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GEOTECHNICAL ENGINEERING REPORT

Harbor Freight
2400 Riverside Drive
Mount Vernon, WA

PSI PROJECT NO.07121398

September 8, 2016

Prepared for:
ADA Architects, Inc.
17710 Detroit Avenue
Cleveland, Ohio

Prepared by:
Professional Service Industries, Inc.
20508 - 56th Ave W, Suite A
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September 8, 2016

ADA Architects, Inc.
17710 Detroit Avenue
Cleveland, Ohio

Attention: Brian Quinn
Phone: (216) 521-5134 x112
Email: BQuinn@adaarchitects.cc

Subject: **Geotechnical Investigation**
Harbor Freight
2400 Riverside Drive
Mount Vernon, WA
PSI Report No. 07121398

Dear Mr. Quinn

Professional Service Industries, Inc. (PSI) is pleased to submit a report based on our previous geotechnical investigation for the proposed Harbor Freight to be located at 2400 Riverside Drive in Mount Vernon, Washington. This report summarizes the work accomplished and provides our geotechnical recommendations and conclusions for support of the proposed improvements.

Based on the results of our field investigation, laboratory testing and engineering analysis, the proposed site is suitable for the construction of the proposed improvements from a geotechnical standpoint, provided the recommendations of this report are followed. Recommendations regarding the geotechnical aspects of project design and construction are presented in the attached report.

PSI appreciates the opportunity to contribute our services and looks forward to working with you during design and construction of this project. Please contact the undersigned directly if you have questions pertaining to this project.

Respectfully Submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.

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FIGURES

LIST OF APPENDICES

1. PROJECT DESCRIPTION

PSI was informed that due to cost associated with the renovations being so high total teardown of the old building and construction of a new 15,000 square foot building is planned. PSI understands that because of the type of remodeling work being conducted a geotechnical investigation of the site is required. PSI understand that the existing building was built prior to many of the modern building codes and structural improvement will be required to bring the building up to the 2012 International Building Code standards. Based on the structural load provided to PSI, column and wall loads will be on the order of 10 kips and 2 kips per foot, respectively. We also anticipate design floor loads of about 150 psf.

2. SITE DESCRIPTION

2.1. General

The site is located at 2400 Riverside Drive in Mount Vernon, Washington. Based on readily available aerial images and observations during our site visit, the site has an existing single story building with a large parking lot surrounding the building. The site is bound by Riverside Drive to the west, Hoag Road followed by the Skagit River to the north, a trailer park to the east, and Pacific Place to the south.

2.2. Topography

Based on available topographic information the site is relatively level and is at an approximate elevation of 30 feet above the mean sea level.

2.3. Geology

Based upon a review of Washington State Department of Natural Resources Interactive Maps (Reference 1) and the results of our field investigation the site is underlain by alluvium deposits. Alluvium in this area typically consists of fine to medium grained sands and silts deposited by moving water.

2.4. Subsurface conditions

Subsurface materials and conditions were investigated with four hand augur borings, designated HAB-1, HAB-2, HAB-3 and HAB-4; 2 Cone Penetrometer Tests (CPTu) and One Seismic Cone Penetrometer Test (SCTPu), designated CPT-1 through CPT-3, on June 7, 2016. The hand auger borings were drilled to a depth of approximately 10 feet while the CPTu's were pushed to depths ranging from 58 to 71 feet below existing ground surface (bgs). The approximate locations of the soil borings, the SCPTu and CPTu's are shown on Figure 2. In general, the soils around the proposed building areas were alluvial deposits consisting of very loose to medium dense sands and very soft to stiff silts within the upper 50 to 60 feet with dense sands underlying them. A description of our field investigation, our boring logs, along with the SCPT and CPT data, and General Notes used to describe materials encountered in the boring logs, are available in Appendix A. A description of the laboratory testing program along with sample test results are available in Appendix B.

2.5. Groundwater

Groundwater was calculated at the site at a depth of approximately 10 feet bgs at the time of our field investigation. PSI anticipates that the groundwater table fluctuates seasonally and in response the water level in the Skagit River and to significant precipitation events.

