Appendix F Geotechnical Engineering Report

Appendix

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NTS GEOTECHNICAL

October 2, 2020
(Revised July 29, 2021)

Project No. 20073

Mr. Rob Bak
Core Spaces
1643 N Milwaukee Ave, 5th Floor
Chicago, IL 60647

Subject: Preliminary Geotechnical Engineering Report
The Hub at Fullerton
2601 to 2751 Chapman Avenue, Fullerton, California

Dear Mr. Bak:

In accordance with your request and authorization, we are presenting the results of our geotechnical investigation for the proposed The Hub at Fullerton project located at 2601 to 2751 Chapman Avenue, in the City of Fullerton, California. The purpose of this investigation has been to evaluate the subsurface conditions at the site and to provide geotechnical engineering recommendations for the proposed construction.

Based on our findings, the proposed project is geotechnically feasible, provided that the recommendations in this report are incorporated into the design and are implemented during construction of the project. This report was prepared in accordance with the requirements of the 2019 California Building Code and the City of Fullerton requirements.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned at (657) 888-4608 or info@ntsgeo.com.

Respectfully submitted,
NTS GEOTECHNICAL, INC.

Nadim Sunna, M.Sc., Q.S.P, P.E., G.E 3172
Principal Engineer

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NTS GEOTECHNICAL

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Attachment(s):

INTRODUCTION

This report presents the results of our geotechnical engineering evaluation performed for the proposed The Hub at Fullerton project located at 2601 Chapman Avenue, in the City of Fullerton, California. See (Plate 1, Location Map). The purpose of this study has been to evaluate the subsurface conditions at the site and to provide geotechnical recommendations related to the design and construction of the proposed structure.

SITE AND PROJECT DESCRIPTION

The project site is located at 2601 Chapman Avenue in the City of Fullerton, California, and it is bound by an existing apartment complex on the north, existing commercial property on the east, Commonwealth Avenue on the west, and Chapman Avenue on the south. The property currently consists of existing two-story office buildings, asphalt-concrete parking lot, planters and trees, and existing flatwork.

It is our understanding that the proposed project consists of the development of a 6-story residential homes and 5-story parking structure. Based on our review of preliminary conceptual design plans, we understand that the structures are planned to be constructed at-grade.

Based on our correspondence with DCI Engineers, the project structural engineers, we understand that the buildings foundations may experience the following preliminary structural loads:

Preliminary Structural Loads

Maximum Column Loads	Dead: 282 kips Live: 89 kips

We have performed our settlement analysis utilizing these preliminary loads. If the actual loads are greater than what was assumed herein, this office should be contacted for additional evaluation.

SCOPE OF WORK

As part of the preparation of this report, we have performed the following tasks:

Background Review

We reviewed readily available background data including in-house geophysical data, geologic maps, topographic maps, and aerial photographs relevant to the subject site in preparation of this report.

Field Exploration

The subsurface conditions were evaluated on April 2, 3 and August 25, 2020 by advancing nine (9) eight-inch diameter, hollow-stem-auger borings and five (5) Cone Penetration Testing (CPT) soundings at various locations across the subject site. The borings were advanced to depths ranging from 5 and 61.5 feet below the existing grade. The CPTs were pushed a maximum depth of 50 feet below the existing grade. The approximate locations of the borings are shown on Figure 2, Geotechnical Map. Detailed exploration information of soils borings is presented in Appendix A, Field Exploration.

Geotechnical Laboratory Testing

Laboratory tests were performed on selected samples obtained from the boring in order to aid in the soil classification and to evaluate the engineering properties of the foundation soils. NTS Geotechnical, Inc. has reviewed the laboratory test results performed by Hushmand and Associates, Inc. and accepts the results for use in our analysis. The following tests were performed in general accordance with ASTM standards:

- In-situ moisture and density;
- #200 sieve wash;
- Direct shear;
- Consolidation;
- Corrosion; and
- R-Value.

A summary of the laboratory test results are presented in Appendix B of this report.

GEOLOGIC FINDINGS

Regional Geologic Setting

According to the Quaternary Geologic Map of the Anaheim and Newport Beach 7.5-Minute Quadrangle, the project site is underlain by younger alluvial fan deposits (Qyf) that are typically comprised of sands, clays, silts and gravel.

Subsurface Materials

Earth materials encountered during our subsurface investigation consisted of approximately 2 to 5 feet of artificial fill (Af) overlaying the young alluvial fan deposits (Qyf) extending to the total depth of exploration. In general, the artificial fill consists of slightly moist, loose to medium dense, silty sand and clayey sands.

The alluvial fan deposits (Qyf) consisted of moist to very moist, very loose to medium dense to dense clayey sand and sands, and, firm to very stiff, clays and silts. The upper approximately 14 feet of the site soils consist of very loose to loose sandy soils that are collapsible and compressible.

Groundwater

Groundwater was not observed during our exploration to a maximum depth of 61.5 feet below the existing grade. The historical high depth to groundwater is reportedly deeper than 70 feet below the existing grade at the project site (CDMG 1997). Groundwater conditions may vary across the site due to stratigraphic and hydrologic conditions, and may change over time as a consequence of seasonal and meteorological fluctuations, or activities by humans at this site and nearby sites. However, based on the above findings, groundwater is unlikely to impact the proposed development.

GEOLOGIC HAZARDS

Faulting and Seismicity

The site is not located within an Alquist-Priolo Earthquake Fault Zone, and no known active faults are shown on the reviewed geologic maps crossing the site, however, the site is located in the seismically active region of Southern California. The nearest known active faults are the Puente Hills and Elsinore fault systems, which are located approximately 0.9 and 4.1 miles from the site, respectively.

Given the proximity of the site to these and numerous other active and potentially active faults, the site will likely be subject to earthquake ground motions in the future. A site PGA_M of 0.78g was calculated for the site in conformance with the 2019 CBC. This PGA_M is primarily dominated by earthquakes with a mean magnitude of 6.7 at a mean distance of 7 miles from the site using the USGS 2014 Interactive Deaggregation website.

Liquefaction and Seismic Settlement

Liquefaction occurs when the pore pressures generated within a soil mass approach the effective overburden pressure. Liquefaction of soils may be caused by cyclic loading such as that imposed by ground shaking during earthquakes. The increase in pore pressure results in a loss of strength, and the soil then can undergo both horizontal and vertical movements, depending on the site conditions. Other phenomena associated with soil liquefaction include sand boils, ground oscillation, and loss of foundation bearing capacity. Liquefaction is generally known to occur in loose, saturated, relatively clean, fine-grained cohesionless soils at depths shallower than approximately 50 feet. Factors to consider in the evaluation of soil liquefaction potential include groundwater conditions, soil type, grain size distribution, relative density, degree of saturation, and both the intensity and duration of ground motion.

Based on our review of the State of California Official Map of Seismic Hazard Zones for the Anaheim and Newport Beach Quadrangle (California Department of Conservation, Division of Mines and Geology, 1997), the site is not located within a zone of required investigation for Liquefaction. Based on the lack of shallow groundwater, the presence of extensive amount of fine-grained soil, the relatively uniform soil stratum across the site, and our liquefaction analysis as presented in Appendix C of this report, it is our professional opinion that the liquefaction potential at the site is very low.

Seismically-induced dry sand settlement is the ground settlement due to densification of loose, dry cohesionless soils during strong earthquake shaking. Based on our liquefaction analysis, we estimate that seismic settlement on the order 2 inches with a differential of 1 inch over a span of 40 feet may occur during seismic shaking.

Landslides

Based on our review of the referenced geologic maps, literature, topographic maps, aerial photographs, and our subsurface evaluation, no landslides or related features underlie or are adjacent to the subject site. Due to the relatively level nature of the site and surrounding areas, the potential for landslides at the project site is considered negligible.

Flooding

The Federal Emergency Management Agency (FEMA) has prepared flood insurance rate maps (FIRMs) for use in administering the National Flood Insurance Program. Based on our review of the FEMA flood map, the site is located in an Area of Minimal Flood Hazard (Zone X). The potential for flooding to impact the proposed development is considered low.

Tsunami and Seiches

Tsunamis are waves generated by massive landslides near or under sea water. The site is not located on any State of California – County of Orange Tsunami Inundation Map for Emergency Planning. The potential for the site to be adversely impacted by earthquake-induced tsunamis is considered to be negligible because the site is located several miles inland from the Pacific Ocean shore, at an elevation exceeding the maximum height of potential tsunami inundation.

Seiches are standing wave oscillations of an enclosed water body after the original driving force has dissipated. The potential for the site to be adversely impacted by earthquake-induced seiches is considered to be negligible due to the lack of any significant enclosed bodies of water located in the vicinity of the site.

GEOTECHNICAL ENGINEERING FINDINGS

Expansive Soil

Based on our evaluation and experience with similar material types, and laboratory testing, the soils encountered near the ground surface at the site exhibit a very low to low expansion potential, however, the clay soils encountered at the bottom of the basement level is anticipated to exhibit a medium expansion potential.

Corrosive Soil

Based on laboratory test results performed for pH, soluble chlorides, sulfate, and minimum resistivity, the on-site soils should be considered to have the following:

- A negligible sulfate exposure to concrete per ACI 318-14, Table 19.3.1.1
- A high minimum resistivity indicating conditions that are mildly corrosive to ferrous metals.
- A low chloride content (potentially corrosive).

Metal structures which will be in direct contact with the soil (i.e., underground metal conduits, pipelines, metal sign posts, etc.) and/or in close proximity to the soil (wrought iron fencing, etc.) may be subject to corrosion. The use of special coatings or cathodic protection around buried metal structures has been shown to be beneficial in reducing corrosion potential. Corrosion of ferrous metal reinforcing elements in structural concrete should be reduced by increasing the thickness of concrete cover and the use of the recommended maximum water/cement ratio for concrete.

The laboratory testing program does not address the potential for corrosion to copper piping. In this regard, a corrosion engineer should be consulted to perform more detailed testing and develop appropriate mitigation measures (if necessary). The above discussion is provided for general guidance in regards to the corrosiveness of the on-site soils to typical metal structures used for construction. Detailed corrosion testing and recommendations for protecting buried ferrous metal and/or copper elements are beyond our purview. If detailed recommendations are required, a corrosion engineer should be consulted to develop appropriate mitigation measures.

Preliminary Infiltration Testing

Two (2) preliminary infiltration tests were performed in general conformance with the County of Orange Technical Guidance Document (TGD). The borings are shown on the attached Plate 2 – Geotechnical Map, were excavated to depths of from approximately 10 feet below the existing grade using a hollow-stem-auger drill rig. The calculated unfactored raw observed infiltration rates are presented in the following table:

Unfactored Raw Infiltration Rates Summary

Boring No.	Depth Below Finish Grade (feet)	Unfactored Raw Observed Infiltration Rates (inches/hour) *
P-1	10.0	0.12
P-2	10.0	0.19

**Rates do not incorporate a factor of safety.*

The results of the infiltration testing indicate that the unfactored raw observed infiltration rates within the southern side of the development range from 0.12 to 0.19 inches per hour, with an average unfactored infiltration of 0.16 inches per hour. Thus, we conclude for the entire site that infiltration rates do not meet the minimum requirement of 0.3 inch/hour when a minimum factor of safety of 2 is applied per the County of Orange TGD manual. The results of the infiltration testing are contained in Appendix D of this report.

Excavation Characteristics

The majority of the soil materials underlying the site can be excavated with excavators and other conventional grading equipment.

GEOTECHNICAL ENGINEERING CONLUSIONS AND RECOMMENDATIONS

Conclusions

Based on the results of our field exploration and engineering analyses, it is our opinion that the proposed development is feasible from a geotechnical standpoint, provided that the recommendations in this report are incorporated into the design plans and are implemented during construction.

Based on the geologic and geotechnical findings, the following is a summary of our conclusions:

- The proposed structures may be supported on one of the following:
  - Shallow spread footings underlain by 12 feet of engineered fill.
  - Shallow spread footings supported by Geopier or equivalent gravel piers.
  - A mat foundation system underlain by engineered fill.

- Groundwater is not anticipated to directly impact the planned precise grading or during the installation of shallow underground utilities.
- There are no known active faults crossing the subject site. The site seismicity is typical for the Fullerton area. Structure design should be in accordance with the current 2019 CBC.
- The magnitude of total seismic settlement beneath the structure that is supported by spread footing is on the order of 2.0 inches with differential settlement of approximately 1 inch over a span of 40 feet.
- The magnitude of total seismic settlement beneath the structure that is supported by a mat foundation is on the order of 2.5 inches with differential settlement of approximately 1.5 inches over a span of 40 feet.
- The magnitude of total static settlements beneath the structure is expected to be less than 1.5 inches for a mat foundation or 1 inch for spread footings supported on engineered fill or rammed aggregate piers.
- The on-site soils are mildly corrosive to ferrous metals and have a negligible sulfate exposure to concrete (i.e., as defined by the CBC) and reinforcement.
- Based on preliminary infiltration testing and calculated infiltration rates, infiltration of storm water into the site soils is deemed not feasible.

Our geotechnical engineering analyses performed for this report were based on the earth materials encountered during the subsurface exploration for the site. If the design substantially changes, then our geotechnical engineering recommendations would be subject to revision based on our evaluation of the changes. The following sections present our conclusions and recommendations pertaining to the engineering design for this project.

Site Preparation

Site preparation should begin with the removal of utility lines, asphalt, concrete, vegetation, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside edges of the proposed excavation and fill areas. We recommend that unsuitable materials such as organic matter or oversized material be selectively removed and disposed offsite. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed at a legal dump site away from the project area.

Corrective Grading

Corrective grading will serve to create a firm and workable platform for construction of the proposed development, and exterior improvements. Due to the presence of compressible/collapsible soil, we recommend corrective grading be performed in order to densify the site soils within the building pads and site improvements. The depth of corrective grading based on each type of foundation system and site improvements are provided below.

It should be noted that the recommendations provided herein are based on our subsurface exploration and knowledge of the on-site geology. Actual removals may vary in configuration and volume based on observations of geologic materials and conditions encountered during grading. The bottom of all corrective grading removals should be observed by a representative of NTS to verify the suitability of in-place soil prior to performing scarification and recompaction. Corrective grading recommendations are outlined below.

<u>Structures Supported on Spread Footings and Engineered Fill</u>

In order to create a firm and stable platform on which to construct the new building foundations that supported directly on engineered fill and without ground improvement, we recommend the following:

- The building pads should be excavated to a depth of at least 12 feet below the bottom of the foundation.
- The bottom of the over excavation should then be scarified to a depth of at least 8 inches, moisture conditioned to 2 percent above optimum moisture content and recompacted to at least 90 percent relative compaction as determined in accordance with ASTM D1557.
- Following the approval of the over-excavation bottom by a representative of NTS, the onsite material may be used as fill material to achieve the planned pad grade.

- The fill material should then be placed in 6- to- 8-inch-thick lifts, moisture conditioned to 2 percent above optimum moisture content and compacted to achieve 90 percent relative compaction.

Structures Supported on Mat Foundation

For buildings that are planned to be supported on a mat foundation system, we recommend the following:

- The building pads should be excavated to a depth of at least 4 feet below the bottom of the mat foundation.
- The bottom of the over excavation should then be scarified to a depth of at least 8 inches, moisture conditioned to 2 percent above optimum moisture content and recompacted to at least 90 percent relative compaction as determined in accordance with ASTM D1557.
- Following the approval of the over-excavation bottom by a representative of NTS, the onsite material may be used as fill material to achieve the planned pad grade.
- The fill material should then be placed in 6- to- 8-inch-thick lifts, moisture conditioned to 2 percent above optimum moisture content and compacted to achieve 90 percent relative compaction.

Alternative 1: Structures Supported on Spread Footings and Geopiers or Equivalent Gravel Piers

For buildings that are planned to be supported on a shallow foundation and Geopiers or equivalent gravel piers system, we recommend the following:

- The building pads should be excavated to a depth of at least 5 feet from finish pad grade and recompacted prior to installation of the Geopiers or equivalent gravel piers to provide support for the slab-on-grade.
- The bottom of the over excavation should then be scarified to a depth of at least 8 inches, moisture conditioned to 2 percent above optimum moisture content and recompacted to at least 90 percent relative compaction as determined in accordance with ASTM D1557.
- Following the approval of the over-excavation bottom by a representative of NTS, the onsite material may be used as fill material to achieve the planned pad grade.
- The fill material should then be placed in 6- to- 8-inch-thick lifts, moisture conditioned to 2 percent above optimum moisture content and compacted to achieve 90 percent relative compaction.

Alternative 2: Structures Supported on Spread Footings and Geopiers or Equivalent Gravel Piers

As a secondary alternative, for buildings that are planned to be supported on shallow foundation and Geopiers or equivalent gravel piers system, and due to the presence of artificial fill material, the proposed building slabs may be supported on a grid of Geopiers or equivalent gravel piers to allow the slab to span the existing undocumented fill. The Geopiers or equivalent gravel piers should be designed by a specialty contractor in such way that the slab does not receive support for the underlying soil.

Pavement / Hardscape

In order to create a firm and stable platform on which to construct the new vehicular pavement and non-vehicular hardscape, we recommend the following:

- The proposed pavement / hardscape should be excavated to the planned subgrade (i.e., bottom of aggregate base for pavement and bottom of concrete for flatwork).
- The bottom of the excavation should then be excavated to a depth of 12 inches below the planned subgrade.
- The bottom of the over excavation should then be scarified to a depth of at least 6 inches, moisture conditioned to 2 percent above optimum moisture content and recompacted to at least 90 percent relative compaction as determined in accordance with ASTM D1557.
- Following the approval of the over-excavation bottom by a representative of NTS, the onsite material may be used as fill material to achieve the planned pad grade.
- The fill material should then be placed in 6- to- 8-inch-thick lifts, moisture conditioned to 2 percent above optimum moisture content and compacted to achieve 90 percent relative compaction.

If the existing loose fill materials are found to be disturbed to depths greater than the proposed remedial grading, then the depth of over-excavation and re-compaction should be increased accordingly in local areas as recommended by a representative of NTS.

Materials for Fill

On-site soils with an organic content of less than 3 percent by volume (or 1 percent by weight) are suitable for use as fill. Soil material to be used as fill should not contain contaminated materials, rocks, or lumps over 6 inches in largest dimension, and not more than 40 percent larger than ¾ inch. Utility trench backfill material should not contain rocks or lumps over 3 inches in largest dimension. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or may be disposed offsite.

Any imported fill material should consist of granular soil having a “very low” expansion potential (that is, expansion index of 20 or less). Import material should also have low corrosion potential (that is, chloride content less than 500 parts per million [ppm], soluble sulfate content of less than 0.1 percent, and pH of 5.5 or higher). Materials to be used as fill should be evaluated by a representative of NTS prior to importing or filling.

Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed excavation bottom by NTS. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of at least 8 inches and watered or dried, as needed, to achieve generally consistent moisture contents approximately 2 percent above the optimum moisture content. The scarified materials should then be compacted to 90 percent relative compaction in accordance with the latest version of ASTM Test Method D1557.

Compacted fill should be placed in horizontal lifts of approximately 6 to 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve near optimum moisture condition, mixed, and then compacted by mechanical methods, using sheepsfoot rollers, multiple-wheel pneumatic-tired rollers, or other appropriate compacting rollers, to a relative compaction of 95 percent as evaluated by ASTM D1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved. Within pavement areas, the upper 12 inches of subgrade soil should be compacted to 95 percent relative compaction evaluated by ASTM D1557.

Personnel from NTS should observe the excavations so that any necessary modifications based on variations in the encountered soil conditions can be made. All applicable safety requirements and regulations, including CalOSHA requirements, should be met.

Excavation Bottom Stability

Based on our subsurface investigation we anticipate that the bottom of the excavation may expose localized areas of saturated clay material. If encountered and schedule does not allow for drying of the material, unstable bottom conditions may be mitigated by overexcavation of the bottom to suitable depths, and/or replacement with a minimum 2-foot-thick aggregate base, or other options may be recommended based on the field evaluation. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by NTS at the time of construction.

Temporary Excavations

Temporary excavations for the demolishing, earthwork, footing and utility trench are expected. We anticipate that unsurcharged excavations with vertical side slopes less than 3 feet high will generally be stable; however, sloughing of cohesionless sandy materials encountered at the site should be expected.

Where the space is available, temporary, unsurcharged excavation sides over 3 feet in height should be sloped no steeper than an inclination of 1.5H:1V (horizontal:vertical). Where sloped excavations are created, the tops of the slopes should be barricaded so that vehicles and storage loads do not encroach within 10 feet of the top of the excavated slopes. A greater setback may be necessary when considering heavy vehicles, such as concrete trucks and cranes. NTS should be advised of such heavy vehicle loadings so that specific setback requirements can be established. If the temporary construction slopes are to be maintained during the rainy season, berms are recommended to be graded along the tops of the slopes in order to prevent runoff water from entering the excavation and eroding the slope faces.

Where space for sloped excavations is not available, temporary shoring may be utilized. Geotechnical recommendations for the design and construction of temporary shoring are presented in the “Temporary Shoring” section of this report. Personnel from NTS should observe the excavation so that any necessary modifications based on variations in the encountered soil conditions can be made. All applicable safety requirements and regulations, including CalOSHA requirements, should be met.

Excavations shall not undermine the existing adjacent building footings. Where space for sloped excavations is not available, temporary shoring may be utilized.

Temporary Shoring

Temporary shoring is anticipated to be placed along the perimeter of the proposed site. Based on the depth of excavation depending on the foundation system selected, we anticipate excavation on the order of 15 feet deep.

Where excavations exceed 15 feet or are surcharged, restrained shoring may be necessary to limit deflections and disruption to nearby improvements. The size of the steel beam, the need for lateral bracing, and the acceptable shoring deflection should be determined by the project shoring engineer.

The shoring design should be provided by a California Registered Civil Engineer experienced in the design and construction of shoring under similar conditions. Once the final excavation and shoring plans are complete, the plans and the design should be reviewed by NTS for conformance with the design intent and recommendations. Further, the shoring system should satisfy applicable requirements of CalOSHA.

Lateral Earth Pressures

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the drained soils, with a level surface behind the cantilevered shoring, will exert an active equivalent fluid pressure of 40 pcf.

Any surcharge (live, including traffic, or dead load) located within 1:1 plane projected upward from the base of the shored excavation, including adjacent structures, should be added to the lateral earth pressures. The lateral contribution of a uniform surcharge load located immediately behind the temporary shoring may be calculated by multiplying the vertical surcharge pressure by 0.30. Lateral load contributions of surcharges located at a distance behind the shored wall may be provided once the load configurations and layouts are known. As a minimum, a 250 psf vertical uniform surcharge is recommended to account for nominal construction and/or traffic loads. More detailed lateral pressure and loading information can be provided, if needed, for specific loading scenarios as recognized through the design process.

Soldier Pile Design

The design embedment of the shoring pile toes must be maintained during excavation activities. The toes of the perimeter shoring piles should be deepened to take into account any required excavations necessary for foundations grading, installation, or drainage systems.

Drilled cast-in-place soldier piles should be placed no closer than 2.5 diameters on center. The minimum diameter of the piles should be 24 inches. Structural concrete should be used for the soldier piles below the excavations; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The lean-mix must be sufficient strength to impart the lateral bearing pressured developed by the wideflange section to the earth materials.

For design purposes, an allowable passive resistance value for the earth materials below the bottom of the excavation may be assumed to be 300 pounds per square foot per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed earth materials.

The frictional resistance between the solider piles and retained earth material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.30 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of the soldier piles below the place of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 340 pounds per square foot. Final embedment of shoring pile below the bottom of the excavation should be determined by the project shoring engineer.

Drilling of the soldier pile shafts can be accomplished using conventional drilling equipment. Additionally, caving should be anticipated within the upper approximately 15 feet below the existing grade, where layers of loose to medium dense sand was encountered during our drilling program. In the event of soil caving, it may be necessary to use casing and/or drilling mud to permit the installation of the soldier piles. Drilled holes for soldier piles should not be left open overnight. Concrete for piles should be placed immediately after the drilling of the hole is complete. The concrete should be pumped to the bottom of the drilled shaft using a tremie. Once concrete pumping is initiated, the bottom of the tremie should remain below the surface of the concrete to prevent contamination of the concrete by soil inclusions. If steel casing is used, the casing should be removed as the concrete is placed.

Lagging

Lagging should be designed for the full design pressure, but be limited to a maximum of 400 psf. NTS representative should observe the installation of lagging to insure uniform support of the excavated embankment. In addition, backfill behind the lagging should consist of a 2 sack, sand-cement slurry, and should be placed immediately once the lagging is installed.

Monitoring

In conjunction with the shoring installation, a monitoring program should be set up and carried out by the contractor to determine the effects of the construction on adjacent buildings and other improvements such as streets, sidewalks, utilities and parking areas. At minimum, we recommend the following:

- Horizontal and vertical surveying of reference points on the shoring and on adjacent streets and buildings, in addition to an initial pre-construction photographic, video and/or survey of adjacent improvements.
- All supported and/or sensitive utilities should be located and monitored by the contractor.
- Reference points should be set up and read prior to the start of construction activities.
- Points should also be set on the shoring as soon as initial installations are made.
- Alternatively, inclinometers could be installed by the contractor at critical locations for a more detailed monitoring of shoring deflections.

- Surveys should be made at least once a week, and more frequently during critical construction activities, or if significant deflections are noted.

Seismic Design

Based on the average standard penetration resistance (N-value) of the upper 100 feet of subsurface soils, the site is designated as Site Class D ("stiff" soil profile). The seismic design parameters based on ASCE 7-16 and 2019 CBC are listed in the following table.

2019 CBC and ASCE 7-16 Seismic Design Parameters

Seismic Item		Design Value	2016 ASCE 7-16 or 2019 CBC Reference
Site Class based on soil profile (ASCE 7-16 Table 20.3-1)		D[(a)]	ASCE 7-16 Table 20.3-1
Short Period Spectral Acceleration S_s		1.661[(a)]	CBC Figures 1613.2.1 (1-8)
1-sec. Period Spectral Acceleration S_1		0.585[(a)]	CBC Figures 1613.2.1 (1-8)
Site Coefficient F_a (2019 CBC Table 1613.2.3(1))		1.000[(a)]	CBC Table 1613.2.3 (1)
Site Coefficient F_v (2019 CBC Table 1613.2.3(2))		1.715[(b)]	CBC Table 1613.2.3 (2)
Short Period MCE* Spectral Acceleration S_{MS}	$S_{MS} = F_a S_s$	1.661[(a)]	CBC Equation 16-36
1-sec. Period MCE Spectral Acceleration S_{M1}	$S_{M1} = F_v S_1$	1.003[(b)]	CBC Equation 16-37
Short Period Design Spectral Acceleration S_{DS}	$S_{DS} = 2/3 S_{MS}$	1.107[(a)]	CBC Equation 16-38
1-sec. Period Design Spectral Acceleration S_{D1}	$S_{D1} = 2/3 S_{M1}$	0.669[(b)]	CBC Equation 16-39
Short Period Transition Period T_S (sec)	$T_S = S_{D1}/S_{DS}$	0.604[(b)]	ASCE 7-16 Section 11.4.6
Long Period Transition Period Tl (sec)		8[(b)]	ASCE 7-16 Figures 22-14 to 22-17
MCE[(c)] Peak Ground Acceleration (PGA)		0.712[(a)]	ASCE 7-16 Figures 22-9 to 22-13
Site Coefficient F_{PGA} (ASCE 7-16 Table 11.8-1)		1.100[(a)]	ASCE 7-16 Table 11.8-1
Modified MCE[(c)] Peak Ground Acceleration (PGA_M)		0.783[(a)]	ASCE 7-16 Equation 11.8-1

(a) Design Values Obtained from USGS Earthquake Hazards Program website that are based on the ASCE-7-16 and 2019 CBC and site coordinates of N33.8744° and W117.8835°.
(b) Design Values Determined per ASCE Table 11.4-2 and CBC Equations 16-36 through 16-39.
(c) MCE: Maximum Considered Earthquake.

Since the Site Class is designated as D and the S1 value is greater than or equal to 0.2, the 2019 CBC requires either a site-specific seismic hazard analysis per Section 21.2 of ASCE 7-16 or the application of Exception 2 of Section 11.4.8 of ASCE 7-16. The project structural engineer should apply all requirements of Section 11.4.8 of ASCE 7-16 to determine if increases to the seismic response coefficient (i.e. increases to the loading of the structure) are required. If increases are required, a site-specific seismic hazard analysis may result in decreased loading and possible cost savings. Please contact NTS if a site-specific seismic hazard analysis is desired.

Per the 2019 CBC and ASCE 7-16, the Design Earthquake peak ground acceleration (PGAD) may be assumed to be equivalent to SDS/2.5; therefore, for the subject site, a PGAD value of 0.44g (1.107/2.5) should be used.

It should be recognized that much of southern California is subject to some level of damaging ground shaking as a result of movement along the major active (and potentially active) fault zones that characterize this region. Design utilizing the 2019 CBC is not meant to completely protect against damage or loss of function. Therefore, the preceding parameters should be considered as minimum design criteria.

Spread Footings on Engineering Fill Design and Construction

A spread/continuous foundation system may be used to support the proposed buildings, provided that the Corrective Grading recommendations are performed and structure can accommodate for the estimate settlement provided below. The spread/continuous footings may be designed using the following recommendations:

Bearing Material	▪ Engineered Fill ▪ 12 feet of compacted fill below bottom of footings
Minimum Footing Dimension	▪ A minimum footing with of 24 inches and footing depth of 24 inches.
Allowable Bearing Capacity	▪ Based on the minimum footing dimension above, an allowable bearing capacity of 2,500 psf may be used. This value may be increased by 100 or each additional footing width, and 400 for each additional footing depth to a maximum allowable of 3,000 psf. ▪ The above value may be increased by 1/3 for temporary loads such as wind or earthquake.
Static Settlement	▪ Total static settlement of 1 inch with differential settlement estimated to be approximately ½ inch over a span of 40 feet.
Seismic Settlement	▪ Total seismic settlement of 2.0 inches with differential settlement of 1.0 inch over a span of 40 feet.
Allowable Lateral Passive Resistance*	• 300 pcf (equivalent fluid pressure)
Allowable Coefficient of Friction *	• 0.35

*These values may be combined without reduction and may be increased by 1/3 for temporary loads such as wind or seismic.

Spread Footings on Geopiers or Equivalent Gravel Piers

Based on the site conditions and depth of excavation and recompaction for shallow spread footings as discussed in the previous sections of this report, it is our opinion that Geopiers or equivalent gravel piers supported shallow foundation may be used for support of the structures. This ground improvement will allow for increase in bearing capacity, typically about 5,000 psf, which result in smaller size of shallow foundations based on assumed structural loads. If this option is selected, we recommend that once a generalized foundation plan is developed, we review the applicability of Geopiers or equivalent gravel piers-supported foundations at this site. We note that the final design of this system is provided by specialty contractor and is reviewed by this office.

Mat Foundation Design and Construction

A mat foundation system may be used for support of the proposed buildings, provided that all the footings are placed on engineered fill prepared as described in the "**Corrective Grading**" section of this report. The preliminary design parameters presented below may be used for foundation structural design.

Bearing Material	▪ Engineered Fill ▪ 4 feet of compacted fill below bottom of footings ▪ A moisture vapor retarder consisting of Stegowrap 15 mil or equivalent should be placed.
Minimum Mat Foundation	▪ Based on an estimated building footprint dimension of 160 feet by 405 feet, estimate that the building load distributed uniformly over the mat foundation footprint may induce an approximate uniform pressure of 400 psf for dead plus live load ▪ Assumed minimum mat thickness of 24 inches. ▪ Final mat foundation thickness should be determined by the structural engineer.
Allowable Bearing Capacity	▪ Based on the assumptions above, the mat foundation estimate of an approximate uniform pressure of 400 psf can also be taken as the allowable bearing capacity. ▪ The above value may be increased by 1/3 for temporary loads such as wind or

	earthquake.
Static Settlement	▪ Total static settlement of 1.5 inches with differential settlement estimated to be approximately ¾ inch over a span of 40 feet.
Seismic Settlement	▪ Total seismic settlement of 2.5 inches with differential settlement of 1.5 inches over a span of 40 feet.
Allowable Lateral Passive Resistance*	• 300 pcf (equivalent fluid pressure)
Allowable Coefficient of Friction *	• 0.35
Modulus of Subgrade Reaction (k)	• 75 pci (static)

*These values may be combined without reduction and may be increased by 1/3 for temporary loads such as wind or seismic.

The mat slab should be designed by the project structural engineer. In addition, in order to finalize the mat foundation recommendations, we recommend that the structural engineer model the mat foundation with all anticipated point loads utilizing the provided Modulus of Subgrade Reaction (k) in this section, and provide this office with the analyses, including bearing pressure and settlement contour under the slab.

Moisture Vapor Retarder

A vapor retarder, such as a 15-mil-thick moisture vapor retarder that meets the requirements of ASTM E1745 Class C (Stego Wrap or equivalent) should be placed directly over the prepared soil subgrade to provide protection against vapor transmission through concrete floor slabs thatare anticipated to receive carpet, tile or other moisture sensitive coverings. The use of moisture vapor retarder should be determined by the project architect. At minimum, the vapor retarder should be installed as follows:

- Per the manufacture's specifications as well as with the applicable recognized installation procedures such as ASTM E1643;
- Joints between the sheets and the openings for utility piping should be lapped and taped. If the barrier is not continuously placed across footings/ribs, the barrier should at minimum be lapped into the side of the footing/rib trenches down to the bottom of the trench; and,
- Punctures in the vapor retarder should be repaired prior to concrete placement.

It should be noted that the moisture retarder is intended only to reduce moisture vapor transmissions from the soil beneath the concrete and is consistent with the current standard of the industry in the building construction in Southern California. It is not intended to provide a “waterproof” or “vapor proof” barrier or reduce vapor transmission from sources above the retarder (i.e., concrete). The evaluation of water vapor from any source and its effect on any aspect of the proposed building space above the slab (i.e., floor covering applicability, mold growth, etc.) is beyond our purview and the scope of this report.

Structural Concrete

Based on Laboratory test results for the site vicinity, the potential of sulfate attack on concrete in contact with the on-site soils is “negligible” based on ACI 318, Table 19.3.1.1. On this basis, we recommend using:

- Type II/V cement with a maximum water to cement ratio of 0.50.

Utilization of the CBC’s moderate sulfate level requirements will also serve to reduce the permeability of the concrete and help reduce the potential of water and/or vapor transmission through the concrete. Wet curing of the concrete per ACI Publication 308 is also recommended.

The aforementioned recommendations in regards to concrete are made from a soils perspective only. Final concrete mix design is beyond our purview. All applicable codes, ordinances, regulations, and guidelines should be followed in regard to the designing a durable concrete with respect to the potential for sulfate exposure from the on-site soils and/or changes in the environment.

Drainage Control

The control of surface water is essential to the satisfactory performance of the building and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath the improvements, even during periods of heavy rainfall. The following recommendations are considered minimal:

- Ponding and areas of low flow gradients should be avoided.
- If bare soil within 5 feet of the structure is not avoidable, then a gradient of 5 percent or more should be provided sloping away from the improvement. Corresponding paved surfaces should be provided with a gradient of at least 1 percent.
- The remainder of the unpaved areas should be provided with a drainage gradient of at least 2 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and to convey water to appropriate discharge points.

- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Brick flatwork should be sealed by mortar or be placed over an impermeable membrane.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.
- Planters should not be located adjacent to the structures wherever possible. If planters are to be located adjacent to the structures, the planters should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to off-site disposal areas by a pipe or concrete swale system.
- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive watering. Sprinkler systems should be checked.

Utility Trench Backfill Considerations

New utility line pipeline trenches should be backfilled with select bedding materials beneath and around the pipes (pipe zone) and compacted soil above the pipe bedding. Recommendations for the types of the materials to be used and the proper placement of these materials are provided in the following sections.

Pipe Zone (Bedding and Shading)

The pipe bedding and shading materials should extend from at least 6 inches below the pipes to at least 12 inches above the crown of the pipes. Pipe bedding and shading should consist of either clean sand with a sand equivalent (SE) of at least 30, or crushed rock. If crushed rock is used, it should consist of ¾-inch crushed rock that conforms to Table 200-1.2.1 (A) of the 2018 “Greenbook.” Pipe bedding and shading should also meet the minimum requirements of the City of Los Angeles. If the requirements of the City are more stringent, they should take precedence over the geotechnical recommendations. Sufficient laboratory testing should be performed to verify the bedding and shading meets the minimum requirements of the Greenbook and City of Fullerton grading codes.

Based on our subsurface exploration and knowledge of the onsite materials, the soils that will be excavated from the pipeline trenches will not meet the recommendations for pipe bedding and shading materials; therefore, imported materials will be required for pipe bedding and shading.

Granular pipe bedding and shading material should be properly placed in thicknesses not exceeding 3 feet, and then sufficiently flooded or jetted in place. Crushed rock, if used, should be capped with filter fabric (Mirafi 160N, or equivalent; Mirafi 140N filter fabric is suitable if available) to prevent the migration of fines into the rock.

Trench Backfill

All existing soil material within the limits of the site are considered suitable for use as trench backfill above the pipe bedding and shading zone if care is taken to remove all significant organic and other decomposable debris, moisture condition the soil materials as necessary, and separate and selectively place and/or stockpile any inert materials larger than 6 inches in maximum diameter.

Imported soils are not anticipated for backfill since the on-site soils are suitable. However, if imported soils are used, the soils should consist of clean, granular materials with physical and chemical characteristics similar to or better than those described herein for on-site soils. Any imported soils to be used as backfill should be evaluated and approved by NTS prior to placement.

Soils to be used as trench backfill should be moistened, dried, or blended as necessary to achieve a minimum of 2 percent over optimum moisture content, placed in lifts which, prior to compaction shall not exceed the thickness specified in Section 306-12.3 of the 2018 “Greenbook” for various types of equipment, and mechanically compacted/densified to at least 90 percent relative compaction as determined by ASTM Test Method D 1557. Jetting is not permitted in this trench zone.

No rock or broken concrete greater than 6 inches in maximum diameter should be utilized in the trench backfills.

Asphalt Concrete Pavement Design

In accordance with Chapter 600 of the Caltrans Highway Design Manual, we have performed pavement structural design utilizing assumed traffic indices (TI) of 4 and 5.5 and our laboratory R-value test result of 15. Based on our analysis, we have developed the pavement structural sections presented in the following table. We note that the assumed TI's should be reviewed by a traffic engineer to confirm their applicability to the project.

