

PRELIMINARY GEOTECHNICAL
ASSESSMENT

25906 GADING ROAD
HAYWARD, CALIFORNIA

The logo for ENGEO, featuring the company name in large, white, 3D block letters. The letters are set against a background that is a horizontal strip of three images: a rocky coastline with waves on the left, a green rolling hill with trees in the center, and a blue sky on the right. Below the letters, the tagline 'Expect Excellence' is written in a white, italicized serif font, flanked by two horizontal lines.

Expect Excellence

Submitted to:
Mr. Tony Dutra
Dutra Land & Consulting Services, Inc.
43430 Mission Blvd, Suite 210
Fremont, CA 94539

Prepared by:
ENGEO Incorporated

September 22, 2015

Project No:
12417.000.000

Project No.
12417.000.000

September 22, 2015

Mr. Tony Dutra
Dutra Land & Consulting Services, Inc.
43430 Mission Blvd., Suite 210
Fremont, CA 94539

Subject: 25906 Gading Road
Hayward, California

PRELIMINARY GEOTECHNICAL ASSESSMENT

Dear Mr. Dutra:

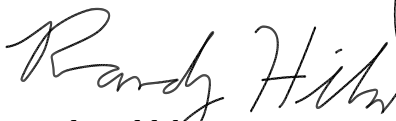
With your authorization and as part of due diligence, we performed a preliminary geotechnical assessment for the property located at 25906 Gading Road in Hayward, California. This report presents our geotechnical observations, as well as our preliminary conclusions and recommendations. We have also provided preliminary site grading, drainage, and foundation recommendations for use during land planning.

Based upon our initial assessment, it is our opinion that the proposed residential development is feasible from a geotechnical standpoint. Design-level exploration(s) should be conducted prior to site development once more detailed land plans have been prepared.

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

Sincerely,

ENGEO Incorporated


Randy Hildebrant, PE




Josef J. Tootle, GE



TABLE OF CONTENTS

Letter of Transmittal

1.0	INTRODUCTION	1
1.1	PURPOSE AND SCOPE	1
1.2	PROJECT LOCATION	1
1.3	PROJECT DESCRIPTION	1
1.4	SITE BACKGROUND	1
2.0	SITE GEOLOGY AND SEISMICITY	2
2.1	REGIONAL AND SITE GEOLOGY	2
2.2	SITE SEISMICITY	2
2.3	SURFACE CONDITIONS	2
2.4	FIELD EXPLORATION.....	3
2.5	LABORATORY TESTING	3
2.6	SUBSURFACE CONDITIONS	3
2.7	GROUNDWATER CONDITIONS.....	3
3.0	DISCUSSION AND PRELIMINARY CONCLUSIONS.....	4
3.1	EXPANSIVE SOILS.....	4
3.2	EXISTING FILL.....	4
3.3	SEISMIC HAZARDS	5
3.3.1	Ground Rupture	5
3.3.2	Ground Shaking	5
3.3.3	Lurching.....	5
3.3.4	Liquefaction/Clay Soil Softening	6
3.3.5	Seismic-Induced Settlement Analyses	6
3.3.5.1	Liquefaction Settlement and Cyclic Softening	6
3.3.5.2	Dynamic Densification Settlement	7
3.3.6	Lateral Spreading and Earthquake-Induced Landsliding.....	7
4.0	PRELIMINARY RECOMMENDATIONS.....	7
4.1	GRADING	7
4.1.1	Demolition and Stripping.....	8
4.1.2	Existing Fill.....	8
4.1.3	Selection of Materials	8
4.1.4	Differential Fill Thickness	9
4.1.5	Fill Placement	9
4.1.6	Surface Venting Mitigation.....	9
4.2	PRELIMINARY BUILDING CODE SEISMIC DESIGN	9
4.3	PRELIMINARY FOUNDATION DESIGN.....	10
4.3.1	Post-Tensioned Mat Foundation Design.....	10
4.4	PRELIMINARY PAVEMENT DESIGN	10
4.5	DRAINAGE.....	11

TABLE OF CONTENTS (Continued)

4.6	STORMWATER BIORETENTION AREAS.....	11
5.0	FUTURE STUDIES.....	13
6.0	LIMITATIONS AND UNIFORMITY OF CONDITIONS.....	13

FIGURES

- Figure 1 - Vicinity Map
- Figure 2 - Site Plan
- Figure 3 - Regional Geologic Map
- Figure 4 - Special Studies Zones Map
- Figure 5 - Seismic Hazard Zones Map
- Figure 6 - Regional Faulting and Seismicity Map

APPENDIX A - Cone Penetration Tests (CPTs)

APPENDIX B - Laboratory Test Data

APPENDIX C - Liquefaction Analysis

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of this preliminary geotechnical assessment, as described in our proposal dated August 12, 2015, is to provide an assessment of the potential geotechnical concerns associated with the use of the site for a residential development. The scope of our services included a site visit, a review of published geologic maps, review of readily available geotechnical and/or environmental reports for the site, advancing three Cone Penetration Tests (CPTs) ranging up to 50 feet deep and preparation of this report identifying potential geotechnical hazards.

This report was prepared for the exclusive use of our client and their consultants for evaluation 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 preliminary 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

The roughly 1.8-acre site is located in Hayward, California as shown on Figures 1 and 2. The site comprises of the parcel at 25906 Gading Road and is bounded to the east by Gading Road as shown on Figure 2. Existing residential parcels bound the site to the north, west, and south. Channelized Zeile Creek is located approximately 290 feet northeast of the project site. Based on regional topographic information, the bottom of the creek channel is approximately one to two feet below the project site elevation. Adjacent to the creek is an existing rail line.

1.3 PROJECT DESCRIPTION

According to the Conceptual Site Plan dated May 19, 2015, the improvements consist of 18 to 19 detached residential structures, a private street, and three private courts. We anticipate two- to three-story buildings of wood-framed construction with light to moderately light building loads and grading to consist of minor cuts and fills on the order of 2 to 3 feet.

1.4 SITE BACKGROUND

According to historical aerial imagery, since the 1946 photograph, the project site has remained relatively unchanged; however, it appears a private residence and detached garage once occupied the western portion of the project site. It appears that they were demolished in 2010.

2.0 SITE GEOLOGY AND SEISMICITY

2.1 REGIONAL AND SITE GEOLOGY

The region is within the Coast Range Province of California, an area dominated by northwest-trending geologic features such as folds and faults. More specifically, the subject site is located on alluvial deposits near the eastern margin of the San Francisco Bay. The San Francisco Bay is located in a fault bound, elongated structural trough that has been filled with a sequence of Quaternary age sedimentary deposits derived from the surrounding Coast Ranges.

Based on mapping by Helley and Graymer (1997), the deposits underlying the subject site comprise Holocene-aged floodplain deposits (Figure 3). Helley and Graymer describe the deposits as medium- to dark-gray, dense, sandy to silty clay. Lenses of coarser material (silt, sand, and pebbles) may be locally present.

2.2 SITE SEISMICITY

The San Francisco Bay Area contains numerous active faults. Figure 6 shows the approximate location of active and potentially active faults and significant historic earthquakes mapped within the San Francisco Bay Region. An active fault is defined by the State as one that has had surface displacement within Holocene time (about the last 11,000 years). Based on the 2010 USGS Quaternary Fault and Fold Database (QFFD), the nearest active fault is the Hayward fault located approximately 0.9 miles northeast of the site. Other active faults located near the site include the Calaveras fault, located approximately 8.9 miles to the east-northeast of the site, and the San Andreas fault, located approximately 17.7 miles to the west-southwest.

The Uniform California Earthquake Rupture Forecast (UCERF3, 2013) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area. The UCERF3 generated an overall probability of 72 percent for the San Francisco Region as a whole, a probability of 14.3 percent for the Hayward fault, 7.4 percent for the Calaveras fault, and 6.4 percent for the northern section of the San Andreas fault.

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone (Figure 4) and no known surface expression of active faults is believed to exist within the site.

2.3 SURFACE CONDITIONS

This site is generally level and the majority of the site contains grass, shrubs, and trees at the ground surface. An existing concrete driveway slab is located at the entrance of the project site.

2.4 FIELD EXPLORATION

Our field exploration included advancing three Cone Penetration Tests (CPTs) at various locations on the site as shown in Figure 2. We performed our field exploration on September 10, 2015.

The location of our explorations are approximate and were estimated by pacing from features shown on the site plan; they should be considered accurate only to the degree implied by the method used.

We retained a CPT rig to push the cone penetrometer to a maximum depth of about 50 feet. The CPT has a 20-ton compression-type cone with a 15-square-centimeter (cm²) base area, an apex angle of 60 degrees, and a friction sleeve with a surface area of 225 cm². The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D-3441. 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). CPT logs are presented in Appendix A.

Prior to performing the CPTs, the upper five feet was cleared for utilities using a hand auger. Near surface samples were collected from the cuttings and submitted to our laboratory for testing.

2.5 LABORATORY TESTING

We performed laboratory tests on selected soil samples to determine the plasticity index of the samples submitted. Individual test results are presented in Appendix B.

2.6 SUBSURFACE CONDITIONS

Our exploration CPTs encountered varying strata of alluvium, including medium stiff to very stiff lean clay and sandy lean clay, silt, and medium dense to dense silty and clayey sand. The near-surface soil consisted of a moderately to highly plastic clay ranging in plasticity index between 29 and 38. The CPTs generally encountered sandy material in the upper 10 to 12 feet, followed by fine-grained soil to about 16 feet below the ground surface. A silty sand to clean sand layer was encountered between 16 and 32 feet below the ground surface and varied in thickness between 8 and 16 feet. Below the sandy material, the CPTs encountered fine-grained material to the maximum depth explored.

The CPT logs include the specific subsurface conditions at the location of the probes. We include our exploration logs in Appendix A.

2.7 GROUNDWATER CONDITIONS

During our exploration, a pore pressure dissipation test was performed in each of the CPTs. The pore pressure dissipation indicated groundwater to range in depth between 14 and 17 feet below the ground surface.

The observations are consistent with the mapped historical groundwater data by the California Geologic Survey (CGS) at a depth of approximately 20 feet below the ground surface. Fluctuations in the level of groundwater may occur due to variations in rainfall, tidal influences, irrigation practice, and other factors.

3.0 DISCUSSION AND PRELIMINARY CONCLUSIONS

Based upon this preliminary study, it is our opinion that the project site is feasible for the proposed residential development from a geotechnical standpoint provided the preliminary recommendations contained in this report and future design-level geotechnical studies are incorporated into the development plans. A site-specific geotechnical exploration should be performed as part of the design process. The exploration would include borings and laboratory soil testing to provide data for preparation of specific recommendations regarding grading, foundation design, and drainage for the proposed development. The exploration will also allow for more detailed evaluations of the geotechnical issues discussed below and afford the opportunity to provide recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical/geological hazards.

Based upon our field exploration and review of readily available published maps and reports for the site, the main geotechnical concerns for the proposed site development include: (1) potentially liquefiable soil, (2) the presence of potentially expansive near-surface soils, and (3) the presence of undocumented fills or buried structures. These items and other geotechnical issues are discussed in the following sections of this report.

3.1 EXPANSIVE SOILS

We observed highly expansive clay near the surface of the site. Successful performance of structures on expansive soils requires special attention during construction. Expansive soils change in volume with changes in moisture. These soils can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction.

We provide preliminary grading recommendations for compaction of clay soil at the site. The purpose of these preliminary recommendations is to reduce the swell potential of the clay by compacting the soil at a high moisture content and controlling the amount of compaction. Preliminary earthwork recommendations are presented in Section 4.1 of this report.

3.2 EXISTING FILL

We could not determine the presence of fill due to our exploration type; however, minor fills likely exist associated with the existing and former structures and associated underground facilities. These fills will likely require subexcavation and placement as engineered fill. The extent and quality of existing fills should be evaluated, and potential mitigation measures recommended, at the time of design-level study.

The presence of existing fill can lead to differential foundation movement due to the unknown density of the fill and due to differences in material properties for structures that span from the fill to native materials. Mitigation can include removal and recompaction of the fill.

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, ground lurching, soil liquefaction, and lateral spreading. These hazards are discussed in the following sections.

Based on topographic and lithologic data, regional subsidence or uplift, tsunamis, landslides and seiches is considered low at the site.

3.3.1 Ground Rupture

The site is not located within a State of California Earthquake Fault Hazard Zone (1982) as shown in Figure 4. Therefore, since no known active faults cross the site, it is our opinion that ground rupture is not likely to occur at the site.

3.3.2 Ground Shaking

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

3.3.3 Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site, as in other geologically similar locations in the Bay Area, but the offset or strain is expected to be low to negligible.

3.3.4 Liquefaction/Clay Soil Softening

The site is located within a State of California Seismic Hazard Zone (CGS, 2003) for areas that may be susceptible to liquefaction (Figure 5). Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands. Empirical evidence indicates that loose to medium dense gravels, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable.

As described previously, layers of sandy soil up to 16 feet thick below the groundwater table were encountered in our exploration locations. We performed a detailed liquefaction potential analysis of the CPT soundings to estimate liquefaction potential using the computer software CLiq Version 1.7 developed by GeoLogismiki. The procedure used in the software is based on the procedure by Boulanger and Idriss (2014). The Cyclic Stress Ratio (CSR) was estimated for a Peak Ground Acceleration (PGA) value of 0.87g as outlined in the ASCE 7-10 and moment magnitude of 7.3. We evaluated the liquefaction potential for the soils encountered below the assumed water table. The results indicate that medium dense sand layers encountered in the CPTs are potentially liquefiable.

3.3.5 Seismic-Induced Settlement Analyses

Seismically induced settlement can be generally subdivided into two categories for granular soils: (1) settlement as a result of liquefaction of saturated or nearly saturated soils and (2) dynamic densification of non-saturated soils. Research has also shown that low-expansive cohesive soils can also undergo post-seismic settlement.