2.6. Seismic Design Values

The nearest mapped fault zone to the site is the Devils Mountain Fault Zone approximately 5.4 miles south of the site. The Devils Mountain Fault Zone is mapped as a late Quaternary age thrust fault, with a northward dip direction and a slip rate on the order of less than 0.2 millimeters per year (Reference 2).

As part of the procedure to evaluate seismic forces, the 2012 IBC requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface. To help define the Site Class for this project PSI utilized shear wave velocities obtained from the Seismic Cone Penetrometer Tests for the upper 60 feet of the site. Based on the obtained shear wave velocities of the site the seismic site class classifies as a site class “D” soil, however since more than 10 feet of potentially liquefiable soil exists on the site the site classifies as a Site Class “F” as defined in Table 20.3-1 of ASCE 07-10. However, the exception in Section 20.3.1 of ASCE 07-10 permits the Site Class to be determined in accordance with Section 20.3 and the corresponding values of Fa and Fv determined from Tables 11.4-1 and 11.4-2. Based on this exception, Site Class E seismic design coefficients can be used and are provided. The associated USGS-NEHRP (2009) probabilistic ground acceleration values and site coefficients for the general site area were obtained from the USGS geo-hazards web page (Reference 3). The calculated seismic design Parameters for an earthquake with a risk targeted 2 percent probability of exceedance in 50 years are presented in Table 1 below:

Table 1: Ground Motion Values*

Period (sec)	Mapped MCE Spectral Response Acceleration (g)		Site Coefficients		Adjusted MCE Spectral Response Acceleration (g)		Design Spectral Response Acceleration (g)	
0.2	S_s	1.071	F_a	0.900	S_{Ms}	0.964	S_{Ds}	0.642
1.0	S_1	0.416	F_v	2.400	S_{M1}	1.0	S_{D1}	0.666

**Risk Targeted 2% Probability of exceedance in 50 years for Latitude 48.44227 and Longitude -122.33436*
MCE = Maximum Considered Earthquake

If the Site Class, as determined from the intended building use and the IBC, is interpreted to be C, D, E or F, the code requires an assessment of slope stability, liquefaction potential, and surface rupture due to faulting or lateral spreading. The following table presents a qualitative *assessment* of these issues considering the site class, the subsurface soil properties, the groundwater elevation, and probabilistic ground motions:

Table 2: Qualitative Seismic Site Assessments

Liquefaction	High	Our liquefaction analysis shows a high probability of seismic induced liquefaction occurring on this site.
Slope Stability	Low	The site is relatively flat with no observed steep slopes in close proximity to the site.
Surface Rupture	Low	No known active faults underlie the site.

2.7.2. Liquefaction Potential

In general, liquefaction is a condition where soils lose intergranular strength due to abrupt increases in pore water pressure. Pore water pressure increases typically occur during dynamic loading such as ground shaking during a seismic event. Liquefaction, should it occur on a site, can induce ground settlement and lateral spreading, which can result in damage to the structures. For liquefaction to occur, the following conditions must be present:

- The soil sediments must be in saturated or near-saturated conditions. At least 80-85 percent saturation is generally considered necessary for the liquefaction to occur.
- The soil must be predominately composed of non-plastic material such as sand or silt.
- The soil must be in a relatively loose state.
- The soil must be subjected to dynamic loading, such as an earthquake.

Based on the subsurface conditions encountered at the site, the potential for liquefaction is considered to be high at the site during a seismic event due to very shallow groundwater and loose sands with low fines content. The site is mapped as having a high liquefaction potential, based on the Washington State Department of Natural Resources Interactive Maps of the area (Reference 1). More information of liquefaction potential and settlement for the site is discussed in section 3.6 of this report.

The estimated liquefaction settlement analysis has been performed based on worst-case scenarios with conservative modeling equations and parameters. Results of our studies indicate that the soils from approximately 10 to 50 feet below ground surface would liquefy under a strong earthquake of magnitude 7.01 at a maximum considered earthquake acceleration of 0.43g, based on data obtained from the USGS 2008 interactive Deaggregations tool (Reference 4). This is illustrated in the liquefaction analysis summary in the Appendix C.

