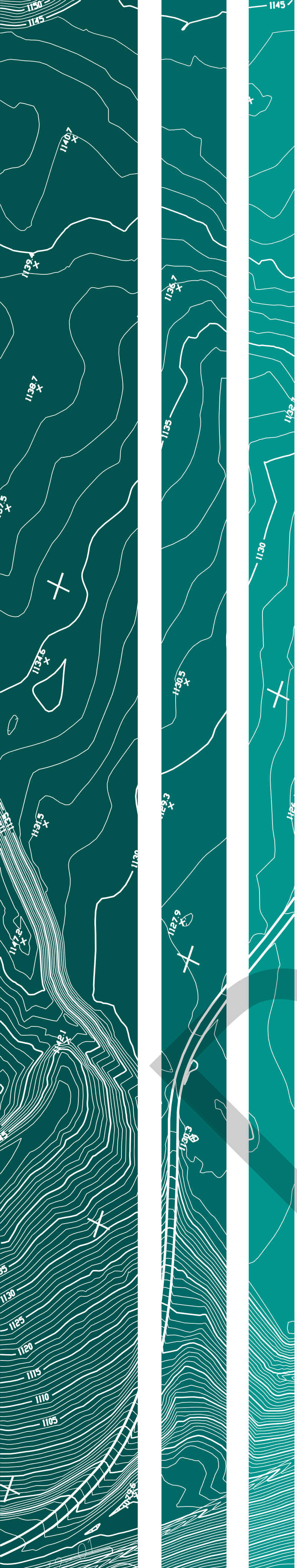

GEOCON

GEOTECHNICAL INVESTIGATION

EUCLID & HEIL SITE
PROPOSED 4-ACRE MULTI-FAMILY RESIDENTIAL DEVELOPMENT
16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN: 144-111-01

MARCH 2025
PROJECT NO. W2045-88-01

PREPARED FOR:
Euclid & Heil FV Owner, LLC
Irvine, California

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. W2045-88-01
March 2025

Mr. Brian Rupp
Euclide & Heil FV Owner, LLC
18565 Jamboree Road, Suite 200
Irvine, CA 92612

Subject: GEOTECHNICAL INVESTIGATION
EUCLID & HEIL SITE
PROPOSED 4-ACRE MULTI-FAMILY RESIDENTIAL DEVELOPMENT
16300 EUCLID STREET, FOUNTAIN VALLEY, CALIFORNIA
APN 144-111-01

Dear Mr. Rupp:

In accordance with your authorization of our proposal dated September 24, 2024, we have performed a geotechnical investigation for the proposed 4-acre multi-family residential development located at 16300 Euclid Street in City of Fountain Valley, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

Disclaimer: This DRAFT is intended for the use of the project team to help with the on-going design of the project and provided as a courtesy for review only. This DRAFT should not be relied upon for final design nor produced and submitted to regulatory agencies until a FINAL document is completed with the signature and stamps of the design professionals.

Petrina Zen	Jelisa Thomas Adams	Gerald A. Kasman
GE 3217	GE 3092	CEG 2251

(EMAIL) Addressee

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed 4-acre multi-family residential development located at 16300 Euclid Street in City of Fountain Valley, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored simultaneously with the adjacent vacant land. The overall site exploration was performed on January 10 and February 21, 2025, by excavating seven 8-inch diameter borings using a truck-mounted, hollow-stem auger drilling machine and two 4-inch diameter borings using a truck-mounted, mud-rotary drilling machine. The borings were drilled to depths between 10½ and 58½ feet below the existing ground surface. Additionally, on February 26, 2025, three 4-inch diameter hand auger borings (borings B7, B10, and B11) were excavated to depths between 6 and 8½ feet below the existing ground surface using manual digging equipment and hand tools. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analyses of the data obtained during our investigation, as well as data from prior investigation report provided for our review, and our experience with similar soil and geologic conditions. The prior report is summarized in Section 3, *Prior Report*. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 16300 Euclid Street in City of Fountain Valley, California. The overall property is approximately 18 acres and is currently unoccupied and undeveloped. The site was formerly used as an agricultural field; a water well associated with the past use is present at the northwest corner of the site. This report pertains to approximately 4 acres of the overall site fronting Euclid Street (see Figure 2). The overall site is bounded by residential structures to the north and east, by Heil Avenue to the south, and by Euclid Street to the west. Local topography is relatively flat with surface water draining by sheet flow to the public streets.

Based on the information provided by the Client, it is our understanding that the subject 4-acre site will be developed with a multi-unit 5-story apartment structure wrapped around a 6.2-story parking garage (see Site Plan, Figure 2). The proposed structures are anticipated to be constructed at or near present site grade.

The remaining 14-acre parcel will be developed separately and is addressed under separate cover.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed apartment structure will be up to 500 kips, and wall loads will be up to 6 kips per linear foot. It is anticipated that the column loads for the proposed parking garage will be up to 975 kips, and the wall loads will be up to 10 kips per linear foot.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. PRIOR REPORT

As a part of the preparation of this report, we reviewed a prior geotechnical report prepared for the project site provided to us by the Client:

> *Due-Diligence Geotechnical Evaluation, Proposed Residential Development, 16300 Euclid Street, Fountain Valley, California,* prepared by Leighton and Associates, Inc., September 30, 2021 (Revised July 2, 2024).

A prior geotechnical investigation of the subject site was performed in 2021 by Leighton and Associates, Inc. (Leighton) and included advancing five Cone Penetrometer Tests (CPTs) to approximate depths of 50 feet below the ground surface. The locations of the CPTs are shown on the Site Plan, Figure 2. Based on review of the CPT interpretations, the soils encountered consist of alternating layers with variable thicknesses of sand, silty sand, sandy silt, silt, silty clay, and clay. Artificial fill was encountered to depths of approximately 2 to 3 feet below ground surface. Groundwater was encountered at depths between 5.9 and 6.4 feet below the ground surface. The raw data files of the CPTs were provided to us by the Client.

Geocon West, Inc. has reviewed the referenced report, and the recommendations presented herein are based on analysis of the subsurface data (i.e. the raw CPT files) obtained from the prior investigation at the site by Leighton, as well as our own subsurface and laboratory data. Furthermore, we assume responsibility for the utilization of the exploration data provided to us by the Client. Geocon West, Inc. is the Geotechnical Consultant of Record and will be providing all necessary geotechnical consultation, plan review, design recommendations, inspection and testing services for this project. Where differing, the recommendations presented herein supersede all previous recommendations. A copy of the prior report is included herein as Appendix C.

4. GEOLOGIC SETTING

The subject site is located on the south-central portion of the Orange County Coastal Plain, a relatively flat-lying alluviated surface with an average slope of less than 20 feet per mile. The lowland surface is bounded by hills and mountains on the north and east, and by the Pacific Ocean to the south and southwest. Prominent structural features within the Orange County Coastal Plain include the central lowland plain, the northwest trending line of low hills and mesas underlain by the Newport-Inglewood Fault Zone along the coast (Newport Mesa, Huntington Beach Mesa, Bolsa Chica Mesa, and Landing Hill), and the San Joaquin Hills to the southeast.

5. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Holocene-age young alluvial fan deposits (CGS, 2003). Detailed stratigraphic profiles are provided on the boring logs in Appendix A and CPT logs in Appendix C.

5.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 4 feet below the existing ground surface, and up to 3 feet below the ground surface within the subject 4-acre site. The fill generally consists of brown sandy silt and silty sand that can be characterized as slightly moist to moist and loose or soft. Construction debris was encountered within the fill and is likely the result of the historic agricultural use of the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

5.2 Young Alluvial Fan Deposits

Holocene-age young alluvial fan deposits was encountered beneath the fill within all borings and to the maximum depth explored of 58½ feet below existing ground surface. The soil consists of brown to reddish and grayish brown silty sand and silt, with lesser amounts of poorly-graded sand and clay. The soils can be characterized as moist to wet and loose to medium dense or soft to firm. These deposits are generally fine- to medium-grained and oxidation stains may be present.

6. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Newport Beach Quadrangle (California Division of Mines and Geology [CDMG], 1997, revised 2001) indicates the historically highest groundwater level in the area is approximately less than 5 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered at depths between 6 and 7 feet below the existing ground surface. The groundwater measurements were performed in a manner that is typical of geotechnical exploration and should not be interpreted as representing a fully equalized water level. Prior explorations at the site encountered groundwater at depths between 5.9 and 6.4 feet below the existing ground surface (Leighton, 2024).

Considering the reported historic high groundwater level in the area (CDMG, 1997, revised 2001), the depth to groundwater encountered in the current and prior explorations, and the depth of the proposed construction, groundwater may be encountered during construction. Also, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 8.20).

7. GEOLOGIC HAZARDS

7.1 Surface Fault Rupture

The numerous faults in Southern California include Holocene-active, pre-Holocene, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018). By definition, a Holocene-active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A pre-Holocene fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not located within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2025a; 2025b; 2017) for surface fault rupture hazards. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Newport-Inglewood Fault Zone located approximately 6.2 miles to the southwest. Other nearby active faults include the Palos Verdes Fault Zone, the Whittier Fault, and the Elsinore Fault Zone located approximately 15 miles southwest, 15 miles northeast, and 19 miles northeast of the site, respectively (USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 44 miles northeast of the site (USGS, 2006).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987, M_w 5.9 Whittier Narrows earthquake and the January 17, 1994, M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

7.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the table below.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Long Beach	March 10, 1933	6.4	8	SSW
Tehachapi	July 21, 1952	7.5	107	NW
San Fernando	February 9, 1971	6.6	54	NW
Whittier Narrows	October 1, 1987	5.9	25	NNW
Sierra Madre	June 28, 1991	5.8	37	N
Landers	June 28, 1992	7.3	92	ENE
Big Bear	June 28, 1992	6.4	71	ENE
Northridge	January 17, 1994	6.7	48	NW
Hector Mine	October 16, 1999	7.1	113	ENE
Ridgecrest China Lake Fault	July 5, 2019	7.1	142	N

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be minimized if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

7.3 Seismic Design Criteria

The following table summarizes the site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application U.S. Seismic Design Maps, provided by the Structural Engineers Association of California (SEAOC). The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented on the following page are for the risk-targeted maximum considered earthquake (MCE_R).

Although there are liquefiable soils underlying the site, we assume that the proposed structures will have fundamental periods of less than 0.5 seconds and therefore will not require a site-response analysis. If proposed structures will have fundamental periods greater than 0.5 seconds, a site-response analysis may need to be performed to further evaluate the seismic response of the site and generate seismic design parameters.

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.331g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.477g	Figure 1613.2.1(3)
Site Coefficient, F_A	1.0	Table 1613.2.3(1)
Site Coefficient, F_V	1.823	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.331g	Section 1613.2.3 (Eqn 16-20)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.869g*	Section 1613.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.887g	Section 1613.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.579g*	Section 1613.2.4 (Eqn 16-23)

*Per Supplement 3 of ASCE 7-16, a ground motion hazard analysis (GMHA) shall be performed for projects on Site Class “D” sites with 1-second spectral acceleration (S_1) greater than or equal to 0.2g, which is true for this site. However, Supplement 3 of ASCE 7-16 provides an exception stating that that the GMHA may be waived provided that the parameter S_{M1} is increased by 50% for all applications of S_{M1}. The values for parameters S_{M1} and S_{D1} presented above have ***not*** been increased in accordance with Supplement 3 of ASCE 7-16.

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.571g	Figure 22-9
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.628g	Section 11.8.3 (Eqn 11.8-1)

Deaggregation of the Maximum Considered Earthquake (MCE) peak ground acceleration was performed using the USGS online Earthquake Hazard Toolbox, NSHM Conterminous U.S. 2018 edition. The result of the deaggregation analysis indicates that the modal earthquake contributing to the MCE peak ground acceleration is characterized as a 7.3 magnitude event occurring at a hypocentral distance of 10.03 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, corresponding to two-thirds of the MCE peak ground acceleration. The result of the analysis indicates that the modal earthquake contributing to the DE peak ground acceleration is characterized as a 6.1 magnitude occurring at a hypocentral distance of 11.99 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

7.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine- to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

Both the State of California Seismic Hazards Program (CGS, 2025b) and the City of Fountain Valley Safety Element (Fountain Valley, 1995) indicate that the site is located in an area designated as having a potential for liquefaction. As previously indicated, the historic high ground level in the site vicinity is reported to be less than 5 feet beneath the existing ground surface (CDMG, 1998). Therefore, we recommend that a groundwater level of 4 feet below the ground surface be assumed for design and construction.

Prior to evaluation of the liquefaction potential, the Standard Penetration Test (SPT) blow counts from boring B6 were compared with the blow counts estimated from CPT 3 based on the proximity of these explorations. SPTs were performed in the boring at intervals of approximately 5 feet. In order to supplement the SPT blow count data, the California Modified Sampler blow count data were converted to equivalent SPT blow counts based on a correlation factor of 0.55 (Rogers, 2006). The field collected blow counts were corrected for hammer efficiency to N60 blow count values. The boring N60 values were compared with the N60 values generated by the program CPet-IT (Version 3.7.1.5). The comparison is shown as Figure 5. It is our opinion that the boring and CPT N60 values show a reasonable correlation and that analysis of the liquefaction potential may be based on the CPT data.

Liquefaction analyses of the CPT soundings were performed using the program CLiq (Version 3.5.2.7). This program utilizes the 2001 NCEER method of analysis. This semi-empirical method is based on correlations with the data collected from the CPT soundings.

The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of 4 feet below the ground surface, a magnitude 6.61 earthquake, and a peak horizontal acceleration of 0.42g (⅔PGA_M). The enclosed liquefaction analyses, included herein for CPTs 1 through 5, indicate that the alluvial soils below the historic high groundwater could be susceptible to up to 2.4 inches of liquefaction induced settlement during Design Earthquake ground motion. A summary of the anticipated liquefaction induced settlements is provided as Figure 6; calculations and output from CLiq are provided as Appendix D.

It is our understanding that the intent of the Building Code is to maintain “Life Safety” during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 4 feet below the ground surface, a magnitude 7.3 earthquake, and a peak horizontal acceleration of 0.63g (PGA_M). The enclosed liquefaction analyses, included herein for CPTs 1 through 5, indicate that the alluvial soils below the historic high groundwater could be susceptible to up to 3.1 inches of liquefaction induced settlement during Maximum Considered Earthquake ground motion. A summary of the anticipated liquefaction induced settlements is provided as Figure 7; calculations and output from CLiq are provided as Appendix D.

7.5 Slope Stability

The site is relatively flat and not within an area identified as having potential for seismic slope instability (CDMG, 1998; CGS, 2025b). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

7.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Orange County Safety Element (2004) indicates that the site is located within the Prado Dam and Santiago Reservoir inundation area. However, these reservoirs, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

7.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up-gradient from the project site. Therefore, flooding resulting from a seismically induced seiche is considered unlikely.

The site is located within Flood Zone X, designated as an area of minimal flooding (FEMA, 2024).

7.8 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and oil or gas wells are not documented in the immediate site vicinity (CalGEM, 2023). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the CalGEM.

As previously indicated, the site is not located within an oilfield. Therefore, the potential for methane at the site is considered low. Should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

7.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Subsidence commonly occurs in such small magnitudes and over such large areas that is it generally imperceptible at an individual locality. Accordingly, it affects only regionally extensive structures sensitive to slight elevation changes, such as canals and pipelines. The rate of elevation change is usually uniform over a large enough area that it does not result in differential settlements that would cause damage to individual buildings. Soils that are particularly subject to subsidence include those with high silt or clay content.

The USGS Areas of Land Subsidence in California, online mapping system (USGS, 2025), indicates that the site is located in an area of known subsidence due to groundwater pumping. Current groundwater basin management practices within southern California account for potential subsidence from groundwater and/or petroleum extraction and take steps to mitigate potential affects. Local agencies maintain groundwater levels by recharging percolation basins that filter water back to the groundwater aquifers in the subsurface alluvial deposits. As long as the water district continues this practice, the potential for ground subsidence at the site is considered low.

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

8.1.2 Up to 3 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 8.5).

8.1.3 Organic testing of the upper site soils indicates soils within the upper 5 feet have organic contents ranging from 0.5 to 2.8 percent (see Figure B41). Based on our experience, the generally acceptable organic content blended into soils used as engineered fill is approximately 5 percent. Therefore, the existing site soils are considered suitable for re-use as engineered fill.

8.1.4 The enclosed liquefaction analyses indicate that the site soils could be susceptible to up to approximately 2.4 inches of total settlement as a result of the Design Earthquake peak ground acceleration (⅔PGA_M). The resulting differential settlement is anticipated to be less than 1.2 inches over a distance of 50 feet. The grading and foundation design recommendations presented herein are intended to reduce the effects of settlement on proposed improvements.

8.1.5 Based on the results of our liquefaction analyses, as well as the results of the laboratory testing, the existing alluvial soils could yield excessive total and differential settlements if subject to liquefaction and/or upon application of foundation loads.

8.1.6 Based on these considerations, it is recommended that soil modification (e.g. stone columns) be performed below the proposed apartment and parking structures. Recommendations for *Stone Columns* are provided in Section 8.7.

8.1.7 Where supported on improved ground and due to the presence of existing artificial fill, it is recommended that the upper 4 feet of existing soils within the footprint area of the proposed structures be excavated and properly compacted for lateral support of foundations and for slab support. The engineered fill blanket should extend at least 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 8.5).

8.1.8 Subsequent to the recommended grading and ground improvement, the proposed structures may be supported on a conventional foundation system deriving support from the improved soils. Recommendations for a conventional foundation system are provided in Section 8.8 of this report.

8.1.9 It should be noted that implementation of the recommendations presented herein is not intended to completely prevent damage to the structure during the occurrence of strong ground shaking as a result of nearby earthquakes. It is intended that the structure be designed in such a way that the amount of damage incurred as a result of strong ground shaking be reduced. Re-leveling of the foundation system and repair of utilities could be necessary following a strong seismic event that triggers liquefaction.

8.1.10 Improvements which are not supported on soil modification, such as walkways, paving, and utilities, may still be subject to seismic and/or static settlement. The client should consider the flexibility of the products and pavements being installed. It is recommended that all utilities traversing through existing site soils utilize flexible connections in order to reduce the damage to underground installations caused by potential soil movements.

8.1.11 Groundwater was encountered at depths between 5.9 and 7 feet below existing ground surface. Based on these considerations, groundwater should be anticipated during construction and the contractor should be prepared for saturated soil and soft/pumping excavation bottoms during grading and earthwork operations. If groundwater is encountered in excavations, temporary dewatering measures will be required to maintain a safe working environment. Recommendations for temporary dewatering are discussed in Section 8.4 of this report.

8.1.12 The alluvial soils anticipated to be exposed at the excavation bottom may be very moist and could be subject to excessive pumping. Operation of rubber tire equipment on the subgrade soils may cause excessive disturbance of the soils. Excavation activities to establish the finished subgrade elevation must be conducted carefully and methodically to avoid excessive disturbance to the subgrade. Stabilization of the excavation bottom will likely be required in order to provide a firm working surface upon which heavy equipment can operate. Recommendations for bottom stabilization and earthwork are provided in the *Grading* section of this report (see Section 8.5).

8.1.13 Based on the relatively shallow groundwater table, the upper alluvial soils have the potential to be very moist and the grading contractor should be aware that the soils may be above optimum moisture content. If the soils are more than 3 percent above the optimum moisture content at the time of construction, the soils will likely require some spreading and drying activities in order to achieve proper compaction.

8.1.14 As an alternative to drying the existing site soils, the soils may be excavated and blended with a dry cement mix or lime, and then placed as properly compacted engineered fill. The dry cement or lime will react with the soils significantly reducing the moisture. Lime and cement treatment will require the retention of an experienced specialty contractor. Recommendations for soil stabilization through the use of lime or cement are provided in the *Grading* section of this report (see Section 8.5).

8.1.15 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 8.18).

8.1.16 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

8.1.17 Where new paving is to be placed, it is recommended that all existing fill soils and soft soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 8.12).

8.1.18 It is recommended that flexible utility connections be considered for all rigid utilities to minimize or prevent damage to utilities from minor differential movements.

8.1.19 Based on the presence of shallow groundwater, infiltration of stormwater into the ground is not feasible and is not recommended for this development. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

8.1.20 Once the design and foundation loading configuration for the proposed structures proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be reevaluated by this office.

8.1.21 Any changes in the design, location or elevation of improvements, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

8.2 Excavation and Soil Characteristics

8.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Caving should be anticipated in unshored excavations, especially where granular and saturated soils are encountered.

8.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of adjacent existing improvements.

8.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 8.18).

8.2.4 The upper 5 feet of existing site soils encountered during this investigation are considered to have a “very low” expansive potential (EI = 0) and are classified as “non-expansive” in accordance with the 2022 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that proposed foundations and slabs will derive support in these materials.

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “moderately corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B40) and should be considered for design of underground structures.

8.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B40) and indicate that the on-site materials possess a sulfate exposure class of “S0” to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-19 Chapter 19.

8.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. A Soil Corrosivity Evaluation Report has been prepared by Project X Corrosion Engineering and is provided as Appendix E. The results presented in the Soil Corrosivity Evaluation Report should be considered for design of underground structures.

8.4 Groundwater and Temporary Dewatering

8.4.1 Based on groundwater depths encountered during site exploration, groundwater should be anticipated during excavation and installation of deeper structures, such as subsurface utilities. The depth to groundwater can be further verified at the time of construction. If groundwater is present above the excavation bottom, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities. Dewatering system design, operation, and proper discharge is the responsibility of the contractor.

8.4.2 Depending on the depth of excavation and groundwater flow levels, it may be feasible to accomplish temporary dewatering by placing a minimum one-foot thick layer of angular gravel along the trench bottom. The gravel layer will function as both a permeable material for dewatering procedures as well as a stable working surface. A dewatering pump can be placed within the gravel layer to remove water from the excavation.

8.4.3 It is recommended that a qualified dewatering consultant be retained to design the dewatering system. Recommendations for design flow rates for the temporary dewatering system should be provided by a qualified contractor or dewatering consultant. The dewatering consultant should also provide the minimum depth that the temporary dewatering will be effective as well as the potential effects of temporary dewatering on adjacent structures and the public right of way. Additional analyses may be required in the future to evaluate the settlement based on the proposed dewatering system and associated drawdown curves.

8.4.4 Structures that are planned below the groundwater elevation should be designed for groundwater uplift loading conditions. The uplift pressure is equal to 62.4H pounds per square foot (psf) where H is the depth below the design groundwater elevation in feet. Based on the depth of water encountered during current and prior site investigation and the reported historic high groundwater, it is recommended that a groundwater depth of 4 feet be used for design.

8.5 Grading

8.5.1 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and, if applicable, building official in attendance. Special soil handling requirements can be discussed at that time.

8.5.2 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soils encountered during exploration is suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches), any encountered deleterious debris is removed, and that site soils are blended so that the engineered fill does not exceed 2 percent organics content. Soils that exhibit high organics content may be removed from the site or stockpiled and utilized as topsoil in proposed landscaping areas, pending the landscape architect’s approval.

8.5.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

8.5.4 Where supported on improved ground (e.g. stone columns) and due to the presence of existing artificial fill, it is recommended that the upper 3 feet of existing soils within the footprint area of the proposed structures be excavated and properly compacted for lateral support of foundations and for slab support. Excavations should be conducted as necessary to remove any encountered artificial fill at the direction of the Geotechnical Engineer (a representative of Geocon). The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. The engineered fill blanket should extend at least 3 feet beyond the edge of foundations or for a distance equal to the depth of fill below the foundations, whichever is greater.

8.5.5 The proposed swimming pool should be designed as a free-standing structure deriving support in newly placed engineered fill and/or the undisturbed alluvial soils found at and below a depth of 3 feet. Based on the depth of groundwater and potential for liquefaction induced settlement, it is strongly suggested that ground improvement be performed below the pool shell/foundation as the equipment and materials will already be mobilized to the site. Recommendations for Ground Improvement are provided in Section 8.7 and recommendations for swimming pool design are provided in Section 8.15.

8.5.6 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

8.5.7 Based on the relatively shallow groundwater table, the upper alluvial soils are currently very moist and have the potential to be very moist at the time of construction. If the soil is more than 3 percent above the optimum moisture content at the time of construction the soils will require spreading and drying efforts to reduce the moisture content in order to achieve proper compaction. Conditions could change seasonally and could worsen if the soil is subjected to heavy precipitation.

8.5.8 If determined to be excessively soft , stabilization of excavation bottoms may be required in order to provide a firm working surface upon which engineered fill can be placed. Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. Track mounted equipment should be considered to minimize disturbance to the soils. It is suggested that excavation and grading be performed during the summer season to promote moisture control of the soils.

8.5.9 One method of subgrade stabilization would consist of introducing a thin lift of 3- to 6-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils.

8.5.10 As an alternative to drying the existing site soils, existing soils may be excavated and blended with a dry cement mix and then placed as properly compacted engineered fill. The dry cement will react with the soils reducing moisture and adding strength. This will require an experienced specialty contractor proficient with cement stabilization.

8.5.11 The soil cement process should consist of the following:

1) Blend dry cement (estimated 4 percent by dry weight) into the existing site soils and properly compact. The blending should extend laterally a minimum distance of 3 feet beyond building footprint areas. The cement placement and blending will require an experienced specialty contractor proficient with cement stabilization. The dry cement will react with the soils reducing moisture and adding strength. Once compaction of the cement stabilized soil has been achieved, the imported fill soils may be placed and compacted to raise grade and achieve finished pad elevation.

2) Prior to the cement treatment, representative samples of the soils should be mixed with various percentages of cement to evaluate the optimal mix proportions. Based on typical cement treatment applications, we anticipate that an application rate of 2 to 5 percent cement (by dry weight of soil) will be needed to provide the intended result. Geocon should conduct trial mixes in our laboratory to determine the actual percent cement required prior to the cement treatment.

3) The soil cement mixture should be performed in accordance with Section 301-3.1 of the Standard Specifications for Public Works Construction (Greenbook). No earthmoving equipment other than mixing equipment should be allowed to pass over the spread cement until the mixing operation is complete. The soil and cement should be mixed a minimum of two times with the approved mixing machine. Water should be added to the subgrade during mixing as necessary to provide a moisture content not less than 1 percent below or 2 percent above the optimum soil-cement mixture to ensure chemical action of the cement and soil.

4) The contractor should regulate the sequencing of the cement treatment such that the final compaction of the soil-cement mixture to the specified density shall begin within ½ hour and be completed within 2½ hours after the initial application of water during the mixing operation. The soil-cement mixture in the equipment pad areas should be compacted to at least 90 percent of the soil-cement maximum dry density per ASTM D-1557. The finished soil-cement should be firm and unyielding and should be given sufficient time to cure before placement of reinforcing or concrete. The soil cement mixture should be protected against drying in accordance with Section 301-3.2.10 of the Greenbook.

5) If soil cement stabilization is performed below hardscape, paving, or pool areas, secondary rolling should be performed after curing and before placing pavements to microcrack the soil cement treated mixture. The microcracking should consist of two to four passes of a heavy steel-wheel vibratory roller. Geocon should be onsite to observe the microcracking.

8.5.12 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near optimum moisture content, and properly compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).

8.5.13 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 8.18).

8.5.14. Where new paving and hardscape is to be placed, it is recommended that all existing fill and soft soils be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 8.12).

8.5.15 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

8.5.16 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Import fill should consist of the characteristics presented in the table below. Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Very Low" (Expansion Index of 20 or less)
Particle Size	Maximum Dimension Less Than 6 Inches Free of Debris
Corrosivity	Equal or Less Detrimental Than Existing Onsite Soils

8.5.17 It is recommended that flexible utility connections be utilized for all rigid utilities to reduce or prevent damage to utilities from minor differential movements. Utility trenches should be properly backfilled in accordance with the following requirements. The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable. Prior to placing any bedding material or pipes, the trench excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

8.5.18 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding sands, fill, steel, gravel, or concrete.

8.6 Shrinkage

8.6.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor between 10 and 20 percent should be anticipated when excavating and compacting the upper 5 feet of existing earth materials on the site to an average relative compaction of 92 percent.

8.6.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). In order to maintain uniformity in the building pad, soils can be borrowed from non-building pad areas and later replaced with imported soils.

8.7 Stone Columns

8.7.1 It is recommended that soil improvement consisting of stone columns be performed below the proposed apartment and parking structures. This type of ground improvement system has multiple trade names and is designed and installed by a specialty contractor. Subsequent to construction of the stone column system, the proposed structures may be supported on a conventional foundation system deriving support in the improved soils. The foundation system should be designed to derive vertical support from the improved soils and may develop lateral resistance at the foundation perimeter, as well as by friction beneath the foundations, if necessary.

8.7.2 The stone columns system is based on soil improvement that consists of installing densified, aggregate columns to depths typically ranging up to 25 feet below the proposed foundations. The system increases density and lateral stress in the surrounding soil and claims improvement in bearing capacity and settlement potential (potential settlement). Stone columns elements are constructed by creating shafts (commonly 30 inches in diameter) by drilling or displacement methods, and backfilling the open shaft with specially rammed/compacted, open graded crushed rock and Class 2 AB in 10- to 12-inch lifts. It should be noted that creating the shaft using the displacement method, advancing the shaft with a displacement mandrel, reduces the soil cuttings generated during the creation of the shaft.

8.7.3 The pattern and depth of ground improvements may vary depending upon the purposes of mitigation and stratigraphic conditions. The ground improvement should be designed to incorporate allowable static and seismic settlements in accordance with the recommendations of the project structural engineer. The ground improvement designer is also responsible for evaluating the post-installation static and dynamic settlement within the ground improvement zone and shall provide this information to the project structural engineer to confirm if the planned structure can tolerate the planned settlements after the installation of the ground improvement.

8.7.4 Spacing and diameter should be selected by the specialty contractor to obtain level of improvement as outlined herein. The ground improvement should extend at least 10 feet laterally outside the edge of planned building structures, where practical.

8.7.5 The ground improvement design should be based on settlement criteria of a maximum combined static and seismic differential settlement of 1 inch between adjacent columns, or as specified by the project structural engineer. The anticipated seismically induced differential settlement should be evaluated as a part of the ground improvement design, as the ground improvement may reduce the potential for liquefaction within the improvement zone.

8.7.6 The ground improvement design package should be submitted to Geocon West, Inc. for review at least two weeks prior to mobilization for construction. Within the design package, the specialty contractor should outline a performance and load testing program to verify the effectiveness of the ground improvement and to confirm the bearing capacity of the improved soils with a full-scale load test. During the load testing, a representative of Geocon should be present to observe the ground improvement installation and testing. The information obtained from the load testing should be used to modify the depth necessary to achieve design capacities, as well as develop installation criteria that can be used during construction.

8.7.7 Geocon should be present continuously during installation of the stone column ground improvements. Geocon's QA/QC observations and documentation will include pier ID, location, depth, diameter, number of lifts, type of aggregate placed, lift thickness, and any changed conditions.

8.8 Foundation Design – Subsequent to Ground Improvement (Stone Columns)

8.8.1 The proposed structure may be supported on a conventional shallow spread foundation system deriving support on the improved soils. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete

8.8.2 Continuous footings should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material. Isolated spread foundations should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

8.8.3 Foundations constructed over ground improvement can achieve relatively high bearing pressures. For preliminary design purposes, a bearing pressure of 5,000 psf may be assumed. Higher bearing capacities with ground improvement are achievable and the design bearing pressure should be provided by the ground improvement contractor.

8.8.4 It is recommended that either a seismic separation or flexible connection be utilized where the apartment structures and parking structure may be attached. The design of the connection is at the discretion of the project structural engineer and should take into account potential differential settlements between structures.

8.8.5 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

8.8.6 If depth increases are utilized for the exterior wall footings, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.8.7 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

8.8.8 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only and are not intended to be used in lieu of those required for structural purposes.

8.8.9 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

8.8.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.8.11 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.8.12 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

8.9 Miscellaneous Foundations

8.9.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

8.9.2 If the soils exposed in the excavation bottom are loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.9.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

8.10 Lateral Design

8.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the undisturbed alluvial soils and newly placed engineered fill. A higher coefficient of friction may be feasible within improved soils and should be provided by the ground improvement designer.

8.10.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill or undisturbed alluvial soils above the groundwater table may be computed as an equivalent fluid having a density of 240 pounds per cubic foot (pcf) with a maximum earth pressure of 2,400 psf. Passive earth pressure for the sides of foundations poured against newly placed engineered fill or undisturbed alluvial soils below the groundwater table may be computed as an equivalent fluid having a density of 120 pcf with a maximum earth pressure of 1,200 psf (values have been reduced for buoyancy). When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.11 Concrete Slabs-on-Grade

8.11.1 Exterior concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 8.12).

8.11.2 Where supported on a conventional foundation system underlain by ground improvement, concrete slabs-on-grade for structures subject to vehicle loading should be a minimum 5 inches of concrete reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint. The slab-on-grade may derive support in the newly placed engineered fill.

8.11.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

8.11.4 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier. A higher coefficient of friction may be feasible within improved soils and should be provided by the ground improvement designer.

8.11.5 Exterior slabs, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to near optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by the project structural engineer.

8.11.6 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.

8.11.7 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

8.12 Preliminary Pavement Recommendations

8.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to near optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

8.12.2 Laboratory testing of the upper existing soils indicates an R-value of 60 and 64 (see Figures 38 and 39). The following pavement sections are based on an R-Value of 50, which is a typical maximum R-value used for design.

8.12.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking and Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	5.0

8.12.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction"* (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation"* (Caltrans). The use of Crushed Miscellaneous Base (CMB) in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction"* (Green Book).

8.12.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).

8.12.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

8.13 Retaining Wall Design

8.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls significantly higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

8.13.2 For the purpose of this report, it is assumed that retaining walls will needed for the proposed swimming pool. If retaining walls are used in other areas of the site, the locations and heights of the walls should be provided to Geocon. Additional recommendations for retaining wall design, including grading and foundation design, must be provided. Depending on the height and location of the retaining wall, retaining wall foundations may need to be underlain by ground improvement.

8.13.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF RETAINING WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 5	30	65

8.13.4 The wall pressures provided above assume that the proposed retaining walls will support newly placed engineered fill or relatively undisturbed alluvial soils. If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

8.13.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, an at-rest equivalent fluid pressure of 95 pcf should be used in design of undrained, restrained walls for the full height of the wall. The value includes hydrostatic pressures plus buoyant lateral earth pressures. If a partially drained wall is proposed, Geocon should be contacted to provide additional recommendations.

8.13.6 Additional pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Surcharges may be evaluated using Section 8.19 of this report. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

8.14 Retaining Wall Drainage

8.14.1 Unless designed for hydrostatic pressures, retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 8). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

8.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 9).

8.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

8.14.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

8.15 Swimming Pool

8.15.1 The proposed swimming pool should be designed as a free-standing structure deriving support in newly placed engineered fill and/or the undisturbed alluvial soils found at and below a depth of 3 feet. Based on the depth of groundwater and potential for liquefaction induced settlement, it is strongly suggested that ground improvement be performed below the pool shell/foundation as the equipment and materials will already be mobilized to the site.

8.15.2 It is assumed that the proposed swimming pool will be 4 feet or less in depth in order to remain above groundwater and avoid dewatering and hydrostatic design. If a deeper pool is planned, Geocon should be contacted to provide additional recommendations.

8.15.3 Swimming pool foundations and walls may be designed in accordance with the foundation design recommendations below and the retaining wall design recommendations provided in Section 8.13. The proposed pool should be constructed utilizing an expansive soils design, and a hydrostatic relief valve should be considered as part of the pool design unless a gravity drain system can be placed beneath the pool shell.

8.15.4 If a spa is proposed, it should be constructed independent of the swimming pool and must not be cantilevered from the swimming pool shell.

8.15.5 A reinforced concrete mat foundation may be utilized for support of the proposed swimming pool. The mat foundation for the pool may derive support in the undisturbed alluvial soils found at and below a depth of 3 feet. If necessary, the mat foundation may derive support in a combination of newly placed engineered fill and undisturbed alluvial soils.

8.15.6 It is anticipated that the proposed mat foundation will impart an average pressure of less than 500 psf. The recommended maximum allowable bearing value is 500 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.15.7 It is recommended that a modulus of subgrade reaction of 50 pounds per cubic inch be utilized for the design of the mat foundation bearing on newly placed engineered fill and/or undisturbed alluvial soils. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where: K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

8.15.8 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

8.15.9 Based on the soil overburden load that will be removed during excavation of the swimming pool, anticipated settlements are expected to be small. We estimate the total settlements for a mat foundation to be less than ½ inch, with differential settlements on the order of ¼ inch over a horizontal distance of 40 feet. Based on seismic considerations, the proposed pool supported on a mat foundation system should be designed for a combined static and seismically induced differential settlement of 1½ inches over a distance of 30 feet.

8.15.10 Foundation excavations should be observed by Geocon, prior to the placement of reinforcing steel and concrete to verify that the exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.16 Elevator Pit Design

8.16.1 The elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pits may be designed in accordance with the recommendations in the *Foundation Design* and *Retaining Wall Design* sections of this report (see Sections 8.8 and 8.13).

8.16.2 Additional pressure should be added for a surcharge condition due to sloping ground, vehicular traffic, or adjacent foundations and should be designed for each condition as the project progresses.

8.16.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Retaining Wall Drainage* section of this report (see Section 8.14). Subdrainage pipes at the base of the retaining wall drainage system should outlet to a location acceptable to the building official.

8.16.4 Proposed structures must be designed for hydrostatic pressure for any portion of the structure below groundwater. Based on the depth of water encountered during current and prior site investigation and the reported historic high groundwater, it is recommended that a groundwater depth of 4 feet be used for design. The hydrostatic design will result in uplift forces on the slab that must be resisted by structural design. The recommended floor slab uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where "H" is the height of the water above the bottom of the foundation in feet.

8.16.5 It is suggested that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

8.17 Elevator Piston

8.17.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation or the drilled excavation could compromise the existing foundation support, especially if the drilling is performed subsequent to the foundation construction.

8.17.2 If the plunger-type elevator piston will be required for this project, and the piston will extend below the groundwater level, the elevator piston should be designed to resist the uplift caused by the buoyant force of the groundwater on the annulus of the elevator piston. Based on the depth of water encountered during current and prior site investigation and the reported historic high groundwater, it is recommended that a groundwater depth of 4 feet be used for design. The recommended uplift pressure to be used in design would be 62.4(H) in units of pounds per square foot (psf), where "H" is the height of the water above the bottom of the elevator piston in feet. If the proposed elevator structure does not provide sufficient dead load to resist the buoyant force then uplift resistance will be required. If it is determined that a plunger-type elevator piston that will extend below the groundwater level will be required for this project, uplift resistance recommendations can be provided under separate cover.

8.17.3 Casing may be required if caving is experienced in the drilled excavation. The contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. The contractor should also be prepared to mitigate buoyant forces during installation of the piston casing. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

8.17.4 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

8.18 Temporary Excavations

8.18.1 Excavations up to 5 feet in height may be required during grading and construction operations. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

8.18.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum height of 8 feet. A uniform slope does not have a vertical portion.

8.18.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for special temporary excavation measures can be provided under separate cover once depths of excavations can be provided to Geocon.

8.18.4 Where temporary construction slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction slopes are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by our personnel and the contractor’s competent person so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

8.19 Surcharge from Adjacent Structures and Improvements

8.19.1 Additional pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses.

8.19.2 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

where x is the distance from the face of the excavation or wall to the vertical line-load, H is the distance from the bottom of the footing to the bottom of excavation or wall, z is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load and $\sigma_H(z)$ is the horizontal pressure at depth z.

8.19.3 It is recommended that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H\ (z) = \ \sigma_H(z) cos^2(1.1\theta)$$

where *x* is the distance from the face of the excavation/wall to the vertical point-load, *H* is distance from the outrigger/bottom of column footing to the bottom of excavation, *z* is the depth at which the horizontal pressure is desired, *Qp* is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth *z*, *Θ* is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth *z*.

8.19.4 In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

8.20 Surface Drainage

8.20.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

8.20.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2022 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

8.20.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

8.20.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

8.21 Plan Review

8.21.1 Grading, foundation, and, if applicable, shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

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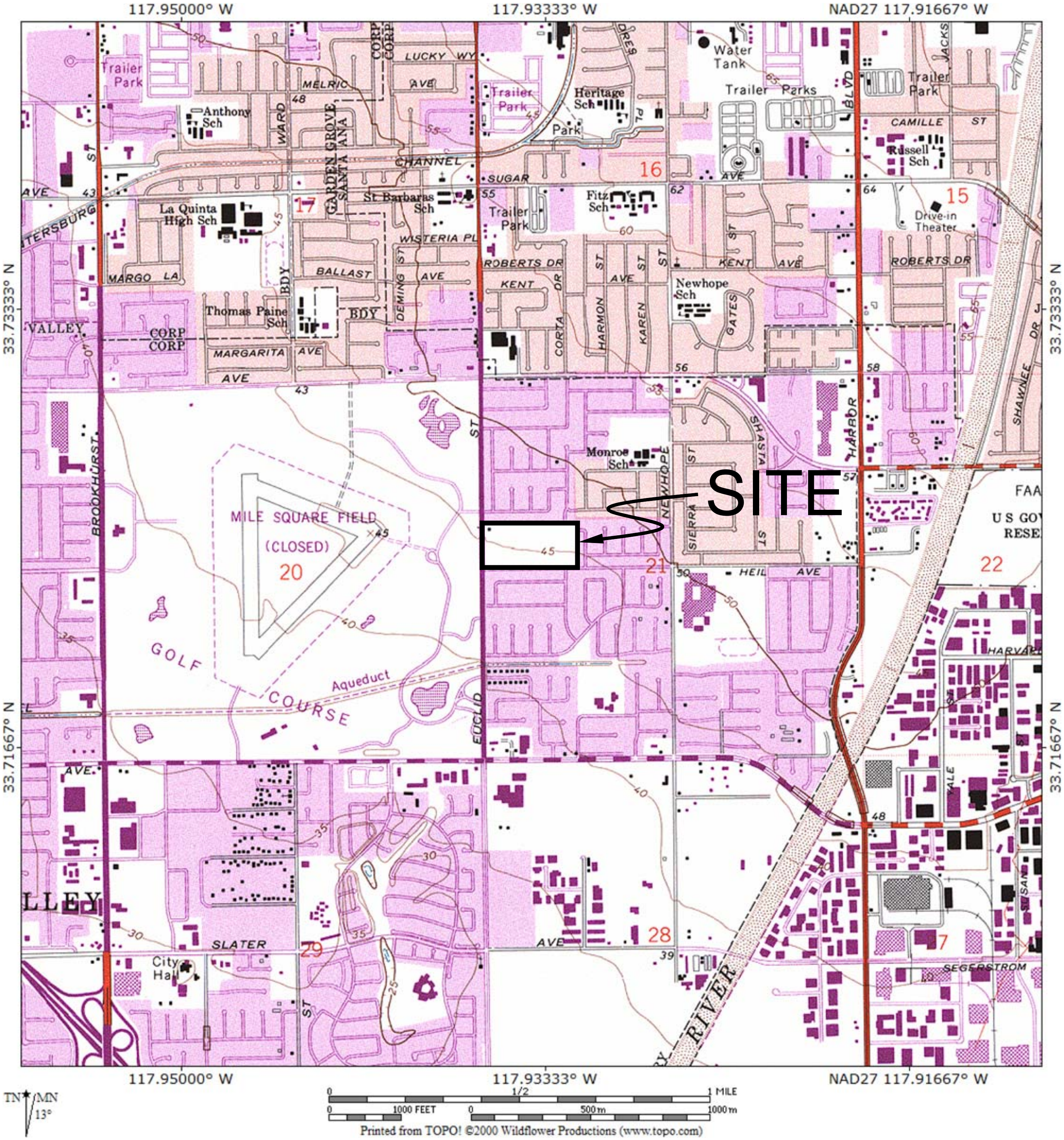

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U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, NEWPORT BEACH, CA QUADRANGLE

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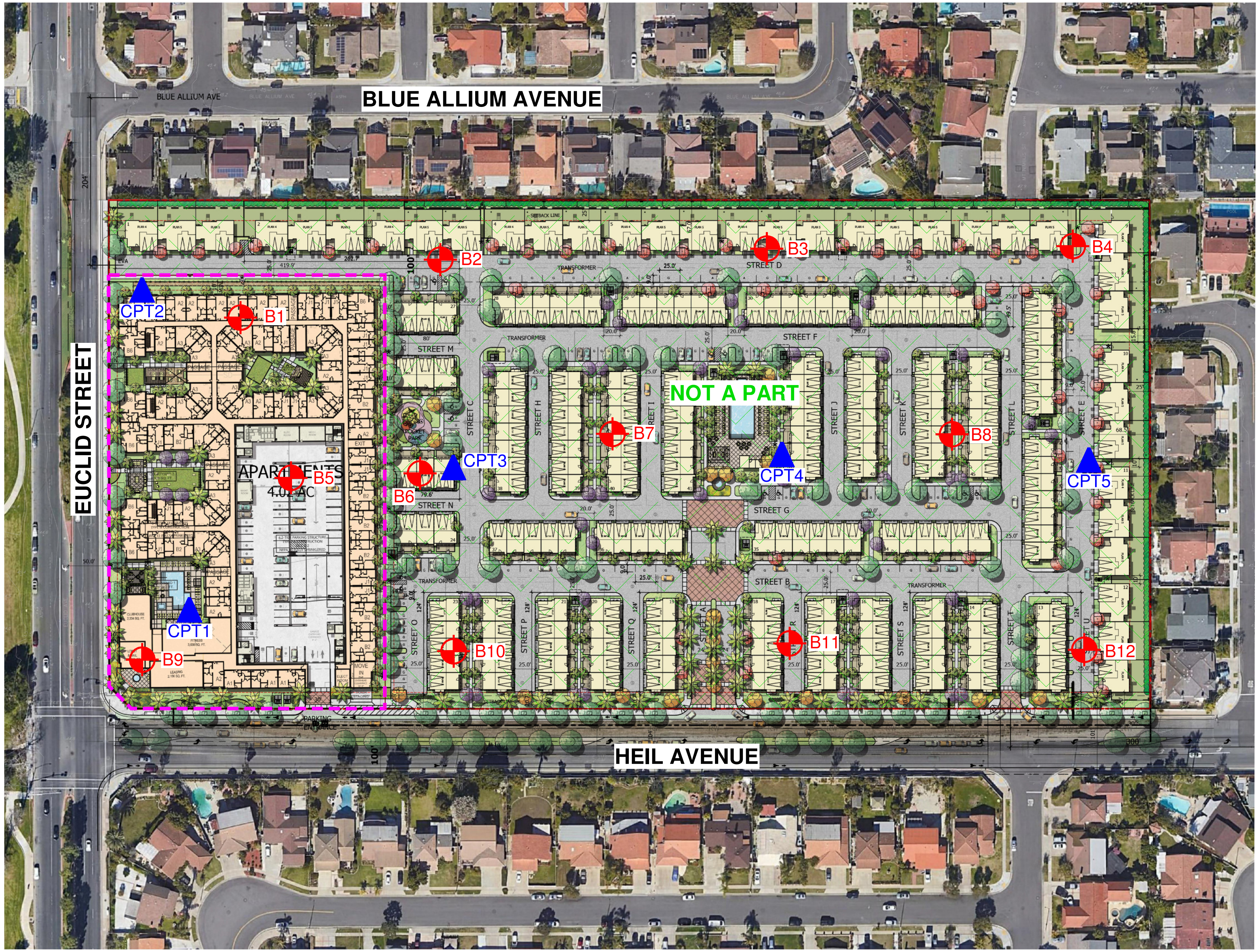
VICINITY MAP

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025 | PROJECT NO. W2045-88-01 | FIG. 1



LEGEND

⌖ B12	Approximate Location of Boring (Geocon, 2025)
▲ CPT5	Approximate Location of CPT (Leighton, 2024)
⬚	Approximate Limits of Proposed Project

NORTH

0 150' 300'

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SITE PLAN

16300 EUCLID STREET
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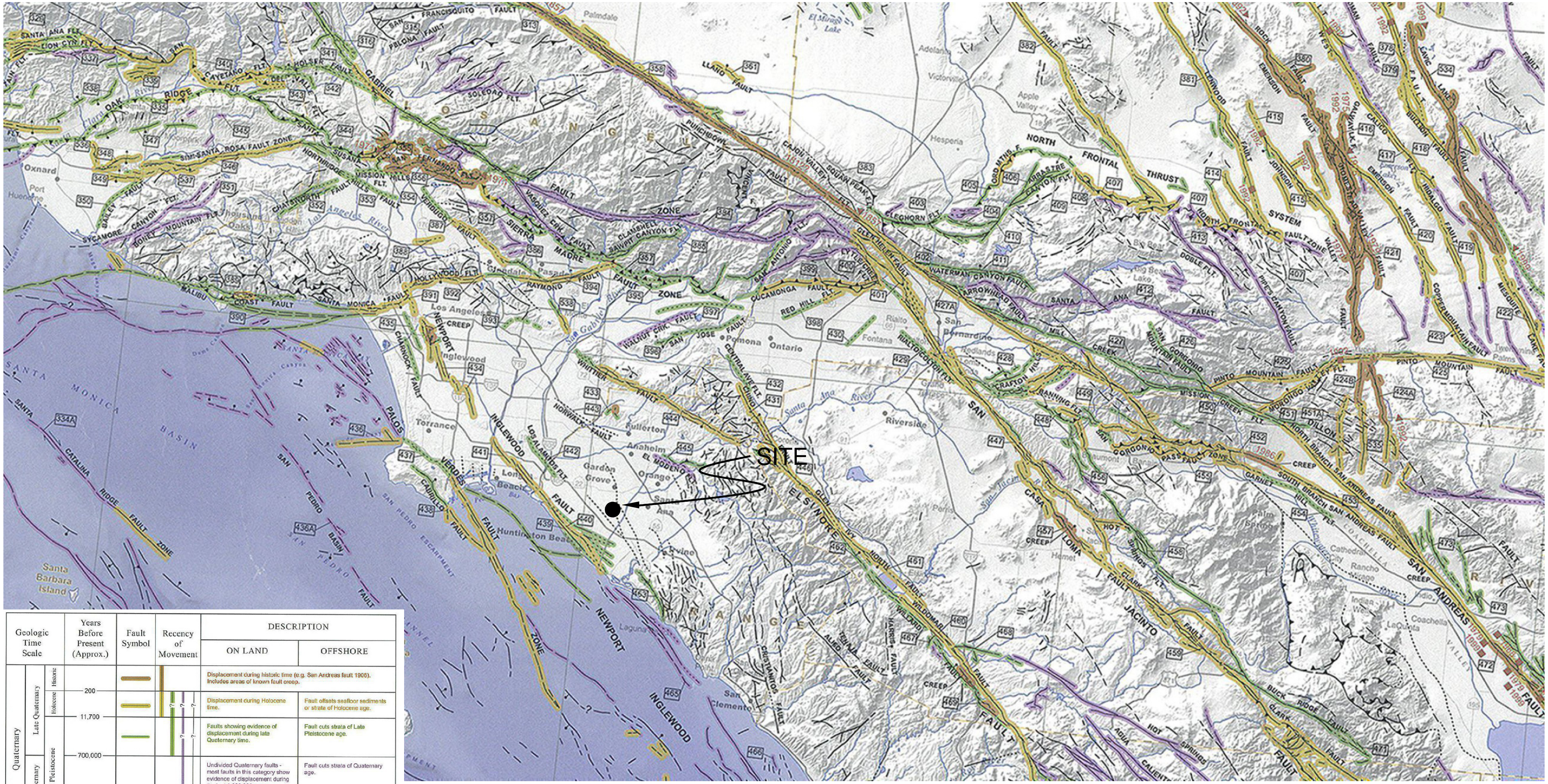
APN 144-11-01

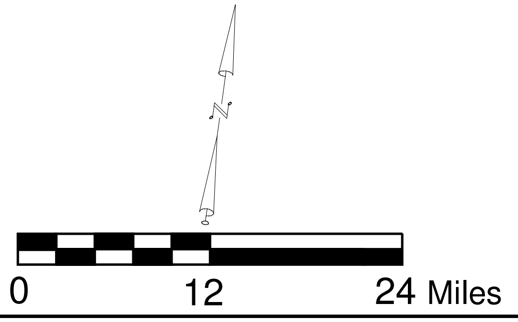
MAR. 2025	PROJECT NO. W2045-88-01	FIG. 2

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION ON LAND	DESCRIPTION OFFSHORE
Quaternary	Late Quaternary	Historic		🟫		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟨	? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩	? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪	?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000* 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.



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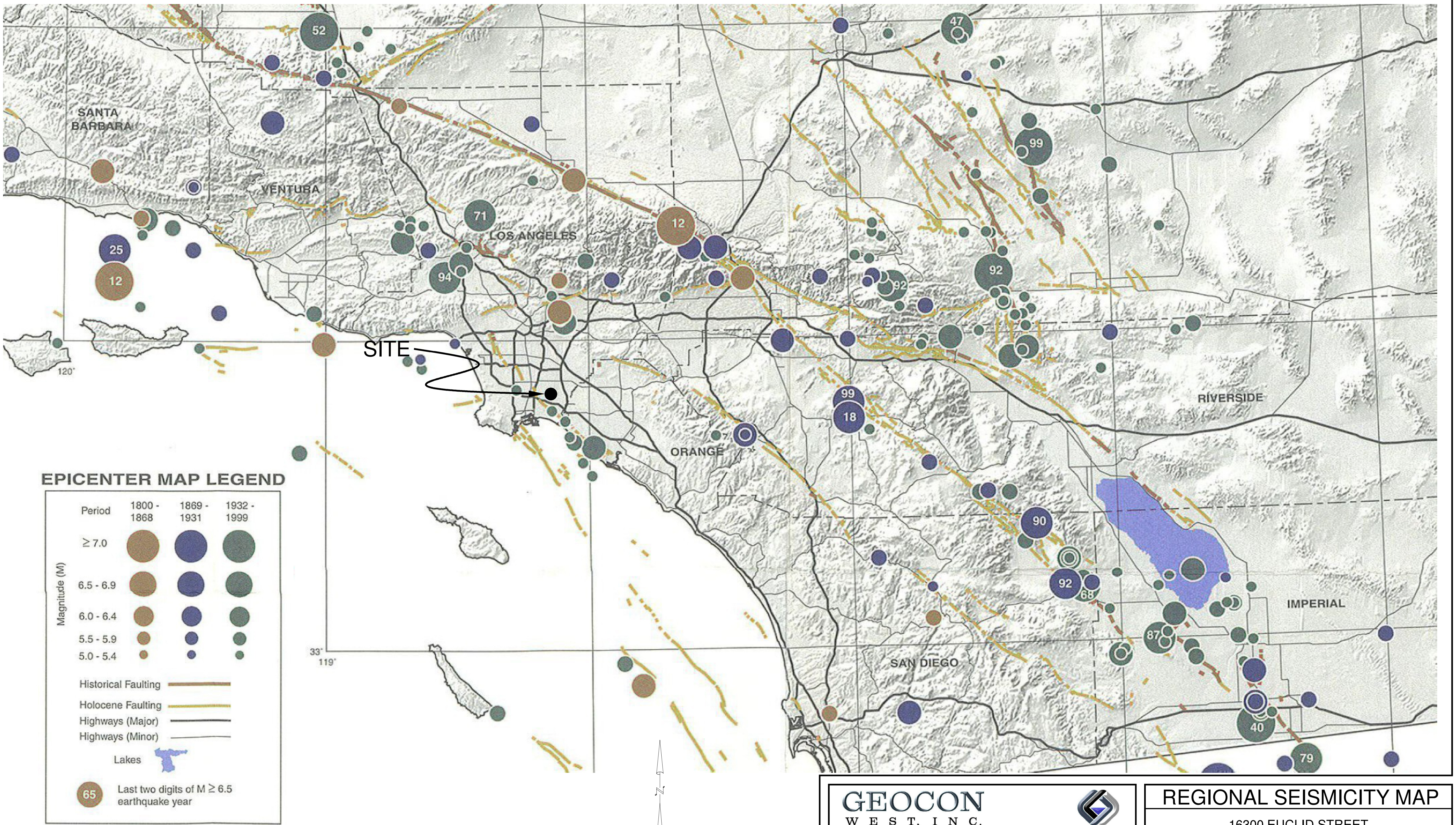
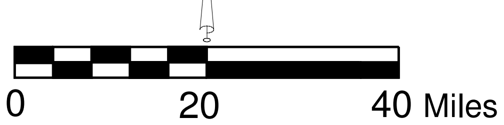
REGIONAL FAULT MAP

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025 | PROJECT NO. W2045-88-01 | FIG. 3



Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M>5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.



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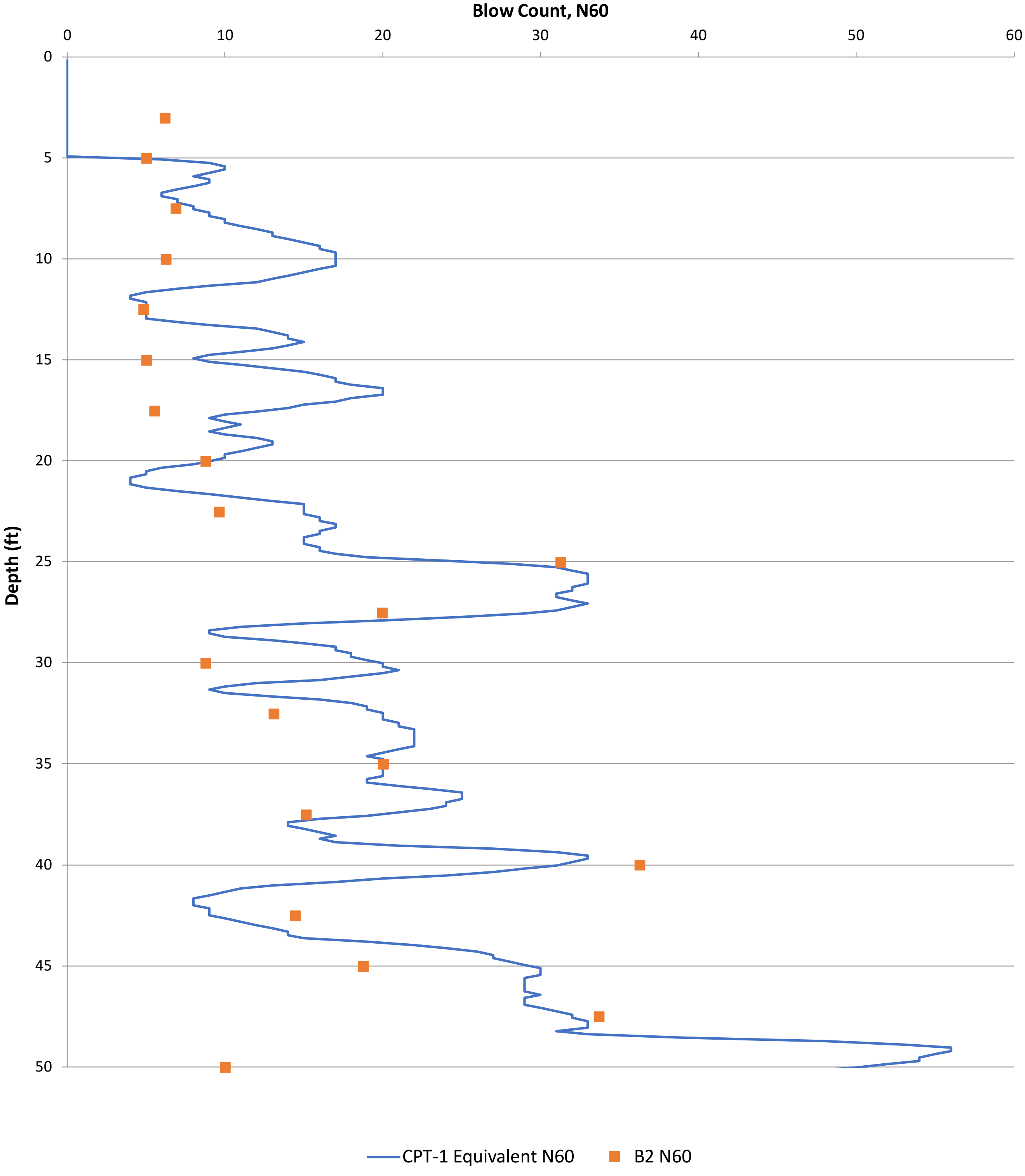
REGIONAL SEISMICITY MAP

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025	PROJECT NO. W2045-88-01	FIG. 4



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CORRELATION OF BORING AND CPT N60

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

DRAFTED BY: PZ	CHECKED BY: JTA	MAR. 2025	PROJECT NO. W2045-88-01	FIG. 5

Project title : W2045-88-01

Location : Euclid and Heil

Overall Vertical Settlements - Design Earthquake

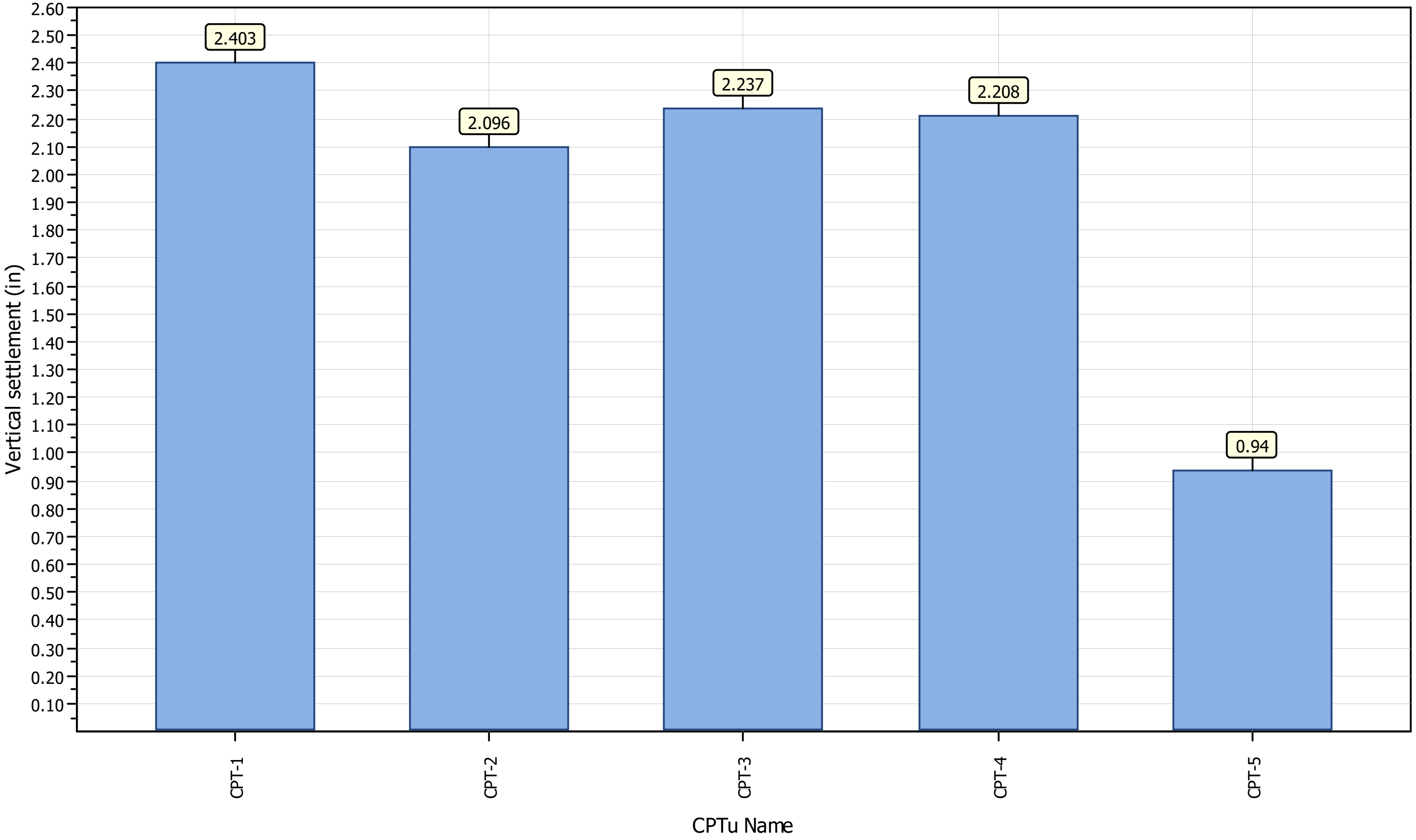
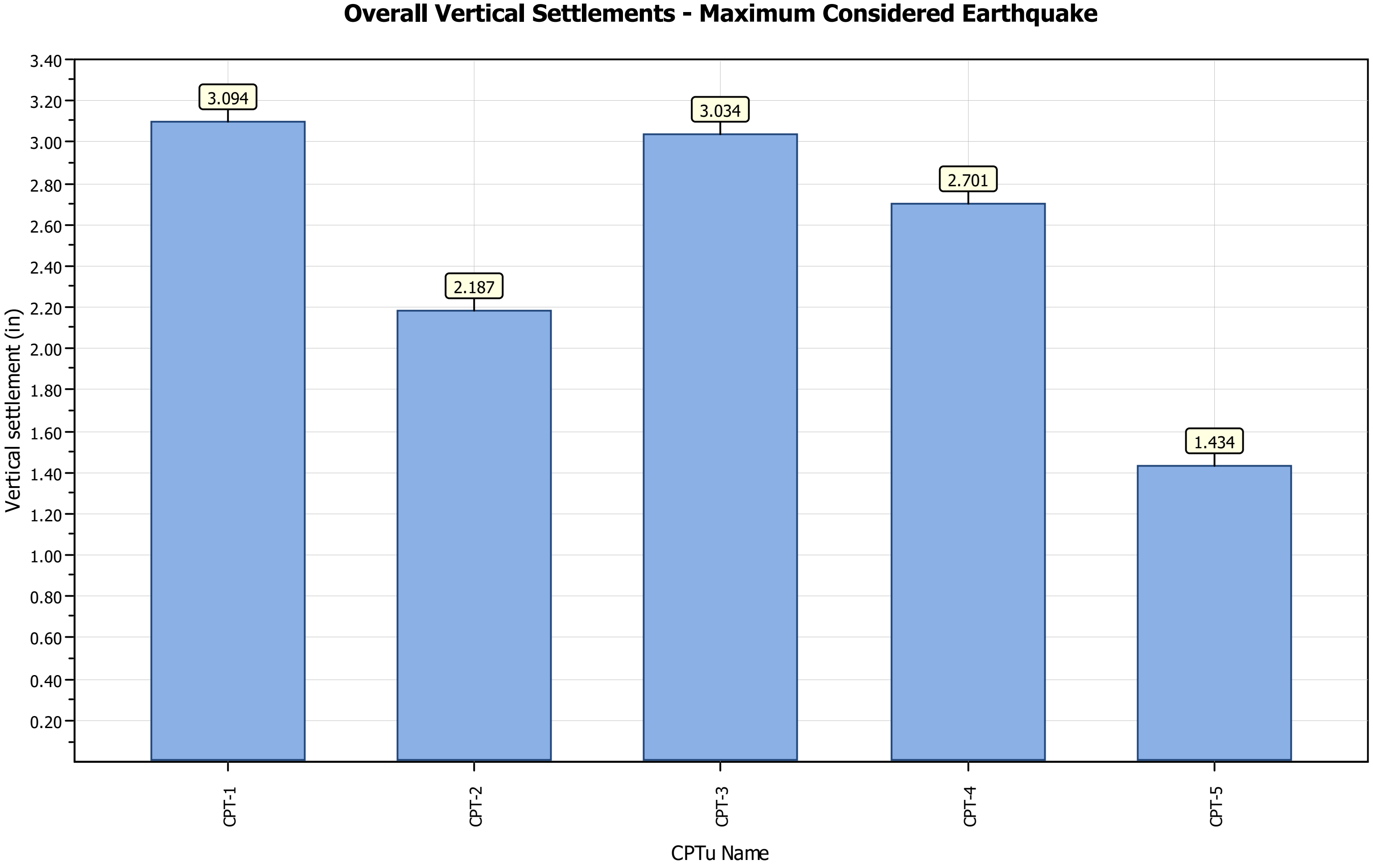
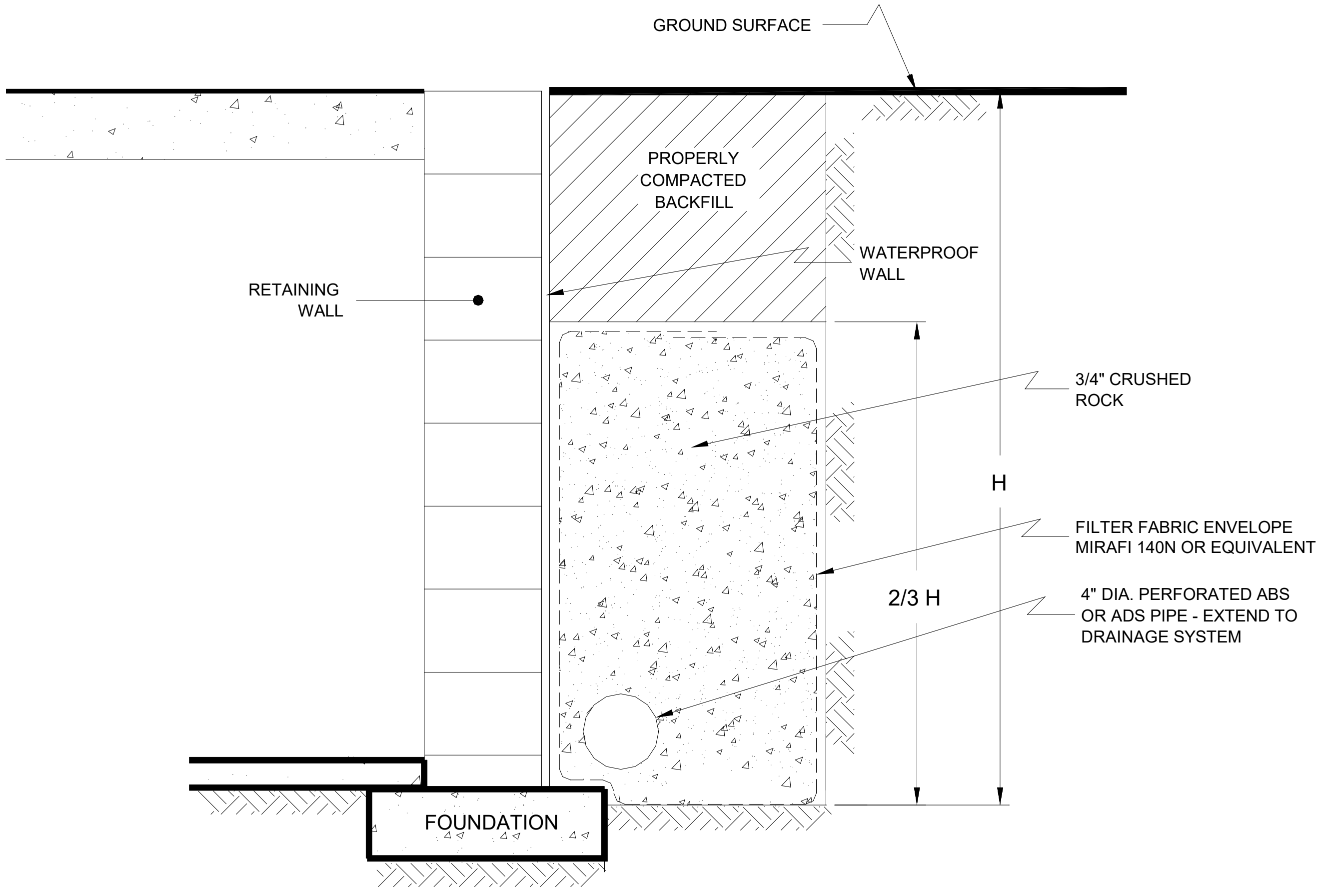


Figure 6

Project title : W2045-88-01

Location : Euclid and Heil





NO SCALE

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RETAINING WALL DRAIN DETAIL

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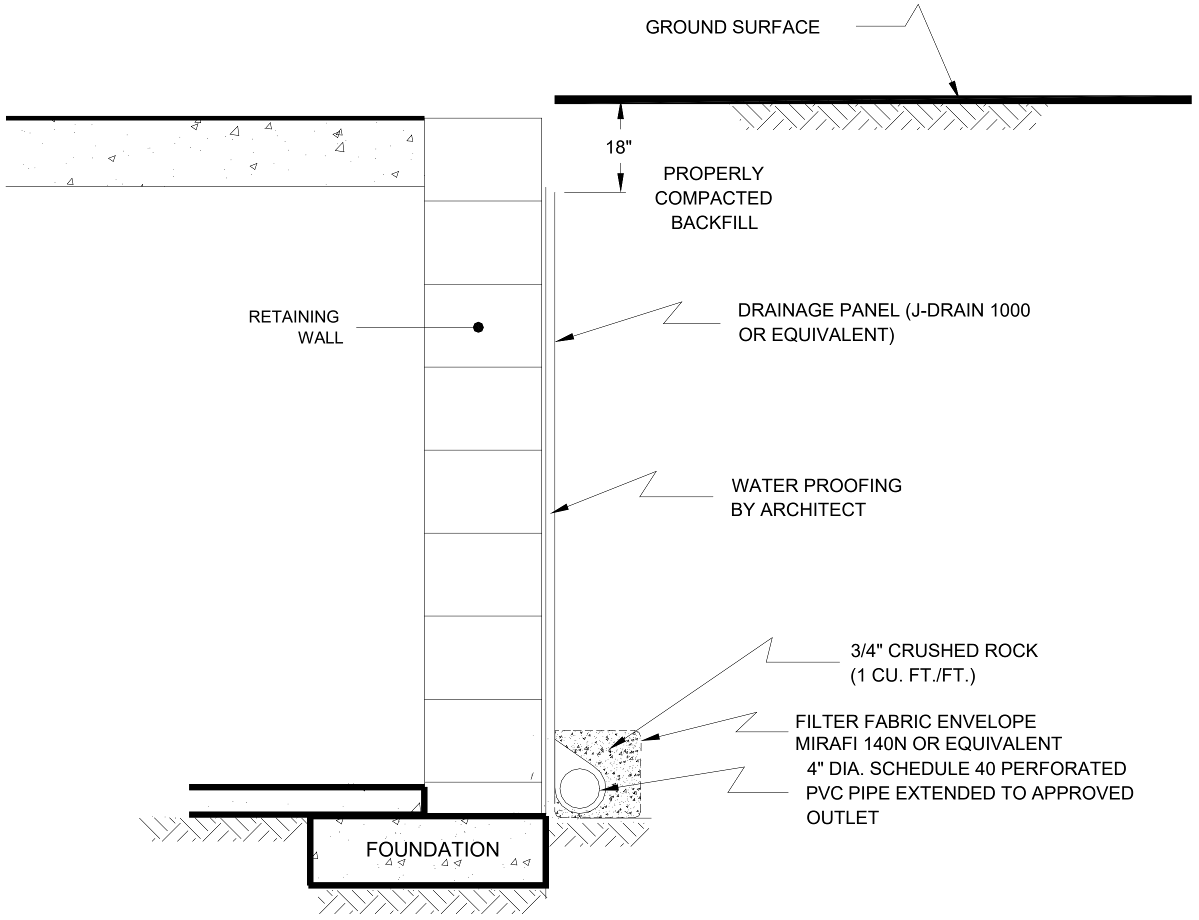
APN 144-11-01

DRAFTED BY: PZ
CHECKED BY: JTA
MAR. 2025
PROJECT NO. W2045-88-01
FIG. 8



NO SCALE

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
2807 MCGAW AVENUE - IRVINE, CA 92614
PHONE (949) 491-6570 - FAX (949) 299-4550

RETAINING WALL DRAIN DETAIL

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

DRAFTED BY: PZ	CHECKED BY: JTA	MAR. 2025	PROJECT NO. W2045-88-01	FIG. 9

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored simultaneously with the adjacent vacant land. The overall site exploration was performed on January 10 and February 21, 2025, by excavating seven 8-inch diameter borings using a truck-mounted, hollow-stem auger drilling machine and two 4-inch diameter borings using a truck-mounted, mud-rotary drilling machine. The borings were drilled to depths between 10½ and 58½ feet below the existing ground surface. Additionally, on February 26, 2025, three 4-inch diameter hand auger borings (borings B7, B10, and B11) were excavated to depths between 6 and 8½ feet below the existing ground surface using manual digging equipment and hand tools. Representative and relatively undisturbed samples were obtained by manually driving a 3-inch O. D. California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches (hollow stem and mud rotary borings) as well as a slide-hammer (hand auger borings). The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained. Standard Penetration Tests were performed in boring B6.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A12. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The approximate locations of the borings are shown on Figure 2.

SOIL BORING NUMBER: B - 1

Page 1 of 1

PROJECT NAME Euclid & Heil
PROJECT NUMBER W2045-88-01
BORING DATE 01/10/2025 **FIGURE NUMBER** A1
LOCATION 16300 Euclid Street, Fountain Valley, CA
DRILLING FIRM BC2 Environmental, LLC
METHOD Cal-Mod **BORING DIAMETER** 8 in

LOGGED BY ACS
LATITUDE / LONGITUDE 33.72508, -117.93654
DEPTH 10.5' **SURFACE ELEVATION** N/A
EQUIPMENT Hollow Stem Auger
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
		▨	Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, slightly moist, brown, fine-grained, plastic tarp, irrigation lines	☒		BULK 0-5'			
2		▨			☒	■	B1@1'	4 5 6	100.0	4.0
4		░	SM	**YOUNG ALLUVIAL FAN DEPOSITS** **Silty SAND**, loose, moist, reddish brown, fine-grained	☒	■	B1@3'	4 5 8	100.3	10.8
6		░		wet, grayish brown, increase in silt		■	B1@5'	3 4 5	82.3	31.7
8	▽	░				■	B1@7'	5 6 7	102.7	23.7
10		░		medium dense		■	B1@10'	5 10 11	97.5	20.4
12										
14										
16										
18										

Total depth of boring: 10.5 feet
Fill to 3 feet.
Groundwater encountered at 7.5 feet.
Backfilled with soil cuttings and tamped.

SOIL BORING NUMBER: B - 2

Page 1 of 1

PROJECT NAME Euclid & Heil

LOGGED BY ACS

PROJECT NUMBER W2045-88-01

LATITUDE / LONGITUDE 33.72525, -117.93569

BORING DATE 01/10/2025 **FIGURE NUMBER** A2

DEPTH 10.5' **SURFACE ELEVATION** N/A

LOCATION 16300 Euclid Street, Fountain Valley, CA

DRILLING FIRM BC2 Environmental, LLC

EQUIPMENT Hollow Stem Auger

METHOD Cal-Mod **BORING DIAMETER** 8 in

HAMMER TYPE Auto

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
0		▩	Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, slightly moist, brown, fine-grained	☒		BULK 0-5'			
2										
2.5		░	SM	**YOUNG ALLUVIAL FAN DEPOSITS** **Silty SAND**, loose, moist, orange brown, fine- to medium-grained		■	B2@2.5'	5 6 8	88.0	18.9
4										
5				wet, grayish brown		■	B2@5'	5 7 8	99.6	22.9
6										
7.5	▽					■	B2@7.5'	4 5 7	100.1	24.1
8										
9.5		░	SP	**SAND**, poorly graded, medium dense, wet, gray, medium- to coarse-grained						
10						■	B2@10'	7 9 10	108.1	20.4
10.5				Total depth of boring: 10.5 feet Fill to 2.5 feet. Groundwater encountered at 7.5 feet. Backfilled with soil cuttings and tamped.						
12										
14										
16										
18										

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING NUMBER: B - 3

Page 1 of 1

PROJECT NAME Euclid & Heil

LOGGED BY ACS

PROJECT NUMBER W2045-88-01

LATITUDE / LONGITUDE 33.7532, -117.93444

BORING DATE 01/10/2025 **FIGURE NUMBER** A3

DEPTH 10.5' **SURFACE ELEVATION** N/A

LOCATION 16300 Euclid Street, Fountain Valley, CA

DRILLING FIRM BC2 Environmental, LLC

EQUIPMENT Hollow Stem Auger

METHOD Cal-Mod **BORING DIAMETER** 8 in

HAMMER TYPE Auto

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
		▨	Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, slightly moist, brown, fine-grained						
		▨				■	B3@1'	3 4 7	94.8	10.2
2		▨								
		░	SM	**YOUNG ALLUVIAL FAN DEPOSITS** **Silty SAND**, loose, moist, grayish brown, fine- to medium-grained, local oxidation						
		░				■	B3@3'	4 5 7	101.6	12.3
4		░								
		░		no recovery		○	B3@5'	3 3 4		
6		░								
	▽	░				■	B3@7'	3 4 7	102.8	24.6
8		░								
10		░				■	B3@10'	7 4 3	104.9	20.2
				Total depth of boring: 10.5 feet Fill to 2.5 feet. Groundwater encountered at 7.5 feet. Backfilled with soil cuttings and tamped.						
12										
14										
16										
18										

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING NUMBER: B - 4

Page 1 of 1

PROJECT NAME Euclid & Heil
LOGGED BY ACS
PROJECT NUMBER W2045-88-01
LATITUDE / LONGITUDE 33.72532, -117.93318
BORING DATE 01/10/2025
FIGURE NUMBER A4
DEPTH 10.5'
SURFACE ELEVATION N/A
LOCATION 16300 Euclid Street, Fountain Valley, CA
DRILLING FIRM BC2 Environmental, LLC
EQUIPMENT Hand Auger
METHOD Cal-Mod
BORING DIAMETER 8 in
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Moisture Content (%)
2		▩	Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, slightly moist, brown, fine-grained	☒		BULK 0-5'		
4		▥	SM	**YOUNG ALLUVIAL FAN DEPOSITS** **Silty SAND**, loose, moist, brown, fine- to medium-grained		■	B4@2.5'	4 6 9	
						■	B4@5'	2 3 4	38.1
6	∇	▥	ML	**Sandy SILT**, soft, wet, brown, fine-grained					
8		▥	SM	**Silty SAND**, loose, wet, gray, fine- to medium-grained		■	B4@7.5'	2 3 4	21.4
10				no recovery		○	B4@10'	3 4 5	
				Total depth of boring: 10.5 feet Fill to 2.5 feet. Groundwater encountered at 7.5 feet. Backfilled with soil cuttings and tamped.					
12									
14									
16									
18									

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING NUMBER: B - 5

PROJECT NAME Euclid & Heil

LOGGED BY JJK

PROJECT NUMBER W2045-88-01

LATITUDE / LONGITUDE 33.72443, -117.93616

BORING DATE 02/21/2025 **FIGURE NUMBER** A5

DEPTH 30.5' **SURFACE ELEVATION** N/A

LOCATION 16300 Euclid Street, Fountain Valley, CA

DRILLING FIRM BC2 Environmental, LLC

EQUIPMENT Mud Rotary

METHOD Cal-Mod **BORING DIAMETER** 4 in

HAMMER TYPE Auto

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
		▨	Fill	**ARTIFICIAL FILL** **Silty SAND**, loose, moist, light brown, fine-grained						
2 4		▥	SM	**YOUNG ALLUVIAL FAN DEPOSITS** **Silty SAND**, medium dense, moist, grayish brown and light brown, fine-grained, trace oxidation staining		■	B5@2.5'	3 6 8	101.9	11.5
		▥	SP-SM	**SAND w/ Silt**, poorly graded, medium dense, moist to wet, grayish brown, fine-grained		■	B5@5'	7 8 10	90.5	11.6
6 8	∇	░	SP	**SAND**, poorly graded, medium dense, wet, grayish brown, fine-grained, trace medium-grained		■	B5@7.5'	7 10 10	98.2	27.6
10 12		▥	SP-SM	**SAND w/ Silt**, poorly graded, medium dense, wet, grayish brown, fine-grained		■	B5@10'	8 8 8	103.9	22.4
14 16 18		▥	SM	**Silty SAND**, loose, wet, dark gray, fine-grained		■	B5@15'	6 3 4	100.4	23.6

GEOCON

SOIL BORING: B-5

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
			ML	**Sandy SILT**, firm, moist to wet, dark gray, fine-grained		■	B5@20'	4 9 10	97.0	27.6
22										
24										
				gray and olive gray		■	B5@25'	4 7 11	96.3	28.7
26										
28										
			CL	**CLAY w/ Sand**, soft, moist to wet, black to light gray, trace fine-grained						
30						■	B5@30'	2 4 5	82.7	40.5
				Total depth of boring: 30.5 feet Fill to 2 feet. Groundwater encountered at 6.5 feet. Backfilled with cement bentonite grout.						
32										
34										
36										
38										

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING NUMBER: B - 6

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PROJECT NAME Euclid & Heil

LOGGED BY JJK

PROJECT NUMBER W2045-88-01

LATITUDE / LONGITUDE 33.72455, -117.93583

BORING DATE 02/21/2025

FIGURE NUMBER A6

DEPTH 58.5'

SURFACE ELEVATION N/A

LOCATION 16300 Euclid Street, Fountain Valley, CA

DRILLING FIRM BC2 Environmental, LLC

EQUIPMENT Mud Rotary

METHOD Cal-Mod

BORING DIAMETER 4 in

HAMMER TYPE Auto

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
			Fill	**ARTIFICIAL FILL** **Silty SAND**, loose, moist, light brown, fine-grained	☒		BULK 0-5'			
2			SM	**YOUNG ALLUVIAL FAN DEPOSITS** **Silty SAND**, loose, moist, grayish brown, fine-grained						
						■	B6@3'	3 4 5	103.4	11.1
4										
						■	B6@5'	1 2 2		23.0
6	▽		SP-SM	**SAND w/ Silt**, poorly graded, very loose, moist to wet, grayish brown, fine-grained						
8				no recovery		○	B6@7.5'	8 4 6		
10			SP	**SAND**, poorly graded, loose, wet, grayish brown, fine-grained		■	B6@10'	4 3 2		25.8
12										
				light gray		■	B6@12.5'	6 4 3	96.0	23.5
14			ML	**Sandy SILT**, soft, moist to very moist, dark gray, fine-grained						
						■	B6@15'	2 1 3		30.2
16										
18				moist to very moist, dark gray		■	B6@17.5'	4 3 5	92.8	31.0

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING: B-6

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
			SM	**Silty SAND**, loose, very moist to wet, dark gray, fine-grained		■	B6@20'	5 4 3		32.5
22										
				increase in silt		■	B6@22.5'	4 6 8	97.9	29.1
24			SP-SM	**SAND w/ Silt**, poorly graded, medium dense, wet, dark gray, fine-grained						
						■	B6@25'	6 11 14		24.3
26										
						■	B6@27.5'	10 17 12	107.4	21.2
28			SM	**Silty SAND**, loose, wet, dark gray, fine-grained						
30						■	B6@30'	1 2 5		26.5
32										
				medium dense, moist		■	B6@32.5'	6 7 12	97.5	29.0
34			ML	**SILT w/ Sand**, firm, moist to wet, dark gray, trace fine-grained sand, trace clay/pasticity						
						■	B6@35'	3 7 9		29.9
36										
			SM	**Silty SAND**, medium dense, moist to wet, dark gray, fine-grained						
38						■	B6@37.5'	9 10 12	95.2	30.5

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

SOIL BORING: B-6

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
			SM			■	B6@40'	11 14 15		31.0
42			CL	**LEAN CLAY**, firm, moist, dark gray and black, some silt		■	B6@42.5'	5 7 14	96.8	28.6
44			CL	**CLAY w/ Sand**, firm, moist to wet, dark gray, trace fine- to medium-grained		■	B6@45'	4 5 10		32.6
46			SM	**Silty SAND**, medium dense, moist, dark gray, fine-grained		■	B6@47.5'	4 21 28		26.3
48										
50				no recovery		○	B6@50'	9 3 5		
52			CL	**CLAY**, stiff, moist to wet, dark gray, trace fine-grained		■	B6@52.5'	5 7 13		29.2
54			CH	**FAT CLAY**, firm, moist, bluish gray and gray		■	B6@55'	3 4 6		40.3
56			SM	**Silty SAND**, dense, moist to wet, dark gray, fine-grained		■	B6@57.5'	12 15 17		25.0
58				Total depth of boring: 58.5 feet Fill to 1.5 feet. Groundwater encountered at 6 feet. Backfilled with cement bentonite grout.						

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING NUMBER: B - 7

Page 1 of 1

PROJECT NAME Euclid & Heil
PROJECT NUMBER W2045-88-01
BORING DATE 02/26/2025 **FIGURE NUMBER** A7
LOCATION 16300 Euclid Street, Fountain Valley, CA
DRILLING FIRM Gold Construction Services
METHOD Cal-Mod **BORING DIAMETER** 3 in

LOGGED BY ACS
LATITUDE / LONGITUDE 33.72463, -117.93508
DEPTH 8' **SURFACE ELEVATION** N/A
EQUIPMENT Hand Auger
HAMMER TYPE Slide
HAMMER WEIGHT / DROP - / -

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Dry Density (pcf)	Moisture Content (%)
		▩	Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, moist, brown, fine-grained, roots					
					☒		BULK 1-1.5'		
2									
		░	SP	**YOUNG ALLUVIAL FAN DEPOSITS** **SAND**, poorly graded, loose, slightly moist, brown, fine- to medium-grained		■	B7@2.5'		31.3
4					☒		BULK 4-4.5'		
						■	B7@5'		11.8
6	▽			moist to wet					
		▥	ML	**Sandy SILT**, soft, wet, black, fine-grained					
						■	B7@7.5'	100.0	22.7
8				Total depth of boring: 8 feet Fill to 2.5 feet. Groundwater encountered at 6.5 feet. Backfilled with soil cuttings and tamped.					
10									
12									
14									
16									
18									

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

SOIL BORING NUMBER: B - 8

Page 1 of 1

PROJECT NAME Euclid & Heil
LOGGED BY ACS
PROJECT NUMBER W2045-88-01
LATITUDE / LONGITUDE 33.72466, -117.93361
BORING DATE 01/10/2025 **FIGURE NUMBER** A8
DEPTH 10.5' **SURFACE ELEVATION** N/A
LOCATION 16300 Euclid Street, Fountain Valley, CA
DRILLING FIRM BC2 Environmental, LLC
EQUIPMENT Hollow Stem Auger
METHOD Cal-Mod **BORING DIAMETER** 8 in
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
2		▨	Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, slightly moist, brown, fine-grained						
4		░	SM	**YOUNG ALLUVIAL FAN DEPOSITS** **Silty SAND**, loose, slightly moist, brown, fine- to medium-grained		■	B8@2.5'	5 7 10	98.5	3.7
6		░		no recovery		○	B8@5'	4 5 7		
8	▽	░		no recovery		○	B8@7.5'	2 3 5		
10		░		no recovery		○	B8@10'	2 3 5		
12 14 16 18										

Total depth of boring: 10.5 feet
Fill to 2.5 feet.
Groundwater encountered at 7 feet.
Backfilled with soil cuttings and tamped.

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

SOIL BORING NUMBER: B - 9

Page 1 of 1

PROJECT NAME Euclid & Heil

PROJECT NUMBER W2045-88-01

BORING DATE 01/10/2025 **FIGURE NUMBER** A9

LOCATION 16300 Euclid Street, Fountain Valley, CA

DRILLING FIRM BC2 Environmental, LLC

METHOD Cal-Mod **BORING DIAMETER** 8 in

LOGGED BY ACS

LATITUDE / LONGITUDE 33.72395, -117.93695

DEPTH 10.5' **SURFACE ELEVATION** N/A

EQUIPMENT Hollow Stem Auger

HAMMER TYPE Auto

HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
		▨	Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, slightly moist, brown, fine-grained	☒		BULK 0-5'			
		▨		moist	☒	■	B9@1'	5 5 6	95.0	6.0
2		▨			☒					
		░	SM	**YOUNG ALLUVIAL FAN DEPOSITS** **Silty SAND**, loose, orange brown, fine- to medium-grained	☒	■	B9@3'	4 5 6	96.4	7.3
4		░			☒					
		░		grayish brown		■	B9@5'	5 7 7	95.4	20.8
6		░								
	▽	░		medium dense, wet		■	B9@7'	5 7 14	102.2	23.4
8		░								
10		░				■	B9@10'	5 9 10	103.2	19.2
				Total depth of boring: 10.5 feet Fill to 2.5 feet. Groundwater encountered at 7.5 feet. Backfilled with soil cuttings and tamped.						
12										
14										
16										
18										

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

SOIL BORING NUMBER: B - 1 0

Page 1 of 1

PROJECT NAME Euclid & Heil

LOGGED BY ACS

PROJECT NUMBER W2045-88-01

LATITUDE / LONGITUDE 33.72395, -117.93566

BORING DATE 02/26/2025 **FIGURE NUMBER** A10

DEPTH 8.6' **SURFACE ELEVATION** N/A

LOCATION 16300 Euclid Street, Fountain Valley, CA

DRILLING FIRM Gold Construction Services

EQUIPMENT Hand Auger

METHOD Cal-Mod **BORING DIAMETER** 3 in

HAMMER TYPE Slide

HAMMER WEIGHT / DROP - / -

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Dry Density (pcf)	Moisture Content (%)
			Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, moist, brown, fine-grained, roots, plants, plastic tarp					
					☒		BULK 1-1.5'		
2									
						■	B10@2.5'	87	33
4			SP	**YOUNG ALLUVIAL FAN DEPOSITS** **SAND**, poorly graded, slightly moist, brown, fine- to medium-grained	☒		BULK 4-4.5'		
						■	B10@5'	100	14
6	▽								
			ML	**Sandy SILT**, soft, moist to wet, black, fine-grained					
			SM	**Silty SAND**, loose, wet, dark brown, fine-grained					
8						■	B10@7.5'	92	32
				Total depth of boring: 8 feet 6 inches Fill to 4 feet. Groundwater encountered at 6.5 feet. Backfilled with soil cuttings and tamped.					
10									
12									
14									
16									
18									

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING NUMBER: B - 1 1

Page 1 of 1

PROJECT NAME Euclid & Heil
LOGGED BY ACS
PROJECT NUMBER W2045-88-01
LATITUDE / LONGITUDE 33.72397, -117.93434
BORING DATE 02/26/2025 **FIGURE NUMBER** A11
DEPTH 6' **SURFACE ELEVATION** N/A
LOCATION 16300 Euclid Street, Fountain Valley, CA
DRILLING FIRM Gold Construction Services
EQUIPMENT Hand Auger
METHOD Cal-Mod **BORING DIAMETER** 3 in
HAMMER TYPE Slide
HAMMER WEIGHT / DROP - / -

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Dry Density (pcf)	Moisture Content (%)
		▩	Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, moist, brown, fine-grained, roots	☒		BULK 1-1.5'		
2									
		░	SP	**YOUNG ALLUVIAL FAN DEPOSITS** **SAND**, poorly graded, loose, slightly moist, brown, fine- to medium-grained		■	B11@2.5'	93.8	3.3
4					☒		BULK 4-4.5'		
						■	B11@5'	92.3	23.2
6	▽			Total depth of boring: 6 feet Fill to 2.5 feet. Groundwater encountered at 6 feet. Backfilled with soil cuttings and tamped.					
8									
10									
12									
14									
16									
18									

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING NUMBER: B - 1 2

Page 1 of 1

PROJECT NAME Euclid & Heil
LOGGED BY ACS
PROJECT NUMBER W2045-88-01
LATITUDE / LONGITUDE 33.72398, -17.93317
BORING DATE 01/10/2025
FIGURE NUMBER A12
DEPTH 10.5'
SURFACE ELEVATION N/A
LOCATION 16300 Euclid Street, Fountain Valley, CA
DRILLING FIRM BC2 Environmental, LLC
EQUIPMENT Hollow Stem Auger
METHOD Cal-Mod
BORING DIAMETER 8 in
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Dry Density (pcf)	Moisture Content (%)
		▩	Fill	**ARTIFICIAL FILL** **Sandy SILT**, soft, slightly moist, brown, fine-grained	☒		BULK 0-5'		
2		▥	SM	**YOUNG ALLUVIAL FAN DEPOSITS** **Silty SAND**, loose, slightly moist, brown, fine- to medium-grained		■	B12@2.5'	100.4	2.6
4		░	SP	**SAND**, poorly graded, very loose, moist, orange brown, fine- to medium-grained		■	B12@5'		6.2
6									
8				no recovery		○	B12@7.5'		
10						■	B12@10'	77.9	10.8
				Total depth of boring: 10.5 feet Fill to 2 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.					
12									
14									
16									
18									

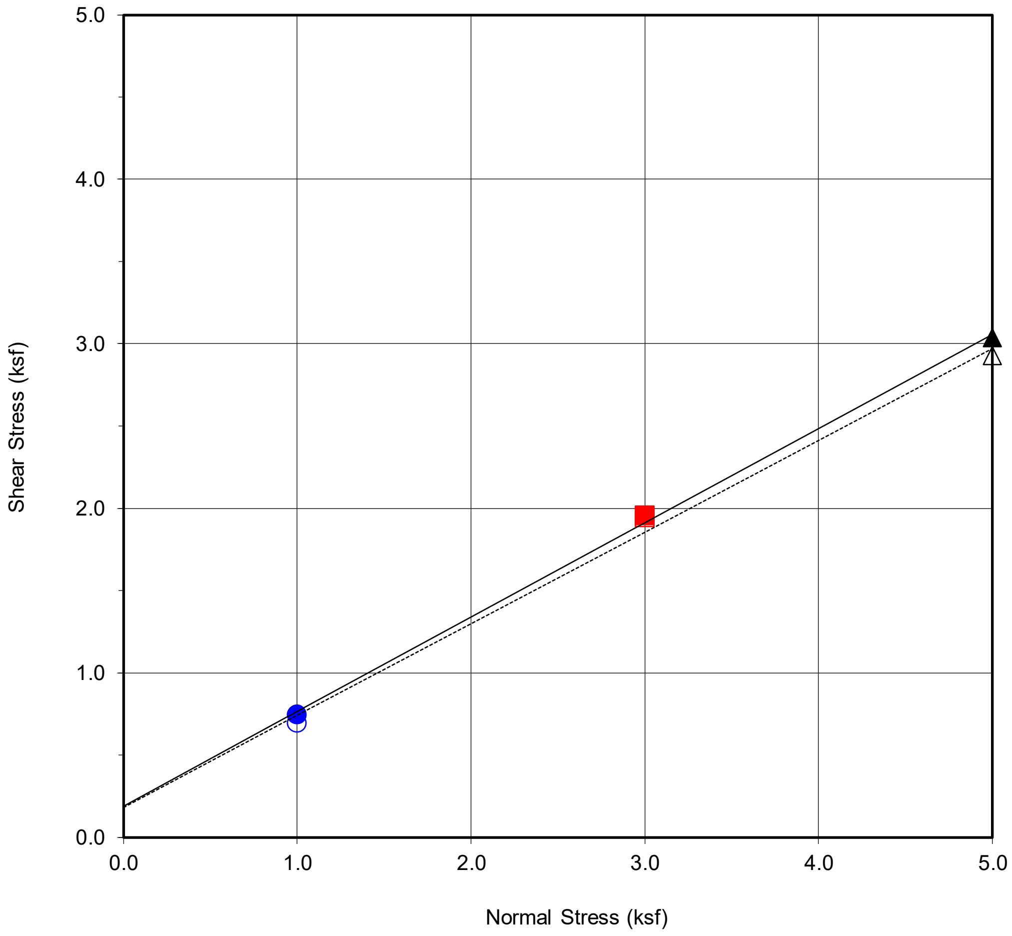
NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, moisture density relationships, Atterberg limits, grain-size, corrosivity, organic content, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B41. The in-place dry density and moisture content of the samples tested are presented in the boring logs, Appendix A.