3.3.5.1 Liquefaction Settlement and Cyclic Softening

Deformation of the ground surface is a common result of liquefaction. Vertical settlement may result from densification of the deposit or volume loss from venting to the ground surface. Densification occurs as excess pore pressures dissipate, resulting as vertical settlement at the ground surface. In addition to the above analysis, we also evaluated the capping effect of any overlying non-liquefiable soils. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a sufficient enough force to break through the overlying soil and vent to the surface resulting in sand boils or fissures.

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

Based on the above studies, the potentially liquefiable soils may not be sufficiently capped by a sufficient thickness of non-liquefiable soils to prevent venting. The settlement estimates provided below assume that the potential surface venting has been mitigated as recommended in

Section 4.1.6. This hazard should be further evaluated during a design level study with additional borings and laboratory testing.

Clay-like (cohesive) soils can also develop pore pressures during cyclic loading, but generally do not reach zero effective stress and are typically considered non-liquefiable (Robertson 2009). However, clay-like soils can deform during cyclic earthquake loading and experience volumetric strains and post-earthquake reconsolidation. The volumetric strains for clay-like soils are generally small compared to cohesionless soils (sand-like), since clay-like soils often retain some original soil structure. Clay reconsolidation was estimated using the program Cliq.

We calculated potential liquefaction-induced settlement estimates using the program Cliq. The procedures used in Cliq are based on the methods published by Zhang, G., Robertson, P.K., and Brachman, R. (2002). Since some of the granular materials were characterized as medium dense and potentially liquefiable and some fine-grained soil is susceptible to soil softening, we estimate the total liquefaction-induced settlements across the site to be less than 2¾ inches. Differential settlement during a liquefaction event is expected to be less than 1½ inch (SCEC, 1999).

3.3.5.2 Dynamic Densification Settlement

Densification of loose granular soils above the water table can cause settlement of the ground surface due to earthquake-induced vibrations. We calculated potential liquefaction-induced settlement estimates using the program Cliq. The procedures used in Cliq are based on the methods published by Zhang, G., Robertson, P.K., and Brachman, R. (2002). Our analysis indicates up to approximately 1¼ inch of settlement may occur due to dynamic densification at the site. Differential settlement is expected to be less than ¾ inch (SCEC, 1999).

3.3.6 Lateral Spreading and Earthquake-Induced Landsliding

Lateral spreading and earthquake-induced landsliding involve lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied sands or weak soils. Due to the relatively flat site topography, distance to free-faces, and depth of liquefiable material, lateral spreading is unlikely at the site.

4.0 PRELIMINARY RECOMMENDATIONS

4.1 GRADING

The following preliminary recommendations are for initial land planning and preliminary estimating purposes. Final recommendations regarding site grading and foundation construction will be provided after additional site-specific exploration has been undertaken.

4.1.1 Demolition and Stripping

Site development will commence with the removal of buried structures, including abandoned utilities and septic tanks and their leach fields, if any exist. All debris should be removed from any location to be graded, from areas to receive fill or structures, or those areas to serve as borrow. The depth of removal of such materials should be determined by the Geotechnical Engineer in the field at the time of grading.

Existing vegetation and pavements (asphalt concrete/concrete and underlying aggregate base) should be removed from areas to receive fill, or structures, or those areas to serve for borrow. Tree roots should be removed down to a depth of at least 3 feet below existing grade. The actual depth of tree root removal should be determined by the Geotechnical Engineer's representative in the field. Subject to approval by the Landscape Architect, strippings and organically contaminated soils can be used in landscape areas. Otherwise, such soils should be removed from the project site. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill.

No loose or uncontrolled backfilling of depressions resulting from demolition and stripping is permitted.

4.1.2 Existing Fill

All existing fill and soft material should be excavated to firm native soils. Excavated material may be used as fill material if it meets the requirements of Section 4.1.3.

4.1.3 Selection of Materials

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

Imported fill materials should meet the above requirements and have a plasticity index less than the on-site soils. ENGEO should sample and test proposed imported fill materials at least 72 hours prior to delivery to the site.

4.1.4 Differential Fill Thickness

Cuts associated with removal of buried structures, foundations, tanks, or undocumented fills could result in differential fill thickness conditions. For subexcavation activities that create a differential fill thickness across a building footprint, mitigation to achieve a similar fill thickness across the pad is beneficial for the performance of a shallow foundation system. We recommend that a differential fill thickness of up to 5 feet is acceptable across a building footprint. For a differential fill thickness exceeding 5 feet across a footprint, we recommend performing subexcavation activities to bring this vertical distance to within the 5-foot tolerance and that the material be replaced as engineered fill. As a minimum, the subexcavation area should include the entire structure footprint plus 5 feet beyond the edges of the building footprint.

4.1.5 Fill Placement

For land planning and cost estimating purposes, the following compaction control requirements should be anticipated for general fill areas:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 4 percentage points above optimum moisture content.
Minimum Relative Compaction:	Between 87 and 92 percent.

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

Additional compaction requirements may be required for deeper fills and retaining wall backfill. These additional requirements will be developed during our detailed exploration.

4.1.6 Surface Venting Mitigation

As previously stated, there may not be a sufficient amount of non-liquefiable soil overlying potentially liquefiable soil to prevent surface venting and volume loss. A potential mitigation option to strengthen the overlying soil is to subexcavate 6 feet deep below building, place biaxial geogrid at 6, 4, and 2 foot depth, and backfill with engineered fill. As an alternative, the building foundation may be designed to accommodate additional differential settlement due to volume loss.

4.2 PRELIMINARY BUILDING CODE SEISMIC DESIGN

We provide the 2013 California Building Code (CBC) seismic parameters in Table 4.2-1 below.

TABLE 4.2-1
2013 CBC Seismic Design Parameters

Parameter	Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_s (g)	2.26
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.94
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.26
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.41
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.51
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.94
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.87
Site Coefficient, F_{PGA}	1.00
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.87
Long period transition-period, T_L	8 sec

4.3 PRELIMINARY FOUNDATION DESIGN

In order to reduce the effects of the potentially expansive soils, the foundations should be sufficiently stiff to move as rigid units with minimum differential movements. This can be accomplished with a post-tensioned mat foundation.

4.3.1 Post-Tensioned Mat Foundation Design

A minimum mat thickness of 12 inches should be anticipated for preliminary purposes. We anticipate that structural mats constructed on swelling soils will move differentially; therefore, structural mats may require stiffening to reduce differential movements due to swelling/shrinkage to a value compatible with the type of structure that will be constructed. The foundations should be designed for 2 inches differential seismic induced settlement over a distance of 30 feet. If the grading mitigation presented in Section 4.1.6 is not performed, at a higher risk to the structure, the foundations may be design to accommodate 3 inches of differential seismic induced settlement.

4.4 PRELIMINARY PAVEMENT DESIGN

The following preliminary pavement section has been determined for an assumed R-value of 5 and in accordance to the design methods contained in Chapter 630 of Caltrans Highway Design Manual.

TABLE 4.4-1
Preliminary Pavement Section

Traffic Index	AC (inches)	AB (inches)
5.0	3.0	10.0
6.0	3.5	13.0
7.0	4.0	16.0

Note: AC – Asphalt Concrete
AB – Caltrans Class 2 aggregate base (R-value of 78 or greater)

The above preliminary pavement sections are provided for estimating only. We recommend the actual subgrade material should be tested for R-value and the Traffic Index and minimum pavement section(s) should be confirmed by the Civil Engineer and the City of Hayward.

4.5 DRAINAGE

The building pads must be positively graded at all times to provide for rapid removal of surface water runoff from the foundation systems and to prevent ponding of water under floors or seepage toward the foundation systems at any time during or after construction. Ponding of stormwater must not be permitted on the building pads during prolonged periods of inclement weather. All surface water should be collected and discharged into the storm drain system. Landscape mounds must not interfere with this requirement.

All roof stormwater should be collected and directed to downspouts. Stormwater from roof downspouts should be directed to a solid pipe that discharges to the street or to an approved outlet or onto an impervious surface, such as pavement that will drain at a 2 percent slope gradient.

Due to the generally high fines content anticipated in the near-surface site materials, the site soils encountered are not expected to have adequate permeability values to handle stormwater infiltration in grassy swales or permeable pavers. Therefore, best management practices should assume that little stormwater infiltration will occur at the site.

4.6 STORMWATER BIORETENTION AREAS

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

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or

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

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

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

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

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

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

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

5.0 FUTURE STUDIES

As previously discussed, a site-specific design-level geotechnical exploration should be performed as part of the design process. The exploration would include borings and laboratory soil testing to provide data for preparation of specific recommendations regarding grading, foundation design, corrosion potential, and drainage for the proposed development. The exploration will also allow for more detailed evaluations of the geotechnical issues discussed in this report and afford the opportunity to provide recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical/geological hazards.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents preliminary geotechnical recommendations for design of the improvements discussed in Section 1.3 for the 25906 Gading Road 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 preliminary 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 preliminary conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

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

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, notify ENGEO immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, 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 written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

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

SELECTED REFERENCES

- American Society of Civil Engineers. Minimum Design Loads for Buildings and other Structures ASCE 7-10. Reston: American Society of Civil Engineers. 2010.
- Boulanger, R. W. and Idriss, I. M. (2014). CPT and SPT Based Liquefaction Triggering and Procedures. Report No. UCD/CGM-14/01.
- California Building Standards Commission, 2013 California Building Code, Volumes 1 and 2. Sacramento, California.
- California Division of Mines and Geology, 1982, Special Studies Zone Maps, Hayward Quadrangle, California, State of California.
- California Department of Transportation, 2010, Highway Design Manual.
- Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, and Zeng, Y., 2013, Uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model: U.S. Geological Survey Open-File Report 2013–1165, 97 p., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, <http://pubs.usgs.gov/of/2013/1165/>
- Hart, E.W. and Bryant, W.A., 1997, Fault rupture hazard in California: Alquist-Priolo earthquake fault zoning act with index to earthquake fault zone maps: California Division of Mines and Geology Special Publication 42.
- Helley, E.J. and Graymer, R. W., 1997, Quaternary geology of Alameda County, and parts of Contra Costa, Santa Clara, San Mateo, San Francisco, Stanislaus, and San Joaquin Counties, California. Open-File Report 97-97 scale 1:100,000.
- International Code Council (2010). California Building Code.
- Idriss, I. M., and Boulanger, R. W. (2008). Soil liquefaction during earthquakes. Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA, 261 pp.
- Ishihara, K. (1985), Stability of Natural Deposits During Earthquakes, Proc 11th International Conference on Soil Mechanics and Foundation Engineering, Vol 1, A. A. Balkema, Rotterdam, The Netherlands, 321-376.
- Ishihara, K. and Ishihara, K. and Yoshimine, M. (1992). “Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes.” Soils and Foundations, Vol. 32, No. 1, March pp. 173-188.

SELECTED REFERENCES (Continued)

Post-Tensioning Institute, 2004, Design of Post-Tensioned Slabs-on-Ground, Third Edition.

Robertson, P. K. and Campanella, R. G., Guidelines for Geotechnical Design Using CPT and CPTU Data.

Robertson, P. K. (2009), Performance based earthquake design using the CPT, Gregg Drilling and Testing, Inc.

State of California, California Geologic Survey (CGS), 2003, Seismic Hazard Zones, Official Map, Hayward Quadrangle, July 2, 2003.

Structural Engineers Association of California (SEAOC) (1996). Recommended Lateral Force Requirements and Tentative Commentary.

Southern California Earthquake Center (1999), Recommended Procedures For Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction in California.

Tokimatsu, K. and Seed, H. B. (1987). "Evaluation of Settlements in Sands due to Earthquake Shaking." Journal of Geotechnical Engineering, Vol. 113, No. 8, pp. 861-878.

Youd, T. L. and C. T. Garris, 1995, Liquefaction induced Ground-Surface Description: Journal of Geotechnical Engineering, Vol. 121, No. 11, pp. 805 – 809.

Youd T. L. et al. (2001) "Liquefaction Resistance of Soils: Summary Report from the NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils." Journal of Geotechnical and Geoenvironmental Engineering., ASCE, 127(10), Oct., pp. 817-833.

Zhang, G. Robertson. P.K, Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180.

