GEOTECHNICAL INVESTIGATION

Viewpoint Old Town Apartments
4620 Pacific Highway, San Diego, California

Prepared for:
Viewpoint Development LLC
1635 Pacific Ranch Drive
Encinitas, CA 92024

NOVA Project No. 2021073
July 18, 2022

NOVA Services

4373 Viewridge Avenue
Suite B
San Diego, California 92123
858.292.7575

944 Calle Amanecer
Suite F
San Clemente, CA 92673
949.388.7710

www.usa-nova.com

Chris Livoni
Viewpoint Development LLC
1635 Pacific Ranch Drive
Encinitas, CA 92024

July 18, 2022
NOVA Project No. 2021073

Subject: Geotechnical Investigation
Viewpoint Old Town Apartments
4620 Pacific Highway, San Diego, California

Dear Mr. Livoni:

NOVA Services, Inc. (NOVA) is pleased to present this report describing the geotechnical investigation performed for the proposed Viewpoint Old Town Apartments project. The scope of work performed for this investigation was in general conformance with the scope of work presented in NOVA's proposal dated July 6, 2022, as authorized on July 17, 2022.

NOVA appreciates the opportunity to be of service to Viewpoint Development LLC on this most interesting project. If you have any questions regarding this report, please call us at 858.292.7575 x 413.

Sincerely,
NOVA Services, Inc.

CERTIFIED ENGINEERING GEOLOGIST
MELISSA STAYNER
2707
STATE OF CALIFORNIA

Melissa Stayner, PG, CEG
Senior Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER
JOHN F. O'BRIEN
651
EXP. 3/31/23
GEOTECHNICAL
STATE OF CALIFORNIA

John F. O'Brien, PE, GE
Principal Geotechnical Engineer

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Viewpoint Old Town Apartments
4620 Pacific Highway, San Diego, California

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1. INTRODUCTION

This report presents the results of the geotechnical investigation performed by NOVA Services, Inc. (NOVA) for the Viewpoint Old Town Apartments project located at 4620 Pacific Highway in San Diego (hereinafter ‘the site’). The project will consist of design and construction of five stories of residential units over a parking podium. The objective of NOVA’s work is to characterize the subsurface in a manner sufficient to develop recommendations for geotechnical-related development 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

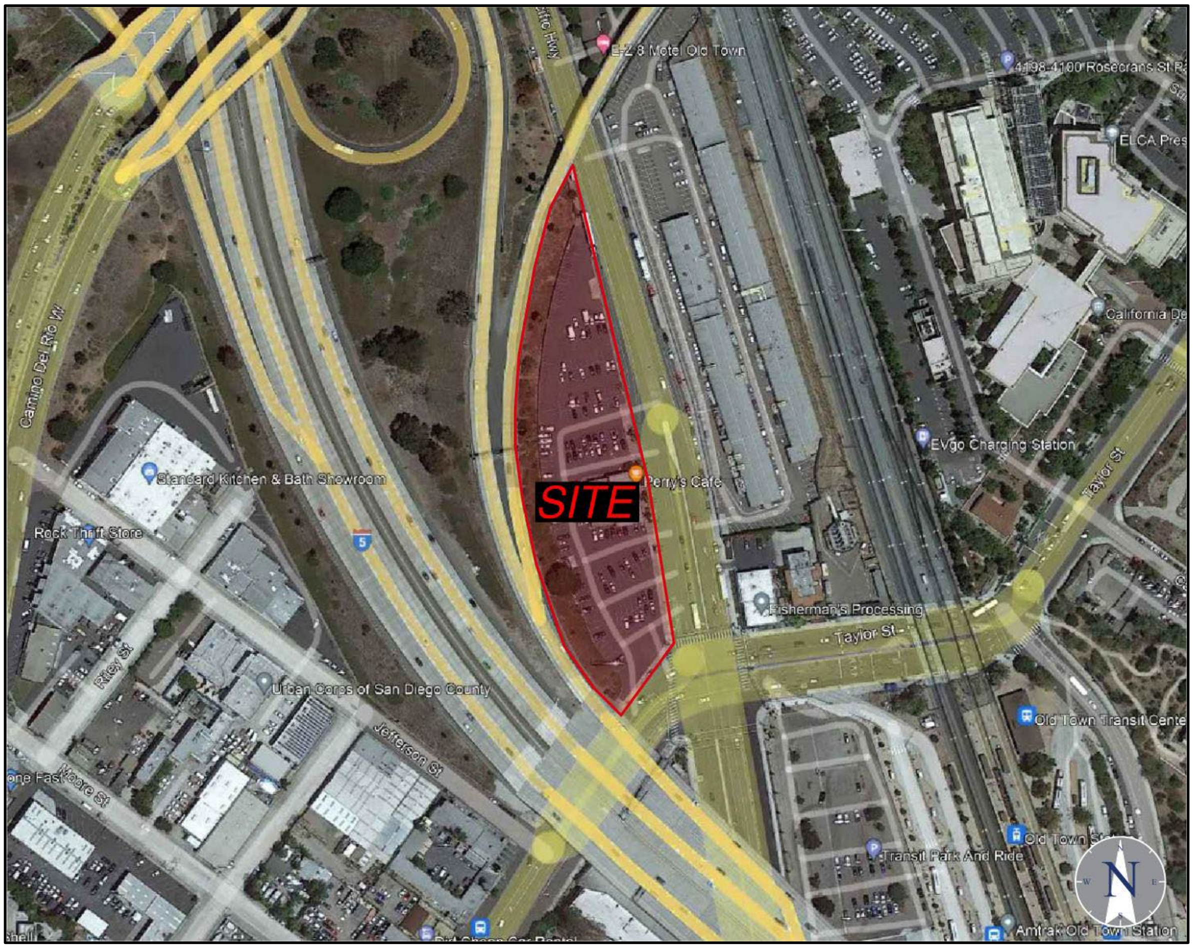

Figure 1-2. Site Location Map
(Source: Google Earth, 2022)

2. SCOPE OF WORK

2.1. Field Investigation

2.1.1 Overview

NOVA's field investigation consisted of a visual reconnaissance of the site and the subsurface exploration summarized below.

- Geotechnical Borings. Two geotechnical borings (B-1 and B-2) were drilled to depths of about 16½ and 71½ feet below the existing ground surface (bgs).
- CPT Soundings. Three cone penetrometer test (CPT) soundings were advanced to depths of between about 40 and 90 feet bgs.
- Geophysical. A shear wave traverse (S-1) was performed to estimate the average shear wave velocity within the top 100 feet (V_{s100}) of the subsurface materials beneath the site.

Figure 2-1 depicts the approximate locations of the subsurface explorations. Plate 1 following the text of the report presents a Subsurface Investigation Map in a larger scale.

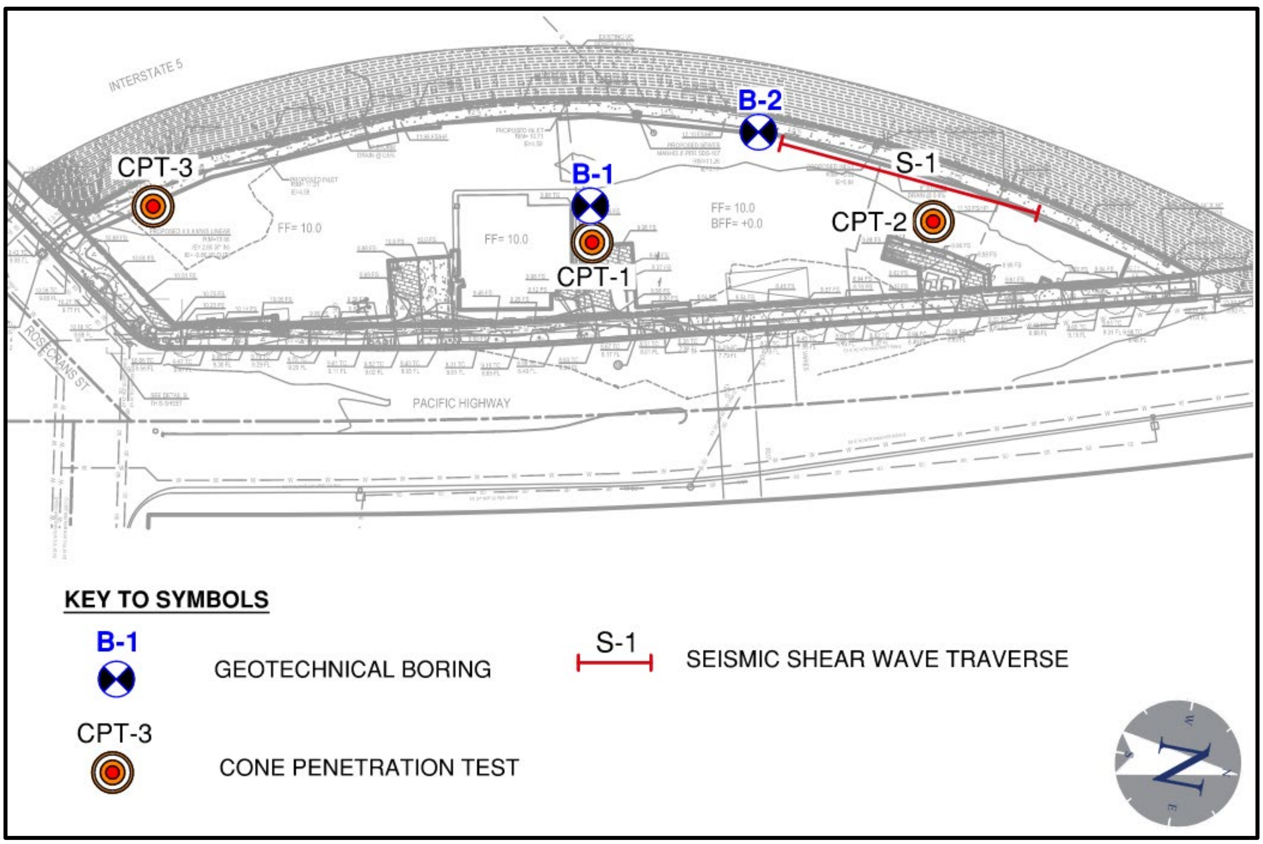

Figure 2-1. Location of Subsurface Explorations

2.1.2 Geotechnical Borings

A NOVA geologist logged the borings and collected samples of the materials encountered for laboratory testing. Relatively undisturbed samples were obtained using a modified California (CAL) sampler, a ring-lined split tube sampler with a 3-inch outer diameter and a 2½-inch inner diameter. Standard Penetration Tests (SPT) were performed in the borings using a 2-inch outer diameter and 1⅜-inch inner diameter split tube sampler. The CAL and SPT samplers were driven using automatic hammers with calibrated Energy Transfer Ratios (ETRs) of about 97%. 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. Soils are classified according to the Unified Soil Classification System.

2.1.3 CPT Soundings

Three CPT soundings in accordance with ASTM D5778 were advanced by a truck-mounted piezocone. Continuous measurements of resistance to penetration of the cone tip (q_c) and the frictional resistance (f_s) were used to evaluate the soil profile, the soil strength and compressibility, and liquefaction potential. Records of the CPT soundings are presented in Appendix C.

2.1.4 Geophysical

A shear wave traverse to estimate the shear wave velocities (V_{s100}) of the subsurface materials was completed by a licensed geophysicist. Shear wave data was used to determine Site Class in accordance with ASCE 7-16 Table 20.3-1, and used in our site-specific ground motion hazard analysis. The shear wave traverse was about 180 feet in length. The approximate alignment of the survey line is shown on Figure 2-1 and Plate 1. Results are presented in Appendix D.

2.2. Laboratory Testing

The strength and compressibility of the dominantly cohesionless subsurface are adequately characterized by the CPT soundings. Accordingly, laboratory testing was limited to index, geochemical and R-Value testing to characterize the NOVA tested select samples to evaluate soil classification and for correlation with engineering properties. The results of the laboratory tests and brief explanations of the test procedures are presented in Appendix E.

2.3. 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 NOVA's findings, conclusions, and recommendations.

3. SITE AND PROJECT DESCRIPTION

3.1. Site Description and Use

3.1.1 Description

The approximately 1.75-acre site is comprised of APN's 442-740-03-00, 442-740-06-00, 442-740-07-00, nominally located 4620 Pacific Highway in San Diego. The site is bounded on the east by Pacific Highway. The arcuate-shaped connector between Interstate 5 North to Interstate 8 East bounds the site to the north and west, with Rosecrans Street to the south.

The site is level, ranging from an elevation of +10 feet mean sea level (msl) on the north side of the site to +11 feet msl on the southern portion of the site.

3.1.2 Use

The site is currently occupied by the single-level Perry's Cafe and a surrounding asphalt parking lot. A 4-foot to 6-foot tall retaining wall bounds the site along the I-5/I-8 connector.

Available historic photography shows that the existing restaurant building was constructed between 1962 and 1964. The site is mapped on the regional geologic map as artificial fill. The 1902 historical topographic map, shows the site is in an area that connected Old Town to Point Loma and is therefore likely composed of alluvium from the San Diego River Delta.

3.2. Proposed Development

3.2.1 Design Basis

NOVA's understanding of current planning for the development is based upon review of permitting drawings (reference, *Site Development Plans, Viewpoint Old Town, 46220 Pacific Hwy, San Diego, CA 92110*, 38 Sheets, carrierjohnson + culture, plot date 3/31/2022, hereinafter 'CJC 2022').

3.2.2 Architectural

Development will consist of constructing five stories of residential units over a podium with mixed uses, residential use, and above-grade parking. Design will provide for one partial level of below-grade parking. The existing Perry's Cafe (constructed in 1966) will be retained and the new structure developed around the restaurant.

The new structure will provide 221 dwelling units, with 32 affordable units. The podium level will include a pool and a variety of other amenities. Three levels of parking will provide 269 parking spaces.

Figure 3-1 reproduces a current architectural schematic.

Figure 3-1. Architectural Schematic
(source: CJC 2022)

3.2.3 Structural

Design is in the preliminary stages. Figure 3-2 (following page) provides an elevation view of the proposed building. As may be seen by review of this graphic, the building will rise seven levels (about 80 feet) above surrounding ground.

By review of Figure 3-1 and Figure 3-2, it can be seen that most of the development will be developed with five levels of apartments and amenities set atop two podium levels of parking. A single level of parking will be developed below ground below a portion of the building, extending to 10 feet below the surrounding ground.

Structural information was not available for this report. However, based upon experience with similar structures, NOVA expects that the building will be developed with 'Type III over Type I' construction. NOVA expects that the below-ground parking and the first level of structure above ground will be constructed in reinforced concrete. The residential levels above the podium will be wood framed.

Preliminary planning indicates that column spacing at the garage level will range to about 30 feet x 40 feet. NOVA expects that column loads (DL+LL) at the garage level may range from about 400 kips to 900 kips.

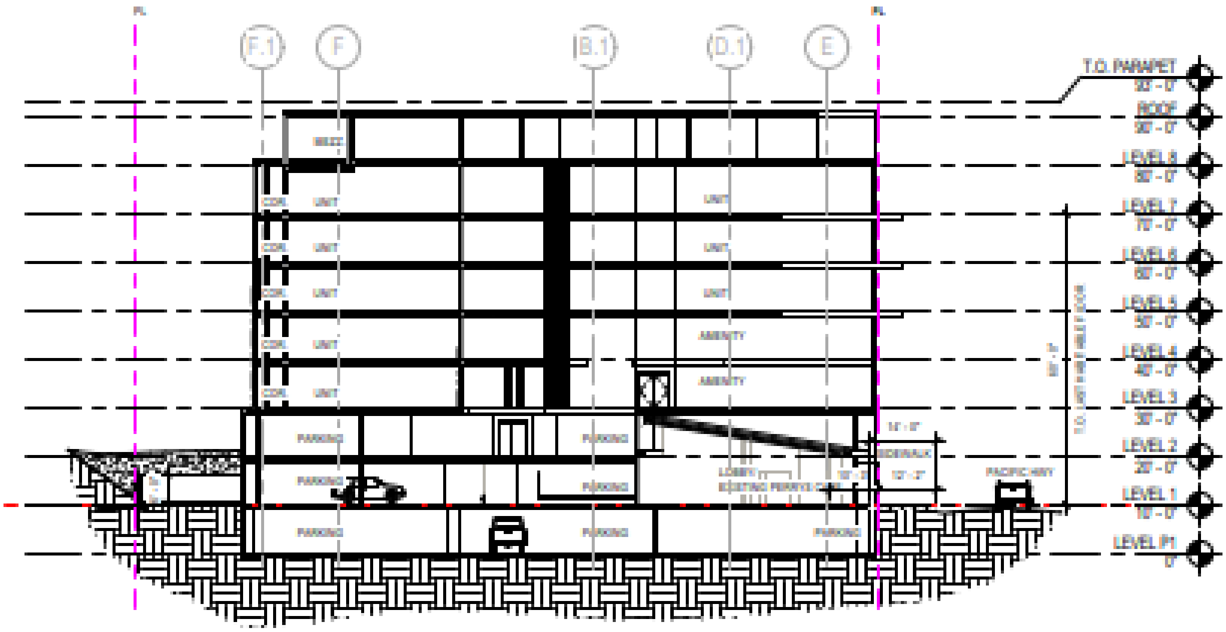

Figure 3-2. West-East Elevation Schematic
(source: CJC 2022, Dwg. A-500, Detail 2)

3.2.4 Civil

Civil drawings are not yet available for review. However, as may be seen by review of Figures 3-1 and 3-2, it is expected that development will include minimal requirements for roadways.

Site improvements may include permanent stormwater Best Management Practices (BMPs) structures, though to NOVA's knowledge such structures have not yet been located.

3.2.5 Potential for Earthwork

With the exception of the partial subterranean garage, site grades will be adapted to the existing groundform, minimizing earthwork. The partial subterranean garage will extend across the west-east limits of the structure between about Column Line 11 and Column Line 19.2, enclosing about 18,500 square feet.

Anticipating soil removal of up to about 12 feet over this area, a neat (dimensional) volume of about 8,200 cy^3 (about 11,500 tons) would be excavated. The depth of this excavation will require temporary shoring. Temporary dewatering will also be required to allow construction in the dry.

4. GEOLOGY AND SUBSURFACE CONDITIONS

4.1. Regional Geology

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 a sequence of fill and/or young alluvial flood plain deposits, Quaternary bay deposits, and Quaternary old paralic deposits.

Figure 4-1 presents the regional geology in the vicinity of the site. Plate 1 following the text of this report presents the geologic cross-section across the site.

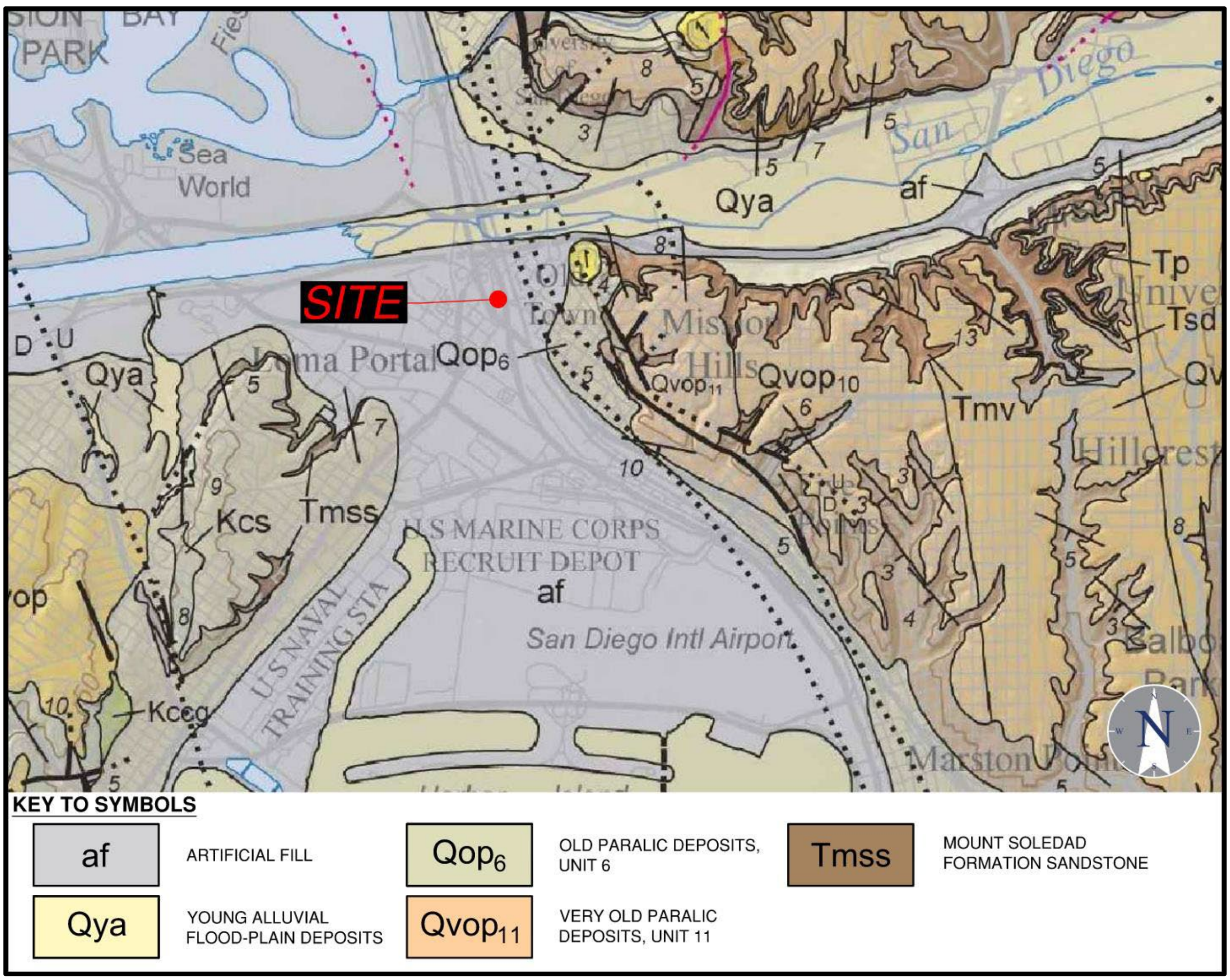

Figure 4-1. Regional Geology Map (Kennedy and Tan, 2008)

4.2. Site-Specific Geology

Descriptions of the materials encountered during the investigation are presented below.

Fill/Quaternary young alluvial flood-plain deposits (af/Qya): Fill/young alluvium was encountered in each of the borings to a depth of about 15 feet bgs. The fill/alluvium generally consisted of loose to medium dense sand with silt, silty sand, and clayey sand. The borings and CPT data indicate that the upper few feet are compacted. Figure 4-2 depicts the fill/alluvium.

Figure 4-2. Fill/Alluvial deposits in Boring B-1

Quaternary bay sediments (Qmo): The fill/alluvium is underlain by about 10 feet of bay sediments, soils that are common to areas of the San Diego shoreline that were developed by hydraulic filling. These soils consisted of medium dense to dense silty sand and medium stiff sandy clay/sandy silt. Figure 4-3 (following page) depicts the bay sediments.

Figure 4-3. Bay Sediments in Boring B-1

Quaternary Old Paralic Deposits (Qop): Late to middle Pleistocene old paralic deposits were encountered beneath the bay deposits at a depth of about 25 feet bgs to the maximum-explored depth. As encountered in Boring B-1, these deposits consisted of medium dense to dense sand with silt, silty sand, and clayey sand. Figure 4-4 depicts the old paralic deposits.

Groundwater: Groundwater was encountered in the borings at depths of about 9½ and 10 feet bgs, corresponding to elevations of about 0 and ½ feet msl. The need for temporary dewatering should be anticipated during construction, as the finished floor of the subsurface parking level is planned to be set at elevation 0 feet msl.

Figure 4-4. Old Paralic deposits in Boring B-1

5. GEOLOGIC, SOIL AND SITING HAZARDS

5.1. Faulting and Surface Rupture

5.1.1 Regional

Major known active faults in the region generally consist of *en echelon*, northwest striking, right-lateral, strike-slip faults. These include the San Andreas, Elsinore, and San Jacinto Faults located northeast of the site, and the San Clemente, San Diego Trough, Agua Blanca-Coronado Bank Faults and Newport-Inglewood-Rose Canyon Fault Zone located to the west of the site.

Earthquake Fault Zones have been established along known active faults in California in accordance with the Alquist-Priolo Earthquake Fault Zoning Act. 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 occupancy 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.

5.1.2 Faulting in the Site Vicinity

The site is not located in an Alquist-Priolo Earthquake Fault Zone. The nearest active fault is located about 1.5 miles south of the site within the Silver Strand section of the Newport-Inglewood-Rose Canyon Fault Zone (NIRC), which is recognized to have the potential for a Magnitude 6.99 seismic event. Evidence of active faulting was not observed at the site during the field investigation. The probability of fault rupture is considered very low.

Figure 5-1 (following page) shows the locations of known faults in the region of the site. Active faults are presented in orange, potentially active faults with displacement dating between 11,700 years and 700,000 years b.p. are presented in green, and undifferentiated Quaternary faults are presented in purple.

5.2. City of San Diego Seismic Safety Study

Figure 5-2 locates the site on the City of San Diego Seismic Safety Study map. The site is in Geologic Hazard Category 31, defined as high potential for liquefaction (City of San Diego, 2008).

NOVA performed a liquefaction analysis for this project, the results of which are discussed in the following section.

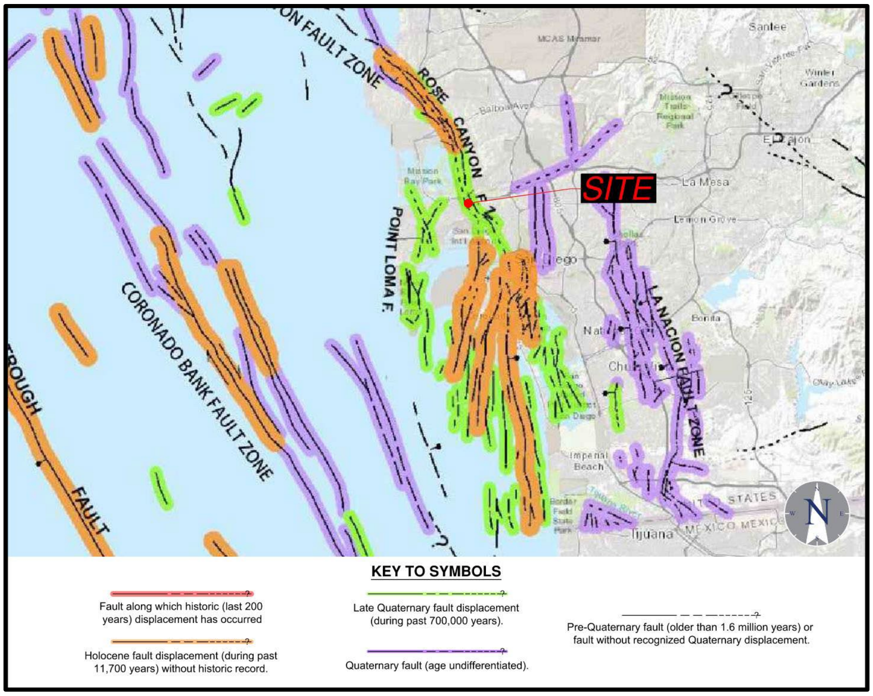

Figure 5-1. Fault Map (CGS, 2022)

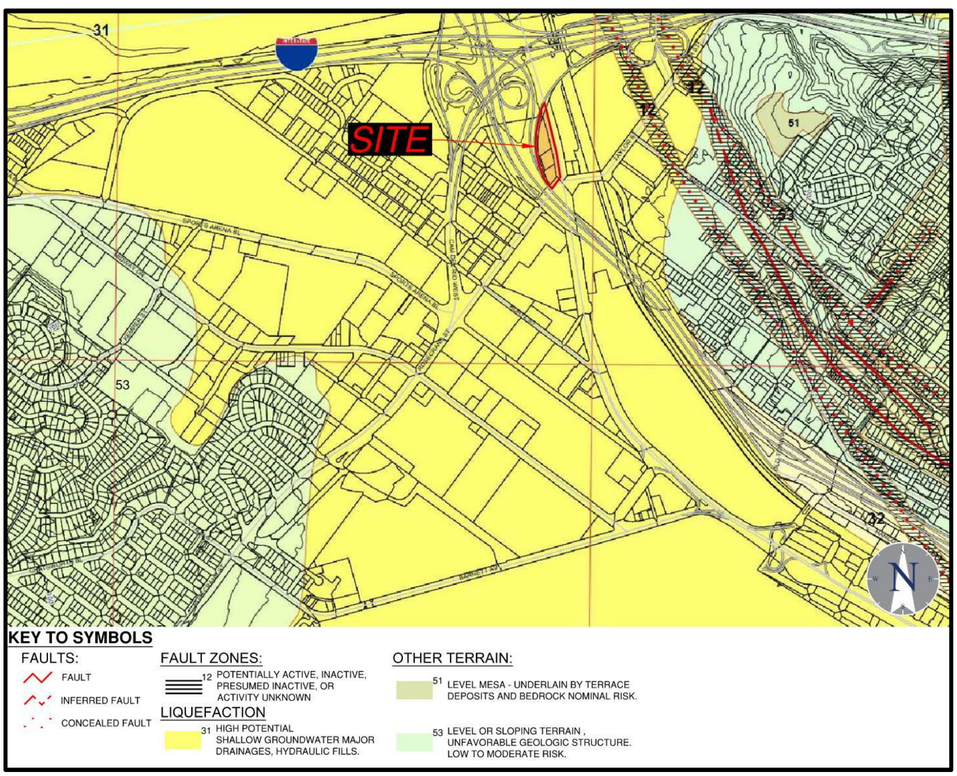

Figure 5-2. Site Location on City of San Diego Seismic Safety Study Map
(source: City of San Diego, 2008)

5.3. Site Class

A geologic hazard likely to affect the project is ground shaking as a result of movement along an active fault zone in the vicinity of the subject site. Based on the shear wave traverse, the site may be classified as Site Class D. The site is subject to liquefaction (Site Class F); however, ground improvements will be performed, which will mitigate the liquefaction settlement, and therefore the site will be Site Class D. For a Site Class D, a site-specific ground motion hazard analysis (GMHA) is required to be performed in accordance with the requirements of 2019 CBC and ASCE 7-16.

A site-specific GMHA was performed as part of the investigation. As part of the analysis, base ground motions were evaluated in conjunction 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, 2022b) was used to develop seismic hazard curves for various periods and the USGS Risk-Targeted Ground Motion Calculator (USGS, 2022c) 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, 2022d) with the ground motion model “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 Rose Canyon (Newport-Inglewood) 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, 2022e), presented in Table 5-1.

The site-specific Risk-Targeted Maximum Considered Earthquake (MCE_R) 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, 2022) to verify that the values obtained from the site-specific analysis are not less than 80 percent of the accelerations obtained from Section 11.4.6. The site coefficients and maximum considered earthquake spectral response acceleration parameters are presented in Table 5-2.

Tabulated values and graphical plots are attached in Appendix D.

Table 5-1. DSHA Input Parameters

Fault: Rose Canyon	
Mw	6.99
Type	Strike-Slip
Dip (°)	90.0
Rake (°)	180
Width (km)	6.93
R_x (km)	0.64
R_{RUP} (km)	0.64
R_{JB} (km)	0.64
V_{s30} (m/s)	213*
$Z_{1.0}$ (km)	N/A
$Z_{2.5}$ (km)	N/A

***Based on S-Wave Measurements Obtained from Seismic Traverse**

Table 5-2. 2019 California Building Code/ASCE 7-16 Site-Specific Parameters

Site Coordinates	
Latitude: 32.75611	Longitude: -117.20161
Site Coefficients and Spectral Response Acceleration Parameters	**Value**
Site Class	D
Site Amplification Factor at 0.2 Second, F_a	1.000
Site Amplification Factor at 1.0 Second, F_v	2.500
Spectral Response Acceleration at Short Period, S_S	1.519g
Spectral Response Acceleration at 1-Second Period, S_1	0.530g
Spectral Response Acceleration at Short Period, Adjusted for Site Class, S_{MS}	1.519g
Spectral Response Acceleration at 1-Second Period, Adjusted for Site Class, S_{M1}	1.326g
Design Spectral Acceleration at Short Period, S_{DS}	1.013g
Design Spectral Acceleration at 1-Second Period, S_{D1}	0.884g
Peak Ground Acceleration, PGA_M	0.693g

5.4. Liquefaction

'Liquefaction' refers to the loss of soil strength during a seismic event. The phenomenon is observed in areas that include geologically 'younger' soils (i.e., soils of Holocene age), shallow water table (less than about 60 feet depth), and cohesionless (i.e., sandy and silty) soils of looser consistency. The seismic ground motions increase soil water pressures, decreasing grain-to-grain contact among the soil particles, which causes the soils to lose strength.

Resistance of a soil mass to liquefaction increases with increasing density, plasticity (associated with clay-sized particles), geologic age, cementation, and stress history.

The CPT data was used in analyses of liquefaction potential using a peak ground acceleration (PGA) of 0.693g, an earthquake magnitude of 7.0, and groundwater depth of 9.7 feet bgs. The analyses indicate that liquefaction of the subsurface will occur in the event of a major earthquake. Appendix F presents the liquefaction analyses. Figure 5-3 depicts the evaluation of liquefaction potential at CPT-1, from which settlement on the order of 3 inches is expected at this location in the design-basis seismic event. Post-liquefaction ground settlement indicated by the three separate soundings range from 2 inches to 3 inches.

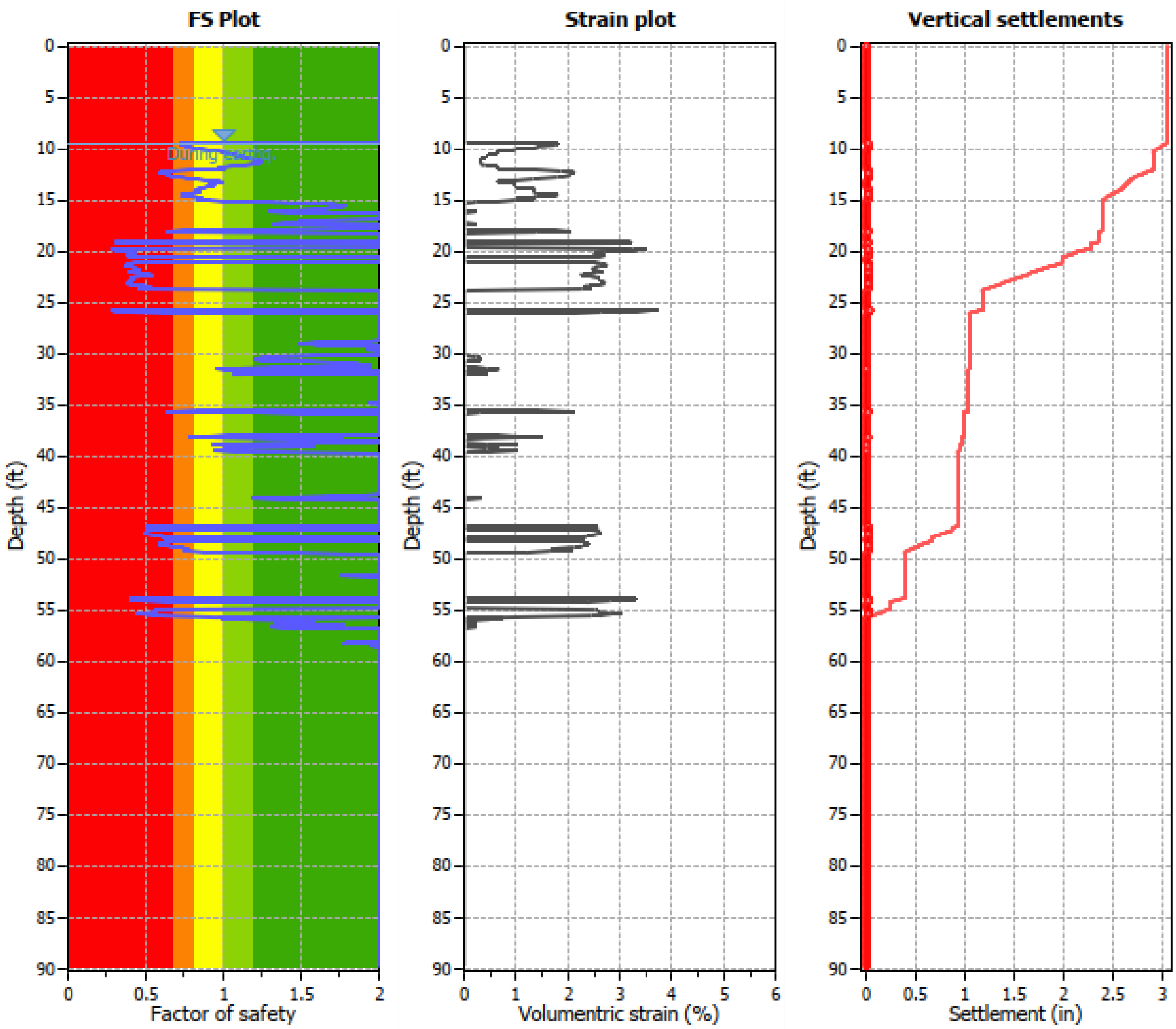

Figure 5-3. Estimate of Post-Liquefaction Settlement, CPT-1

As shown in the liquefaction-related settlement depicted on Figure 5-3, about ⅔ of the settlement occurs over the interval from the groundwater level (about 10 feet depth) to about 25 feet bgs. The remainder of the settlement occurs below this level, extending to about 55 feet bgs.