Minimum Asphalt Concrete Pavement Structural Sections

Location	Traffic Index	Asphalt Concrete (in.)	Aggregate Base (in.)*
Parking Stalls	4.0	3.0	4.0
Driveway	5.5	4.0	8.0

The above design sections will need to be verified based on additional testing performed at the completion of future precise grading of the specific locations.

The planned pavement structural sections should consist of the following:

- Aggregate Base materials (AB) consisted of either Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB).
- Asphalt Concrete (AC) material of a type meeting the minimum City of Fullerton standards.
- The subgrade soils should be moisture conditioned to a minimum of 2 percent above optimum moisture content to a depth of at least 18 inches and compacted to 90 percent relative compaction.
- The AB and AC should be compacted to at least 95 percent relative compaction.

Exterior Flatwork/Hardscape Design Considerations

For exterior flatwork and hardscape planned as part of the proposed development, the following design may be considered by the project civil engineer. These recommendations may be considered as minimal design based on the soils conditions encountered during our investigation. Final design of the proposed flatwork and hardscape area should be provided by the project civil engineer. Based on the conditions encountered, we recommend that the subgrade for the subject concrete flatwork and hardscape be moisture conditioned to 2 percent over optimum to a depth of 18 inches below finish subgrade elevation and compacted to 90 percent relative compaction. A Type II/V cement may be used from a geotechnical perspective. Our flatwork and hardscape design considerations are presented in the table below.

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Concrete Flatwork Table

Description	Subgrade Preparation (1)	Minimum Concrete Thickness	Cut-Off Barrier Or Edge Thickness	Reinforcement(2)	Joint Spacing (Maximum)	Concrete(3)
Concrete Sidewalks and Walkways (4)	1) 2% over optimum to 18"(1), 2) 2" of sand or well graded rock (i.e., Class II base or equiv.) above moisture conditioned subgrade.	4 inches	Not Required	No. 3 bars @ 18"o.c.b.w. and dowel into building and curb using No. 3 bars @ 18"o.c (5)	5 feet	Type II/V
Concrete Driveways(4)	1) 2% over optimum to 18"(1), 2) 2" of sand or well graded rock (i.e., Class II base or equiv.) above moisture conditioned subgrade.	8 inches	Where adjacent to landscape areas – 12" from adjacent finish grade. Min. 8" width	1) Slab – No. 3 bars @ 18" o.c. (2) bent into cut-off; 2) where adjacent to curbs use dowels: No. 3 bars @ 18" o.c. (5)	10 feet	Type II/V

(1) The moisture content of the subgrade must be verified by the geotechnical consultant prior to sand/rock placement.
(2) Reinforcement to be placed at or above the mid-point of the slab (i.e., a minimum of 2.0 to 2.5 inches above the prepared subgrade).
(3) The site has negligible levels of sulfates as defined by the CBC. Concrete mix design is outside the geotechnical engineer's purview.
(4) Where flatwork is adjacent a stucco surface, a ¼" to ½" foam separation/expansion joint should be used.
(5) If dowels are placed in cored holes, the core holes shall be placed at alternating in-plane angles (i.e., not cored straight into slab).

Planters and Trees

Where new trees or large shrubs are to be located in close proximity to new concrete flatwork, rigid moisture/root barriers should be placed around the perimeter of the flatwork to at least 12 inches in depth in order to offer protection to the adjacent flatwork against potential root and moisture damage. Existing mature trees near flatwork areas should also incorporate a rigid moisture/root barrier placed at least 2 feet in depth below the top of the flatwork.

Plans and Specifications Review

The recommendations presented in this report are contingent upon review of final plans and specifications for the project by NTS. NTS Geotechnical, Inc. should review and verify in writing the compliance of the final grading plan and the final foundation plans with the recommendations presented in this report.

Construction Observation and Testing

It is recommended that NTS be retained to provide continuous Geotechnical Consulting services during the earthwork operations (i.e., shoring, rough grading, utility trench backfill, subgrade preparation for slabs-on-grade, finish grading, etc.) and foundation installation process. This is to observe compliance with the design concepts, specifications and recommendations and to allow for design changes in the event that subsurface conditions differ from those anticipated during our subsurface investigation.

LIMITATIONS

All parties reviewing or utilizing this report should recognize that the findings, conclusions, and recommendations presented represent the results of our professional geological and geotechnical engineering efforts and judgments. Due to the inexact nature of the state of the art of these professions and the possible occurrence of undetected variables in subsurface conditions, we cannot guarantee that the conditions actually encountered during grading and site construction will be identical to those observed, sampled, and interpreted during our study, or that there are no unknown subsurface conditions which could have an adverse effect on the use of the property. We have exercised a degree of care comparable to the standard of practice presently maintained by other professionals in the fields of geotechnical engineering and engineering geology, and believe that our findings present a reasonably representative description of geotechnical conditions and their probable influence on the grading and use of the property.

Our conclusions and recommendations are based on the assumption that our firm will act as the geotechnical engineer of record during construction and grading of the project to observe the actual conditions exposed, to verify our design concepts and the grading contractor's general compliance with the project geotechnical specifications, and to provide our revised conclusions and recommendations should subsurface conditions differ significantly from those used as the basis for our conclusions and recommendations presented in this report. Since our conclusions and recommendations are based on a limited amount of current and previous geotechnical exploration and analysis, all parties should recognize the need for possible revisions to our conclusions and recommendations during grading of the project.

It should be further noted that the recommendations presented herein are intended solely to minimize the effects of post-construction soil movements. Consequently, minor cracking and/or distortion of all on-site improvements should be anticipated.

This report has not been prepared for the use by other parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

REFERENCES

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California Department of Conservation, Division of Mines and Geology, 1997, Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangle, Orange County, California: Seismic Hazard Zone Report 03.

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Coduto, Donald P., 1994, Foundation Design: Principles and Practices: Prentice-Hall, Inc, Englewood Cliffs, New Jersey.

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Southern California Earthquake Center, 1999, Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction in California: dated March, 63 pp.

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LOCATION MAP

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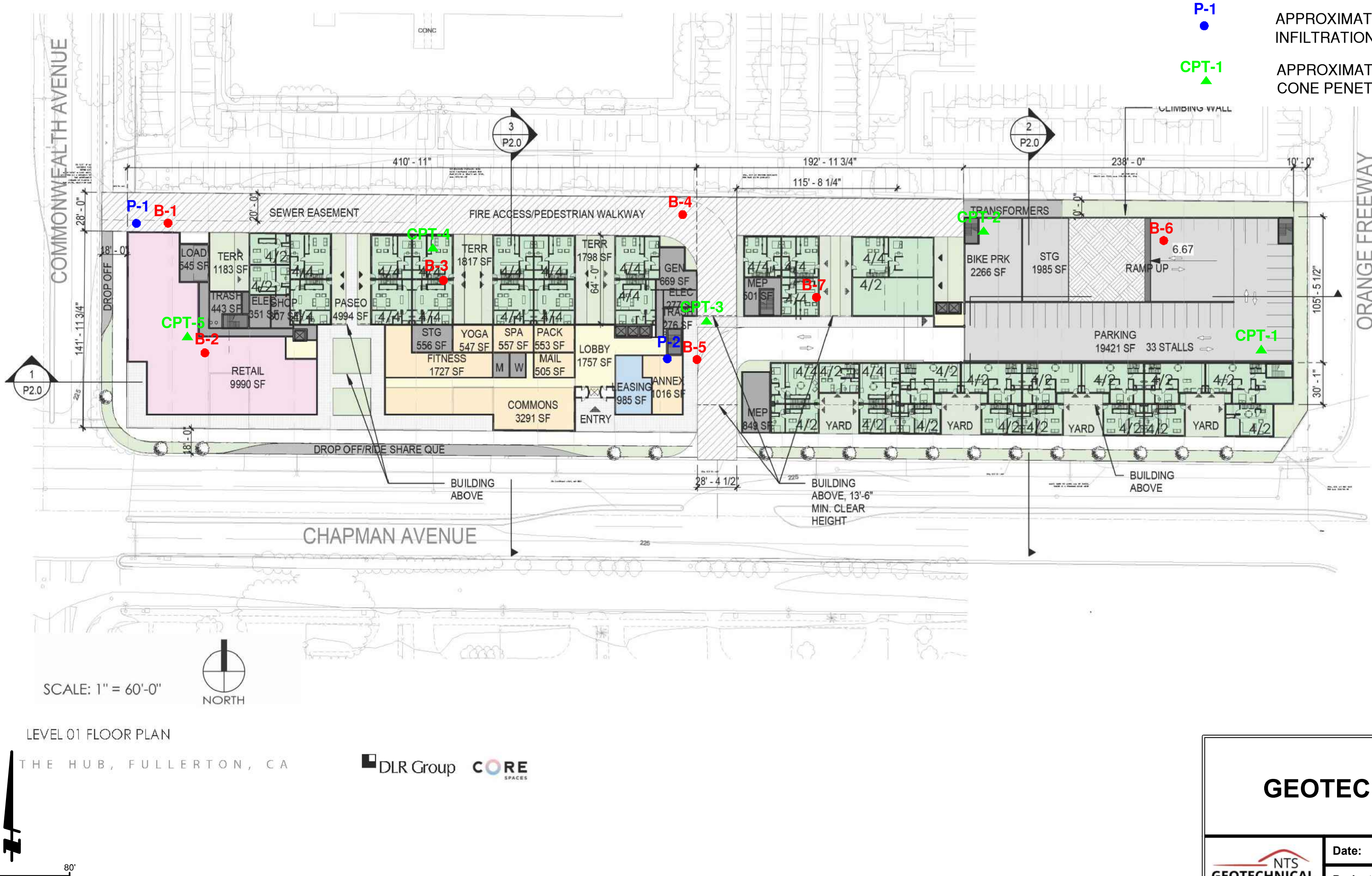
Date:	OCTOBER 2, 2020	Plate 1
Project No.:	20073	



GEOTECHNICAL MAP

NTS GEOTECHNICAL

Date:	SEPTEMBER 30, 2020	Plate 2
Project No.:	20073	

DRAWING: c:\users\nsunna\sync\nadim\nadim\nts geotechnical\04 projects\2020\20073 – 2601 chapman ave, fullerton\plate 2 – geotechnical map_r1.dwg PLOTTED: 9/28/2020 2:14 PM BY: Nadim Sunna

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APPENDIX A

Field Exploration

Appendix A
Field Exploration

The subsurface exploration program for the proposed project consisted of advancing seven (7) 8-inch-diameter, hollow-stem-auger drill rig borings and five (5) Cone Penetration Testing (CPT) soundings at the subject site. The borings were advanced to depths ranging from 10 to 61.5 feet below the existing grade and CPT's were advanced to a maximum depth of 50 feet below the existing grade. The CPT logs are presented within Appendix A-1.

The Boring Logs are presented as Figures A-3 to A-11. The Boring Logs describe the earth materials encountered, samples obtained, and show the field and laboratory tests performed. The log also shows the boring number, drilling date, and the name of the logger and drilling subcontractor. The borings were logged by an engineer using the Unified Soil Classification System. The boundaries between soil types shown on the logs are approximate because the transition between different soil layers may be gradual. Drive and bulk samples of representative earth materials were obtained from the borings.

Disturbed samples were obtained using a Standard Penetration Sampler (SPT). This sampler consists of a 2-inch O.D., 1.4-inch I.D. split barrel shaft that is advanced into the soil at the bottom of the drilled hole a total of 18 inches. The number of blows required to drive the sampler 18 inches is presented on the boring logs. Soil samples obtained by the SPT were retained in plastic bags. A California modified sampler was used to obtain drive samples of the soil encountered. This sampler consists of a 3-inch outside diameter (O.D.), 2.4-inch inside diameter (I.D.) split barrel shaft that was driven a total of 12-inches into the soil at the bottom of the boring by a safety hammer weighing 140 pounds at a drop height of approximately 30 inches. The soil was retained in brass rings for laboratory testing. Additional soil from each drive remaining in the cutting shoe was usually discarded after visually classifying the soil. The number of blows required to drive the sampler 18 inches is presented on the boring logs.

Upon completion of the borings, the boreholes were backfilled with soil from the cuttings.

UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

COARSE-GRAINED SOILS			FINE-GRAINED SOILS	
Relative Density	SPT (blows/ft)	Relative Density (%)	Consistency	SPT (blows/ft)
Very Loose	<4	0 - 15	Very Soft	<2
Loose	4 - 10	15 - 35	Soft	2 - 4
Medium Dense	10 - 30	35 - 65	Medium Stiff	4 - 8
Dense	30 - 50	65 - 85	Stiff	8 - 15
Very Dense	>50	85 - 100	Very Stiff	15 - 30
			Hard	>30

NOTE: SPT blow counts based on 140 lb. hammer falling 30 inches

Sample Symbol	Sample Type	Description
	SPT	1.4 in I.D., 2.0 in. O.D. driven sampler
	California Modified	2.4 in. I.D., 3.0 in. O.D. driven sampler
	Bulk	Retrieved from soil cuttings
	Thin-Walled Tube	Pitcher or Shelby Tube

LABORATORY TESTING ABBREVIATIONS

ATT	Atterberg Limits
C	Consolidation
CORR	Corrosivity Series
DS	Direct Shear
EI	Expansion Index
GS	Grain Size Distribution
K	Permeability
MAX	Moisture/Density (Modified Proctor)
O	Organic Content
RV	Resistance Value
SE	Sand Equivalent
SG	Specific Gravity
TX	Triaxial Compression
UC	Unconfined Compressior

BORING LOGS EXPLANATION

2601 – 2751 Chapman Ave
Fullerton, California

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FIGURE A-2

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SUBSURFACE EXPLORATION LOG
BORING NO. B-1

				Project No.:	20073
Project Name:	2601 Chapman Ave	Date:	4/2/2020	Logged By:	LB
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Elevation:	~225 ft. MSL
Drill Hole Dia.:	8"	Drop:	30"	Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
								ASPHALT CONCRETE / AGGREGATE BASE (~10 inches)
1			SM			Af		**ARTIFICIAL FILL** SILTY SAND, brown, damp to moist, fine-grained sand, loose
2								
3			CL			Qyf		**YOUNG ALLUVIAL FAN DEPOSITS** SANDY CLAY, dark brown, moist, fine-grained sand
4								
5	S	2						
		2						
6		3	ML/SM			Qyf		SANDY SILT TO SILTY SAND, light brown, damp to moist, loose
7								
8								
9								
10	R	4						loose
		4						
11		9	SM			Qyf		light brown, moist, stiff, fine-grained sand, some clay
12								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

'e A-3 (Sheet 1 of 6)

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SUBSURFACE EXPLORATION LOG
BORING NO. B-1

				Project No.:	20073
Project Name:	2601 Chapman Ave	Date:	4/2/2020	Logged By:	LB
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Elevation:	~225 ft. MSL
Drill Hole Dia.:	8"	Drop:	30"	Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
13			CL			Qyf		SANDY CLAY, brown, moist to very moist, stif to very stiff
14								
15	S	9	CL			Qyf		stiff
		4						
16		5						
17								
18								
19								
20	R	3	CL	116.2	12.8	Qyf		stiff to very stiff
		6						
21		20						
22								
23								
24								

S - SPT Sample   R - Ring Sample   B - Bulk Sample   D - Disturbed Sample

Figure A-3 (Sheet 2 of 6)

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SUBSURFACE EXPLORATION LOG
BORING NO. B-1

Project Name:	2601 Chapman Ave	Date:	4/2/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
25	S	8	CL			Qyf		very stiff
		11						red brown
26		11						
27								
28								
29								
30	R	7	CL	116.0	14.8	Qyf		stiff to very stiff
		17						
31		23						
32								
33			SC			Qyf		CLAYEY SAND, red brown, moist, medium dense, fine-grained sand
34								
35	S	12	SC			Qyf		medium dense
		12						
36		14						

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

'e A-3 (Sheet 3 of 6)

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SUBSURFACE EXPLORATION LOG
BORING NO. B-1

Project Name:	2601 Chapman Ave	Date:	4/2/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
37								
38								
39								
40	R	4	SC	110.3	15.1	Qyf		medium dense
		4						
41		10	CL			Qyf		SANDY CLAY, brown, moist
42			SC					CLAYEY SAND, brown to light brown, moist
43								
44								
45	S	7	SC			Qyf		medium dense
		9						
46		12						
47			CL					SANDY CLAY, brown, moist
48								

S - SPT Sample   R - Ring Sample   B - Bulk Sample   D - Disturbed Sample

'e A-3 (Sheet 4 of 6)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-1

Project Name:	2601 Chapman Ave	Date:	4/2/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
49								
50	R	9						
		9						
51		16	CL	102.9	21.3	Qyf		brown, very moist, stiff to very stiff
52								
53								
54								
55	S	4	CL			Qyf		
		10						olive brown, moist, fine- to- coarse-grained sand, stiff to very stiff
56		5						
57								
58								
59								
60	R		CL			Qyf		brown, very moist, fine-grained sand

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

'e A-3 (Sheet 5 of 6)

DRAFT

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-1

				Project No.:	20073
Project Name:	2601 Chapman Ave	Date:	4/2/2020	Logged By:	LB
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Elevation:	~225 ft. MSL
Drill Hole Dia.:	8"	Drop:	30"	Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
		7	CL	107.2	18.8	Qyf		very stiff
		11						
61		18						
62								Total Depth = 61.5 feet Groundwater not encountered Backfilled with soil from cuttings and capped with AC cold patch
63								
64								
65								
66								
67								
68								
69								
70								
71								
72								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

'e A-3 (Sheet 6 of 6)

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-2

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	31.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
0								**ASPHALT CONCRETE / AGGREGATE BASE (~10 inches)**
1			SC			Af		**ARTIFICIAL FILL** CLAYEY SAND, brown, slightly moist
2								
3								
4								
5	R	4	CL	108.9	15.1	Qyf		**YOUNG ALLUVIAL FAN DEPOSITS** SANDY CLAY, dark brown, moist, firm, fine-grained sand
		5						
6		5						
7			ML					SANDY SILT, olive brown, very moist, firm, fine-grained sand
8								
9								
10	S	2	ML/SM			Qyf		interlayer of sandy silt and silty sand, loose
		2						
11		3						
12								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-4 (Sheet 1 of 3)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-2

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	31.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
13			CL					SANDY CLAY, brown, moist, stiff to very stiff
14								
15	R	4 7 12	CL	110.4	19.7	Qyf		dark brown, very moist, stiff
16								
17								
18								
19								
20	S	2 5 10	CL			Qyf		
21								
22								
23								
24								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-4 (Sheet 2 of 3)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-2

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	31.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
25	R	5	CL	102.8	25.5	Qyf		very moist, stiff
		6						
26		11						
27								
28								
29								
30	S	2	SC			Qyf		CLAYEY SAND, brown, moist, medium dense
		4						
31		9						
32								Total Depth = 31.5 feet Groundwater not encountered Backfilled with soil from cuttings and capped with AC cold patch
33								
34								
35								
36								

S - SPT Sample   R - Ring Sample   B - Bulk Sample   D - Disturbed Sample

Figure A-4 (Sheet 3 of 3)

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-3

				Project No.:	20073
Project Name:	2601 Chapman Ave	Date:	4/3/2020	Logged By:	LB
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Elevation:	~226 ft. MSL
Drill Hole Dia.:	8"	Drop:	30"	Depth of Boring (ft.):	26.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
0								**ASPHALT CONCRETE / AGGREGATE BASE (~10 inches)**
1			SC			Af		**ARTIFICIAL FILL** CLAYEY SAND, brown, slightly moist, loose
2								
3								
4								
5	S	1	SM			Qyf		**YOUNG ALLUVIAL FAN DEPOSITS** SILTY SAND, olive brown, slightly moist, trace clay, very loose to loose
		1						
6		2						
7								
8								
9								
10	R	5	SP-SM	93.3	4.3	Qyf		POORLY GRADED SAND WITH SILT, light brown, damp, fine- to-coarse-grained sand, loose
		6						
11		8						
12								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-5 (Sheet 1 of 3)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-3

				Project No.:	20073
Project Name:	2601 Chapman Ave	Date:	4/3/2020	Logged By:	LB
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Elevation:	~225 ft. MSL
Drill Hole Dia.:	8"	Drop:	30"	Depth of Boring (ft.):	26.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
13								
14								
15	S	1	CL			Qyf		SANDY CLAY, dark brown, very moist, very soft to soft
		1						
16		1						
17								
18								
19								
20	R	5	CL	107.6	20.0	Qyf		stiff
		8						
21		11						
22								
23								
24								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-5 (Sheet 2 of 3)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-3

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	26.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
25	S	4	CL			Qyf		stiff to very stiff
		5						
26		10						
27								Total Depth = 26.5 feet Groundwater not encountered Backfilled with soil from cuttings and capped with AC cold patch
28								
29								
30								
31								
32								
33								
34								
35								
36								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-5 (Sheet 3 of 3)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-4

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	31.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
								ASPHALT CONCRETE / AGGREGATE BASE (~10 inches)
1			SM			Af		**ARTIFICIAL FILL** SILTY SAND, brown, slightly moist, loose
2			SC			Qyf		**YOUNG ALLUVIAL FAN DEPOSITS** CLAYEY SAND, brown, slightly moist
3								
4								
5	R	2 2 5	CL	104.3	13.9	Qyf		SANDY CLAY, dark brown, moist, firm, fine-grained sand
6								
7								
8			SP-SM			Qyf		POORLY GRADED SAND WITH SILT, olive brown, slightly moist, loose fine- to- medium coarse-grained sand
9								
10	S	3 4 5	SP-SM			Qyf		loose
11								
12								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-6 (Sheet 1 of 3)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-4

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	31.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
13								
14								
15	R	6	ML	108.4	15.9	Qyf		SANDY SILT, olive brown, moist, stiff
		6						
16		14						
17								
18								
19								
20	S	4	CL			Qyf		SANDY CLAY, brown, moist, very stiff
		6						
21		10						
22								
23								
24								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

'e A-6 (Sheet 2 of 3)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-4

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	31.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
25	R	8	CL	106.7	13.9	Qyf		very stiff
		12						
26		14						
27								
28								
29								
30	S	8						
		8	SC			Qyf		CLAYEY SAND, brown, moist, medium dense
31		11						
32								Total Depth = 31.5 feet Groundwater not encountered Backfilled with soil from cuttings and capped with AC cold patch
33								
34								
35								
36								

S - SPT Sample   R - Ring Sample   B - Bulk Sample   D - Disturbed Sample

Figure A-6 (Sheet 3 of 3)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-5

Project Name:	2601 Chapman Ave	Date:	4/2/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
0								**ASPHALT CONCRETE / AGGREGATE BASE (~12 inches)**
1			SC			Qyf		**ARTIFICIAL FILL** CLAYEY SAND, brown, slightly moist, loose to medium dense
5	S	2 3 3	CL			Qyf		**YOUNG ALLUVIAL FAN DEPOSITS** SANDY CLAY, dark brown, moist, soft to firm, fine-grained sand
8								POORLY GRADED SAND WITH SILT, olive brown, slightly moist, medium dense, fine- to- medium coarse-grained sand
10	R	4 7 9	SP-SM	93.4	5.9	Qal		loose to medium dense
12								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-7 (Sheet 1 of 6)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-5

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
13								
14								
15	S	1	ML			Qyf		SANDY SILT, olive brown, very moist, soft, fine-grained sand
		1						
16		2	CL			Qyf		SANDY CLAY, brown, moist, stiff
17								
18								
19								
20	R	4						
		6						
21		10	CL	115.3	16.0	Qyf		
22								
23								
24								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-7 (Sheet 2 of 6)

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-5

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
25	S	3	CL			Qyf		very moist, stiff to very stiff
		7						
26		10						
27								
28								
29								
30	R	10						
		13	SC	115.0	9.2	Qyf		CLAYEY SAND, brown, moist, medium dense
31		21						
32								
33								
34								
35	S	3	CL			Qyf		SANDY CLAY, brown, moist, stiff
		4						
36		8						

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-7 (Sheet 3 of 6)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-5

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
37								
38								
39								
40	R	5	CL	98.3	28.0	Qyf		increase in sand, very moist, stiff
		7						
41		12						
42								
43								
44								
45	S	4	CL			Qyf		very moist, stiff
		4						
46		6						
47								
48								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-7 (Sheet 4 of 6)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-5

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
49								
50	R	7	CL	105.4	22.5	Qyf		
		14						olive brown, moist, very stiff, fine- to- coarse-grained sand
51		22						
52								
53								
54								
55	S	4	CL			Qyf		stiff
		8						
56		14						
57								
58								
59								
60								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-7 (Sheet 5 of 6)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-5

Project Name:	2601 Chapman Ave	Date:	4/3/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	LB
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	61.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
60	R	50/6"	SP	98.2	2.6	Qyf		POORLY GRADED SAND, light brown, dry, very dense coarse-grained sand
61								
62								Total Depth = 61.5 feet Groundwater not encountered Backfilled with soil from cuttings and capped with AC cold patch
63								
64								
65								
66								
67								
68								
69								
70								
71								

S - SPT Sample   R - Ring Sample   B - Bulk Sample   D - Disturbed Sample

Figure A-7 (Sheet 6 of 6)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-6

				Project No.:	20073
Project Name:	2601 Chapman Ave	Date:	8/25/2020	Logged By:	RA
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Elevation:	~225 ft. MSL
Drill Hole Dia.:	8"	Drop:	30"	Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
								ASPHALT CONCRETE
			SM			Qaf		**ARTIFICIAL FILL** SILTY SAND, brown, moist, some gravel
1								
2								
	R	4	SM	106.7	7.1	Qaf		loose
3		4						
		5						
4								
5	S	3	SP-SM			Qyf		**YOUNG ALLUVIAL FAN DEPOSITS** POORLY GRADED SAND WITL SILT, light brown, slightly moist, fine- to- coarse-grained sand, loose
		3						
6		3						
7								
	R	7	SP-SM	112.5	8.2	Qyf		loose to medium dense
8		7						
		10						
9								
10	S	2	SP-SM			Qyf		loose
		2						
11		4						
12								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-8 (Sheet 1 of 5)

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SUBSURFACE EXPLORATION LOG
BORING NO. B-6

Project Name:	2601 Chapman Ave	Date:	8/25/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	RA
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
13								
14								
15	R	7	SP-SM	101.7	1.8	Qyf		dry, medium dense
		10						
16		13	CL			Qyf		SANDY CLAY, red brown, moist, fine-grained, stiff
17								
18								
19								
	S	3	CL			Qyf		stiff
20		4						
		5						
21								
22								
23								
24								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-8 (Sheet 2 of 5)

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-6

Project Name:	2601 Chapman Ave	Date:	8/25/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	RA
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
25	R	7	CL	113.6	14.0	Qyf		very stiff to hard
		14						
26		36						
27								
28								
29								
30	S	7	CL			Qyf		very stiff
		8						
31		18						
32								
33			SC			Qyf		CLAYEY SAND, brown, moist
34								
35	R	7	SC	114.3	4.6	Qyf		dry, medium dense
		14						
36		27						

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-8 (Sheet 3 of 5)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-6

Project Name:	2601 Chapman Ave	Date:	8/25/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	RA
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
37								
38								
39								
40	S	5	CL			Qyf		SANDY CLAY, brown, moist, fine-grained sand, very stiff
		6						
41		10	SC			Qyf		CLAYEY SAND, brown, moist, medium dense to dense
42								
43								
44								
45	R	8						
		14	CL	105.5	13.6	Qyf		very stiff
46		17						
47								
48								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-8 (Sheet 4 of 5)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-6

Project Name:	2601 Chapman Ave	Date:	8/25/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	RA
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
49								
50	S	4	CL			Qy		stiff
		6						
51		7						
52								Total Depth = 51.5 feet Groundwater not encountered Backfilled with soil from cuttings
53								
54								
55								
56								
57								
58								
59								
60								

S - SPT Sample　R - Ring Sample　B - Bulk Sample　D - Disturbed Sample

Figure A-8 (Sheet 5 of 5)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-7

Project Name:	2601 Chapman Ave	Date:	8/25/2020
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs
Drill Hole Dia.:	8"	Drop:	30"

Project No.:	20073
Logged By:	RA
Elevation:	~225 ft. MSL
Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
								ASPHALT CONCRETE
								ARTIFICIAL FILL
1			SC/CL			Qaf		SANDY CLAY/CLAYEY SAND, brown, moist
2								
	S	1	SC/CL					very loose
3		1						
		1						
4								**YOUNG ALLUVIAL FAN DEPOSITS**
								SILTY SAND, brown to dark brown, moist
5	R	2	SM	106.8	9.0	Qyf		loose
		5						
6		5						
7								
	S	3	SM			Qyf		loose
8		3						
		5						
9								
10	R	8	SM	102.2	4.7	Qyf		dry, loose to medium dense
		8						
11		14						
12								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-9 (Sheet 1 of 5)

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-7

				Project No.:	20073
Project Name:	2601 Chapman Ave	Date:	8/25/2020	Logged By:	RA
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Elevation:	~225 ft. MSL
Drill Hole Dia.:	8"	Drop:	30"	Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
	S	2						very loose
13		1						
		2						
14								
15	R	5	SM	117.8	11.3	Qyf		
		10						
16		18	CL			Qyf		SANDY CLAY, brown, moist
17								
18								
19								
20	S	8	CL			Qyf		very stiff
		7						
21		12						
22								
23								
24								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-9 (Sheet 2 of 5)

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-7

				Project No.:	20073
Project Name:	2601 Chapman Ave	Date:	8/25/2020	Logged By:	RA
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Elevation:	~225 ft. MSL
Drill Hole Dia.:	8"	Drop:	30"	Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
25	R	7	CL	124.7	10.3	Qyf		slightly moist, hard
		14						
26		36						
27								
28								
29								
30	S	7	CL			Qyf		very stiff
		8						
31		18						
32								
33								
34								
35	R	7	CL	126.9	9.1	Qyf		very stiff
		14						
36		27						

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-9 (Sheet 3 of 5)

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NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-7

Project Name:	2601 Chapman Ave	Date:	8/25/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	RA
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
			SC			Qy		CLAYEY SAND, brown, moist
37								
38								
39								
40	S	5	SC			Qyf		medium dense
		6						
41		10						
42								
43								
44								
45	R	8	CL	103.4	4.4	Qyf		SANDY CLAY, brown, dry, fine-grained, very stiff
		14						
46		17						
47								
48								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-9 (Sheet 4 of 5)

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. B-7

Project Name:	2601 Chapman Ave	Date:	8/25/2020	Project No.:	20073
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs	Logged By:	RA
Drill Hole Dia.:	8"	Drop:	30"	Elevation:	~225 ft. MSL
				Depth of Boring (ft.):	51.5

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
49								
50	S	4	CL			Qyf		stiff
		6						
51		7						
52								Total Depth = 51.5 feet Groundwater not encountered Backfill with soil from cuttings
53								
54								
55								
56								
57								
58								
59								
60								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-9 (Sheet 5 of 5)

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. P-1

Project Name:	2601 Chapman Ave	Project No.:	20073
Date:	4/2/2020	Logged By:	LB
Type of Rig:	Hollow-Stem-Auger	Elevation:	~225 ft. MSL
Drive Wt.:	140 lbs		
Drill Hole Dia.:	8"	Depth of Boring (ft.):	10
Drop:	30"		

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
0								**ASPHALT CONCRETE / AGGREGATE BASE (~10 inches)**
1			SM			Af		**ARTIFICIAL FILL** SILTY SAND, brown, damp to moist, fine-grained sand, loose
2								
3			CL			Qyf		**YOUNG ALLUVIAL FAN DEPOSITS** SANDY CLAY, dark brown, moist, fine-grained sand
4								
5								
6			SM			Qyf		SILTY SAND, light brown, damp to moist, loose
7								
8								
9								
9.5			ML			Qyf		SANDY SILT, light brown, moist, stiff, fine-grained sand, some clay
10								Total Depth = 10 feet Groundwater not encountered Backfilled with soil from cuttings and capped with AC cold patch
11								
12								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-10 (Sheet 1 of 1)

NTS GEOTECHNICAL

SUBSURFACE EXPLORATION LOG
BORING NO. P-2

Project Name:	2601 Chapman Ave	Date:	4/2/2020
Type of Rig:	Hollow-Stem-Auger	Drive Wt.:	140 lbs
Drill Hole Dia.:	8"	Drop:	30"

Project No.:	20073
Logged By:	LB
Elevation:	~225 ft. MSL
Depth of Boring (ft.):	10

Depth (ft.)	Sample Type	No. of Blows per 6"	Soil Classification	Dry Density (lb/ft3)	Moisture Content (%)	Lithology	Groundwater	Description
0								**TOPSOIL**
1			SC			Qyf		CLAYEY SAND, brown, slightly moist, loose to medium dense
2								
3								
4								
5			CL			Qyf		**YOUNG ALLUVIAL FAN DEPOSITS** SANDY CLAY, dark brown, moist, soft to firm, fine-grained sand
6								
7								
8								
9								
10								
11								Total Depth = 10 feet Groundwater not encountered Backfilled with soil from cuttings and capped with AC cold patch
12								

S - SPT Sample R - Ring Sample B - Bulk Sample D - Disturbed Sample

Figure A-11 (Sheet 1 of 1)

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APPENDIX A-1

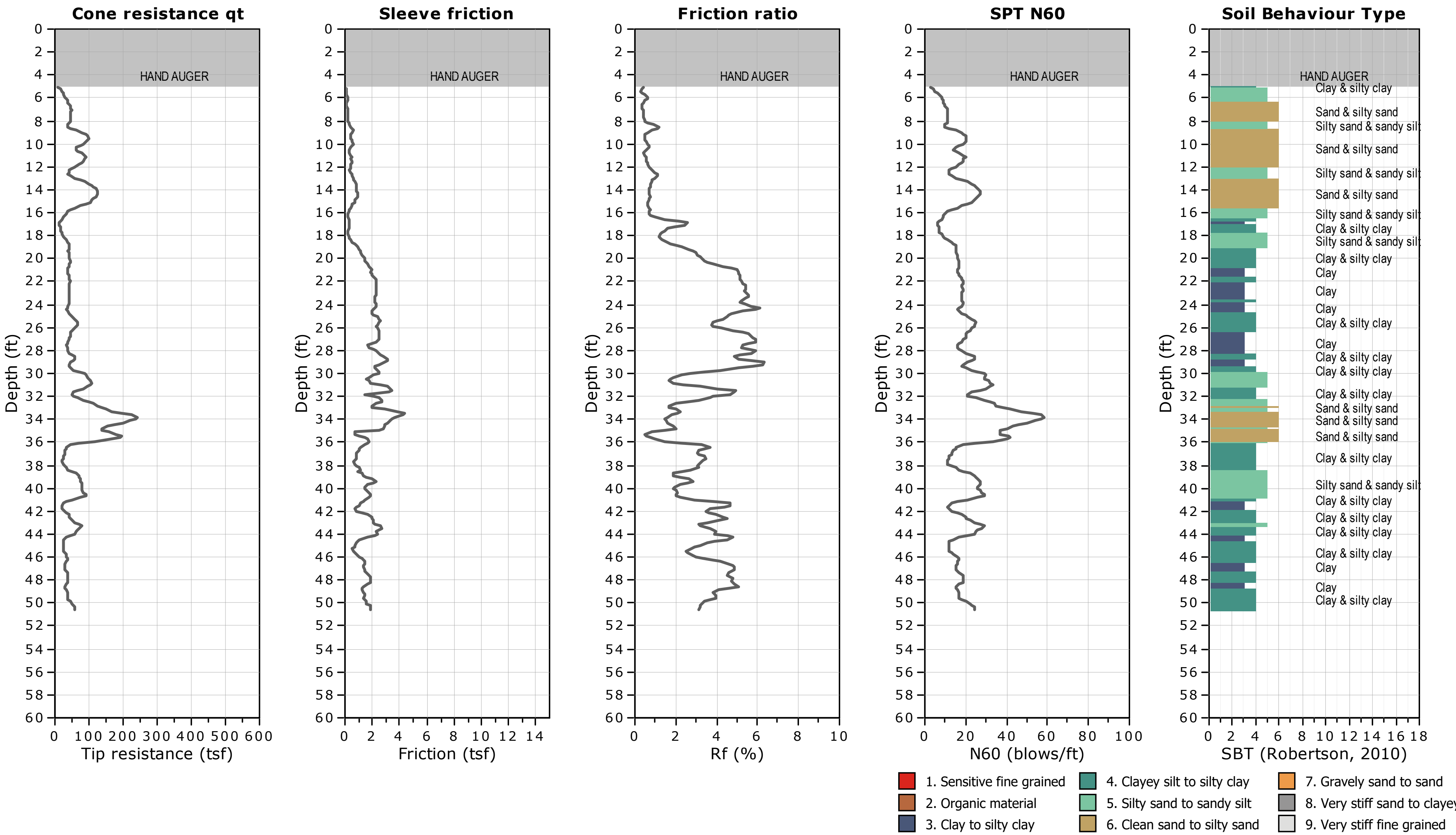
Cone Penetration Testing Logs

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-1

CLIENT: NTS GEOTECHNICAL
SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM
Total depth: 50.52 ft, Date: 8/25/2020