Boring No.	**B1**
Sample No.	**B1@0-5'**
Depth (ft)	**0-5'**
Sample Type:	REMOLD

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	193	30
Ultimate	182	29

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.74	■ 1.96	▲ 3.04
Shear Stress @ End of Test (ksf)	○ 0.70	□ 1.94	△ 2.93
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.8	14.4	14.0
Initial Dry Density (pcf)	105.0	104.5	104.8
Initial Degree of Saturation (%)	61.6	63.6	62.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.5	21.3	20.8

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

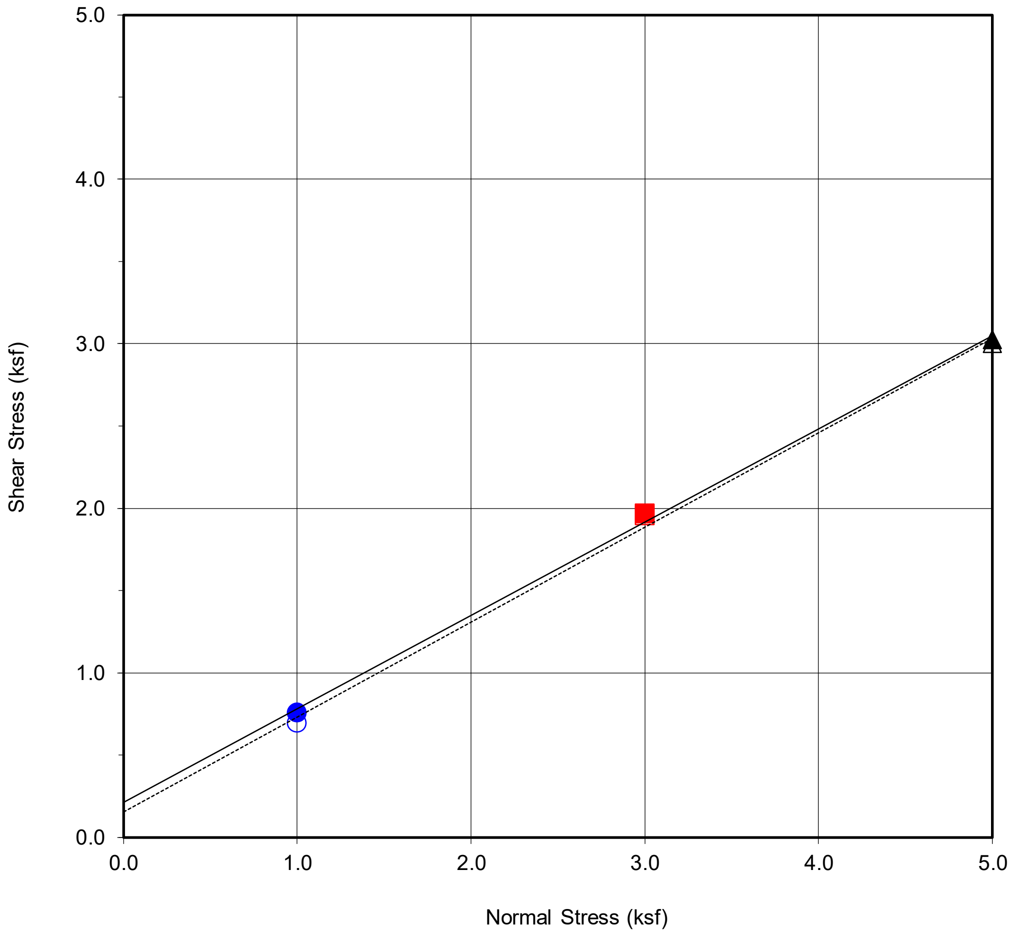
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B1



Boring No.	**B9**
Sample No.	**B9@0-5'**
Depth (ft)	**0-5'**
Sample Type:	REMOLD

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	215	30
Ultimate	156	30

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.76	■ 1.97	▲ 3.02
Shear Stress @ End of Test (ksf)	○ 0.70	□ 1.96	△ 3.00
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.6	13.6	14.6
Initial Dry Density (pcf)	102.1	102.3	101.3
Initial Degree of Saturation (%)	56.6	56.7	59.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	20.6	20.4	20.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

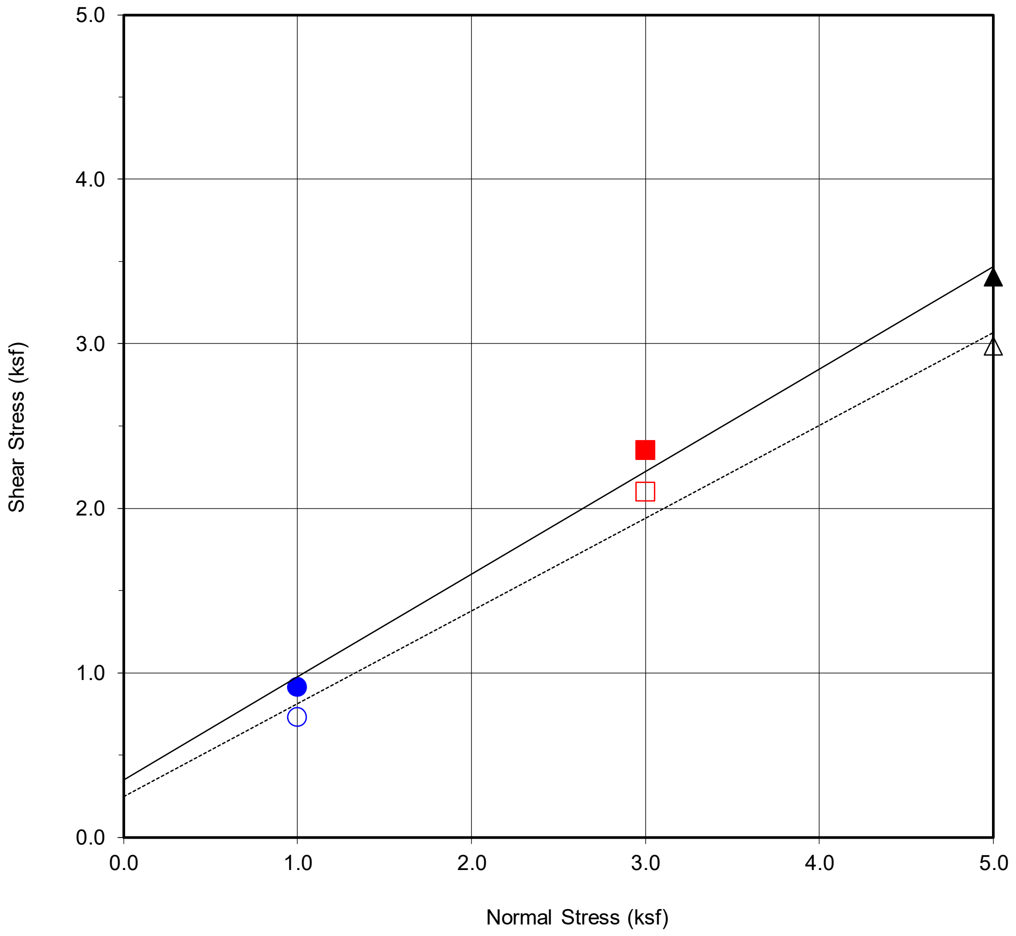
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B2



Boring No.	**B8**
Sample No.	**B8@2.5'**
Depth (ft)	**2.5'**
Sample Type:	RING

Soil Identification:

Silty Sand (SM)

Strength Parameters

	C (psf)	ϕ (°)
Peak	352	32
Ultimate	248	29

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.91	■ 2.35	▲ 3.41
Shear Stress @ End of Test (ksf)	○ 0.73	□ 2.10	△ 2.99
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	12.8	10.8	17.4
Initial Dry Density (pcf)	89.0	88.8	85.3
Initial Degree of Saturation (%)	38.5	32.4	48.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	26.6	25.5	24.9

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

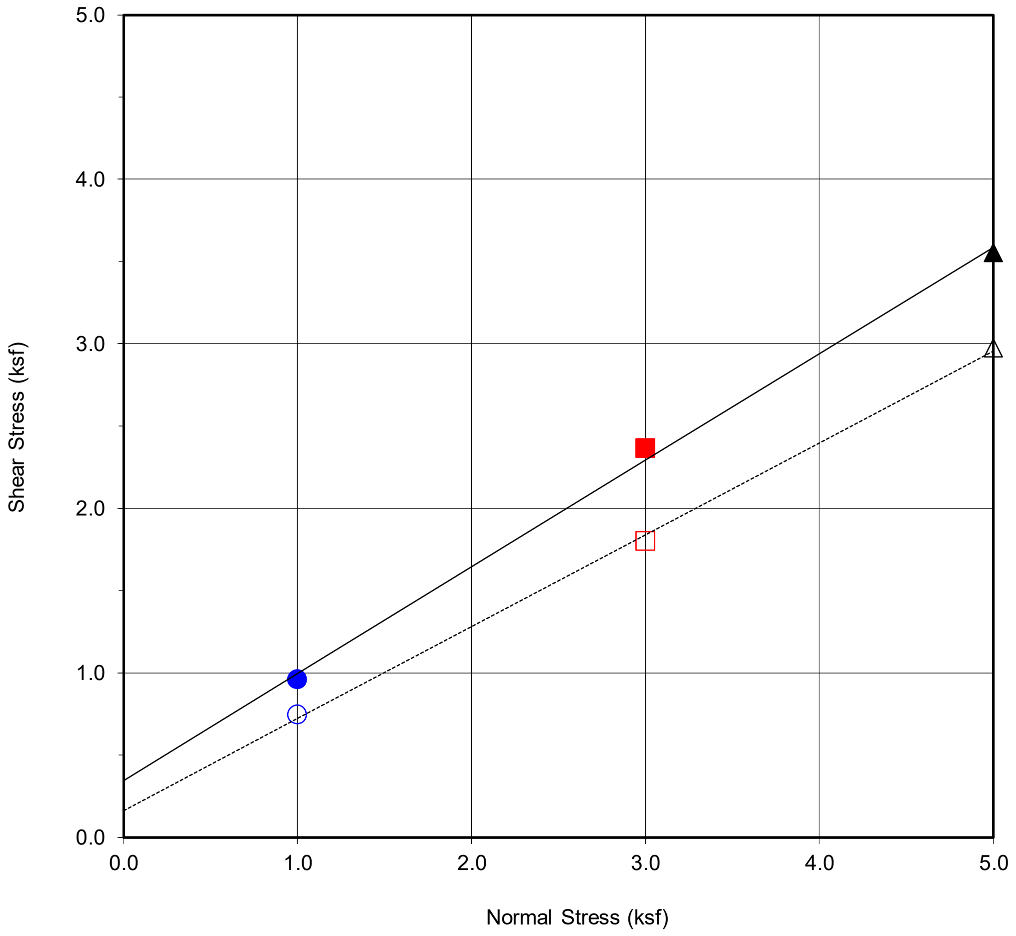
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B3



Boring No.	**B2**
Sample No.	**B2@5'**
Depth (ft)	**5'**
Sample Type:	RING

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	348	33
Ultimate	166	29

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.96	■ 2.36	▲ 3.55
Shear Stress @ End of Test (ksf)	○ 0.74	□ 1.80	△ 2.98
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	31.6	34.4	33.2
Initial Dry Density (pcf)	95.2	93.3	93.6
Initial Degree of Saturation (%)	110.6	115.1	112.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	27.0	26.7	26.1

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

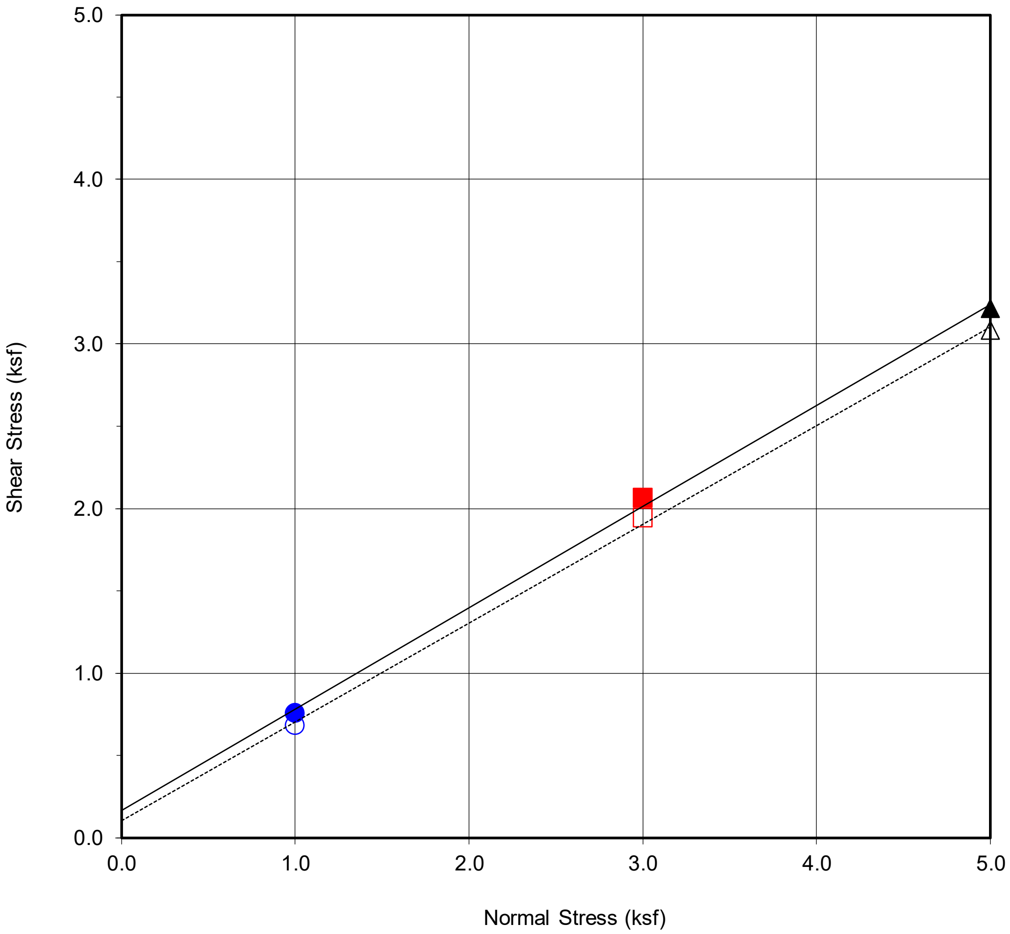
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B4



Boring No.	**B4**
Sample No.	**B4@5'**
Depth (ft)	**5'**
Sample Type:	RING

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	167	32
Ultimate	104	31

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.76	■ 2.06	▲ 3.22
Shear Stress @ End of Test (ksf)	○ 0.68	□ 1.94	△ 3.08
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	37.9	41.1	47.0
Initial Dry Density (pcf)	87.7	85.5	77.2
Initial Degree of Saturation (%)	111.0	114.4	107.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	33.4	34.4	40.1

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

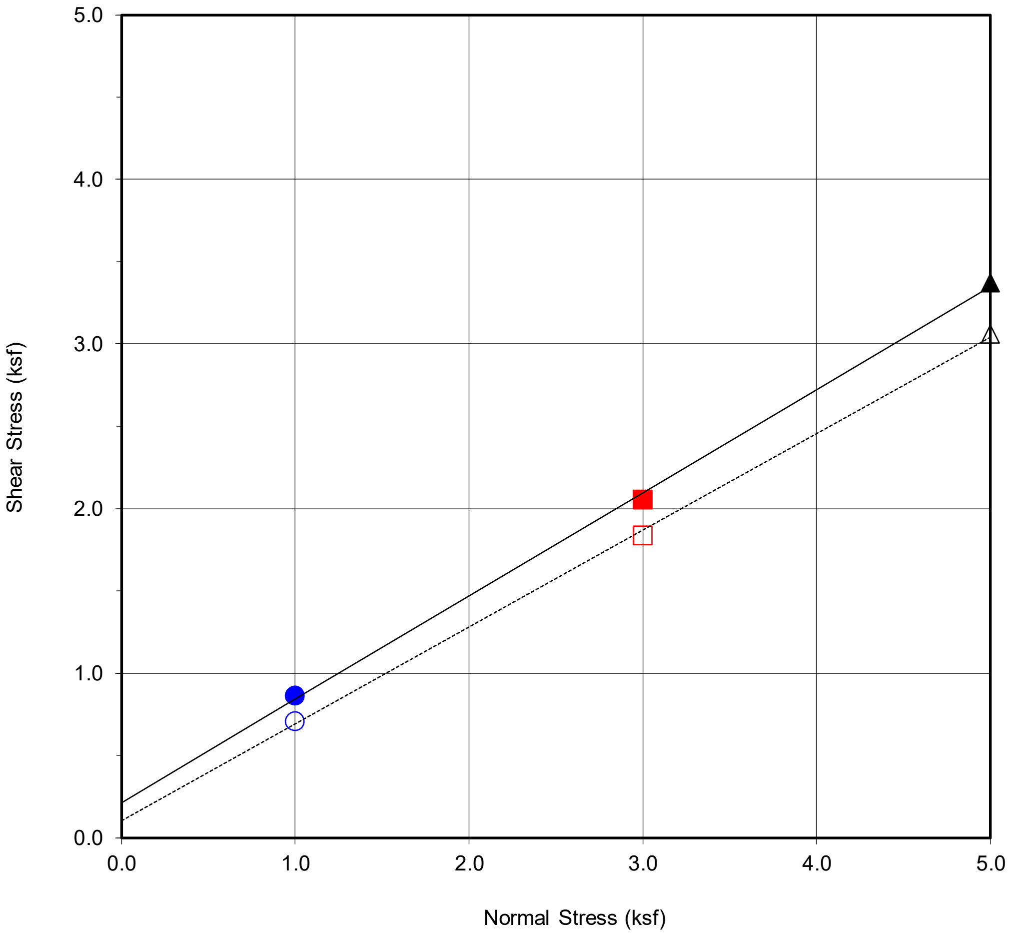
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B5



Boring No.	**B5**
Sample No.	**B5@5'**
Depth (ft)	**5'**
Sample Type:	RING

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	215	32
Ultimate	104	30

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.86	■ 2.05	▲ 3.37
Shear Stress @ End of Test (ksf)	○ 0.71	□ 1.84	△ 3.06
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	17.5	15.1	13.5
Initial Dry Density (pcf)	100.6	99.2	99.6
Initial Degree of Saturation (%)	69.8	58.3	52.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	24.3	22.4	23.8

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: PZ

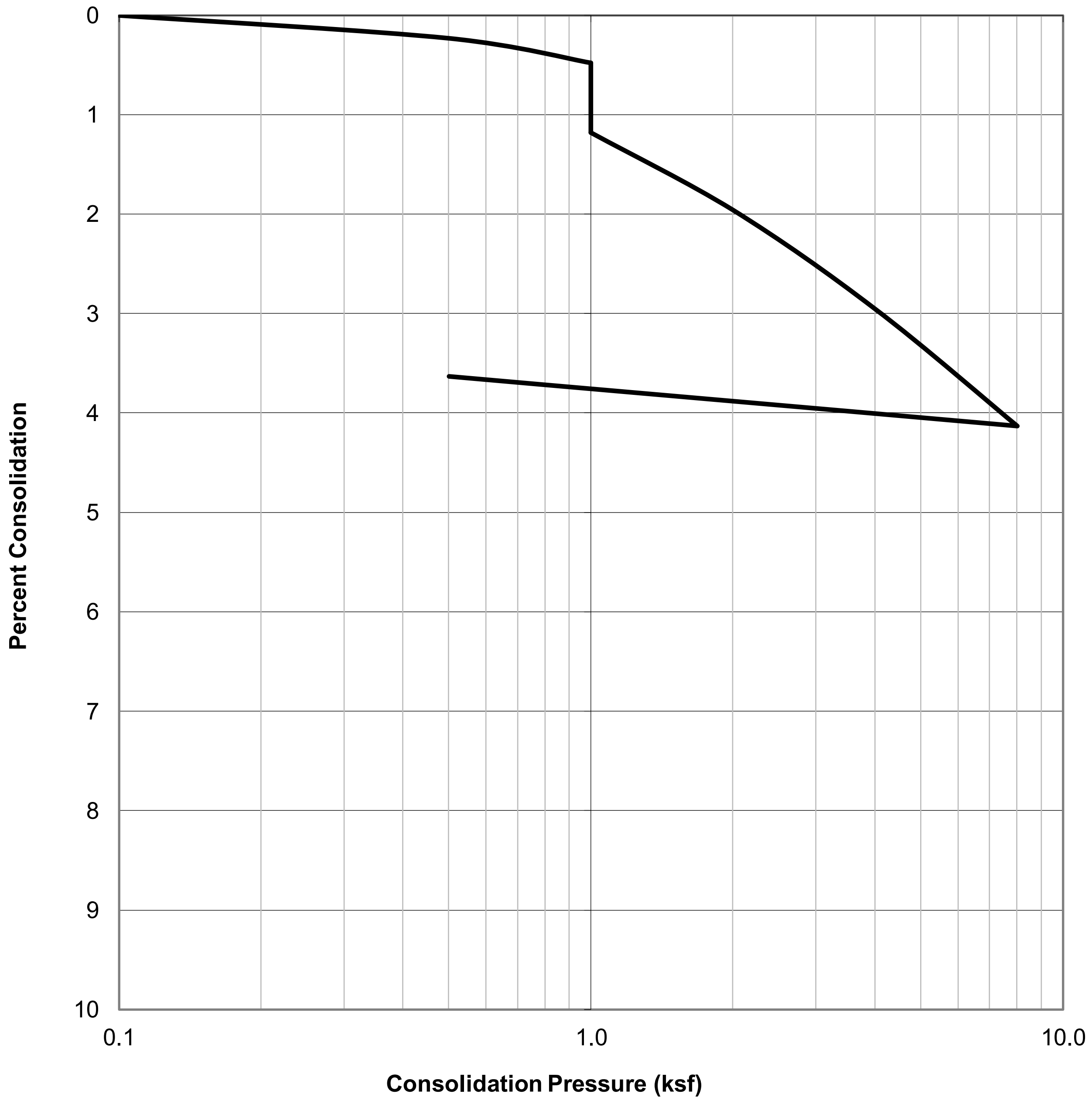
Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B6

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B12@2.5'	Silty Sand (SM)	93.0	8.1	26.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

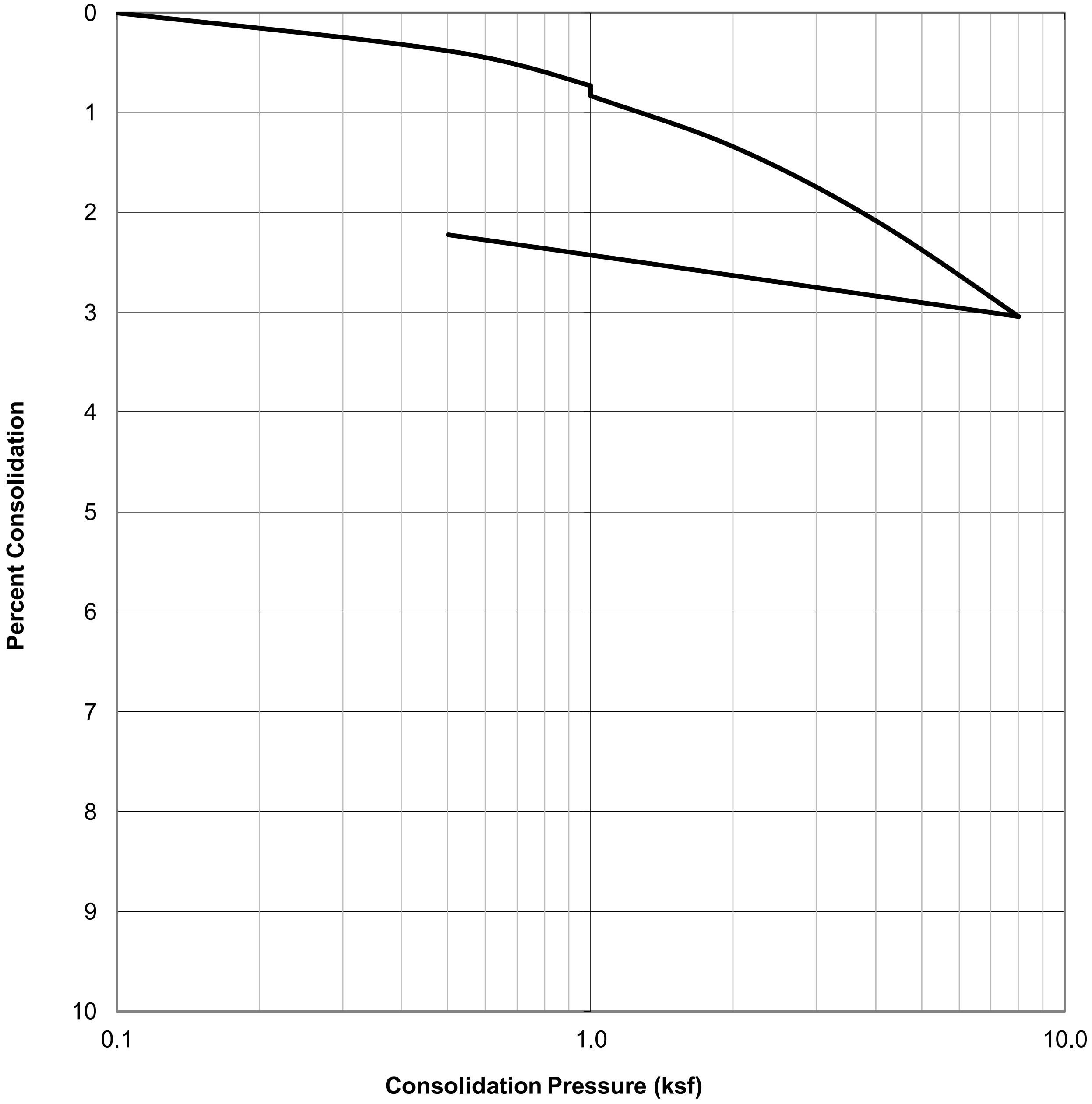
Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B7

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@3'	Silty Sand (SM)	99.4	14.8	23.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

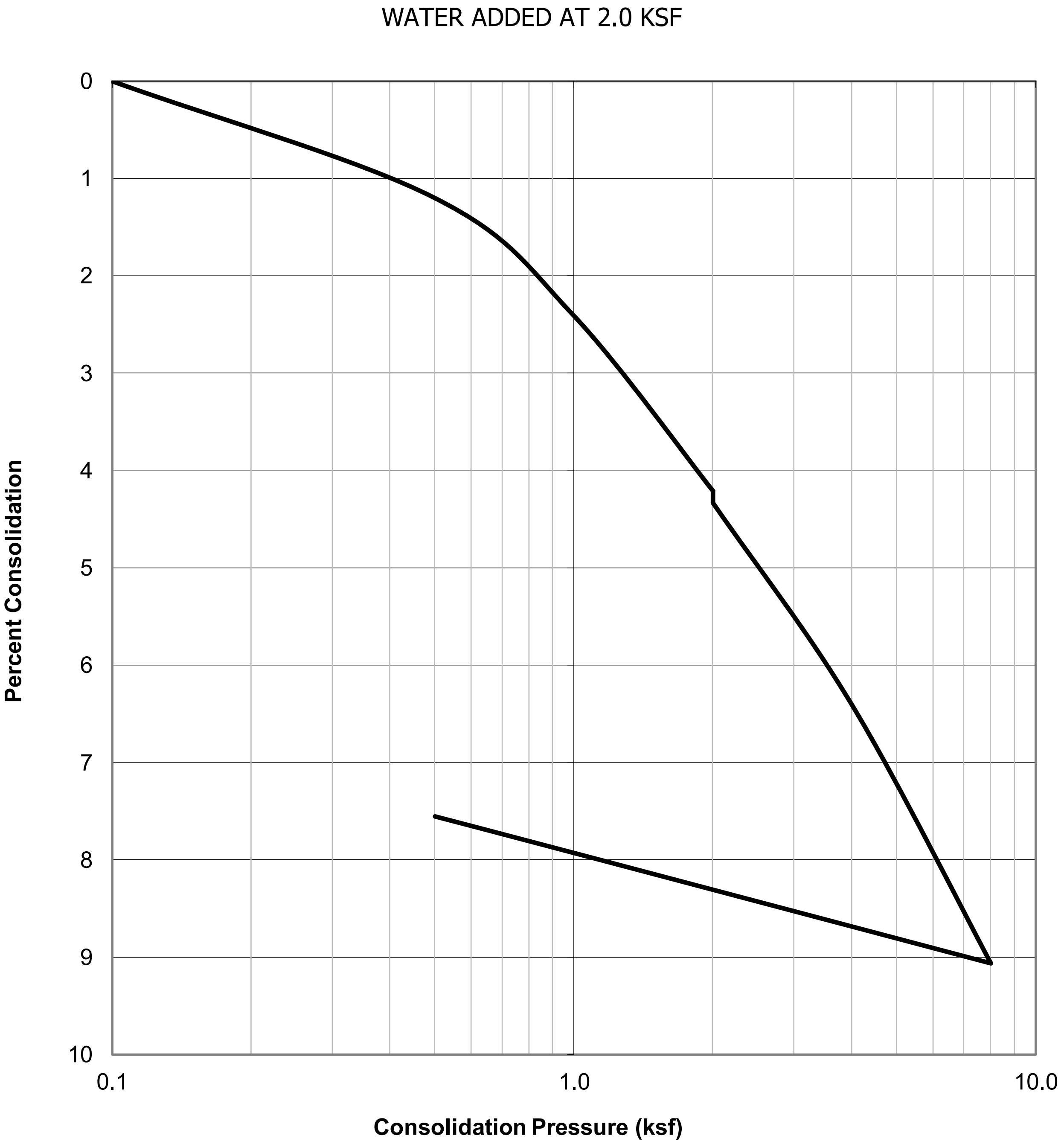
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B8



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@5'	Silty Sand (SM)	86.2	36.7	31.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

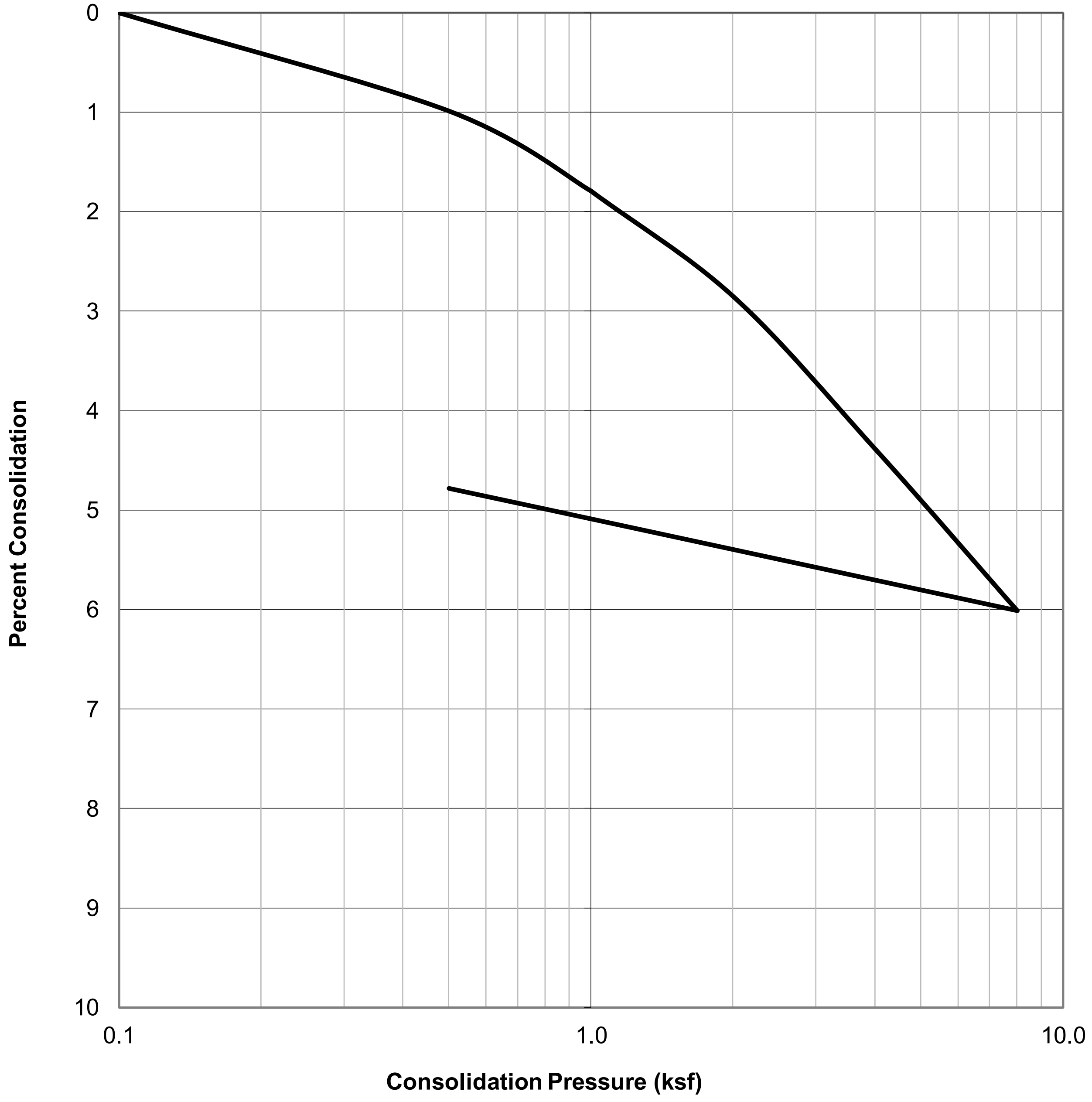
Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B9

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@5'	Silty Sand (SM)	88.5	36.5	33.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

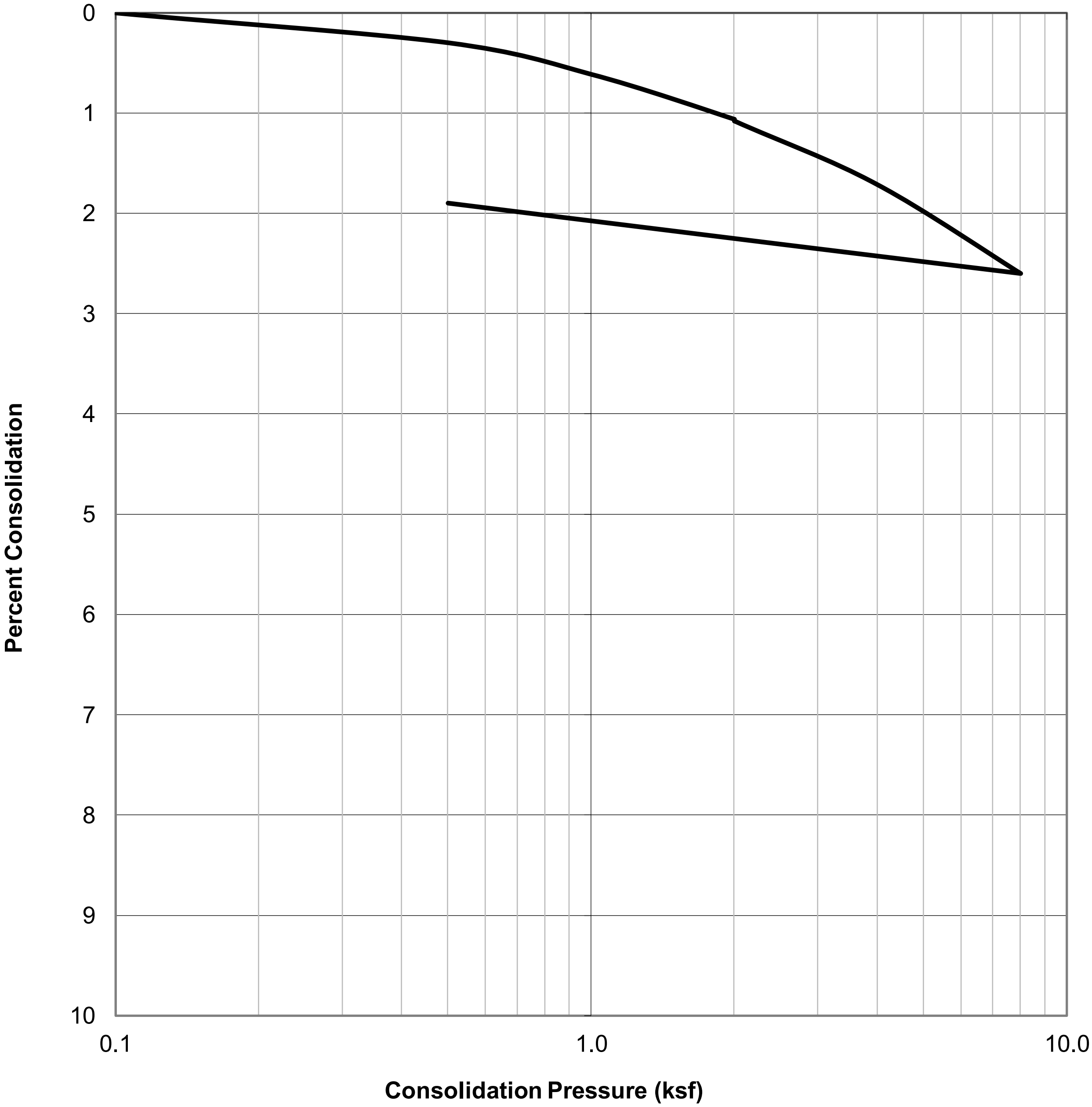
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@5'	Silty Sand (SM)	103.9	16.9	21.9

GEOCON

CONSOLIDATION TEST RESULTS

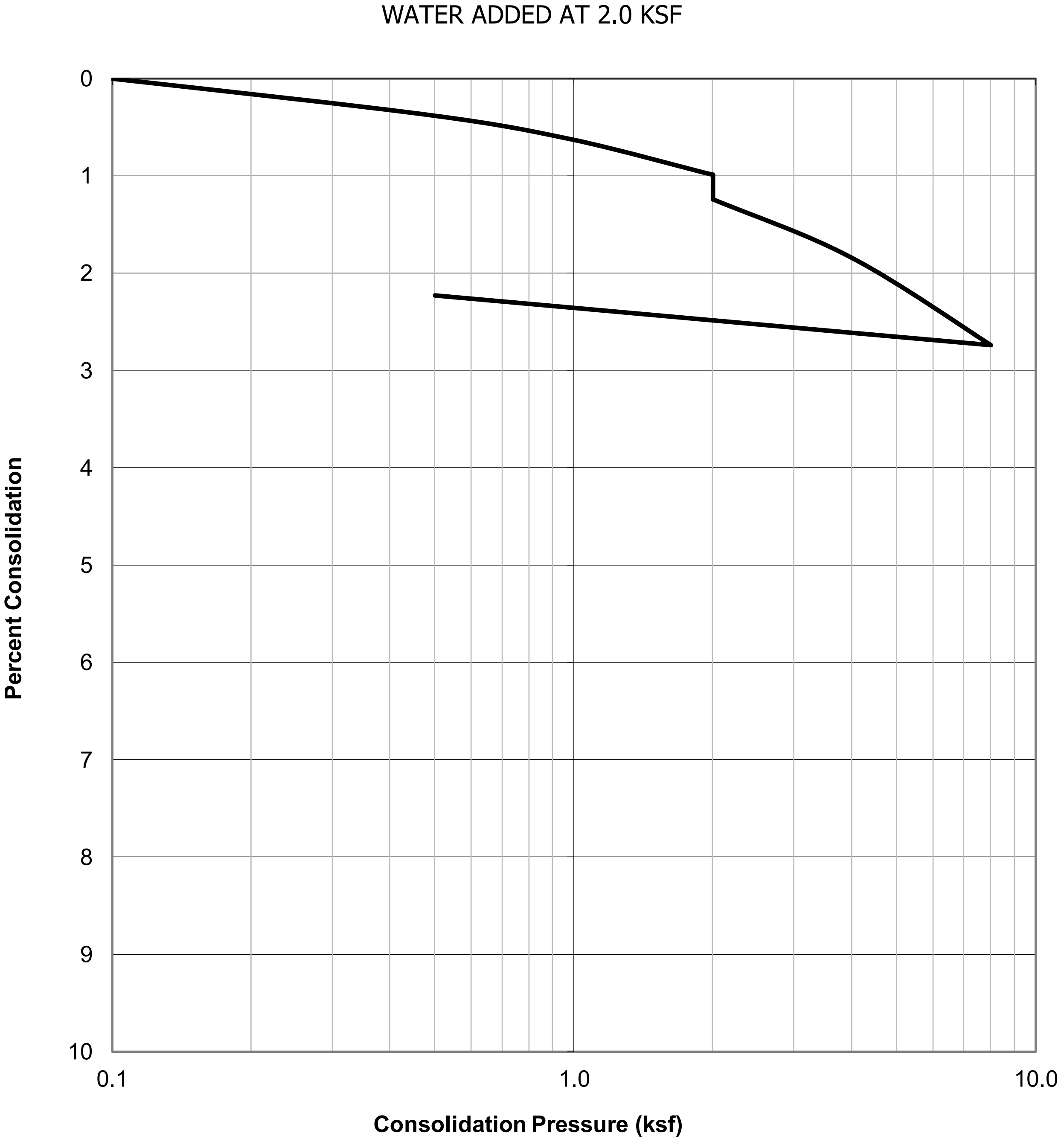
ASTM D-2435

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@5'	Silty Sand (SM)	95.0	17.1	25.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

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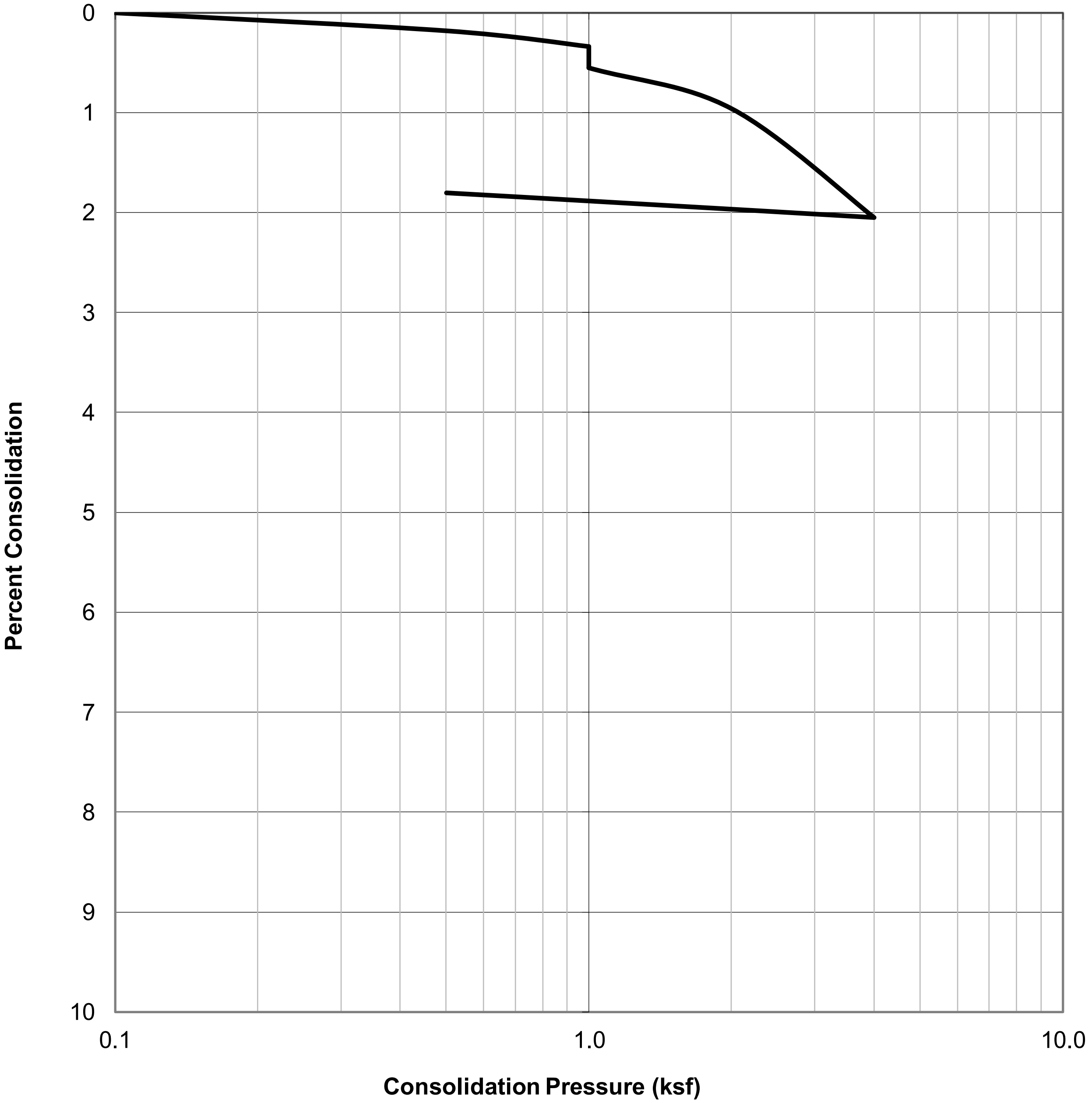
Figure B12

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B11@5'	Poorly Graded Sand (SP)	95.8	23.2	25.1

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

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16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
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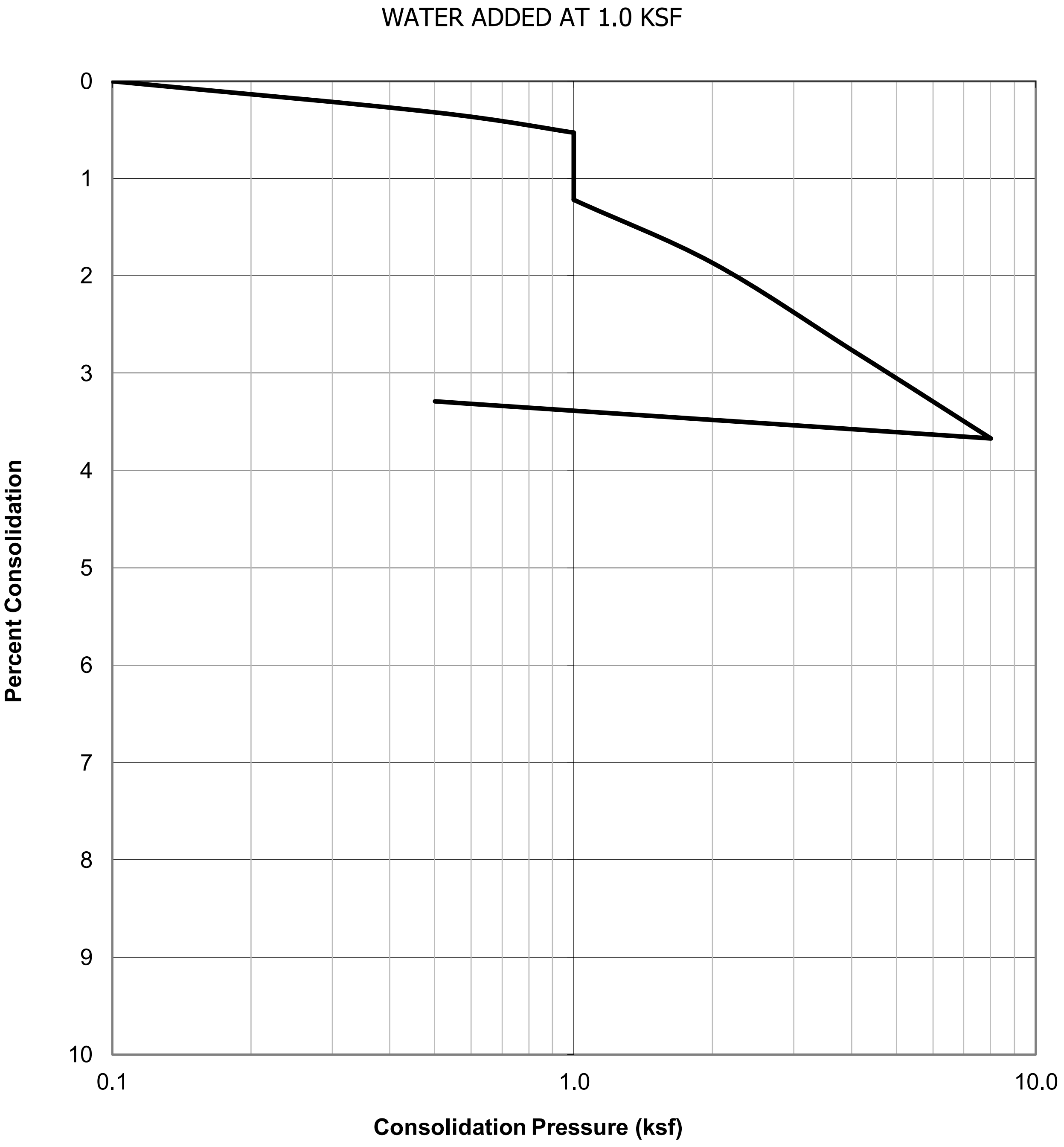


SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3@7'	Silty Sand (SM)	98.1	32.9	26.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

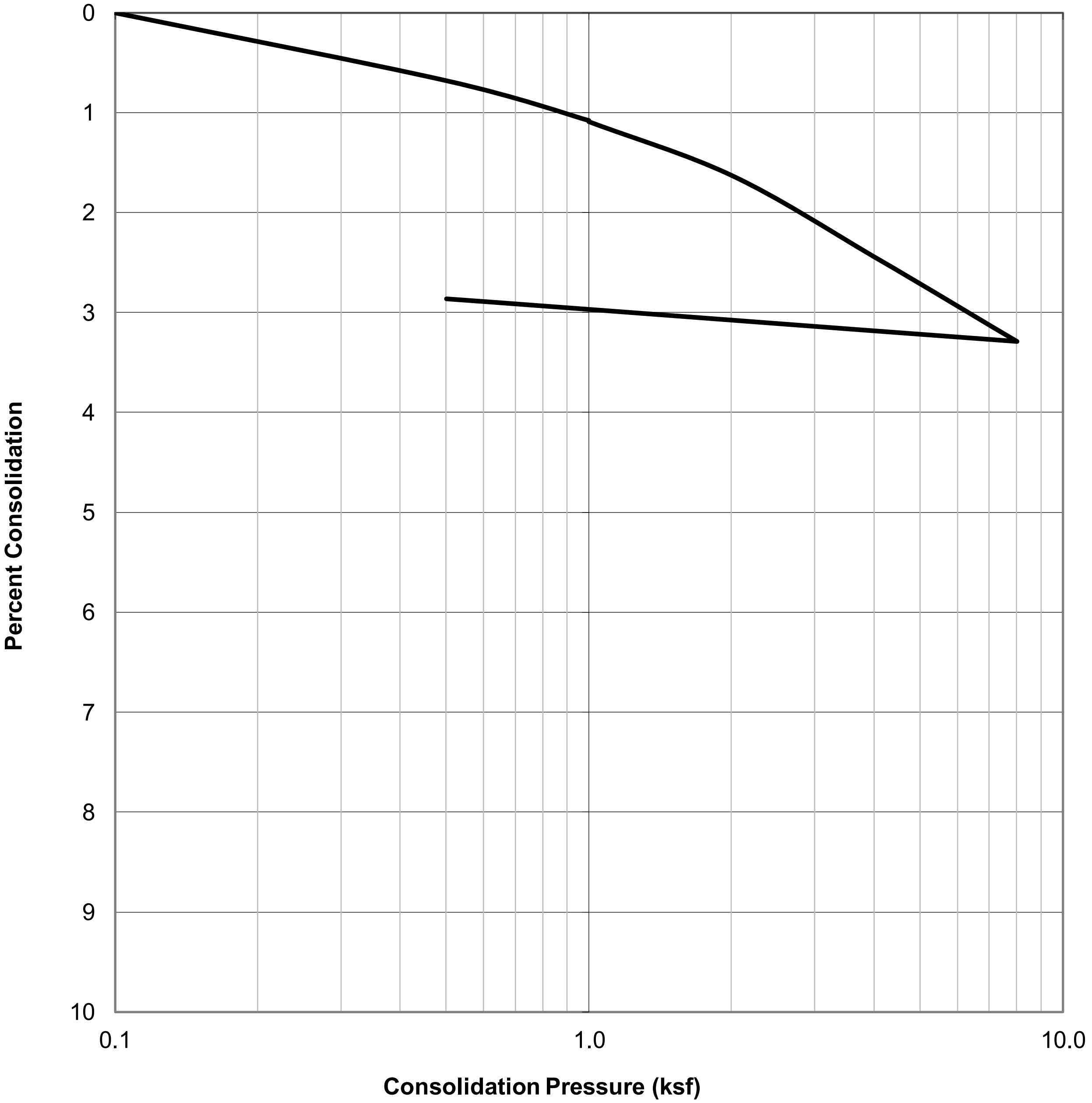
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@7.5'	Silty Sand (SM)	110.9	21.6	19.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

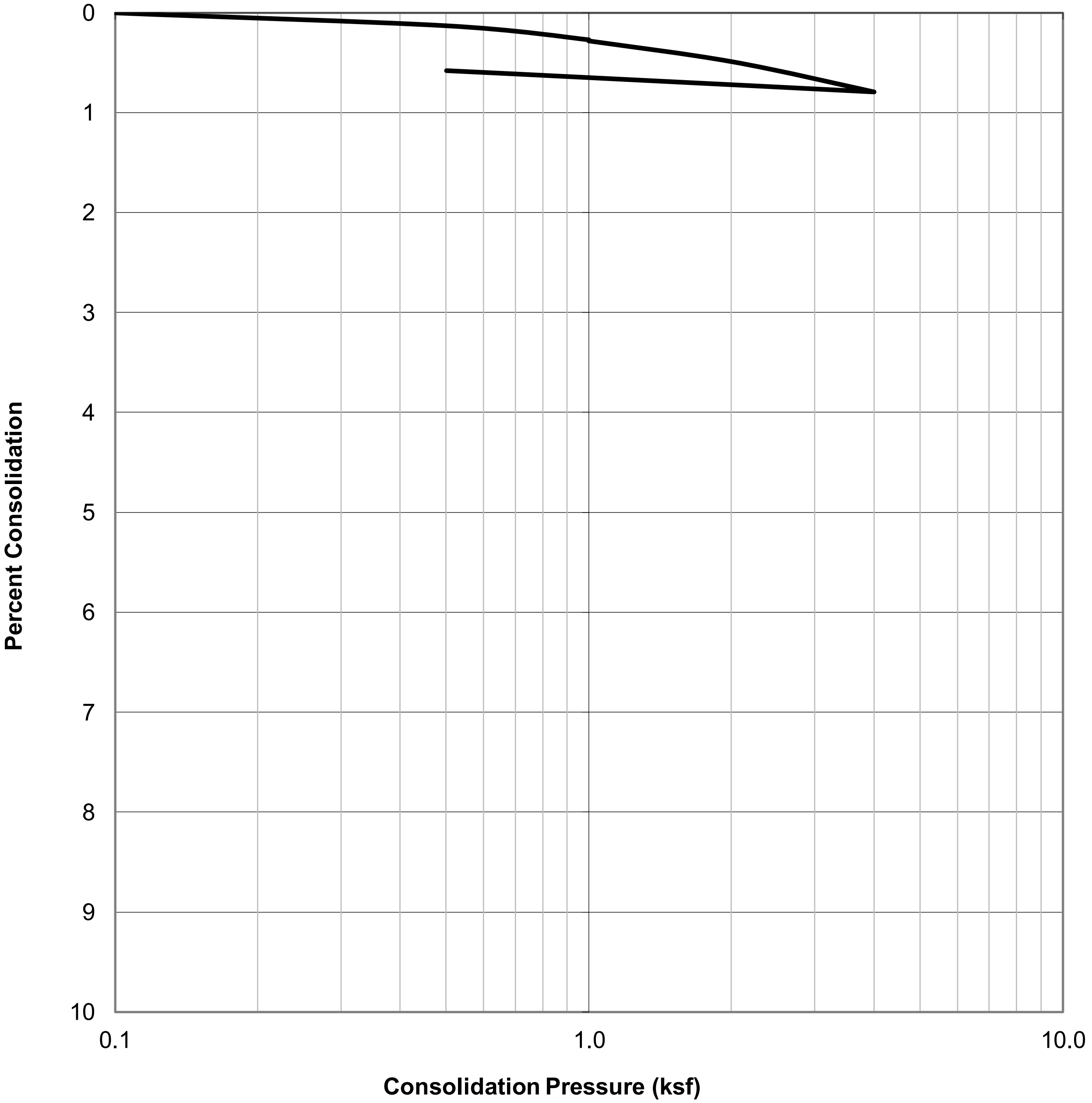
Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B15

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7@7.5'	Sandy Silt (ML)	109.1	21.7	21.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

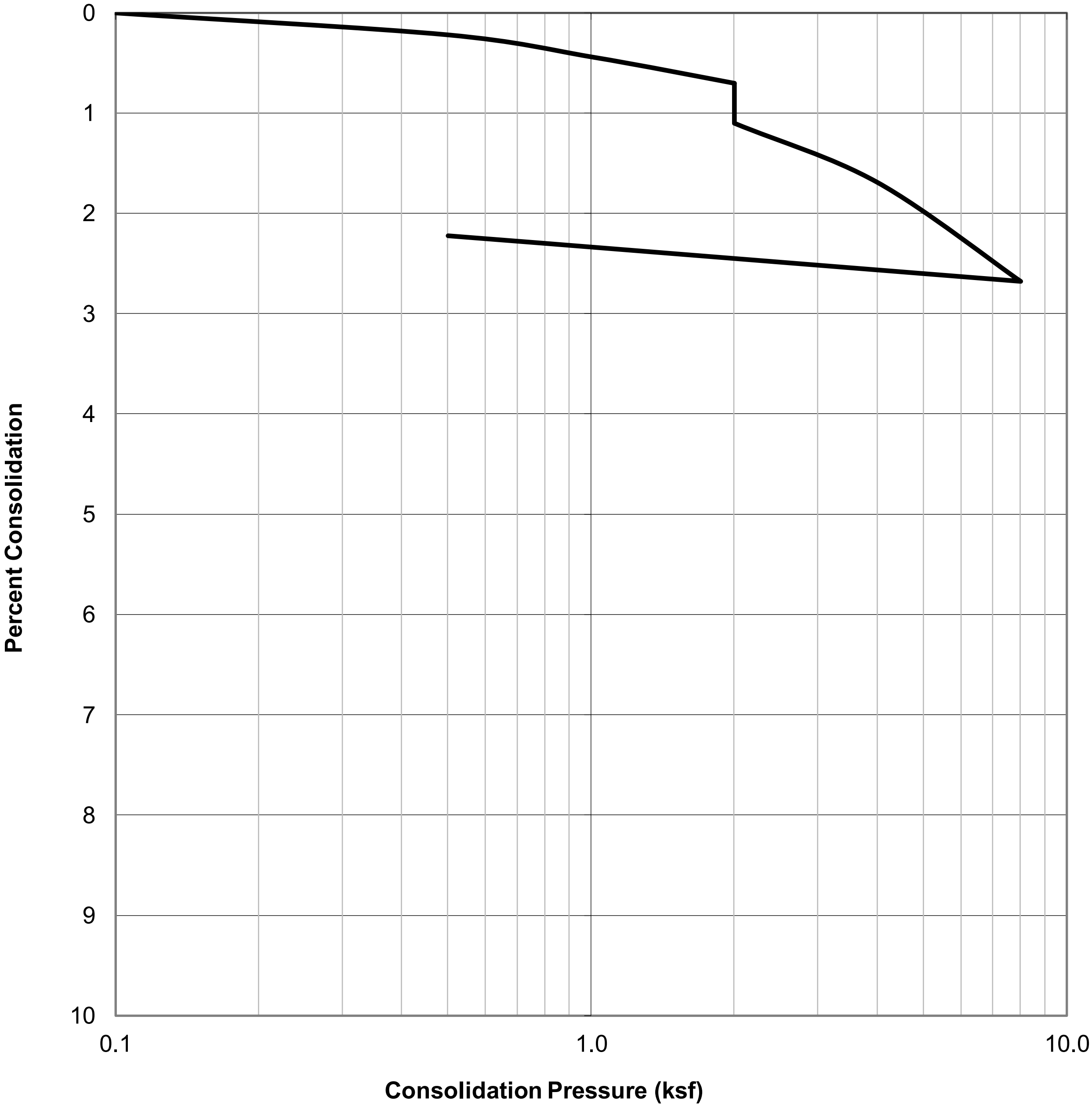
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@10'	Silty Sand (SM)	101.7	27.0	21.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

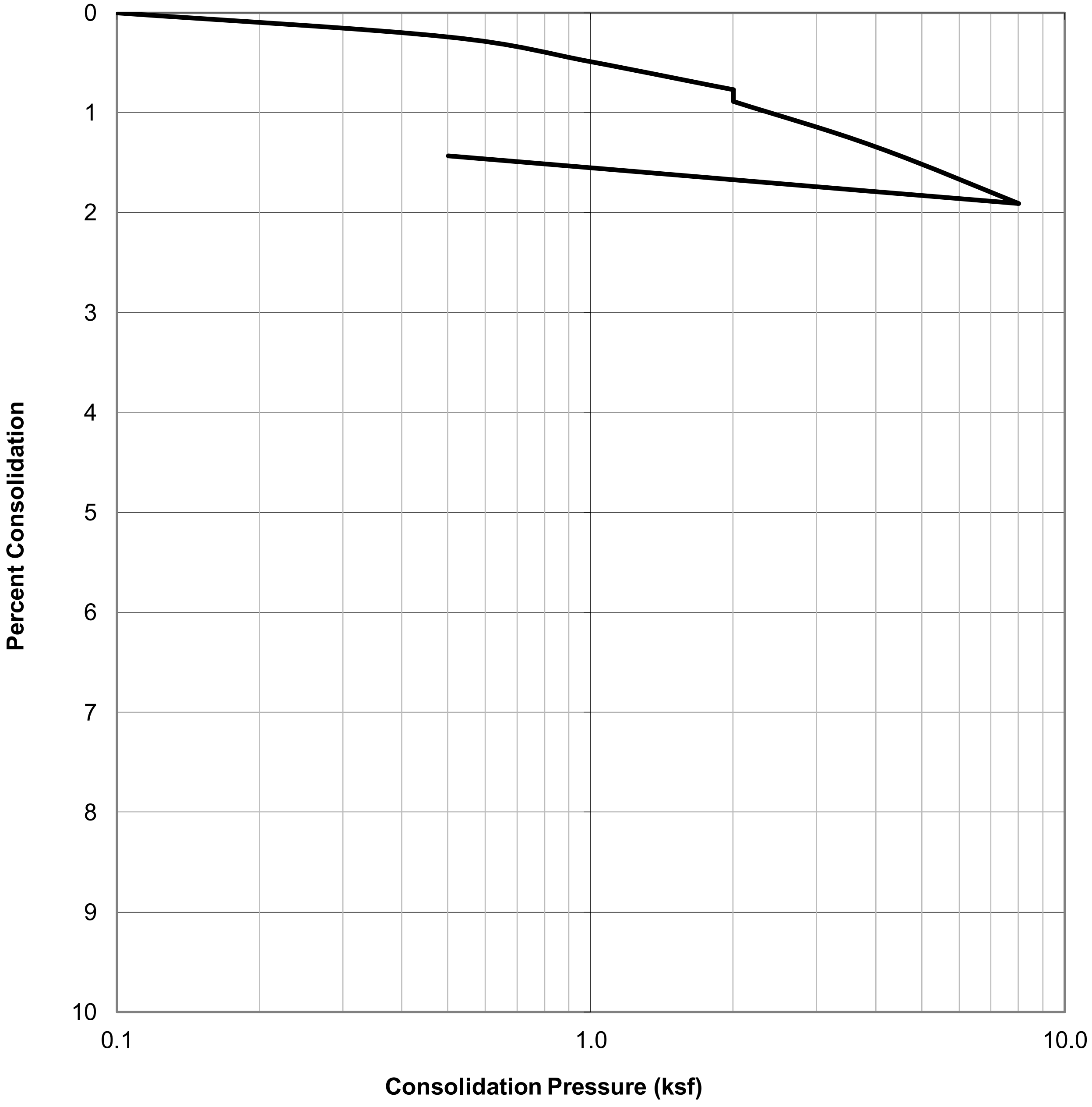
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@10'	Poorly Graded Sand with Silt (SP-SM)	103.9	25.7	24.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

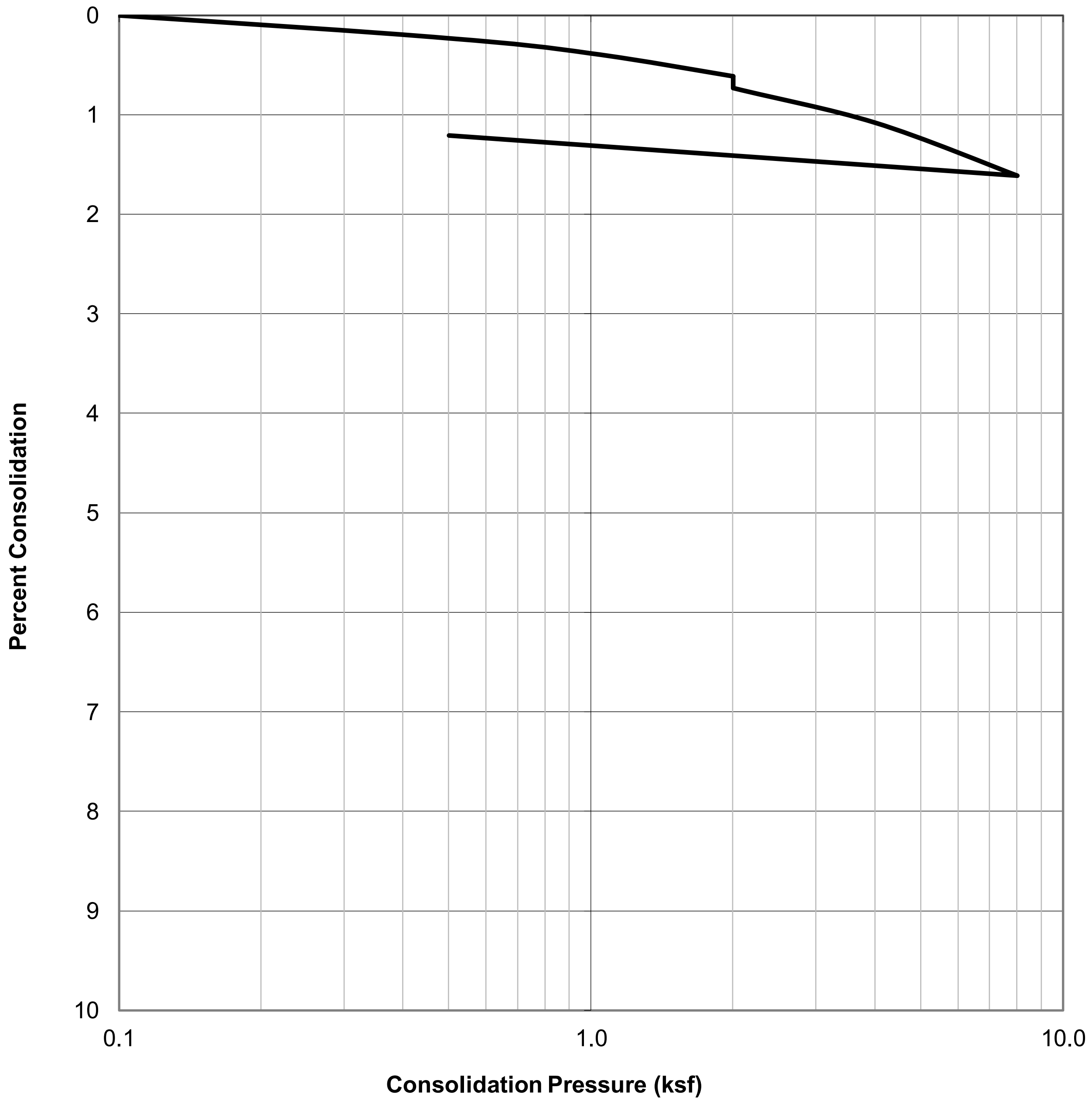
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@10'	Silty Sand (SM)	104.3	22.7	22.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

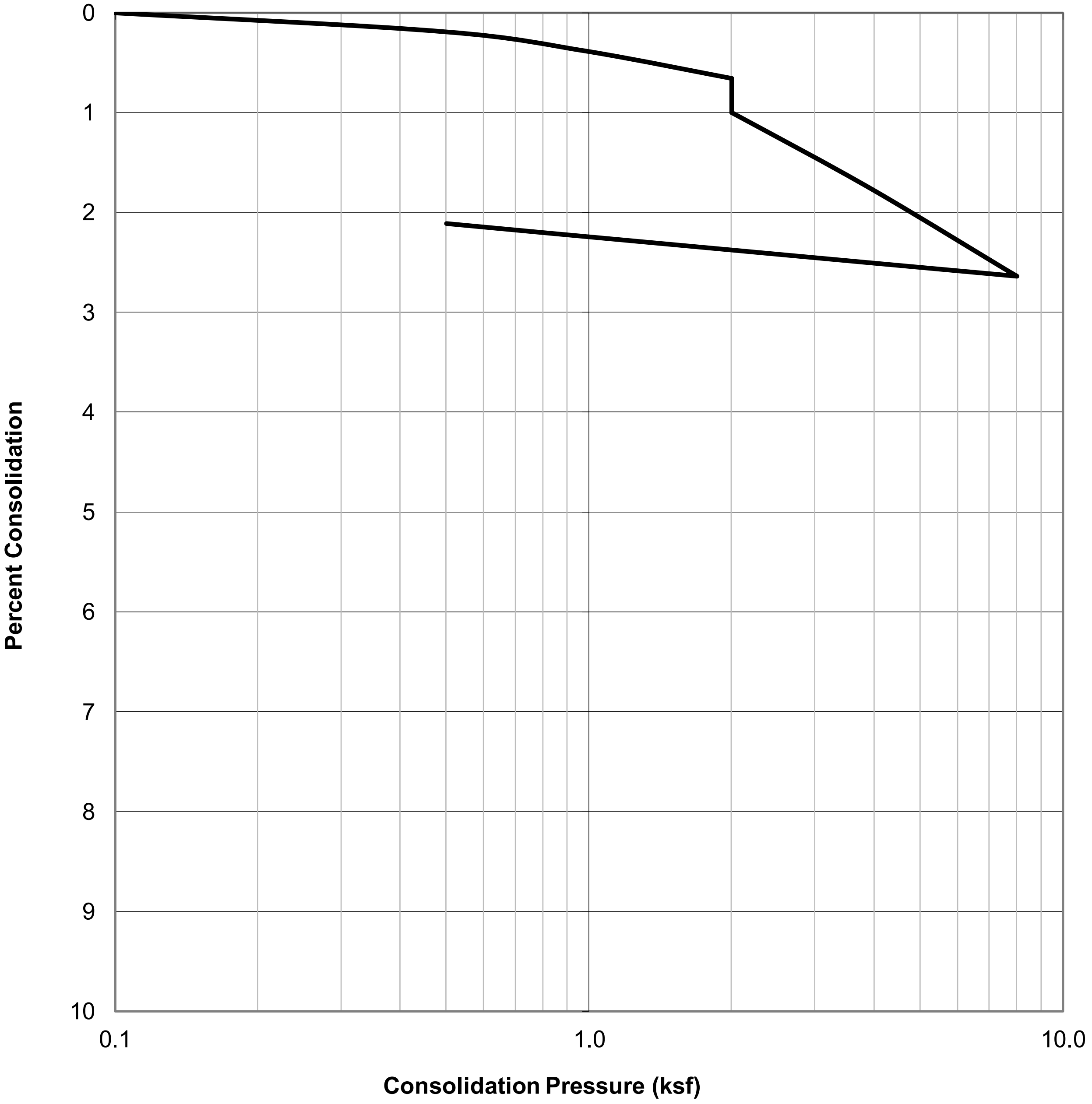
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@12.5'	Poorly Graded Sand (SP)	98.0	29.8	26.7

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

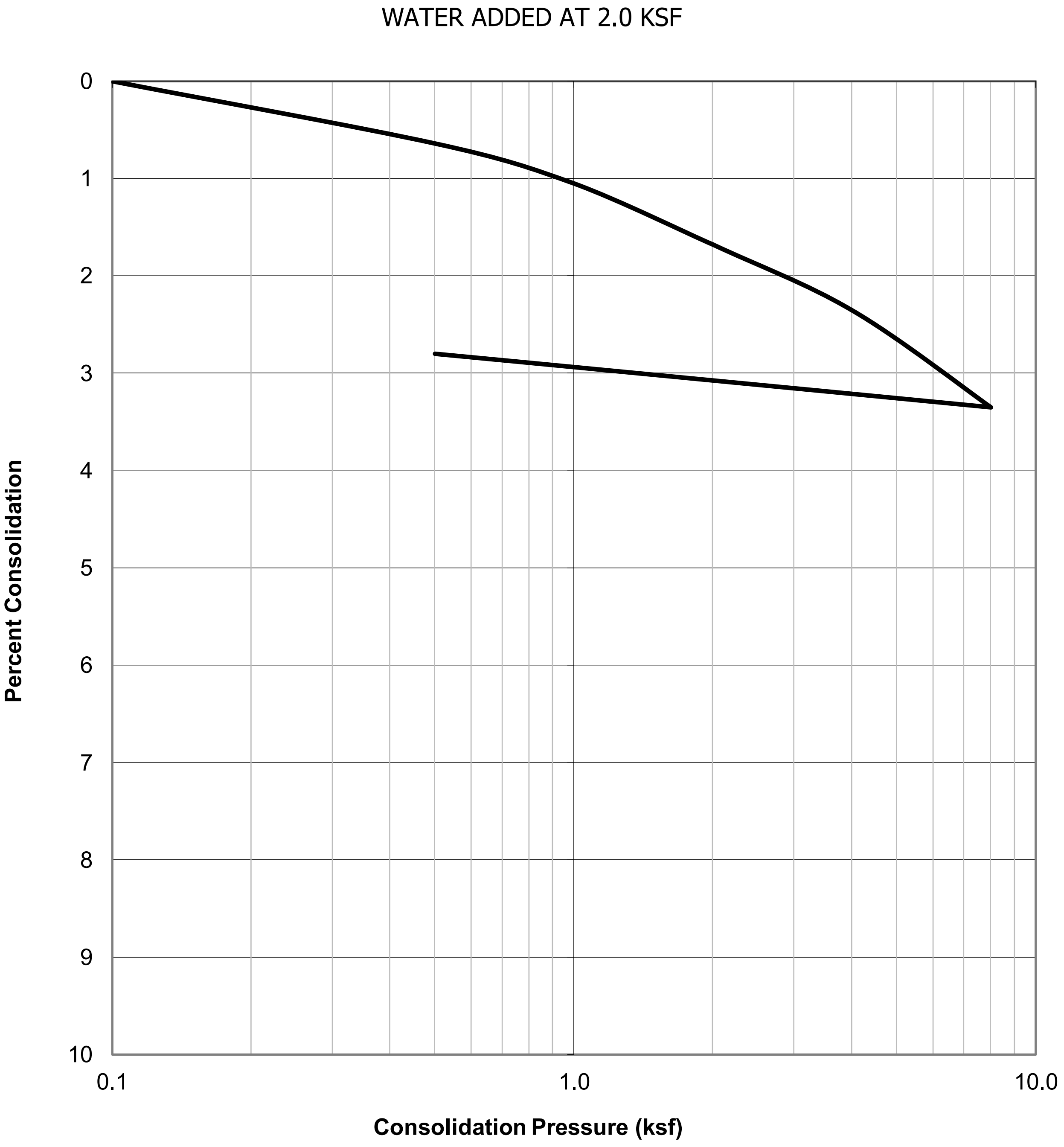
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B20



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@15'	Silty Sand (SM)	105.4	25.1	23.1

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

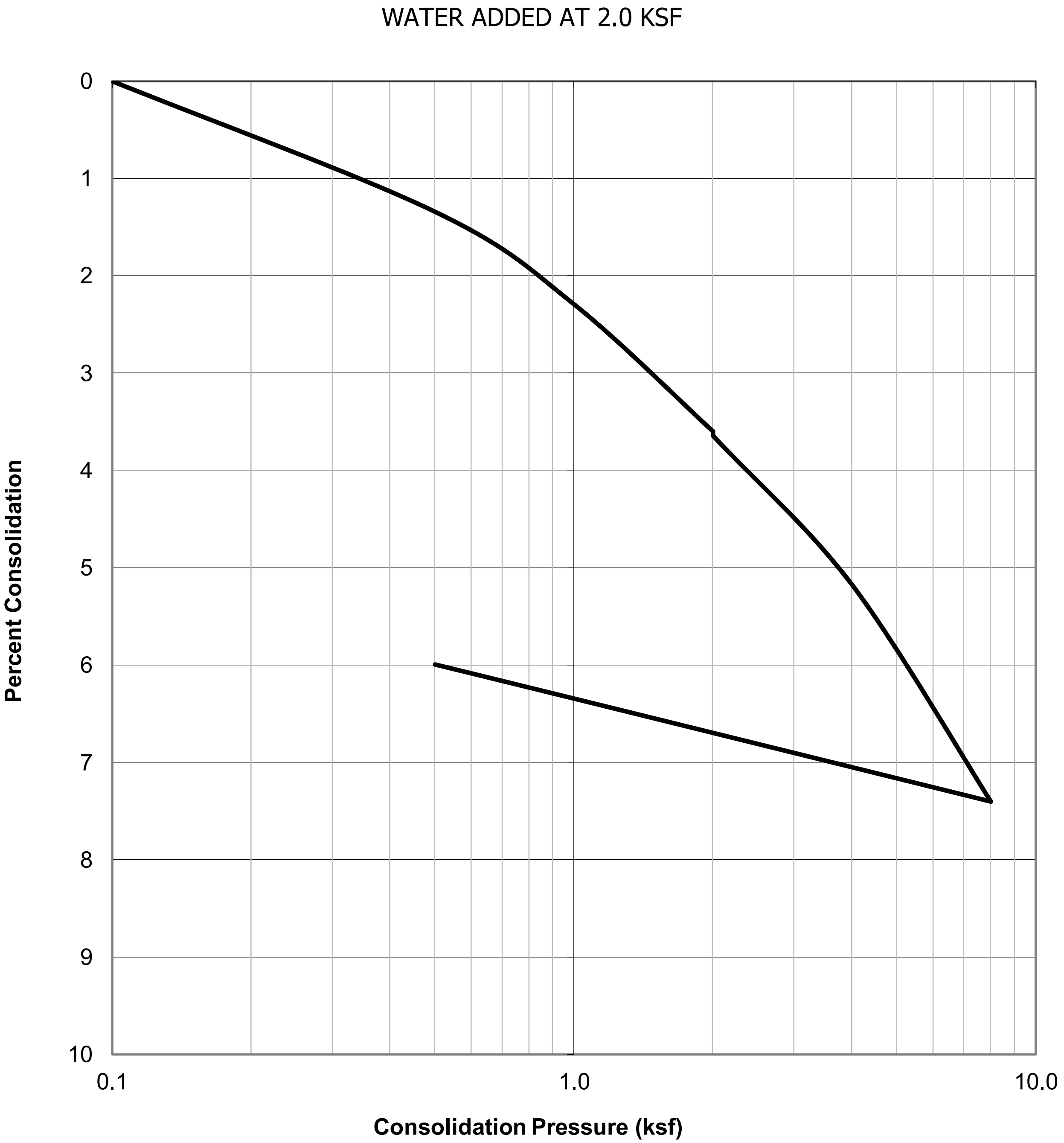
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B21



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@17.5'	Silt (ML)	90.9	34.6	30.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

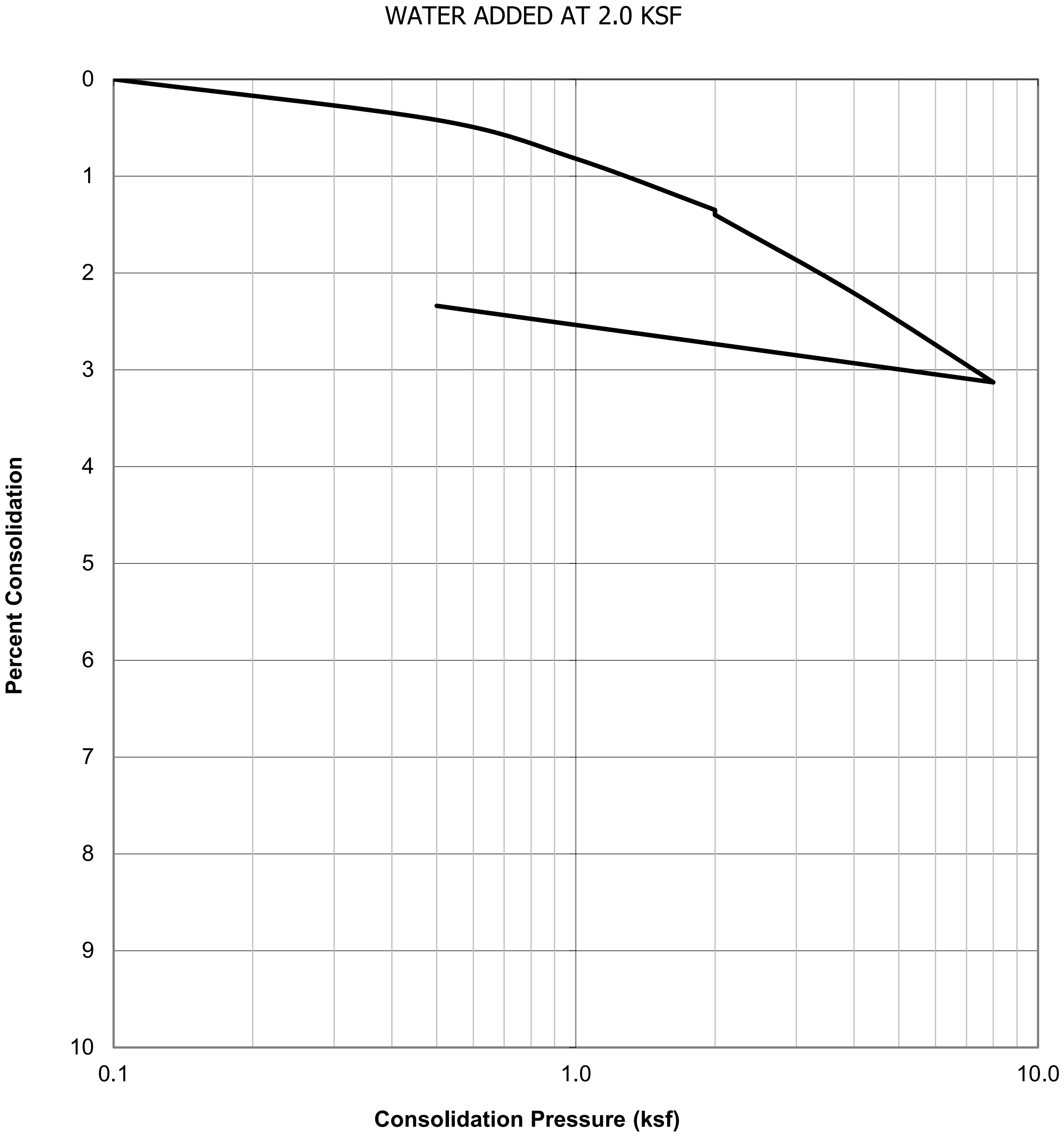
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B22



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@20'	Sandy Silt (ML)	96.8	28.9	28.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

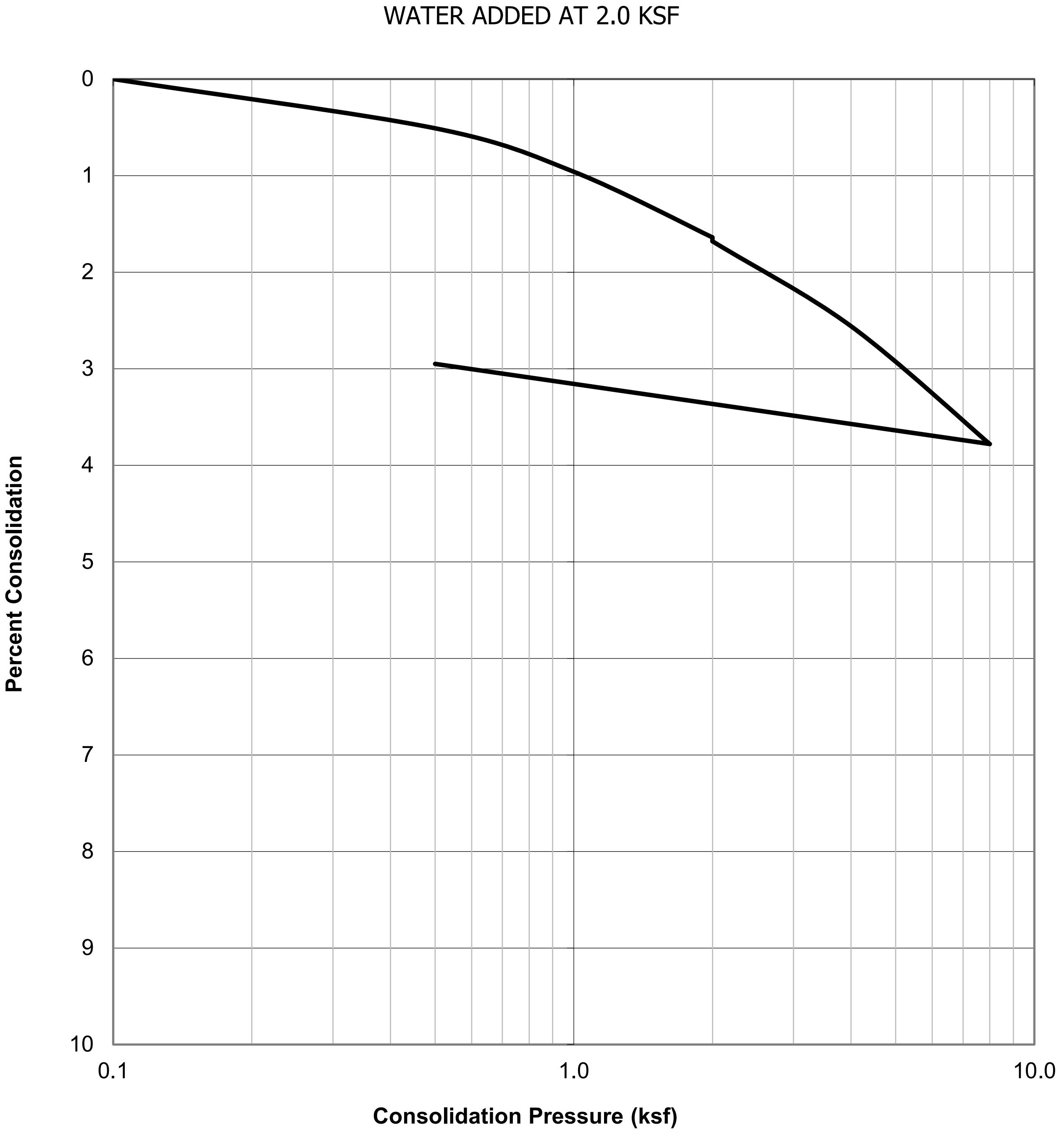
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B23



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@22.5'	Silty Sand (SM)	97.2	31.8	28.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

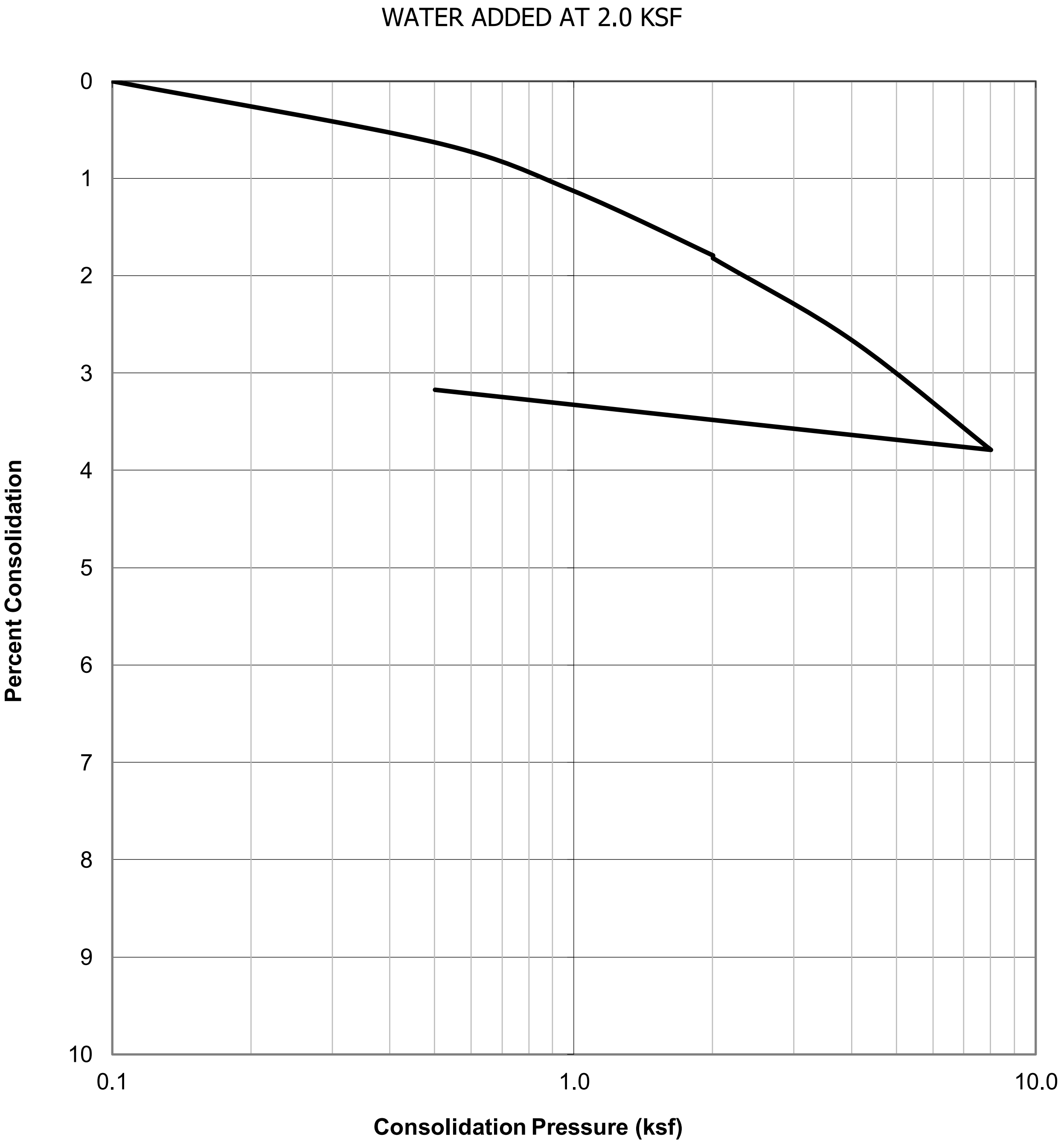
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B24



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@25'	Sandy Silt (ML)	98.0	29.5	25.6

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
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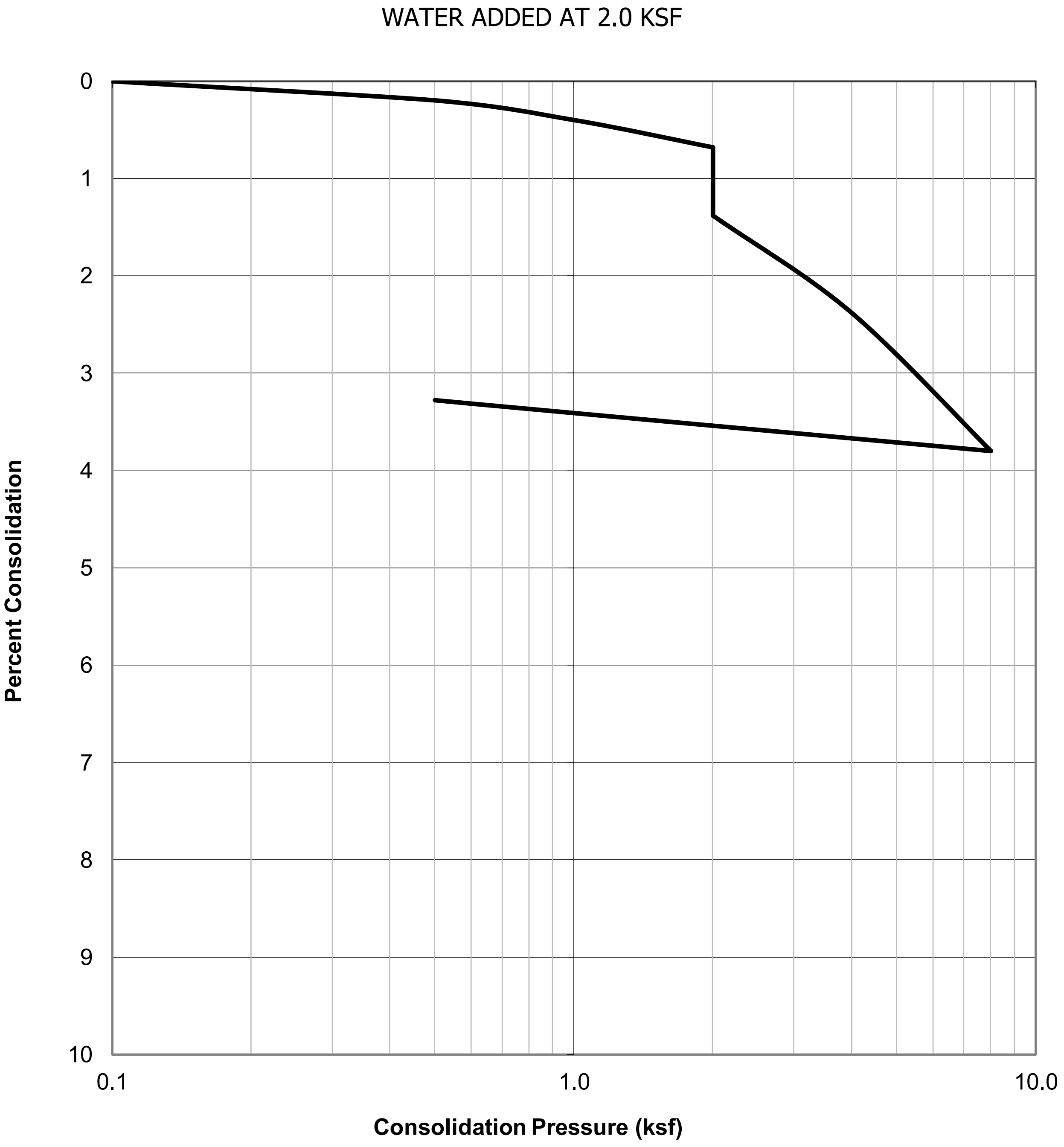
MAR. 2025

Figure B25



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@27.5'	Poorly Graded Sand with Silt (SP-SM)	105.2	26.7	23.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

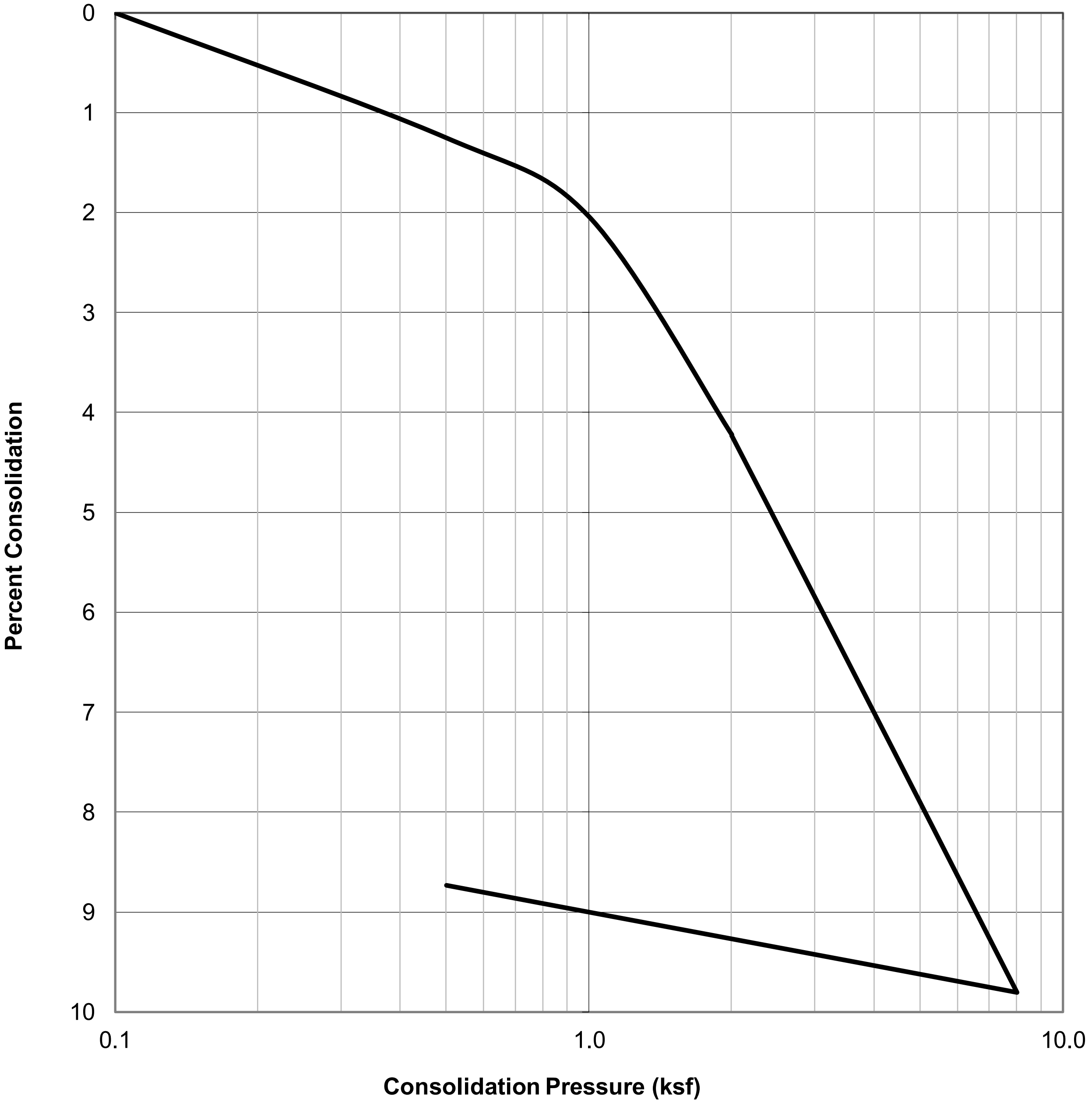
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@30'	Clay (CL), black	83.1	39.9	32.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

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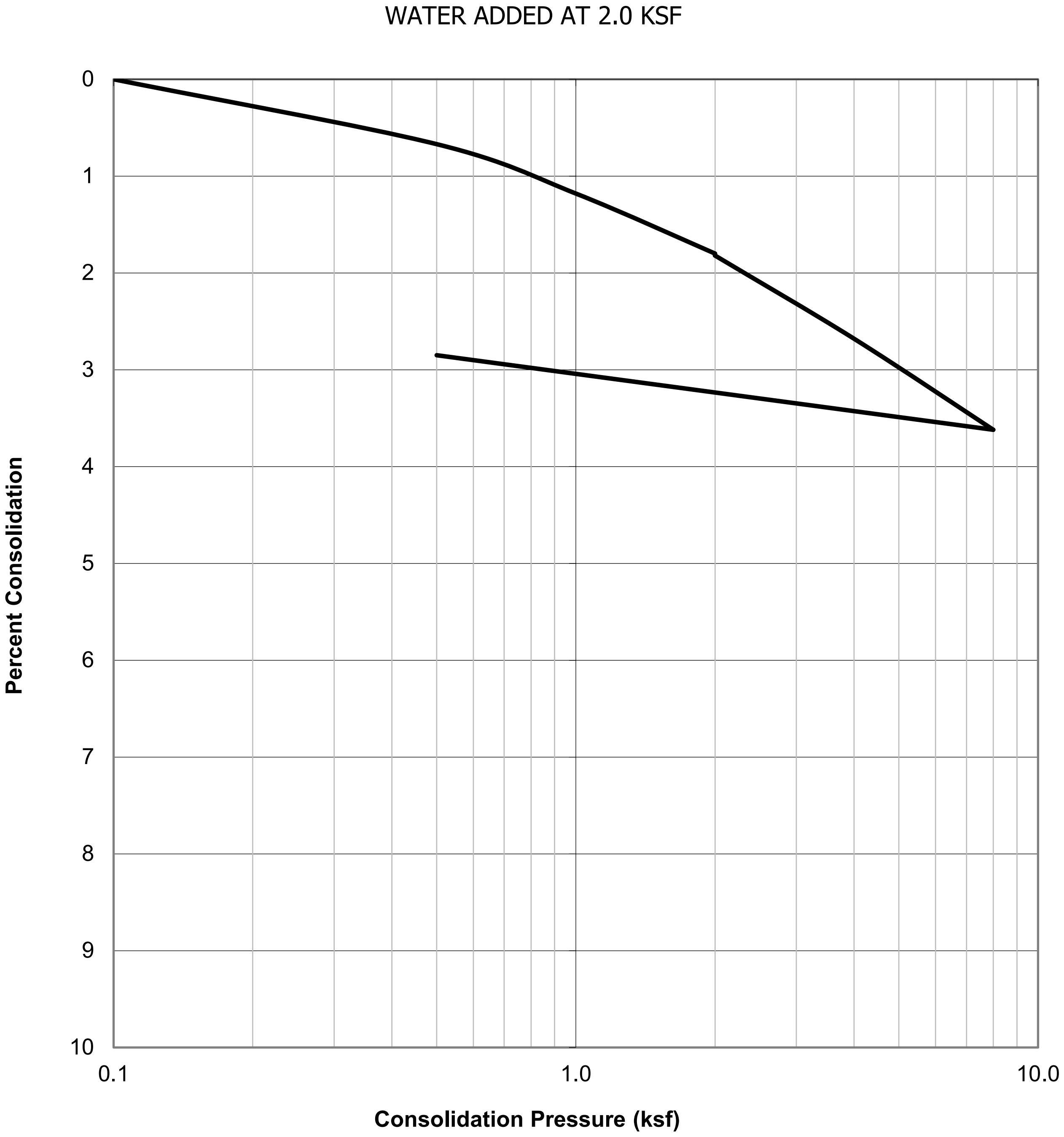


SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@32.5'	Silty Sand (SM)	95.5	30.5	30.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

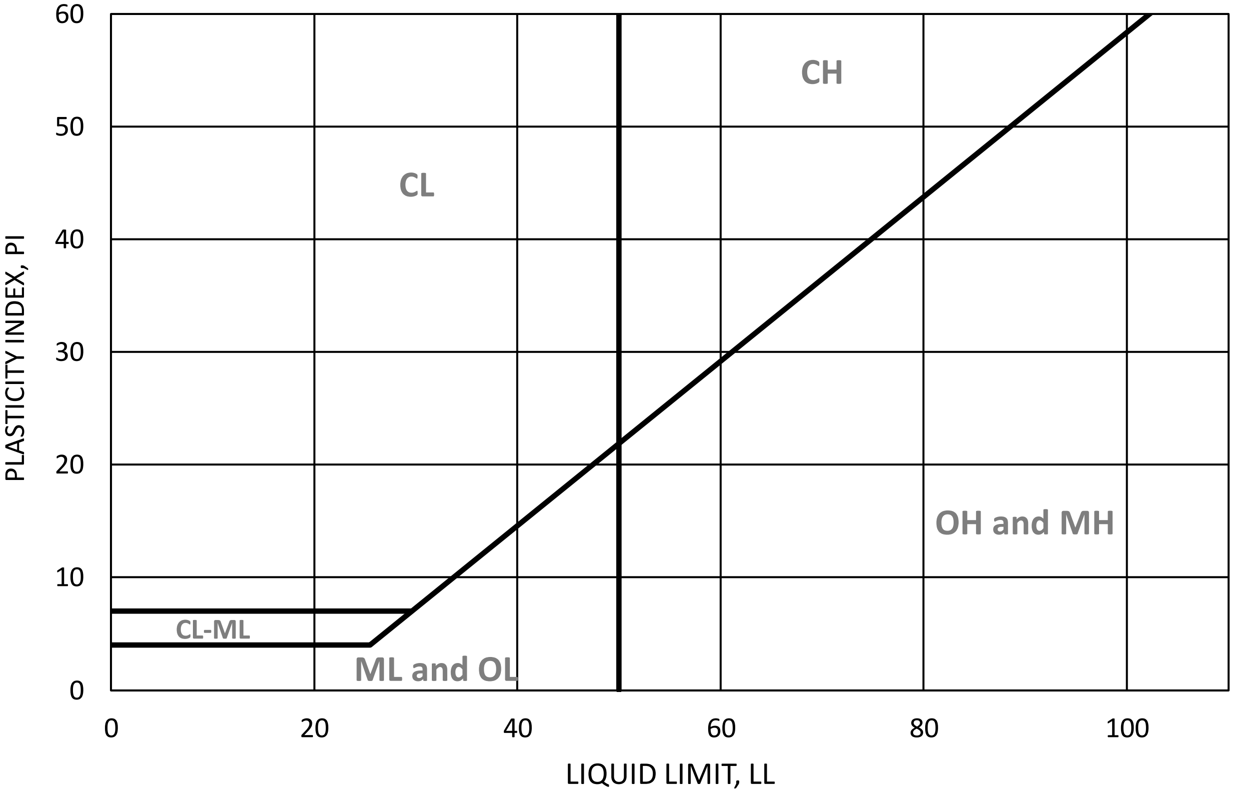
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B28



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B6	3'	N/P	N/P	N/P		N/P
◆	B6	15'	N/P	N/P	N/P		N/P
▲	B6	20'	N/P	N/P	N/P		N/P
●	B6	22.5'	N/P	N/P	N/P		N/P
□	B6	30'	N/P	N/P	N/P		N/P
◇	B6	35'	N/P	N/P	N/P		N/P
△	B6	37.5'	N/P	N/P	N/P		N/P
○	B6	40'	N/P	N/P	N/P		N/P

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

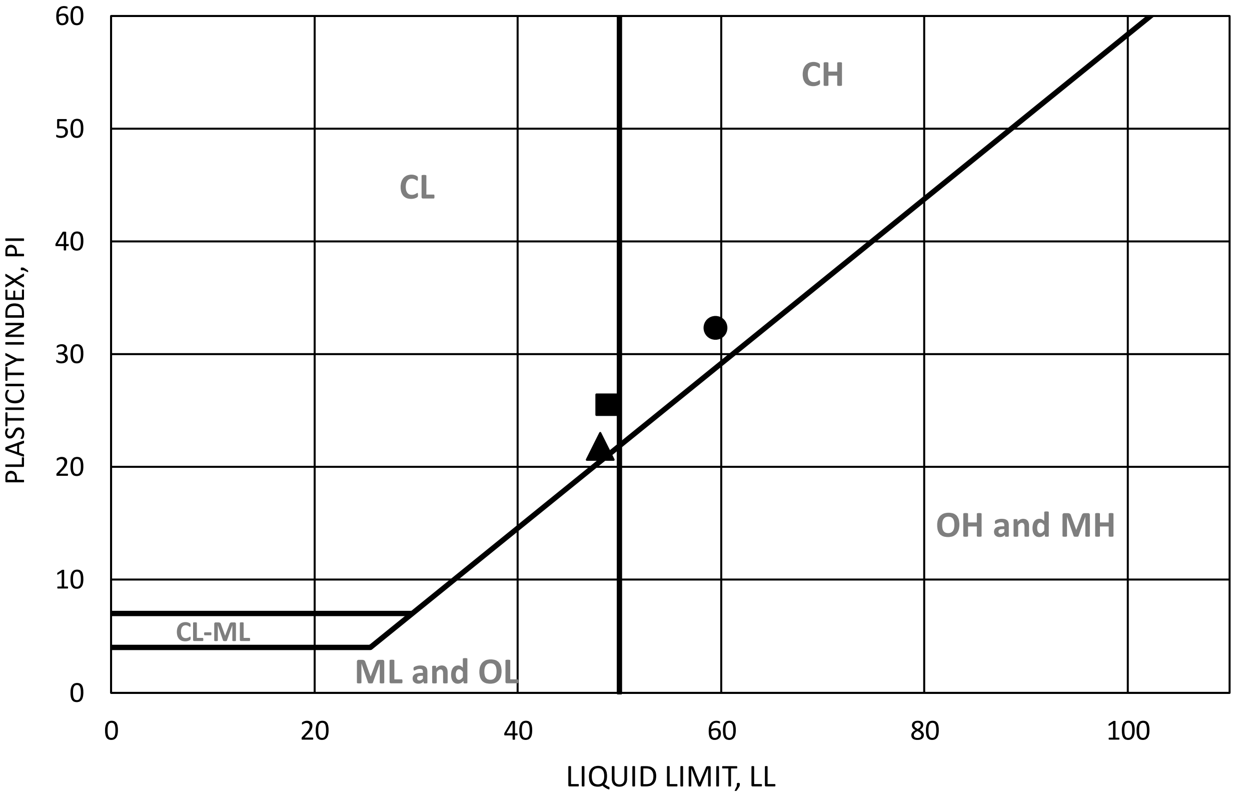
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B29



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B6	42.5'	49	23	25		CL
◆	B6	45'	N/P	N/P	N/P		N/P
▲	B6	52.5'	48	26	22		CL
●	B6	55'	60	27	32		CH
□	B6	57.5'	N/P	N/P	N/P		N/P
◇							
△							
○							

N/P = Non-Plastic

GEOCON

ATTERBERG LIMITS

ASTM D-4318

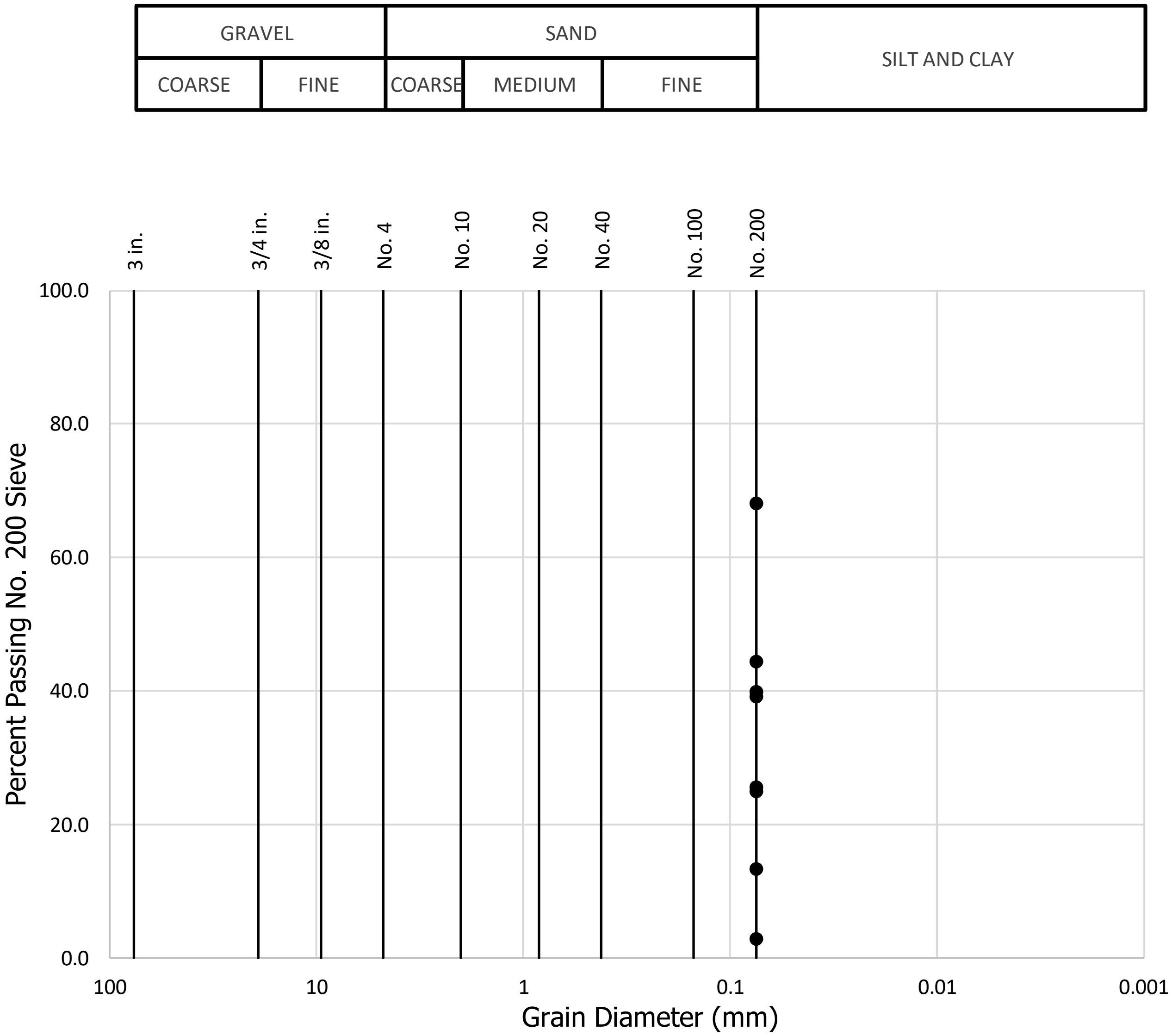
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B30



Sample No.	Percent Passing No. 200 Sieve
B6 @ 3'	39.2
B6 @ 5'	39.9
B6 @ 10'	2.9
B6 @ 15'	68.1
B6 @ 20'	25.0
B6 @ 22.5'	44.4
B6 @ 25'	13.4
B6 @ 30'	25.6

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

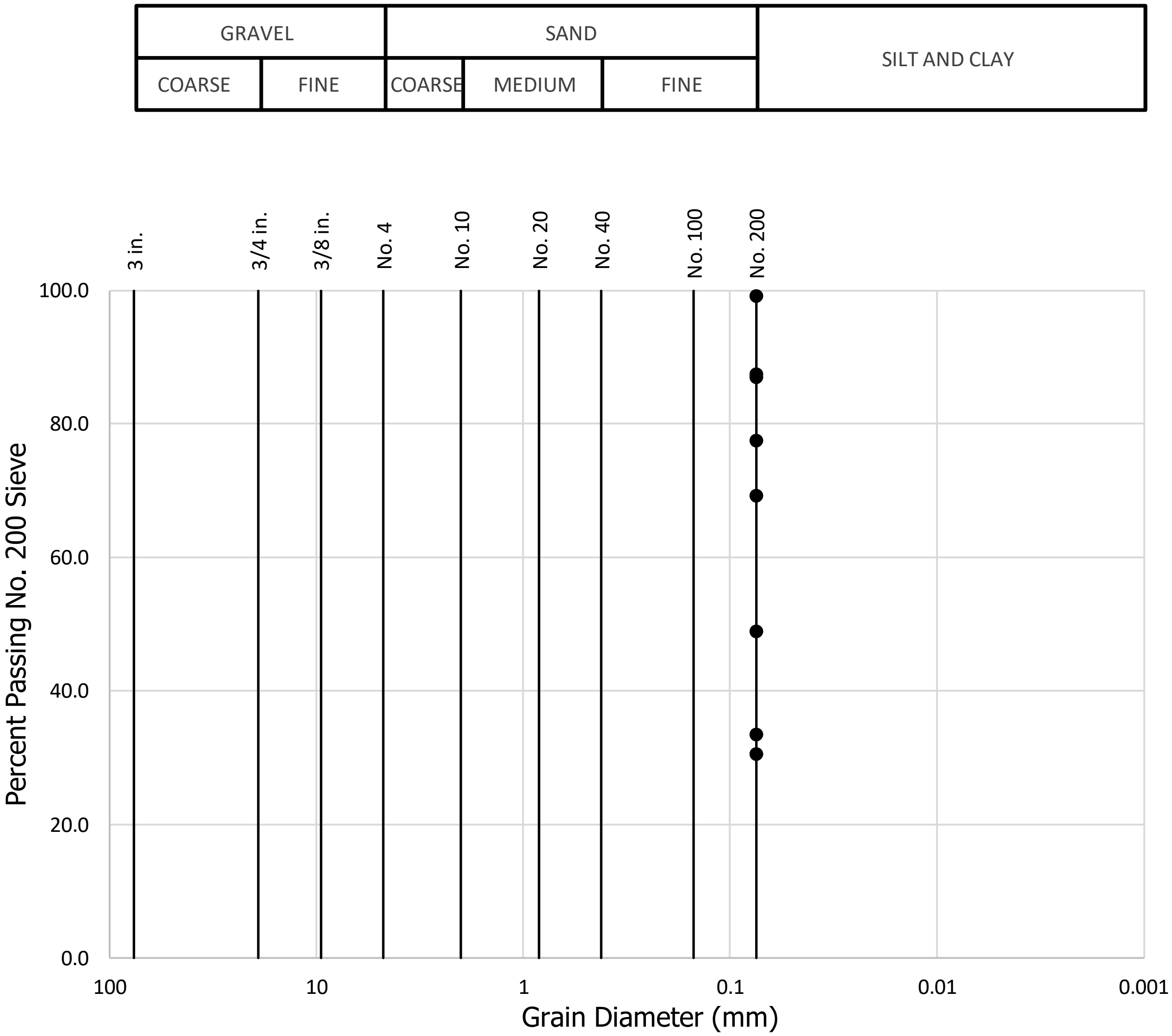
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B31



Sample No.	Percent Passing No. 200 Sieve
B6 @ 35'	77.5
B6 @ 37.5'	48.9
B6 @ 40'	33.5
B6 @ 42.5'	87.0
B6 @ 45'	69.3
B6 @ 47.5'	30.6
B6 @ 52.5'	87.4
B6 @ 55'	99.1

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

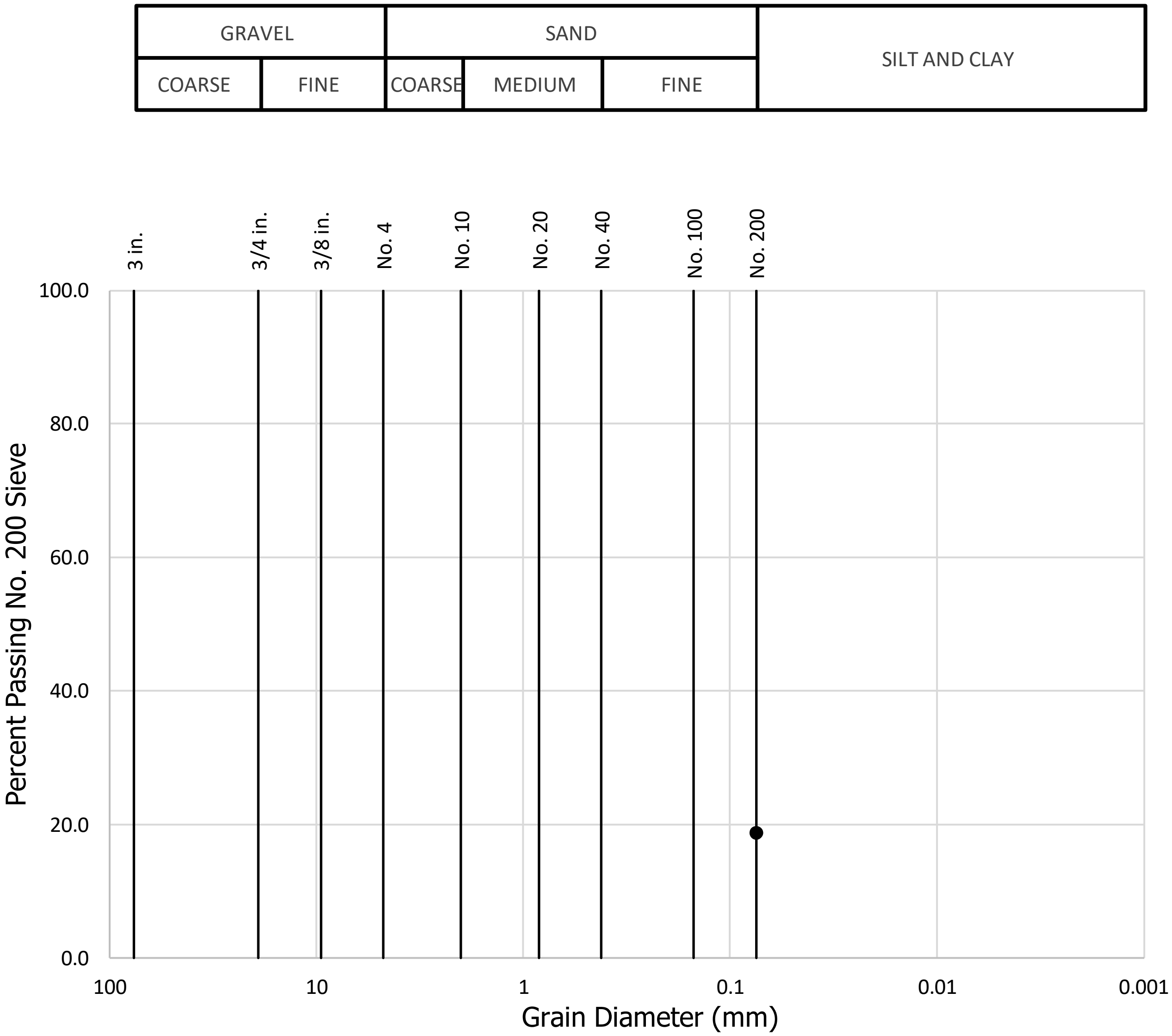
Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B32



Sample No.	Percent Passing No. 200 Sieve
B6 @ 57.5'	18.8

GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B33

B1@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	590.2	612.1
Wt. of Mold	(gm)	201.2	201.2
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	421.6	612.1
Dry Wt. of Soil + Cont.	(gm)	390.7	348.9
Wt. of Container	(gm)	121.6	201.2
Moisture Content	(%)	11.5	17.8
Wet Density	(pcf)	117.3	123.8
Dry Density	(pcf)	105.2	105.1
Void Ratio		0.6	0.6
Total Porosity		0.4	0.4
Pore Volume	(cc)	77.8	77.8
Degree of Saturation	(%) [S_{meas}]	52.0	79.7

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
2/10/2025	10:00	1.0	0	0.4055
2/10/2025	10:10	1.0	10	0.405
Add Distilled Water to the Specimen				
2/11/2025	10:00	1.0	1430	0.4052
2/11/2025	11:00	1.0	1490	0.4052

Expansion Index (EI meas) =	0.2
Expansion Index (Report) =	**0**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2022 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B34

B9@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	569.4	583.0
Wt. of Mold	(gm)	176.3	176.3
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	421.0	583.0
Dry Wt. of Soil + Cont.	(gm)	393.7	357.4
Wt. of Container	(gm)	121.0	176.3
Moisture Content	(%)	10.0	13.8
Wet Density	(pcf)	118.6	122.5
Dry Density	(pcf)	107.8	107.7
Void Ratio		0.6	0.6
Total Porosity		0.4	0.4
Pore Volume	(cc)	74.6	74.3
Degree of Saturation	(%) [S_{meas}]	48.3	66.4

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
2/10/2025	10:00	1.0	0	0.4113
2/10/2025	10:10	1.0	10	0.4111
Add Distilled Water to the Specimen				
2/11/2025	10:00	1.0	1430	0.4097
2/11/2025	11:00	1.0	1490	0.4097

Expansion Index (EI meas) =	-1.4
Expansion Index (Report) =	**0**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2022 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

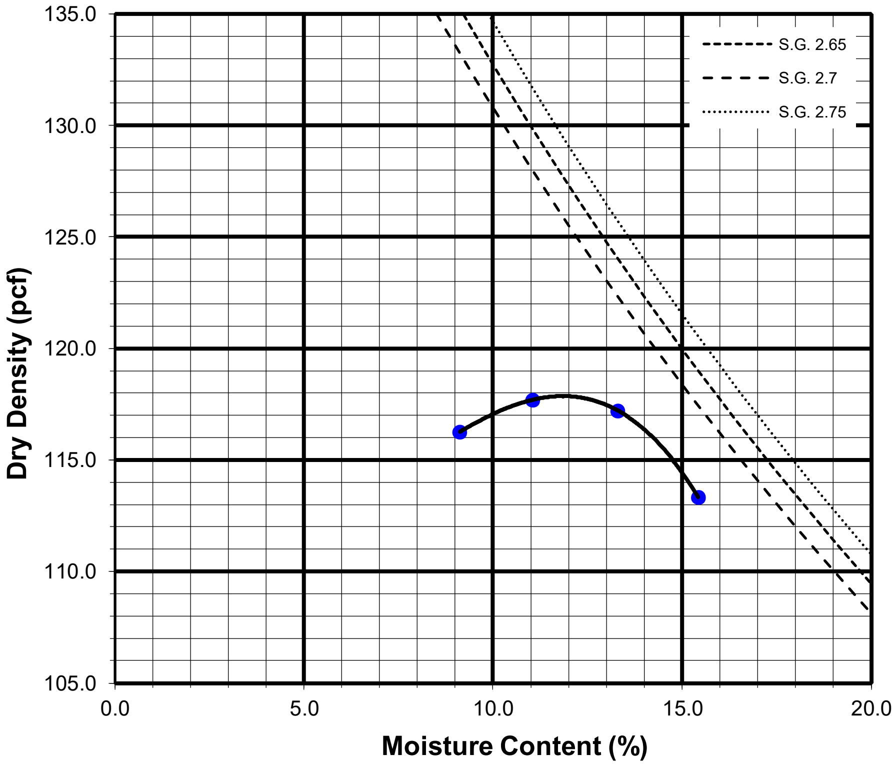
Figure B35

Sample No:

B1@0-5'	Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6198	6256	6288	6258		
Weight of Mold	(g)	4282	4282	4282	4282		
Net Weight of Soil	(g)	1916	1974	2006	1976		
Wet Weight of Soil + Cont.	(g)	2338.2	2361.7	2396.3	2365.9		
Dry Weight of Soil + Cont.	(g)	2177.1	2164.4	2159.2	2100.4		
Weight of Container	(g)	409.7	377.3	376.3	378.8		
Moisture Content	(%)	9.1	11.0	13.3	15.4		
Wet Density	(pcf)	126.8	130.7	132.8	130.8		
Dry Density	(pcf)	116.3	117.7	117.2	113.3		

Maximum Dry Density (pcf)	**118.3**	**Optimum Moisture Content (%)**	**12.2**



Preparation Method: A

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: PZ

GEOCON

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

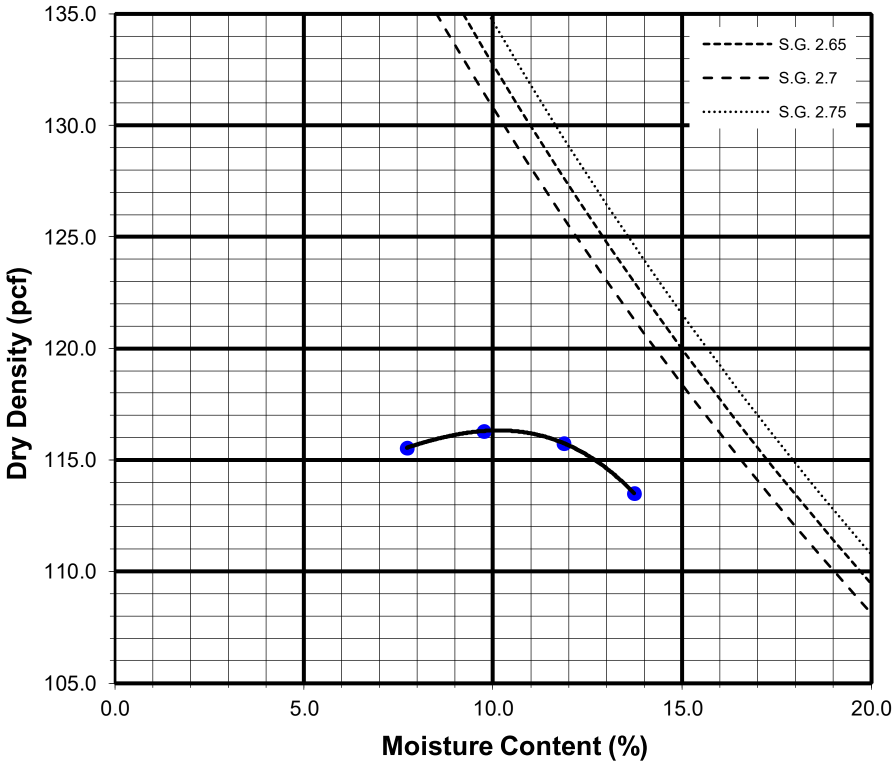
Figure B36

Sample No:

B9@0-5'	Silty Sand (SM)

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	6162	6210	6238	6232		
Weight of Mold (g)	4282	4282	4282	4282		
Net Weight of Soil (g)	1880	1928	1956	1950		
Wet Weight of Soil + Cont. (g)	2272.5	2341.7	2345.1	2341.9		
Dry Weight of Soil + Cont. (g)	2136.8	2170.0	2136.1	2104.8		
Weight of Container (g)	378.6	409.6	376.5	377.3		
Moisture Content (%)	7.7	9.8	11.9	13.7		
Wet Density (pcf)	124.5	127.6	129.5	129.1		
Dry Density (pcf)	115.5	116.3	115.7	113.5		

Maximum Dry Density (pcf)	**116.5**	**Optimum Moisture Content (%)**	**10.8**



Preparation Method: A

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: PZ

GEOCON

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

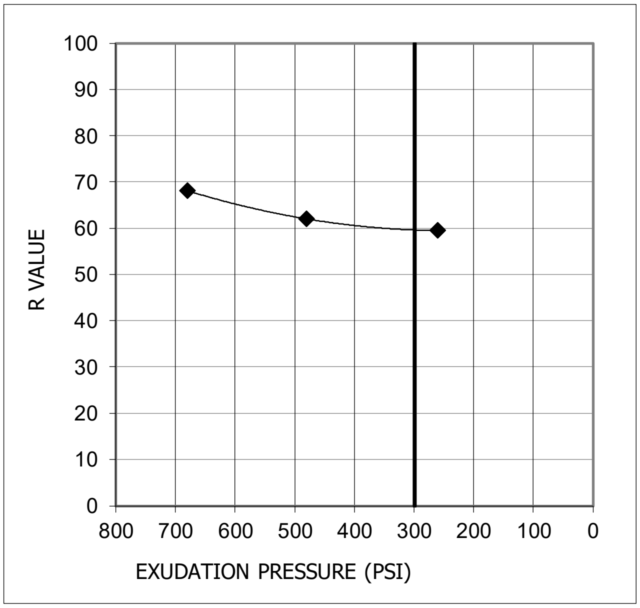
MAR. 2025

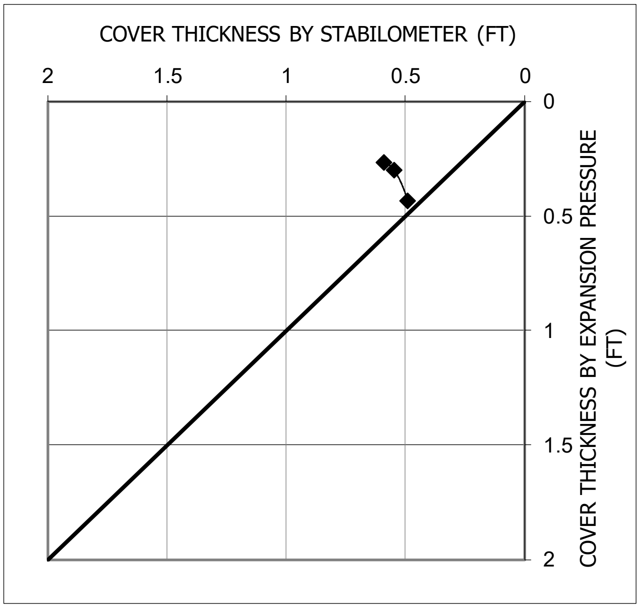
Figure B37

Mold ID		A	B	C
Exudation Pressure	(psi)	680	480	260
Expansion Dial	(.0001")	13	9	8
Expansion Pressure	(psf)	56.3	39.0	34.6
Resistance 'R' Value	(psi)	68	62	59
Moisture Content	(%)	10.5	11.5	12.5
Dry Density	(pcf)	112	111.7	109.3

Sample ID:
B2@0-5'
Sample Description:
SM

R-Value by Expansion:	68
R-Value by Exudation:	60
R-Value by Equilibrium:	**60**





GEOCON

R-VALUE TEST RESULTS

ASTM D-2844

Checked by: PZ

Project No.: W2045-88-01

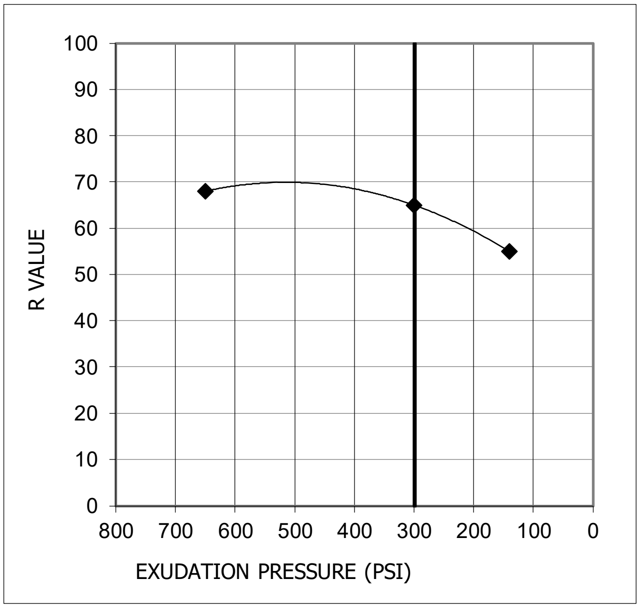
16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

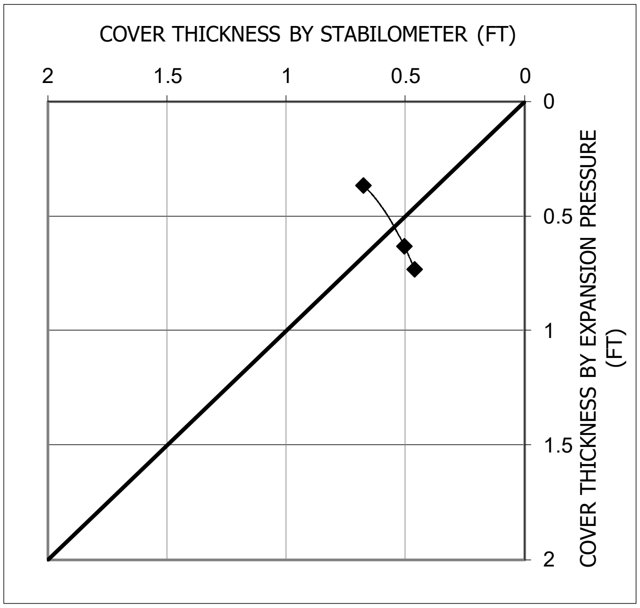
MAR. 2025

Mold ID		A	B	C
Exudation Pressure	(psi)	140	300	650
Expansion Dial	(.0001")	11	19	22
Expansion Pressure	(psf)	47.6	82.3	95.3
Resistance 'R' Value	(psi)	55	65	68
Moisture Content	(%)	12.8	11.8	11.3
Dry Density	(pcf)	114.7	115.2	114.1

Sample ID:
B4@0-5'
Sample Description:
SM

R-Value by Expansion:	64
R-Value by Exudation:	65
R-Value by Equilibrium:	**64**





GEOCON

R-VALUE TEST RESULTS

ASTM D-2844

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B39

SUMMARY OF LABORATORY
POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
AASHTO T289 ASTM D4972 and AASHTO T288 ASTM G187

Sample No.	pH	Resistivity (ohm centimeters)
B1@0-5'	8.7	5100 (Moderately Corrosive)
B9@0-5'	8.5	3700 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
B1@0-5'	0.008
B9@0-5'	0.007

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
AASHTO T290 ASTM C1580

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure
B1@0-5'	0.000	S0
B9@0-5'	0.001	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

Figure B40

Sample No.	Organic Content (%)
B1 @ 1'	1.1
B1 @ 3'	0.9
B1 @ 5'	2.8
B3 @ 1'	1.2
B3 @ 3'	1.0
B9 @ 1'	1.3
B9 @ 3'	0.8
B9 @ 5'	0.6
B12 @ 2.5'	0.5
B12 @ 5'	1.1

GEOCON

ORGANIC CONTENT

Checked by: PZ

Project No.: W2045-88-01

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA
APN 144-11-01

MAR. 2025

APPENDIX C

APPENDIX C

PRIOR REPORT

DUE-DILIGENCE GEOTECHNICAL EVALUATION
PROPOSED RESIDENTIAL DEVELOPMENT
16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA

Prepared For

SHOPOFF ADVISORS, LP
2 PARK PLAZA, SUITE 700
IRVINE, CALIFORNIA 92614

Prepared By

LEIGHTON AND ASSOCIATES, INC.
2600 MICHELSON DRIVE, SUITE 400
IRVINE, CALIFORNIA 92612

13255.001

September 20, 2021

(Revised July 2, 2024)

Leighton
a verdantas company

Leighton and Associates, Inc.