FIGURES

Figure 1 - Vicinity Map

Figure 2 - Site Plan

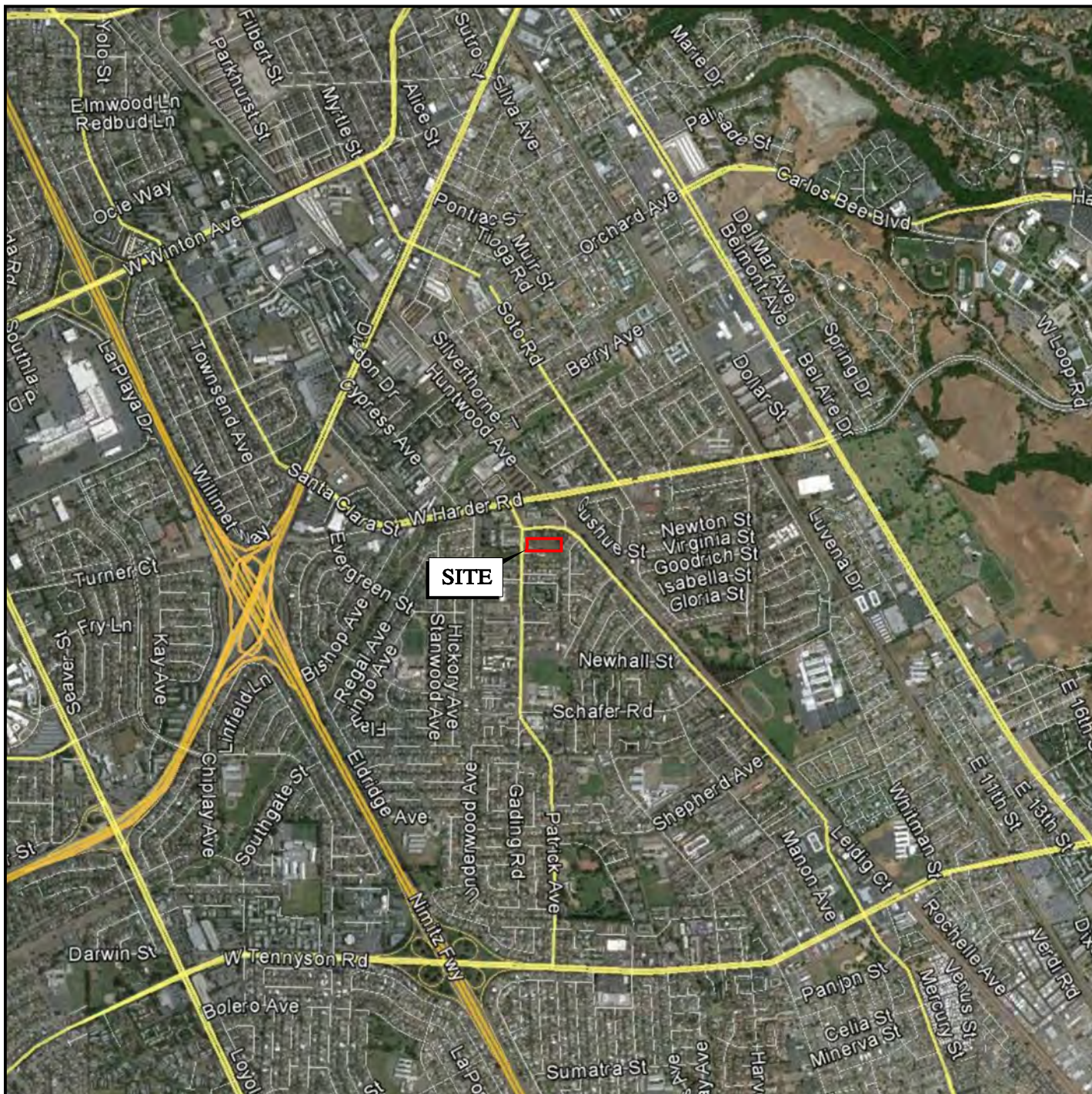
Figure 3 - Regional Geologic Map (Graymer)

Figure 4 - Special Studies Zones Map

Figure 5 - Seismic Hazard Zones Map

Figure 6 - Regional Faulting and Seismicity Map





BASE MAP SOURCE: GOOGLE EARTH PRO



VICINITY MAP
25906 GADING ROAD
HAYWARD, CALIFORNIA

PROJECT NO.: 12417.000.000

SCALE: AS SHOWN

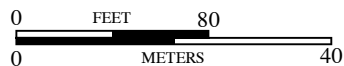
DRAWN BY: SRP

CHECKED BY: JJT

FIGURE NO.

1

G:\Drafting\DRAWING2\DWG\10000 to 12999\2417\000\2417000000-2-SitePlan-0915.dwg Plot Date: 9-14-15 spotters



BASE MAP SOURCE: GOOGLE EARTH PRO



SITE PLAN
25906 GADING ROAD
HAYWARD, CALIFORNIA

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

1-CPT3  CONE PENETRATION TEST

PROJECT NO.: 12417.000.000

SCALE: AS SHOWN

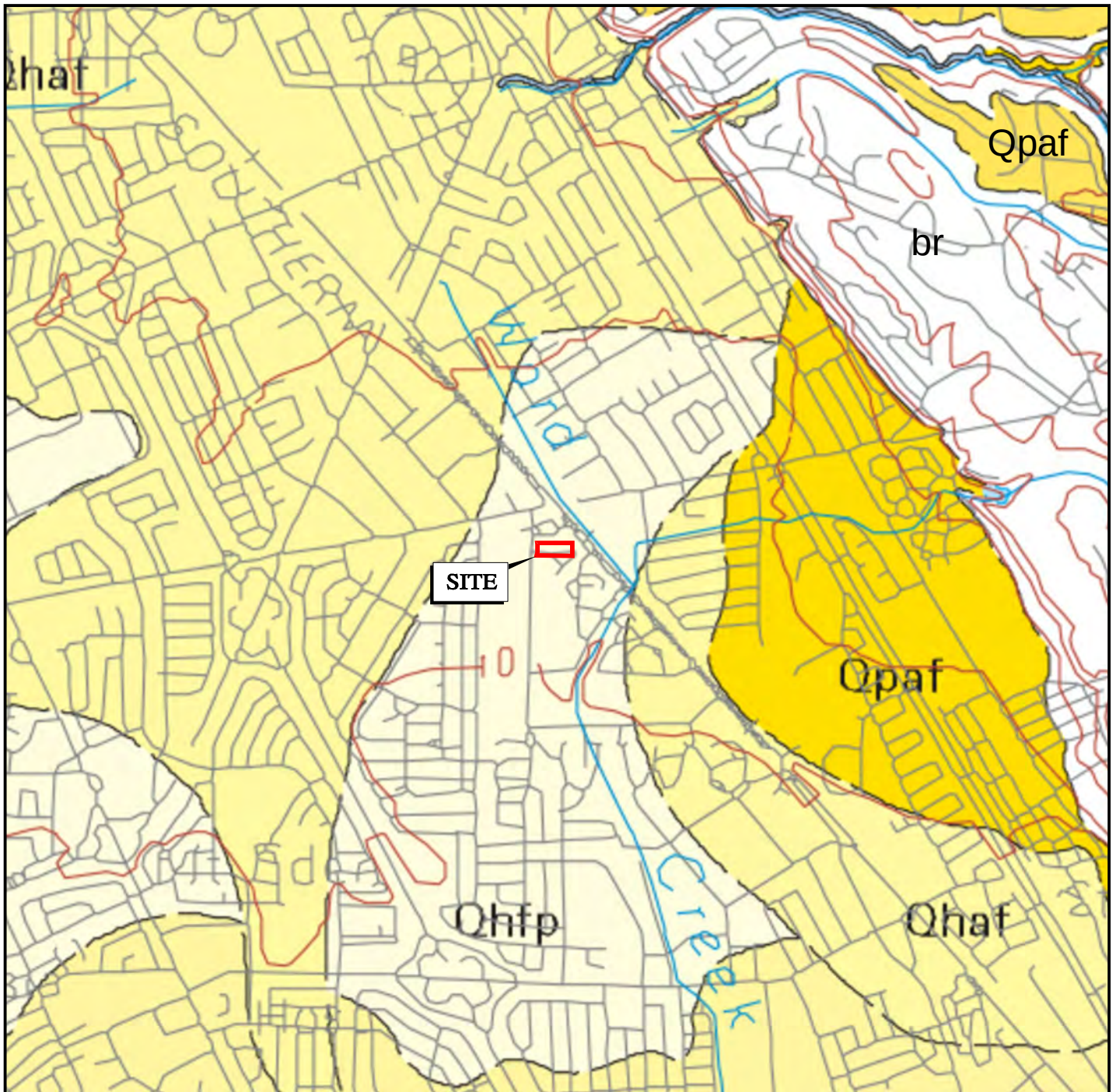
DRAWN BY: SRP

CHECKED BY: JJT

FIGURE NO.

2

ORIGINAL FIGURE PRINTED IN COLOR



0 FEET 2000
0 METERS 1000

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- Qhaf ALLUVIAL FAN DEPOSIT (HOLOCENE)
Qhfp FLOODPLAIN DEPOSIT (HOLOCENE)
Qpaf ALLUVIAL FAN DEPOSITS (PLEISTOCENE)
br UNDIFFERENTIATED BEDROCK (PLEISTOCENE AND OTHER)

BASE MAP SOURCE: GRAYMER, 1997



REGIONAL GEOLOGIC MAP

25906 GADING ROAD
HAYWARD, CALIFORNIA

PROJECT NO.: 12417.000.000

SCALE: AS SHOWN

DRAWN BY: SRP

CHECKED BY: JJT

FIGURE NO.

3

The legend consists of two parts. The top part shows four horizontal line styles: a solid line labeled 'C' above it, a long dashed line labeled '?' above it, a short dashed line, and a dotted line. To the left of these lines is the year '1906'. The bottom part shows a symbol consisting of a circle connected by a horizontal line to another circle.

1906 C
? ———
- - -
.....

Faults considered to have been active during Holocene time and to have a relatively high potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by creep or possible creep

○ ——— ○ Earthquake fault zone boundaries; delineated as straight-line segments that connect encircled turning points so as to define earthquake fault zone segments

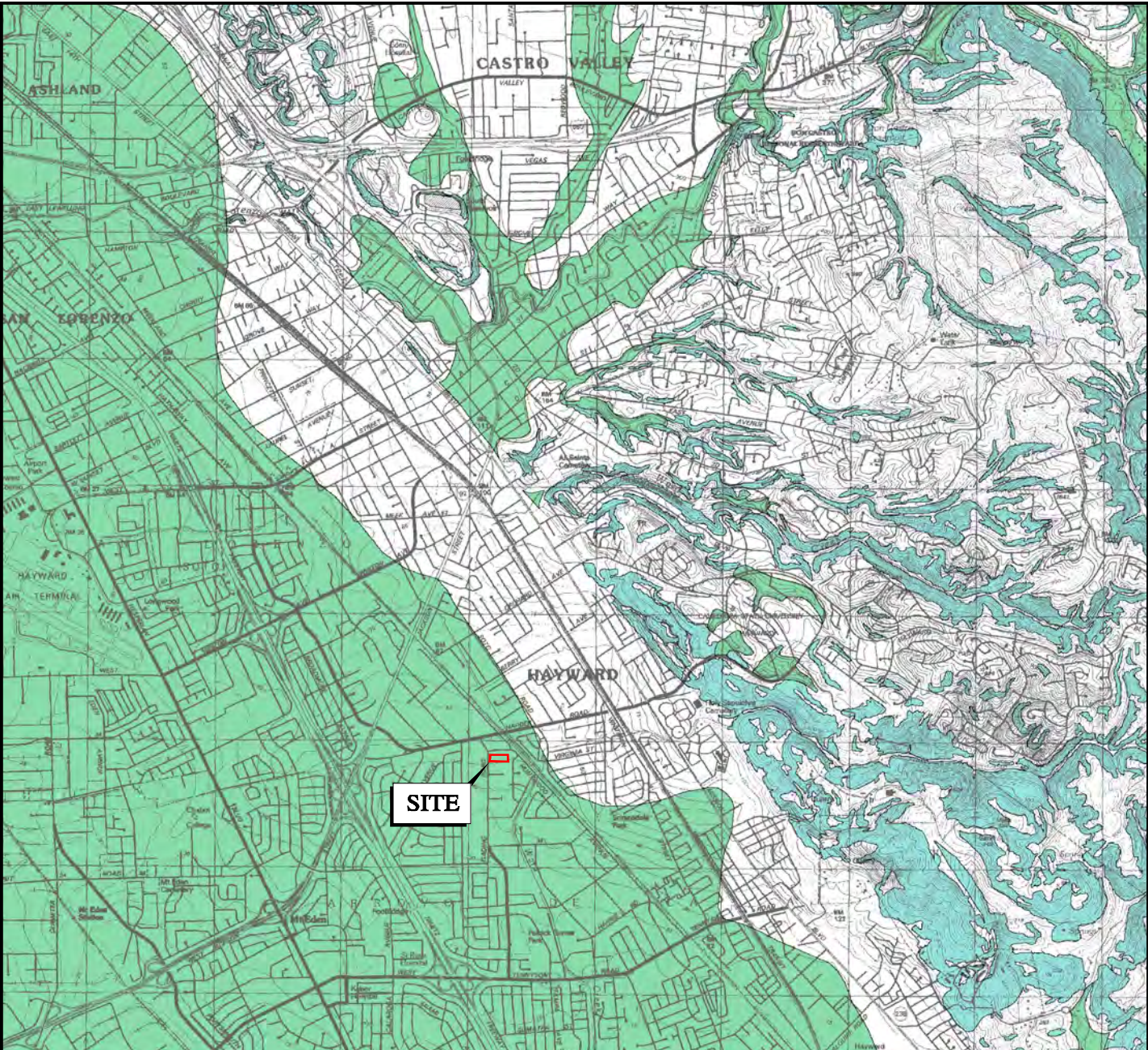


25906 GADING ROAD
HAYWARD, CALIFORNIA

DRAWN BY: SRP	CHECKED BY: JJT
----------------------	------------------------

4

COPYRIGHT © 2015 BY ENGEO INCORPORATED. THIS DOCUMENT MAY NOT BE REPRODUCED IN WHOLE OR IN PART BY ANY MEANS WHATSOEVER, NOR MAY IT BE QUOTED OR EXCERPTED WITHOUT THE EXPRESS WRITTEN CONSENT OF ENGEO INCORPORATED.