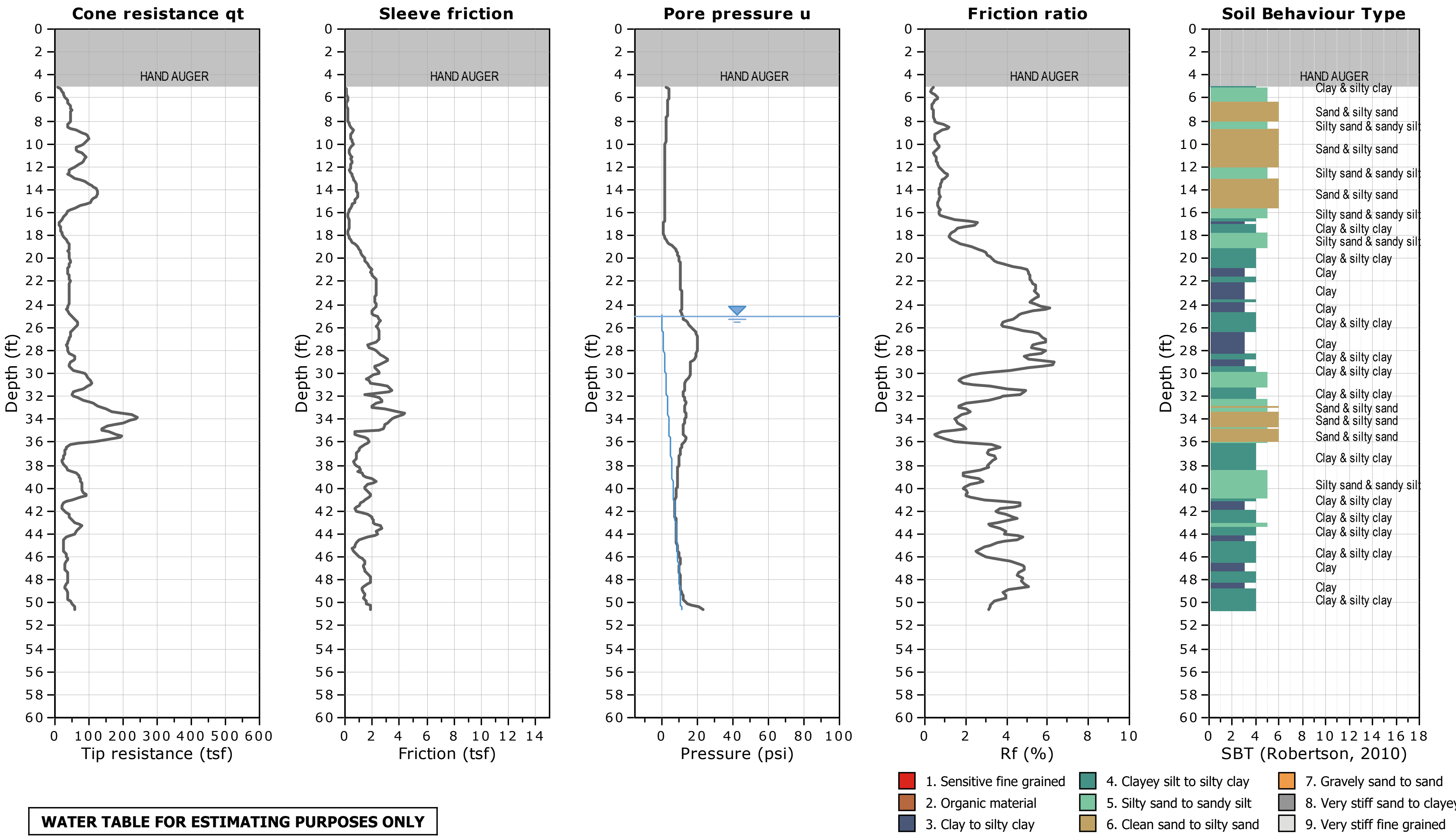


GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-1

CLIENT: NTS GEOTECHNICAL
SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM
Total depth: 50.52 ft, Date: 8/25/2020



WATER TABLE FOR ESTIMATING PURPOSES ONLY

CPeT-IT v.19.0.1.24 - CPTU data presentation & interpretation software - Report created on: 8/26/2020, 11:39:30 AM
Project file: C:\CPT-2020\205084SH\REPORT\205084sh.cpt

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

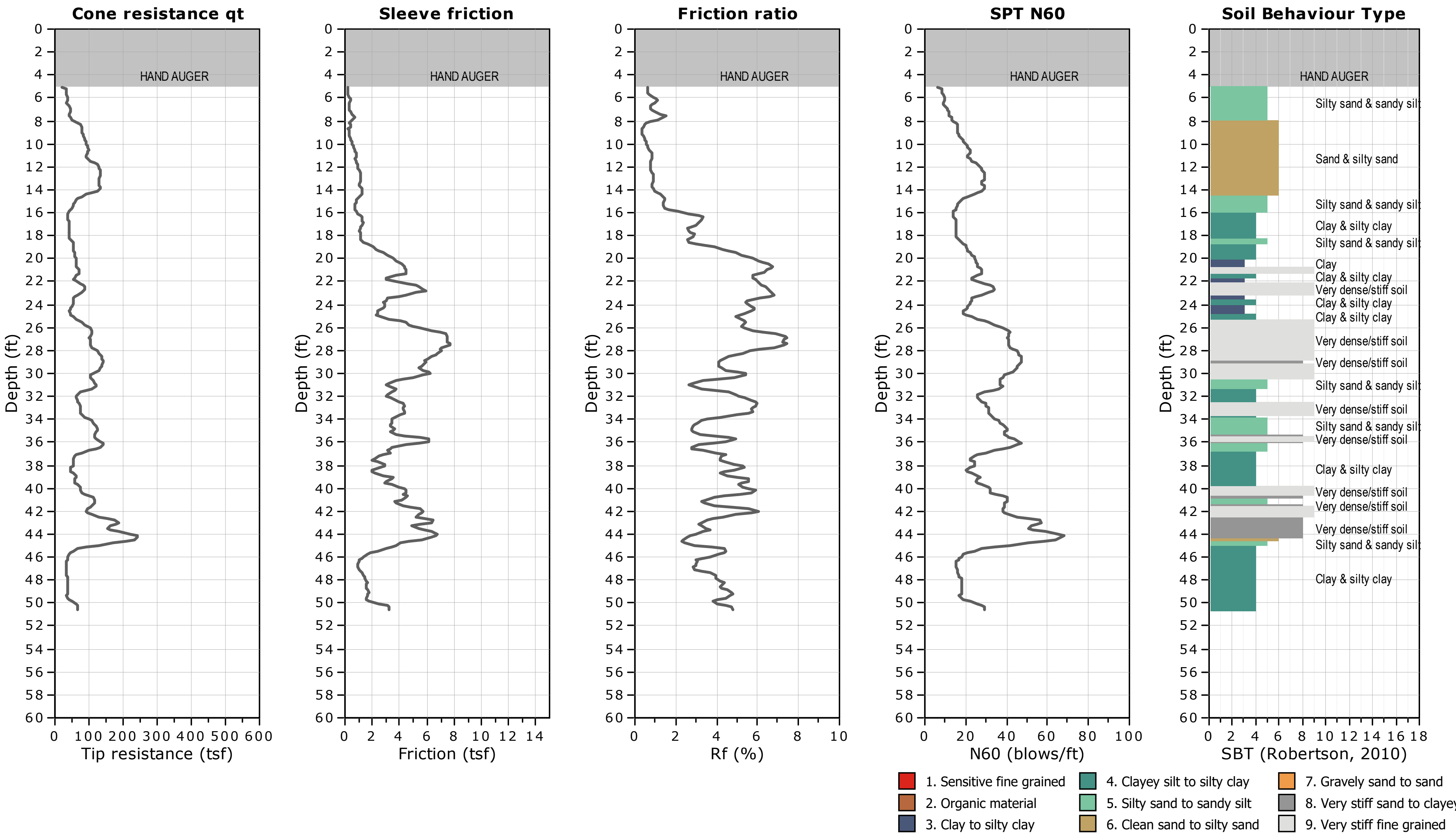
CPT: CPT-2

CLIENT: NTS GEOTECHNICAL

SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM

Total depth: 50.52 ft, Date: 8/25/2020



CPeT-IT v.19.0.1.24 - CPTU data presentation & interpretation software - Report created on: 8/26/2020, 11:39:30 AM

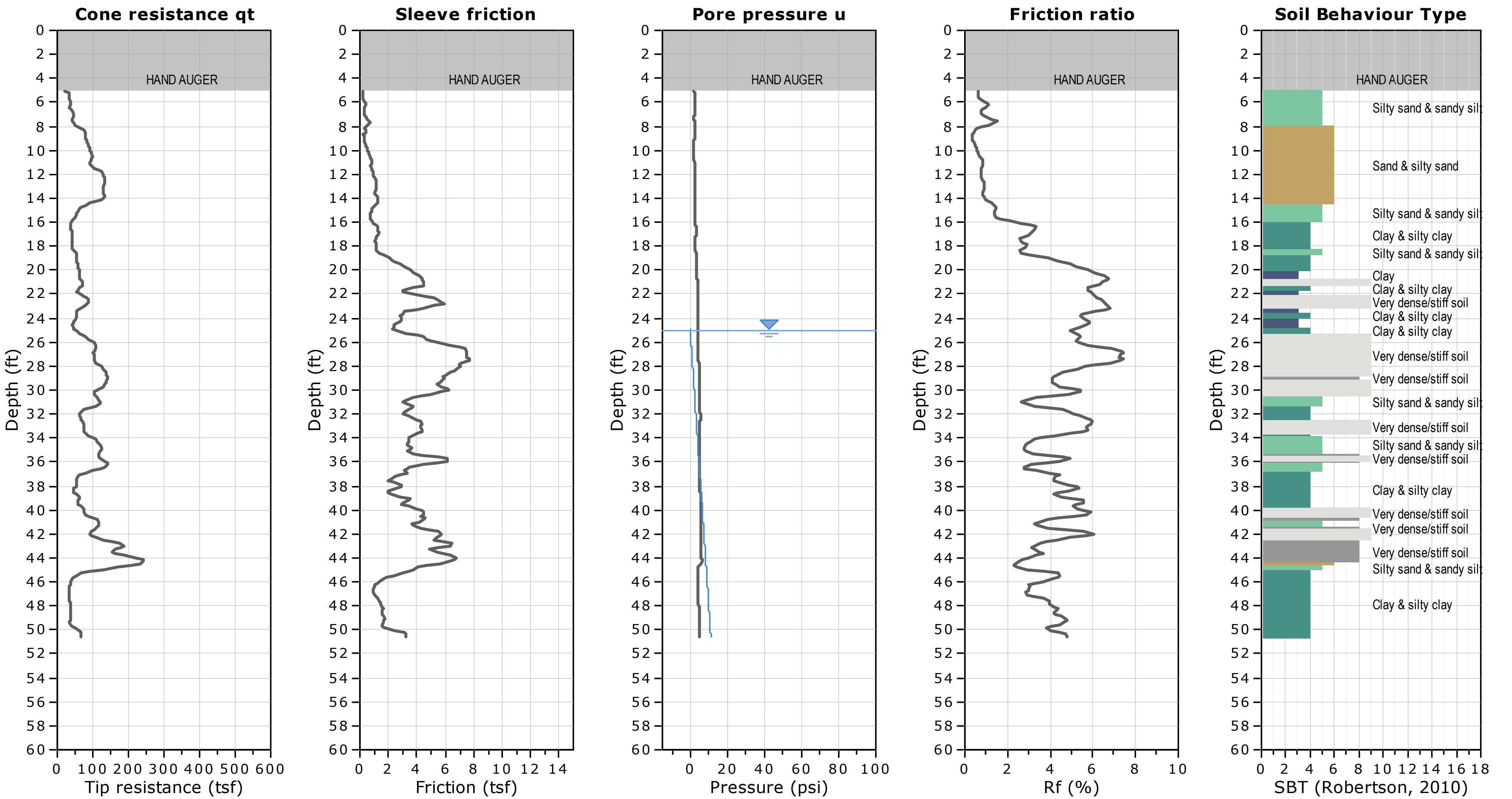
Project file: C:\CPT-2020\205084SH\REPORT\205084sh.cpt

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-2

CLIENT: NTS GEOTECHNICAL
SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM
Total depth: 50.52 ft, Date: 8/25/2020



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

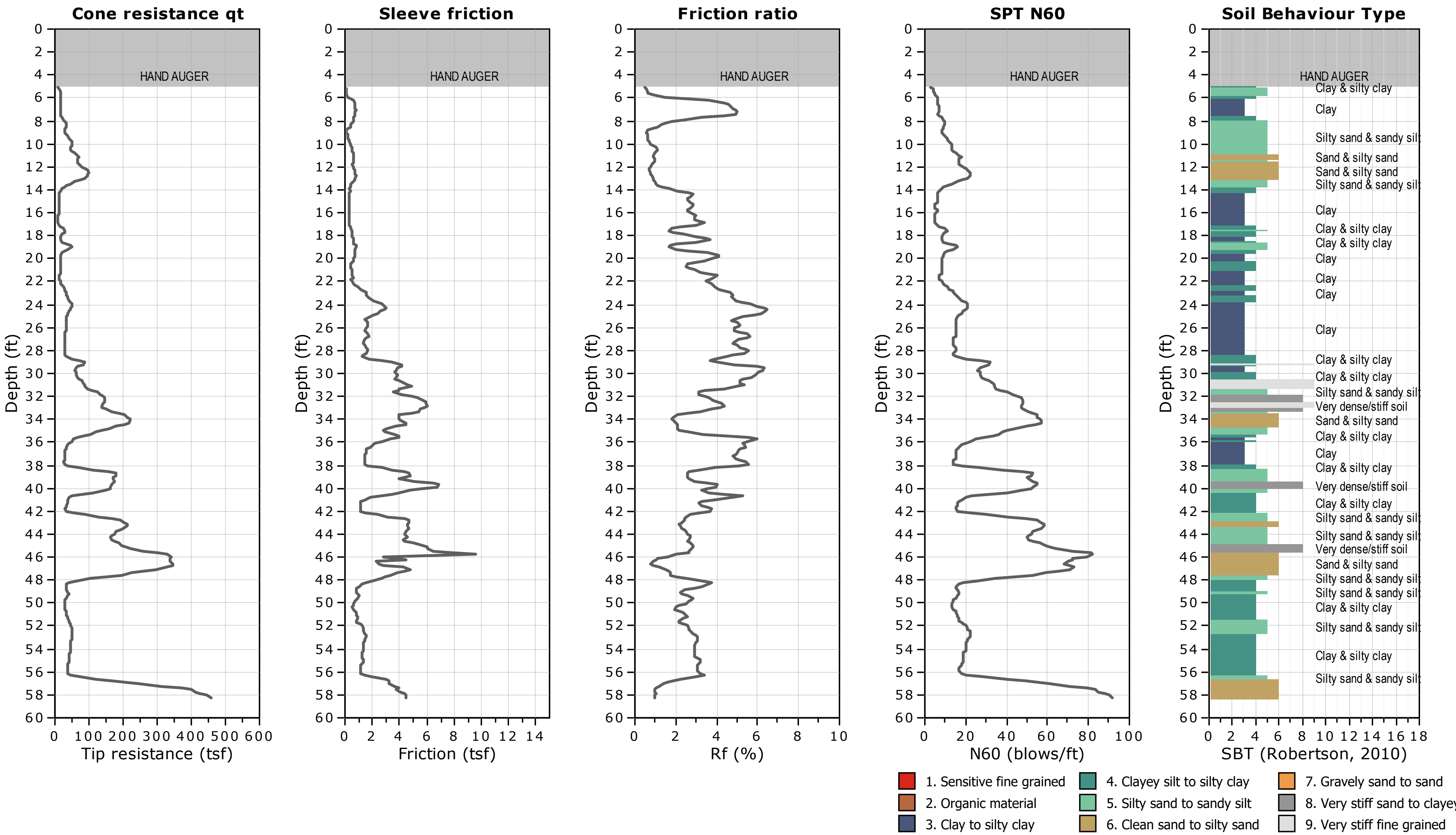
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-4

CLIENT: NTS GEOTECHNICAL
SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM
Total depth: 58.23 ft, Date: 8/25/2020

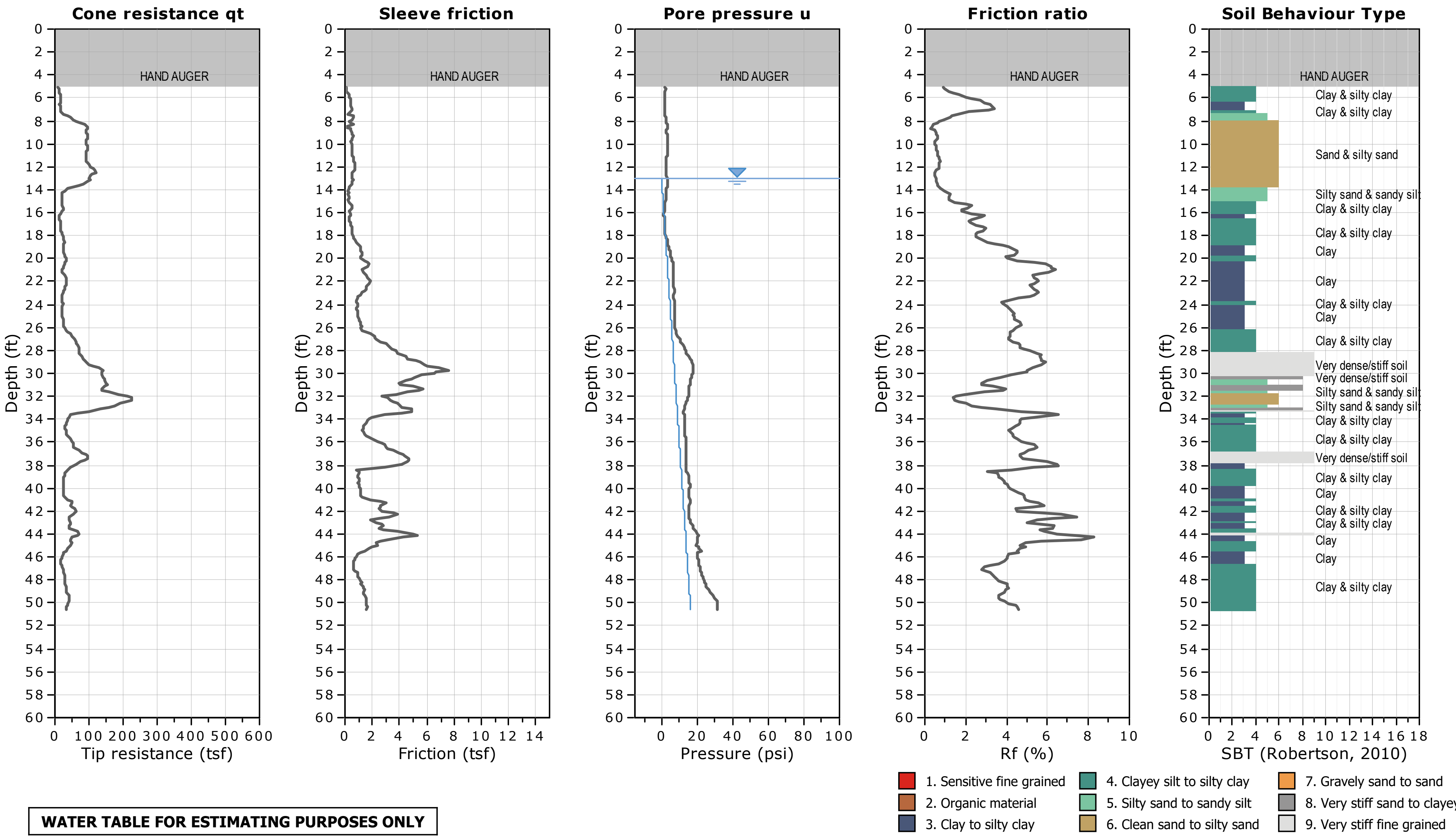


GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: SCPT-3

CLIENT: NTS GEOTECHNICAL
SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM
Total depth: 50.52 ft, Date: 8/25/2020



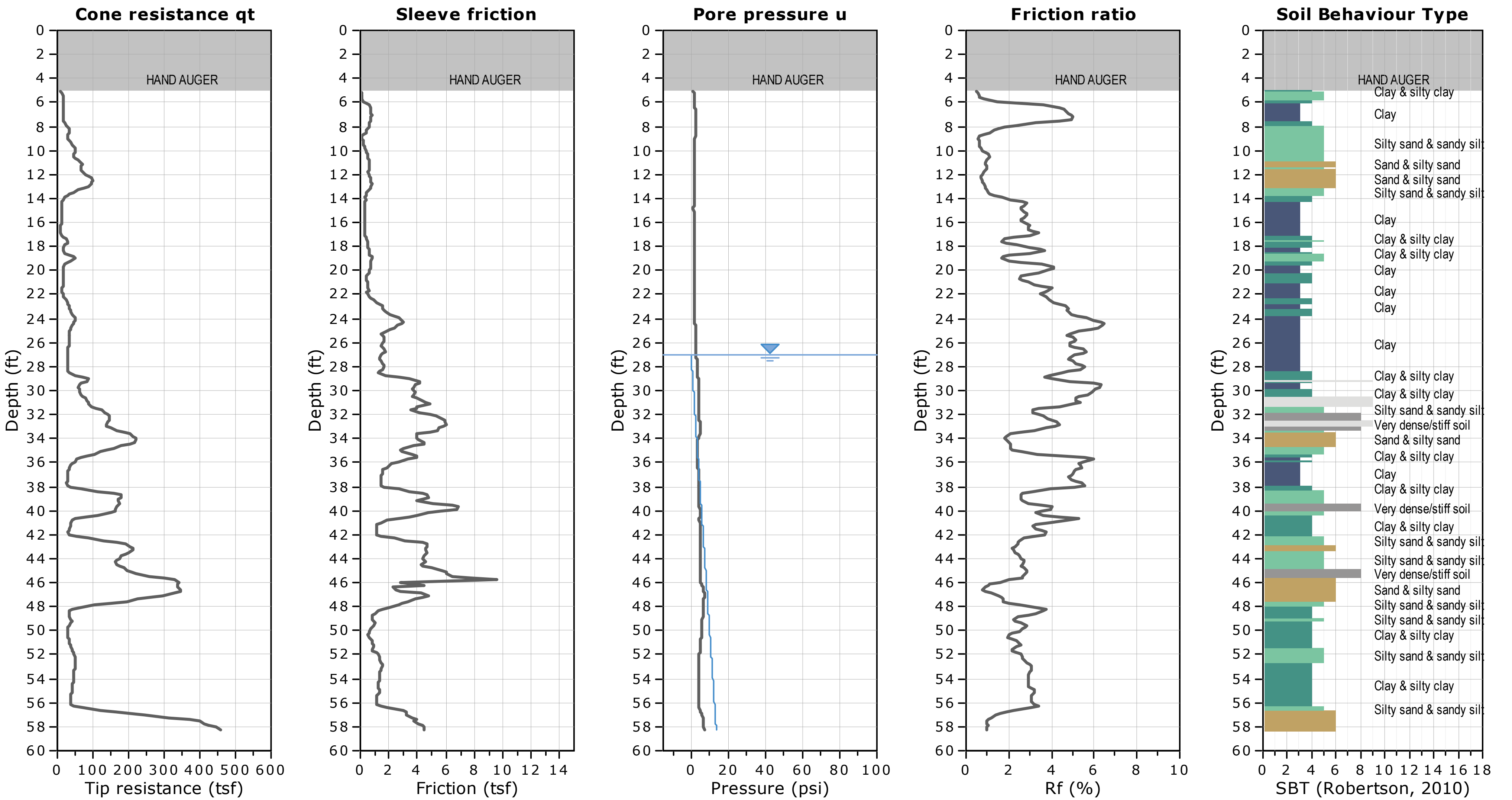
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-4

CLIENT: NTS GEOTECHNICAL
SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM
Total depth: 58.23 ft, Date: 8/25/2020



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

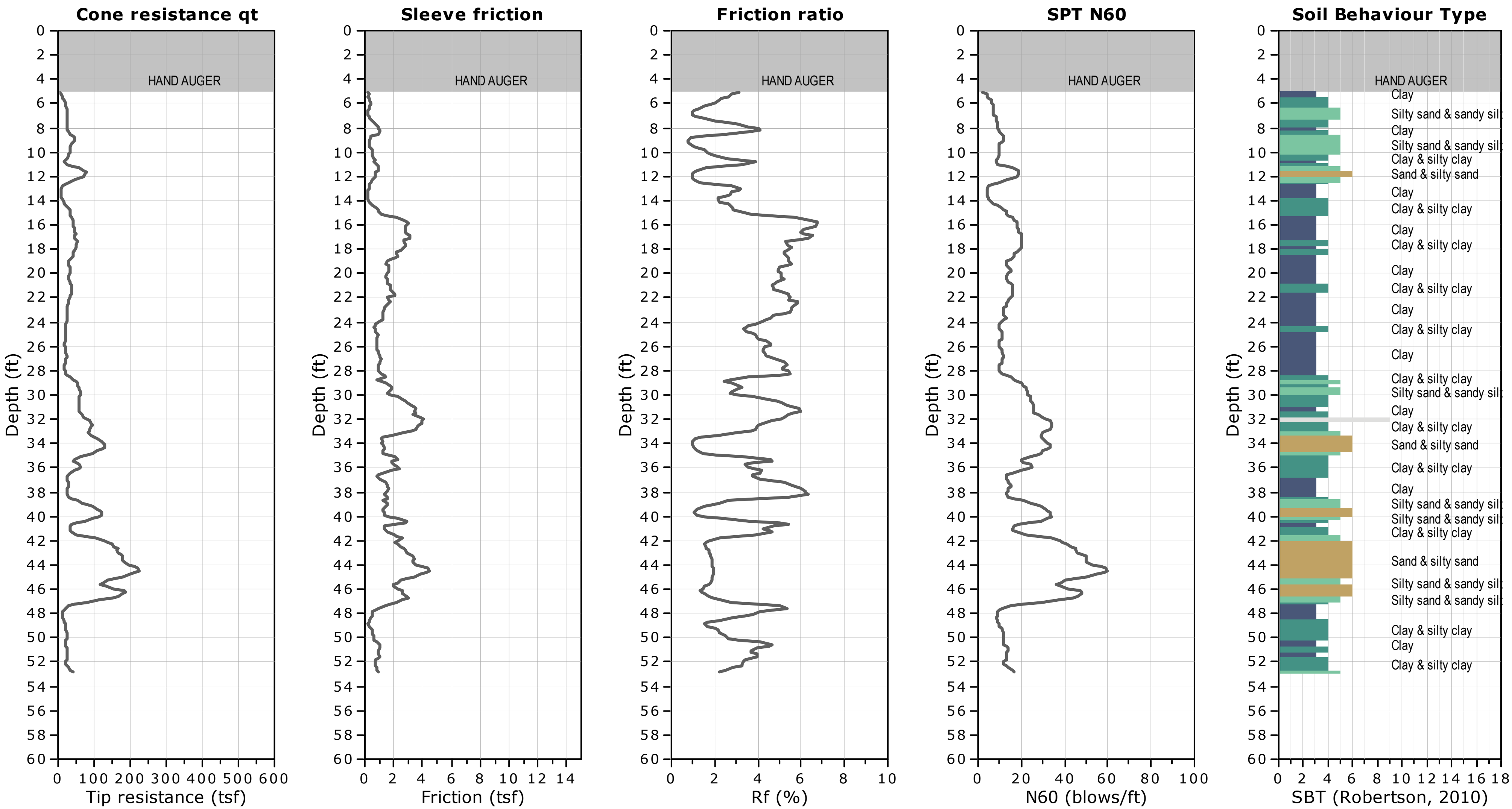
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-5

CLIENT: NTS GEOTECHNICAL
SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM
Total depth: 52.82 ft, Date: 8/25/2020



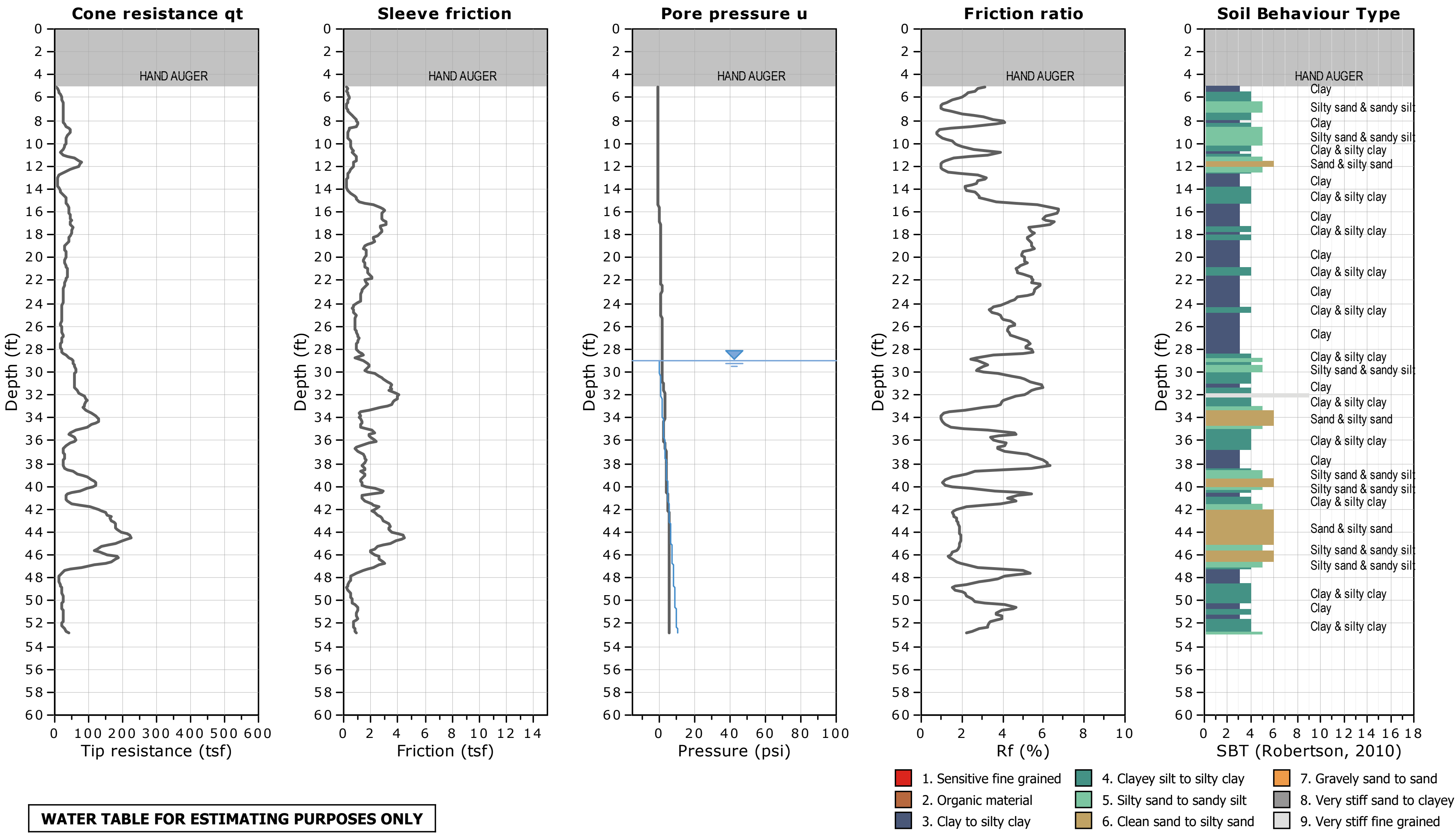
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey
9. Very stiff fine grained

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: CPT-5

CLIENT: NTS GEOTECHNICAL
SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM
Total depth: 52.82 ft, Date: 8/25/2020



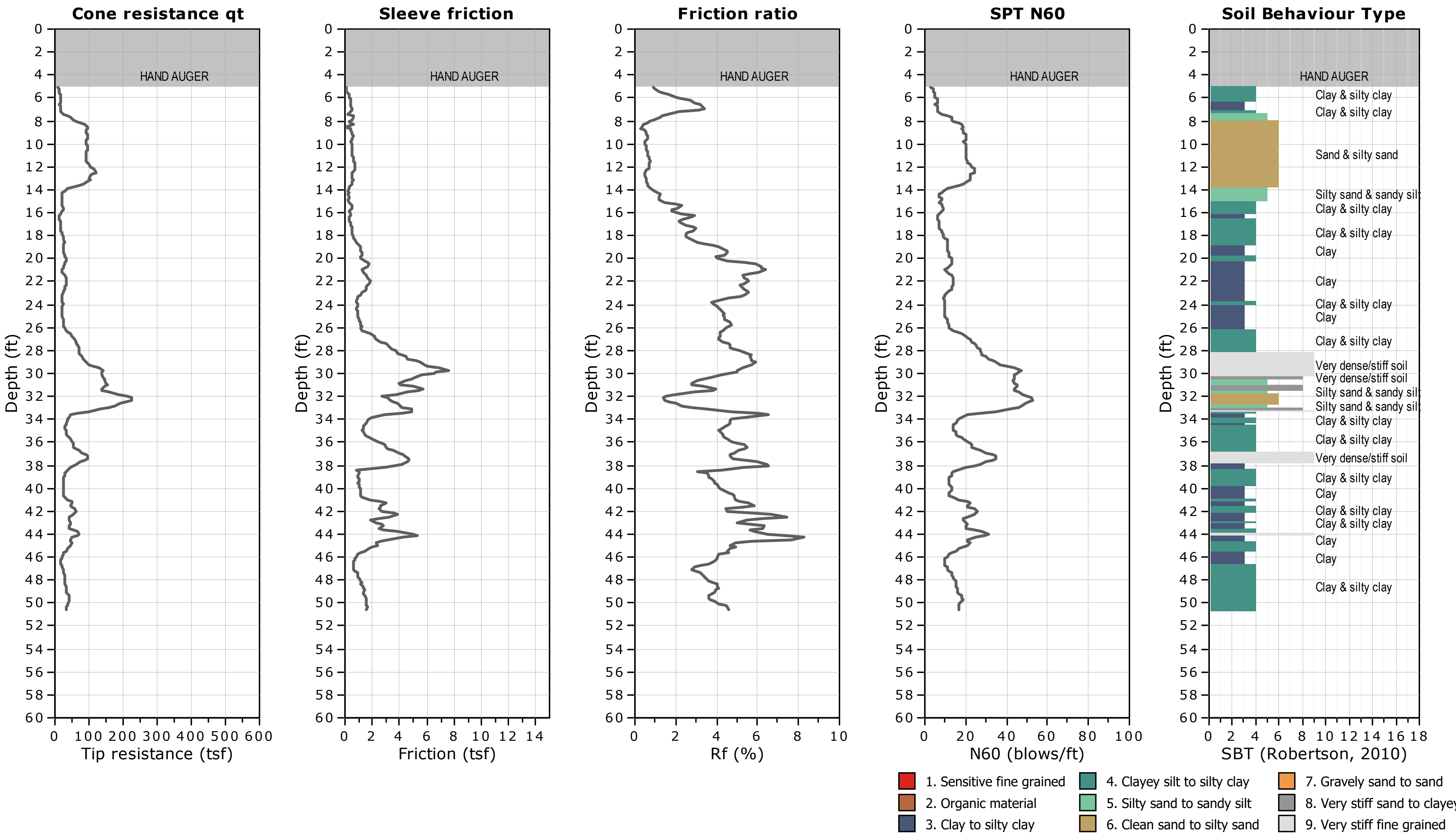
WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, LLC
WWW.GREGGDRILLING.COM

CPT: SCPT-3

CLIENT: NTS GEOTECHNICAL
SITE: HUB @ FULLERTON, CA

FIELD REP: NADIM
Total depth: 50.52 ft, Date: 8/25/2020



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NTS GEOTECHNICAL

APPENDIX B

Geotechnical Laboratory Testing

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NTS GEOTECHNICAL

Appendix B
Geotechnical Laboratory Testing

Laboratory Moisture Content and Density Tests

The moisture content and dry densities of selected driven samples obtained from the exploratory borings were evaluated in general accordance with the latest version of ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A

Wash Sieve

The amount of fines passing the No. 200 sieve was evaluated by the wash sieve. The test procedure was in general accordance with ASTM D 1140. The results are presented in the table below:

Boring No.	Depth	Fines Passing No. 200 Sieve
B-1	5.0	53.0
B-1	15.0	54.9
B-1	25.0	49.1
B-1	35.0	25.2
B-1	45.0	30.2
B-1	55.0	82.4
B-2	10.0	77.1
B-2	20.0	69.2
B-2	30.0	37.6
B-3	5.0	41.3
B-3	15.0	26.8
B-3	25.0	21.9
B-4	10.0	5.9
B-4	20.0	55.4
B-4	30.0	49.3
B-5	5.0	55.3
B-5	15.0	59.2
B-5	25.0	74.5
B-5	35.0	55.8
B-5	45.0	87.8
B-5	55.0	23.8
B-6	15.0	6.2
B-6	35.0	14.6

B-6	45.0	31.1
B-7	15.0	31.2

Atterberg Limits

As part of the engineering classification of the soil material, some samples of the on-site soil material were tested to determine relative plasticity. This relative plasticity is based on the Atterberg limits determined in general accordance with ASTM Test Method D 4318. The results of these tests are summarized in the table below:

Boring No.	Depth	LL	PL	PI	USCS Classification
B-1	15.0	30	25	5	ML
B-2	20.0	36	16	20	CL
B-6	20.0	37	15	22	CL
B-7	15.0	32	13	19	CL

Direct Shear Tests

Direct shear tests were performed on selected remolded and relatively undisturbed soil samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the materials. The samples were inundated during shearing to represent adverse field conditions. Direct shear testing was performed by Hushmand Associates and NOVA Geotechnical, and the test results are attached to this Appendix B.

Consolidation Test

Consolidation tests was performed on a selected driven soil sample in general accordance with the latest version of ASTM D2435. The sample was inundated during testing to represent adverse field conditions. The percent consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. Consolidation testing was performed by Hushmand Associates and NOVA Geotechnical, and the test results are attached to this Appendix B.

Corrosion Suite

The corrosion potential of typical on-site materials under long-term contact with both metal and concrete was determined by chemical and electrical resistance tests. The soluble sulfate test for potential concrete corrosion was performed in general accordance with California Test Method 417, the minimum resistivity test for potential metal corrosion was performed in general accordance with California Test Method 643, and the concentration of soluble chlorides was determined in general accordance with

California Test Method 422. Test was performed by Anaheim Laboratory and test results are attached to this Appendix B.

R-Value Test

A bulk sample representative of the underlying on-site materials was tested to measure the response of a compacted sample to a vertically applied pressure under specific conditions. The R-value of a material is determined when the material is in a state of saturation such that water will be exuded from the compacted test specimen when a 16.8 kN load (2.07 MPa) is applied. The result of this test is presented in the table below.

Boring No.	Depth	R-Value
B-1	0.0 – 5.0	15

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

Hushmand Associates, Inc.
250 Goddard, Irvine,
CA 92618

p. (949) 777-1274
w. haieng.com
e. hai@haieng.com

May 5, 2020

NTS Geotechnical
15333 Culver Dr.,
Suite 340
Irvine, CA 92604

Attention: Mr. Lee Bainer

SUBJECT: Laboratory Test Result
Project Name: 2601 Chapman Ave. Fullerton -
Project No.: NTS 20073
HAI Project No.: TWI-20-005

Dear Mr. Bainer:

Enclosed is the result of the laboratory testing program conducted on samples from the above referenced project. The testing performed for this program was conducted in general accordance with the following test procedure:

Type of Test	Test Procedure
Direct Shear (Consolidated & Drained)	ASTM D3080
Consolidation	ASTM D2435

Attached are: two (2) 3-point Direct shear test results; and two (2) Consolidation test results.

We appreciate the opportunity to provide our testing services to Twining Inc. If you have any questions regarding the test results, please contact us.

