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Tanager Battery Energy Storage System Project

Preliminary Geotechnical Engineering Report

DISCLAIMER

This report is submitted to the California Energy Commission solely for informational purposes in connection with the Tanager Battery Energy Storage System ("BESS") Project (the "Project"). The raw, empirical data collected in connection with the preparation of this report, including but not limited to field measurements, laboratory analytical results, cone penetration tests ("CPTs"), and boring logs may be evaluated on their own merits by the CEC or any other party in any subsequent environmental review or CEQA proceeding, subject to the limitations noted in the report. However, the interpretive analyses, professional judgments, conclusions, impact assessments, and mitigation recommendations derived from the underlying data (the "Findings") set forth herein are based on a preliminary project design that does not reflect, and should not be construed as reflecting, the final design decisions for the Project. The project design remains subject to ongoing revision, refinement, and modification as engineering, environmental, and regulatory review proceeds. In other words, the Findings are preliminary in nature and are contingent upon design parameters, assumptions, and configurations that may differ from those ultimately adopted in the final project design. Submission of this report to the CEC does not constitute a waiver of any right to present updated analyses, revised impact assessments, or alternative mitigation recommendations based on the final project design in any subsequent environmental review document or administrative proceeding. Notwithstanding the foregoing, it is worth noting that the overarching conclusion from the report is that the site can be made

"suitable for the proposed BESS and substation based upon geotechnical conditions encountered in the test borings and CPTs".

Calpine Tanager Energy Storage Project

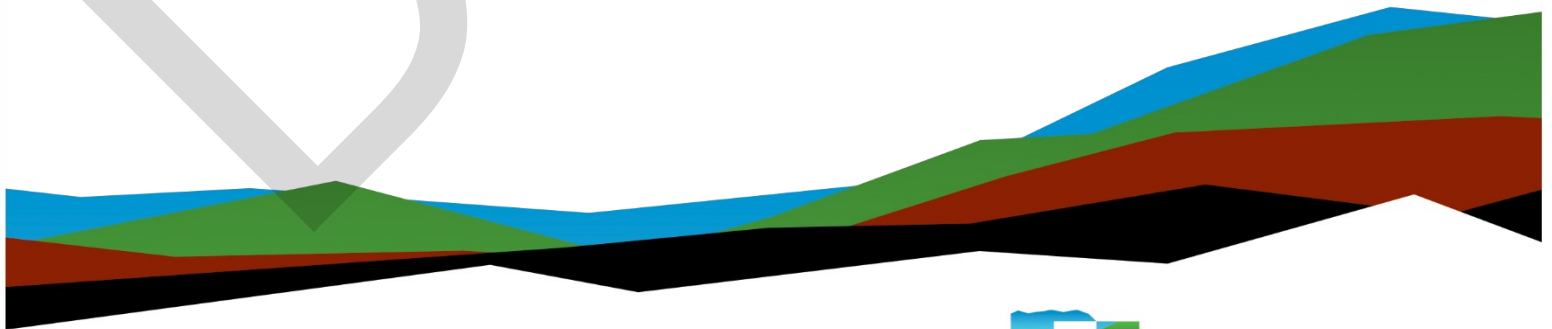
Preliminary Geotechnical Engineering Report

San Jose, Santa Clara County, California

August 29, 2025 | Terracon Project No. R1245076

Prepared for:

Calpine Corporation
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August 29, 2025

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Attn: Eli Helou
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Re: Preliminary Geotechnical Engineering Report
Calpine Tanager Energy Storage Project
800 Thomas Foon Chew Way
San Jose, Santa Clara County, California
Terracon Project No. R1245076

Dear Mr. Helou:

We have completed the scope of Preliminary Geotechnical Engineering services for the referenced project in general accordance with Terracon Proposal No. PR1245076 dated May 5, 2025. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations for the proposed energy storage project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,

Terracon

Ramsey S. Zeidan, P.E.
Senior Engineer



Robert Fosse, P.E., G.E.
Principal



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Note: This report was originally delivered in a web-based format. **Blue Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the  logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

Refer to each individual Attachment for a listing of contents.

Report Summary

Topic ¹	Overview Statement ²
Project Description	The project includes the development of a new battery energy storage (BESS) facility and collector substation which is expected to include battery containers, utility inverters, and auxiliary power distribution equipment. Site improvements that include a water tank, retention pond, and various access roads are also expected as part of the proposed development.
Geotechnical Characterization	Subgrade soil encountered in our test borings and cone penetration tests (CPTs) generally consisted of 6 to 9 feet of uncontrolled fill consisting of soft to very stiff lean clay and loose clayey sand underlain by soft to very stiff lean to fat clay to a depth of about 30 feet below the ground surface (bgs). The fill in the BESS area was generally mixed with granular soils and contained deleterious materials including plastic debris. Below the upper clay stratum, the borings and CPTs generally encountered interbedded loose to dense clayey sand and soft to stiff clay to the maximum depths explored of about 51½ feet bgs in the test borings and 100 feet in the CPTs. CPT-03 was terminated due to practical refusal conditions in very dense soils at a depth of about 84 feet bgs. Based on dissipation tests conducted in the CPTs, groundwater was measured at depths ranging from about 8 to 9½ feet bgs. Mapping indicates the historical high groundwater in the vicinity of the site is as shallow as 5 feet bgs. Groundwater was not observed and/or could not be measured in the test borings due to the drilling method used.
Earthwork	Uncontrolled fill should be removed and replaced with structural fill in accordance with the recommendations in Earthwork. The near surface fat clay is highly expansive and is not suitable for reuse as structural fill. Cuts and fills on the order of 2 feet or less are anticipated to develop final grades. Soils with high fines content are sensitive to moisture variation. Dewatering of excavations that extend below groundwater should be anticipated.
Shallow Foundations	Shallow foundation recommendations provided below only apply to substation location. Refer to BESS Foundation Options for

	more information regarding the use of shallow foundations in the BESS area. Allowable bearing pressure for Mat Foundations: 1,400 psf; for mat foundations up to 10 feet wide 1,000 psf; for mat foundations up to 14 feet wide 550 psf; for mat foundations up to 30 feet wide Allowable bearing pressure for Spread Footing Foundations: 1,250 psf for columns up to 15 feet wide Expected settlements: < 1-inch total, < 1/2-inch differential
Deep Foundations	Deep foundations are recommended for the site. Please refer to Deep Foundations for preliminary recommendations for support of the BESS and Substation structures. Allowable axial compression capacity graphs for drilled cast-in-place shafts are included in Supporting Information.
Below-Grade Structures	Cantilevered or gravity wall design can be designed using the resistance values in the Shallow Foundation section of the report. Earth pressures acting are provided in the Lateral Earth Pressure section. Fat clay soils should not be used as backfill.
Access Roads	With subgrade prepared as noted in Earthwork. 12 inches minimum aggregate surfacing.
General Comments	This section contains important information about the limitations of this geotechnical engineering report.

1. If the reader is reviewing this report as a pdf, the topics in the table can be used to access the appropriate section of the report by simply clicking on the topic itself.
2. This summary is for convenience only. It should be used in conjunction with the entire report for design purposes.

Introduction

This report presents the results of our subsurface exploration and Preliminary Geotechnical Engineering services performed for the proposed energy storage project located at 800 Thomas Foon Chew Way in San Jose, Santa Clara County, California. The purpose of these services was to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Seismic site classification per the 2022 California Building Code (CBC)

- Site preparation and earthwork
- Demolition considerations
- Foundation design and construction
- Pavement design and construction
- Stormwater detention considerations
- Liquefaction potential
- Corrosivity test results

The geotechnical engineering Scope of Services for this project included the advancement of test borings/CPTs, laboratory testing, engineering analysis, and preparation of this report.

Drawings showing the site and boring/CPT locations are shown on the [Site Location](#) and [Exploration Plan](#), respectively. The results of the laboratory testing performed on soil samples obtained from the site during our field exploration are included on the boring logs and/or as separate graphs in the [Exploration and Laboratory Results](#) section.

Project Description

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	An original email request for proposal was provided by Mr. Eli Helou of Calpine on July 10, 2024, with subsequent discussions with the project team through August 1, 2024. On April 9, 2025, Terracon was contacted to provide an updated proposal for a revised scope of work, as requested by Calpine. Documents provided to Terracon included: ■ Tanager Energy Storage Project, Bess and Substation Geotechnical Investigation Specification, No. T-6750, dated February 1, 2024, by Calpine. ■ Calpine 230 kV Tanager Gen Tie, Geotechnical Investigation Specification, dated July 3, 2024, by Power Engineers. ■ Plan of general arrangement, sheet G-100, dated June 26, 2024. ■ Plan of general arrangement (hithium layout), sheet G-100, dated March 25, 2025. ■ Google Earth KMZ files with requested boring locations at BESS and substation tie-in areas. During preparation of this report, the following previous geotechnical reports were provided by Eli Helou: ■ Geotechnical Investigation, North San Jose Energy Center, San Jose California, prepared by Lowney Associates, dated July 23, 2021. ■ Supplemental Geotechnical Recommendations Letter, prepared by Lowney Associates, dated August 1, 2001. ■ Geotechnical Investigation, Los Esteros Critical Energy Facility – Phase 2, prepared by TRC, dated April 25, 2011.
Project Description	The project includes the development of a new battery energy storage (BESS) facility and collector substation which is expected to include battery containers, utility inverters, and auxiliary power distribution equipment. Site improvements that include a water tank, retention pond, and various access roads are also expected as part of the proposed development.

Item	Description
Proposed Construction	Based on the information provided, we understand the proposed BESS facility will include arrays of batteries supported by either mat slab foundations, helical anchors or driven steel piles. The substation facility will include electrical structures including duct banks and large electrical equipment supported by either shallow spread footings and mat slabs, or drilled cast-in-place shaft deep foundation systems.
Maximum Loads	The following estimated structural loads were provided: ■ Each Battery System: 60 kips ■ Mat Slab Foundations: 100 to 300 kips ■ Electrical Structures: 1 to 10 kips axial; 1 to 10 kips shear; and 10 to 500 kip-ft overturning
Grading	A preliminary grading plan was not available for review at the time this report was prepared. Finished grades are expected to be within 2 feet of existing site grades. We anticipate that proposed grades for associated transmission and underground lines will follow existing ground levels with minimal earthwork. We assume cuts and fills on the order of about 2 feet or less will be required to achieve final grade in the BESS areas, excluding remedial grading requirements.
Below-Grade Structures	An underground vault structure is anticipated near the northeast corner of the substation tie-in area.
Free-Standing Retaining Walls	Not anticipated.
Access Roads	Access roads may be constructed as part of the BESS and substation development. We understand the access roads will consist of both gravel access roads. We assume post construction traffic with an allowable rut depth of 2 inches. We anticipate low-volume, aggregate-surfaced and native soil access roads will have primarily light maintenance vehicles. The substation access road should be able to support heavy vehicle deliveries (HS-20 loading) throughout the design life to replace electrical equipment.
Building Code	2022 California Building Code (CBC)

Terracon should be notified if any of this information is inconsistent with the planned construction, especially the grading limits, as modifications to our recommendations may be necessary.

Site Conditions

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The project is located at 800 Thomas Foon Chew Way in San Jose, Santa Clara County, California, immediately south of the Los Esteros Energy Center and substation. The site is bordered by Thomas Foon Chew Way to the west, Alviso Milpitas Road and Highway 237 to the south, and undeveloped land and Coyote Creek further to the east. APN: 01531072 (Approximately 33 acres in size) BESS area approximately 12 acres Substation area approximately 3 acres Latitude/Longitude (approximate center of BESS area) 37.4232° N, 121.9314° W Latitude/Longitude (approximate center of substation tie-in area) 37.4252° N, 121.9307° W See Site Location
Existing Improvements	The proposed BESS development area is currently undeveloped. The Los Esteros Energy Center and substation are located immediately to the north of the proposed BESS site.
Current Ground Cover	The BESS area currently consists of an unpaved, lightly to moderately vegetated lot with small to large shrubs and trees. The Los Esteros substation area is partially paved with concrete and asphalt pavement and aggregate surfacing.
Existing Topography	The BESS site is located in a gently sloping area ranging from about elevation +18 to +23 feet (NAVD 88). Coyote Creek is located roughly 500 feet south, and about 1,200 feet east of the subject site.

Geotechnical Characterization

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting, and our understanding of the project. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of the site. Conditions observed at each exploration point are indicated on the individual logs. The individual logs can be found in

the **Exploration and Laboratory Results** and the GeoModel can be found in the **Figures** attachment of this report.

As part of our analyses, we identified the following model layers within the subsurface profile. For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

Model Layer	Layer Name	General Description
1	Existing Fill	Soft to very stiff, Lean Clay with variable amounts of sand, gravel, and debris, and Loose Clayey Sand with Gravel
2	Lean Clay	Soft to stiff Lean Clay with variable amounts of sand and gravel
3	High Plasticity Clay	Soft to very stiff Fat Clay
4	Clayey Sand	Loose to Dense Clayey Sand with variable fines

Practical refusal was encountered in CPT-03 at a depth of about 84 feet bgs. Refusal is defined as the depth below the ground surface at which the CPT probe can no longer be advanced. Refusal is subjective and is based on the type of probing equipment used and the effort exerted by the CPT rig.

Additional borings, auger probes, test pits, or geophysical testing could be performed to obtain more specific subgrade information.

Groundwater Conditions

The borings were advanced in the dry using a drilling technique that allows for short term groundwater observations to be made while drilling. Rotary wash drilling techniques were also performed due to the presence of caving sands in the boreholes. Due to the low permeability of soils encountered in the borings, a relatively long period may be necessary for a groundwater level to develop and stabilize in a borehole; therefore, actual groundwater levels were difficult to evaluate at the time of drilling. Long term observations in piezometers or observation wells sealed from the influence of surface water are often required to define groundwater levels in materials of this type. Long-term groundwater monitoring was outside the scope of services for this project. Terracon is experienced in installing groundwater monitoring wells/piezometers to provide more groundwater data prior to construction if required.

Pore pressure dissipation tests were performed in the CPTs to help determine groundwater levels. The water levels observed in the CPTs can be found on the CPT logs in **Exploration and Laboratory Results** and are summarized in the following table.

Boring/CPT Number	Approximate Depth to Groundwater while Drilling/Testing ¹ (feet)
CPT-01	9.6
CPT-02	7.8
CPT-03	8.0
CPT-04	8.2

1. Below ground surface.

Groundwater conditions may be different at the time of construction. The Seismic Hazard Zone Report for the Milpitas 7.5-Minute Quadrangle¹ indicates the historic high groundwater level at the site is about 5 feet bgs. In addition, based on a local observation well located about 1,500 feet east, and an elevation difference of about 3 feet lower, of the subject site (Local Well Name: 06S01W12G005)² groundwater was measured at depths varying from about 5 to 8 feet bgs from September 2008 to May 2024.

Groundwater conditions may change because of seasonal variations in rainfall, runoff, tide, and other conditions not apparent at the time the borings and CPTs were performed. Therefore, groundwater levels during construction or at other times in the life of the proposed structures may be higher or lower than the levels indicated on the logs. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project. We recommend a groundwater level of 5 feet bgs be used for design and planning purposes.

¹ California Geological Survey. (2001). *Seismic Hazard Zone Report for the Milpitas 7.5-Minute Quadrangle, Alameda and Santa Clara Counties, California (SHZR 051)*. California Department of Conservation. <https://www.conservation.ca.gov/cgs>

² California Department of Water Resources. (n.d.). *Water Data Library Map Viewer*. Retrieved July 21, 2025, from <https://wdbeta.water.ca.gov/Map.aspx>

Dewatering should be considered and planned for in proposed excavations deeper than about 5 feet bgs. The depth of dewatering below the bottom of excavations should be determined by the contractor and/or designer. Pump tests for dewatering were not included in the scope of work for this report.

Geologic Setting

Geologic maps indicate subsurface conditions at the site consist of Holocene age alluvial fan deposits consisting of clay, sand, and gravel. Subgrade soils encountered in our borings and CPTs were generally consistent with mapped geology and our experience in the area.

Geologic Hazards

Faulting and Estimated Ground Motions

The site is located in the southern Bay Area, which is a relatively high seismicity region. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, the intensity, and the magnitude of the seismic event. The following table indicates the distance of the fault zones and the associated maximum credible earthquake that can be produced by nearby seismic events, as calculated using the USGS Earthquake Hazard Toolbox. Segments of the Hayward fault, which are located approximately 8 kilometers from the site, are considered to have the most significant effect on the site from a design standpoint.

Fault Name	Approximate Contribution (%)	Approximate Distance to Site (kilometers)	Maximum Credible Earthquake (MCE) Magnitude
Hayward (south) (1)	50.65	7.97	7.03
San Andreas (Peninsula) (2)	9.17	22.89	7.84

Based on the ASCE 7-16 Standard, the peak ground acceleration (PGA_M) at the subject site is approximately 0.806g. Based on the USGS 2023 interactive disaggregations, the PGA at the subject site for a 2% probability of exceedance in 50 years (return period of

2475 years) is expected to be about 0.988g. The site is not located within an Alquist-Priolo Earthquake Fault Zone based on our review of the State Fault Hazard Maps.¹

Liquefaction

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils or low plasticity fine grained soils exist below groundwater. The California Geological Survey (CGS) has designated certain areas within California as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table. The project site is located within a mapped CGS liquefaction hazard zone. Therefore, a liquefaction evaluation was performed to estimate the potential for liquefaction induced settlement. The results of our liquefaction evaluation are summarized in **Liquefaction Evaluation**.

Flooding

Based on a review of the Federal Emergency Management Agency (FEMA) National Flood Layer (NFHL), the project site is located within a mapped flood zone (Zone X) with 0.2 percent annual change flood hazard.

Seismic Considerations

The 2022 California Building Code (CBC) Seismic Design Parameters have been generated using the ASCE Hazard Tool. This web-based software application calculates seismic design parameters in accordance with ASCE 7-16, and 2022 CBC. The 2022 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped S_1 value greater than or equal 0.2.

However, an exception to this requirement is included in Supplement 3 to ASCE 7 that states: "A ground motion hazard analysis is not required where the value of the

¹ California Geological Survey. *California Earthquake Hazards Zone Application (EQ Zapp)*. Updated November 21, 2024. California Department of Conservation. <https://www.conservation.ca.gov/cgs/geohazards/eq-zapp>

parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in this standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in the Standard.”

Based on this exception, the spectral response accelerations presented in the following table were calculated using the site coefficients (F_a and F_v) from Tables 1613.2.3(1) and 1613.2.3(2) presented in Section 16.4.4 of the 2022 CBC.

Description	Value
2022 California Building Code (CBC) Site Classification ¹	F ²
Risk Category	III
Site Latitude ³	37.4232°
Site Longitude ³	-121.9314°
S_s , Spectral Acceleration for a Short Period ⁴	1.741
S_1 , Spectral Acceleration for a 1-Second Period ⁴	0.661
F_a , Site Coefficient	1.0
F_v , Site Coefficient (1-Second Period)	1.7
S_{DS} , Spectral Acceleration for a Short Period	1.161
S_{D1} , Spectral Acceleration for a 1-Second Period	1.124 ⁶

1. Seismic site soil classification in general accordance with the 2022 California Building Code, which refers to ASCE 7-16. Site Classification is required to determine the Seismic Design Category for a structure.
2. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of shear wave velocity in accordance with Section 20.4 of ASCE 7-16 and the 2022 CBC. CPTs were extended to a maximum depth of approximately 100 feet bgs at the site and shear wave velocities were measured at approximately 5-foot intervals. The weighted average shear wave velocity estimated based on the data was estimated to be about 796 ft/s.
3. Provided coordinates represent a point located at the general center of the site.
4. These values were obtained using online seismic design maps and tools provided by the ASCE Hazard Tool.
5. This site qualifies as a Site Class F due to the presence of liquefiable soils. Site Class D was used to develop the listed seismic design parameters due to the shear wave velocity measurements obtained from the CPTs. Based on the

Description	Value
exception for liquefiable soils provided in ASCE 7-16 Section 20.3.1, structures may use the listed design parameters provided they have a period of 0.5s or less. Please note that this exception only applies to the seismic design parameters; other code requirements for Site Class F still apply to this site. Should the anticipated structure have a period greater than 0.5s, a site-specific ground motion analysis is required to develop seismic design parameters. Terracon is qualified to perform such an analysis.	
6. Includes 50% increase in accordance with the exception in ASCE 7-16, Supplement 3 Section 11.4.8.1.	

Typically, a site-specific ground motion study may reduce construction costs. We recommend consulting with a structural engineer to evaluate the need for such a study and its potential impact on construction costs. Terracon should be contacted if a site-specific ground motion study is desired.

Liquefaction Evaluation

We performed a liquefaction hazard evaluation in general compliance with the California Geological Survey (CGS) Special Publication 117A (2008); Southern California Earthquake Center "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California," 1999 report; and the Seismic Hazard Zone Report for the Milpitas 7.5-Minute Quadrangle, Contra Costa County, California (SHZR 051).

As recommended in these reports, we performed a screening analysis to determine if there is potential for liquefaction to occur at the site. We evaluated the soils encountered in our borings and CPTs advanced to a maximum depth of 100 feet below the existing ground surface (bgs). We evaluated these soils based on soil classification, corrected SPT blow counts, water content, Atterberg limits, groundwater elevation, shear strength, peak ground acceleration, and CPT data. In our screening investigation we evaluated the Atterberg limits for cohesive soils in the upper 50 feet of our soil borings. The Atterberg limits for these cohesive soils exhibited liquid limits ranging from 29 to 61 percent and plasticity indices ranging from 12 to 41. We also calculated the ratio of the in-situ moisture content to the liquid limit. This data was then compared to the criteria by Idriss and Boulanger (2006) and Bray and Sancio (2006) for potential liquefaction or cyclic softening of fine-grained soils. The cohesive soils classify as "clay-like" and non-liquefiable by Idriss/Boulanger. Additionally, due to the in-situ moisture content to the liquid limit being less than 85 percent in the cohesive soils, we believe the cohesive soils have a low susceptibility to liquefaction. Cohesionless soils were present in the soil profile and have the potential for liquefaction. As a result, we performed a quantitative

evaluation of the potential for liquefaction to occur considering the cohesionless soils and the effects if liquefaction were to occur on this project.

We reviewed the shear wave velocities measured during advancement of our CPTs. The shear wave velocities measured in the upper 50 feet of the CPTs generally varied from 330 feet per second (ft/s) to 995 ft/s. Based research by Andrus et al. (2009), it has been accepted that soils with shear wave velocities greater than 720 ft/s are generally not susceptible to liquefaction.

A Peak Ground Acceleration (PGA) of 0.81g and an earthquake magnitude of 7.0 for the project site was used in our evaluation. Seismic Hazard Zone Report 051 for the Milpitas Quadrangle indicates the historical high groundwater depth for this site is about 5 feet bgs. Nearby monitoring wells have also recorded groundwater depths as shallow as 5 feet. As a result, a groundwater depth of 5 feet was utilized in our evaluation.

The liquefaction study utilized the software "CLiq" by GeoLogismiki Geotechnical Software. Our analysis was based on the soil data obtained from the CPT soundings supplemented by laboratory data performed on samples obtained from our borings. CPT calculations were assessed using the Robertson (NCEER 2001) method. A factor of safety of 1.3 was used against liquefaction.

Based on our review of the calculations, the anticipated potential total liquefaction-induced settlement across the site is estimated to be on the order of about $\frac{1}{4}$ to $\frac{3}{4}$ inches. The soil layers contributing to the majority of potential liquefaction settlement were encountered between depths of approximately 28 and 50 feet bgs. We estimate the differential liquefaction-induced settlement may be about $\frac{1}{2}$ -inch over a span of about 50 feet. Due to the thickness of the upper clay layer across the site, we believe the probability for liquefaction to manifest at the surface is low.

With regards to lateral spreading, we note that the site and surrounding area is relatively level. The western bank of Coyote Creek is approximately 1,200 feet from the eastern edge of the project site at its closest point, and based on the existing conditions and ground surface elevations the base of Coyote Creek is about 7 to 8 feet deep. Therefore, we estimate that the potential for lateral spread occurring at the site is low. Summary results of our liquefaction evaluation are included in [Supporting Information](#).

The proposed BESS and substation structures should be designed to accommodate the potential liquefaction induced settlement. If the proposed structures cannot be designed to accommodate the estimated liquefaction settlement or if a higher degree of performance is warranted, we recommend that [Ground Improvement](#) be implemented to mitigate the liquefaction potential at the site. For planning purposes, [Ground Improvement](#) should extend to a depth of at least 40 feet below existing site grades.

Corrosivity

The following table lists the results of laboratory soluble sulfate, soluble chloride, electrical resistivity, and pH testing. The values may be used to estimate potential corrosive characteristics of the on-site soils with respect to contact with the various underground materials which will be used for project construction.

Corrosivity Test Results Summary

Boring	Sample Depth (feet)	Soil Description	Soluble Sulfate (%)	Soluble Chloride (%)	Electrical Resistivity (Ω -cm)	pH
B1	0 – 5	Lean Clay (CL)	0.012	0.003	1,400	8.10

Results of soluble sulfate testing can be classified in accordance with ACI 318 – Building Code Requirements for Structural Concrete. Numerous sources are available to characterize corrosion potential to buried metals using the parameters presented in the previous table. ANSI/AWWA is commonly used for ductile iron, while threshold values for evaluating the effect on steel can be specific to the buried feature (e.g., piling, culverts, welded wire reinforcement, etc.) or agency for which the work is performed. Imported fill materials may have significantly different properties than the site materials noted in the table and should be evaluated if expected to be in contact with metals used for construction. Consultation with a NACE certified corrosion professional is recommended for buried metals on the site.

Field Electrical Resistivity

In-situ electrical resistivity testing was performed during our field exploration in general accordance with ASTM G57, utilizing the Wenner Four-Electrode Method. The testing consisted of three arrays oriented N-S and E-W with a common midpoint. Within the test array, potential electrodes are created on a transverse line between the current electrodes. An equal “A” spacing between electrodes is maintained. Individual in-situ electrical resistivity values at various “A” spacings along each array are summarized in the table provided in the results. Field Electrical Resistivity Test Data Sheets (Project No. NS245159, Dated June 12, 2025) presenting the field test results and approximate test locations are presented in the [Exploration and Laboratory Results](#) section of this report.

Thermal Resistivity

We understand that new underground utilities will be installed as part of the proposed project. In accordance with IEEE Standard 442-2017, thermal resistivity testing was performed on bulk subgrade samples collected from Borings B2 and B4 from 0 to 5 feet bgs, and on Tube samples obtained at depths of 5, 10, and 15 feet bgs, as requested by Calpine. The samples were tested at 90 percent of the maximum dry density using ASTM D698 (Standard Proctor), as requested. A dry-out curve was performed for each thermal resistivity test. The test results are presented in the [Exploration and Laboratory Results](#) section of this report.

Geotechnical Overview

The site appears suitable for the proposed BESS and substation based upon geotechnical conditions encountered in the test borings and CPTs, provided that the recommendations provided in this report are implemented in the design and construction phases of the project.

The subject site has geotechnical considerations that will affect the construction and performance of the proposed improvements that are discussed in this report. The primary geotechnical considerations that have been identified at the subject site that will affect development are the following:

- Soft/Unstable subgrade potential
- Liquefaction settlement
- Expansive soils
- Pre-Existing fill
- Consolidation settlement

Soft/Unstable Subgrade Potential

Soft to medium stiff clay was noted near the surface in our soil borings at the BESS and substation locations. These soils may become unstable when disturbed. During periods of dry weather, these soils may be stable upon initial exposure; however, these soils could become relatively soft and unstable under construction traffic. Furthermore, depending upon site conditions during construction, over-excavation, or stabilization of the subgrade and/or base of over-excavations may be needed to achieve a suitable working surface. Accordingly, we recommend that the owner budget for the possibility that over-excavation and/or stabilization may be required, and contractors be prepared to handle potentially unstable and/or soft conditions. Stabilization may consist of aerating/drying the soil, a composite section of aggregate base and geogrid, or chemical stabilization using high calcium quick lime or Portland cement.

Liquefaction Settlement

The primary seismic hazards for the site are the potential for strong to very strong earthquake shaking within the lifetime of the structures, and the potential for liquefaction to occur at the site. Seismically induced settlements of up to $\frac{3}{4}$ -inch should be expected in the event of liquefaction during the design earthquake. However, potentially liquefiable soils are not continuous or of uniform thickness across the site and ground settlements are expected to be irregular around and below the improvements.

Expansive Soils

Expansive soils are present on this site. This report provides recommendations to help mitigate the effects of soil shrinkage and expansion. However, even if these procedures are followed, some movement and (at least minor) cracking in structures should be anticipated. The severity of cracking and other damage such as uneven slabs will probably increase if modification of the site results in excessive wetting or drying of the expansive soils. Eliminating the risk of movement and distress may not be feasible, but it may be possible to further reduce the risk of movement if significantly more expensive measures are used during construction. Some of these options are discussed in this report such as complete replacement of expansive soils, using a structural slab, or supporting the improvements on **Deep Foundations**.

The near surface, soft to medium stiff lean clay and high plasticity fat clay could become unstable with typical earthwork and construction traffic, especially after precipitation events. The effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. If possible, the grading should be performed during the warmer and drier times of the year. If grading is performed during the winter months, an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

The soils which form the bearing stratum for shallow foundations are plastic and exhibit potential for shrink-swell movements with changes in moisture. Additional areas of localized highly plastic soils are likely present where borings were not performed. Maintaining above optimum moisture conditions in the bearing soils and a minimum dead load pressure on footings should reduce the anticipated swell movements to tolerable levels.

Pre-Existing Fill

Approximately up to 9 feet of uncontrolled pre-existing fill was encountered in our borings at the BESS site, and up to 6½ feet of fill at the substation site. The fill generally

consisted of lean clay with variable amounts of coarse-grained soil and plastic debris in Borings B1, B2 and B3 advanced in the proposed BESS area, and clayey sand with gravel and lean clay in Borings B4, B5, and B6 in the substation areas. Specific excavation, backfill, and site grading details for the fill are not known. The thickness of fill across the site may vary, and the thicknesses of fill encountered in our borings and CPTs should not be construed as a minimum or maximum. Additional fill may be present in areas where borings and/or CPTs were not performed.

No compaction records were located or made available for review for the fill. The density of the fill encountered was generally loose sands and soft to stiff lean clay indicating the fill may not have been placed with much compaction effort, if any. As a result, we have considered the fill to be undocumented and uncontrolled. Such uncontrolled fill can result in excessive erratic and differential settlements causing damage to proposed structures supported on shallow foundations, hardscapes, and pavements relying on the fill for structural support. Subsequently, the pre-existing surficial uncontrolled fills across the site are not suitable for the support of shallow spread footing foundations and slabs, which would be subject to unacceptable amounts of settlement.

Support of slabs and pavements on or above existing fill materials is discussed in this report. However, even with the recommended construction procedures, an inherent risk remains for the owner that compressible fill or unsuitable material, within or buried by the fill, will not be discovered. This risk of unforeseen conditions cannot be eliminated without completely removing the existing fill but can be reduced by following the recommendations contained in this report. To take advantage of the cost benefit of not removing the entire amount of undocumented fill, the owner must be willing to accept the risk of increased differential performance which can result in increased cracking and abrupt differential settlement.

Consolidation Settlement

The site is underlain by moderately compressible clay. CPT data and current laboratory testing indicated the clay layers are generally over-consolidated. New fill placed across the site may cause consolidation settlement of the underlying moderately compressible clay deposits.

We anticipate final site grades will generally be within 2 feet of existing site grade with 2 feet or less of new site grading fill being required to achieve final grade. For new site grading fills placed, we estimate on the order of ½-inch of additional site settlement (50-year) for each foot of new fill placed above existing site grades. A proposed grading plan was not available at the time this report was prepared. The project grading plan, once developed, should be provided to Terracon for review to help confirm the estimated consolidation settlement that may be anticipated due to proposed fill placement.

The site settlements should occur most rapidly immediately after site fills are placed and gradually the rates of settlement will decrease. The estimated settlements are based on the existing subgrade conditions encountered in our test borings and CPTs at the time they were performed and could be significantly higher in areas where borings were not performed. Further evaluation of site settlement due to proposed site grading should be considered during final design, when final site grading plans and construction schedule and sequencing are defined in more detail.

Differential settlements between adjacent ground surfaces and the improvements supported on **Shallow Foundations** over soils mitigated by **Ground Improvement**, or **Deep Foundations** should be anticipated. Special design details should be considered for underground utility lines; site development such as hardscape, entrances, and pavement adjacent to the improvements supported on **Shallow Foundations** over soils mitigated by **Ground Improvement** with or **Deep Foundations**; and site drainage. In areas not improved by **Ground Improvement** methods, the anticipated differential movement should be considered when planning development in these areas. It is recommended utilities and piping be designed with flexible connections and/or other means to accommodate such soil movement to reduce the potential for damage. Utility and drain lines designed for gravity flow should consider and account for anticipated settlements.

The recommendations contained in this report are based upon the results of field and laboratory testing (presented in the **Exploration and Laboratory Results**), engineering analyses, and our current understanding of the proposed project. The **General Comments** section provides an understanding of the report limitations.

Earthwork

We anticipate grading may consist of cuts and fills on the order of 2 feet or less and that site grades will remain at the same elevation as existing. Specific site grading information was unavailable at the time this report was prepared. If elevation and site grading differ from our stated assumptions, Terracon should be contacted to determine if additional earthwork recommendations are warranted, particularly with regard to potential ground settlement.

Earthwork is anticipated to include clearing and grubbing, excavations, and engineered fill placement. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for foundations, slabs, and pavements.

Site Preparation

Prior to placing fill, existing vegetation, topsoil, and root mats should be removed. Complete stripping of the topsoil should be performed in the proposed construction areas. Stripping should extend laterally a minimum of 5 feet beyond the limits of proposed improvements. If unexpected underground facilities are encountered, such features should be removed, and the excavation thoroughly cleaned prior to backfill placement and/or construction.

Existing Fill

As noted in **Geotechnical Characterization**, our borings encountered previously placed fill up to 9 feet thick in the BESS area and up to 6½ feet thick in the substation area. The density/stiffness of the fill encountered in our borings was loose/soft indicating the fill may not have been placed with much compaction effort, if any. We have no records to indicate the degree of control under which the fill was placed, and consequently, the fill is considered unreliable for support of foundation loads. Support of slabs and pavements on or above existing fill soils is discussed in this report. However, even with the recommended construction procedures, inherent risk exists for the owner that compressible fill or unsuitable material, within or buried by the fill will, not be discovered. This risk of unforeseen conditions cannot be eliminated without completely removing the existing fill but can be reduced by following the recommendations contained in this report.

If the owner elects to construct pavements and miscellaneous minor improvements on the existing fill, the following protocol should be followed. The fill below pavements and exterior slab areas should be over-excavated to a depth of 2 feet and the resulting subgrade should be scarified to a minimum depth of 12 inches, moisture conditioned, and compacted per the recommendations in **Fill Placement and Compaction Requirements**. Following compaction of the subgrade, the over-excavated areas may be backfilled with compacted structural fill. A representative from Terracon should be present to observe the over-excavation of fill. Once the planned subgrade elevation has been reached, the entire pavement area should be proofrolled. Areas of soft or otherwise unsuitable material should be undercut and replaced with either new structural fill or suitable, existing on-site materials.

Subgrade Preparation

After clearing, any required cuts and overexcavation should be made. The presence of a high volume of organic material due to any prior agricultural usage may warrant additional cuts or over-excavation at the time of grading operations.

Once cuts and over-excavation operations are complete, the resulting subgrade should be proofrolled with an adequately loaded vehicle such as a fully-loaded tandem-axle dump truck. The proofrolling should be performed under the observation of the Geotechnical Engineer or their representative. Areas excessively deflecting under the proofroll should be delineated and subsequently addressed by the Geotechnical Engineer. Such areas should either be removed or modified by stabilizing as noted in the **Soil Stabilization** section of this report. Excessively wet or dry material should either be removed, or moisture conditioned and recompacted.

Once proof rolling has been performed, and prior to placing any fill, the subgrade soils should be scarified and compacted. The depth of scarification of subgrade soils and moisture conditioning of the subgrade is highly dependent upon the time of year of construction and the site conditions that exist immediately prior to construction. If construction occurs during the winter or spring, when the subgrade soils are typically already in a moist condition, scarification and compaction may only be 12 inches. If construction occurs during the summer or fall when the subgrade soils have been allowed to dry out deeper, the depth of scarification and moisture conditioning may be as much as 18 inches or more. A representative from Terracon should be present during earthwork to observe the exposed subgrade and confirm the depth of scarification and moisture conditioning required.

Following scarification, moisture conditioning, and compaction of the subgrade soils, compacted structural fill soils should then be placed to the proposed design grade and the moisture content and compaction of subgrade soils should be maintained until foundation, slab, or access road construction.

Soft soil conditions could be encountered in the bottom of excavations. These soils may be unworkable. The contractor may utilize dry crushed rock or clean granular fill material placed over a geotextile such as Mirafi RS580i or equivalent to stabilize wet subgrade materials in the bottom of the excavations prior to backfill. If further soil stabilization is needed or another method is preferred or desired, Terracon should be consulted to evaluate the situation as needed.

Based upon the subsurface conditions determined from the geotechnical exploration, subgrade soils exposed during construction are anticipated to be relatively workable; however, the workability of the subgrade may be affected by precipitation, repetitive construction traffic or other factors. If unworkable conditions develop, workability may be improved by scarifying and drying.

Excavation

We anticipate that excavations for the proposed construction can be accomplished with conventional earthmoving equipment. The bottom of excavations should be thoroughly

cleaned of loose soils and disturbed materials prior to backfill placement and/or construction.

Soil Stabilization

Methods of subgrade stabilization, as described in this section, could include scarification, moisture conditioning and recompaction, removal of unstable materials and replacement with granular fill (with or without geosynthetics), and chemical stabilization. The appropriate method of improvement, if required, would be dependent on factors such as schedule, weather, the size of area to be stabilized, and the nature of the instability. More detailed recommendations can be provided during construction as the need for subgrade stabilization occurs. Performing site grading operations during warm seasons and dry periods would help reduce the amount of subgrade stabilization required.

If the exposed subgrade is unstable during proofrolling operations, it could be stabilized using one of the following methods:

- **Scarification and Recompaction** - It may be feasible to scarify, dry, and recompact the exposed soils. The success of this procedure would depend primarily upon favorable weather and sufficient time to dry the soils. Stable subgrades likely would not be achievable if the thickness of the unstable soil is greater than about 1 foot, if the unstable soil is at or near groundwater levels, or if construction is performed during a period of wet or cool weather when drying is difficult.
- **Aggregate Base** - The use of Caltrans Class II aggregate base is a common procedure to improve subgrade stability. Typical undercut depths would be expected to range from about 12 inches to 18 inches below finished subgrade elevation. The use of high modulus geosynthetics (i.e., engineering fabric or geogrid) could also be considered after underground work such as utility construction is completed. Prior to placing the fabric or geogrid, we recommend that all below grade construction, such as utility line installation, be completed to avoid damaging the fabric or geogrid. Equipment should not be operated above the fabric or geogrid until one full lift of aggregate base is placed above it. The maximum particle size of granular material placed over geotextile fabric or geogrid should meet the manufacturer's specifications.
- **Chemical Stabilization** - Improvement of subgrades with Portland cement or quicklime could be considered for improving unstable soils. Chemical stabilization should be performed by a pre-qualified contractor who has experience with successfully stabilizing subgrades in the project area on similar sized projects with similar soil conditions. The hazards of chemicals blowing across the site or onto adjacent property should also be considered. Additional testing would be needed to develop specific recommendations to improve subgrade stability by

blending chemicals with the site soils. Additional testing could include, but not be limited to, determining the most suitable stabilizing agent, the optimum amounts required, and the presence of sulfates in the soil. If this method is chosen to stabilize subgrade soils the actual amount of high calcium quicklime/Portland cement to be used should be determined by Terracon and by laboratory testing at least three weeks prior to the start of grading operations.

Further evaluation of the need and recommendations for subgrade stabilization can be provided during construction as the geotechnical conditions are exposed.

Fill Material Types

Fill required to achieve design grade should be classified as structural fill and general fill. Structural fill is material used below, or within 10 feet of structures and pavements. General fill is material used to achieve grade outside of these areas.

Reuse of On-Site Soil: Excavated on-site clay soil is not suitable for reuse as structural fill and should not be placed beneath settlement sensitive structures and within foundation bearing zones. Portions of the on-site soil have an elevated fines content and will be sensitive to moisture conditions (particularly during seasonally wet periods) and may not be suitable for reuse when above optimum moisture content.

Material property requirements for on-site soil for use as general fill and structural fill are noted in the following table:

Property	General Fill	Structural Fill
Composition	Free of deleterious material	Free of deleterious material
Maximum particle size	6 inches (or 2/3 of the lift thickness)	3 inches
Fines content	Not limited	Less than 40% Passing No. 200 sieve
Plasticity	Not limited	Maximum liquid limit of 30 Maximum plasticity index of 10
GeoModel Layer Expected to be Suitable ¹	1, 2, 4	4

1. Based on subsurface exploration. Actual material suitability should be determined in the field at time of construction.

Imported Fill Materials: Imported fill materials should meet the following material property requirements. Regardless of its source, compacted fill should consist of approved materials that are free of organic matter and debris. For all import material, the contractor shall submit current verified reports from a recognized analytical laboratory indicating that the import has a “not applicable” (Class S0) potential for sulfate attack based upon current ACI criteria and is “mildly corrosive” to ferrous metal and copper. The reports shall be accompanied by a written statement from the contractor that the laboratory test results are representative of all import material that will be brought to the project.

Soil Type ¹	USCS Classification	Acceptable Parameters (for Structural Fill)
Low Plasticity	CL, SC	Liquid Limit less than 30 Plasticity index less than 10
Granular ²	GM, GC, SW, SM	Less than 40% passing No. 200 sieve

1. Structural and general fill should consist of approved materials free of organic matter and debris and should contain no material larger than 3 inches and 6 inches in greatest dimension, respectively. A sample of each material type should be submitted to the Geotechnical Engineer for evaluation at least two weeks prior to use on this site. Additional geotechnical consultation should be provided prior to use of uniformly graded gravel on the site.
2. Caltrans Class II aggregate base may be used for this material. Recycled aggregate base should not be used without prior approval by the Geotechnical Engineer.

Fill Placement and Compaction Requirements

Compacted native soil and structural and general fill should meet the following compaction requirements.

Item	Structural Fill	General Fill
Maximum Lift Thickness	8 inches or less in loose thickness when heavy, self-propelled compaction equipment is used 4 to 6 inches in loose thickness when hand-guided equipment (i.e. jumping jack or plate compactor) is used	Same as structural fill

Item	Structural Fill	General Fill
Minimum Compaction Requirements ^{1,2}	95% of max. for structural fill below foundations and slabs, within 1 foot of finished pavement subgrade, for aggregate base and chemically treated soil, and for fills deeper than 5 feet 90% of max. for all other locations	90% of max.
Water Content Range ¹	Low plasticity cohesive: +1% to +3% above optimum Granular: -2% to +2% of optimum	As required to achieve min. compaction requirements

1. Maximum density and optimum water content as determined by the Modified Proctor test (ASTM D 1557).
2. If the granular material is a coarse sand or gravel, or of a uniform size, or has a low fines content, compaction comparison to relative density may be more appropriate. In this case, granular materials should be compacted to at least 70% relative density (ASTM D 4253 and D 4254). Materials not amenable to density testing should be placed and compacted to a stable condition observed full time by the Geotechnical Engineer or representative.

Utility Trench Backfill

Any soft or unsuitable materials encountered at the bottom of utility trench excavations should be removed and replaced with structural fill or bedding material in accordance with public works specifications for the utility to be supported. This recommendation is particularly applicable to utility work requiring grade control and/or in areas where subsequent grade raising could cause settlement in the subgrade supporting the utility. Trench excavation should not be conducted below a downward 1:1 projection from existing foundations without engineering review of shoring requirements and geotechnical observation during construction.

It is recommended utilities and piping be designed with flexible connections and/or other means to accommodate soil movement to preclude damage due to excessive settlement from consolidation. Utility and drain lines designed for gravity flow should consider steeper gradients to account for these settlements, especially where such lines enter improvements supported over soil mitigated by **Ground Improvement** or on **Deep Foundations**.

On-site materials are considered suitable for backfill of utility and pipe trenches from 1 foot above the top of the pipe to the final ground surface, provided the material is free of organic matter and deleterious substances. Where trenches are placed beneath slabs

or footings, the backfill should satisfy the gradation and Atterberg limit requirements for structural fill discussed in this report.

Trench backfill should be mechanically placed and compacted as discussed earlier in this report. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. Flooding or jetting for placement and compaction of backfill is not recommended.

All trench excavations should be made with sufficient working space to permit construction including backfill placement and compaction. If utility trenches are backfilled with relatively clean granular material, they should be capped with at least 18 inches of cementitious flowable fill or cohesive fill in non-pavement areas to reduce the infiltration and conveyance of surface water through the trench backfill. Attempts should also be made to limit the amount of fines migration into the clean granular material. Fines migration into clean granular fill may result in unanticipated localized settlements over a period of time. To help limit the amount of fines migration, Terracon recommends the use of a geotextile fabric that is designed to prevent fines migration in areas of contact between clean granular material and fine-grained soils. Terracon also recommends that clean granular fill be tracked or tamped in place where possible to limit the amount of future densification which may cause localized settlements over time.

Grading and Drainage

All grades must provide effective drainage away from the structures during and after construction and should be maintained throughout the life of the structures. Water retained next to the structures can result in soil movements greater than those discussed in this report. Greater movements can result in unacceptable differential slab and/or foundation movements.

Exposed ground should be sloped and maintained at a minimum 5 percent away from the structures for at least 10 feet beyond the perimeter. If a minimum 5 percent slope cannot be achieved due to site grades, a minimum 2½ percent slope could be used provided pavement or hardscape surrounds the structures, or a subdrain could be installed around the perimeter of the foundations that carries water away. Locally, flatter grades may be necessary to transition to ADA access requirements for flatwork. After construction and landscaping have been completed, final grades should be verified to document effective drainage has been achieved. Grades around the structures should also be periodically inspected and adjusted, as necessary, as part of the structures' maintenance programs. Where paving or flatwork abuts a structure, a maintenance program should be established to effectively seal and maintain joints and prevent surface water infiltration.

Any planters and/or bio-swales located within 10 feet of the improvements should be self-contained or lined with an impermeable membrane to prevent water from accessing subgrade soils below the improvements. Sprinkler mains and spray heads should be located a minimum of 5 feet away from the foundation lines.

Implementation of adequate drainage for this project can affect the surrounding developments. Consequently, in addition to designing and constructing drainage for this project, the effects of site drainage should be taken into consideration for the planned structures on this property, the undeveloped portions of this property, and surrounding sites. Extra care should be taken to ensure irrigation and drainage from adjacent areas do not drain onto the project site or saturate the construction area.

Earthwork Construction Considerations

Shallow excavations for the proposed structures are anticipated to be accomplished with conventional construction equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of grade-supported improvements such as floor slabs, exterior hardscape, and pavements. Construction traffic over the completed subgrades should be avoided to the extent practical. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade should become desiccated, saturated, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to construction.

Groundwater may be encountered as shallow as 5 feet bgs. The groundwater table could affect excavation efforts, especially for overexcavation and replacement of lower strength soils. A temporary dewatering system consisting of sumps with pumps may be necessary to achieve required depths of excavations depending on groundwater conditions at the time of construction. **Dewatering should be considered and planned for in proposed excavations if the excavations will extend near the groundwater levels encountered during our field exploration. The depth of dewatering below the bottom of excavations should be determined by the contractor and/or designer.** Pump tests for dewatering were not included in the scope of work for this report.

We recommend that the earthwork portion of this project be completed during extended periods of dry weather if possible. If earthwork is completed during the wet season (typically November through April) it may be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork operations may require additional mitigation measures beyond that which would be expected during the drier summer and fall months. This could include ground stabilization utilizing chemical treatment of the subgrade, diversion of surface runoff around exposed soils, and

draining of ponded water on the site. Once subgrades are established, it may be necessary to protect the exposed subgrade soils from construction traffic.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with any applicable local and/or state regulations. Stockpiles of soil, construction materials, and construction equipment should not be placed near trenches or excavations. ***The Contractor is responsible for maintaining the stability of adjacent structures during construction.***

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety or the contractor's activities; such responsibility shall neither be implied nor inferred.

