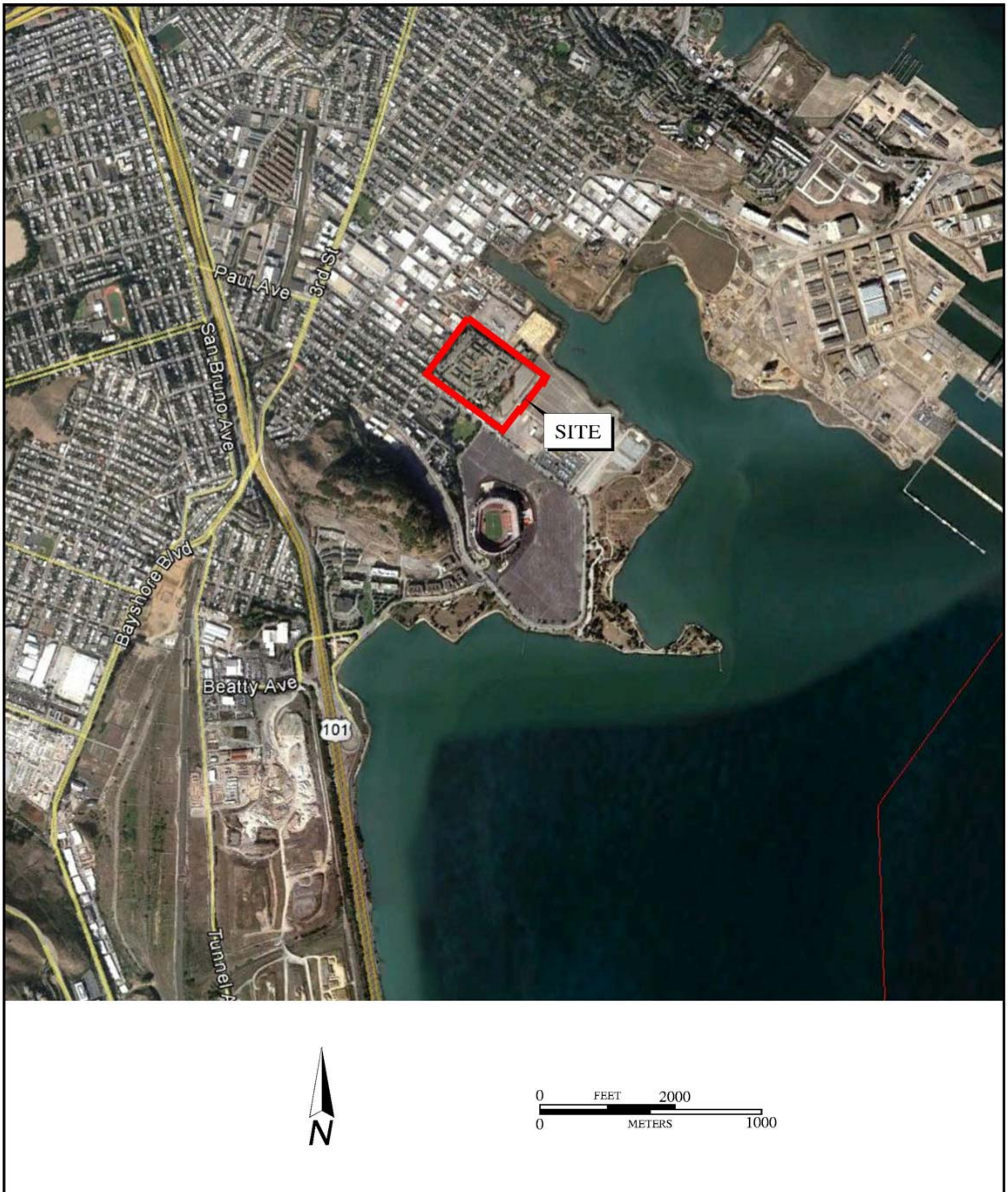
FIGURES

F I G U R E S



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BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO — Expect Excellence	VICINITY MAP ALICE GRIFFITH SAN FRANCISCO, CALIFORNIA	PROJECT NO.: 8472.001.001 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: LC	FIGURE NO. 1

G:\Drafting\DRAFTING2_Dwg\8472\001\GEX\Alice Griffith\8472001001-GEX-1-VicMap-0613.dwg Plot Date: 9-17-13 llee

ORIGINAL FIGURE PRINTED IN COLOR

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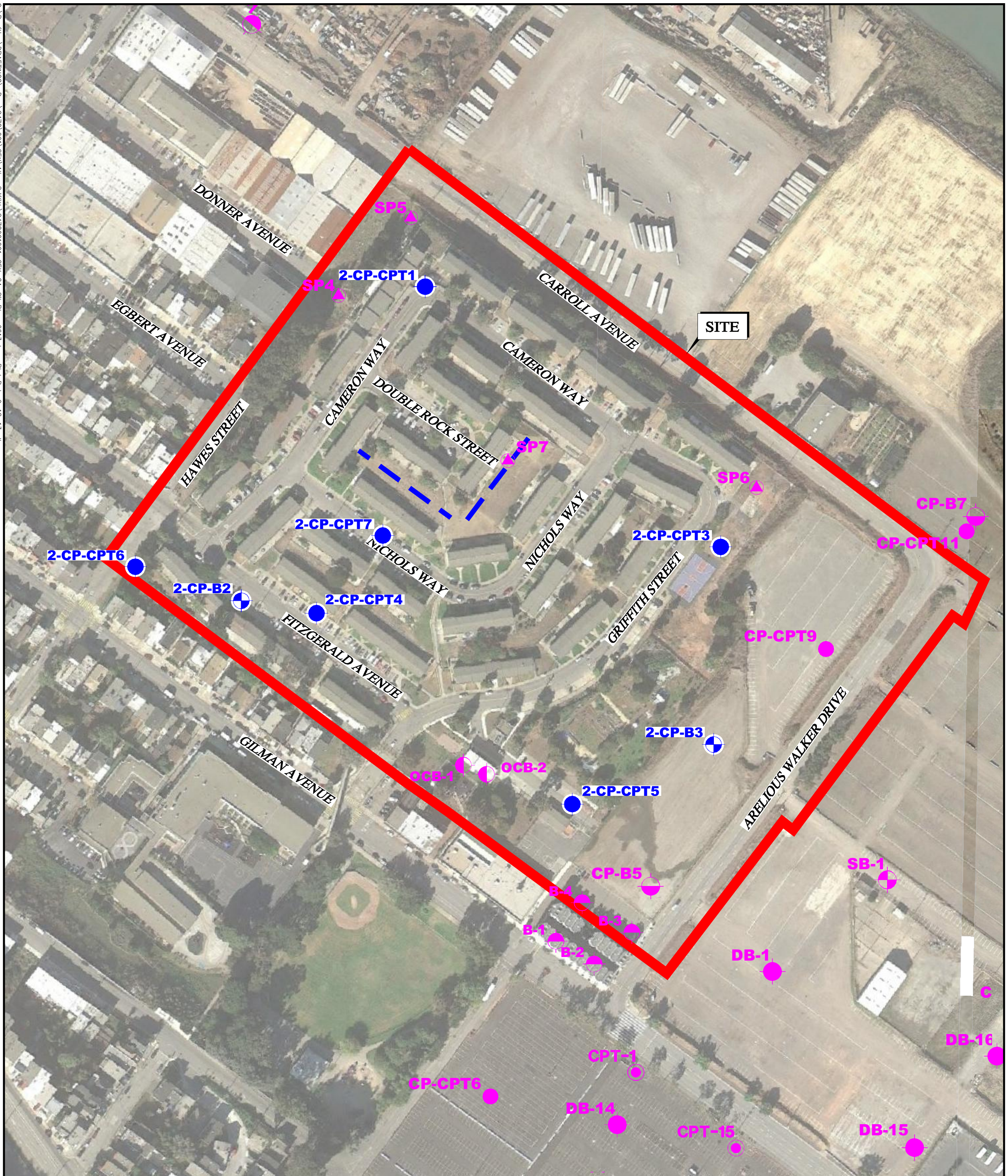

EXPLANATION

APPROXIMATE LOCATION OF SEISMIC SURVEY LINE

2-CP-CPT7 APPROXIMATE LOCATION OF CONE PENETRATION TEST (PUSH PROBE WITH NO SPOILS) (ENGEO)

2-CP-B3 APPROXIMATE LOCATION OF BORING (ENGEO)

CP-B7 APPROXIMATE LOCATION OF PREVIOUS EXPLORATORY BORING (ENGEO)

CP-CPT11 APPROXIMATE LOCATION OF PREVIOUS CONE PENETRATION TEST (ENGEO)

SP10 APPROXIMATE LOCATION OF PREVIOUS SURFACE SAMPLING (ENGEO)

P2 APPROXIMATE LOCATION OF PREVIOUS PERCOLATION TEST (ENGEO)

CPT-18 APPROXIMATE LOCATION OF PREVIOUS CONE PENETRATION TEST (TREADWELL & ROLLO)

DB-16 APPROXIMATE LOCATION OF PREVIOUS BORING (TREADWELL & ROLLO)

B-4 APPROXIMATE LOCATION OF PREVIOUS BORING (TREADWELL & ROLLO)

OCB-1 APPROXIMATE LOCATION OF PREVIOUS BORING (OLIVIA CHEN)

N

0 FEET 200
0 METERS 100

BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO
Expect Excellence

EXISTING SITE PLAN ALICE GRIFFITH SAN FRANCISCO, CALIFORNIA	PROJECT NO.: 8472.001.001		FIGURE NO.
	SCALE: AS SHOWN		2A
	DRAWN BY: LL	CHECKED BY: LC	

ORIGINAL FIGURE PRINTED IN COLOR

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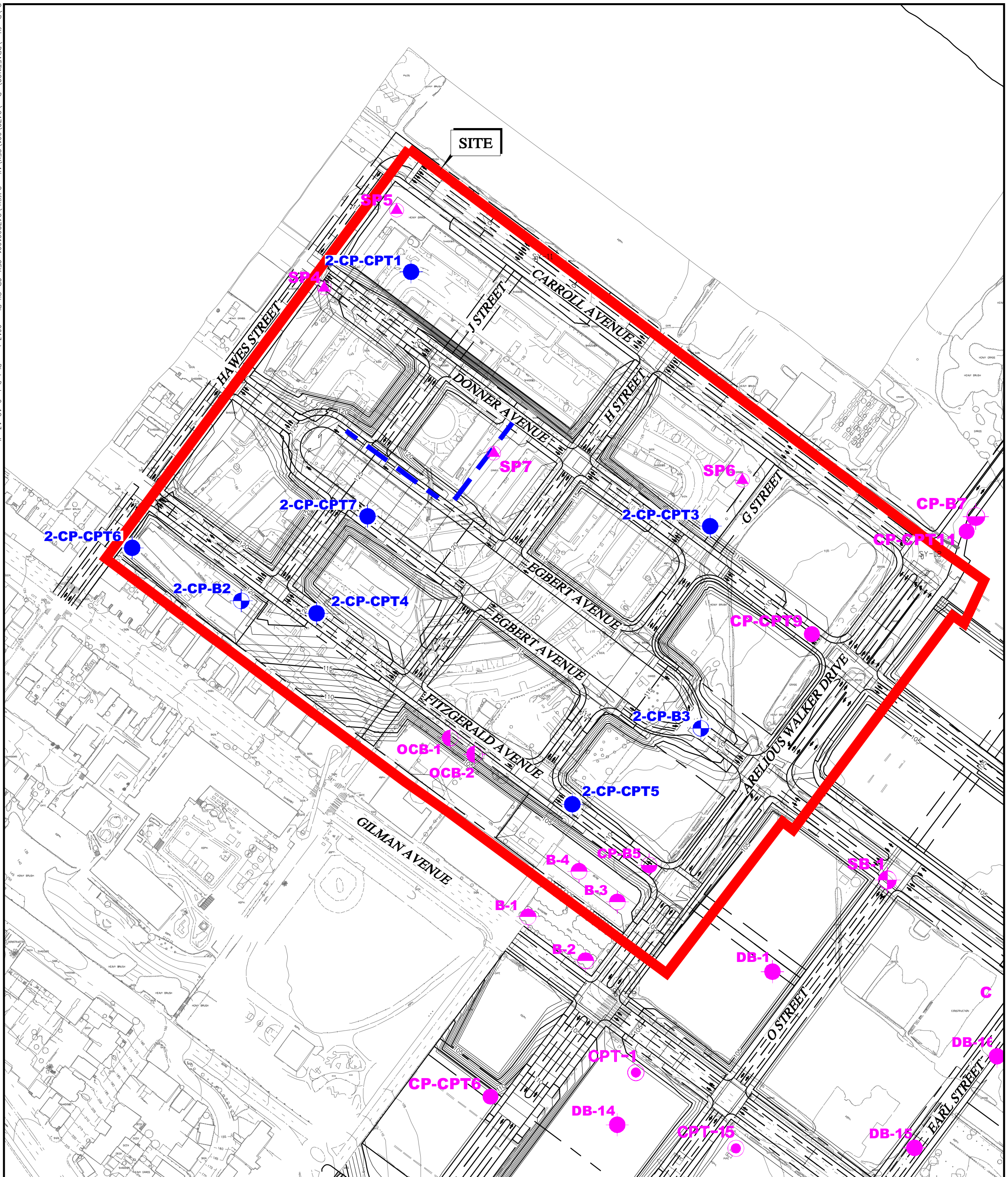

EXPLANATION

- APPROXIMATE LOCATION OF SEISMIC SURVEY LINE
- 2-CP-CPT7 — APPROXIMATE LOCATION OF CONE PENETRATION TEST (PUSH PROBE WITH NO SPOILS) (ENGEO)
- 2-CP-B3 — APPROXIMATE LOCATION OF BORING (ENGEO)
- CP-B7 — APPROXIMATE LOCATION OF PREVIOUS EXPLORATORY BORING (ENGEO)
- CP-CPT11 — APPROXIMATE LOCATION OF PREVIOUS CONE PENETRATION TEST (ENGEO)
- SP10 — APPROXIMATE LOCATION OF PREVIOUS SURFACE SAMPLING (ENGEO)
- P2 — APPROXIMATE LOCATION OF PREVIOUS PERCOLATION TEST (ENGEO)
- CPT-18 — APPROXIMATE LOCATION OF PREVIOUS CONE PENETRATION TEST (TREADWELL & ROLLO)
- DB-16 — APPROXIMATE LOCATION OF PREVIOUS BORING (TREADWELL & ROLLO)
- B-4 — APPROXIMATE LOCATION OF PREVIOUS BORING (TREADWELL & ROLLO)
- OCB-1 — APPROXIMATE LOCATION OF PREVIOUS BORING (OLIVIA CHEN)

N

0 FEET 200
0 METERS 100

BASE MAP SOURCE: RADMAN AERIAL SURVEYS, 2013

ENGEO
Expect Excellence

PROPOSED SITE PLAN
ALICE GRIFFITH
SAN FRANCISCO, CALIFORNIA

PROJECT NO.: 8472.001.001		FIGURE NO.
SCALE: AS SHOWN		2B
DRAWN BY: LL	CHECKED BY: LC	

ORIGINAL FIGURE PRINTED IN COLOR

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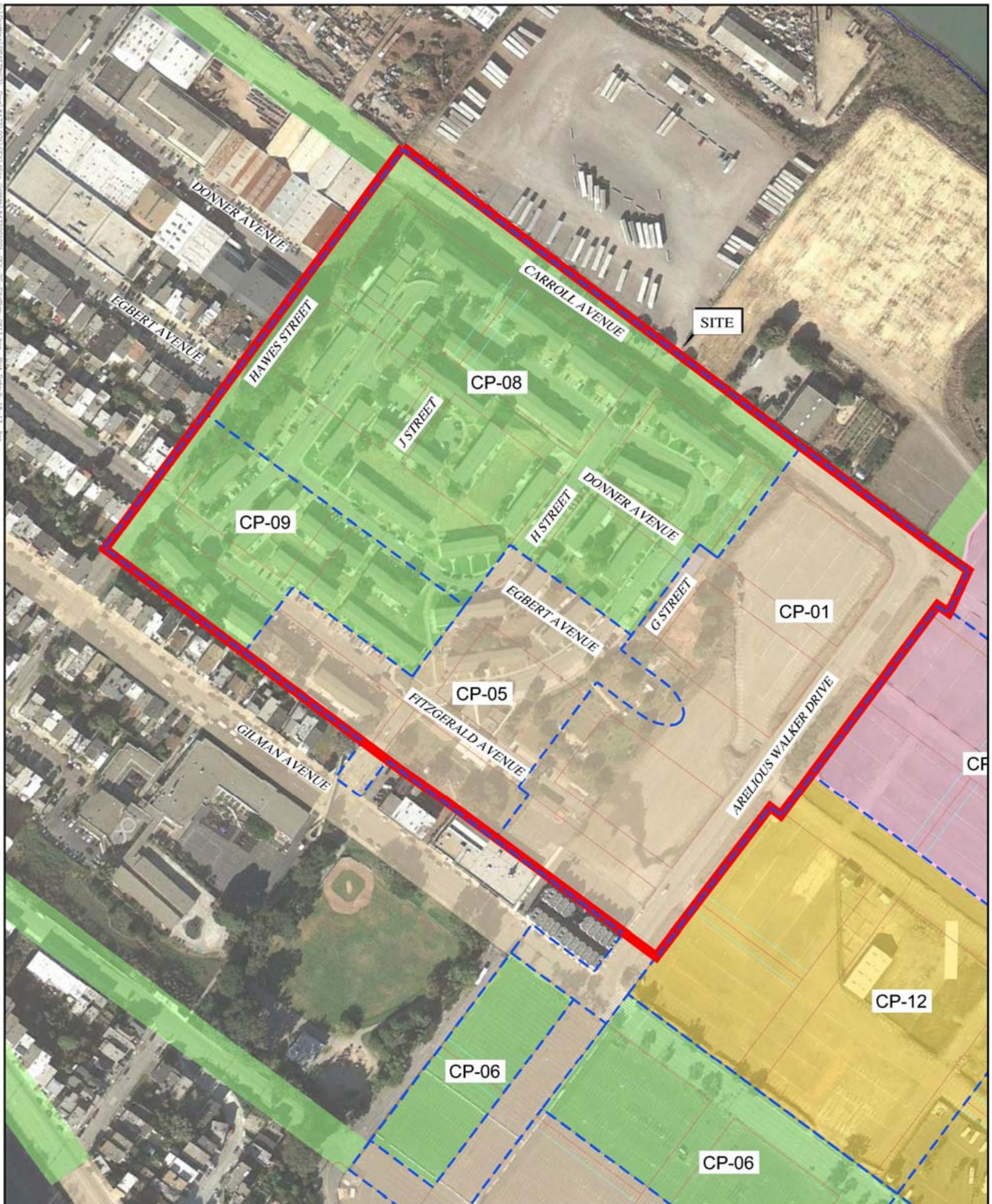


EXPLANATION

- SUB PHASE BOUNDARY LINE
- MAJOR PHASE 1 AREA
- MAJOR PHASE 2 AREA
- MAJOR PHASE 3 AREA
- MAJOR PHASE 4 AREA

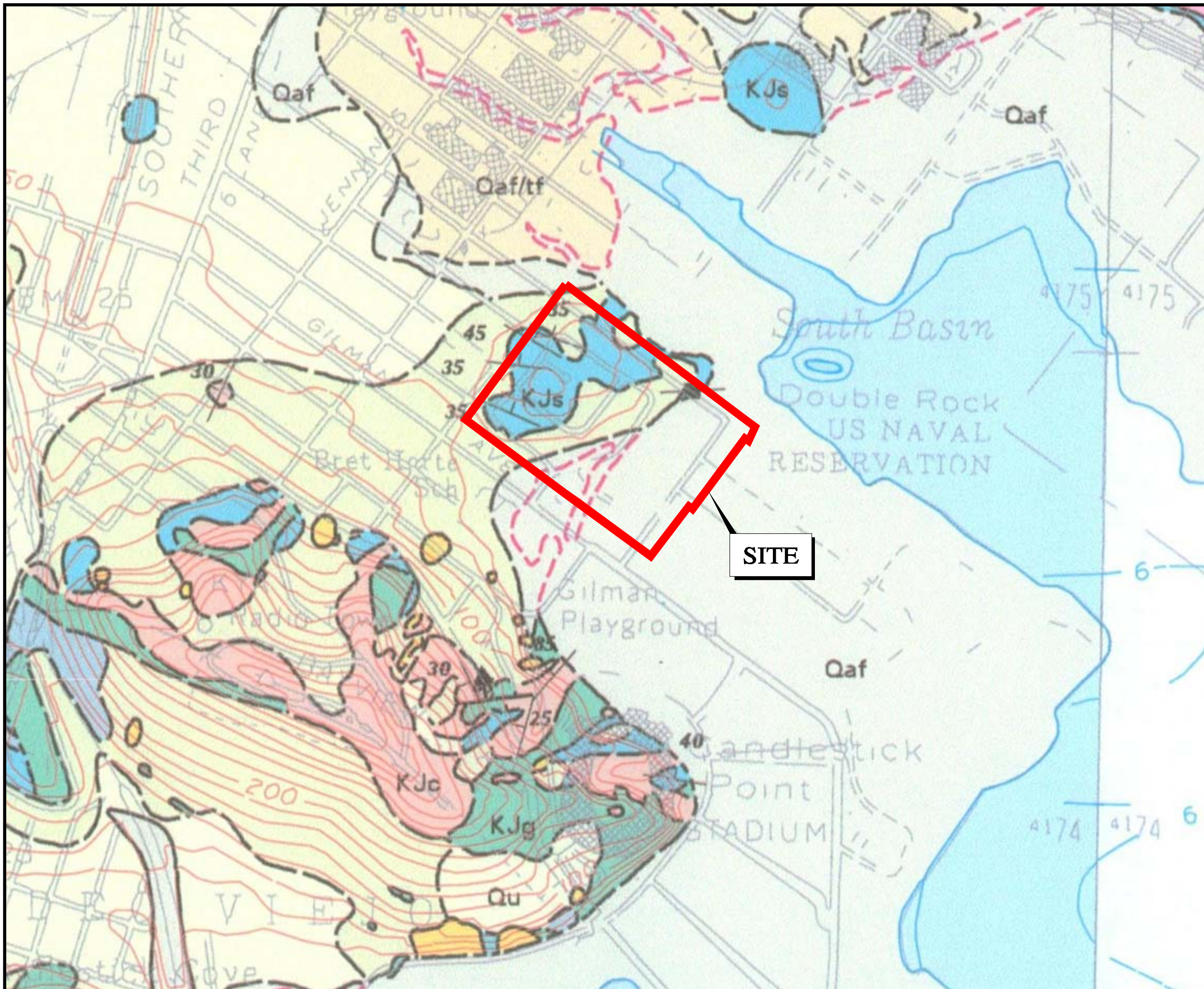
N

0 FEET 200
0 METERS 100

BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO Expect Excellence	SITE PLAN WITH MAJOR AND SUB PHASE DESIGNATION ALICE GRIFFITH SAN FRANCISCO, CALIFORNIA	PROJECT NO.: 8472.001.001 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: LC	FIGURE NO. 2C

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

———	CONTACT, CERTAIN	Qaf	ARTIFICIAL FILL
— — —	CONTACT, APPROXIMATELY LOCATED	Qaf/tf	ARTIFICIAL FILL OVER TIDAL FLAT
		Qsr	SLOPE DEBRIS AND RAVINE FILL
		Qu	SEDIMENTARY DEPOSITS, UNDIFFERENTIATED
		KJs	SANDSTONE AND SHALE
		KJc	CHERT
		KJg	GREENSTONE
		sp	SERPENTINE

N

0 FEET 1000
0 METERS 500

BASE MAP SOURCE: BONILLA, 1998

ENGEO
Expect Excellence

REGIONAL GEOLOGIC MAP
ALICE GRIFFITH
SAN FRANCISCO, CALIFORNIA

PROJECT NO.: 8472.001.001	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: LC

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ORIGINAL FIGURE PRINTED IN COLOR

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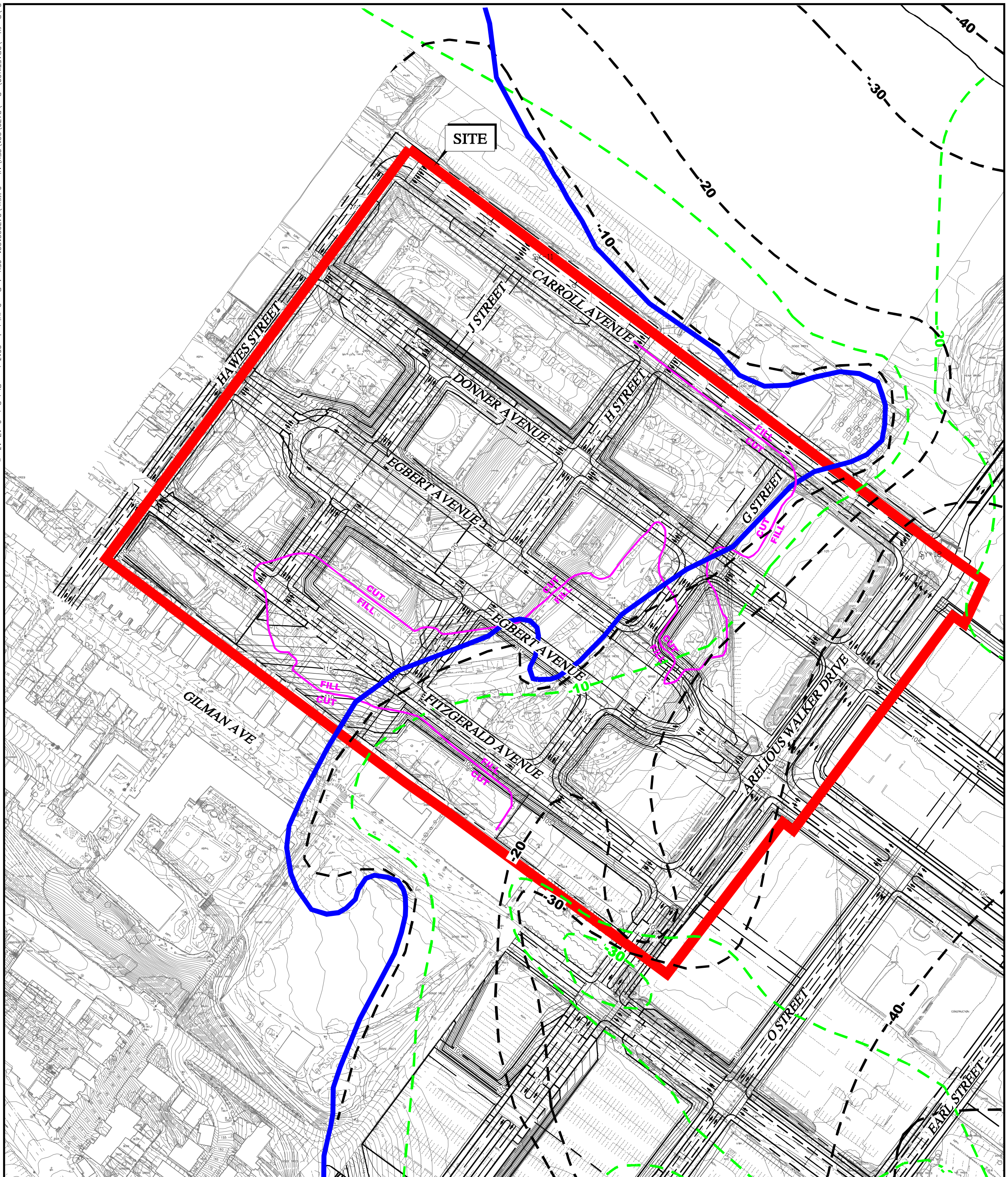

EXPLANATION

SHORELINE FROM 1903 MAPPING AND 1938 AERIAL PHOTOGRAPHY

-70 CONTOURS ON THE BASE OF BAY FILL DEPOSITS*

-80 CONTOURS ON THE BASE OF THE YOUNGER BAY MUD*

CUT / FILL — CUT AND FILL LINE

*ELEVATIONS BASED ON CCSF (CITY AND COUNTY OF SAN FRANCISCO DATUM)

N

0 FEET 200
0 METERS 100

BASE MAP SOURCE: RADMAN AERIAL SURVEYS, 2013

ENGEO Expect Excellence	PROPOSED GRADING WITH HISTORIC SHORELINE ALICE GRIFFITH SAN FRANCISCO, CALIFORNIA	PROJECT NO.: 8472.001.001 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: LC	FIGURE NO. 4

ORIGINAL FIGURE PRINTED IN COLOR

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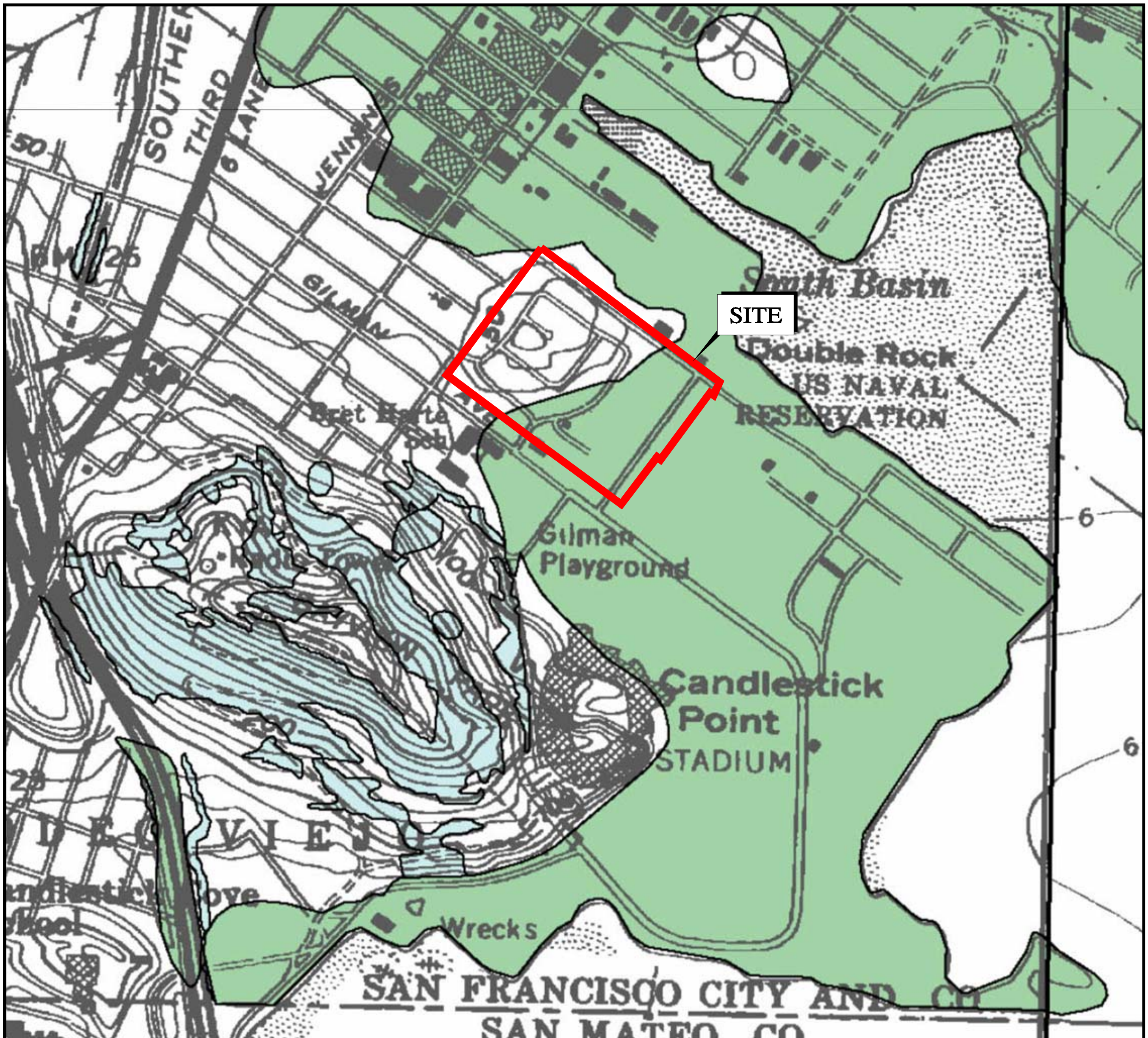


EXPLANATION

LIQUEFACTION

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

EARTHQUAKE-INDUCED LANDSLIDES

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

N

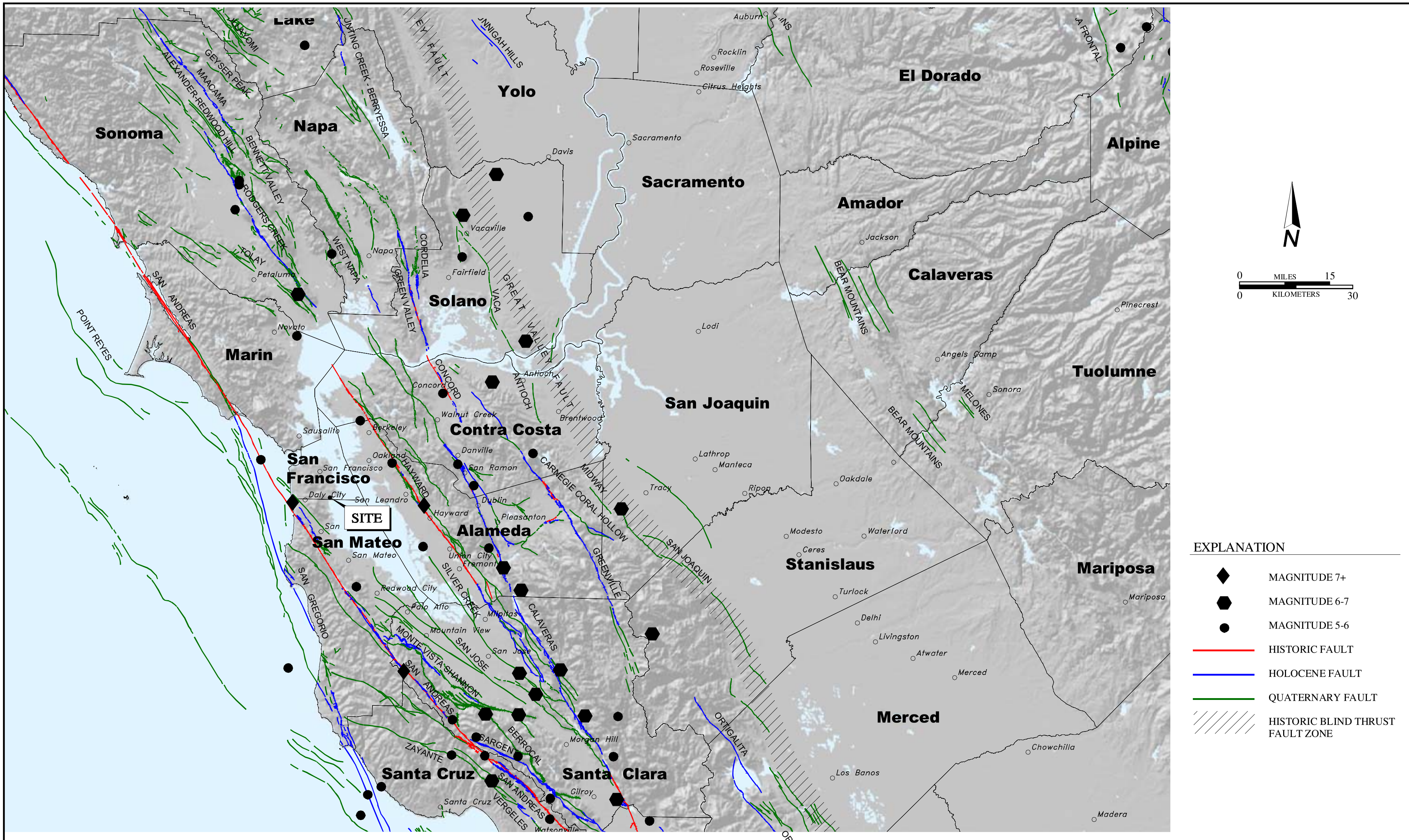
0 FEET 4000
0 METERS 2000

BASE MAP SOURCE: DEPARTMENT OF CONSERVATION, 2001

ENGEO Expect Excellence	SEISMIC HAZARD ZONE MAP ALICE GRIFFITH SAN FRANCISCO, CALIFORNIA	PROJECT NO.: 8472.001.001 SCALE: AS SHOWN DRAWN BY: LL CHECKED BY: LC	FIGURE NO. 5

G:\Drafting\DRAFTING2_Dwg\8472\001\GEX\Alice Griffith\8472001001-GEX-5-SeisHazZone-0613.dwg Plot Date: 9-17-13 llee

ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE:
HILLSHADE IMAGE BASED ON 1-ARC SECOND S.R.T.M. DATABASE
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

ENGEO — Expect Excellence —

REGIONAL FAULTING AND SEISMICITY
ALICE GRIFFITH
SAN FRANCISCO, CALIFORNIA

PROJECT NO.: 8472.001.001		FIGURE NO.
SCALE: AS SHOWN		6
DRAWN BY: LL	CHECKED BY: LC	

ORIGINAL FIGURE PRINTED IN COLOR

G:\Drafting\DRAFTING2_Dwg\8472\001\GEX\Alice Griffith\8472001001-GEX-6-RegFaultSies-0613.dwg Plot Date:9-17-13 ilee

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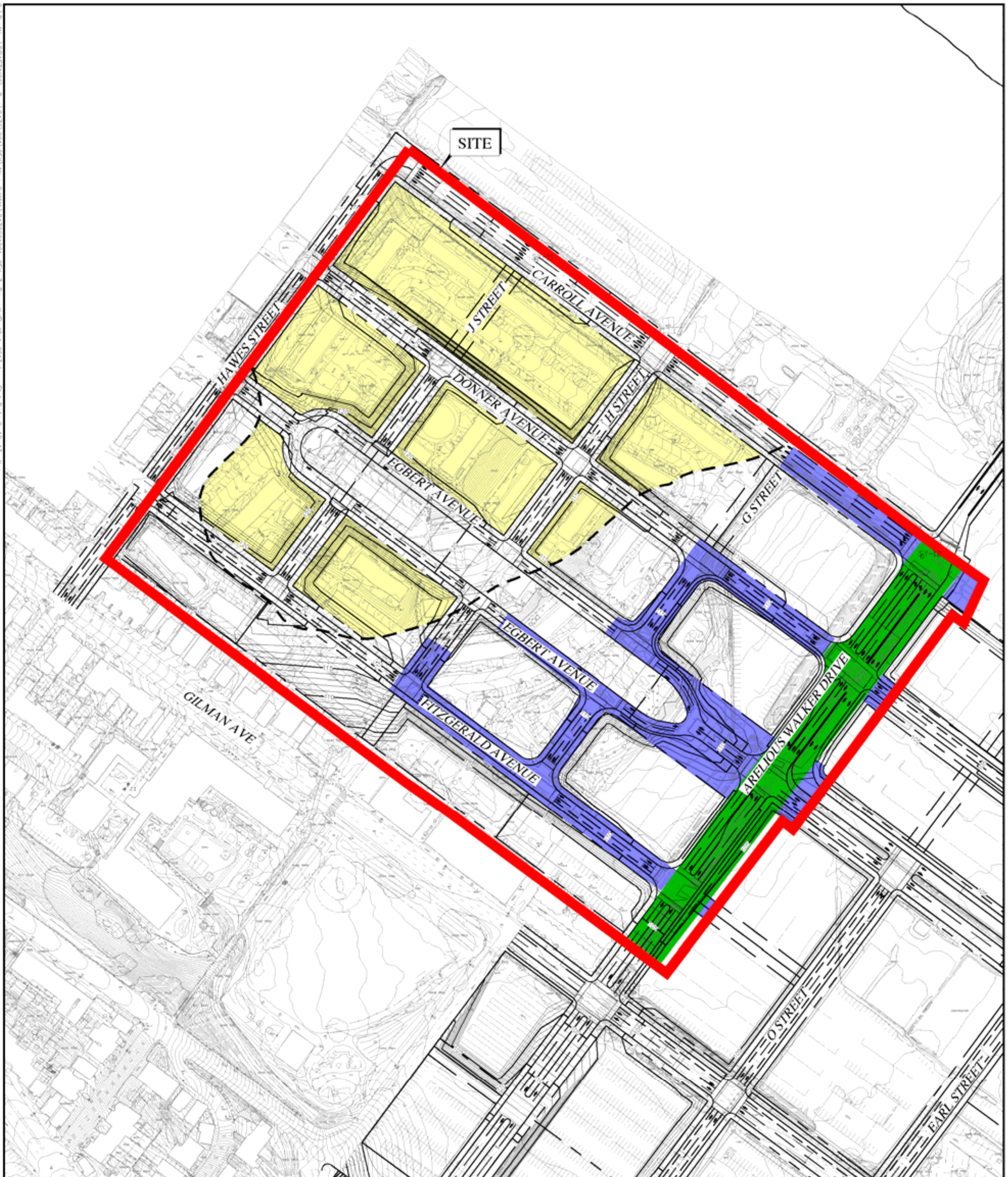


EXPLANATION

— — — APPROXIMATE CONTACT WHERE PROPOSED GRADING IS ANTICIPATED TO ENCOUNTER BEDROCK IN CUTS

APPROXIMATE LIMITS OF SURCHARGE IMPROVEMENT AREAS

APPROXIMATE LIMITS OF LIGHTWEIGHT FILL IN IMPROVEMENT AREAS

APPROXIMATE LIMITS OF RECOMMENDED 5 FOOT REMOVAL AND RECONSTRUCTION IN BUILDING PADS

N

0 FEET 200
0 METERS 100

BASE MAP SOURCE: RADMAN AERIAL SURVEYS, 2013

ENGEO — Expect Excellence —

AREA OF ANITICPATED SHALLOW BEDROCK
REQUIRING REMEDIAL PAD GRADING
ALICE GRIFFITH
SAN FRANCISCO, CALIFORNIA

PROJECT NO.: 8472.001.001	FIGURE NO. 7
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: LC

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

Exploration Logs (This Study)

Boring Logs
CPT Logs

APPENDIX A

LOG OF BORING 2-CP-B2

ENGEO INCORPORATED

Geotechnical Exploration
Alice Griffith
San Francisco, CA
8472.002.000

DATE DRILLED: 8/23/2013
HOLE DEPTH: Approx. 12¾ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (): Approx. 36 ft.

LOGGED / REVIEWED BY: A. Salehian / LC
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	DESCRIPTION	Blow Count/Foot
		3 inches thick AC	
		GREYWACKE, light brown with dark gray	
	1		
5			30/6"
	2		
		Grayish brown	50/6"
10	3		
		Yellowish brown	50/3"
		Bottom of boring at approximately 12.75 feet Groundwater was not encountered during drilling	

Column headings printed with no entries: Log Symbol, Water Level, Atterberg Limits (Liquid Limit, Plastic Limit, 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.

LOG OF BORING 2-CP-B3

ENGEO INCORPORATED

Geotechnical Exploration
Candlestick Park
San Francisco, CA
8472.001.002

DATE DRILLED: 3/12/2013
HOLE DEPTH: Approx. 54 ft.
HOLE DIAMETER: 8.0 in.
SURF ELEV (): Approx. 18 ft.

LOGGED / REVIEWED BY: A. Salehian / LC
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: 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
			GRAVELLY ELASTIC SILT (ML), dark brown, moist, with subangular fine to coarse gravel, (FILL)												
	1		SILT (ML), yellowish red, very stiff, moist, 5 to 10% subangular fine to coarse gravel, (FILL)			18	27	15	12		10.2	122.2		3.0*	PP
5	2		POORLY GRADED GRAVEL WITH CLAY (GP), brown, medium dense, moist, (FILL)			17				15					
			WELL GRADED GRAVEL WITH CLAY (GW), brown, loose, moist, with fine to coarse-grained sand, (FILL)			11									
10	3		POORLY GRADED SAND (SP), brown, very loose, wet, 5 to 10% subanguar fine gravel, (FILL)			3				9	16.8				
	4														
15			Wood, tire, debris, <5% seashells, (FILL)												
	5														
20	6		5 to 10% clay			2				37					
			FAT CLAY (CH), dark gray, medium stiff, wet, (Young Bay Mud)										615*		TV
	7					100 psi	102	36	66				560		LVS
25													1025*		TV
	8					100 psi								0.75*	PP
						200 psi									
30	9		POORLY GRADED SAND (SP), dark gray, wet, <5% clay, (Alluvium)											3.75*	PP

LOG OF BORING 2-CP-B3

Geotechnical Exploration
Candlestick Park
San Francisco, CA
8472.001.002

DATE DRILLED:	3/12/2013
HOLE DEPTH:	Approx. 54 ft.
HOLE DIAMETER:	8.0 in.
SURF ELEV ():	Approx. 18 ft.
LOGGED / REVIEWED BY:	A. Salehian / LC
DRILLING CONTRACTOR:	Pitcher Drilling
DRILLING METHOD:	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			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
							Liquid Limit	Plastic Limit	Plasticity Index						
	10		BECOMES greenish gray, dense			33									
35	11														
			BECOMES medium dense			17									
40	12														
	13		FAT CLAY (CH), light gray, stiff, wet, (Old Bay Mud)			16							1830	1.75* 2.75*	PP UU PP
45	14		POORLY GRADED SAND (SP), greenish gray, dense, wet, (Alluvium)												
			CLAY (CL), greenish gray, very stiff, wet			50/5"									
50	15														
	16		Contains fine-grained sand			11									

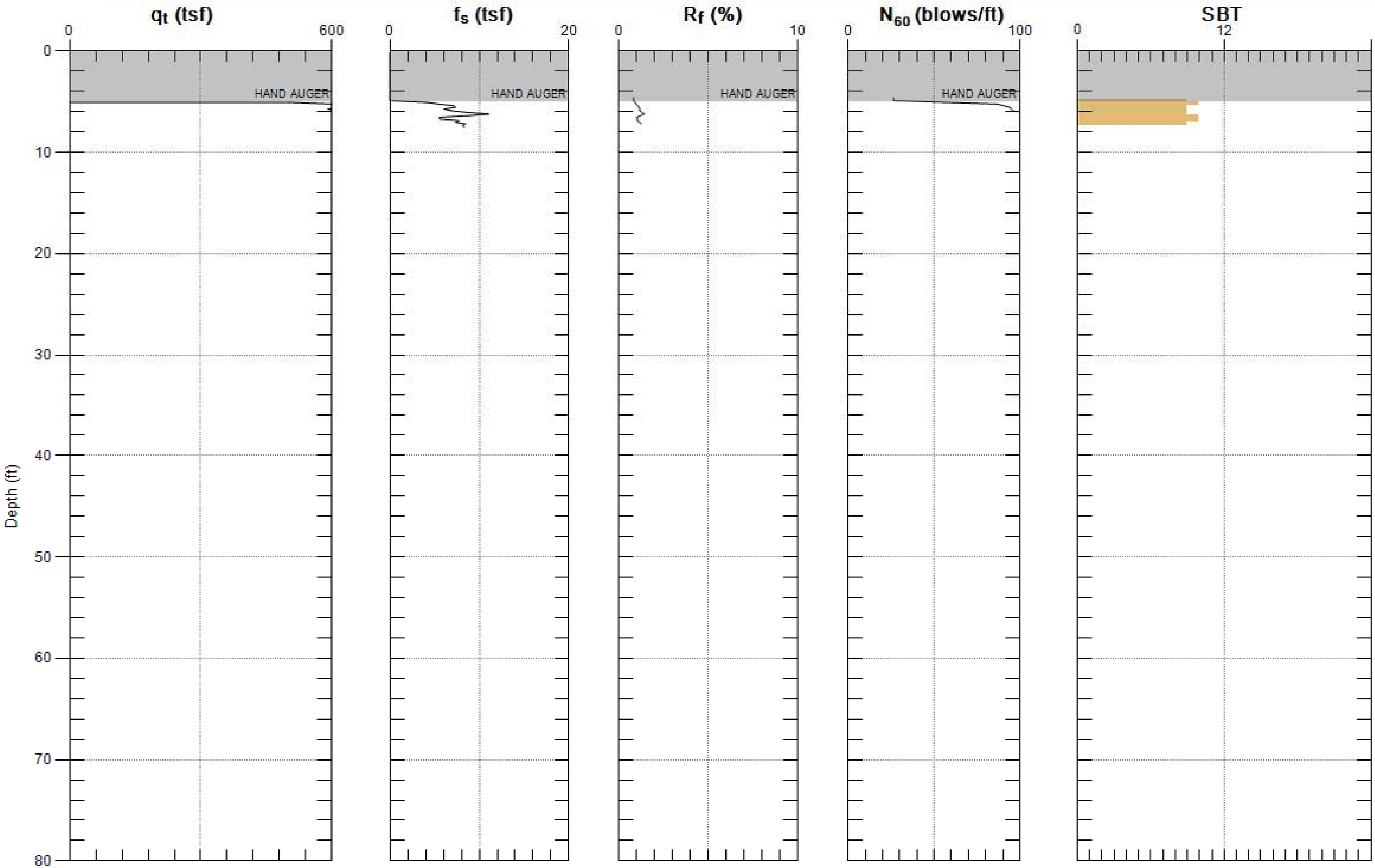
Bottom of boring at approximately 54 feet
Groundwater level was not encountered due to drilling method

LOG - SHEAR AND UNCONF STRENGTH 8472001002 CP BORING LOGS 03-22-13.GPJ ENGEO INC.GDT 6/14/13

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-1 Date: 5/28/2013 02:25



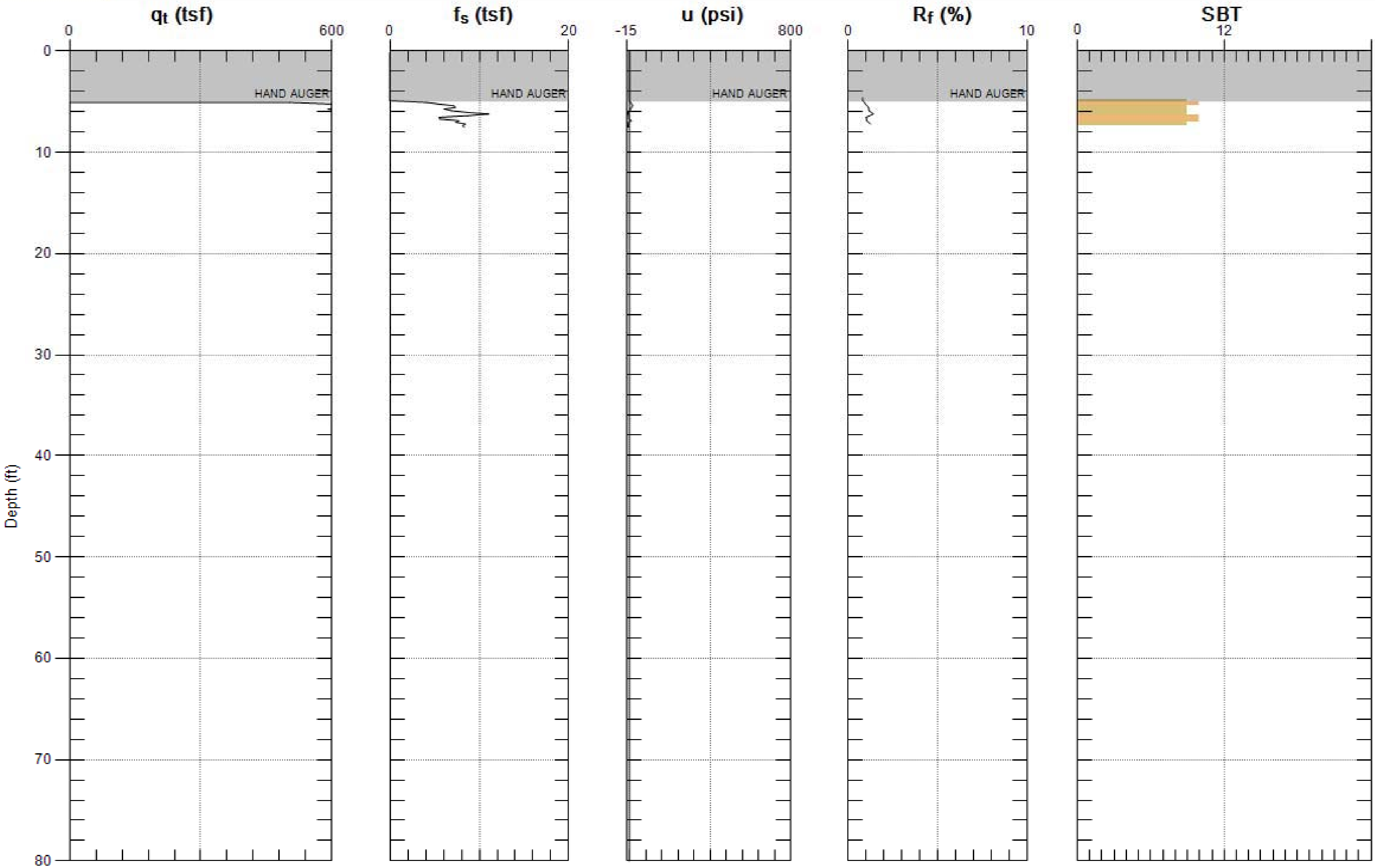
Max. Depth: 7.546 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-1 Date: 5/28/2013 02:25



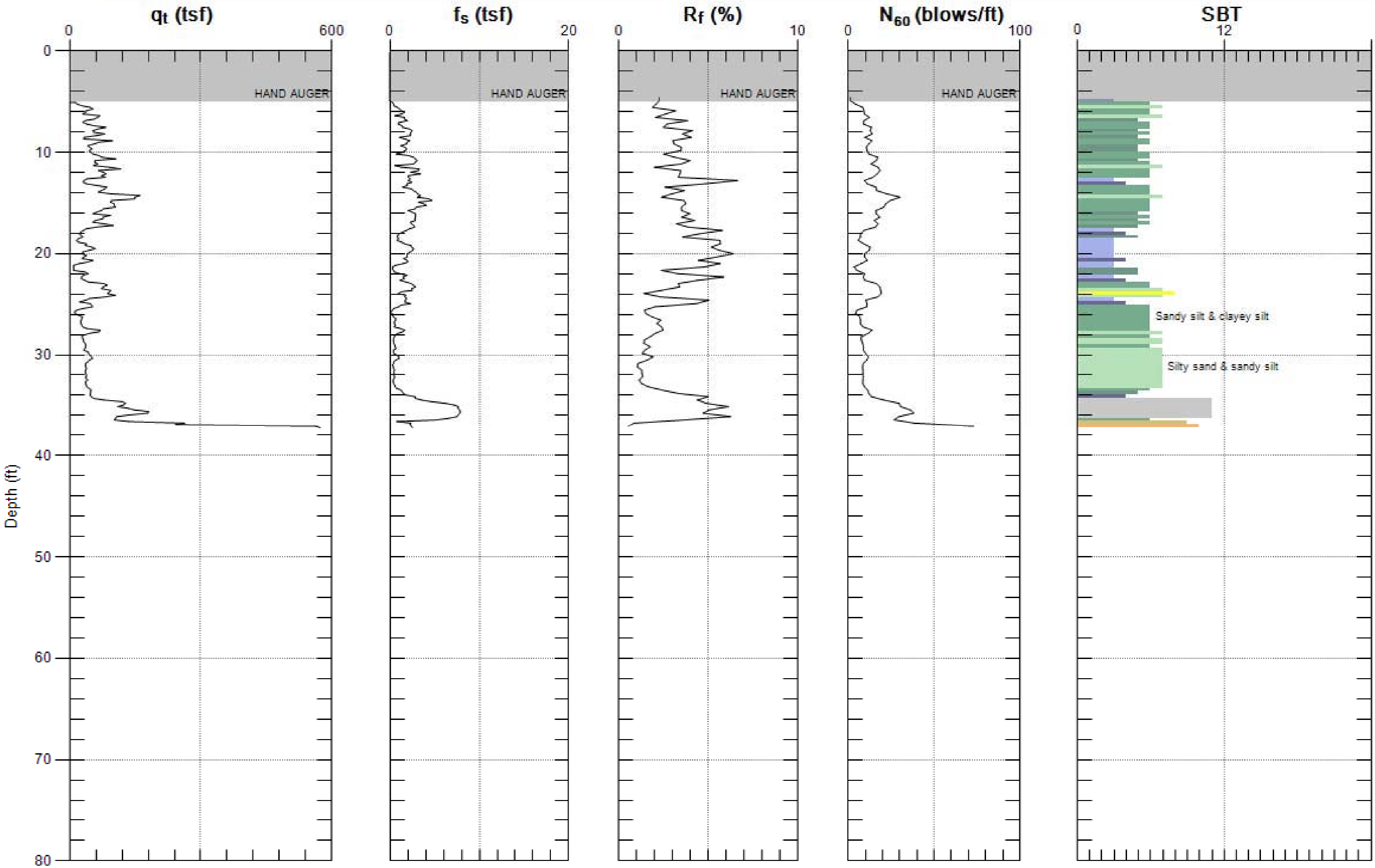
Max. Depth: 7.546 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-3 Date: 5/28/2013 12:08