September 20, 2021
(Revised July 2, 2024)

Project No. 13255.001

Shopoff Advisors, LP
2 Park Plaza, Suite 700
Irvine, California 92614

Attention: Mr. Blair Ruffner

Subject: **Due-Diligence Geotechnical Evaluation**
Proposed Residential Development
16300 Euclid Street
Fountain Valley, California

In accordance with our proposal, dated August 16, 2021, Leighton and Associates, Inc. (Leighton) has performed a due-diligence level geotechnical evaluation for the subject project. The purpose of this study is to provide you with our professional opinion with respect to the feasibility of developing the site for its intended use from a geotechnical standpoint, and to provide you with the required information for budgeting and cost analysis. Our study included five (5) Cone Penetrometer Test (CPT) soundings to preliminarily evaluate the liquefaction and settlement potential at the site. In addition, our study included limited geotechnical laboratory testing of near-surface site soils to evaluate soil expansion and corrosion potential.

We appreciate the opportunity to provide our services for this project. We trust that the information contained herein meets your objectives. If you have any questions or concerns, please contact us at your convenience. The undersigned can be reached at **(866) *LEIGHTON***, specifically at the phone extension and e-mail address listed below.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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1.0 INTRODUCTION

1.1 Site Description and Proposed Development

The project site is located at the northeast corner of Euclid Street and Heil Avenue in the City of Fountain Valley, California. The site location (latitude 33.724564°, longitude -117.934992°) and immediate vicinity are shown on Figure 1, *Site Location Map.* The project site covers approximately 19 acres and is currently used for agricultural purposes. The Orange County Assessor's Office identifies the site as Assessor Parcel Numbers (APN) 144-111-01. The site is bordered by single-family residential homes to the north and east, Heil Avenue to the south, and Euclid Street to the west.

Based on review of aerial photographs (NETR, 2021), the site appears to have been used for agricultural purposes since at least 1953.

Review of the United States Geological Survey (USGS) *7.5-Minute Newport Beach Quadrangle* (USGS, 1978) indicates the site is relatively flat at approximate elevation (El.) +45 feet mean sea level (msl) with sheet flow generally directed to the southwest.

Based on review of the *Architectural Site Plan (P01)* prepared by JZMK Partners, dated May 17, 2024, we understand the proposed development will consist of (36) two-story triplex units, (183) three-story townhome units, and a five-story apartment building with 407 units wrapped around a 6½-level parking structure. When plans become available, they should be provided to the geotechnical engineer for review to ensure the recommendations contained herein remain applicable to site development.

1.2 Purpose and Scope

The purpose of our work was to evaluate the subsurface conditions at the site relative to the proposed development and provide preliminary geotechnical recommendations to aid in project planning. The scope of this evaluation included the following tasks:

- *Background Review* – We reviewed readily available geotechnical reports, literature, aerial photographs, and maps relevant to the site available from our in-house library or in the public domain. We evaluated geological hazards and potential geotechnical issues that may significantly impact the site. The documents reviewed are listed in Appendix A, *References*.

- *Field Exploration* – Our field exploration was performed on September 9, 2021 and consisted of five (5) Cone Penetrometer Test (CPT) soundings advanced at the site (designated CPT-1 through CPT-5), each to an approximate depth of 50 feet below ground surface (bgs). Shear wave velocity measurements were recorded at CPT-4 to develop seismic design parameters. Pore pressure dissipation tests were recorded at CPT-1 and CPT-4 to evaluate the groundwater conditions at the site. In addition, the groundwater level was physically measured at each CPT location after completion of each CPT prior to backfill. The approximate locations of the CPTs are shown on Figure 2, *Exploration Location Map*. Prior to the field exploration, the CPT locations were marked and Underground Service Alert (USA) was notified for utility clearance. In addition, we hand-augered the upper 5 feet at CPT-2 to CPT-4 prior to collect a representative near-surface bulk soil sample from the site for laboratory testing.

  The CPT soundings were performed in accordance with ASTM D5778 advanced by a 30-ton CPT rig in which a standard Cone equipped with a 15 cm^2 tip advanced at a constant rate of approximately 1 inch per second. The CPT provides a continuous record of the subsurface stratigraphy via data regarding tip and sleeve resistance which is continuously recorded electronically as the probe is advanced through the subsurface stratigraphy. The recorded data is processed yielding interpretations of soil type based upon the anticipated engineering behavior of the various soil strata though which the probe penetrates. Graphical logs of the interpreted soil conditions at the CPT sounding locations are included in Appendix B, *Cone Penetrometer Test (CPT) Logs*.

- *Laboratory Testing* – Laboratory tests were performed on the collected bulk soil sample from CPT-2 at our in-house geotechnical laboratory to preliminarily evaluate the near-surface site soils for expansion potential and corrosivity. The following laboratory tests were performed:

  - Expansion Index (ASTM D4829); and
  - Corrosivity Suite - pH, Sulfate, Chloride, and Resistivity (California Test Methods 417, 422, and 532/643).

The results of the laboratory tests are presented in Appendix C, *Laboratory Test Results*.

- *Engineering Analysis* – The data obtained from our background review and subsurface field exploration were evaluated and analyzed to develop conclusions and preliminary recommendations for the proposed development.
- *Report Preparation* – This report presents our findings, conclusions, and preliminary recommendations for the conceptually proposed development

2.0 GEOTECHNICAL FINDINGS

2.1 Regional Geology

The project site is located on the lowest reach of the Santa Ana River basin within the Peninsular Ranges geomorphic province. The Peninsular Ranges geomorphic province extends southward from the Los Angeles Basin to the tip of Baja California (Yerkes et al., 1965) and is characterized by elongated northwest-trending mountain ranges separated by sediment-floored valleys. The most dominant structural features of the province are the northwest-trending fault zones, most of which die out, merge with, or are terminated by the steep reverse faults at the southern margin of the Transverse Ranges geomorphic province. East of the site are the northwest-trending Santa Ana Mountains, a large range that has been uplifted on its eastern side along the Whittier-Elsinore Fault Zone, producing a tilted, irregular highland that slopes westward toward the sea.

Approximately 65 million years ago (at the end of the Cretaceous Period) a deep, structural trough existed off the coast of southern California (Yerkes, 1972). Over time the trough was filled with sediments eroded from the surrounding highlands and mountains. About 7 million years ago, the boundary between the Pacific and North American plates shifted to its present position and the geologically modern Los Angeles basin began to form. The deepest part of the Los Angeles basin contains Tertiary to Quaternary-aged (65 million years and younger) marine and non-marine sedimentary rocks that are about 24,000 feet thick (Yerkes, et al, 1965; Wright, 1991). During the Pleistocene epoch (the last two million years), the region was flooded as the sea level rose in response to the worldwide melting of the Pleistocene glaciers depositing sediments across the Los Angeles Basin during transgression and regression of sea level.

The area south and west of the Santa Ana Mountains is generally characterized as a broad, complex, alluvial fan that receives sediments from the Santa Ana River and its tributaries draining the Santa Ana and San Bernardino Mountains. These sediments are comprised of relatively flat-lying, unconsolidated to loosely consolidated clastic deposits that are approximately 3,000 feet thick beneath the site (Sprotte et al., 1980, and Real, 1985). The project site is located approximately 0.9 mile west of the Santa Ana River. Regional geologic mapping of the project site and vicinity indicates that near-surface native soils beneath the site consist of Quaternary age (Holocene) young alluvial fan deposits consisting of varying proportions of sand, silt, and clay (Morton and Miller, 2006; Bedrossian and

Roffers, 2010). The surficial geologic units mapped in the vicinity of the project site are shown on Figure 3, *Regional Geology Map*.

2.2 Subsurface Conditions

Based on past site usage and development, the site is likely underlain by a relatively thin layer of undocumented artificial fill materials overlying Quaternary-age young alluvial fan deposits. The artificial fill materials at the site are likely associated with the current and historical agricultural site use, and generally anticipated to be on the order of approximately 2 to 3 feet in thickness. The stratigraphy of the subsurface soils as interpreted in each CPT is presented on the logs included in Appendix B, and a general description of the interpreted earth materials as encountered during the CPT exploration are described below.

2.2.1 Subsurface Soils

Based on review of the CPT interpretations (CPT-1 through CPT-5), the subsurface soils to the depth explored (50 feet bgs) contain alternating sequences with variable thicknesses of sand, silty sand, sandy silt, silt, silty clay, and clay.

Artificial fill materials in the upper 2 to 3 feet across the site generally consist of silty sand. Localized thicker accumulations of fill materials should be anticipated during future earthwork construction.

2.2.2 Expansive Soil Characteristics

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subject to uplifting forces caused by the swelling. Without proper mitigation measures, heaving and cracking of both building foundations and slabs-on-grade could result. Based on our exploration, the near surface (upper 5 feet) onsite soils consist predominantly of sand and silty sand. The laboratory test result of representative near-surface (upper 5 feet) bulk soil sample from CPT-2 indicates very low expansion potential when wetted (EI = 0), or non-expansive. Accordingly, we recommend that the upper onsite soils be assumed to have very low expansion potential.

Variance in expansion potential of onsite soil is anticipated; therefore, additional testing is recommended during future geotechnical exploration and/or upon completion of rough grading to confirm the expansion potential result presented in this report. Standard engineering and earthwork construction practices, such as proper foundation design and controlled moisture conditioning will reduce impacts associated with expansive soils.

2.2.3 Soil Corrosivity

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 4.3 of ACI 318 (ACI, 2014). The 2022 California Building Code (CBC) provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm). The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover or plain steel substructures, such as steel pipes, is 500 ppm per California Test 532. Concentrations of chloride ions above the stated concentration or other characteristics such as soil resistivity or redox potential may warrant special corrosion protection measures.

For screening purposes, representative near-surface (upper 5 feet) bulk soil sample from CPT-2 was tested to provide a preliminary evaluation of corrosivity. The test results indicate a soluble sulfate concentration of 115 ppm, chloride content of 100 ppm, pH value of 7.52 and minimum resistivity value of 1,250 ohm-cm.

The results of the resistivity test indicate that the underlying soil is severely corrosive to buried ferrous metals per ASTM STP 1013. Based on the measured water-soluble sulfate content from the soil sample, concrete in contact with the soil is expected to have negligible exposure to sulfate attack per ACI 318-14. The sample tested for water-soluble chloride content indicate a low potential for corrosion of steel in concrete due to the chloride content of the soil. The chemical analysis test results for the onsite soil from our geotechnical exploration are included in Appendix C of this report.

2.2.4 Shear Wave Velocity Profile

Shear wave velocities were measured in CPT-4, see Figure 2 for location. Results are presented in Appendix B. The average shear wave velocity the ground surface down to about 45 feet bgs is about 587 feet per, which falls just under the lower threshold of Site Class D. However, considering that shear wave velocity generally increases with depth, we assumed that Site Class D can be justified with additional exploration. Accordingly, the seismic site class is characterized as Site Class D for preliminary design purposes.

2.2.5 Groundwater Conditions

Groundwater was encountered at the locations of CPT-1 through CPT-5 at depths between 5.9 and 6.4 feet bgs. According to groundwater information obtained through the California Geological Survey (CGS) and presented in the Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangles (CGS, 1997), the historically shallowest groundwater depth in the vicinity of the project site is approximately 4 feet bgs. In addition, based on review of available groundwater information from the California Department of Water Resources Water Data Library (DWR, 2021) for a nearby groundwater monitoring well located approximately 0.4 mile to the southwest of the project site (State Well # 05S10W21M002S), the shallowest groundwater level measured for a monitoring period between November 1969 and October 2010 was approximately 12.3 feet bgs.

Based on the currently proposed development scheme, groundwater will likely pose a constraint during and after construction. We recommend that a groundwater level at 4 feet bgs be assumed for preliminary design and construction.

Seasonal fluctuations in groundwater level, localized zones of perched water including water due to nearby landscaping, and an increase in soil moisture should be anticipated during and following locally intense rainfall or stormwater runoff. More detailed evaluation of the current groundwater level at the site should be performed during future subsurface exploration of the site in support of design level foundation studies

2.3 Surface Fault Rupture

Our review of available in-house literature indicates that no known active faults have been mapped across the site, and the site is not located within a designated Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007; CGS, 1986 and 2018).

Therefore, the potential for surface fault rupture at the site is expected to be low and a surface fault rupture hazard evaluation is not mandated for this site.

The location of the closest active faults to the site was evaluated using the United States Geological Survey (USGS) Earthquake Hazards Program National Seismic Hazard Maps (USGS, 2008). The closest active faults to the site with the potential for surface rupture are the Newport-Inglewood Fault Zone (NIFZ), Elsinore Fault, and Palos Verdes Fault located approximately 4.7 miles, 14.8 miles, and 15.5 miles from the site, respectively. The Puente Hills and San Joaquin Hills faults are located approximately 2.0 and 10.1 miles from the site, respectively; however, these are blind thrust faults that are concealed at depth without the potential for surface fault rupture. The San Andreas Fault, which is the largest active fault in California, is approximately 46 miles northeast of the site. Major regional faults with surface expression in proximity to the site are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

2.4 Seismicity and Ground Shaking

The principal seismic hazard to the site is ground shaking resulting from an earthquake occurring along any of several major active and potentially active faults in southern California. The intensity of ground shaking at a given location depends primarily upon the earthquake magnitude, the distance from the seismic source, and the site response characteristics. The site should be expected to experience strong ground shaking after the proposed project is developed resulting from an earthquake occurring along one or more of the major active faults (Figure 4). Accordingly, the project should be designed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). The 2022 edition of the California Building Code (CBC) is the current edition of the code. Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals for the project, potential effects relating to seismic shaking can be reduced.

The following parameters should be considered for design under the 2022 CBC:

Table 1 – 2022 CBC Based Ground Motion Parameters (Mapped Values)

Categorization/Coefficients	Code-Based
Site Longitude (decimal degrees) West	-117.934992°
Site Latitude (decimal degrees) North	33.724564°
Site Class	D (default)
Mapped Spectral Response Acceleration at 0.2s Period, S_s	1.331
Mapped Spectral Response Acceleration at 1s Period, S_1	0.477
Short Period Site Coefficient at 0.2s Period, F_a	1.2
Long Period Site Coefficient at 1s Period, F_v	null[1]
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS}	1.597
Adjusted Spectral Response Acceleration at 1s Period, S_{M1}	null[1]
Design Spectral Response Acceleration at 0.2s Period, S_{DS}	1.065
Design Spectral Response Acceleration at 1s Period, S_{D1}	null[1]
Site-adjusted geometric mean Peak Ground Acceleration, PGA_m	0.685

[1] Per Exception 2 in Section 11.4.8 of ASCE 7-16, seismic response coefficient C_S to be determined by Eq. 12.8-2 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5T_S$ or Eq. 12.8-4 for $T > T_L$

2.5 Liquefaction Potential

The term liquefaction is generally referenced to loss of strength and stiffness in soils due to build-up of pore water pressure when subject to cyclic or monotonic loading. Both sandy and clayey soils are susceptible to loss of strength and stiffness. Because of the difference in strength characteristic and methods for evaluating strength loss potential for granular and clayey soils, the term liquefaction is used for granular soils while cyclic softening is used for fine-grained soils (i.e. clays and plastic silts).

In general, adverse effects of liquefaction or cyclic softening include excessive ground settlement, loss of bearing support for structural foundations, and seismically-induced lateral ground deformations such as lateral spreading. Depending upon the relative thickness of the liquefied strata with respect to overlying non-liquefiable soils, other potentially adverse effects such as ground oscillation and ground fissuring may occur.

As shown on the *Seismic Hazard Zones* map for the Newport Beach Quadrangle (CGS, 1998), the project site **is** located within an area that has been identified by the State of California as being potentially susceptible to liquefaction (Figure 5, *Seismic Hazard Map*). In addition, the historically shallowest depth to groundwater at the site is approximately 4 feet bgs (CGS, 1997). Groundwater was also encountered during our subsurface exploration at depths between approximately 5.9 and 6.4 feet bgs.

As a part of this due-diligence geotechnical evaluation, we have evaluated the liquefaction potential at the site using the data obtained from the CPTs with the computer program *Cliq* (v.3.0.3.4). Based on our evaluation using the using the Maximum Considered Earthquake (MCE) and a design groundwater level of 4 feet bgs for the CPTs performed at the site, the potential for liquefaction to occur at the site is very high. The results of our analysis are presented in Appendix D, *Liquefaction Analysis.*

2.6 Seismically-Induced Settlement

Seismically-induced settlement consists of dynamic settlement of unsaturated soil (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within low density sandy soil due to reduction in volume during and shortly after an earthquake event.

Based on the results of our preliminary analysis, total seismically-induced settlement due to dry dynamic settlement (above groundwater) and liquefaction settlement (below groundwater) is estimated to range between 1.4 and 3.1 inches across the site. The differential settlement can be taken as one-half the total estimated settlement over a horizontal distance of 30 feet.

2.7 Seismically-Induced Lateral Ground Displacements

Liquefaction may also cause lateral spreading. For lateral spreading to occur, the liquefiable zone must be continuous, unconstrained laterally, and free to move along gently sloping ground toward an unconfined area. Since the site is essentially flat and constrained laterally, the potential for lateral spreading due to liquefaction is also considered low.

2.8 Seismically-Induced Landsliding

As shown on the *Seismic Hazard Zones* map for the Newport Beach Quadrangle (CGS, 1998), the project site is **not** located within a seismically-induced landslide hazard zone identified by the State of California (Figure 5, *Seismic Hazard Map*). In addition, due to project site being relatively flat, it is our opinion that the potential for seismically-induced landslide hazard at the site is negligible.

2.9 Flooding

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2009), the project site is located within a flood hazard area identified as "Zone X", which is defined as area with reduced flood risk due to levee. As shown on Figure 6, *Flood Hazard Zone Map*, the site is **not** located within a 100-year or 500-year flood hazard zone.

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. The project site **is** located within a flood impact zone from Prado Dam as indicated on Figure 7, *Dam Inundation Map*. However, due to the location and distance of the site from Prado Dam, the potential for earthquake-induced flooding to occur due to a failure of this dam is considered low. In addition, catastrophic failure of this dam is expected to be a very unlikely event in that dam safety regulations exist and are enforced by the Division of Safety of Dams, Army Corp of Engineers and Department of Water Resources. Inspectors may require dam owners to perform work, maintenance or implement controls if issues are found with the safety of the dam.

2.10 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the absence of an enclosed water body near the site and the inland location of the site, seiche and tsunami risks at the site are considered negligible.

2.11 Sedimentation and Erosion

The erosion characteristics of the unconsolidated alluvial deposits exposed on any future slopes onsite are expected to be moderately susceptible to erosion. These materials will be particularly prone to erosion during excavation and site development, especially during heavy rains.

The potential for erosion can be mitigated through the application of best management practices (BMPs) and other Storm Water Pollution Prevention Plan (SWPPPs), such as temporary catchment basins and/or sandbagging to control runoff and contain sediment transport within the project site during construction. Following completion of the project, the site is anticipated to be improved with structures, hardscape, landscaping, and appropriate drainage infrastructure. Therefore, sedimentation and erosion impacts upon completion of construction are considered less than significant.

3.0 FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

Presented below is a summary of findings based upon the results of our due-diligence geotechnical evaluation of the site:

- The site is likely underlain by a relatively thin veneer of undocumented artificial fill generally anticipated to be on the order of approximately 2 to 3 feet overlying Quaternary age (Holocene) young alluvial fan deposits. Localized thicker accumulations of fill materials should be anticipated during future earthwork construction.

- Based on review of groundwater information obtained through the California Geological Survey (CGS) and presented in the Seismic Hazard Zone Report for the Newport Beach Quadrangle, the historically shallowest groundwater depth in the vicinity of the project site is approximately 4 feet bgs.

- The site is **not** located in a designated Alquist-Priolo Earthquake Fault Zone. The nearest fault to the site with the potential for ground surface rupture is the Newport-Inglewood Fault Zone which is located approximately 4.7 miles from the site. The site is expected to experience moderate to strong ground shaking resulting from an earthquake from one of the major regional faults located in the southern California region.

- The site **is** located within an area shown as susceptible to liquefaction based on the State of California Seismic Hazard Zones map for the Newport Beach Quadrangle. Based on the results of our preliminary analysis of data obtained from our CPTs performed at the site, the potential for liquefaction to occur at the site is very high. Total seismically-induced settlement potential is estimated to range between 1.4 and 3.1 inches across the site.

- The site is essentially flat and is **not** located within an area shown as susceptible to seismically-induced landslides based on the California Seismic Hazard Zones map for the Newport Beach Quadrangle; therefore, the potential for this hazard to occur at the site is negligible.

- The project site is **not** located within a 100-year or 500-year flood hazard zone; however, the site **is** located within a flood impact zone from Prado Dam.

It should be noted that a design-level geotechnical exploration, including soil borings, laboratory testing, and engineering analysis is recommended and will be required by the reviewing agency during design phase of the project to confirm our preliminary findings and to develop design geotechnical recommendations for the project suitable for submittal to the reviewing agency in pursuit of building permits. In addition, the geotechnical consultant should review the grading plan, foundation plan and specifications as they become available to verify that the geotechnical recommendations presented in the future design-level studies for the project have been incorporated into the plans. Subsurface conditions encountered during field exploration indicate the soils underlying the site may be potentially compressible when subject to heavy structural loads. Some of the clayey soil may also be susceptible to settlement over time. The settlement of the native soil under the new buildings or engineered fill should be evaluated when the building and grading plans become available. Building loads should be provided when available for the various structures such that settlement analysis can be performed during design level foundation studies.

4.0 PRELIMINARY RECOMMENDATIONS

Presented below are the preliminary geotechnical recommendations for planning purposes. A geotechnical investigation that includes additional subsurface explorations will be required by the reviewing agency once the proposed design, building loads and project plans become available for review. Design of the project in accordance with standard engineering practice, including requirements of the 2022 California Building Code (CBC), and the recommendations of the project civil and structural engineers, geotechnical consultant and others will reduce the potential for adverse geotechnical conditions impacting the proposed improvements.

The conceptually proposed residential structures (both single-family and multi-family) may be supported on shallow spread-type foundations if the seismic and static settlements can be accommodated. Ground improvement will likely be required to enable support of planned podium structure on spread footings. There may be existing underground utilities that will also be impacted. Information on these utilities should be provided to Leighton for evaluation. All existing undocumented fill is recommended to be removed beneath any planned improvements. Excavations may require dewatering during construction if it extends more than approximately 4 feet bgs. Conversely, saturated subgrade soils may be encountered at this depth (or shallower) requiring mitigation. Any planned basement floor slab and basement walls will have to be designed for hydrostatic pressure if the basement level extends more than 4 feet bgs. A permanent dewatering system may be implemented as an alternative.

4.1 Earthwork

All site earthwork grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professionals.

4.1.1 Site Preparation

Prior to construction, the site should be cleared of any existing improvements, vegetation, trash and/or debris within the area of proposed grading. These materials should be removed from the site. Any underground obstructions onsite should be removed. Efforts should be made to locate any existing utility lines to be removed or rerouted where interfering with the proposed construction. Any resulting cavities should be properly backfilled and compacted. After the site is cleared, the soils should be carefully observed for the removal of all unsuitable deposits. All unsuitable deposits should be excavated and removed from proposed building/structure footprint prior to fill placement.

4.1.2 Site Grading

A majority of the project area is likely covered with artificial fill anticipated on the order of up to approximately 2 to 3 feet bgs. Localized thicker accumulations of undocumented fill materials and possible foundation remnants should be anticipated during future earthwork construction. To provide a uniform support and reduce the potential for differential settlement, all existing fill should be removed to expose suitable native soils and replaced as engineered fill to provide supports for the proposed building and other structural improvements.

Where ground improvement is not performed, removals should be performed such that a minimum of 3 feet of engineered fill is established below the bottom of all new foundations. Where feasible, overexcavation and recompaction should extend a minimum horizontal distance of 3 feet from the edges of the foundations (i.e., a 1:1 projection line down from the bottom edges of the foundations).

4.1.3 Excavation Bottom Preparation

After excavating as recommended, the moisture content of the soils should be determined, and the soils slowly and uniformly moistened (or dried) as necessary to bring the soils to a uniform moist condition. The moisture content of the clayey soils should be brought to about 4 percent over optimum moisture content to a depth of 18 inches. The moisture content of any relatively non-expansive and predominantly granular soils should be brought to about 2 percent over optimum moisture content to a depth of 18 inches. The moisture content of the subgrade should be checked and approved by Leighton prior to placing the required fill. In addition, placement of a gravel or rock layer may be needed to stabilize the excavation bottoms, reduce the potential for piping, and provide a firm working surface for heavy equipment. If a rock or gravel layer is placed, a layer of nonwoven filter fabric such as Mirafi 140N or equivalent, should be placed over the gravel/rock layer to reduce the potential for migration of sediments into the void space between the coarse aggregate.

All concrete slabs on grade, including floor slabs and Portland cement concrete paving, should be underlain by at least 2 feet of non-expansive fill (EI<20). It is anticipated that existing near-surface soil onsite may be suitable for this purpose. Additional geotechnical testing is warranted to confirm expansive potential for reuse below concrete during future design level study of the site.

4.1.4 Fill Materials

On-site soil that is free of construction debris, organics, cobbles, boulders, rubble, or rock larger than 4-inches in largest dimension is suitable to be used as fill for support of structures. Any imported fill soil should be approved by the geotechnical engineer prior to placement as fill.

4.1.5 Fill Placement and Compaction

Fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned to at least 2 percent above optimum moisture content for sandy soils and at least 4 percent above optimum moisture content for clayey soils, and compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM Test Method D 1557. Aggregate base should be compacted to a minimum of 95 percent relative compaction.

When grading is interrupted by heavy rains, fill operations should not be resumed until the moisture content and the dry density of the placed fill are satisfactory.

4.1.6 Construction Dewatering

Due to the shallow groundwater conditions that exist at the site and the reported historic high groundwater level at the site as shallow as 4 feet bgs (CGS, 1997), temporary dewatering may be required during earthwork construction and/or construction of subterranean portions of the project (if applicable). To minimize the potential for impacting the surrounding improvements, we recommend using localized sump pumps within the excavation to remove the groundwater that enters the excavation. It is the responsibility of the contractor to design and install the dewatering system should it be required. The contractor should anticipate that continuous pumping of groundwater may be required during the excavation. Discharge of groundwater during excavation should comply with all environmental regulations and under proper National Pollutant Discharge Elimination System (NPDES) permitting. Additional site-specific information to better characterize the groundwater levels at the site should be obtained during the design-level geotechnical investigation recommended to be performed during the design phase of the project.

4.2 Preliminary Foundation Design

Shallow foundations may be used to support the proposed structures if the seismic and static settlements can be accommodated. Due to anticipated dead-plus-live column loads exceeding 500 kips, ground improvement will likely be required to enable support of planned podium structure on spread footings.

The ground improvement system should be designed by a specialty contractor specializing in design and construction of ground improvement techniques. Feasible alternatives for ground improvement at this site that may be considered are Geopiers® or rammed aggregate piers, drilled displacement columns, and stone columns. The performance target for ground improvement is an allowable bearing capacity of at least 5,000 pounds per square foot (psf) and a reduction in total static plus seismically-induced settlement of less than 1½ inches.

4.2.1 Conventional Spread Footings over Improved Ground

Footings should be embedded a minimum 18 inches below the lowest adjacent grade. An allowable soil bearing pressure of 5,000 psf may be used for footings with a minimum width of 12 inches for continuous footings and 18 inches for isolated footings. A one-third increase in the bearing value for short duration loading, such as wind or seismic forces may be used. The ultimate bearing capacity can be taken as 15,000 psf, which does not incorporate a factor of safety. A resistance factor of 0.45 should be used for initial bearing capacity evaluation with factored loads.

The recommended bearing values are net values, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot (pcf); the weight of soil backfill can be neglected when determining the downward loads

The allowable bearing capacity for shallow footings is based on a total static settlement of 1 inch. Differential settlement can be taken as half the total settlement over a horizontal distance of 30 feet. Since settlement is a function of footing size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists. Leighton should review the settlement estimates when final foundation plans and loads for the proposed structures become available.

Resistance to lateral loads will be provided by a combination of friction between the soil and structure interface and passive pressure acting against the vertical portion of the footings structures. For calculating lateral resistance above the design groundwater at 4 feet bgs, a passive pressure of 300 pcf and a frictional coefficient of 0.30 may be used. Below groundwater, the passive resistance should be reduced to 200 pcf to a maximum of 3,000 psf. Note that the passive and frictional coefficients do not include a factor of safety. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

4.2.2 Mat Foundations

A mat foundation established on engineered fill may be used to support the proposed single-family homes and townhouses if seismically-induced settlement of up to 3 inches can be accommodated. Mat foundations may be designed using an allowable bearing capacity 2,000 pounds per square foot (psf) and a modulus of subgrade reaction of 25 pounds per cubic inch (pci). Differential settlement of the mat foundation due to the static and seismic loads is expected to be on the order of 1½ inches over a distance of 30 feet. The bearing capacity may be increased by one-third for wind or seismic loading. The perimeter of the mat foundation should have a minimum embedment of 12 inches below the lowest adjacent grade.

The ultimate bearing capacity can be taken as 4,500 psf, which does not incorporate a factor of safety. A resistance factor of 0.45 should be used for initial bearing capacity evaluation with factored loads. The recommended bearing values are net values, and the weight of concrete in the mat foundation can be taken as 50 pcf; the weight of soil backfill can be neglected when determining the downward loads.

Resistance to lateral loads will be provided by a combination of friction between the soil and structure interface and passive pressure acting against the vertical portion of the footings structures. For calculating lateral resistance above the design groundwater at 4 feet bgs, a passive pressure of 300 pcf and a frictional coefficient of 0.30 may be used. Below groundwater, the passive resistance should be reduced to 200 pcf to a maximum of 3,000 psf. Note that the passive and frictional coefficients do not include a factor of safety. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

4.2.3 Post-Tensioned Slab-on-Ground Foundations

Post-tensioned slab-on-ground (PT-slab) foundations over engineered fill may also be used to support the proposed single-family homes and townhouses if seismically-induced settlement of up to 3 inches can be accommodated. The following table provides a design summary for foundations for the apartment buildings underlain engineered fill consisting of on-site materials.

Table 2 - Geotechnical Parameters for Post-Tensioned Slab Design

Design Criteria		Design Value
Edge Moisture Variation, e_m (feet)	Center Lift	9
	Edge Lift	5
Differential Soil Movement, y_m (inch)	Center Lift	0.2
	Edge Lift	0.4
Minimum Perimeter Foundation Embedment Depth (feet)		2
Allowable Bearing Pressure		2,000 psf*
Modulus of Subgrade Reaction		25 pci**

*pounds per square foot
**pounds per cubic inch

The above recommended slab design parameters are based on Post-Tensioning Institute's *Design of Post-Tensioned Slabs-on Ground*, 3rd Edition with 2008 supplement (PTI DC10.1-08). Local agencies, the structural engineer or the California Building Code may have requirements that are more stringent.

Differential settlement of the PT-slab foundation due to the static and seismic loads is expected to be on the order of 1½ inches over a distance of 30 feet. The bearing capacity may be increased by one-third for wind or seismic loading. Leighton should review the settlement estimates when final foundation plans and loads for the proposed structures become available.

Resistance to lateral loads will be provided by a combination of friction between the soil and structure interface and passive pressure acting against the vertical portion of the foundations. For calculating lateral resistance, a passive pressure of 250 psf per foot of depth to a maximum of 2,500 psf and a frictional coefficient of 0.30 may be used. Note that the passive and frictional coefficients do not include a factor of safety. The frictional resistance and the passive resistance of the soils can be combined without reduction in determining the total lateral resistance.

The post-tensioned slabs are recommended to be underlain by a synthetic sheeting to serve as a retarder to moisture vapor transmission in areas where a moisture-sensitive floor covering (such as vinyl, tile, or carpet) or equipment is planned. The sheeting is recommended to be a minimum 15-mil thick and consist of polyethylene or similar material that meets or exceeds specifications for ASTM E1745 Class A vapor retarders. Where the vapor retarder is used, 2 inches of sand should be placed above and below to prevent punctures and to aid in the concrete cure. Vapor barrier seams should be overlapped a minimum of 6 inches and taped or otherwise sealed. Floor covering manufacturers should be consulted for specific recommendations based on type of product used.

Soil-moisture change below slabs is the major factor in expansive soil problems. The appropriate personnel involved in the day to day maintenance of the site improvements should be made aware of the potential negative consequences of both excessive watering, as well as allowing expansive soils to become too dry (i.e., the soil will undergo shrinkage as it dries up, followed by swelling during the winter, rainy season or when irrigation is resumed, resulting in distress to improvements and structures). Planters should not be located adjacent to foundations unless they are properly designed such that they do not pond water.

Our experience indicates that use of post-tension slab foundations can generally reduce the potential for drying and shrinkage cracking. However, some cracking should be expected as the concrete cures. Minor cracking is considered normal; however, it is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low slump concrete can reduce the potential for shrinkage cracking.

4.3 Slabs-on-Grade

Concrete slabs may be designed using a modulus of subgrade reaction of 100 pci provided the subgrade is prepared as described in Section 4.1. From a geotechnical standpoint, we recommend slab-on-grade be a minimum 5 inches thick with No. 3 rebar placed at the center of the slab at 24 inches on center in each direction. The structural engineer should design the actual thickness and reinforcement based on anticipated loading conditions. Where moisture-sensitive floor coverings or equipment is planned, the slabs should be protected by a minimum 10-mil-thick vapor barrier between the slab and subgrade. A coefficient of friction of 0.35 can be used between the floor slab and the vapor barrier.

Minor cracking of concrete after curing due to drying and shrinkage is normal and should be expected; however, concrete is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking. Additionally, our experience indicates that the use of reinforcement in slabs and foundations can generally reduce the potential but not eliminate for concrete cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or weakened plane joints at frequent intervals. Joints should be laid out to form approximately square panels.

4.4 Temporary Excavation and Shoring Design

All temporary excavations, including utility trenches, retaining wall excavations and foundation excavations should be performed in accordance with project plans, specifications, and all OSHA requirements. Excavations 5 feet or deeper should be laid back or shored in accordance with OSHA requirements before personnel are allowed to enter.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the cut, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structure.

Typical cantilever shoring for drained conditions should be designed based on the active fluid pressure of 35 pcf. If excavations are braced at the top and at specific design intervals, the active pressure for drained conditions may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to 25H, where H is equal to the depth of the excavation being shored.

Adjacent to existing buildings, shoring should be designed to accommodate the surcharge pressure from existing foundations. A uniform horizontal pressure equal to ½ of the foundation bearing pressure may be assumed for preliminary design.

4.5 Drainage and Landscaping

Building walls below grade should be waterproofed or at least dampproofed, depending upon the degree of moisture protection desired and the depth below grade. Surface drainage should be designed to direct water away from foundations and toward approved drainage devices. Irrigation of landscaping should be controlled to maintain, as much as possible, consistent moisture content sufficient to provide healthy plant growth without overwatering.

5.0 LIMITATIONS

Leighton's work was performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in California at this time. No other warranty, express or implied, is made as to the conclusions and professional opinions included in this report.

This report is issued with the understanding that it is the responsibility of the owner or a duly authorized agent acting on behalf of the owner, to ensure that information and preliminary recommendations contained herein are brought to the attention of the necessary design consultants for this project and incorporated into plans and specifications.

Until reviewed and accepted by the local governing Agency, this report may be subject to change. Changes may be required as part of the Agency review process. Leighton assumes no risk or liability for consequential damages that may arise due to design work progressing before this report is reviewed and accepted by the reviewing Agency.

The findings of this report are considered valid as of the present date. However, changes in the condition of a property can occur with the passage of time, whether due to natural processes or the work of man on the subject or adjacent properties. In addition, changes in standards of practice may occur from legislation or the broadening of knowledge. Accordingly, the findings of this report may at some future time be invalidated wholly or partially by changes outside Leighton's control. Conditions revealed in construction excavations may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by Leighton and additional recommendations may be warranted based on additional observations and findings.

The conclusions and recommendations in this report are based in part upon data that were obtained from a necessarily limited number of observations, site visits, excavations, samples and testes. Such information can be obtained only with respect to the specific locations explored, and therefore may not completely define all subsurface conditions throughout the site. The nature of many sites is that differing geotechnical and/or geological conditions can occur within small distances and under varying climatic conditions. Furthermore, changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report should be considered preliminary if unanticipated conditions are encountered and additional explorations, testing and analyses may be necessary to develop alternative recommendations.

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Project: 13235.001
Eng/Geol: JMP
Scale: 1 " = 2,000 '
Date: September 2021
Base Map: ESRI ArcGIS Online 2021
Author: Leighton Geomatics (btran)

SITE LOCATION MAP

Proposed Residential Development
16300 Euclid Street
Fountain Valley, California

FIGURE 1

Leighton

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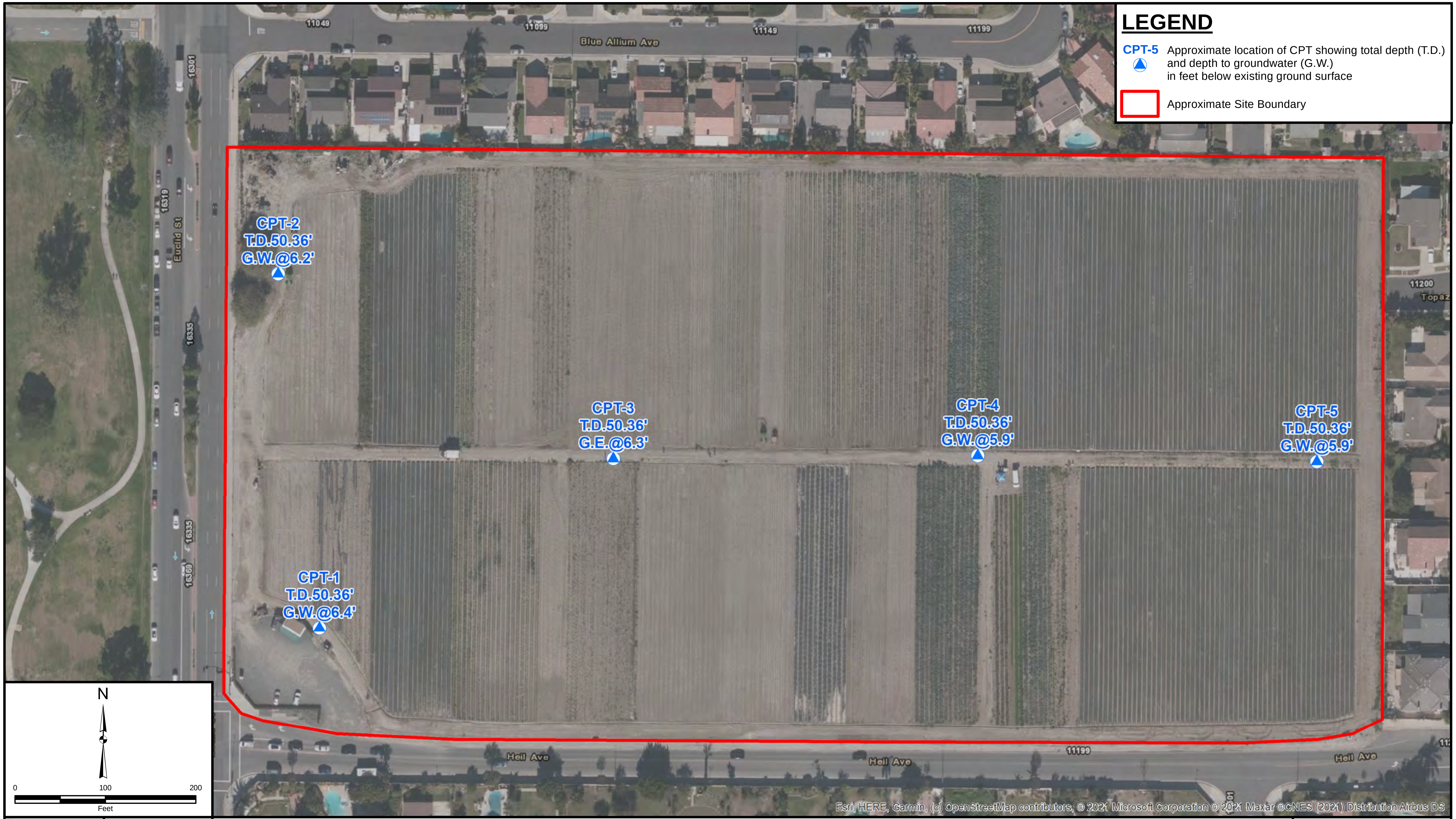

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Base Map: ESRI ArcGIS Online 2021

Author: Leighton Geomatics (btran)

BORING LOCATION MAP

Proposed Residential Development
16300 Euclid Street
Fountain Valley, California

FIGURE 2

Leighton

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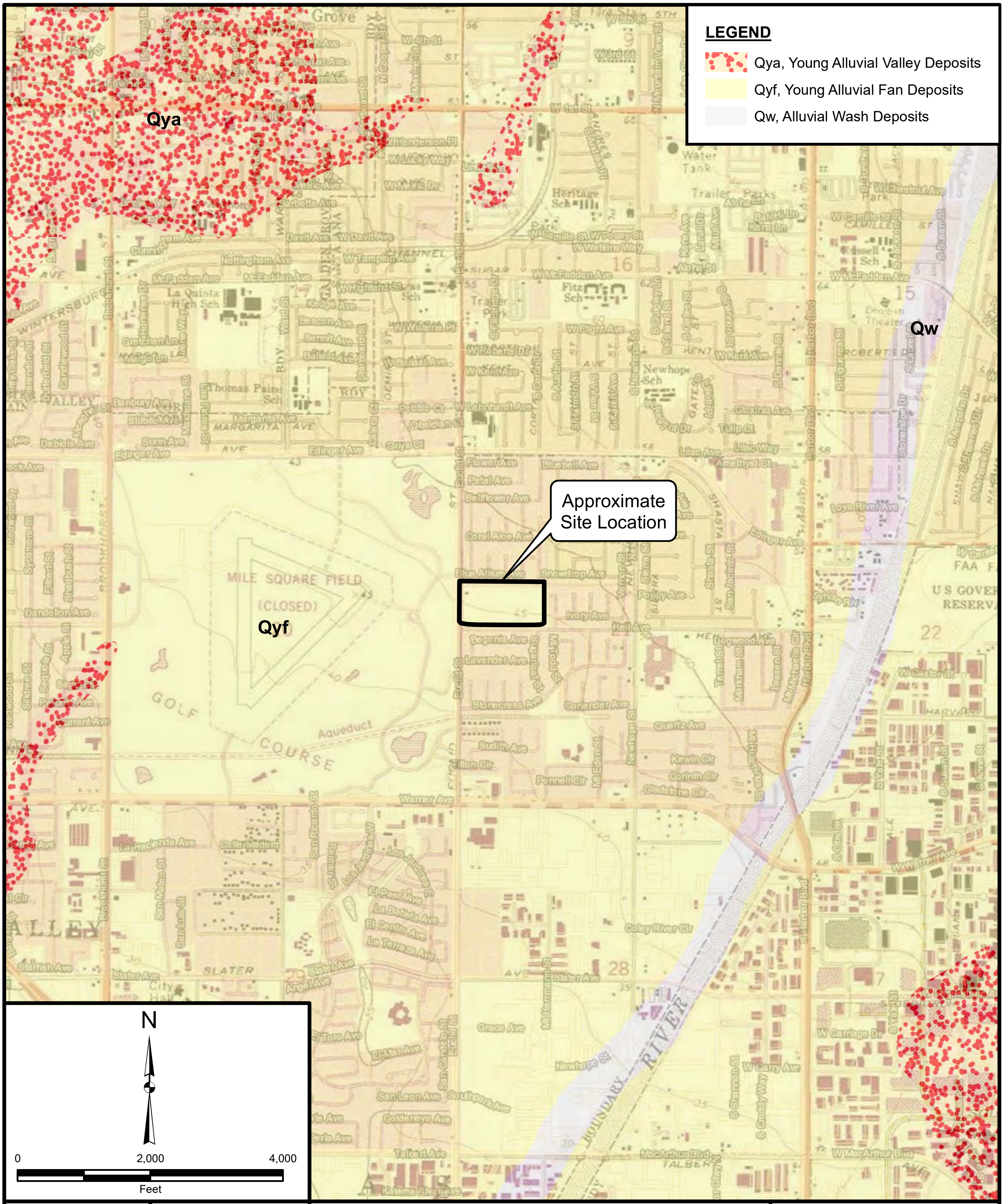

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Reference: ESRI ArcGIS Online 2021, USGS

Author: Leighton Geomatics (btran)

REGIONAL GEOLOGY MAP

Proposed Residential Development
16300 Euclid Street
Fountain Valley, California

FIGURE 3

Leighton

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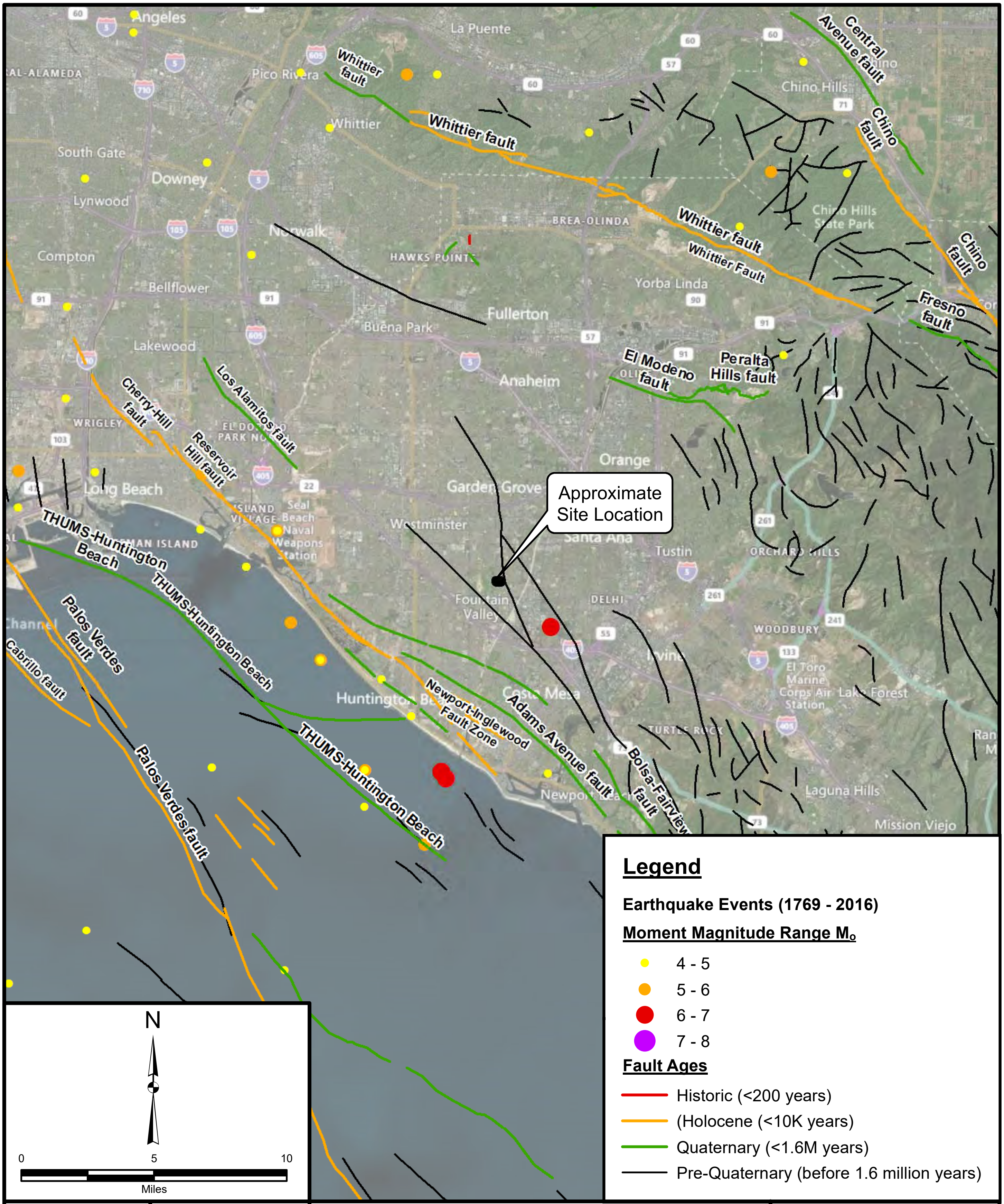

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Reference: ESRI ArcGIS Online 2021
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Author: Leighton Geomatics (btran)

REGIONAL FAULT AND HISTORICAL SEISMICITY MAP

Proposed Residential Development
16300 Euclid Street
Fountain Valley, California

FIGURE 4

Leighton

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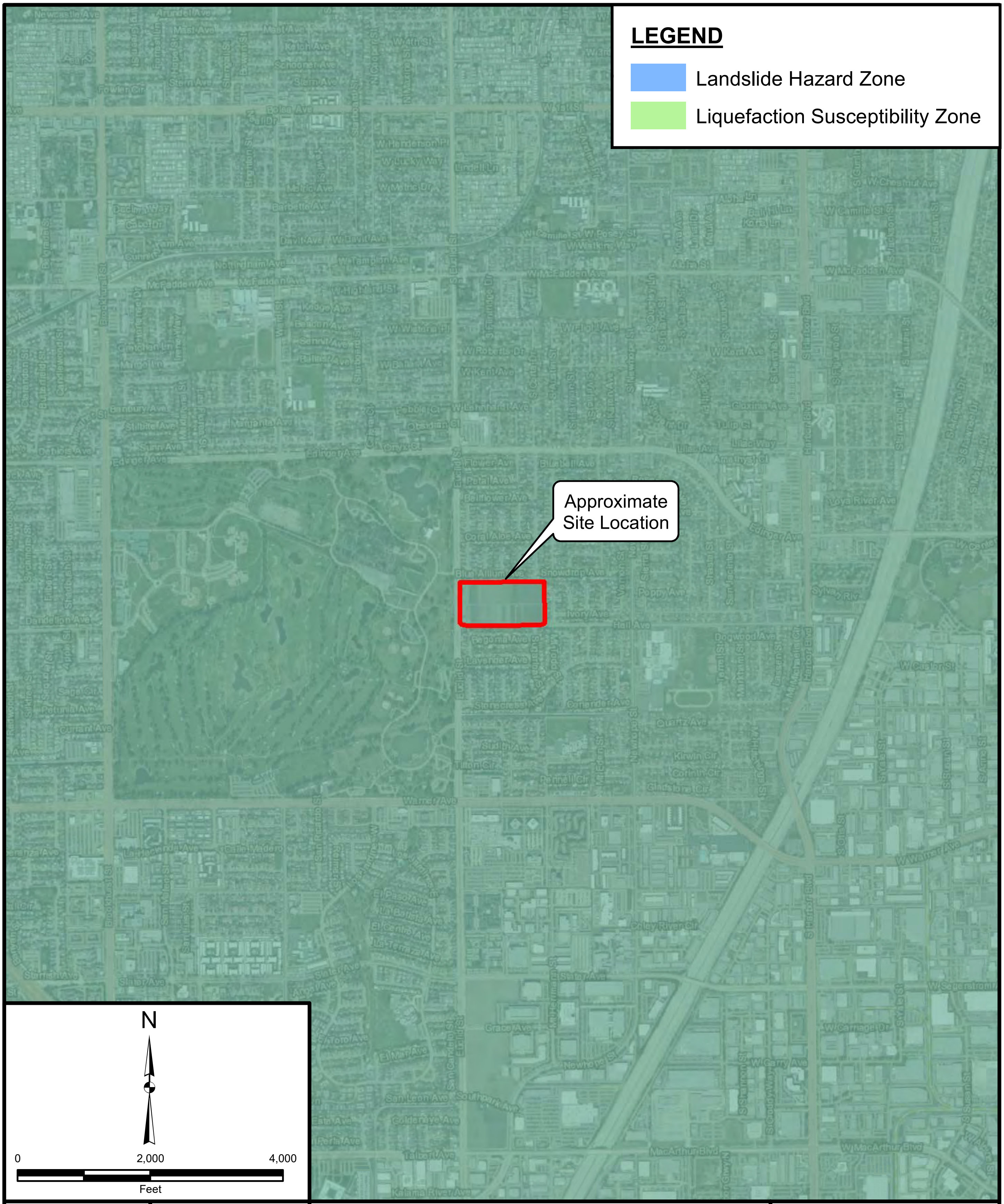

LEGEND

🟦 Landslide Hazard Zone

🟩 Liquefaction Susceptibility Zone

Project: 13255.001	Eng/Geol: JMP
Scale:1 " = 2,000 '	Date: September 2021

Base Map: ESRI ArcGIS Online 2021

Author: Leighton Geomatics (btran)

SEISMIC HAZARD MAP

Proposed Residential Development
16300 Euclid Street
Fountain Valley, California

FIGURE 5

Leighton

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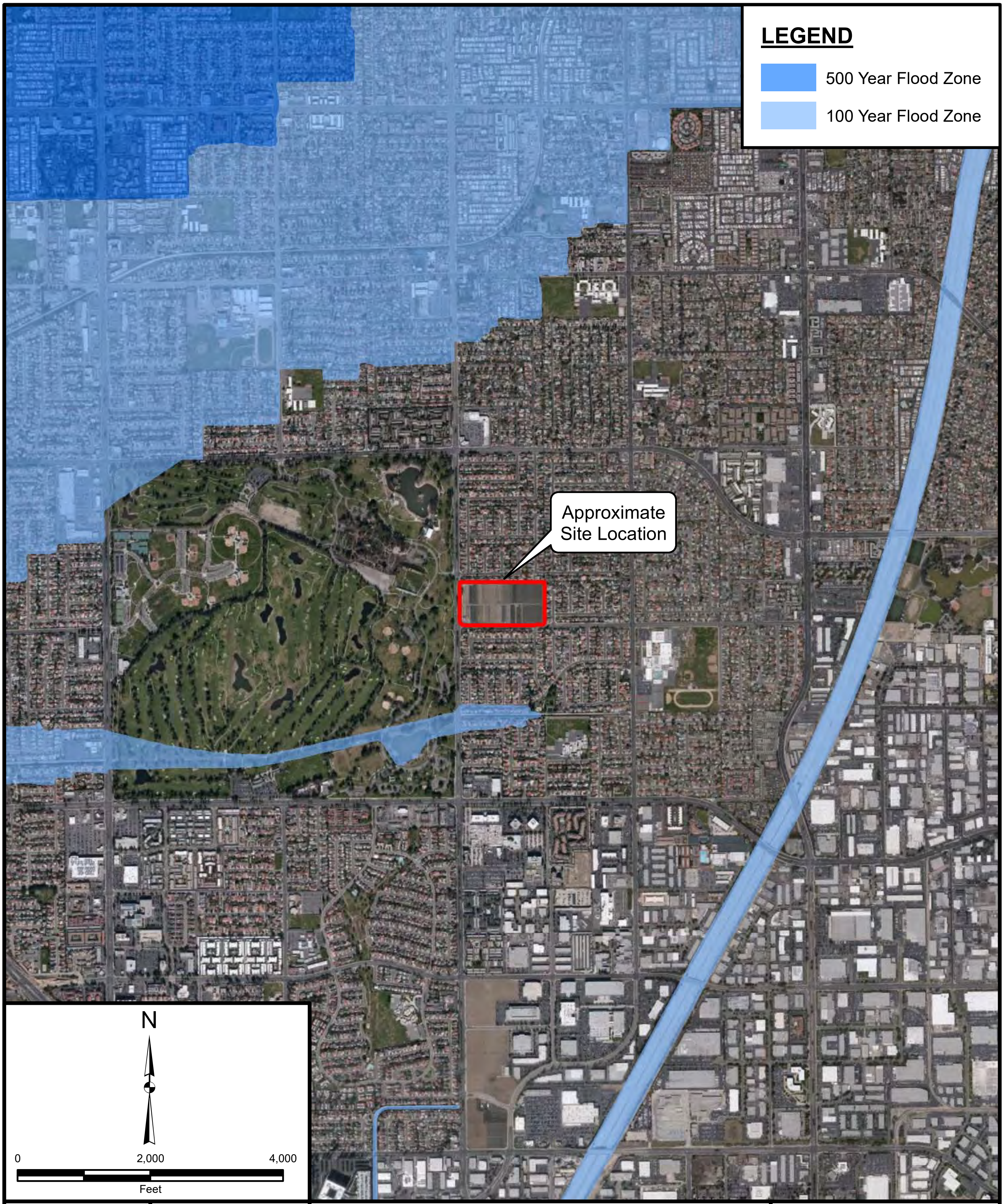

Project: 13255.001

Eng/Geol: JMP

Scale:1 " = 2,000 '

Date: September 2021

Base Map: ESRI ArcGIS Online 2021
Reference: CA DWR, FEMA
Author: Leighton Geomatics (btran)

FLOOD HAZARD ZONE MAP

Proposed Residential Development
16300 Euclid Street
Fountain Valley, California

FIGURE 6

Leighton

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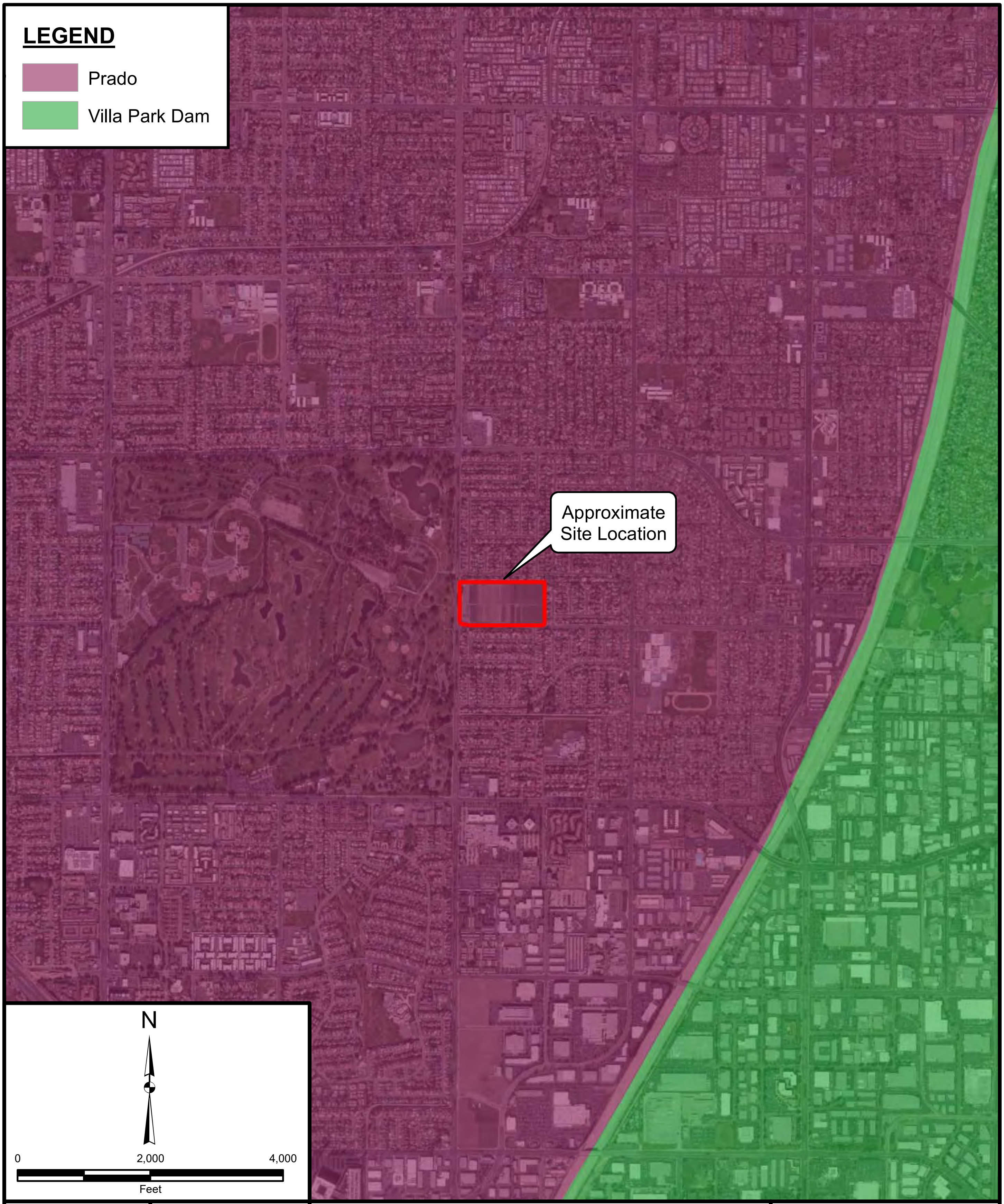

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Eng/Geol: JMP
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Date: September 2021
Base Map: ESRI ArcGIS Online 2021
Reference: CA DWR, FEMA
Author: Leighton Geomatics (btran)

DAM INUNDATION MAP

Proposed Residential Development
16300 Euclid Street
Fountain Valley, California

FIGURE 7

Leighton

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APPENDIX A

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APPENDIX A

REFERENCES

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APPENDIX B

CONE PENETROMETER TEST (CPT) LOGS

GREGG

GREGG DRILLING, LLC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

September 3, 2021

Leighton Consulting
Attn: Michelle M.

Subject: CPT Site Investigation
Shop Off
Fountain Valley, California
GREGG Project Number: D1215078

Dear Michelle:

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

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

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact me at 949-903-6873.

Sincerely,
Gregg Drilling, LLC.

CPT Reports Team
Gregg Drilling, LLC.

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
CPT-01	9/3/2021	50.36	-	-	50.4
CPT-02	9/3/2021	50.36	-	-	18.7
CPT-03	9/3/2021	50.36	-	-	-
CPT-05	9/3/2021	50.36	-	-	-
SCPT-04	9/3/2021	50.36	-	-	23.1

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

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (*PPDT*). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

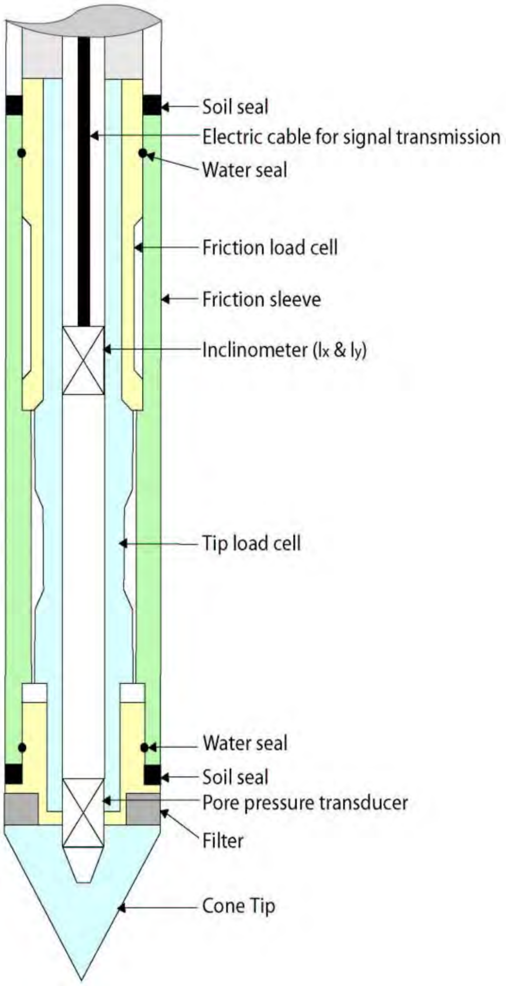


Figure CPT

Gregg 15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.80
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

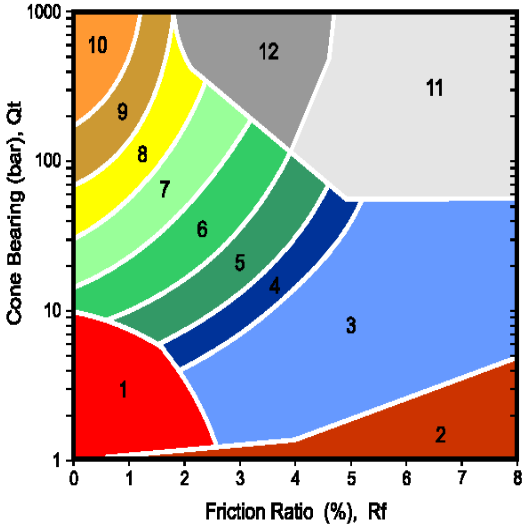
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



ZONE		SBT
1		🟥 Sensitive, fine grained
2		🟫 Organic materials
3		🟦 Clay
4		🟦 Silty clay to clay
5		🟩 Clayey silt to silty clay
6		🟩 Sandy silt to clayey silt
7		🟨 Silty sand to sandy silt
8		🟨 Sand to silty sand
9		🟫 Sand
10		🟧 Gravely sand to sand
11		⬜ Very stiff fine grained*
12		⬛ Sand to clayey sand*

*over consolidated or cemented

Figure SBT (After Robertson et al., 1986) – Note: Colors may vary slightly compared to plots

Cone Penetration Test (CPT) Interpretation

Gregg uses a proprietary CPT interpretation and plotting software. The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

1 Units for display (Imperial or metric) (atm. pressure, p_a = 0.96 tsf or 0.1 MPa)
2 Depth interval to average results (ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
3 Elevation of ground surface (ft or m)
4 Depth to water table, z_w (ft or m) – input required
5 Net area ratio for cone, a (default to 0.80)
6 Relative Density constant, C_{Dr} (default to 350)
7 Young's modulus number for sands, α (default to 5)
8 Small strain shear modulus number
    a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
    b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
9 Undrained shear strength cone factor for clays, N_{kt} (default to 15)
10 Over Consolidation ratio number, k_{ocr} (default to 0.3)
11 Unit weight of water, (default to γ_w = 62.4 lb/ft^3 or 9.81 kN/m^3)

Column

1 Depth, z, (m) – CPT data is collected in meters
2 Depth (ft)
3 Cone resistance, q_c (tsf or MPa)
4 Sleeve resistance, f_s (tsf or MPa)
5 Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
6 Other – any additional data
7 Total cone resistance, q_t (tsf or MPa) $q_t = q_c + u\,(1\text{-}a)$

8	Friction Ratio, R_f (%)	$R_f = (f_s/q_t) \times 100\%$
9	Soil Behavior Type (non-normalized), SBT	see note
10	Unit weight, γ (pcf or kN/m^3)	based on SBT, see note
11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \sigma\ z$
12	In-situ pore pressure, u_o (tsf)	$u_o = \gamma_w\ (z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u - u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with I_c)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60}, blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r, (%)	see note
24	Estimated Friction Angle, ϕ', (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, Go (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u/σ_v'
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

1 Soil Behavior Type (non-normalized), SBT (Lunne et al., 1997 and table below)

2 Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)

3 Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)

4 SBT_n Index, I_c $I_c = ((3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$

5 Normalized Cone resistance, Q_{tn} (n varies with Ic)

$Q_{tn} = ((q_t - \sigma_{vo})/pa)\ (pa/(\sigma'_{vo})^n$ and recalculate I_c, then iterate:

When $I_c < 1.64$, $n = 0.5$ (clean sand)
When $I_c > 3.30$, $n = 1.0$ (clays)
When $1.64 < I_c < 3.30$, $n = (I_c - 1.64)0.3 + 0.5$
Iterate until the change in n, $\Delta n < 0.01$

6 Estimated permeability, k_{SBT} based on Normalized SBT_n (Lunne et al., 1997 and table below)

7 Equivalent SPT N_{60}, blows/ft Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1 - \frac{I_c}{4.6}\right)$$

8 Equivalent SPT $(N_1)_{60}$ blows/ft $(N_1)_{60} = N_{60}\,C_{N,}$
where $C_N = (pa/\sigma'_{vo})^{0.5}$

9 Relative Density, D_r, (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

10 Friction Angle, ϕ', (degrees) $\tan\phi' = \frac{1}{2.68}\left[\log\left(\frac{q_c}{\sigma'_{vo}}\right) + 0.29\right]$

Only SBT_n 5, 6, 7 & 8 *Show'N/A' in zones 1, 2, 3, 4 & 9*

11 Young's modulus, E_s $E_s = \alpha\, q_t$
Only SBT_n 5, 6, 7 & 8 *Show 'N/A' in zones 1, 2, 3, 4 & 9*

12 Small strain shear modulus, Go
a. $G_o = S_G\,(q_t\ \ \sigma'_{vo}\ pa)^{1/3}$ *For SBT_n 5, 6, 7*
b. $G_o = C_G\, q_t$ *For SBT_n 1, 2, 3& 4*
Show 'N/A' in zones 8 & 9

13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

14 Over Consolidation ratio, OCR $OCR = k_{ocr}\, Q_{t1}$
Only SBT_n 1, 2, 3, 4 & 9 *Show 'N/A' in zones 5, 6, 7 & 8*

The following updated and simplified SBT descriptions have been used in the software:

SBT Zones		**SBT_n Zones**	
1	sensitive fine grained	1	sensitive fine grained
2	organic soil	2	organic soil
3	clay	3	clay
4	clay & silty clay	4	clay & silty clay
5	clay & silty clay		
6	sandy silt & clayey silt		

7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

*heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print ‘clays & silty clays’)

Estimated Permeability (see Lunne et al., 1997)

SBT_n	Permeability (ft/sec)	(m/sec)
1	$3x\ 10^{-8}$	$1x\ 10^{-8}$
2	$3x\ 10^{-7}$	$1x\ 10^{-7}$
3	$1x\ 10^{-9}$	$3x\ 10^{-10}$
4	$3x\ 10^{-8}$	$1x\ 10^{-8}$
5	$3x\ 10^{-6}$	$1x\ 10^{-6}$
6	$3x\ 10^{-4}$	$1x\ 10^{-4}$
7	$3x\ 10^{-2}$	$1x\ 10^{-2}$
8	$3x\ 10^{-6}$	$1x\ 10^{-6}$
9	$1x\ 10^{-8}$	$3x\ 10^{-9}$

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft^3)	(kN/m^3)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0

Pore Pressure Dissipation Tests (PPDT)

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u)* with time is measured behind the tip of the cone and recorded.

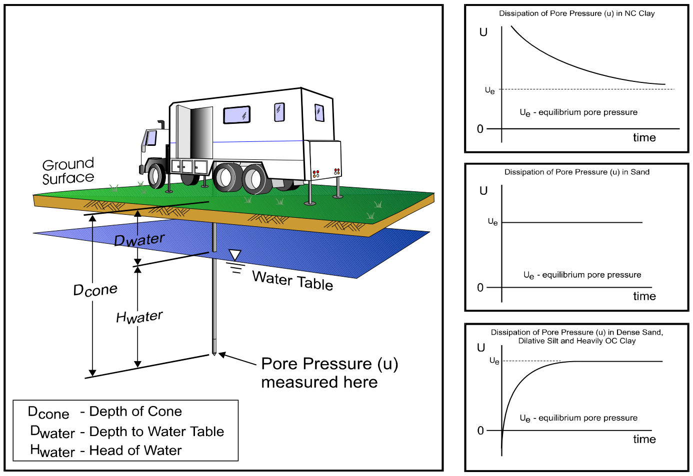
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests are summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors:
1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Seismic Cone Penetration Testing (SCPT)

Seismic Cone Penetration Testing (SCPT) can be conducted at various intervals during the Cone Penetration Test. Shear wave velocity (Vs) can then be calculated over a specified interval with depth. A small interval for seismic testing, such as 1-1.5m (3-5ft) allows for a detailed look at the shear wave profile with depth. Conversely, a larger interval such as 3-6m (10-20ft) allows for a more average shear wave velocity to be calculated. Gregg's cones have a horizontally active geophone located 0.2m (0.66ft) behind the tip.

To conduct the seismic shear wave test, the penetration of the cone is stopped and the rods are decoupled from the rig. An automatic hammer is triggered to send a shear wave into the soil. The distance from the source to the cone is calculated knowing the total depth of the cone and the horizontal offset distance between the source and the cone. To calculate an interval velocity, a minimum of two tests must be performed at two different depths. The arrival times between the two wave traces are compared to obtain the difference in time (Δt). The difference in depth is calculated (Δd) and velocity can be determined using the simple equation: $v = \Delta d / \Delta t$

Multiple wave traces can be recorded at the same depth to improve quality of the data.

A complete reference on seismic cone penetration tests is presented by Robertson et al. 1986 and Lunne et al. 1997.

A summary the shear wave velocities, arrival times and wave traces are provided with the report.

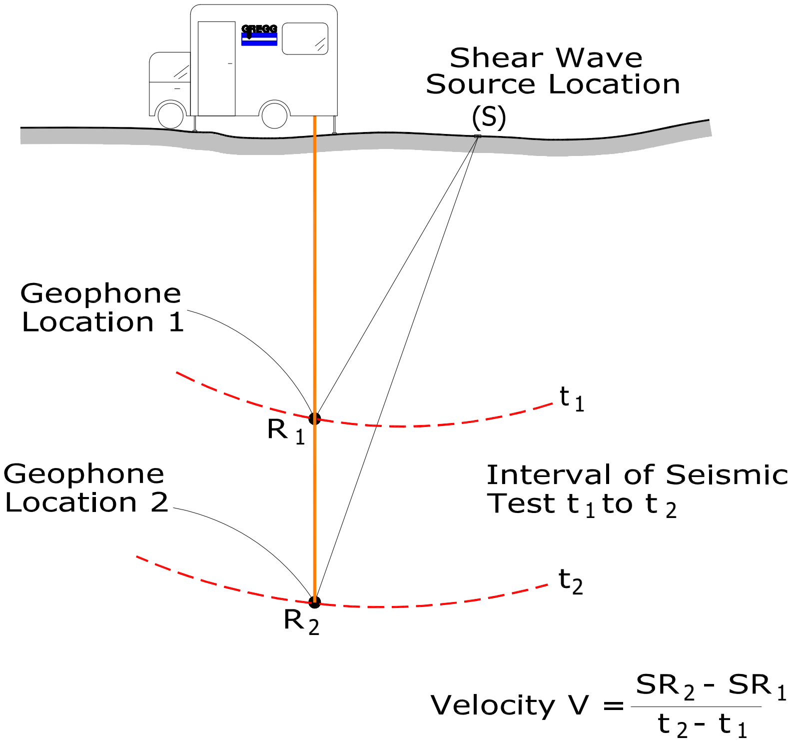


Figure SCPT

Groundwater Sampling

Gregg Drilling & Testing, Inc. conducts groundwater sampling using a sampler as shown in *Figure GWS*. The groundwater sampler has a retrievable stainless steel or disposable PVC screen with steel drop off tip. This allows for samples to be taken at multiple depth intervals within the same sounding location. In areas of slower water recharge, provisions may be made to set temporary PVC well screens during sampling to allow the pushing equipment to advance to the next sample location while the groundwater is allowed to infiltrate.

The groundwater sampler operates by advancing 44.5mm (1¾ inch) hollow push rods with the filter tip in a closed configuration to the base of the desired sampling interval. Once at the desired sample depth, the push rods are retracted; exposing the encased filter screen and allowing groundwater to infiltrate hydrostatically from the formation into the inlet screen. A small diameter bailer (approximately ½ or ¾ inch) is lowered through the push rods into the screen section for sample collection. The number of downhole trips with the bailer and time necessary to complete the sample collection at each depth interval is a function of sampling protocols, volume requirements, and the yield characteristics and storage capacity of the formation. Upon completion of sample collection, the push rods and sampler, with the exception of the PVC screen and steel drop off tip are retrieved to the ground surface, decontaminated and prepared for the next sampling event.

For a detailed reference on direct push groundwater sampling, refer to Zemo et. al., 1992.

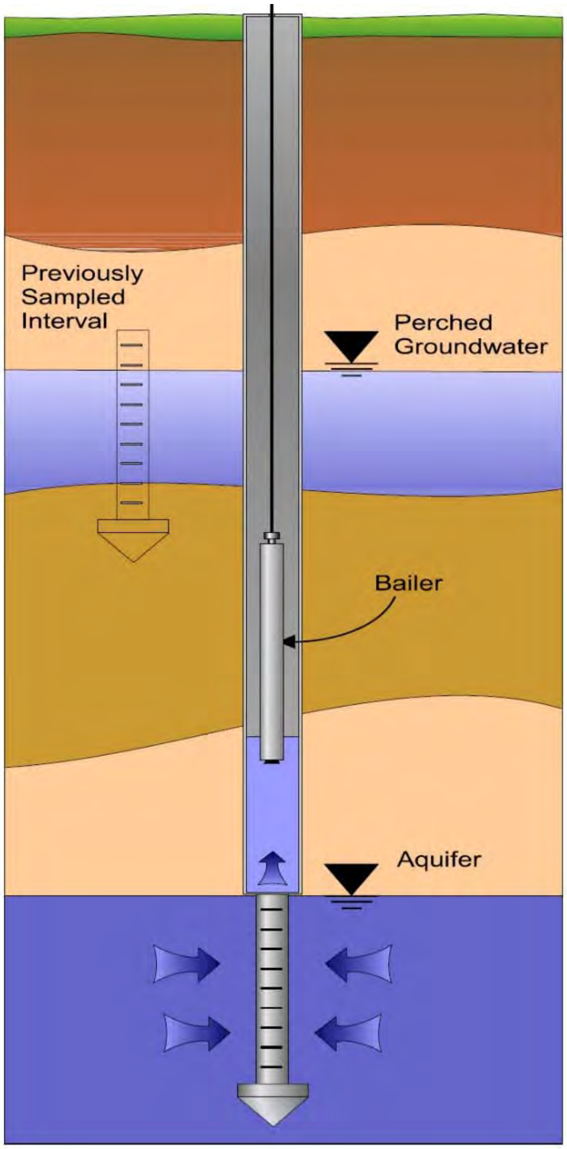


Figure GWS

Soil Sampling

Gregg Drilling & Testing, Inc. uses a piston-type push-in sampler to obtain small soil samples without generating any soil cuttings, *Figure SS*. Two different types of samplers (12 and 18 inch) are used depending on the soil type and density. The soil sampler is initially pushed in a "closed" position to the desired sampling interval using the CPT pushing equipment. Keeping the sampler closed minimizes the potential of cross contamination. The inner tip of the sampler is then retracted leaving a hollow soil sampler with inner 1¼" diameter sample tubes. The hollow sampler is then pushed in a locked "open" position to collect a soil sample. The filled sampler and push rods are then retrieved to the ground surface. Because the soil enters the sampler at a constant rate, the opportunity for 100% recovery is increased. For environmental analysis, the soil sample tube ends are sealed with Teflon and plastic caps. Often, a longer "split tube" can be used for geotechnical sampling.

For a detailed reference on direct push soil sampling, refer to Robertson et al, 1998.

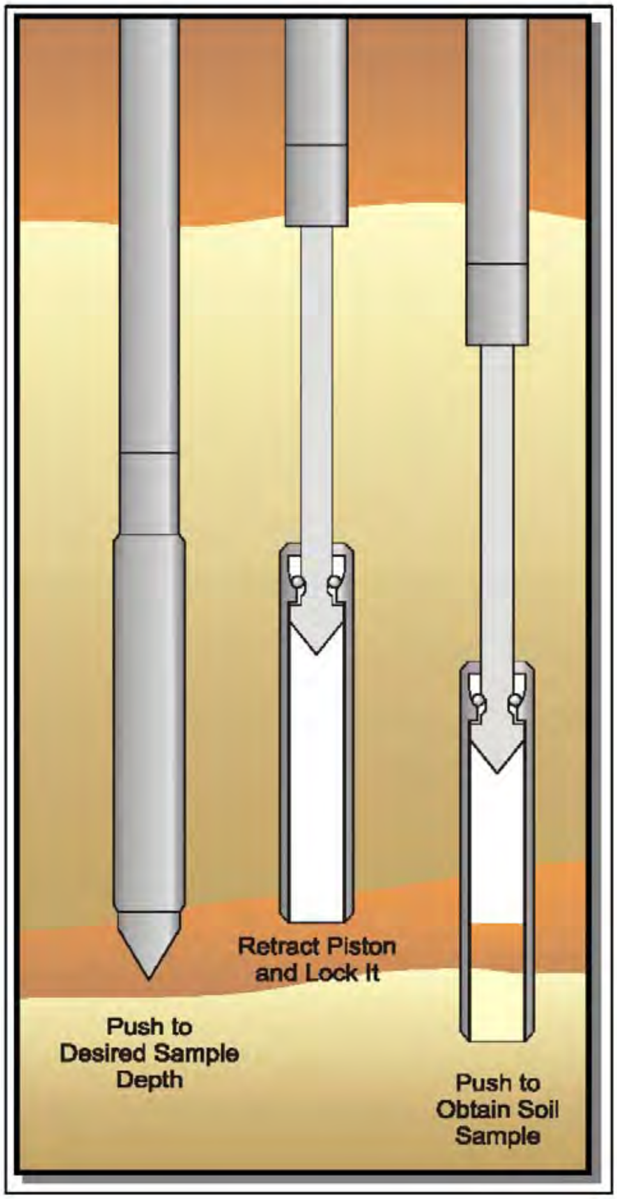


Figure SS

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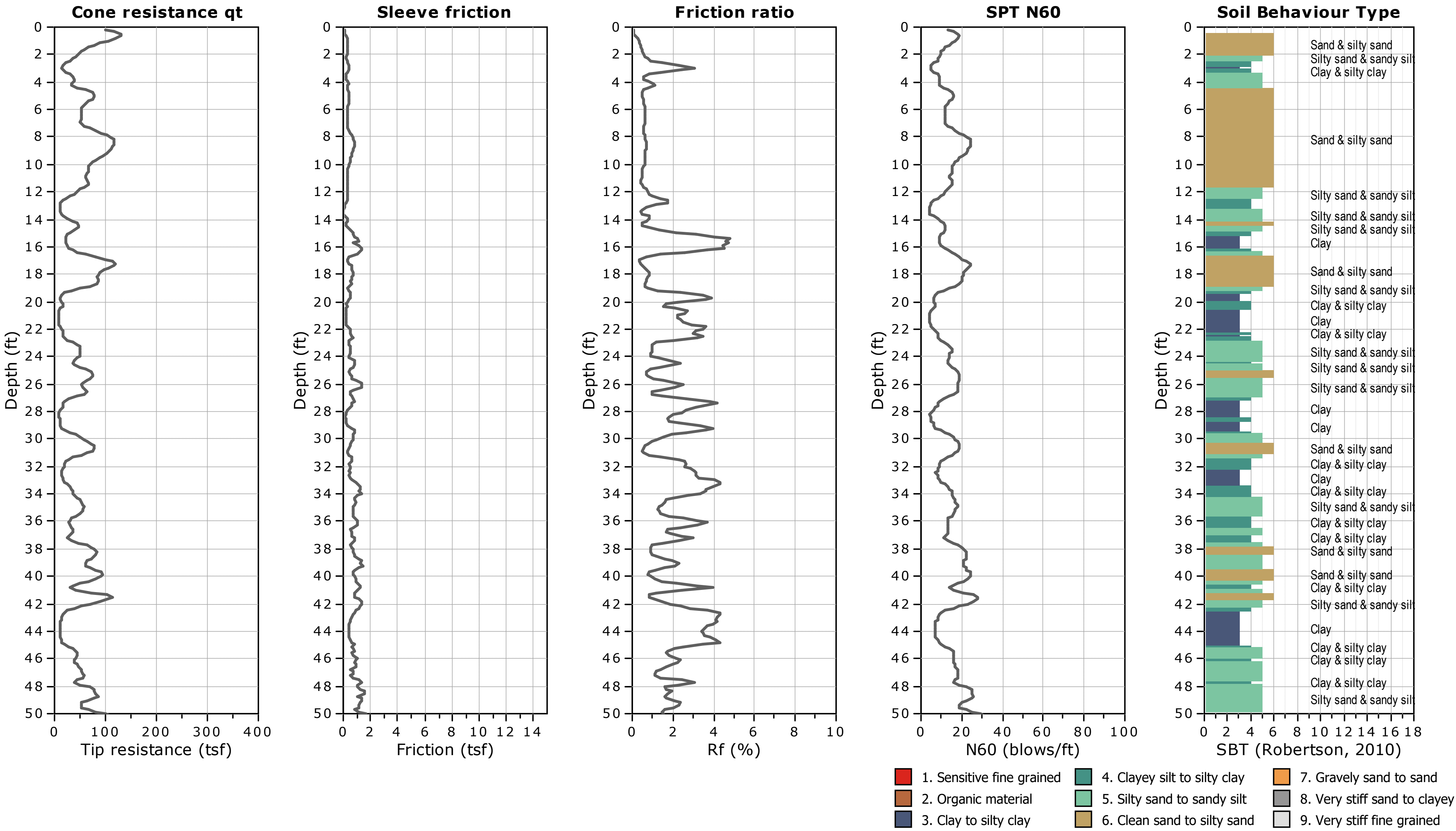
CPT: CPT-01

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



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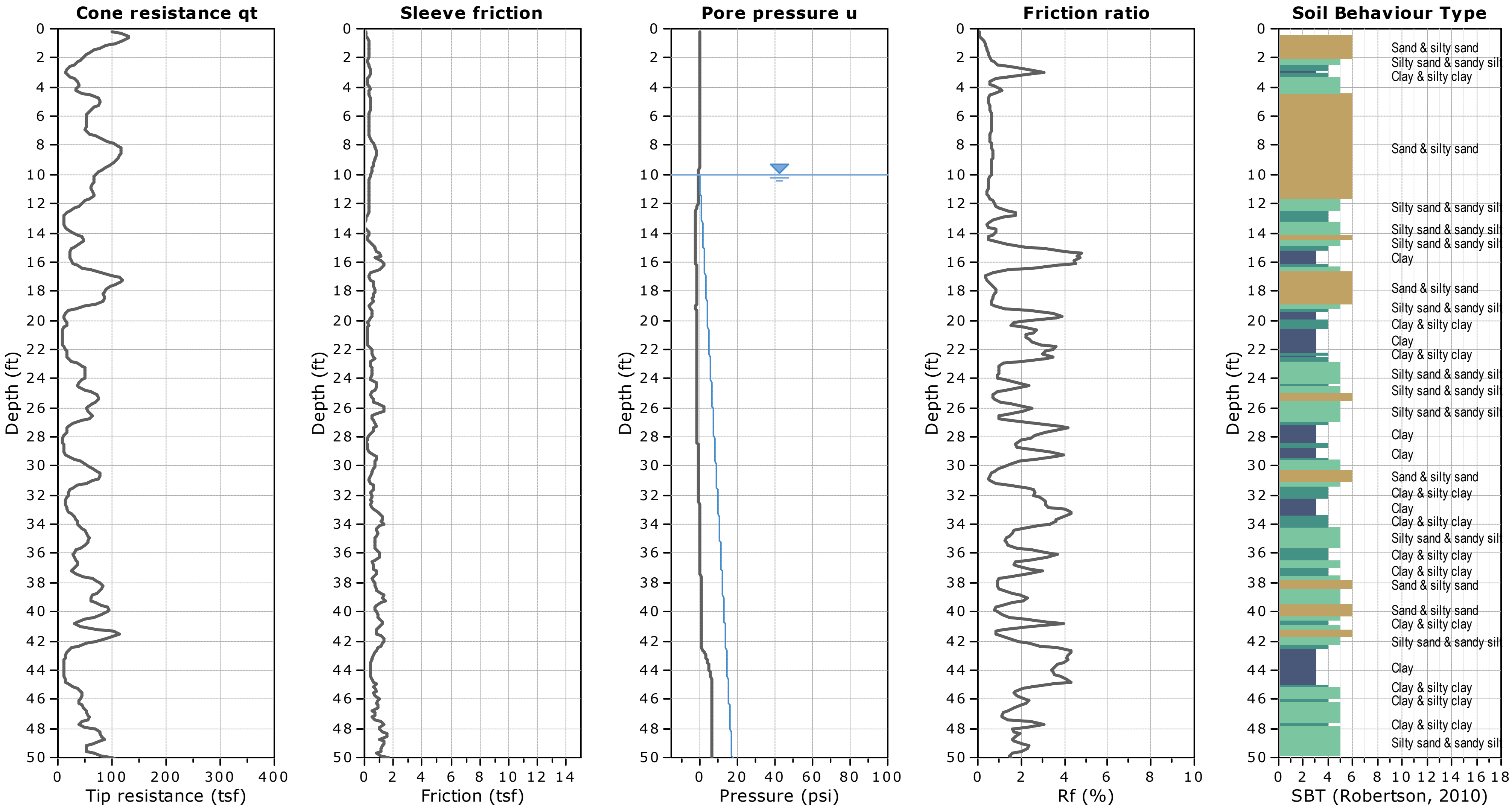
CPT: CPT-01

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey
⬜ 9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

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Project file: C:\CPT-2021\5078SH\REPORT\215078SH.cpt

GREGG DRILLING, LLC
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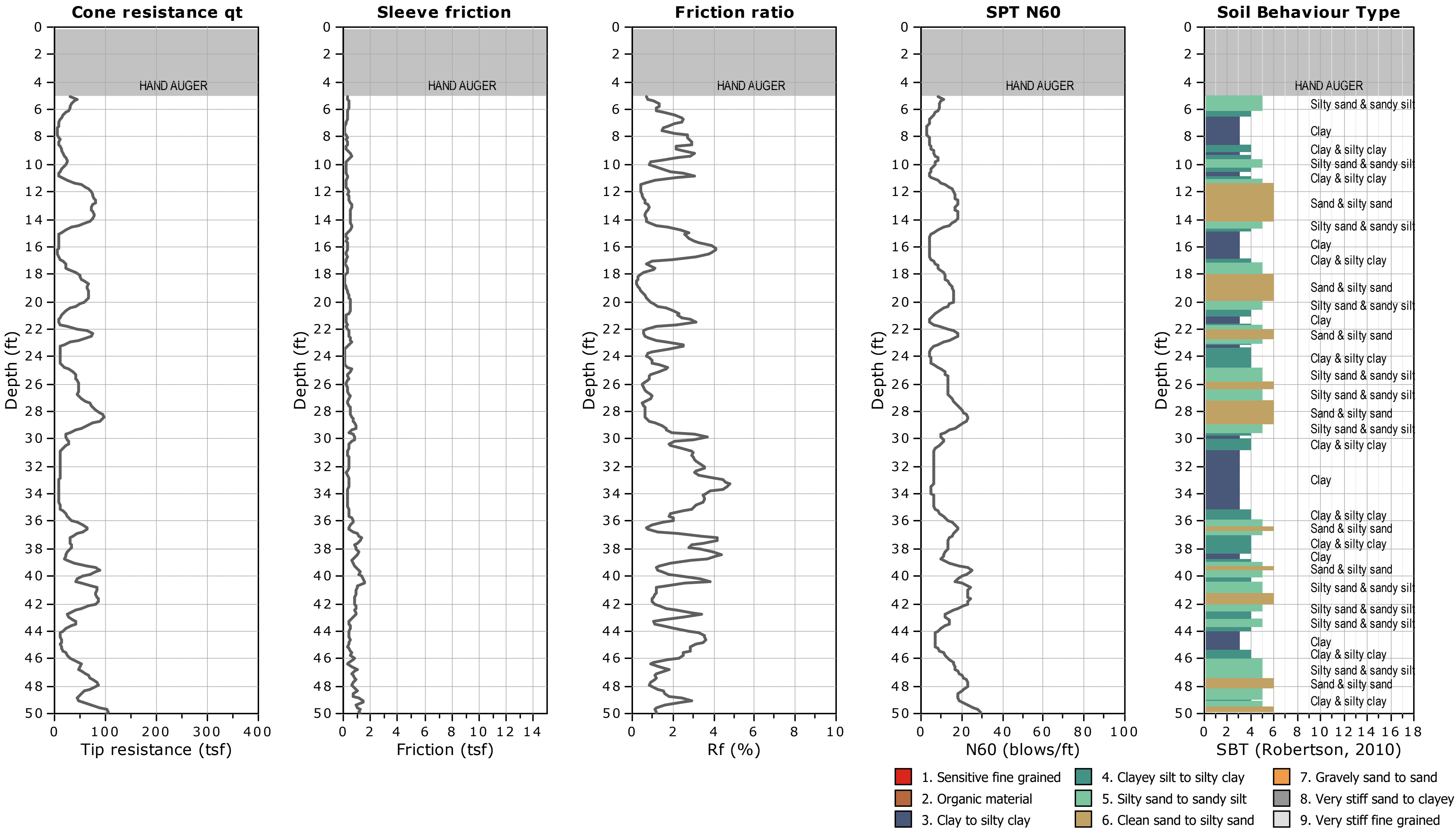
CPT: CPT-02

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



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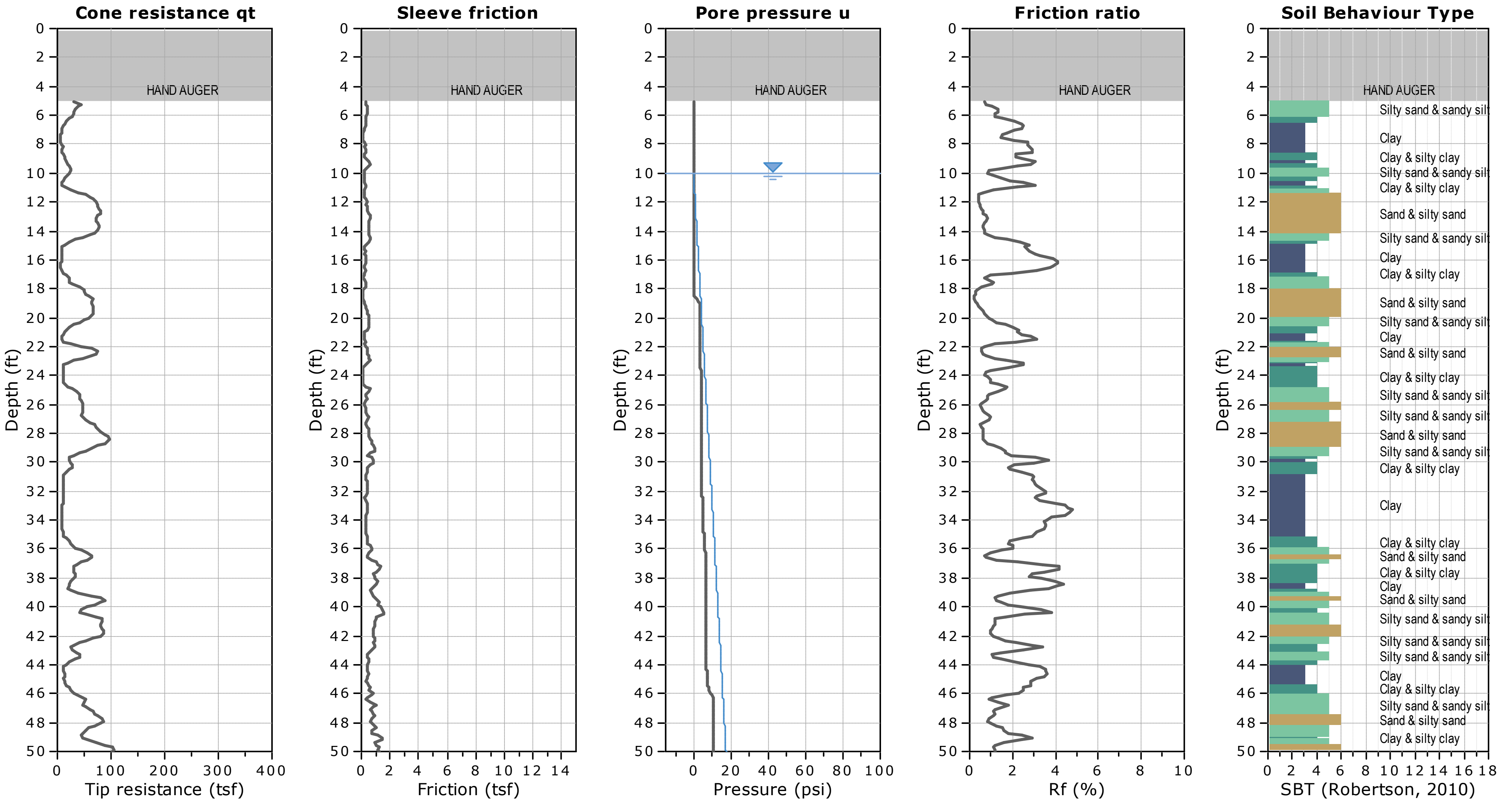
CPT: CPT-02

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



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

WATER TABLE FOR ESTIMATING PURPOSES ONLY

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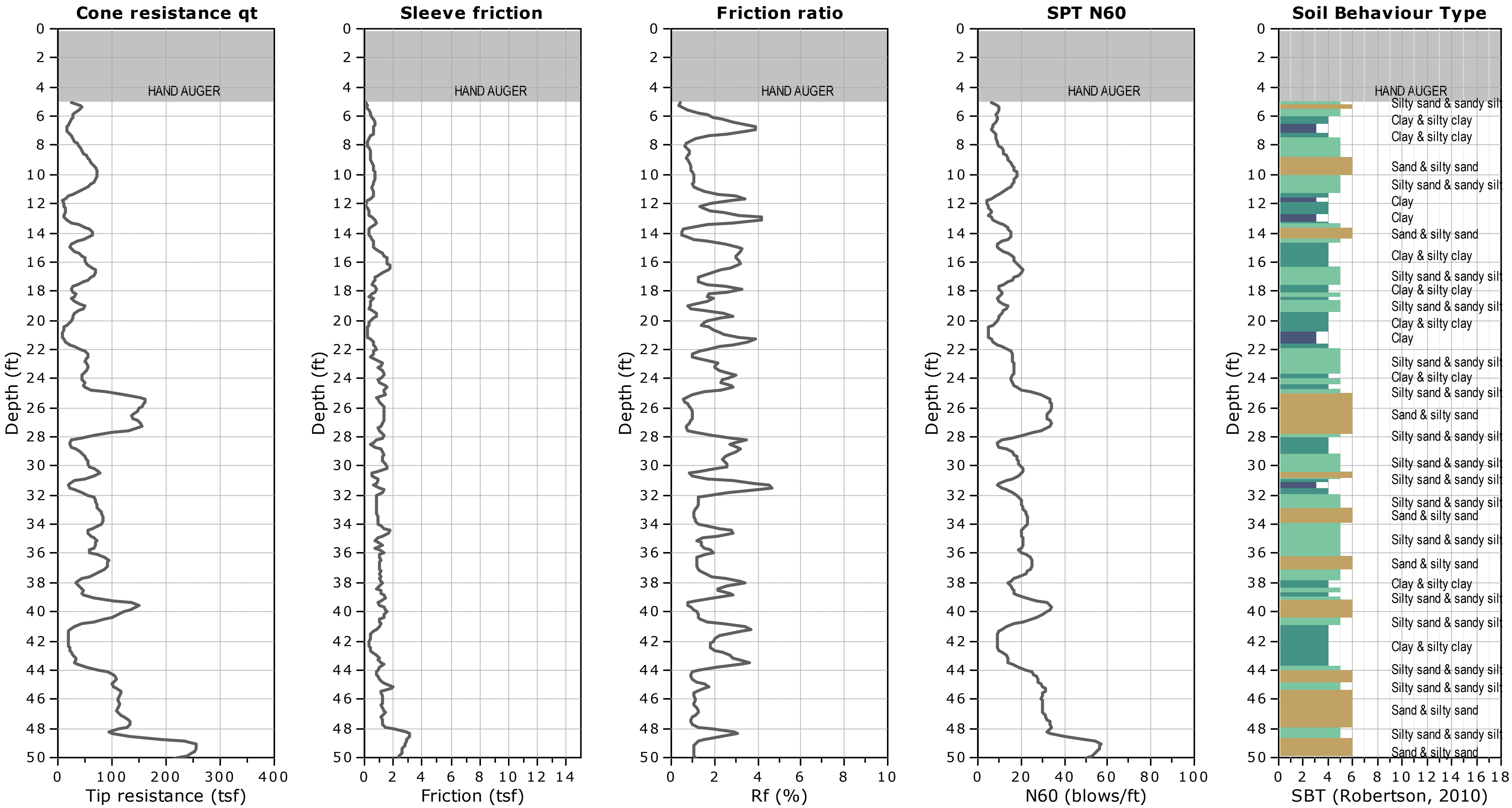
CPT: CPT-03

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty clay	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

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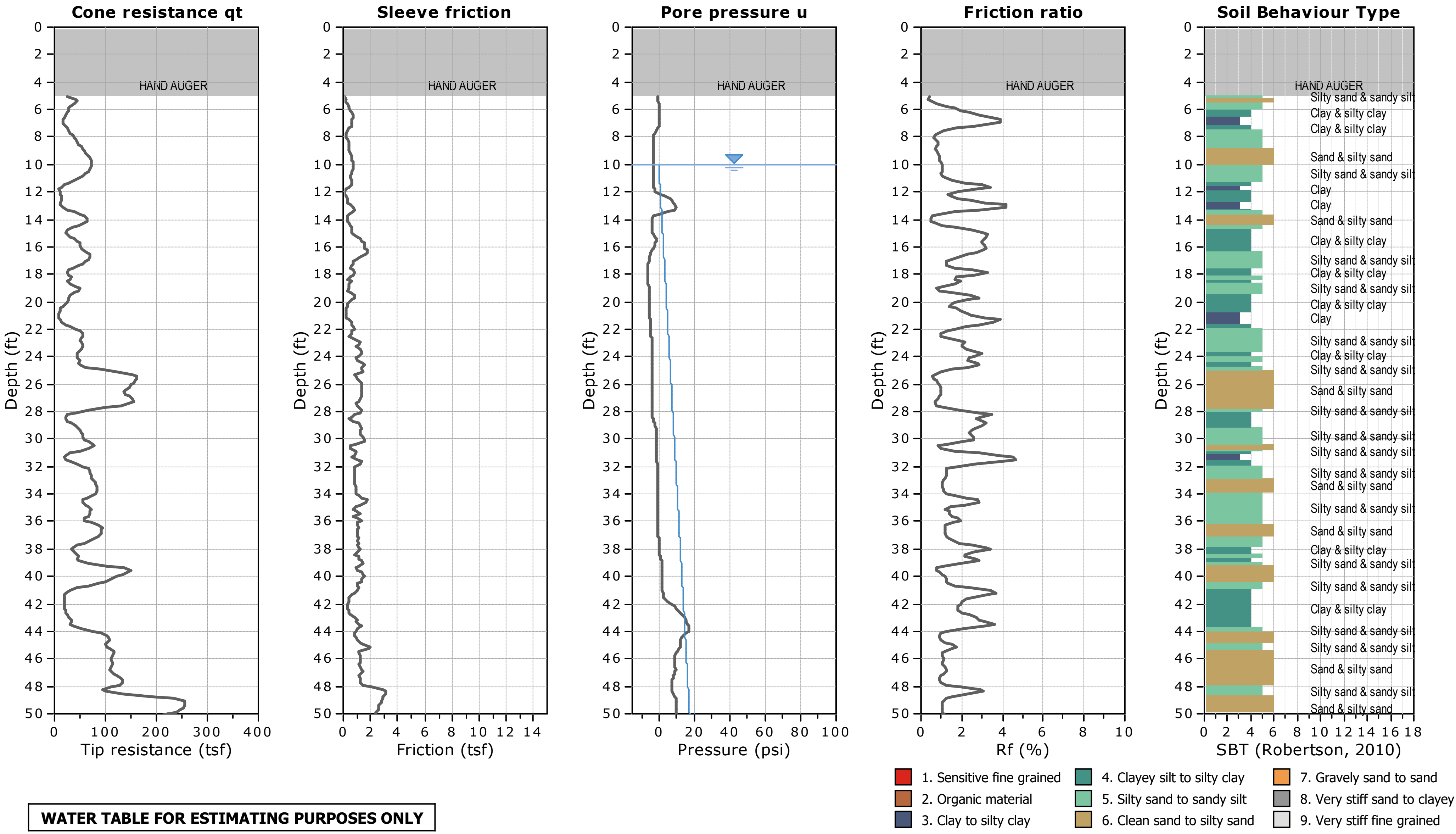
CPT: CPT-03

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



WATER TABLE FOR ESTIMATING PURPOSES ONLY

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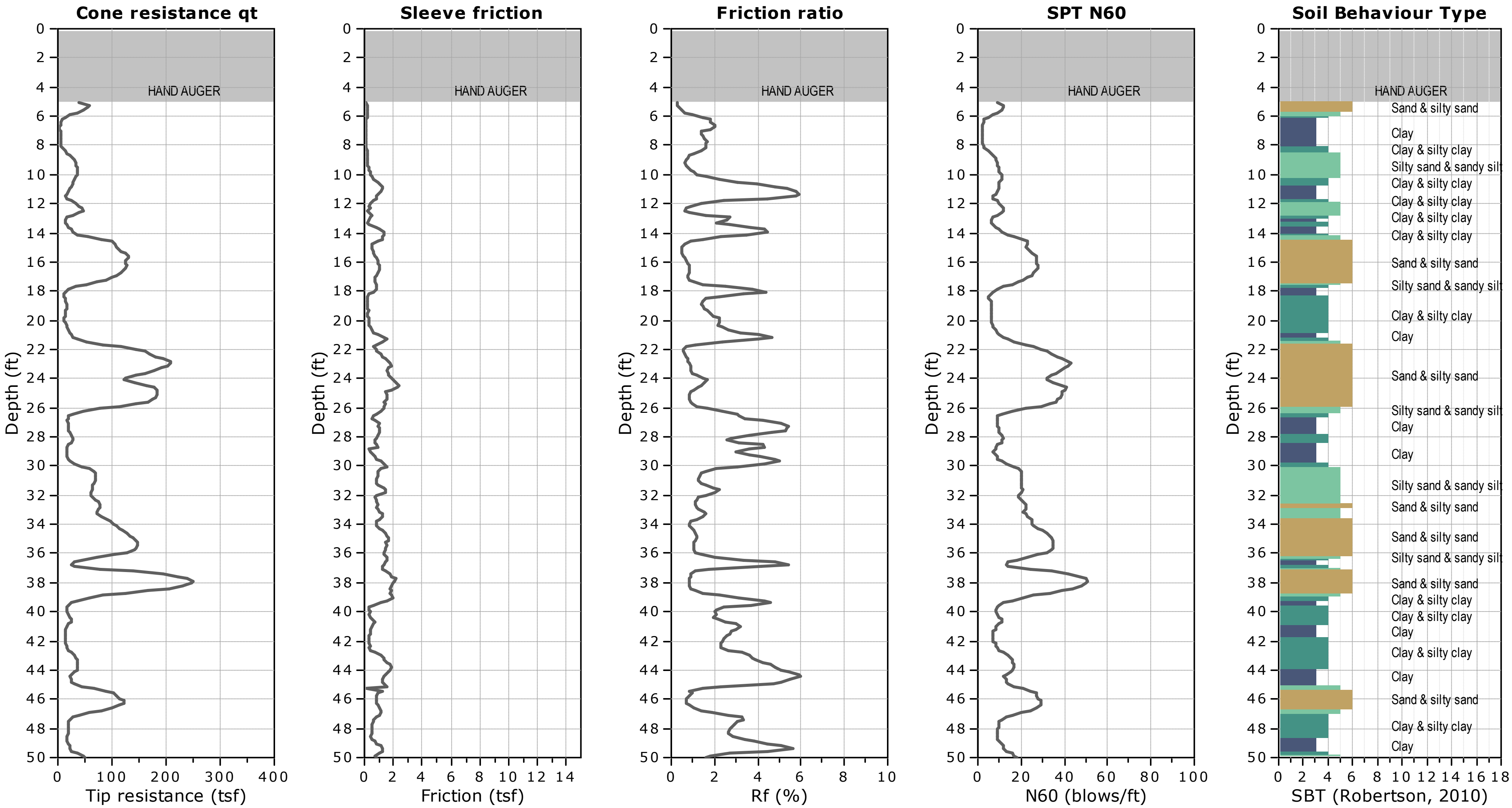
CPT: CPT-05

