UPDATE GEOTECHNICAL REPORT

Proposed Eddy Jones Industrial Distribution
260 Eddy Jones Way, Oceanside, CA

RAF Pacifica Group
315 South Coast Highway 101, Suite U-12
Encinitas, CA 92024

NOVA Project No. 2021176
April 8, 2024

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April 8, 2024
NOVA Project No. 2021176

Subject: Update Geotechnical Report
Proposed Eddy Jones Industrial Distribution
260 Eddy Jones Way, Oceanside, California

Dear Mr. Jacob:

NOVA Services, Inc. (NOVA) is pleased to present our update report describing the geotechnical investigation performed for the proposed development at 260 Eddy Jones Way in Oceanside, California. We conducted the geotechnical investigation in general conformance with the scope of work presented in our proposal dated June 1, 2021, and authorized on August 5, 2021.

Revised project design concepts were provided to NOVA in February 2024. This update report provides recommendations to address the currently proposed development. This report also provides updated seismic design criteria per the 2022 California Building Code (CBC), effective January 1, 2023. This site is considered geotechnically suitable for the proposed development provided the recommendations contained in this report are followed.

NOVA appreciates the opportunity to be of service to RAF Pacifica Group. If you have any questions regarding this report, please do not hesitate to call us at 858.292.7575 x 406.

Sincerely,
NOVA Services, Inc.

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1. INTRODUCTION

This report presents the results of the geotechnical investigation NOVA performed for the proposed industrial distribution development located at 260 Eddy Jones Way in Oceanside, California. We understand the project will consist of design and construction of an approximately 566,905-square-foot (sf) concrete tilt-up industrial building. The height of the proposed building will reach 32 feet at the south dock and 36 feet at the remainder of the building. Associated improvements will include a floodwall with a height of about 6 feet, parking bays and drive isles around the site perimeter, and a detention basin for stormwater management. The purpose of our work is to provide conclusions and recommendations regarding the geotechnical aspects of the project. Figure 1-1 presents a site vicinity map. Figure 1-2 presents a site location map.

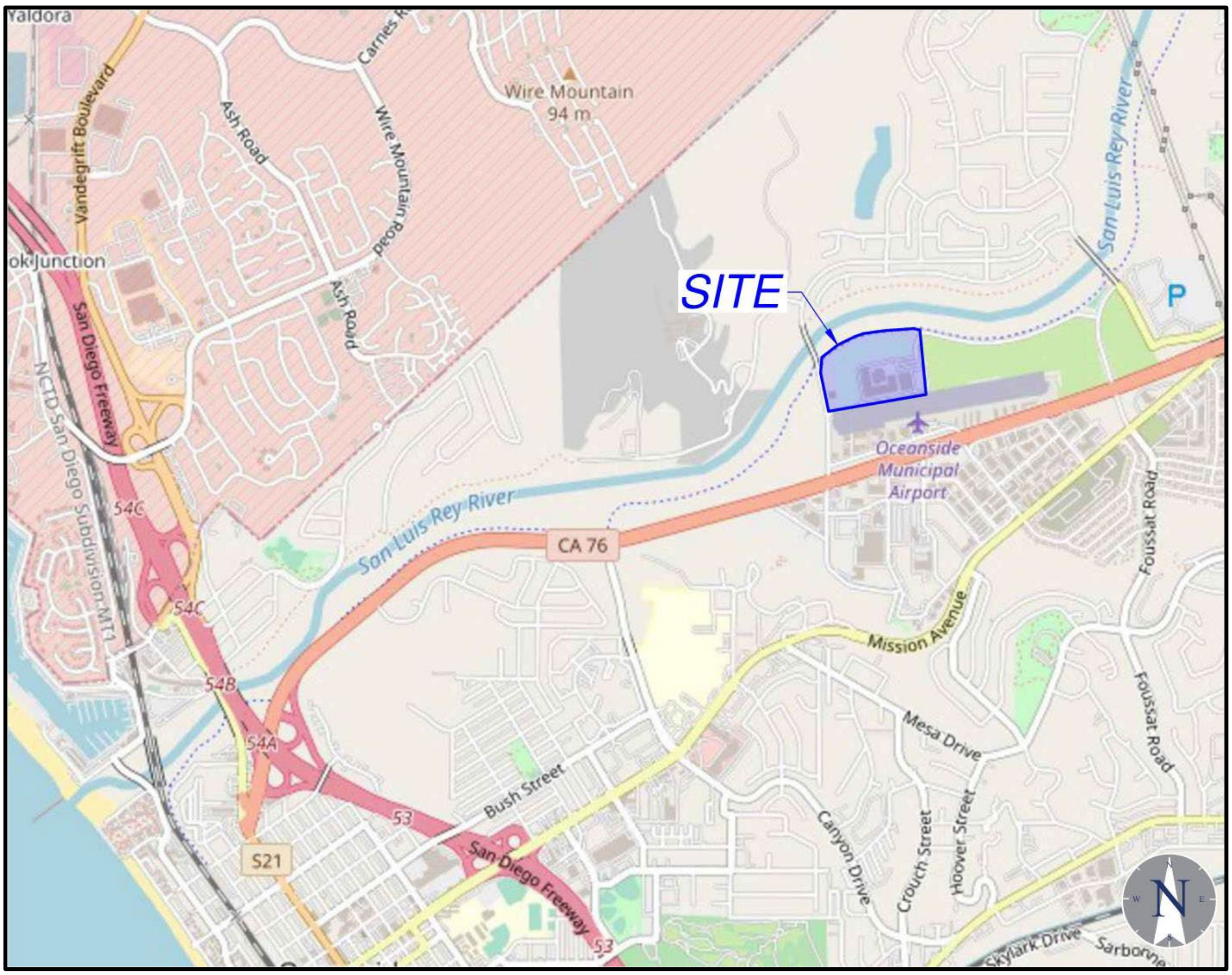

Figure 1-1. Site Vicinity Map

Figure 1-2. Site Location Map

2. SCOPE OF WORK

NOVA's scope of work consisted of reviewing our previous investigation (NOVA, 2021), conducting a brief site reconnaissance, and evaluating the geotechnical aspects of the currently proposed development. No additional subsurface exploration or laboratory testing were performed. After submitting our 2021 geotechnical report, NOVA was requested on site to observe and test the fill placed within a demolished basement as described in Section 3.1 of this report and summarized in our referenced report (NOVA, 2024).

2.1. Previous Field Investigation

NOVA's field investigation consisted of a visual reconnaissance of the site and drilling four geotechnical borings (B-1 through B-4) to depths of about 21½ and 51½ feet below the ground surface (bgs) and two percolation test borings (P-1 and P-2) to depths of about 5 feet bgs using a truck-mounted drill rig equipped with a hollow stem auger. The percolation test borings were drilled within areas of potential BMP locations to evaluate stormwater infiltration feasibility. Additionally, four Cone Penetrometer Test (CPT) soundings were advanced to depths of about 70 and 95 feet bgs to evaluate liquefaction potential. Figure 2-1 presents the approximate locations of the subsurface explorations.

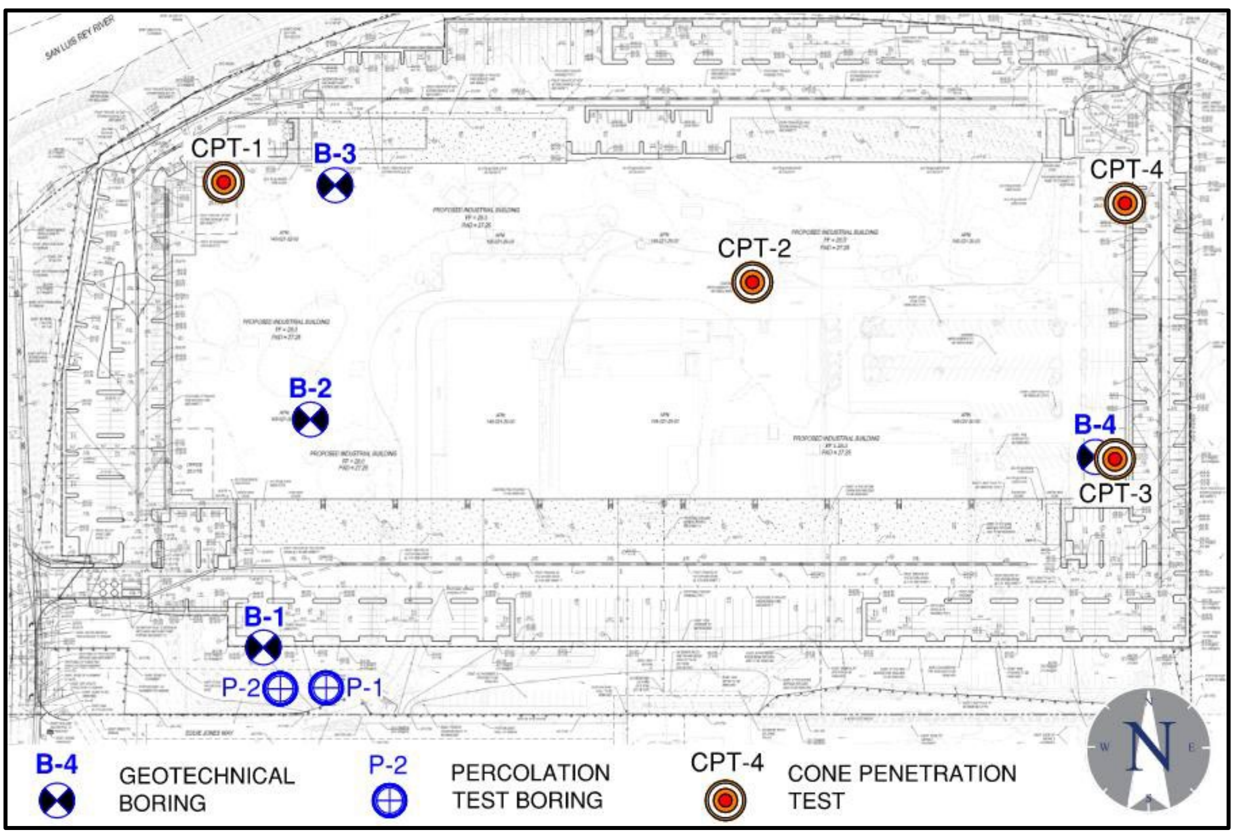

Figure 2-1. Subsurface Exploration Map

A NOVA geologist logged the borings and collected samples of the materials encountered for laboratory testing. Standard Penetration Tests (SPT) were performed in the borings using a 2-inch outer diameter and 1⅜-inch inner diameter split tube sampler. The SPT samplers were driven using an automatic hammer with a calibrated Energy Transfer Ratio (ETR) of 70.6%. The number of blows needed to drive the sampler the final 12 inches of an 18-inch drive is noted on the logs. The field blow counts, N, were corrected to a standard hammer (cathead and rope) with a 60% ETR. The corrected blow counts are noted on the boring logs as N_{60}. Disturbed bulk samples were obtained from the SPT sampler and the drill cuttings. Logs of the borings are presented in Appendix B. Logs of the CPTs are presented in Appendix C. Soils are classified according to the Unified Soil Classification System.

2.2. Previous Laboratory Testing

NOVA tested select samples to evaluate soil classification and engineering properties and develop geotechnical conclusions and recommendations. The laboratory tests consisted of particle-size distribution, Atterberg limits, expansion index, R-value, and corrosivity. The results of the laboratory tests and brief explanations of the test procedures are presented in Appendix D.

2.3. Previous Borehole Percolation Testing

NOVA performed borehole percolation testing in accordance with the test method described in the City of Oceanside Stormwater Standards BMP Design Manual, January 2022 Edition (hereinafter ‘BMP Manual’). The procedure is discussed in Section 8 of this report and Appendix E presents the infiltration test results.

2.4. Analysis and Report Preparation

The results of the field and laboratory testing were evaluated to develop conclusions and recommendations regarding the geotechnical aspects of the proposed construction. This report presents our findings, conclusions, and recommendations.

3. SITE AND PROJECT DESCRIPTION

3.1. Site Description

The proposed development will be located in an approximately 31.7-acre site at 260 Eddy Jones Way corresponding to APNs 145-021-29-00, 145-021-030-00, and 145-021-032-00 in Oceanside, California. The site is bounded by the San Luis Rey River to the north, Oceanside Municipal Airport to the south, Benet Road to the west, and open space to the east. Since NOVA's investigation in 2021, demolition of the vacant buildings has commenced. The site is relatively level, with elevations ranging from about +25 feet NAVD 88 (North American Vertical Datum of 1988) to about +30 feet NAVD 88.

A review of historic aerial photography dating back to 1938, the earliest available historical imagery, indicates that the site was undeveloped until about 1953. Mass grading of the site took place sometime between 1953 and 1964. The southern and western portions of the main building had been in place since at least 1967 and the building to the east was built by 2005. The site had been occupied its previous configuration (prior to the start of demolition in late 2021) since at least 2005. Review of historical topography dating back to 1893 shows that the north and east portions of the site were once occupied by the San Luis Rey River channel until development occurred around 1967, at which point the river was diverted to the north.

During demolition of the existing building, a relatively small basement was discovered. NOVA was requested on site to observe removal of the basement slab and placement and compaction of the basement fill. NOVA's observation and testing services were provided between February 17 and April 11, 2023. The site grading consisted of draining standing water from the bottom of excavation, removing about 1½ feet of saturated material from the excavation bottom, and scarifying the excavation bottom to a depth of about 1 foot. About 2 feet of onsite crushed material was worked into the scarified excavation bottom, manually compacted, and densified until the soils probed firm and unyielding. The excavation bottom elevation was about +23 feet NAVD 88. Once a firm excavation bottom was achieved, about 4 feet of on-site crushed material was moisture conditioned to near optimum moisture content, spread in loose lifts of typically 4 to 8 inches, and compacted to at least 90% relative compaction following ASTM D1557.

3.2. Proposed Construction

Based on review of the updated architectural plans (WM, 2022) and grading plans (PLSA, undated), NOVA understands that the proposed construction will consist of a 566,905-sf concrete tilt-up industrial building. The building height will be 32 feet at the south dock and 36 feet at the remainder of the building. Associated improvements will include a floodwall with a height of about 6 feet, parking bays and drive isles around the site perimeter, and a detention basin for stormwater management. As currently planned, site grading will consist of maximum cut heights of 11½ feet and maximum fill heights of 6 feet. A total of 60,000 cubic yards of cut and 40,000 cubic yards of fill are anticipated, resulting in an estimated export of 20,000 cubic yards for the site.

4. GEOLOGY AND SUBSURFACE CONDITIONS

The site is located within the Peninsular Ranges Geomorphic Province of California, which stretches from the Los Angeles basin to the tip of Baja California in Mexico. This province is characterized as a series of northwest-trending mountain ranges separated by subparallel fault zones, and a coastal plain of subdued landforms. The mountain ranges are underlain primarily by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the Southern California batholith, while the coastal plain is underlain by subsequently deposited marine and nonmarine sedimentary formations. The site is located within the coastal plain portion of the province and is underlain by fill and Quaternary young alluvial flood-plain deposits (map unit Qya). Descriptions of the materials are presented below. Figure 4-1 presents the regional geology in the vicinity of the site. Plate 1 following provides a geotechnical map and geologic cross-sections.

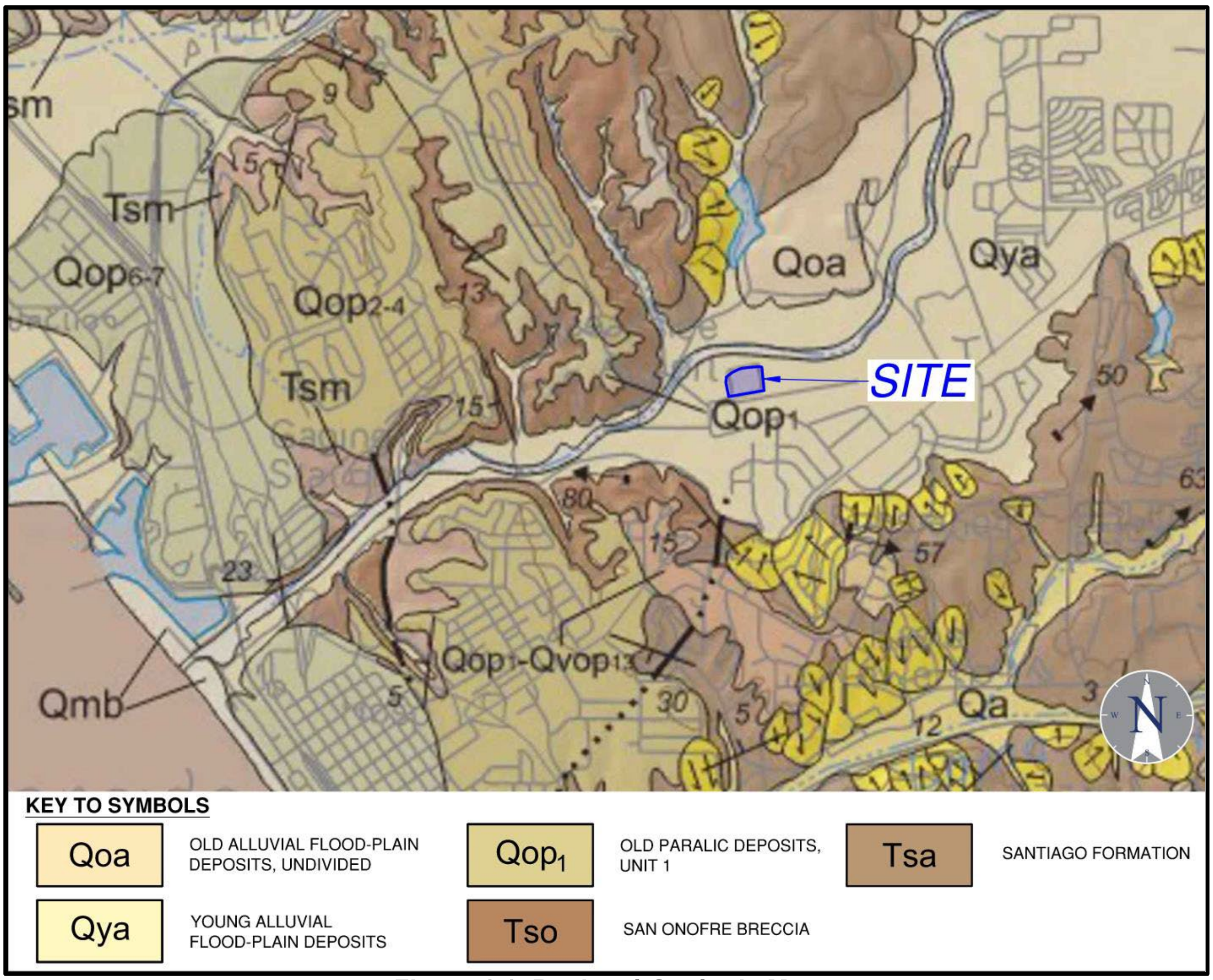

Figure 4-1. Regional Geologic Map
(Source: Kennedy, M.P. and Tan 2007)

Fill (af): As previously mentioned, fill derived from onsite crushed material was placed to backfill the demolished basement. As observed during construction, the fill generally consisted of well graded gravel with silt.

Young Alluvial Flood-Plain Deposits (Qya): Young alluvial flood-plain deposits were encountered to the maximum-explored depth of about 95 feet bgs. The alluvial deposits generally consisted of moist to wet, loose to medium dense, poorly graded sand, poorly graded sand with silt, silty sand, silty, clayey sand and medium stiff to very stiff sandy silt.

Groundwater: Groundwater was encountered at depths between about 7 and 7½ feet bgs, corresponding to elevations between about +17½ and +20 feet NAVD 88 (North American Vertical Datum of 1988). Groundwater levels may fluctuate in the future due to rainfall, irrigation, broken pipes, or changes in site drainage. Groundwater should be anticipated during design and construction of the proposed development.

5. GEOLOGIC HAZARDS

5.1. Faulting and Surface Rupture

California is known to contain active faults that can potentially cause significant damage during earthquakes. The Alquist-Priolo Earthquake Fault Zoning Act was implemented in 1972 to prevent the development over the surface trace of active faults. California Geologic Survey Special Publication 42 was created to provide guidance for following and implementing the law requirements. Special Publication 42 was most recently revised in 2018 (CGS, 2018). The State Geologist defines an "active" fault as one which has had surface rupture within recent geologic time (i.e., Holocene time, <11,700 years b.p.). Earthquake Fault Zones have been delineated to encompass traces of known, Holocene-active faults to address hazards associated with fault surface rupture within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault evaluations be performed so that engineering geologists can identify the locations of active faults and recommend setbacks from locations of possible surface fault rupture. The site is not located within an Earthquake Fault Zone. No faults were identified on the site during the site evaluation; therefore, the possibility of damage due to surface rupture is considered low. The nearest active fault is located about 4¾ miles southwest of the site within the Oceanside section of the Newport-Inglewood-Rose Canyon Fault Zone, which is recognized to have the potential for a Magnitude 6.99 seismic event. Figure 5-1 shows the locations regional faulting in the vicinity of the site.

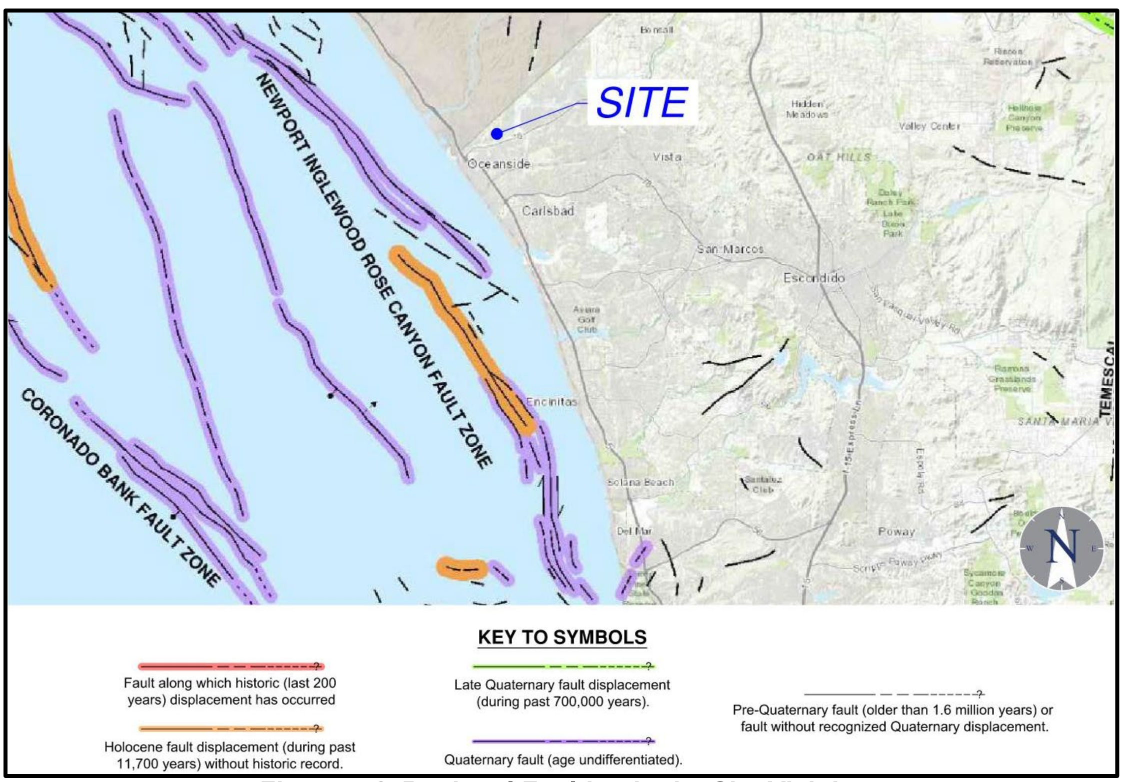

Figure 5-1. Regional Faulting in the Site Vicinity

5.2. Liquefaction and Dynamic Settlement

Liquefaction is a process in which soil grains in a saturated deposit lose contact after the occurrence of earthquakes or other sources of ground shaking. The soil deposit temporarily behaves as a viscous fluid; pore pressures rise, and the strength of the deposit is greatly diminished. Liquefiable soils typically consist of cohesionless sands and silts that are loose to medium dense, and saturated. Recent studies also show that some relatively soft cohesive soils can be subject to cyclic softening during significant earthquake shaking. To liquefy, saturated soils must be subjected to ground shaking of sufficient magnitude and duration. For our analysis we used a PGA of 0.500g, an earthquake magnitude of 7.0, and groundwater depth of 7 feet bgs.

Based on our analysis, there is a potential for liquefaction to occur within the loose to medium dense alluvial sands and silts underlying the site. Dynamic and post-liquefaction settlements are estimated to be about 10 to 12 inches total and about 5 to 6 inches differential across the structure. Lateral spreading is estimated to be about 15 to 20 inches. We that understand ground improvement will be performed to reduce settlements to 2 inches total and 1-inch differential over a distance of 40 feet.

5.3. Site Class

Based on the seismic shear wave velocity in the upper 100 feet (V_{s100}) measured during the investigation, the site is classified as Site Class D. A site-specific ground motion hazard analysis (GMHA) was performed in accordance with the requirements of updated 2022 California Building Code (CBC), American Society of Civil Engineers (ASCE) 7-16, and ASCE 7-16 supplemental publications. Table 5-1 presents the results of seismic CPT testing, which indicates an average V_{s100} of about 706 ft/sec.

Table 5-1. Summary of the Seismic CPT Testing, CPT-1

Loc.	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT-1	10.04	9.04	9.26	14.24	650	-
	20.05	19.05	19.15	32.76	585	534
	30.02	29.02	29.09	47.96	607	654
	40.03	39.03	39.08	60.68	644	786
	50.03	49.03	49.07	76.88	638	617
	60.04	59.04	59.07	89.38	661	800
	70.05	69.05	69.08	103.72	666	698
	80.09	79.09	79.12	114.24	693	954
	90.03	89.03	89.05	123.00	724	1134

5.4. Site-Specific Ground Motion Hazard Analysis

For Site Class D, a site-specific ground motion hazard analysis (GMHA) is required to be performed based on the requirements of the 2022 CBC and ASCE 7-16. As part of the analysis, base ground motions were evaluated with both a Probabilistic Seismic Hazard Analysis (PSHA) and a Deterministic Seismic Hazard Analysis (DSHA) to characterize earthquake ground shaking that may occur at the site during future seismic events.

The PSHA is based on an assessment of the recurrence of earthquakes on potential seismic sources in the region and on ground motion prediction models of different seismic sources in the region. The United States Geological Survey (USGS) Unified Hazard Tool (USGS, 2024b) was used to develop seismic hazard curves for various periods, and the USGS Risk-Targeted Ground Motion Calculator (USGS, 2024c) was used to analyze ground motions for each corresponding period. Maximum directional scale factors were applied to the results to develop the probabilistic ground motion response spectrum specific to this site.

The DSHA is represented by the 84th percentile of the spectral accelerations for different periods. The logarithmic means and standard deviations of various periods were calculated using the USGS Response Spectra Tool (USGS, 2024d) with ground motion model(s) “Combined: WUS 2018 (5.0, deep basins).” This combined model utilizes attenuation relationships of Abrahamson-et al (2014) NGA West 2, Boore-et al (2014) NGA West 2, Campbell & Bozorgnia (2014) NGA West 2, and Chiou & Youngs (2014) NGA West 2.

The deterministic ground motions are controlled by the Newport-Inglewood (Offshore) Fault. Input parameters were obtained from the USGS Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) model, and USGS Earthquake Scenario Map (BSSC 2014) (USGS, 2024e), as presented in Table 5-2.

Table 5-2. DSHA Input Parameters

Fault: Newport-Inglewood (Offshore)	
Mw	7.02
Type	Strike-Slip
Dip (°)	90.0
Rake (°)	180.0
Width (km)	9.18
R_x (km)	7.70
R_{RUP} (km)	7.70
R_{JB} (km)	7.70
V_{s30} (m/s)	215*

*Measured

The site-specific Risk-Targeted Maximum Considered Earthquake (MCER) was taken as the lesser of the spectral response accelerations determined from the PSHA and DSHA for each period. The site-specific design response spectral accelerations were compared to the design response spectrum from ASCE 7-16, Section 11.4.6 (SEAOC, 2024) to verify that the values obtained from the site-specific analysis are not less than 80% of the accelerations obtained from Section 11.4.6. The site coefficients and maximum considered earthquake spectral response acceleration parameters are presented in Table 5-3. Tabulated values and graphical plots are attached in Appendix F.

Table 5-3. 2022 CBC and ASCE 7-16 Seismic Design Parameters

Site Coordinates	
Latitude: 33.22000°	Longitude: -117.35494°
Site Coefficients and Spectral Response Acceleration Parameters	**Value**
Site Class	D
Site Amplification Factor at 0.2 Second, F_a	1.109
Site Amplification Factor at 1.0 Second, F_v	1.940
Spectral Response Acceleration at Short Period, S_S	1.165g
Spectral Response Acceleration at 1-Second Period, S_1	0.527g
Spectral Response Acceleration at Short Period, Adjusted for Site Class, S_{MS}	1.292g
Spectral Response Acceleration at 1-Second Period, Adjusted for Site Class, S_{M1}	1.023g
Design Spectral Acceleration at Short Period, S_{DS}	0.861g
Design Spectral Acceleration at 1-Second Period, S_{D1}	0.682g
Peak Ground Acceleration, PGA_M	0.518g

5.5. Landslides and Slope Stability

Evidence of landslides, deep-seated landslides, or slope instabilities were not observed at the time of the field investigation. Additionally, there are no mapped landslides in the vicinity of the project site. The site is relatively level and the potential for landslides or slope instabilities to occur at the site is considered very low.

5.6. Flooding, Tsunamis, and Seiches

The site is located within Zone A99, mapped as a special flood hazard area without a base flood elevation (FEMA, 2012; 2019). The southern portion of the site, although mapped as Zone A99, is mapped as an area with a reduced flood risk due to a levee. The site is not located within a mapped inundation area on the State of California Tsunami Inundation Maps (CGS, 2022); therefore, damage due to tsunamis is considered negligible. Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The site is not located adjacent to any lakes or confined bodies of water; therefore, the potential for a seiche to affect the site is considered negligible.

5.7. Subsidence

The site is not located in an area of known subsidence associated with fluid withdrawal (groundwater or petroleum); therefore, the potential for subsidence due to the extraction of fluids is considered negligible.

5.8. Hydro-Consolidation

Hydro-consolidation can occur in recently deposited sediments (less than 10,000 years old) that were deposited in a semi-arid environment. Examples of such sediments are eolian sands, alluvial fan deposits, and mudflow sediments deposited during flash floods. The pore spaces between the particle grains can re-adjust when inundated by groundwater, causing the material to consolidate. The loose, unsaturated alluvial soils are considered susceptible to hydro-consolidation. The proposed remedial grading and ground improvement should effectively mitigate this hazard.

6. CONCLUSIONS

Based on the results of our investigation, we consider the proposed construction feasible from a geotechnical standpoint provided the recommendations contained in this report are followed. Geotechnical conditions exist that should be addressed prior to construction. Geotechnical design and construction considerations include the following.

- There are no known active faults underlying the site. The main seismic hazard at the site is the potential for moderate to severe ground shaking in response to large-magnitude earthquakes generated during the lifetime of the proposed construction. The risk of strong ground motion is common to all construction in southern California and is typically mitigated through building design in accordance with the CBC.
- The site is underlain by relatively deep, saturated alluvial deposits that are potentially liquefiable should a significant seismic event occur. Seismic settlements on the order of 10 to 12 inches total and 5 to 6 inches differential are estimated. Mitigation of potentially liquefiable soils typically consists of ground improvement or deep foundations. We understand that ground improvement consisting of stone columns will be used to mitigate the liquefaction hazard and the resulting settlements to acceptable levels.
- The unsaturated soils above groundwater are potentially compressible. To improve subgrade support and reduce the potential for settlement, remedial grading of the upper soils will need to be performed. Remedial grading recommendations are provided herein.
- Based on our expansion index (EI) testing, the on-site soils have a very low expansion potential. These soils are suitable for reuse as compacted fill. Clays, if encountered, are not suitable for direct support of buildings or heave-sensitive improvements. Recommendations for expansive soils are provided herein.
- In general, excavations should be achievable using standard heavy earthmoving equipment in good working order with experienced operators.
- Following ground improvement and mitigation of seismic settlements to acceptable levels, the proposed building can be supported on shallow spread footings with bottom levels bearing on stone columns. Foundation recommendations are provided herein.
- Flooding after periods of rainfall can occur due to the site's proximity to the San Luis Rey River. A floodwall will be constructed to mitigate the flooding hazard. We understand the floodwall will be constructed using sheet piles. Floodwall recommendations are provided herein.
- Groundwater was encountered at depths between about 7 and 7½ feet bgs, corresponding to elevations of about +17½ to +20 feet msl, and should be anticipated during construction.
- The infiltration feasibility condition category is "No Infiltration" within the Quaternary Alluvial Flood-Plain deposits due to increased risk of geotechnical hazards. Infiltration is discussed further in Section 8 of this report.

7. RECOMMENDATIONS

The remainder of this report presents recommendations regarding earthwork construction as well as preliminary geotechnical recommendations for the design of the proposed improvements. These recommendations are based on empirical and analytical methods typical of the standard of practice in southern California. If these recommendations appear not to address a specific feature of the project, please contact our office for additions or revisions to the recommendations. The recommendations presented herein may need to be updated once final plans are developed.

7.1. Earthwork

Grading and earthwork should be conducted in accordance with the CBC and the recommendations of this report. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on field conditions observed by our offices during grading.

7.1.1 Site Preparation

Site preparation should begin with the removal of existing improvements, vegetation, and debris. Subsurface improvements that are to be abandoned should be removed, and the resulting excavations should be backfilled and compacted in accordance with the recommendations of this report. Pipeline abandonment can consist of capping or rerouting at the project perimeter and removal within the project perimeter. If appropriate, abandoned pipelines can be filled with grout or slurry as recommended by and observed by the geotechnical consultant.

7.1.2 Remedial Grading – Building Pad

To improve building support and reduce the potential for static settlement, the top 5 feet of existing soil beneath the proposed building pad should be excavated. Horizontally, excavations should extend at least 5 feet outside the planned perimeter foundations or up to existing improvements or the project boundary, whichever is less. NOVA should observe conditions exposed in the bottom of the excavation to evaluate if additional excavation is required. The resulting excavation should then be filled to the finished pad grade with compacted fill having an EI of 50 or less. We anticipate that the excavated soils will generally be suitable for reuse as compacted fill.

7.1.3 Ground Improvement

Various ground improvement methods are available to mitigate liquefaction and the resulting settlements to acceptable levels in saturated granular soils. Methods include stone columns and pressure grouting. The specifications are unique to the method used and to the contractor performing the work, as each contractor's methods and equipment vary. The only control is to perform post-treatment testing to verify that the soils have been densified as required to mitigate the potential for liquefaction. Verification testing should be performed after ground improvement is completed. We understand stone columns will be used for ground improvement. Stone columns should be designed to target maximum total and differential settlements of 2 inches and 1 inch over a distance of 40 feet, respectively. Following ground improvement installation and verification testing indicating that the liquefaction potential has been mitigated to acceptable levels, the planned building can be supported on shallow spread footings with bottoms levels bearing on stone columns. NOVA should observe the ground improvement operations.