EXPLANATION

LIQUEFACTION

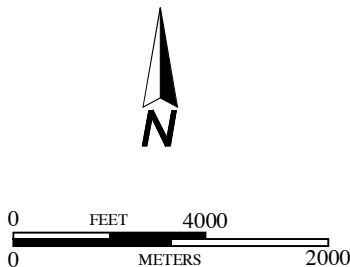


AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

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



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



SEISMIC HAZARD ZONES MAP
25906 GADING ROAD
HAYWARD, CALIFORNIA

PROJECT NO.: 12417.000.000

SCALE: AS SHOWN

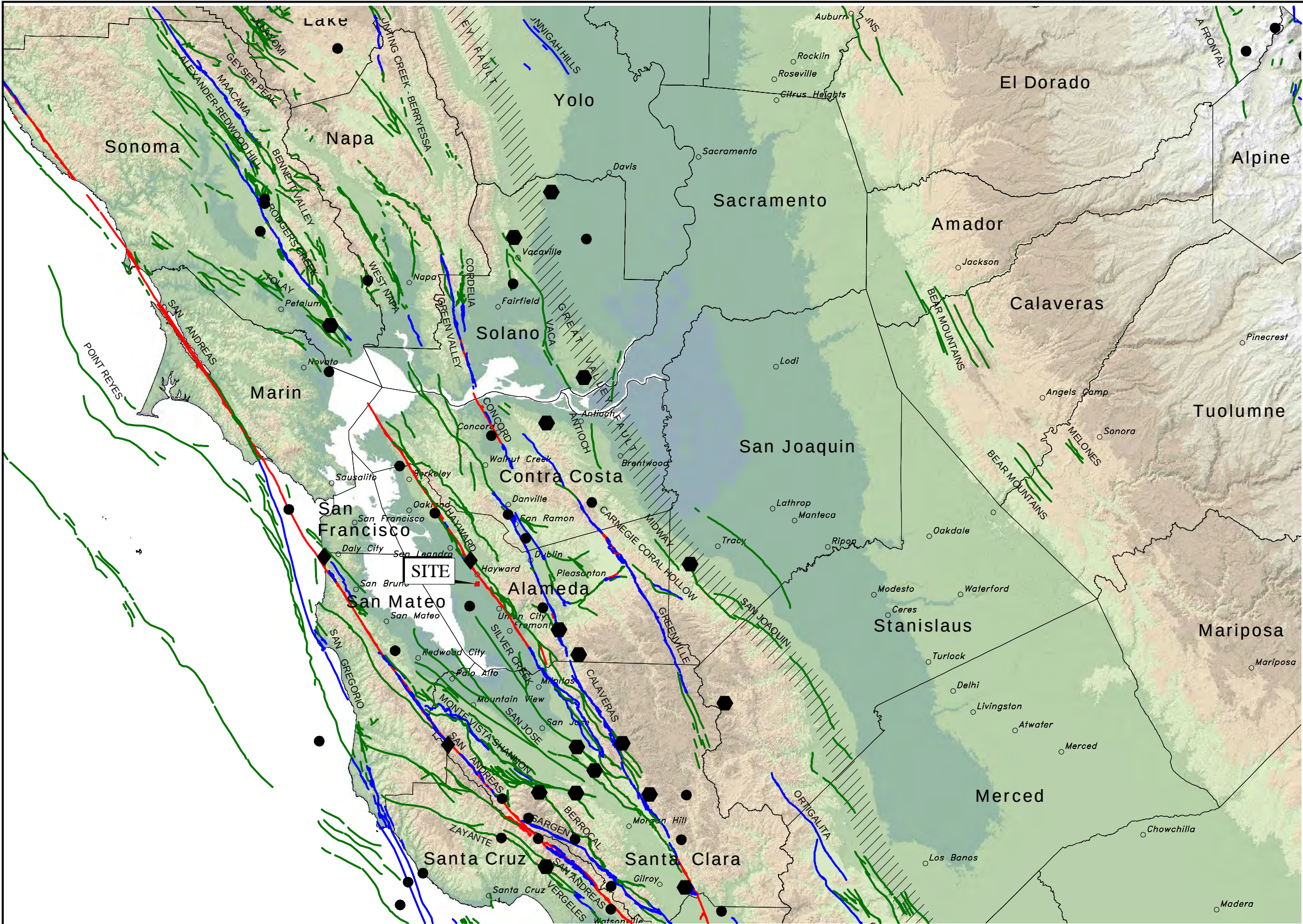
DRAWN BY: SRP

CHECKED BY: JJT

FIGURE NO.

5

COPYRIGHT © 2015 BY ENGEO INCORPORATED. THIS DOCUMENT MAY NOT BE REPRODUCED IN WHOLE OR IN PART BY ANY MEANS WHATSOEVER, NOR MAY IT BE QUOTED OR EXCERPTED WITHOUT THE EXPRESS WRITTEN CONSENT OF ENGEO INCORPORATED.



EXPLANATION

◆	MAGNITUDE 7+
⬡	MAGNITUDE 6-7
●	MAGNITUDE 5-6
— (Red)	HISTORIC FAULT
— (Blue)	HOLOCENE FAULT
— (Green)	QUATERNARY FAULT
///	HISTORIC BLIND THRUST FAULT ZONE

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



REGIONAL FAULTING AND SEISMICITY
25906 GADING ROAD
HAYWARD, CALIFORNIA

PROJECT NO.: 12417.000.000
SCALE: AS SHOWN
DRAWN BY: SRP CHECKED BY: JJT

FIGURE NO
6

APPENDIX A
Cone Penetration Tests
(CPTs)





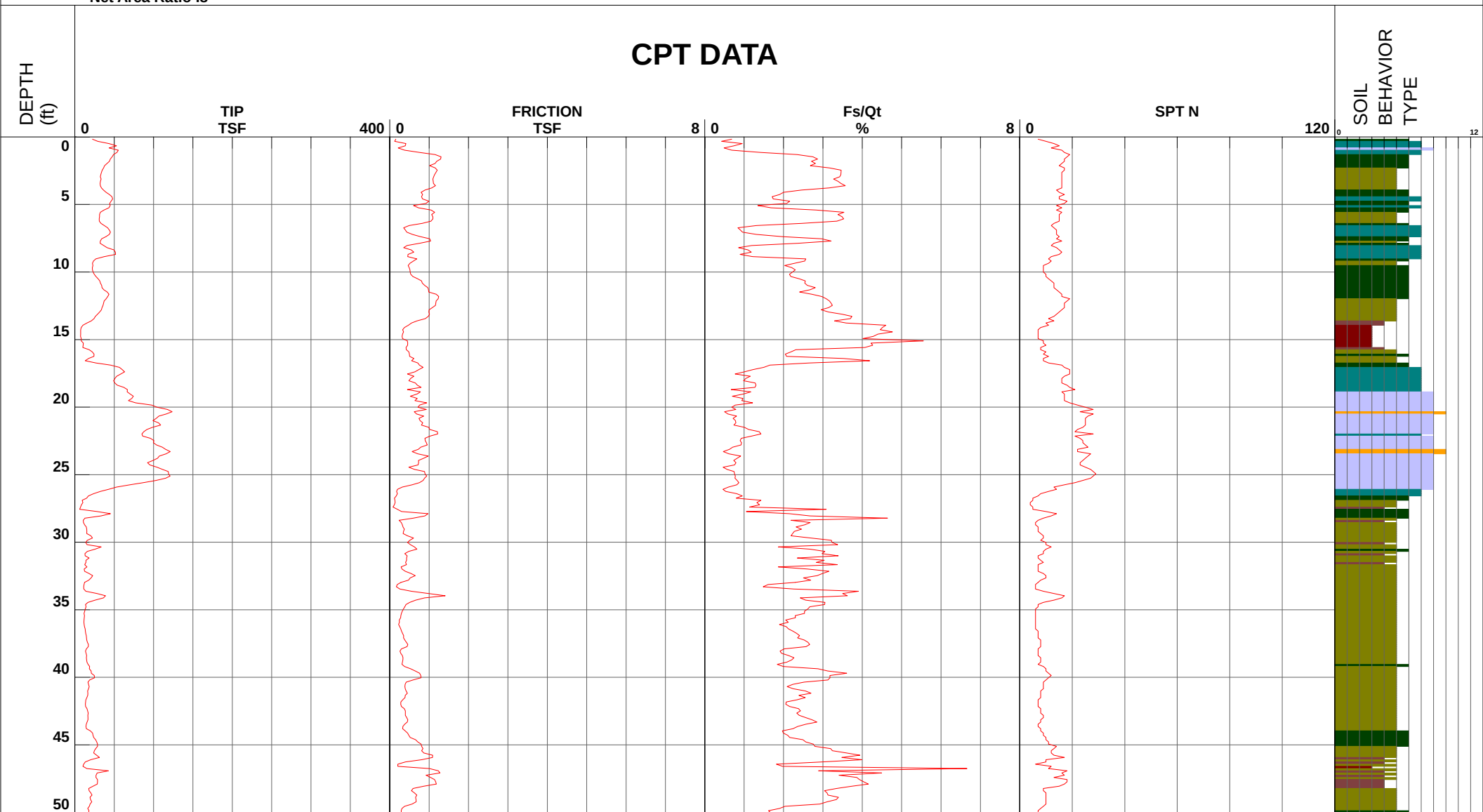
Engeo Inc

Project 25906 Gading Road
Job Number 12417.000.000
Hole Number CPT-01
EST GW Depth During Test

Operator CB
Cone Number DDG1281
Date and Time 9/10/2015 7:41:51 AM
14.00 ft

Filename SDF(139).cpt
GPS
Maximum Depth 50.52 ft

Net Area Ratio .8



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

Cone Size 10cm squared

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

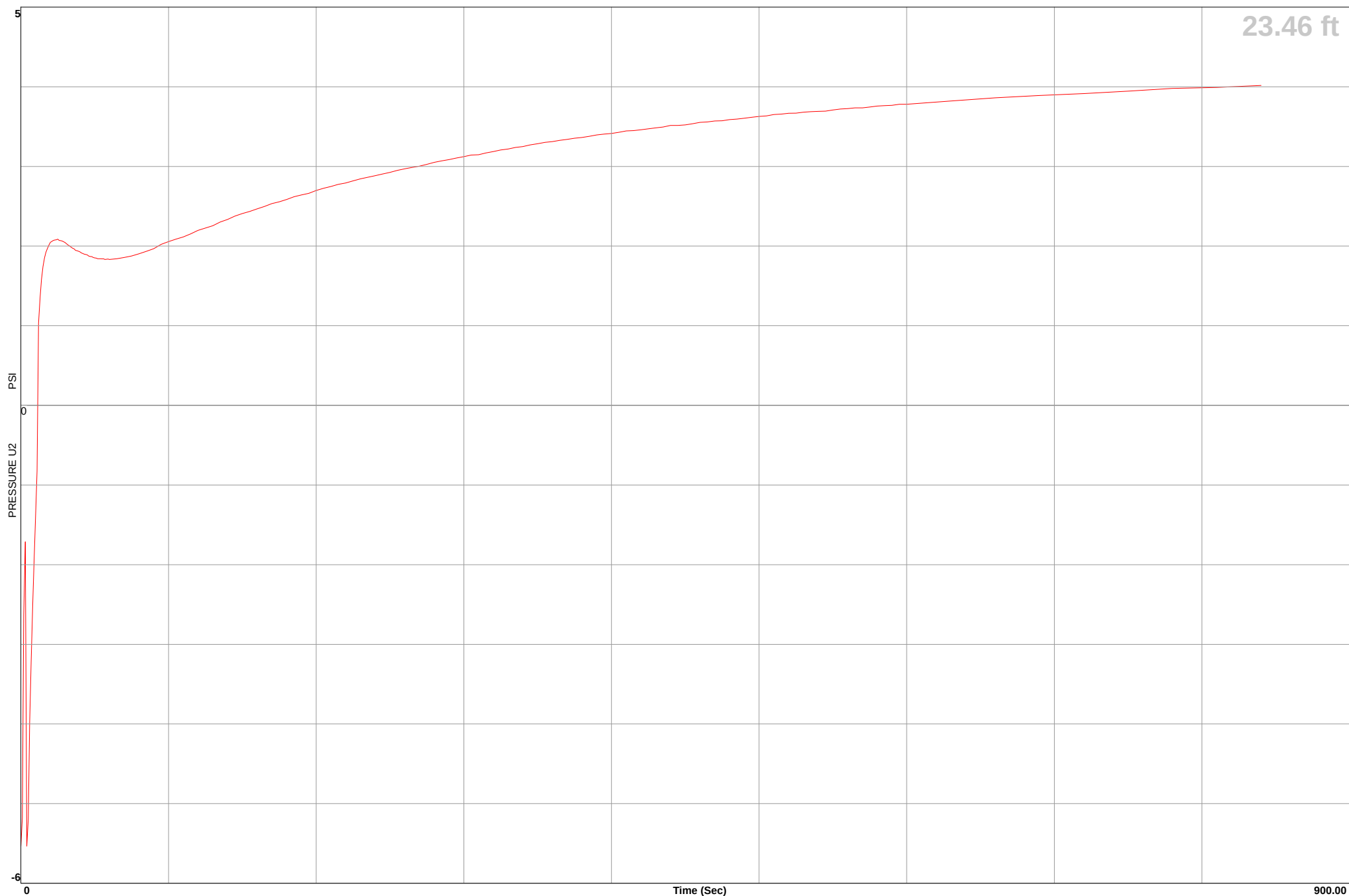


Engeo Inc

Location 25906 Gading Road
Job Number 12417.000.000
Hole Number CPT-02
Equilized Pressure 4.0

Operator CB
Cone Number DDG1281
Date and Time 9/10/2015 8:17:21 AM
EST GW Depth During Test 14.1