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey
⬜ 9. Very stiff fine grained

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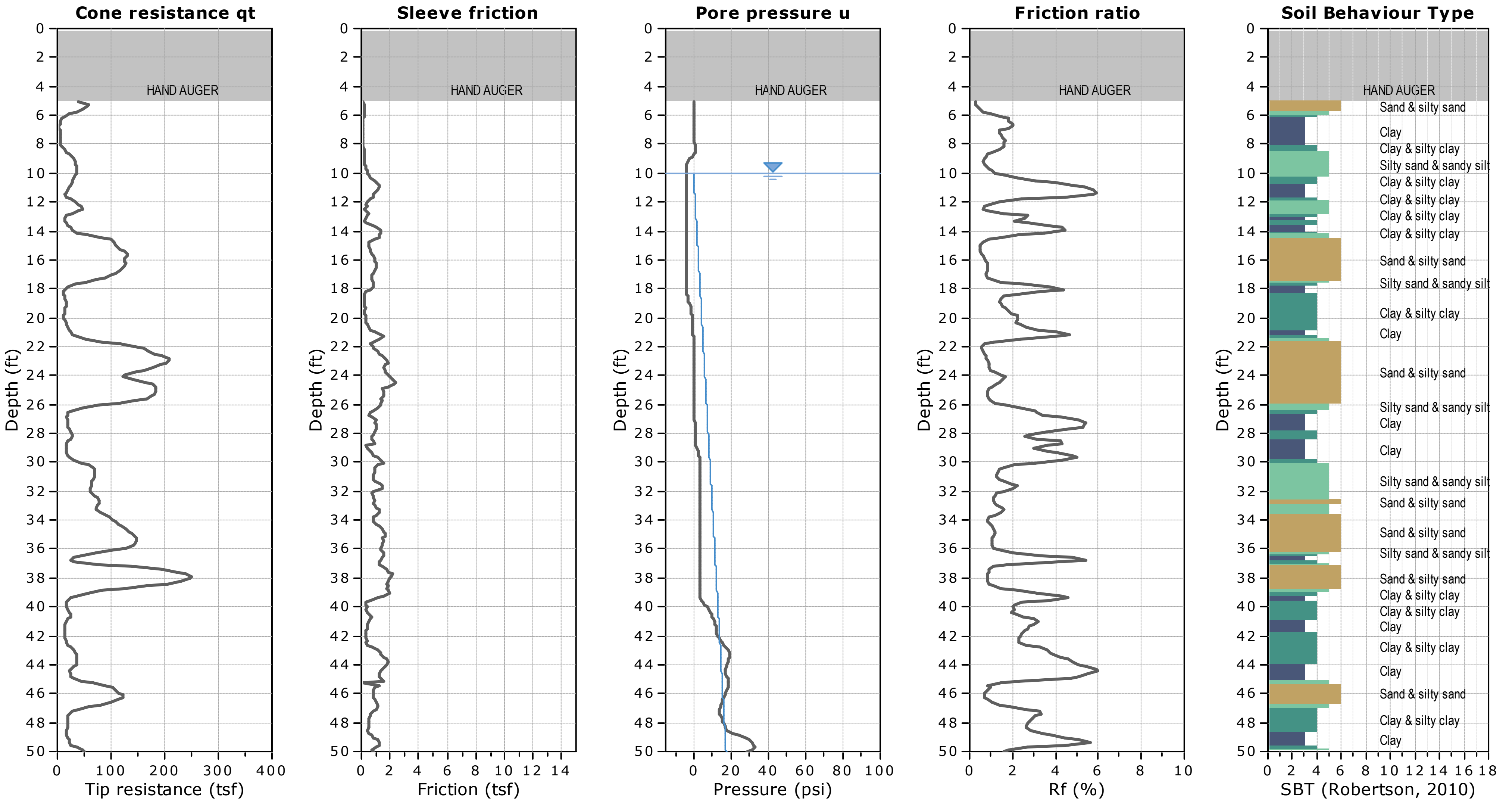
CPT: CPT-05

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey
⬜ 9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

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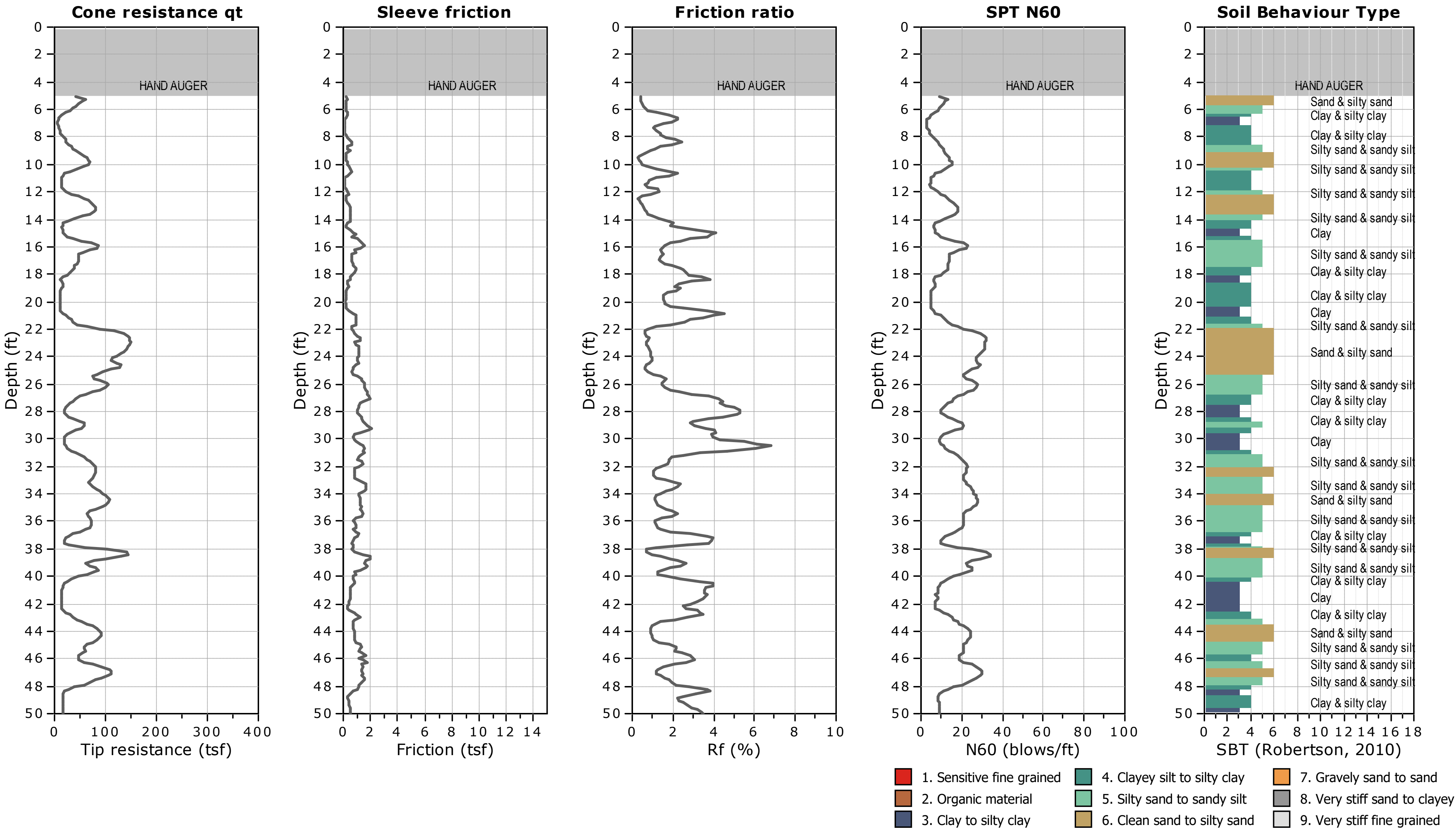
CPT: SCPT-04

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



GREGG DRILLING, LLC
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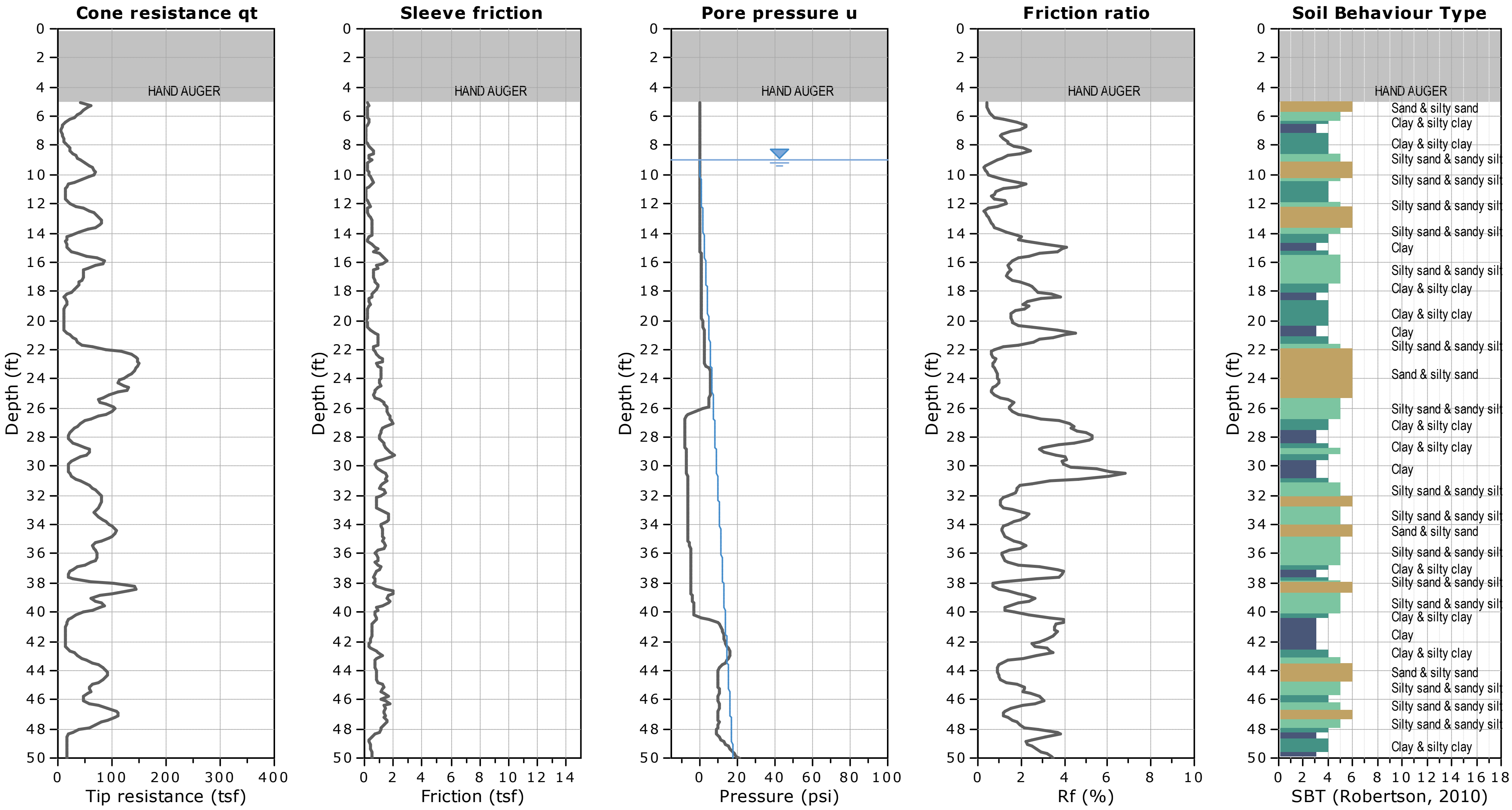
CPT: SCPT-04

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



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

WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
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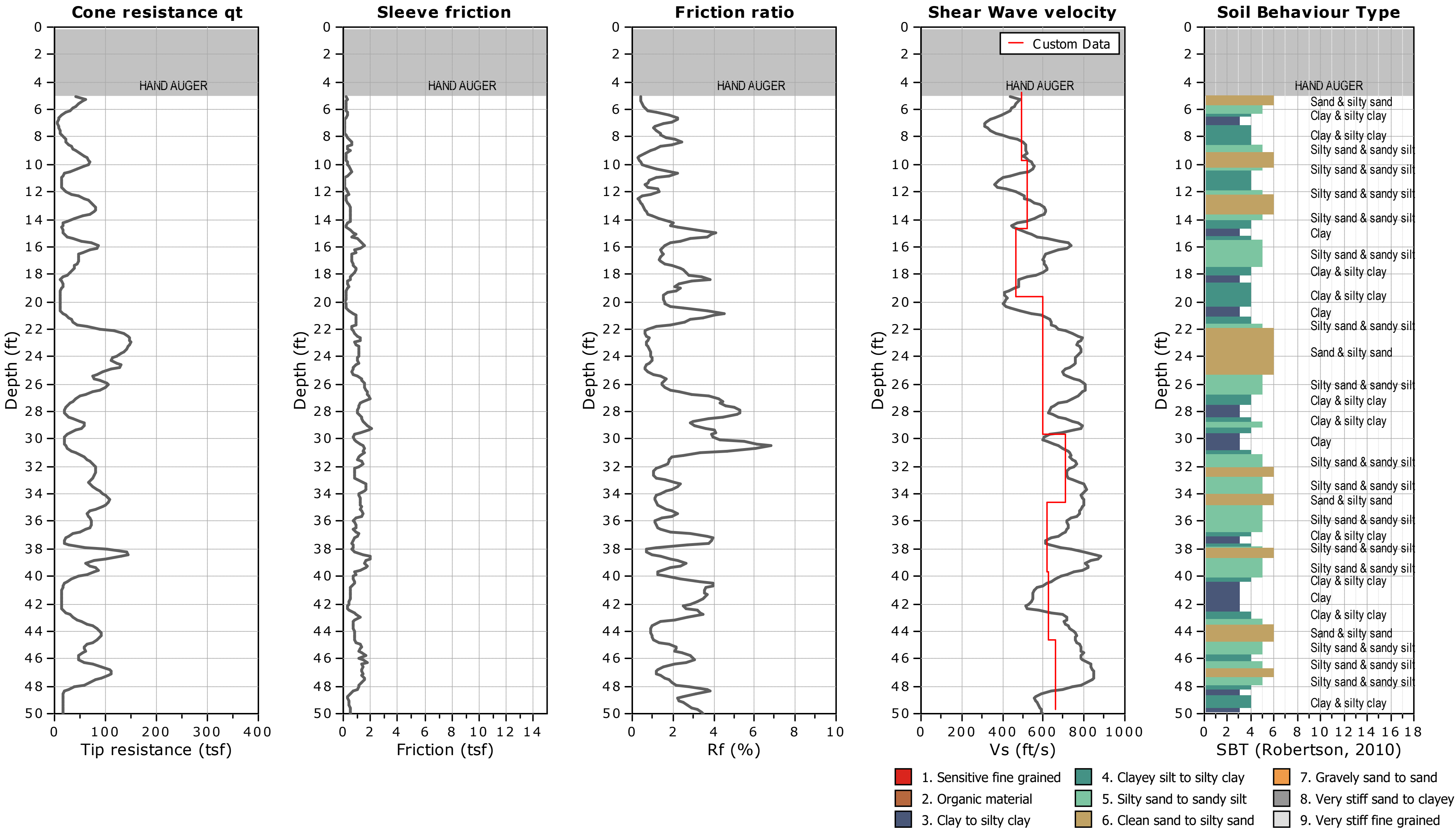
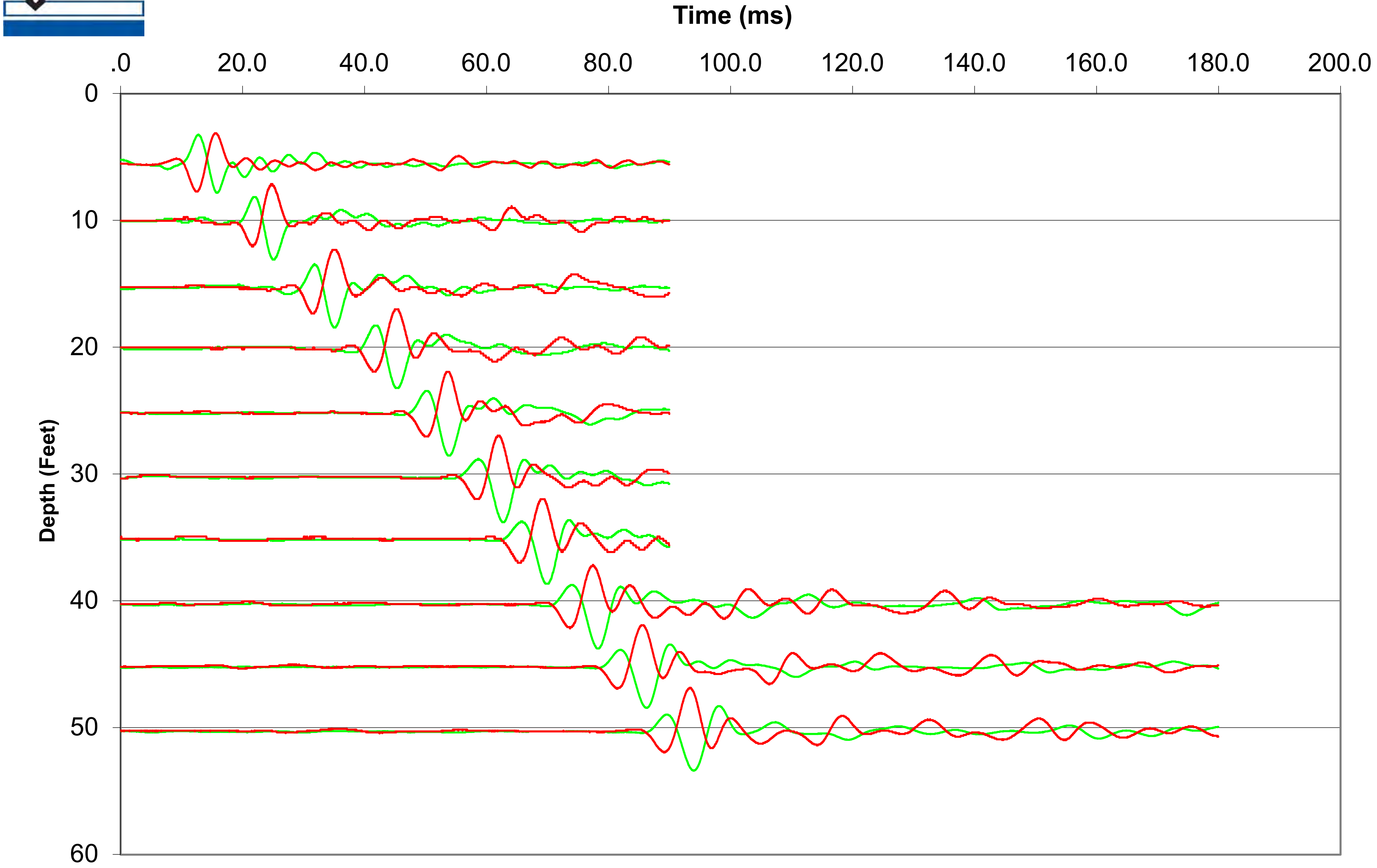
CPT: SCPT-04

CLIENT: LEIGHTON CONSULTANTS
SITE: SHOP OFF, FOUNTAIN VALLEY, CA

FIELD REP: MICHELLE M.
Total depth: 50.36 ft, Date: 9/3/2021



Waveforms for Sounding SCPT-04



Shear Wave Velocity Calculations

SHOPOFF FV

SCPT-04

Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

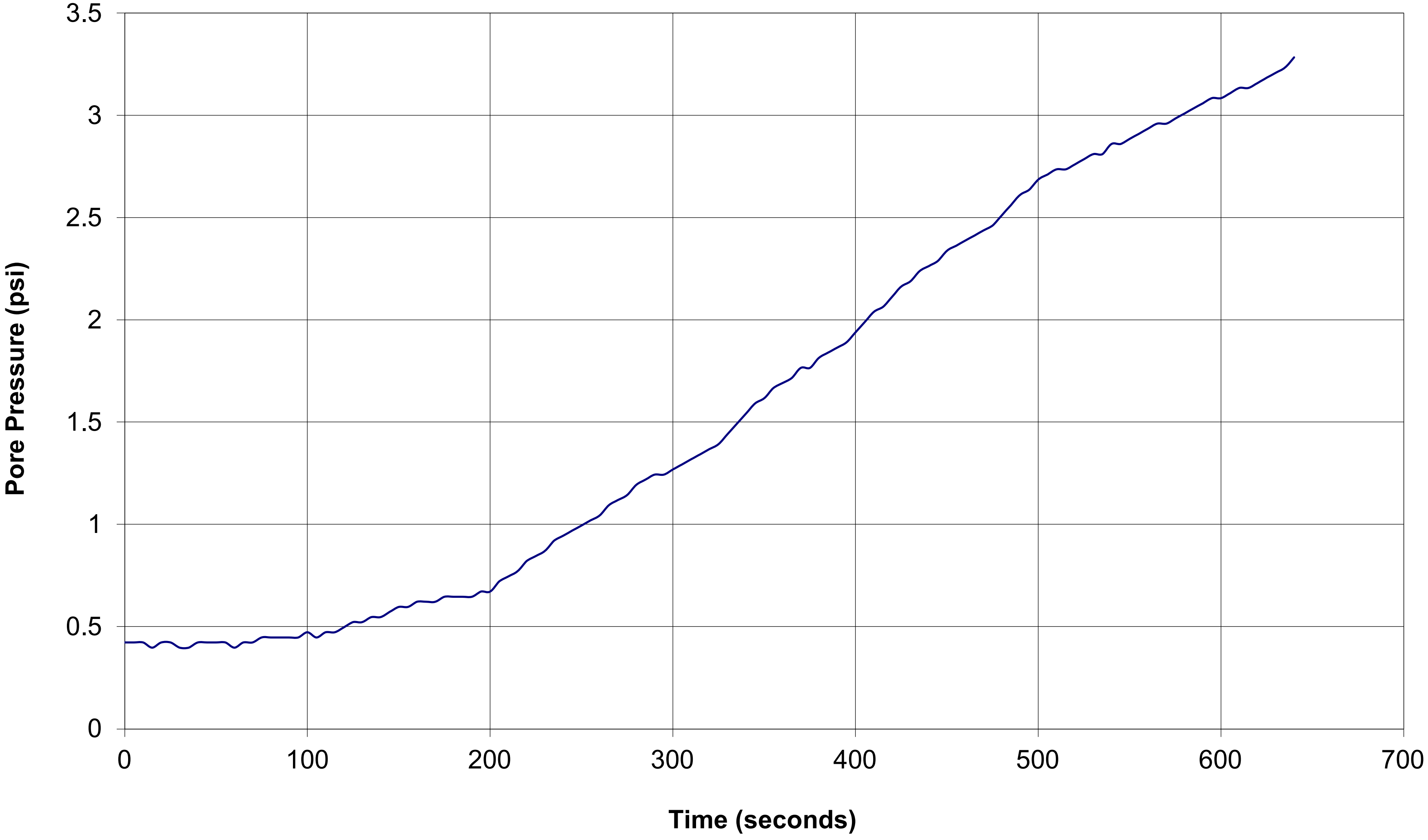
09/03/21

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
5.58	4.92	5.19	5.19	14.1500			
10.17	9.51	9.66	4.46	23.1500	9.0000	495.9	7.21
15.42	14.76	14.85	5.20	33.1000	9.9500	522.4	12.14
20.18	19.52	19.59	4.73	43.3500	10.2500	461.9	17.14
25.26	24.60	24.66	5.07	51.8000	8.4500	600.1	22.06
30.35	29.69	29.73	5.08	60.3000	8.5000	597.1	27.15
35.27	34.61	34.65	4.91	67.2500	6.9500	707.1	32.15
40.35	39.69	39.73	5.08	75.5000	8.2500	615.8	37.15
45.28	44.62	44.65	4.92	83.3500	7.8500	626.4	42.15
50.36	49.70	49.73	5.08	91.0500	7.7000	660.0	47.16

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

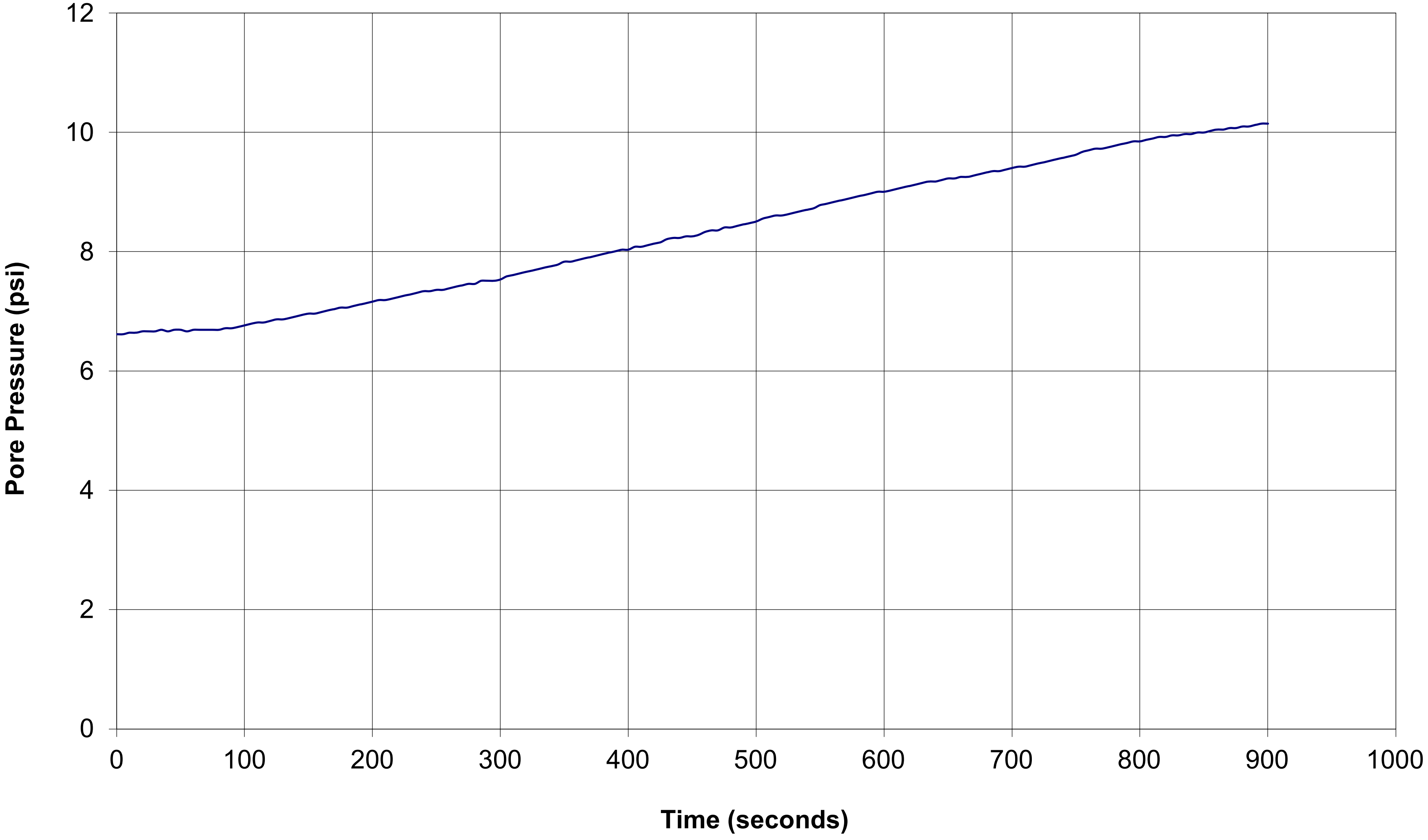
Sounding: CPT-01
Depth (ft): 18.70
Site: SHOPOFF FV
Engineer: MICHELLE M.



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

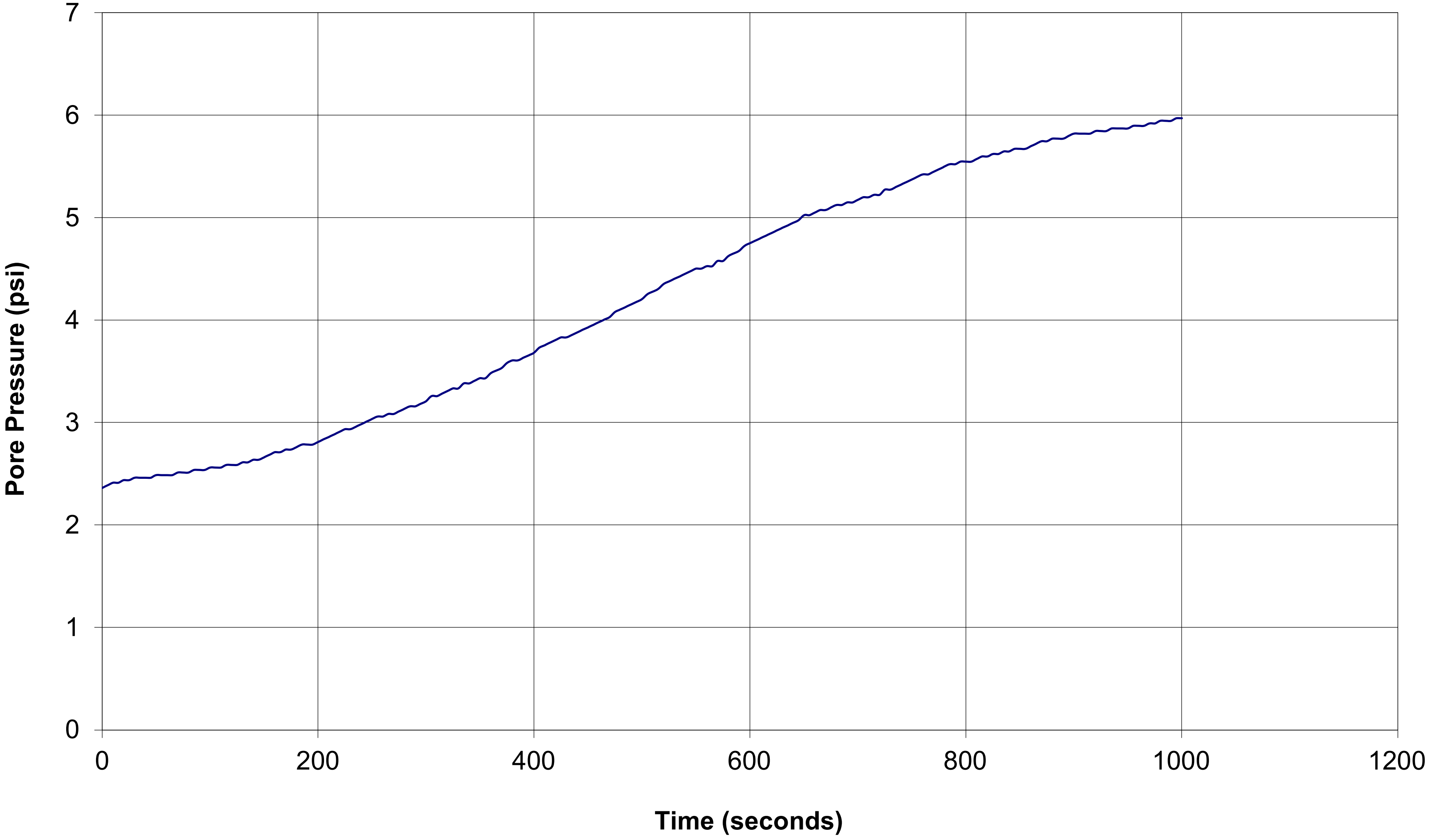
Sounding: CPT-01
Depth (ft): 50.36
Site: SHOPOFF FV
Engineer: MICHELLE M.



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: SCPT-04
Depth (ft): 23.13
Site: SHOPOFF FV
Engineer: MICHELLE M.



APPENDIX C

LABORATORY TEST RESULTS

EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	Shopoff Fountain Valley	Tested By:	G. Berdy	Date:	09/08/21
Project No.:	13255.001	Checked By:	A. Santos	Date:	09/09/21
Boring No.:	CPT-2	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Pale olive silty sand (SM)				

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN		Before Test	After Test
Specimen Diameter	(in.)	4.01	4.01
Specimen Height	(in.)	1.0000	1.0005
Wt. Comp. Soil + Mold	(g)	565.20	391.70
Wt. of Mold	(g)	201.30	0.00
Specific Gravity (Assumed)		2.70	2.70
Container No.		O	O
Wet Wt. of Soil + Cont.	(g)	733.50	593.00
Dry Wt. of Soil + Cont.	(g)	646.20	521.92
Wt. of Container	(g)	0.00	201.30
Moisture Content	(%)	13.51	22.17
Wet Density	(pcf)	109.8	118.1
Dry Density	(pcf)	96.7	96.7
Void Ratio		0.743	0.744
Total Porosity		0.426	0.427
Pore Volume	(cc)	88.3	88.3
Degree of Saturation (%) [S meas]		**49.1**	80.5

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
09/08/21	13:19	1.0	0	0.5815
09/08/21	13:29	1.0	10	0.5815
Add Distilled Water to the Specimen				
09/08/21	14:30	1.0	61	0.5820
09/09/21	5:07	1.0	938	0.5820
09/09/21	6:10	1.0	1001	0.5820

Expansion Index (EI meas) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**0**

Leighton

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Shopoff Fountain Valley　Tested By : ACS/GEB　Date: 09/07/21

Project No. : 13255.001　Checked By: A. Santos　Date: 09/09/21

Boring No.	CPT-2			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Pale olive silty sand (SM)			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.03			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	95			
Crucible No.	19			
Furnace Temperature (°C)	860			
Time In / Time Out	6:30/7:15			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	19.8617			
Wt. of Crucible (g)	19.8589			
Wt. of Residue (g) (A)	0.0028			
PPM of Sulfate (A) x 41150	115.22			
PPM of Sulfate, Dry Weight Basis	**115**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.7			
PPM of Chloride (C -0.2) * 100 * 30 / B	100			
PPM of Chloride, Dry Wt. Basis	**100**			

pH TEST, DOT California Test 643

pH Value	**7.52**			
Temperature °C	20.1			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Shopoff Fountain Valley	Tested By :	G. Berdy	Date: 08/03/21
Project No. :	13255.001	Checked By:	A. Santos	Date: 09/09/21
Boring No.:	CPT-2	Depth (ft.) :	0-5	
Sample No. :	B-1			

Soil Identification:* Pale olive silty sand (SM)

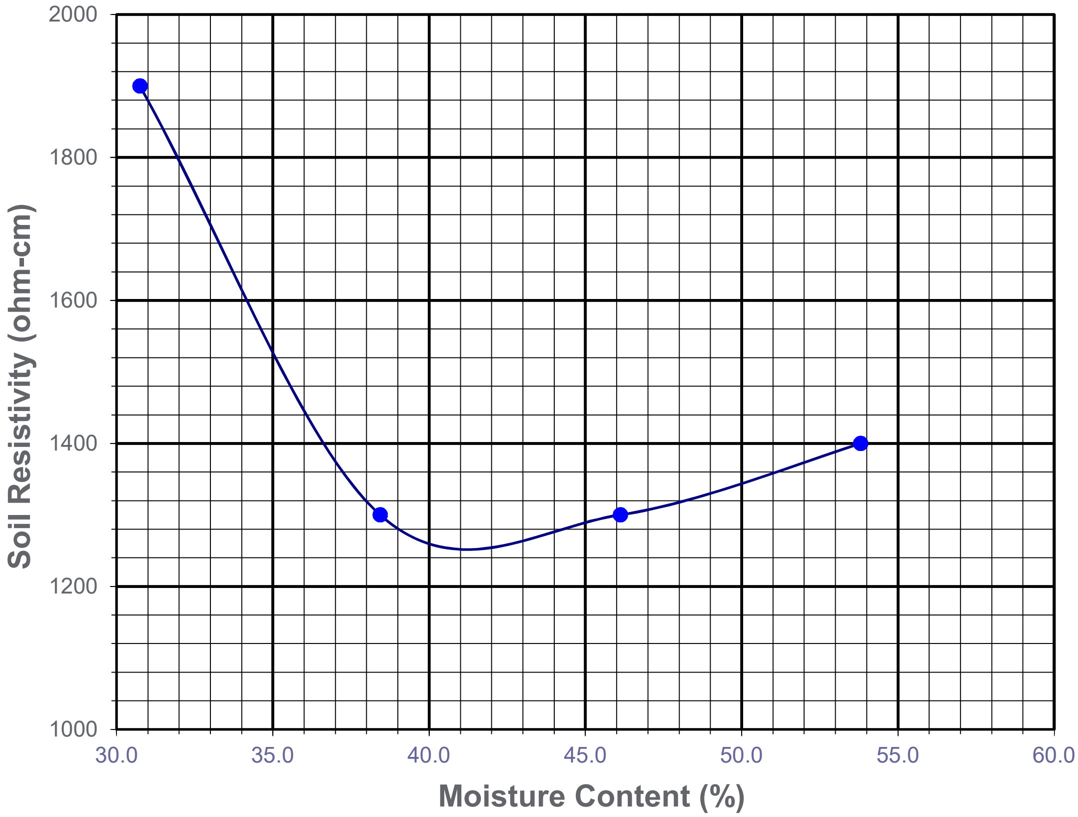
*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	40	30.75	1900	1900
2	50	38.43	1300	1300
3	60	46.12	1300	1300
4	70	53.80	1400	1400
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.10
Box Constant	1.000

MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
1250	**41.5**	**115**	**100**	**7.52**	**20.1**



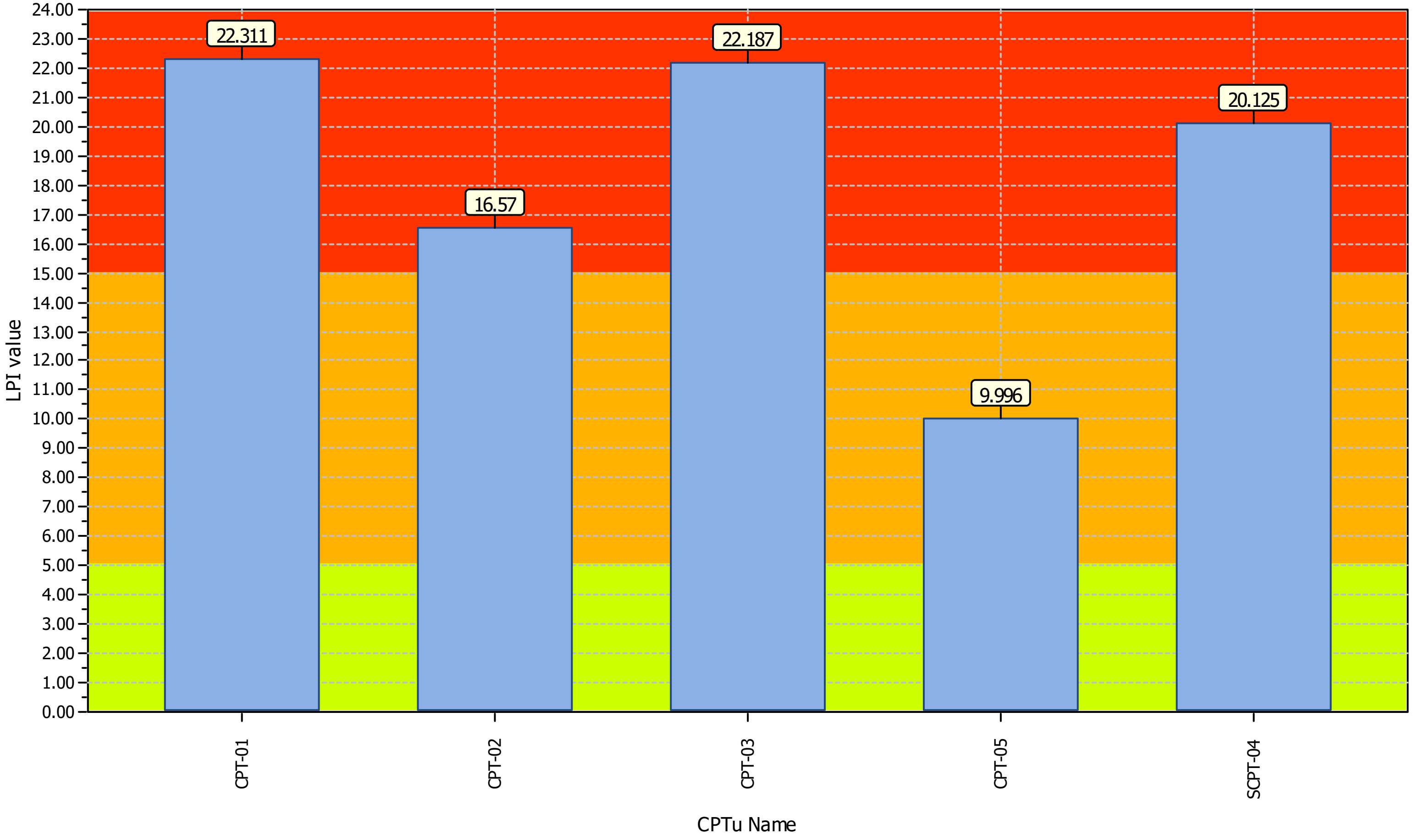
APPENDIX D

LIQUEFACTION ANALYSIS

Project title : 13255.001 Shopoff Fountain Valley

Location : 16300 Euclid Street, Fountain Valley, CA

Overall Liquefaction Potential Index report



LPI color scheme

🟥 Very high risk
🟧 High risk
🟩 Low risk

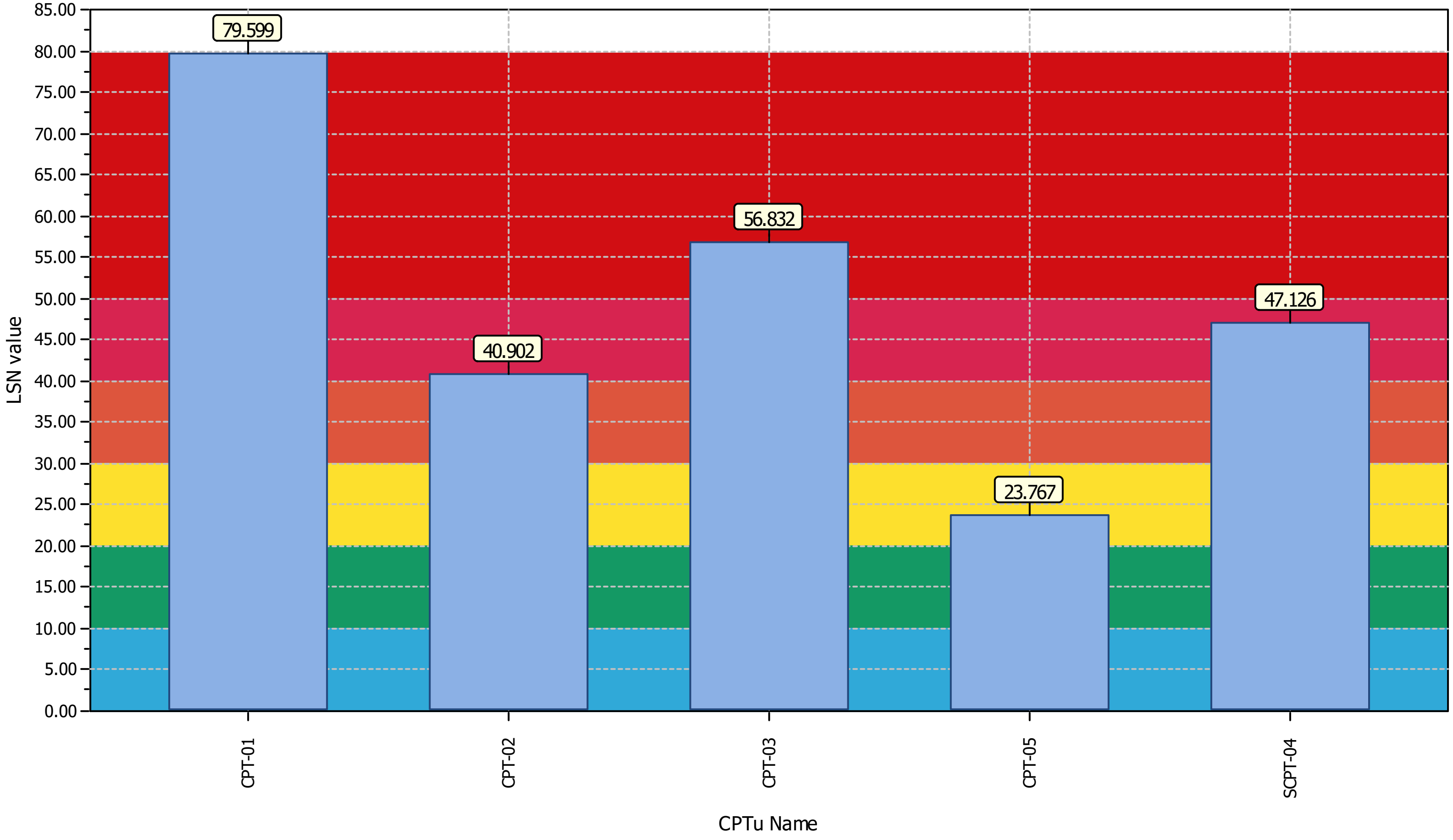
Basic statistics

Total CPT number: 5
0% low risk
20% high risk
80% very high risk

Project title : 13255.001 Shopoff Fountain Valley

Location : 16300 Euclid Street, Fountain Valley, CA

Overall Liquefaction Severity Number report



LSN color scheme

- 🟥 Severe damage
- 🟥 Major expression of liquefaction
- 🟧 Moderate to severe exp. of liquefaction
- 🟨 Moderate expression of liquefaction
- 🟩 Minor expression of liquefaction
- 🟦 Little to no expression of liquefaction

Basic statistics

Total CPT number: 5

0% little liquefaction

0% minor liquefaction

20% moderate liquefaction

0% moderate to major liquefaction

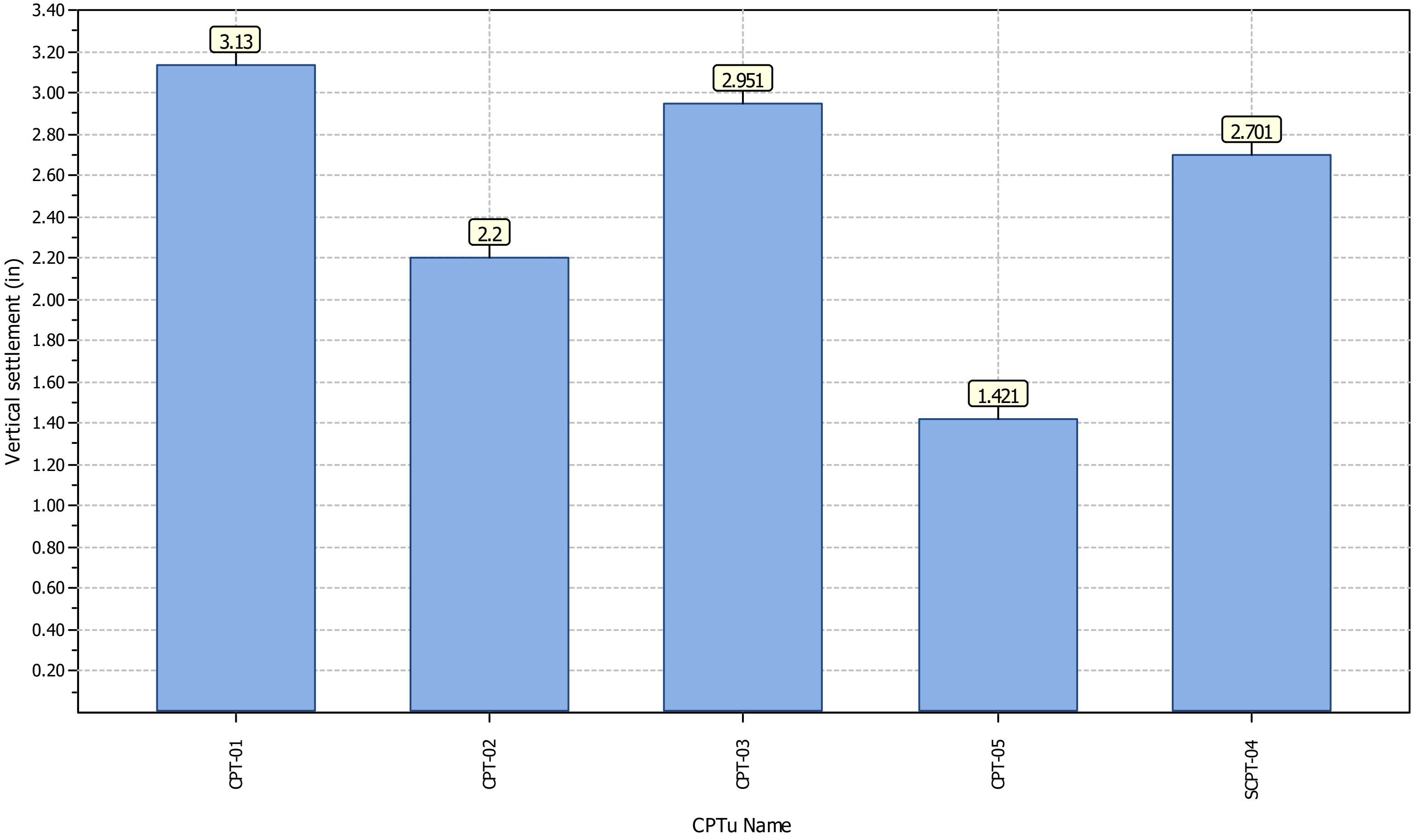
40% major liquefaction

40% severe liquefaction

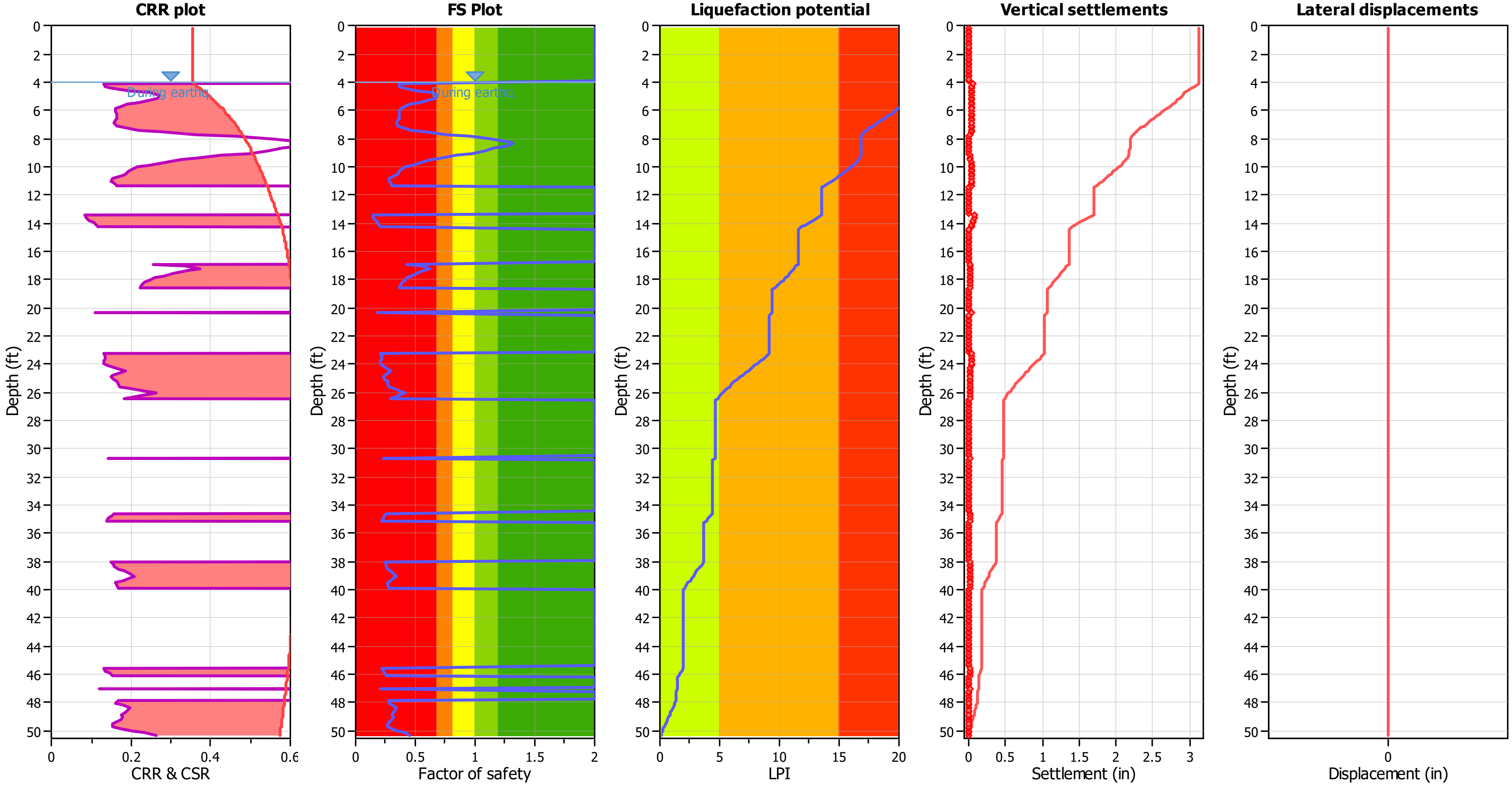
Project title : 13255.001 Shopoff Fountain Valley

Location : 16300 Euclid Street, Fountain Valley, CA

Overall vertical settlements report



Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.89	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

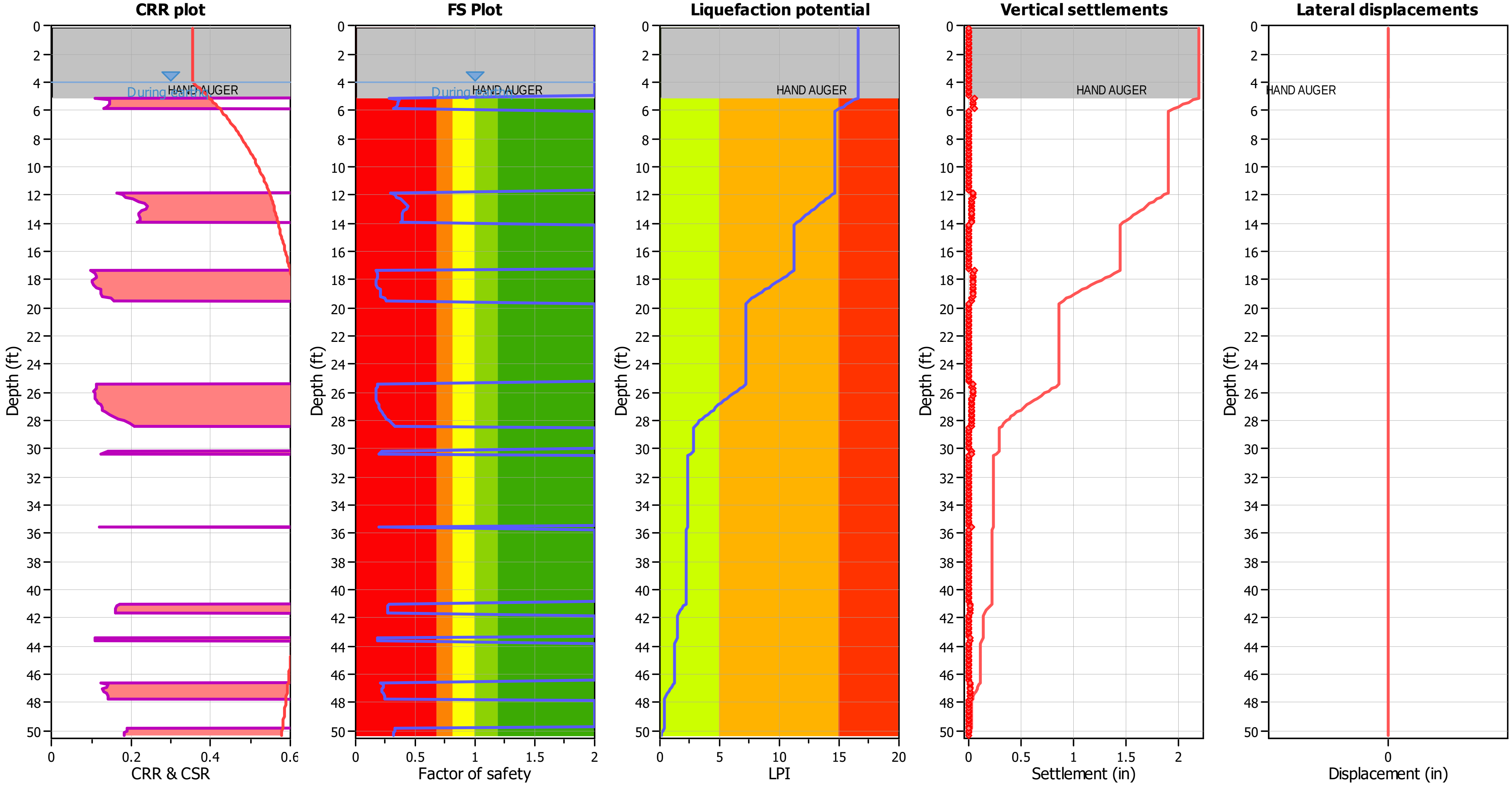
F.S. color scheme

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

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis overall plots



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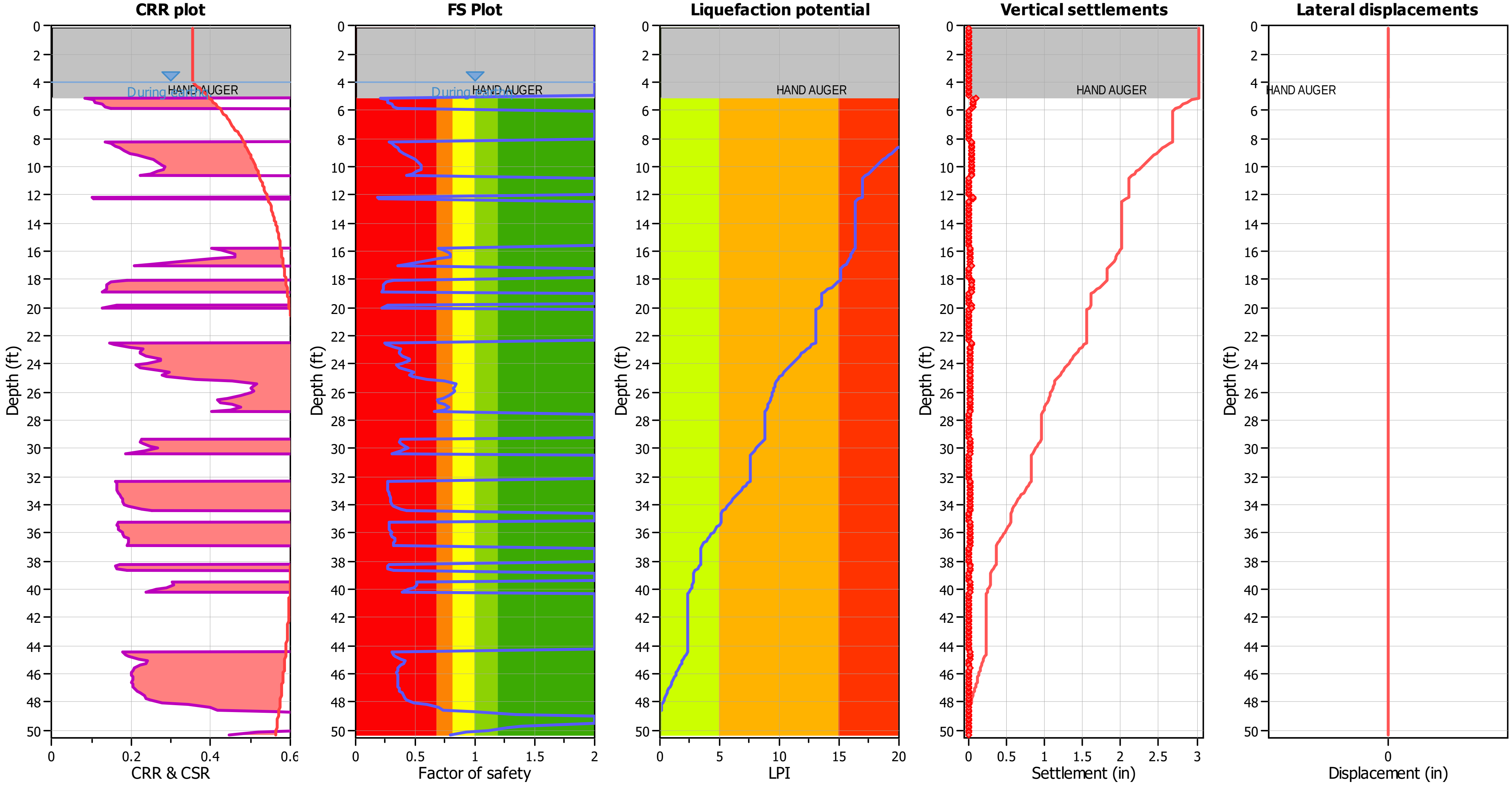
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.89	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
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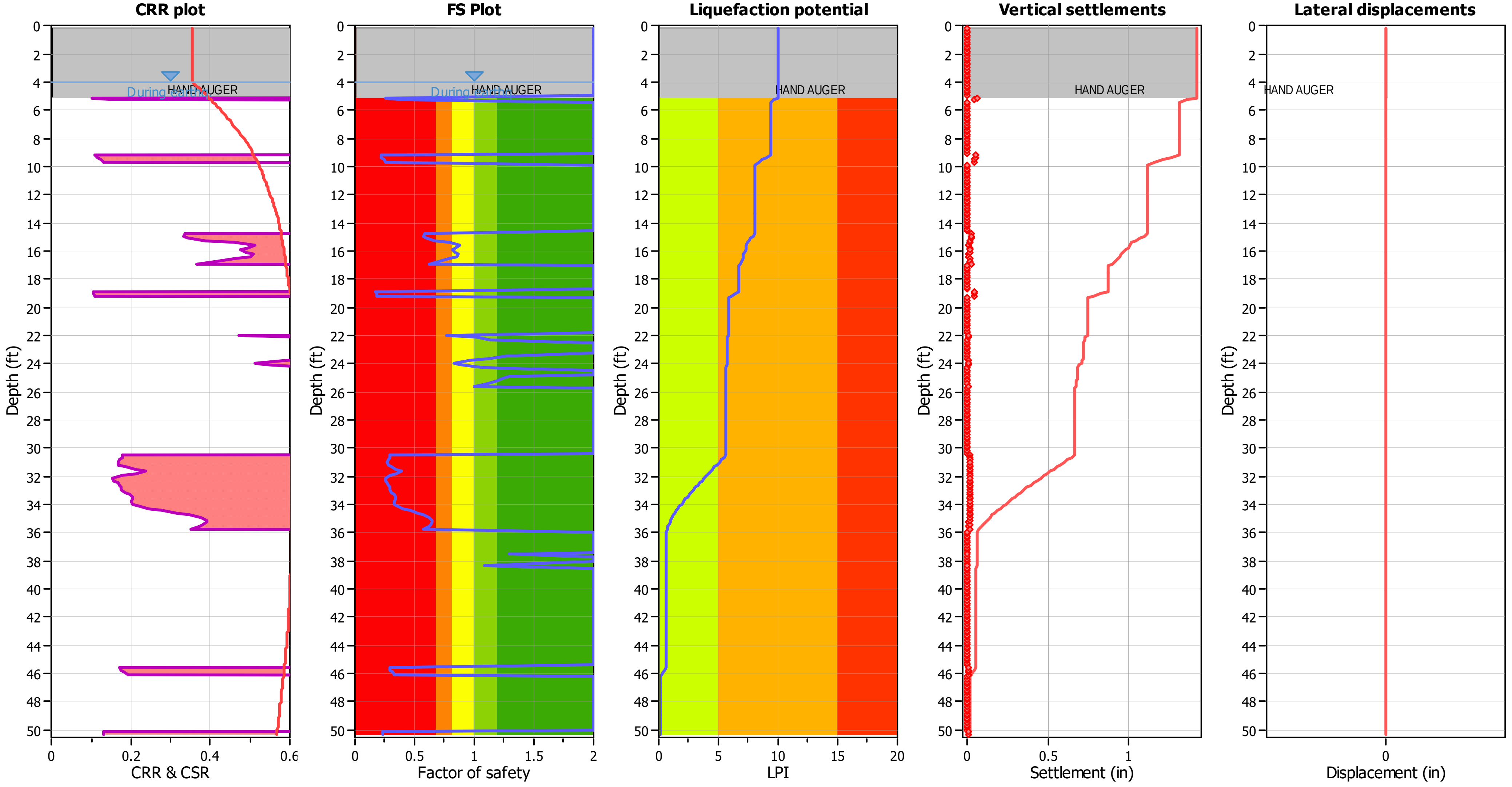
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.89	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

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LPI color scheme

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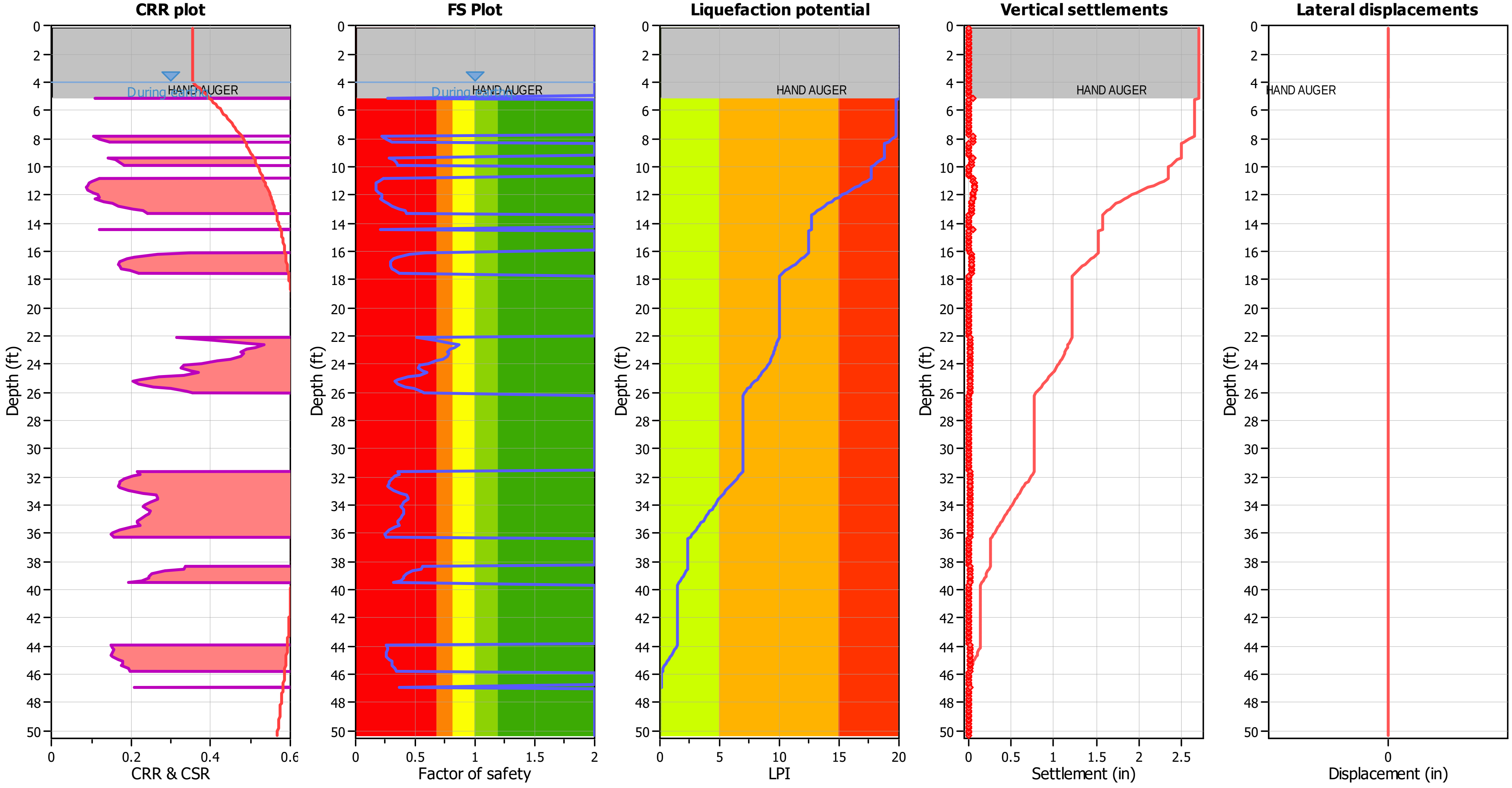
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.89	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.68	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

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LPI color scheme

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APPENDIX D

APPENDIX D

ANALYSES OF LIQUEFACTION POTENTIAL

TABLE OF CONTENTS

Geocon West, Inc.
2807 McGaw Avenue
Irvine, CA 92614
www.geoconinc.com

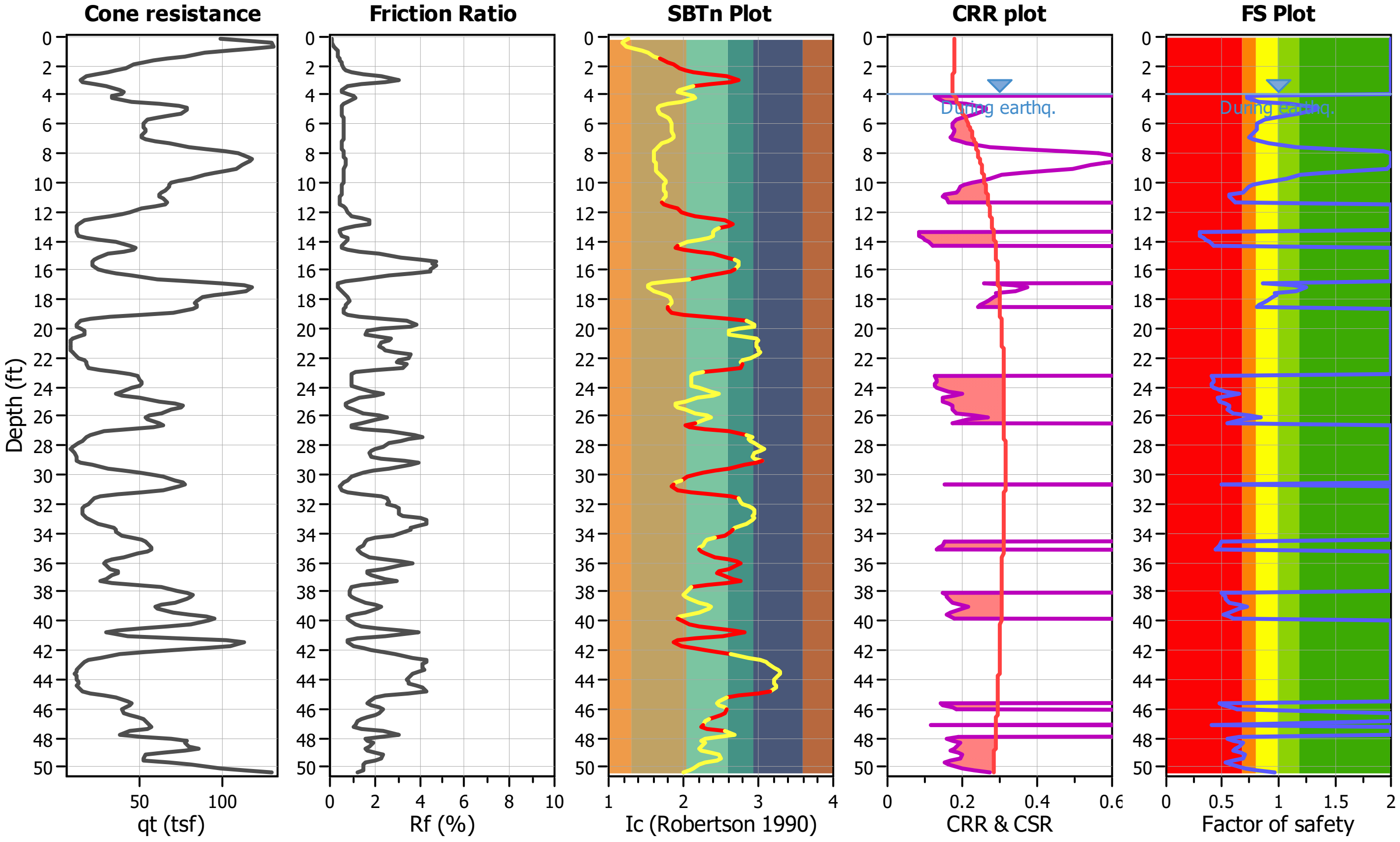
LIQUEFACTION ANALYSIS REPORT

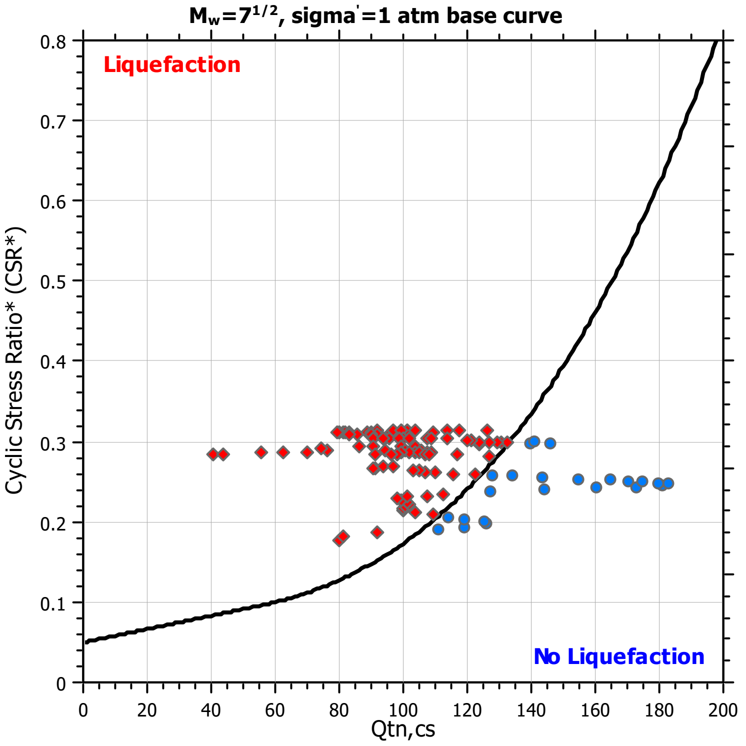
Project title : W2045-88-01 **Location : Euclid and Heil**

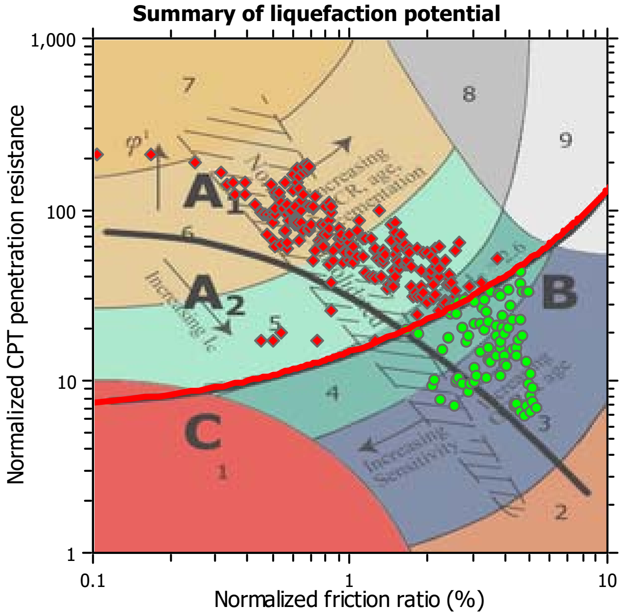
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.42	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

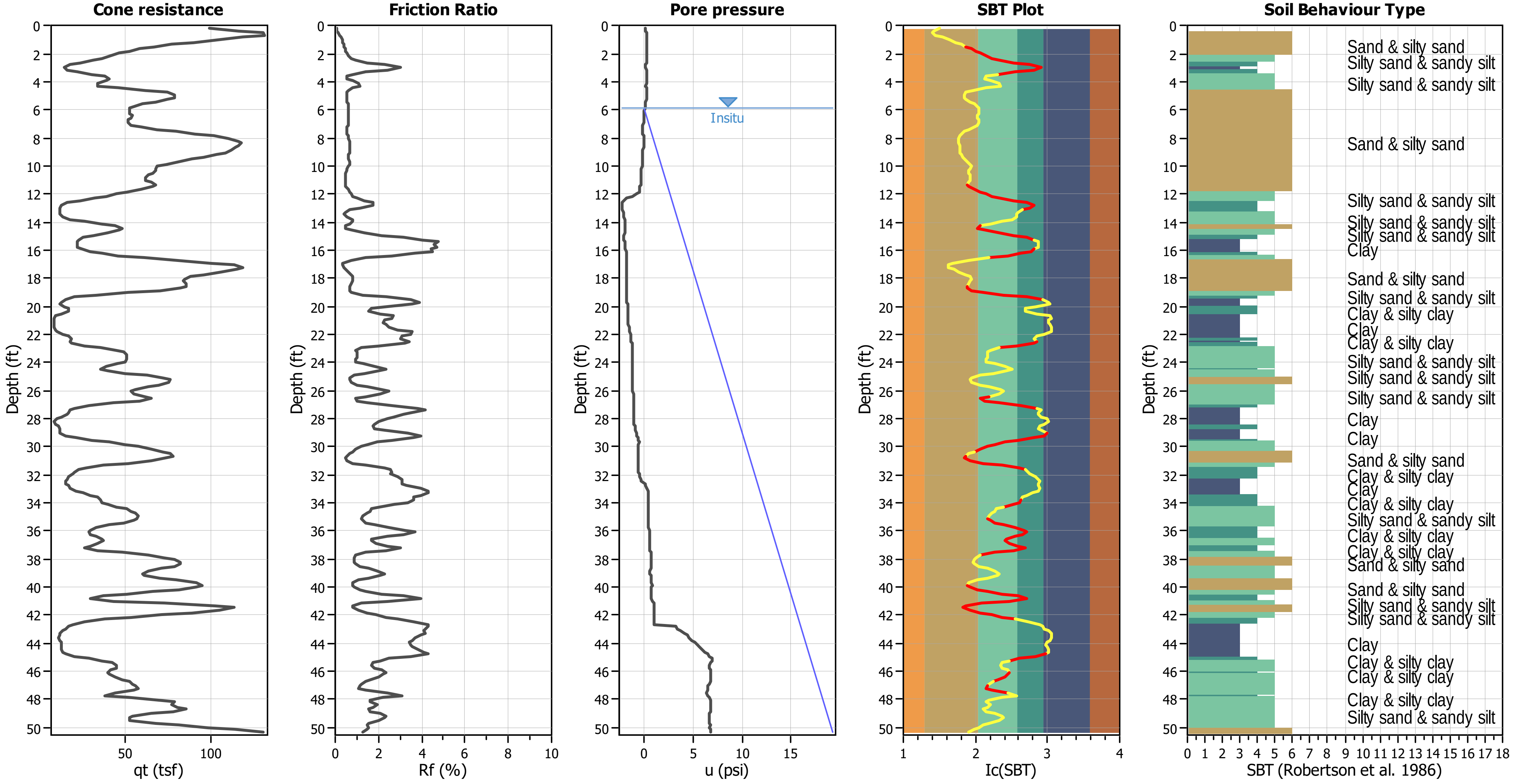






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.10
Peak ground acceleration: 0.42
Depth to water table (insitu): 5.90 ft

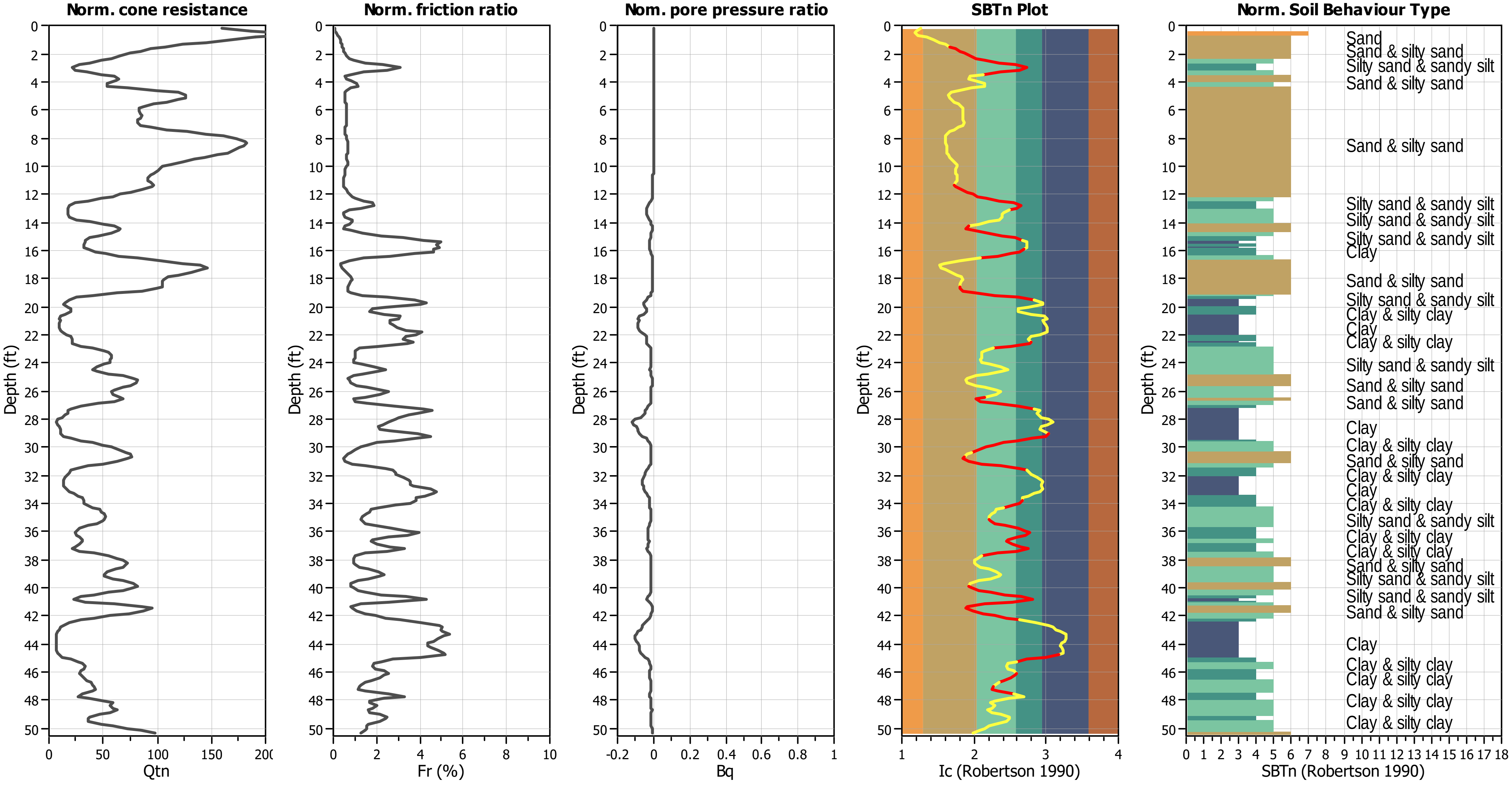
Depth to water table (erthq.): 4.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

SBT legend

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

CPT basic interpretation plots (normalized)



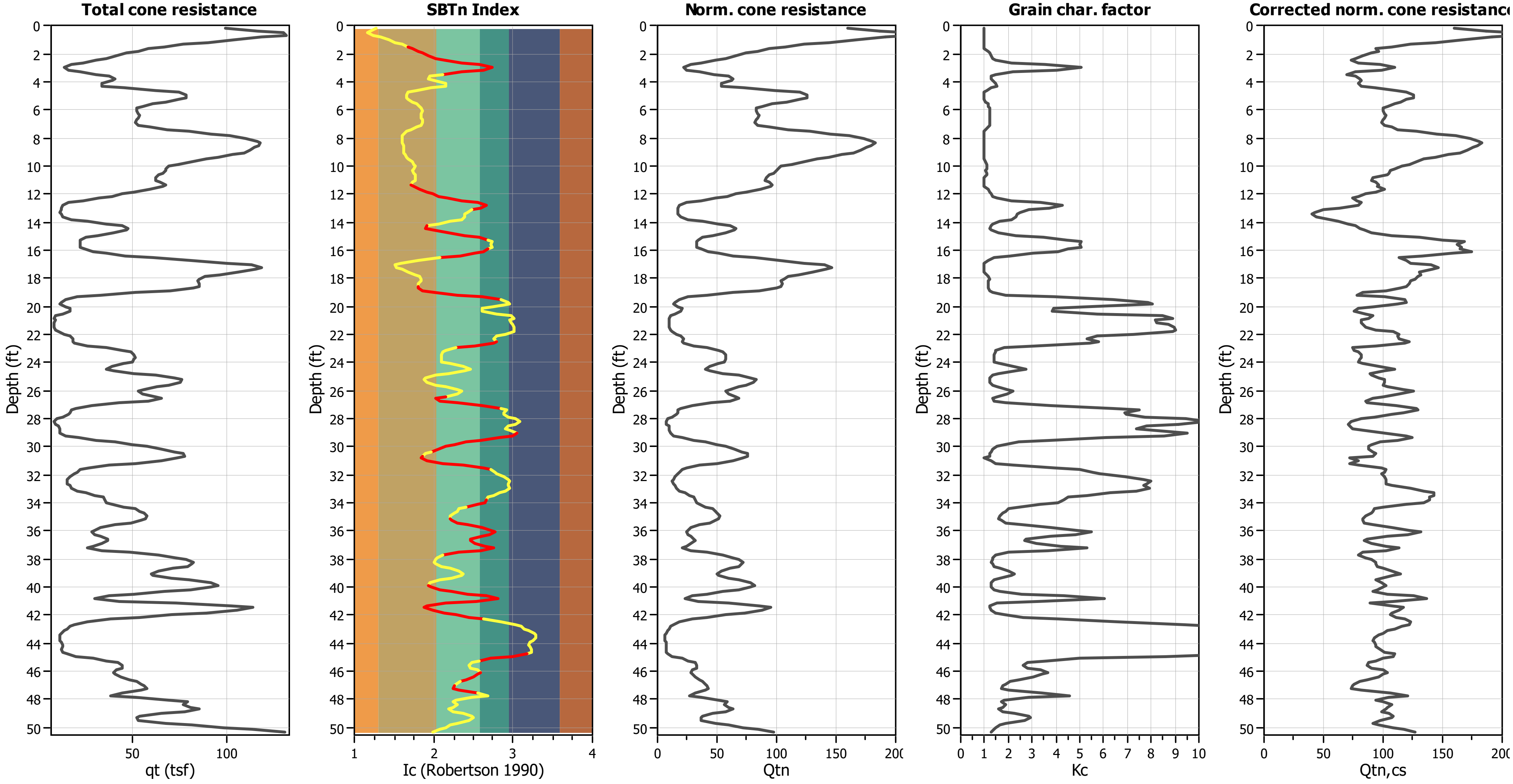
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

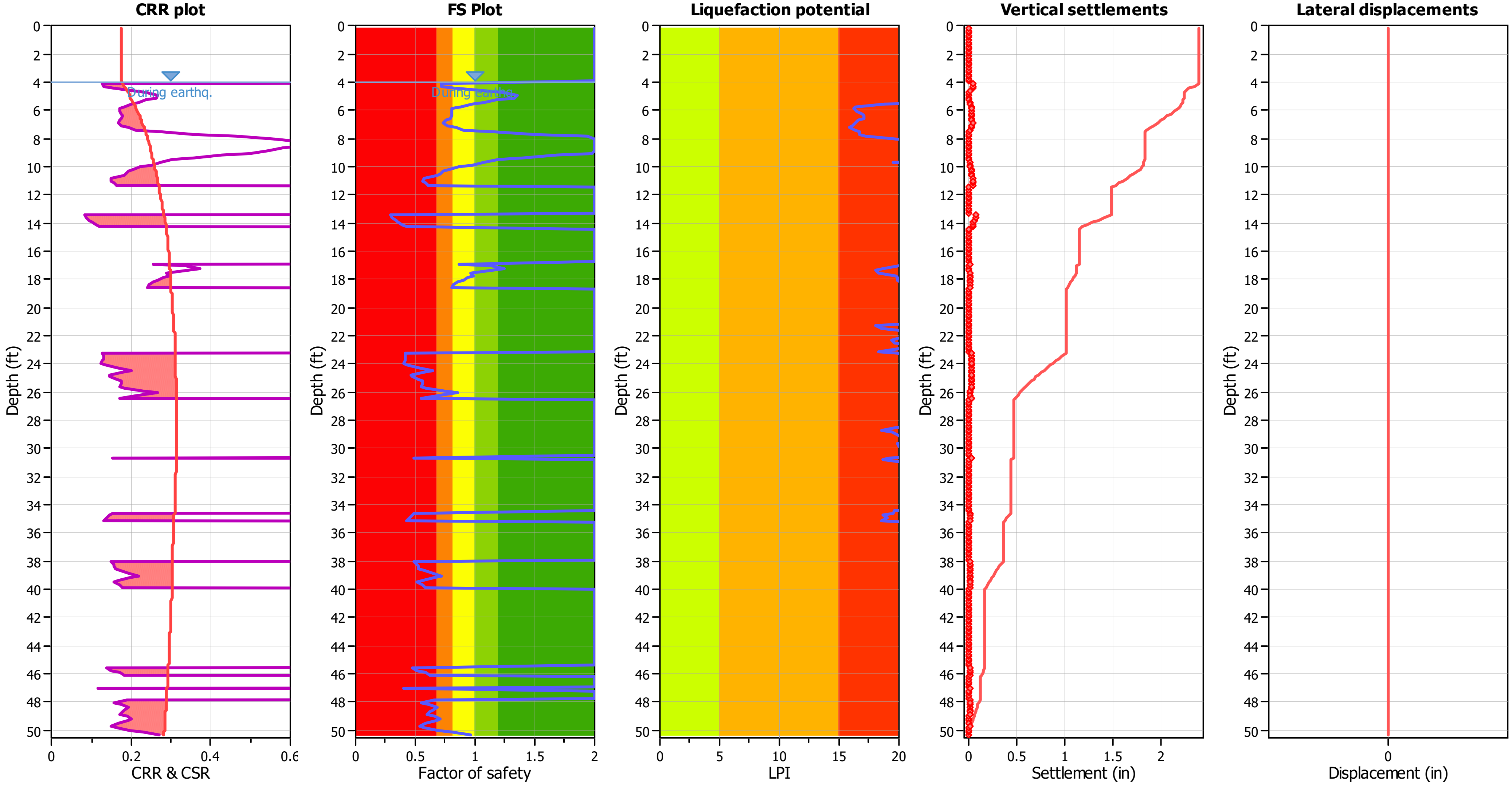
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

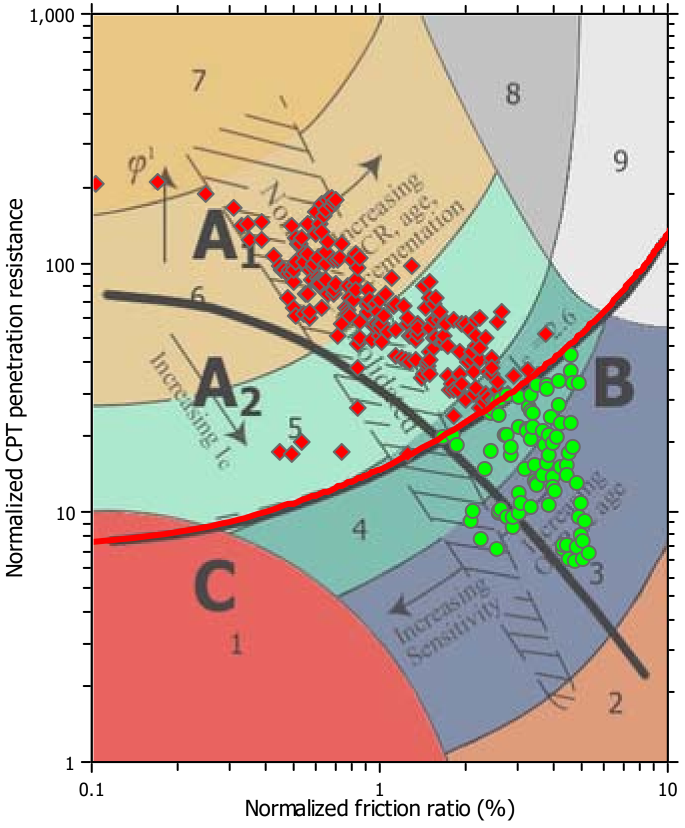
F.S. color scheme

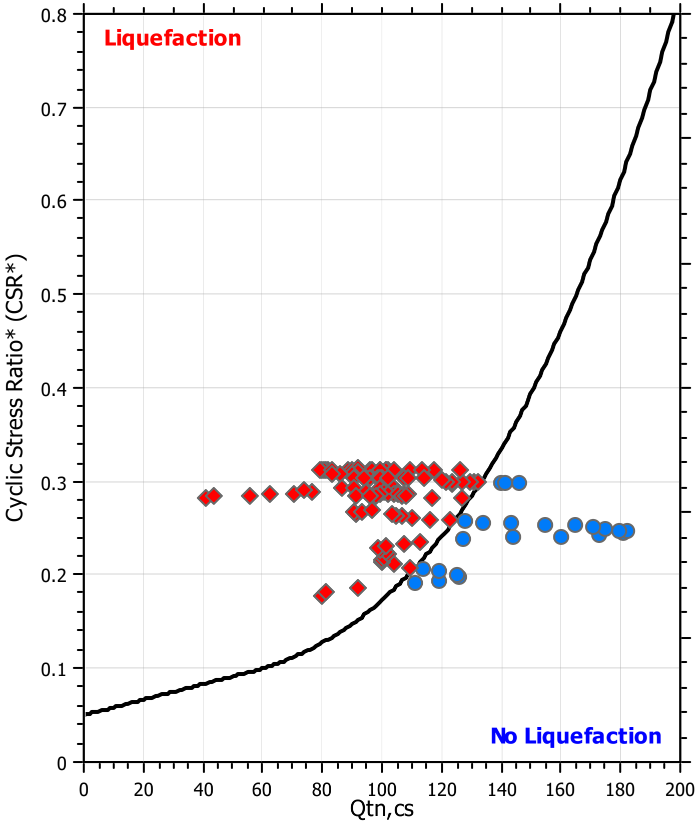
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

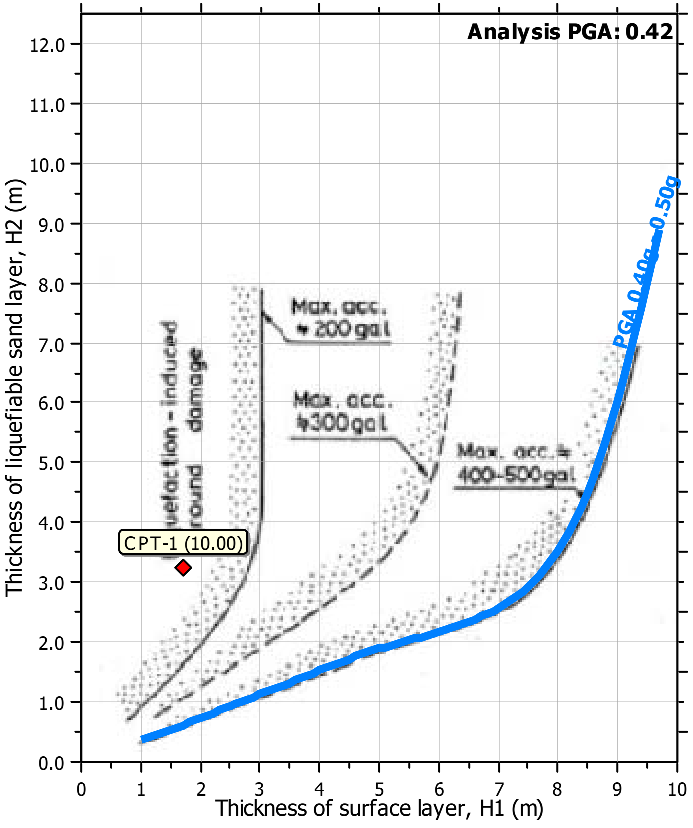
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



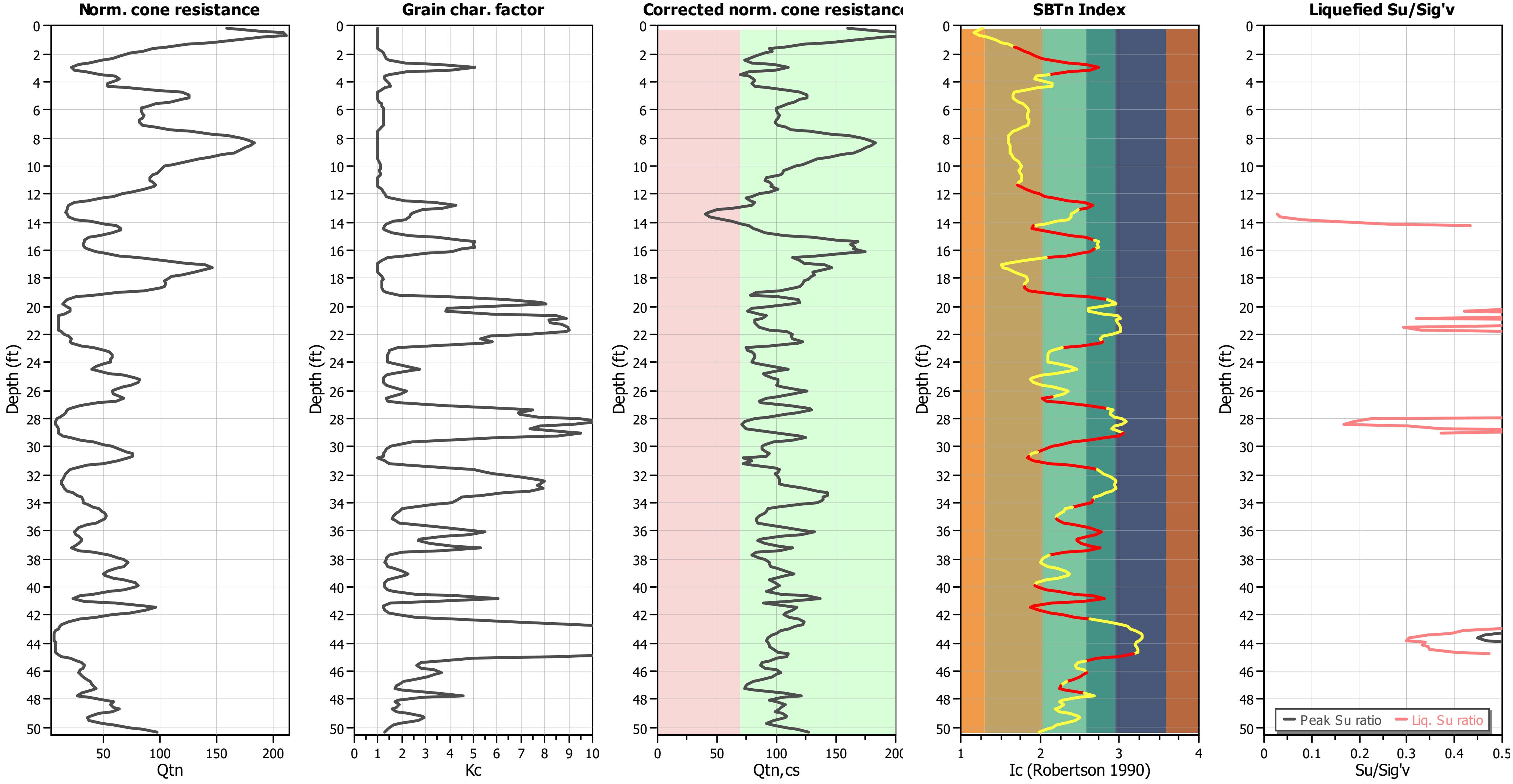




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.10
Peak ground acceleration: 0.42
Depth to water table (insitu): 5.90 ft

Depth to water table (erthq.): 4.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

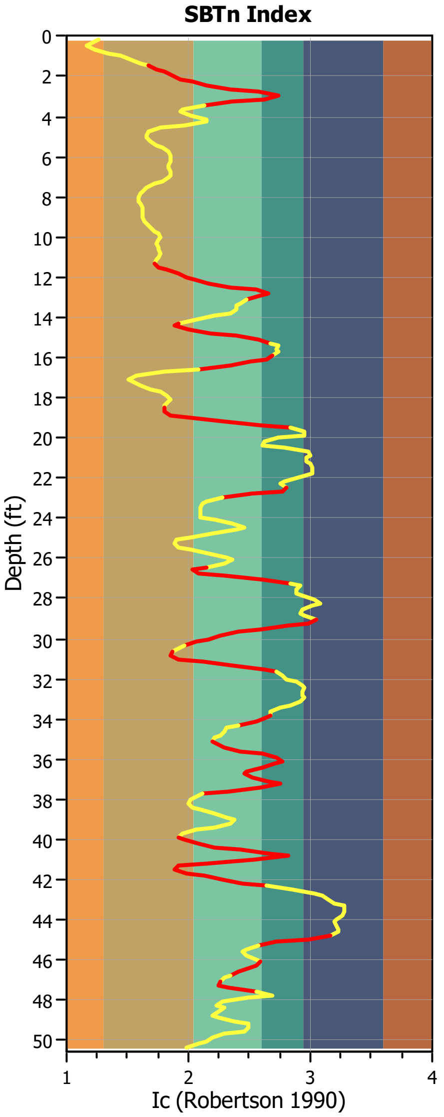
Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

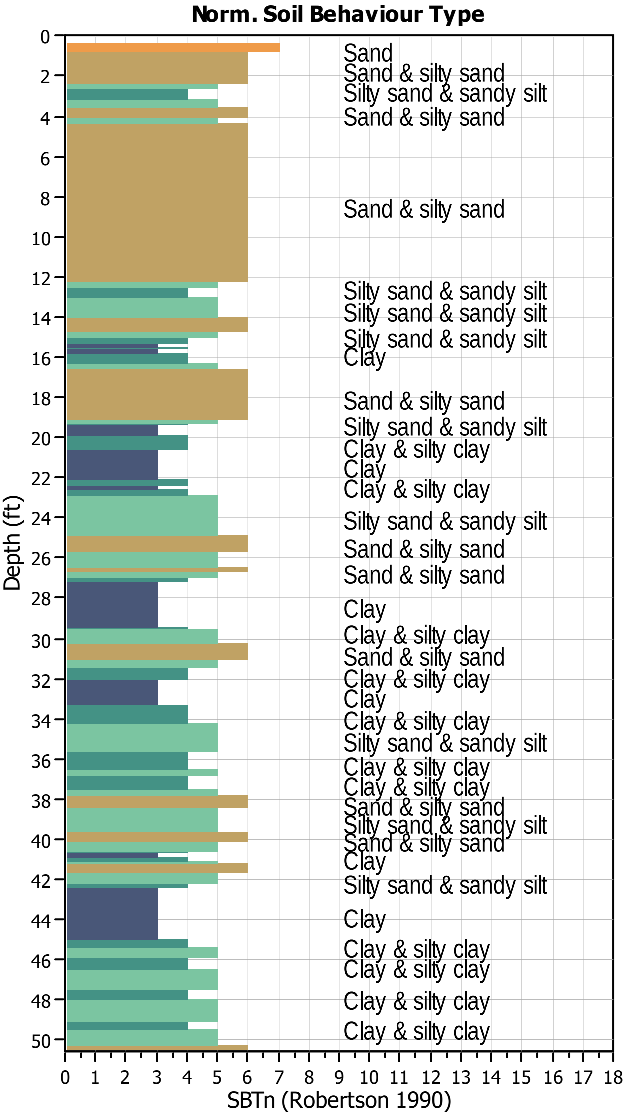
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





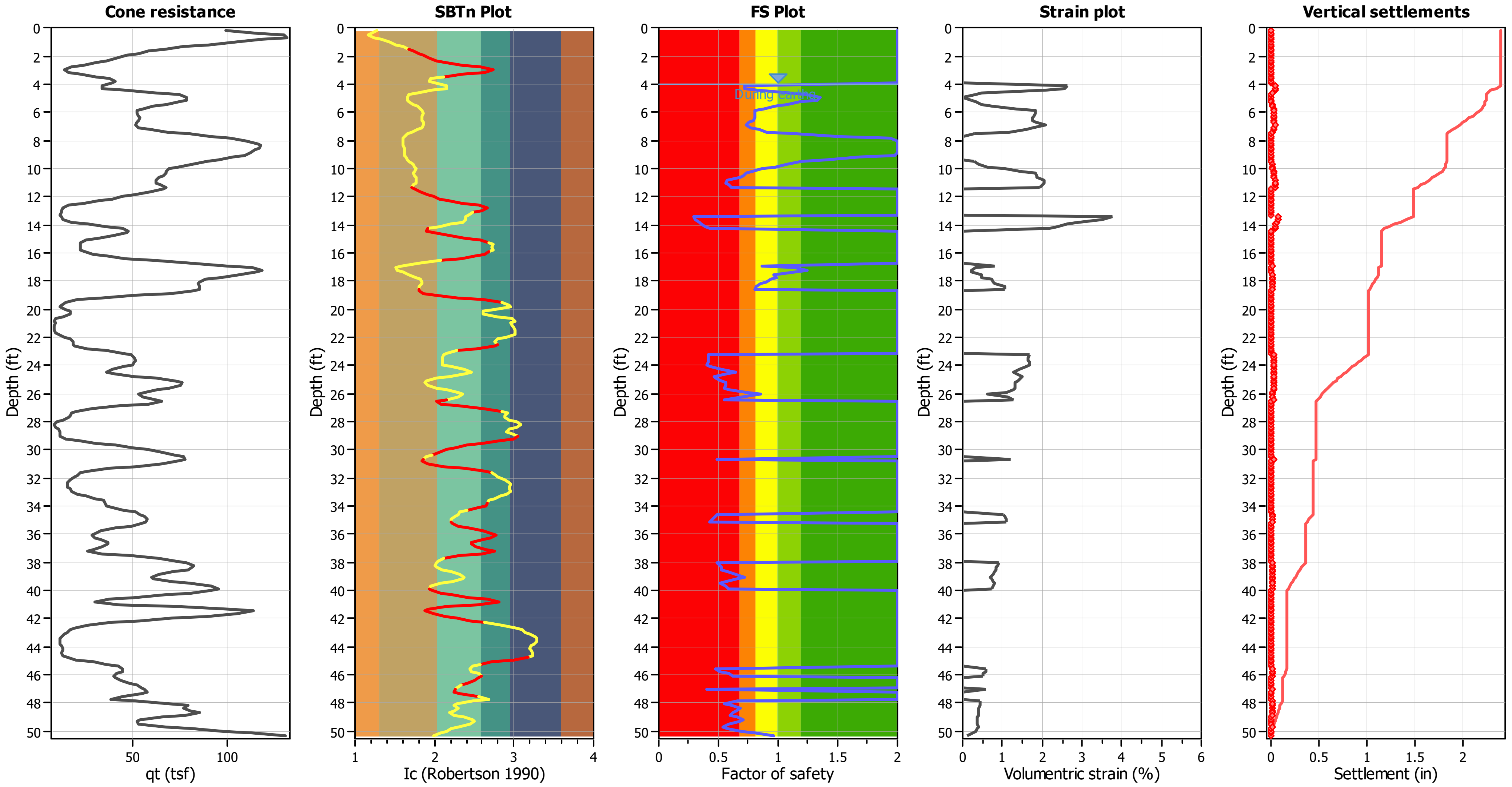
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	307
Total points excluded:	125
Exclusion percentage:	40.72%
Number of layers detected:	22

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

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LIQUEFACTION ANALYSIS REPORT

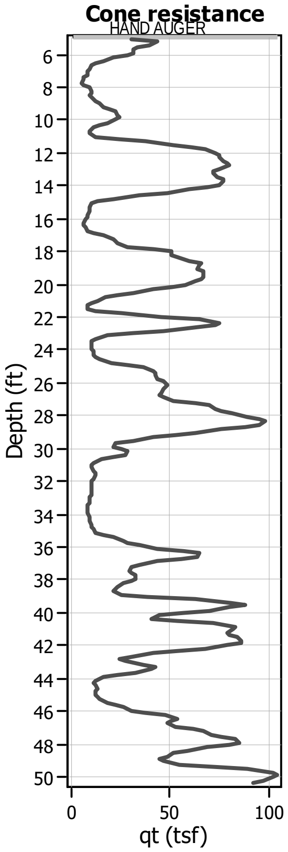
Project title : W2045-88-01 **Location : Euclid and Heil**

CPT file : CPT-2

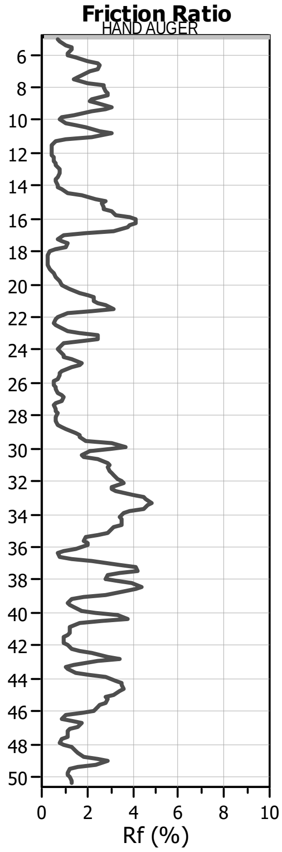
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.42	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

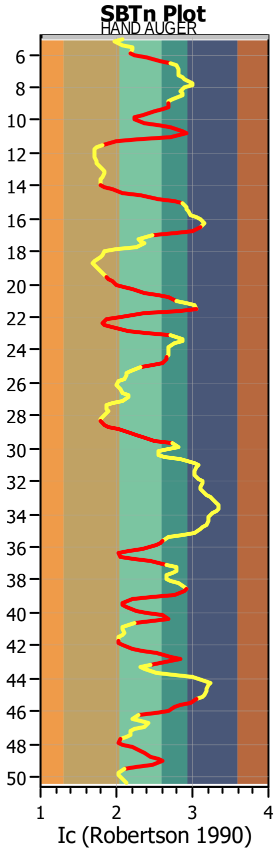
Cone resistance



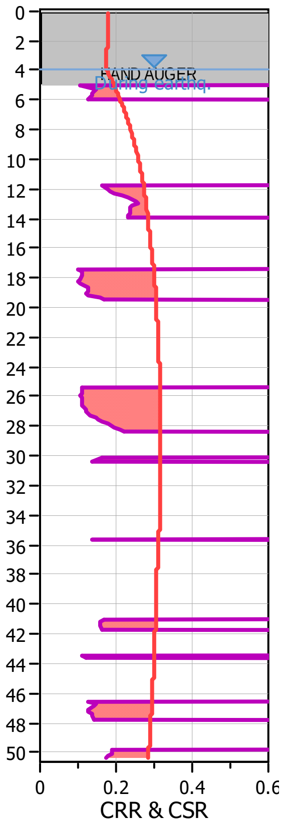
Friction Ratio



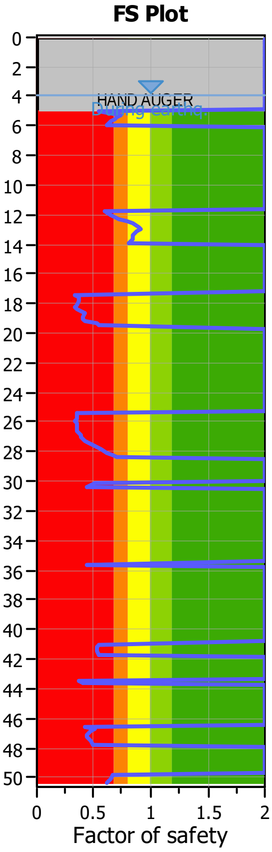
SBTn Plot



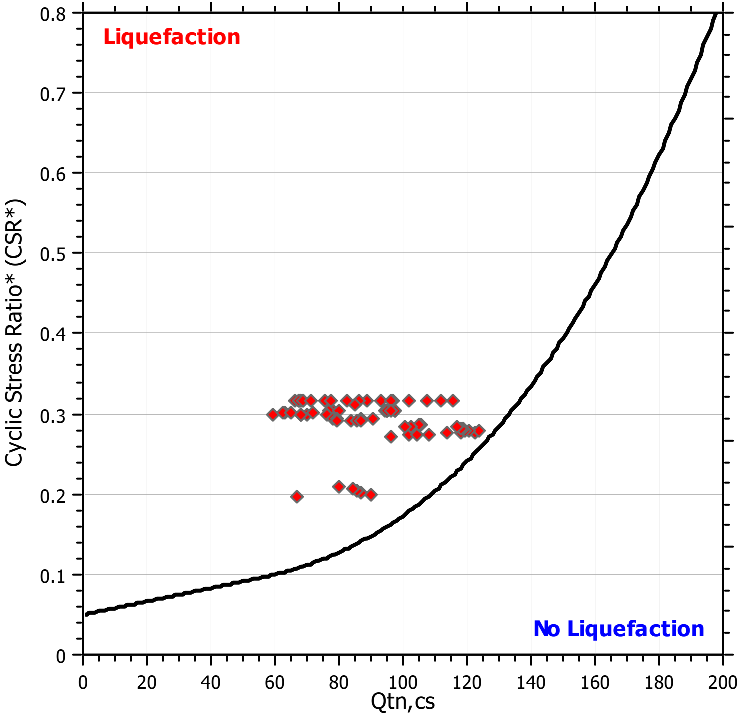
CRR plot



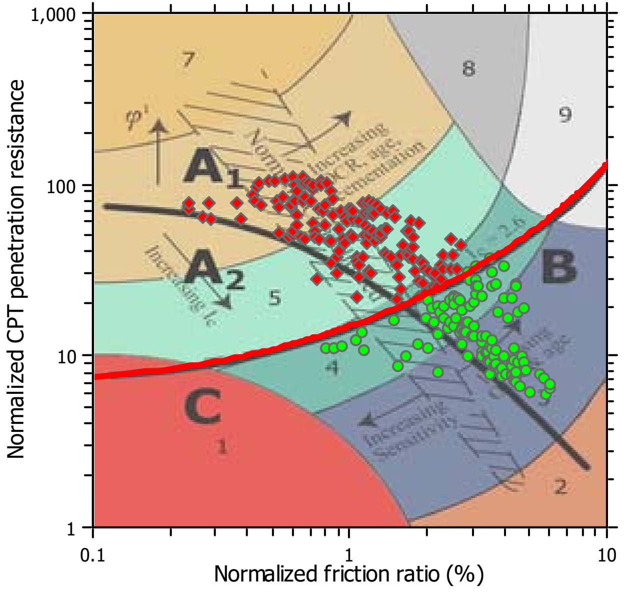
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve

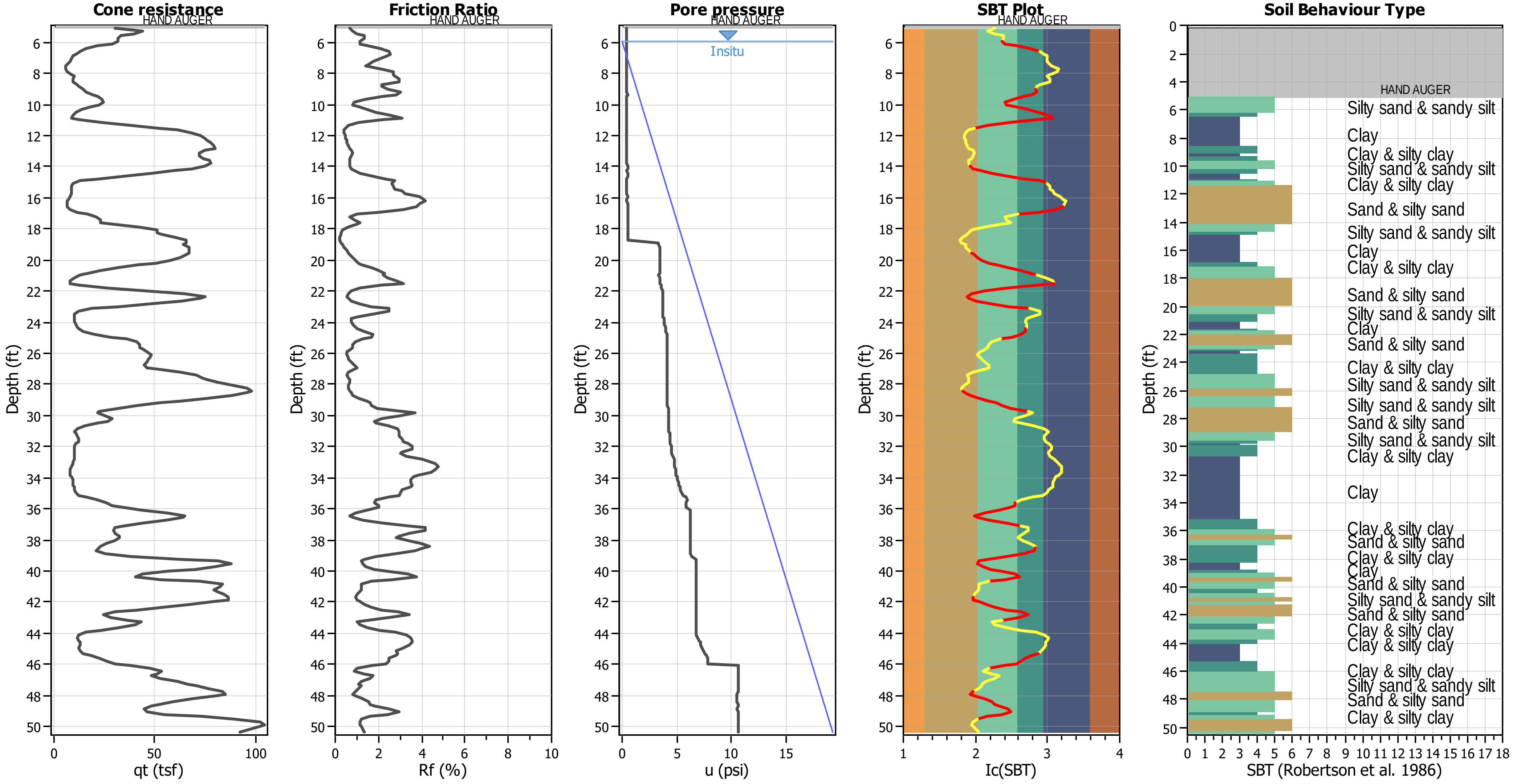


Summary of liquefaction potential



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

CPT basic interpretation plots



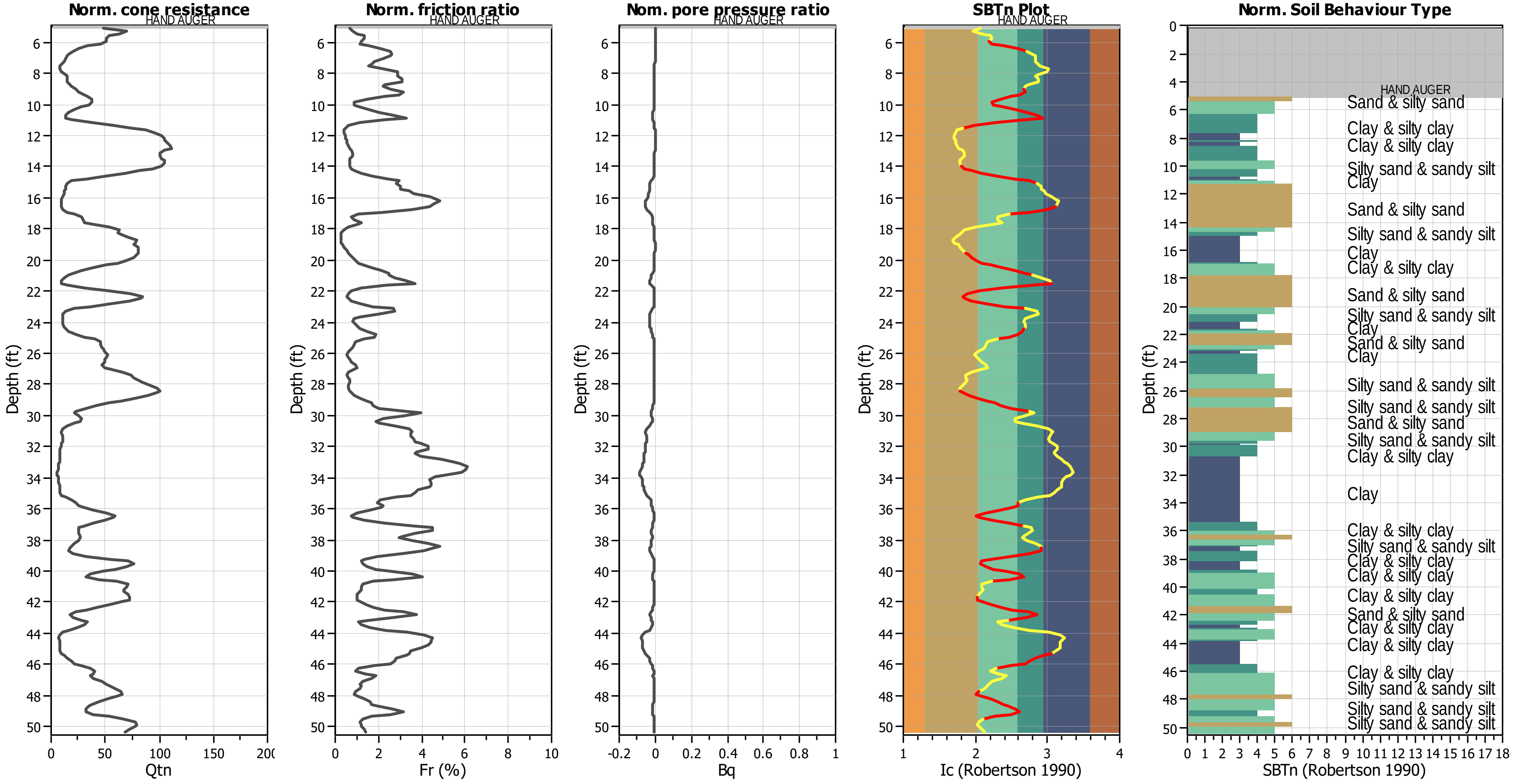
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to
⬜ 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.10
Peak ground acceleration: 0.42
Depth to water table (insitu): 5.90 ft

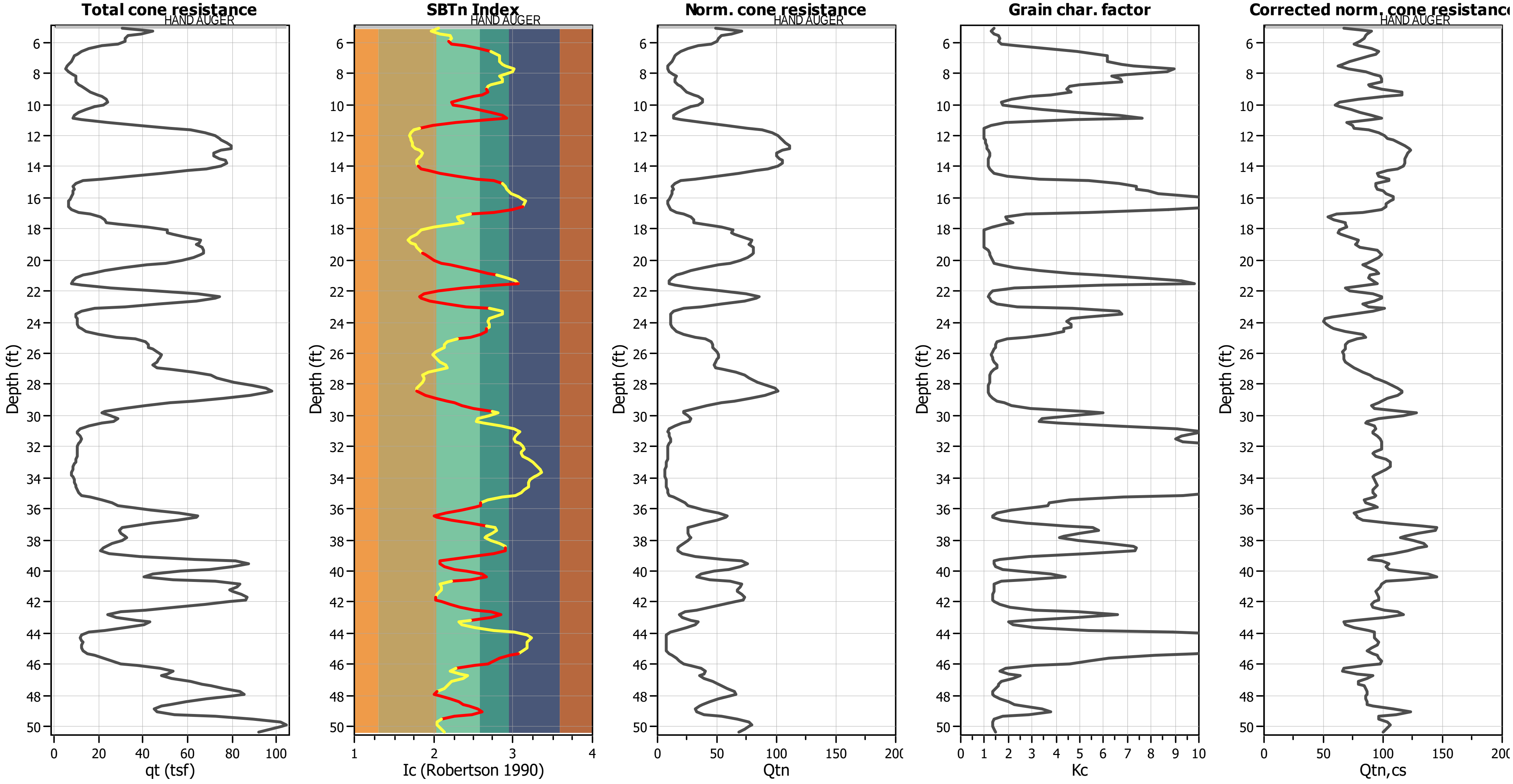
Depth to water table (erthq.): 4.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

SBTn legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

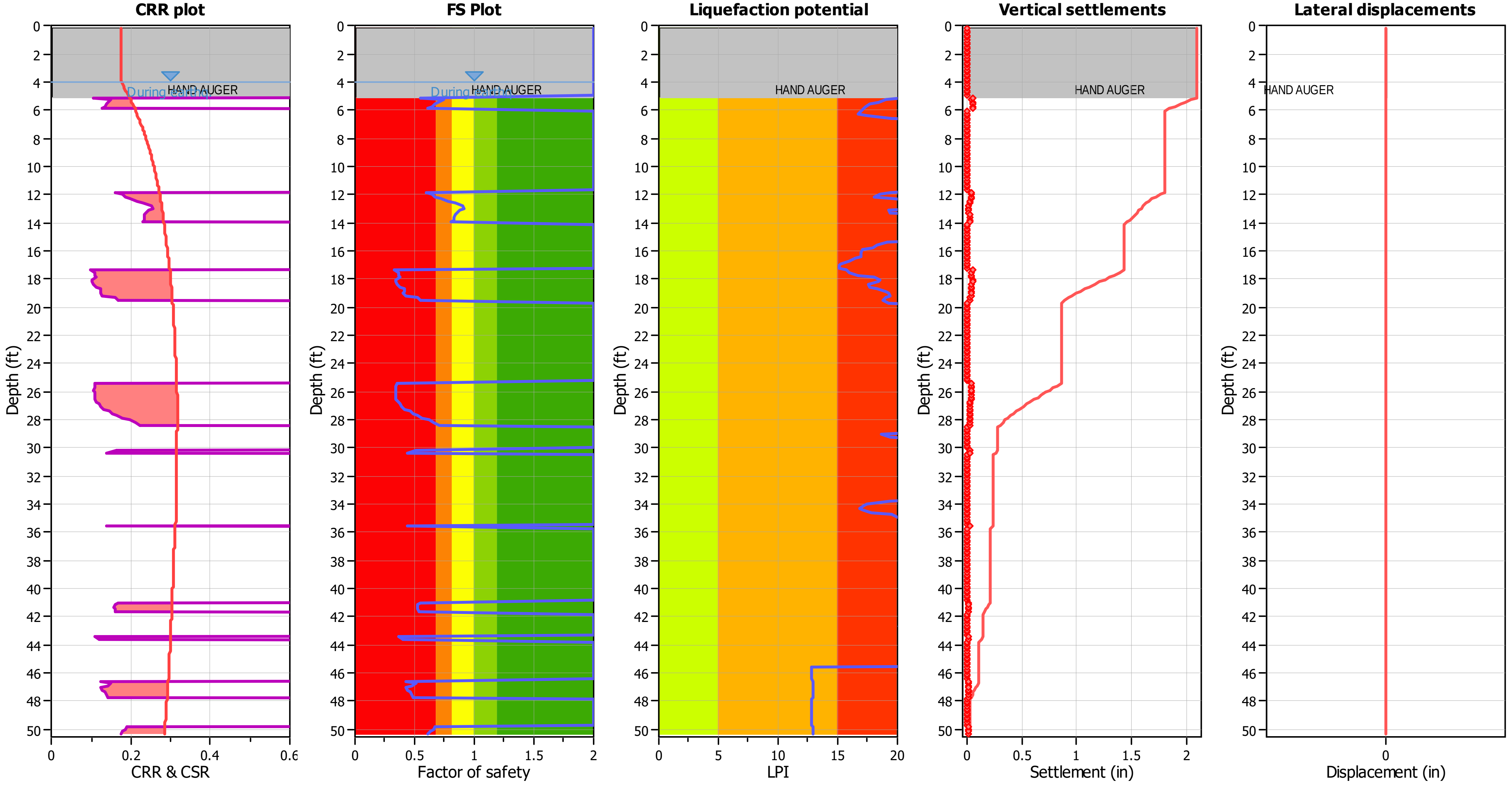
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

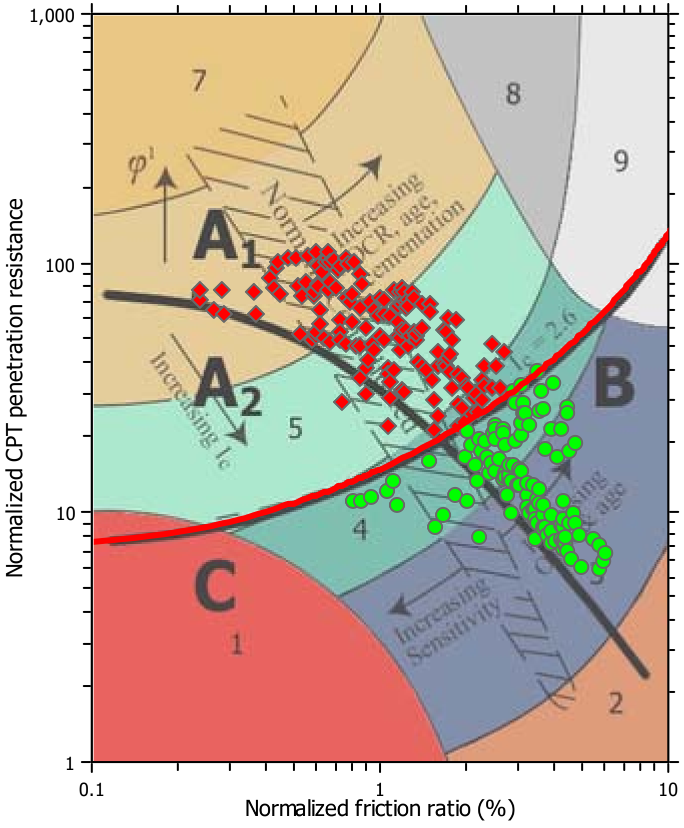
F.S. color scheme

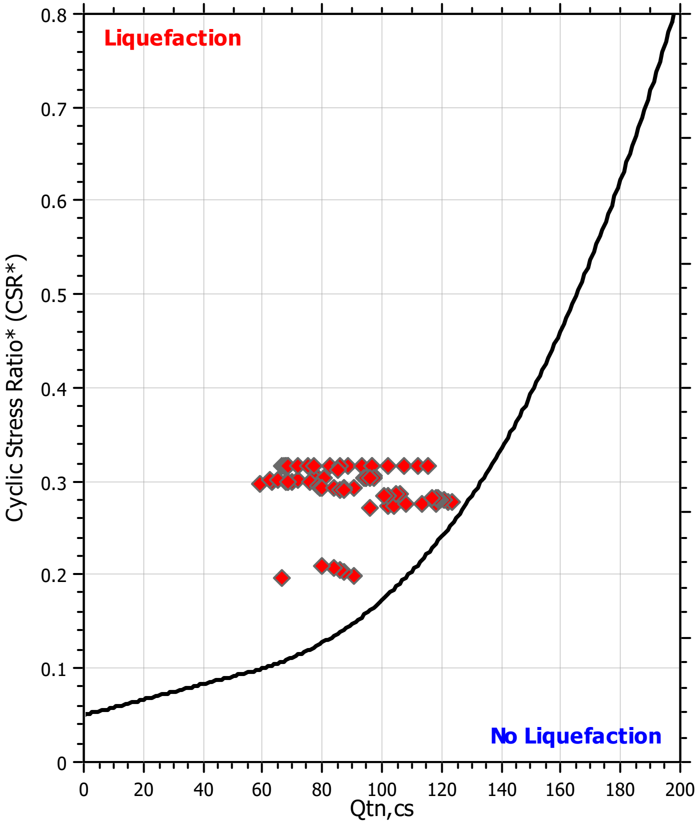
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

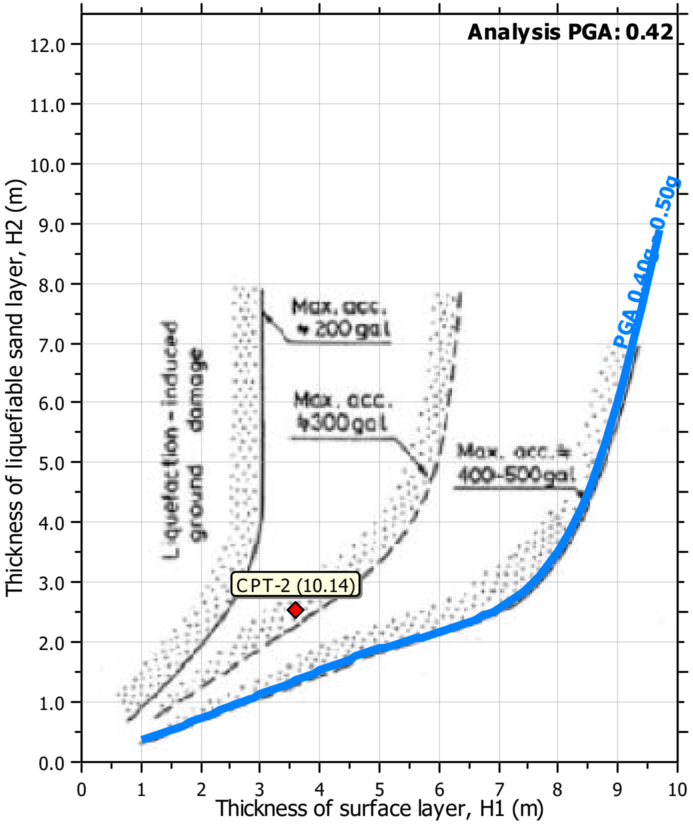
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



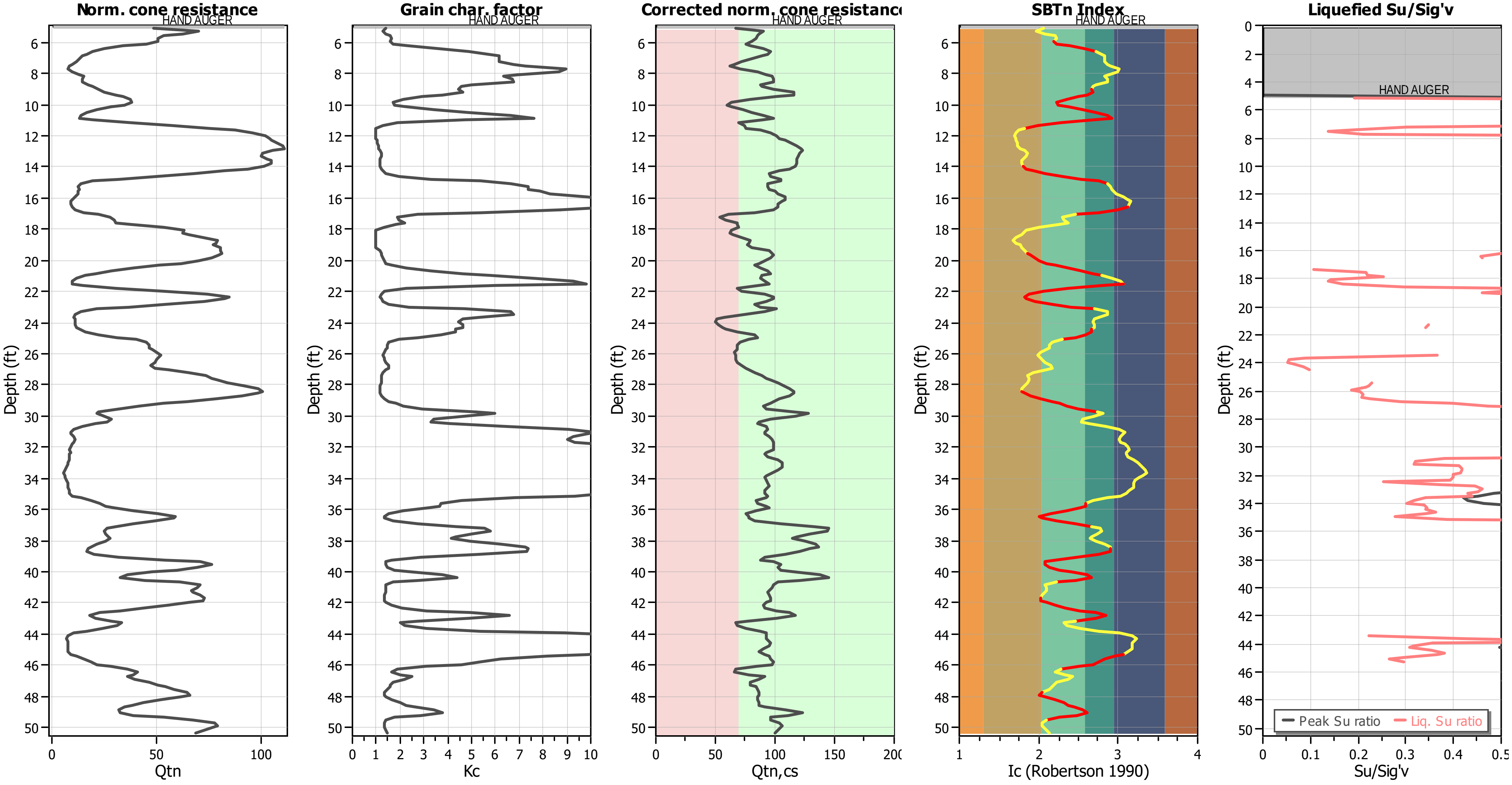




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

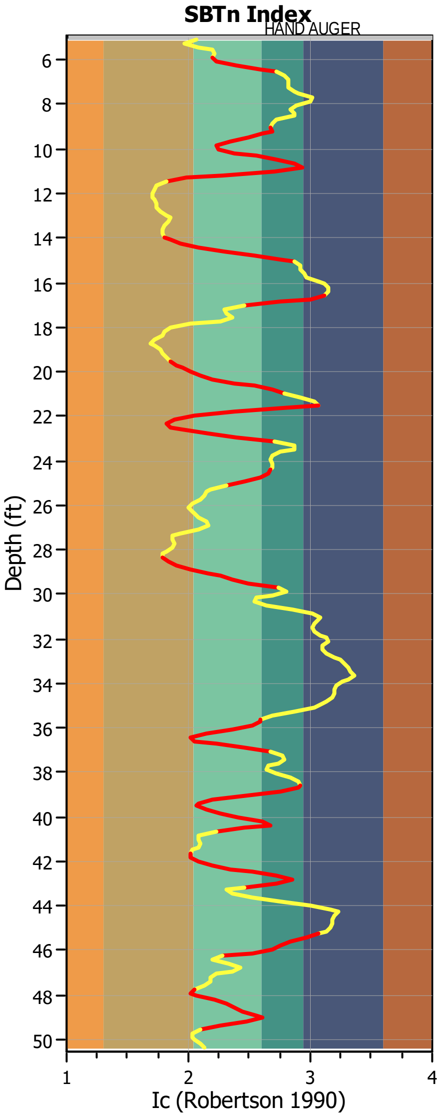
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

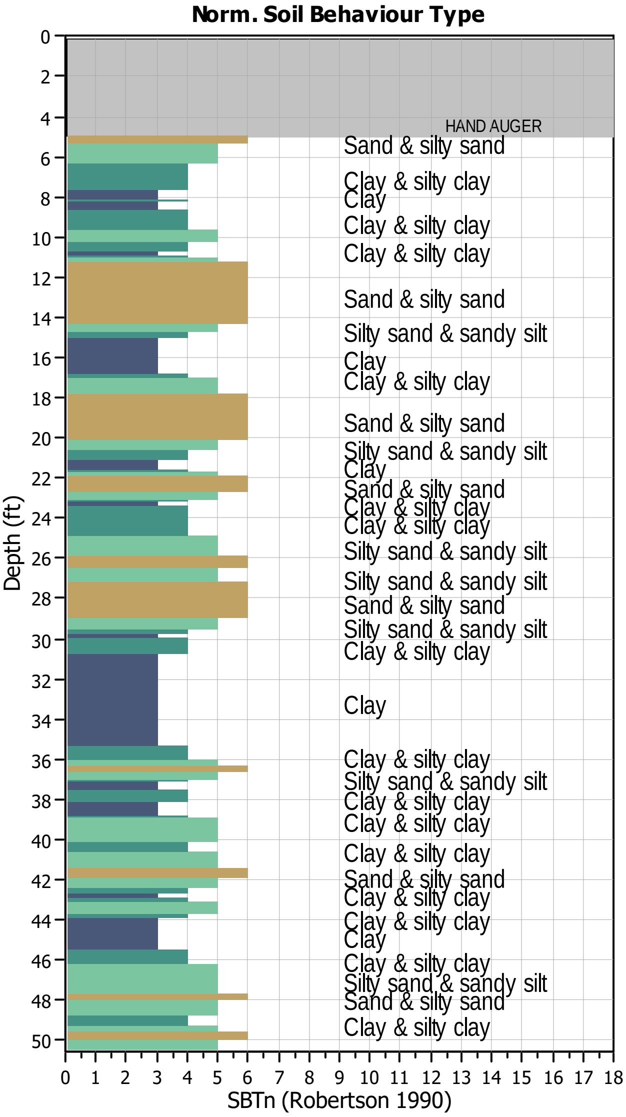
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





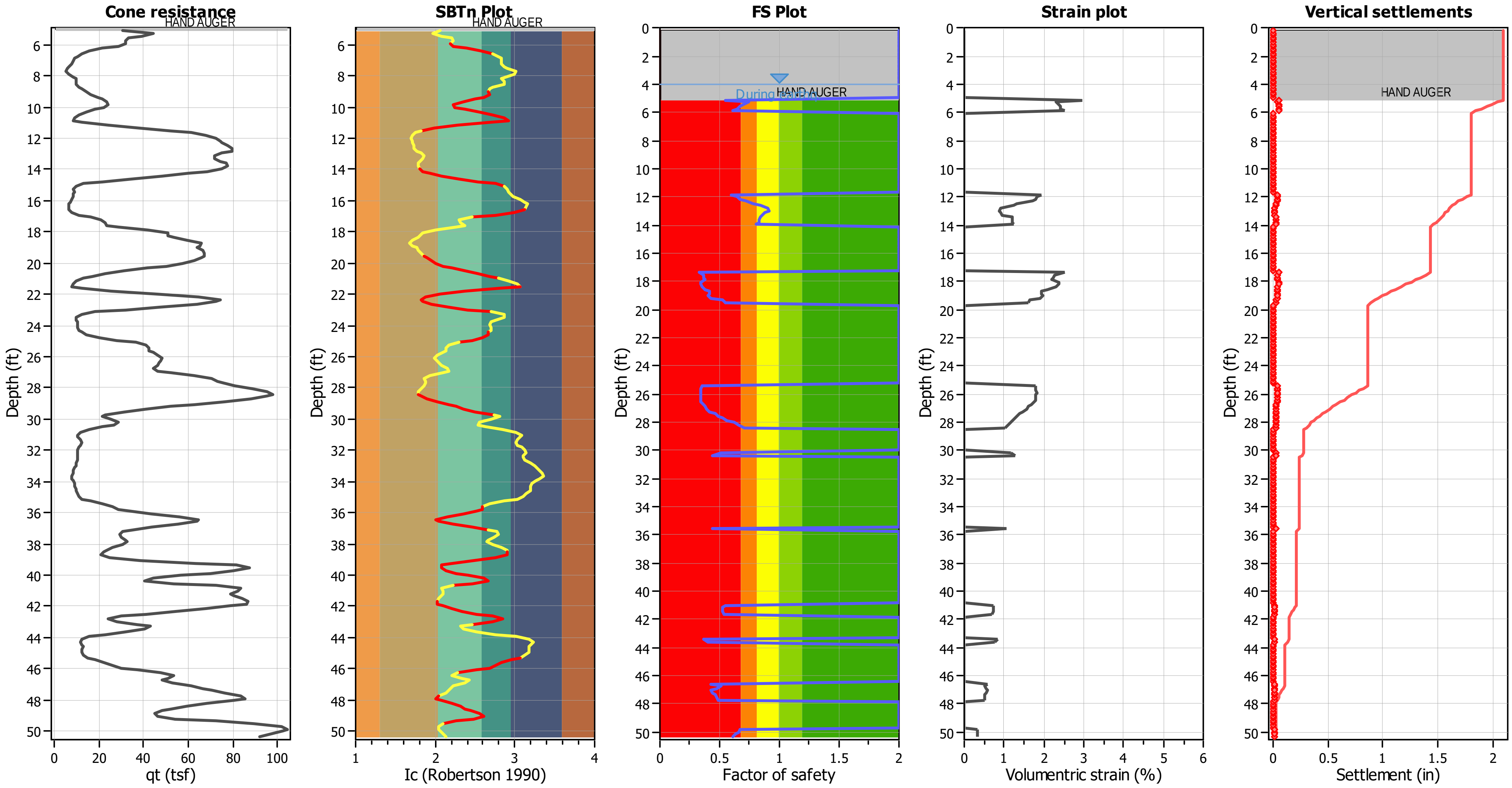
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	307
Total points excluded:	124
Exclusion percentage:	40.39%
Number of layers detected:	21

Estimation of post-earthquake settlements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
FS: Calculated Factor of Safety against liquefaction
Volumentric strain: Post-liquefaction volumentric strain

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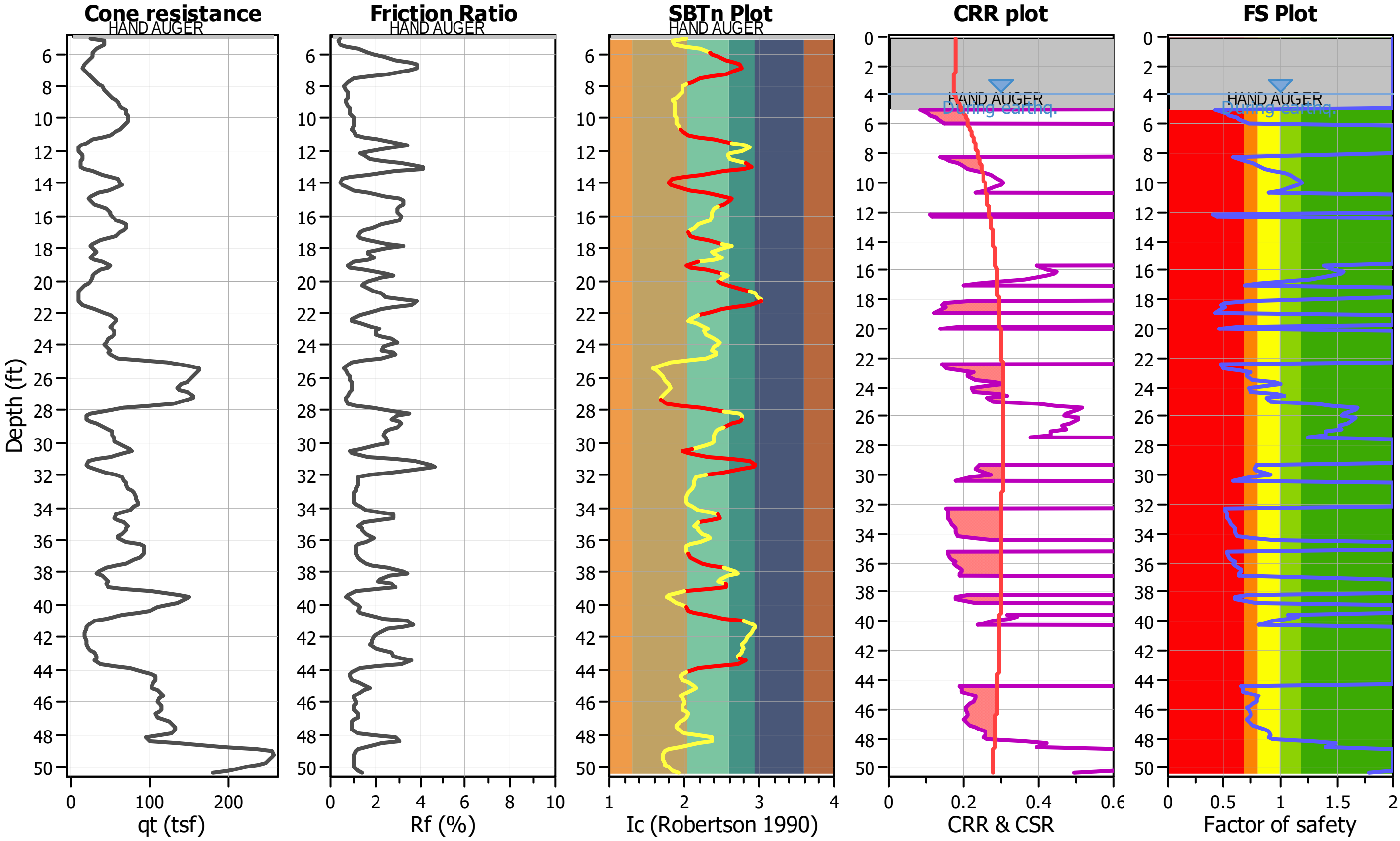
LIQUEFACTION ANALYSIS REPORT

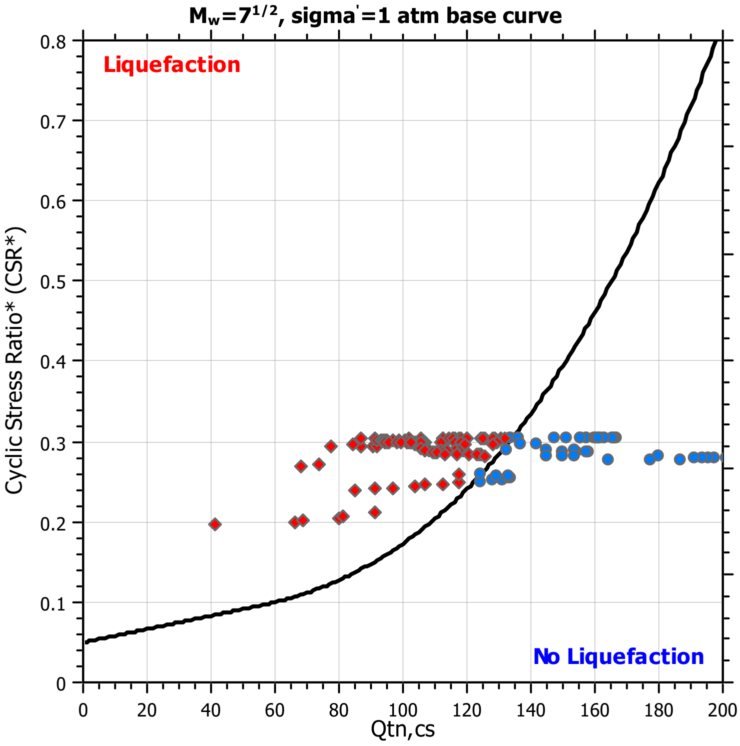
Project title : W2045-88-01 **Location : Euclid and Heil**

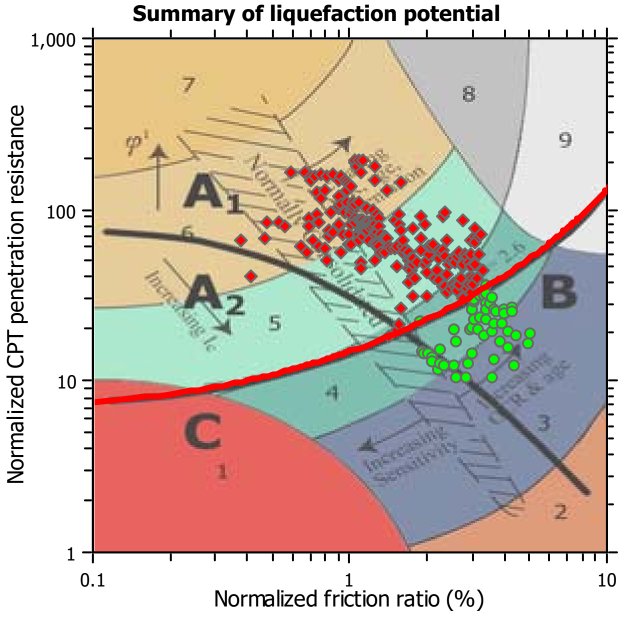
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.42	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

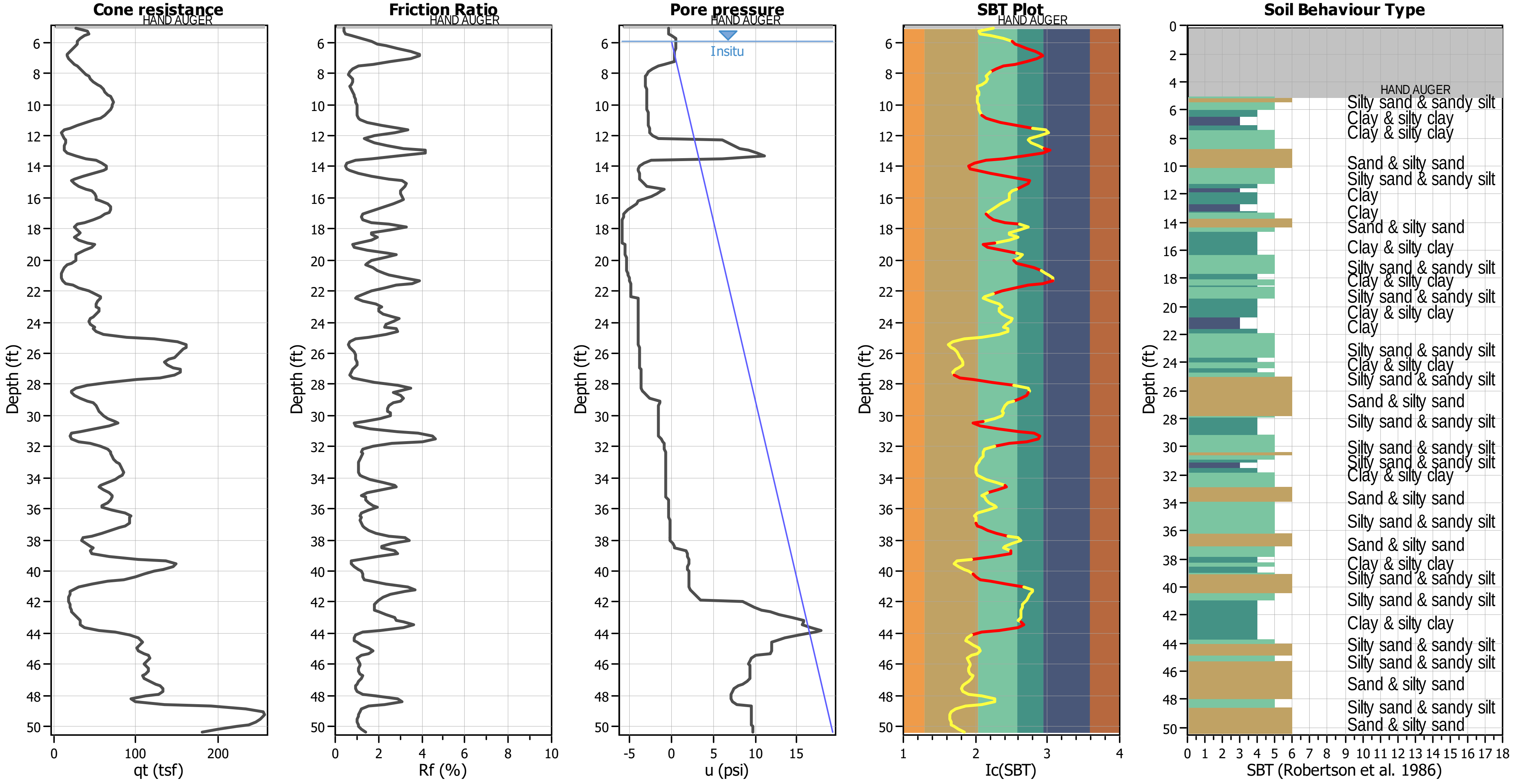






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



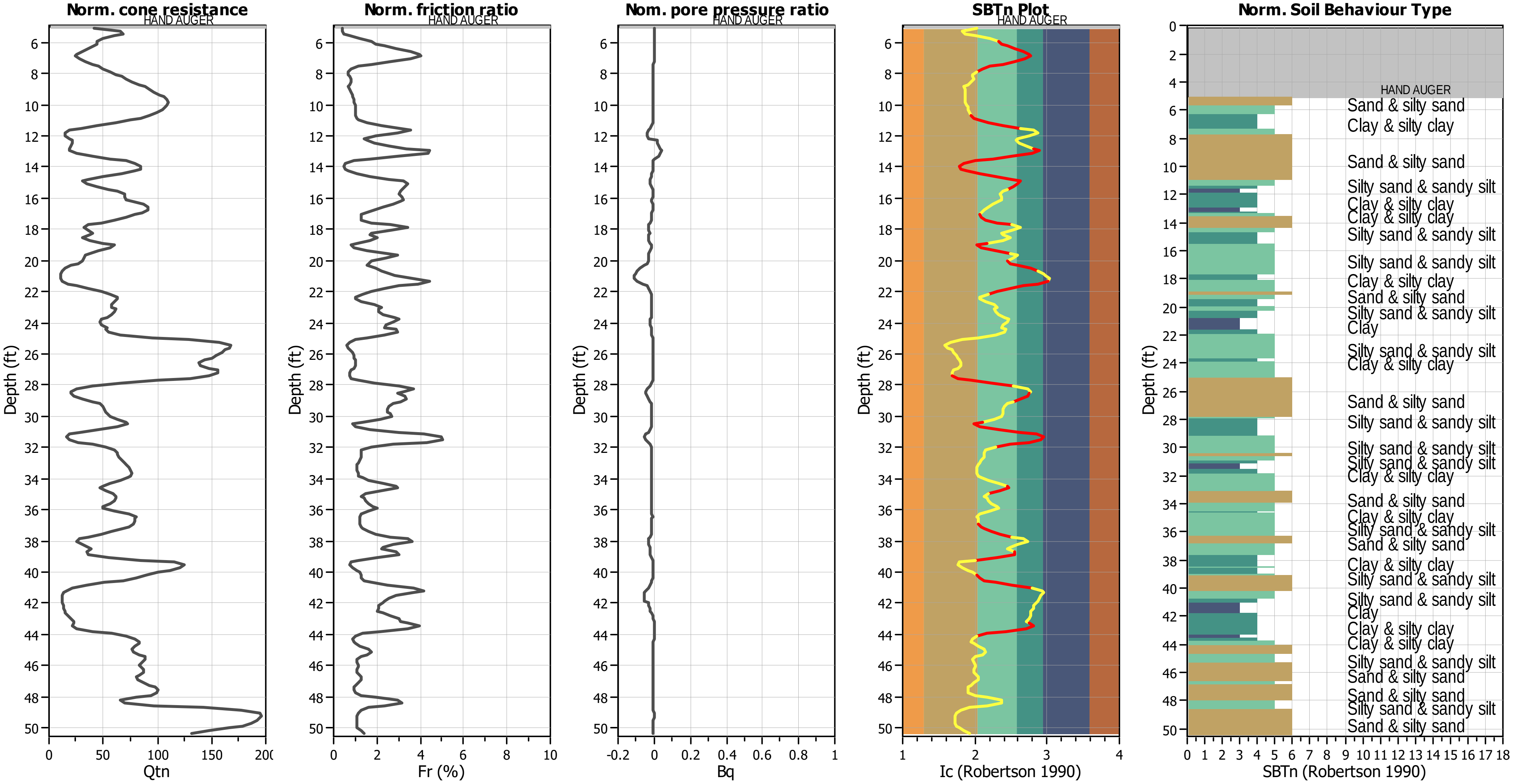
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



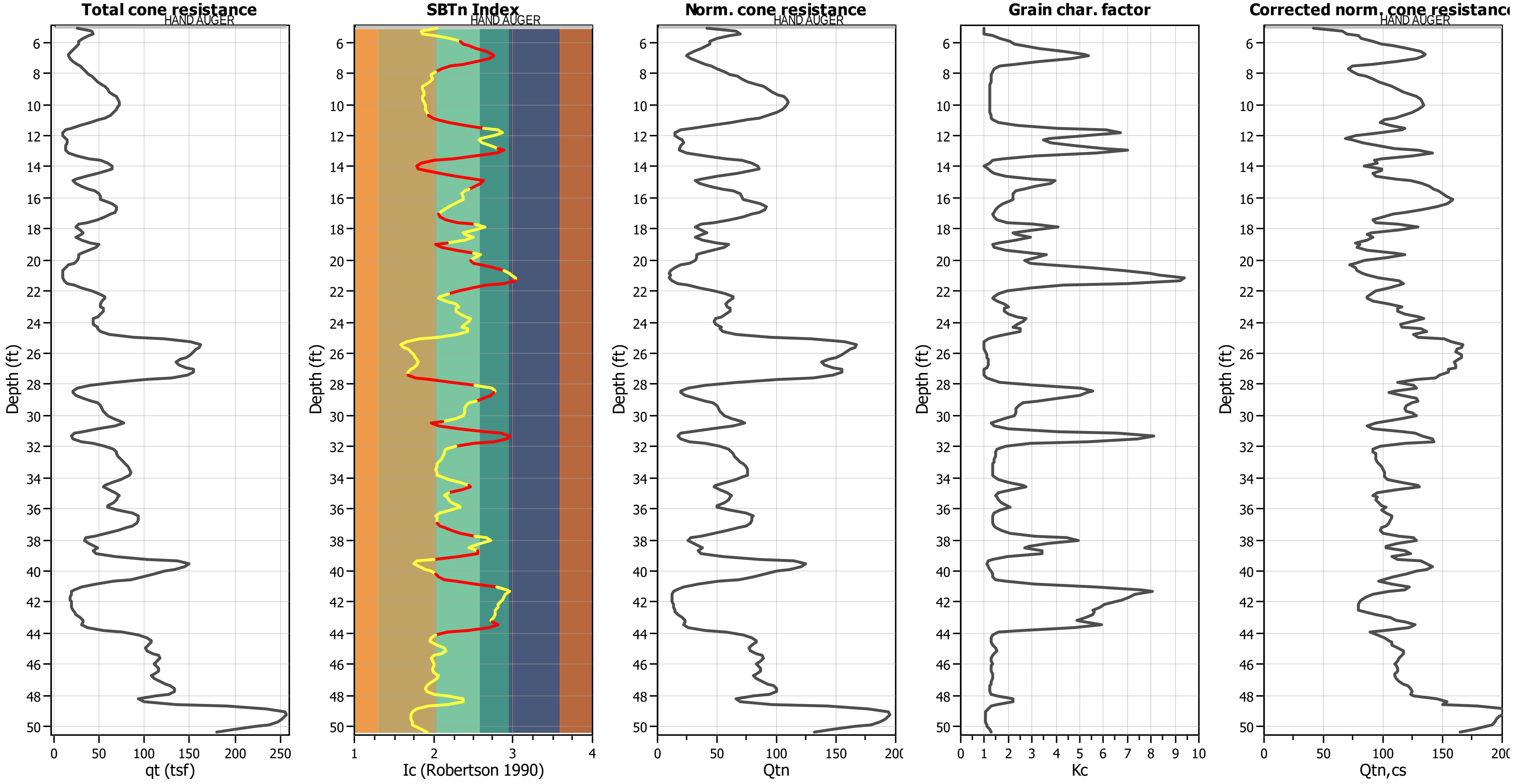
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

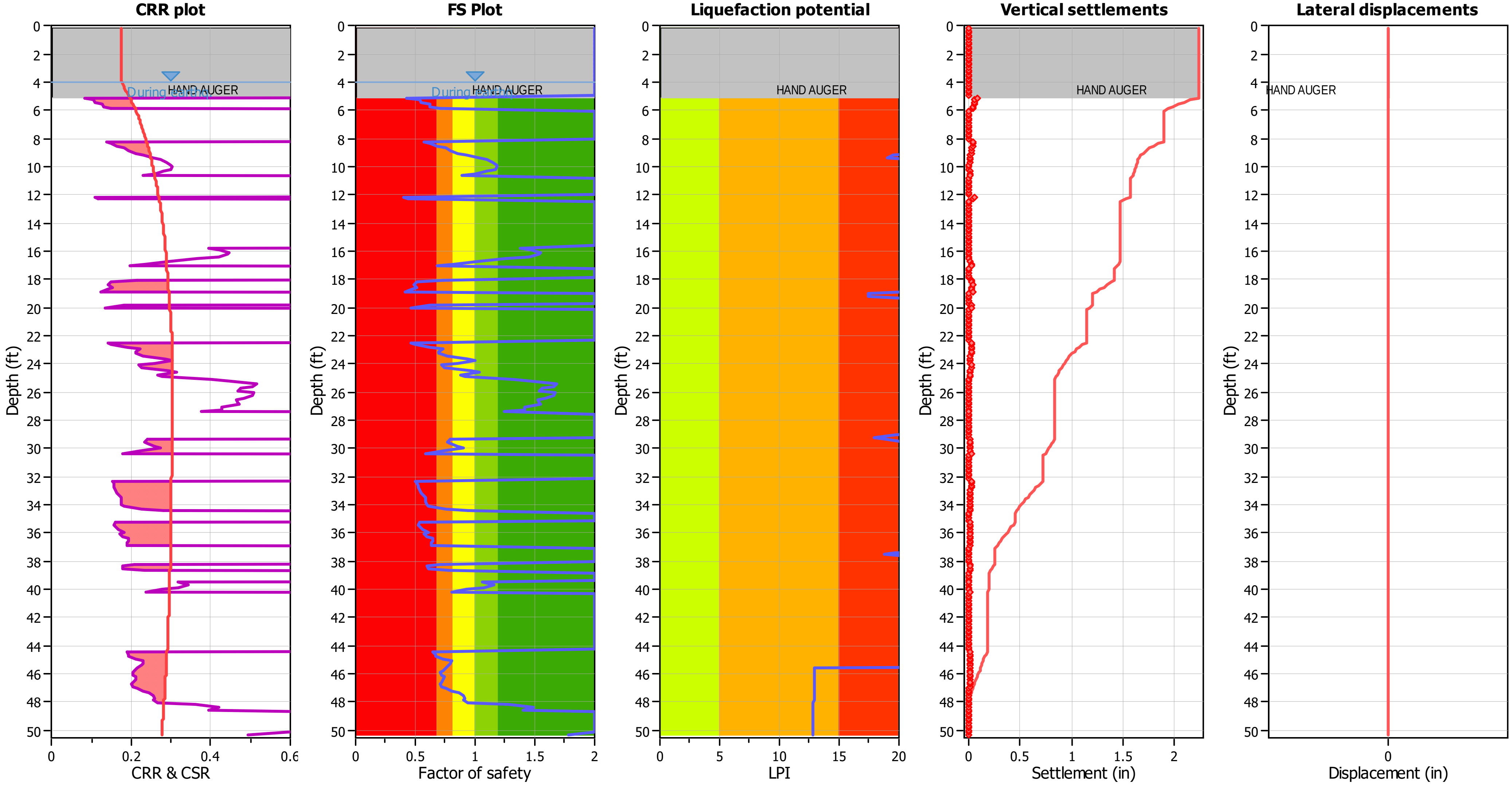
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

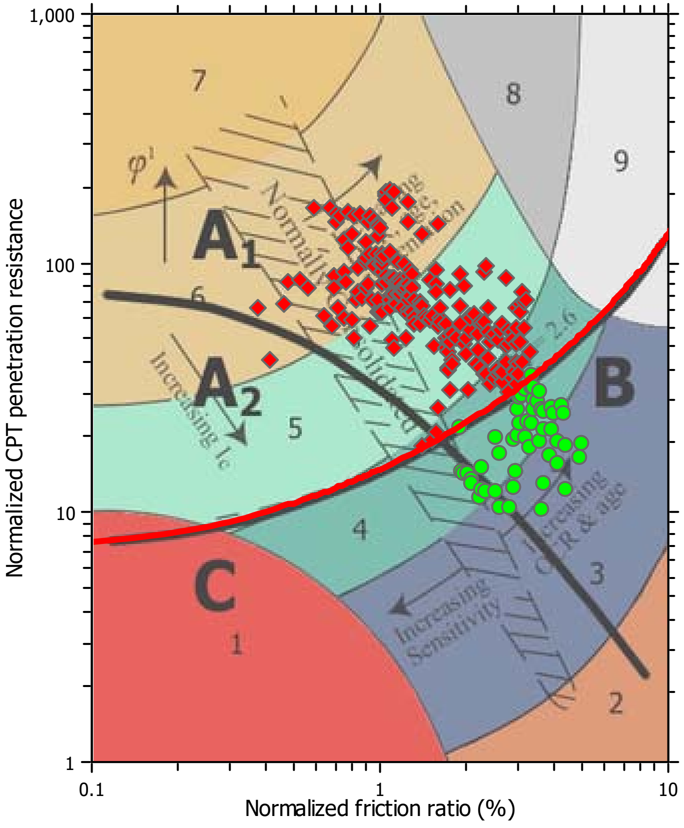
F.S. color scheme

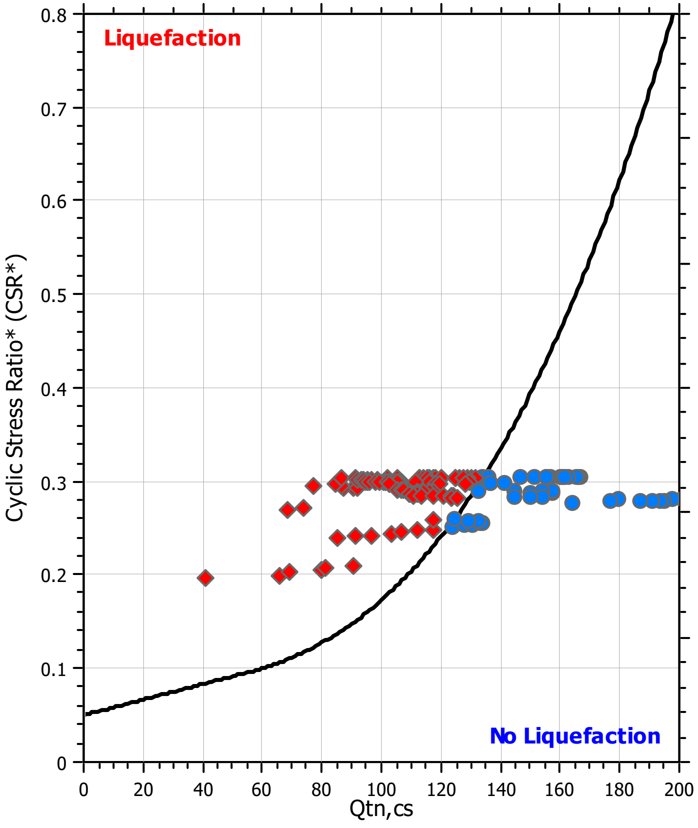
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

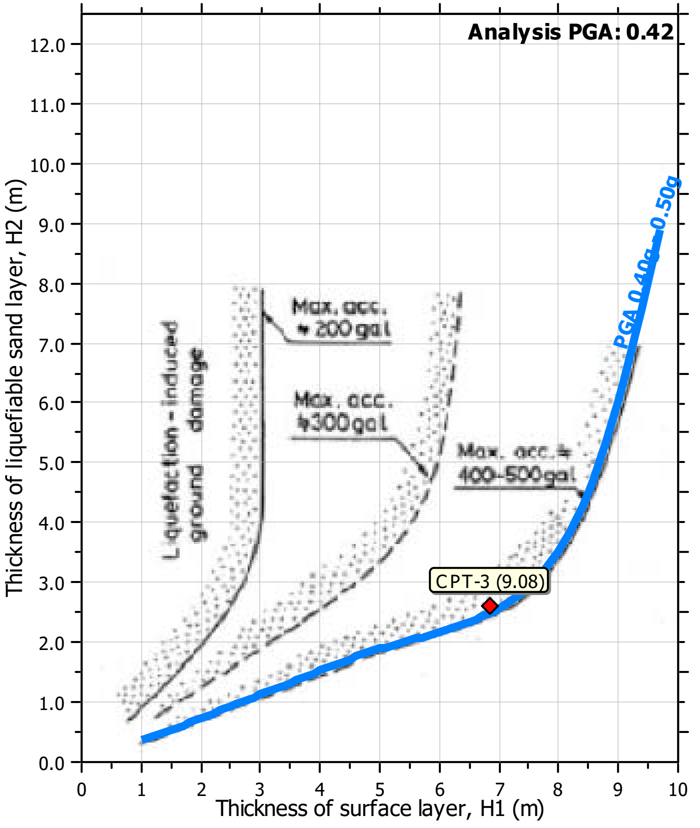
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



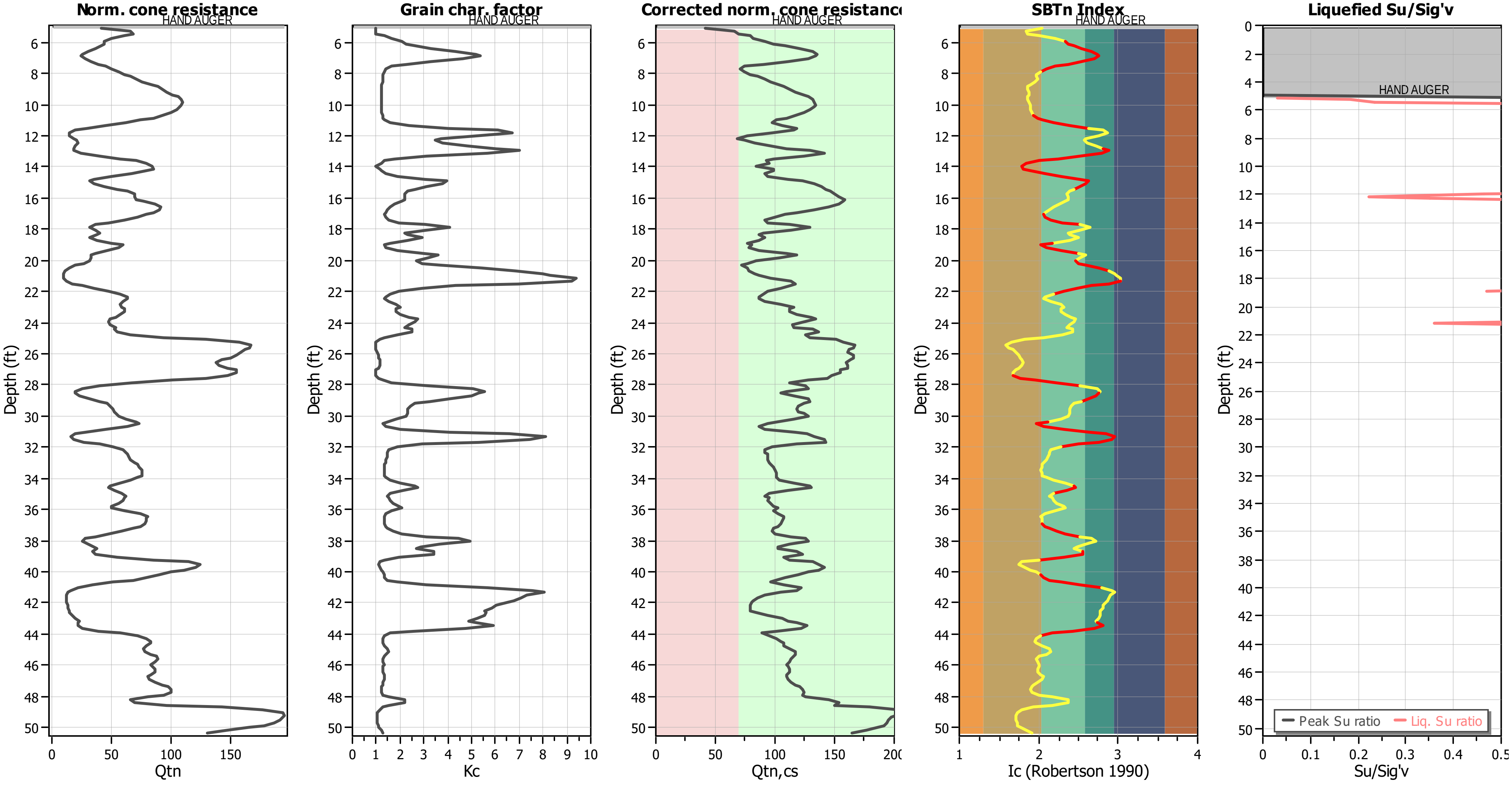




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

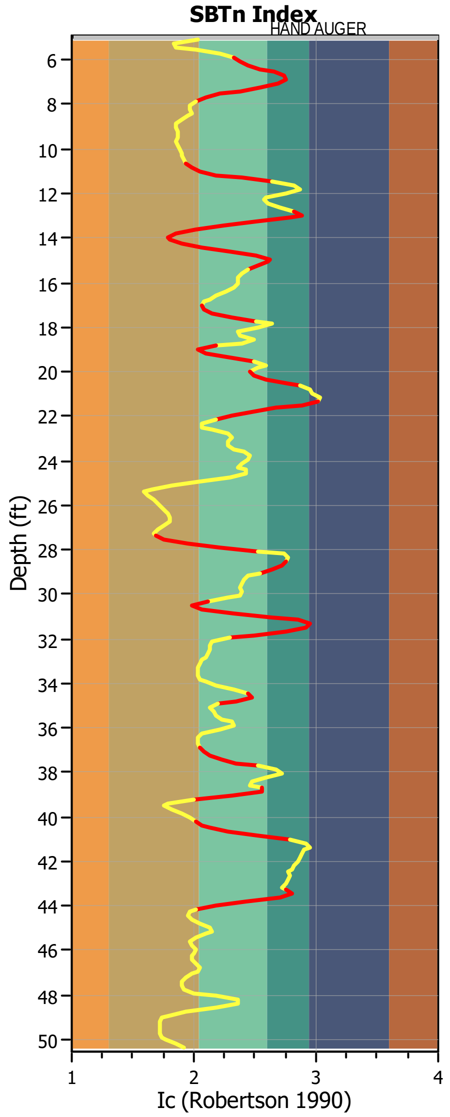
TRANSITION LAYER DETECTION ALGORITHM REPORT

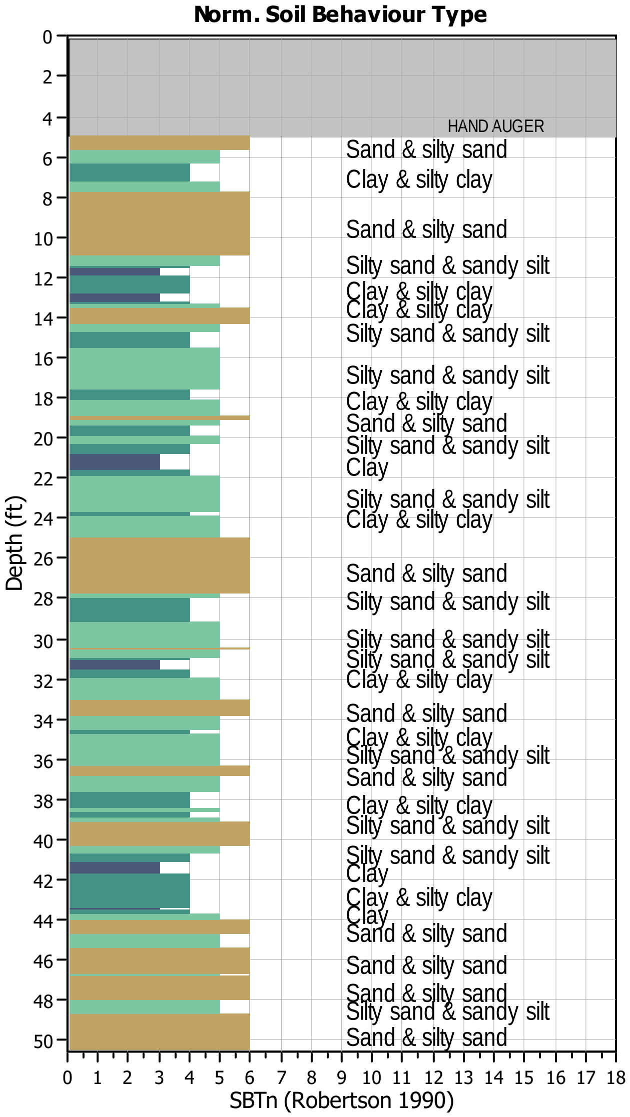
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between 1.80 < I_c < 3.0) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.


SBTn Index


Norm. Soil Behaviour Type

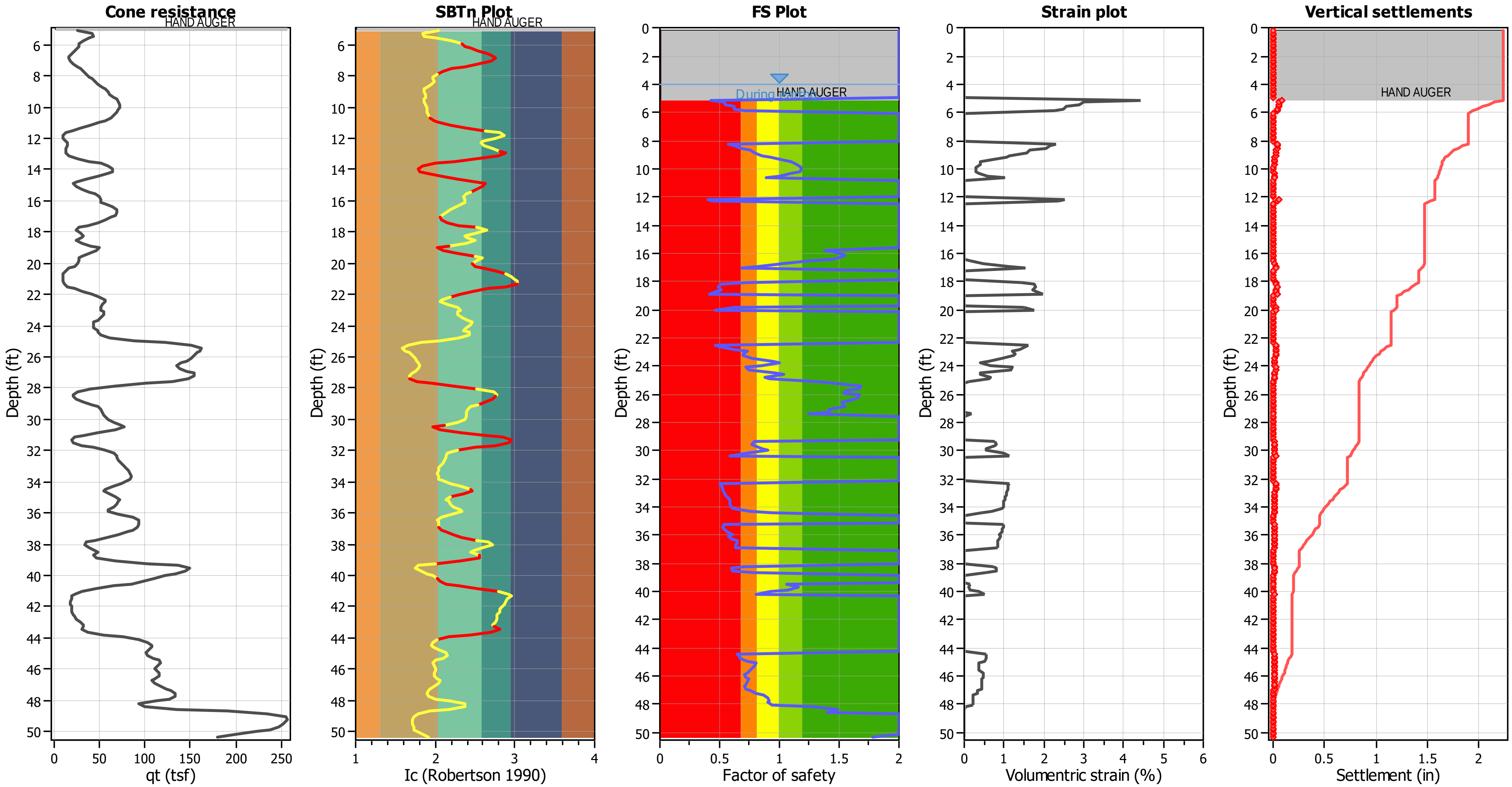
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	307
Total points excluded:	103
Exclusion percentage:	33.55%
Number of layers detected:	19

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

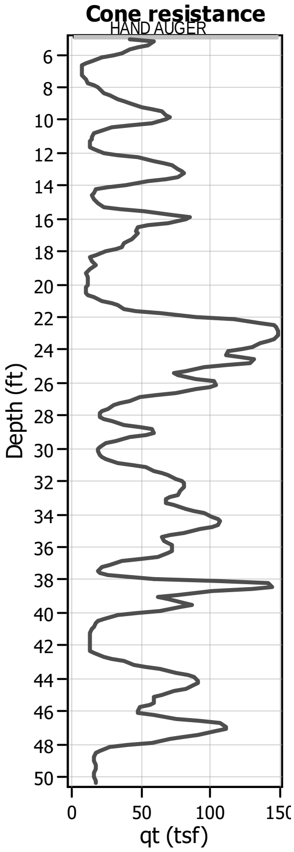
LIQUEFACTION ANALYSIS REPORT

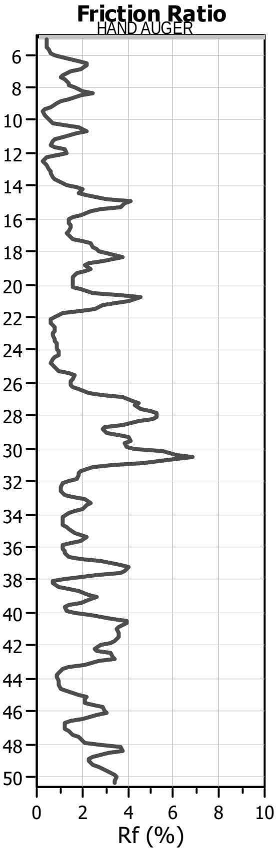
Project title : W2045-88-01 **Location : Euclid and Heil**

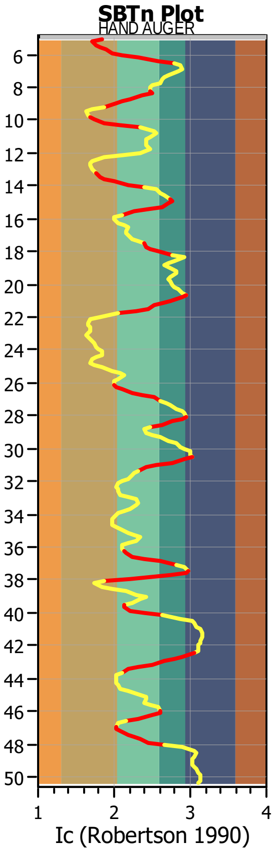
CPT file : CPT-4

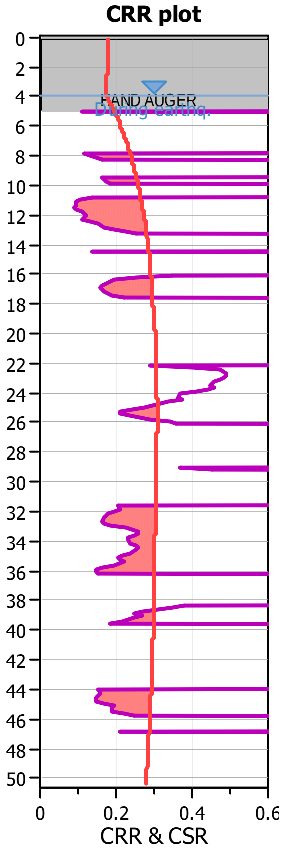
Input parameters and analysis data

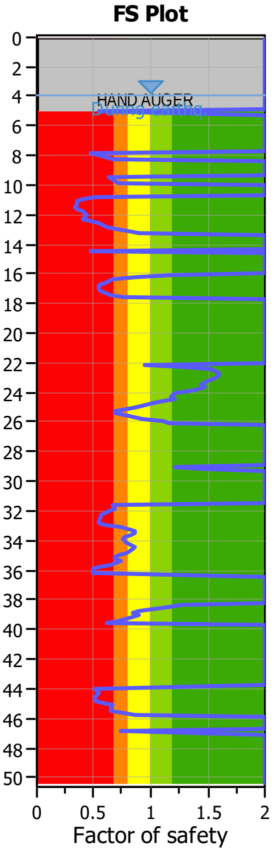
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.42	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

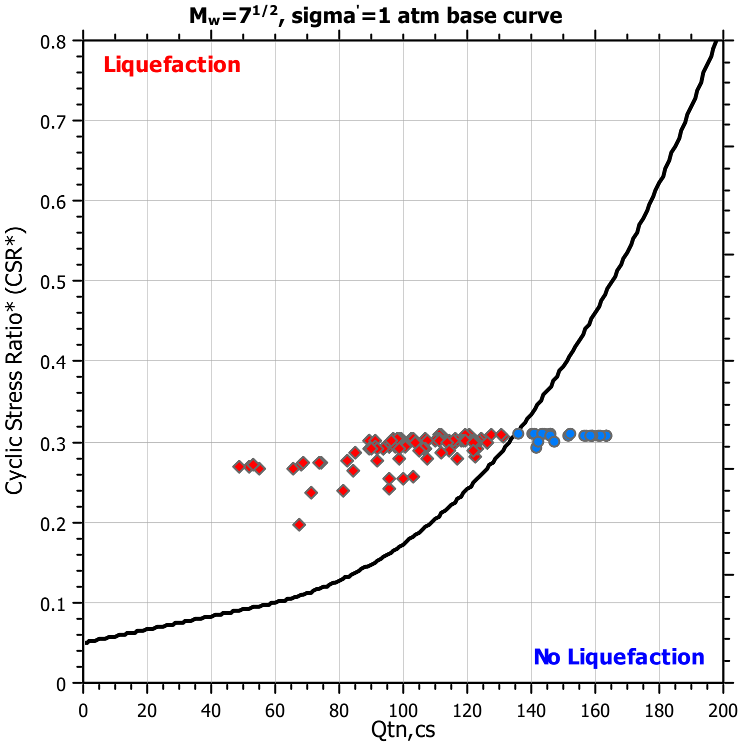


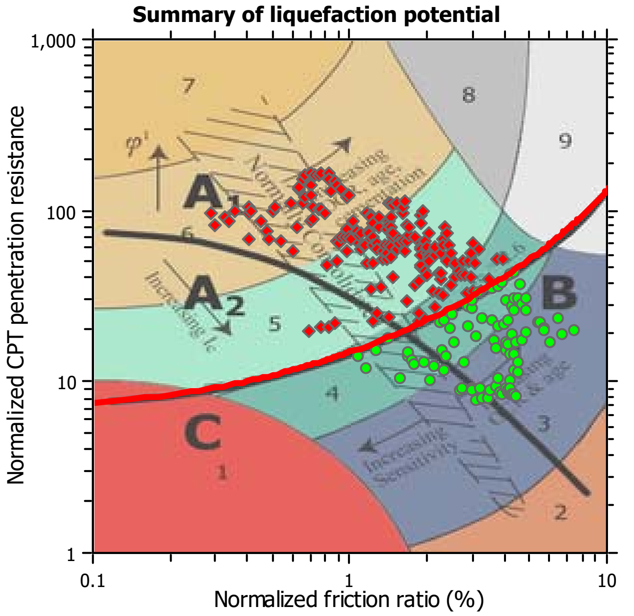






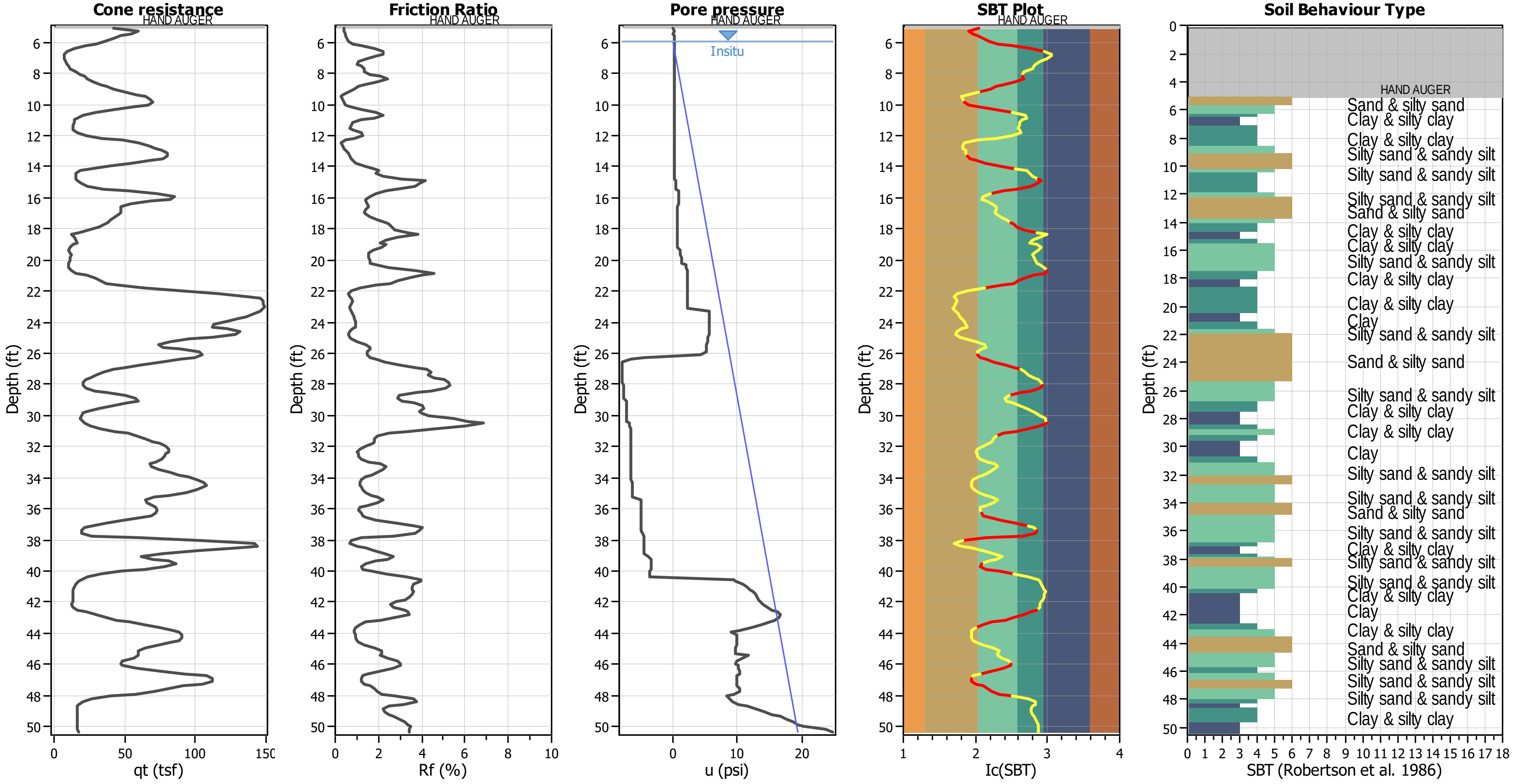






Zone A$_1$: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A$_2$: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 6.10
Peak ground acceleration: 0.42
Depth to water table (insitu): 5.90 ft

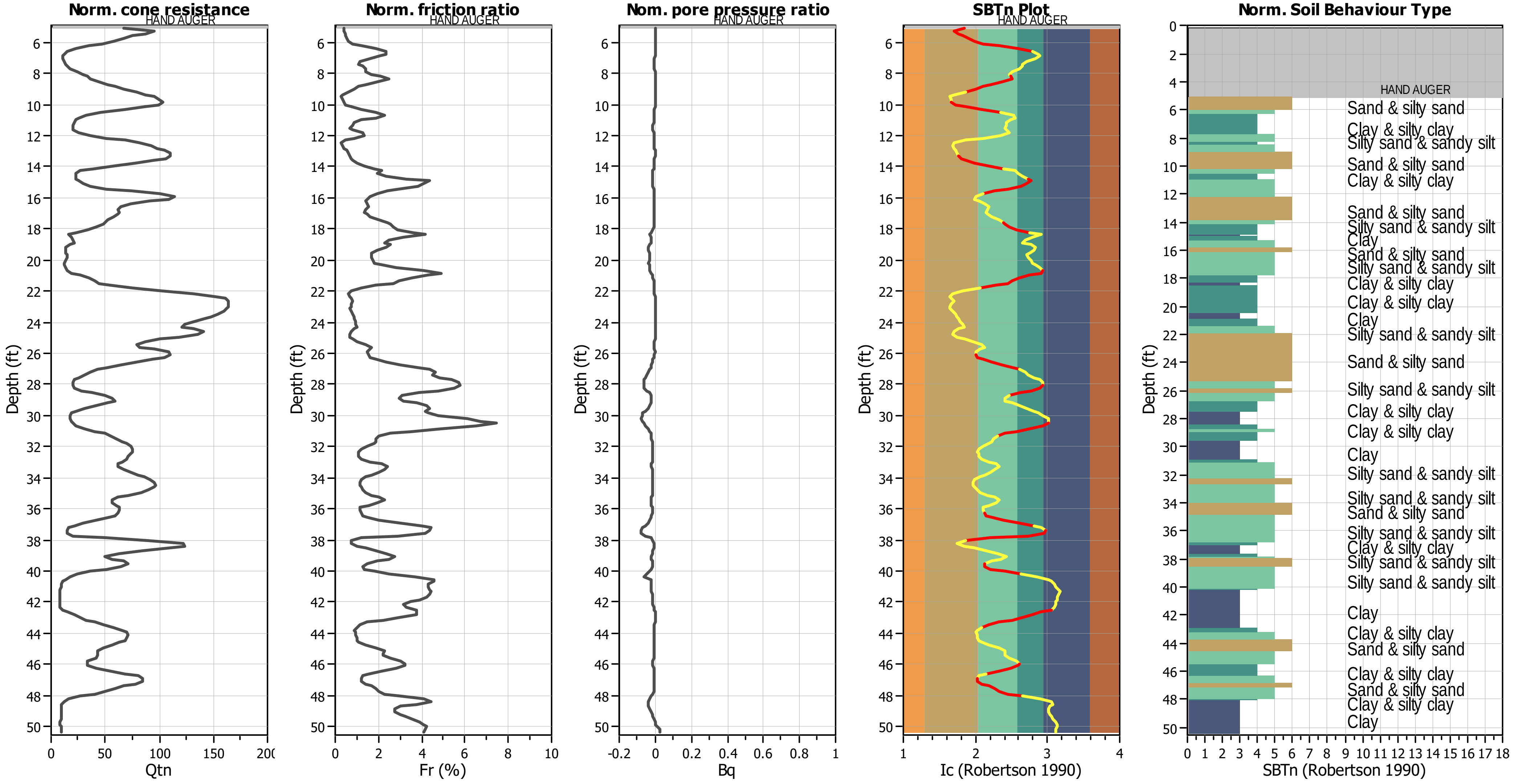
Depth to water table (erthq.): 4.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



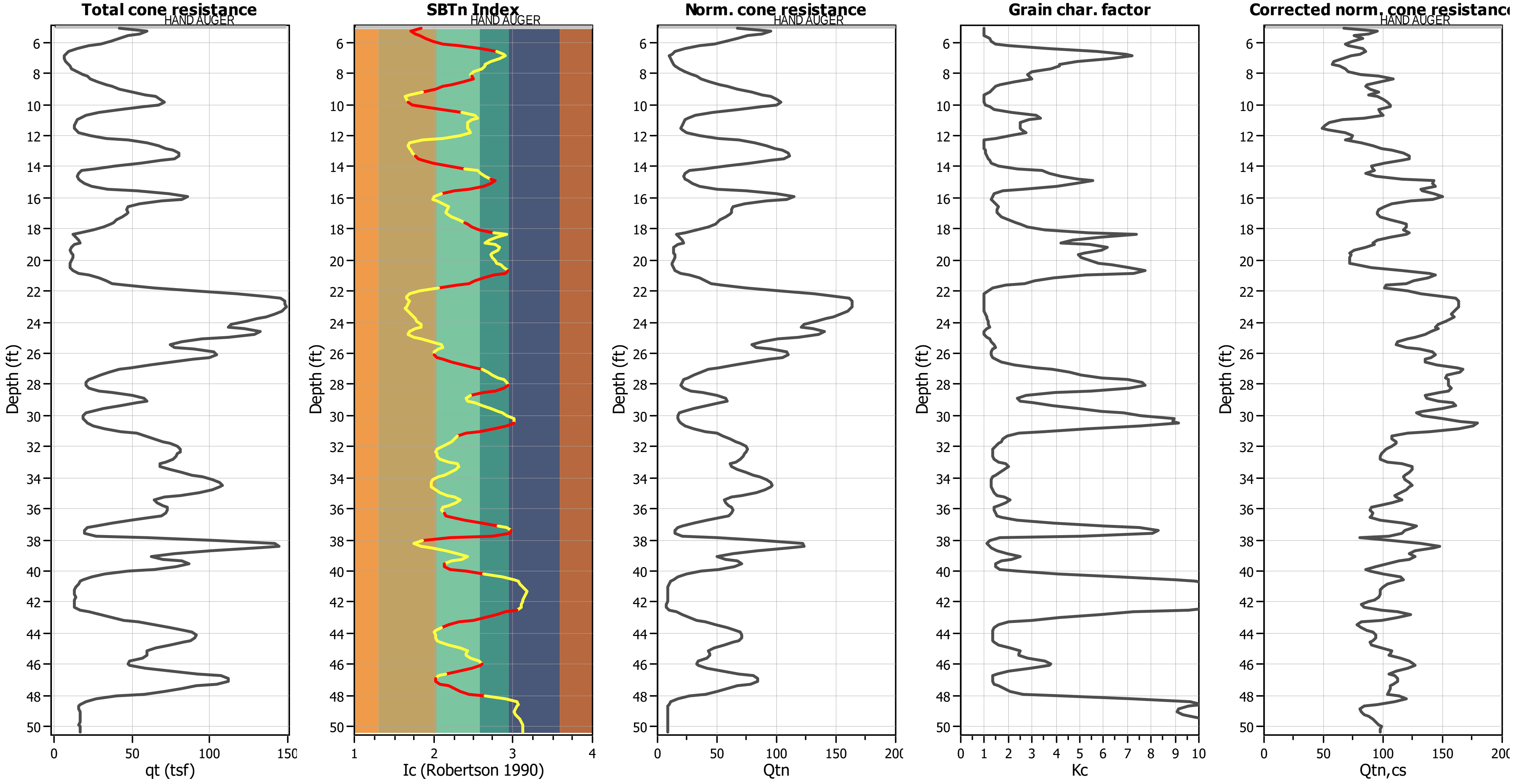
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

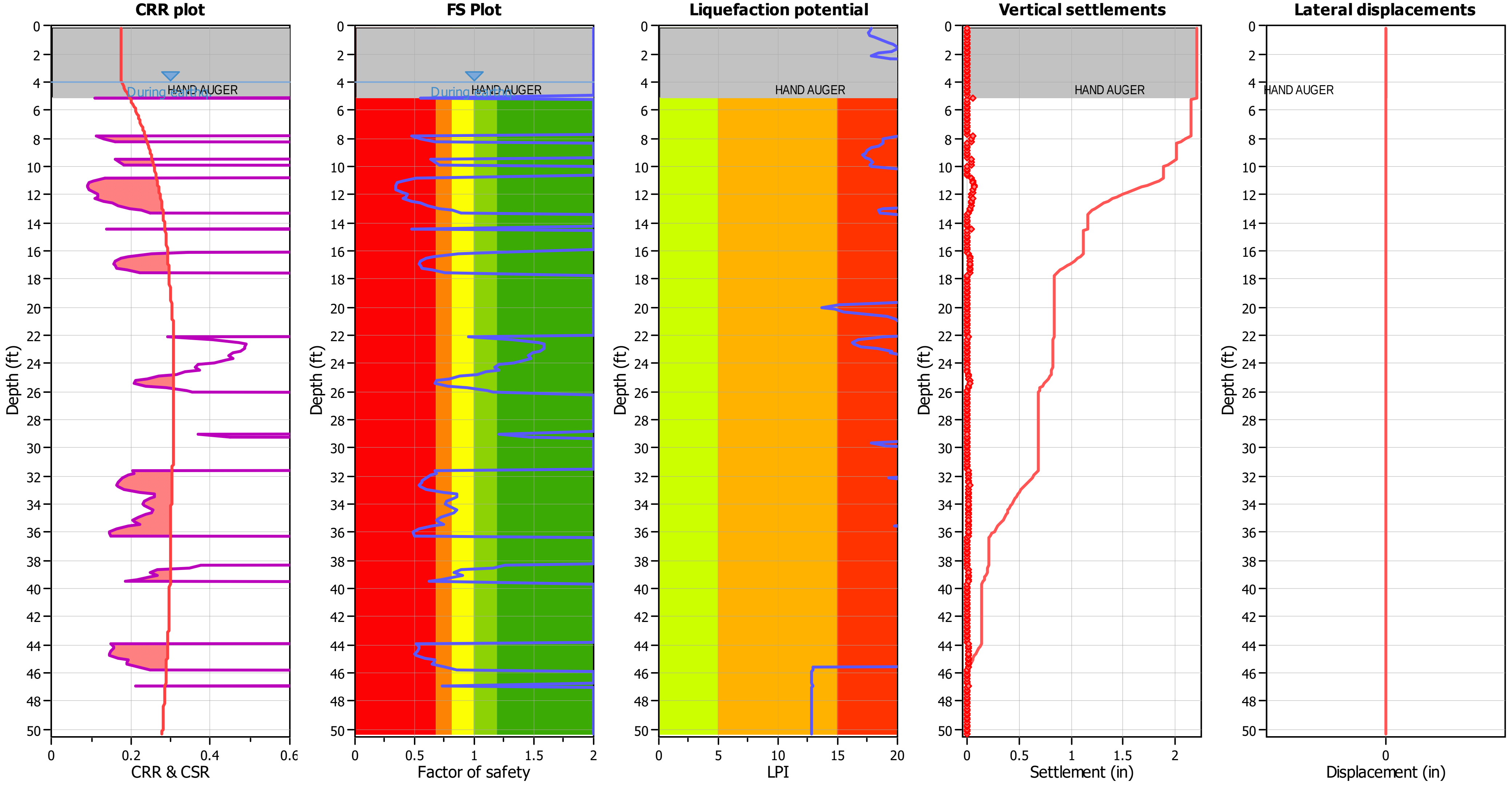
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

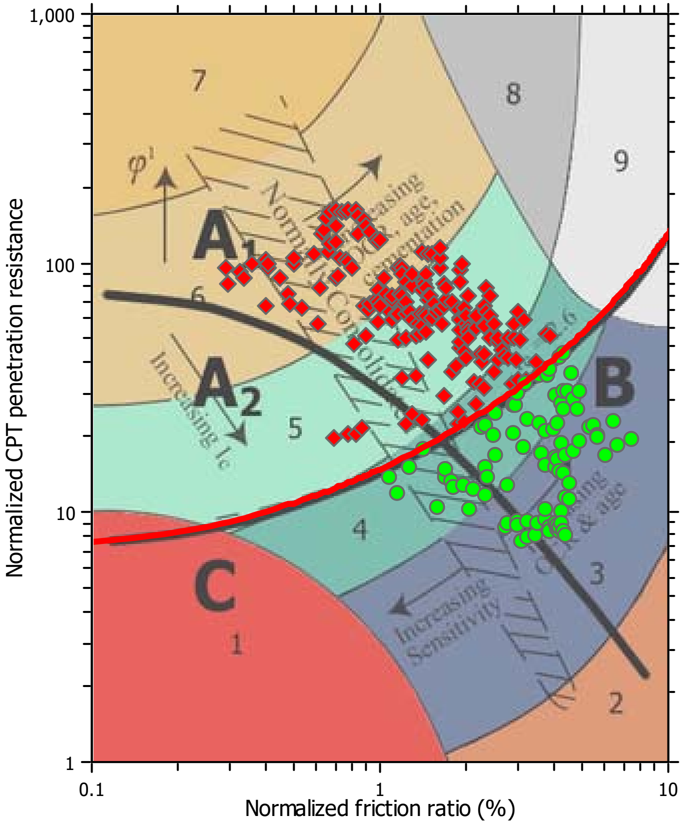
F.S. color scheme

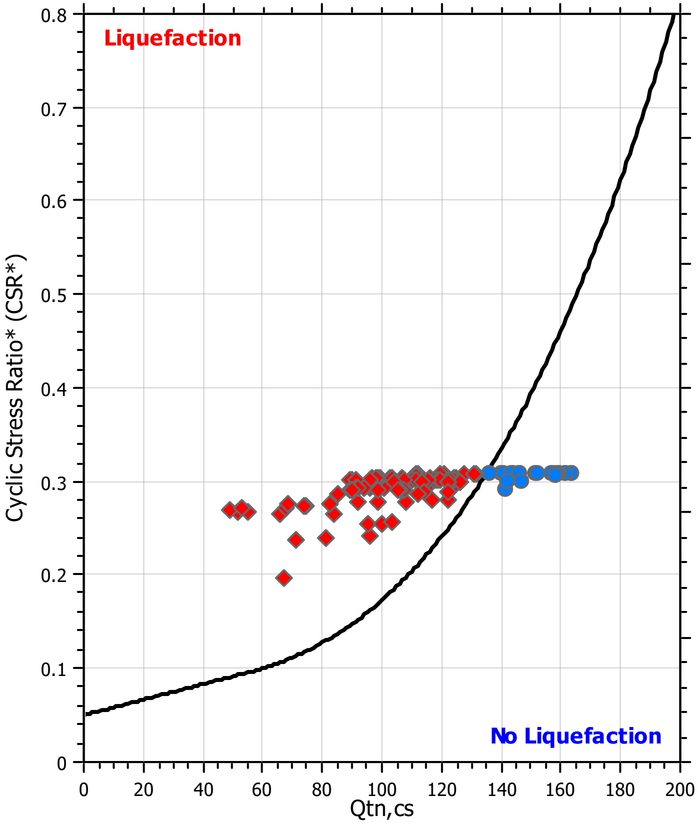
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

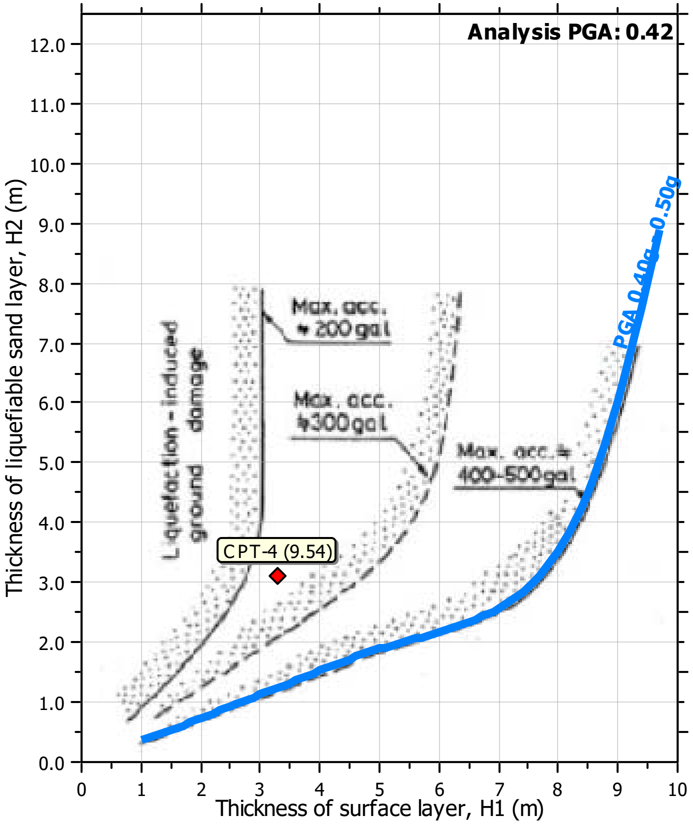
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

Liquefaction analysis summary plots



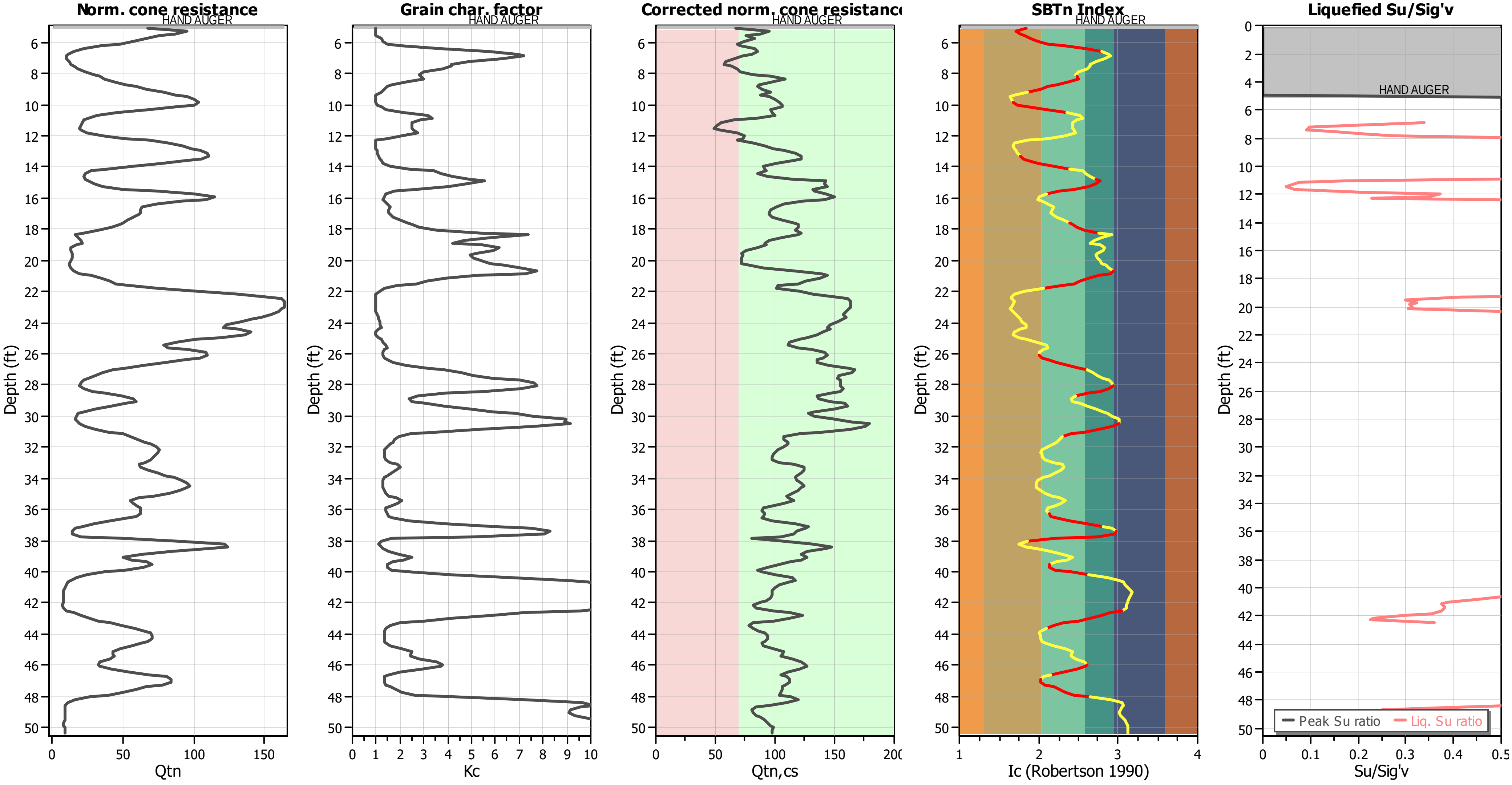




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

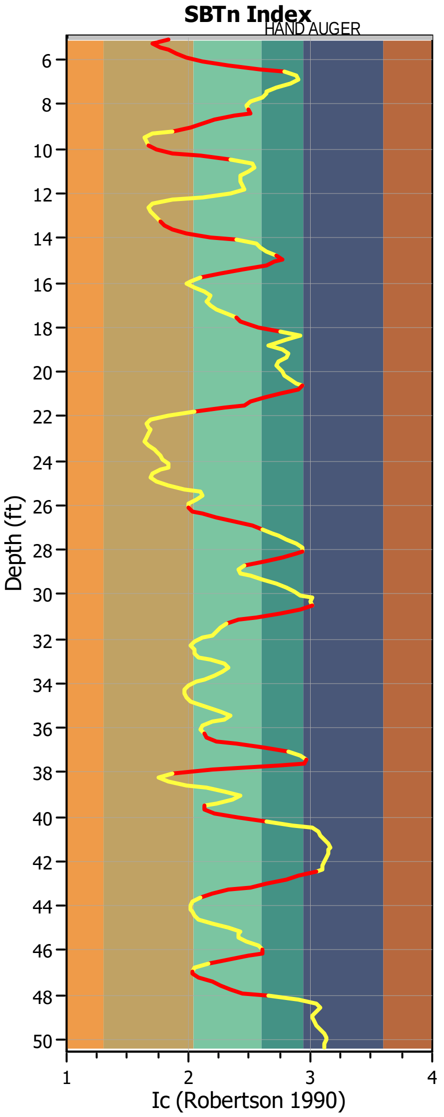
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

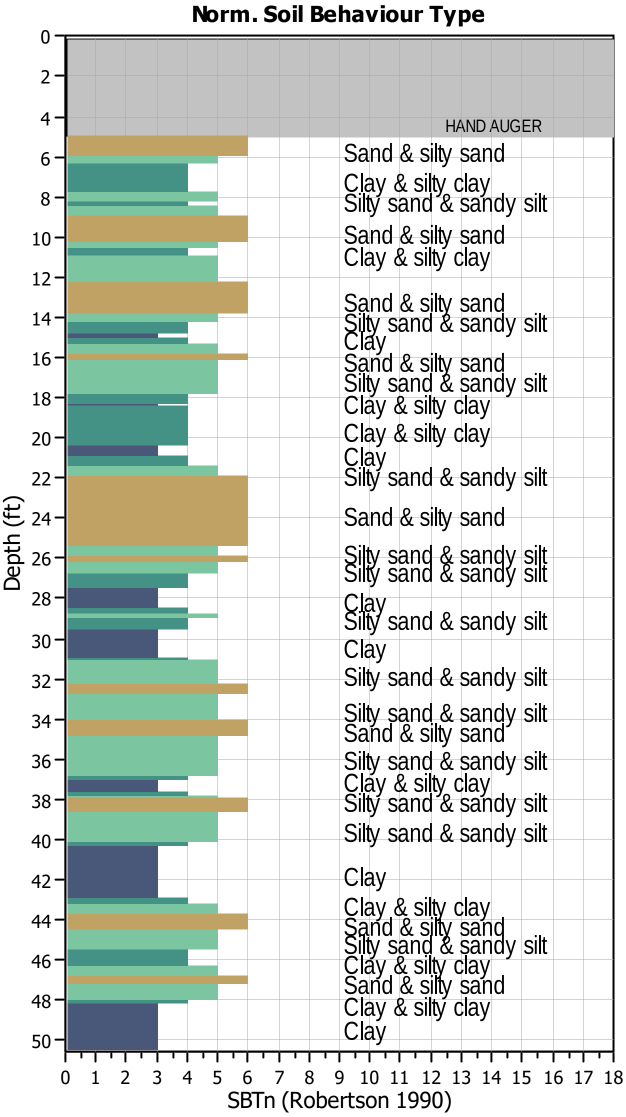
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between 1.80 < I_c < 3.0) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





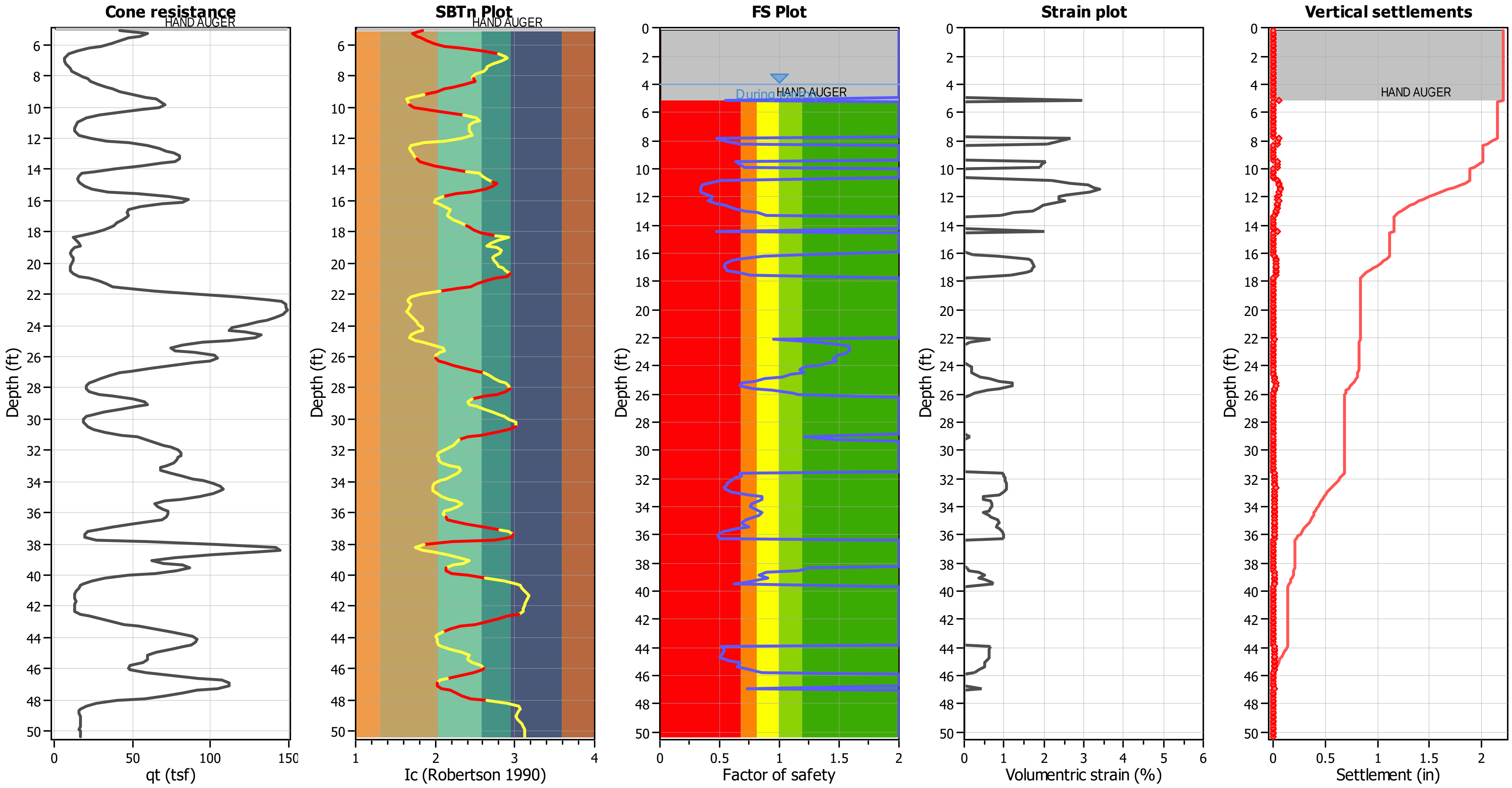
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	307
Total points excluded:	103
Exclusion percentage:	33.55%
Number of layers detected:	16

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

GEOCON

Geocon West, Inc.
2807 McGaw Avenue
Irvine, CA 92614
www.geoconinc.com

LIQUEFACTION ANALYSIS REPORT

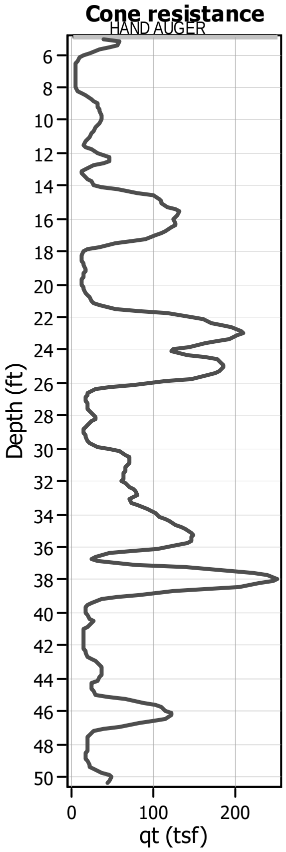
Project title : W2045-88-01 **Location : Euclid and Heil**

CPT file : CPT-5

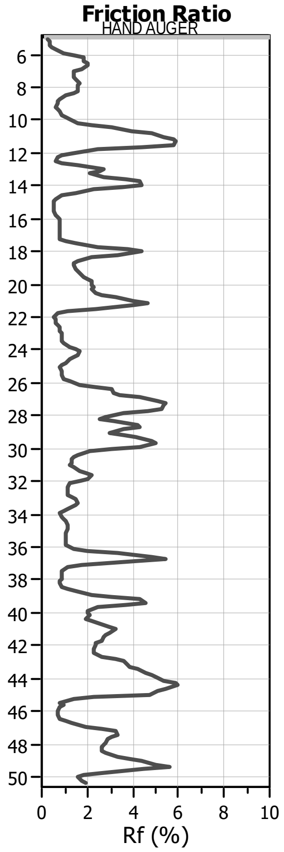
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No		
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Clay like behavior applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.10	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.42	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