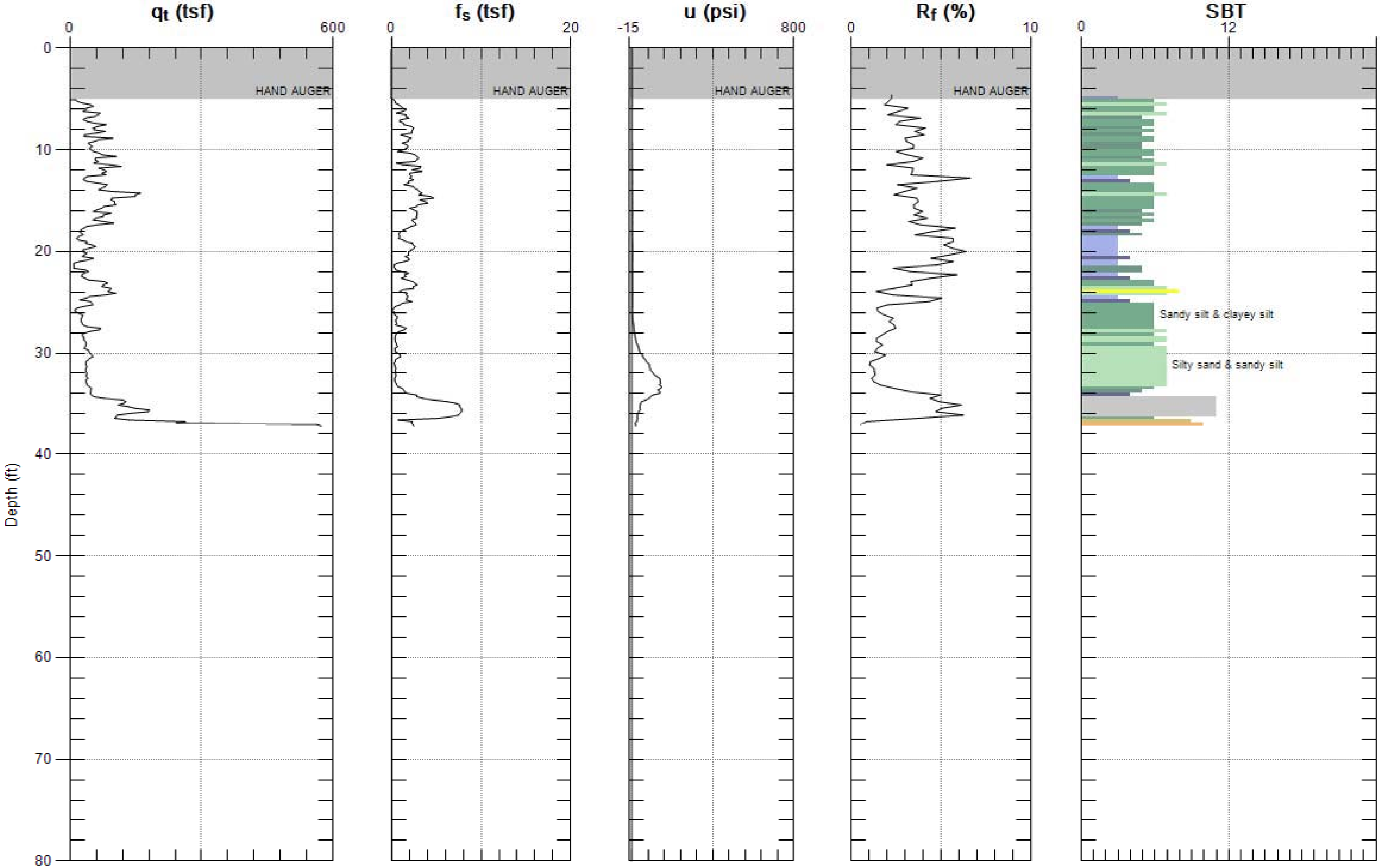
Max. Depth: 37.238 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-3 Date: 5/28/2013 12:08



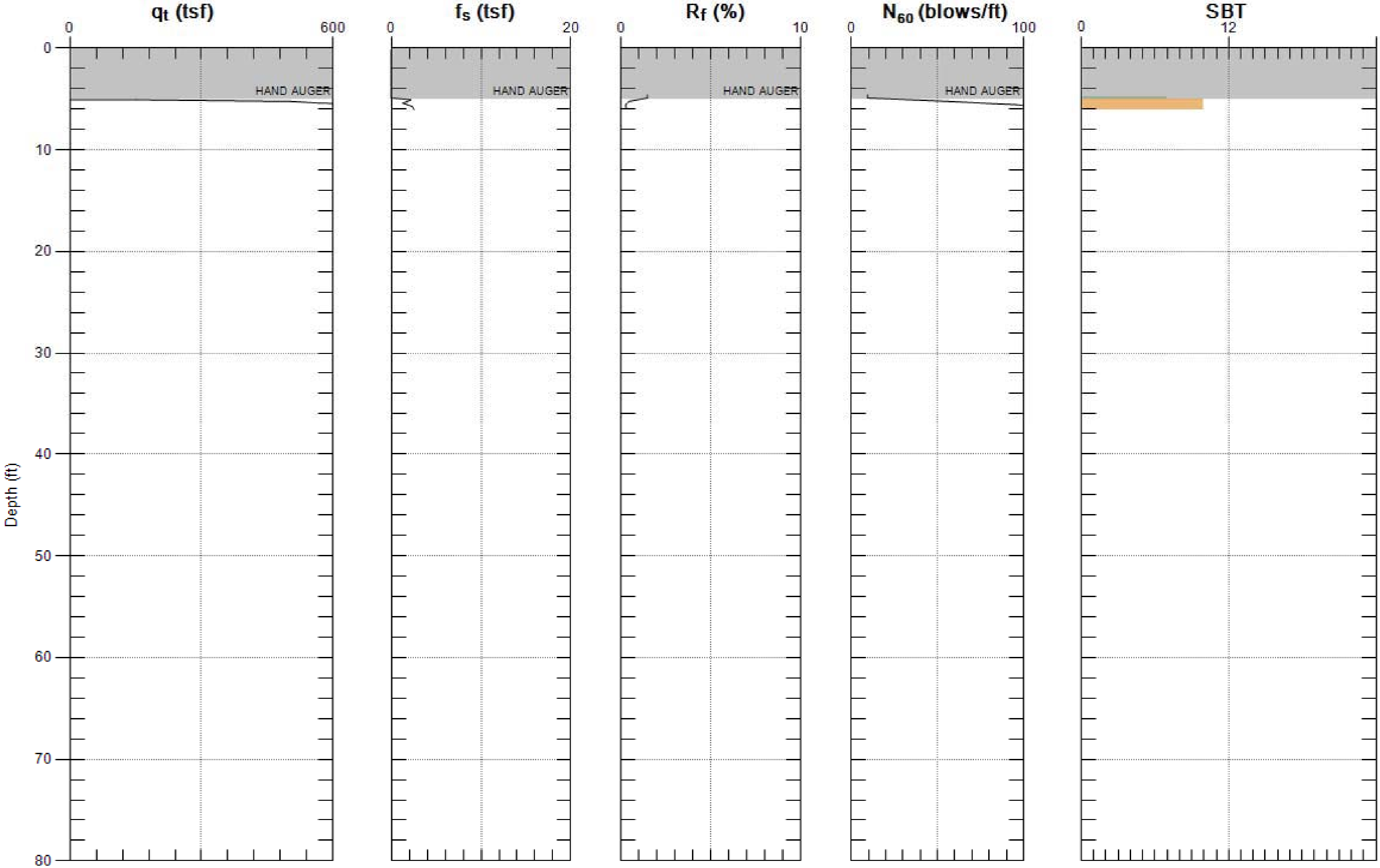
Max. Depth: 37.238 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-4 Date: 5/28/2013 10:40



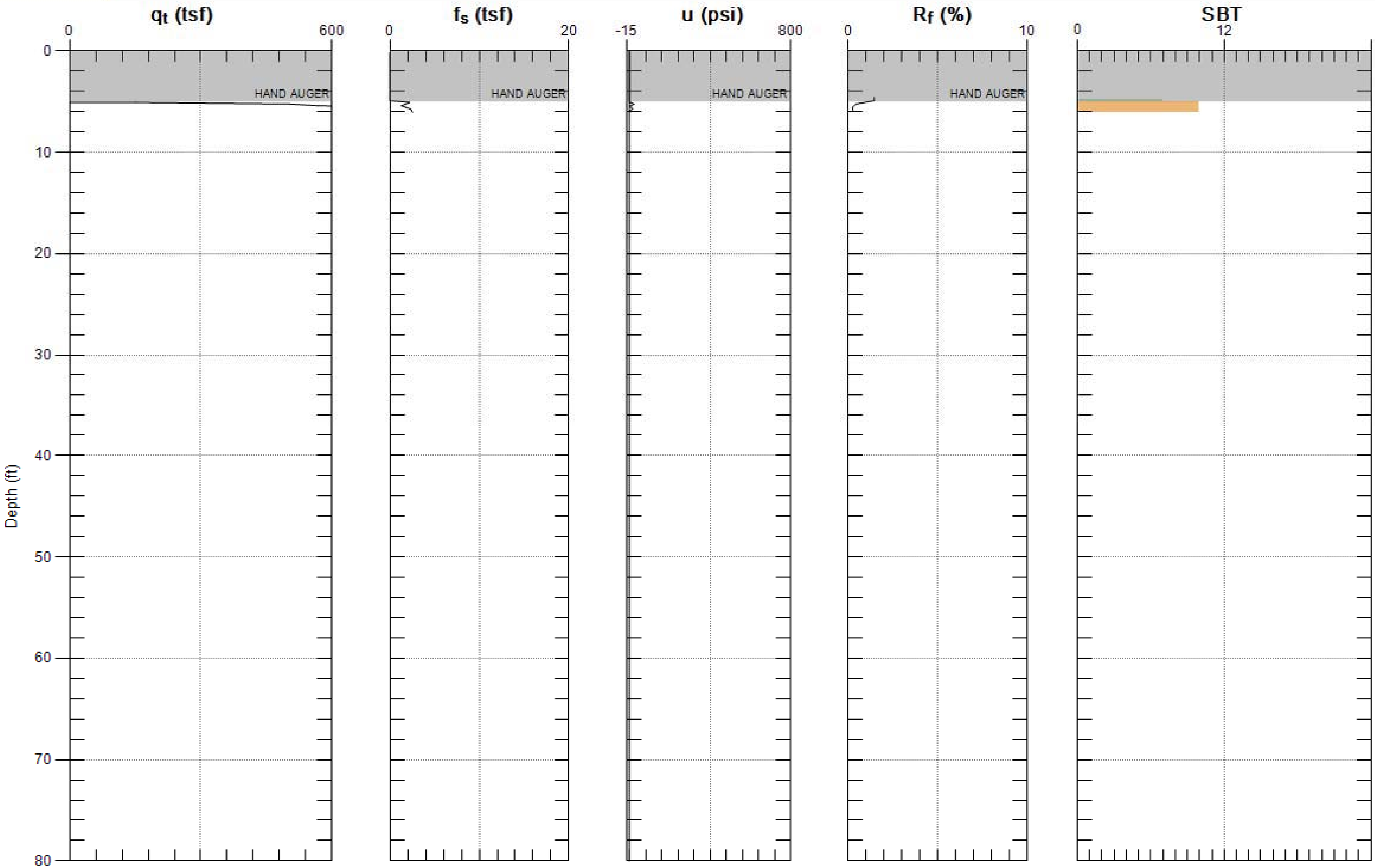
Max. Depth: 6.070 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-4 Date: 5/28/2013 10:40



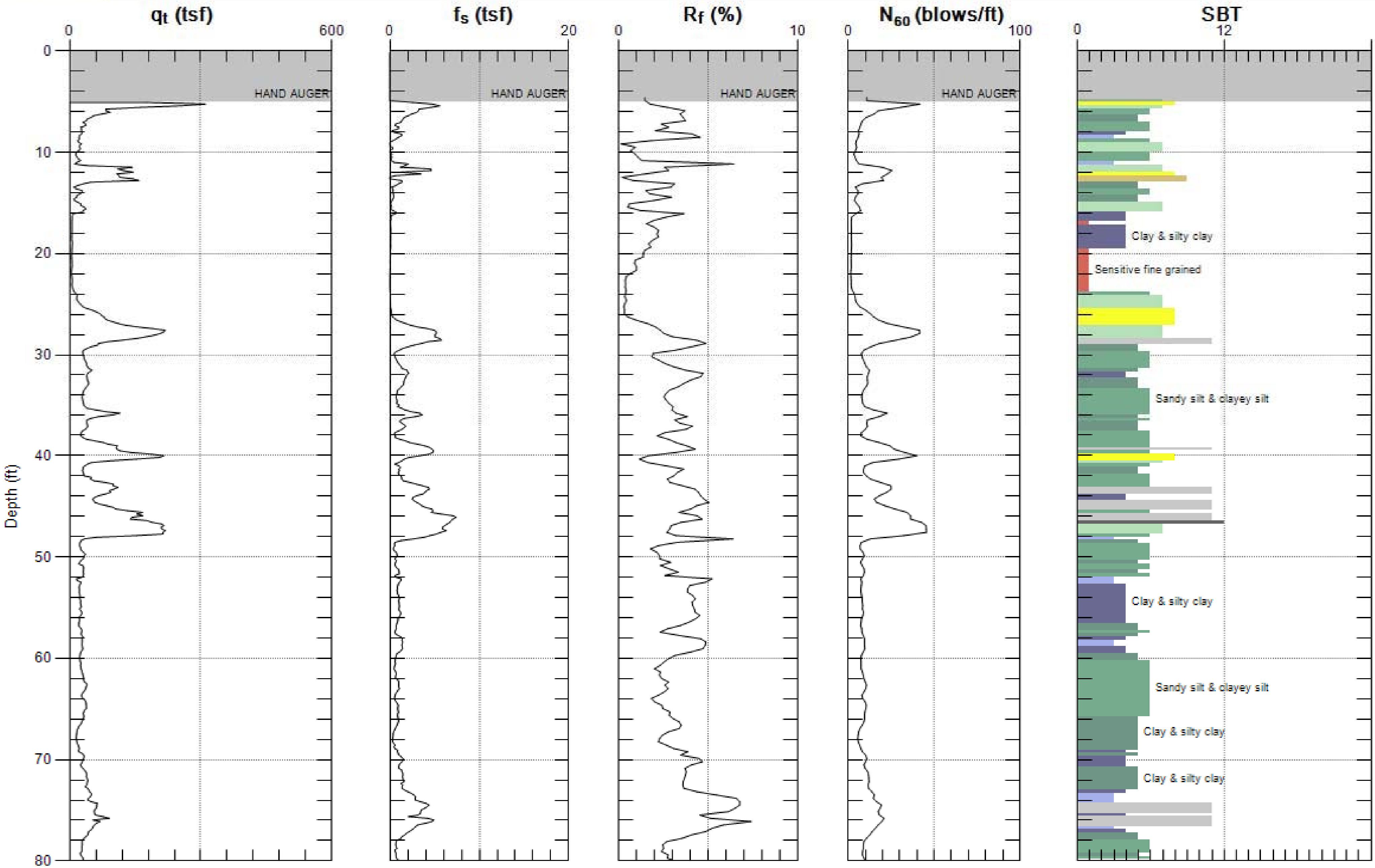
Max. Depth: 6.070 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-5 Date: 5/28/2013 05:00



Max. Depth: 80.545 (ft)
Avg. Interval: 0.328 (ft)

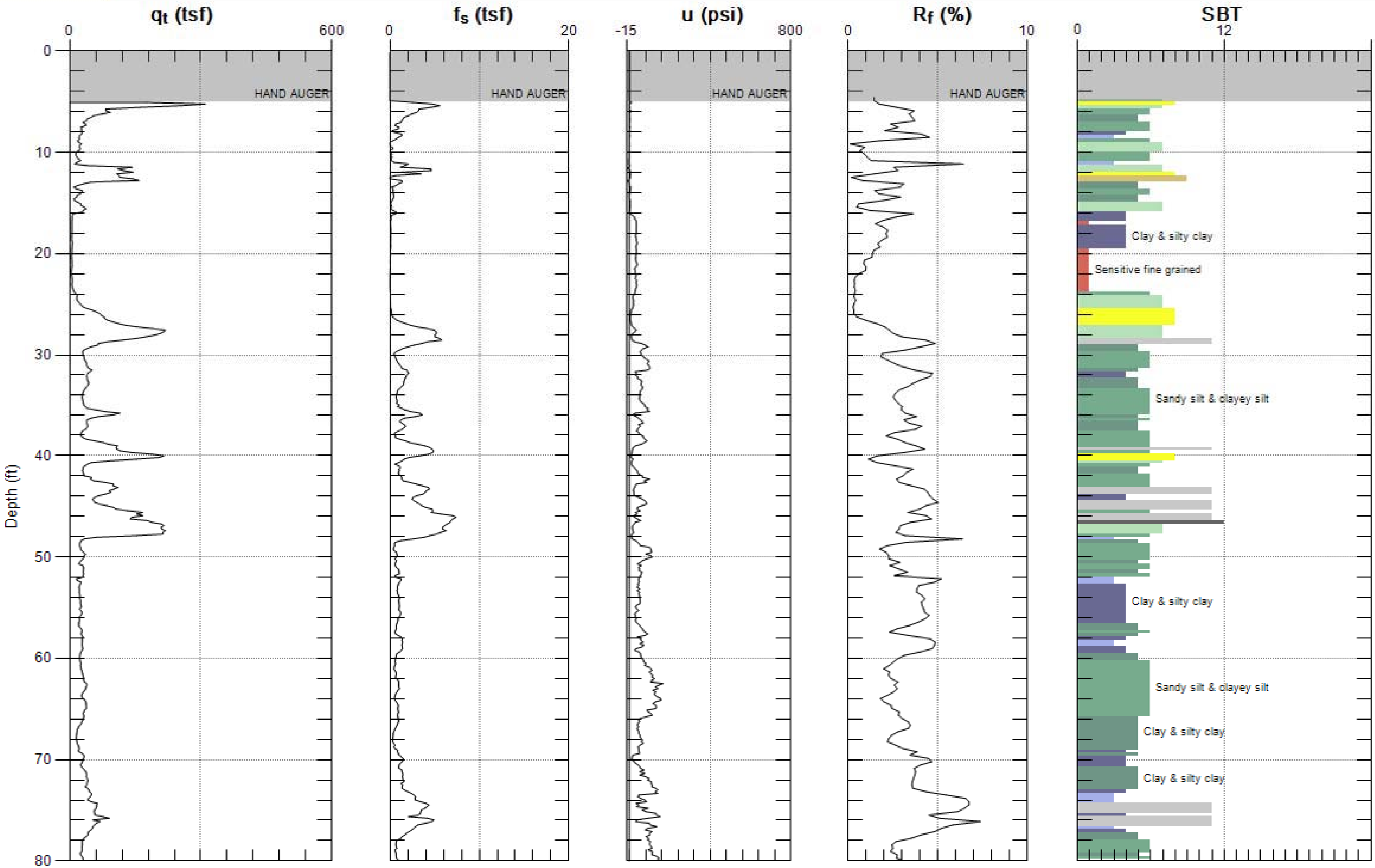
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-5 Date: 5/28/2013 05:00



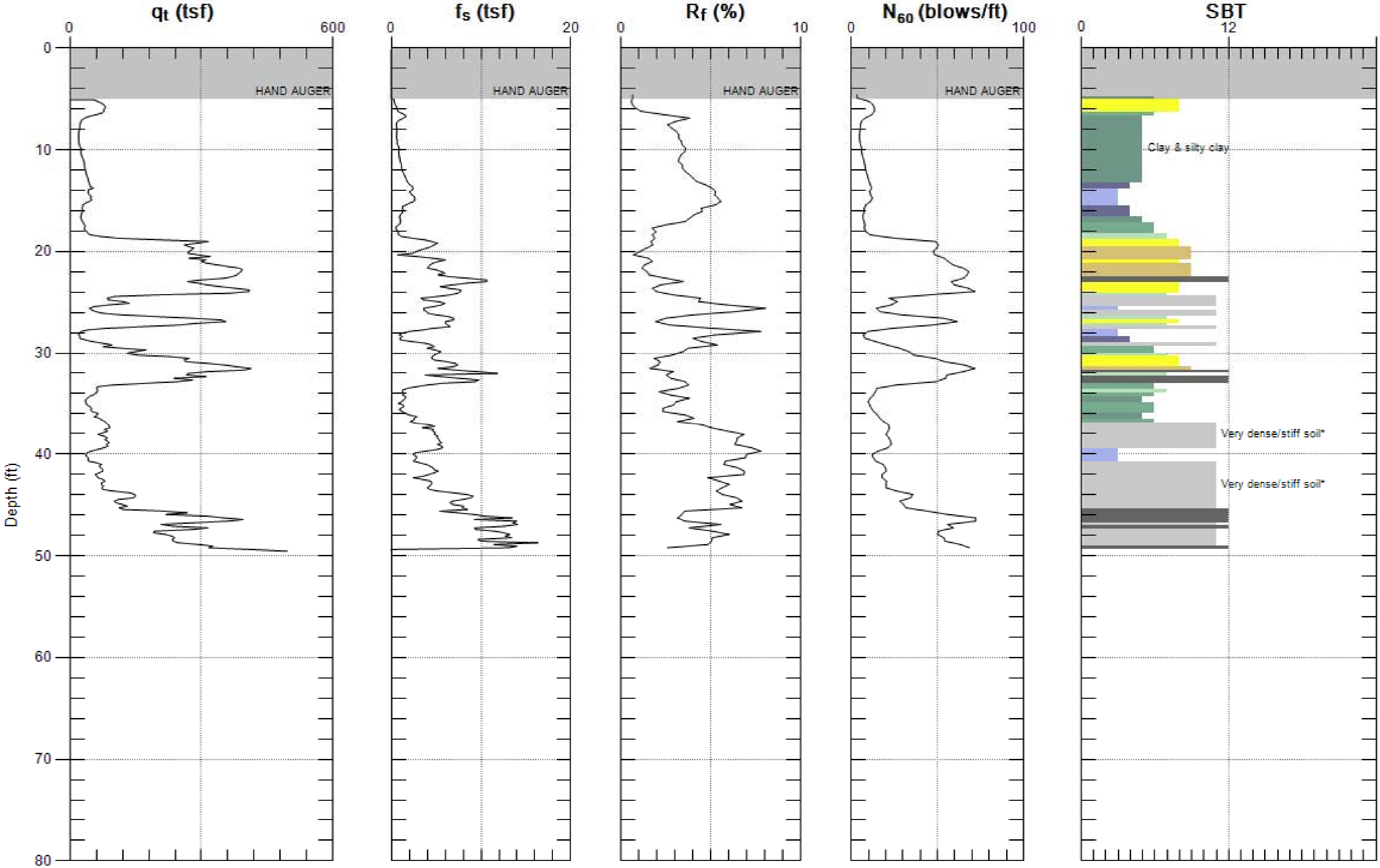
Max. Depth: 80.545 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-6 Date: 5/28/2013 08:38



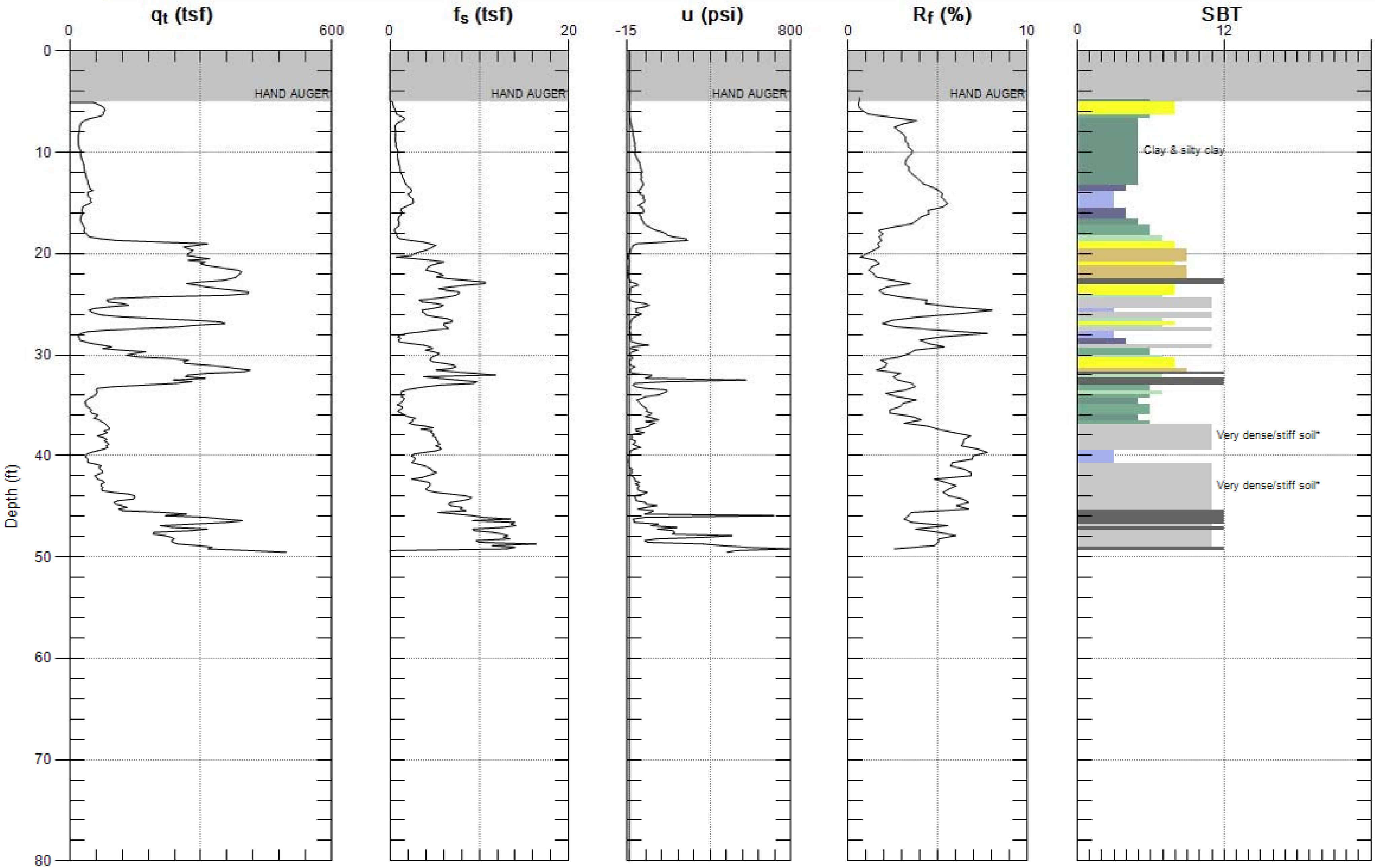
Max. Depth: 49.541 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-6 Date: 5/28/2013 08:38



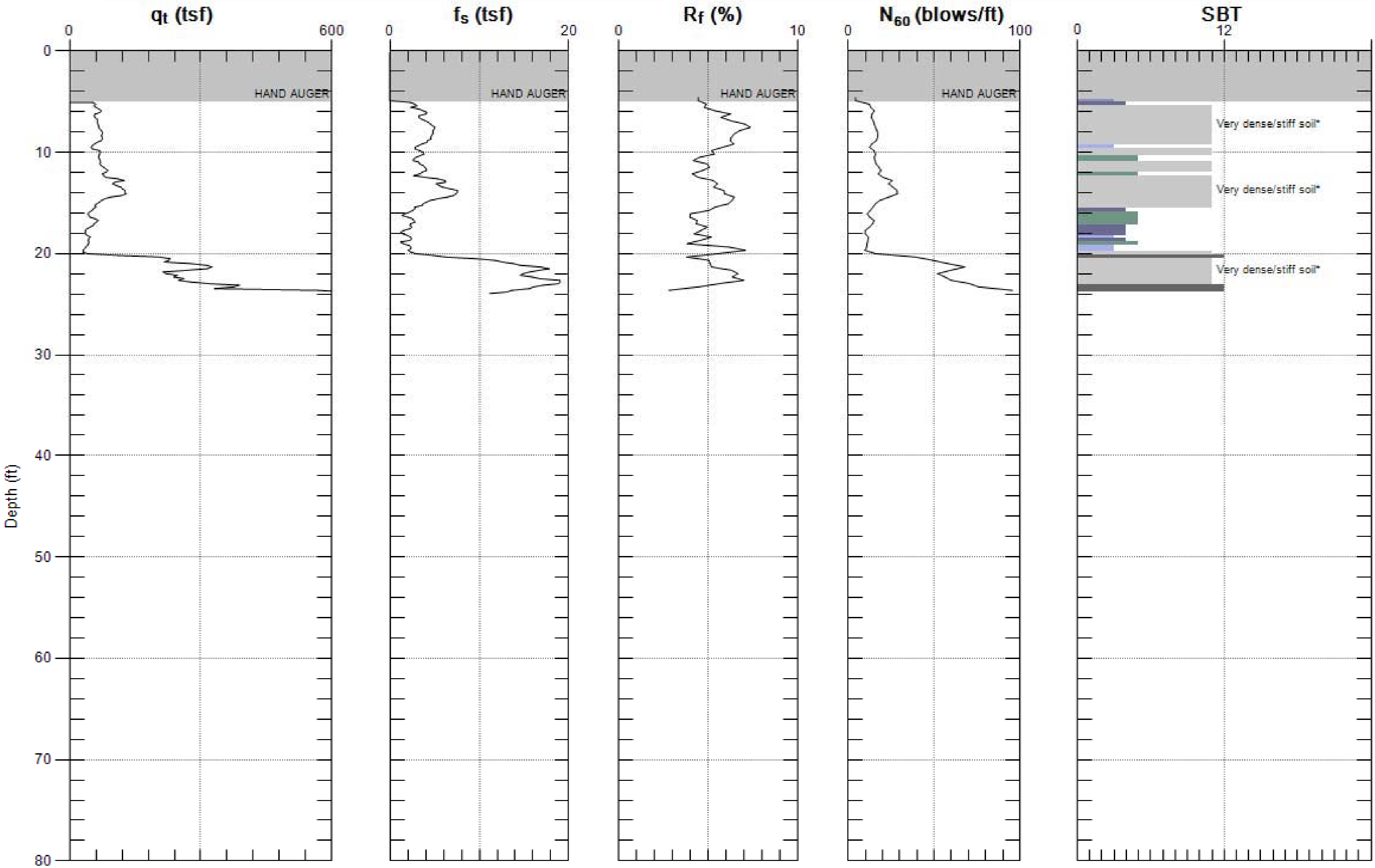
Max. Depth: 49.541 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-8 Date: 5/28/2013 03:02



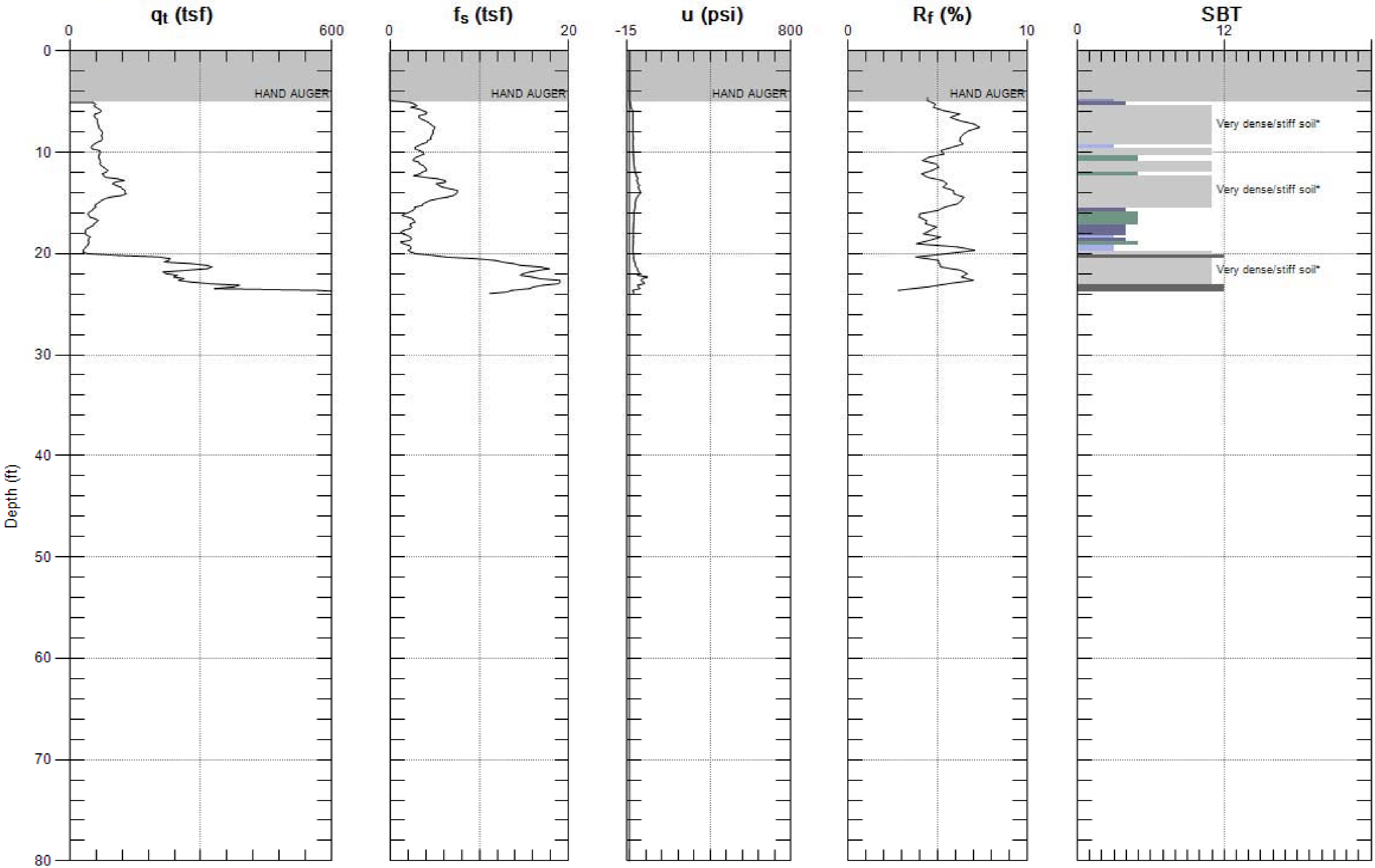
Max. Depth: 23.950 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG ENGEO

Site: HUNTERS P./CANDLESTICK Engineer: L.CHAN

Sounding: 2-CP-CPT-8 Date: 5/28/2013 03:02



Max. Depth: 23.950 (ft)
Avg. Interval: 0.328 (ft)

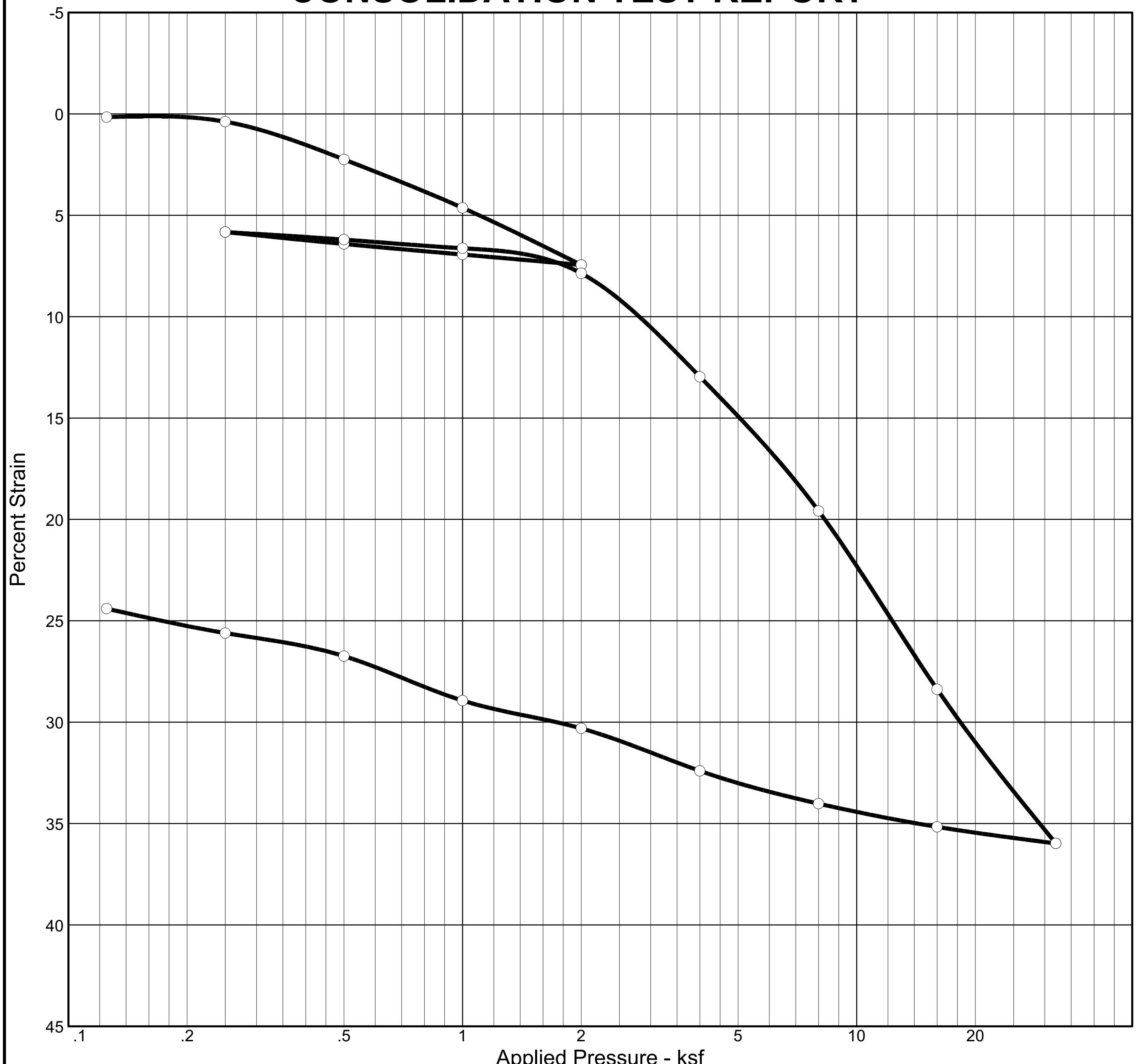
SBT: Soil Behavior Type (Robertson 1990)

APPENDIX B

Laboratory Test Results

APPENDIX B

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
96.0 %	76.9 %	53.5	102	66	2.732			2.189

MATERIAL DESCRIPTION
See exploration logs

Project No. 8472.001.001 **Client:** Lennar Urban

Project: HPS2/CP Geotechnical Exploration, SF, CA

Source: 2-CP-B3 **Sample No.:** 2-CP-B3@24 **Elev./Depth:** 24.0 feet

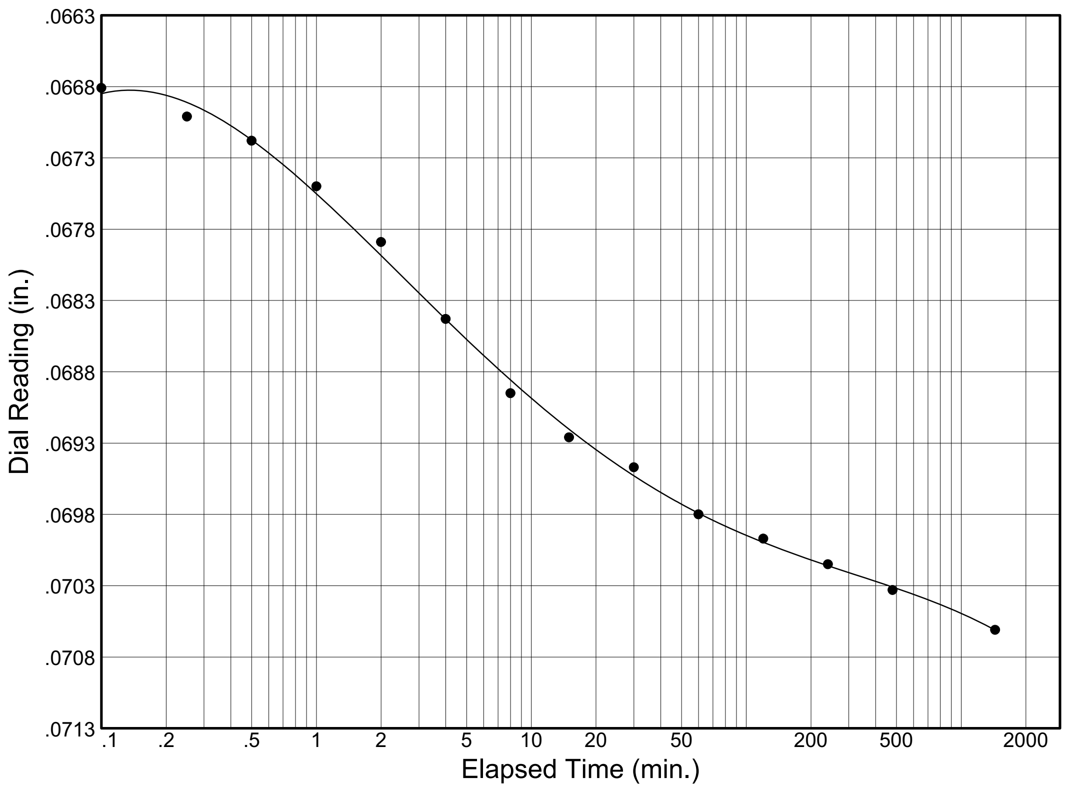
Remarks:
ASTM D2435, Method A; Initial height = 0.76388 in; Initial dial reading = 0.02000

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Dial Reading vs. Time

Project No.: 8472.001.001
Project: HPS2/CP Geotechnical Exploration, SF, CA

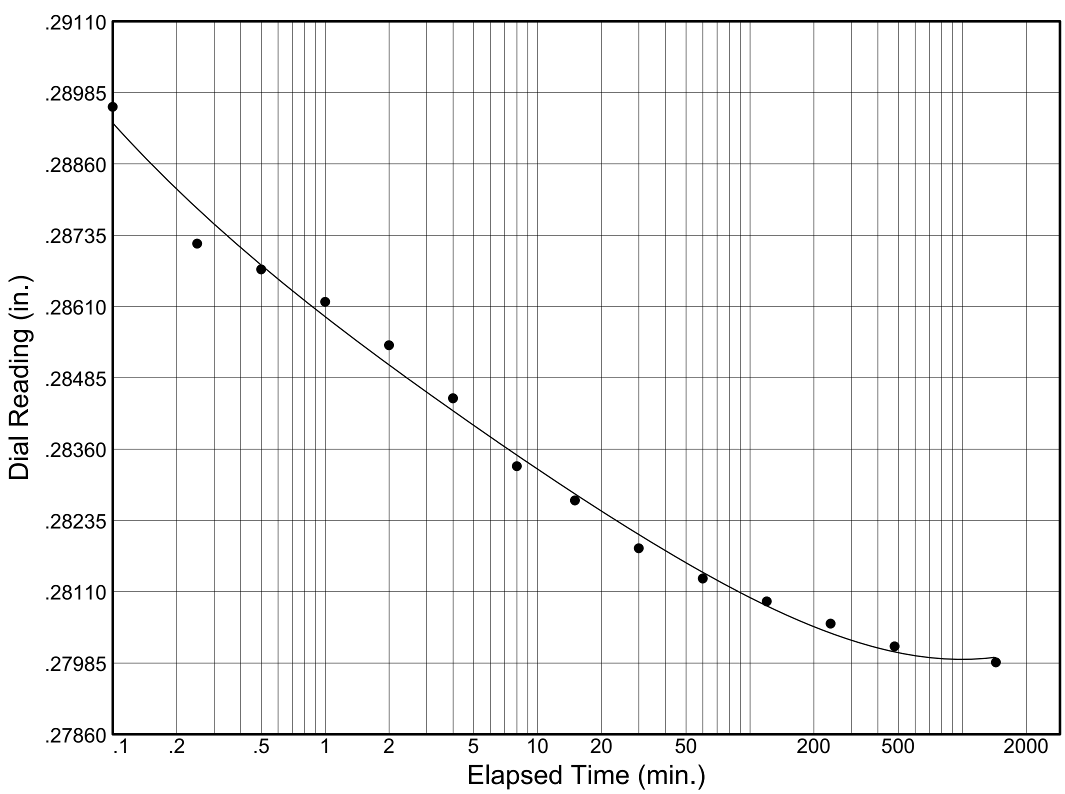
Source: Sample No.: 2-CP-B3@24 Elev./Depth: 24.0 feet



Load No.= 10
Load= 1.00 ksf
D_0 = 0.06735
D_{50} = 0.06837
D_{100} = 0.06940
T_{50} = 3.65 min.

C_v @ T_{50}
0.07 ft.^{2}/day

C_α = 0.001



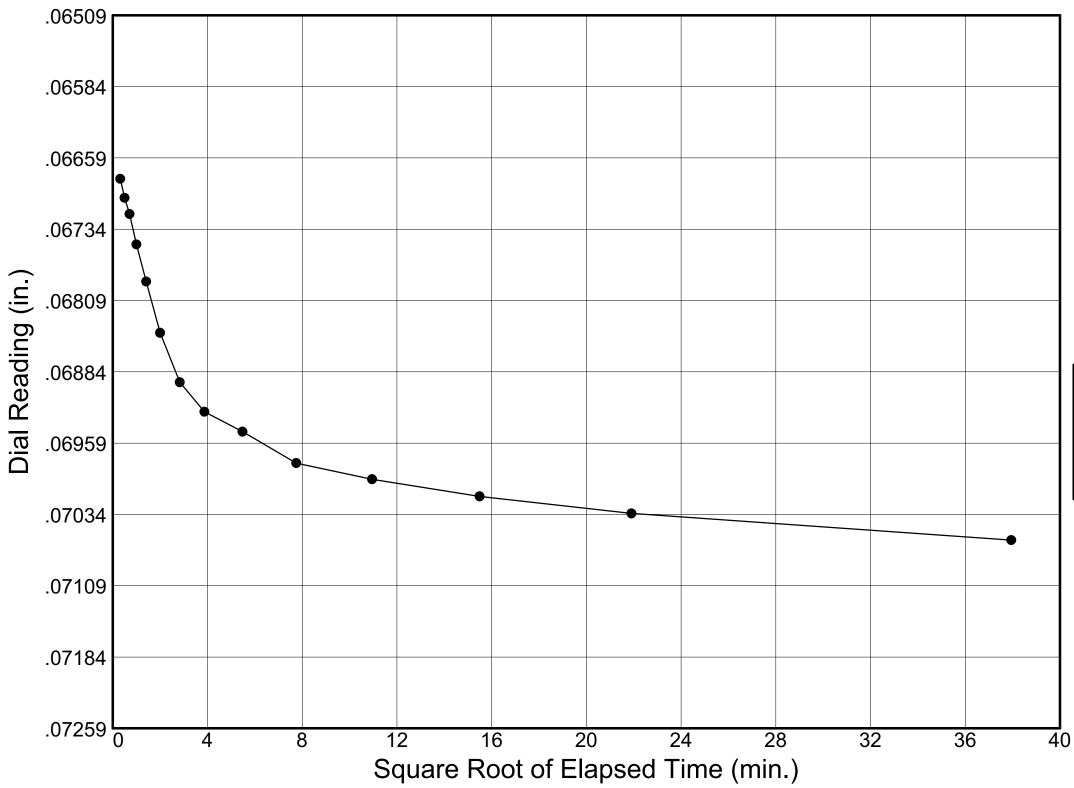
Load No.= 17
Load= 8.00 ksf
D_0 = 0.28862
D_{50} = 0.28454
D_{100} = 0.28047
T_{50} = 3.15 min.

C_v @ T_{50}
0.04 ft.^{2}/day

Dial Reading vs. Time

Project No.: 8472.001.001
Project: HPS2/CP Geotechnical Exploration, SF, CA

Source: Sample No.: 2-CP-B3@24 Elev./Depth: 24.0 feet



Load No.= 10

Load= 1.00 ksf

D_0 = 0.06659

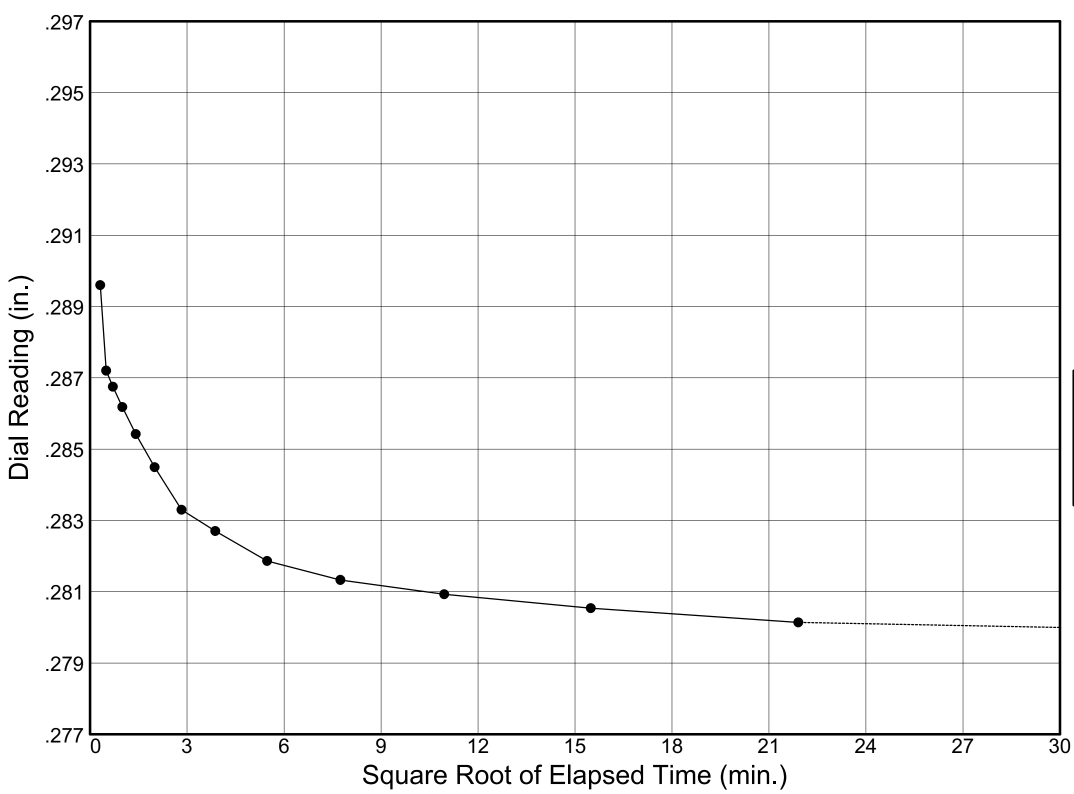
D_{90} = 0.06909

D_{100} = 0.06937

T_{90} = 10.94 min.

C_v @ T_{90}

0.10 ft.^{2}/day



Load No.= 17

Load= 8.00 ksf

D_0 = 0.29373

D_{90} = 0.28700

D_{100} = 0.28625

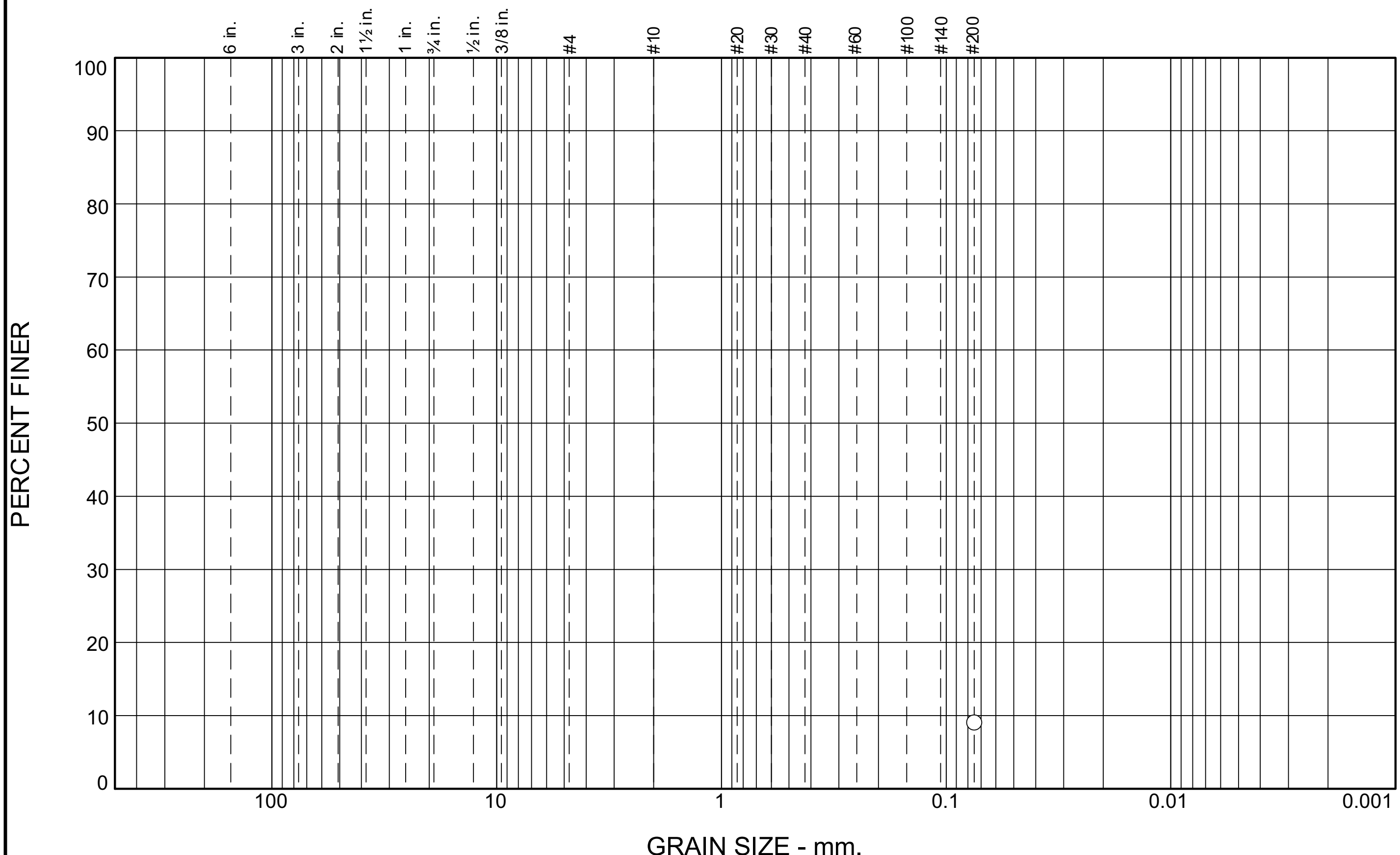
T_{90} = 0.35 min.