7.1.4 Remedial Grading – Pedestrian Hardscape

Beneath proposed pedestrian hardscape areas, the on-site soils should be excavated to a depth of at least 2 feet below planned subgrade elevation. Horizontally, excavations should extend at least 2 feet outside the planned hardscape or up to existing improvements, whichever is less. NOVA should observe the conditions exposed at the bottom of excavations to evaluate whether additional excavation is recommended. The resulting surface should then be scarified to a depth of 6 to 8 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. The excavation should be filled with compacted fill having an EI of 50 or less.

7.1.5 Remedial Grading – Vehicular Pavements

Beneath proposed vehicular pavement areas, the existing soils should be excavated to a depth of at least 1 foot below planned subgrade elevation. Horizontally, excavations should extend at least 2 feet outside the planned pavement or up to existing improvements, whichever is less. NOVA should observe the conditions exposed in the bottom of excavations to evaluate whether additional excavation is recommended. The resulting surface should then be scarified to a depth of 6 to 8 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. The excavation should be filled with material suitable for reuse as compacted fill.

7.1.6 Remedial Grading – Conventional Site Walls and Retaining Walls

Beneath proposed conventional site walls and retaining walls not connected to buildings, the existing fill should be excavated to a depth of at least 2 feet below bottom of footing. Horizontally, the excavations should extend at least 2 feet outside the planned hardscape, wall footing, or up to existing improvements, whichever is less. NOVA should observe the conditions exposed at the bottom of excavations to evaluate whether additional excavation is recommended. Any required fill should have an EI of 50 or less.

7.1.7 Remedial Grading – Floodwall

Prior to installing sheet piles for the proposed floodwall, site preparation should be performed along the floodwall alignment as described in Section 7.1.3. The removals should include the areas within the limits of proposed backfill behind the floodwall. Once the sheet piles are driven and the floodwall has achieved adequate structural strength, granular and free-draining soil having an EI of 20 or less can be placed and compacted. Lateral deflection of the floodwall should be monitored during backfilling.

7.1.8 Expansive Soil

The on-site soils tested had EIs of 0 and 2, classified as very low expansion potential. To reduce the potential for expansive heave, the top 2 feet of material beneath building footings, concrete slabs-on-grade, hardscape, and site and retaining wall footings should have an EI of 50 or less. Horizontally, the soils having an EI of 50 or less should extend at least 5 feet outside the planned perimeter building foundations, at least 2 feet outside hardscape and site/retaining wall footings, or up to existing improvements, whichever is less. NOVA anticipates that the on-site soil will generally meet the EI criterion. Clays, if encountered, are not expected to meet the EI criterion.

7.1.9 Compacted Fill

Excavated soils free of organic matter, construction debris, rocks greater than 6 inches, and expansive soils described above can be used as compacted fill. Areas to receive fill should be scarified to a depth of 6 to 8 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. Fill and backfill should be placed in 6- to 8-inch-thick loose lifts, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. The top 12 inches of pavement subgrade should be compacted to at least 95% relative compaction. The maximum density and optimum moisture content for the evaluation of relative compaction should be determined in accordance with ASTM D1557.

7.1.10 Imported Soil

Imported soil should consist of predominately granular soil, free of organic matter and rocks greater than 6 inches. Imported soil should be observed and, if appropriate, tested by NOVA prior to transport to the site to evaluate suitability for the intended use.

7.1.11 Subgrade Stabilization

Excavation bottoms should be firm and unyielding prior to placing fill. In areas of saturated or yielding subgrade, a reinforcing geogrid such as Tensar® Triax® TX-5 or equivalent can be placed on the excavation bottom, and then at least 12 inches of aggregate base placed and compacted. Once the surface of the aggregate base is firm enough to achieve compaction, then the remaining excavation should be filled to finished pad grade with suitable material.

7.1.12 Excavation Characteristics

It is anticipated that excavations can be achieved with conventional earthwork equipment in good working order. Groundwater may be encountered in the excavations.

7.1.13 Oversized Material

Excavations may generate oversized material. Oversized material is defined as rocks or cemented clasts greater than 6 inches in largest dimension. Oversized material should be broken down to no greater than 6 inches in largest dimension for use in fill, use as landscape material, or disposed of off-site.

7.1.14 Temporary Excavations

Temporary excavations 3 feet deep or less can be made vertically. Deeper temporary excavations in fill and alluvium should be laid back no steeper than 1:1 (horizontal:vertical) (h:v). Deeper temporary excavations in soils consisting of cohesionless clean sands (SP, SW) should be laid back no steeper than 1½:1 (h:v). The faces of temporary slopes should be inspected daily by the contractor's

Competent Person before personnel are allowed to enter the excavation. Any zones of potential instability, sloughing, or raveling should be brought to the attention of the engineer and corrective action implemented before personnel begin working in the excavation. Excavated soils should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. NOVA should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of slopes to prevent runoff water from entering the excavation and eroding the slope faces.

Slopes steeper than those described above will require shoring. Additionally, temporary excavations that extend below a plane inclined at 1½:1 (h:v) downward from the outside bottom edge of existing structures or improvements will require shoring. Soldier piles and lagging, internally braced shoring, or trench boxes could be used. If trench boxes are used, the soil immediately adjacent to the trench box is not directly supported. Ground surface deformations immediately adjacent to the pit or trench could be greater where trench boxes are used compared to other methods of shoring.

7.1.15 Temporary Shoring

For design of cantilevered shoring with level backfill, an active earth pressure equal to a fluid weighing 40 pounds per cubic foot (pcf) can be used. An additional 20 pcf should be added for 2:1 (h:v) sloping ground. The surcharge loads on shoring from traffic and construction equipment working adjacent to the excavation can be modeled by assuming an additional 2 feet of soil behind the shoring. For design of soldier piles, an allowable passive pressure of 300 pounds per square foot (psf) per foot of embedment above groundwater or 150 psf below groundwater can be used over two times the pile diameter up to a maximum of 2,000 psf. Soldier piles should be spaced at least three pile diameters, center to center. Continuous lagging will be required throughout. The soldier piles should be designed for the full anticipated lateral pressure; however, the pressure on the lagging will be less due to arching in the soils. For design of lagging, the earth pressure can be limited to a maximum of 400 psf.

7.1.16 Slopes

Permanent slopes should be constructed no steeper than 2:1 (h:v). Faces of fill slopes should be compacted either by rolling with a sheepsfoot roller or other suitable equipment, or by overfilling and cutting back to design grade. Fills should be benched into sloping ground inclined steeper than 5:1 (h:v). In our opinion, slopes constructed no steeper than 2:1 (h:v) will possess an adequate factor of safety. An engineering geologist should observe cut slopes during grading to ascertain that no unforeseen adverse geologic conditions are encountered that require revised recommendations. Slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the top of slope. Additionally, slopes should be planted with vegetation that will reduce the potential for erosion.

7.1.17 Groundwater

As previously mentioned, groundwater was encountered at depths between about 7 and 7½ feet bgs and should be anticipated in excavations. Groundwater levels may fluctuate in the future due to rainfall, irrigation, broken pipes, or changes in site drainage. If dewatering is necessary, the dewatering method should be evaluated and implemented by an experienced dewatering subcontractor.

7.1.18 Surface Drainage

Final surface grades around structures should be designed to collect and direct surface water away from structures, including retaining walls, and toward appropriate drainage facilities. The ground around the structure should be graded so that surface water flows rapidly away from the structure without ponding. In general, we recommend that the ground adjacent to the structure slope away at a gradient of at least 2%. Densely vegetated areas where runoff can be impaired should have a minimum gradient of at least 5% within the first 5 feet from the structure. Roof gutters with downspouts that discharge directly into a closed drainage system are recommended on structures. Drainage patterns established at the time of fine grading should be maintained throughout the life of the proposed structures. Site irrigation should be limited to the minimum necessary to sustain landscape growth. Should excessive irrigation, impaired drainage, or unusually high rainfall occur, saturated zones of perched groundwater can develop.

7.1.19 Grading Plan Review

NOVA should review the grading plans and earthwork specifications to ascertain whether the intent of the recommendations contained in this report have been implemented, and that no revised recommendations are needed due to changes in the development scheme.

7.2. Foundations

The foundation recommendations provided herein are considered generally consistent with methods typically used in southern California. Other alternatives may be available. Our recommendations are only minimum criteria based on geotechnical factors and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or by the structural engineer. The design of the foundation system should be performed by the project structural engineer, incorporating the geotechnical parameters described herein and the requirements of applicable building codes.

7.2.1 Spread Footings

Following ground improvement and mitigation of seismic settlements to acceptable levels, the proposed building can be supported on shallow spread footings with bottom levels bearing on stone columns. Footings should extend at least 24 inches below lowest adjacent finished grade. A minimum width of 12 inches is recommended for continuous footings and 24 inches for isolated or wall footings. An allowable bearing capacity of 5,000 psf can be used. The bearing value can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. Footings located adjacent to or within slopes should be extended to a depth such that a minimum horizontal distance of 10 feet exists between the lower outside footing edge and the face of the slope.

Lateral loads will be resisted by friction between the bottoms of footings and passive pressure on the faces of footings and other structural elements below grade. An allowable coefficient of friction of 0.35 can be used. An allowable passive pressure of 350 psf per foot of depth below the ground surface can be used for level ground conditions. The allowable passive pressure should be reduced for sloping ground conditions. The passive pressure can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

7.2.2 Settlement Characteristics

We understand that the ground improvement program will be designed to target maximum total and differential settlements of 2 inches and 1 inch over a distance of 40 feet, respectively, for static and seismic conditions.

7.2.3 Foundation Plan Review

NOVA should review the foundation plans to ascertain that the intent of the recommendations in this report has been implemented and that revised recommendations are not necessary as a result of changes after this report was completed.

7.2.4 Foundation Excavation Observations

A representative from NOVA should observe the foundation excavations prior to forming or placing reinforcing steel.

7.3. Interior Slabs-On-Grade

Interior concrete slabs-on-grade should be underlain by at least 2 feet of material with an EI of 20 or less. The top 12 inches of subgrade soils should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. Subgrade preparation should be performed immediately prior to placement of the concrete slab.

We recommend that concrete slabs-on-grade be at least 5 inches thick and reinforced with at least No. 4 bars at 18 inches on center each way. To reduce the potential for excessive cracking, concrete slabs should be provided with construction or ‘weakened plane’ joints at frequent intervals. The project structural engineer should design on-grade building slabs and joint spacing.

Moisture protection should be installed beneath slabs where moisture-sensitive floor coverings will be used. The project architect should review the tolerable moisture transmission rate of the proposed floor covering and specify an appropriate moisture protection system. Typically, a plastic vapor barrier is used. Minimum 15-mil plastic is recommended. The plastic should comply with ASTM E1745. The vapor barrier installation should comply with ASTM E1643. The slab can be placed directly on the vapor barrier.

7.4. Pedestrian Hardscape

Pedestrian hardscape should be underlain by at least 2 feet of material with an EI of 20 or less. The top 12 inches of subgrade soils should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. If competent formational sandstone is exposed, scarification, and recompaction need not be performed. Subgrade preparation should be performed immediately prior to placement of the hardscape.

Exterior slabs should be at least 4 inches in thickness and reinforced with at least No. 3 bars at 18 inches on center each way. Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. The project architect should select the final joint patterns. A 1-inch maximum size aggregate mix is recommended for concrete for exterior slabs. The corrosion potential of on-site soils with respect to reinforced concrete will need to be taken into account in concrete mix design. Coarse and fine aggregate in concrete should conform to the “Greenbook” specifications.

7.5. Conventional Retaining Walls

Conventional retaining walls can be supported on shallow spread footings. The recommendations for spread footings provided in the foundation section of this report are also applicable to conventional retaining walls.

The active earth pressure for the design of unrestrained retaining walls with level backfill can be taken as equivalent to the pressure of a fluid weighing 35 pcf. The at-rest earth pressure for the design of restrained retaining wall with level backfill can be taken as equivalent to the pressure of a fluid weighing 55 pcf. These values assume a granular and drained backfill condition. Higher lateral earth pressures would apply if walls retain clay soils. An additional 20 pcf should be added to these values for walls with 2:1 (h:v) sloping backfill. An increase in earth pressure equivalent to an additional 2 feet of retained soil can be used to account for surcharge loads from light traffic. The above values do not include a factor of safety. Appropriate factors of safety should be incorporated into the design. If any other surcharge loads are anticipated, NOVA should be contacted for the necessary increase in soil pressure.

If required, the seismic earth pressure can be taken as equivalent to the pressure of a fluid pressure weighing 16 pcf. This value is for level backfill and does not include a factor of safety. Appropriate factors of safety should be incorporated into the design. This pressure is in addition to the un-factored, active earth pressure. The total equivalent fluid pressure can be modeled as a triangular pressure distribution with the resultant acting at a height of H/3 up from the base of the wall, where H is the retained height of the wall. The passive pressure and bearing capacity can be increased by ⅓ in determining the seismic stability of the wall.

Retaining walls should be provided with a backdrain to reduce the accumulation of hydrostatic pressures or be designed to resist hydrostatic pressures. Backdrains can consist of a 2-foot-wide zone of ¾-inch crushed rock. The crushed rock should be separated from the adjacent soils using a non-woven filter fabric, such as Mirafi 140N or equivalent. A perforated pipe should be installed at the base of the backdrain and sloped to discharge to a suitable storm drain facility, or weep holes should be provided. As an alternative, a geocomposite drainage system such as Miradrain 6000 or equivalent placed behind the wall and connected to a suitable storm drain facility can be used. The project architect should provide dampproofing/waterproofing specifications and details. Figure 7-1 presents typical conventional retaining wall backdrain details. Note that the guidance provided on Figure 7-1 is conceptual. Other options are available.

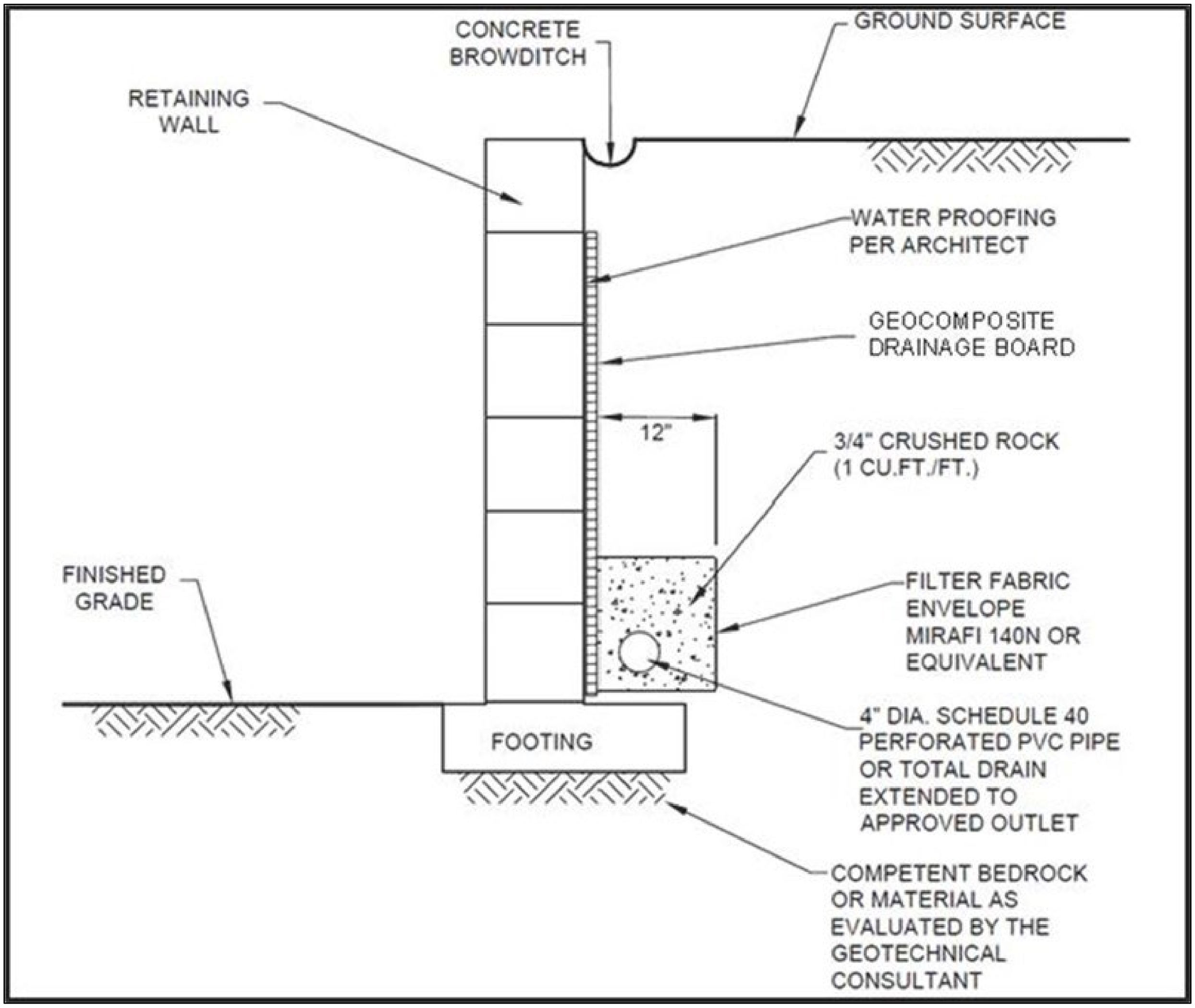

Figure 7-1. Typical Conventional Retaining Wall Backdrain Detail

Wall backfill should consist of granular, free-draining material having an EI of 20 or less. The backfill zone is defined by a 1:1 plane projected upward from the heel of the wall. Expansive or clayey soil should not be used. Additionally, backfill within 3 feet from the back of the wall should not contain rocks greater than 3 inches in dimension. Backfill should be compacted to at least 90% relative compaction. Backfill should not be placed until walls have achieved adequate structural strength. Compaction of wall backfill will be necessary to minimize settlement of the backfill and overlying settlement sensitive improvements. However, some settlement should still be anticipated. Provisions should be made for some settlement of concrete slabs and pavements supported on backfill. Additionally, any utilities supported on backfill should be designed to tolerate differential settlement.

7.6. Floodwall

We understand the proposed floodwall will be constructed using steel sheet piles. The active earth pressure for the design of unrestrained sheet piles can be taken as equivalent to the pressure of a fluid weighing 35 pcf. If required, the seismic earth pressure can be taken in addition to the active earth pressure as equivalent to the pressure of a fluid pressure weighing 16 pcf. These values are for level backfill and do not include a factor of safety. Appropriate factors of safety should be incorporated into the design. The total equivalent fluid pressure can be modeled as a triangular pressure distribution with the resultant acting at a height of H/3 up from the base of the wall, where H is the retained height of the wall.

For level ground conditions above groundwater, an allowable passive pressure of 300 psf per foot of depth below the ground surface can be used. For level ground conditions below groundwater, an allowable passive pressure of 150 psf per foot of depth below the ground surface can be used. The allowable passive pressure values should be reduced for sloping ground conditions. The passive pressure can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

To reduce the potential for water intrusion, a sheet piling interlock sealant such as WADIT is recommended. The sealant is typically applied to the interlocks prior to driving the sheets in general accordance with the product manufacturer's recommendations.

The floodwall should be provided with a backdrain to reduce the accumulation of hydrostatic pressures or be designed to resist hydrostatic pressures. The backdrain can consist of a 4-inch diameter perforated PVC pipe surrounded by crushed rock wrapped with filter fabric such as Mirafi 140N, and outlet through solid PVC pipe to the storm drain system.

Wall backfill should consist of granular, free-draining material having an EI of 20 or less. The backfill and compaction equipment will load the sheet pile floodwall, which may result in lateral deflection. Floodwall deflection should be evaluated by the design engineer to confirm that the sheet piles will contain adequate moment capacity. The actual deflection should be monitored weekly during the backfill process using surveyed monuments to confirm that deflection remains within tolerable limits defined by the structural engineer.

Sheet piles are typically installed by vibratory driving, impact driving, and/or hydraulic pushing. In general, vibratory driving is the most efficient method of installing sheet piles in granular soils such as the on-site soils. Most of the alluvium is anticipated to be relatively easily penetrated; however, localized layers of dense sands may result in driving difficulties. The contractor should select the appropriate driving methods and equipment to achieve the required penetration without damaging the sheet piles.

7.7. Pipelines

For level ground conditions, a passive earth pressure of 300 psf per foot of depth below the lowest adjacent final grade can be used to compute allowable thrust block resistance. A value of 150 psf per foot should be used below groundwater level.

A modulus of soil reaction (E') of 1,500 psi can be used to evaluate the deflection of buried flexible pipelines. This value assumes that granular bedding material is placed adjacent to the pipe and is compacted to at least 90% relative compaction.

Pipe bedding as specified in the "Greenbook" Standard Specifications for Public Works Construction can be used. Bedding material should consist of clean sand having a sand equivalent not less than 20 and should extend to at least 12 inches above the top of pipe. Alternative materials meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The on-site materials are not expected to meet "Greenbook" bedding specifications. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the pipe haunches. Ponding or jetting the pipe bedding should not be allowed.

Where pipeline inclinations exceed 15%, cutoff walls are recommended in trench excavations. Additionally, we do not recommend that open graded rock be used for pipe bedding or backfill because of the potential for piping erosion. The recommended bedding is clean sand having a sand equivalent not less than 20 or 2-sack sand/cement slurry. If sand/cement slurry is used for pipe bedding to at least 1 foot over the top of the pipe, cutoff walls are not considered necessary. The need for cutoff walls should be further evaluated by the project civil engineer designing the pipeline.

7.8. Pavement Section Recommendations

The pavement support characteristics of the soils encountered during NOVA's investigation are considered low to medium. An R-value of 39 was assumed for design of preliminary pavement sections. The actual R-value of the subgrade soils should be determined after grading, and the final pavement sections should be provided. Based on an R-value of 39, the following preliminary pavement structural sections are provided for the assumed Traffic Indexes on Table 7-1.

Table 7-1. AC and PCC Pavement Sections

Traffic Type	Traffic Index	Asphalt Concrete (inches)	Portland Cement Concrete (inches)
Parking Stalls	4.5	3 AC / 4 AB	6 PCC
Driveways	6.0	4 AC / 5 AB	6½ PCC
Fire Lanes	7.5	5 AC / 7 AB	7½ PCC

AC: Asphalt Concrete
AB: Aggregate Base
PCC: Portland Cement Concrete

Subgrade preparation should be performed immediately prior to placement of the pavement section. The upper 12 inches of subgrade should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction. All soft or yielding areas should be stabilized or removed and replaced with compacted fill or aggregate base. Aggregate base and asphalt concrete should conform to the Caltrans Standard Specifications or the "Greenbook" and should be compacted to at least 95% relative compaction. Aggregate base should have an R-value of not less than 78. All materials and methods of construction should conform to good engineering practices and the minimum local standards.

7.9. Working Platforms

Construction may utilize equipment (cranes, concrete pumps, concrete trucks, etc.) that impose high ground bearing pressures. The contractor is solely responsible for design, construction, and maintenance of working platforms that safely support this equipment (see OSHA Construction Standard Subpart R, CFR 1926.752). As necessary, working platforms and/or paths of heavy equipment travel should be designed in conformance with appropriate guidance (see, for example, Guide to Working Platforms, 1st Edition, January 2020, by the Deep Foundations Institute).

7.10. Corrosivity

Representative samples of the on-site soils were tested to evaluate corrosion potential. The test results are presented in Appendix C. The project design engineer can use the sulfate results in conjunction with ACI 318 to specify the water/cement ratio, compressive strength, and cementitious material types for concrete exposed to soil. A corrosion engineer should be contacted to provide specific corrosion control recommendations.

8. INFILTRATION FEASIBILITY

Final stormwater infiltration Best Management Practices ('stormwater BMP') locations were not identified at the time of the investigation; however, NOVA coordinated with the project architect to provide infiltration testing in the areas most likely to have BMPs.

Two percolation test borings (P-1 and P-2) were constructed following the recommendations for percolation testing presented in the City of Oceanside BMP Design Manual (hereinafter, 'the BMP Manual').

The percolation test borings were drilled with a truck-mounted, 8-inch hollow stem auger to depths of about 5 feet bgs. Field measurements were taken to confirm that the boring was excavated to about 8 inches in diameter. The borings were logged by a NOVA geologist, who observed and logged the exposed soil cuttings and the boring conditions.

Once the boring was drilled to the desired depth, the boring was converted to a percolation test boring by placing an approximately 2-inch layer of ¾-inch gravel on the bottom, then extending 3-inch diameter Schedule 40 perforated PVC pipe to the ground surface. The ¾-inch gravel was used to partially fill the annular space around the perforated pipe below existing finish grade to minimize the potential of soil caving.

The percolation test well was pre-soaked by filling the hole with water to the ground surface level and testing commenced within a 26-hour window. On the day of testing, two 25-minute trials were conducted in the well.

In the percolation borings, the pre-soak water did not percolate over 6 inches into the soil unit within 25 minutes. Based on the results of the trials, water levels were recorded every 30 minutes for 6 hours. At the beginning of each test interval, the water level was raised to approximately the same level as the previous tests, in order to maintain a near-constant head during all test periods.

The percolation rate of a soil profile is not the same as its infiltration rate ('I'). Therefore, the field percolation rate was converted to an estimated infiltration rate utilizing the Porchet Method in accordance with guidance contained in the BMP Manual. The table below provides a summary of the infiltration rates determined by the percolation testing.

Table 8-1. Infiltration Rate Test Results

Test Location	Test Depth (feet)	Material at Test Depth	Infiltration Rate (in/hr, FS=2)
P-1	5	Young Alluvial Flood-Plain Deposits: Poorly Graded Sand with Silt	0.45
P-2	5	Young Alluvial Flood-Plain Deposits: Poorly Graded Sand	0.12

Note: 'FS' indicates 'Factor of Safety'

As shown in Table 8-1, a factor of safety (FS) is applied to the infiltration rate (I) determined by the percolation testing. This factor of safety, at least FS = 2 in local practice, considers the nature and variability of subsurface materials, as well as the natural tendency of infiltration structures to become less efficient with time. The infiltration rate after applying FS = 2 is I > 0.01 inch per hour but less than 0.5 inches per hour. Partial infiltration BMPs are typically suitable with these rates, however, not without increasing the geotechnical hazards.

8.1. Infiltration Restrictions

Form 4 from the BMP Design Manual is intended to present the mandatory and optional criteria when considering stormwater infiltration feasibility at a site. NOVA has selected *Restricted* on this form due to the shallow groundwater at the site, highly liquefiable soils, the hydric soils, and the proposed proximity of the BMPs to the structure and walls. Form 4 is presented below.

Appendix D: Approved Infiltration Rate Assessment Methods

Infiltration Restrictions		**Form 4**	
Retention is required at the project site to the maximum extent practicable. Complete this form to summarize applicable infiltration restrictions. Supporting documentation must be provided in the Attachments.			
Restriction Element		**Applicable?**	
Mandatory Considerations	BMP is within 100 feet of contaminated soils	☐ Yes	☒ No
	BMP is within 100 feet of industrial activities lacking source control	☐ Yes	☒ No
	BMP is within 100 feet of well/groundwater basin	☐ Yes	☒ No
	BMP is within 50 feet of septic tanks/leach fields	☐ Yes	☒ No
	BMP is within 10 feet of structures/tanks/walls	☒ Yes	☐ No
	BMP is within 10 feet of sewer utilities	☐ Yes	☒ No
	BMP is within 10 feet of groundwater table	☒ Yes	☐ No
	BMP is within hydric soils	☒ Yes	☐ No
	BMP is within highly liquefiable soils and has connectivity to structures	☒ Yes	☐ No
	BMP is within 1.5 times the height of adjacent steep slopes (≥25%)	☐ Yes	☒ No
	City staff has assigned "Restricted" Infiltration Category	☐ Yes	☒ No
Optional Considerations	BMP is within predominantly Type D soil	☐ Yes	☒ No
	BMP is within 10 feet of property line	☐ Yes	☒ No
	BMP is within fill depths of ≥5 feet (existing or proposed)	☐ Yes	☒ No
	BMP is within 10 feet of underground utilities	☐ Yes	☒ No
	BMP is within 250 feet of ephemeral stream	☐ Yes	☒ No
	Other (provide detailed geotechnical support in Attachment 6)	☐ Yes	☒ No
Result	**Unrestricted** – No restriction elements are applicable	☐	
	Restricted – One or more restriction elements are applicable	☒	

8.2. Conclusions and Recommendations

The tested infiltration rates do support reliable stormwater infiltration in an appreciable quantity, however, based on the potential for liquefaction of the underlying soils and distance to groundwater, it is NOVA's judgment that the site is not suitable for permanent infiltration BMPs. Based on the test results, the infiltration feasibility condition category is "No Infiltration." BMP facilities should be lined throughout with an impermeable geomembrane to reduce the potential for water-related distress to adjacent structures or improvements. A subdrain system should be installed at the bottom of BMP facilities. Additionally, BMP facilities should be kept at least 10 feet from structural foundations.

9. CLOSURE

NOVA should review project plans and specifications prior to bidding and construction to check that the intent of the recommendations in this report has been incorporated. Observations and tests should be performed during construction. If the conditions encountered during construction differ from those anticipated based on the subsurface exploration program, the presence of personnel from our offices during construction will enable an evaluation of the exposed conditions and modifications of the recommendations in this report or development of additional recommendations in a timely manner.

NOVA should be advised of changes in the project scope so that the recommendations contained in this report can be evaluated with respect to the revised plans. Changes in recommendations will be verified in writing. The findings in this report are valid as of the date of this report. Changes in the condition of the site can, however, occur with the passage of time, whether they are due to natural processes or work on this or adjacent areas. In addition, changes in the standards of practice and government regulations can occur. Thus, the findings in this report may be invalidated wholly or in part by changes beyond our control. This report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations to site conditions at that time.

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the boring locations and that our data, interpretations, and recommendations are based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

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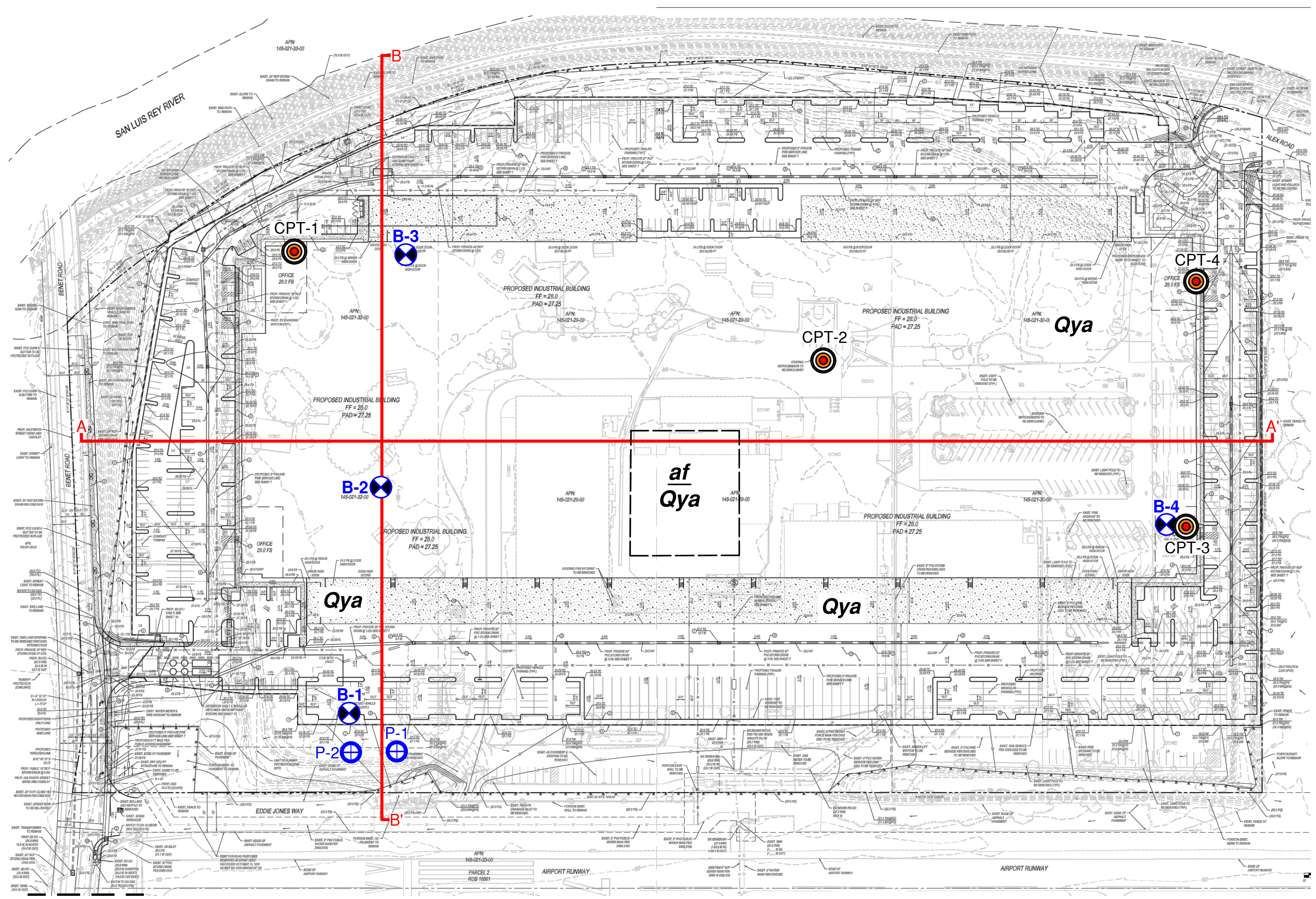
PLATES



KEY TO SYMBOLS

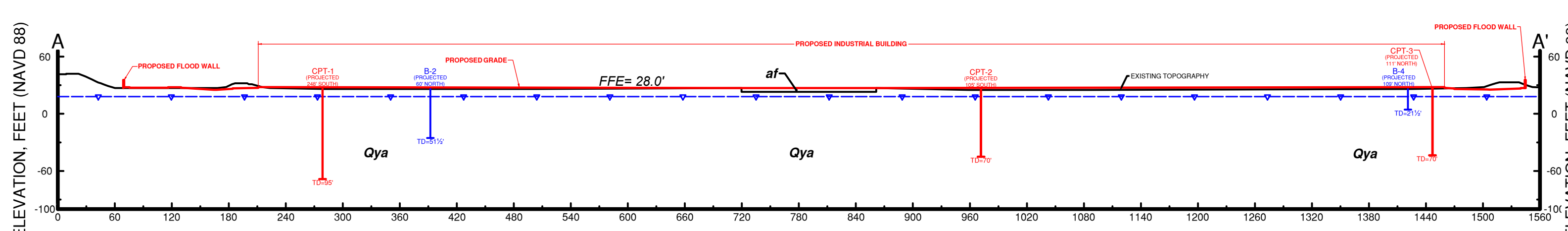
af	FILL
Qya	YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS
B-4	GEOTECHNICAL BORING
P-2	PERCOLATION TEST BORING
CPT-4	CONE PENETRATION TEST
B B'	GEOLOGIC CROSS-SECTION
	GEOLOGIC CONTACT

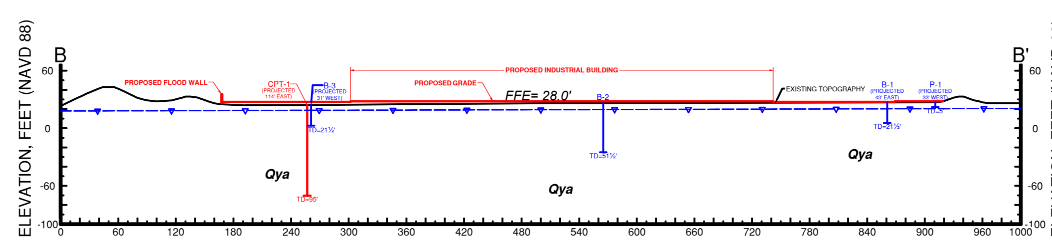
0 200' 400'

BACKGROUND PROVIDED BY:

PASCO LARET SUITER & ASSOCIATES

San Diego | Encinitas | Orange County
Phone 858.259.8212 | www.plsaengineering.com





KEY TO SYMBOLS

af	FILL
Qya	YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS
B-4	GEOTECHNICAL BORING
P-2	PERCOLATION TEST BORING
CPT-4	CONE PENETRATION TEST
	GROUNDWATER
	GEOLOGIC CONTACT
FFE=	FINISHED FLOOR ELEVATION

0 200' 400'

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PROJECT NO.:	2021176
DATE:	APRIL 2024
DRAWN BY:	DJ
REVIEWED BY:	AN/GD
SCALE:	1"=200'
DRAWING TITLE:	

GEOTECHNICAL MAP & CROSS-SECTIONS AA' & BB'

PLATE NO. 1 OF 1

APPENDIX A
USE OF THE GEOTECHNICAL REPORT

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

APPENDIX B
BORING LOGS

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVEL MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVEL WITH LESS THAN 15% FINES	GW	WELL-GRADED GRAVEL WITH OR WITHOUT SAND
			GP	POORLY GRADED GRAVEL WITH OR WITHOUT SAND
		GRAVEL WITH 15% OR MORE FINES	GM	SILTY GRAVEL WITH OR WITHOUT SAND
			GC	CLAYEY GRAVEL WITH OR WITHOUT SAND
	SAND MORE THAN HALF COARSE FRACTION IS FINER THAN NO. 4 SIEVE SIZE	CLEAN SAND WITH LESS THAN 15% FINES	SW	WELL-GRADED SAND WITH OR WITHOUT GRAVEL
			SP	POORLY GRADED SAND WITH OR WITHOUT GRAVEL
		SAND WITH 15% OR MORE FINES	SM	SILTY SAND WITH OR WITHOUT GRAVEL
			SC	CLAYEY SAND WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	SILT WITH OR WITHOUT SAND OR GRAVEL
			CL	LEAN CLAY WITH OR WITHOUT SAND OR GRAVEL
			OL	ORGANIC SILT OR CLAY OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	ELASTIC SILT WITH OR WITHOUT SAND OR GRAVEL
			CH	FAT CLAY WITH OR WITHOUT SAND OR GRAVEL
			OH	ORGANIC SILT OR CLAY OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GROUNDWATER / STABILIZED

PERCHED GROUNDWATER

BULK SAMPLE

SPT SAMPLE (ASTM D1586)

MOD. CAL. SAMPLE (ASTM D3550)

* NO SAMPLE RECOVERY

GEOLOGIC CONTACT

SOIL TYPE CHANGE

LAB TEST ABBREVIATIONS

CR	CORROSIVITY
MD	MAXIMUM DENSITY
DS	DIRECT SHEAR
EI	EXPANSION INDEX
AL	ATTERBERG LIMITS
SA	SIEVE ANALYSIS
RV	RESISTANCE VALUE
CN	CONSOLIDATION
SE	SAND EQUIVALENT

RELATIVE DENSITY OF COHESIONLESS SOILS

RELATIVE DENSITY	SPT N60 BLOWS/FOOT
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY OF COHESIVE SOILS

CONSISTENCY	SPT N60 BLOWS/FOOT	POCKET PENETROMETER MEASUREMENT (TSF)
VERY SOFT	0 - 2	0 - 0.25
SOFT	2 - 4	0.25 - 0.50
MEDIUM STIFF	4 - 8	0.50 - 1.0
STIFF	8 - 15	1.0 - 2.0
VERY STIFF	15 - 30	2.0 - 4.0
HARD	OVER 30	OVER 4.0

NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).
IF THE SEATING INTERVAL (1st 6 INCH INTERVAL) IS NOT ACHEIVED, N IS REPORTED AS REF.

