

A P P E N D I X D

G E O T E C H N I C A L R E P O R T

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GEOTECHNICAL ENGINEERING
REPORT FOR THE PROPOSED
HOTEL INDIGO



**GEOTECHNICAL ENGINEERING
REPORT**

For the
PROPOSED HOTEL INDIGO

**501 Industrial Road,
San Carlos, California**



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REPORT**

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PROPOSED HOTEL INDIGO

**501 Industrial Road,
San Carlos, California**

Prepared for

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PSI PROJECT NO. 575-1547-1

October 21, 2019





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October 21, 2019

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Subject: Geotechnical Engineering Report
Proposed Hotel Indigo
501 Industrial Road, San Carlos, California
PSI Project No. 575-1547-1

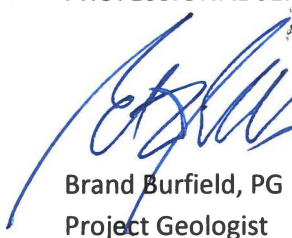
Dear Mr. Wu:

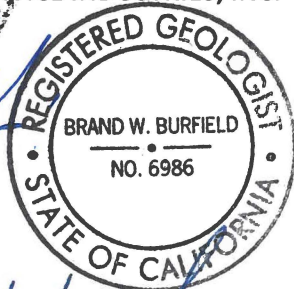
Professional Service Industries, Inc. (PSI), an Intertek company, is pleased to submit our Geotechnical Engineering Report for the above-referenced project in San Carlos, California. This report includes the results of field and laboratory testing and geotechnical recommendations for foundations, as well as general site development.

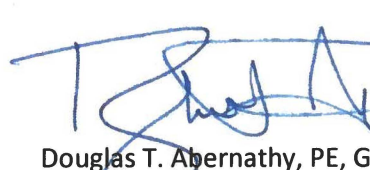
We appreciate the opportunity to perform this Geotechnical Study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office.

Respectfully Submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.


Brand Burfield, PG
Project Geologist




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Reviewed by: Robert R. Russell, PE, GE
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FIGURES

Figure 1: Site Location Map

Figure 2: Site Plan and Boring Location Map

APPENDIXES

Appendix A – Exploration Logs and Drilling Permit

Appendix B – Laboratory Test Results

Appendix C – Liquefaction Analysis



1.0 PROJECT INFORMATION

1.1 Project Authorization

Professional Service Industries, Inc. (PSI) is pleased to submit our Geotechnical Engineering Report for the proposed Hotel Indigo in San Carlos, California. Our work was performed in general accordance with the scope of work as outlined in our Proposal Number 575-286055, dated August 14, 2019. Our services were authorized by Mr. Chao Wu of Holly Hotel Group, LLC, who provided a signed copy of our proposal on August 15, 2019.

1.2 Site Location and Description

The subject site is located southwest of Highway 101 at the eastern corner of the intersection of Holly Street and Industrial Road, with a street address of 501 Industrial Road in San Carlos, California (see Figure 1). The site is an irregularly-shaped property measuring approximately 2.2 acres in plan area. At the time of our fieldwork, the site was developed with a number of commercial and light industrial warehouses, storage structures and sheds surrounded by both unpaved and asphalt-paved parking and drive areas (see Figure 2). The pavements were observed to be in generally fair to poor condition.

The regional area has a gentle slope toward the northeast (toward the San Francisco Bay), however; the subject site is relatively level, with slight engineered slopes in paved areas for drainage toward Industrial Road. The site elevations obtained from Google Earth range from between approximately 8 and 11 feet above mean sea level (AMSL). This agrees with a site elevation of less than 10 AMSL estimated from the San Mateo, California USGS topographic map.

1.3 Project Understanding

A conceptual site plan for the proposed development with floor plans, perspective and section views (C.Y. Lee, 2019) was provided by you for our use in preparing this report. Based on the provided information, PSI understands that it is planned to demolish the existing structures at the subject property and develop the site with a multi-story structure of about 27,000 square feet (sf) in plan area for use as a hotel. The structure will be surrounded by grade-level parking and landscaped areas. The south wing of the building (measuring about 10,000 sf) is planned to be 3 stories, while the north wing (about 17,000 sf) is planned to have 6 stories. The information provided indicates that the structure will consist of a reinforced concrete podium (expected 16 ft. high) on the ground level, with wood-framed construction on the upper stories. A subsequent plan set provided (RYS, 2019) indicates that a below-grade parking level (expected concrete construction) is being considered either under the garden courtyard or below the 3-story portion of the hotel. Other improvements are to include subsurface utilities and concrete flatwork. The approximate location of the proposed structure is presented in Figure 2.

Expected structural loading was not made available at the time this report was prepared but based on the type of development planned and on our previous experience with similar projects, PSI assumes, for the purpose of this report, maximum point and wall loads on the order of 450 kips and 20 kips per foot, respectively. The finished floor elevations of the buildings were not furnished to us, but for the purposes of this report, we have assumed finish exterior grades are to be near (+/- two feet) existing grades, but cuts of up to about 10 to 12 feet may be expected beneath the structure if a subterranean parking level



is constructed. A site plan with the locations of the existing and proposed structures is presented as Figure 2. Should any of the above information or assumptions made by PSI be inconsistent with the planned construction, we request that you contact us immediately to allow us to make any necessary modifications to our recommendations.

1.4 Purpose and Scope of Services

The purpose of our geotechnical evaluation was to assess the subsurface soil conditions at the site in order to provide appropriate recommendations for site preparation and foundation design. Our evaluation was in general accordance with the scope of work outlined in our Proposal Number 575-286055, dated August 14, 2019. Our scope of services included 9 soil explorations (5 Cone Penetration Tests and 4 soil borings) for the property in various areas at the subject site.

This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface conditions, and presents geotechnical recommendations regarding the following:

- Site topographic information and surface conditions;
- Review of subsurface conditions including groundwater;
- Review of field and laboratory test procedures and test data;
- Information on potentially expansive, deleterious, chemically active or corrosive materials;
- Figures including a site plan with boring locations and boring logs;
- California Building Code (CBC) site class and seismic site coefficients for use in seismic design (CBSC, 2016);
- An assessment of the geologic hazards that could impact the site including a detailed liquefaction evaluation;
- Site preparation and grading considerations, including recommended fill material characteristics and compaction requirements for general site fill and slab/pavement subgrades, including an assessment as to the suitability of on-site soils for use as fill;
- Recommendations pertaining to design and construction of foundations, retaining walls, floor slabs, and pavements, including ground improvement, allowable soil bearing pressures, anticipated bearing depths and estimated settlements; and,
- Comments regarding factors that may impact construction and performance of the proposed construction.

The scope of services did not include an environmental assessment for determining the presence or absence of wetlands or hazardous or toxic materials in the soil, bedrock, surface water, groundwater, or air on or below, or around this site. A Phase I Environmental Site Assessment was prepared by PSI and provided under separate cover. Any statements in this report or on the boring logs regarding odors, colors, and unusual or suspicious items or conditions are strictly for information purposes only.



2.0 SUBSURFACE EXPLORATION

2.1 Site Geology

The subject site is located within a large region known as the Coast Ranges geomorphic province. This province is characterized by extensively folded, faulted, and fractured earth materials. These structural features trend in a northwesterly direction and make up the prominent system of northwest-trending mountain ranges separated by sediment filled valleys (CGS, 2002).

The subject site is situated near the southwestern edge of the San Francisco Bay. Our observations and analysis of readily available, pertinent geologic literature indicate that the subject site is underlain by artificial fill. The material is described as loose to very well consolidated gravel, sand, silt, clay, rock fragments, organic matter, and man-made debris in various combinations, with variable thickness that may exceed 30 meters (Brabb, 1998). This generally agrees with the San Mateo County Geotechnical Hazard Synthesis Map which shows the site in an area of younger estuarine mud and artificial fill (Leighton, 1976).

2.2 Pre-Field Activities

Prior to initiation of field drilling activities, PSI outlined the site in white paint, staked the proposed boring locations and contacted Underground Service Alert (USA) a minimum of 48 hours prior to beginning work to locate any potential buried utilities. The USA inquiry identification number for the utility locate request was #W923500498. Also prior to drilling, PSI obtained a drilling permit (No. 19-1098) from the San Mateo County Environmental Health Services Division (SMCEH). A copy of the permit is included in Appendix A.

2.3 Subsurface Conditions

In order to evaluate soil conditions at the site, PSI advanced 9 soil explorations for the property in various areas at the subject site. The following is a description of the explorations;

- 1 Cone Penetration Test (CPT) to 100 feet below the ground surface (bgs) for seismic classification and building evaluation;
- 4 CPTs to 70 feet bgs for building evaluation;
- 1 auger boring to 51½ feet bgs for liquefaction and building evaluation, and;
- 3 auger borings to 6½ feet bgs for parking lot evaluation.

Approximate locations of the soil explorations, as well as existing and proposed improvements, are shown on Figure 2. Logs of the CPTs and soil borings are presented in Appendix A.

2.3.1 Cone Penetration Test (CPT) Explorations

The CPT explorations, designated CPT-1 through CPT-5, were performed by Gregg Drilling of Martinez, California using a 20-ton, truck-mounted CPT rig. The CPT explorations were advanced in the proposed new building area; CPT-1 was advanced to about 100 feet bgs, while CPT-2 through CPT-5 were advanced to about 70 feet bgs. CPT-2 included pore pressure testing at a depth of 35½ feet bgs and seismic sounding at approximately 5-foot intervals to a depth of 50 feet bgs. CPT-1 included seismic sounding at 5-foot



intervals below a depth of 50 feet. At the completion of each CPT, the resultant hole was backfilled with cement grout to match the existing surface grade in accordance with SMCEH permit requirements.

2.3.2 Auger Borings

The soil borings, designated B-1 through B-4, were drilled using a CME-75 drill rig operated by HEW Drilling of East Palo Alto, California using both hollow-stem and solid-flight auger drilling methods to a depth of about 51½ feet bgs for B-1 and to about 6½ feet bgs for borings B-2 through B-4. At the completion of drilling, each boring was backfilled with cement grout to match the existing surface grade in accordance with SMCEH permit requirements.

During the sampling procedure, Standard Penetration Tests (SPT) were performed in accordance with ASTM D1586 and relatively undisturbed samples were obtained in general accordance with ASTM D3550. The SPT for soil borings were performed by driving a 2-inch diameter split-spoon sampler into the undisturbed subsurface materials located at the bottom of the advanced borehole with repeated blows of a 140-pound hammer falling a vertical distance of 30 inches. The number of blows required to drive the sampler the last 12 inches of an 18-inch penetration depth is a measure of the soil consistency. For ASTM D3550 (California Modified Sampler) the split barrel sampler possesses a 3-inch outside diameter and is driven in the same manner as the SPT. The field blow counts obtained from the California Modified sampler, as indicated in the boring logs, are multiplied by a factor of 0.69 to obtain an approximate correlation to SPT blow counts (SPT-N value). Samples were identified in the field, placed in sealed containers, and transported to the laboratory for further classification and testing.

2.3.3 Soil Description

In the areas explored at B-1, B-3 and B-4, the site was surfaced with a pavement section generally consisting of approximately 2 inches of asphalt over approximately 0 to 11 inches of aggregate base. Boring location B-2 was surfaced with gravel. Beneath these surface materials, the site is underlain by about 11 feet of lean clay (CL) and fat clay (CH), which appears to be fill material. These upper clays are underlain by layers of sandy silt (ML), silty sand (SM), lean clay and fat clay to the total depth explored with our soil borings of approximately 50 feet bgs.

The consistency of the clay and silt was observed to be generally medium stiff to stiff in the upper 33 feet, and very stiff below 33 feet, while the sand and gravel encountered was generally medium dense to dense in terms of the standard penetration tests performed. Bedrock was not encountered in our borings.

The CPT results suggest relatively low-density clay and silt until about 32 feet bgs, where a layer of dense sand was encountered, which was underlain by higher density clay and silt to the maximum depth explored of 100 feet bgs.

2.3.4 General Boring Notes

The above subsurface information is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring logs, included in Appendix A, should be reviewed for specific information at the boring locations. These records include soil descriptions,



stratification, penetration resistance, locations of the samples and laboratory test data. The stratification shown on the logs represent the conditions only at the actual location at the time of our exploration. Variations may occur and should be expected between exploration locations. The stratification that represents the approximate boundary between subsurface materials and the actual transition may be gradual. It should be noted that, although the test borings are drilled and sampled by experienced professionals, it is sometimes difficult to record changes in stratification within narrow limits, especially at great depths. In the absence of foreign substances, it is also sometimes difficult to distinguish between discolored soils and clean fill soil.

2.4 Groundwater

Groundwater was encountered during drilling only in soil boring B-1 at a depth of about 7.5 feet bgs. Additionally, a pore pressure dissipation test was performed at CPT-2 and groundwater was calculated to be at approximately 6 feet bgs. Based on our review of logs of nearby soil borings available from the State Water Resource Board GeoTracker environmental database, historic high groundwater is estimated to be at about 5 feet bgs. It is possible that transient, saturated ground conditions at shallower depths could develop during periods of heavy precipitation, landscape watering, leaking water lines, or other unforeseen causes. Variations in groundwater levels should be expected seasonally, annually, and from location to location.

Groundwater is expected to be a consideration during the construction of the proposed structure, and may significantly impact the proposed construction, dependent upon the depth of any utilities and whether a subgrade parking level is planned. Due to the shallow groundwater encountered at the site, it is expected that a subgrade parking level (even a half-sunken parking level) would likely encounter groundwater, requiring specialized construction techniques and the use of drains, sump pumps, and working mats to manage the shallow groundwater conditions. Additionally, it is possible that collected water would require treatment and offsite disposal. As such, a subgrade parking level is not recommended.

2.5 Laboratory Evaluation

Selected samples of the subsurface soils encountered were returned to our laboratory for further evaluation to aid in classification of the materials, and to help assess their strength and expansive nature. The laboratory evaluation consisted of visual and textural examinations, moisture and density tests, Expansion Index testing, Atterberg Limits testing, R-value testing and particle size analysis (including percent passing the No. 200 sieve). Sulfate, chloride, pH, and electrical resistivity testing were also performed to assist in evaluating the corrosive potential of the site soils. A brief discussion of the laboratory tests performed, and the test results are presented in Appendix B or on the boring logs in Appendix A. Samples that were not altered by laboratory testing will be retained for 30 days from the date of this report and will then be discarded.



3.0 SEISMIC CONSIDERATIONS

3.1 Regional Seismicity

Generally, seismicity within California can be attributed to faulting due to regional tectonic movement. This includes the San Andreas Fault and most parallel and subparallel faulting within the region. The portion of California which includes the subject site is considered seismically active. Seismic hazards within the site can be attributed to potential groundshaking resulting from earthquake events along nearby or more distant faulting.

According to regional geologic literature (Blake, 2000; USGS, 2019), the closest known late Quaternary fault is the San Andreas Fault, located approximately 4.4 miles (7.1 km) southwest of the site. The Special Studies Zones map for the San Mateo quadrangle (CDMG, 1974) indicate that the San Andreas Fault is mapped as close as about 3.8 miles from the site. Several potentially active and pre-Quaternary faults also occur within the regional vicinity. There are no faults mapped as crossing the site (Brabb, 1998). The site is subject to a Maximum Magnitude Event of 7.9 Magnitude along the San Andreas Fault. The Maximum Magnitude Event is defined as the maximum earthquake that appears capable of occurring under the presently known tectonic framework.

3.2 Seismic Analysis

According to the Alquist-Priolo Special Studies Zones Act of 1972 (revised 1994), active faults are those that have been shown to display surface rupture during the last 11,000 years (i.e., Holocene time). This site is not currently situated within a mapped Earthquake Fault Zone (CDMG, 1974) and PSI did not observe any mapped faults crossing the site on readily available resources (Brabb, 1998; USGS, 2019).

The site will be affected by seismic shaking as a result of earthquakes on major active faults located throughout the northern California area. As part of the current, 2016 California Building Code (CBC), the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site. As part of the procedure to evaluate seismic forces, the code requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface.

To define the Site Class for this project, we interpreted the results of our soil CPT probes advanced within the project site to a depth of up to 100 feet bgs and performed shear wave velocity measurements at approximately 5-foot intervals in probes CPT-1 and CPT-2. The average shear wave velocity in the upper 100 feet of the soil column was determined to be about 910 feet per second. The data are presented after the CPT logs in Appendix A. To evaluate the Site Class, we also took into account data available in published geologic reports as well as our experience with subsurface conditions in the general site area. Based upon this, the subsurface conditions within the site are consistent within the characteristics of **Site Class D** (stiff soil profile).



In accordance with the 2016 California Building Code (CBC), the USGS probabilistic ground acceleration values (ASCE 7 Chapter 20, 2010) for latitude 37.5119° and longitude -122.2570° obtained from the U.S. Seismic Design Maps webpage (SEAOC-OSHPD, 2019) are presented in the following table;

Ground Motion Values*

Period (sec)	Mapped MCE Spectral Response Acceleration** (g)		Site Coefficients		Adjusted MCE Spectral Response Acceleration (g)		Design Spectral Response Acceleration (g)	
0.2	S_s	1.759	F_a	1.000	S_{Ms}	1.759	S_{Ds}	1.173
1.0	S_1	0.814	F_v	1.500	S_{M1}	1.221	S_{D1}	0.814

*2% Probability of Exceedance in 50 years

**At B-C interface (i.e. top of bedrock)

MCE = Maximum Considered Earthquake

The Site Coefficients, F_a and F_v presented in the above table were also obtained from the noted Seismic Design Maps webpage as a function of the site classification and mapped spectral response acceleration at the short (S_s) and 1-second (S_1) periods but can also be interpolated from CBC Tables 1613.3.3(1) and 1613.3.3(2). Based on the seismic parameters noted above, a ***Seismic Design Category of E*** is believed appropriate for this site.

Design of structures should comply with the requirements of the governing jurisdiction's building codes and standard practices of the Structural Engineering Association of California.

3.3 Hazard Assessment

Seismically-Induced Dry Settlement of Soils – The materials encountered within the test borings were observed to consist primarily of clay and silt, which were generally medium stiff to hard, and sands, which were medium dense to very dense at the time of our exploration. Based on the anticipated earthquake effect, the stratigraphy of the site, and depth to groundwater, only negligible seismically-induced dry settlement is estimated to occur. Such settlement will probably affect relatively large areas so that differential settlements over short distances are likely to be very small.

Shallow Ground Rupture – Evidence of active fault rupture was not observed within the explored areas of the site at the time of our subsurface exploration and as noted above, PSI did not observe any mapped faults crossing the site in readily available resources. As such, the potential for ground rupture from faulting at the site is considered to be very low.

Liquefaction – Soil liquefaction and seismically-induced settlement typically occur in saturated loose to medium dense cohesionless soils; and in clays and silts with low plasticity indexes and with moistures near their liquid limits, due to cyclic softening where the groundwater is relatively shallow (within 50 feet of the ground surface). During an earthquake, ground shaking causes a rapid increase in the porewater pressure within the soil mass under undrained conditions. The generation of excess porewater pressures causes a



corresponding decrease in the soil's effective stress, which can result in a sudden loss of soil bearing strength and ground surface settlement within the liquefied (and softened) soil layers. Soil liquefaction potential is generally affected by soil types, mineral contents, ground acceleration, duration of shaking, and frequency content of the earthquake ground motion, among other factors.

The subject site lies within a zone of required investigation for liquefaction hazard for the San Mateo Quadrangle (CGS, 2018). Additionally, the San Mateo County Geotechnical Hazard Synthesis Map (Leighton, 1976) indicates that the site is within an area of variable liquefaction potential and that a detailed liquefaction study is required prior to construction. PSI performed a detailed evaluation of soil liquefaction potential based on the collected CPT data, which included the advancement of 5 CPT probes to depths between approximately 70 and 100 feet bgs. A pore-pressure dissipation test performed in CPT-2 indicated a groundwater level of approximately 6 feet bgs at the time of our field exploration. The historic high groundwater is estimated at 5 feet bgs.

Soil liquefaction potential and seismically-induced settlement was estimated using computer program CLiq (version V.2.3.1.15), developed by Geologismiki Geotechnical Software. The program estimates the extent and depth of liquefaction within the CPT subsurface profile using the method developed by NCEER (1998), corresponding to input ground surface acceleration and earthquake magnitudes consistent with a design-level earthquake event. A magnitude 7.9 earthquake was used for this analysis along with a horizontal ground acceleration of 0.70g and a groundwater level of 5 feet below the existing surface grade. The design-level earthquake magnitude and peak ground acceleration are based on requirements of the 2016 CBC.

The results of this analysis are presented in Appendix C, which showed that several layers of the underlying soils are susceptible to soil liquefaction upon application of the design horizontal ground acceleration. Due to the relative thicknesses of non-liquefiable soils over potentially liquefiable soils, it is PSI's opinion that the chances of surface manifestations in terms of sand boils and loss of bearing, etc. are low. However, soil liquefaction is expected to cause aerial settlements resulting from the induced volumetric strains within the potentially liquefiable soil layers.

The estimated seismically-induced total settlements due to soil liquefaction range from about 0.3 to 3 inches with differential settlements from soil liquefaction estimated up to about 1½ inch over a span of approximately 40 feet. These order of magnitude estimates of settlements that may be induced by soil liquefaction are within tolerable limits for use with conventional shallow footings as noted by the California Geological Survey (CGS) Special Publication 117A.

Landsliding and Slope Stability – Seismically induced landsliding is not considered a hazard on, or adjacent to the project site due to the absence of significant steep slopes in or around the project area.

Flooding - The current Federal Emergency Management Agency flood zone map (FEMA, 2012) indicates that the site is in a 100-year flood zone.



Tsunamis and Seiches – Inundation by tsunamis (seismic or "tidal waves") is not considered to be a significant threat to the subject site due to the fact that San Francisco Bay is a mostly enclosed body of water. The "Mare Island" earthquake in 1898 is reported to have caused a seiche in the northern part of the San Francisco Bay but there are no reports of damage associated with this event. Inundation by seiches ("tidal waves" in confined bodies of water) is not considered to be a significant threat to the subject site due to the absence of historic evidence of this type of event in the area of the site. This generally agrees with the San Mateo County Geotechnical Hazard Synthesis Map which shows the site outside of mapped areas of tsunamis and seiche hazard (Leighton, 1976).

Lurching and Lateral Spreading – Due to the limited thickness of potentially liquefiable soil, the absence of a free face, embankment or scarp, and the relative consistency of the encountered soils, it is our opinion that the potential for lurching and lateral spreading at this site is low.



4.0 GEOTECHNICAL RECOMMENDATIONS

4.1 General

Soil deposits, generally consisting of lean and fat clay, silty sand and sandy silt were encountered at the site. Fine grained soils were observed to be medium stiff to stiff in the upper 30 feet, while coarse soils were observed to be generally medium dense to dense.

The primary geotechnical hazards to the proposed development identified at the site are static and seismic settlements and low bearing soils. Based on the assumed structural loads and the soils encountered, we estimate a static settlement of about 2 to 4 inches, which is in-excess of expected tolerable limits. These hazards are common to sites in the vicinity of the project site and can be mitigated with appropriate engineered design and ground improvement. It is our opinion that the hazards identified should not preclude the development of the proposed structures and that the site is suitable to receive the proposed improvements as long as the recommendations presented in this report are incorporated into design and construction.

While the existing soil conditions at the site are considered unsuitable for the use of shallow foundations for the support of the buildings, it is our opinion that the site soils can be improved to support the proposed buildings on conventional continuous strip or isolated pad footings after an appropriate ground improvement program.

The proposed construction at the site should be performed in accordance with the following recommendations, the current edition of the California Building Code, and local governmental standards which have jurisdiction over this project. Our recommendations have been developed on the basis of the described project characteristics and subsurface conditions encountered. If there are any changes in these project criteria, including project location on the site, a review should be made by PSI to determine if modifications to the recommendations are warranted.

Once final design plans and specifications are available, a general review by PSI is recommended to check that the evaluations made in preparation of this report are correct and that earthwork and foundation recommendations are properly interpreted and implemented.

4.2 Site Preparation

Initial site preparation should include demolition of the existing buildings and site improvements in conflict with the new planned improvements. Demolition should include removal of all conflicting foundations, floor slabs and any other below-grade construction. The locations of any existing underground utility lines within the construction area should be established. Provisions should be made to relocate any interfering utility lines within the construction area to appropriate locations. The development area should also be cleared of surface vegetation, trees, shrubs, and debris. As a minimum, it is recommended the clearing operations extend at least five feet beyond the proposed building and pavement perimeters, where possible. We recommend that at the time of initial site stripping and grading, that PSI be retained to observe the subgrade conditions to verify that no potentially deleterious soils are present. All materials generated by the demolition and stripping operations should be legally



disposed off-site. All grading operations should be performed in accordance with our recommendations, the requirements of the current edition of the CBC, and local governmental standards which have jurisdiction over this project.

4.2.1 Ground Improvement

Due to the expected high structural loads and excessive settlement of the underlying soil, we recommend ground improvement be performed within the building envelope to increase the strength of the relatively low bearing soils. As previously noted, without the recommended ground improvement, an estimated total static settlement on the order of 2 to 4 inches would be expected.

Ground improvement systems such as rammed aggregate piers (RAP's), controlled modulus columns (CMC's), or other similar systems may be implemented. Such systems are typically designed as a performance specification approach by a specialty ground-improvement designer-installer. We recommend the performance-based specification be designed to limit total and differential static settlements to 1-inch and ½-inch over a 40-foot span, respectively. If this option is selected, we recommend that the ground modification plan be reviewed by PSI prior to implementation. Post evaluation is also recommended to confirm that the recommended performance specification static settlement of 1 inch total and ½ inch differential has been achieved. Following successful implementation of the ground improvement program, the planned building could be supported by conventional shallow foundations.

As an alternate to a ground improvement program, the planned building could be supported by a deep foundation system (piles or piers) that is expected would need to extend below a below of 50 feet below grade. Although this option is considered feasible, it is not anticipated to be a cost-effective alternative.

4.2.2 Proof-Rolling of Pavement Subgrade Soils

In proposed new pavement areas, after site clearing and cutting to subgrade level, we recommend the subgrade soils be proof rolled with a heavy rubber-tired piece of construction equipment (minimum 15 ton) approved by and in the presence of the geotechnical engineer. Proof rolling is recommended to include at least 2 passes, one in each perpendicular direction. Any soil that excessively yields or ruts during the proof roll operation should be removed as recommended by the geotechnical engineer.

Following proof rolling and any needed over-excavation, the subgrade soil should be scarified to a minimum depth of 12 inches, moisture-conditioned to 0 to 4 percent above optimum moisture content and recompacted to a minimum 95% of the soil's maximum dry density (based on ASTM Test Method D1557).

4.3 Engineered Fill

Engineered Fill material required at this site should not contain rocks greater than 3-inches in diameter or have greater than 30 percent retained on the ¾-inch sieve and should not contain more than 3 percent (by weight) of organic matter or other unsuitable material. All fill soil should possess a low expansion potential, with an Expansion Index (EI) not exceeding 50. Based on our laboratory testing, which indicates an EI of 36, the near-surface existing on-site clay soils are anticipated to be suitable for use as engineered fill, however, this should be confirmed by a PSI representative during grading. Also, we anticipate that



some drying of the site soils will be necessary prior to their use as engineered fill. Samples of on-site and import materials should be tested and approved by the Geotechnical Engineer prior to use as Engineered Fill on-site.

Engineered Fill should be compacted to at least 90 percent of the maximum dry density as determined by the modified Proctor (ASTM D1557). The moisture content of Engineered Fill should be maintained within about 0 to 4 percent above the fill's optimum moisture content (clayey soils), as determined by the same index during compaction. If the Engineered Fill is too dry, water should be uniformly applied across the affected fill area. If the Engineered Fill is too wet, it must be dried. In either event, the Engineered Fill should be thoroughly mixed by diskings to obtain relatively uniform moisture content throughout the lift immediately prior to compaction.

Engineered Fill should be placed in maximum lifts of 8-inches of loose material. Each lift of Engineered Fill should be tested by a PSI soils technician, working under the direction of our Project Geotechnical Engineer, prior to placement of subsequent lifts.

Compaction of the backfill should be checked with a sufficient number of density tests by a representative of the Geotechnical Engineer to determine if adequate compaction is being achieved by the contractor. The properly compacted Engineered Fill should extend horizontally outward beyond the exterior perimeter of the foundations, or pavements, a distance equal to the height of fill prior to significant sloping.

4.4 Excavations

Excavation and construction operations for the foundations may expose the on-site soils to inclement weather conditions. The stability of exposed soils will rapidly deteriorate due to drying or wetting or the action of heavy or repeated construction traffic. Accordingly, foundation area excavations and pavement subgrade areas should be adequately protected from the elements, and from the action of repetitive or heavy construction loading.

4.4.1 Excavations/Slopes

Any permanent cut or fill slopes should not exceed 2 Horizontal to 1 Vertical (2H:1V). Excavations extending below a 1H:1V plane extending down from any adjacent footings should be shored for safety. All excavations should be inspected by a representative of the geotechnical engineer during construction to allow any modifications to be made due to variation in the soil types. All work should be performed in accordance with Department of Labor Occupational Safety and Health Administration (OSHA) guidelines. Job site safety is the responsibility of the project contractor.

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, subpart P". This document was issued to better insure the safety of personnel entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations, or footing excavations, be constructed in accordance with the new OSHA guidelines.



The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person," as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal state regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

4.4.2 Utility Trench Backfill

Excavations should be performed in accordance with applicable Federal and State Occupational Safety and Health Administration regulations. Utility trenches in the soils at the site below a depth of about 4 feet will need to be sloped or shored from the ground surface due to the potential for caving. Actual inclinations will ultimately depend on the soil conditions encountered during earthwork. While we may provide certain approaches for trench excavations, the contractor should be responsible for selecting the excavation technique, monitoring the trench excavations for safety, and providing shoring, as required, to protect personnel and adjacent improvements. The information provided below is for use by the owner and engineer and should not be interpreted to mean that PSI is assuming responsibility for the contractor's actions or site safety. The soils encountered in our subsurface explorations should be classified as Type C soil for temporary slopes, provided groundwater is not observed. Though not anticipated, if groundwater is observed in excavation areas PSI should be notified and allowed to reassess our temporary sloping recommendations. In our opinion, excavations should be safely sloped or shored. The contractor should be aware that excavation and shoring should conform to the requirements specified in the applicable local, state, and federal safety regulations, such as OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations. We understand that such regulations are being strictly enforced, and if not followed, the contractor may be liable for substantial penalties.

Excavation and construction operations may expose the on-site soils to inclement weather conditions. The stability of exposed soils may deteriorate due to a change in moisture content or the action of heavy or repeated construction traffic. Accordingly, foundation and pavement area excavations should be protected from the elements and from the action of repetitive or heavy construction loadings.

Utility trenches below the slab, footings and pavement areas should be backfilled with engineered fill above bedding and shading fill. Bedding and shading fill around utilities is typically performed with granular soil according to local recommendations. Where utilities cross building perimeters, slurry or concrete should be used for backfill around the utility to prevent moisture from migrating along the utility trench and entering the building envelope.



4.4.3 Dewatering

Based on the presence of shallow groundwater and dependent upon the planned excavation depths, some dewatering may be necessary. For excavations extending not more than about 2 feet below the water table, the use of conventional sump pumps placed in lined pits is anticipated to be suitable. Excavations extending more than a couple feet below the water table may require a more elaborate dewatering system, such as multiple well points. If needed, specialized dewatering system should be determined by a dewatering contractor.

4.5 Foundations

Following site preparation and ground improvement, as recommended in Section 4.2, it is our opinion that conventional continuous and isolated spread footings are suitable for the support of the proposed 3- to 6-story structure. Continuous footings should be founded at least 18 inches below lowest adjacent finished grade and have a minimum width of 18 inches. Isolated spread footings should be at least 24 inches wide and supported at a depth of 24 inches below the lowest adjacent grade. The allowable soil bearing capacity to be used for foundation support is typically determined by the specialty contractor based on the ground improvement program implemented. Typically, an allowable soil bearing pressure in the range of 3,000 to 6,000 psf is suitable following ground improvement. It is recommended that PSI review the planned ground improvement program and the allowable soil bearing capacity to be used for design.

The allowable soil bearing pressure may be increased by one-third for loads of short duration, including wind and seismic forces. Appropriate foundation reinforcement and foundation dimensions should be provided in accordance with the Structural Engineer's design and in consideration of the low expansion potential of the site soils.

Lateral loads may be resisted by any rational method, which incorporates sliding friction and/or passive earth pressure. The design may incorporate an allowable passive earth pressure of 250 psf per foot of depth below a depth of one foot, provided that the footing concrete is poured tightly against firm native soil or properly compacted fill materials. An allowable friction coefficient of 0.30 may be used at the concrete-soil interface. No reductions are necessary when combining the frictional and passive resistance of the soils to determine the total lateral resistance.

The foundation excavations should be observed by a representative of PSI prior to steel or concrete placement to assess that the foundation materials are capable of supporting the design loads and are consistent with the materials discussed in this report. Soft or loose soil zones encountered at the bottom of the foundation excavations should be removed as directed by the geotechnical engineer.

After opening, foundation excavations should be observed by PSI and concrete placed as quickly as possible to avoid exposure of the foundation bottoms to wetting and drying. Surface run-off water should be drained away from the excavations and not be allowed to pond. If possible, the foundation concrete should be placed during the same day the excavation is made. If it is required that foundation excavations be left open for more than one day, they should be protected to reduce evaporation or entry of moisture.



We anticipate that foundations supported on improved ground, designed and constructed in accordance with the above recommendations, and following post-installed verification testing, will experience an estimated total static settlement on the order of 1-inch with a differential settlement on the order of ½-inch over a distance of approximately 40 feet within the building area. While settlement of this magnitude is generally considered tolerable for structures of the type proposed, the design of masonry walls should include provisions for liberally-spaced, vertical control joints to minimize the effects of cosmetic "cracking". The tolerance of the proposed structures to the predicted total and differential settlements should be confirmed by the Structural Engineer. Utilities connections should consist of flexible joints to account for earthquake-induced settlements below the treated zone.

4.6 Concrete Slabs-On-Grade

Following ground improvement, the building floor slab may consist of a conventional slab supported by the underlying improved soil. As a minimum, we recommend that the concrete floor slab possess a minimum thickness of 4 inches and be reinforced as determined necessary by the structural engineer. If desired, the slab section may be designed by the structural engineer using a coefficient of subgrade reaction of 150 pci based on a one-foot plate bearing test. Care should be taken by the contractor to ensure that the reinforcement is placed and maintained at slab mid-height. Floor slabs should be suitably reinforced and jointed so that a small amount of independent movement can occur without causing damage.

Should a below grade level floor slab be planned, the installation of an under slab drainage system is recommended. A typical underdrain system would include a minimum 6-inch-thick, free-draining aggregate base course. The base course will serve as a drainage course and it should, therefore, underlie the entire floor slab and it should be continuous to an interior footing drain pipes so that water below the slab adequately discharges into the drainage system. The need and specific recommendation for the slab under drain system should be determined once final base floor elevations are known.

Interior floor slabs should be underlain by a capillary break at least 4 inches in thickness, consisting of clean sand. In moisture-sensitive flooring areas or areas used to store moisture-sensitive materials, such as carpeted or linoleum covered areas or storage areas, a 15-mil visqueen moisture retarder should be placed beneath the slab at mid-height within the capillary break material. The visqueen sheet should be sealed along the edges to prevent lateral migration of soil moisture from adjacent non-visqueen areas. Prior to placement of clean sand and slab-on-grade, the visqueen sheet should be thoroughly inspected for cracks, punctures, tears, and holes. If necessary, the visqueen should be replaced or patched to assure a fully functional entity.

Some minor cracking of slabs can be expected due to shrinkage. The potential for this slab cracking can be reduced by careful control of water/cement ratios in the concrete. The contractor should take appropriate curing precautions during the pouring of concrete in hot weather to reduce cracking of slabs. We recommend that a slipsheet (or equivalent) be utilized if grouted fill, tile, or other moisture-sensitive floor covering is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations. The floor slab should be liberally jointed in accordance with American Concrete Institute (ACI) guidelines to help control cracking, resulting from differential movement and concrete shrinkage.



4.7 Below Grade Walls and Retaining Walls

Although not anticipated, retaining walls may be supported by conventional shallow continuous (strip) footings bearing on ground improved soil and may be designed as recommended for the building.

The following lateral earth pressures should be used for the design of retaining walls and below grade walls backfilled with suitable granular soils.

Table of Equivalent Fluid Weight (pcf)		
Wall Type	Level Backfill	2:1 Sloped Backfill (Ascending)
Active	38	60
At-Rest (fixed at top)	55	70
Passive	250	175

The above values assume backfill soils will have a very low expansion potential ($EI < 20$) and free-draining condition. If conditions other than those covered herein are anticipated, the geotechnical engineer should provide the equivalent fluid pressures on an individual basis.

Below-grade walls and retaining walls should include a positive foundation drainage system. A typical wall drain consists of a minimum 4-inch diameter rigid perforated pipe surrounded by at least one foot of $\frac{3}{4}$ -inch crushed rock per foot and wrapped in a non-woven geotextile fabric (consisting of Mirafi 140N or approved equivalent). This system typically is installed directly on top of the retaining wall footing on the retained soil side of the wall. Perforations in the drain pipe should be placed facing down. The gravel pack around the pipe should be brought up to within one foot of the soil surface. The subsurface drainage system should be tied to the storm drainage system, allowed to daylight down slope, or collected in a sump and pumped out. Cleanouts should be installed at regular intervals and each bend of the drainage pipe.

Retaining wall backfill should consist of approved very low expansive ($EI < 20$) granular material. This fill material should be compacted to at least 90% of the maximum dry density (as determined by ASTM D1557). Flooding or jetting of the backfill should not be permitted. Granular backfill should be capped with relatively impervious fill or pavement to seal the backfill and reduce the potential for saturation.

Cantilever or restrained walls subject to uniform surcharge loads should be designed for an additional uniform lateral pressure equal to one-third the anticipated surcharge pressure in the case of the cantilevered walls (active conditions). It should be noted that the use of heavy compaction equipment in close proximity to retaining structures can result in wall pressures exceeding design values and corresponding wall movement greater than normally associated with the development of active conditions. In this regard, the contractor should take appropriate precautions during the backfill placement.

Lateral soil resistance developed against lateral structural movement should be calculated in accordance with the recommendations in the shallow foundations section herein.



If retaining walls over 6 feet are proposed, the CBC requires that seismic loading must be considered. Based on a PGA_M value of 0.7 and using the Mononobe-Okabe method, we recommend an incremental seismic pressure of 21.9 pcf be incorporated if/where needed. This seismic pressure can be applied as a conventional triangular distribution.

4.8 Drainage Considerations

Surface water must not be allowed to pond or discharge adjacent to the foundations. To preclude drainage problems, we recommend continuous roof gutters for the proposed structure. We recommend that roof drains be connected to a tight-line pipe leading to storm drain facilities or other suitable discharge locations. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend that ground surfaces adjacent to buildings be sloped to facilitate positive drainage away from the buildings.

A positive slope gradient of 5 percent down and away from the building perimeters should be applied to the finished subgrade (inclusive of topsoil). This slope should extend no less than 10 feet away from the outside building perimeter, with drainage swales provided to remove runoff from around the structure. Any utility trench that enters the perimeter of a structures should be excavated with a slight slope down and away from the perimeter of the structure.

Landscaped or planted areas and irrigation should not be placed within 10 feet of the footings of the proposed structure. Trees and shrubs should be positioned away from the building equal to half of their mature height. Hardscape should surround the building as close as possible. Where concrete flat work such as sidewalks are placed next to the structure, concrete should be placed adjacent to the foundation to prevent a planter strip that would trap surface water between the foundation and the sidewalk. For vegetation planted near the building, plants that require very little moisture should be used. Irrigation systems (drip and/or sprinkler heads) should not direct water where it could saturate foundation soil. If landscaping is desired closer to the building, moisture barriers may be added to limit moisture infiltration beneath the building.

Water should not be allowed to collect in the foundation excavation, on floor slab areas, or on prepared subgrades of the construction area either during or after construction. Positive site drainage away from excavation areas should be established to minimize the flow of surface runoff or rainwater into the excavations. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff.



4.9 Pavement Recommendations

Preparation of the subgrade soils for new pavements should be prepared in general accordance with the site preparation recommendations (Section 4.2). While specific traffic loads and volumes for the project have not been provided, we are providing recommended light-duty and medium to heavy-duty pavement sections, which have been successfully utilized for this type of development in the project area with similar traffic loading. For these pavement sections, we have utilized a Traffic Index of 5.0 and 7.0 for the light duty and medium to heavy-duty sections, respectively. R-value testing of a representative bulk soil sample, collected from B-4 at 0 to 4 feet, indicates an R-Value for the near-surface clay of 9.

