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160 Industrial Street
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June 4, 2020
P/W 1908-04
Report No. 1908-04-B-2

Attention: **Terry Mathew**

Subject: ***Preliminary Geotechnical Investigation, Gas Station & Convenience Store, South East Corner of Twin Oaks Valley Road and Borden Road, San Marcos, California***

References: Appendix A

Gentleperson:

Pursuant to your request, Advanced Geotechnical Solutions, Inc. (AGS) has prepared this preliminary geotechnical investigation and associated design recommendations for the proposed gas station, car wash and convenience store located at the southeast corner of Borden Road and Twin Oaks Valley Road, City of San Marcos, California. In this report, AGS presents a summary of results for our recent geotechnical investigation. In addition, we discuss geologic/geotechnical issues associated with grading and development of the proposed project.

AGS appreciates the opportunity to provide you with geotechnical consulting services on this project. If you have questions concerning this report, please do not hesitate to contact the undersigned.

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ATTACHMENTS:

Preliminary Geotechnical Investigation for Gas Station & Convenience Store South East Corner of Twin Oaks Valley Road and Borden Road San Marcos, California

1.0 INTRODUCTION

1.1. Purpose and Background

This study is aimed at providing geologic and geotechnical information and recommendations for the development of the proposed gas station, car wash and convenience store located at the southeast corner of Borden Road and Twin Oaks Valley Road in the City of San Marcos, California. This report has been prepared in a manner consistent with City of San Marcos geotechnical report guidelines and the current standard of practice. Geotechnical conclusions and recommendations are presented herein. Items addressed include: 1) Unsuitable soil removals; 2) Preliminary cut, fill, and natural slope stability; 3) Cut/fill pad over-excavation criteria; 4) Remedial grading recommendations and shallow groundwater; and 5) Preliminary foundation design recommendations based upon anticipated as-graded soil conditions.

1.2. Scope of Study

This study is aimed at providing geotechnical/geologic conclusions and recommendations associated with developing the site to support the proposed gas station structures and associated improvements. The scope of this study included the following tasks:

- Review of readily available maps, literature, aerial photographs, and previous studies (Appendix A);
- Conduct field exploration consisting of six (6) exploratory test pits and two cone penetration tests (CPTs).
- Conduct laboratory testing on representative soil samples recovered from the test pits;
- Prepare a geotechnical/geologic map of the site depicting the approximate exploratory locations and distribution of geologic units;
- Prepare geologic cross sections depicting existing and proposed design grades and interpreted geologic contacts;
- Analysis and discussion of geologic/geotechnical conditions onsite, excavation characteristics of onsite materials and groundwater conditions as they relate to the Preliminary Grading Plan and proposed improvements,
- Recommendations for remedial grading and stabilization of saturated soils;
- Limited seismic hazard analysis and evaluation of seismic settlement potential;
- Preliminary foundation design recommendations based upon the anticipated site geotechnical conditions;
- Determine seismic design parameters in accordance with 2019 *California Building Code* and mapped spectral acceleration parameters (United States Geological Survey, 2019)*;*
- Determine preliminary earth pressures for retaining structures;
- Prepare preliminary pavement section recommendations; and,
- Prepare this report summarizing our findings and recommendations.

2.0 SITE DESCRIPTION

The rectangular shaped property covers approximately 1.8 acres and is bounded to the west by North Twin Oaks Valley Road, to the north by Borden Road, to the east by Twin Oaks Creek and to the south by existing commercial buildings as shown in Figure 1 - Site Location Map. The majority of the site is currently covered by a light growth of grasses and weeds with the eastern portion covered with dense chaparral and trees adjacent to the existing Twin Oaks Creek. The site can be accessed from the west by a driveway off of North Twin Oaks Valley Road. Current grades range from El. 597 above mean sea level (msl) on the northwest corner of the site to El. 588 on the eastern limits.

2.1. Geotechnical Study Limitations

The conclusions and recommendations in this report are professional opinions based on the data developed during this investigation and the current design as reflected on the 20-scale Preliminary Grading Plan prepared by Howes/Weiler/Landy Planning & Engineering (HWL) dated February 25, 2020. Pertinent geotechnical information has been superimposed on this plan and is presented as Plate 1 - Geologic Map and Exploration Location Plan. If significant changes to the grading plans occur, further review by AGS may be necessary.

The materials immediately adjacent to or beneath those observed may have different characteristics than those observed. No representations are made as to the quality or extent of material not observed. Any evaluation regarding the presence or absence of hazardous material is beyond the scope of this firm's services.

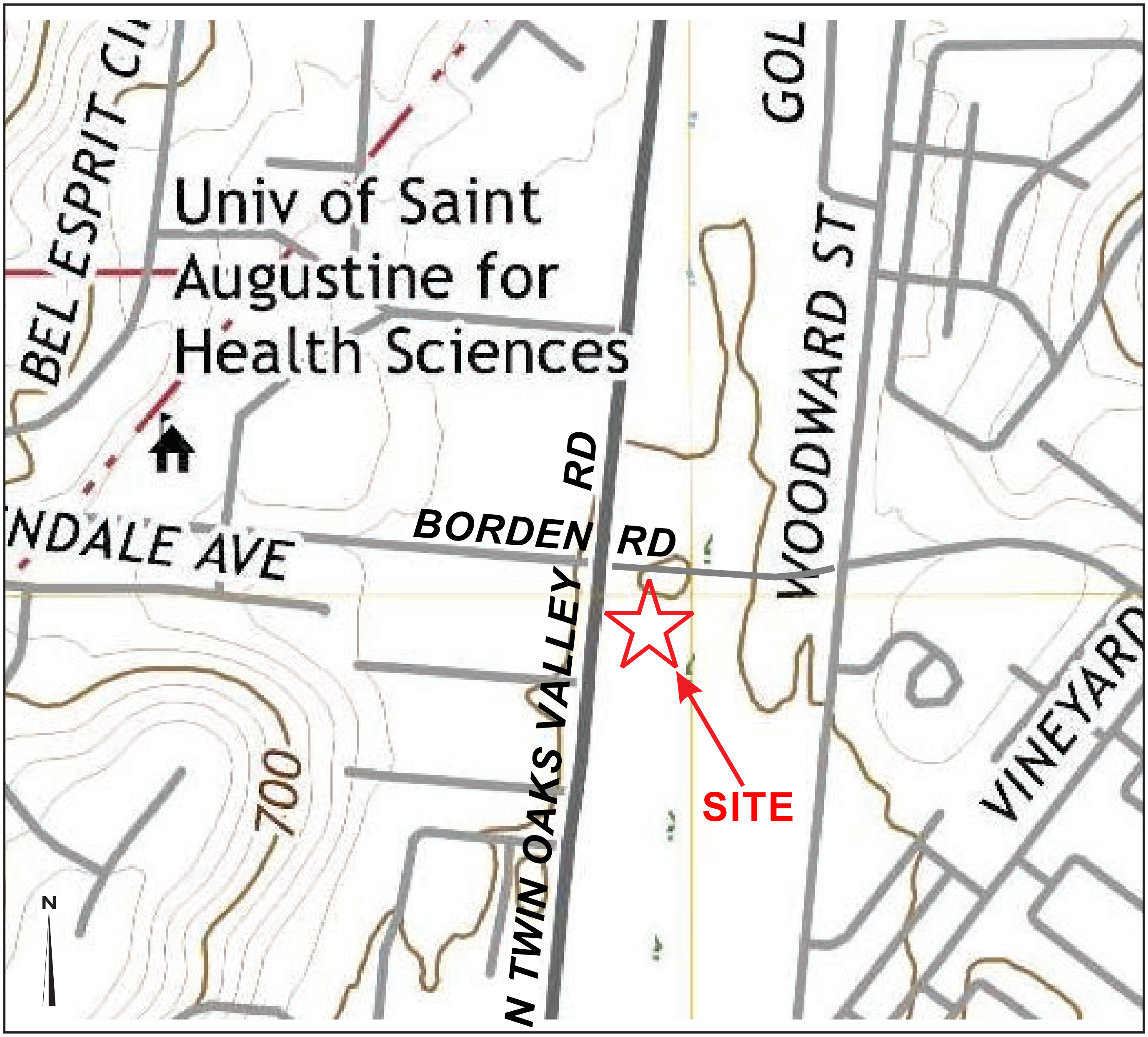
3.0 PROPOSED DEVELOPMENT

Based on our cursory review of the Preliminary Grading Plan (Plate 1), cuts and fills are expected to range from a few feet to as much as 1 to 6 feet. It is our understanding that the site will support: 1) A one-story Convenience Store at the southern portion of the property; 2) A carwash at the northern portion of the property; 3) A fuel pumping area with underground storage tanks (20K gal. Unleaded Gas; 12K gal. Leaded Gas; 12K gal. Diesel; and a 10K gal. E 85 tank) at the center of the site. Based upon the current plans, the buried tanks will range in length from 28 to 40 feet with diameters of up to 10 feet. It is anticipated that the proposed buildings will be wood-framed structures supported by conventional shallow foundation elements. Associated improvements will include: driveways and parking areas; several retaining walls ranging in height from approximately 1 to 8 (?) feet; buried "wet and dry" utilities; and hydro-modification/BMP devices to be located at the southern end of the property.

Based upon past experience with projects utilizing similar buried fuel tanks it is anticipated the proposed tanks will have a minimum of 5 to 10 feet of cover. Although detailed plans have not been developed, cuts to a maximum depth of 15 to 20 feet below design grade are anticipated where the proposed fuel tanks will be located.

4.0 FIELD AND LABORATORY INVESTIGATION

AGS conducted an initial geotechnical investigation of the project site in September 24, 2019. As part of our study, six (6) exploratory test pits were excavated within the project limits utilizing a JD 580 ExtendaHoe equipped with an 18-inch wide bucket to approximate depths ranging between 8.5 and 15.5 feet. During test pit excavation samples of the various geologic units were obtained. In addition, on May 14, 2020 AGS advanced two cone penetration tests (CPTs) to an approximate depth of 20 feet using a truck



SITE LOCATION MAP
GAS STATION - TWIN OAKS VALLEY ROAD AND BORDEN ROAD
SAN MARCOS, CALIFORNIA

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FIGURE 1

SOURCE MAP - U.S.G.S. TOPOGRAPHIC MAP OF THE SAN MARCOS 7.5 MINUTE QUADRANGLE, SAN DIEGO COUNTY, CALIFORNIA

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mounted rig. Locations of exploratory test and CPTs are shown on Plate 1 with associated subsurface logs presented in Appendix B.

AGS performed laboratory testing on representative soil samples from the test pits which included: in-situ moisture and density, hydrometer analysis, expansion index, direct shear strength, corrosivity, maximum density and optimum moisture content. Laboratory test results are presented in Appendix C.

5.0 ENGINEERING GEOLOGY

5.1. Regional Geologic and Geomorphic Setting

The subject site is situated within the Peninsular Ranges Geomorphic Province. The Peninsular Ranges province occupies the southwestern portion of California and extends southward to the southern tip of Baja California. In general, the province consists of young, steeply sloped, northwest trending mountain ranges underlain by metamorphosed Late Jurassic to Early Cretaceous-aged extrusive volcanic rock and Cretaceous-aged igneous plutonic rock of the Peninsular Ranges Batholith. The westernmost portion of the province is predominantly underlain by younger marine and non-marine sedimentary rocks. The Peninsular Ranges' dominant structural feature is northwest-southeast trending crustal blocks bounded by active faults of the San Andreas transform system.

5.2. Regional Geologic Map

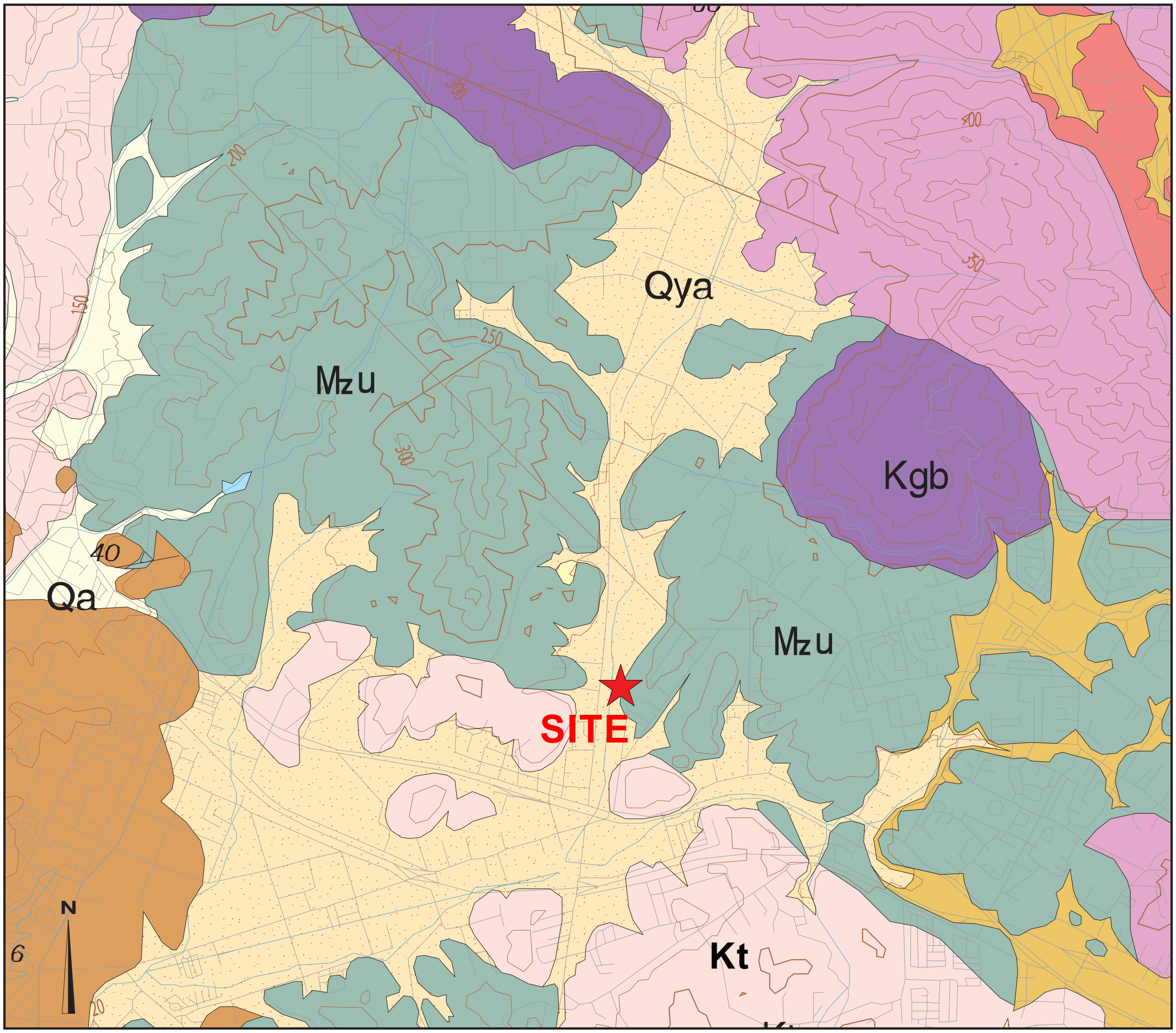
Current published regional geologic maps indicate the site is underlain by young alluvial flood plain deposits (Qya) which are subsequently underlain at depth by Cretaceous-age granitic bedrock (Mzu or Kt) as shown in Figure 2 - Geologic Map.

5.3. Site Geology and Stratigraphy

Based upon our field exploration, the site is underlain by alluvial flood plain deposits (Qal) which are underlain by Older Terrace deposits (Qt). Along the southern, western and northern boundaries of the site, relatively thin veneers of surficial undocumented fill soils locally mantle the alluvial deposits. These undocumented fills are associated with the original grading of Twin Oaks Valley Road to the west, Borden Road to the north and the existing commercial development along the southern property line. The approximate distribution of the geologic units is shown on the enclosed Plate 1 - Geologic Map and Exploration Location Plan. Geologic cross sections A-A' and B-B present the interpreted geologic profile at the site in Plate 2. The following is a brief description of each geologic unit listed from youngest to oldest.

5.3.1. Artificial Fill - Undocumented (Map Symbol afu)

Undocumented fill consists of clayey silt to silty clay and gravelly silty medium to fine grained sand, medium brown to grey in color, slightly moist, soft to loose, with sub-angular gravel to 1-inch diameter, occasional angular rock to 12 inches, and scattered wood debris present at the surface. Undocumented artificial fill soils extended to depths of 2 to 5 feet in localized areas along the north, east, and south perimeter of the property.



GEOLOGIC MAP
GAS STATION - TWIN OAKS VALLEY ROAD AND BORDEN ROAD
SAN MARCOS, CALIFORNIA

Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Kt	Tonalite, undivided (mid-Cretaceous)
Mzu	Metasedimentary and metavolcanic rocks, undivided (Mesozoic)

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FIGURE 2

SOURCE MAP - GEOLOGIC MAP OF OCEANSIDE 30'X60' QUADRANGLE, CALIFORNIA. KENNEDY & TAN 2007.

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5.3.2. Alluvial Flood Plain Deposits (Map Symbol Qal)

Alluvial deposits were encountered at the surface or underlying artificial fill in all test pits and extended to depths ranging from 8.5 to 9 feet from existing grade. In general, the upper alluvial deposits are characterized as sandy clay to silty clay, dark grey brown to tan, saturated, and soft. Deeper deposits consist of brown to gray, interbedded medium to coarse grained sand and gravel observed near the contact with the underlying Terrace Deposits. Where these coarse materials were in contact with the underlying Terrace deposits groundwater and caving soils were encountered.

5.3.3. Terrace Deposits/Older Alluvium (Map Symbol Qt)

Terrace deposits/older alluvial flood-plain deposits were encountered beneath young alluvium. The upper portion of terrace/older alluvial deposits consist of mottled grey to brown and red brown, very moist to saturated, loose, silty sand to clayey sand, and soft sandy clay to silty clay to approximate depths ranging from 9 to 14.5 feet. The underlying terrace/older alluvial deposits become medium dense to dense and very stiff at depth. Moderately hard sandy claystone was encountered at 14.5 feet depth in TP-1.

5.4. Geologic Structure and Tectonic Setting

The site is located within the Peninsular Ranges geomorphic province, which extends south into Baja California and terminates in the north against the Transverse Ranges province (Jennings, 1985). The tectonically active Elsinore Fault zone is located approximately 13 miles northeast of the proposed project. No faults have been mapped within or projecting into the site or the immediate site vicinity. Review of historic aerial photographs did not show any well-developed lineaments.

5.5. Groundwater

Groundwater was encountered onsite during our subsurface exploration at depths ranging from 6.0 feet to 11.0 feet from existing grades. For design purposes, a groundwater level at elevation 587 feet msl is recommended. It should be noted that localized perched groundwater elevations may vary at a later date, due to fluctuations in precipitation, irrigation practices, or factors not evident at the time of our field explorations.

5.6. Non-Seismic Hazards

5.6.1. Mass Wasting

No evidence of mass wasting was observed onsite nor was any noted on the reviewed maps.

5.6.2. Flooding

Based on our review of FEMA (2012) flood map, the site is located within Zone AE, a Special Flood Hazard area with Base Flood Elevations (BFE) of 594 feet to 598 feet (see Figure 3, Flood Hazard Map) during the 1% annual chance flood (100-year flood). A regulatory floodway exists along Twin Oaks Creek basin to the east of the site.

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FLOOD HAZARD MAP
GAS STATION TWIN OAKS VALLEY ROAD AND BORDEN ROAD, SAN MARCOS, CALIFORNIA

ZONE X	Area of minimal flood hazard risk
ZONE X	Area of minimal 0.2% annual chance of flood hazard risk
ZONE AE	Special flood hazard zone
	Regulatory floodway

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FIGURE 3

SOURCE MAP - F.E.M.A. (2012), NATIONAL FLOOD INSURANCE PROGRAM, CITY OF SAN MARCOS, MAP NUMBER 06073C0793G

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5.6.3. Subsidence and Ground Fissuring

Due to the presence of the relatively dense underlying Terraces deposits materials, and the lack of deep unconsolidated soils, the potential for subsidence and ground fissuring due to settlement is unlikely.

5.7. Seismic Hazards

The project is located in the tectonically active southern California and will likely experience some effects from future earthquakes. The type or severity of seismic hazards affecting the site is chiefly dependent upon the distance to the causative faults, the intensity and duration of the seismic events, and the onsite soil characteristics. The seismic hazard may be primary, such as surface rupture and/or ground shaking, or secondary, such as liquefaction or landsliding.

The following is a site-specific discussion of earthquake-induced/seismic hazards and proposed mitigations, if necessary, to reduce the hazard to an acceptable level of risk.

5.7.1. Surface Fault Rupture

No known active faults have been mapped within the project site. The nearest known active surface fault is the Rose Canyon fault (Oceanside section) approximately 13 miles west of the project site. Accordingly, the potential for fault surface rupture on the subject site is very low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

5.7.2. Seismicity

As noted, the site is within the tectonically active southern California area. The potential exists for strong ground motion that may affect future improvements. At this point in time, non-critical structures (commercial, residential, and industrial) are designed according to the guidelines of the California Building Code (2019) and the controlling local agency.

5.7.3. Liquefaction Potential

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure and causes the soil to behave as a fluid for a short period of time. Post liquefaction effects at a site can manifest in several ways, and may include ground deformations, loss of bearing strength, lateral spreading, flow failure, and dynamic settlement. Due to the presence of saturated alluvial deposits, it is our opinion that the potential for liquefaction onsite is high.

5.7.4. Dynamic Settlement

Seismic settlement can occur when loose to medium dense granular materials densify during seismic shaking and liquefaction. Seismically-induced settlement may occur in dry, unsaturated, as well as saturated soils. Liquefaction is generally known to occur in loose, saturated, relatively clean, fine-grained cohesionless soils at depths shallower than approximately 50 feet. Factors to consider in the evaluation of soil liquefaction potential include groundwater conditions, soil type, grain size distribution, relative density, degree of saturation, and both the intensity and duration of ground motion. Other phenomena associated with soil liquefaction include sand boils, ground oscillation, and loss of foundation bearing capacity.