Based on our analysis of the soils encountered during our investigation, the soils encountered are susceptible to liquefaction, with a potential for liquefaction-induced settlement on the order of approximately 3½ to 5½ inches during a major seismic event with the liquefaction occurring between 10 and 50 feet bgs. Base on the data from the three CPT locations PSI anticipates differential liquefaction settlements to be on the order of approximately 2 inches over a 100-foot span.

3. CONCLUSIONS AND RECOMMENDATIONS

3.1. General

Our previous subsurface explorations for this investigation indicate the presence of approximately 50 feet of potentially liquefiable soil across the site. The groundwater table is located at a depth of at least 10 feet and the anticipated liquefaction that may occur resulting from the design earthquake is anticipated to result in 2 inches of differential settlement across a 100-foot span. We understand that the existing building appears to be supported by conventional spread footings and the expected loads for the new building will be on the order of 10 kips and 2 kips per linear foot for column and perimeter footing respectively.

3.2. Site Preparation

We anticipate that the removal of existing structures, foundations and utilities will disturb the upper 2 to 4 feet of soils across the site. Any large debris encountered below the existing site structures should be removed. Once the existing site structures are removed the site surfaces should be compacted to provide suitable access for equipment. Compacted soils should be proof rolled using a loaded tandem axle dump truck. If the surface fails the proof roll and cannot be repaired suitably to allow for heavy equipment, or if the work is to occur during the wet season, then it may be necessary for the upper 12 inches of soil to be removed and replaced with a crushed rock that has been approved by PSI.

3.3. Structural Fill

All fill placed beneath building, sidewalk, and pavement areas should be installed as compacted structural fill. The onsite soils are suitable for use as structural fill, provided they can be suitably moisture conditioned to meet the required compaction results. We recommend that imported structural fill should consist of pit-run or quarry-run rock, crushed rock, crushed gravel, or sand. It should be fairly well-graded between coarse and fine material and have less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. The material should be placed in lifts with a maximum un-compacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density as determined by ASTM D1557.

The condition of the subgrade should be evaluated by a PSI representative before fill placement or construction begins. Fill compaction should be evaluated by in-place density tests performed during fill placement so that adequacy of soil compaction efforts may be evaluated as earthwork progresses.

3.4. Utility Trench Excavations and Backfill

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

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

Utility trenches within the building, pavement, and sidewalk areas should be backfilled with granular structural fill such as the onsite soil that can be properly compacted, or imported sand, sand and gravel, fragmental rock, or recycled concrete of up to 2 inches' maximum particle size with less than 5 percent passing the No. 200 sieve (washed analysis). Granular backfill should be placed in lifts and compacted to not less than 95 percent of the maximum dry density as determined by ASTM D 1557.

3.5. Foundations

The site is anticipated to experience up to 5½ inches of total settlement and up to 2 inches of differential settlement as a result of liquefaction during the maximum conceived design event. Several foundation options are presented below to mitigate the sites seismic and subsurface soil conditions. In each foundation option PSI recommends that exterior footings extend at least 18 inches below existing site grades to protect against frost heave.

3.5.1. Shallow Spread footing

If the liquefaction induced settlement is determined to be within acceptable limits any new foundation elements can be founded on at least 24 inches of suitably compacted structural fill. This may include an over-excavation of two feet of soil immediately below the proposed footings and compaction of either suitable onsite soils or approved imported materials as structural fill in accordance with section 3.3 of this report. A bearing pressure of 1,500 psf can be used for these footings. This value applies to the total of dead load and/or frequently applied live load and can be increased by one-third for the total of all loads; dead, live and wind or seismic. Static settlement with this option have been calculated to yield less than 1 inch of total settlement with an anticipated differential settlement of less than ½ inch over a 40 foot span.