C_v @ T_{90}

1.51 ft.^{2}/day

ENGEO, INC.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						9.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	9.0		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Sample Number: 2-CP-B3 @ 11 **Depth:** 11.0 feet

Date: 4.4.13

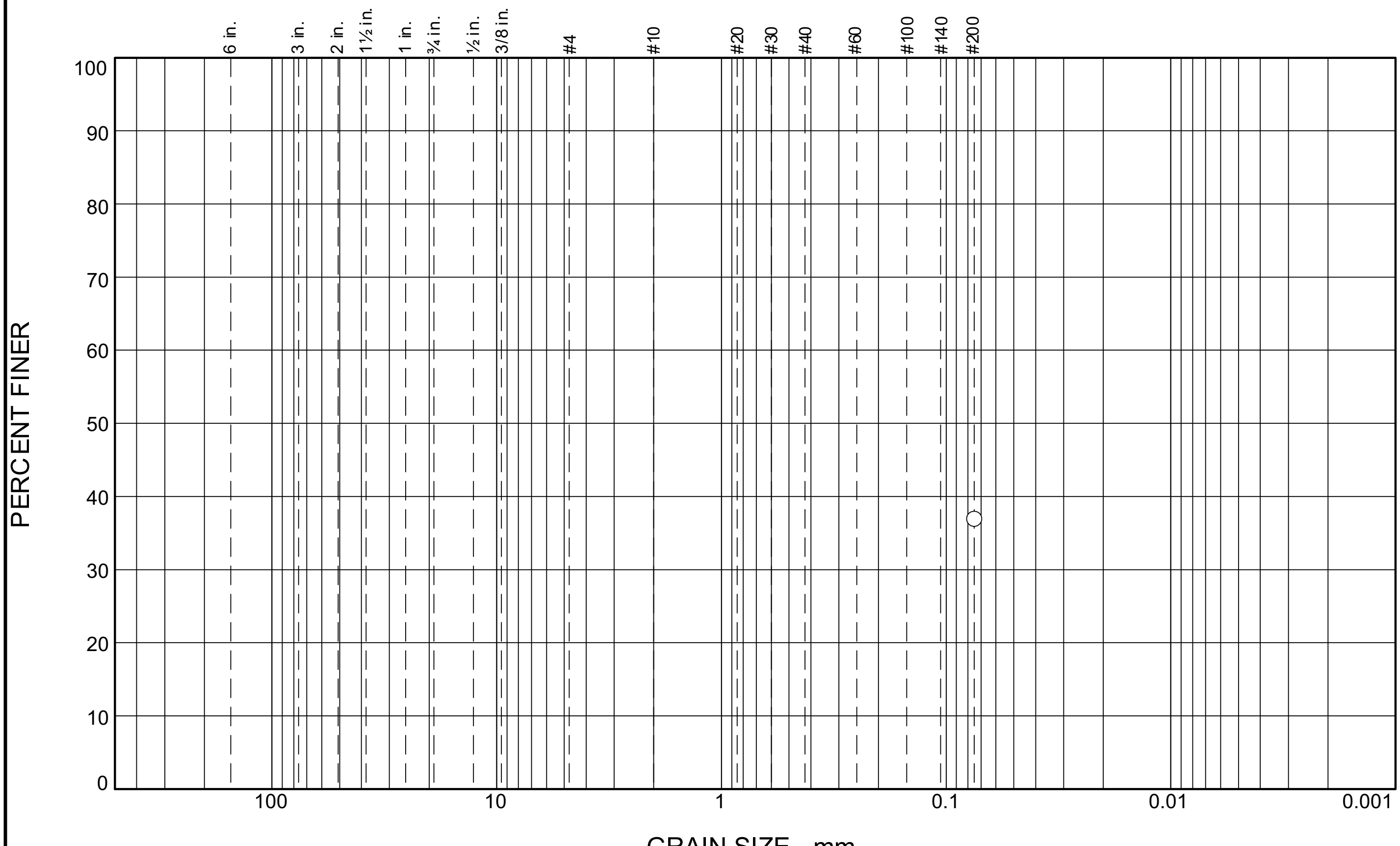
ENGEO INCORPORATED

Client: Lennar Urban
Project: HPS2/CP Geotechnical Exploration, SF, CA

Project No: 8472.001.001

Tested By: AV **Checked By:** DS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						37.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	37.0		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Sample Number: 2-CP-B3 @ 20.5 **Depth:** 20.5 feet

Date: 4.4.13

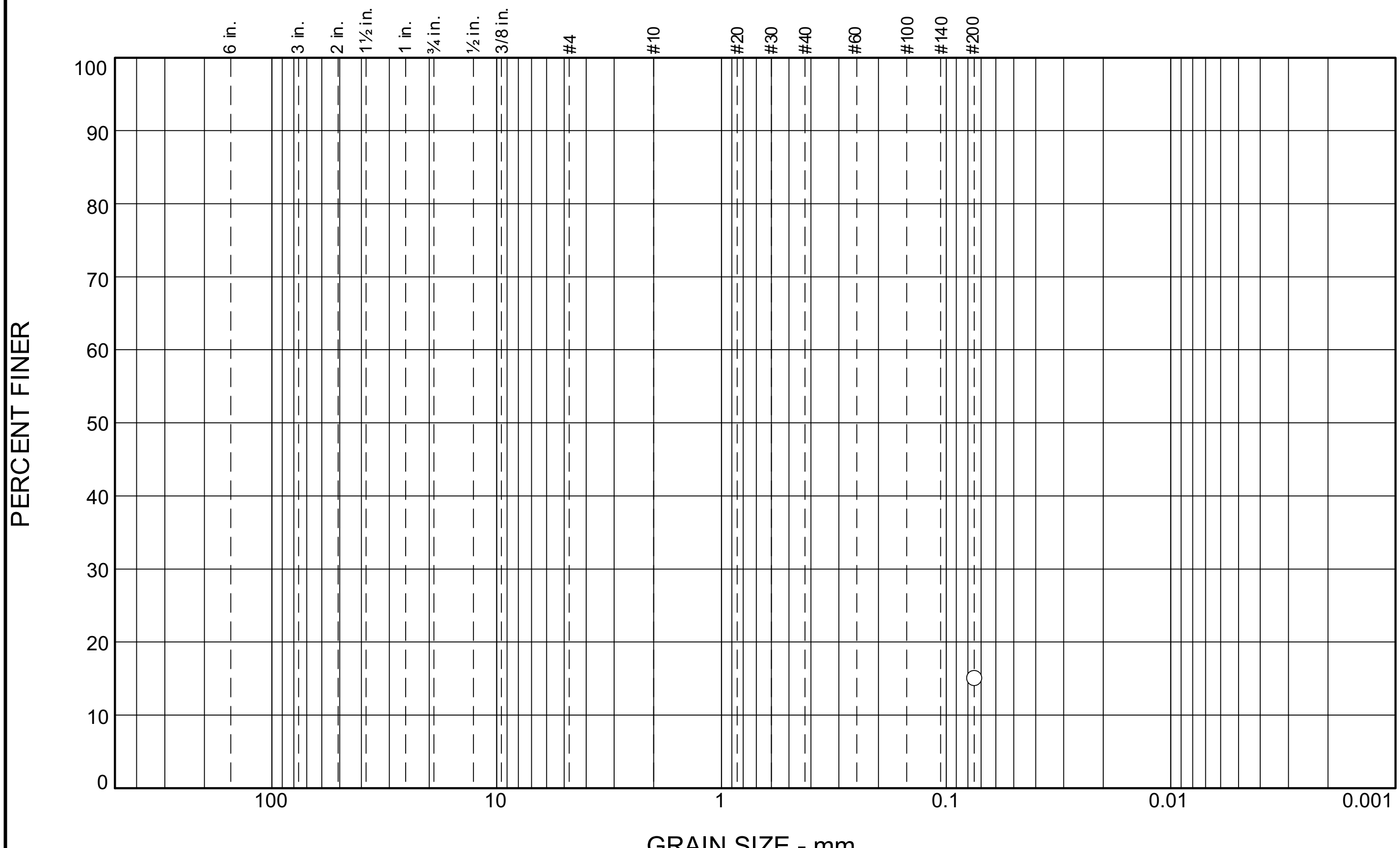
ENGEO INCORPORATED

Client: Lennar Urban
Project: HPS2/CP Geotechnical Exploration, SF, CA

Project No: 8472.001.001

Tested By: AV **Checked By:** DS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						15.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	15.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Sample Number: 2-CP-B3 @ 6 **Depth:** 6.0 feet

Date: 4.4.13

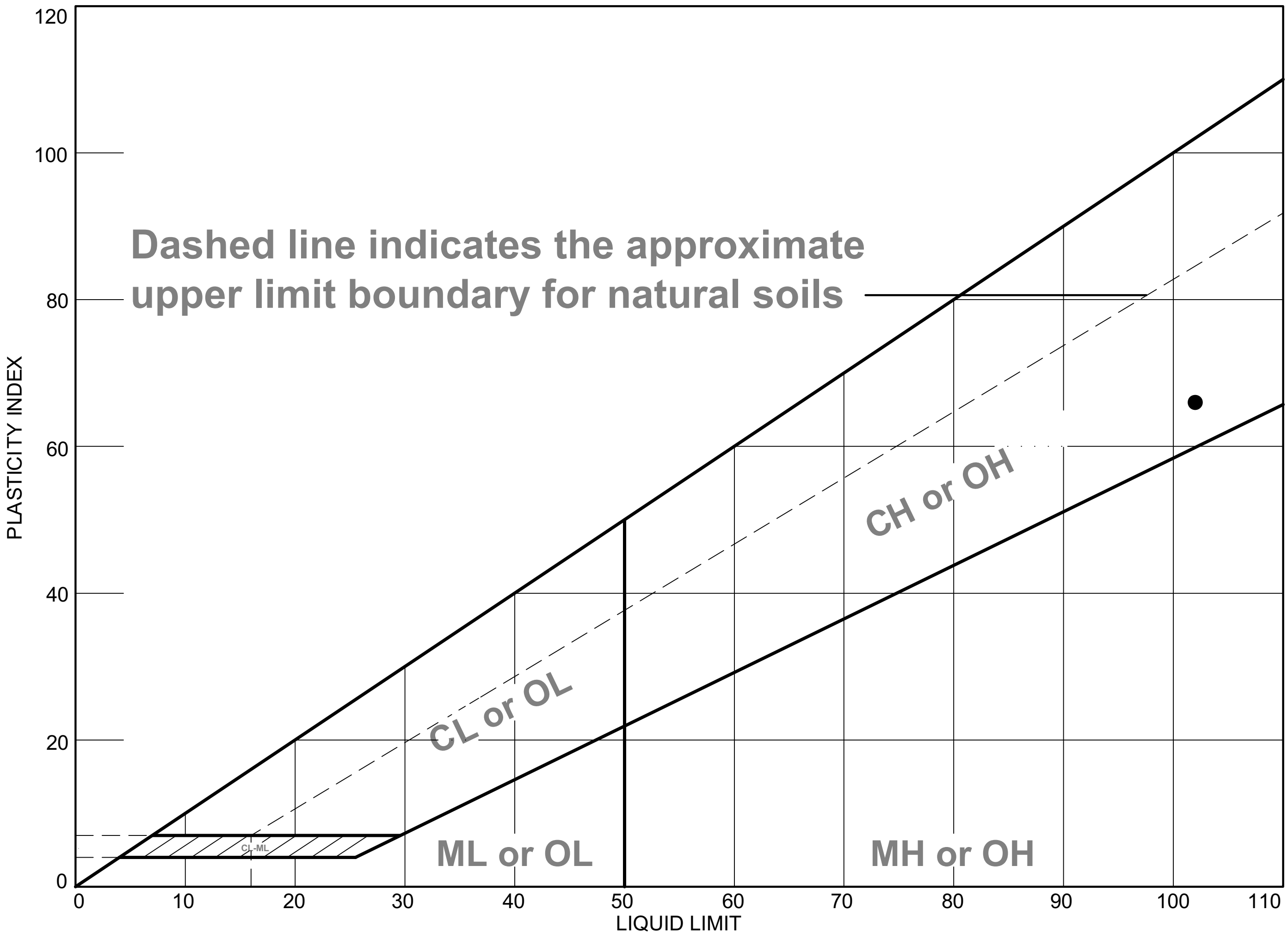
ENGEO INCORPORATED

Client: Lennar Urban
Project: HPS2/CP Geotechnical Exploration, SF, CA

Project No: 8472.001.001

Tested By: AV **Checked By:** DS

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	102	36	66			

Project No. 8472.001.001 **Client:** Lennar Urban

Project: HPS2/CP Geotechnical Exploration, SF, CA

● **Depth:** 24.0 feet **Sample Number:** 2-CP-B3 @ 24

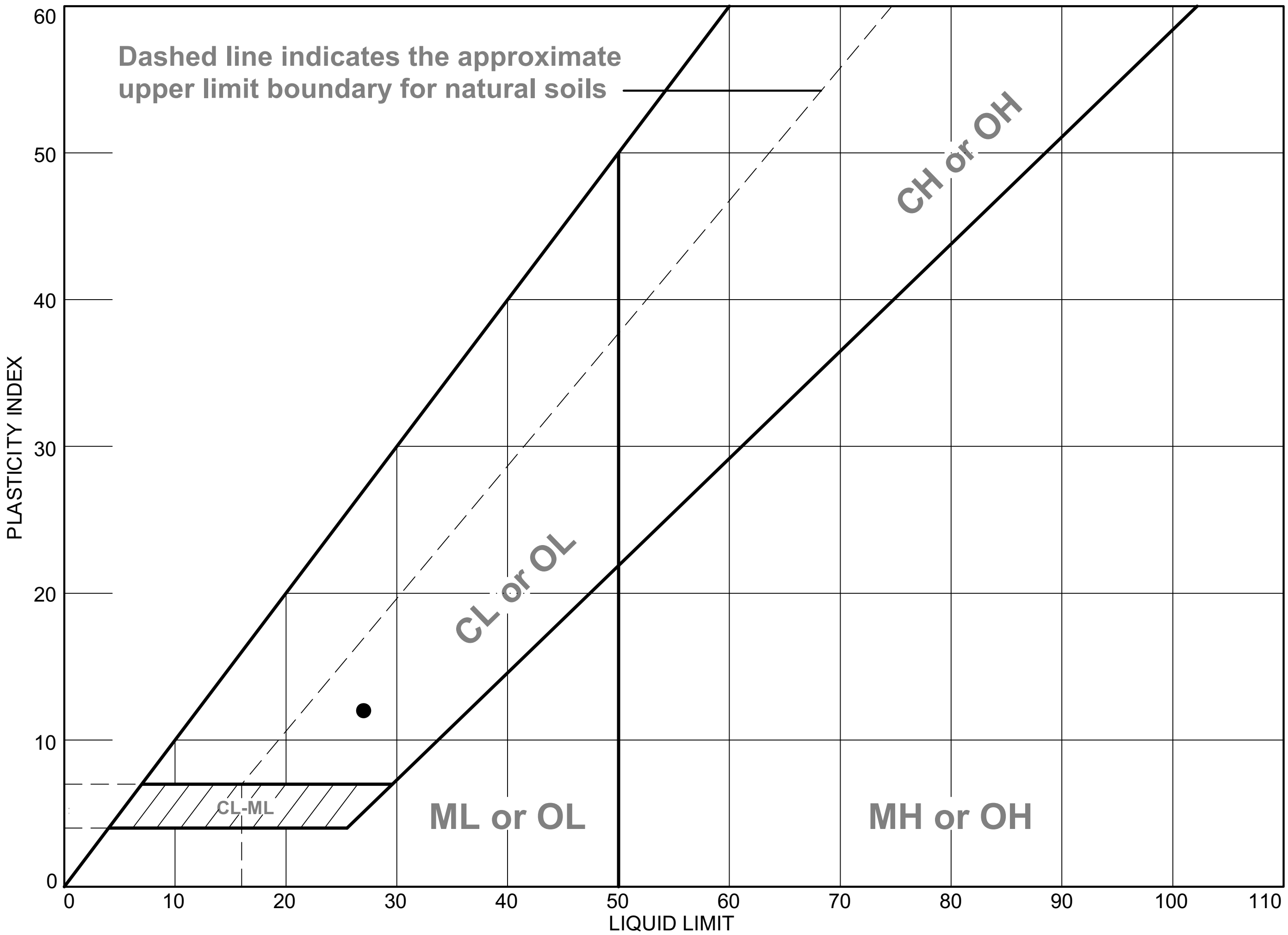
ENGEO INCORPORATED

Remarks:

● PI: ASTM D4318

Tested By: GC **Checked By:** DS

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	27	15	12			

Project No. 8472.001.001 **Client:** Lennar Urban

Project: HPS2/CP Geotechnical Exploration, SF, CA

● **Depth:** 3.5 feet **Sample Number:** 2-CP-B3 @ 3.5

Remarks:

● PI: ASTM D4318

ENGEO INCORPORATED

Tested By: AV **Checked By:** DS

EN GEO

Unconsolidated Undrained Triaxial Test (ASTM D2850)

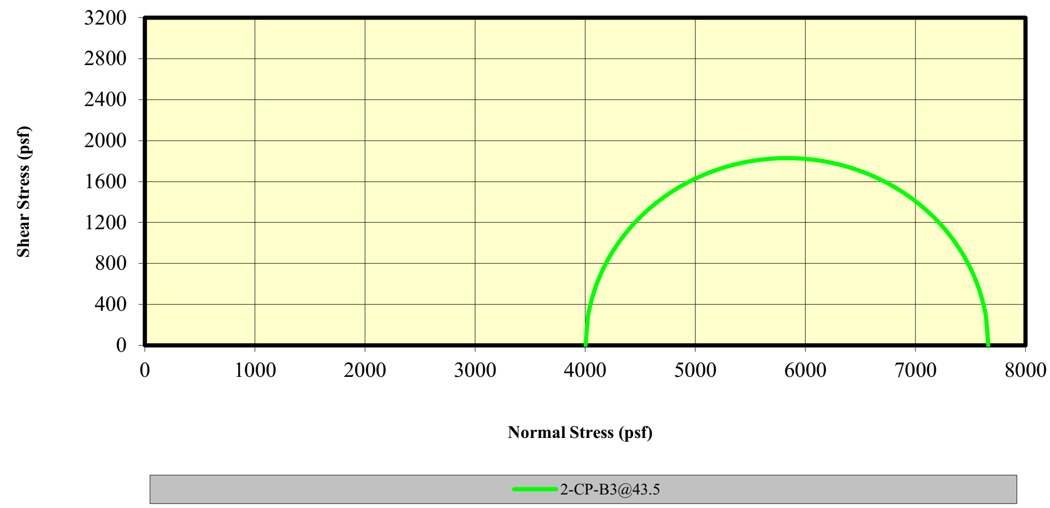
Date: 4.4.13

Checked By: D. Seibold

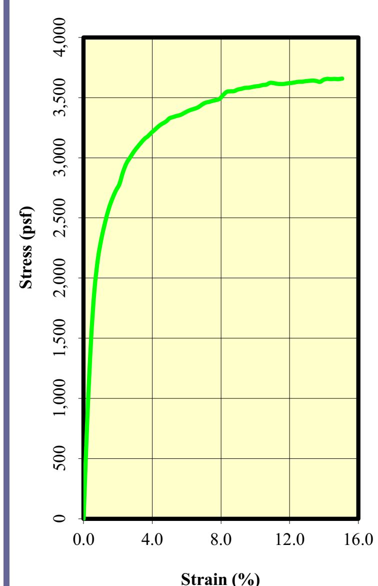
Date: 4.4.13

Tested By: G. Criste

Mohr Circles



Stress-Strain Curve



Before Test	Specimen B3@43.5			
Water Content (%)	22.58			
Dry Density (pcf)	106.22			
Saturation (%)	99.60			
Void Ratio	0.63			
Diameter (in)	2.407			
Height (in)	5.010			
Liquid Limit	-			
Plastic Limit	-			
Specific Gravity	2.770			
Height-to-Diam. Ratio	2.081			
After Test	**B3@43.5**			
Water Content (%)	22.58			
Saturation (%)	99.60			
Strain Rate (in/min)	0.05			
Peak Deviator Stress (psf)	3657.7			
Axial Strain @ Failure (%)	15.068			
	Cell Pressure			
Cell (psf)	4003.2			
Back (psf)	n/a			
	Principle Stresses at Failure			
σ1 (psf)	7660.9			
σ3 (psf)	4003.2			

Mohr-Coulomb Strength Parameters		Sample Description
Cohesion, c (psf)	1828.9	See Exploration Logs
Friction Angle Ø	0.00	

Project Information			
Project Name:	Allice Griffith		
Project Number:	8472.001.001 Phase AG	Job Number:	8472.001.001
Location:	Candlestick Park	Boring Number:	2-CP-B3
Client:	Lennar Urban	Sample Number:	2-CP-B3
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-CP-B3 @ 24.0	N	24.0	3	558

PROJECT NAME: Candlestick Park Retail
DATE: 04/03/13
PROJECT NUMBER: 8472.001.001
CLIENT: Lennar Urban
PHASE NUMBER: 002

Tested by: GC
Reviewed by: DS

ENGEO INCORPORATED

APPENDIX C

Seismic Refraction Report (NorCal Geophysical)

APPENDIX C

NORCAL GEOPHYSICAL CONSULTANTS, INC.

June 26, 2013

ENGEO
332 Pine Street, Suite 300
San Francisco, California 94104

Subject: Seismic Refraction Survey
Candlestick Point
San Francisco, California
NORCAL Project No. 13-241.51

Attention: Mr. Leroy Chan

This report presents the findings of a seismic refraction survey performed by NORCAL Geophysical Consultants, Inc. for ENGEO at Candlestick Point in San Francisco, California. The geophysical survey was conducted on May 28, 2013 by California Professional Geophysicist William E. Black (PGp No. 843) and Senior Geophysical Technician Travis W. Black. Site logistical support and property owner liaison was provided by Messrs. Eugenio Diaz and Leroy Chan of Engeo.

The seismic refraction survey was conducted at two sites within the Candlestick Point area of San Francisco, California. One is the stadium at Candlestick Park and the other is a tenement housing area referred to as the "Alice Griffith Parcel". The general locations of both sites are shown on the index maps included on Plates 1 and 3. The purpose of this seismic refraction survey is to measure the depth, configuration, and seismic compressional (P-) wave velocity of the bedrock underlying both sites.

1.0 METHODOLOGY

The seismic refraction method is used to determine the seismic velocity structure of the subsurface. Compressional (P) wave energy generated by an impulsive source at the surface propagates into the earth. When the P-waves encounter an increase in seismic velocity, they are refracted along the interface and back to the surface where they are detected by a collinear array of geophones. The detected signals are recorded on a multi-channel seismograph and are analyzed to determine the shot point-to-geophone travel times. These data can be used along with the corresponding shot point-to-geophone distances to determine the depth, thickness, and velocity of subsurface seismic layers.

2.0 DATA ACQUISITION

We collected seismic refraction data using 24-geophones and 7-shot points distributed in collinear arrays (spreads). Seismic energy was produced at each shot point using multiple impacts with either a 16# sledge hammer, or an accelerated weight drop, against a metal plate placed on the ground surface. The device used depended on site access. The resulting

compressional (P-) wave energy was detected by ***OYO Geospace*** geophones with a natural frequency of 8-Hz and recorded using a Geometrics ***Geode*** distributed array seismic system (photo at left). This instrument has 24-channels with 24 bit A/D converters. However, it can be networked with additional units to expand the system up to 120-channels. The Geode was networked to a field computer which was used to display the recorded wave forms and to archive the seismic data.

3.0 DATA PROCESSING

The seismic refraction data were processed using the computer program ***SeisImager*** by Geometrics, Inc. This is an interactive program that is used to determine the shot point to geophone travel times, and to compute a 2D model based on those times. Once we had determined the travel times for a given line, we used the programs time-term algorithm to compute a preliminary 2D seismic model. We then used that model as input for the programs tomographic routine. Using this procedure, the program divided the starting model into a network of cells and assigned velocities to those cells based on the starting model. The program then traced refracted seismic travel paths through those cells and computed the associated travel times. It then compared the computed travel times with the measured times and adjusted the velocities of the appropriate cells to improve the fit. We programmed the software to continue this procedure for 30-iterations. At the end of the 30-iterations the travel times associated with the computed model matched the observed travel times to an accuracy of 1.5 milliseconds (msec) or better. Once a satisfactory model was computed, we used the computer program ***Surfer 11.0*** by Golden Software, Ltd of Golden, Colorado to contour the modeled data to produce a color contoured cross-section illustrating the distribution of seismic velocity vs. depth and distance.

The seismic refraction surveys conducted at Candlestick Park and the Alice Griffith Parcel are described in the following sections. Each section includes a description of the specific data acquisition procedures used at each site, a presentation of the results and a discussion regarding our interpretation of the results.

4.0 CANDLESTICK PARK SEISMIC REFRACTION SURVEY

4.1 SITE DESCRIPTION

Candlestick Park consists of a football stadium, the surrounding paved parking lots and a nearby camp ground. An aerial photograph of the stadium is shown on Plate 1. The general location of the stadium is shown in the vicinity map included on the plate. The seismic refraction survey was conducted inside the stadium and extended out through one of the tunnels into the parking lot. The floor of the stadium was flat and open and covered by closely cropped grass. The floor of the tunnel was concrete paved and sloped slightly towards the northeast. The parking lot outside the stadium was basically flat, open and asphalt paved.

Two test borings were installed inside the stadium by others. The locations of these borings, labeled 2-CP-B4 and 2-CP-B5, are shown on Plate 1. The geologic logs from the borings, as provided by Engeo, indicate that the upper 10- to 16-ft of the subsurface consists of 2- to 4-ft of poorly graded sand overlying Franciscan Formation bedrock. In B-4 the rock consisted of serpentine from depths of 2- to 9-ft overlying shale to the bottom of the boring at 10-ft. In B-5 the rock consisted of greywacke from a depth of 4-ft to the bottom of the boring at 16-ft.

4.2 DATA ACQUISITION

We collected seismic refraction data along a 710-ft long transect (line) that trended diagonally (southwest to northeast) across the football field and through a tunnel into the parking lot on the northeast side of the stadium. The location of the line, labeled Line 1, is represented by the red line shown on Plate 1. Line 1 comprised three end-to-end seismic spreads. Each spread consisted of 24-geophones and 7-shot points distributed in a collinear array. The geophones were distributed at 10-ft intervals and the shot points were distributed at 40- to 45-ft intervals beginning 10-ft from the first geophone in the array and ending 10-ft past the last geophone. The spreads were distributed so that the last geophone in the first spread became the first geophone in the second spread. Similarly, the last geophone in the second spread served as the first geophone in the last spread. A portion of the geophone array is shown in Figure 1.

Seismic energy was produced at each shot point using a Digipulse AWD-100 mounted on the back of an all-terrain vehicle (ATV), as shown in Figure 2. This device consists of a cylindrically shaped 100# weight that is dropped from a height of approximately 24-inches onto a 12" x 12" aluminum plate on the ground surface. Large elastic bands attached to the weight cause it to accelerate as it drops. The weight is lifted into place for each drop by a battery powered hydraulic ram. An accelerometer attached to the strike plate transmits a triggering pulse to the seismograph each time the weight strikes the plate.

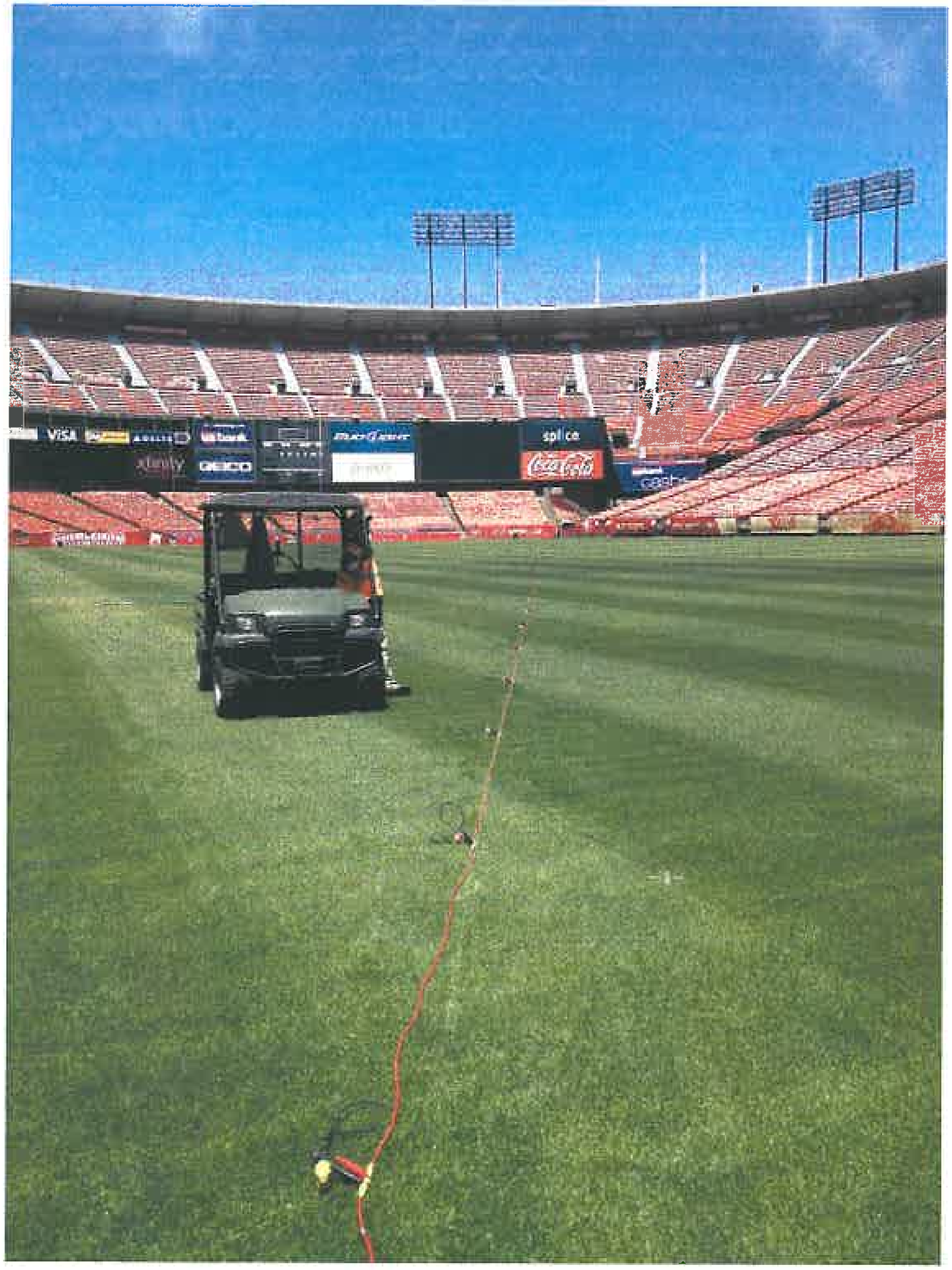


Figure 1: A portion of Line 1 looking southwest. Yellow objects on ground surface are geophones.

Figure 2: AWD-100 mounted on ATV

4.3 RESULTS

The results of the seismic refraction survey are illustrated by the seismic velocity cross-section (profile) for Line 1 shown on Plate 2. The color shaded contours on this profile indicate the distribution of P-wave velocities with depth and distance beneath the seismic line. The relationship between contour color and velocity is indicated by the velocity scale shown at the bottom of the profile. The P-wave velocity contours are relatively uniform and flat-lying except over the last 250-ft of the line where they dip to the northeast. The contours also indicate that the P-wave velocity increases rapidly with depth, from as low as 1,000-ft/sec at the surface to in excess of 14,000-ft/sec at depth. Correlation of the P-wave contours with the geologic logs from borings 2-CP-B4 and 2-CP-B5 suggest that velocities less than 5,000-ft/sec (brown to yellow contours) probably represent poorly graded sand and that the remainder of the profile (green, blue and purple contours) represents bedrock. That being the case, the variations in velocity within the bedrock sequence are probably related to variations in the degree to which the rock is weathered. This relationship is that the higher the velocity of the rock, the less weathered it is. The wide range of velocities suggests a wide variation in weathering, from moderately weathered (green to blue contours) at the top of the bedrock sequence to little weathered (purple contours) at depths ranging from about 5- to 40-ft.

5.0 ALICE GRIFFITH PARCEL SEISMIC REFRACTION SURVEY

5.1 SITE DESCRIPTION

The Alice Griffith Parcel (AGP) is a tenement housing project located in the Candlestick Point area of San Francisco, California. An aerial photograph of the area is shown on Plate 3.The vicinity map included on the plate shows the general location of the AGP. The seismic refraction survey was conducted in a "V" shaped lot between tenement buildings enclosed by Nichols Street on the northeast, southeast and southwest and by Cameron Way on the northwest. Each arm of the V is approximately 250-ft long and 45- to 65-ft wide. The ground surface within the west arm slopes downward to the southwest and the surface in the east arm slopes downward to the southeast. The ground cover in the survey area ranged from sparse, short, dry grass to exposed soil or rock.

5.2 DATA ACQUISITION

Seismic refraction data were collected along two transects (lines) labeled Line 2 and Line 3, as shown on Plate 3. Each line was 200-ft long and consisted of 7-shot points and 24-geophones distributed in a collinear array. The geophones were distributed at 8-ft intervals and the shot points were distributed at 32- to 36-ft intervals starting 8-ft from the first geophone and ending 8-ft beyond the last geophone in the array. Seismic energy was produced at each shot point through multiple impacts with a 16#-sledge hammer against a metal plate placed on the ground surface.

5.3 RESULTS

The results of the seismic refraction survey are illustrated by the seismic velocity cross-sections (profiles) for Lines 2 and 3 shown on Plate 4. The color shaded contours on these profiles indicate the distribution of P-wave velocities with depth and distance beneath the seismic lines. The relationship between contour color and velocity is indicated by the velocity scale shown at the bottom of the plate. The contours are relatively uniform and flat-lying and indicate a gradual increase in velocity with depth; from around 1,000-ft/sec at the surface to over 11,000-ft/sec at depths of 40- to 45-ft. Since rock is exposed at the surface in the survey area, specifically along Line 2, it is our interpretation that the seismic velocity profiles shown on Plate 2 primarily represent bedrock and that the variations in velocity are related to variations in the degree to which the rock is weathered. This relationship is that the higher the velocity of the rock, the less it is weathered. The wide range of velocities suggests a wide variation in weathering, from decomposed at the surface (brown to yellow contours) to little weathered at depth (purple contours).

6.0 EXCAVATION CHARACTERISTICS

We specifically differentiated the velocity ranges comprising the P-wave velocity models shown on Plates 2 and 4 to coincide with the excavation characteristics (rippability) of the subsurface materials. This is based on information published by the Caterpillar Tractor Company for different types of excavating equipment (rippers) operating in different types of materials. The geologic logs from borings 2-CP-B4 and 2-CP-B5 at Candlestick Park indicate that the bedrock consists of Franciscan greywacke, serpentine and shale. We assume that the bedrock at the Alice Griffith Parcel is essentially the same. However, which ripper will be used for excavation is unknown at this time. Therefore, we have assumed, for estimating purposes, that the ripping equipment will consist of a Caterpillar multi or single shank No. 9 Ripper (D9R/D9T). Given these parameters, P-wave velocities ranging from 1,000- to 7,500-ft/sec (yellow to green contours) are rippable, velocities ranging from 7,500- to 9,500-ft/sec (blue contours) are marginally rippable and velocities in excess of 9,500-ft/sec (purple contours) are non-rippable. These velocity ranges should only be used as a general guideline. Actual ripping performance will depend on the bedding, induration, fracturing and jointing of the rock, the condition of the equipment and the skill of the operator.

6.1 Candlestick Park

According to the P-wave velocity profile shown on Plate 2, rippable material (green to yellow contours) is only about 10-ft thick beneath the first 460-ft of the profile. However, continuing northeastward it thickens considerably, reaching a thickness of about 40-ft at the northeast end of the line. Marginally rippable rock (blue contours) ranges in thickness from about 5- to 15-ft over the first 450-ft of the line where it is about 10-ft deep. However, proceeding northeastward the marginally rippable rock thickens to as much as 20-ft as its depth increases to 40-ft. Finally, non-rippable rock (purple contours) is 10- to 20-ft deep beneath the first 440-ft of the line. However, northeast of that point it drops off steeply, reaching a depth of 55-ft by Station 550-ft and maintaining that depth to the northeast end of the line.

6.2 Alice Griffith Parcel

According to the P-wave velocity profiles shown on Plate 4, rippable material (yellow to green contours) is 15- to 20-ft thick beneath most of Line 2 except at the southeast end of the line where it decreases in thickness to about 3-ft. Marginally rippable rock (blue contours) is about 17- to 22-ft deep beneath most of the Line 2 but comes within 3-ft of the surface at the southeast end of the line. Non-rippable rock (purple contours) is at depths ranging from 25- to 30-ft beneath the entire length of the line. A similar situation exists beneath Line 3, except at greater depths. Here, the rippable rock (yellow to green contours) is 25- to 30-ft thick beneath most of the line except at the northeast end where it thins to about 5-ft. Marginally rippable rock (blue contours) is 5- to 15-ft thick and occurs at depths of 25- to 30-ft beneath most of the line except for the northeast end where it rises to within 5-ft of the surface. Non-rippable rock (purple contours) is only defined beneath the central to northeast portion of the line where it ranges in depth from 35-to 45-ft.

7.0 LIMITATIONS

It should be noted that the seismic refraction technique is based on the assumption that seismic velocity increases with depth. Any layers representing a decrease in velocity with depth, otherwise known as a velocity inversion, will not be defined and will result in the over-estimation of the depth of deeper, higher velocity layers. In addition, relatively thin layers might not be individually resolved and might, instead, be lumped together with other layers. Hard and soft zones within a given seismic layer will tend to be averaged into the velocity of that layer. Finally, there is not necessarily a one-to-one relationship between lithologic layers and seismic layers. It is entirely possible that two different types of material could have the same velocity. Alternatively, a change in velocity can occur within a single lithologic unit.

8.0 STANDARD CARE AND WARRANTY

The scope of NORCAL's services for this project consisted of using geophysical methods to characterize the shallow subsurface. The accuracy of our findings is subject to specific site conditions and limitations inherent to the techniques used. We performed our services in a manner consistent with the level of skill ordinarily exercised by members of the profession currently employing similar methods. No warranty, with respect to the performance of services or products delivered under this agreement, expressed or implied, is made by NORCAL.

We appreciate the opportunity to provide our services to Engeo, Inc. on this project. If you have any questions, or require additional geophysical services, please do not hesitate to call.

Sincerely,

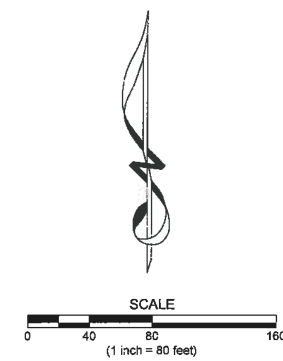
NORCAL Geophysical Consultants, Inc.

William E. Black
Professional Geophysicist PGp-843

WEB/tlt

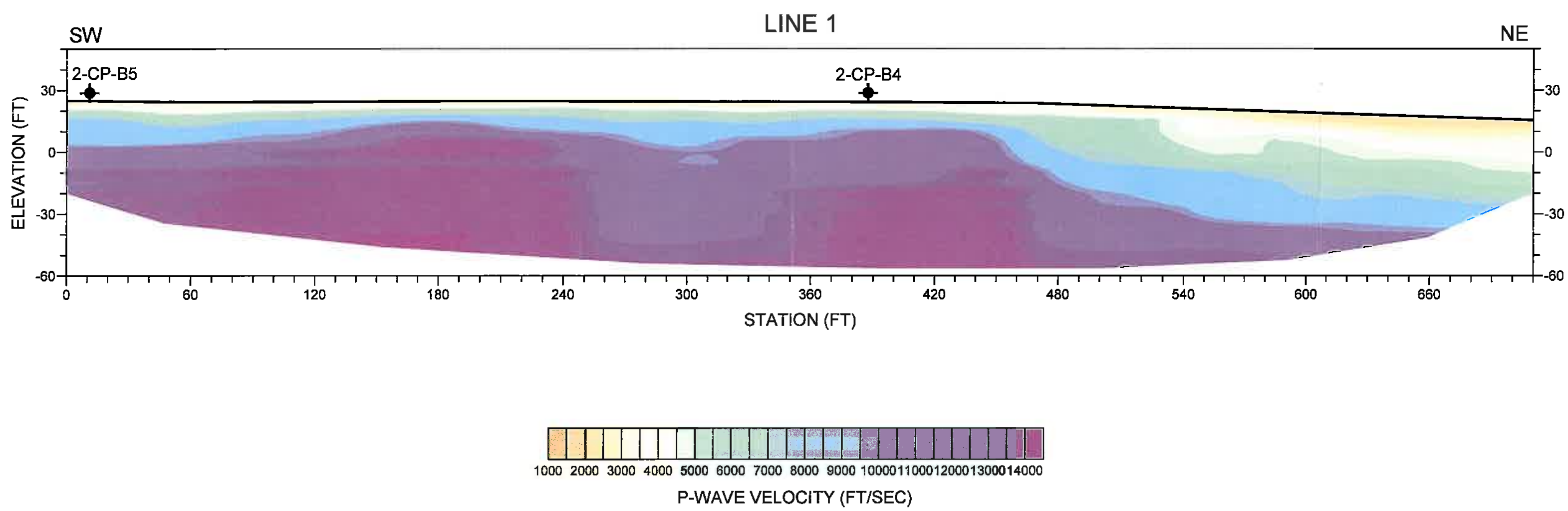
Enclosures: Plates 1 – 4

VICINITY MAP



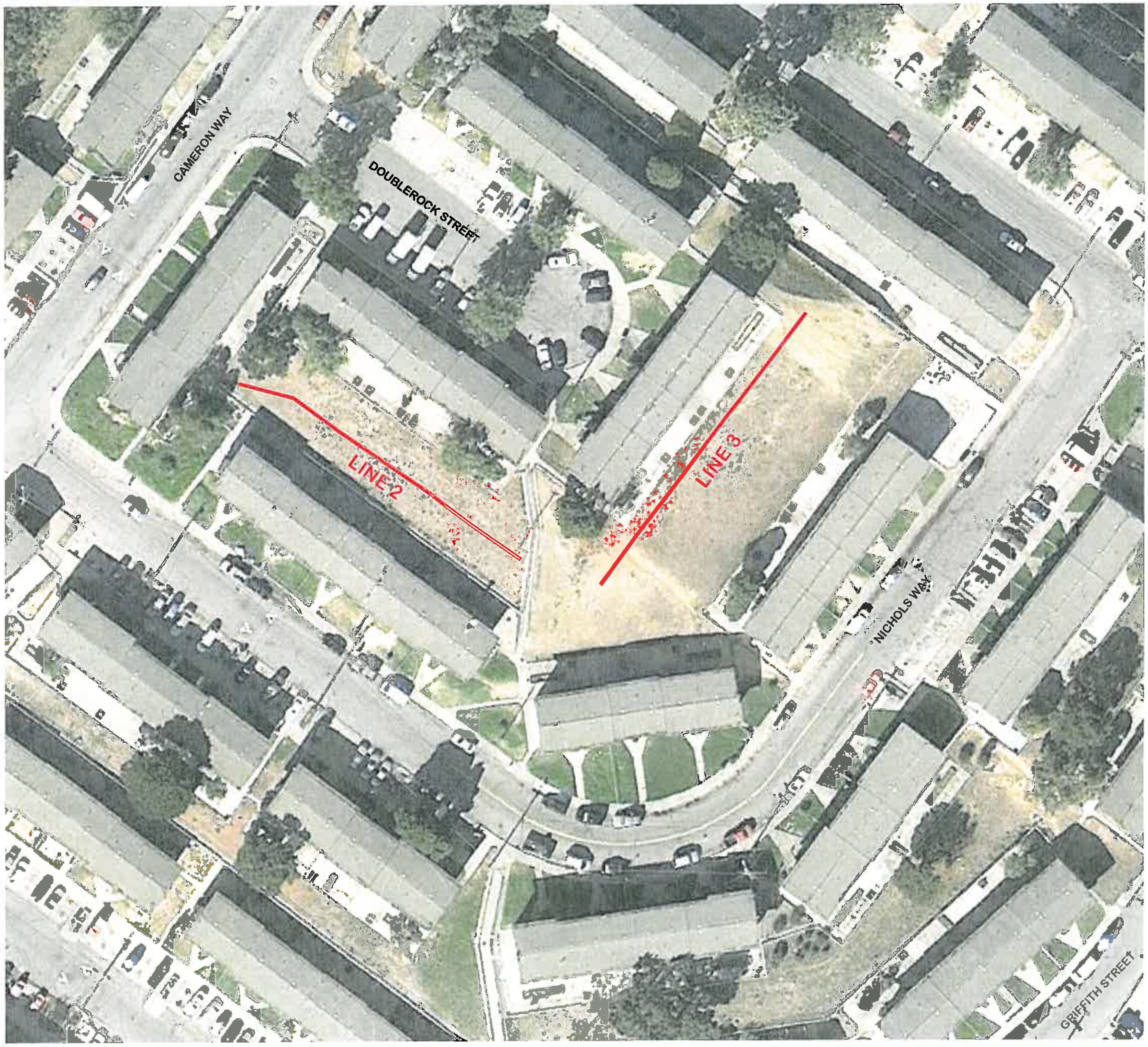
LEGEND	
(red line)	SEISMIC REFRACTION LINE
(boring symbol)	BORING

NORCAL	SITE LOCATION MAP SEISMIC REFRACTION SURVEY CANDLESTICK PARK	
	LOCATION: SAN FRANCISCO, CALIFORNIA	
	CLIENT: ENGEO	PLATE 1
JOB #: 13-241.51	NORCAL GEOPHYSICAL CONSULTANTS INC.	
DATE: JUN. 2013	DRAWN BY: G.RANDALL APPROVED BY: WEB	

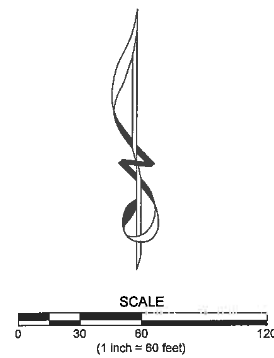


LEGEND	
✦	BORING

NORCAL	LINE 1 SEISMIC REFRACTION SURVEY CANDLESTICK PARK	
	LOCATION: SAN FRANCISCO, CALIFORNIA	
	CLIENT: ENGEO	PLATE
JOB #: 13-241.51	NORCAL GEOPHYSICAL CONSULTANTS INC.	2
DATE: JUN. 2013	DRAWN BY: G.RANDALL / APPROVED BY: WEB	



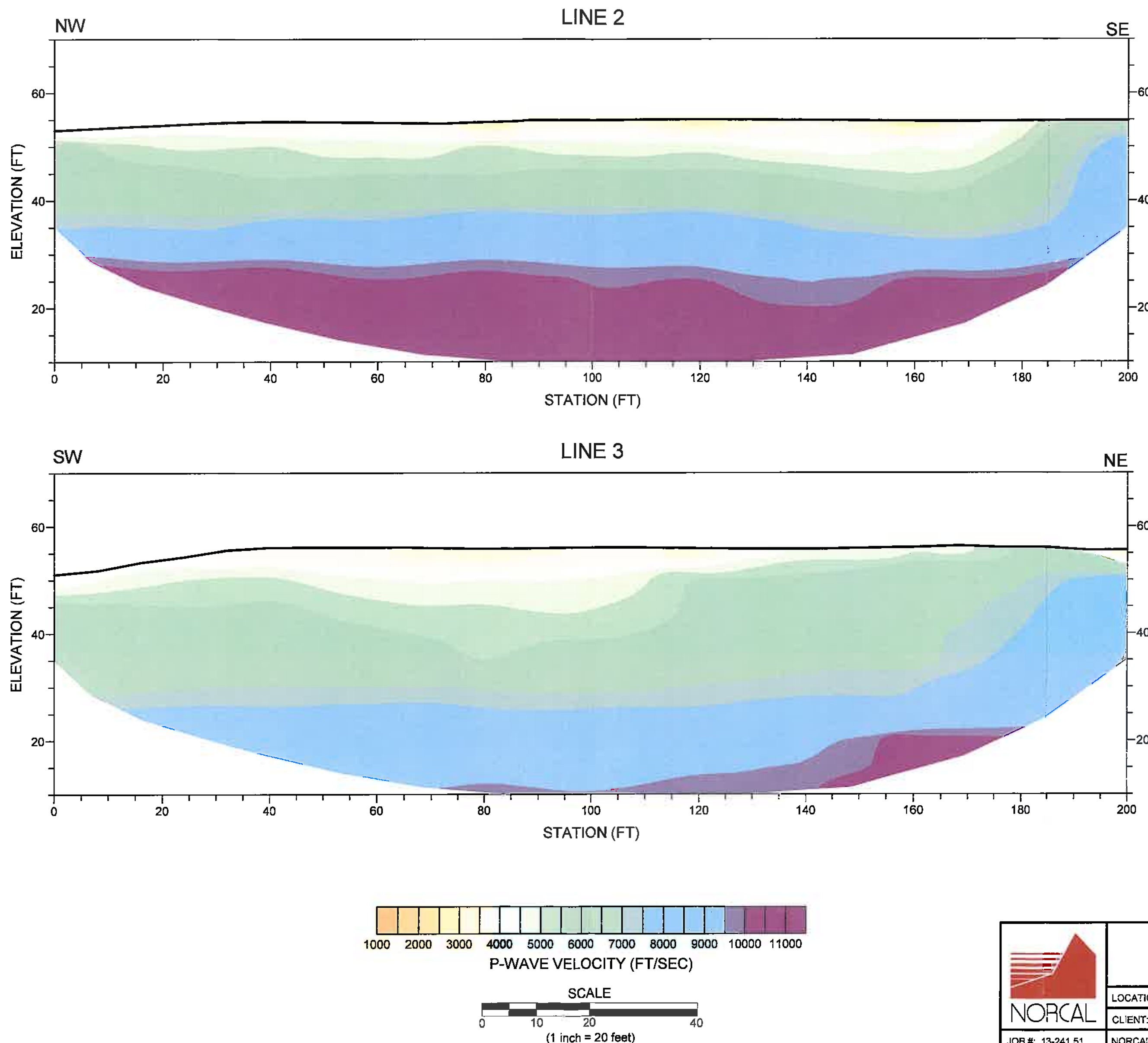
VICINITY MAP



LEGEND	
	SEISMIC REFRACTION LINE

SITE LOCATION MAP
SEISMIC REFRACTION SURVEY
ALICE GRIFFITH PARCEL

NORCAL	LOCATION: SAN FRANCISCO, CALIFORNIA		
	CLIENT: ENGEO		PLATE 3
JOB #: 13-241.51	NORCAL GEOPHYSICAL CONSULTANTS INC.		
DATE: JUN. 2013	DRAWN BY: G.RANDALL	APPROVED BY: WEB	



NORCAL	LINES 2 & 3 SEISMIC REFRACTION SURVEY ALICE GRIFFITH PARCEL	
	LOCATION: SAN FRANCISCO, CALIFORNIA	
	CLIENT: ENGEO	PLATE
JOB #: 13-241.51	NORCAL GEOPHYSICAL CONSULTANTS INC.	4
DATE: JUN. 2013	DRAWN BY: G.RANDALL APPROVED BY: WEB	

APPENDIX D

Previous Explorations (ENGEO)

A P P E N D I X D

LOG OF BORING CP-B5

ENGEO INCORPORATED

Geotechnical Feasibilty
CANDLESTICK POINT
SAN FRANCISCO, CA
8472.000.001

DATE DRILLED: 11/19/2009
HOLE DEPTH: Approx. 30½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV: Approx. 2½ ft.

LOGGED / REVIEWED BY: L. Chan / BHBJ
DRILLING CONTRACTOR: WDC Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
		CLAYEY SILT (ML), yellow-brown, moist, with subrounded fine gravel, with coarse subangular sand. (FILL)								
	1	Trace fill gravel, with coarse subangular sand.								
5	2	SANDY SILT (ML), brown, moist, loose, medium sand, trace subround fine gravel. (FILL)	12					9.5		1.5
10	3	SILTY SAND (SM), brown, wet, very loose, coarse grained sand, with fine subangular gravel. (FILL)	5	21	25	NP	19	11.9	93.5	
	4		7							
15	5	FAT CLAY (CH), olive grey, wet, soft. (YOUNG BAY MUD)								
		No Recovery with Shelby Tube. SILTY SAND (SM), grey, wet, loose, medium sand. (ALLUVIUM)	9							
20	6	Heaving sand.	16	21	18	3	8			
		SAND (SP-SM), yellow brown, wet, medium dense, with silt. (ALLUVIUM)								
25	7, 8	SAND (SW-SM), dark brown, wet, very dense, fine to coarse grained sand, trace fine gravel. (ALLUVIUM)	46				10			

LOG - GEOTECHNICAL 8472000001 GINT LOGS.GPJ ENGEO INC.GDT 9/30/11

LOG OF BORING CP-B5

ENGEO INCORPORATED

Geotechnical Feasibilty
CANDLESTICK POINT
SAN FRANCISCO, CA
8472.000.001

DATE DRILLED: 11/19/2009
HOLE DEPTH: Approx. 30½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV: Approx. 2½ ft.

LOGGED / REVIEWED BY: L. Chan / BHBJ
DRILLING CONTRACTOR: WDC Exploration
DRILLING METHOD: 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)	Unconfined Strength (tsf) *field approx
30	9		SAND (SW-SM), dark brown, wet, dense, fine to coarse grained sand, trace fine gravel. (ALLUVIUM)			31							
			Bottom of Boring at approximately 30½ feet below ground surface. Groundwater encountered at approximately 11 feet below ground surface at time of drilling.										

ENGEO INCORPORATED

LOG OF BORING CP-B7

Geotechnical Feasibilty
CANDLESTICK POINT
SAN FRANCISCO, CA
8472.000.001

DATE DRILLED: 11/25/2009
HOLE DEPTH: Approx. 65½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV: Approx. 4 ft.

LOGGED / REVIEWED BY: L. Chan / BHBJ
DRILLING CONTRACTOR: WDC Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Depth in Meters	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
		SANDY SILT (ML), brown, moist, trace coarse sand, trace brick fragments. (FILL)								
	1									
5		SILTY CLAY (CL), reddish brown, moist, stiff, with coarse subrounded sand, some greenstone fragments. (FILL)	13					11.3	107.6	2.50*
	2	Fragments of asphalt from cuttings.								
10	3		23	25	13	12				
	4	SILTY GRAVEL (GM), olive grey to black, wet, medium dense, with sand, lots of organics and wood debris, with concrete fragments. Highly organic. (FILL)								
15	5		21	26	18	8	23			
20	6	SANDY CLAY (CH), dark grey, wet, soft, fine sand, trace shell fragments, trace sandstone fragments. (YOUNG BAY MUD)	5					17.6	115.2	
	7	CLAY (CH), olive grey, wet, soft. (YOUNG BAY MUD)								
25	8	TX UU - su = 662 psf								3.50*

ENGEO INCORPORATED

LOG OF BORING CP-B7

Geotechnical Feasibilty
CANDLESTICK POINT
SAN FRANCISCO, CA
8472.000.001

DATE DRILLED: 11/25/2009
HOLE DEPTH: Approx. 65½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV: Approx. 4 ft.