Asphaltic Concrete (AC):

Light Duty (Automobile Parking; TI=5.0)

3 inches Asphalt Concrete (Caltrans Standard Specs. Section 39)

9 inches Class II Aggregate Base (Caltrans Standard Specs. Section 26)

Medium to Heavy-Duty (Entrance and Drive Lanes; TI=7.0)

4 inches Asphalt Concrete (Caltrans Standard Specs. Section 39) over

15 inches Class II Aggregate Base (Caltrans Standard Specs. Section 26)

or

5 inches Asphalt Concrete (Caltrans Standard Specs. Section 39) over

13 inches Class II Aggregate Base (Caltrans Standard Specs. Section 26)

All aggregate base and the upper 12 inches of subgrade should be compacted to at least 95 percent of the maximum dry density as determined by ASTM Designation D1557. All materials and methods of construction should conform to good engineering practices and be in conformance with the requirements of the local jurisdiction.

As an alternate, concrete pavement could also be used at the site. A rigid pavement section merits consideration for areas to receive relatively high concentrated sustained loads such as loading / storage areas or trash dumpster approaches and enclosures. Based on the near surface soil encountered in the borings, it is our opinion that a modulus of subgrade reaction (k) of 125 pounds per square inch per inch (psi/in) is suitable for all the concrete pavement sections, given the presence of the underlying base course. Based on this, we offer the following concrete pavement recommendations:

Portland Cement Concrete (PCC):

Light Duty Section (TI=5.0)

- 5½ inches Portland Cement Concrete
- 4 inches Class II Aggregate Base

Medium to Heavy Duty Section (TI=7.0)

- 6 inches Portland Cement Concrete
- 4 inches Class II Aggregate Base



Based on our local experience, rigid concrete pavements are considered to be a part of the civil site work package and the concrete mix design specifications and rebar reinforcement detailing is developed as part of the project specifications, typically by the Civil Engineer. Minimum cement contents and cementitious material replacement specifications should consider the time of year for concrete placement for optimal material performance. The design project engineer of record is best qualified to be familiar with the project schedule and to establish those parameters. Making some typical assumptions, however, PSI provides the following recommendations.

PSI recommends that the concrete should have a minimum 28-day compressive strength of 4,000 psi. The aggregate base course and the upper 12 inches of subgrade should be compacted to at least 95% of the soil's maximum dry density, per ASTM D-1557. The concrete pavements should be properly reinforced and jointed (per ACI requirements).

Saw cut control joints should be placed at maximum 15-foot intervals and should be cut at a depth of at least one-third of the pavement thickness. Saw cut control joints spaced at 10 feet usually control cracking better than the 15-foot interval. Joints should be sawed within 12 hours of concrete placement, and preferably sooner. All joint spacing in large pavement areas should be placed in accordance with the ACI standard or other local requirements, if stricter than those set by ACI.

Expansion joints should be used wherever the pavement will abut a structural element subject to a different magnitude of movement, such as: light poles, retaining walls, or manholes. Expansion joints should be sealed with a polyurethane sealant so that moisture infiltration into the subgrade soils and resultant concrete deterioration at the joints is minimized.

4.9.1 General Pavement Notes

The above recommended pavement sections represent minimum design thicknesses and, as such, periodic maintenance should be anticipated. Also, these recommended pavement sections should be confirmed or modified by your Civil Engineer, based on actual traffic and the owner's requirements. The pavement section materials and construction should comply with Caltrans Standard Specifications and local municipality requirements.

Where pavement areas are adjacent to heavily watered landscaping areas, we recommend some measure of moisture control be taken to prevent the subgrade soils from becoming saturated. It is recommended that the concrete curbing adjacent to landscape areas extend into the prepared subgrade to reduce the potential for irrigation water to saturate subgrade soils.

The pavement thicknesses shown on the boring logs are approximate measurements and represent the conditions only at the actual boring locations at the time of our exploration. Variations may occur and should be expected between boring locations. Additionally, geotextile paving fabric in the pavement section is difficult to discern during drilling and may or may not be present in the existing pavements. As such, the pavement description given in this report is not intended to be used to evaluate the suitability or estimate the quantity for disposal.



4.10 Corrosivity

Testing was performed to evaluate the corrosivity of the on-site soils and the potential for attack on concrete and subsurface utility pipes, specifically cast iron and ductile iron. The testing included pH, sulfate, chloride and electrical resistivity. The results of the chemical analysis are as follows:

Boring Number	Sample Depth (feet)	pH	Resistivity (ohm-cm)	Water Soluble Sulfate (ppm)	Water Soluble Chloride (ppm)
B-1	0 to 4	7.6	1,200	202	17.1

Laboratory testing of a selected soil sample indicates that the on-site soils possess a **negligible** sulfate and chloride exposure and slightly alkaline pH, indicating a **low degree** of corrosivity with respect to concrete, based on ACI 318. Based on this result, it is our opinion that special sulfate-resistant concrete mix designs are not warranted and that the use of Type I or II cement is suitable for concrete in contact with on-site soils. Final concrete mix designs should be evaluated after sulfate tests have been performed on the actual subgrade material.

Corrosivity testing was also performed to determine whether the on-site soils have the potential to attack subsurface utility pipes, specifically cast iron and ductile iron based on chloride and resistivity testing. While the chloride levels suggest that the site soils are non-corrosive according to Caltrans specifications, based on the resistivity test results, the soils are characterized as being **highly corrosive** to cast iron or ductile iron piping (Roberge, 2000). PSI does not practice in the field of corrosion engineering. We recommend that a qualified corrosion engineer be consulted to determine if special corrosion protection is warranted for this site. Testing for corrosivity of any fill soils should be conducted during site grading to verify our recommendations.

4.11 Construction Monitoring

It is recommended that PSI be retained to examine and identify soil exposures created during project construction to document that soil conditions are as anticipated. We further recommend that any Engineered Fills be continuously observed and tested by our representative to evaluate the thoroughness and uniformity of their compaction. If possible, samples of fill materials should be submitted to our laboratory for evaluation prior to placement of fills on site. Costs for the recommended observations during construction are beyond the scope of this current consultation.



5.0 GENERAL

Our conclusions and recommendations described in this report are subject to the following general conditions:

5.1 Use of Report

This report is for the exclusive use of Holly Hotel Group, LLC and their representatives to use for the design of the proposed structures described herein and preparation of construction documents. The data, analyses, and recommendations may not be appropriate for other structures or purposes. We recommend that parties contemplating other structures or purposes contact us. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report.

After the plans and specifications are more complete, the geotechnical engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents.

5.2 Limitations

The recommendations contained in this report are based on the available subsurface information obtained by PSI, and design details furnished for the proposed project. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation recommendations are required. If PSI is not retained to perform these functions, PSI will not be responsible for the impact of those conditions on the project.

PSI did not provide any service to investigate or detect the presence of moisture, mold or other biological contaminants in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. Client acknowledges that mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Client further acknowledges that site conditions are outside of PSI's control, and that mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot and shall not be held responsible for the occurrence or recurrence of mold amplification.

Services performed by PSI for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in this area. No warranty, expressed or implied, is made.



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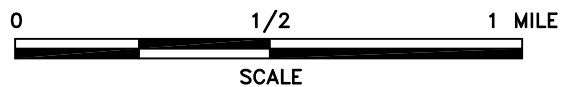
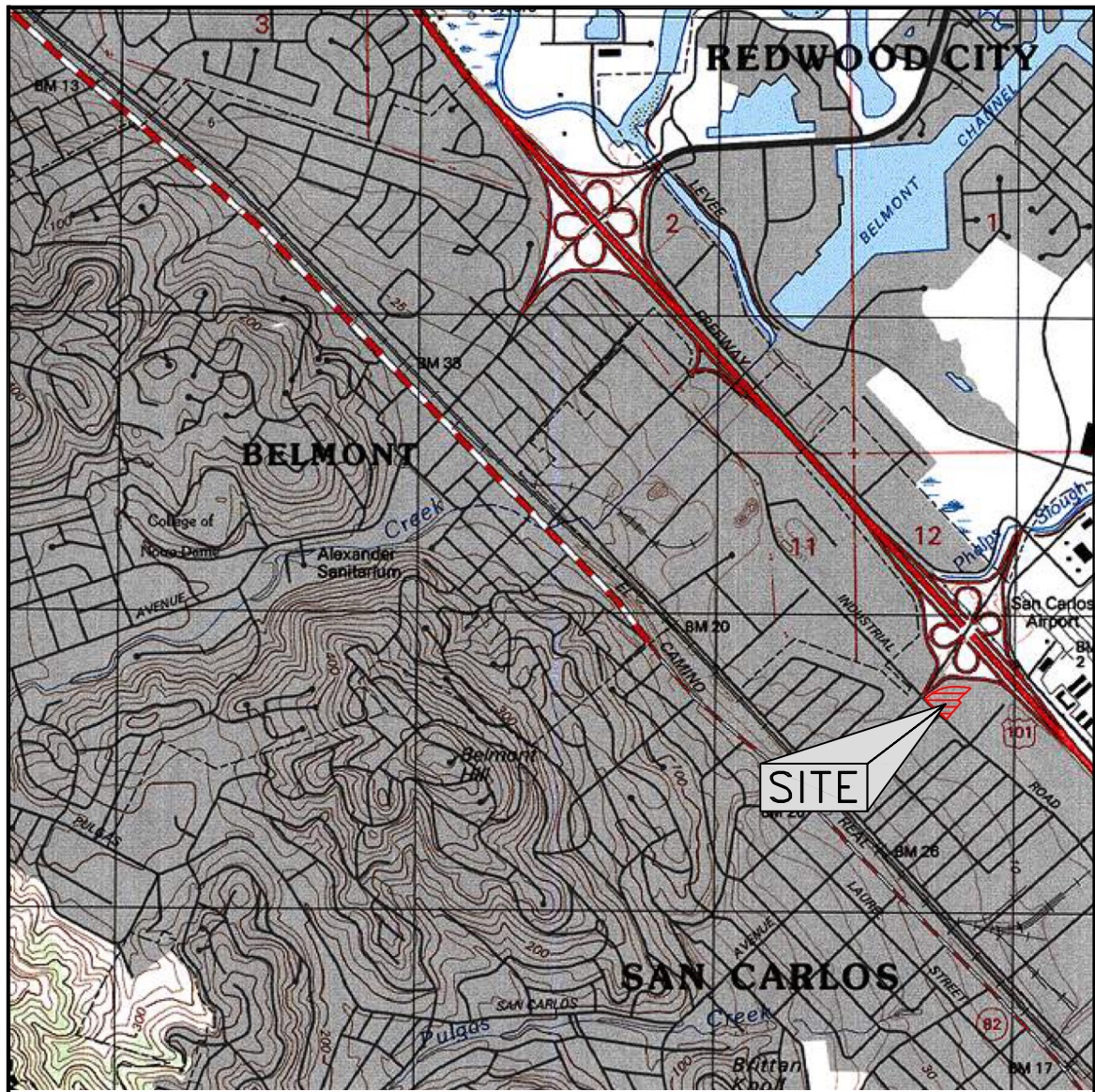
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FIGURES



REFERENCE:

U.S.G.S. SAN MATEO,
CALIFORNIA, 7.5 MINUTE
SERIES TOPOGRAPHIC MAP,
DATED 1997.

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Total Quality. Assured.

4703 Tidewater Avenue, Suite B
Oakland, California 94601
(510) 434-9200

Project Name: PROPOSED HOTEL
501 INDUSTRIAL ROAD, SAN CARLOS, CALIFORNIA

Title: SITE LOCATION MAP

Drawn By:
Z.J.

Date:
9/19

File No.:
1547-1-1

Figure No.:

Approved By:
D.A.

Project No.:
575-1547-1

1



LEGEND

-  - PROPERTY BOUNDARY
-  - PROPOSED BUILDING ENVELOPE
-  - BORING LOCATION
-  - CONE PENETRATION TEST LOCATION

0 90 180ft



APPROXIMATE SCALE

NOTES

1. PROPOSED IMPROVEMENTS TAKEN FROM C.Y. LEE & PARTNERS, "CONCEPT DESIGN, HOLLY GARDEN INN, SAN CARLOS, CALIFORNIA," UNDATED.
2. ALL LOCATIONS SHOWN ARE APPROXIMATE.

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Oakland, California 94601
(510) 434-9200

PROPOSED HOTEL
501 INDUSTRIAL ROAD, SAN CARLOS, CALIFORNIA

SITE PLAN AND BORING
LOCATION MAP

Drawn By:
Z.J.

Date:
9/19

File No.:
1547-1-2

Approved By:
D.A.

Project No.:
575-1547-1

Figure No.:
2



APPENDIX A

EXPLORATION LOGS AND DRILLING PERMIT

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.	SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
HSA: Hollow Stem Auger - typically 3 1/4" or 4 1/4" I.D. openings, except where noted.	ST: Shelby Tube - 3" O.D., except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry	BS: Bulk Sample
R.C.: Diamond Bit Core Sampler	PM: Pressuremeter
H.A.: Hand Auger	CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings
P.A.: Power Auger - Handheld motorized auger	

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N ₆₀ : A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q _u : Unconfined compressive strength, TSF
Q _p : Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL), %
DD: Dry unit weight, pcf
▼, ▼, ▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS ANGULARITY OF COARSE-GRAINED PARTICLES

Relative Density	N - Blows/foot	Description	Criteria
Very Loose	0 - 4	Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Loose	4 - 10	Subangular:	Particles are similar to angular description, but have rounded edges
Medium Dense	10 - 30	Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Dense	30 - 50	Rounded:	Particles have smoothly curved sides and no edges
Very Dense	50+		

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (3/4 in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to 3/4 in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

<u>Q_u - TSF</u>	<u>N - Blows/foot</u>	<u>Consistency</u>
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Medium Stiff
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30+	Hard

MOISTURE CONDITION DESCRIPTION

<u>Description</u>	<u>Criteria</u>
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

<u>Descriptive Term</u>	<u>% Dry Weight</u>
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

<u>Description</u>	<u>Criteria</u>	<u>Description</u>	<u>Criteria</u>
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick	Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick	Lensed:	Inclusion of small pockets of different soils
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing	Layer:	Inclusion greater than 3 inches thick (75 mm)
Slickensided:	Fracture planes appear polished or glossy, sometimes striated	Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
		Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

<u>Q_u - TSF</u>	<u>Consistency</u>
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

<u>Description</u>	<u>Criteria</u>
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

<u>Voids</u>	<u>Void Diameter</u>
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)	
<u>Component</u>	<u>Size Range</u>
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

<u>Rock Mass Description</u>	<u>RQD Value</u>
Excellent	90 - 100
Good	75 - 90
Fair	50 - 75
Poor	25 - 50
Very Poor	Less than 25

DEGREE OF WEATHERING

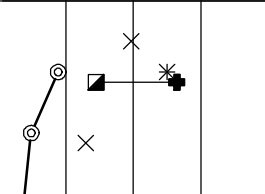
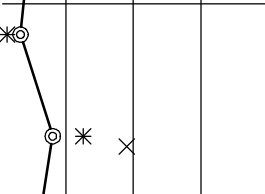
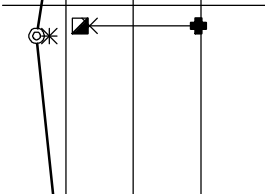
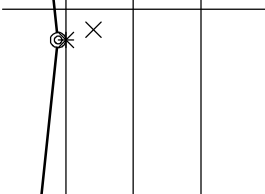
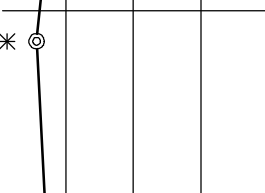
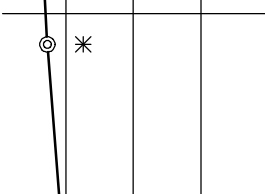
Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

Water		While Drilling	7.5 feet
			
			

STANDARD PENETRATION TEST DATA N in blows/ft ☉				Additional Remarks
× Moisture ▣ PL ⊕ LL				
0 25 50				
STRENGTH, tsf				
▲ Qu * Qp				
0 2.0 4.0				
				LL = 33 PL = 18 Expansion Index=36 DD = 116 pcf
				DD = 102 pcf
				LL = 37 PL = 15
				Passing #200=54%
				
				



DATE STARTED: 8/29/19 DATE COMPLETED: 8/29/19 COMPLETION DEPTH: 51.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 37.5119° LONGITUDE: -122.2569° STATION: N/A OFFSET: N/A REMARKS:		DRILL COMPANY: HEW Drilling DRILLER: Perfecto LOGGED BY: Z. Jacob DRILL RIG: CME 75 DRILLING METHOD: Hollow Stem Auger SAMPLING METHOD: SS; 3" SPT & SS HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: B. Burfield		BORING B-1 Water ▽ While Drilling ▼ ▽ 7.5 feet								
BORING LOCATION:												
Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft @ × Moisture ■ PL ▲ Qu * Qp 0 25 50 0 2.0 4.0		Additional Remarks
	30			9	18	Fat CLAY, medium olive, moist, stiff, few sand	CH	3-4-8 N=12	19	LL = 51 PL = 15		
	35			10	18	becomes very stiff		4-10-17 N=27				
						Silty SAND, light brown, very moist, medium dense, fine to medium sand	SM					
	40			11	18	Sandy SILT, olive, wet, very stiff, fine sand, some clay	ML	5-5-15 N=20				
	45			12	18	becomes very moist		3-10-15 N=25	19			
	50			13	18	Silty SAND, dark olive, very moist, dense, fine to coarse sand	SM	7-17-30 N=47		Passing #200=50%		
					End of boring at 51-1/2 feet below grade. Groundwater was encountered at 7-1/2 feet. Borehole was backfilled with cement grout. Calif. sampler N Values were converted to SPT-equivalent (Calif. *0.69).							

Professional Service Industries, Inc.
 4703 Tidewater Avenue, Suite B
 Oakland, CA 94601
 Telephone: (510) 434-9200

PROJECT NO.: 575-1547
PROJECT: Holly Hotel Group, LLC
LOCATION: Proposed Hotel Indigo
 501 Industrial Road
 San Carlos, California

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	STANDARD PENETRATION TEST DATA N in blows/ft ©				Additional Remarks	
									Moisture, %	STRENGTH, tsf				
										×	Moisture	Moisture		Moisture
	0			1	18	Sandy GRAVEL, grayish-brown, damp, very dense, fine to coarse gravel (FILL)	GP	27-70-40 N=76						
				2	18	Fat CLAY, dark brown, very moist, medium stiff, few decayed organics, trace coarse sand	CH	2-3-4 N=7	42					
				3	18	becomes moist, medium stiff		2-3-5 N=5	30					
	5					End of boring at 6-1/2 feet below grade. Groundwater was not encountered. Borehole was backfilled with cement grout. Calif. sampler N Values were converted to SPT-equivalent (Calif. *0.69).								

	Professional Service Industries, Inc.	PROJECT NO.:	575-1547
	4703 Tidewater Avenue, Suite B	PROJECT:	Holly Hotel Group, LLC
	Oakland, CA 94601	LOCATION:	Proposed Hotel Indigo
	Telephone: (510) 434-9200		501 Industrial Road San Carlos, California

DATE STARTED: 8/29/19 DATE COMPLETED: 8/29/19 COMPLETION DEPTH: 6.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 37.512° LONGITUDE: -122.2562° STATION: N/A OFFSET: N/A REMARKS:		DRILL COMPANY: HEW Drilling DRILLER: Perfecto LOGGED BY: Z. Jaco DRILL RIG: CME 75 DRILLING METHOD: Solid Flight Auger SAMPLING METHOD: SS; 3" SPT & SS HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: B. Burfield		BORING B-3 Water ▽ ▼ ▽ Not Encountered	
BORING LOCATION:					

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft @				Additional Remarks
										Moisture, % 0 25 50 X Moisture PL LL				
										STRENGTH, tsf ▲ Qu * Qp 0 2.0 4.0				
0		2 inches asphalt												
		Silty SAND, dark brown, damp to moist, medium dense (FILL)		1	18		SM	9-9-16 N=25						
		Fat CLAY, medium to dark olive, moist, medium stiff, few organics		2	18		CH	3-3-3 N=4	29			X		DD = 65 pcf
5		some roots and organics		3	18			3-4-4 N=6	41			X		DD = 81 pcf Odoriferous
						End of boring at 6-1/2 feet below grade. Groundwater was not encountered. Borehole was backfilled with cement grout. Calif. sampler N Values were converted to SPT-equivalent (Calif. *0.69).								

	Professional Service Industries, Inc. 4703 Tidewater Avenue, Suite B Oakland, CA 94601 Telephone: (510) 434-9200	PROJECT NO.: 575-1547 PROJECT: Holly Hotel Group, LLC LOCATION: Proposed Hotel Indigo 501 Industrial Road San Carlos, California
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DATE STARTED: 8/29/19 DATE COMPLETED: 8/29/19 COMPLETION DEPTH: 6.5 ft BENCHMARK: N/A ELEVATION: N/A LATITUDE: 37.5115° LONGITUDE: -122.2568° STATION: N/A OFFSET: N/A REMARKS:		DRILL COMPANY: HEW Drilling DRILLER: Perfecto LOGGED BY: Z. Jacob DRILL RIG: CME 75 DRILLING METHOD: Solid Flight Auger SAMPLING METHOD: SS; 3" SPT & SS HAMMER TYPE: Automatic EFFICIENCY: N/A REVIEWED BY: B. Burfield		BORING B-4 Water ▽ Not Encountered ▽ ▽ BORING LOCATION:	
---	--	--	--	--	--

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft @	Additional Remarks
0		2 inches asphalt									
				1	18	Lean CLAY, dark olive, damp to moist, very stiff, some organics, few coarse sand (FILL)		11-16-10 N=26			
				2	18	becomes moist to very moist, medium stiff, trace fine sand	CL	4-5-6 N=7			LL = 41 PL = 18 R-Value = 9
				3	18	becomes dark brown, moist, stiff, trace fine to medium sand, few organics		4-9-12 N=14			DD = 85 pcf
						End of boring at 6-1/2 feet below grade. Groundwater was not encountered. Borehole was backfilled with cement grout. Calif. sampler N Values were converted to SPT-equivalent (Calif. *0.69).					DD = 106 pcf

	Professional Service Industries, Inc. 4703 Tidewater Avenue, Suite B Oakland, CA 94601 Telephone: (510) 434-9200		PROJECT NO.: 575-1547 PROJECT: Holly Hotel Group, LLC LOCATION: Proposed Hotel Indigo 501 Industrial Road San Carlos, California
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Cone Penetration Testing Procedure (CPT)

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

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

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

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

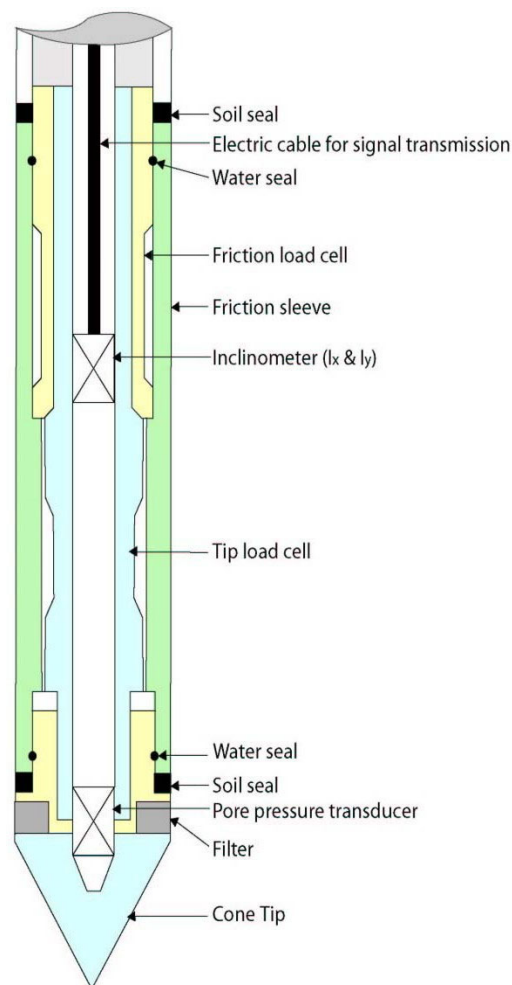


Figure CPT

Gregg 15cm² Standard Cone Specifications

Dimensions	
Cone base area	15 cm ²
Sleeve surface area	225 cm ²
Cone net area ratio	0.85
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

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

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (2009 & 2010). Typical plots display SBT based on the non-normalized charts of Robertson (2010). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (2009) which can be displayed as SBTn, upon request. The report can also include spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Robertson and Cabal (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface. Note that it is not always possible to clearly identify a soil type based solely on q_t , f_s , and u_2 . In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.

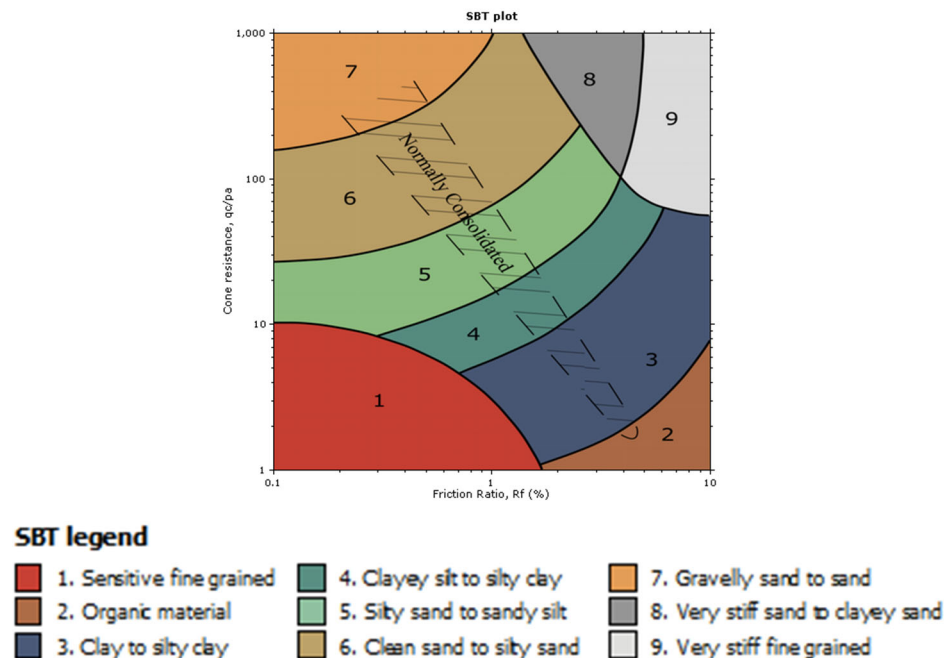


Figure SBT (After Robertson, 2010) – Note: Colors may vary slightly compared to plots

Cone Penetration Test (CPT) Interpretation

Gregg uses a commercial CPT interpretation and plotting software (CPeT-IT <https://geologismiki.gr/products/cpet-it/>). The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997) and updated by Robertson and Cabal (2015). The interpretation is presented in tabular format. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

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

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log\left(\frac{q_t}{p_a}\right) + 1.236 \right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$$I_c < 3.27 \text{ and } I_c > 1.00 \text{ then } k = 10^{0.952 - 3.04 I_c}$$

$$I_c \leq 4.00 \text{ and } I_c > 3.27 \text{ then } k = 10^{-4.52 - 1.37 I_c}$$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{p_a} \right) \cdot \frac{1}{10^{1.1268 - 0.2817 I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268 - 0.2817 I_c}}$$

:: Young's Modulus, E_s (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 I_c + 1.68}$$

(applicable only to $I_c < I_{c_cutoff}$)

:: Relative Density, D_r (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{OCR}}} \quad \text{(applicable only to SBT}_n\text{: 5, 6, 7 and 8 or } I_c < I_{c_cutoff}\text{)}$$

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,CS})$$

:: Drained Friction Angle, ϕ (°) ::

$$\phi = \phi'_{cv} + 15.94 \cdot \log(Q_{tn,CS}) - 26.88$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$

$a = 14$ for $Q_{tn} > 14$

$a = Q_{tn}$ for $Q_{tn} \leq 14$

$M_{CPT} = a \cdot (q_t - \sigma_v)$

If $I_c \leq 2.20$

$$M_{CPT} = 0.03 \cdot (q_t - \sigma_v) \cdot 10^{0.55 I_c + 1.68}$$

:: Small strain shear Modulus, G_0 (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 I_c + 1.68}$$

:: Shear Wave Velocity, V_s (m/s) ::

$$V_s = \left(\frac{G_0}{\rho} \right)^{0.50}$$

:: Undrained peak shear strength, S_u (kPa) ::

$$N_{kt} = 10.50 + 7 \cdot \log(F_r) \text{ or user defined}$$

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Remolded undrained shear strength, $S_u(rem)$ (kPa) ::

$$S_{u(rem)} = f_s \quad \text{(applicable only to SBT}_n\text{: 1, 2, 3, 4 and 9 or } I_c > I_{c_cutoff}\text{)}$$

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 + 7 \cdot \log(F_r))} \right]^{-1.25} \text{ or user defined}$$

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: In situ Stress Ratio, K_0 ::

$$K_0 = (1 - \sin \phi') \cdot OCR^{\sin \phi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_s}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c_cutoff}$)

:: Peak Friction Angle, ϕ' (°) ::

$$\phi' = 29.5^\circ \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

ASTM D5778-12, 2012, Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils. ASTM West Conshohocken, USA

Lunne, T., Robertson, P.K. and Powell, J.J.M., 1997. Cone Penetration Testing in Geotechnical Practice.

Robertson, P.K., 1990. Soil Classification using the Cone Penetration Test. Canadian Geotechnical Journal, Volume 27: 151-158

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Robertson, P.K., 2010, "Soil Behavior type from the CPT: an update", 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, Vol.2. pp 575-583

Robertson, P.K. and Cabal, K.L., "Guide to Cone Penetration Testing for Geotechnical Engineering", 6th Edition, 2015, 145 p. Free online, <http://www.greggdrilling.com/technical-guides>.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, "Seismic CPT to Measure In-situ Shear Wave Velocity", Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 8, pp. 791-803, 1986.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.



CPT BORINGS

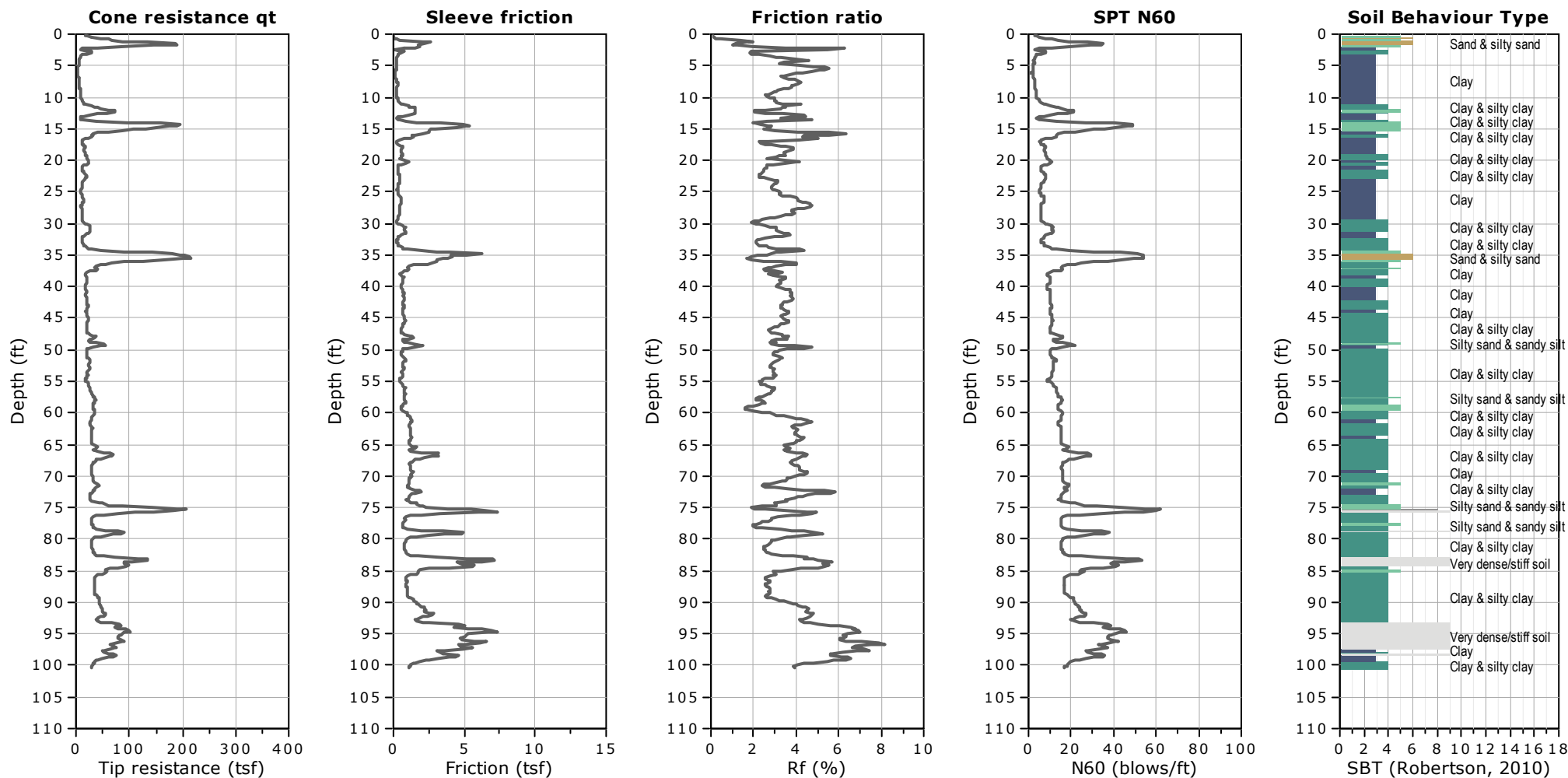


CLIENT: INTERTEK

SITE: 501 INDUSTRIAL ROAD, SAN CARLOS, CA

FIELD REP: BRAND BURFIELD

Total depth: 100.39 ft, Date: 9/5/2019



SBTn legend

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

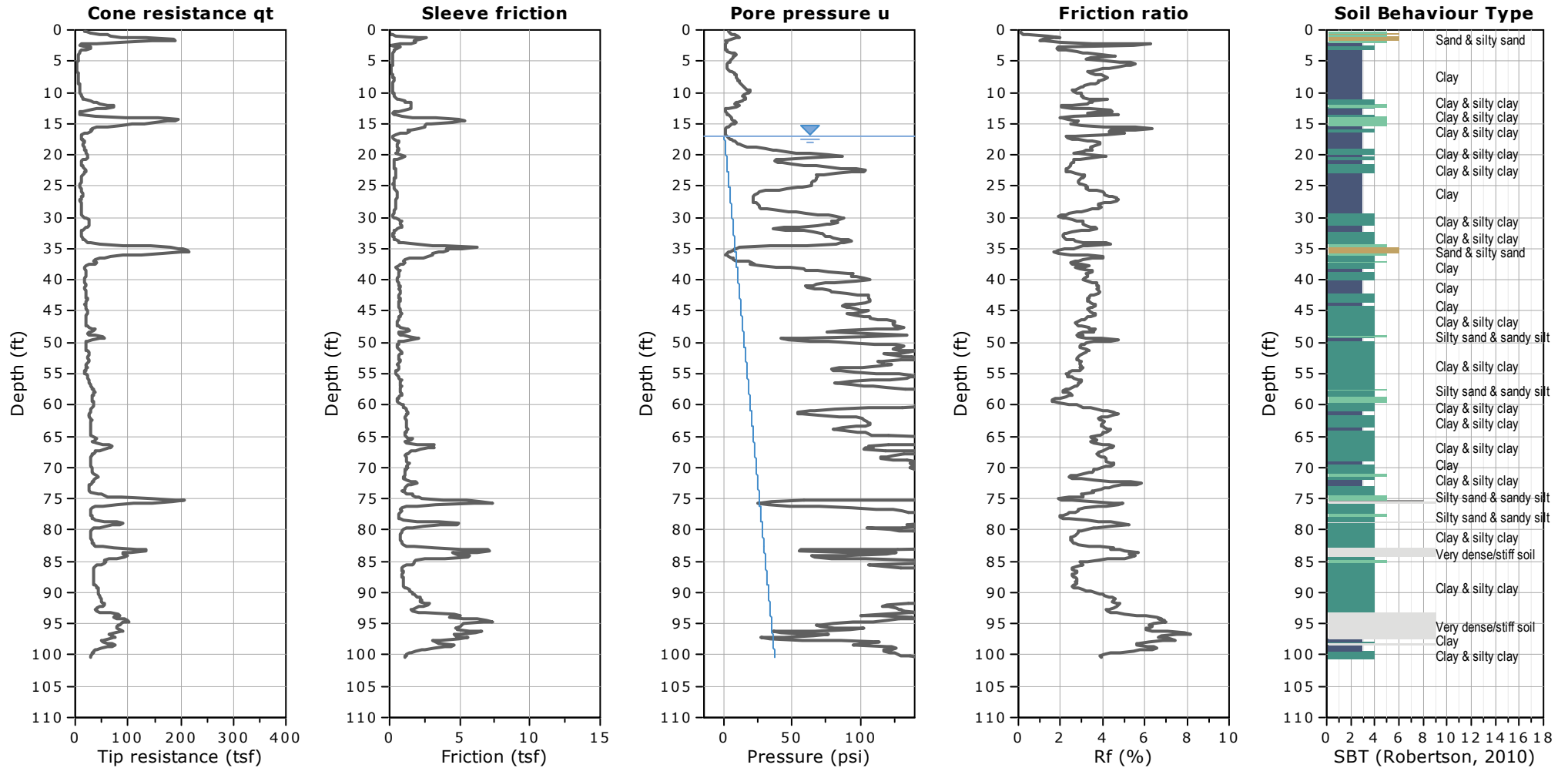


CLIENT: INTERTEK

SITE: 501 INDUSTRIAL ROAD, SAN CARLOS, CA

Field Rep: BRAND BURFIELD

Total depth: 100.39 ft, Date: 9/5/2019



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WATER TABLE FOR ESTIMATING PURPOSES ONLY