Excavations or other activities resulting in ground disturbance have the potential to affect adjoining properties and structures. Our scope of services does not include review of available final grading information or consider potential temporary grading performed by the contractor for potential effects such as ground movement beyond the project limits. A preconstruction/precondition survey should be conducted to document nearby property/infrastructure prior to any site development activity. Excavation or ground disturbance activities adjacent or near property lines should be monitored or instrumented for potential ground movements that could negatively affect adjoining property and/or structures.

Construction Observation and Testing

The earthwork efforts should be observed by the Geotechnical Engineer (or others under their direction). Observation should include documentation of adequate removal of surficial materials (vegetation, topsoil, debris, and pavements), evaluation and remediation of existing fill materials, as well as proofrolling and mitigation of unsuitable areas delineated by the proofroll.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, as recommended by the Geotechnical Engineer prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency of at least one test for every 2,500 square feet of compacted fill in the building areas and 5,000 square feet in pavement areas. Where not specified by local ordinance, one density and water content test should be performed for every 100 linear feet of compacted utility trench backfill and a minimum of one test performed for every 12 vertical inches of compacted backfill.

In areas of foundation excavations, the bearing subgrade should be evaluated by the Geotechnical Engineer. If unanticipated conditions are observed, the Geotechnical Engineer should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the Geotechnical Engineer into the construction phase of the project provides the continuity to maintain the Geotechnical Engineer's evaluation of subsurface conditions, including assessing variations and associated design changes.

Foundations

BESS Foundation Options

As discussed above, our borings and CPTs advanced in the BESS area generally encountered variable subsurface conditions consisting of 6 to 9 feet of uncontrolled fill and soft to medium stiff native clay. Uncontrolled pre-existing fill and soft clay soils are not recommended for shallow foundation support due to the increased potential for differential settlement. Additional settlement on the order of 1 to 2 inches should also be expected if site grades are raised due to consolidation of the soft clay layers from new fill placement above the existing ground surface in the BESS area. In addition, we estimate up to about $\frac{3}{4}$ -inch of total settlement, and $\frac{1}{2}$ -inch of differential settlement over a span of about 50, from liquefaction occurring due to a major earthquake.

In general, due to the thickness of the fill and anticipated differential settlements, shallow foundation support for the planned BESS structures is not recommended for this site unless the subgrade soils are improved with **Ground Improvement** techniques as discussed below. It may be possible to support the BESS structures on shallow foundations supported on a section of structural fill in-lieu of ground improvement; however, the owner should be aware and willing to accept a higher level of risk of differential movement. Terracon can provide alternative recommendations for this option once more specific project information is available, if requested.

In order to mitigate the effects of the total and associated differential settlements on the proposed BESS structures from uncontrolled fill settlement, consolidation of the soft clay, and liquefaction, we recommend the following two options for foundation support for the proposed BESS improvements.

Option 1 – Shallow Foundations over Ground Improvement

The BESS structures may be supported by shallow foundations provided **Ground Improvement** is performed to mitigate the anticipated settlement. For this option, ground improvement should be performed to a depth of about 40 feet bgs. The slab

foundations for the BESS structures should be underlain by minimum 18 inches of chemically treated material or LVC structural fill to help mitigate the effects of moderate to high volume change clays.

Option 2 – Deep Foundations

As an alternative to shallow foundations supported on soil subgrade mitigated by ground improvement, the proposed improvements may be supported by **Deep Foundations** to help protect the improvements against the estimated total and differential settlements. The deep foundation systems currently being considered for the BESS structures include driven W-section steel piles or helical anchor piles. For this option, **Deep Foundations** will be required to be designed to account for drag loads imposed on the foundations as a result from liquefaction-induced settlement and consolidation settlement due to site grading fill placement. Equipment or structure floor slabs should be designed as structural slabs that span between foundation elements and rely on the foundations for support.

However, while the use of deep foundations will provide improved performance with regards to fill and consolidation settlement, the deep foundations will need to be designed to accommodate the estimated liquefaction-induced settlement provided they are terminated above liquefiable layers.

Substation Foundation Options

We expect the substation to include several structures that will house electrical equipment that will be constructed as part of the substation portion of the project. The recommendations provided are based on the test borings and CPTs advanced in the planned substation location and assumed structural loads (see **Project Description** section of this report). Due to the presence of up to about 6½ feet of uncontrolled fill consisting of loose clayey sand and stiff lean clay, the proposed substation improvements should be supported on engineered fill in accordance with the recommendations in **Earthwork**. Once loading for these ancillary structures is better known, additional analyses can be performed to confirm shallow foundation acceptability. Substation structures may also be supported on drilled cast-in-place shaft foundations designed using the soil properties presented in **Deep Foundations**.

Preliminary BESS Recommendations

BESS Option 1 - Ground Improvement

Ground improvement may be utilized to help mitigate the anticipated excessive settlement due to liquefaction, undocumented fill and consolidation of soft compressible

clay layers. As an alternative to supporting the BESS structure on deep foundations or replacing the fill, the BESS structures could be supported on lower strength/lower density native soils if ground improvement methods are utilized. Ground improvement methods such as aggregate piers and drilled displacement columns (DDC) are proprietary systems designed by licensed contractors who could provide further information regarding support options. Considering the various methods available for ground improvement, it is our opinion aggregate piers or drilled displacement columns (DDCs) would be suitable options for ground improvement at this site. However, if the Contractor or Structural Engineer have worked with a different ground improvement method that has proven successful to mitigate the hazards present at this site with similar subgrade soil conditions, Terracon could consider such options if desired.

Aggregate Piers

As a way to mitigate the presence of undocumented fill and the potential for consolidation settlement of the compressible clay layers, the subgrade soils could be improved with aggregate piers installed on a grid pattern. This option would allow for the use of shallow foundations over the aggregate pier-reinforced subgrade. Aggregate pier systems are typically installed after clearing. Aggregate piers can be used to densify the undocumented fill and potentially liquefiable cohesionless soils.

Aggregate piers are typically constructed by advancing a drill or mandrel to design depths, then building a bottom bulb of clean, open-graded stone. The pier is built on top of the bottom bulb, using graded aggregate placed in thin lifts (12 to 24 inches compacted thickness). We anticipate the aggregate piers would need to extend to a depth of at least twice the width of the proposed BESS foundation or 40 feet bgs, whichever is deeper, at the site depending on the soil layers targeted for improvement. The result is a reinforced zone of soils directly under the foundations, which allows for the design and construction of foundations for relatively higher bearing pressures and with lower anticipated settlements. Aggregate piers can also be installed where differential movement is a concern between underground utility lines; site development such as hardscape, entrances, and pavements adjacent to structures supported by ground improvement or deep foundations and site drainage.

We anticipate foundations supported over aggregate piers installed following fill placement may be designed using an allowable bearing pressure of 2,500 to 3,000 pound per square foot (psf) for dead plus live loads. However, the final design allowable bearing pressure should be confirmed by the design-build contractor installing the aggregate piers and coordinated with the structural engineer. The aggregate pier ground improvement system for this project should meet the following design criteria:

Bearing Capacity Factor of Safety = 2.0

Global Stability (static) = 1.3

Global Stability (dynamic) = 1.1

Post-construction Settlement: <1 inch for combined static and liquefaction settlement

Post-construction Differential Settlement: < ½ inch / 40 feet for combined static and liquefaction settlement

Aggregate pier systems should be designed and constructed by a specialty ground improvement contractor. At least one load test should be performed in the footprint of each improvement to confirm the aggregate pier design capacity prior to full production of aggregate piers. Since this would be specialty work, we recommend consideration of using a design-build process if this alternative is selected. A design and installation package including a quality control plan for aggregate pier installation should be prepared by the design-build contractor license in the State of California and submitted to Terracon for review and approval prior to construction. The package should also include information regarding load testing such as proposed test location, set-up, and testing parameters. Terracon should be present on-site during load test and production to observe installation and testing of the aggregate piers.

Drilled Displacement Columns

DDCs would help increase the bearing capacity of the subgrade soils while reducing settlement and help mitigate potential settlement below the improvements by transferring their loads to deeper, more competent soils and improving the soil around the DDCs. DDCs are constructed by using a displacement auger to create a soil shaft filled with Controlled Low Strength Material (CLSM) injected under pressure as the displacement auger is withdrawn. DDCs generate a minimal amount of soil cuttings compared to other foundation and ground improvement methods. The diameter of DDCs typically vary between 18 to 36 inches and based on foundation load requirements the strength of CLSM can typically vary between 100 to 500 psi at 28 days.

We anticipate foundations supported on DDCs may be designed using an allowable bearing pressure of 3,000 to 3,500 pounds per square foot (psf) for dead plus live loads. However, the final design allowable bearing pressure should be confirmed by the design-build contractor installing the DDCs and coordinated with the structural engineer. The DDC ground improvement system for this project should meet the following design criteria:

- Bearing Capacity Factor of Safety = 2.0
- Global Stability (static) = 1.3
- Global Stability (dynamic) = 1.1
- Post-construction Settlement: < 1-inch for combined static and liquefaction settlement
- Post-construction Differential Settlement: < ½ -inch / 40 feet for combined static and liquefaction settlement

DDC capacities and settlement based on anticipated loading should be evaluated by a design-build contractor. Terracon should be afforded the opportunity to review the bearing capacity and settlement evaluation.

We anticipate the DDCs would need to extend to a depth of at least twice the width of the proposed BESS foundation or 40 feet bgs, whichever is deeper, depending on the soil layers targeted for improvement. Minimum DDC lengths shall be confirmed by Terracon during construction. A 12 to 24-inch cushion of Caltrans Class II aggregate base should be installed between the top of the DDCs and the bottom of foundations and slabs.

Design and installation of DDCs shall be performed by a qualified design-build contractor. At least one load test should be performed in the footprint of each improvement to confirm DDC capacities prior to installing production columns. Axial compression load tests shall be performed according to ASTM D1143. Only columns meeting the load testing criteria shall be used as final production columns. A design and installation package including a quality control plan for DDC installation should be prepared by the design-build contractor and submitted to Terracon for review and approval prior to construction. The package should also include information regarding load test such as proposed test location, set-up, and testing parameters. Terracon should be present on-site during load test and production to observe installation and testing of the DDCs.

BESS Option 2 - Deep Foundations

Preliminary Driven Pile Design Parameters

Proposed battery storage units can be supported on driven steel W-section foundations (assumed to be W6x9, W6x15 or similar) in general accordance with the following sections.

As discussed in [Liquefaction Evaluation](#), we estimate up to about ¾-inch of total settlement and about ½-inch of differential settlement over a span of about 50 feet, from liquefaction. The pile foundations should be designed to accommodate the estimated liquefaction settlement from the design level earthquake. If the foundations cannot be designed to accommodate the estimated liquefaction settlement, we recommend that ground improvement be performed to mitigate the liquefaction potential across the site.

The design capacity of a single-driven pile is a function of several factors including:

- Size and type of pile;
- Type and capacity of pile installation equipment;
- Pile integrity after installation; and
- Engineering properties of the subsurface soils.

Based on specific conditions encountered on site, the soils are generally considered drivable for pile installation. The most effective means of verifying pile drivability and capacities for either tension or lateral loads is through pile load tests. Pile foundation design parameters have been based upon correlated capacities utilizing soil strength criteria determined from the soil borings, CPTs, and laboratory testing.

The axial uplift and compression capacity of driven piles supporting the BESS structures may be estimated based on skin friction developed along the perimeter of the pile. When determining embedment depths, the perimeter of a wide flange beam should be taken as twice the sum of the flange width and section depth. The upper undocumented fill (9 feet thick) should be neglected in the axial capacity analyses. The ultimate axial capacity of driven steel piles may be calculated using the skin friction values as presented in the following table:

Axial Compression Design Parameters		
Pile Embedment Depth (feet-bgs)	Ultimate Compression Skin Friction (psf)^{1, 2}	Recommended Factor of Safety
9	0	0
9 to 15	200	2.0
15 to 20	500	2.0

1. Ultimate uplift capacity is on the order of 60% of the compression capacity values in the table. The values provided should be multiplied by the box perimeter of the pile times the depth. The box perimeter is considered two times the width of the flange plus two times the depth of the web.
2. The values provided should be multiplied by the box area of the pile and be used for compression resistance only.

Final pile design should be determined by the Geotechnical and Structural Engineer based upon information contained in this geotechnical report and independent pile load testing.

Driven Piles Construction Considerations

Based on the field exploration and laboratory testing, it is our opinion that the soils on the site are suitable for pile installation into native soils. It is anticipated that predrilling may be required (depending on the driven deep foundation size and type selected) to penetrate debris and/or obstructions in the existing site fills. The drill auger and predrill hole diameter should not be greater than the diameter of the piles.

A geotechnical engineer should be engaged to make periodic observations of pile driving operations during construction. Each pile should be observed and checked for buckling, crimping and alignment in addition to recording penetration resistance, depth of embedment, and general pile driving operations.

As part of the overall quality control program, the time rate of installation (seconds per foot of embedment) should be recorded during production post driving. As a direct extension of the design process, additional “proof” testing should be performed on a representative number of production posts that do not meet the minimum installation rate criteria outlined in this report.

Preliminary Helical Pile Anchor Recommendations

The proposed BESS structures may be supported on helical pile anchor foundations extending to competent native soil. Helical piles are a foundation system that consists of one or more helical steel plates welded to a central steel shaft which is screwed into the ground instead of driving. It is our understanding that the design team is currently considering helical piles with 5.5-inch to 9.625-inch steel pipe diameters, with 16-inch to 24-inch helixes. We recommend extending piles through the soft clay layers in the upper 10 to 15 feet and bear into stiffer/denser soils below. Test piles should be installed to determine actual embedment depths required to achieve capacity.

As discussed in [Liquefaction Evaluation](#), we estimate up to about ¾-inch of total settlement and about ½-inch of differential settlement over a span of about 50 feet, from liquefaction. The pile foundations should be designed to accommodate the estimated liquefaction settlement from the design level earthquake. If the foundations cannot be designed to accommodate the estimated liquefaction settlement, we recommend that ground improvement be performed to mitigate the liquefaction potential across the site.

Helical pile anchors are a proprietary foundation system which should be designed by a qualified design-build specialty contractor in consultation with the structural engineer to provide engineering analysis and project specific design information

- Helical pile anchors should be spaced a minimum of four helical bearing plate diameters on center to avoid group effects on axial capacity. Design spacing of the piers should be determined by the structural engineer based on structural demands and grade beam design.
- We do not recommend using vertically installed helical pile anchors to resist lateral loads without approved lateral load test data, as these types of foundations are typically designed to resist axial loads. For battered piers, only the horizontal component of the allowable axial load should be considered to resist the lateral loading and only in the direction of the batter.
- The helical pile designer should select the size and number of helical bearing plates for each helical pier based on planned loads and bearing materials.

- A minimum of two load tests should be performed in the footprint of the BESS structure area to confirm helical pile capacities and calibrate torque correlations prior to installing production piles. Axial load tests shall be performed according to ASTM D3689. Only piles meeting the load testing criteria shall be used as final production piles.
- A design and installation package including a quality control plan for helical pile installation should be prepared by the design-build contractor and submitted to Terracon for review and approval prior to construction. The package should also include information regarding load testing such as proposed test location, set-up, and testing parameters. Terracon should be present on-site during load test and production to observe installation and testing of the helical piles.
- Torque measurements during installation of all production helical piles should be used to verify the axial capacity of the helical piles.
- Terracon should be provided with the torque to capacity relationships for each type of pile used on the project for our review and comment prior to mobilization to the site.
- The helical pile foundations should be installed per the manufacturer's recommendations.
- Terracon should be retained to observe helical pier installation to verify that proper bearing materials have been encountered during installation.

Preliminary Substation Recommendations

The substation facility will include electrical structures including duct banks and large electrical equipment supported by either shallow spread footings and mat slabs, or drilled cast-in-place shaft deep foundation systems. The recommendations provided are based on the test borings and CPTs advanced in the planned substation location and assumed structural loads (see [Project Description](#) section of this report). Due to the presence of up to about 6½ feet of uncontrolled fill consisting of loose clayey sand and stiff lean clay, the proposed substation improvements should be supported on engineered fill in accordance with the recommendations in [Earthwork](#). Once loading for these ancillary structures is better known, additional analyses can be performed to confirm shallow foundation system feasibility. Substation structures may also be supported on drilled cast-in-place shaft foundations designed using the parameters presented in [Deep Foundations](#).

Shallow Foundations

The proposed improvements planned for the substation may be supported by shallow foundations consisting of either spread footings or mat slab foundations. If the site has been prepared in accordance with the requirements noted in [Earthwork](#), the following design parameters are applicable for the design of shallow and mat foundations. Uncontrolled fill up to about 6½ feet was encountered in our test borings at the substation area and should not be relied upon for support; therefore, we recommend that the planned structures be supported on granular structural fill or Controlled Low Strength Material (CLSM) in accordance with the recommendations below. Shallow foundations should also be designed to accommodate the estimated total and differential settlement from liquefaction as discussed in [Liquefaction Evaluation](#).

Mat Foundation Design Parameters – Compressive Loads

The design of mat foundations is typically performed in an iterative process between the Geotechnical Engineer and Structural Engineer. Use of finite element analysis is often performed when loads acting on the mat are asymmetric or non-uniform, but estimates of soil pressure and resulting settlement can converge quickly absent of sophisticated design procedures in cases where uniform loads are acting upon the mat. This procedure is outlined in ACI 336.2 “Suggested Analysis and Design Procedure for Combined Footings and Mats”.

For the initial iteration of this design, we have assumed that the pressure acting on the soil is uniform across the footprint of the mat. This assumption would only be true for a flexible foundation or thin mat where load redistribution cannot be achieved through the mat foundation. The resulting settlement from this assumption would produce a dish-shaped settlement profile with the settlement at the center of the mat approximately twice that of the edges.

Item	Description
Maximum Net Allowable Bearing Pressure^{1, 2}	1,400 psf; for mat foundations up to 10 feet wide 1,000 psf; for mat foundations up to 14 feet wide 550 psf; for mat foundations up to 30 feet wide
Required Bearing Stratum³	At least 5 feet of granular structural fill or CLSM
Design Modulus of Subgrade Reaction, k⁹	For 1,400 psf bearing pressure for noted maximum foundation width: 9.7 psi/in – mat interior 14.6 psi/in – edge of foundation to 0.25*B toward center For 1,000 psf bearing pressure for noted maximum foundation width: 6.9 psi/in – mat interior

Item	Description
	10.4 psi/in – edge of foundation to 0.25*B toward center For 550 psf bearing pressure for noted maximum foundation width: 3.8 psi/in - mat interior 5.7 psi/in – edge of foundation to 0.25*B toward center
Ultimate Passive Resistance^{4, 8} (equivalent fluid pressures)	350 pcf
Sliding Resistance^{5, 8}	0.35 allowable coefficient of friction
Minimum Embedment below Finished Subgrade⁶	18 inches
Estimated Total Settlement from Structural Loads at the center of the mat²	About 1 inch or less
Estimated Differential Settlement⁷	About 1/2 of total settlement

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the foundation base elevation. This bearing pressure can be increased by 1/3 for transient loads unless those loads have been factored to account for transient conditions.
2. Maximum allowable pressure is based on both calculated bearing capacities and allowable settlements.
3. Uncontrolled fill, expansive soils or soft soils should be overexcavated and replaced per the recommendations presented in [Earthwork](#).
4. Use of passive earth pressures require the sides of the excavation for the foundation to be nearly vertical and the concrete placed neat against these vertical faces or that the foundation forms be removed and compacted structural fill be placed against the vertical foundation face. Assumes no hydrostatic pressure.
5. Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Frictional resistance for granular materials is dependent on the bearing pressure which may vary due to load combinations.
6. Embedment necessary to minimize the effects of seasonal water content variations.
7. Differential settlements are noted for equivalent-loaded foundations and bearing elevation as measured over a span of 40 feet. **Estimated settlements do not include settlement from liquefaction.**
8. Passive Resistance and Sliding Resistance may be combined to resist sliding provided the Passive Resistance is reduced by 50 percent.

Item	Description
9.	We anticipate that the Structural Engineer will design the mat to provide some level of rigidity either through thickness or ribs/cellular construction techniques to promote more uniform settlement. The perfectly rigid foundation with uniform settlement has loads redistributed to the margins. The provided moduli of subgrade reaction are recommended to analyze mat stresses for this condition. The Structural Engineer should provide our Geotechnical Engineer with the anticipated foundation stresses so that settlement and the corresponding soil modulus can be subsequently estimated and compared to the values in our current analysis. If these values deviate significantly, revised estimates of soil modulus will be provided for further iteration until convergence is attained.

Spread Footing Design Parameters – Compressive Loads

Item	Description
Maximum Net Allowable Bearing Pressure ^{1, 2}	1,250 psf
Required Bearing Stratum ³	At least 5 feet of granular structural fill or CLSM
Minimum Foundation Dimensions	Per CBC 1809.7
Maximum Foundation Dimensions	Columns: 15 feet
Ultimate Passive Resistance (equivalent fluid pressures) ^{4, 8}	350 pcf
Sliding Resistance ^{5, 8}	0.35 allowable coefficient of friction
Minimum Embedment below Finished Subgrade ⁶	18 inches
Estimated Total Settlement from Structural Loads ²	About 1 inch or less
Estimated Differential Settlement ^{2, 7}	About 1/2 of total settlement

1. The maximum net allowable bearing pressure is the pressure in excess of the minimum surrounding overburden pressure at the footing base elevation. This bearing pressure can be increased by 1/3 for transient loads unless those loads have been factored to account for transient conditions.
2. Values provided are for maximum loads noted in [Project Description](#). Additional geotechnical consultation will be necessary if higher loads are anticipated.
3. Uncontrolled fill, expansive soils or soft soils should be overexcavated and replaced per the recommendations presented in [Earthwork](#).
4. Use of passive earth pressures require the sides of the excavation for the spread footing foundation to be nearly vertical and the concrete placed neat against these vertical

Item	Description
	faces or that the footing forms be removed and compacted structural fill be placed against the vertical footing face. Assumes no hydrostatic pressure.
5.	Can be used to compute sliding resistance where foundations are placed on suitable soil/materials. Frictional resistance for granular materials is dependent on the bearing pressure which may vary due to load combinations.
6.	Embedment necessary to minimize the effects of seasonal water content variations.
7.	Differential settlements are noted for equivalent-loaded foundations and bearing elevation as measured over a span of 40 feet. Estimated settlements do not include settlement from liquefaction.
8.	Passive Resistance and Sliding Resistance may be combined to resist sliding provided the Passive Resistance is reduced by 50 percent.

Design Parameters – Overturning and Uplift Loads

Shallow foundations subjected to overturning loads should be proportioned such that the resultant eccentricity is maintained in the center-third of the foundation (e.g., $e < b/6$, where b is the foundation width). This requirement is intended to keep the entire foundation area in compression during the extreme lateral/overturning load event. Foundation oversizing may be required to satisfy this condition.

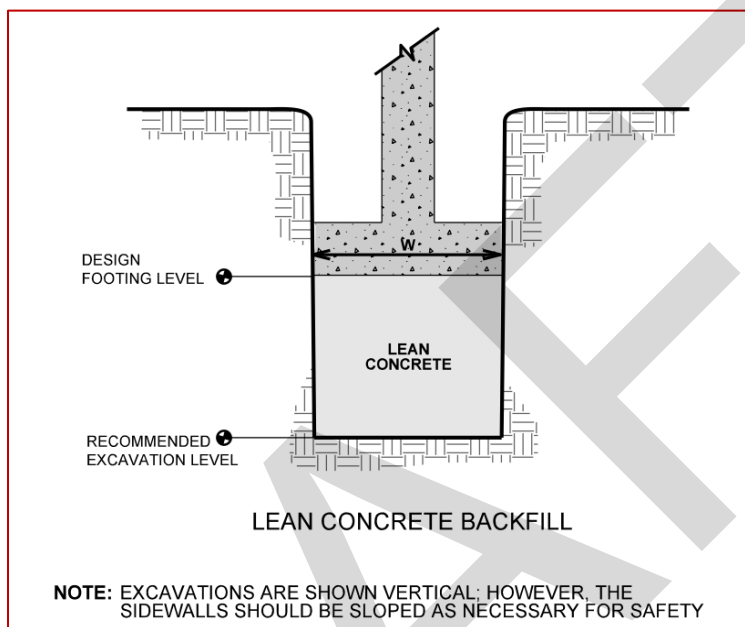
Uplift resistance of spread footings can be developed from the effective weight of the footing and the overlying soils with consideration to the 2022 CBC basic load combinations.

Item	Description
Soil Moist Unit Weight	110 pcf
Soil Effective Unit Weight	48 pcf
Soil weight included in uplift resistance	Soil included within the prism extending up from the top perimeter of the footing at an angle of 20 degrees from vertical to ground surface

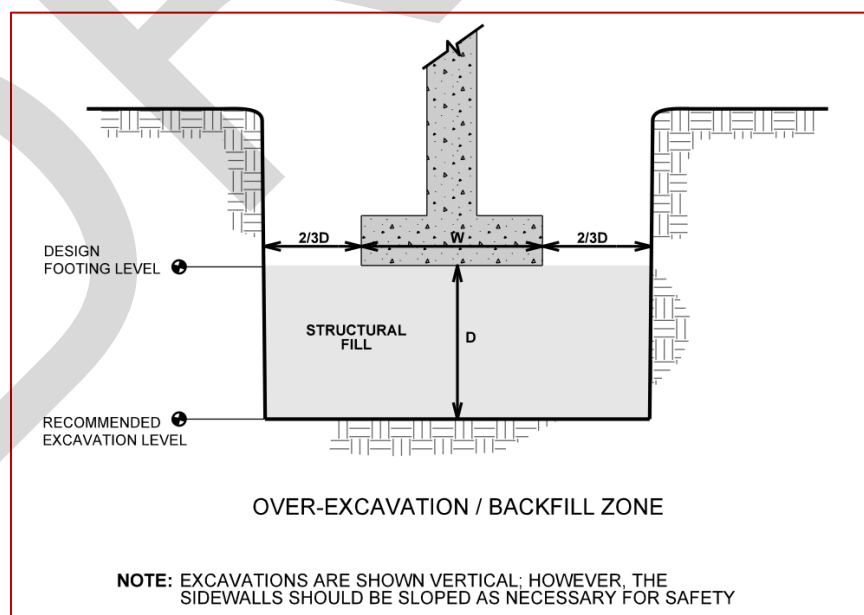
Foundation Construction Considerations

As noted in [Earthwork](#), the foundation excavations should be evaluated under the observation of the Geotechnical Engineer. The base of all foundation excavations should be free of water and loose soil, prior to placing concrete. Concrete should be placed soon after excavating to reduce bearing soil disturbance. Care should be taken to prevent wetting or drying of the bearing materials during construction. Excessively wet or dry material or any loose/disturbed material in the bottom of the foundation excavations should be removed/reconditioned before foundation concrete is placed.

If unsuitable bearing soils are observed at the base of the planned foundation excavations, the excavations should be extended deeper to suitable soils, and the foundations could bear directly on these soils at the lower level or on lean concrete backfill placed in the excavations. The lean concrete replacement zone is illustrated on the following sketch.



Overexcavation for structural fill placement below foundations should be conducted as shown in the following sketch. The overexcavation should be backfilled up to the foundation base elevation, with structural fill placed, as recommended in the [Earthwork](#) section.



To ensure foundations have adequate support, special care should be taken when foundations are located adjacent to trenches. The bottom of such foundations should be at least 1 foot below an imaginary plane with an inclination of 1.5 horizontal to 1.0 vertical extending upward from the nearest edge of the adjacent trench.

Deep Foundations

Preliminary Drilled Shaft Axial Loading

The proposed BESS structures and associated improvements in the BESS area may be supported by a deep foundation system consisting of drilled cast-in-place shafts. The design shaft capacities and lengths should be determined by the Geotechnical and Structural Engineers during final design. For preliminary design, an estimated allowable compression capacity versus depth chart for shaft diameters ranging from 24 to 48 inches is provided in [Supporting Information](#).

We recommend that the deep foundation system be designed to develop axial compression through skin friction only, and end-bearing should be neglected. Shaft uplift capacity should also be derived from skin friction only. The allowable uplift capacities should only be based on 60% of the allowable side friction of the shaft; however, the weight of the foundation should be added to these values to obtain the actual allowable uplift capacities for drilled shafts. Tensile reinforcement should extend to the bottom of shafts subjected to uplift loading. The allowable capacity values are based on a factor of safety of 2.5.

Negative skin friction, or drag loads, as a result of consolidation and liquefaction-induced settlement should also be considered in the design of the deep foundation system. This settlement will result in additional loading on the shaft foundations which should be considered, in addition to the long-term loads, for evaluating the structural capacity of the shaft foundations.

Terracon should be contacted to provide estimates of drag loads when structural design information and shaft foundation system type and geometry are developed during final design. The project Structural Engineer should confirm combined drag and design loads do not exceed the structural capacity of the drilled shaft foundations.

Buoyant unit weights of the soil and concrete should be used in the calculations below the highest anticipated groundwater elevation of 5 feet bgs.

Drilled shaft should have a minimum (center-to-center) spacing of three diameters. Closer spacing may require a reduction in axial load capacity. Axial capacity reduction can be determined by comparing the allowable axial capacity determined from the sum of individual piles in a group versus the capacity calculated using the perimeter and base of the pile group acting as a unit. The lesser of the two capacities should be used in design.

Total settlement of an individual shaft foundation will vary depending on the dimensions of the foundation and the actual load supported. However, we anticipate settlements of drilled shafts designed and constructed as described in this report are estimated to range from about $\frac{1}{2}$ to $\frac{3}{4}$ inch. Differential settlement between individual shafts is expected to be $\frac{1}{2}$ to $\frac{2}{3}$ of the total settlement.

Lateral Loading

The following table lists input values for use in LPILE analyses and may be use for both driven steel piles and drilled shaft foundations. Such analysis should be considered if lateral loads exceed 15 kips. Modern versions of LPILE provide estimated default values of k_h and E_{50} based on strength and are recommended for the project. Since deflection or a service limit criterion will most likely control lateral capacity design, no safety/resistance factor is included with the parameters.

Stratigraphy ¹		L-Pile Soil Model	S_u (psf) ²	ϕ ²	γ' (pcf) ²	E_{50}	K (pci)	
Depth	Material						Static	Cyclic
5 ³ - 8	Lean Clay Fill	Stiff Clay with Free Water	800	---	58	Use Default Value		
8 - 20	Fat Clay	Stiff Clay with Free Water	1,200	---	63	Use Default Value		
20 - 35	Fat Clay	Stiff Clay with Free Water	1,500	---	63	Use Default Value		
35 - 60	Fat Clay	Stiff Clay with Free Water	2,000	---	63	Use Default Value		

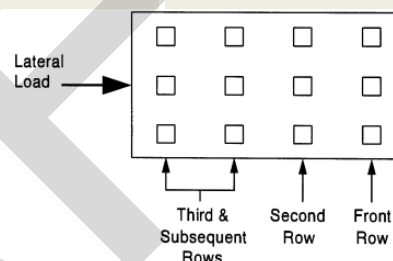
1. See Subsurface Profile in [Geotechnical Characterization](#) for more details on Stratigraphy.
2. Definition of Terms:
 S_u : Undrained shear strength
 ϕ : Internal friction angle
 γ' : Effective unit weight
3. The upper 5 feet of the drilled shaft should be neglected from design.

When shafts/piles are used in groups, the lateral capacities of the shafts/piles in the second, third, and subsequent rows of the group should be reduced as compared to the capacity of a single, independent shaft/pile. Guidance for applying p-multiplier factors to

the p values in the p-y curves for each row of shaft foundations within a shaft group are as follows:

Center to Center Shaft Spacing ^{1,2}	P-Multiplier, P_m ³		
	Front Row	Second Row	Third and Subsequent Rows
3B	0.8	0.4	0.3
4B	0.9	0.65	0.5
5B	1.0	0.85	0.7
6B	1.0	1.0	1.0

1. Spacing in the direction of loading. B = shaft diameter
2. For the case of a single row of shafts/piles supporting a laterally loaded grade beam, group action for lateral resistance of shafts/piles would need be considered when spacing is less than three shaft diameters (measured center-to-center).
3. See adjacent figure for definition of front, second and third rows.



Spacing closer than 3D (where D is the diameter of the shaft) is not recommended without additional geotechnical consultation due to potential for the installation of a new shaft disturbing an adjacent installed shaft likely resulting in axial capacity reduction.

Drilled Shaft Construction Considerations

The drilling contractor should be experienced in the subsurface conditions observed at the site, and the excavations should be performed with equipment capable of providing a clean bearing surface free of any loose material. The drilled straight-shaft foundation system should be installed in general accordance with the procedures presented in "Standard Specification for the Construction of Drilled Piers", ACI Publication No. 336.1-01.

The contractor is generally expected to use conventional "dry" techniques for installation of the drilled shafts. Subsurface water was encountered as shallow as 5 feet bgs in our borings. Subsurface water levels are influenced by seasonal and climatic conditions, which result in fluctuations in subsurface water elevations. Additionally, it is common for water to be present after periods of significant rainfall. Casing or slurry drilling procedures could be required in soils zones of higher sand content to reduce the potential for excavation sidewall collapse.

To prevent collapse of the sidewalls and/or to control groundwater seepage, the use of temporary steel casing and/or slurry drilling procedures may be required for construction of the drilled shaft foundations. Significant seepage could occur in case of excavations penetrating water-bearing sandy soil. The drilled shaft contractor and foundation design engineer should be informed of these risks.

While withdrawing casing, care should be exercised to maintain concrete inside the casing at a sufficient level to resist earth and hydrostatic pressures acting on the casing exterior. Arching of the concrete, loss of seal and other problems can occur during casing removal and result in contamination of the drilled shaft. These conditions should be considered during the design and construction phases. Placement of loose soil backfill should not be permitted around the casing prior to removal.

Care should be taken to not disturb the sides and bottom of the excavation during construction. The bottom of the shaft excavation should be free of loose material before concrete placement. Concrete should be placed as soon as possible after the foundation excavation is completed, to reduce potential disturbance of the bearing surface.

Formation of mushrooms or enlargements at the tops of shafts should be avoided during shaft drilling. If mushrooms develop at the tops of the shafts during drilling, sono-tubes should be placed at the shaft tops to help isolate the shafts.

The drilled shaft installation process should be performed under the observation of the Geotechnical Engineer. The Geotechnical Engineer should document the shaft installation process including soil/rock and groundwater conditions observed, consistency with expected conditions, and details of the installed shaft.

Below-Grade Structures

We understand that the project may include an underground vault structure near the northeast corner of the site and the substation area. The below recommendations are preliminary and should be refined once specific details for the underground structures become available.

Underground Vault Structure

Based on the subsurface conditions encountered in Boring B6, in the vicinity of the planned underground vault, we anticipate that relatively soft clay soils will be encountered at the bottom of the vault excavation depending on the actual depth of the excavation.

Groundwater may be encountered as shallow as 5 feet bgs. The groundwater table could affect excavation efforts, especially for overexcavation and replacement of lower

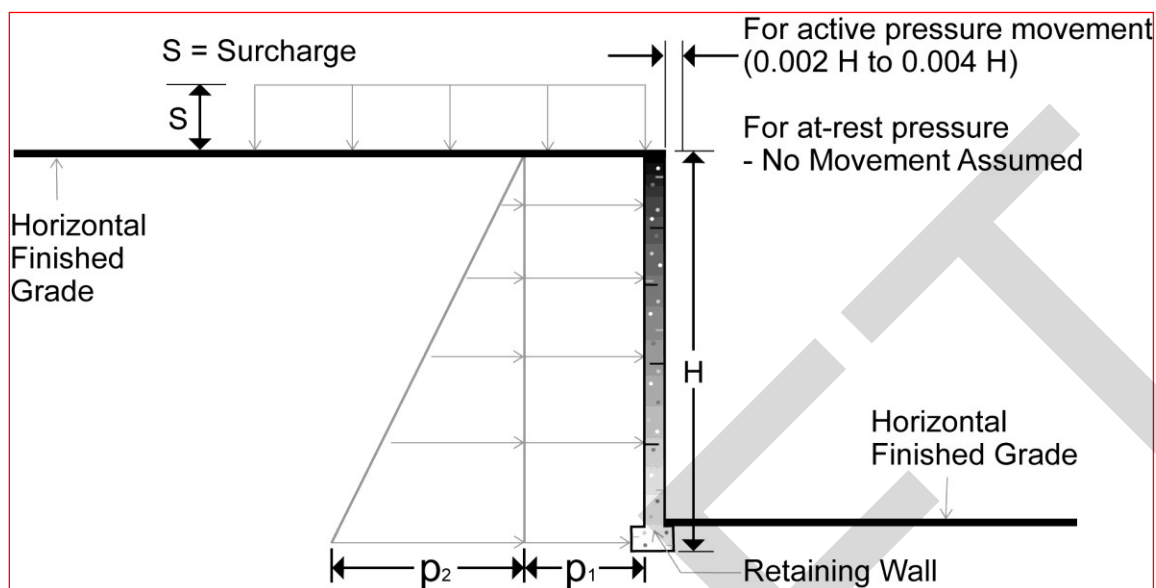
strength soils. A temporary dewatering system consisting of sumps with pumps may be necessary to achieve required depths of excavations depending on groundwater conditions at the time of construction. **Dewatering should be considered and planned for in proposed excavations if the excavations will extend near the groundwater levels encountered during our field exploration. The depth of dewatering below the bottom of excavations should be determined by the contractor and/or designer.** Pump tests for dewatering were not included in the scope of work for this report.

Based on the depth of the underground structure, uplift pressures due to hydrostatic loading should be accounted for during design of the underground structure.

Lateral Earth Pressures

Design Parameters

Below-grade construction is expected to be limited to utility vaults. We have assumed the pits/vaults will be 5 feet deep or less. Structures with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to values indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction, and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown in the following diagram. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement and is commonly used for basement walls, loading dock walls, or other walls restrained at the top. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls (unless stated).



Lateral Earth Pressure Design Parameters

Earth Pressure Condition ¹	Coefficient for Backfill Type ²	Surcharge Pressure ³ p ₁ (psf)	Equivalent Fluid Pressures (psf) ^{2,4}	
			Unsaturated ⁵	Submerged ⁵
Active (K _a)	Granular - 0.27	(0.27)S	(33)H	(78)H
	Cohesive - 0.41	(0.41)S	(45)H	(82)H
At-Rest (K _o)	Granular - 0.43	(0.43)S	(51)H	(87)H
	Cohesive - 0.58	(0.58)S	(64)H	(90)H

1. For active earth pressure, wall must rotate about base, with top lateral movements 0.002 H to 0.004 H, where H is wall height. Fat clay or other expansive soils should not be used as backfill behind the wall.
2. Uniform, horizontal backfill, with a maximum unit weight of 110 pcf for cohesive soils and 120 pcf for granular soils.
3. Uniform surcharge, where S is surcharge pressure.
4. Loading from heavy compaction equipment is not included.
5. To achieve "Unsaturated" conditions, follow guidelines in the following **Subsurface Drainage for Below-Grade Walls** or **Retaining Wall Drainage** section of this report. "Submerged" conditions are recommended when drainage behind walls is not incorporated into the design.
6. Values in the table are for flat backfill only.

Backfill placed against structures should consist of granular soils or low plasticity cohesive soils. For the granular values to be valid, the granular backfill must extend out and up from the base of the wall at an angle of at least 45 degrees from vertical for the active case.

Heavy equipment should not operate within a distance closer than the exposed height of retaining walls to prevent lateral pressures more than those provided. Compaction of each lift adjacent to wall should be accomplished with hand-operated tampers for other lightweight compactors. Over-compaction may cause excessive lateral earth pressures which could result in wall movement.

Footings, slabs or other loads bearing on backfill behind walls may have a significant influence on the lateral earth pressure. Placing footings within wall backfill and in the zone of active soil influence on the wall should be avoided unless structural analyses indicate the wall can safely withstand the increased pressure.

The lateral earth pressure recommendations given in this section are applicable to the design of rigid retaining walls subject to slight rotation, such as cantilever, or gravity type concrete walls. These recommendations are not applicable to the design of modular block - geogrid reinforced backfill walls (also termed MSE walls). Recommendations covering these types of wall systems are beyond the scope of services for this assignment. However, we would be pleased to develop a proposal for evaluation and design of such wall systems upon request.

Subsurface Drainage for Below-Grade Walls

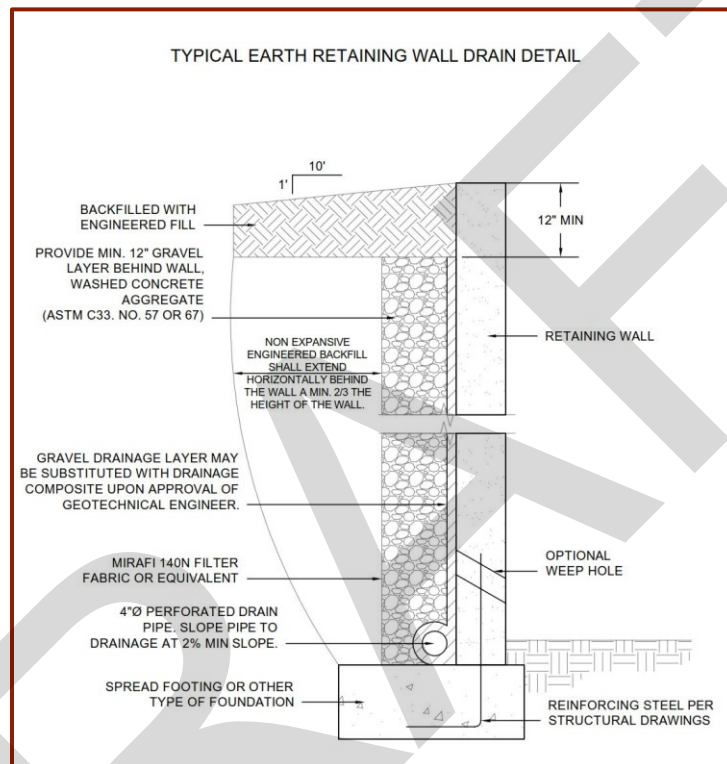
A perforated rigid plastic drain line installed behind the base of walls and extends below adjacent grade is recommended to prevent hydrostatic loading on the walls. The invert of a drain line around a below-grade building area or exterior retaining wall should be placed near foundation bearing level. The drain line should be sloped to provide positive gravity drainage to daylight or to a sump pit and pump. The drain line should be surrounded by clean, free-draining granular material having less than 5% passing the No. 200 sieve, such as No. 57 aggregate. The free-draining aggregate should be encapsulated in a filter fabric. The granular fill should extend to within 2 feet of final grade, where it should be capped with compacted cohesive fill to reduce infiltration of surface water into the drain system.

As an alternative to free-draining granular fill, a prefabricated drainage composite may be used. A prefabricated drainage composite is a plastic drainage core or mesh which is covered with filter fabric to prevent soil intrusion and is fastened to the wall prior to placing backfill.

Retaining Wall Drainage

To control hydrostatic pressure behind the wall we recommend that a drain be installed at the bottom of the wall with a collection pipe leading to a reliable discharge. The drainage should consist of either a prefabricated drainage composite or a 12-inch-thick free draining gravel blanket. Free draining gravel should consist of Caltrans Class II

permeable material or $\frac{3}{4}$ inch clean gravel wrapped in Mirafi 140N filter fabric or equivalent. The drainage should extend from the bottom of the wall to within 12 inches of the top of the wall. The drainage should be capped with 12 inches of compacted cohesive soil. The collection pipe should be designed by the Civil Engineer but should be a minimum 4-inch diameter perforated Schedule 40 PVC or ABS drain pipe and should slope to an existing drainage system or to a positive gravity outlet. A typical earth retaining wall drain detail is illustrated on the following sketch.



Gravel-Surfaced Drives and Parking

Subgrade Preparation

On most project sites, the site grading is accomplished relatively early in the construction phase. Fills are typically placed and compacted in a uniform manner. However, as construction proceeds, the subgrade may be disturbed due to utility excavations, construction traffic, desiccation, or rainfall. As a result, the aggregate-surfaced roadway or parking area subgrade may not be suitable for construction and corrective action will be required. The subgrade should be carefully evaluated at the time of construction for signs of disturbance or instability. We recommend the subgrade be thoroughly proof-rolled with a loaded tandem-axle dump truck prior to final grading. All aggregate-surfaced roadway or parking subgrade areas should be moisture

conditioned and properly compacted in accordance with the recommendations provided in the **Earthwork**.

Design Recommendations

Design of aggregate-surfaced roadways for the project has been based in general accordance with the "Aggregate-Surfaced Road Design Catalog" subsection of the 1993 AASHTO "Guide for the Design of Pavement Structures" and based on subsurface conditions observed at the site and laboratory test results.

Our analysis has assumed a "medium" traffic volume which is appropriate for 30,000 to 60,000 equivalent single axle loads (ESALs) over the life of the gravel surfacing. This traffic volume should be confirmed by the design team; additional geotechnical consultation and revision of recommendations could be necessary with higher traffic volumes.

A bulk sample of the near surface native soils were collected to perform Hveem Stabilometer (R-Value) testing. The testing resulted in an R-Value of 13. Additional R-Value testing may be performed following rough grading of the site on the subgrade soils that will ultimately support proposed pavements in order to determine if a more favorable R-Value result may be used in design reducing planning roadway sections.

Recommended minimum aggregate surfacing thickness is provided in the following table:

Recommended Gravel Surface Thickness

Locations	Relative Quality of Roadway Subgrade	Traffic Category	Minimum Aggregate-Surfacing Material Thickness, in. ¹
Parking and Roadways	Poor to Fair	Medium	12

1. Materials should meet the requirements of the current Caltrans Highway Design Manual specifications for Aggregate Base Course.

Quality roadway surfacing materials should consist of a blend of gravel, sand, and fines (clay and silt). The maximum size particle should not exceed 1 inch in diameter and the gravel should be crushed with angular edges (not rounded). The blend of materials should be selected to allow for easy compaction resulting in a firm, low permeable surface promoting surface drainage off the roadway surface. Materials meeting Caltrans Class 5 or 6 specifications can be used for aggregate surfacing material. Aggregate surfacing material should be placed in lifts not exceeding 6 inches and compacted to a minimum of 95 percent of the maximum dry unit weight as determined by ASTM D1557.

A quality roadway surfacing material should also contain approximately 10 to 25 percent fines (silt and clay-sized particles passing the No. 200 sieve). The fines should exhibit

low to moderate plasticity (plastic index less than 15) and will act as a binder to help reduce risk for wash boarding. If the fines content of a roadway surfacing material is comprised mostly of silt, the fines will be non-plastic and the surfacing materials will not have the benefit of the binder or cohesive aspects.

In order to reduce dust, reclaimed asphalt pavement (RAP) may be used as the upper 2 to 4 inches of the aggregate-surfacing. The RAP should be graded to the specified limits for Caltrans Class 5 or 6 aggregate base course but modified to contain 10 to 25 percent fines and properly compacted. Periodic spraying (1 to 2 times a year following maintenance grading) of the surface with magnesium chloride or other dust suppressant may also be considered to reduce dust and wash boarding.

Aggregate-surfaced roadway performance is affected by its surroundings. In addition to providing preventive maintenance, the Civil Engineer should consider the following recommendations in the design and layout of aggregate-surfaced roadways:

- Site grades should slope a minimum of 10 percent away from the roadways;
- The subgrade and the aggregate-surfaced roadways have a minimum 10 percent slope to promote proper surface drainage;
- Consider appropriate edge drainage; and
- Install pavement drainage in surrounding areas anticipated for frequent wetting.

Maintenance

Periodic maintenance extends the service life of the aggregate-surfaced roadways and parking areas and should include re-grading and replacement of aggregate base course in any deteriorated areas. Thicker aggregate base course sections could be used to reduce the required maintenance and extend the service life of the aggregate-surfaced roadways. Design alternatives which could reduce the risk of subgrade saturation and improve long-term performance include installing surface drains next to any areas where surface water could pond. Properly designed and constructed subsurface drainage will reduce the time subgrade soils are saturated and can also improve subgrade strength and performance.

General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide

further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials, or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly affect excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety and cost estimating including excavation support and dewatering requirements/design are the responsibility of others. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a preconstruction/precondition survey of surrounding development. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing. This report should not be used after 3 years without written authorization from Terracon.