Sincerely,

Kang C. Lin, BS, EIT
Laboratory Manager

Woongju (MJ) Mun, PhD
Senior Staff Engineer

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

DIRECT SHEAR TEST
ASTM D3080

HAI Project No.: TWI-20-005

Client: NTS Geotechnical
Project Name: 2601Chapman Ave. Fullerton
Project Number: -
Boring No.: B1
Sample No.: R
Sample Type: Undistured Tube
Depth (ft): 20
Soil Description: Brown, Sandy Clay (CL)

Tested by: KL
Checked by: MJ
Date: 4/27/2020

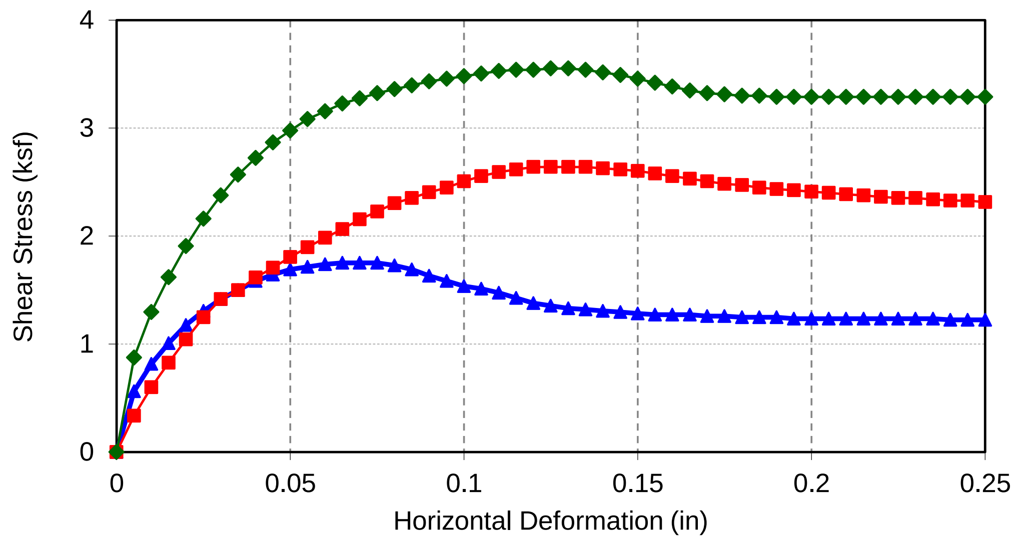
Type of test: **Consolidated, Drained**

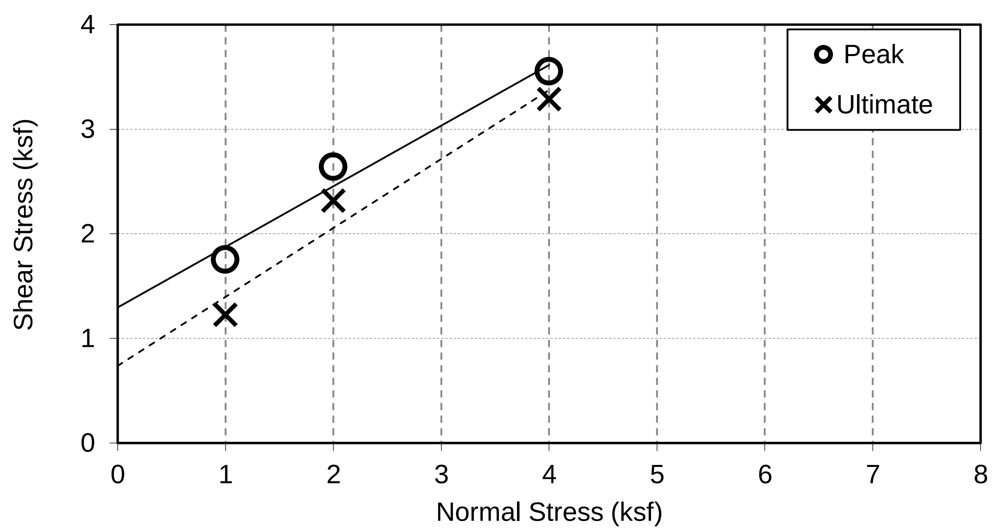
Test No.	1	2	3
Symbol	▲	■	◆
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	**O**	**1.75**	**2.64**	**3.55**
Shear Stress @ End of Test (ksf)	**X**	**1.22**	**2.32**	**3.29**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	0.9867	0.9765	0.9759
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**11.7**	**11.7**	**11.7**
Final Moisture Content (%)	**14.8**	**14.3**	**14.7**
Dry Density (pcf)	**118.8**	**119.9**	**118.6**

Strength Properties	Peak	Ultimate
Cohesion (psf)	**1300**	**740**
Friction Angle (degrees)	**30**	**33**





DIRECT SHEAR TEST

ASTM D3080

HUSHMAND ASSOCIATES, INC.
Geotechnical and Earthquake Engineers

HAI Project No.: TWI-20-005

Client: NTS Geotechnical

Project Name: 2601 Chapman Ave. Fullerton

Project Number: -

Boring No.: B3

Sample No.: R

Sample Type: Undistured Tube

Depth (ft): 10

Soil Description: Yellowish Brown, Poorly graded Sand With Silt (SP-SM)

Type of test: **Consolidated, Drained**

Tested by: KL

Checked by: MJ

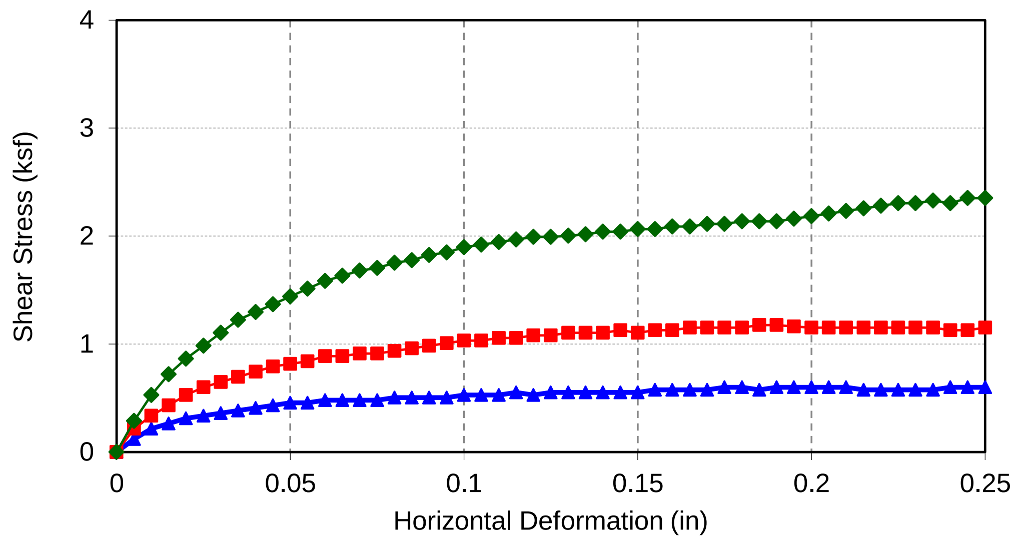
Date: 4/27/2020

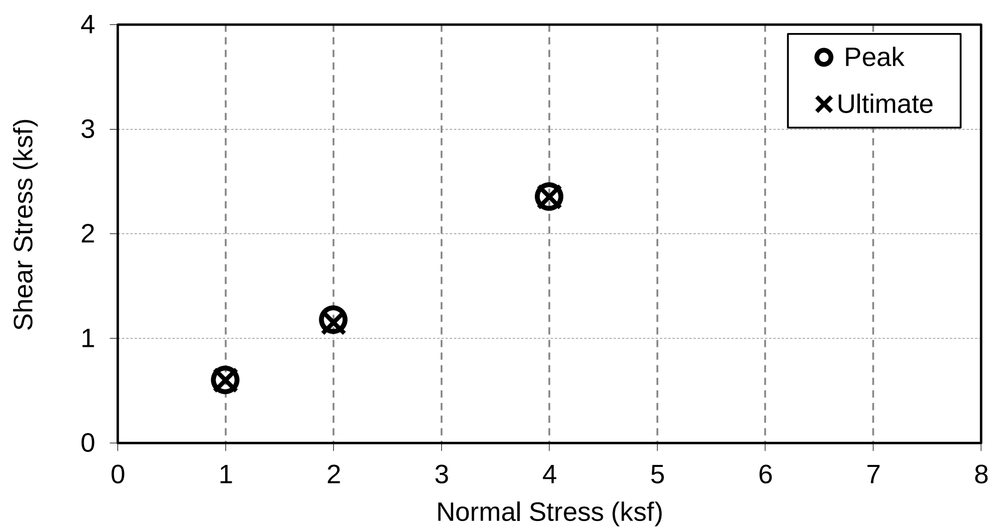
Test No.	1	2	3
Symbol	▲	■	◆
Normal Stress (ksf)	**1**	**2**	**4**
Deformation Rate (in/min)	**0.002**	**0.002**	**0.002**

Peak Shear Stress (ksf)	**O**	**0.60**	**1.18**	**2.35**
Shear Stress @ End of Test (ksf)	**X**	**0.60**	**1.15**	**2.35**

Initial Height of Sample (in)	1.000	1.000	1.000
Height of Sample before Shear (in)	0.9717	0.9644	0.9529
Diameter of Sample (in)	2.416	2.416	2.416
Initial Moisture Content (%)	**4.3**	**4.3**	**4.3**
Final Moisture Content (%)	**26.5**	**25.1**	**26.8**
Dry Density (pcf)	**79.8**	**82.6**	**84.3**

Strength Properties	Peak	Ultimate
Cohesion (psf)	**10**	**0**
Friction Angle (degrees)	**30**	**30**





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CONSOLIDATION TEST

ASTM D2435

Client :	NTS Geotechnical	*HAI Project No.:*	TWI-20-005
Project Name:	2601 Chapman Ave. Fullerton	*Tested by:*	KL
Project Number:	NTS 20073	*Checked by:*	MJ
Boring No.:	B1	*Date:*	04/27/20
Sample No.:	R		
Type of Sample:	Undisturbed Tube		
Depth (ft):	10		
Soil Description:	Light Brown, Sandy Silt with some clay (ML)		

Initial Total Weight	Final Total Weight	Final Dry Weight
(g)	(g)	(g)
111.47	131.42	98.25

			Initial Conditions	Final Conditions
Height	H	(in)	1.027	0.934
Height of Solids	H_s	(in)	0.490	0.490
Height of Water	H_w	(in)	0.176	0.442
Height of Air	H_a	(in)	0.361	0.002
Dry Density		**(pcf)**	**79.5**	**93.6**
Water Content		**(%)**	**13.5**	**33.8**
Saturation		**(%)**	**32.8**	**99.5**

* Saturation is calcualted based on Gs= 2.67

Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consol. (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0270	0.537	1.097	0			
0.25	0.0028	1.0242	0.534	1.091	0.3	2.4E-02	1.1E-02	
0.5	0.0097	1.0173	0.528	1.077	0.9	5.6E-02	2.7E-02	
0.5	0.0122	1.0148	0.525	1.072	1.2	Water Added		
1	0.0212	1.0058	0.516	1.053	2.1	3.7E-02	1.8E-02	
2	0.0439	0.9831	0.493	1.007	4.3	4.6E-02	2.3E-02	
4	0.0800	0.9470	0.457	0.933	7.8	3.7E-02	1.9E-02	
6	0.1009	0.9261	0.436	0.891	9.8	2.1E-02	1.1E-02	
4	0.1002	0.9268	0.437	0.892	9.8	Unloaded		
2	0.0975	0.9295	0.440	0.898	9.5			
1	0.0933	0.9337	0.444	0.906	9.1			

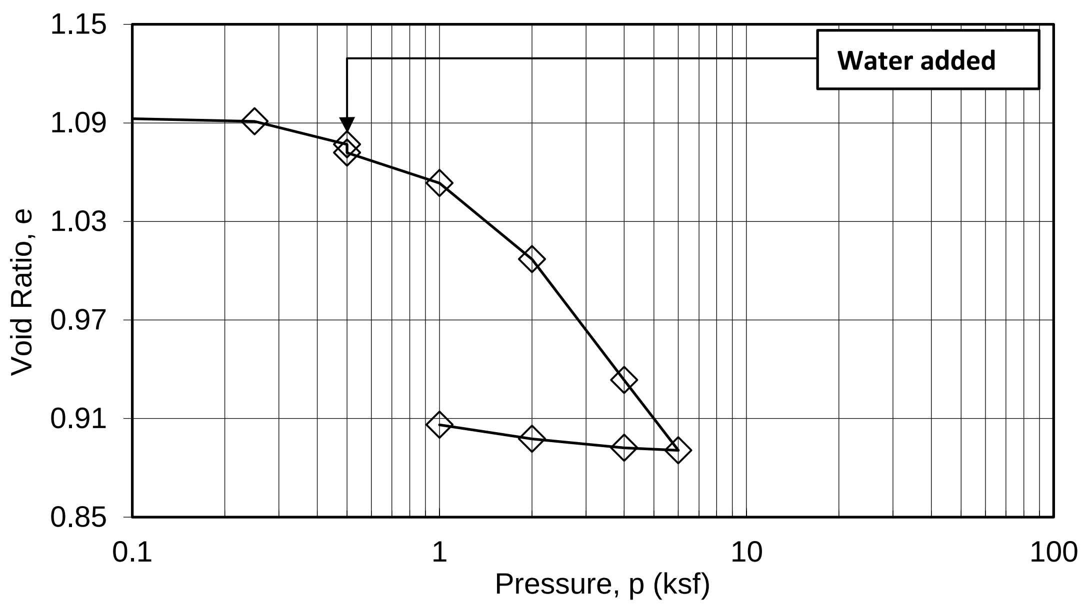
DRAFT

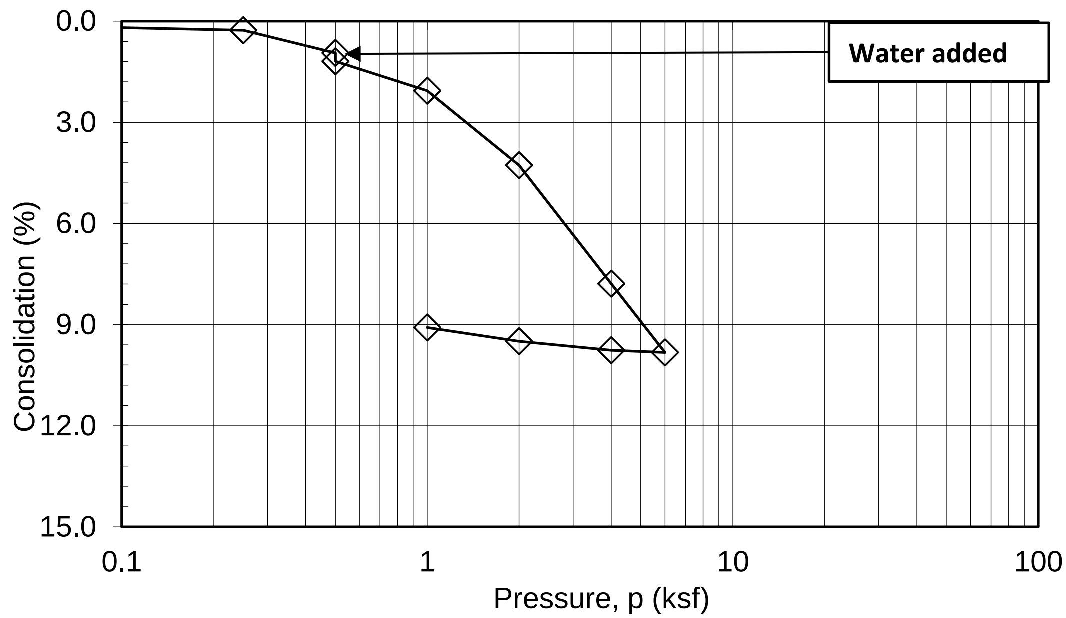
CONSOLIDATION TEST

ASTM D2435

Client : NTS Geotechnical
Project Name: 2601 Chapman Ave. Fullerton
Project Number: NTS 20073
Boring No.: B1
Sample No.: R
Type of Sample: Undisturbed Tube
Depth (ft): 10
Soil Description: Light Brown, Sandy Silt with some clay (CL)

HAI Project No.: TWI-20-005
Tested by: KL
Checked by: MJ
Date: 04/27/20





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CONSOLIDATION TEST

ASTM D2435

Client : NTS Geotechnical
Project Name: 2601 Chapman Ave. Fullerton NTS 20073
Project Number: B5
Boring No.: R
Sample No.: Undisturbed Tube
Type of Sample: 20
Depth (ft): Reddish Brown, Sandy Clay (CL)
Soil Description:

HAI Project No.: TWI-20-005
Tested by: KL
Checked by: MJ
Date: 04/27/20

Initial Total Weight	Final Total Weight	Final Dry Weight
(g)	(g)	(g)
163.92	164.98	141.80

			Initial Conditions	Final Conditions
Height	H	(in)	1.029	1.007
Height of Solids	H_s	(in)	0.697	0.697
Height of Water	H_w	(in)	0.294	0.309
Height of Air	H_a	(in)	0.038	0.002
Dry Density		**(pcf)**	**114.5**	**116.0**
Water Content		**(%)**	**15.6**	**16.3**
Saturation		**(%)**	**88.6**	**99.2**

* Saturation is calcualted based on Gs= 2.71

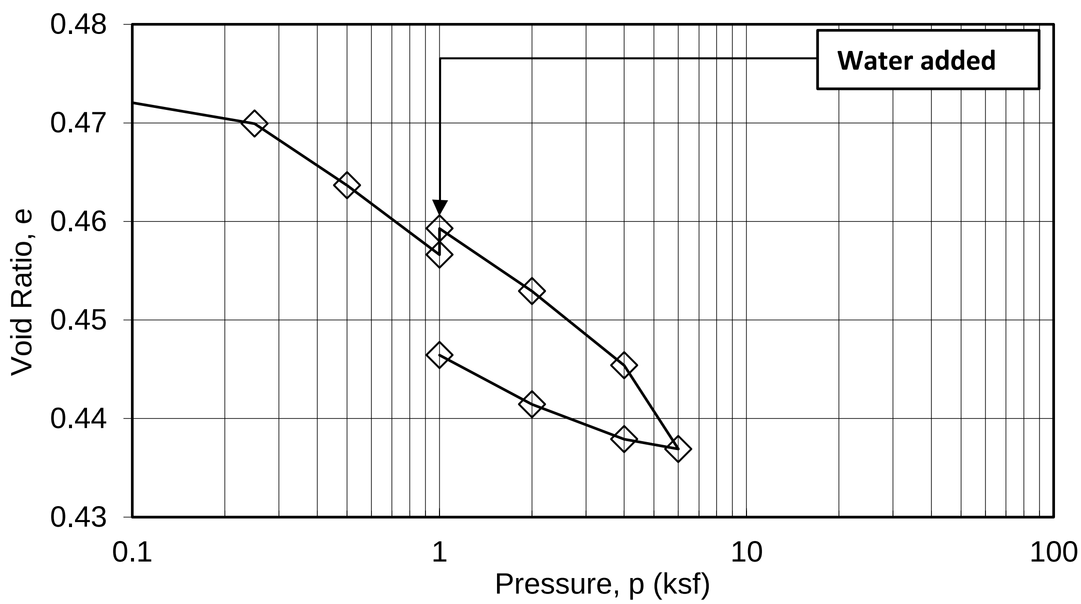
Load (ksf)	δH (in)	H (in)	Voids (in)	e	Consol. (%)	a_v (ksf^{-1})	M_v (ksf^{-1})	Comment
0.01	-------	1.0290	0.332	0.477	0			
0.25	0.0052	1.0238	0.327	0.470	0.5	3.1E-02	2.1E-02	
0.5	0.0096	1.0195	0.323	0.464	0.9	2.5E-02	1.7E-02	
1	0.0145	1.0146	0.318	0.457	1.4	1.4E-02	9.7E-03	
1	0.0126	1.0164	0.320	0.459	1.2	Water Added		
2	0.0170	1.0120	0.315	0.453	1.7	6.4E-03	4.4E-03	
4	0.0223	1.0067	0.310	0.445	2.2	3.8E-03	2.6E-03	
6	0.0282	1.0008	0.304	0.437	2.7	4.2E-03	2.9E-03	
4	0.0275	1.0015	0.305	0.438	2.7	Unloaded		
2	0.0250	1.0040	0.307	0.441	2.4			
1	0.0216	1.0075	0.311	0.446	2.1			

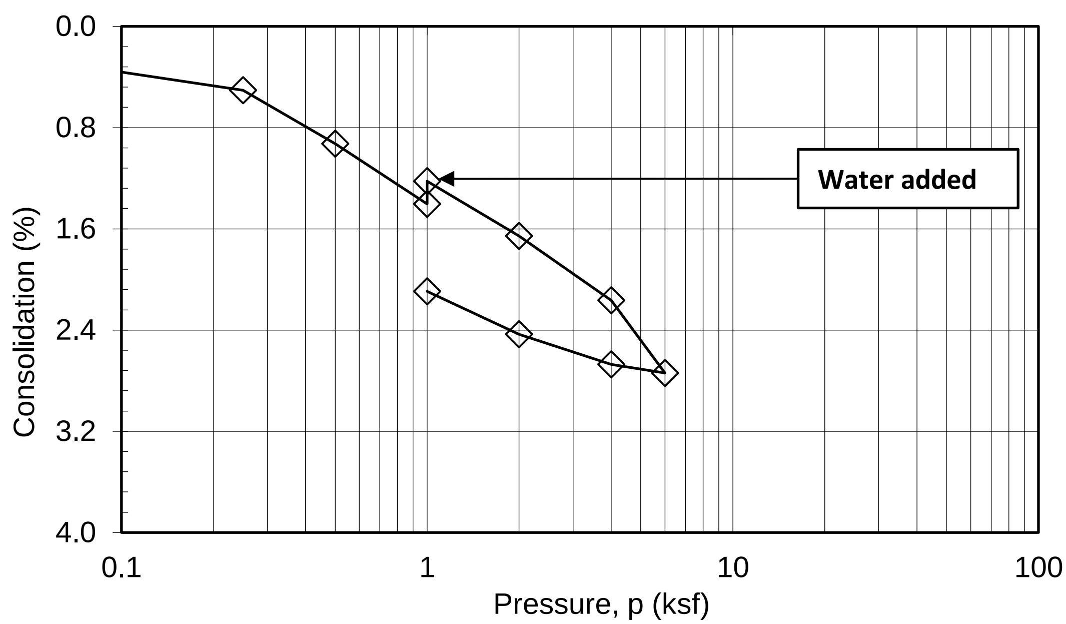
DRAFT

CONSOLIDATION TEST

ASTM D2435

Client :	NTS Geotechnical	*HAI Project No.:*	TWI-20-005
Project Name:	2601 Chapman Ave. Fullerton	*Tested by:*	KL
Project Number:	NTS 20073	*Checked by:*	MJ
Boring No.:	B5	*Date:*	04/27/20
Sample No.:	R		
Type of Sample:	Undisturbed Tube		
Depth (ft):	20		
Soil Description:	Reddish Brown, Sandy Clay (CL)		





DRAFT

Job No.: SCG-20-028 | Sample No.: B-6 @ 7.5

Client Name: NTS Geotechnical | Sampled By: R.A

☒ Split Sieve ☐ Total Wash Sieve

Sieve Analysis- ASTM C117, C136

Sieve Passing	Indiv. Wt. Retained	Accum. Wt. Retained	Accum. % Retained	Accum. % Passing	Specifications min.	max.
6-inch						
4-inch						
3 1/2-inch						
3-inch						
2 1/2-inch						
2-inch						
1 1/2-inch						
1-inch						
3/4inch						
1/2-inch						
3/8-inch						
No. 4	0.0	0.0	0.0	100.0		
WW of -NO.4	172.2		W,W, Bef. Wash		172.2	
DW of -No.4	159.1		D.W. Bef, Wash		159.1	
DW of Total	159.1		D.W. Aft. Wash		101.5	
WW of Total	172.2		%Loss *	1.43	min	max
No. 8	0.0	0.0	0.0	100		
No. 10	0.1	0.1	0.0	100		
No. 16	0.0	0.1	0.1	100		
No. 30	0.3	0.4	0.2	100		
No. 40	4.0	4.3	2.7	97		
No. 50	10.6	14.9	9.4	91		
No.100	46.4	61.4	38.6	61		
No. 200	36.1	97.5	61.3	37.3		
Pan	1.7	99.2		Moisture Data:		
Fineness Modulus:				Wet Wt.	90.8	
		Results	Maximum	Dry Wt.	83.9	
Liquid Limit				Wt of Water	6.9	
Plasticity Index				% Moisture	8.2	

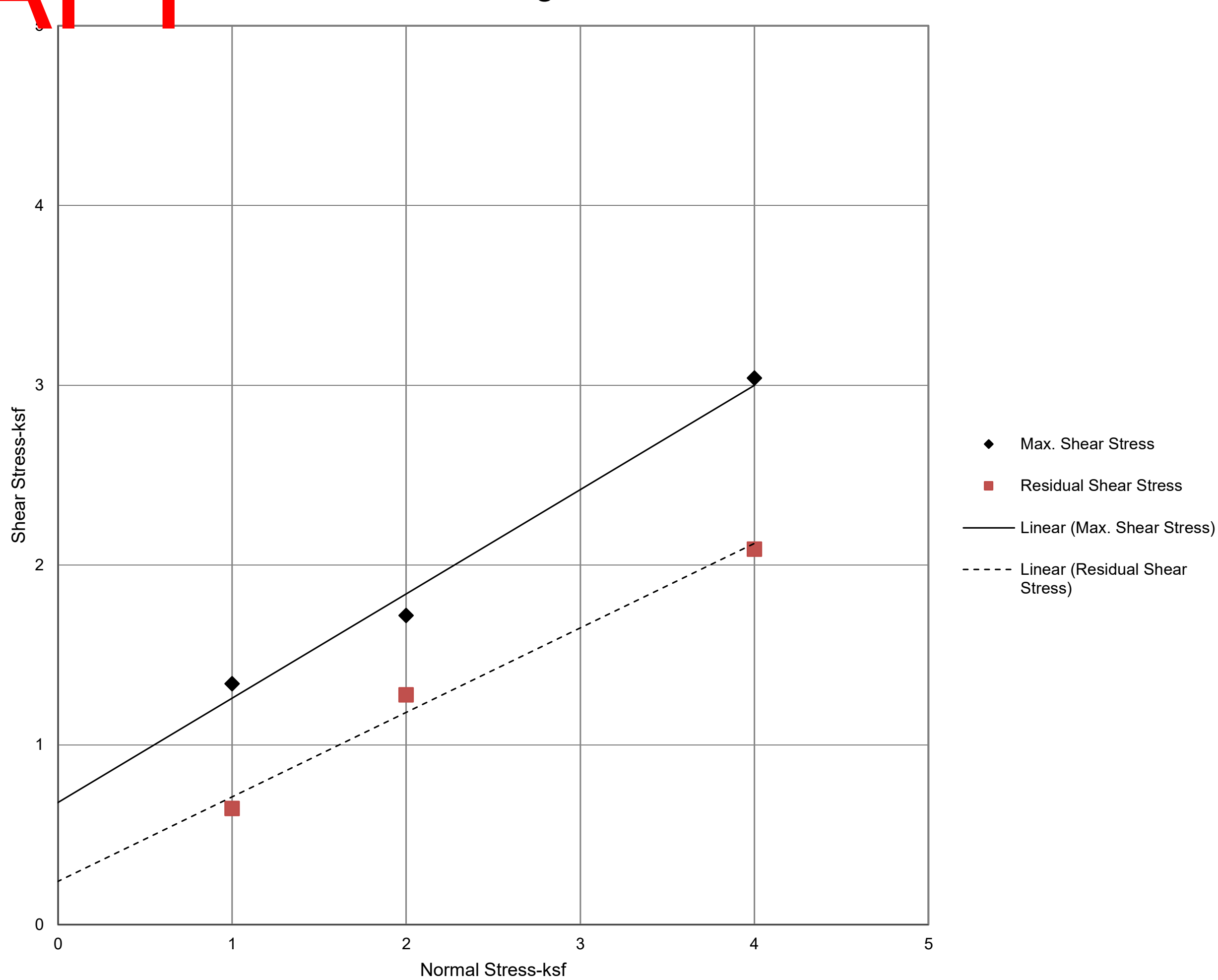
% Gravel	0.0
% Sand	62.7
% Silt & Clay	37.3
Total	100.0%

Note: NDOT Dense Graded Plantmix must have #10 and #40 Sieves

*Loss must be less than or equal to 0.3% of sample.

DRAFT

Friction Angle Determination



Maximum Density= pcf Optimum Moisture %

Sample No.	Normal Stress (ksf)	Maximum Shear Stress (ksf)	Residual Shear Stress (ksf)	Wet Density (psf)	Moisture Content (%)	Dry Density (psf)	Compaction (%)
B-1 @ 5	1.0	1.3	0.6	118.5	N/A	N/A	N/A
B-1 @ 5	2.0	1.7	1.3	118.5	N/A	N/A	N/A
B-1 @ 5	4.0	3.0	2.1	118.5	N/A	N/A	N/A

Sample Type: CAL RING Samples **Test Condition:** In-situ **Sample Location:** B-6@2.5 ft.

Maximum Shear Stress Test Results

Cohesion (psf):	680
Friction Angle (degrees):	30
Shear Rate (in/min)	**0.02**

Residual Shear Stress Test Results

Cohesion (psf):	241
Friction Angle (degrees):	25
Shear Rate (in/min)	**0.02**

Lab ID: B-1 @ 5 ft.

Sample Date: 8-25-2020

Project No.

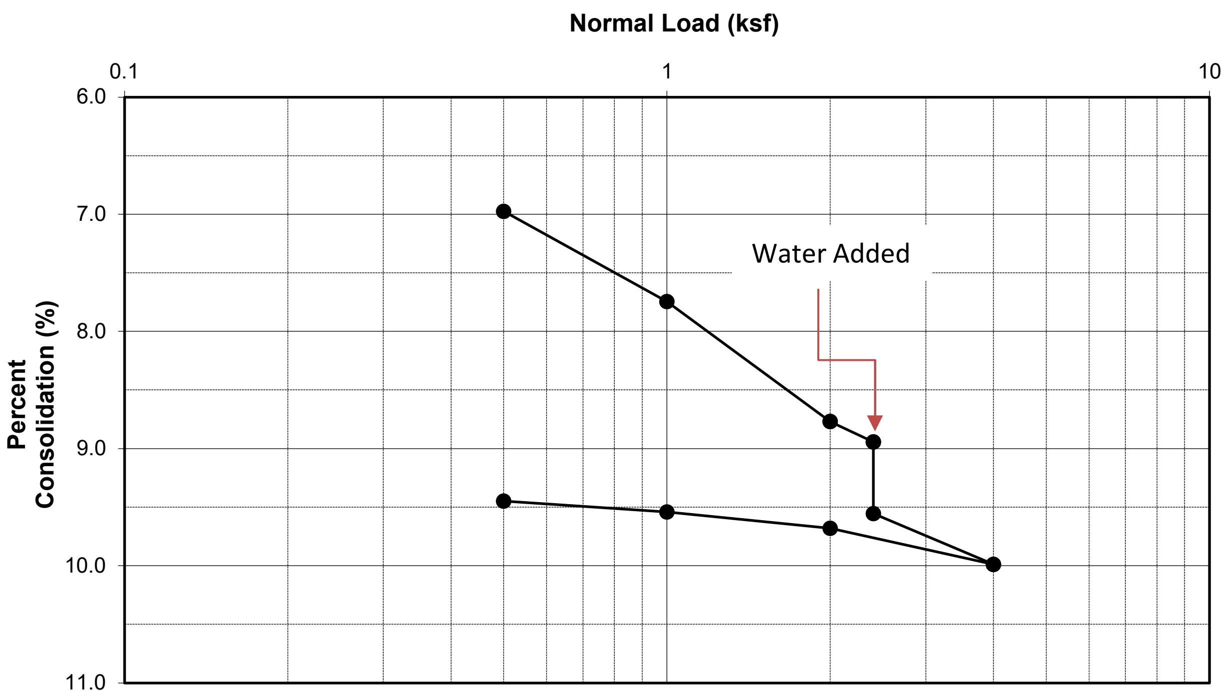
SCG-20-028

NOVA GEOTECHNICAL & inspection services
16 Technology Dr. Ste 139
Irvine, CA 92618
949-537-3222

Reviewed By:____________________________

Rouzbeh Afshar, Ph.D., P.E.
Geotechnical Department Manager

Project: 2601-2751 Chapman Ave
Boring No: B-6
Soil Type: CL
Tested By: RA
Project No. SCG-20-028
Depth: 15 feet
Date: 9/7/2020



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Project: 2601-2751 Chapman Ave

Boring No: B-7

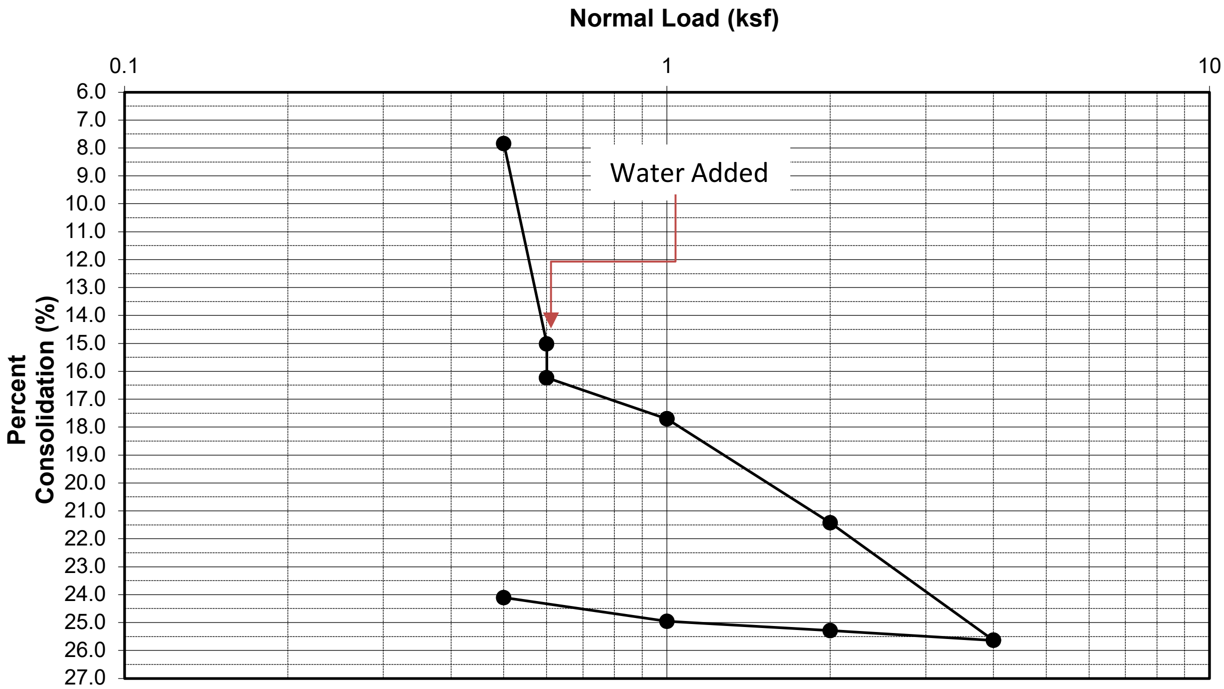
Soil Type: SM

Tested By: RA

Project No. SCG-20-028

Depth: 5 feet

Date: 9/8/2020



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Project: 2601-2751 Chapman Ave