LOGGED / REVIEWED BY: L. Chan / BHBJ
DRILLING CONTRACTOR: WDC Exploration
DRILLING METHOD: 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)	Unconfined Strength (tsf) *field approx
			SANDY CLAY (CH), dark grey, wet, soft, fine sand. (YOUNG BAY MUD)										
	9												
30													
						7							0.50*
	10												
35			SILTY SAND (SM), olive grey, wet, loose, fine sand. (ALLUVIUM)										
	11												
	12												
			SILTY SAND (SM), yellow brown, wet, loose, fine sand. (ALLUVIUM)										
40							19	19	NP	17			
			SILTY SAND (SM), yellow brown, wet, dense, fine sand. (ALLUVIUM)										
						52							
							18	18	NP	19			
	13												
						8							
45							24	17	7	34			
			CLAYEY SAND (SC), yellow brown, wet, loose. (ALLUVIUM)										
	14												
			SILTY SAND (SM), light olive grey, wet, medium dense, medium sand. (ALLUVIUM)										
	15												
						30							
50													
	16												
			Stiffer drilling.										
			GREYWACKE, weak, highly weathered, very thin bedding. (FRANCISCAN BEDROCK)										
55													
	17												

LOG OF BORING CP-B7

ENGEO INCORPORATED

Geotechnical Feasibilty
CANDLESTICK POINT
SAN FRANCISCO, CA
8472.000.001

DATE DRILLED: 11/25/2009
HOLE DEPTH: Approx. 65½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV: Approx. 4 ft.

LOGGED / REVIEWED BY: L. Chan / BHBJ
DRILLING CONTRACTOR: WDC Exploration
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

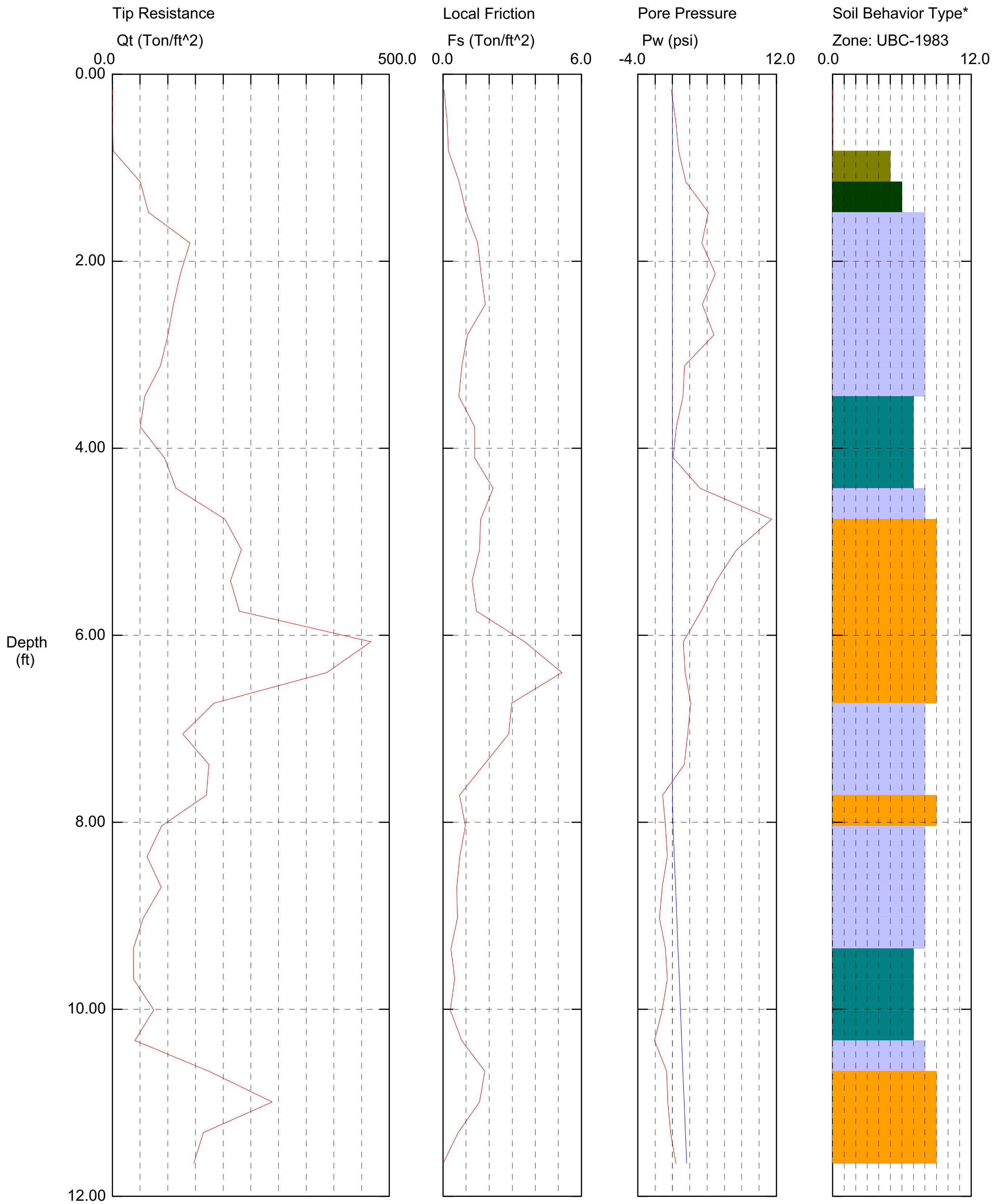
Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			GREYWACKE, weak, highly weathered, very thin bedding. (FRANCISCAN BEDROCK)										
	18		No recovery.			50/3"							
60						50/5"							
	19												
65						50/3"							
			Bottom of boring at approximately 65½ feet below ground surface. Groundwater not encountered during drilling.										

LOG - GEOTECHNICAL 8472000001 GINT LOGS.GPJ ENGEO INC.GDT 9/30/11

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

Candlestick Park CPT

Operator: JT	CPT Date/Time: 11-20-09 08:55
Sounding: CPT-06	Location: CPT-06
Cone Used: 4583.124	Job Number:



Maximum Depth = 11.65 feet

Depth Increment = 0.33 feet

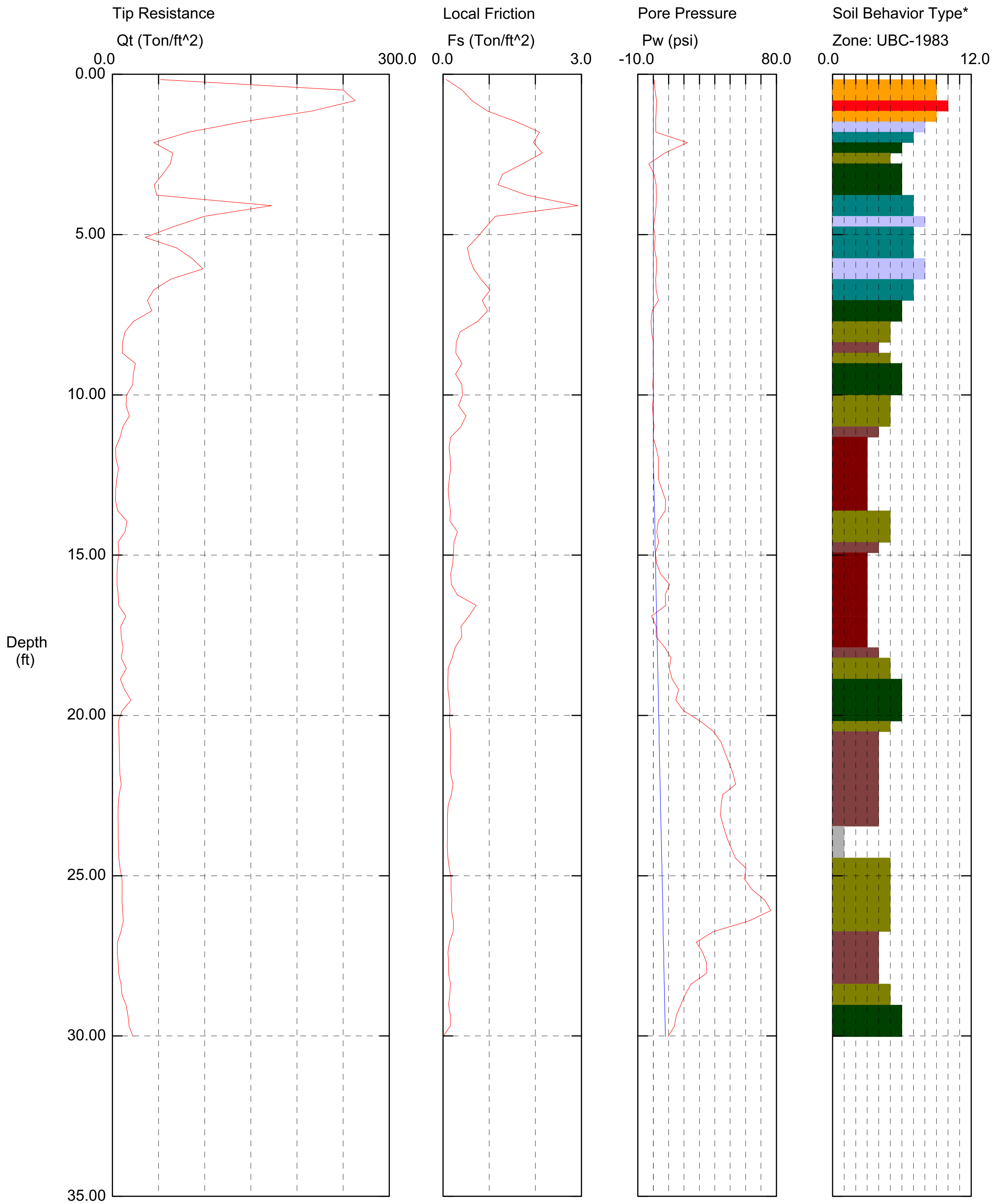
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

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

Candlestick Park CPT

Operator: JT
Sounding: CPT-09
Cone Used: 4583.124

CPT Date/Time: 11-25-09 12:10
Location: CPT-09
Job Number:



Maximum Depth = 30.02 feet

Depth Increment = 0.33 feet

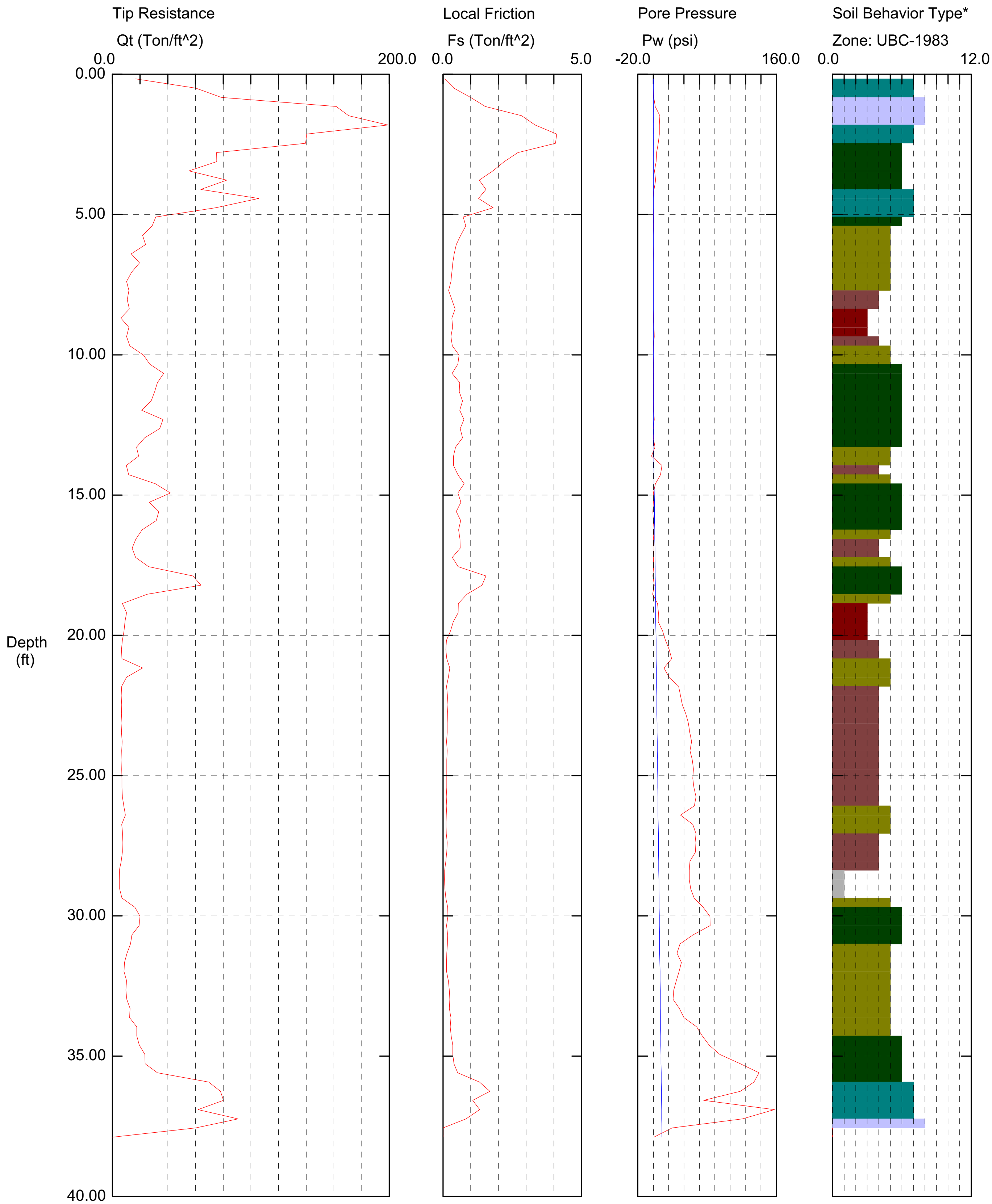
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

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

Candlestick Park CPT

Operator: JT
Sounding: CPT-11
Cone Used: 4583.124

CPT Date/Time: 11-22-09 08:56
Location: CPT-11
Job Number:



Maximum Depth = 37.89 feet

Depth Increment = 0.33 feet

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

APPENDIX E

Previous Explorations (Other Consultants)

APPENDIX E

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-1

Boring location: See Figure 2

Date started: 8/25/97 | Date finished: 8/25/97

Drilling method: Rotary wash

Hammer weight/drop: 140 lbs/30 in | Hammer type: Safety hammer

Sampler: SPT, S&H, D&M piston

Logged by M. McKee

Ground Surface Elevation: 3.5 feet, SFCD

DEPTH (feet)	Sampler Type	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
0–5			CL	SANDY CLAY (CL), dark reddish-brown, very stiff to hard, moist	FILL						
5–10	S&H (6)	18	SC	CLAYEY SAND (SC), dark yellowish-brown, medium dense, moist, with sandstone and serpentinite fragments	FILL						
10				▼ 7:45 am 8/25/97							
10–22	S&H (11)	8	CL/SC	SANDY CLAY/CLAYEY SAND (CL/SC), yellowish-brown with reddish-brown mottling, medium stiff, wet, with fine gravel, trace wood fragments	FILL	TXUU	1000	1120		17.8	115
12				▼ 7:50 am 8/25/97							
16	SPT	55/11"	CL/SC	grades sandier, with brick, concrete, and wood debris sampled concrete debris	FILL						
22–30			CH	CLAY (CH), dark greenish-gray, medium stiff, moist to wet, with some shell fragments, trace sand	BAY MUD						
28	D&M	110 psi	CH		BAY MUD	TV		1020		55.4	68

PROJECT: SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS San Francisco, California	Log of Boring DB-1 PAGE 2 OF 6

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
31–39				CH	CLAY (CH), continued; grades sandy (at 37 ft). BAY MUD (to 39 ft)						
39–55	D&M (41–42.5)		37	CL	SANDY CLAY (CL), greenish-gray with black, reddish-brown, and olive mottling, very stiff, moist	PP		3000			
55–60	S&H (56–57.5)		23	CH	CLAY (CH), greenish-gray with yellowish-brown mottling, very stiff, moist	TXUU	1500	1640		28.3	97

DEPTH (feet)	Sampler Type	Sample	Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
61–67				CH	CLAY (CH), continued						
68–70				CH	sand and gravel lens from 68 to 70 feet						
72	S&H		29	CH	LL = 72; PI = 39	PP		3400		36.0	86
81				GP	SANDY GRAVEL (GP), fine- to medium-grained						
86–87	D&M		225 psi	CH	CLAY (CH), dark gray, stiff, moist, with trace chert fragments Cec = 0.42; Cer = 0.07; Cv = 4.2 ft^2/yr	TXUU TV	2500	2230 1400		59.8	65

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-1

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
91–94				CH	CLAY (CH), continued						
94–106				CL	CLAY (CL), reddish-brown with yellowish-brown mottling, hard, moist, with sand						
101–102					grades sandy from 101 to 102 feet						
102	S&H		40		LL = 43; PI = 23					23.9	104
106–116				SM	SILTY SAND (SM), gray, fine- to medium-grained						
111–112					clay lens from 111 to 112 feet						
116–120				CH	CLAY (CH), dark greenish-gray, hard, moist						
117	S&H		40			TXUU	3000	2910		23.7	104
120					softer drilling at 120 feet						

Treadwell&Rollo | Project No. 2149.02 | Figure A-1d

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
121					CLAY (CH), continued						
122					grades very stiff, with some fine sand, trace chert fragments						
123											
124											
125											
126											
127											
128											
129											
130											
131				CH							
132	SPT		39								
133											
134					drilling hard at 134 feet						
135											
136											
137											
138											
139											
140											
141											
142											
143											
144					SANDY CLAY (CL), reddish-brown with red and black specks, hard, moist, with trace gravel						
145											
146	SPT		54/5"	CL							
147											
148											
149											
150											

PROJECT:	SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS San Francisco, California	Log of Boring DB-1	PAGE 6 OF 6

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
151–161				CL	SANDY CLAY (CL), continued grades very stiff, with some fine sand, trace chert fragments						
161–164.5	SPT		89		SANDSTONE and CHERT, yellow, brown, and dark reddish-brown, clayey, deeply weathered, crushed BEDROCK						
164.5–180					Boring terminated at a depth of 164.5 feet. Groundwater encountered at a depth of 12 feet during drilling and rose to 10 feet within a few minutes. Boring backfilled with grout.						

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-14

Boring location: See Figure 2	Logged by M. McKee
Date started: 8/22/97	Date finished: 8/22/97
Drilling method: Rotary wash	
Hammer weight/drop: 140 lbs/30 in	Hammer type: Safety hammer
Sampler: SPT, S&H, D&M piston	

DEPTH (feet)	Sampler Type	Sample	Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
					Ground Surface Elevation: 2.0 feet, SFCD						
0					3-1/2 inches asphalt concrete						
1–3				SC	CLAYEY SAND with GRAVEL (SC), dark reddish-brown, medium dense, moist, sand is coarse-grained (FILL)						
5–6	S&H		16		grades more gravelly						
6					▼ 8:55 am 8/22/97						
9					▼ 8:45 am 8/22/97						
10–11	S&H		6		grades less gravelly, wet				28.0	20.9	106
15–16.5	SPT		11								

Boring terminated at a depth of 16.5 feet.
Groundwater encountered at a depth of 9-1/2 feet during drilling and rose to 6 feet 10 minutes later.
Boring backfilled with grout.

Treadwell&Rollo | Project No. 2149.02 | Figure A-14a

PROJECT: SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS San Francisco, California	Log of Boring DB-14A PAGE 1 OF 4

Boring location:	See Figure 2		Logged by M. McKee
Date started:	8/22/97	Date finished: 8/22/97	
Drilling method:	Rotary wash		
Hammer weight/drop:	140 lbs/30 in	Hammer type: Safety hammer	
Sampler: SPT, S&H, D&M piston			

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ foot [1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 2.0 feet, SFCD)		LABORATORY TEST DATA: Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
0–1					3-1/2 inches asphalt concrete	FILL						
1–9				GC	CLAYEY SAND and GRAVEL (GC), reddish-brown, with some chert and serpentinite fragments	FILL						
9–10				GC	▼ 11:00 am 8/22/97	FILL						
15				GC	grades less clayey, more gravelly	FILL						
27–30				CH	CLAY (CH), dark greenish-gray, soft, wet, trace shells	BAY MUD						

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
31				CH	CLAY (CH), continued — BAY MUD						
32–33	D&M		400 psi	CL	CLAY (CL), yellowish-red with black and gray mottling, very stiff, moist, with some chert fragments to 1/8", with sand						
35–36	S&H		13		LL = 45; PI = 24					26.1	100
44–45					grades dark reddish-brown						
50–51	S&H		20		grades yellowish-brown with trace black organics and chert fragments to 1/16", with trace sand	TXUU	2000	1540		25.8	100
55–60				CH	CLAY (CH), bluish-gray, very stiff to hard, moist						

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-14A

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
61				CH	CLAY with GRAVEL (CH), greenish-gray, hard, moist, with angular chert fragments to 3/4"						
66	SPT		36								
69				SM	SILTY SAND and GRAVEL (SM), grayish- to reddish-brown, gravel consists of angular chert and serpentinite fragments						
76				CL	CLAY (CL), reddish-brown, hard, wet, with some chert and serpentinite fragments to 1/8"						
81	S&H		42			TXUU	3000	1550		21.5	110
90					grades gravelly at 90 feet						

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-14A

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
91				CL	CLAY (CL), continued						
92											
93											
94				CL-CH	CLAY (CL-CH), greenish-gray and brown mottled, hard, moist, with coarse angular sand						
95											
96	SPT	◩	50								
97											
98											
99											
100											
101											
102											
103											
104											
105				CL	SANDY CLAY (CL), yellowish-brown, hard, moist						
106											
107											
108					RESIDUAL SOIL						
109											
110											
111	SPT	◩	66		SERPENTINITE, olive and yellowish-brown, deeply weathered, crushed — BEDROCK						
112					Boring terminated at a depth of 111.5 feet. Groundwater encountered at a depth of 9-1/2 feet during drilling. Boring backfilled with grout.						
113											
114											
115											
116											
117											
118											
119											
120											

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-15

Boring location:	See Figure 2
Date started:	9/3/97
Date finished:	9/3/97
Drilling method:	Rotary wash
Hammer weight/drop:	140 lbs/30 in
Hammer type:	Safety hammer
Sampler:	SPT, S&H, D&M piston
Logged by	M. McKee

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ foot [1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
					Ground Surface Elevation: 3.0 feet, SFCD							
0					1 inch asphalt concrete	FILL						
1–5				SM	SILTY SAND (SM), dark brown, medium dense, moist, with trace fine gravel, with clay							
4.5					trace asphalt, glass, chert fragments 4 to 5-1/2 feet							
6	S&H		17	SC	CLAYEY SAND (SC), dark orange-brown, very stiff, moist, with some sand lenses						13.4	117
9					▼ 7:45 am 9/3/97							
9–13				SM	SILTY SAND with GRAVEL (SM), dark gray, loose, wet, with some creosote wood debris, and serpentinite fragments							
11	S&H		5							13.0	19.5	
13–15				GC	CLAYEY GRAVEL (GC), gray, medium dense, wet, with serpentinite fragments							
15–29				SC	CLAYEY SAND with GRAVEL (SC), yellowish-brown, medium dense, wet, medium- to coarse-grained							
16	SPT		18									
20					grades loose at 20 feet							
21	SPT		7							18.9		
25					grades medium dense							
26	SPT		15									
29–30				CH	CLAY (CH), bluish-gray, soft, wet, with some white shell fragments	BAY MUD						

Treadwell&Rollo | Project No. 2149.02 | Figure A-15a

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
31–33				CH	CLAY (CH), continued	BAY MUD						
34–35	D&M		175 psi	CH	$C_{ec} = 0.29$; $C_{er} = 0.06$; $C_v = 16.1\ ft^2/yr$	BAY MUD	TV		470		60.2	64
44–45	D&M		150 psi	CH	grades medium stiff at 44 feet with fine sand, no shells $C_{ec} = 0.30$; $C_{er} = 0.05$; $C_v = 16.3\ ft^2/yr$	BAY MUD	TV		600		58.1	66
54.5–60	S&H		30/4"	CL	SANDY CLAY (CL), light greenish-gray, hard, moist		PP		4000		16.1	118

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
60–70				CH	CLAY (CH), yellowish-brown, very stiff, moist						
70–90				CH	CLAY (CH), bluish-gray, very stiff, moist						
77	S&H		29			TXUU	2000	1310		35.2	87

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-15

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
91–95				CH	CLAY (CH), continued						
95–104				CH	CLAY (CH), greenish-gray with red mottling, stiff, moist, with organics and rock fragments						
97	S&H		13			PP TXUU	2500	1700 1340		40.1	81
104–110				CL	SANDY CLAY (CL), olive-gray, very stiff, moist						
110–116				SM	SILTY SAND (SM), yellowish-brown, dense, wet, fine- to medium-grained						
116–120				SM	SILTY SAND (SM), bluish-gray, very dense, wet, fine- to medium-grained grades with clay at 118 feet						
117	SPT		76								

Log of Boring DB-15

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
121					CLAY (CH), bluish-gray, very stiff, wet, with fine sand lenses						
122											
123											
124											
125											
126											
127											
128											
129											
130											
131											
132				CH							
133											
134											
135											
136											
137	D&M		275 psi			TXUU	3500	3380		54.1	69
138					grades with some organics at 137-1/2 feet						
139											
140											
141											
142											
143											
144											
145											
146					grades with chert fragments						
147											
148											
149											
150											

PROJECT: SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS San Francisco, California	Log of Boring DB-15 PAGE 6 OF 7

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
151–155				CH	CLAY (CH), continued						
156–174	S&H (156)		30/2"	SC	CLAYEY SAND with GRAVEL (SC), dark yellowish-brown, very dense, wet						
175–180	SPT (176)		49	CL	SANDY CLAY (CL), reddish-brown, hard, moist, with some fine gravel						

DEPTH (feet)	Sampler Type	Sample	Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
181–195				CL	SANDY CLAY (CL), continued						
195–200					SHALE, yellowish-brown and dark gray, deeply weathered, crushed — BEDROCK						
200	SPT		100/2"		Boring terminated at a depth of 200.2 feet. Groundwater encountered at a depth of 9 feet during drilling. Boring backfilled with grout.						

PROJECT: SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS San Francisco, California	**Log of Boring DB-16** PAGE 1 OF 8
Boring location: See Figure 2	Logged by M. McKee
Date started: 9/2/97 — Date finished: 9/2/97	
Drilling method: Rotary wash	
Hammer weight/drop: 140 lbs/30 in — Hammer type: Safety hammer	
Sampler: SPT, S&H, D&M piston	

Ground Surface Elevation: 3.0 feet, SFCD

DEPTH (feet)	Sampler Type	Blows/ foot[1]	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
0				4 inches asphalt concrete	FILL						
1–3			CL	SANDY CLAY (CL), dark reddish-brown, medium stiff to stiff, moist, with chert and serpentinite fragments							
4–5			SM	SILTY SAND (SM), dark gray, loose to medium dense, moist, with clay, and serpentinite, brick, and concrete fragments							
6	S&H	10	ML	SANDY SILT (ML), dark gray and dark greenish-gray mottled, stiff, moist, with serpentinite and brick fragments						18.6	96
9				▼ 8:45 am 9/2/97							
10–11	S&H	7	SM	SILTY SAND (SM), black, loose, wet, some decayed wood fragments, trace brick and serpentinite fragments							
15–16	SPT	23		grades medium dense							
18–30			CH	CLAY (CH), black, soft, wet, with trace roots and woody debris	BAY MUD						

Treadwell&Rollo | Project No. 2149.02 | Figure A-16a

PROJECT:	SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS San Francisco, California	Log of Boring DB-16 PAGE 2 OF 8

DEPTH (feet)	Sampler Type	Sample	Blows/foot 1	LITHOLOGY	MATERIAL DESCRIPTION		Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
31–32				CH	CLAY (CH), continued	BAY MUD						
33–34	D&M		175 psi	CH	grades dark greenish-gray, with trace shells at 33 feet	BAY MUD	TV			700	60.1	64
35–51				CH		BAY MUD						
51–55				CL	SANDY CLAY (CL), gray, with trace chert, fine sand							
55–58				CH	CLAY (CH), olive, stiff, moist, with fine sand							
58–60				CH	CLAY (CH), yellowish-brown and gray mottled, hard, moist to wet							

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS San Francisco, California**

Log of Boring DB-16

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot 1	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
61					CLAY (CH), continued						
62											
63	S&H		35								
64				CH		PP			3200	27.9	97
65											
66											
67											
68											
69					CLAYEY SAND (SC), olive-gray, dense, moist						
70											
71											
72				SC							
73											
74											
75											
76					CLAY (CH), gray, very stiff, moist						
77											
78											
79											
80											
81					grades with fine sand						
82				CH							
83											
84					fine sand and chert fragments at 84 feet						
85											
86											
87											
88											
89											
90											

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-16

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
91–96				CH	CLAY (CH), continued						
96–102.5				SM	SILTY SAND with GRAVEL (SM), gray and red mottled, chert and serpentinite fragments						
102.5–106				CH	CLAY (CH), gray, stiff, moist						
106–111				SM	SILTY SAND (SM), gray, dense, moist						
111–120				SC	CLAYEY SAND (SC), yellowish- to reddish-brown, dense, moist grades gray, more clayey sand lens from 117 to 119 feet						

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-16

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
121				SC	CLAYEY SAND (SC), continued						
122											
123					CLAY (CH), gray, hard, with fine sand						
124											
125											
126											
127											
128											
129				CH							
130					grades with fine chert fragments at 130 feet						
131											
132											
133											
134											
135					sand and gravel layer at 135 feet						
136											
137											
138					CLAY (CH), gray with red mottling, stiff, moist, with organics						
139		☒									
140		☒									
141											
142											
143											
144				CH							
145											
146											
147					grades dark olive with abundant peat at 147 to 148 feet						
148					grades gray at 148 feet						
149											
150											

PROJECT: SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS San Francisco, California	Log of Boring DB-16 PAGE 6 OF 8

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
151–157				CH	CLAY (CH), continued grades with fine to medium sand at 150 feet						
157–161				CL	CLAY (CL), dark gray to dark bluish-gray, very stiff to hard, moist, with chert fragments						
161–163		X		SM	SILTY SAND and GRAVEL (SM), gray, sand is coarse-grained, gravel is fine-grained						
163–180		X (168–169)		CL	CLAY (CL), dark yellowish-brown, very stiff to hard, moist grades with more chert fragments harder drilling at 174 feet						

Depth scale: 151, 152, 153, 154, 155, 156, 157, 158, 159, 160, 161, 162, 163, 164, 165, 166, 167, 168, 169, 170, 171, 172, 173, 174, 175, 176, 177, 178, 179, 180

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-16

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/foot 1	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
181					CLAY (CL), continued						
182					chert fragment layer from 182 to 183-1/2 feet						
183											
184											
185											
186											
187					hard drilling, abundant rock fragments from 187 to 192-1/2 feet						
188											
189											
190											
191											
192											
193				CL							
194											
195											
196											
197					grades with more chert fragments from 197 to 199 feet						
198											
199											
200											
201											
202											
203											
204											
205											
206											
207											
208											
209											
210											

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS**
San Francisco, California

Log of Boring DB-16

DEPTH (feet)	Sampler Type	Sample	Blows/foot [1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
211					CLAY (CL), continued drilling very hard from 211 feet						
212											
213											
214											
215				CL							
216	SPT		100/4"		chert fragments and slough recovered						
217											
218											
219											
220											
221											
222					SHALE, black, deeply weathered, crushed						
223					BEDROCK						
224											
225	SPT		100/6"								
226					Boring terminated at a depth of 225.5 feet. Groundwater encountered at a depth of 9 feet during drilling. Boring backfilled with grout.						
227											
228											
229											
230											
231											
232											
233											
234											
235											
236											
237											
238											
239											
240											

PROJECT: **SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS** **San Francisco, California**	# Log of Boring SB-1 PAGE 1 OF 1

Boring location: See Figure 2		Logged by M. McKee
Date started: 8/27/97	Date finished: 8/27/97	
Drilling method: Rotary wash		
Hammer weight/drop: 140 lbs/30 in	Hammer type: Safety hammer	
Sampler: none		

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ foot [1]	LITHOLOGY	MATERIAL DESCRIPTION (Ground Surface Elevation: 4.5 feet, SFCD)	Type of Strength Test	Test Surcharge Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content	Dry Density Lbs/Cu Ft
0–2		X		SM	SILTY SAND and GRAVEL (SM), grayish-brown, dry, gravel up to 1"						
2–4		X		SM	SILTY SAND (SM), gray, with brick debris						
4–5		X		SM	SILTY SAND and GRAVEL (SM), grayish-brown, dry to moist						
5–30					Boring terminated at a depth of 5 feet. Groundwater not encountered during drilling. Boring backfilled with grout.						

LABORATORY TEST DATA: none reported.

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes: U.S. Standard Sieve Size	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
Silt and Clay	Below No. 200	Below 0.074

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with split-barrel sampler other than Standard Penetration Test sampler. Darkened area indicates sample obtained

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Groundwater level at the time and date indicated

SAMPLER TYPE

C Core barrel

CA California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

S&H Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

ST Shelby tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

SAN FRANCISCO 49ers STADIUM AND CANDLESTICK MILLS
San Francisco, California

Treadwell&Rollo

CLASSIFICATION CHART

Project No. 2149.02 | Figure A-20

I CONSOLIDATION OF SEDIMENTARY ROCKS: usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

II BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

III FRACTURING

Intensity	Size of Places in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

IV HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visibly after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

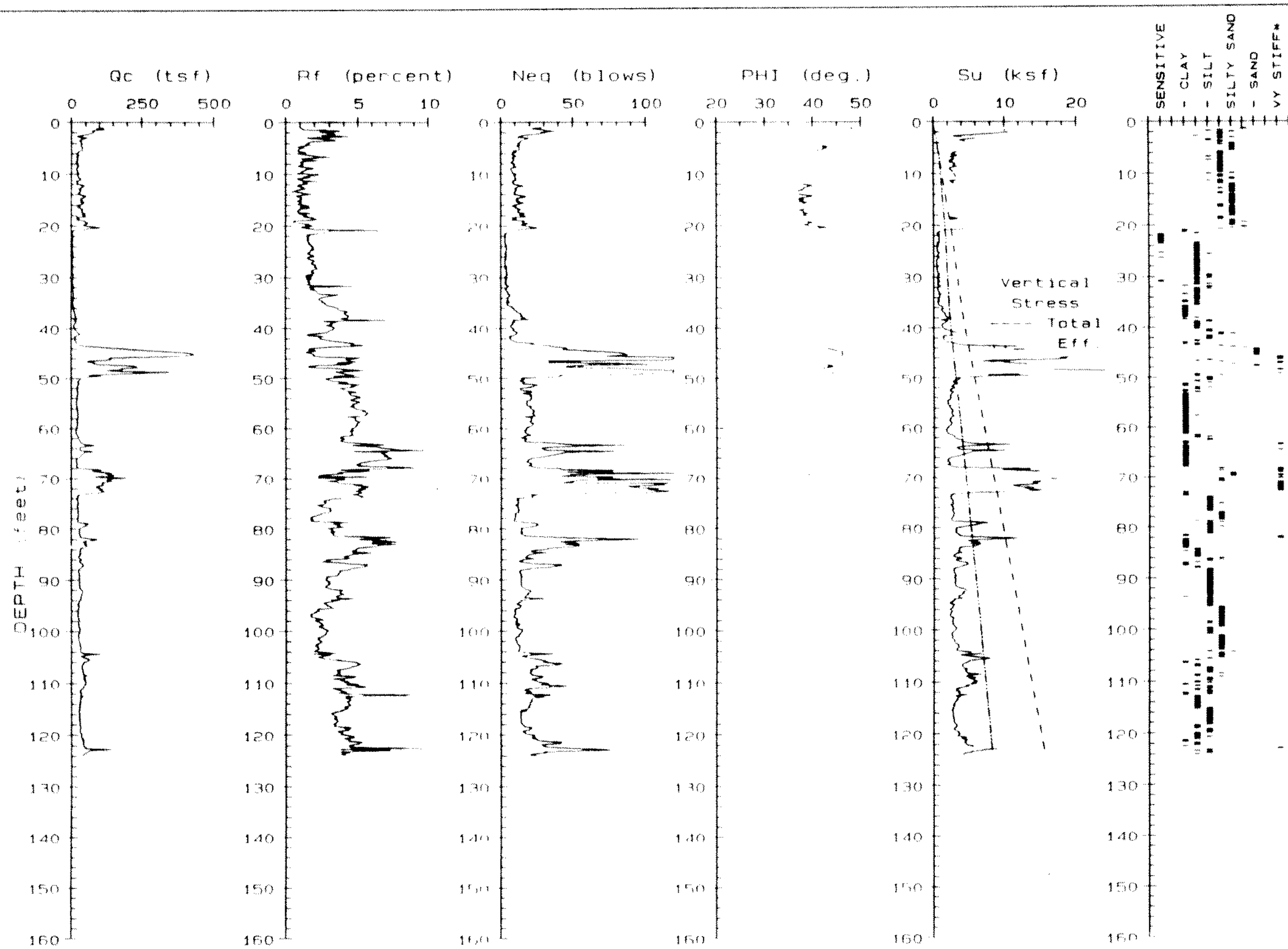
V STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

VI WEATHERING - The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing .

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

SAN FRANCISCO 49ers STADIUM AND CANDLESTICK MILLS San Francisco, California	**PHYSICAL PROPERTIES CRITERIA FOR ROCK DESCRIPTIONS**
Treadwell&Rollo	Project No. 2149.02 / Figure A-21



Terminated at 124.0 feet

Groundwater measured at 7.2 feet

Date: 8/26/97

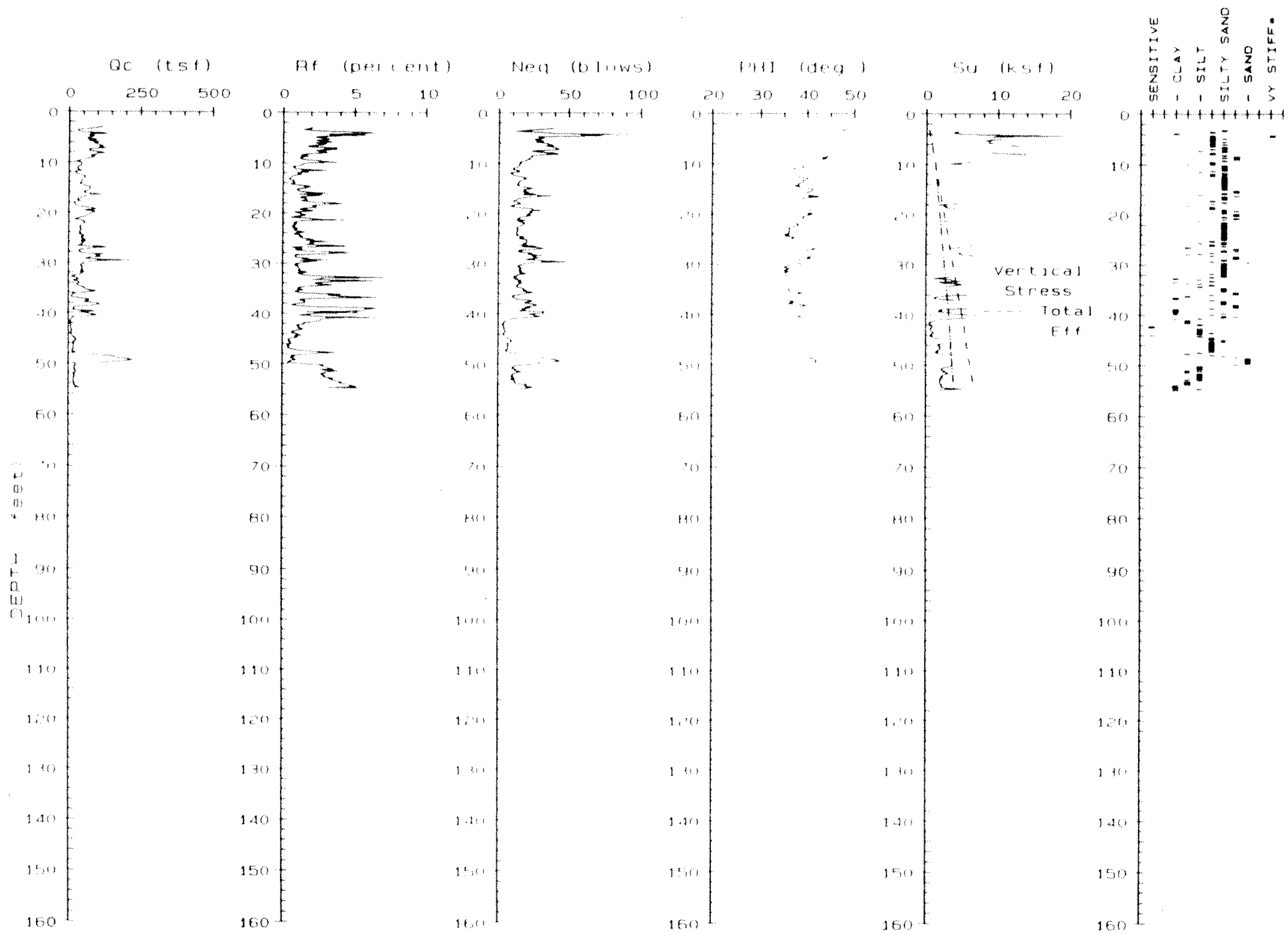
Elevation of ground surface (approximate, SFCD): 1.0

SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS
San Francisco, California

CONE PENETRATION TEST RESULTS CPT-1

Project No. 2149.02	Figure B-1

Treadwell&Rollo



Terminated at 55.0 feet

Groundwater measured at 10.3 feet

Date: 8/27/97

Elevation of ground surface (approximate, SFCD): 4.5

SAN FRANCISCO 49ERS STADIUM AND CANDLESTICK MILLS
San Francisco, California

CONE PENETRATION TEST RESULTS CPT-16

Project No. 2149.02 | Figure B-15

Treadwell&Rollo

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR MH	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR MH	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

BLOW COUNT

THE NUMBER OF BLOWS OF THE SAMPLING HAMMER REQUIRED TO DRIVE THE SAMPLER THROUGH EACH OF THREE 6-INCH INCREMENTS. LESS THAN THREE INCREMENTS MAY BE REPORTED IF MORE THAN 50 BLOWS ARE COUNTED FOR ANY INCREMENT. THE NOTATION 50/5" INDICATES 5 INCHES OF PENETRATION ACHIEVED IN 50 BLOWS.

N-VALUE

THE NUMBER OF BLOWS REQUIRED TO DRIVE A STANDARD SPT SPLIT SPOON THE LAST 12 OF 18 INCHES. IF LESS THAN 18 INCHES WERE DRIVEN, AN EXTRAPOLATED VALUE IS REPORTED ON THE BORING LOG. IF A 2.5-INCH MODIFIED CALIFORNIA SAMPLER IS DRIVEN, THE VALUE REPORTED ON THE LOG HAS BEEN REDUCED USING A 0.65 CONVERSION FACTOR.

CEMENTATION

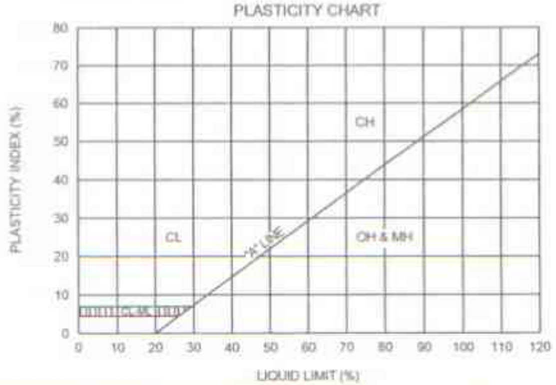
WEAK - CRUMBLES OR BREAKS WITH HANDLING OR LITTLE FINGER PRESSURE

MODERATE - CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE

STRONG - WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

SOURCE: U.S. DEPARTMENT OF INTERIOR, BUREAU OF RECLAMATION; PUB. YEAR, NOT AVAILABLE; ENGINEERING GEOLOGY FIELD MANUAL, P. 37.

PLASTICITY CHART



SAMPLE TYPES

MODIFIED CALIFORNIA

STANDARD PENETRATION

PUSHED SHELBY TUBE

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY) COMPLETED. REFER TO LABORATORY RESULTS.

CD - CONSOLIDATED DRAINED (CD) TRIAXIAL TEST COMPLETED. REFER TO LABORATORY RESULTS.

CN - CONSOLIDATION TEST COMPLETED. REFER TO LABORATORY RESULTS.

CU - CONSOLIDATED UNDRAINED (CU) TRIAXIAL TEST COMPLETED. REFER TO LABORATORY RESULTS.

DS - RESULTS OF DIRECT SHEAR TEST, IN TERMS OF TOTAL COHESION (C, KSF) OR EFFECTIVE COHESION AND FRICTION ANGLE (C',KSF, AND Ø', DEGREES).

PP - RESULTS OF POCKET PENETROMETER TEST, IN TERMS OF SHEAR STRENGTH (KSF).

SW - SWELL TEST COMPLETED. REFER TO LABORATORY RESULTS.

TC - CYCLIC TRIAXIAL COMPLETED. REFER TO LABORATORY RESULTS.

TV - RESULTS OF TORVANE SHEAR TEST, IN TERMS OF UNDRAINED SHEAR STRENGTH (KSF).

UC - RESULTS OF UNCONFINED COMPRESSION TEST, IN TERMS OF UNDRAINED SHEAR STRENGTH (KSF).

UU - UNCONSOLIDATED UNDRAINED (UU) TRIAXIAL TEST COMPLETED, IN TERMS OF UNDRAINED SHEAR STRENGTH (KSF).

RV - RESULTS OF R-VALUE TEST.

▽ - WATER LEVEL (REFER TO END NOTES FOR DATE OF MEASUREMENT)

PENETRATION RESISTANCE (RECORDED AS BLOWS / 0.5 FT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	N-VALUE (BLOWS/FOOT)*	CONSISTENCY	N-VALUE (BLOWS/FOOT)*	COMPRESSIVE STRENGTH (TSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.50
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.50 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

Olivia Chen Consultants

Job No. 1420.05

LEGEND TO SOIL DESCRIPTIONS

FIGURE A-1

OCC STD LEGEND L:\SFHA\TASK ORDER 5 ALICE GRIFFITH GEOTECHNICAL\06 DATA & CALCS\BORING LOGS\A_GRIFFITH.GPJ OCC_STD.GDT 3/30/05

BORING LOG

Logged By: CBK	Edited By: CBK	Hole Diameter: 5 inches	Drilling Fluid: Polymer Fluid
Start Date: 3/4/05 Time: 8:30 AM	Finish Date: 3/4/05 Time: 12:20 PM	Backfill Method: Cement Grout	Date: 3/4/05
Boring Location: Lat. N37.7186 Long. W122.3873	Boring Elevation: Street Level	Drilling Company: Pitcher Drilling Co.	Driller: Lee
		Rig Type: Failing 1500	Drilling Method: Mud Rotary
Datum: WGS84	Datum:	Hammer Type: Safety Drive Method: Rope/Cathead	Weight: 140 lb. Drop: 30 inches
Source: Approx. from USGS Map	Source:	SPT Split Spoon Type: Room for Liners/Liners Not Used	Modified California Liner Diameter: 2.5 inches

Depth (feet)	Sample No.	Sample Interval	Sample Type	Recovery (%)	Blow Counts/6-Inch or Sample Pressure	Blowcount	Graphic Log	GROUP NAME (GROUP SYMBOL) color, moisture condition, consistency/density, other description (local name or material type)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing No. 200 Sieve	Other
	1		MC	83	5 14 20	34		3 Inches Peat (TOPSOIL). Lean Clay (CL), orange brown, very stiff, moist (FILL).			26	10		
	2		SPT	94	12 10 14	24				12.4				PP: Su = 4.3 ksf
								Gravelly Lean Clay (CL), orange brown, very stiff, moist (FILL).						
5	3		MC	56	4 19 23	42				12.9				PP: Su > 4.5 ksf
								Clayey Sand with Gravel (SC), green, very dense, moist (FILL).						
10	4		SPT		16 26 27	53								
15	5		MC	50	10 3 3	6		Lean Clay (CL), gray, soft, moist (YOUNG BAY DEPOSITS).		63.2				PP: Su = 0.25 ksf
20	6		MC	56	2 3 3	6				20.9	23	9		
	7		ST	73	200 psi									
								Clayey Sand (SC), orange, dense to very dense, moist.						
25	8		MC	80	50/5"	120								
	9		SPT		10 16	36		Increased clay content.		22				

Olivia Chen Consultants	Project Name: SFHA Alice Griffith Portable Building	Job Number: 1420.05	Boring Number: B-1
	Location: Griffith St. at Fitzgerald, SF, CA	Approved: EW Date: 3/30/05	Sheet 1 of 2

L:\SFHA\TASK ORDER 5 ALICE GRIFFITH GEOTECHNICAL\06 DATA & CALCS\BORING LOGS\A_GRIFFITH.GPJ

BORING LOG

Depth (feet)	Sample No.	Sample Interval	Sample Type	Recovery (%)	Blow Counts/6-Inch or Sample Pressure	Blowcount	Graphic Log	GROUP NAME (GROUP SYMBOL) color, moisture condition, consistency/density, other description (local name or material type)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing No. 200 Sieve	Other
					20									
								Silty Sand (SM), orange brown, very dense, moist.						
35	10		SPT		32 40 50/4"	75								
								Faster drilling.						
40	11		SPT	0	21 23 28	51		No recovery.						
	12		SPT	83	29 26 31	57		Used sand catch.		19.8			18.6	
45														
								Clay in cuttings.						
								Lean Clay (CL), gray, hard, moist (OLD BAY CLAY).						
	13		MC		18 32 50	82								
								Bottom of boring at 49 feet below ground surface.						
50								Boring backfilled with cement grout on March 4, 2005.						
								Groundwater level could not be determined due to drilling methods used.						
55														
60														
65														

Olivia Chen Consultants

Project Name: SFHA Alice Griffith Portable Building

Job Number: 1420.05

Location: Griffith St. at Fitzgerald, SF, CA

Approved: EW

Date: 3/30/05

Boring Number: B-1

BORING LOG

Logged By: CBK	Edited By: CBK	Hole Diameter: 5 inches	Drilling Fluid: Polymer Fluid
Start Date: 3/4/05 Time: 12:50 PM	Finish Date: 3/4/05 Time: 3:40 PM	Backfill Method: Cement Grout	Date: 3/4/05
Boring Location: Lat. N37.7185 Long. W122.3872	Boring Elevation: Street Level	Drilling Company: Pitcher Drilling Co.	Driller: Lee
		Rig Type: Failing 1500	Drilling Method: Mud Rotary
Datum: WGS84	Datum:	Hammer Type: Safety Drive Method: Rope/Cathead	Weight: 140 lb. Drop: 30 inches
Source: Approx. from USGS Map	Source:	SPT Split Spoon Type: Room for Liners/Liners Not Used	Modified California Liner Diameter: 2.5 inches

Depth (feet)	Sample No.	Sample Interval	Sample Type	Recovery (%)	Blow Counts/6-Inch or Sample Pressure	Blowcount	Graphic Log	GROUP NAME (GROUP SYMBOL) color, moisture condition, consistency/density, other description (local name or material type)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing No. 200 Sieve	Other
	1		MC	38	4 15 50/4	90		4 Inches Peat (TOPSOIL). Gravelly Lean Clay (CL), brown and black, very stiff to hard, moist, roots, concrete rubble at 1.3 feet (FILL).		10.9	33	12		
	2		SPT	33	23 15 10	25		No roots after 2 feet.						
5	3		SPT	44	11 18 27	45		Grades to brown and orange.		12.0				PP: Su = 4.5 ksf
10	4		SPT	33	9 11 23	34				18.5				
								Pea gravel in cuttings.						
15	5		SPT	0	26 30 7	37		No recovery due to gravel blocking of sampler tip.						
								Lean Clay (CL), gray, soft, moist (YOUNG BAY DEPOSITS).						
20	6		ST	100	150 psi			Clayey Sand (SC), gray, dense to very dense, moist.						
25	7		MC	83	17 31 38	69			117	17.0			39.8	PP: Su = 4.0 ksf
								Increased sand content in cuttings.						
	8		SPT	100	19 30	69		Grades to brown and orange.		18.0				

Olivia Chen Consultants	Project Name: SFHA Alice Griffith Portable Building	Job Number: 1420.05	Boring Number: B-2
	Location: Griffith St. at Fitzgerald, SF, CA	Approved: EW Date: 3/30/05	Sheet 1 of 2

BORING LOG

Depth (feet)	Sample No.	Sample Interval	Sample Type	Recovery (%)	Blow Counts/6-Inch or Sample Pressure	Blowcount	Graphic Log	GROUP NAME (GROUP SYMBOL) color, moisture condition, consistency/density, other description (local name or material type)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Percent Passing No. 200 Sieve	Other
					39									
								Faster drilling 32-34 feet.						
35	9		SPT	89	6 11 23	34								
40	10		SPT	0	17 19 20	39		No recovery.						
45	11		SPT		5 26 30	56								
								Lean Clay (CL), gray, hard, moist (OLD BAY CLAY).						
50	12		MC	92	24 50/6"	100								
								Bottom of boring at 51 feet below ground surface. Boring backfilled with cement grout on March 4, 2005. Groundwater level could not be determined due to drilling methods used.						
55														
60														
65														

Olivia Chen Consultants

Project Name: SFHA Alice Griffith Portable Building

Job Number: 1420.05

Location: Griffith St. at Fitzgerald, SF, CA

Approved: EW

Date: 3/30/05

Boring Number: B-2

Sheet 2 of 2

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

Olivia
Chen
Consultants

APPENDIX B GEOTECHNICAL LABORATORY TESTING PROGRAM

The purpose of the laboratory testing program was to provide data to assist in the evaluation of the physical and mechanical properties of the soils underlying the site.

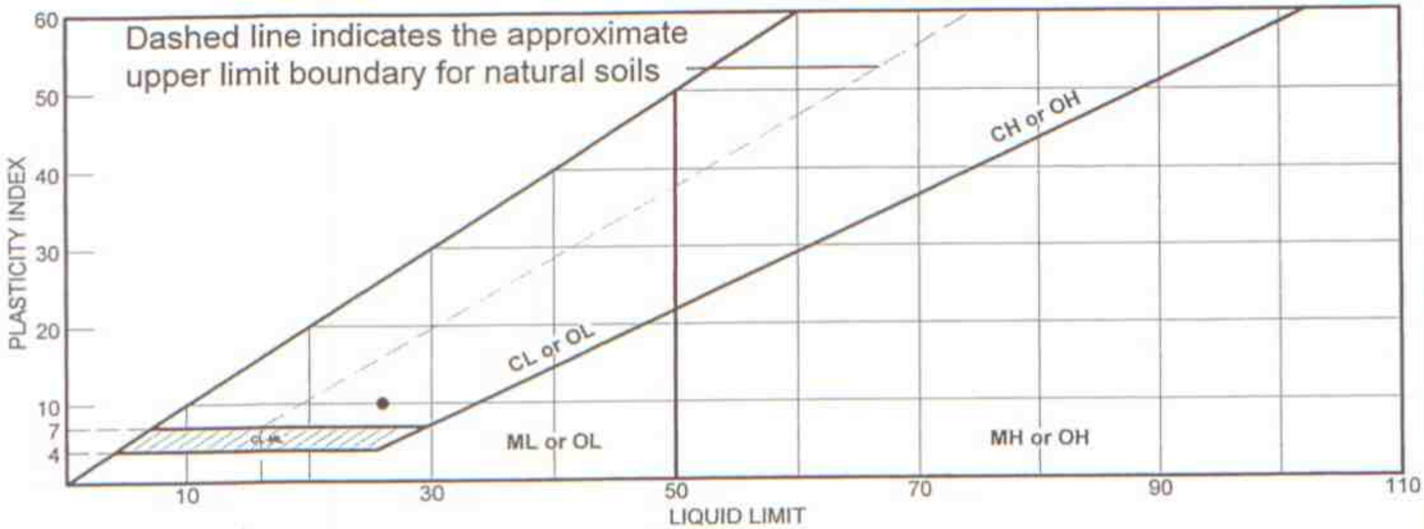
The natural water content was determined on 11 samples of the materials recovered from the borings in accordance with ASTM Test Method D2216. These water contents are recorded on the boring log at the appropriate sample depths.