Preliminary Geotechnical Engineering Report

Calpine Tanager Energy Storage Project | San Jose, Santa Clara County, California

August 29, 2025 | Terracon Project No. R1245076

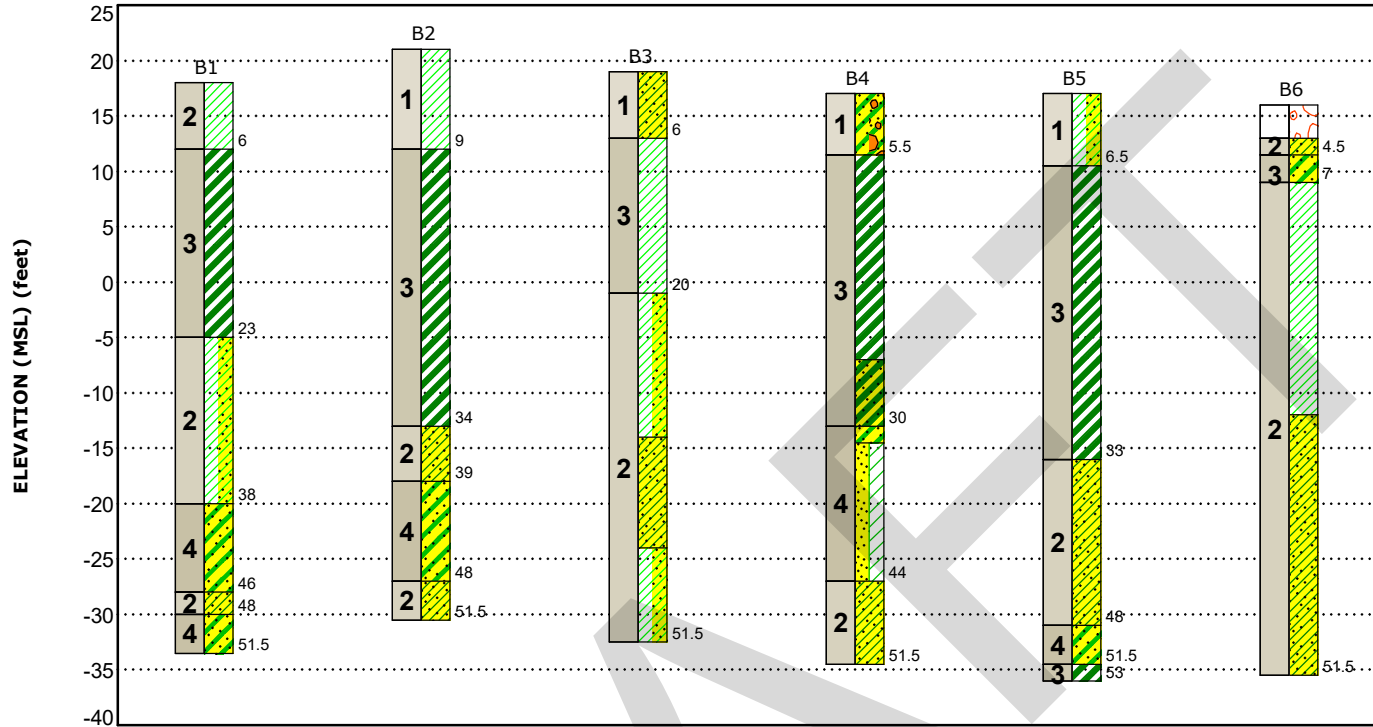


Figures

Contents:

GeoModel
BESS Section
Substation Section

GeoModel



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description	Legend	
1	Existing Fill	Soft to very stiff, Lean Clay with variable amounts of sand, gravel, and debris, and Loose Clayey Sand with Gravel	Lean Clay	Fat Clay
2	Lean Clay	Soft to stiff Lean Clay with variable amounts of sand and gravel	Lean Clay with Sand	Clayey Sand
3	High Plasticity Clay	Soft to very stiff Fat Clay	Sandy Lean Clay	Clayey Sand with Gravel
4	Clayey Sand	Loose to Dense Clayey Sand	Sandy Fat Clay	Poorly-graded Sand with Clay
			Aggregate Base Course	

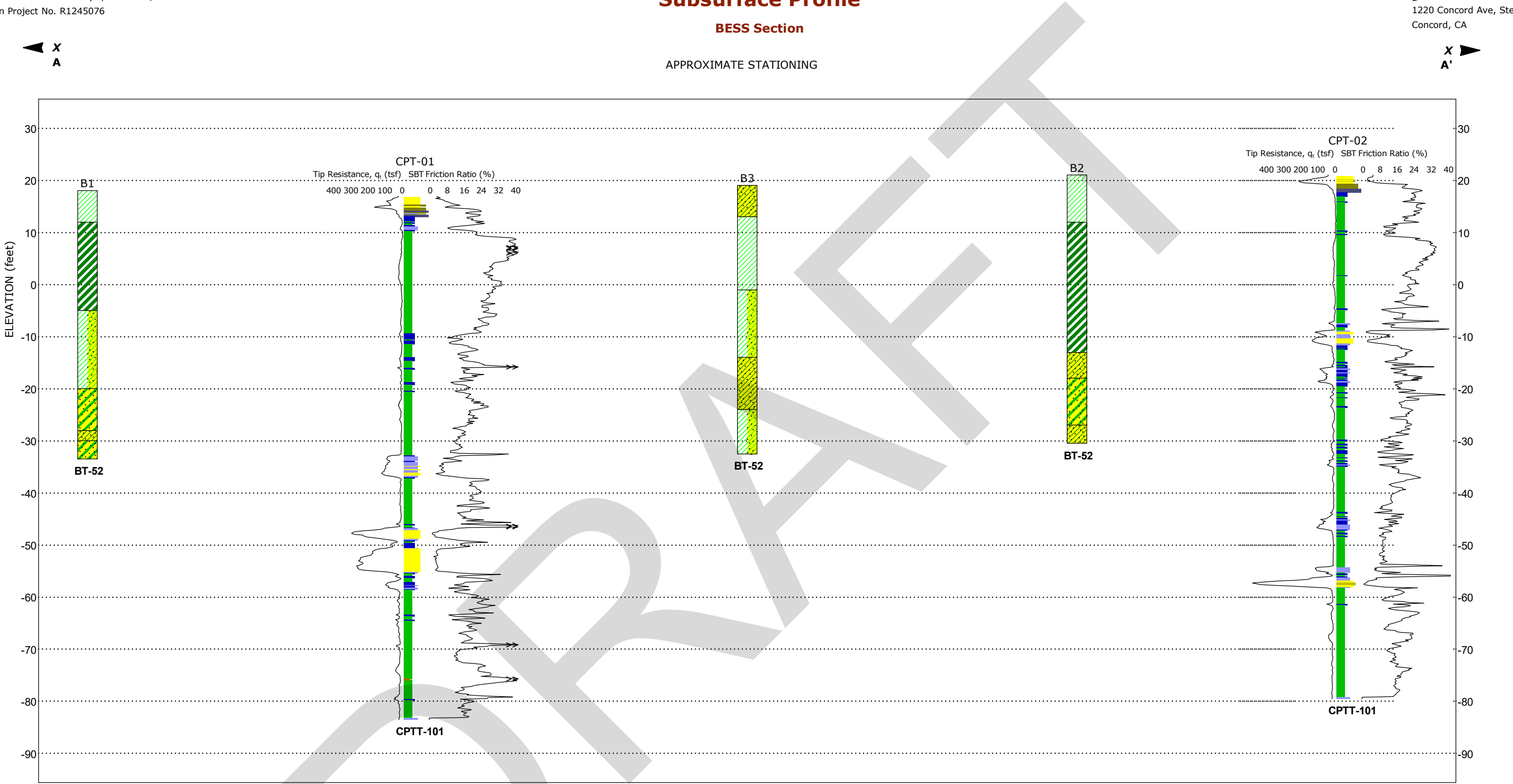
NOTES:

Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

Subsurface Profile

BESS Section

APPROXIMATE STATIONING

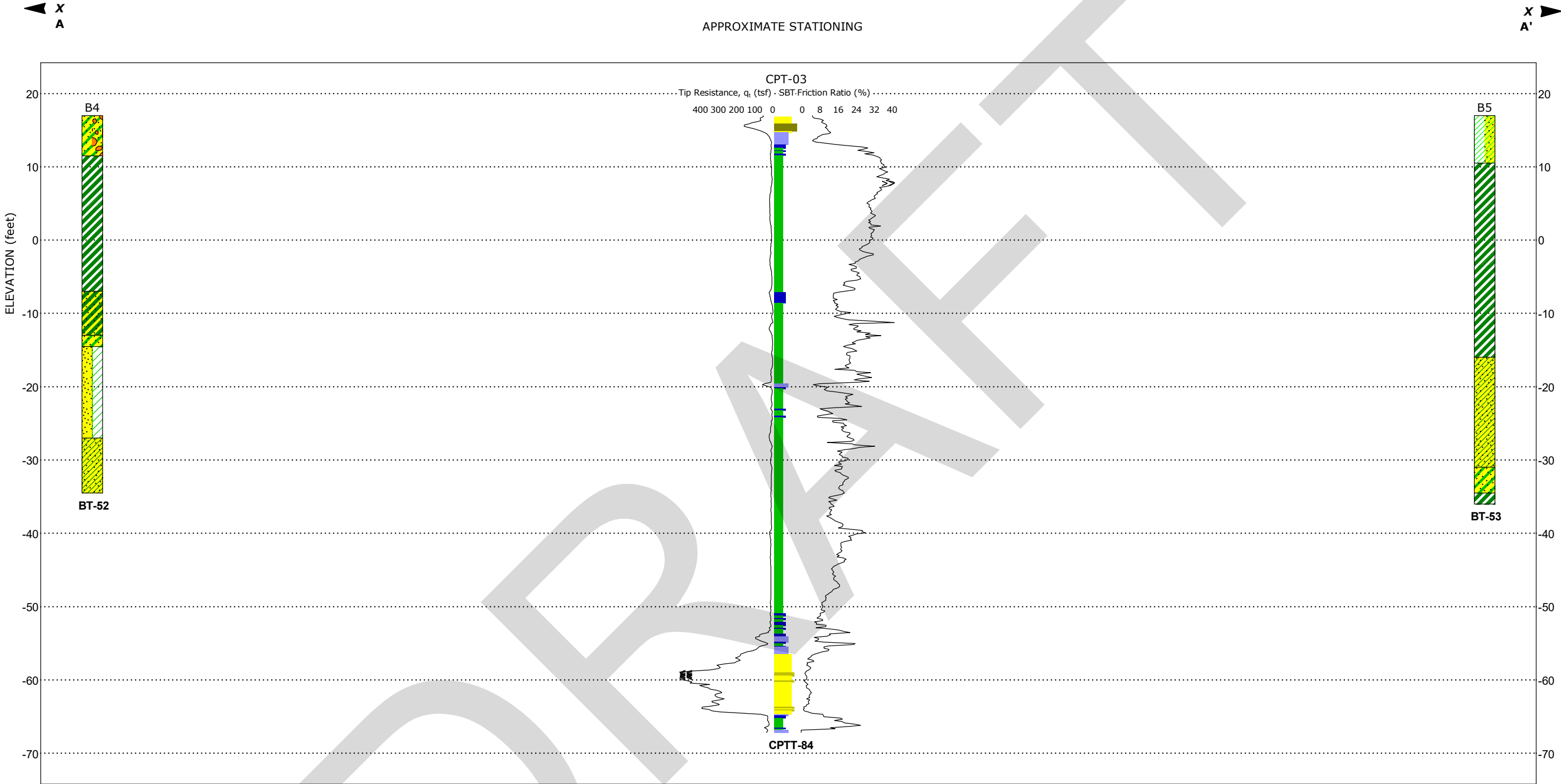


Notes	Water Level Observations	Explanation	Material Legend	CPT Soil Classification Graphic Color Code
See Exploration Plan for orientation of soil profile. See General Notes in Supporting Information for symbols and soil classifications. Soils profile provided for illustration purposes only. Soils between borings may differ AR - Auger Refusal BT - Boring Termination	Water Level Reading at time of drilling. Water Level Reading after drilling.	Moisture Content — %w Sampling (See General Notes) B1 LL PL Borehole Lithology AR BT Borehole Number Liquid and Plastic Limits Borehole Termination Type	Lean Clay Fat Clay Lean Clay with Sand Clayey Sand Sandy Lean Clay	1 Sensitive, fine grained 2 Organic soils - clay 3 Clay - silty clay to clay 4 Silt mixtures - clayey silt to silty clay 5 Sand mixtures - silty sand to sandy silt 6 Sands - clean sand to silty sand 7 Gravelly sand to dense sand 8 Very stiff sand to clayey sand 9 Very stiff fine grained

Subsurface Profile

Substation Section

APPROXIMATE STATIONING



Notes	Water Level Observations	Explanation	Material Legend	CPT Soil Classification Graphic Color Code
See Exploration Plan for orientation of soil profile. See General Notes in Supporting Information for symbols and soil classifications. Soils profile provided for illustration purposes only. Soils between borings may differ AR - Auger Refusal BT - Boring Termination	Water Level Reading at time of drilling. Water Level Reading after drilling.	Moisture Content — %w Sampling (See General Notes) B4 AR BT Borehole Number LL PL Borehole Lithology Borehole Termination Type	Clayey Sand with Gravel Fat Clay Sandy Fat Clay Clayey Sand Poorly-graded Sand with Clay Sandy Lean Clay Lean Clay with Sand	1 Sensitive, fine grained 2 Organic soils - clay 3 Clay - silty clay to clay 4 Silt mixtures - clayey silt to silty clay 5 Sand mixtures - silty sand to sandy silt 6 Sands - clean sand to silty sand 7 Gravelly sand to dense sand 8 Very stiff sand to clayey sand 9 Very stiff fine grained

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Attachments

Exploration and Testing Procedures

Field Exploration

Number of Borings/CPTs	Approximate Boring/CPT Depth (feet)	Location
3 borings	50	BESS Area
2 CPTs	100	
3 Borings	50	Substation Area
2 CPTs	100	

Boring Layout and Elevations: Terracon personnel provided the boring/CPT layout using handheld GPS equipment (estimated horizontal accuracy of about ± 10 feet) and referencing existing site features. Approximate ground surface elevations were estimated using Google Earth. If elevations and a more precise boring/CPT layout are desired, we recommend the exploration locations be surveyed.

Subsurface Exploration Procedures: We advanced the borings with a truck-mounted drill rig with continuous flight hollow stem augers and rotary wash boring techniques due to the presence of shallow groundwater and caving sand in the boreholes. Four samples were obtained in the upper 10 feet of each boring and at intervals of 5 feet thereafter. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. A 3-inch O.D. split-barrel sampling spoon with 2.5-inch I.D. tube lined (6-inch tubes) sampler was used for sampling in the upper 20 feet. We observed and recorded groundwater levels during drilling and sampling. For safety purposes, all borings and CPTs were backfilled with cement-grout after their completion in accordance with County requirements.

We also observed the boreholes while drilling and at the completion of drilling for the presence of groundwater. The groundwater levels are shown on the attached boring and CPT logs.

The sampling depths, penetration distances, and other sampling information was recorded on the field boring logs. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by a Geotechnical Engineer. Our exploration team prepared field boring logs as part of the drilling operations. These field logs included visual classifications of the materials observed during drilling and our interpretation of the subsurface conditions between samples. Final boring logs were

prepared from the field logs. The final boring logs represent the Geotechnical Engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

For the cone penetrometer testing, the CPT rig hydraulically pushed an instrumented cone through the soil while nearly continuous readings were recorded to a portable computer. The cone was equipped with electronic load cells to measure tip resistance and sleeve resistance and a pressure transducer to measure the generated ambient pore pressure. The face of the cone has an apex angle of 60° and an area of 15 cm². Digital data representing the tip resistance, friction resistance, pore water pressure, and probe inclination angle were recorded about every 2 centimeters while advancing through the ground at a rate between 1½ and 2½ centimeters per second. These measurements were correlated to various soil properties used for geotechnical design. In addition, seismic shear-wave velocities were measured every 1.5 to 3 meters (5 to 10 feet) of probe advancement in CPT-01. No soil samples were gathered through this subsurface investigation technique. CPT testing was conducted in general accordance with ASTM D5778 "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils."

Electrical Resistivity Testing: Soil electrical resistivity data was be obtained in accordance with ASTM G57 Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method. At the test location, two near perpendicular lines was tested. Electrode "a" spacings are summarized in the following table. Electrode spacing was adjusted to conform to site conditions.

No. of Test Locations	Electrode "a" Spacing (feet) ¹	Location
ER-1 and ER-2	0.5, 1, 1.5, 2, 3, 5, 10, 15, 20, 30, 45, 100, 150, and 200 feet.	BESS Areas
ER-3	0.5, 1, 1.5, 2, 3, 5, 10, 15, 20, 30, 45, 100, and 150 feet	Substation Area

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests. The laboratory testing program included the following types of tests:

- Moisture Content
- Dry Unit Weight
- Unconfined Compression

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- Atterberg Limits
- Grain Size Distribution
- One-Dimensional Consolidation
- Hveem Stabilometer (R-Value)
- Corrosivity Analysis

The laboratory testing program included examination of soil samples by an engineer. Based on the results of our field and laboratory programs, we described and classified the soil samples in accordance with the Unified Soil Classification System.

Thermal Resistivity: Laboratory thermal resistivity testing was performed by Geotherm on soil samples obtained during our field explorations from Borings B2 and B4 from 0 to 5 feet bgs, and on Tube samples obtained at depths of 5, 10, and 15 feet bgs, as requested by Calpine. The thermal resistivity testing was performed in general accordance with the IEEE standard. The samples were tested at 90 percent of the maximum dry density using ASTM D698 (Standard Proctor), as requested. A dry-out curve was performed for each thermal resistivity test.

Site Location and Exploration Plans

Contents:

Site Location Plan

Exploration Plan

Note: All attachments are one page unless otherwise noted.

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Site Location

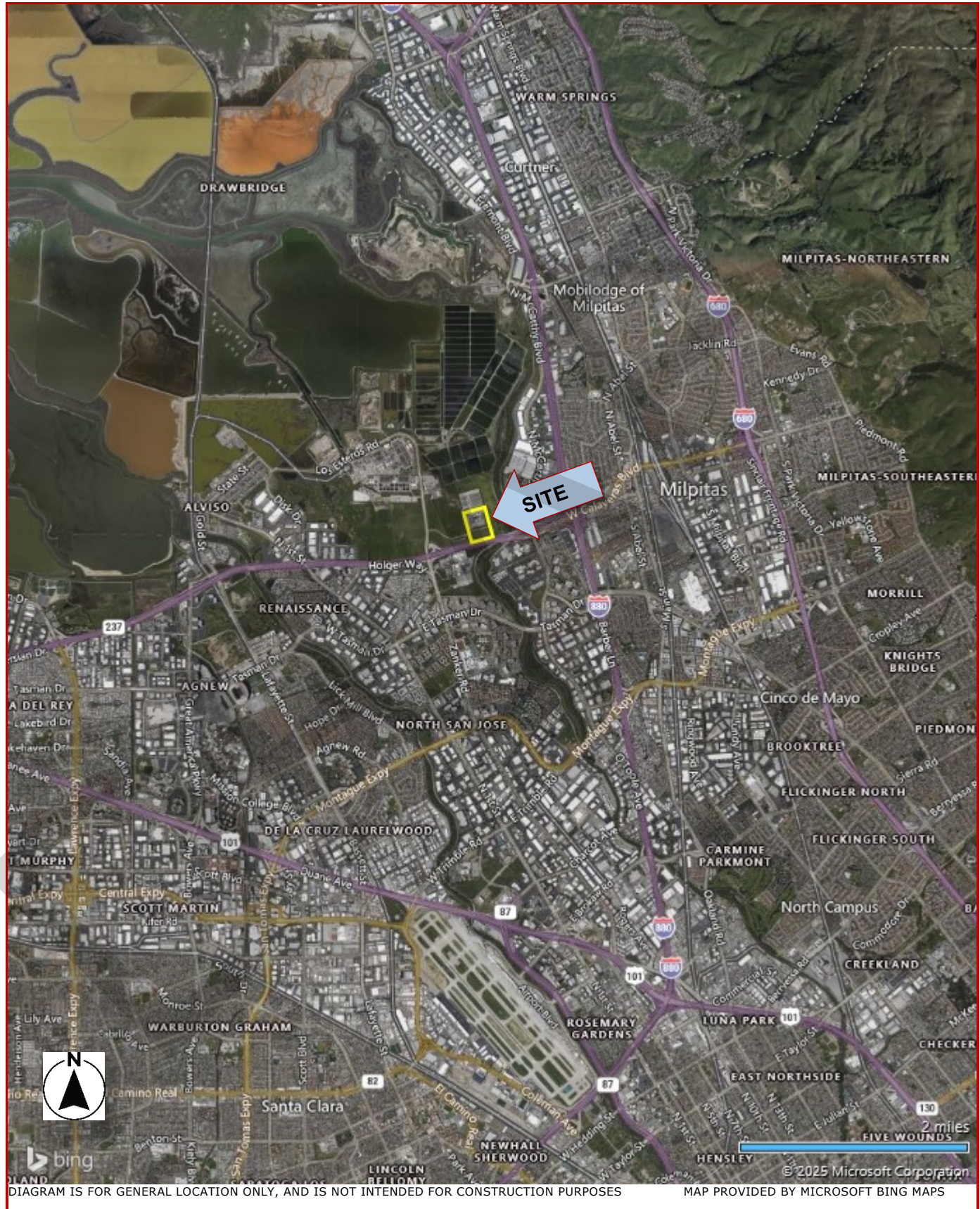


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

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Exploration Plan



Exploration and Laboratory Results

Contents:

Boring Logs (B1 through B6)
CPT Logs (CPT-01 through CPT-04)
Atterberg Limit Results
Grain Size Distribution (3 pages)
Unconfined Compression Tests (6 pages)
One-Dimensional Consolidation Tests (2 pages)
R-Value Test
Results of Corrosion Analysis
Electrical Resistivity Surveys (ER-1, ER-2, and ER-3)
Thermal Resistivity Report (4 pages)
Seismic Shear Wave Velocities

Note: All attachments are one page unless otherwise noted.

Boring Log No. B1

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4232° Longitude: -121.9323° Depth (Ft.) Approximate Elevation: 18 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	Percent Fines
											LL-PL-PI	
2		LEAN CLAY (CL) , trace fine sand, brown, medium stiff	5			4-5-6	2.5		18.6	93		
3		FAT CLAY (CH) , trace fine sand, dark brown, stiff, fine sand brown	10			2-4-6	2.0	1.09	21.6	92	35-19-16	92.1
			15			6-8-12	2.5		21.8	100		
2		LEAN CLAY WITH SAND (CL) , brown, stiff	25			5-8-11	3.5		21.9	100		
2		LEAN CLAY WITH SAND (CL) , brown, stiff	20			7-8-11	2.0		26.3	92	53-18-35	
2		LEAN CLAY WITH SAND (CL) , brown, stiff	23.0			2-3-4 N=7			32.9			
2		LEAN CLAY WITH SAND (CL) , brown, stiff	25			2-4-6 N=10			19.1		25-13-12	75.5

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-19-2025

Boring Completed

06-19-2025

Notes

Advancement Method

6-Inch Hollow Stem Auger to 15' bgs, then Mud Rotary to Depth

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B1

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4232° Longitude: -121.9323° Depth (Ft.) Approximate Elevation: 18 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	Percent Fines
											LL-PL-PI	
2		LEAN CLAY WITH SAND (CL) , brown, stiff (continued)										
		medium stiff	30			3-3-3 N=6			20.8			
		stiff	35			4-6-7 N=13			22.3			
		38.0 -20										
4		CLAYEY SAND (SC) , fine grained, brown, loose	40			2-2-2 N=4						45.0
		medium dense	45			3-7-11 N=18			21.7			
2		SANDY LEAN CLAY (CL) , with fine sand, brown, very stiff	46.0 -28									
		48.0 -30										
4		CLAYEY SAND (SC) , fine grained, brown, medium dense	50			5-9-12 N=21			21.9			
		51.5 -33.5										
		Boring Terminated at 51.5 Feet										

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-19-2025

Boring Completed

06-19-2025

Notes

Advancement Method

6-Inch Hollow Stem Auger to 15' bgs, then Mud Rotary to Depth

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B2

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4238° Longitude: -121.9304° Depth (Ft.) Approximate Elevation: 21 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	Percent Fines
											LL-PL-PI	
1		FILL - LEAN CLAY (CL) , with fine sand, dark brown, medium stiff no sample recovery due to 3 inch cobble in sampler plastic debris	5			9-7-6	0.5			27.4	84	
						2-2-3						
						2-2-2						
3		FAT CLAY (CH) , trace fine sand, brown to gray, soft to medium stiff, carbonates stiff rust, carbonates medium stiff	9.0	12								
			10									
			15			6-11-10	1.75			19.8	103	55-16-39
			20			4-8-11	3.5			23.0	98	
			25			1-2-2 N=4				33.0		

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-20-2025

Boring Completed

06-20-2025

Notes

Advancement Method

6-Inch Hollow Stem Auger to 20' bgs, then Mud Rotary to Depth

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B2

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4238° Longitude: -121.9304° Depth (Ft.) Approximate Elevation: 21 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	Percent Fines
											LL-PL-PI	
3		FAT CLAY (CH) , trace fine sand, brown to gray, soft to medium stiff, carbonates (<i>continued</i>)										
			30			2-3-4 N=7			26.7			
2		SANDY LEAN CLAY (CL) , with fine grained sand, brown, medium stiff										
			35			2-2-5 N=7			23.2			63.0
4		CLAYEY SAND (SC) , fine to coarse grained, brown to gray, medium dense, trace fine gravel up to 1/4-inch										
			40			10-10-13 N=23			25.7			18.7
			45			8-13-15 N=28			14.5			
			50			4-3-3 N=6			16.6			
		Boring Terminated at 51.5 Feet										

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-20-2025

Boring Completed

06-20-2025

Notes

Advancement Method

6-Inch Hollow Stem Auger to 20' bgs, then Mud Rotary to Depth

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B3

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4228° Longitude: -121.9311° Depth (Ft.) Approximate Elevation: 19 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	Percent Fines
											LL-PL-PI	
1		FILL - SANDY LEAN CLAY (CL) , fine sand, brown, very stiff plastic debris	5			6-11-11			13.3	102		
						7-8-8			17.4	101		
						4-6-10			23.1	96		
3		LEAN CLAY (CL) , trace fine sand, dark brown, stiff reddish brown, very stiff	10			5-8-11	3.0	3.08	20.8	108	49-18-31	95.6
						8-10-17	4.25		18.6	104		
2		LEAN CLAY WITH SAND (CL) , with fine sand stiff soft, clayey sand seam	20			5-6-8	1.0		23.3		35-15-20	
						2-1-1 N=2			23.4			

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-20-2025

Boring Completed

06-20-2025

Notes

Advancement Method

6-Inch Hollow Stem Auger to 20' bgs, then Mud Rotary to Depth

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B3

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4228° Longitude: -121.9311° Depth (Ft.) Approximate Elevation: 19 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	Percent Fines
											LL-PL-PI	
2		LEAN CLAY WITH SAND (CL) , with fine sand (continued) very low recovery	30			1-1-1 N=2			29.6			
		33.0	35			3-1-3 N=4			22.5			
		SANDY LEAN CLAY (CL) , with fine sand, brown, soft to medium stiff										
			40			1-2-3 N=5			25.1			
		43.0	45			2-2-3 N=5			34.5			76.2
		LEAN CLAY WITH SAND (CL) , with fine sand, brown, soft to medium stiff										
		51.5	50			2-2-2 N=4			20.0			
		Boring Terminated at 51.5 Feet										

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-20-2025

Boring Completed

06-20-2025

Notes

Advancement Method

6-Inch Hollow Stem Auger to 20' bgs, then Mud Rotary to Depth

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B4

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4246° Longitude: -121.9305° Depth (Ft.) Approximate Elevation: 17 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	
											LL-PL-PI	Percent Fines
1		FILL - CLAYEY SAND WITH GRAVEL (SC) , fine to coarse grained, brown, loose, subrounded gravel up to 1 inch	5.5			2-2-2			7.6	101		
3		FAT CLAY (CH) , trace fine sand, dark brown, stiff very stiff gray with red, rust	11.5			2-5-7	2.5		21.2	94		
			10			6-6-10	2.75	2.31	23.1	100	61-20-41	98.7
			15			6-12-12	3.5		21.9	98		
			20			6-8-9 N=17			23.2			
			24.0			2-3-3 N=6			24.5			
		SANDY FAT CLAY (CH) , with fine sand, olive brown, medium stiff	-7									

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig
CME 75

Hammer Type
Automatic

Driller
Terracon

Logged by
EF

Boring Started
06-19-2025

Boring Completed
06-19-2025

Notes

Advancement Method

6-Inch Hollow Stem Auger to 35' bgs, then Mud Rotary to Depth

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B4

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4246° Longitude: -121.9305° Depth (Ft.) Approximate Elevation: 17 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	
											LL-PL-PI	Percent Fines
3		SANDY FAT CLAY (CH) , with fine sand, olive brown, medium stiff (<i>continued</i>)	30.0 -13									
		CLAYEY SAND (SC) , fine grained, gray, dense	31.5 -14.5			5-17-16 N=33			19.7			
4		POORLY GRADED SAND WITH CLAY (SP-SC) , fine grained, gray, dense, heaving sand				11-24-18 N=42			24.8			
		loose	40			4-5-3 N=8			25.2			11.6
			44.0 -27			3-3-3 N=6			21.1			
2		SANDY LEAN CLAY (CL) , fine sand, light brown, medium stiff				9-10-7 N=17			22.5			
		gray, very stiff	51.5 -34.5									
		Boring Terminated at 51.5 Feet										

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-19-2025

Boring Completed

06-19-2025

Notes

Advancement Method

6-Inch Hollow Stem Auger to 35' bgs, then Mud Rotary to Depth

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B5

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4256° Longitude: -121.9308° Depth (Ft.) Approximate Elevation: 17 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	
											LL-PL-PI	Percent Fines
1		FILL - LEAN CLAY WITH SAND (CL) , with fine sand, brown, stiff, trace roots	5			24-11-9			6.7	101		
						3-4-8 N=12			19.7			
3		FAT CLAY (CH) , trace fine sand, brown, very stiff	6.5			6-10-12	3.5	3.13	19.6	104	52-16-36	92.9
			10									
			15			5-4-5	1.0		23.8	97		
			20			5-7-8	1.5		20.3	103		
			25			3-4-5 N=9			25.6			

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-18-2025

Boring Completed

06-18-2025

Notes

Advancement Method

Mud Rotary

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B5

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4256° Longitude: -121.9308° Depth (Ft.) Approximate Elevation: 17 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	
											LL-PL-PI	Percent Fines
3		FAT CLAY (CH) , trace fine sand, brown, very stiff (continued)										
		stiff	30			3-4-4 N=8			23.6			
2		SANDY LEAN CLAY (CL) , with fine sand, light brown, stiff	33.0 -16									
			35			7-6-6	0.75	0.59	20.9	110	29-14-15	65.9
		medium stiff	40			2-2-3 N=5			20.7			
		stiff	45			3-7-7 N=14			17.2			
4		CLAYEY SAND (SC) , fine grained, gray, loose	48.0 -31									
			50			8-5-4 N=9			19.3			
3		FAT CLAY (CH) , trace fine sand, gray, stiff	51.5 -34.5			6-6-7 N=13			31.3			
		Boring Terminated at 53 Feet	53.0 -36									

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-18-2025

Boring Completed

06-18-2025

Notes

Advancement Method

Mud Rotary

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B6

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4264° Longitude: -121.9311° Depth (Ft.) Approximate Elevation: 16 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	Percent Fines
											LL-PL-PI	
		FILL - AGGREGATE BASE COURSE , Approximately 3 feet										
		3.0 13										
2		SANDY LEAN CLAY (CL) , with fine to medium sand, dark brown, medium stiff				7-5-5	1.5		18.8	96		
		4.5 11.5										
3		CLAYEY SAND (SC) , fine to coarse grained, dark brown, loose, trace fine subangular gravel up to 1/4-inch	5			2-3-2			14.4	103		
		7.0 9										
		LEAN CLAY (CL) , trace fine sand, dark brown, stiff				5-8-10	2.0		21.1	102		
		brown, very stiff	10			7-11-16	3.5		20.2	105		
		reddish brown, medium stiff	15			4-4-4	0.5		25.6	93	38-16-22	
2												
		stiff	20			6-8-10	2.0		26.6			
			25			5-6-7 N=13			21.6			

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-18-2025

Boring Completed

06-19-2025

Notes

Advancement Method

Mud Rotary

Abandonment Method

Boring backfilled with cement grout upon completion

Boring Log No. B6

Model Layer	Graphic Log	Location: See Exploration Plan Latitude: 37.4264° Longitude: -121.9311° Depth (Ft.) Approximate Elevation: 16 (Ft.)	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	HP (tsf)	Unconfined Compressive Strength (tsf)	Water Content (%)	Dry Unit Weight (pcf)	Atterberg Limits	
											LL-PL-PI	Percent Fines
2		LEAN CLAY (CL) , trace fine sand, dark brown, stiff (<i>continued</i>)	28.0									
		SANDY LEAN CLAY (CL) , with fine sand, brown, medium stiff	-12									
			30	X		1-2-4 N=6			22.9			58.5
			35	X		2-2-3 N=5			20.7			
		stiff	40	X		2-4-6 N=10			23.7			
		gray	45	X		6-7-7 N=14			18.2			
			50	X		4-5-6 N=11			24.8			
		51.5	-35.5									
		Boring Terminated at 51.5 Feet										

See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data (If any).

See [Supporting Information](#) for explanation of symbols and abbreviations.

Elevation Reference: Estimated From Google Earth Pro

Water Level Observations

Water level not determined due to drilling method

Drill Rig

CME 75

Hammer Type

Automatic

Driller

Terracon

Logged by

EF

Boring Started

06-18-2025

Boring Completed

06-19-2025

Notes

Advancement Method

Mud Rotary

Abandonment Method

Boring backfilled with cement grout upon completion

CPT Sounding ID CPT-01

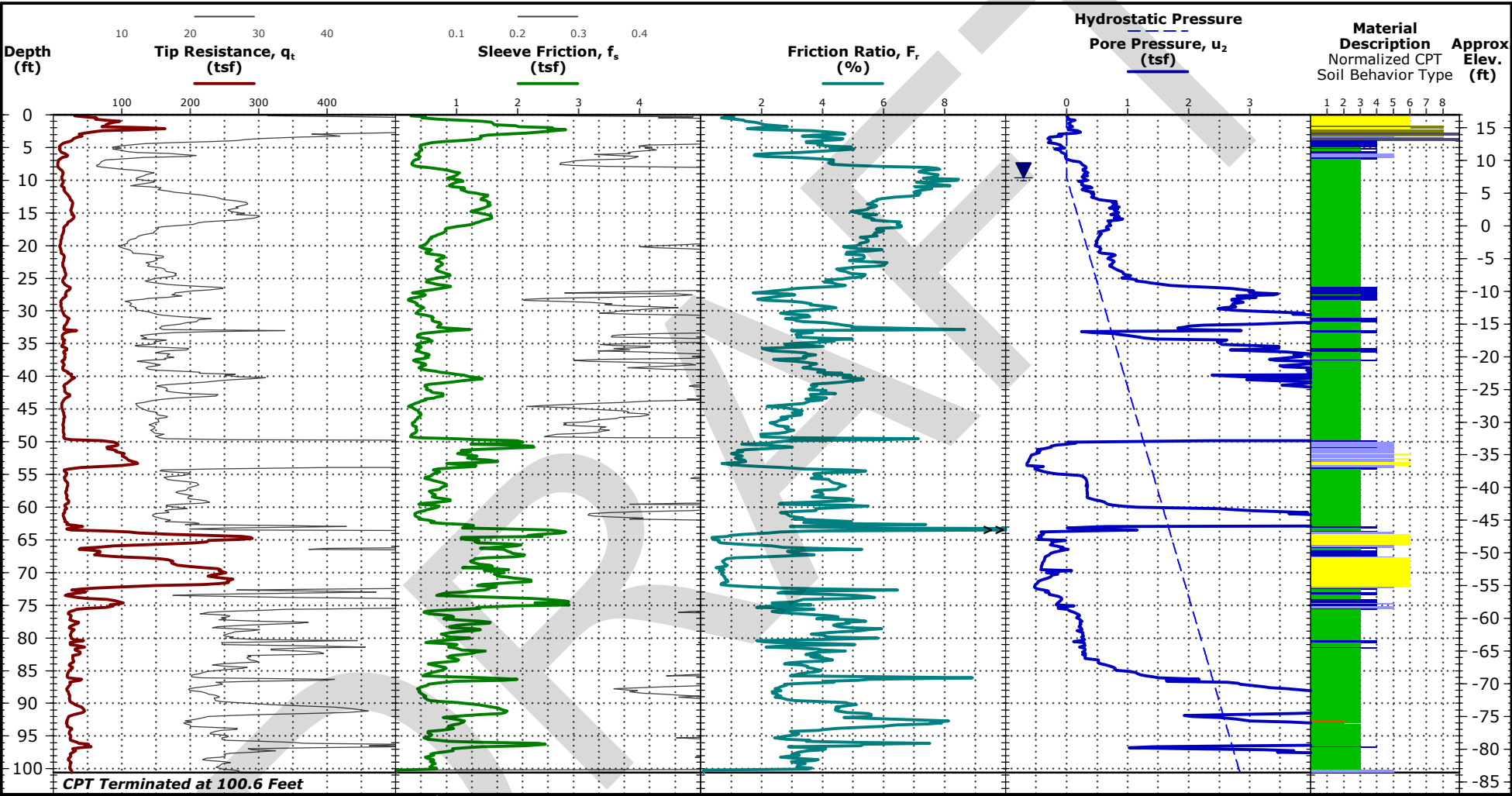
Latitude: 37.4226° Longitude: -121.9320°



1220 Concord Ave, Ste 450
Concord, CA

CPT Started: 6/18/2025
CPT Completed: 6/18/2025

Elevation: 17 (ft) +/-
Elevation Reference: Estimated From Google Earth Pro



See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data, if any.
See [Supporting Information](#) for explanation of symbols and abbreviations.

Notes
Test Location: See [Exploration Plan](#)

CPT Equipment
CPT Rig: 25-Ton Truck
Operator: Middle Earth
CPT sensor calibration reports available upon request
Probe No. DPG1556

Water Level Observation
▼ 9.6 ft estimated water depth
(used in normalizations and correlations)

Normalized Soil Behavior Type (Robertson 1990)

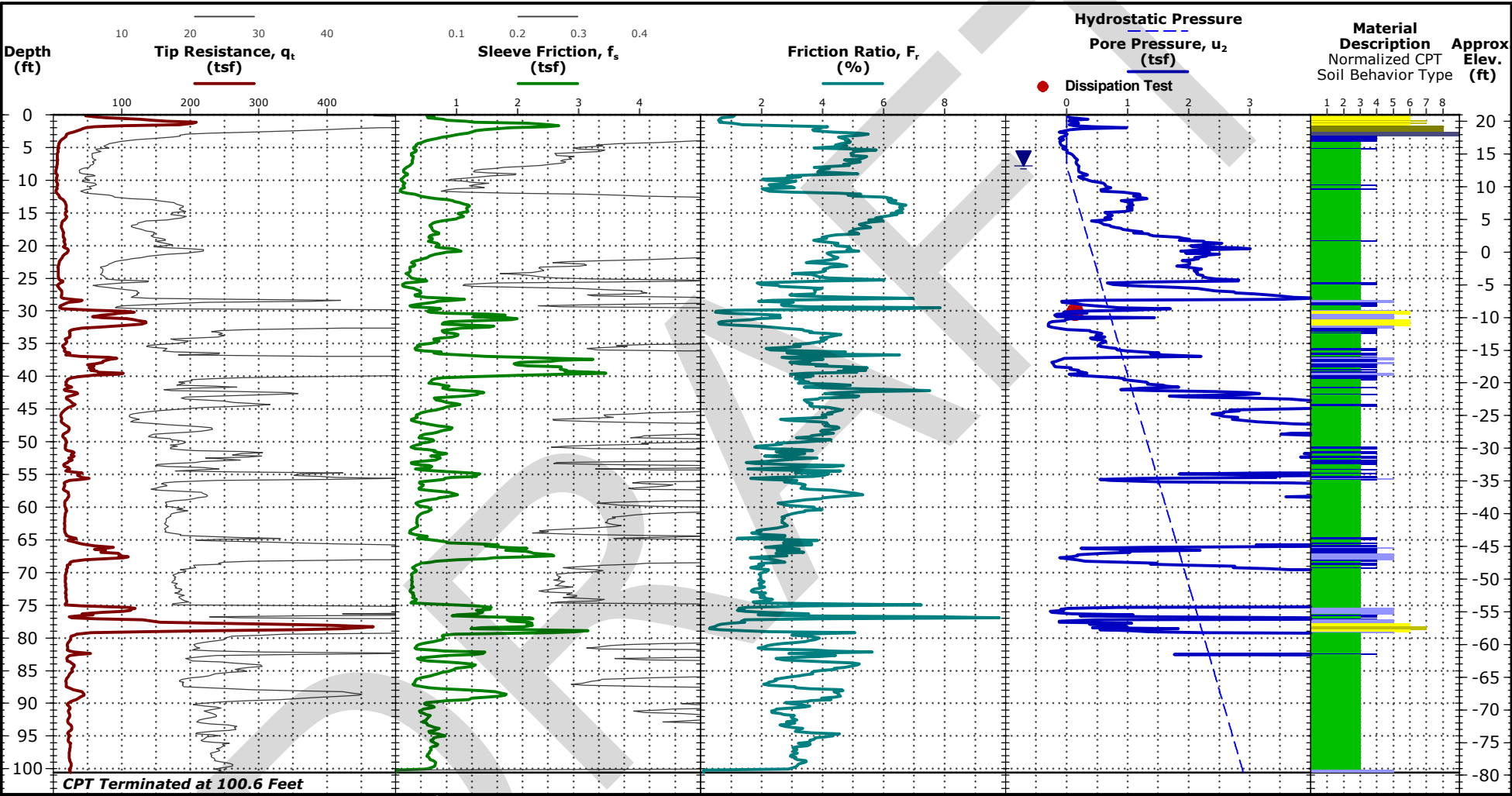
1	Sensitive, fine grained
2	Organic soils - clay
3	Clay - silty clay to clay
4	Silt mixtures - clayey silt to silty clay
5	Sand mixtures - silty sand to sandy silt
6	Sands - clean sand to silty sand
7	Gravelly sand to dense sand
8	Very stiff sand to clayey sand
9	Very stiff fine grained

CPT Sounding ID CPT-02

Latitude: 37.4236° Longitude: -121.9303°

Elevation: 21 (ft) +/-
Elevation Reference: Estimated From Google Earth Pro

CPT Started: 6/18/2025
CPT Completed: 6/18/2025



See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data, if any.
See [Supporting Information](#) for explanation of symbols and abbreviations.

Notes

Test Location: See [Exploration Plan](#)

CPT Equipment

CPT Rig: 25-Ton Truck
Operator: Middle Earth
CPT sensor calibration reports available upon request
Probe No. DPG1556

Water Level Observation

▼ 7.8 ft estimated water depth
(used in normalizations and correlations)

Normalized Soil Behavior Type (Robertson 1990)

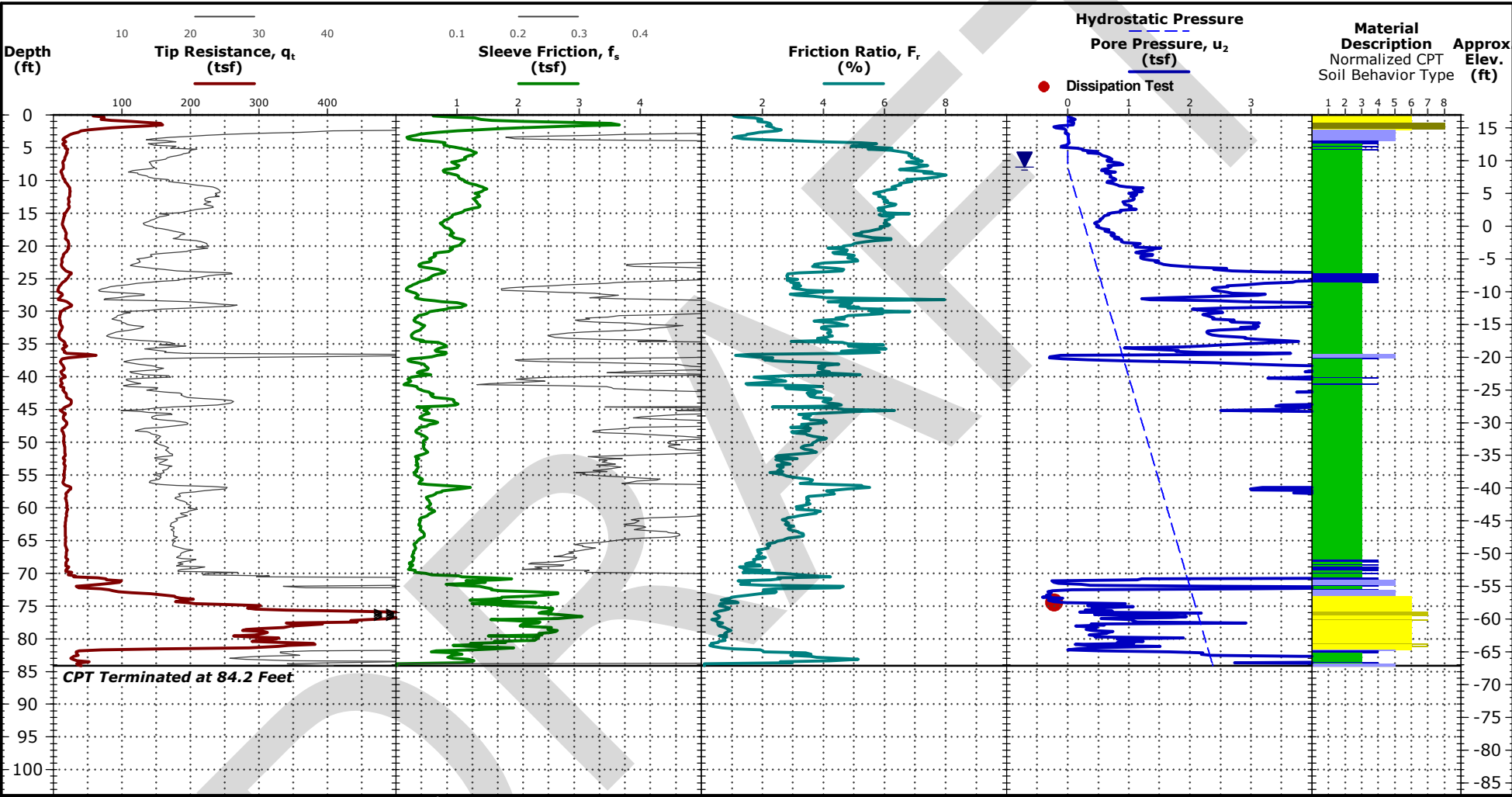
- 1 Sensitive, fine grained
- 2 Organic soils - clay
- 3 Clay - silty clay to clay
- 4 Silt mixtures - clayey silt to silty clay
- 5 Sand mixtures - silty sand to sandy silt
- 6 Sands - clean sand to silty sand
- 7 Gravely sand to dense sand
- 8 Very stiff sand to clayey sand
- 9 Very stiff fine grained

CPT Sounding ID CPT-03

Latitude: 37.4249° Longitude: -121.9306°

Elevation: 17 (ft) +/-
Elevation Reference: Estimated From Google Earth Pro

CPT Started: 6/18/2025
CPT Completed: 6/18/2025



See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data, if any.
See [Supporting Information](#) for explanation of symbols and abbreviations.