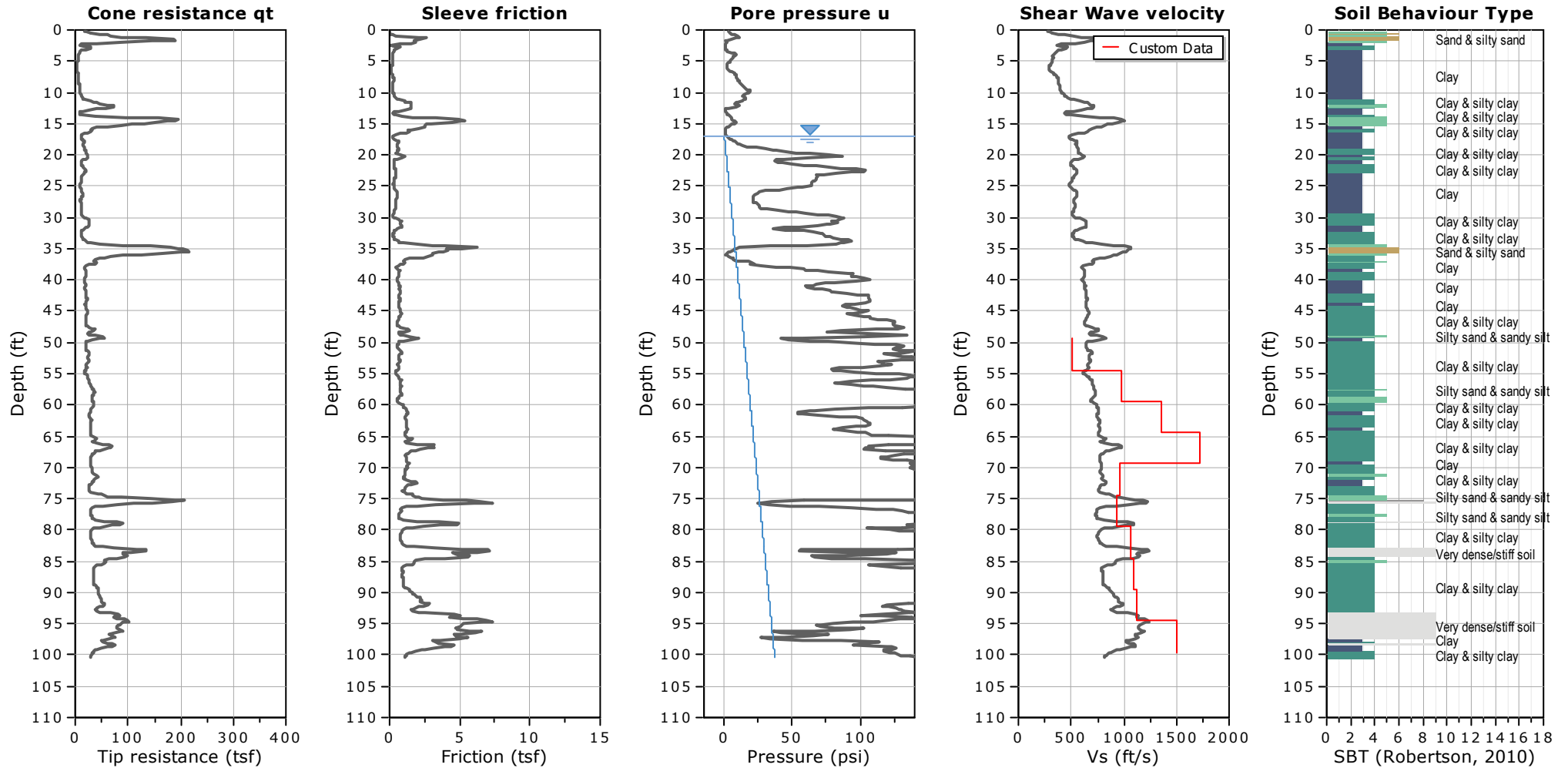


CLIENT: INTERTEK

SITE: 501 INDUSTRIAL ROAD, SAN CARLOS, CA

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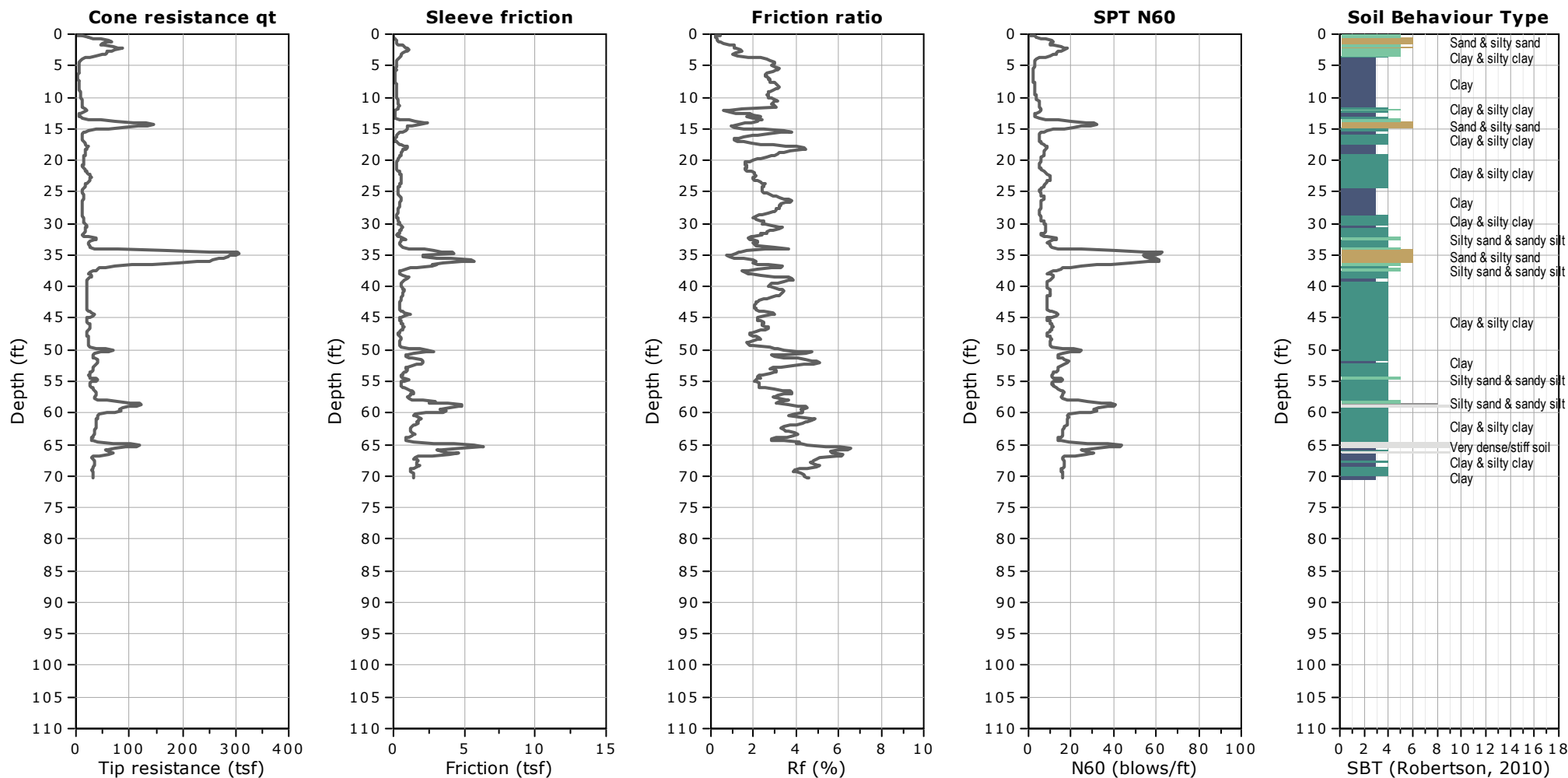


CLIENT: INTERTEK

SITE: 501 INDUSTRIAL ROAD, SAN CARLOS, CA

FIELD REP: BRAND BURFIELD

Total depth: 70.37 ft, Date: 9/5/2019



SBTn legend

- | | | |
|---------------------------|------------------------------|-----------------------------------|
| 1. Sensitive fine grained | 4. Clayey silt to silty clay | 7. Gravely sand to sand |
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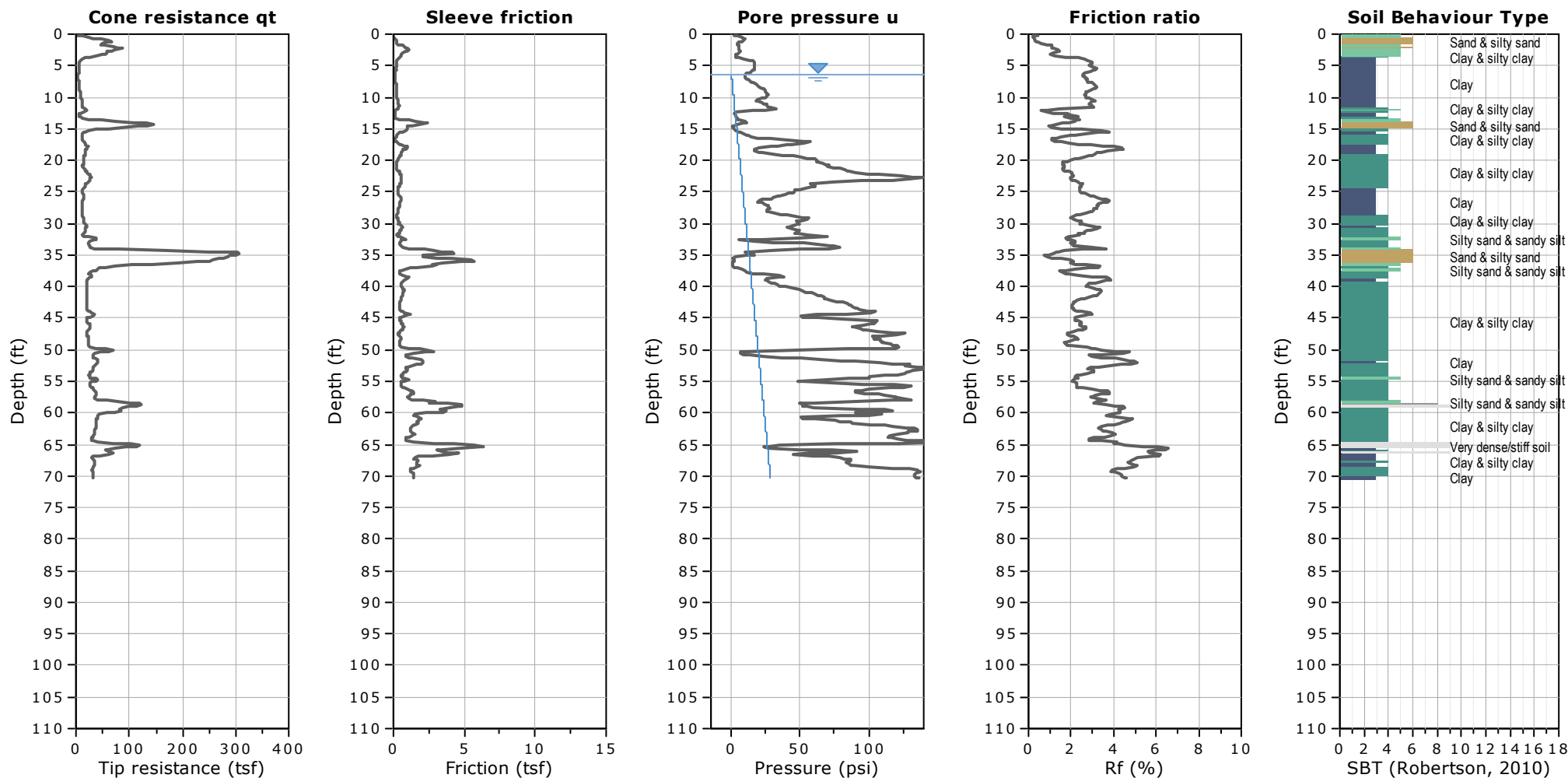


CLIENT: INTERTEK

SITE: 501 INDUSTRIAL ROAD, SAN CARLOS, CA

Field Rep: BRAND BURFIELD

Total depth: 70.37 ft, Date: 9/5/2019



WATER TABLE FOR ESTIMATING PURPOSES ONLY

SBTn legend

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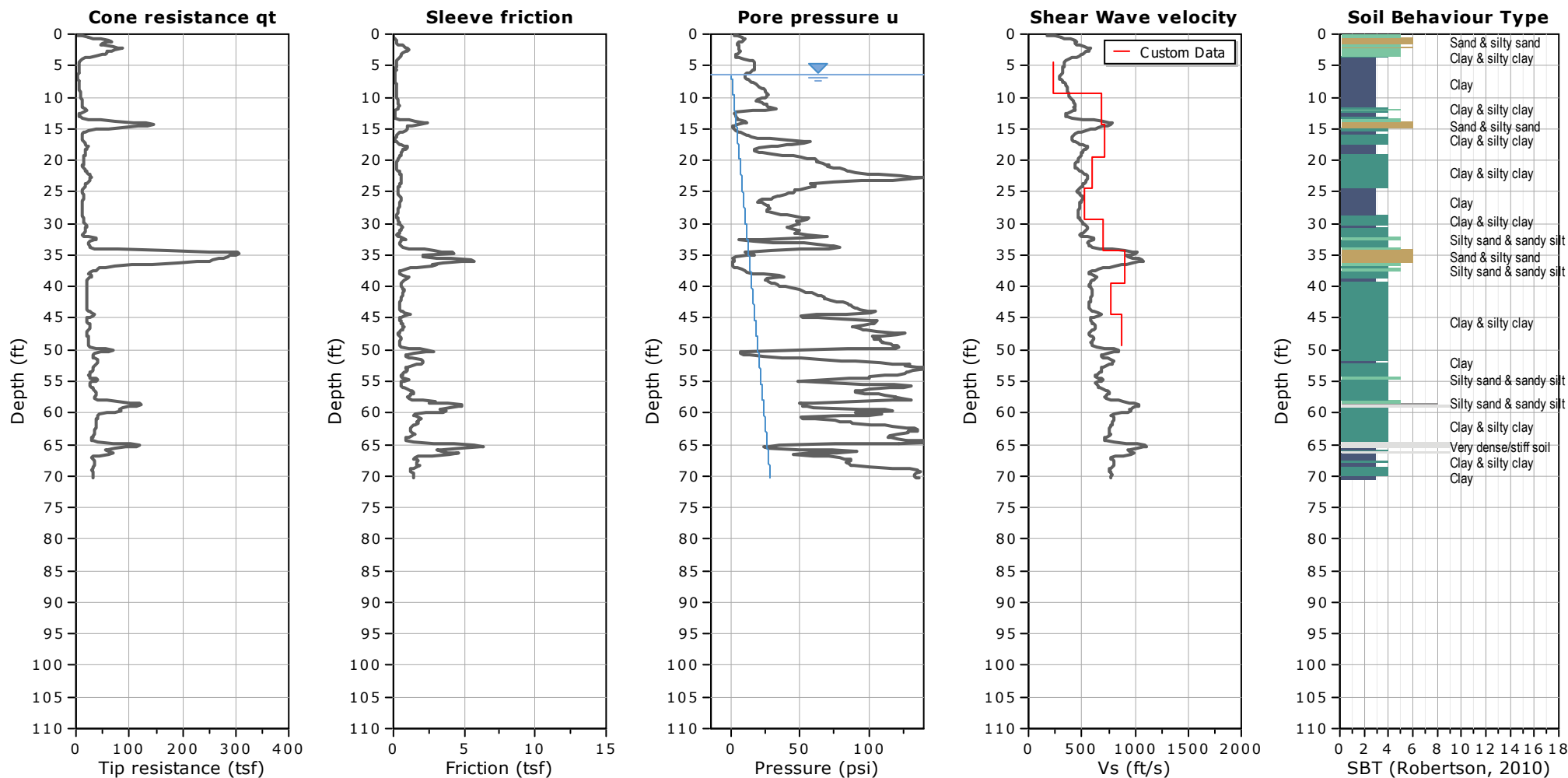


CLIENT: INTERTEK

SITE: 501 INDUSTRIAL ROAD, SAN CARLOS, CA

Field Rep: BRAND BURFIELD

Total depth: 70.37 ft, Date: 9/5/2019



WATER TABLE FOR ESTIMATING PURPOSES ONLY

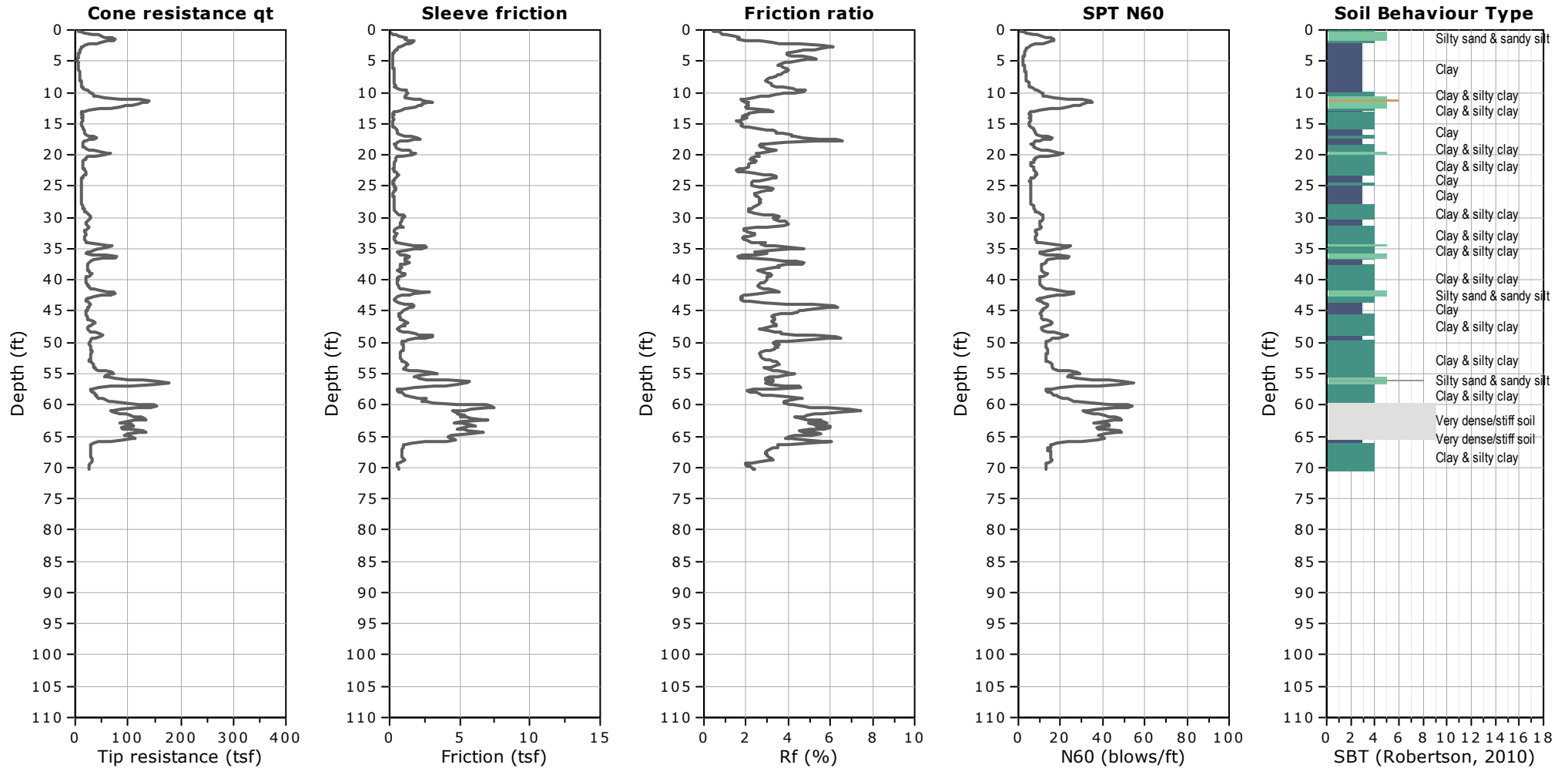


CLIENT: INTERTEK

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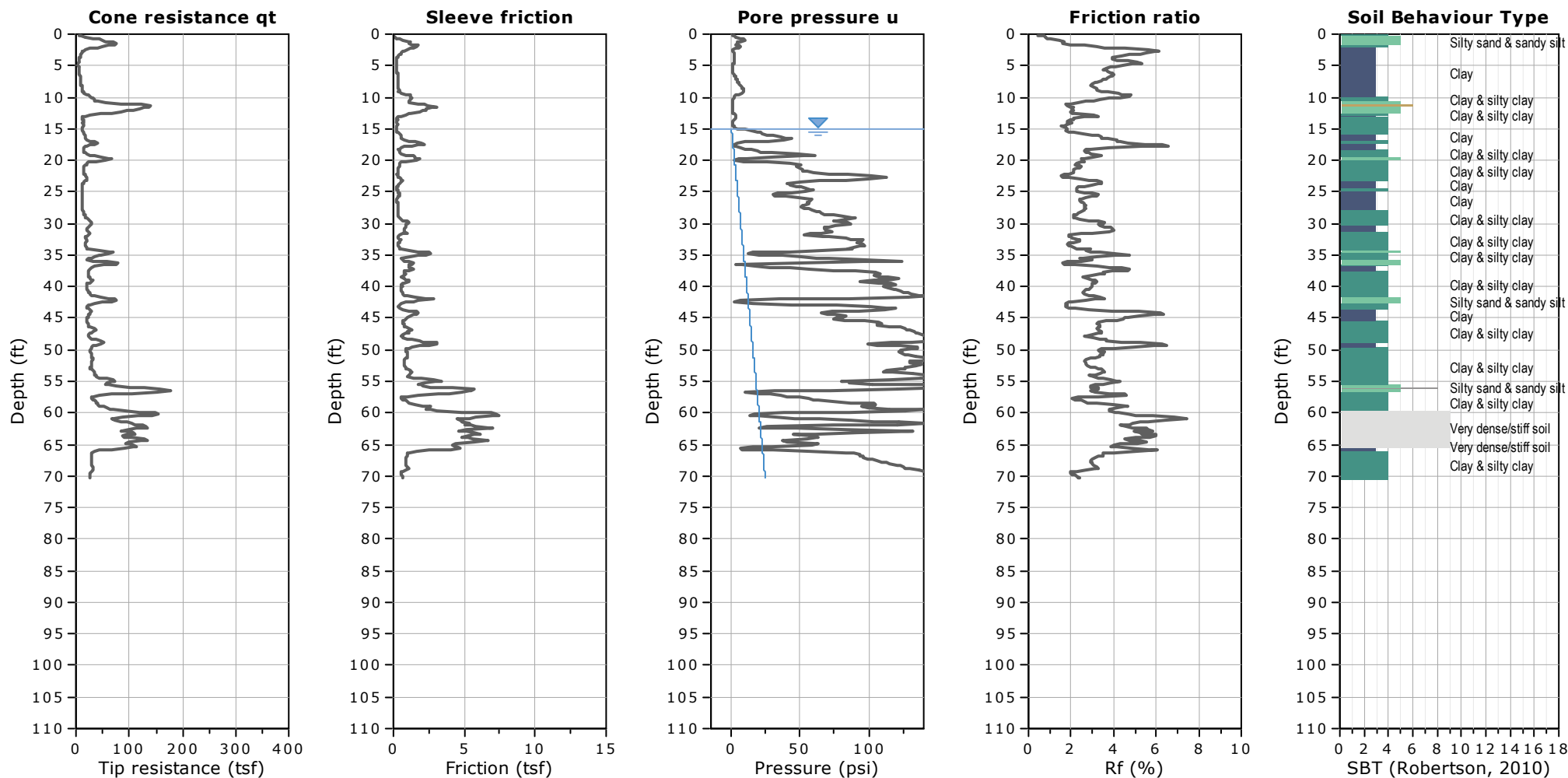


CLIENT: INTERTEK

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Field Rep: BRAND BURFIELD

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WATER TABLE FOR ESTIMATING PURPOSES ONLY

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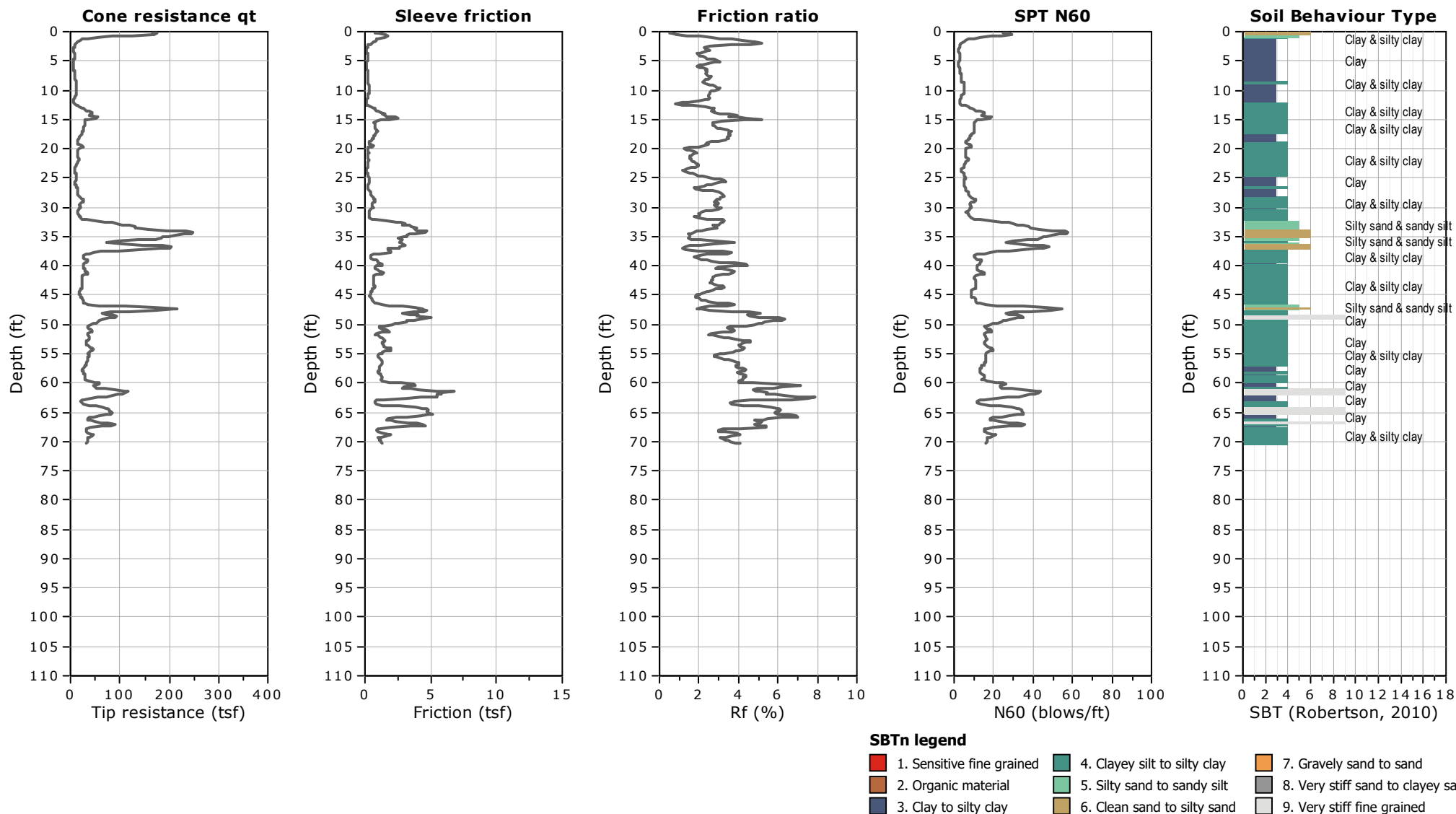


CLIENT: INTERTEK

SITE: 501 INDUSTRIAL ROAD, SAN CARLOS, CA

FIELD REP: BRAND BURFIELD

Total depth: 70.37 ft, Date: 9/5/2019



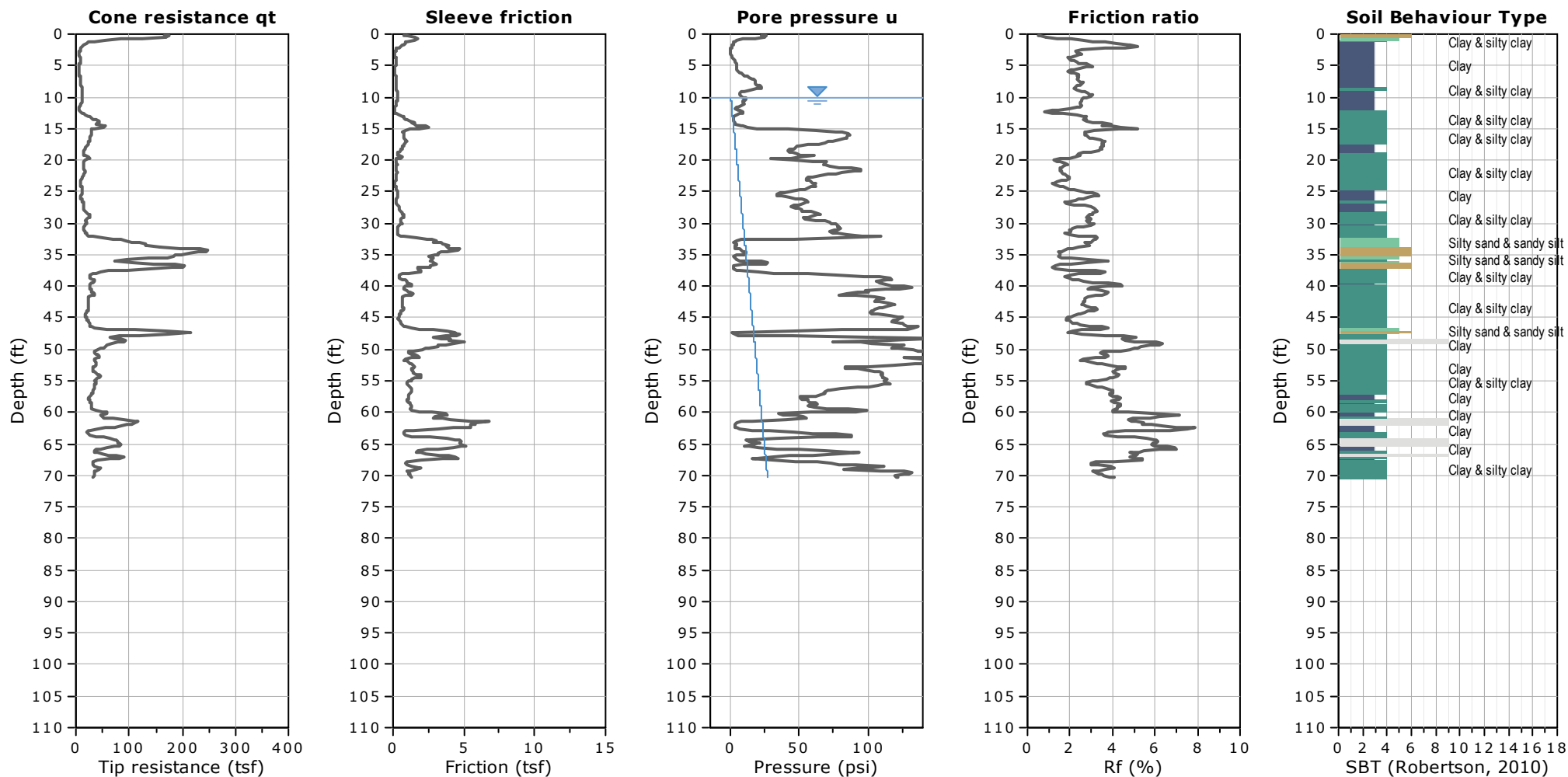


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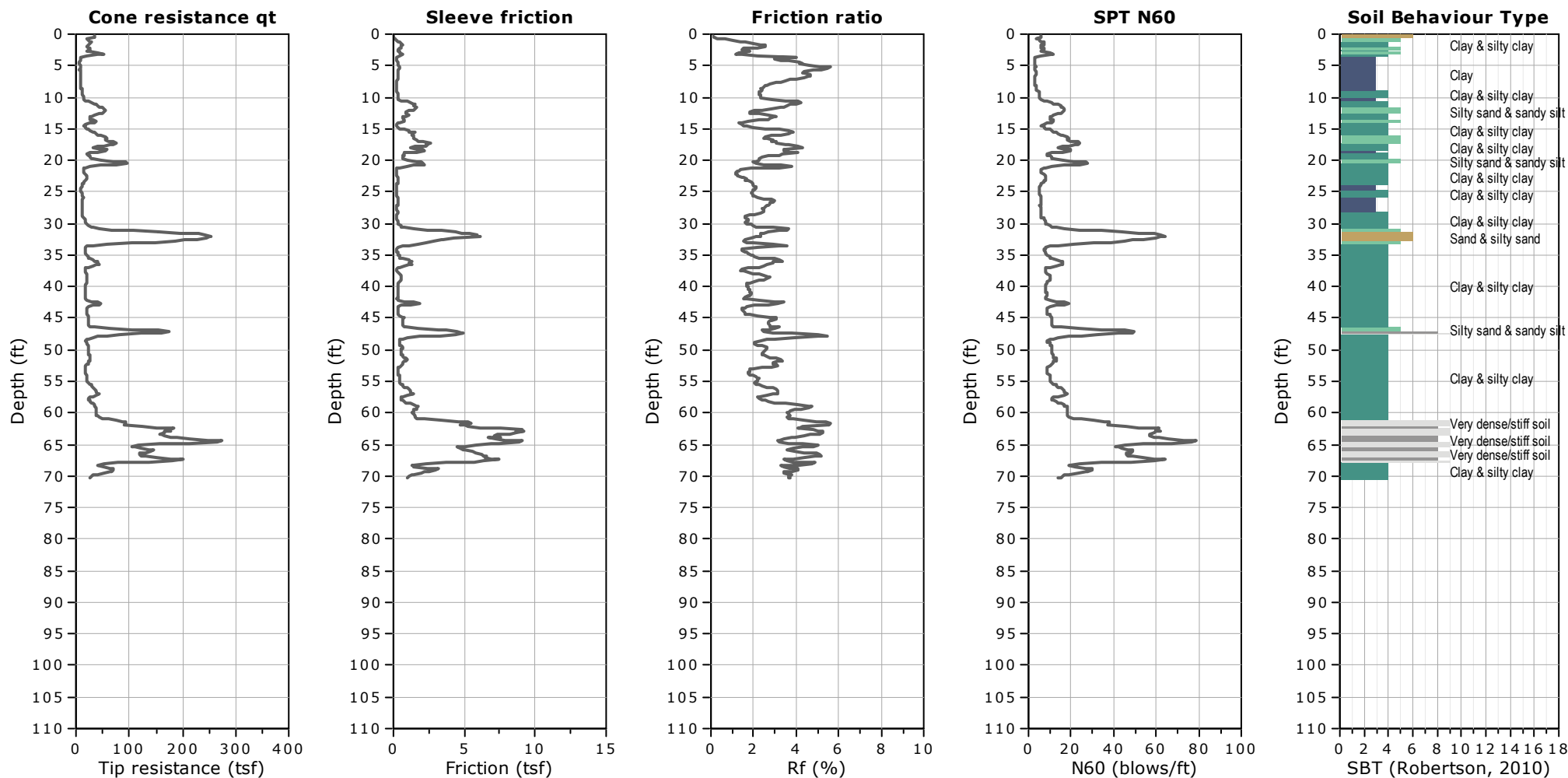


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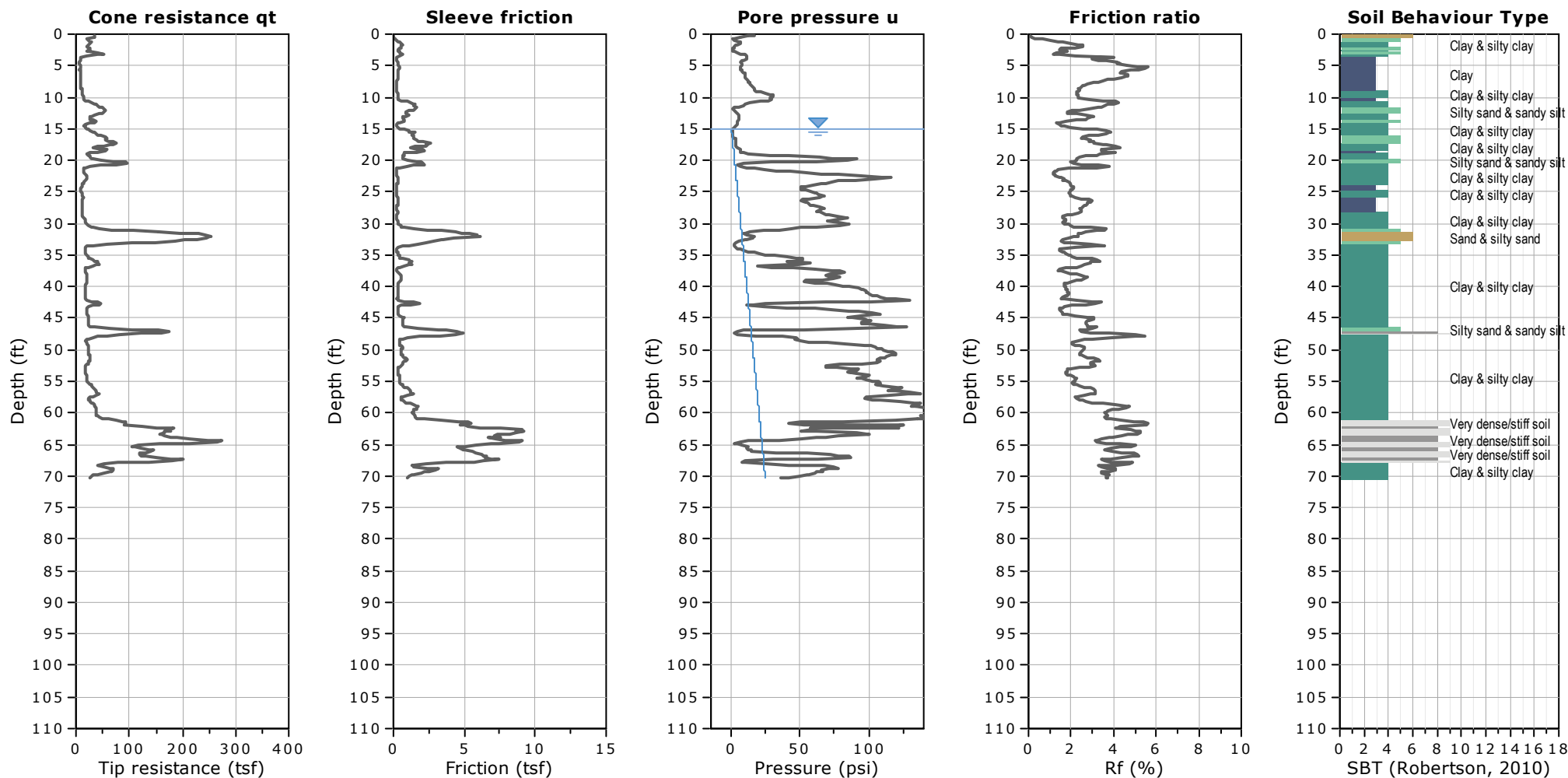


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PORE PRESSURE DISSIPATION

Pore Pressure Dissipation Tests (PPDT)

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

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

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

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

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

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

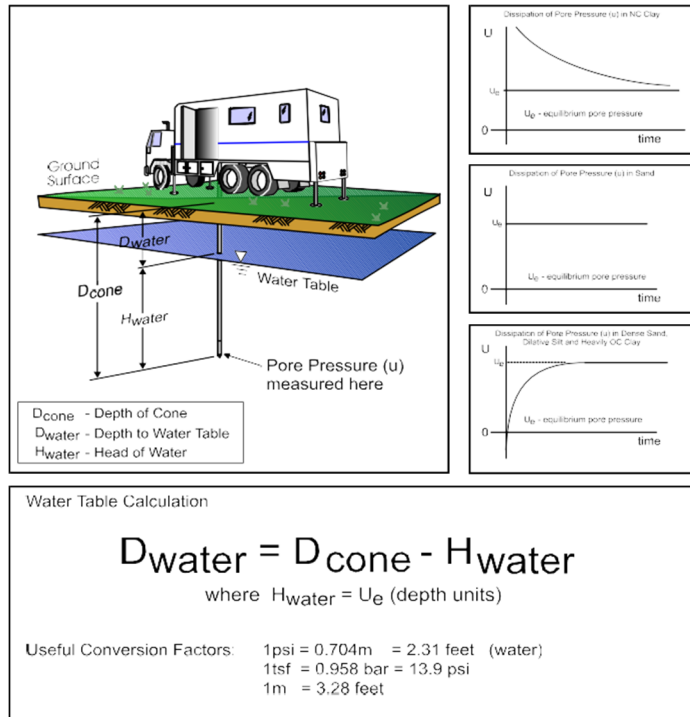


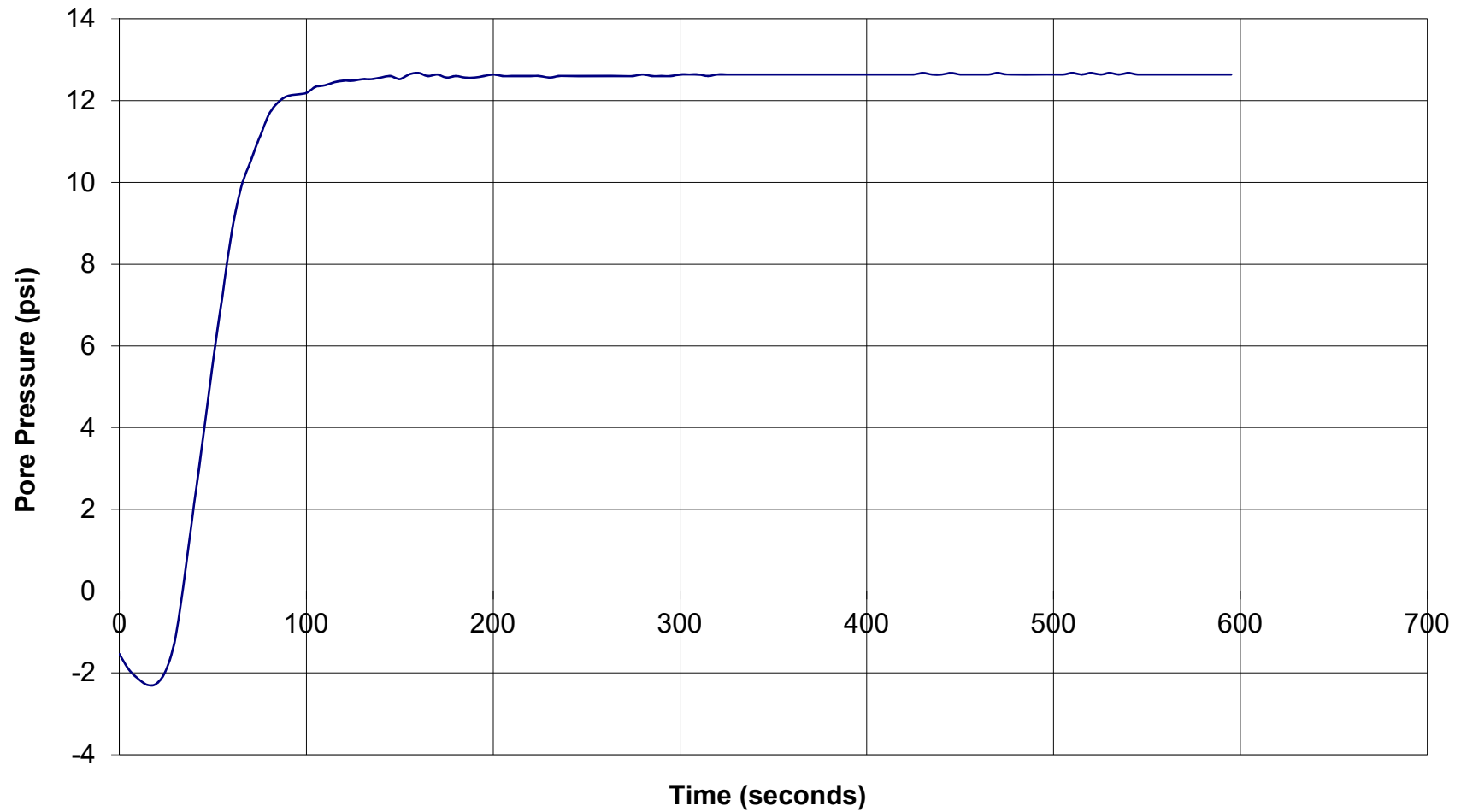
Figure PPDT



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: CPT-2
Depth: 35.432964
Site: 501 Industrial
Engineer: Brand Burfield