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SUBSURFACE EXPLORATION LEGEND

LOG OF BORING B-1

DATE DRILLED: OCTOBER 4, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 26½ FT NAVD88 **DRILLING EQUP.:** CME 95 **GROUNDWATER DEPTH:** 7½ FT

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTOMATIC) **NOTES:** ETR~70.6%, N_{60} ~ 70.6/60*N~1.17*N

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	SOIL DESCRIPTION *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0								*VEGETATED SURFACE*	
	X						SP	**YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS (Qya):** *POORLY GRADED SAND; OLIVE BROWN TO GRAY, MOIST, LOOSE, FINE TO MEDIUM GRAINED, MICACEOUS*	RV
5	X	X	7	8				*VERY MOIST, LOOSE*	
7½	X							*GROUNDWATER ENCOUNTERED*	
10	X	X	8	9			SM	*SILTY SAND; GRAY, WET, LOOSE, FINE TO MEDIUM GRAINED, MICACEOUS*	
15		X	7	8					
								BROWN SILT LENSE	
20		X	20	23			SM	*SILTY SAND; GRAY, WET, MEDIUM DENSE*	
								BORING TERMINATED AT 21½ FT. GROUNDWATER ENCOUNTERED AT 7½ FT. CAVING AT 7½ FT.	
25									
30									

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FIGURE B.1

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LOG OF BORING B-2

DATE DRILLED: OCTOBER 4, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 26 FT NAVD88 **DRILLING EQUP.:** CME 95 **GROUNDWATER DEPTH:** 7 FT

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTOMATIC) **NOTES:** ETR~70.6%, N_{60} ~ 70.6/60*N~1.17*N

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0								*VEGETATED SURFACE*	
							SM	**YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS (Qya):** *SILTY SAND; OLIVE BROWN TO GRAY, MOIST, LOOSE, FINE TO MEDIUM GRAINED, MICACEOUS*	SA AL EI CR
5			7	8			SP	*POORLY GRADED SAND; DARK GRAY, MOIST, LOOSE, FINE TO MEDIUM GRAINED*	
								GROUNDWATER ENCOUNTERED	
10			4	5				*BROWNISH GRAY, WET*	
15			9	11				*DARK GRAY, MEDIUM DENSE*	
20			19	22				*DARK GRAY TO GRAY, FINE GRAINED*	
25			15	18			SM/SP	*SILTY SAND/POORLY GRADED SAND; GRAY, WET, MEDIUM DENSE, FINE GRAINED, MICACEOUS*	
30									

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FIGURE B.2

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LOG OF BORING B-2

DATE DRILLED: OCTOBER 4, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 26 FT NAVD88 **DRILLING EQUP.:** CME 95 **GROUNDWATER DEPTH:** 7 FT

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTOMATIC) **NOTES:** ETR~70.6%, N_{60} ~ 70.6/60*N~1.17*N

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
30		☐	16	19			SP-SM	**YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS (Qya) CONTINUED:** *POORLY GRADED SAND WITH SILT; GRAY, WET, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
35		☐	7	8				*DARK GRAY, LOOSE, FINE GRAINED, MICACEOUS*	
40		☐	22	26			ML	*SANDY SILT; DARK GRAY, WET, VERY STIFF, FINE GRAINED*	
45		☐	10	12			SM/ML	*SILTY SAND/SANDY SILT; DARK GRAY, WET, MEDIUM DENSE/STIFF, FINE GRAINED SAND, MICACEOUS*	
50		☐	10	12			ML	*SANDY SILT; DARK GRAY, WET, STIFF, FINE TO MEDIUM GRAINED SAND, SOFT SEDIMENT DEFORMATION*	
								BORING TERMINATED AT 51½ FT. GROUNDWATER ENCOUNTERED AT 7 FT. CAVING TO 7 FT.	
55									
60									

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LOG OF BORING B-3

DATE DRILLED:	OCTOBER 4, 2021	**DRILLING METHOD:**	HOLLOW STEM AUGER		
ELEVATION:	± 25 FT NAVD88	**DRILLING EQUP.:**	CME 95	**GROUNDWATER DEPTH:**	7½ FT
SAMPLE METHOD:	HAMMER: 140 LBS., DROP: 30 IN (AUTOMATIC)	**NOTES:**	ETR~70.6%, N_{60} ~ 70.6/60*N~1.17*N		

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0								*VEGETATED SURFACE*	
	X						SP	**YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS (Qya):** *POORLY GRADED SAND; OLIVE BROWN TO GRAY, MOIST, LOOSE, FINE TO MEDIUM GRAINED*	
5	X	⧄	8	9			SP-SM	*POORLY GRADED SAND WITH SILT; DARK GRAY, VERY MOIST, LOOSE*	
▼	X							*GROUNDWATER ENCOUNTERED*	
10		⧄	9	11			SP	*POORLY GRADED SAND; GRAY, WET, MEDIUM DENSE, MEDIUM GRAINED, MICACEOUS, IRON OXIDE STAINING*	
15		⧄	5	6			ML	*SANDY SILT; GRAY, WET, MEDIUM STIFF, FINE GRAINED, IRON OXIDE STAINING, SOME CLAY*	
20		⧄	8	9			SM	*SILTY SAND; GRAY, WET, LOOSE, FINE GRAINED, SCATTERED ORGANIC MATERIAL, MICACEOUS*	
								BORING TERMINATED AT 21½ FT. GROUNDWATER ENCOUNTERED AT 7½ FT. CAVING TO 7½ FT.	
25									
30									

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LOG OF BORING B-4

DATE DRILLED: OCTOBER 4, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 27 FT NAVD88 **DRILLING EQUP.:** CME 95 **GROUNDWATER DEPTH:** 7 FT

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTOMATIC) **NOTES:** ETR~70.6%, N_{60} ~ 70.6/60*N~1.17*N

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0	X							*2½ INCHES OF ASPHALT CONCRETE OVER 3 INCHES OF AGGREGATE BASE*	
	X						SC-SM	**YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS (Qya):** *SILTY, CLAYEY SAND; DARK GRAY, MOIST, LOOSE, FINE TO COARSE GRAINED*	SA AL EI RV CR
5	X	X	13	15			SP-SM	*POORLY GRADED SAND WITH SILT; DARK GRAY, VERY MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
7 ▼	X							*GROUNDWATER ENCOUNTERED*	
10		X	15	18				*GRAY TO BROWN, MEDIUM GRAINED*	
15		X	14	16				*GRAY, FINE TO MEDIUM GRAINED, MICACEOUS*	
20		X	19	22			SP	*POORLY GRADED SAND; GRAY, WET, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
								BORING TERMINATED AT 21½ FT. GROUNDWATER ENCOUNTERED AT 7 FT. CAVING TO 7 FT.	
25									
30									

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FIGURE B.5

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LOG OF PERCOLATION BORING P-1

DATE DRILLED: OCTOBER 4, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 27 FT NAVD88 **DRILLING EQUP.:** CME 95 **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: BULK SAMPLES **NOTES:** N/A

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0								*VEGETATED SURFACE*	
	X						SP-SM	**YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS (Qya):** *POORLY GRADED SAND WITH SILT; OLIVE BROWN TO GRAY, MOIST, LOOSE, FINE TO MEDIUM GRAINED, MICACEOUS*	SA
5								*BORING TERMINATED AT 5 FT AND CONVERTED TO PERCOLATION TEST WELL. GROUNDWATER NOT ENCOUNTERED.*	
10									
15									
20									
25									
30									

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FIGURE B.6

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LOG OF PERCOLATION BORING P-2

DATE DRILLED: OCTOBER 4, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: ± 27 FT NAVD88 **DRILLING EQUP.:** CME 95 **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: BULK SAMPLES **NOTES:** N/A

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0								*VEGETATED SURFACE*	
	X						SP	**YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS (Qya):** *POORLY GRADED SAND; OLIVE BROWN TO GRAY, MOIST, LOOSE, FINE TO MEDIUM GRAINED, MICACEOUS*	
5								*BORING TERMINATED AT 5 FT AND CONVERTED TO PERCOLATION TEST WELL. GROUNDWATER NOT ENCOUNTERED.*	
10									
15									
20									
25									
30									

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FIGURE B.7

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APPENDIX C
CPT DATA AND LIQUEFACTION ANALYSIS

TABLE OF CONTENTS

NOVA Services, Inc.
4373 Viewridge Avenue, Suite B
San Diego, CA 92123

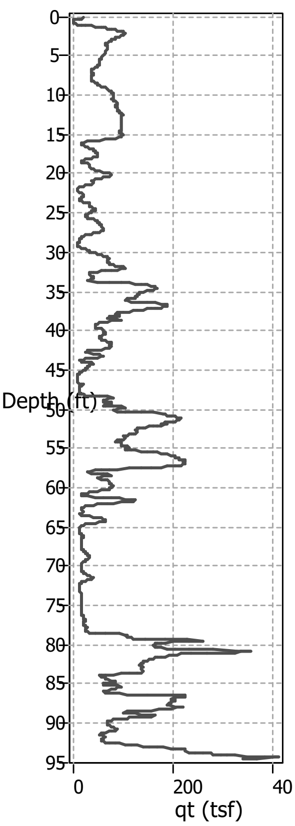
LIQUEFACTION ANALYSIS REPORT

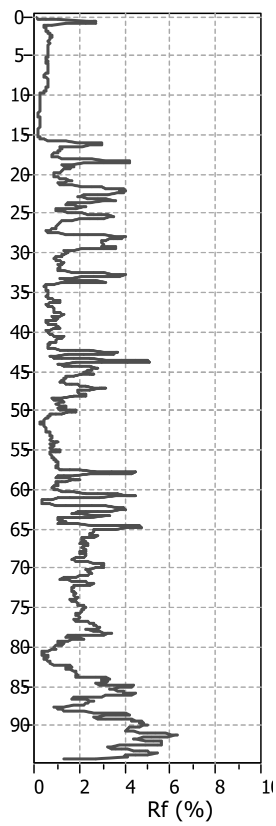
Project title : Eddy Jones Warehouse **Location : 630 Eddy Jones, Oceanside, CA**

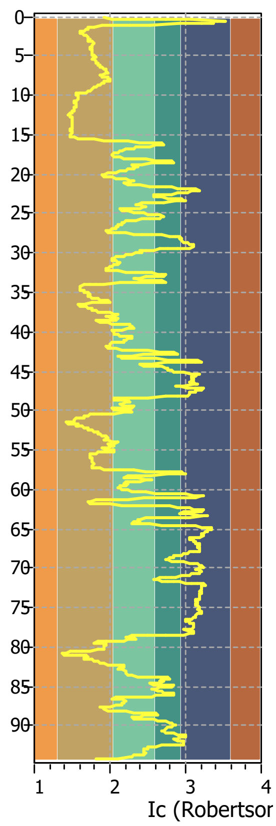
CPT file : CPT-1

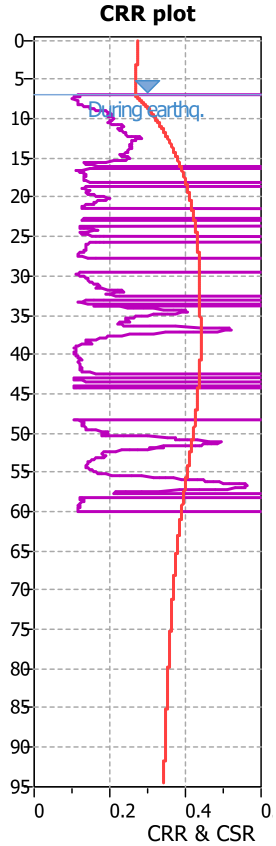
Input parameters and analysis data

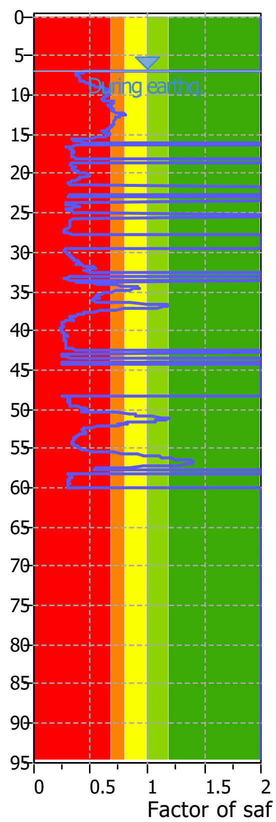
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

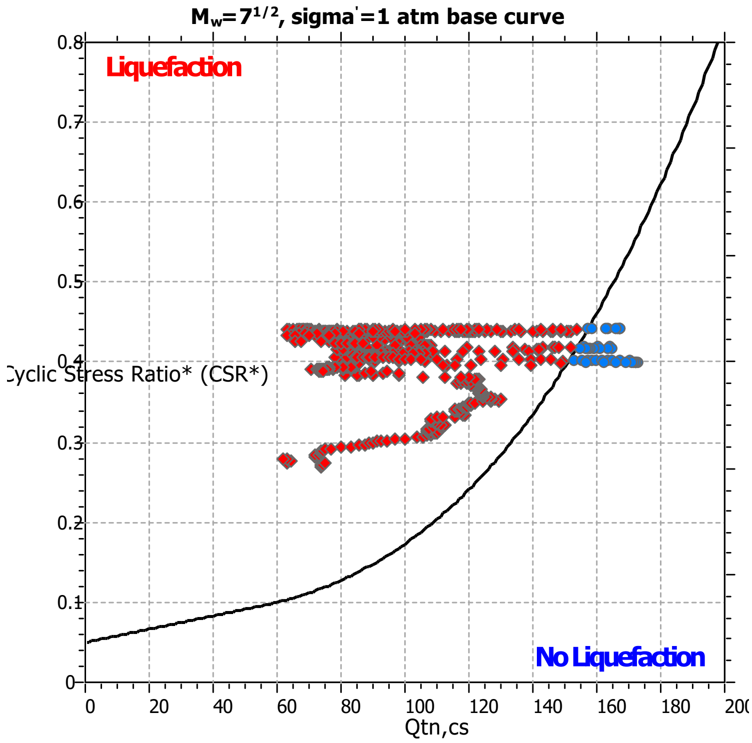


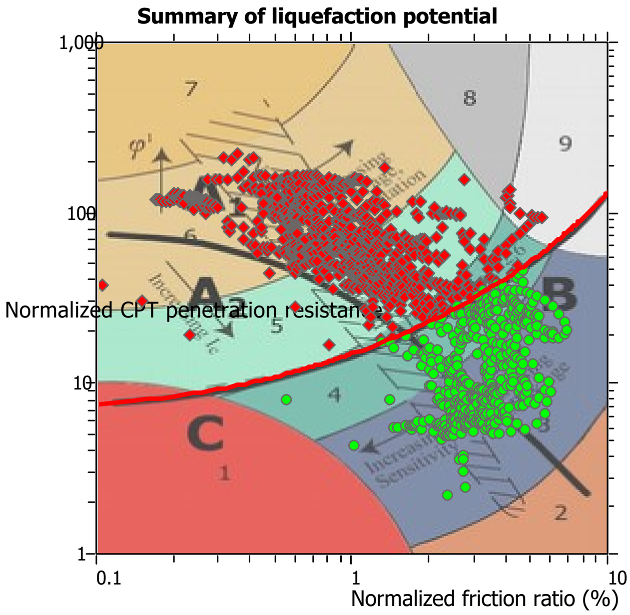






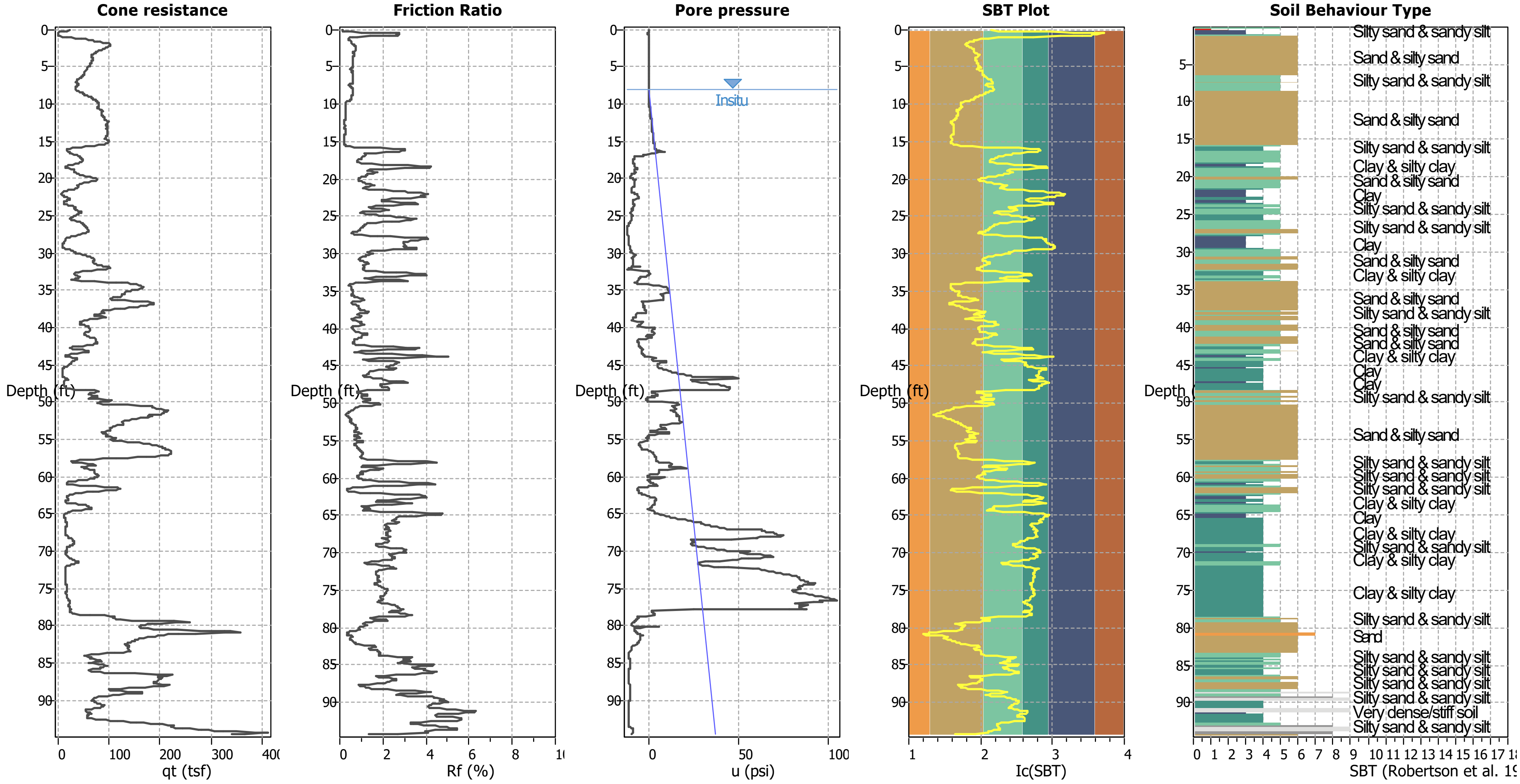






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



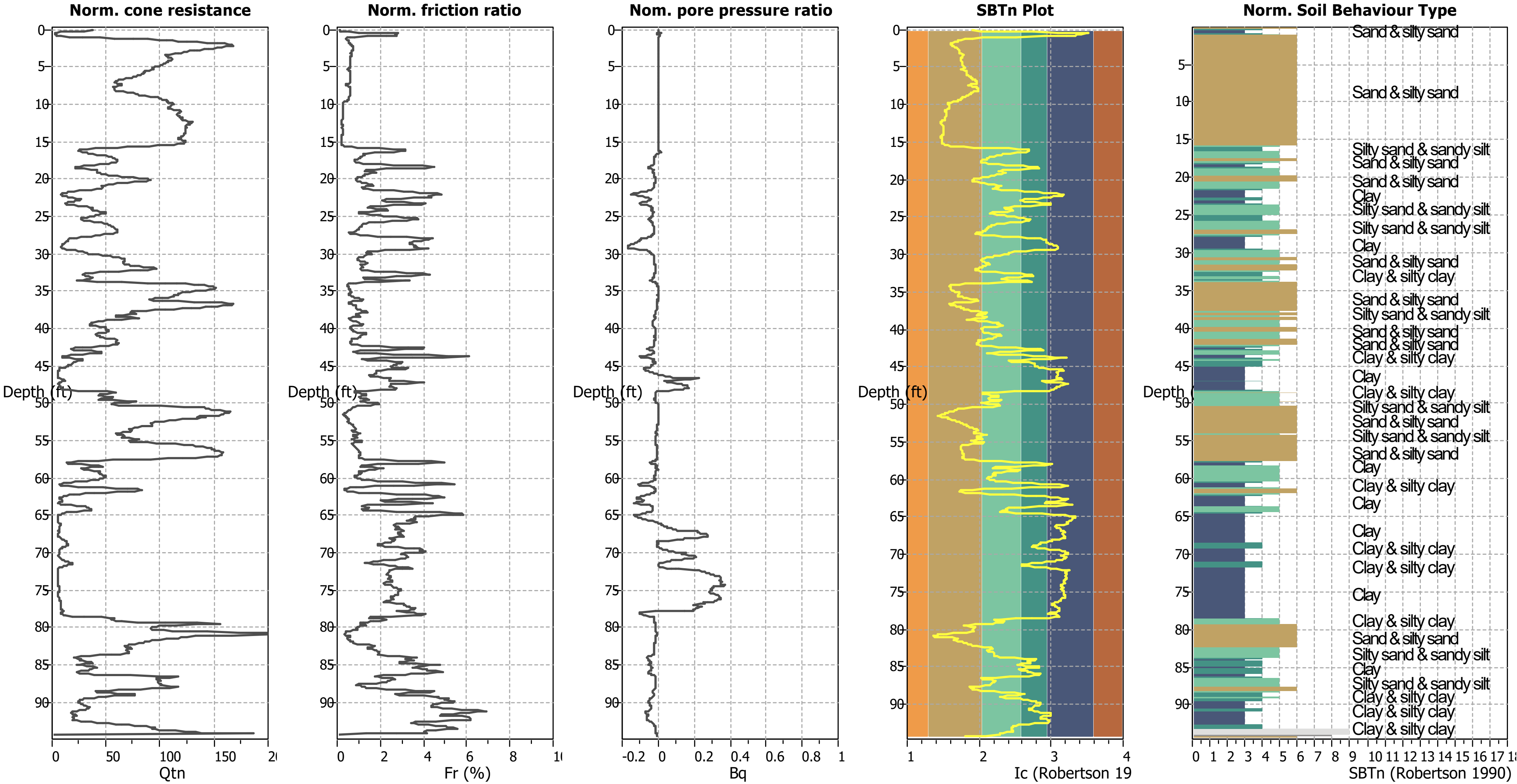
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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)



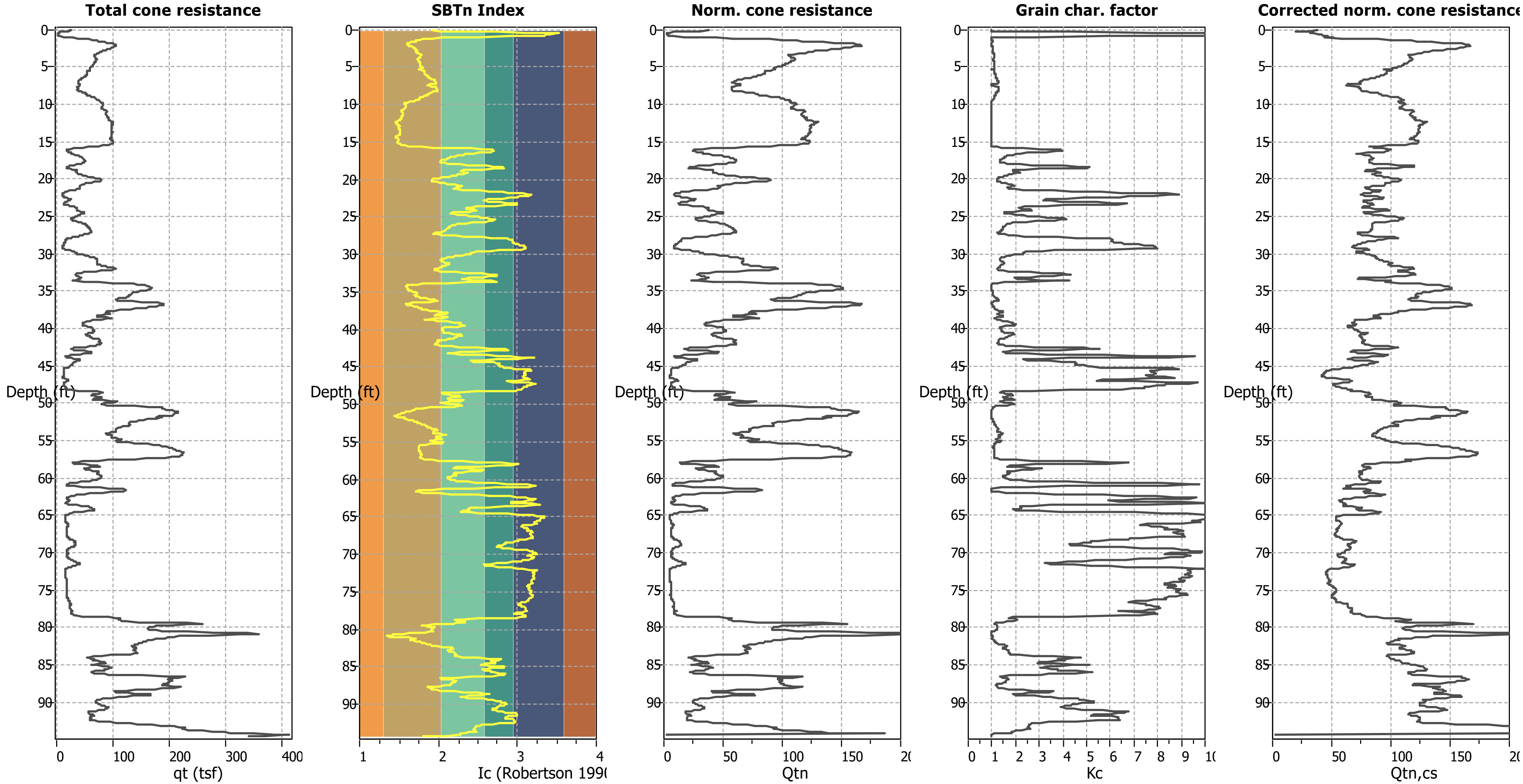
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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

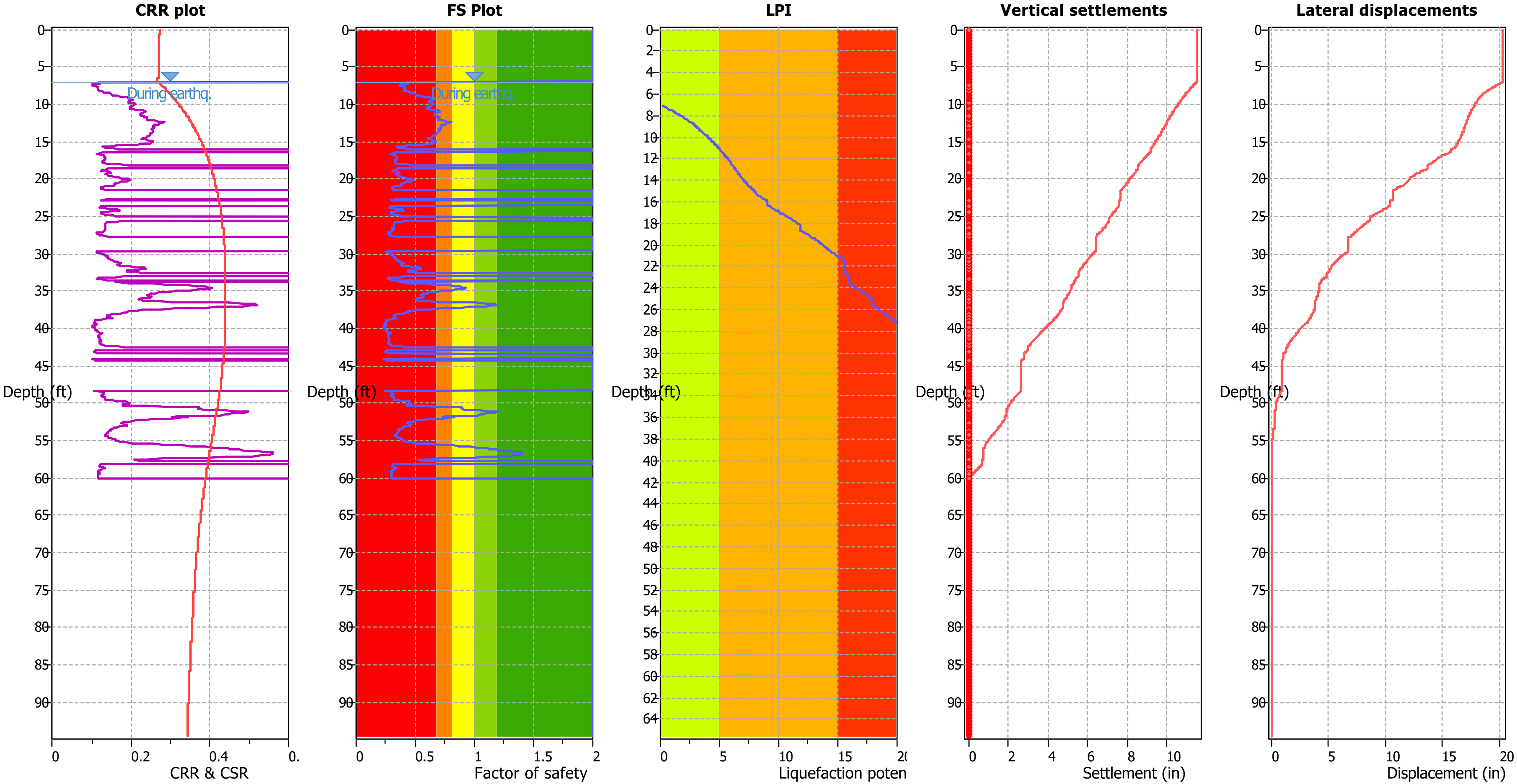
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