Estimating liquefaction-related ground settlement is complex and inexact. To address this uncertainty, data obtained from the CPT soundings considered estimates of liquefaction-related settlement using varying procedures. Figure 5-4 provides a graphic summarizing the results of these analyses, considering liquefaction as it could occur in subsurface conditions represented by each CPT sounding. As may be seen by review of this graphic, it is estimated that settlements in the range 2 inches to 5 inches could occur across the site. NOVA recommends an expected ground settlement of about 2 to 4 inches.

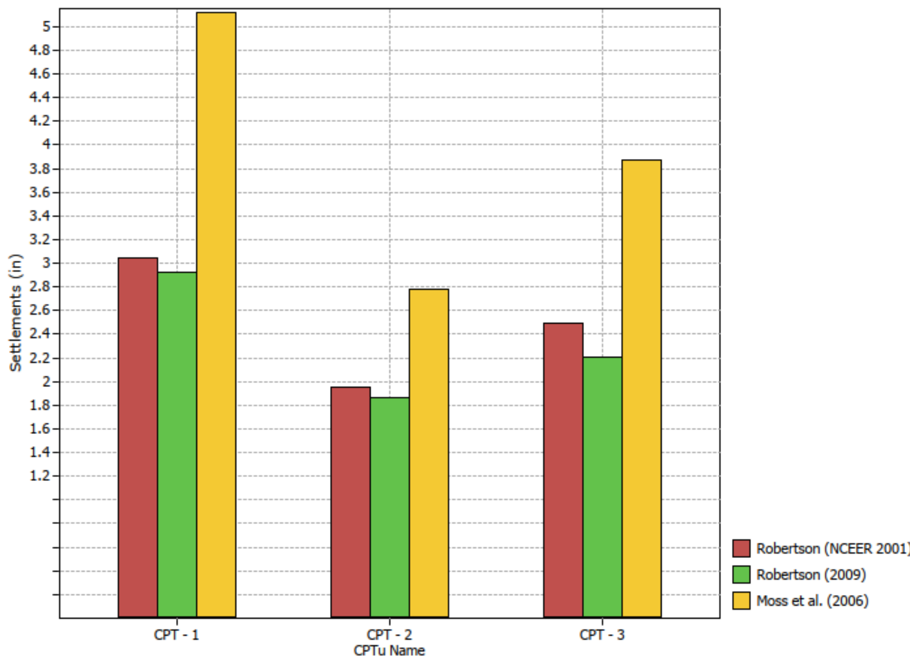

Figure 5-4. Estimates of Liquefaction-Related Settlement, PGA_M = 0.69 g

The estimates provided in Figure 5-4 assume a ground surface acceleration (a) of a = 0.69g. The potential for liquefaction-related settlement to occur at lower levels of ground surface acceleration was also considered. Figure 5-5 provides a summary of this evaluation, from which it can be seen that liquefaction-related settlement on the order of 1 inch will occur at PGA ~ 0.4g

It is the judgment of NOVA that there is a potential for liquefaction to occur within the loose to medium dense alluvial sand and bay sediments underlying the site as a consequence of the design seismic event. Post-liquefaction settlements are estimated to be in range from about 2 inches to 5 inches. Because of the shallow-seated nature of the liquefaction, differential settlement at the ground surface may be high, on the order of 2 inches over a distance of 30 feet.

Despite the liquefaction seismic hazard there is no risk of related phenomena, to include Lateral spreading and seismic compression. Section 7 provides recommendations for ground improvement to mitigate this hazard.

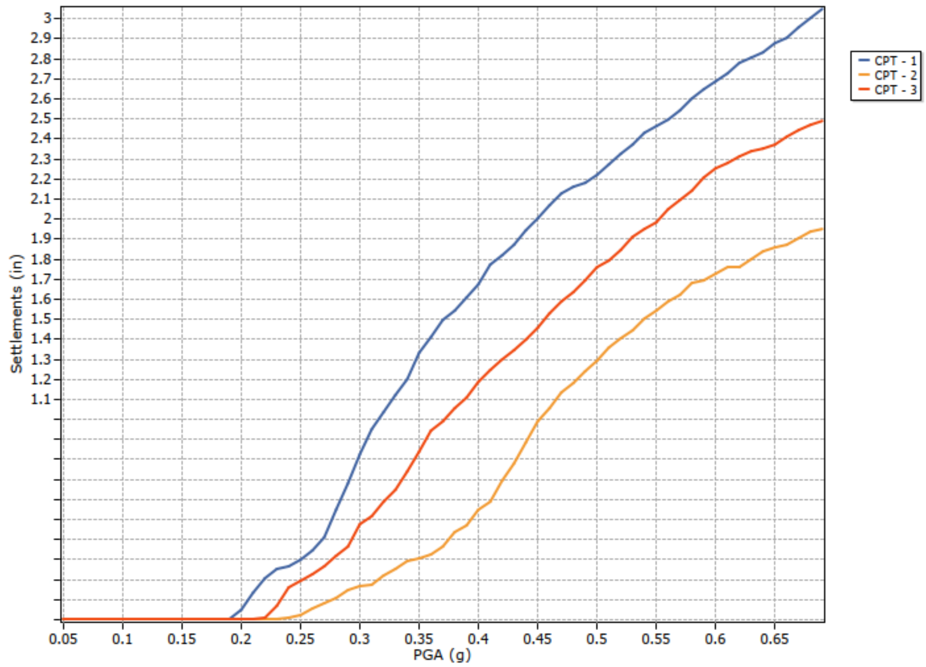

Figure 5-5. Estimates of Post Liquefaction-Related Settlement for Varying Ground Accelerations

5.5. Landslides and Slope Stability

The potential for landslides or slope instabilities to occur at the site is considered negligible given the flat topography and flat-lying geological structure below the site.

5.6. Flooding, Tsunamis, and Seiches

The site is mapped within Zone X (FEMA, 2012), which are areas of minimal flood hazard. As such, the probability for a flood to affect the site is considered low.

The site is not located within a mapped area on the State of California Tsunami Inundation Maps (Cal EMA, 2009); 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 fill/young alluvium unit is considered subject to hydro-consolidation unless it is improved per the ground improvement recommendations within Section 7 of this report.

6. CONCLUSIONS

Based on the results of this investigation, NOVA considers 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 those listed below.

- There are no known active or potentially active faults underlying the site. The primary 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 fill/young alluvial flood-plain deposits and saturated bay deposits to a depth of about 25 feet bgs. Old paralic deposits were encountered at 25 feet bgs to the maximum depth explored. The upper two units are potentially liquefiable should a significant seismic event occur. Liquefaction-related settlements on the order of 2 to 5 inches are estimated. Mitigation of potentially liquefiable soils typically consists of ground improvement or deep foundations. Ground improvement by means of aggregate piers or deep soil mixing may be used to mitigate this hazard. Section 7 addresses these considerations.

- The unsaturated soils above groundwater are potentially compressible. Ground improvement is recommended to improve subgrade support and reduce the potential for settlement. Section 7 addresses these considerations.

- The on-site soils are anticipated to have a very low to 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.

- Excavations should be achievable using standard heavy earthmoving equipment in good working order with experienced operators. Excavation bracing may be required.

- Following ground improvement to limit of both static and liquefaction-related settlements to acceptable levels, the proposed building can be supported on shallow foundations. Foundation recommendations are provided in Section 7.

- Groundwater was encountered at a depth of 9.7 feet bgs, corresponding to elevations of about +0.3 feet msl, and dewatering operations should be anticipated during construction.

- The infiltration feasibility condition category is “No Infiltration” within the fill/young 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. If these recommendations appear not to address a specific feature of the project, please contact NOVA for additions or revisions to the recommendations. The recommendations presented herein may need to be updated once final plans are developed.

7.1. Earthwork

7.1.1 General

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.

7.1.2 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 criteria 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 and observed by the geotechnical consultant.

7.1.3 Compacted Fill

Engineered fill/backfill should be a mineral soil free of organics, regulated chemicals, or otherwise toxic constituents, with the materials characteristics listed below:

- at least 40% by weight finer than ¼-inch;
- classified as GW, GM, GC, SW, SM, or SC after ASTM D2487;
- maximum particle size of 6 inches; and,
- expansion index (EI) of less than 20 (i.e., EI < 20, after ASTM D4829).

Much of the existing fill and alluvium will conform to the above criteria.

Compacted fill beneath structures should be moisture conditioned to just above its optimum moisture content, placed in 6- to 8-inch-thick loose lifts, then densified to at least 95% relative compaction after ASTM D1557 (the 'modified Proctor'). Outside the structures, utility trench backfill and subgrade soils beneath pedestrian hardscape should be compacted to at least 90% relative compaction. The top 12 inches of subgrade soils beneath vehicular pavements should be compacted to at least 95% relative compaction.

7.1.4 Imported Soil

Any imported soil should conform to the criteria for engineered fill cited above. The source(s) of imported soil should be observed and, if appropriate, tested by NOVA prior to transport to the site to evaluate suitability for the intended use.

7.1.5 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.

7.1.6 Excavation Characteristics

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

7.1.7 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, used as landscape material, or disposed of off-site.

7.1.8 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. Ground Improvement

7.2.1 Potentially Applicable Ground Improvement Technologies

Ground improvement to mitigate liquefaction risk and diminish compressibility of a soil mass is widely applied. In particular, the liquefaction hazard at hundreds of sites within the continental United States has been addressed by ground improvement.

A variety of ground improvement technologies can be applied to conditions comparable to those found at this site. Figure 7-1 depicts the variety of alternatives are available for ground improvement, comparing the adaptability of these alternatives to dominantly sandy soils that underlie this site.

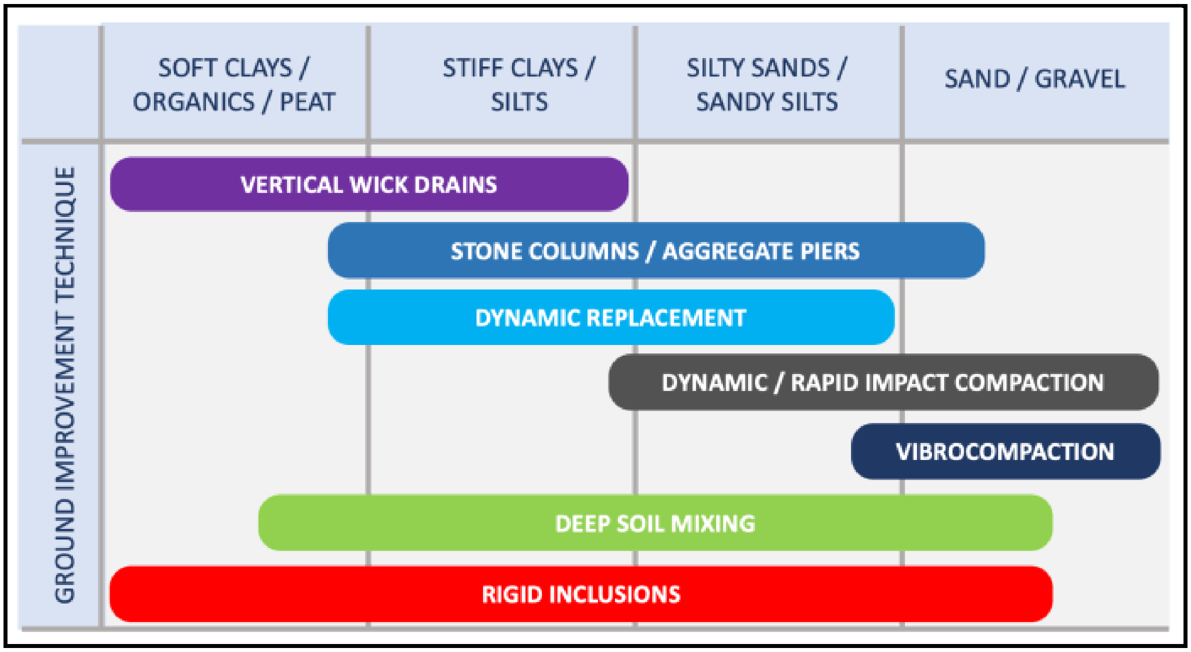

Figure 7-1. Ground Improvement Techniques for Soils of Varying Gradation
(source: Civil + Structural Engineer, March 2021)

This evaluation considered both deep soil mixing ('DSM') and aggregate piers ('Vibro Piers') as alternatives for ground improvement. Both technologies are widely applied in this area of California.

1. Deep Soil Mixing. DSM is a ground improvement technology that employs *in-situ* mixing of soil with cementitious material (most commonly, cement) to harden and stiffen the ground. the technology is vended by a variety of specialty contractors, each with their own specialty equipment and means of soil mixing.

   As applied in this instance, DSM would involve construction of an in-ground grid of soil cement shear walls. The grid constrains the enclosed soil against developing shear strains and related excess pore water pressures that can effect liquefaction. Figure 7-2 (following page) depicts the DSM grid enclosing a soil at risk for liquefaction.

   The grid pattern for DSM is usually expressed in the form of an 'area replacement ratio' (A_r). Initial evaluations for this site anticipate A_r in the range A_r = 30% - 40%. As applied in this instance, mixing would extend over a depth interval of about 20 feet, from about El +5 feet msl to El-15 feet msl. The DSM grid might be on the order of 15 feet x 15 feet in plan dimension across the limits of the planned building. Ground improved by DSM will support shallow foundations with net allowable bearing (q_a) on the order of q_a ~ 6,000 psf.

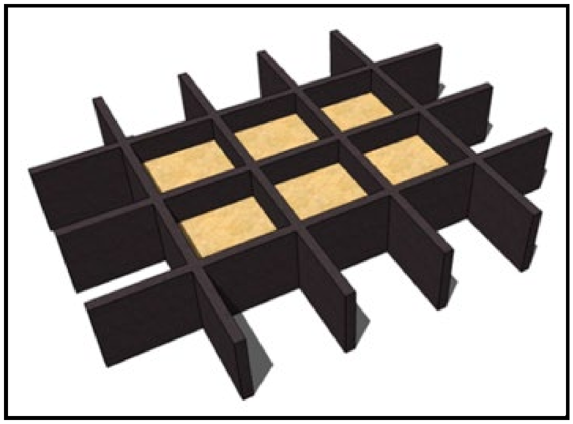

Figure 7-2. Idealized DSM Grid Pattern
(source: Nguyen, *et al*., 2013)

2. Aggregate Piers ('Vibro Piers'). The vibro-compaction technique utilizes a heavy, high-energy vibrator to penetrate the soil to the design depth. At sites such as this, with a relatively high groundwater level, penetration of the vibrator will be supported by displacement of the soil water jetting out the tip. Once the vibratory compactor reaches the design depth, crushed stone is added at the ground surface to the annular space around the vibrator. The stone falls through the space to the vibrator tip and fills the void created as the vibrator is lifted several feet. The vibrator is lowered, densifying and displacing the underlying stone. The vibro replacement process is repeated in lifts until a dense stone column is constructed to the ground surface.

   The technology is reliant upon the ability of the soil mass to respond to the vibratory energy. Though several variables affect this response, the principal variable in this regard is the 'fines content' of the soil mass; that is the portion of the soil mass that is silt and clay-sized, as described by the fraction finer than the U.S. No. 200 sieve, 0.075 mm. Figure 7-3 depicts this relationship. The gradation of the soils at this site largely conforms to the gradation limits of the white-shaded area of Figure 7-3, suggesting that the technology would be successful at this site.

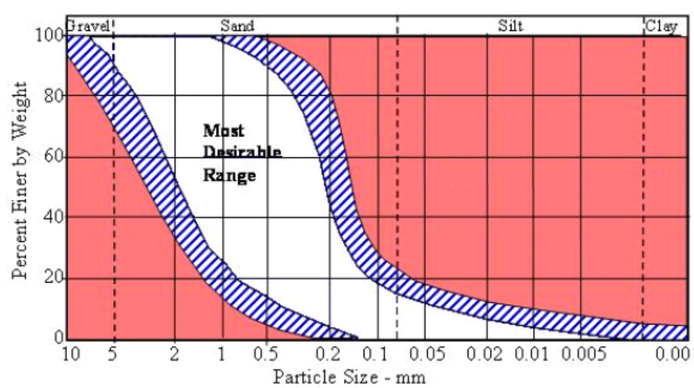

Figure 7-3. Gradation of Soils Most Adaptable to Vibratory Compaction

A field of aggregate piers can support shallow foundations with net allowable bearing on the order of q_a ~ 6,000 psf. As employed in this instance, it is expected that a field of 36-inch diameter vibro piers placed on an 8' grid (A_r ~10%) would extend from the base of foundations to about El -20 feet msl.

7.2.2 Preferred Ground Improvement Technology

The aggregate pier ('vibro pier') alternative will likely offer a marginal cost savings over the DSM alternative for what will largely be similar foundation performance. However, several aspects of design and construction that will be particular to this site diminish this apparent advantage. These considerations are discussed below.

1. Site Limitations to Aggregate Piers. As is discussed in Section 3 (and evident by review of Figure 3-2), the east side of the building extends to the property line. It is normal that aggregate piers extend at least half their penetration depth beyond the limits of the structure for which ground treatment is undertaken. This requirement would complicate the use of aggregate piers, likely adding cost.

2. Savings On Dewatering. DSM creates a low permeability soil mass. As such, the technology can be used in the partial below-grade garage to limit (and practically eliminate) the need for dewatering. This action alone could lead to a consequential cost savings depending upon the efficiency with which the garage excavation and construction is completed and the efficiency of the dewatering system. Elimination of the risk of dewatering removes a considerable site development risk. Dewatering is among the most claims-prone elements of civil construction.

3. Savings on Excavation Bracing. If aggregate piers are employed, the excavation for the partial underground garage will be required to be shored with a 'soldier beam and lagging' system. If DSM is employed, the soil treatment can be adapted to eliminate the need for shoring, creating a stabilized wall that will allow an unbraced excavation.

In consideration of the foregoing, it is the judgment of NOVA that DSM is preferred over aggregate piers its expected superior performance.

Final design for implementation of either DSM or aggregate pier construction would be completed by a specialty contractor, providing ground improvement on a 'design-build' basis. NOVA will coordinate with you in identifying prospective contractors, obtaining rough-order-of-magnitude contractor's estimates, developing outline specifications for implementation, and developing bid requests. These activities should proceed as structural and civil-related designs become more developed.

7.3. Temporary Excavations

7.3.1 Responsibility

The recommendations provided in this section are intended to provide guidance for development of both unretained ('unbraced') and retained ('braced') excavations.

It is the sole responsibility of the contractor to provide an excavation that is safe, with deflections that do not damage nearby structures or utilities. If braced excavations are developed, this design of temporary shoring should be performed by a qualified shoring engineer. When excavations are active, the contractor should provide a properly trained and empowered Competent Person for temporary excavation safety.

7.3.2 Unbraced Excavations

Temporary excavations 3 feet deep or less can be made vertically. Deeper temporary excavations in fill should be laid back no steeper than 1:1 (horizontal:vertical). The faces of temporary slopes should be inspected daily by the contractor's Competent Person before personnel are allowed to enter the excavation. Corrective action should be implemented to address any zones of potential instability, sloughing, or raveling should be brought to the attention of the engineer and 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.

7.3.3 Braced Excavations

For design of cantilevered shoring with level backfill, an active earth pressure equal to a fluid weighing 35 pounds per cubic foot (pcf) can be used. For design of tied-back shoring with level backfill, a rectangular earth pressure distribution with a maximum pressure of 23H pounds per square foot (psf), where H is the height of shoring in feet, can be used. Alternatively, a trapezoidal pressure distribution with a maximum pressure of 28H psf at 0.1H down from the top of shoring and 0.2H up from the base of shoring can be used. The surcharge loads from traffic and construction equipment adjacent to the shored excavation can be modeled by assuming an additional 2 feet of soil behind the shoring. An additional 20 pcf should be added for 2:1 (h:v) sloping ground.

For design of soldier piles, an allowable passive pressure of 350 pounds per square foot (psf) per foot of embedment above groundwater or 250 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.

7.4. Construction Dewatering

Groundwater was encountered at an elevation of approximately +0.3 feet msl. If DSM is not undertaken, excavations below groundwater will require dewatering during the construction period. An experienced dewatering subcontractor should evaluate, design, and implement the dewatering system.

NOVA anticipates that a system of shallow wells and well points will be adequate to lower and maintain the groundwater level below the excavation to provide a stable excavation during construction. Dewatering rates, water volumes, drawdown time, radius of influence, and equipment requirements should be considered in the design. Pumping tests to evaluate the hydraulic parameters for the dewatering system design may be required. An NPDES permit from the Regional Water Quality Control Board will have to be obtained by the Contractor for discharge of the dewatering effluent.

Groundwater should be drawn down at least 5 feet below the bottom of the deepest planned excavation to reduce the possibility of wet, unstable soils. Groundwater must remain at this depressed level during construction until structure loads and uplift resistance are sufficient to counteract buoyant forces with groundwater at historic levels.

The Contractor should provide monitoring during construction (e.g., monitoring wells) to ensure that the design depressed groundwater level is maintained during construction. Nuisance groundwater that enters the excavation can typically be removed by a gravel sump pump collection system. The dewatering system should be integrated with the shoring system.

Dewatering will affect the water level outside the excavation. Lowering the water table will result in effective stress increases of the soil supporting nearby structures or improvements, which could result in ground settlement and distress to those structures or improvements. Adjacent structures and improvements should be surveyed by the contractor prior to dewatering and monitored during construction.

7.5. Permanent Slopes and Surface Drainage

7.5.1 Permanent 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, any slopes should be planted with vegetation that will reduce the potential for erosion.

7.5.2 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 should discharge directly into a closed drainage system.

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.

7.6. Shallow Foundations

7.6.1 General

If ground improvement by either DSM or vibro piers is undertaken, the building may be supported on shallow foundations in conformance with the geotechnical criteria provided in this section. Note that these 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 structural engineer, incorporating the geotechnical parameters described herein and the requirements of applicable building codes.

7.6.2 Spread Footings

Following ground improvement, the proposed building can be supported on shallow spread footings with bottom levels bearing on the improved ground. Footings that are a minimum width of 12 inches set at least 24 inches below lowest adjacent finished grade may be designed for a net allowable bearing capacity of 6,000 psf can be used. This bearing value can be increased by ⅓ when considering the total of all loads, including wind or seismic forces.

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 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.

7.6.3 Interior Slabs-On-Grade

The ground level of the building may be supported on conventionally reinforced on-grade concrete slabs founded atop at least 2 feet of fill compacted to at least 90% relative compaction after ASTM D1557. Conventional concrete slab-on-grade floors should be at least 5 inches thick and reinforced with at least No. 4 bars at 18 inches on center each way. Actual slab thickness and reinforcement should be designed by the structural engineer using a modulus of subgrade reaction (k) of k = 100 lb/in^3.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or 'weakened plane' joints at frequent intervals

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.

7.6.4 Foundation Settlement

Supported on ground improved by either DSM or aggregate piers, foundations will settle on the order of 1 inch or less. This movement will be elastic- occurring approximately as load is applied- such that about 70% of the settlement will be complete during the construction period. Angular distortion due to differential settlement of adjacent, unevenly loaded footings will be less than 1 inch in 40 feet (i.e., Δ./L less than 1:480).

The above estimate is for the static case only. About 1 inch of settlement will occur following a liquefaction event related to the design basis earthquake. Differential movement of this deeper-seated settlement will effect only small (i.e., Δ./L less than 1:480) differential movement at the ground surface.

7.6.5 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.

7.6.6 Foundation Excavation Observations

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

7.7. Hardscape

7.7.1 Subgrade Preparation

The on-site soils beneath hardscape should be excavated to a depth of at least 2 feet below planned hardscape surface. 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 backfilled with soil having an expansion index of 20 or less and compacted to at least 90% relative compaction after ASTM D1557.

7.7.2 Hardscape Section

Exterior concrete slabs should be at least 4 inches thick 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" Standard Specifications for Public Works Construction.

7.8. Conventional Retaining Walls

7.8.1 Foundation Preparation

Conventional retaining walls founded on ground improved as described in Section 7.2 can be supported on shallow spread footings designed as described in Section 7.6.

The ground beneath site walls and retaining walls not connected to buildings, the existing soils should be excavated to a depth of at least 2 feet below bottom of footing. Horizontally, these excavation should extend at least 2 feet outside the planned wall footing, or up to existing improvements, whichever is less. If competent formational materials are exposed, excavation need not be performed. NOVA should observe the conditions exposed in the bottom of excavations to evaluate whether additional excavation is recommended. Any required fill or backfill should have an EI of 20 or less.

7.8.2 Wall Pressures

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 a wall extends below groundwater and cannot be drained, the wall should be designed to resist the incremental hydrostatic pressure. Consideration should also be given to positive side (i.e., the wet face) waterproofing to limit moisture accumulation inside the elevator pit, anticipating water level rise to perhaps El +6 feet msl.

7.8.3 Seismic Increment

Walls taller than 6 feet should include a seismic increment. The seismic load increment (ΔP_E) can be computed for the different conditions of wall yield that are described below.

- Basement wall (i.e., fixed), level backfill: $\Delta P_E = \frac{1}{2}\,\gamma\,H^2\,(0.68)\,(PGA)$ (PGA = 0.69g)
- Cantilever wall, level backfill: $\Delta P_E = \frac{1}{2}\,\gamma\,H^2\,(0.42)\,(PGA)$ (PGA = 0.69g)
- Cantilever wall with sloping backfill: $\Delta P_E = \frac{1}{2}\,\gamma\,H^2\,(0.70)\,(PGA)$ (PGA = 0.69g)

In each of the above cases the resultant acts at 0.33H above the base of the wall.

7.8.4 Drainage

The recommendation for lateral wall loads assumes walls are provided with a backdrain to reduce the accumulation of 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. Alternatively, 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 7-4 (following page) presents typical retaining wall backdrain details. Note that the guidance provided on Figure 7-4 is conceptual. Other options are available.

7.8.5 Backfill

Wall backfill should consist of granular, free-draining material having an expansion index 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.

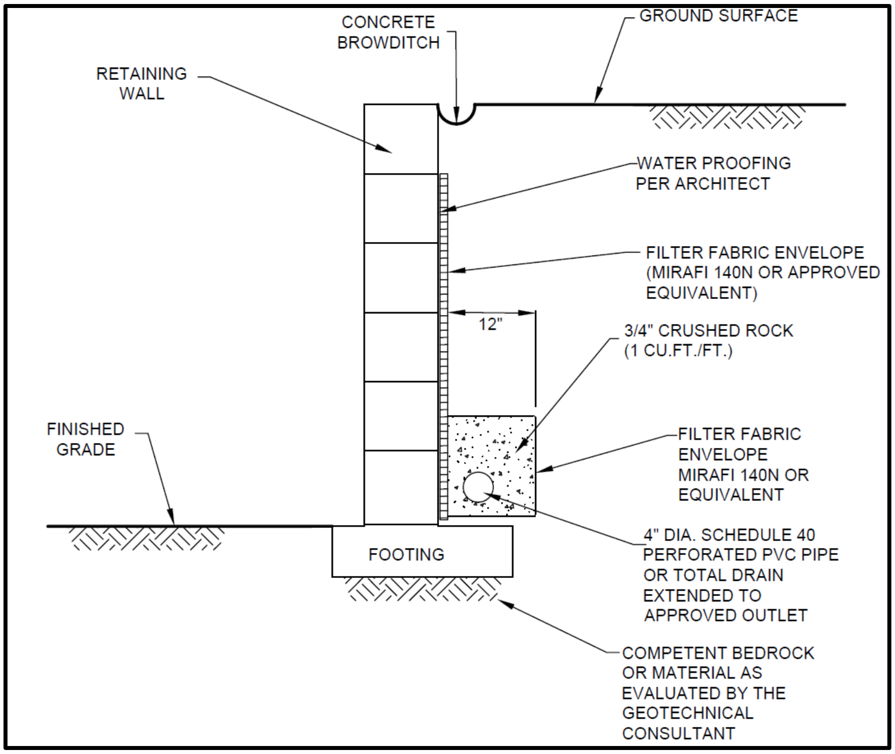

Figure 7-4. Typical Conventional Retaining Wall Backdrain Detail

7.8.6 Elevator Pits

It is expected that the building will include several elevator pits that will extend perhaps 6 feet deeper than the slab around it, bearing on a ground-supported slab.

An elevator pit slab and related retaining wall footings will derive suitable support from the sandy soils around it. Design for the elevator pit walls should consider the circumstances and conditions described below.

1. Wall Yield. NOVA expects that proper function of the elevator pit should not allow yielding of the elevator pit walls. As such, walls should be designed to resist 'at rest' lateral soil pressures and seismic pressures provided above, also allowing for any structural and hydrostatic surcharge.
2. Construction. It is common that construction of elevator walls precedes much of the construction around them. Design of the elevator pit walls should include consideration for surcharge conditions that will occur during construction. Such conditions may include, but not be limited to, surcharges from vehicle traffic, sloping ground above and around the walls, etc.
3. Moisture. Where applicable, consideration should be given to positive side (i.e., the wet face) waterproofing to limit moisture accumulation inside the elevator pit, anticipating water level rise to perhaps El +6 feet msl.

4. Piston. If the elevator pit includes a plunger-type elevator piston, a deeper drilled excavation may be required. NOVA should be consulted regarding recommendations for development of a plunger-type elevator piston.

7.9. Pipelines

For level ground conditions, a passive earth pressure of 350 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, if encountered.

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. Open graded rock should not 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 civil engineer designing the pipeline.

7.10. Pavements

7.10.1 Subgrade Preparation

Soils beneath proposed vehicular pavement areas should be excavated to a depth of at least 2 feet below the planned base course 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 necessary. 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. All soft or yielding areas should be stabilized or removed and replaced with compacted fill or aggregate base.

The excavation should then be backfilled filled with material suitable for reuse as compacted fill.

7.10.2 Pavement Sections

Based upon the indications of laboratory testing, an R-value of 50 may be assumed for preliminary design of pavement sections. The actual R-value of the subgrade soils should be determined after grading, and the final pavement sections provided. Based on an R-value of 50, Table 7-1 provides preliminary pavement structural sections for the assumed Traffic Indexes.

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 / 4 AB	6½ PCC
Heavy Traffic Areas	7.5	5 AC / 6 AB	7 PCC

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

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.

7.11. Corrosivity

Representative samples of the on-site soils were tested to evaluate corrosion potential. The test results are presented in Appendix E.

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.

It should be noted that elevated levels of chloride (0.118% or 1180 parts per million) and low resistivity (240 Ohm-cm) were detected in one of the tested samples. The project architect and/or design engineer should review and consider the chloride content in the project design. A corrosion engineer should be contacted to provide specific corrosion control recommendations.

8. INFILTRATION FEASIBILITY

Full or partial infiltration of stormwater is not recommended for this site, as the fill/young alluvial soils are hydro-collapsible, and the site is in an area designated by the City's Seismic Safety Study as having a high liquefaction potential, with high groundwater and deep hydraulic fill.

Appendix G provides the Infiltration Feasibility Condition Letter for the site.

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. NOVA 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.