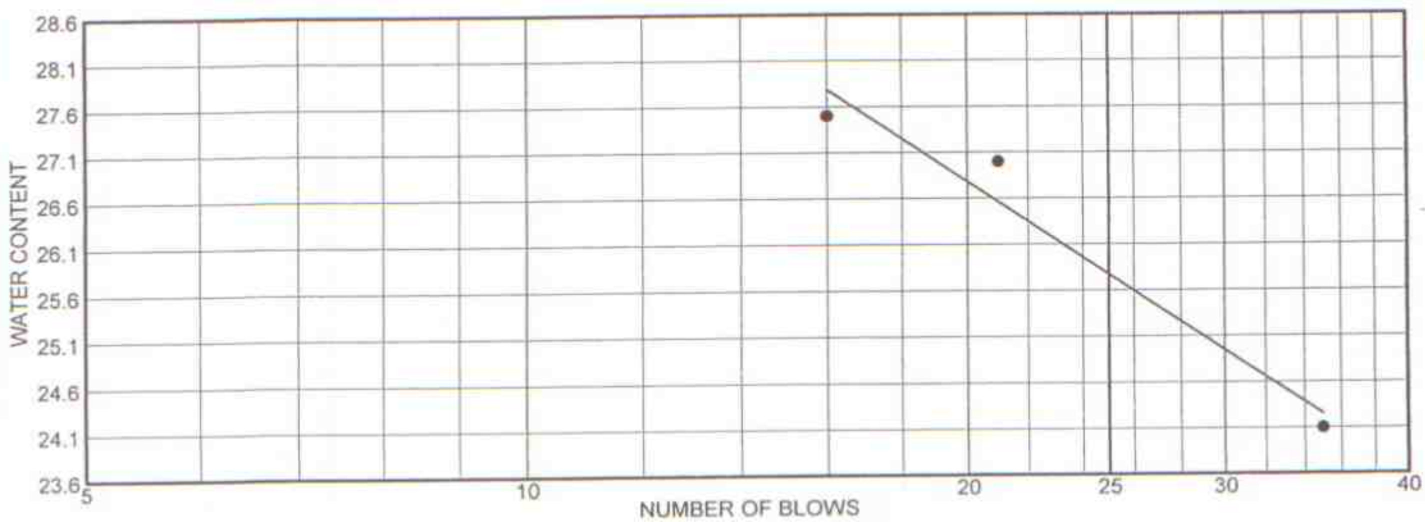
Dry density determination was performed on one sample. The result of this test is recorded on the boring log at the appropriate sample depth.

Particle size (-200) analysis was performed on two samples to evaluate the fines content of the soil and to aid in soil classification. The test was performed in accordance with ASTM Method D1140. The results of these tests are recorded on the boring logs at the appropriate sample depths.

Atterberg Limits determinations were performed on three samples. The Atterberg Limits were determined in accordance with ASTM Test Methods D428 and D424. These values are used to classify the soil in accordance with the Unified Soil Classification System and to indicate the soil's compressibility and expansion potentials. The results of these tests are presented on boring logs at the appropriate sample depths and in this appendix.

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Yellowish brown sandy clay with gravel	26	16	10			

Project No. 1420.05 **Client:** Olivia Chen Consultants

Project: Alice Griffith Portable Building

● **Source:** B-1 **Elev./Depth:** 1.5-3

Remarks:

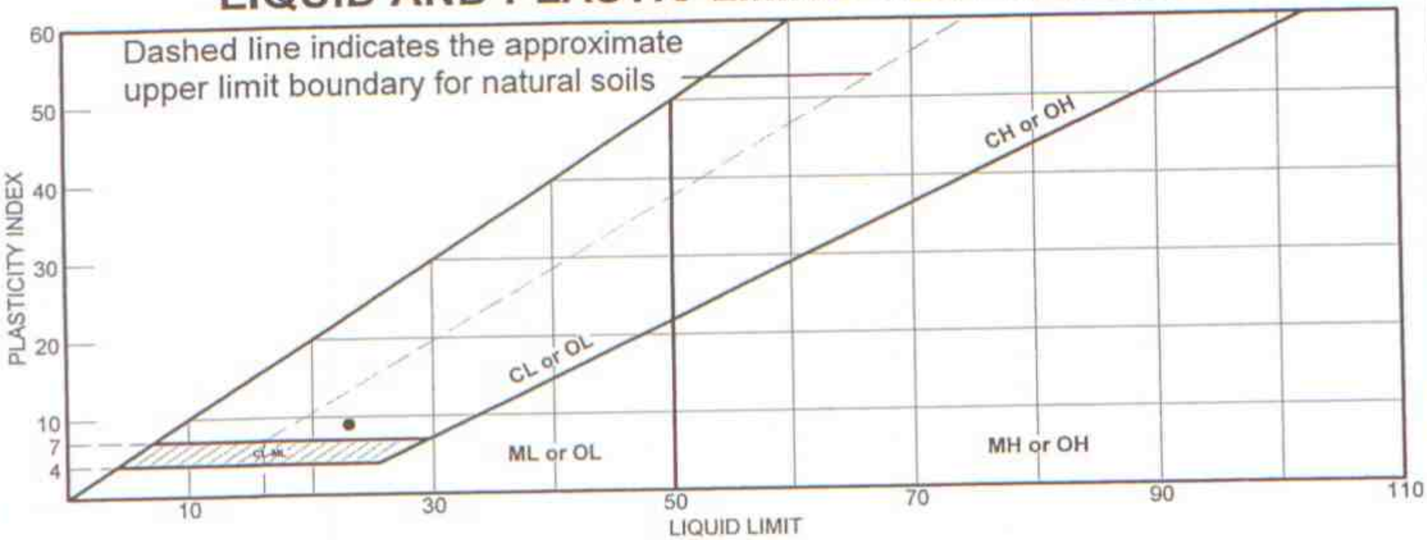
●

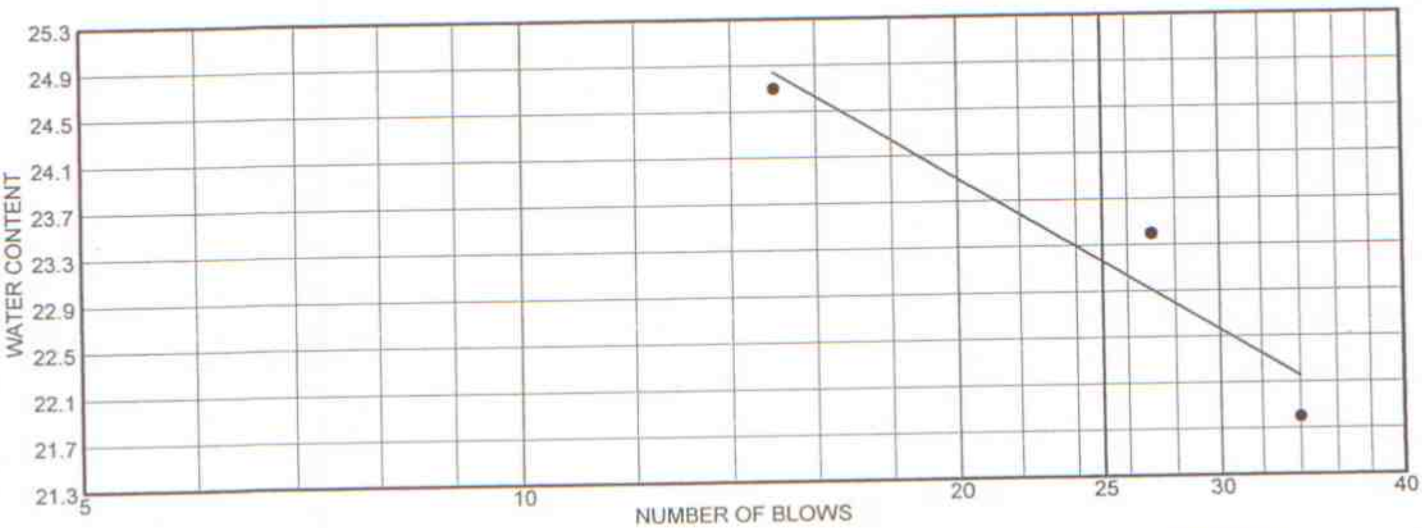
LIQUID AND PLASTIC LIMITS TEST REPORT

SIGNET TESTING LABS, INC.

Plate

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark gray sandy clay	23	14	9			

Project No. 1420.05 **Client:** Olivia Chen Consultants

Project: Alice Griffith Portable Building

● **Source:** B-1 **Elev./Depth:** 19-20.5

Remarks:

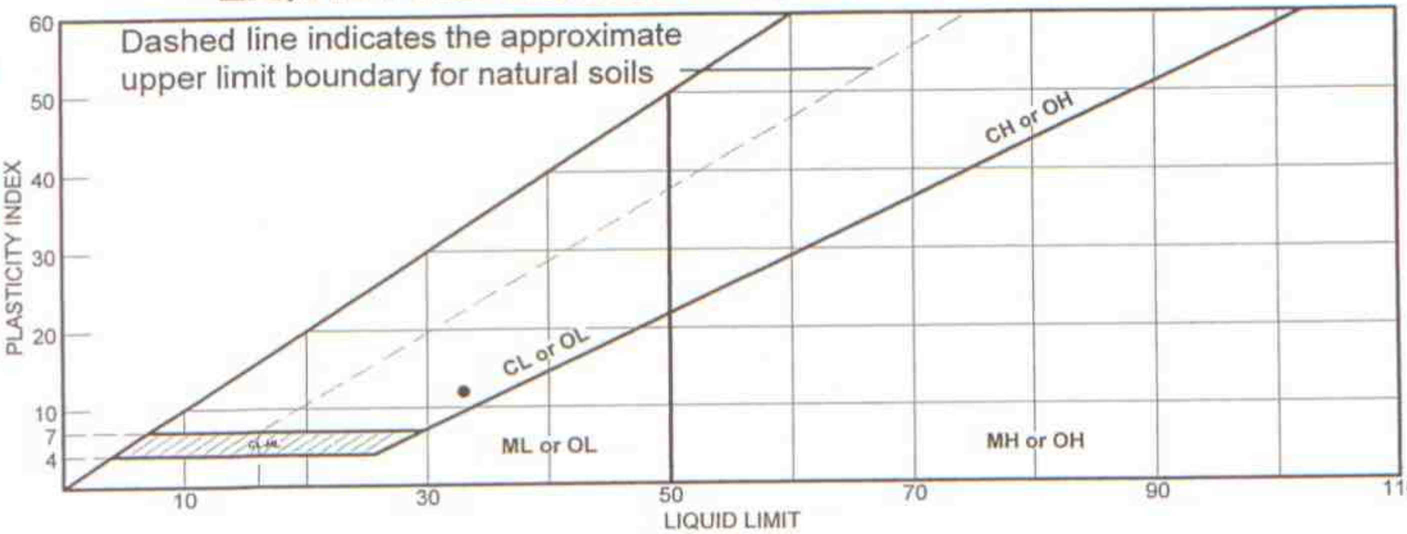
●

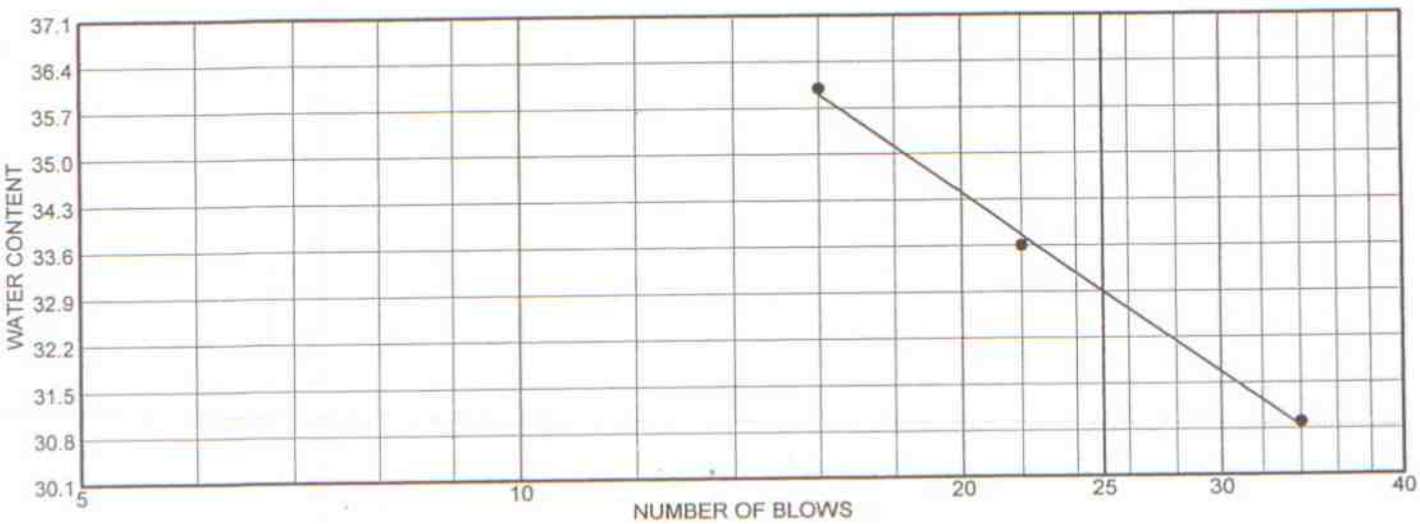
LIQUID AND PLASTIC LIMITS TEST REPORT

SIGNET TESTING LABS, INC.

Plate

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Mottled brown sandy clay with gravel	33	21	12			

Project No. 1420.05 **Client:** Olivia Chen Consultants

Project: Alice Griffith Portable Building

● **Source:** B-2 **Elev./Depth:** 0-1.5

Remarks:

●

LIQUID AND PLASTIC LIMITS TEST REPORT

SIGNET TESTING LABS, INC.

Plate

MOISTURE & DENSITY TEST

Client : Olivia Chen Consultants　Project : Alice Griffith Portable Building　Job no : 1420.05

Boring #	B-1	B-1	B-1	B-1	B-1	B-1	B-2	B-2	
Sample #									
Depth (ft.)	1.5-3	5-7.5	14-15.5	19-20.5	29-30.5	41-42.5	0-1.5	5-6.5	
Soil type: (visual)	Yellowish brown sandy clay with gravel	Brown sandy clay with gravel	Dark olive gray silty clay	Dark gray sandy clay	Olive brown sand	Olive brown sand with silt	Mottled brown sandy clay with gravel	Olive brown clay with sand and gravel	
1. Date tested:	03/10/05	03/10/05	03/10/05	03/10/05	03/10/05	03/10/05	03/10/05	03/10/05	1.
2. Tested by:	JH	JH	JH	JH	JH	JH	JH	JH	2.
3. Specimen height (in.)									3.
4. Wt. of specimen + tare (gm)									4.
5. Tare wt. (gm)									5.
6. Diameter (in.)									6.
7. Wet wt. of soil + dish wt. (gm)	165.05	306.42	170.57	214.91	232.15	234.00	227.88	280.56	7.
8. Dry wt. of soil + dish wt. (gm)	152.57	276.11	121.51	185.18	199.68	202.17	209.38	255.08	8.
9. Wt. of dish (gm)	51.99	40.40	43.85	43.12	51.89	41.77	40.23	42.95	9.
10. Dish ID									10.

Wet Density (pcf)								
Dry Density (pcf)								
Moisture Content (%)	12.4	12.9	63.2	20.9	22.0	19.8	10.9	12.0

Gs (Assumed)	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Void Ratio								
Saturation (%)								

Additional data:

Wt. of dry soil + dish before washing (gm)								
Wt. of dry soil + dish after washing (gm)								
% Passing # 200 sieve								
USCS symbol								

MOISTURE & DENSITY TEST

Client : Olivia Chen Consultants Project : Alice Griffith Portable Building Job no : 1420.05

	Boring #	B-2	B-2	B-2					
	Sample #								
	Depth (ft.)	9-10.5	25	29-30.5					
	Soil type: (visual)	Olive orangish brown clay with sand	Olive gray clayey sand	Olive brown sand with clay					
1.	Date tested:	03/10/05	03/10/05	03/10/05					1.
2.	Tested by:	JH	JH	JH					2.
3.	Specimen height (in.)		5.97						3.
4.	Wt. of specimen + tare (gm)		985.5						4.
5.	Tare wt. (gm)		0.00						5.
6.	Diameter (in.)		2.42						6.
7.	Wet wt. of soil + dish wt. (gm)	177.60	247.88	206.82					7.
8.	Dry wt. of soil + dish wt. (gm)	156.33	218.07	183.10					8.
9.	Wt. of dish (gm)	41.31	43.21	51.46					9.
10.	Dish ID								10.

Wet Density (pcf)		136.6						
Dry Density (pcf)		116.7						
Moisture Content (%)	18.5	17.0	18.0					
Gs (Assumed)	2.70	2.70	2.70	2.70	2.70	2.70	2.70	2.70
Void Ratio		0.444						
Saturation (%)		103.8						

Additional data:

Wt. of dry soil + dish before washing (gm)								
Wt. of dry soil + dish after washing (gm)								
% Passing # 200 sieve								
USCS symbol								

ASTM D-1140
PERCENT PASSING NO. 200 SIEVE REPORT

Method A

Specimens Soaked Overnight without Deflocculating Agent

Dry Mass Determined Directly

Client Name Olivia Chen Consultants

Project Name Alice Griffith Portable Building

Project Number 1420.05

Boring Number	B-1	B-2			
Sample Number					
Depth (ft)	41-42.5	25			
Percent of Soil Finer than No. 200 Sieve	18.6	39.8			
Visual Classification	Olive brown sand with silt	Olive gray clayey sand			
Date	03/10/05	03/10/05			
Weight of Dry Soil + Pan (before wash)	202.2	218.1			
Weight of Dry Soil + Pan (after wash)	172.3	148.4			
Weight of Pan	41.8	43.2			

PROJECT: **BAYVIEW HOPE DEVELOPMENT**
San Francisco, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2
Logged by: C. Brown
Date started: 5/14/02 | Date finished: 5/14/02
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30-inches | Hammer type: Safety
Sampler: Sprague & Henwood (S&H), Shelby Tube (ST)

Surface Elevation: 2 feet[2]

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Fines %	Natural Moisture Content, %
0				2-inches Asphalt Concrete		
0				3- 4-inches Baserock		
1				SAND with GRAVEL and CLAY gray, dry		
2			CL	CLAY with GRAVEL, SAND, SILT and RUBBLE (cl) red brown to red, very stiff, moist		
3	S&H	22				
5				with concrete charcoal and metal		
6	S&H	14	SP	SAND with GRAVEL (SP) dark brown, medium dense, moist		
9			SM	(05/14/02 at 8:25 AM) SILTY SAND (SM) brown with gray mottling, loose, wet		
10					15.6	20.5
11	S&H	5	ML	SILT with SAND (ML) black, soft, wet, with rubble		
14			SP	SAND (SP) black and tan, medium dense, wet		
16	S&H	16				
20					2.2	13.1
21	S&H	5		GRAVEL dark green, loose, wet, with shells		
22				CLAY dark gray, soft, wet, with thin interbedded sandy layers and shells		
23	ST	50 to 75 psi				
26	ST	75 psi				

Other laboratory test data columns (no entries): Type of Strength Test; Confining Pressure Lbs/Sq Ft; Shear Strength Lbs/Sq Ft; Dry Density Lbs/Cu Ft.

FILL (0–21 ft); BAY MUD (21–30 ft)

Treadwell&Rollo
Project No.: 3242.02 | Figure: A-1a

PROJECT: **BAYVIEW HOPE DEVELOPMENT**
San Francisco, California

Log of Boring B-1

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31–32	ST		75 psi	CL	CLAY with SAND dark gray, soft to medium stiff, wet (BAY MUD)						
35–37	S&H		14	SC	CLAYEY SAND (SC) light to dark gray, medium dense, wet, with occasional gravel LL = 28, PI = 13					19.2	112
40–41	S&H		22	SP	SAND (SP) orange brown to gray, medium dense, wet						
41–46				CL	SANDY CLAY (CL) gray with brown mottling, very stiff, wet						
45–46	S&H		22	SC	CLAYEY SAND (SC) dark gray, dense, wet; grading less clay						
50–51.5	S&H		21	CL	CLAY (CL) dark gray, very stiff, wet						

Boring terminated at 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at 9 feet during drilling.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

Treadwell&Rollo

Project No.: 3242.02 | Figure: A-1b

PROJECT: **BAYVIEW HOPE DEVELOPMENT**
San Francisco, California

Log of Boring B-2

Boring location: See Site Plan, Figure 2

Logged by: C. Brown

Date started: 5/14/02 | Date finished: 5/14/02

Drilling method: Rotary Wash

Hammer weight/drop: 140 lbs./30-inches | Hammer type: Safety

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

Surface Elevation: 2 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5				1-inch Asphalt concrete; 4-inches Aggregate Base						
0.5–2			CL	CLAY with SAND (CL) brown, stiff, dry, with rubble and gravel						
2–3	S&H	18	GC	GRAVEL with CLAY (GC) brown, stiff, dry, with sand and rubble						
3–4			CL	CLAY (CL) red brown, stiff, dry, with rubble						
4–15			SM	Mixed SAND, SILT, and CLAY (SM) brown to red, medium dense to stiff, dry to wet, with extensive brick and concrete debris						
6	SPT	23								
7				concrete block						
8				marble and brick						
9	SPT	13								
12	SPT	16		encountered steel plate						
15–20			SM	SILTY SAND (SM) black to brown, loose to medium dense, wet, with thin interbedded clay lenses				31.8	27.3	
16	SPT	6								
20–25			SP	SAND (SP) brown to red brown, medium dense, wet, with thin interbedded gravel layers				3.0	16.8	
21	SPT	15								
25–28			SP	SAND (SP) brown to red brown, dense, wet, with trace silt						
26	SPT	43								
28–30			GP	GRAVEL (GP) brown, dense, wet						

FILL (0–15 feet)

LABORATORY TEST DATA

Treadwell&Rollo

Project No.: 3242.02 | Figure: A-2a

TEST GEOTECH LOG 324202.GPJ TR.GDT 6/25/02

PROJECT: **BAYVIEW HOPE DEVELOPMENT**
San Francisco, California

Log of Boring B-2

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30–31	SPT	68	GP							
31–32			SP	SAND with GRAVEL (SP) brown, very dense, wet						
32–37			CH	CLAY (CH) dark gray, soft, wet						
36	ST	100 psi		Consolidation Test, See Figure B-1					23.5	102
37–48			SP-SC	SAND with CLAY (SP-SC) brown with olive-gray mottling, dense to very dense, wet, with thin interbedded, finely graded gravel layers						
40–41	S&H	30/ 6"								
45–46	S&H	43								
48–51.5			SC	CLAYEY SAND (SC) gray to olive gray, dense, wet, with trace gravel						
50–51	S&H	38								

FILL (to approx. 32 ft); BAY MUD (approx. 32–37 ft)

Boring terminated at 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling fluid.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

Treadwell&Rollo

Project No.: 3242.02 | Figure: A-2b

PROJECT: **BAYVIEW HOPE DEVELOPMENT**
San Francisco, California

Log of Boring B-3

Boring location: See Site Plan, Figure 2	Logged by: C. Brown
Date started: 5/15/02	Date finished: 5/15/02
Drilling method: Rotary Wash	
Hammer weight/drop: 140 lbs./30-inches	Hammer type: Safety
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)	

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Surface Elevation: 2 feet[2])		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5				6-inches Asphalt Concrete	FILL						
0.5–6	S&H (3)	15	CL	GRAVELLY CLAY with SAND (CL) red brown to orange, stiff to very stiff, dry, with brick and asphalt	FILL						
	SPT (6)	22									
6–8.5			GC	CLAYEY GRAVEL with SAND (GC) dark brown, medium dense, moist	FILL						
8.5–10.5			CL	CLAY with SAND (CL) brown to red brown, stiff, wet	FILL						
10.5–13.5	S&H (11)	24	CL	CLAY (CL) gray, very stiff, wet, with concrete debris	FILL						
13.5–18	SPT (14)	16	CH	CLAY (CH) black, soft, wet; grading with shells	BAY MUD						
	ST (17)	50 to 200 psi									
18–28			SP	SAND (SP) brown, dense, wet							
	SPT (22)	41		grading with trace gravel							
	SPT (26)	45		grading with trace silt							
28–30			GP	GRAVEL (GP) brown to red, dense, wet							

PROJECT:	BAYVIEW HOPE DEVELOPMENT San Francisco, California	Log of Boring B-3	PAGE 2 OF 2

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
30.5	SPT	30	GP							
31			GC	CLAYEY GRAVEL with SAND (GC) gray, medium dense, wet						
33.5			CL	SANDY CLAY (CL) yellow brown with gray mottling, hard, wet						
36	S&H	34	CL							
38			SP	SAND (SP) gray brown, very dense, wet						
41	SPT	50/6"	SP							
42			SC	CLAYEY SAND (SC) brown, medium dense, wet						
46	S&H	24	SC	LL = 24, PI = 8					23.7	105
46.5			CL	SANDY CLAY (CL) olive gray, very stiff, moist						
51	SPT	16	CL	gray, very stiff						

Boring terminated at 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling fluid.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

Treadwell&Rollo

Project No.: 3242.02

Figure: A-3b

TEST GEOTECH LOG 324202.GPJ TR.GDT 6/20/02

PROJECT: **BAYVIEW HOPE DEVELOPMENT**
San Francisco, California

Log of Boring B-4

Boring location: See Site Plan, Figure 2
Logged by: C. Brown
Date started: 5/15/02 Date finished: 5/15/02
Drilling method: Rotary Wash
Hammer weight/drop: 140 lbs./30-inches Hammer type: Safety
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)

DEPTH (feet)	Sampler Type	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION (Surface Elevation: 2 feet[2])		Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1				6-inches Asphalt GRAVEL with SAND and CLAY (GP-GC) red brown, medium dense, moist	FILL						
2–3	S&H	10	GP-GC								
3–5			GC	CLAYEY GRAVEL (GC) yellow brown, medium dense, moist							
5–6	SPT	18	CL	CLAY with SAND (CL) red brown, very stiff, moist							
6–8			GP	GRAVEL with SAND (GP) brown, medium dense, moist, with brick debris							
8–15				SILTY-CLAYEY SAND (SM-SC) black, loose, wet, with trace wood and brick							
10–11	SPT	5	SM-SC								
12.5				grading with gravel and rubble							
13–14	S&H	6		PI = 6, LL = 23					23.0	17.4	
15–16.5			ML	SILT (ML) gray, medium stiff, wet							
16–17	SPT	8									
16.5–17.5			SP	SAND (SP) black, loose, wet, with silt and organics	BAY MUD						
17.5–30			CH	CLAY (CH) gray, soft, wet							
19–21	ST	50 psi		Consolidation Test, See Figure B-2		TXUU	1,500	560		81.6 85.7	52 50
25–27	ST	50 psi		grading sandy							
30											

TEST GEOTECH LOG 324202.GPJ TR.GDT 6/20/02

Treadwell&Rollo

Project No.: 3242.02 Figure: A-4a

PROJECT: **BAYVIEW HOPE DEVELOPMENT**
San Francisco, California

Log of Boring B-4

DEPTH (feet)	Sampler Type	Sample	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
			50		CLAY (CH) (continued)						
31	ST		to 300 psi	SP-SC	SAND with CLAY (SP-SC) yellow brown and orange brown with gray mottling, very dense, wet						
32											
33											
34											
35					grading with trace gravel						
36	S&H		52								
37											
38											
39				SC	CLAYEY SAND (SC) orange-brown to yellow-brown, medium dense, wet						
40											
41	SPT		15								
42											
43											
44				SP	SAND (SP) gray, dense, wet						
45											
46	S&H		21	CL	SANDY CLAY (CL) gray to tan, very stiff, wet						
47											
48											
49					grading sandy						
50											
51	S&H		21	CL	CLAY (CL) gray, very stiff, wet						
52											
53											
54											
55											
56											
57											
58											
59											
60											

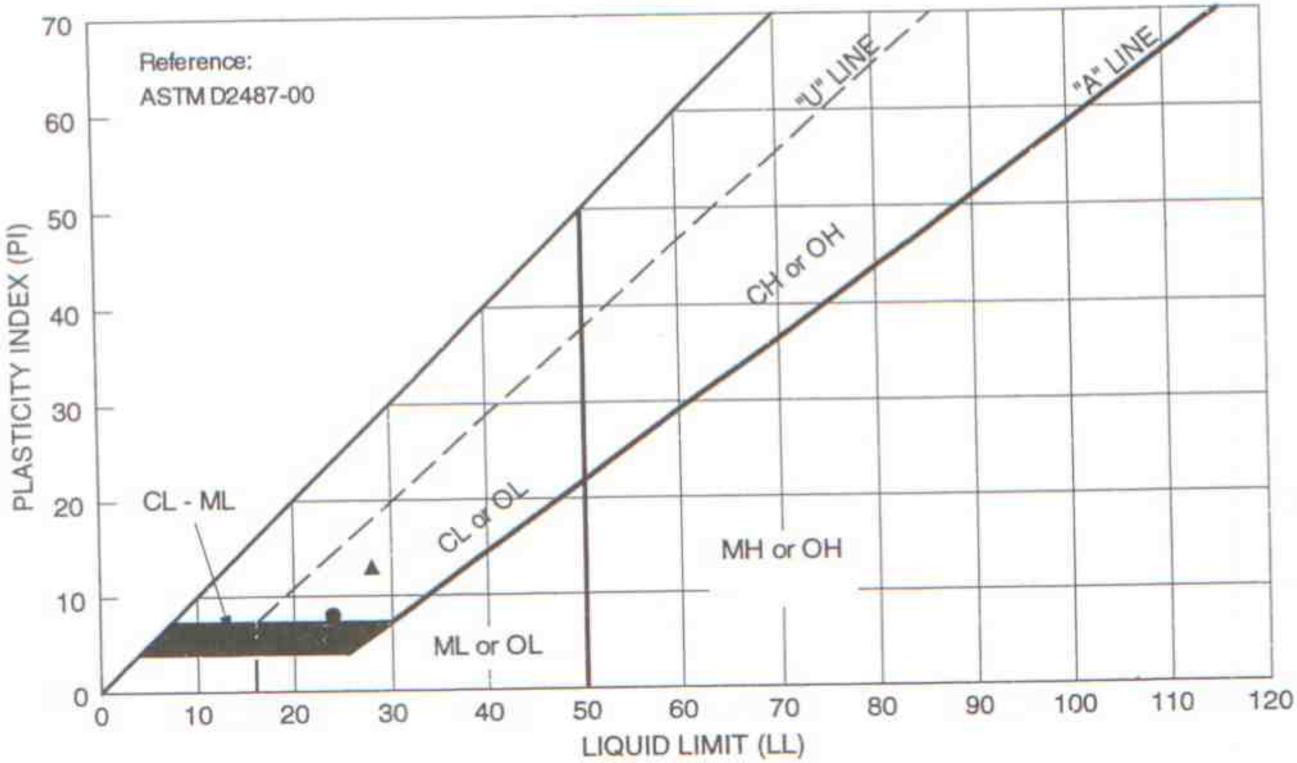
Boring terminated at 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater obscured by drilling fluid.

[1] S&H blow counts converted to SPT N-Values using a factor of 0.6.
[2] Elevations based on San Francisco City datum.

Treadwell&Rollo

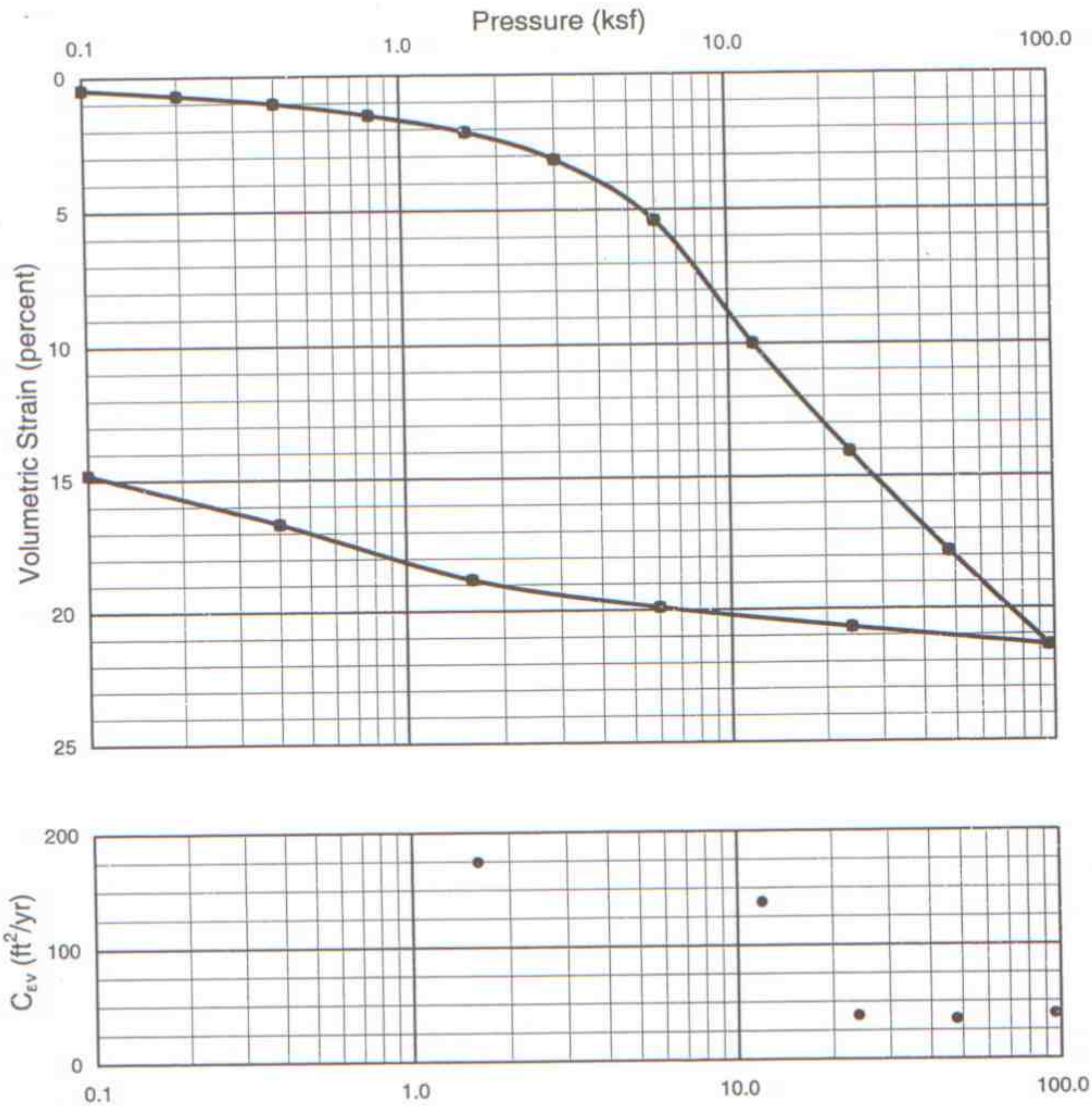
Project No.: 3242.02

Figure: A-4b



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 36 feet	CLAYEY SAND (SC), light to dark gray	17.4	23	6	23.0
▲	B-3 at 46 feet	CLAYEY SAND (SC), brown	19.2	28	13	—
■	B-4 at 13 feet	SILTY-CLAYEY SAND (SM-SC), black	23.7	24	8	—

BAYVIEW HOPE DEVELOPMENT San Francisco, California	**PLASTICITY CHART**
Treadwell&Rollo	Date 06/18/02 · Project No. 3242.02 · Figure B-1



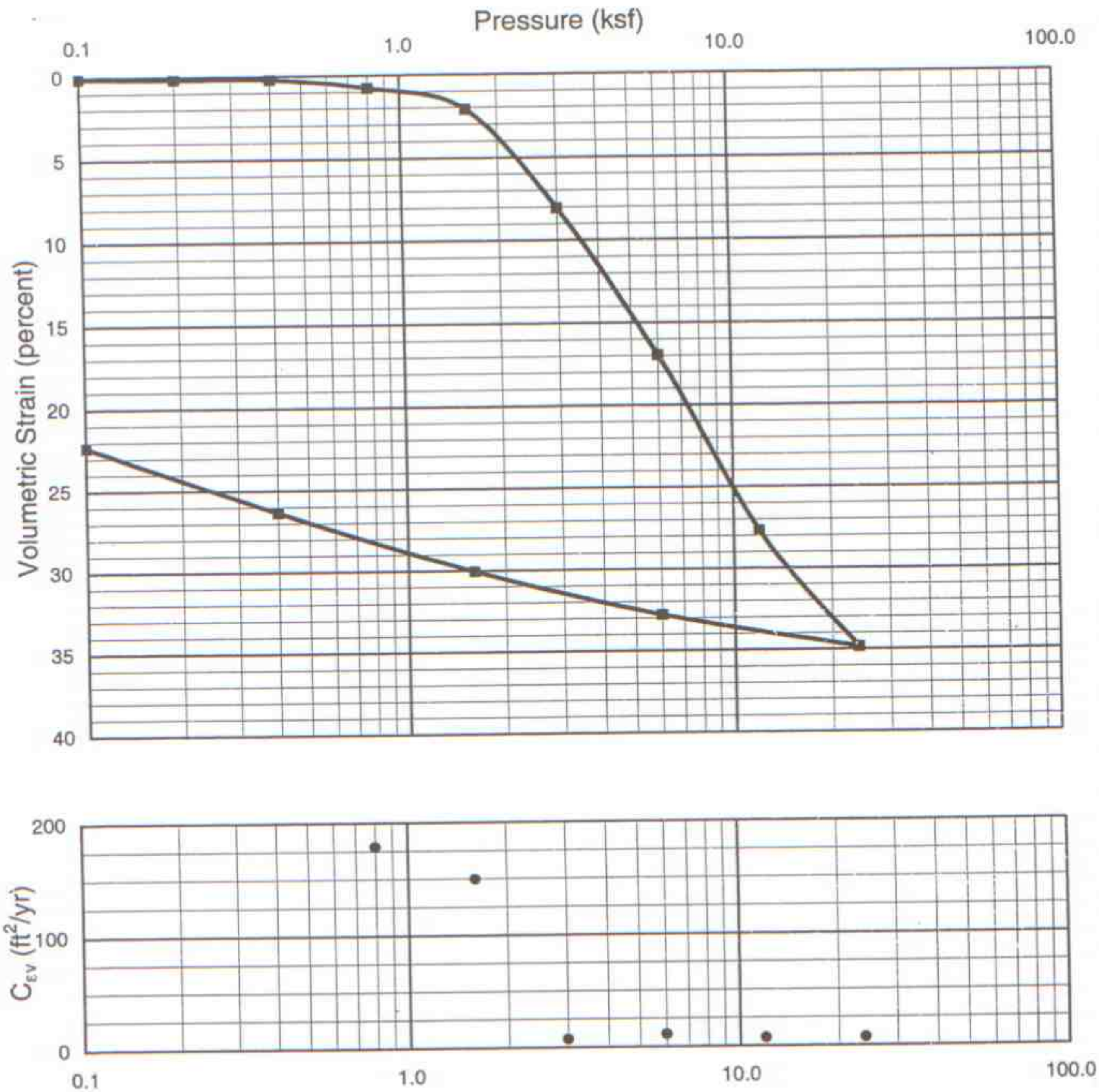
Sampler Type: Shelby Tube		Condition		Before Test		After Test	
Diameter (in) 2.42	Height (in) 1.00	Water Content	w_o	23.5 %	w_f	16.4 %	
Overburden Pressure, p_o	2,600 psf	Void Ratio	e_o	0.66	e_f	0.41	
Preconsol. Pressure, p_c	5,100 psf	Saturation	S_o	97 %	S_f	100 %	
Compression Ratio, $C_{\varepsilon c}$	0.14	Dry Density	γ_d	102 pcf	γ_d	120 pcf	
LL	PL	PI			G_s	2.70	(assumed)
Classification CLAY (CH), dark gray			Source	B-2	at	36	feet

BAYVIEW HOPE DEVELOPMENT
San Francisco, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date 06/25/02 | Project No. 3242.02 | Figure B-2



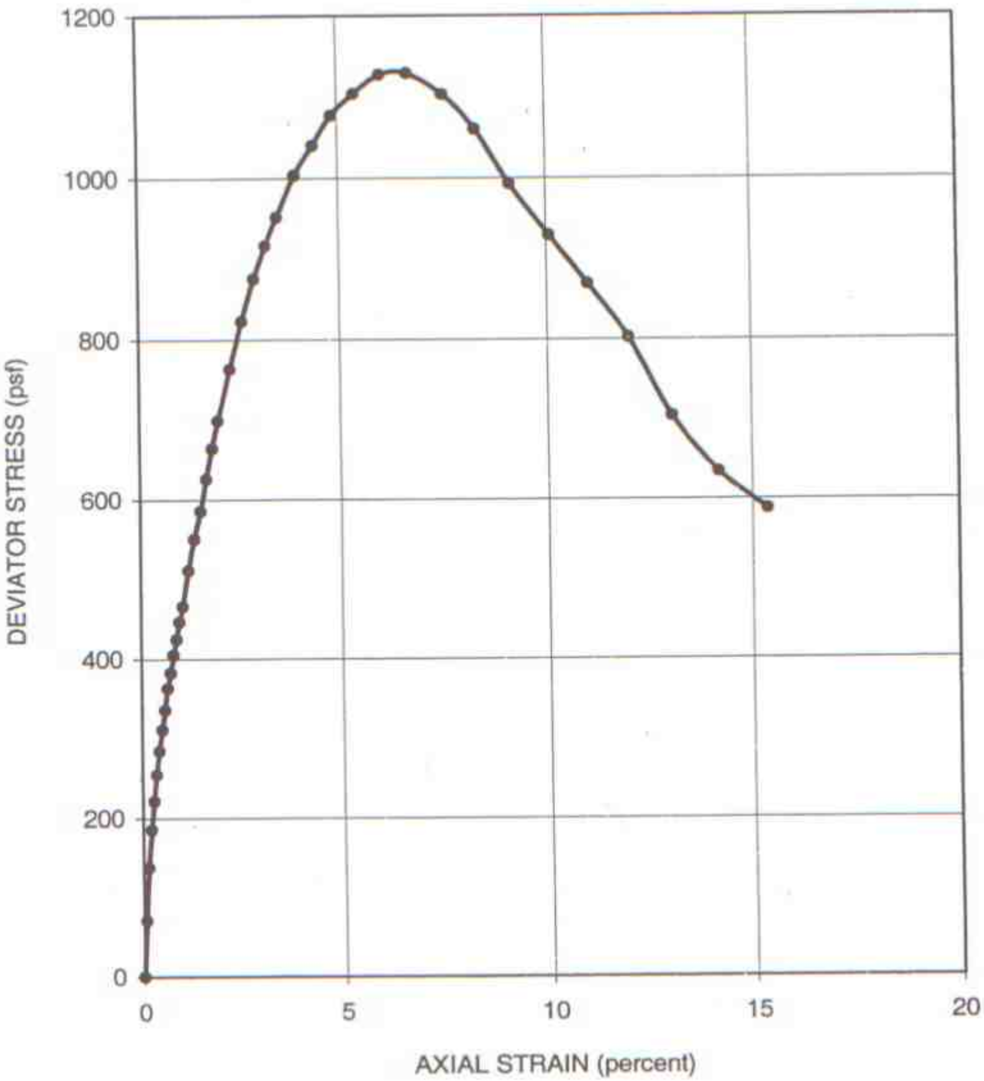
Sampler Type: Shelby Tube				Condition	Before Test			After Test		
Diameter (in)	2.42	Height (in)	1.00	Water Content	w_o	81.6	%	w_f	56.3	%
Overburden Pressure, p_o	1,700		psf	Void Ratio	e_o	2.28		e_f	1.54	
Preconsol. Pressure, p_c	2,100		psf	Saturation	S_o	99	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.32			Dry Density	γ_d	52	pcf	γ_d	67	pcf
LL	PL			PI			G_s	2.75	(assumed)	
Classification CLAY (CH), gray				Source		B-4	at	20	feet	

BAYVIEW HOPE DEVELOPMENT
San Francisco, California

Treadwell&Rollo

CONSOLIDATION TEST REPORT

Date	06/25/02	Project No.	3242.02	Figure	B-3



SAMPLER TYPE	Shelby Tube			SHEAR STRENGTH	560	psf
DIAMETER (in.)	2.860	HEIGHT (in.)	6	STRAIN AT FAILURE	6.7	%
MOISTURE CONTENT		85.7	%	CONFINING PRESSURE	1,500	psf
DRY DENSITY		50	pcf	STRAIN RATE	0.67	% / min
DESCRIPTION	CLAY (CH), dark gray			SOURCE	B-4 @ 19 feet	

BAYVIEW HOPE DEVELOPMENT
San Francisco, California

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Treadwell&Rollo

Date 06/20/02 | Project No. 3242.02 | Figure B-4

APPENDIX F

Liquefaction Analyses

APPENDIX F

ENGEO
332 Pine Street, Suite 300
San Francisco, CA 94104

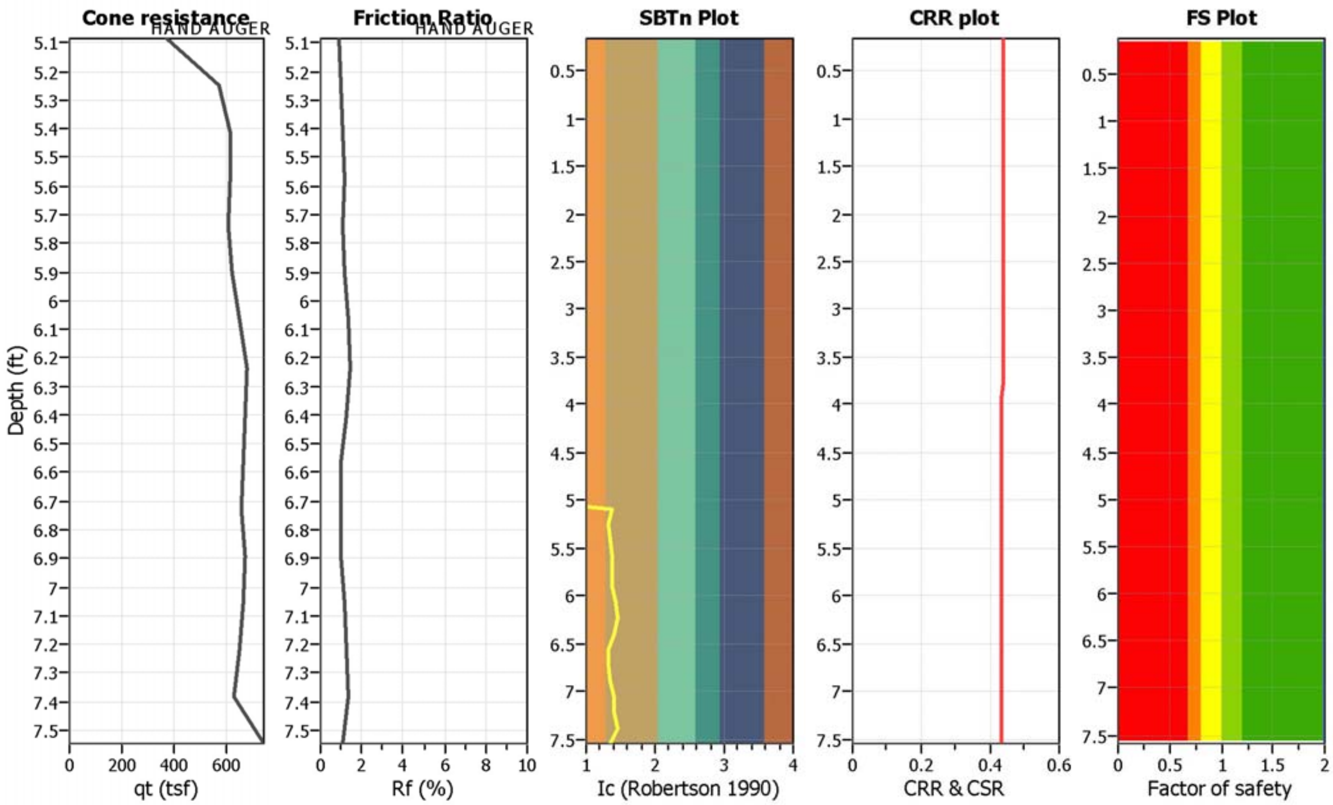
LIQUEFACTION ANALYSIS REPORT

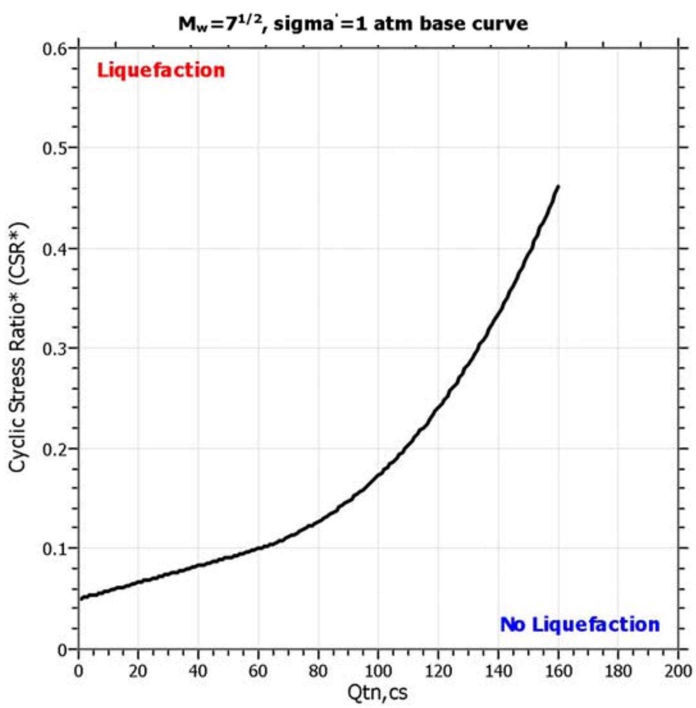
Project title : Alice Griffith Residential Development **Location : San Francisco, California**

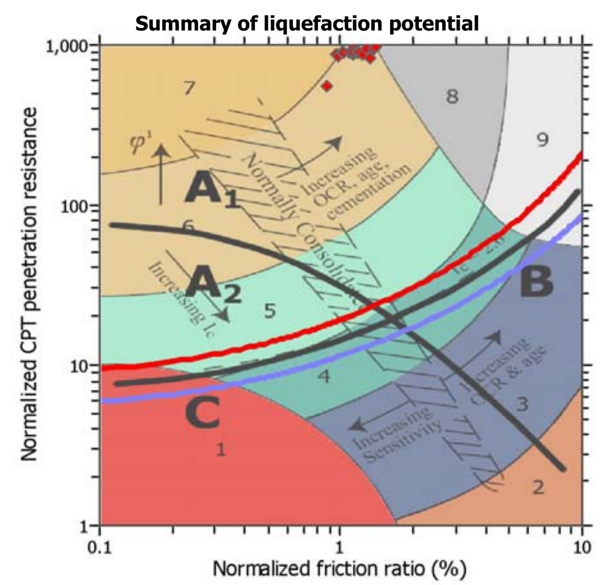
CPT file : 2-CP-CPT1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	No		

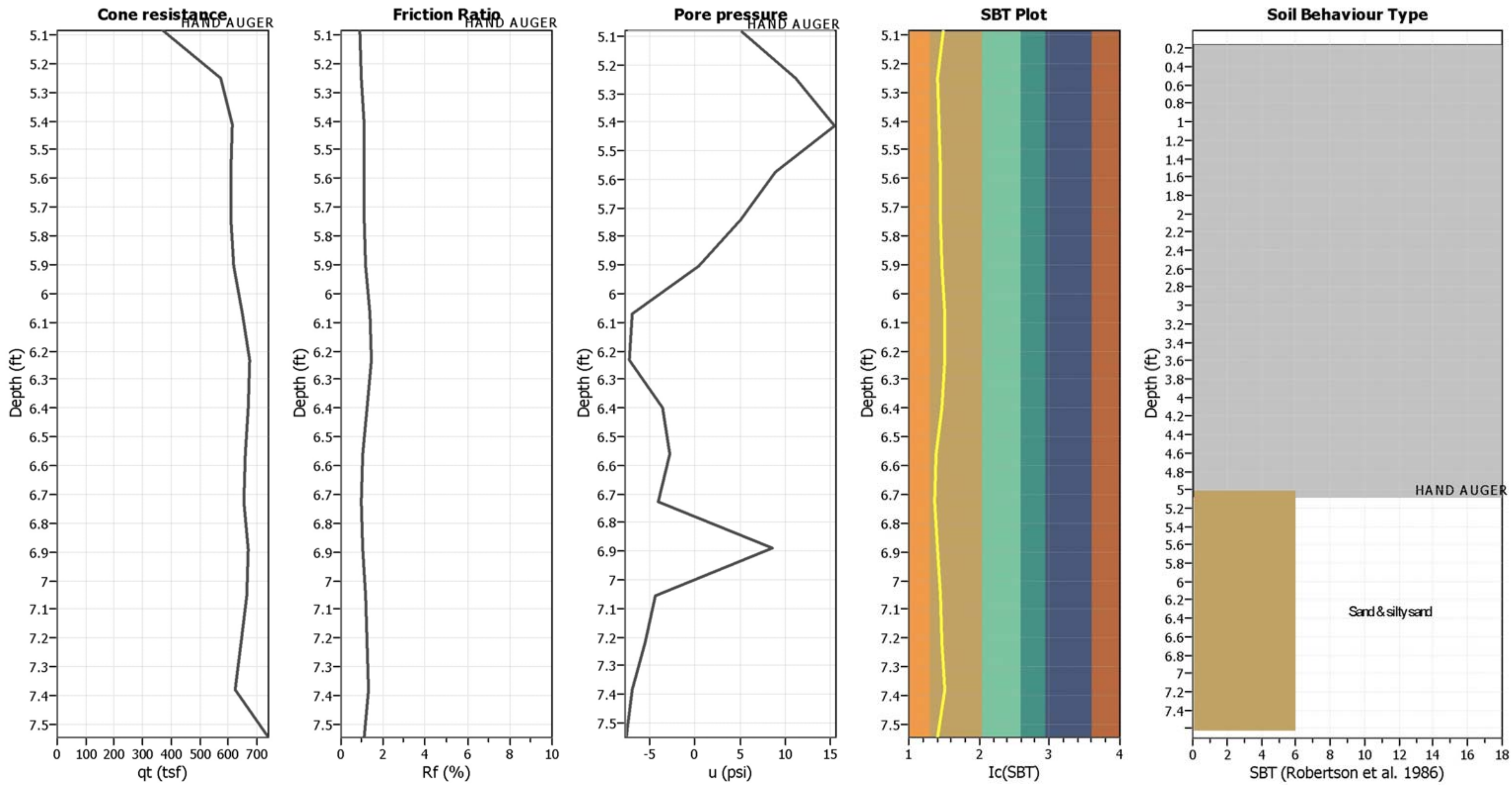






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



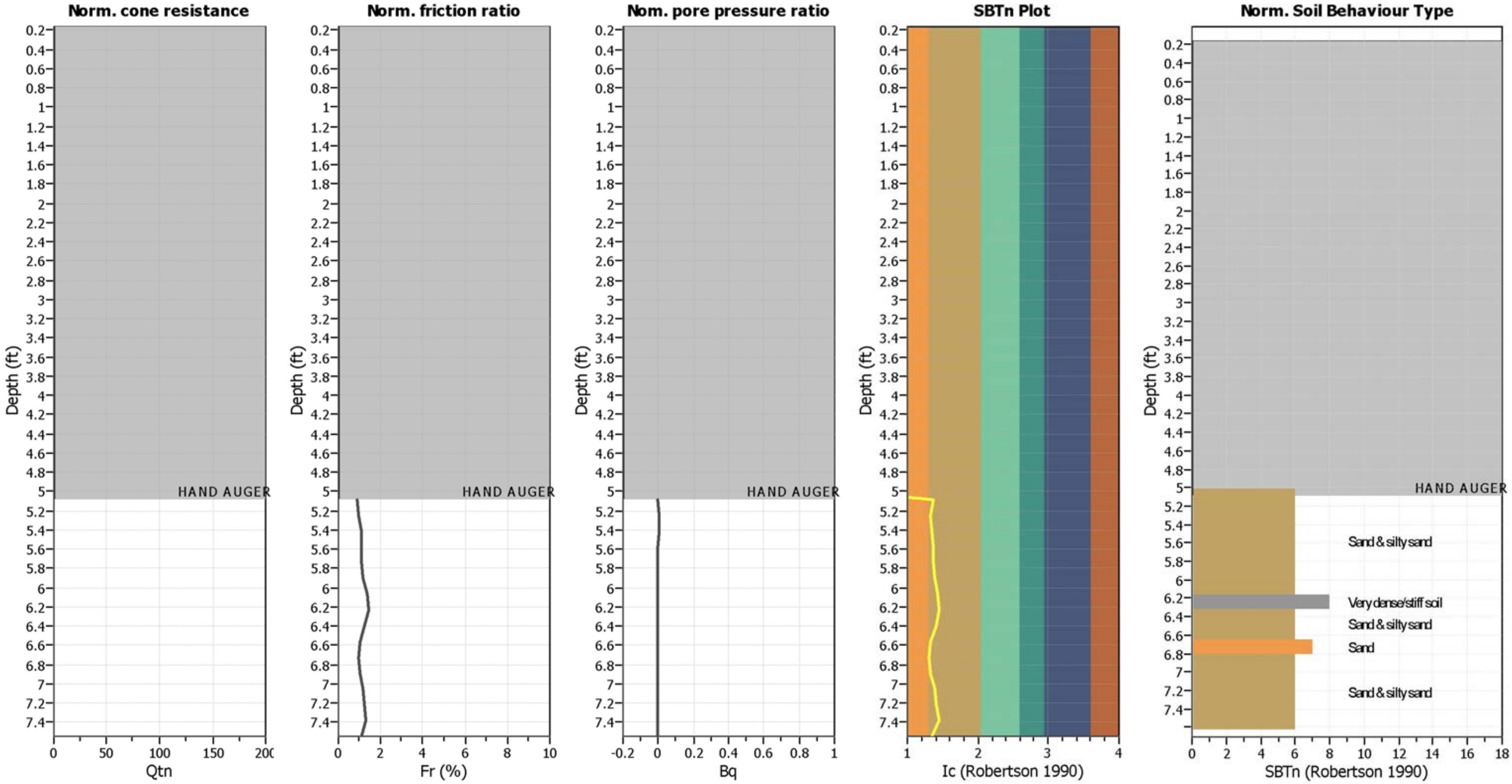
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