3.5.2. Mat Foundation

Mat foundations can be used to support the proposed building. Mat foundations do not resist total settlement, but can be effective at limiting differential settlement since they can be designed to bridge over the estimated static and seismically-induced settlements. The material removed from the demolition would need to be removed prior to any additional work taking place. If the methods described in this section are performed PSI anticipates that differential settlements on this site will be on the order of less than ½ inch over a 40 foot span.

PSI recommends that a minimum 12 inches of soil immediately below the mat foundation be structural fill compacted to at least 95% of modified proctor (ASTM D1557), or native soil compacted to a firm and unyielding state and observed by a representative of the geotechnical engineer. If soft or loose soils are encountered at the subgrade, over-excavation for one additional foot may be required. The over-excavation and re-compacted areas should extend at least 5 feet beyond the maximum lateral extent of the footing elements.

Allowable Bearing Pressure

The mats should be founded a minimum 1½ feet below the lowest exterior site grade. The bearing capacity of large mats is not the governing criteria for design. The settlement of the mat usually governs the allowable load on the mats. We evaluated the proposed mat foundation for limiting the static settlement to less than 1/2-inch based on an allowable bearing pressure of 1,000 psf. The allowable load can be increased if the structure can tolerate higher settlements. Seismically-induced settlements should be added to the static settlement in the mat foundation design. Maximum differential settlements are expected to be less than half of the total settlement.

Coefficient of Subgrade Reaction

The coefficient of subgrade reaction (Ks) is the unit pressure required to produce a unit settlement in soils. The Ks is generally used for the structural design of the mat foundation. Factors such as size of foundation and shape affect the value of Ks. A general equation to include the effect of size for square footings on granular soils is given by:

$$K_s = K_1 \left[\frac{B+1}{2B} \right]^2 \text{ (Reference : Bowles,1988)}$$

where, B = width of footing in feet
K_1 = Coefficient of subgrade reaction for a one-foot square footing.

For mat foundation over compacted fill, K1 may be taken as 200 pounds per cubic inches (pci), provided subgrade soil are prepared in the manner discussed in this report.

Resistance to Lateral Loads

Resistance to lateral loads can be provided by passive earth pressure against the side of mat foundations and by friction at the base.

Passive earth pressure may be used for the sides of mats poured against properly compacted fill or competent site soils. An equivalent fluid pressure of 250 psf can be used for ultimate passive resistance, not to exceed 3,500 psf. These values do not include a safety factor. Top one foot of passive resistance should be neglected unless the soil is confined by pavement or slab.

An ultimate friction coefficient of 0.4 can be used between the contact of concrete mat and compacted sandy soils. Friction should be applied to net dead normal load only. A minimum factor of safety of 1.5

and 1.1 should be used for sliding resistance for static and seismic cases, respectively. If passive pressure and friction are combined when evaluating the lateral resistance of a mat foundation, a factor of safety of 1.5 should be used to reduce the contribution from passive pressure.

3.5.3. Grade Beams

Grade beams may be used to interconnect footing element of the building and restrict the differential settlement of structure. Grade Beams can be designed with an allowable bearing pressure of 1,500 psf. This value applies to the total of dead load and/or frequently applied live load and can be increased by one-third for the total of all loads; dead, live and wind or seismic. Beams should bear on at least 12 inches of suitably compacted structural fill or firm and unyielding native soil. This subgrade should be observed by the geotechnical engineer prior to grade beam installation. Beams should be installed separate from the slab on grade floors to limit the amount of cracking resulting from differential settlements which may occur between them. Recommendations for resistance to lateral loading of grade beams are the same as for the mat foundation, see section 3.2.1. If the methods described in this section are performed PSI anticipates that differential settlements on this site will be on the order of less than ½ inch over a 40 foot span