10. REFERENCES

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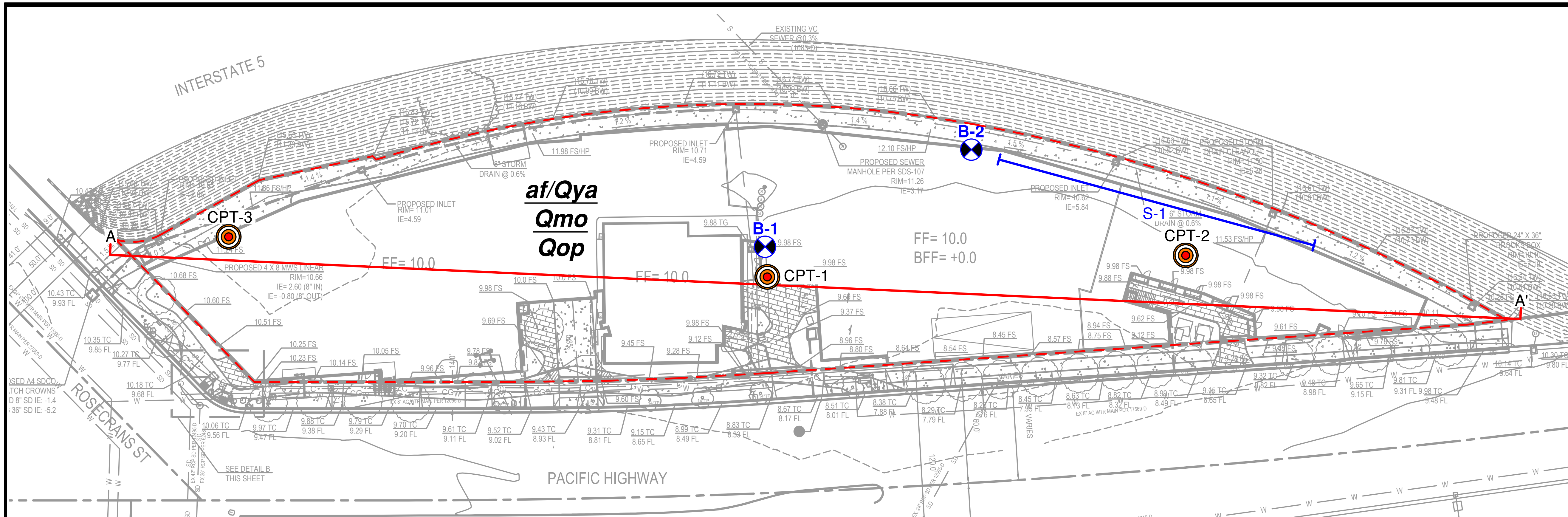
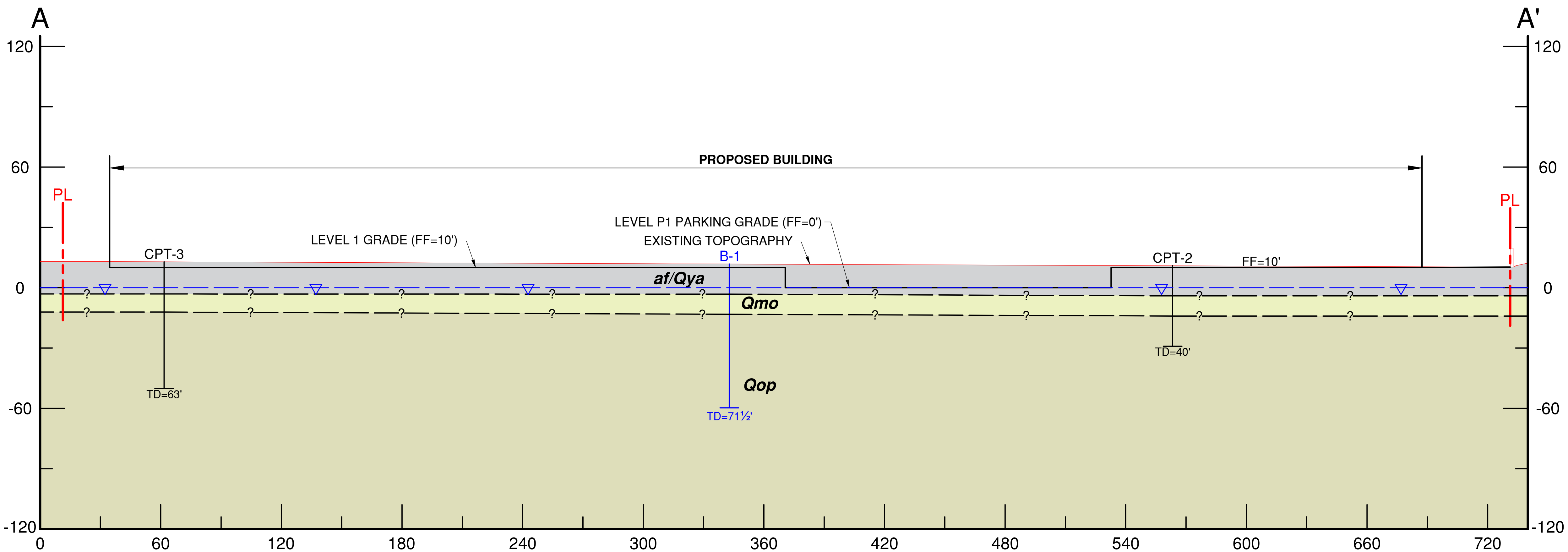
PLATES





KEY TO SYMBOLS

af/Qya	FILL/YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS	B-2 / B-2	GEOTECHNICAL BORING	—?—	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
Qmo	UNDIVIDED MARINE DEPOSITS IN OFFSHORE REGION	CPT-3 / CPT-3	CONE PENETRATION TEST		GROUNDWATER
Qop	OLD PARALIC DEPOSITS				ANTICIPATED LIMITS OF GRADING
		S-1	SEISMIC SHEAR WAVE TRAVERSE	A A'	GEOLOGIC CROSS-SECTION

0 60' 120'

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PROJECT NO.:	2021073
DATE:	JULY 2022
DRAWN BY:	DTJ
REVIEWED BY:	MS
SCALE:	1"=60'
DRAWING TITLE:	

SUBSURFACE INVESTIGATION MAP AND GEOLOGIC CROSS-SECTION A-A'

PLATE NO. 1 OF1

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
	GROUNDWATER SEEPAGE
☒	BULK SAMPLE
⧄	SPT SAMPLE (ASTM D1586)
■	MOD. CAL. SAMPLE (ASTM D3550)
*	UNRELIABLE BLOW COUNTS
——	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: MARCH 26, 2021 **DRILLING METHOD:** 6-INCH HOLLOW STEM AUGER/MUD ROTARY

ELEVATION: ± 10 FT **DRILLING EQUP.:** YETI M10 **GROUNDWATER DEPTH:** 10 FT

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTOMATIC) **NOTES:** ETR~96.5%, $N_{60} \sim \frac{96.5}{60}*N \sim 1.61*N$

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								*5½ IN OF ASPHALT CONCRETE OVER 4½ IN OF AGGREGATE BASE*	
							SM	**FILL/ QUATERNARY YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS (af/Qya):** *SILTY SAND; LIGHT BROWN TO BROWN, SLIGHTLY MOIST, LOOSE, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL*	
			10	11	14.9	103.2		*MEDIUM DENSE*	MD SA RV CR
5			4	6				*LOOSE*	
			3*	5*					
10			7	11			SP-SM	*POORLY GRADED SAND WITH SILT; GRAYISH BROWN, WET, MEDIUM DENSE, FINE GRAINED, SWITCHED TO ROTARY DRILLING*	SA
15			7	11			SM/SC	**QUATERNARY BAY SEDIMENTS (Qmo):** *SILTY SAND/CLAYEY SAND; DARK GRAY, SATURATED, MEDIUM DENSE, FINE GRAINED*	SA
20			5	8			CL/ML	*SANDY CLAY/SANDY SILT; DARK GRAY/BLACK, WET, MEDIUM STIFF, FINE GRAINED, ABUNDANT MICA*	SA
25			19	31			SP-SM	**QUATERNARY OLD PARALIC DEPOSITS (Qop):** *POORLY GRADED SAND WITH SILT; DARK GRAY, WET, DENSE, FINE GRAINED*	SA
30									

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SAN DIEGO, CALIFORNIA

BY: AR	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: B.1

CONTINUED LOG OF BORING B-1

DATE DRILLED: MARCH 26, 2021 **DRILLING METHOD:** 6-INCH HOLLOW STEM AUGER/MUD ROTARY

ELEVATION: ± 10 FT **DRILLING EQUP.:** YETI M10 **GROUNDWATER DEPTH:** 10 FT

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTOMATIC) **NOTES:** ETR~96.5%, $N_{60} \sim \frac{96.5}{60} * N \sim 1.61 * N$

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	26			SP-SM	**QUATERNARY OLD PARALIC DEPOSITS (Qop):** *POORLY GRADED SAND WITH SILT; DARK GRAY, WET, MEDIUM DENSE, FINE GRAINED*	SA
35			21	34			SM	*SILTY SAND; DARK GRAY, WET, DENSE, FINE GRAINED*	SA
40			19	31					SA
45									
50			15	24			ML	*SANDY SILT; DARK GRAY, WET, MEDIUM DENSE, FINE GRAINED*	SA
55									
60									

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SAN DIEGO, CALIFORNIA

BY: AR	REVIEWED BY: MS	PROJECT: 2021073	APPENDIX: B.2

CONTINUED LOG OF BORING B-1

DATE DRILLED: MARCH 26, 2021 **DRILLING METHOD:** 6-INCH HOLLOW STEM AUGER/MUD ROTARY

ELEVATION: ± 10 FT **DRILLING EQUP.:** YETI M10 **GROUNDWATER DEPTH:** 10 FT

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTOMATIC) **NOTES:** ETR~96.5%, $N_{60} \sim \frac{96.5}{60}$*N~1.61*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
60			29	47			SP-SM	**QUATERNARY OLD PARALIC DEPOSITS (Qop):** *SAND WITH SILT; DARK GRAY, WET, DENSE, FINE TO MEDIUM GRAINED*	SA
65									
70			16	26			SM/SC	*SILTY SAND/CLAYEY SAND; DARK GRAY, WET, MEDIUM DENSE, FINE GRAINED*	SA
								BORING TERMINATED AT 71½ FT. GROUNDWATER ENCOUNTERED AT 10 FT.	
75									
80									
85									
90									

LOG OF BORING B-2

DATE DRILLED: JULY 11, 2022 **DRILLING METHOD:** 6-INCH HOLLOW STEM AUGER/MUD ROTARY

ELEVATION: ± 10 FT **DRILLING EQUP.:** CME 75 **GROUNDWATER DEPTH:** 9.7 FT

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN (AUTOMATIC) **NOTES:** ETR~73.9%, $N_{60} \sim \frac{73.9}{60}$*N~1.23*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								*3 IN OF ASPHALT CONCRETE OVER 6 IN OF AGGREGATE BASE*	
	X						SC	**FILL/ QUATERNARY YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS (af/Qya):** *CLAYEY SAND; YELLOW BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, SOME CLAY BLEBS, FEW GRAVEL*	
5	X	■	19	15			SM	*SILTY SAND; YELLOW BROWN, MOIST, MEDIUM DENSE, MEDIUM TO COARSE GRAINED*	
								DARK GRAY, FINE TO MEDIUM GRAINED, MICACEOUS	
10		/	6	7			SC	*CLAYEY SAND; DARK GRAY, WET, LOOSE, FINE GRAINED, SOME INTERBEDDED CLAY LENSES*	
15	X	■	7	6			SM	*SILTY SAND; MOTTLED YELLOW BROWN AND DARK GRAY, WET, LOOSE, FINE TO MEDIUM GRAINED*	CR
		/	26	32			SM	**QUATERNARY BAY SEDIMENTS (Qmo):** *SILTY SAND; GRAY, WET, DENSE, FINE GRAINED*	
20								*BORING TERMINATED AT 16½ FT. GROUNDWATER ENCOUNTERED AT 9 FT AND STABILIZED AT 9.7 FT. BACKFILLED WITH BENTONITE.*	
25									
30									

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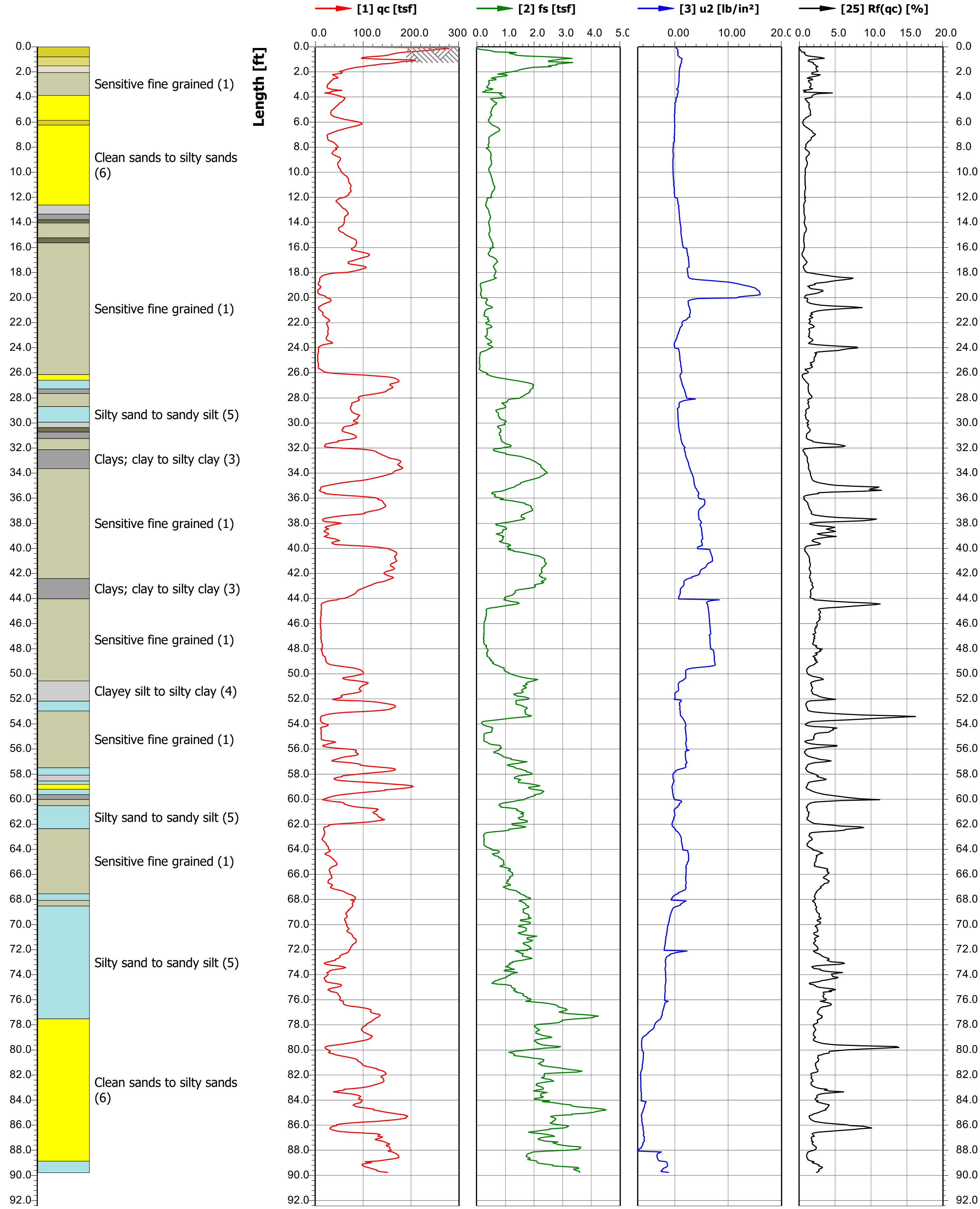
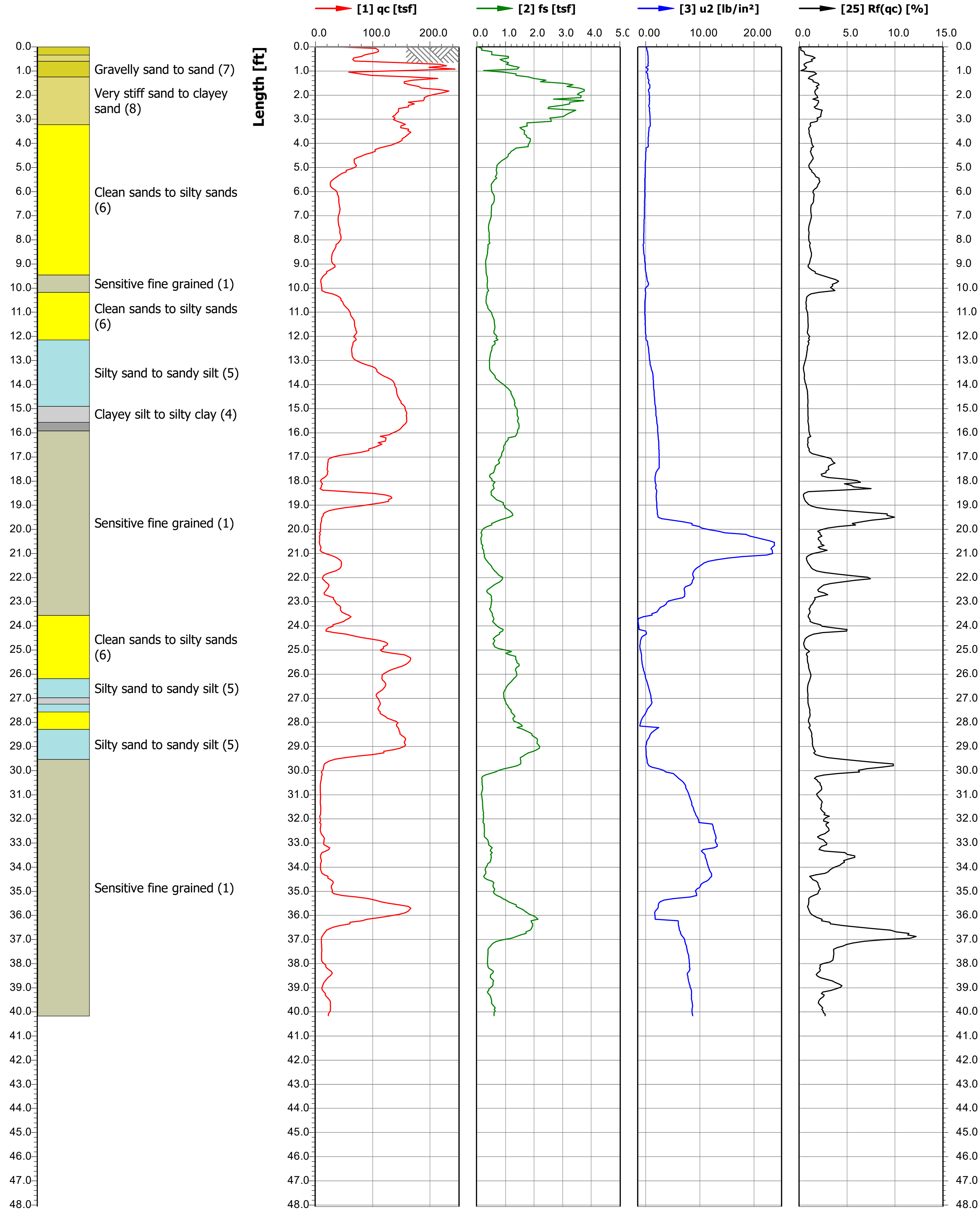
BY: SA	REVIEWED BY: MS	PROJECT: 2021073	APPENDIX: B.4

APPENDIX C
LOGS OF CPT SOUNDINGS

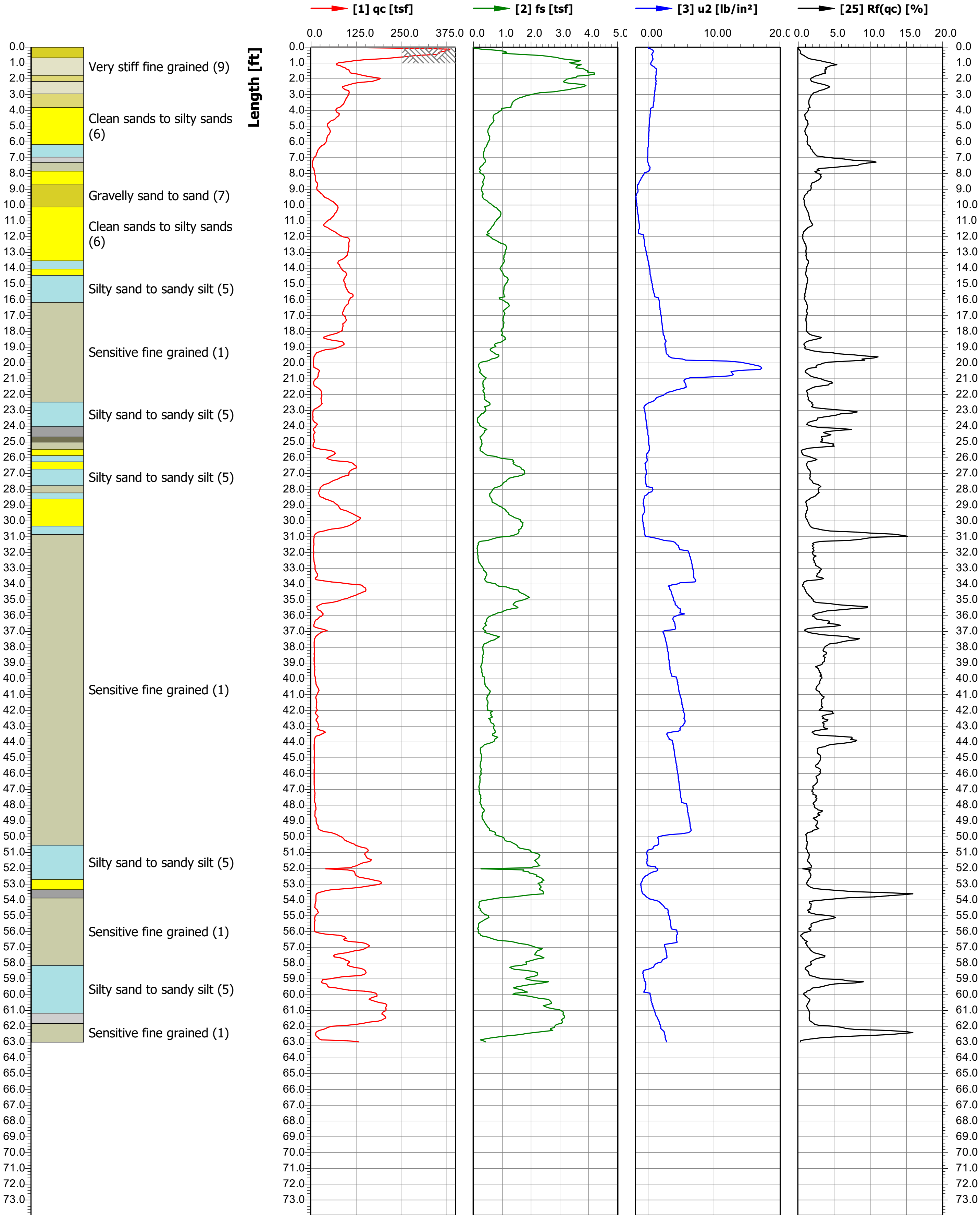
Cone no: 5238	Location:	Position: X: 0.00 ft Y: 0.00 ft	Ground level: 0.00	Test no: CPT 1
Tip/sleeve area [cm^2]: 10 / 150	Project ID:	Client: NOVA	Date: 3/26/2021	Standard / class: /
Area factor a/b: 838.000 / 0.000	Project: Perry's Cafe		Page: 1/1	Scale: 1:125
Pore pressure: U2			Coordinate system:	



Cone no: 5238	Location:	Position: X: 0.00 ft Y: 0.00 ft	Ground level: 0.00	Test no: CPT #2
Tip/sleeve area [cm²]: 10 / 150	Project ID:	Client: NOVA	Date: 3/26/2021	Standard / class: /
Area factor a/b: 838.000 / 0.000	Project: Perry's Cafe		Page: 1/1	Scale: 1:65
Pore pressure: U2			Coordinate system:	

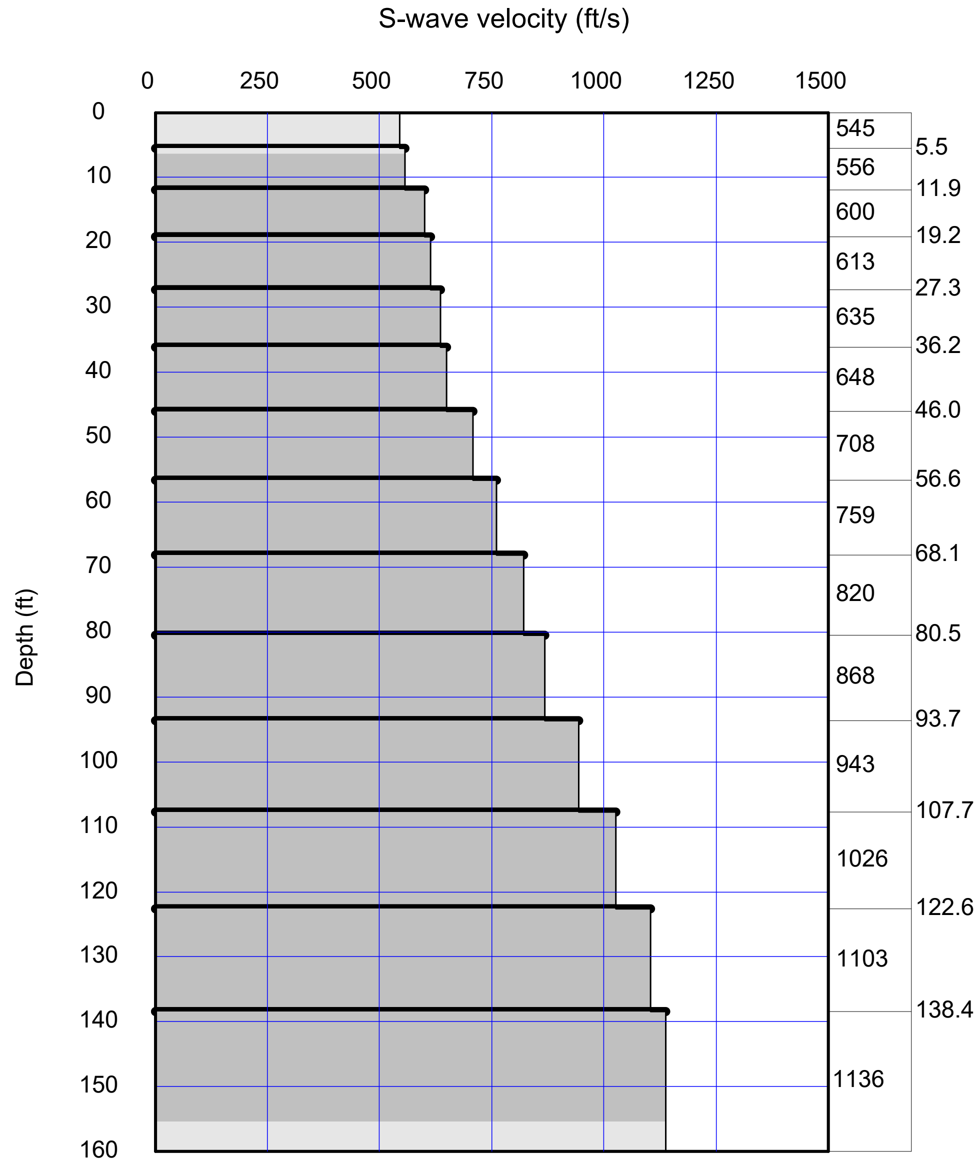


Cone no: 5238	Location:	Position: X: 0.00 ft Y: 0.00 ft	Ground level: 0.00	Test no: CPT #3
Tip/sleeve area [cm²]: 10 / 150	Project ID:	Client: NOVA	Date: 3/26/2021	Standard / class: /
Area factor a/b: 838.000 / 0.000	Project: Perry's Cafe		Page: 1/1	Scale: 1:100
Pore pressure: U2			Coordinate system:	



APPENDIX D
RESULTS OF SHEAR WAVE TRAVERSE AND SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS

SEISMIC LINE SW-1



SHEAR-WAVE VELOCITY MODEL: Average Vs 100ft = 698.6 ft/sec

Site Classification (ASCE 7-16 Ch. 20)- "D" (Stiff Soil profile)

Client: **Nova Services, Inc.** **NOVA Project No. 2021073**

Project Name: **Viewpoint Old Town, San Diego, California**

Survey Line End Coordinates: **32.757079, -117.201864 / 32.756578, -117.201939**

Date: **7/3/22**

PROFESSIONAL GEOPHYSICIST
DONN C. SCHWARTZKOPF
No. 1002
STATE OF CALIFORNIA

TERRA GEOSCIENCES
P.O. Box 1090
Loma Linda, CA 92354

TG Project No. 223847-1

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

Project:	Viewpoint Old Town	**Latitude:**	32.75611	deg		**Calculated By:**	GLC
Client:	Viewpoint Development	**Longitude:**	-117.20161	deg		**Date:**	July 2022
Job No:	2021073	**Vs_{30}:**	213	m/s	(Measured)		

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 Accelaration (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	0.693	0.613	1.1	0.674	0.743	1.1	0.817	0.398	0.318	0.674	0.450	---
0.10	1.106	1.005	1.1	1.106	1.027	1.1	1.130	0.883	0.707	1.106	0.737	---
0.20	1.505	1.353	1.1	1.488	1.367	1.1	1.504	0.994	0.795	1.488	0.992	---
0.30	1.679	1.500	1.125	1.688	1.683	1.125	1.893	0.994	0.795	1.688	1.125	---
0.50	1.624	1.436	1.175	1.687	1.899	1.175	2.231	0.994	0.795	1.687	1.125	---
0.75	1.334	1.175	1.2375	1.454	1.747	1.2375	2.162	0.814	0.651	1.454	0.969	---
1.00	1.097	0.968	1.3	1.258	1.605	1.3	2.087	0.610	0.488	1.258	0.839	0.839
2.00	0.551	0.491	1.35	0.663	1.014	1.35	1.369	0.305	0.244	0.663	0.442	0.884
3.00	0.339	0.304	1.4	0.426	0.658	1.4	0.921	0.203	0.163	0.426	0.284	0.851
4.00	0.225	0.202	1.45	0.293	0.428	1.45	0.621	0.153	0.122	0.293	0.195	0.781
5.00	0.161	0.144	1.5	0.216	0.293	1.5	0.440	0.122	0.098	0.216	0.144	0.720

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

Site Class=	D	
F_a=	1.000	Short Period Site Coefficient
S_S=	1.492	Mapped MCE_R, 5% Damped at T=0.2s
S_1=	0.512	Mapped MCE_R, 5% Damped at T=1s
S_{DS}=	0.994	Design, 5% Damped at Short Periods
S_{MS}=	1.492	The MCE_R, 5% Damped at Short Periods
T_L (sec)=	8.0	Long Period Transition (Sect 11.4.6)
F_{PGA} (g)=	1.1	Site Coefficient for PGA
PGA_M (g)=	0.750	
F_v=	1.788	Used Only for Calculation of T_o and T_s
S_{M1}=	0.915	
S_{D1}=	0.610	Design, 5% Damped at T=1s
T_o (sec)=	0.123	Defined in ASCE 7-16 Sect 11.4.6
T_S (sec)=	0.614	Defined in ASCE 7-16 Sect 11.4.6

SITE-SPECIFIC DESIGN PARAMETERS

S_{DS}=	**1.013**	90% of max S_a (ASCE 7-16 Sect 21.4)
S_{MS}=	**1.519**	MCE_R, 5% Damped, adjusted for Site Class
S_{D1}=	**0.884**	Design, 5% Damped, at T=1s (Sect 11.4.5)
S_{M1}=	**1.326**	MCE_R, 5% Damped, at T=1s, adjusted for Site
F_a=	**1.000**	Short Period Site Coefficient
F_v=	**2.500**	Long Period Site Coefficient (7-16 Sect 21.3)
S_S=	**1.519**	MCE_R, 5% Damped at T=0.2s
S_1=	**0.530**	MCE_R, 5% Damped at T=1s
$PGA_{Probabilistic}$ (g)=	**0.693**	Peak Ground Acceleration, Probabilistic
$PGA_{Deterministic}$ (g)=	**0.743**	Peak Ground Acceleration, Deterministic
F_{PGA} (g)=	**1.1**	Site Coefficient for PGA
0.5*F_{PGA} (g)=	**0.550**	OK (Check $PGA_{Deterministic}$ > 0.5 x F_{PGA})
0.8*PGA_M (g)=	**0.600**	PGA_M (g) (Determined from ASCE 7-16 Eq. 11.8-1)
Site Specific PGA_M (g) =	**0.693**	(Check $PGA_{Site\ Specific}$ > 0.8 x PGA_M)

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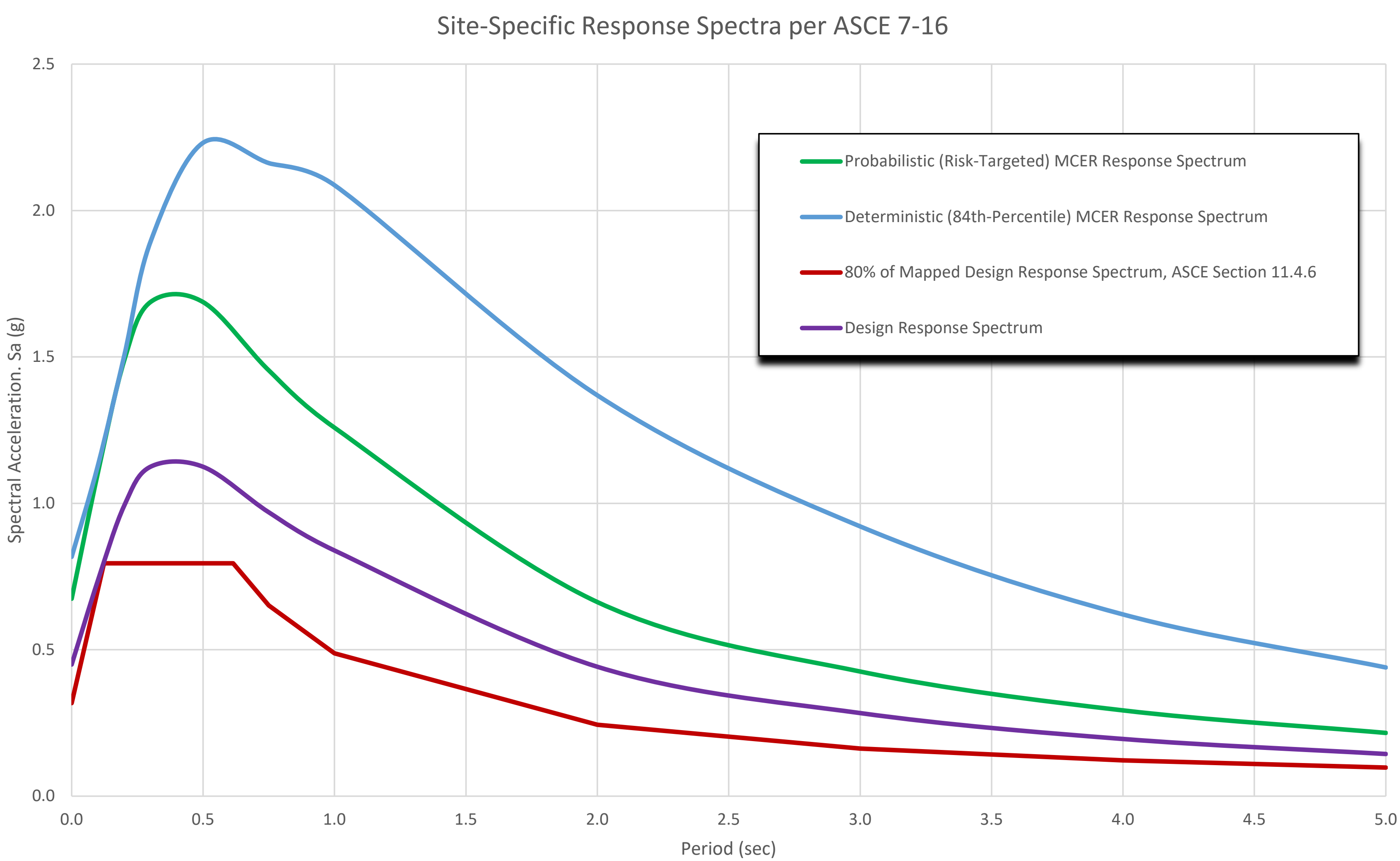
Viewpoint Old Town San Diego, California			
By:	GLC	Date:	July 2022
Job Number:	2021073	Figure:	D.2



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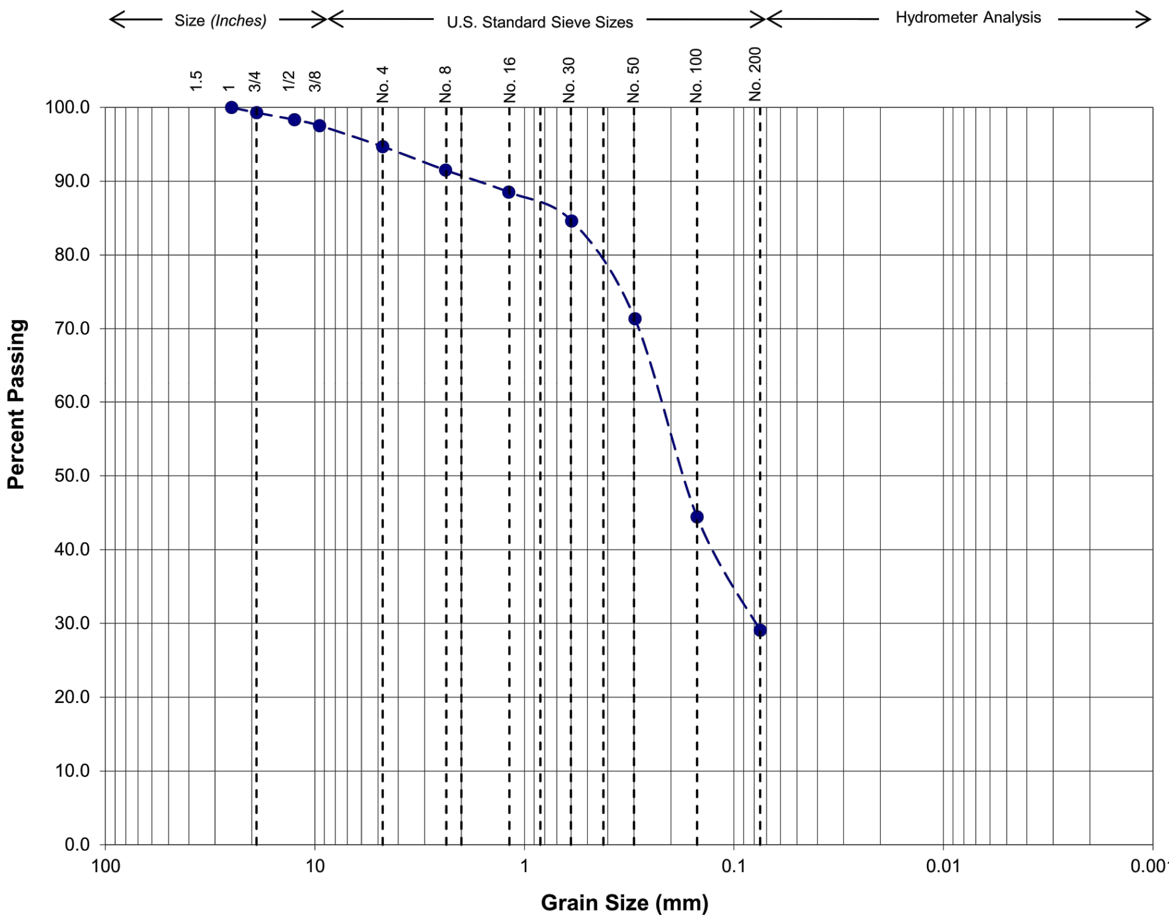
Viewpoint Old Town San Diego, California			
By:	GLC	Date:	July 2022
Job Number:	2021073	Figure:	D.3

APPENDIX E
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.*
- **MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT (ASTM D 1557 METHOD A,B,C):** *The maximum dry density and optimum moisture content of typical soils were determined in the laboratory in accordance with ASTM Standard Test D 1557, Method A, Method B, Method C.*
- **IN-PLACE MOISTURE AND DENSITY OF SOIL (ASTM D3550):** *In-place moisture contents and dry densities were determined for representative soil samples. This information was an aid to classification and permitted recognition of variations in material consistency with depth. The dry unit weight is determined in pounds per cubic foot, and the in-place moisture content is determined as a percentage of the soil's dry weight. The results are summarized in the exploration logs presented in Appendix B.*
- **GRADATION ANALYSIS (ASTM D6913):** *Tests were performed on selected representative soil samples in general accordance with ASTM D422. The grain size distributions of selected samples were determined in accordance with ASTM D6913.*
- **R-VALUE (CT 301 and ASTM D 2844):** *The resistance Value, or R-Value, for near-surface site soils were evaluated in general accordance with California Test (CT) 301 and ASTM D 2844. The sample was 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 content of the selected samples were evaluated in general accordance with CT 417 and CT 422, respectively.*

Soil samples not tested are now stored in our laboratory for future reference and evaluation, if needed. Unless notified to the contrary, samples will be disposed of 90 days from the date of this report.