Liquefaction analyses were performed in accordance with the National Center for Earthquake Engineering Research (NCEER) procedure by Youd et al., (2001) using the computer program CLiq v. 3.0 (Geoligismiki, 2019), and subsurface data obtained from our CPTs. The analyses considered an earthquake moment magnitude of 7.1, peak ground acceleration PGA_M of 0.50g, and anticipated historic high groundwater level at 5 feet depth. Our analyses indicate that liquefaction may occur at intermittent layers within the soil column to an approximate depth of 16 feet.

According to our analyses, the estimated dynamic settlement of liquefied soil layers ranges between 1.4 and 2.1 inches during a seismic event as shown in Appendix D. Liquefaction mitigation measures are discussed in Section 7.1 of this report.

5.7.5. Lateral Spreading

Liquefaction-induced lateral spreading is defined as the finite, lateral displacement of sloping ground as a result of pore pressure build-up or liquefaction in a shallow underlying deposit during an earthquake. Due to the sloping conditions in the vicinity of Twin Oaks Creek and the liquefaction potential, lateral spreading is considered a seismic hazard at the site. Densification of the upper portion of site soils may be used to mitigate the lateral spreading potential at the site.

5.7.6. Seiches and Tsunamis

A seiche is a free- or standing-wave oscillation on the surface of water in an enclosed or semi-enclosed basin. The wave can be initiated by an earthquake and can vary in height from several centimeters to a few meters. The potential for a seiche impacting the property is considered to be unlikely as there are no upstream large bodies of water. The potential for tsunami is negligible due to the inland location of the site.

5.8. Seismic Design Parameters

Based on our subsurface exploration, the site has been classified as Seismic Site Class D - Default consisting of a stiff soil profile with average SPT N blowcount between 15 and 50 blows per foot and assumed Vs30 of 259 m/s. Table 5.8 presents seismic design parameters in accordance with 2019 CBC and mapped spectral acceleration parameters (United States Geological Survey, 2019) utilizing site coordinates of Latitude 33.1503°N and Longitude 117.1614°W. The seismic provisions of 2019 CBC are significantly different from the previous version and require a site-specific seismic hazard analysis (SHA) for most sites located in Site Class D soil conditions.

TABLE 5.8 2019 CALIFORNIA BUILDING CODE DESIGN PARAMETERS	
Seismic Site Class	D
Mapped Spectral Acceleration Parameter at Period of 0.2-Second, S_s	0.900g
Mapped Spectral Acceleration Parameter at Period 1-Second, S_1	0.331g
Site Coefficient, F_a	1.200
Site Coefficient, F_v	N/A[3]
Adjusted MCE_R[1] Spectral Response Acceleration Parameter at Short Period, S_{MS}	1.081g
1-Second Period Adjusted MCE_R[1] Spectral Response Acceleration Parameter, S_{M1}	N/A[3]
Short Period Design Spectral Response Acceleration Parameter, S_{DS}	0.720g
1-Second Period Design Spectral Response Acceleration Parameter, S_{D1}	N/A[3]
Peak Ground Acceleration, PGA_M[2]	0.471g
Seismic Design Category	N/A[3]

Notes: [1] Risk-Targeted Maximum Considered Earthquake
[2] Peak Ground Acceleration adjusted for site effects
[3] Requires Site Specific Ground Motion Hazard Analysis per ASCE 7-16 Section 11.4.8

5.9. <u>Site Specific Ground Motion Hazard Analysis</u>

The site-specific ground motion hazard analysis was performed in accordance with Section 21.1 of ASCE Standard 7-16. Probabilistic and deterministic maximum considered earthquake (MCE) response accelerations were evaluated in order to develop the site-specific design response spectrum. The derivation of the site-specific design response spectra including the probabilistic and deterministic seismic hazard analyses are described below.

5.9.1. Probabilistic Seismic Hazard Analysis

A site-specific probabilistic seismic hazard analysis was performed to evaluate the spectral response accelerations represented by a 5-percent-damped acceleration response spectrum having a 2 percent probability of exceedance within a 50-year period. The probabilistic seismic hazard analysis was performed using the Java program OpenSHA (http://www.OpenSHA.org), developed jointly by the Southern California Earthquake Center (SCEC) and the United States Geological Survey (USGS). The probabilistic seismic hazard analyses used the next generation attenuation (NGA) relationships by Abrahamson, Silva & Kamai (2014); Boore, Stewart, Seyhan & Atkinson (2014); Campbell and Bozorgnia (2014) and Chiou and Youngs (2014). The resulting median geometric-mean acceleration response spectra were used to create a probabilistic response spectrum based on the average spectral acceleration at each period, and then converted into maximum rotated components of ground motion using applicable scale factors.

5.9.2. Deterministic Seismic Hazard Analysis

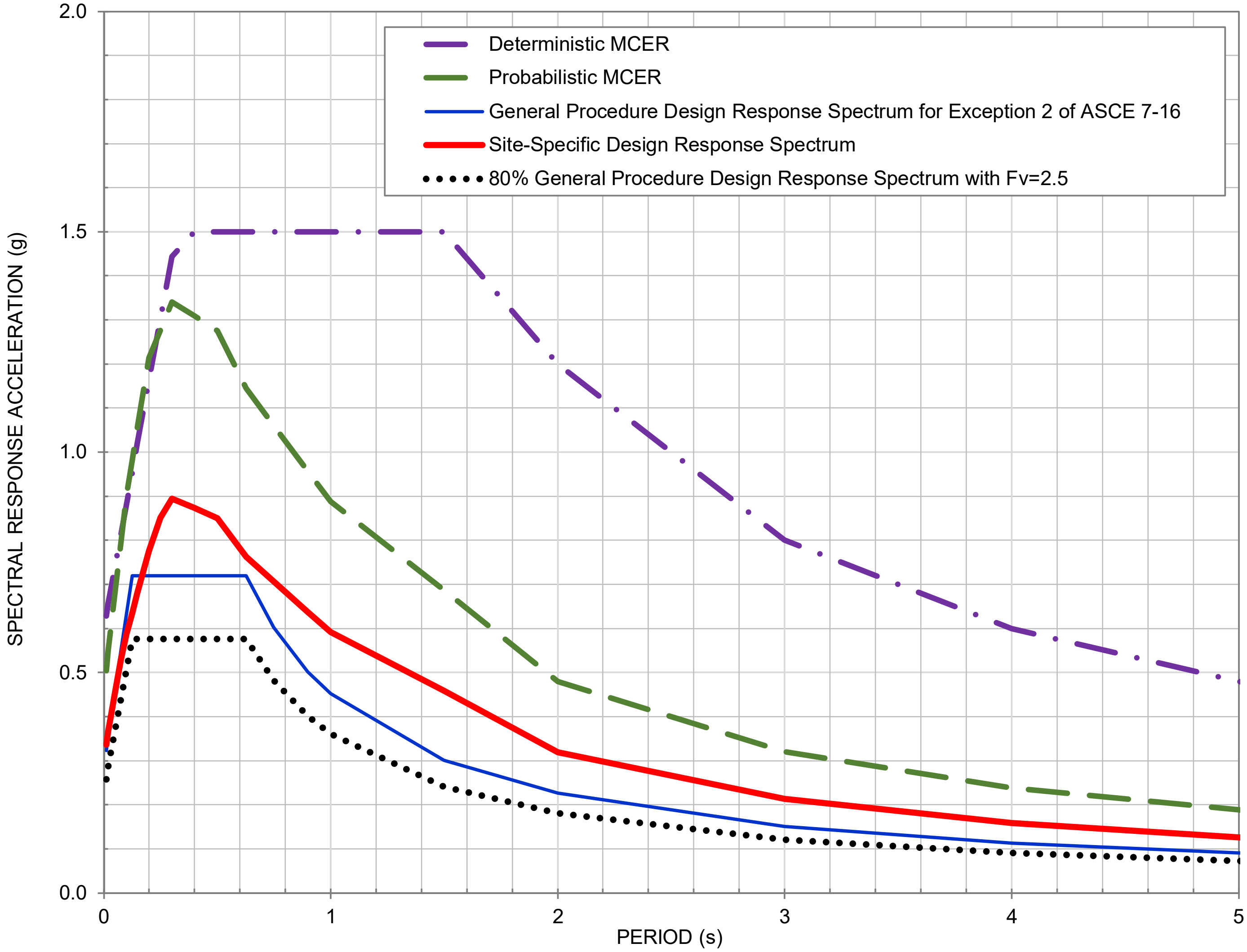
A site-specific deterministic seismic hazard analysis was performed to evaluate the MCE response acceleration. The deterministic MCE response acceleration at specified periods was calculated as the 84th percentile of the maximum rotated component of ground motion computed at each period for characteristic earthquakes on known active faults within the region. Initially we performed an evaluation of potentially damaging earthquake sources by reviewing published geologic maps and sources that contribute to the probabilistic hazard analysis, according to the deaggregation results obtained using the USGS unified hazard tool website (https://earthquake.usgs.gov/hazards/ interactive/). Based on our evaluation, we selected three “controlling” sources and seismic events: the Rose Canyon fault (Oceanside section), the Elsinore fault (Julian section), and the Elsinore fault (Temecula Section). Subsequently we used the NGA Models by Abrahamson, Silva & Kamai (2014); Boore, Stewart, Seyhan & Atkinson (2014); Campbell and Bozorgnia (2014) and Chiou and Youngs (2014) to estimate the ground motion distribution for each earthquake. The 5-percent-damped pseudo-absolute acceleration response spectrum was calculated for each earthquake using an Excel spreadsheet issued by the Pacific Earthquake Engineering Research Center (http://peer.berkeley.edu/ngawest2/ databases/). Earthquake source and site characteristic parameters were evaluated using the California Geological Survey earthquake source database and the CalTrans ARS Online web-based tool (http://dap3.dot.ca.gov/ARS_Online). Distances to faults were evaluated using the USGS unified hazard tool website. The resulting median geometric-mean acceleration response spectra were used to create a deterministic MCE response spectrum based on the greatest spectral acceleration at each period, and then converted into maximum rotated components of ground motion using applicable scale factors. The final deterministic spectral response accelerations were taken to be not lower than the deterministic lower limit as calculated using Figure 21.2-1 of ASCE 7-16, Chapter 21.

5.9.3. Site-Specific Design Response Spectrum

The site-specific MCER spectral response acceleration was calculated at each period to be the lesser of the spectral response accelerations from the probabilistic and deterministic MCE. Finally, the design spectral response acceleration at each period was calculated as two-thirds of the site-specific MCE spectral response acceleration, but not less than 80 percent of the spectral response acceleration evaluated in accordance with Section 11.4.5 of ASCE 7-16. In order to calculate the 80 percent lower limit, mapped values from USGS Seismic Design Maps (http://earthquake.usgs.gov/designmaps/us) were used to calculate SDS, SD1 and the design spectrum in accordance with Section 21.4 of ASCE 7-16. Applicable response spectra data are presented in Table 5.9.3A and on Figure 4, Site-Specific Design Response Spectrum.

PERIOD (s)	SITE-SPECIFIC DESIGN SPECTRAL ACCELERATION Sa, (g)
0.01	0.336
0.02	0.366
0.03	0.395
0.05	0.454
0.075	0.528
0.10	0.588
0.125	0.635
0.15	0.681
0.2	0.775
0.25	0.851
0.3	0.894

PERIOD (s)	SITE-SPECIFIC DESIGN SPECTRAL ACCELERATION Sa, (g)
0.4	0.873
0.5	0.850
0.627	0.763
0.8	0.707
0.9	0.638
1.0	0.592
1.5	0.458
2.0	0.319
3.0	0.214
4.0	0.159
5.0	0.125



Note: See report for values of the various curves.

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SITE-SPECIFIC DESIGN RESPONSE SPECTRUM

GAS STATION
TWIN OAKS VALLEY ROAD AND BORDEN (SE CORNER)
SAN MARCOS, CALIFORNIA

PROJECT NO.	DATE	FIGURE 4
1908-04	06/20	

TABLE 8.3.3A
SITE-SPECIFIC DESIGN RESPONSE SPECTRUM DATA

Period (sec)	General Procedure Design Response Spectrum for Exception 2 of ASCE 7-16	Risk Coeff. C_R	Site-Specific Ground Motion Analysis Spectral Accelerations (g)							
			Maximum direction 2%-in-50-yr Probabilistic Spectrum	Probabilistic MCE_R	Maximum direction 84th-percentile Deterministic Spectrum	Deterministic Lower Limit	Deterministic MCE_R	Site Specific MCE_R	80% General Procedure Design Response Spectrum with Fv=2.5	Site-Specific Design Response Spectrum
0.01	0.323	0.922	0.547	0.504	0.427	0.628	0.628	0.504	0.258	0.336
0.02	0.357	0.922	0.595	0.548	0.425	0.656	0.656	0.548	0.286	0.366
0.03	0.392	0.922	0.643	0.593	0.431	0.684	0.684	0.593	0.313	0.395
0.05	0.461	0.922	0.739	0.681	0.479	0.741	0.741	0.681	0.368	0.454
0.075	0.547	0.922	0.859	0.792	0.580	0.811	0.811	0.792	0.437	0.528
0.1	0.633	0.922	0.979	0.903	0.683	0.881	0.881	0.881	0.506	0.588
0.1254	0.720	0.922	1.065	0.981	0.785	0.953	0.953	0.953	0.576	0.635
0.15	0.720	0.922	1.148	1.058	0.839	1.022	1.022	1.022	0.576	0.681
0.2	0.720	0.922	1.317	1.213	0.950	1.163	1.163	1.163	0.576	0.775
0.25	0.720	0.922	1.385	1.277	1.033	1.303	1.303	1.277	0.576	0.851
0.3	0.720	0.922	1.455	1.341	1.097	1.444	1.444	1.341	0.576	0.894
0.4	0.720	0.922	1.420	1.310	1.122	1.500	1.500	1.310	0.576	0.873
0.5	0.720	0.922	1.383	1.275	1.101	1.500	1.500	1.275	0.576	0.850
0.6268	0.720	0.922	1.241	1.145	1.003	1.500	1.500	1.145	0.576	0.763
0.75	0.602	0.923	1.149	1.060	0.943	1.500	1.500	1.060	0.482	0.707
0.9	0.502	0.923	1.036	0.956	0.871	1.500	1.500	0.956	0.401	0.638
1	0.451	0.923	0.962	0.888	0.823	1.500	1.500	0.888	0.361	0.592
1.5	0.301	0.923	0.745	0.687	0.619	1.500	1.500	0.687	0.241	0.458
2	0.226	0.923	0.519	0.479	0.484	1.200	1.200	0.479	0.181	0.319
3	0.150	0.923	0.348	0.321	0.343	0.800	0.800	0.321	0.120	0.214
4	0.113	0.923	0.258	0.238	0.257	0.600	0.600	0.238	0.090	0.159
5	0.090	0.923	0.204	0.188	0.198	0.480	0.480	0.188	0.072	0.125

The site-specific design response parameters are provided in Table 5.9.3B. These parameters were evaluated from Design Response Spectra values presented above in accordance with ASCE 7-16 Section 21.4 guidelines.

TABLE 8.3.3B
SITE-SPECIFIC SEISMIC DESIGN PARAMETERS

Spectral Response Acceleration 0.2-second period, S_{MS}	1.207g
Spectral Response Acceleration 1-second period, S_{M1}	1.031g
Design Spectral Response Acceleration for short period, S_{DS}	0.805g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.687g
MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.497g

6.0 GEOTECHNICAL ENGINEERING

Presented herein is a general discussion of the geotechnical properties of the various soil types and the analytic methods used in this report.

6.1. Material Properties

6.1.1. Excavation Characteristics

It is anticipated that excavation of surficial soils, undocumented artificial fills and upper portion of alluvium can likely be conducted with conventional grading equipment. Excavations in the vicinity and below the groundwater level may not be feasible due to soft and caving condition of the soil.

6.1.2. Compressibility

Onsite materials that are significantly compressible in their current condition include undocumented fill, topsoil, young alluvial deposits and upper portion of terrace/older alluvial deposits. These materials will require complete removal prior to placement of fill, where exposed at design grade. Recommended removal depths are presented in Section 7.1 and earthwork adjustment estimates are presented in Section 6.1.9.

6.1.3. Collapse Potential/Hydro-Consolidation

Given the relatively thin veneer of undocumented fill soils, the clayey nature of the formational materials and the removals proposed herein, the potential for hydro-consolidation is considered to be “very low”.

6.1.4. Expansion Potential

Based upon our observations and preliminary testing, the expansion potential of the onsite materials will range from "very low" to "medium" when classified in accordance with ASTM D4829.

6.1.5. Shear Strength

Based upon our familiarity with similar projects and the onsite geologic units, AGS has summarized the recommended shear strengths in Table 6.1.5 for compacted fill and the various geologic units onsite.

TABLE 6.1.5 SHEAR STRENGTH		
Material	**Cohesion (psf)**	**Friction Angle (degrees)**
Artificial Fill - Compacted (afc)	100	31
Competent Terrace Deposits/Older Alluvium (Qt)	200	28

6.1.6. Concrete Mix Design

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical deterioration. Laboratory testing was not conducted as the site will ultimately require imported soils to achieve design grade. Accordingly, additional sulfate testing should be conducted prior to the importation of these materials. It is recommended that all import soils should have sulfate content testing prior to importation to verify that these materials are sulfate exposure Class S0 – Not Applicable (sulfate content below 0.1%) per ACI 318 (2014).

6.1.7. Earthwork Adjustments

In consideration of the proposed grading to develop the project as currently shown on the 20-scale Preliminary Grading Plan, the following average earthwork adjustment factors presented in Table 6.1.7 have been formulated for use in the earthwork design of the project.

TABLE 6.1.7 EARTHWORK ADJUSTMENTS	
Geologic Unit (Map Symbol)	**Adjustment Factor**
Undocumented Fill (afu), Topsoil and Alluvial Deposits (Qal)	10% - 15% Shrink
Terrace Deposits/Older Alluvium (Qt)	0% - 5% Shrink

These values may be used in an effort to balance the earthwork quantities. As is the case with every project, contingencies should be made to adjust the earthwork balance when grading is in progress and actual conditions are better defined.

6.2. Bearing Capacity and Lateral Earth Pressures

Ultimate bearing capacity values were obtained using the graphs and formulas presented in NAVFAC DM-7.1. Allowable bearing was determined by applying a factor of safety of at least three (3) to the ultimate bearing capacity.

Static lateral earth pressures were calculated using Rankine methods for active and passive cases. If it is desired to use Coulomb forces, a separate analysis specific to the application can be conducted.

6.3. Pavement Support Characteristics

It is anticipated that the onsite soils will have poor to moderate support characteristics. Depending upon the final distribution of site soils, pavement support characteristics could vary. If structural pavements are to be constructed (concrete or asphaltic concrete), an "R"-value of 10 can be utilized for the preliminary design of pavements. Final pavement design and subgrade stabilization recommendations should be based upon site observations and representative sampling of as-graded soils.

7.0 GRADING RECOMMENDATIONS

Development of the subject property as proposed is considered feasible, from a geotechnical standpoint, provided that the conclusions and recommendations presented herein are incorporated into the design and construction of the project. Presented below are issues identified by this study or previous studies as possibly impacting site development. Recommendations to mitigate these issues and geotechnical recommendations for use in planning and design are presented in the following sections of this report.

7.1. Site Preparation

Site preparation should begin with the removal of existing structures, utility lines, asphalt, concrete, and other deleterious debris from areas to be graded. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. Debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dumpsite away from the project area. Abandoned utilities should be removed and/or backfilled with slurry in accordance with local regulations.

7.2. Unsuitable Soil Removals

Undocumented fill material and loose to medium dense Old Paralic deposits are not considered suitable for large structural loads in their present condition. According to our liquefaction analysis, densification of loose soils to an approximate depth of 16 feet will be required under settlement sensitive structures and improvements such as retaining walls. Due to the presence of groundwater at approximate depth of 6 feet, removal and recompaction of loose soils is not considered feasible. Ground improvement alternatives are discussed below.

7.3. Ground Improvement

Ground improvement construction techniques can be used to replace, densify, or solidify in-situ soils to increase liquefaction resistance, reduce static compressibility, and increase strength. Common ground improvement techniques that are considered potentially technically suitable for this site include (listed in order of lowest to highest relative cost) surcharge loading, vibro-replacement stone columns and compaction grouting. These techniques and their applicability to the project are discussed below.

7.3.1. Surcharge Loading

Surcharge loading may be used to densify soils in areas where other densification techniques are not applied. Surcharge loads are usually applied by placing an embankment fill on the soft soil area. The underlying soft soils are monitored with settlement monuments to evaluate the time required to achieve primary consolidation settlement.

7.3.2. Vibro-Replacement Stone Columns

Vibro-replacement stone columns is a densification and reinforcement technique wherein a vibratory probe ("vibroflot") is advanced vertically into the ground. As the probe advances, it displaces and densifies the soil laterally. After the probe has reached its intended depth, gravel is introduced from the probe tip or from the ground surface as the probe is withdrawn. The probe is reinserted in 1- or 2-foot (0.3- to 0.6-meter) increments as it is withdrawn to further compact the gravel and surrounding soil. Vibro methods are most commonly used for liquefaction mitigation in sandy to silty material. In addition to the densification of the native soil, the stone columns can act as drains to assist in relieving pore water pressure buildup during earthquake shaking.

7.3.3. Compaction Grouting

Compaction grouting is a densification and reinforcement technique that consists of injecting low slump mortar into soil under relatively low pressure. The grout expands in a bulb against the surrounding soil causing densification and displacement of the soil around the grout bulbs. The grout tube is advanced into the ground by drilling and/or vibrating. The probe is raised incrementally, and successive, adjacent grout bulbs are constructed, resulting in a compaction grout column. A triangular or square array of columns results in a composite mass of improved ground composed of the grout columns and densified native soil between the columns. The strength of the overall soil mass increases due to the increased density of the soil between the grout bulbs, and the reinforcement of the soil mass by the grout columns. Compaction grouting is applicable to a wide range of soils including sands, silts, and clays.