Notes
Test Location: See [Exploration Plan](#)

CPT Equipment
CPT Rig: 25-Ton Truck
Operator: Middle Earth
CPT sensor calibration reports available upon request
Probe No. DPG1556

Water Level Observation
▼ 8 ft estimated water depth
(used in normalizations and correlations)

Normalized Soil Behavior Type
(Robertson 1990)

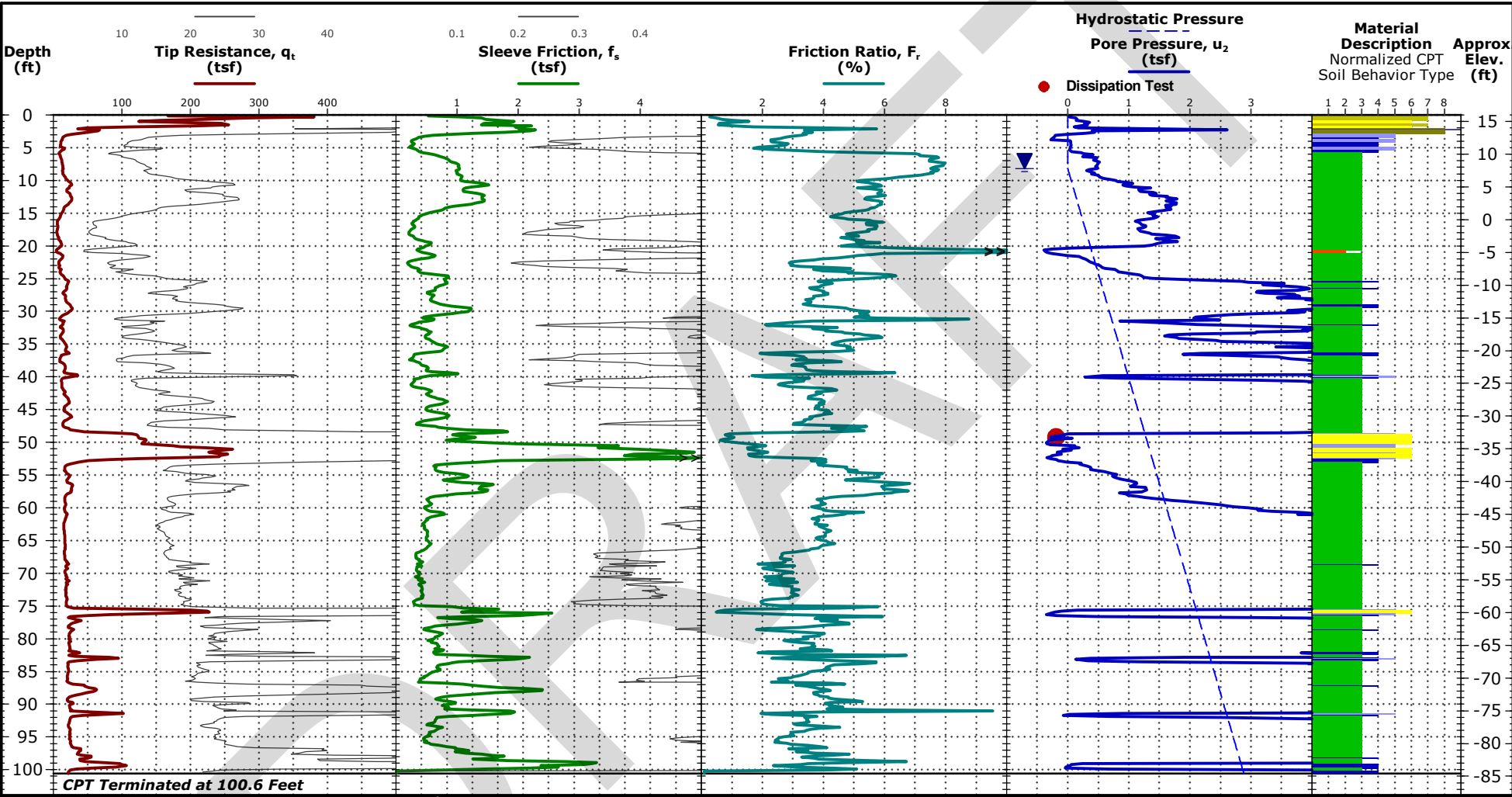
- 1 Sensitive, fine grained
- 2 Organic soils - clay
- 3 Clay - silty clay to clay
- 4 Silt mixtures - clayey silt to silty clay
- 5 Sand mixtures - silty sand to sandy silt
- 6 Sands - clean sand to silty sand
- 7 Gravely sand to dense sand
- 8 Very stiff sand to clayey sand
- 9 Very stiff fine grained

CPT Sounding ID CPT-04

Elevation: 16 (ft) +/-
Elevation Reference: Estimated From Google Earth Pro

Latitude: 37.4264° Longitude: -121.9311°

CPT Started: 6/18/2025
CPT Completed: 6/18/2025



See [Exploration and Testing Procedures](#) for a description of field and laboratory procedures used and additional data, if any.
See [Supporting Information](#) for explanation of symbols and abbreviations.

Notes

Test Location: See [Exploration Plan](#)

CPT Equipment

CPT Rig: 25-Ton Truck
Operator: Middle Earth
CPT sensor calibration reports available upon request
Probe No. DPG1556

Water Level Observation

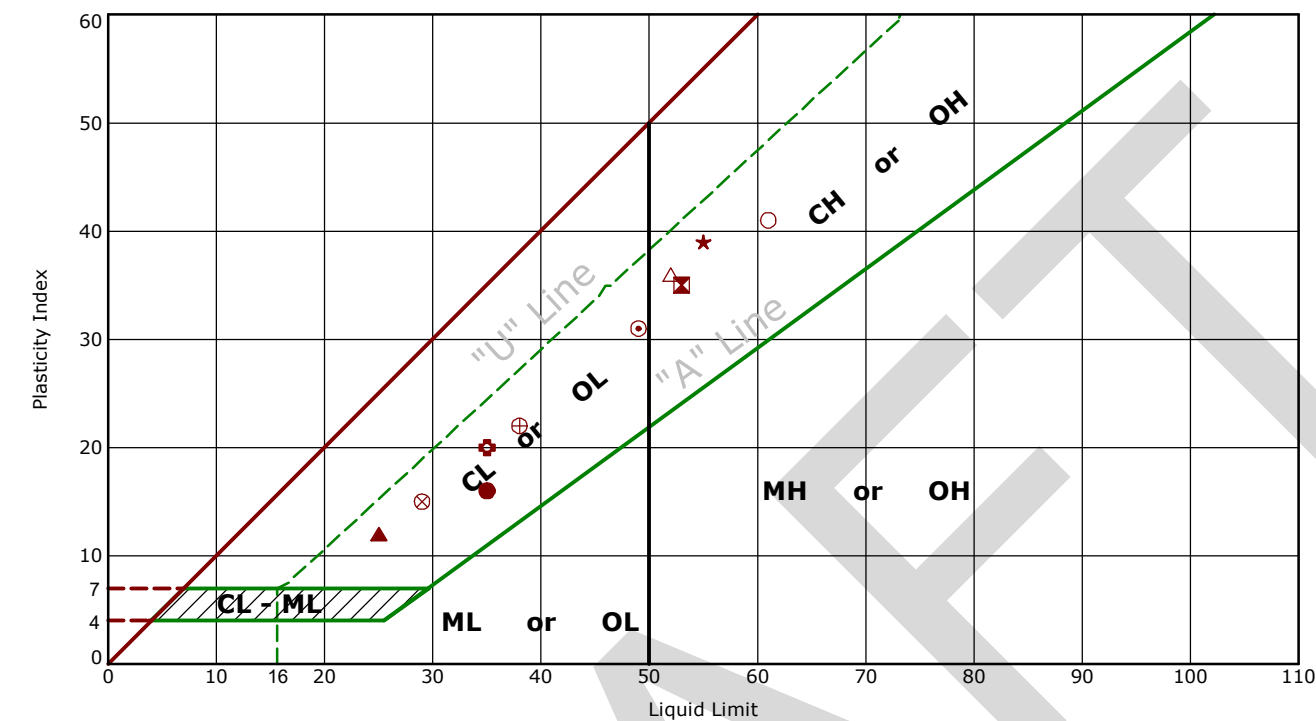
8.2 ft estimated water depth
(used in normalizations and correlations)

Normalized Soil Behavior Type (Robertson 1990)

- 1 Sensitive, fine grained
- 2 Organic soils - clay
- 3 Clay - silty clay to clay
- 4 Silt mixtures - clayey silt to silty clay
- 5 Sand mixtures - silty sand to sandy silt
- 6 Sands - clean sand to silty sand
- 7 Gravely sand to dense sand
- 8 Very stiff sand to clayey sand
- 9 Very stiff fine grained

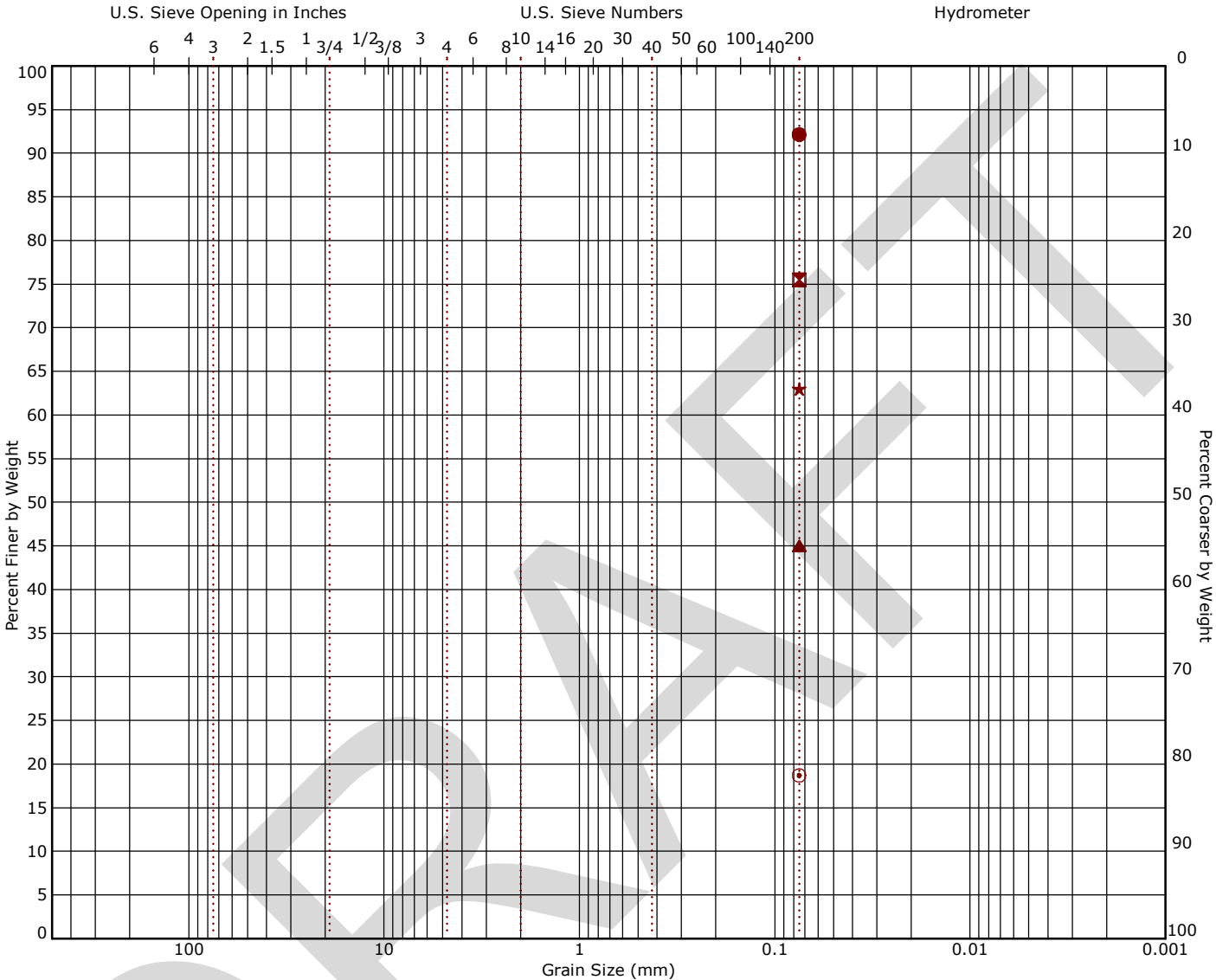
Atterberg Limit Results

ASTM D4318

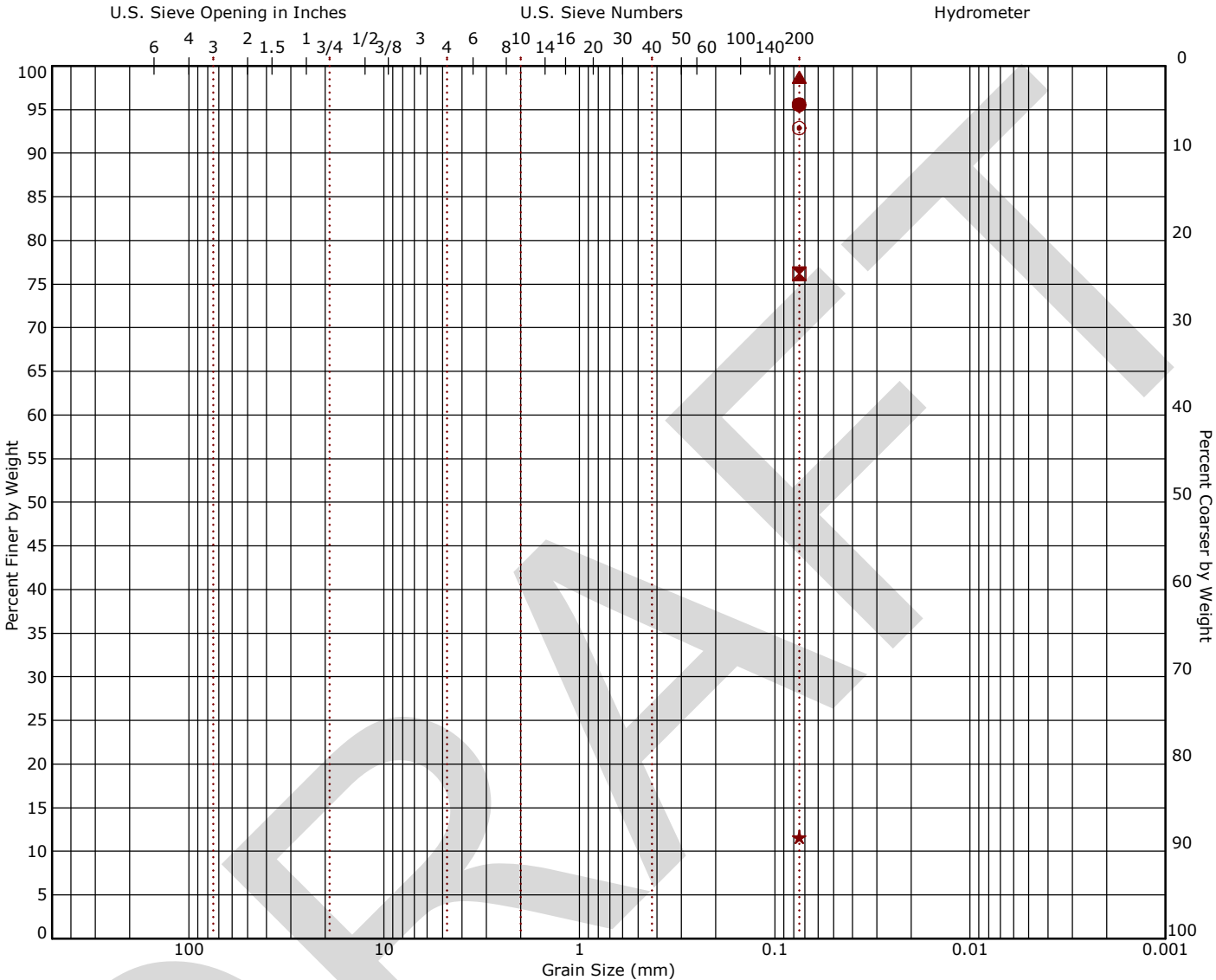


	Boring ID	Depth (Ft)	LL	PL	PI	Fines	USCS	Description
●	B1	5 - 6.5	35	19	16	92.1	CL	LEAN CLAY
⊠	B1	15 - 16.5	53	18	35		CH	FAT CLAY
▲	B1	25 - 26.5	25	13	12	75.5	CL	LEAN CLAY with SAND
★	B2	15 - 16.5	55	16	39		CH	FAT CLAY
⊙	B3	7.5 - 9	49	18	31	95.6	CL	LEAN CLAY
⊕	B3	20 - 21.5	35	15	20		CL	LEAN CLAY
○	B4	7.5 - 9	61	20	41	98.7	CH	FAT CLAY
△	B5	7.5 - 9	52	16	36	92.9	CH	FAT CLAY
⊗	B5	35 - 36.5	29	14	15	65.9	CL	SANDY LEAN CLAY
⊕	B6	15 - 16.5	38	16	22		CL	LEAN CLAY

Grain Size Distribution
ASTM D422 / ASTM C136 / AASHTO T27

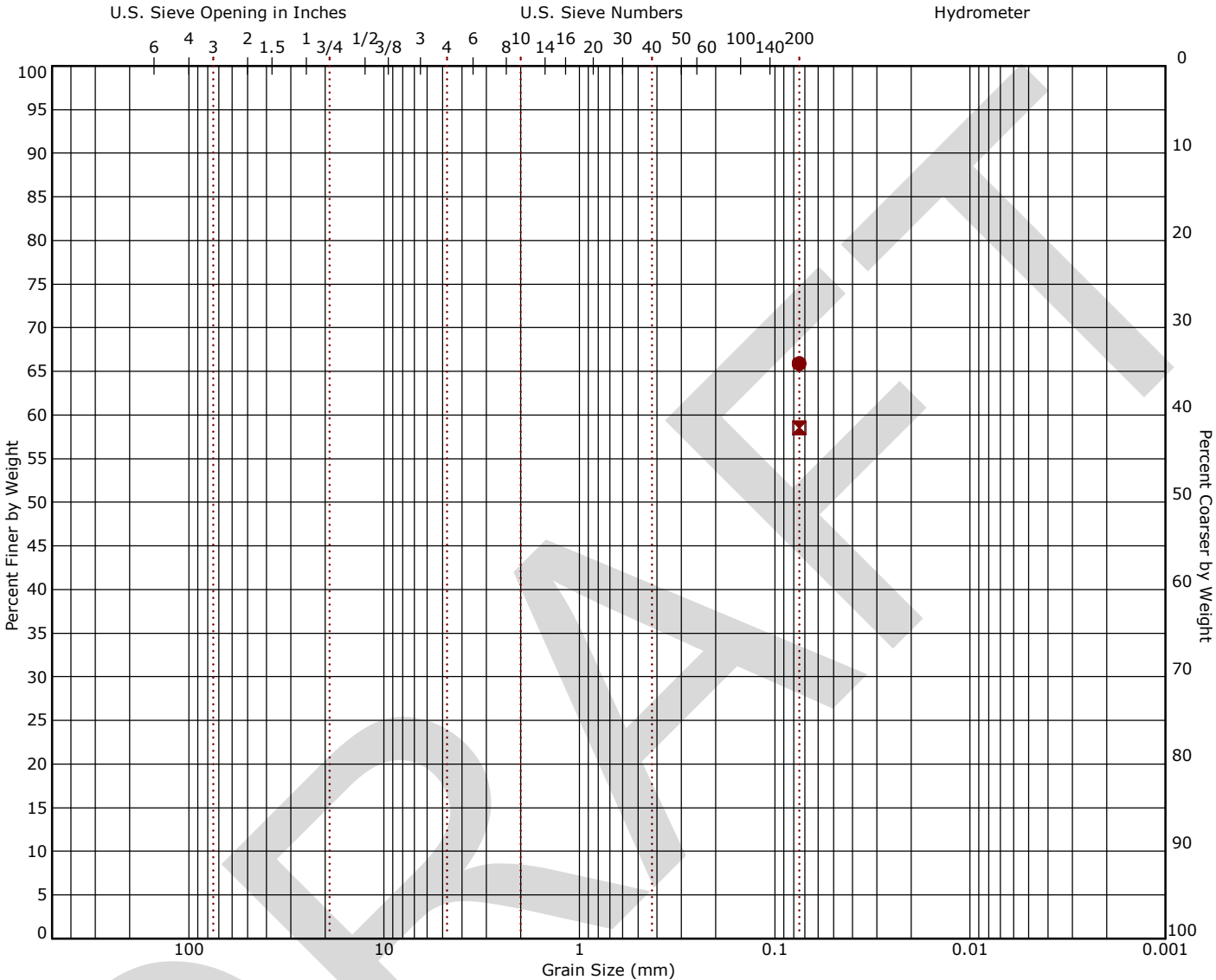


Grain Size Distribution
ASTM D422 / ASTM C136 / AASHTO T27



		Cobbles		Gravel		Sand			Silt or Clay					
			coarse		fine	coarse	medium	fine						
Boring ID		Depth (Ft)	Description						USCS	LL	PL	PI	Cc	Cu
●	B3	7.5 - 9	LEAN CLAY						CL	49	18	31		
⊠	B3	45 - 46.5	SANDY LEAN CLAY						CL					
▲	B4	7.5 - 9	FAT CLAY						CH	61	20	41		
★	B4	40 - 41.5	POORLY GRADED SAND with CLAY						SP-SC					
⊙	B5	7.5 - 9	FAT CLAY						CH	52	16	36		
Boring ID		Depth (Ft)	D ₁₀₀	D ₆₀	D ₃₀	D ₁₀	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay		
●	B3	7.5 - 9	0.075							95.6				
⊠	B3	45 - 46.5	0.075							76.2				
▲	B4	7.5 - 9	0.075							98.7				
★	B4	40 - 41.5	0.075							11.6				
⊙	B5	7.5 - 9	0.075							92.9				

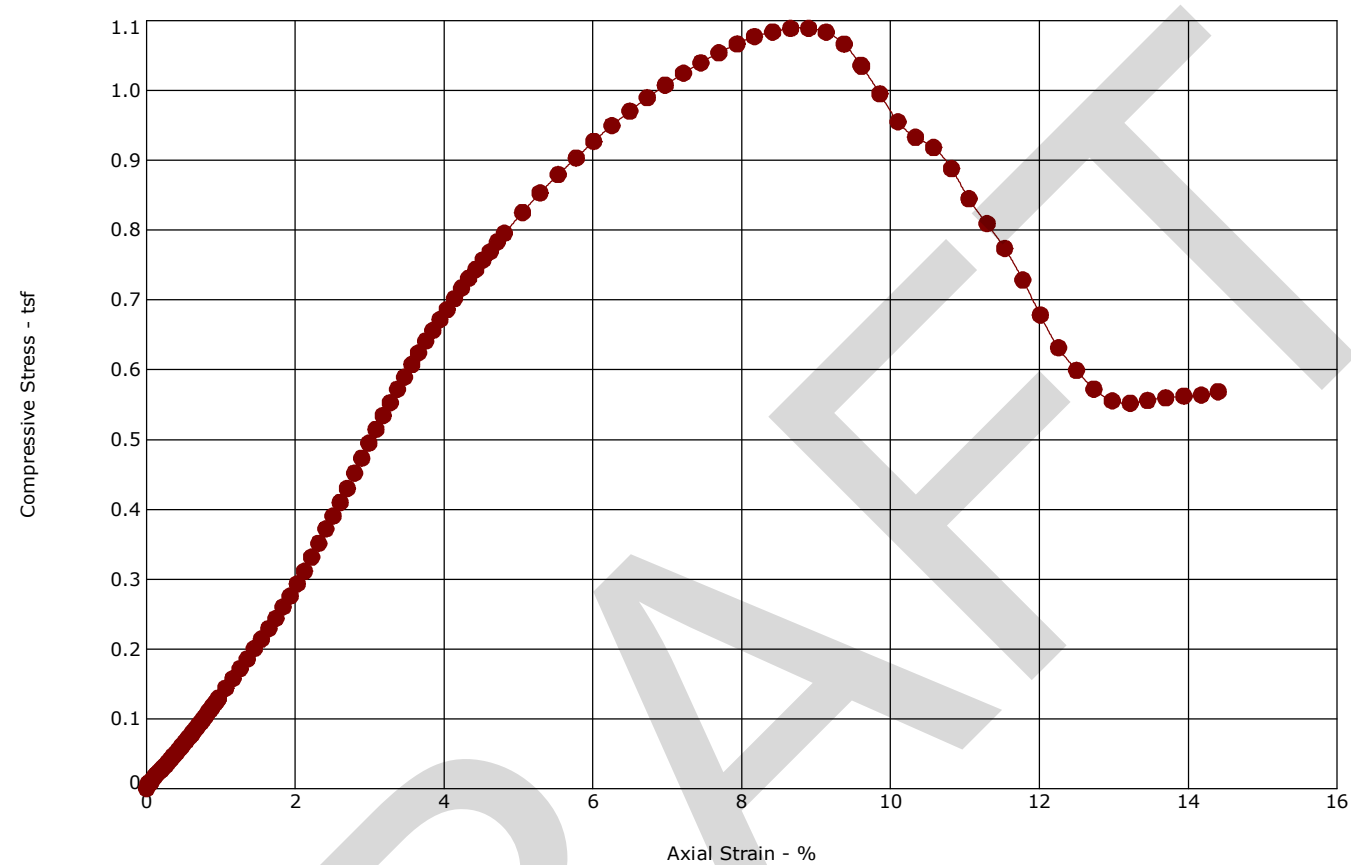
Grain Size Distribution
ASTM D422 / ASTM C136 / AASHTO T27



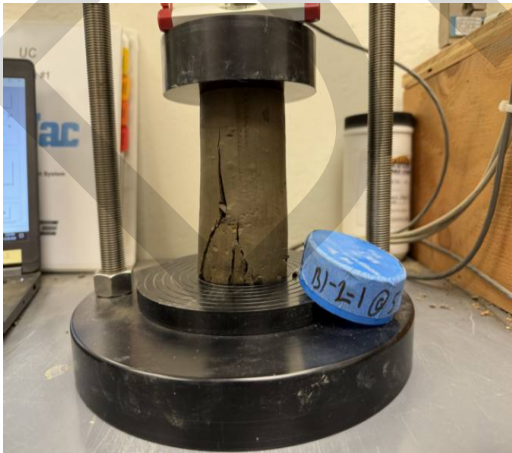
Cobbles		Gravel		Sand			Silt or Clay						
		coarse	fine	coarse	medium	fine							
Boring ID	Depth (Ft)	Description						USCS	LL	PL	PI	Cc	Cu
● B5	35 - 36.5	SANDY LEAN CLAY						CL	29	14	15		
☒ B6	30 - 31.5	SANDY LEAN CLAY						CL					
Boring ID	Depth (Ft)	D ₁₀₀	D ₆₀	D ₃₀	D ₁₀	%Cobbles	%Gravel	%Sand	%Fines	%Silt	%Clay		
● B5	35 - 36.5	0.075							65.9				
☒ B6	30 - 31.5	0.075							58.5				

Unconfined Compression Test

ASTM D2166

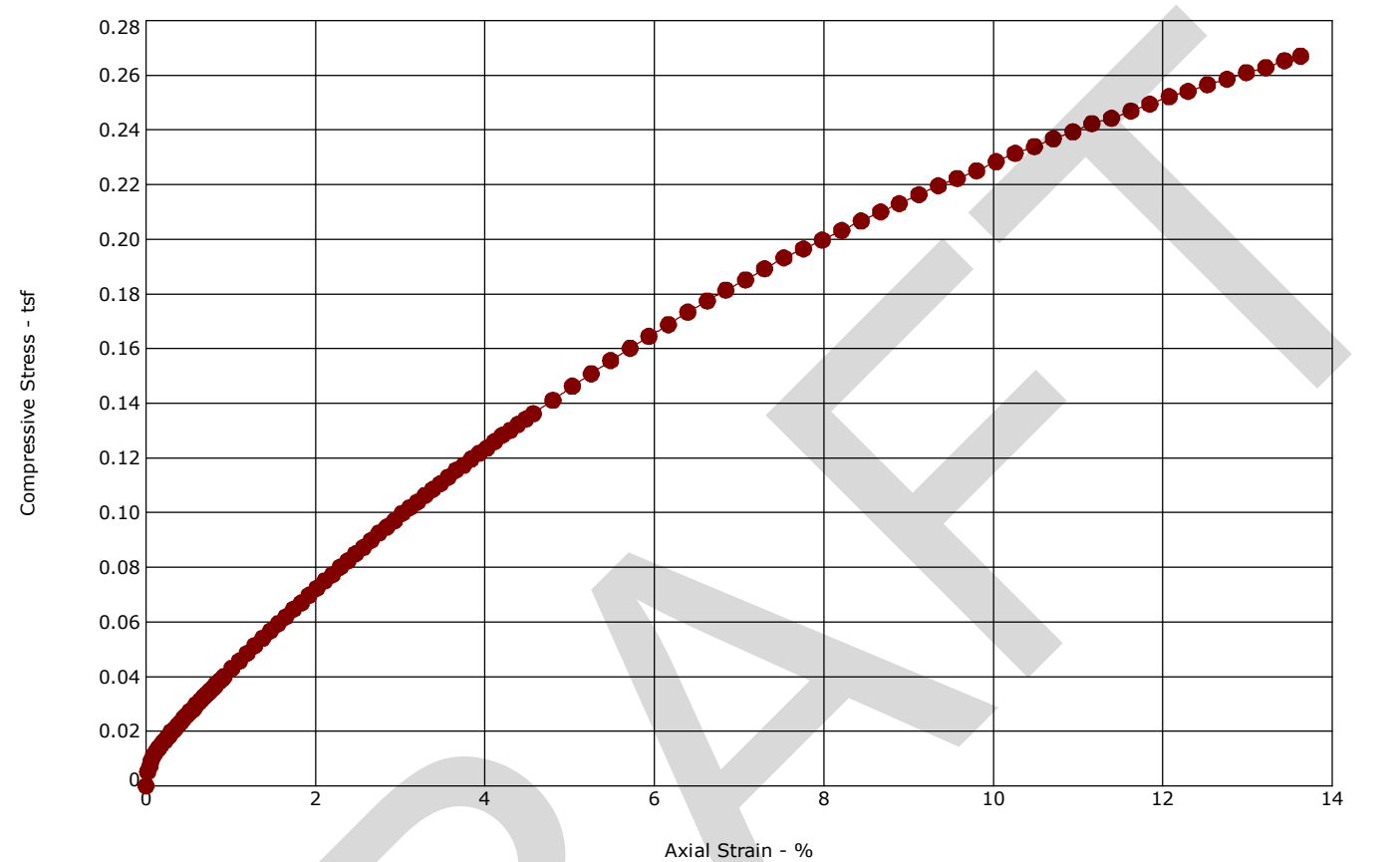


Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B1	5 - 6.5	CARS	35	19	16	92.1	LEAN CLAY (CL)

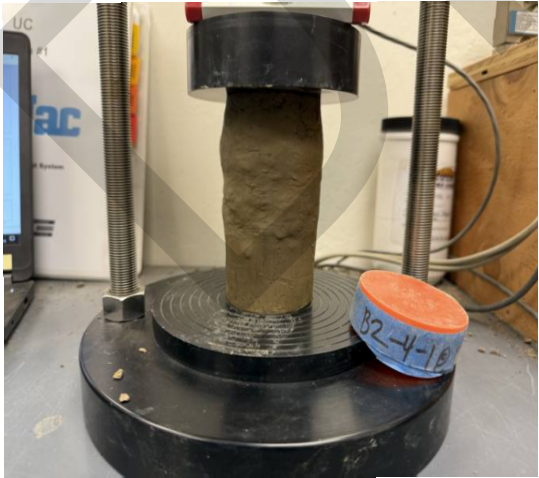
Specimen Failure Mode	Specimen Test Data	
	Moisture Content (%):	21.6
	Dry Density (pcf):	104
	Diameter (in.):	2.37
	Height (in.):	5.21
	Height / Diameter Ratio:	2.20
	Calculated Saturation (%):	93.08
	Calculated Void Ratio:	0.63
	Assumed Specific Gravity:	2.7
	Failure Strain (%):	8.90
	Unconfined Compressive Strength (tsf):	1.09
Undrained Shear Strength (tsf): 0.54		
Strain Rate (in/min):		
Remarks: Assumed Value for Specific Gravity		

Unconfined Compression Test

ASTM D2166

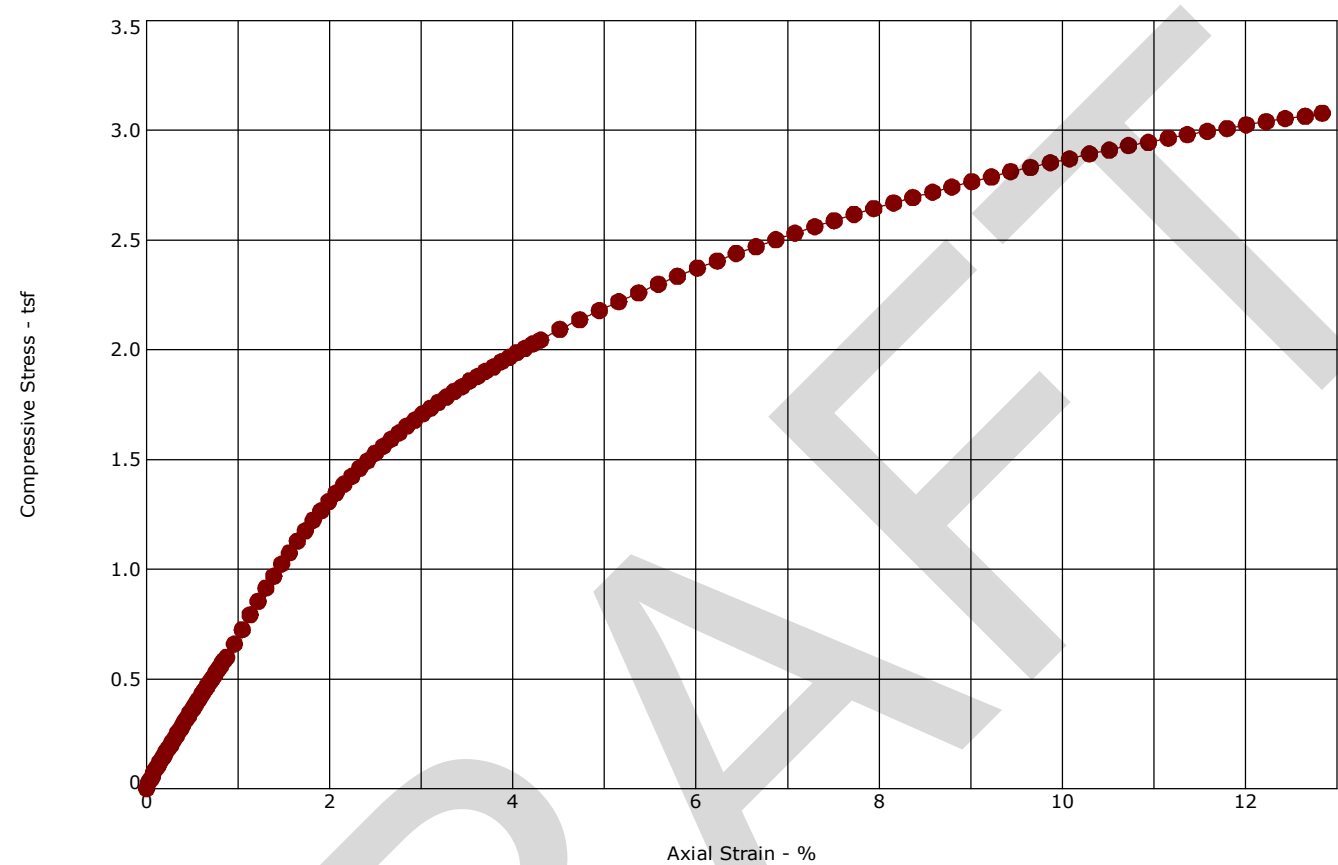


Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B2	10 - 11.5	CARS					FAT CLAY (CH)

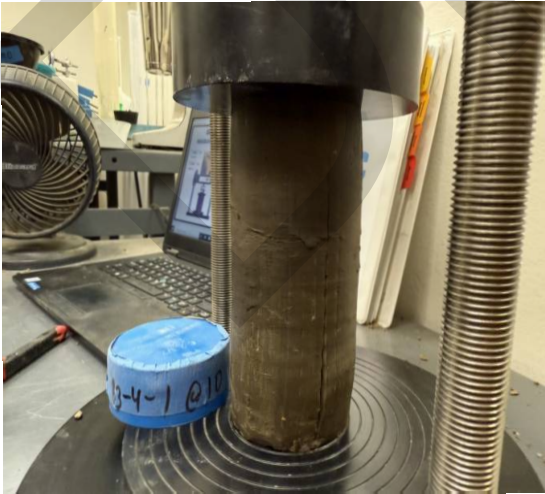
Specimen Failure Mode	Specimen Test Data	
	Moisture Content (%):	20.1
	Dry Density (pcf):	108
	Diameter (in.):	2.38
	Height (in.):	5.51
	Height / Diameter Ratio:	2.31
	Calculated Saturation (%):	95.89
	Calculated Void Ratio:	0.56
	Assumed Specific Gravity:	2.7
	Failure Strain (%):	13.63
	Unconfined Compressive Strength (tsf):	0.27
Undrained Shear Strength (tsf):		0.13
Strain Rate (in/min):		
Remarks: Assumed Value for Specific Gravity		

Unconfined Compression Test

ASTM D2166

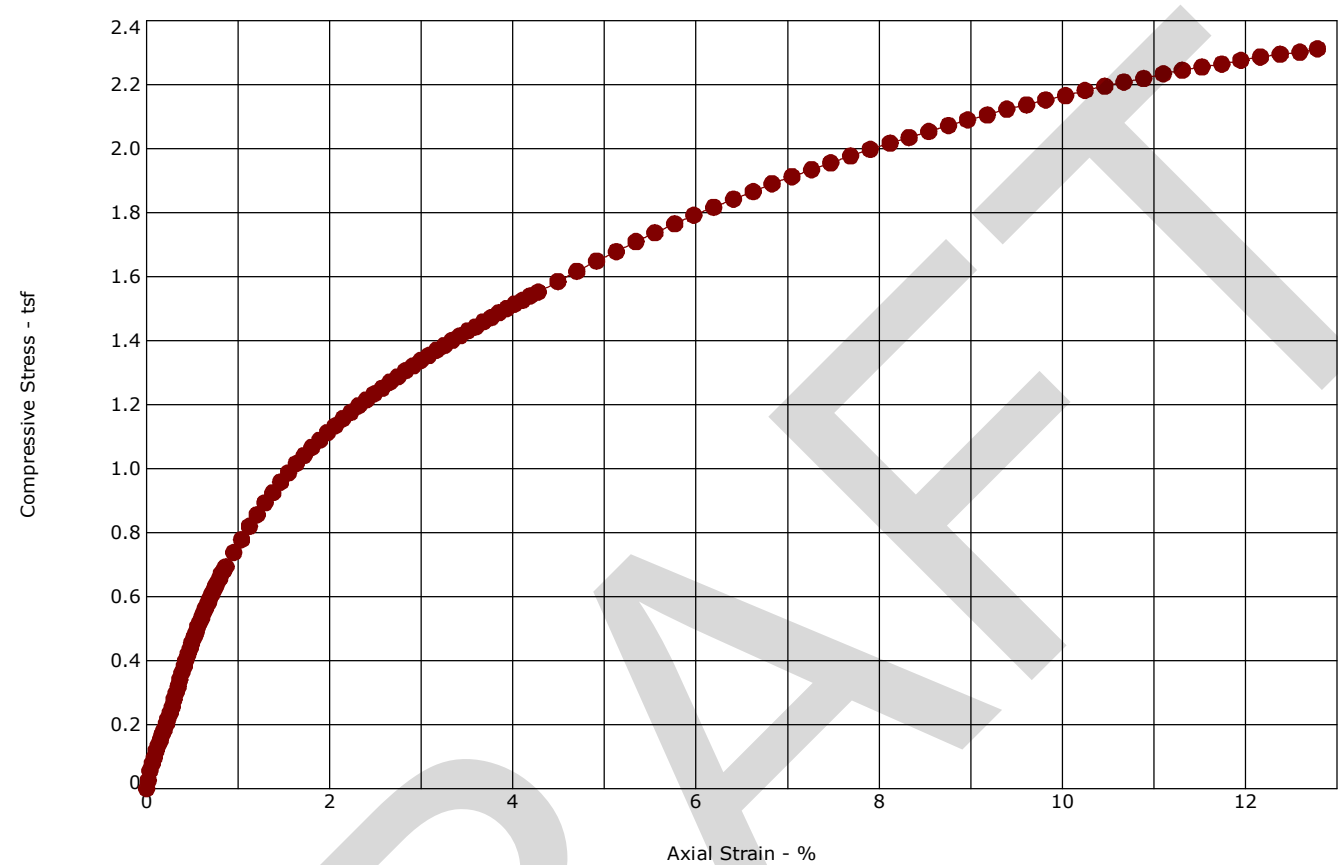


Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B3	10 - 11.5	CARS					LEAN CLAY (CL)

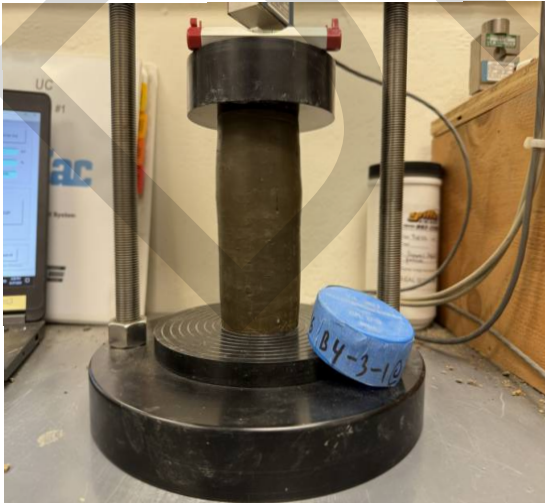
Specimen Failure Mode	Specimen Test Data	
	Moisture Content (%):	20.8
	Dry Density (pcf):	108
	Diameter (in.):	2.38
	Height (in.):	5.84
	Height / Diameter Ratio:	2.45
	Calculated Saturation (%):	99.32
	Calculated Void Ratio:	0.57
	Assumed Specific Gravity:	2.7
	Failure Strain (%):	12.84
	Unconfined Compressive Strength (tsf):	3.08
Undrained Shear Strength (tsf):		1.54
Strain Rate (in/min):		
Remarks: Assumed Value for Specific Gravity		

Unconfined Compression Test

ASTM D2166

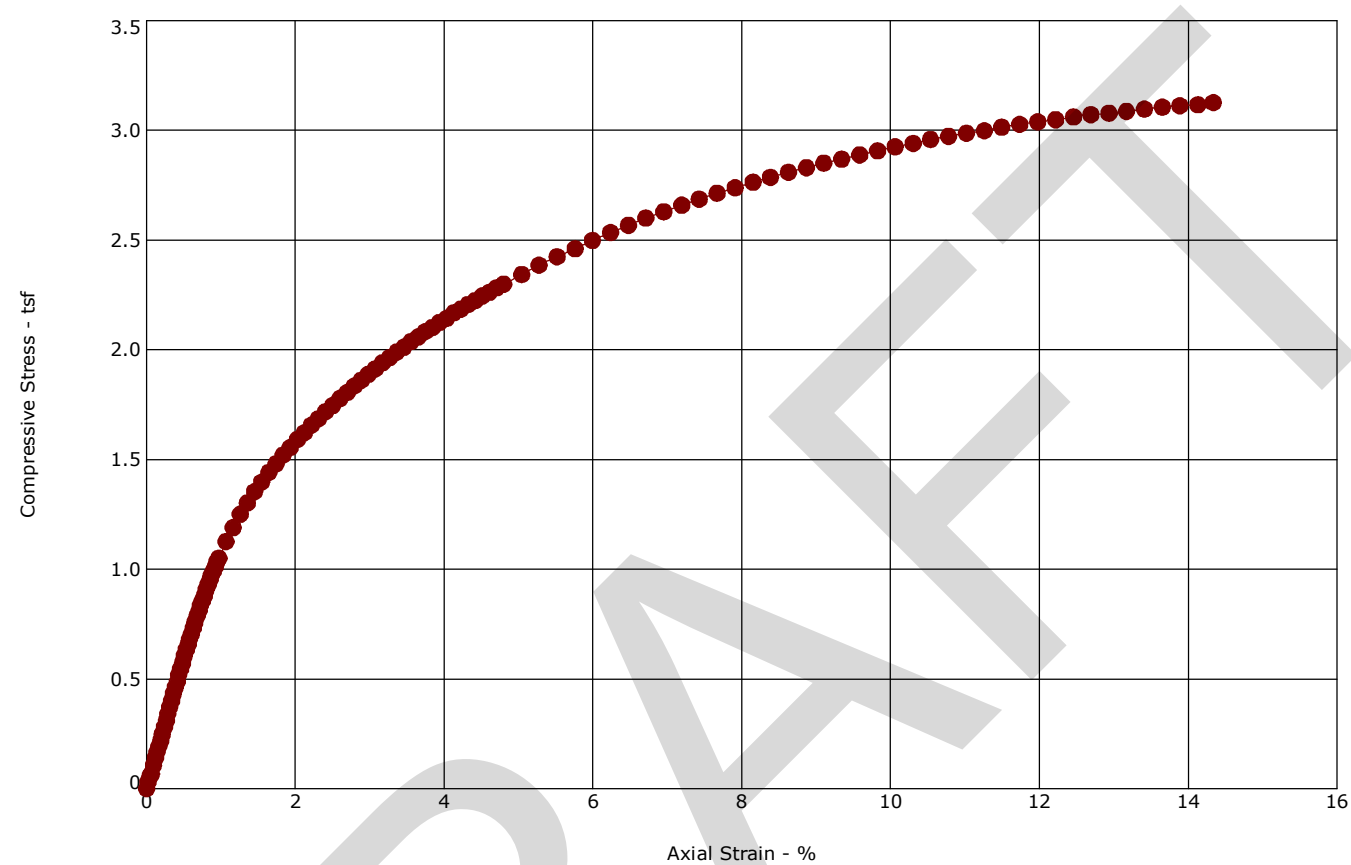


Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B4	7.5 - 9	CARS	61	20	41	98.7	FAT CLAY (CH)

Specimen Failure Mode	Specimen Test Data	
	Moisture Content (%):	23.1
	Dry Density (pcf):	105
	Diameter (in.):	2.38
	Height (in.):	5.87
	Height / Diameter Ratio:	2.47
	Calculated Saturation (%):	103.42
	Calculated Void Ratio:	0.60
	Assumed Specific Gravity:	2.7
	Failure Strain (%):	12.79
	Unconfined Compressive Strength (tsf):	2.31
Undrained Shear Strength (tsf): 1.16		
Strain Rate (in/min):		
Remarks: Assumed Value for Specific Gravity		

Unconfined Compression Test

ASTM D2166

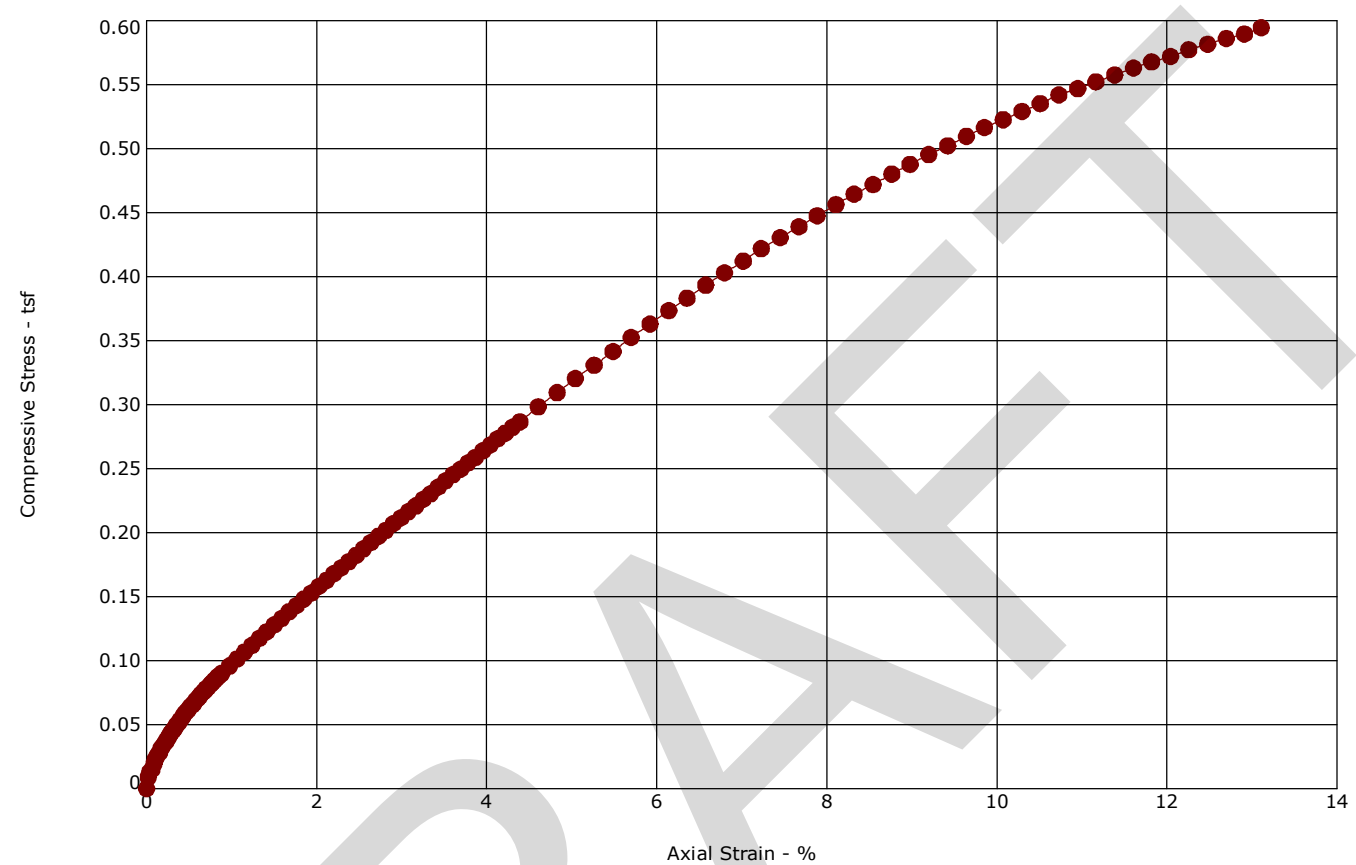


Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B5	7.5 - 9	CARS	52	16	36	92.9	FAT CLAY (CH)