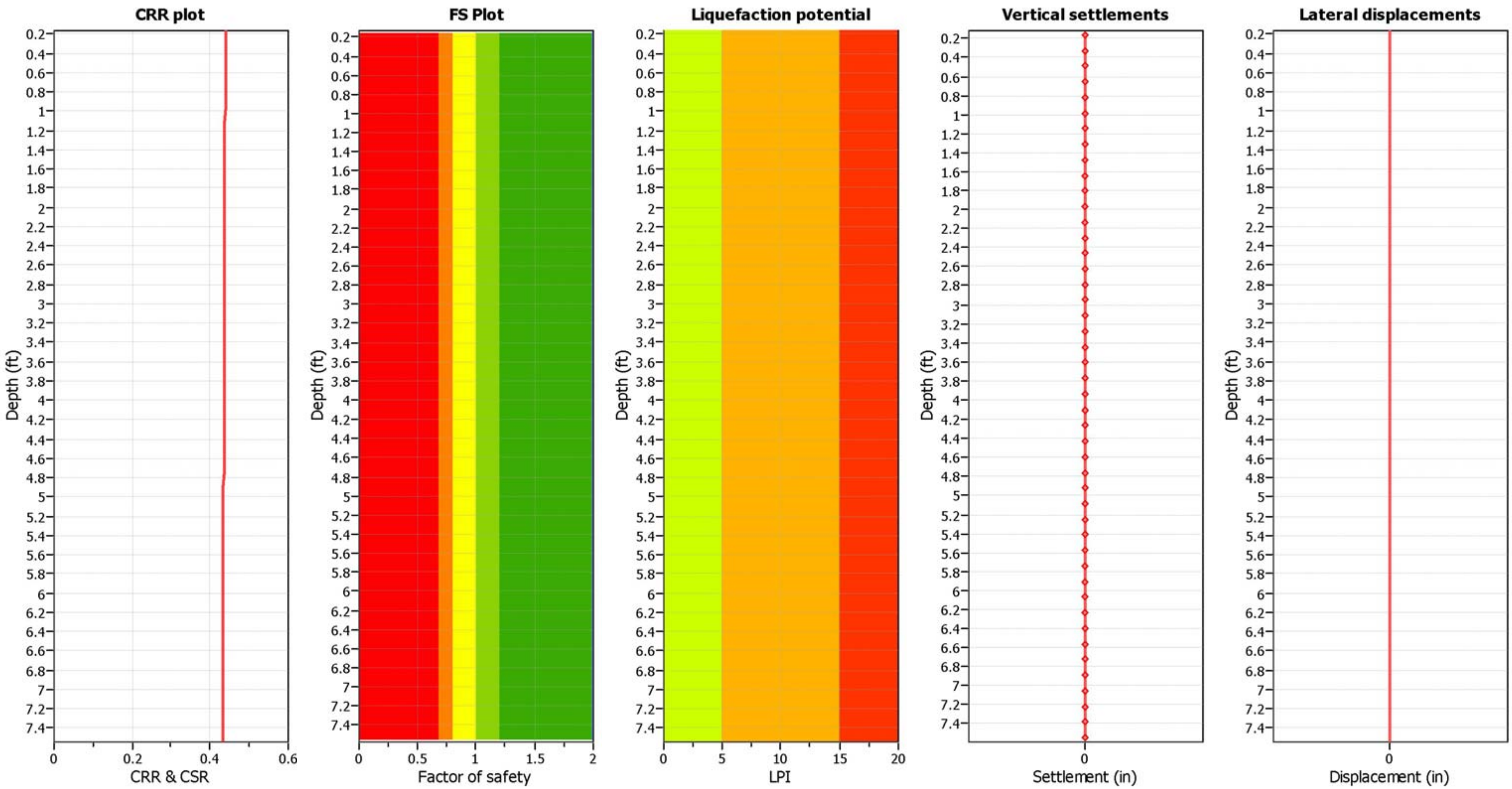
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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 liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

ENGEO
332 Pine Street, Suite 300
San Francisco, CA 94104

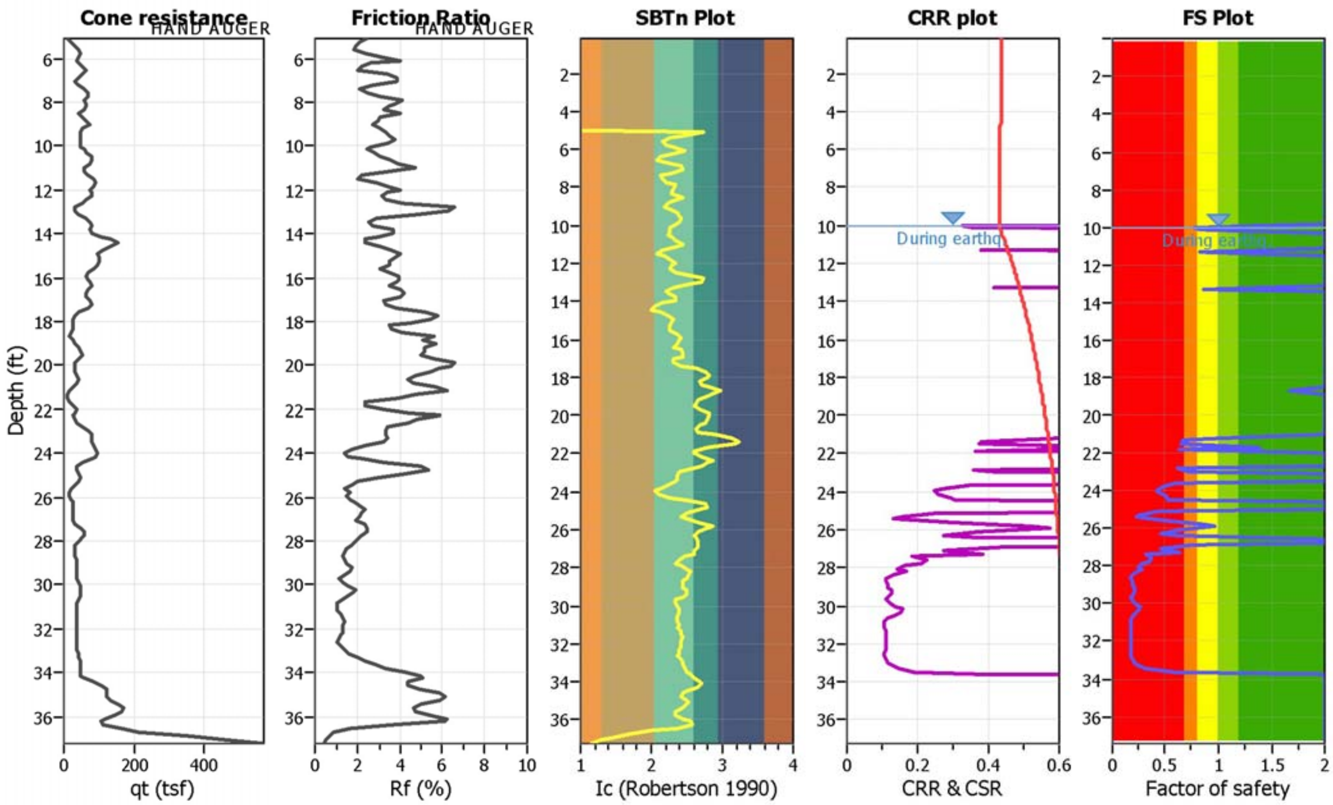
LIQUEFACTION ANALYSIS REPORT

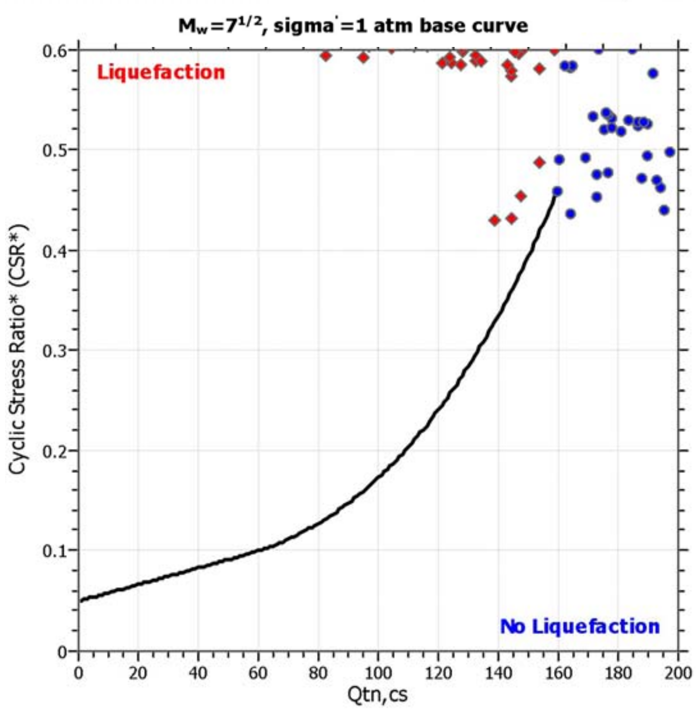
Project title : Alice Griffith Residential Development **Location : San Francisco, California**

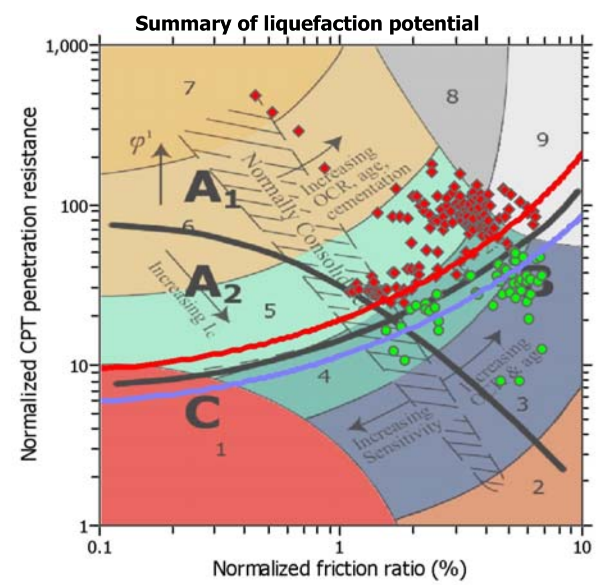
CPT file : 2-CP-CPT3

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	No		

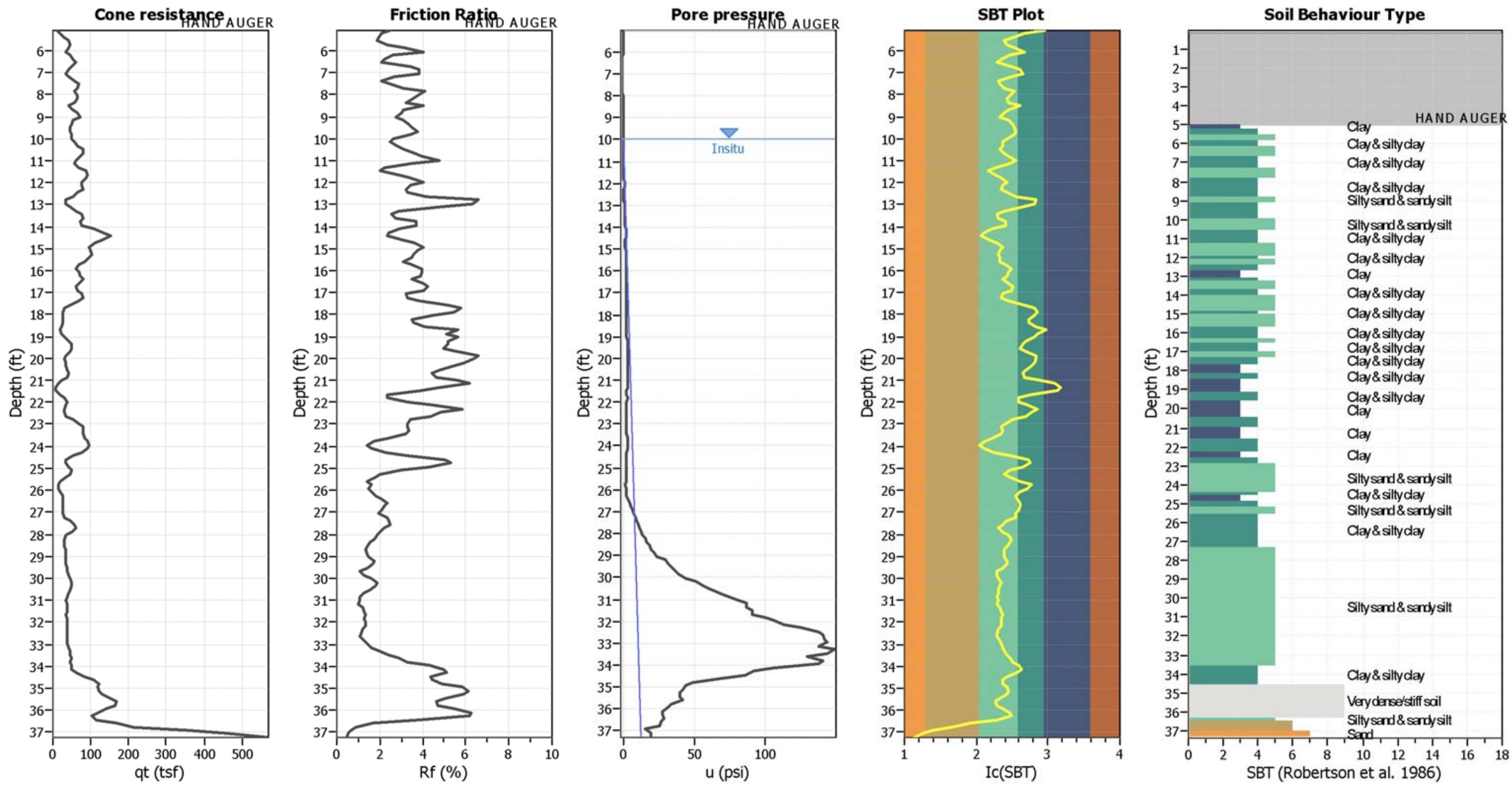






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



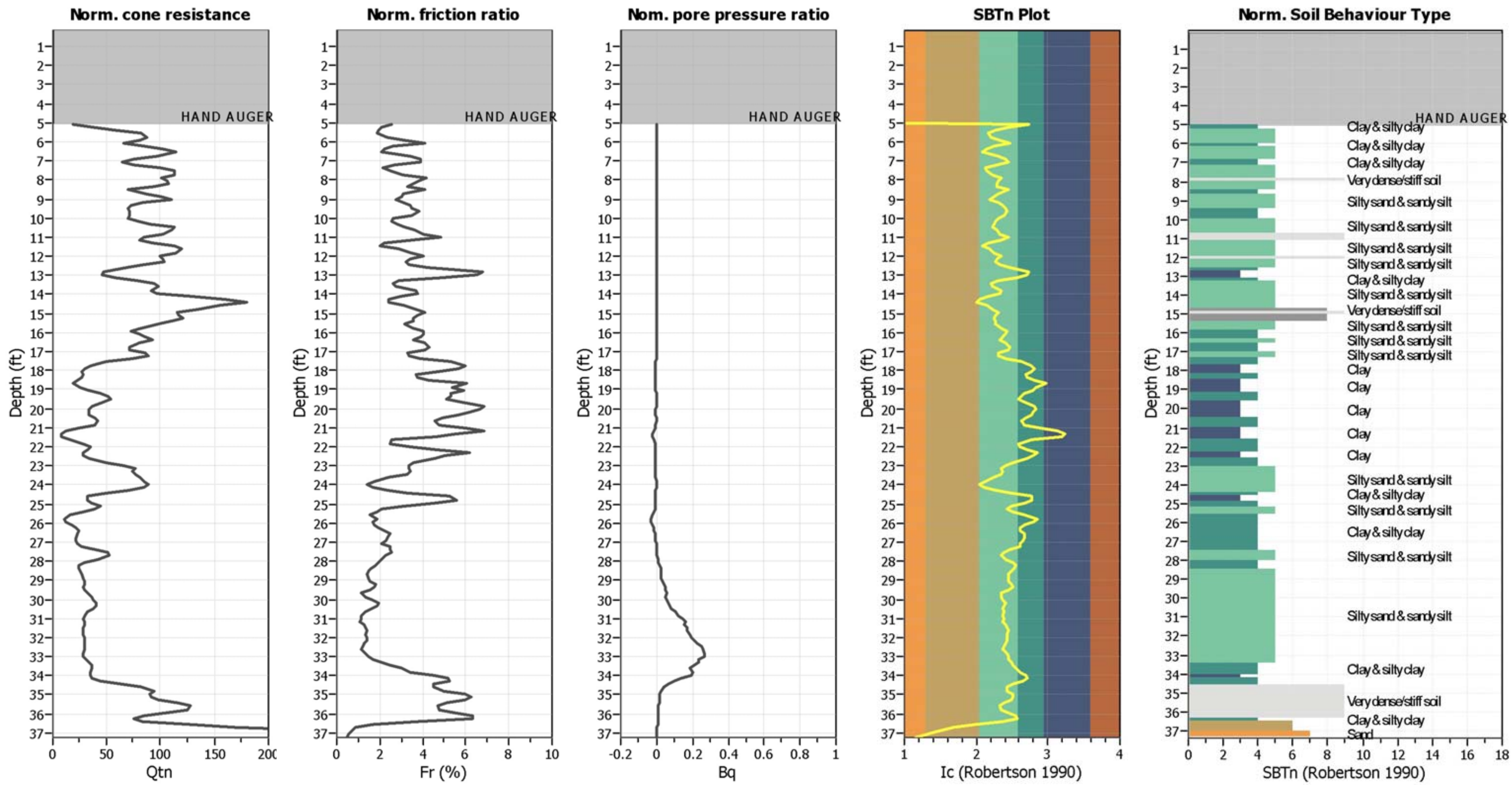
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

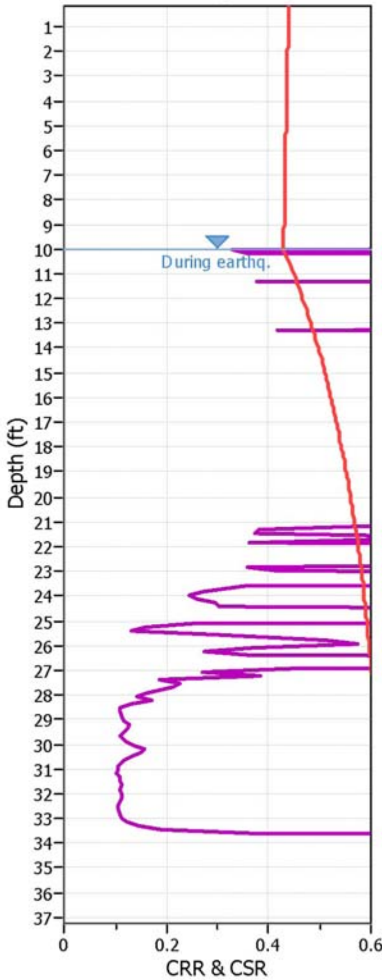
Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

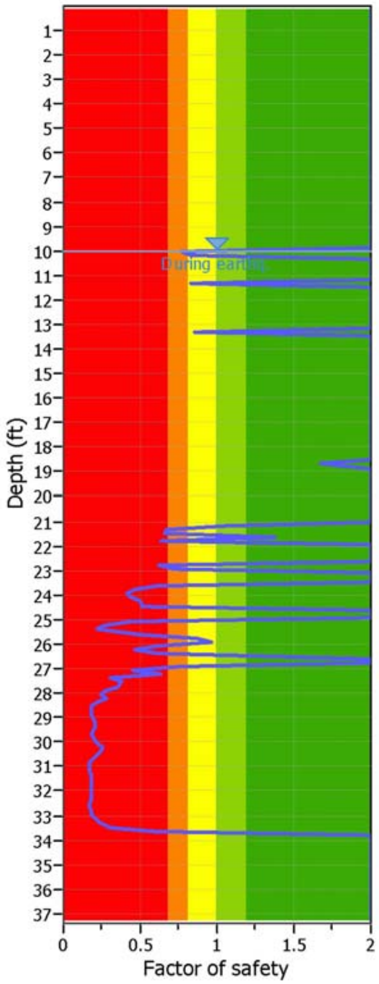
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots

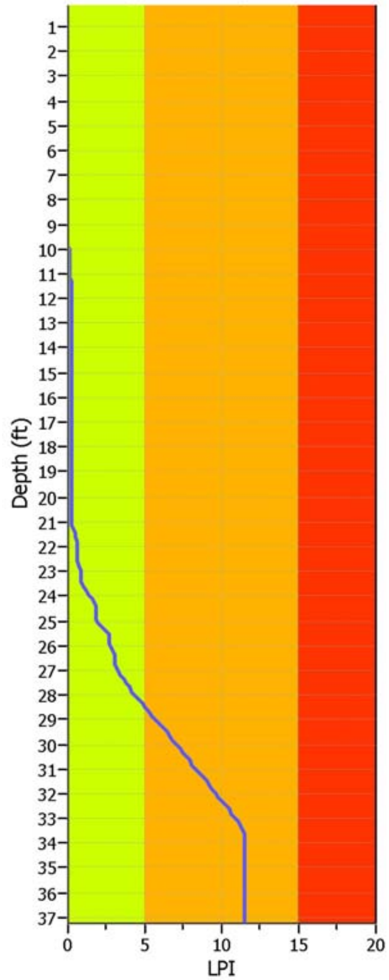
CRR plot



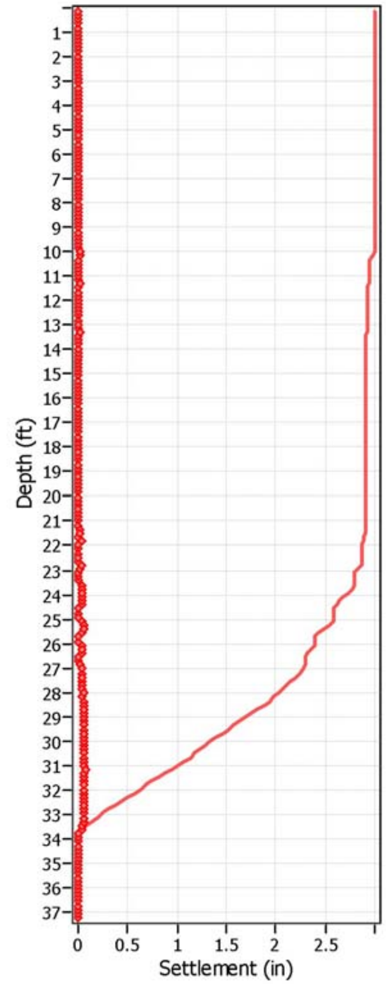
FS Plot



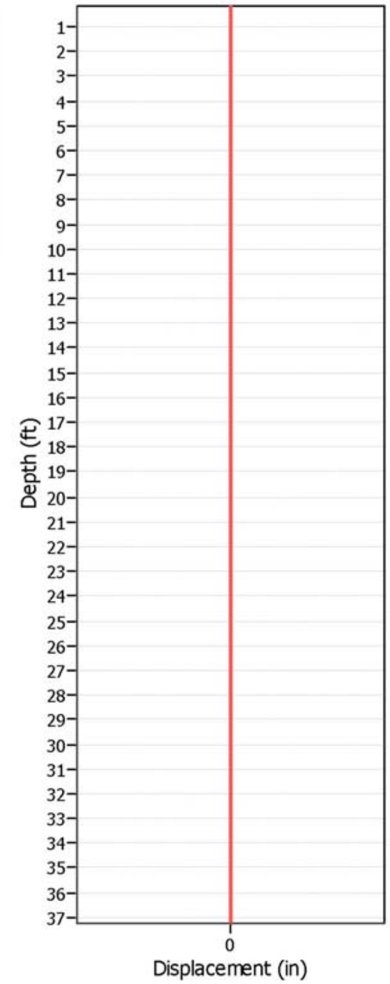
Liquefaction potential



Vertical settlements



Lateral displacements



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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 liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO
332 Pine Street, Suite 300
San Francisco, CA 94104

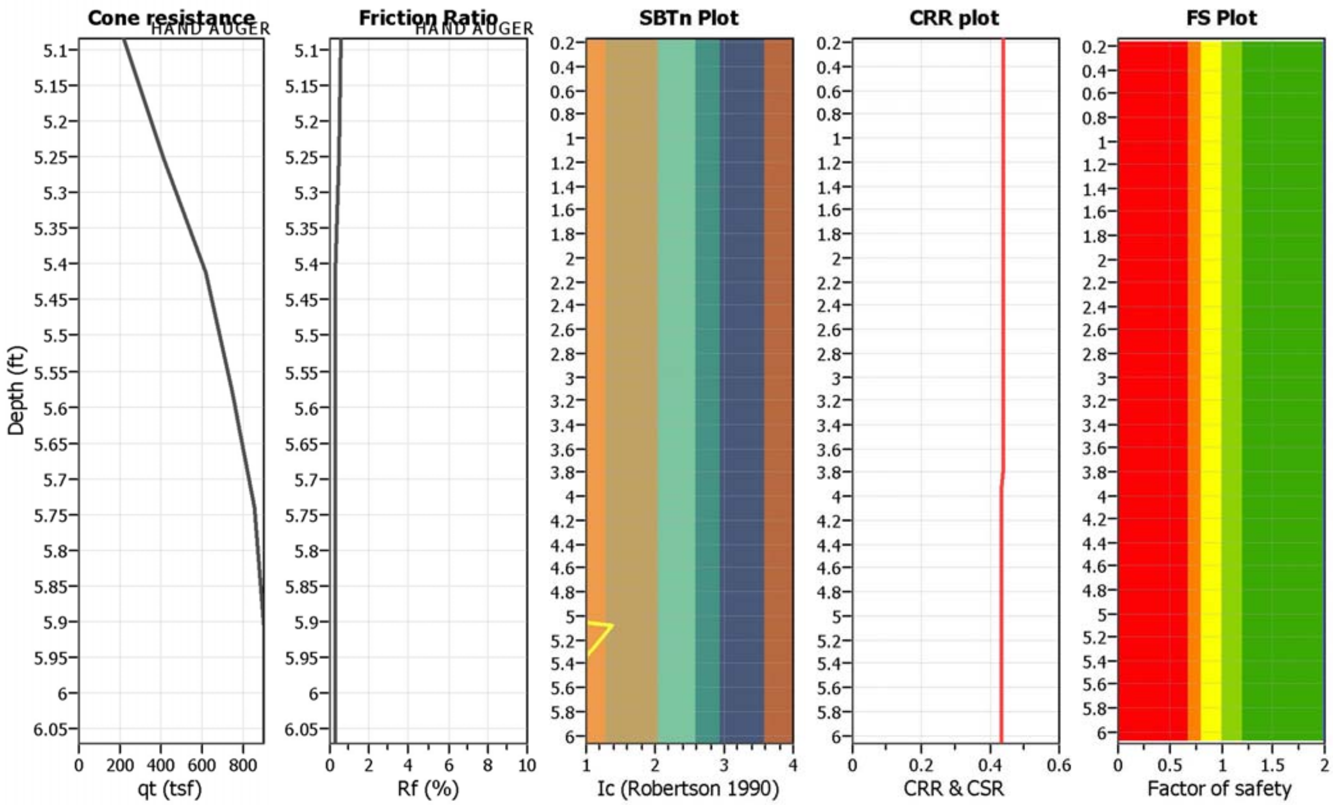
LIQUEFACTION ANALYSIS REPORT

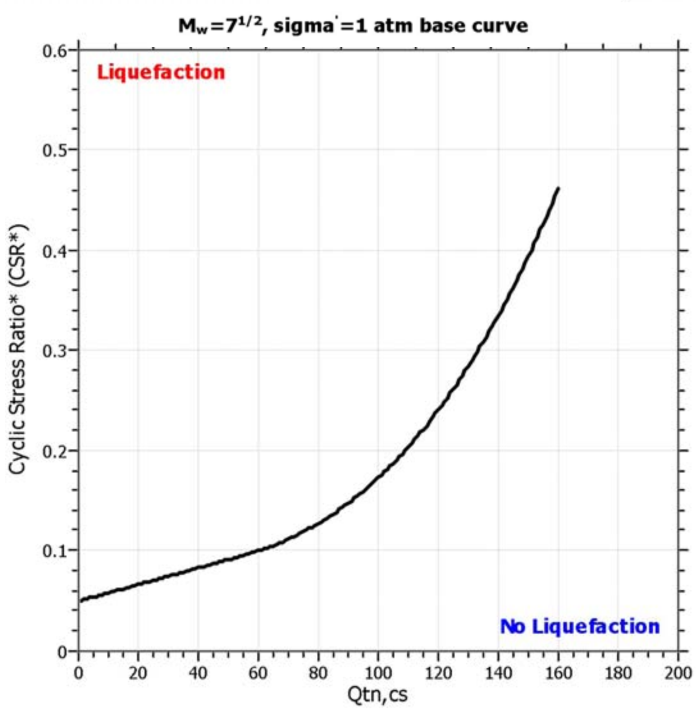
Project title : Alice Griffith Residential Development **Location : San Francisco, California**

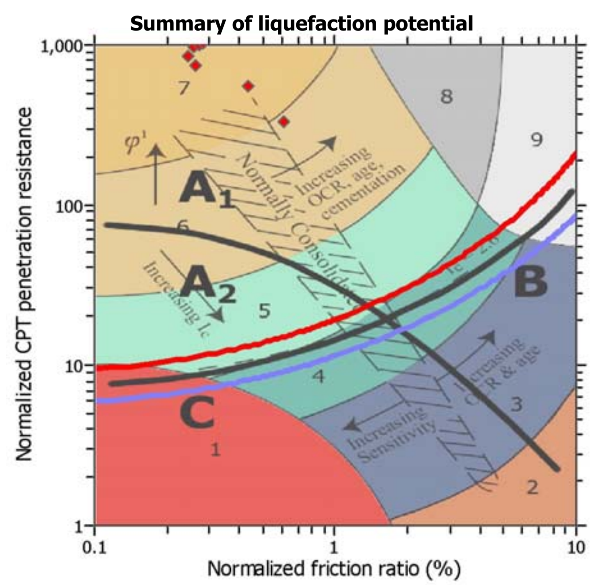
CPT file : 2-CP-CPT4

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	No		

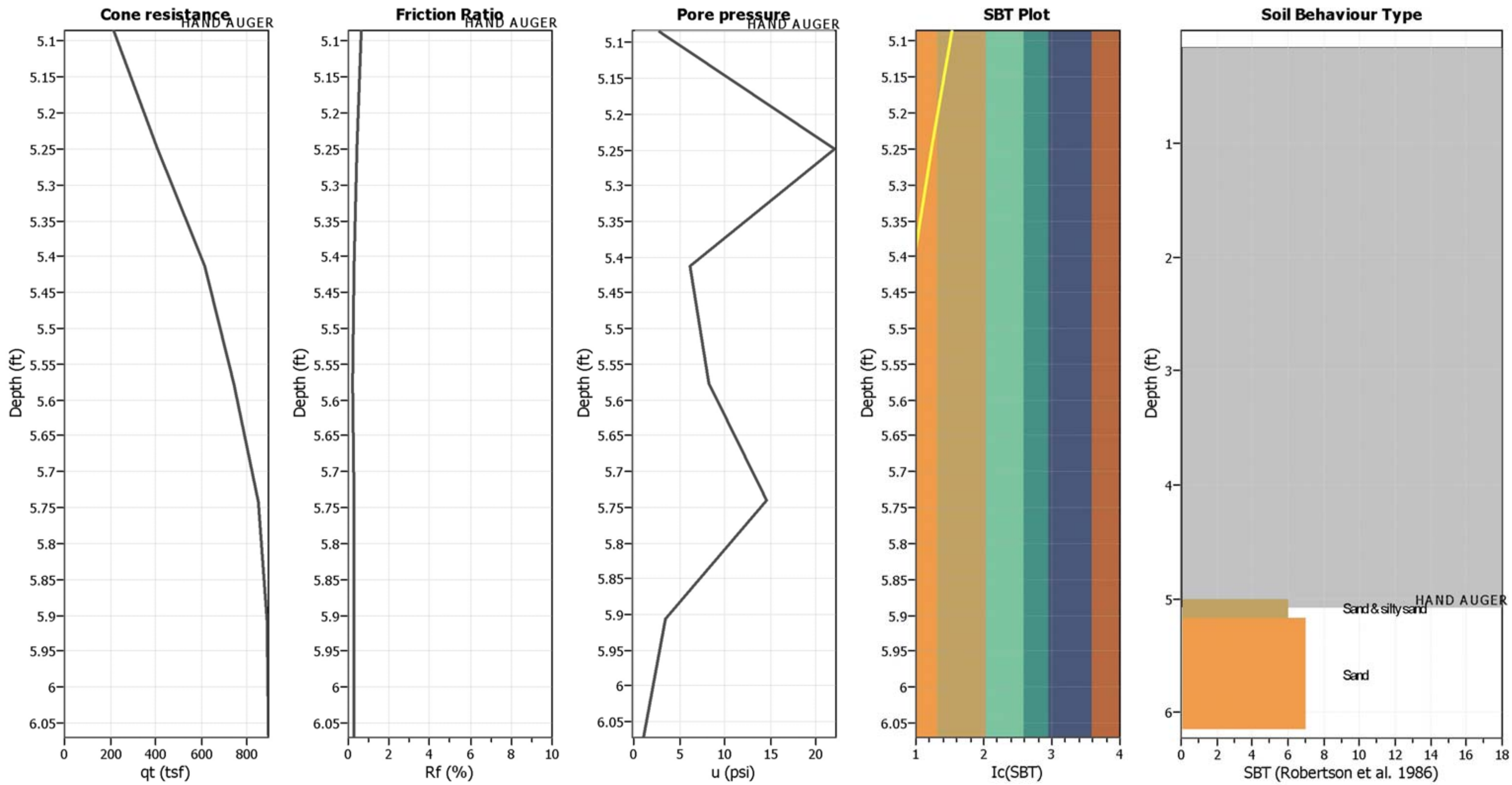






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



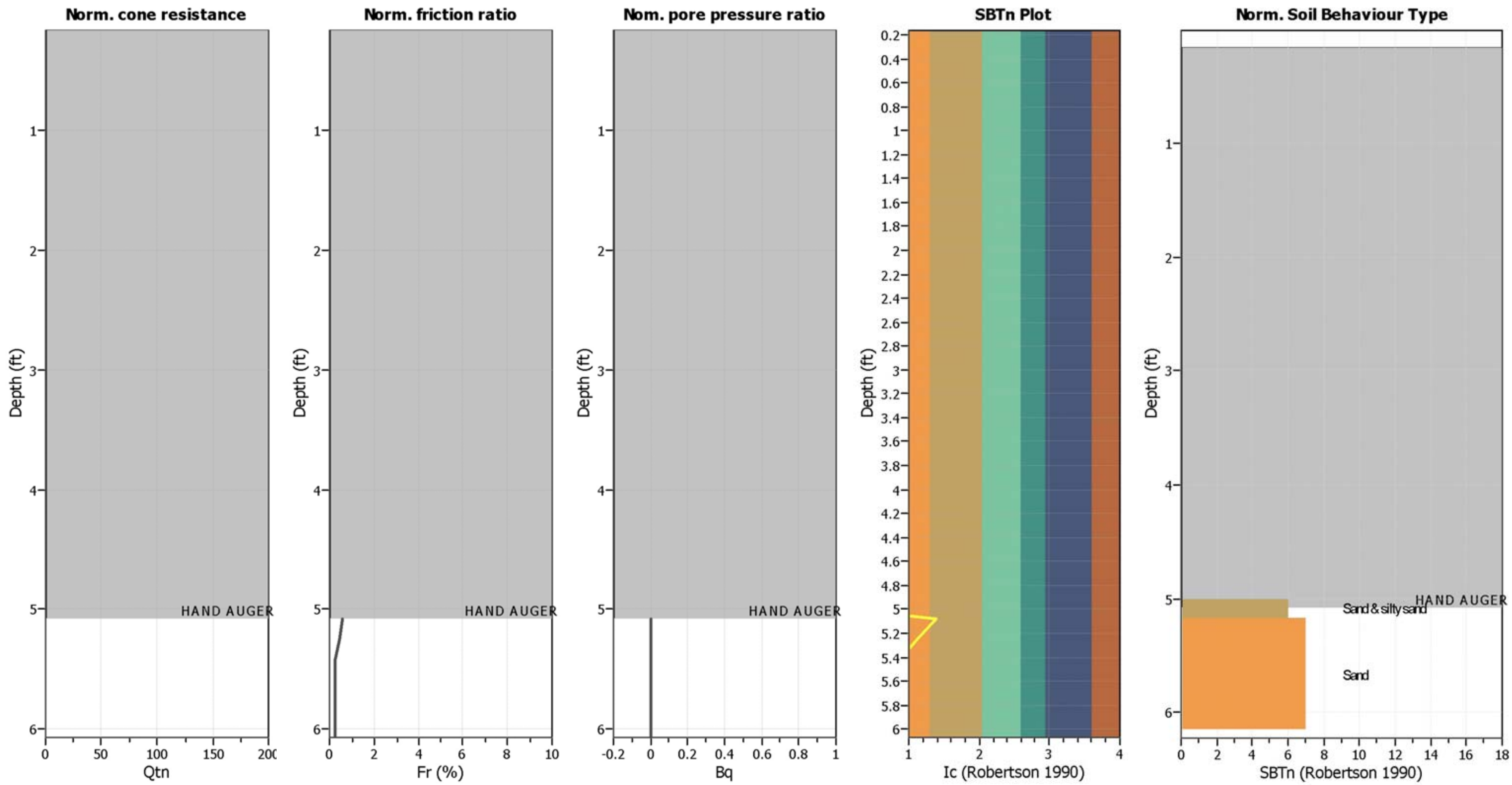
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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

CPT basic interpretation plots (normalized)



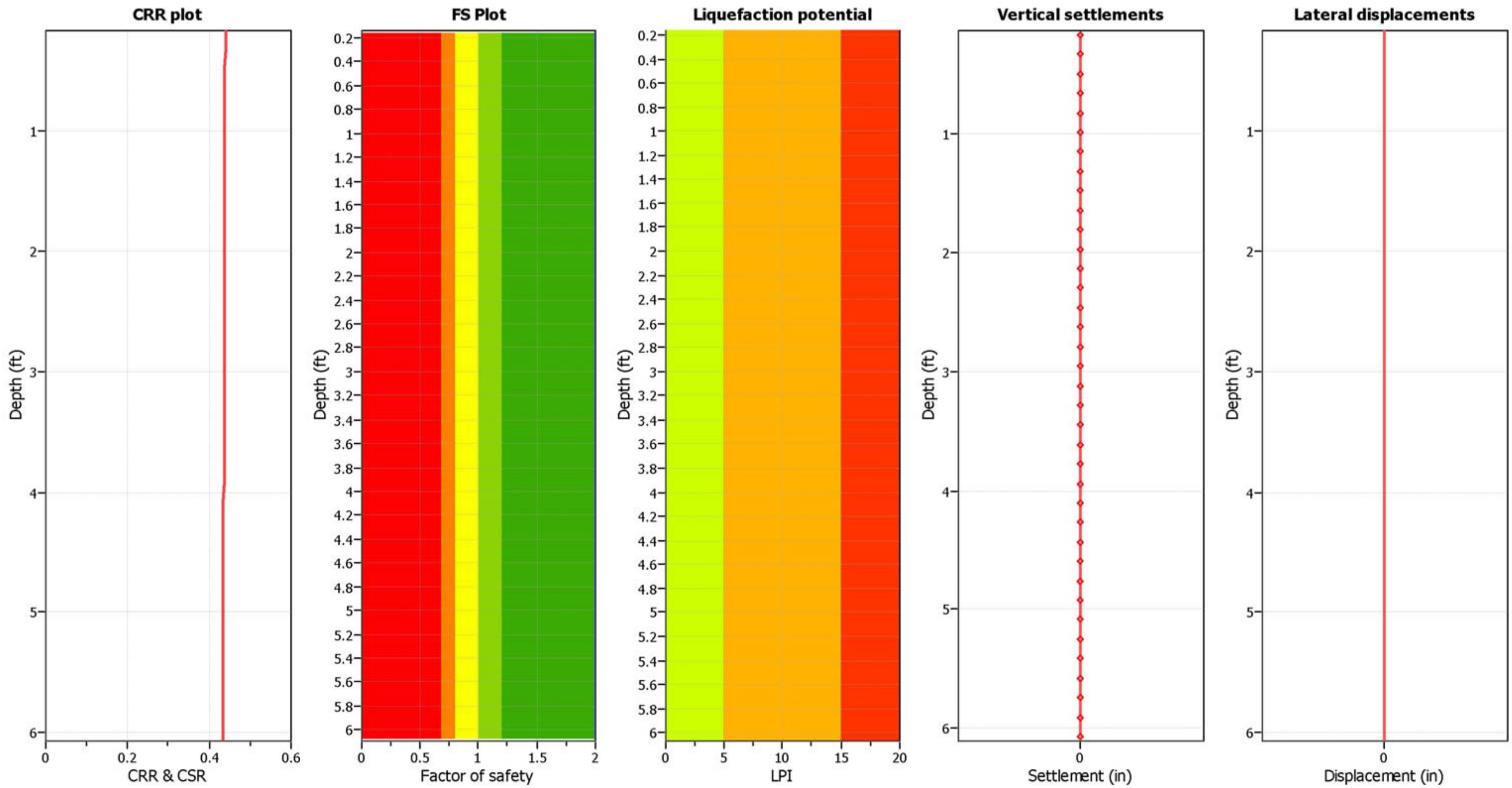
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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 liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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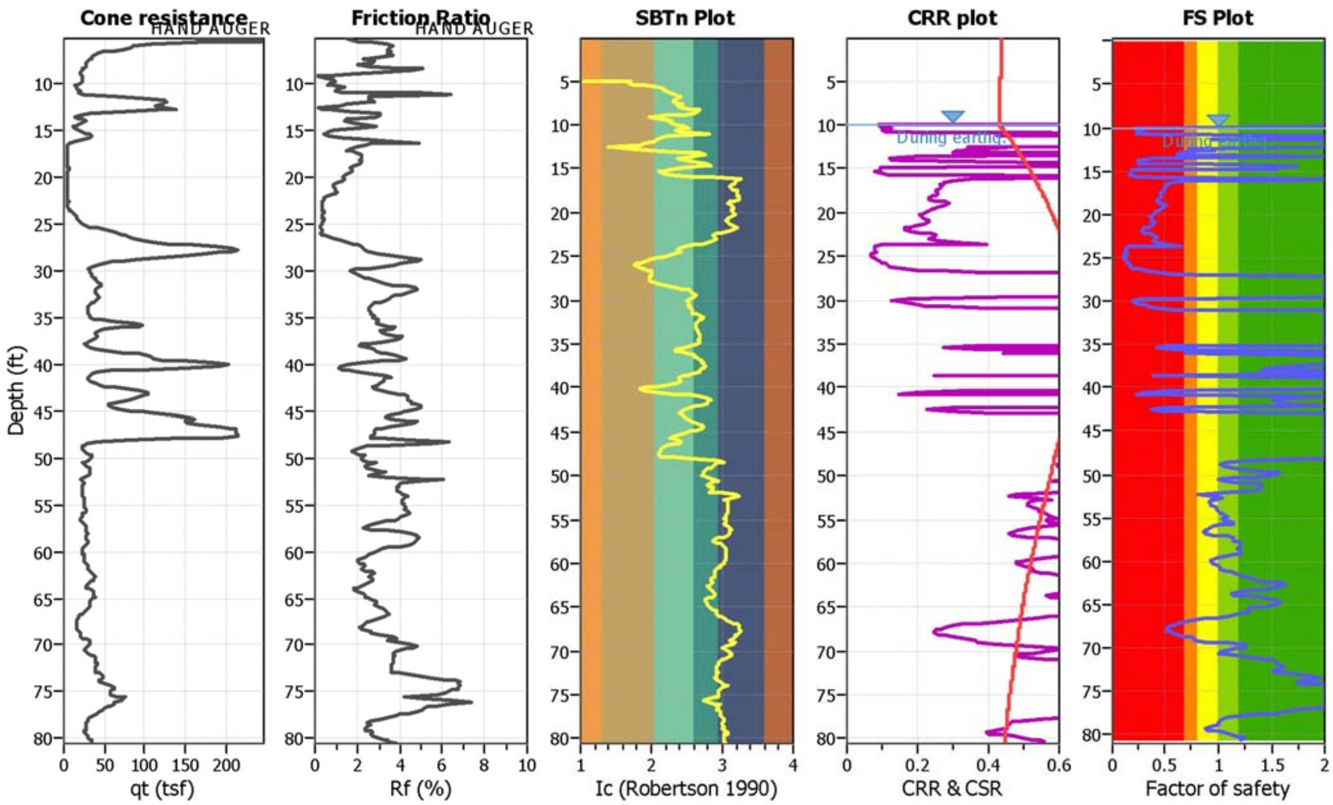
LIQUEFACTION ANALYSIS REPORT

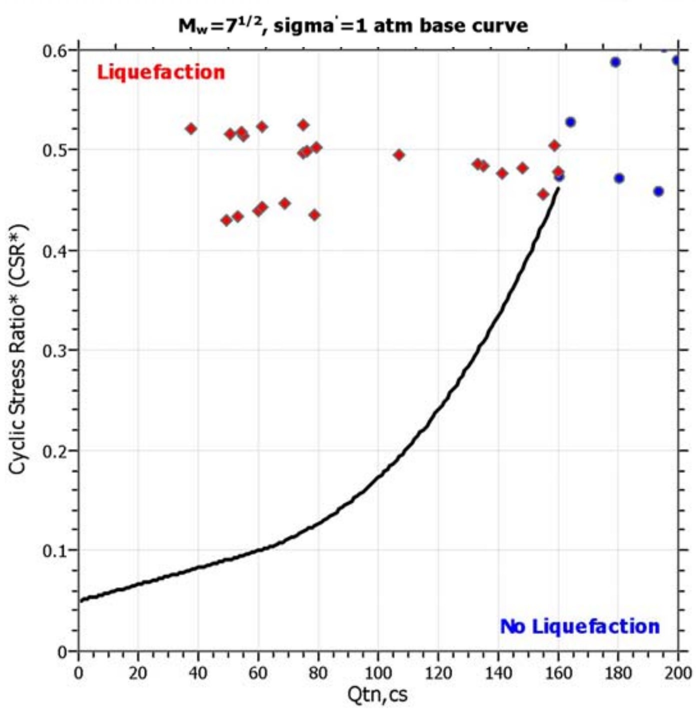
Project title : Alice Griffith Residential Development **Location : San Francisco, California**

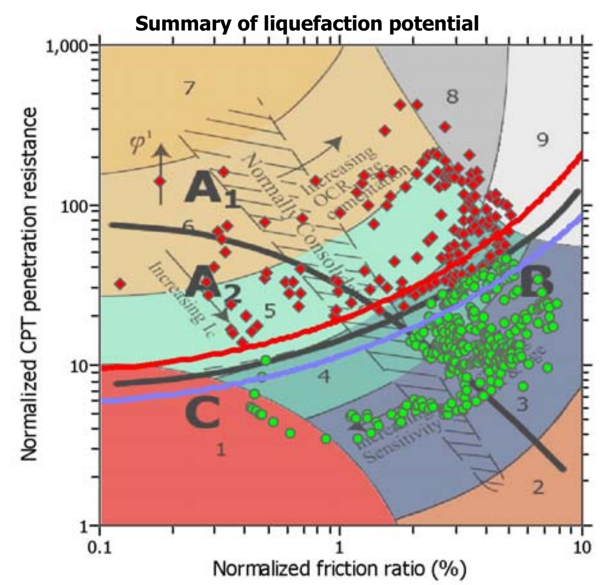
CPT file : 2-CP-CPT5

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	No		

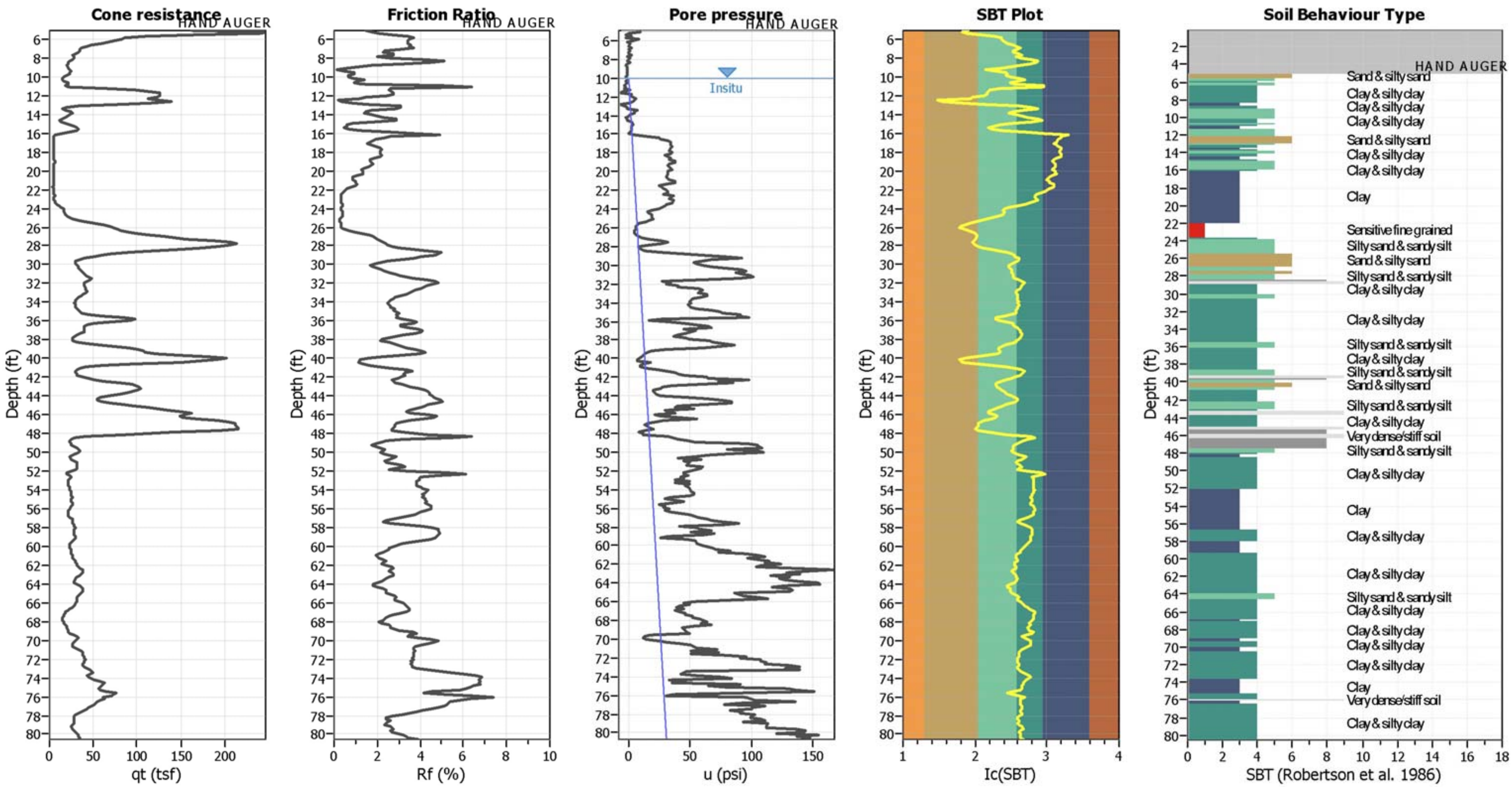






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



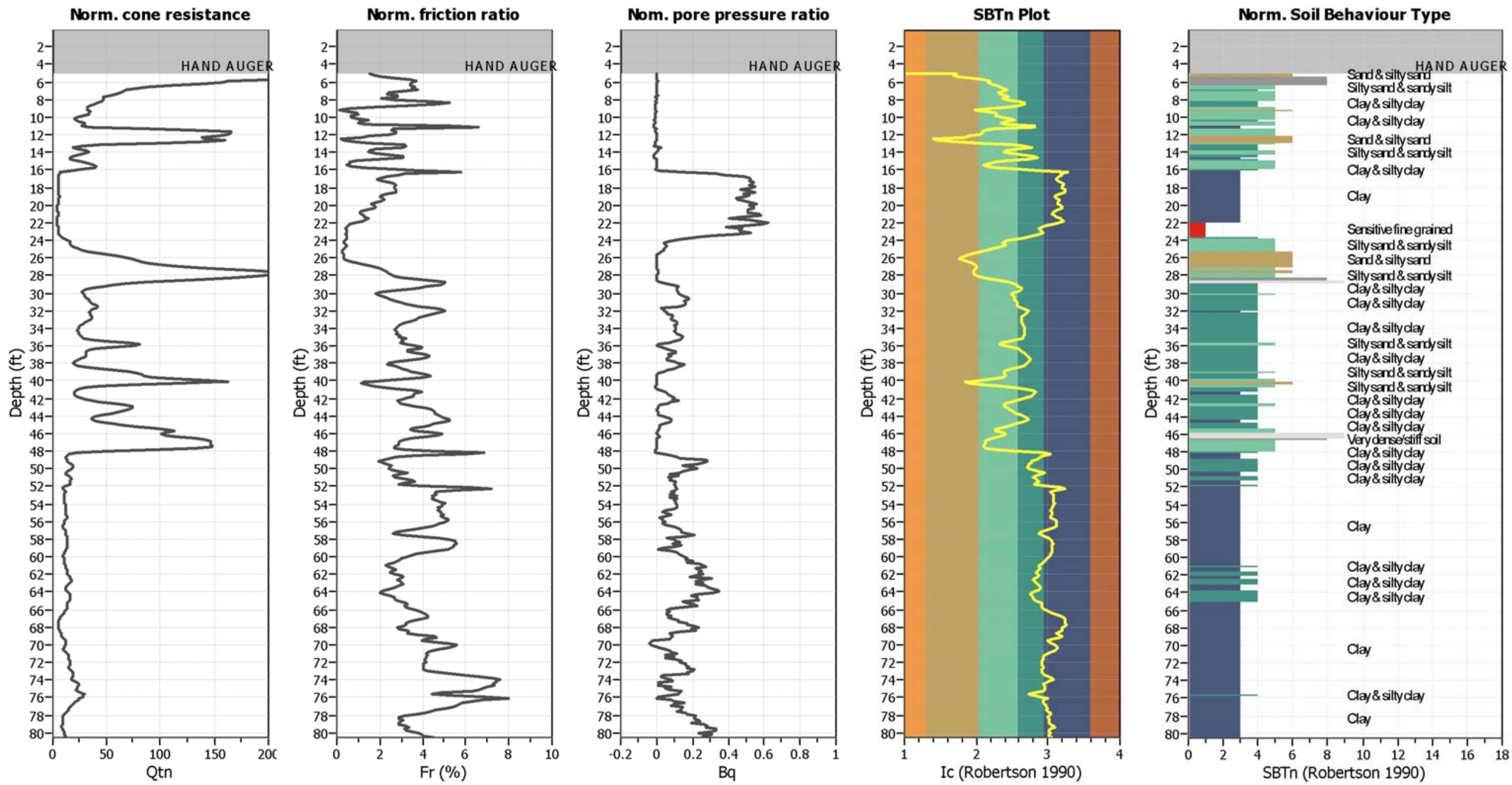
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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