3.6. Floor Support

We recommend that floor subgrades be proof rolled to verify subgrade suitability and/or observed by the Geotechnical Engineer or their representative prior to additional fill placement. PSI recommends the installation of an 8-inch thick granular base course beneath the floor slab to provide uniform support and a capillary break between the slab and the subgrade soil, the capillary. The base course should consist of crushed rock of up to 1 inch size and having less than about 2% passing the No. 200 sieve (washed analysis). Crushed rock ¾- to ¼ -inch gradation is often used for this purpose. The base course material should be installed in a single lift and compacted to at least 95% of the maximum density as determined by AS1M D 1557. In our opinion, it is appropriate to assume a coefficient of subgrade reaction, k, of 200 pci for the design of floor slabs constructed as recommended above. It may also be appropriate to install a vapor-retarding membrane beneath slabs that will receive floor coverings or will be used to store moisture-sensitive materials. The membrane should be installed in accordance with manufacturer's recommendations. Unless the mat foundation option is selected, PSI recommends that slab on grade floors be placed independent of footings to limit any damage that may result from differential settlement between the floors and the footings.

3.7. Drainage

We recommend footing drains be placed around the exterior of the building foundation to reduce the potential for lateral migration of moisture into the building envelope. We recommend that all roof drains be connected to a tight-line pipe leading to storm drain facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend that ground surfaces adjacent to buildings be sloped to facilitate positive drainage away from the buildings.

PSI recommends that any infiltration system used on this site be placed at least 4 feet above the groundwater table, which will limit the depth the infiltration system to no more than 6 feet below existing site grades.

3.8. Pavement

For automobile parking areas, we recommend a pavement section consisting of 3 inches of asphaltic concrete (AC) over 6 inches of crushed rock base (CRB). For truck traffic areas, the pavement section should consist of 4 inches of AC over 8 inches of CRB. These preliminary pavement sections are based on a pavement design using the site sand subgrade, a desired pavement life of 20 years, and a terminal serviceability index of 2.0. The pavement section described above are based on ESAL's of 2,600 and 42,000 for automotive parking areas and truck traffic areas respectively. If concrete pavements are to be used in truck traffic areas or near site dumpsters, the section should consist at least a 5 inch section of concrete with at least 4 inches of CRB below it. Concrete used should have a 28 day break strength of at least 4,000 psi. These estimates and recommendations should be revised if design traffic information is shown to be different that described above.

Exposed soil subgrades should be compacted to a firm and unyielding state and proof-rolling should be used to evaluate pavement subgrade. Any soft areas disclosed by proof-rolling will likely require over-excavation and replacement with structural fill. Some contingency should be provided for the repair of any soft areas.

Permanent, properly installed drainage is also an essential aspect of pavement design and construction. All paved areas should have positive drainage to prevent ponding of surface water and saturation of the base course. This is particularly important in cut sections or at low points within the paved areas, such as in sunken loading dock areas or around stormwater catch basins. Effective means to prevent saturation of the base course including installing subdrain systems below sunken loading docks and weep holes in the sidewalls to catch basins.

4. DESIGN REVIEW AND CONSTRUCTION MONITORING

We welcome the opportunity to review and discuss construction plans and specifications as they are being developed. We are of the opinion that to observe compliance with the design concepts, specifications, and recommendations, construction operations dealing with earthwork and foundations should be observed by a qualified geotechnical engineer. We would be pleased to provide these services to you.

5. REPORT LIMITATIONS

The recommendations submitted in this report are based on the subsurface information obtained by PSI and design details furnished by representatives of the client, ADA Architects Inc, for the proposed improvements at 2400 Riverside Drive in Mount Vernon, Washington. If there are any revisions to the plans for this project, or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine if changes in the foundation and/or pavement recommendations are required. If PSI is not retained to review these changes, PSI will not be responsible for the impact of those conditions on the project.