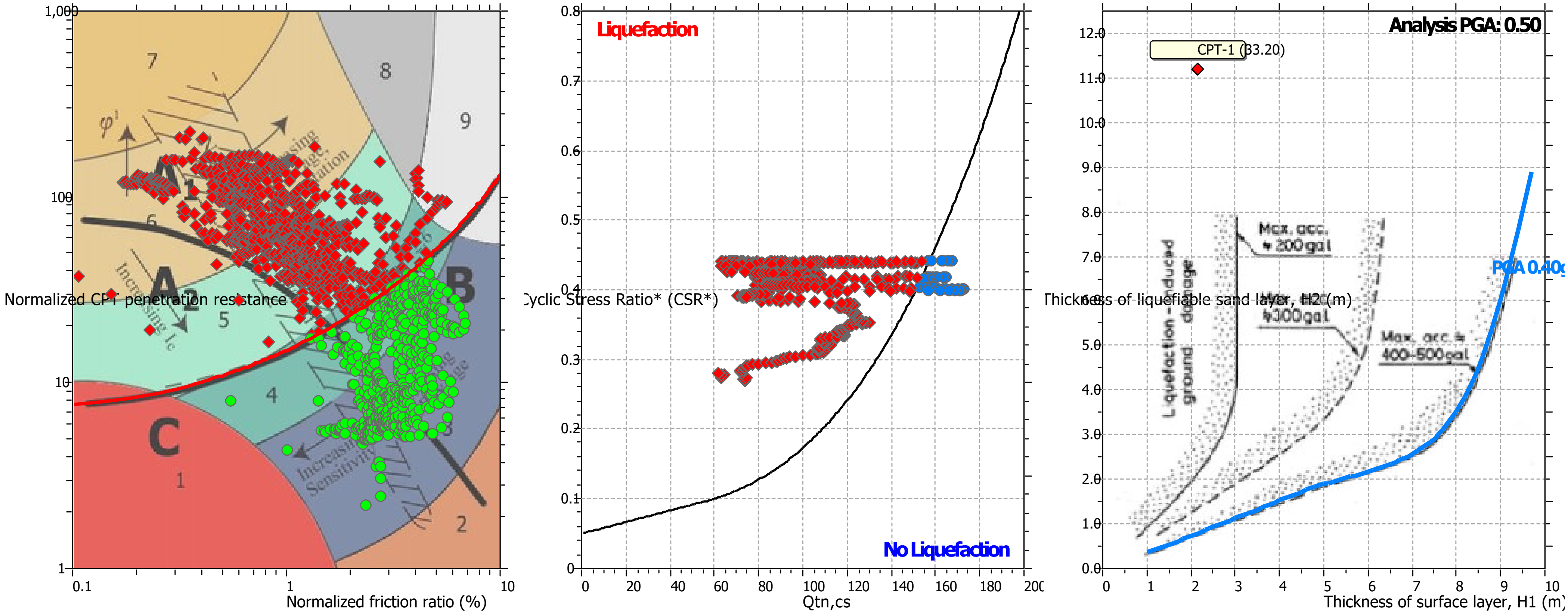
F.S. color scheme

- 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

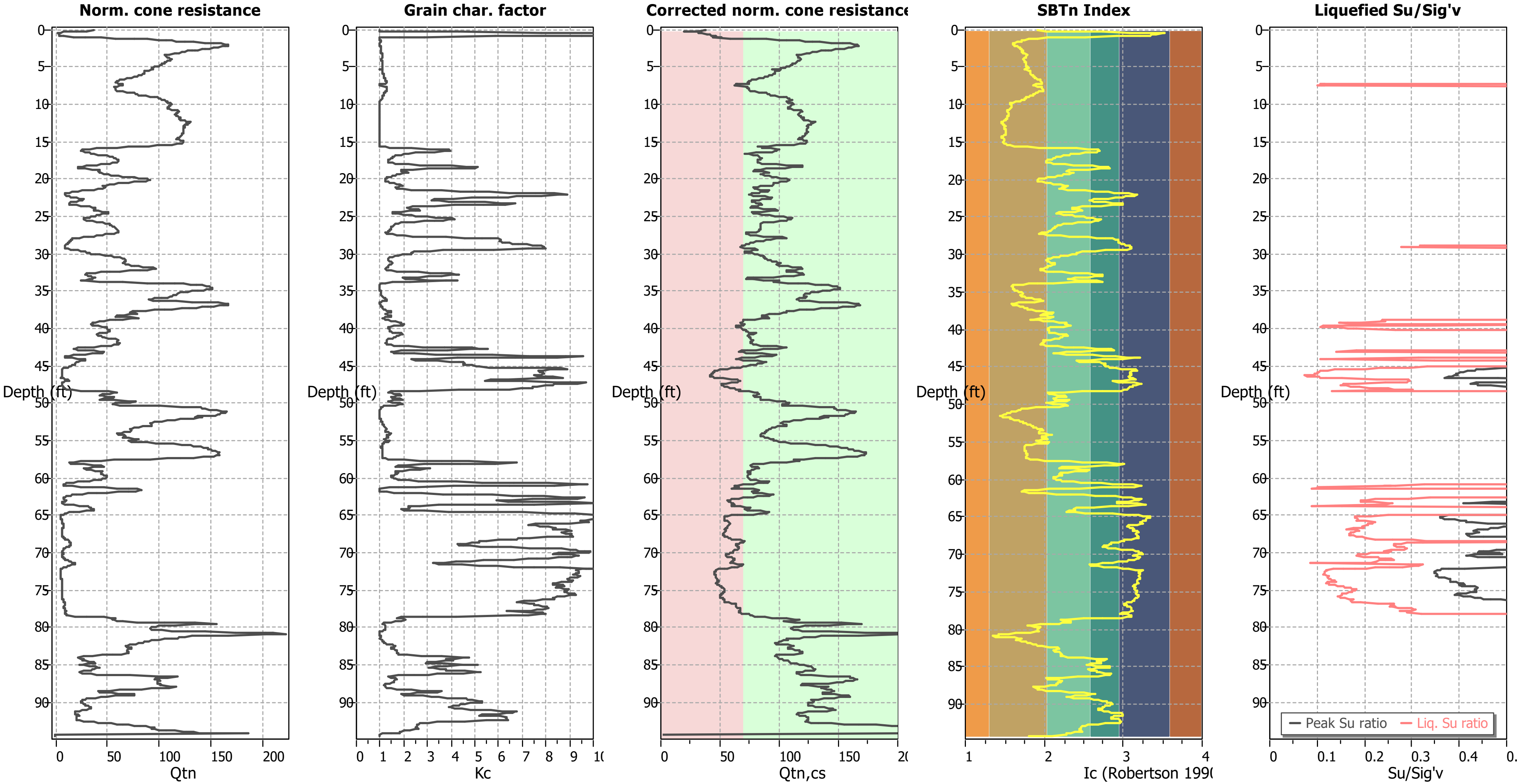
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

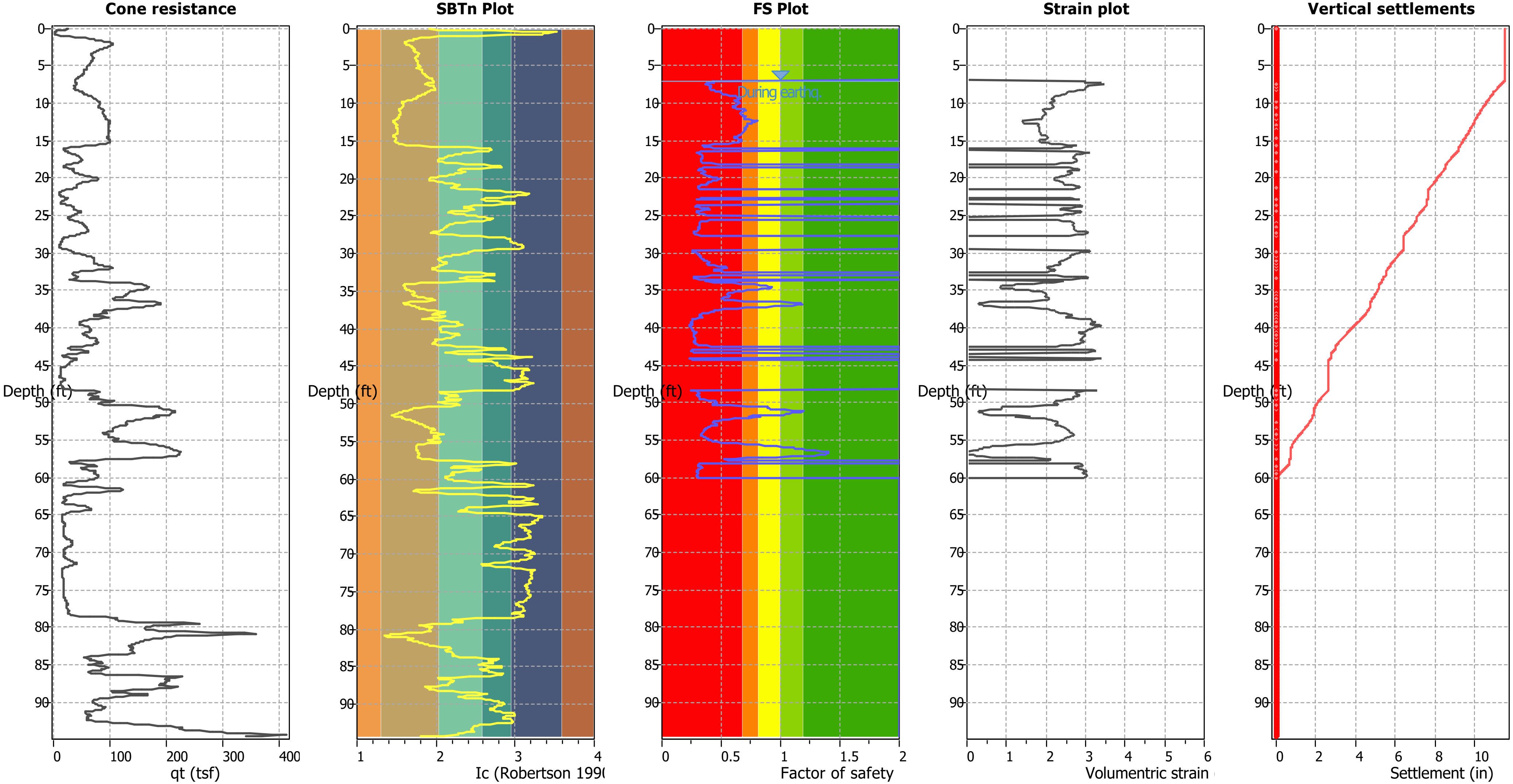
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements

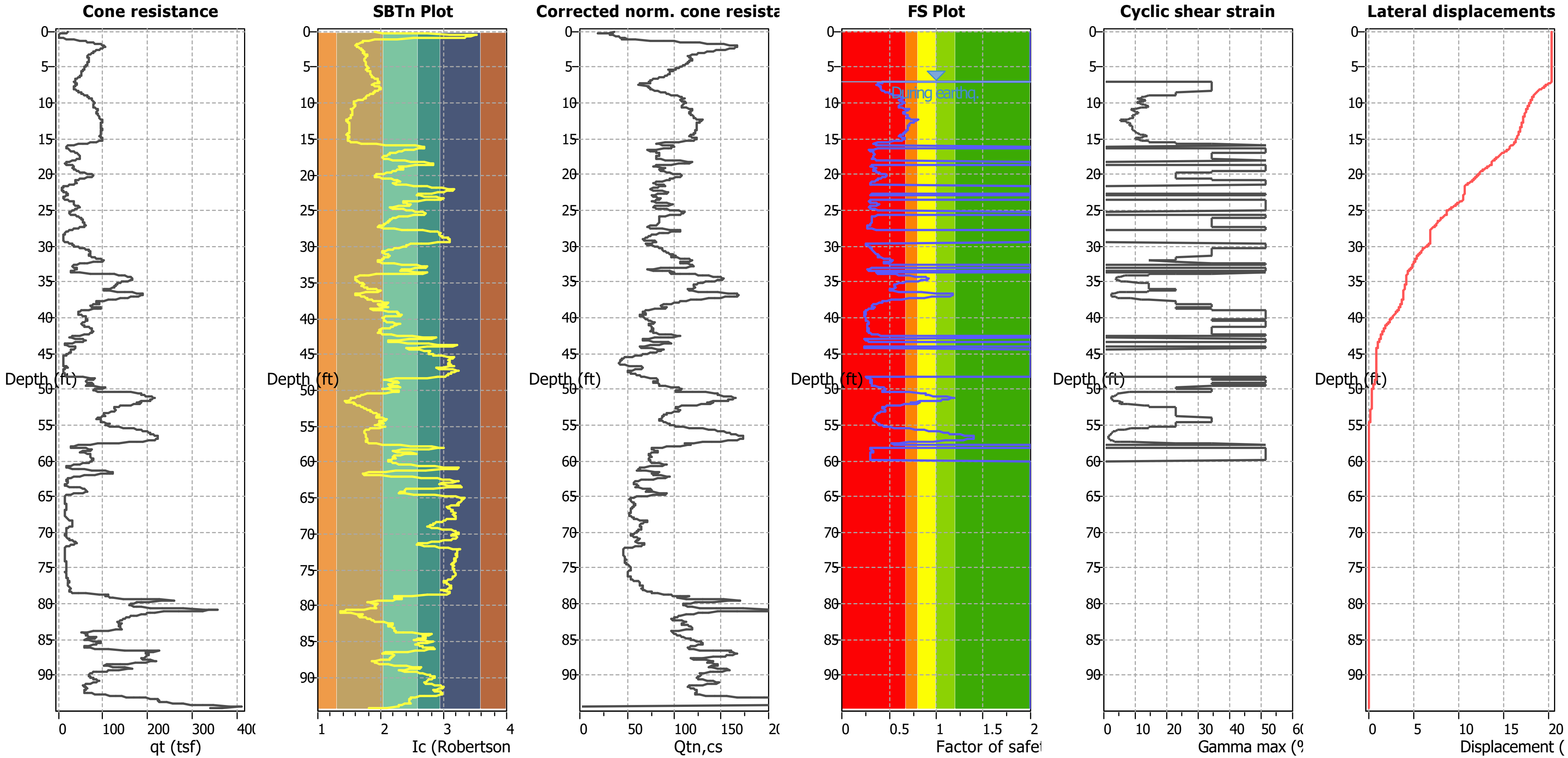


Abbreviations

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

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 0.12 %)

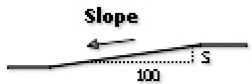


Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



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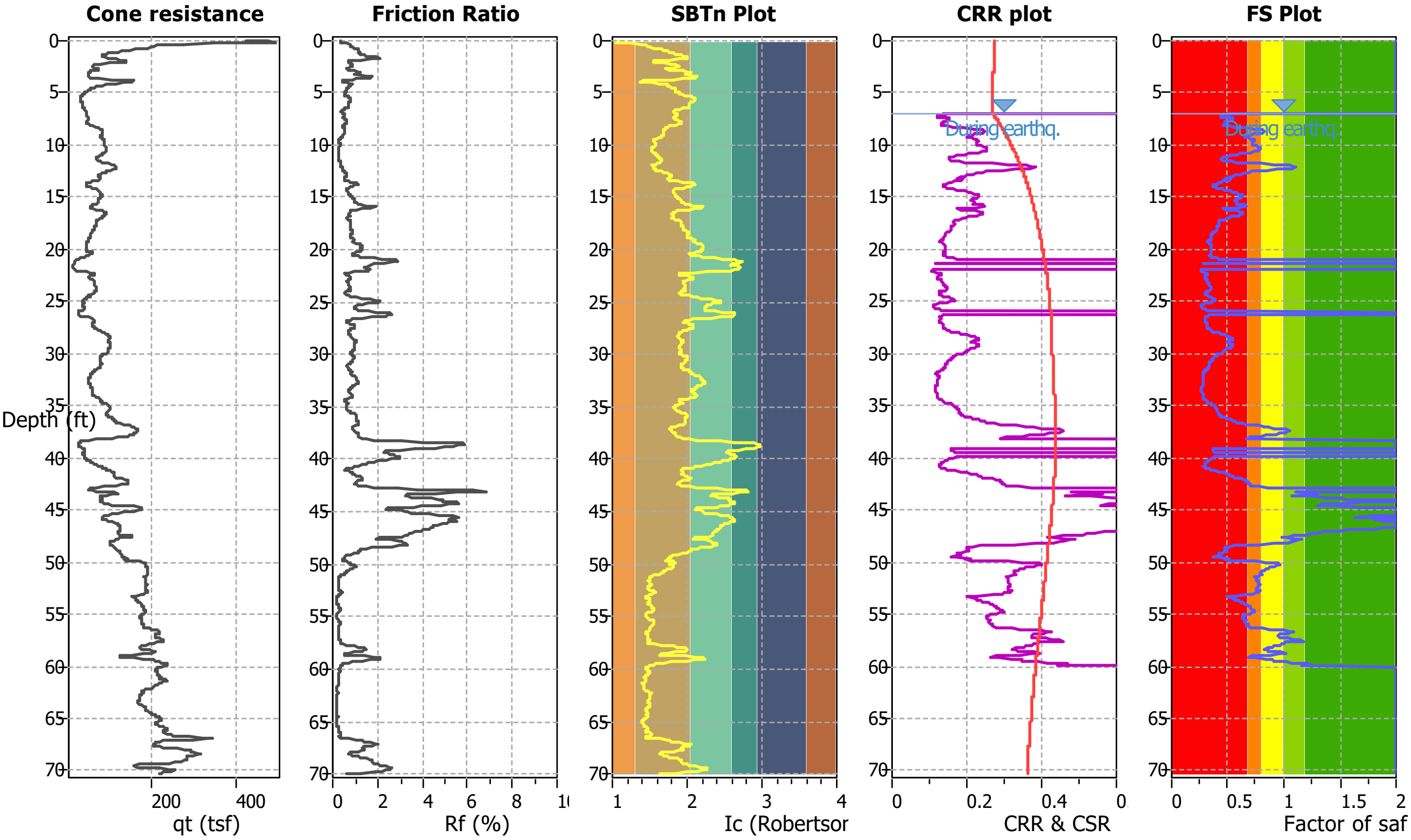
LIQUEFACTION ANALYSIS REPORT

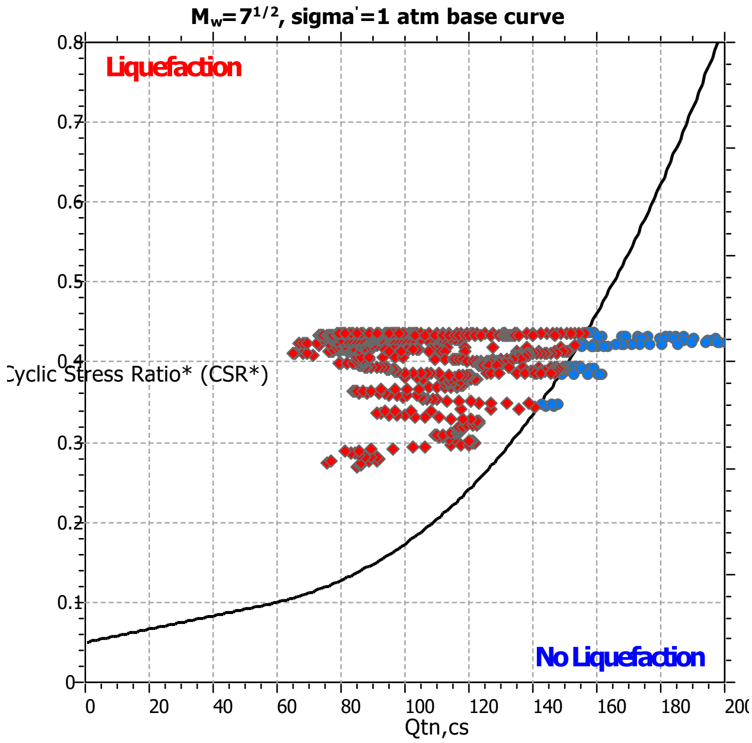
Project title : Eddy Jones Warehouse **Location : 630 Eddy Jones, Oceanside, CA**

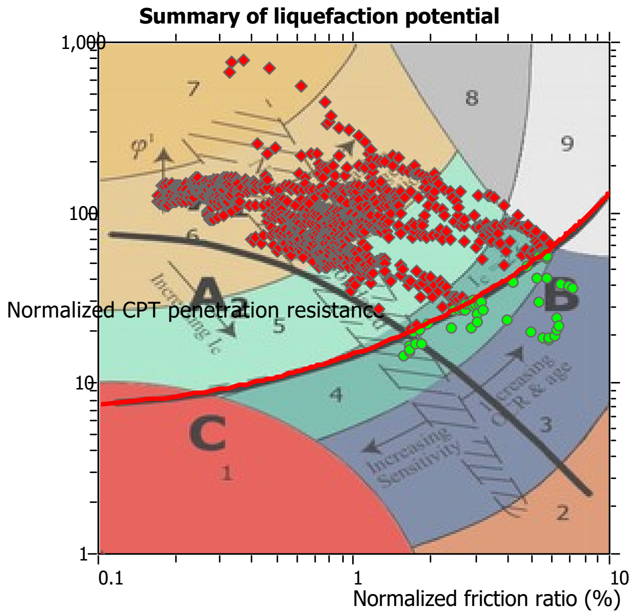
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

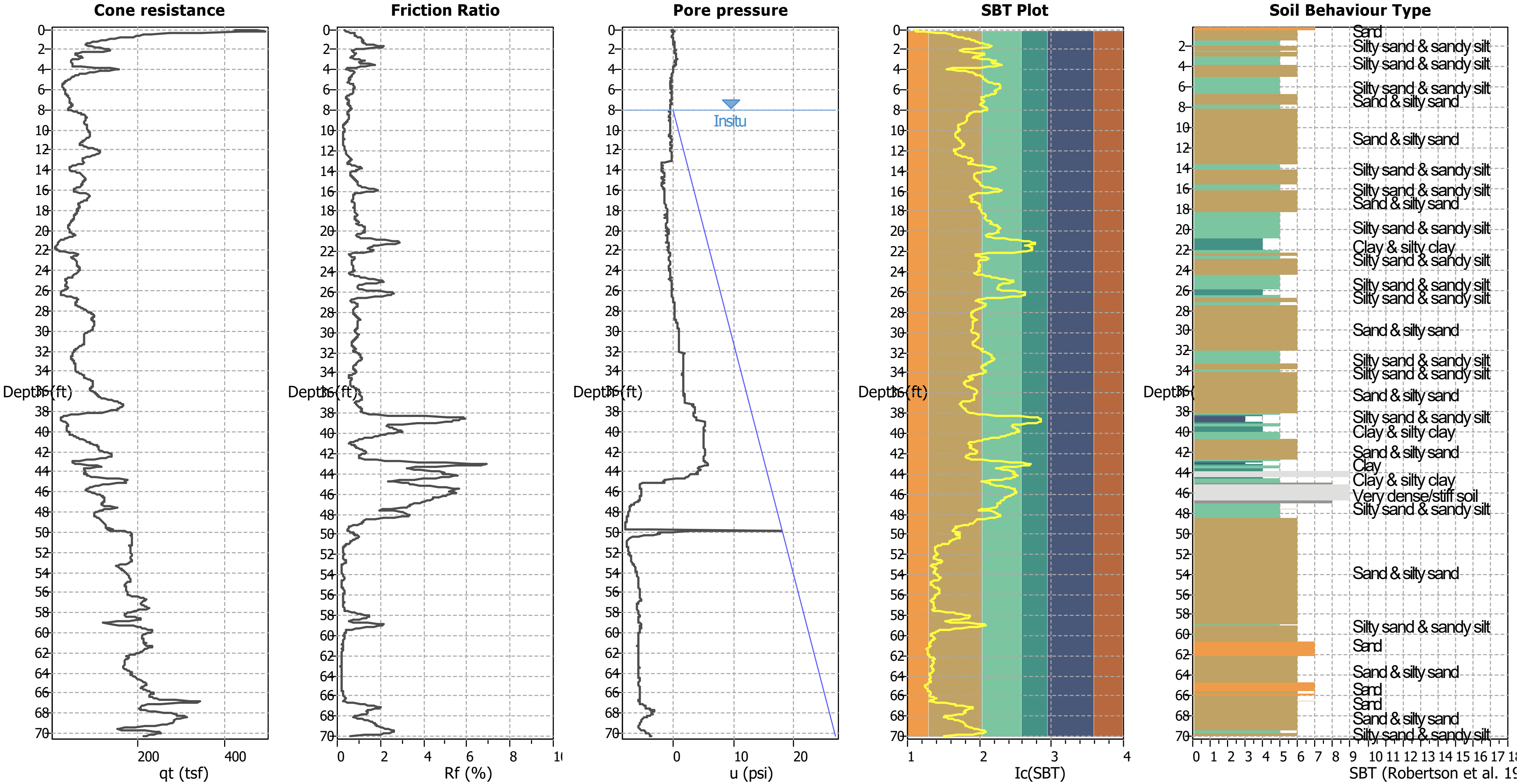






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



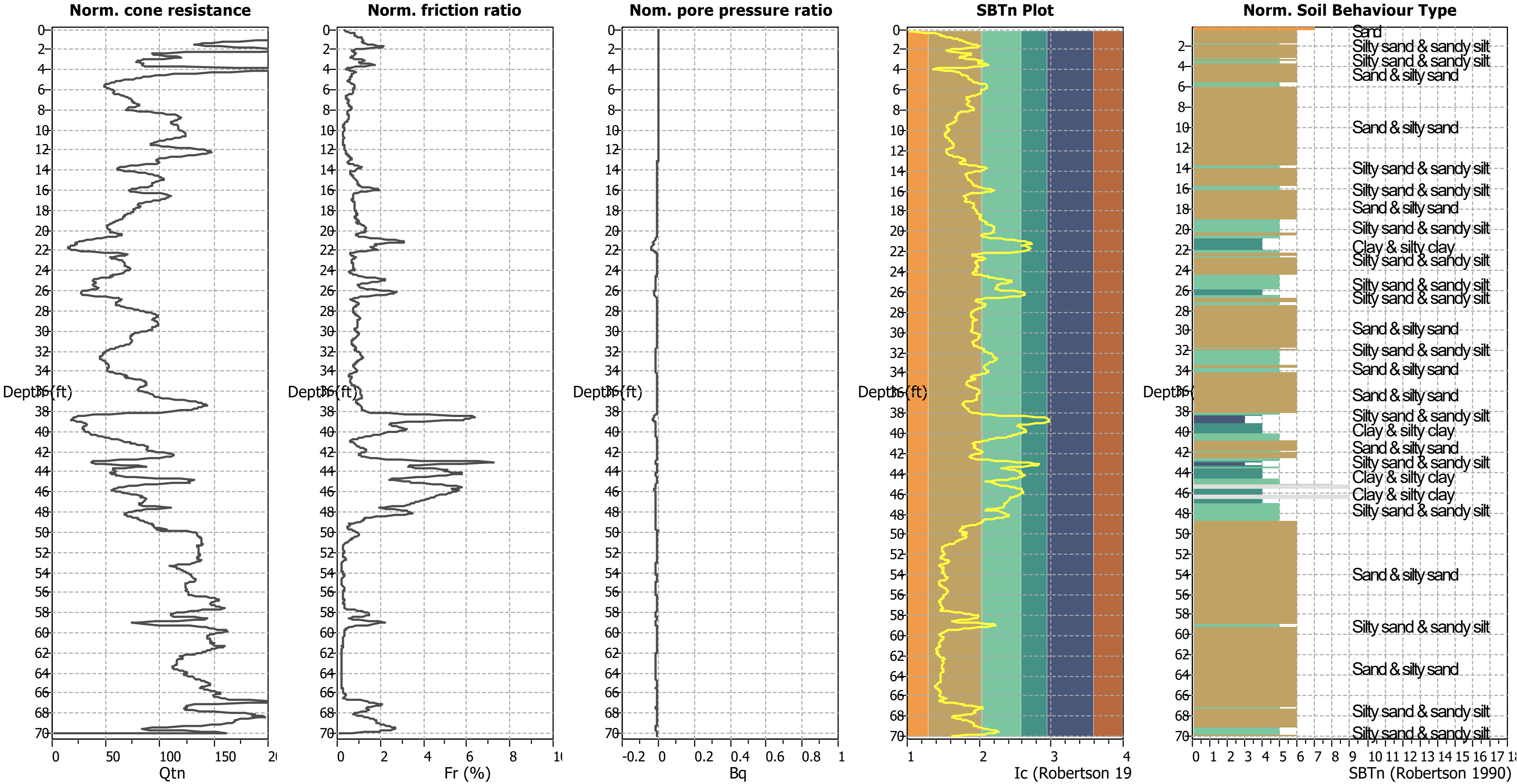
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

- 1. Sensitive fine grained
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- 3. Clay to silty clay
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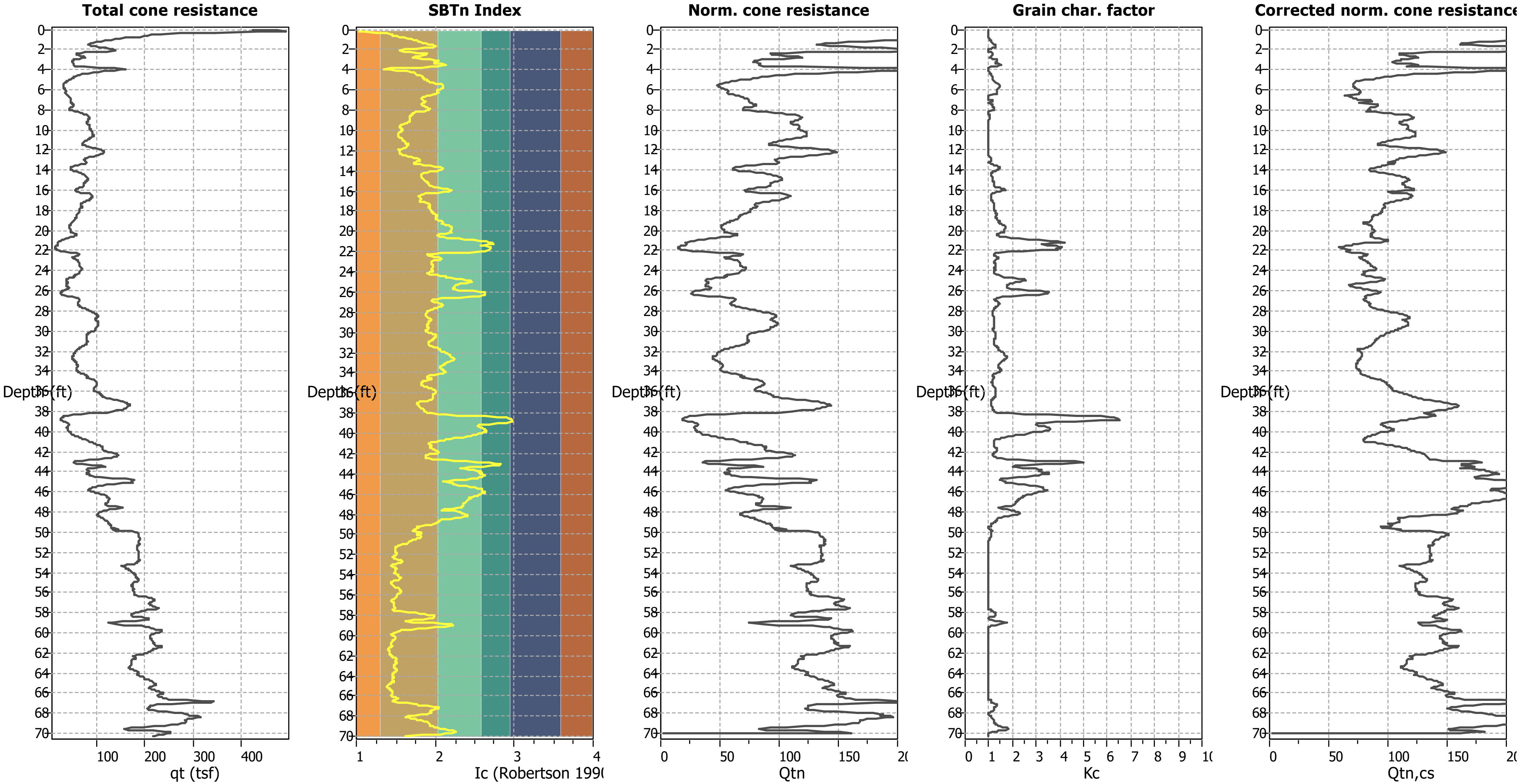
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
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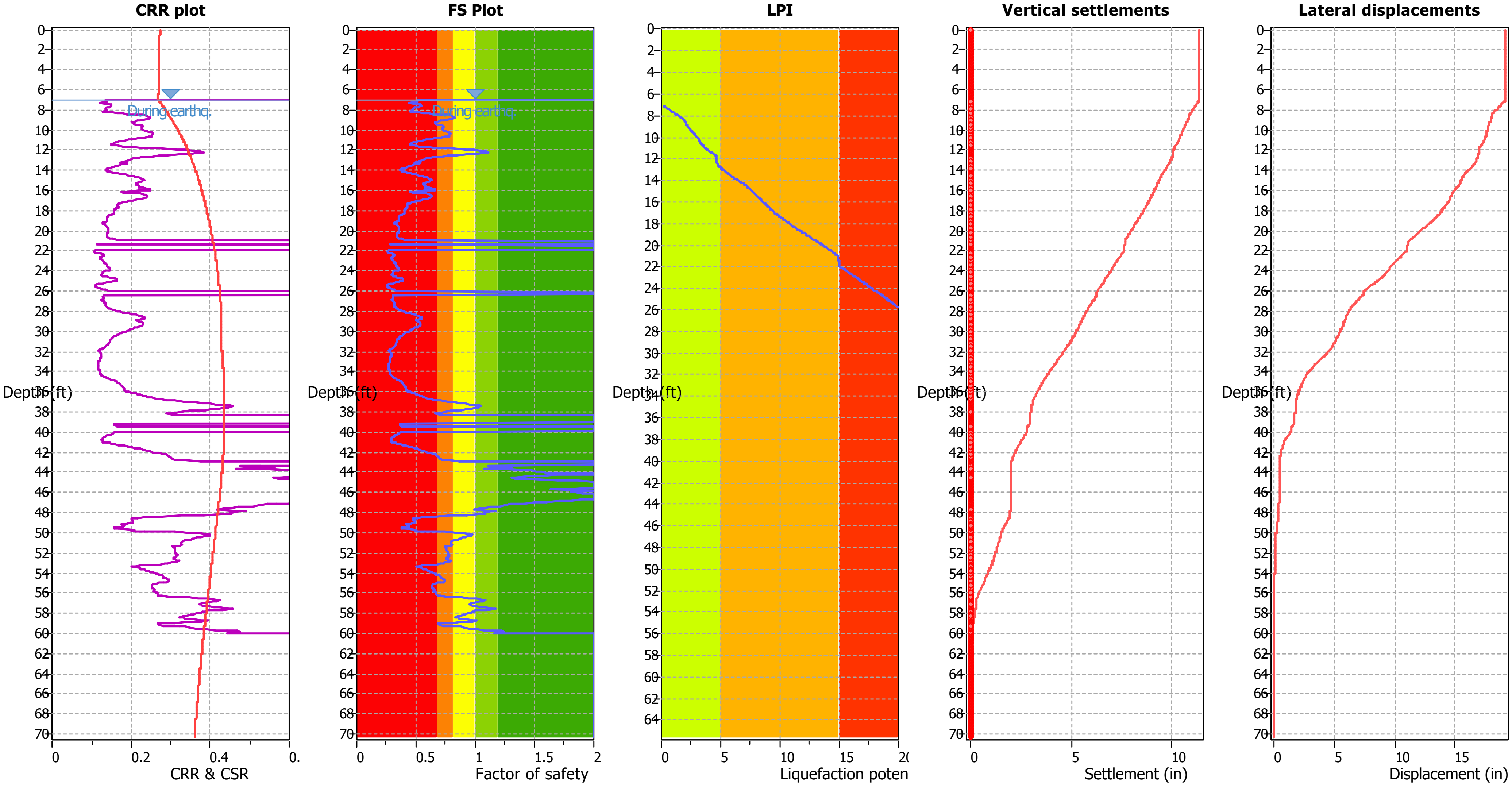
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