GPS _____





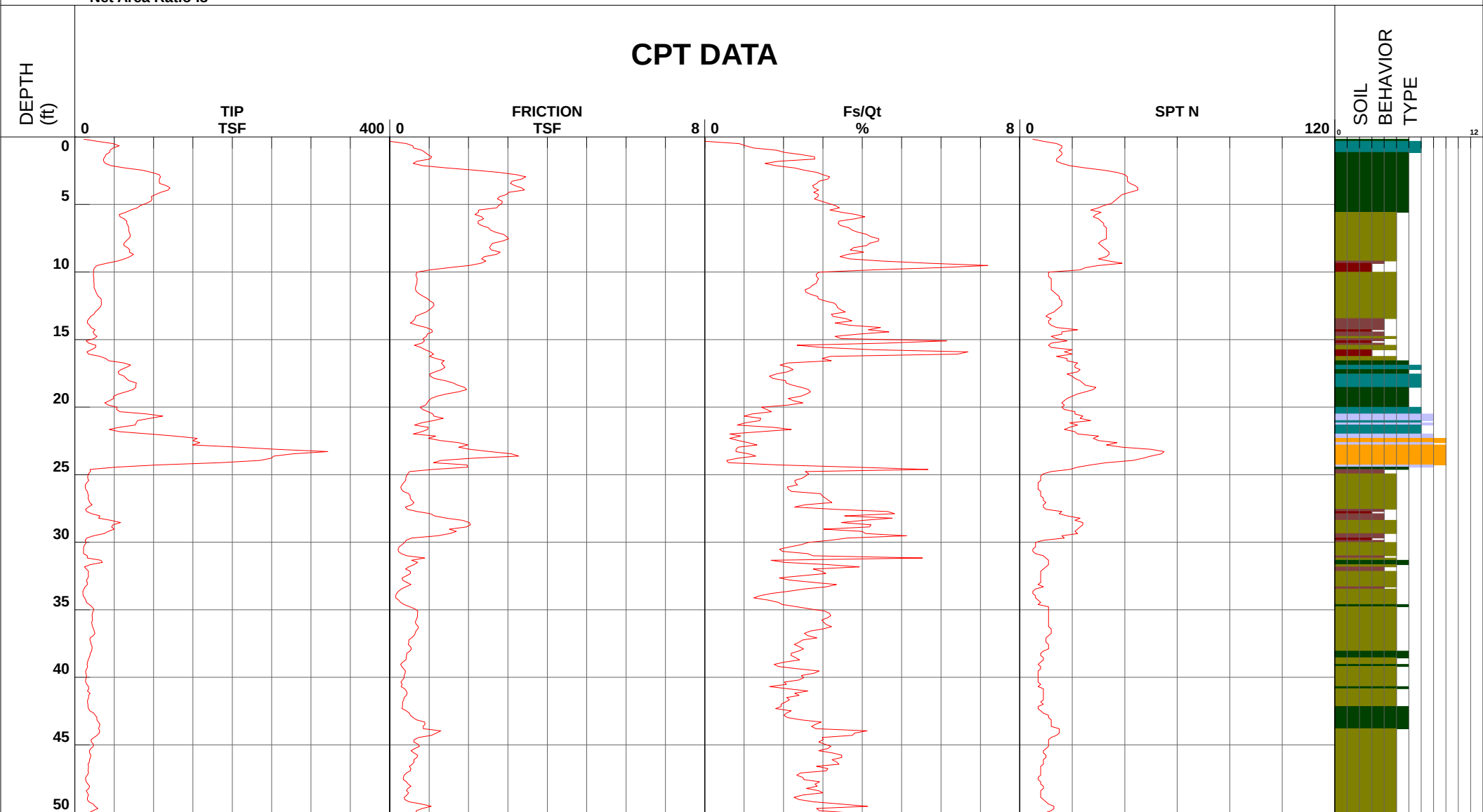
Engeo Inc

Project 25906 Gading Road
Job Number 12417.000.000
Hole Number CPT-02
EST GW Depth During Test

Operator CB
Cone Number DDG1281
Date and Time 9/10/2015 8:17:21 AM
14.10 ft

Filename SDF(140).cpt
GPS
Maximum Depth 50.52 ft

Net Area Ratio .8



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

Cone Size 10cm squared

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

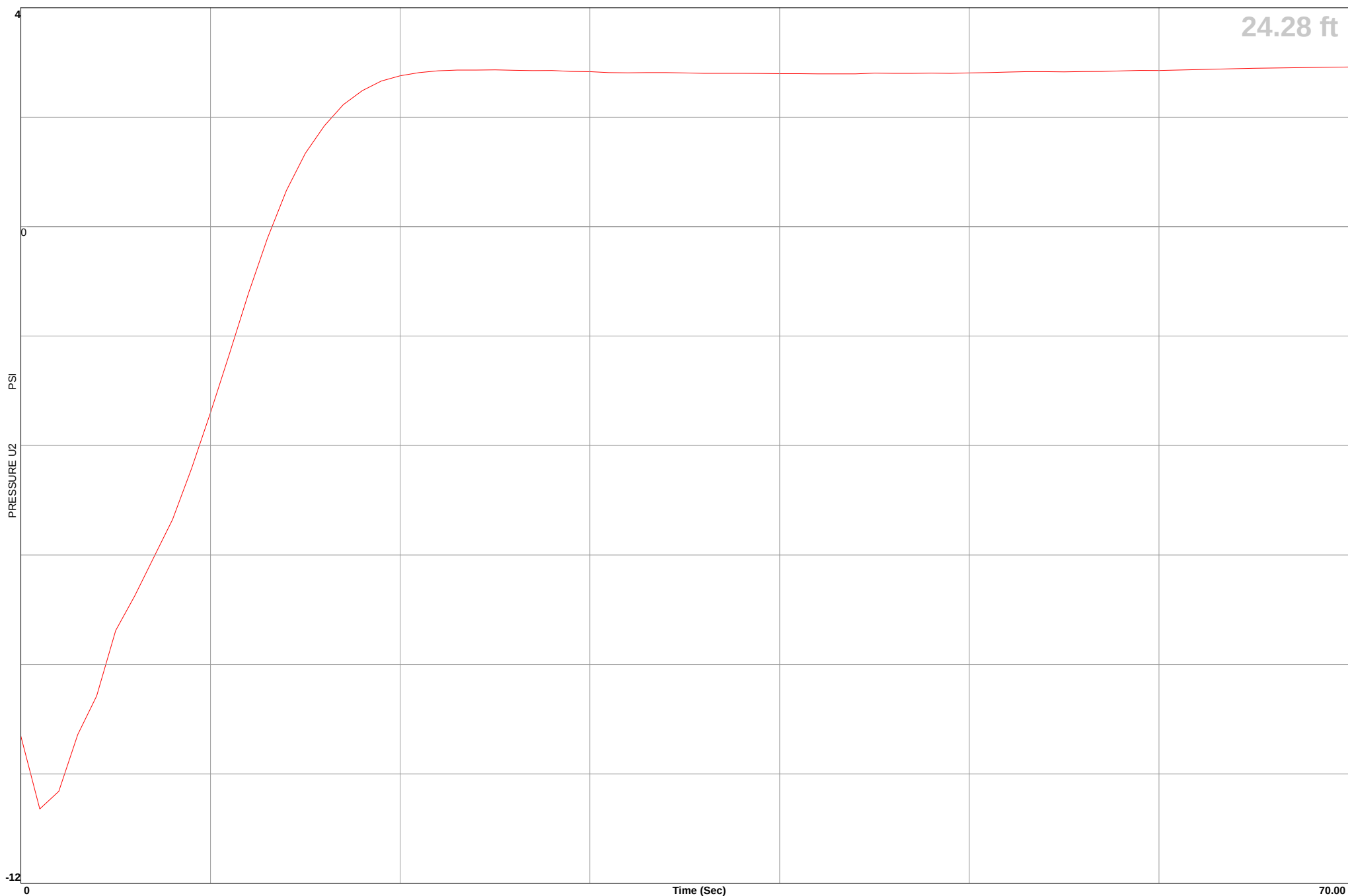


Engeo Inc

Location 25906 Gading Road
Job Number 12417.000.000
Hole Number CPT-03
Equilized Pressure 2.8

Operator CB
Cone Number DDG1281
Date and Time 9/10/2015 9:06:20 AM
EST GW Depth During Test 17.7

GPS _____





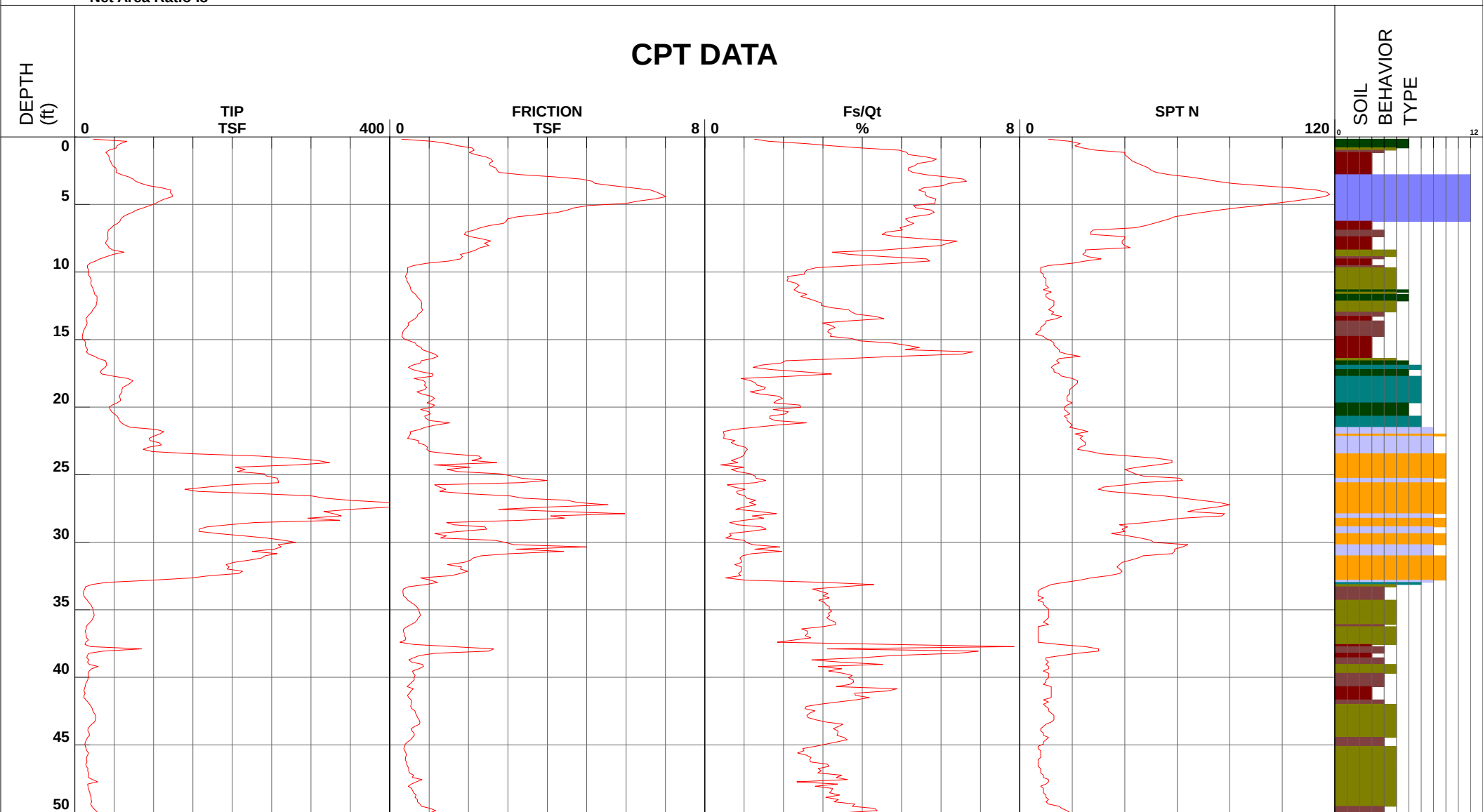
Engeo Inc

Project 25906 Gading Road
Job Number 12417.000.000
Hole Number CPT-03
EST GW Depth During Test

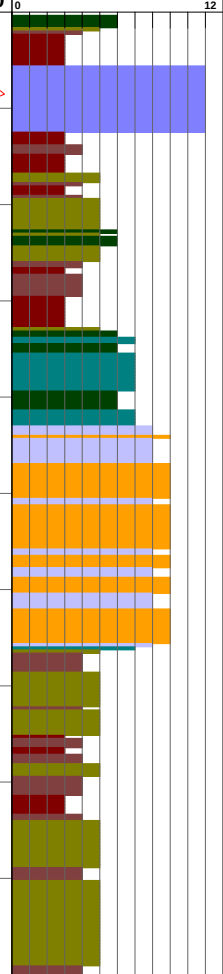
Operator CB
Cone Number DDG1281
Date and Time 9/10/2015 9:06:20 AM
17.70 ft