The geotechnical engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

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

REFERENCES

Reference 1: Washington Department of Natural Recourses Interactive Geologic Map:
http://www.dnr.wa.gov/researchscience/topics/geosciencesdata/pages/geology_portal.aspx

Reference 2: U.S. Geological Survey, 2010, Quaternary fault and fold database for the United States, accessed November 10, 2010, from USGS web site:
http://earthquake.usgs.gov/hazards/qfaults/

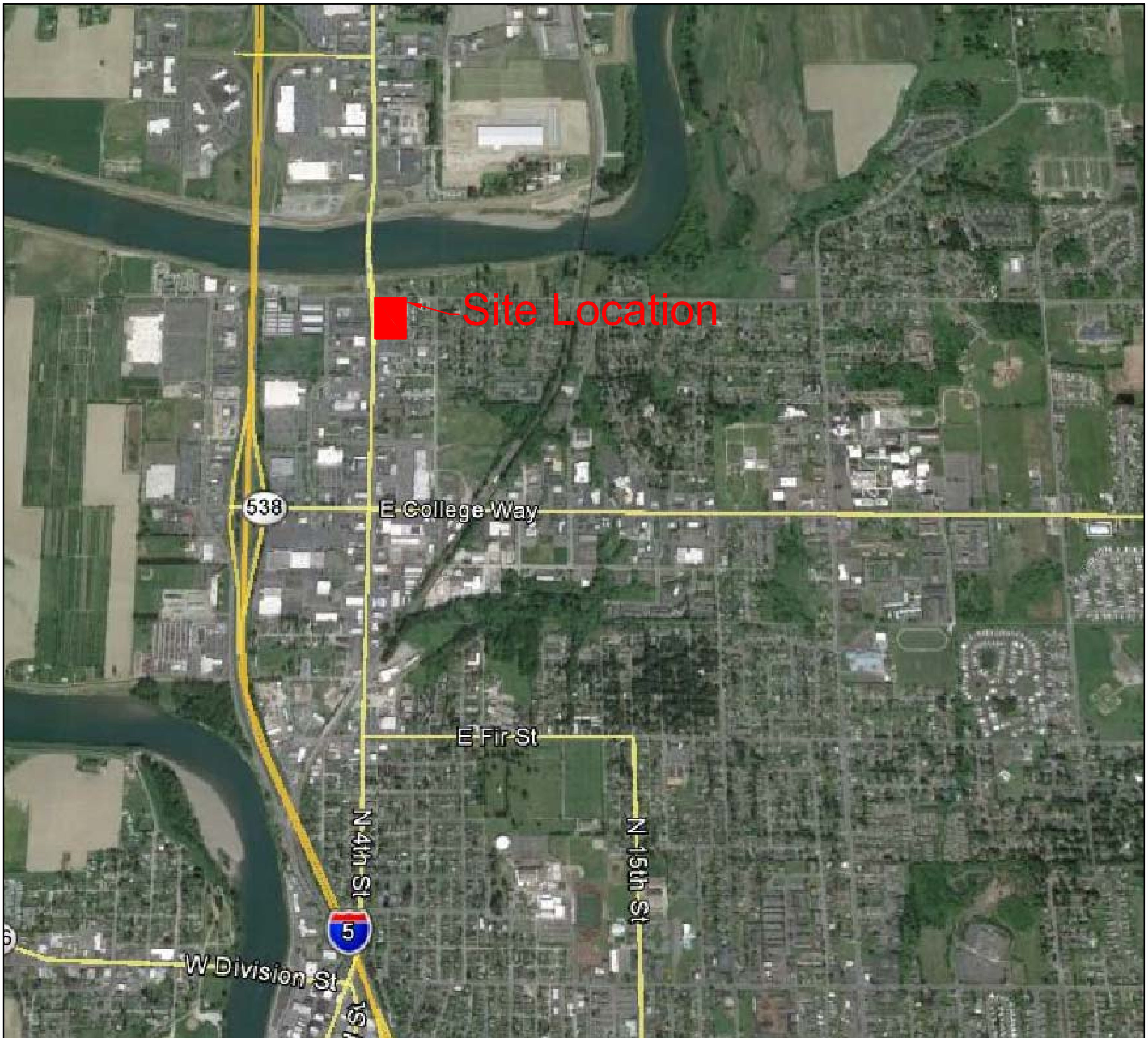
Reference 3: USGS Seismic Design Maps. http://earthquake.usgs.gov/designmaps/us/application.php