SEISMIC DATA

Seismic Cone Penetration Testing (SCPT)

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

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

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

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

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

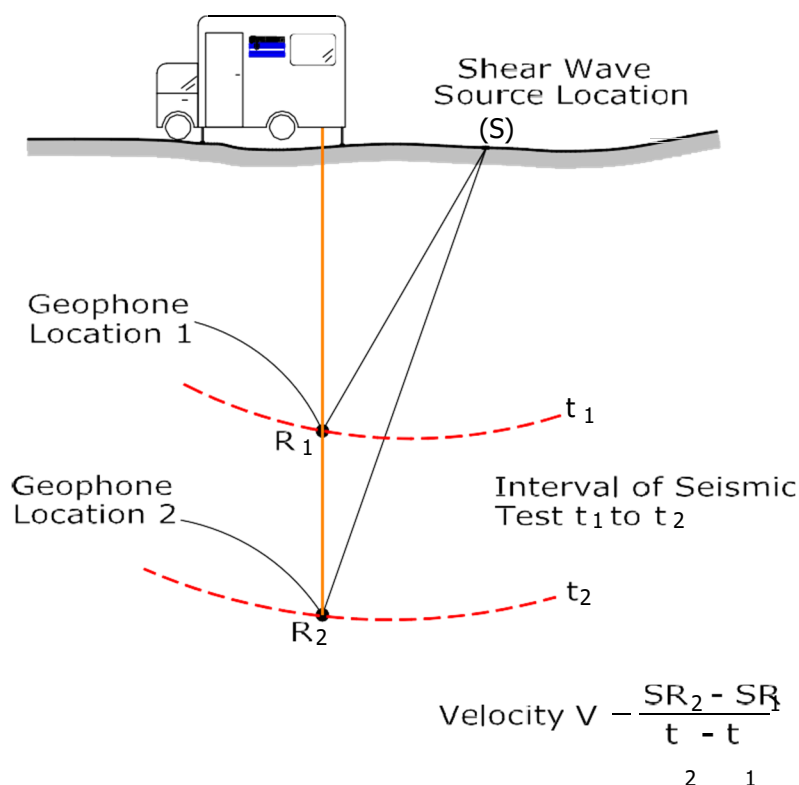


Figure SCPT



Shear Wave Velocity Calculations

501 Industrial

CPT-2

Geophone Offset: 0.66 Feet

Source Offset: 1.67 Feet

09/05/19

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
5.25	4.59	4.88	4.88	9.2500			
10.01	9.35	9.49	4.61	28.9000	19.6500	234.6	6.97
15.09	14.43	14.53	5.03	36.3000	7.4000	680.2	11.89
20.01	19.35	19.42	4.90	43.2000	6.9000	709.7	16.89
25.10	24.44	24.50	5.07	51.7500	8.5500	593.0	21.90
30.02	29.36	29.41	4.91	61.0000	9.2500	531.0	26.90
35.10	34.44	34.49	5.08	68.3000	7.3000	695.7	31.90
40.03	39.37	39.40	4.92	73.7000	5.4000	910.4	36.91
45.11	44.45	44.48	5.08	80.3000	6.6000	769.9	41.91
50.03	49.37	49.40	4.92	85.9000	5.6000	878.2	46.91

Waveforms for Sounding CPT-2

Time (ms)





Shear Wave Velocity Calculations

501 Industrial
CPT-1

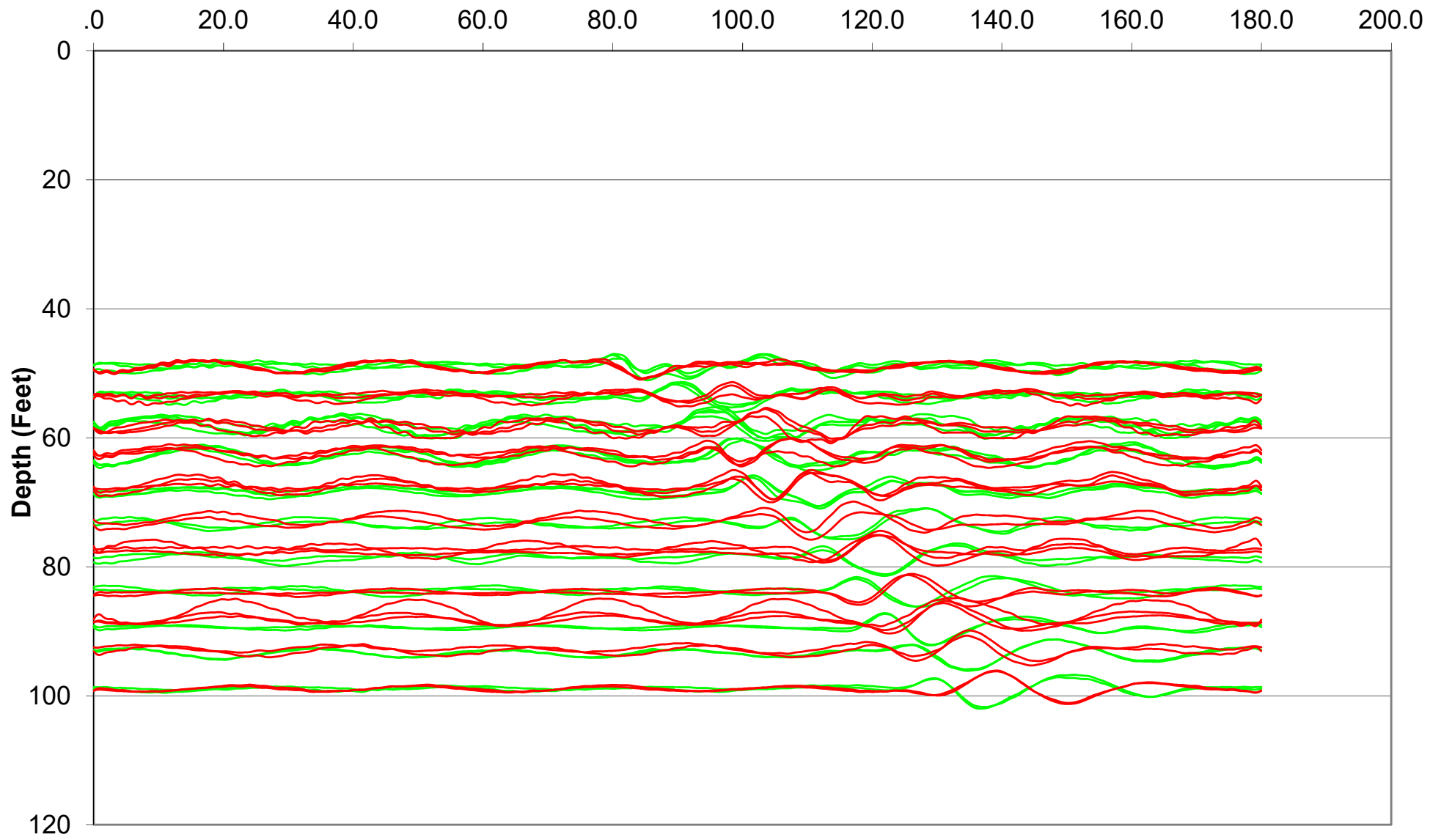
Geophone Offset: 0.66 Feet
Source Offset: 1.67 Feet

09/05/19

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
50.03	49.37	49.40	49.40	80.4000			
55.12	54.46	54.48	5.08	90.2500	9.8500	516.0	51.92
60.04	59.38	59.40	4.92	95.2500	5.0000	983.8	56.92
65.12	64.46	64.49	5.08	99.0000	3.7500	1355.6	61.92
70.05	69.39	69.41	4.92	101.8500	2.8500	1726.2	66.93
75.13	74.47	74.49	5.08	107.1000	5.2500	968.4	71.93
80.22	79.56	79.57	5.08	112.5000	5.4000	941.5	77.01
85.14	84.48	84.49	4.92	117.1000	4.6000	1069.6	82.02
90.22	89.56	89.58	5.08	121.7500	4.6500	1093.4	87.02
95.14	94.48	94.50	4.92	126.1000	4.3500	1131.1	92.02
100.39	99.73	99.75	5.25	129.6000	3.5000	1499.6	97.11

Waveforms for Sounding SCPT-1

Time (ms)



ORDINANCE: 04023



SAN MATEO COUNTY HEALTH
**ENVIRONMENTAL
HEALTH SERVICES**

Permit 19-1098

PE: 2013 GEOTECHNICAL - ANNUAL SOIL BORING PERMIT

FACILITY:

SAN MATEO COUNTY 48 HOUR NOTICE

OWNER:

PROFESSIONAL SERVICE INDUSTRIES INC
4703 TIDEWATER AVE STE B
OAKLAND

SR0028738
AMOUNT PAID:

\$934.00

CONTRACTOR:

CONSULTANT:
BURFIELD, BRAND

TERMS & CONDITIONS:

ANNUAL GEOTECHNICAL PERMIT

DATE ISSUED: 6/19/2019

KIAN ATKINSON

ENVIRONMENTAL HEALTH SPECIALIST

EXPIRATION DATE: 6/19/2020

THIS CERTIFICATE IS NONTRANSFERABLE AND MUST BE POSTED ON-SITE IN A CONSPICUOUS PLACE.



SAN MATEO COUNTY HEALTH
**ENVIRONMENTAL
HEALTH SERVICES**

PAID VIA MAIL
\$934.
CK# 3645

Environmental Health Services
Groundwater Protection Program
2000 Alameda de las Pulgas, Suite #100
San Mateo, CA 94403
Phone: (650) 372-6200 | Fax: (650) 627-8244
smchealth.org/gpp

ANNUAL GEOTECHNICAL DRILLING PERMIT APPLICATION

ALLOW THREE (3) WORKING DAYS FOR PROCESSING PERMIT, SEPERATE OFFICE SPECIFIC NOTIFICATION FORMS MUST BE FULLY COMPLETED FOR EACH JOB AND SUBMITTED (drilling@smcgov.org) AT LEAST TWO (2) WORKING DAYS (48 HOURS) IN ADVANCE, ONE APPLICATION PER OFFICE LOCATION. Visit smchealth.org/ehfees for Groundwater Protection Fees

Consultant Company: (one per office) Professional Service Industries, Inc.

Primary Contact: Brand Burfield

Email: brand.burfield@intertek.com

Mailing Address: 4703 Tidewater Avenue, Suite B

City, State, Zip: Oakland, CA 94601

Telephone: 510-434-9200

GENERAL CONDITIONS:

1. Only **GEOTECHNICAL** investigations will be performed under this permit. Other investigations including, but not limited to, environmental investigations and grab soil or water sampling for activities other than geotechnical investigation, require a Subsurface Drilling Permit Application with the appropriate fee.
2. Any drilling within a landfill or former landfill requires a Subsurface Drilling Permit Application with the appropriate fee. A work plan is required to be submitted concurrently and must be approved by San Mateo County Local Enforcement Agency (LEA) and Groundwater Protection Program (GPP) prior to drilling.
3. Any well installations, including wells for geotechnical investigations, require a separate Subsurface Drilling Permit Application with the appropriate fee.
4. Written notification to Groundwater Protection Program (GPP) staff, on GPP-supplied forms, must occur at least two (2) working days prior to each drilling event.
5. An accurate & correct map of proposed boring locations must be included with the written notification. The boring location map must be to scale and include north arrow, existing site features, approximate property lines and any other pertinent existing & historic features and information.
6. Boring construction and destruction under this permit is subject to the Standards for the Construction of Wells in San Mateo County, County Groundwater Protection Program (GPP) Guidelines, Policies & Procedures, the State Water Well Standards, and any instructions by County GPP staff.
7. All borings must be properly destroyed (grouted/sealed), using neat cement grout with maximum of 5% bentonite or sand cement (as specified in California Well Standards), within 24 hours of drilling unless special conditions are approved in writing as part of this permit.
8. Drilling under this permit must be under the supervision of a California Professional Geologist or Civil Engineer.
9. If contamination is discovered during drilling, verbal notification to County GPP by the Responsible Professional is required within 72 hours of discovery. A written report is required within 30 days.
10. The consultant company named above assumes responsibility for all activities and uses under the permit, including compliance with Workmen's Compensation Laws, and indemnifies, defends and saves the County of San Mateo, its officers, agents and employees, free and harmless from any and all expense, cost, or liability in connection with or resulting from work or stopped-work associated with the permit, including, but not limited to, property damage, personal injury, wrongful death, and loss of income.
11. Permit is valid for one calendar year only from the permit issue date.
12. Violation of any requirement or general or special permit condition as listed on this permit application or on the ANNUAL GEOTECHNICAL PERMIT DRILLING NOTIFICATION may result in an order by GPP staff to cease work under this permit and correct the violation. An office hearing may be held to determine if Consulting Company will be allowed to continue to have and apply for future Annual Geotechnical Drilling Permits.
13. If there is a change in Responsible Professional, I will notify San Mateo County GPP staff.

Responsible Professional's Signature: _____

Responsible Professional's Name: Brand Burfield

California Professional
Geologist (PG) No. 6986

or Civil Engineer
(PE) No. _____

County Approval: _____

Date: 6/17/19

DRILLING NOTIFICATION FORM FOR ANNUAL GEOTECHNICAL DRILLING PERMIT

SAN MATEO COUNTY ENVIRONMENTAL HEALTH SERVICES DIVISION
2000 ALAMEDA DE LAS PULGAS, SUITE 100, SAN MATEO, CA. 94403
VOICE (650) 372-6200 FAX (650) 627-8244 WWW.SMCHEALTH.ORG

An accurate & correct map of proposed boring locations must be included with notification.

Notification is hereby given under Annual Geotechnical Drilling Permit No. AGDP-19-1098, with expiration date June 19, 2020 that Professional Service Industries Inc will be drilling for soil boring geotechnical investigation only, not permanent structures or for environmental investigations, as described below.

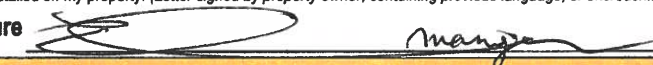
ALL DRILLING MUST BE SCHEDULED WITH COUNTY STAFF (drilling@smcgov.org) AT LEAST TWO (2) WORKING DAYS (48 HOURS) IN ADVANCE		
DRILLING WILL BEGIN ON: August 30, 2019	AT: 7am	(AM/PM) NO. OF BORINGS 5
BORING DESIGNATIONS# CPT1-5		
DRILLING INFORMATION (MUST BE FILLED OUT COMPLETELY)		
SITE NAME Holly Hotel - San Carlos	ASSESSOR'S PARCEL # (REQUIRED) 046-090-410	(one per permit)
DRILLING LOCATION ADDRESS 501 Industrial Road	CITY San Carlos	ZIP 94070
Borings To Be Constructed In: ☐ Public Property ☒ Private Property ☐ Refuse ☐ Other		
Maximum Proposed Depth Wells/Borings 100 Feet	(feet)	Drilling Method Cone Penetrometer Tests (CPT)
Boring Diameter 3 inches	Grout Material: use 6 gallons water max per 94 lb cement, can add up to 5% bentonite	
BORING OWNER (BORING OWNER NAME OR CONTACT NAME SHOULD MATCH SIGNATURE)		
NAME Holly Hotel Group, LLC	CONTACT PERSON Mr. Chao Wu	
ADDRESS 991 W Hedding Street, Suite 103	CITY, STATE, ZIP San Jose, California, 95126	
TELEPHONE 408-835-6931	EMAIL chaowu13@gmail.com	
(Letter signed by boring owner attesting to knowledge of all permit requirements and conditions, may be substituted for signature on permit application.)		
Boring Owner's Signature	Date 8-20-2019	
PROPERTY OWNER (NAME AS APPEARS ON ASSESSOR'S ROLES SHOULD MATCH SIGNATURE)		
NAME Holly Hotel Group, LLC	CONTACT PERSON Mr. Chao Wu	
ADDRESS 991 W Hedding Street, Suite 103	CITY, STATE, ZIP San Jose, California, 95126	
TELEPHONE 408-835-6931	EMAIL chaowu13@gmail.com	
I understand that a boring(s) is being installed on my property. (Letter signed by property owner, containing previous language, or encroachment permit may be substituted for signature on permit application.)		
Property Owner's Signature	Date 8-20-2019	
DRILLING COMPANY		
DRILLING COMPANY Gregg Drilling	CONTACT PERSON Chris Pruner	
ADDRESS 950 Howe Road	CITY, STATE, ZIP Martinez, California, 94553	
TELEPHONE 925-313-5800	C 57 LICENSE # 1044456	E-MAIL cpruner@greggdrilling.com
I certify that borings under this notification will be constructed/destroyed in compliance with the conditions of the Annual Geotechnical Drilling Permit listed above, the San Mateo County Ordinance, and the State Water Well Standards, and that the license listed above is considered current and active by the Contractor's State License Board.		
Driller's Signature	Date	
CONSULTANT COMPANY		
CONSULTANT COMPANY Intertek PSI	PROJECT MANAGER Frank Poss	
ADDRESS 4703 Tidewater Avenue	TELEPHONE # 510-434-9200	
CITY, STATE, ZIP Oakland, California, 94601	E-MAIL frank.poss@intertek.com	
I certify that this notification is correct to the best of my knowledge. I certify that the geotechnical borings under this notification will be constructed/destroyed in compliance with the conditions of the Annual Geotechnical Drilling Permit listed above, the San Mateo County Ordinance, and the State Water Well Standards. I certify if I indicated the purpose of drilling is geotechnical, then no one will use the boring to collect any samples for environmental analyses. (Responsible Professional must be a California Professional Geologist or Civil Engineer.)		
Responsible Professional's Name (Please print legibly) BRAND BURFIED		
Responsible Professional's Signature	Date 8/20/19	
California Professional Geologist (PG) No. 6986	or Civil Engineer (PE) No.	

DRILLING NOTIFICATION FORM FOR ANNUAL GEOTECHNICAL DRILLING PERMIT

SAN MATEO COUNTY ENVIRONMENTAL HEALTH SERVICES DIVISION
2000 ALAMEDA DE LAS PULGAS, SUITE 100, SAN MATEO, CA. 94403
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DRILLING WILL BEGIN ON: August 30, 2019	AT: 7am (AM/PM) NO. OF BORINGS 4
BORING DESIGNATIONS# B1-4	
DRILLING INFORMATION (MUST BE FILLED OUT COMPLETELY)	
SITE NAME Holly Hotel - San Carlos	ASSESSOR'S PARCEL # (REQUIRED) 046-090-410 (one per permit)
DRILLING LOCATION ADDRESS 501 Industrial Road	CITY San Carlos ZIP 94070
Borings To Be Constructed In: ☐ Public Property ☒ Private Property ☐ Refuse ☐ Other	
Maximum Proposed Depth Wells/Borings 50 Feet (feet)	Drilling Method Hollow Stem Auger (HSA)
Boring Diameter 8 inches	Grout Material: use 6 gallons water max per 94 lb cement, can add up to 5% bentonite
BORING OWNER (BORING OWNER NAME OR CONTACT NAME SHOULD MATCH SIGNATURE)	
NAME Holly Hotel Group, LLC	CONTACT PERSON Mr. Chao Wu
ADDRESS 991 W Hedding Street, Suite 103	CITY, STATE, ZIP San Jose, California, 95126
TELEPHONE 408-835-6931	EMAIL chaowu13@gmail.com
(Letter signed by boring owner attesting to knowledge of all permit requirements and conditions, may be substituted for signature on permit application.)	
Boring Owner's Signature 	Date 8-20-2019
PROPERTY OWNER (NAME AS APPEARS ON ASSESSOR'S ROLES SHOULD MATCH SIGNATURE)	
NAME Holly Hotel Group, LLC	CONTACT PERSON Mr. Chao Wu
ADDRESS 991 W Hedding Street, Suite 103	CITY, STATE, ZIP San Jose, California, 95126
TELEPHONE 408-835-6931	EMAIL chaowu13@gmail.com
(I understand that a boring(s) is being installed on my property. (Letter signed by property owner, containing previous language, or encroachment permit may be substituted for signature on permit application.)	
Property Owner's Signature 	Date 8-20-2019
DRILLING COMPANY	
DRILLING COMPANY HEW Drilling	CONTACT PERSON Evelyn Lee Jung
ADDRESS 1045 Weeks Street	CITY, STATE, ZIP East Palo Alto, California, 94303
TELEPHONE 650-322-2851	C 57 LICENSE # E-MAIL evelyn@hew-gwp.com
I certify that borings under this notification will be constructed/destroyed in compliance with the conditions of the Annual Geotechnical Drilling Permit listed above, the San Mateo County Ordinance, and the State Water Well Standards, and that the license listed above is considered current and active by the Contractor's State License Board.	
Driller's Signature	Date
CONSULTANT COMPANY	
CONSULTANT COMPANY Intertek PSI	PROJECT MANAGER Frank Poss
ADDRESS 4703 Tidewater Avenue	TELEPHONE # 510-434-9200
CITY, STATE, ZIP Oakland, California, 94601	E-MAIL frank.poss@intertek.com
I certify that this notification is correct to the best of my knowledge. I certify that the geotechnical borings under this notification will be constructed/destroyed in compliance with the conditions of the Annual Geotechnical Drilling Permit listed above, the San Mateo County Ordinance, and the State Water Well Standards. I certify if I indicated the purpose of drilling is geotechnical, then no one will use the boring to collect any samples for environmental analyses. (Responsible Professional must be a California Professional Geologist or Civil Engineer.)	
Responsible Professional's Name (Please print legibly) BRAND BURRELL	
Responsible Professional's Signature 	Date 8/20/19
California Professional Geologist (PG) No. 6986	or Civil Engineer (PE) No.

Page 2 of 2 ANNUAL GEOTECHNICAL DRILLING PERMIT NOTIFICATION

REQUIREMENTS:

An accurate & correct map of existing and proposed boring locations **must** be included with the drilling notification. The boring location map **must** include the following.

1. North arrow, existing site features, wells, approximate property lines, closest street and cross-street, and any other pertinent existing & historic information.
2. Proposed boring locations to scale.

Upon review of information in this drilling notification and subject to approval, no additional documents or verbal correspondence may be issued notifying boring owner, driller, and responsible professional (consultant) to perform the specified work. The annual geotechnical permit is subject to General Conditions stated in the permit. A copy of the approved Annual Geotechnical Drilling Permit and the submitted Drilling Notification **must** be available on site while work related to the permit is being performed. This drilling notification **must** be submitted to County staff at least two (2) working days in advance of field work. Drilling may begin at the notified date and time whether County staff is present or not. However, should the field work be cancelled or delayed, notification must be given to GPP staff up to 1 minute before the listed start time. If GPP staff attempt to perform an inspection and was not properly notified of a cancellation, then the Consultant will be billed an Inspection Cancellation fee of \$342.

DRILLING NOTIFICATION INSTRUCTIONS

Section 1: Drilling Date and Time

The **date** and **time** of the start of the geotechnical drilling must be entered. Circle either AM or PM for the start time. Indicate the **number of borings** and list the preliminary **boring designations**. This is to avoid confusion regarding geotechnical borings which may have already been performed at this site as indicated on the site map.

Section 2: Drilling Information

All applicable spaces must be filled in. **Site name** refers to the name of the project under which the investigation is being conducted. **Assessor's parcel number** is the 9-digit number corresponding to the specific private property drilling is proposed to be conducted on (County Assessor's web site wecare.co.sanmateo.ca.us under Property Assessment or www.sanmateocountytaxcollector.org under Secured Property Taxes). Each notification must include only one assessor's parcel number. If the drilling is to be conducted only in public right-of-ways, then the assessor's parcel number space should be filled in with NA for not applicable. If drilling is to occur on both a private property and a contiguous public right-of-way, then two notifications (one for the private property and one for the public right-of-way) must be filled out. **Address** and **City** refer to the location of the specific property drilling is proposed to be conducted on. The Address for a public right-of-way would simply be the name of the specific section of the public right-of-way (such as 100s block of Main Street). **Borings to be Constructed in** must have one box selected. Again, this differentiates between a public right-of-way and a private property. **Refuse** is a special land use designation which needs to be indicated on the drilling notification. The rest of this section is self-explanatory, may change in the field based on conditions encountered, and must be filled in. Drilling in the City of Daly City is permitted by the City of Daly City Water and Wastewater Department personnel at (650) 991-8200.

Grout Material requires the use of a maximum of 6 gallons of water per 94 pounds of cement. This measurement (for both water and cement) must be able to be demonstrated in the field upon request from the inspector (such as using a 5-gallon bucket for measuring the water and using entire bags of cement). Grout must be tremied into the boring if water is present in 10% of the boring or greater or the total depth of the boring is deeper than 30 feet.

Section 3: Boring Owner

The **name** of the entity owning the borings must be listed along with their contact person (if different from the name of the boring owner), address, telephone number, and email address. The **contact person** must be directly associated with or an agent of the entity owning the borings such as a property manager, real estate manager, contractor, or lawyer but not the geotechnical consultant listed on the permit application in Section 6. A **telephone** number and an **email** address must be provided to allow the County inspector to contact the boring owner to verify information if necessary. The drilling notification must be **signed** and **dated** by either the entity listed as the owner of the wells and borings or the contact person. **Signatures (Sections 3 through 6)** do not need to be wet; however, one copy of the permit application must contain all of the information besides the signatures in a legible format. ALL SIGNATURES REQUIRED (SECTIONS 3 THROUGH 6) DO NOT NEED TO BE ON THE SAME COPY OF THE PERMIT APPLICATION.

Section 4: Property Owner

The **name** of the entity owning the property must be listed and needs to match the name listed with the County Assessor for this property. The **contact person** must be directly associated with or an agent of the entity owning the property such as a property manager, real estate manager, contractor, or lawyer but not the environmental consultant listed on the permit application in Section 6. A **telephone** number and an **email** address must be provided to allow the County inspector to contact the property owner to verify information if necessary. The drilling notification must be signed and dated by the entity listed as the property owner only. AGENTS CAN NOT SIGN FOR THE PROPERTY OWNER. For public rights-of-way, a copy of the Encroachment Permit can be substituted for the property owner signature. It is known by San Mateo County that the City of San Mateo will not issue an encroachment permit until the drilling permit is issued, but the City of San Mateo will issue a letter of intent to issue an encroachment permit which is acceptable to San Mateo County as a substitute for the property owner signature in City of San Mateo rights-of-way.

Section 5: Drilling Company

The **name** of the company proposed to drill the borings must be listed along with the drilling company **contact person**, address, telephone number, and email address. In addition, the **driller's C57 license number** must be provided. A **telephone** number and an **email** address must be provided to allow the County inspector to contact the drilling company to verify information if necessary. The drilling notification must be signed and dated by the driller's contact person. If the drilling company changes, then a new drilling notification should be filled out completely except for Sections 3, 4, and 6.

Section 6: Consulting Company

The **name** of the company overseeing the proposed drilling of the borings must be listed along with the **project manager**, address, **telephone** number, and **email** address. The responsible professional overseeing the work must **print** their name legibly, **sign** their name and date, and provide either their **California Professional Geologist** number or **California Civil or Geotechnical Engineering** number.



Q = Proposed Boring, S



APPENDIX B

LABORATORY TEST RESULTS



LABORATORY TEST RESULTS

Laboratory tests were performed on representative soil samples to determine their relative engineering properties. Tests were performed in general accordance with test methods of the American Society for Testing Materials or other accepted standards. The following presents a brief description of the various test methods used.

Classification - Soils were classified visually according to the Unified Soil Classification System. Visual classifications were supplemented by laboratory testing of selected samples in general accordance with ASTM D2487. The soil classifications are shown on the boring logs in Appendix A.

In-Situ Moisture/Density - The in-place moisture content and dry unit weight of selected samples were determined using relatively undisturbed samples from the linear rings of a 2.38-inch I.D. modified California Sampler. The moisture content of representative SPT samples was also determined. The dry unit weight and moisture contents are shown on the boring logs.

Atterberg Limits – The liquid limit, plastic limit, and plasticity index of a selected representative sample was determined in accordance with ASTM D4318. The liquid limit and plastic limit are shown on the boring logs and below in this appendix.

Expansion Index - Expansion index testing was performed on a representative sample of the on-site soils in general accordance with ASTM Test method D4829. The result of this test is provided in the text of this report, on the boring logs and below in this appendix.

Sieve Analysis – Grain size testing was performed on representative samples of the on-site soil in general accordance with ASTM D422. The percent passing the #200 sieve is shown on the boring logs.

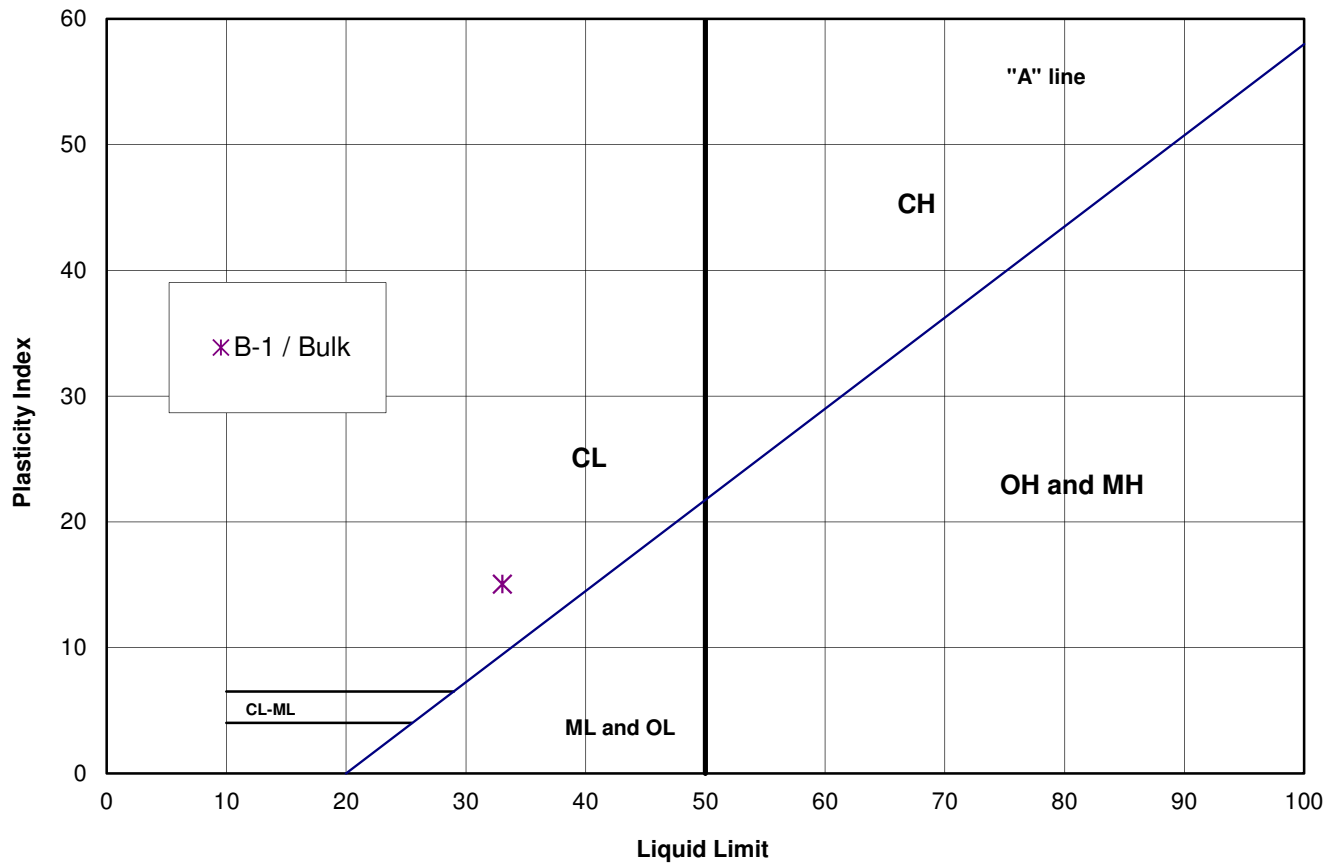
Resistance R-Value – R-Value testing was performed on representative soils anticipated to be exposed at subgrade level in paved parking areas and/or drive lanes in accordance with California Test Method 301. The results of the tests are presented in the text of this report and below in this appendix.

Soil Sulfate / Chloride Test – In order to estimate the concrete degradation potential of soils, the soluble sulfate and chloride content of a representative sample of the on-site soil, provided in the text of this report, was determined in accordance with EPA Test Method 300.0.

pH (Potential of Hydrogen) – The measure of acidity or alkalinity of a material is referred to as the pH factor, which increases with alkalinity and decreases with acidity. The corrosivity potential of iron increases with low pH (4-5) while the corrosivity potential of copper increases with high pH (10-11). The pH value of a representative sample of the on-site soil, provided in the text of this report, was determined in accordance with EPA Test Method 9045B.

Resistivity – The electrical resistivity of a soil is a measure of its resistance to electrical current flow. Corrosion of buried ferrous metals is an electrochemical process which is related to the flow of electrical current from the metal to the soil. Lower electrical resistivity (higher currents) result from higher moisture and chemical contents in the soil. Resistivity is minimal when the soil is saturated. The resistivity of a representative sample of the on-site soil, provided in the text of this report, was determined in accordance with AASHTO Test Method T 288-91.

PLASTICITY INDEX _ ASTM D4318



Sample	Depth	LL	PL	PI	USCS	Material Description
B-1 / Bulk	0 - 4'	33	18	15	CL	

Job Name: PSI # 575-1547

Date: 9/11/19

Job No.: 2015-0152

EXPANSION INDEX - UBC 18-2 & ASTM D 4829-88

PROJECT PSI # 575-1547

JOB NO. 2015-0152

Sample <u>B-1 / Bulk (0'-4')</u> By <u>LD</u>					Sample _____ By _____				
Ini. W.C.% <u>12.6</u>					Sta. No. _____				
Soil Type <u>Brown, Sandy Clay</u>					Soil Type _____				
Date	Time	Dial Reading	Wet+Tare	609.4	Date	Time	Dial Reading	Wet+Tare	
9/10/2019	13:00	0.3386	Tare	217.9				Tare	
		H2O	Net Weight	391.5				Net Weight	
9/11/2019	10:00	0.3022	% Water	10				% Water	
			Dry Dens.	107.8				Dry Dens.	
			% Max					% Max	
			Wet+Tare	651.5				Wet+Tare	
			Tare	217.9				Tare	
			Net Weight	433.6				Net Weight	
INDEX	36	3.6%	% Water	21.8	INDEX			% Water	

Sample _____ By _____					Sample _____ By _____				
Sta. No. _____					Sta. No. _____				
Soil Type _____					Soil Type _____				
Date	Time	Dial Reading	Wet+Tare		Date	Time	Dial Reading	Wet+Tare	
			Tare					Tare	
			Net Weight					Net Weight	
			% Water					% Water	
			Dry Dens.					Dry Dens.	
			% Max					% Max	
			Wet+Tare					Wet+Tare	
			Tare					Tare	
			Net Weight					Net Weight	
INDEX			% Water		INDEX			% Water	

Project Name: Holly Hotel Group - San Carlos

Project Number: 575-1547

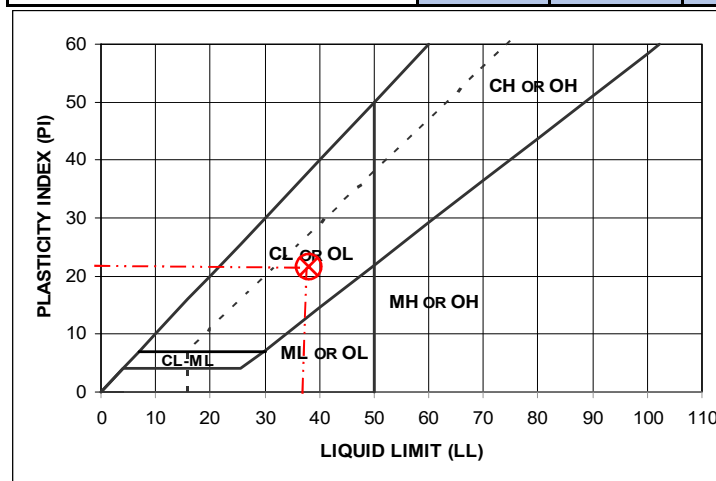
Laboratory Number: 0575 (Oakland, CA)

Date Tested: 9/29/2019

Sample Description: B-1 (10.5')

Equipment List:	Scale: 01PS575/02PS575	Oven: VWR-1675	Liquid Limit Cup: 01LL575
-----------------	------------------------	----------------	---------------------------

	LIQUID LIMIT				PLASTIC LIMIT		
CAN NUMBER	2	5	1		1	6	20
WEIGHT OF CAN	21.18	22.31	17.52		17.51	24	125.52
WEIGHT OF CAN + WET SOIL	35.42	34.74	33.63		26.53	36.73	139.4
WEIGHT OF CAN + DRY SOIL	31.67	31.37	29.32		25.4	35.06	137.58
WEIGHT OF WATER	3.75	3.37	4.31		1.13	1.67	1.82
WEIGHT OF DRY SOIL	10.49	9.06	11.8		7.89	11.06	12.06
MOISTURE CONTENT	35.7	37.2	36.5		14.3	15.1	15.1
NUMBER OF BLOWS	34	26	18				



Boring Number	B1
Sample Depth (ft.)	10.5
Density	
Weight of Sample + Ring [a]	
Weight of Ring [b]	
Weight of Sample [c=a-b]	0
Diameter of Ring	2.4
Area of Ring [A]	4.52
Height of the Sample [h]	1
Wet Density of the Sample [d=(c*3.81)/(A*h)]	0.00
Moisture Content	
Tare Number	
Tare Weight (g) [e]	
Wet Weight + Tare (g) [f]	
Dry Weight + Tare (g) [g]	
Weight of Water (g) [h=f-g]	0
Weight of Dry Sample (g) [i=g-e]	0
Moisture Content (%) [j=(h/i)*100]	
Dry Density [k=d/(1+j/100)]	

Liquid Limit	37
Plastic Limit	15
Plasticity Index	22

Equation of "A" - Line
Horizontal at PI=4 to LL=25.5,
Then PI= 0.73 (LL - 20)
Equation of "U" - Line
Vertical at LL=16 to PI=7,
Then PI= 0.9 (LL - 8)

Project Name: Holly Hotel Group - San Carlos

Project Number: 575-1547

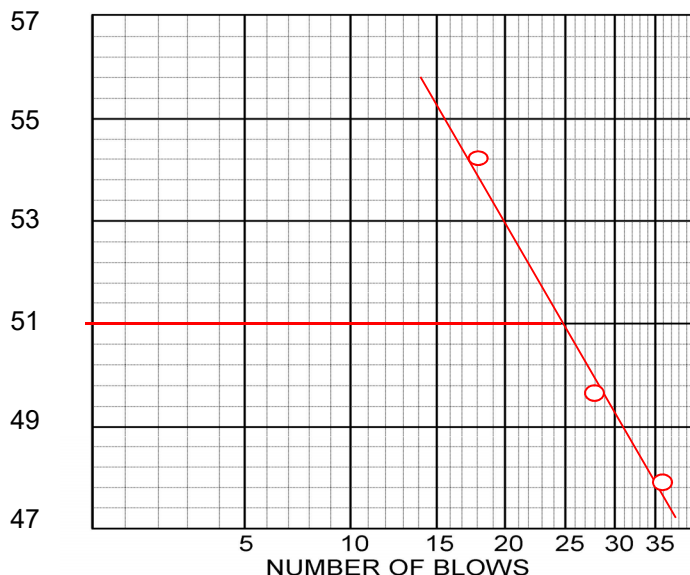
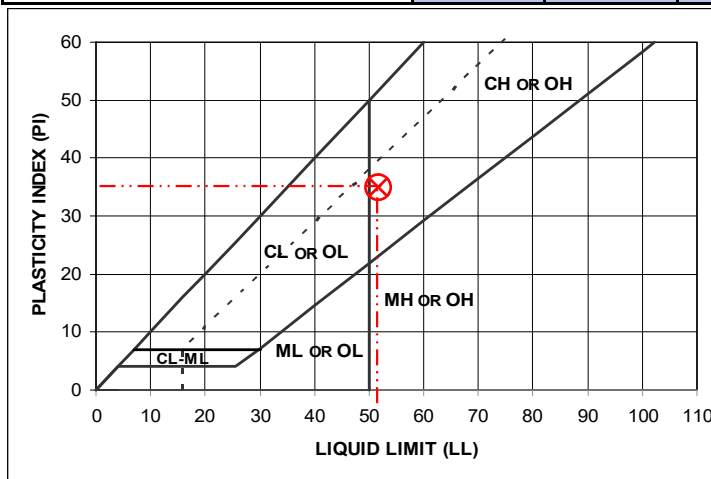
Laboratory Number: 0575 (Oakland, CA)