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 1

Depth (ft): 0 - 8

USCS Soil Type: ML

Passing No. 200 (%): 29

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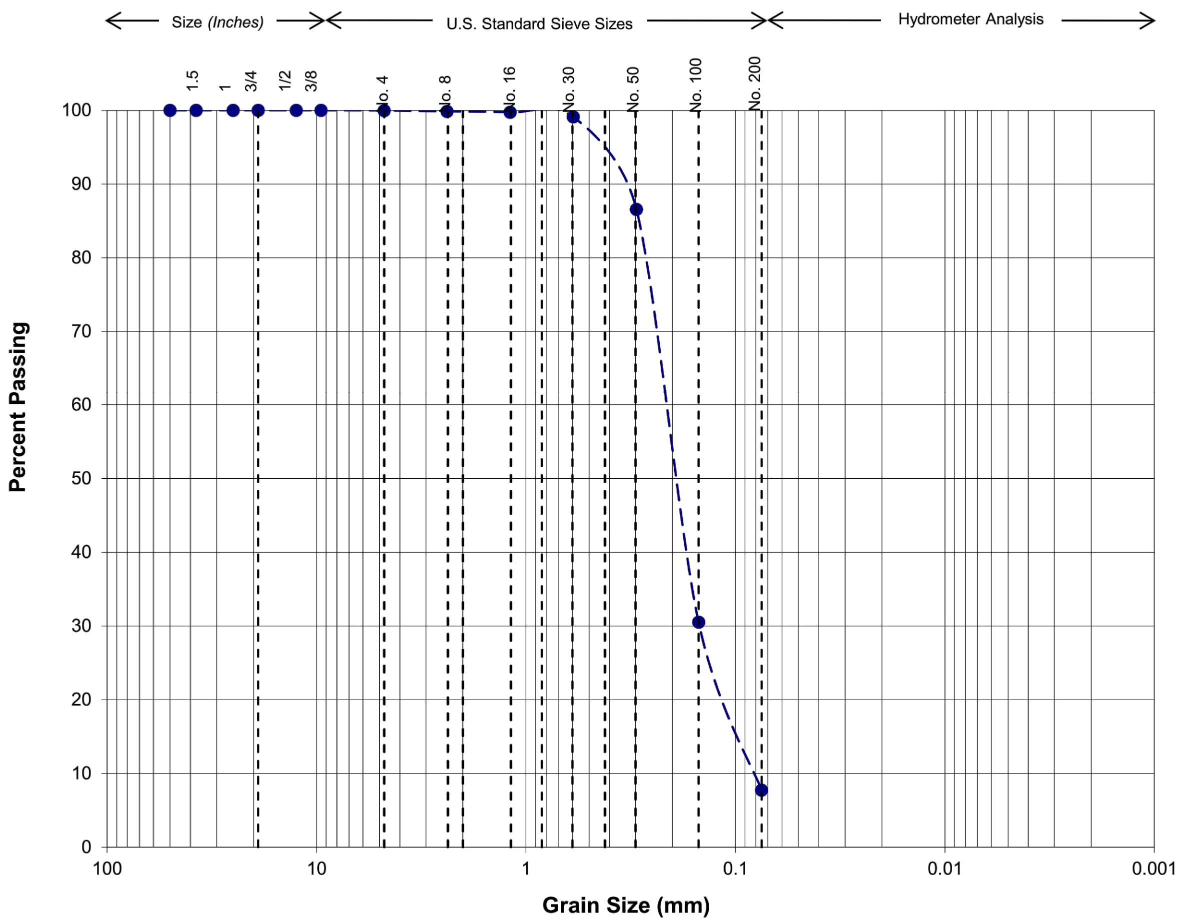
GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN

4620 PACIFIC HIGHWAY

SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.2



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 1

Depth (ft): 10

USCS Soil Type: SP-SM

Passing No. 200 (%): 8

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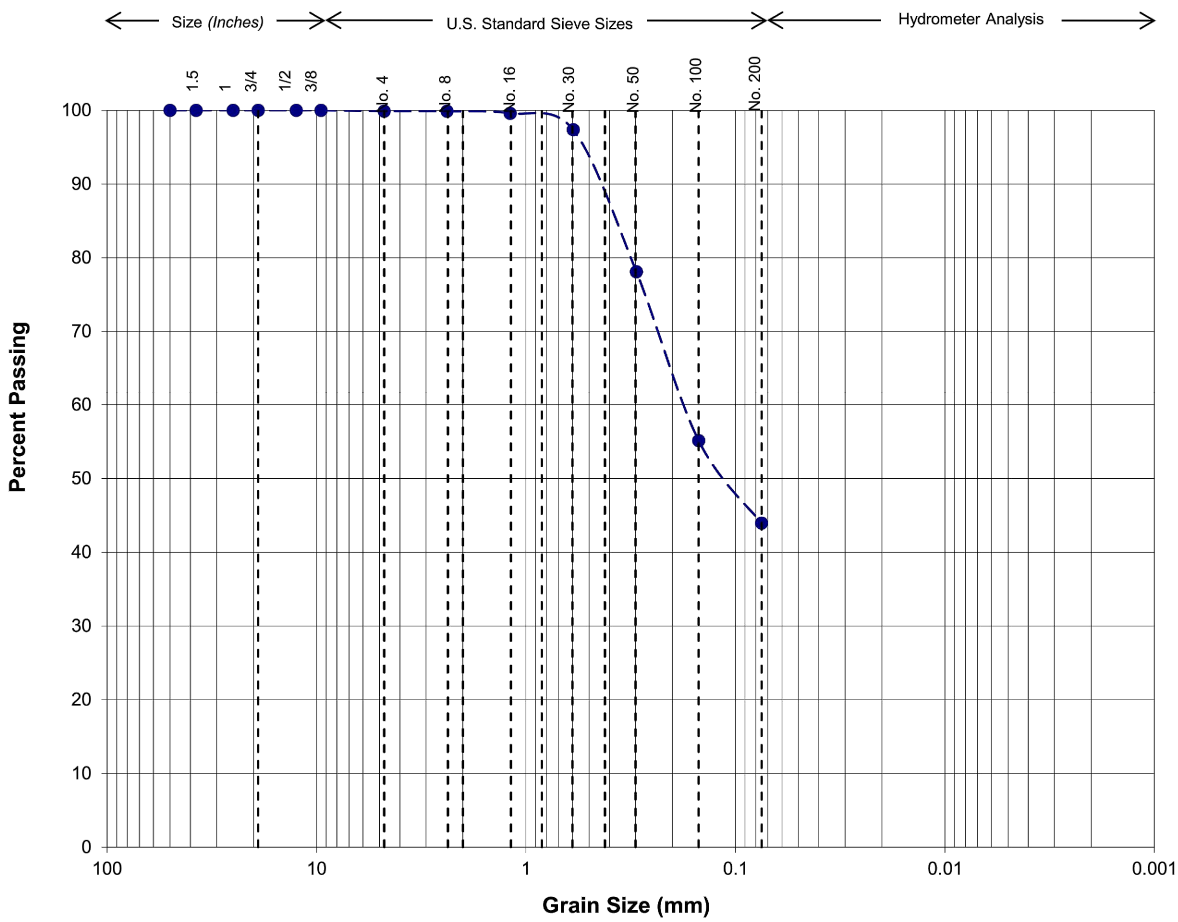
GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN

4620 PACIFIC HIGHWAY

SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.3



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 1

Depth (ft): 15

USCS Soil Type: SM/SC

Passing No. 200 (%): 44

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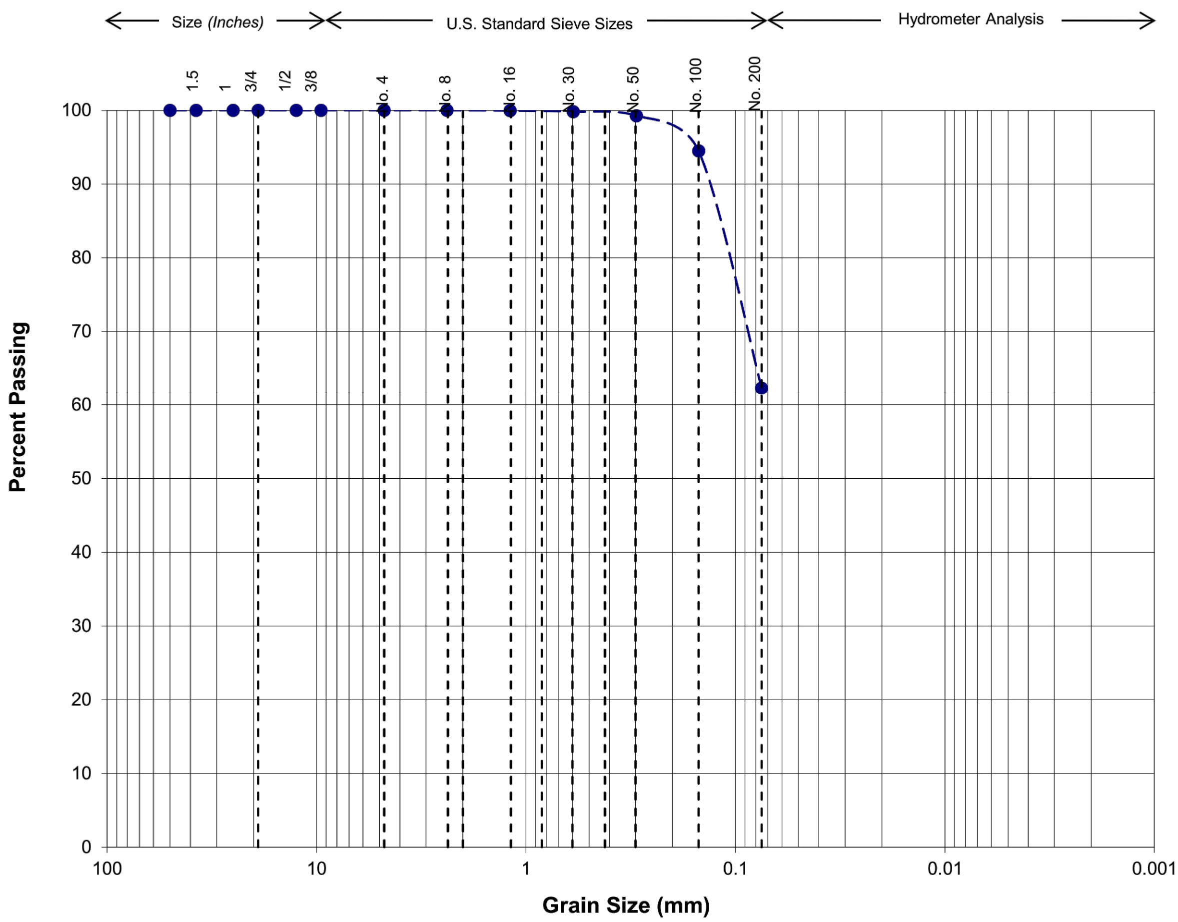
GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN

4620 PACIFIC HIGHWAY

SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.4



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 1

Depth (ft): 20

USCS Soil Type: CL/ML

Passing No. 200 (%): 62

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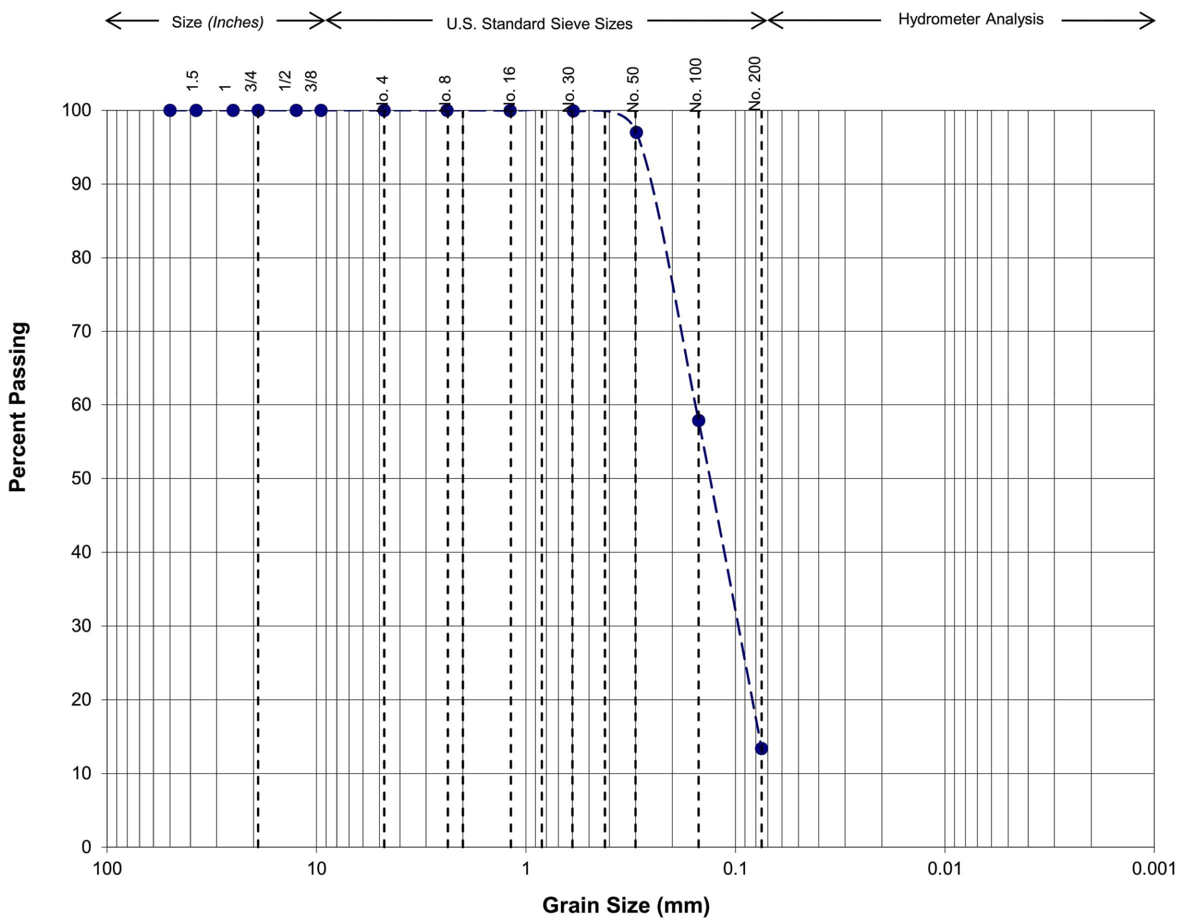
GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN
4620 PACIFIC HIGHWAY
SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.5



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 1

Depth (ft): 25

USCS Soil Type: SP-SM

Passing No. 200 (%): 13

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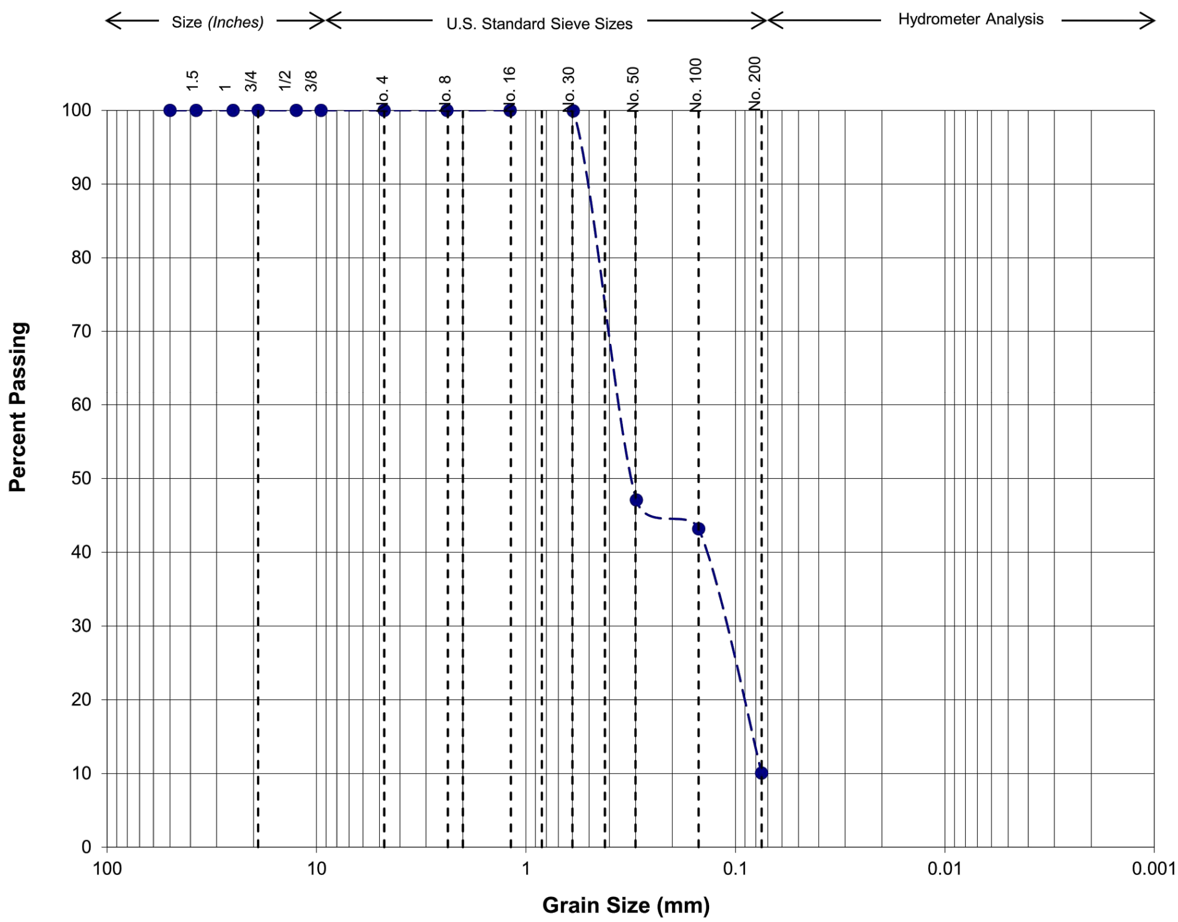
GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN

4620 PACIFIC HIGHWAY

SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.6



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<tr><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Sample Location:	B - 1
Depth (ft):	30
USCS Soil Type:	SP-SM
Passing No. 200 (%):	10

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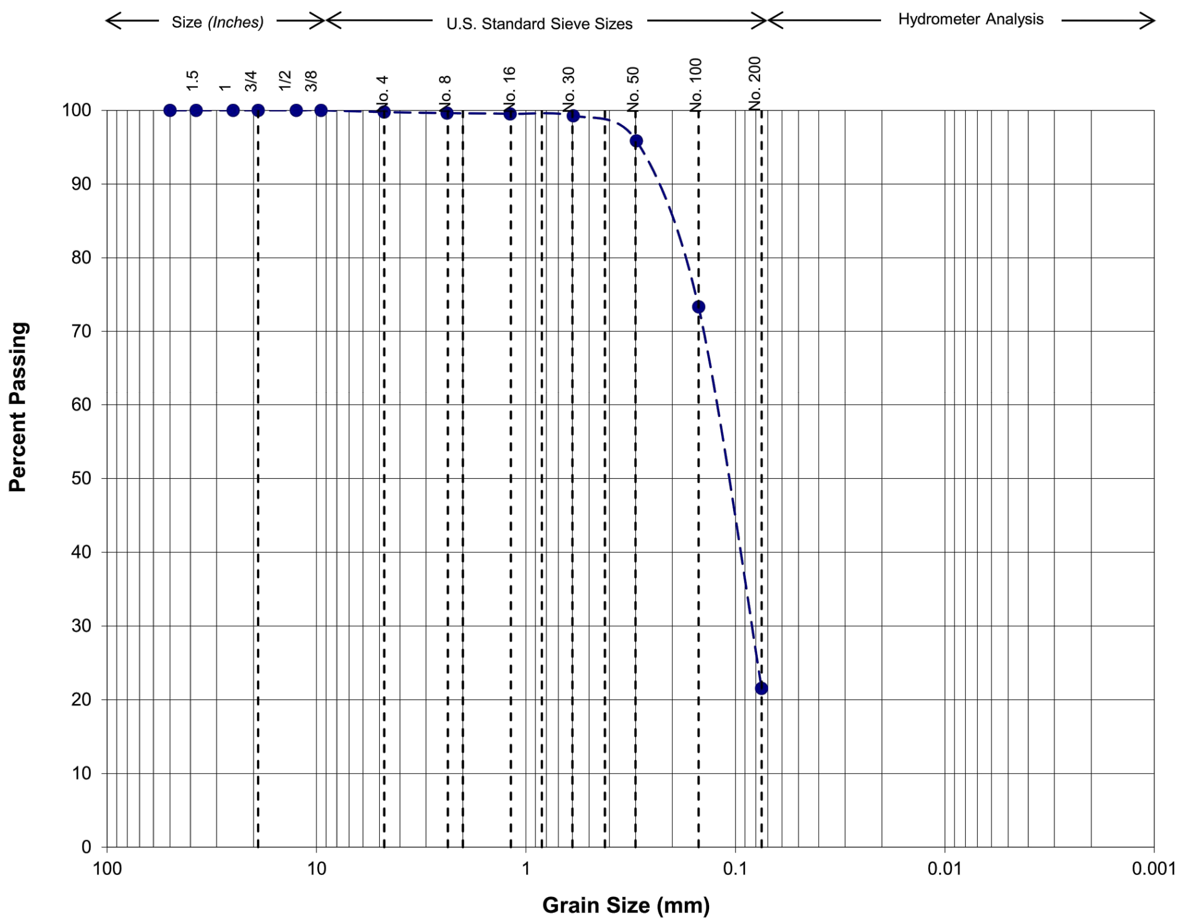
GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN
4620 PACIFIC HIGHWAY
SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.7



<table>
<tr><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Sample Location:	B - 1
Depth (ft):	35
USCS Soil Type:	SM
Passing No. 200 (%):	22

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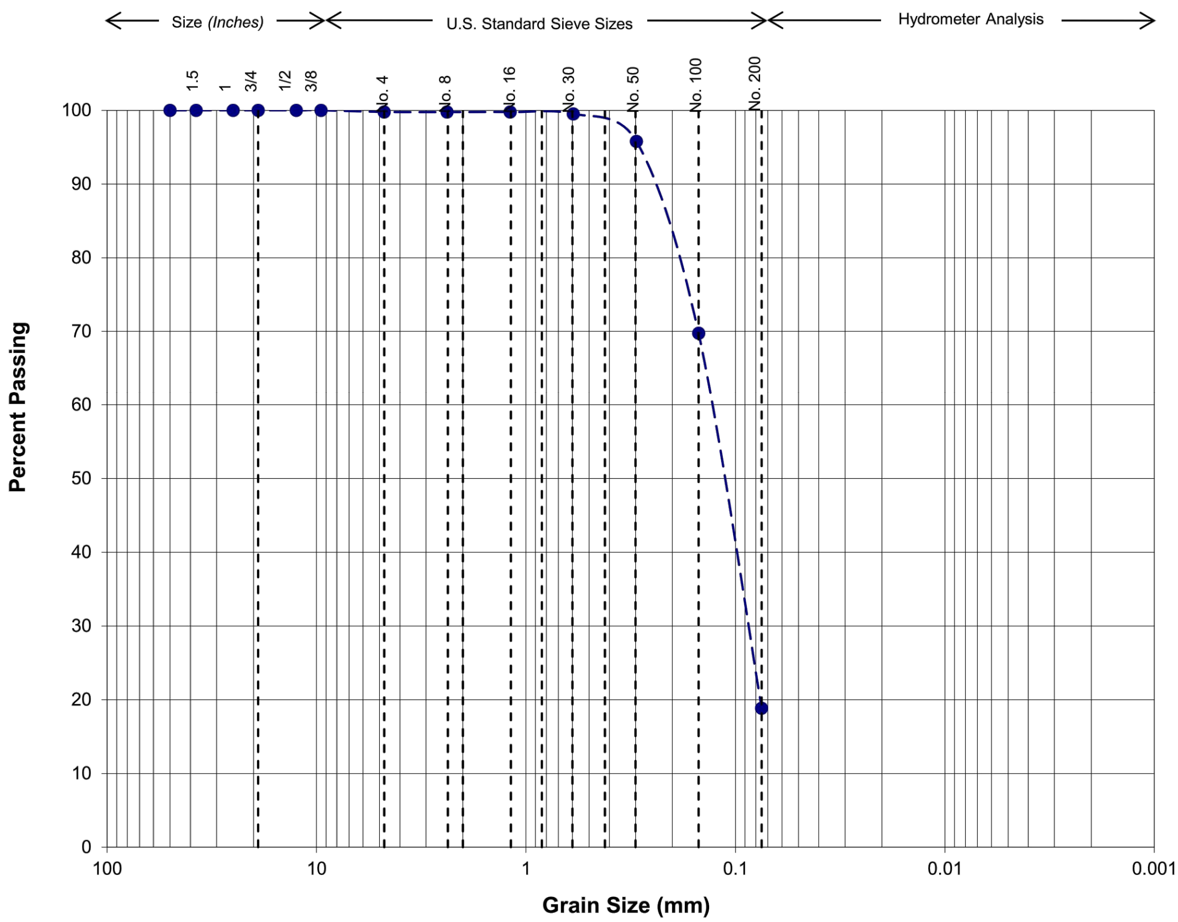
GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN
4620 PACIFIC HIGHWAY
SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.8



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 1

Depth (ft): 40

USCS Soil Type: SM

Passing No. 200 (%): 19

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GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN

4620 PACIFIC HIGHWAY

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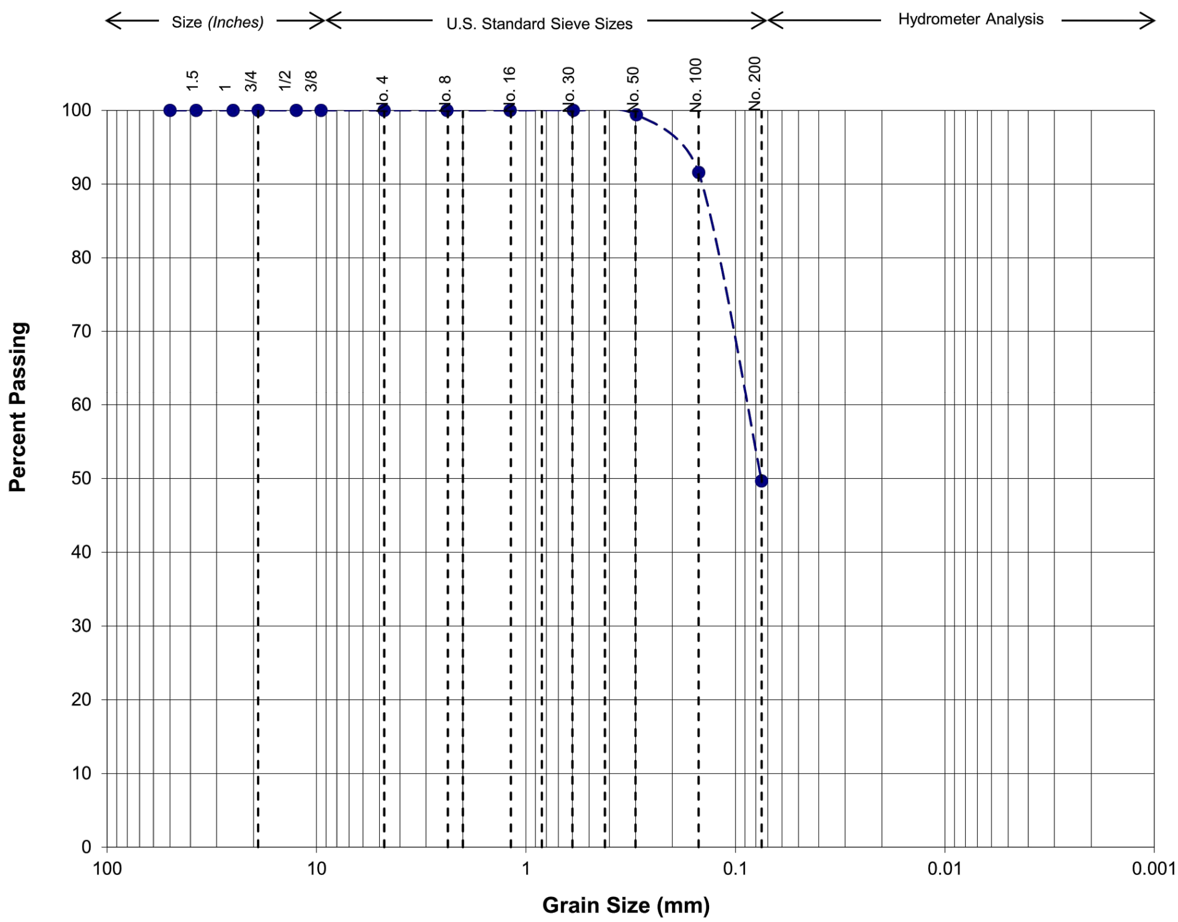
BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.9



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 1

Depth (ft): 50

USCS Soil Type: SM-ML

Passing No. 200 (%): 50

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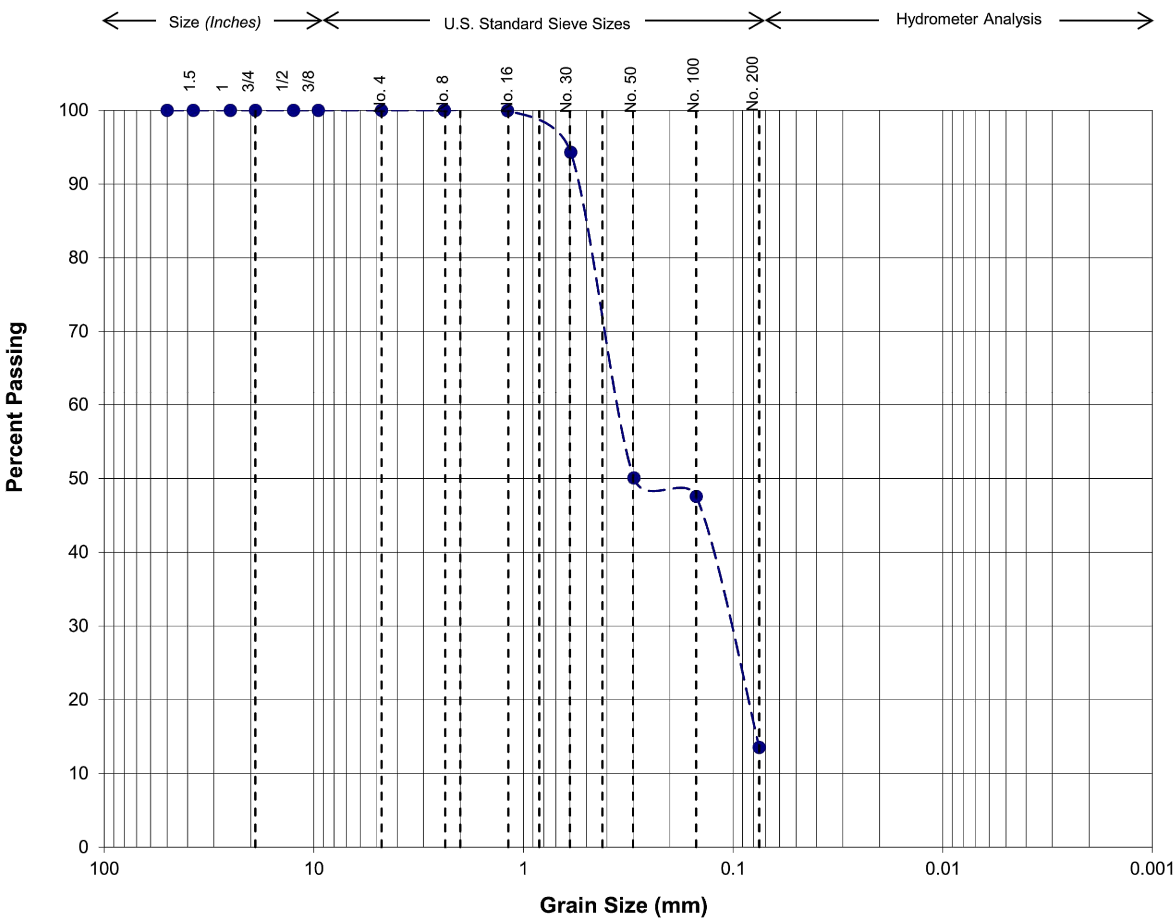
GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN
4620 PACIFIC HIGHWAY
SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.10



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 1

Depth (ft): 60

USCS Soil Type: SP-SM

Passing No. 200 (%): 14

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GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN

4620 PACIFIC HIGHWAY

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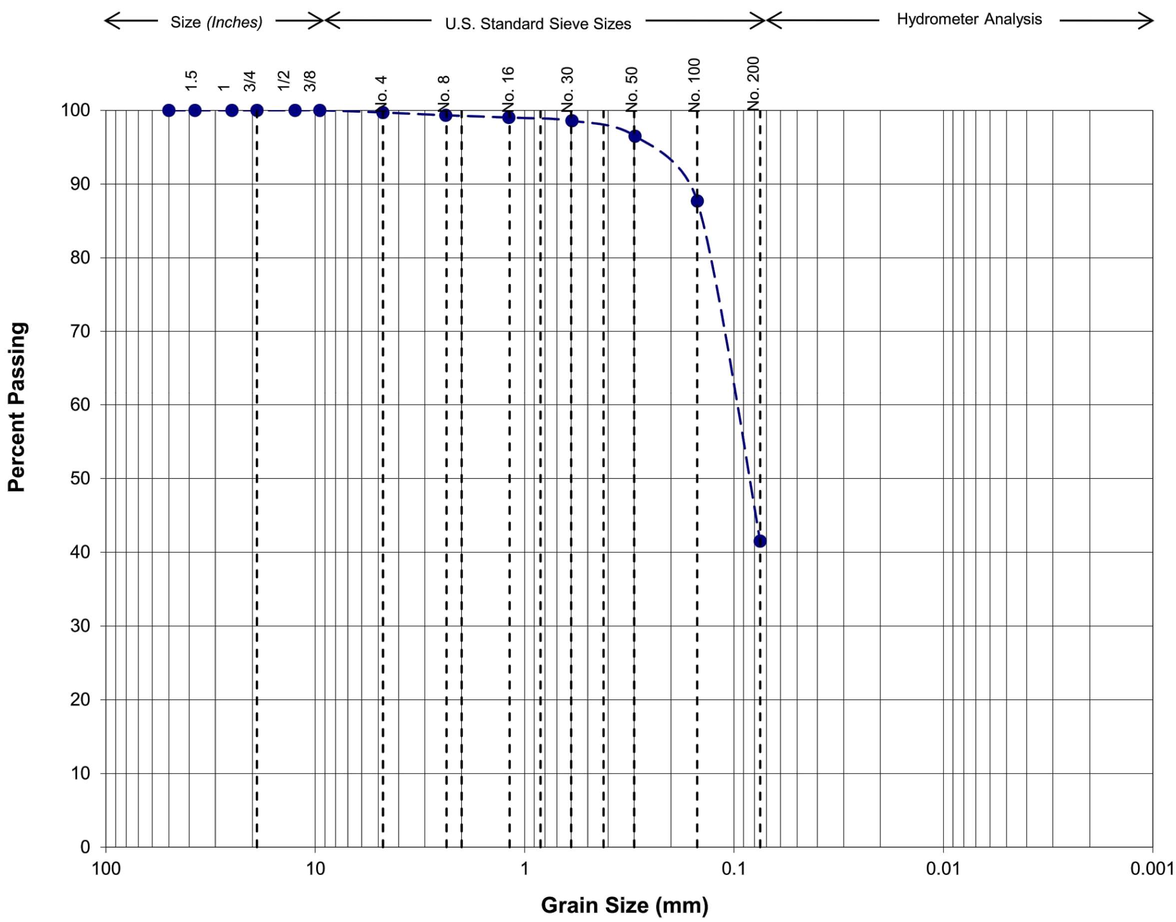
BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.11



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B - 1

Depth (ft): 70

USCS Soil Type: SM/SC

Passing No. 200 (%): 42

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GRADATION ANALYSIS TEST RESULTS

VIEWPOINT OLD TOWN

4620 PACIFIC HIGHWAY

SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.12

Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

Sample Location	Sample Depth (ft.)	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B - 1	2 - 5	133.0	10.1

R-Value (Cal. Test Method 301 & ASTM D2844)