Reference 4: USGS 2008 Interactive Deaggregations:
http://geohazards.usgs.gov/deaggint/2008/

FIGURES

VICINITY MAP

SITE EXPLORATION LOCATION MAP



LEGEND:

= Site Location

NOTES

SIte underlay provided by Google Earth

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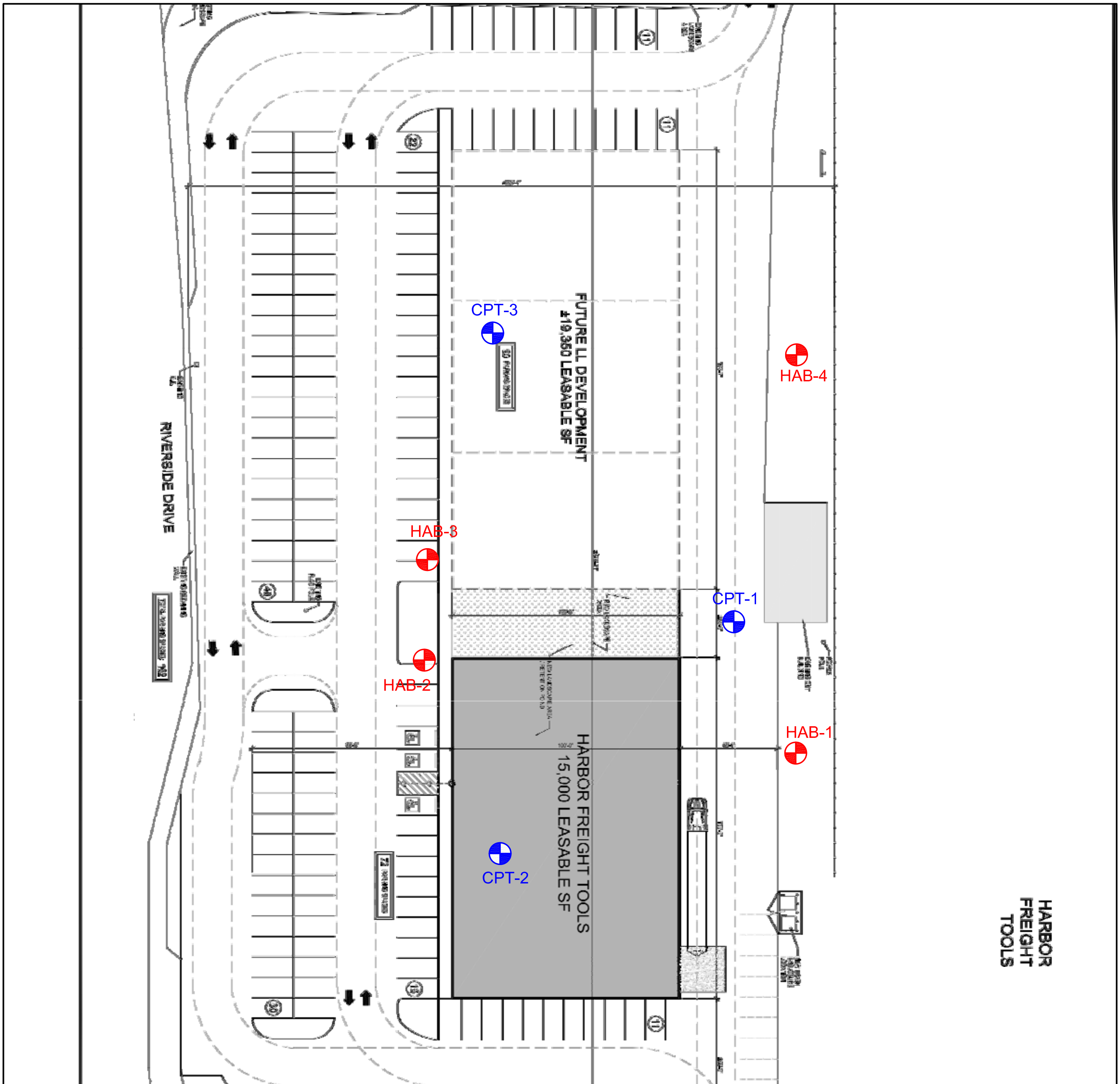
psi Information To Build On Engineering • Consulting • Testing	PROJECT NAME: Harbor Freight 2400 Riverside Drive Mt Vernon, Washington	DRAWN BY: MSP	DATE: September, 2016	FIGURE: 1
20508 56th Ave W Suite A **Lynwood, WA 98036** **(425)409-2504**	DESCRIPTION: Vicinity Map	APPROVED BY: MSP	PSI PROJECT NUMBER: 07121398	