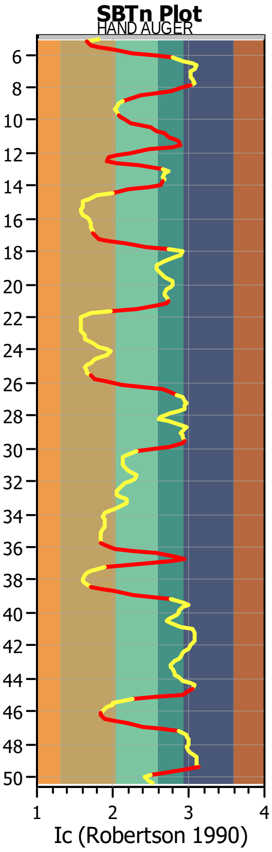
Cone resistance



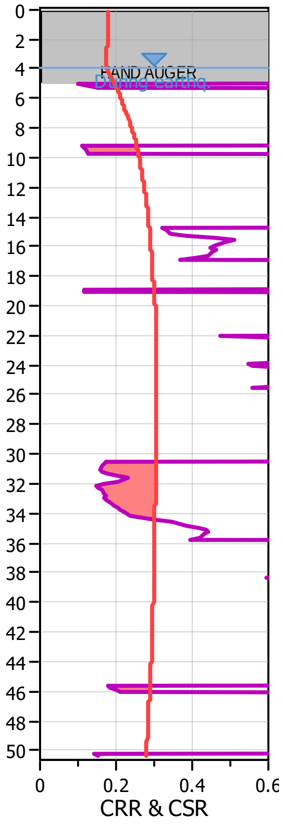
Friction Ratio



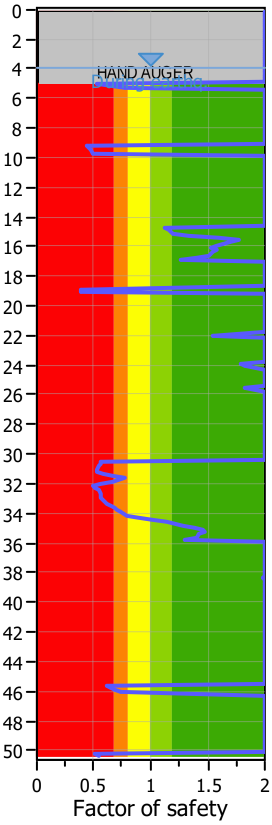
SBTn Plot



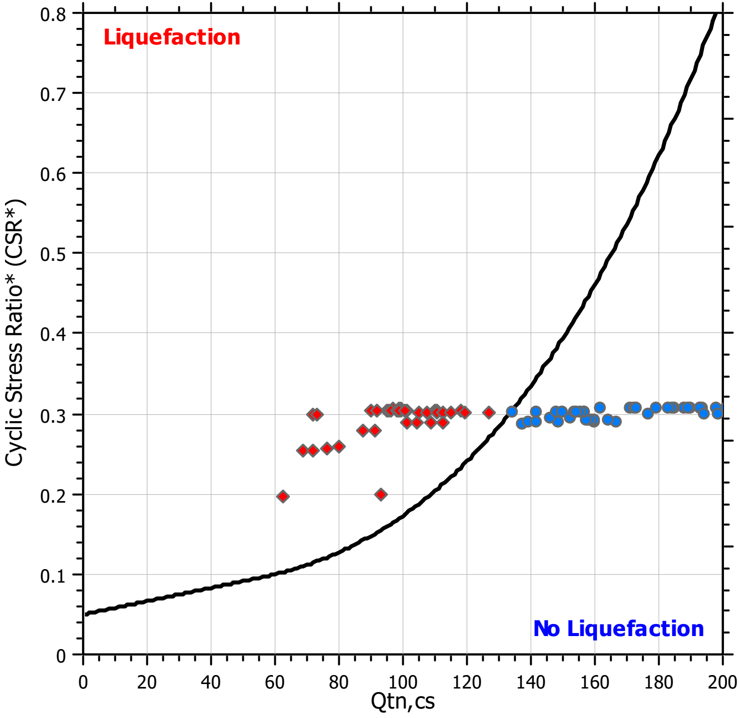
CRR plot



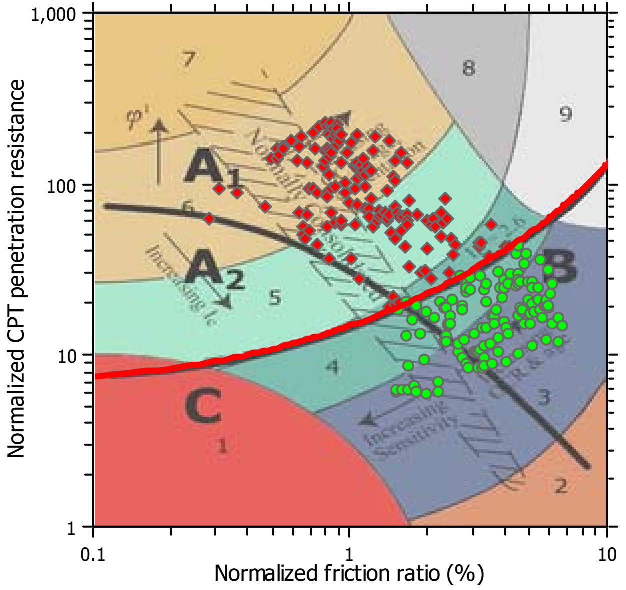
FS Plot



$M_w=7^{1/2}$, sigma'=1 atm base curve

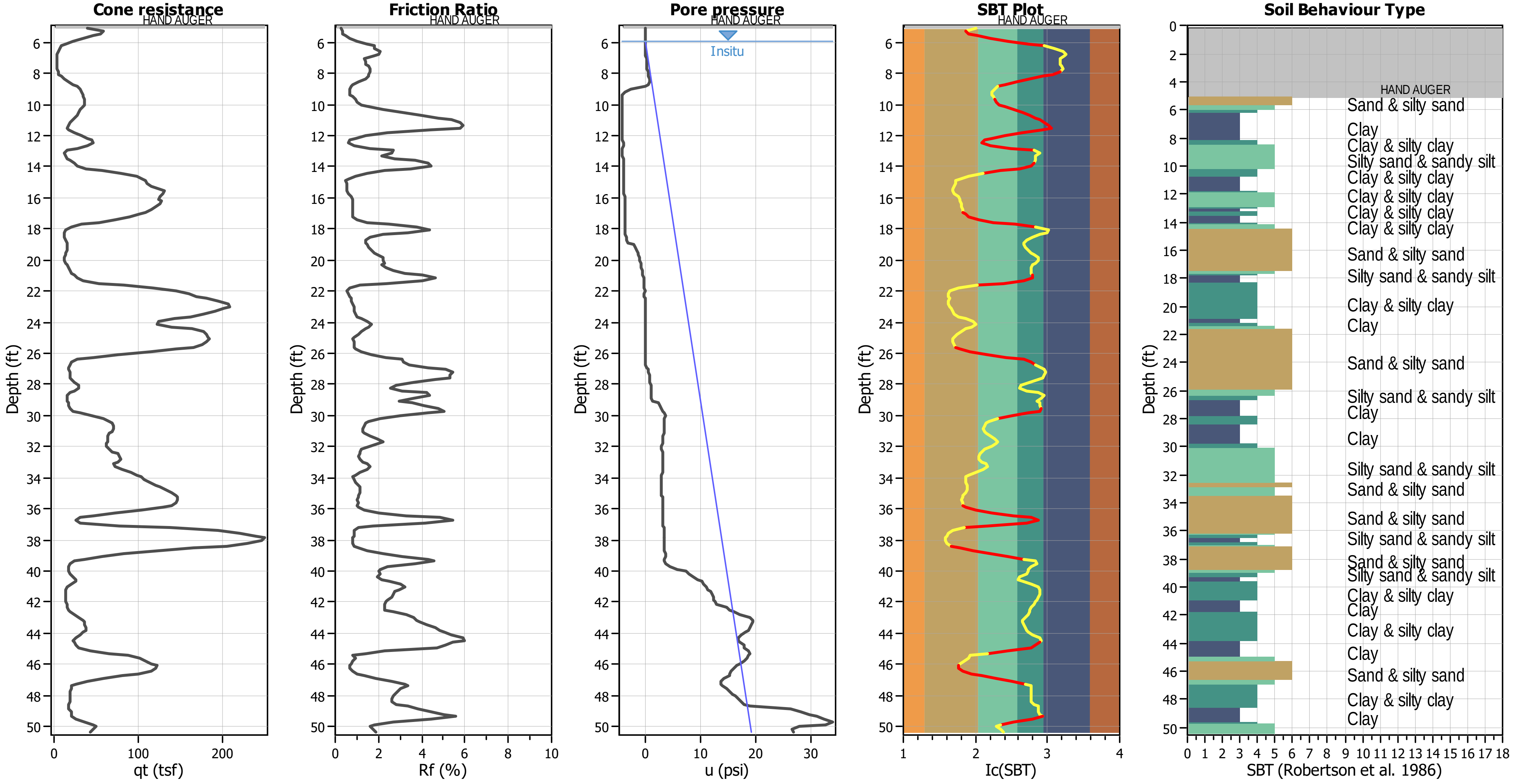


Summary of liquefaction potential



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

CPT basic interpretation plots



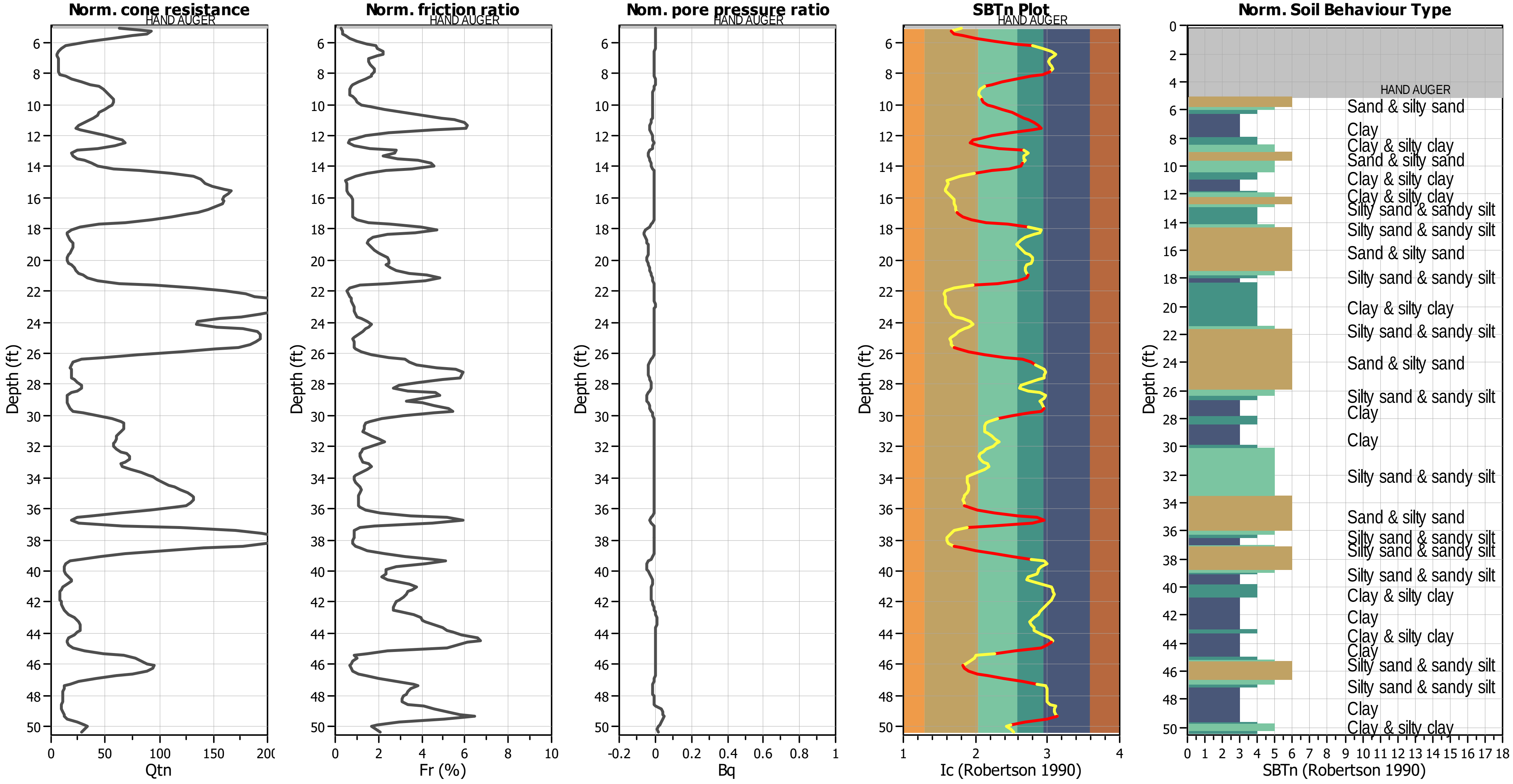
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



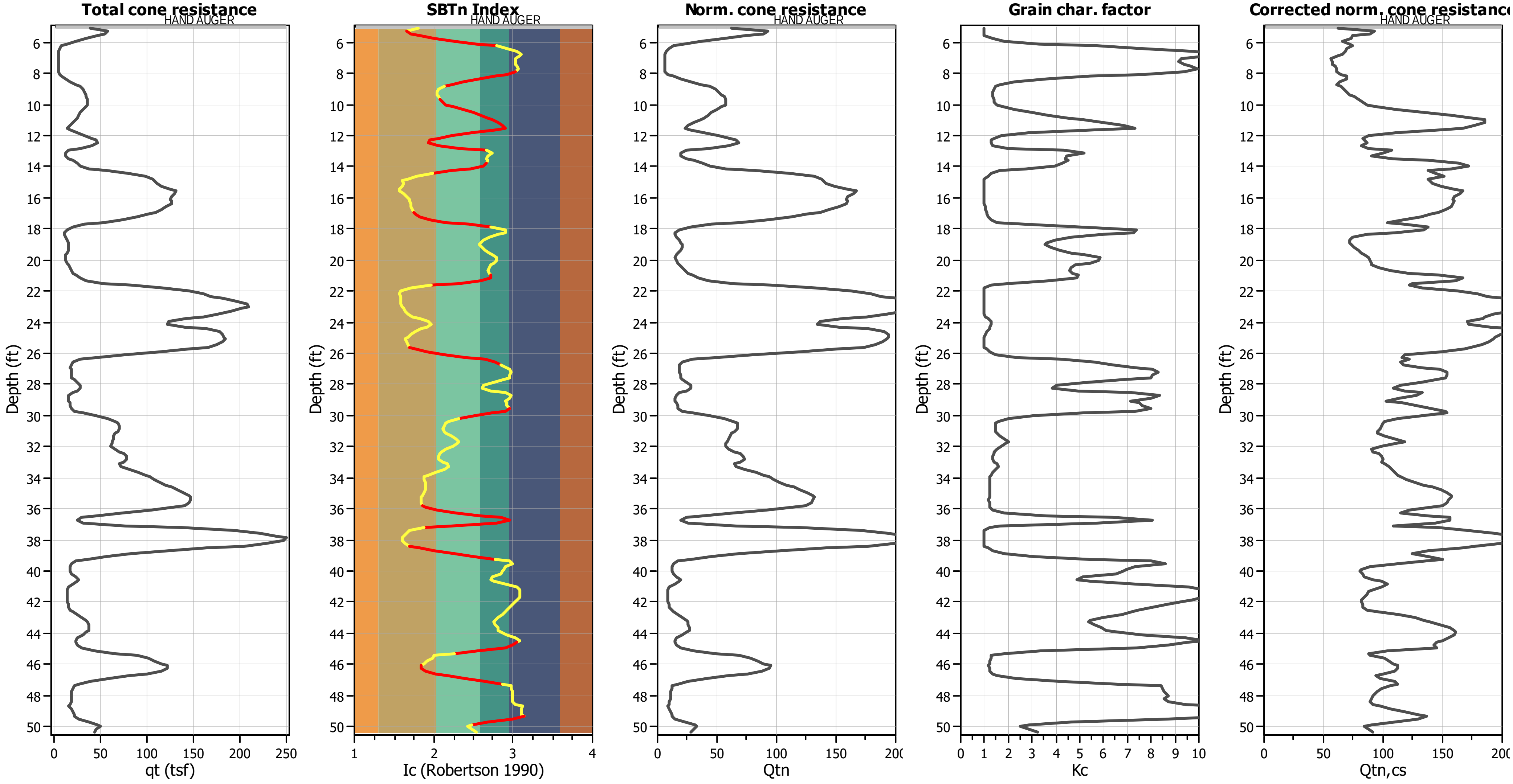
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

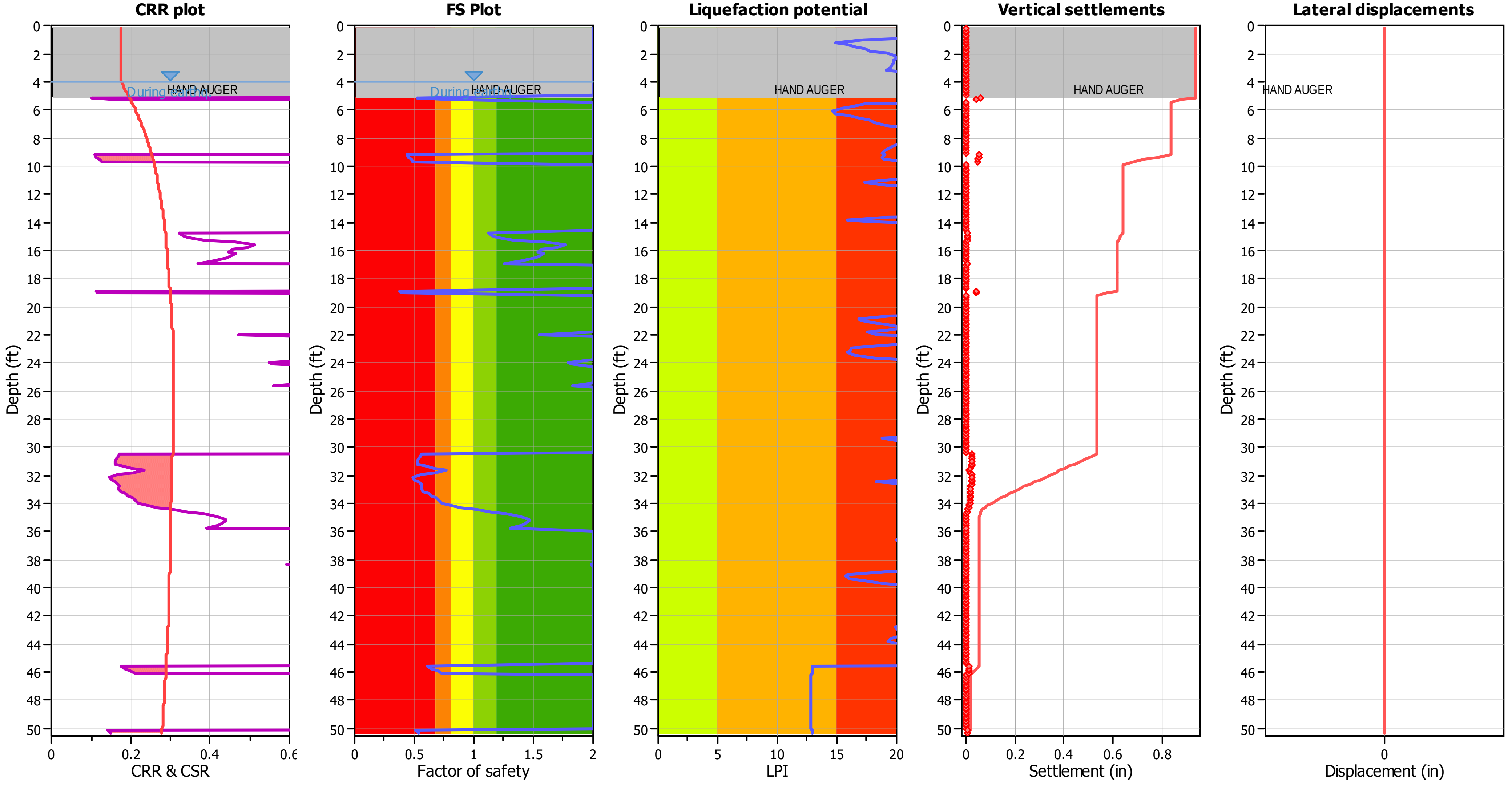
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

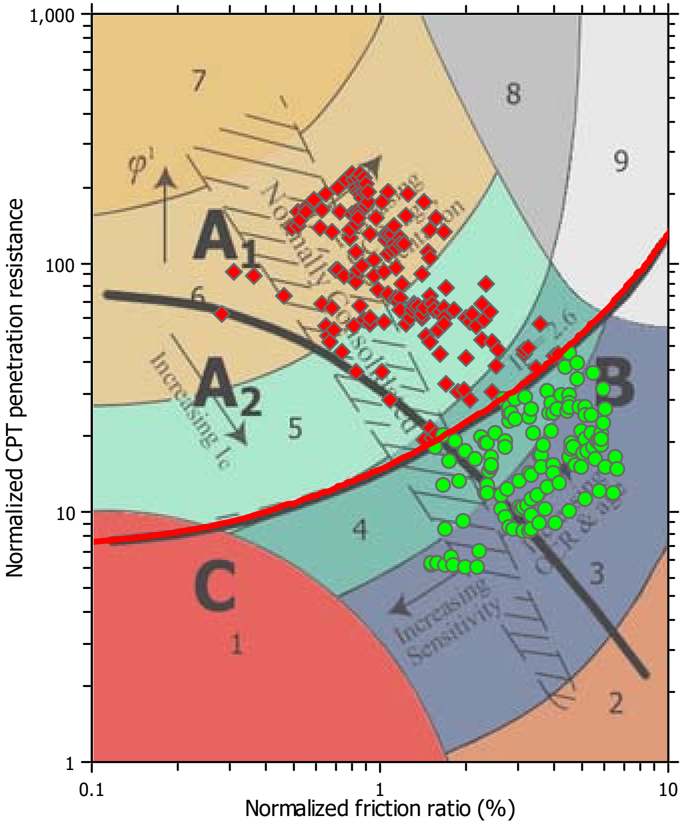
F.S. color scheme

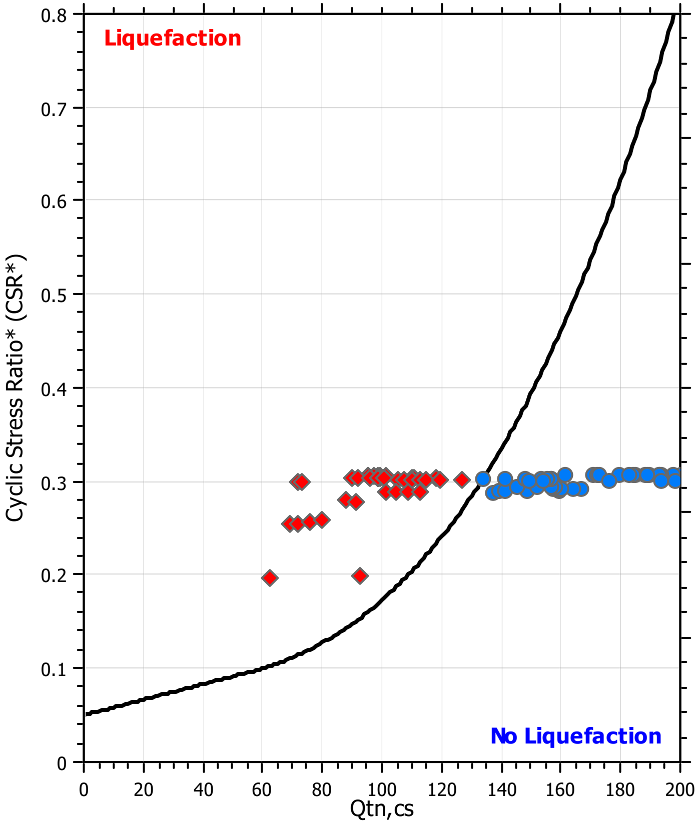
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

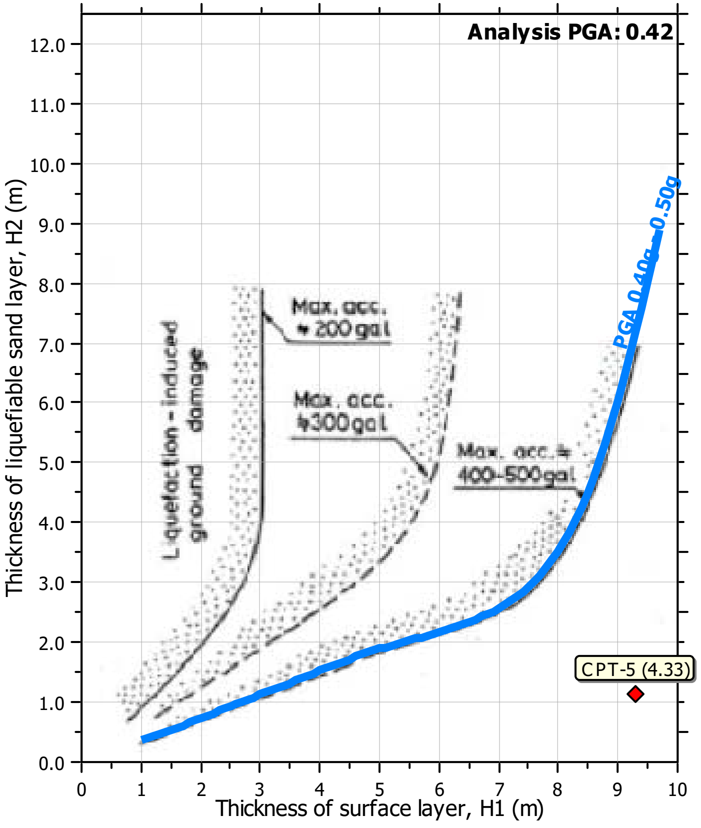
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



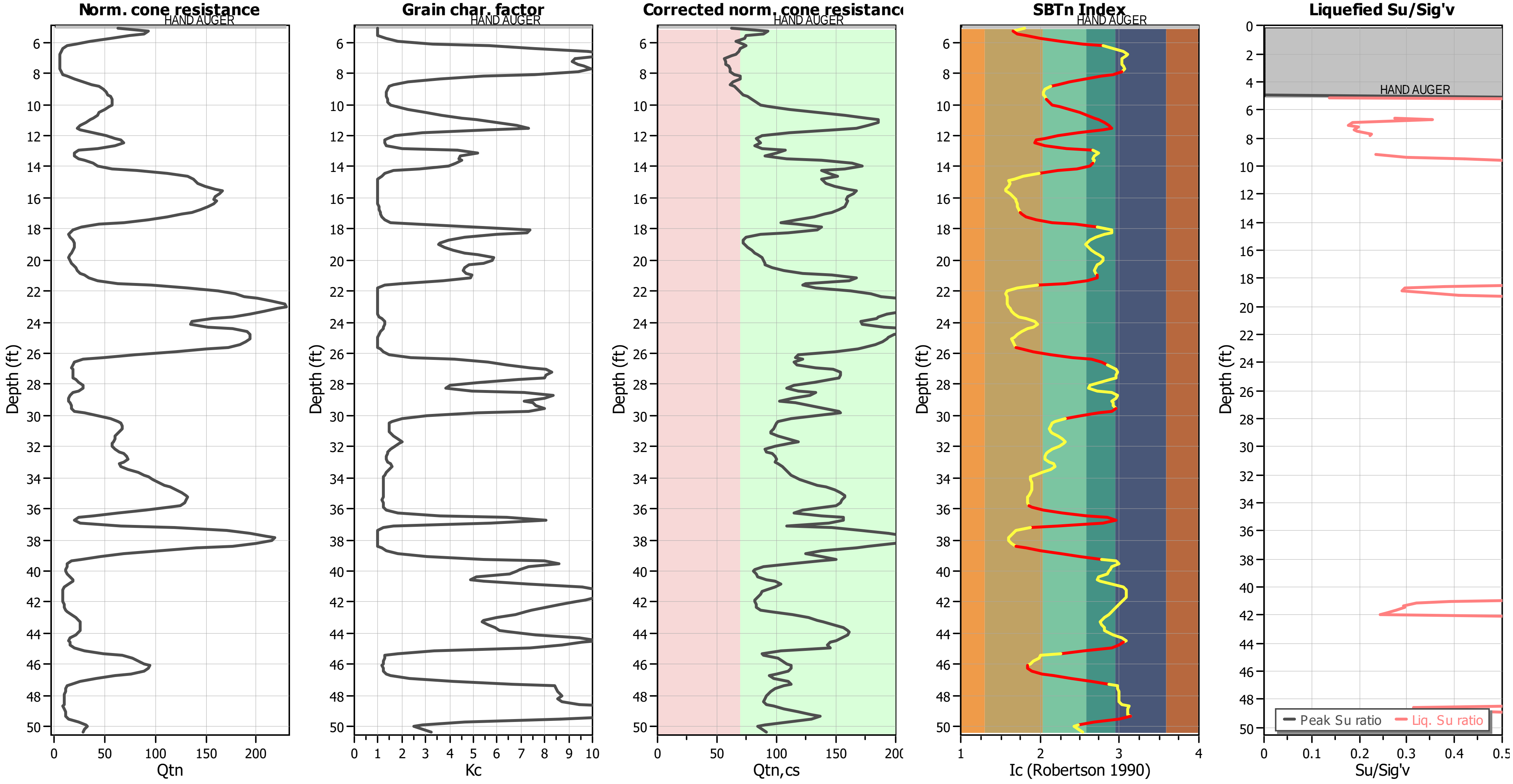




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

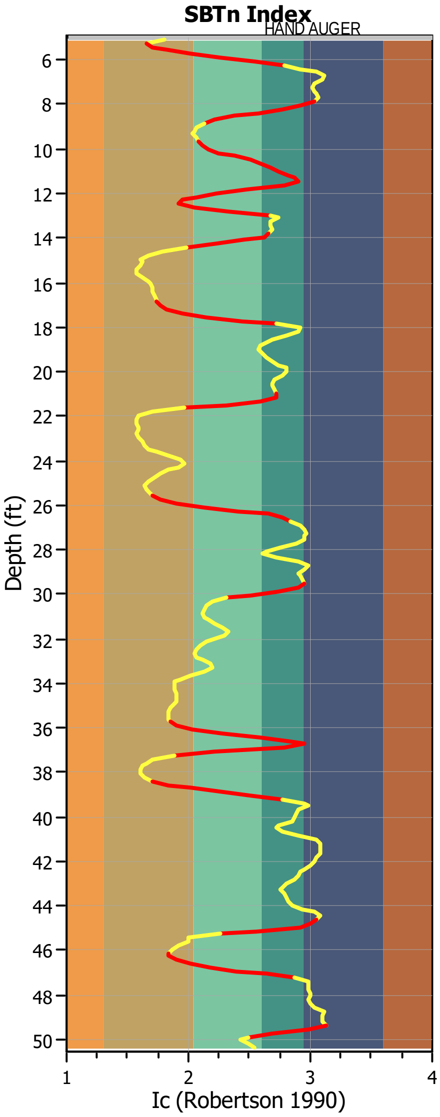
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.42	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

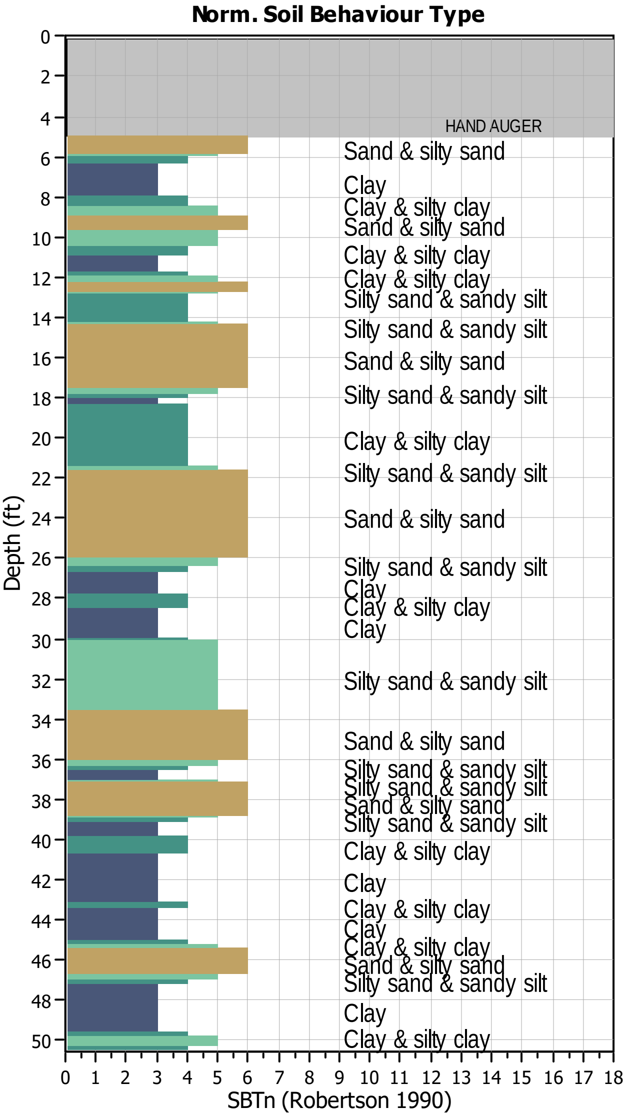
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

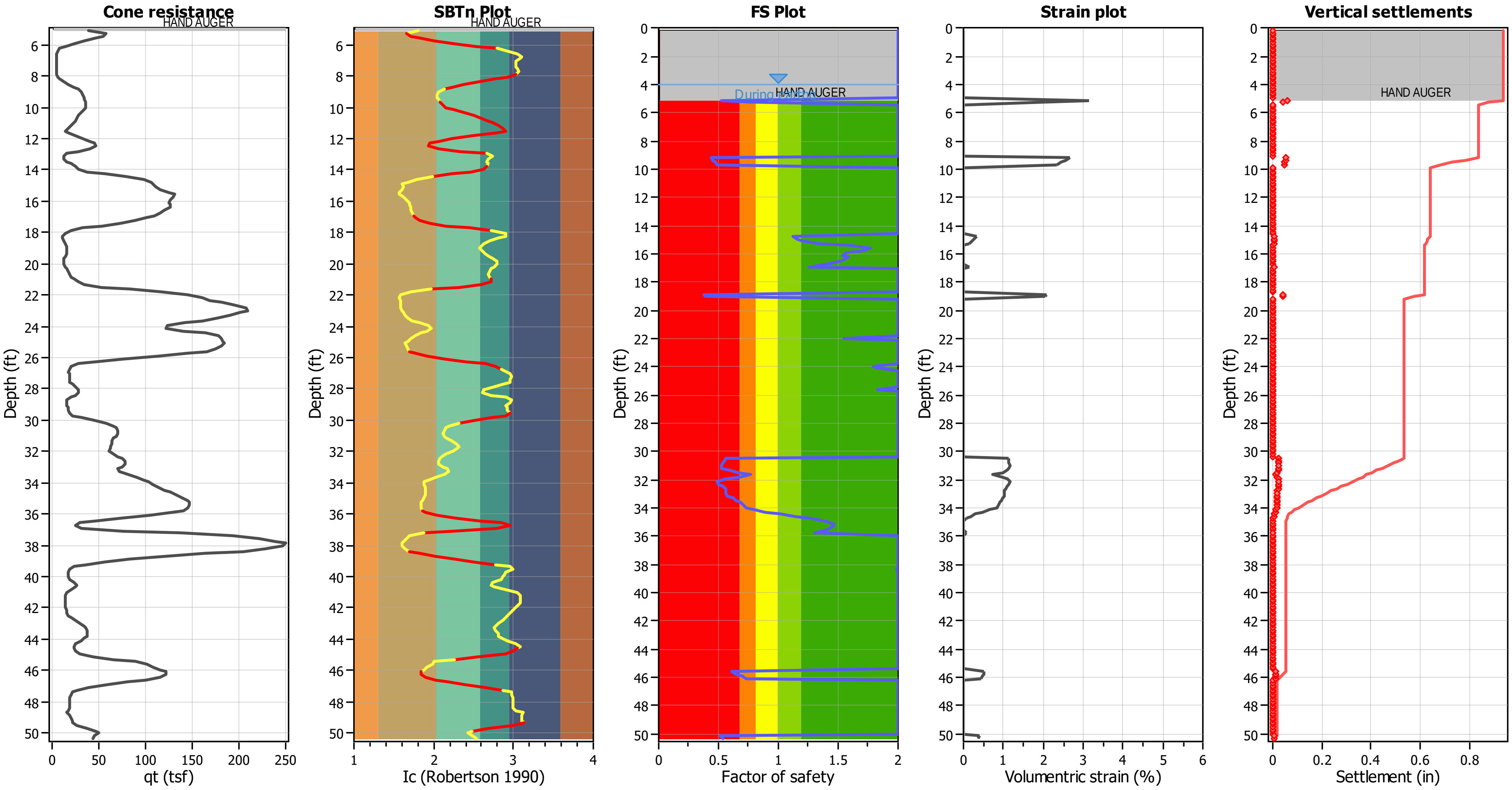
The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





Transition layer algorithm properties		**General statistics**	
I_c minimum check value:	1.70	Total points in CPT file:	307
I_c maximum check value:	3.00	Total points excluded:	99
I_c change ratio value:	0.0250	Exclusion percentage:	32.25%
Minimum number of points in layer:	4	Number of layers detected:	16

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

TABLE OF CONTENTS

Geocon West, Inc.
2807 McGaw Avenue
Irvine, CA 92614
www.geoconinc.com

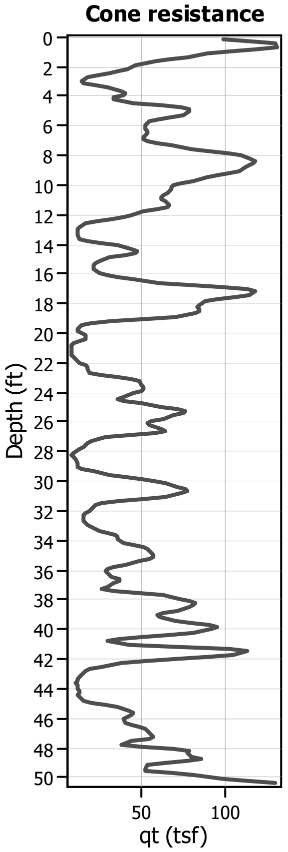
LIQUEFACTION ANALYSIS REPORT

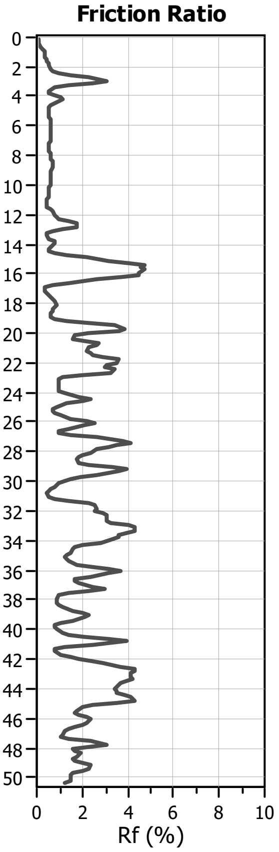
Project title : W2045-88-01 **Location : Euclid and Heil**

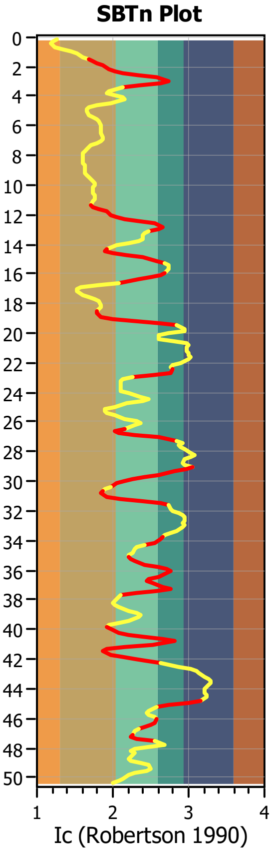
CPT file : CPT-1

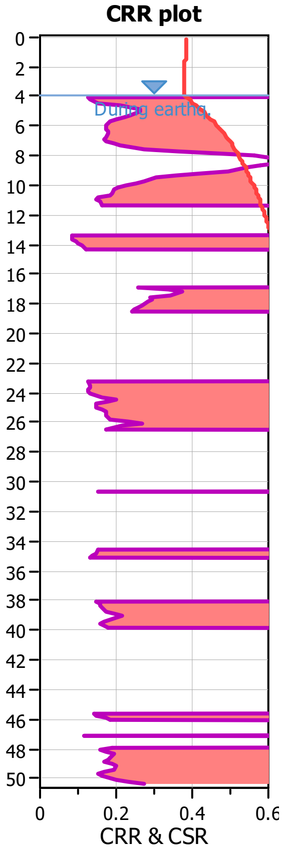
Input parameters and analysis data

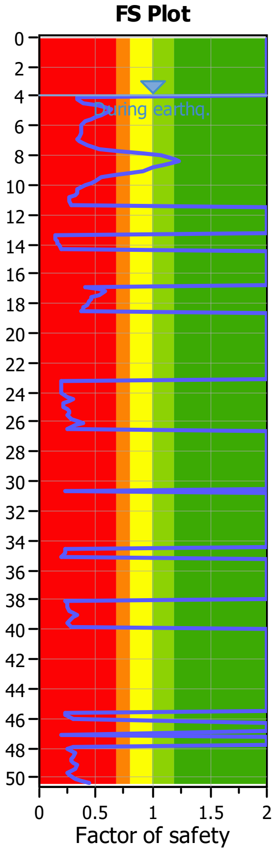
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.63	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

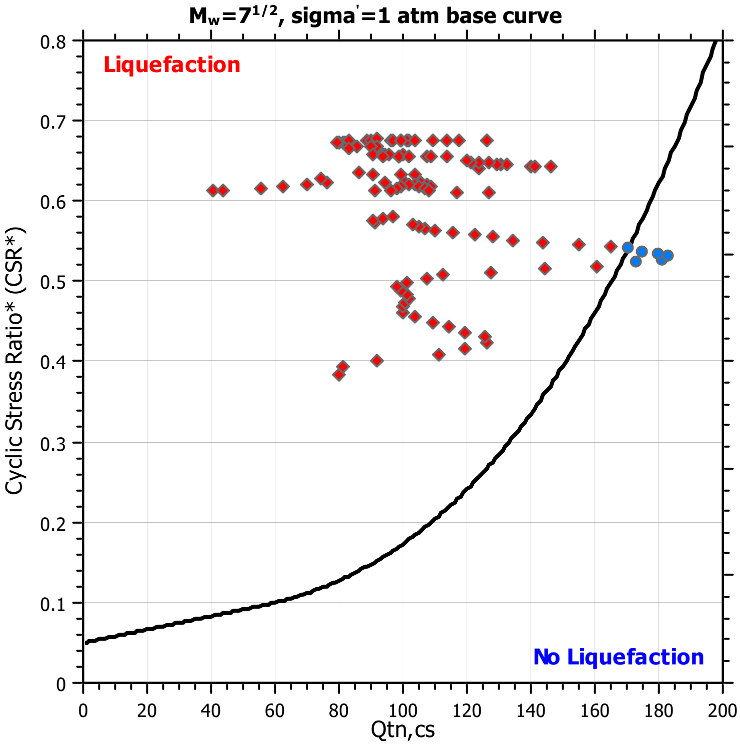

Cone resistance

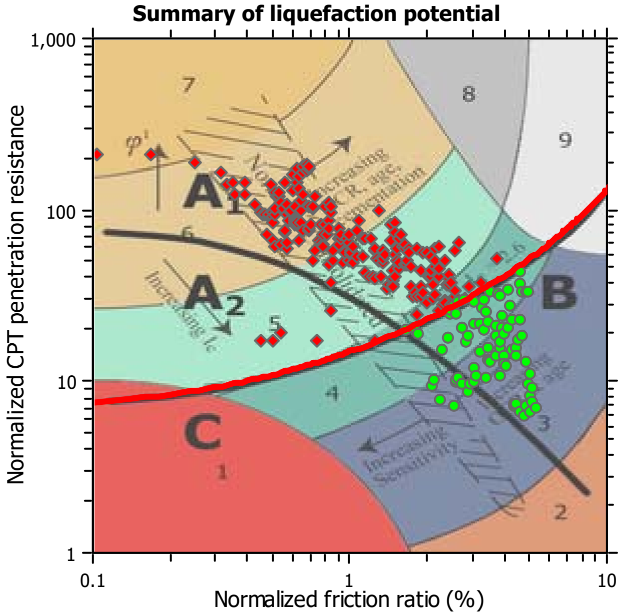

Friction Ratio


SBTn Plot


CRR plot


FS Plot


M_w=$7^{1/2}$, sigma'=1 atm base curve


Summary of liquefaction potential

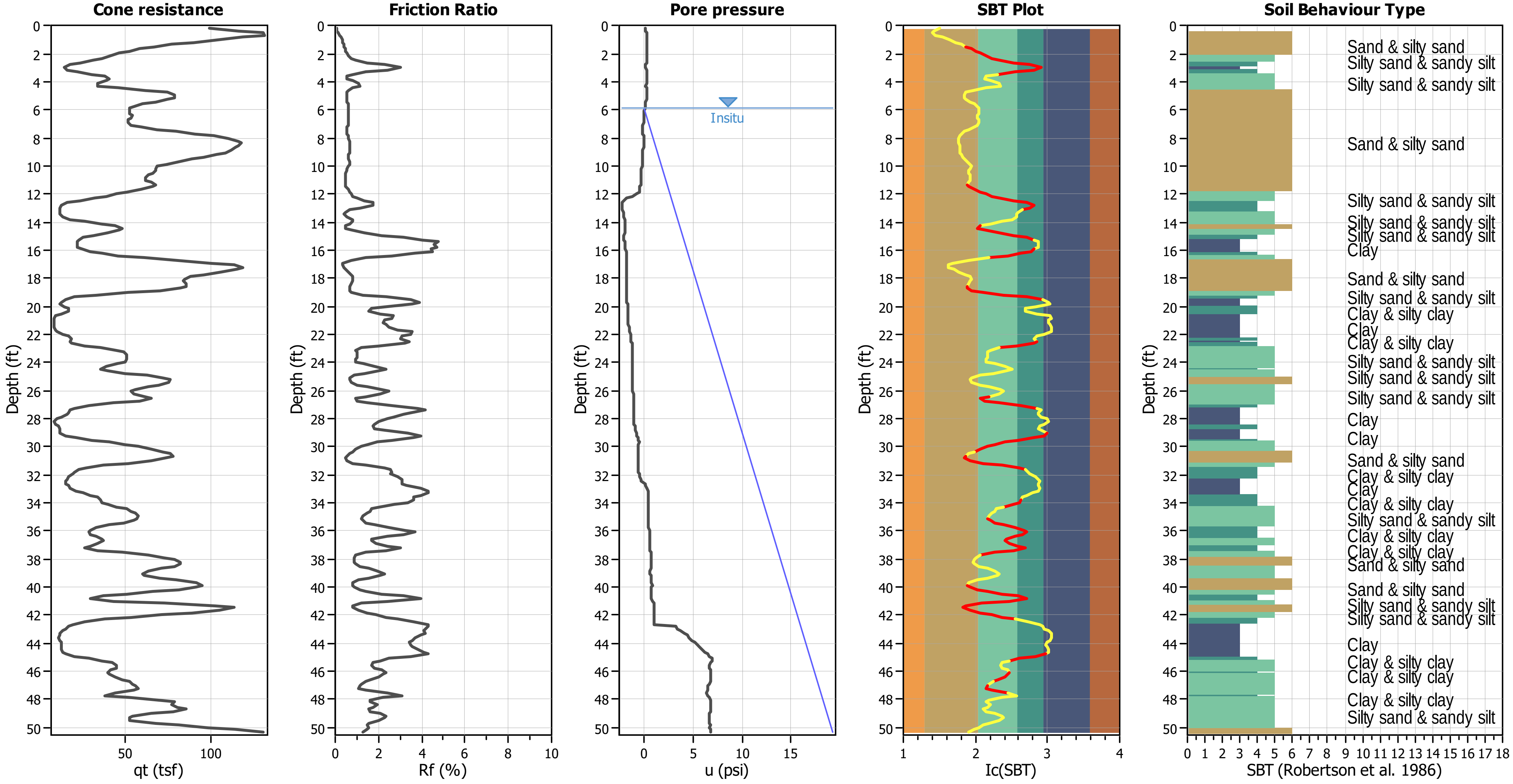
Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading

Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry

Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening

Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



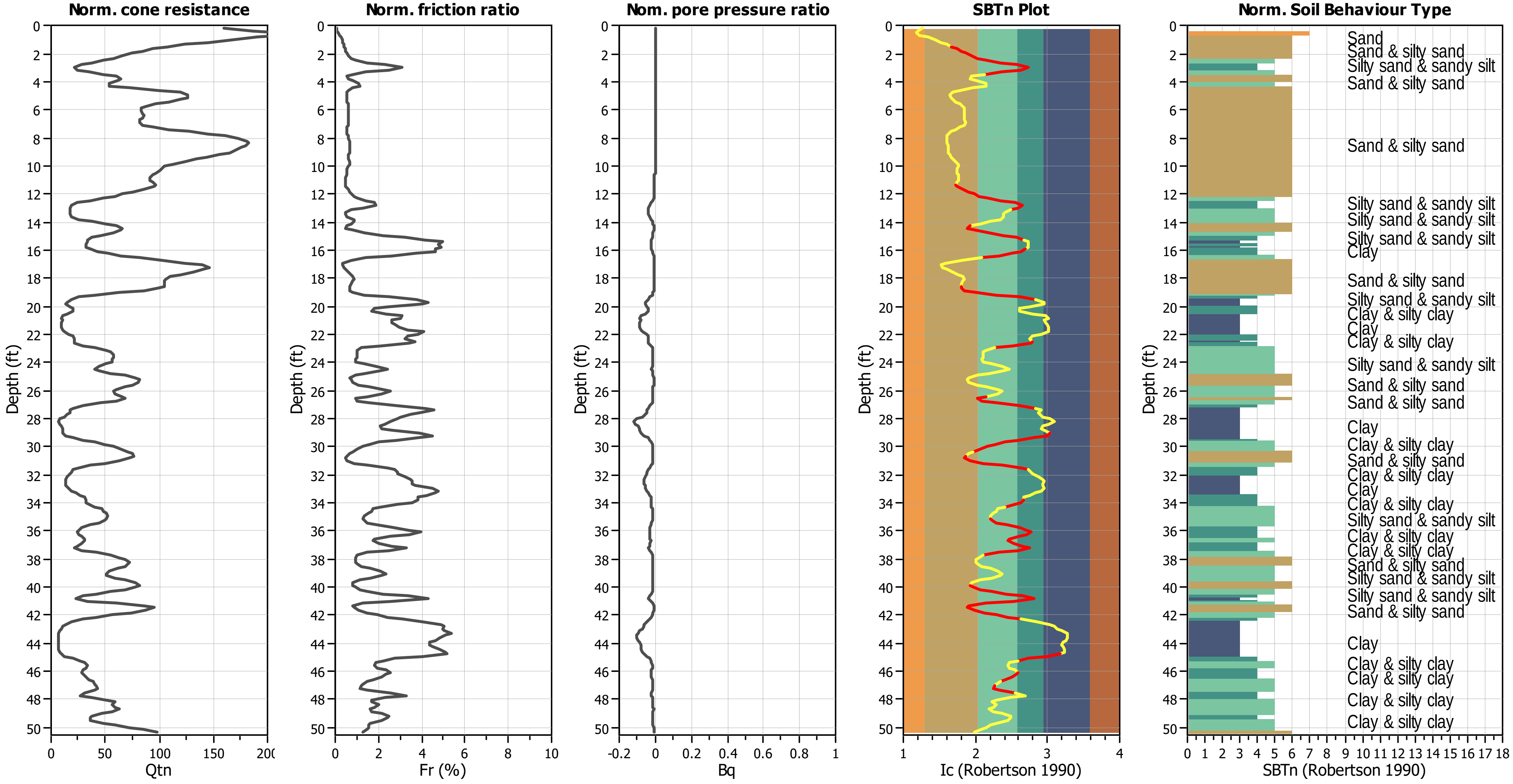
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



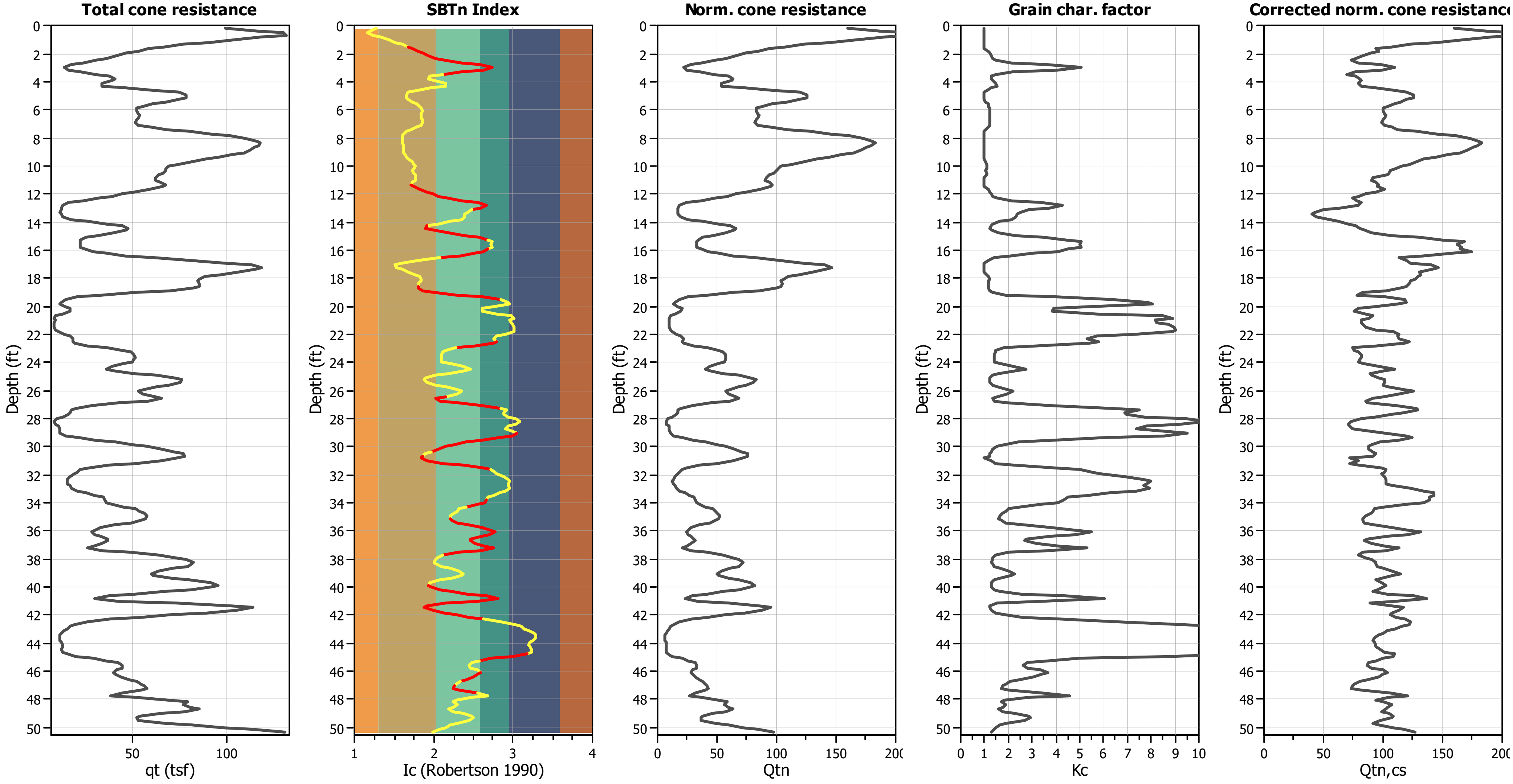
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

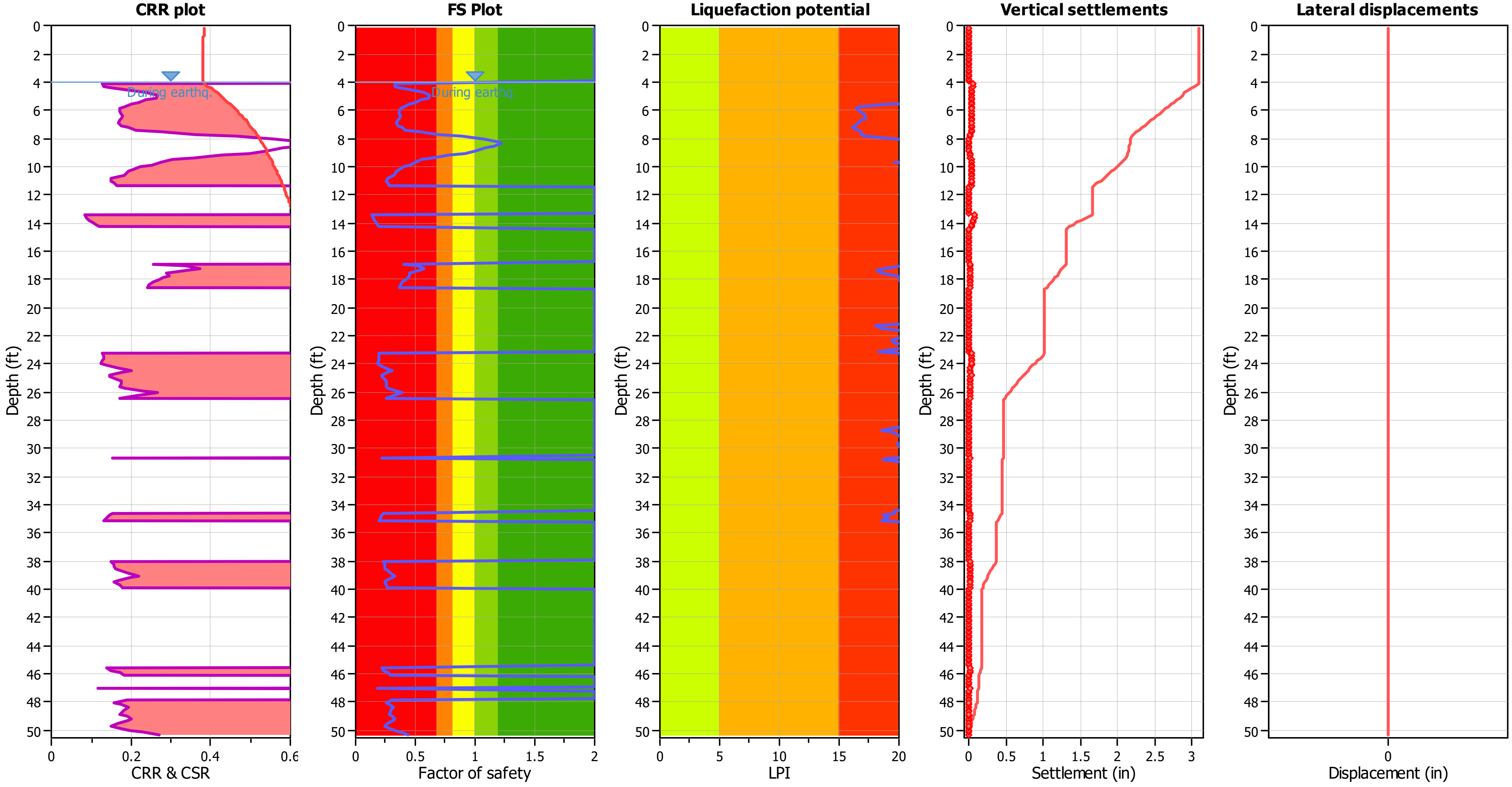
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: NCEER (1998)
Fines correction method: NCEER (1998)
Points to test: Based on Ic value
Earthquake magnitude M_w: 7.30
Peak ground acceleration: 0.63
Depth to water table (insitu): 5.90 ft

Depth to water table (erthq.): 4.00 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: No
Limit depth: N/A

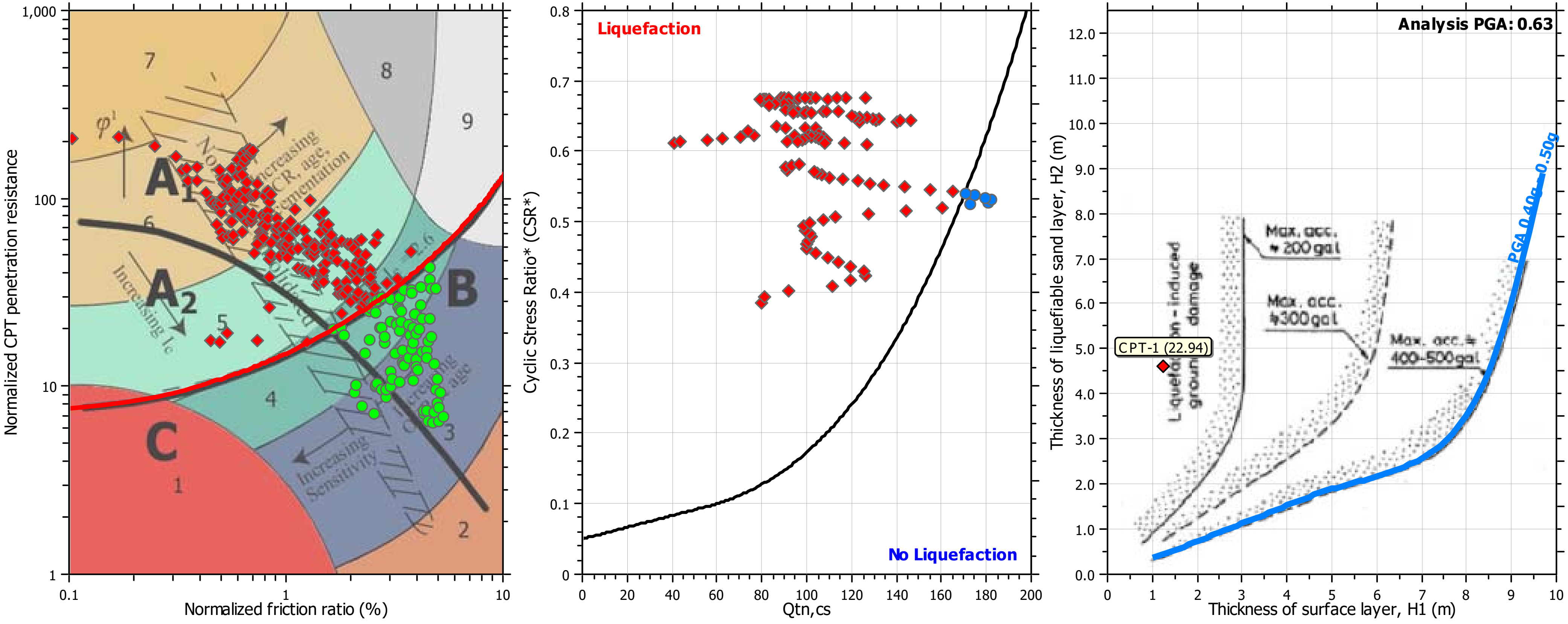
F.S. color scheme

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

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

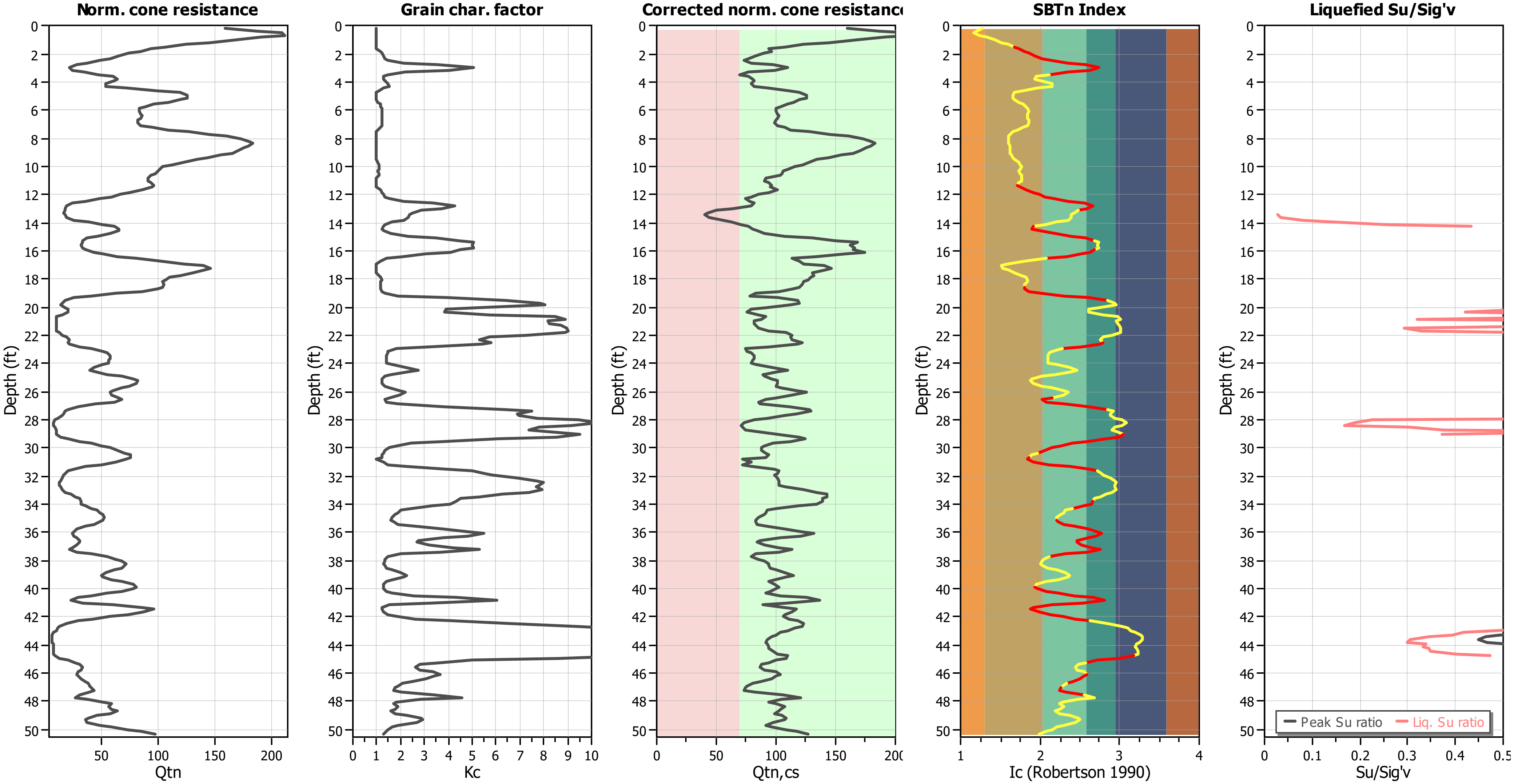
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_{σ} applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

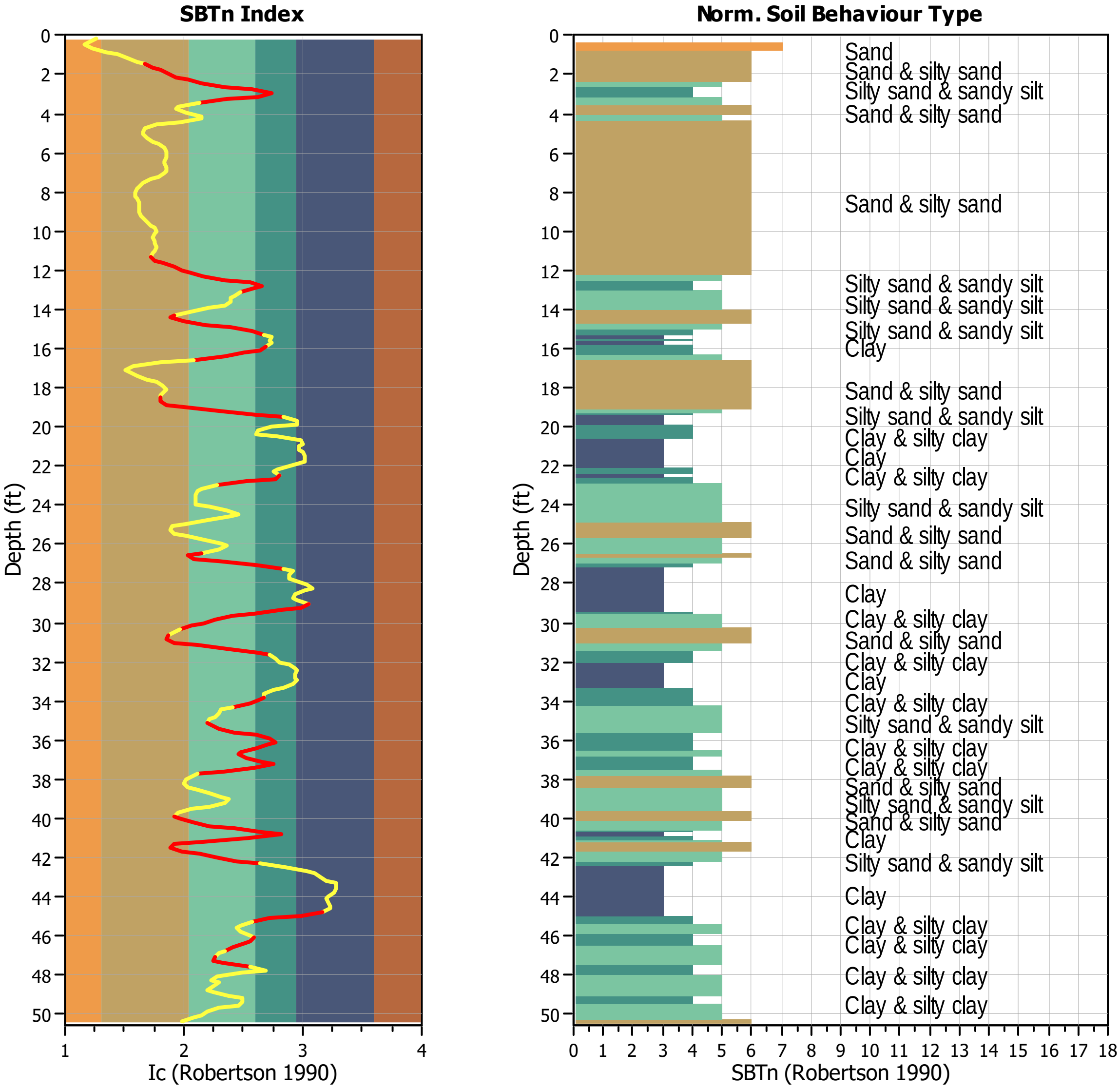
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.



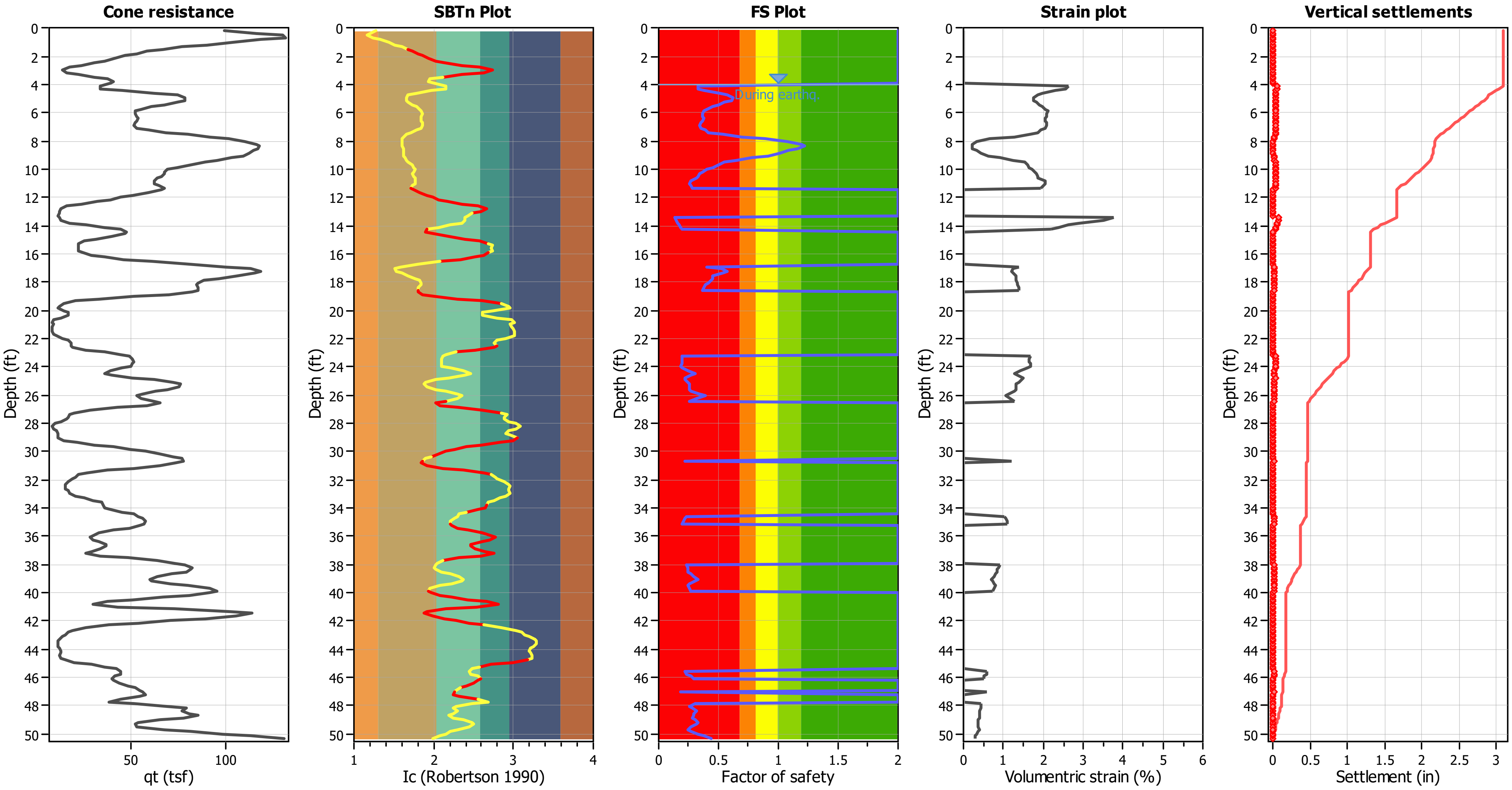
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	307
Total points excluded:	125
Exclusion percentage:	40.72%
Number of layers detected:	22

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

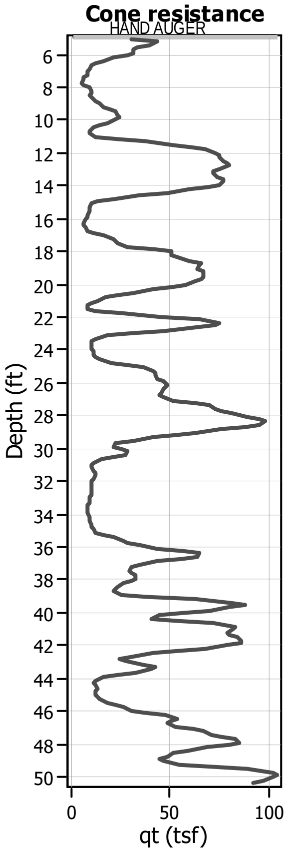
LIQUEFACTION ANALYSIS REPORT

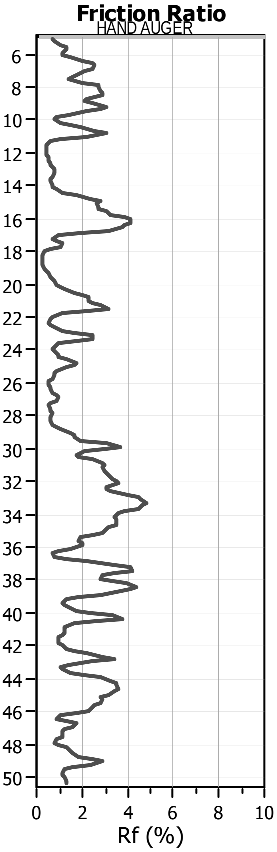
Project title : W2045-88-01 **Location : Euclid and Heil**

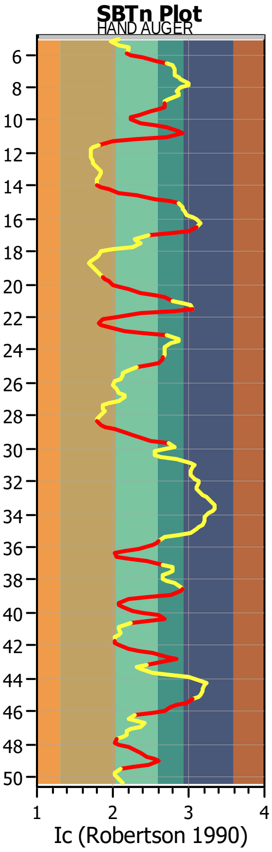
CPT file : CPT-2

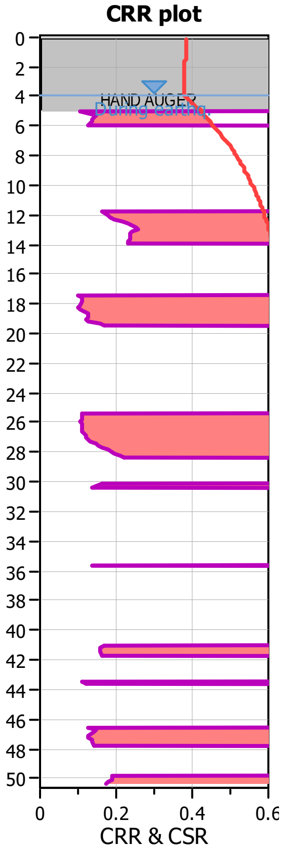
Input parameters and analysis data

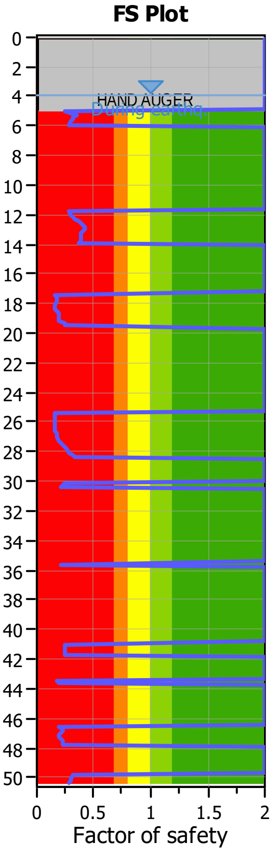
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.63	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

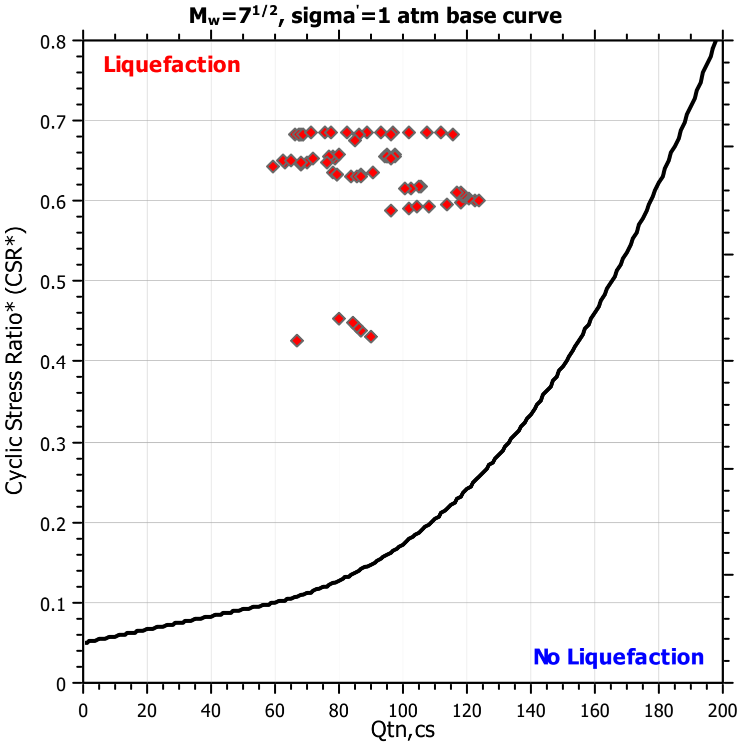


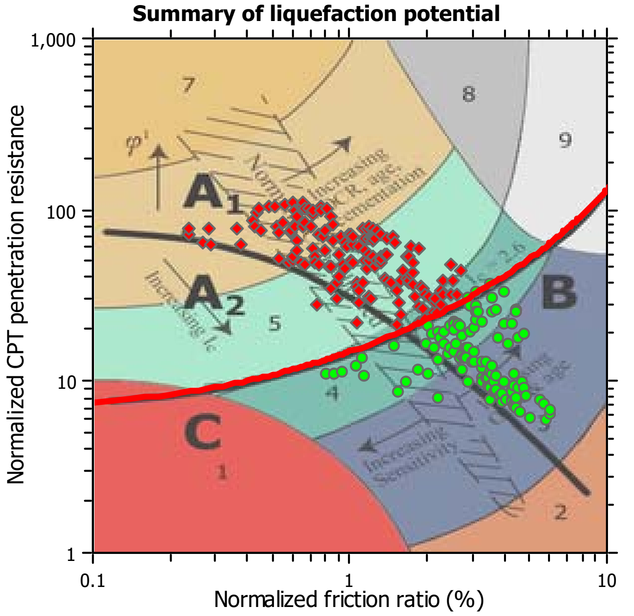






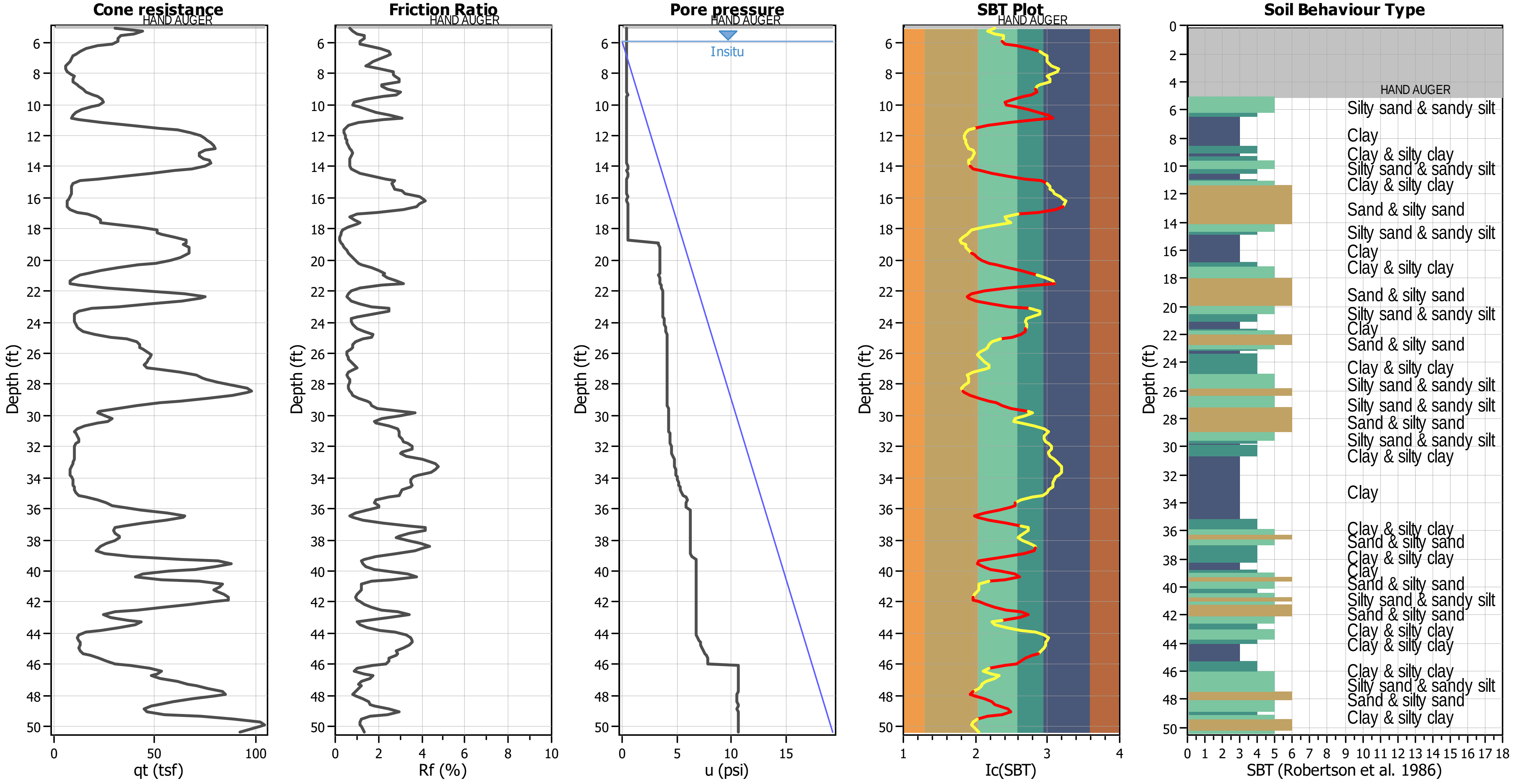






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



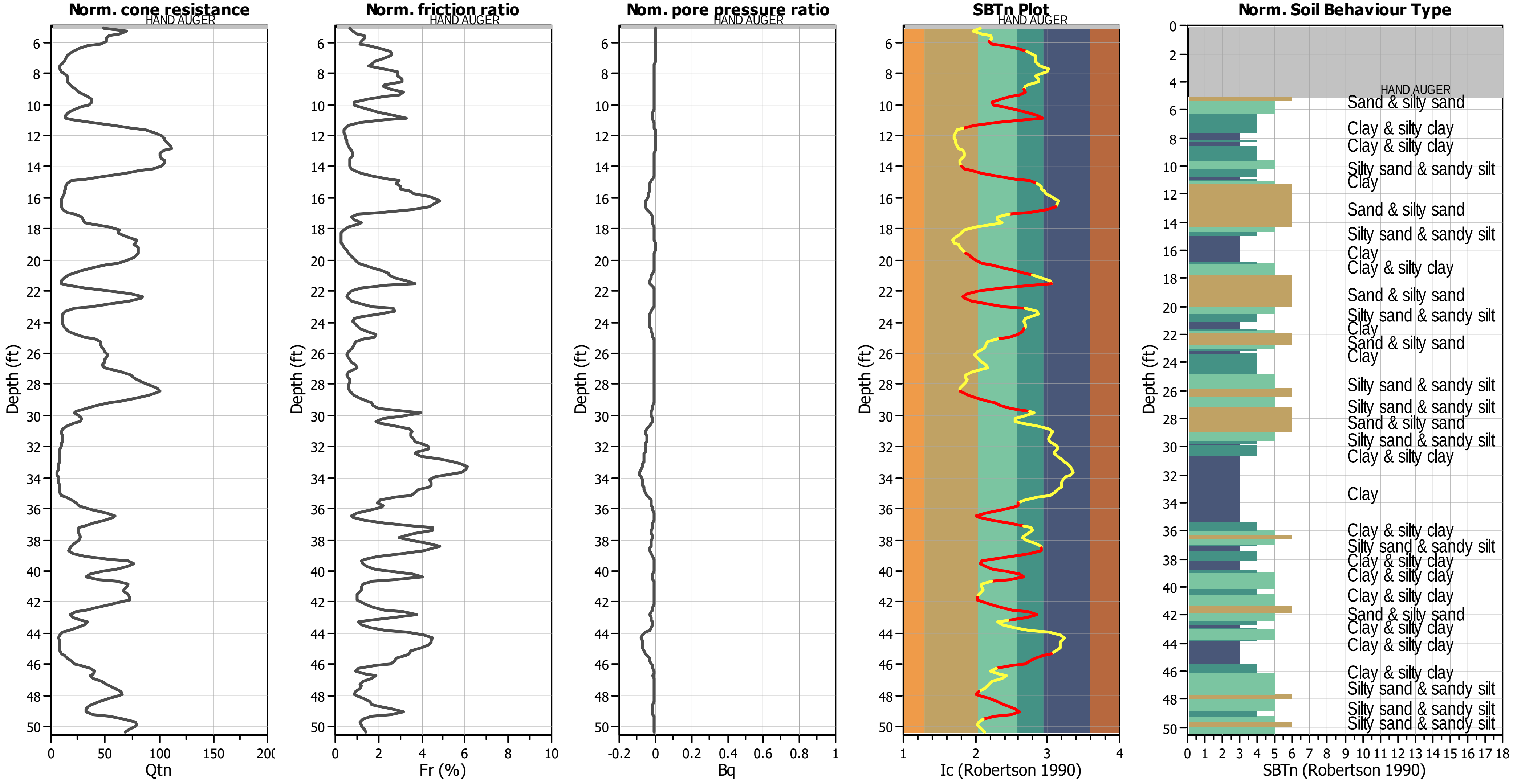
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to
- ⬜ 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



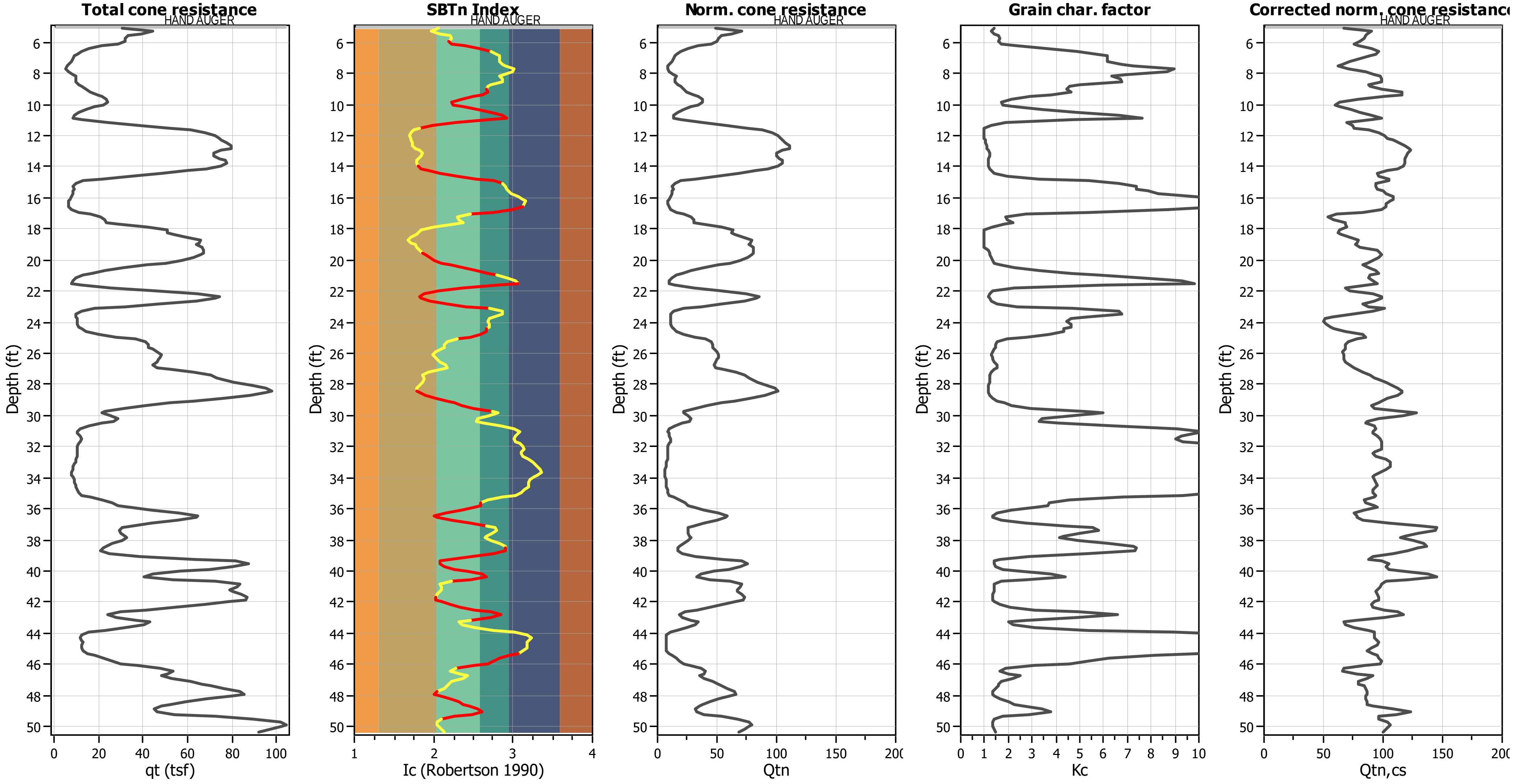
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

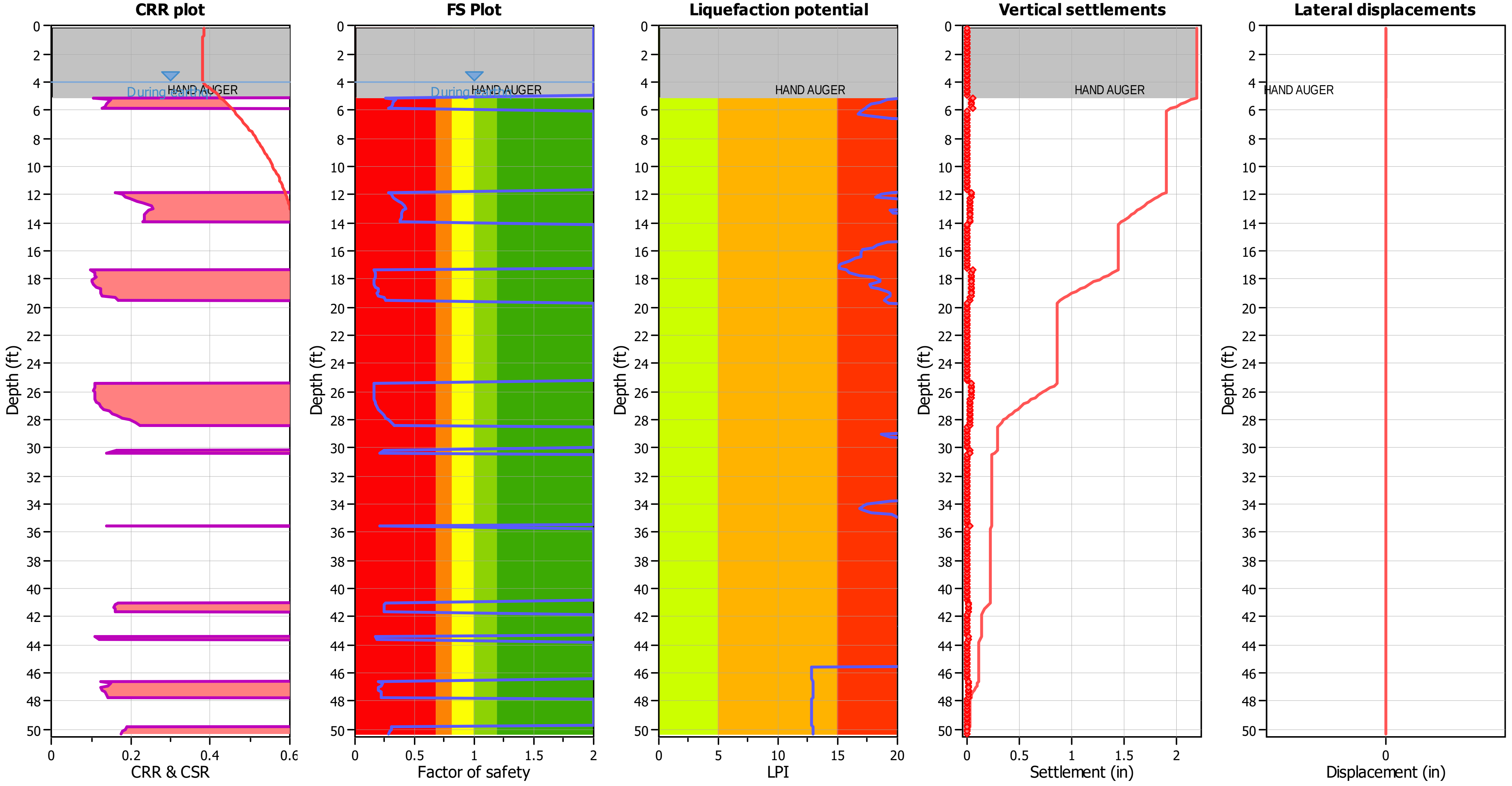
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

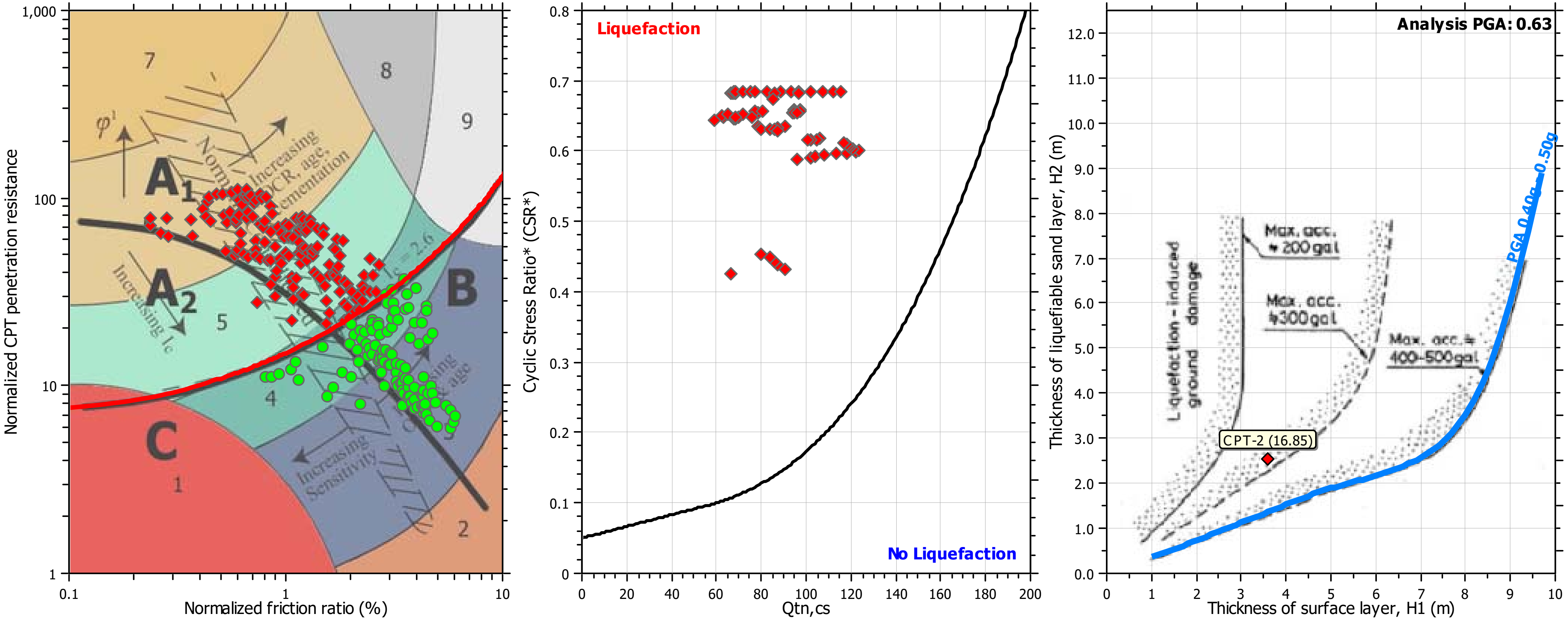
F.S. color scheme

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

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

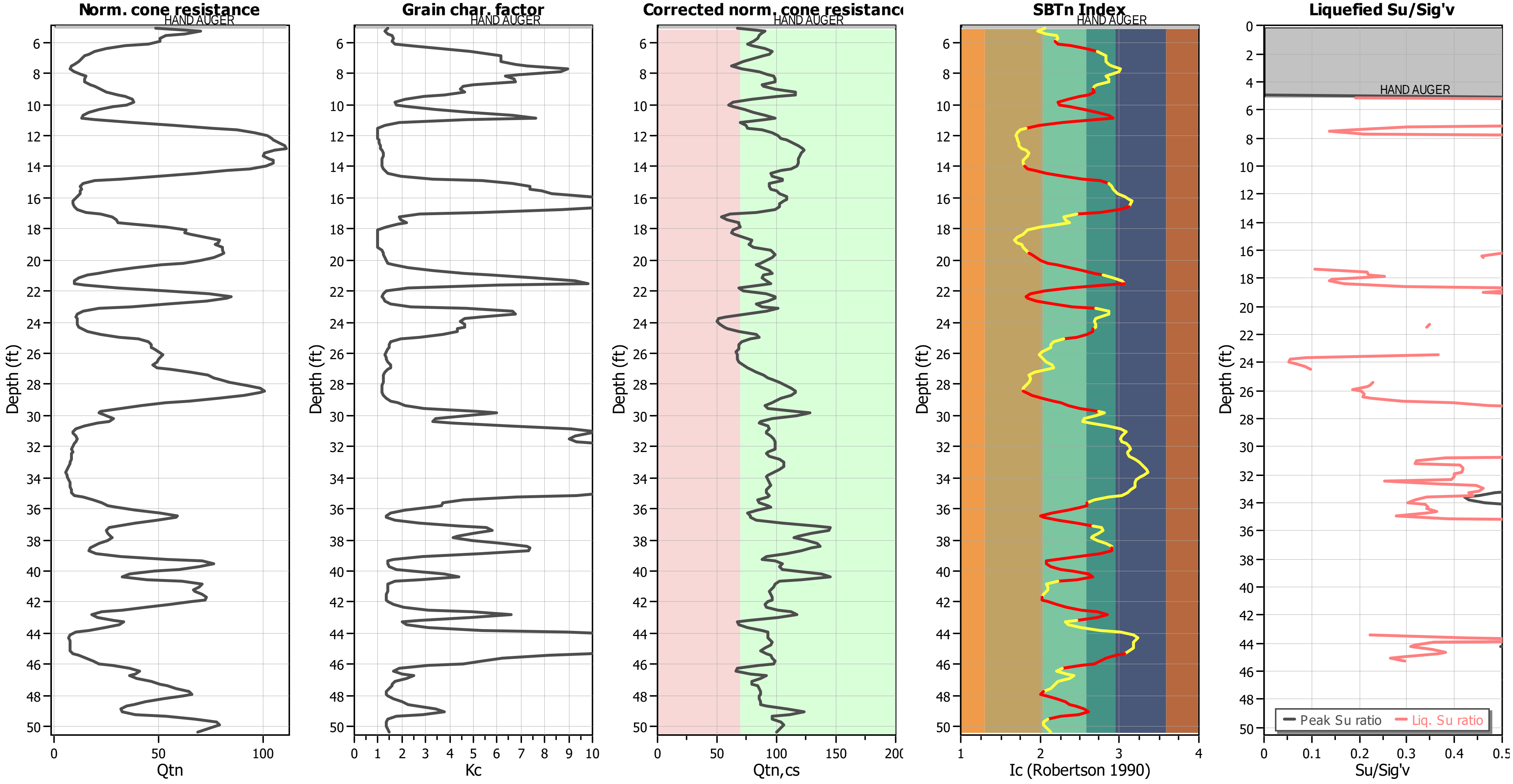
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

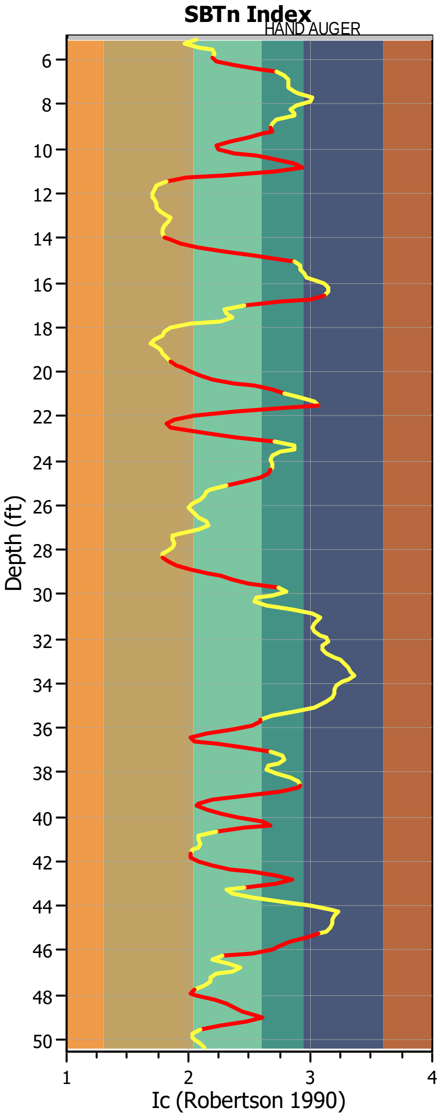
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

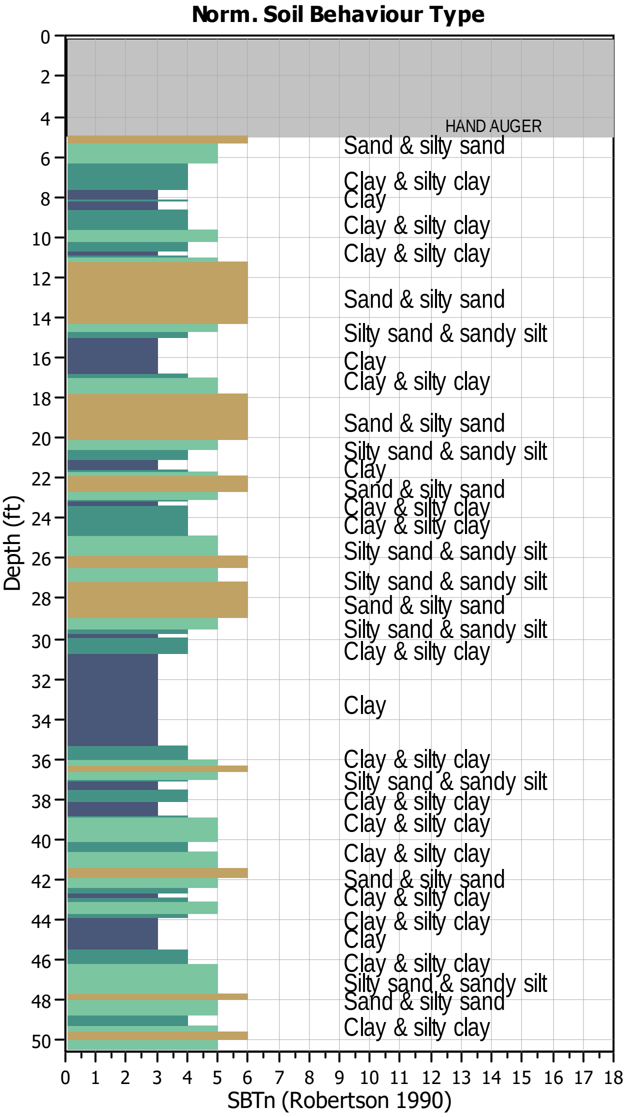
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between 1.80 < I_c < 3.0) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





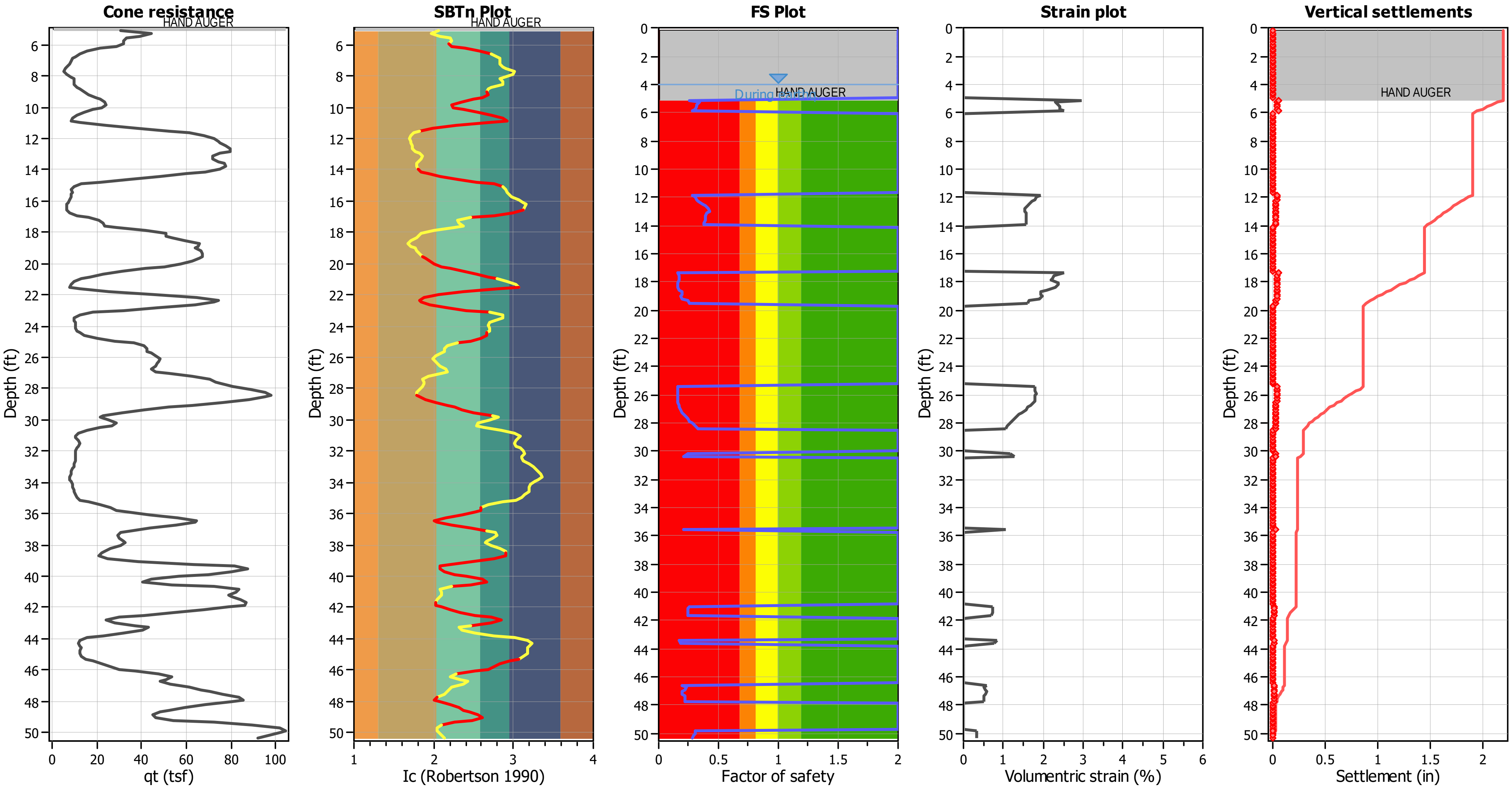
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	307
Total points excluded:	124
Exclusion percentage:	40.39%
Number of layers detected:	21

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

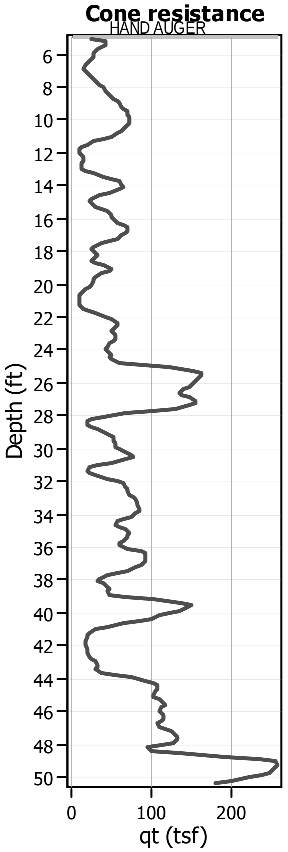
LIQUEFACTION ANALYSIS REPORT

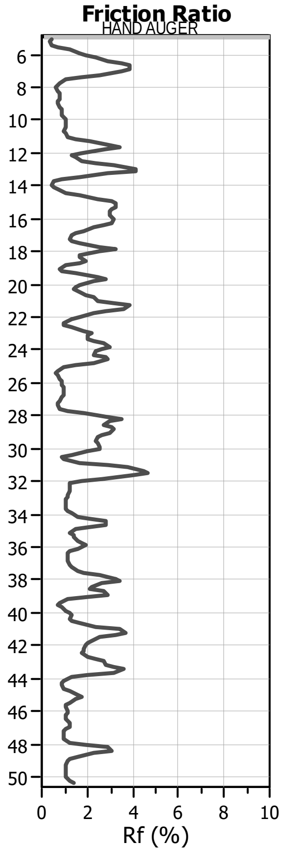
Project title : W2045-88-01 **Location : Euclid and Heil**

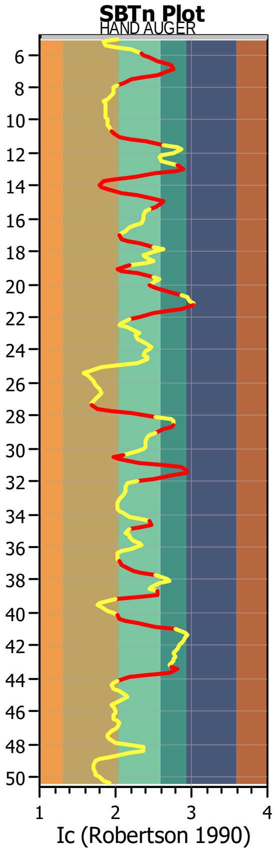
CPT file : CPT-3

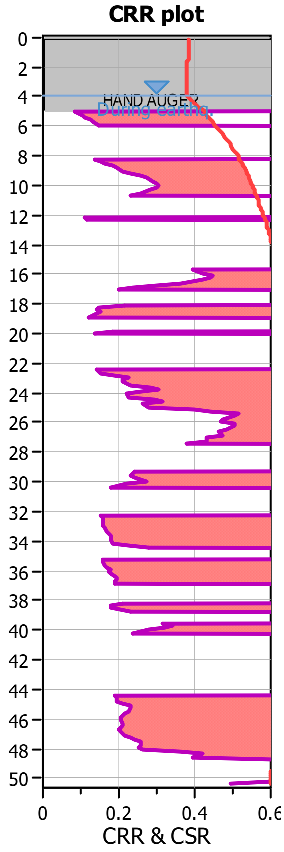
Input parameters and analysis data

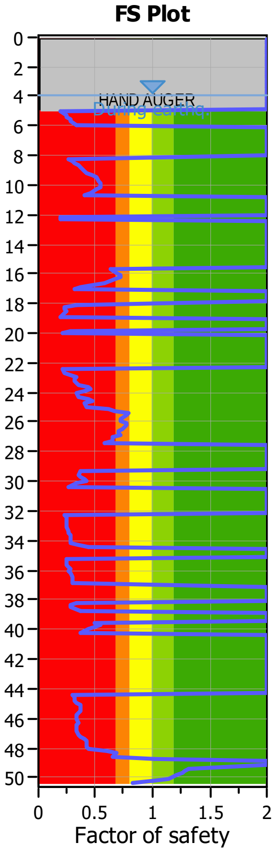
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.63	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