Sample Location	Sample Depth (ft.)	R-Value
B - 1	0 - 8	60

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

Sample Location	Sample Depth (ft.)	pH	Resistivity (Ohm-cm)	Sulfate Content (ppm)	Sulfate Content (%)	Chloride Content (ppm)	Chloride Content (%)
B - 1	1 - 6	8.6	2400	84	0.008	53	0.005
B - 2	13½ - 15	8.2	1400	90	0.009	170	0.017

Water-Soluble Sulfate Exposure (ACI 318 Table 19.3.1.1 and Table 19.3.2.1)

Water-Soluble Sulfate (SO_4) in Soil (% by Weight)	Exposure Severity	Exposure Class	Cement Type (ASTM C150)	Max. W/C	Min. f_c' (psi)
$SO_4 < 0.10$	N/A	S0	No type restriction	N/A	2,500
$0.10 \leq SO_4 < 0.20$	Moderate	S1	II	0.50	4,000
$0.20 \leq SO_4 \leq 0.20$	Severe	S2	V	0.45	4,500
$SO_4 > 2.00$	Very Severe	S3	V plus pozzolan or slag cement	0.45	4,500

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LAB TEST RESULTS

VIEWPOINT OLD TOWN
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SAN DIEGO, CALIFORNIA

BY: GN	REVIEWED BY: MS	PROJECT: 2021073	FIGURE: E.13

APPENDIX F
LIQUEFACTION ANALYSES

T A B L E   O F   C O N T E N T S

NOVA Services, Inc.
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San Diego, CA 92123

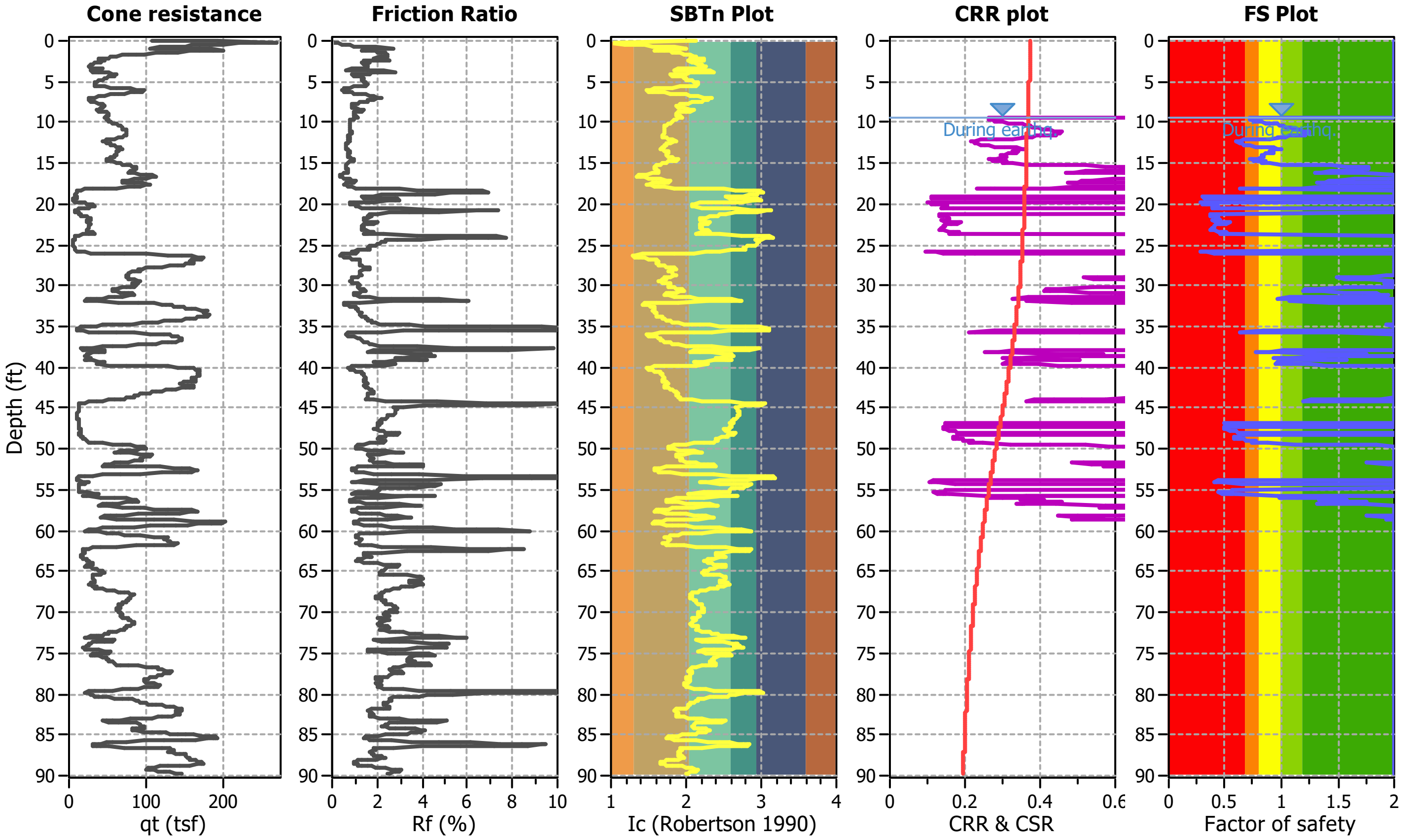
LIQUEFACTION ANALYSIS REPORT

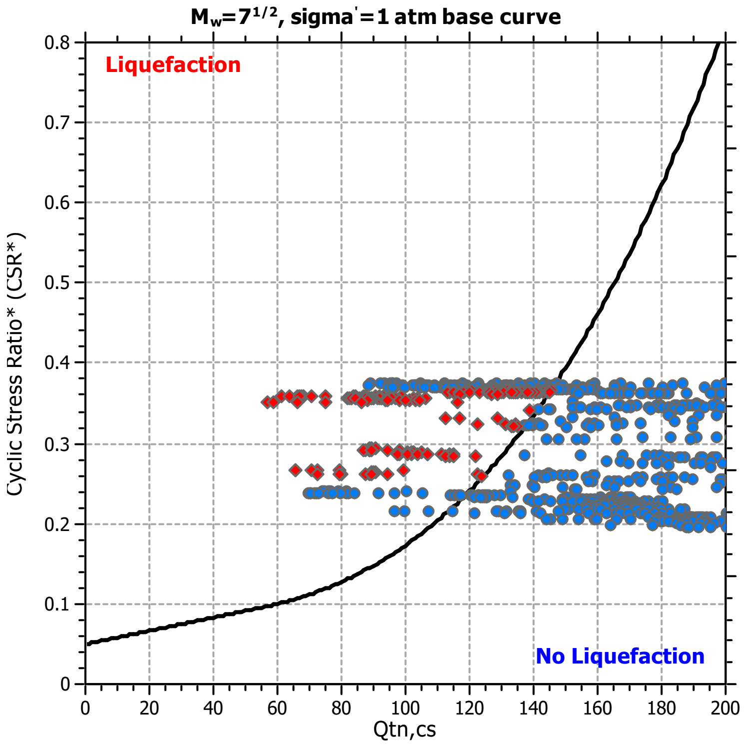
Project title : Old Town Viewpoint **Location : San Diego, California**

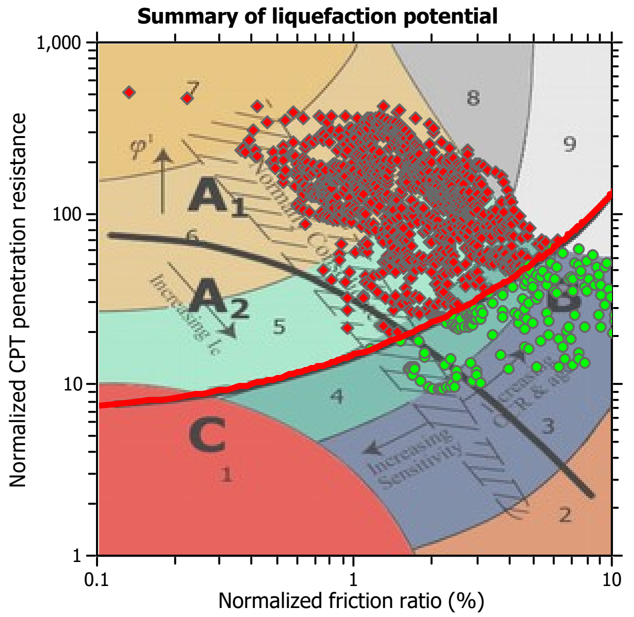
CPT file : CPT - 1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	9.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	9.50 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:	6.99	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.69	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

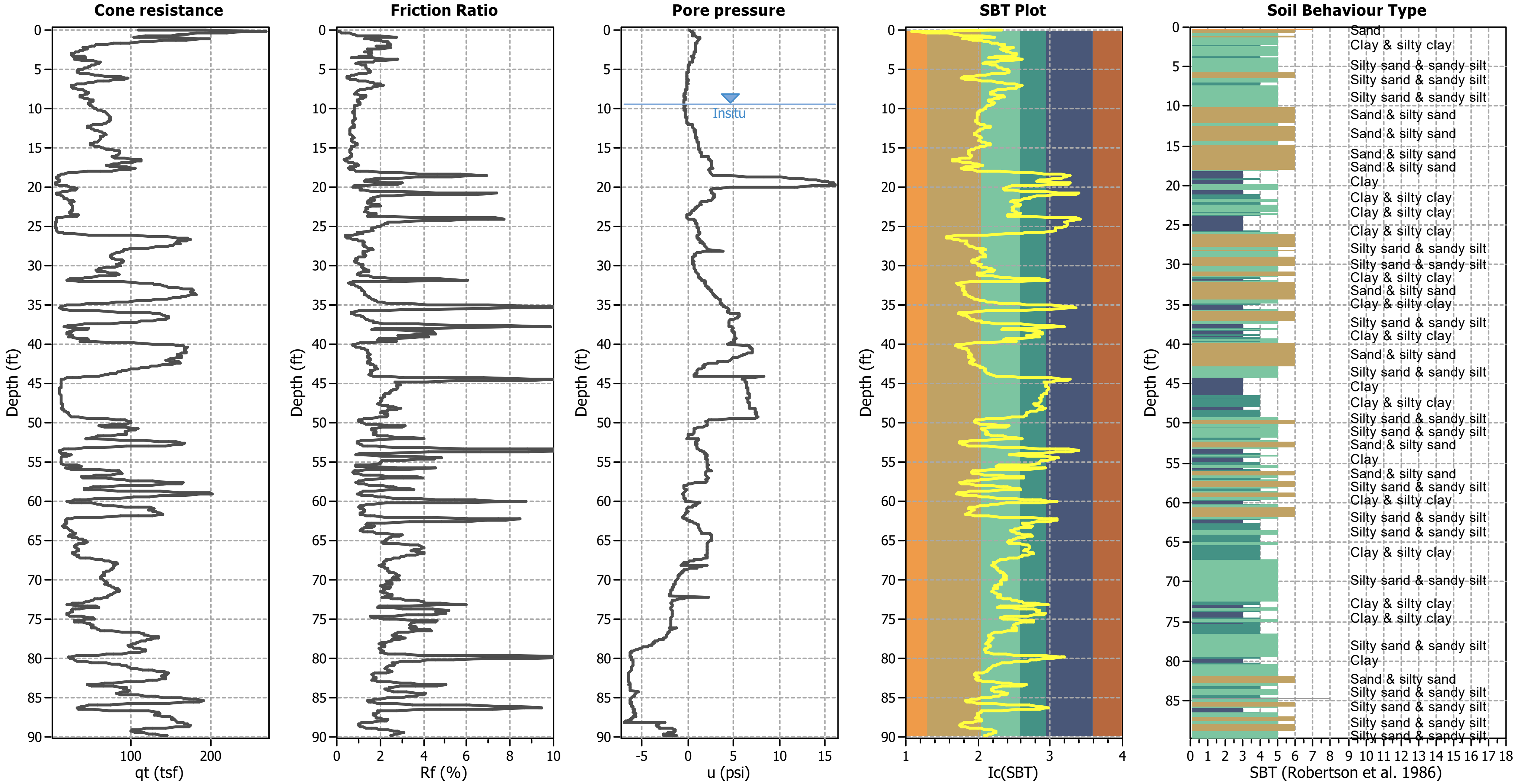






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



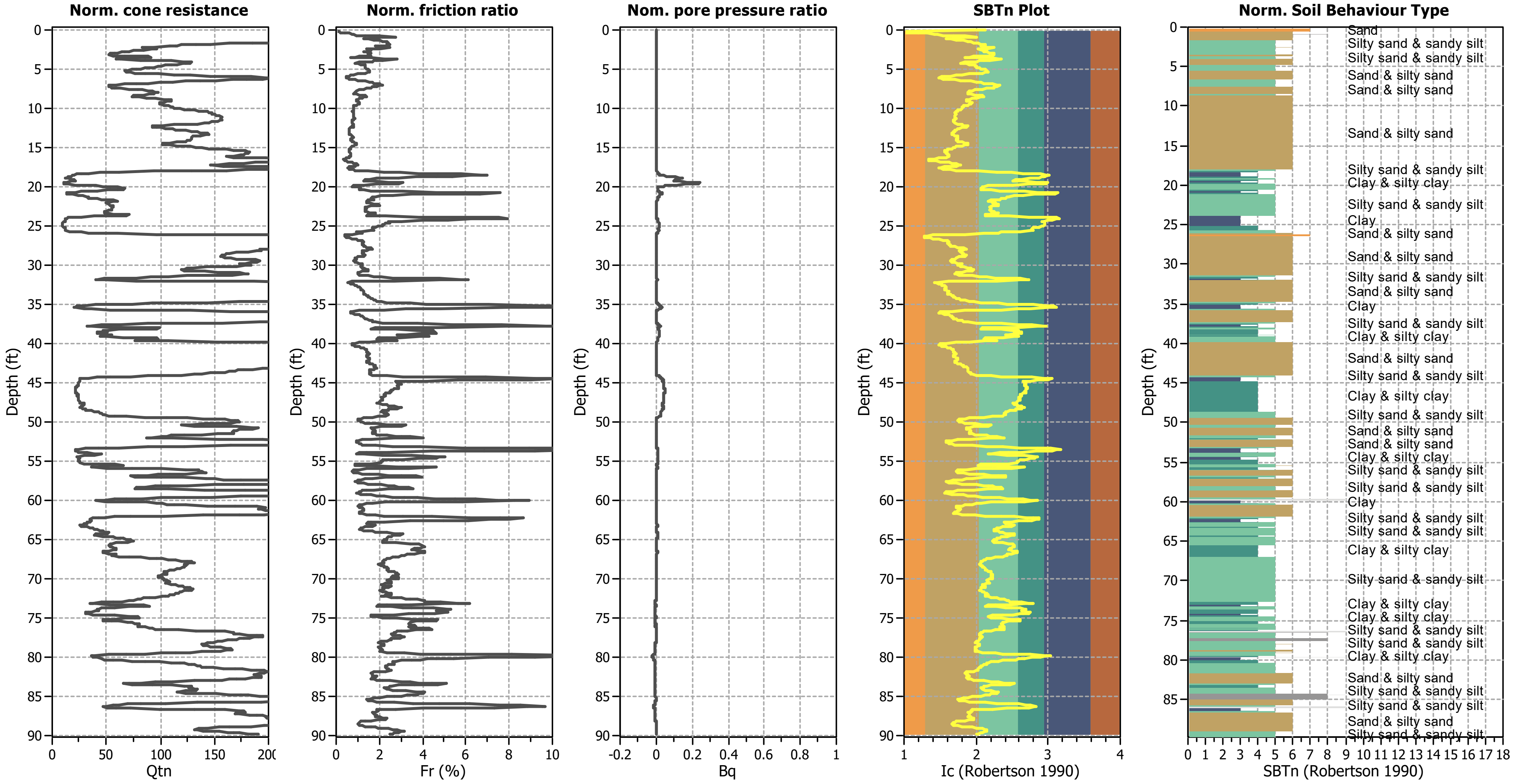
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 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)



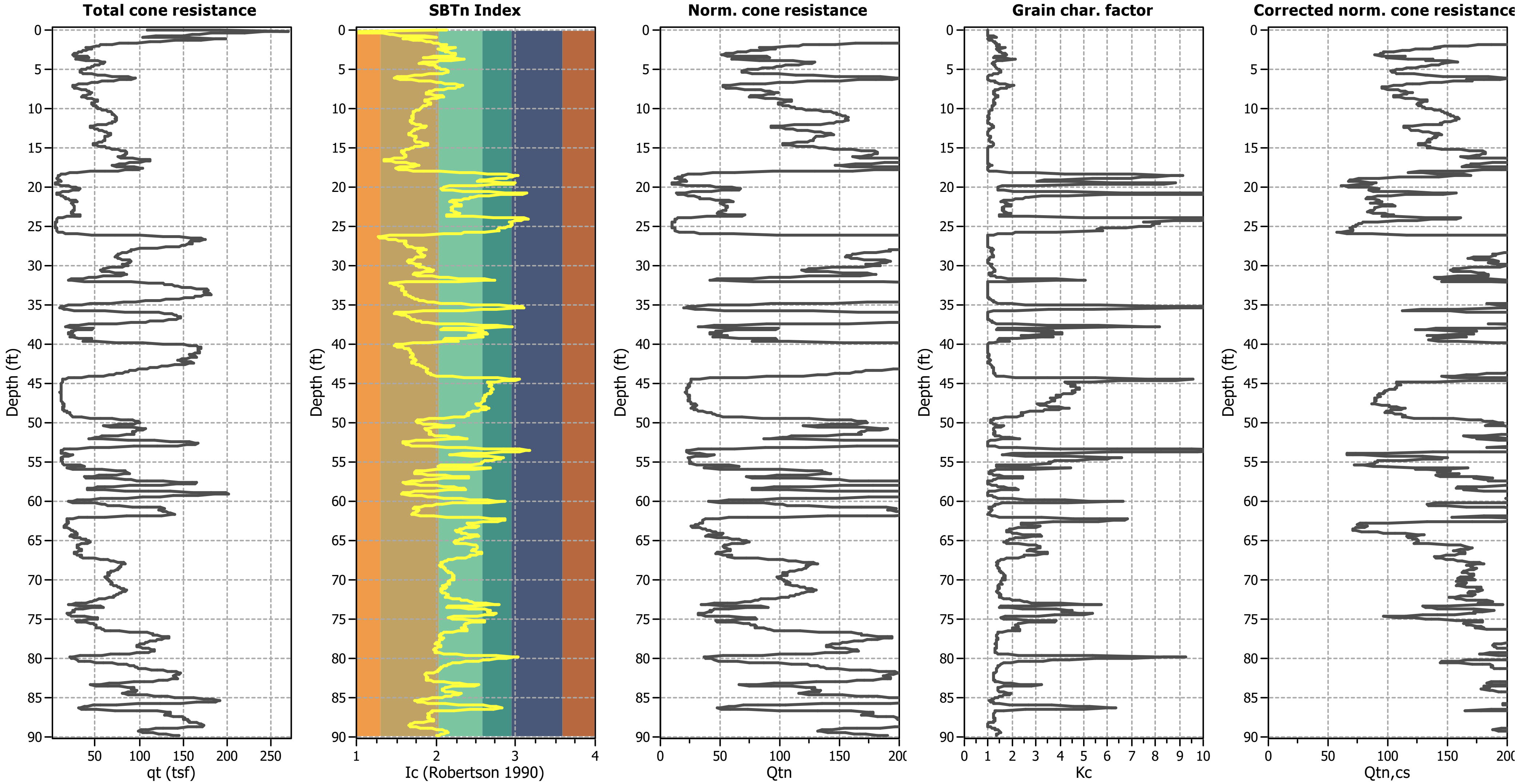
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 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

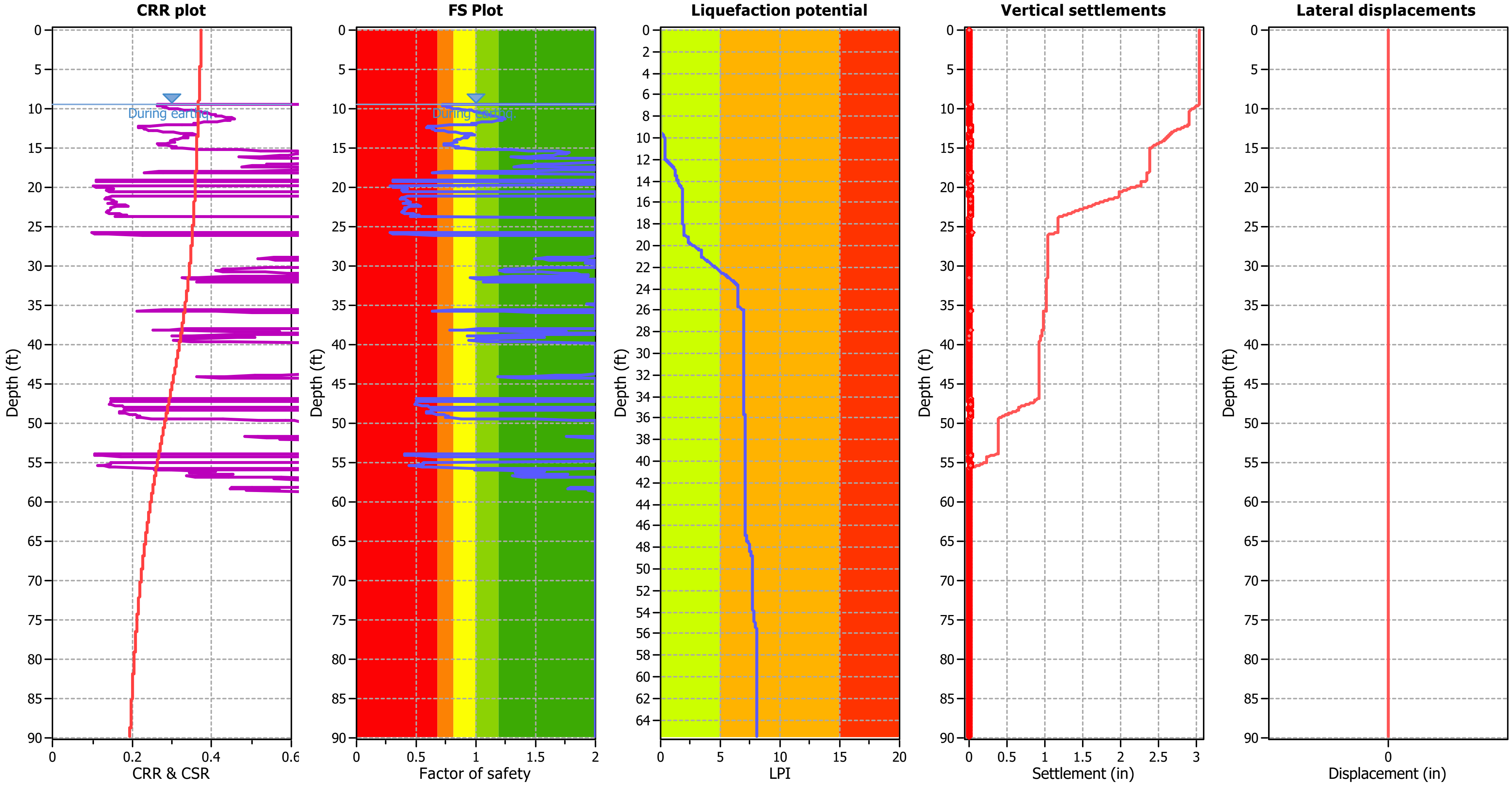
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 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.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

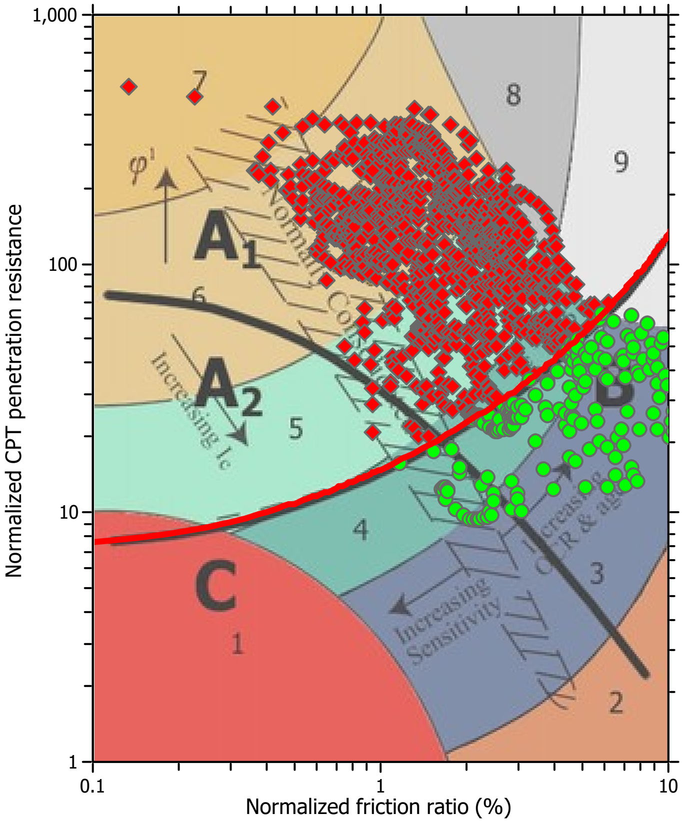
F.S. color scheme

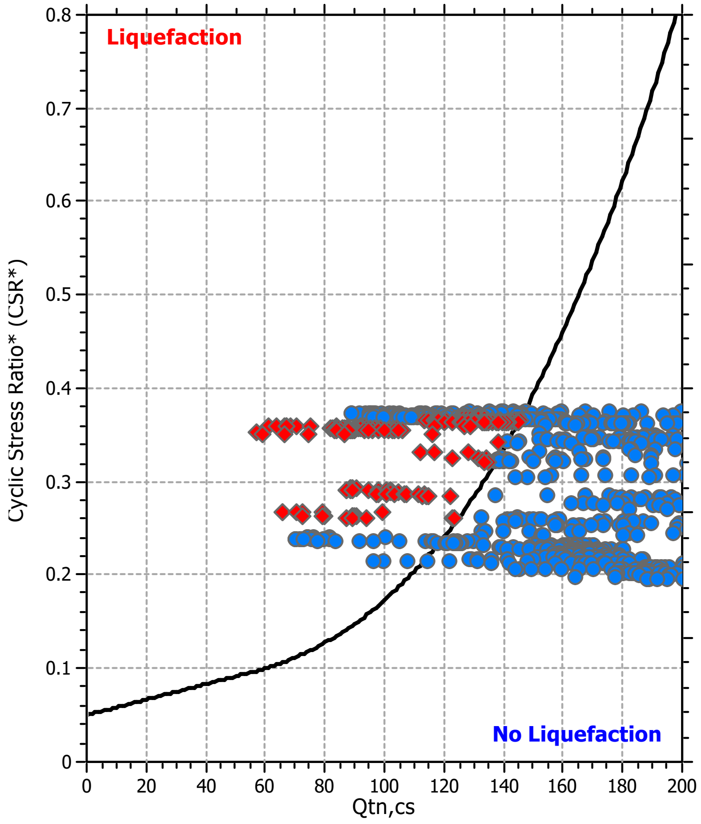
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

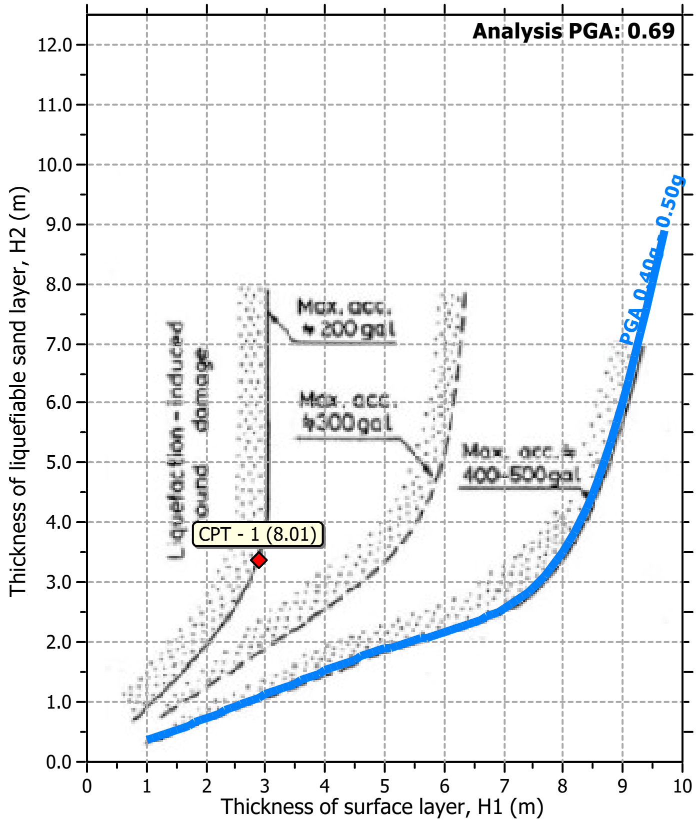
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



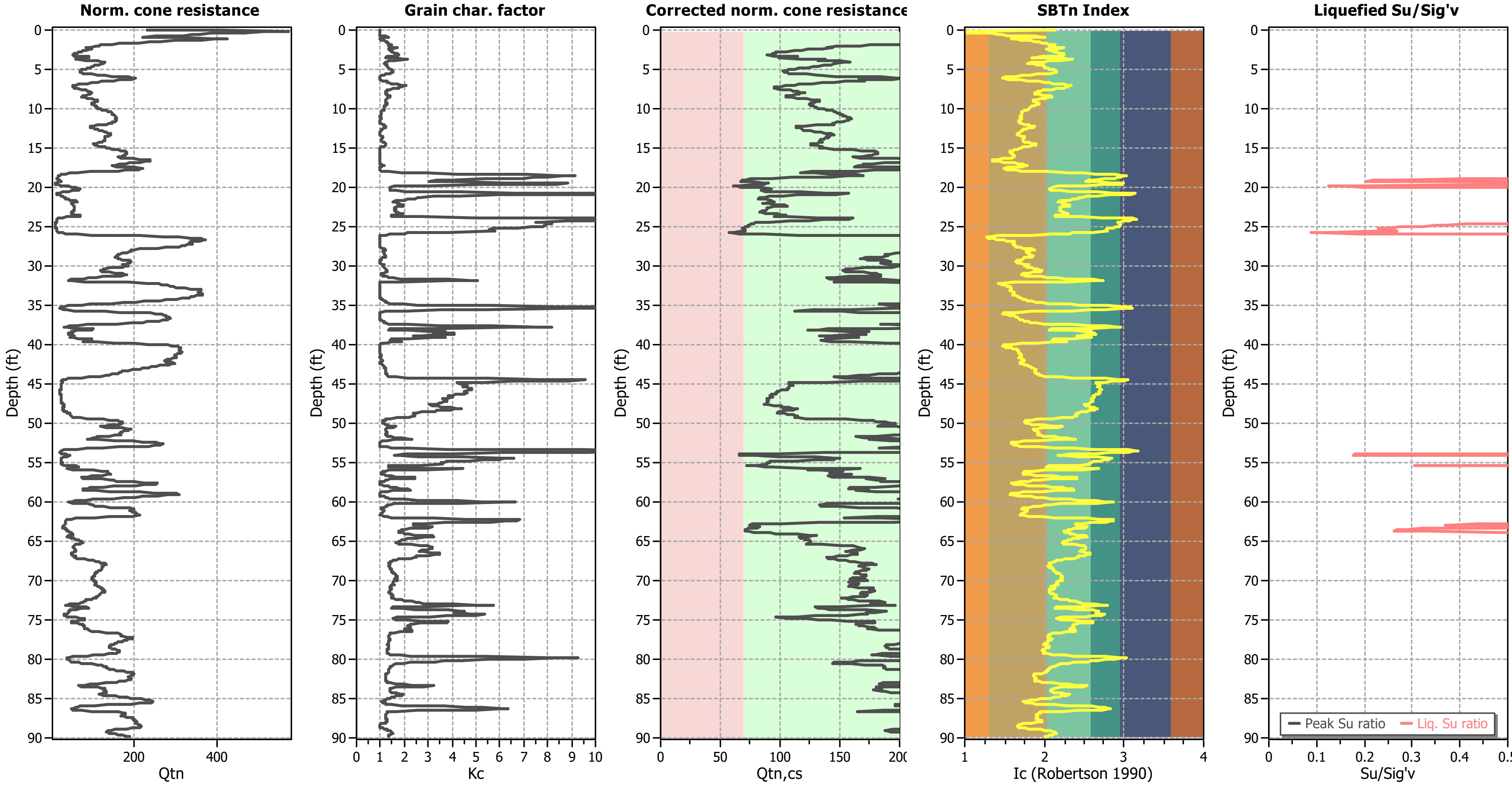




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

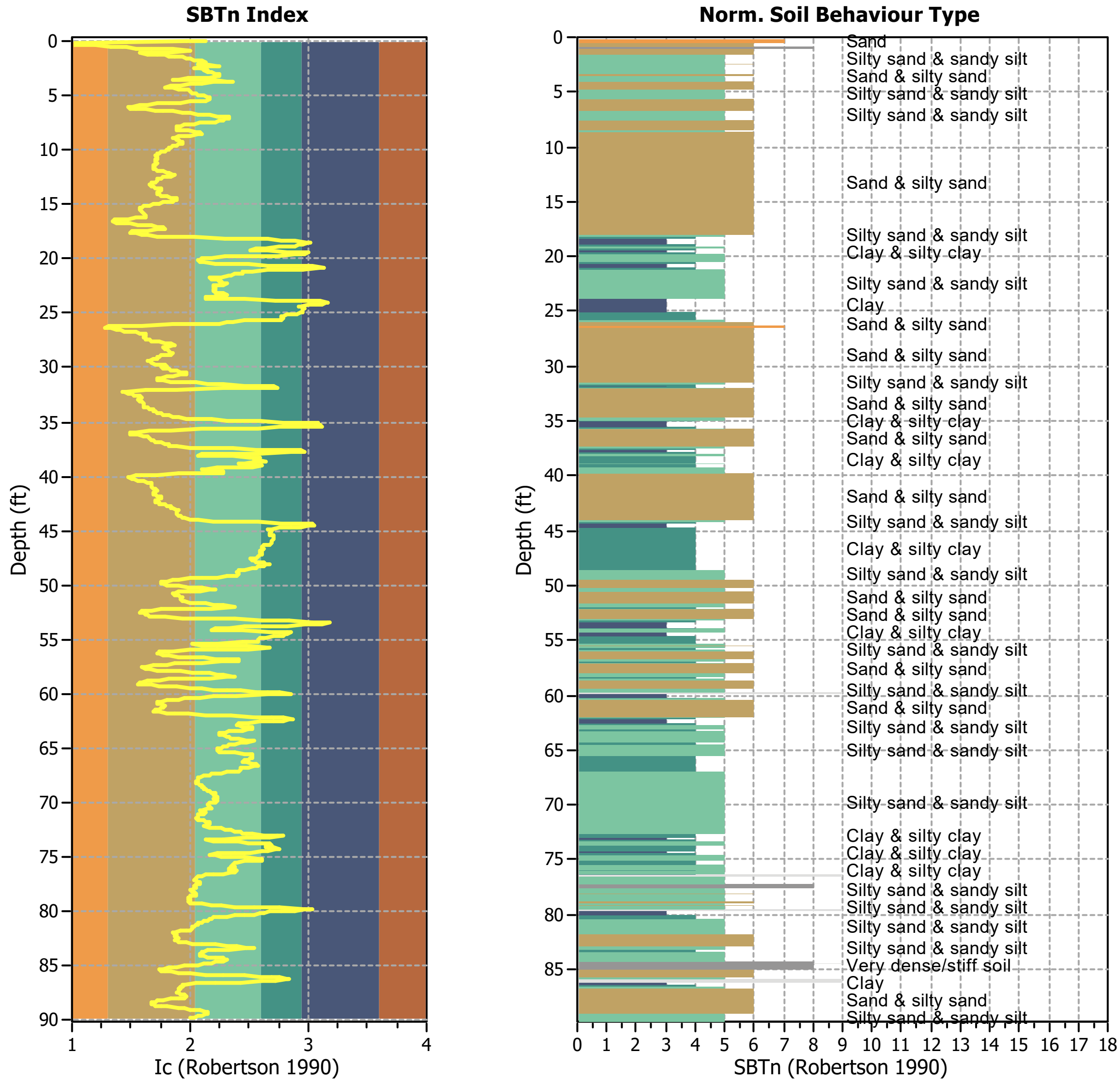
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