Compaction grouting can be performed with low overhead equipment that could operate within the height constraints at the site. Compaction grouting does not involve vibrations, and thus typically does not result in settlement of adjacent ground. Near the ground surface, compaction grouting can result in ground heave. However, this is typically controlled by active survey monitoring during the grouting process. Ground heave threshold levels are set beforehand (depending upon the sensitivity of existing improvements) that trigger cessation of the grouting if they are reached.

7.4. Slope Stability and Remediation

At this time, it is our understanding that cut and fill slopes will be designed at 2:1 slope ratios. Close geologic inspection should be conducted during grading to observe if soil and geologic conditions differ significantly from those anticipated. Should field conditions dictate, modifications to the recommendations presented herein may be necessary and should be based upon conditions exposed in the field during grading activities.

7.5. Excavation and Temporary Cut Slopes

All excavations should be shored or laid back in accordance with applicable Cal-OSHA standards. Topsoil/alluvium is considered Type "C" soil. Any temporary excavation greater than 5 feet in depth should be laid back at the appropriate slope ratio. These excavations should not become saturated or allowed to dry out. Surcharge loads should not be permitted within a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 10 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 10 feet from an existing surface improvement should be temporarily shored in accordance with applicable OSHA codes and regulations.

7.6. Dewatering

Dewatering may be necessary to accomplish deeper excavations for utilities, tanks and foundation elements. Dewatering can create subsidence outside of the area of work and create distress to adjacent improvements. Adjacent improvements should be inventoried prior to dewatering and observed periodically to determine if the dewatering is creating settlement outside of the work area. Discharge of groundwater generated during the dewatering process will require a discharge permit in accordance with NPDES permits. Accordingly, water testing and possible treatment of the discharge water will be necessary.

7.7. Earthwork Considerations

7.7.1. Compaction Standards

Fill and processed natural ground shall be compacted to a minimum relative compaction of 90 percent as determined by ASTM Test Method: D 1557. Compaction shall be achieved at slightly above the optimum moisture content.

7.7.2. Mixing and Moisture Control

In order to prevent layering of different soil types and/or different moisture contents, mixing and moisture control of materials may be necessary. The preparation of the earth materials through mixing and moisture control should be accomplished prior to and as part of the compaction of each fill lift. Water trucks or other water delivery means may be necessary for moisture control. Discing may be required when either excessively dry or wet materials are encountered.

7.7.3. Oversize Rock

Oversized rock material [i.e., rock fragments greater than eight (8) inches] may be produced during grading. Provided that the procedure is acceptable to the owner and governing agency, this rock may be incorporated into the compacted fill section to within three (3) feet of finish grade and to two (2) foot below the deepest utility and buried fuel storage tanks. Variances to the above rock hold-down must be approved by the owner, geotechnical consultant and governing agencies.

7.7.4. Utility Trench Excavation and Backfill

All utility trenches should be shored or laid back in accordance with applicable Cal/OSHA standards. Excavations in bedrock areas should be made in consideration of underlying geologic structure. The geotechnical consultant should be consulted on these issues during construction.

Mainline and lateral utility trench backfill should be compacted to at least 90 percent of maximum dry density as determined by ASTM D 1557. Onsite soils will not be suitable for use as bedding material but will be suitable for use in backfill, provided oversized materials are removed. No surcharge loads should be imposed above excavations. This includes spoil piles, lumber, concrete trucks or other construction materials and equipment. Drainage above excavations should be directed away from the banks. Care should be taken to avoid saturation of the soils.

Compaction should be accomplished by mechanical means. Jetting of native soils will not be acceptable. To reduce moisture penetration beneath the slab-on-grade areas, shallow utility trenches should be backfilled with lean concrete or concrete slurry where they intercept the foundation perimeter. As an alternative, such excavations can be backfilled with native soils, moisture-conditioned to over optimum, and compacted to a minimum of 90 percent relative compaction.

8.0 DESIGN RECOMMENDATIONS

8.1. Structural Design

Detailed foundation plans are not currently available; however, it is our understanding that the proposed structures and retaining walls will be supported by conventional shallow foundation systems placed on densified and/or reinforced soil. It is anticipated that the majority of the onsite soils will generally vary from "Very Low" to "Medium" in expansion potential when tested in general accordance with ASTM D 4829.

8.1.1. Foundation Design

Gas station structures can be supported on conventional shallow foundation systems. The design of foundation systems should be based on as-graded conditions as determined after grading completion. The following values may be used in preliminary foundation design:

Allowable Bearing:	2,500 psf
Lateral Bearing:	300 lbs./sq.ft. at a depth of 12 inches plus 150 lbs./sq.ft. for each additional 12 inches embedment to a maximum of 2500 lbs./sq.ft.
Sliding Coefficient:	0.35
Settlement:	Total = 3/4 inch
Differential:	3/8 inch in 20 feet

The above values may be increased as allowed by Code to resist transient loads such as wind or seismic. Building code and structural design considerations may govern. Depth and reinforcement requirements and should be evaluated by a qualified engineer.

8.1.2. Conventional Slab Recommendations

Based upon the anticipated lot categories and preliminary expansion potential of "Very Low" to "Medium" for the onsite soil conditions and information supplied by the CBC 2019, conventional foundation systems should be designed in accordance with Section 8.1.1 and Table 8.1.2.

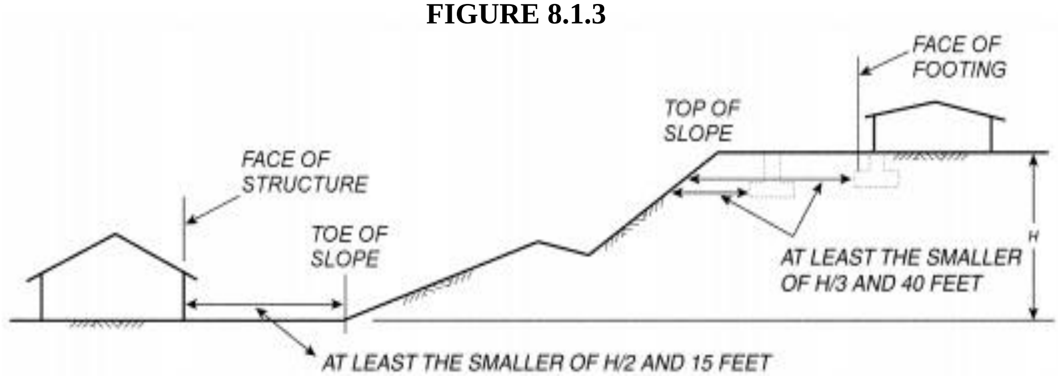
TABLE 8.1.2
CONVENTIONAL SLAB ON GRADE FOUNDATION DESIGN RECOMMENDATIONS

	CAR WASH	CONVENIENCE STORE
Expansion Potential	Low to Medium	Low to Medium
Embedment (One-story)	22 inches	18 inches
Footing Width	12 inches	12 inches
Footing Reinforcement	No. 4 rebar, two (2) on top and two (2) on bottom or No. 5 rebar one (1) on top and one (1) on bottom	
Slab Thickness	10 inches (actual)	4 inches (actual)
Slab Reinforcement	No. 4 rebar spaced 12 inches on center, each way	No. 3 rebar spaced 18 inches on center, each way or 10x10 welded wire mesh
Slab Underlayment	Stego Wrap Vapor Barrier (or equivalent) (15mil) in moisture sensitive areas	
Slab Subgrade Moisture	Minimum of 110% of optimum moisture 24 hours prior to placing concrete.	
Footing Embedment Next to Swales and Slopes	If exterior footings adjacent to drainage swales are to exist within five (5) feet horizontally of the swale, the footing should be embedded sufficiently to assure embedment below the swale bottom is maintained. Footings adjacent to slopes should be embedded such that a least seven (7) feet are provided horizontally from edge of the footing to the face of the slope.	
Isolated Spread Footings	Isolated spread footings should be embedded a minimum of 18 inches below lowest adjacent finish grade and should at least 24 inches wide. A grade beam should also be constructed for interior and exterior spread footings and should be tied into the structure in two orthogonal directions, footing dimensions and reinforcement should be similar to the aforementioned continuous footing recommendations. Final depth, width and reinforcement should be determined by the structural engineer.	

8.1.3. Deepened Footings and Structural Setbacks

It is generally recognized that improvements constructed in proximity to natural slopes or properly-constructed, manufactured slopes can, over a period of time, be affected by natural processes including gravity forces, weathering of surficial soils, and long-term (secondary) settlement. Most building codes, including the 2019CBC, require that structures be set back or footings deepened, where subject to the influence of these natural processes. For the subject site, where foundations for structures are to exist in proximity to slopes, the footings should be embedded to satisfy the requirements presented in Figure 8.1.3.

FIGURE 8.1.3



8.1.4. Miscellaneous Foundation Design Recommendations

Soils from the footing excavations should not be placed in slab-on-grade areas unless properly compacted and tested. The excavations should be cleaned of all loose/sloughed materials and be neatly trimmed at the time of concrete placement.

8.1.5. Earth Pressures for Design of Buried Structures

The recommended active, passive and at rest earth Rankine earth pressures for artificial compacted fills, which may be utilized for design of buried structures with level and 2:1 backfill are as follows:

Level Backfill	**Rankine Coefficients**	**Equivalent Fluid Pressure (psf/lin.ft.)**
Coefficient of Active Pressure:	$K_a = 0.32$	38
Coefficient of Passive Pressure:	$K_p = 3.12$	375
Coefficient of At Rest Pressure:	$K_o = 0.48$	58

2:1 Backfill	**Rankine Coefficients**	**Equivalent Fluid Pressure (psf/lin.ft.)**
Coefficient of Active Pressure:	$K_a = 0.50$	60
Coefficient of Passive Pressure: (Descending)	$K_p = 1.18$	142
Coefficient of At Rest Pressure:	$K_o = 0.88$	105

For rigid restrained walls it is recommended that “At-Rest” values be used. For cantilever retaining walls which can undergo minor rotations active pressures can be used.

The above values may be increased by 1/3 as allowed by Code to resist transient loads. Building Code and structural design considerations may govern.

In addition to the above static pressures, unrestrained retaining walls should be designed to resist seismic loading as required by the 2019 CBC. The seismic load can be modeled as a thrust load applied at a point 0.6H above the base of the wall, where H is equal to the height of the wall.

The seismic load (in pounds per lineal foot of wall) is represented by the following equation:

$$Pe = \tfrac{3}{8} * \gamma * H^2 * k_h$$

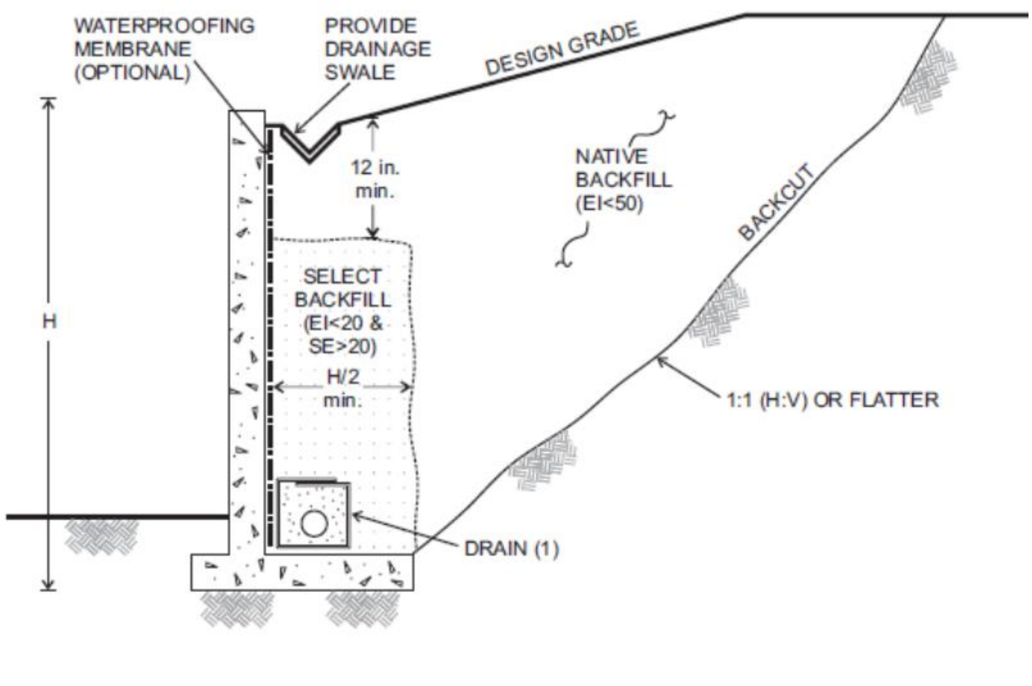
Where:	Pe	=	Seismic thrust load
	H	=	Height of the wall (feet)
	γ	=	soil density = 120 pounds per cubic foot (pcf)
	k_h	=	seismic pseudostatic coefficient = 0.5 * PGA_M

Walls should be designed to resist the combined effects of static pressures and the above seismic thrust load.

8.1.6. Retaining Wall Backfill and Drainage

Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic pressures. To relieve the potential for hydrostatic pressure wall backfill should consist of a free draining soil (sand equivalent "SE" >20) and a heel drain should be constructed (see Figure 8.1.6). The heel drain should consist of a 4-inch diameter perforated pipe (SDR35 or SCHD 40) surrounded by 4 cubic feet of crushed rock (3/4-inch) per lineal foot, wrapped in filter fabric (Mirafi® 140N) or approved equivalent.

FIGURE 8.1.6
Retaining Wall Backfill and Drainage



NOTES: (1) DRAIN: 4-INCH PERFORATED ABS OR PVC PIPE OR APPROVED EQUIVALENT SUBSTITUTE PLACED PERFORATIONS DOWN AND SURROUNDED BY A MINIMUM OF 1 CUBIC FEET OF 3/4 INCH ROCK OR APPROVED EQUIVALENT SUBSTITUTE AND WRAPPED IN MIRAFI 140 FILTER FABRIC OR APPROVED EQUIVALENT SUBSTITUTE

Proper drainage devices should be installed along the top of the wall backfill, which should be properly sloped to prevent surface water ponding adjacent to the wall In addition to the wall drainage system, for building perimeter walls extending below the finished grade, the wall should be waterproofed and/or damp-proofed to effectively seal the wall from moisture infiltration through the wall section to the interior wall face.

The wall should be backfilled with granular soils placed in loose lifts no greater than 8-inches thick, at or near optimum moisture content, and mechanically compacted to a minimum 90 percent of the maximum dry density as determined by ASTM D1557. Flooding or jetting of backfill materials generally do not result in the required degree and uniformity of compaction and, therefore, is not recommended. No backfill should be placed against concrete until minimum design strengths are achieved as verified by compression tests of cylinders. The geotechnical consultant should observe the retaining wall footings, back drain installation, and be present during placement of the wall backfill to confirm that the walls are properly backfilled and compacted.

8.2. Civil Design Recommendations

8.2.1. Drainage

Final site grading should assure positive drainage away from structures, and positive drainage away from structures should be maintained. The use of gutters and down spouts to carry roof drainage well away from structures is recommended. Planter areas should be provided with area drains to transmit irrigation and rain water away from structures. Raised planters should be provided with a positive means to remove water through the face of the containment wall.

8.2.2. Infiltration

An infiltration feasibility study was not performed in conjunction with this geotechnical investigation. According to our observations it is our opinion that onsite infiltration is not recommended due to the shallow depth to groundwater, and the fine-grained clayey nature of the onsite soils. Based on the anticipated as-graded conditions it is our professional opinion that proposed storm water BMPs should be designed for a *no infiltration condition*.

8.2.3. Concrete Flatwork and Lot Improvements

- In an effort to minimize shrinkage cracking, concrete flatwork should be constructed of uniformly cured, low-slump concrete and should contain sufficient control/contraction joints (typically spaced at 8 to 10 feet, maximum).
- Concrete flatwork should be designed utilizing 4-inch minimum thickness.
- Consideration should be given to reinforcing any exterior flatwork.
- Consideration should be given to construct a thickened edge (scoop footing) at the perimeter of slabs and walkways adjacent to landscape areas to minimize moisture variation below these improvements. The thickened edge (scoop footing) should extend approximately 8 inches below concrete slabs and should be a minimum of 6 inches wide.

- Additional provisions need to be incorporated into the design and construction of all improvements exterior to the proposed structures (pools, spas, walls, patios, walkways, planters, etc.) to account for the hillside nature of the project, as well as being designed to account for potential expansive soil conditions. Design considerations on any given lot may need to include provisions for differential bearing materials (bedrock vs. compacted fill), ascending/descending slope conditions, bedrock structure, perched (irrigation) water, special surcharge loading conditions, potential expansive soil pressure, and differential settlement/heave.

- All exterior improvements should be designed and constructed by qualified professionals using appropriate design methodologies that account for the onsite soils and geologic conditions. The aforementioned considerations should be used when designing, constructing, and evaluating long-term performance of the exterior improvements on the lots.

8.2.4. Preliminary Pavement Design

For preliminary design and estimating purposes, the following pavement structural sections can be used for the range of likely traffic indices. The structural sections are based upon an assumed "R"-Value of 10.

TABLE 8.4.4
PRELIMINARY PAVEMENT SECTIONS

Traffic Type	Traffic Index (TI)	Asphaltic Concrete AC (inch)	Class II Aggregate Base AB (inch)
Auto (Light)	5.0	3.0	9.0
Truck Traffic	6.0	4.0	11.0

It is recommended that the Portland cement concrete (PCC) pavement section consist of 6-inch thick PCC with a flexural strength of 650 psi placed over compacted subgrade.

If soft subgrade areas are exposed during grading, these areas can be remediated by removing the upper 1 to 2 feet and replacing with gravel/rock and geogrid reinforcement. Subgrade soils should be compacted to at least 95 percent of maximum density as determined by ASTM D-1557. Aggregate base materials should be compacted to at least 95 percent of maximum density as determined by California Test 216.

Final pavement design and subgrade stabilization recommendations should be based upon site conditions and representative sampling of as-graded soils in accordance with City of San Marcos guidelines.

8.2.5. Buried Fuel Tanks

It is our understanding that the four buried fuel tanks (Unleaded Gas 20,000 gal, Leaded Gas 12,000 gal, Diesel 10,000 gal, and the 10,000 gal E85) could be embedded as deep as 18 feet from finished grade. It is anticipated that these tanks will be installed below the existing groundwater level. For design, it is recommended to use groundwater elevation of

587 feet msl. It should be anticipated that underpinning with holddown piers will likely be necessary to resist the potential uplift forces.

9.0 FUTURE STUDY NEEDS

This report represents a geotechnical review of the 20-scale Site Plan. As the project design progresses, additional site specific geologic and geotechnical issues may need to be considered in the ultimate design and construction of the project. Consequently, future geotechnical studies and reviews may be necessary. These may include:

- Review of final Grading Plans
- Review of Foundation plans
- Review of Retaining Wall plans

As plans are refined, they should be forwarded to the project geotechnical engineer/geologist for evaluation and comment, as necessary.

10.0 CLOSURE

10.1. Geotechnical Review

As is the case in any grading project, multiple working hypotheses are established utilizing the available data, and the most probable model is used for the analysis. Information collected during the grading and construction operations is intended to evaluate the hypotheses, and some of the assumptions summarized herein may need to be changed as more information becomes available. Some modification of the grading and construction recommendations may become necessary, should the conditions encountered in the field differ significantly than those hypothesized to exist.

AGS should review the pertinent plans and sections of the project specifications, to evaluate conformance with the intent of the recommendations contained in this report.

If the project description or final design varies from that described in this report, AGS must be consulted regarding the applicability of, and the necessity for, any revisions to the recommendations presented herein. AGS accepts no liability for any use of its recommendations if the project description or final design varies and AGS is not consulted regarding the changes.

10.2. Limitations

This report is based on the project as described and the information obtained from our investigation and the referenced reports. The findings are based on the review of the field and laboratory data provided combined with an interpolation and extrapolation of conditions between and beyond the reviewed exploratory excavations. The results reflect an interpretation of the direct evidence obtained. Services performed by AGS have been conducted in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions. No other representation, either expressed or implied, and no warranty or guarantee is included or intended.

The recommendations presented in this report are based on the assumption that an appropriate level of field review will be provided by geotechnical engineers and engineering geologists who are familiar with the design and site geologic conditions. That field review shall be sufficient to confirm that geotechnical and geologic conditions exposed during grading are consistent with the geologic representations and corresponding recommendations presented in this report. AGS should be notified of any pertinent changes in the project plans or if subsurface conditions are found to vary from those described herein. Such changes or variations may require a re-evaluation of the recommendations contained in this report.

The data, opinions, and recommendations of this report are applicable to the specific design of this project as discussed in this report. They have no applicability to any other project or to any other location, and any and all subsequent users accept any and all liability resulting from any use or reuse of the data, opinions, and recommendations without the prior written consent of AGS.

AGS has no responsibility for construction means, methods, techniques, sequences, or procedures, or for safety precautions or programs in connection with the construction, for the acts or omissions of the CONTRACTOR, or any other person performing any of the construction, or for the failure of any of them to carry out the construction in accordance with the final design drawings and specifications.

APPENDIX A

REFERENCES

REFERENCES

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APPENDIX B

SUBSURFACE EXPLORATION

APPENDIX B

SUBSURFACE EXPLORATION

Test Pits

AGS representatives observed the excavation of six test pits with a Case 580 Extenda Hoe equipped with an 18” bucket. The approximate locations of the test pits are shown on Plate 1, and the test pit logs are attached.

Representative bulk soil samples were obtained from the test pits at selected depths. The bulk samples were transported to AGS laboratory for testing. Laboratory testing procedures and test results are presented in Appendix C of this report.