Filename SDF(141).cpt
GPS
Maximum Depth 50.52 ft

Net Area Ratio .8



SOIL
BEHAVIOR
TYPE



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

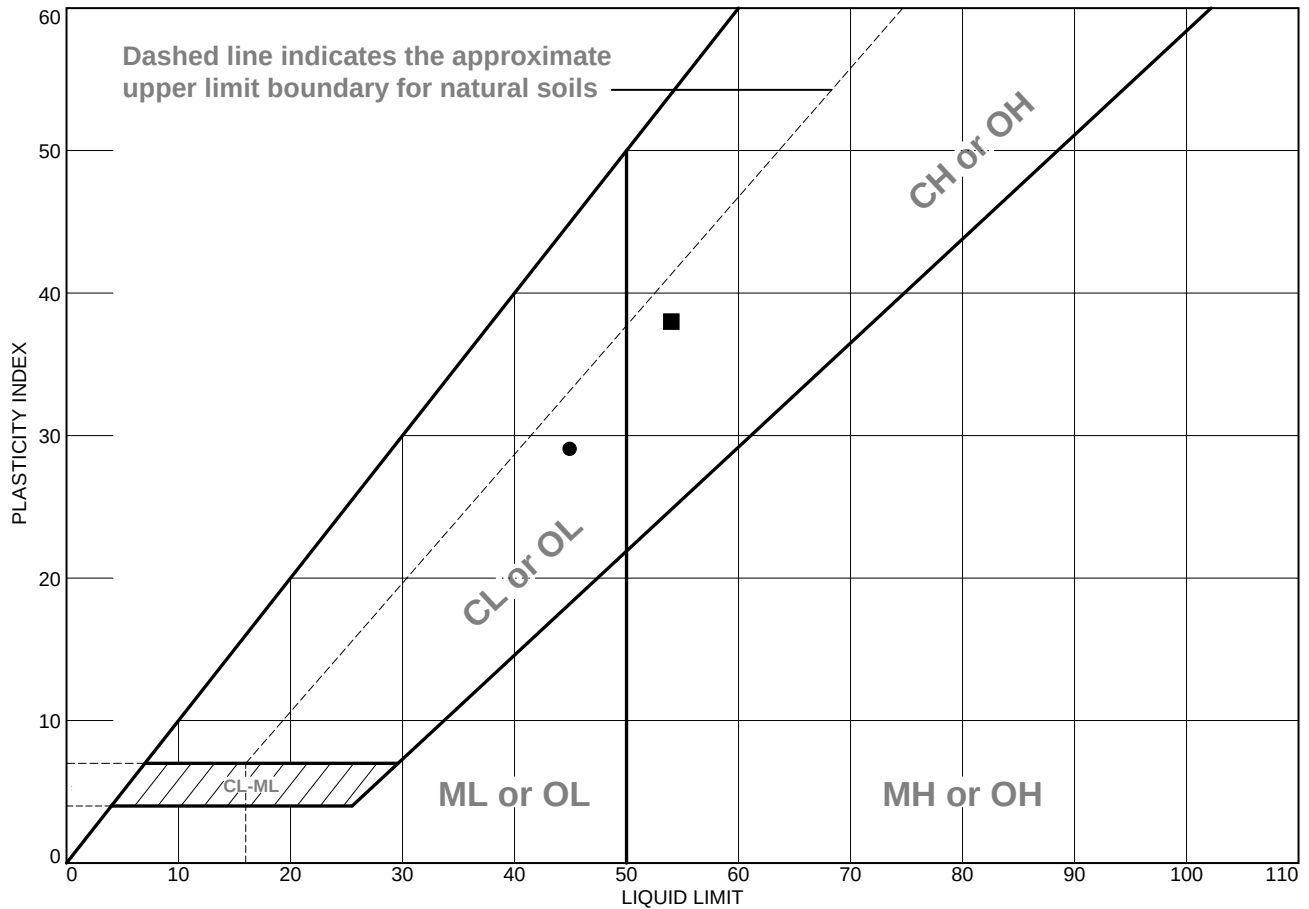
Cone Size 10cm squared

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

APPENDIX B
Laboratory Test Results



LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	45	16	29			
■	See exploration logs	54	16	38			

Project No. 12417.000.000 **Client:** Dutra Land & Consulting Services, Inc.

Project: 25906 Gading Road

● **Sample Number:** 1-CPT1

■ **Sample Number:** 1-CPT3

Remarks:

- ASTM D4318, Wet method
- ASTM D4318, Wet method



Tested By: G. Criste **Checked By:** D. Seibold

APPENDIX C
Liquefaction Analysis



LIQUEFACTION ANALYSIS REPORT

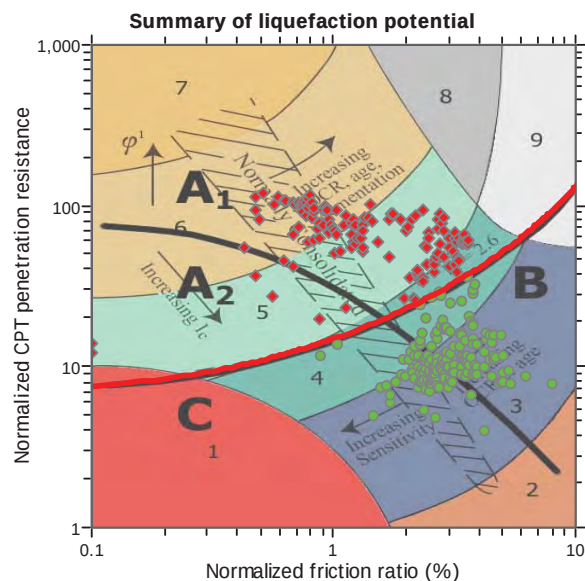
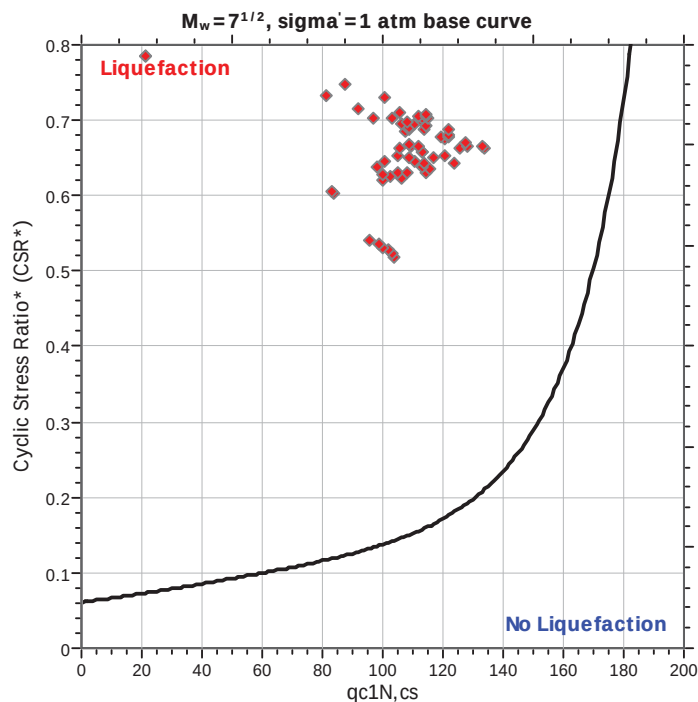
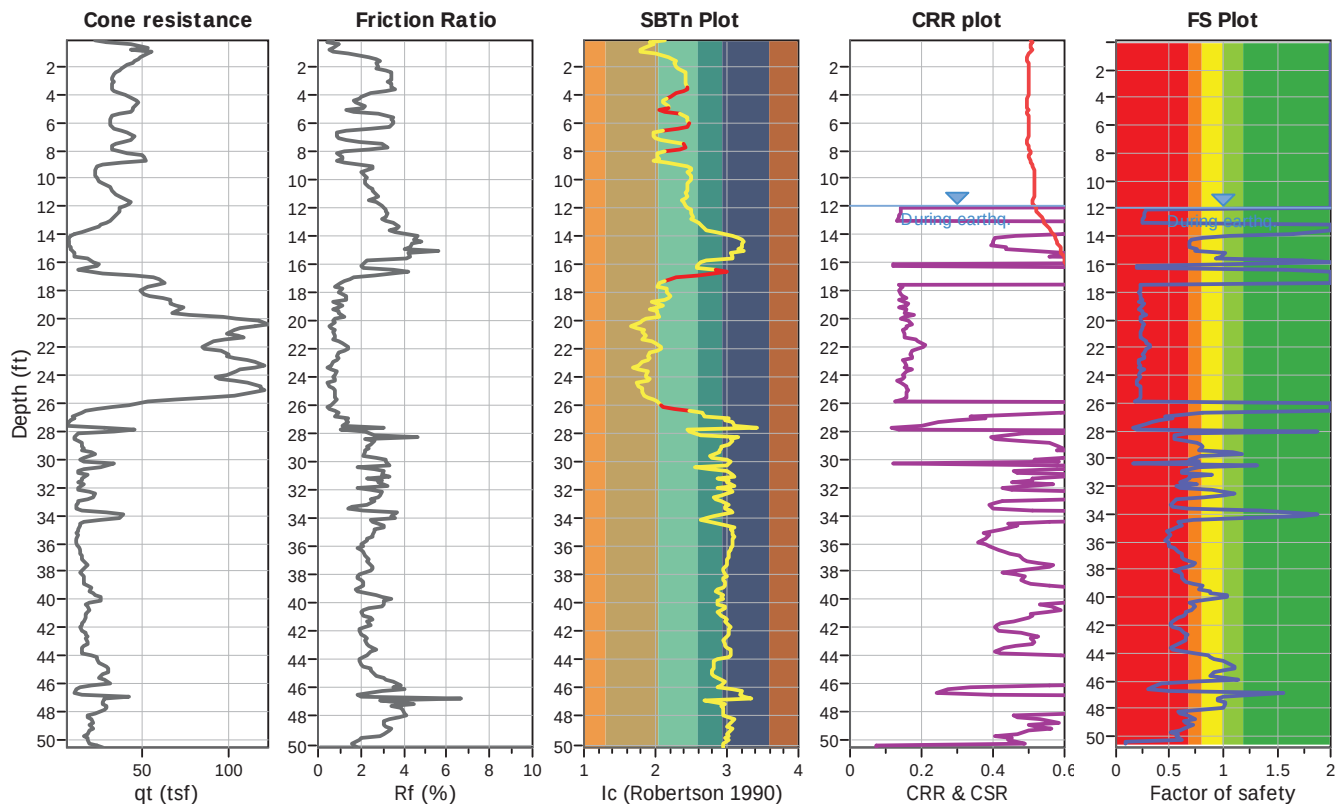
Project title :

Location :

CPT file : CPT-01

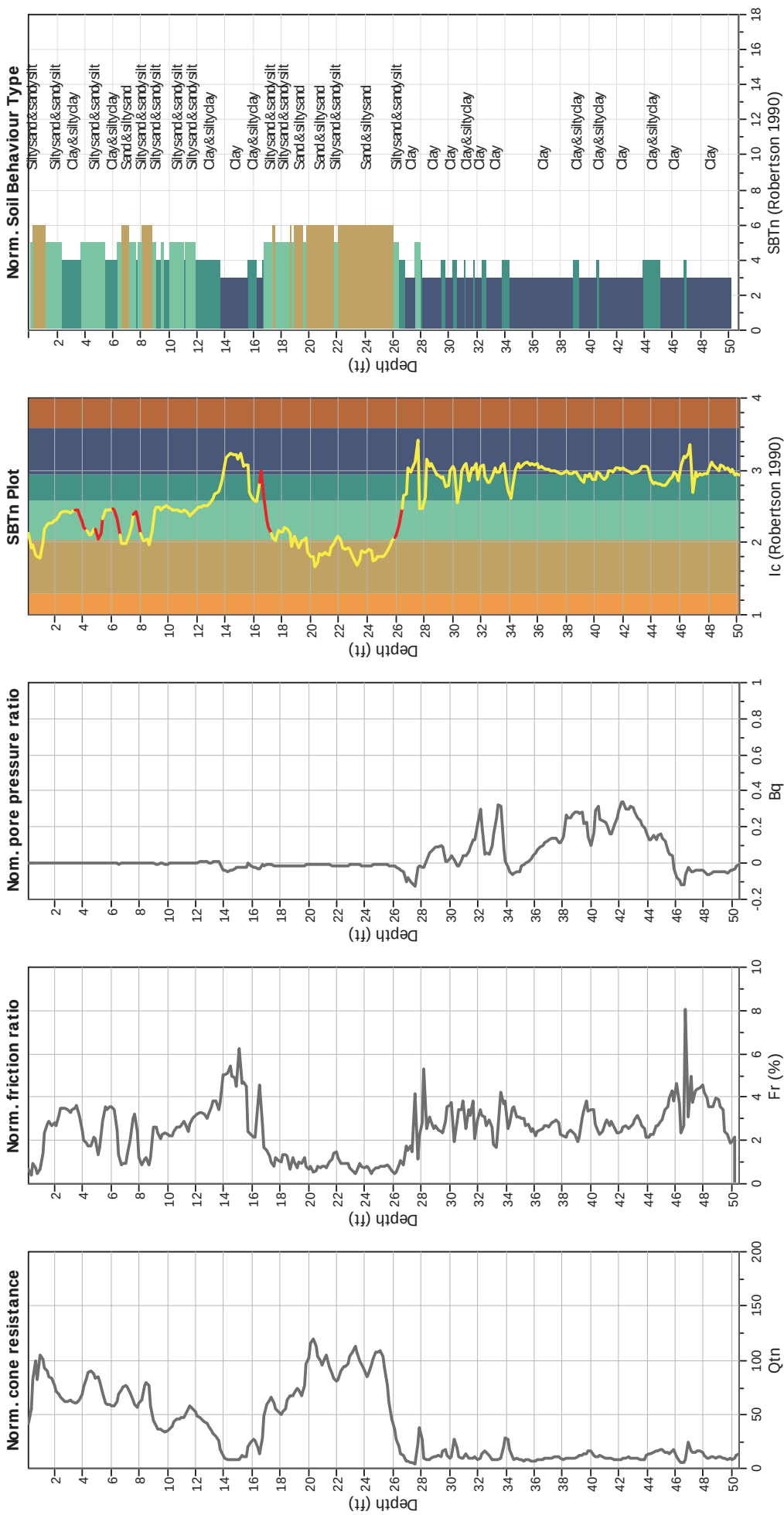
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	14.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	12.00 ft	Fill height:	N/A	applied:	Sand & Clay
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.87	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