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

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



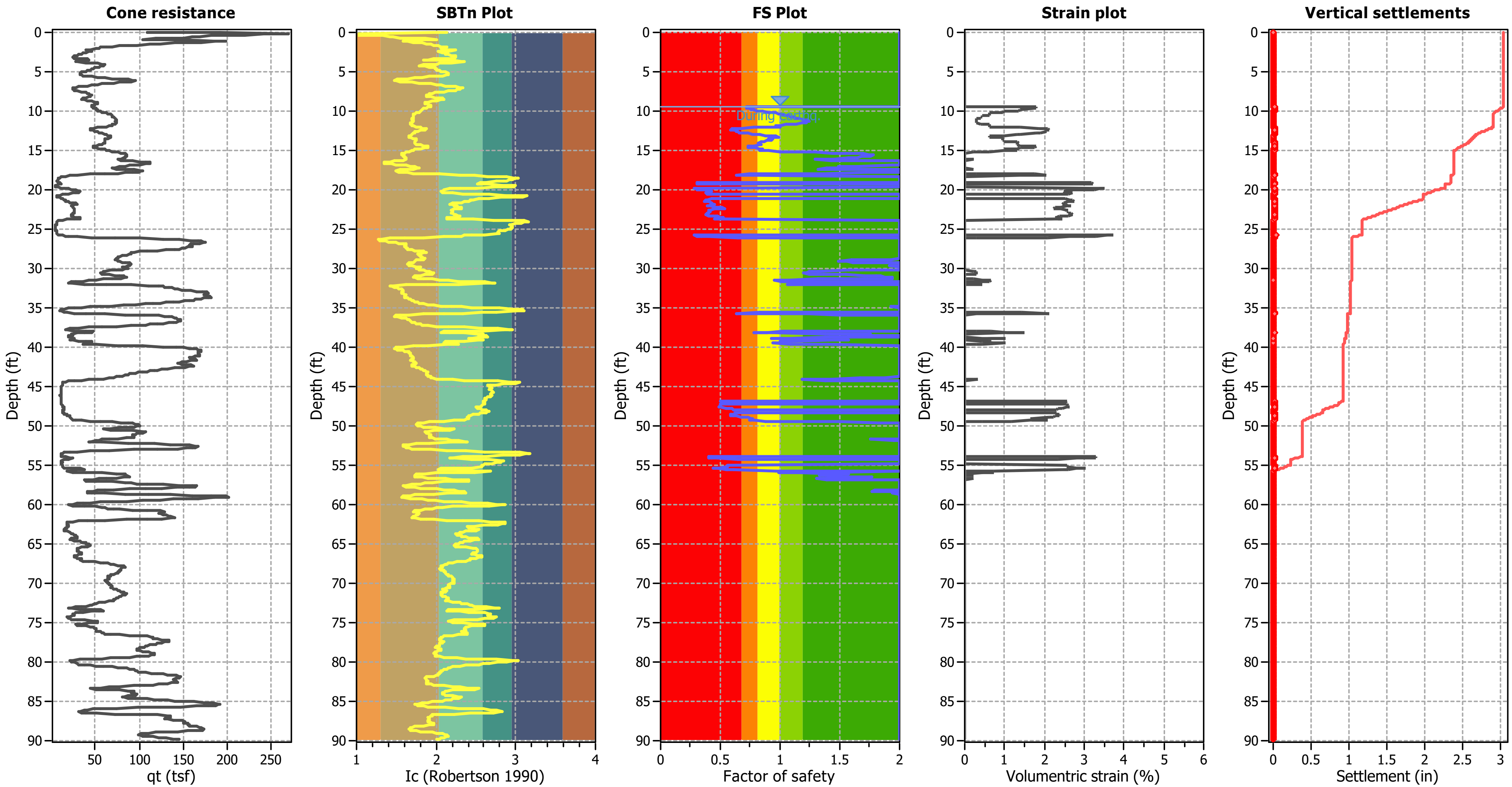
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	1.70
I_c change ratio value:	3.0000
Minimum number of points in layer:	-26215

General statistics

Total points in CPT file:	1369
Total points excluded:	0
Exclusion percentage:	0.00%
Number of layers detected:	0

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

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

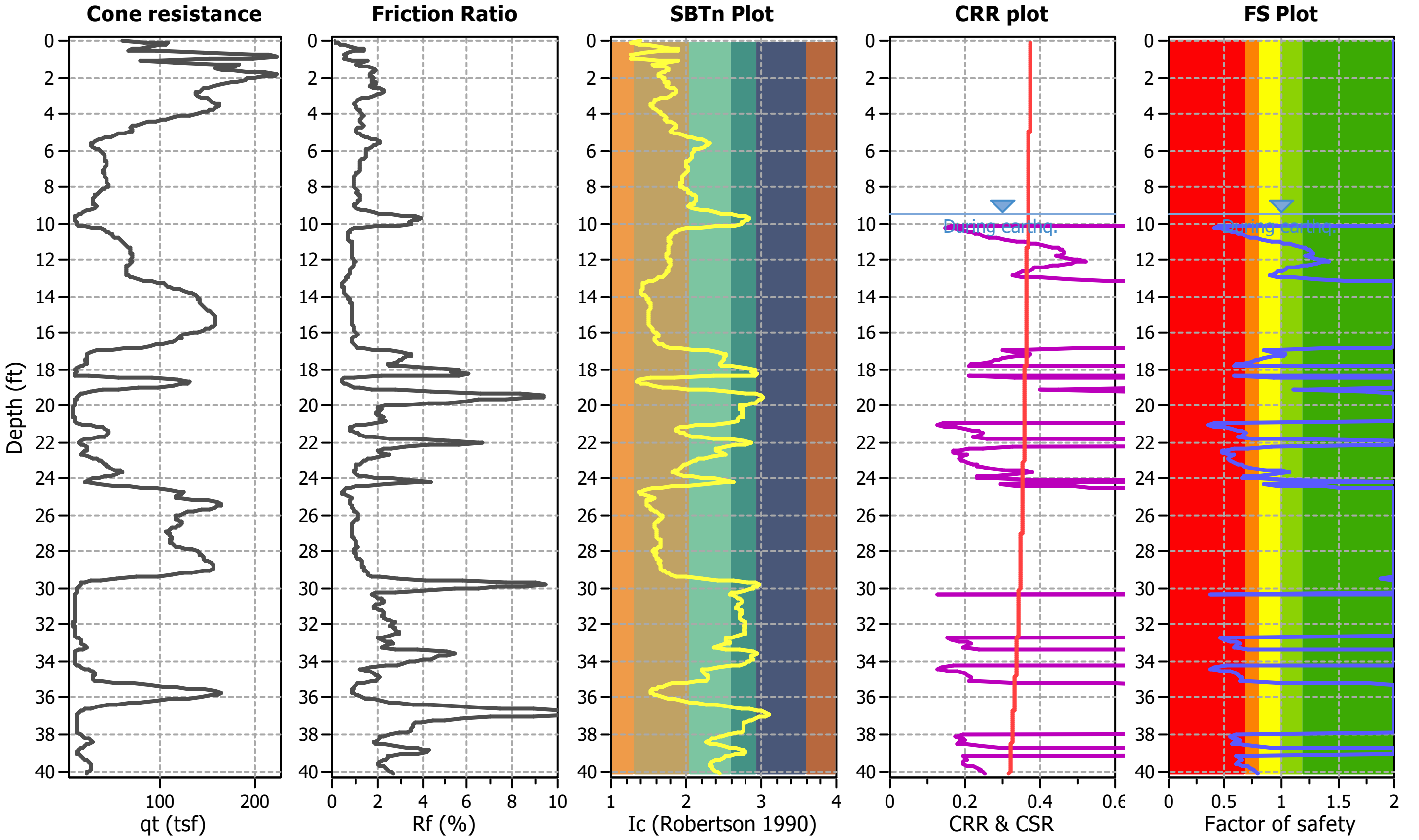
LIQUEFACTION ANALYSIS REPORT

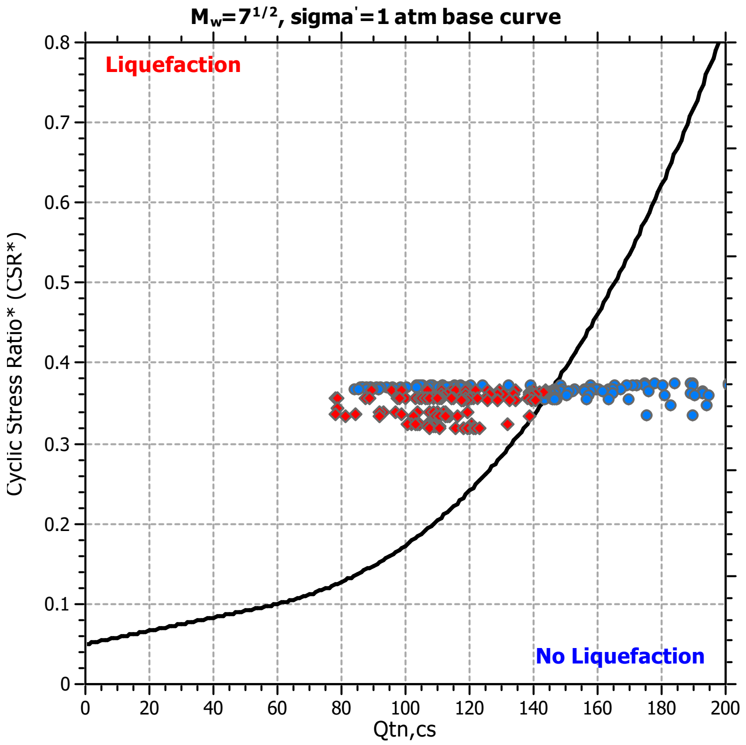
Project title : Old Town Viewpoint **Location : San Diego, California**

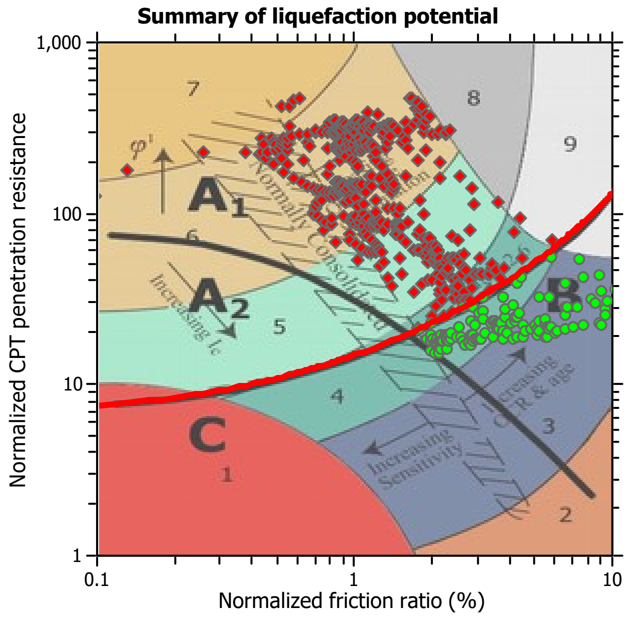
CPT file : CPT - 2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	9.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	9.50 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	6.99	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	60.00 ft
Peak ground acceleration:	0.69	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

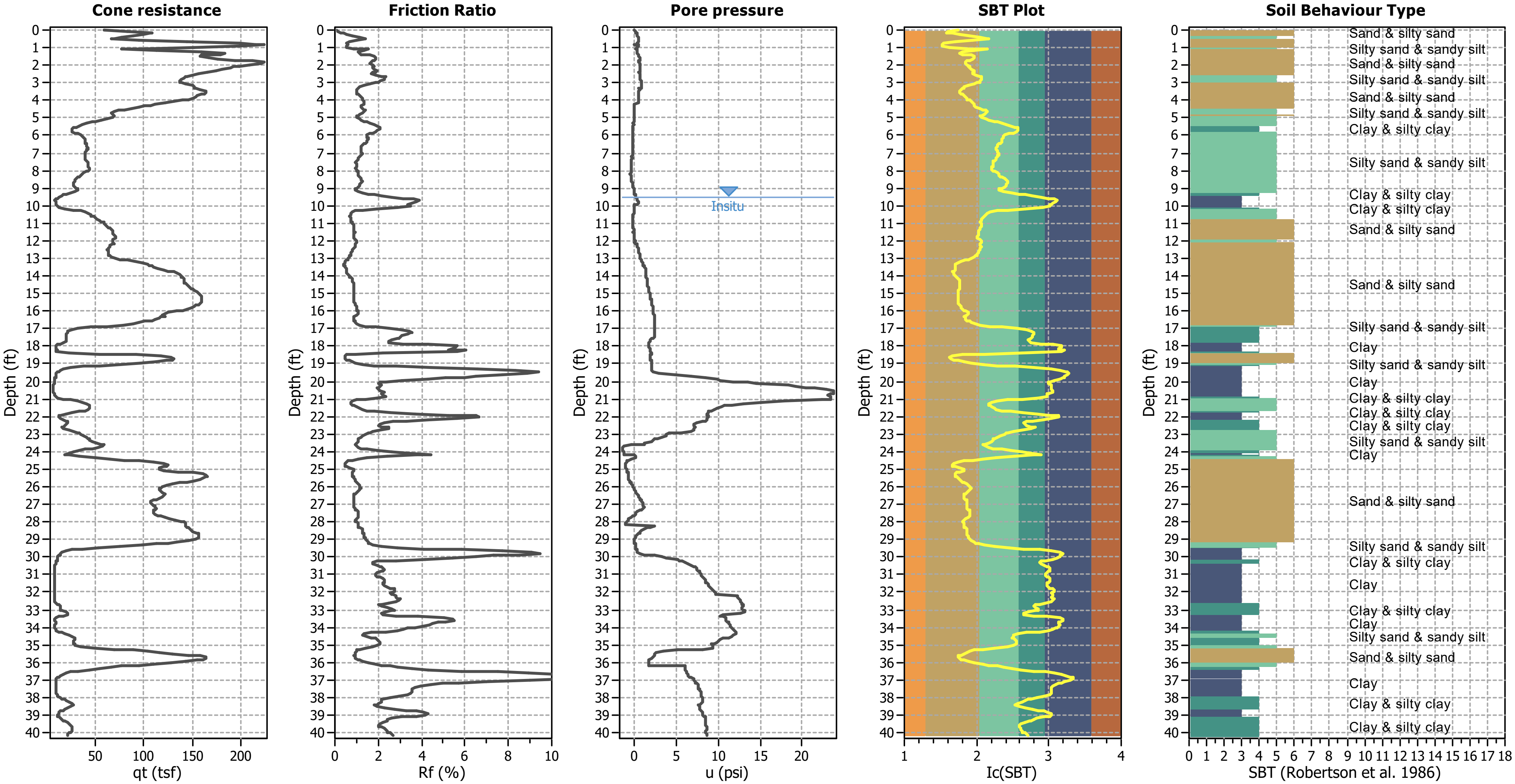






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



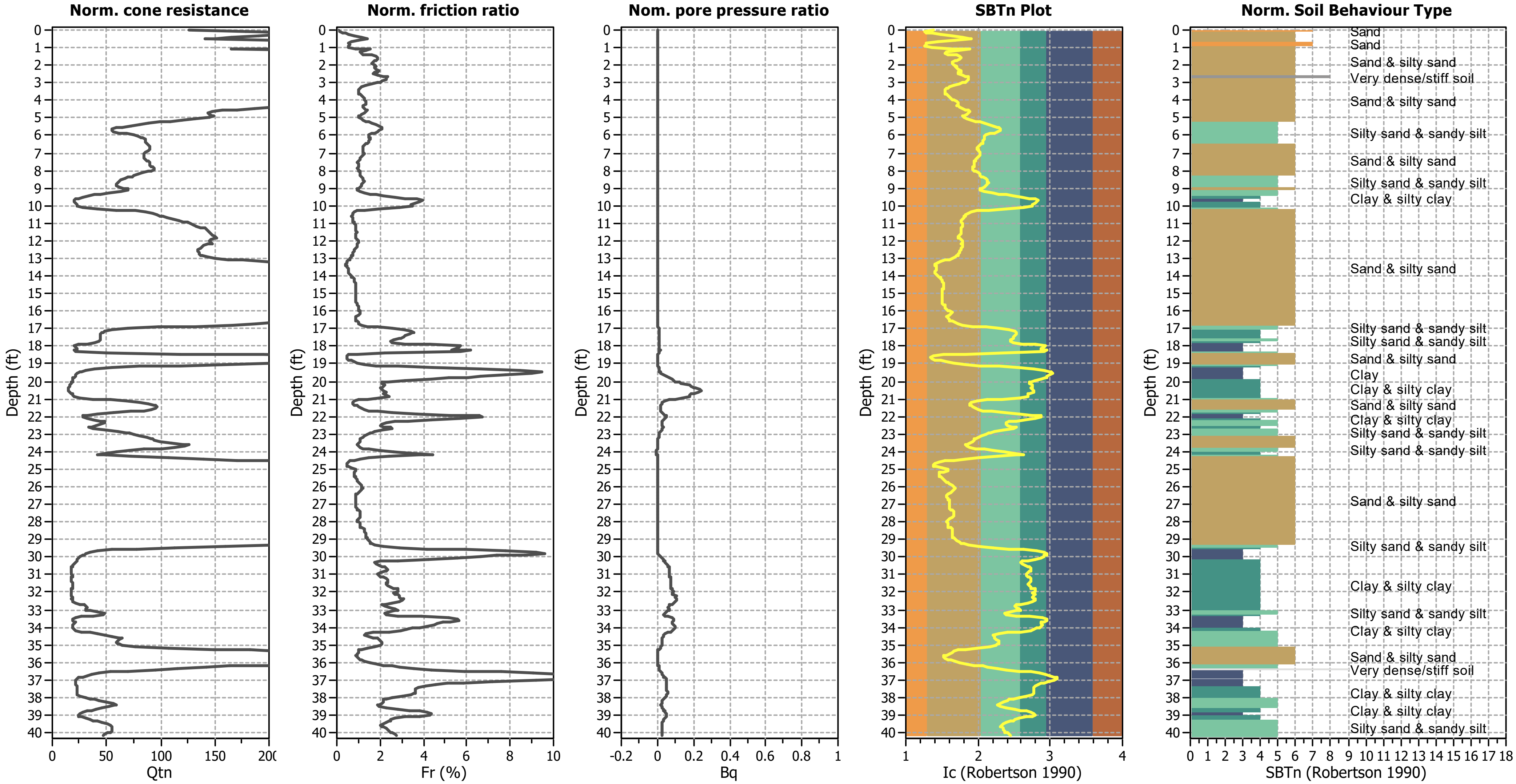
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 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)



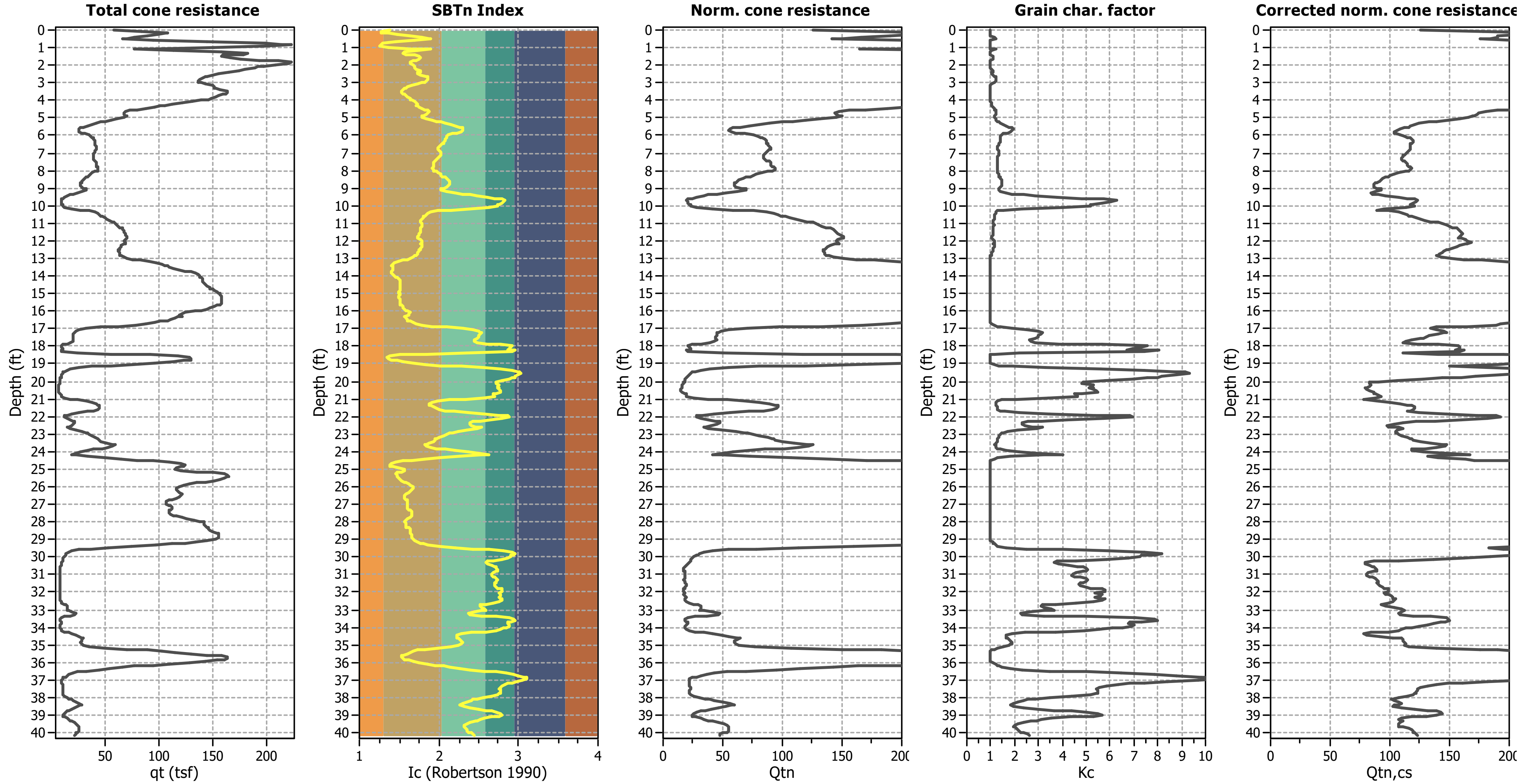
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 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

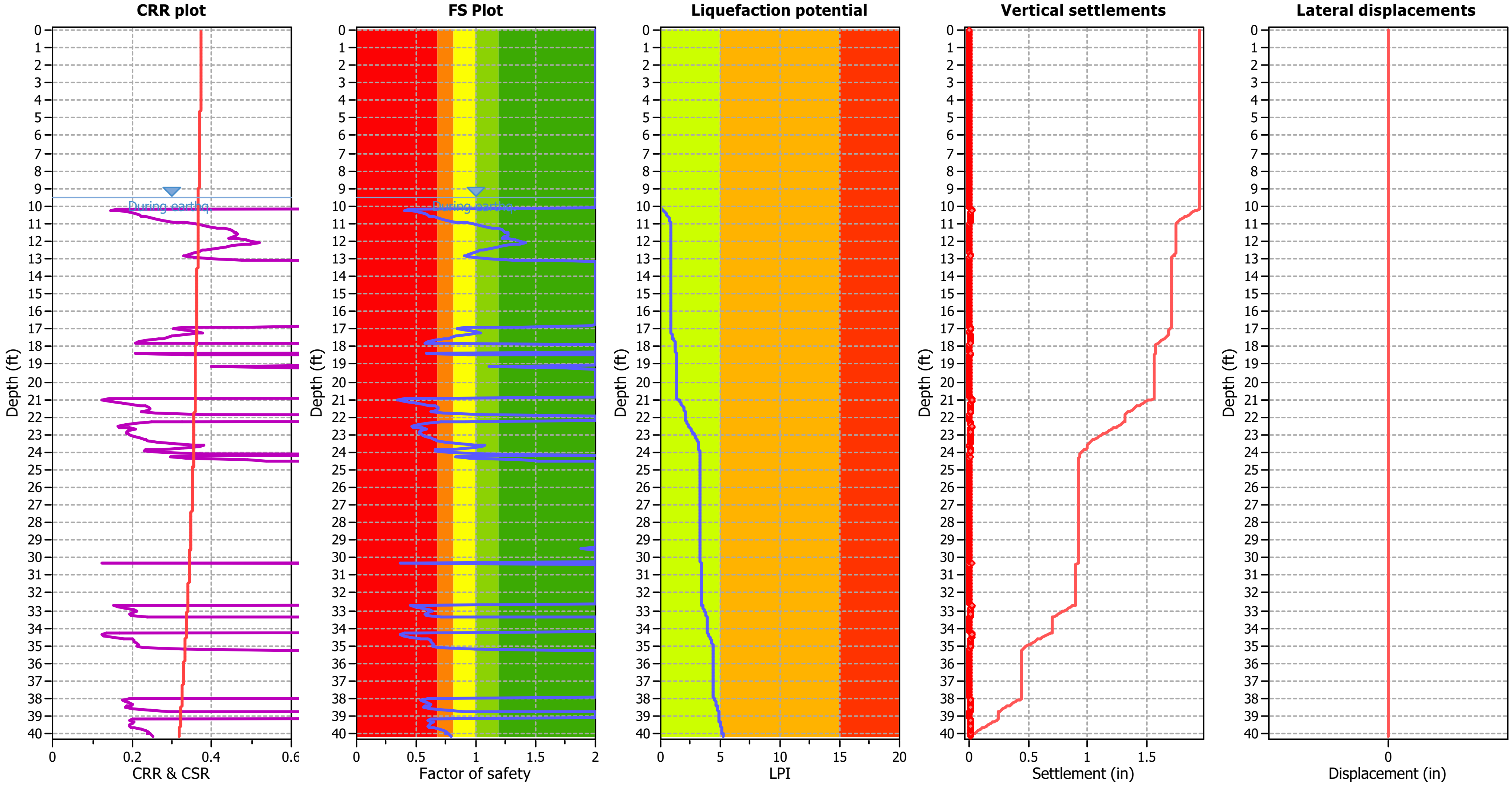
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 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.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

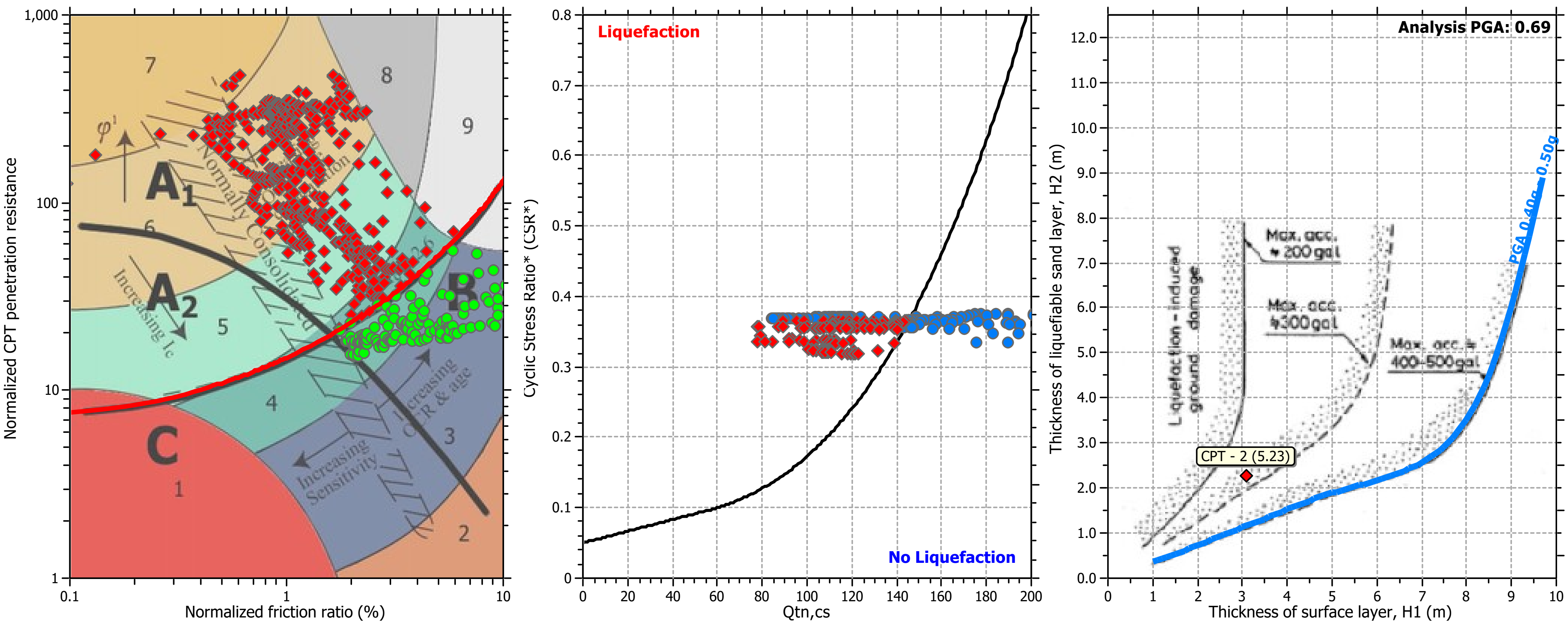
F.S. color scheme

- 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

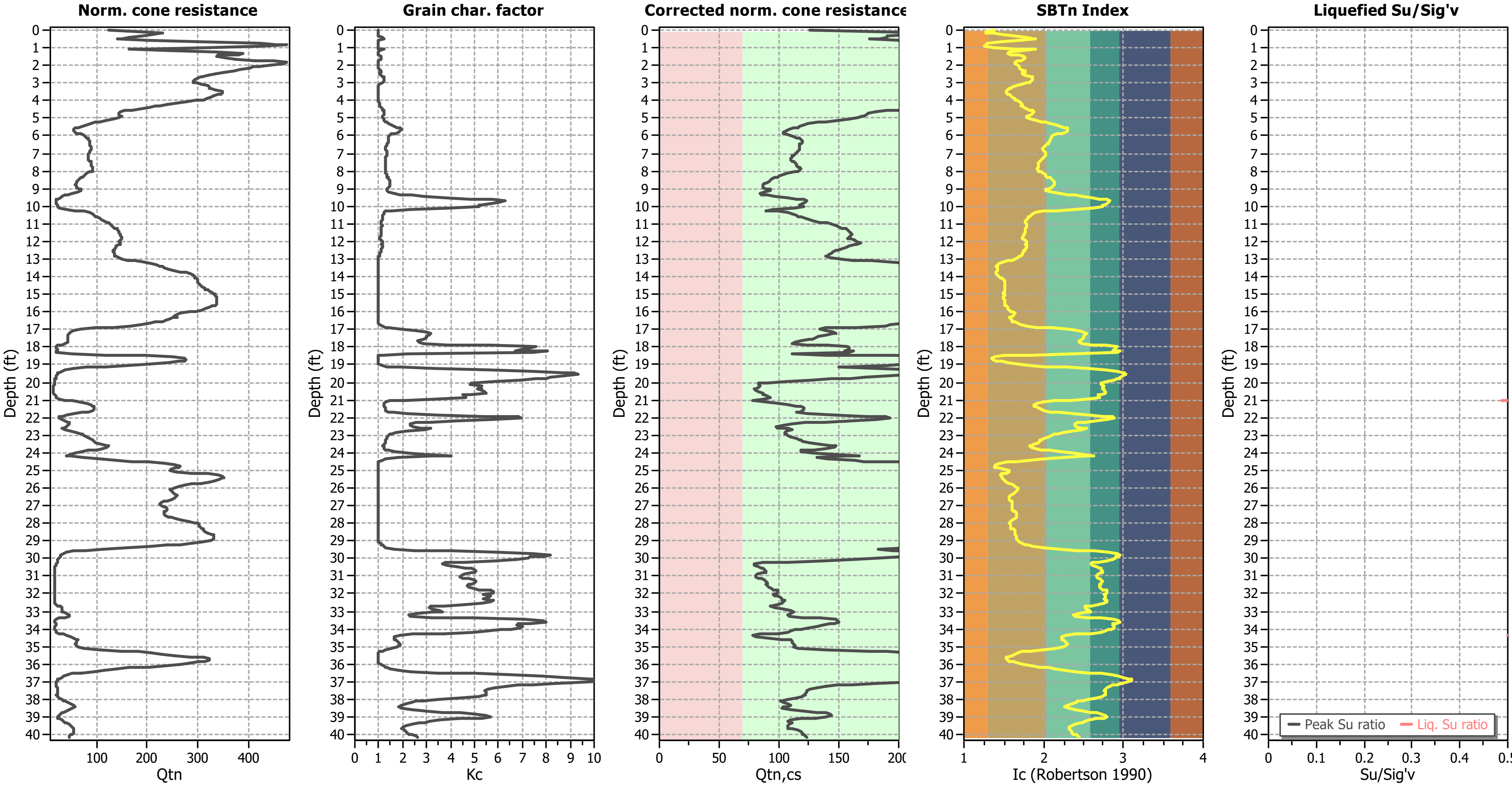
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

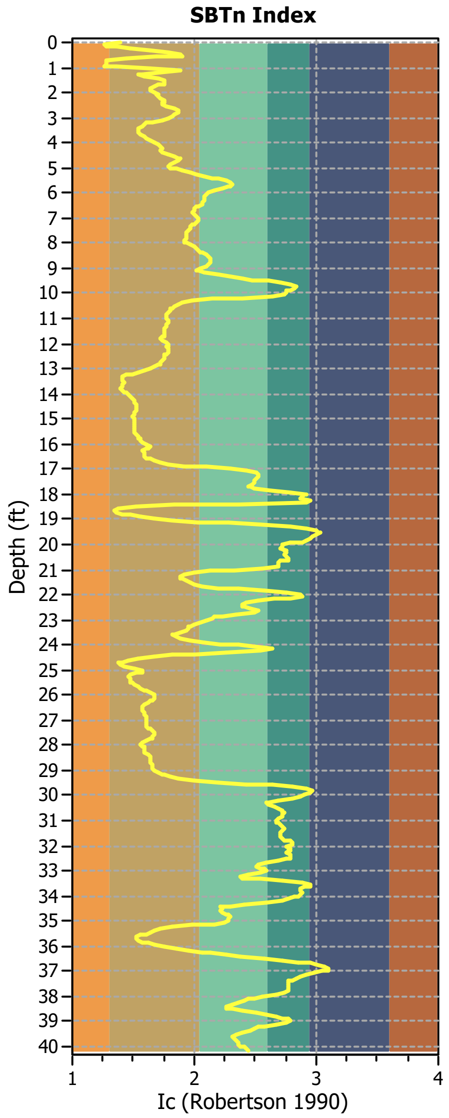
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

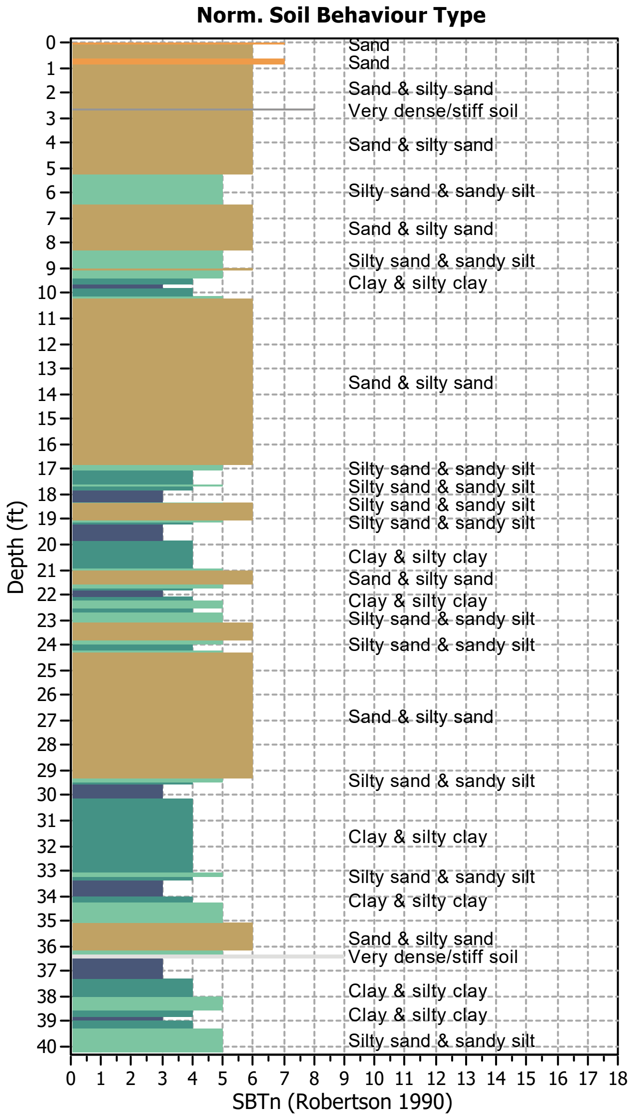
TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