Date Tested: 9/29/2019

Sample Description: B-1 (30.5')

Equipment List:	Scale: 01PS575/02PS575	Oven: VWR-1675	Liquid Limit Cup: 01LL575
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	LIQUID LIMIT				PLASTIC LIMIT		
CAN NUMBER	16	21	10		17	11	7
WEIGHT OF CAN	20.93	18.91	22.64		20.30	23.86	22.63
WEIGHT OF CAN + WET SOIL	30.62	27.17	31.04		30.91	33.3	29.86
WEIGHT OF CAN + DRY SOIL	27.48	24.43	28.09		29.48	32.05	28.90
WEIGHT OF WATER	3.14	2.74	2.95		1.43	1.25	0.96
WEIGHT OF DRY SOIL	6.55	5.52	5.45		9.18	8.19	6.27
MOISTURE CONTENT	47.9	49.6	54.1		15.6	15.3	15.3
NUMBER OF BLOWS	36	28	18				



Boring Number	B-1
Sample Depth (ft.)	30.5
Density	
Weight of Sample + Ring [a]	
Weight of Ring [b]	
Weight of Sample [c=a-b]	0
Diameter of Ring	2.4
Area of Ring [A]	4.52
Height of the Sample [h]	1
Wet Density of the Sample [d=(c*3.81)/(A*h)]	0.00
Moisture Content	
Tare Number	42
Tare Weight (g) [e]	133.65
Wet Weight + Tare (g) [f]	227.92
Dry Weight + Tare (g) [g]	212.64
Weight of Water (g) [h=f-g]	15.28
Weight of Dry Sample (g) [i=g-e]	78.99
Moisture Content (%) [j=(h/i)*100]	19.3
Dry Density [k=d/(1+j/100)]	0.00

Liquid Limit	51
Plastic Limit	15
Plasticity Index	36

Equation of "A" - Line
Horizontal at PI=4 to LL=25.5,
Then PI= 0.73 (LL - 20)
Equation of "U" - Line
Vertical at LL=16 to PI=7,
Then PI= 0.9 (LL - 8)

Project Name: Holly Hotel Group - San Carlos

Project Number: 575-1547

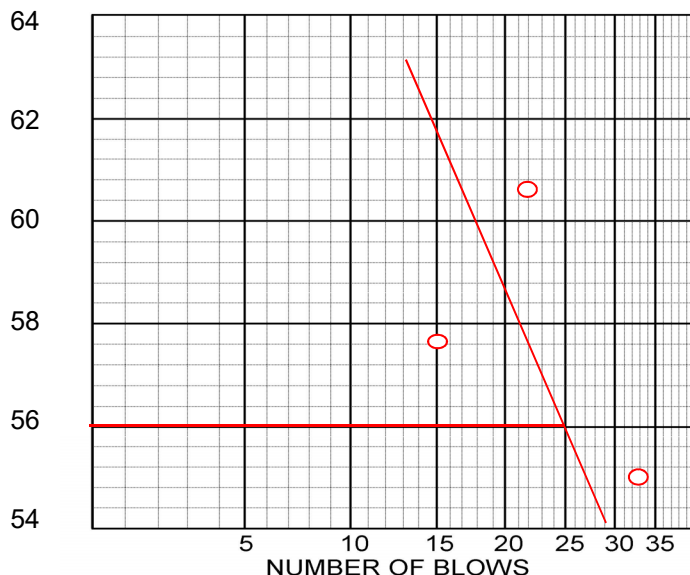
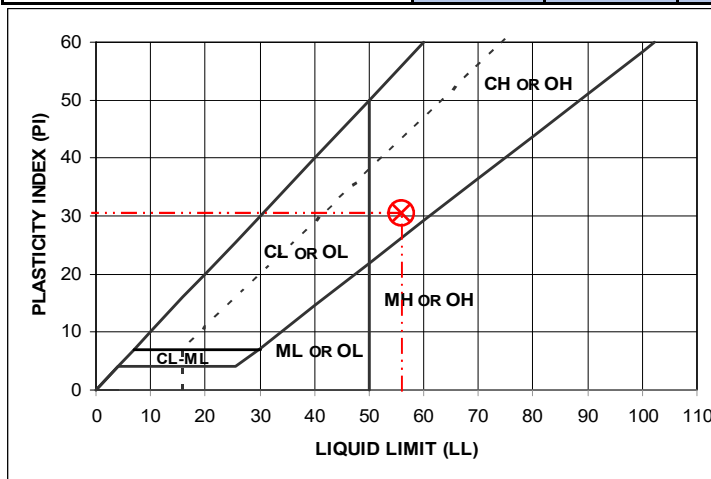
Laboratory Number: 0575 (Oakland, CA)

Date Tested: 9/29/2019

Sample Description: B-2 (3.0')

Equipment List:	Scale: 01PS575/02PS575	Oven: VWR-1675	Liquid Limit Cup: 01LL575
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	LIQUID LIMIT				PLASTIC LIMIT		
CAN NUMBER	18	14	9		13	8	3
WEIGHT OF CAN	20.71	22.39	20.19		23.97	22.00	23.73
WEIGHT OF CAN + WET SOIL	32.88	33.02	31.14		27.57	27.63	31.90
WEIGHT OF CAN + DRY SOIL	28.56	29.01	27.14		26.86	26.51	30.29
WEIGHT OF WATER	4.32	4.01	4.00		0.71	1.12	1.61
WEIGHT OF DRY SOIL	7.85	6.62	6.95		2.89	4.51	6.56
MOISTURE CONTENT	55.0	60.6	57.6		24.6	24.8	24.5
NUMBER OF BLOWS	33	22	15				



Boring Number	B-2
Sample Depth (ft.)	3.0
Density	
Weight of Sample + Ring [a]	
Weight of Ring [b]	
Weight of Sample [c=a-b]	0
Diameter of Ring	2.4
Area of Ring [A]	4.52
Height of the Sample [h]	1
Wet Density of the Sample [d=(c*3.81)/(A*h)]	0.00
Moisture Content	
Tare Number	79
Tare Weight (g) [e]	129.44
Wet Weight + Tare (g) [f]	237.99
Dry Weight + Tare (g) [g]	206.00
Weight of Water (g) [h=f-g]	31.99
Weight of Dry Sample (g) [i=g-e]	76.56
Moisture Content (%) [j=(h/i)*100]	41.8
Dry Density [k=d/(1+j/100)]	0.00

Liquid Limit	56
Plastic Limit	25
Plasticity Index	31

Equation of "A" - Line
Horizontal at PI=4 to LL=25.5,
Then PI= 0.73 (LL - 20)
Equation of "U" - Line
Vertical at LL=16 to PI=7,
Then PI= 0.9 (LL - 8)

Project Name: Holly Hotel Group - San Carlos

Project Number: 575-1547

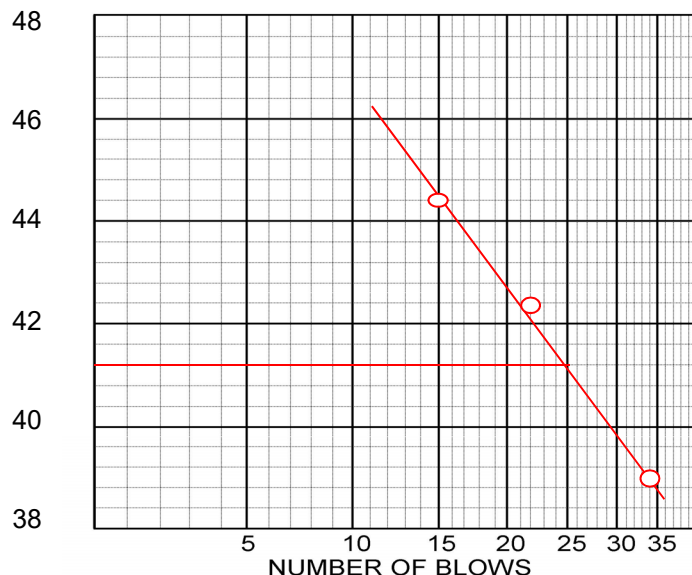
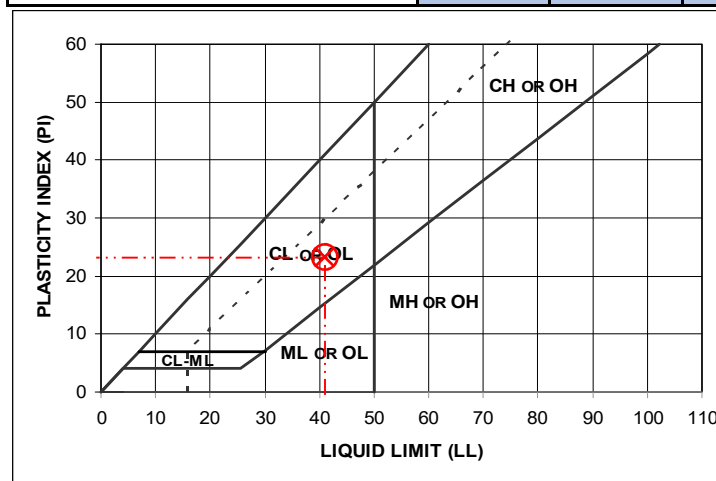
Laboratory Number: 0575 (Oakland, CA)

Date Tested: 9/29/2019

Sample Description: B-4-Bulk (0-4')

Equipment List:	Scale: 01PS575/02PS575	Oven: VWR-1675	Liquid Limit Cup: 01LL575
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	LIQUID LIMIT				PLASTIC LIMIT		
CAN NUMBER	15	22	2		24	20	4
WEIGHT OF CAN	20.05	21.58	21.25		23.02	19.41	19.71
WEIGHT OF CAN + WET SOIL	31.28	34.59	32.89		36.11	32.81	31.38
WEIGHT OF CAN + DRY SOIL	28.13	30.72	29.31		34.11	30.75	29.56
WEIGHT OF WATER	3.15	3.87	3.58		2	2.06	1.82
WEIGHT OF DRY SOIL	8.08	9.14	8.06		11.09	11.34	9.85
MOISTURE CONTENT	39.0	42.3	44.4		18.0	18.2	18.5
NUMBER OF BLOWS	34	22	15				



Boring Number	B-4
Sample Depth (ft.)	(0-4')
Density	
Weight of Sample + Ring [a]	
Weight of Ring [b]	
Weight of Sample [c=a-b]	0
Diameter of Ring	2.4
Area of Ring [A]	4.52
Height of the Sample [h]	1
Wet Density of the Sample [d=(c*3.81)/(A*h)]	0.00
Moisture Content	
Tare Number	43
Tare Weight (g) [e]	133.31
Wet Weight + Tare (g) [f]	333.38
Dry Weight + Tare (g) [g]	314.65
Weight of Water (g) [h=f-g]	18.73
Weight of Dry Sample (g) [i=g-e]	181.34
Moisture Content (%) [j=(h/i)*100]	10.3
Dry Density [k=d/(1+j/100)]	0.00

Liquid Limit	41
Plastic Limit	18
Plasticity Index	23

Equation of "A" - Line
Horizontal at PI=4 to LL=25.5,
Then PI= 0.73 (LL - 20)
Equation of "U" - Line
Vertical at LL=16 to PI=7,
Then PI= 0.9 (LL - 8)

'R' VALUE CA 301

Client: PSI

Date: 9/22/19

By: LD

Client's Job No.: 575-1547

Sample No.: B-4 / Bulk @ 0 - 4'

GLA Reference: 2015-0152

Soil Type: Brown, Clay

TEST SPECIMEN		A	B	C	D
Compactor Air Pressure	psi	60	150	100	
Initial Moisture Content	%	10.4	10.4	10.4	
Water Added	ml	80	50	65	
Moisture at Compaction	%	18.5	15.5	17.0	
Sample & Mold Weight	gms	3170	3193	3188	
Mold Weight	gms	2098	2090	2098	
Net Sample Weight	gms	1072	1103	1090	
Sample Height	in.	2.455	2.46	2.48	
Dry Density	pcf	111.7	117.7	113.9	
Pressure	lbs	3420	6455	4920	
Exudation Pressure	psi	272	514	392	
Expansion Dial	x 0.0001	0	65	31	
Expansion Pressure	psf	0	281	134	
Ph at 1000lbs	psi	67	38	51	
Ph at 2000lbs	psi	142	105	121	
Displacement	turns	4.46	3.35	3.88	
R' Value		7	28	17	
Corrected 'R' Value		7	28	17	

FINAL 'R' VALUE	
By Exudation Pressure (@ 300 psi):	9
By Expansion Pressure :	N/A
TI = 5	

Sieve Analysis of
Fine and Coarse Aggregates
(ASTM C136/C117, AASHTO T27/T11)

Project Name: Holly Hotel Group - San Carlos Project Number: 575-1547
Laboratory Number: 0575 (Oakland, CA) Date Tested: 9/18/2019
Sample Description: B-1 (15.5') Reviewed By:
Equipment List: Oven: VWR-1675 Scale: 01PS575/02PS575
Sieve Set: 01SV575 through 12SV575

	Sieve Size		Sample (circle one) :		Washed		Not Washed
	English	Metric	Tare	Tare + Sample	Wt. Retained	% Retained	% Pass
Complete Stack + Sample Before							
Complete Stack + Sample After							
Coarse Sieves	3"				0.0	0.0%	100.0%
	1.5"	37.5mm			0.0	0.0%	100.0%
	3/4"	19mm			0.0	0.0%	100.0%
	3/8"	9.5mm			0.0	0.0%	100.0%
	#4	4.75mm		13.0	13.0	5.6%	94.4%
Coarse Pan Weight (a)							
Fines Split Weight (b)			0.0				
Factor = (a/b)							
Fine Sieves	#10	2.00mm		6.1	6.1	2.6%	91.8%
	#20	0.85mm		5.7	5.7	2.4%	89.4%
	#40	0.425mm		7.2	7.2	3.1%	86.3%
	#100	0.15mm		38.4	38.4	16.5%	69.8%
	#200	0.075mm		33.4	33.4	14.4%	55.4%
	Fine Catch Pan + Wash				3.8	128.8	55.4%
Totals			0.0	107.5	232.6	100.0%	

Moisture Content	
Pan #	34
Pan Wt	253.6
Wet Sample + Pan	525.7
Dry Sample + Pan	484.9
% Moisture Content	18%

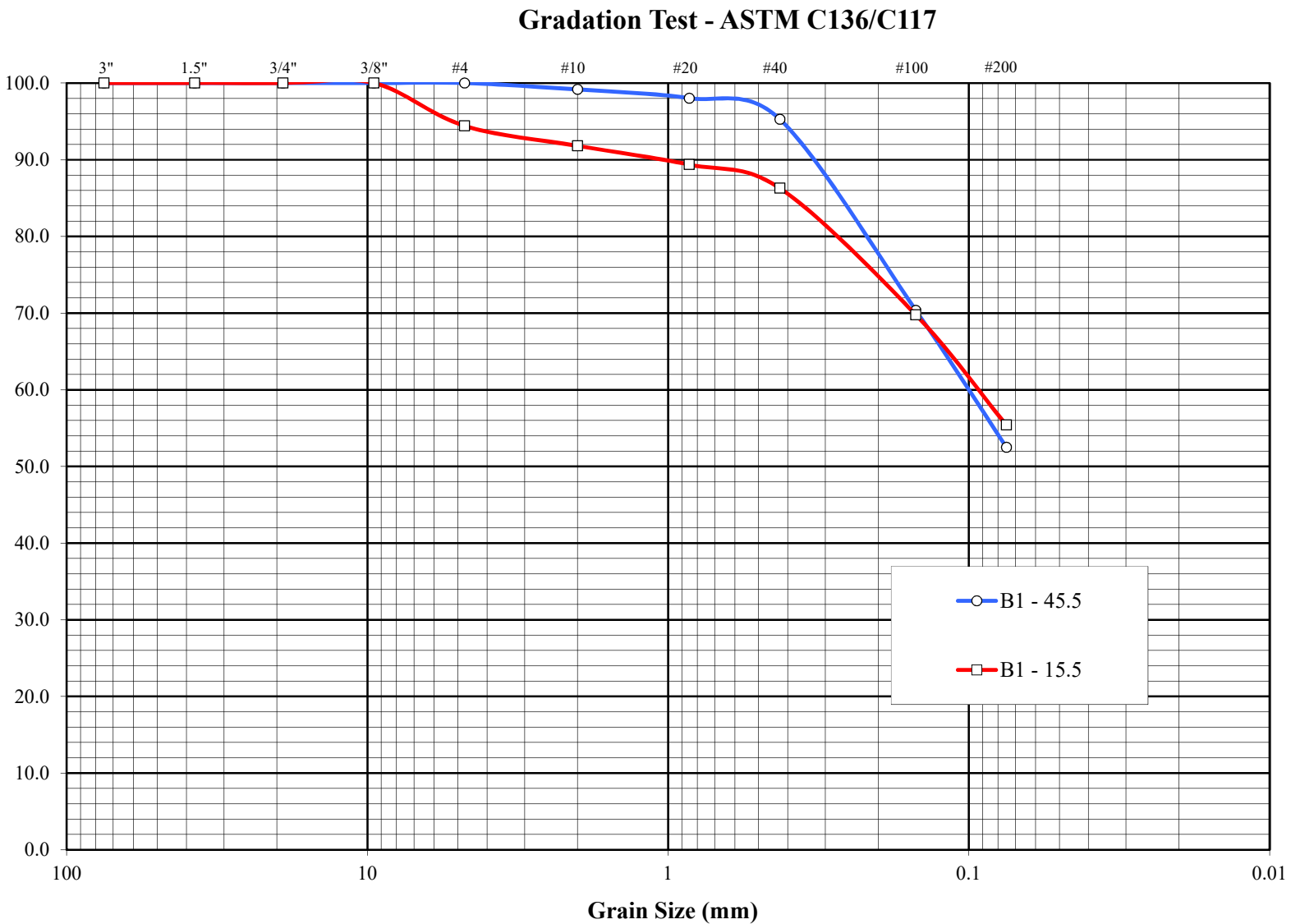
Wash #200	
Pan #	34
Pan Wt	253.6
Initial Dry Wt + Pan	484.9
Dry Wt after Wash + Pan	359.87
% Loss in Wash	54%

Project Name: Holly Hotel Group - San Carlos Project Number: 575-1547
Laboratory Number: 0575 (Oakland, CA) Date Tested: 9/18/2019
Sample Description: B-1 (45.5') Reviewed By:
Equipment List: Oven: VWR-1675 Scale: 01PS575/02PS575
Sieve Set: 01SV575 through 12SV575

	Sieve Size		Sample (circle one) :		Washed	Not Washed	
	English	Metric	Tare	Tare + Sample	Wt. Retained	% Retained	% Pass
Complete Stack + Sample Before							
Complete Stack + Sample After							
Coarse Sieves	3"				0.0	0.0%	100.0%
	1.5"	37.5mm			0.0	0.0%	100.0%
	3/4"	19mm			0.0	0.0%	100.0%
	3/8"	9.5mm			0.0	0.0%	100.0%
	#4	4.75mm			0.0	0.0%	100.0%
Coarse Pan Weight (a)			0.0	0.0			
Fines Split Weight (b)			0.0	114.2			
Factor = (a/b)				0.00			
Fine Sieves	#10	2.00mm		1.9	1.9	0.8%	99.2%
	#20	0.85mm		2.6	2.6	1.1%	98.0%
	#40	0.425mm		6.3	6.3	2.7%	95.3%
	#100	0.15mm		57.2	57.2	24.9%	70.3%
	#200	0.075mm		40.9	40.9	17.9%	52.5%
	Fine Catch Pan + Wash			5.3	120.3	52.5%	0.0%
Totals			0.0	114.2	229.3	100.0%	

Moisture Content	
Pan #	32
Pan Wt	273.8
Wet Sample + Pan	544.6
Dry Sample + Pan	501.94
% Moisture Content	19%

Wash #200	
Pan #	32
Pan Wt	273.8
Initial Dry Wt + Pan	501.94
Dry Wt after Wash + Pan	386.88
% Loss in Wash	50%



Professional Service Industries, Inc.

4703 Tidewater Avenue, Suite B
Oakland, California 94601

Gradation Test ASTM C136/C117

Holly Hotel Group
San Carlos, California

Project No. 575-1547

Date: Oct, 2019

Project Name: Holly Hotel Group - San Carlos

Project Number: 575-1547

Laboratory Number: 0575 (Oakland, CA)

Date Tested: 9/18/2019

Sample Description: B-1 Bulk (0-4')

Equipment List

Meter: H-4385

Scale: 01PS587

Initial Data	
Air Dried Mass of Sample (g)	1,483
Mass of Water Added (mL)	148
Amount of Time Hydrated (hrs)	24
Test Data	Reading on Resistivity Meter (Ω)
Initial	3,400
100 mL of Water	1,650
200 mL of Water	1,400
300 mL of Water	1,350
400 mL of Water	1,350
500 mL of Water	1,200
600 mL of Water	1,200
700 mL of Water	1,200 (Saturated)
800 mL of Water	
Final Test Results	
Lowest Reading	1,200

Tested By: Z.J.
Date: 9/18/2019

Reviewed By: B.B.
Date: 10/3/2019



25712 Commercentre Drive
Lake Forest, California 92630
949.297.5020 Phone
949.297.5027 Fax

16 September 2019

Brand Burfield
PSI -- Oakland
4703 Tidewater Ave Ste B
Oakland, CA 94601
RE: Holly Hotel - San Carlos

Enclosed are the results of analyses for samples received by the laboratory on 09/07/19 09:00. If you have any questions concerning this report, please feel free to contact me.

Sincerely,

Mike Jaroudi
Project Manager



25712 Commercentre Drive
Lake Forest, California 92630
949.297.5020 Phone
949.297.5027 Fax

PSI -- Oakland
4703 Tidewater Ave Ste B
Oakland CA, 94601

Project: Holly Hotel - San Carlos
Project Number: 575-1547
Project Manager: Brand Burfield

Reported:
09/16/19 09:08

ANALYTICAL REPORT FOR SAMPLES

Sample ID	Laboratory ID	Matrix	Date Sampled	Date Received
B1-BULK	T193140-01	Soil	08/29/19 00:00	09/07/19 09:00

SunStar Laboratories, Inc.

The results in this report apply to the samples analyzed in accordance with the chain of custody document. This analytical report must be reproduced in its entirety.

Mike Jaroudi, Project Manager



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PSI -- Oakland
4703 Tidewater Ave Ste B
Oakland CA, 94601

Project: Holly Hotel - San Carlos
Project Number: 575-1547
Project Manager: Brand Burfield

Reported:
09/16/19 09:08

DETECTIONS SUMMARY

Sample ID: B1-BULK

Laboratory ID: T193140-01

Analyte	Reporting		Units	Method	Notes
	Result	Limit			
pH	7.6	0.1	pH Units	EPA 9045B	O-04
Chloride	17.1	10.0	mg/kg	EPA 300.0	
Sulfate as SO4	202	10.0	mg/kg	EPA 300.0	

SunStar Laboratories, Inc.

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Mike Jaroudi, Project Manager



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PSI -- Oakland
4703 Tidewater Ave Ste B
Oakland CA, 94601

Project: Holly Hotel - San Carlos
Project Number: 575-1547
Project Manager: Brand Burfield

Reported:
09/16/19 09:08

B1-BULK
T193140-01 (Soil)

Analyte	Result	Reporting Limit	Units	Dilution	Batch	Prepared	Analyzed	Method	Notes
---------	--------	--------------------	-------	----------	-------	----------	----------	--------	-------

SunStar Laboratories, Inc.

Conventional Chemistry Parameters by APHA/EPA/ASTM Methods

pH	7.6	0.1	pH Units	1	9090911	09/09/19	09/09/19	EPA 9045B	O-04
----	-----	-----	----------	---	---------	----------	----------	-----------	------

Anion Scan by EPA Method 300.0

Chloride	17.1	10.0	mg/kg	1	9090915	09/09/19	09/09/19	EPA 300.0	
Sulfate as SO4	202	10.0	"	"	"	"	"	"	

SunStar Laboratories, Inc.

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Mike Jaroudi, Project Manager



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4703 Tidewater Ave Ste B
Oakland CA, 94601

Project: Holly Hotel - San Carlos
Project Number: 575-1547
Project Manager: Brand Burfield

Reported:
09/16/19 09:08

Conventional Chemistry Parameters by APHA/EPA/ASTM Methods - Quality Control

SunStar Laboratories, Inc.

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
---------	--------	--------------------	-------	----------------	------------------	------	----------------	-----	--------------	-------

Batch 9090911 - General Preparation

Duplicate (9090911-DUP1)

Source: T193140-01

Prepared & Analyzed: 09/09/19

pH	7.63	0.1	pH Units	7.62	0.131	20	O-04
----	------	-----	----------	------	-------	----	------

SunStar Laboratories, Inc.

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Mike Jaroudi, Project Manager



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PSI -- Oakland
4703 Tidewater Ave Ste B
Oakland CA, 94601

Project: Holly Hotel - San Carlos
Project Number: 575-1547
Project Manager: Brand Burfield

Reported:
09/16/19 09:08

Anion Scan by EPA Method 300.0 - Quality Control

SunStar Laboratories, Inc.

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
---------	--------	-----------------	-------	-------------	---------------	------	-------------	-----	-----------	-------

Batch 9090915 - General Preparation

Blank (9090915-BLK1)

Prepared & Analyzed: 09/09/19

Chloride	ND	10.0	mg/kg							
Sulfate as SO4	ND	10.0	"							

LCS (9090915-BS1)

Prepared & Analyzed: 09/09/19

Chloride	258	10.0	mg/kg	250		103	70-130			
Sulfate as SO4	279	10.0	"	250		112	70-130			

Matrix Spike (9090915-MS1)

Source: T193140-01

Prepared & Analyzed: 09/09/19

Chloride	264	10.0	mg/kg	250	17.1	98.6	70-130			
Sulfate as SO4	362	10.0	"	250	202	64.1	70-130			QM-07

Matrix Spike Dup (9090915-MSD1)

Source: T193140-01

Prepared & Analyzed: 09/09/19

Chloride	243	10.0	mg/kg	231	17.1	97.7	70-130	8.01	20	
Sulfate as SO4	412	10.0	"	231	202	90.6	70-130	12.8	20	

SunStar Laboratories, Inc.

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Mike Jaroudi, Project Manager



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PSI -- Oakland
4703 Tidewater Ave Ste B
Oakland CA, 94601

Project: Holly Hotel - San Carlos
Project Number: 575-1547
Project Manager: Brand Burfield

Reported:
09/16/19 09:08

Notes and Definitions

QM-07 The spike recovery and or RPD was outside acceptance limits for the MS and/or MSD. The batch was accepted based on acceptable LCS recovery.

O-04 This sample was received and analyzed outside the EPA recommended holding time.

DET Analyte DETECTED

ND Analyte NOT DETECTED at or above the reporting limit

NR Not Reported

dry Sample results reported on a dry weight basis

RPD Relative Percent Difference

SunStar Laboratories, Inc.

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Mike Jaroudi, Project Manager



Chain of Custody Record

Date: 9/5/19 Page: 1 Of 1
Project Name: HOLLY HOTEL - SAN CARLOS
Collector: Z. JACO Client Project #: 575-1547
Batch #: 7193140 EDF #: _____

[illegible]

Sample disposal Instructions: Disposal @ \$2.00 each _____ Return to client _____ Pickup _____

COC 170385



APPENDIX C

LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS REPORT

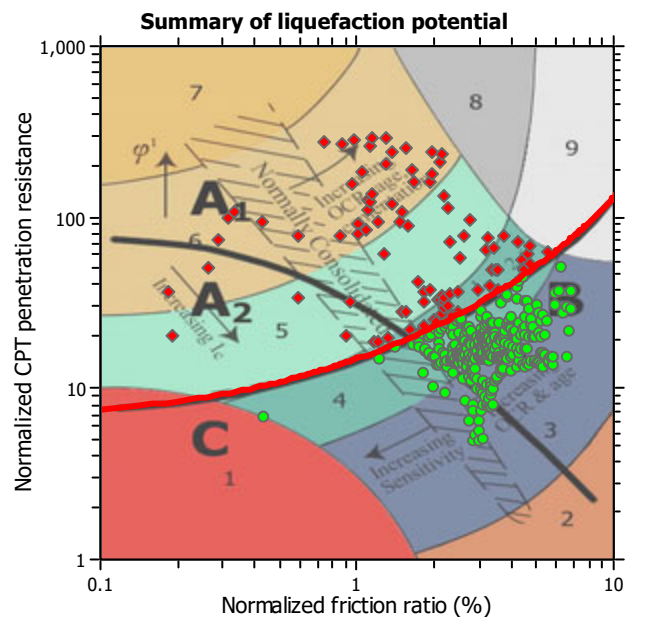
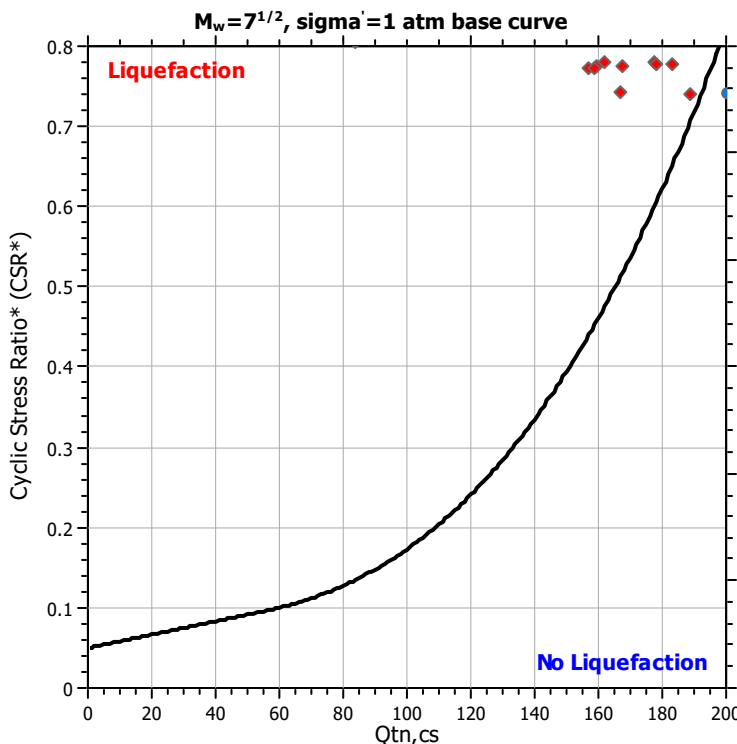
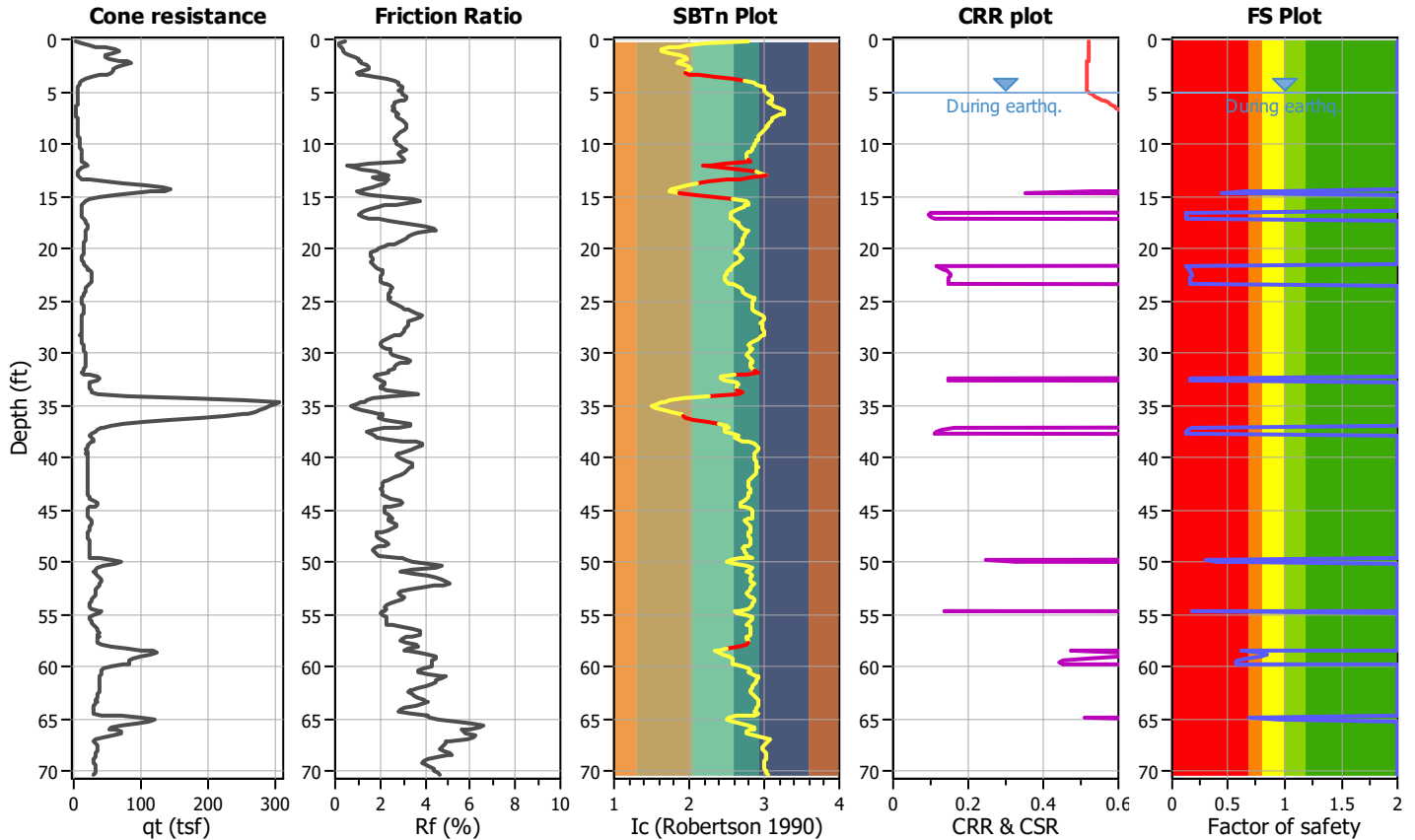
Project title : Holly Hotel Group

Location : 501 INDUSTRIAL ROAD, SAN CARLOS, CA

CPT file : CPT-2

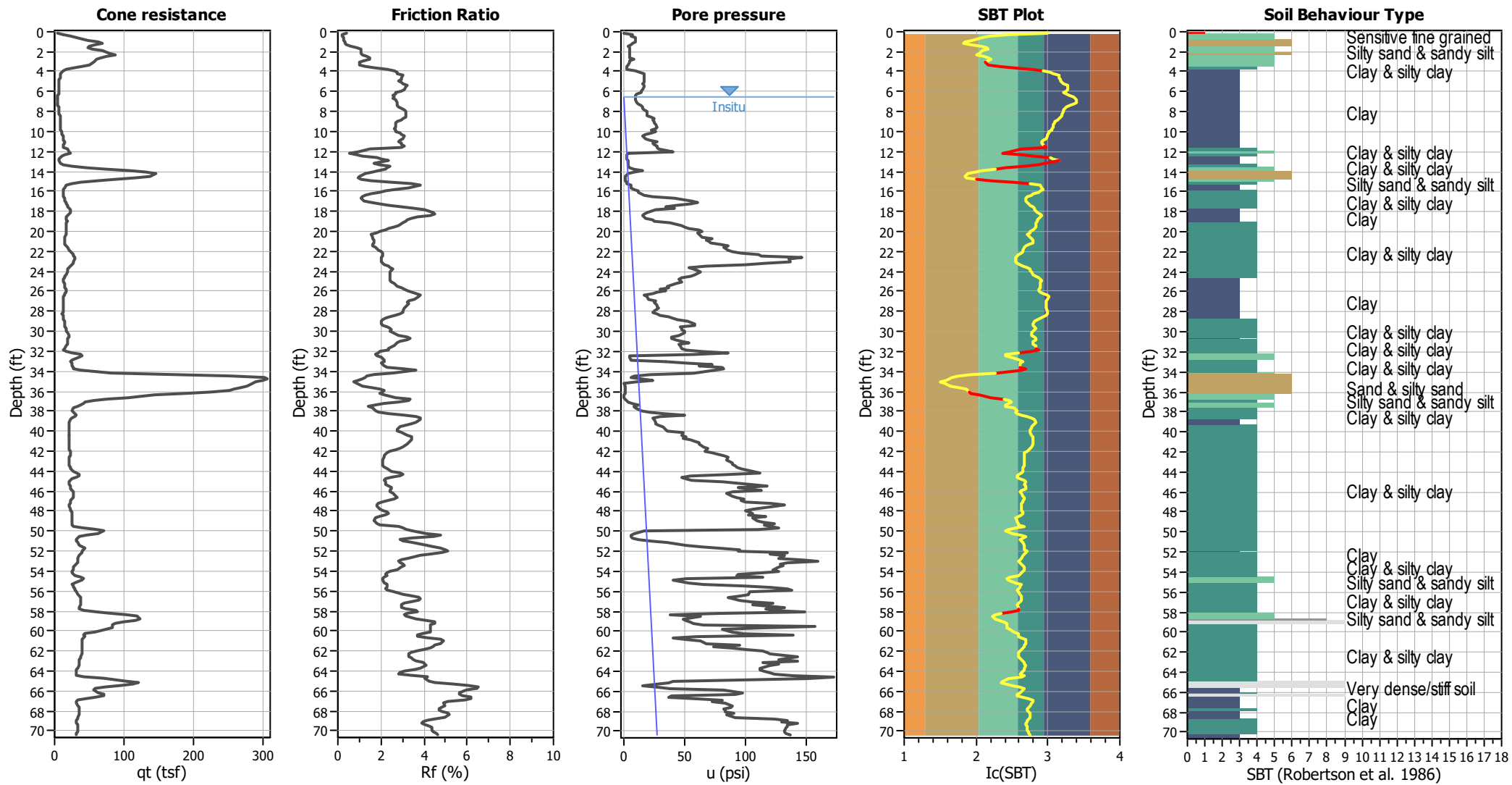
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	6.50 ft	Use fill:	No	Clay like behavior	
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.70	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots

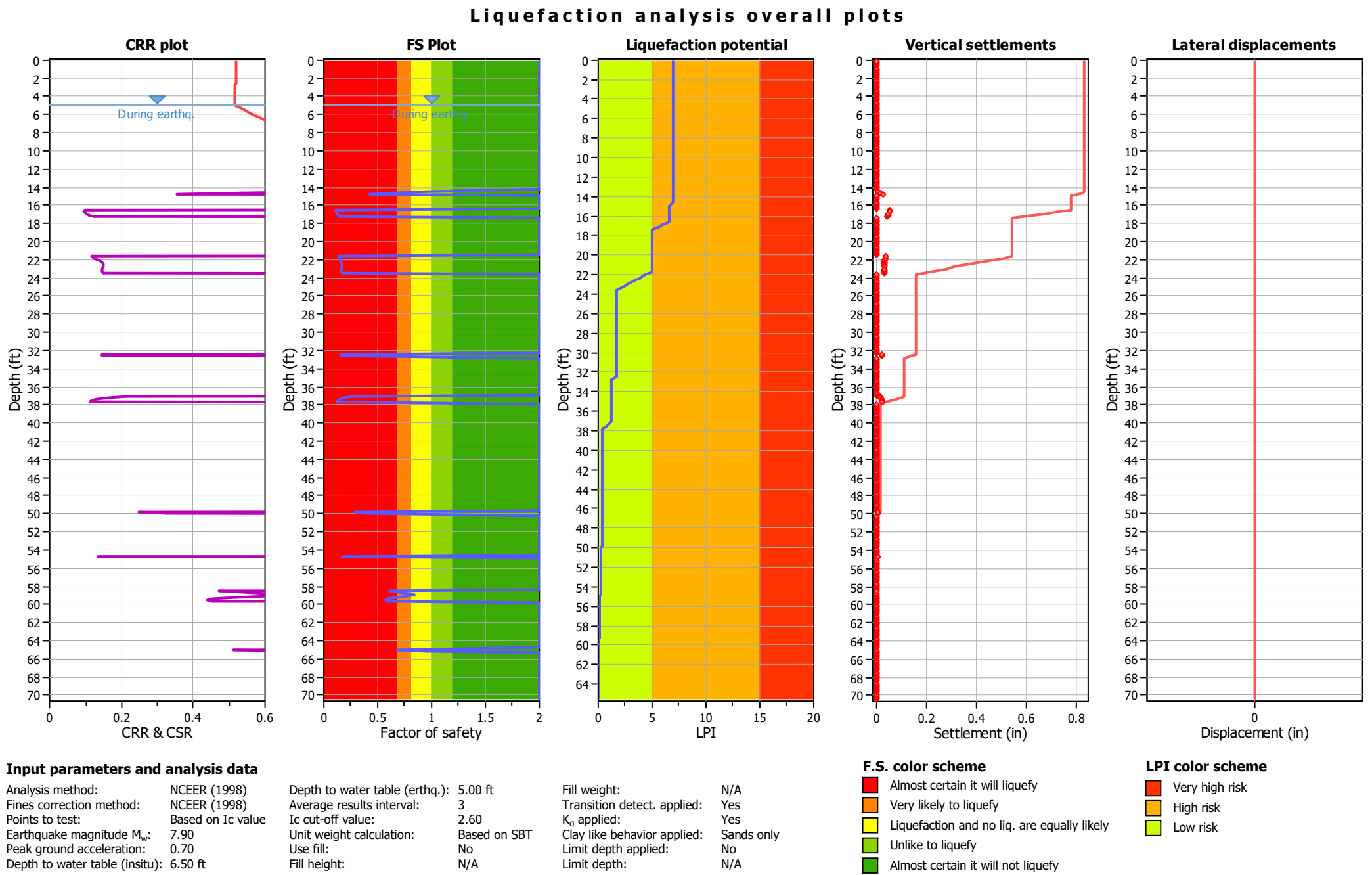


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.50 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