CPT basic interpretation plots (normalized)



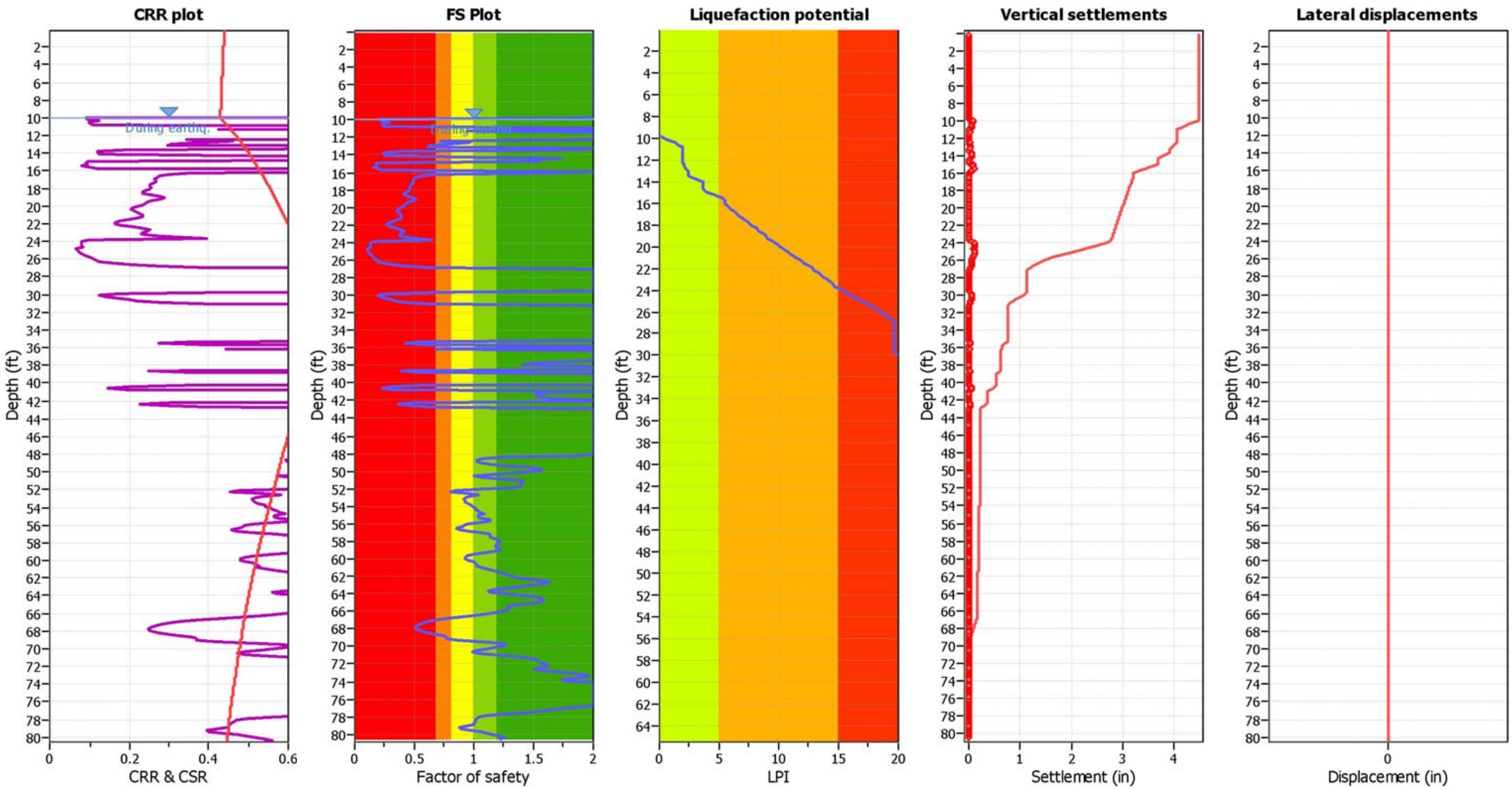
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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 liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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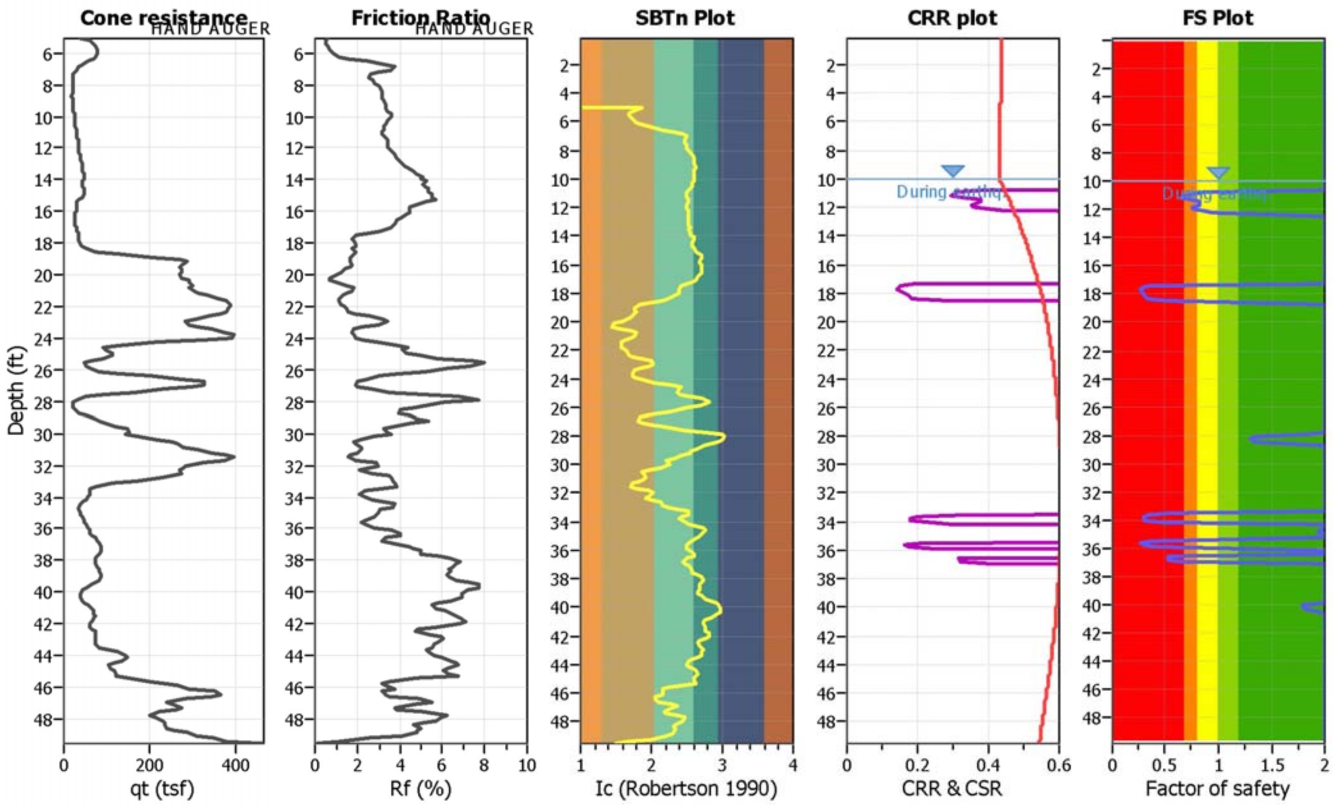
LIQUEFACTION ANALYSIS REPORT

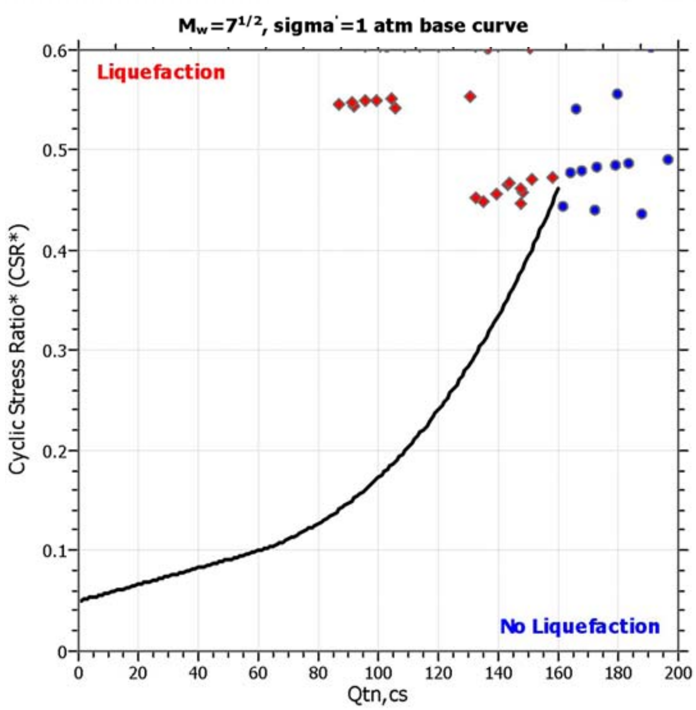
Project title : Alice Griffith Residential Development **Location : San Francisco, California**

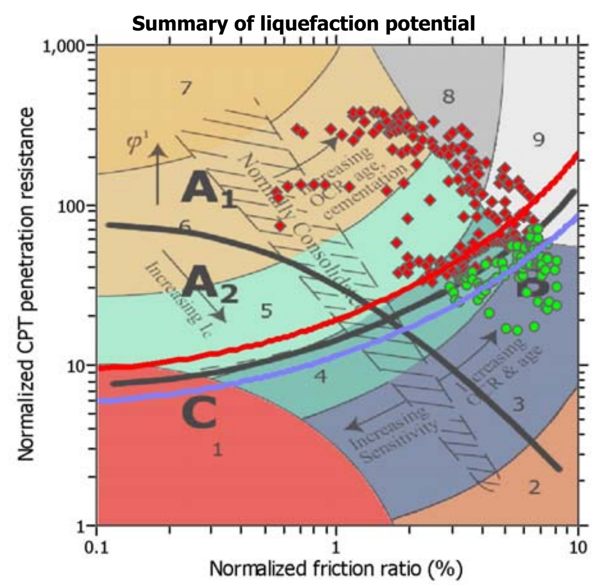
CPT file : 2-CP-CPT6

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	No		

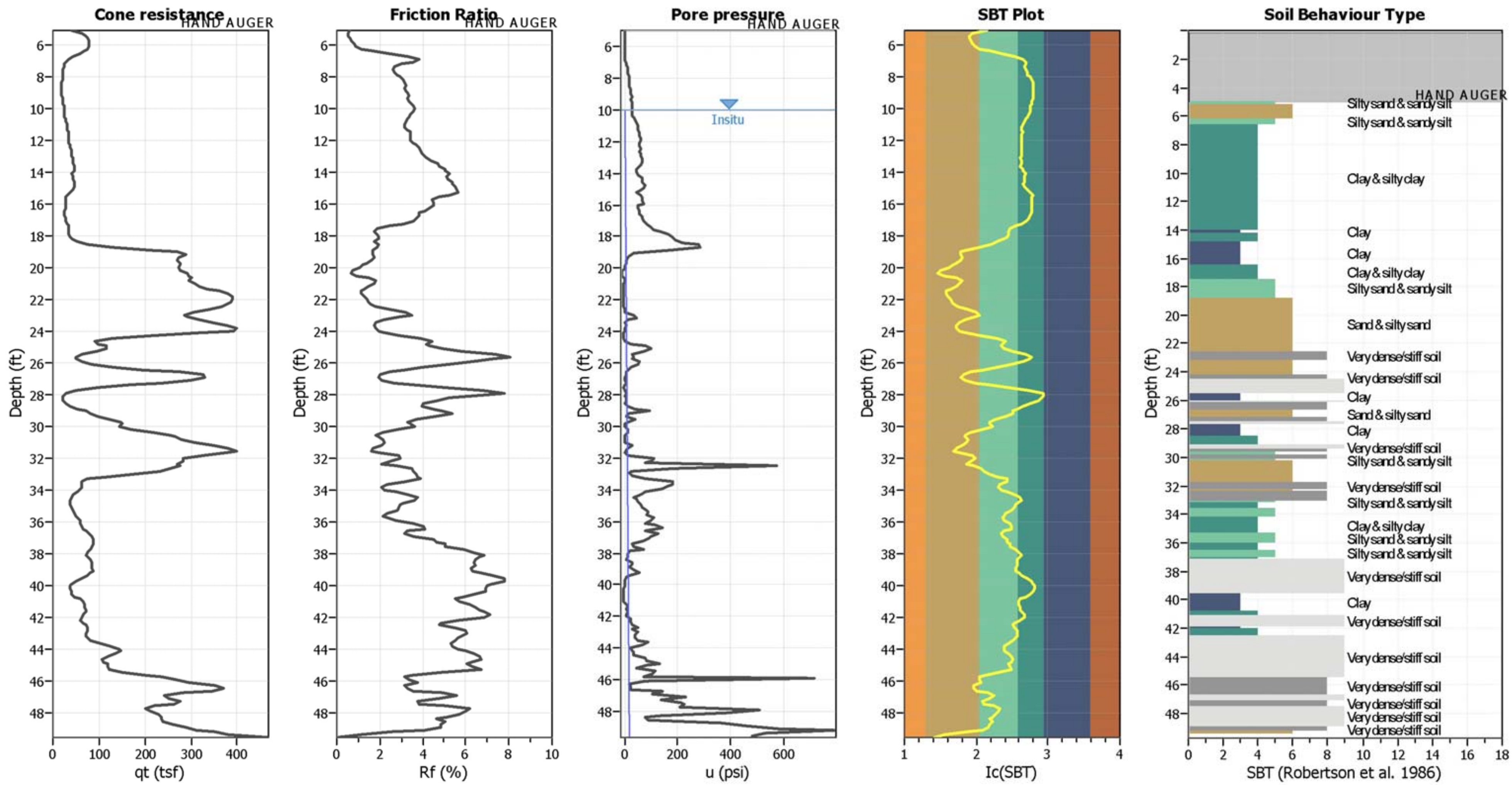






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



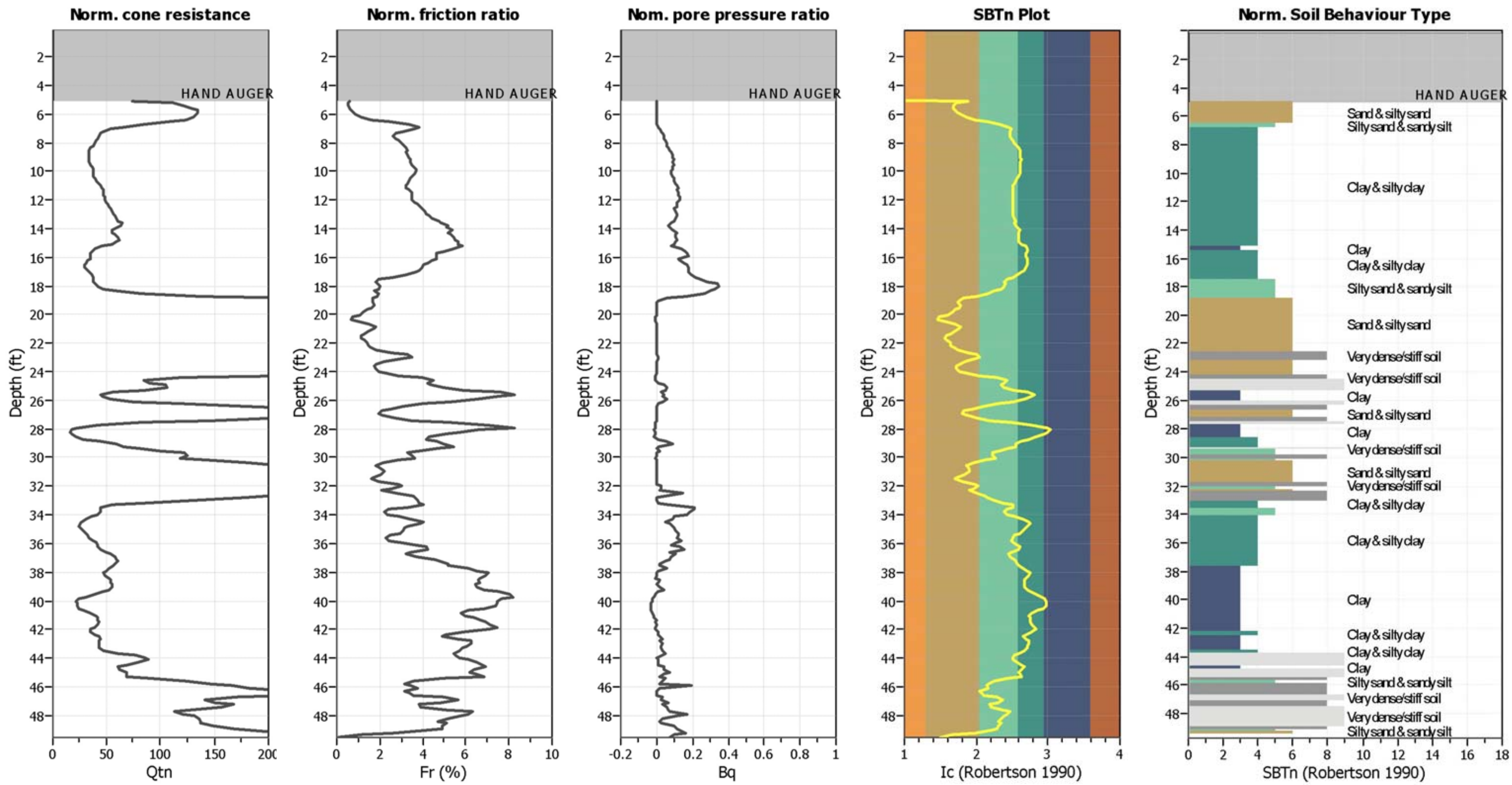
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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

CPT basic interpretation plots (normalized)



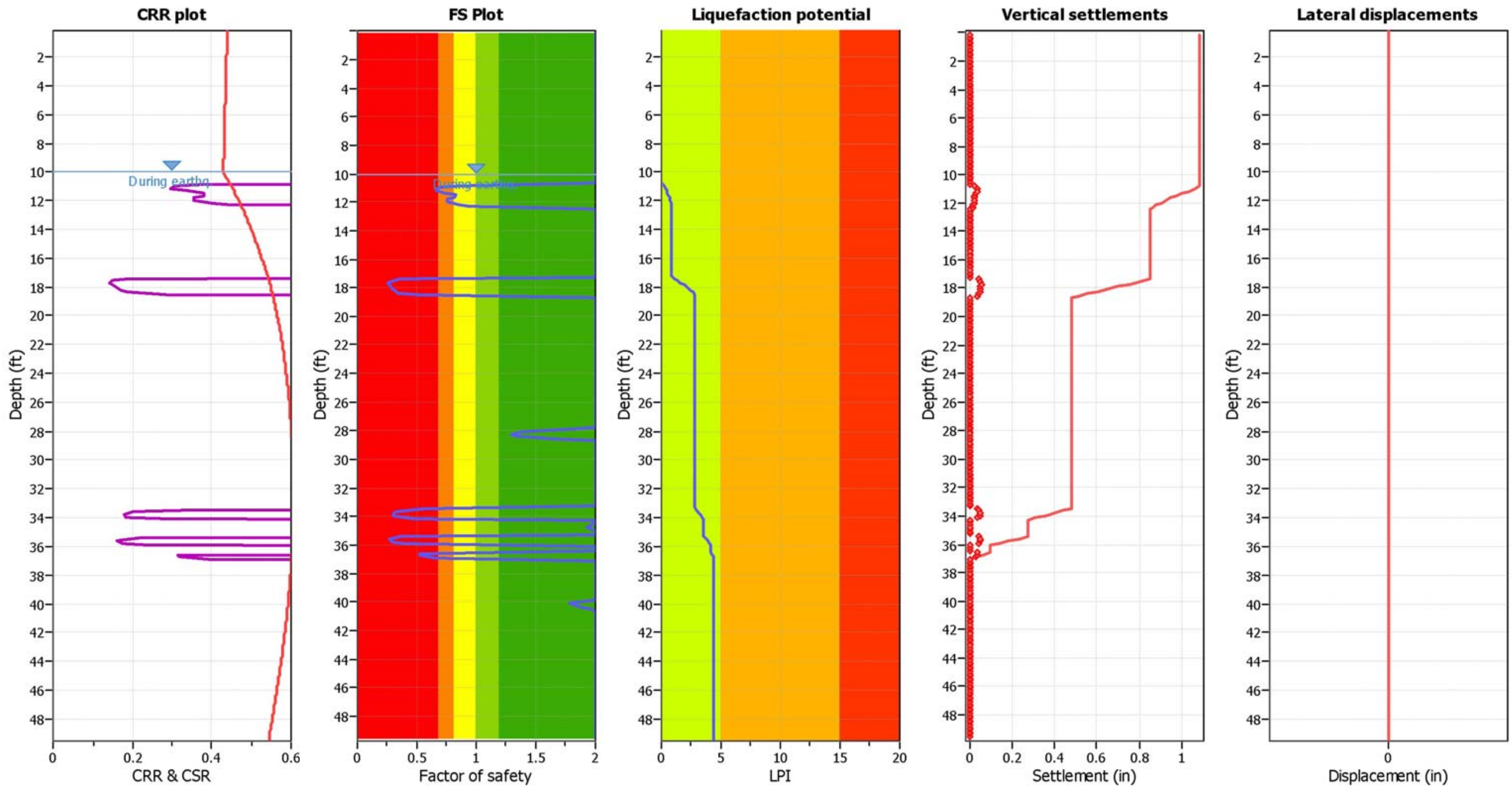
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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 liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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San Francisco, CA 94104

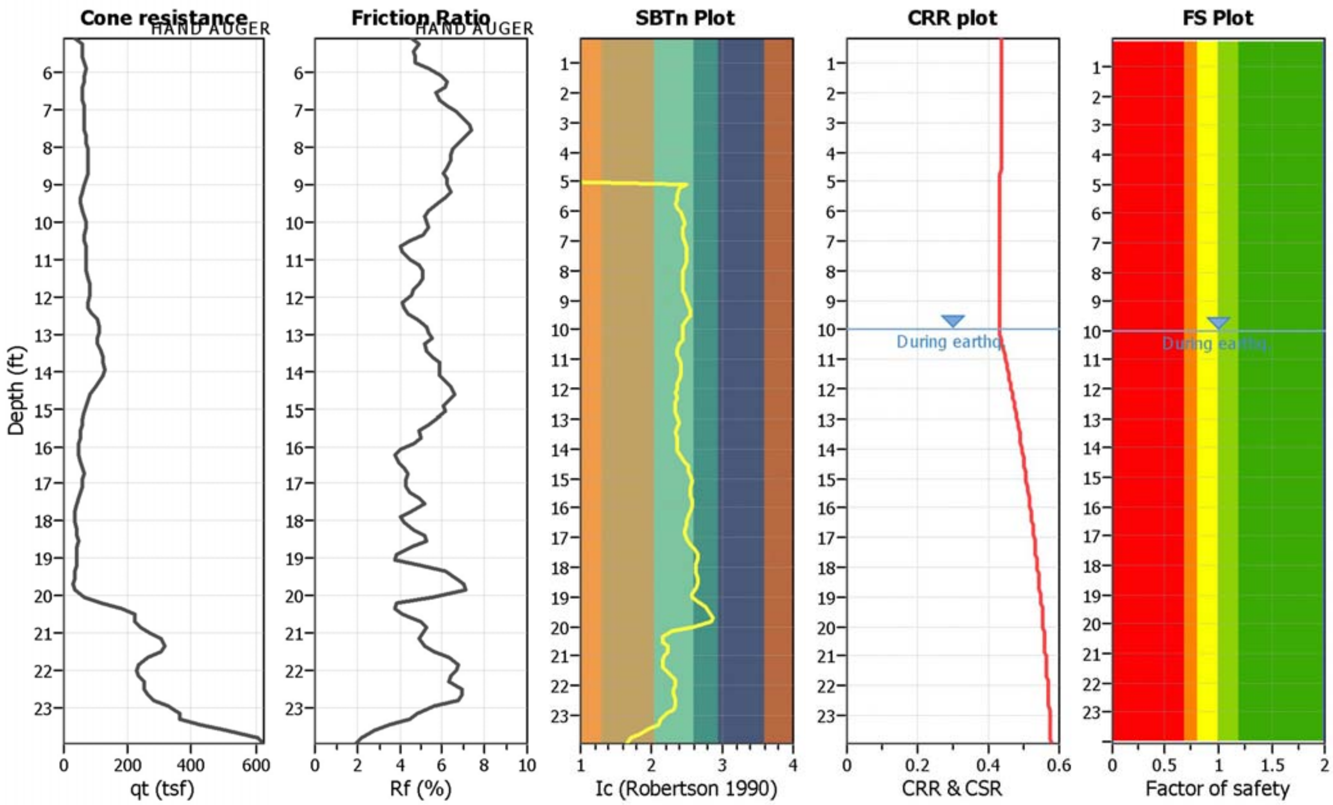
LIQUEFACTION ANALYSIS REPORT

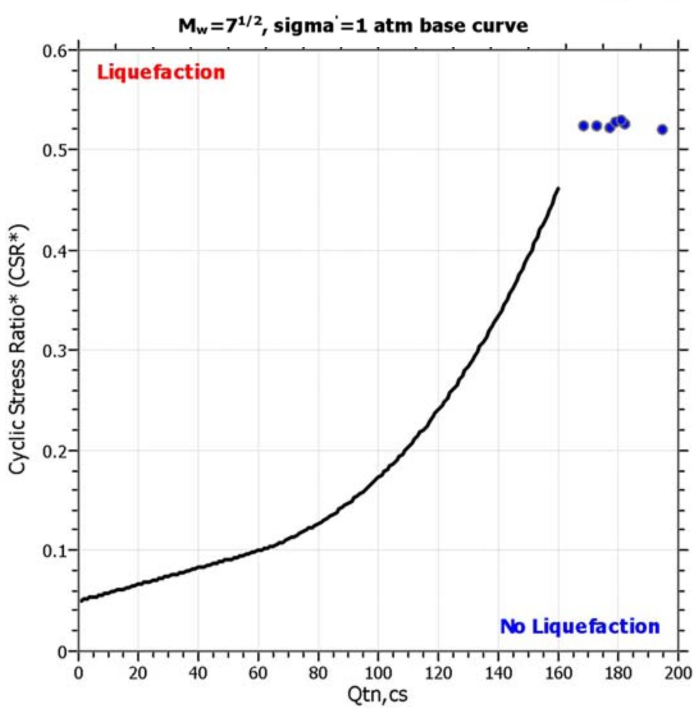
Project title : Alice Griffith Residential Development **Location : San Francisco, California**

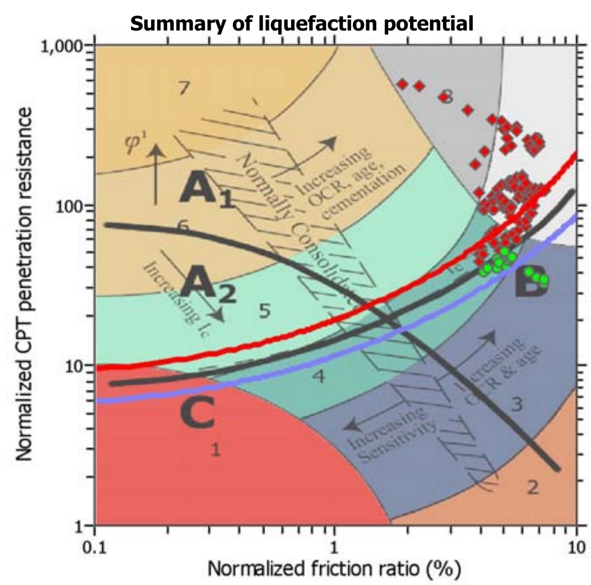
CPT file : 2-CP-CPT7

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	No		

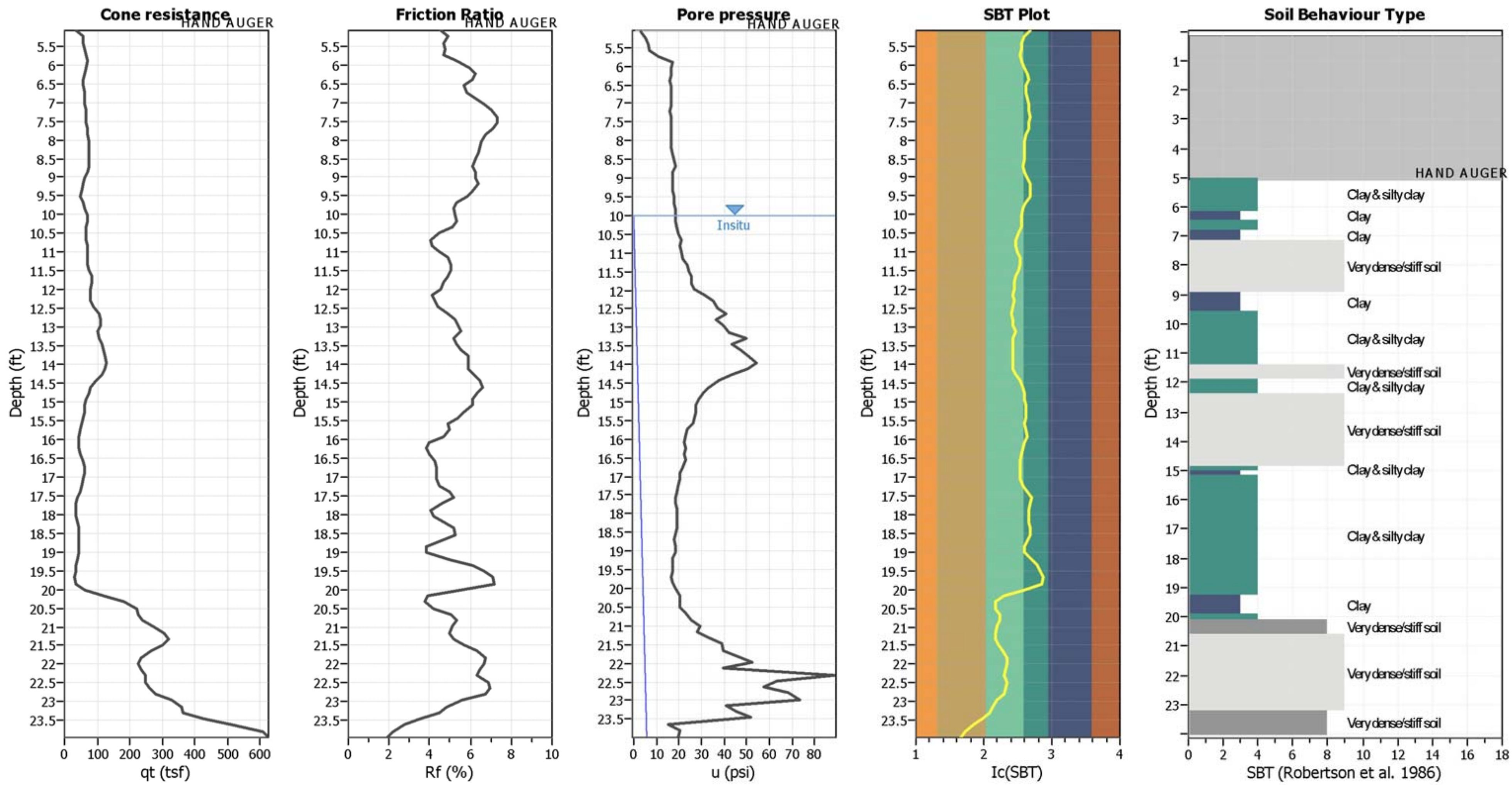






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



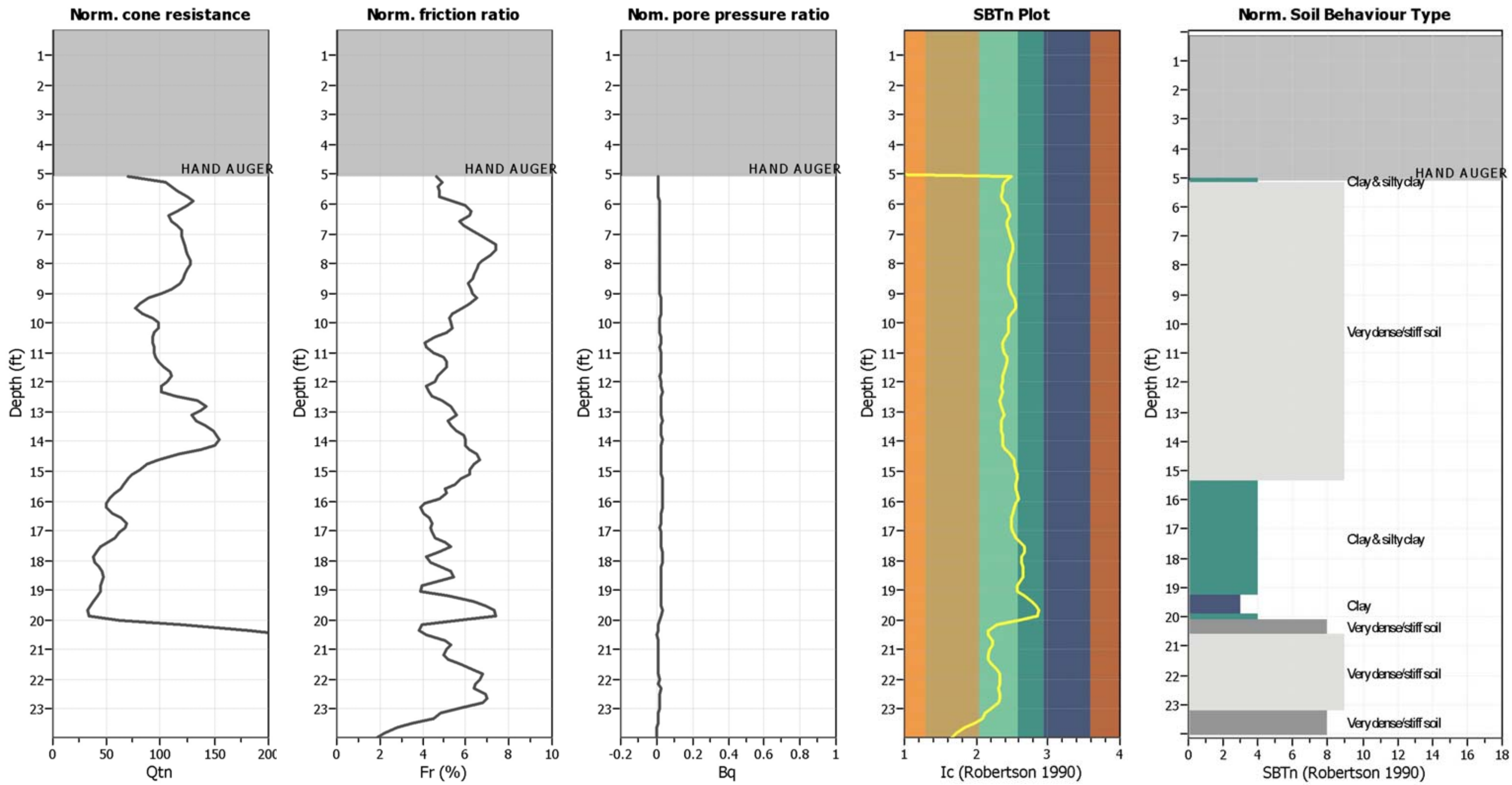
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

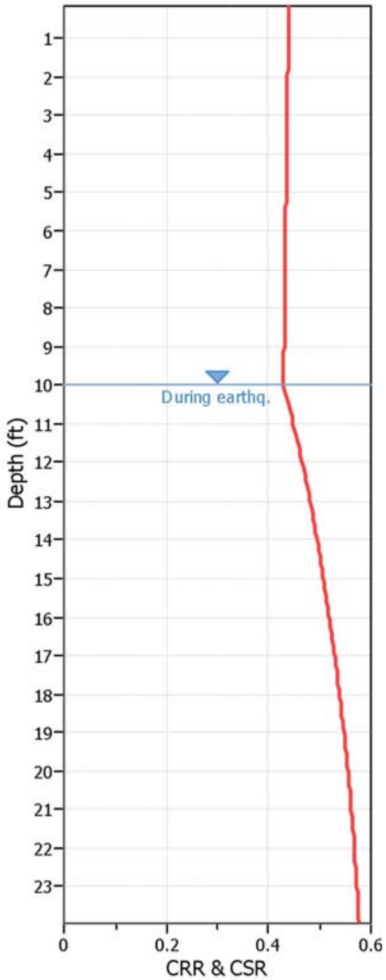
Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

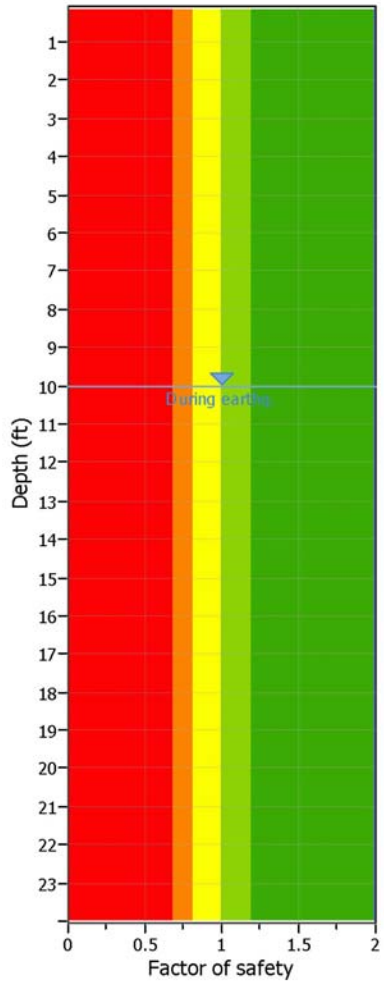
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots

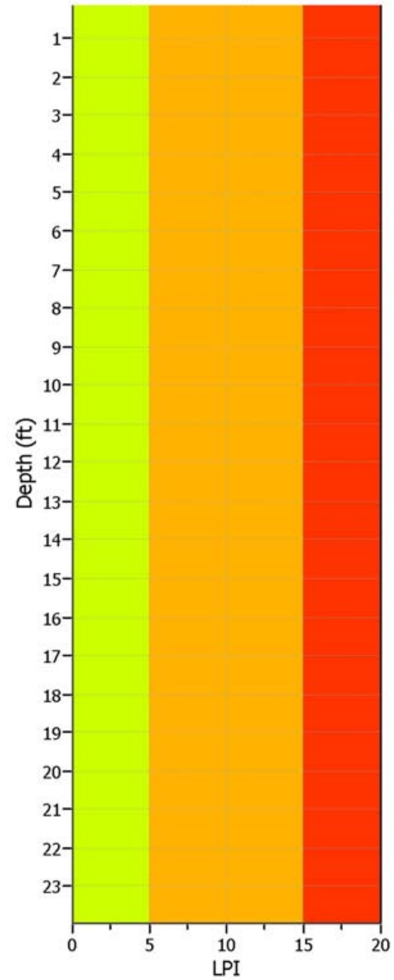
CRR plot



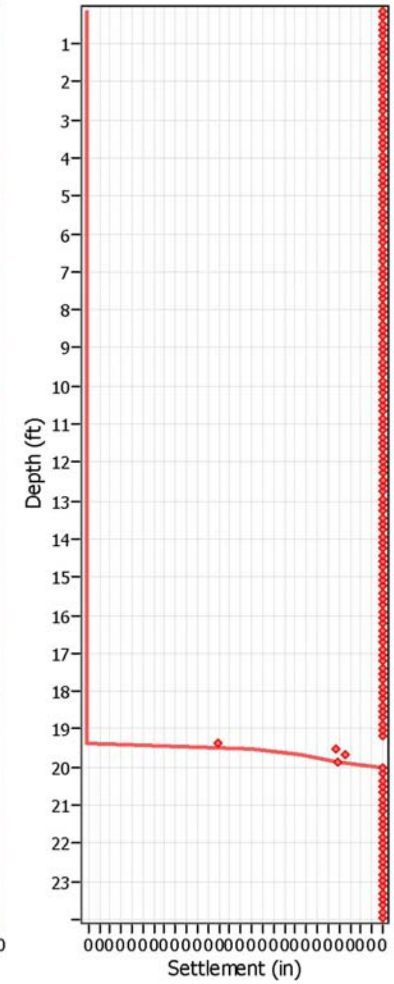
FS Plot



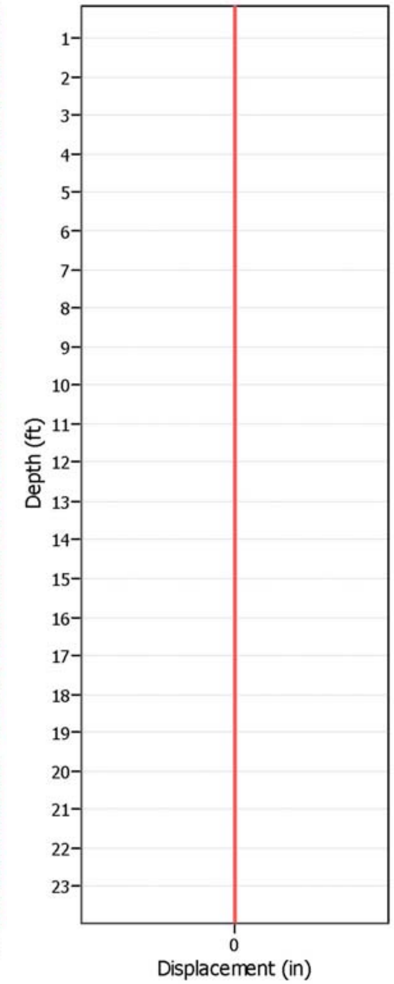
Liquefaction potential



Vertical settlements



Lateral displacements



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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 liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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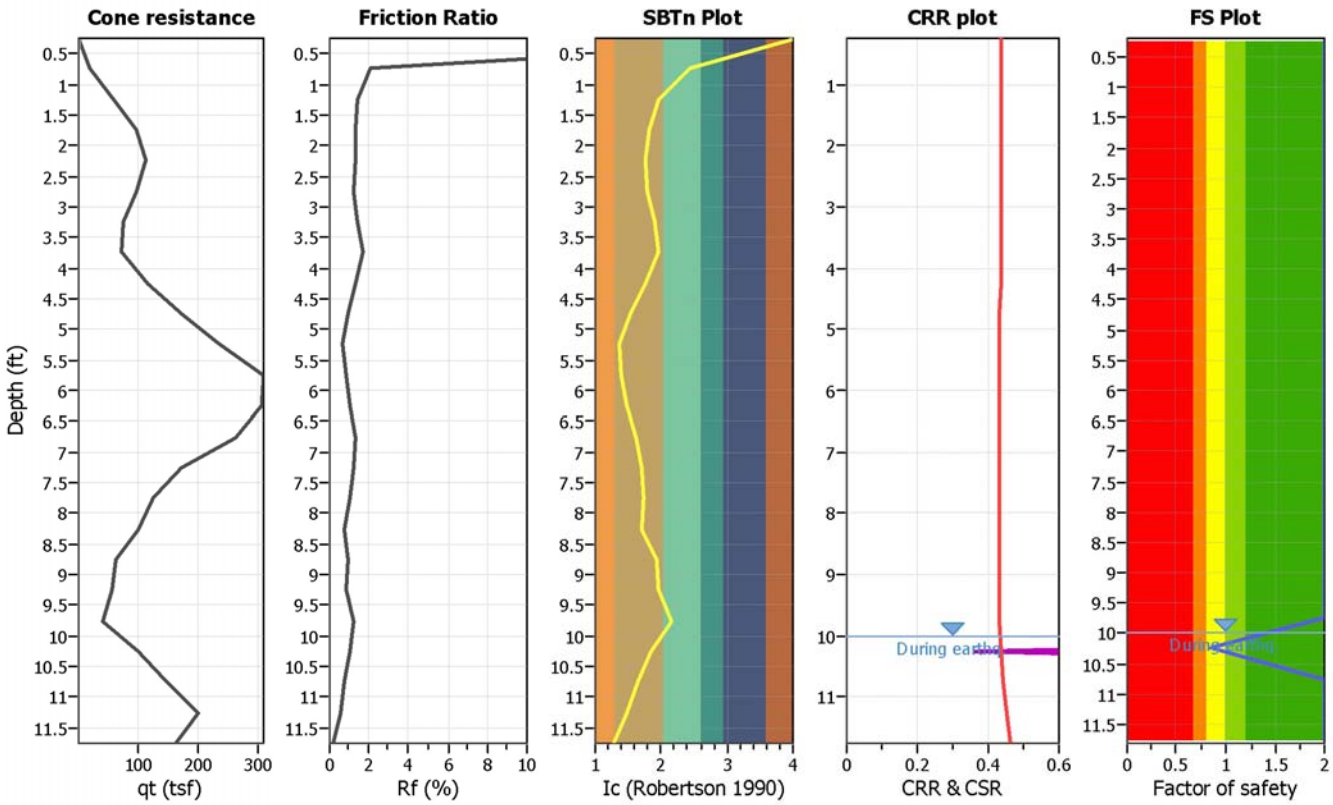
LIQUEFACTION ANALYSIS REPORT

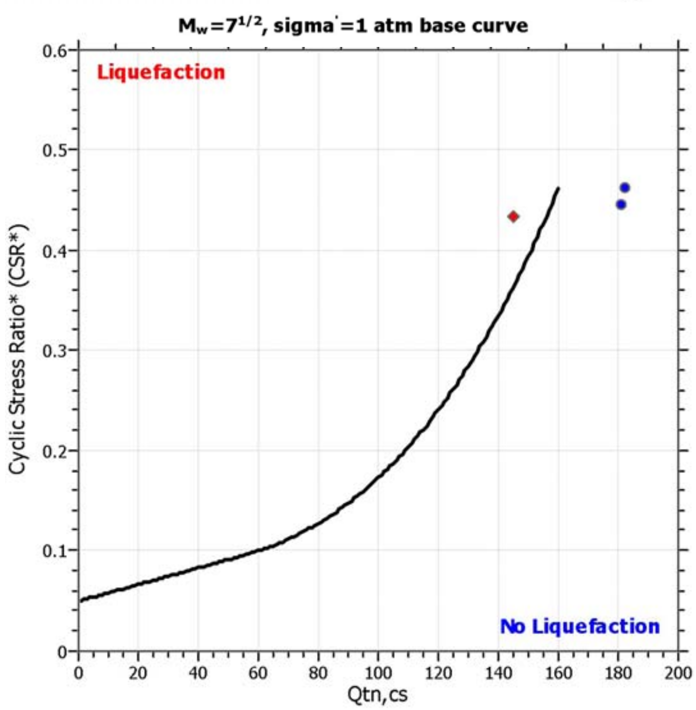
Project title : Alice Griffith Residential Development **Location : San Francisco, California**

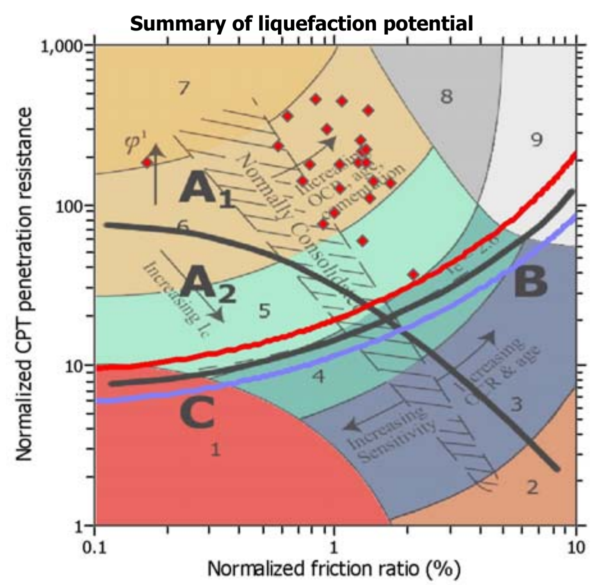
CPT file : CP-CPT6

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	No		

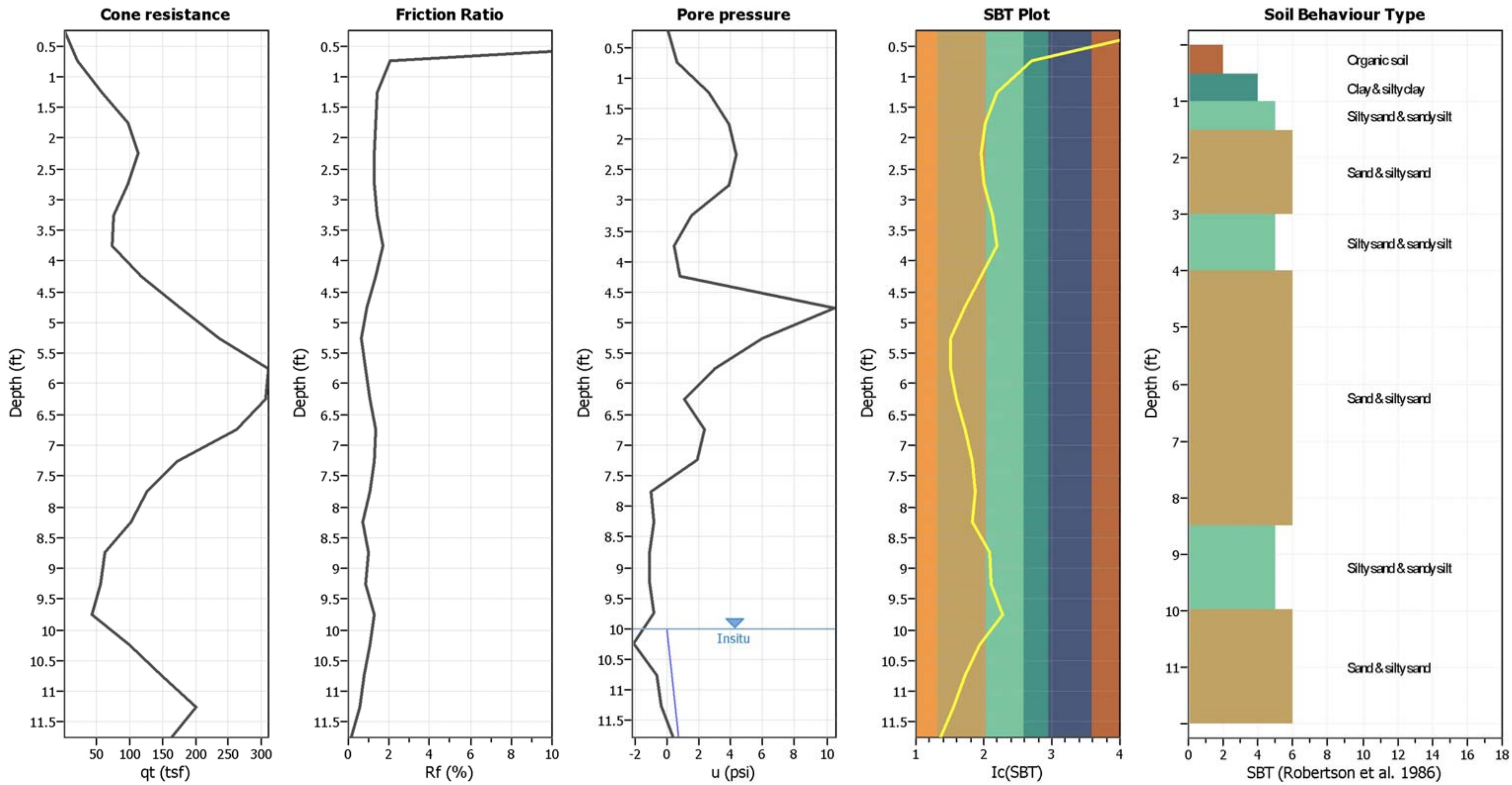






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



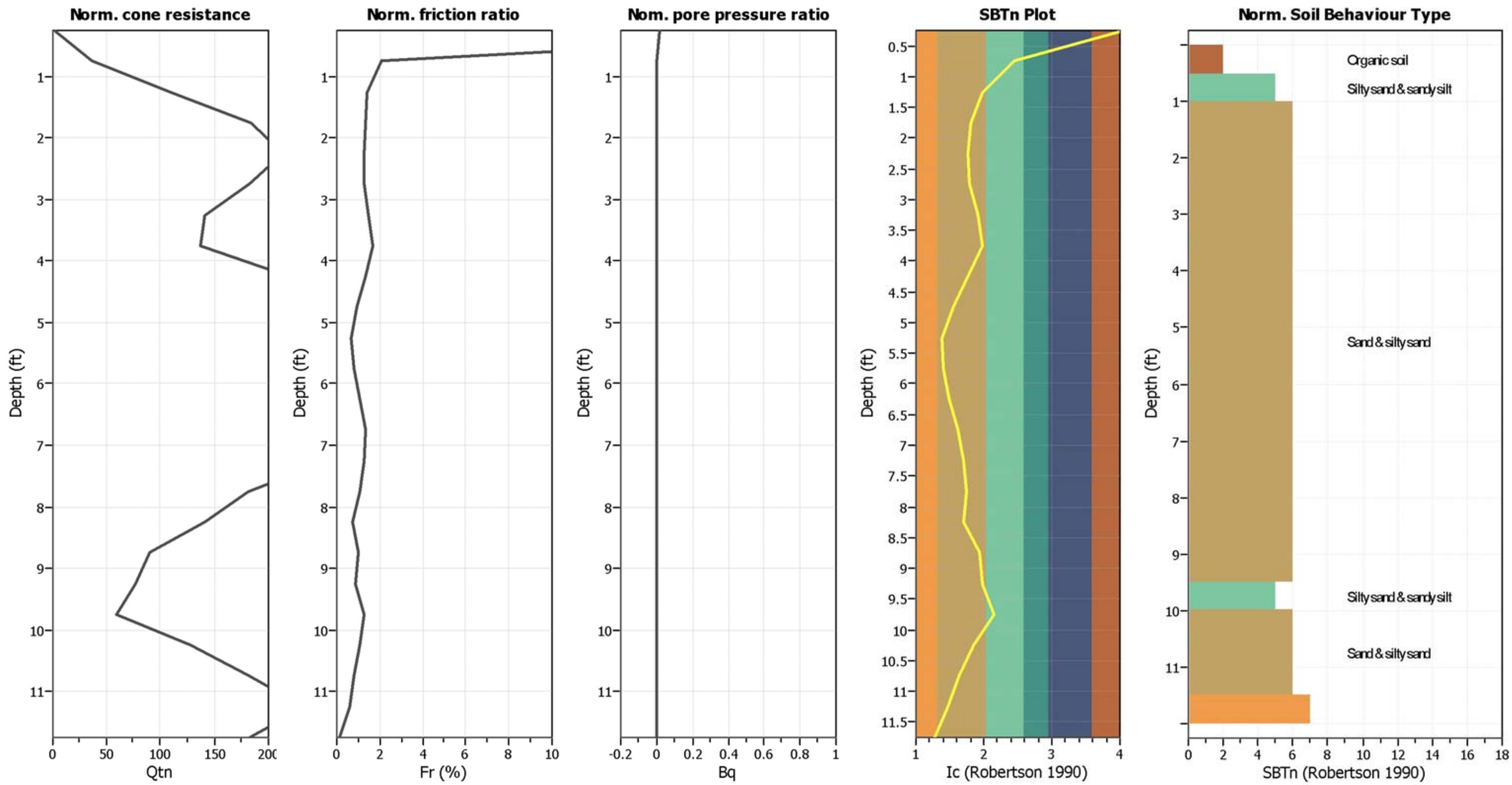
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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