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

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





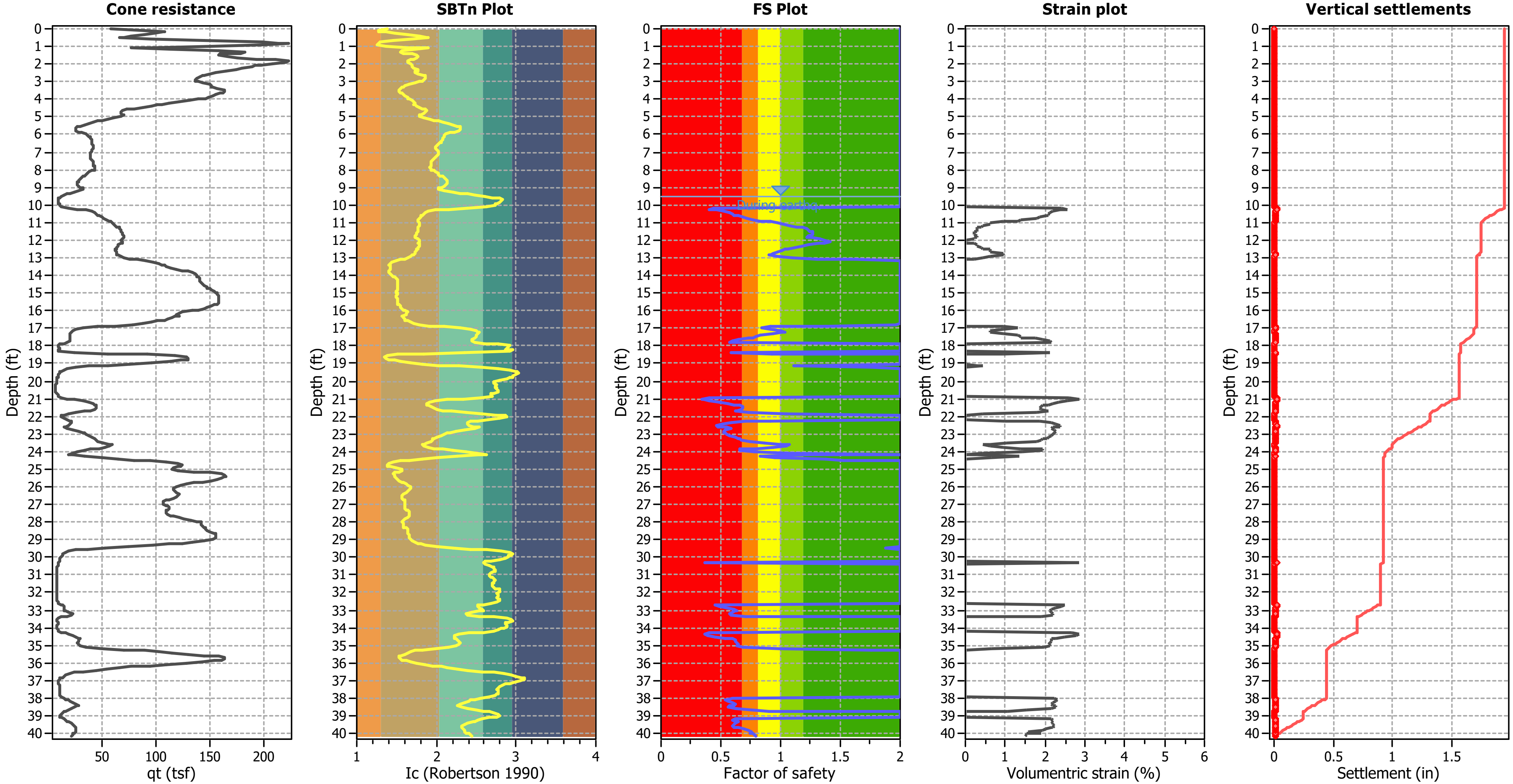
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	1.70
I_c change ratio value:	3.0000
Minimum number of points in layer:	-26215

General statistics

Total points in CPT file:	613
Total points excluded:	0
Exclusion percentage:	0.00%
Number of layers detected:	0

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

NOVA

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

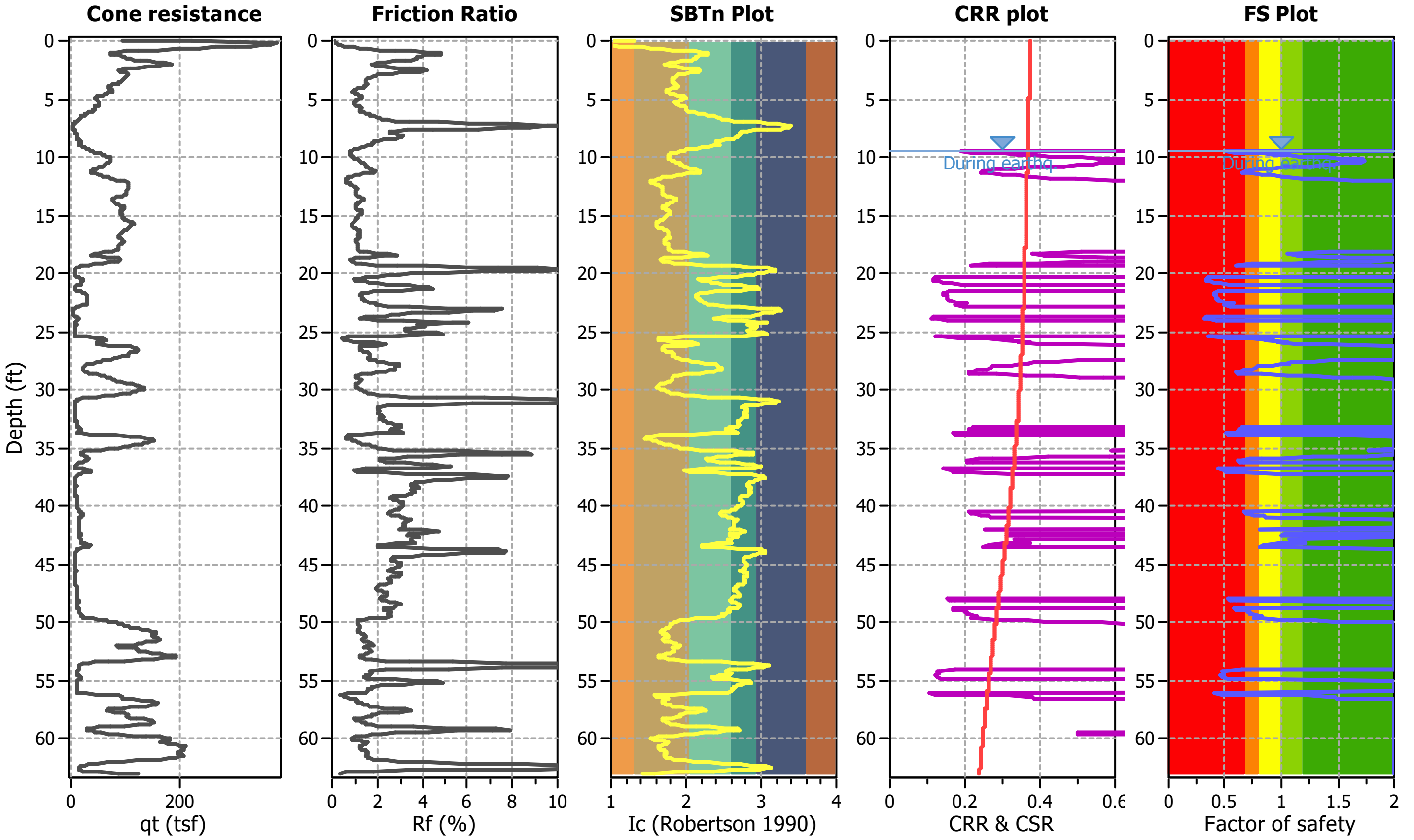
LIQUEFACTION ANALYSIS REPORT

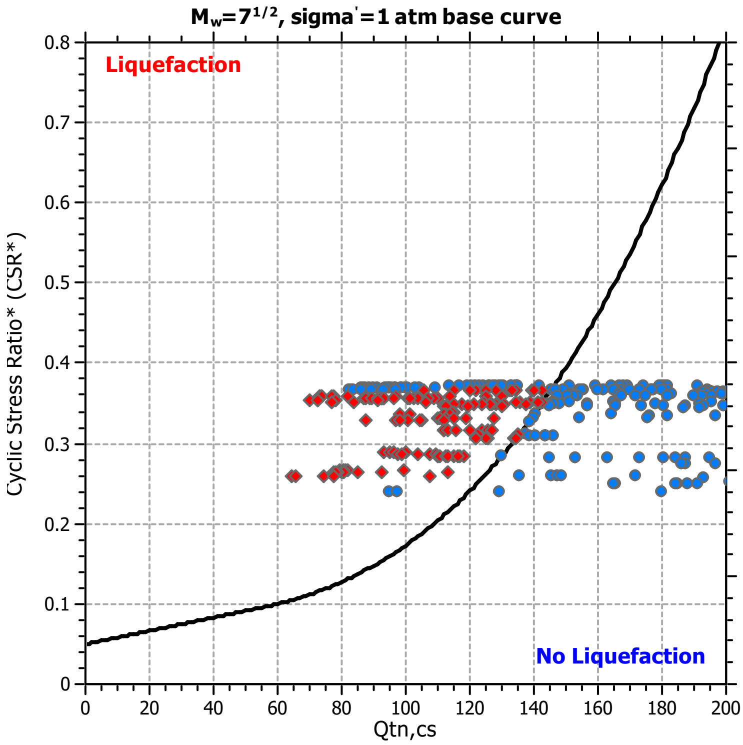
Project title : Old Town Viewpoint **Location : San Diego, California**

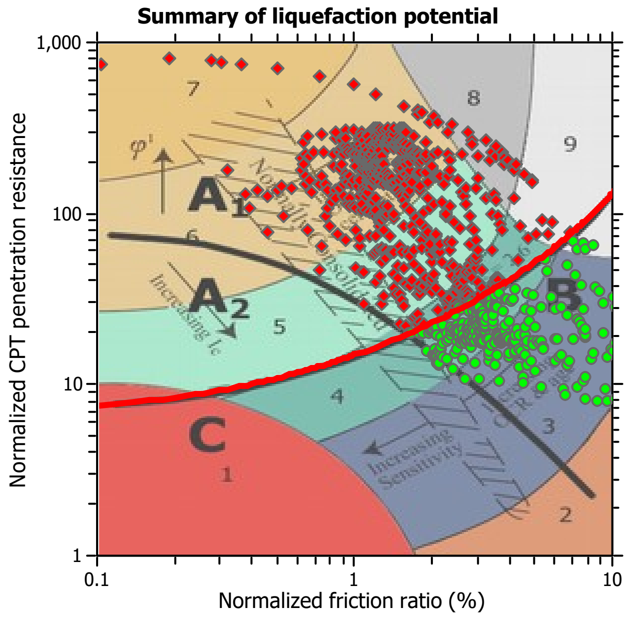
CPT file : CPT - 3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	9.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	9.50 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:	6.99	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.69	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

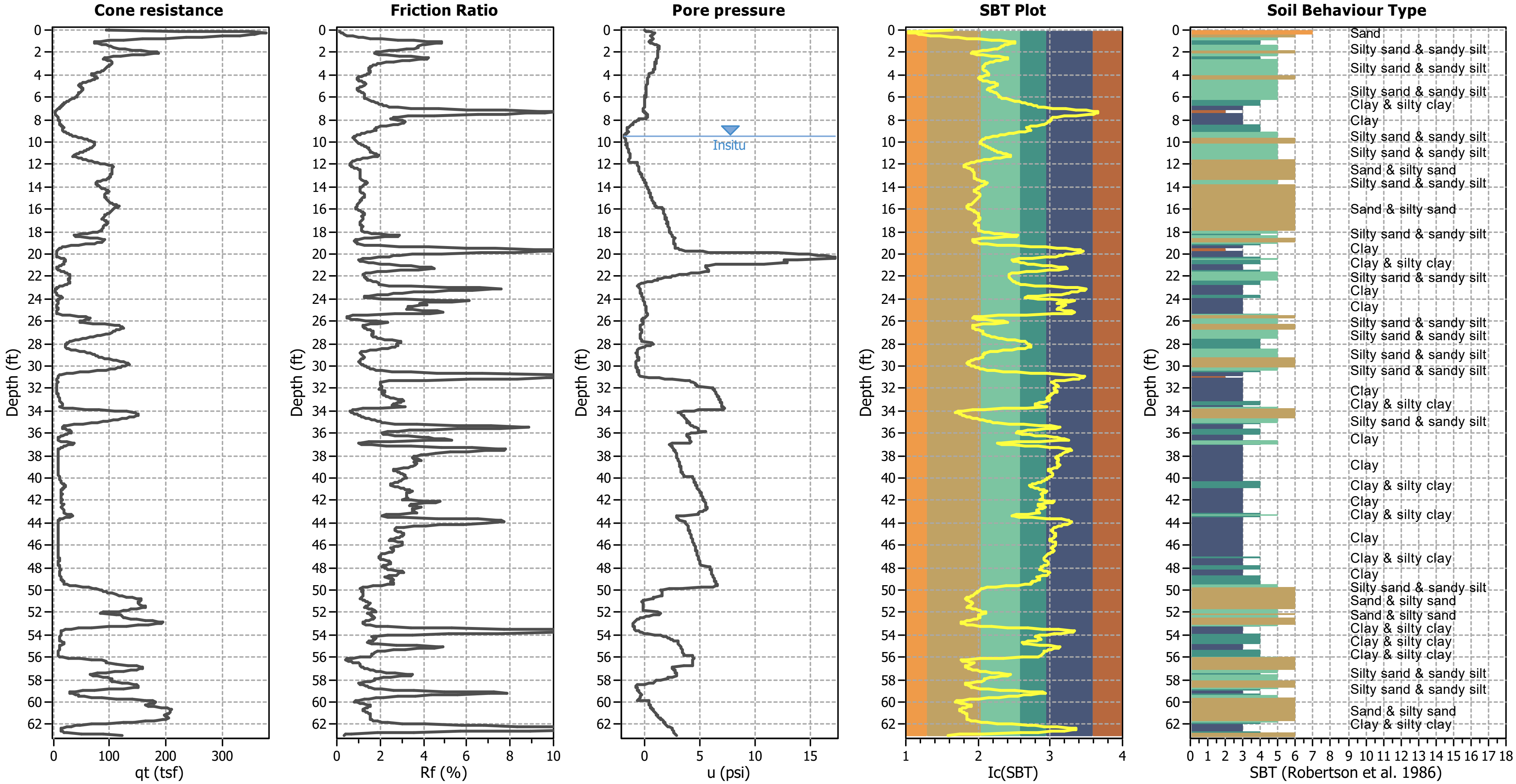






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



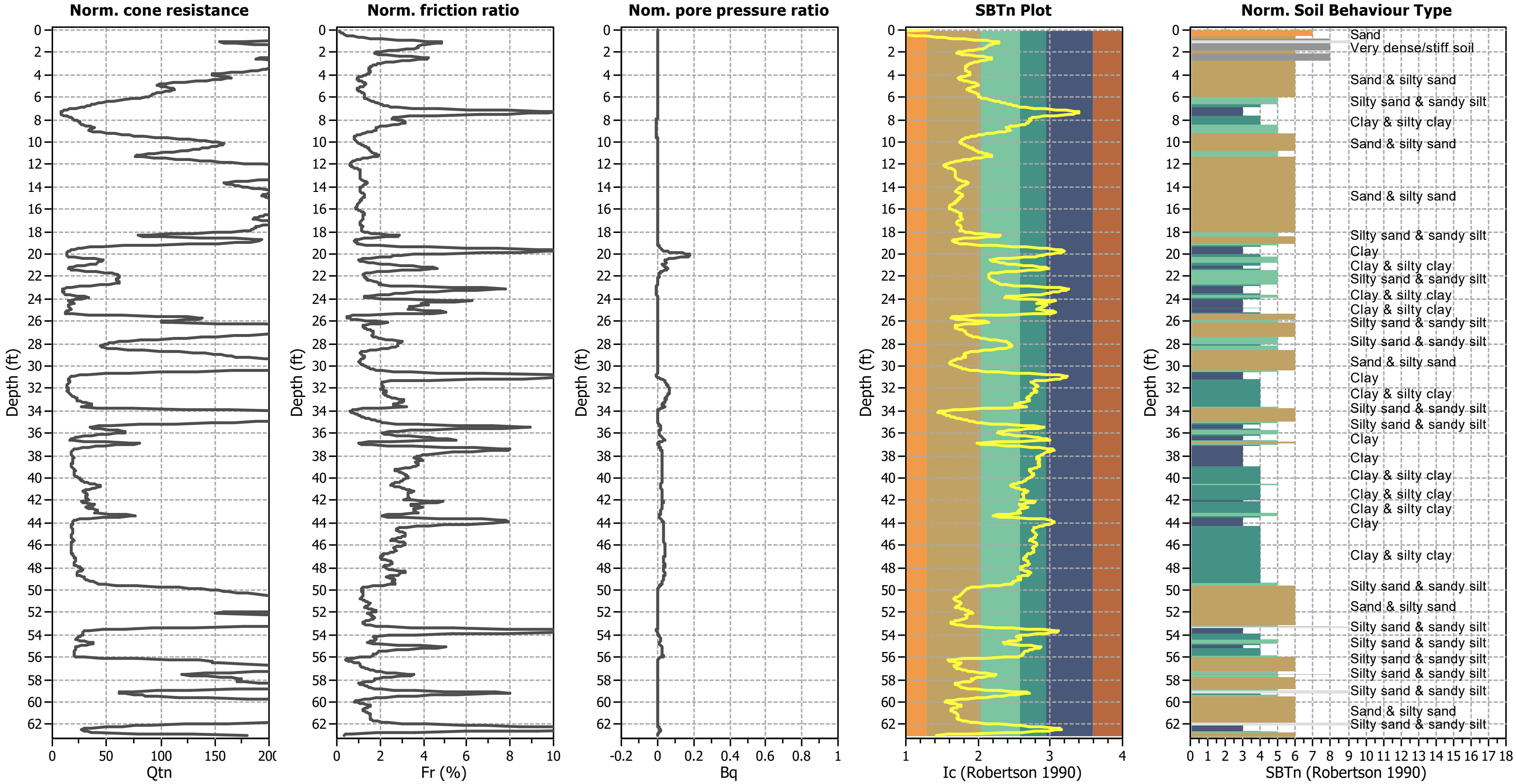
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 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)



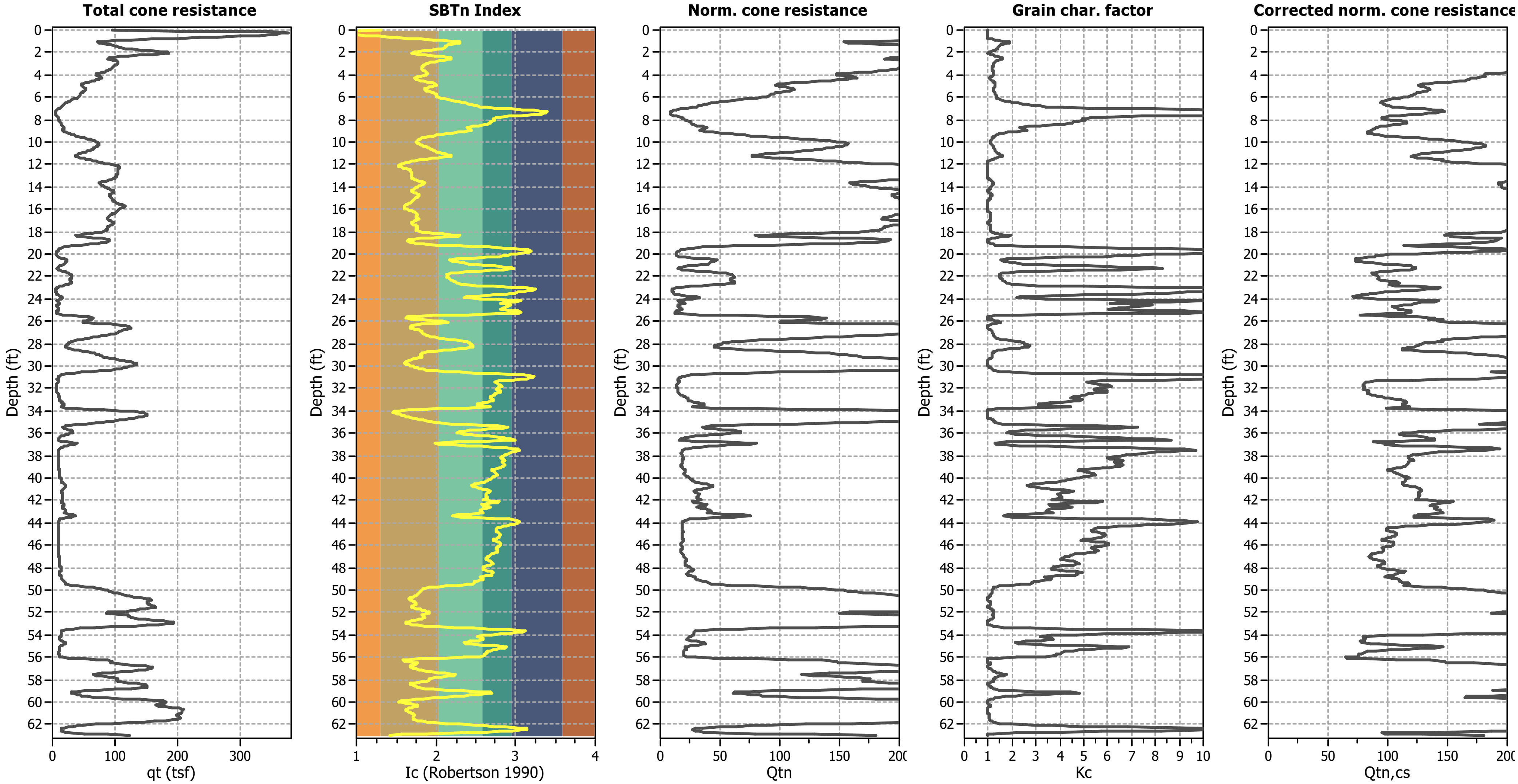
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

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

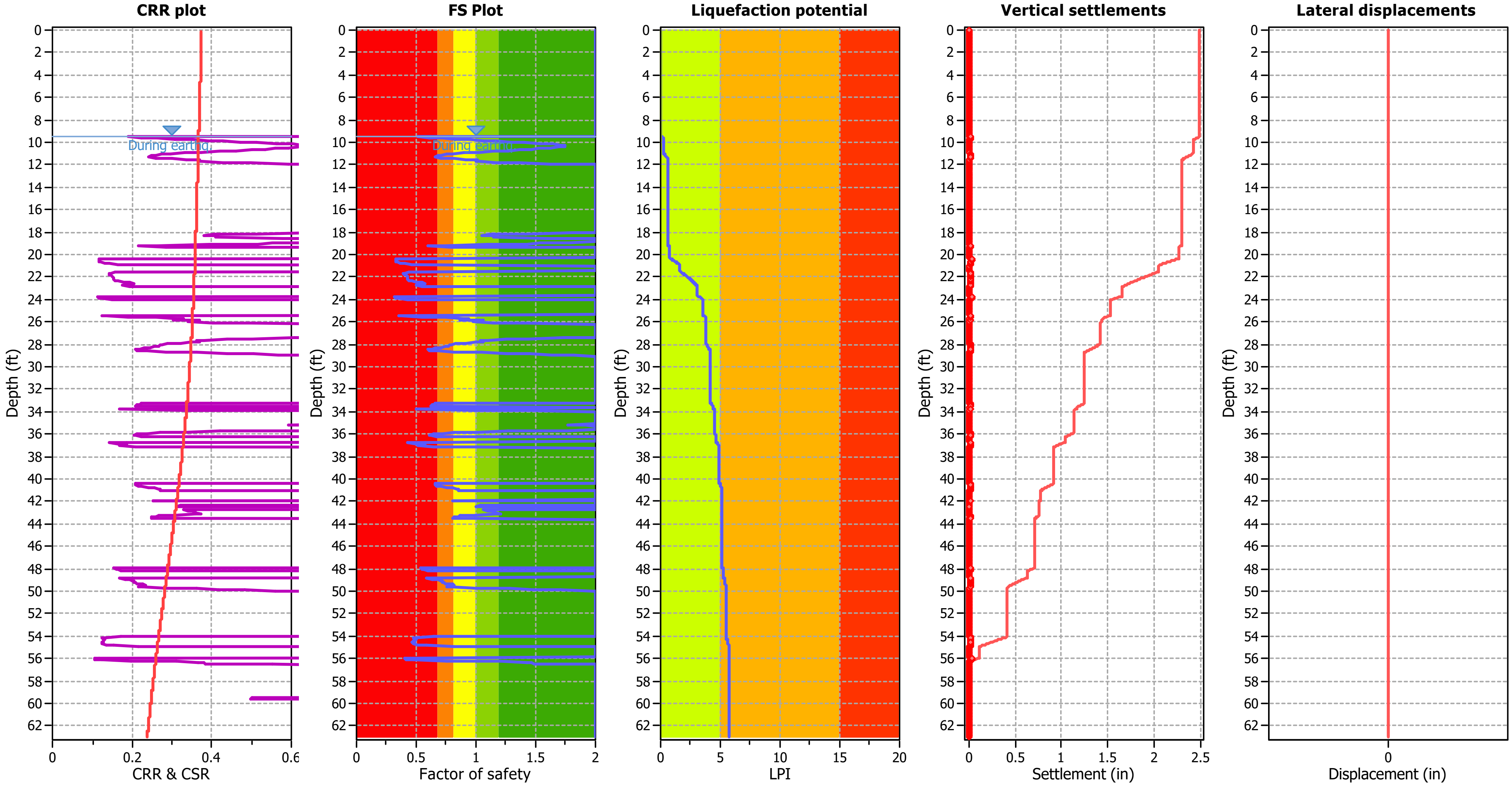
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 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.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

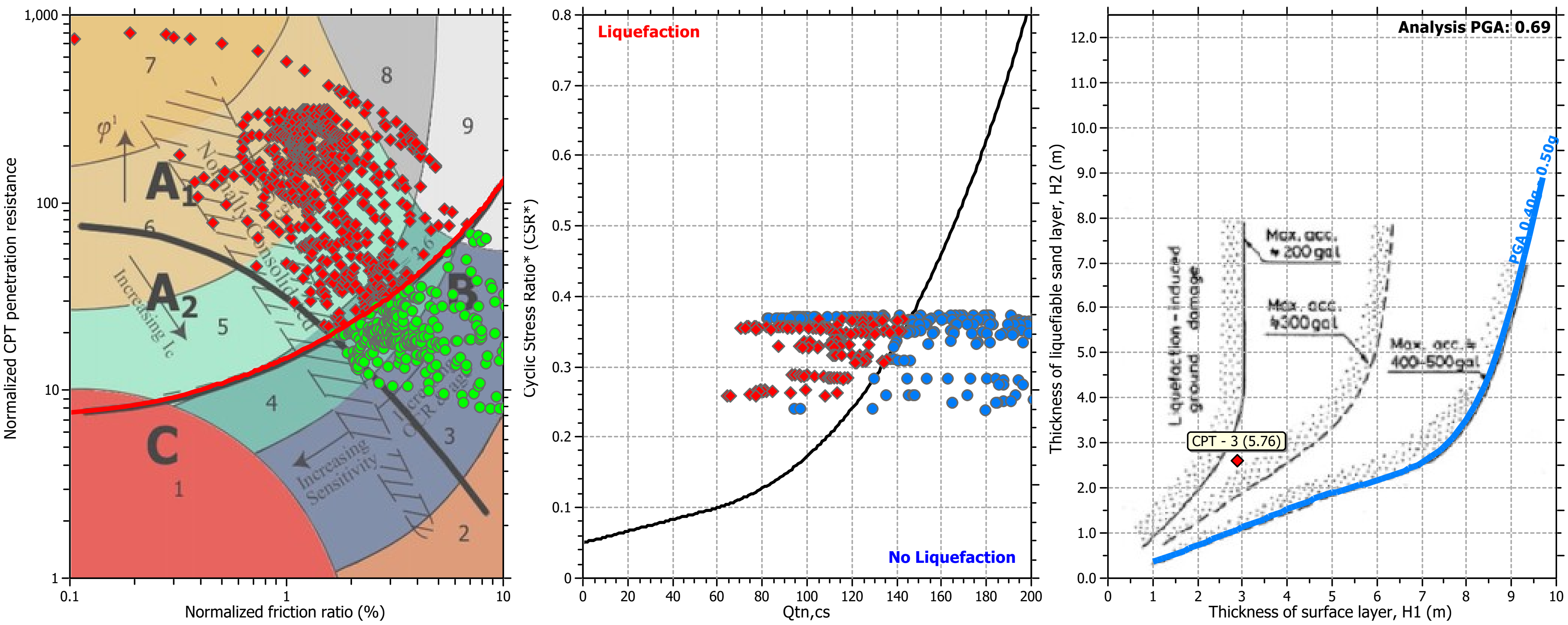
F.S. color scheme

- 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

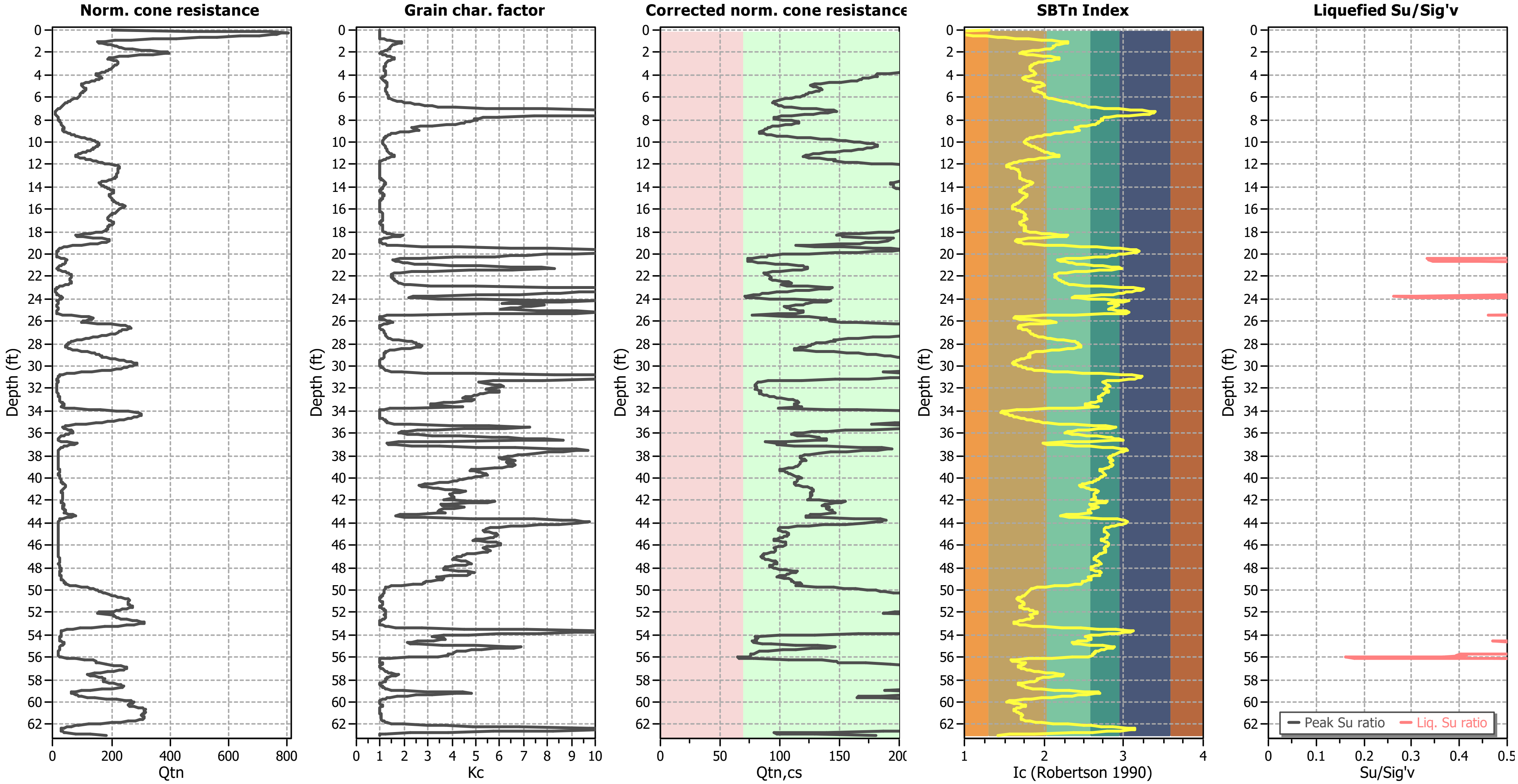
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

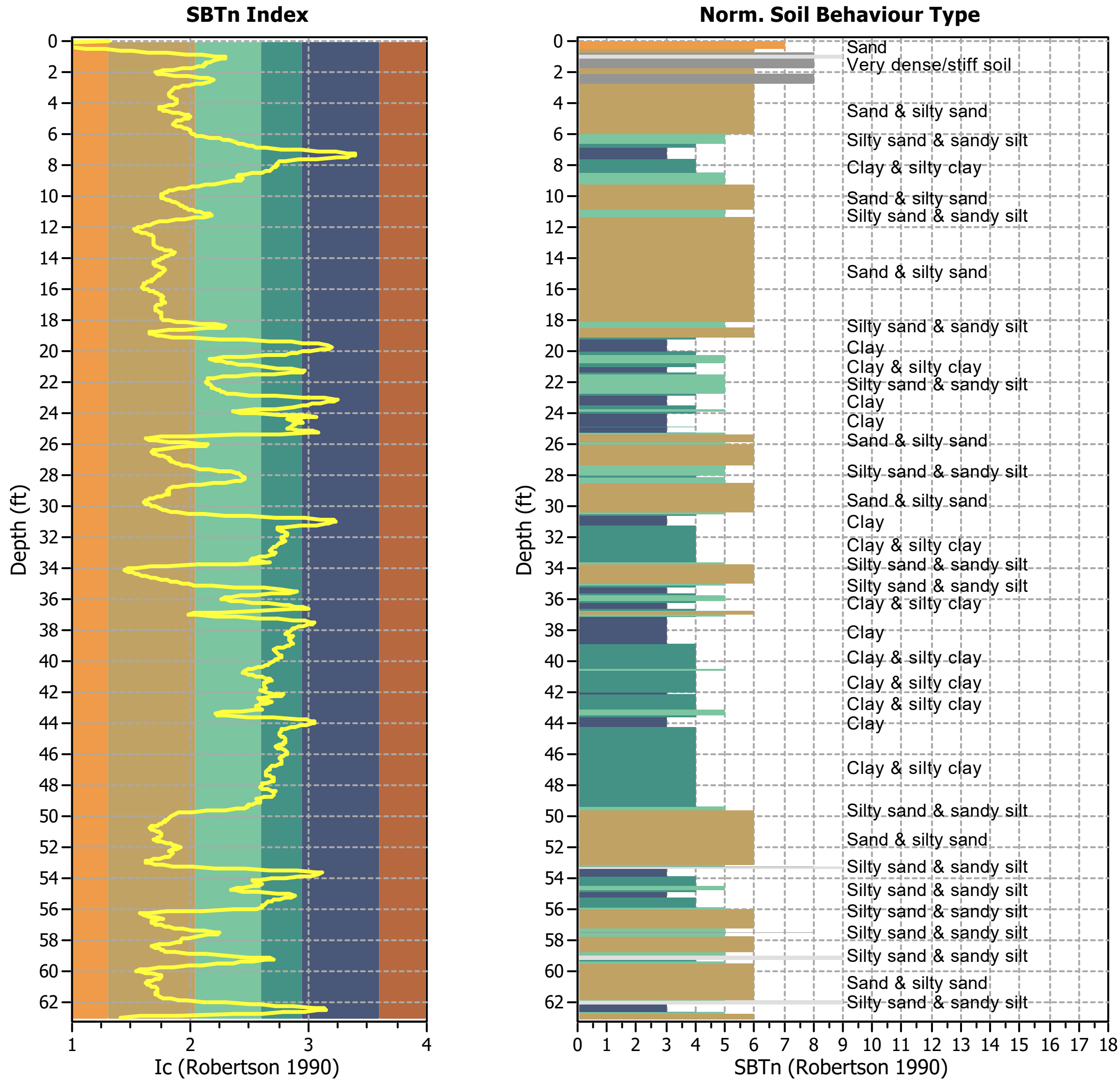
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	9.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.99	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.69	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	9.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