LIQUEFACTION ANALYSIS REPORT

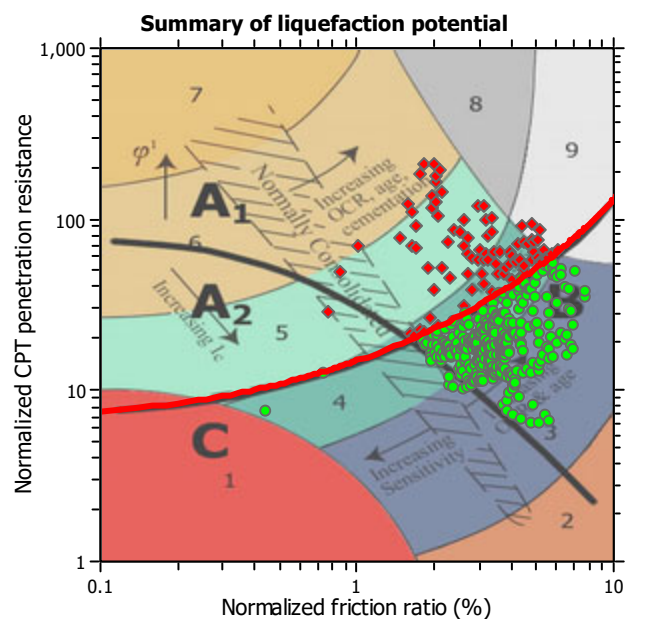
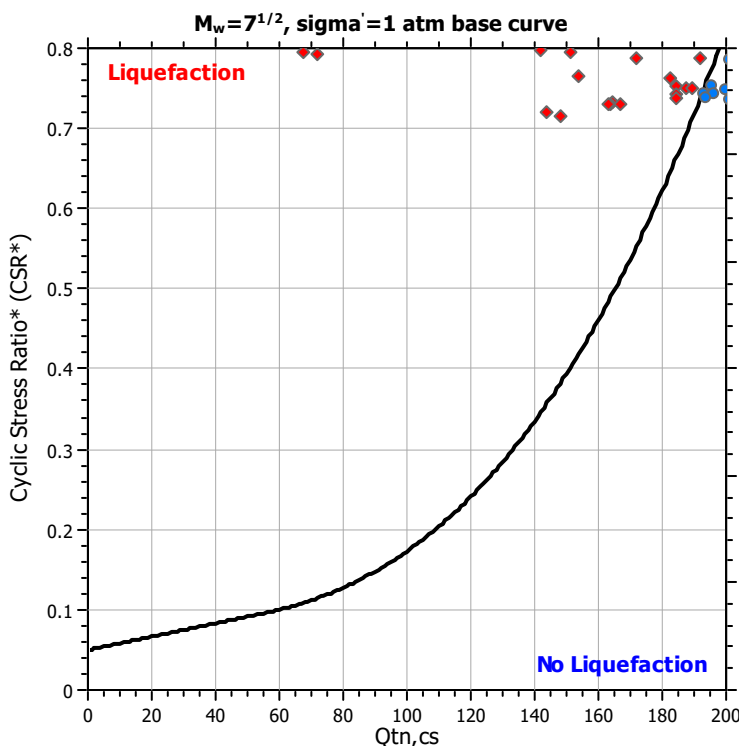
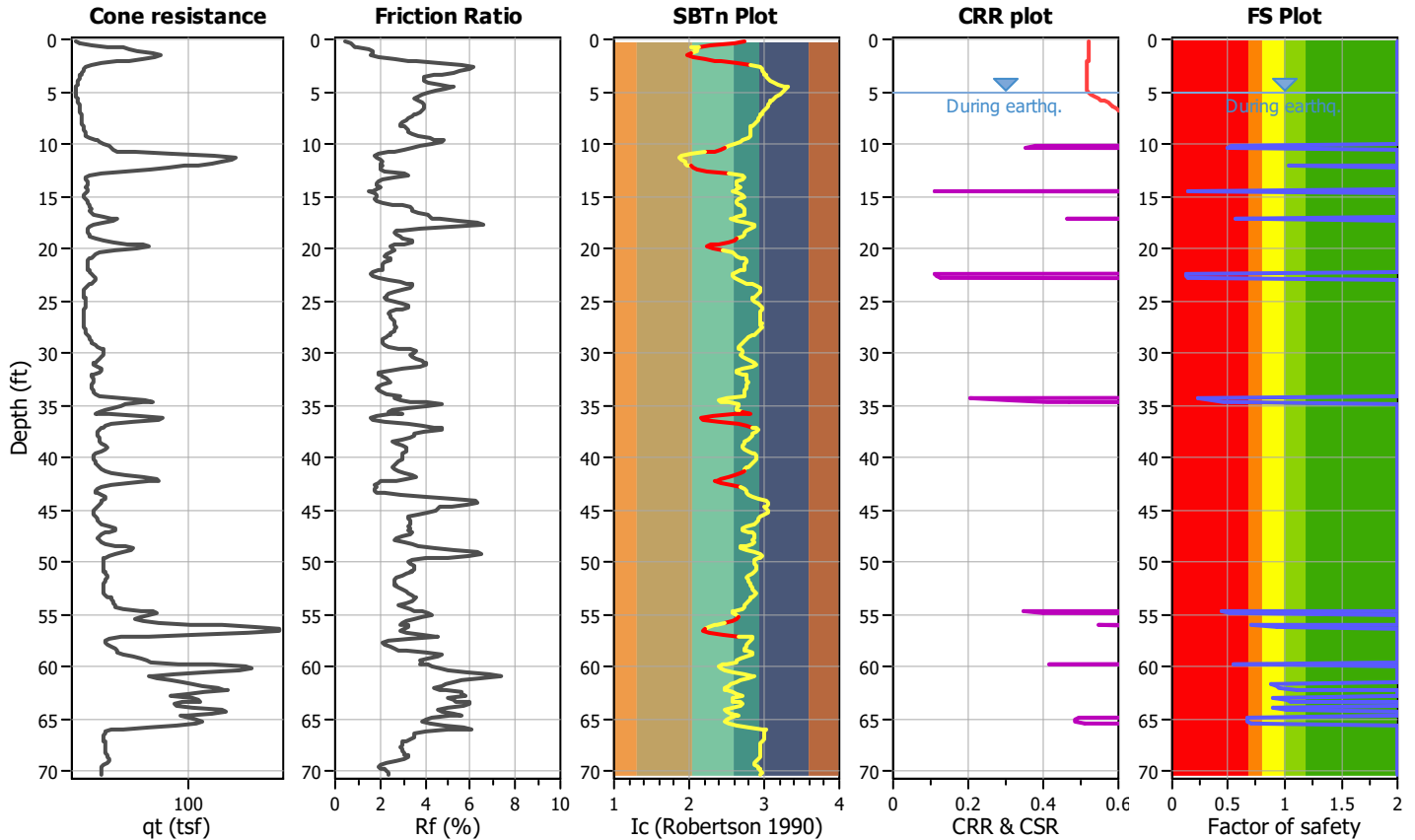
Project title : Holly Hotel Group

Location : 501 INDUSTRIAL ROAD, SAN CARLOS, CA

CPT file : CPT-3

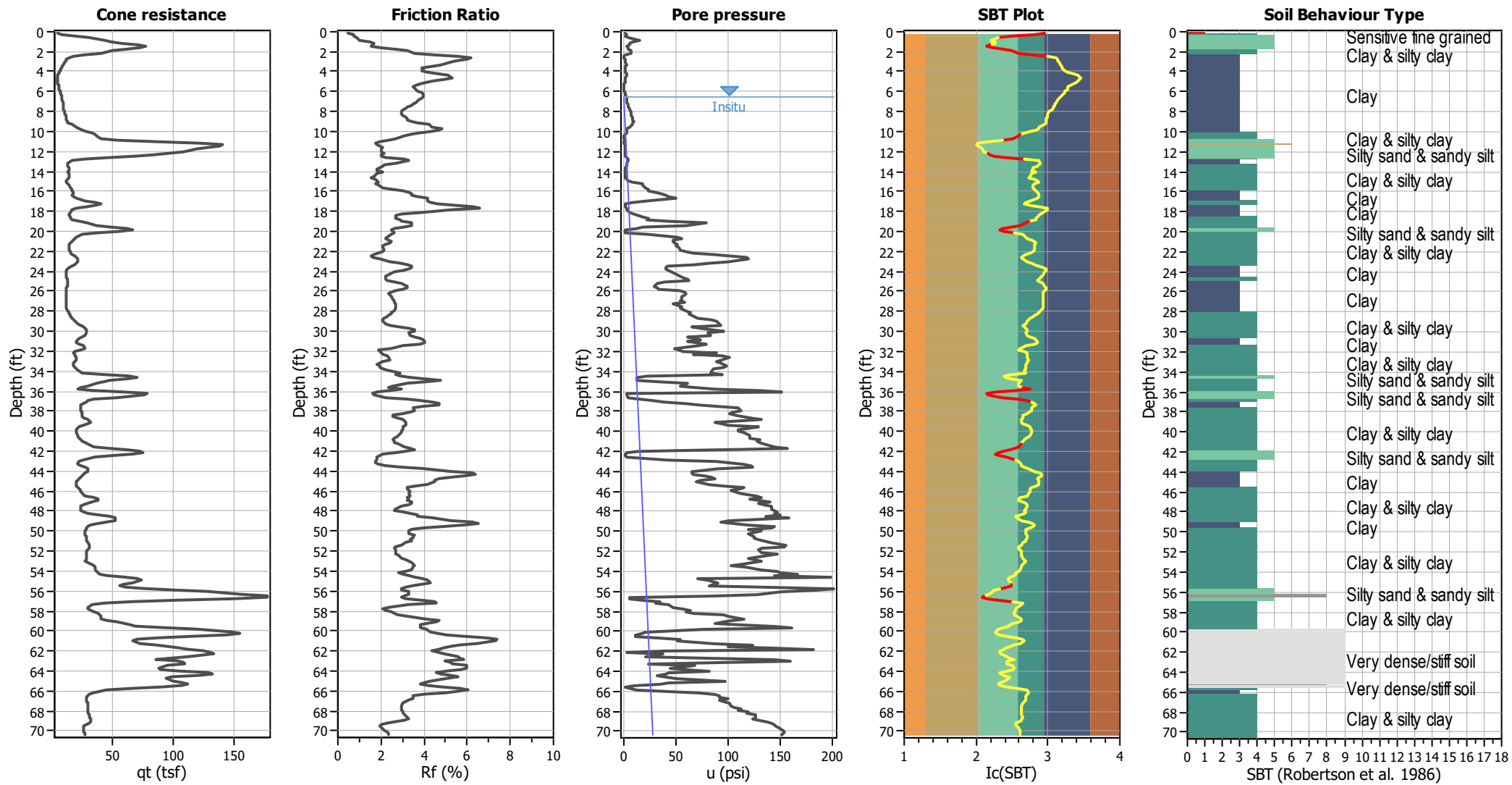
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	6.50 ft	Use fill:	No	Clay like behavior	
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.70	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots

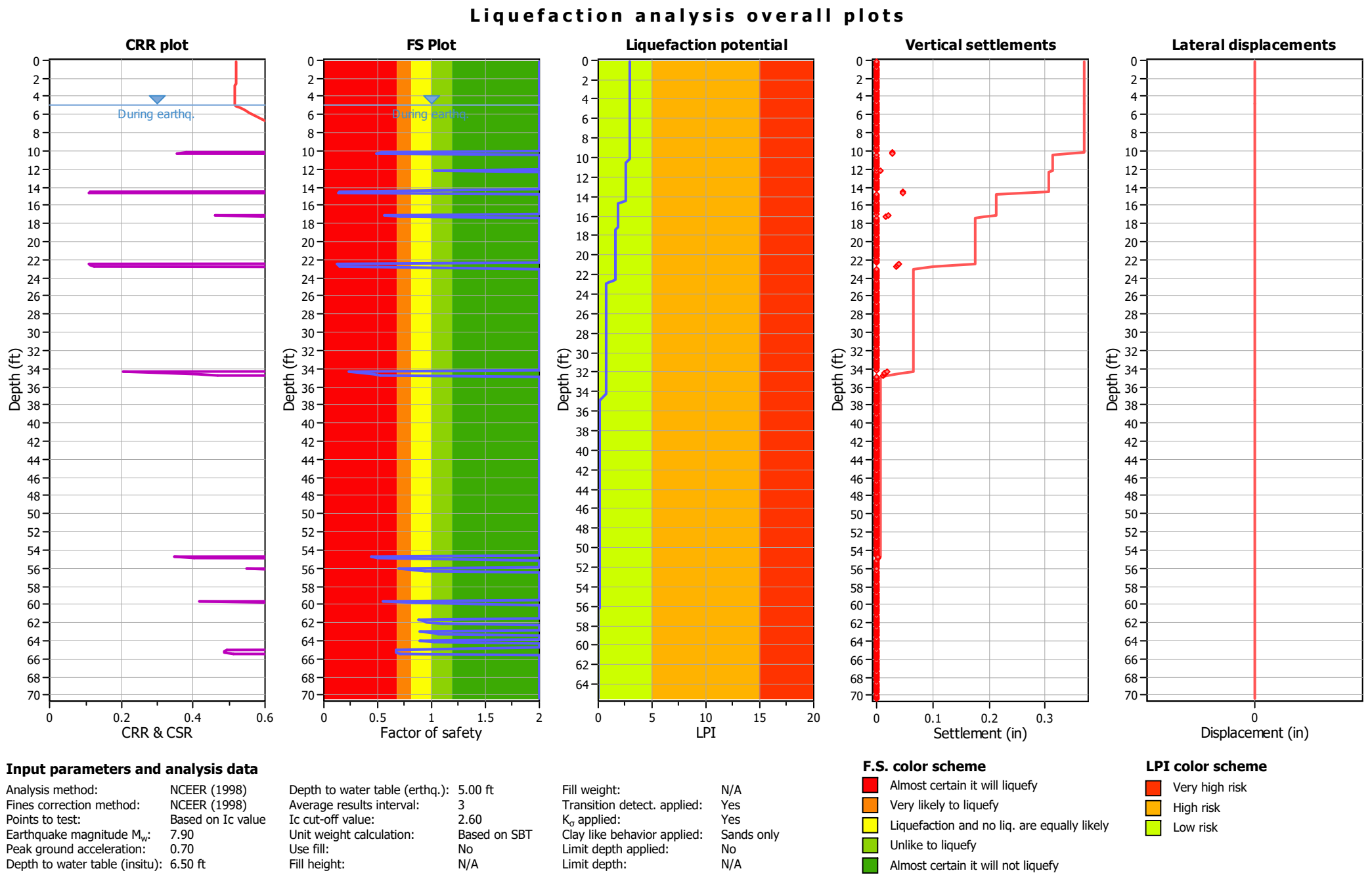


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _σ applied:	Yes
Earthquake magnitude M _w :	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.50 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



LIQUEFACTION ANALYSIS REPORT

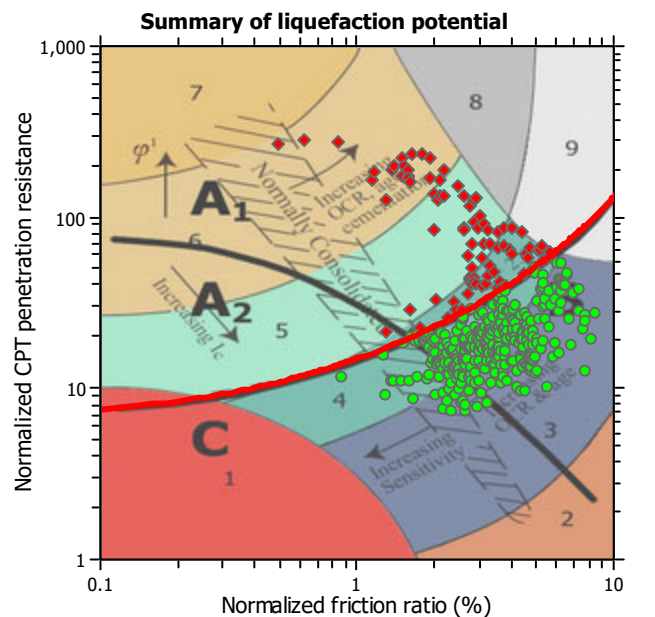
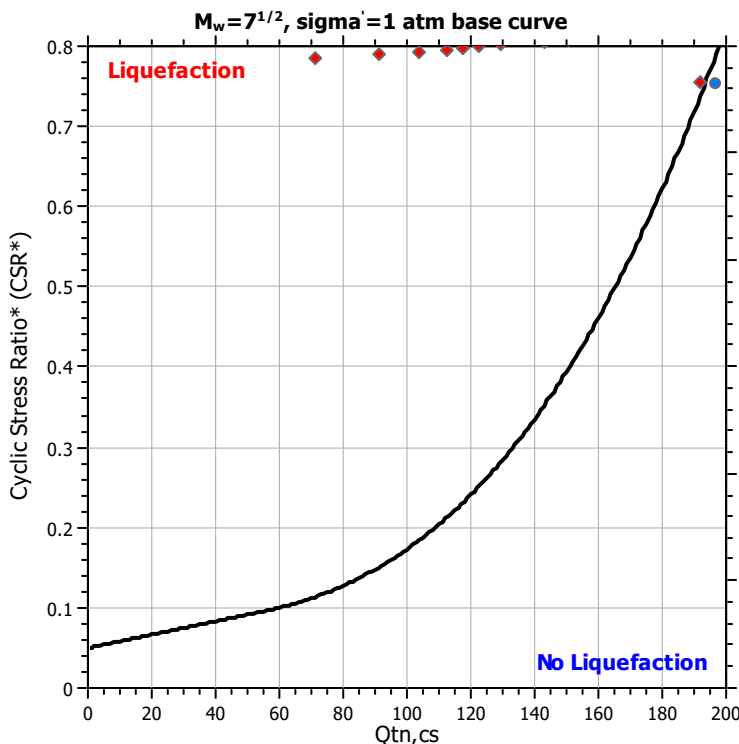
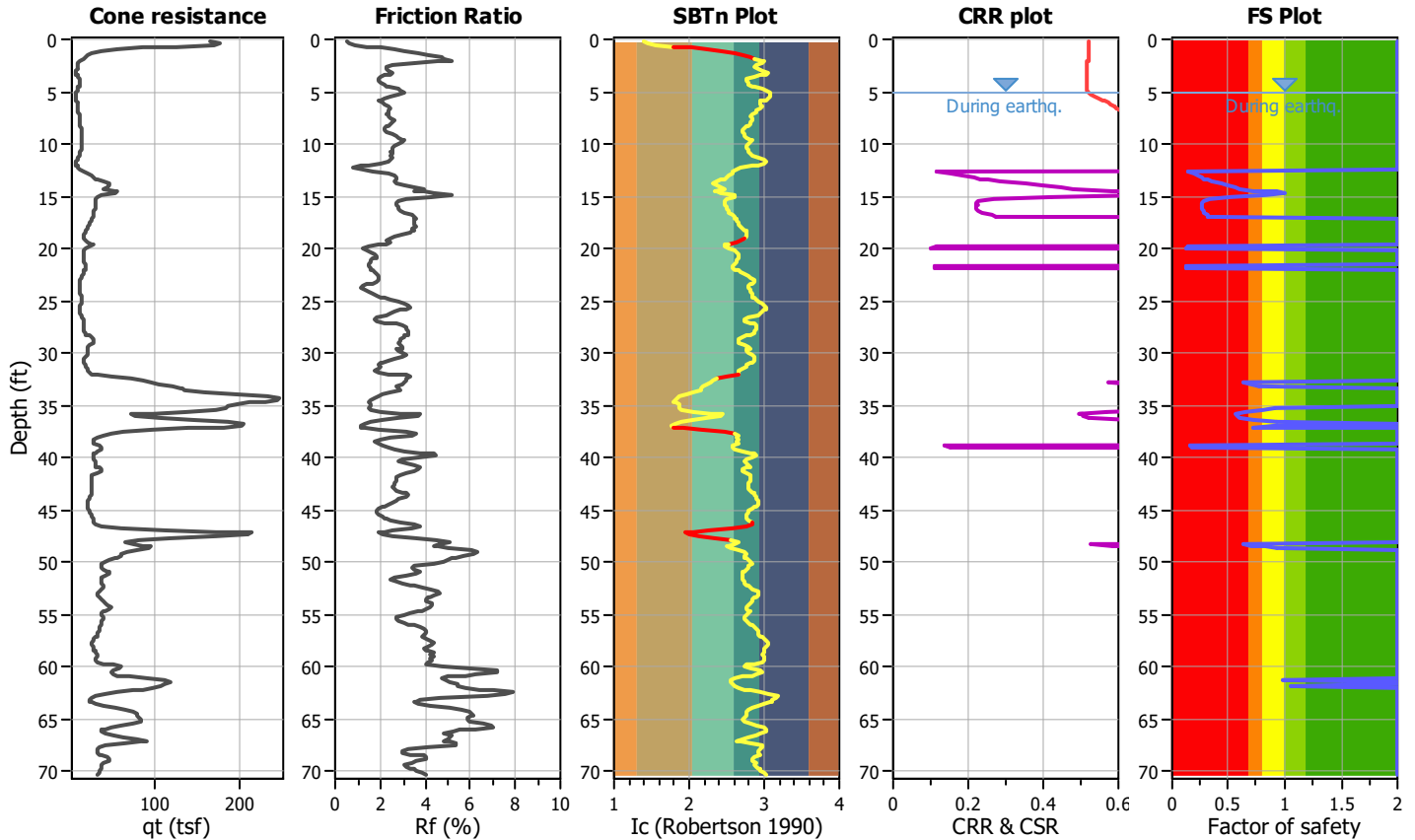
Project title : Holly Hotel Group

Location : 501 INDUSTRIAL ROAD, SAN CARLOS, CA

CPT file : CPT-4

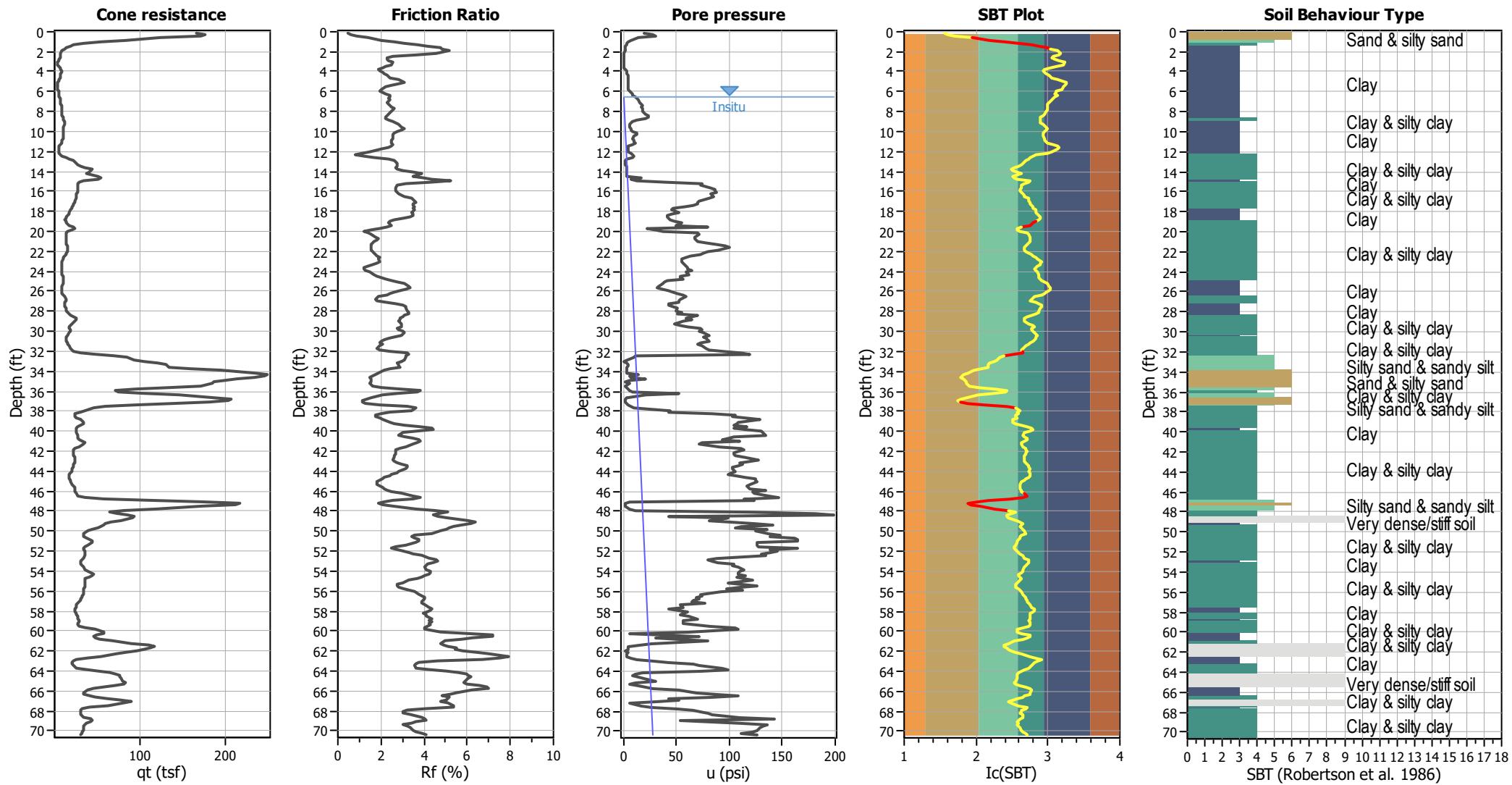
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	6.50 ft	Use fill:	No	Clay like behavior	
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.70	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based



Zone A1: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots

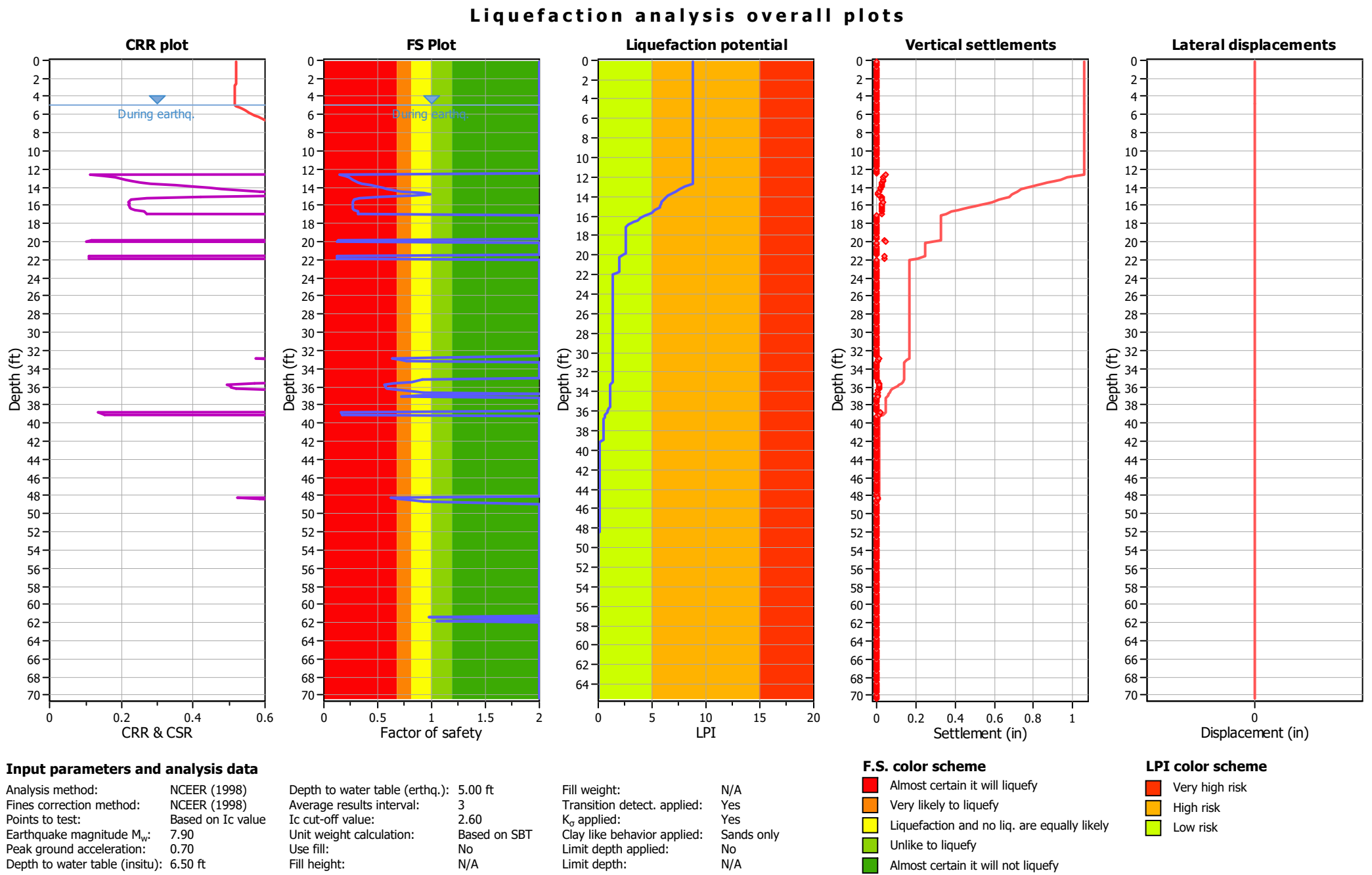


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _g applied:	Yes
Earthquake magnitude M _w :	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.50 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



LIQUEFACTION ANALYSIS REPORT

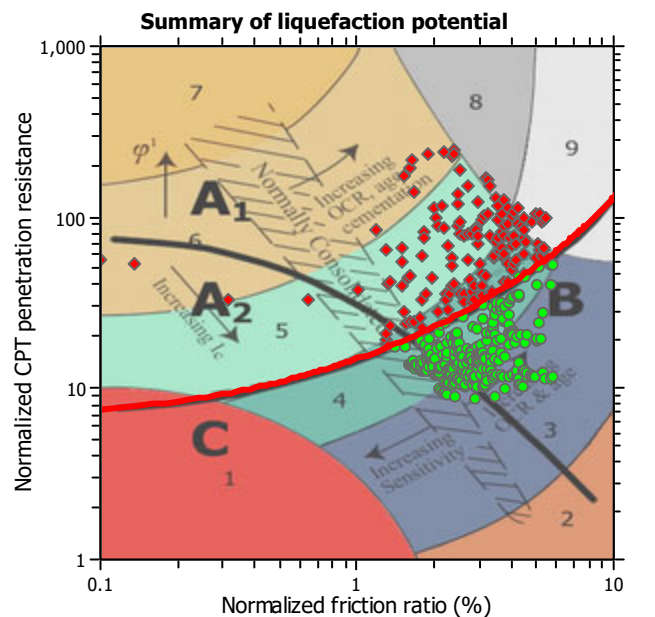
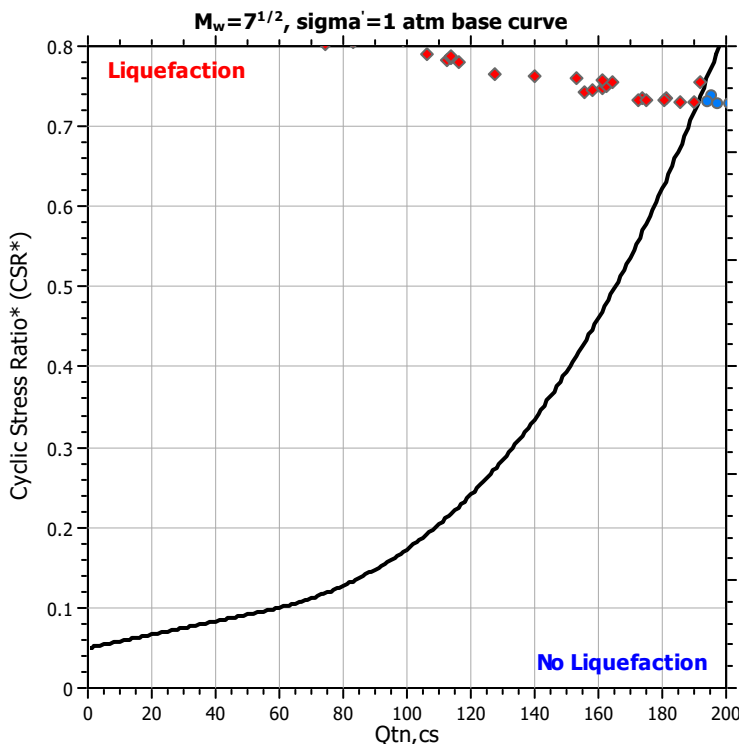
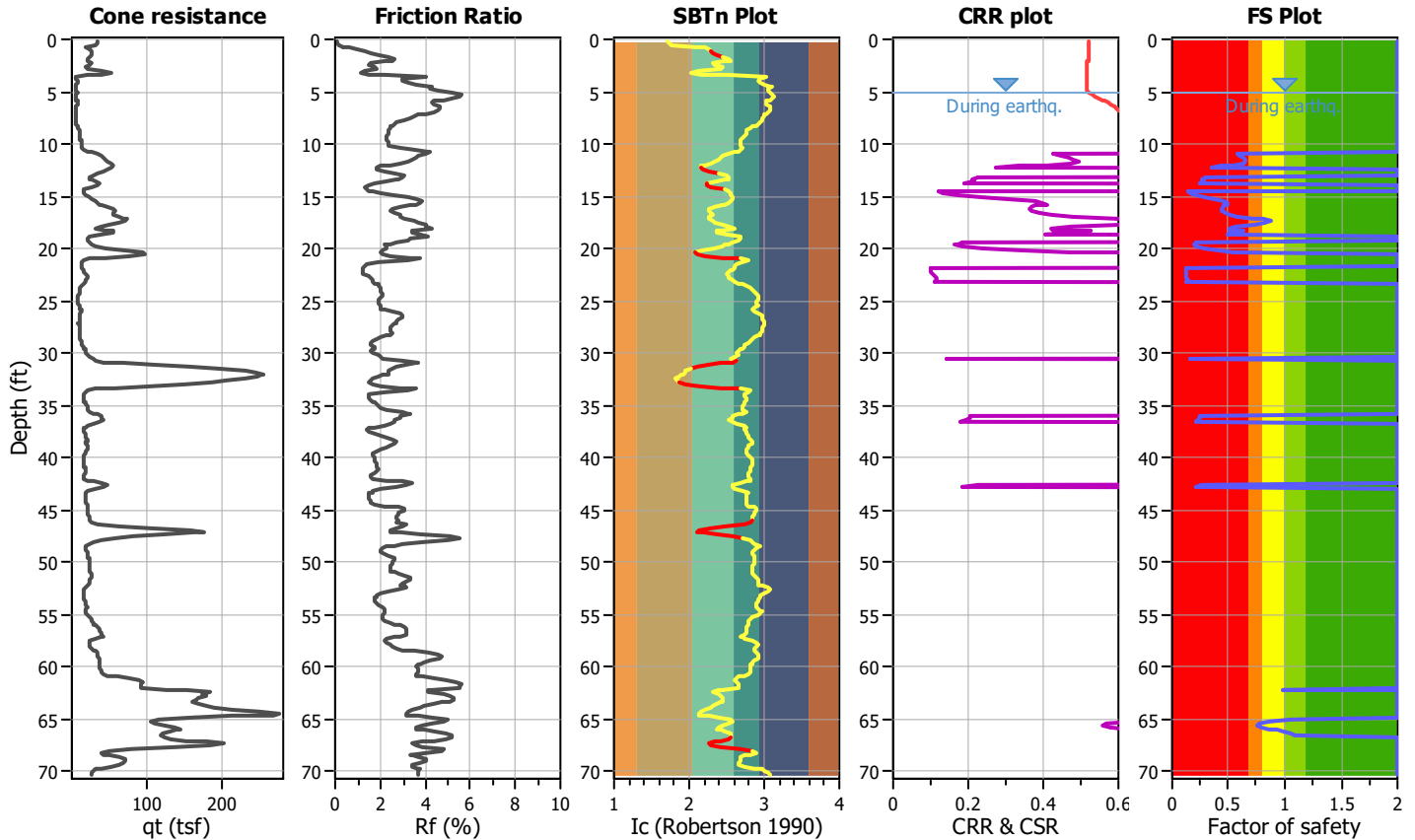
Project title : Holly Hotel Group

Location : 501 INDUSTRIAL ROAD, SAN CARLOS, CA

CPT file : CPT-5

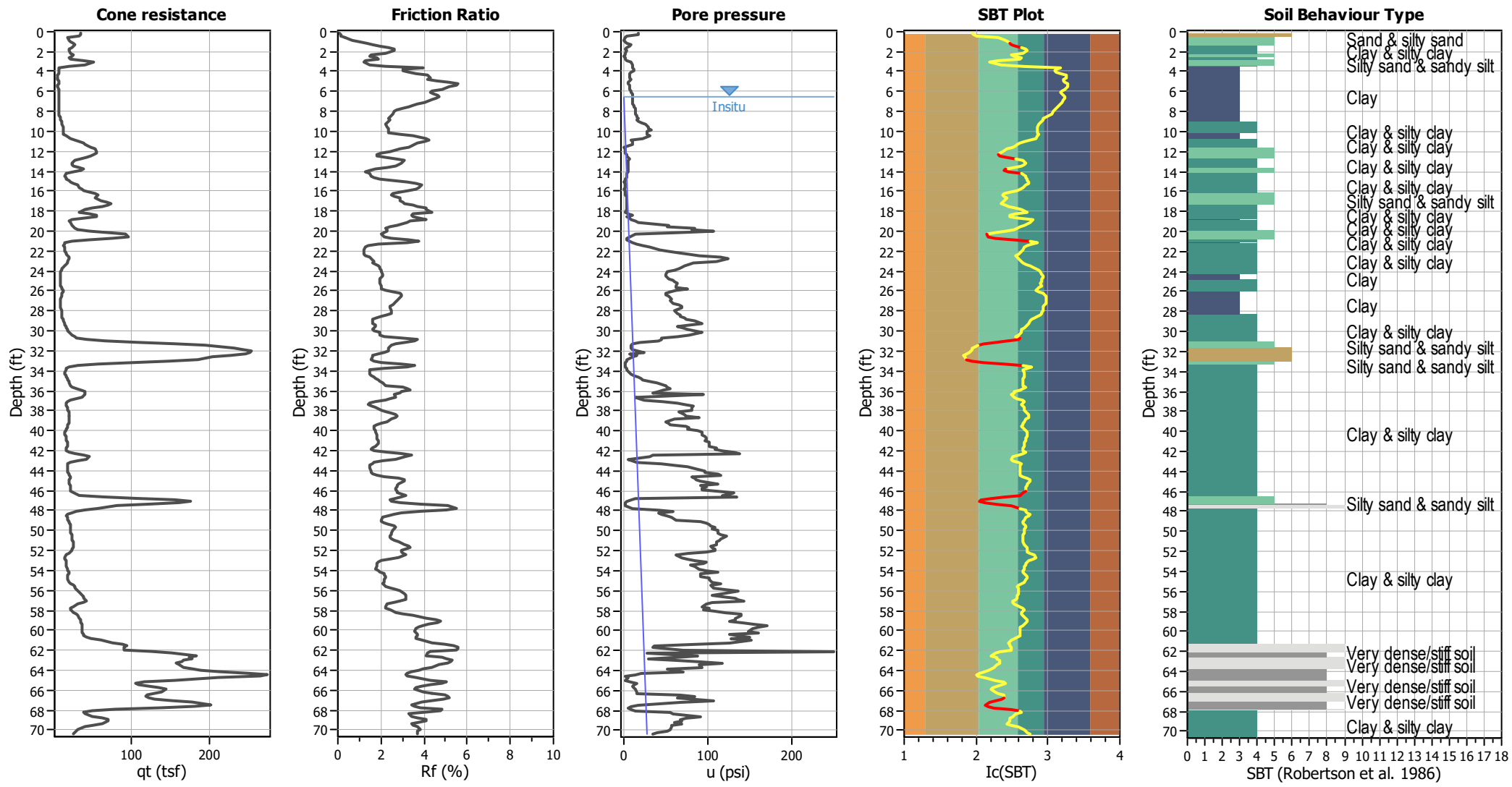
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	6.50 ft	Use fill:	No	Clay like behavior	
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.70	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based