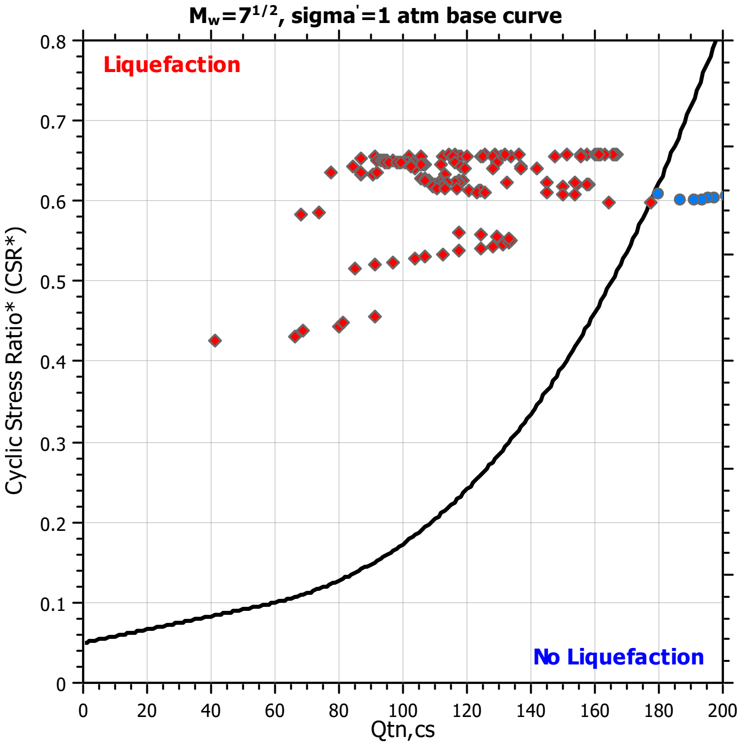


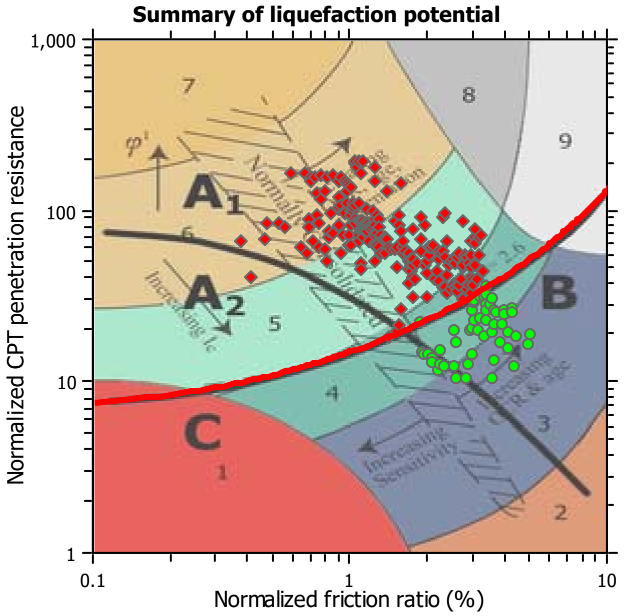






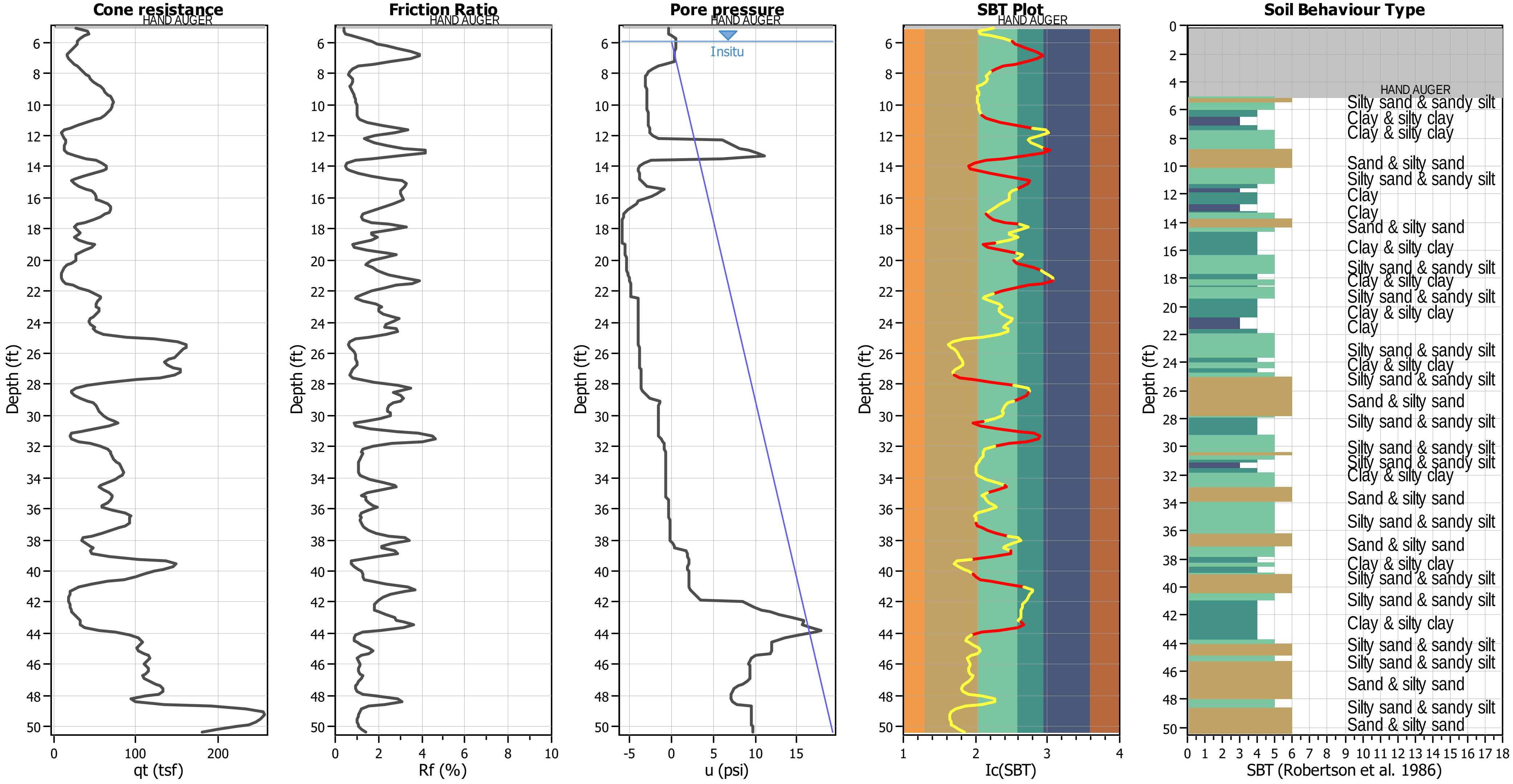






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



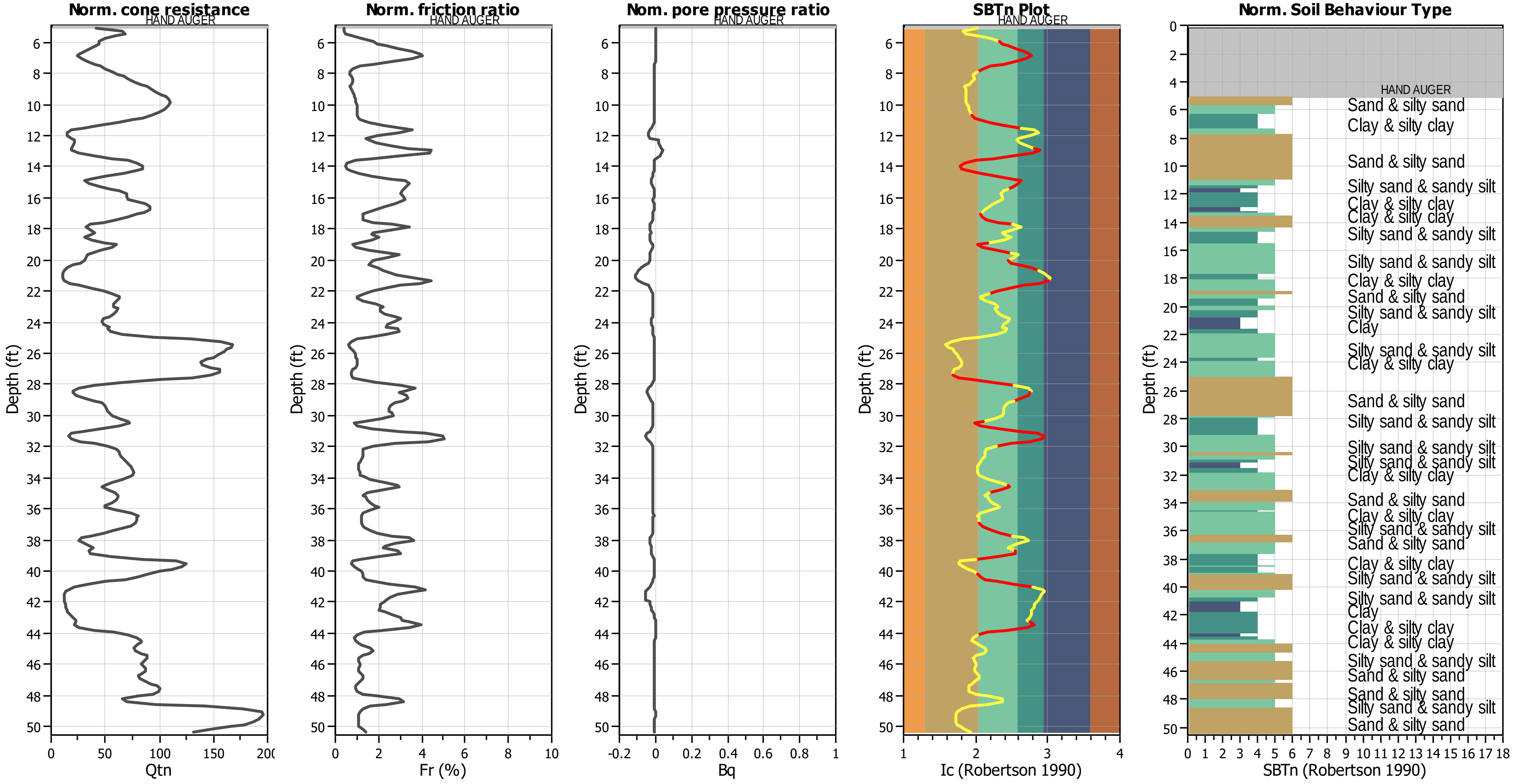
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



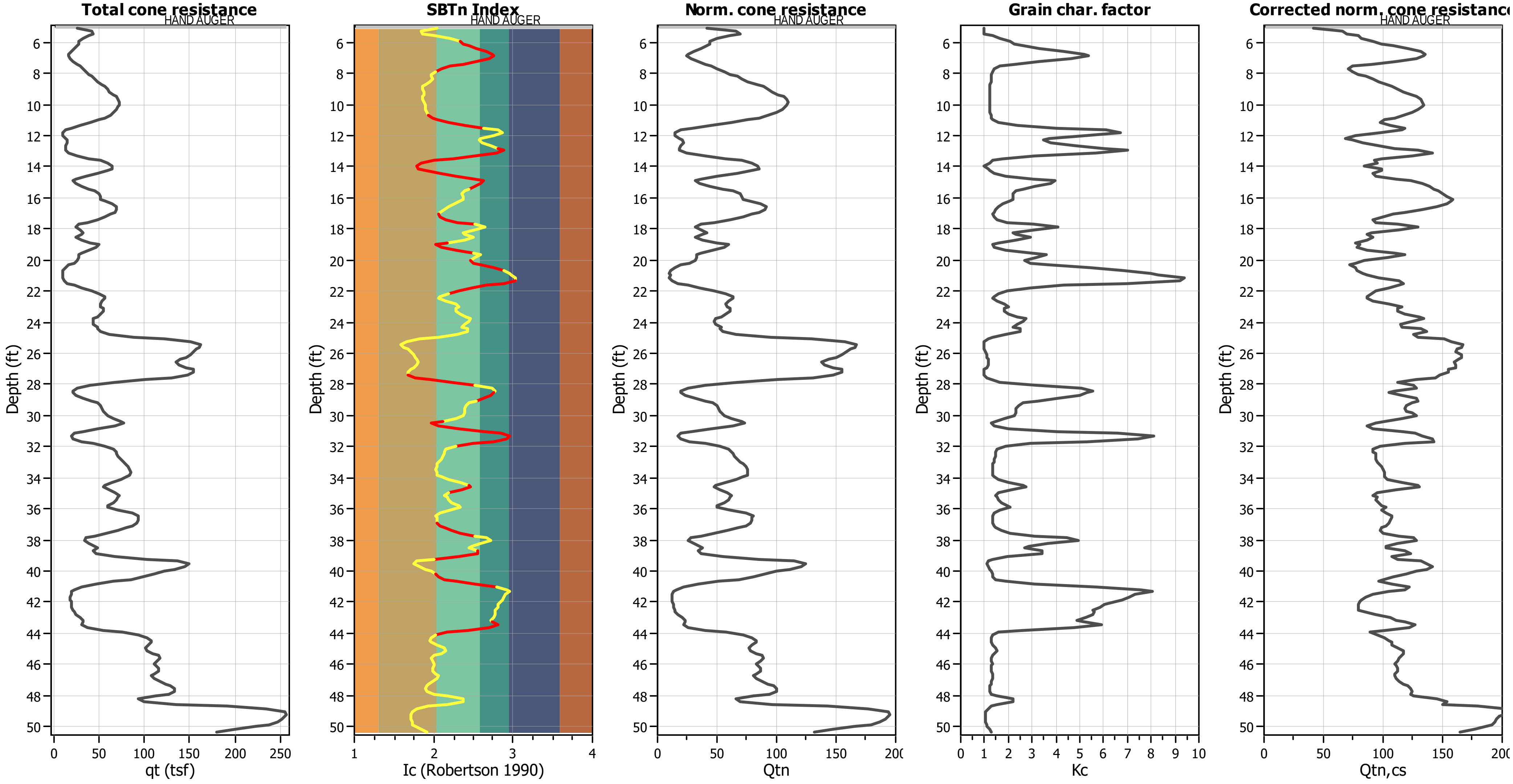
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty
🟩 5. Silty sand to sandy silt
🟨 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to
⬜ 9. Very stiff fine grained

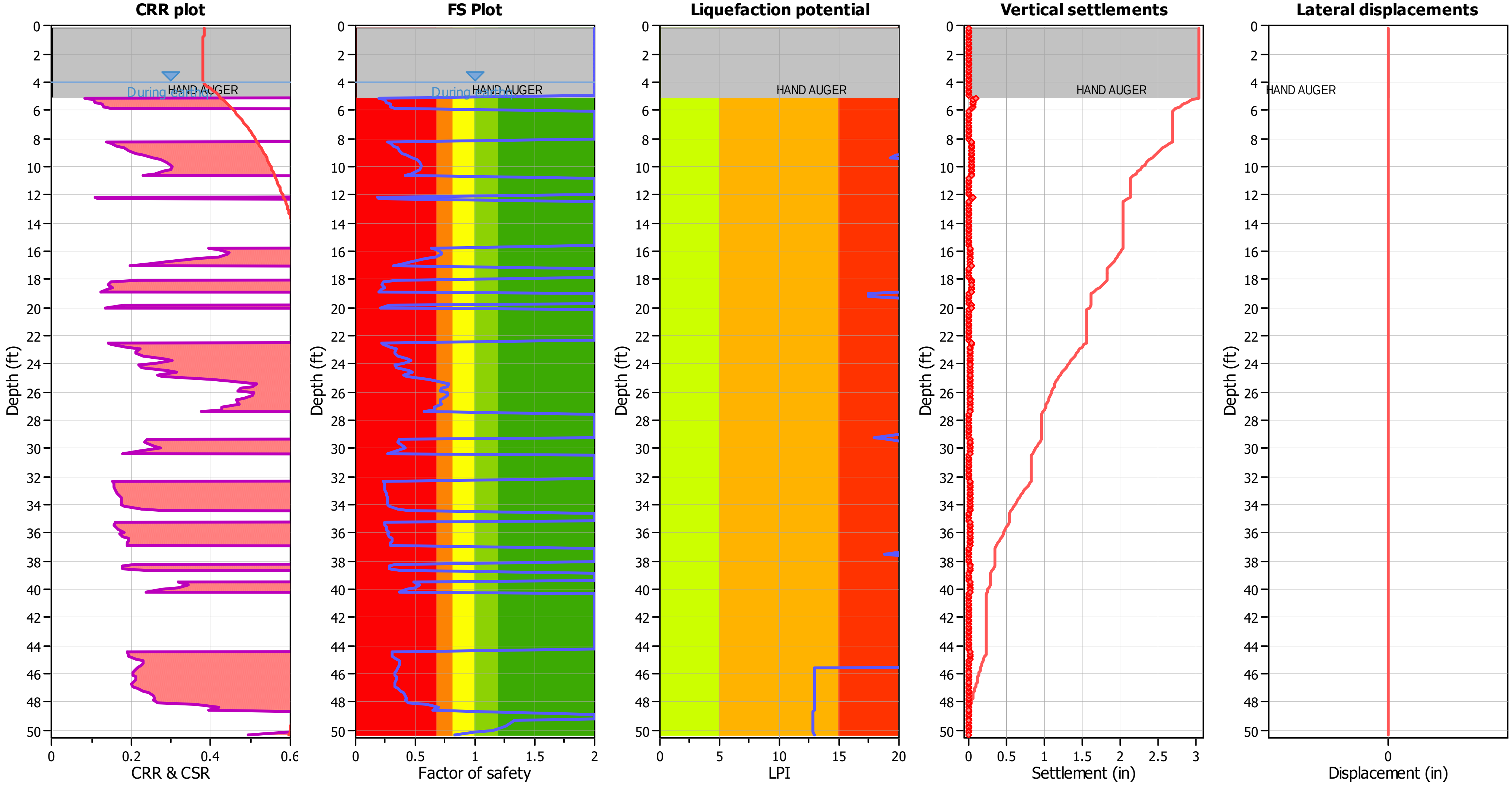
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_{σ} applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

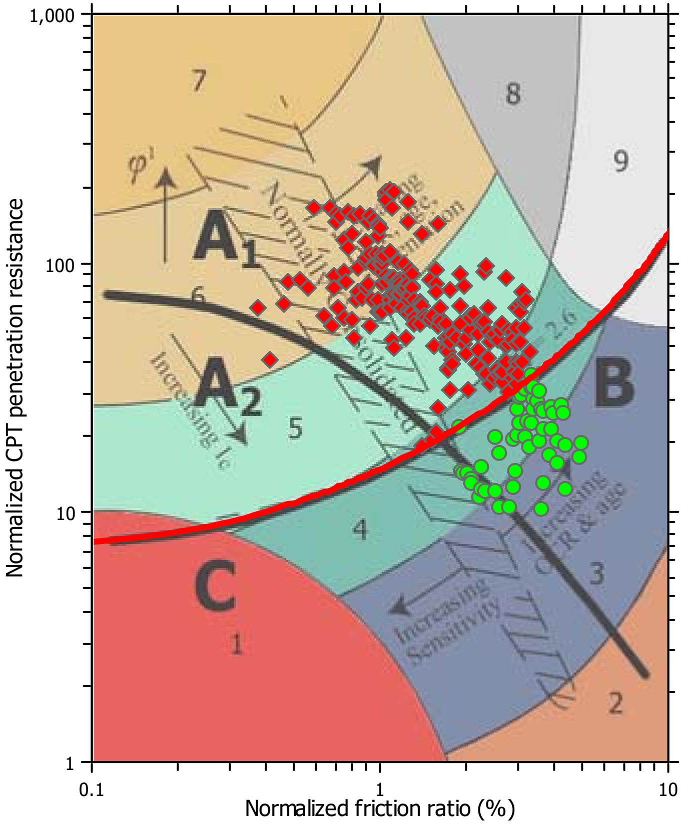
F.S. color scheme

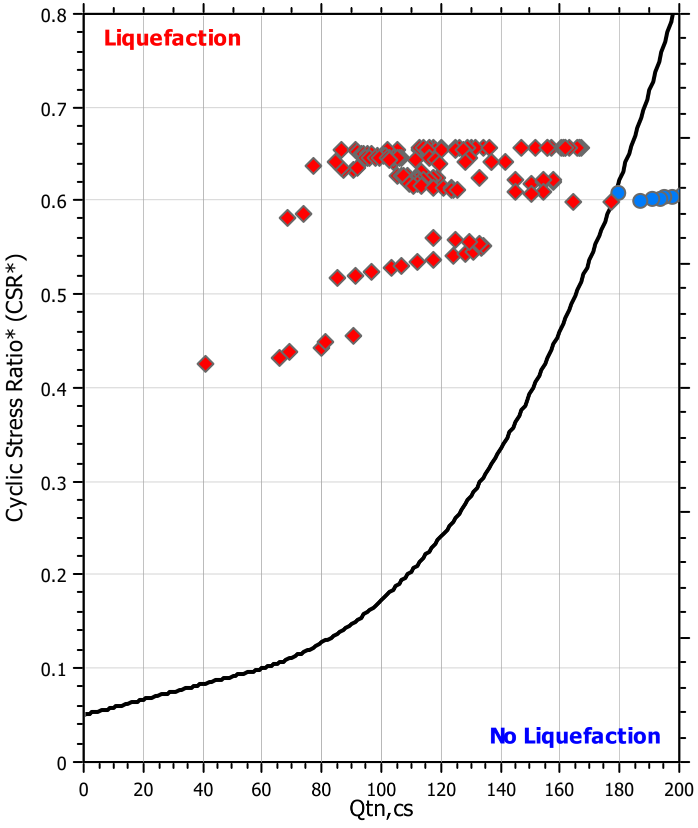
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

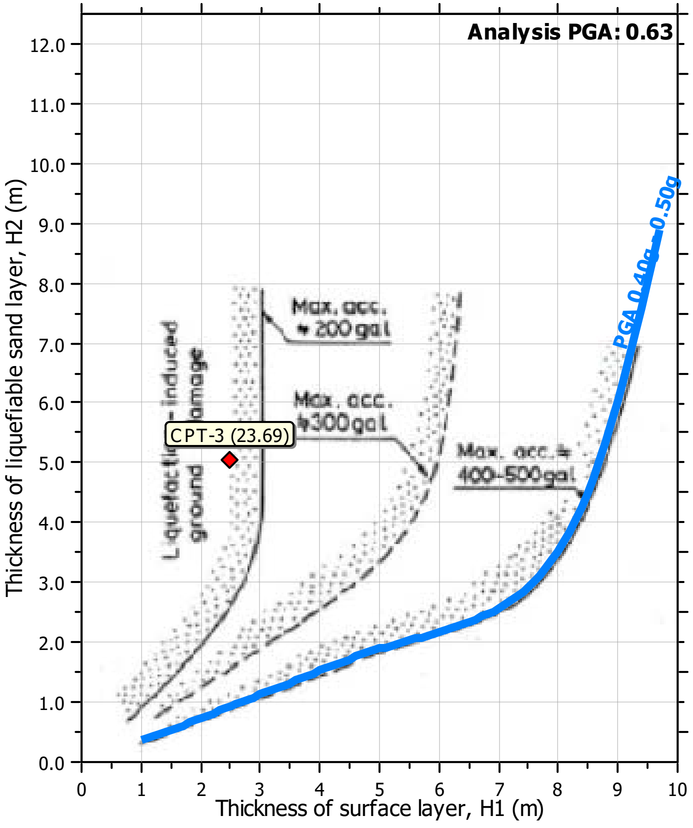
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟩 Low risk

Liquefaction analysis summary plots



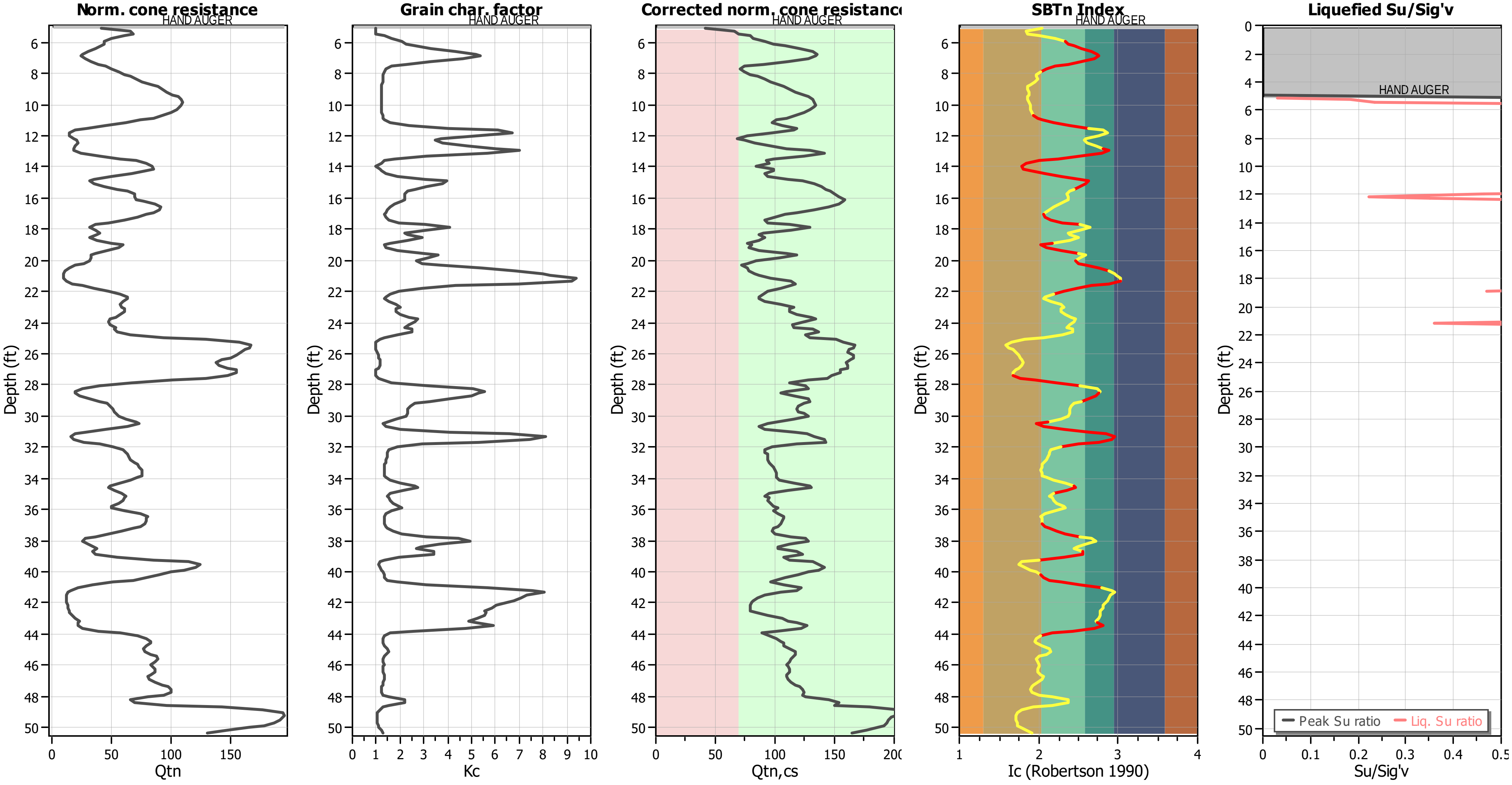




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

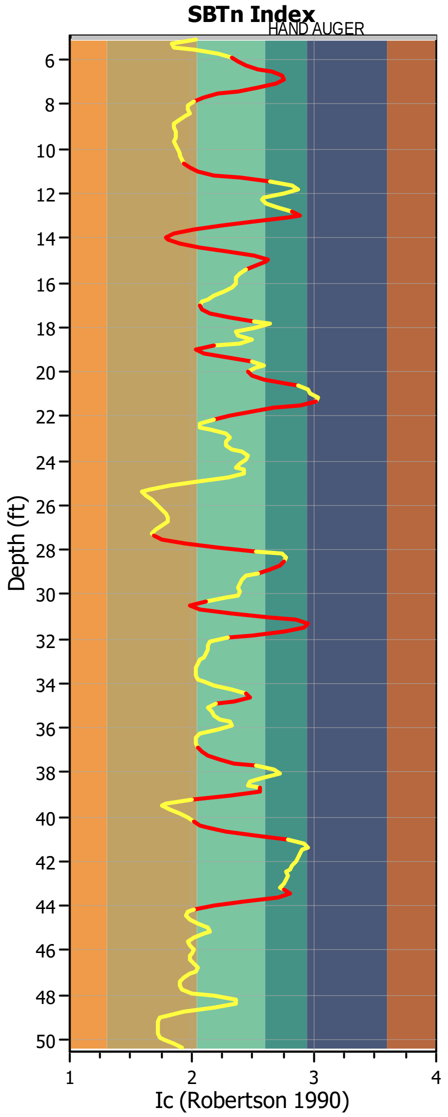
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

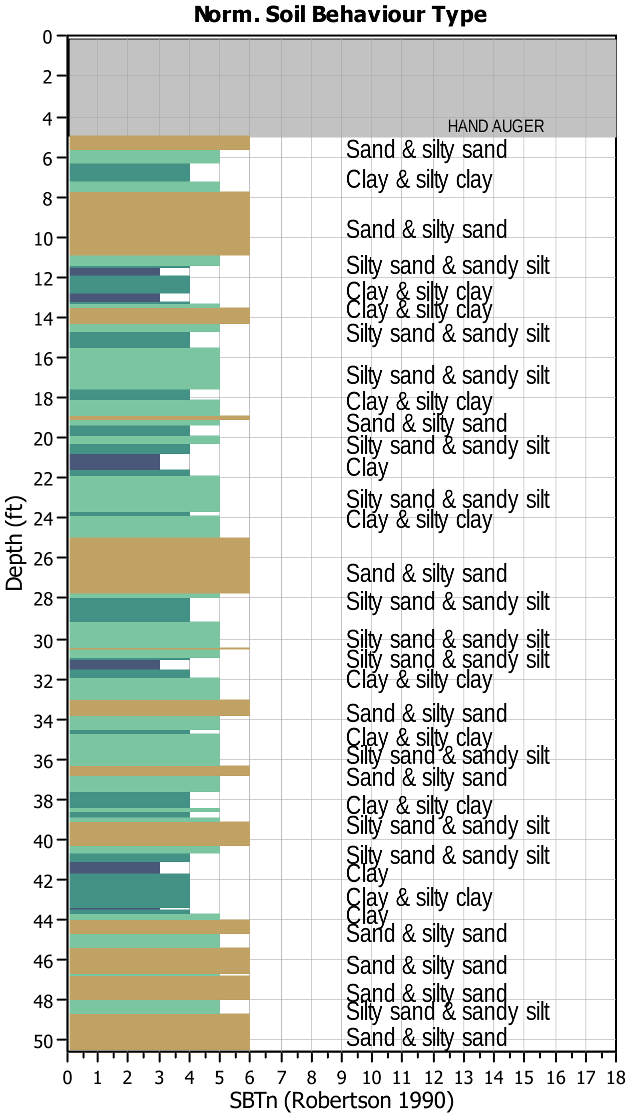
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





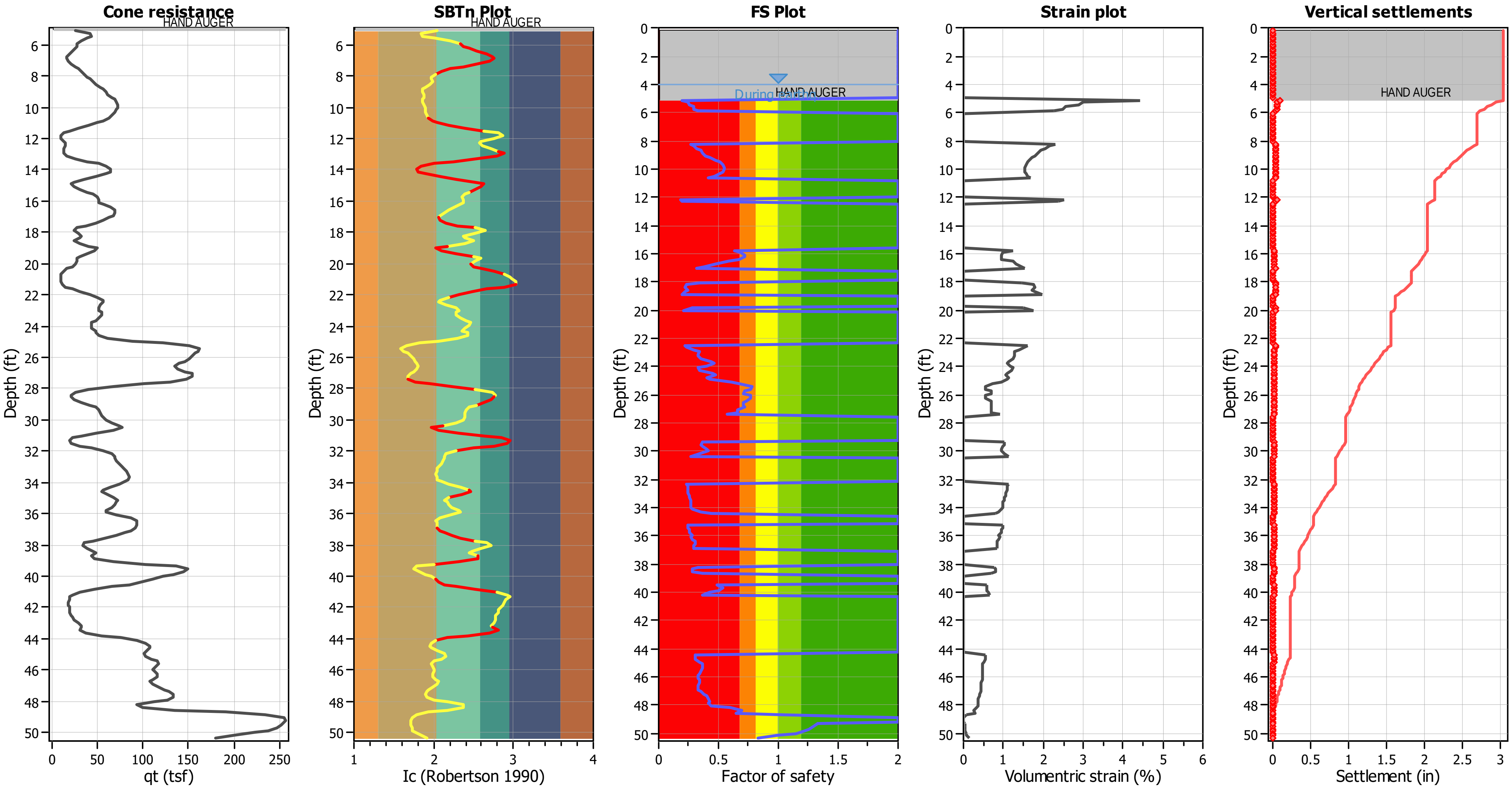
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	307
Total points excluded:	103
Exclusion percentage:	33.55%
Number of layers detected:	19

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

GEOCON
Geocon West, Inc.
2807 McGaw Avenue
Irvine, CA 92614
www.geoconinc.com

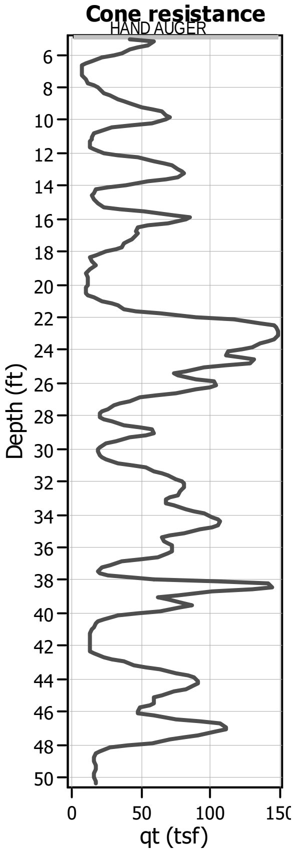
LIQUEFACTION ANALYSIS REPORT

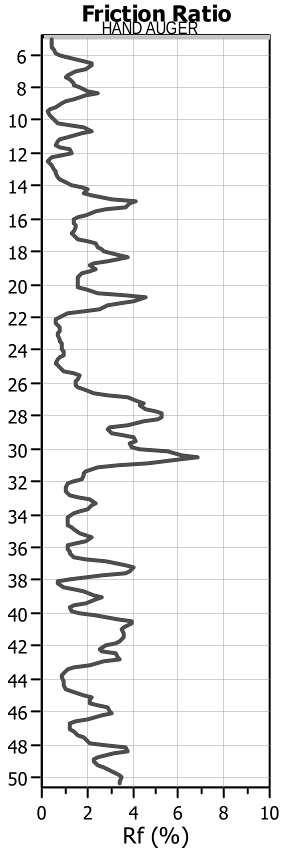
Project title : W2045-88-01 **Location : Euclid and Heil**

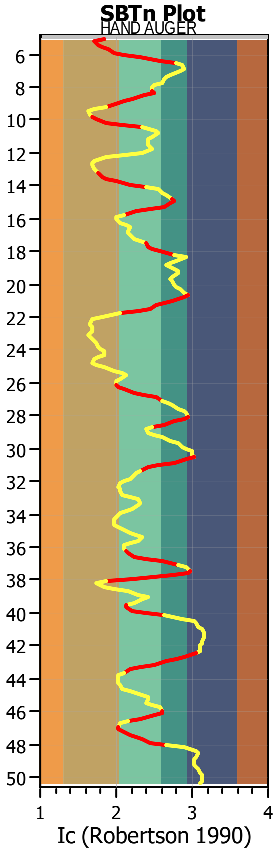
CPT file : CPT-4

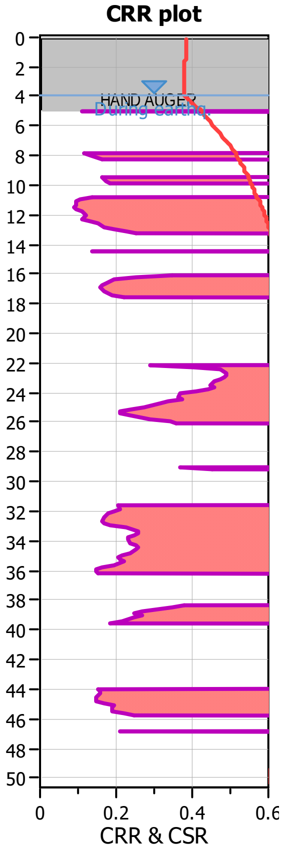
Input parameters and analysis data

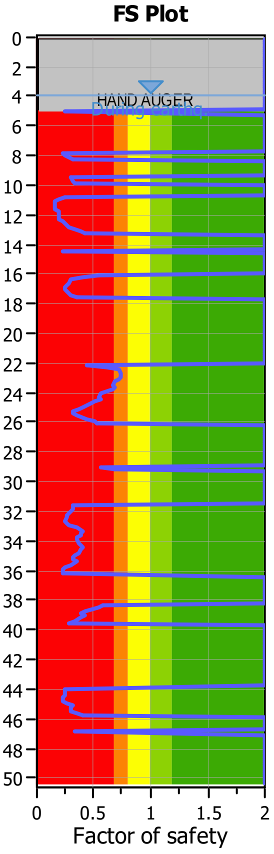
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.63	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

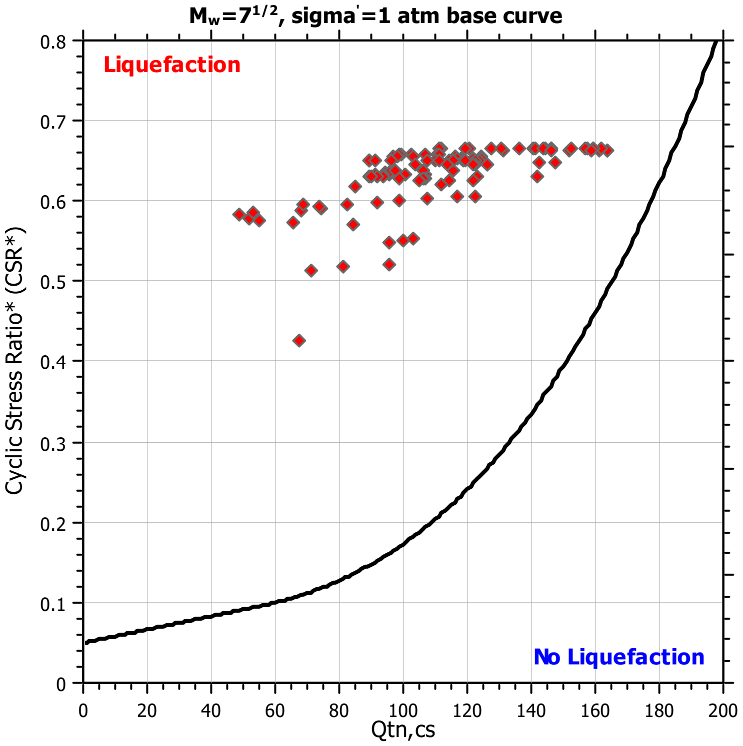


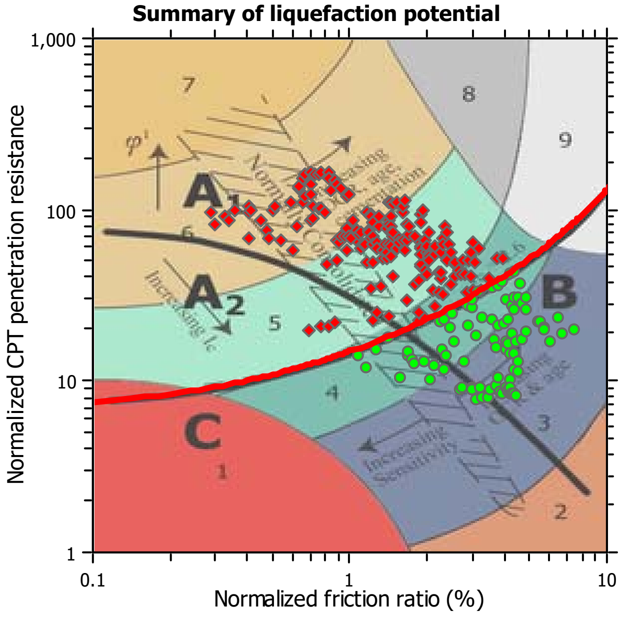






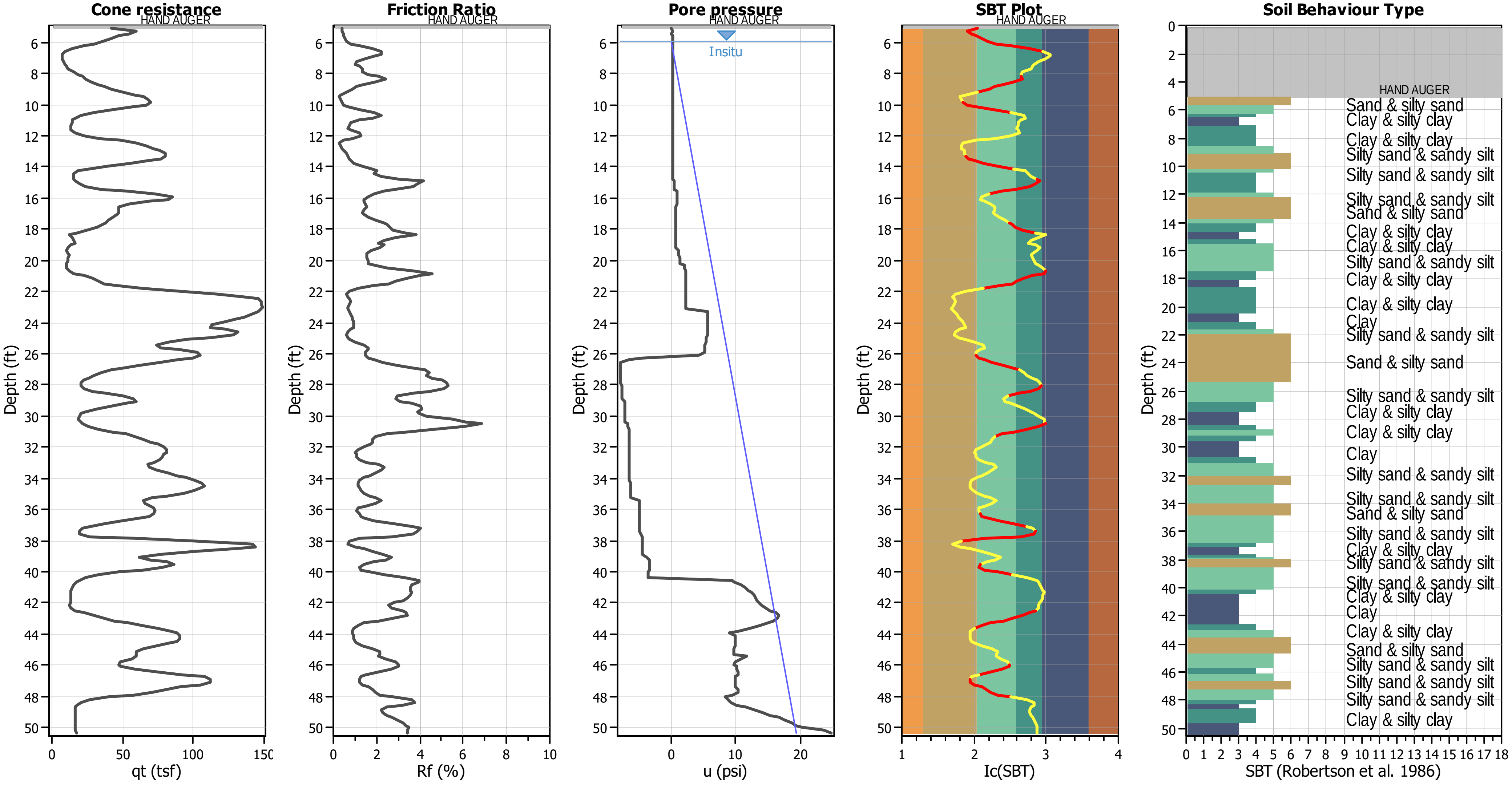






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



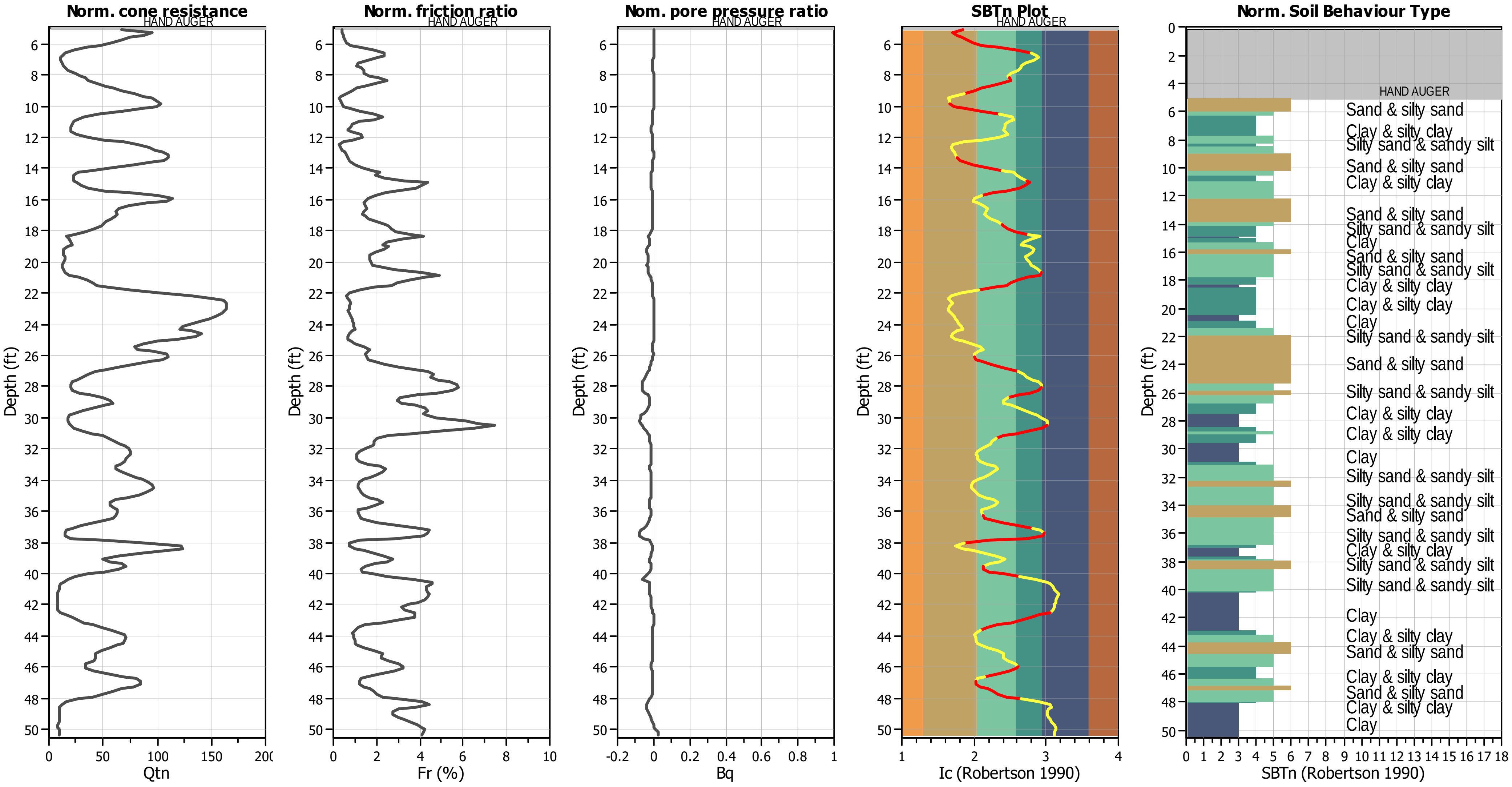
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



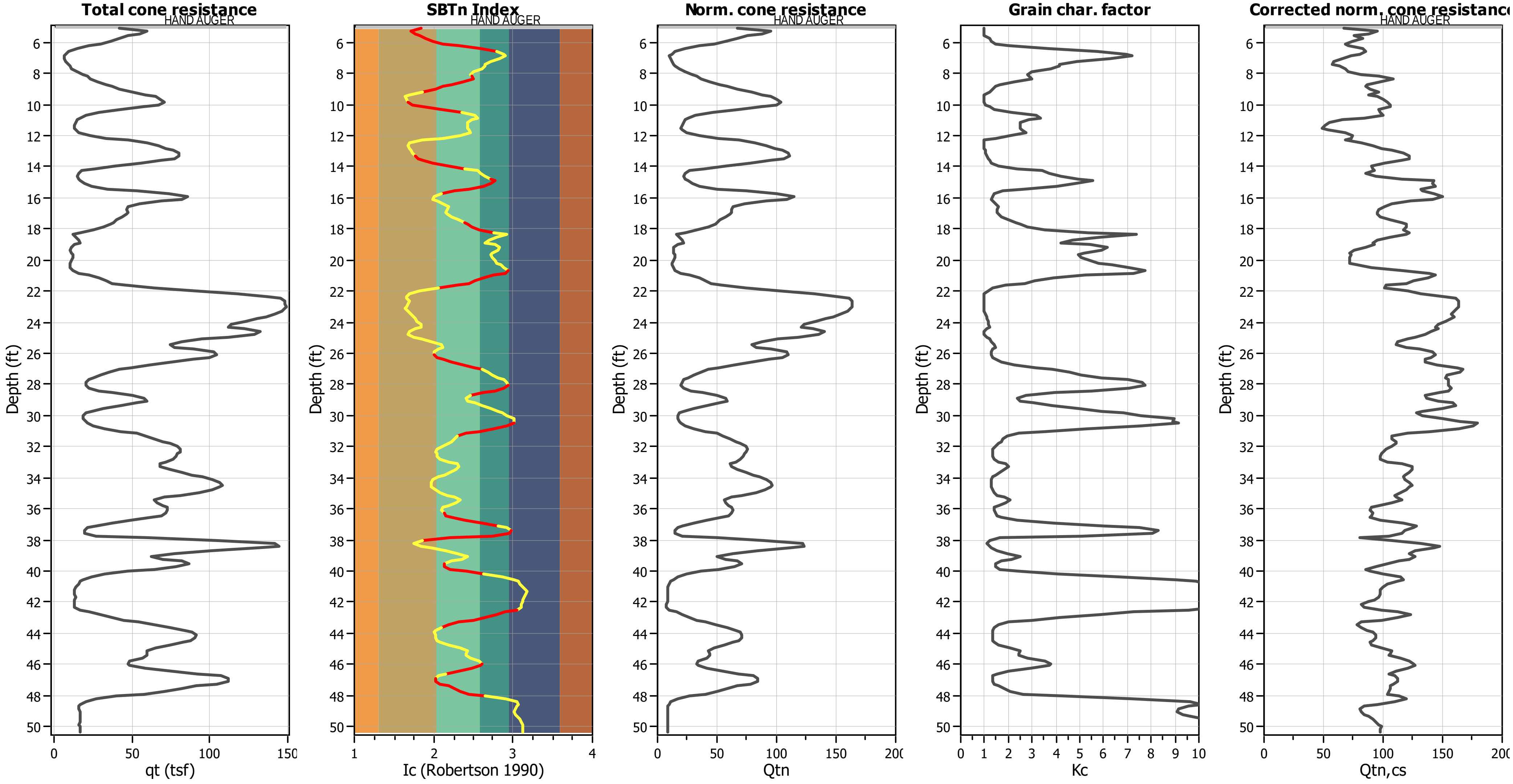
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

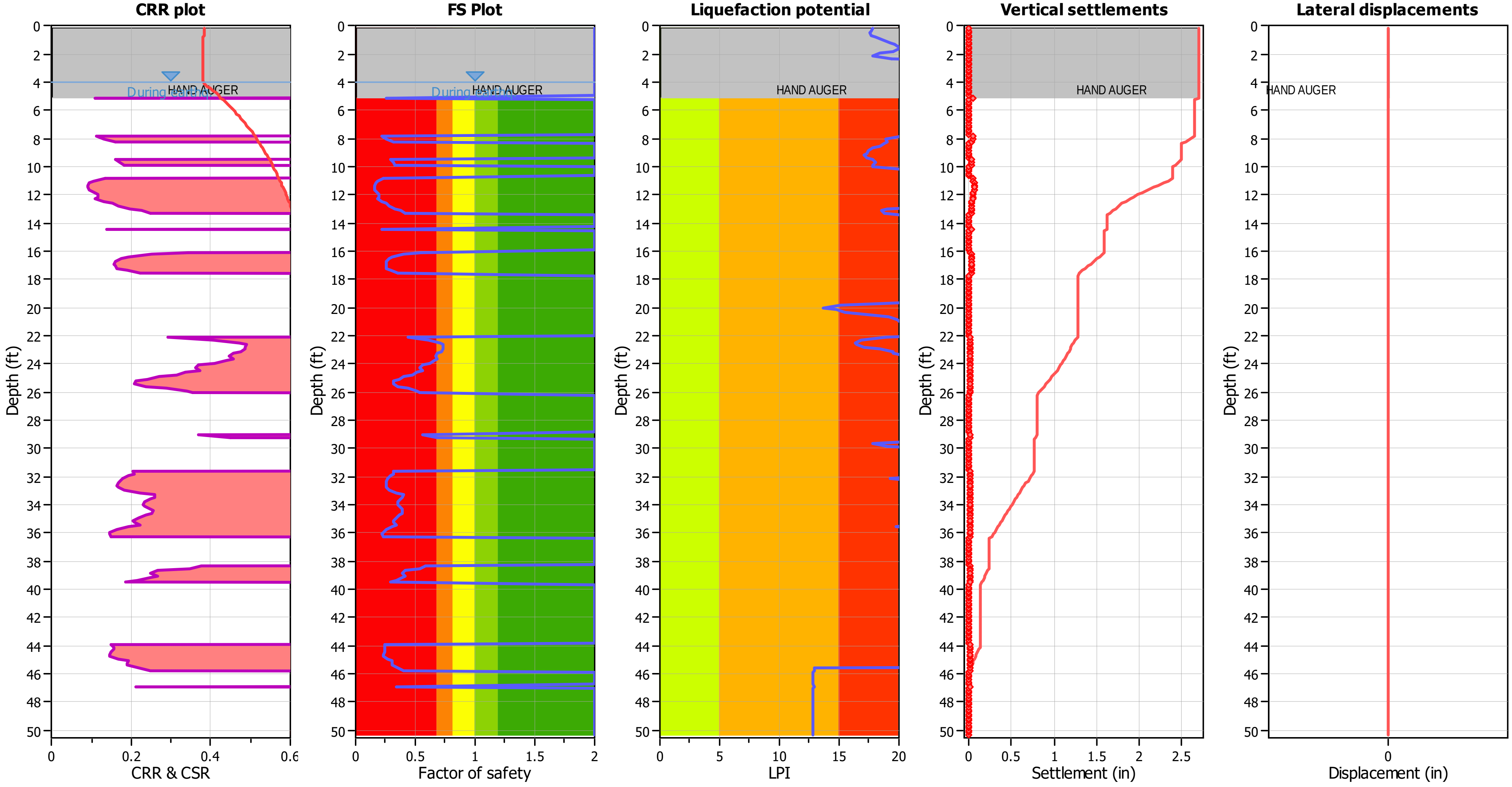
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

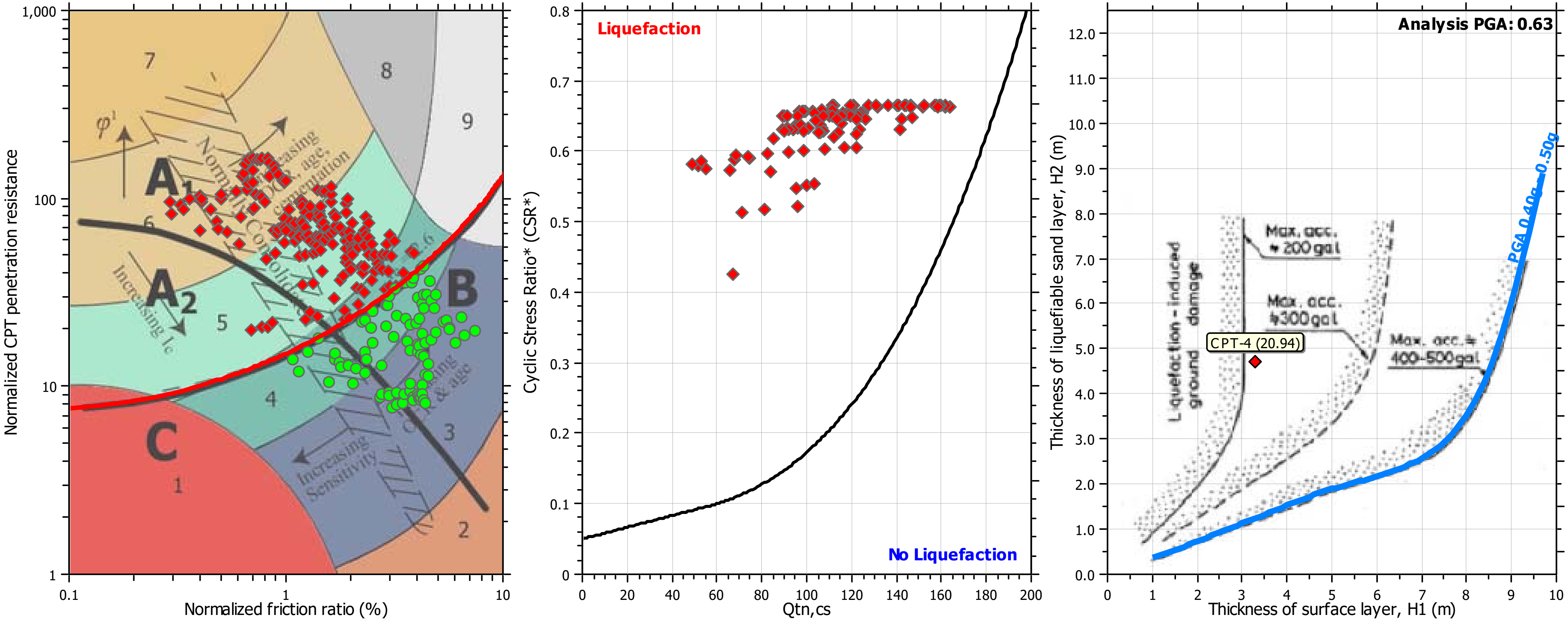
F.S. color scheme

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

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

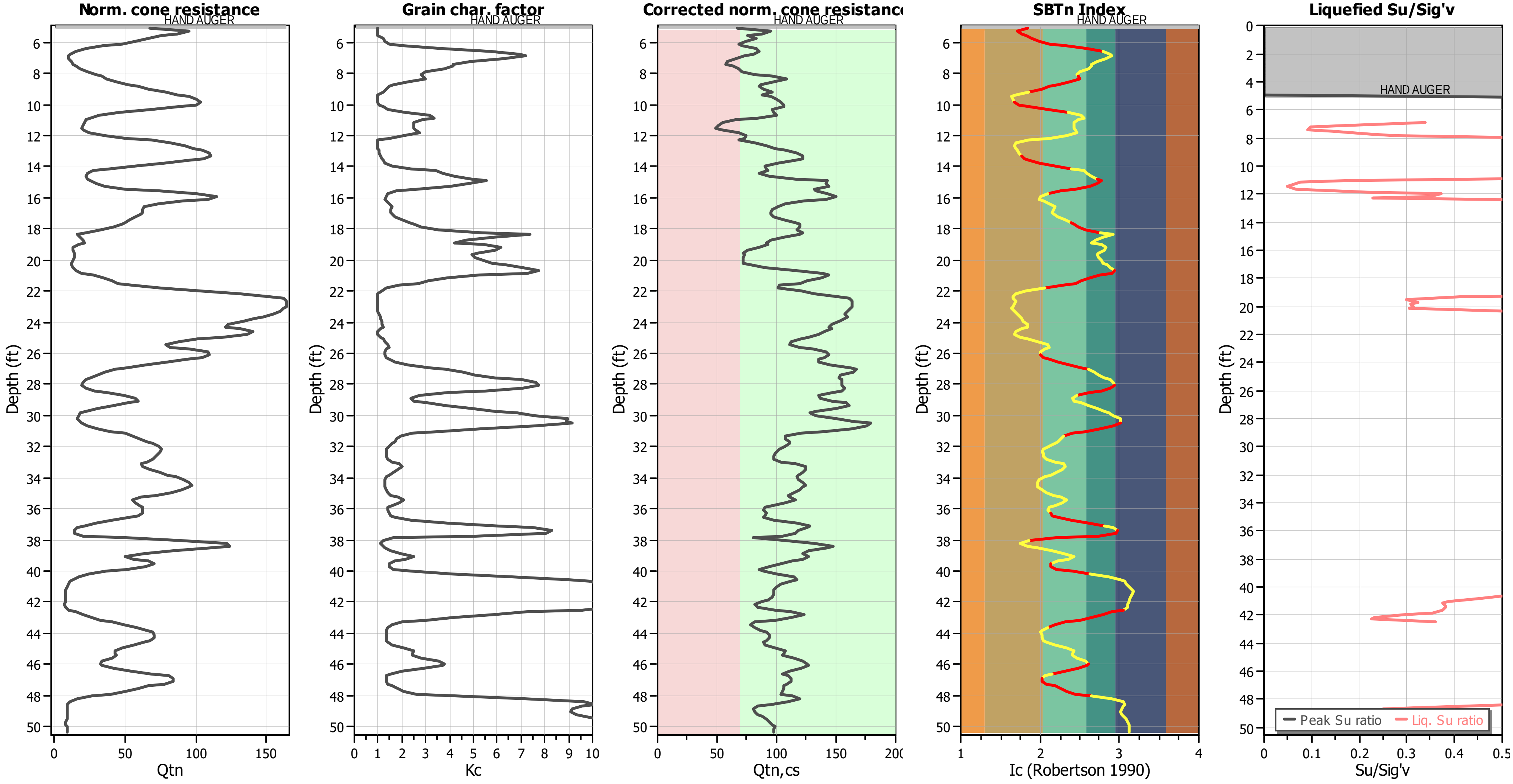
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_{σ} applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

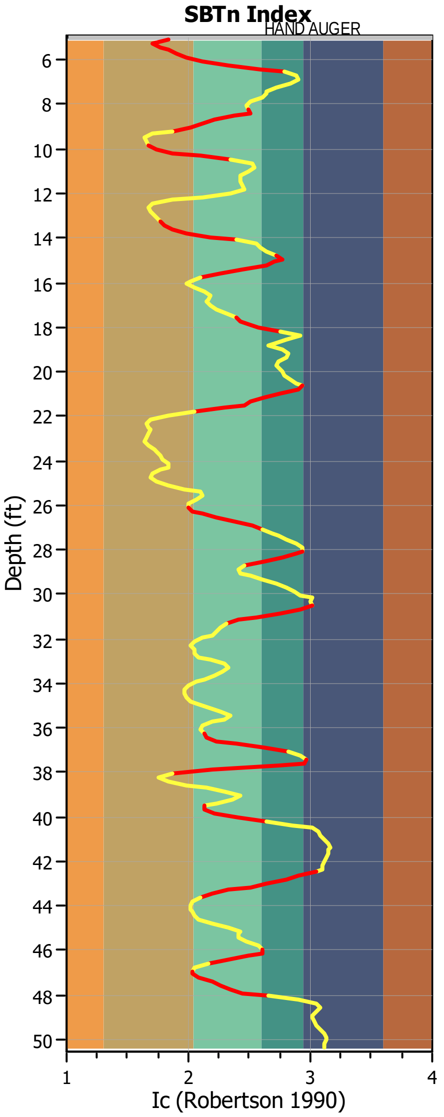
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

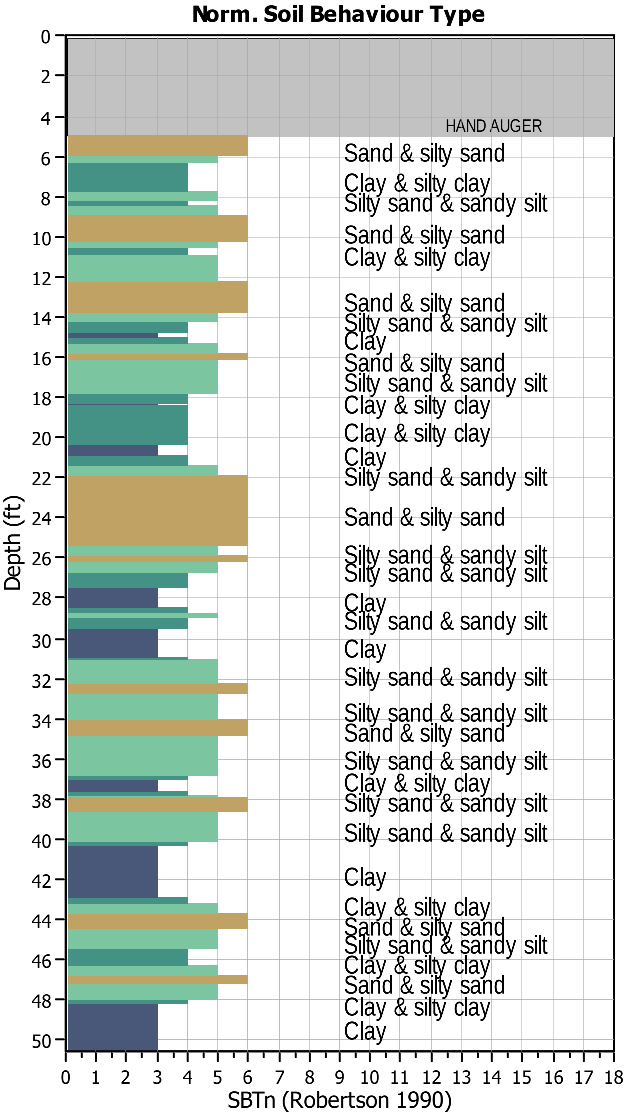
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

The software will delete data when the cone is in transition from either clay to sand or vise-versa. To do this the software requires a range of I_c values over which the transition will be defined (typically somewhere between $1.80 < I_c < 3.0$) and a rate of change of I_c. Transitions typically occur when the rate of change of I_c is fast (i.e. delta I_c is small).

The SBT_n plot below, displays in red the detected transition layers based on the parameters listed below the graphs.





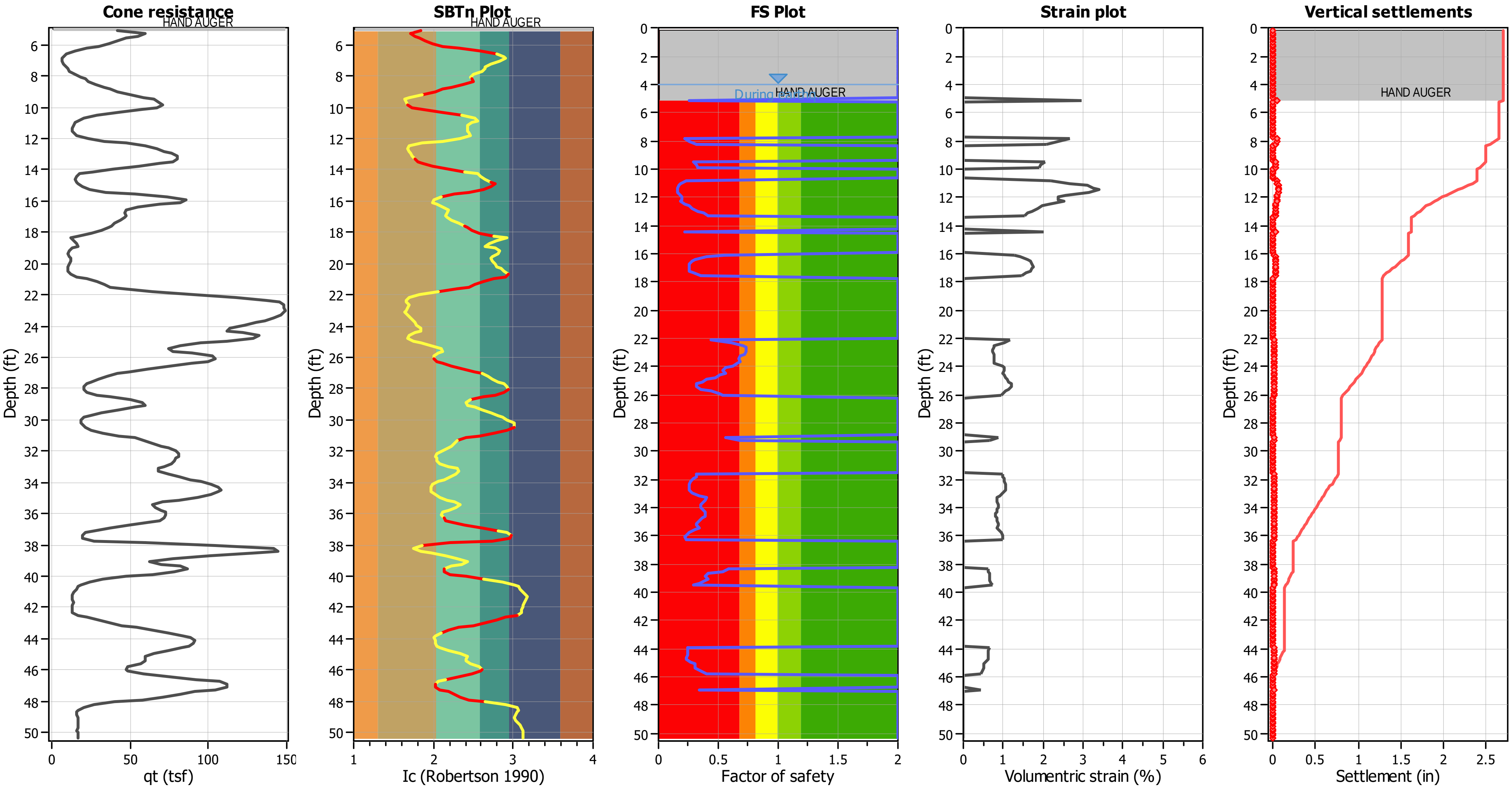
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	307
Total points excluded:	103
Exclusion percentage:	33.55%
Number of layers detected:	16

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

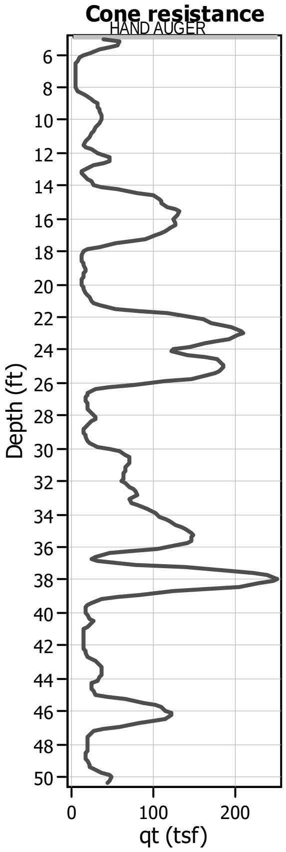
LIQUEFACTION ANALYSIS REPORT

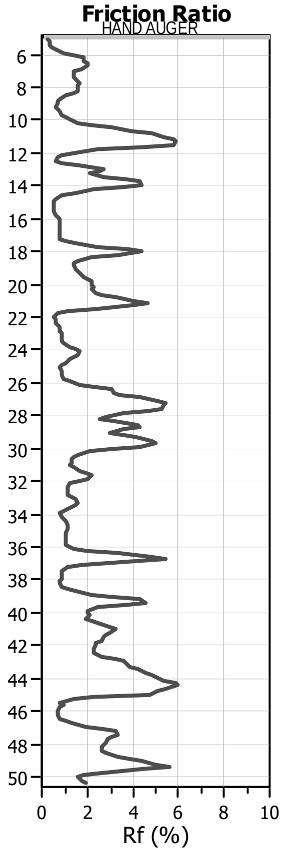
Project title : W2045-88-01 **Location : Euclid and Heil**

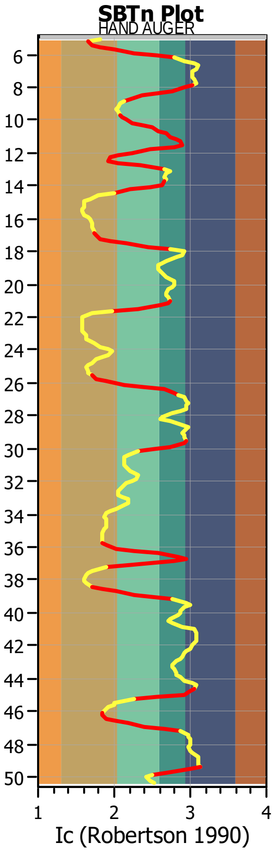
CPT file : CPT-5

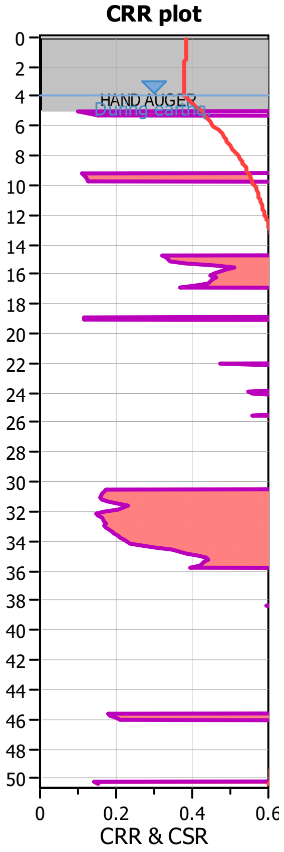
Input parameters and analysis data

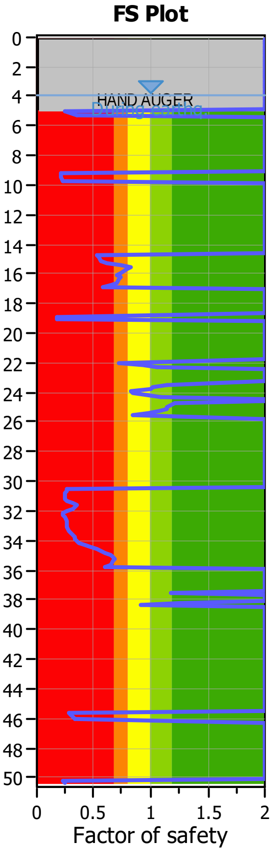
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	5.90 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	4.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.30	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.63	Unit weight calculation:	Based on SBT	K_{σ} applied:	Yes		

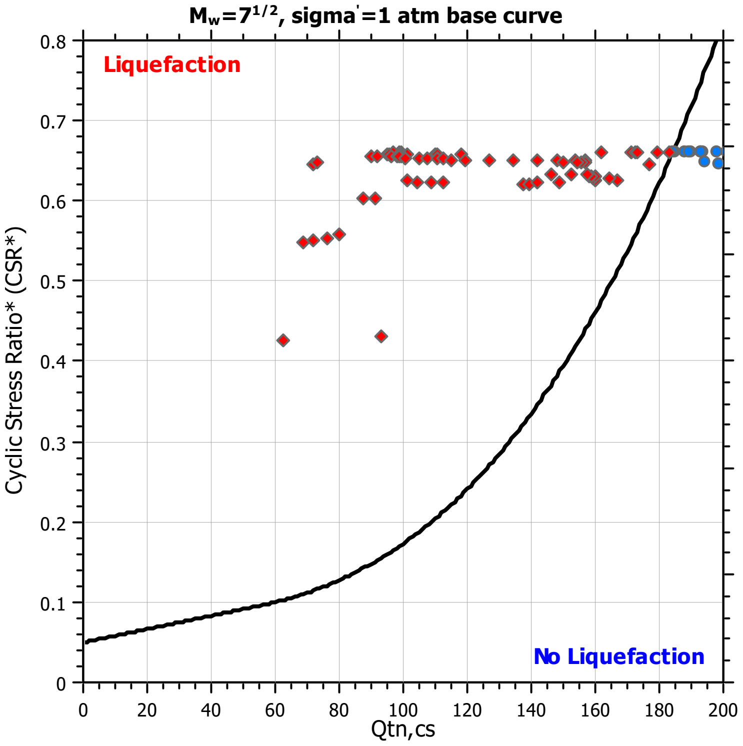


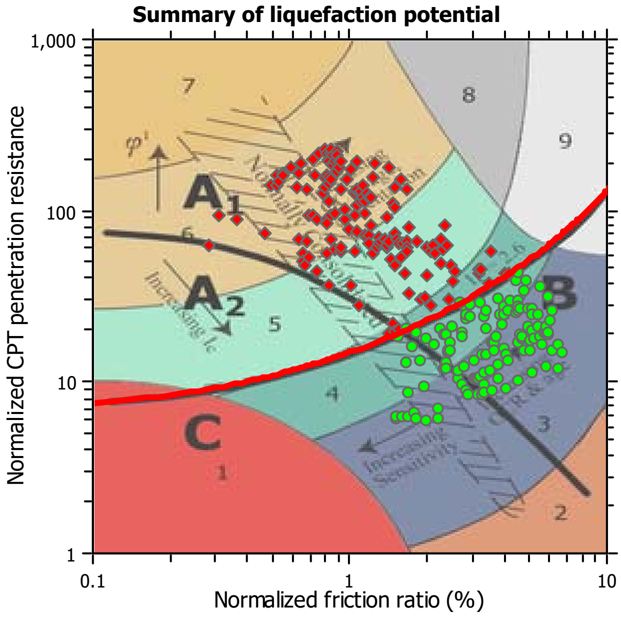






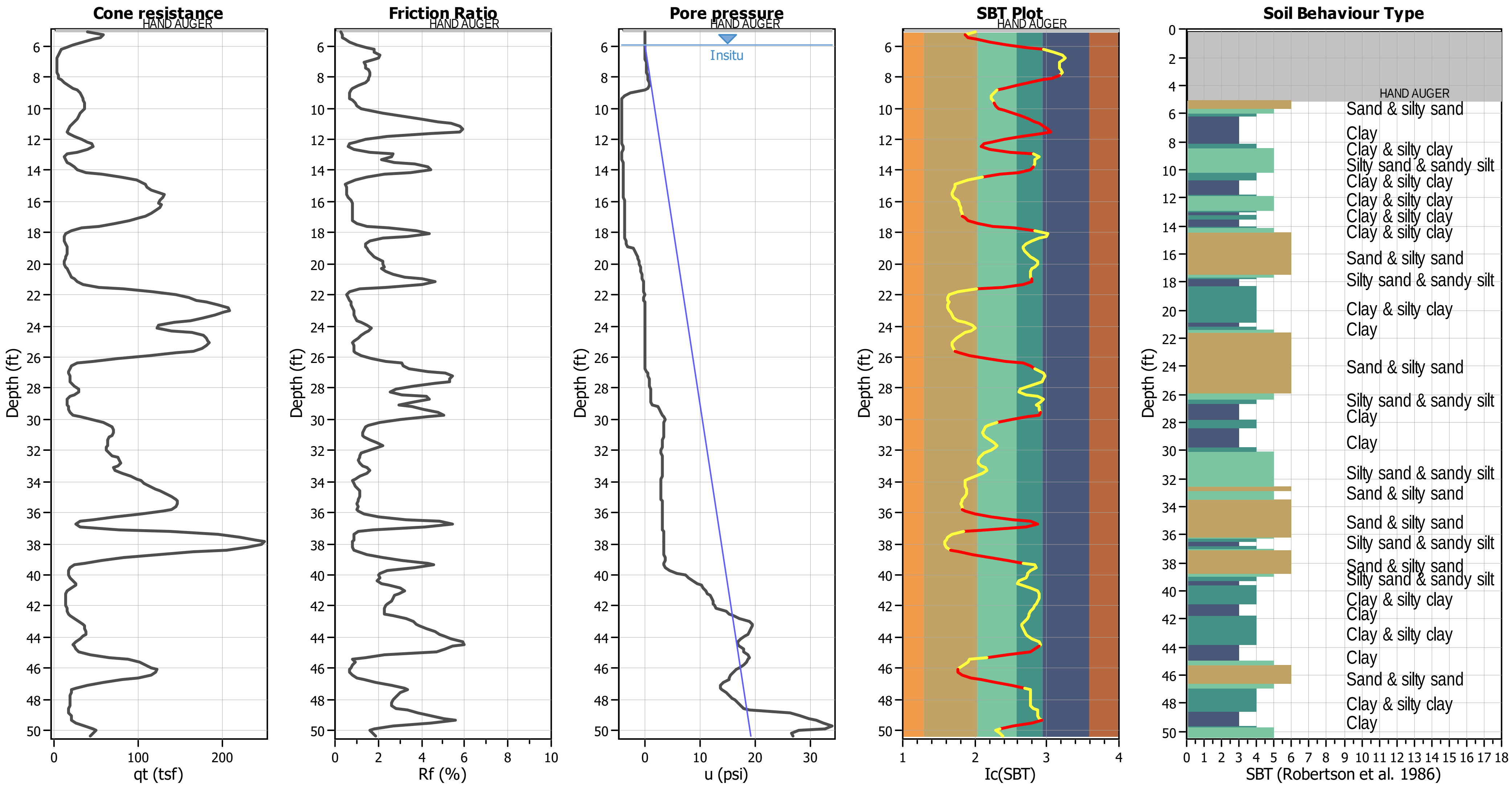






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



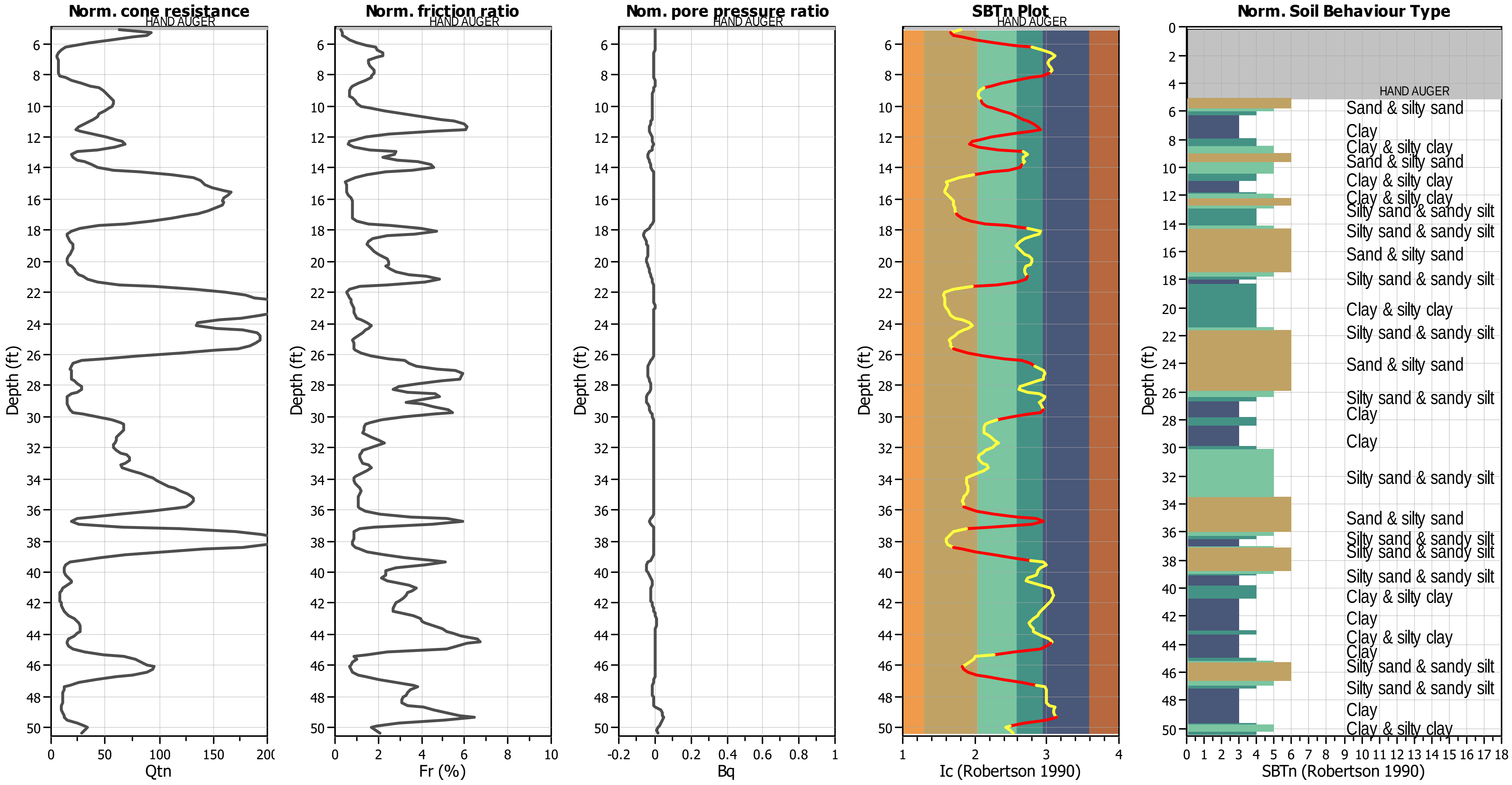
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



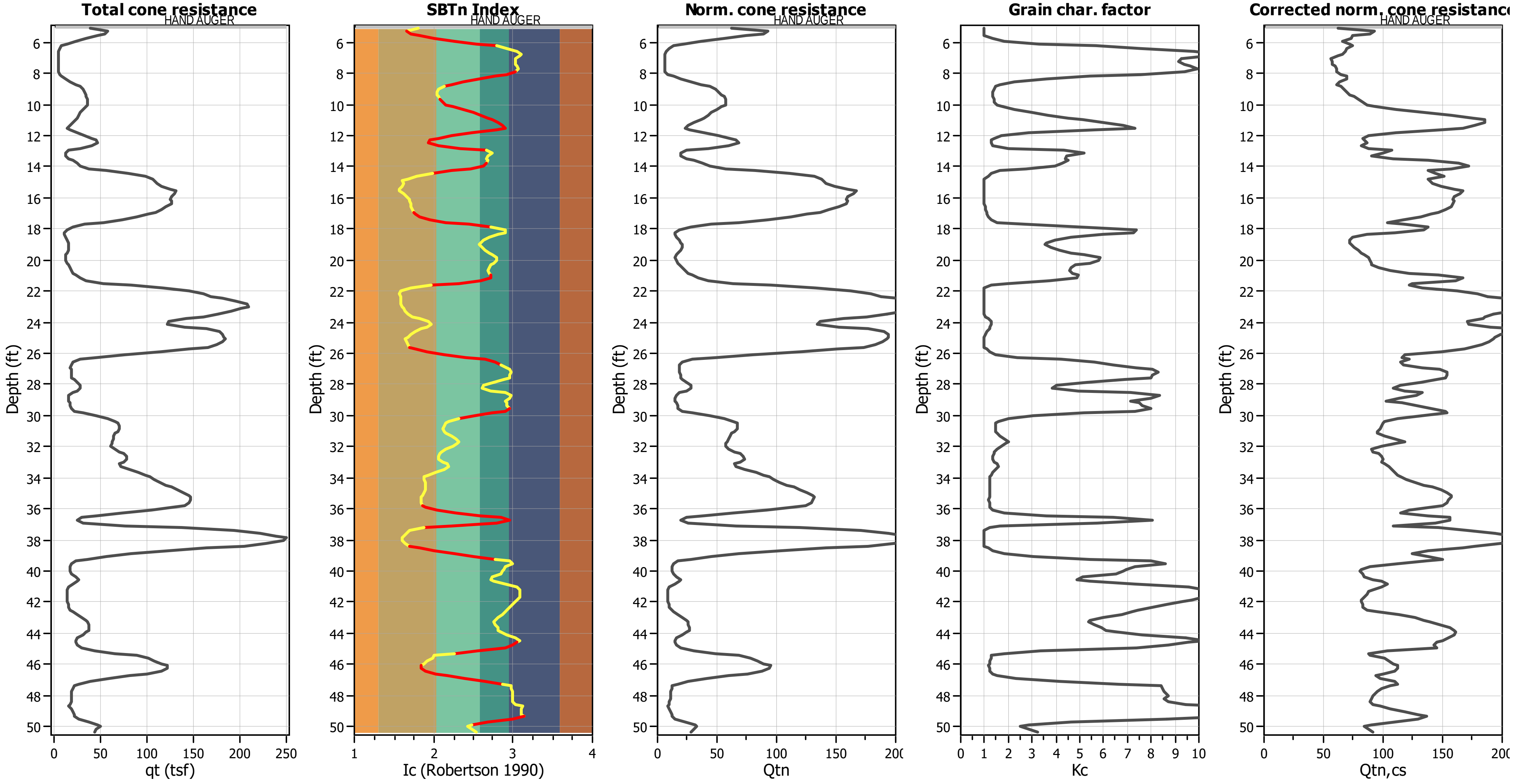
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

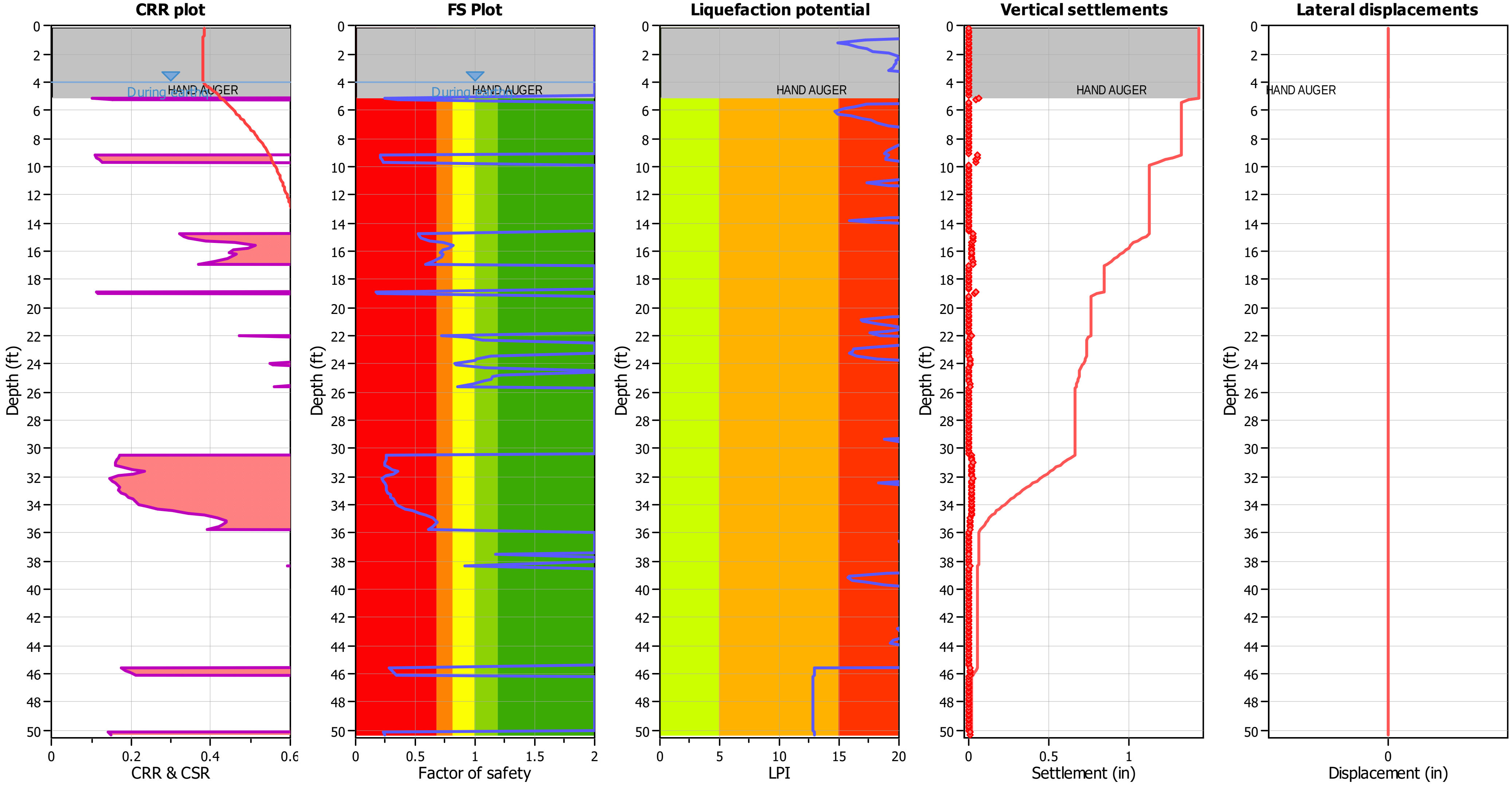
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

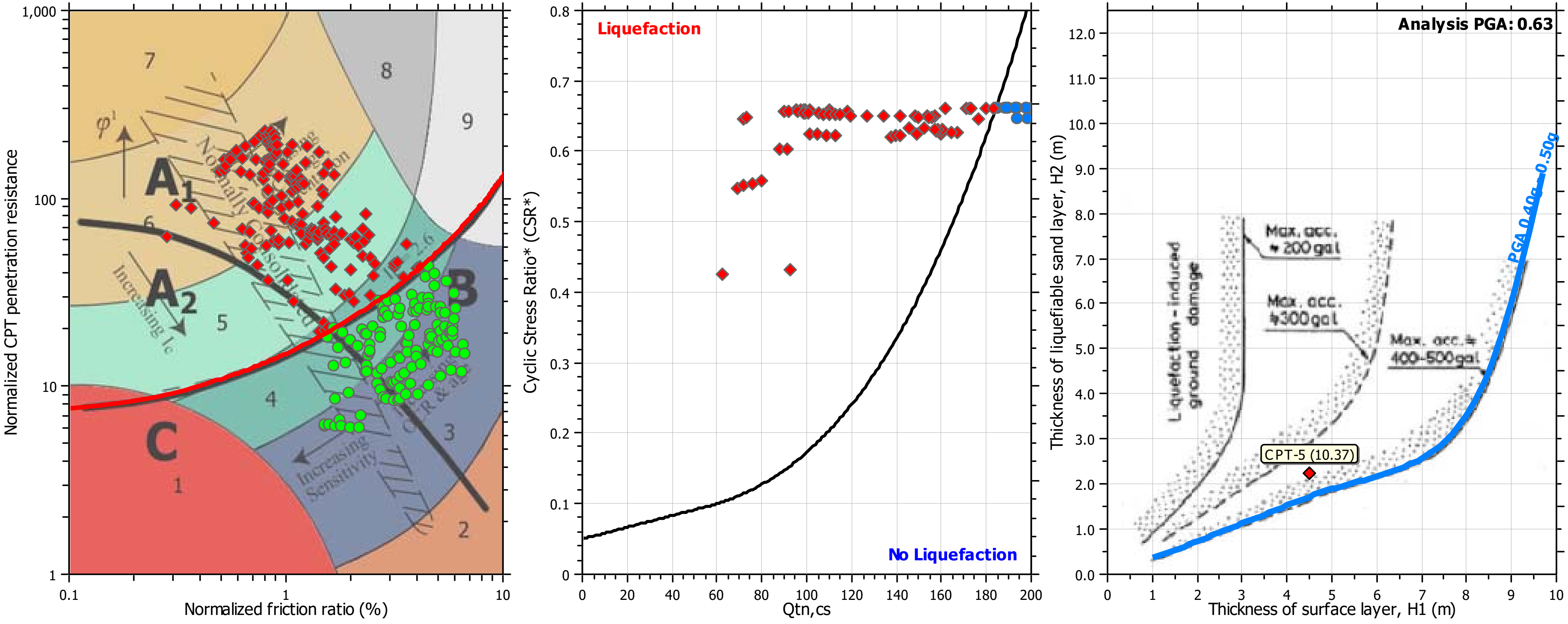
F.S. color scheme

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

LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

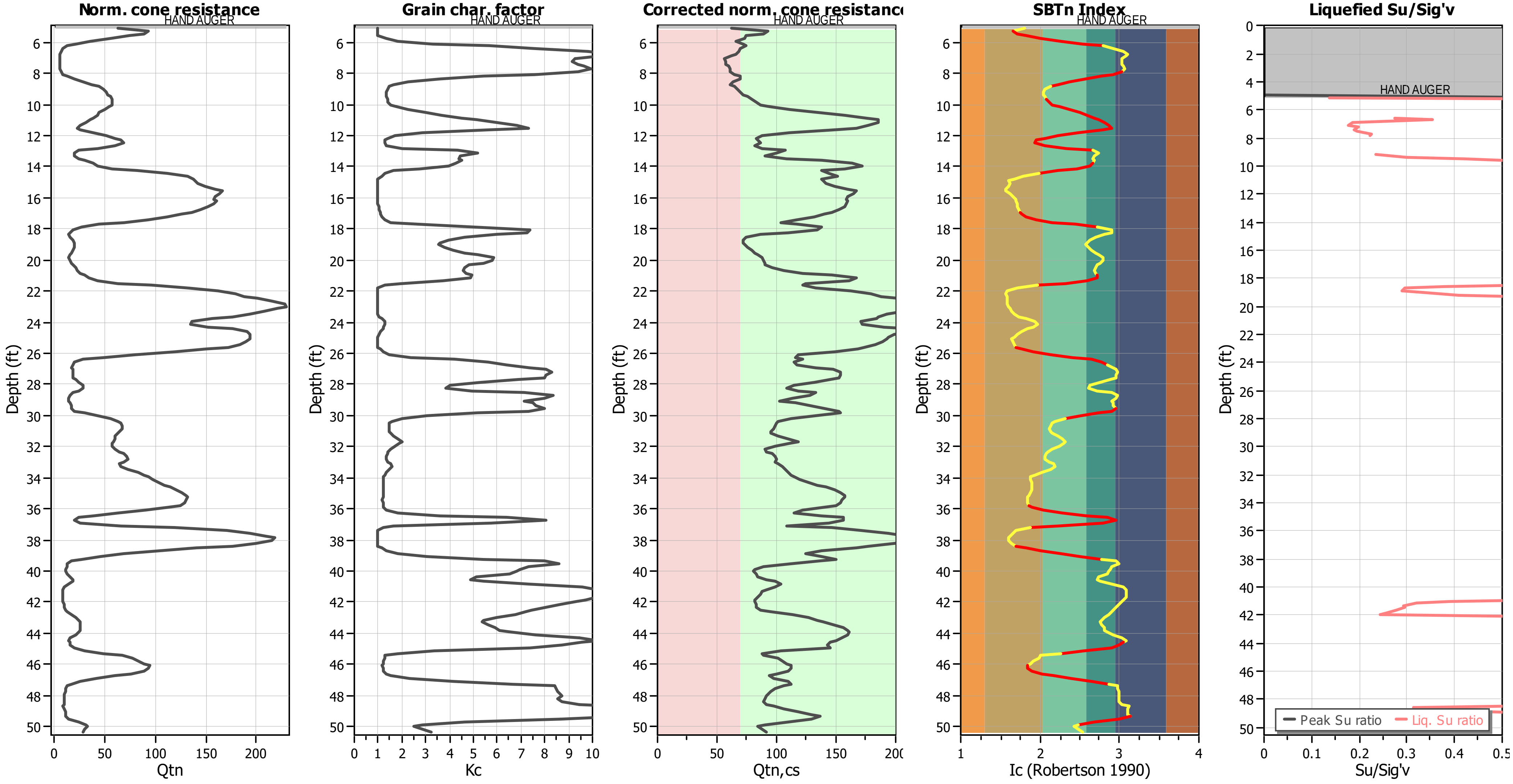
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
Earthquake magnitude M_w:	7.30	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.63	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	4.00 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_σ applied:	Yes
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Depth to water table (insitu):	5.90 ft	Fill height:	N/A	Limit depth:	N/A

TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

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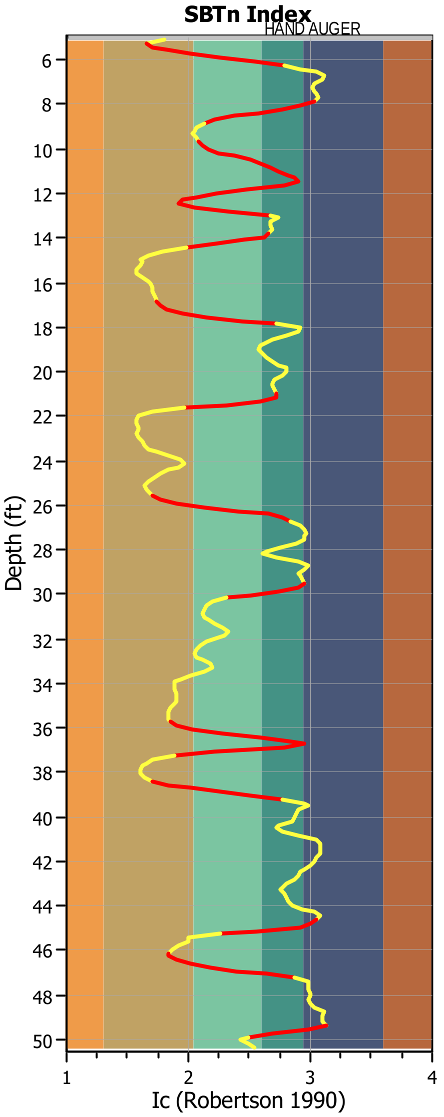
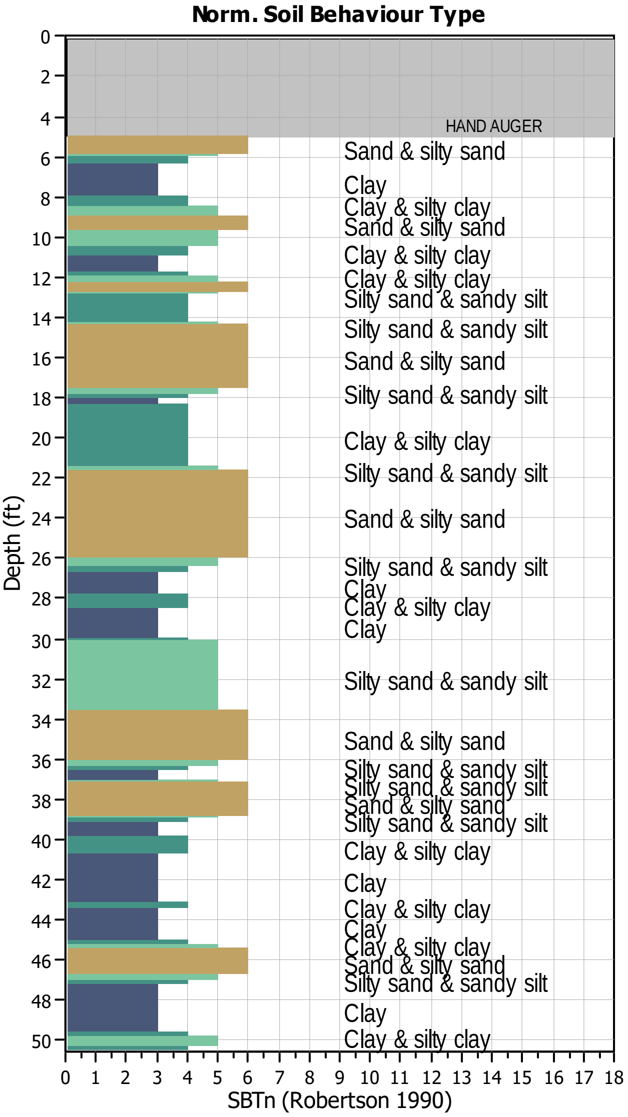




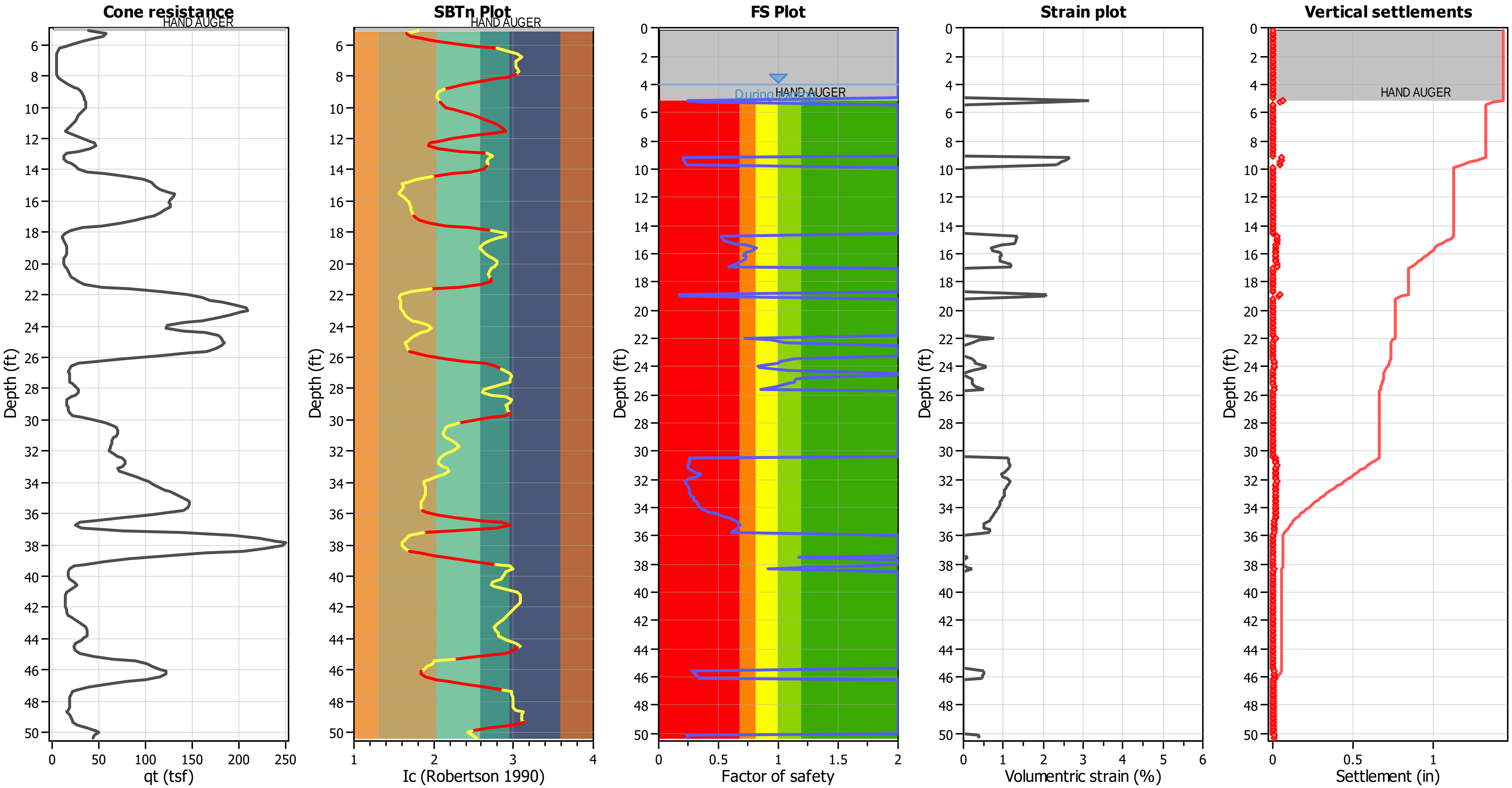
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	3.00
I_c change ratio value:	0.0250
Minimum number of points in layer:	4

General statistics

Total points in CPT file:	307
Total points excluded:	99
Exclusion percentage:	32.25%
Number of layers detected:	16

Estimation of post-earthquake settlements

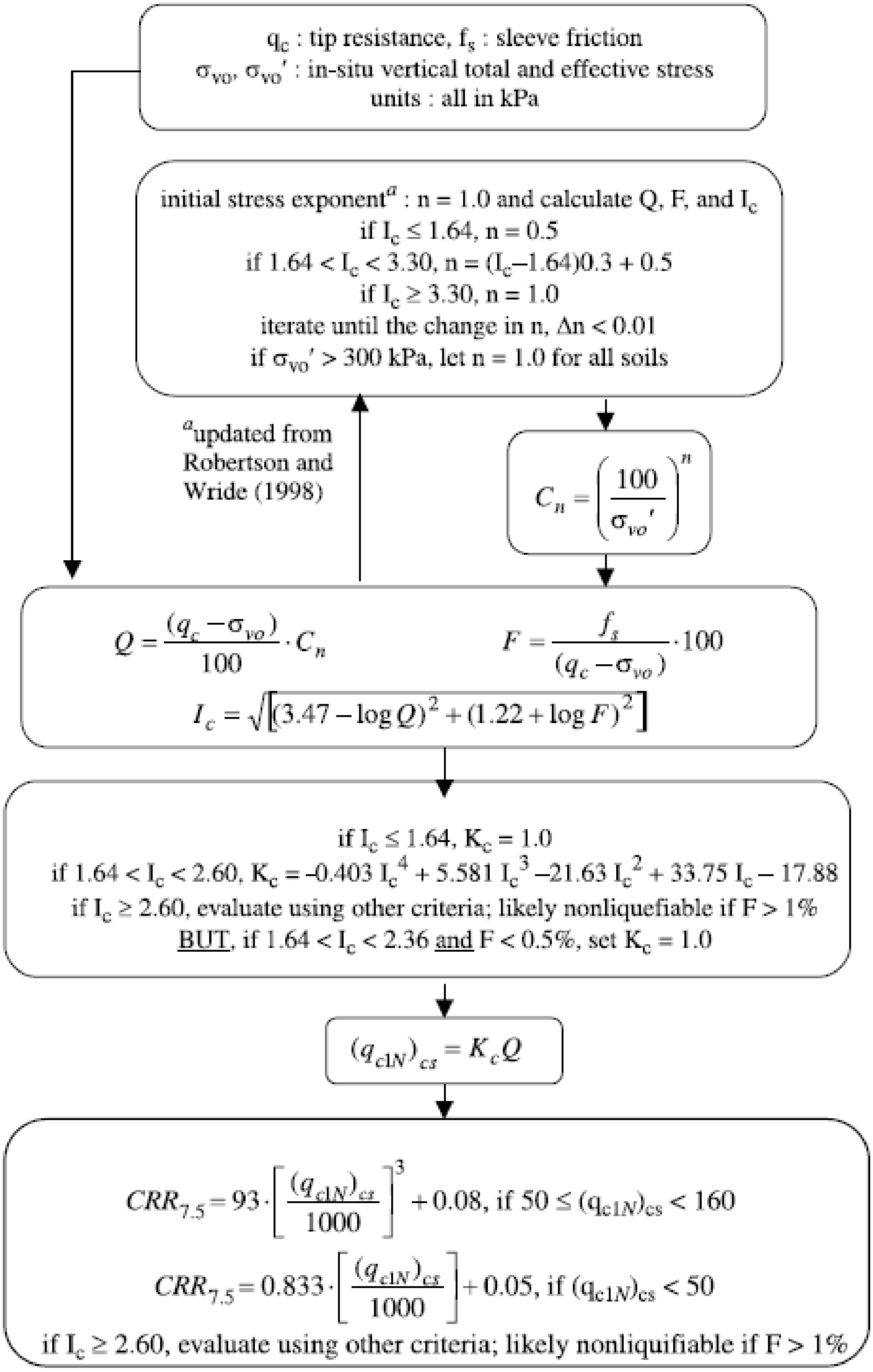


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



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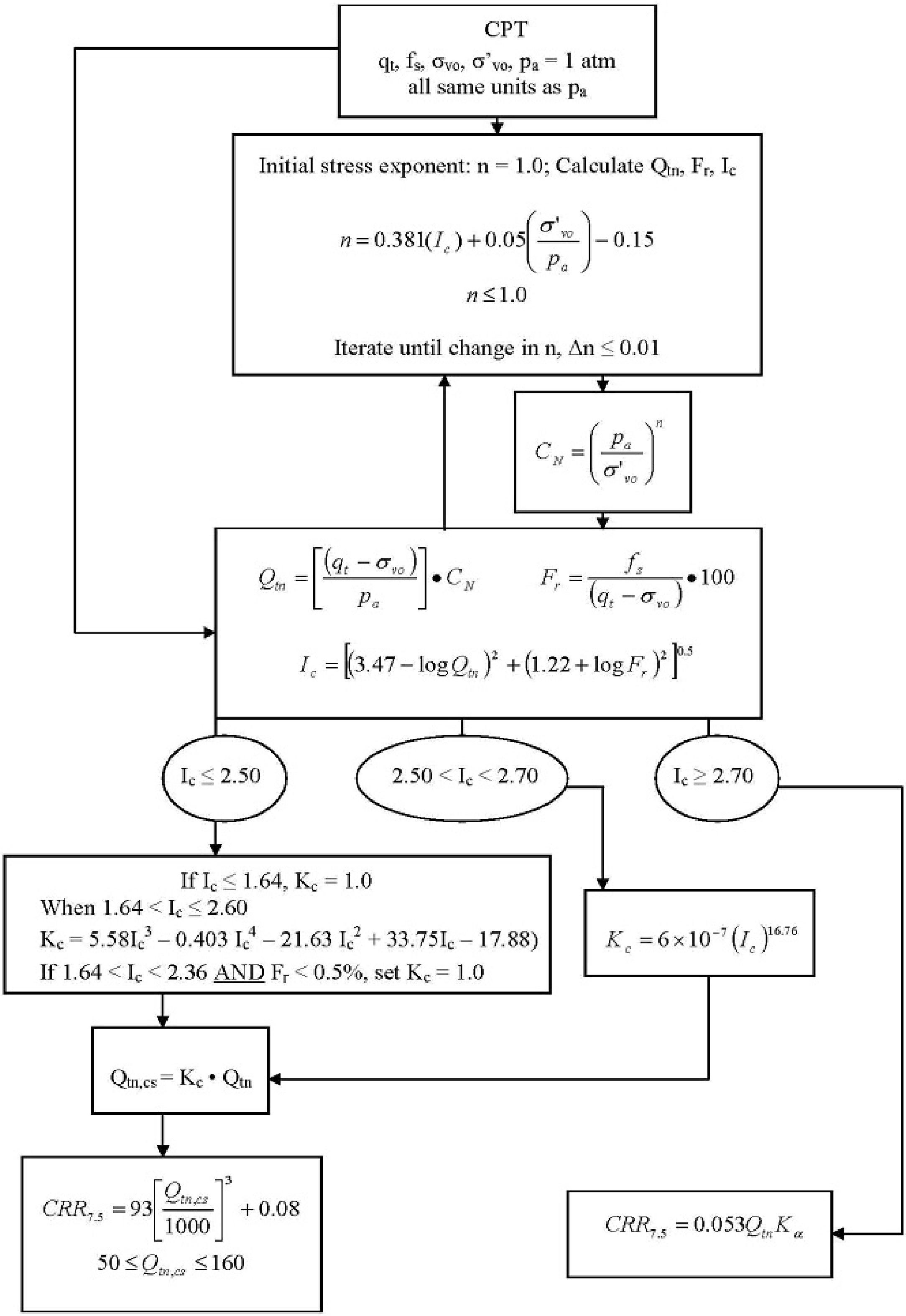
[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