CPT Soundings

Kehoe Testing and Engineering performed two CPT soundings to a maximum depth of approximately 20 feet below existing grade. The soil conditions encountered during the field investigation were automatically logged in a continuous profile of penetration resistance as each CPT sounding was being conducted. The recorded tip stress, sleeve stress, and pore pressure of the soil was used to develop a stratigraphic interpretation of the soil profile. CPT data provided by Kehoe is presented in this Appendix.

Date Excavated: 9/24/19
Logged by: JAC
Equipment: Case 580 Extenda Hoe With 18" bucket

LOG OF TEST PITS

Depth (ft.)	USCS	Description
		TP-1
0.0 - 5.0	SM	**Artificial Fill –Undocumented (afu):** Gravelly Silty Sand, medium to fine grained; medium brown, slightly moist, soft to loose, sub-angular rock to 1-inch diameter
5.0 - 8.5	ML/CL	**Alluvium (Qal):** Silty to Sandy Clay, fine grained, greyish tan, moist to very moist, soft (Bulk @ 5.0 ft.) @ 7.5 ft., tan to red brown, moist to wet, soft, micaceous (Small Bulk @ 8.5 ft.)
8.5 - 15.5	CL	**Terrace Deposits (Qt):** Sandy Clay, brown to red brown, very moist to saturated, soft, caving @ 14.5 ft. Medium to Fine grained Sandy Claystone, mottled dark brown to brown, moist to very moist, soft to moderately hard (Small Bulk @ 14.5ft.) Total Depth: 15.5 ft. Groundwater @ 11 ft. after 10 minutes
		TP-2
0.0 - 1.0	ML/CL	**Topsoil** Silty to Sandy Clay, fine grained, dark brown, slightly moist, soft
1.0 - 9.0	CL/SC	**Alluvium (Qal):** Sandy Clay to Clayey Sand, fine grained, greyish tan, moist to very moist, soft (Bulk @ 1 - 4 ft.) @ 3.0 ft. very moist @ 6.0 ft. groundwater
	SP	@ 8.0 ft. medium to coarse grained Sand, red brown to dark brown, saturated, loose to medium dense, caving
9.0 - 9.5	CL/SC	**Terrace Deposits (Qt):** Medium to Coarse grained Sandy Clay to Clayey Sand, mottled red/brown, moist to very moist, medium dense to firm (Small Bulk @ 9.5 ft) Total Depth: 9.5 ft Groundwater @ 9 ft. after 10 minutes Caving 8 to 9 ft.

LOG OF TEST PITS

Depth (ft.)	USCS	Description
		TP- 3
0.0 - 9.0	SM	**Alluvium (Qal):** Silty Sand, medium to fine grained; dark brown, moist, loose. @ 2.5 ft. becoming moist,
	SP	@ 8.5 ft. Gravelly Sand, medium brown to brown, moist to very moist, soft, occasional sub-angular gravel 1/3 to ¼ inch @8.5 ft Medium to coarse grained Gravelly Sand, medium brown, saturated, moderately dense to loose, caving from 8.5 to 9 ft.
9.0 - 10.5	SM/SC	**Terrace Deposits** Fine grained Silty Sand to Clayey Sand, mottled red brown to grey, moist, dense Total Depth: 10.5 ft. Groundwater @ 8.5 ft. Caving 8 to 8.5 ft.
		TP-4
0.0 - 8.5	SM	**Alluvium (Qal):** Silty Sand, medium to fine grained; dark brown, slightly moist, loose. @ 2.5 ft. moist
8.5 - 9.0	GP	@ 8.5. to 9.0 ft. Medium to coarse grained Sandy Gravel, brown, saturated moist, moderately dense, occasional sub-angular gravel 1/3 to ¼ inch
9.0 - 10.5	SM/SC	**Terrace Deposits (Qt):** Silty Sand to Clayey Sand, mottled red brown and gray saturated, dense, caving Total Depth: 10.5 ft. Groundwater @ 8.5 ft. Caving 9 to 10.5 ft.

LOG OF TEST PITS

Depth (ft.)	USCS	Description
		TP-5
0.0 –9.0	CL	**Alluvium (Qal):** Fine grained Sandy Clay to Silty Clay; dark brown, slightly moist, soft @ 5.0 ft. dark grey brown, saturated, soft
	SP	@ 6.0 ft. to 6.5 ft. Medium to coarse grained Sand with Gravel, brown, saturated, moderately dense, sub-angular gravel from 1/3 to ¼ inch diameter, caving
	CL	@6.5 ft. Sandy Clay to Silty Clay, dark grey brown, saturated, soft
9.0- 10.5	SM/SC	**Terrace Deposits (Qt):** Silty Sand to Clayey Sand, mottled red brown and gray, moist to very moist, medium dense
		Total Depth: 10.5 ft Groundwater @ 6.0 ft.
		TP-6
0.0 –2.0	ML-CL	**Undocumented Fill (afu):** Clayey Silt to Silty Clay; light grey, dry, soft, occasional angular rock to 12 inches, with scattered wood debris.
2.0-9.0	CL	**Alluvium (Qal):** Silty Clay; dark brown to black, dry, soft to firm. @ 4.5 saturated, soft @ 6.0 Medium to coarse grained Sand, grey, saturated, loose, caving
9.0- 10.5	CL/SC	**Terrace Deposits (Qt):** Sandy Clay to Clayey Sand, mottled red brown and gray, moist to very moist, soft to firm.
		Total Depth: 10.5 ft Groundwater @ 6.0 ft.

SUMMARY

OF

CONE PENETRATION TEST DATA

Project:

W. Borden Avenue & N. Twin Oaks Valley Road
San Marcos, CA
May 14, 2020

Prepared for:

Mr. Andres Bernal
Advanced Geotechnical Solutions, Inc. (AGS)
485 Corporate Drive, Ste B
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Prepared by:

KTE

KEHOE TESTING & ENGINEERING
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www.kehoetesting.com

TABLE OF CONTENTS

SUMMARY

OF

CONE PENETRATION TEST DATA

1. INTRODUCTION

This report presents the results of a Cone Penetration Test (CPT) program carried out for the project located at W. Borden Avenue & N. Twin Oaks Valley Road in San Marcos, California. The work was performed by Kehoe Testing & Engineering (KTE) on May 14, 2020. The scope of work was performed as directed by Advanced Geotechnical Solutions, Inc. (AGS) personnel.

2. SUMMARY OF FIELD WORK

The fieldwork consisted of performing CPT soundings at two locations to determine the soil lithology. A summary is provided in **TABLE 2.1**.

LOCATION	DEPTH OF CPT (ft)	COMMENTS/NOTES:
CPT-1	20	
CPT-2	20	

TABLE 2.1 - Summary of CPT Soundings

3. FIELD EQUIPMENT & PROCEDURES

The CPT soundings were carried out by **KTE** using an integrated electronic cone system manufactured by Vertek. The CPT soundings were performed in accordance with ASTM standards (D5778). The cone penetrometers were pushed using a 30-ton CPT rig. The cone used during the program was a 15 cm^2 cone and recorded the following parameters at approximately 2.5 cm depth intervals:

- Cone Resistance (qc)
- Sleeve Friction (fs)
- Dynamic Pore Pressure (u)
- Inclination
- Penetration Speed

The above parameters were recorded and viewed in real time using a laptop computer. Data is stored at the KTE office for up to 2 years for future analysis and reference. A complete set of baseline readings was taken prior to each sounding to determine temperature shifts and any zero load offsets. Monitoring base line readings ensures that the cone electronics are operating properly.

4. CONE PENETRATION TEST DATA & INTERPRETATION

The Cone Penetration Test data is presented in graphical form in the attached Appendix. These plots were generated using the CPeT-IT program. Penetration depths are referenced to ground surface. The soil behavior type on the CPT plots is derived from the attached CPT SBT plot (Robertson, “Interpretation of Cone Penetration Test…”, 2009) and presents major soil lithologic changes. The stratigraphic interpretation is based on relationships between cone resistance (qc), sleeve friction (fs), and penetration pore pressure (u). The friction ratio (Rf), which is sleeve friction divided by cone resistance, is a calculated parameter that is used along with cone resistance to infer soil behavior type. Generally, cohesive soils (clays) have high friction ratios, low cone resistance and generate excess pore water pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate little (or negative) excess pore water pressures.

The CPT data files have also been provided. These files can be imported in CPeT-IT (software by GeoLogismiki) and other programs to calculate various geotechnical parameters.

It should be noted that it is not always possible to clearly identify a soil type based on qc, fs and u. In these situations, experience, judgement and an assessment of the pore pressure data should be used to infer the soil behavior type.

If you have any questions regarding this information, please do not hesitate to call our office at (714) 901-7270.

Sincerely,

KEHOE TESTING & ENGINEERING

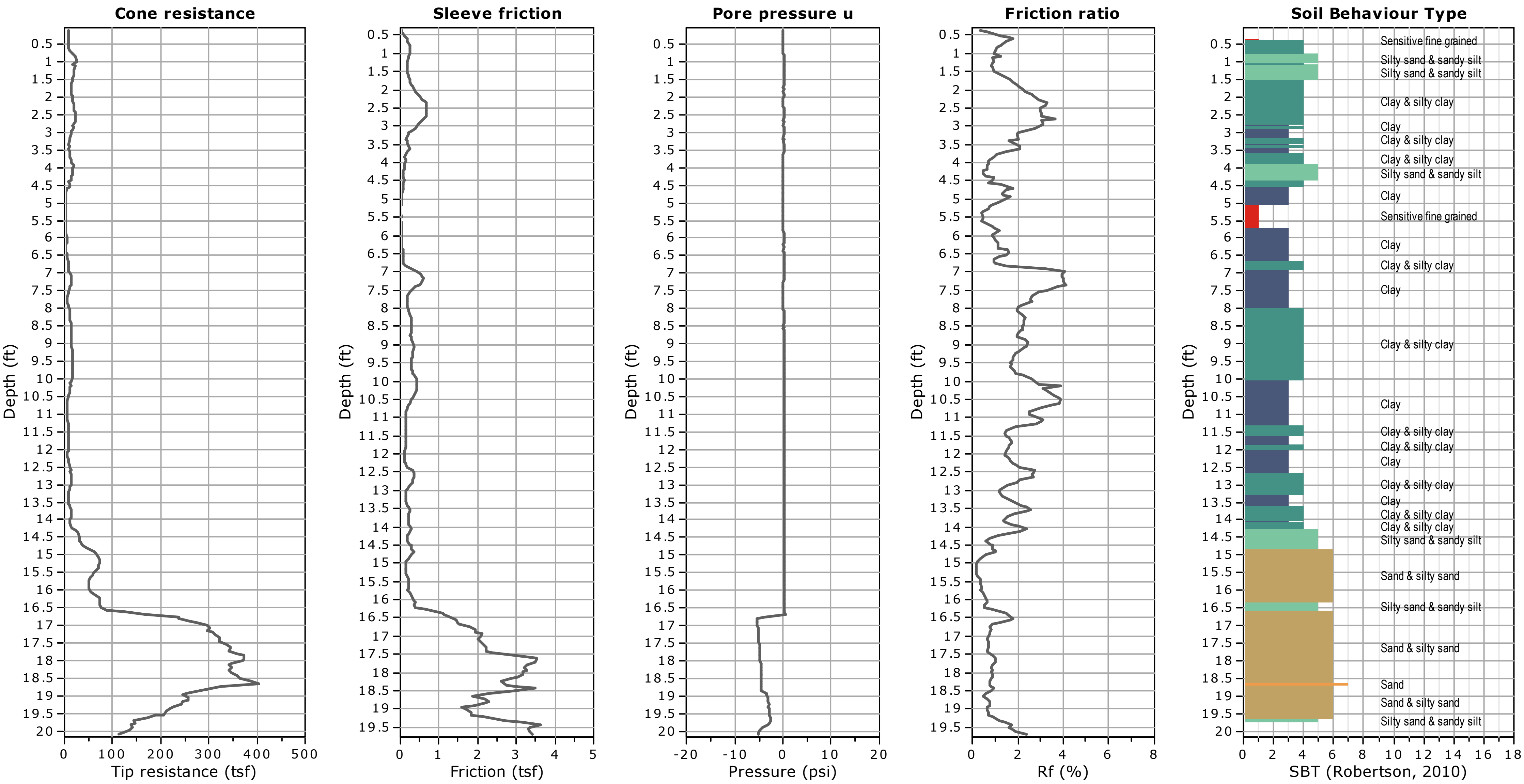
Steven P. Kehoe
President

05/18/20-wt-1643-2

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Advanced Geotech **CPT-1**

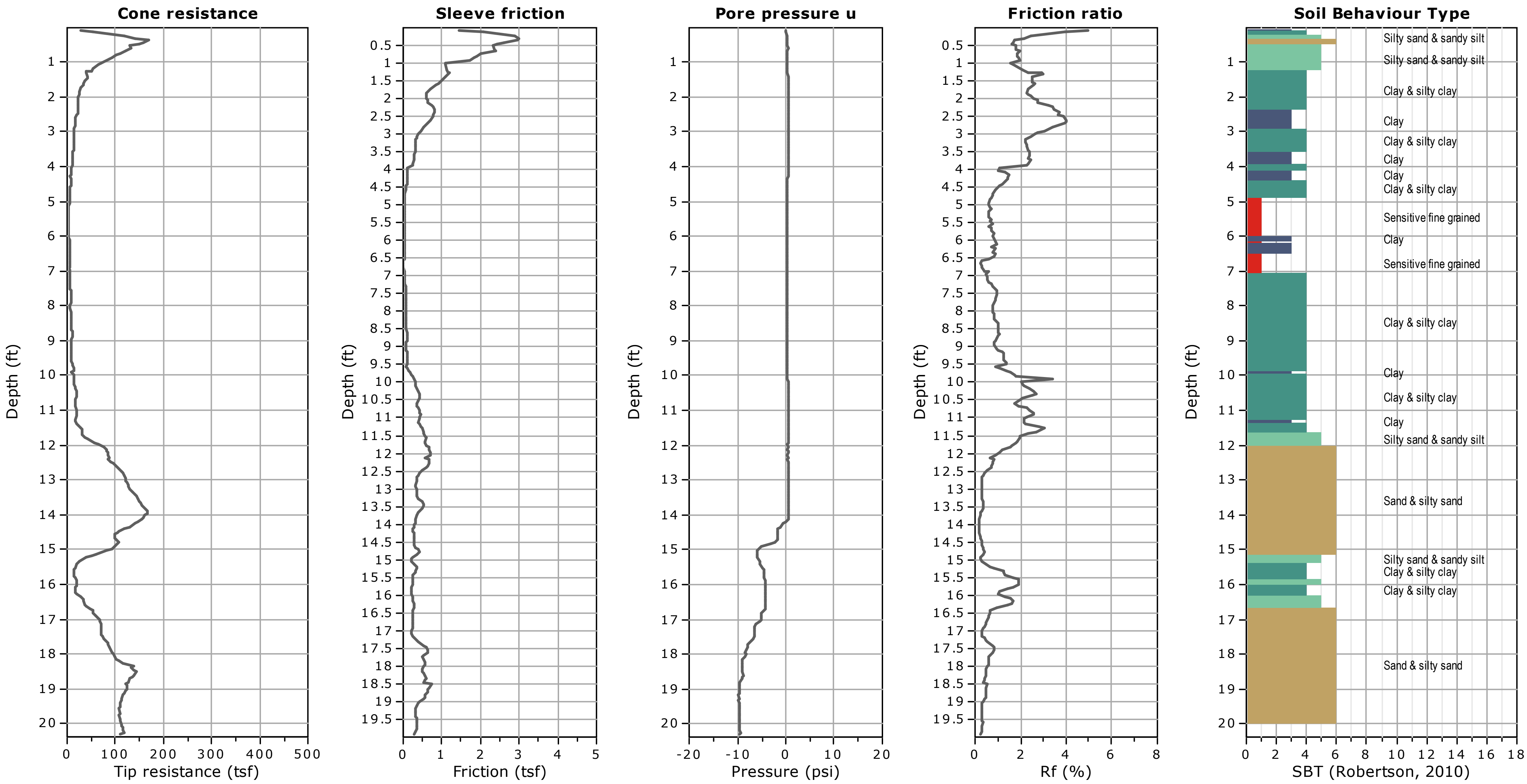
Location: Borden Ave & Twin Oaks Valley Rd, San Marcos, CA Total depth: 20.09 ft, Date: 5/14/2020



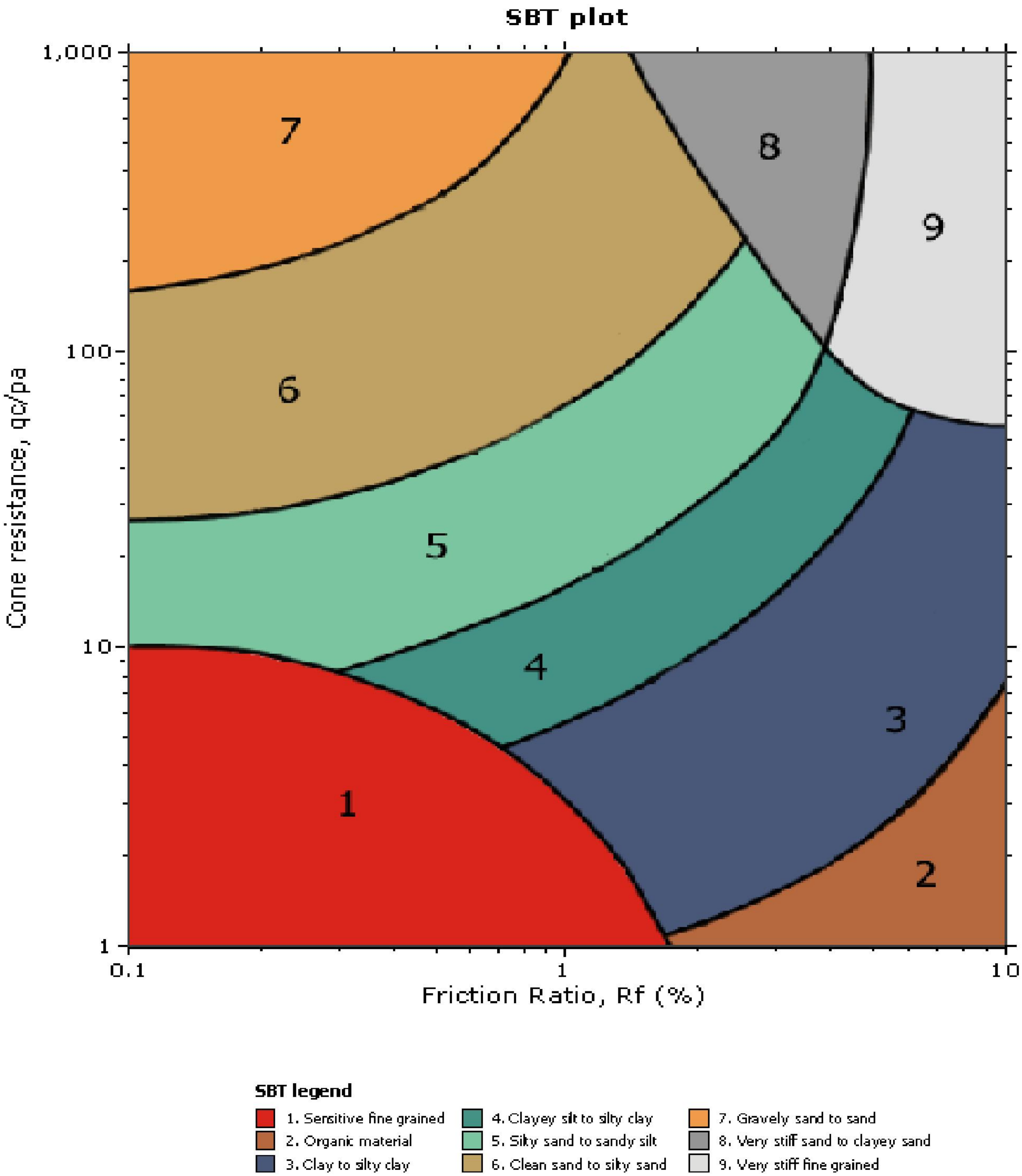
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Advanced Geotech
Location: Borden Ave & Twin Oaks Valley Rd, San Marcos, CA

CPT-2
Total depth: 20.30 ft, Date: 5/14/2020



Kehoe Testing & Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com



APPENDIX C

LABORATORY TEST RESULTS

APPENDIX C
LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D2488. Soil classifications are indicated on the test pit logs in Appendix B.

Expansion Index

The expansion index of selected materials was evaluated in general accordance with ASTM D4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (±1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure C-1.

Hydrometer

The gradation of soil samples was evaluated by the hydrometer analysis in general accordance with ASTM D 7928. The results are presented on the boring logs and on Figures C-2 and C-3.

Modified Proctor Density

The maximum dry density and optimum moisture content of a selected representative soil sample was evaluated using the Modified Proctor method in general accordance with ASTM D1557. The results of these tests are summarized on Figure C-4.

Direct Shear

Direct shear tests were performed on remolded samples in general accordance with ASTM D3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figure C-5.

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ADVANCED GEOTECHNICAL SOLUTIONS, INC.