Specimen Failure Mode	Specimen Test Data	
	Moisture Content (%):	19.6
	Dry Density (pcf):	110
	Diameter (in.):	2.39
	Height (in.):	5.24
	Height / Diameter Ratio:	2.19
	Calculated Saturation (%):	98.54
	Calculated Void Ratio:	0.54
	Assumed Specific Gravity:	2.7
	Failure Strain (%):	14.34
	Unconfined Compressive Strength (tsf):	3.13
Undrained Shear Strength (tsf):		1.56
Strain Rate (in/min):		
Remarks: Assumed Value for Specific Gravity		

Unconfined Compression Test

ASTM D2166

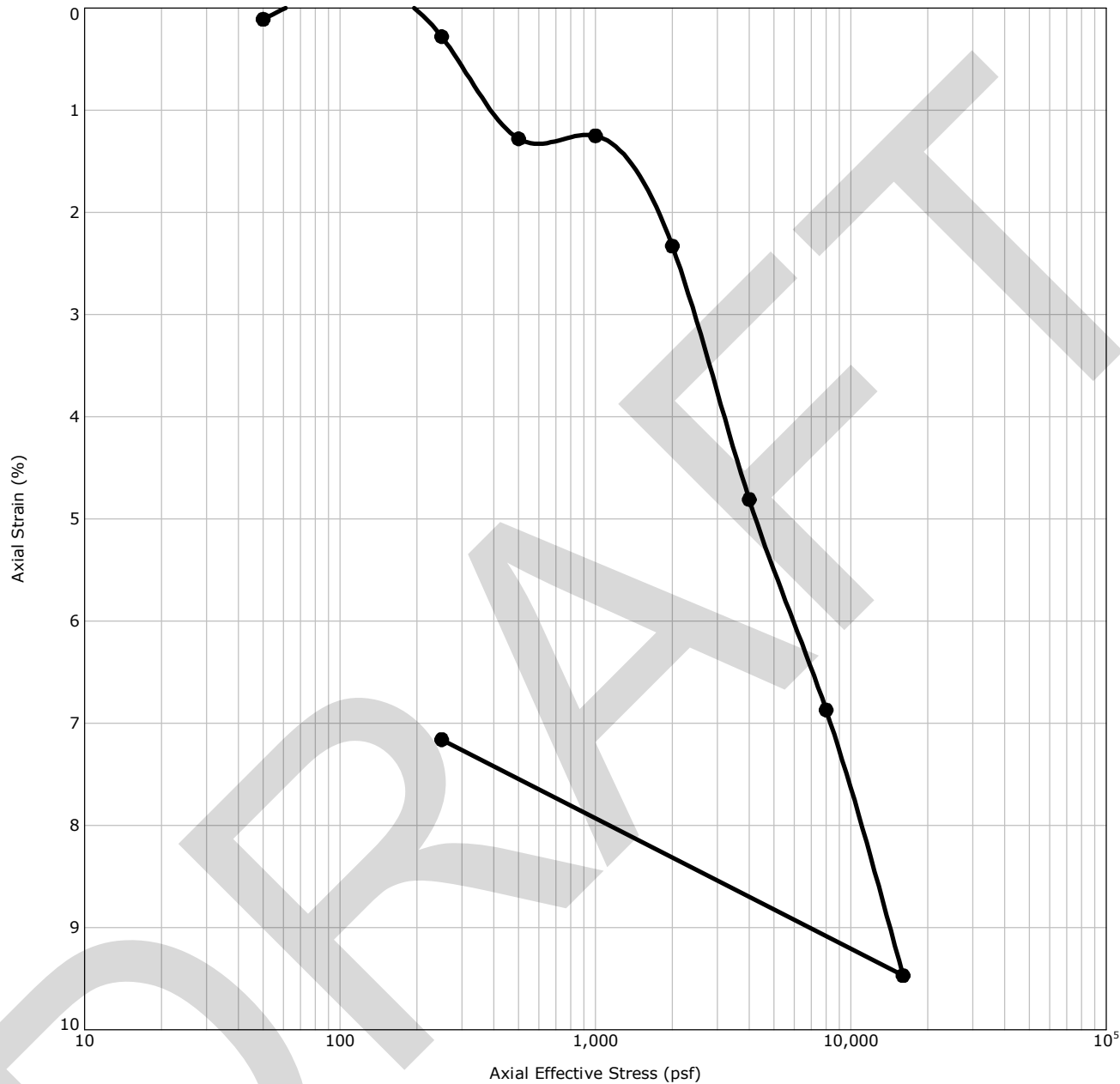


Boring ID	Depth (Ft)	Sample type	LL	PL	PI	Fines (%)	Description
B5	35 - 36.5	CARS	29	14	15	65.9	SANDY LEAN CLAY (CL)

Specimen Failure Mode	Specimen Test Data	
	Moisture Content (%):	20.9
	Dry Density (pcf):	110
	Diameter (in.):	2.38
	Height (in.):	5.72
	Height / Diameter Ratio:	2.41
	Calculated Saturation (%):	105.90
	Calculated Void Ratio:	0.53
	Assumed Specific Gravity:	2.7
	Failure Strain (%):	13.11
	Unconfined Compressive Strength (tsf):	0.59
Undrained Shear Strength (tsf):		0.30
Strain Rate (in/min):		
Remarks: Assumed Value for Specific Gravity		

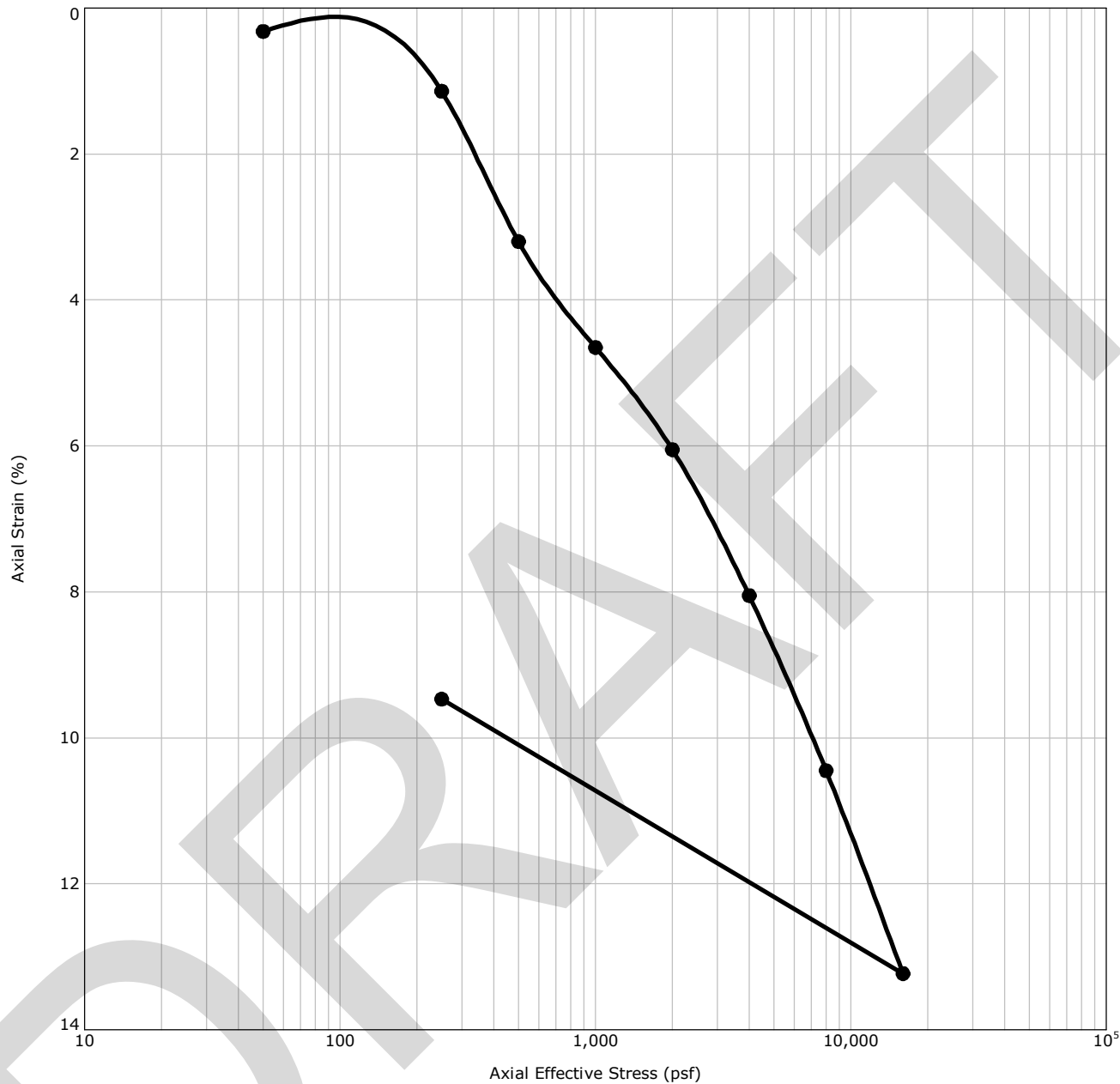
One-Dimensional Consolidation Test

ASTM D2435



Boring ID	Depth (Ft)	Specimen #	Material Description							USCS	AASHTO
B3	20 - 21.5	6	LEAN CLAY							CL	
Natural			Initial Dry Density (pcf)	LL	PI	Specific Gravity	Overburden (psf)	P _c (psf)	C _c (% / log stress)	C _r (% / log stress)	Initial Void Ratio
Saturation (%)		Moisture (%)									
98.9		23.3	103.0	35	20	2.70	2400				
Notes: Drive Sample is Disturbed											

One-Dimensional Consolidation Test
ASTM D2435



Boring ID	Depth (Ft)	Specimen #	Material Description							USCS	AASHTO
B6	15 - 16.5	5	LEAN CLAY							CL	
Natural			Initial Dry Density (pcf)	LL	PI	Specific Gravity	Overburden (psf)	P _c (psf)	C _c (% / log stress)	C _r (% / log stress)	Initial Void Ratio
Saturation (%)		Moisture (%)									
99.8		25.6	99.5	38	22	2.70	1800				
Notes: Drive Sample is Disturbed											

902 Industrial Way
Lodi, California 95240
(209) 367-3701



Client

Calpine Corporation

Project

Calpine Tanager Energy Storage Project

Sample Submitted By: Terracon (R1) **Date Received:** 6/30/2025 **Project Number:** R1245076

Results of Corrosion Analysis

Sample Type GRAB

Sample Location B1

Sample Depth (ft.) 0.0 - 5.0

pH Analysis, ASTM G 51 8.10

Water Soluble Sulfate (SO4), ASTM D516 (%) 0.012

Sulfides, AWWA 4500-S²⁻ D, (ppm) <0.001

Chlorides, AWWA 4500-CL⁻ E, (%) 0.003

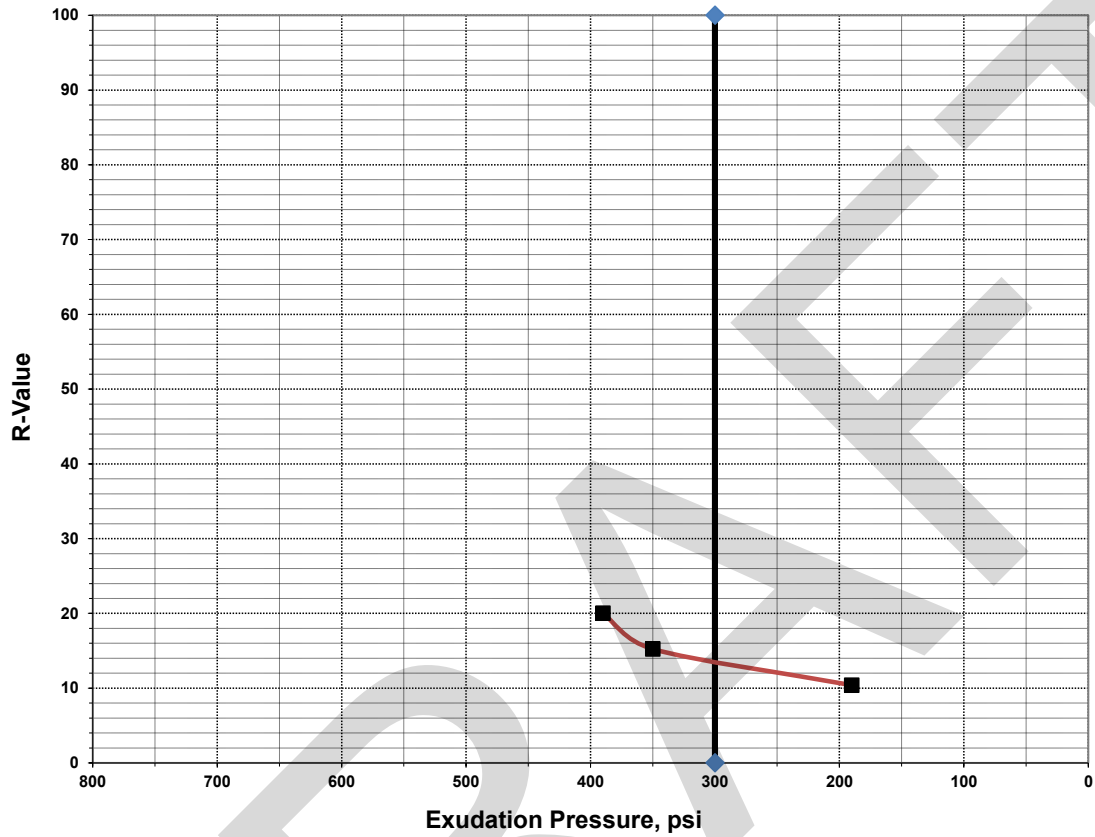
Red-Ox, ASTM G 200, (mV) +142

Total Salts, AWWA 2520 B, (mg/kg) 360

Saturated Minimum Resistivity, ASTM G 57, (ohm-cm) 1,400

Reviewed By: Stella Oliveira
Stella Oliveira
Lab Coordinator

The tests were performed in general accordance with applicable ASTM and AWWA test methods. This report is exclusively for the use of the client indicated above and shall not be reproduced except in full without the written consent. Test results transmitted herein are only applicable to the actual samples tested at the location(s) referenced and are not necessarily indicative of the properties of other apparently similar or identical materials.



Specimen Identification	Compaction Pressure (psi)	R-Value at 300 psi
B1 (0 to 5 ft)	140.0	13

R-Value Test

Client: Calpine Corporation
Project: Calpine Tanager Energy Storage Project
Site: 800 Thomas Foon Chew Way, San Jose 95134
Project No.: R1245076

FIELD ELECTRICAL RESISTIVITY DATA SHEET

Calpine BESS ■ San Jose, CA

June 12, 2025 ■ Terracon Project No. NS245159

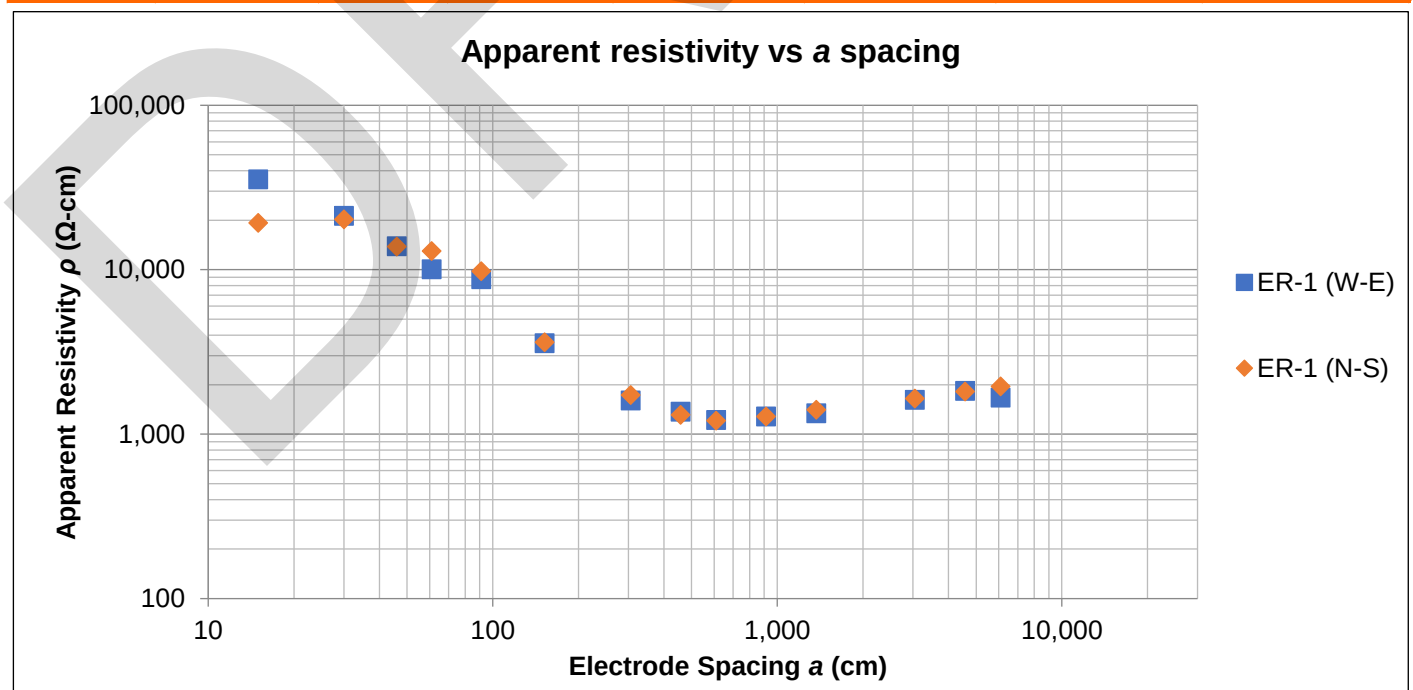


Array Loc.	ER-1 (37.423117°N, 121.931957°W)		
Instrument	AGI SuperSting R1	Weather	Overcast, high of 59°F.
Serial #	SP0303161	Ground Cond.	Tilled AG field
Cal. Check	6/12/2024	Tested By	T.Black
Test Date	June 12, 2025	Method	Wenner 4-pin ASTM G57-06 (2012); IEEE 81-2012
Notes & Conflicts	Arrays share a common midpoint and are oriented N-S and W-E.		

Apparent resistivity ρ is calculated as :

$$\rho = \frac{4\pi aR}{1 + \frac{2a}{\sqrt{a^2 + 4b^2}} - \frac{a}{\sqrt{a^2 + b^2}}}$$

Electrode Spacing a		Electrode Depth b		ER-1 (W-E)		ER-1 (N-S)	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance R	Apparent Resistivity ρ	Measured Resistance R	Apparent Resistivity ρ
				Ω	(Ω -cm)	Ω	(Ω -cm)
0.5	15	2	5	322.50	35440	175.10	19240
1	30	2	5	107.800	21270	102.500	20220
1.5	46	2	5	47.070	13880	46.860	13820
2	61	3	8	25.530	10070	32.940	13000
3	91	4	10	15.010	8760	16.830	9820
5	152	6	15	3.6820	3580	3.7270	3620
10	305	12	30	0.8233	1600	0.8860	1730
15	457	12	30	0.4732	1370	0.4544	1310
20	610	12	30	0.3181	1220	0.3154	1210
30	914	12	30	0.2224	1280	0.2218	1280
45	1372	12	30	0.1548	1340	0.1631	1410
100	3048	12	30	0.0844	1620	0.0861	1650
150	4572	12	30	0.0637	1830	0.0635	1820
200	6096	12	30	0.0437	1670	0.0508	1950



FIELD ELECTRICAL RESISTIVITY DATA SHEET

Calpine BESS ■ San Jose, CA

June 12, 2025 ■ Terracon Project No. NS245159

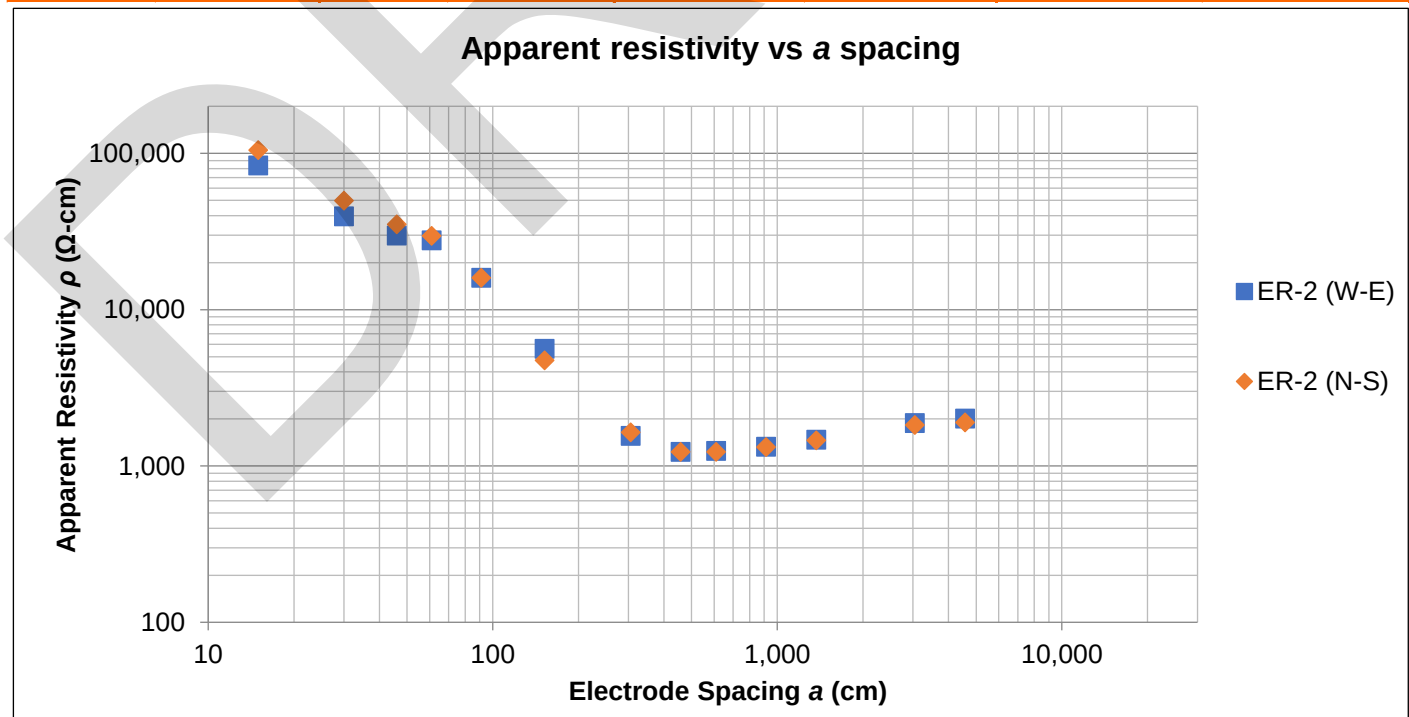


Array Loc.	ER-2 (37.4233417°N, 121.930499°W)		
Instrument	AGI SuperSting R1	Weather	Overcast, high of 72°F.
Serial #	SP0303161	Ground Cond.	Dry grass field
Cal. Check	6/12/2024	Tested By	T.Black
Test Date	June 12, 2025	Method	Wenner 4-pin ASTM G57-06 (2012); IEEE 81-2012
Notes & Conflicts	Arrays share a common midpoint and are oriented N-S and W-E.		

Apparent resistivity ρ is calculated as :

$$\rho = \frac{4\pi aR}{1 + \frac{2a}{\sqrt{a^2 + 4b^2}} - \frac{a}{\sqrt{a^2 + b^2}}}$$

Electrode Spacing a		Electrode Depth b		ER-2 (W-E)		ER-2 (N-S)	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance R	Apparent Resistivity ρ	Measured Resistance R	Apparent Resistivity ρ
				Ω	(Ω -cm)	Ω	(Ω -cm)
0.5	15	2	5	762.50	83790	956.20	105070
1	30	2	5	200.900	39630	252.700	49850
1.5	46	2	5	101.300	29870	119.400	35210
2	61	3	8	70.500	27810	75.270	29690
3	91	4	10	27.470	16030	27.420	16000
5	152	6	15	5.7980	5630	4.8680	4730
10	305	12	30	0.8026	1560	0.8418	1640
15	457	12	30	0.4264	1230	0.4246	1230
20	610	12	30	0.3256	1250	0.3185	1230
30	914	12	30	0.2319	1330	0.2301	1320
45	1372	12	30	0.1708	1470	0.1690	1460
100	3048	12	30	0.0983	1880	0.0954	1830
150	4572	12	30	0.0701	2010	0.0662	1900



FIELD ELECTRICAL RESISTIVITY DATA SHEET

Calpine BESS ■ San Jose, CA

June 12, 2025 ■ Terracon Project No. NS245159

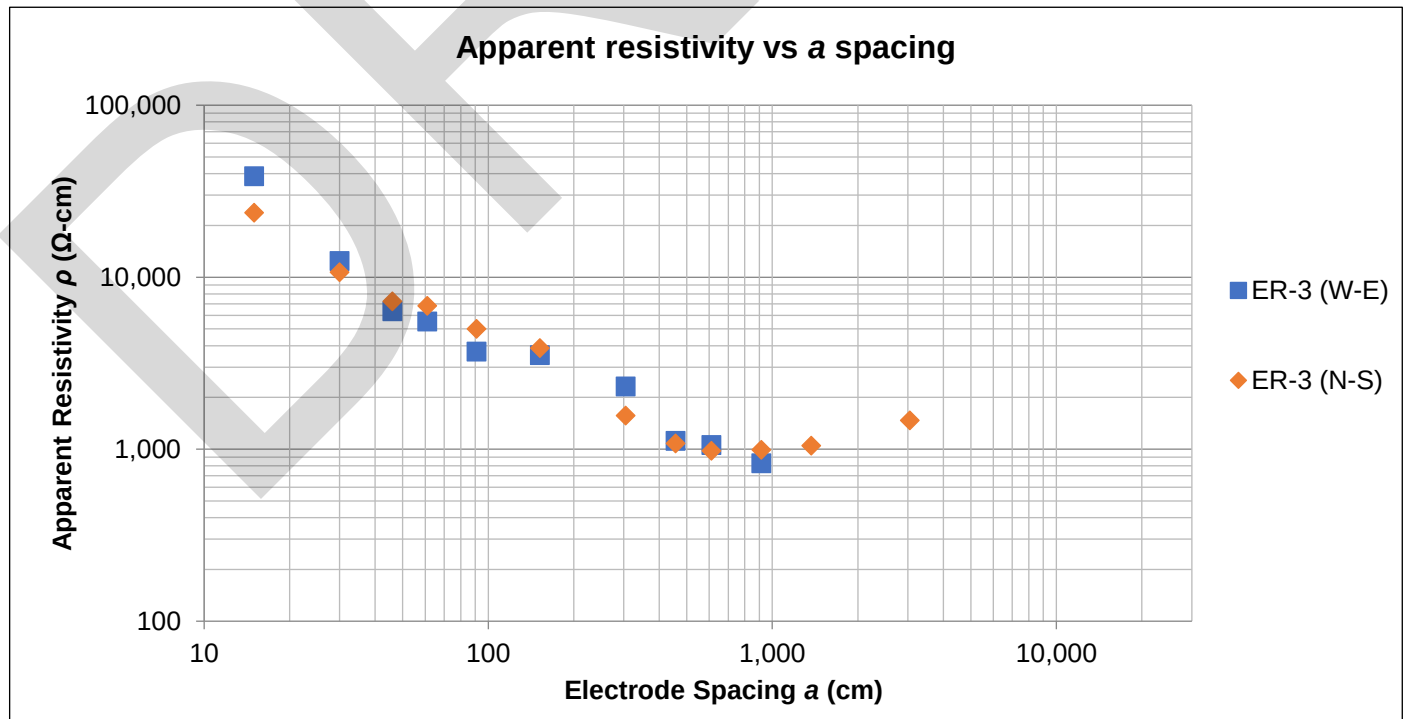


Array Loc.	ER-3 (37.424751°N, 121.930488°W)		
Instrument	AGI SuperSting R1	Weather	Overcast, high of 72°F.
Serial #	SP0303161	Ground Cond.	Dry grass field
Cal. Check	6/12/2024	Tested By	T.Black
Test Date	June 12, 2025	Method	Wenner 4-pin ASTM G57-06 (2012); IEEE 81-2012
Notes & Conflicts	Arrays share a common midpoint and are oriented N-S and W-E.		

Apparent resistivity ρ is calculated as :

$$\rho = \frac{4\pi a R}{1 + \frac{2a}{\sqrt{a^2 + 4b^2}} - \frac{a}{\sqrt{a^2 + b^2}}}$$

Electrode Spacing a		Electrode Depth b		ER-3 (W-E)		ER-3 (N-S)	
(feet)	(centimeters)	(inches)	(centimeters)	Measured Resistance R Ω	Apparent Resistivity ρ (Ω-cm)	Measured Resistance R Ω	Apparent Resistivity ρ (Ω-cm)
0.5	15	2	5	352.20	38700	215.60	23690
1	30	2	5	63.040	12440	54.230	10700
1.5	46	2	5	21.530	6350	24.550	7240
2	61	3	8	13.990	5520	17.310	6830
3	91	4	10	6.342	3700	8.608	5020
5	152	6	15	3.6200	3520	3.9930	3880
10	305	12	30	1.1900	2320	0.8060	1570
15	457	12	30	0.3873	1120	0.3748	1080
20	610	12	30	0.2754	1060	0.2536	980
30	914	12	30	0.1450	830	0.1712	990
45	1372	12	30	0.0000	0	0.1214	1050
100	3048	12	30	0.0000	0	0.0769	1470
150	4572	12	30	0.0000	0	0.0000	0





July 25, 2025

Terracon

1220 Concord Ave, Ste 450
Concord, CA 94520
Attn: Ramsey Zeidan, P.E.

Thermal Resistivity Report (Project No. R1245076)
Calpine Tanager Energy Storage Project – San Jose, CA

The following is the report of thermal dryout characterization tests conducted on six (6) tube samples and two (2) bulk samples of native soil from the referenced project sent to our laboratory.

Thermal Resistivity Tests: The tube samples were tested 'as is.' The bulk samples were tested at the 'optimum' moisture content and at 90% of the standard Proctor dry density ***provided by Terracon***. The tests were conducted in accordance with the **IEEE standard 442-2017**. The results are tabulated below and the thermal dry out curves are presented in **Figures 1 and 2**.

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

Sample ID	Depth (ft)	Effort (%)	Description (Terracon)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft³)
				Wet	Dry		
B2-Bulk	0 - 5	90	Sandy Lean Clay (CL)	70	153	13	106
B2-2-11	5	Tube	Sandy Lean Clay (CL)	60	193	26	95
B2-4-11	10	Tube	Fat Clay (CH)	62	168	24	101
B2-5-11	15	Tube	Fat Clay (CH)	64	158	22	104
B4-Bulk	0 - 5	90	Clayey Sand with Gravel (SC)	78	169	17	99
B4-2-II	5	Tube	Fat Clay (CH)	68	174	23	96
B4-4-11	10	Tube	Fat Clay (CH)	67	143	17	109
B4-5-11	15	Tube	Fat Clay (CH)	64	156	21	107



Comments: The thermal characteristic depicted in the dryout curves apply for the soils at their respective test dry density.

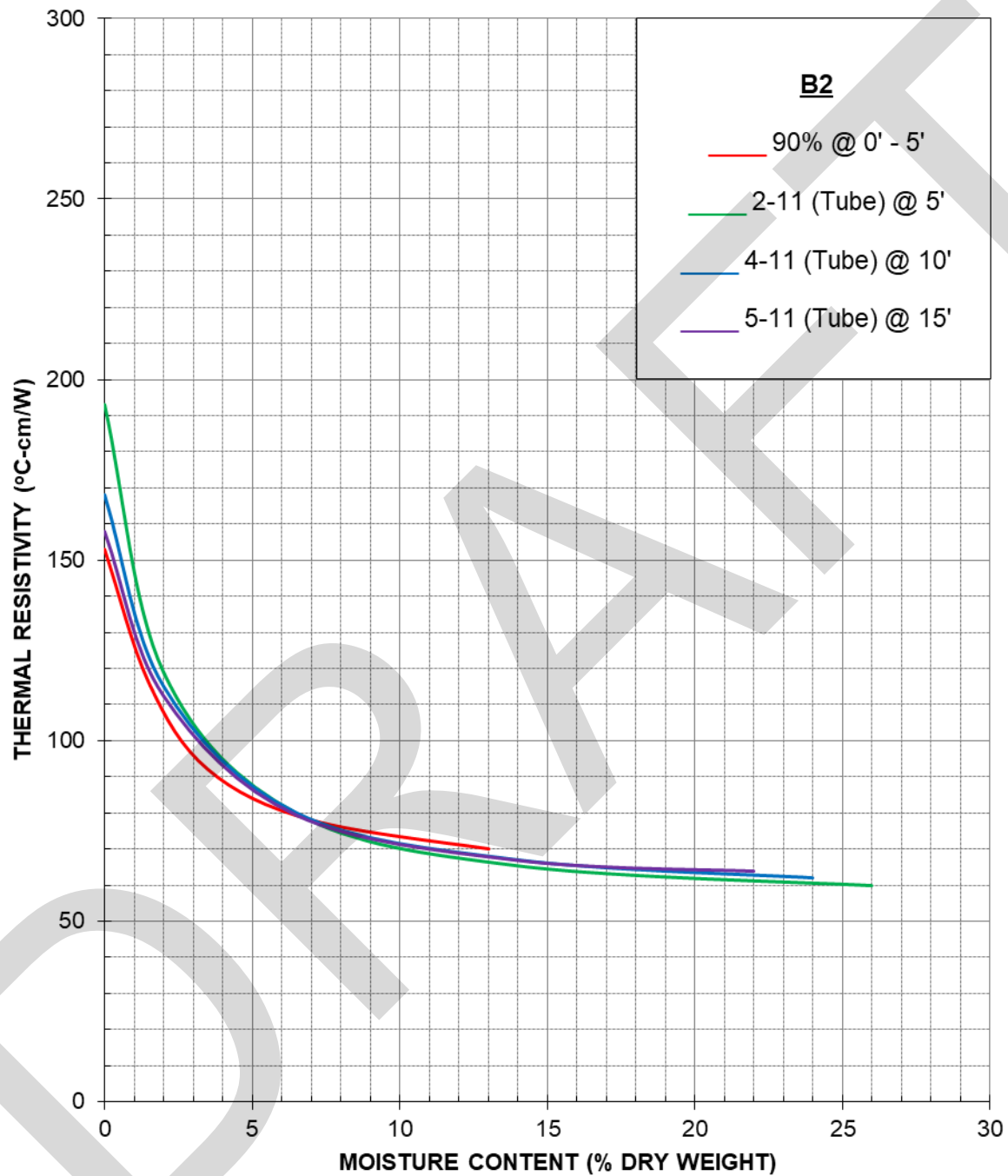
Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA, LLC

A handwritten signature in black ink, appearing to read "Nimesh Patel", written in a cursive style.

Nimesh Patel

THERMAL DRYOUT CURVES



Terracon (Project No. R1245076)

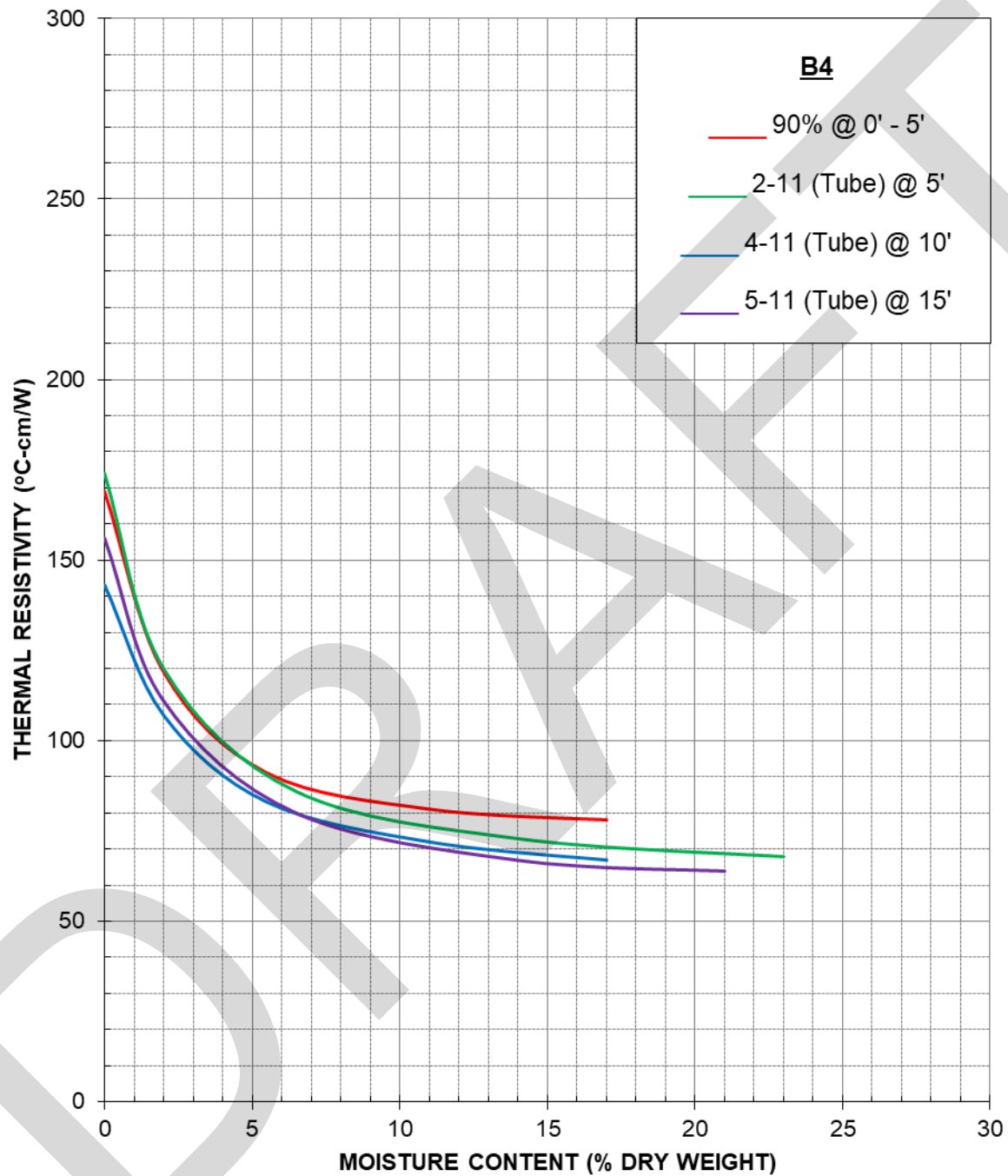
Calpine Tanager Energy Storage Project – San Jose, CA

Thermal Resistivity Report

July 2025

Figure 1

THERMAL DRYOUT CURVES



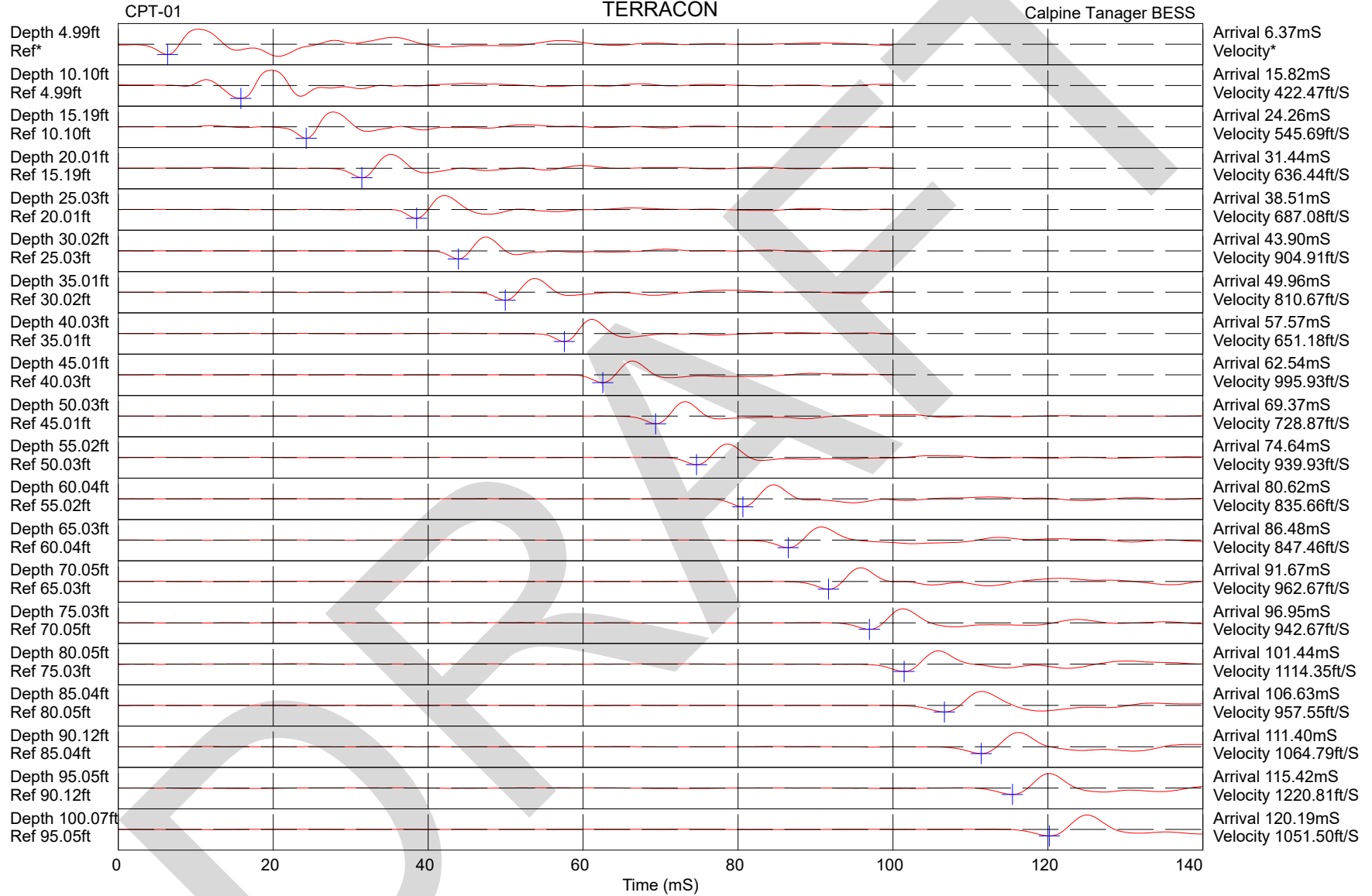
Terracon (Project No. R1245076)

Calpine Tanager Energy Storage Project – San Jose, CA

Thermal Resistivity Report

July 2025

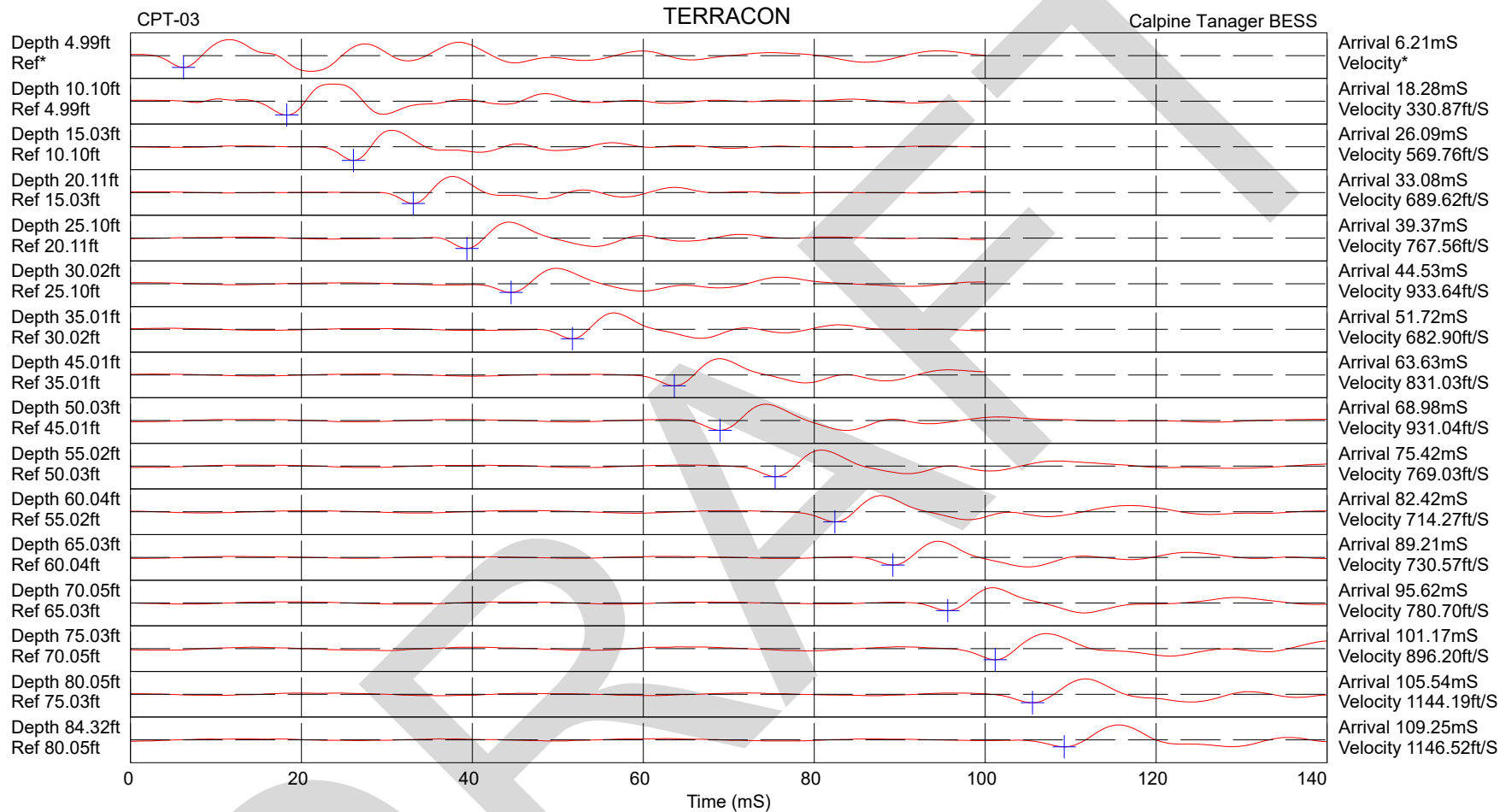
Figure 2



Hammer to Rod String Distance (ft): 5.83

* = Not Determined

COMMENT:



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

Supporting Information

Contents:

General Notes
Unified Soil Classification System
SHAFT Allowable Axial Capacity Curves
Liquefaction Analysis Results

Note: All attachments are one page unless otherwise noted.