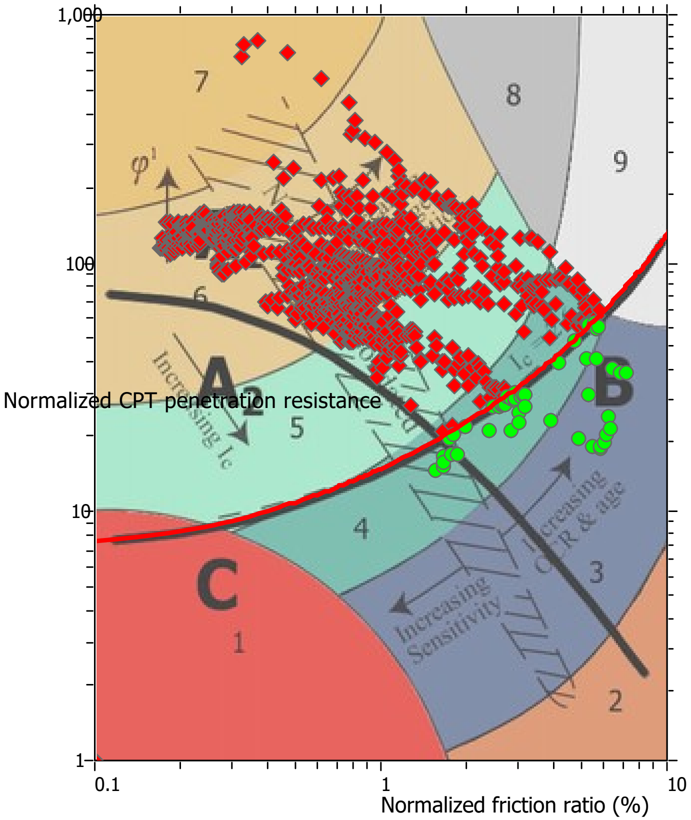
F.S. color scheme

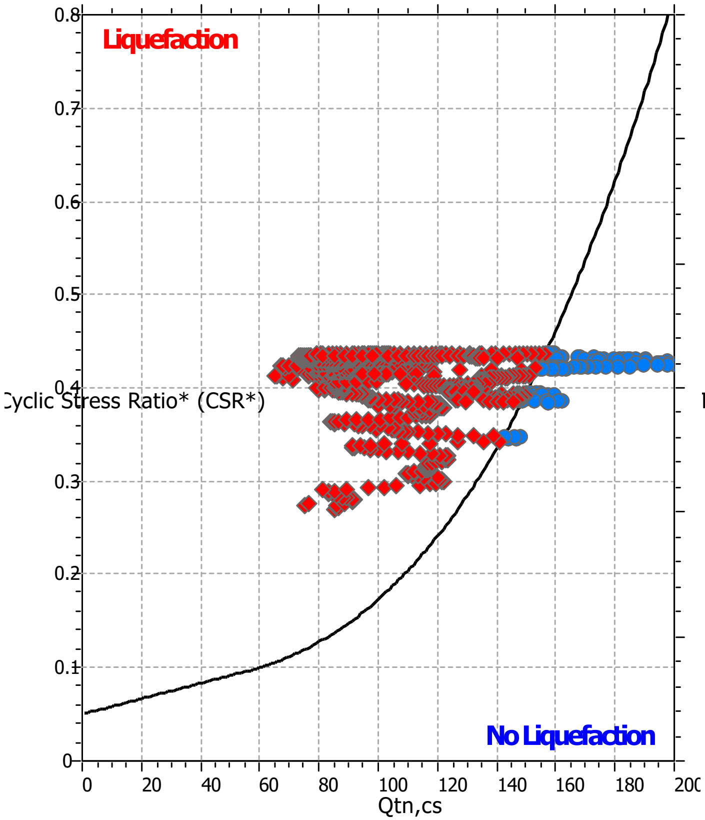
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

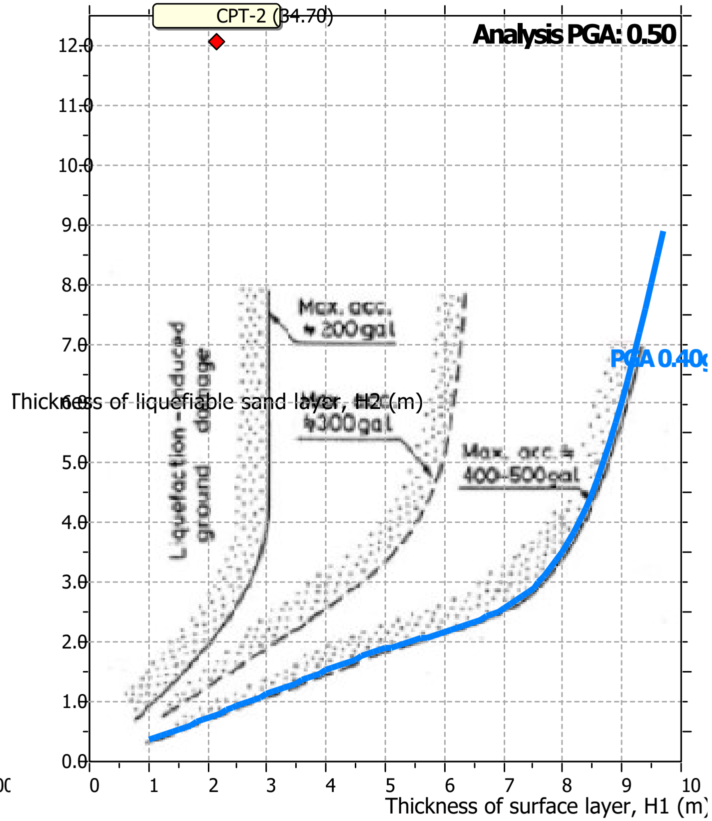
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



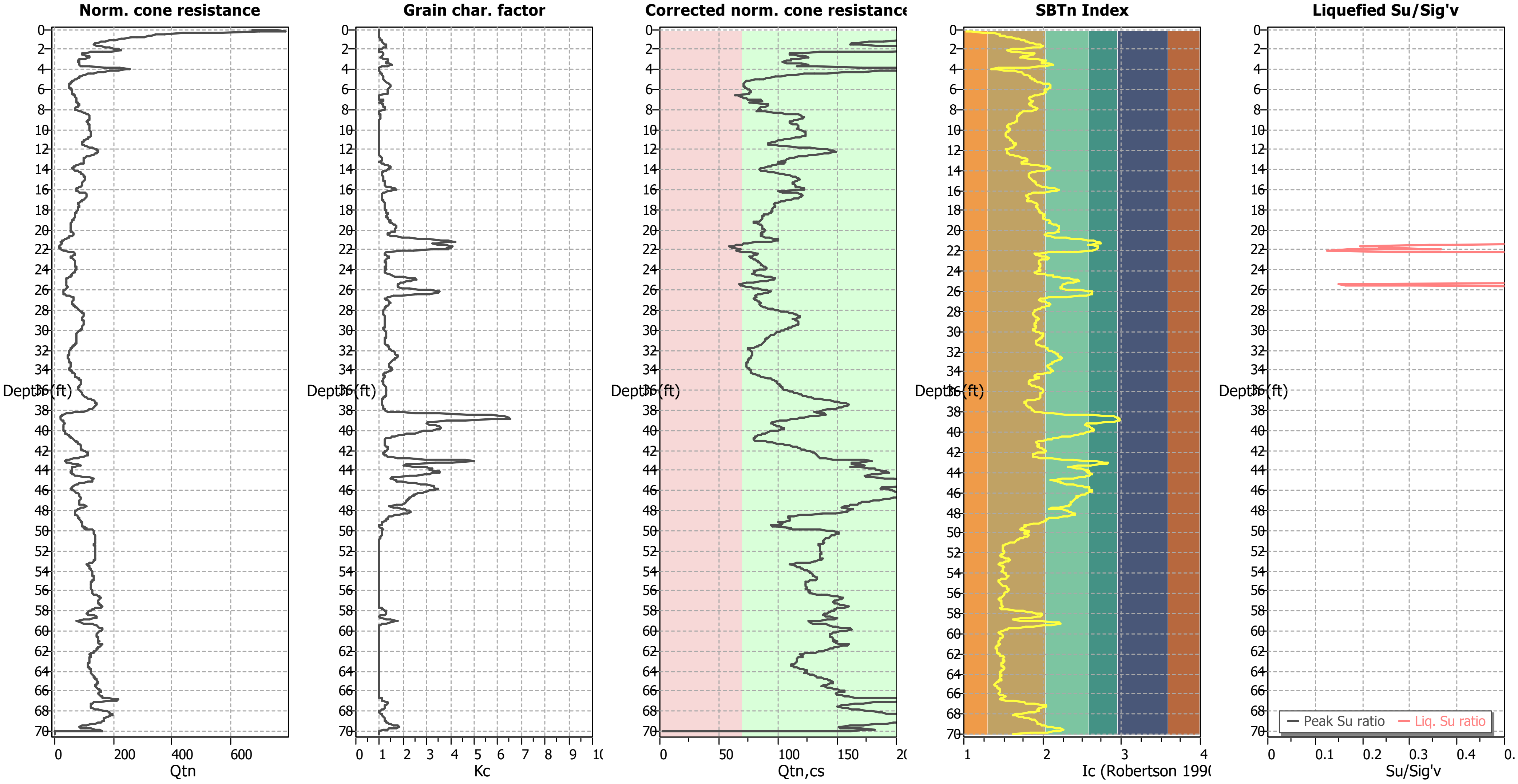




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

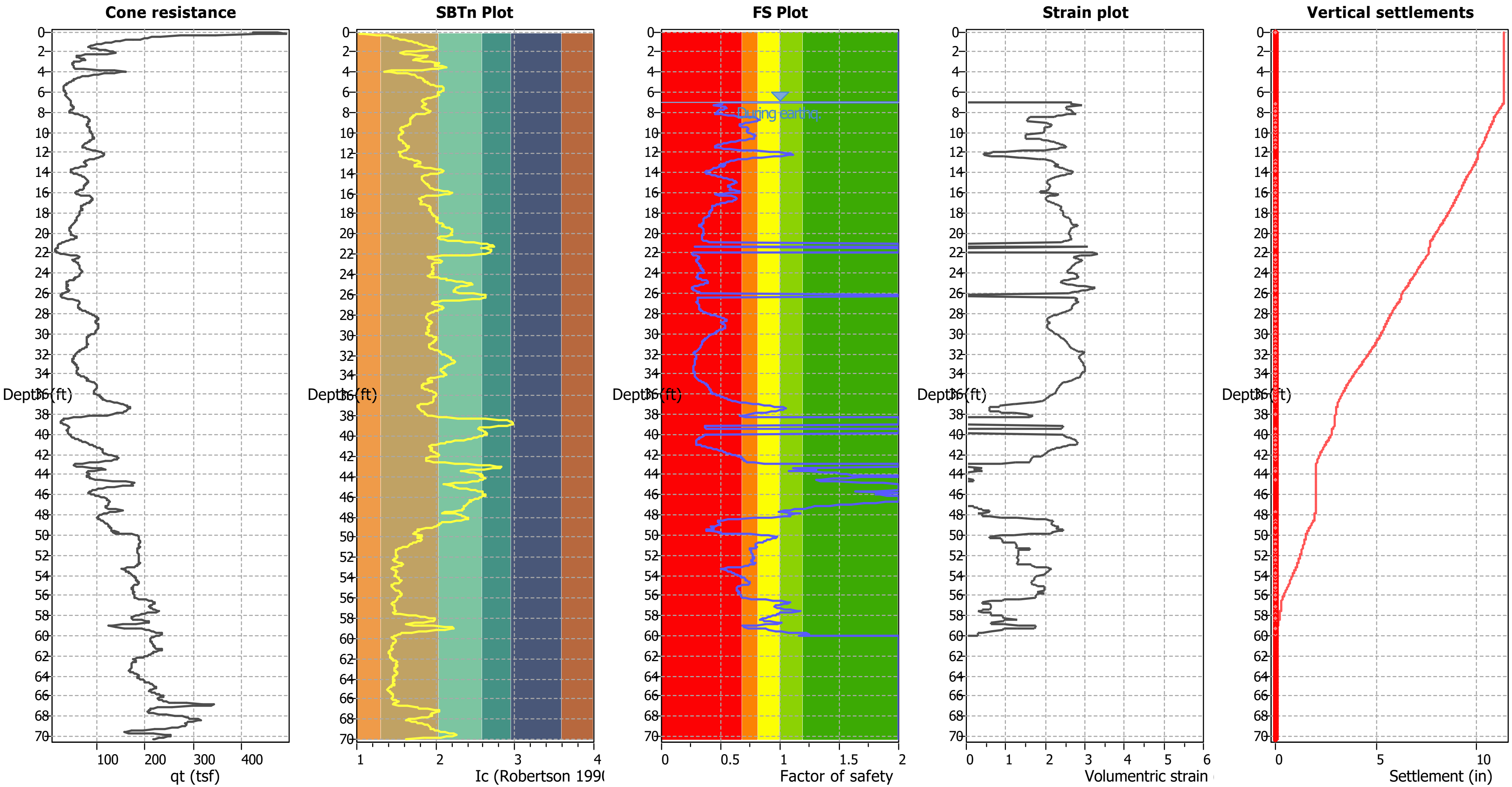
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements

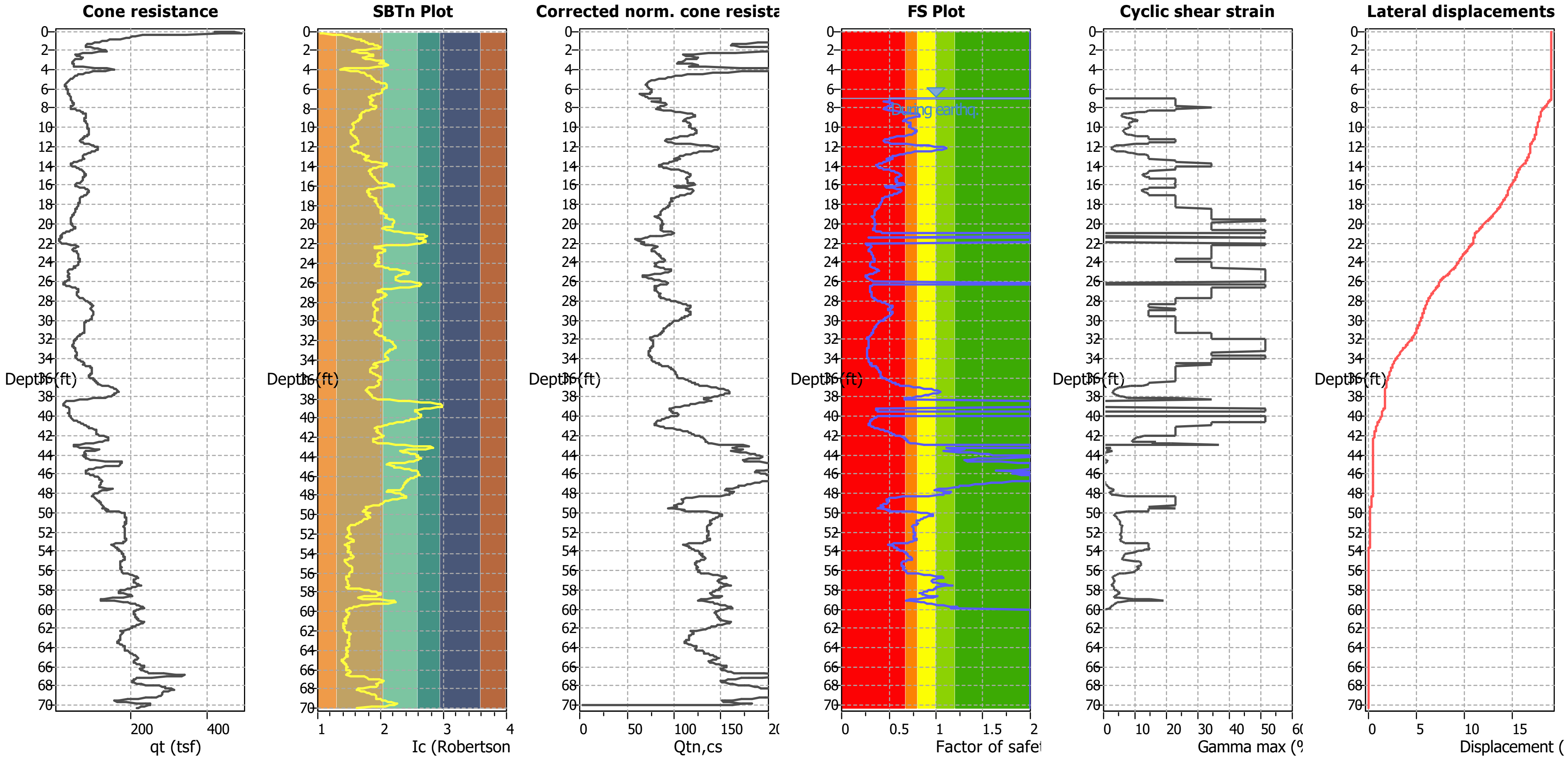


Abbreviations

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

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 0.12 %)

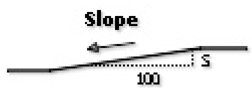


Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition



NOVA

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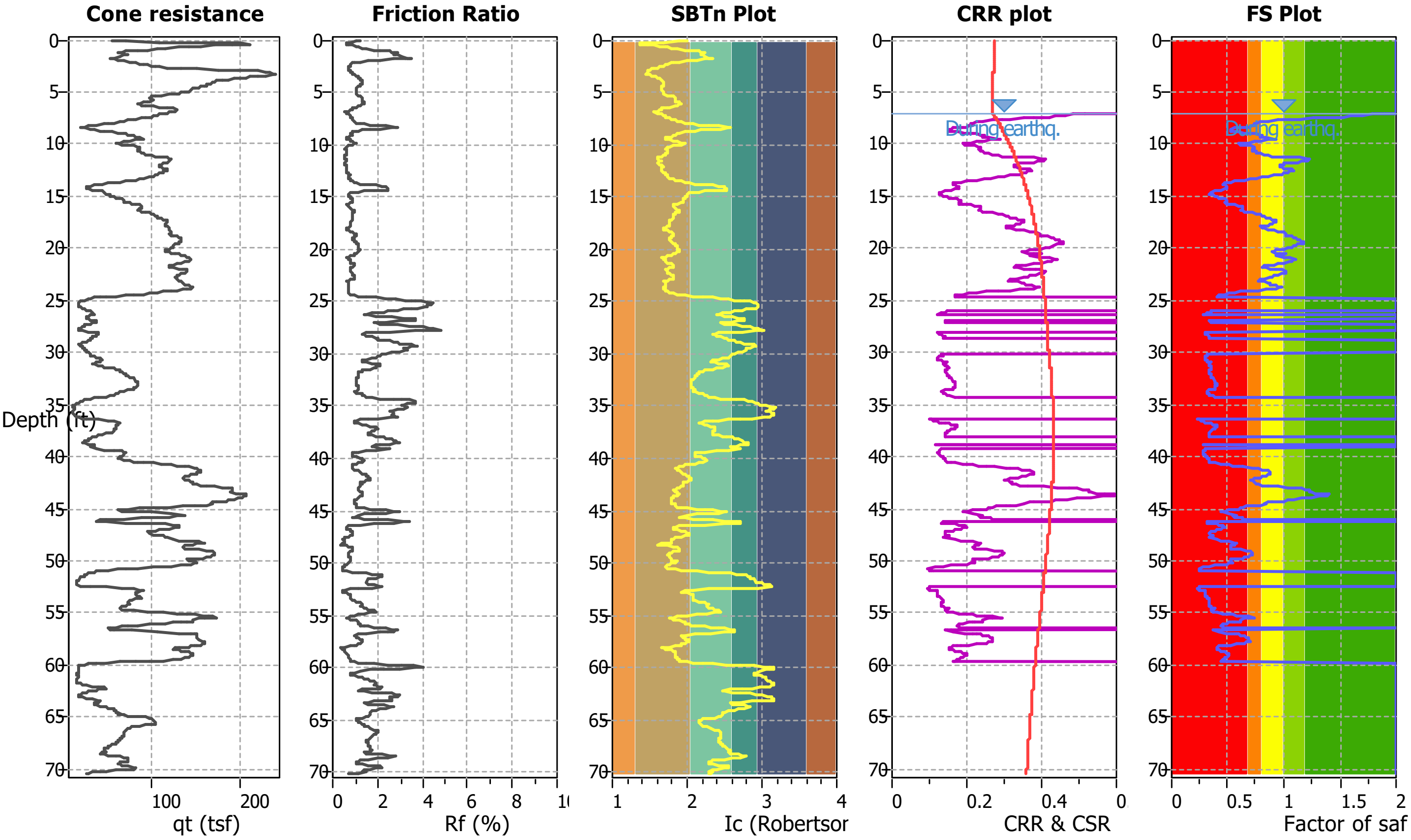
LIQUEFACTION ANALYSIS REPORT

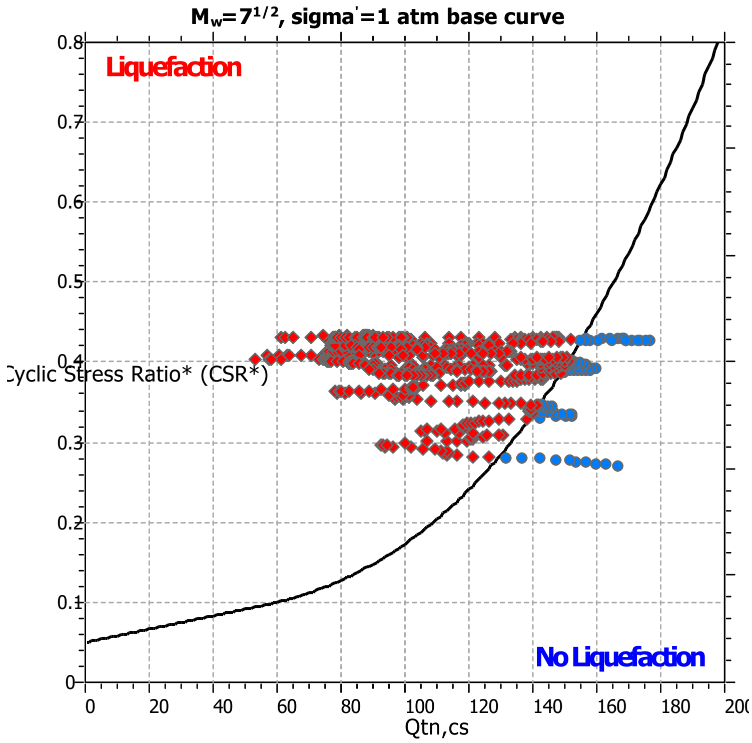
Project title : Eddy Jones Warehouse **Location : 630 Eddy Jones, Oceanside, CA**

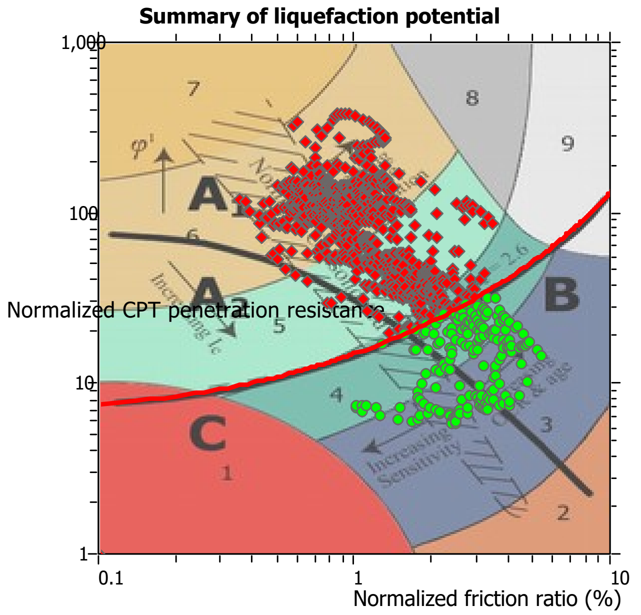
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

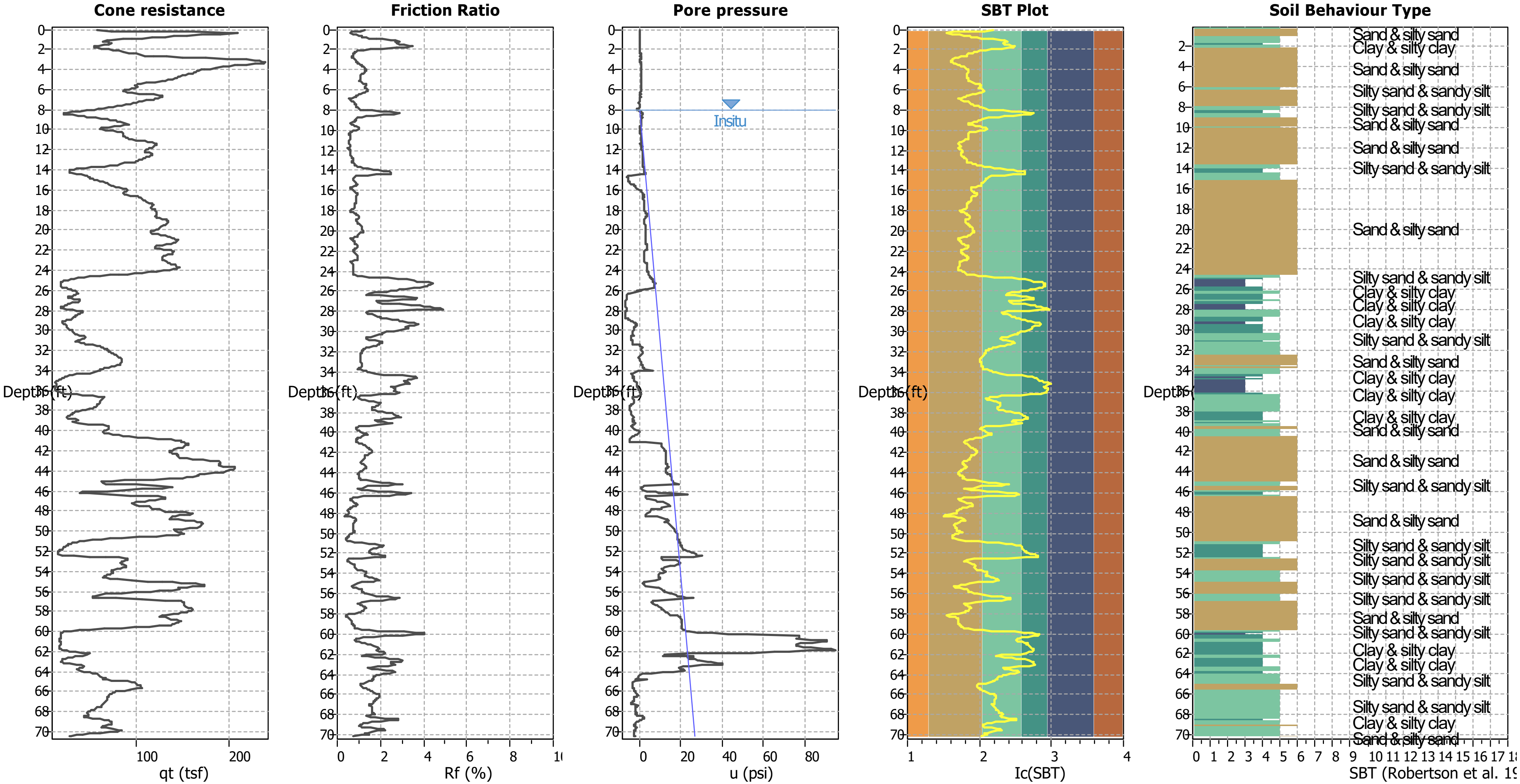






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



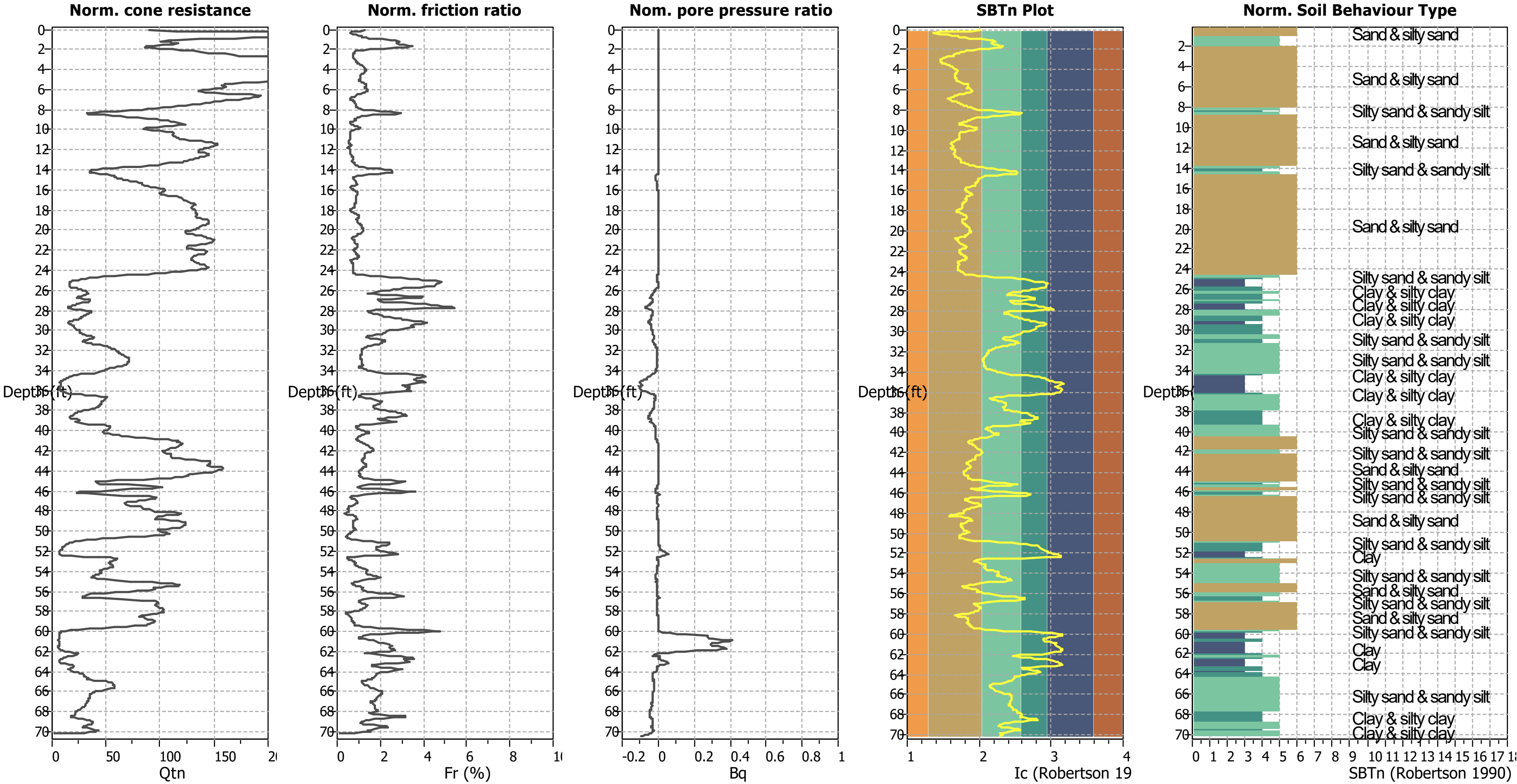
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



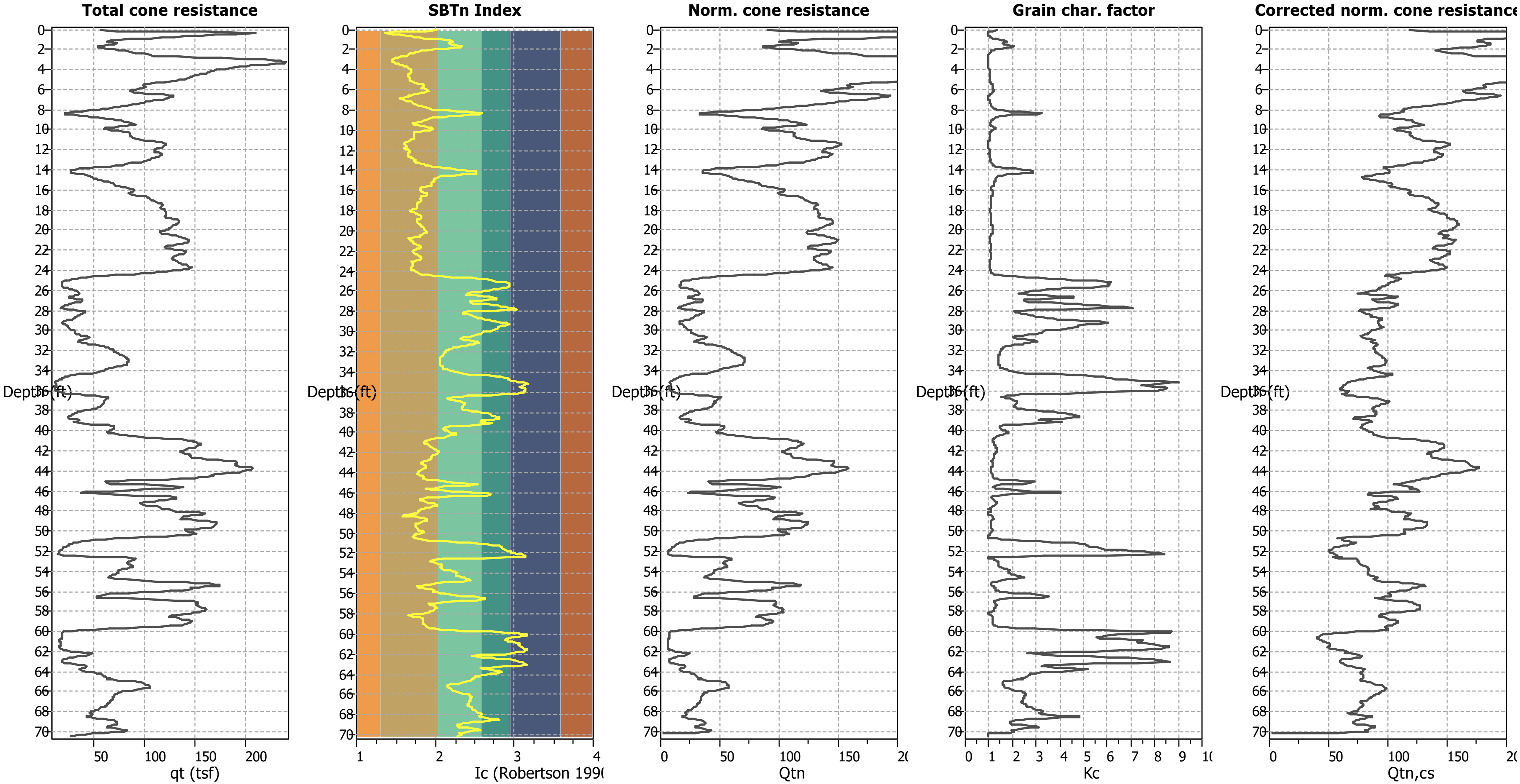
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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