Soil Type: CL

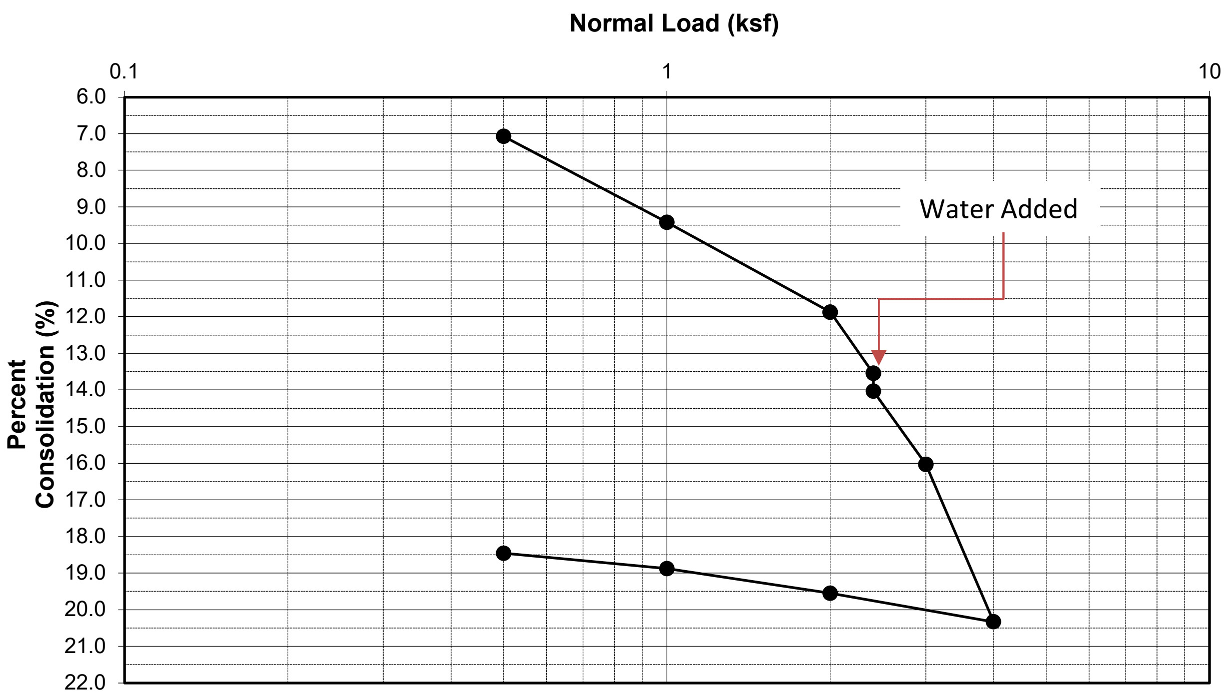
Project No. SCG-20-028

Date: 9/8/2020

Boring No: B-7

Tested By: RA

Depth: 25 feet



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NTS GEOTECHNICAL

APPENDIX C

CPT Liquefaction Analysis

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http://www.geologismiki.gr

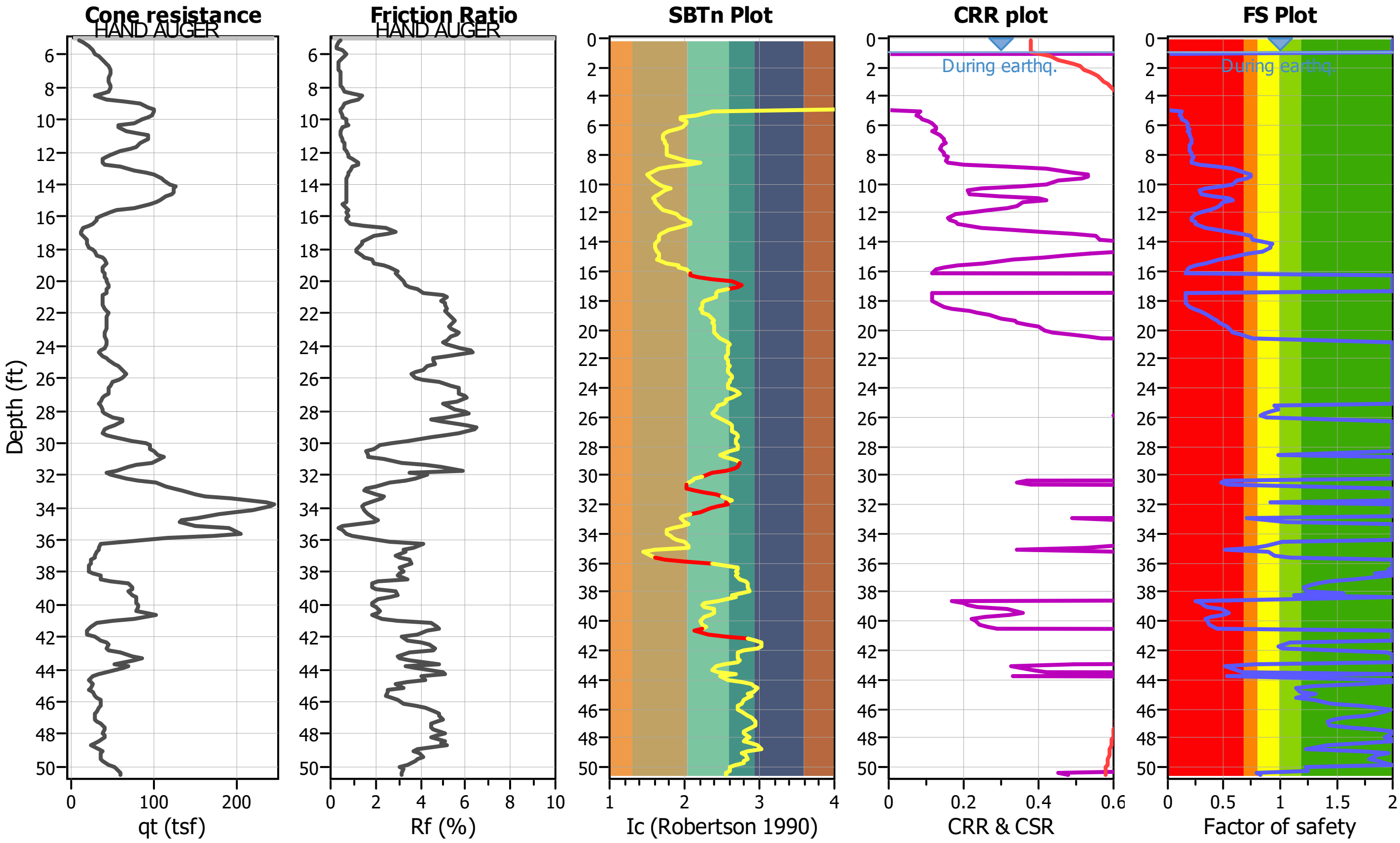
LIQUEFACTION ANALYSIS REPORT

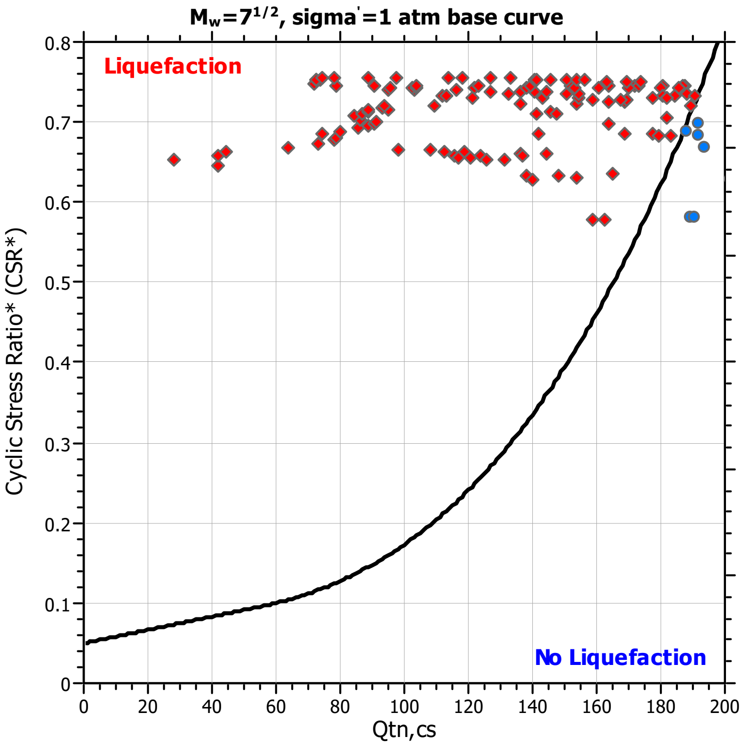
Project title : **Location :**

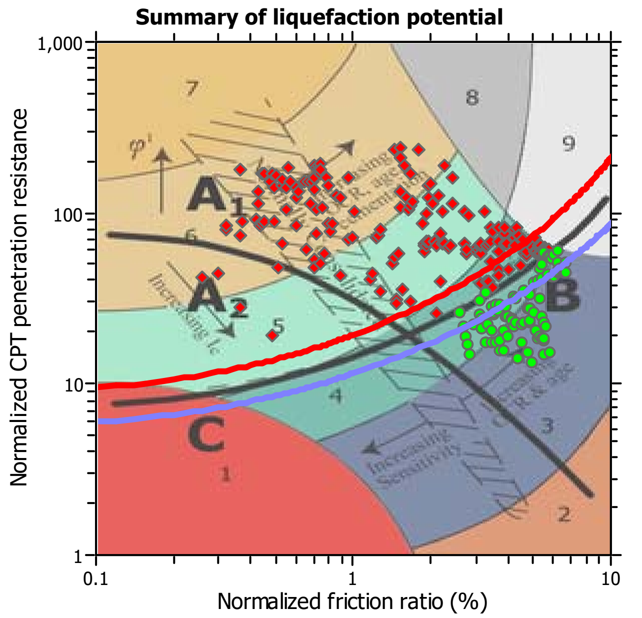
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	1.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	1.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	No		

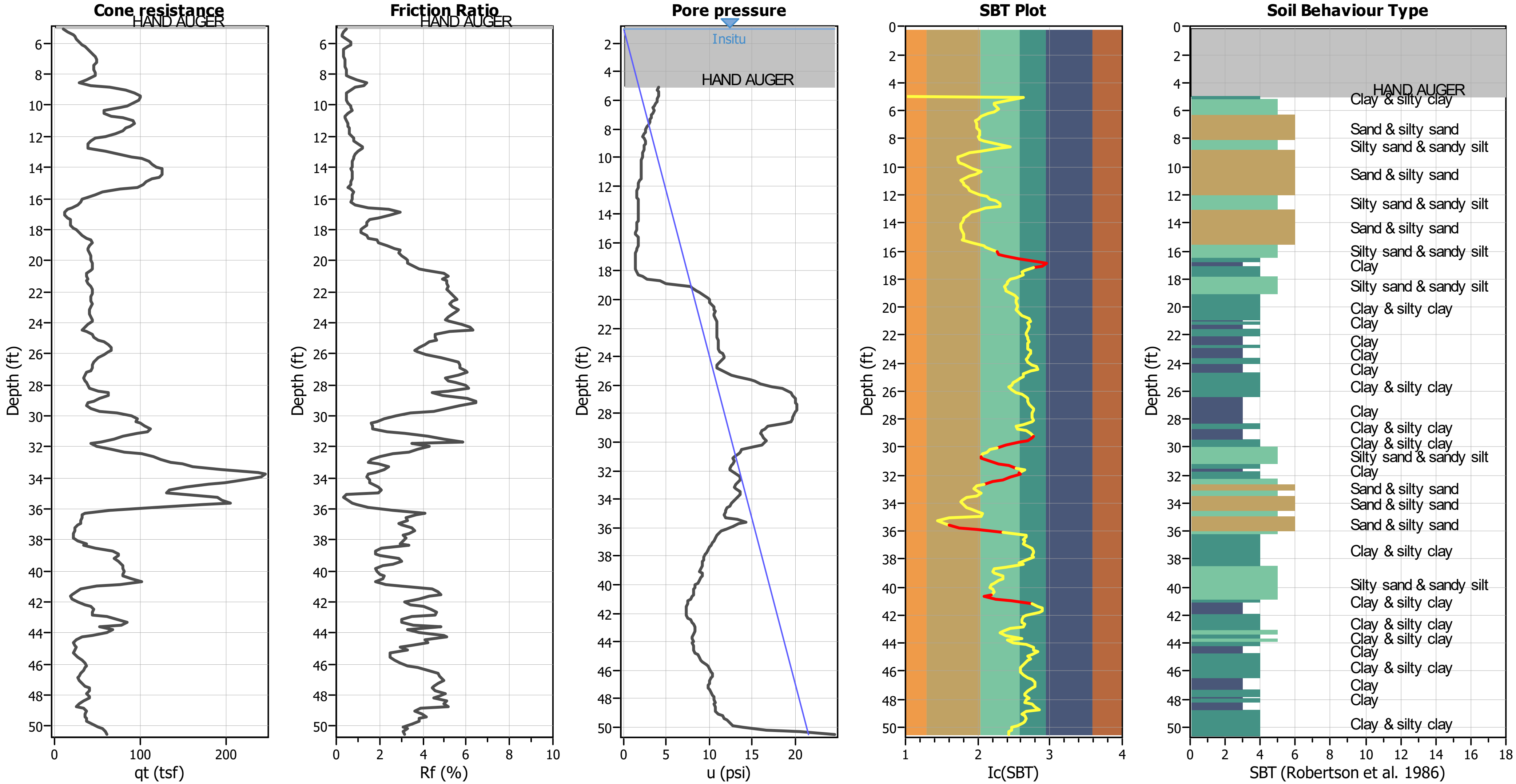






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



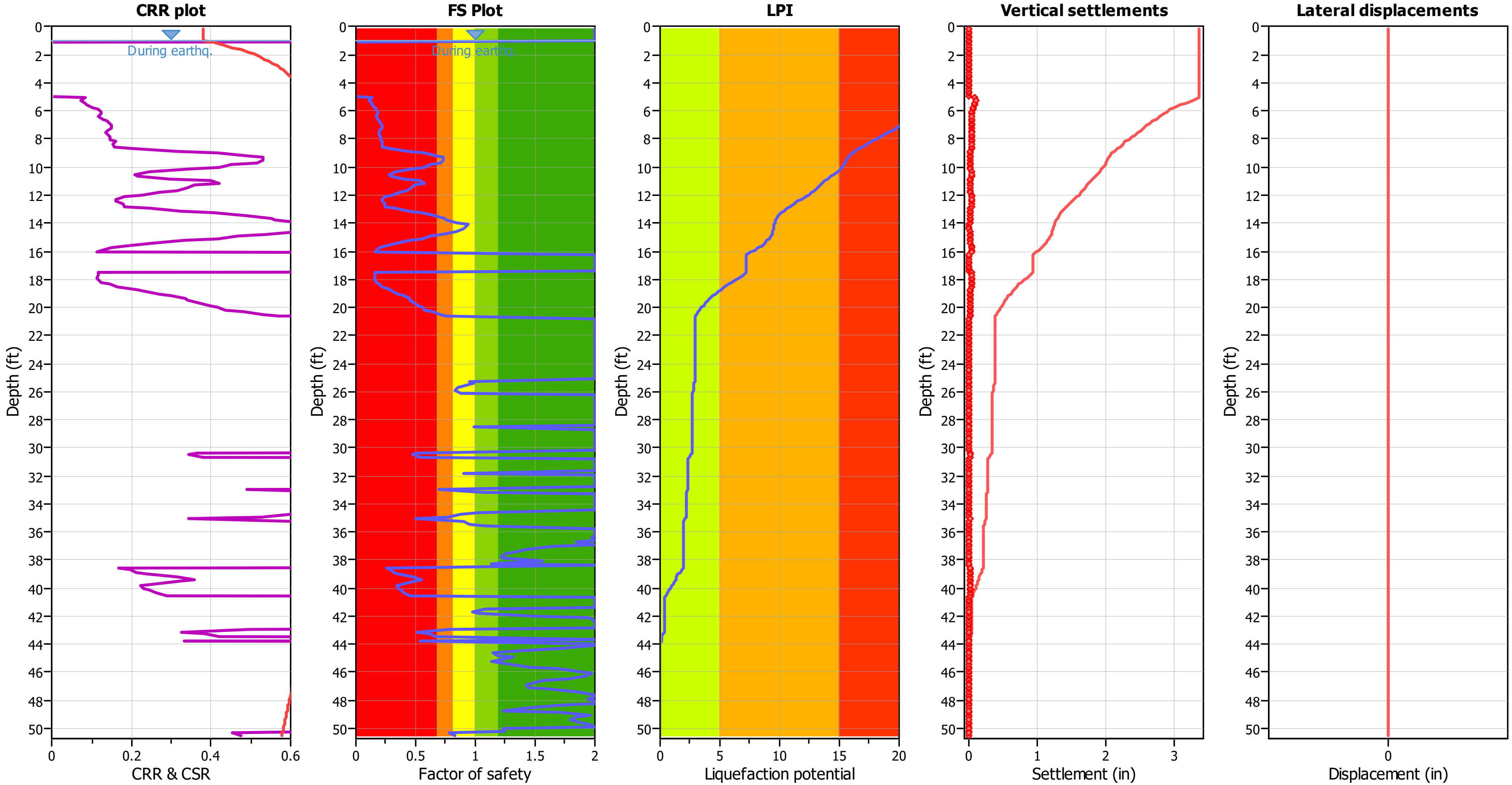
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

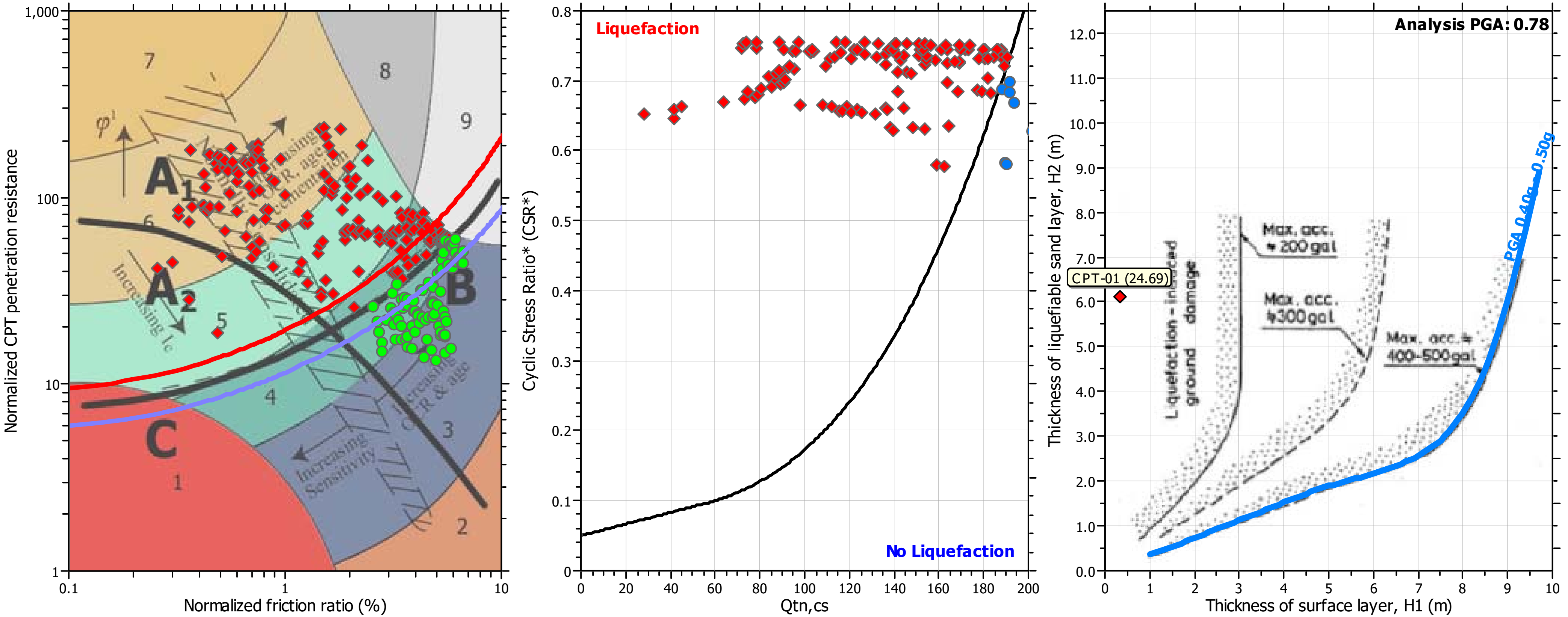
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

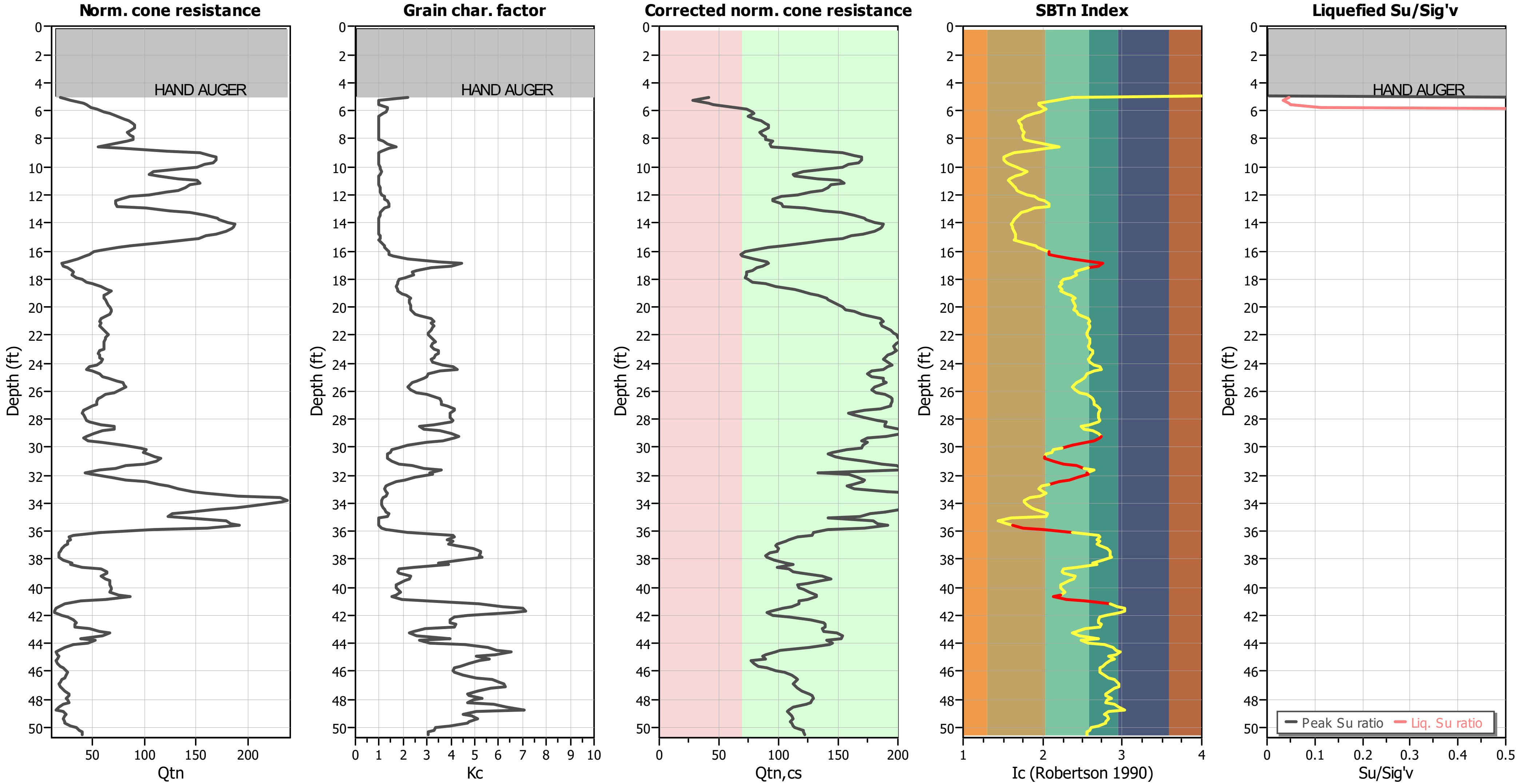
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

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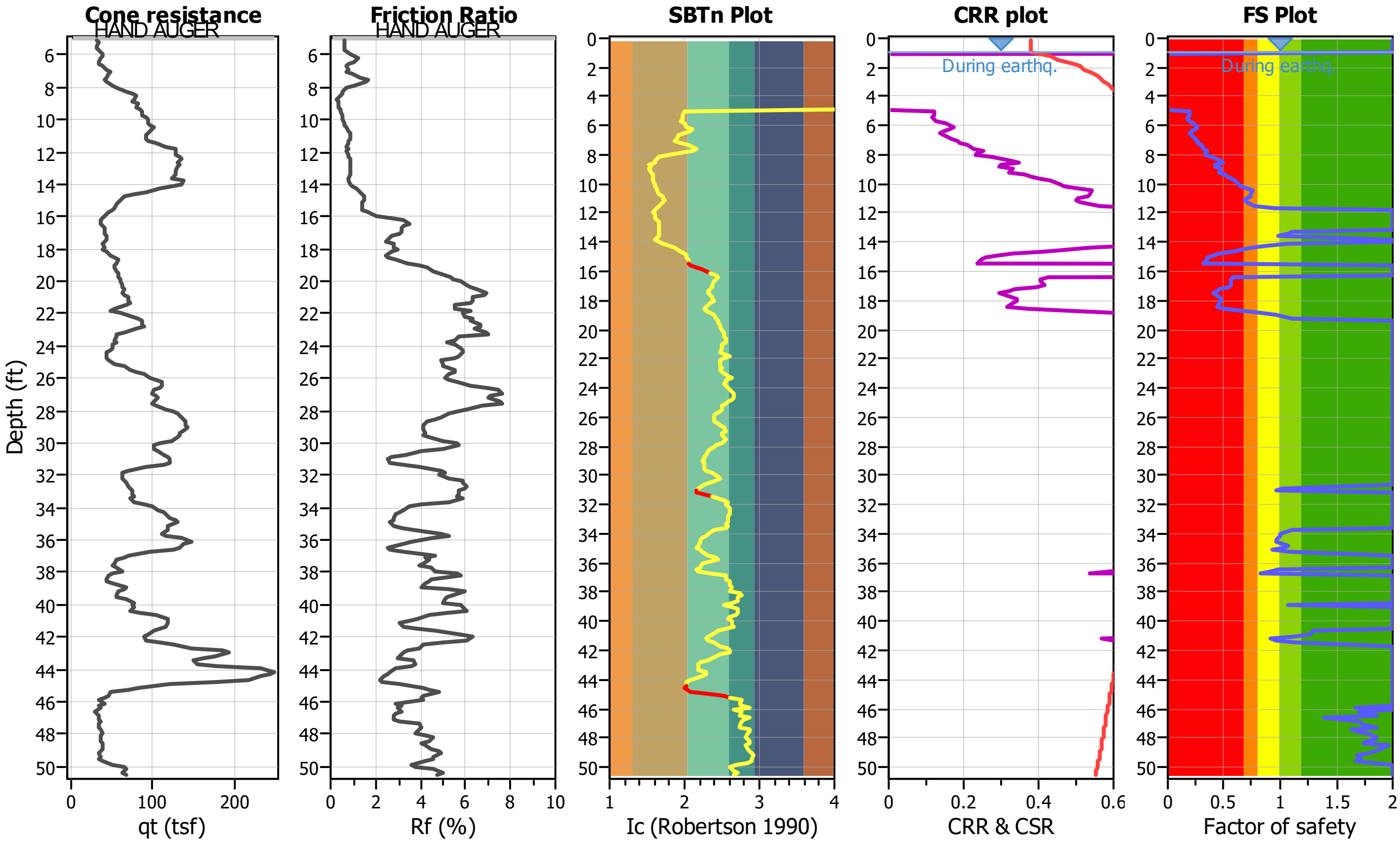
LIQUEFACTION ANALYSIS REPORT

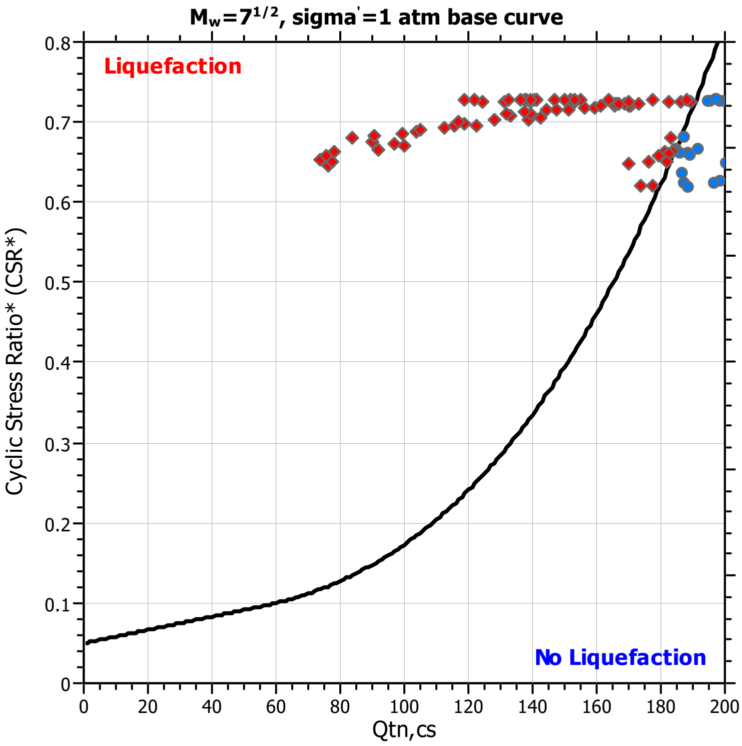
Project title : **Location :**

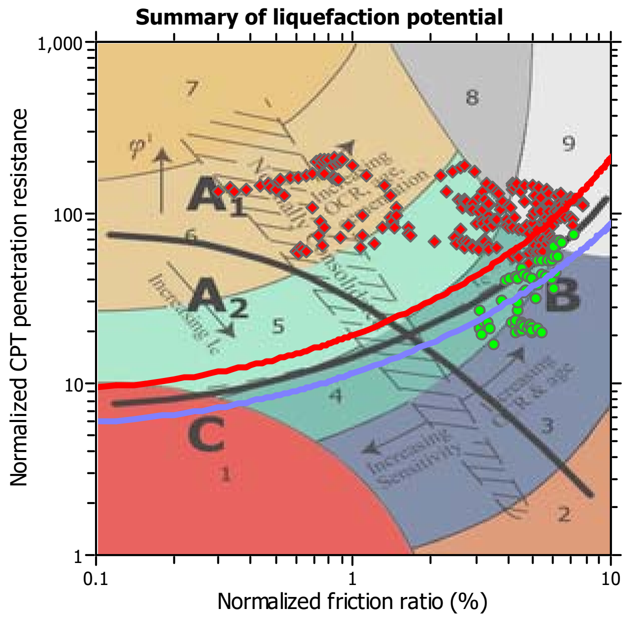
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	1.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	1.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	No		

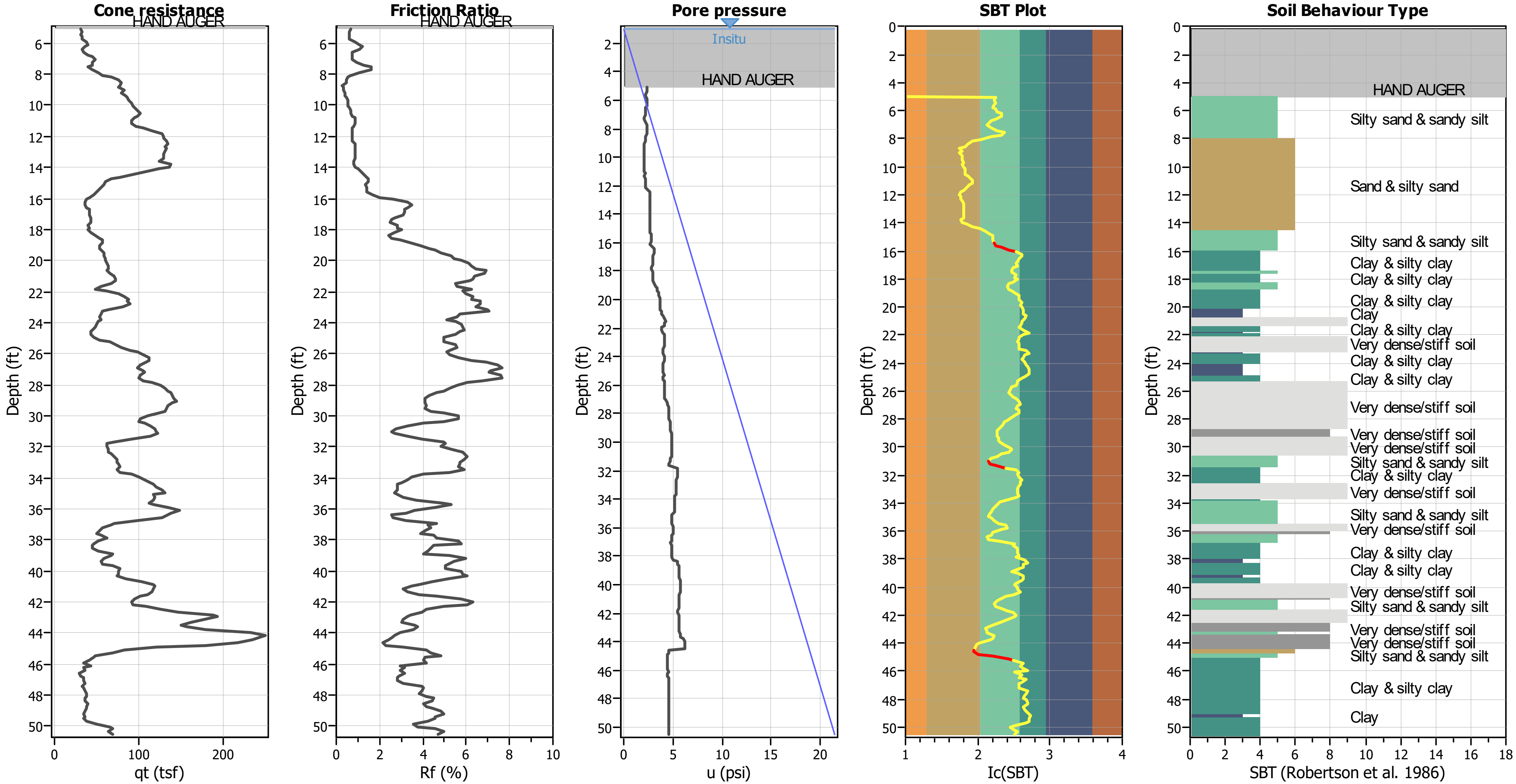






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



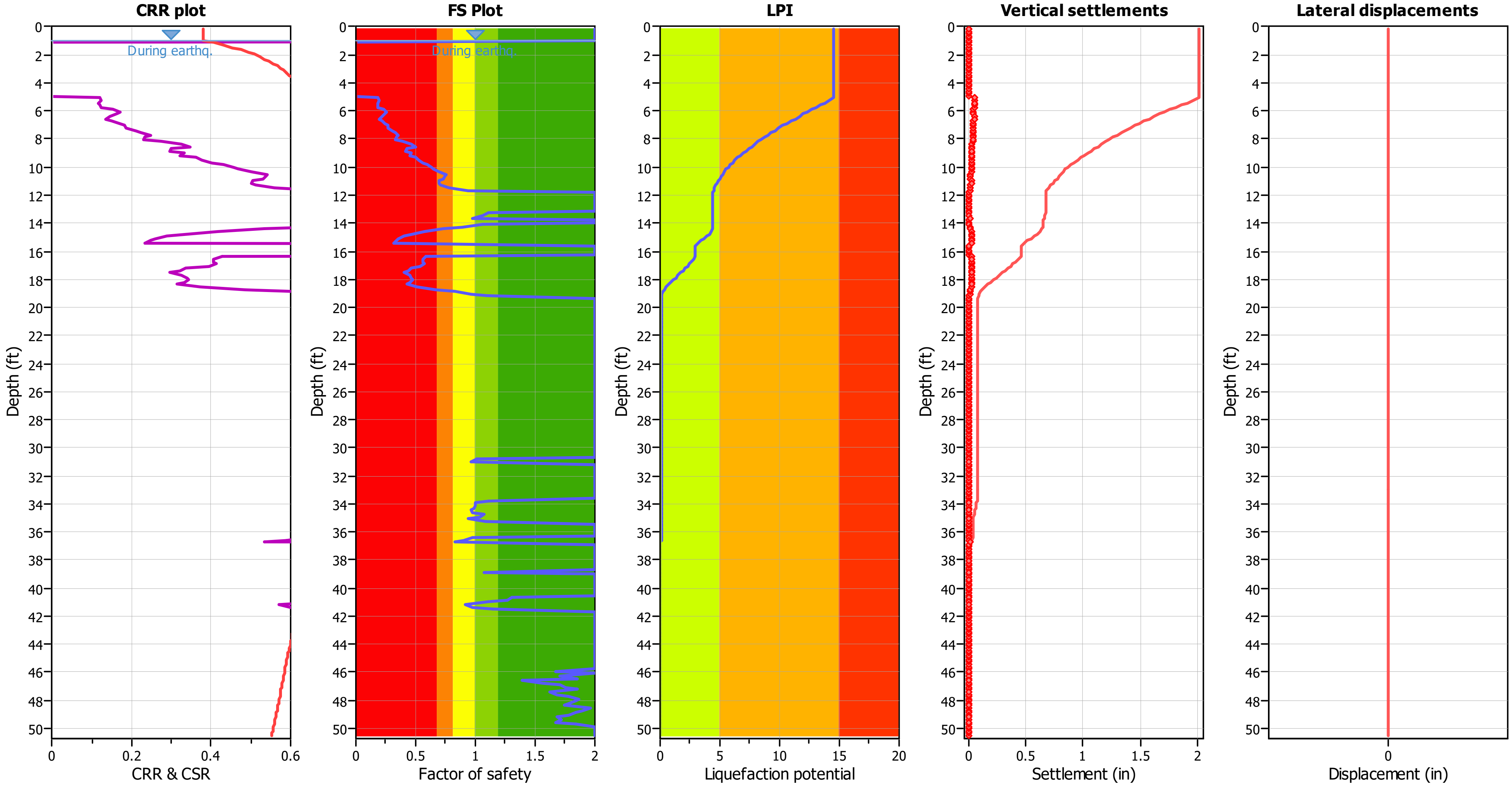
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

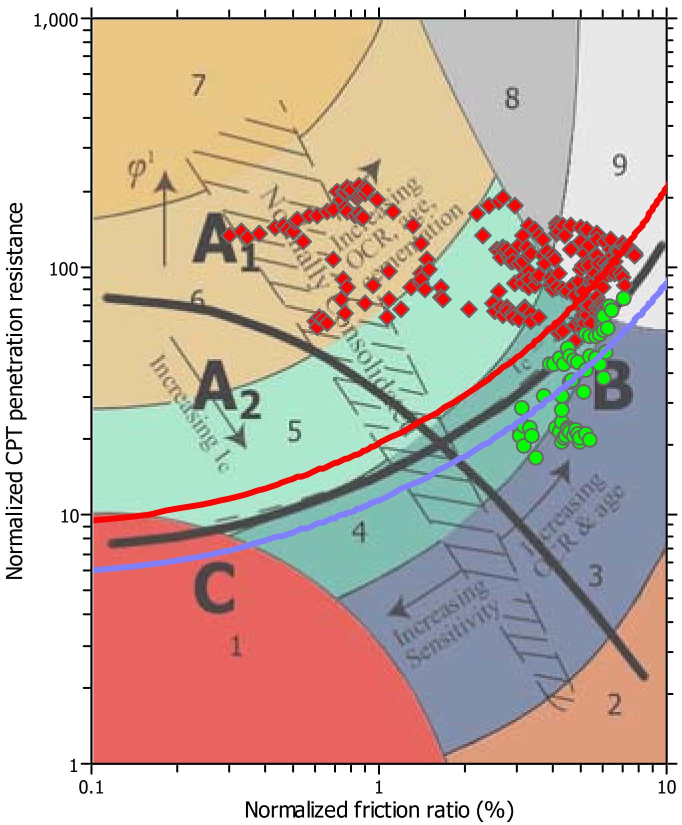
F.S. color scheme

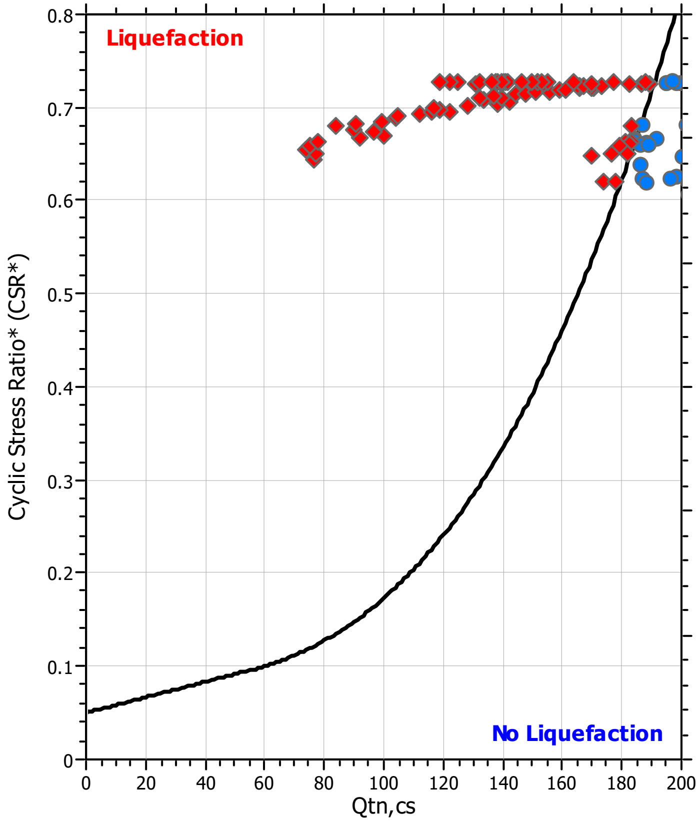
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