Zone A1: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots

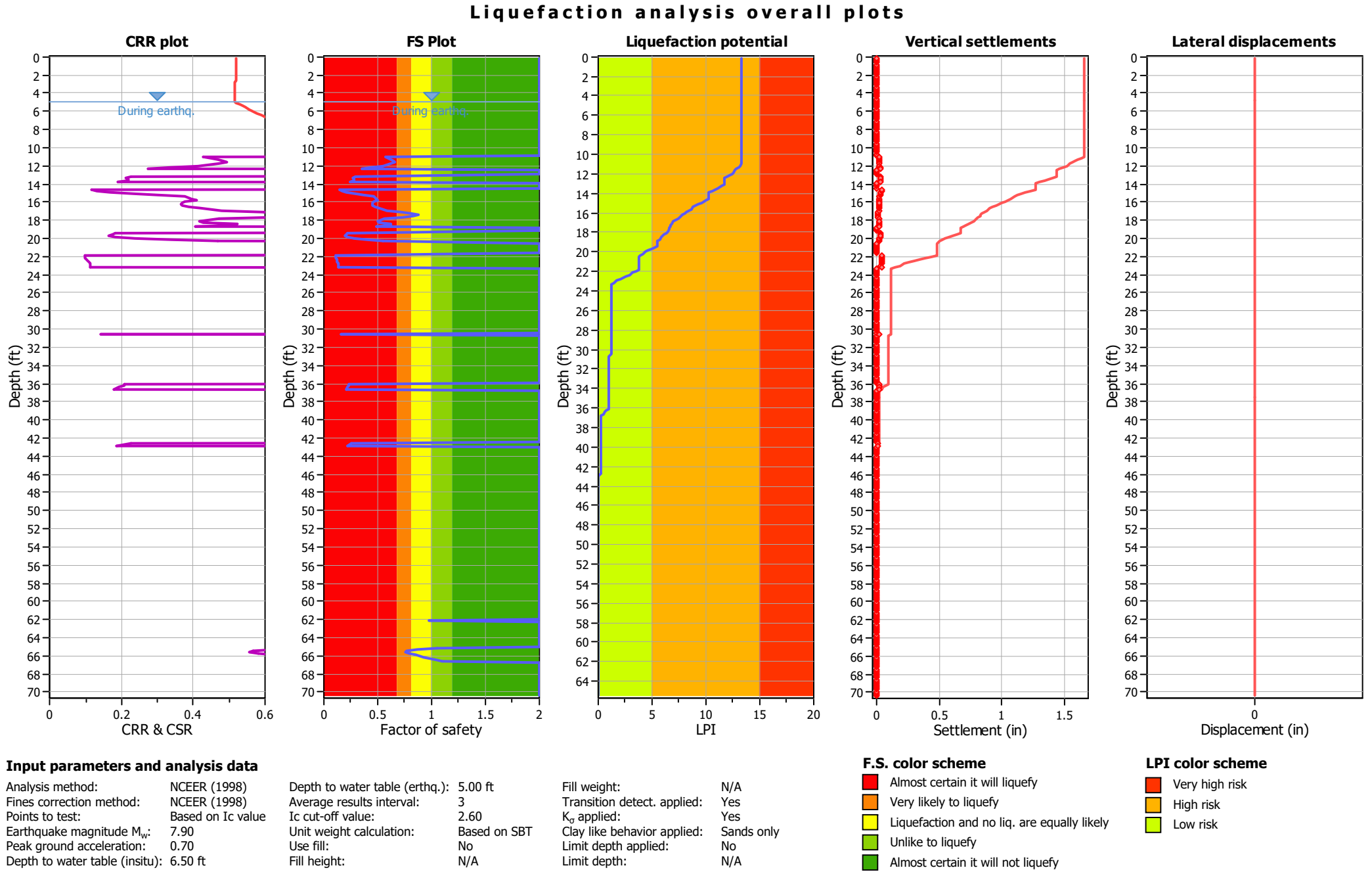


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on I_c value	I_c cut-off value:	2.60	K_g applied:	Yes
Earthquake magnitude M_w :	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.50 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



LIQUEFACTION ANALYSIS REPORT

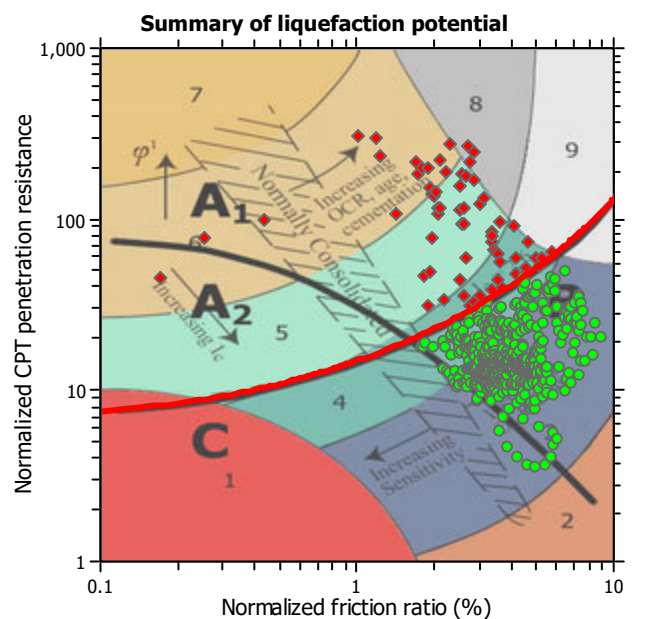
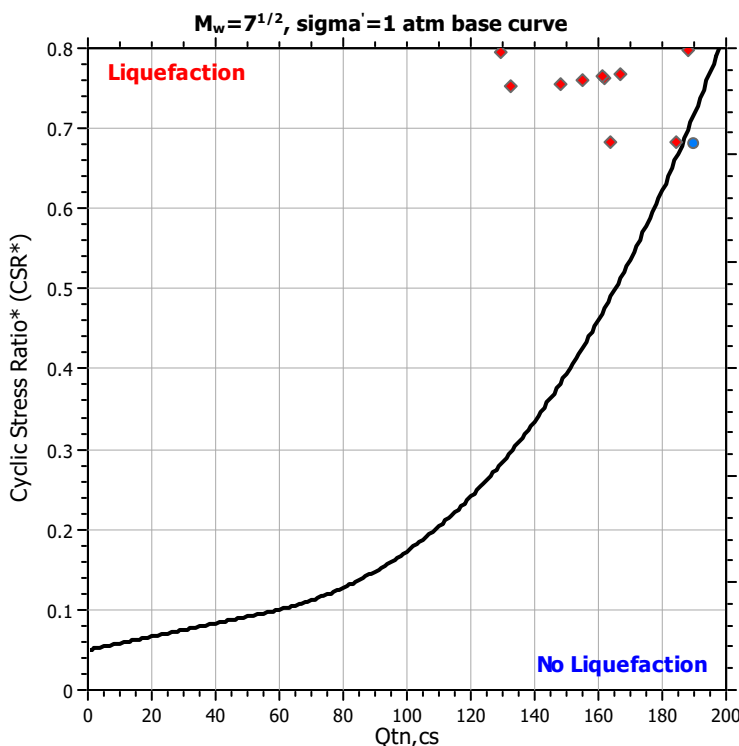
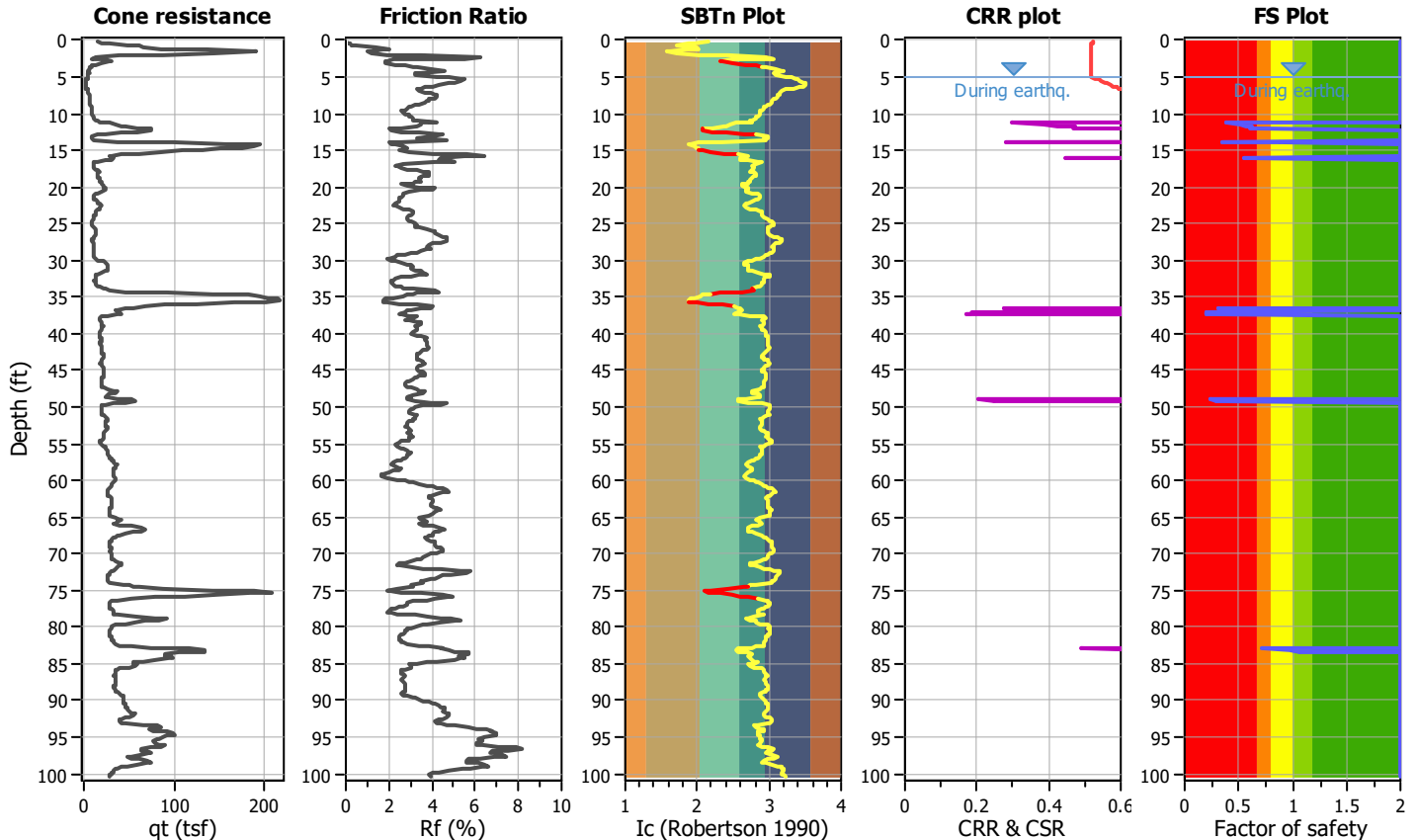
Project title : Holly Hotel Group

Location : 501 INDUSTRIAL ROAD, SAN CARLOS, CA

CPT file : SCPT-1

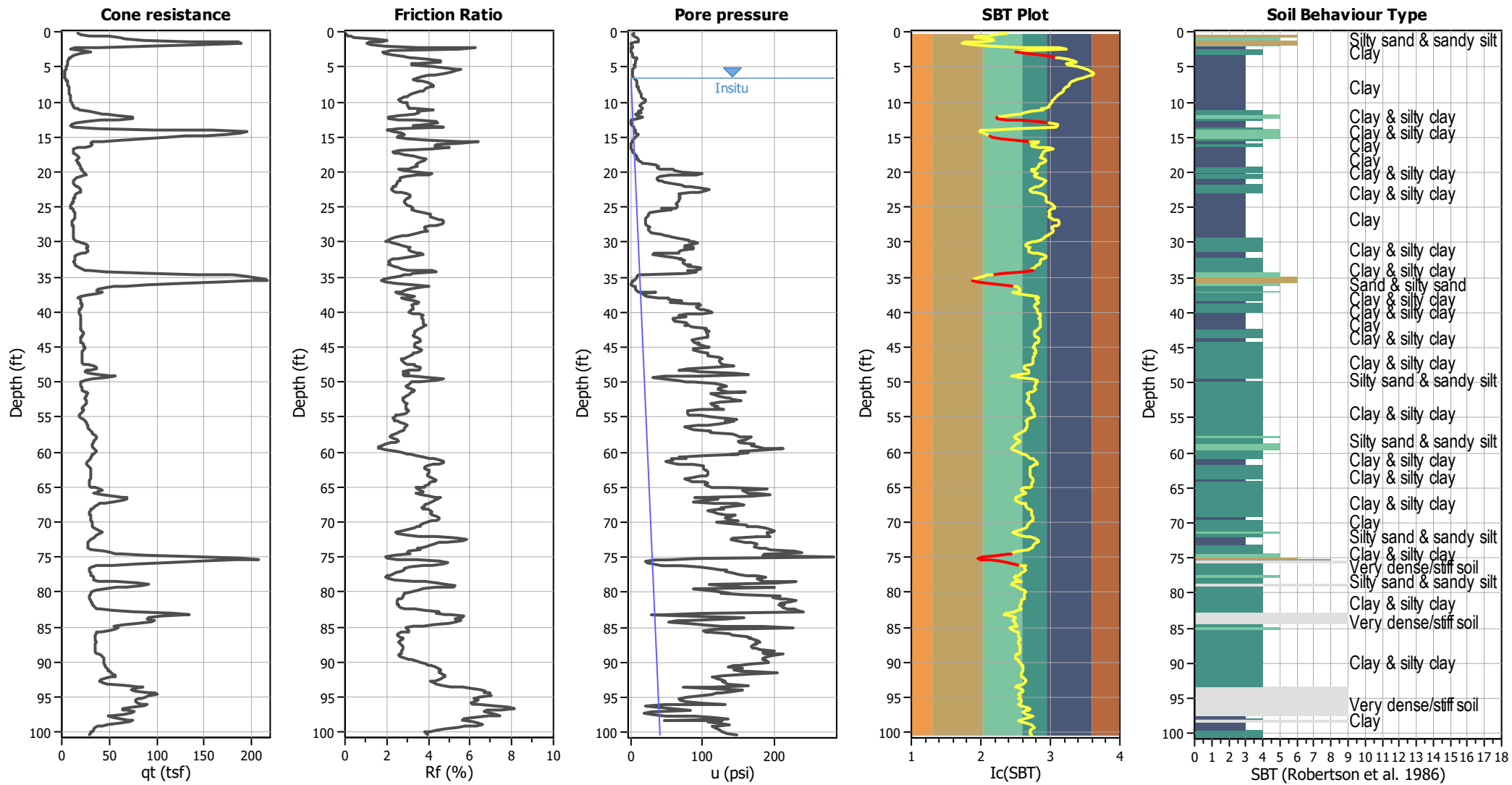
Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	6.50 ft	Use fill:	No	Clay like behavior	
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	5.00 ft	Fill height:	N/A	applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.70	Unit weight calculation:	Based on SBT	K_0 applied:	Yes	MSF method:	Method based



Zone A₁: Cyclic liquefaction likely depending on size and duration of cyclic loading
 Zone A₂: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
 Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
 Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots

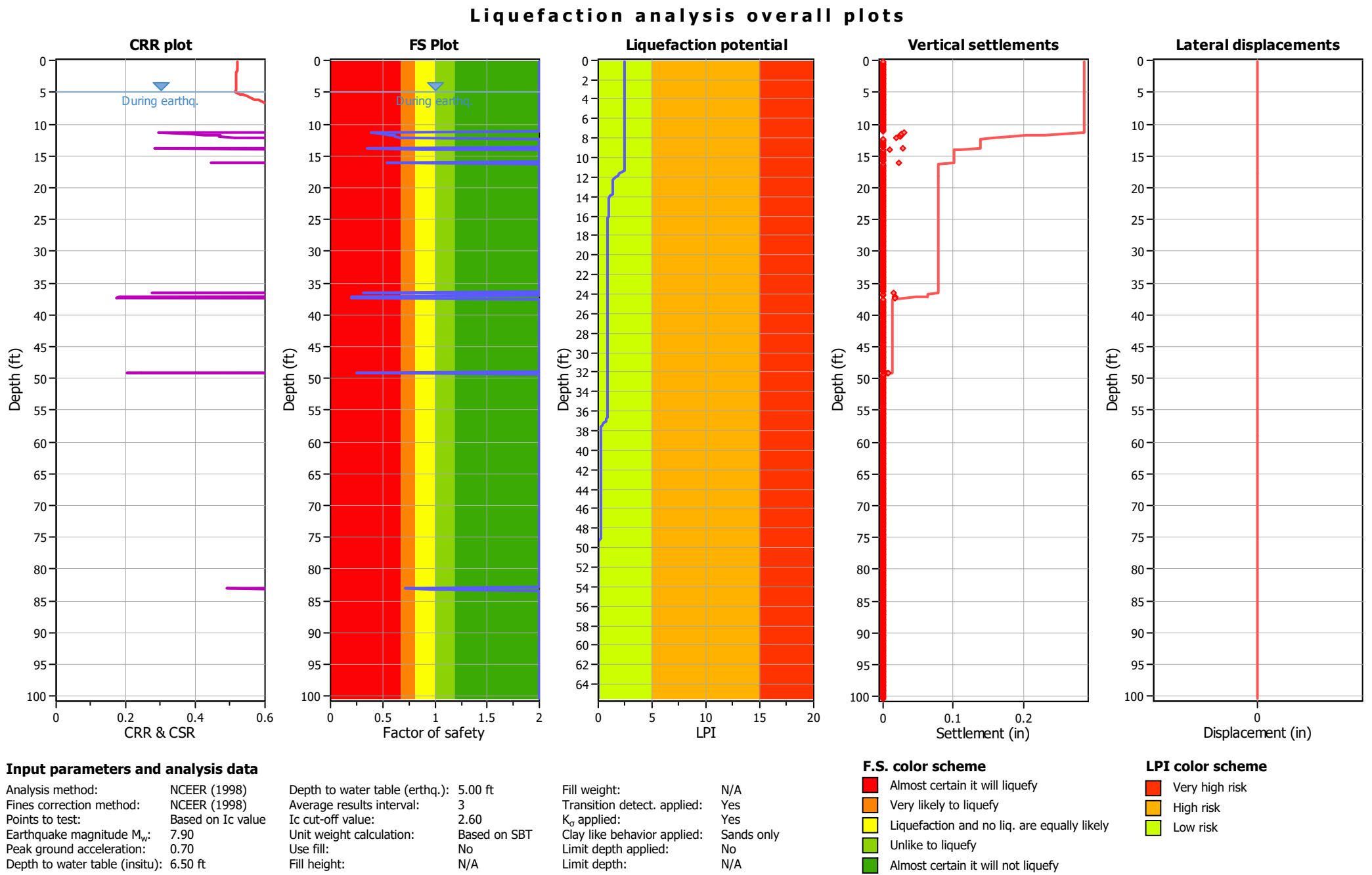


Input parameters and analysis data

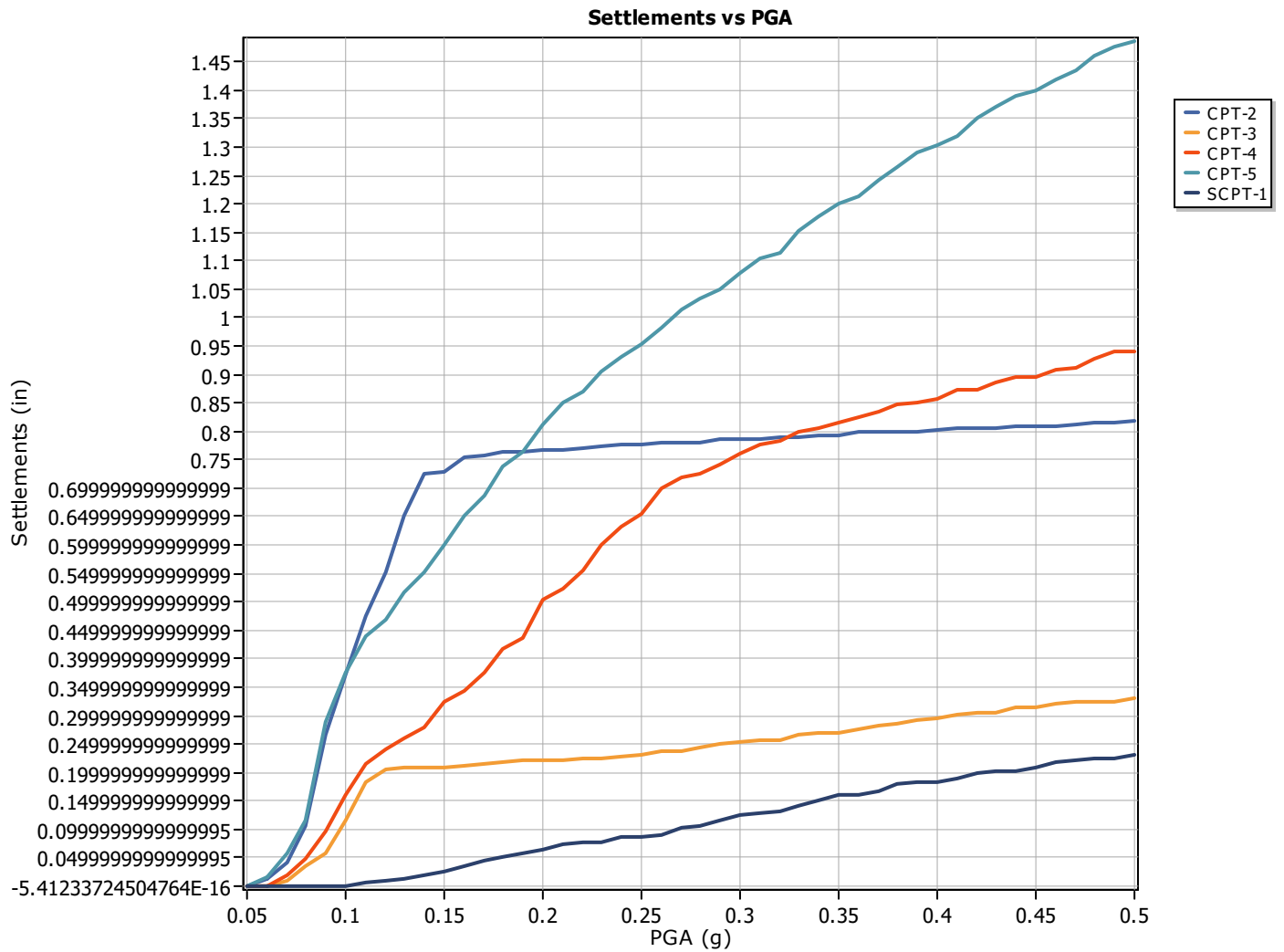
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	5.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _g applied:	Yes
Earthquake magnitude M _w :	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.70	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	6.50 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



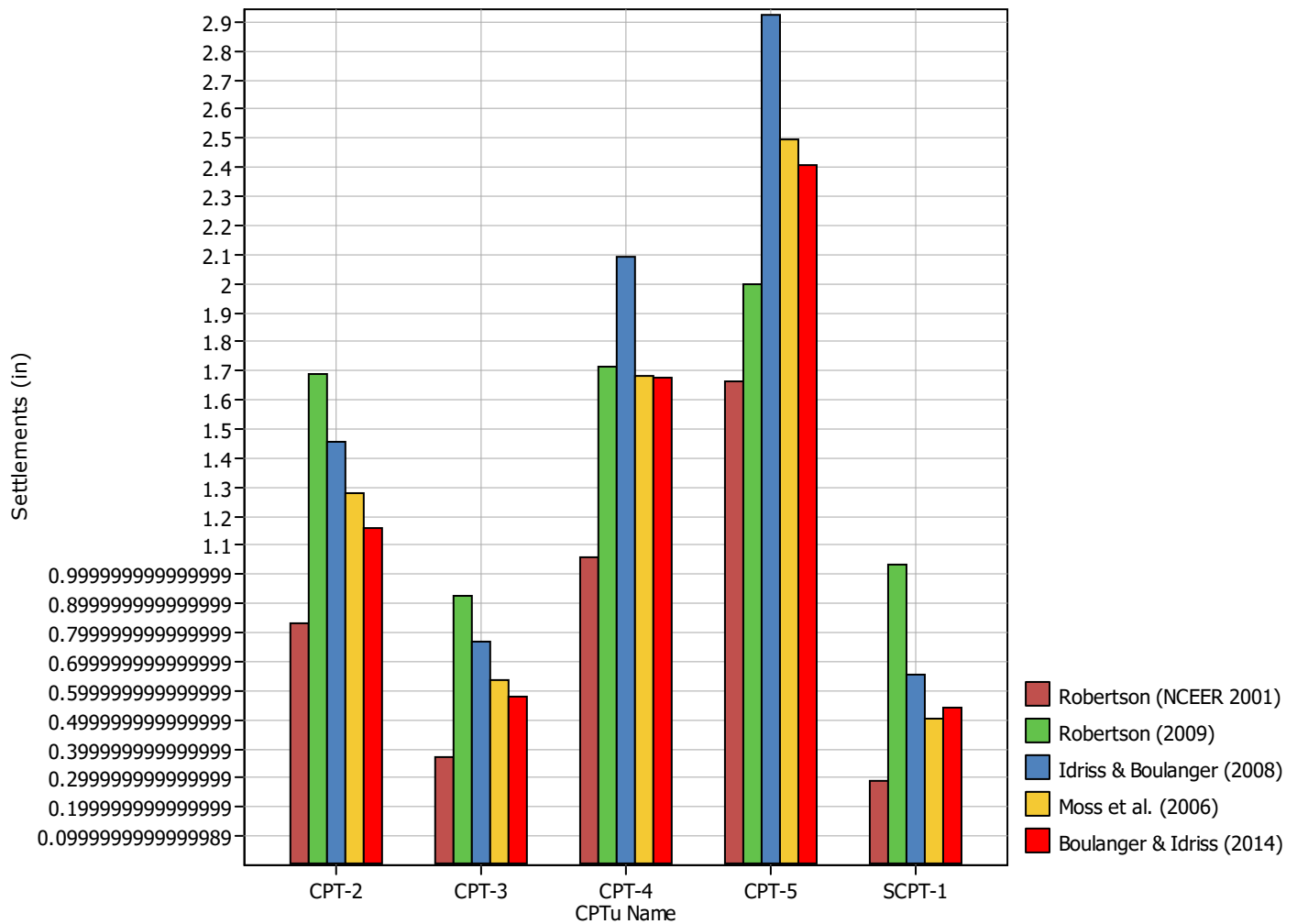
PGA Based Parametric Analysis



:: CPT main liquefaction parameters details ::

CPT Name	Assesment method	Earthquake Mag.	GWT in situ (ft)	GWT earthq. (ft)
CPT-2	NCEER (1998)	7.90	6.50	5.00
CPT-3	NCEER (1998)	7.90	6.50	5.00
CPT-4	NCEER (1998)	7.90	6.50	5.00
CPT-5	NCEER (1998)	7.90	6.50	5.00
SCPT-1	NCEER (1998)	7.90	6.50	5.00

Overall Parametric Assessment Method



:: CPT main liquefaction parameters details ::

CPT Name	Earthquake Mag.	Earthquake Accel.	GWT in situ (ft)	GWT earthq. (ft)
CPT-2	7.90	0.70	6.50	5.00
CPT-3	7.90	0.70	6.50	5.00
CPT-4	7.90	0.70	6.50	5.00
CPT-5	7.90	0.70	6.50	5.00
SCPT-1	7.90	0.70	6.50	5.00

Vs BASED LIQUEFACTION ANALYSIS REPORT (NCEER 1998)

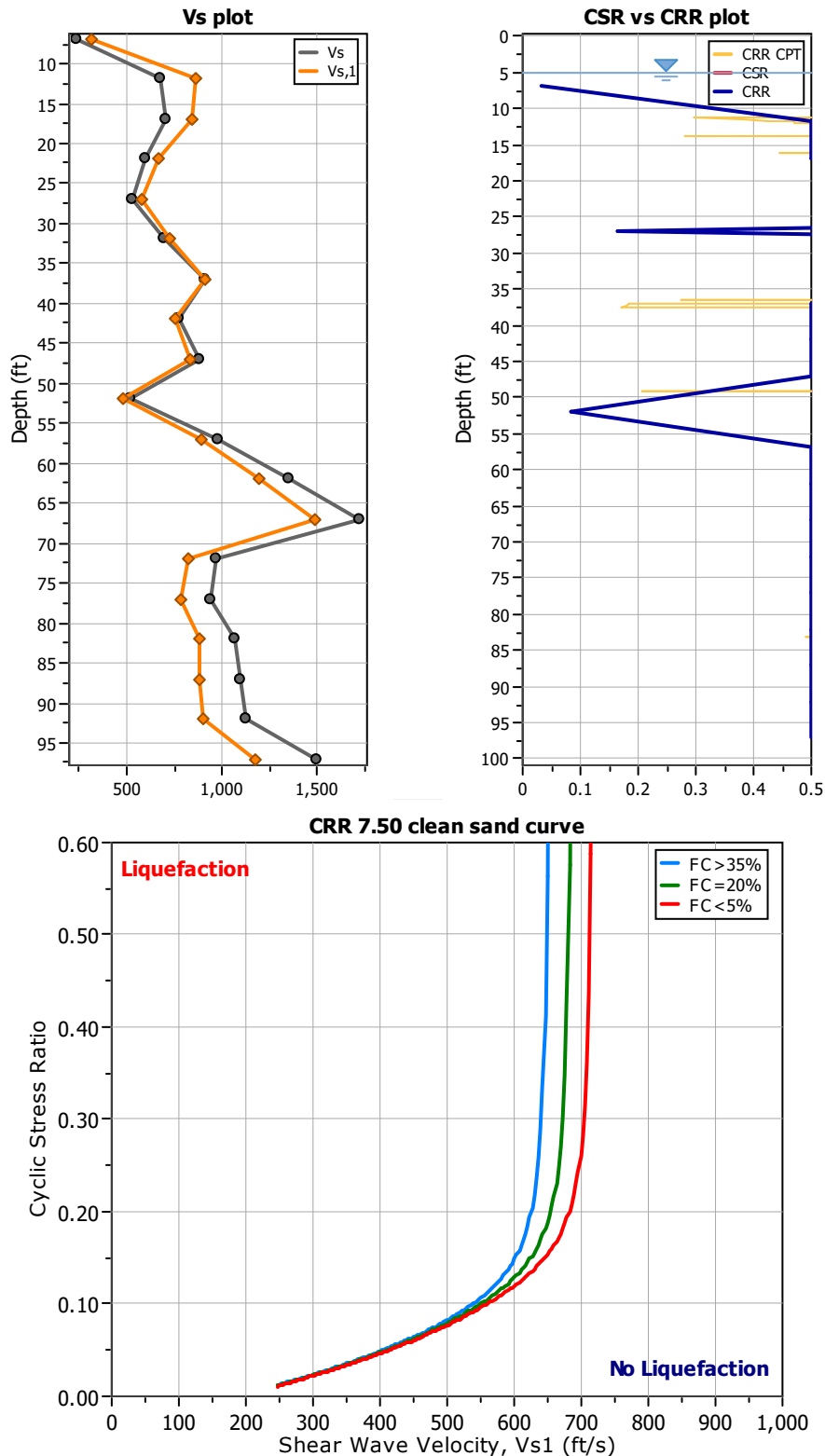
Project title : Holly Hotel Group

Location : 501 INDUSTRIAL ROAD, SAN CARLOS, CA

CPT file : SCPT-1

:: Input parameters and analysis properties ::

Calculation method: NCEER (1998)
G.W.T. (in-situ): 6.50 ft
G.W.T. (earthq.): 5.00 ft
Earthquake magnitude M_w : 7.90
Peak ground acceleration: 0.70g



:: Cyclic Stress Ratio fully adjusted (CSR*) numeric results ::

No	Depth (ft)	Weight (pcf)	u_0 (tsf)	Ext. Load (tsf)	σ_v (tsf)	σ'_v (tsf)	r_d	CSR	K_σ	MSF	CSR*	Can Liquefy
1	6.95	97.00	0.06	0.00	0.34	0.28	0.98	0.546	1.00	0.88	0.624	Yes
2	11.85	100.00	0.21	0.00	0.58	0.37	0.97	0.699	1.00	0.88	0.799	Yes
3	16.90	110.00	0.37	0.00	0.86	0.49	0.96	0.769	1.00	0.88	0.879	Yes
4	21.90	113.00	0.53	0.00	1.14	0.62	0.95	0.802	1.00	0.88	2.000	Yes
5	26.90	109.00	0.68	0.00	1.41	0.73	0.94	0.825	1.00	0.88	0.942	Yes
6	31.90	110.00	0.84	0.00	1.69	0.85	0.92	0.827	1.00	0.88	2.000	Yes
7	36.90	120.00	1.00	0.00	1.99	0.99	0.87	0.796	1.00	0.88	0.910	Yes
8	41.95	113.00	1.15	0.00	2.28	1.12	0.83	0.769	0.99	0.88	0.889	Yes
9	46.95	112.00	1.31	0.00	2.56	1.25	0.79	0.740	0.97	0.88	0.873	Yes
10	51.95	124.00	1.46	0.00	2.87	1.40	0.75	0.700	0.95	0.88	0.846	Yes
11	56.95	120.00	1.62	0.00	3.17	1.54	0.71	0.664	0.93	0.88	0.818	Yes
12	61.95	121.00	1.78	0.00	3.47	1.69	0.67	0.626	0.91	0.88	0.786	Yes
13	66.95	122.00	1.93	0.00	3.77	1.84	0.63	0.588	0.90	0.88	0.750	Yes
14	71.95	120.00	2.09	0.00	4.07	1.98	0.59	0.551	0.88	0.88	0.713	Yes
15	77.05	116.00	2.25	0.00	4.37	2.12	0.56	0.521	0.87	0.88	0.685	Yes
16	82.05	117.00	2.40	0.00	4.66	2.26	0.54	0.511	0.86	0.88	0.680	Yes
17	87.05	118.00	2.56	0.00	4.96	2.40	0.53	0.500	0.85	0.88	0.673	Yes
18	92.05	127.00	2.72	0.00	5.27	2.56	0.52	0.487	0.84	0.88	0.665	Yes
19	97.10	132.00	2.87	0.00	5.61	2.73	0.51	0.473	0.83	0.88	0.654	Yes

Abbreviations

Depth: Depth relative to final conditions free surface (ft) during eq.
 u_0 : Water pressure at test point (tsf) during eq.
 σ_v : Total overburden pressure at test point (tsf) during eq.
 σ'_v : Effective overburden pressure based on GWT during earthquake (tsf) during eq.
 r_d : Nonlinear shear mass factor
CSR: Cyclic Stress Ratio
MSF: Effective overburden stress factor
 K_σ : Magnitude Scaling Factor
CSR*: CSR fully adjusted

:: Cyclic Resistance Ratio (CRR) numeric results ::

No	Depth (ft)	V_s (ft/s)	Fines %	n	V_{s1} (ft/s)	V_{s1c} (ft/s)	CRR _{7.5}	F.S.	Can Liquefy
1	6.95	234.60	77.00	1.35	315.61	656.17	0.033	0.05	Yes
2	11.85	680.20	20.00	1.26	859.44	680.77	0.500	0.63	Yes
3	16.90	709.70	34.00	1.19	841.49	657.81	0.500	0.57	Yes
4	21.90	593.00	32.00	1.12	666.80	661.09	4.000	2.00	Yes
5	26.90	531.00	56.00	1.08	573.37	656.17	0.164	0.17	Yes
6	31.90	695.70	45.00	1.04	724.96	656.17	4.000	2.00	Yes
7	36.90	910.40	32.00	1.00	914.04	661.09	0.500	0.55	Yes
8	41.95	769.90	45.00	0.98	750.94	656.17	0.500	0.56	Yes
9	46.95	878.20	46.00	0.95	835.25	656.17	0.500	0.57	Yes
10	51.95	516.00	51.00	0.92	477.15	656.17	0.084	0.10	Yes
11	56.95	983.80	47.00	0.90	888.41	656.17	0.500	0.61	Yes
12	61.95	1,355.60	60.00	0.88	1197.50	656.17	0.500	0.64	Yes
13	66.95	1,726.20	43.00	0.87	1493.84	656.17	0.500	0.67	Yes
14	71.95	968.40	50.00	0.85	822.77	656.17	0.500	0.70	Yes
15	77.05	941.50	57.00	0.84	787.00	656.17	0.500	0.73	Yes
16	82.05	1,069.60	54.00	0.82	880.53	656.17	0.500	0.74	Yes
17	87.05	1,093.40	54.00	0.81	887.03	656.17	0.500	0.74	Yes
18	92.05	1,131.10	54.00	0.80	903.05	656.17	0.500	0.75	Yes

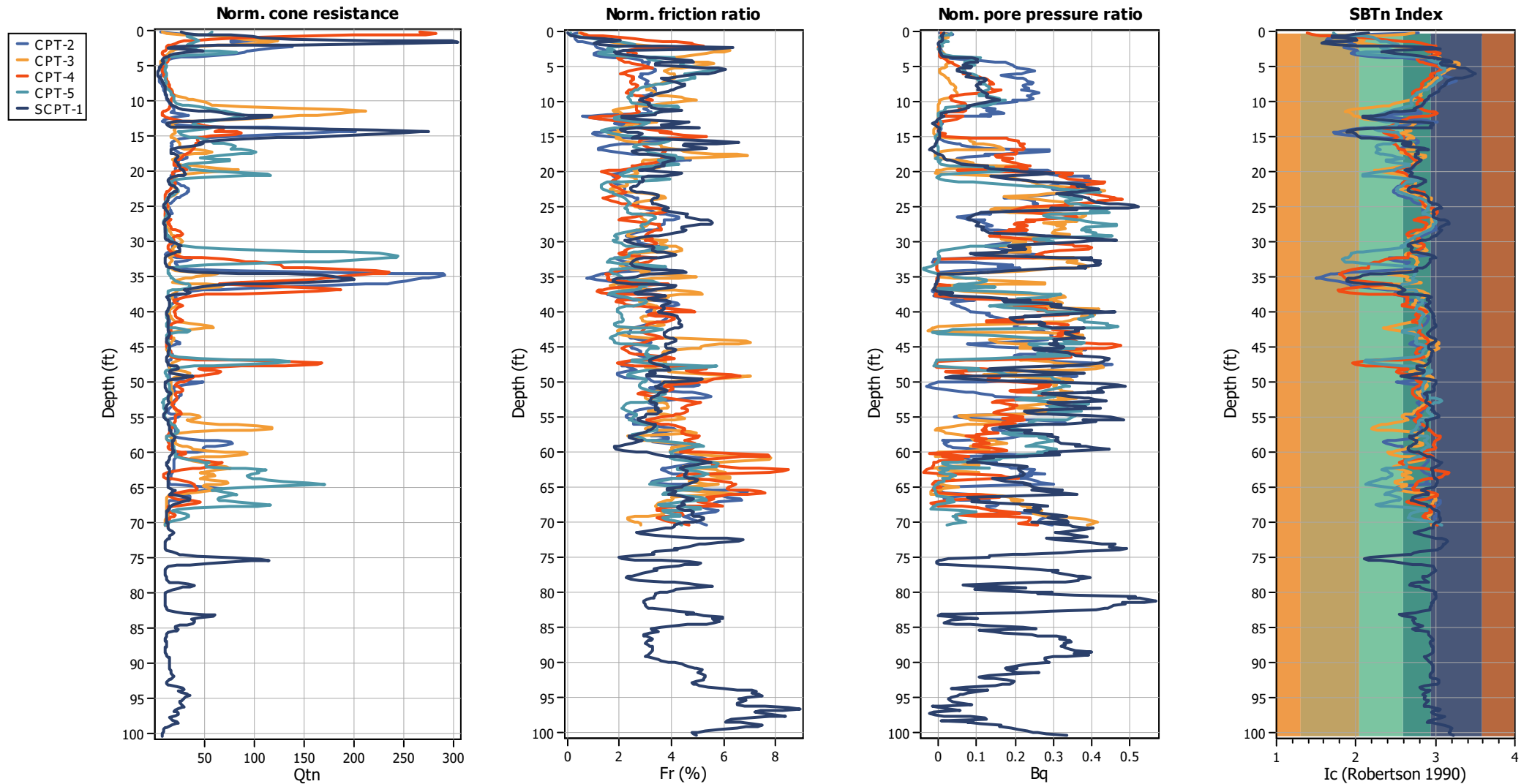
:: Cyclic Resistance Ratio (CRR) numeric results ::									
No	Depth (ft)	V _s (ft/s)	Fines %	n	V _{s1} (ft/s)	V _{s1c} (ft/s)	CRR _{7.5}	F.S.	Can Liquefy
19	97.10	1,499.60	55.00	0.79	1177.87	656.17	0.500	0.76	Yes

Abbreviations

Depth: Depth from free surface where Vs was performed (ft)
 V_s: Estimated Vs (ft/s)
 n: Stress exponent normalization factor
 V_{s1}: Normalized Vs (ft/s)
 V_{s1c}: Critical value of Vs1, which separates contractive and dilative behavior (tsf)
 CRR_{7.5}: Cyclic Resistance Ratio for M_w 7.50
 F.S.: Factor of safety against liquefaction