EXPANSION INDEX - ASTM D4829

AGS FORM E-6

Project Name:	CCI/Namou Group	Excavation/Tract:	TP-2
Location:	San Marcos	Depth/Lot:	1-4 ft
P/W:	1908-04	Description:	Brown SC
Date:	10/2/19	Tested by:	FV
		Checked by:	AB

Expansion Index - ASTM D4829	
Initial Dry Density (pcf):	87.1
Initial Moisture Content (%):	17.3
Initial Saturation (%):	50.1
Final Dry Density (pcf):	86.1
Final Moisture Content (%):	35.3
Final Saturation (%):	99.4
Expansion Index:	13
Potential Expansion:	Very Low

ASTM D4829 - Table 5.3	
Expansion Index	Potential Expansion
0 - 20	Very Low
21 - 50	Low
51 - 90	Medium
91 - 130	High
>130	Very High

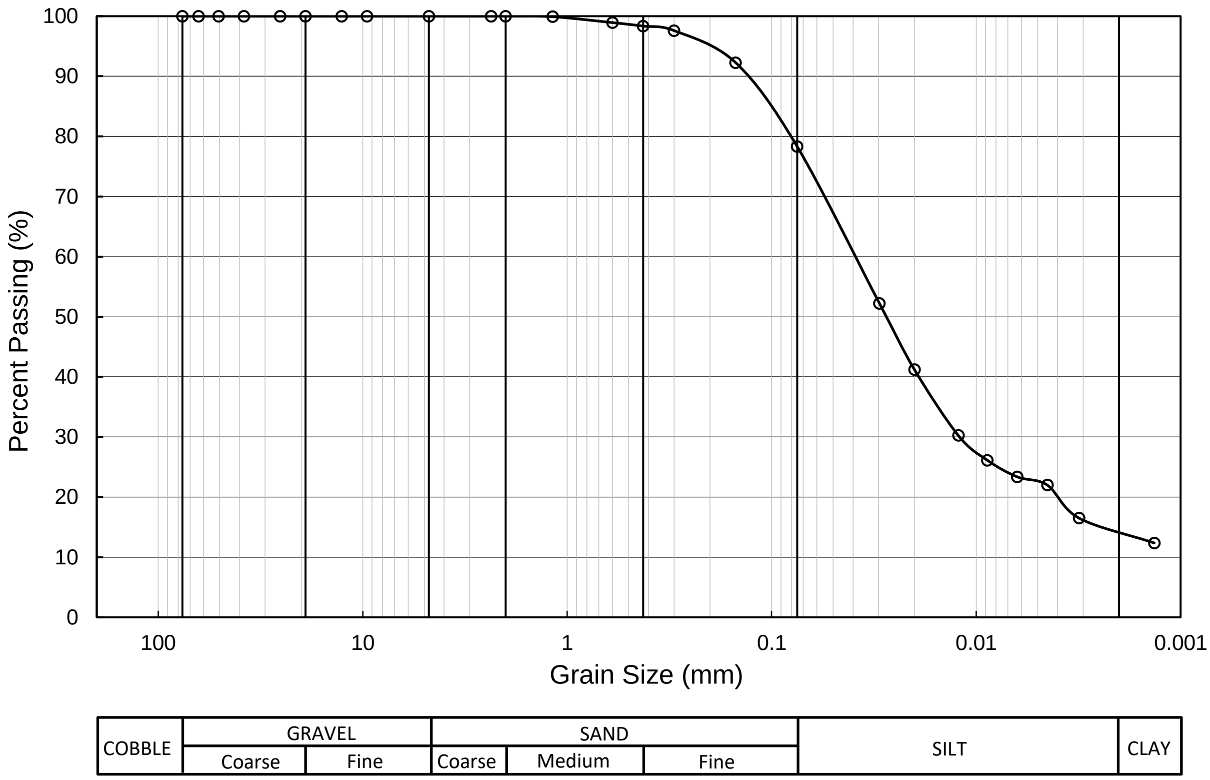
FIGURE C-1

ADVANCED GEOTECHNICAL SOLUTIONS, INC.

PARTICLE SIZE ANALYSIS - ASTM D422

Project Name: CCI/Namou Group
Location: Borden Road, San Marcos
Project No.: 1903-04
Date: 10/2/2019

Excavation: TP-1
Depth: 8.5 ft
Tested by: FV
Checked by: AB



Grain Size (in/#)	Grain Size (mm)	Amount Passing (%)
3 "	76.20	100
2 1/2 "	63.50	100
2 "	50.80	100
1 1/2 "	38.10	100
1 "	25.40	100
3/4 "	19.05	100
1/2 "	12.70	100
3/8 "	9.53	100
# 4	4.75	100
# 8	2.36	100
#10	2.00	100
#16	1.18	100
# 30	0.60	98.9
# 40	0.425	98.4
# 50	0.30	97.6
# 100	0.15	92.3
# 200	0.075	78.3
Hydro	0.0297	52.2
Hydro	0.0201	41.2
Hydro	0.0122	30.2
Hydro	0.0088	26.1
Hydro	0.0063	23.4
Hydro	0.0045	22.0
Hydro	0.0031	16.5
Hydro	0.0013	12.4

Summary	
% Gravel =	0.0
% Sand =	21.7
% Fines =	78.3
Sum =	100.0

Soil Type: CL

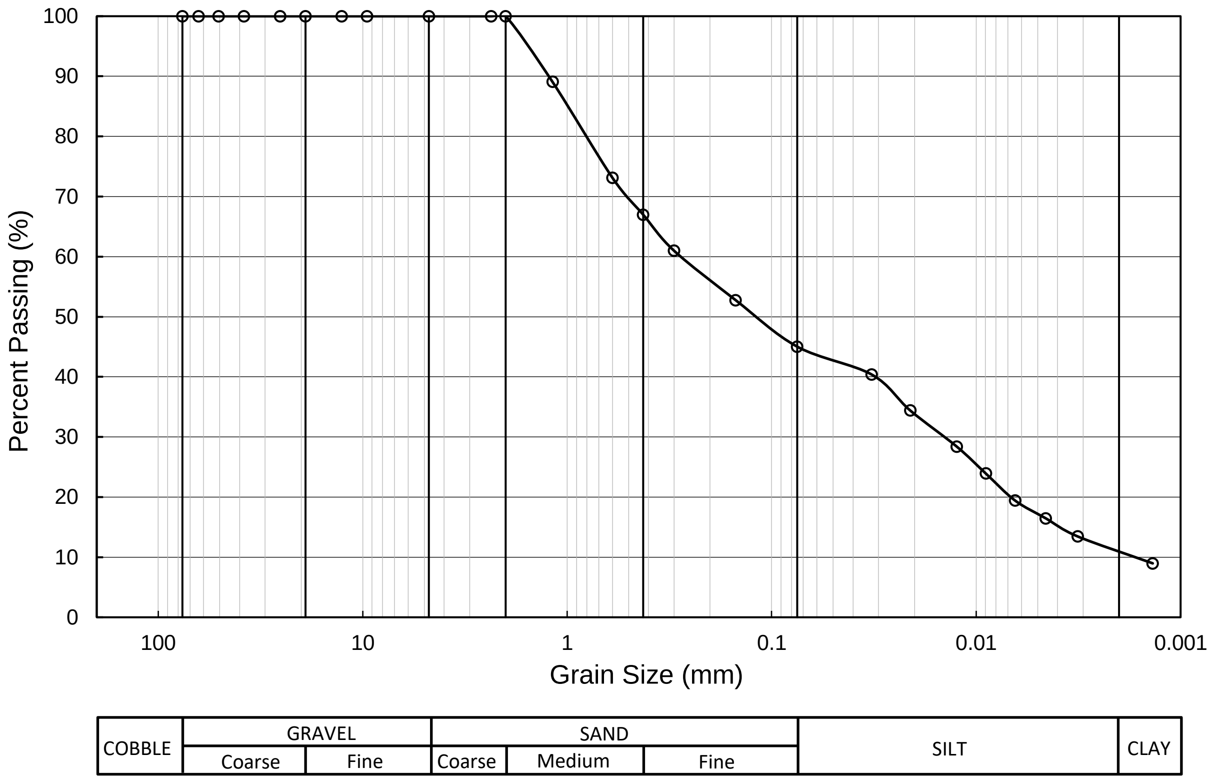
FIGURE C-2

ADVANCED GEOTECHNICAL SOLUTIONS, INC.

PARTICLE SIZE ANALYSIS - ASTM D422

Project Name: CCI/Namun Group
Location: Borden Road, San Marcos
Project No.: 1908-04
Date: 10/2/2019

Excavation: TP-2
Depth: 9.5 ft
Tested by: FV
Checked by: AB



Grain Size (in/#)	Grain Size (mm)	Amount Passing (%)
3 "	76.20	100
2 1/2 "	63.50	100
2 "	50.80	100
1 1/2 "	38.10	100
1 "	25.40	100
3/4 "	19.05	100
1/2 "	12.70	100
3/8 "	9.53	100
# 4	4.75	100
# 8	2.36	100
#10	2.00	100
#16	1.18	89
# 30	0.60	73.1
# 40	0.425	67.0
# 50	0.30	61.0
# 100	0.15	52.8
# 200	0.075	45.0
Hydro	0.0324	40.4
Hydro	0.0210	34.4
Hydro	0.0125	28.4
Hydro	0.0090	23.9
Hydro	0.0065	19.4
Hydro	0.0046	16.5
Hydro	0.0032	13.5
Hydro	0.0014	9.0

Summary	
% Gravel =	0.0
% Sand =	55.0
% Fines =	45.0
Sum =	100.0

Soil Type: SC/CL

FIGURE C-3

ADVANCED GEOTECHNICAL SOLUTIONS, INC.

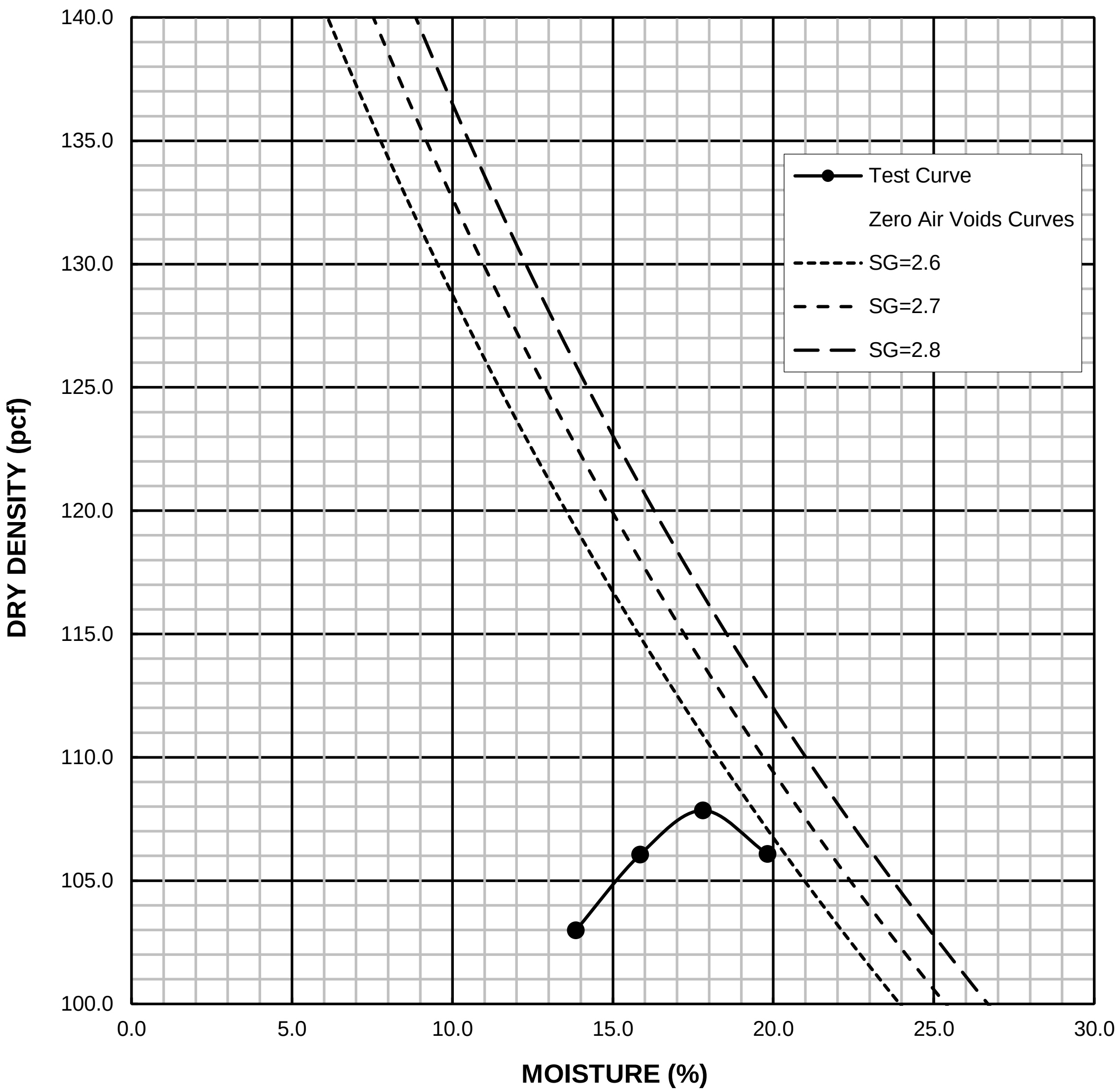
MAXIMUM DENSITY - ASTM D1557

AGS FORM E-8

Project Name:	CCI/Namon Group	Excavation:	TP-1
Location:	Borden Road, San Marcos	Depth:	5 ft
P/W No.:	1908-04	Soil Type:	ML/CL
Date:	10-2019	Tested by:	FV
		Checked by:	AB

Method:	A	Oversize Retained: 0 %		
Point No.	1	2	3	4
Dry Density (pcf)	103.0	106.1	107.8	106.1
Moisture Content (%)	13.8	15.9	17.8	19.8

MAXIMUM DENSITY CURVE



Corrected Max. Dry Density	**107.8 pcf**	Corrected Moisture	**17.8**	%
Max. Dry Density	**107.8 pcf**	Optimum Moisture	**17.8**	%

FIGURE C-4

ADVANCED GEOTECHNICAL SOLUTIONS, INC.

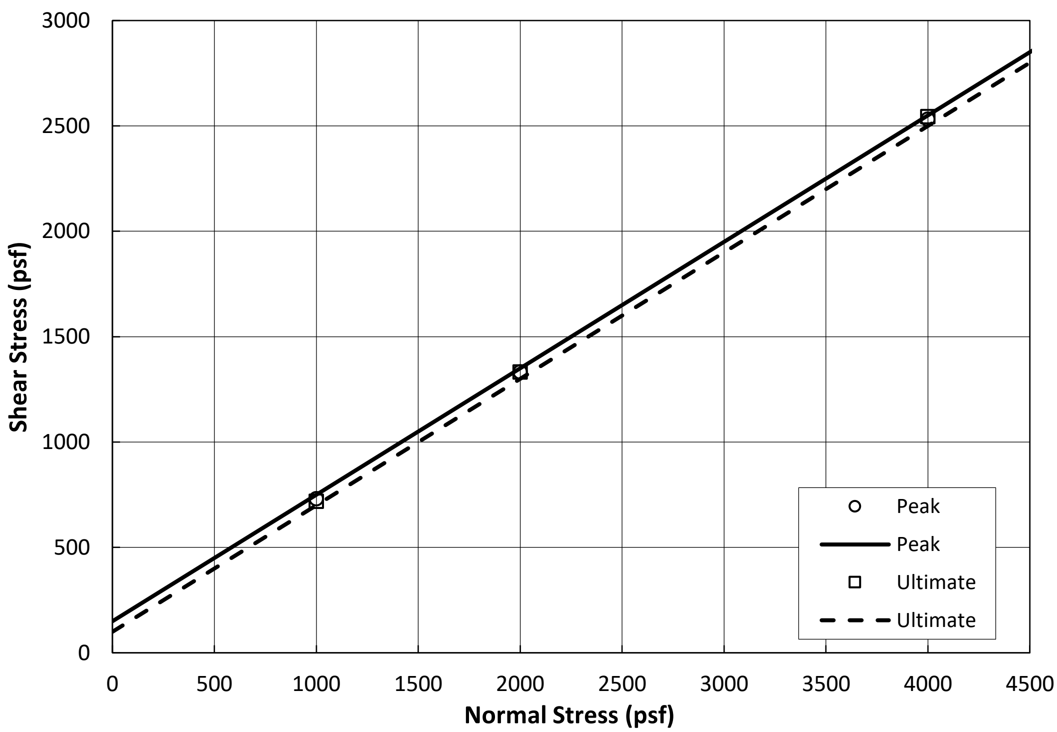
DIRECT SHEAR - ASTM D3080

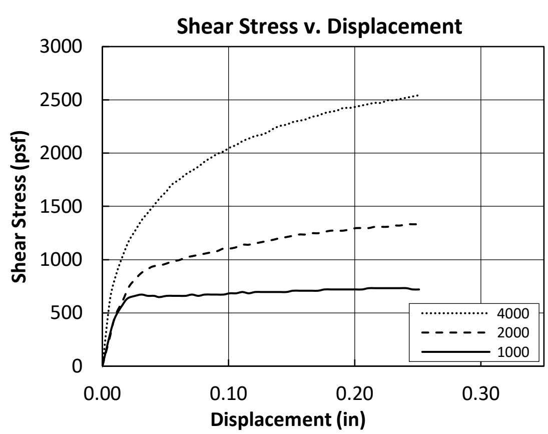
Project Name:	CCI/Namon Group	Excavation:	TP-2
Location:	San Diego	Depth:	5 ft
Project No.:	1908-04	Tested by:	FV
Date:	10/2/2019	Reviewed by:	AB

Samples Tested	1	2	3
Intial Moisture (%)	17.5	17.5	17.5
Initial Dry Density (pcf)	97.2	97.2	97.2
Normal Stress (psf)	1000	2000	4000
Peak Shear Stress (psf)	732	1332	2532
Ult. Shear Stress (psf)	720	1332	2544

Soil Type:	SC
Test:	Remolded 90%
Method:	Drained
Consolidation:	Yes
Saturation:	Yes
Shear Rate ($^{in}/_{min}$):	0.01

Strength Parameters	Peak	Ultimate
Friction Angle, phi (deg)	31	31
Cohesion (psf)	150	100





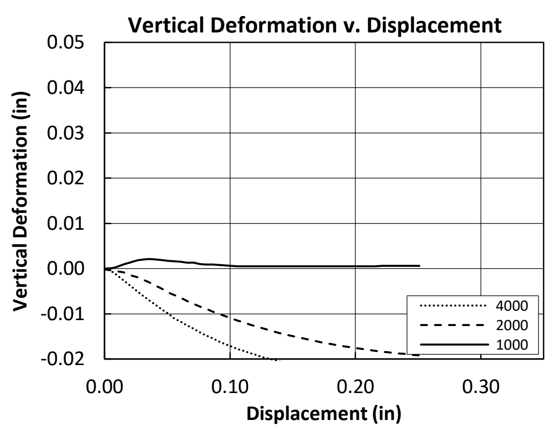


FIGURE C-5

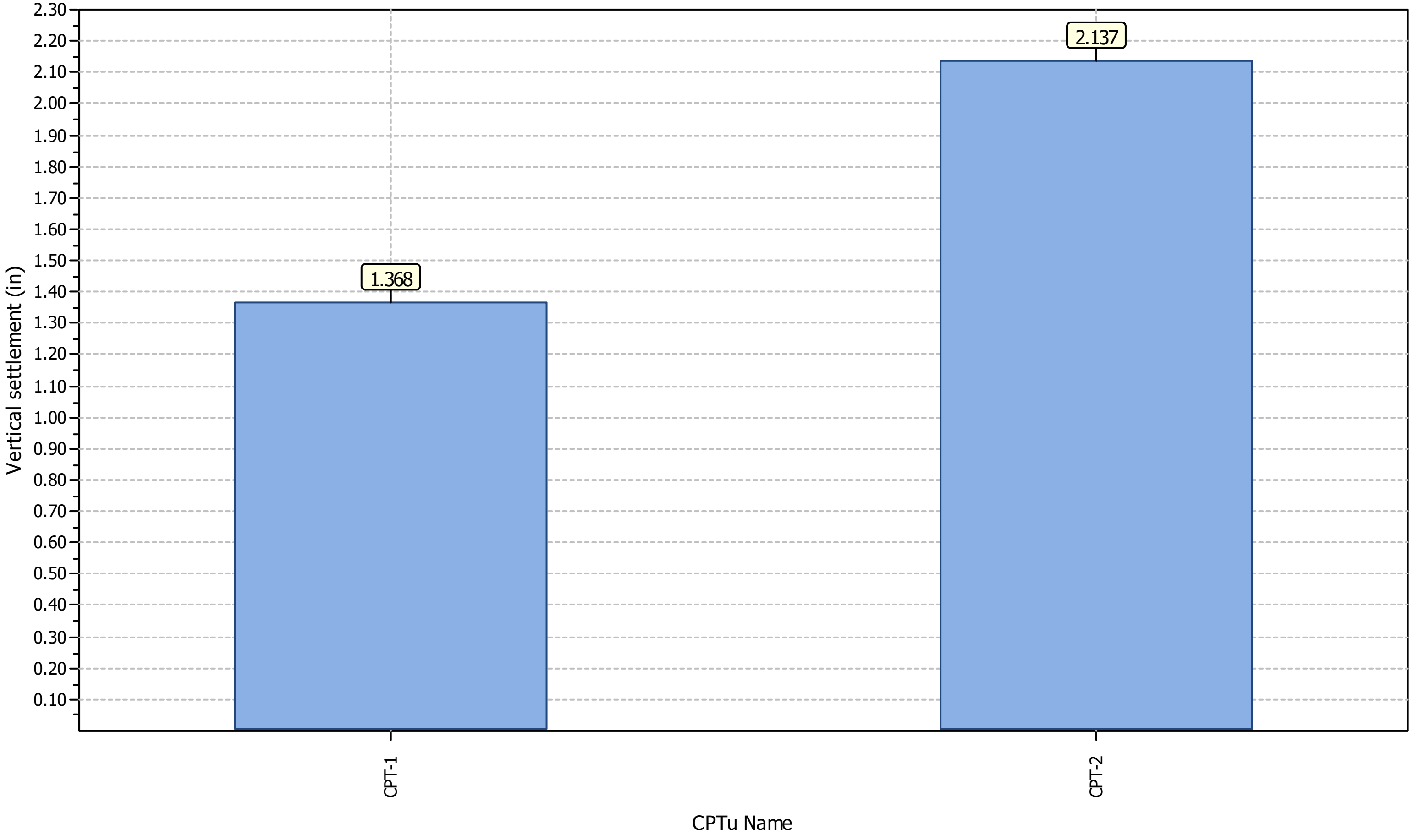
APPENDIX D

LIQUEFACTION ANALYSES

Project title :