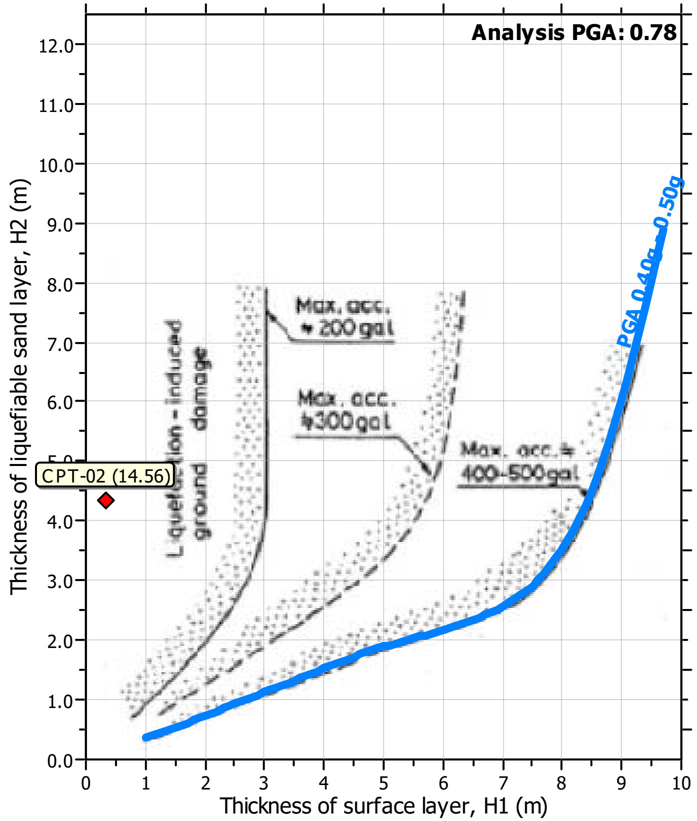
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



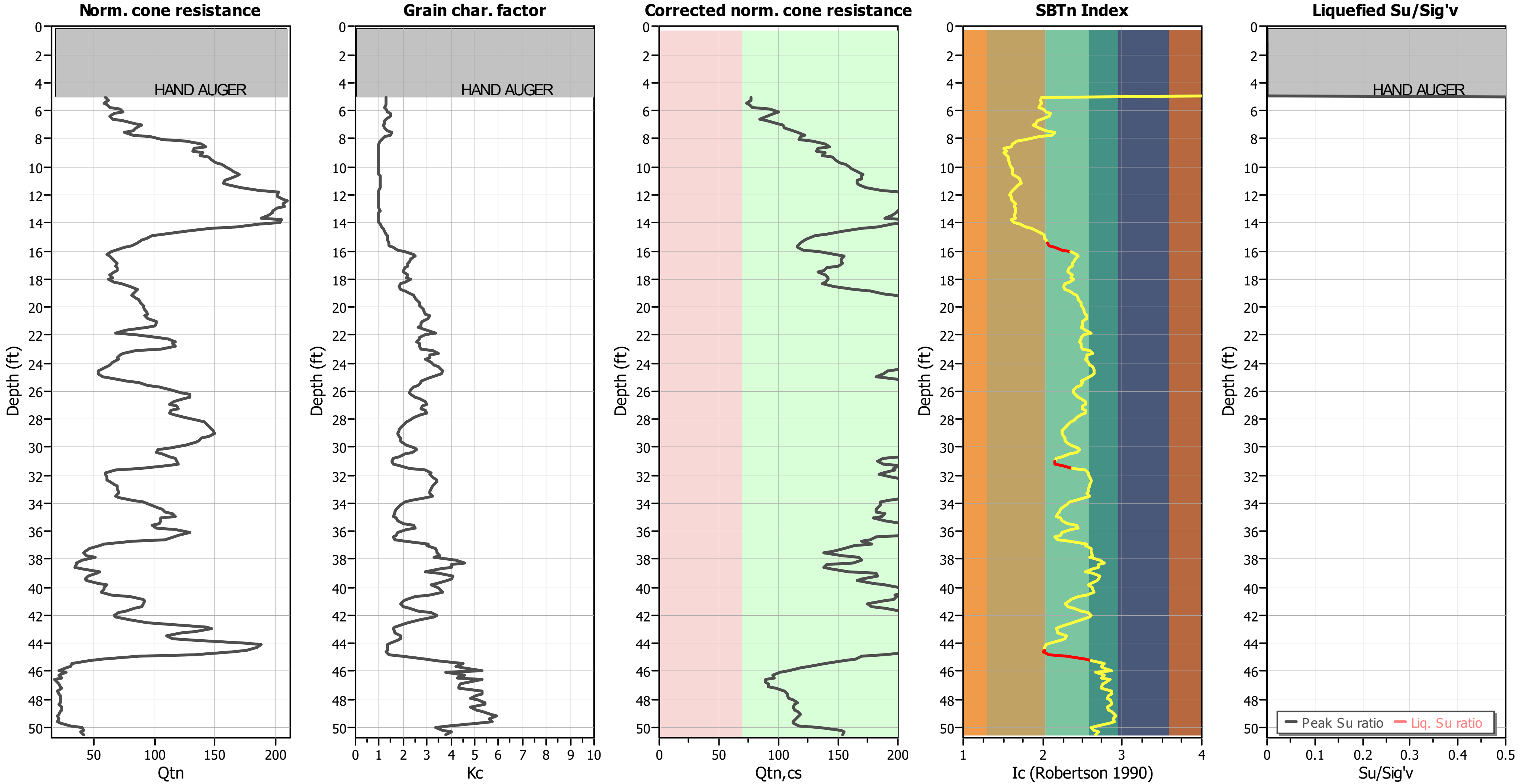




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

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Merarhias 56
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DRAFT

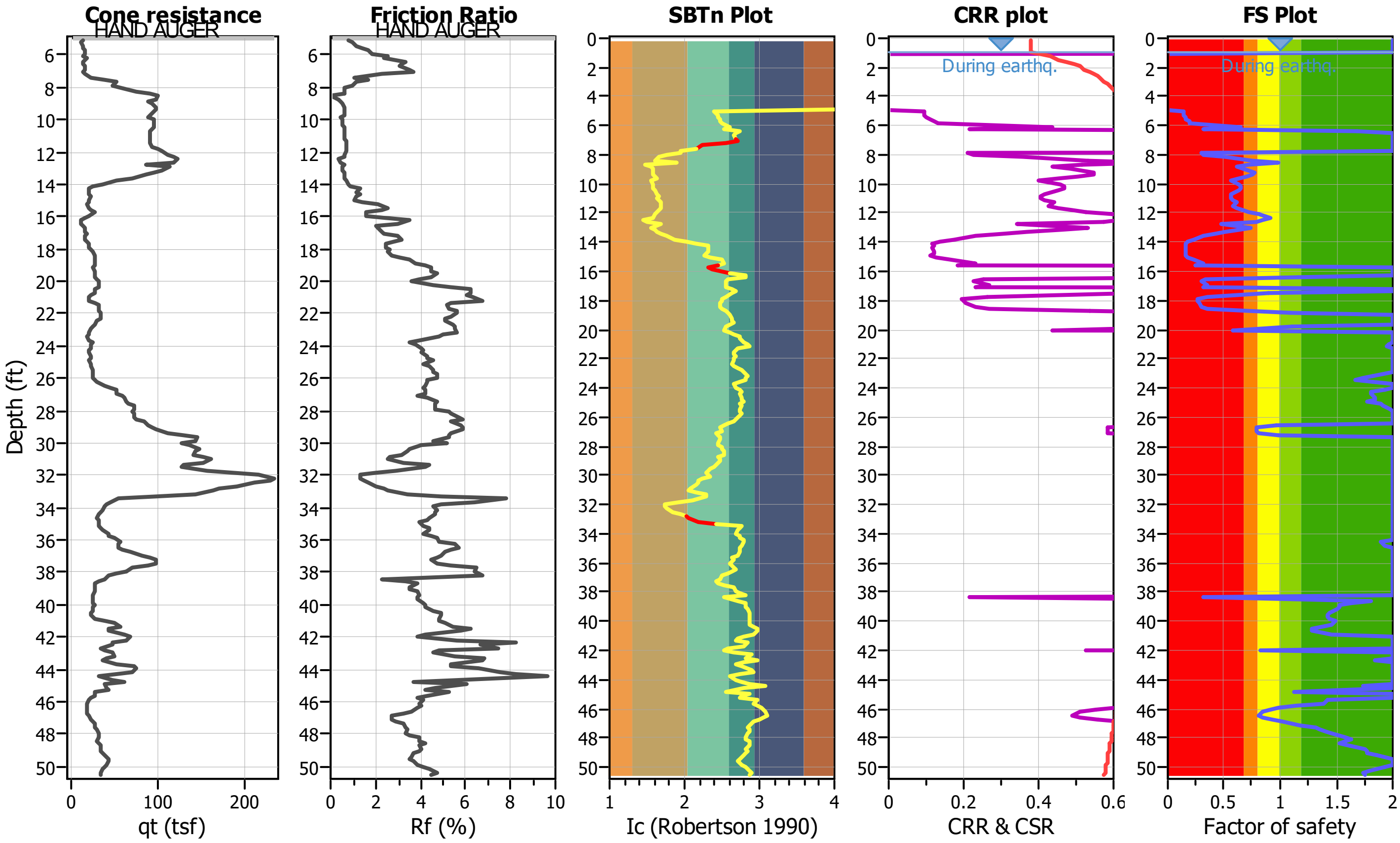
LIQUEFACTION ANALYSIS REPORT

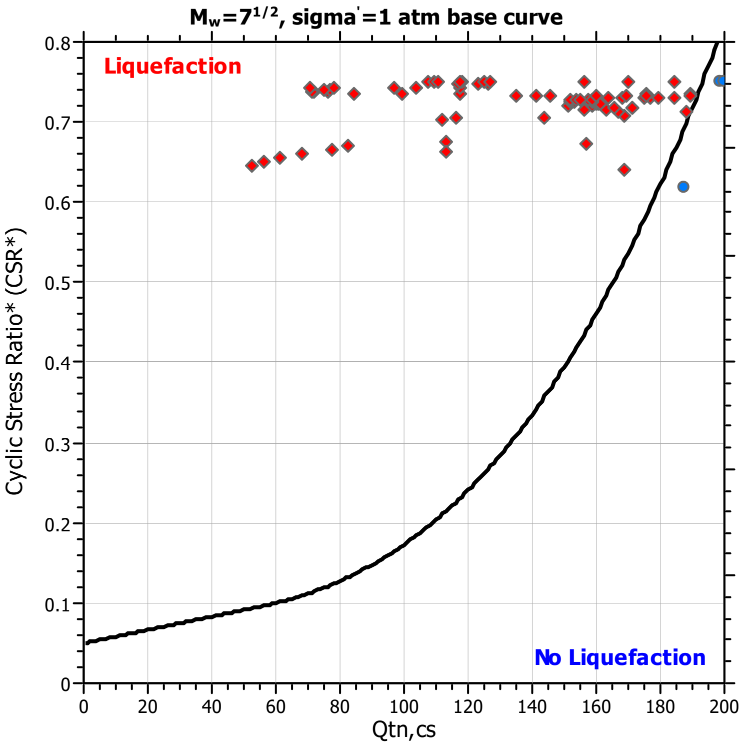
Project title : **Location :**

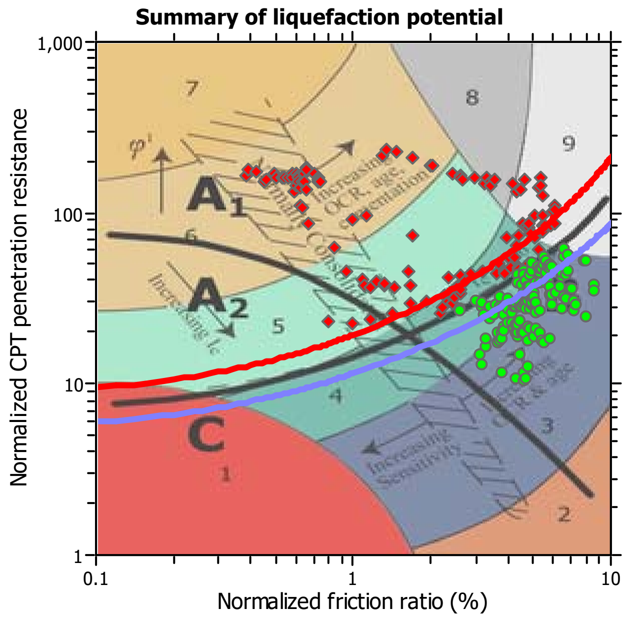
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	1.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	1.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	No		

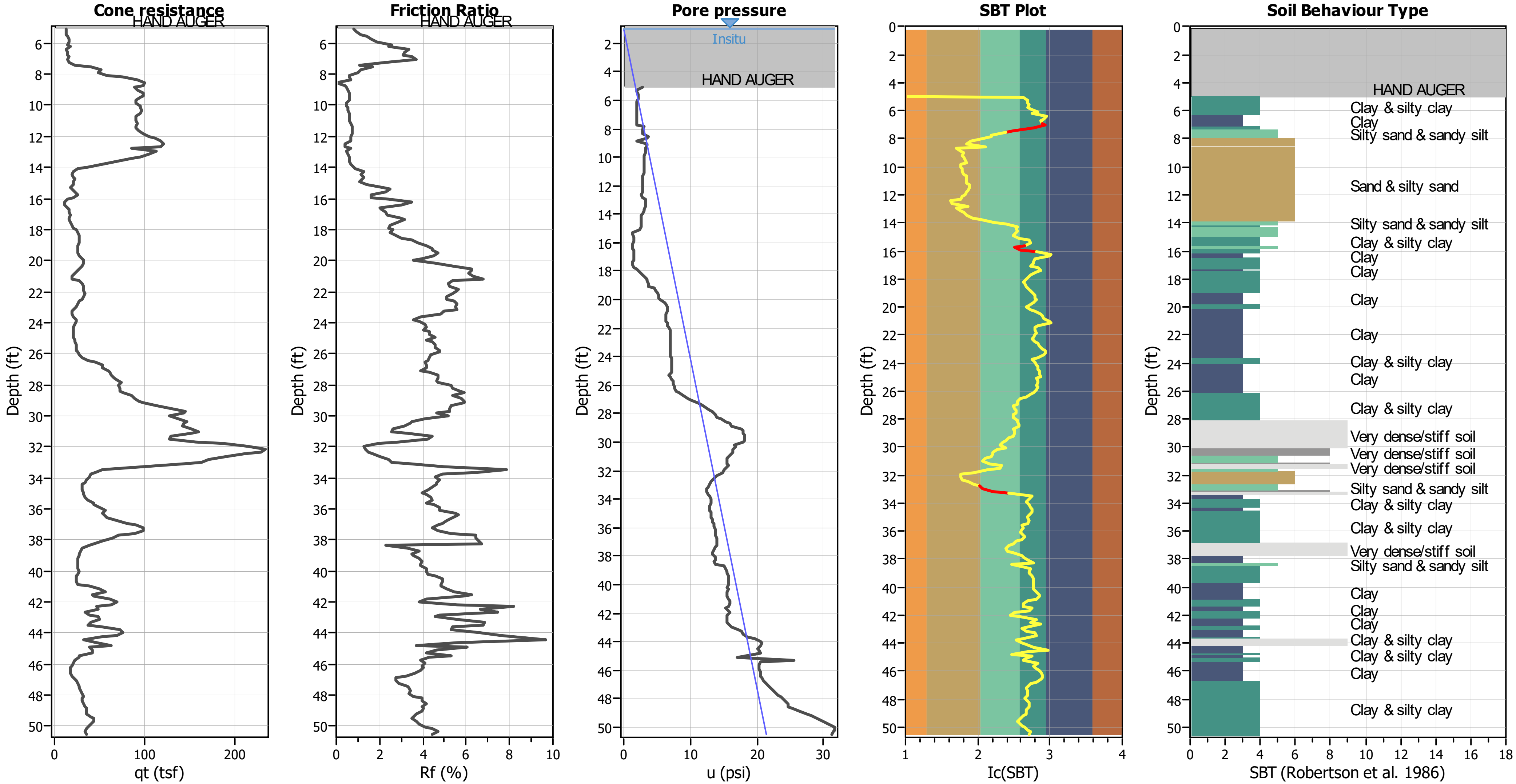






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



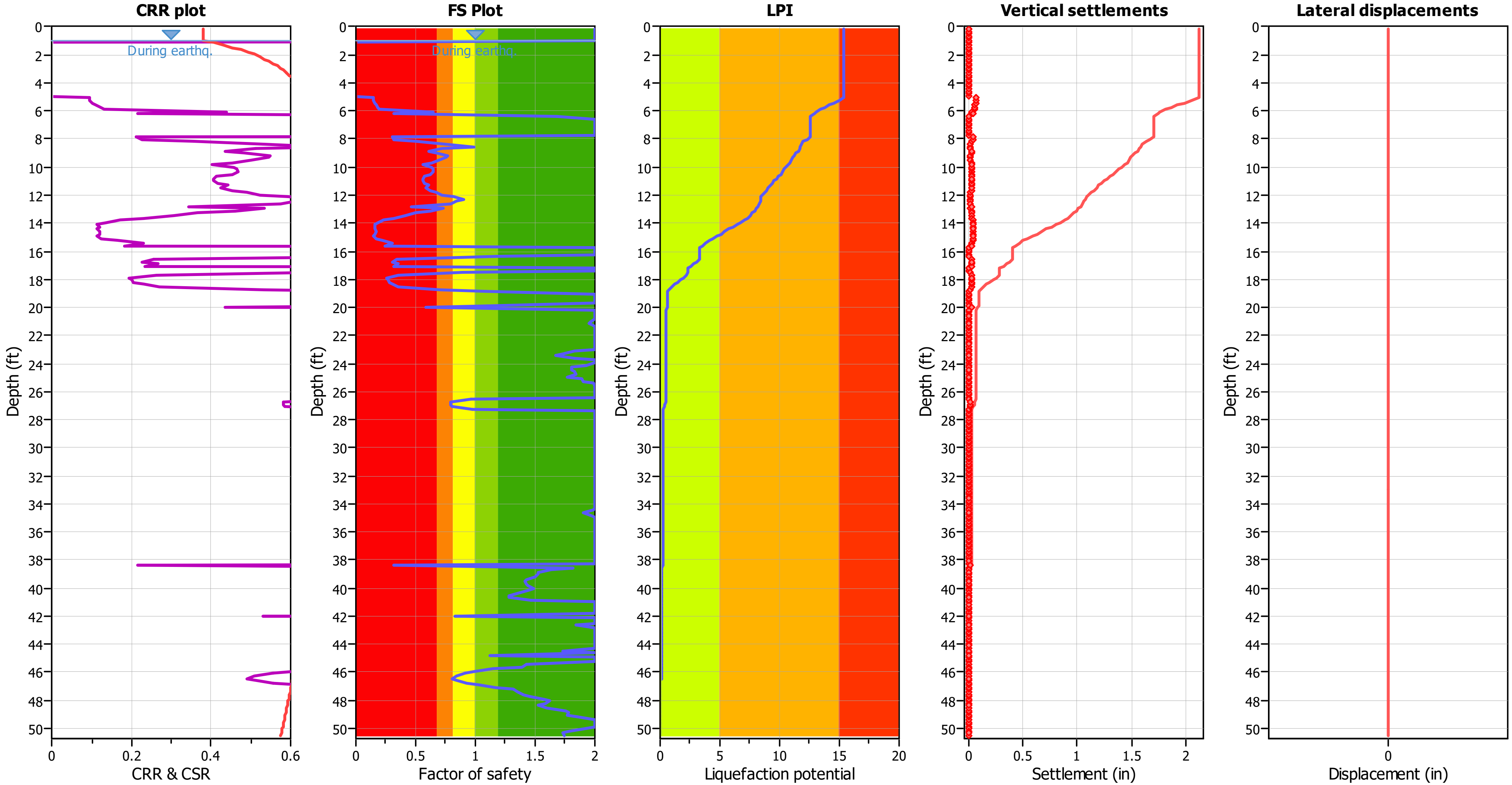
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

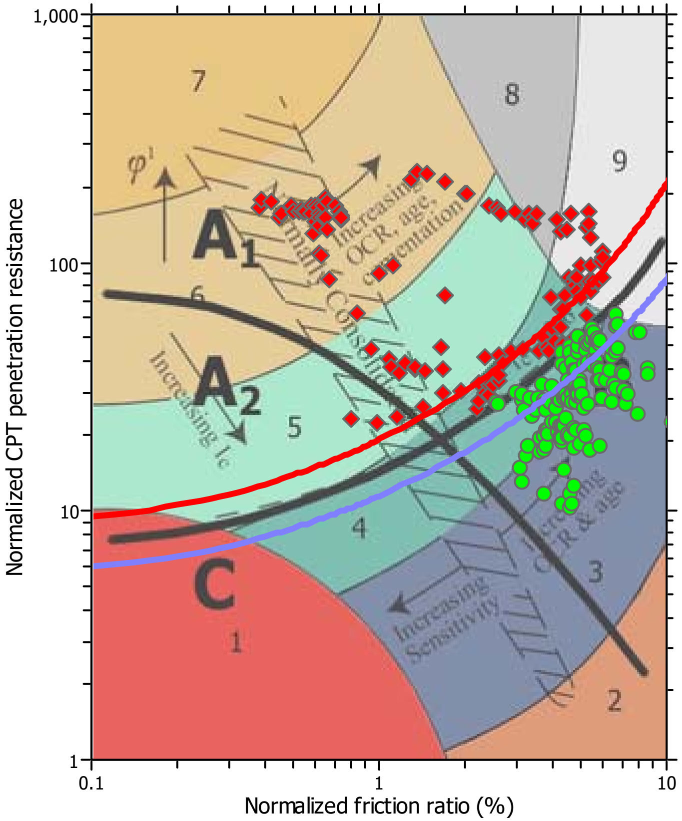
F.S. color scheme

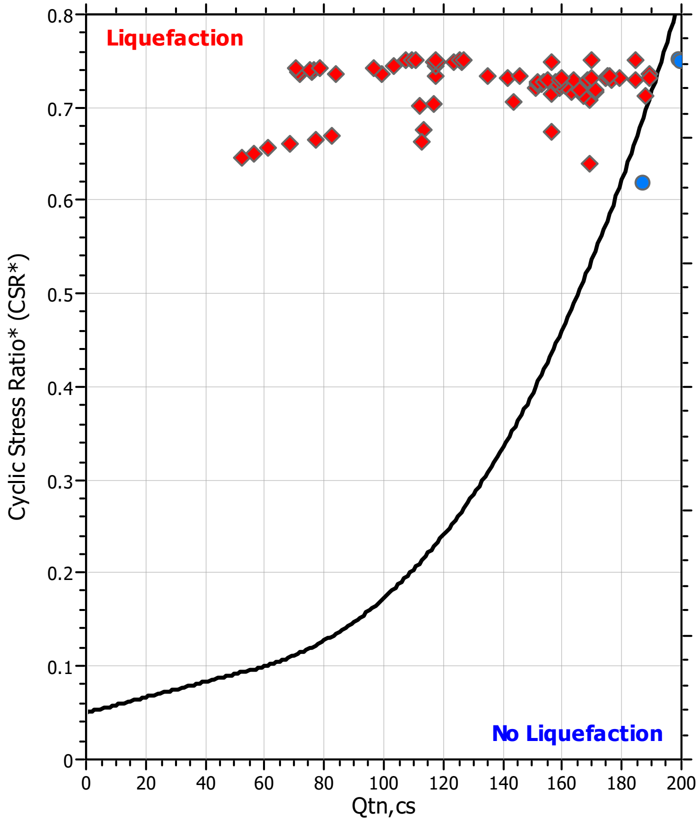
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

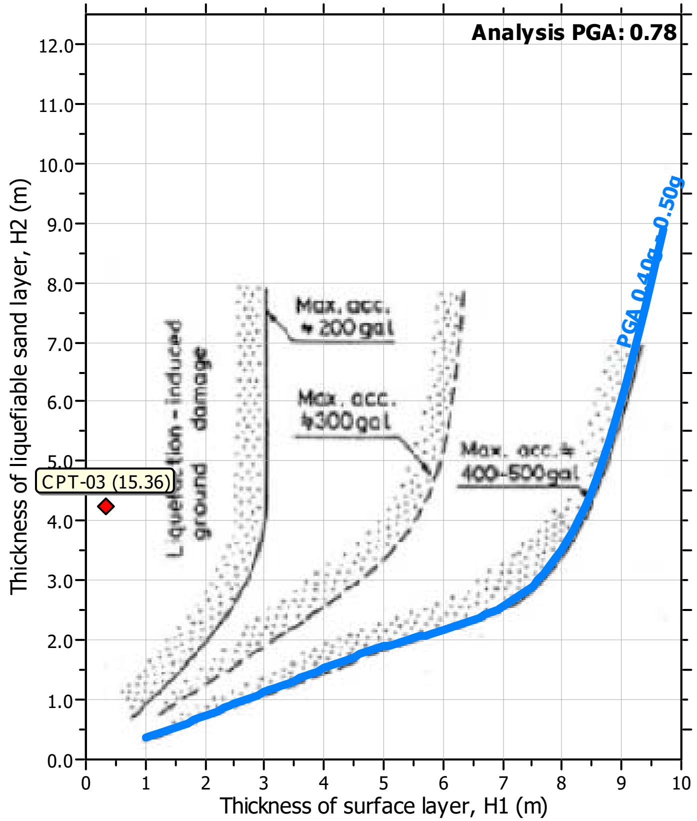
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



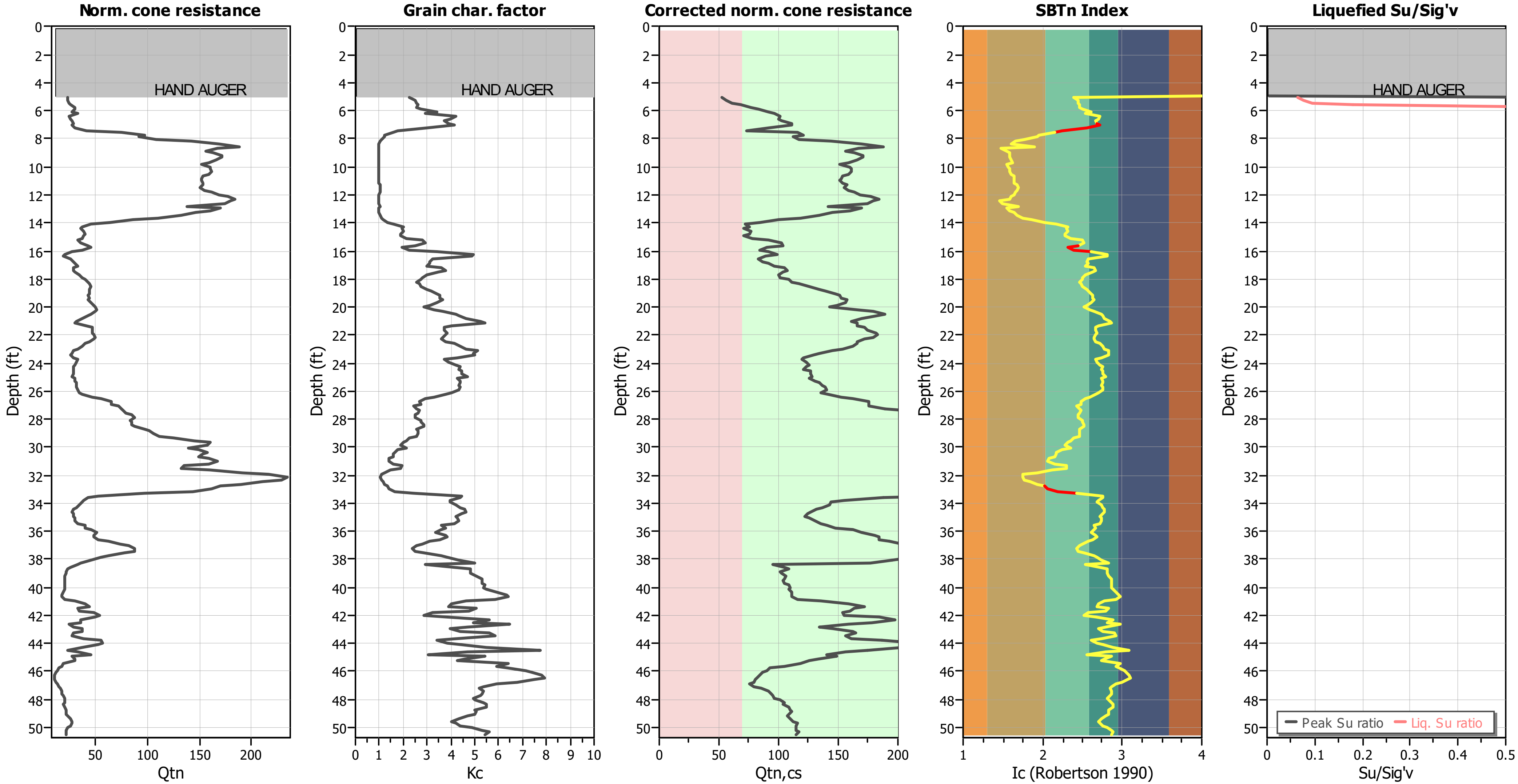




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

DRAFT

LIQUEFACTION ANALYSIS REPORT

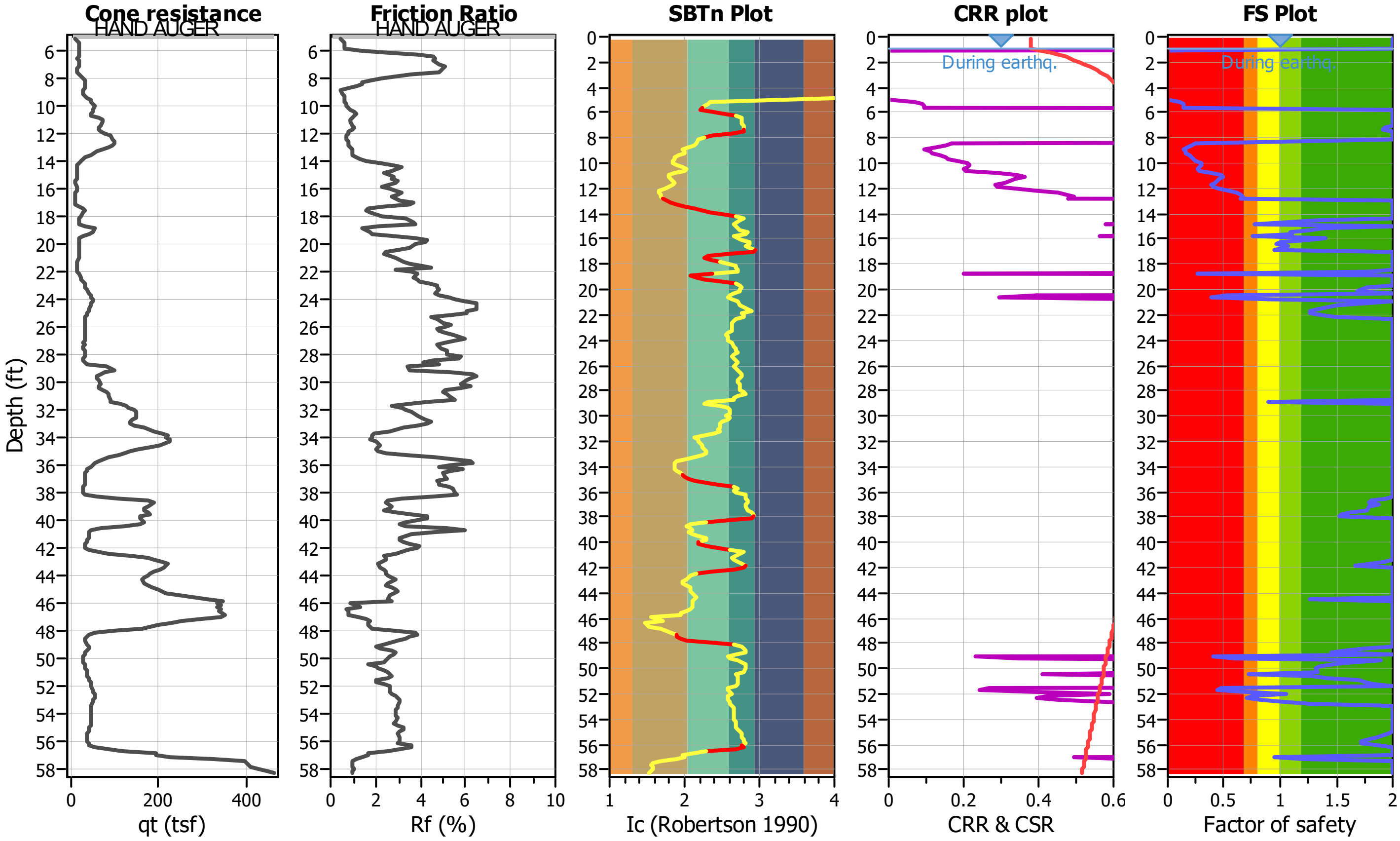
Project title : **Location :**

CPT file : CPT-04

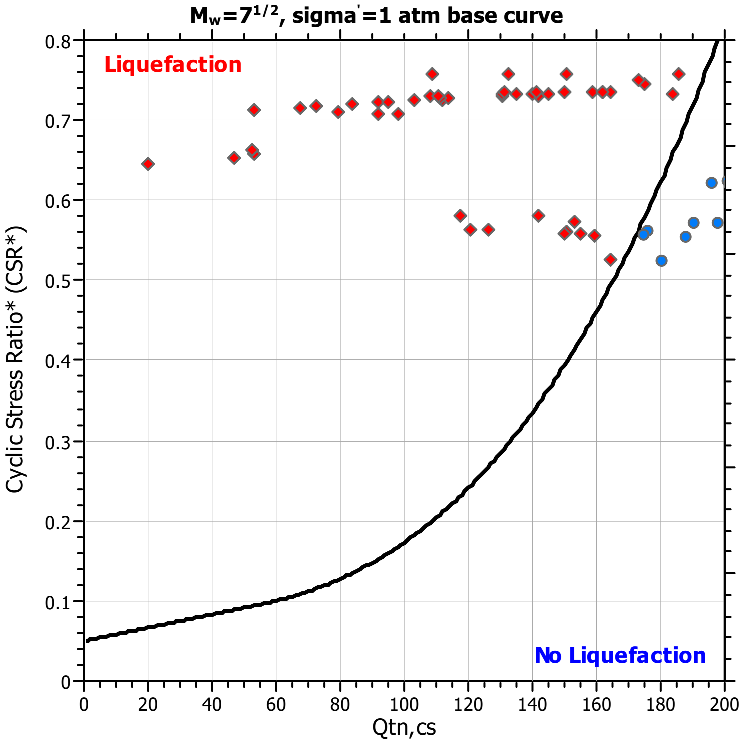
Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	1.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	1.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	No		

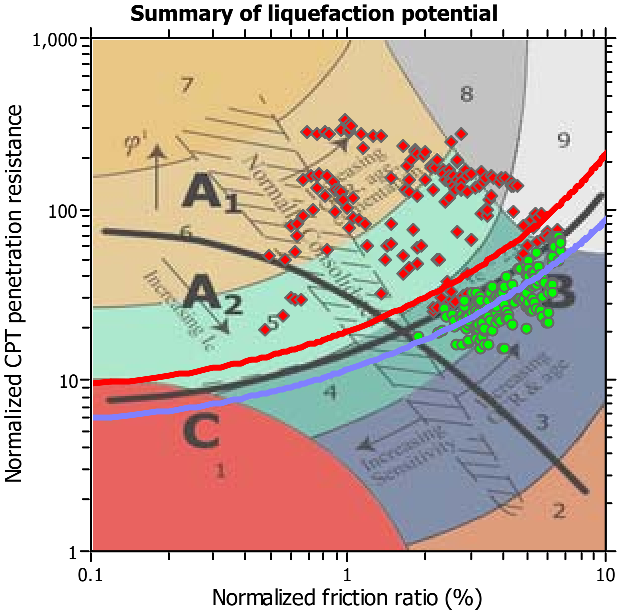
Cone resistance | Friction Ratio | SBTn Plot | CRR plot | FS Plot