General Notes

Sampling	Water Level	Field Tests
 Modified California Ring Sampler  Grab Sample  Standard Penetration Test	 Water Initially Encountered  Water Level After a Specified Period of Time  Water Level After a Specified Period of Time  Cave In Encountered Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.	N Standard Penetration Test Resistance (Blows/Ft.) (HP) Hand Penetrometer (T) Torvane (DCP) Dynamic Cone Penetrometer UC Unconfined Compressive Strength (PID) Photo-Ionization Detector (OVA) Organic Vapor Analyzer

Descriptive Soil Classification

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

Location And Elevation Notes

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

Strength Terms

Relative Density of Coarse-Grained Soils (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance			Consistency of Fine-Grained Soils (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance			
Relative Density	Standard Penetration or N-Value (Blows/Ft.)	Ring Sampler (Blows/Ft.)	Consistency	Unconfined Compressive Strength Qu (tsf)	Standard Penetration or N-Value (Blows/Ft.)	Ring Sampler (Blows/Ft.)
Very Loose	0 - 3	0 - 5	Very Soft	less than 0.25	0 - 1	< 3
Loose	4 - 9	6 - 14	Soft	0.25 to 0.50	2 - 4	3 - 5
Medium Dense	10 - 29	15 - 46	Medium Stiff	0.50 to 1.00	4 - 8	6 - 10
Dense	30 - 50	47 - 79	Stiff	1.00 to 2.00	8 - 15	11 - 18
Very Dense	> 50	≥ 80	Very Stiff	2.00 to 4.00	15 - 30	19 - 36
			Hard	> 4.00	> 30	> 37

Relevance of Exploration and Laboratory Test Results

Exploration/field results and/or laboratory test data contained within this document are intended for application to the project as described in this document. Use of such exploration/field results and/or laboratory test data should not be used independently of this document.

DESCRIPTION OF MEASUREMENTS AND CALIBRATIONS

To be reported per ASTM D5778:

- Uncorrected Tip Resistance, q_c
Measured force acting on the cone divided by the cone's projected area
- Corrected Tip Resistance, q_t
Cone resistance corrected for porewater and net area ratio effects
 $q_t = q_c + u_2(1 - a)$
Where a is the net area ratio, a lab calibration of the cone typically between 0.70 and 0.85

- Pore Pressure, u
Pore pressure measured during penetration
 u_1 - sensor on the face of the cone
 u_2 - sensor on the shoulder (more common)

- Sleeve Friction, f_s
Frictional force acting on the sleeve divided by its surface area

- Normalized Friction Ratio, F_r
The ratio as a percentage of f_s to q_t , accounting for overburden pressure

To be reported per ASTM D7400, if collected:

- Shear Wave Velocity, V_s
Measured in a Seismic CPT and provides direct measure of soil stiffness

DESCRIPTION OF GEOTECHNICAL CORRELATIONS

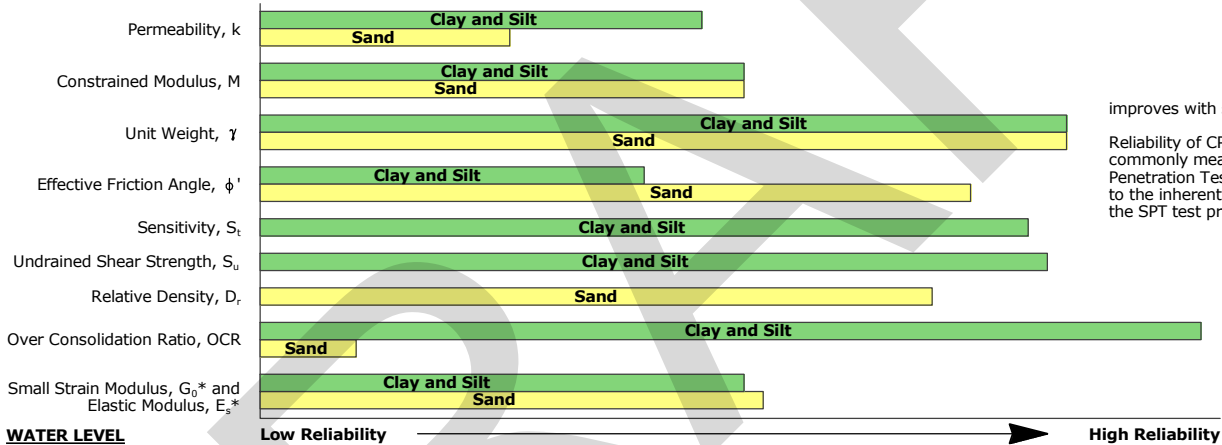
- Normalized Tip Resistance, Q_{tn}
 $Q_{tn} = ((q_t - \sigma_{v0})/P_a)(P_a/\sigma'_{v0})^n$
 $n = 0.381(I_c) + 0.05(\sigma'_{v0}/P_a) - 0.15$
- Over Consolidation Ratio, OCR
 $OCR(1) = 0.25(Q_{tn})^{1.25}$
 $OCR(2) = 0.33(Q_{tn})$
- Undrained Shear Strength, S_u
 $S_u = Q_{tn} \times \sigma'_{v0}/N_{kt}$
 N_{kt} is a soil-specific factor (shown on S_u plot)
- Sensitivity, S_t
 $S_t = (q_t - \sigma_{v0}/N_{kt}) \times (1/f_s)$
- Effective Friction Angle, ϕ'
 $\phi'(1) = \tan^{-1}(0.373[\log(q_t/\sigma'_{v0}) + 0.29])$
 $\phi'(2) = 17.6 + 11[\log(Q_{tn})]$
- Unit Weight, γ
 $\gamma = (0.27[\log(F_r)] + 0.36[\log(q_t/atm)] + 1.236) \times \gamma_{water}$
 σ_{v0} is taken as the incremental sum of the unit weights
- Small Strain Shear Modulus, G_0
 $G_0(1) = \rho V_s^2$
 $G_0(2) = 0.015 \times 10^{(0.55I_c + 1.68)}(q_t - \sigma_{v0})$

- Soil Behavior Type Index, I_c
 $I_c = [(3.47 - \log(Q_{tn}))^2 + (\log(F_r) + 1.22)^2]^{0.5}$
- SPT N_{60}
 $N_{60} = (q_t/atm) / 10^{(1.1268 - 0.2817I_c)}$
- Elastic Modulus, E_s (assumes $q/q_{ultimate} \sim 0.3$, i.e. $FS = 3$)
 $E_s(1) = 2.6\psi G_0$ where $\psi = 0.56 - 0.33\log Q_{tn, clean sand}$
 $E_s(2) = G_0$
 $E_s(3) = 0.015 \times 10^{(0.55I_c + 1.68)}(q_t - \sigma_{v0})$
 $E_s(4) = 2.5q_t$
- Constrained Modulus, M
 $M = \alpha_M(q_t - \sigma_{v0})$
For $I_c > 2.2$ (fine-grained soils)
 $\alpha_M = Q_{tn}$ with maximum of 14
For $I_c < 2.2$ (coarse-grained soils)
 $\alpha_M = 0.0188 \times 10^{(0.55I_c + 1.68)}$
- Hydraulic Conductivity, k
For $1.0 < I_c < 3.27$ $k = 10^{(0.952 - 3.04I_c)}$
For $3.27 < I_c < 4.0$ $k = 10^{(-4.52 - 1.37I_c)}$
- Relative Density, D_r
 $D_r = (Q_{tn} / 350)^{0.5} \times 100$

REPORTED PARAMETERS

CPT logs as provided, at a minimum, report the data as required by ASTM D5778 and ASTM D7400 (if applicable). This minimum data include q_t , f_s , and u . Other correlated parameters may also be provided. These other correlated parameters are interpretations of the measured data based upon published and reliable references, but they do not necessarily represent the actual values that would be derived from direct testing to determine the various parameters. To this end, more than one correlation to a given parameter may be provided. The following chart illustrates estimates of reliability associated with correlated parameters based upon the literature referenced below.

RELATIVE RELIABILITY OF CPT CORRELATIONS



improves with seismic V_s measurements

Reliability of CPT-predicted N_{60} values as commonly measured by the Standard Penetration Test (SPT) is not provided due to the inherent inaccuracy associated with the SPT test procedure.

WATER LEVEL

The groundwater level at the CPT location is used to normalize the measurements for vertical overburden pressures and as a result influences the normalized soil behavior type classification and correlated soil parameters. The water level may either be "measured" or "estimated."

Measured - Depth to water directly measured in the field

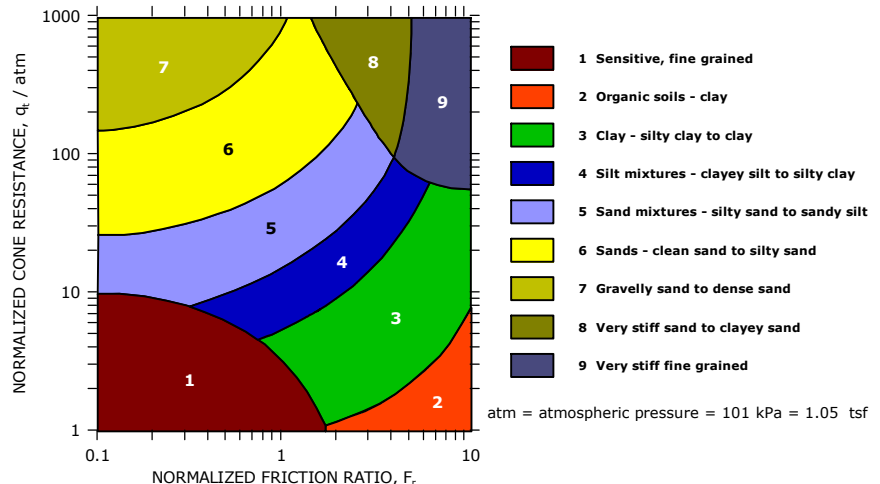
Estimated - Depth to water interpolated by the practitioner using pore pressure measurements in coarse grained soils and known site conditions

While groundwater levels displayed as "measured" more accurately represent site conditions at the time of testing than those "estimated," in either case the groundwater should be further defined prior to construction as groundwater level variations will occur over time.

CONE PENETRATION SOIL BEHAVIOR TYPE

The estimated stratigraphic profiles included in the CPT logs are based on relationships between corrected tip resistance (q_t), friction resistance (f_s), and porewater pressure (u_2). The normalized friction ratio (F_r) is used to classify the soil behavior type.

Typically, silts and clays have high F_r values and generate large excess penetration porewater pressures; sands have lower F_r 's and do not generate excess penetration porewater pressures. The adjacent graph (Robertson *et al.*) presents the soil behavior type correlation used for the logs. This normalized SBT chart, generally considered the most reliable, does not use pore pressure to determine SBT due to its lack of repeatability in onshore CPTs.



REFERENCES

- Kulhawy, F.H., Mayne, P.W., (1997). "Manual on Estimating Soil Properties for Foundation Design," Electric Power Research Institute, Palo Alto, CA.
- Mayne, P.W., (2013). "Geotechnical Site Exploration in the Year 2013," Georgia Institute of Technology, Atlanta, GA.
- Robertson, P.K., Cabal, K.L. (2012). "Guide to Cone Penetration Testing for Geotechnical Engineering," Signal Hill, CA.
- Schmertmann, J.H., (1970). "Static Cone to Compute Static Settlement over Sand," *Journal of the Soil Mechanics and Foundations Division*, 96(SM3), 1011-1043.

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests ^A				Soil Classification	
				Group Symbol	Group Name ^B
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	Gravels: More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels: Less than 5% fines ^C	Cu≥4 and 1≤Cc≤3 ^E	GW	Well-graded gravel ^F
			Cu<4 and/or [Cc<1 or Cc>3.0] ^E	GP	Poorly graded gravel ^F
		Gravels with Fines: More than 12% fines ^C	Fines classify as ML or MH	GM	Silty gravel ^{F, G, H}
			Fines classify as CL or CH	GC	Clayey gravel ^{F, G, H}
	Sands: 50% or more of coarse fraction passes No. 4 sieve	Clean Sands: Less than 5% fines ^D	Cu≥6 and 1≤Cc≤3 ^E	SW	Well-graded sand ^I
			Cu<6 and/or [Cc<1 or Cc>3.0] ^E	SP	Poorly graded sand ^I
		Sands with Fines: More than 12% fines ^D	Fines classify as ML or MH	SM	Silty sand ^{G, H, I}
			Fines classify as CL or CH	SC	Clayey sand ^{G, H, I}
Fine-Grained Soils: 50% or more passes the No. 200 sieve	Silts and Clays: Liquid limit less than 50	Inorganic:	PI > 7 and plots above “A” line ^J	CL	Lean clay ^{K, L, M}
			PI < 4 or plots below “A” line ^J	ML	Silt ^{K, L, M}
		Organic:	$\frac{LL \text{ oven dried}}{LL \text{ not dried}} < 0.75$	OL	Organic clay ^{K, L, M, N} Organic silt ^{K, L, M, O}
	Silts and Clays: Liquid limit 50 or more	Inorganic:	PI plots on or above “A” line	CH	Fat clay ^{K, L, M}
			PI plots below “A” line	MH	Elastic silt ^{K, L, M}
		Organic:	$\frac{LL \text{ oven dried}}{LL \text{ not dried}} < 0.75$	OH	Organic clay ^{K, L, M, P} Organic silt ^{K, L, M, Q}
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

^A Based on the material passing the 3-inch (75-mm) sieve.

^B If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

^C Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

^D Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.

^E $Cu = D_{60}/D_{10}$ $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

^F If soil contains $\geq 15\%$ sand, add "with sand" to group name.

^G If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

^H If fines are organic, add "with organic fines" to group name.

^I If soil contains $\geq 15\%$ gravel, add "with gravel" to group name.

^J If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

^K If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

^L If soil contains $\geq 30\%$ plus No. 200 predominantly sand, add "sandy" to group name.

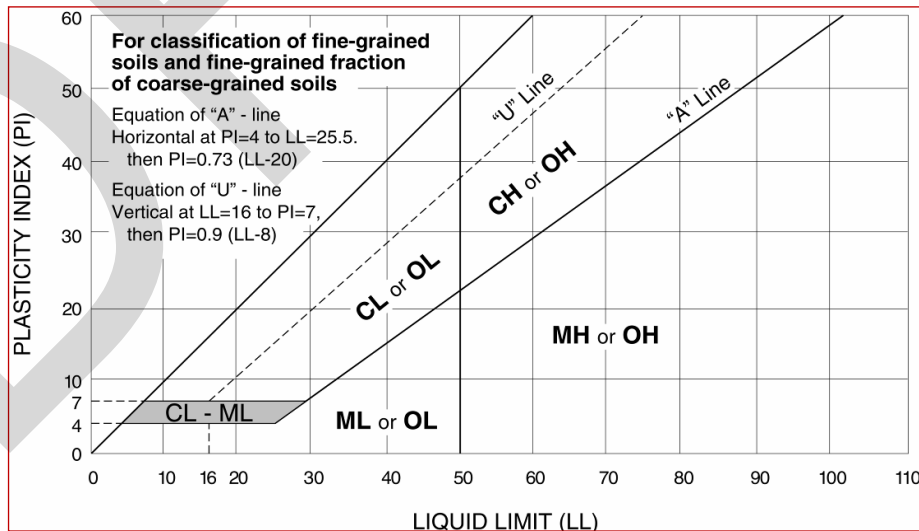
^M If soil contains $\geq 30\%$ plus No. 200, predominantly gravel, add "gravelly" to group name.

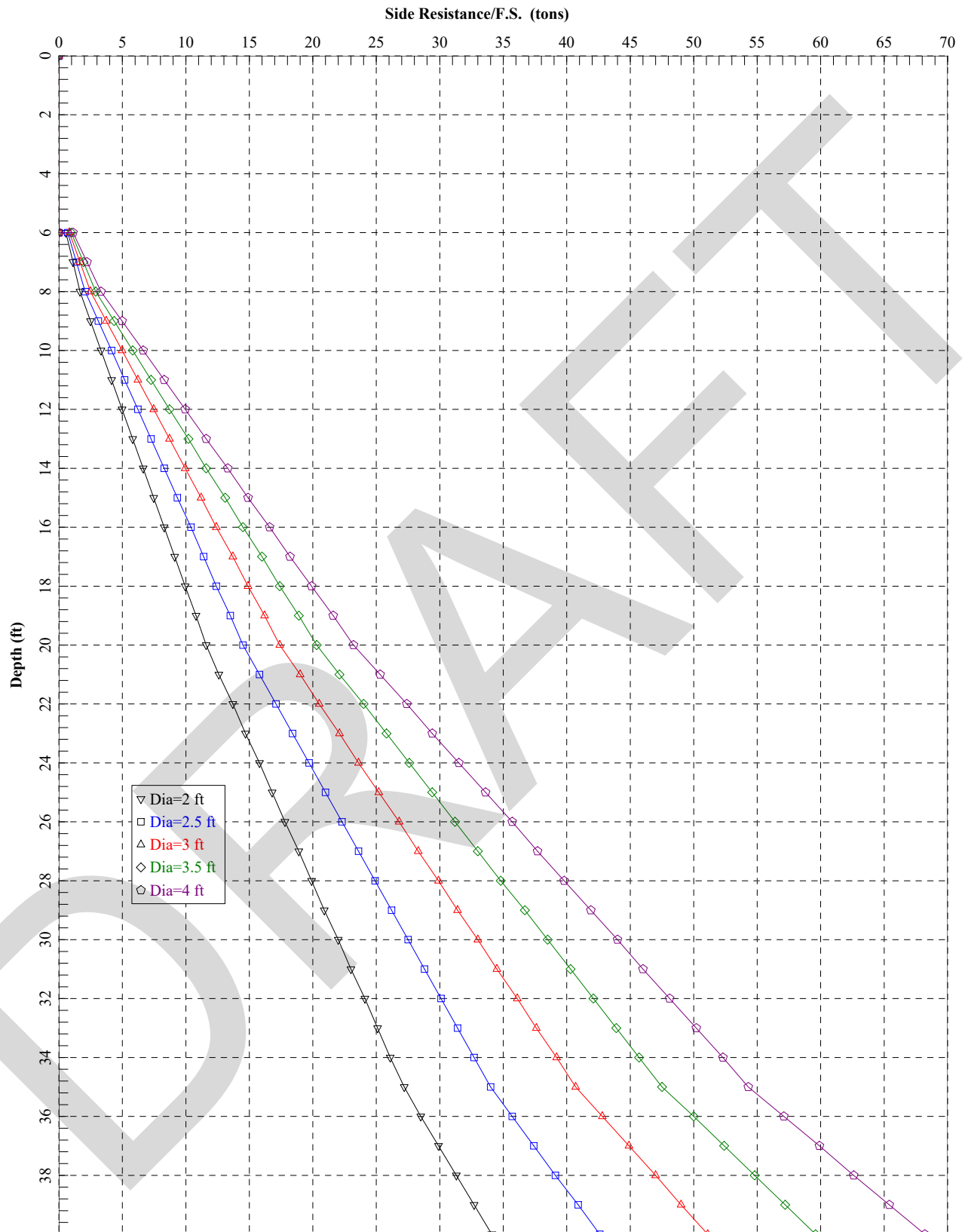
^N $PI \geq 4$ and plots on or above "A" line.

^O $PI < 4$ or plots below "A" line.

^P PI plots on or above "A" line.

^Q PI plots below "A" line.





R1245076-Calpine Tanager - Drilled Cast-in-Place Shaft - Allowable Axial Compression Capacity

LIQUEFACTION ANALYSIS REPORT

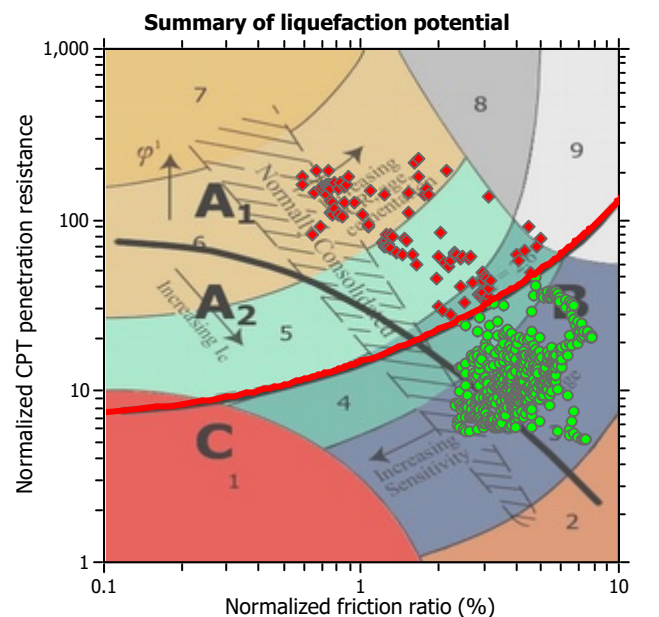
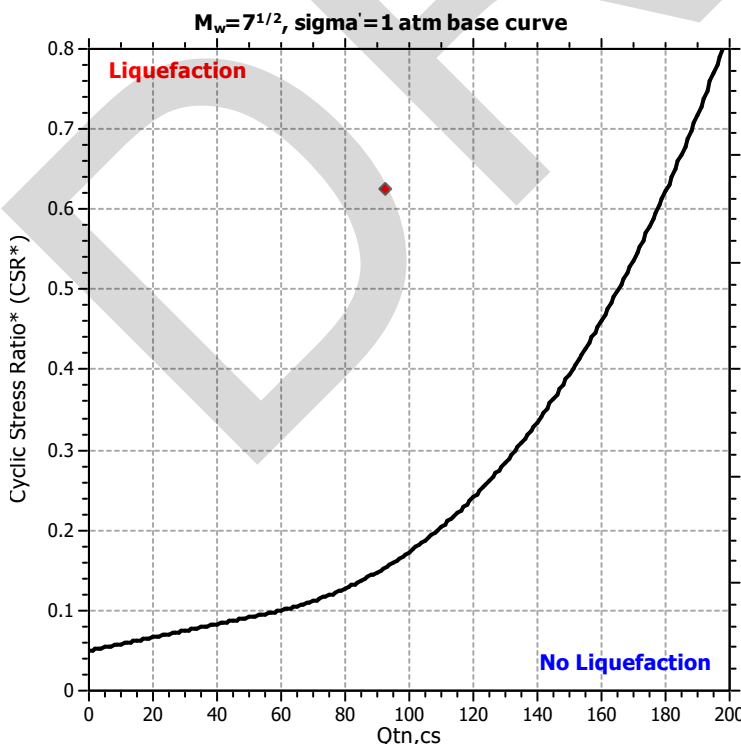
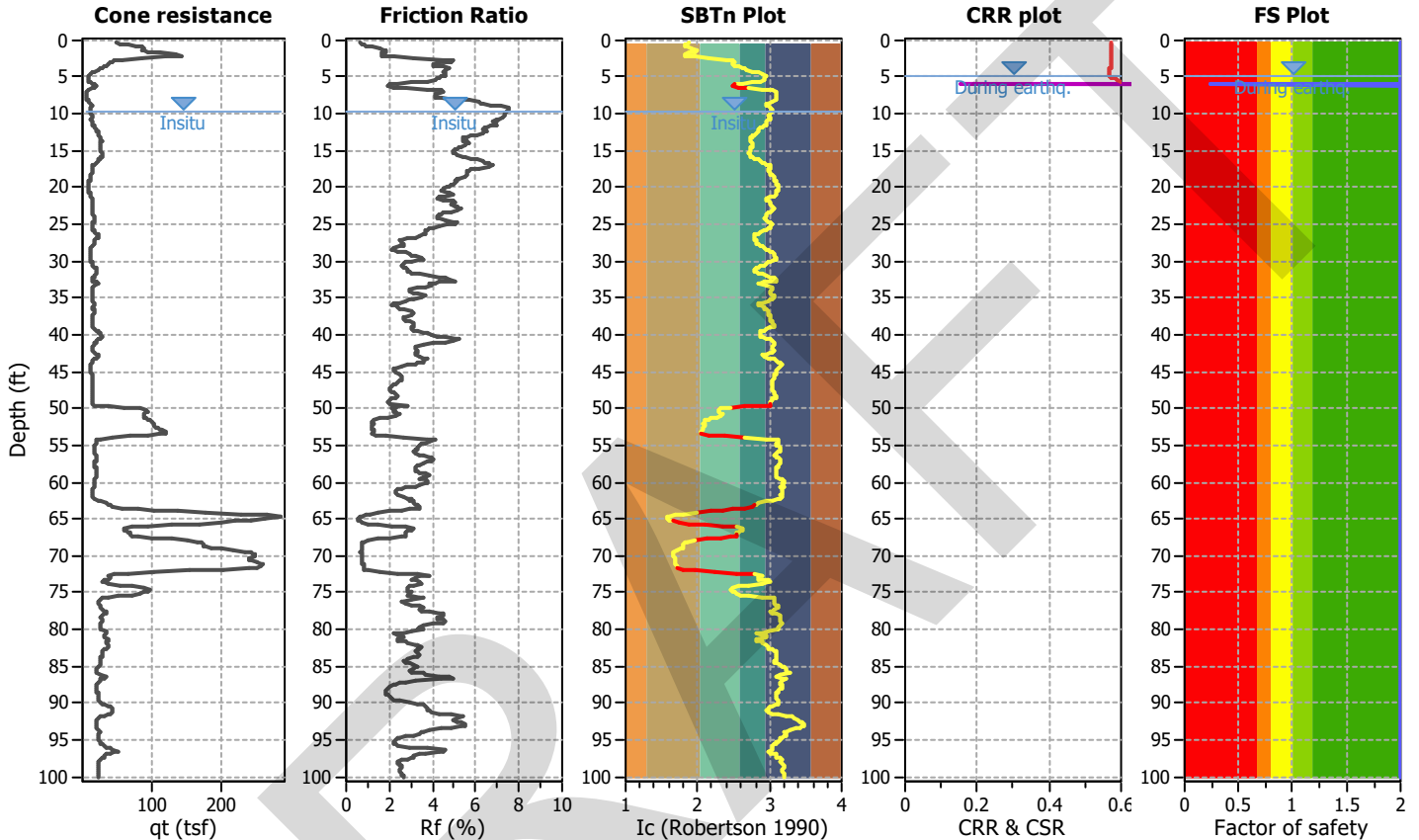
Project title : Calpine Tanager Energy Storage Project

Location : San Jose, CA

CPT file : CPT-01

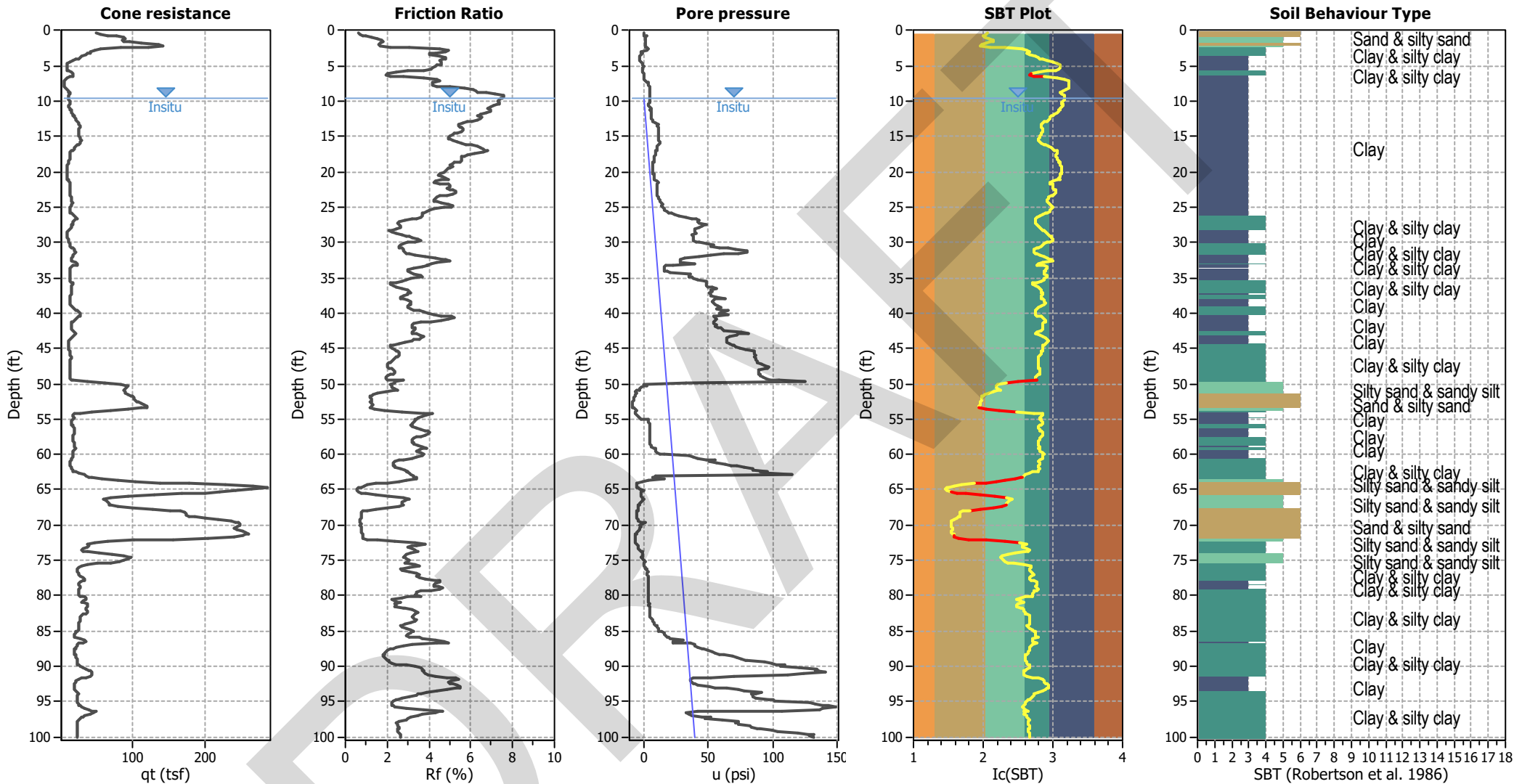
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	9.60 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w :	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.81	Unit weight calculation:	Based on SBT	K_o applied:	Yes		



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



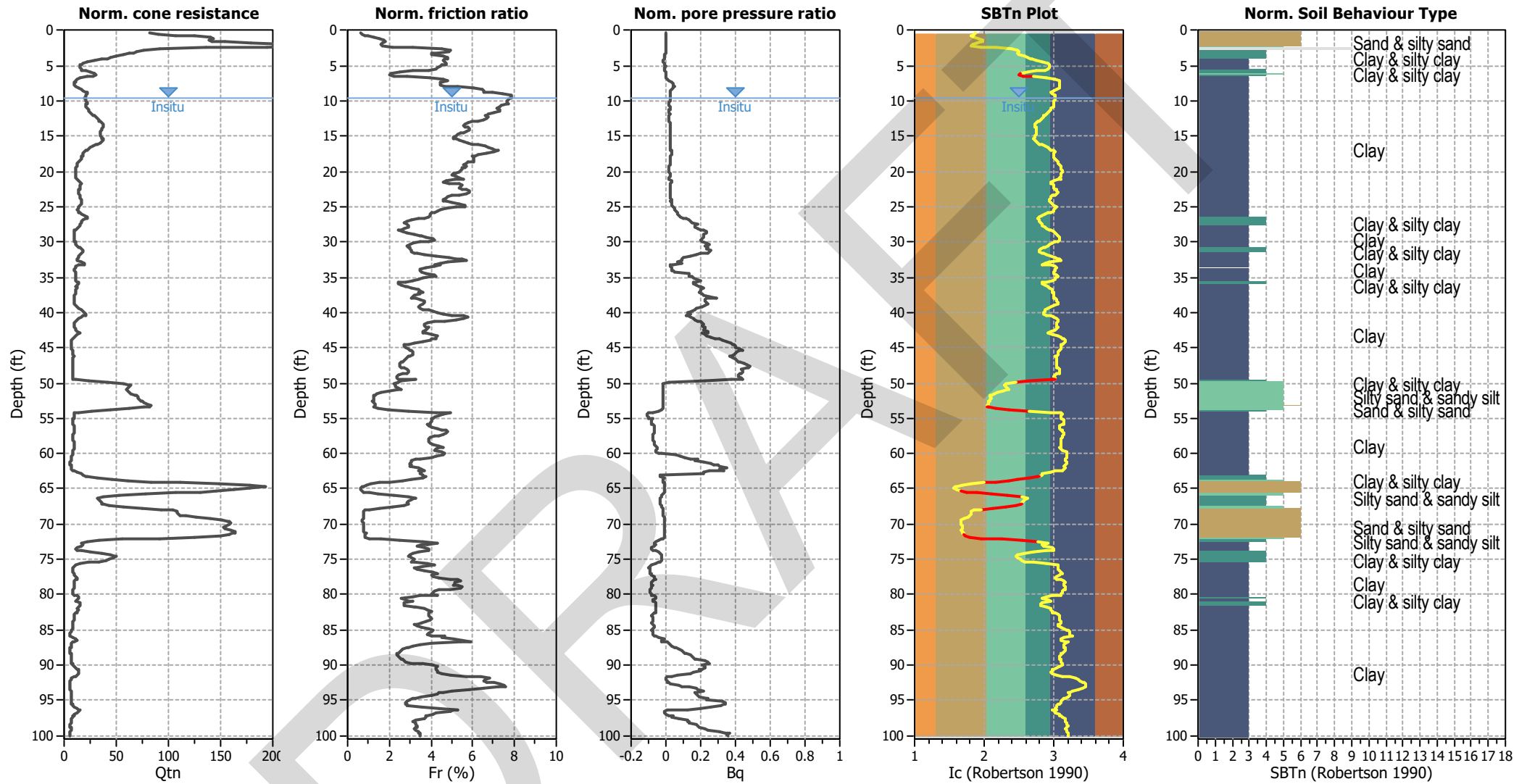
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.60 ft	Fill height:	N/A	Limit depth:	50.00 ft

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

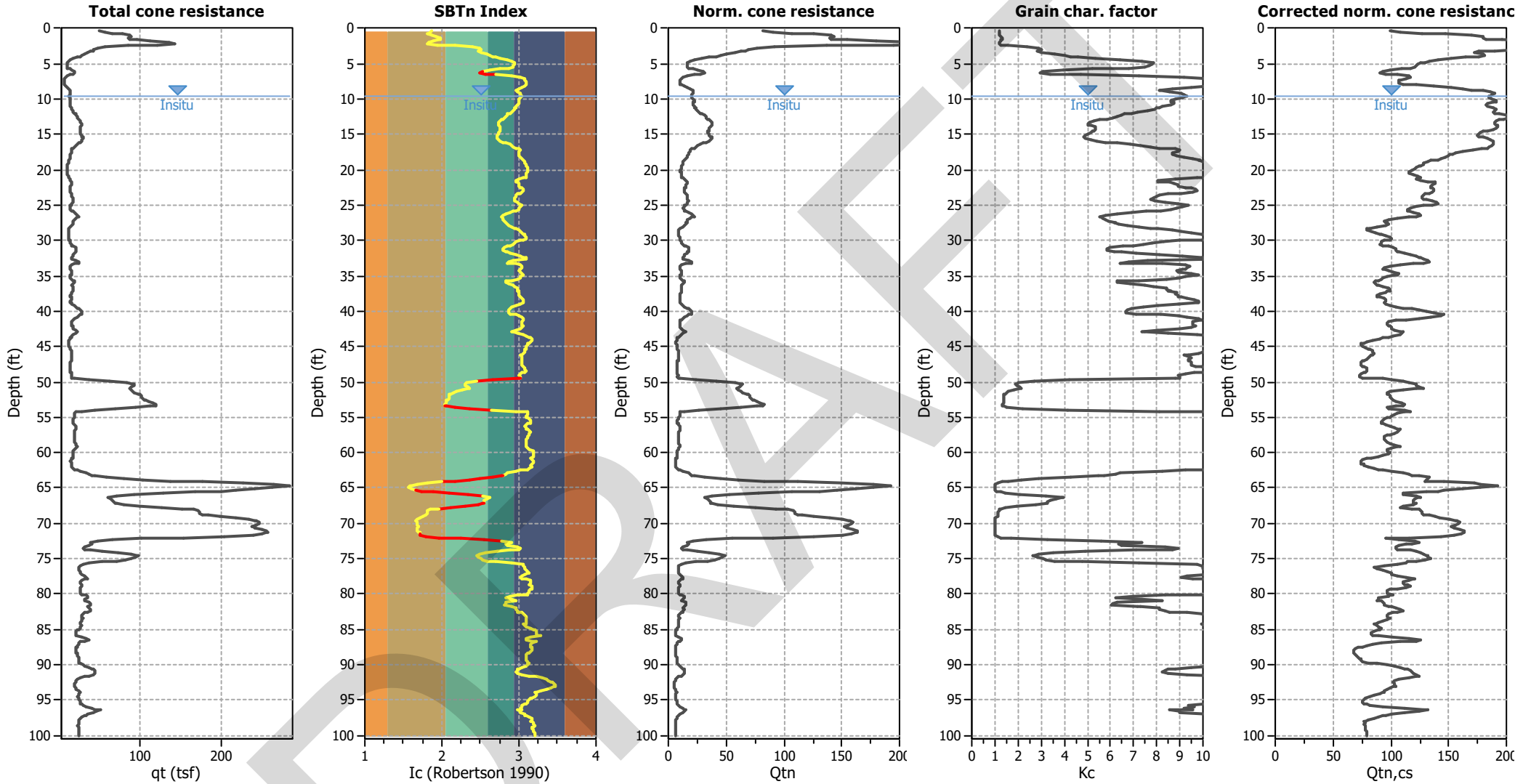
CPT basic interpretation plots (normalized)



Input parameters and analysis data

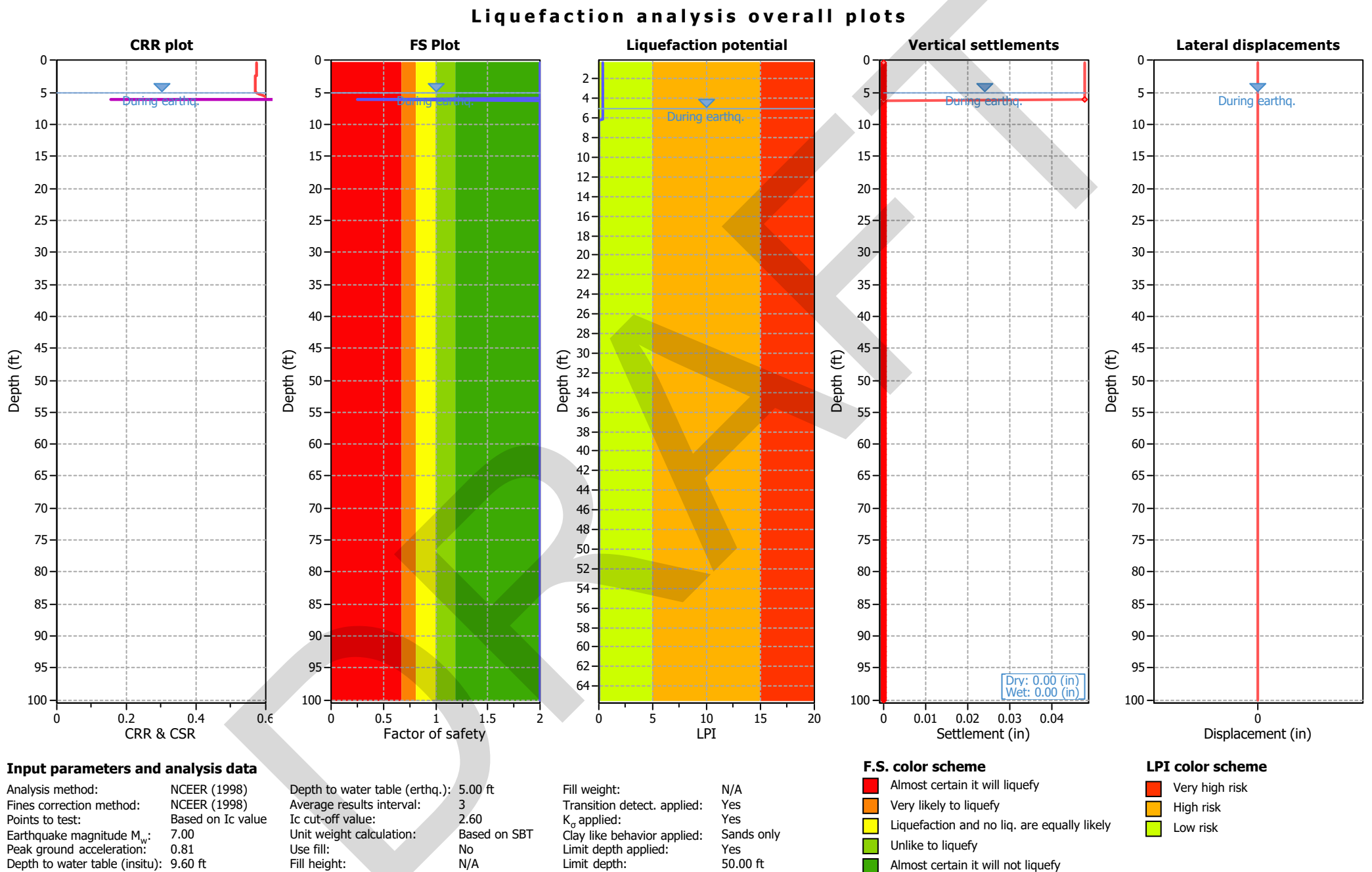
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.60 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots (intermediate results)

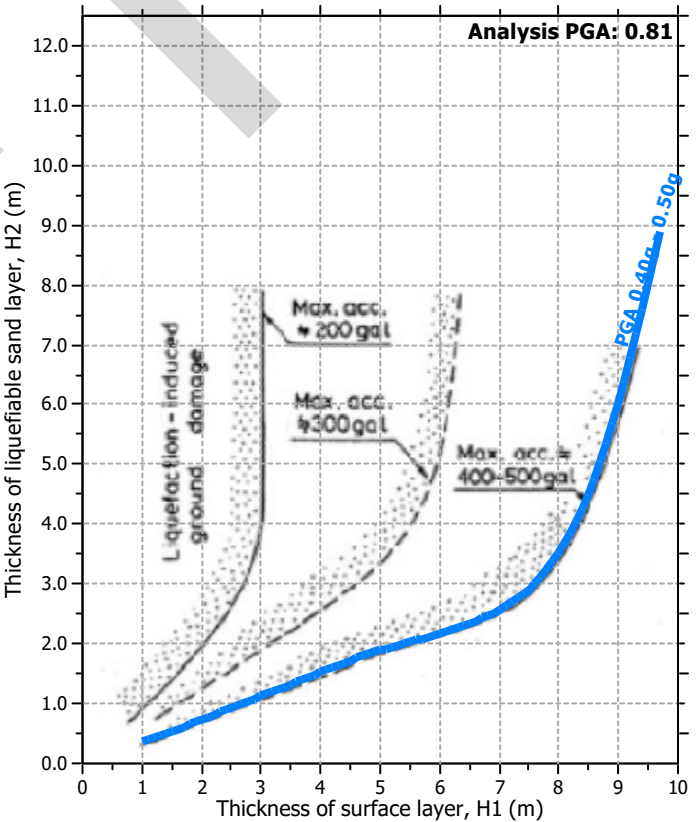
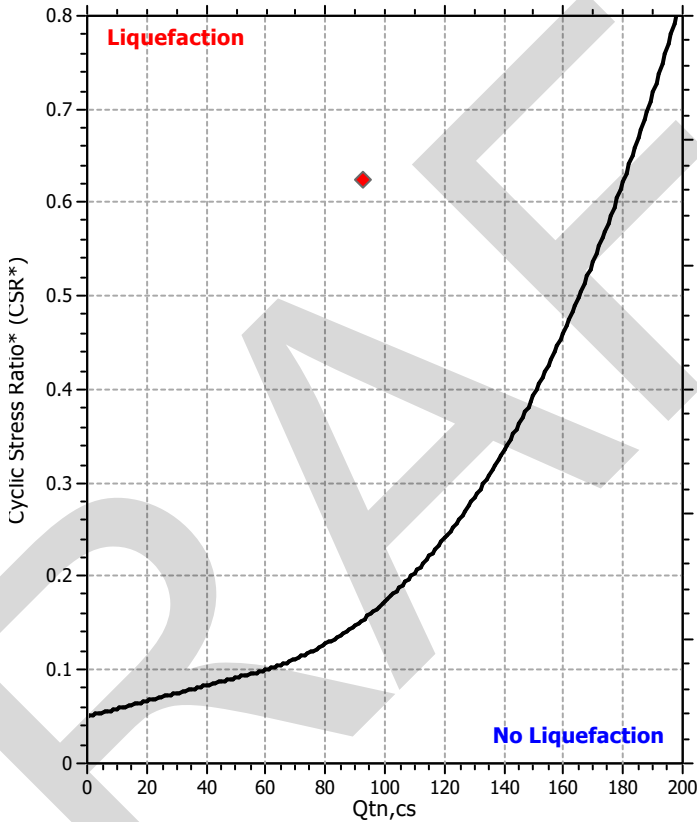
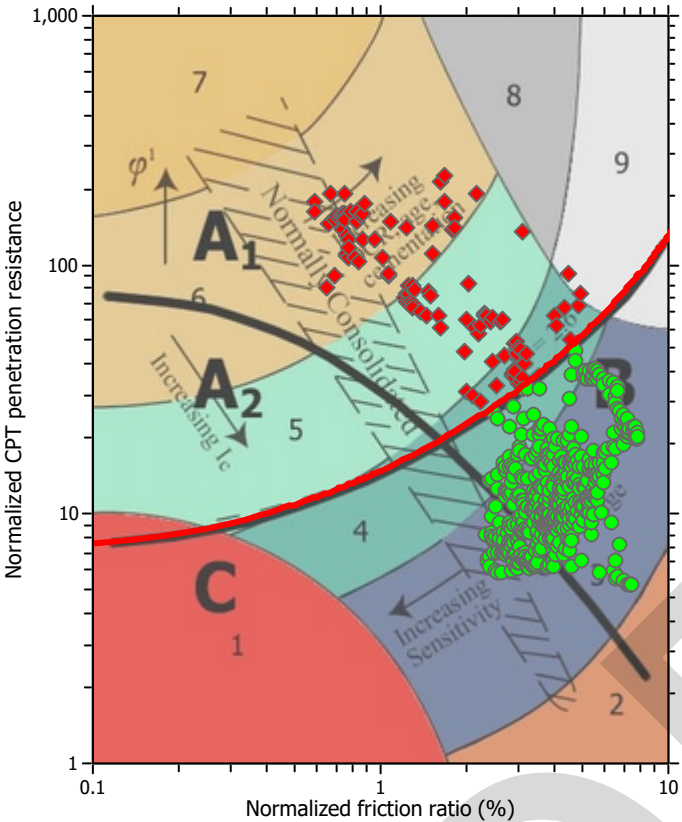


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.60 ft	Fill height:	N/A	Limit depth:	50.00 ft



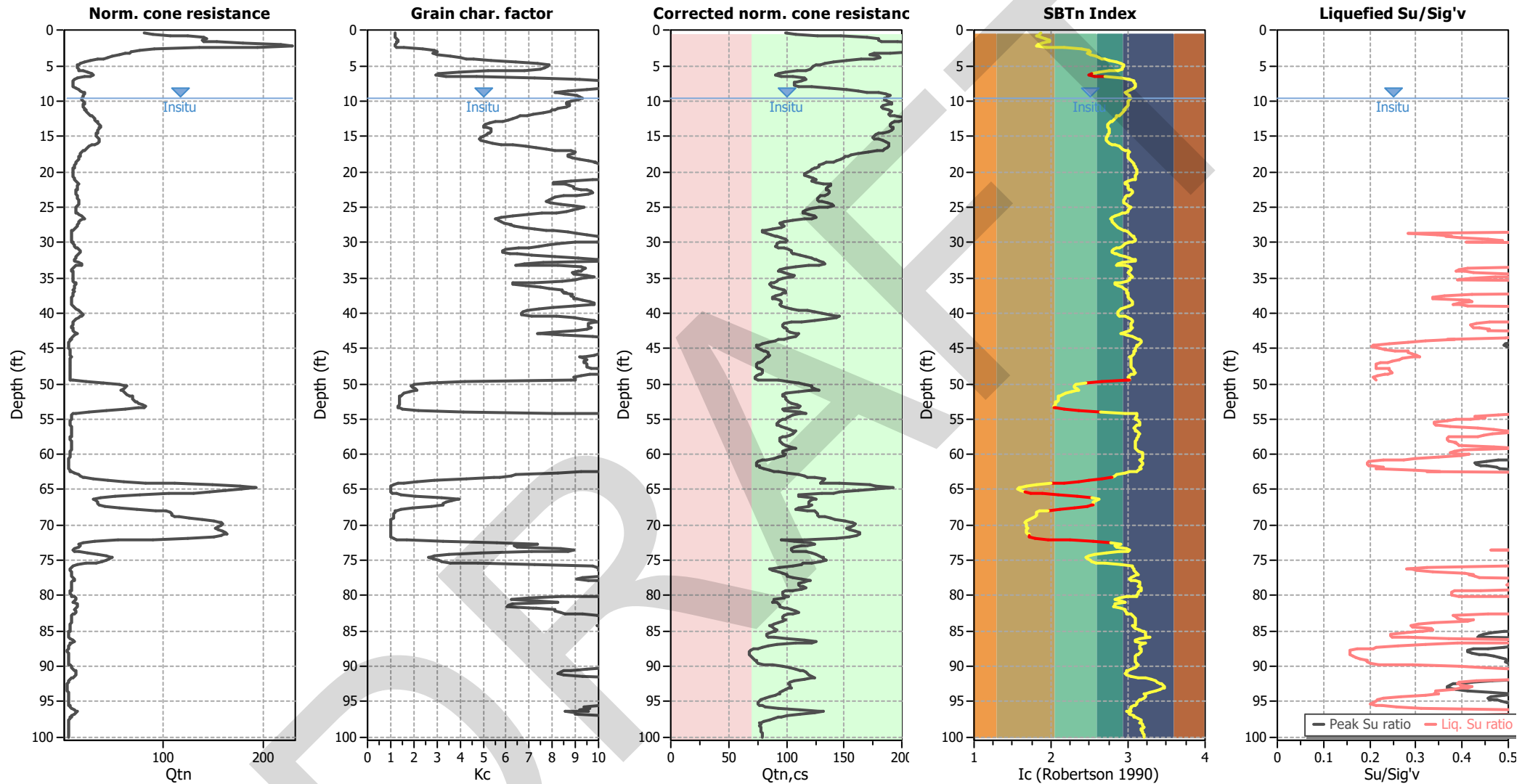
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.60 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.60 ft	Fill height:	N/A	Limit depth:	50.00 ft