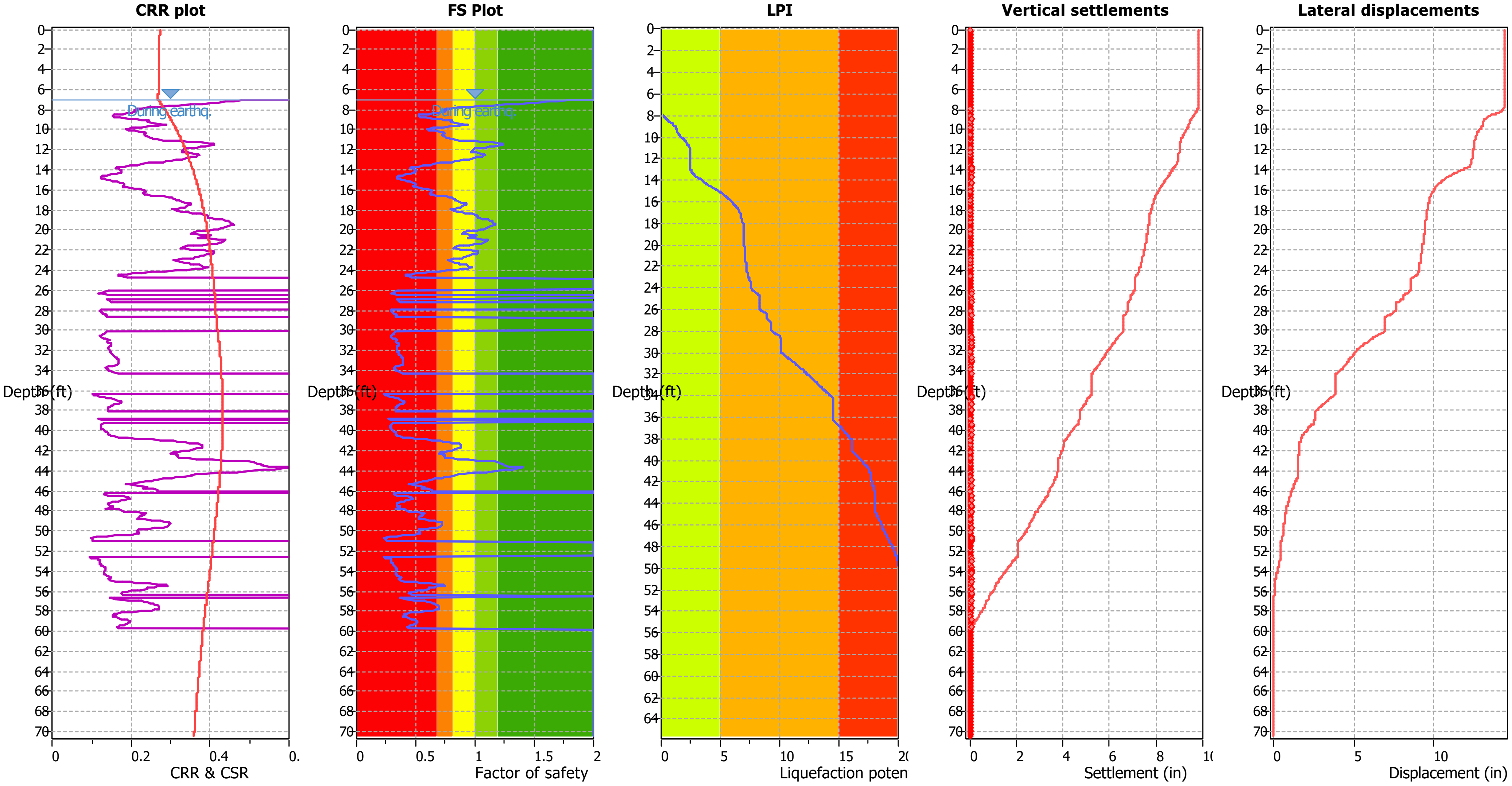
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

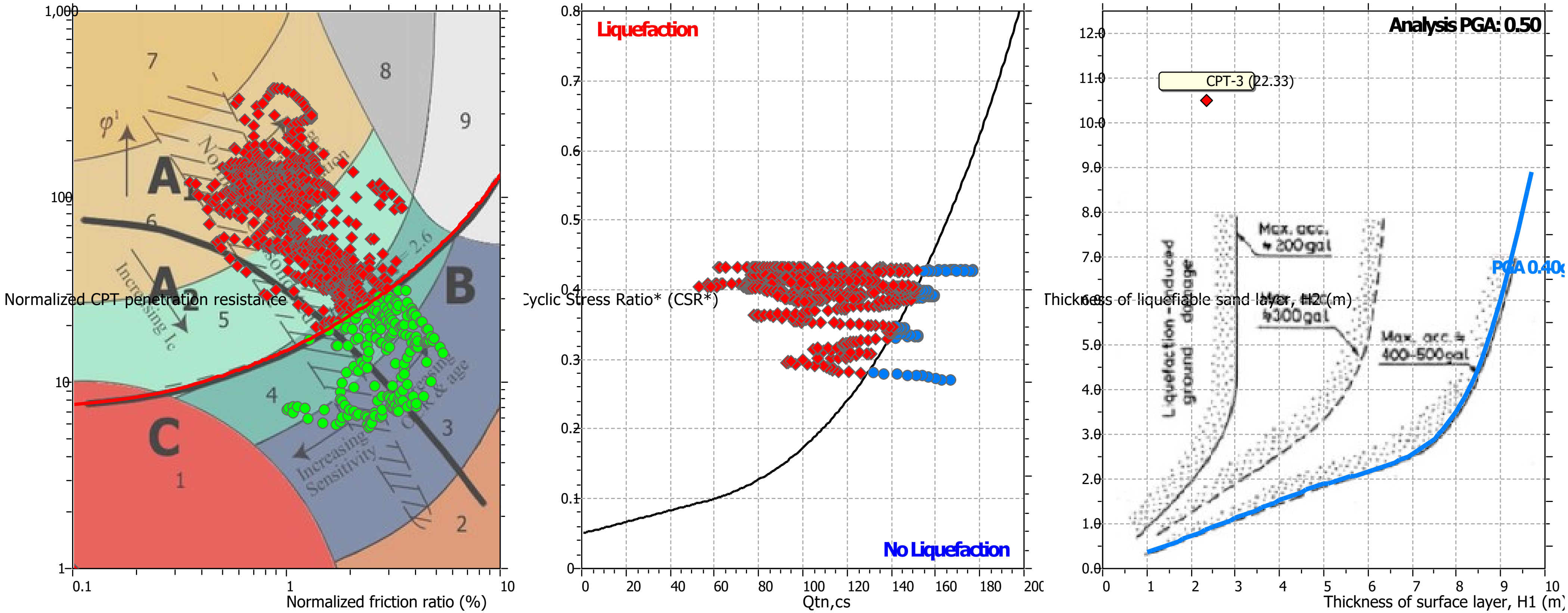
F.S. color scheme

- 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

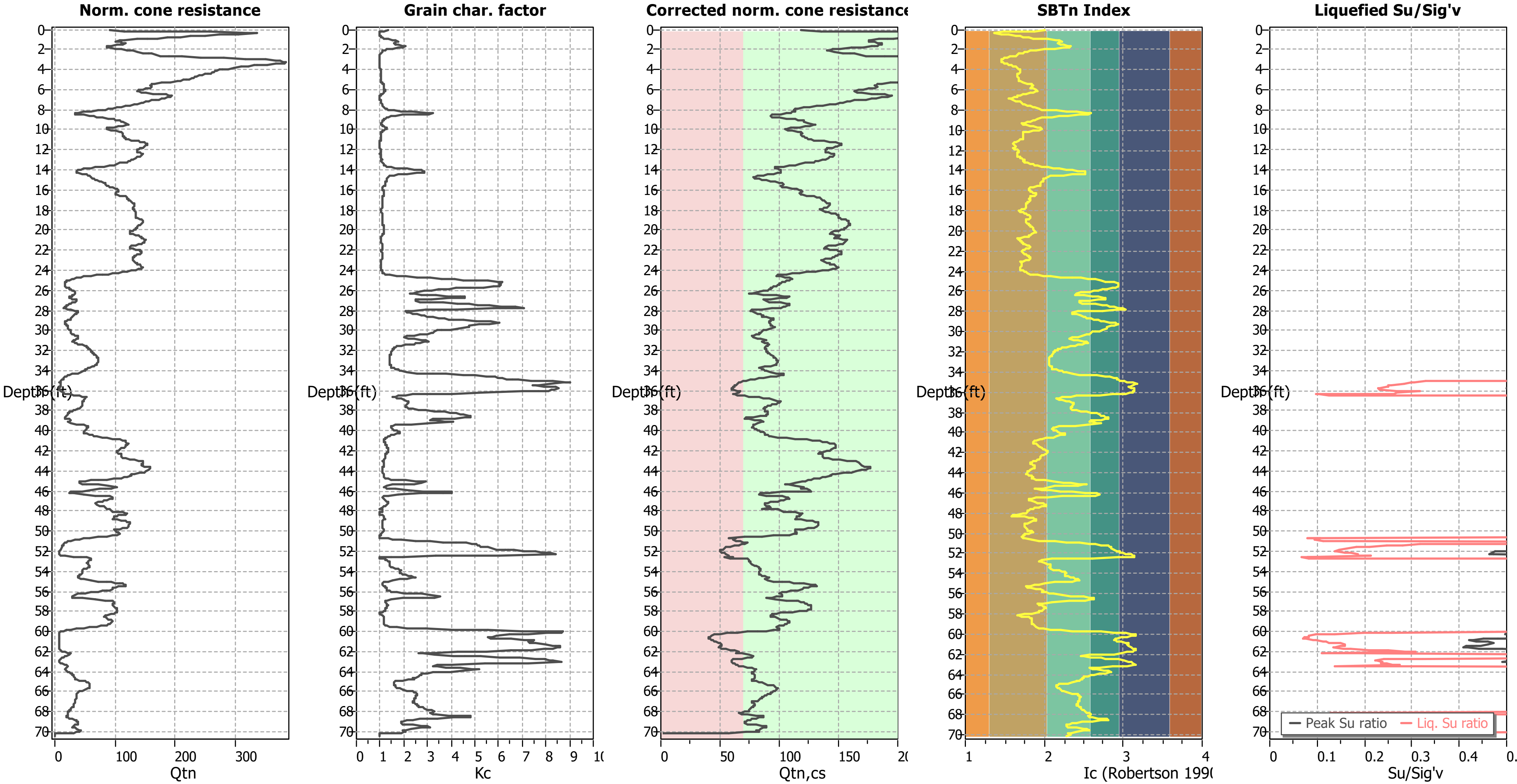
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
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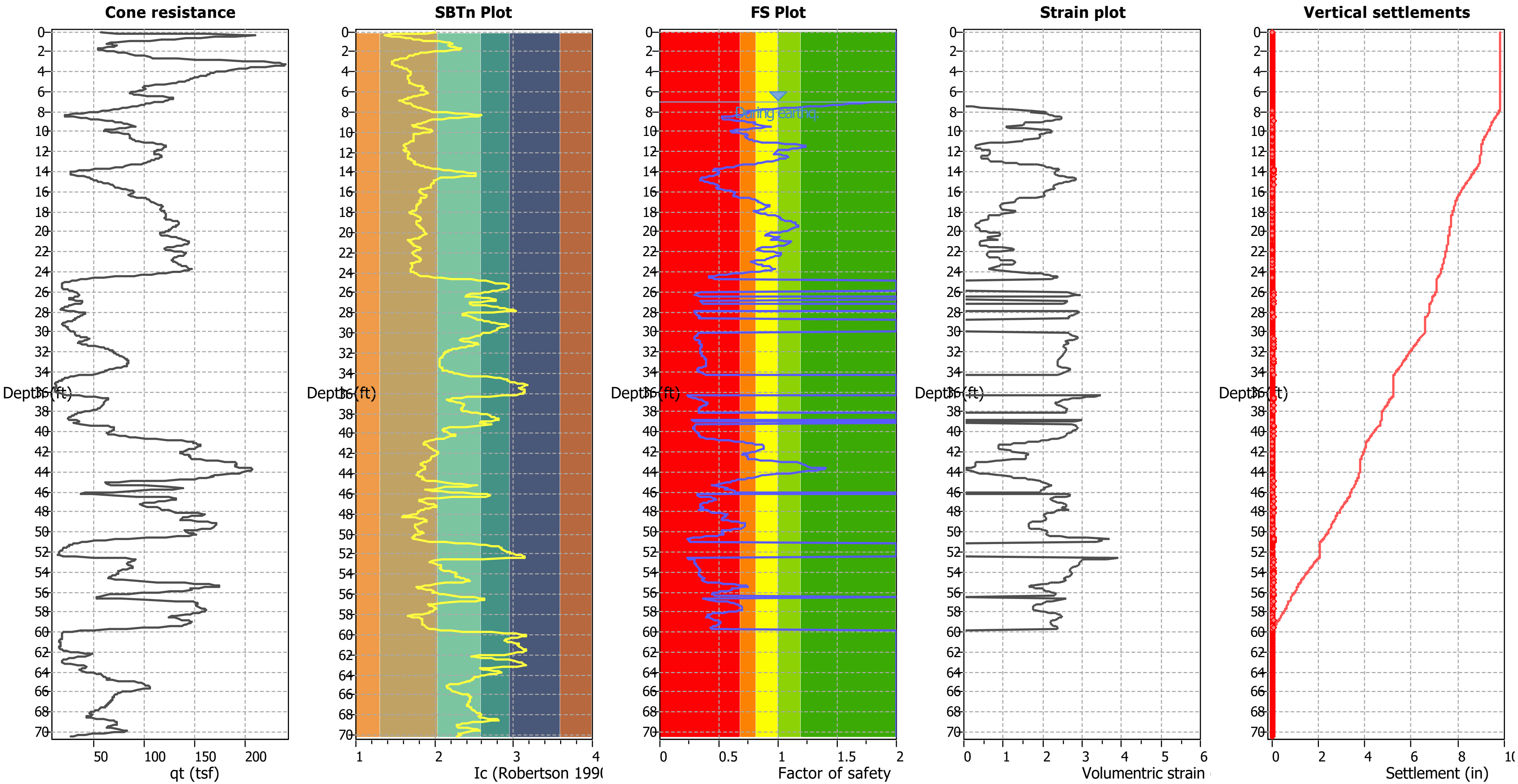
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements

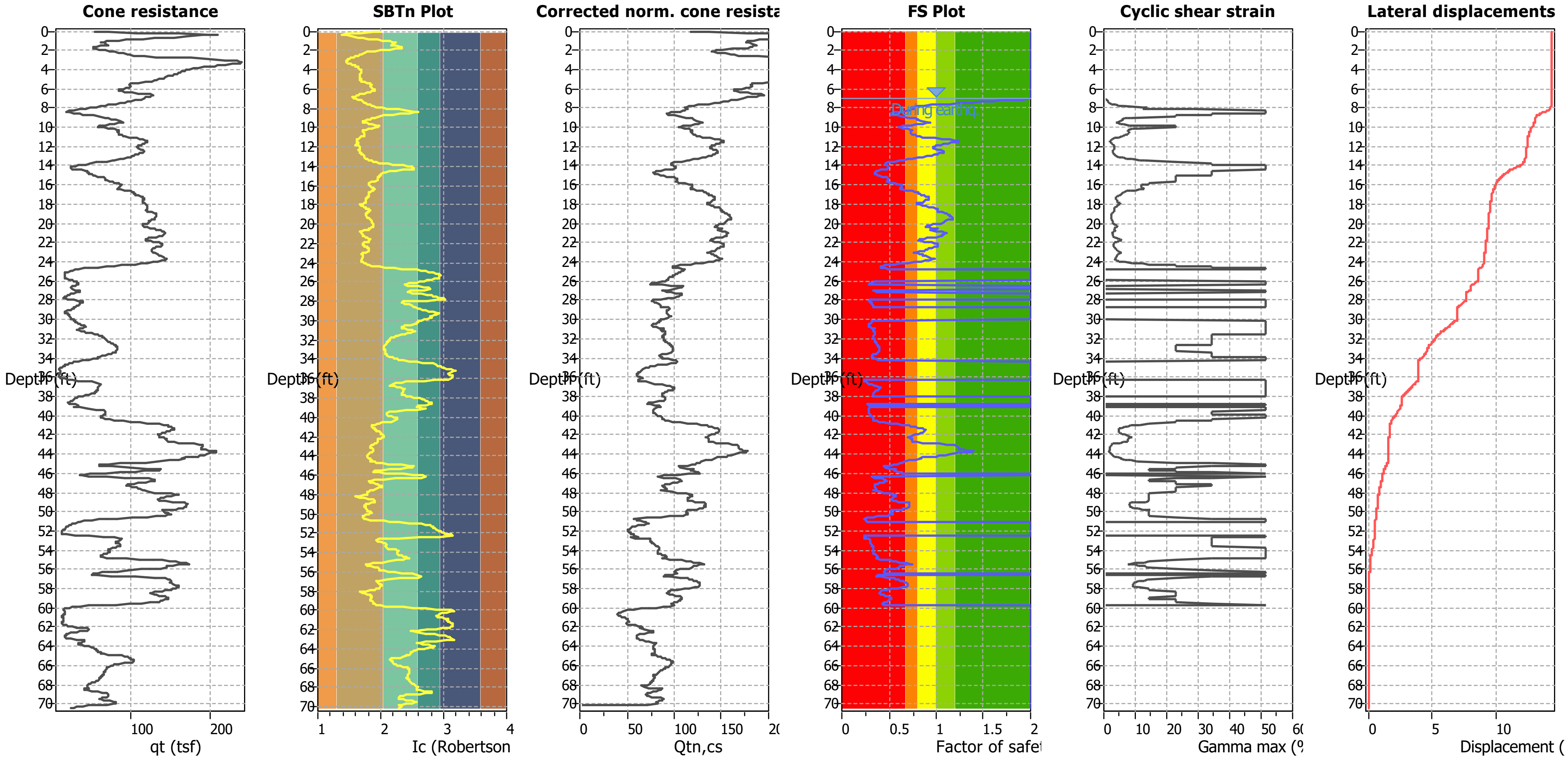


Abbreviations

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

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 0.12 %)



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition

NOVA Services, Inc.
4373 Viewridge Avenue, Suite B
San Diego, CA 92123

LIQUEFACTION ANALYSIS REPORT

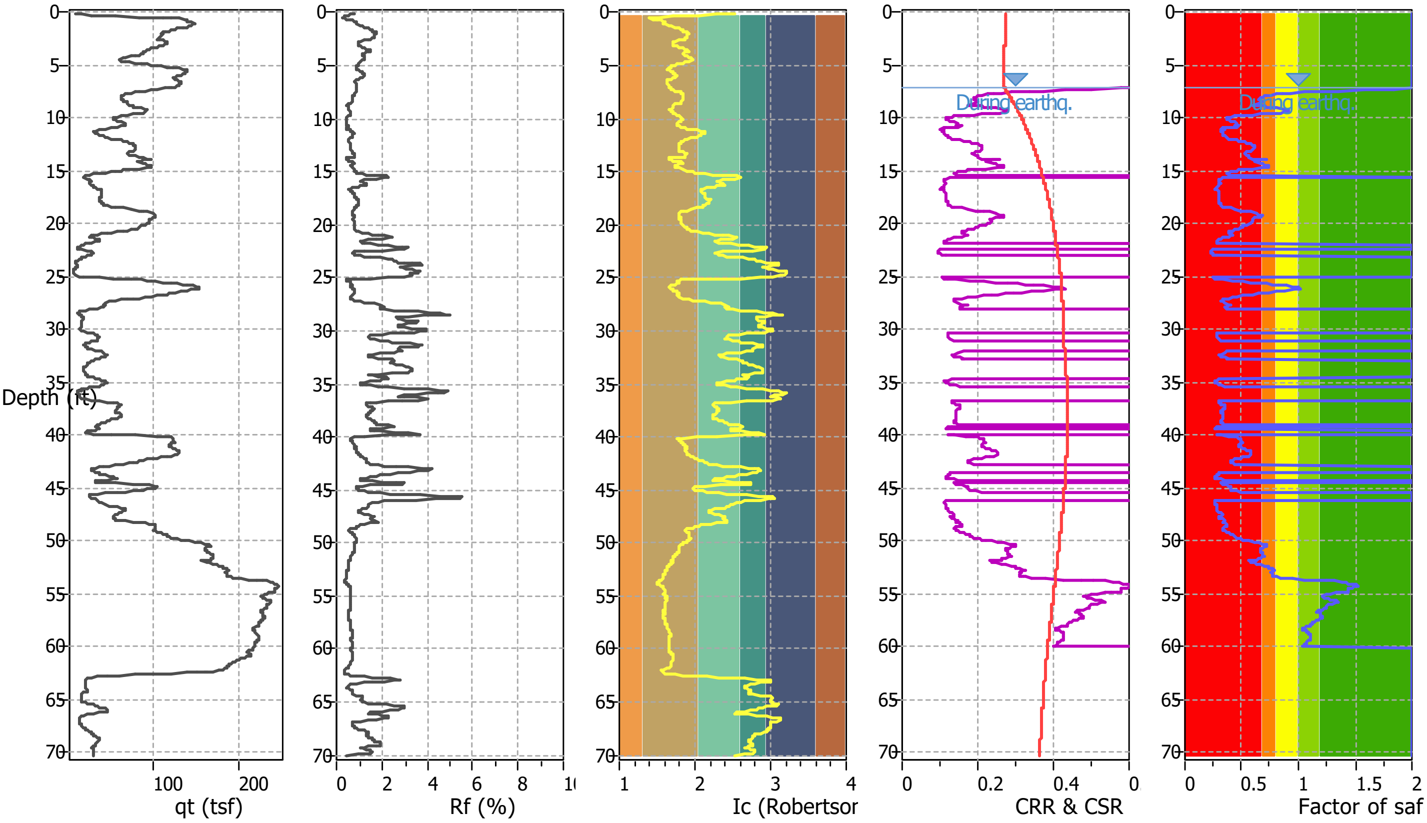
Project title : Eddy Jones Warehouse **Location : 630 Eddy Jones, Oceanside, CA**

CPT file : CPT-4

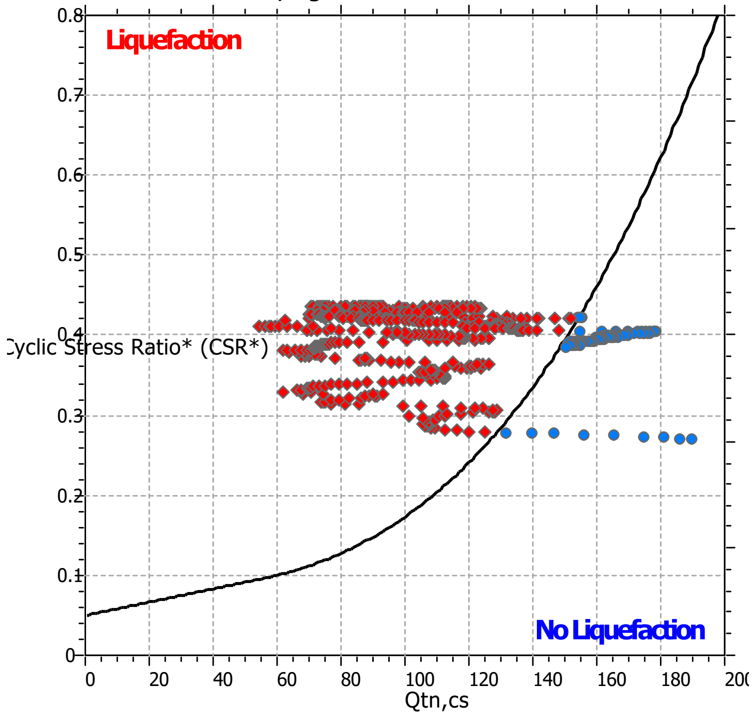
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.00	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

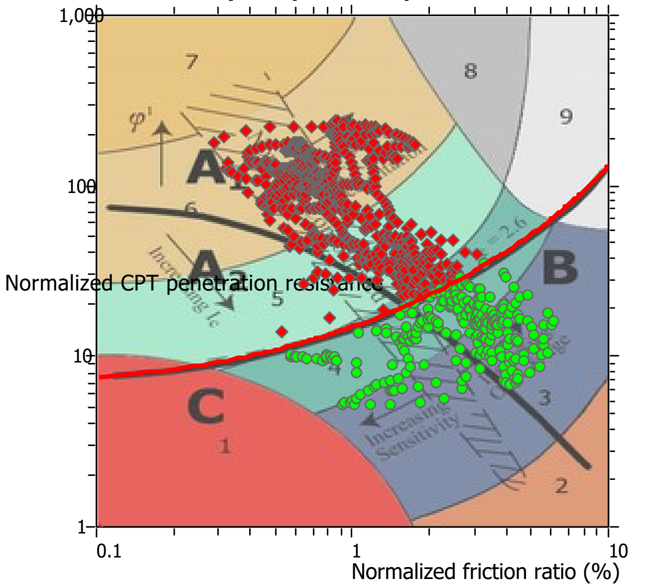
Cone resistance **Friction Ratio** **SBTn Plot** **CRR plot** **FS Plot**



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

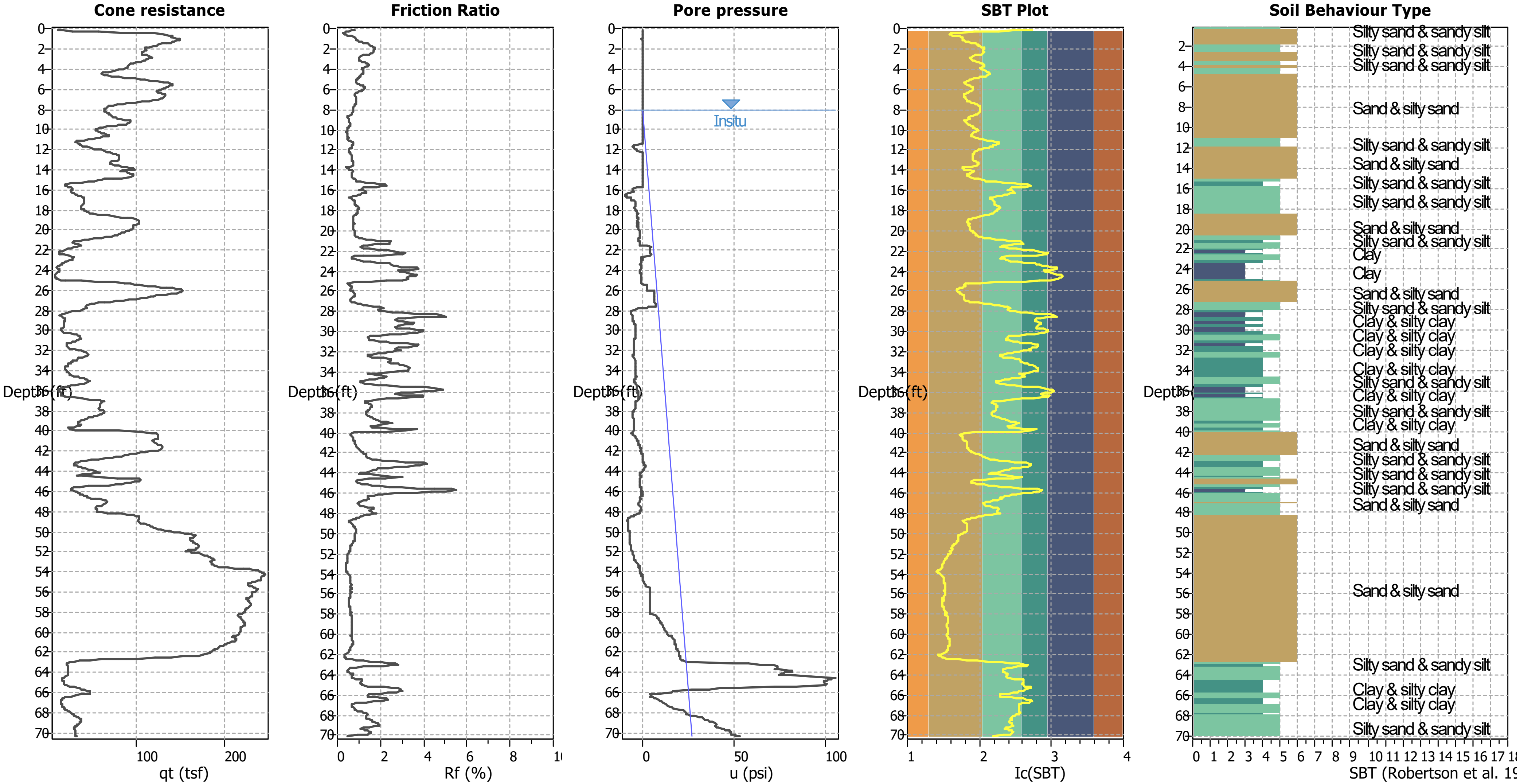


Summary of liquefaction potential



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



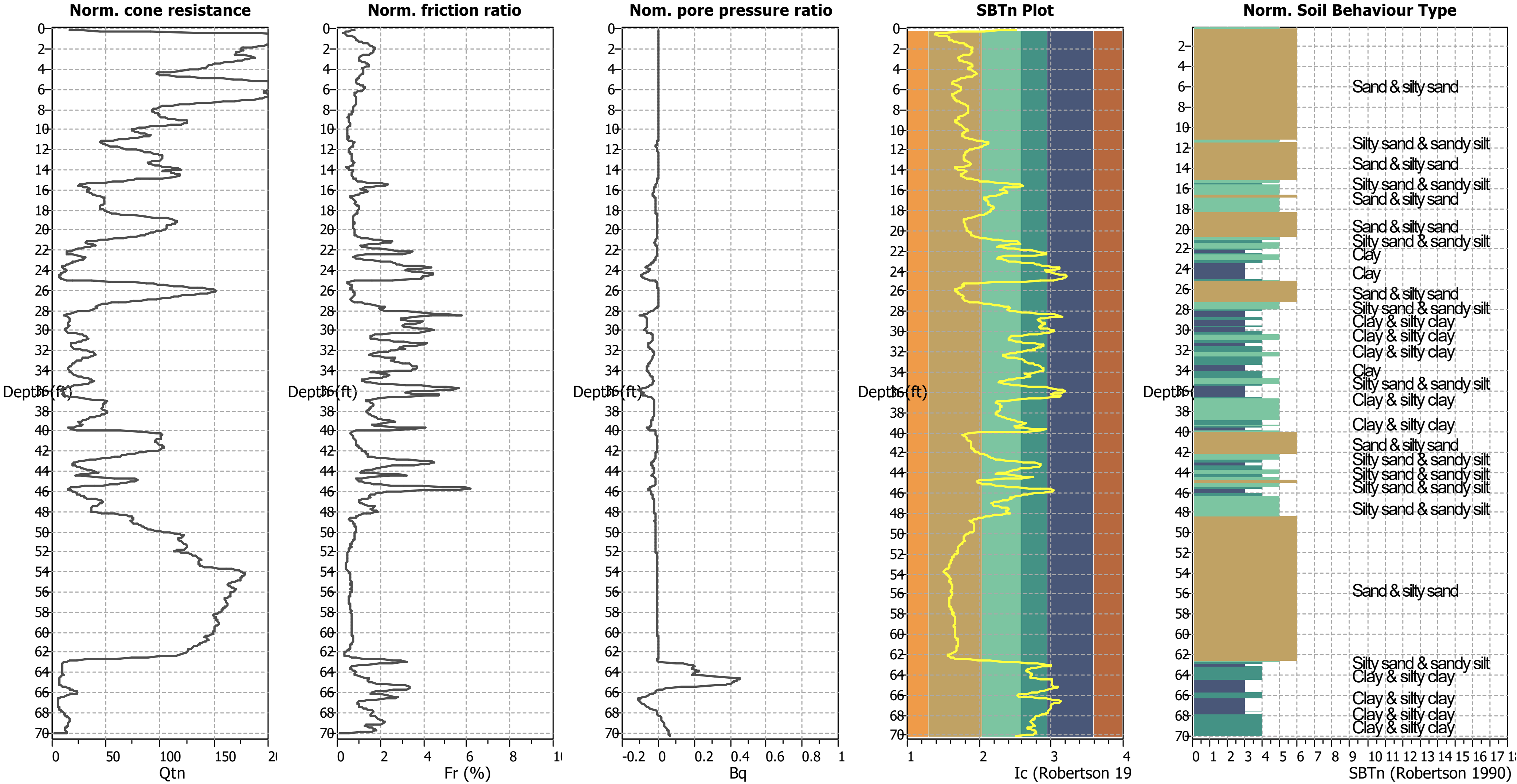
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A		
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes		
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes		
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft		

SBT legend

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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
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Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

- 1. Sensitive fine grained
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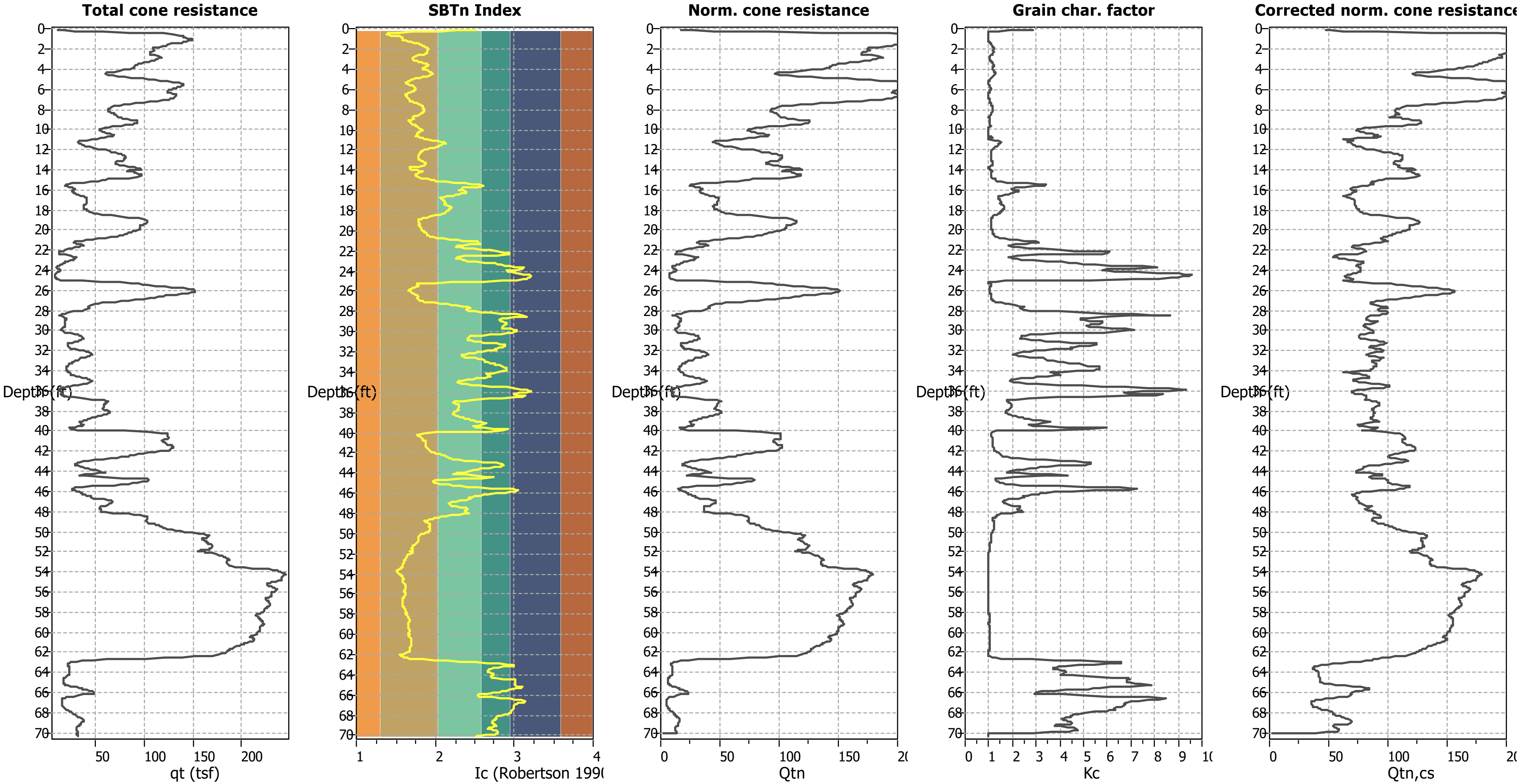
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