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

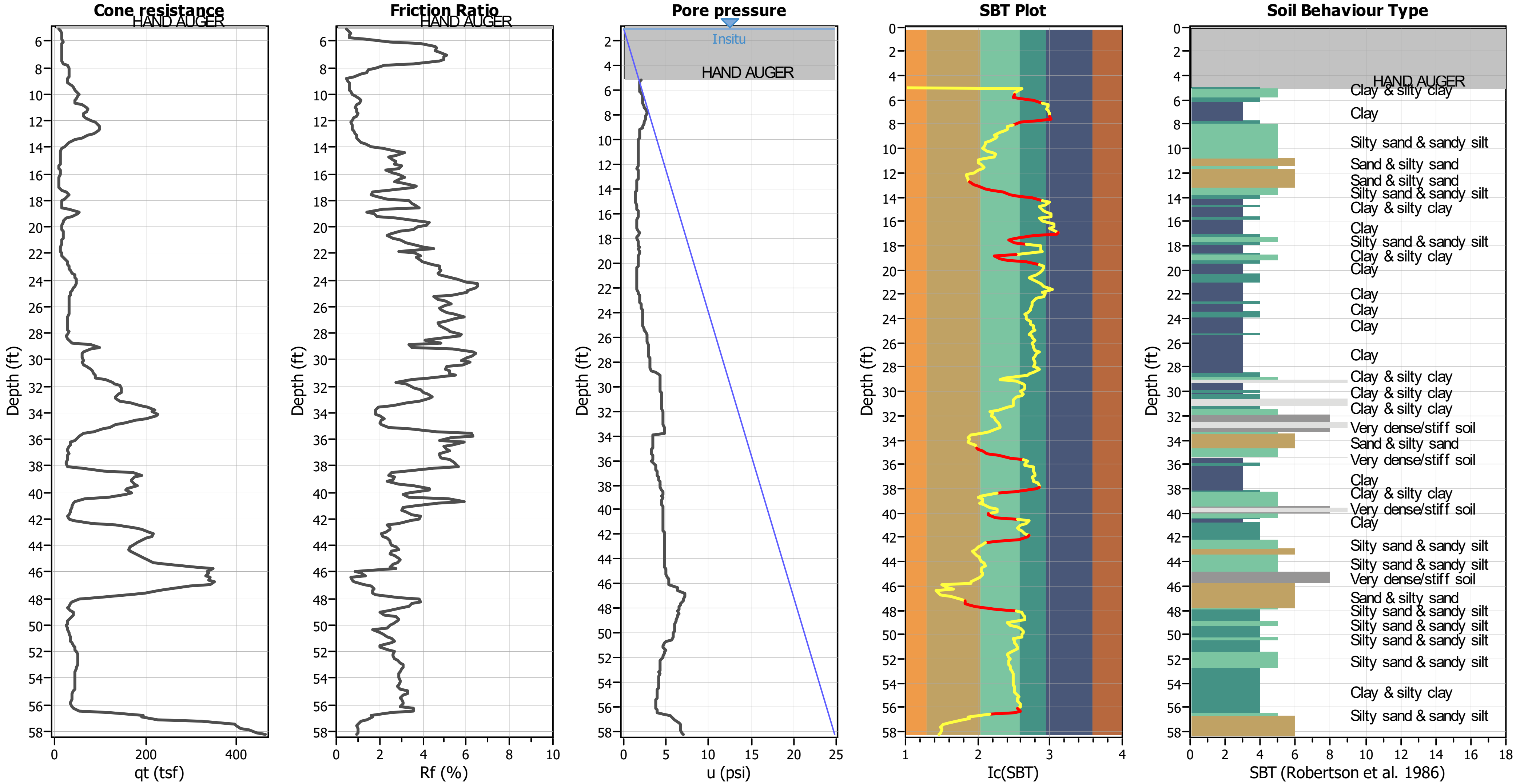


Summary of liquefaction potential



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

CPT basic interpretation plots



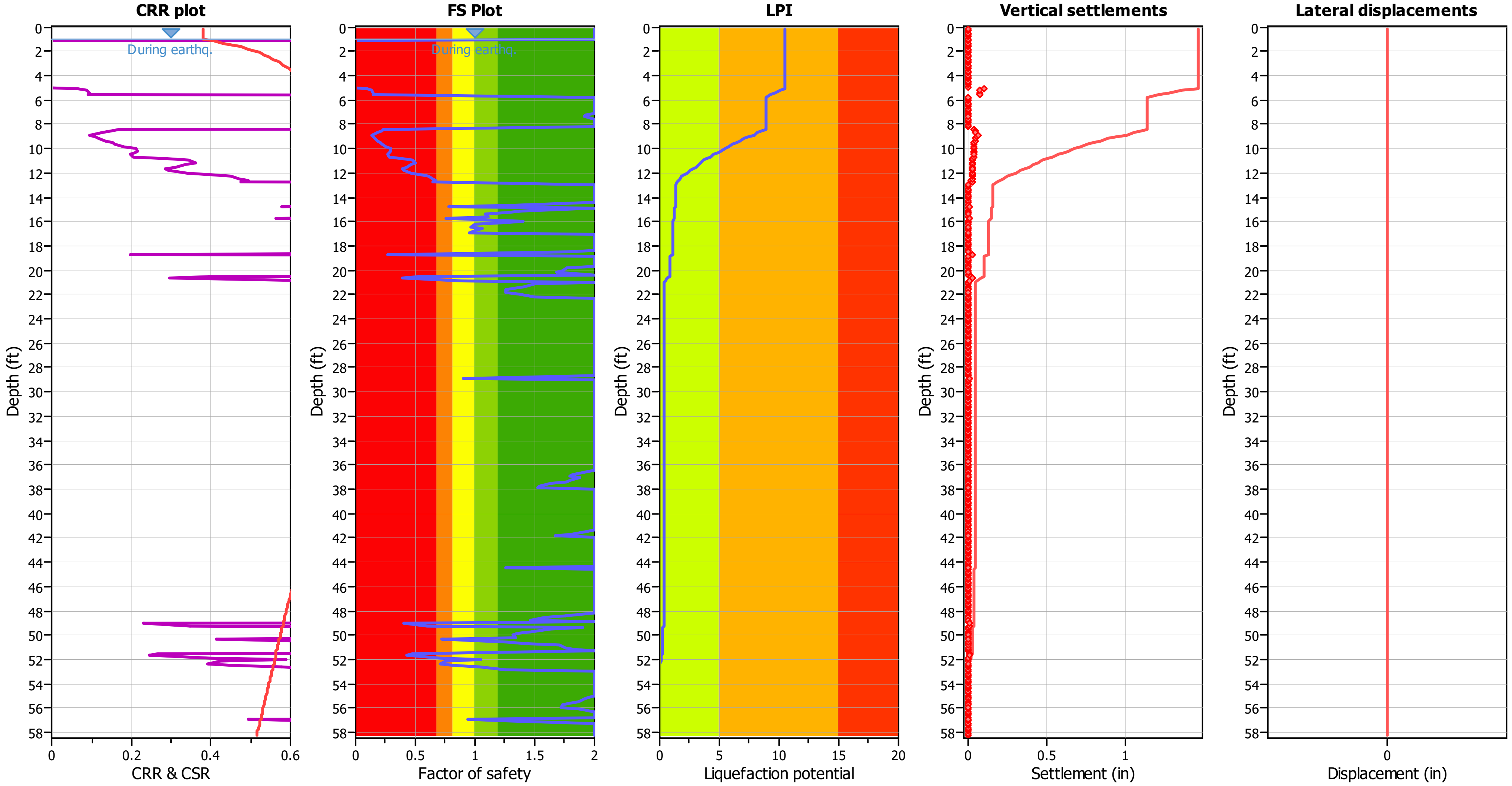
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

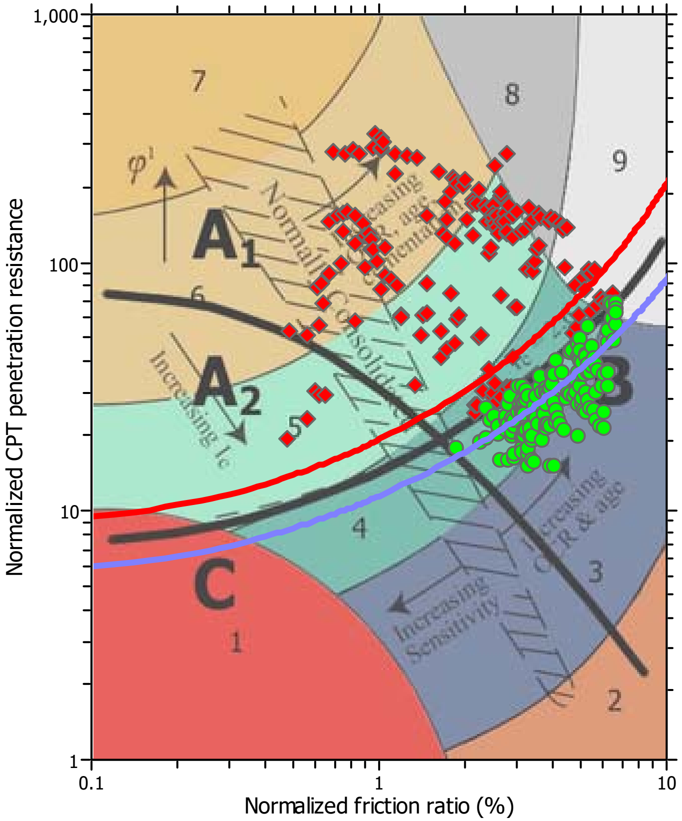
F.S. color scheme

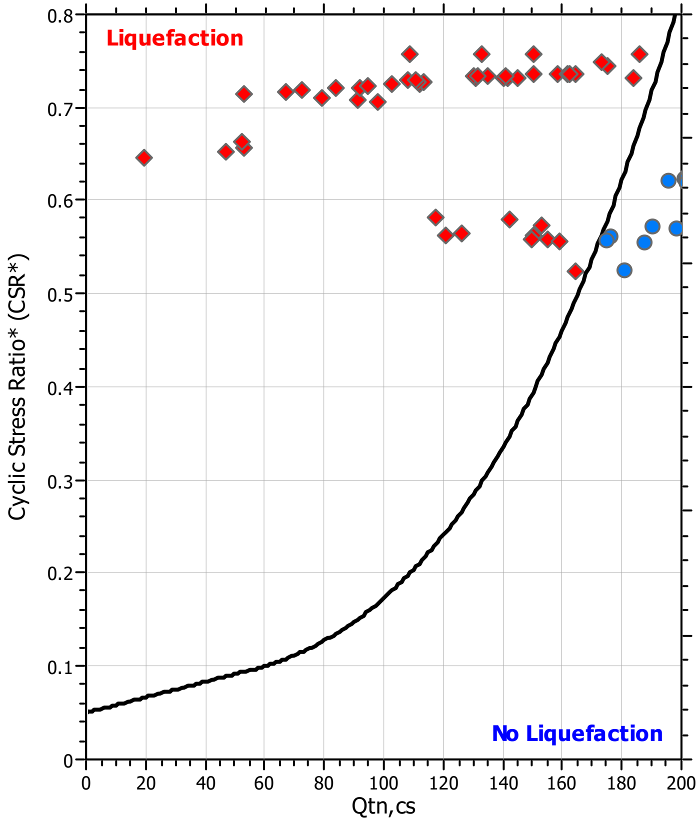
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

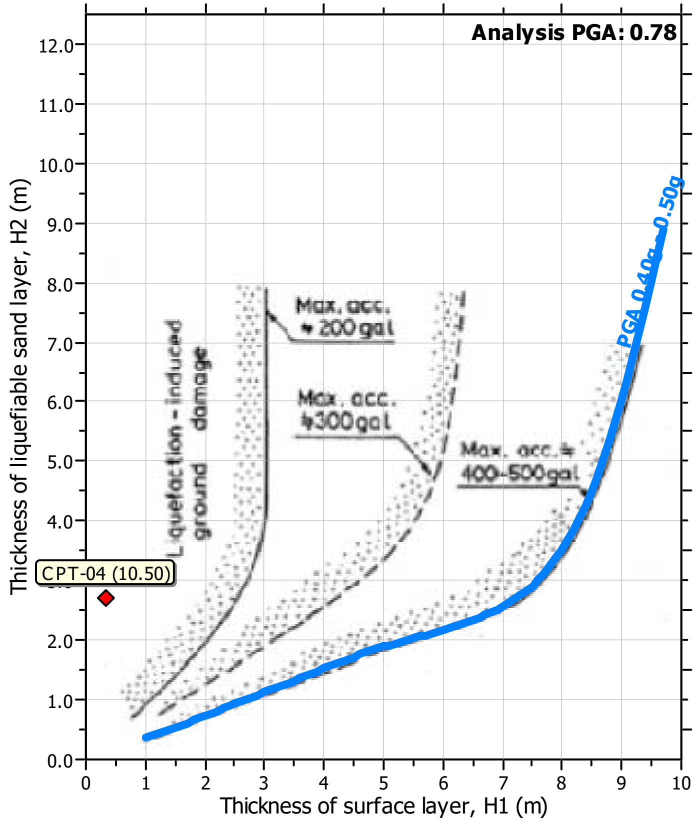
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



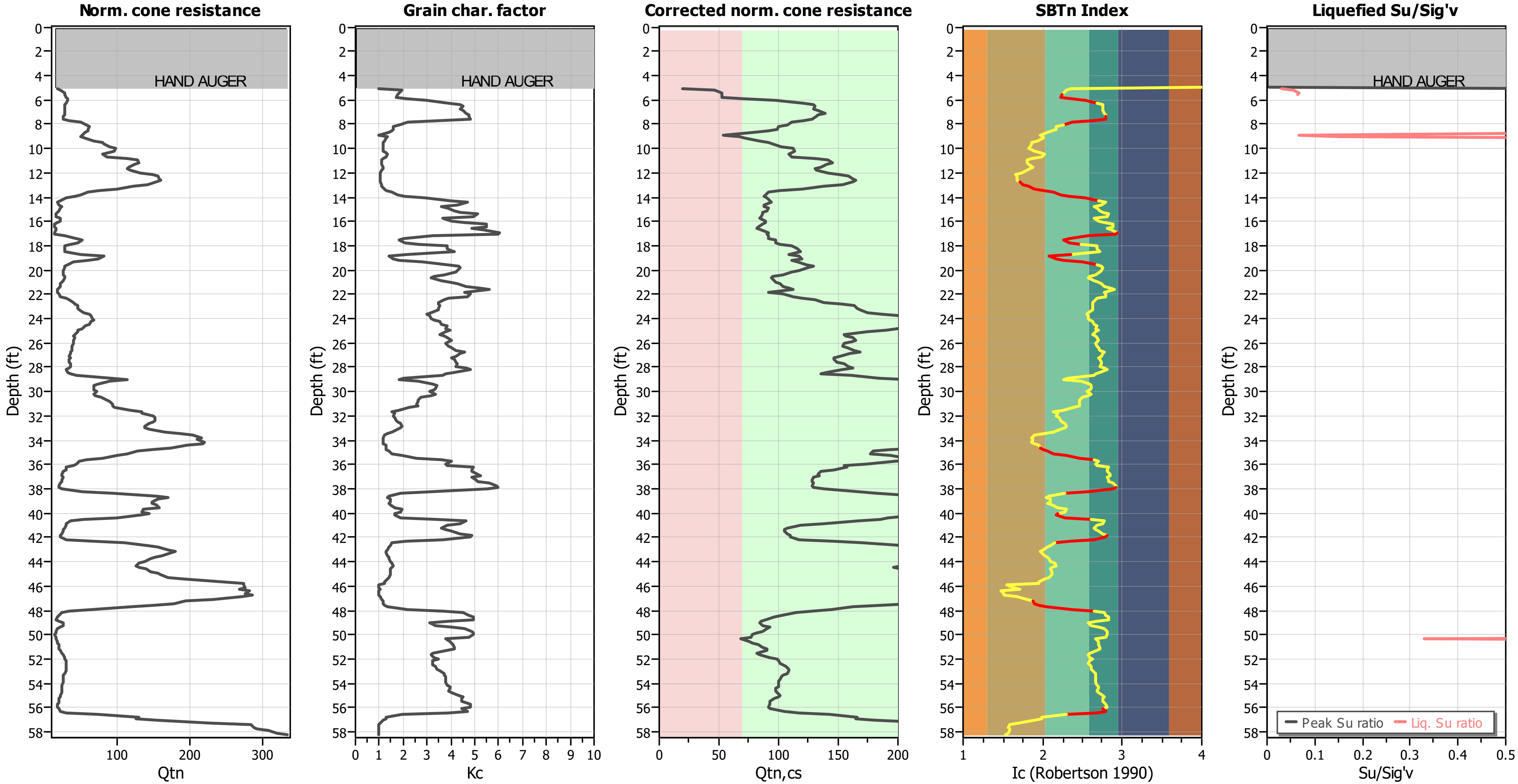




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

DRAFT

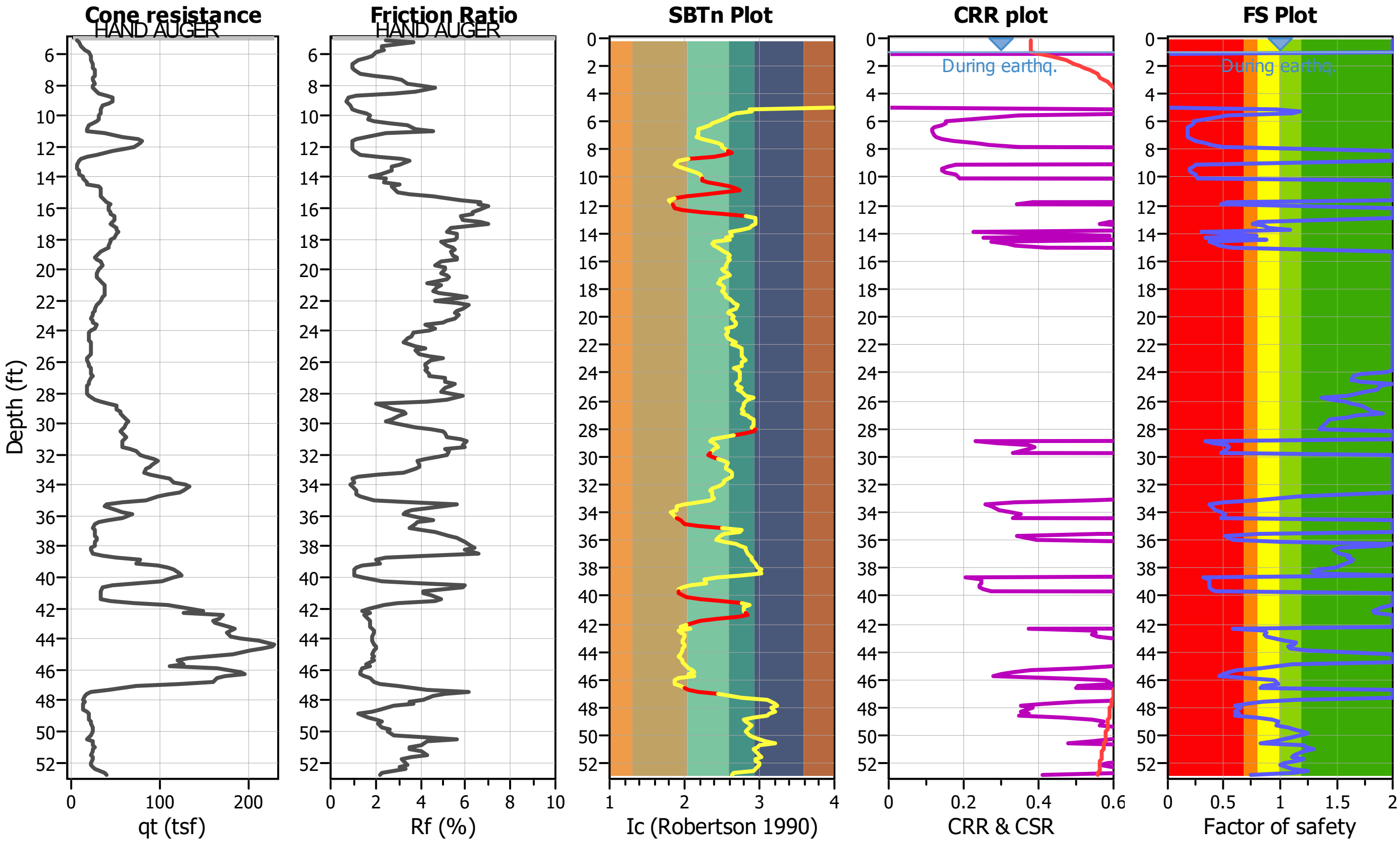
LIQUEFACTION ANALYSIS REPORT

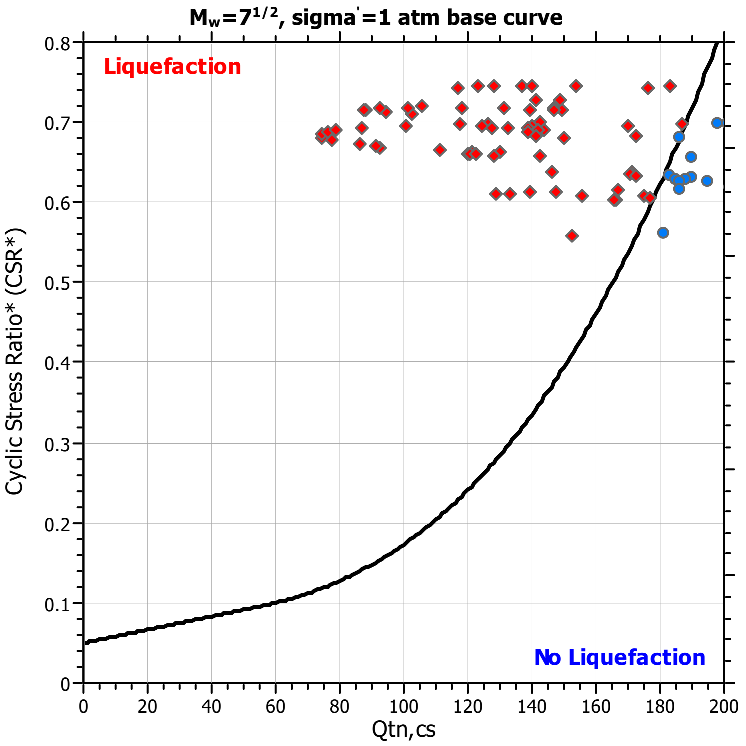
Project title : **Location :**

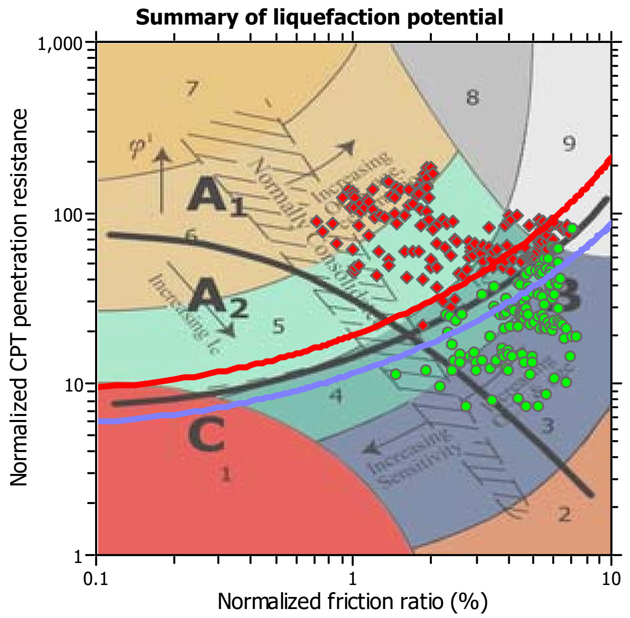
CPT file : CPT-05

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	1.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	1.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.78	Unit weight calculation:	Based on SBT	K_σ applied:	No		

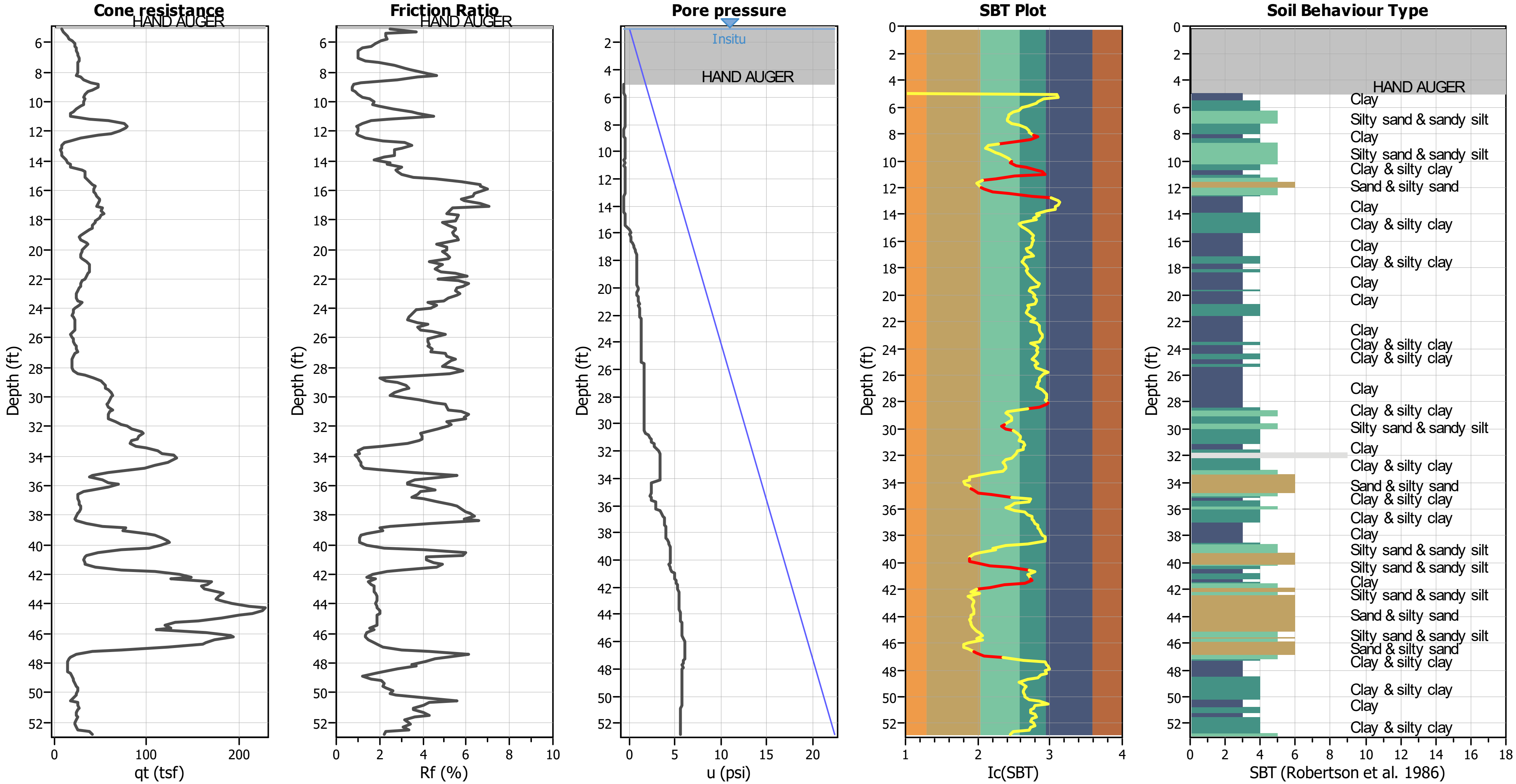






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



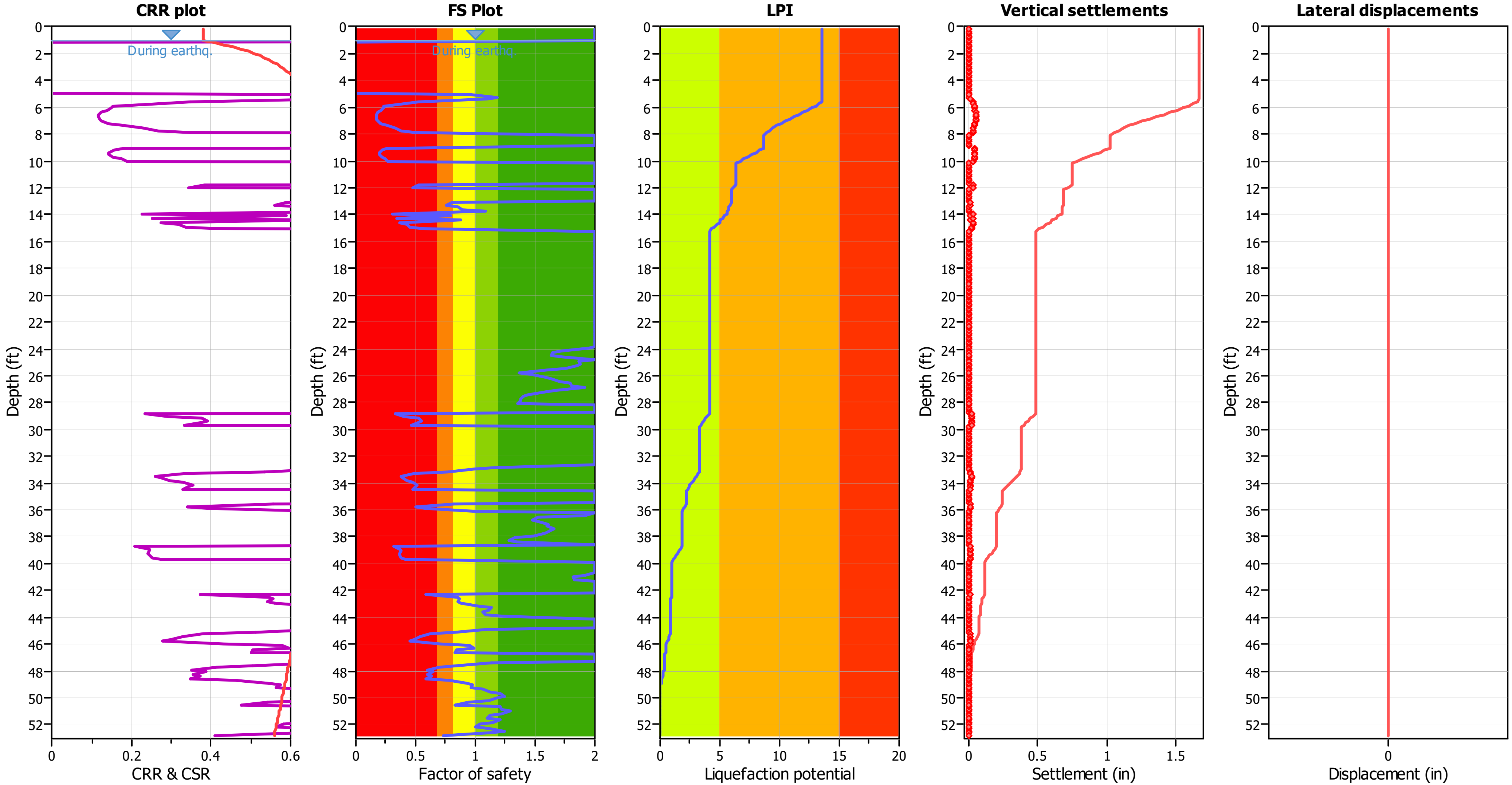
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

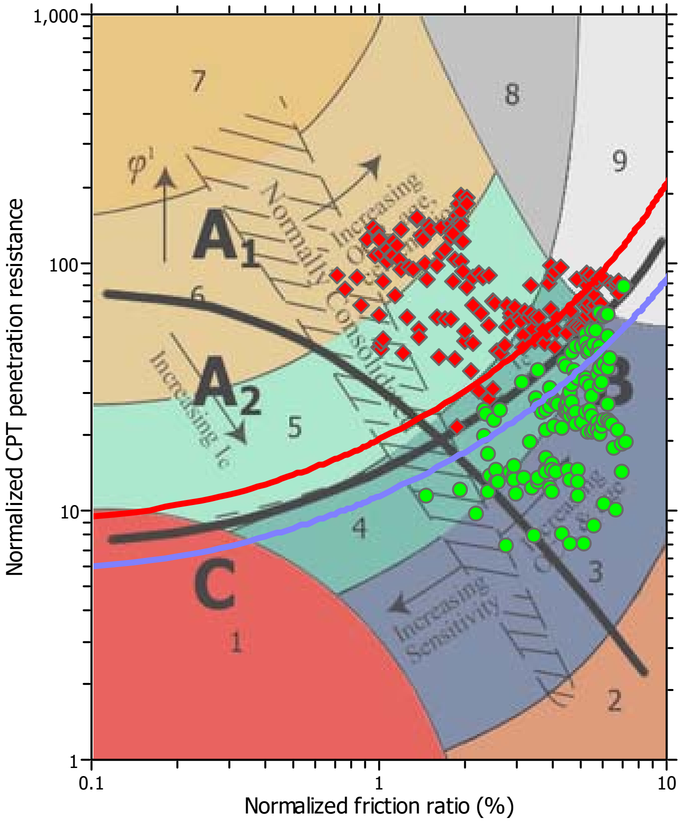
F.S. color scheme

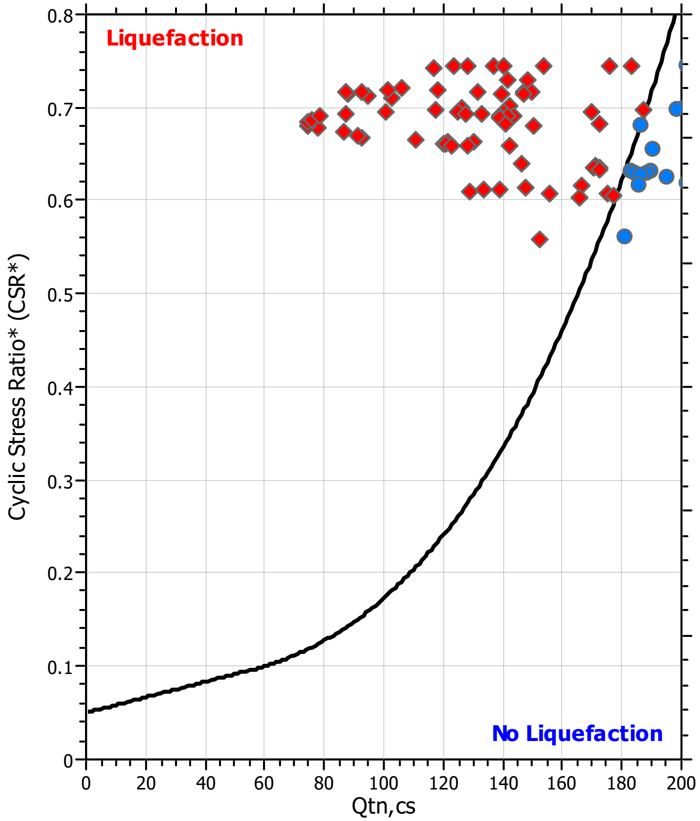
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

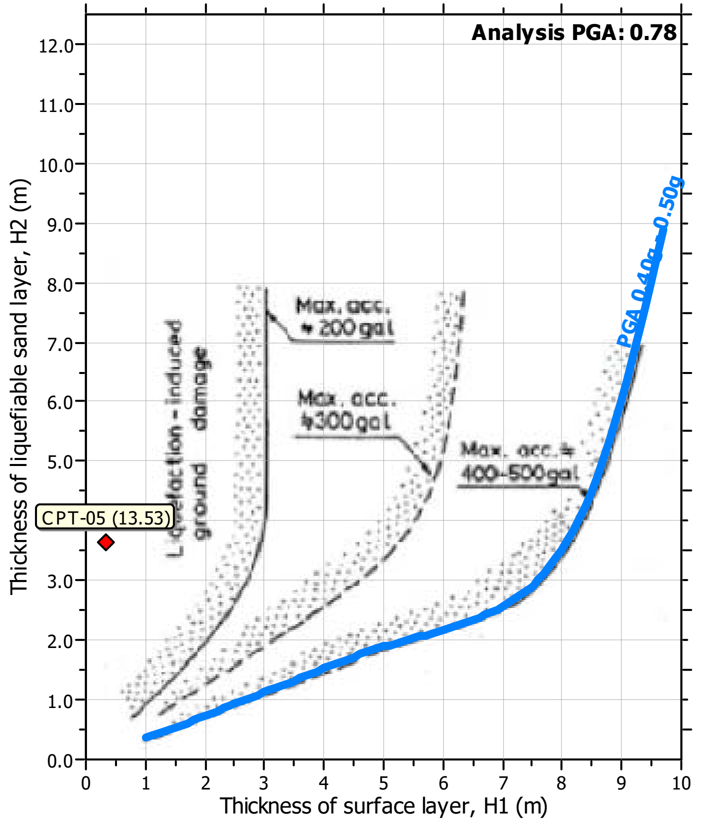
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots







Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
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Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
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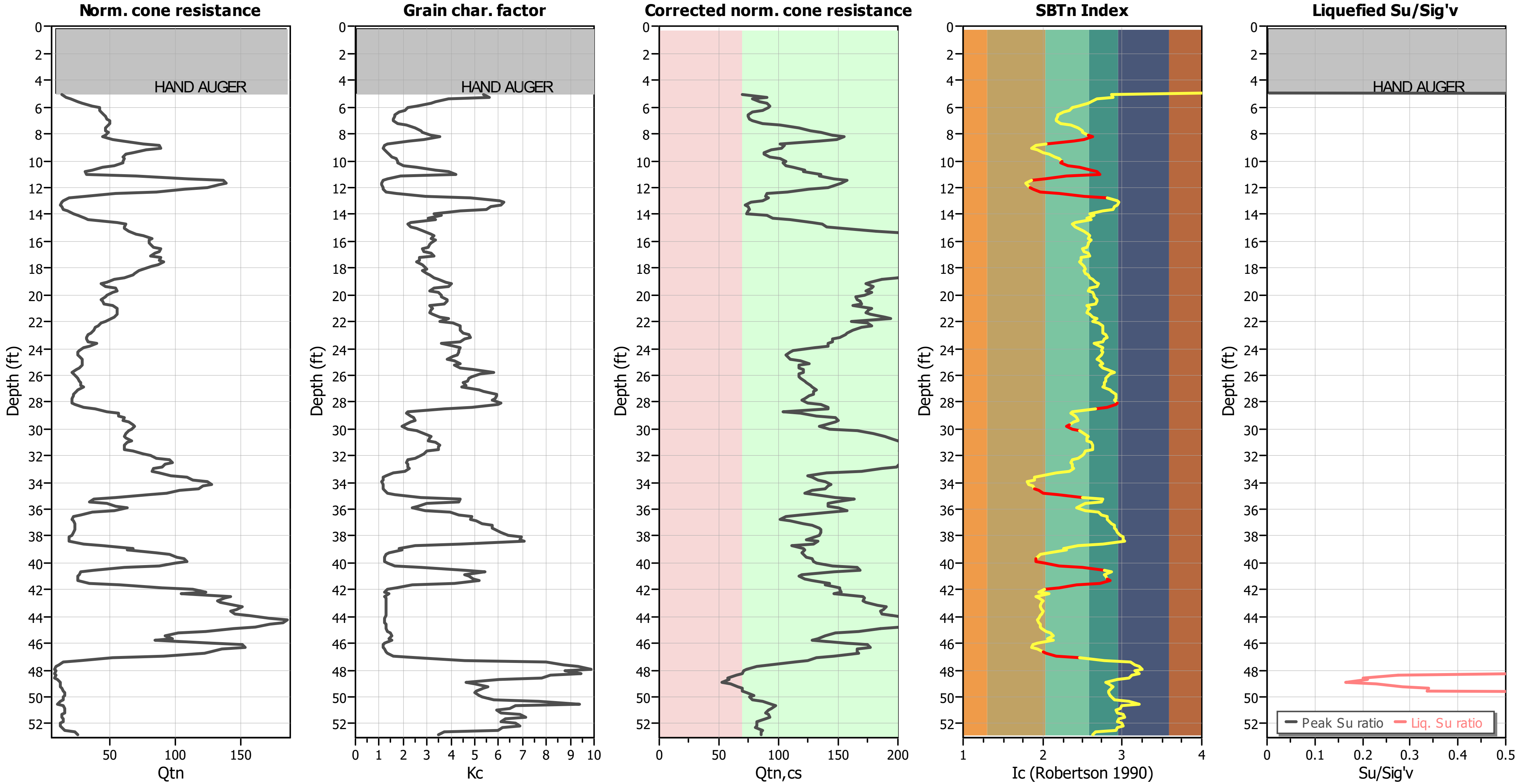
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	1.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	1	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	6.70	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.78	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	1.00 ft	Fill height:	N/A	Limit depth:	N/A