LIQUEFACTION ANALYSIS REPORT

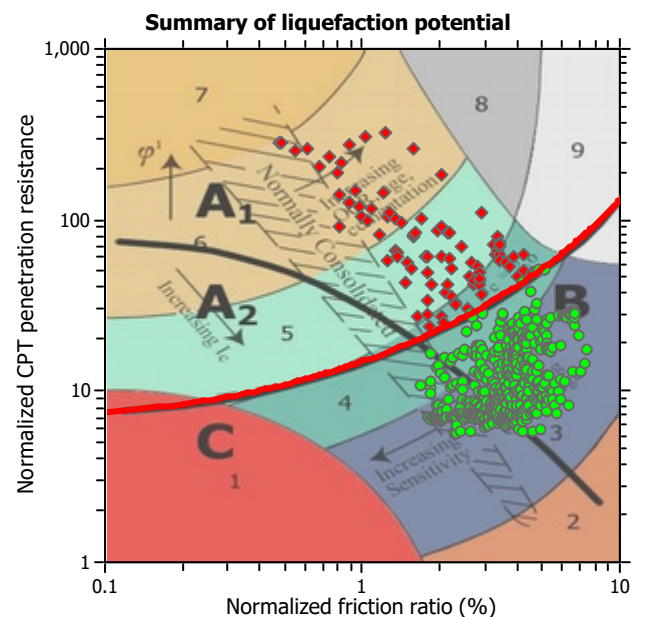
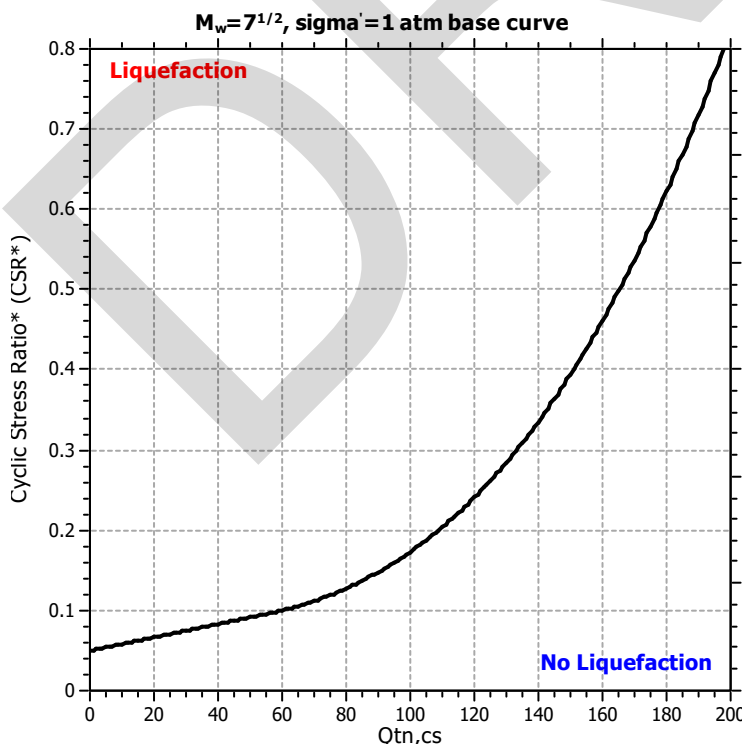
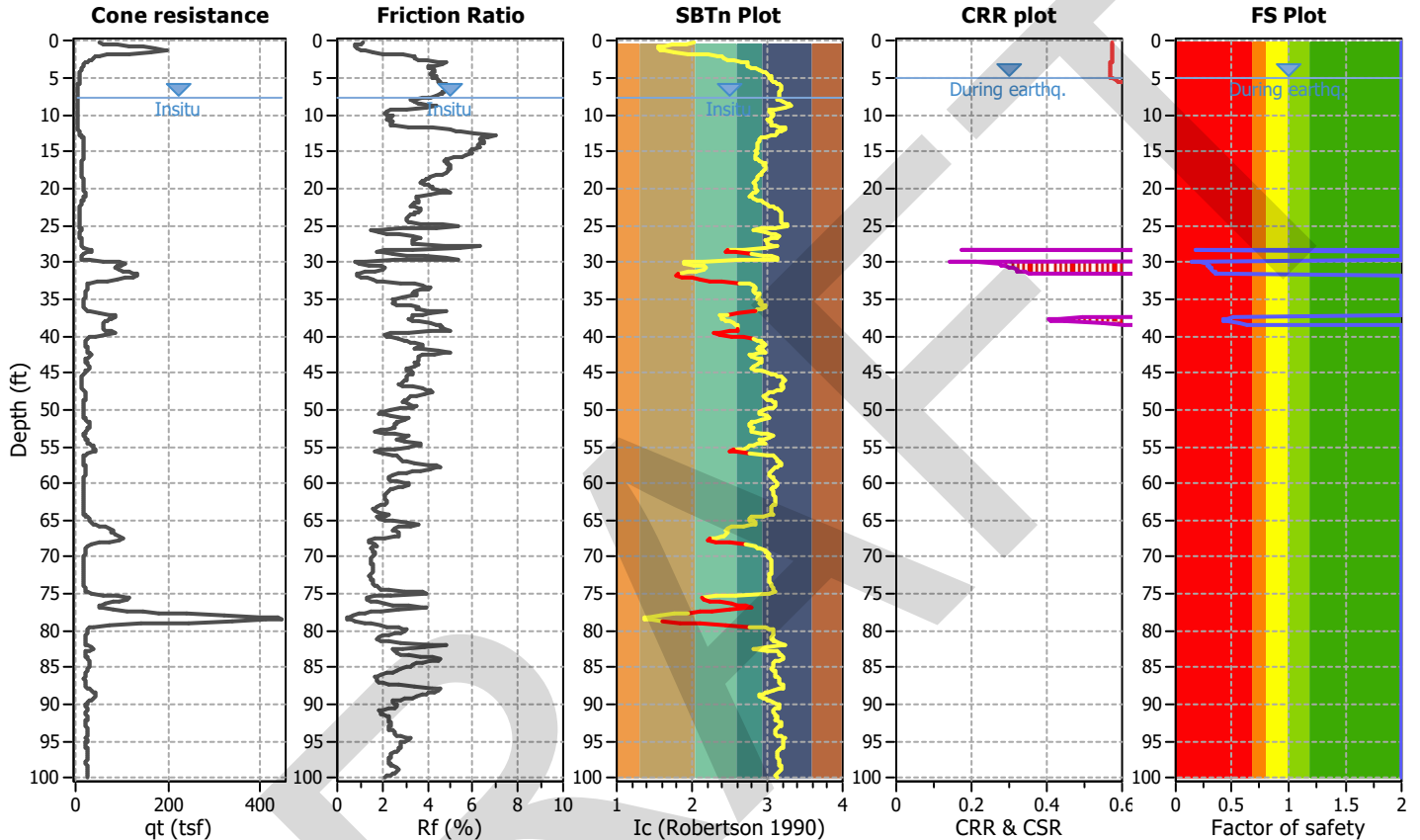
Project title : Calpine Tanager Energy Storage Project

Location : San Jose, CA

CPT file : CPT-02

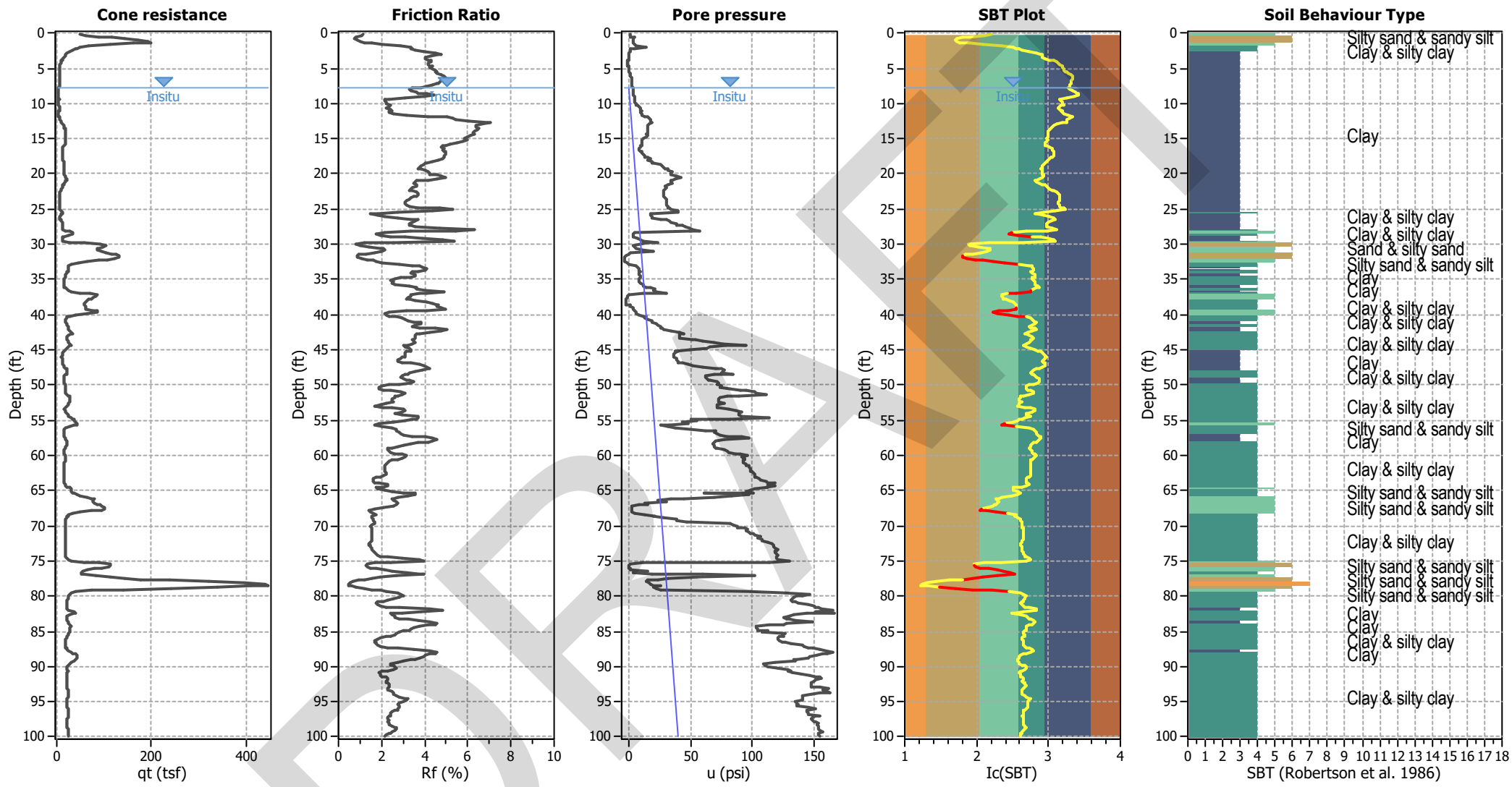
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.80 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w :	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.81	Unit weight calculation:	Based on SBT	K_o applied:	Yes		



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



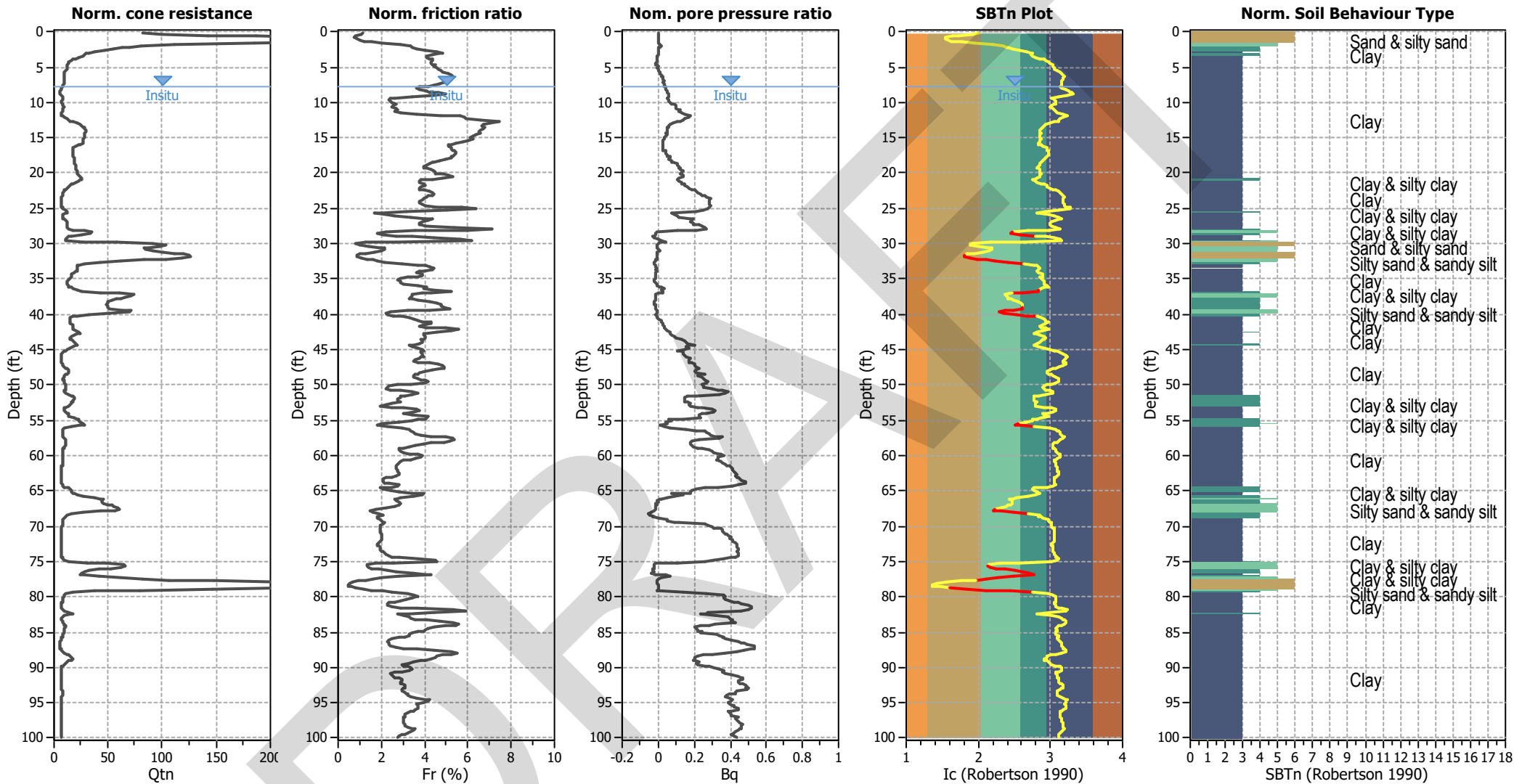
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.80 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

1. Sensitive fine grained	4. Clayey silt to silty	7. Gravelly 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)



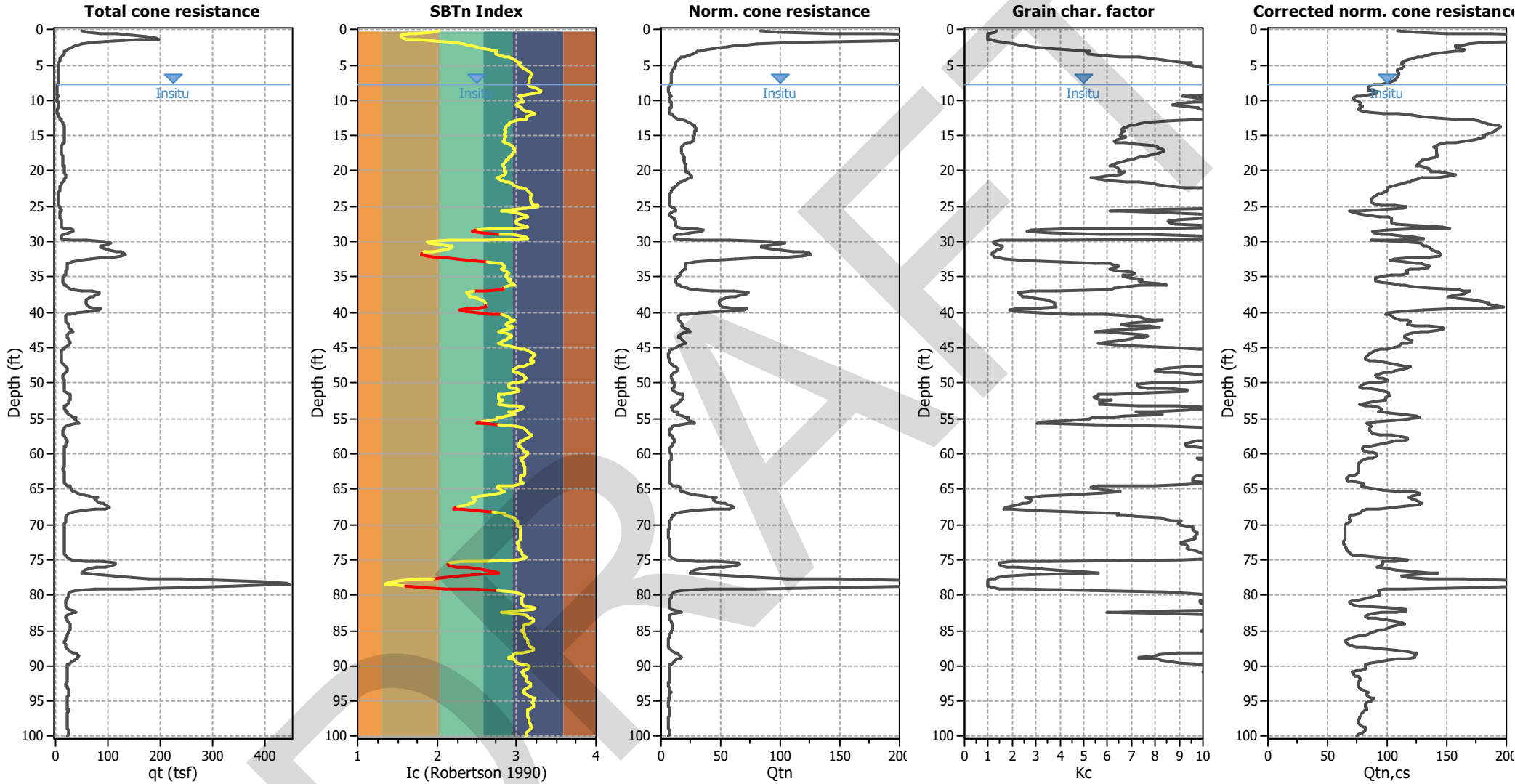
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.80 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBTn legend

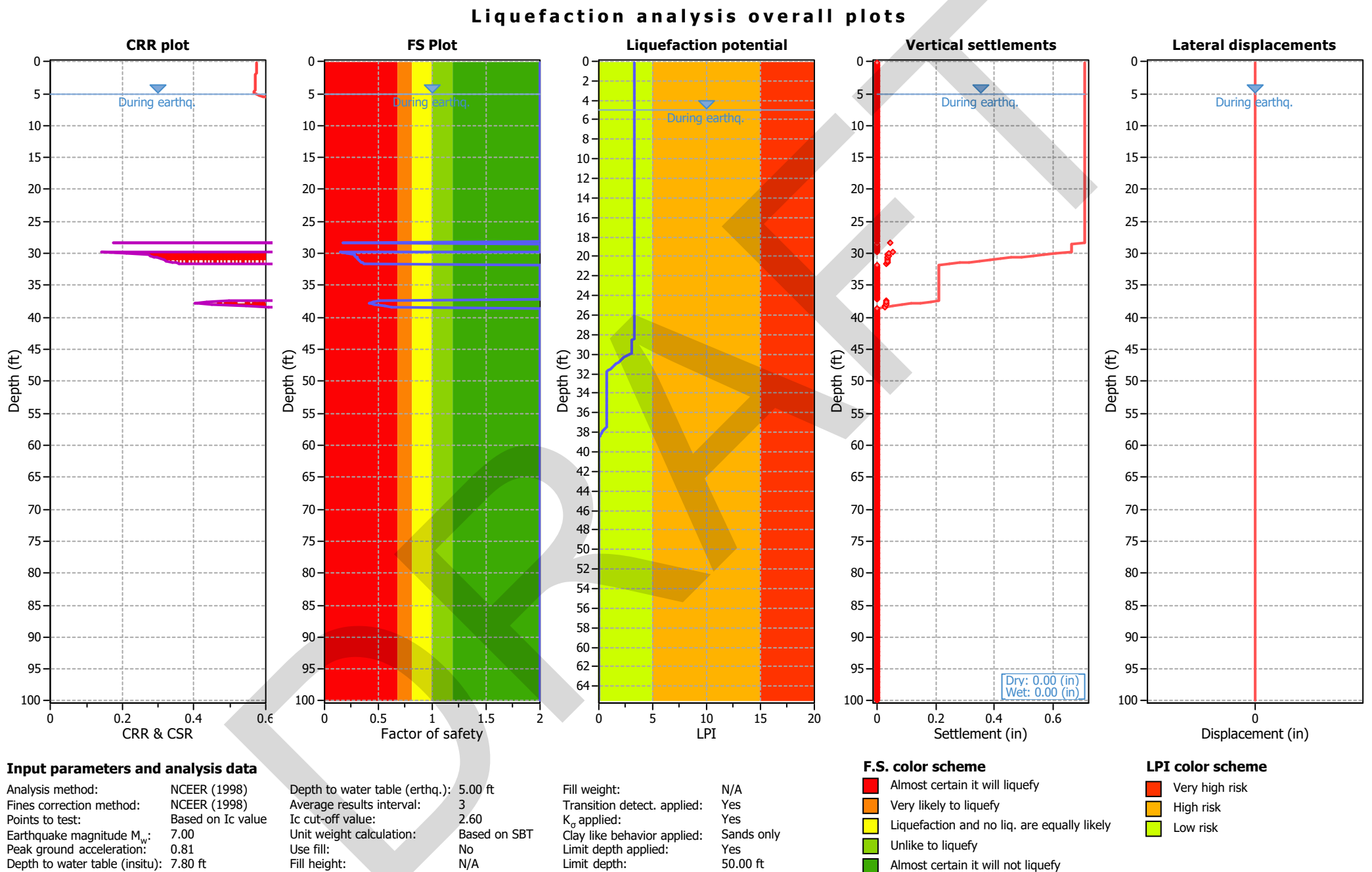
1. Sensitive fine grained	4. Clayey silt to silty	7. Gravely sand to sand
2. Organic material	5. Silty sand to sandy silt	8. Very stiff sand to
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)

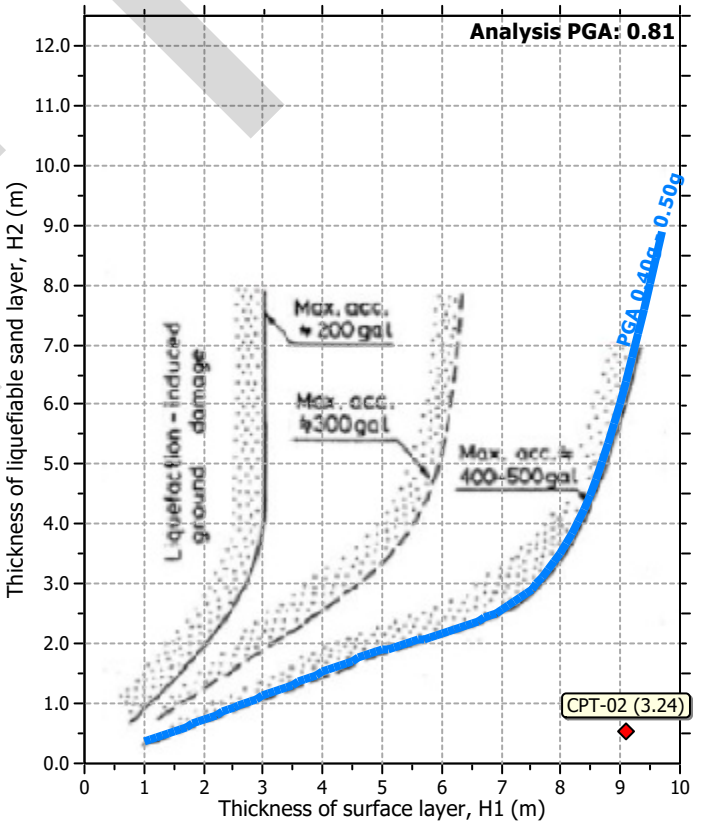
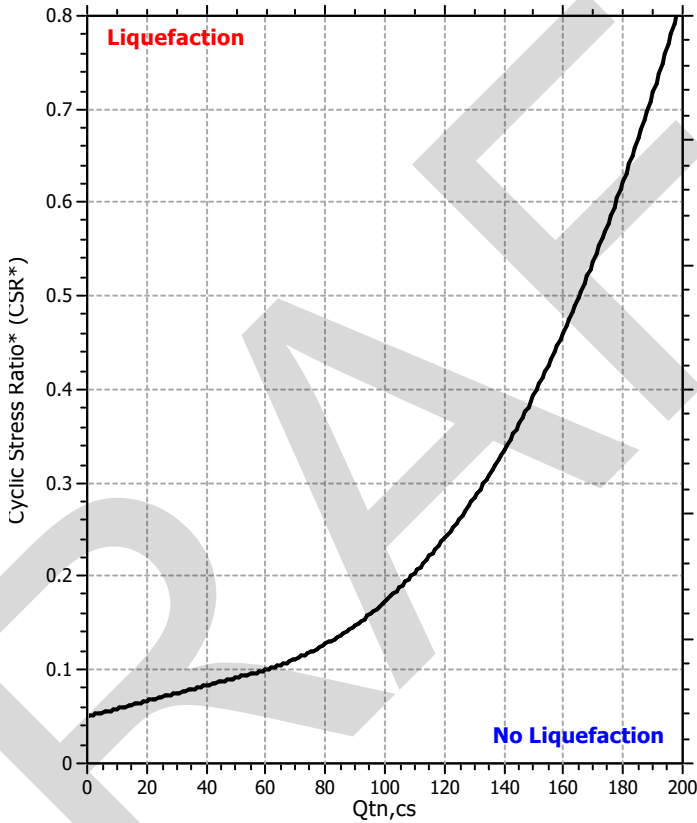
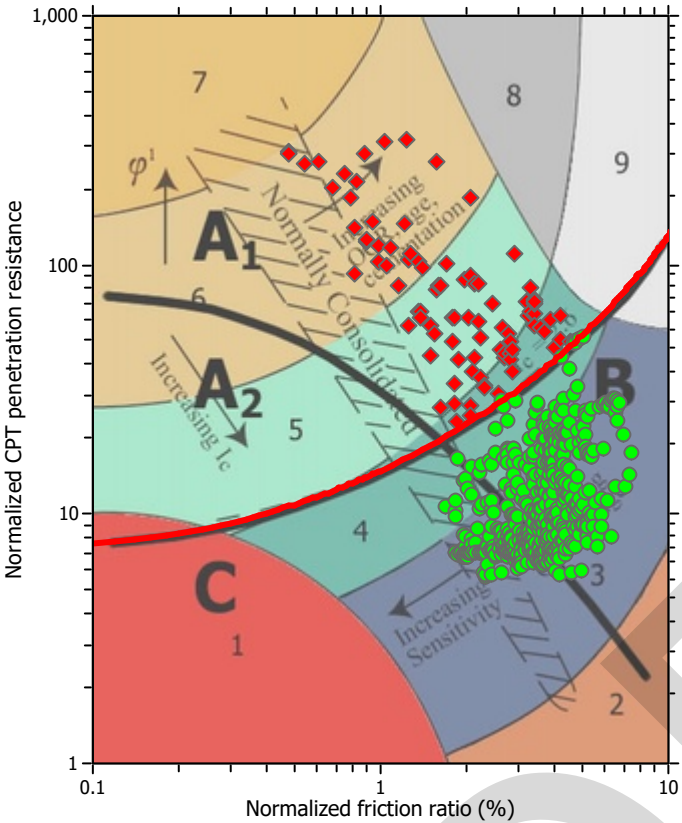


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.80 ft	Fill height:	N/A	Limit depth:	50.00 ft



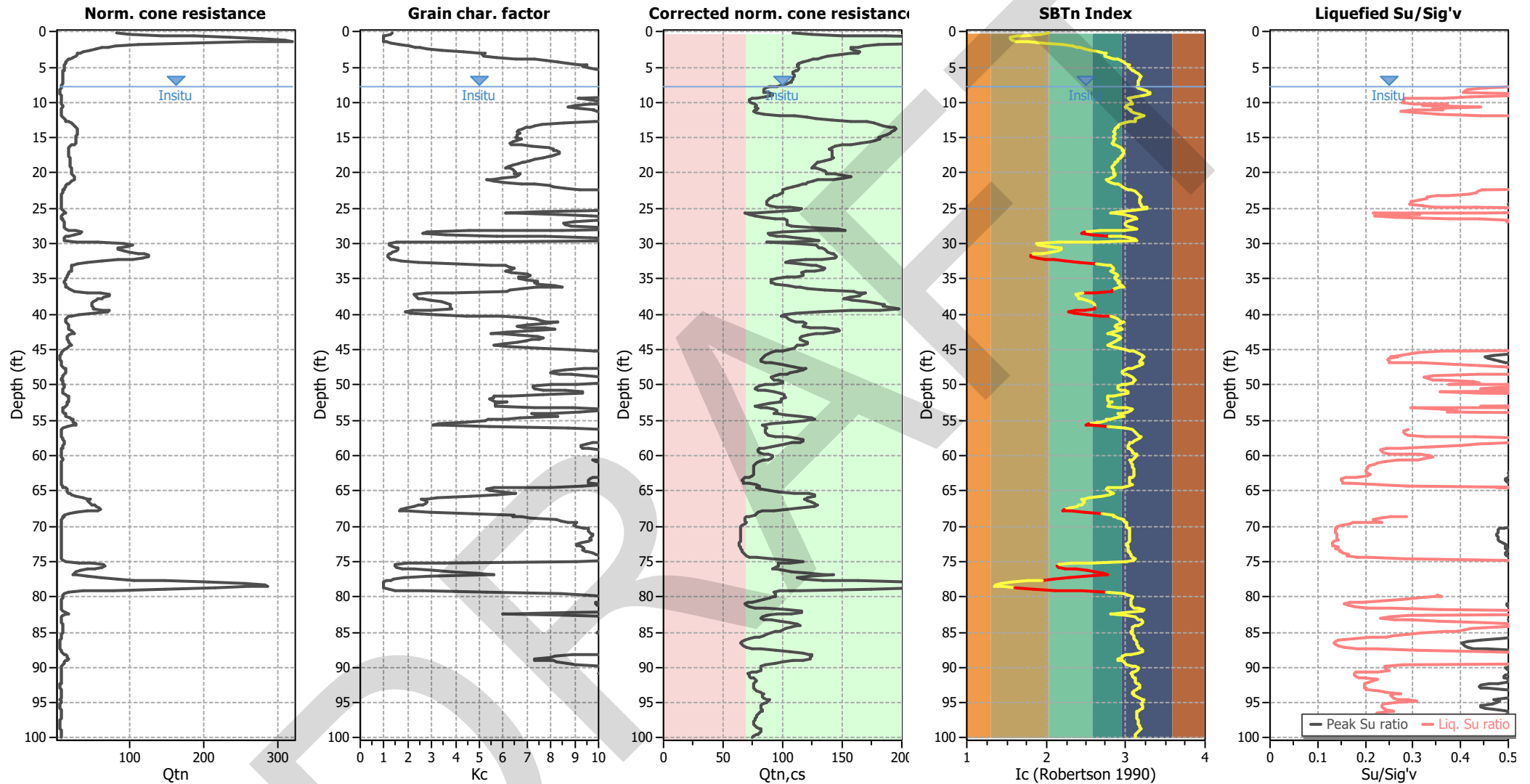
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.80 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.80 ft	Fill height:	N/A	Limit depth:	50.00 ft

LIQUEFACTION ANALYSIS REPORT

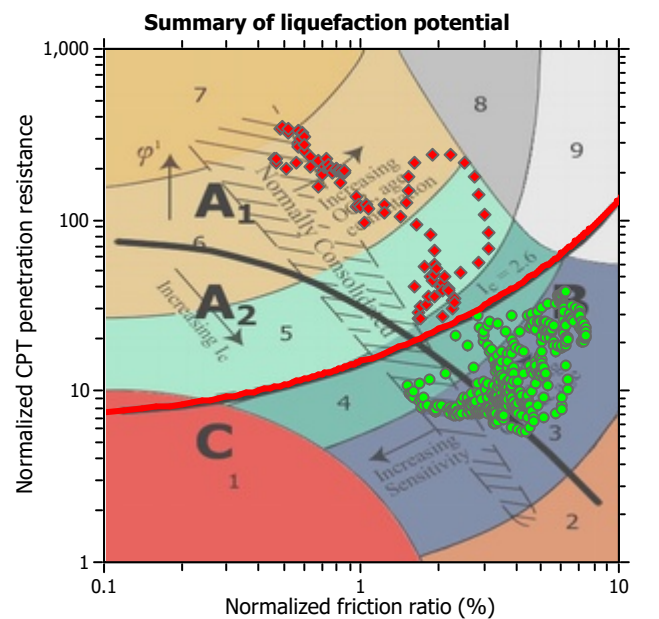
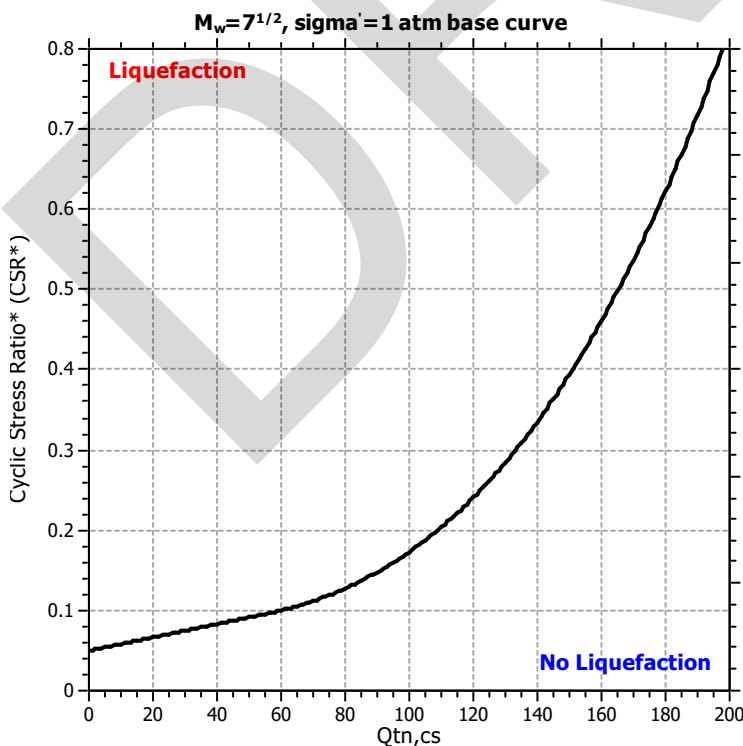
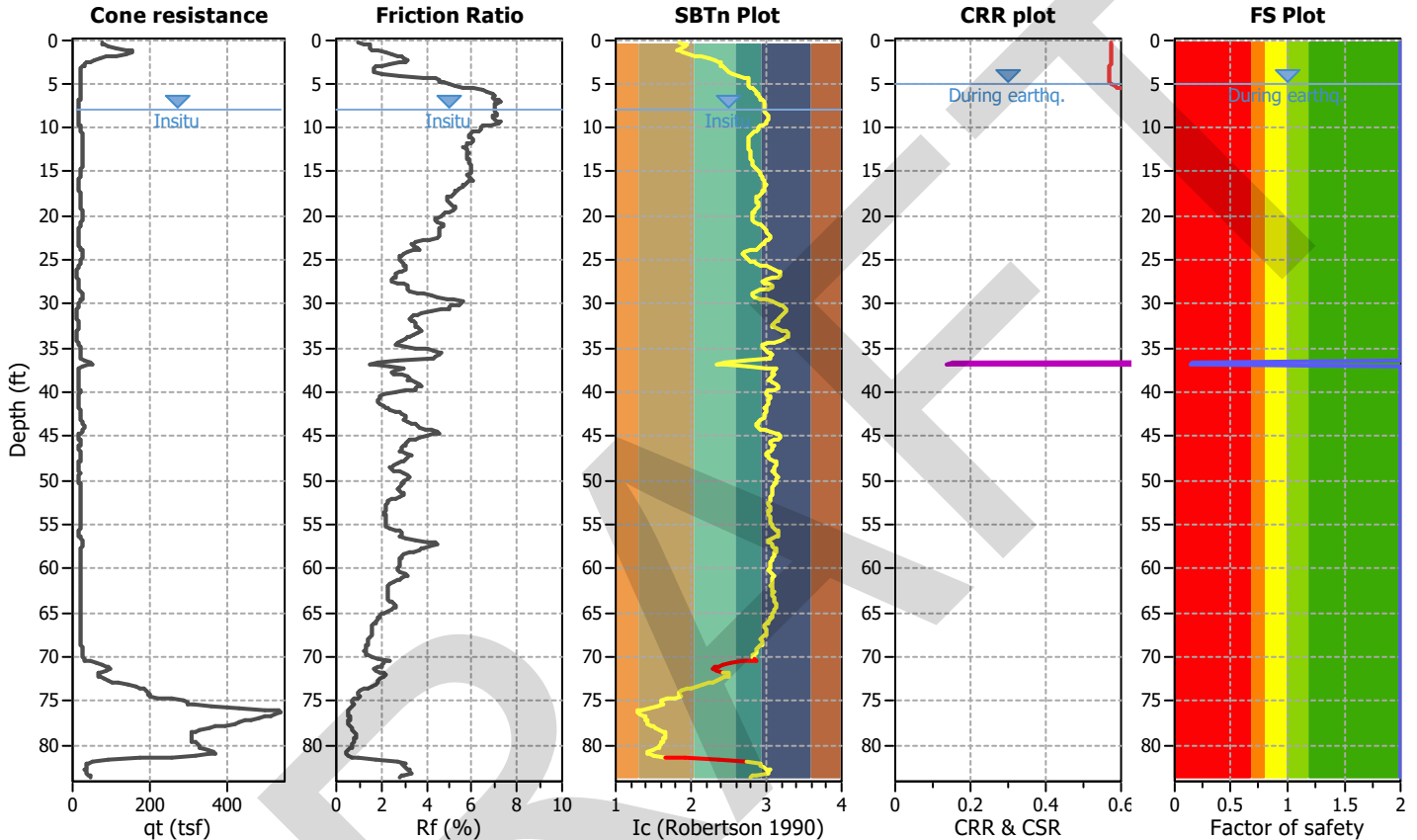
Project title : Calpine Tanager Energy Storage Project

Location : San Jose, CA

CPT file : CPT-03

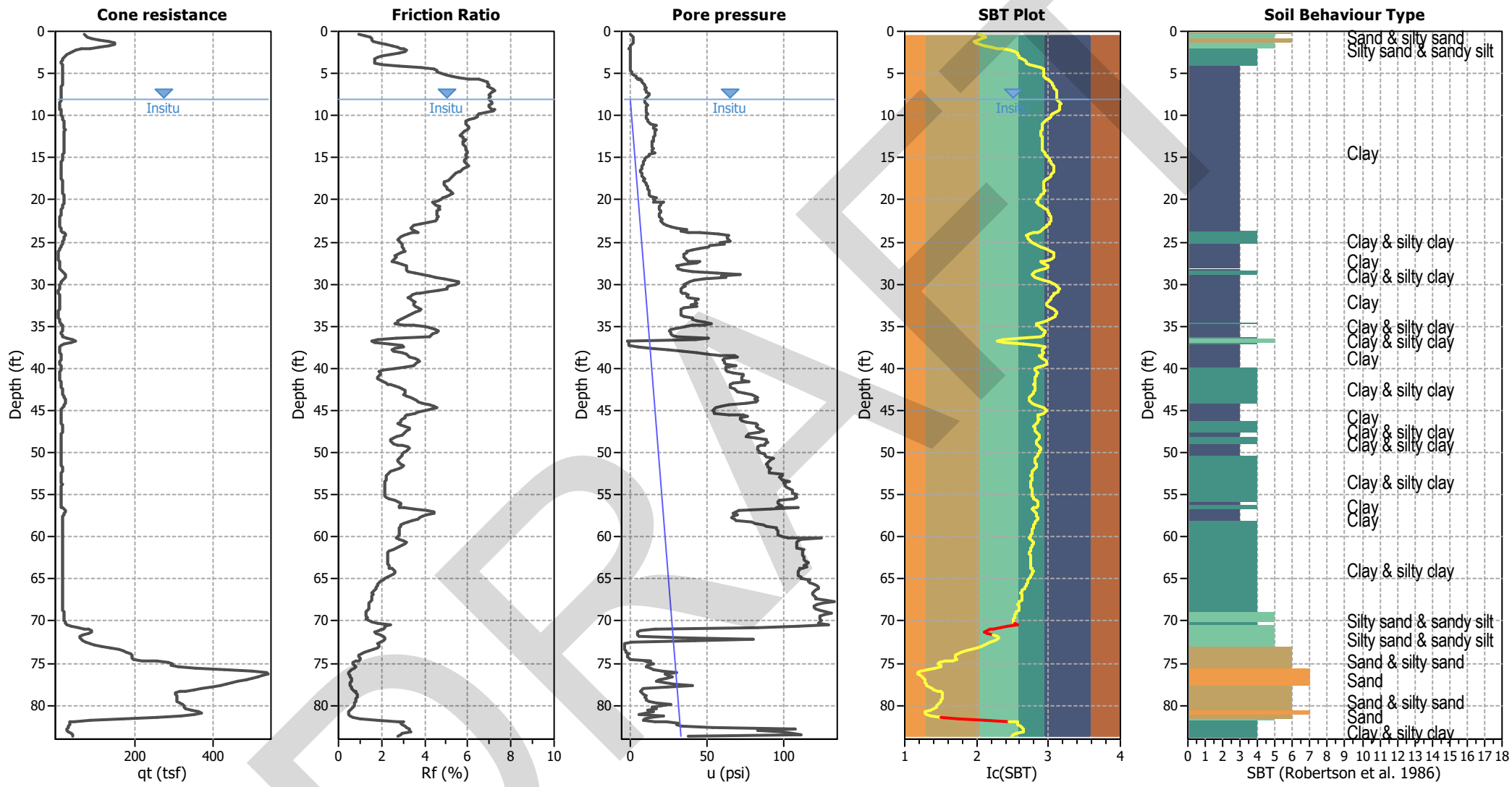
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w :	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.81	Unit weight calculation:	Based on SBT	K_o applied:	Yes		



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



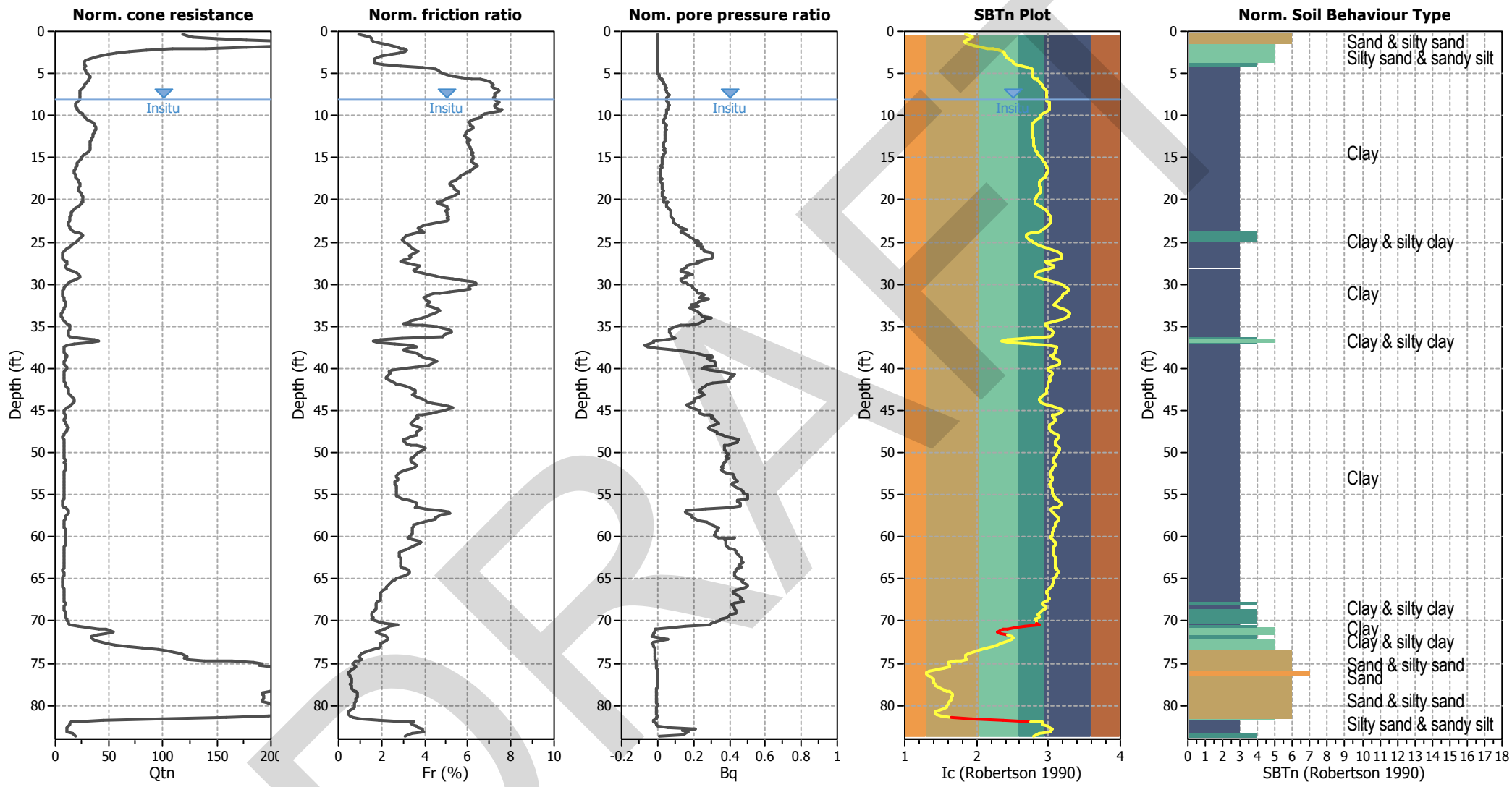
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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)