Location :

Overall vertical settlements report



GEOLOGISMIKI Geotechnical Software

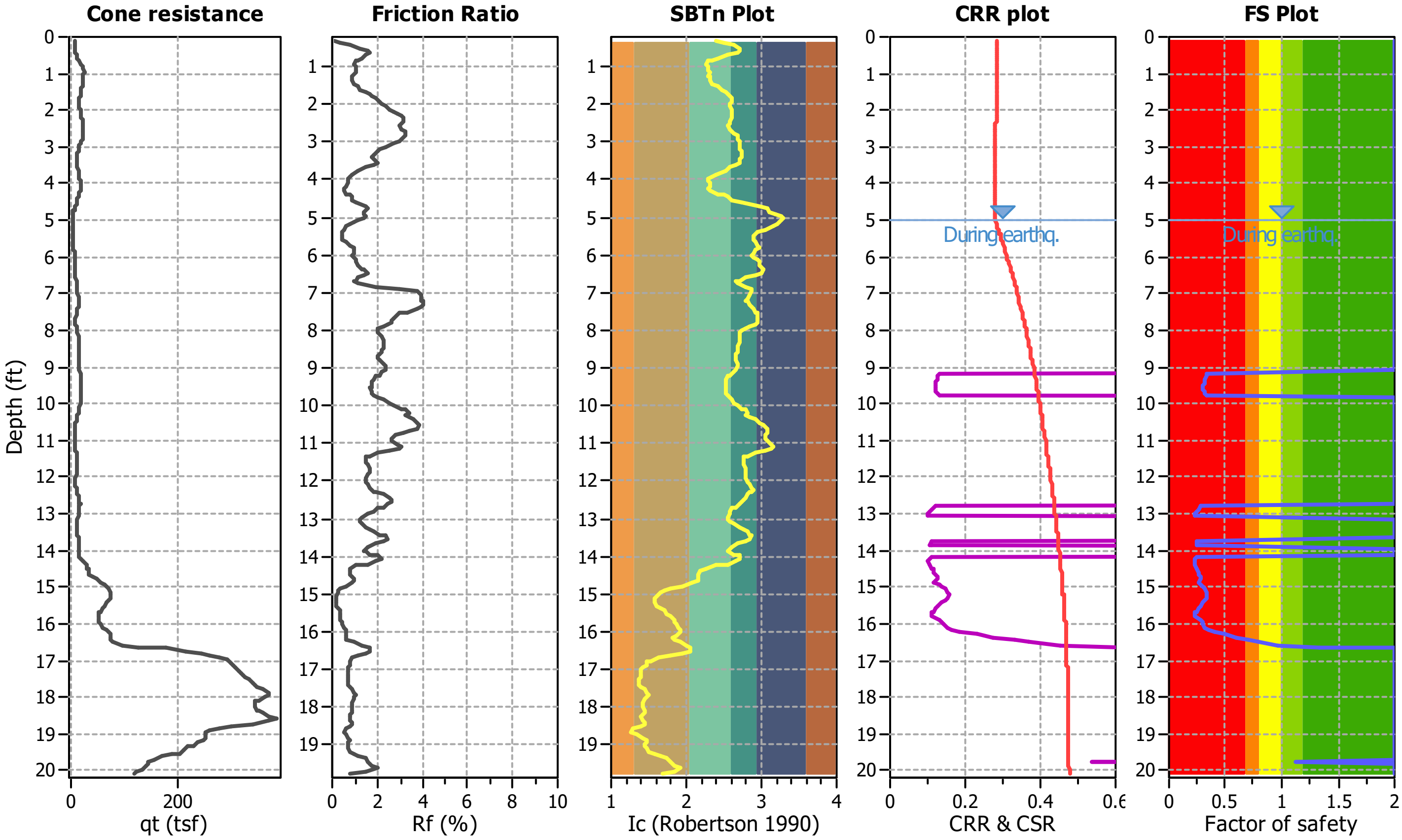
LIQUEFACTION ANALYSIS REPORT

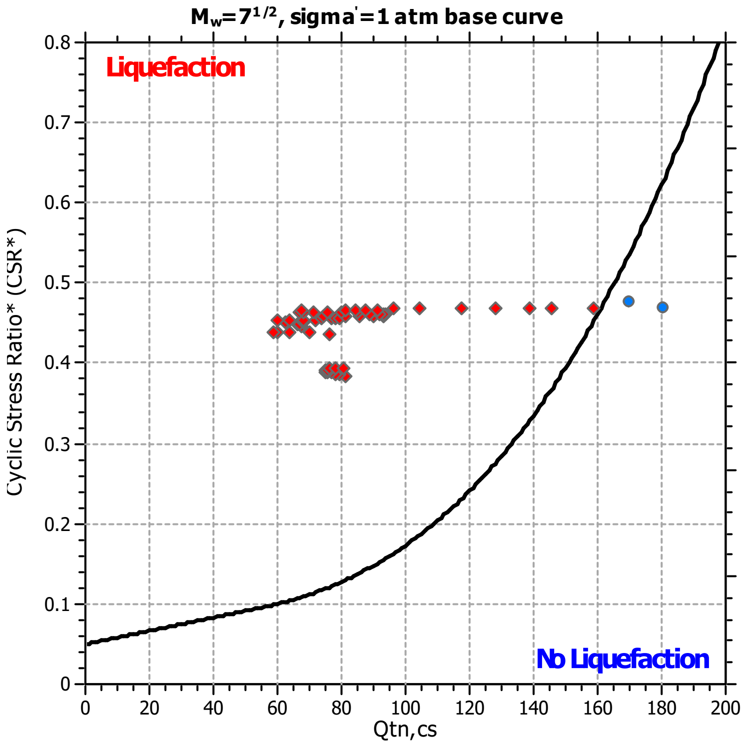
Project title : **Location :**

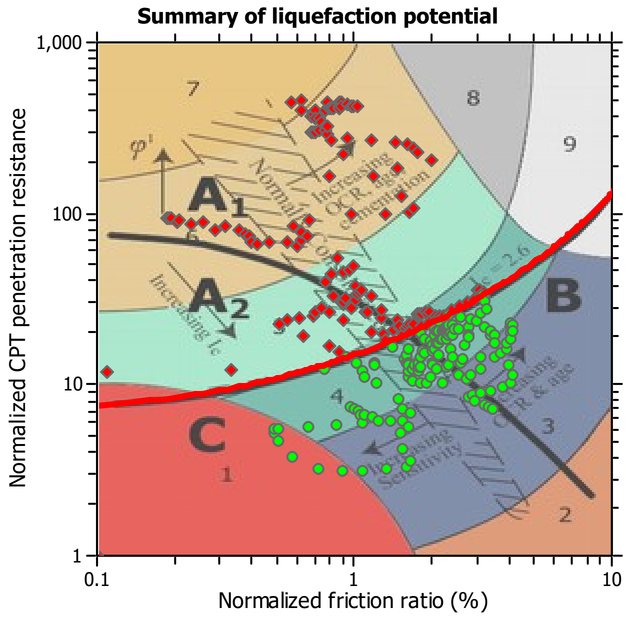
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

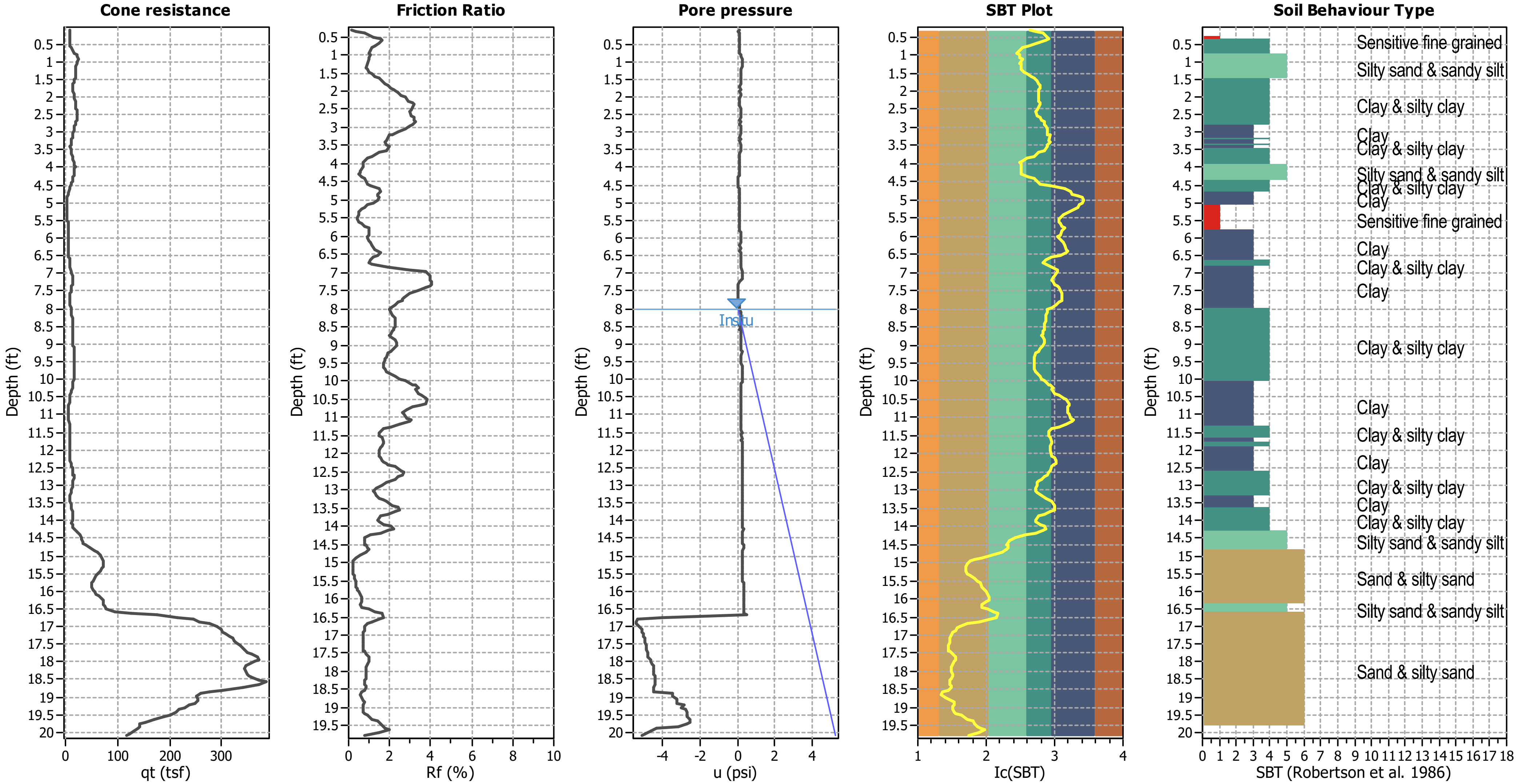






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



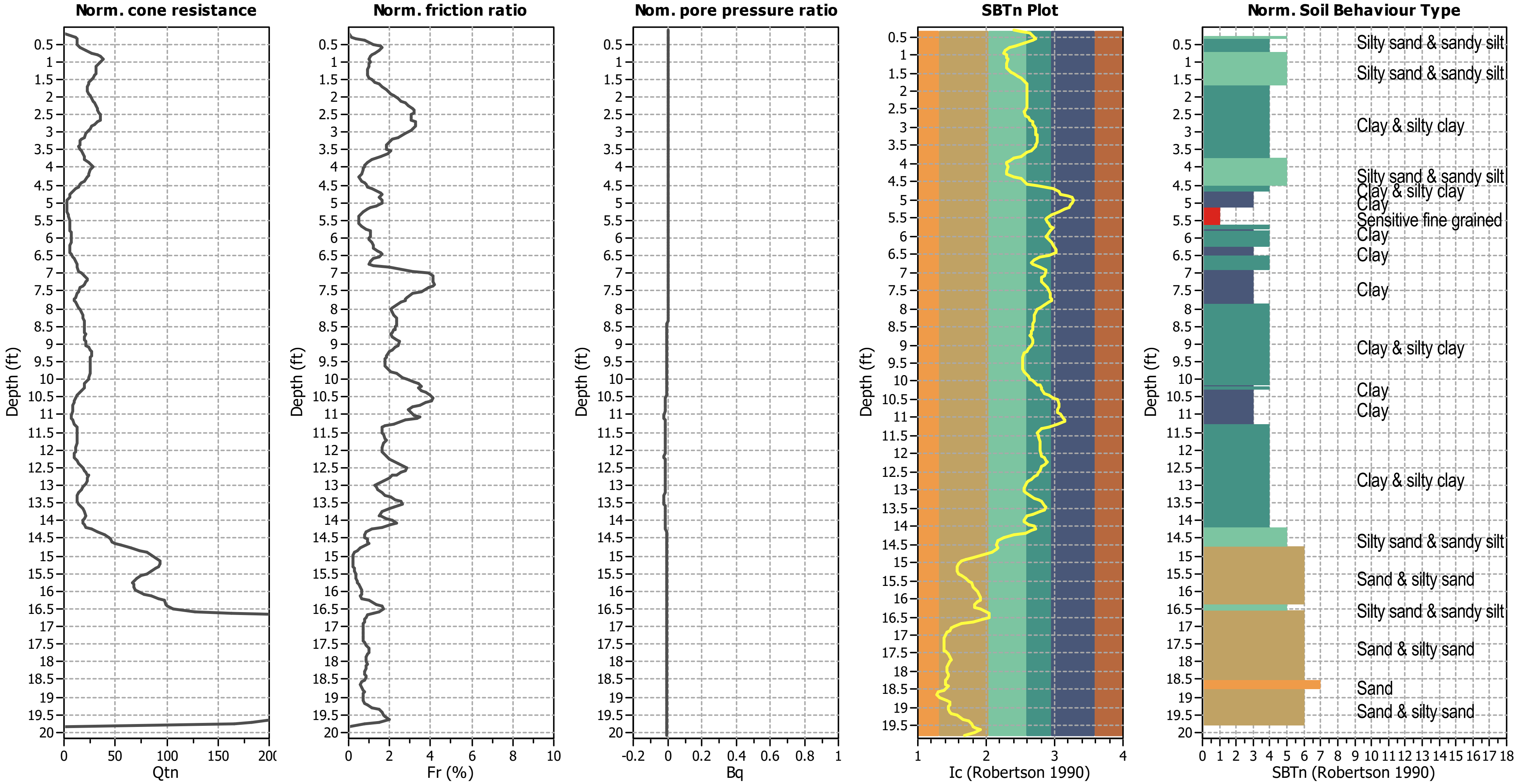
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



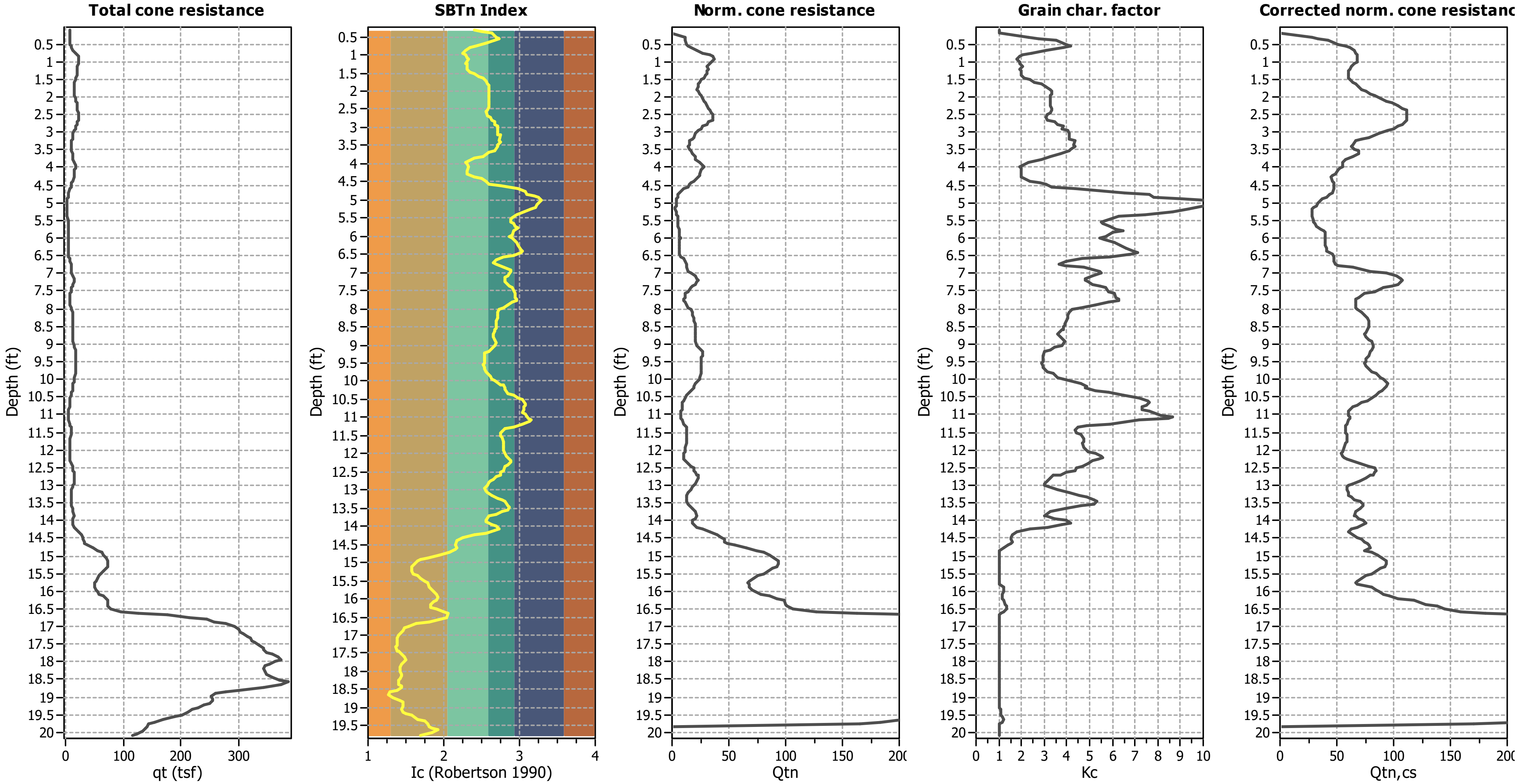
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A		
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes		
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A		

SBTn legend

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

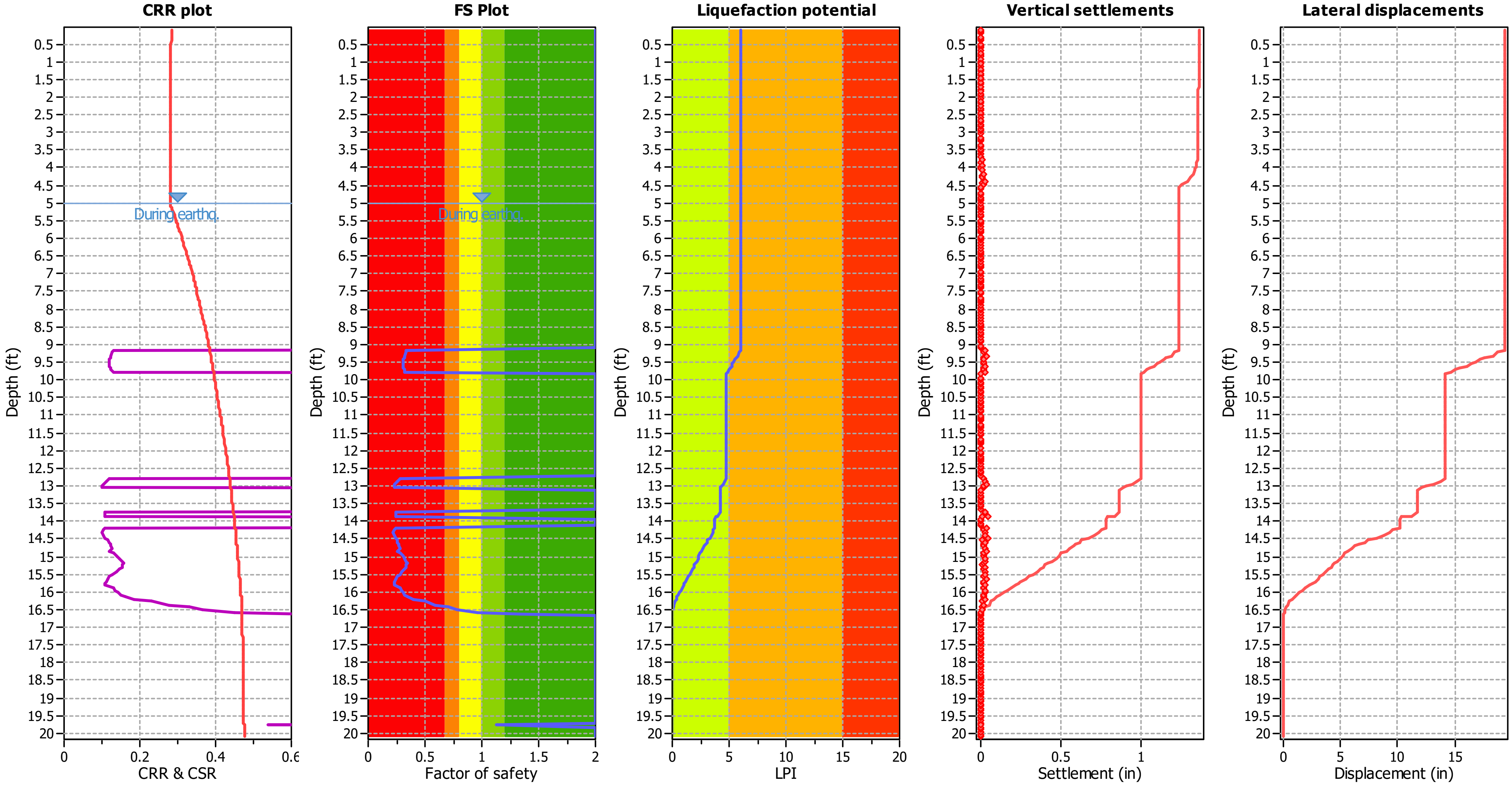
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