DRAFT

NTS GEOTECHNICAL

APPENDIX C-1

SPT Liquefaction Analysis

NTS GEOTECHNICAL, INC.
15333 CULVER DR., SUITE 340
IRVINE, CA 92604
WWW.NTSGEO.COM

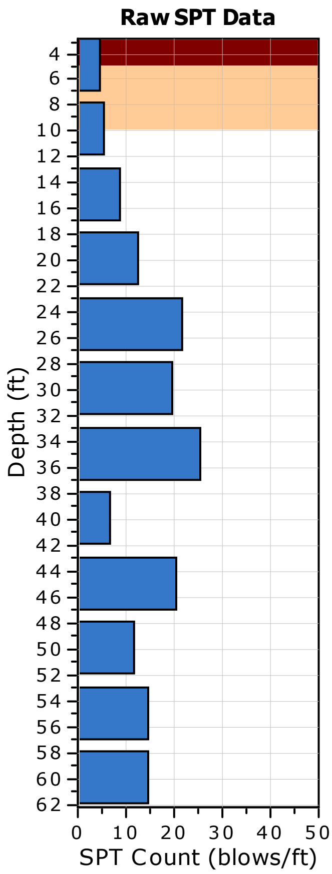
SPT BASED LIQUEFACTION ANALYSIS REPORT

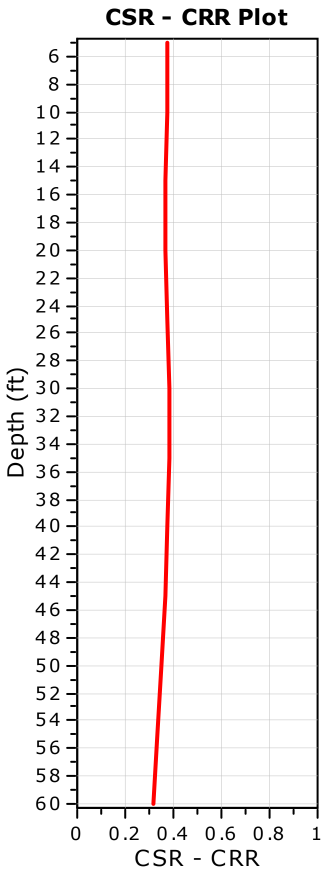
Project title : The Club at Fullerton **SPT Name: B-1**

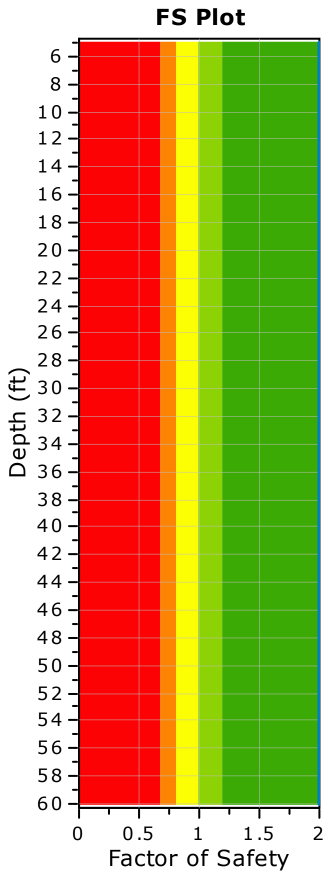
Location : 2601 - 2651 Chapman Ave

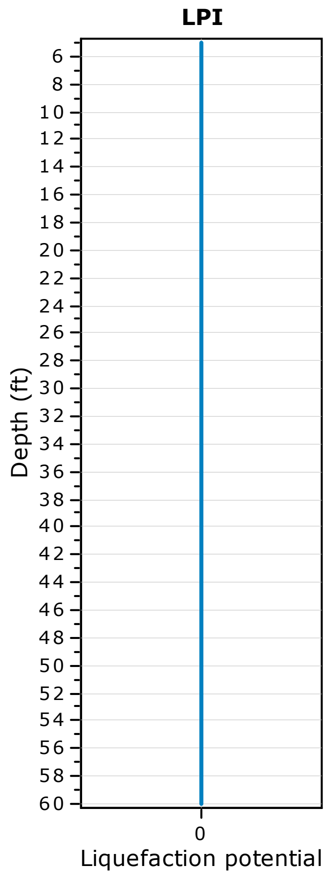
:: Input parameters and analysis properties ::

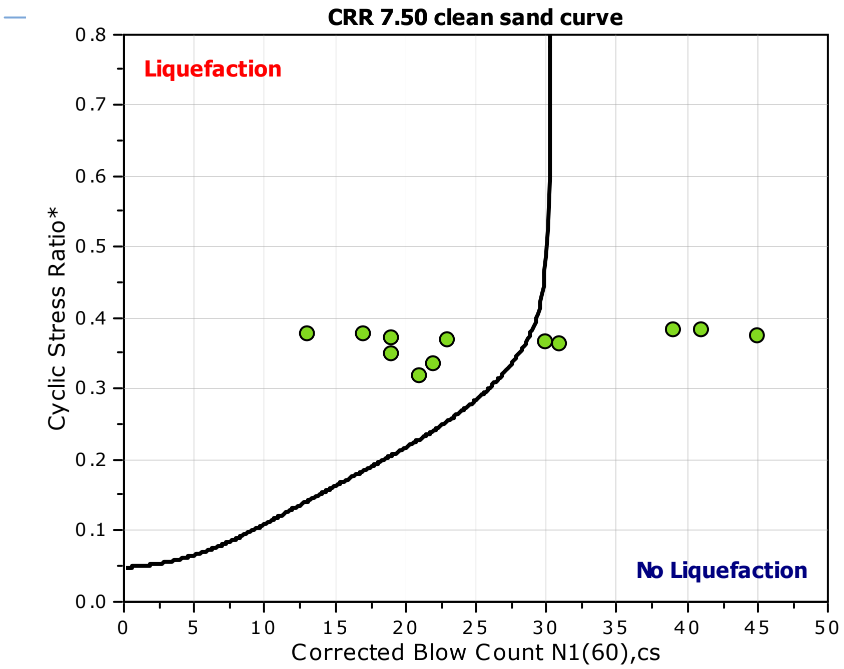
Analysis method:	NCEER 1998	G.W.T. (in-situ):	70.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	70.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	6.70
Borehole diameter:	200mm	Peak ground acceleration:	0.78 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.28		











F.S. color scheme

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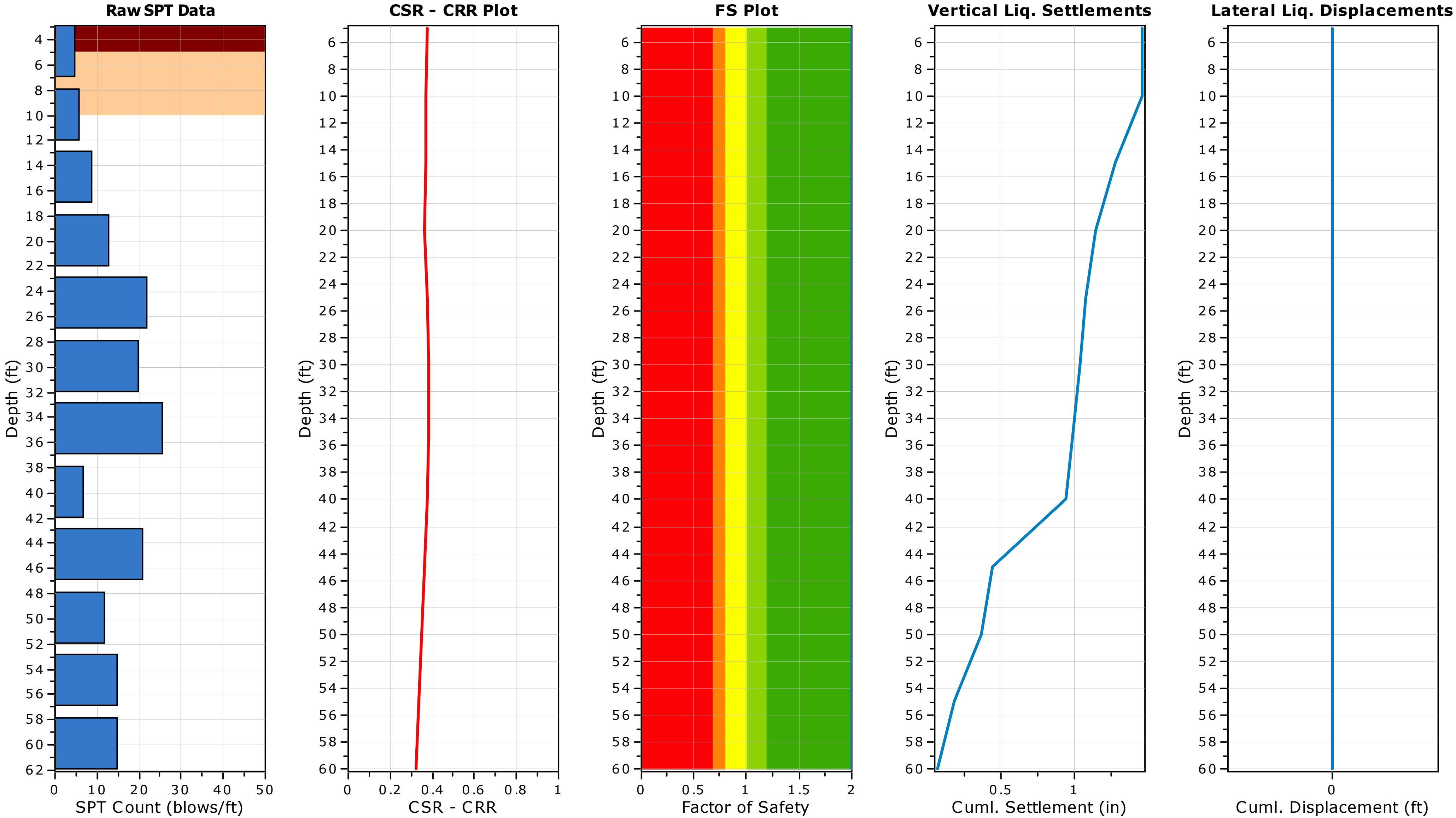
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	5	53.00	125.00	8.25	No
10.00	6	55.00	95.00	5.00	Yes
15.00	9	55.00	95.00	5.00	Yes
20.00	13	53.00	95.00	5.00	Yes
25.00	22	49.00	130.00	5.00	Yes
30.00	20	49.00	125.00	5.00	Yes
35.00	26	25.00	125.00	5.00	Yes
40.00	7	25.00	125.00	5.00	Yes
45.00	21	30.00	125.00	5.00	Yes
50.00	12	82.00	125.00	5.00	Yes
55.00	15	82.00	125.00	5.00	Yes
60.00	15	82.00	125.00	3.25	Yes

Abbreviations

Depth:	Depth at which test was performed (ft)
SPT Field Value:	Number of blows per foot
Fines Content:	Fines content at test depth (%)
Unit Weight:	Unit weight at test depth (pcf)
Infl. Thickness:	Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy:	User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	5	125.00	0.31	0.00	0.31	1.47	1.28	1.15	0.75	1.20	10	53.00	5.00	1.20	17	4.000
10.00	6	95.00	0.55	0.00	0.55	1.28	1.28	1.15	0.85	1.20	12	55.00	5.00	1.20	19	4.000
15.00	9	95.00	0.79	0.00	0.79	1.13	1.28	1.15	0.85	1.20	15	55.00	5.00	1.20	23	4.000
20.00	13	95.00	1.02	0.00	1.02	1.01	1.28	1.15	0.95	1.20	22	53.00	5.00	1.20	31	4.000
25.00	22	130.00	1.35	0.00	1.35	0.89	1.28	1.15	0.95	1.20	33	49.00	5.00	1.20	45	4.000
30.00	20	125.00	1.66	0.00	1.66	0.79	1.28	1.15	1.00	1.20	28	49.00	5.00	1.20	39	4.000
35.00	26	125.00	1.98	0.00	1.98	0.72	1.28	1.15	1.00	1.20	33	25.00	4.29	1.12	41	4.000
40.00	7	125.00	2.29	0.00	2.29	0.65	1.28	1.15	1.00	1.20	8	25.00	4.29	1.12	13	4.000
45.00	21	125.00	2.60	0.00	2.60	0.60	1.28	1.15	1.00	1.20	22	30.00	4.71	1.15	30	4.000
50.00	12	125.00	2.91	0.00	2.91	0.56	1.28	1.15	1.00	1.20	12	82.00	5.00	1.20	19	4.000
55.00	15	125.00	3.23	0.00	3.23	0.52	1.28	1.15	1.00	1.20	14	82.00	5.00	1.20	22	4.000
60.00	15	125.00	3.54	0.00	3.54	0.48	1.28	1.15	1.00	1.20	13	82.00	5.00	1.20	21	4.000

Abbreviations

σ_v:	Total stress during SPT test (tsf)
u_o:	Water pore pressure during SPT test (tsf)
σ'_{vo}:	Effective overburden pressure during SPT test (tsf)
C_N:	Overburden corretion factor
C_E:	Energy correction factor
C_B:	Borehole diameter correction factor
C_R:	Rod length correction factor
C_S:	Liner correction factor
$N_{1(60)}$:	Corrected N_{SPT} to a 60% energy ratio
α, β:	Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$:	Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$:	Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	125.00	0.31	0.00	0.31	0.99	1.00	0.502	1.33	0.376	1.00	0.376	2.000
10.00	95.00	0.55	0.00	0.55	0.98	1.00	0.496	1.33	0.372	1.00	0.372	2.000
15.00	95.00	0.79	0.00	0.79	0.97	1.00	0.491	1.33	0.368	1.00	0.368	2.000
20.00	95.00	1.02	0.00	1.02	0.96	1.00	0.485	1.33	0.364	1.00	0.364	2.000
25.00	130.00	1.35	0.00	1.35	0.94	1.00	0.478	1.33	0.358	0.95	0.376	2.000
30.00	125.00	1.66	0.00	1.66	0.92	1.00	0.467	1.33	0.350	0.91	0.383	2.000
35.00	125.00	1.98	0.00	1.98	0.89	1.00	0.452	1.33	0.338	0.88	0.383	2.000
40.00	125.00	2.29	0.00	2.29	0.85	1.00	0.431	1.33	0.323	0.86	0.377	2.000
45.00	125.00	2.60	0.00	2.60	0.80	1.00	0.407	1.33	0.305	0.84	0.366	2.000
50.00	125.00	2.91	0.00	2.91	0.75	1.00	0.382	1.33	0.286	0.82	0.350	2.000
55.00	125.00	3.23	0.00	3.23	0.70	1.00	0.357	1.33	0.267	0.80	0.334	2.000
60.00	125.00	3.54	0.00	3.54	0.66	1.00	0.334	1.33	0.250	0.79	0.319	2.000

Abbreviations

$\sigma_{v,eq}$:	Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$:	Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$:	Effective overburden pressure, during earthquake (tsf)
r_d :	Nonlinear shear mass factor
α:	Improvement factor due to stone columns
CSR :	Cyclic Stress Ratio (adjusted for improvement)
MSF :	Magnitude Scaling Factor
$CSR_{eq,M=7.5}$:	CSR adjusted for M=7.5
K_{sigma}:	Effective overburden stress factor
CSR^*:	CSR fully adjusted (user FS applied)***
FS:	Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00
55.00	2.000	0.00	1.62	5.00	0.00
60.00	2.000	0.00	0.86	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	8.25	0.000
10.00	12	0.27	0.37	724.06	0.15	9161.41	0.00	0.00	8.63	0.15	5.00	0.184
15.00	15	0.39	0.53	923.38	0.15	7386.35	0.00	0.00	8.63	0.11	5.00	0.127
20.00	22	0.50	0.69	1163.66	0.16	6305.88	0.00	0.00	8.63	0.06	5.00	0.070
25.00	33	0.64	0.90	1512.11	0.18	5345.39	0.00	0.00	8.63	0.03	5.00	0.035
30.00	28	0.78	1.11	1599.85	0.19	4717.62	0.00	0.00	8.63	0.04	5.00	0.049
35.00	33	0.89	1.32	1773.06	0.20	4254.41	0.00	0.00	8.63	0.04	5.00	0.045
40.00	8	0.99	1.53	1301.19	0.21	3895.50	0.00	0.01	8.63	0.42	5.00	0.499
45.00	22	1.06	1.74	1833.18	0.23	3607.41	0.00	0.00	8.63	0.06	5.00	0.076
50.00	12	1.11	1.95	1666.21	0.24	3369.93	0.00	0.00	8.63	0.15	5.00	0.174
55.00	14	1.15	2.16	1841.13	0.25	3170.02	0.00	0.00	8.63	0.10	5.00	0.117
60.00	13	1.18	2.37	1898.60	0.26	2998.90	0.00	0.00	8.63	0.10	3.25	0.076

Cumulative settlemetns: 1.452

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
5.00	10	44.27	0.00	8.25	0.000	0.00
10.00	12	48.50	0.00	5.00	0.000	0.00
15.00	15	54.22	0.00	5.00	0.000	0.00
20.00	22	65.67	0.00	5.00	0.000	0.00
25.00	33	80.42	0.00	5.00	0.000	0.00
30.00	28	74.08	0.00	5.00	0.000	0.00
35.00	33	80.42	0.00	5.00	0.000	0.00
40.00	8	39.60	0.00	5.00	0.000	0.00
45.00	22	65.67	0.00	5.00	0.000	0.00
50.00	12	48.50	0.00	5.00	0.000	0.00
55.00	14	52.38	0.00	5.00	0.000	0.00
60.00	13	50.48	0.00	3.25	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

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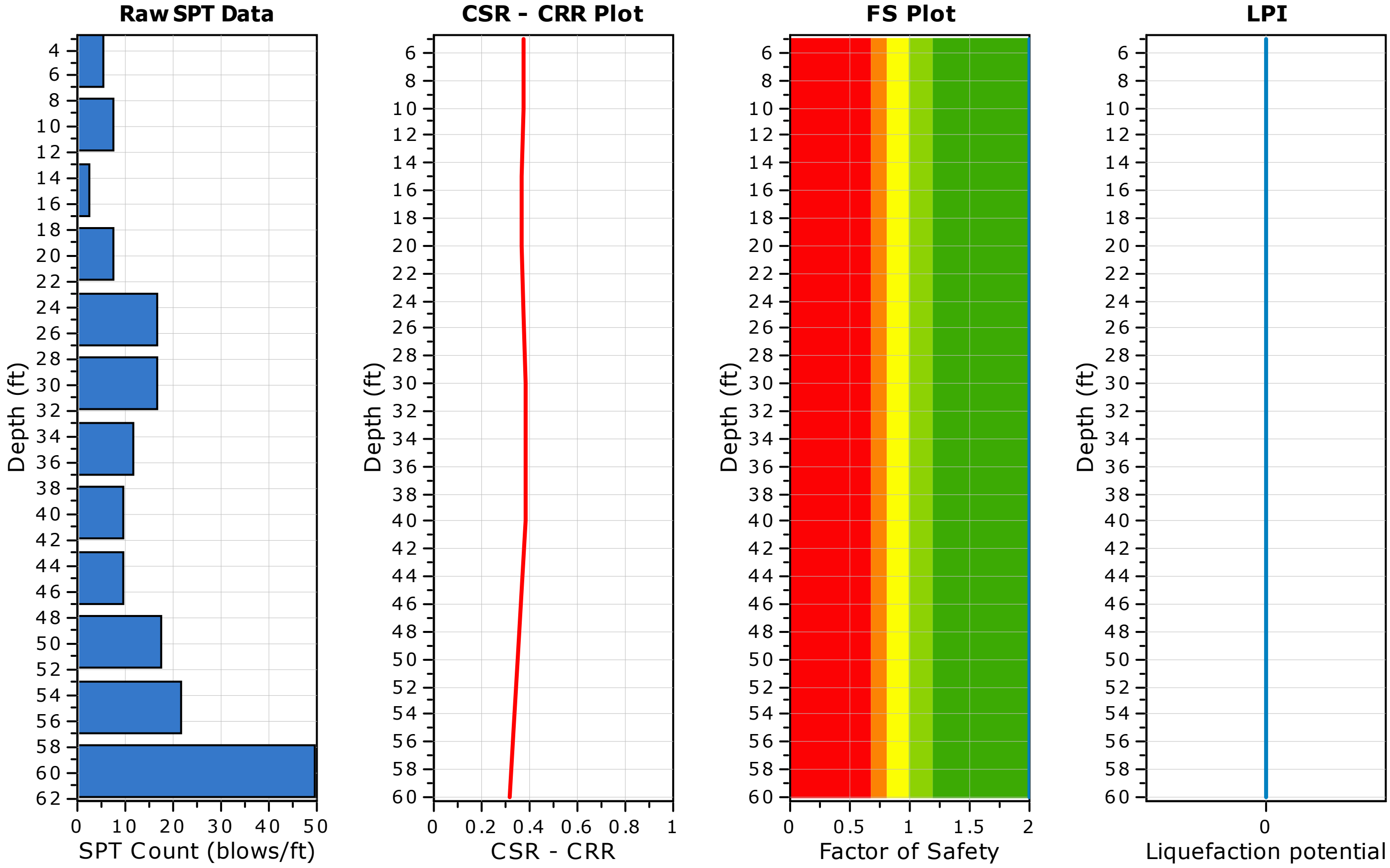
SPT BASED LIQUEFACTION ANALYSIS REPORT

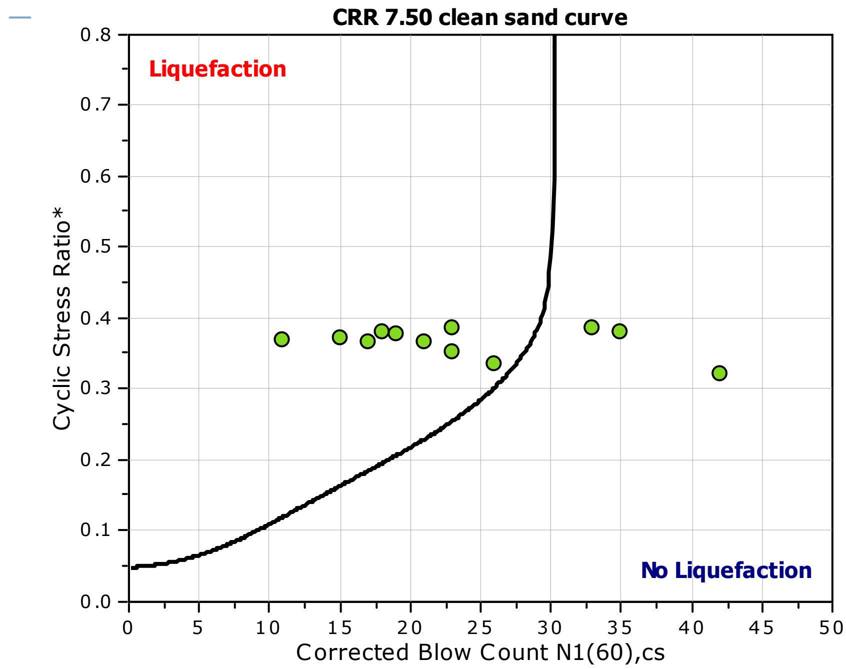
Project title : The Club at Fullerton **SPT Name: B-5**

Location : 2601 - 2651 Chapman Ave

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	70.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	70.00 ft
Sampling method:	Sampler wo liners	Earthquake magnitude M_w:	6.70
Borehole diameter:	200mm	Peak ground acceleration:	0.78 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.28		

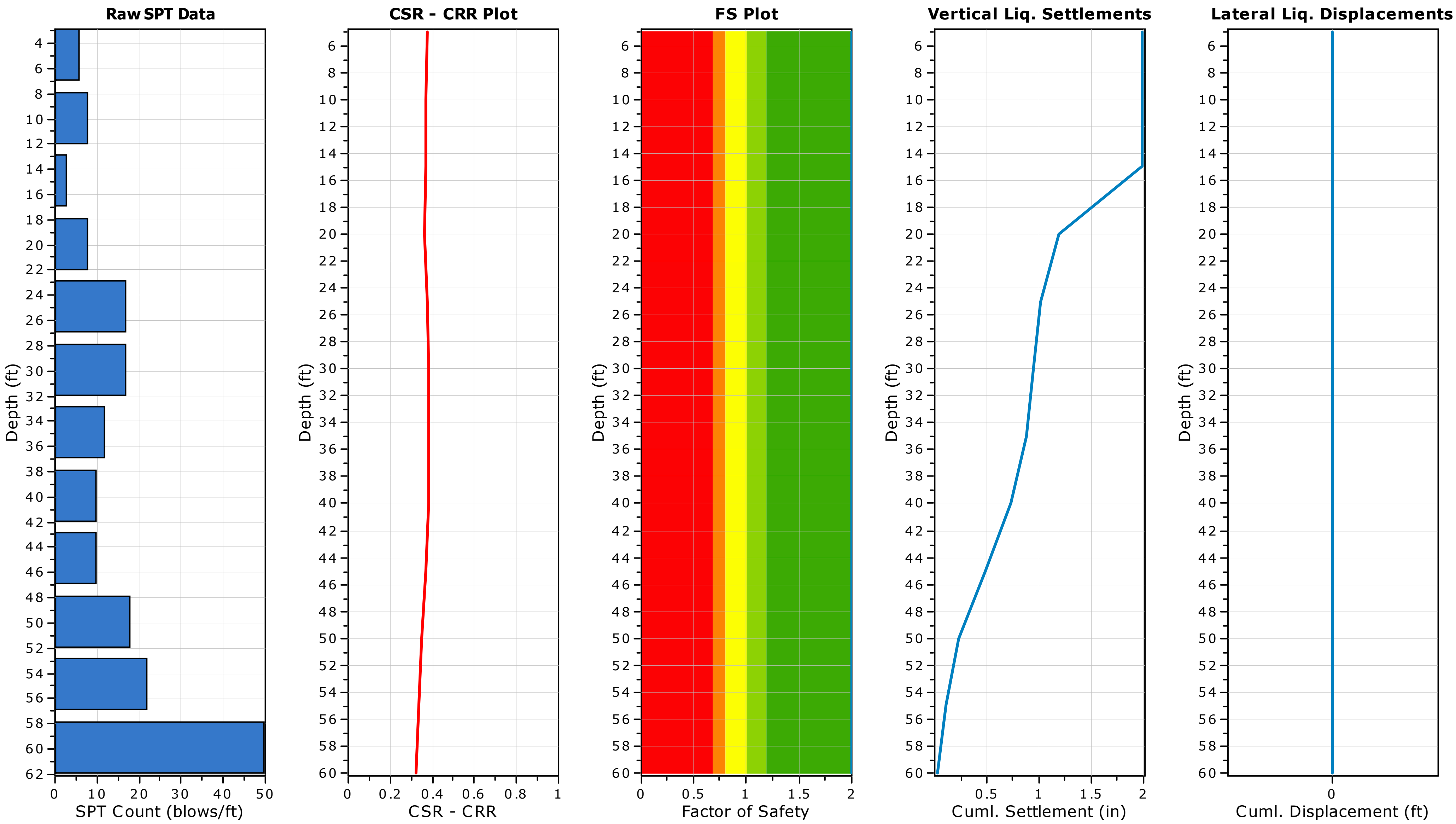




F.S. color scheme
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme
- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	6	55.00	125.00	8.25	No
10.00	8	5.00	90.00	5.00	No
15.00	3	55.00	90.00	5.00	Yes
20.00	8	59.00	130.00	5.00	Yes
25.00	17	75.00	130.00	5.00	Yes
30.00	17	49.00	125.00	5.00	Yes
35.00	12	56.00	125.00	5.00	Yes
40.00	10	56.00	125.00	5.00	Yes
45.00	10	88.00	125.00	5.00	Yes
50.00	18	24.00	125.00	5.00	Yes
55.00	22	24.00	125.00	5.00	Yes
60.00	50	5.00	125.00	5.00	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	6	125.00	0.31	0.00	0.31	1.47	1.28	1.15	0.75	1.20	12	55.00	5.00	1.20	19	4.000
10.00	8	90.00	0.54	0.00	0.54	1.29	1.28	1.15	0.85	1.20	15	5.00	0.00	1.00	15	4.000
15.00	3	90.00	0.76	0.00	0.76	1.15	1.28	1.15	0.85	1.20	5	55.00	5.00	1.20	11	4.000
20.00	8	130.00	1.09	0.00	1.09	0.99	1.28	1.15	0.95	1.20	13	59.00	5.00	1.20	21	4.000
25.00	17	130.00	1.41	0.00	1.41	0.87	1.28	1.15	0.95	1.20	25	75.00	5.00	1.20	35	4.000
30.00	17	125.00	1.73	0.00	1.73	0.78	1.28	1.15	1.00	1.20	23	49.00	5.00	1.20	33	4.000
35.00	12	125.00	2.04	0.00	2.04	0.70	1.28	1.15	1.00	1.20	15	56.00	5.00	1.20	23	4.000
40.00	10	125.00	2.35	0.00	2.35	0.64	1.28	1.15	1.00	1.20	11	56.00	5.00	1.20	18	4.000
45.00	10	125.00	2.66	0.00	2.66	0.59	1.28	1.15	1.00	1.20	10	88.00	5.00	1.20	17	4.000
50.00	18	125.00	2.98	0.00	2.98	0.55	1.28	1.15	1.00	1.20	17	24.00	4.18	1.11	23	4.000
55.00	22	125.00	3.29	0.00	3.29	0.51	1.28	1.15	1.00	1.20	20	24.00	4.18	1.11	26	4.000
60.00	50	125.00	3.60	0.00	3.60	0.48	1.28	1.15	1.00	1.20	42	5.00	0.00	1.00	42	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
C_N: Overburden coretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	125.00	0.31	0.00	0.31	0.99	1.00	0.502	1.33	0.376	1.00	0.376	2.000
10.00	90.00	0.54	0.00	0.54	0.98	1.00	0.496	1.33	0.372	1.00	0.372	2.000
15.00	90.00	0.76	0.00	0.76	0.97	1.00	0.491	1.33	0.368	1.00	0.368	2.000
20.00	130.00	1.09	0.00	1.09	0.96	1.00	0.485	1.33	0.364	0.99	0.366	2.000
25.00	130.00	1.41	0.00	1.41	0.94	1.00	0.478	1.33	0.358	0.94	0.379	2.000
30.00	125.00	1.73	0.00	1.73	0.92	1.00	0.467	1.33	0.350	0.91	0.386	2.000
35.00	125.00	2.04	0.00	2.04	0.89	1.00	0.452	1.33	0.338	0.88	0.386	2.000
40.00	125.00	2.35	0.00	2.35	0.85	1.00	0.431	1.33	0.323	0.85	0.379	2.000
45.00	125.00	2.66	0.00	2.66	0.80	1.00	0.407	1.33	0.305	0.83	0.367	2.000
50.00	125.00	2.98	0.00	2.98	0.75	1.00	0.382	1.33	0.286	0.81	0.352	2.000
55.00	125.00	3.29	0.00	3.29	0.70	1.00	0.357	1.33	0.267	0.80	0.335	2.000
60.00	125.00	3.60	0.00	3.60	0.66	1.00	0.334	1.33	0.250	0.78	0.320	2.000

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d : Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio (adjusted for improvement)
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR^*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	5.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00
55.00	2.000	0.00	1.62	5.00	0.00
60.00	2.000	0.00	0.86	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	8.25	0.000
10.00	15	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.00	0.000
15.00	5	0.37	0.51	710.55	0.15	7530.71	0.00	0.01	8.63	0.67	5.00	0.800
20.00	13	0.53	0.73	1052.69	0.17	6085.87	0.00	0.00	8.63	0.14	5.00	0.171
25.00	25	0.67	0.95	1422.42	0.18	5202.20	0.00	0.00	8.63	0.05	5.00	0.060
30.00	23	0.81	1.16	1541.38	0.19	4614.31	0.00	0.00	8.63	0.06	5.00	0.070
35.00	15	0.92	1.37	1485.26	0.20	4175.62	0.00	0.00	8.63	0.13	5.00	0.151
40.00	11	1.01	1.57	1469.95	0.22	3833.00	0.00	0.00	8.63	0.20	5.00	0.243
45.00	10	1.08	1.78	1535.11	0.23	3556.36	0.00	0.00	8.63	0.21	5.00	0.249
50.00	17	1.14	1.99	1794.72	0.24	3327.27	0.00	0.00	8.63	0.10	5.00	0.120
55.00	20	1.17	2.20	1965.33	0.25	3133.72	0.00	0.00	8.63	0.07	5.00	0.086
60.00	42	1.20	2.41	2413.12	0.26	2967.55	0.00	0.00	8.63	0.03	5.00	0.033

Cumulative settlemetns: 1.982

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
5.00	12	48.50	0.00	8.25	0.000	0.00
10.00	15	54.22	0.00	5.00	0.000	0.00
15.00	5	31.30	0.00	5.00	0.000	0.00
20.00	13	50.48	0.00	5.00	0.000	0.00
25.00	25	70.00	0.00	5.00	0.000	0.00
30.00	23	67.14	0.00	5.00	0.000	0.00
35.00	15	54.22	0.00	5.00	0.000	0.00
40.00	11	46.43	0.00	5.00	0.000	0.00
45.00	10	44.27	0.00	5.00	0.000	0.00
50.00	17	57.72	0.00	5.00	0.000	0.00
55.00	20	62.61	0.00	5.00	0.000	0.00
60.00	42	90.73	0.00	5.00	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

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DRAFT

NTS GEOTECHNICAL

APPENDIX D

Infiltration Test Result

NTS GEOTECHNICAL

Falling Head Borehole Infiltration Test

Project Name:	2601 Chapman Ave		**Date:**	4/3/2020	
Project Number:	20073		**Tested By:**	LB	
Test Hole Number:	P-1		**USCS Soil Classification:**	CL/SM	
Total Depth :	10.00	feet	**Water Temperature:**	76	°F
Test Hole Diameter:	8.00 inches	radius= 4	inches		

DRAFT

Trial	Start Time	End Time	ΔT	Total Time	Initial Depth of Water	Final Depth of Water	H_0	H_f	ΔH	Havg	Infiltration Rate
			(min)	*(min)*	*(ft)*	*(ft)*	*(in)*	*(in)*	*(in)*	*(in.)*	*(in/hour)*
1	7:55	8:25	30.0	30.0	5.58	6.16	66.96	73.92	6.96	70.44	0.38
2	8:25	8:55	30.0	60.0	6.16	6.74	73.92	80.88	6.96	77.40	0.35
3	8:55	9:25	30.0	90.0	6.74	7.32	80.88	87.84	6.96	84.36	0.32
4	9:25	9:55	30.0	120.0	7.32	8.07	87.84	96.84	9.00	92.34	0.38
5	9:55	10:25	30.0	150.0	8.07	8.65	96.84	103.80	6.96	100.32	0.27
6	10:25	10:55	30.0	180.0	8.65	9.03	103.80	108.36	4.56	106.08	0.17
7	10:55	11:25	30.0	210.0	9.03	9.40	108.36	112.80	4.44	110.58	0.16
8	11:25	11:55	30.0	240.0	9.40	9.75	112.80	117.00	4.20	114.90	0.14

WATER TEMPERATURE CORRECTION FACTOR:	0.84
SAFETY FACTOR*:	2
UNFACTORED INFILTRATION RATE (IN/HR):	0.12

Factor Category	Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) = w x v
Suitability Assessment	Soil assessment methods	0.25	3	0.75
	Predominant soil texture	0.25	2	0.5
	Site soil variablity	0.25	2	0.5
	Depth to groundwater	0.25	1	0.25
			Geotechnical Factor of Safety (SA)*:	2

***Factor of safety should not be less than 2. Additional factor of safety in accordance with Table D-7 of the South Orange County Technical Guidance Document should be applied by the project civil engineer.**

NTS GEOTECHNICAL

Falling Head Borehole Infiltration Test

Project Name:	2601 Chapman Ave		**Date:**	4/3/2020	
Project Number:	20073		**Tested By:**	LB	
Test Hole Number:	P-2		**USCS Soil Classification:**	CL/SM/ML	
Total Depth :	10.00	feet	**Water Temperature:**	76	°F
Test Hole Diameter:	8.00 inches	radius= 4	inches		

DRAFT

Trial	**Start Time**	**End Time**	**ΔT** *(min)*	**Total Time** *(min)*	**Initial Depth of Water** *(ft)*	**Final Depth of Water** *(ft)*	**H_0** *(in)*	**H_f** *(in)*	**ΔH** *(in)*	**Havg** *(in.)*	**Infiltration Rate** *(in/hour)*
1	7:55	8:25	30.0	30.0	3.58	3.83	42.96	45.96	3.00	44.46	0.26
2	8:25	8:55	30.0	60.0	3.83	4.00	45.96	48.00	2.04	46.98	0.17
3	8:55	9:25	30.0	90.0	4.00	4.25	48.00	51.00	3.00	49.50	0.23
4	9:25	9:55	30.0	120.0	4.25	4.67	51.00	56.04	5.04	53.52	0.36
5	9:55	10:25	30.0	150.0	4.67	5.03	56.04	60.36	4.32	58.20	0.29
6	10:25	10:55	30.0	180.0	5.03	5.36	60.36	64.32	3.96	62.34	0.25
7	10:55	11:25	30.0	210.0	5.36	5.69	64.32	68.28	3.96	66.30	0.23
8	11:25	11:55	30.0	240.0	5.69	6.03	68.28	72.36	4.08	70.32	0.23

WATER TEMPERATURE CORRECTION FACTOR:	0.84
SAFETY FACTOR*:	2
UNFACTORED INFILTRATION RATE (IN/HR):	0.19

Factor Category	**Factor Description**	**Assigned Weight (w)**	**Factor Value (v)**	**Product (p) = w x v**
Suitability Assessment	Soil assessment methods	0.25	3	0.75
	Predominant soil texture	0.25	2	0.5
	Site soil variablity	0.25	2	0.5
	Depth to groundwater	0.25	1	0.25
			Geotechnical Factor of Safety (SA):	2

***Factor of safety should not be less than 2. Additional factor of safety in accordance with Table D-7 of the South Orange County Technical Guidance Document should be applied by the project civil engineer.**