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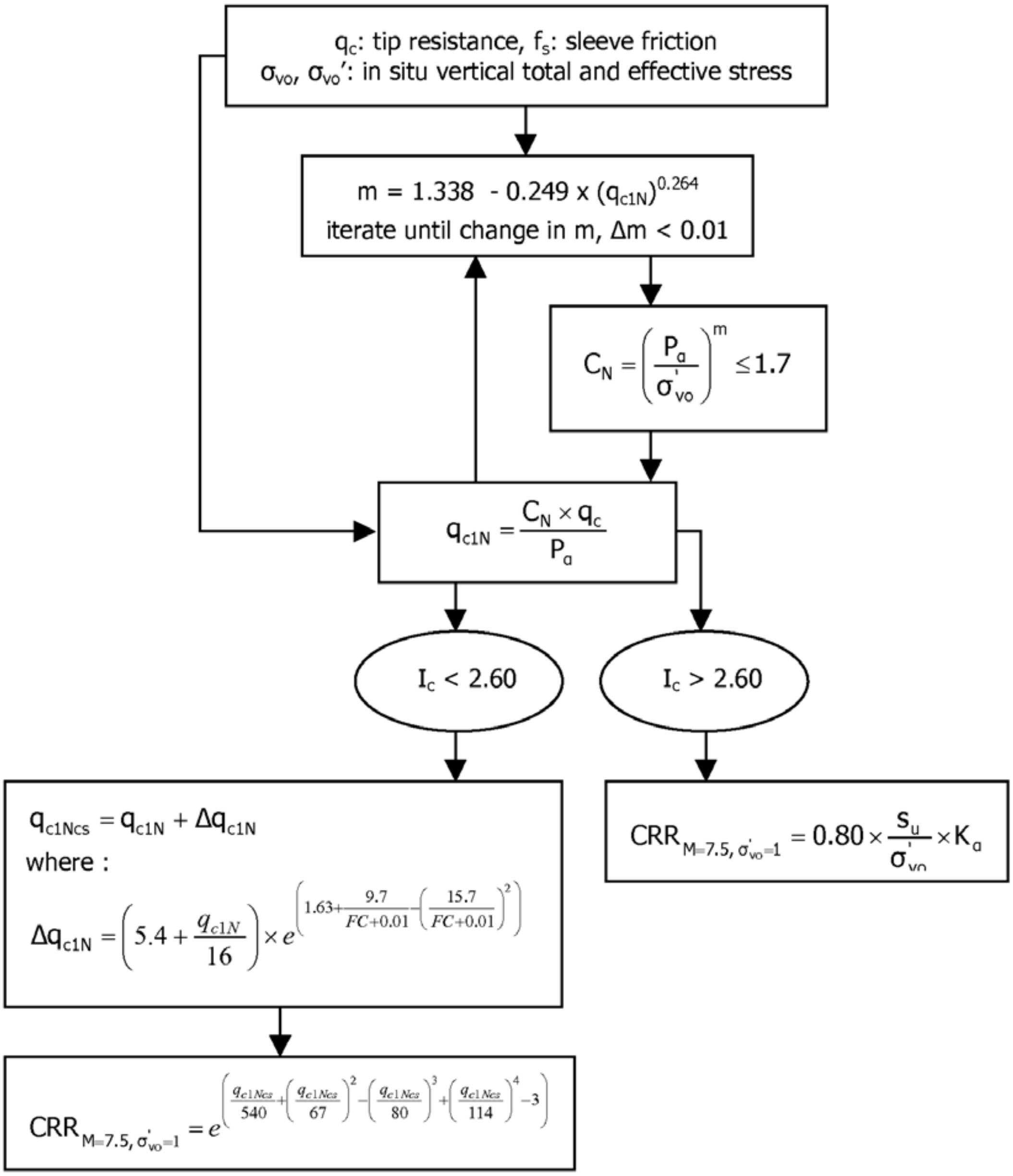
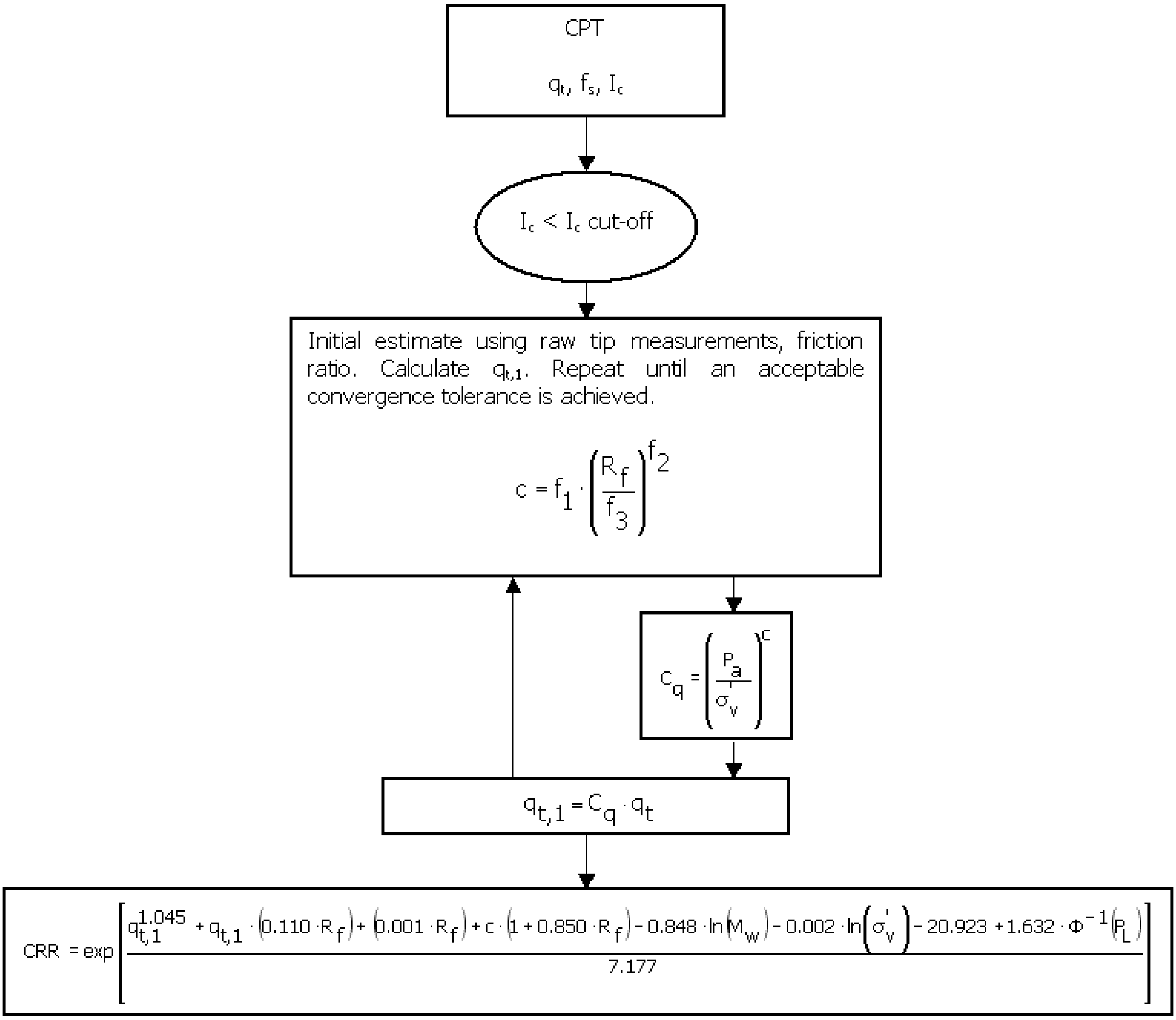
[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

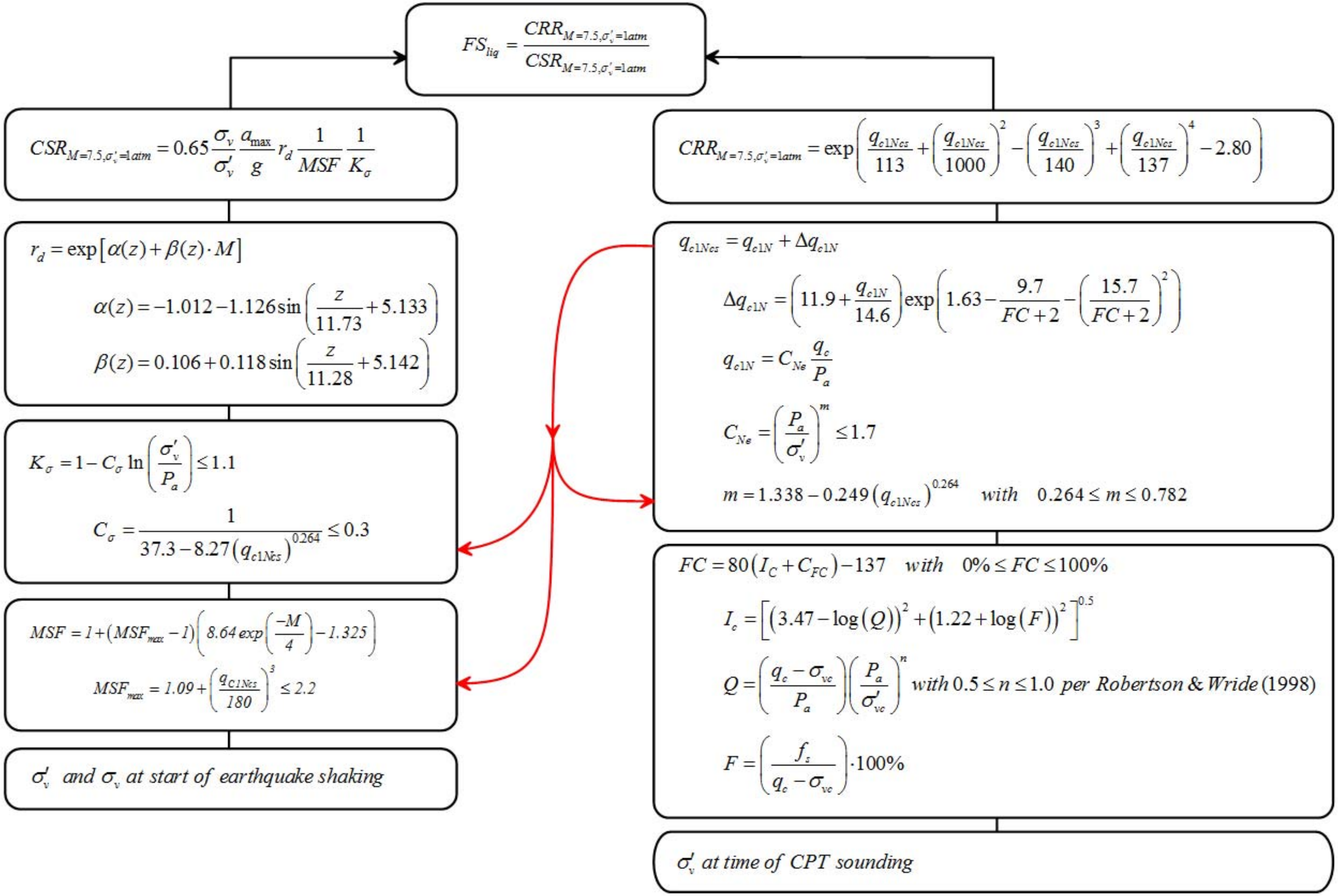
Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)



Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)



$$FS_{liq} = \frac{CRR_{M=7.5,\sigma_v'=1atm}}{CSR_{M=7.5,\sigma_v'=1atm}}$$

$$CSR_{M=7.5,\sigma_v'=1atm} = 0.65\frac{\sigma_v}{\sigma_v'}\frac{a_{\max}}{g}r_d\frac{1}{MSF}\frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z)+\beta(z)\cdot M\right]$$

$$\alpha(z) = -1.012 - 1.126\sin\left(\frac{z}{11.73}+5.133\right)$$

$$\beta(z) = 0.106 + 0.118\sin\left(\frac{z}{11.28}+5.142\right)$$

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma_v'}{P_a}\right) \le 1.1$$

$$C_\sigma = \frac{1}{37.3-8.27\left(q_{c1Ncs}\right)^{0.264}} \le 0.3$$

$$MSF = 1+(MSF_{max}-1)\left(8.64\,exp\left(\frac{-M}{4}\right)-1.325\right)$$

$$MSF_{max} = 1.09+\left(\frac{q_{c1Ncs}}{180}\right)^3 \le 2.2$$

$$\sigma_v' \text{ and } \sigma_v \text{ at start of earthquake shaking}$$

$$CRR_{M=7.5,\sigma_v'=1atm} = \exp\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2.80\right)$$

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9+\frac{q_{c1N}}{14.6}\right)\exp\left(1.63-\frac{9.7}{FC+2}-\left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{N\sigma}\frac{q_c}{P_a}$$

$$C_{N\sigma} = \left(\frac{P_a}{\sigma_v'}\right)^m \le 1.7$$

$$m = 1.338 - 0.249\left(q_{c1Ncs}\right)^{0.264} \quad with \quad 0.264 \le m \le 0.782$$

$$FC = 80\left(I_C + C_{FC}\right) - 137 \quad with \quad 0\% \le FC \le 100\%$$

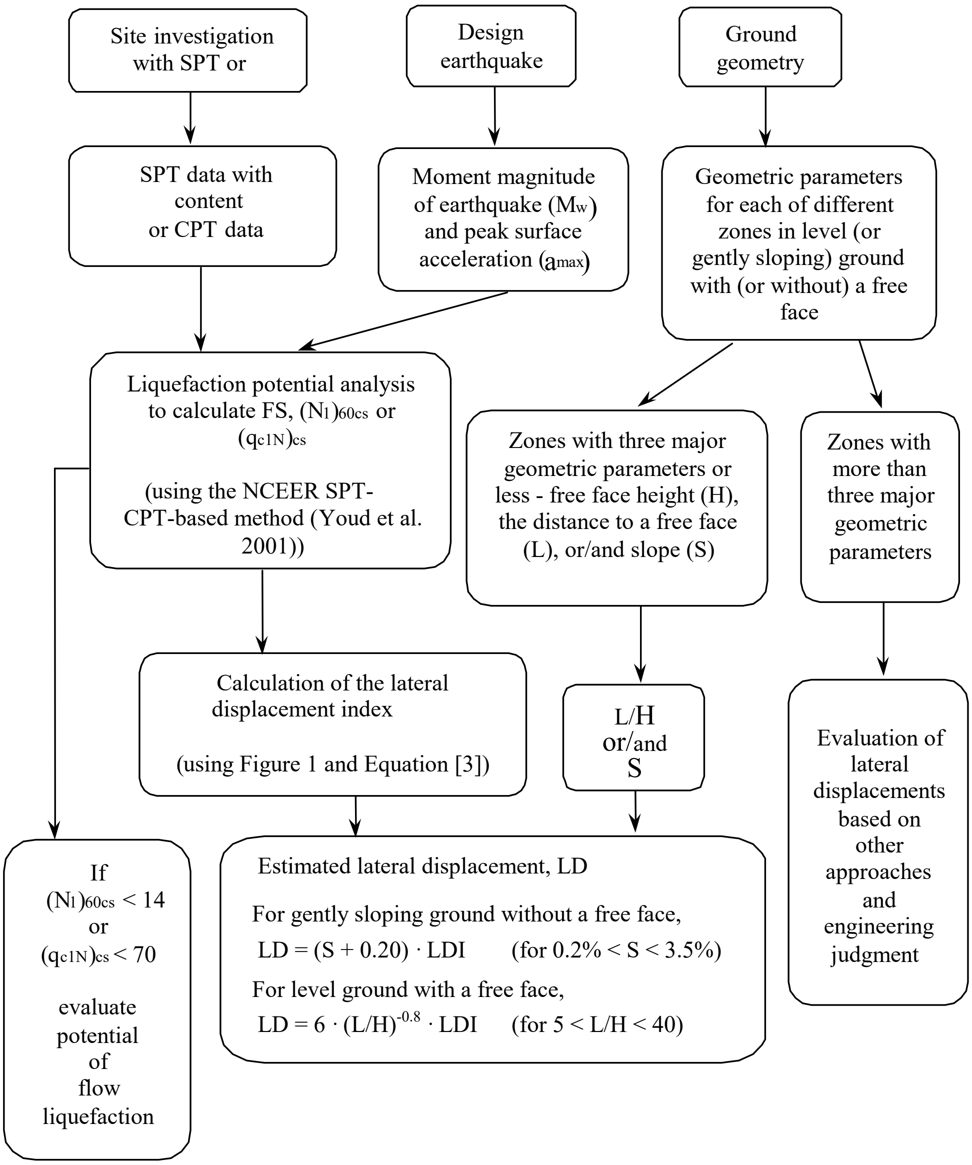
$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c-\sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma_{vc}'}\right)^n \; with\, 0.5 \le n \le 1.0 \; per \; Robertson \,\&\, Wride\,(1998)$$

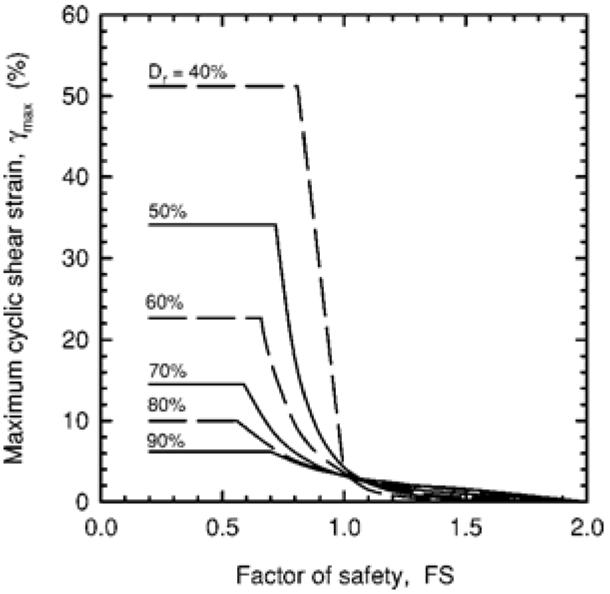
$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

$$\sigma_v' \text{ at time of CPT sounding}$$

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



[1] **Figure 1**

$$LDI=\int_0^{Z_{max}} \gamma_{max} dz$$

[1] **Equation [3]**

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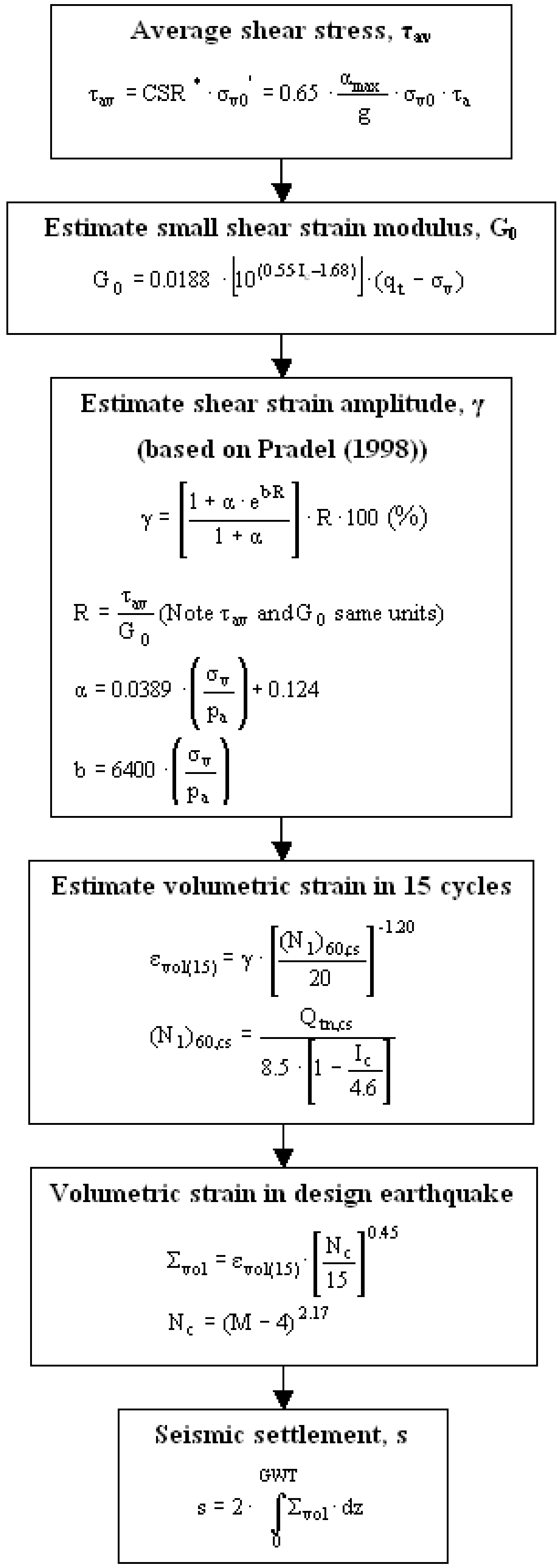
[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands



Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_{v0} \cdot \tau_{a}$$

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

Estimate shear strain amplitude, γ (based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

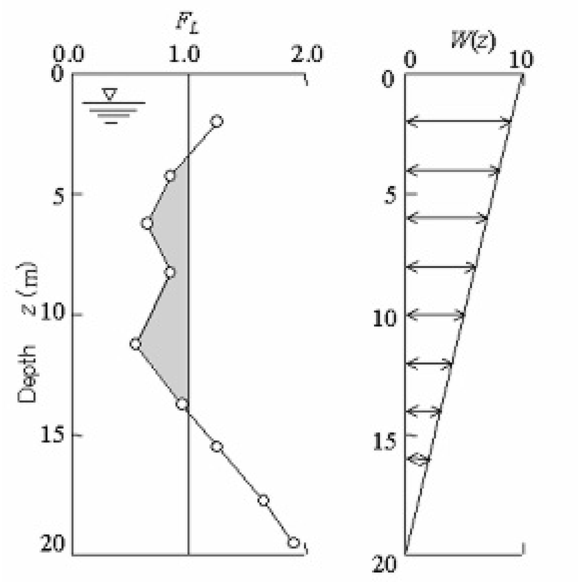
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_{0}^{20} (10 - 0{,}5_{z}) \times F_{I} \times d_{z}$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

Shear-Induced Building Settlement (Ds) calculation procedure

The shear-induced building settlement (Ds) due to liquefaction below the building can be estimated using the relationship developed by Bray and Macedo (2017):

$$Ln(Ds) = c1 + c2 * LBS + 0.58 * Ln\left(Tanh\left(\frac{HL}{6}\right)\right) +$$

$$4.59 * Ln(Q) - 0.42 * Ln(Q)^2 - 0.02 * B +$$
$$0.84 * Ln(CAVdp) + 0.41 * Ln(Sa1) + \varepsilon$$

where Ds is in the units of mm, c1= -8.35 and c2= 0.072 for LBS ≤ 16, and c1= -7.48 and c2= 0.014 otherwise. Q is the building contact pressure in units of kPa, HL is the cumulative thickness of the liquefiable layers in the units of m, B is the building width in the units of m, CAVdp is a standardized version of the cumulative absolute velocity in the units of g-s, Sa1 is 5%-damped pseudo-acceleration response spectral value at a period of 1 s in the units of g, and ε is a normal random variable with zero mean and 0.50 standard deviation in Ln units. The liquefaction-induced building settlement index (LBS) is:

$$LBS = \sum W * \frac{\varepsilon_{shear}}{z} dz$$

where z (m) is the depth measured from the ground surface > 0, W is a foundation-weighting factor wherein W = 0.0 for z less than Df, which is the embedment depth of the foundation, and W = 1.0 otherwise. The shear strain parameter (ε_shear) is the liquefaction-induced free-field shear strain (in %) estimated using Zhang et al. (2004). It is calculated based on the estimated Dr of the liquefied soil layer and the calculated safety factor against liquefaction triggering (FSL).

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APPENDIX E

APPENDIX E

CORROSION REPORT

Soil Corrosivity Evaluation Report for 16300 Euclid St, Fountain Valley, CA

March 13, 2025

Prepared for:
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Project X Job #: S250205A
Client Job or PO #: W2047-88-01

Contents

1 Executive Summary

A corrosion evaluation of the soils at 16300 Euclid St, Fountain Valley, CA was performed to provide corrosion control recommendations for general construction materials. The site is located at 16300 Euclid Street Fountain Valley, CA 92708. Twenty Four (24) samples were tested to a depth of 4.0 ft. Site ground water and topography information was provided by Geocon West Inc. Groundwater depth was determined to be GW 6 to 7 feet feet below finished grade.

Every material has its weakness. Aluminum alloys, galvanized/zinc coatings, and copper alloys do not survive well in very alkaline or very acidic pH environments. Copper and brasses do not survive well in high nitrate or ammonia environments. Steels and irons do not survive well in low soil resistivity and high chloride environments. High chloride environments can even overcome and attack steel encased in normally protective concrete. Concrete does not survive well in high sulfate environments. And nothing survives well in high sulfide and low redox potential environments with corrosive bacteria. This is why Project X tests for these 8 factors to determine a soil's corrosivity towards various construction materials. **Depending solely on soil resistivity or Caltrans corrosion guidelines (which concentrate on concrete/steel highways), will over-simplify descriptions as corrosive or non-corrosive. This approach will not detect these other factors attacking other metals because it is possible to have bad levels of corrosive ions and still have greater than 1,100 ohm-cm soil resistivity. We have observed this fact on thousands of soil samples tested in our laboratory.**

It should not be forgotten that import soil should also be tested for all factors to avoid making your site more corrosive than it was to begin with.

The recommendations outlined herein are not a substitute for any design documents previously prepared for the purpose of construction and apply only to the depth of samples collected.

Soil samples were tested for minimum resistivity, pH, chlorides, sulfates, ammonia, nitrates, sulfides and redox.

As-Received soil resistivities ranged between 938 ohm-cm and 154,100.0 ohm-cm. This data would be similar to a Wenner 4 pin test in the field and used in the design of a cathodic protection or grounding bed system. This resistivity can change seasonally depending on the weather and moisture in the ground. This is why minimum resistivity is more important for categorizing soil corrosivity. An as-received reading alone can be misleading because condensation or minor water leaks will occur underground along pipe surfaces creating a saturated soil environment in the trench on infrastructure surfaces. This is why minimum or saturated soil resistivity measurements are more important than as-received resistivities. This is also mentioned in AWWA C105 Appendix A "*The interpretation of the results of resistivity measurements is extremely important. A determination based on a four-pin reading with dry topsoil averaged with wetter subsoil would probably be inaccurate. Only by determining the resistivity in soil at pipe depth can an accurate interpretation be made. Also, the local situation should be determined concerning groundwater table, the presence of shallow groundwater, and the approximate percentage of time the soil is likely to be water saturated.*

In making field determinations of resistivity, temperature is important. Resistivity increases as the temperature decreases. As the water in the soil approaches freezing, resistivity increases greatly and, therefore, is not reliable. Field determinations under frozen soil conditions should *be avoided. Reliable results under these conditions can be obtained only by the collection of suitable subsoil samples for analysis in laboratory conditions at a proper temperature.*”.

Saturated soil resistivities ranged between 623 ohm-cm to 21,440 ohm-cm. The worst of these values is considered to be corrosive to general metals.

PH levels ranged between 6.6 to 8.5 pH. PH levels were determined to be at levels not detrimental to copper or aluminum alloys. The pH of these samples can allow corrosion of steel and iron in moist environments.

Chlorides ranged between 10 mg/kg to 763 mg/kg. Chloride levels in these samples are low and may cause insignificant corrosion of metals. One sample (B-11 one ft depth) had significantly high chlorides but it also had high sulfates, Sodium, Magnesuim and Calcium. We believe this sample is an outlier contaminated surface.

Sulfates ranged between 32 mg/kg to 952 mg/kg. Sulfate levels in these samples are negligible for corrosion of cement. Any type of cement can be used that does not contain encased metal.

Ammonia ranged between 0.3 mg/kg to 10.3 mg/kg. Nitrates ranged between 1.0 mg/kg to 70.7 mg/kg. Concentrations of these elements were not high enough to cause accelerated corrosion of copper and copper alloys such as brass.

Sulfides presence was determined to be negative. REDOX ranged between + 151 mV to + 195 mV. The probability of corrosive bacteria was determined to be low due to the sulfide and positive REDOX levels determined in these samples.

Import soil should ideally have the following properties to avoid significant corrosion controls:

1. A minimum resistivity greater than 3,000 ohm-cm
2. Sulfates less than 1,000 mg/kg
3. Chlorides less than 300 mg/kg
4. pH between 6.5 and 8.5
5. Ammonia less than 10 mg/kg
6. Nitrates less than 50 mg/kg
7. Sulfides less than 1 mg/kg
8. REDOX potential greater than 100 mV.

2 Corrosion Control Recommendations

The following recommendations are based upon the results of soil testing.

2.1 Cement

The highest reading for sulfates was 952 mg/kg or 0.0952 percent by weight.

Per ACI 318-14, Table 19.3.1.1, sulfate levels in these samples categorized as S0 and are negligible for corrosion of metals and cement. Per ACI 318-14 Table 19.3.2.1 any type of cement not containing steel or other metal can be used.

2.2 Steel Reinforced Cement/ Cement Mortar Lined & Coated (CML&C)

Chlorides in soil can overcome the corrosion inhibiting property of cement for steel, as it can also break through passivated surfaces of aluminum and stainless steels.[1,2] The highest concentration of chlorides was 763 mg/kg.

Chloride levels in these samples are not significantly corrosive to metals not in tension. Standard cement cover may be used in these soils.

Though soils at some locations are significantly corrosive to various metals, per ACI 318-14 Chapter 19 Table 19.3.1.1, all slabs on this site exposure categories and class for **Corrosion Protection of Reinforcement (C) would be considered C1** as Concrete exposed to moisture [mud/rain] (slab sides and bottom) but not to an external source of chlorides. Though there are chlorides in the soil, ACI 318's definition of "external source of chlorides" consists of deicing chemicals, salt, brackish water, seawater, or spray from these sources. The chloride levels in seawater are typically over 19,000 mg/L or 19,000 ppm.

When concrete is tested for water-soluble chloride ion content, the tests should be made at an age of 28 to 42 days. The limits in Per ACI 318-14 Table 5.3.2.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.[3]

2.3 Stainless Steel Pipe/Conduit/Fittings

Stainless steels derive their corrosion resistance from their chromium content and oxide layer which needs oxygen to regenerate if damaged. Thus stainless steel is not good for deep soil applications where oxygen levels are extremely low. Stainless steels should not be installed deeper than a plant root zone. Stainless steels typically have the same nobility as copper on the galvanic series and can be connected to copper. If stainless steel must be used, it must be backfilled with soil having greater than 10,000 ohm-cm resistivity and excellent drainage. 304 Stainless steel will also corrode if in contact with carbon materials such as activated carbon. Stainless steel welds should be pickled.

The soil at this site has low probability for anaerobic corrosive bacteria and low chloride levels. Per Nickel Institute guidelines, 304 or 316 Stainless steels can be used in these soils.

2.4 Steel Post Tensioning Systems

The proper sealing of stressing holes is of utmost importance in PT Systems. Cut off excess strand 1/2" to 3/4" back in the hole. Coat or paint exposed anchorage, grippers, and stub of strands with "Rust-o-leum" or equal. After tendons have been coated, the cement contractor shall dry pack blockouts within ten (10) days. A non-shrink, non-metallic, non-porous moisture-insensitive grout (Master EMACO S 488 or equivalent), or epoxy grout shall be used for this purpose. If an encapsulated post-tension system is used, regular non-shrink grout can be used.

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[1] Design Manual 303: Cement Cylinder Pipe. Ameron. p.65

[2] *Chapter 19, Table 1904.2.2(1), 2012 International Building Code*

[3] ACI 381-14., BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI 318-14) AND COMMENTARY (ACI 318R-14)

Due to the low chloride concentrations measured on samples obtained from this site, post-tensioned slabs should be protected in accordance with soil considered normal (non-corrosive).[4,5] Addition of grease caps to the cut strand at live end anchors can deter construction defect accusations but are not needed.

2.5 Steel Piles

Steel piles are most susceptible to corrosion in disturbed soil where oxygen is available. Further, a dissimilar environment corrosion cell would exist between the steel embedded in cement, such as pile caps and the steel in the soil. In the cell, the steel in the soil is the anode (corroding metal), and the steel in cement is the cathode (protected metal). This cell can be minimized by coating the part of the steel piles that will be embedded in cement to prevent contact with cement and reinforcing steel.

Piles driven into soils without disturbing soils will avoid oxygen introduction and low corrosion rates unless there is a probability for corrosive anaerobic bacteria. Galvanized steel's zinc coating can provide significant protection for driven piles. In corrosive soils in which normal zinc coatings are not enough, the life of piles can be extended by increasing zinc coating thickness, using sacrificial metal, or providing a combination of epoxy coatings and cathodic protection. Corrosion has been observed to be extremely localized even at and below underground water tables. Pit depths of this magnitude do not have an appreciable effect on the strength or useful life of piling structures because the reduction in pile cross section is not significant.[6] Pitting is of more importance to pipes transporting liquids or gases which should not be leaked into the ground.

The following recommendations are recommended to achieve desired life. We defer to structural engineers to use our estimated corrosion rates and to choose from the corrosion control options listed below.

1) Sacrificial metal by use of thicker piles per non-disturbed soil corrosion rates, or
2) Galvanized steel piles per non-disturbed soil corrosion rates, or
3) Combination of galvanized and sacrificial metal per non-disturbed soil corrosion rates, or
4) For no loss of metal, coat entire pile with abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent, or
5) Use high yield steel which will corrode at the same rate as mild steel but have greater yield strength and thus be able to suffer more material loss than mild steel.

2.5.1 Expected Corrosion Rate of Steel and Zinc in disturbed soil

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next,

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[4] *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, PTI DC10.5-12,Table 4.1, pg 16*

[5] *Specification for Unbonded Single Strand Tendons. Post-tensioning Institute (PTI), Phoenix, AZ, 2000.*

[6] Melvin Romanoff, Corrosion of Steel Pilings in Soils, National Bureau of Standards Monograph 58, pg 20.

especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils will also be considered disturbed simply because of their nature from dry to wet seasons.

In Melvin Romanoff's NBS Circular 579, the corrosion rates of carbon steels and various metals was studied over long term periods. Various metals were placed in various soil types to gather corrosion rate data of all metals in all soil types. Samples were collected and material loss measured over the course of 20 years in some sites. The following corrosion rates were estimated by comparing the worst results of soils tested with similar soils in Romanoff's studies and Highway Research Board's publications.[7] The corrosion rate of zinc in disturbed soils is determined per Romanoff studies and King Nomograph.[8]

Expected Corrosion Rate for Steel = 0.91 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.17 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

In undisturbed soils, a corrosion rate of 0.91 mil/year for steel is expected with little change in the corrosion rate of zinc due to it's low nobility in the galvanic series.

Per CTM 643: Years to perforation of corrugated galvanized steel culverts

54.2	Years to Perforate 18 gage (0.052in) metal
70.5	Years to Perforation for a 16 gage metal culvert
86.8	Years to Perforation for a 14 gage metal culvert
119.3	Years to Perforation for a 12 gage metal culvert
151.8	Years to Perforation for a 10 gage metal culvert
184.4	Years to Perforation for a 8 gage metal culvert

2.5.2 Expected Corrosion Rate of Steel and Zinc in Undisturbed soil

Expected Corrosion Rate for Steel = 0.91 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.17 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

2.6 Steel Storage tanks

Underground fuel tanks must be constructed and protected in accordance with California Underground Storage Tank Regulations, CCR, Title 23, Division 3, Chapter 16. Metals should be protected with cathodic protection or isolated from backfill material with an epoxy coating.

2.7 Steel Pipelines

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket

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[7] Field test for Estimating Service Life of Corrugated Metal Culverts, J.L. Beaton, Proc. Highway Research Board, Vol 41, P. 255, 1962

[8] King, R.A. 1977, Corrosion Nomograph, TRRC Supplementary Report, British Corrosion Journal

joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286 to avoid galvanic corrosion cells. **These are especially important for fire risers.**

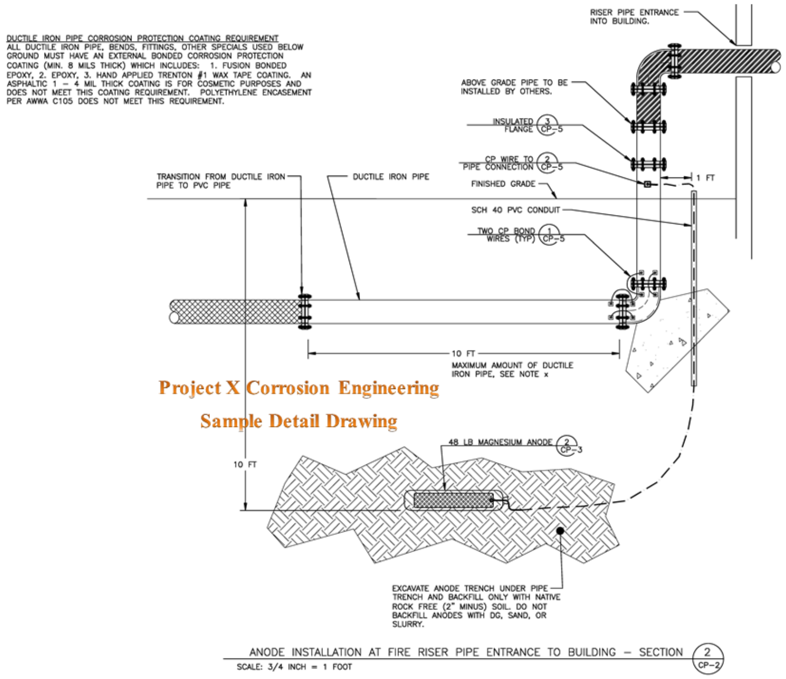

Figure 1- SAMPLE Fire Riser Detail: Install Isolation joint at red arrow

The corrosivity at this site is corrosive to steel. Any piping that must be jack-bored should use abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Wax tape per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213, or
5) For bare steel surfaces, such as welded pipe joints, apply 3 inch thick field coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide. (For CML&C pipes, CML&C factory applied 3/4 inch thick coating is equivalent and needs no extra thickness added.)

It is critical for the life of the pipe that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.8 Steel Fittings

The corrosivity at this site is corrosive to steel. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Tape coating system per AWWA C214, or
3) Wax tape all metallic surfaces per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.9 Ductile Iron (DI) & Cast Iron Fittings

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. It is a tool to help in deciding whether or not to use polyethylene encasement [AWWA C105 Appendix A]. The 10-point system does not, and was never intended to; quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 15 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials and would recommend the use of polyethylene encasement or other coating. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for underground corrosion protection.[9]

The corrosivity at this site is corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Wax tape all metallic surfaces per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213
5) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

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[9] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

2.10 Ductile Iron & Cast Iron Pipe

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 15 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[10]

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection. **If using thermite, perform one test bond using a half-charge then pressure test to confirm excess heat and pinholes were not created.**

Pea gravel is used by plumbers to lay pipes and establish slopes. If the gravel has more than 200 ppm chlorides or is not tested, a 25 mil plastic should be placed between the gravel and pipe to avoid corrosion.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines

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[10] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286**. These are especially important for fire risers.**

The corrosivity at this site is corrosive to iron. Any piping that must be jack-bored should use abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, and install cathodic protection system per NACE SP0169, or
2) Wax tape all metallic surfaces per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213
5) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11 Copper Materials

Copper is an amphoteric material which is susceptible to corrosion at very high and very low pH. It is one of the most noble metals used in construction thus typically making it a cathode when connected to dissimilar metals. Copper’s nobility can change with temperature, similar to the phenomenon in zinc. When zinc is at room temperature, it is less noble than steel and can provide cathodic protection to steel. But when zinc is at a temperature above 140F such as in a water heater, it becomes more noble than the steel and the steel becomes the sacrificial anode. This is why zinc is not used in steel water heaters or boilers. Cold copper has one native potential, but when heated it develops a more electronegative electro-potential aka open circuit potential. Thus hot and cold copper pipes should be electrically isolated from each other to avoid creation of a thermo-galvanic corrosion cell.

2.11.1 Copper Pipes

The lowest pH for this area was measured to be 6.6. Copper is greatly affected by pH, ammonia and nitrate concentrations[11]. The highest nitrate concentration was 70.7 mg/kg and the highest ammonia concentration was 12.6 mg/kg at this site.

These soils were determined mildly corrosive to copper and copper alloys such as brass.

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[11] Corrosion Data Handbook, Table 6, Corrosion Resistance of copper alloys to various environments, 1995

Underground, aboveground, cold water, and hot water pipes should be electrically isolated from each other by use of dielectric unions and plastic in-wall pipe supports per NACE SP0286. The following are corrosion control options for underground copper water pipes.

1) Cover cold copper piping with minimum 8 mil polyethylene and backfill with clean sand with 2 inch minimum cover above and below tubing. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm
2) Heat increases corrosion rates. Hot water pipes should be installed within PVC piping to prevent soil contact, or
3) Cover hot water pipes with minimum 8 mil polyethylene sleeve or incase in double 4-mil thick polyethylene sleeves over a suitable primer

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.2 <u>Brass Fittings</u>

Brass fittings should be electrically isolated from dissimilar metals by use of dielectric unions or isolation joint kits per NACE SP0286.

These soils were determined to be mildly corrosive to copper and copper alloys such as brass.

The following are corrosion control options for underground brass.

1) Cover with minimum 10 mil polyethylene or other impermeable coating and backfill with clean sand with 4 inch minimum cover above and below brass. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm, or
2) Wrap fitting or valves in wax tape

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.3 <u>Bare Copper Grounding Wire</u>

It is assumed that corrosion will occur at all sides of the bare wire, thus the corrosion rate is calculated as a two sided attack determining the time it takes for the corrosion from two sides to meet at the center of the wire. The estimated life of bare copper wire for this site is the following:[12]

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[12] Soil-Corrosion studies 1946 and 1948: Copper Alloys, Lead, and Zinc, Melvin Romanoff, National Bureau of Standards, Research Paper RP2077, 1950

Size (AWG)	**Diameter (mils)**	**Est. Time to penetration (Yrs)**
14	64.1	1068.3
13	72	1200.0
12	80.8	1346.7
11	90.7	1511.7
10	101.9	1698.3
9	114.4	1906.7
8	128.5	2141.7
7	144.3	2405.0
6	162	2700.0
5	181.9	3031.7
4	204.3	3405.0
3	229.4	3823.3
2	257.6	4293.3
1	289.3	4821.7

If the bare copper wire is being used as a grounding wire connected to less noble metals such as galvanized steel or carbon steel, the less noble metals will provide additional cathodic protection to the copper reducing the corrosion rate of the copper.

It is recommended that a corrosion inhibiting and water-repelling coating be applied to aboveground and belowground copper-to-dissimilar metal connections to reduce risk of dissimilar corrosion. This can be wax tape, or other epoxy coating.

Tinned copper wiring or laying copper wire in conductive concrete can protect against chemical attack in soils with high nitrates, ammonia, sulfide and severely low soil electrical resistivity.

2.12 Aluminum Pipe/Conduit/Fittings

Aluminum is an amphoteric material prone to pitting corrosion in environments that are very acidic or very alkaline or high in chlorides.

Conditions at this site are safe for aluminum.

Aluminum derives its corrosion resistance from its oxide layer which needs oxygen to regenerate if damaged, similar to stainless steels. Thus aluminum is not good for deep soil applications. Since aluminum corrodes at very alkaline environments, it cannot be encased or placed against cement or mortar such as brick wall mortar up against an aluminum window frame.

Aluminum is also very low on the galvanic series scale making it most likely to become a sacrificial anode when in contact with dissimilar metals in moist environments. Avoid electrical continuity with dissimilar metals by use of insulators, dielectric unions, or isolation joints per NACE SP0286. Pooling of water at post bottoms or surfaces should be avoided by integrating good drainage.

2.13 Carbon Fiber or Graphite Materials

Carbon fiber or other graphite materials are extremely noble on the galvanic series and should always be electrically isolated from dissimilar metals. They can conduct electricity and will create corrosion cells if placed in contact within a moist environment with any metal.

2.14 Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping from a corrosion viewpoint.

Protect all metallic fittings and pipe restraining joints with wax tape per AWWA C217, cement if previously recommended, or epoxy.

3 CLOSURE

In addition to soils chemistry and resistivity, another contributing influence to the corrosion of buried metallic structures is stray electrical currents. These electrical currents flowing through the earth originate from buried electrical systems, grounding of electrical systems in residences, commercial buildings, and from high voltage overhead power grids. Therefore, it is imperative that the application of protective wraps and/or coatings and electrical isolation joints be properly applied and inspected.

It is the responsibility of the builder and/or contractor to closely monitor the installation of such materials requiring protection in order to assure that the protective wraps or coatings are not damaged.

The recommendations outlined herein are in conformance with current accepted standards of practice that meet or exceed the provisions of the Uniform Building Code (UBC), the International Building Code (IBC), California Building Code (CBC), the American Cement Institute (ACI), Nickel Institute, National Association of Corrosion Engineers (NACE International), Post-Tensioning Institute Guide Specifications and State of California Department of Transportation, Standard Specifications, American Water Works Association (AWWA) and the Ductile Iron Pipe Research Association (DIPRA).

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,

Ed

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Professional Engineer
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EDUARDO HERNANDEZ
No. M37102

4 SOIL ANALYSIS LAB RESULTS

Client: Geocon West Inc.
Job Name: 16300 Euclid St, Fountain Valley, CA
Client Job Number: W2047-88-01
Project X Job Number: S250205A
March 13, 2025

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** **As Rec'd**	**Minimum**	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{2-}	**Phosphate** PO_4^{3-}
	(ft)	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(Ω-cm)**	**(Ω-cm)**		**(mV)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**
B1	1.0	43.3	0.0043	16.5	0.0016	154,100	19,430	8.1	170	0.51	1.2	0.3	ND	21.9	4.0	13.1	37.4	0.7	ND
B1	3.0	70.1	0.0070	32.3	0.0032	11,390	5,293	7.9	176	0.42	15.9	5.6	ND	41.5	6.8	14.6	44.7	1.9	3.9
B3	1.0	31.8	0.0032	16.2	0.0016	18,090	7,370	8.4	170	0.45	22.2	7.1	ND	66.5	9.1	44.3	238.5	3.3	1.5
B3	3.0	102.4	0.0102	60.4	0.0060	1,943	1,809	8.2	189	0.51	27.0	5.8	ND	123.6	12.5	47.7	296.3	3.8	3.7
B9	1.0	97.5	0.0097	58.1	0.0058	16,080	5,025	7.4	181	0.24	2.2	4.3	ND	70.0	13.2	45.3	303.5	1.9	4.5
B9	3.0	65.2	0.0065	29.0	0.0029	48,910	12,730	7.8	176	0.33	5.7	5.8	ND	120.8	8.5	29.0	192.8	2.1	3.7
B2	1.0	78.0	0.0078	39.7	0.0040	18,090	670	8.5	178	0.12	19.6	3.4	ND	65.0	7.6	30.0	93.5	5.7	7.9
B2	4.0	103.2	0.0103	56.4	0.0056	22,110	6,633	8.3	175	ND	3.9	3.9	ND	67.9	6.7	22.2	75.6	4.6	6.3
B4	1.0	98.1	0.0098	54.1	0.0054	4,355	2,479	7.9	188	ND	44.3	10.3	ND	75.5	7.9	22.9	91.4	6.2	4.1
B4	4.0	95.0	0.0095	53.6	0.0054	22,780	6,432	8.2	177	ND	4.9	4.8	ND	77.1	6.2	17.8	80.1	7.0	8.4
B5	1.0	140.4	0.0140	69.0	0.0069	5,896	3,283	8.0	187	ND	32.7	3.5	ND	88.4	6.5	21.5	108.5	8.3	1.6
B5	4.0	77.2	0.0077	45.8	0.0046	6,298	3,886	7.0	175	ND	7.1	4.4	ND	42.9	4.1	16.1	86.7	8.4	8.9
B6	1.0	133.0	0.0133	134.0	0.0134	3,082	1,742	6.7	180	ND	58.8	3.0	ND	104.2	18.4	19.1	113.2	7.1	1.8
B6	4.0	129.5	0.0130	106.4	0.0106	2,747	2,211	6.9	178	ND	5.5	5.5	ND	89.2	7.8	17.5	111.8	9.6	3.1
B7	1.0	197.0	0.0197	83.8	0.0084	2,814	2,144	7.0	181	ND	26.8	3.9	ND	114.9	9.7	17.1	116.1	7.2	0.4
B7	4.0	45.7	0.0046	25.2	0.0025	55,610	8,040	7.4	166	ND	2.1	3.5	ND	34.3	4.8	10.0	71.6	5.5	4.1
B8	1.0	59.8	0.0060	32.0	0.0032	7,370	4,288	7.4	175	ND	70.7	3.4	ND	59.7	5.8	15.6	108.2	5.4	3.7
B8	4.0	68.9	0.0069	36.4	0.0036	127,300	17,420	7.4	151	ND	3.4	6.3	ND	55.7	7.4	10.6	77.0	2.7	2.7
B10	1.0	72.4	0.0072	22.8	0.0023	18,760	5,695	6.8	164	ND	37.9	7.4	ND	52.8	9.8	13.6	100.1	3.9	5.4
B10	4.0	134.8	0.0135	72.3	0.0072	14,740	5,762	6.6	167	ND	12.0	4.6	ND	72.6	3.5	13.3	99.9	7.2	8.6
B11	1.0	951.9	0.0952	762.9	0.0763	938	623	6.6	195	ND	16.6	7.0	ND	393.6	10.7	54.4	334.4	7.3	5.5

	Method	**ASTM D4327**		**ASTM D4327**		**ASTM G187**		**ASTM G51**	**ASTM G200**	**SM 4500-D**	**ASTM D4327**	**ASTM D6919**	**ASTM D6919**	**ASTM D6919**	**ASTM D6919**	**ASTM D6919**	**ASTM D6919**	**ASTM D4327**	**ASTM D4327**
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** **As Rec'd**	**Minimum**	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{2-}	**Phosphate** PO_4^{3-}
	(ft)	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(Ω-cm)**	**(Ω-cm)**		**(mV)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**
B11	4.0	78.1	0.0078	42.1	0.0042	38,860	8,040	7.1	162	ND	1.0	2.9	ND	53.4	1.5	13.4	89.6	4.7	9.5
B12	1.0	39.8	0.0040	25.4	0.0025	23,450	6,700	7.2	171	0.24	29.0	3.0	ND	54.1	5.5	16.7	117.9	7.5	0.9
B12	4.0	32.9	0.0033	10.2	0.0010	134,000	21,440	7.6	166	ND	3.3	2.6	ND	28.5	1.7	10.5	77.7	4.8	7.2

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract
Anions and Cations tested via Ion Chromatograph except Sulfide.

Figure 2- Soil Sample Locations, 16300 Euclid Street Fountain Valley, CA 92708

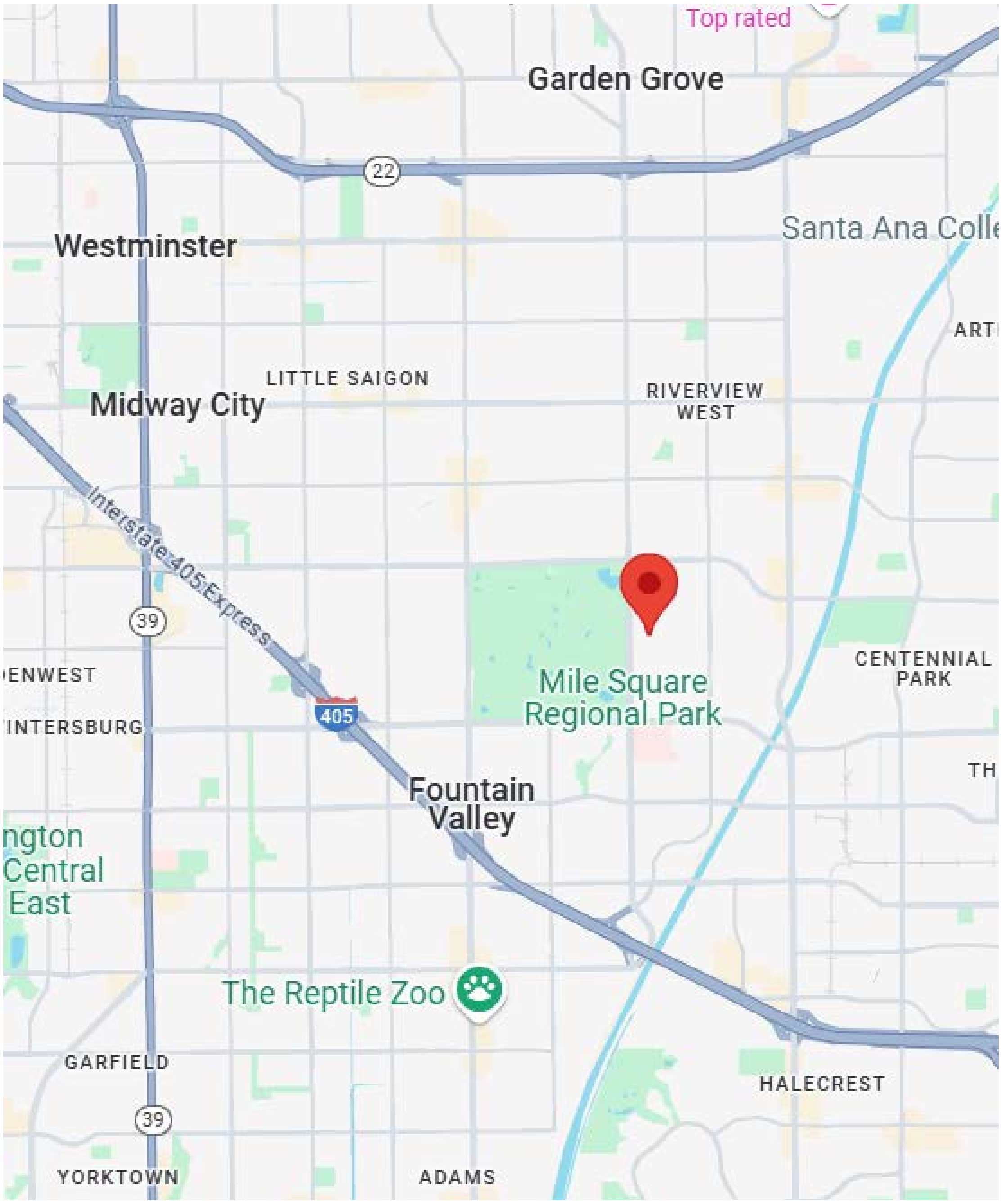


Figure 3- Vicinity Map, 16300 Euclid Street Fountain Valley, CA 92708

5 Corrosion Basics

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils should be considered disturbed simply because of their nature from dry to wet seasons.

5.1 Pourbaix Diagram – In regards to a material’s environment

All metals are unique and have a weakness. Some metals do not like acidic (low pH) environments. Some metals do not like alkaline (high pH) environments. Some metals don’t like either high or low pH environments such as aluminum. These are called amphoteric materials. Some metals become passivated and do not corrode at high pH environments such as steel. These characteristics are documented in Marcel Pourbaix’s book “Atlas of electrochemical equilibria in aqueous solutions”

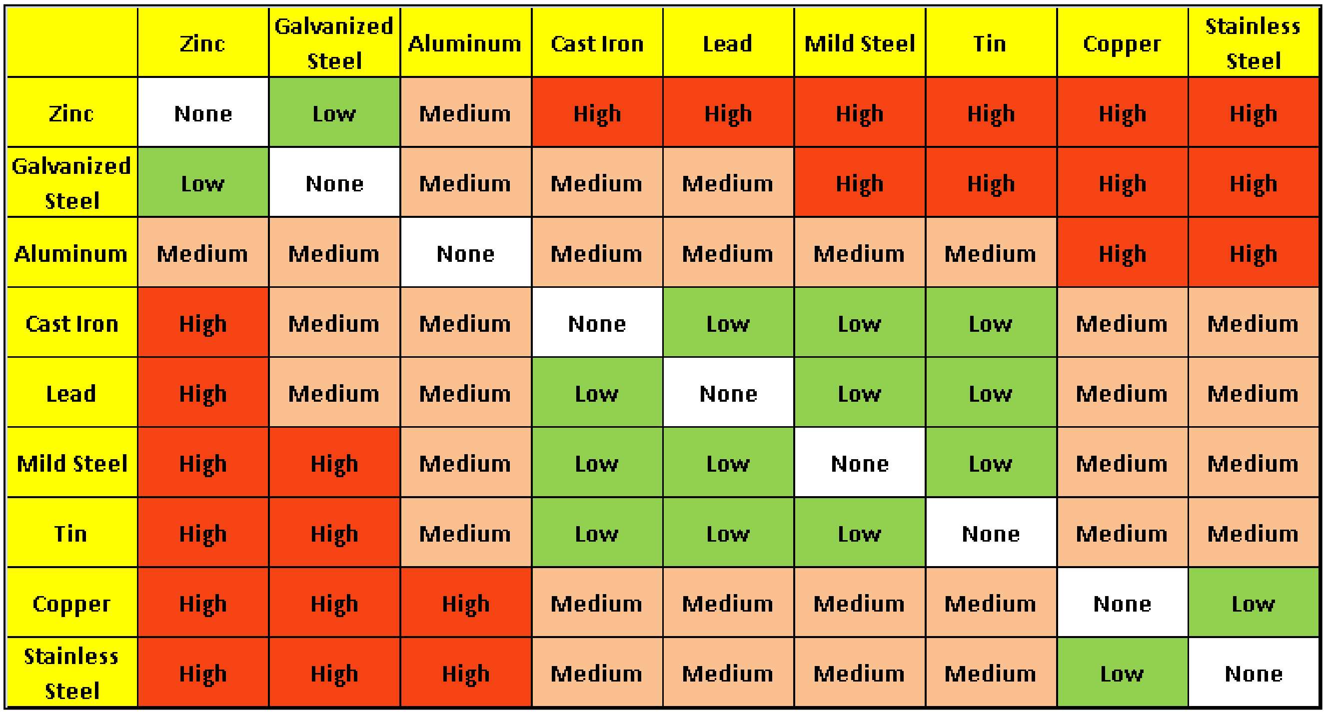
In the mid 1900’s, Marcel Pourbaix developed the Pourbaix diagram which describes a metal’s reaction to an environment dependent on pH and voltage conditions. It describes when a metal remains passive (non-corroding) and in which conditions metals become soluble (corrode). Steels are passive in pH over 12 such as the condition when it is encased in cement. If the cement were to carbonate and its pH reduce to below 12, the cement would no longer be able to act as a corrosion inhibitor and the steel will begin to corrode when moist.

Some metals such as aluminum are amphoteric, meaning that they react with acids and bases. They can corrode in low pH and in high pH conditions. Aluminum alloys are generally passive within a pH of 4 and 8.5 but will corrode outside of those ranges. This is why aluminum cannot be embedded in cement and why brick mortar should not be laid against an aluminum window frame without a protective barrier between them.

5.2 Galvanic Series – In regards to dissimilar metal connections

All metals have a natural electrical potential. This electrical potential is measured using a high impedance voltmeter connected to the metal being tested and with the common lead connected to a copper copper-sulfate reference electrode (CSE) in water or soil. There are many types of reference electrodes. In laboratory measurements, a Standard Hydrogen Electrode (SHE) is commonly used. When different metal alloys are tested they can be ranked into an order from most noble (less corrosion), to least noble (more active corrosion). When a more noble metal is connected to a less noble metal, the less noble metal will become an anode and sacrifice itself through corrosion providing corrosion protection to the more noble metal. This hierarchy is known as the galvanic series named after Luigi Galvani whose experiments with electricity and muscles led Alessandro Volta to discover the reactions between dissimilar metals leading to the early battery. The greater the voltage difference between two metals, the faster the corrosion rate will be.

Table 1- Dissimilar Metal Corrosion Risk



	Zinc	Galvanized Steel	Aluminum	Cast Iron	Lead	Mild Steel	Tin	Copper	Stainless Steel
Zinc	None	Low	Medium	High	High	High	High	High	High
Galvanized Steel	Low	None	Medium	Medium	Medium	High	High	High	High
Aluminum	Medium	Medium	None	Medium	Medium	Medium	Medium	High	High
Cast Iron	High	Medium	Medium	None	Low	Low	Low	Medium	Medium
Lead	High	Medium	Medium	Low	None	Low	Low	Medium	Medium
Mild Steel	High	High	Medium	Low	Low	None	Low	Medium	Medium
Tin	High	High	Medium	Low	Low	Low	None	Medium	Medium
Copper	High	High	High	Medium	Medium	Medium	Medium	None	Low
Stainless Steel	High	High	High	Medium	Medium	Medium	Medium	Low	None

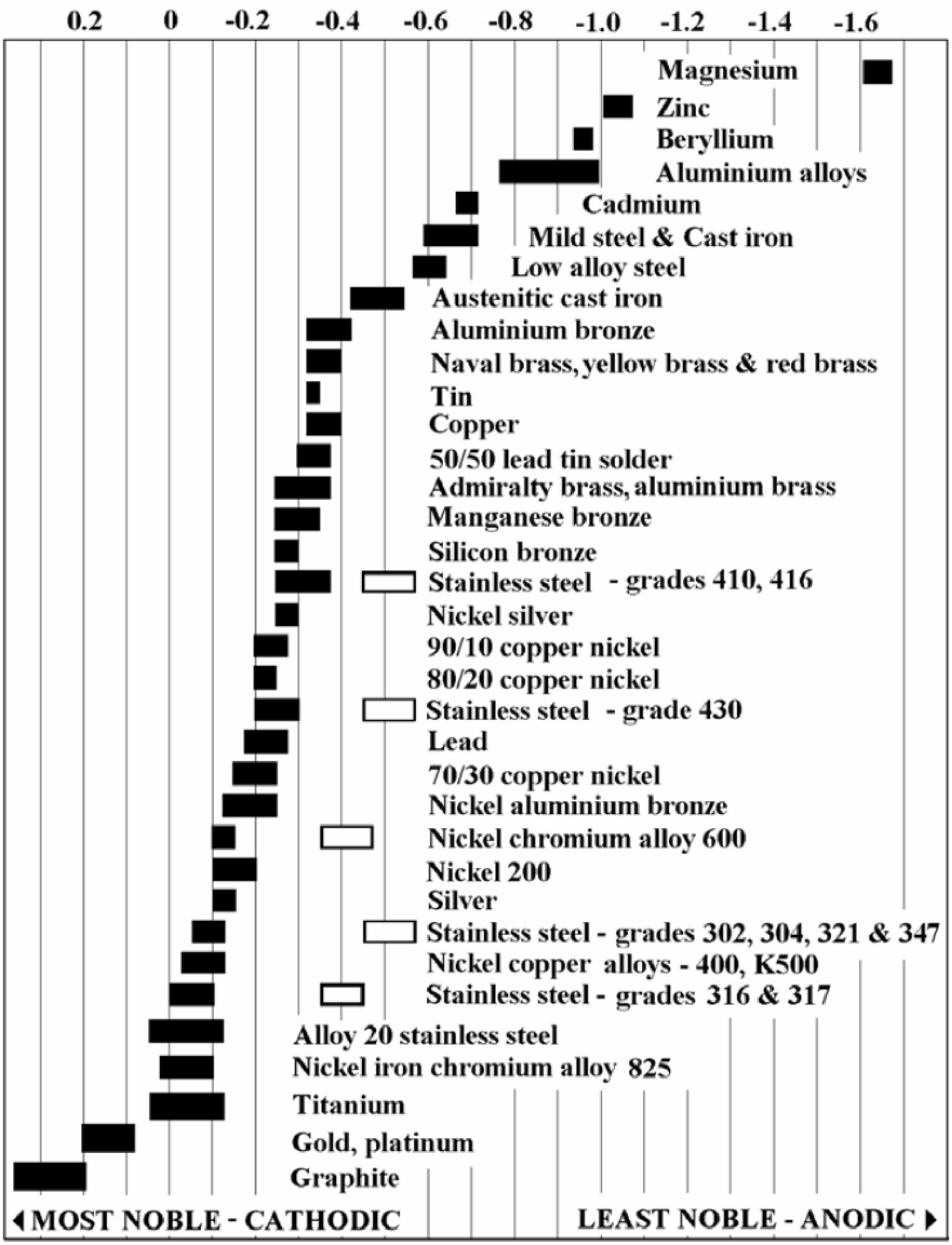
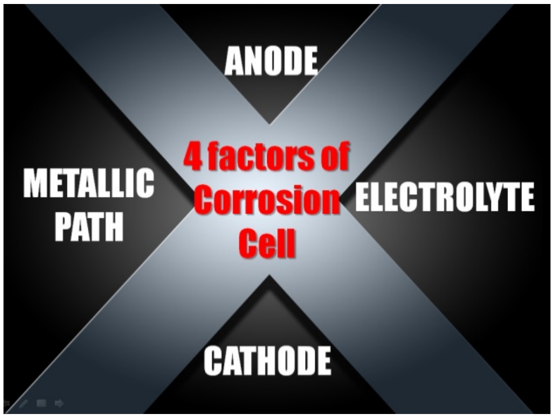


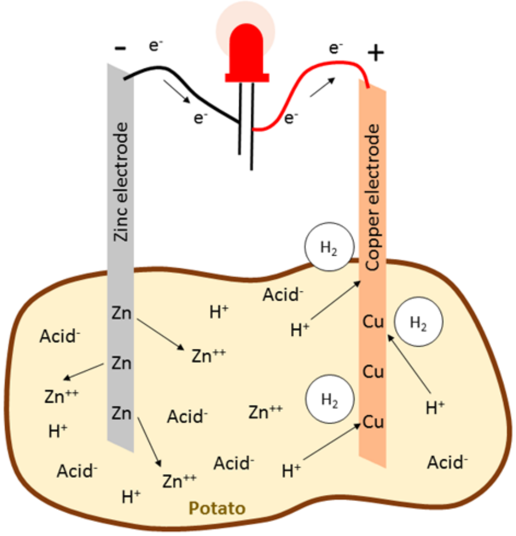
Figure 4 - Galvanic series of metals relative to CSE half cell.

5.3 Corrosion Cell



In order for corrosion to occur, four factors must be present. (1) The anode (2) the cathode (3) the electrolyte and (4) the metallic or conductive path joining the anode and the cathode. If any one of these is removed, corrosion activity will stop. This is how a simple battery produces electricity. An example of a non-metallic yet conductive material is graphite. Graphite is similar in nobility to gold. Do not connect graphite to anything in moist environments.

The anode is where the corrosion occurs, and the cathode is the corrosion free material. Sometimes the anode and cathode are different materials connected by a wire or union. Sometimes the anode and cathode are on the same pipe with one area of the pipe in a low oxygen zone while the other part of the pipe is in a high oxygen zone. A good example of this is a post in the ocean that is repeatedly splashed. Deep underwater, corrosion is minimal, but at the splash zone, the corrosion rate is greatest.



Low oxygen zones and crevices can also harbor corrosive bacteria which in moist environments will lead to corrosion. This is why pipes are laid on backfill instead of directly on native cut soil in a trench. Filling a trench slightly with backfill before installing pipe then finishing the backfill creates a uniform environment around the entire surface of the pipe.

The electrolyte is generally water, seawater, or moist soil which allows for the transfer of ions and electrical current. Pure water itself is not very conductive. It is when salts and minerals dissolve into pure water that it becomes a good conductor of electricity and chemical reactions. Metal ores are turned into metal alloys which we use in construction. They naturally want to return to their natural metal ore state but it requires energy to return to it. The corrosion cell, creates the energy needed to return a metal to its natural ore state.

The metallic or conductive path can be a wire or coupling. Examples are steel threaded into a copper joint, or an electrician grounding equipment to steel pipes inadvertently connecting electrical grid copper grounding systems to steel or iron underground pipes.

The ratio of surface area between the anode and the cathode is very important. If the anode is very large, and the cathode is very small, then the corrosion rate will be very small and the anode may live a long life. An example of this is when short copper laterals were connected to a large and long steel pipeline. The steel had plenty of surface area to spread the copper's attack, thus corrosion was not noticeable. But if the copper was the large pipe and the steel the short laterals, the steel would corrode at an amazing rate.

5.4 Design Considerations to Avoid Corrosion

The following recommendations are based upon typical observations and conclusions made by forensic engineers in construction defect lawsuits and NACE International (Corrosion Society) recommendations.

5.4.1 Testing Soil Factors (Resistivity, pH, REDOX, SO, CL, NO3, NH3)

As previously mentioned, different factors can cause corrosion. The most useful and common test for categorizing a soil's corrosivity has been the measure of soil resistivity which is typically measured in units of (ohm-cm) by corrosion engineers and geologists. Soil resistivity is the ability of soil to conduct or resist electrical currents and ion transfer. The lower the soil resistivity, the more conductive and corrosive it is. The following are "generally" accepted categories but keep in mind, the question is not "Is my soil corrosive?", the question should be, "What is my soil corrosive to?" and to answer that question, soil resistivity and chemistry must be tested. Though **soil resistivity is a good corrosivity indicator for steel materials, high chlorides or other corrosive elements do not always lower soil resistivity, thus if you don't test for chlorides and other water soluble salts, you can get an unpleasant surprise.** The largest contributing factor to a soil's electrical resistivity is its clay, mineral, metal, or sand make-up.

Table 2 - Corrosion Basics- An Introduction, NACE, 1984, pg 191

(Ohm-cm)	Corrosivity Description
0-500	Very Corrosive
500-1,000	Corrosive
1,000-2,000	Moderately Corrosive
2,000-10,000	Mildly Corrosive
Above 10,000	Progressively less corrosive

Testing a soil's pH provides information to reference the Pourbaix diagram of specific metals. Some elements such as ammonia and nitrates can create localized alkaline conditions which will greatly affect amphoteric materials such as aluminum and copper alloys.

Excess sulfates can break-down the structural integrity of cement and high concentrations of chlorides can overcome cement's corrosion inhibiting effect on encased ferrous metals and break down protective passivated surface layers on stainless steels and aluminum.

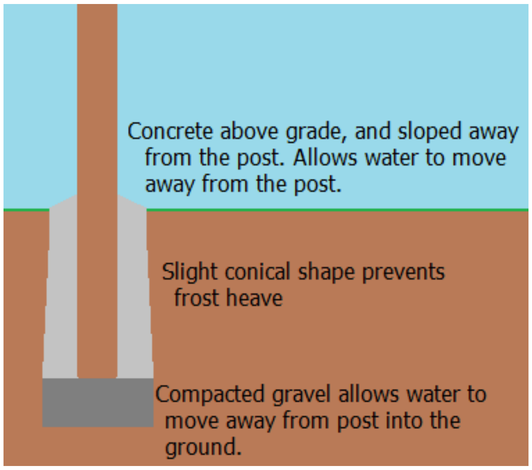
Corrosive bacteria are everywhere but can multiply significantly in anaerobic conditions with plentiful sulfates. The bacteria themselves do not eat the metal but their by-products can form corrosive sulfuric acids. The probability of corrosive bacteria is tested by measuring a soil's oxidation-reduction (REDOX) electro-potential and by testing for the presence of sulfides.

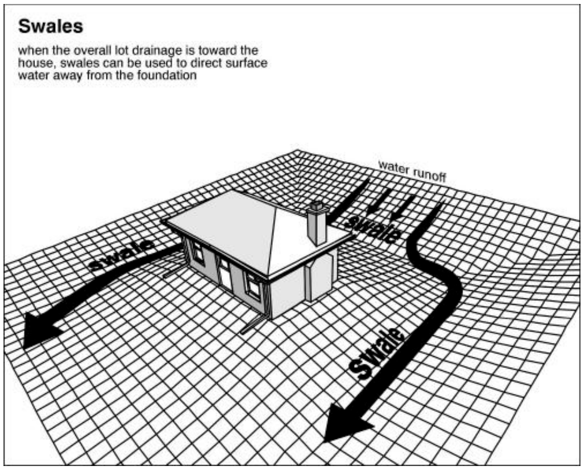
Only by testing a soil's chemistry for minimum resistivity, pH, chlorides, sulfates, sulfides, ammonia, nitrate, and redox potential can one have the information to evaluate the corrosion risk to construction materials such as steel, stainless steel, galvanized steel, iron, copper, brass, aluminum, and concrete.

5.4.2 *Proper Drainage*

It cannot be emphasized enough that pooled stagnant water on metals will eventually lead to corrosion. This stands for internal corrosion and external corrosion situations. In soils, providing good drainage will lower soil moisture content reducing corrosion rates. Attention to properly sealing polyethylene wraps around valves and piping will avoid water intrusion which would allow water to pool against metals. Above ground structures should not have cupped or flat surfaces that will pond water after rain or irrigation events.

Buildings typically are built on pads and have swales when constructed to drain water away from buildings directing it towards an acceptable exit point such as a driveway where it continues draining to a local storm drain. Many homeowners, landscapers and flatwork contractors appear to not be aware of this and destroy swales during remodeling. The majority of garage floor and finished grade elevations are governed by drainage during design. [13,14]





5.4.3 *Avoiding Crevices*

Crevices are excellent locations for oxygen differential induced corrosion cells to begin. Crevices can also harbor corrosive bacteria even in the most chemically treated waters. Crevices will also gather salts. If water's total alkalinity is low, its ability to maintain a stable pH can also become more difficult within a crevice allowing the pH to drop to acidic levels continuing a pitting process. Welds in extremely corrosive environments should be complete and well filleted without sharp edges to avoid crevices. Sharp edges should be avoided to allow uniform coating of protective epoxy. Detection of crevices in welds should be treated immediately. If pressures and loads are low, sanding and rewelding or epoxy patching can be suitable repairs. Damaged coatings can usually be repaired with Direct to Metal paints. **Scratches and crevice corrosion are like infections, they should not be left to fester or the infection will spread making things worse.**

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[13] https://www.fencedaddy.com/blogs/tips-and-tricks/132606467-how-to-repair-a-broken-fence-post

[14] http://southdownstudio.co.uk/problme-drainage-maison.html

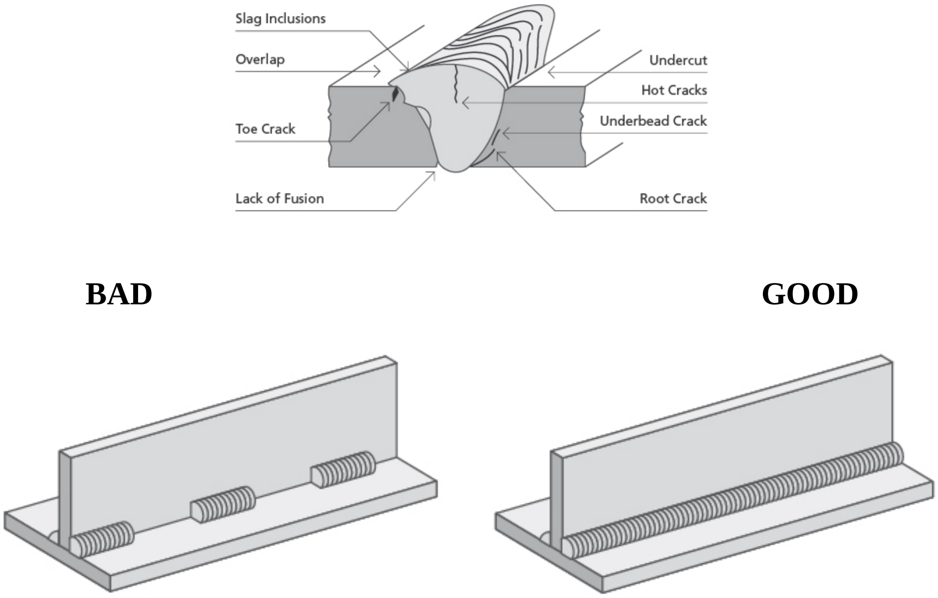

Figure 5- Defects which form weld crevices[15]

5.4.4 *Coatings and Cathodic Protection*

When faced with a corrosive environment, the best defense against corrosion is removing the electrolyte from the corrosion cell by applying coatings to separate the metal from the soil. During construction and installation, there is always some scratch or damage made to a coating. NACE training recommends that coatings be used as a first line of defense and that sacrificial or impressed current cathodic protection is used as a 2nd line of defense to protect the scratched areas. Use of a good coating dramatically reduces the amount of anodes a CP system would need. If CP is not installed as a 2nd line of defense in an extremely corrosive environment, the small scratched zones will suffer accelerated corrosion. CP details such as anode installation instructions must be designed by corrosion engineers or vessel manufacturers on a per project basis because it depends on electrolyte resistivity, surface area of infrastructure to be protected, and system geometry.

There are two types of cathodic protection systems, a Galvanic Anode Cathodic Protection (GACP) system and an Impressed Current Cathodic Protection (ICCP) system. A Galvanic Anode Cathodic Protection (GACP) system is simpler to install and maintain than an Impressed Current Cathodic Protection (ICCP) system. To protect the metals, they must all be electrically continuous to each other. In a GACP system, sacrificial zinc or magnesium anodes are then buried at locations per the CP design and connected by wire to a structure at various points in system. At the connection points, a wire connecting to the structure and the wire from the anode are joined in a Cathodic Protection Test Station hand hole which looks similar in size and shape to an irrigation valve pull box. By coating the underground structures, one can reduce the number of anodes needed to provide cathodic protection by 80% in many instances.

An ICCP system requires a power source, a rectifier, significantly more trenching, and more expensive type anodes. These systems are typically specified when bare metal is requiring protection

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[15] http://www.daroproducts.co.uk/makes-good-weld/

in severely corrosive environments in which galvanic anodes do not provide enough power to polarize infrastructure to -850 mV structure-to-soil potential or be able to create a 100 mV potential shift as required by NACE SP169 to control corrosion. In severely corrosive environments, a GACP system simply may not last a required lifetime due to the high rate of consumption of the sacrificial anodes. ICCP system rectifiers must be inspected and adjusted quarterly or at a minimum bi-annually per NACE recommendations. Different anode installations may be possible but for large sites, anodes are placed evenly throughout the site and all anode wires must be trenched to the rectifier. For a large site, it may be beneficial to use two or more rectifiers to reduce wire lengths or trenching.

To simplify, a GACP system can be installed and practically forgotten with minor trenching because the anodes can be installed very close to the structures. An ICCP system must be inspected annually and anode wires run back to the rectifier which itself connects to the pile system. If any type of trenching or development is expected to occur at the site during the life of the site, it is a good idea to inspect the anode connections once a year to make sure wires are not cut and that the infrastructure is still being provided adequate protection. A common situation that occurs with ICCP systems is that a contractor accidently cuts the wires during construction then reconnects them incorrectly, turning the once cathode, into a sacrificing anode.

Design of a cathodic protection system protecting against soil side corrosion requires that Wenner Four Pin ground resistance measurements per ASTM G57 be performed by corrosion engineers at various locations of the site to determine the best depths and locations for anode installations. Ideally, a sample pile is installed and experiments determining current requirement are conducted. Using this data, the decision is made whether a GACP system is feasible or if an ICCP must be used.

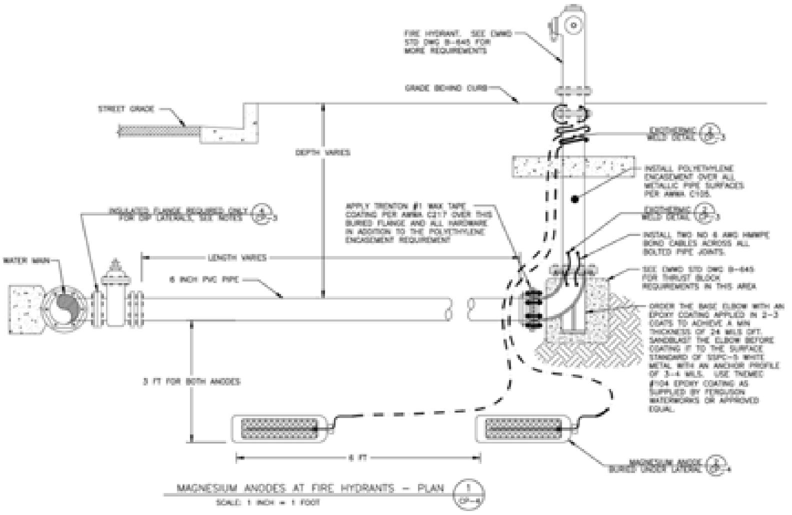


Figure 6- Sample anode design for fire hydrant underground piping

Vessels such as water tanks will have protective interior coatings and anodes to protect the interior surfaces. Anodes can also be buried on site and connected to system skid supports to protect the metal in contact with soil. A good example of a vessel cathodic protection system exists in all home water heaters which contain sacrificial aluminum or magnesium anodes. In environments that exceed 140F, zinc anodes cannot be used with carbon steel because they become the aggressor (Cathodic) to the steel instead of sacrificial (anodic). Anodes in vessels containing extremely brackish water with chloride levels over 2,000 ppm should inspect or change out their anodes every 6 months.

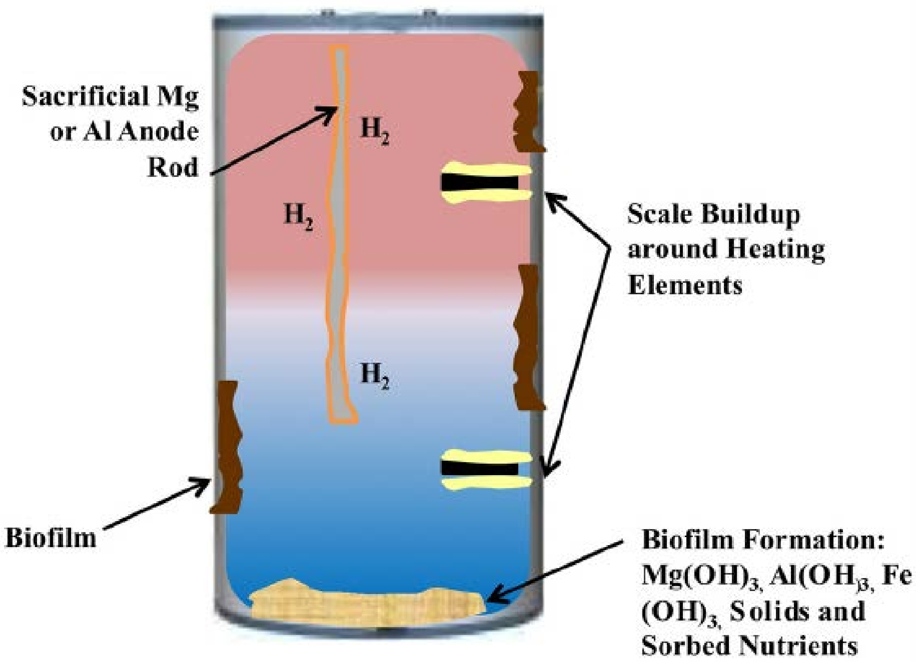


Figure 7- Cross section of boiler with anode

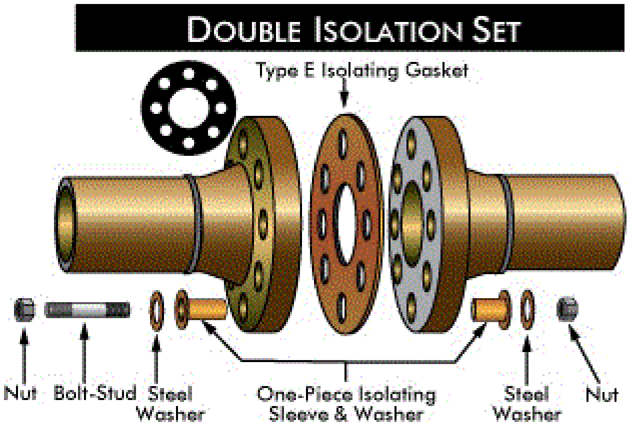
Cathodic protection can only protect a few diameters within a pipeline thus it is not recommended for small diameter pipelines and tubing internal corrosion protection. Anodes are like a lamp shining light in a room. They can only protect along their line of sight.

5.4.5 Good Electrical Continuity

In order for cathodic protection to protect a long pipeline or system of pipes from external soil side corrosion, they must all be electrically continuous to each other so that the electric current from the anode can travel along the pipes, then return through the earth to the anode. Electrical continuity is achieved by welding or pin brazing #8 AWG copper strand bond cable to the end of pipe sticks which have rubber gaskets at bell and spigots. If steel pipes are joined by full weld, bonding wires are not needed.

Electrical continuity between dissimilar metals is not desirable. Isolation joints or di-electric unions should be installed between dissimilar metals, such as steel pipes connecting to a brass valve per NACE SP0286. Bonding wires should then be welded onto the steel pipes by-passing the brass valve so that the cathodic protection system's current can continue to travel along the steel piping but isolate the brass valve from the steel pipeline. Another option would be to provide a separate cathodic protection system for steel pipes on both sides of the brass valve.

Typically, water heater inlets and outlets, gas meters and water meters have dielectric unions installed in them to separate utility property from homeowner property. This also protects them in the case that a home owner somehow electrically connects water pipes or gas pipes to a neighborhood electrical grounding system which can potentially have less noble steel in soil now connected to much more noble copper in soil which will then create a corrosion cell. This is exactly how a lemon powered clock works when a galvanized zinc nail and a steel nail are inserted into a lemon then connected to a clock. The clock is powered by the corrosion cell created.



5.4.6 ***Bad Electrical Continuity***

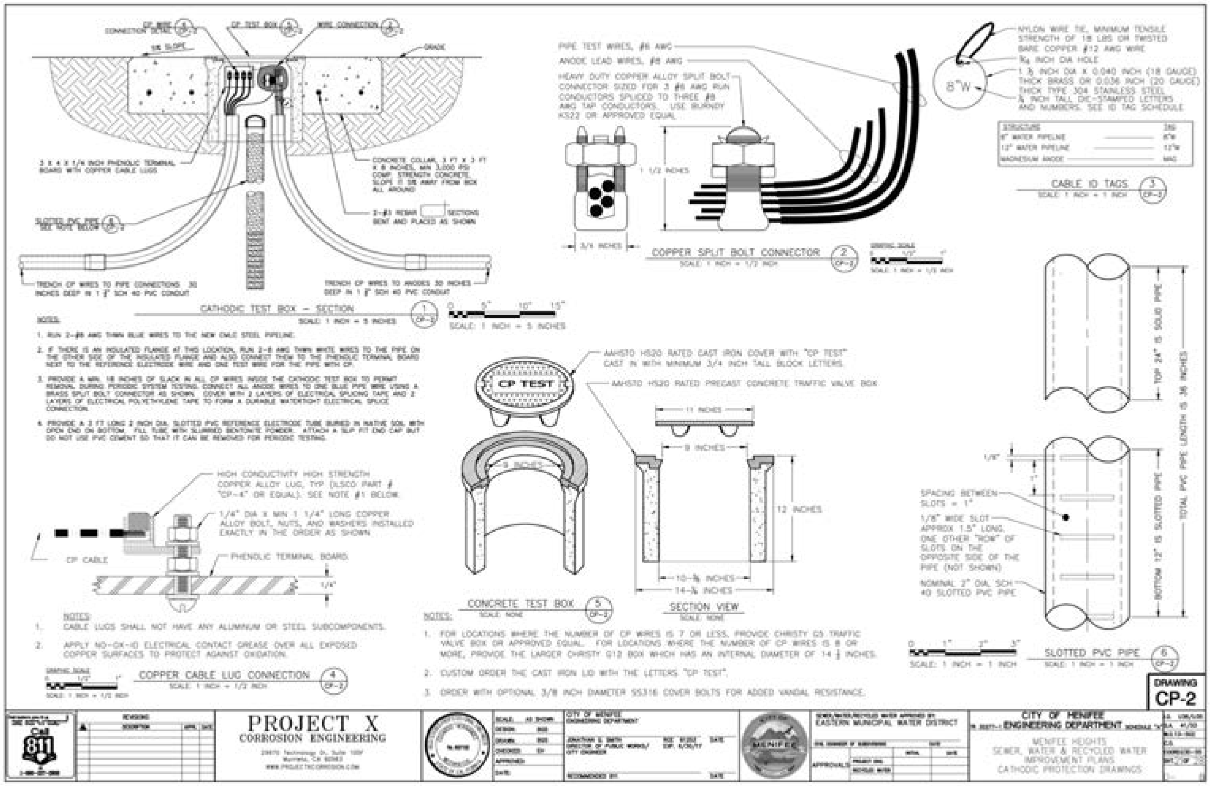
Bad electrical continuity is when two different materials or systems are made electrically continuous (aka shorted) when they were not designed to be electrically continuous. Examples of this would be when gas lines are shorted to water lines or to electrical grounding beds. Very often, fire risers are shorted to electrical grounding systems, and water pipes at business parks. Since fire risers usually have a very short ductile iron pipe in the ground which connects to PVC pipe systems, they tend to experience leaks after 7 to 10 years of being attacked by underground copper systems.

It is absolutely imperative that any copper water piping or other metal conduits penetrating cement slab or footings, not come in contact with the reinforcing steel or post-tensioning tendons to avoid creation of galvanic corrosion cells.

5.4.7 ***Corrosion Test Stations***

Corrosion test stations should be installed every 1,000 feet along pipelines in order to measure corrosion activity in the future. For a simple pipeline, two #8 AWG copper strand bond cable welded or pin brazed onto the pipeline are run up to finished grade and left in a hand hole. Corrosion test stations are used to measure pipe-to-soil electro potential relative to a copper copper-sulfate reference electrode to determine if the pipe is experiencing significant corrosion activity. By measuring test stations along a pipeline, hot spots can be determined, if any. The wires also allow for electrical continuity testing, condition assessment, and a multitude of other types of tests.

At isolation joints and pipe casings, two wires should be welded to either side of the isolation joint for a total of 4 wires to be brought up to the hand hole. This allows for future tests of the isolation joint, casing separation confirmation, and pipe-to-soil potential readings during corrosion surveys.



NOTES:

1. RUN 2-#8 AWG THWN BLUE WIRES TO THE NEW CMLC STEEL PIPELINE.
2. IF THERE IS AN INSULATED FLANGE AT THIS LOCATION, RUN 2-8 AWG THWN WHITE WIRES TO THE PIPE ON THE OTHER SIDE OF THE INSULATED FLANGE AND ALSO CONNECT THEM TO THE PHENOLIC TERMINAL BOARD NEXT TO THE REFERENCE ELECTRODE WIRE AND ONE TEST WIRE FOR THE PIPE WITH CP.
3. PROVIDE A MIN. 18 INCHES OF SLACK IN ALL CP WIRES INSIDE THE CATHODIC TEST BOX TO PERMIT REMOVAL DURING PERIODIC SYSTEM TESTING. CONNECT ALL ANODE WIRES TO ONE BLUE PIPE WIRE USING A BRASS SPLIT BOLT CONNECTOR AS SHOWN. COVER WITH 2 LAYERS OF ELECTRICAL SPLICING TAPE AND 2 LAYERS OF ELECTRICAL POLYETHYLENE TAPE TO FORM A DURABLE WATERTIGHT ELECTRICAL SPLICE CONNECTION.
4. PROVIDE A 3 FT LONG 2 INCH DIA. SLOTTED PVC REFERENCE ELECTRODE TUBE BURIED IN NATIVE SOIL WITH OPEN END ON BOTTOM. FILL TUBE WITH SLURRIED BENTONITE POWDER. ATTACH A SLIP FIT END CAP BUT DO NOT USE PVC CEMENT SO THAT IT CAN BE REMOVED FOR PERIODIC TESTING.

NOTES:

1. CABLE LUGS SHALL NOT HAVE ANY ALUMINUM OR STEEL SUBCOMPONENTS.
2. APPLY NO-OX-ID ELECTRICAL CONTACT GREASE OVER ALL EXPOSED COPPER SURFACES TO PROTECT AGAINST OXIDATION.

NOTES:

1. FOR LOCATIONS WHERE THE NUMBER OF CP WIRES IS 7 OR LESS, PROVIDE CHRISTY G5 TRAFFIC VALVE BOX OR APPROVED EQUAL. FOR LOCATIONS WHERE THE NUMBER OF CP WIRES IS 8 OR MORE, PROVIDE THE LARGER CHRISTY G12 BOX WHICH HAS AN INTERNAL DIAMETER OF 14 1/2 INCHES.
2. CUSTOM ORDER THE CAST IRON LID WITH THE LETTERS "CP TEST".
3. ORDER WITH OPTIONAL 3/8 INCH DIAMETER SS316 COVER BOLTS FOR ADDED VANDAL RESISTANCE.

Figure 8- Sample of corrosion test station specification drawing

5.4.8 Excess Flux in Plumbing

Investigations of internal corrosion of domestic water plumbing systems almost always finds excess flux to be the cause of internal pitting of copper pipes. Some people believe that there is no such thing as too much flux. Flux runs have been observed to travel up to 20 feet with pitting occurring along the flux run. Flushing a soldered plumbing system with hot water for 15 minutes can remove significant amounts of excess flux left in the pipes. If a plumbing system is expected to be stagnant for some time, it should be drained to avoid stagnant water conditions that can lead to pitting and dezincification of yellow brasses.

5.4.9 Landscapers and Irrigation Sprinkler Systems

A significant amount of corrosion of fences is due to landscaper tools scratching fence coatings and irrigation sprinklers spraying these damaged fences. Recycled water typically has a higher salt content than potable drinking water, meaning that it is more corrosive than regular tap water. The same risk from damage and water spray exists for above ground pipe valves and backflow preventers. Fiber glass covers, cages, and cement footings have worked well to keep tools at an arm’s length.

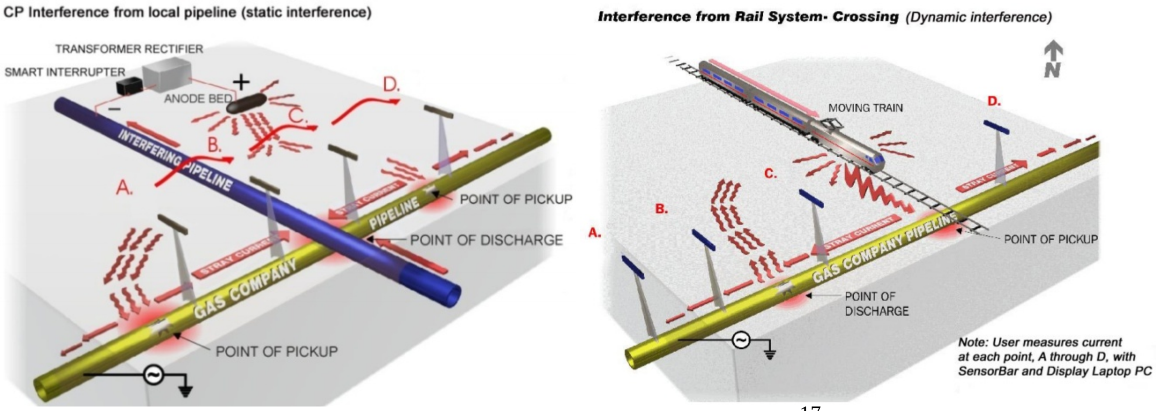
5.4.10 Roof Drainage splash zones

Unbelievably, even the location where your roof drain splashes down can matter. We have seen drainage from a home’s roof valley fall directly down onto a gas meter causing it’s piping to corrode at an accelerated rate reaching 50% wall thickness within 4 years. It is the same effect as a splash zone in the ocean or in a pool which has a lot of oxygen and agitation that can remove material as it corrodes.

5.4.11 Stray Current Sources

Stray currents which cause material loss when jumping off of metals may originate from direct-current distribution lines, substations, or street railway systems, etc., and flow into a pipe system or other steel structure. Alternating currents may occasionally cause corrosion. The corrosion resulting from stray currents (external sources) is similar to that from galvanic cells (which generate their own current) but different remedial measures may be indicated. In the electrolyte and at the metal-electrolyte interfaces, chemical and electrical reactions occur and are the same as those in the galvanic cell; specifically, the corroding metal is again considered to be the anode from which current leaves to flow to the cathode. Soil and water characteristics affect the corrosion rate in the same manner as with galvanic-type corrosion.

However, stray current strengths may be much higher than those produced by galvanic cells and, as a consequence, corrosion may be much more rapid. Another difference between galvanic-type currents and stray currents is that the latter are more likely to operate over long distances since the anode and cathode are more likely to be remotely separated from one another. Seeking the path of least resistance, the stray current from a foreign installation may travel along a pipeline causing severe corrosion where it leaves the line. Knowing when stray currents are present becomes highly important when remedial measures are undertaken since a simple sacrificial anode system is likely to be ineffectual in preventing corrosion under such circumstances.[16] Stray currents can be avoided by installing proper electrical shielding, installation of isolation joints, or installation of sacrificial jump off anodes at crossings near protected structures such as metal gas pipelines or electrical feeders.


CP Interference from local pipeline (static interference)

Interference from Rail System- Crossing *(Dynamic interference)*

Note: User measures current at each point, A through D, with SensorBar and Display Laptop PC

Figure 9- Examples of Stray Current[17]

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[16] http://corrosion-doctors.org/StrayCurrent/Introduction.htm

[17] http://www.eastcomassoc.com/

PG 1

Project X
Corrosion Engineering
Corrosion Control - Soil, Water and Metallurgy Lab

Lab Request Sheet Chain of Custody
Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com
Ship Samples To: 29990 Technology Dr, Suite 13, Murrieta, CA 92563

Project X Job Number S250205A Geocon W2045-88-01 Shopoff 18 Full+Rpt DB

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name: Geocon West Inc.
Contact Name: Andrew Sover
Phone No: 925 701-1040
Mailing Address: 2807 McGaw Avenue, Irvine, CA 92614
Contact Email: sover@geoconinc.com
Accounting Contact:
Invoice Email: sover@geoconinc.com
Client Project No: W2045-88-01
Project Name: Shopoff Euclid and Heil
P.O. #:

	3-5 Day Standard	3 Day Guarantee 50% mark-up	24 Hour RUSH 100% mark-up
(Business Days) Turn Around Time:	●		

For Corrosion Control Recommendations (350g soil sample):
NEED (1) Groundwater depth and
(2) Soil Sample Locations Map
Gw 6 to 7 feet

FOR THERMAL RESISTIVITY PROVIDE (1,500g soil sample):
(1) Optimal Moisture %
(2) Dry Density{PCF}
(3) Desired Compaction
Date & Received By:

METHOD ANALYSIS REQUESTED (Please circle)

		SAMPLE ID - BORE # - Description	DEPTH (ft)	DATE COLLECTED	Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	Sulfide	Ammonia	Nitrate	Flouride	Phosphate	Lithium	Sodium	Potassium	Magnesium	Calcium	Full Corrosion Series	Soil Corrosivity Evaluation Report	Water Corrosivity Mini Report	Moisture Content	Total Alkalinity	Soil Remolding	Thermal Resistivity	(ANC) Alk Neut Cap	(SRB) Sulfate Reducing	Puckorius Index	XRF Elemental Analysis	Water Hardness
					Full Corrosion Series																Reports										
					Geo Quad																										
					Caltrans CTM643	Caltrans CTM643	Caltrans CTM417	Caltrans C1422												350g Sample	*Req: Min. 3 Samples, site map, and groundwater info					IEEE 442 6" 3 lbs/1.5kg	1,500g Sample			ASTM A751	
					AASHTO T2888	AASHTO T289	AASHTO T290	AASHTO T291	SM 2580B		SM 4500-NH3	SM 4500-NO3													ASTM D2216					ASTM E322	
		Default Method			ASTM G187	ASTM G51	ASTM D4327	ASTM D4327	ASTM G200	SM 4500-S2	ASTM D4327	ASTM D4327	SM 2320B	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327				ASTM D2216	SM 2320B	ASTM D698	ASTM 3lbs D5334 8"		HACH BART		ASTM E1621	
413	1	B2	1	2/26/25																●	●										
414	2	B2	4	2/26/25																●											
415	3	B4	1	2/26/25																●											
416	4	B4	4	2/26/25																●											
417	5	B5	1	2/26/25																●											
418	6	B5	4	2/26/25																●											
419	7	B6	1	2/26/25																●											
420	8	B6	4	2/26/25																●											
421	9	B7	1	2/26/25																●											
22	10	B7	4	2/26/25																●											
23	11	B8	1	2/26/25																●											
24	12	B8	4	2/26/25																●											
25	13	B10	1	2/26/25																●											
26	14	B10	4	2/26/25																●											
	15																														

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Project X
Corrosion Engineering
Corrosion Control - Soil, Water and Metallurgy Lab

Lab Request Sheet Chain of Custody
Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com
Ship Samples To: 29990 Technology Dr, Suite 13, Murrieta, CA 92563

Project X Job Number S2502054

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name:	Geocon West Inc.	**Contact Name:**	Andrew Sover	**Phone No:**	925 701-1040
Mailing Address:	2807 McGaw Avenue, Irvine, CA 92614	**Contact Email:**	sover@geoconinc.com		
Accounting Contact:		**Invoice Email:**	sover@geoconinc.com		
Client Project No:	W2045-88-01	**Project Name:**	Shopoff Euclid and Heil		

P.O. #:		**3-5 Day Standard**	**3 Day Guarantee** 50% mark-up	**24 Hour RUSH** 100% mark-up
(Business Days) Turn Around Time:		●		

For Corrosion Control Recommendations **(350g soil sample):**
NEED (1) Groundwater depth and
(2) Soil Sample Locations Map
Gw 6 to 7 feet

FOR THERMAL RESISTIVITY PROVIDE **(1,500g soil sample):**
(1) Optimal Moisture %
(2) Dry Density{PCF}
(3) Desired Compaction
Date & Received By:

METHOD ANALYSIS REQUESTED (Please circle)

		SAMPLE ID - BORE # - Description	DEPTH (ft)	DATE COLLECTED	Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	Sulfide	Ammonia	Nitrate	Flouride	Phosphate	Lithium	Sodium	Potassium	Magnesium	Calcium	Full Corrosion Series	Soil Corrosivity Evaluation Report	Water Corrosivity Mini Report	Moisture Content	Total Alkalinity	Soil Remolding	Thermal Resistivity	(ANC) Alk Neutr Cap	(SRB) Sulfate Reducing	Puckorius Index	XRF Elemental Analysis	Water Hardness
					Caltrans CTM643	Caltrans CTM643	Caltrans CTM417	Caltrans CT422												350g Sample	*Req: Min. 3 Samples, site map, and groundwater info					IEEE 442 6" 3 lbs/1.5kg	1,500g Sample			ASTM A751	
					AASHTO T288	AASHTO T289	AASHTO T290	AASHTO T291	SM 2580B		SM 4500-NH3	SM 4500-NO3													ASTM D2216					ASTM E322	
				Default Method	ASTM G187	ASTM G51	ASTM D4327	ASTM D4327	ASTM G200	SM 4500-S2	ASTM D4327	ASTM D4327	SM 2320B	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327				ASTM D2216	SM 2320B	ASTM D698	ASTM 3lbs D5334 8"		HACH BART		ASTM E1621	
					Full Corrosion Series																Reports										
					Geo Quad																										
427	1	B11	1	2/26/25																●											
428	2	B11	4	2/26/25																●											
429	3	B12	1	2/26/25																●											
430	4	B12	4	2/26/25																●											
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Project X
Corrosion Engineering
Corrosion Control · Soil, Water, and Metallurgy Lab

Lab Request Sheet Chain of Custody
Phone (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com
Ship Samples To: 29990 Technology Dr, Suite 13, Murrieta, CA 92563

Project X Job Number
S250205A GEOCON WEST W2045.88.01 SHOPOFF LFULL + RPT

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name:	Geocon West Inc.	**Contact Name:**	Andrew Sover	**Phone No:**	925 701-1040
Mailing Address:	2807 McGaw Avenue, Irvine, CA 92614	**Contact Email:**	sover@geoconinc.com		
Accounting Contact:		**Invoice Email:**	sover@geoconinc.com		
Client Project No:	W2045-88-01	**Project Name:**	Shopoff Euclid and Heil		

P.O. #:		**3-5 Day Standard**	**3 Day Guarantee** 50% mark-up	**24 Hour RUSH** 100% mark-up
(Business Days) Turn Around Time:		●		

For Corrosion Control Recommendations (350g soil sample):
NEED (1) Groundwater depth and
(2) Soil Sample Locations Map
Gw 6 to 7 feet

FOR THERMAL RESISTIVITY PROVIDE (1,500g soil sample):
(1) Optimal Moisture %
(2) Dry Density{PCF}
(3) Desired Compaction
Date & Received By:

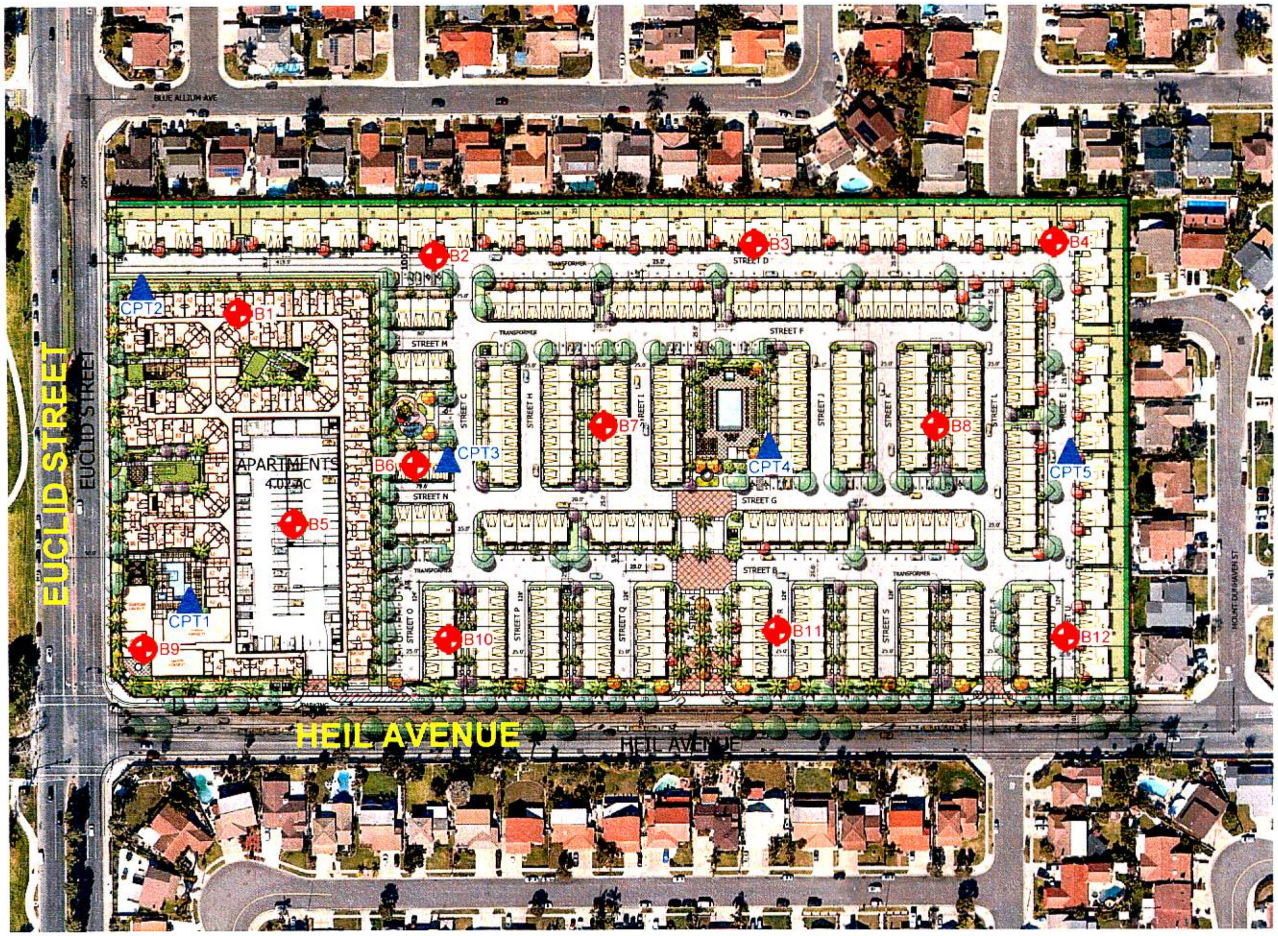
METHOD ANALYSIS REQUESTED (Please circle)

Full Corrosion Series — **Geo Quad** (Soil Resistivity, pH, Sulfate, Chloride) — 350g Sample

Reports — *Req: Min. 3 Samples, site map, and groundwater info

Thermal Resistivity: IEEE 442 6" 3lbs 1.5kg — 1,500g Sample

		SAMPLE ID - BORE # - Description	DEPTH (ft)	DATE COLLECTED	Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	Sulfide	Ammonia	Nitrate	Flouride	Phosphate	Lithium	Sodium	Potassium	Magnesium	Calcium	Full Corrosion Series	Soil Corrosivity Evaluation Report	Water Corrosivity Mini Report	Moisture Content	Total Alkalinity	Soil Remolding	Thermal Resistivity	(ANC) Alk Neutr Cap	(SRB) Sulfate Reducing	Puckorius Index	XRF Elemental Analysis	Water Hardness
				Default Method	ASTM G187	ASTM G51	ASTM D4327	ASTM D4327	ASTM G200	SM 4500-S2	ASTM D4327	ASTM D4327	SM 2320B	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327				ASTM D2216	SM 2320B	ASTM D698	ASTM 3lbs D5334 8"		HACH BART		ASTM E1621	
30	1	B1	1	1/10/25																●	●										
31	2	B1	3	1/10/25																●											
32	3	B3	1	1/10/25																●											
33	4	B3	3	1/10/25																●											
34	5	B9	1	1/10/25																●											
35	6	B9	3	1/10/25																●											
	7																														
	8																														
	9																														
	10																														
	11																														
	12																														
	13	Note: Additonal samples will be sent later. Please prepare one soil report for all 12 borings in the end																													
	14	18 additonal samples to be sent.																													
	15																														



LEGEND

⊕ B12	Boring Locations (Geocon 2025)
▲ CPT5	CPT Locations (Leighton 2012)

GEOCON
W E S T, I N C.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
2807 MCGAW BOULEVARD - IRVINE, CA 92618
PHONE (949) 491-6570 · FAX (949) 299-4550

DRAFTED BY: ACS	CHECKED BY: JTA

SITE PLAN

16300 EUCLID STREET
FOUNTAIN VALLEY, CALIFORNIA

FEB 2025	PROJECT NO. W2045-88-01	FIG 2

GEOCON
WEST, INC.

SOIL LABORATORY

RECEIVING ORDER AND SCHEDULE OF TEST

Project Name: EUCLID AND HEIL
Sample By: ACS
Sample Date: 1/10/25
LAB NUMBER:
Project Number: W2045-88-01
Date To Be Completed:
Engineer/Geologist: JELISA THOMAS ADAMS
Sheet 1 **of** 1
Number of : ____ **Bulk Bags,** ____ **Boxes,** ____ **Cylinders,** ____ **Ring Cans,** ____ **Buckets**

NOTES:

SAMPLE NUMBER	SOIL CLASSIFICATION	SAMPLE TYPE	MOISTURE & DENSITY - RING (M&D)	WAX DENSITY	PLASTIC INDEX (PI)	HYDRO-GRAIN SIZE ANALYSIS	SIEVE ANALYSIS	#200 WASH	SPECIFIC GRAVITY - SOIL	SPECIFIC GRAVITY - AGGREGATE	DIRECT SHEAR (KIPS/SQFT.) NORMAL PRESSURE 1	DIRECT SHEAR (KIPS/SQFT.) NORMAL PRESSURE 3	DIRECT SHEAR (KIPS/SQFT.) NORMAL PRESSURE 5	RESIDUAL SHEAR	EXPANSION INDEX	Consolidation MAX. LOAD(KIPS)	Consolidation UNLOADING(CHECK)	Consolidation SATURATE AT(KIPS)	SATURATED	PH & RESISTIVITY	CHLORIDE	SULFATE	COMPACTION CURVE - MAX DENSITY	SAND EQUIVALENT	ORGANICS	R-VALUE (SAN DIEGO LAB)	ASPHALT - (SAN DIEGO LAB)	REMARKS
B1@1'																									X			SHIPPING VIA
B1@3'																									X			GLS 2/3/25
B3@1'																									X			TRACKING:
B3@3'																									X			562588136
B9@1'																									X			CONFIRMATION:
B9@3'																									X			4424562

****PLEASE INDICATE IN REMARKS SOURCE/SUPPLIER AND LOCATION OF USE****