TRANSITION LAYER DETECTION ALGORITHM REPORT
Summary Details & Plots

Short description

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

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



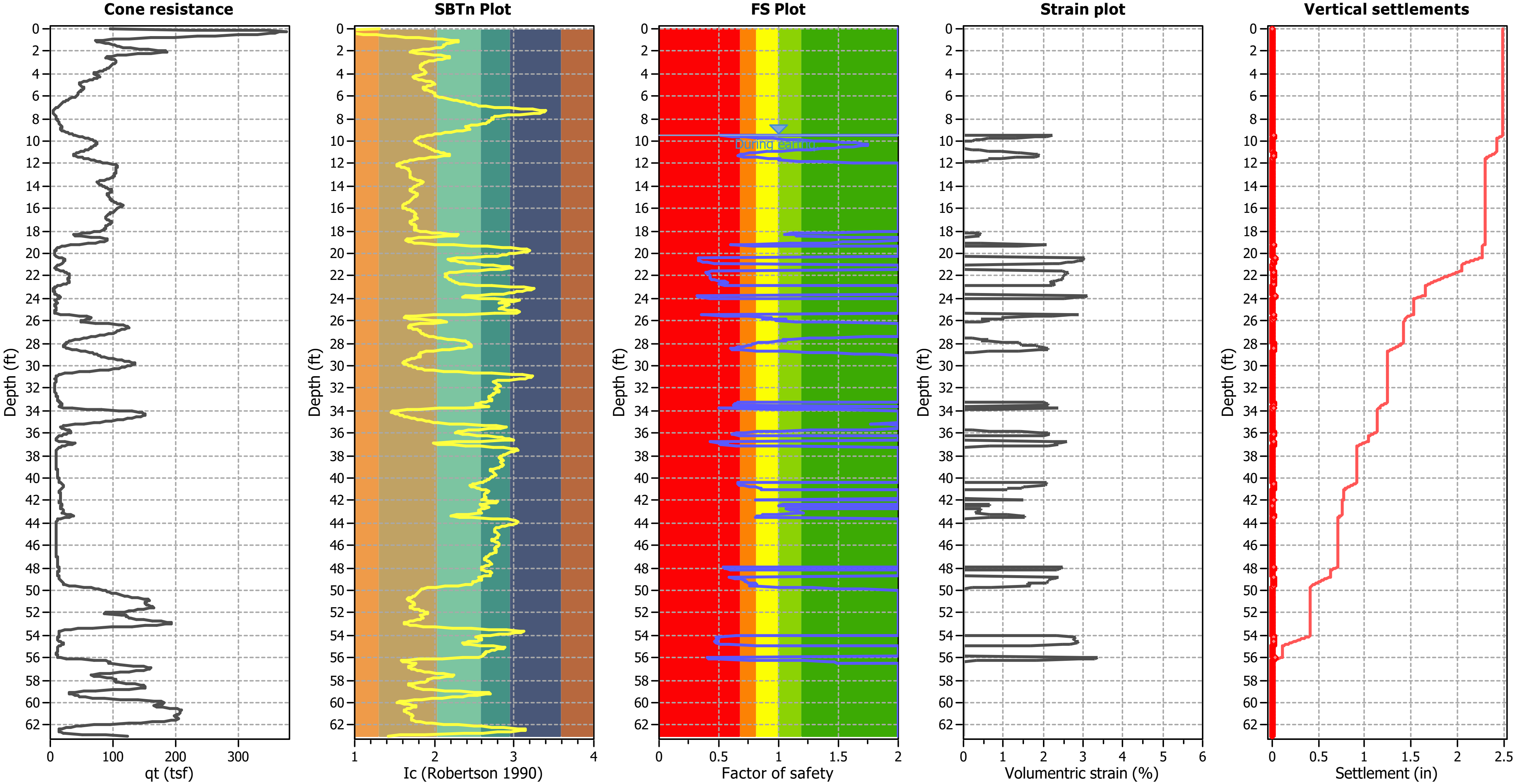
Transition layer algorithm properties

I_c minimum check value:	1.70
I_c maximum check value:	1.70
I_c change ratio value:	3.0000
Minimum number of points in layer:	-26215

General statistics

Total points in CPT file:	961
Total points excluded:	0
Exclusion percentage:	0.00%
Number of layers detected:	0

Estimation of post-earthquake settlements

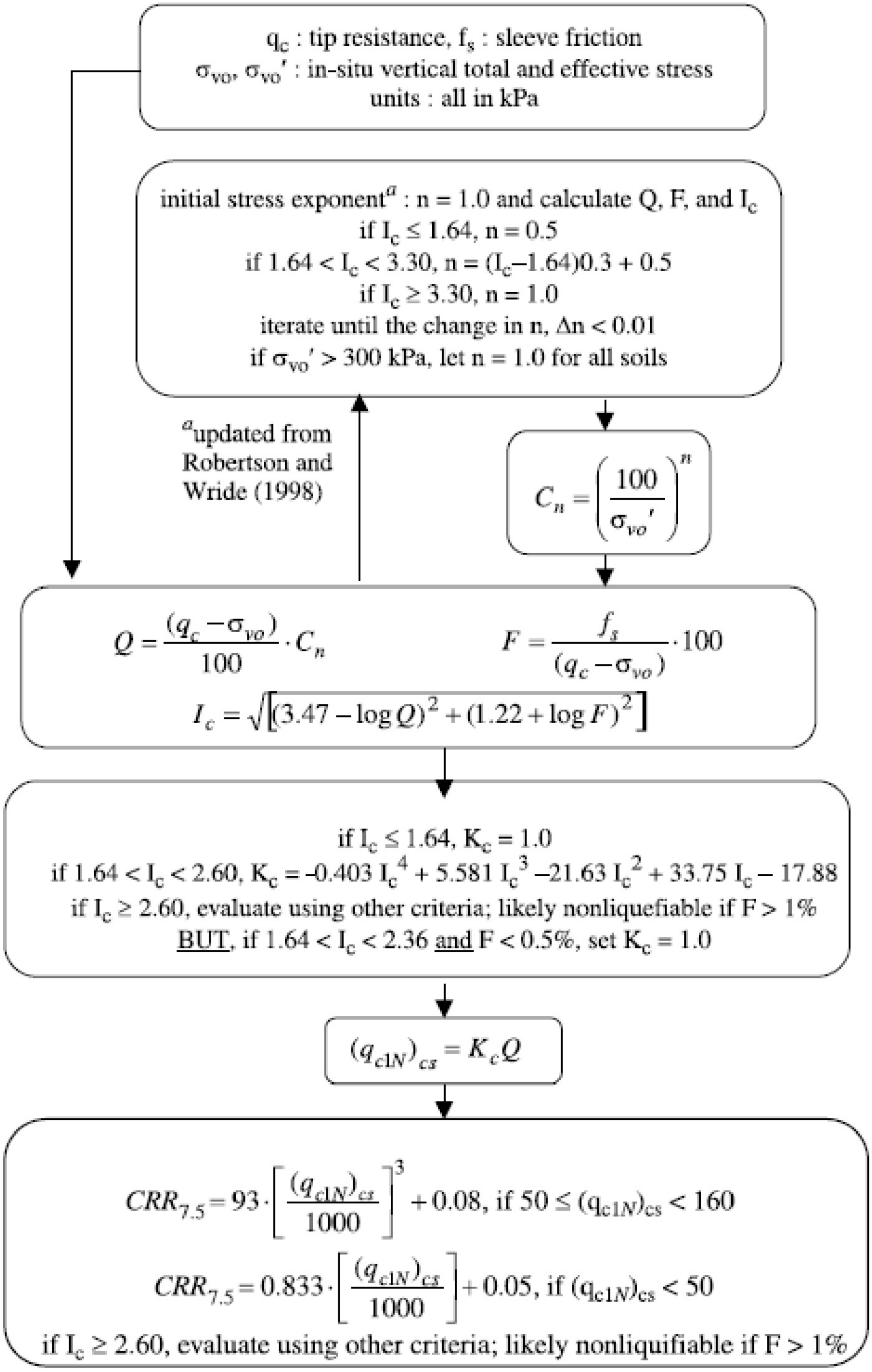


Abbreviations

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

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

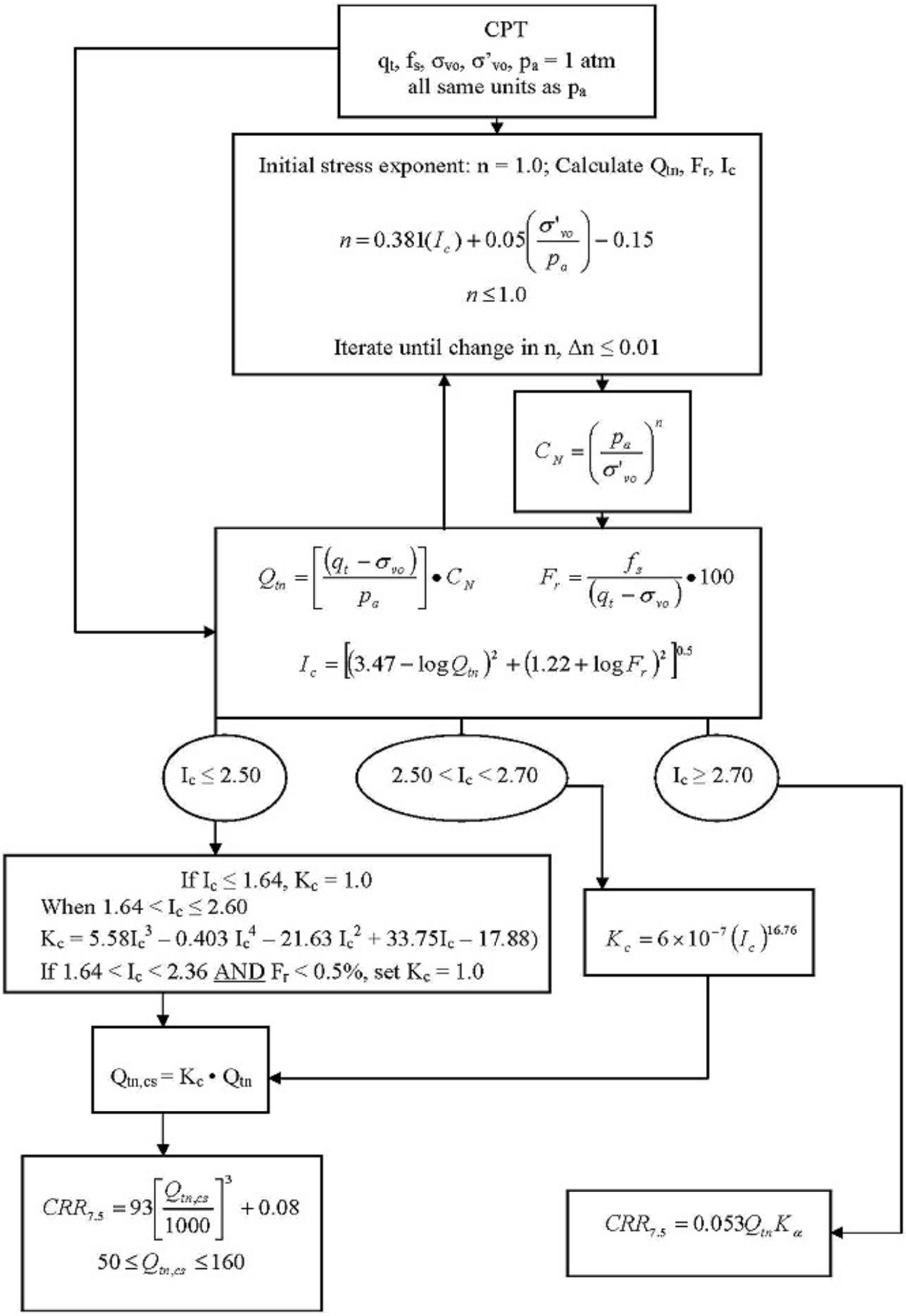
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

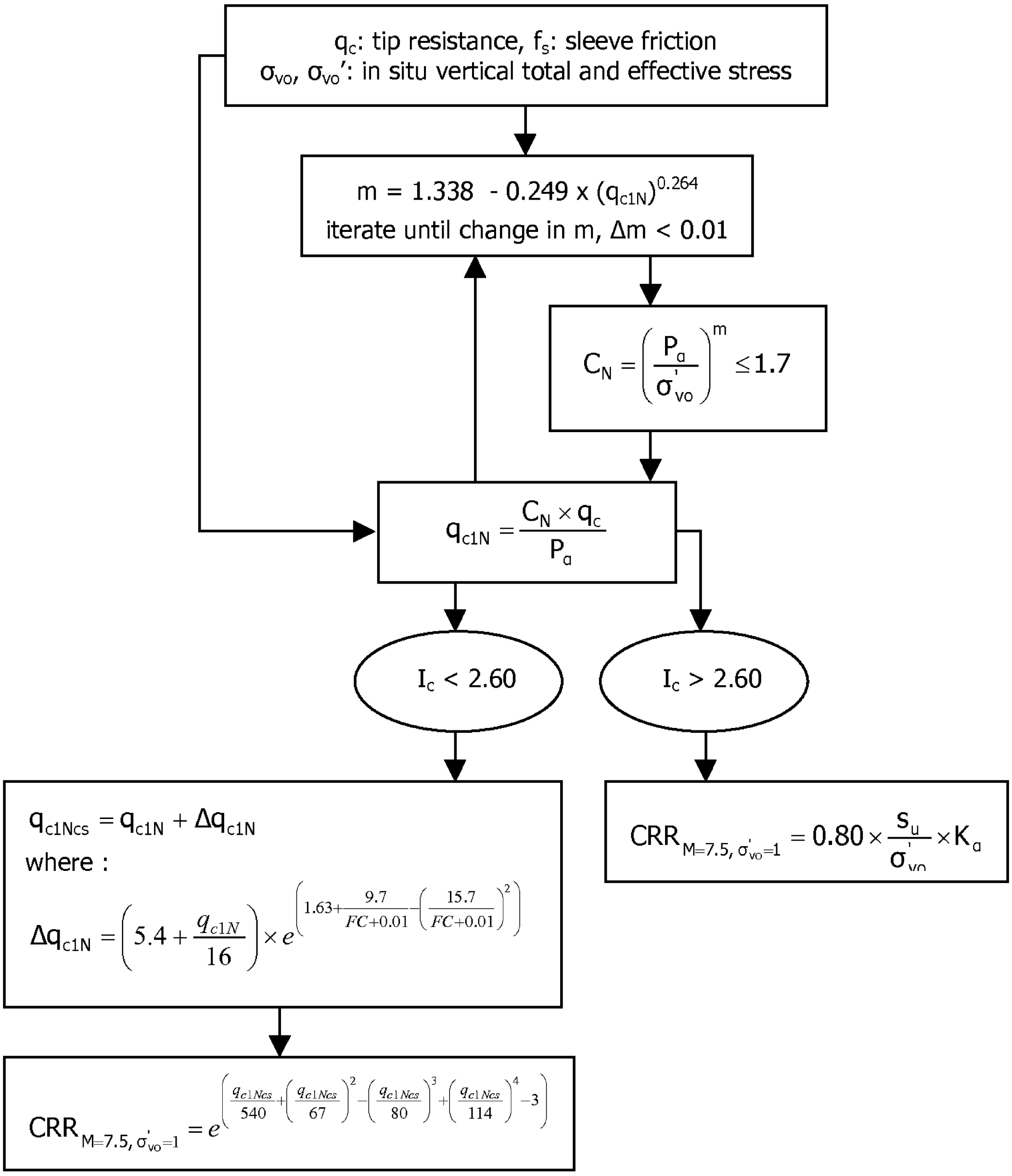
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[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)

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

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

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

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

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

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

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

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

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

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

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

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

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

$$q_{c1N} = C_{N\sigma}\frac{q_c}{P_a}$$

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

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

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

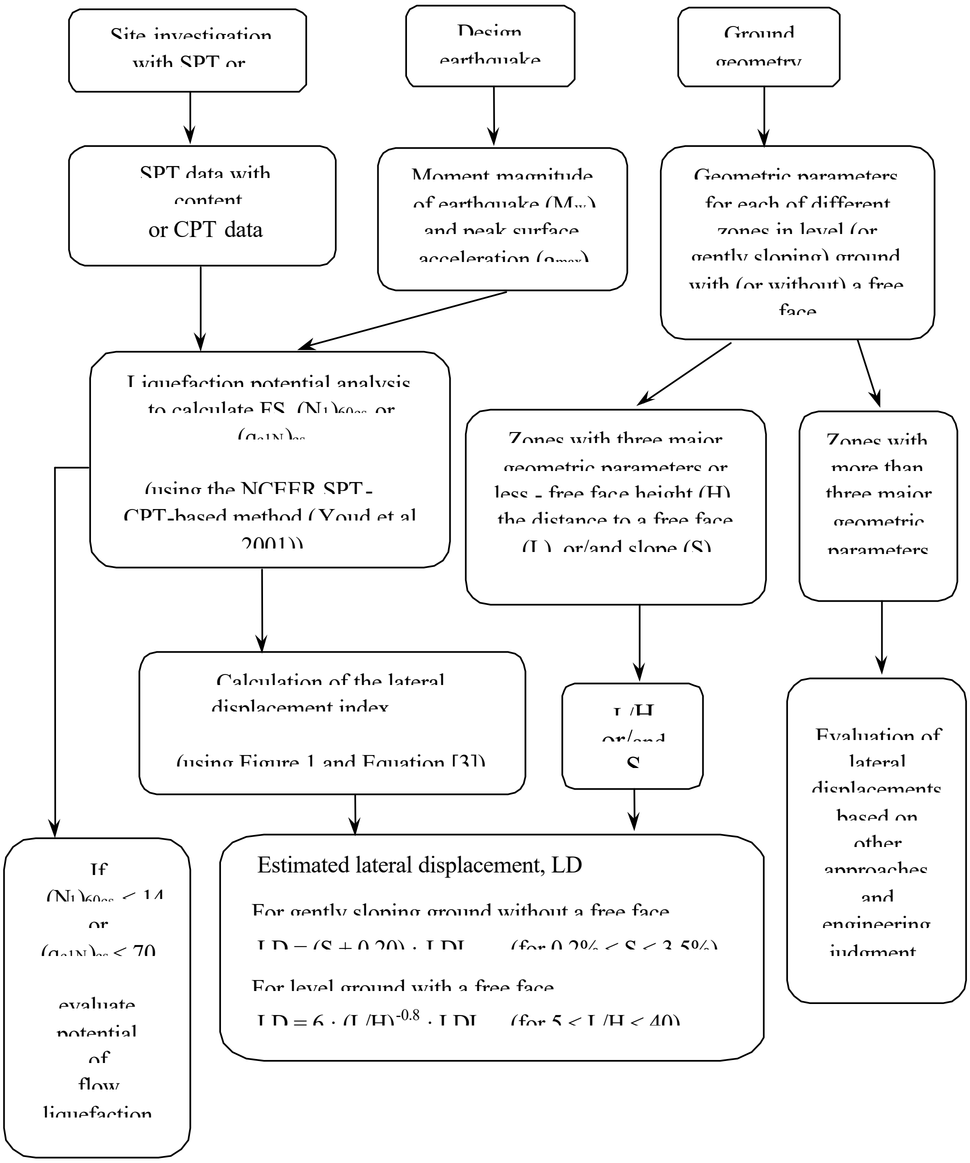
$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

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

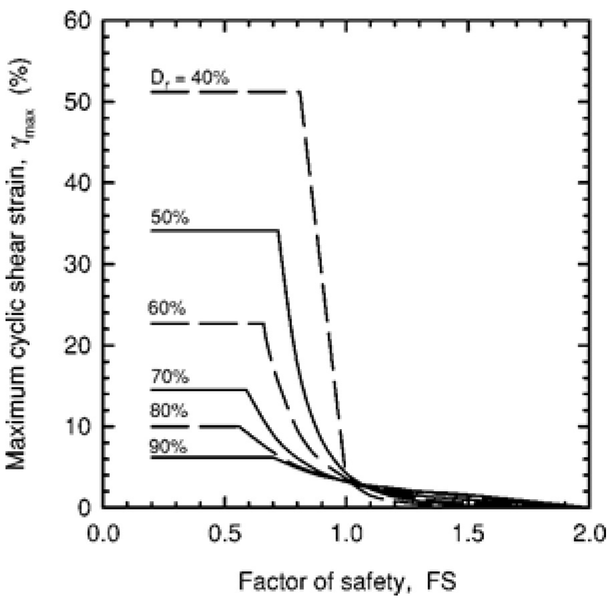
$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

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**

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

[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))

$$\gamma = \left[\frac{1 + a \cdot e^{bR}}{1 + a}\right] \cdot R \cdot 100 \ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$a = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

↓

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.

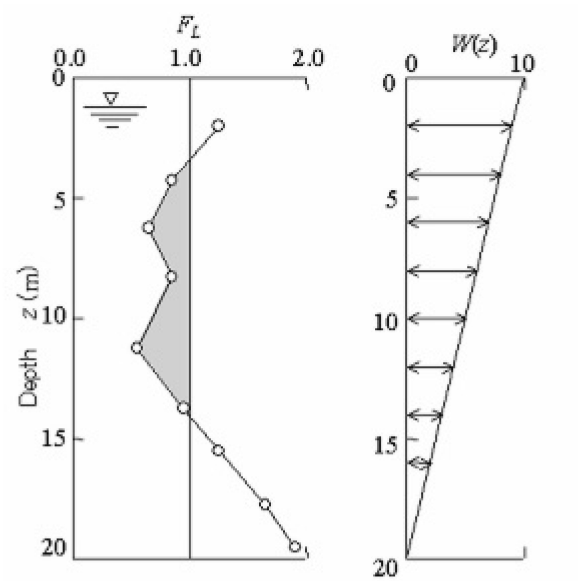
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

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

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 G
INFILTRATION FEASIBILITY CONDITION LETTER

Viewpoint Development LLC
Mr. Chris Livoni
1635 Pacific Ranch Drive
Encinitas, CA 92024

July 18, 2022
NOVA Project No. 2021073

Subject: Infiltration Feasibility Condition Letter
Viewpoint Old Town Apartments
4620 Pacific Highway, San Diego, California

References: NOVA Services, Inc., 2022. *Report Geotechnical Investigation, Viewpoint Old Town Apartments, 4620 Pacific Highway, San Diego, California*, NOVA Project No. 2021069, July 18, 2022.

carrierjohnson + culture (CJC), 2022, *Viewpoint Old Town, 46220 Pacific Hwy, San Diego, CA 92110,* 38 Sheets, Plot Date 3/31/2022.

City San Diego, 2021, Stormwater Standards Manual, Effective Date: May 2021.

City of San Diego. 2008, Seismic Safety Study, Grid 20, dated April 3.

Dear Mr. Livoni,

The intent of this letter is to provide the findings of an assessment by NOVA Services, Inc. (NOVA) of the infiltration conditions and related feasibility for permanent stormwater Best Management Practices ('stormwater BMPs') for drainage management areas (DMAs) at the above-referenced site.

The assessment has been prepared by NOVA for the Viewpoint Old Town Apartments. NOVA is retained by Viewpoint Development as Geotechnical Engineer-of-Record (GEOR) for the project.

The assessment provides an analysis of the infiltration feasibility in accordance with the criteria detailed in Section C.1.1 Simple Feasibility Criteria of the referenced City of San Diego BMP Design Manual (San Diego 2021). Based on these criteria, it is NOVA's opinion that this site should be considered to have a 'no-infiltration' condition.

EXISTING GEOLOGIC AND GEOTECHNICAL CONDITIONS

Section C.1 of the BMP Manual states that if one of the standard setbacks listed cannot be achieved, the DMA may classify as a 'no infiltration condition'. Consideration of the existing fill thickness across the site and the location of the proposed BMPs, preclude the implementation of infiltration for the proposed BMPs.

As reported in NOVA 2022 and presented in Figure 1, the entire site is mapped on the regional geologic map as "af" a deep layer of undocumented artificial fill. Based on our subsurface investigation, this layer is approximately 15 feet deep. The BMP manual states that full and partial infiltration BMPs should not be placed within existing fill soils greater than 5 feet thick.

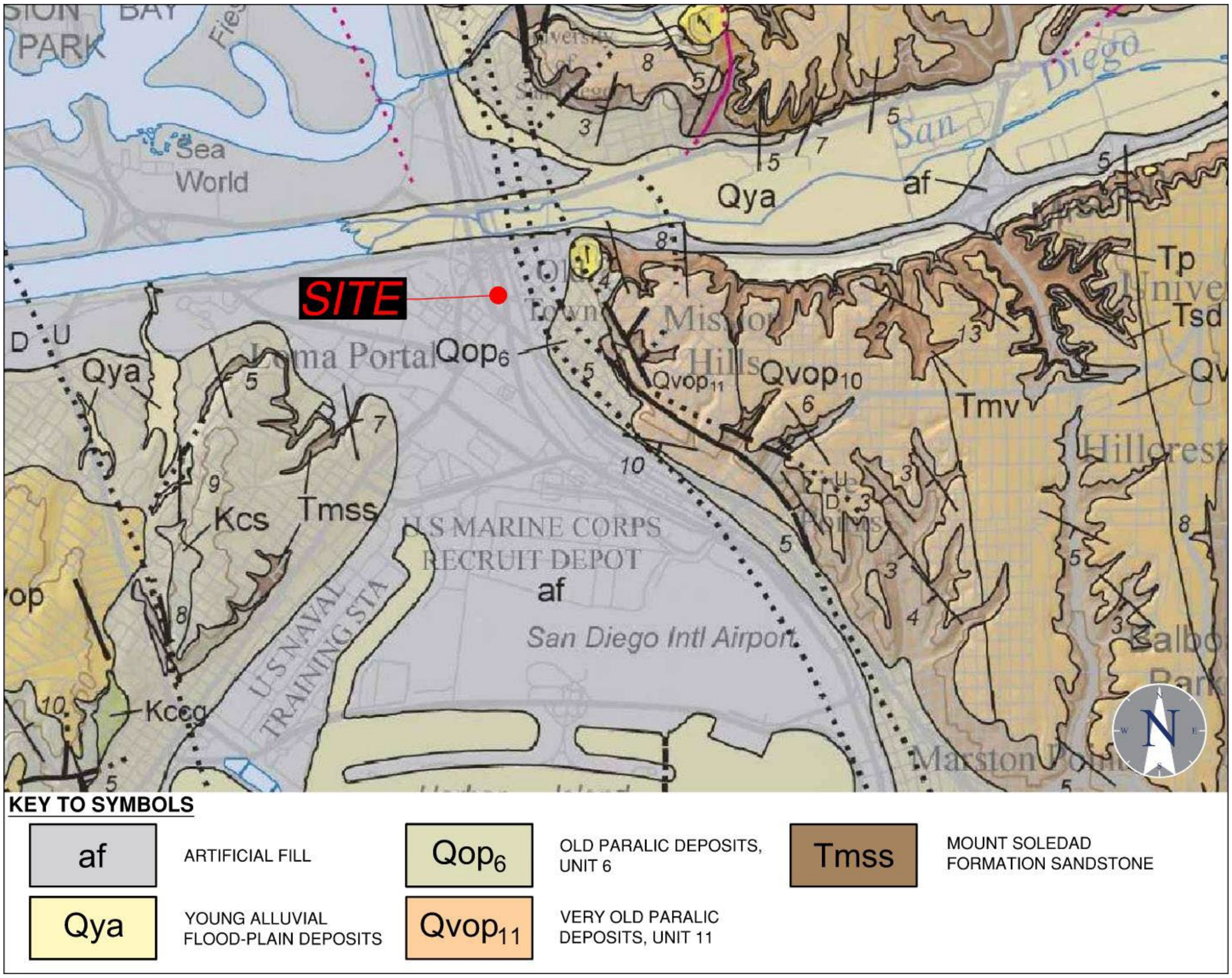

Figure 1. Regional Geology Map (Kennedy and Tan, 2008)

In addition, groundwater was measured at elevation +0.3 feet mean sea level- 9.7 feet below the existing ground surface. If infiltration were to be allowed, the infiltration surface would be far less than the recommended 10 feet of vertical separation between the infiltration surface and groundwater.

Finally, as shown in Figure 2, this site is mapped by the City of San Diego Seismic Safety Study as an area highly susceptible to liquefaction. NOVA has provided a liquefaction analysis on the site and determined that ground improvements or deep foundations are necessary to mitigate settlement caused by liquefaction.

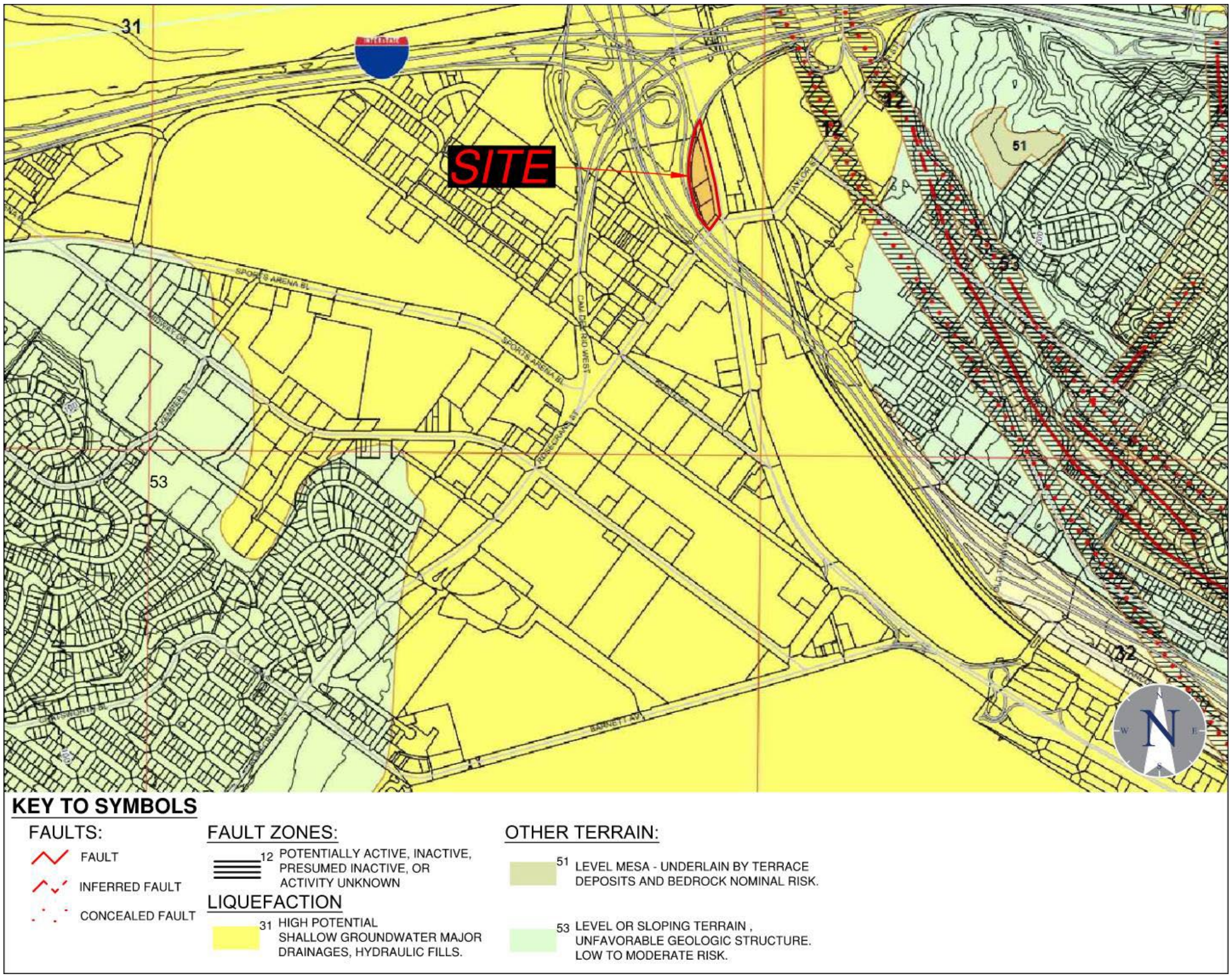

Figure 2. Site Location on City of San Diego Seismic Safety Study Map
(source: City of San Diego, 2008)

INFILTRATION FEASIBILITY CRITERIA FROM C.1.1

The following text reproduces the discussion points from Appendix C.1.1 in the referenced City of San Diego BMP Design Manual (San Diego 2021) for an infiltration feasibility condition letter. The discussion points from San Diego 2021 are reproduced below in italics, following which a response is provided by NOVA.

- *The phase of the project in which the geotechnical engineer first analyzed the site for infiltration feasibility.*

  The project is currently in the planning phase of the site's development.

- *Results of previous geotechnical analyses conducted in the project area, if any.*

  NOVA is not aware of previous geotechnical investigations at this site.

- *The development status of the site prior to the project application (i.e., new development with raw ungraded land, or redevelopment with existing graded conditions).*

  The approximately 1.75-acre site is comprised of APN's 442-740-03-00, 442-740-06-00, 442-740-07-00, nominally located at 4620 Pacific Highway in San Diego. The site is bounded on the east by Pacific Highway. The arcuate-shaped connector between Interstate 5 North to Interstate 8 East bounds the site to the north and west, with Rosecrans Street to the south.

  The site is level, ranging from an elevation of +10 feet mean sea level (msl) on the north side of the site to +11 feet msl on the southern portion of the site. The site is currently occupied by the single-level Perry's Cafe and a surrounding asphalt parking lot. A 4-foot to 6-foot tall retaining wall bounds the site along the Caltrans I-5/I-8 connector.

  Available historic photography indicates that the grading for the existing restaurant building and parking lot was completed between 1962 and 1964.

- *The history of design discussions for the project footprint, resulting in the final design determination.*

  NOVA has not been involved in design discussions pertaining to the project footprint. The footprint appears to maximize the available area for use as apartment units and the associated parking.

- *Full/partial infiltration BMP standard setbacks to underground utilities, structures, retaining walls, fill slopes, and natural slopes applicable to the DMA that prevent full/partial infiltration.*

  As discussed previously, based on the BMP Manual, full and partial BMPs should not be sited within existing fill soils greater than 5 feet thick. As may be seen by a review of Figure 1 and boring logs in NOVA 2022, the site is covered by fill soils greater than 5 feet in thickness.

- *The physical impairments (i.e., fire road egress, public safety considerations, etc.) that prevent full/partial infiltration.*

  The addition of stormwater into liquefiable soils is a risk to public safety.

- *The consideration of site design alternatives to achieve partial/full infiltration within the BMP.*

  Based on high groundwater, deep fills, and liquefiable soils, stormwater infiltration should not be performed at this site. There are no viable design alternatives, as these conditions are uniform across the site.

- *The extent site design BMP requirements were included in the overall design.*

  The Site Development Plan indicates that four DMAs are included in this project. Three are roof filtration systems and one is hardscape (CJC, 2022).

- *Conclusion of recommendation from the geotechnical engineer regarding the DMA's infiltration condition.*

  In conclusion, given the deep fill condition, the shallow groundwater, and the liquefiable nature of the soils, it is NOVA's opinion that the risk of geologic or geotechnical hazards cannot be reasonably mitigated to an acceptable level at the site.

- *An Exhibit for all applicable DMAs that clearly labels:*
  - *Proposed development areas and development type.*
  - *All applicable features and setbacks that prevent partial or full infiltration, including underground utilities, structures, retaining walls, fill slopes, natural slopes, and existing fill materials greater than 5 feet.*
  - *Potential locations for structural BMPs.*
  - *Areas where full/partial infiltration BMPs cannot be proposed*.

  See Plate 1 within NOVA 2022 for development areas and a cross-section of the proposed development. The development is five stories of residential apartments over one at-grade podium level with a partial subterranean parking level. Fill between 15 to 16 feet is mapped below the site, groundwater is located less than 10 feet below ground surface and the soils are liquefiable, therefore infiltration BMPs may not be proposed anywhere at this site.

CLOSURE

NOVA appreciates the opportunity to be of service to Viewpoint Development on this project. Should you have any questions regarding this letter or other matters, please contact the undersigned at 858.292.7575 x 413.

Sincerely,
NOVA Services, Inc.

John F. O'Brien

REGISTERED PROFESSIONAL ENGINEER
JOHN F. O'BRIEN
651
EXP. 3/31/23
GEOTECHNICAL
STATE OF CALIFORNIA

John F. O'Brien, PE, GE
Principal Engineer

Melissa Stayner

CERTIFIED ENGINEERING GEOLOGIST
MELISSA STAYNER
2707
STATE OF CALIFORNIA

Melissa Stayner, PG, CEG
Senior Engineering Geologist