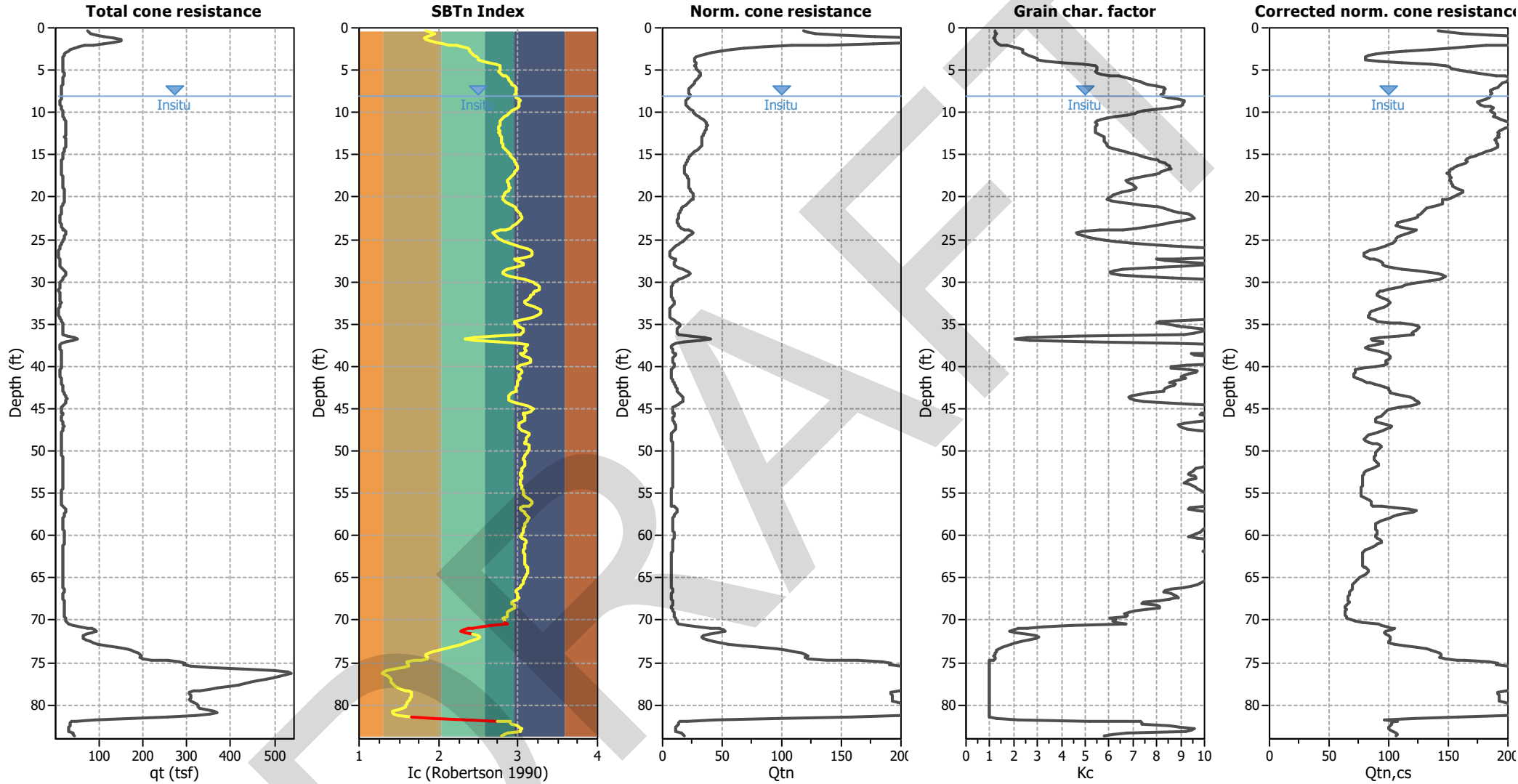
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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

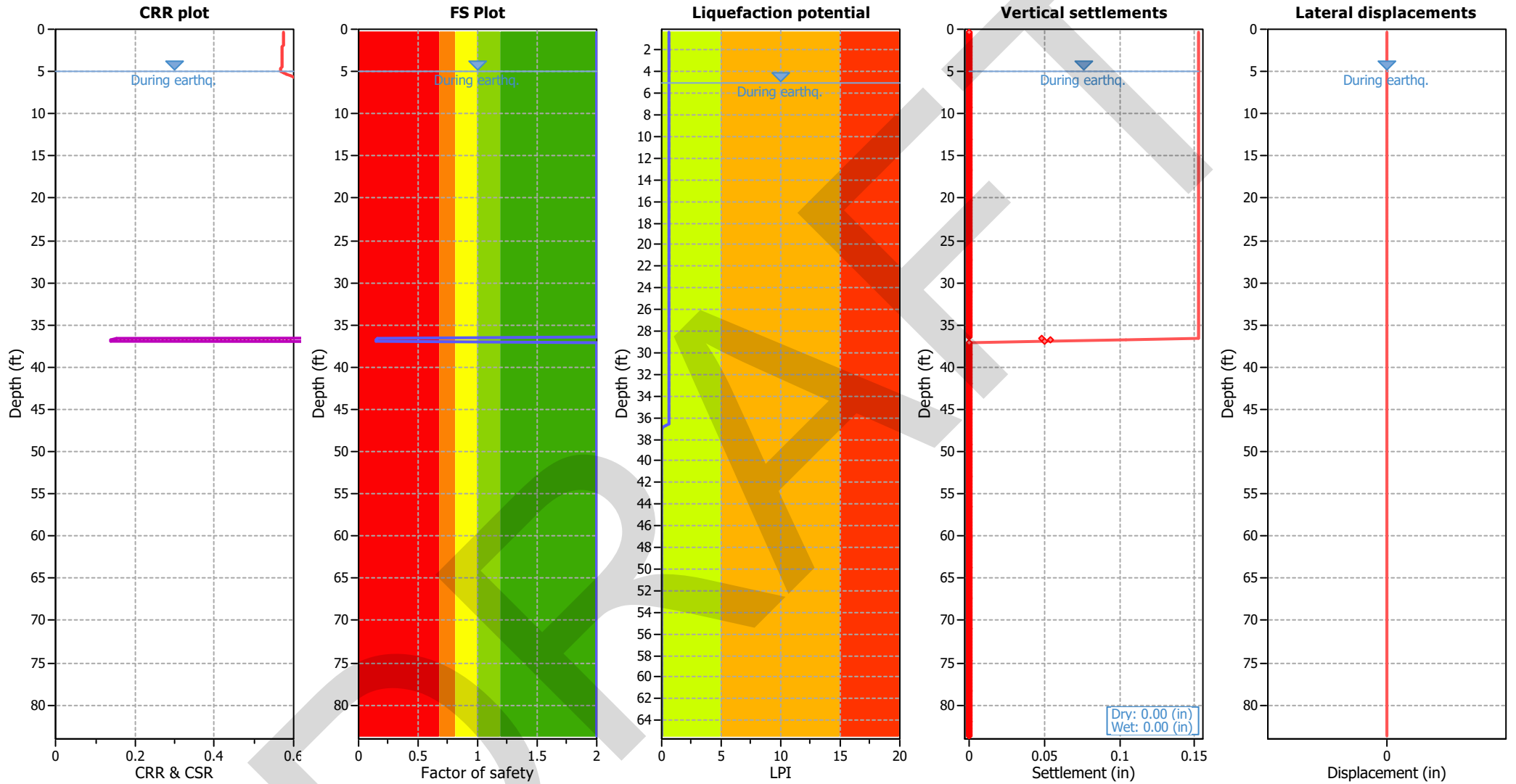
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

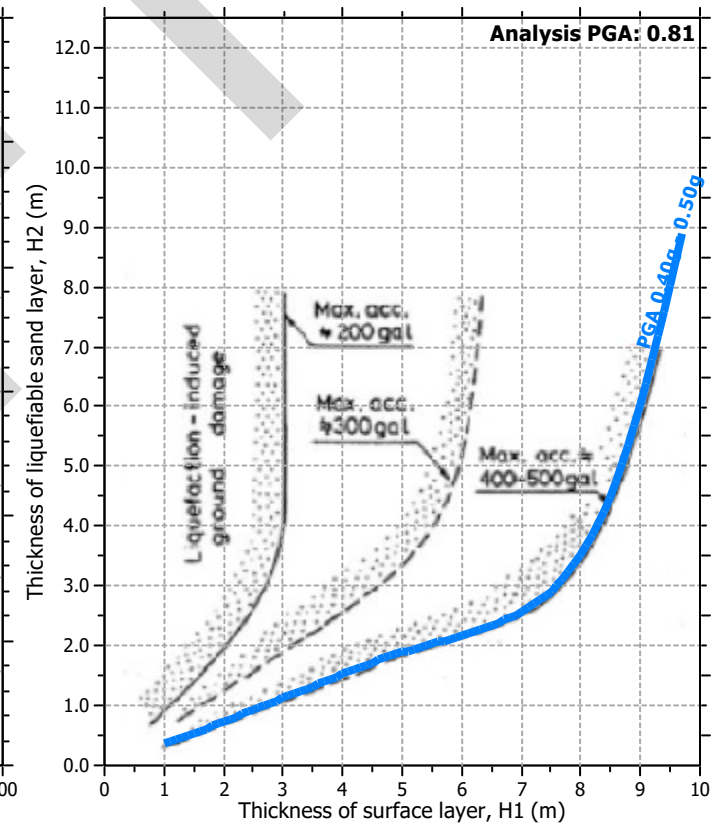
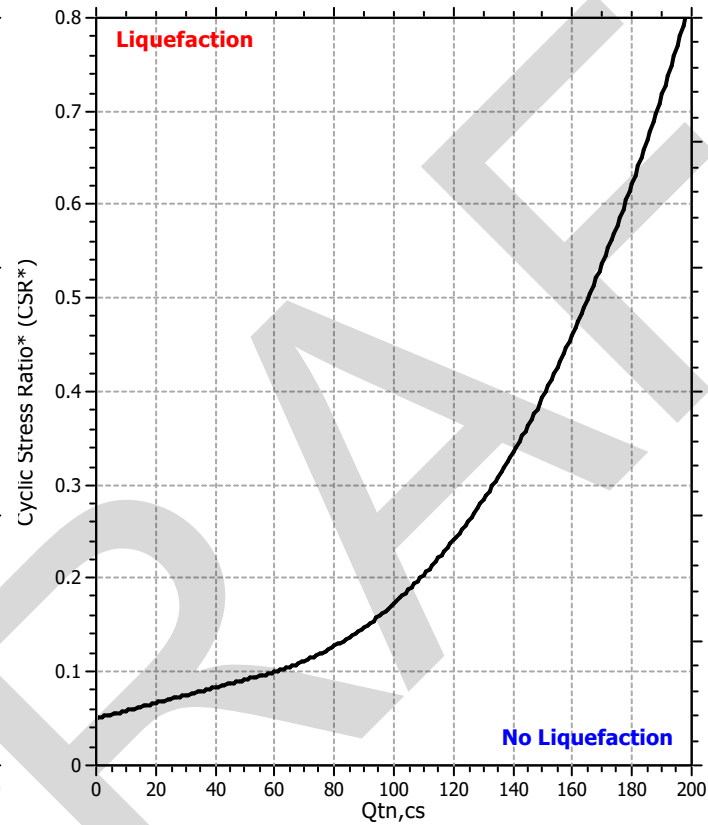
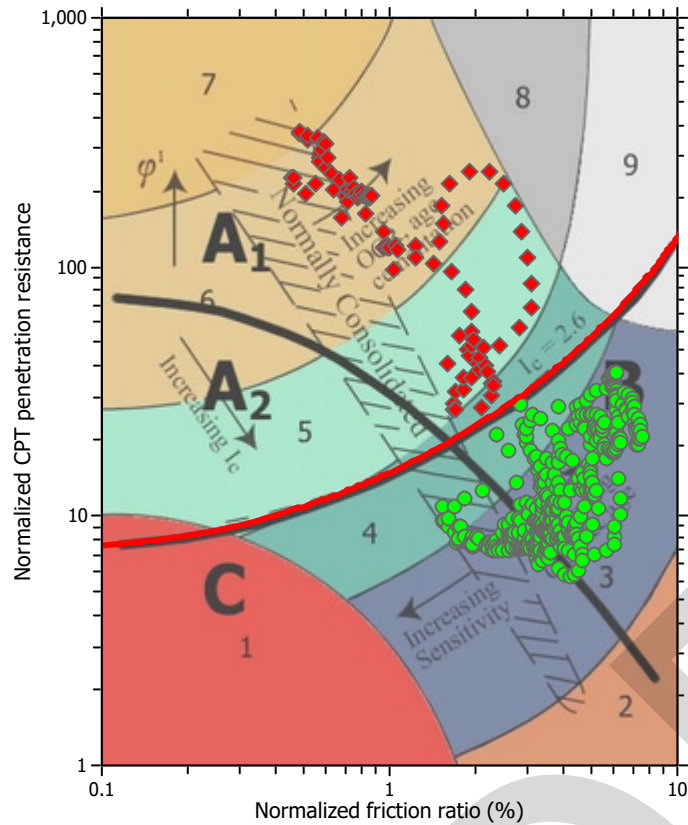
F.S. color scheme

Red	Almost certain it will liquefy
Orange	Very likely to liquefy
Yellow	Liquefaction and no liq. are equally likely
Green	Unlike to liquefy
Dark Green	Almost certain it will not liquefy

LPI color scheme

Red	Very high risk
Orange	High risk
Yellow	Low risk

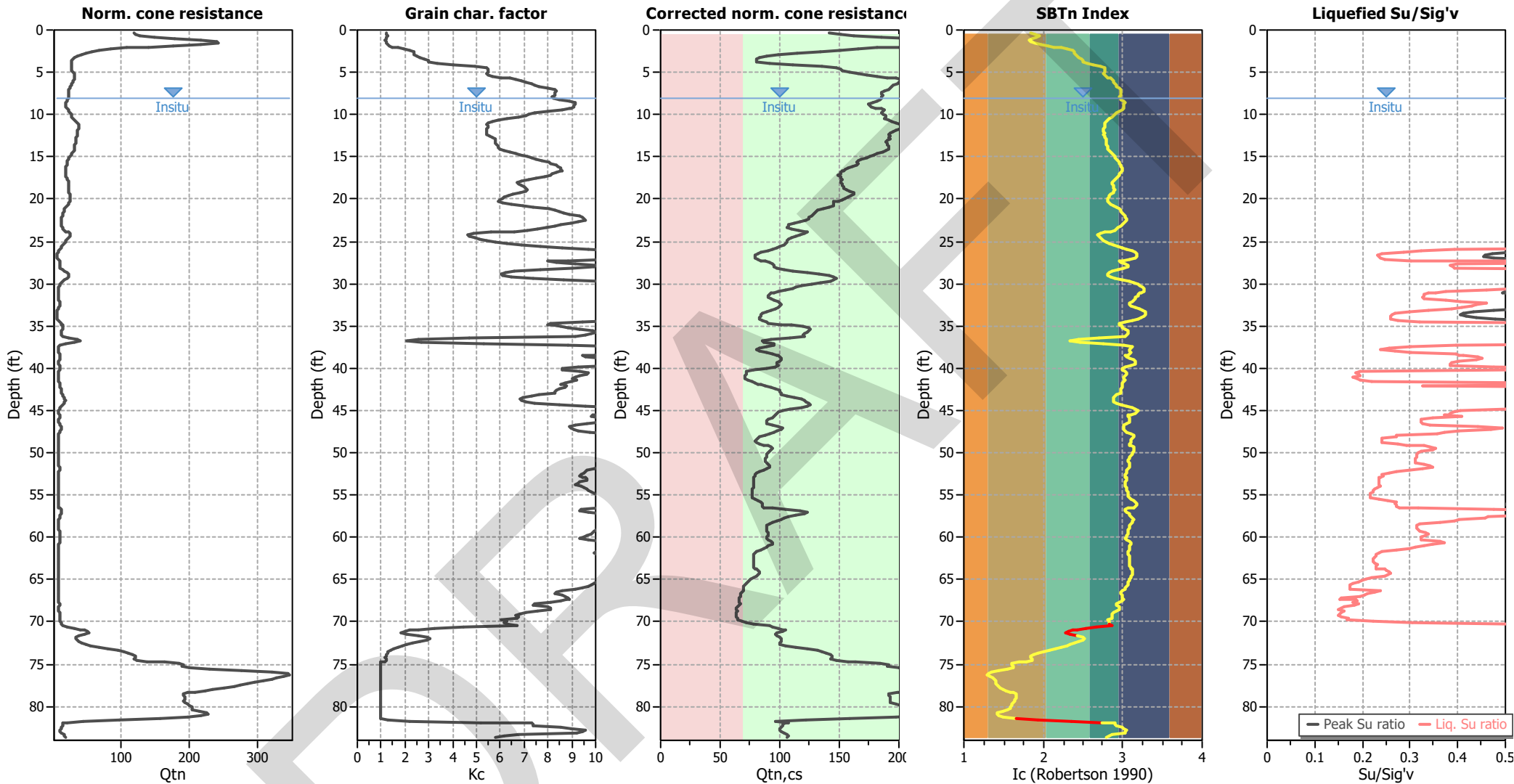
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	IC cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

LIQUEFACTION ANALYSIS REPORT

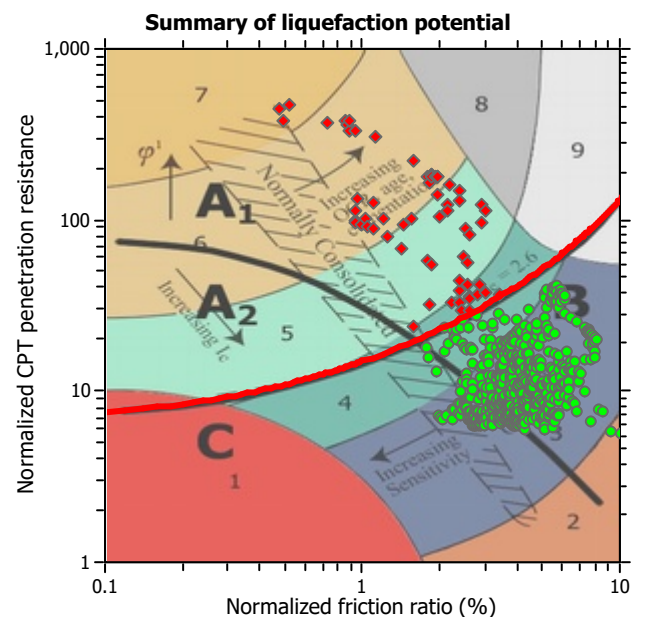
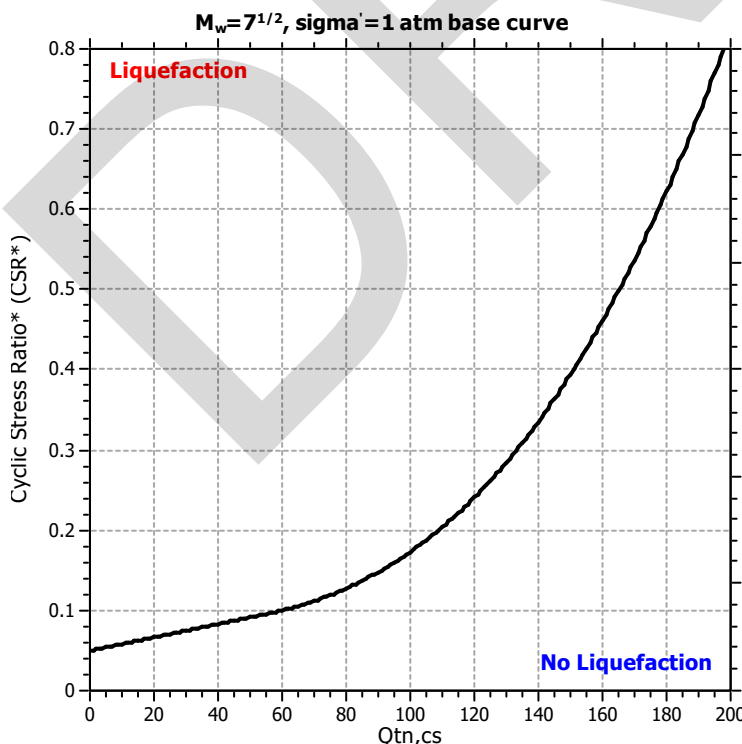
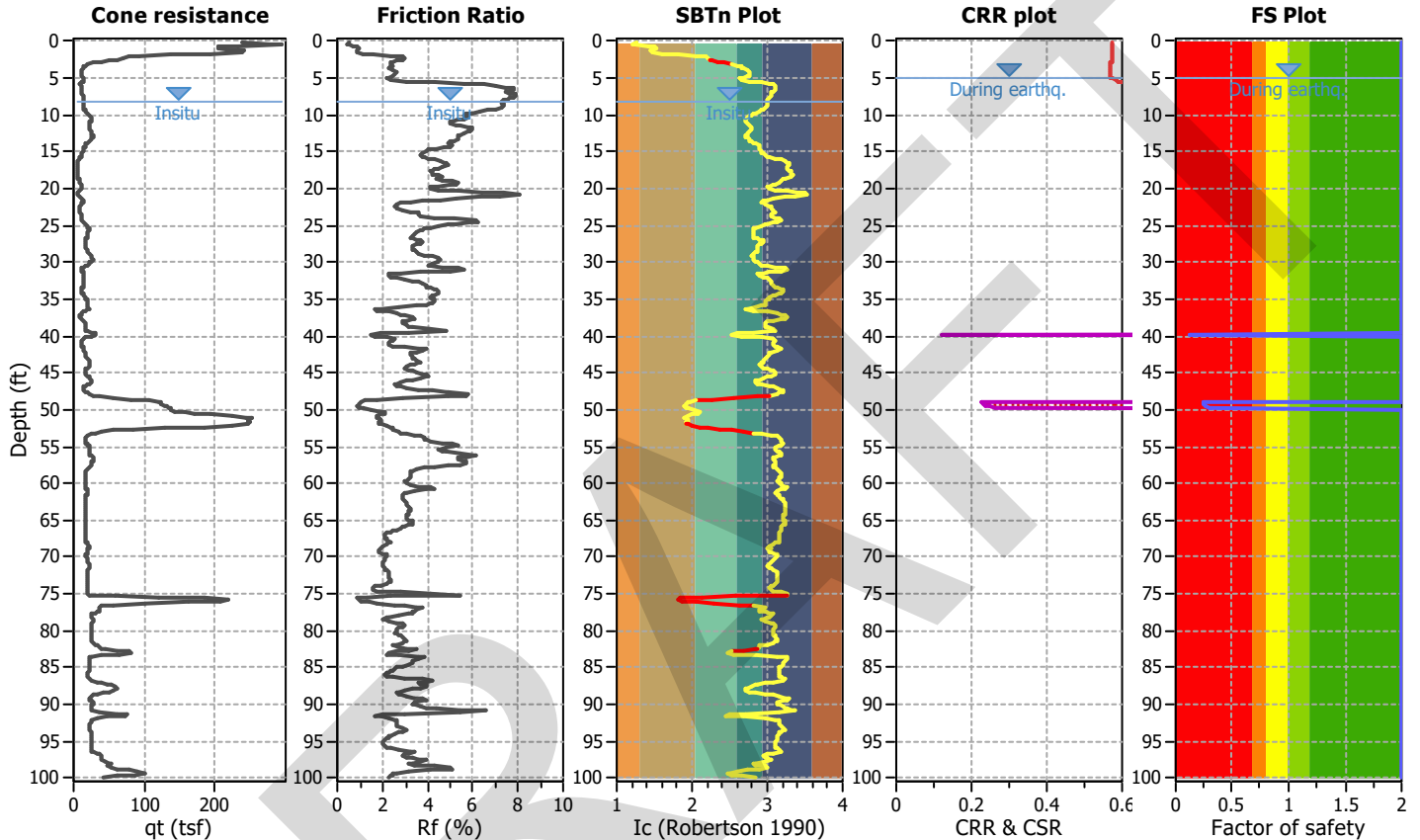
Project title : Calpine Tanager Energy Storage Project

Location : San Jose, CA

CPT file : CPT-04

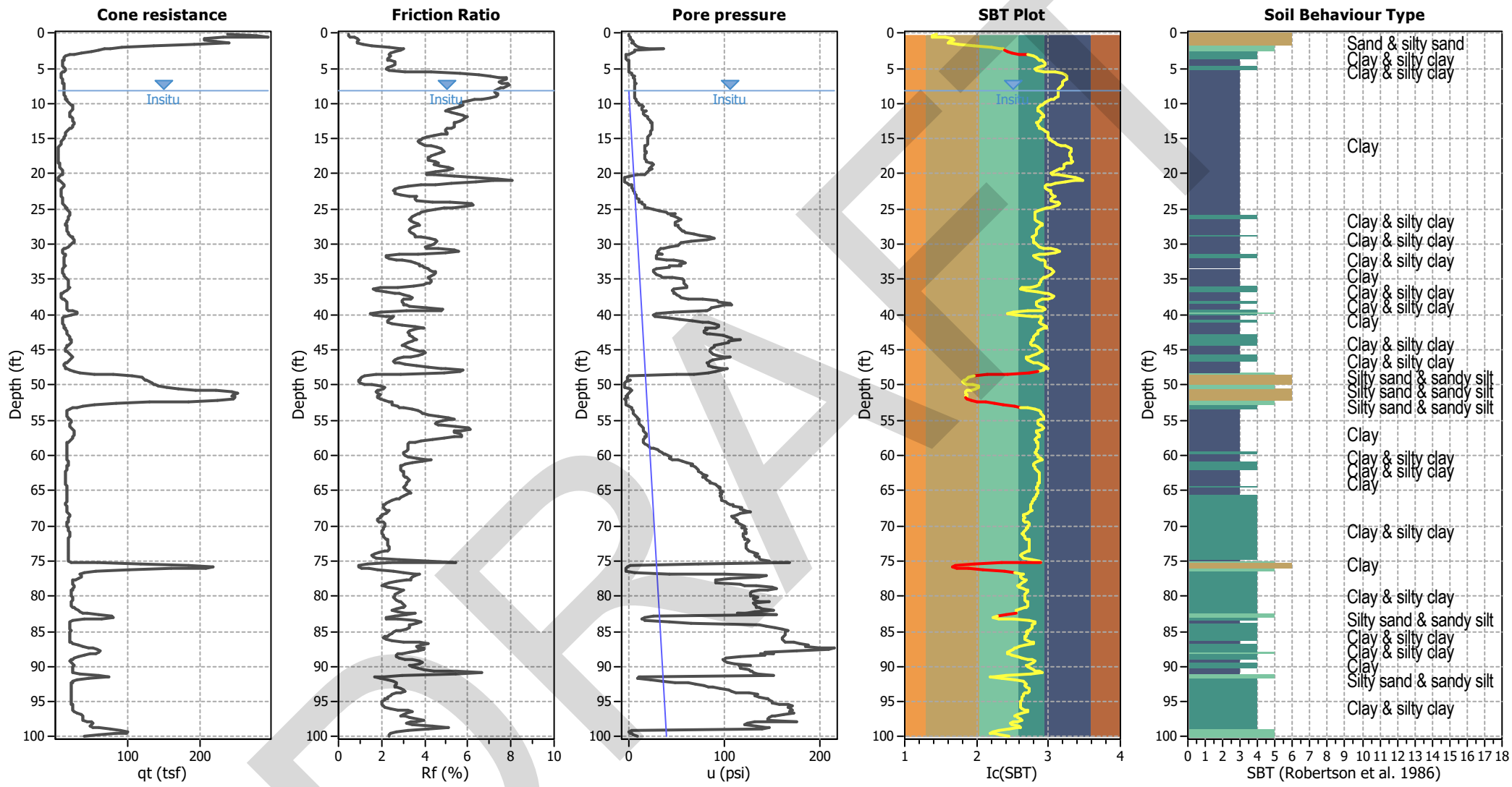
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.20 ft	Use fill:	No	Clay like behavior	
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w :	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	50.00 ft
Peak ground acceleration:	0.81	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



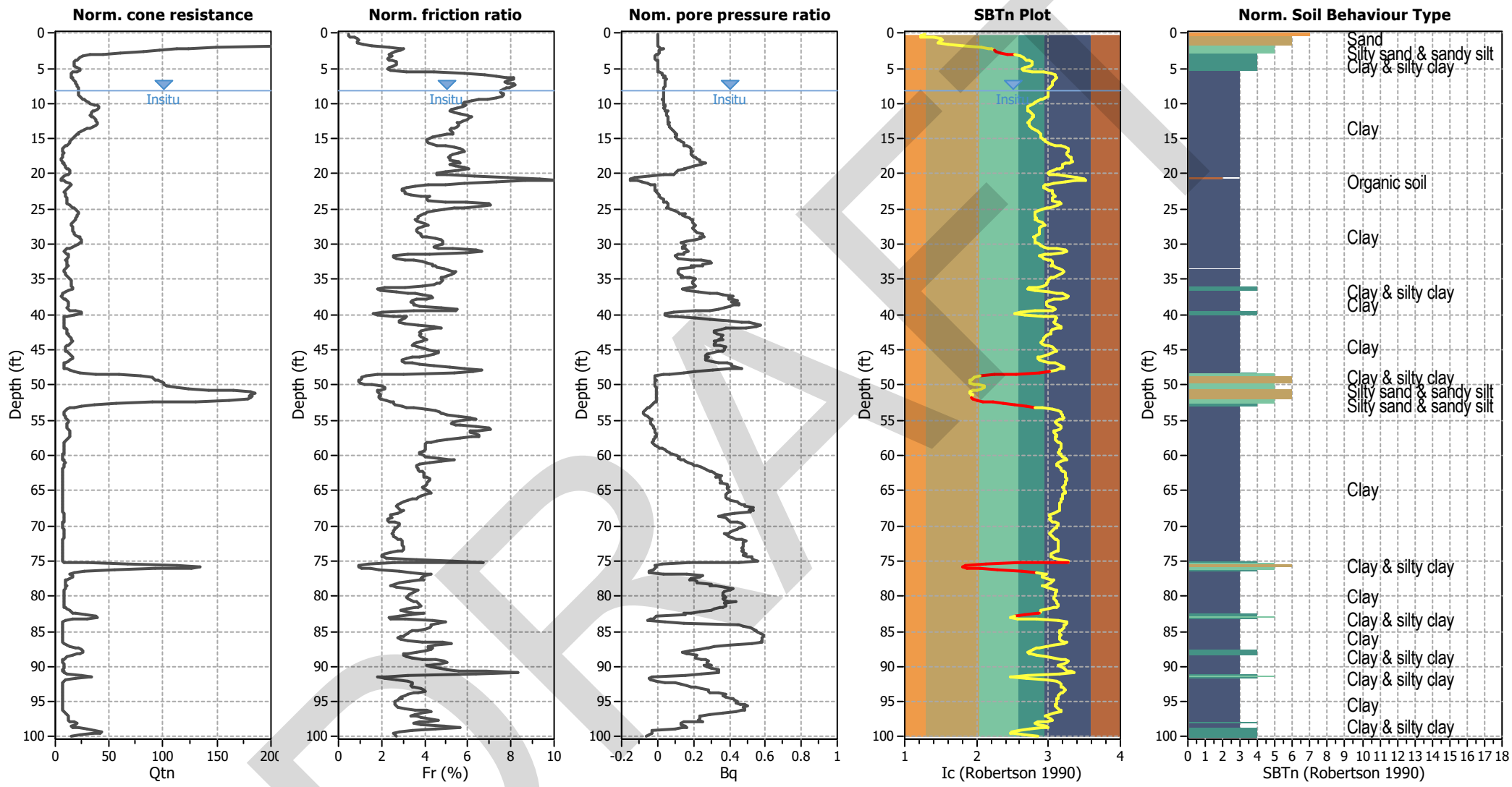
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.20 ft	Fill height:	N/A	Limit depth:	50.00 ft

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)

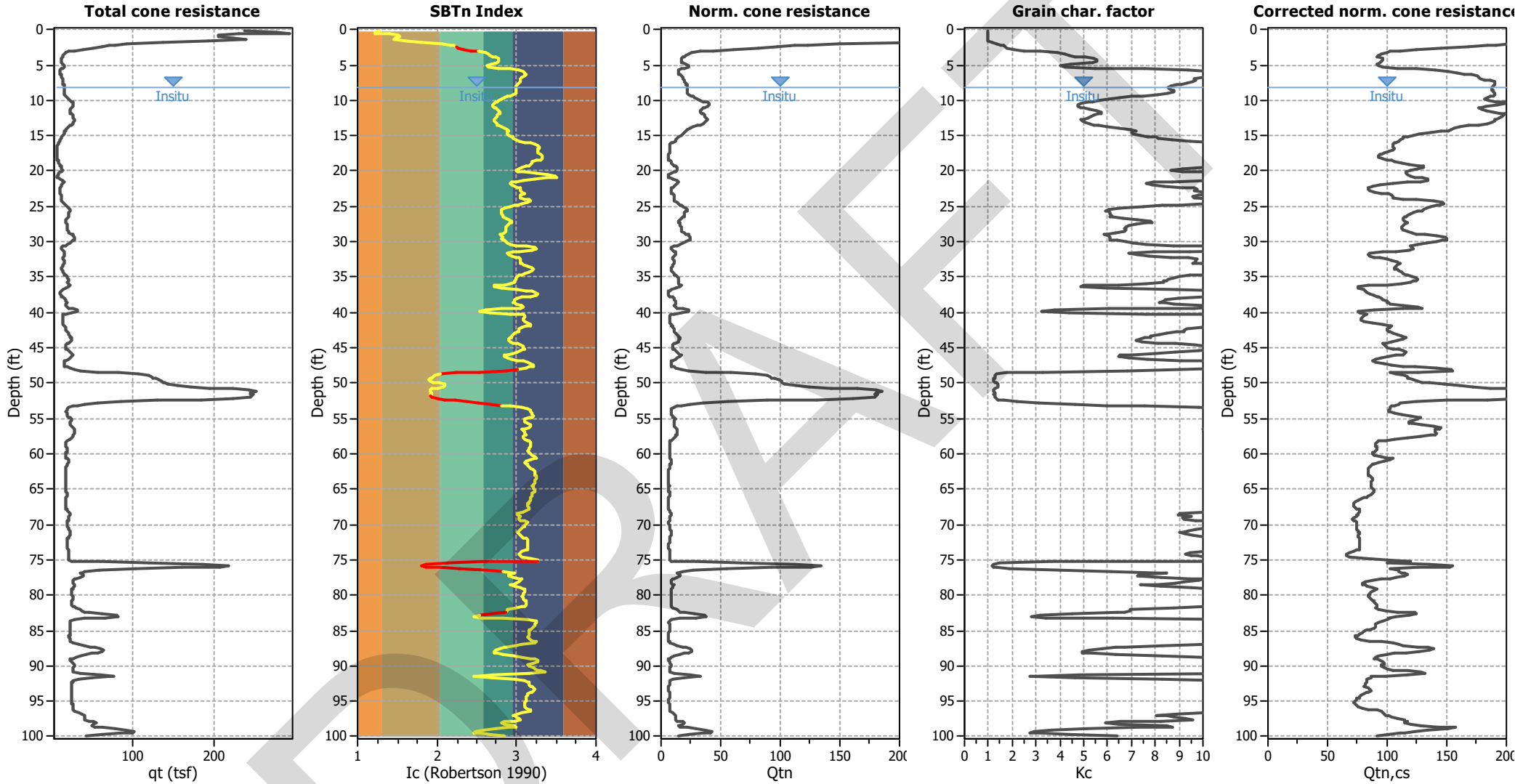


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.20 ft	Fill height:	N/A	Limit depth:	50.00 ft

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

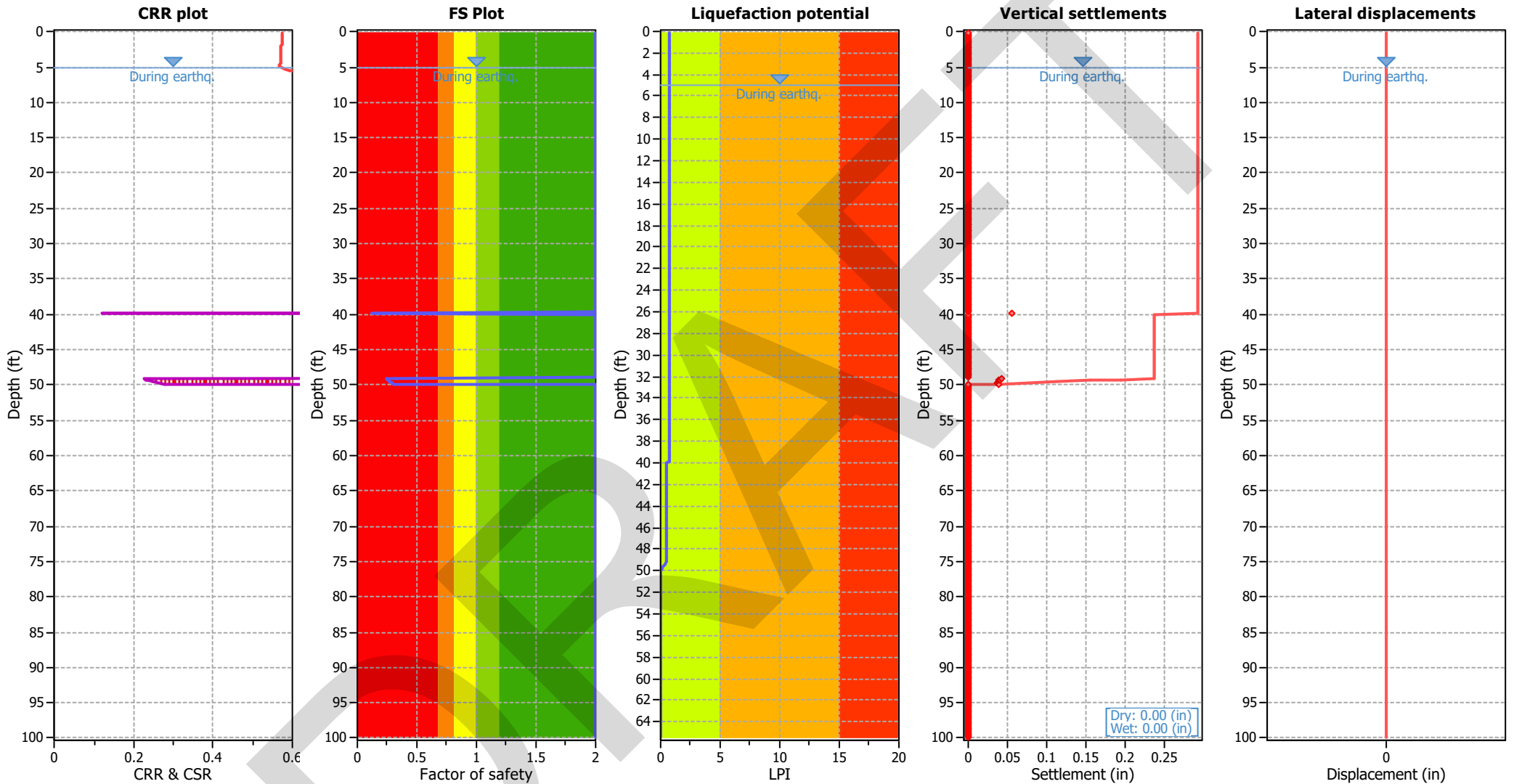
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.20 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (earthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	Yes
Earthquake magnitude M _w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.20 ft	Fill height:	N/A	Limit depth:	50.00 ft

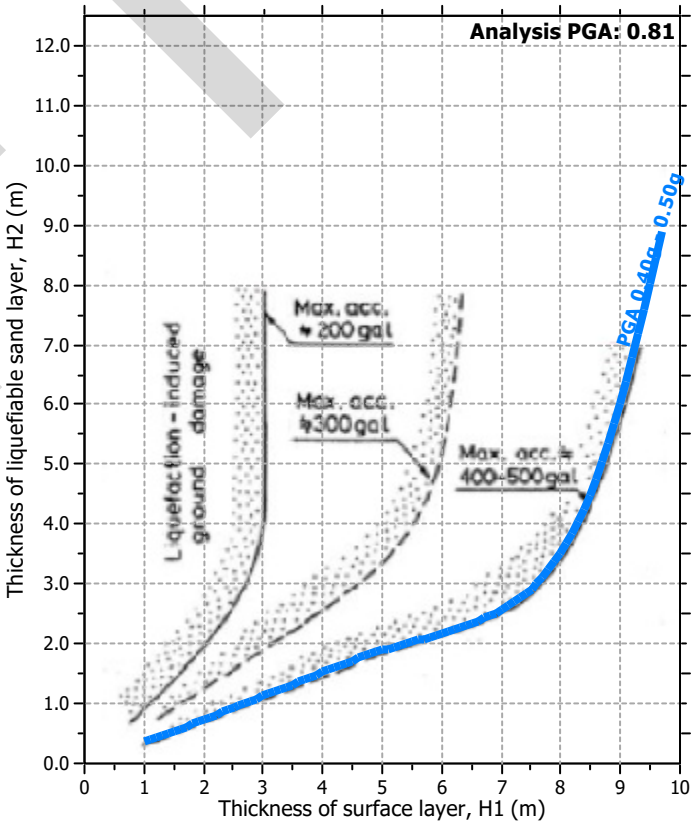
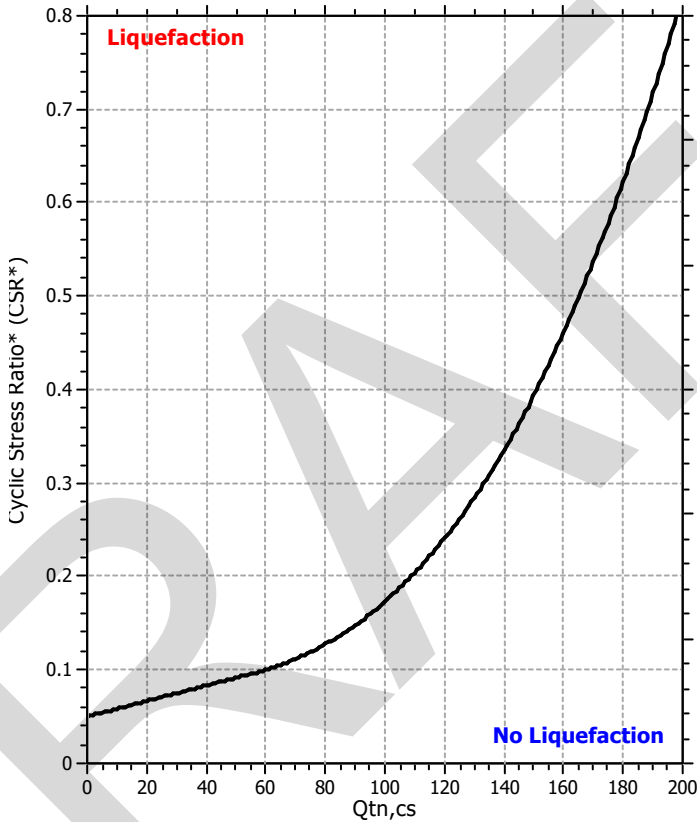
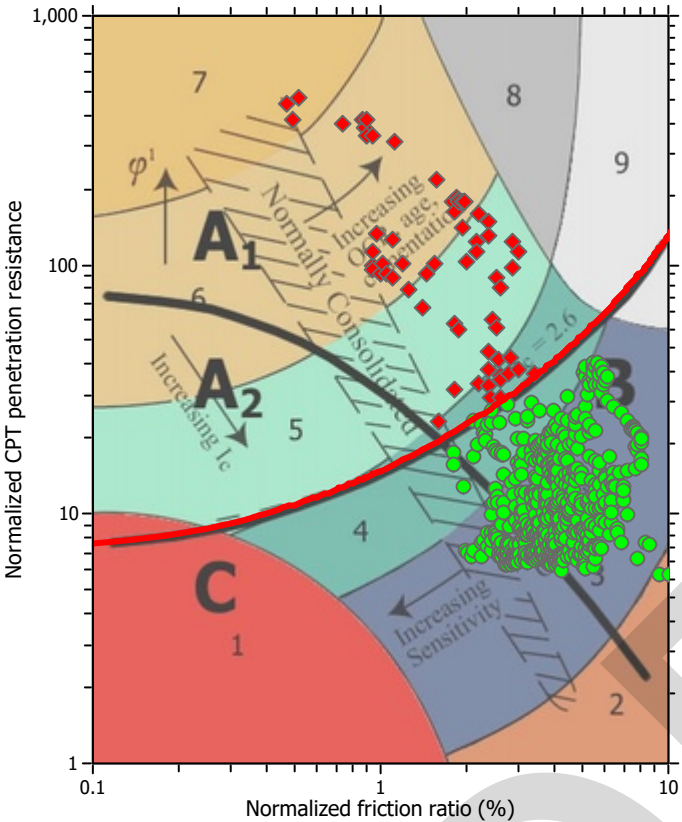
F.S. color scheme

Red	Almost certain it will liquefy
Orange	Very likely to liquefy
Yellow	Liquefaction and no liq. are equally likely
Light Green	Unlike to liquefy
Dark Green	Almost certain it will not liquefy

LPI color scheme

Red	Very high risk
Orange	High risk
Yellow	Low risk

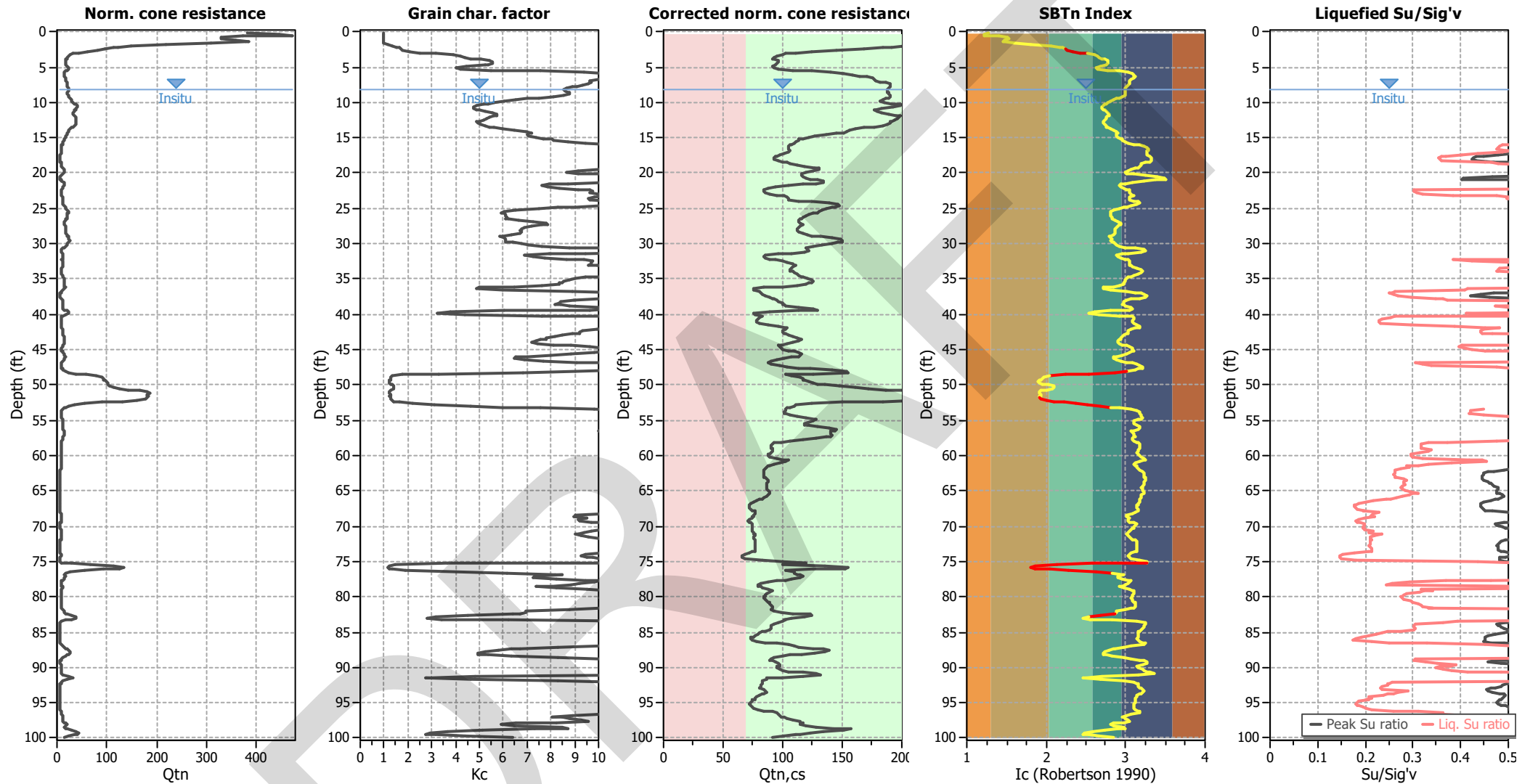
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_0 applied:	Yes
Earthquake magnitude M_w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.20 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w :	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.81	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.20 ft	Fill height:	N/A	Limit depth:	50.00 ft