CPT basic interpretation plots (normalized)



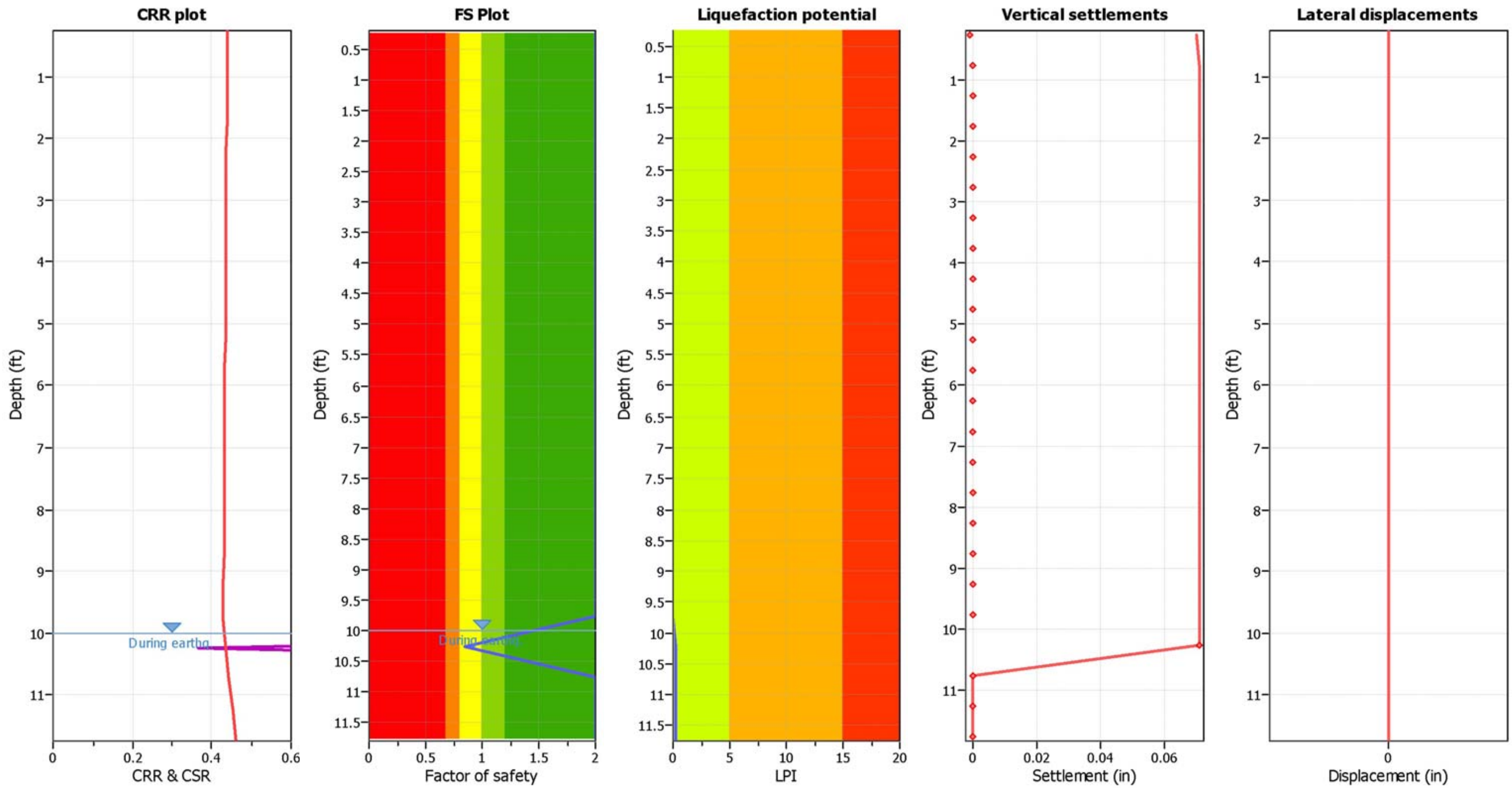
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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 liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

ENGEO
332 Pine Street, Suite 300
San Francisco, CA 94104

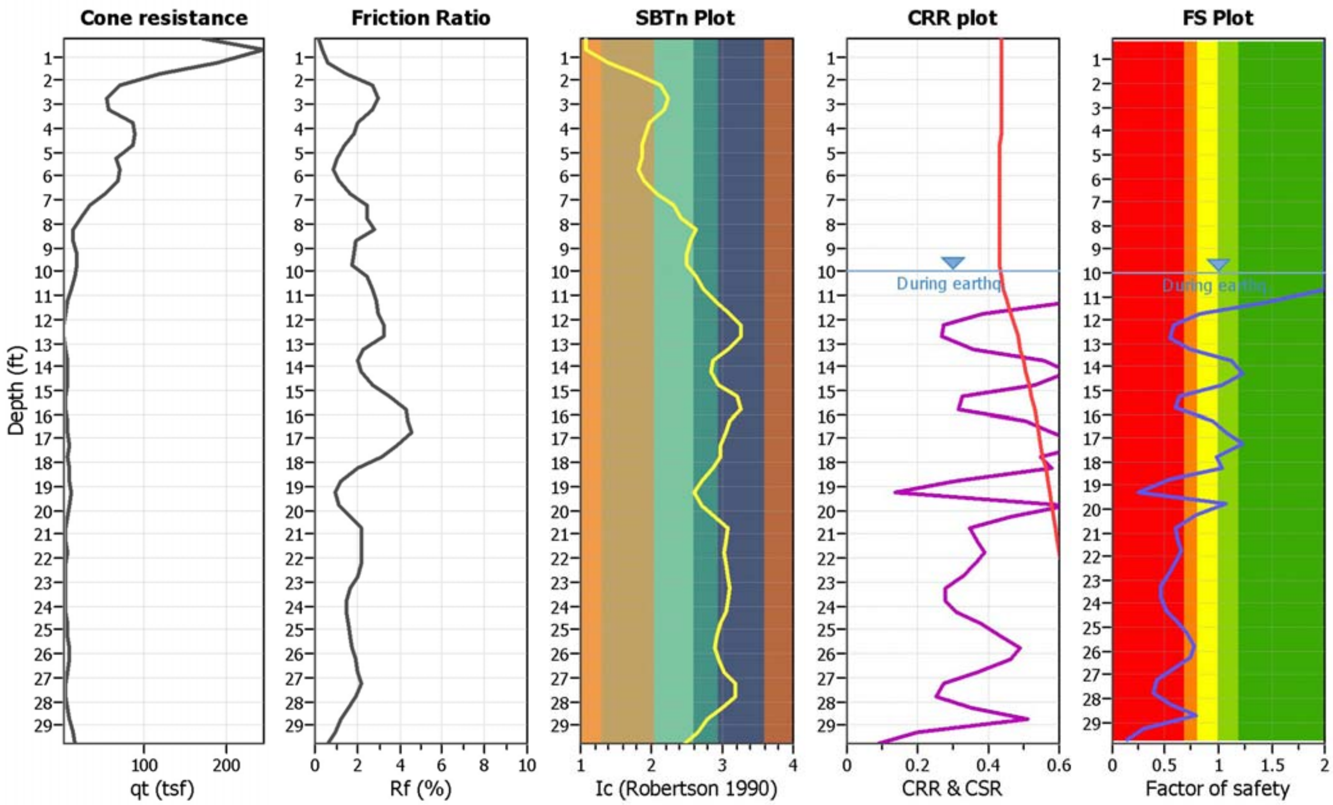
LIQUEFACTION ANALYSIS REPORT

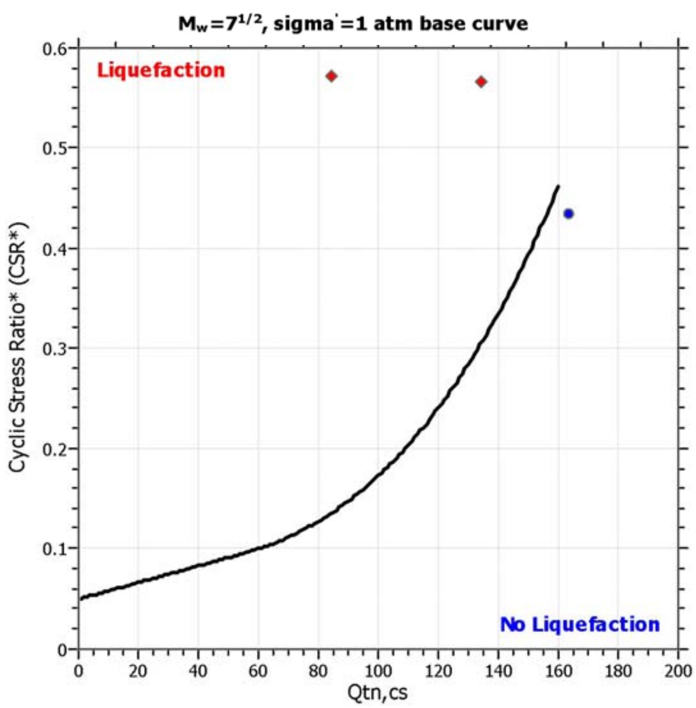
Project title : Alice Griffith Residential Development **Location : San Francisco, California**

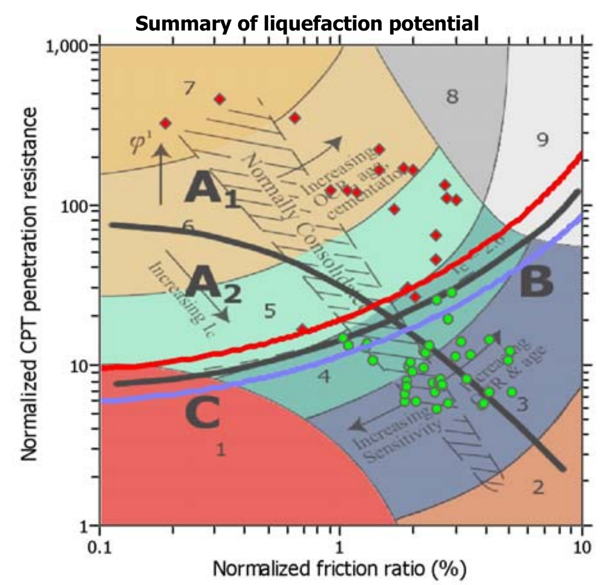
CPT file : CP-CPT9

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	No		

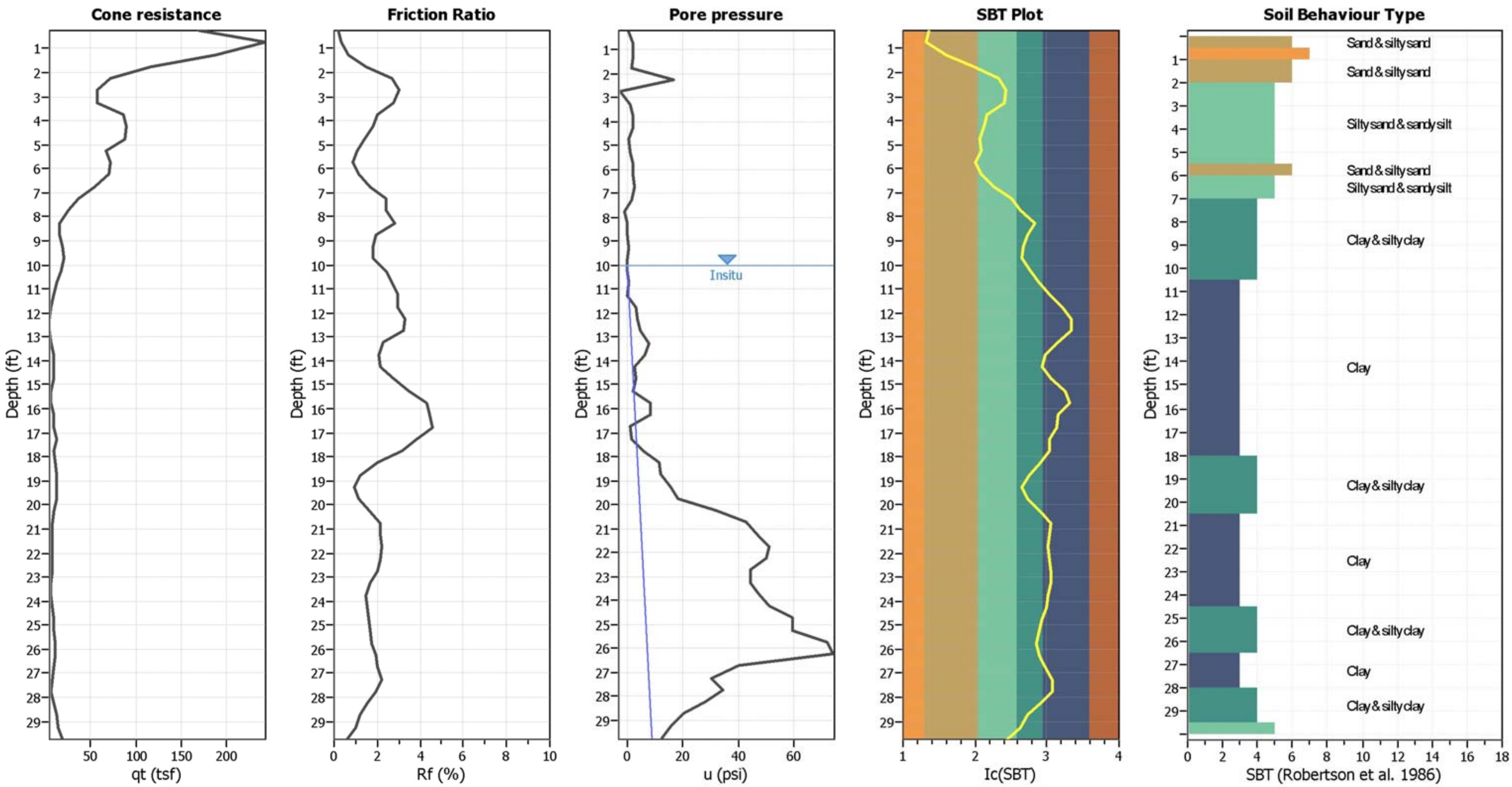






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



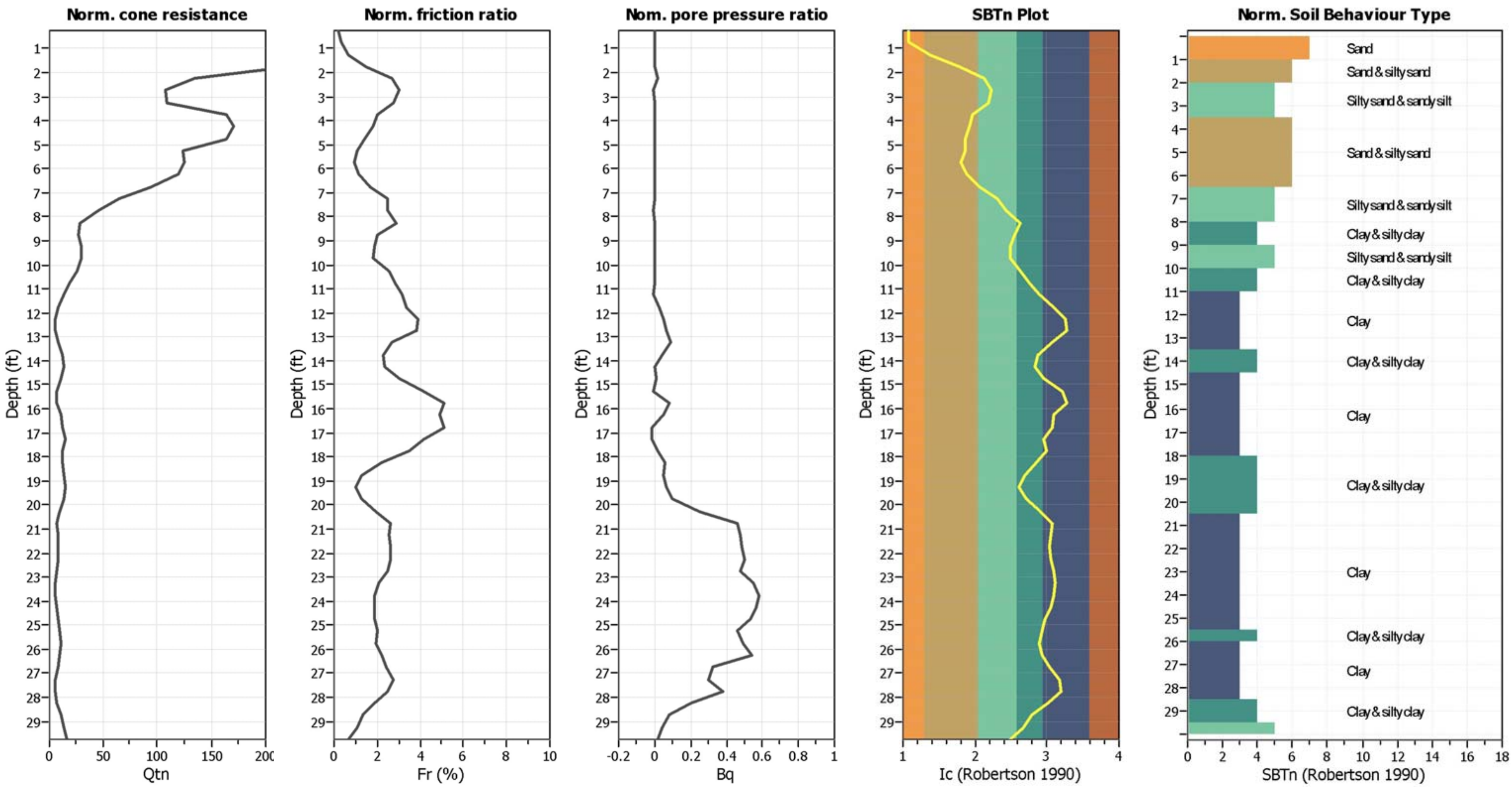
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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

CPT basic interpretation plots (normalized)



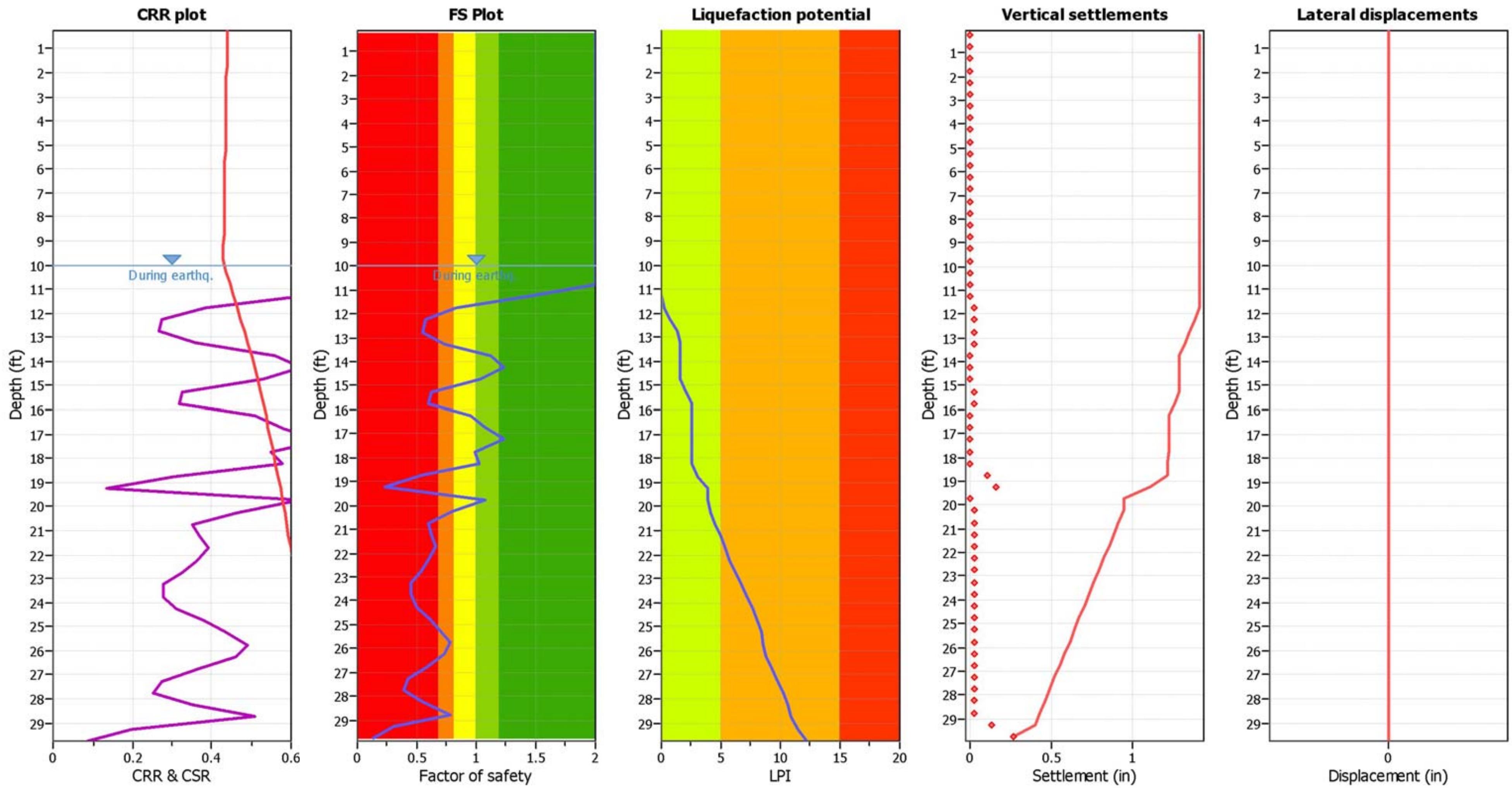
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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 liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

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San Francisco, CA 94104

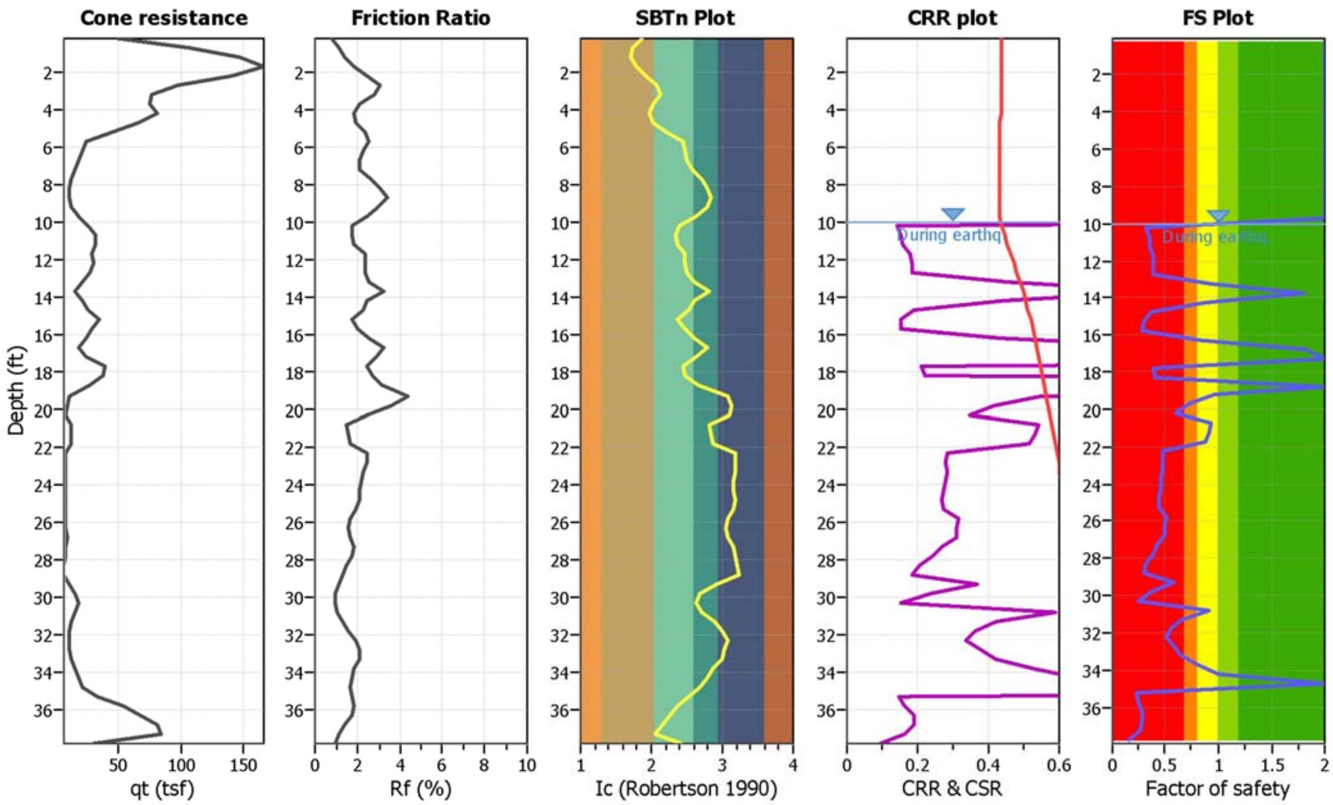
LIQUEFACTION ANALYSIS REPORT

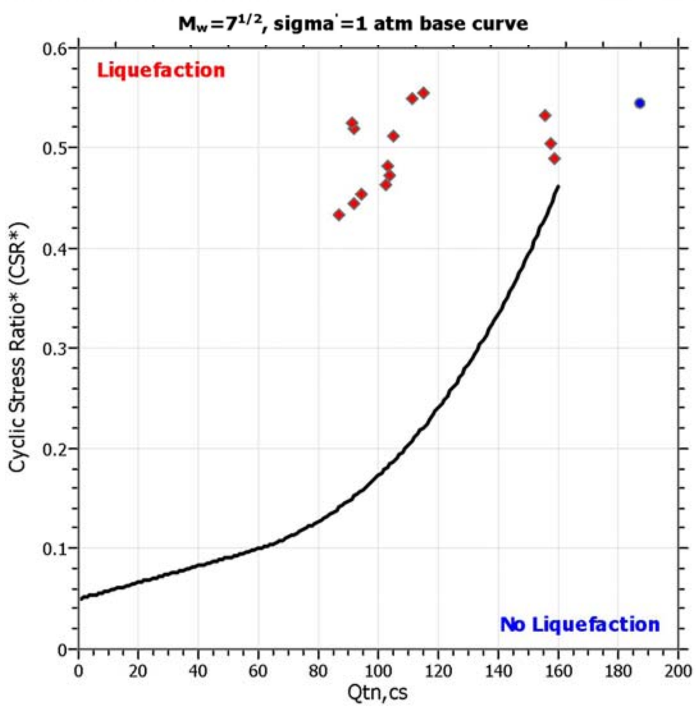
Project title : Alice Griffith Residential Development **Location : San Francisco, California**

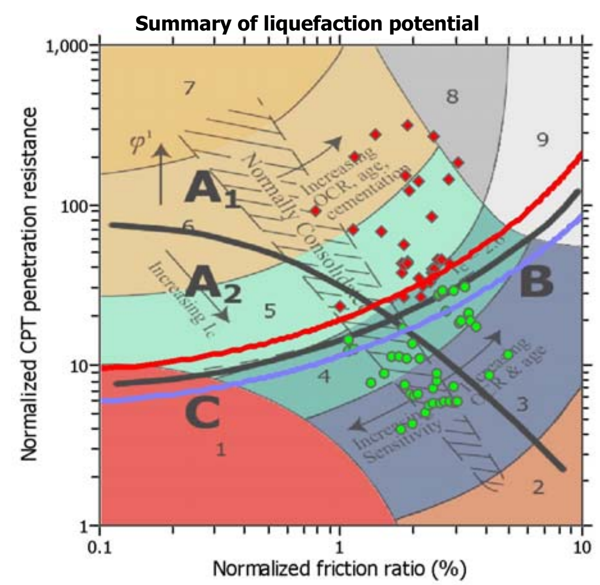
CPT file : CP-CPT11

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No		
Peak ground acceleration:	0.59	Unit weight calculation:	Based on SBT	K_σ applied:	No		

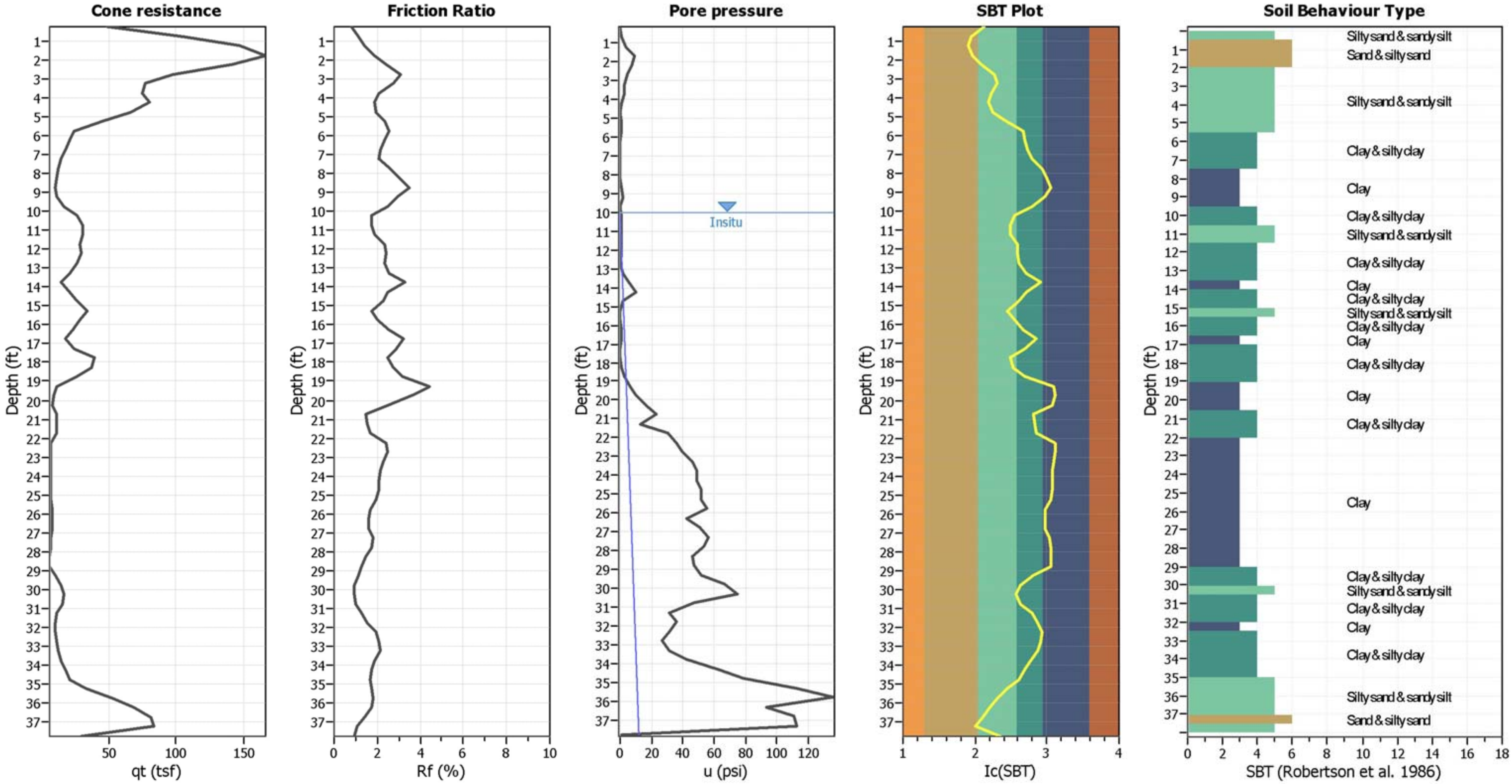






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



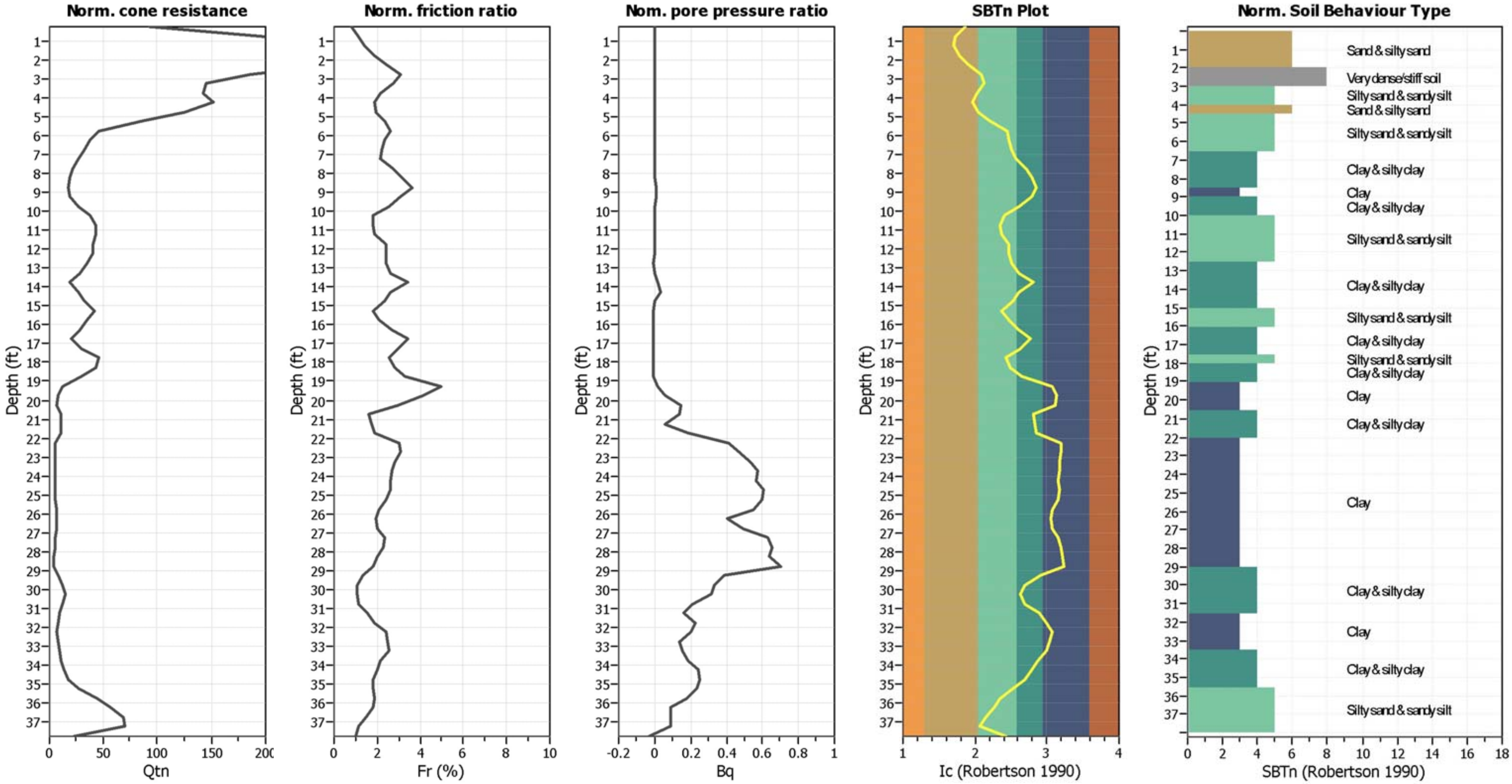
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method: Robertson (2009)
Fines correction method: Robertson (2009)
Points to test: Based on Ic value
Earthquake magnitude M_w: 7.90
Peak ground acceleration: 0.59
Depth to water table (insitu): 10.00 ft

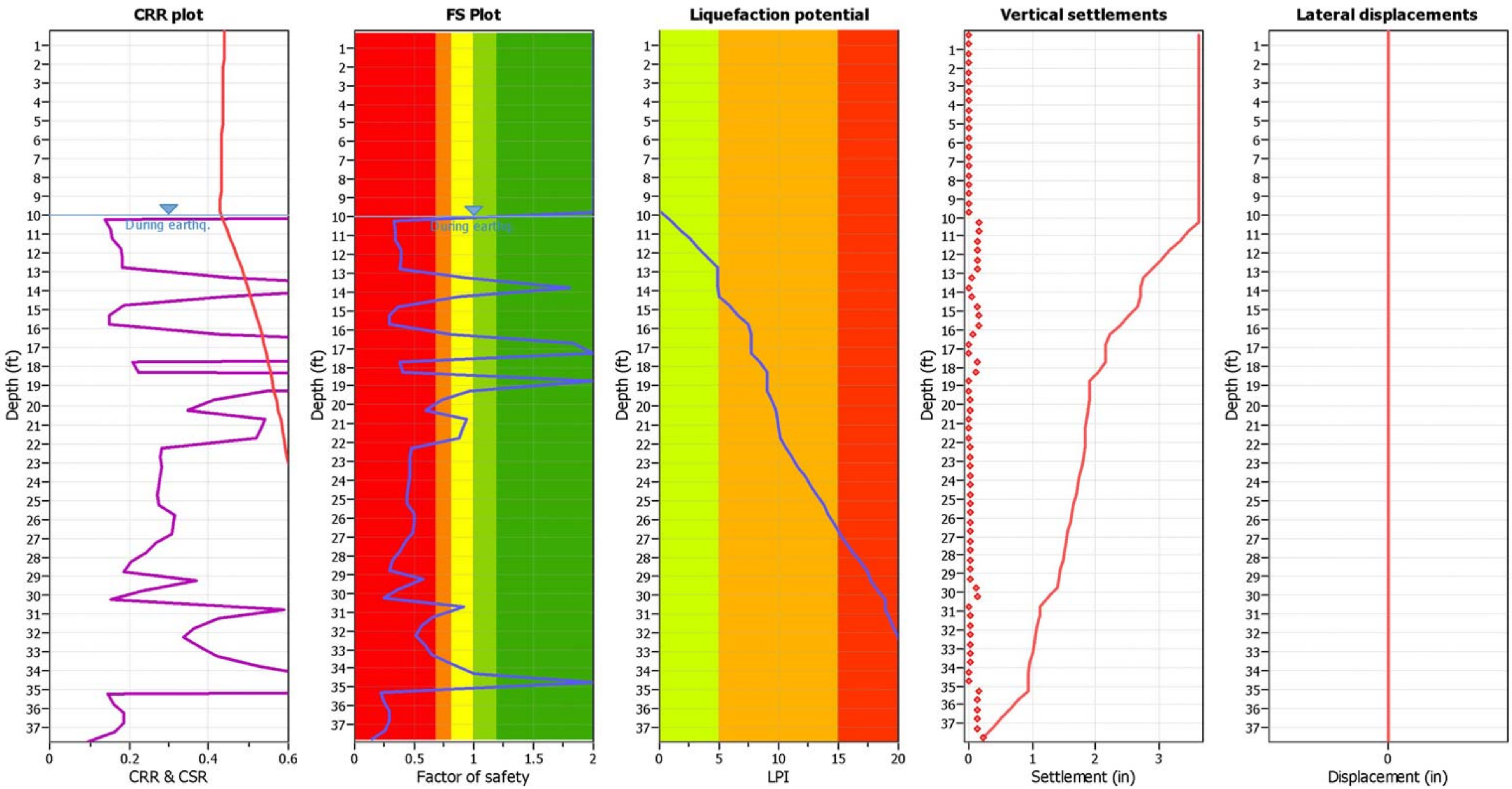
Depth to water table (erthq.): 10.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: No
K_σ applied: No
Clay like behavior applied: All soils
Limit depth applied: No
Limit depth: N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.59	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 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 liquefaction are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Candlestick Point Borings

Liquefaction Evaluation - Youd 2001, Seed 2003, I&B 2008 Methods -

Note, if sloping ground and non-zero statis shear stress exist, user may chose to change value of kalpha

Input

Yellow cells are calculated

Green cells require user input - reference respective papers for details
Corrdction factors on "Driving Force" and "Resisting Force" sheets require user input

Water Table depth at time of Exploration	Water Table depth at time of Liquefaction	amax/g	Mw	$V_{s^*40'}$
5	5	0.59	7.9	400

* V_{S40} = Avg shear wave velocity in upper 40 feet expressed in ft/s

					At time of Exploration		At time of Liquefaction	
Boring Designation	**Depth [ft]**	**Soil Type**	**N_m [Blows/ft]**	**FC**	**Total Stress [psf]**	**Effective Stress [psf]**	**Total Stress [psf]**	**Effective Stress [psf]**
CP-B5	12	SM	7	19	1440	1003.2	1440	1003.2
CP-B5	18	SM	9	8	1980	1168.8	1980	1168.8
CP-B7	14	GM	15	23	1080	518.4	1080	518.4
CP-B7	38	SM	8	19	4560	2500.8	4560	2500.8
2-CP-B3	1	ML	12	55	120	120	120	120
2-CP-B3	6	GP	11	15	720	657.6	720	657.6
2-CP-B3	8	GW	7	15	960	772.8	960	772.8
2-CP-B3	11	SP	3	0	1320	945.6	1320	945.6

N_m = Measured SPT Blow Count

YOUD 2001 Methodology Results

Boring Designation	Depth	CRR	CSR	FS
CP-B5	12	0.13	0.54	0.24
CP-B5	18	0.12	0.62	0.19
CP-B7	14	TDL	0.78	TDL
CP-B7	38	0.12	0.60	0.19
2-CP-B3	1	0.30	0.38	0.78
2-CP-B3	6	0.20	0.41	0.48
2-CP-B3	8	0.13	0.47	0.27
2-CP-B3	11	0.06	0.52	0.11
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!

TDL = Too Dense to Liquefy based on blowcount criteria

Candlestick Point Borings

SEED 2003 Methodology Results

Boring Designation	Depth	CRR	CSR			Calculated FS		
			mean rd	rd + sigma	rd - sigma	mean rd	rd + sigma	rd - sigma
CP-B5	12	0.08	0.37	0.40	0.34	0.23	0.21	0.24
CP-B5	18	0.09	0.40	0.45	0.35	0.23	0.20	0.26
CP-B7	14	0.32	0.46	0.50	0.42	0.70	0.65	0.77
CP-B7	38	0.06	0.37	0.50	0.24	0.16	0.12	0.24
2-CP-B3	1	0.39	0.18	0.18	0.18	2.20	2.18	2.21
2-CP-B3	6	0.16	0.29	0.30	0.28	0.54	0.52	0.56
2-CP-B3	8	0.09	0.32	0.34	0.31	0.29	0.28	0.31
2-CP-B3	11	0.05	0.35	0.38	0.33	0.14	0.13	0.15
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
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0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

THC = CRR capped at 4, in high seismicity cases, verify

Idriss & Boulanger 2008 Methodology Results

Boring Designation	Depth	CRR	CSR	FS
CP-B5	12	0.15	0.56	0.27
CP-B5	18	0.14	0.66	0.21
CP-B7	14	0.48	0.79	0.61
CP-B7	38	0.13	0.71	0.19
2-CP-B3	1	0.27	0.39	0.69
2-CP-B3	6	0.21	0.42	0.50
2-CP-B3	8	0.15	0.48	0.31
2-CP-B3	11	0.08	0.55	0.15
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!

THC = CRR capped at 4, in high seismicity cases, verify

Liquefaction Evaluation - Driving Force

Boring No.	0

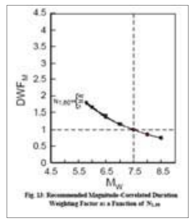
Youd 2001

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	rd	CSR
CP-B5	12	1440	1003	0.975	0.54
CP-B5	18	1980	1169	0.961	0.62
CP-B7	14	1080	518	0.970	0.78
CP-B7	38	4560	2501	0.864	0.60
2-CP-B3	1	120	120	1.000	0.38
2-CP-B3	6	720	658	0.988	0.41
2-CP-B3	8	960	773	0.983	0.47
2-CP-B3	11	1320	946	0.977	0.52
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!
0	0	0	0	1.000	#DIV/0!

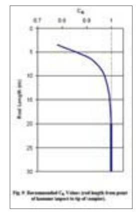
SEED 2003

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	CN	(N1)60	Ce	Cb	Cr	Cs
CP-B5	12	1440	1003	1.41	10	1.16	1	0.85	1
CP-B5	18	1980	1169	1.31	12	1.16	1	0.9	1
CP-B7	14	1080	518	1.60	24	1.16	1	0.85	1
CP-B7	38	4560	2501	0.89	8	1.16	1	0.98	1
2-CP-B3	1	120	120	1.60	17	1.16	1	0.8	1
2-CP-B3	6	720	658	1.60	16	1.16	1	0.8	1
2-CP-B3	8	960	773	1.60	11	1.16	1	0.8	1
2-CP-B3	11	1320	946	1.45	4	1.16	1	0.8	1
0	0	0	0	#DIV/0!	#DIV/0!	1.16	1		1
0	0	0	0	#DIV/0!	#DIV/0!	1.16	1		1
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				
0	0	0	0	#DIV/0!	#DIV/0!				

DWF
0.93



Rod Length	CR
9	0.8
12	0.85
15	0.87
18	0.9
21	0.92
24	0.94
27	0.96
30	0.97
33	0.975
36	0.98
39	0.985
45	0.99
60	1
100	1



Boring Designation	Depth	Cfines	(N1)60cs	rd	sigma	rd +sigma	rd -sigma	f	K sigma	K alpha ~
CP-B5	12	1.17	11	0.811	0.060	0.870	0.751	0.66	1.29	1.00
CP-B5	18	1.06	13	0.698	0.084	0.782	0.614	0.67	1.22	1.00
CP-B7	14	1.14	27	0.773	0.068	0.841	0.705	0.73	1.45	1.00
CP-B7	38	1.19	10	0.462	0.159	0.620	0.303	0.65	0.94	1.00
2-CP-B3	1	1.24	22	0.988	0.007	0.995	0.981	0.71	2.31	1.00
2-CP-B3	6	1.11	18	0.917	0.033	0.950	0.884	0.69	1.44	1.00
2-CP-B3	8	1.13	12	0.883	0.042	0.925	0.841	0.66	1.41	1.00
2-CP-B3	11	1.00	4	0.829	0.055	0.885	0.774	0.62	1.36	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00
0.00	0	1.00	#DIV/0!	1.000	0.000	1.000	1.000	#DIV/0!	#DIV/0!	1.00

Boring Designation	mean value of rd				rd + sigma				rd - sigma			
	CSReq	CSRN	CSR*	CSR*α>0	CSReq	CSRN	CSR*	CSR*α>0	CSReq	CSRN	CSR*	CSR*α>0
CP-B5	0.45	0.48	0.37	0.37	0.48	0.52	0.40	0.40	0.41	0.45	0.34	0.34
CP-B5	0.45	0.49	0.40	0.40	0.51	0.55	0.45	0.45	0.40	0.43	0.35	0.35
CP-B7	0.62	0.66	0.46	0.46	0.67	0.72	0.50	0.50	0.56	0.61	0.42	0.42
CP-B7	0.32	0.35	0.37	0.37	0.43	0.47	0.50	0.50	0.21	0.23	0.24	0.24
2-CP-B3	0.38	0.41	0.18	0.18	0.38	0.41	0.18	0.18	0.38	0.41	0.18	0.18
2-CP-B3	0.38	0.41	0.29	0.29	0.40	0.43	0.30	0.30	0.37	0.40	0.28	0.28
2-CP-B3	0.42	0.45	0.32	0.32	0.44	0.47	0.34	0.34	0.40	0.43	0.31	0.31
2-CP-B3	0.44	0.48	0.35	0.35	0.47	0.51	0.38	0.38	0.41	0.45	0.33	0.33
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0.00	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

~ K alpha = 1.0 for level ground conditions only (no static shear stress)

I&B 2008

MSF
0.90

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	rd	CSR	Csigma	K sigma	K alpha ~	CSR7.5
CP-B5	12	1440	1003	0.983	0.54	0.092	1.07	1.00	0.56
CP-B5	18	1980	1169	0.968	0.63	0.101	1.06	1.00	0.66
CP-B7	14	1080	518	0.978	0.78	0.163	1.10	1.00	0.79
CP-B7	38	4560	2501	0.905	0.63	0.086	0.99	1.00	0.71
2-CP-B3	1	120	120	1.004	0.39	0.126	1.10	1.00	0.39
2-CP-B3	6	720	658	0.996	0.42	0.120	1.10	1.00	0.42
2-CP-B3	8	960	773	0.992	0.47	0.096	1.10	1.00	0.48
2-CP-B3	11	1320	946	0.985	0.53	0.074	1.06	1.00	0.55
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!
0	0	0	0	1.006	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!

~ K alpha = 1.0 for level ground conditions only (no static shear stress)

Liquefaction Evaluation - Resisting Force

Boring No.	0

YOUD 2001

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	CN	Ce	Cb	Cr	Cs	(N1)60
CP-B5	12	1440	1003	1.41	1.16	1	0.85	1	10
CP-B5	18	1980	1169	1.31	1.16	1	0.9	1	12
CP-B7	14	1080	518	1.70	1.16	1	0.85	1	25
CP-B7	38	4560	2501	0.89	1.16	1	0.98	1	8
2-CP-B3	1	120	120	1.70	1.16	1	0.8	1	18
2-CP-B3	6	720	658	1.70	1.16	1	0.8	1	17
2-CP-B3	8	960	773	1.61	1.16	1	0.8	1	11
2-CP-B3	11	1320	946	1.45	1.16	1	0.8	1	4
0	0	0	0	#DIV/0!	1.16	1	0	1	#DIV/0!
0	0	0	0	#DIV/0!	1.16	1	0	1	#DIV/0!
0	0	0	0	#DIV/0!					#DIV/0!
0	0	0	0	#DIV/0!					#DIV/0!
0	0	0	0	#DIV/0!					#DIV/0!

MSF
0.87513418

Boring Designation	Depth	alpha	beta	(N1)60cs	f	K sigma	K alpha ~	CRR-B	CRR-M
CP-B5	12	3.43	1.07	14	0.67	1.28	1.00	0.15	0.13
CP-B5	18	0.30	1.01	13	0.66	1.22	1.00	0.14	0.12
CP-B7	14	4.06	1.10	32	0.76	1.41	1.00	0.65	TDL
CP-B7	38	3.43	1.07	12	0.66	0.94	1.00	0.13	0.12
2-CP-B3	1	5.00	1.20	27	0.74	2.13	1.00	0.34	0.30
2-CP-B3	6	2.50	1.05	21	0.70	1.41	1.00	0.23	0.20
2-CP-B3	8	2.50	1.05	14	0.67	1.40	1.00	0.15	0.13
2-CP-B3	11	0.00	1.00	4	0.62	1.36	1.00	0.07	0.06
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!	#DIV/0!
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!	1.00	#DIV/0!	#DIV/0!
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!		#DIV/0!	#DIV/0!
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!		#DIV/0!	#DIV/0!
0.00	0	0.00	1.00	#DIV/0!	#DIV/0!	#DIV/0!		#DIV/0!	#DIV/0!

~ K alpha = 1.0 for level ground conditions only (no static shear stress)

SEED 2003

Boring Designation	Depth	CRR
CP-B5	12	0.08
CP-B5	18	0.09
CP-B7	14	0.32
CP-B7	38	0.06
2-CP-B3	1	0.39
2-CP-B3	6	0.16
2-CP-B3	8	0.09
2-CP-B3	11	0.05
0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!
0	0	#DIV/0!

I&B 2008

Boring Designation	Depth	Total Stress [psf]	Effective Stress [psf]	(N)60	Ce	Cb	Cr	Cs
CP-B5	12	1440	1003	7	1.16	1	0.85	1
CP-B5	18	1980	1169	9	1.16	1	0.9	1
CP-B7	14	1080	518	15	1.16	1	0.85	1
CP-B7	38	4560	2501	9	1.16	1	0.98	1
2-CP-B3	1	120	120	11	1.16	1	0.8	1
2-CP-B3	6	720	658	10	1.16	1	0.8	1
2-CP-B3	8	960	773	7	1.16	1	0.8	1
2-CP-B3	11	1320	946	3	1.16	1	0.8	1
0	0	0	0	0	1.16	1	0	1
0	0	0	0	0	1.16	1	0	1
0	0	0	0	0				
0	0	0	0	0				
0	0	0	0	0				

Iterations of CN Value

Boring Designation	Depth	CN i	N1-1	Cn1	N1-2	Cn2	N1-3	Cn3	N14	CN	(N1)60
CP-B5	12	1.49	10.3142319	1.45	9.99972565	1.45	10.0259054	1.45	10.023708	1.45	10
CP-B5	18	1.34	12.6160229	1.32	12.3652802	1.32	12.3833896	1.32	12.3820747	1.32	12
CP-B7	14	1.70	25.05919	1.70	25.05919	1.70	25.05919	1.70	25.05919	1.70	25
CP-B7	38	0.88	8.03830915	0.88	8.01344694	0.88	8.0128435	0.88	8.01282884	0.88	8
2-CP-B3	1	1.70	18.45792	1.70	18.45792	1.70	18.45792	1.70	18.45792	1.70	18
2-CP-B3	6	1.70	17.43248	1.67	17.1686635	1.68	17.2152388	1.68	17.2069811	1.68	17
2-CP-B3	8	1.70	11.27984	1.65	10.9420513	1.66	10.9826154	1.65	10.9777034	1.65	11
2-CP-B3	11	1.63	4.55023891	1.59	4.43023088	1.59	4.43745403	1.59	4.4370162	1.59	4
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
0	0	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!

Boring Designation	Depth	Delta N	(N1)60cs	CRR-B
CP-B5	12	4.30	14	0.15
CP-B5	18	0.37	13	0.14
CP-B7	14	4.88	30	0.48
CP-B7	38	4.30	12	0.13
2-CP-B3	1	5.61	24	0.27
2-CP-B3	6	3.26	20	0.21
2-CP-B3	8	3.26	14	0.15
2-CP-B3	11	0.00	4	0.08
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!
0.00	0	0.00	#DIV/0!	#DIV/0!