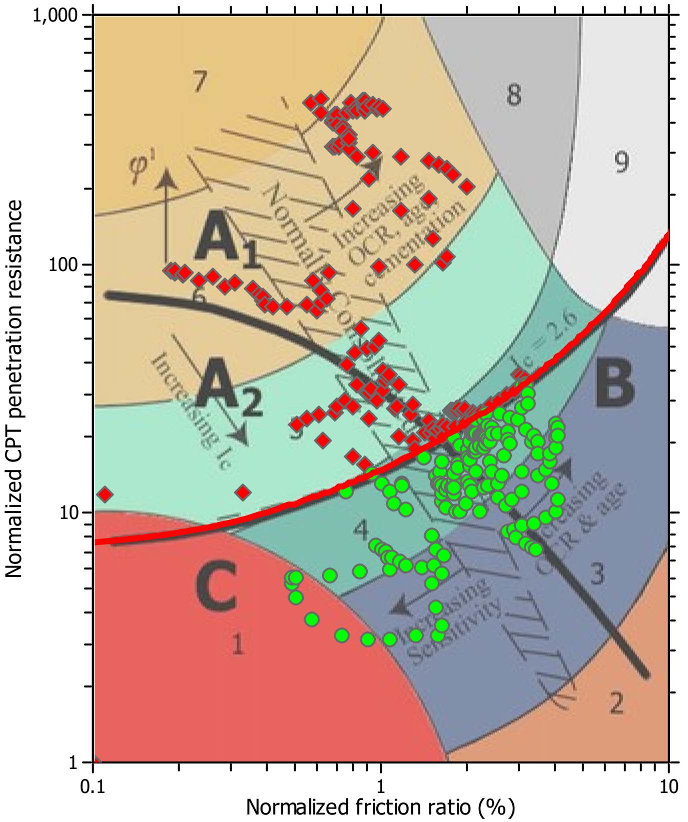
F.S. color scheme

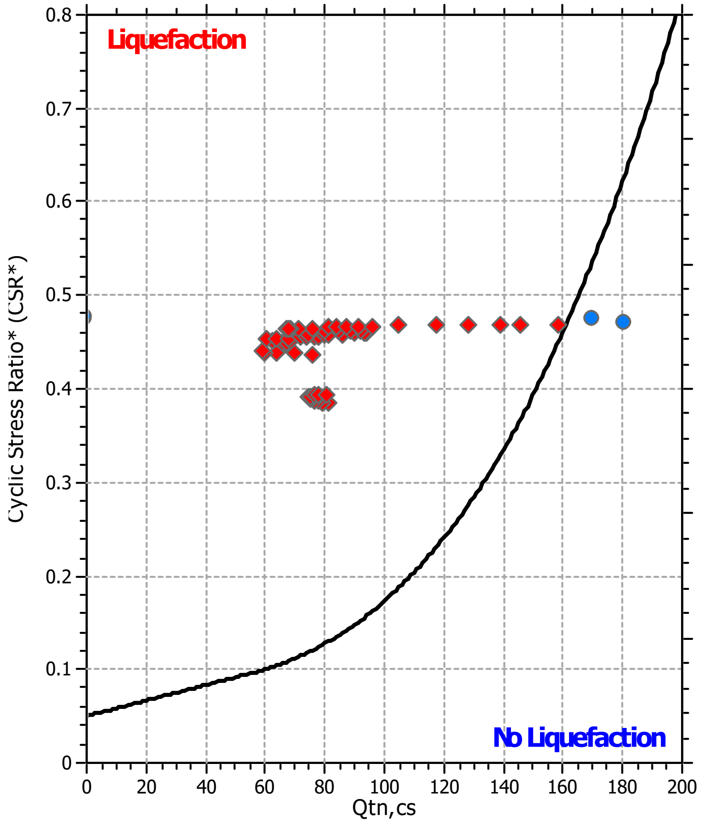
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

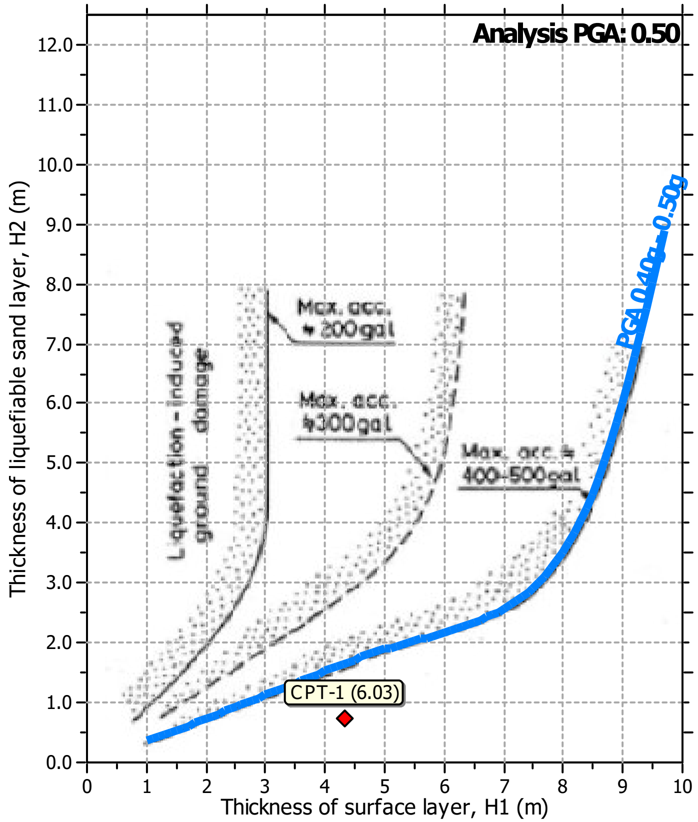
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



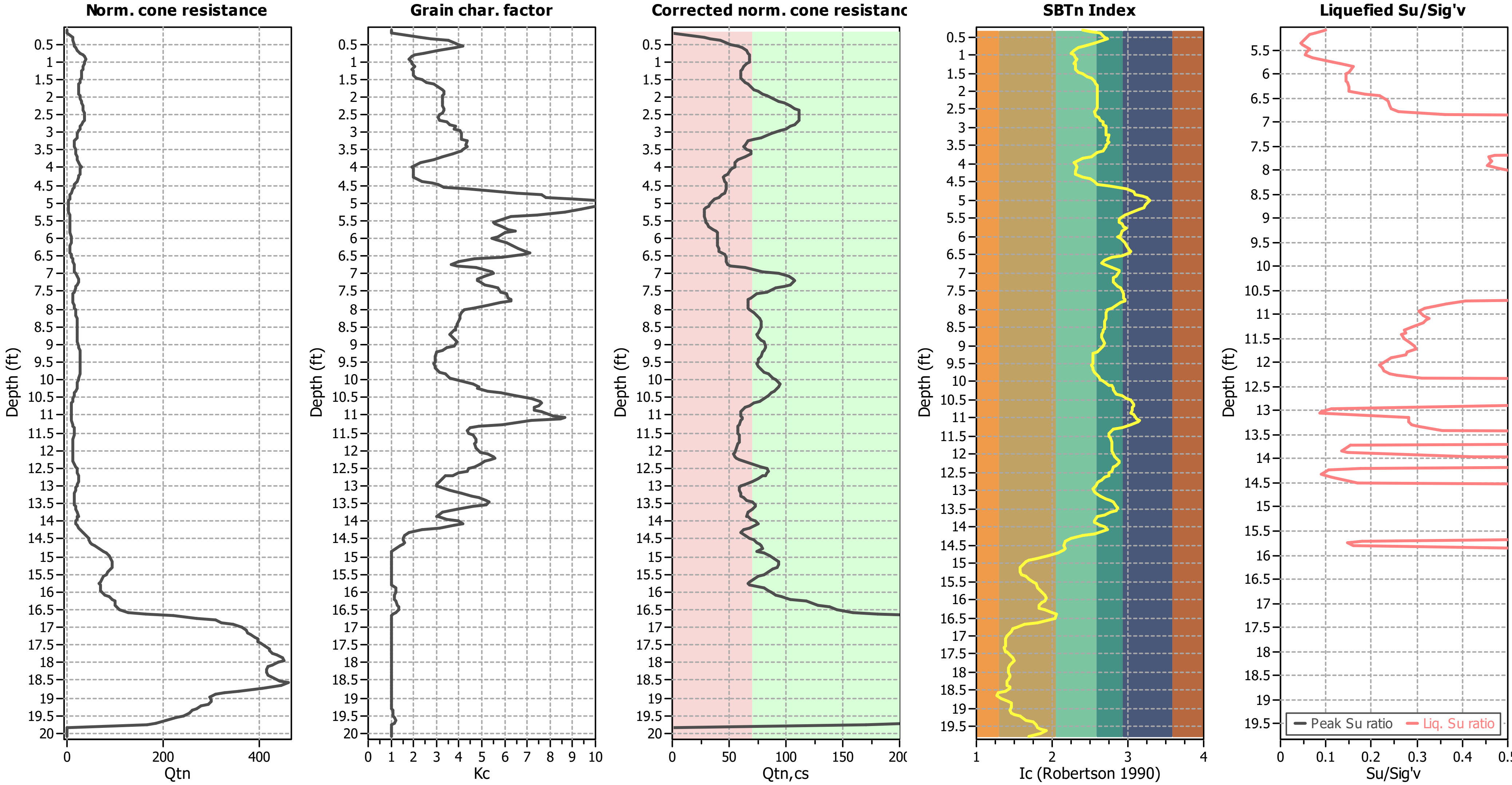




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

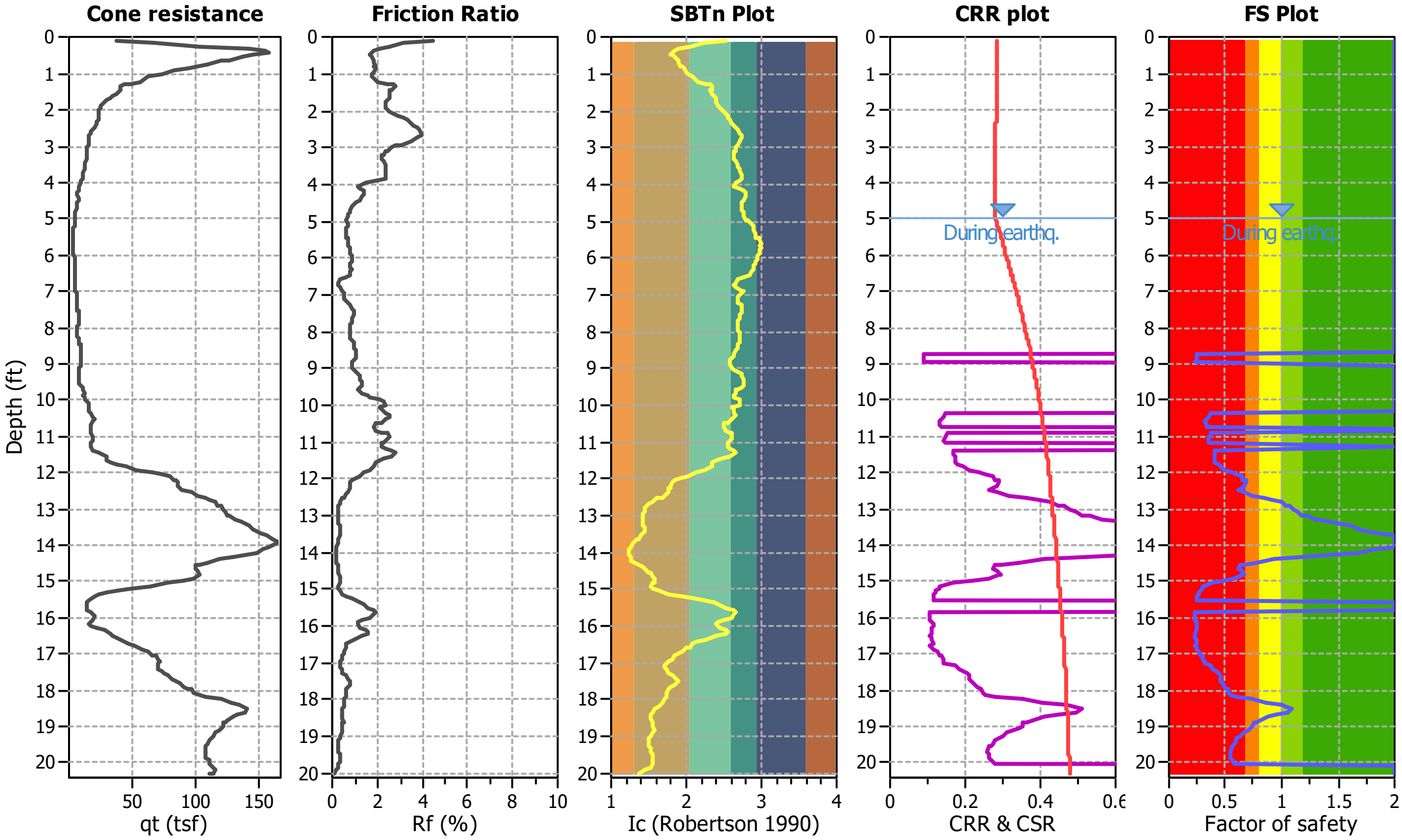
LIQUEFACTION ANALYSIS REPORT

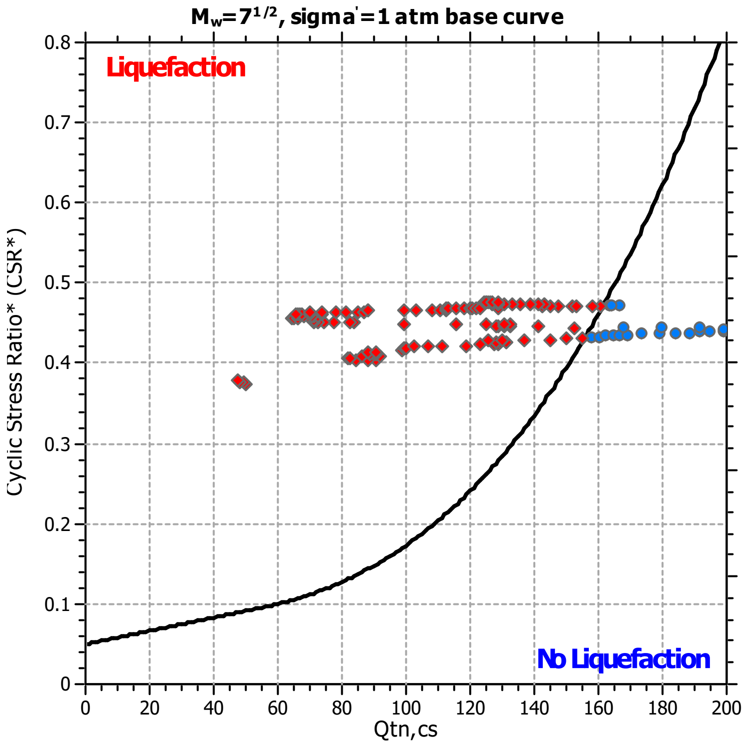
Project title : **Location :**

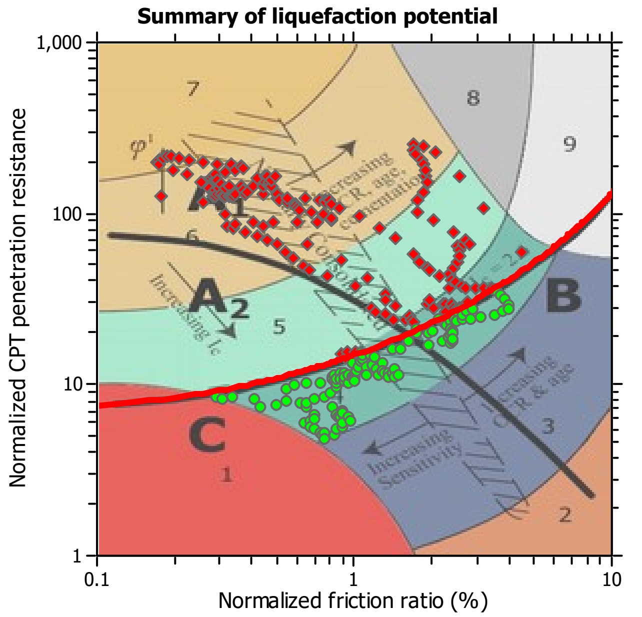
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	7.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	Limit depth:	N/A
Peak ground acceleration:	0.50	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

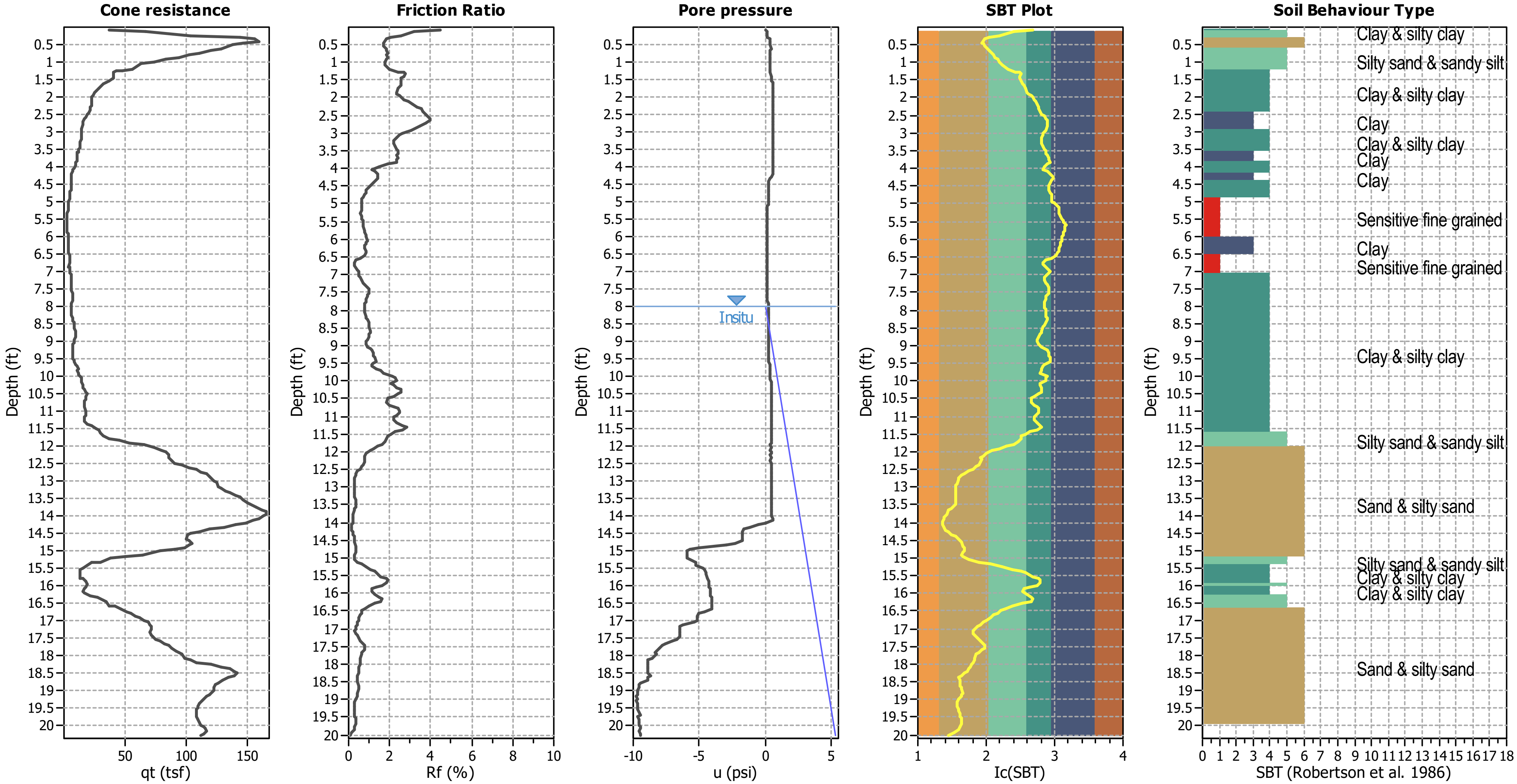






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



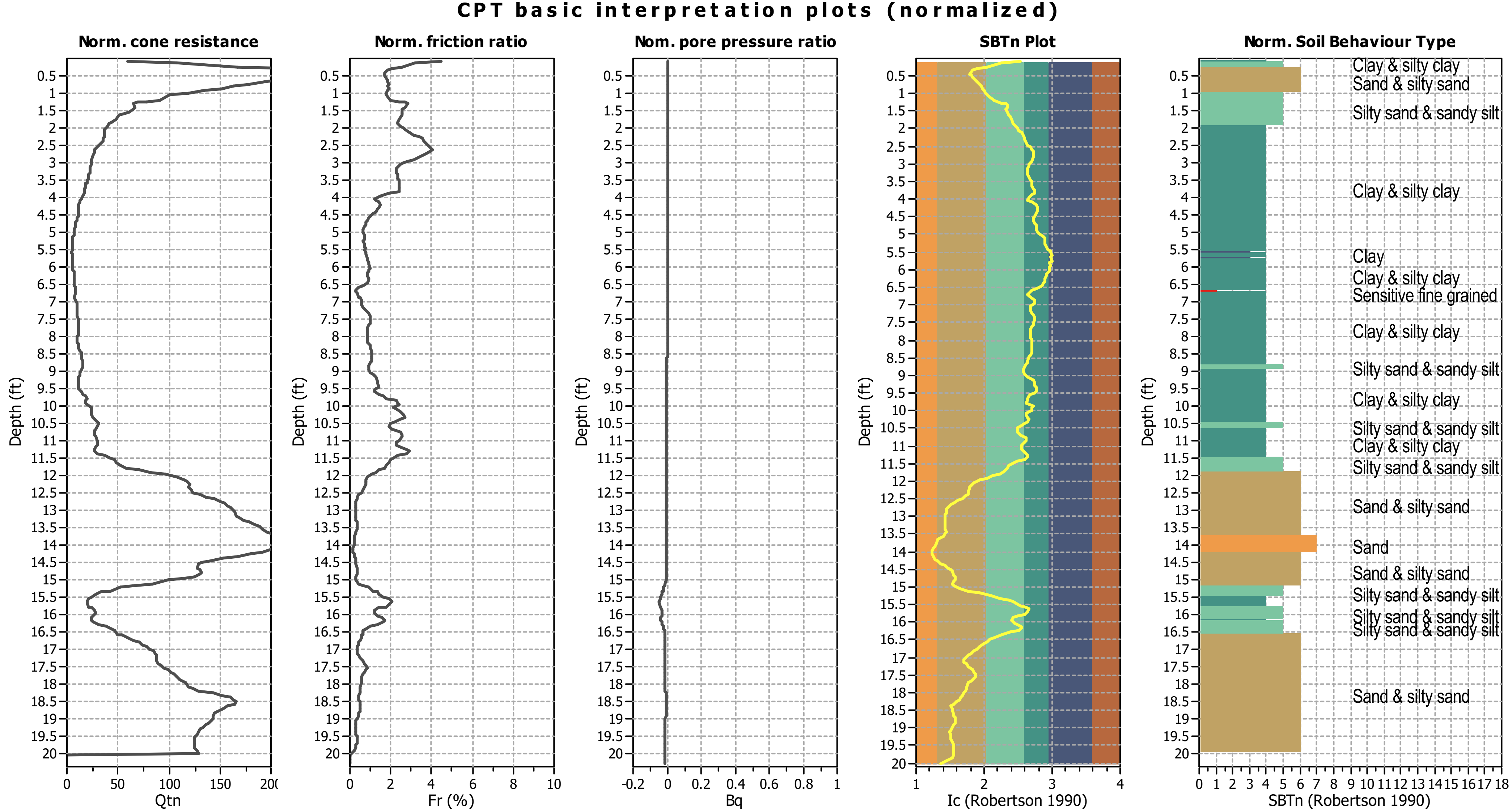
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