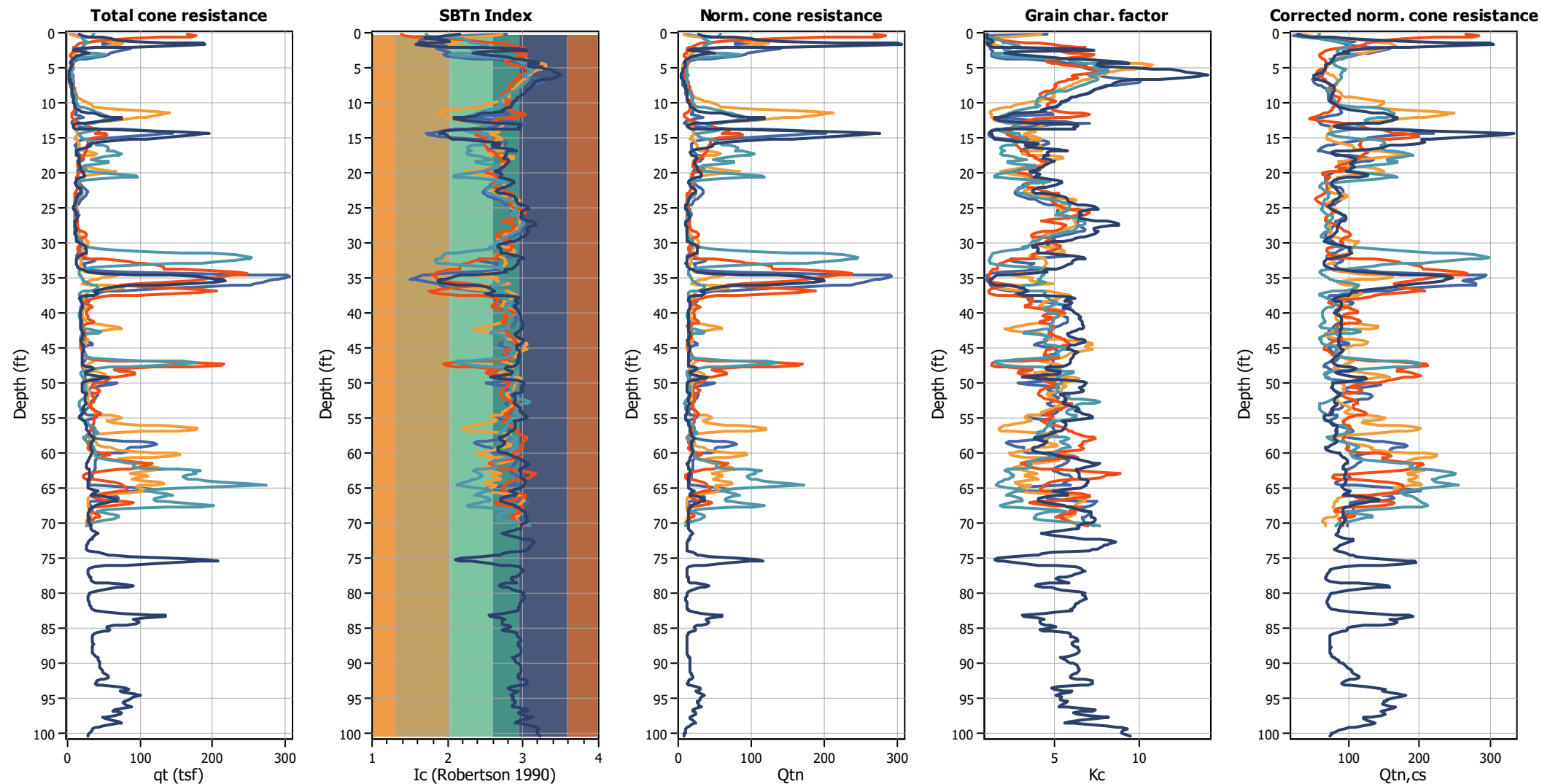
Project: Holly Hotel Group

Overlay Normalized Plots



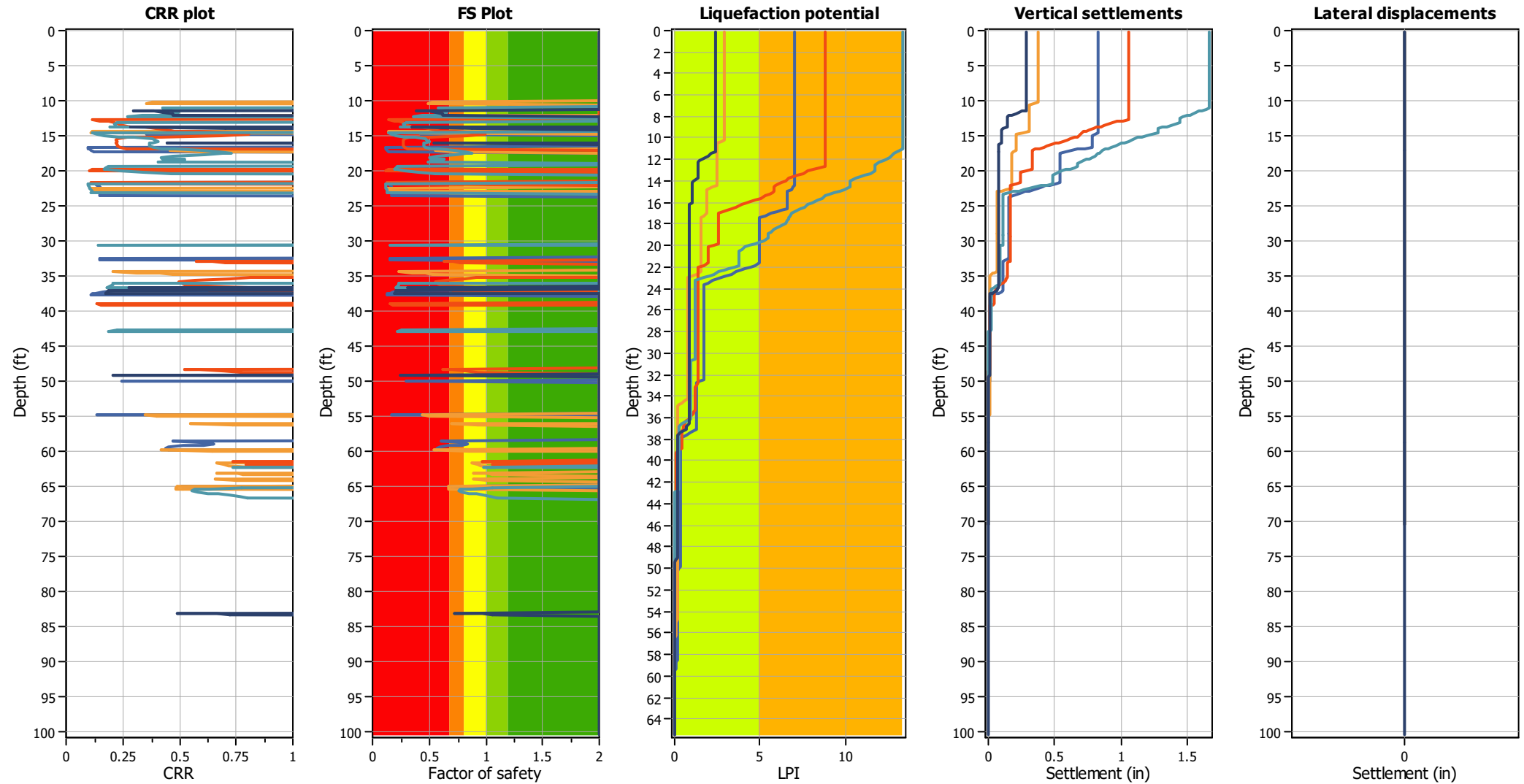
Project: Holly Hotel Group

Overlay Intermediate Results



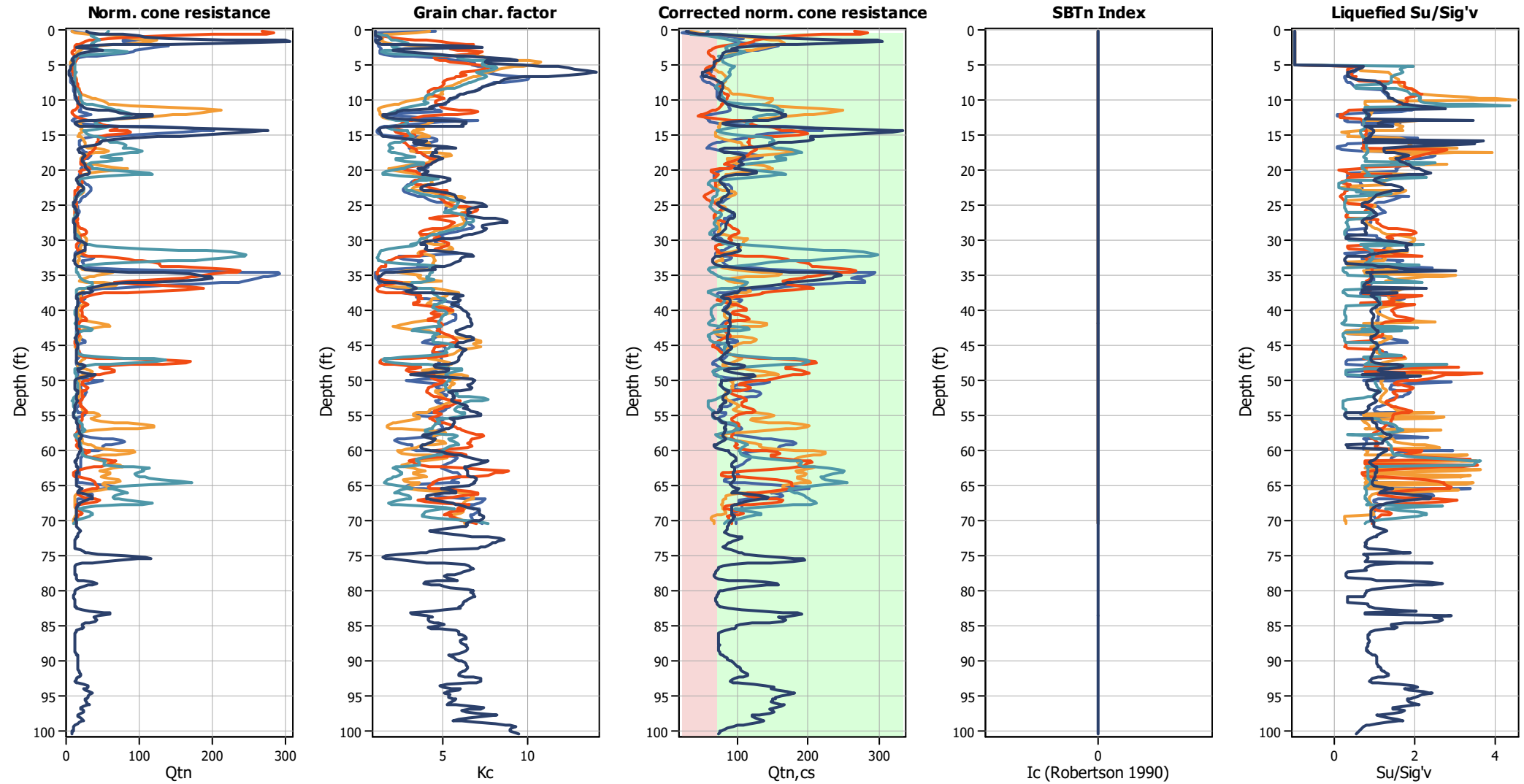
Project: Holly Hotel Group

Overlay Cyclic Liquefaction Plots



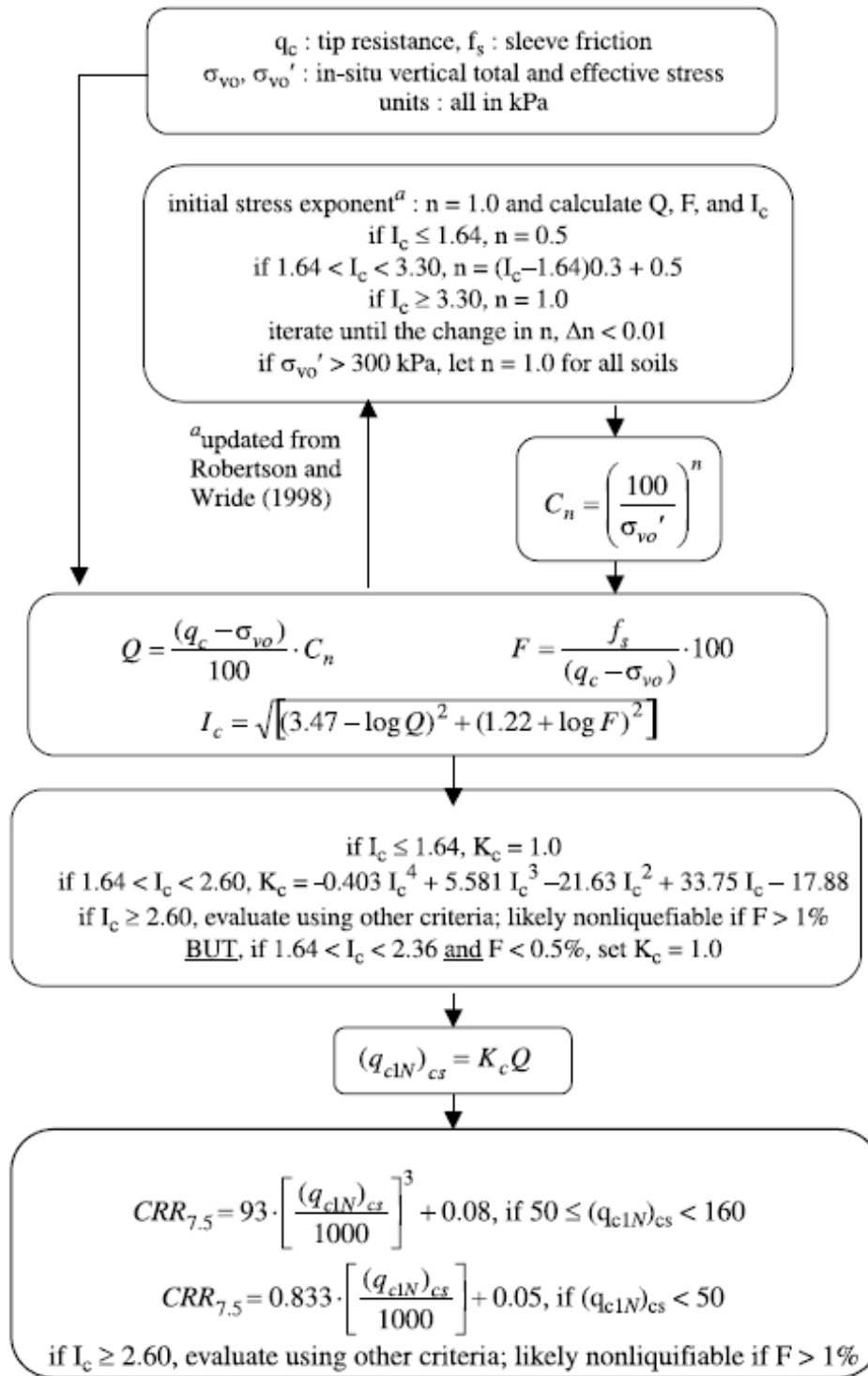
Project: Holly Hotel Group

Overlay Strength Loss Plots



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

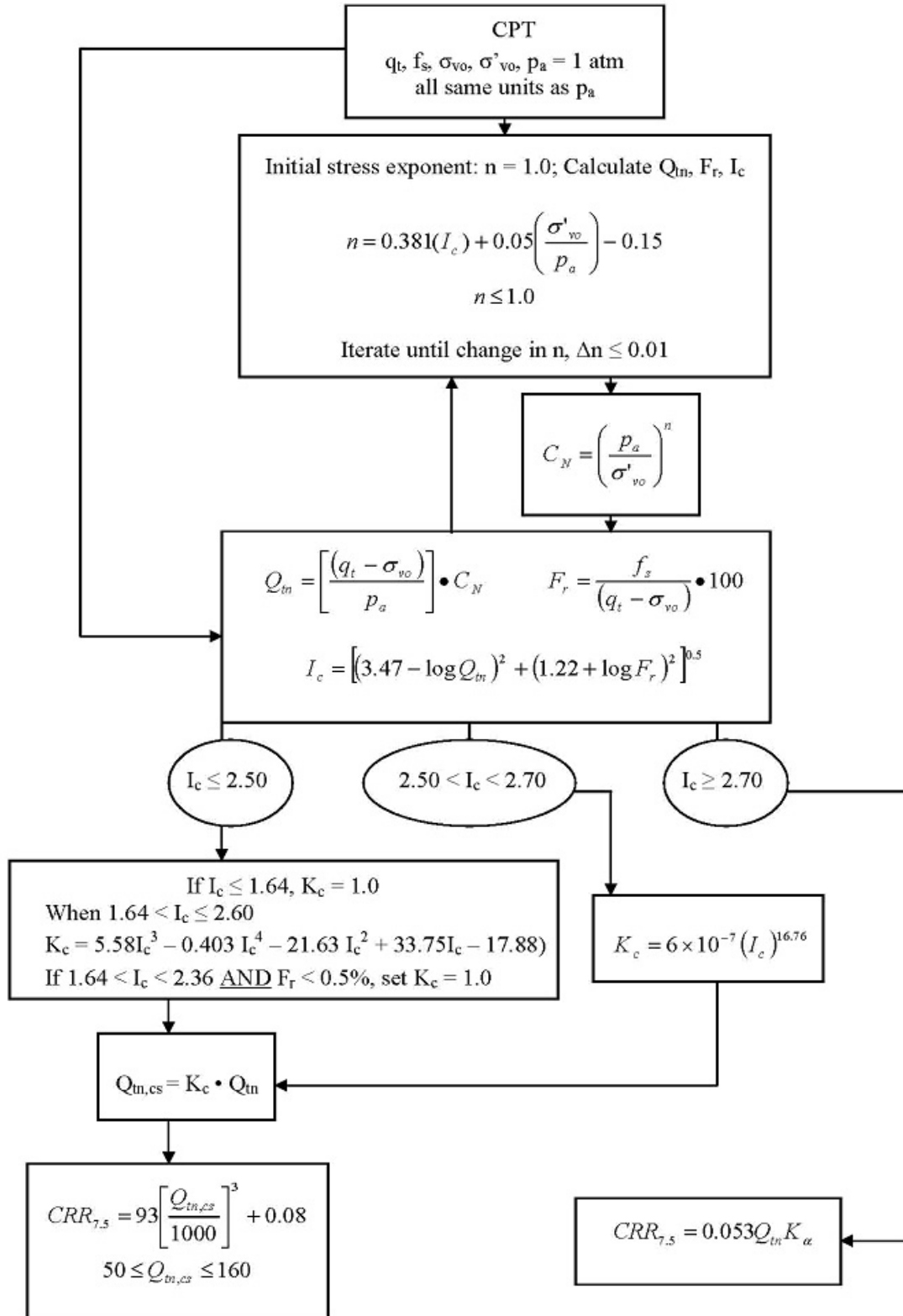
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart¹:



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

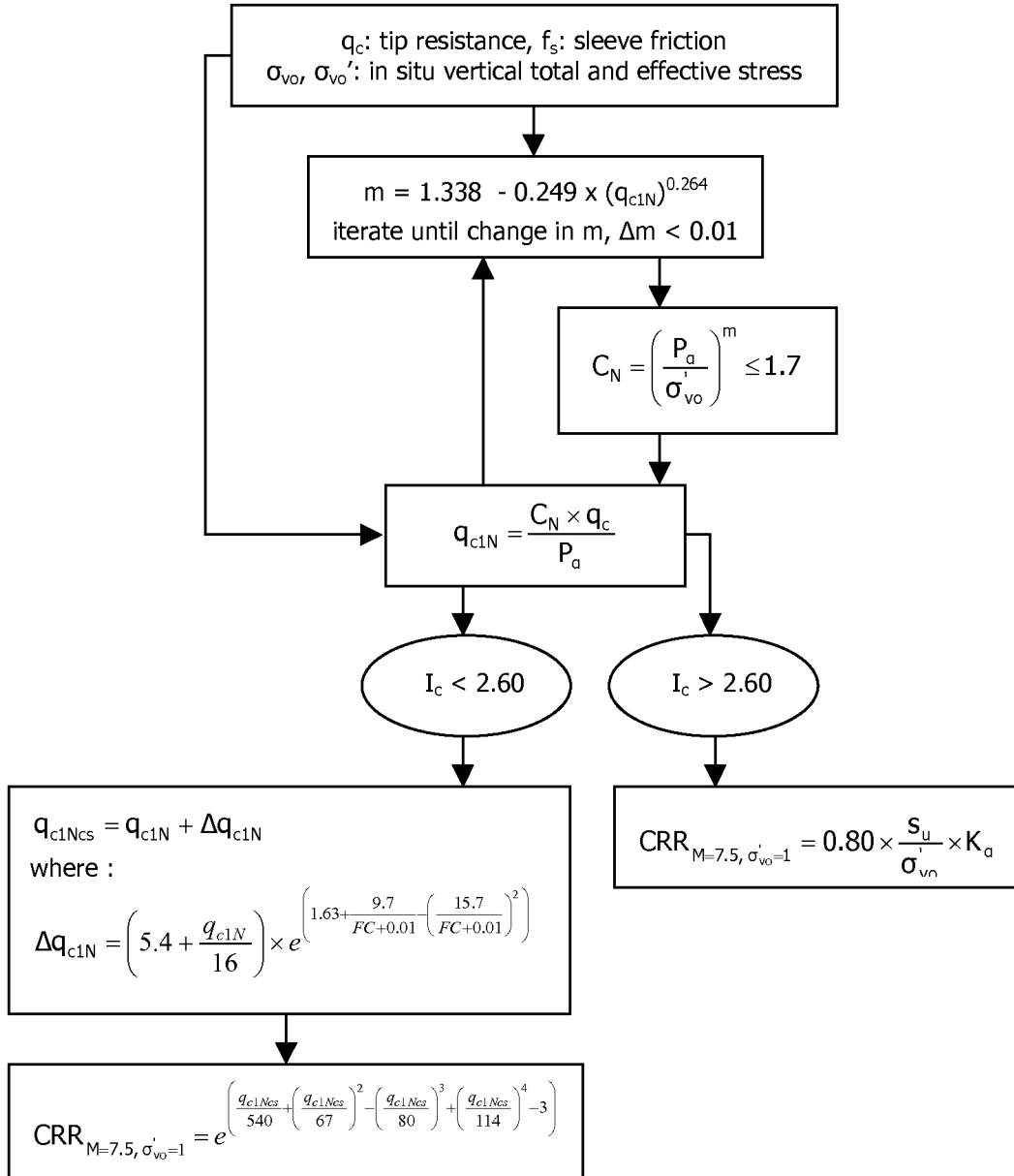
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart¹:

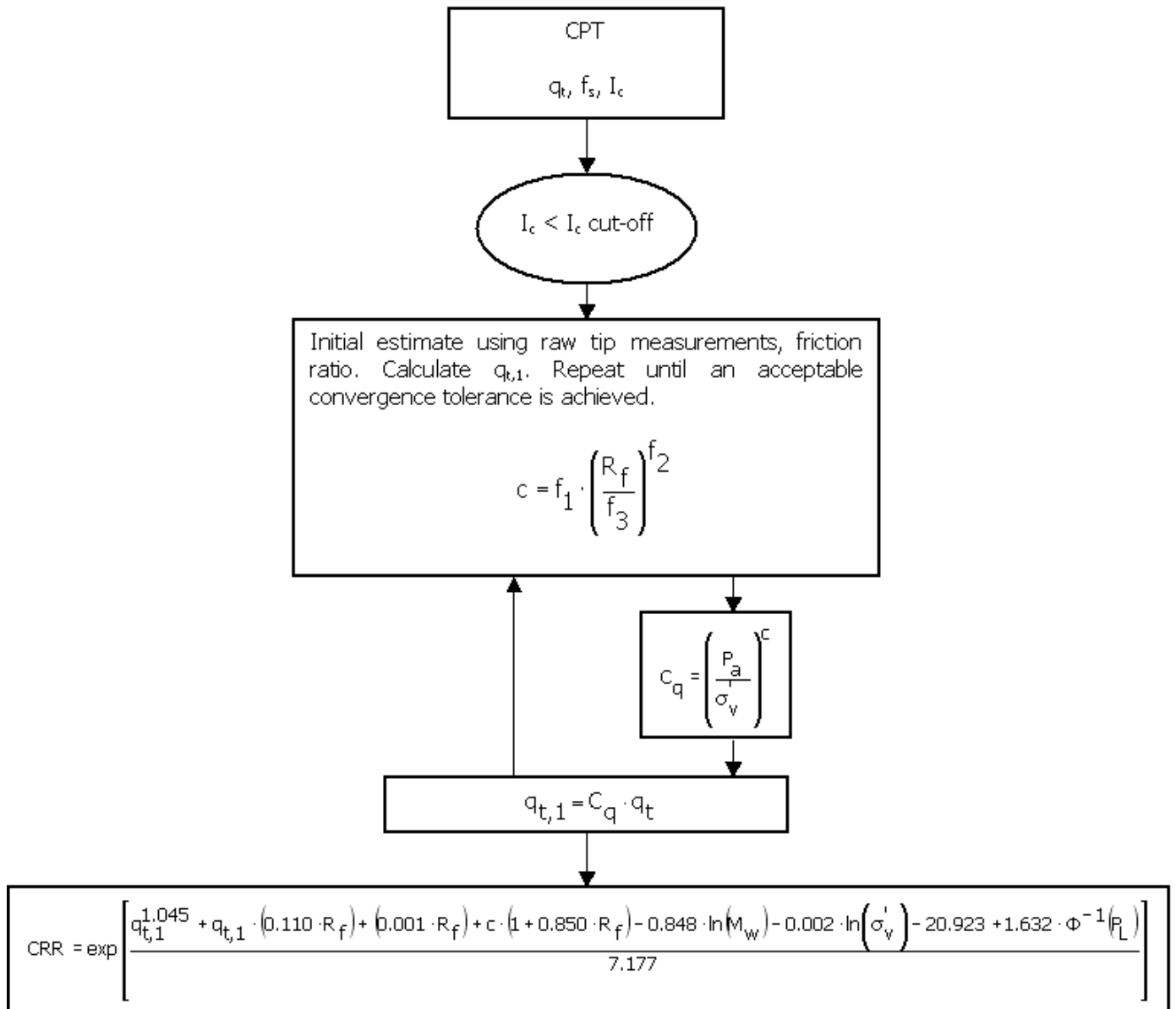


¹ P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

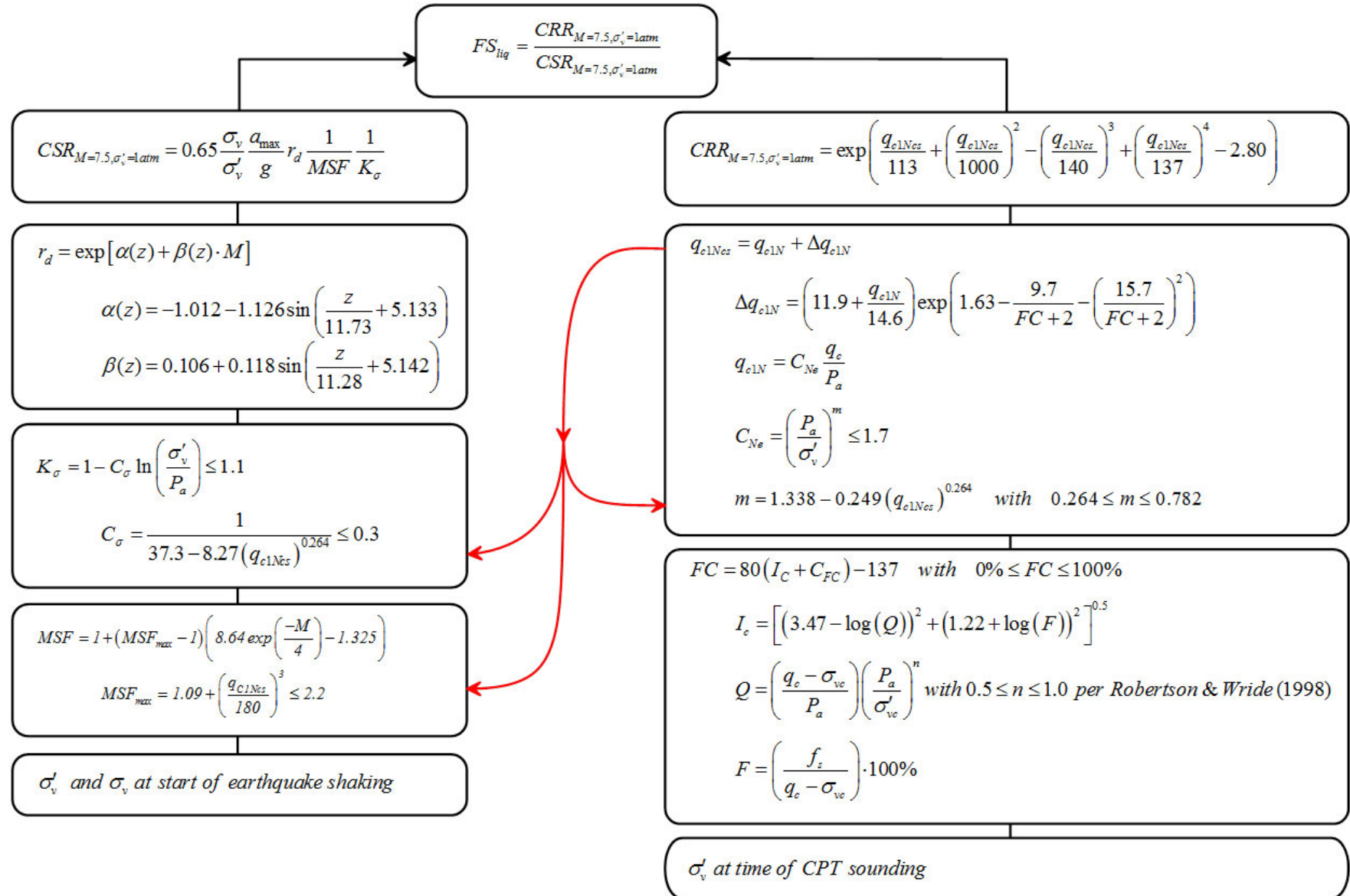
Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



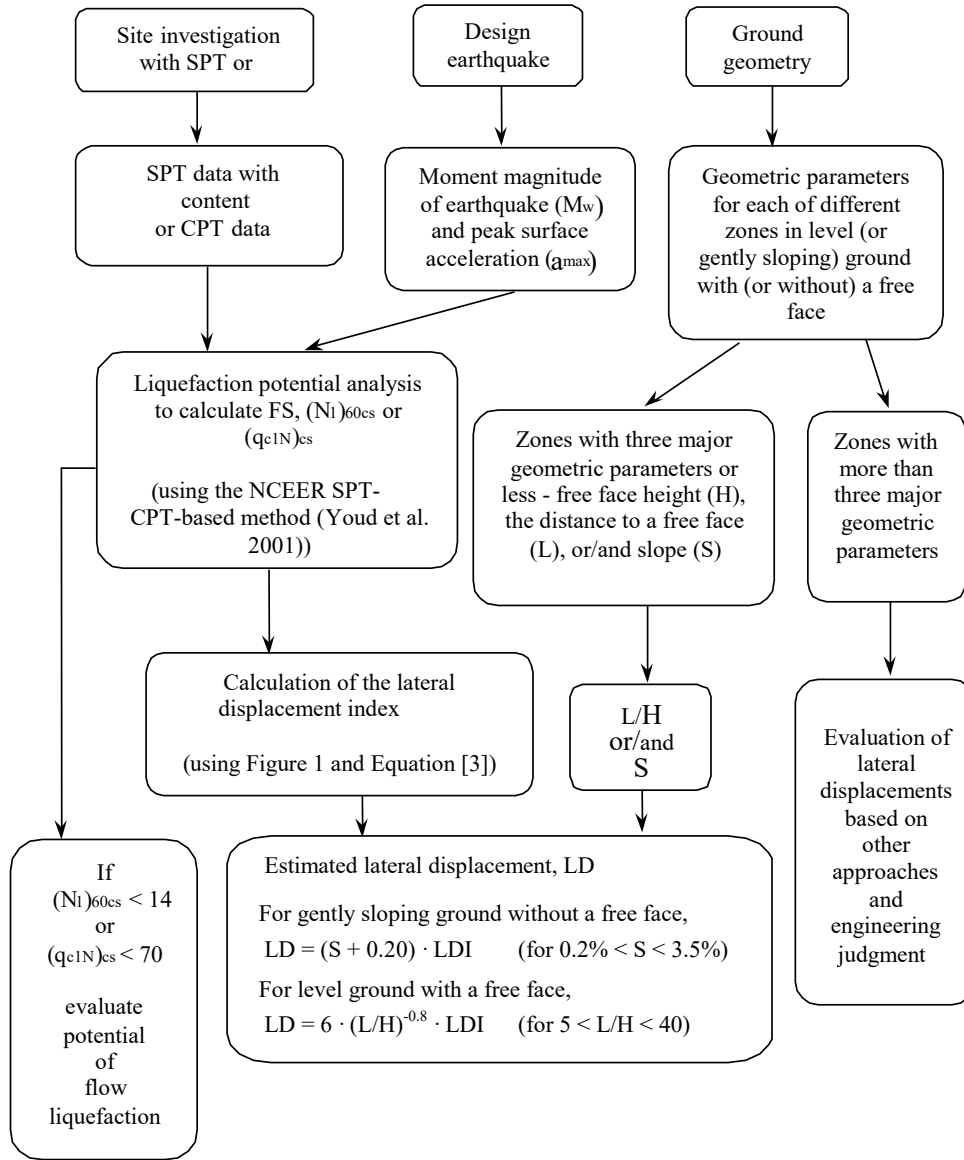
Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)



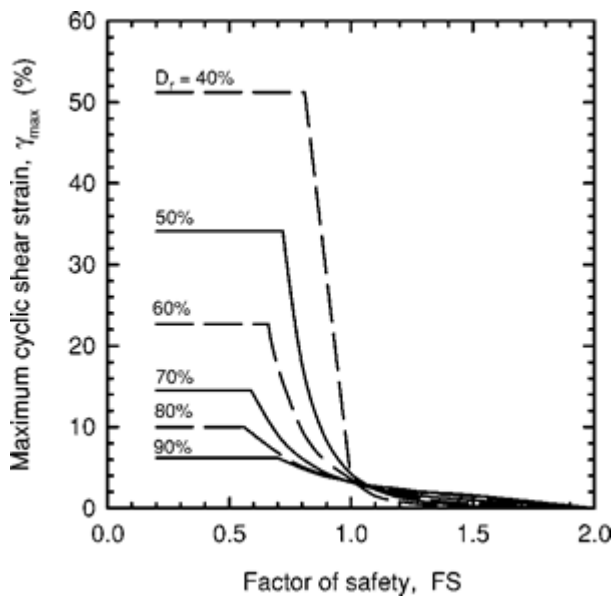
Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)



Procedure for the evaluation of liquefaction-induced lateral spreading displacements



¹ Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



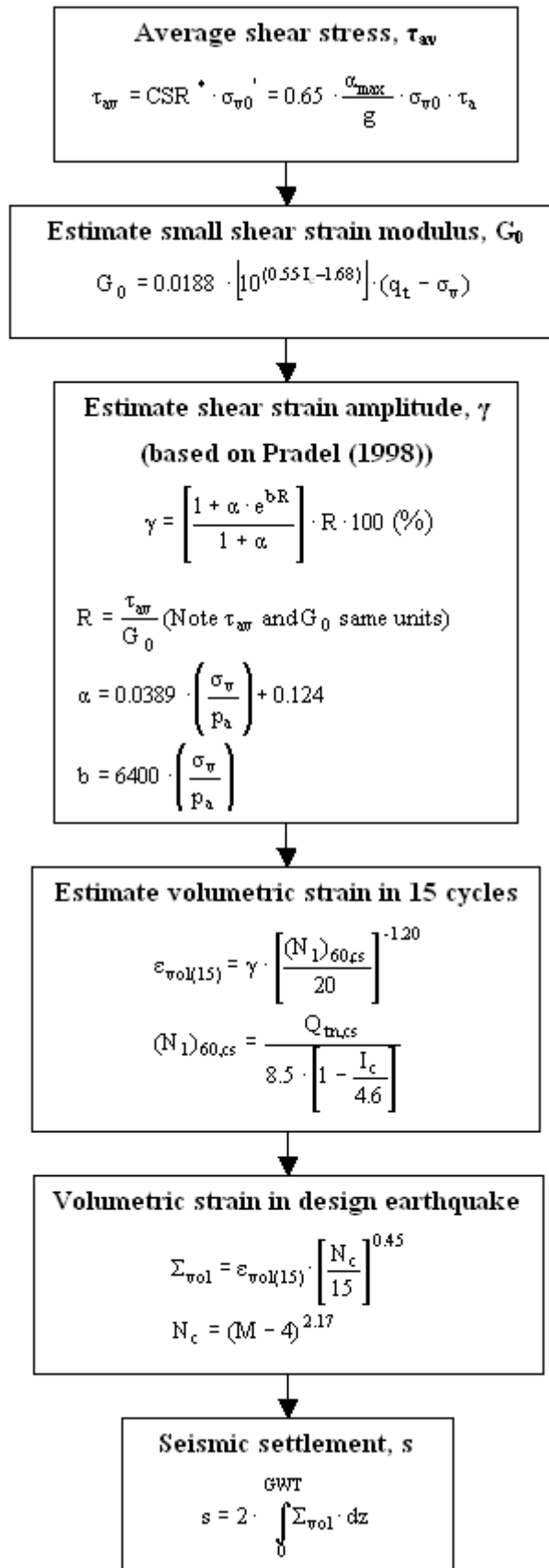
¹ Figure 1

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

¹ Equation [3]

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

Procedure for the estimation of seismic induced settlements in dry sands



Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methodology developed by Iwasaki (1982) and is adopted by AFPS.

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

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

where:

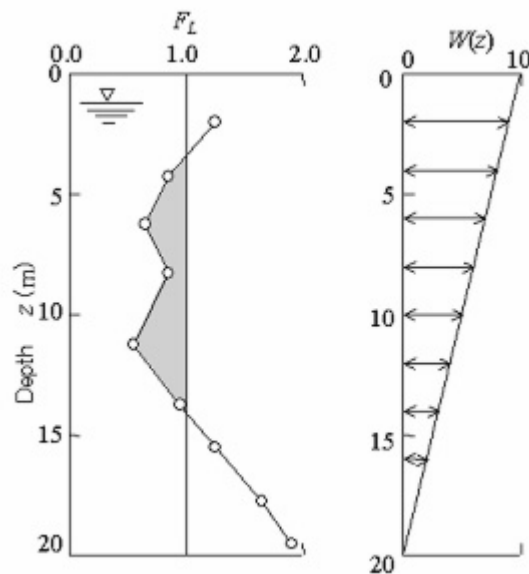
$F_L = 1 - F.S.$ when F.S. less than 1

$F_L = 0$ when F.S. greater than 1

z depth of measurement in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- $LPI = 0$: Liquefaction risk is very low
- $0 < LPI \leq 5$: Liquefaction risk is low
- $5 < LPI \leq 15$: Liquefaction risk is high
- $LPI > 15$: Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

Shear-Induced Building Settlement (Ds) calculation procedure

The shear-induced building settlement (Ds) due to liquefaction below the building can be estimated using the relationship developed by Bray and Macedo (2017):

$$\begin{aligned} \ln(D_s) = & c_1 + c_2 * LBS + 0.58 * \ln\left(\tanh\left(\frac{HL}{6}\right)\right) + \\ & 4.59 * \ln(Q) - 0.42 * \ln(Q)^2 - 0.02 * B + \\ & 0.84 * \ln(CAVdp) + 0.41 * \ln(Sa1) + \varepsilon \end{aligned}$$

where Ds is in the units of mm, c1= -8.35 and c2= 0.072 for LBS ≤ 16, and c1= -7.48 and c2= 0.014 otherwise. Q is the building contact pressure in units of kPa, HL is the cumulative thickness of the liquefiable layers in the units of m, B is the building width in the units of m, CAVdp is a standardized version of the cumulative absolute velocity in the units of g-s, Sa1 is 5%-damped pseudo-acceleration response spectral value at a period of 1 s in the units of g, and ε is a normal random variable with zero mean and 0.50 standard deviation in Ln units. The liquefaction-induced building settlement index (LBS) is:

$$LBS = \sum W * \frac{\varepsilon_{shear}}{z} dz$$

where z (m) is the depth measured from the ground surface > 0, W is a foundation-weighting factor wherein W = 0.0 for z less than Df, which is the embedment depth of the foundation, and W = 1.0 otherwise. The shear strain parameter (ε_{shear}) is the liquefaction-induced free-field shear strain (in %) estimated using Zhang et al. (2004). It is calculated based on the estimated Dr of the liquefied soil layer and the calculated safety factor against liquefaction triggering (FSL).

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