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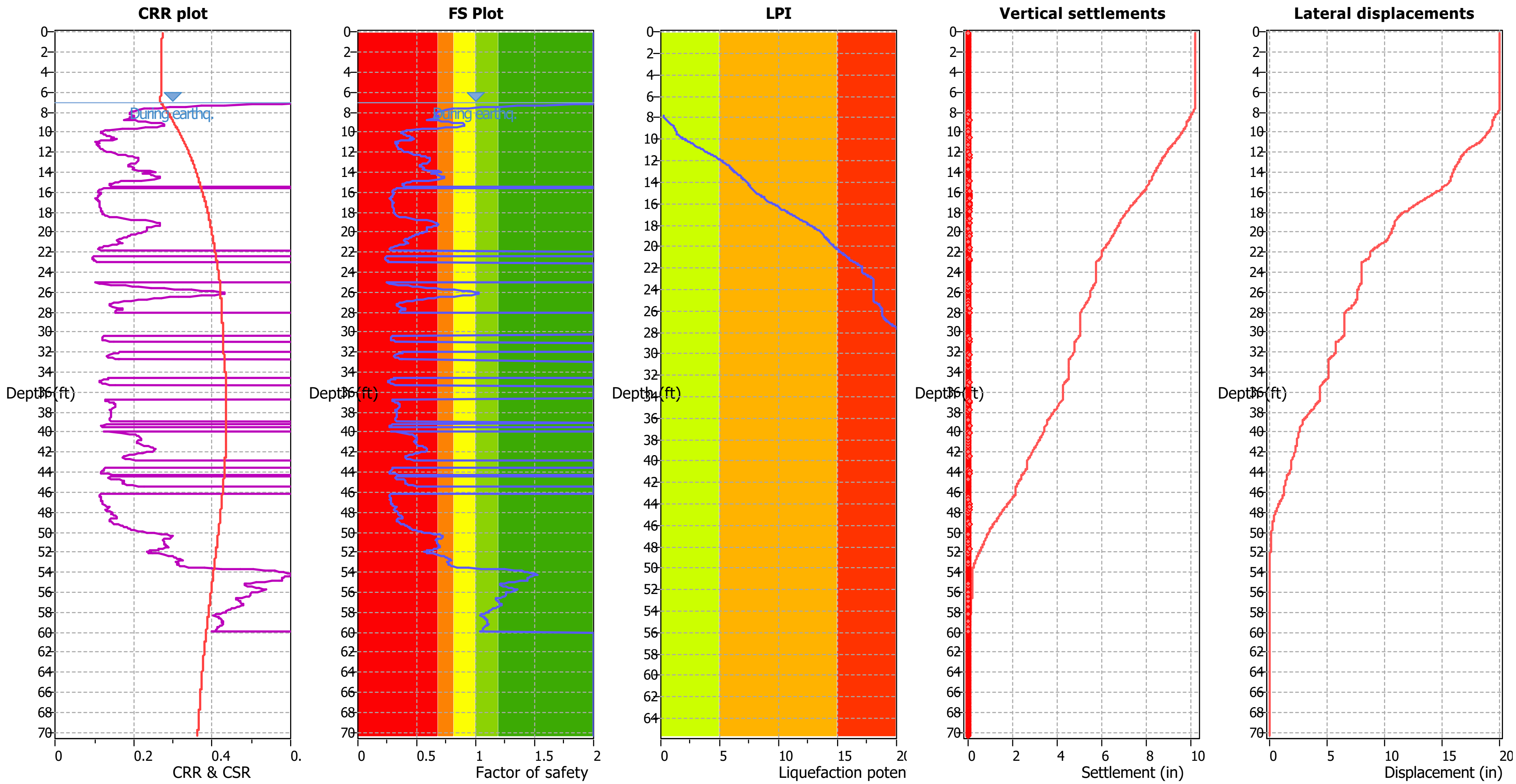
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

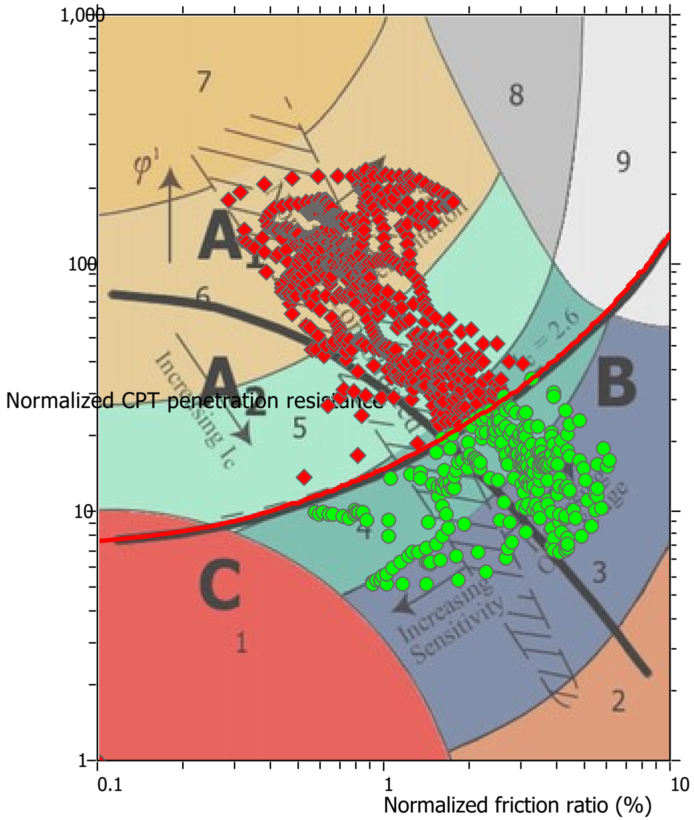
F.S. color scheme

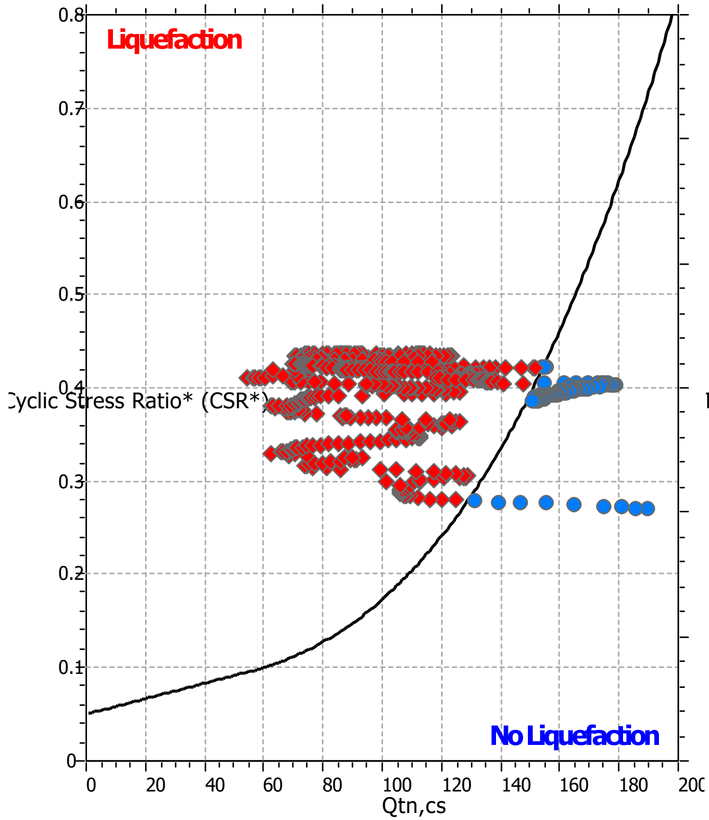
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

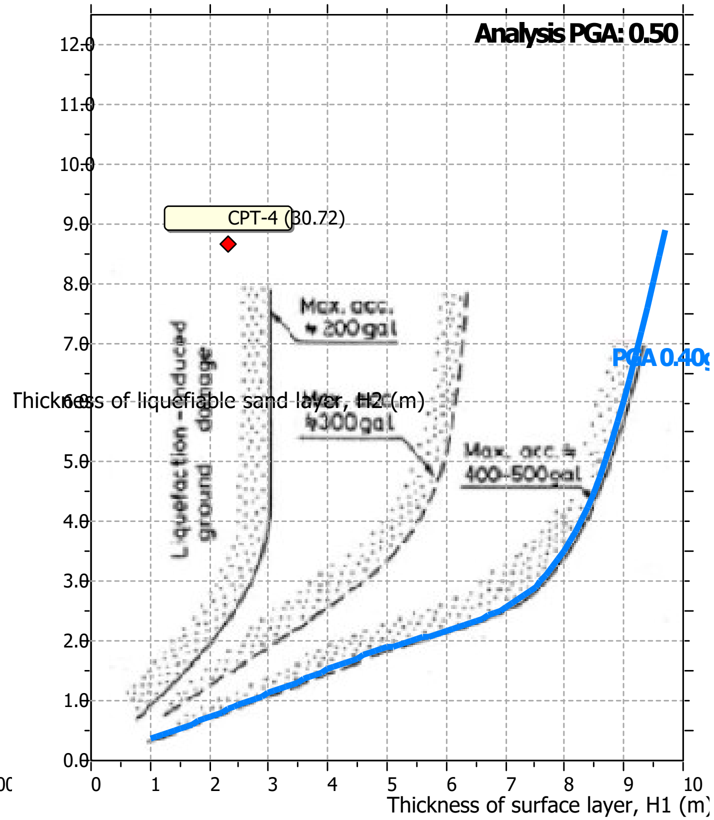
LPI color scheme

- 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.):	7.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
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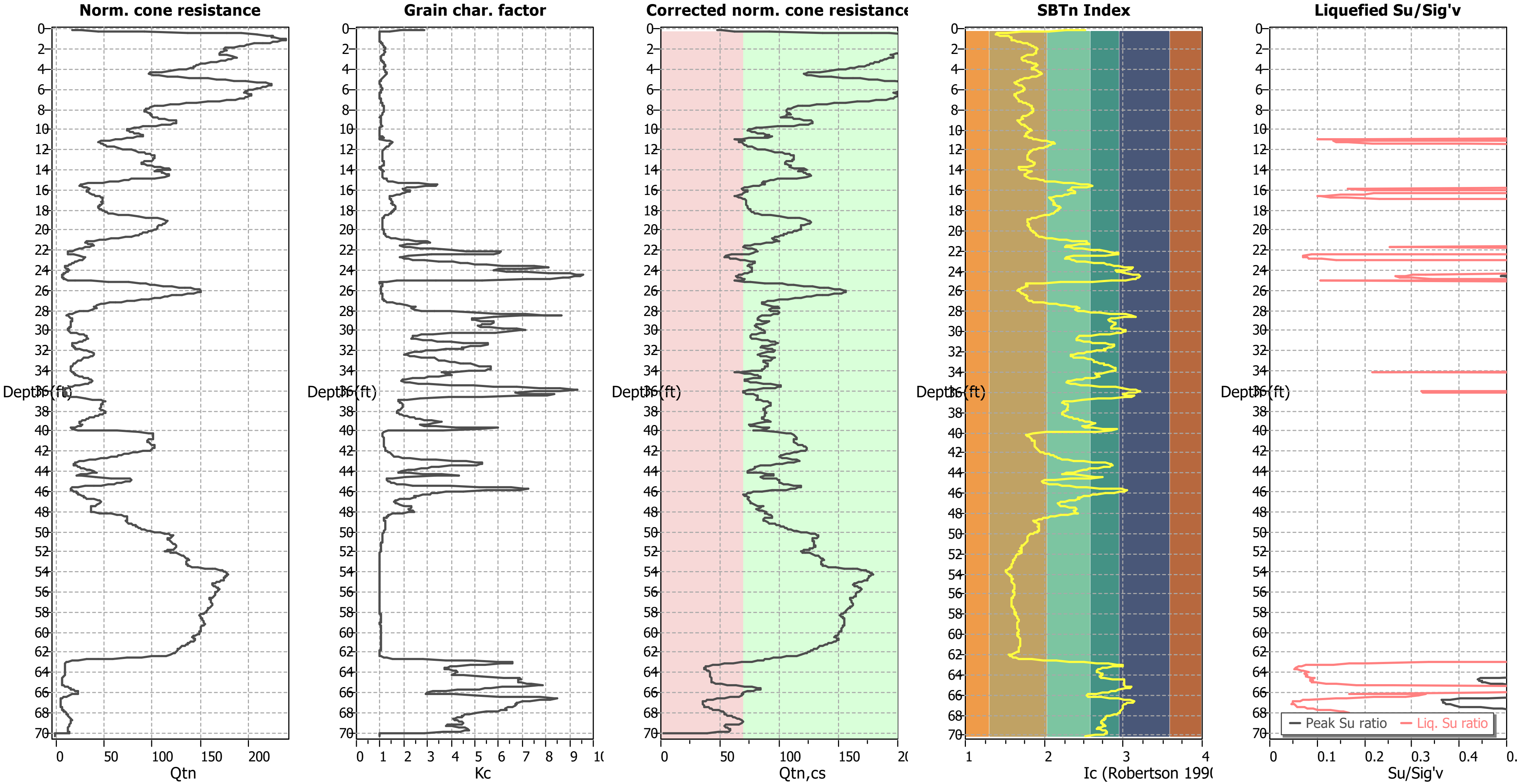

Input parameters and analysis data

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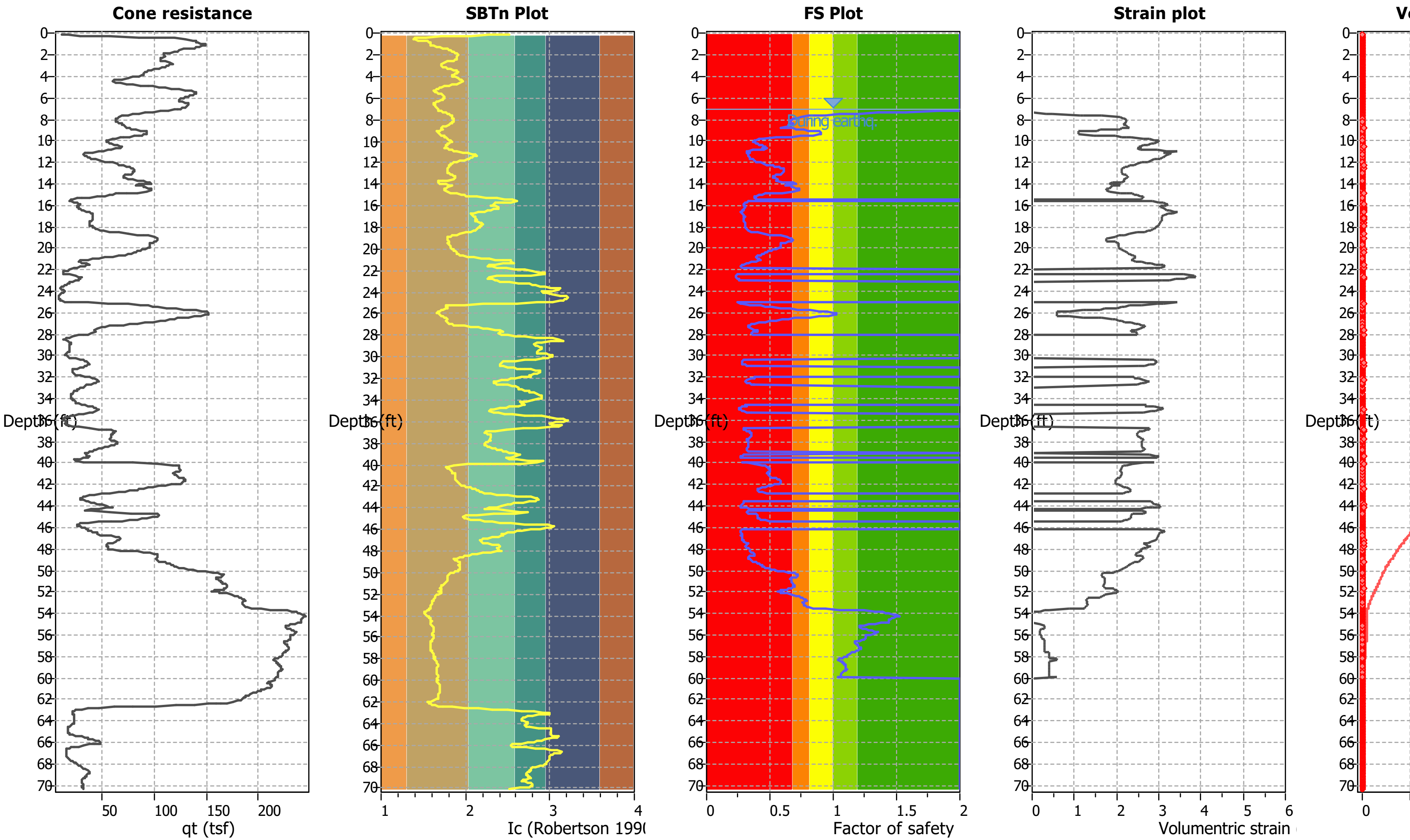
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

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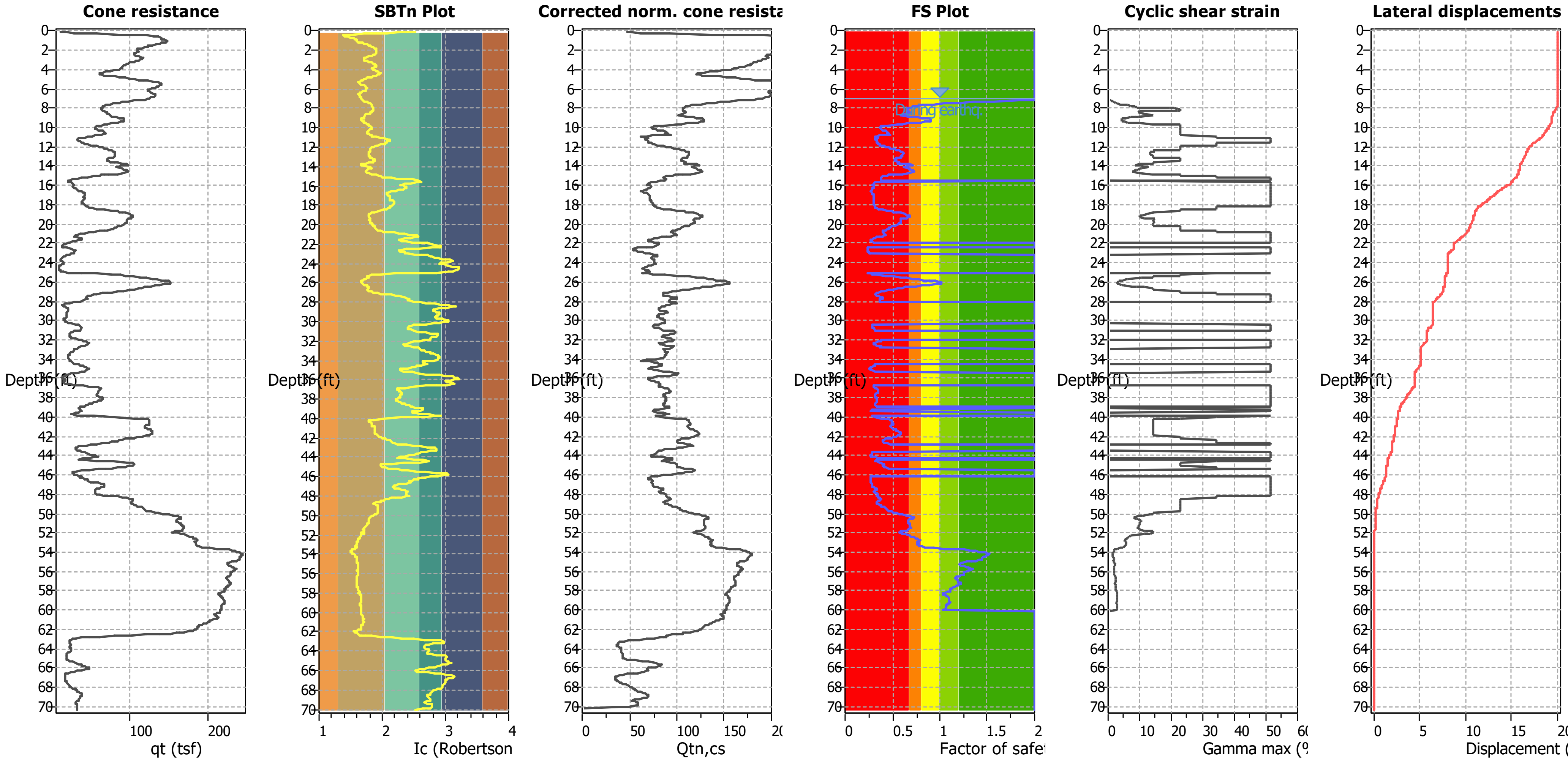
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements

Geometric parameters: Gently sloping ground without free face (Slope 0.12 %)



Abbreviations

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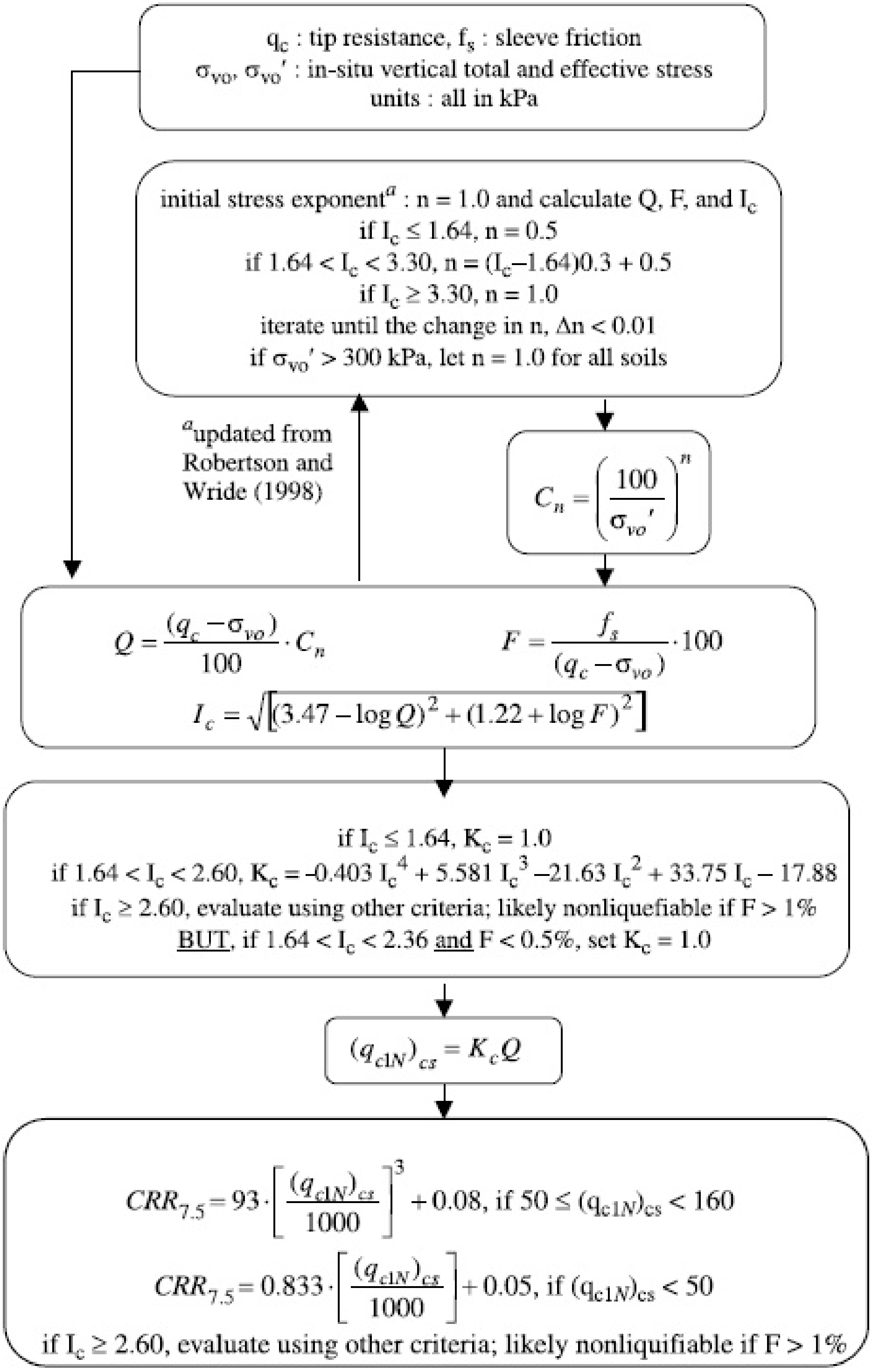
I_c: Soil Behaviour Type Index
$Q_{tn,cs}$: Equivalent clean sand normalized CPT total cone resistance

F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Surface condition

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]:



[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)

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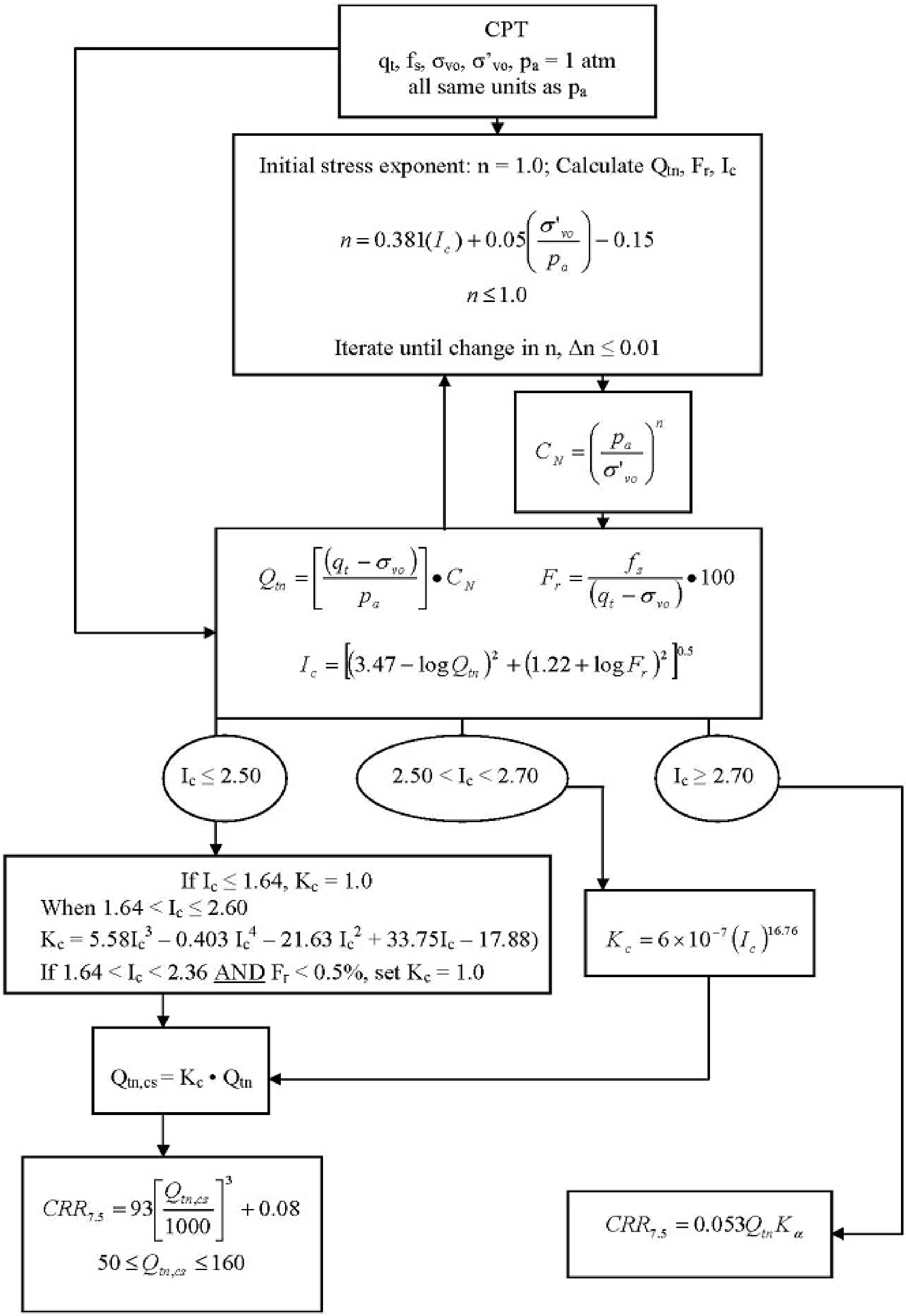
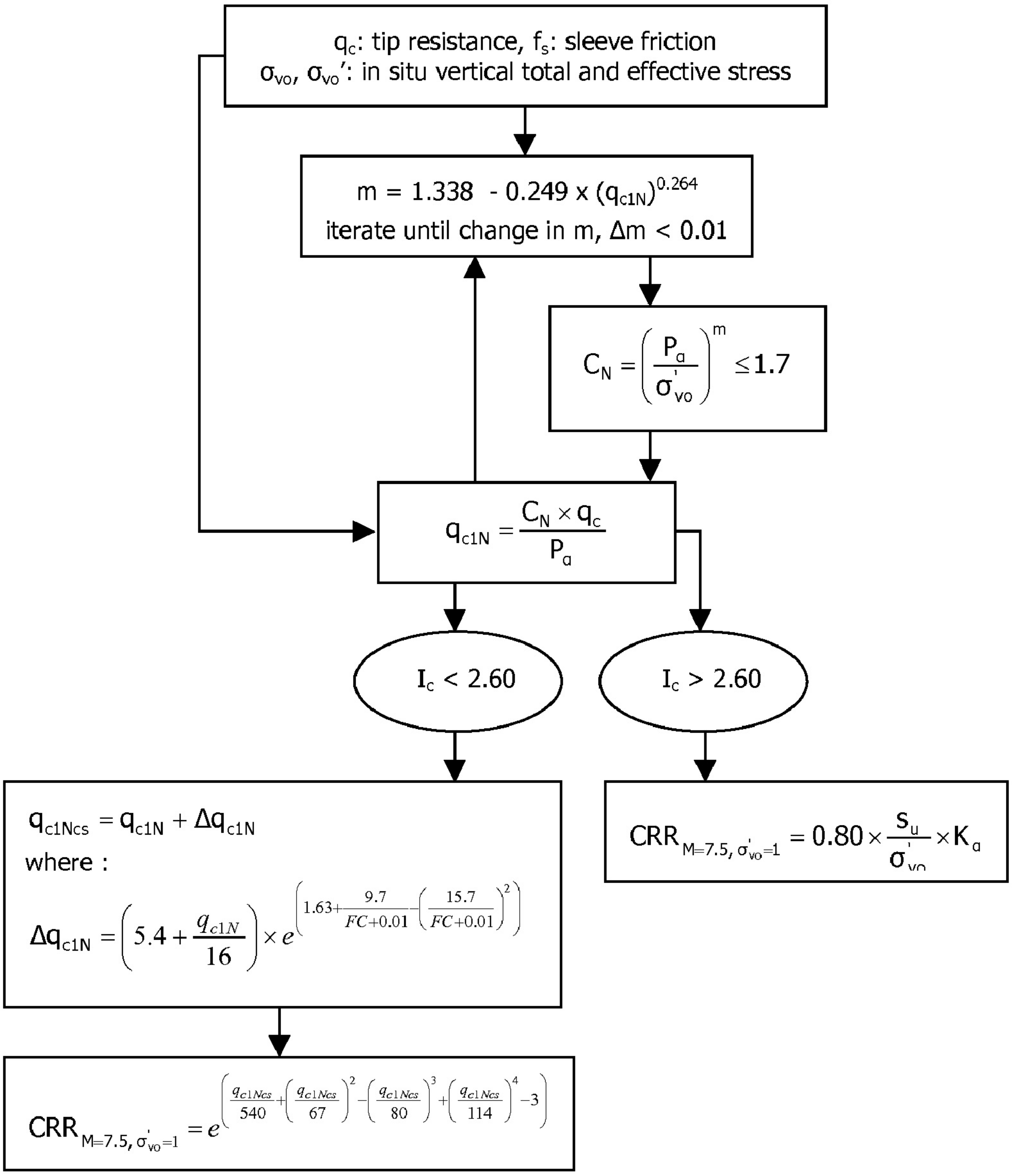


[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

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)

CPT

q_t, f_s, I_c

$I_c < I_c$ cut-off

Initial estimate using raw tip measurements, friction ratio. Calculate $q_{t,1}$. Repeat until an acceptable convergence tolerance is achieved.

$$c = f_1 \cdot \left(\frac{R_f}{f_3}\right)^{f_2}$$

$$C_q = \left(\frac{P_a}{\sigma'_v}\right)^{c}$$

$$q_{t,1} = C_q \cdot q_t$$

$$CRR = \exp\left[\frac{q_{t,1}^{1.045} + q_{t,1} \cdot (0.110 \cdot R_f) + (0.001 \cdot R_f) + c \cdot (1 + 0.850 \cdot R_f) - 0.848 \cdot \ln(M_w) - 0.002 \cdot \ln(\sigma'_v) - 20.923 + 1.632 \cdot \Phi^{-1}(P_L)}{7.177}\right]$$

Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'_v=1atm}}{CSR_{M=7.5,\sigma'_v=1atm}}$$