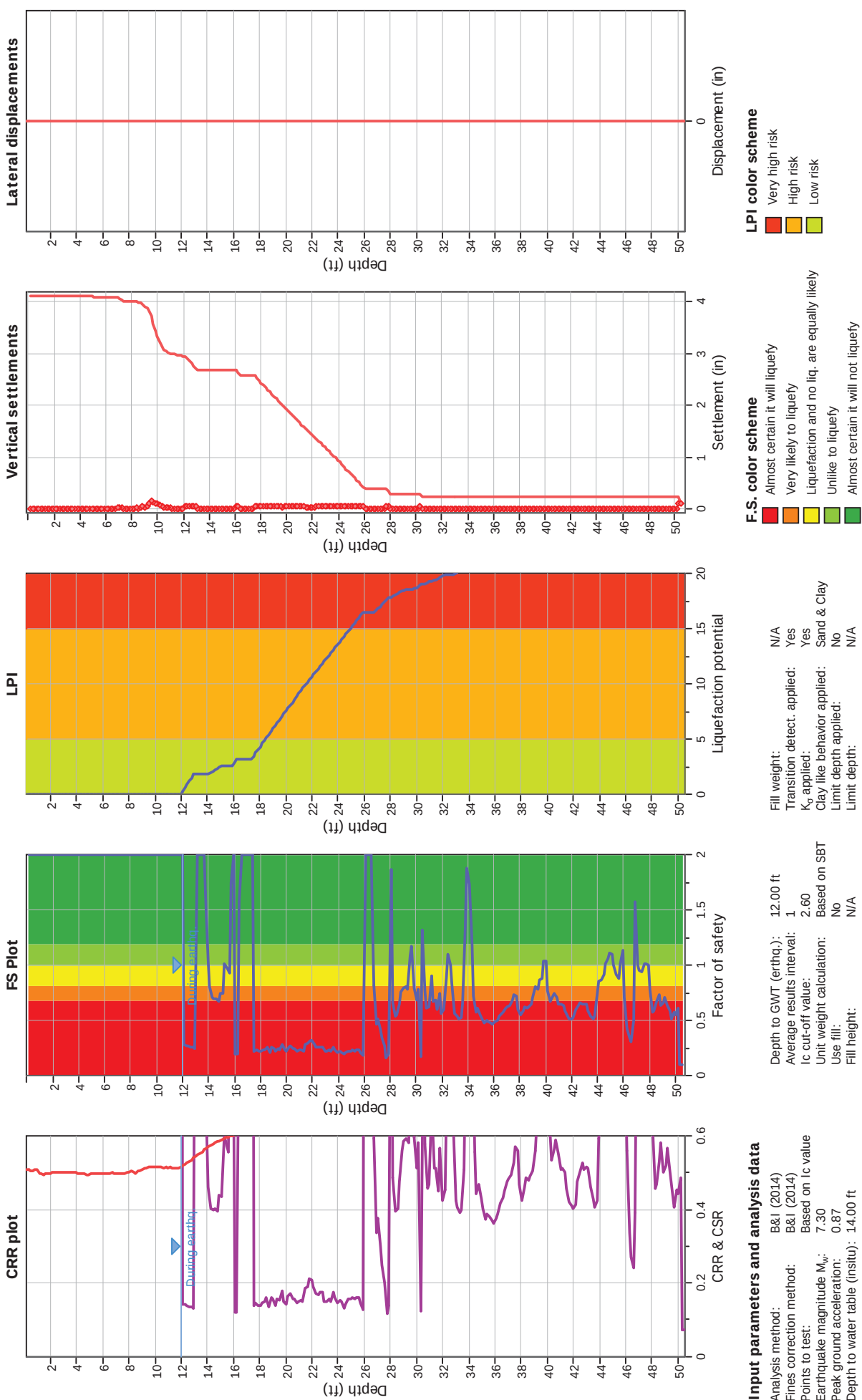
CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWL (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	1	Transition detect:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.87	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



LIQUEFACTION ANALYSIS REPORT

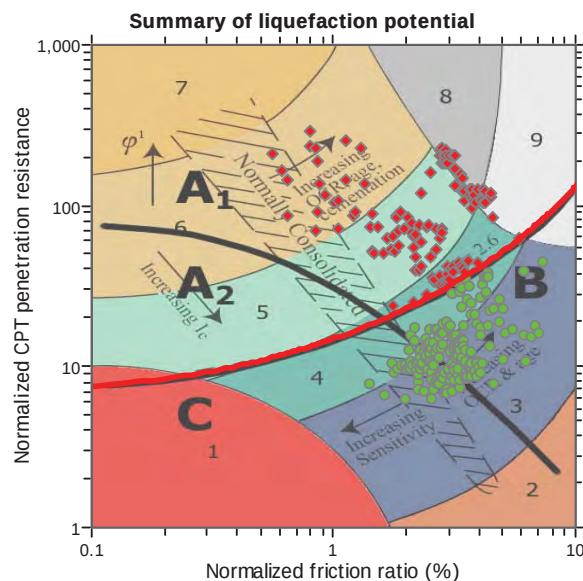
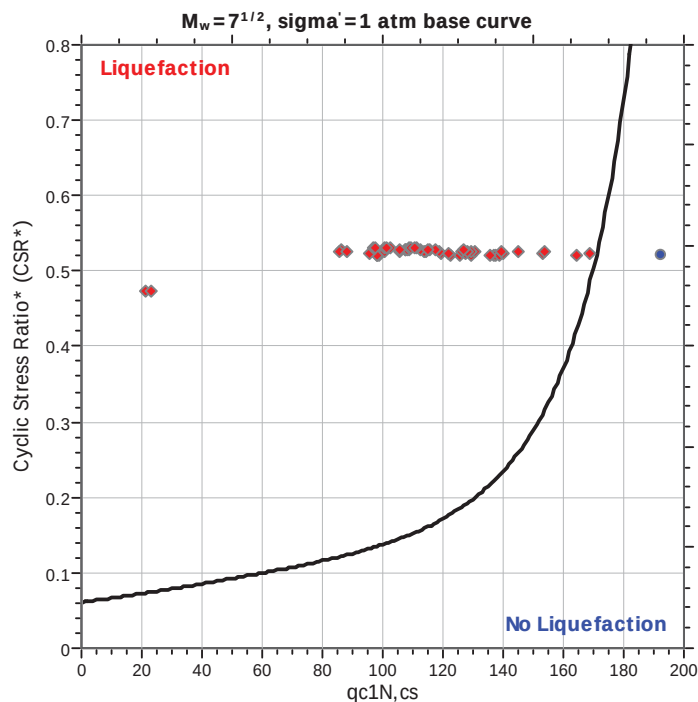
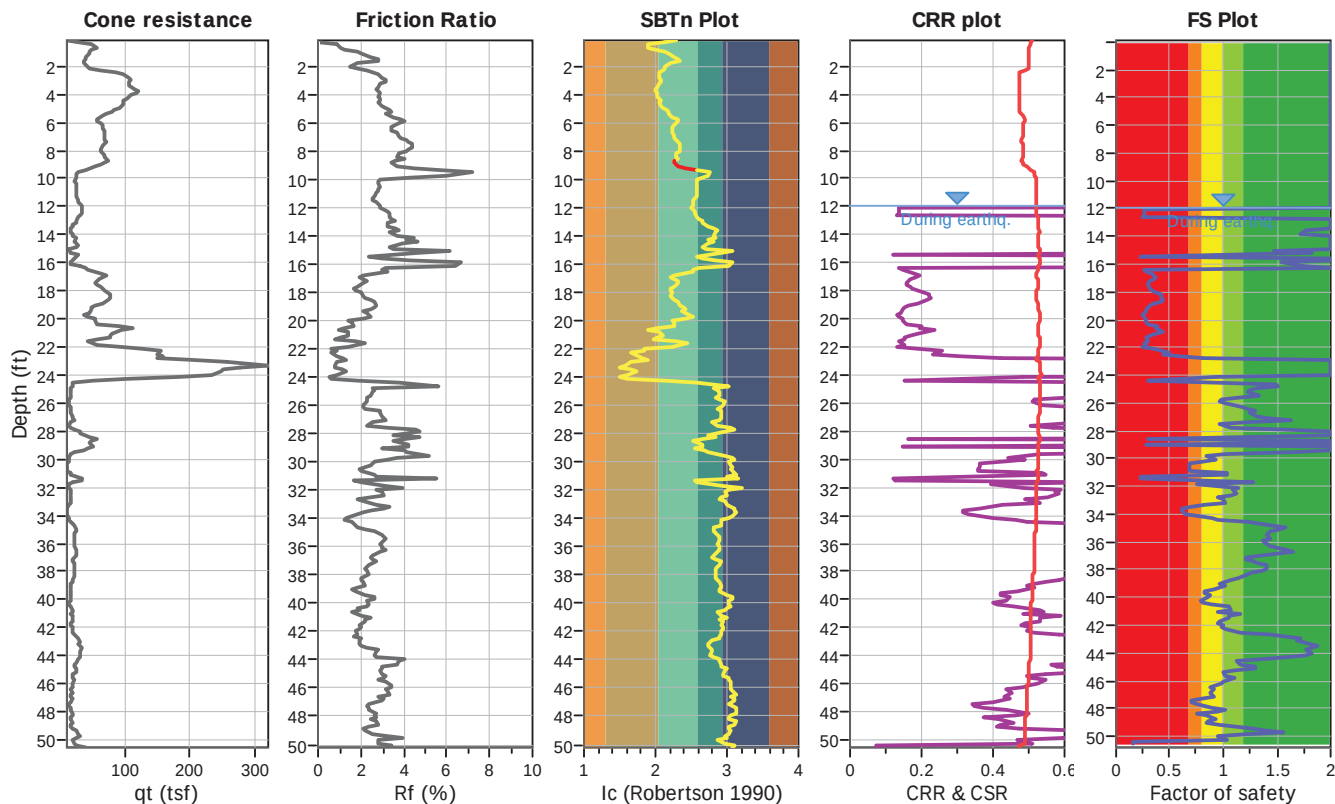
Project title :

Location :

CPT file : CPT-02

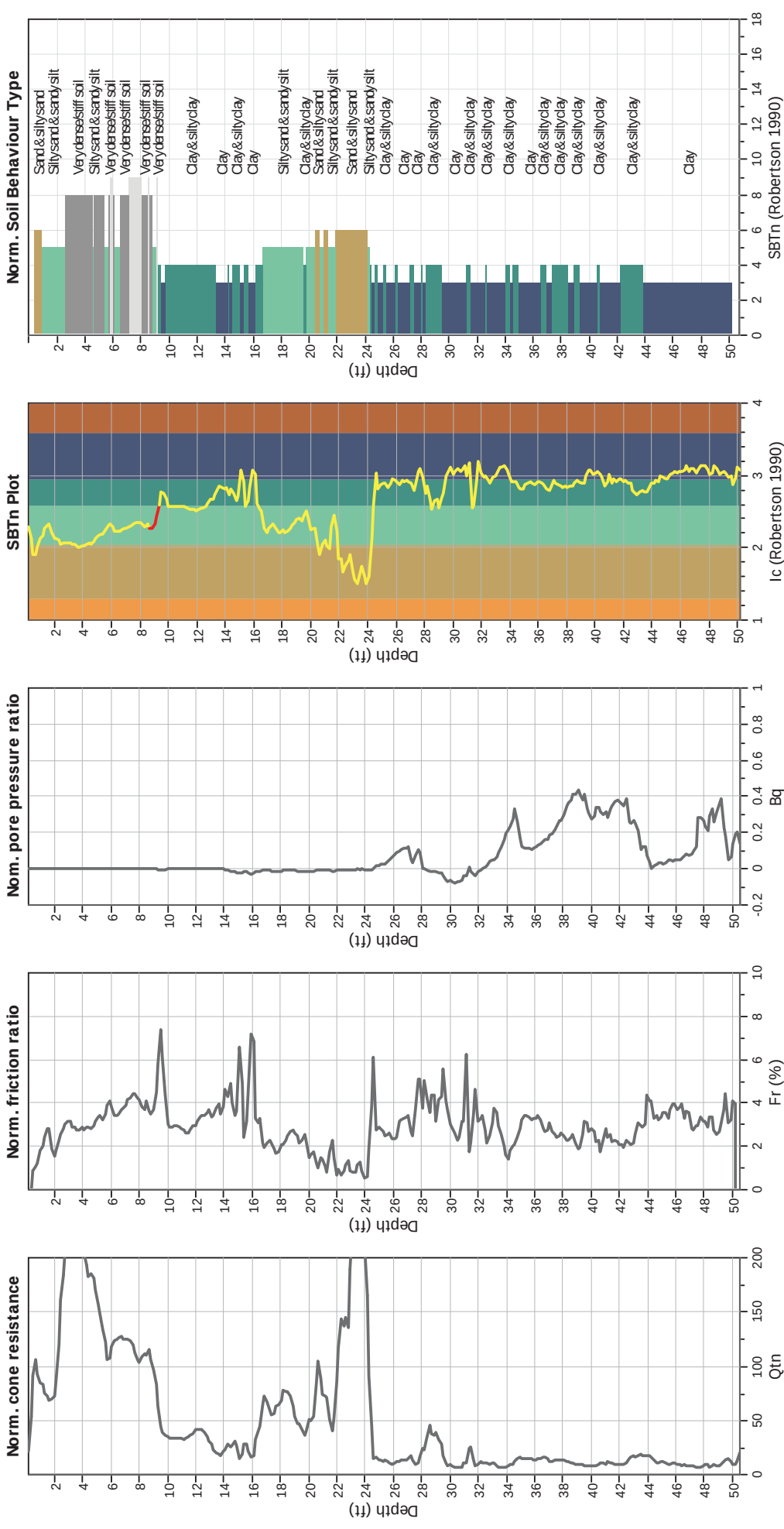
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	14.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	12.00 ft	Fill height:	N/A	applied:	Sand & Clay
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.87	Unit weight calculation:	Based on SBT	K_o applied:	Yes	MSF method:	Method based