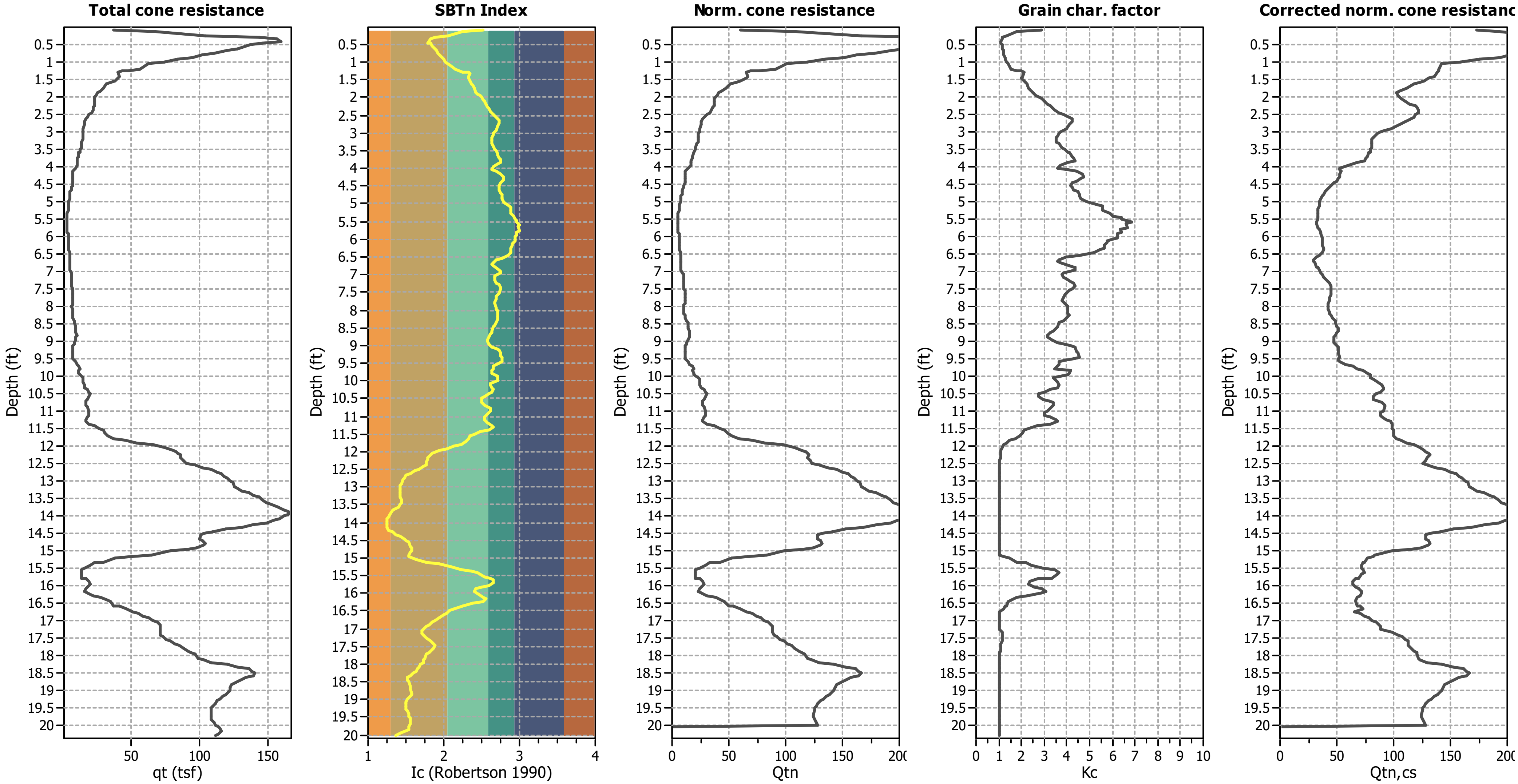
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

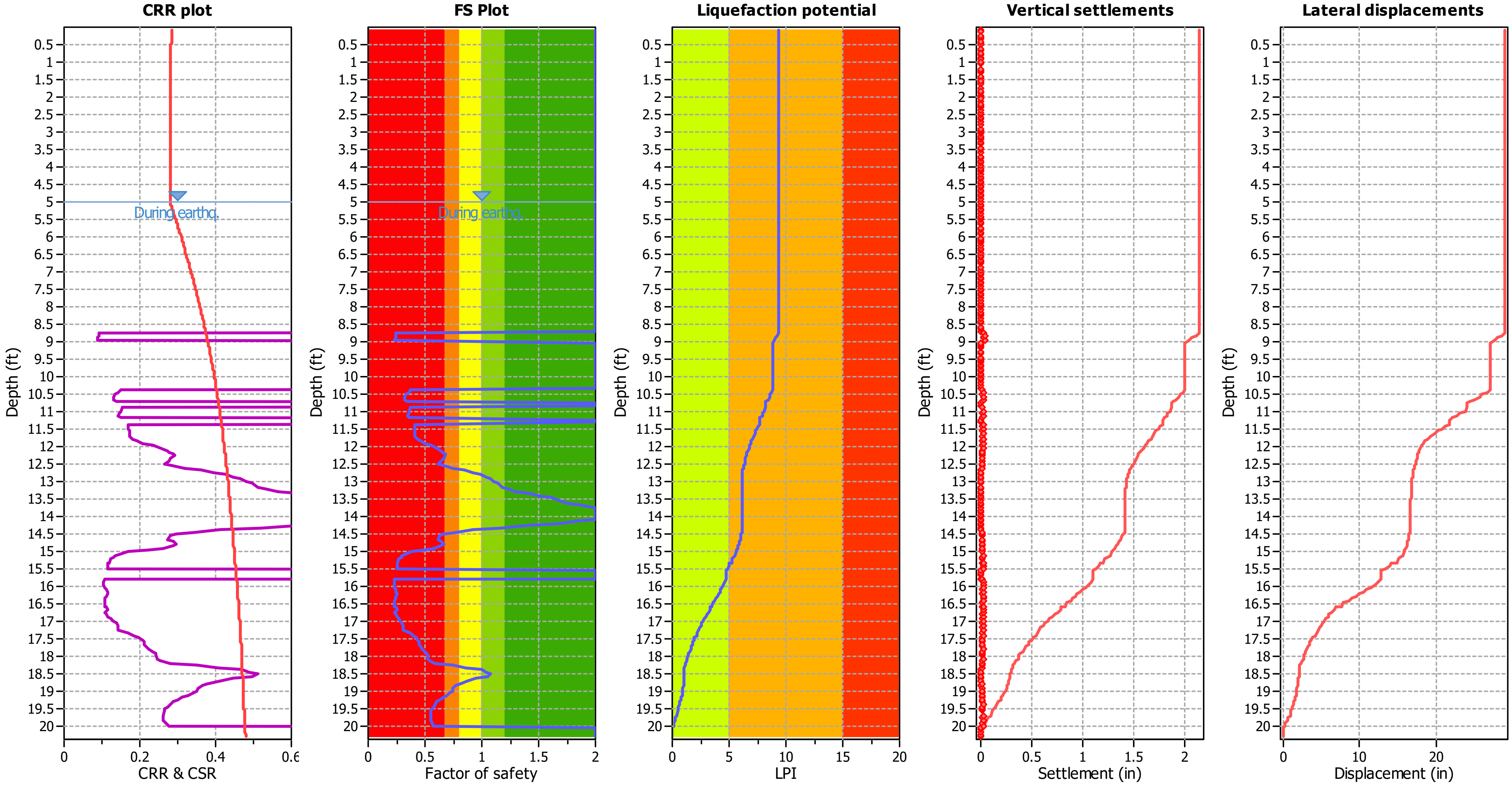
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

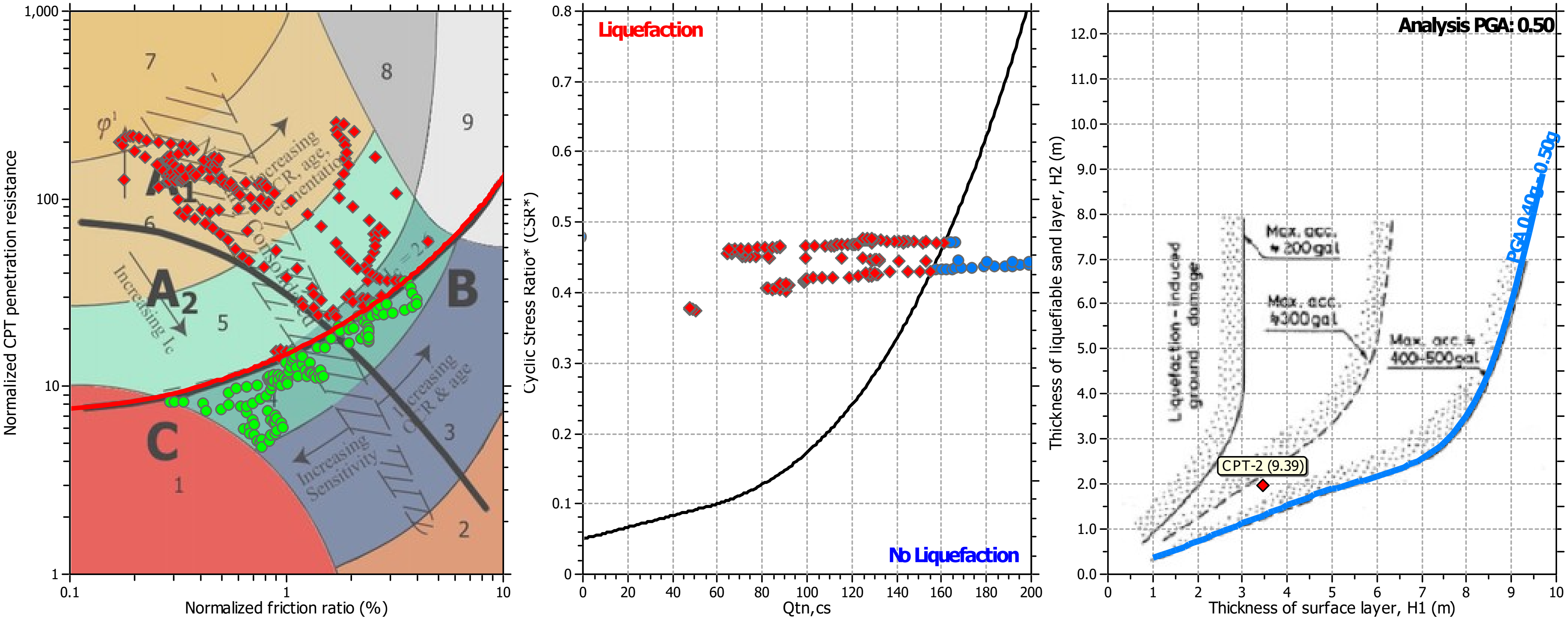
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

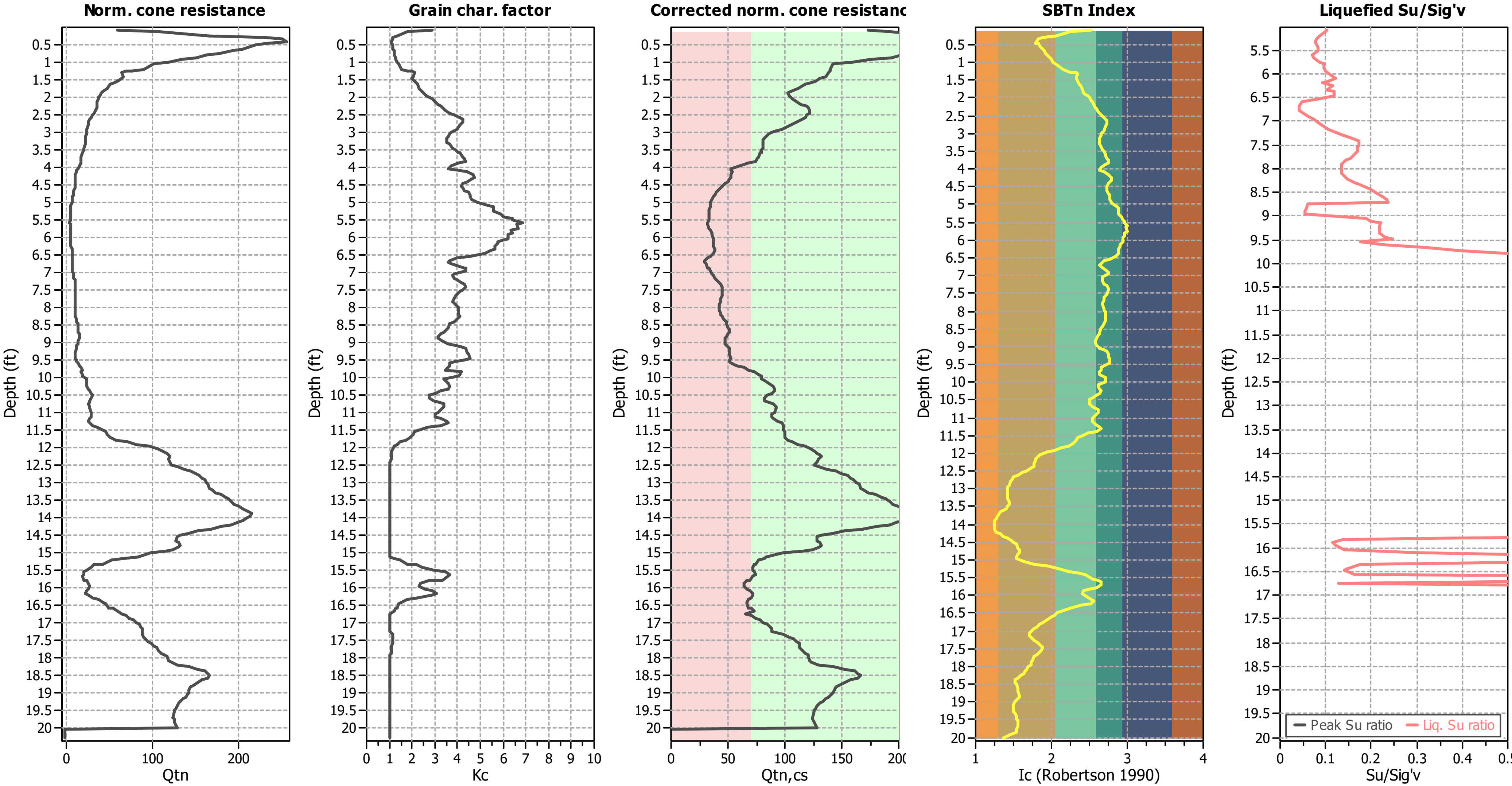
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A

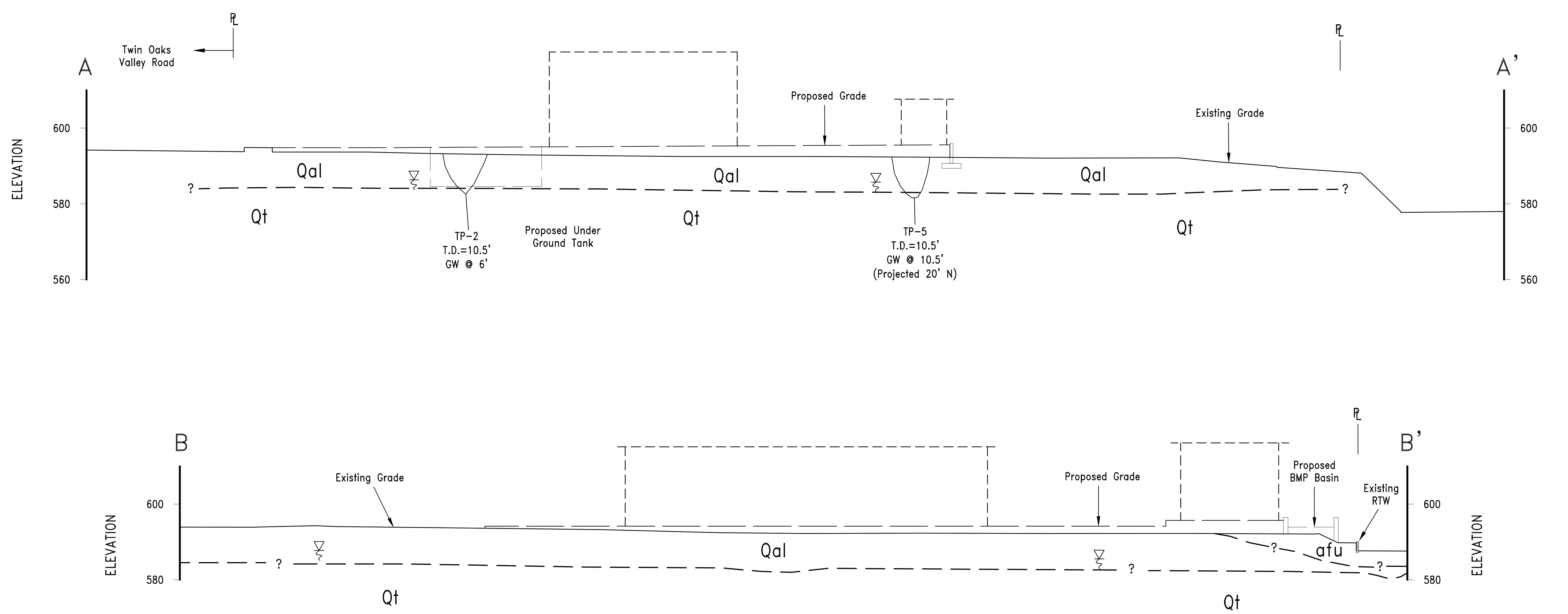
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.50	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	8.00 ft	Fill height:	N/A	Limit depth:	N/A


BORDEN ROAD
TWIN OAKS VALLEY RD.
TP-1
0-5.0' afu
5.0'-8.5' Qal
8.0'-15.5' Qt
▽@ 11.0'
T.D.=15.5'
TP-2
0-9.0' Qal
9.0'-10.5' Qt
▽ @ 6.0'
T.D.=10.5'
TP-3
0-9.0' Qal
9.0'-10.5' Qt
▽ @ 8.5'
T.D.=10.5'
TP-4
0-9.0' Qal
9.0'-10.5' Qt
▽ @ 8.5'
T.D.=10.5'
TP-5
0-9.0' Qal
9.0'-10.5' Qt
▽ @ 6.0'
T.D.=10.5'
TP-6
0-2.0' afu
2.0'-9.0' Qal
9.0-10.5' Qt
▽ @ 6.0'
T.D.=10.5'
CPT-1
CPT-2
afu (Qal) ((Qt))
Qal (Qt)
A
A'
B
B'
LEGEND
afu UNDOCUMENTED FILL
Qal ALLUVIUM
Qt TERRACE DEPOSIT
BRACKETED WHERE BURIED
APPRROXIMATE LOCATIONS OF TEST PITS
APPROXIMATE LOCATION OF GEOLOGIC CONTACT
CROSS SECTION
APPROXIMATE LOCATION OF CONE PENETRATION TEST
PLATE 1
Geologic Map and Exploration Location Plan
AGS
ADVANCED GEOTECHNICAL SOLUTIONS, INC.
Project: P/W 1908-04
Report: 1908-04-B-1
Date: May 2020
SCALE: 1" = 20'
HOWES | WEILER | LANDY
PLANNING & ENGINEERING
Tel. 760.929.2288 Fax. 760.929.2287
2888 LOKER AVENUE EAST SUITE 217
CARLSBAD, CA 92010
NTI FUEL STATION & CAR WASH
SEC TWIN OAKS VALLEY RD. &
BORDEN RD. SAN MARCOS, CA 92069
DATE: 02/25/20
SHEET 1 OF 1



CROSS–SECTIONS A–A', and B–B'

SCALE 1"=20'

PLATE 2

AGS ADVANCED GEOTECHNICAL SOLUTIONS, INC.
485 Corporate Drive, Suite B
Escondido, California 92029
Telephone: (619) 867-0487

Project#	Report#	Date:
P/W 1908–04	1908–04–B–1	May 2020