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65\frac{\sigma_v}{\sigma'_v}\frac{a_{\max}}{g}r_d\frac{1}{MSF}\frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z)+\beta(z)\cdot M\right]$$

$$\alpha(z) = -1.012-1.126\sin\left(\frac{z}{11.73}+5.133\right)$$

$$\beta(z) = 0.106+0.118\sin\left(\frac{z}{11.28}+5.142\right)$$

$$K_\sigma = 1-C_\sigma\ln\left(\frac{\sigma'_v}{P_a}\right)\le 1.1$$

$$C_\sigma = \frac{1}{37.3-8.27\left(q_{c1Ncs}\right)^{0.264}}\le 0.3$$

$$MSF = 1+\left(MSF_{max}-1\right)\left(8.64\exp\left(\frac{-M}{4}\right)-1.325\right)$$

$$MSF_{max} = 1.09+\left(\frac{q_{c1Ncs}}{180}\right)^3\le 2.2$$

σ'_v *and* σ_v *at start of earthquake shaking*

$$CRR_{M=7.5,\sigma'_v=1atm} = \exp\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2.80\right)$$

$$q_{c1Ncs} = q_{c1N}+\Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9+\frac{q_{c1N}}{14.6}\right)\exp\left(1.63-\frac{9.7}{FC+2}-\left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{Nq}\frac{q_c}{P_a}$$

$$C_{Nq} = \left(\frac{P_a}{\sigma'_v}\right)^m\le 1.7$$

$$m = 1.338-0.249\left(q_{c1Ncs}\right)^{0.264}\quad with\quad 0.264\le m\le 0.782$$

$$FC = 80\left(I_C+C_{FC}\right)-137\quad with\quad 0\%\le FC\le 100\%$$

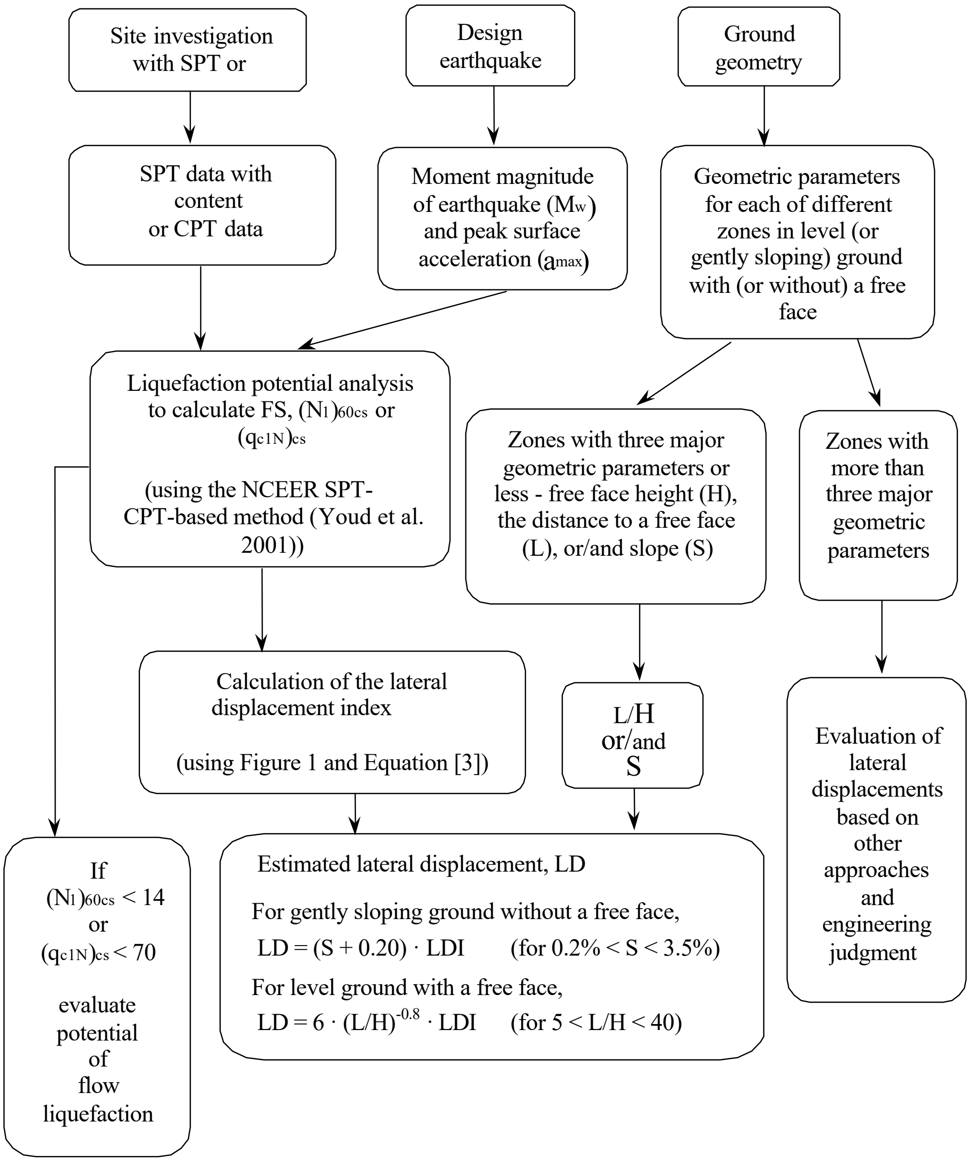
$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c-\sigma_{vo}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vo}}\right)^n\ with\ 0.5\le n\le 1.0\ per\ Robertson\ \&\ Wride\ (1998)$$

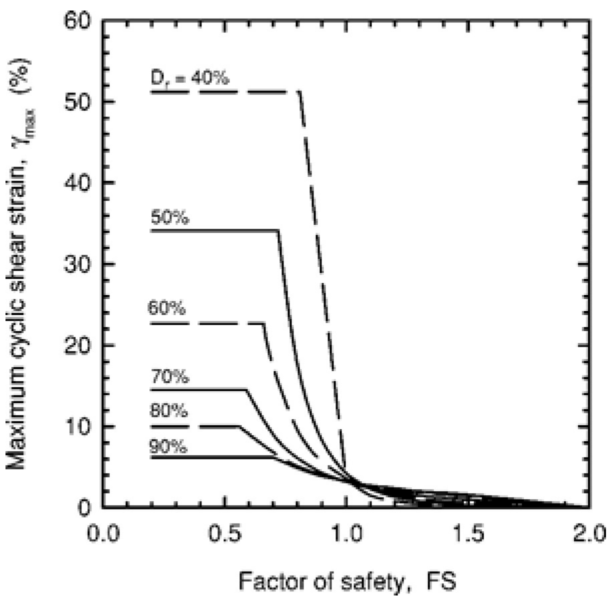
$$F = \left(\frac{f_s}{q_c-\sigma_{vo}}\right)\cdot 100\%$$

σ'_v *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**

$$\mathrm{LDI} = \int_0^{Z_{\max}} \gamma_{\max}\, dz$$

[1] **Equation [3]**

[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}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_{v0} \cdot r_d$$

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.

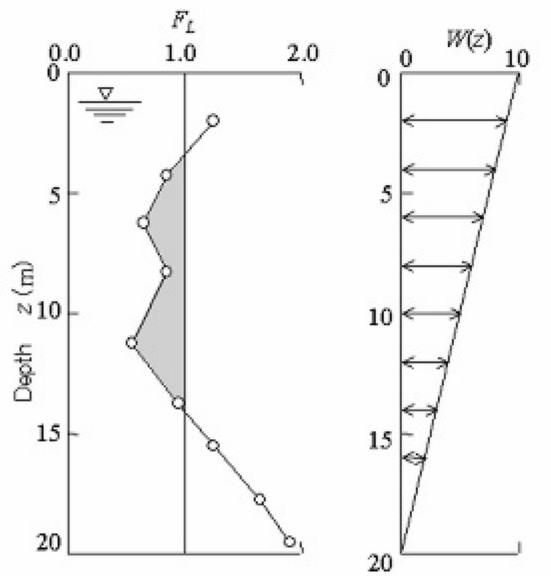
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_z) \times F_L \times d_z$$

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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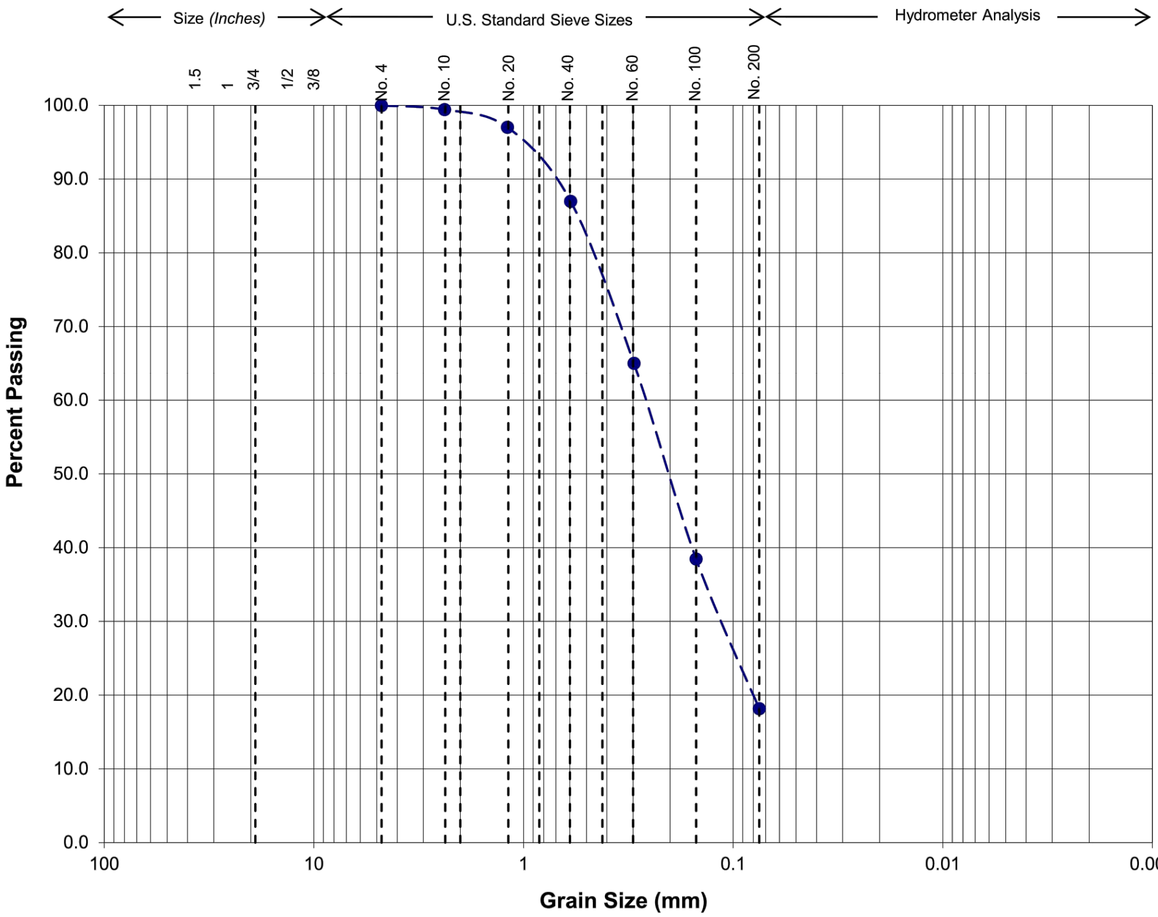
APPENDIX D
LABORATORY TESTING

Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. Brief descriptions of the tests performed are presented below:

- **CLASSIFICATION:** *Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soils Classification System and are presented on the exploration logs in Appendix B.*
- **GRADATION ANALYSIS (ASTM D6913):** *Gradation analyses were performed on representative soil samples in general accordance with ASTM D422. The grain size distributions of the samples were determined in accordance with ASTM D6913.*
- **ATTERBERG LIMITS (ASTM D4318):** *Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limits, plastic limits, and plasticity indexes in general accordance with ASTM D4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System.*
- **EXPANSION INDEX (ASTM D4829):** *The expansion indexes of selected materials were evaluated in general accordance with ASTM D4829. The specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with distilled water. Readings of volumetric swell were made for a period of 24 hours.*
- **R-VALUE (CT 301 and ASTM D2844):** *The resistance values, or R-Values, for near-surface site soils were evaluated in general accordance with California Test (CT) 301 and ASTM D2844. The samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-Value is reported as the lesser or more conservative of the two calculated results.*
- **CORROSIVITY TEST (CAL. TEST METHOD 417, 422, 643):** *Soil pH and minimum resistivity tests were performed on representative soil samples in general accordance with test method CT 643. The sulfate and chloride contents of the selected samples were evaluated in general accordance with CT 417 and CT 422, respectively.*



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location:	B - 2
Depth (ft):	0 - 5
USCS Soil Type:	SM
Passing No. 200 (%):	18

Atterberg Limits (ASTM D4318):

Liquid Limit, LL:	NP
Plastic Limit, PL:	NP
Plasticity Index, PI:	NP

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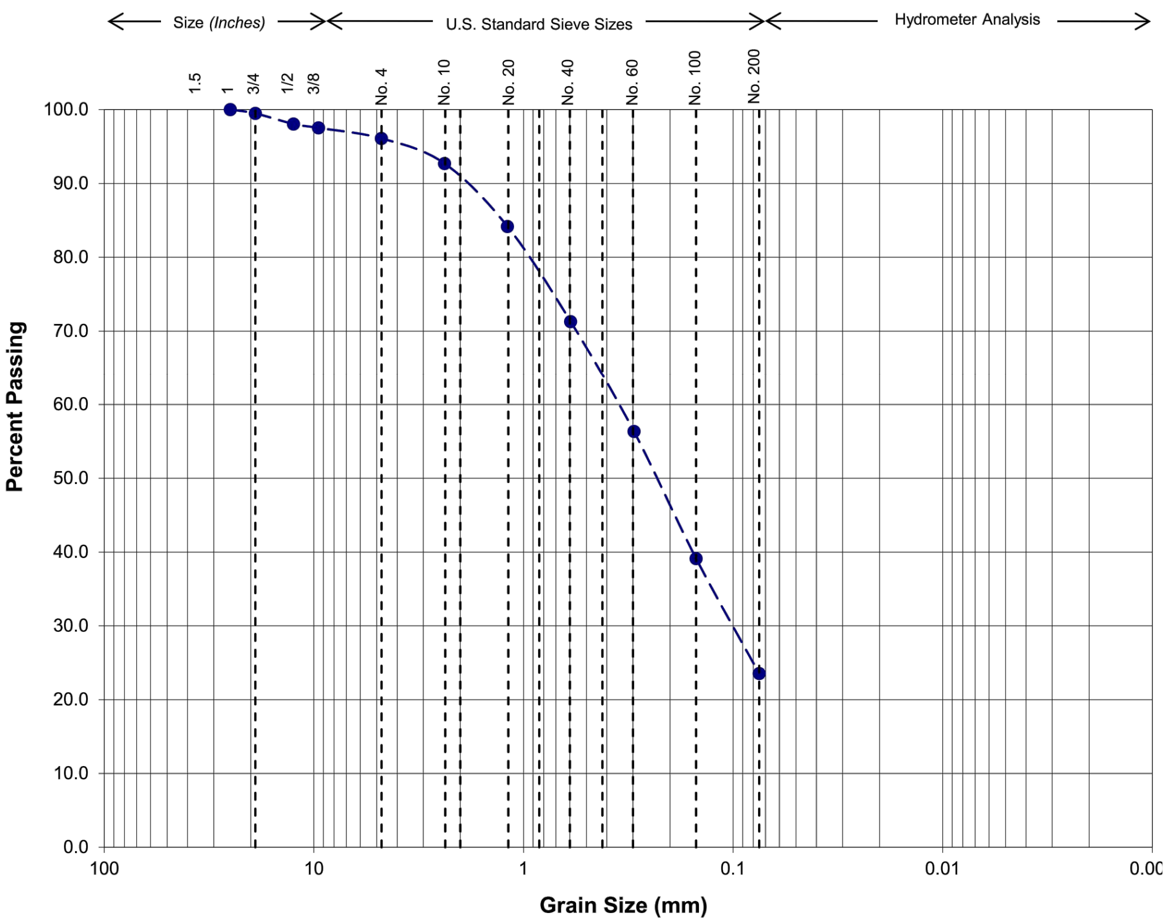
CLASSIFICATION TEST RESULTS

PROPOSED EDDY JONES INDUSTRIAL DISTRIBUTION

260 EDDY JONES WAY

OCEANSIDE, CA

BY: GN	DATE: APR 2024	PROJECT: 2021176	FIGURE: D.2



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 4

Depth (ft): 0 - 5

USCS Soil Type: SC-SM

Passing No. 200 (%): 24

Atterberg Limits (ASTM D4318):

Liquid Limit, LL: 24

Plastic Limit, PL: 19

Plasticity Index, PI: 5

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CLASSIFICATION TEST RESULTS

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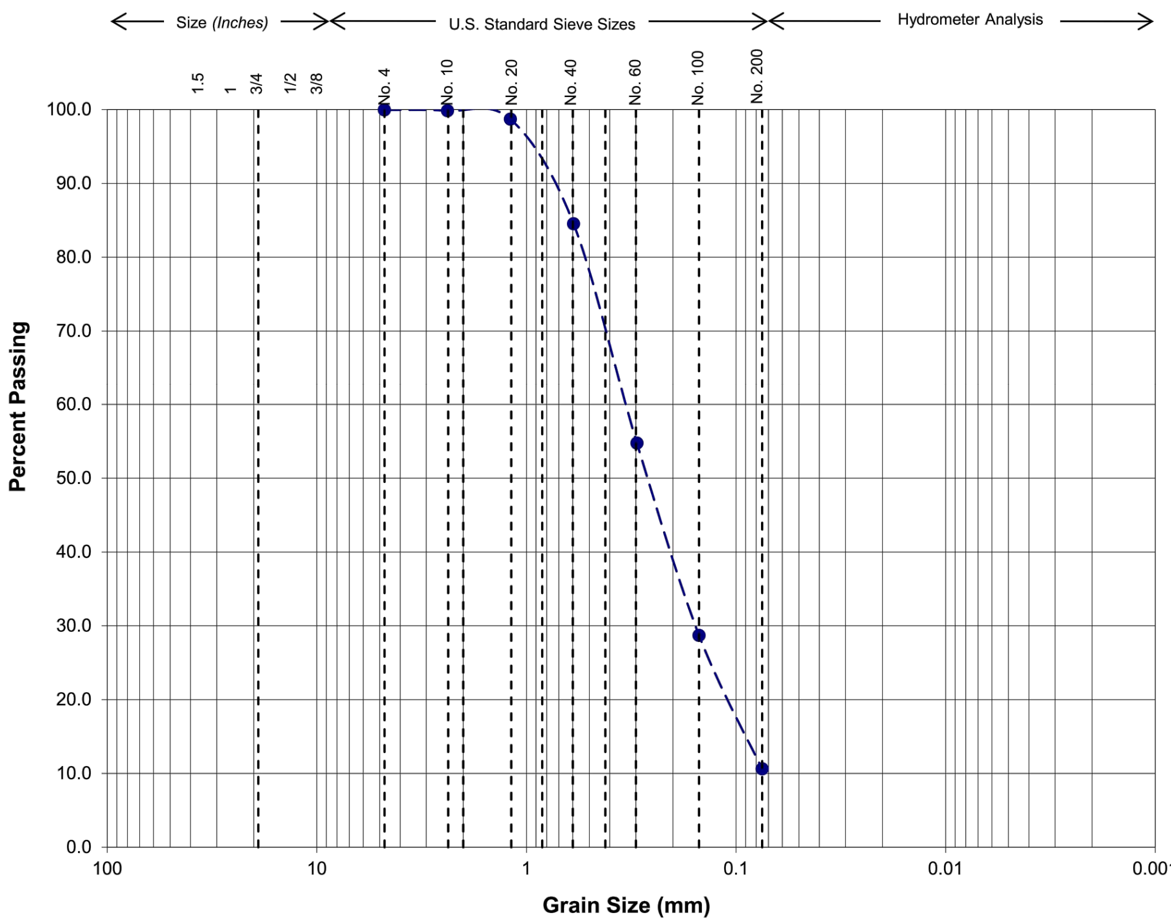
OCEANSIDE, CA

BY: GN	DATE: APR 2024	PROJECT: 2021176	FIGURE: D.3



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: P - 1

Depth (ft): 0 - 5

USCS Soil Type: SP-SM

Passing No. 200 (%): 11

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CLASSIFICATION TEST RESULTS

PROPOSED EDDY JONES INDUSTRIAL DISTRIBUTION

260 EDDY JONES WAY

OCEANSIDE, CA

BY: GN	DATE: APR 2024	PROJECT: 2021176	FIGURE: D.4

Expansion Index (ASTM D4829)

Sample Location	Sample Depth (ft.)	Expansion Index	Expansion Potential
B - 2	0 - 5	0	Very Low
B - 4	0 - 5	2	Very Low

Resistance Value (Cal. Test Method 301 & ASTM D2844)

Sample Location	Sample Depth (ft.)	R-Value
B - 1	0 - 5	54
B - 4	0 - 5	39

Corrosivity (Cal. Test Method 417,422,643)

Sample Location	Sample Depth (ft.)	pH	Resistivity	Sulfate Content		Chloride Content	
			(Ohm-cm)	(ppm)	(%)	(ppm)	(%)
B - 2	0 - 5	7.4	10,000	42	0.004	11	0.001
B - 4	0 - 5	8.3	1,500	90	0.009	180	0.018

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LAB TEST RESULTS

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BY: GN	DATE: APR 2024	PROJECT: 2021176	FIGURE: D.5

APPENDIX E
INFILTRATION TEST RESULTS

REPORT OF BOREHOLE PERCOLATION TESTING
Storm Water Infiltration

Project Name:	Proposed Eddy Jones Industrial Distribution	Test Number:	**P-1**
Job Number:	2021176	Tested By:	DM
Date Drilled:	10/4/2021	Date Tested:	10/5/2021
Drilling Method:	8-inch Hollow Stem Auger	Presoak Time:	24 HR
Drilled Depth (feet):	5.0		
Test Hole Diameter (inches):	8		
Gravel Pack:	Y		
Pipe Diameter (inches):	3		

Trial No.	Time	Time Interval, ΔT (min)	Initial Water Height, H_o (ft)	Final Water Height, H_f (ft)	Change in Water Height, ΔH (in)	Percolation Rate (min/in)
1	11:21 11:51	0:30	3.08	1.53	18.60	2
2	11:51 12:21	0:30	3.58	2.73	10.20	3
3	12:21 12:51	0:30	2.73	2.08	7.80	4
4	12:51 13:21	0:30	2.08	1.58	6.00	5
5	13:21 13:51	0:30	3.58	2.78	9.60	3
6	13:51 14:21	0:30	2.78	2.18	7.20	4
7	14:21 14:51	0:30	2.18	1.68	6.00	5
8	14:51 15:21	0:30	3.58	2.83	9.00	3
9	15:21 15:51	0:30	2.83	2.28	6.60	5
10	15:51 16:21	0:30	2.28	1.78	6.00	5
11	16:21 16:51	0:30	2.28	1.78	6.00	5
12	16:51 17:21	0:30	2.28	1.78	6.00	5
				Observed Percolation Rate:	5 min/in 12.0 in/hr	
				Tested Infiltation Rate Using Porchet Method, I_t:	**0.91 in/hr**	
				Infiltation Rate with Factor of Safety FS=2:	**0.45 in/hr**	

PORCHET METHOD CALCULATION:

$$I_t = \frac{\Delta H(60r)}{\Delta T(r + 2H_{avg})}$$

	Values from Last Trial
ΔH = Change in water head height over the time interval	= 6.0 in
r = Test hole radius	= 4 in
ΔT = Time interval	= 30 min
H_{avg}= Average water height over time interval=12(H_o + H_f)/2	= 24.40 in

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OCEANSIDE, CA

By:	HP	Date:	March 2024
Job No:	2021176	Appendix	E.1

REPORT OF BOREHOLE PERCOLATION TESTING
Storm Water Infiltration

Project Name:	Proposed Eddy Jones Industrial Distribution	Test Number:	**P-2**
Job Number:	2021176	Tested By:	DM
Date Drilled:	10/4/2021	Date Tested:	10/5/2021
Drilling Method:	8-inch Hollow Stem Auger	Presoak Time:	24 HR
Drilled Depth (feet):	5.0		
Test Hole Diameter (inches):	8		
Gravel Pack:	Y		
Pipe Diameter (inches):	3		

Trial No.	Time	Time Interval, ΔT (min)	Initial Water Height, H_o (ft)	Final Water Height, H_f (ft)	Change in Water Height, ΔH (in)	Percolation Rate (min/in)
1	12:21 12:51	0:30	3.00	2.73	3.24	9
2	12:51 13:21	0:30	3.00	2.78	2.64	11
3	13:21 13:51	0:30	3.00	2.76	2.88	10
4	13:51 14:21	0:30	3.00	2.77	2.76	11
5	14:21 14:51	0:30	2.50	2.33	2.04	15
6	14:51 15:21	0:30	2.50	2.33	2.04	15
7	15:21 15:51	0:30	2.50	2.33	2.04	15
8	15:51 16:21	0:30	2.50	2.34	1.92	16
9	16:21 16:51	0:30	2.50	2.34	1.92	16
10	16:51 17:21	0:30	2.50	2.34	1.92	16
11	17:21 17:51	0:30	2.50	2.35	1.80	17
12	17:51 18:21	0:30	2.50	2.35	1.80	17
				Observed Percolation Rate:	17 min/in 3.6 in/hr	
				Tested Infiltation Rate Using Porchet Method, I_t:	**0.23 in/hr**	
				Infiltation Rate with Factor of Safety FS=2:	**0.12 in/hr**	

PORCHET METHOD CALCULATION:

$$I_t = \frac{\Delta H(60r)}{\Delta T(r + 2H_{avg})}$$

	Values from Last Trial
ΔH = Change in water head height over the time interval	= 1.8 in
r = Test hole radius	= 4 in
ΔT = Time interval	= 30 min
H_{avg}= Average water height over time interval=12(H_o + H_f)/2	= 29.10 in

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By:	GN	Date:	March 2024
Job No:	2021176	Appendix	E.2

APPENDIX F
SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS

SITE-SPECIFIC GROUND MOTION ANALYSIS (ASCE 7-16)

Project:	Proposed Eddy Jones Industrial Distribution	**Latitude:**	33.22000	deg		**Calculated By:**	IB
Client:	RAF Pacifica Group	**Longitude:**	-117.35494	deg		**Checked By:**	GD
Job No:	2021176	**Vs_{30}:**	215	m/s	(Measured)	**Date:**	2/19/24

Period T (sec)	PROBABILISTIC (RISK-TARGETED) GROUND MOTION ANALYSIS				DETERMINISTIC (84TH-PERCENTILE) GROUND MOTION ANALYSIS			CODE-BASED (LOWER LIMIT) ASCE 7-16 SECTION 11.4.6		SITE-SPECIFIC DESIGN RESPONSE		
	Uniform Hazard Ground Motion (g)	Risk Targeted Ground Motion (g)	Maximum Direction Scale Factor	Maximum Directional Probabilistic Sa (g)	84th Percentile Spectral Acceleration (g)	Maximum Direction Scale Factor	Maximum Directional Deterministic Sa (g)	Code Based S_a (g)	80% of Code Based S_a (g)	Design S_{aM} (g)	Design S_a (g)	T x S_a (T>1s)
PGA	0.518	0.492	1.1	0.541	0.546	1.1	0.601	0.289	0.231	0.541	0.361	---
0.10	0.914	0.878	1.1	0.966	0.813	1.1	0.894	0.626	0.501	0.894	0.596	---
0.20	1.232	1.189	1.1	1.308	1.130	1.1	1.243	0.723	0.578	1.243	0.829	---
0.30	1.345	1.276	1.125	1.436	1.335	1.125	1.502	0.723	0.578	1.436	0.957	---
0.50	1.240	1.163	1.175	1.367	1.398	1.175	1.643	0.723	0.578	1.367	0.911	---
0.75	0.986	0.912	1.2375	1.129	1.193	1.2375	1.476	0.621	0.497	1.129	0.752	---
1.00	0.790	0.731	1.3	0.950	1.048	1.3	1.362	0.466	0.372	0.950	0.634	0.634
2.00	0.415	0.379	1.35	0.512	0.623	1.35	0.841	0.233	0.186	0.512	0.341	0.682
3.00	0.267	0.243	1.4	0.340	0.403	1.4	0.564	0.155	0.124	0.340	0.227	0.680
4.00	0.190	0.172	1.45	0.249	0.270	1.45	0.392	0.116	0.093	0.249	0.166	0.665
5.00	0.143	0.130	1.5	0.195	0.189	1.5	0.284	0.093	0.074	0.195	0.130	0.650

INPUT PARAMETERS - SEAOC (https://seismicmaps.org/)

Site Class=	D	
F_a=	1.109	Short Period Site Coefficient
S_S=	0.978	Mapped MCE_R, 5% Damped at T=0.2s
S_1=	0.360	Mapped MCE_R, 5% Damped at T=1s
S_{DS}=	0.723	Design, 5% Damped at Short Periods
S_{MS}=	1.084	The MCE_R, 5% Damped at Short Periods
T_L (sec)=	8.0	Long Period Transition (Sect 11.4.6)
F_{PGA} (g)=	1.175	Site Coefficient for PGA
PGA_M (g)=	0.500	
F_v=	1.940	Used Only for Calculation of T_o and T_s
S_{M1}=	0.698	
S_{D1}=	0.466	Design, 5% Damped at T=1s
T_o (sec)=	0.129	Defined in ASCE 7-16 Sect 11.4.6
T_S (sec)=	0.644	Defined in ASCE 7-16 Sect 11.4.6

SITE-SPECIFIC DESIGN PARAMETERS

S_{DS}=	**0.861**	90% of max S_a (ASCE 7-16 Sect 21.4)
S_{MS}=	**1.292**	MCE_R, 5% Damped, adjusted for Site Class
S_{D1}=	**0.682**	Design, 5% Damped, at T=1s (Sect 11.4.5)
S_{M1}=	**1.023**	MCE_R, 5% Damped, at T=1s, adjusted for Site
F_a=	**1.109**	Short Period Site Coefficient
F_v=	**1.940**	Long Period Site Coefficient (7-16 Sect 21.3)
S_S=	**1.165**	MCE_R, 5% Damped at T=0.2s
S_1=	**0.527**	MCE_R, 5% Damped at T=1s
$PGA_{Probabilistic}$ (g)=	**0.518**	Peak Ground Acceleration, Probabilistic
$PGA_{Deterministic}$ (g)=	**0.588**	Peak Ground Acceleration, Deterministic
F_{PGA} (g)=	**1.175**	Site Coefficient for PGA
0.5*F_{PGA} (g)=	**0.588**	OK (Check $PGA_{Deterministic}$ > 0.5 x F_{PGA})
0.8*PGA_M (g)=	**0.400**	PGA_M (g) (Determined from ASCE 7-16 Eq. 11.8-1)
Site Specific PGA_M (g) =	**0.518**	(Check $PGA_{Site\ Specific}$ > 0.8 x PGA_M)

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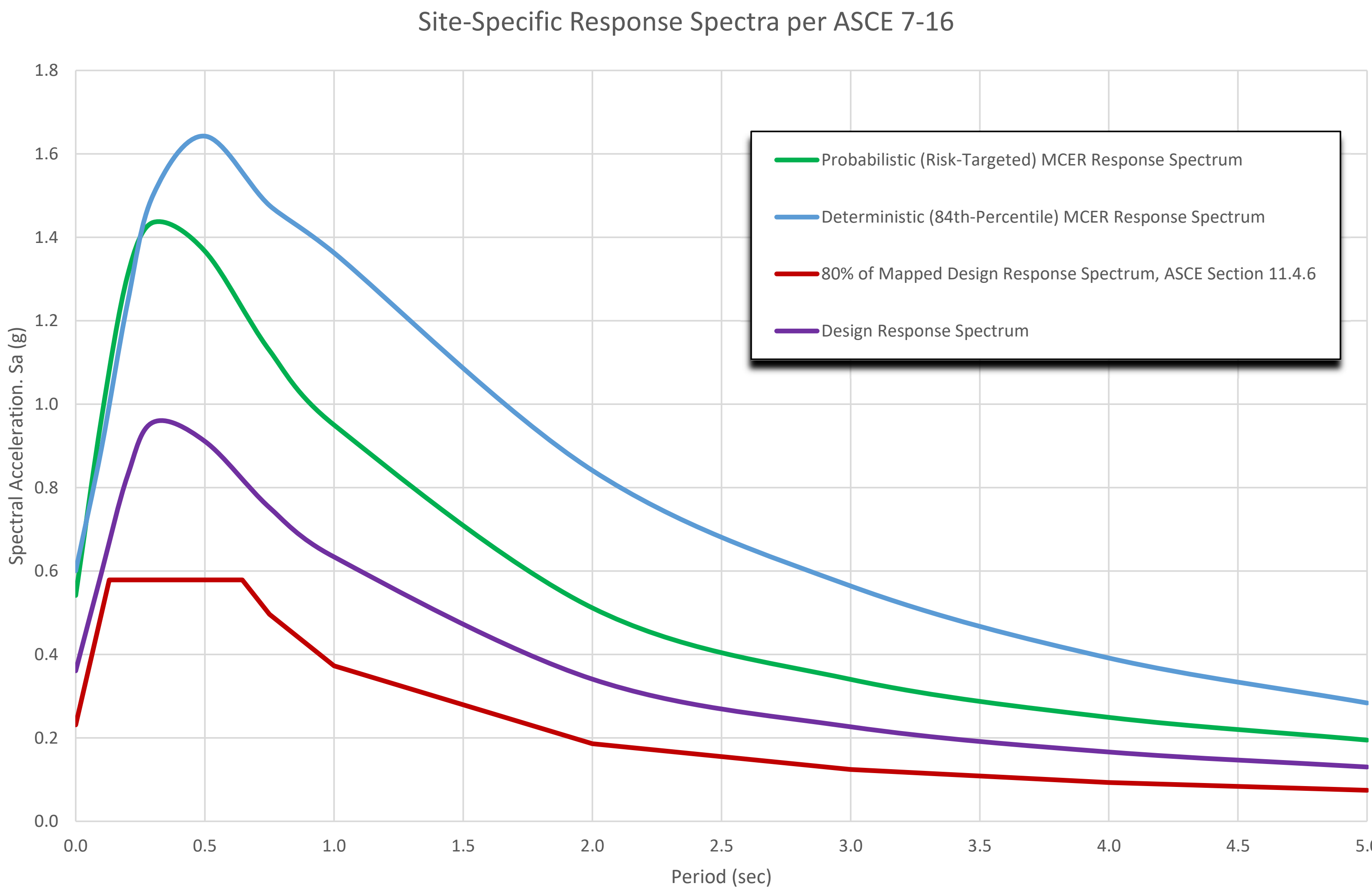
Proposed Eddy Jones Industrial Distribution 260 Eddy Jones Way, Oceanside, CA			
By:	IB/GCD	Date:	April 2024
Job Number:	2021176	Figure:	F.1

Period T (sec)	84th Percentile Spectral Acceleration (g) (Newport Inglewood Offshore)	84th Percentile Spectral Acceleration (g) (Rose Canyon)	84th Percentile Spectral Acceleration (g) (Oceanside alt1)	Deterministic Spectral Acceleration (g)
PGA	0.546	0.469	0.456	0.546
0.10	0.813	0.724	0.700	0.813
0.20	1.130	1.019	0.990	1.130
0.30	1.335	1.179	1.162	1.335
0.50	1.398	1.194	1.186	1.398
0.75	1.193	0.991	0.987	1.193
1.00	1.048	0.852	0.853	1.048
2.00	0.623	0.488	0.499	0.623
3.00	0.403	0.313	0.322	0.403
4.00	0.270	0.210	0.218	0.270
5.00	0.189	0.148	0.155	0.189

NOVA Services

GEOTECHNICAL
MATERIALS
SPECIAL INSPECTION

Proposed Eddy Jones Industrial Distribution 260 Eddy Jones Way, Oceanside, CA			
By:	IB/GD	Date:	April 2024
Job Number:	2021176	Figure:	F.2



NOVA Services

GEOTECHNICAL
MATERIALS
SPECIAL INSPECTION

Proposed Eddy Jones Industrial Distribution 260 Eddy Jones Way, Oceanside, CA			
By:	IB/GD	Date:	April 2024
Job Number:	2021176	Figure:	F.3