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



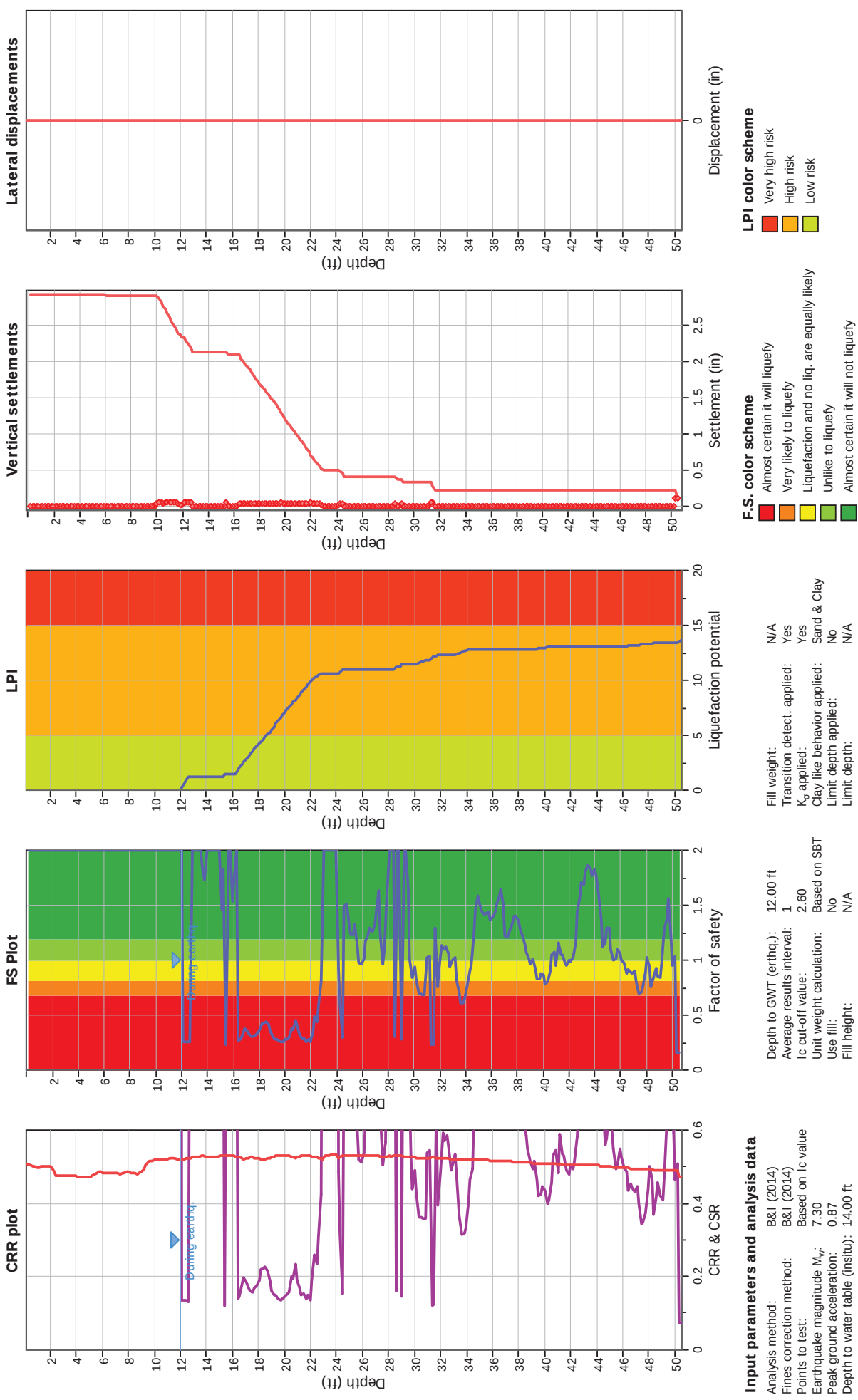
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWL (erthq.):	12.00 ft	Fill weight:	N/A
Finest correction method:	B&I (2014)	Average results interval:	1	Transition detect:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.87	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	14.00 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



LIQUEFACTION ANALYSIS REPORT

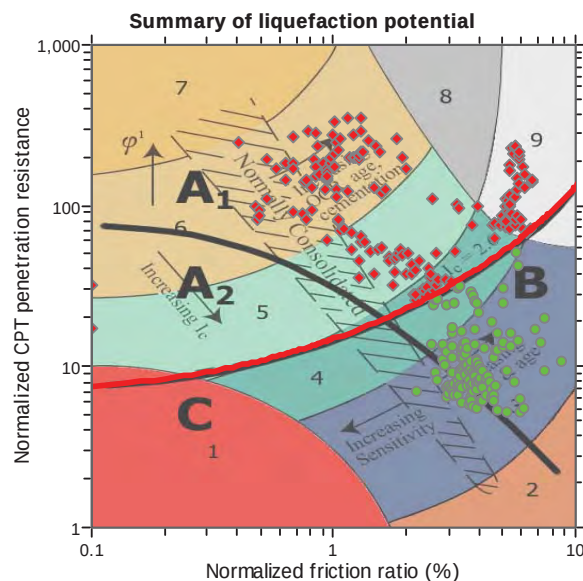
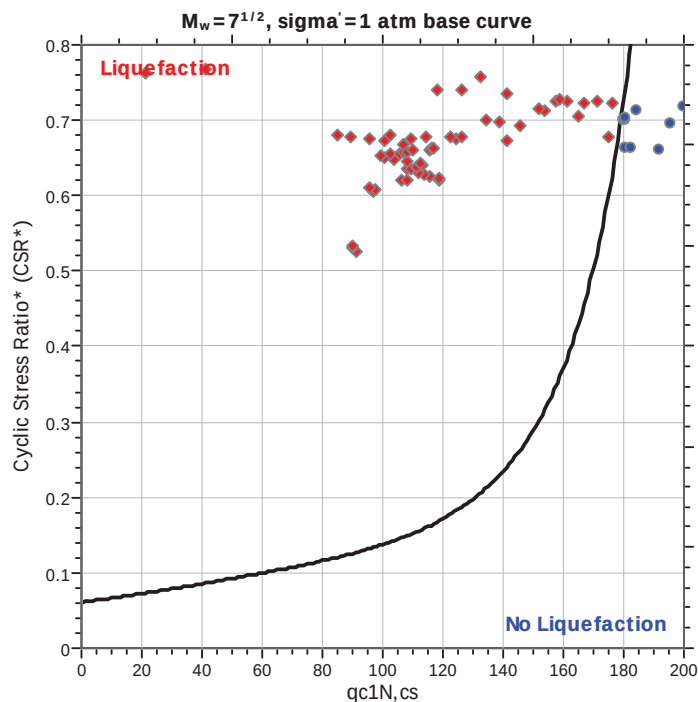
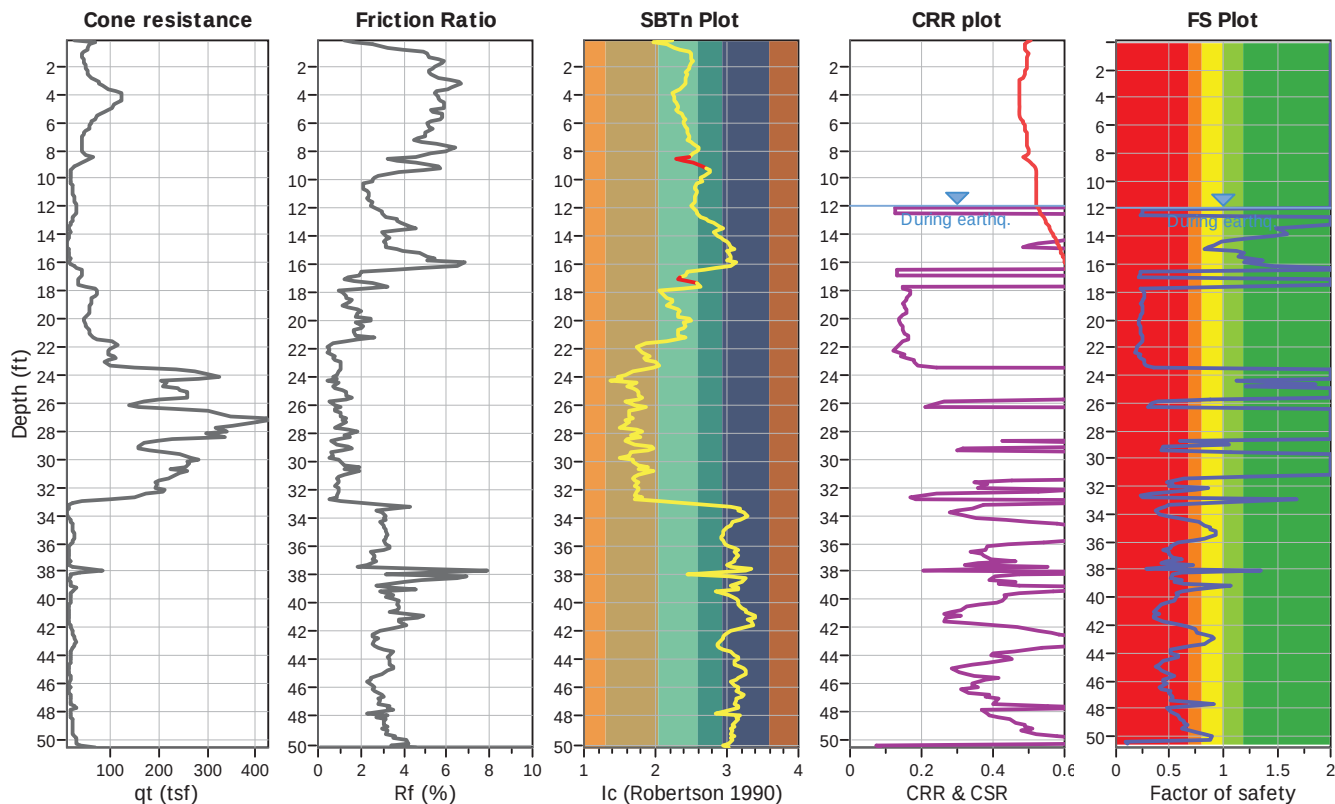
Project title :

Location :

CPT file : CPT-03

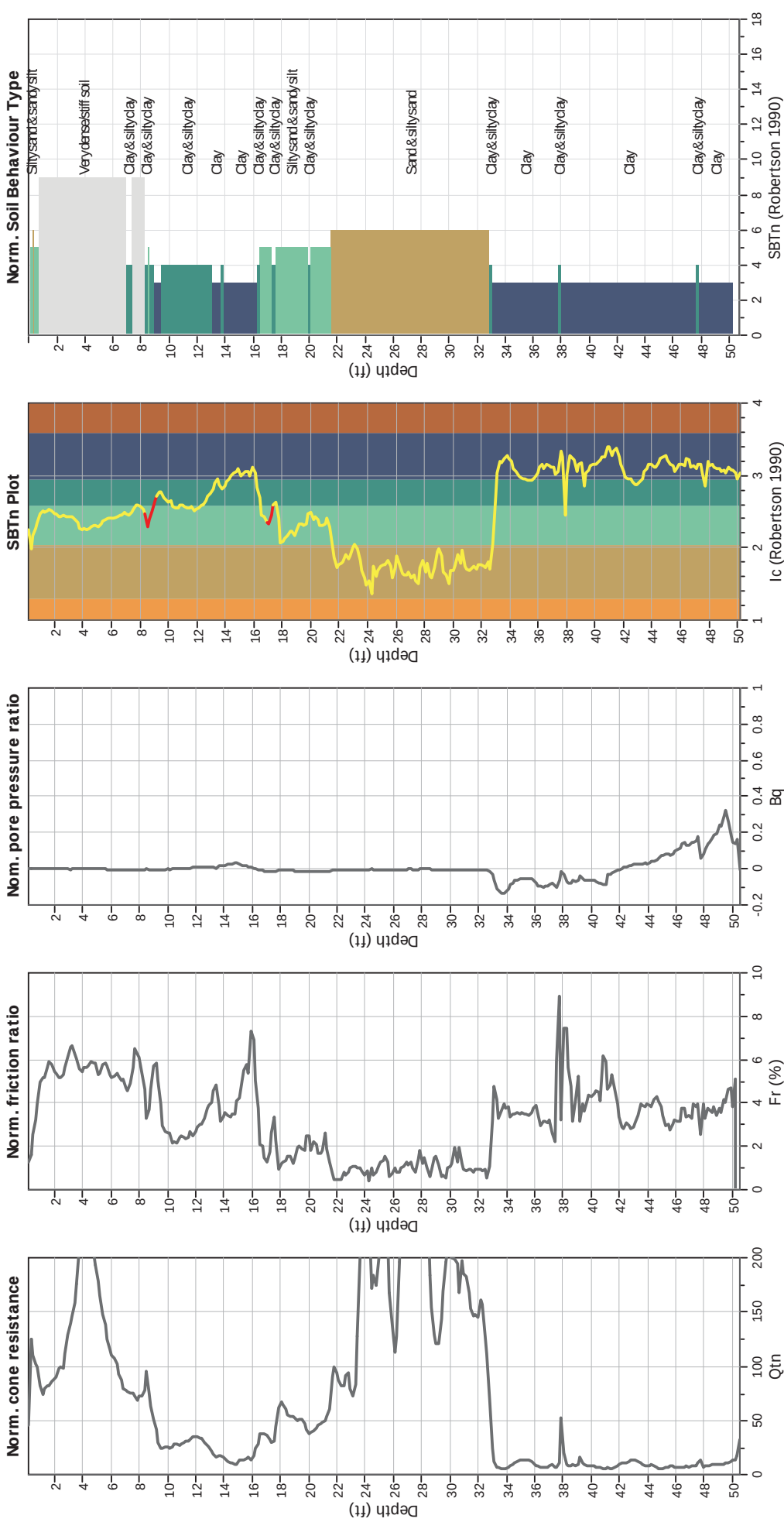
Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	17.70 ft	Use fill:	No	Clay like behavior	
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	12.00 ft	Fill height:	N/A	applied:	Sand & Clay
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.87	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	1	Transition detect:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sand & Clay
Peak ground acceleration:	0.87	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	17.70 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