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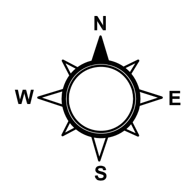
Hand Auger Boring Location

CPT Location

NOTES

Site underlay provided by ADA Architects Inc.



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20508 56th Ave W Suite A Lynwood, WA 98036 (425) 409-2504	PROJECT NAME: Harbor Freight 2400 Riverside Drive Mt Vernon, Washington	DRAWN BY: MSP	DATE: September, 2016	FIGURE: 2
	DESCRIPTION: Site Exploration Map	APPROVED BY: MSP	PSI PROJECT NUMBER: 07121398	

APPENDIX A

FIELD EXPLORATION PROGRAM

GENERAL NOTES

SOIL CLASSIFICATION CHART

BORING LOGS

FIELD EXPLORATION PROGRAM

General

We explored the site by drilling four hand auger soil borings (HAB1-HAB4) to depths ranged from approximately 6½ feet to 10 feet bgs and 3 CPT to depths ranging 58 to 71 feet bgs using a truck mounted CPT rig, on June 7, 2016. Prior to performing all hand auger borings, PSI performed Dynamic Cone penetrometers (DCP) tests which are used to determine blow counts of the soil and thereby provide relative density/relative constancy of the subsurface soils. DCP test involves taking a one-inch diameter probe and a 35-pound slide hammer that is manually lifted 15 inches and dropped vertically onto the top of the probe. DCP blow counts are recorded every 10 centimeters to provide blow counts, to determine the relative density/relative consistency of a material. The locations of the borings are shown on Figure 2. A representative of PSI's geotechnical staff was present during the explorations to record soil and groundwater conditions encountered in the exploration and to obtain soil and rock samples for laboratory testing.

Sampling Procedures

Throughout the drilling operation, soil samples were obtained from the hand auger borings using a 5-inch hand auger. The soils were observed continuously throughout the drilling process and samples were collected when changes in material were observed.

The DCP's and hand auger borings were conducted to observe the stratigraphy, density, and variability of subsurface soil conditions.

No samples can be collected form CPT's. Logs form CPT's are shown in this Appendix A.

Field Classification

Soil samples were initially classified visually in the field. Consistency, color, relative moisture, degree of plasticity, peculiar odors and other distinguishing characteristics of the soil samples were noted. The terminology used in the soil and rock classifications and other modifiers are defined in the General Notes in this Appendix A.

Exploration Logs

Summary boring log follows in this appendix. The left-hand portion of the boring log gives our interpretation of the soil encountered in the soil boring, sample locations and depths, and groundwater information. The right-hand portion of the log shows the results of the sample water contents, and other laboratory information.

The soil profile shown on the boring logs represent the conditions only at actual exploration location. Variations may occur and should be expected. The stratifications represent the approximate boundary between subsurface materials; the actual transition may be gradual.

GENERAL NOTES

SAMPLE IDENTIFICATION

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

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
ST: Shelby Tube - 3" O.D., except where noted.
BS: Bulk Sample
PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

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

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

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

RELATIVE PROPORTIONS OF FINES

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

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Medium Stiff
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

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

RELATIVE PROPORTIONS OF SAND AND GRAVEL

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

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

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

ROCK BEDDING THICKNESSES

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

ROCK VOIDS

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

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

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

DEGREE OF WEATHERING

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

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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

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LOG OF HAB-1

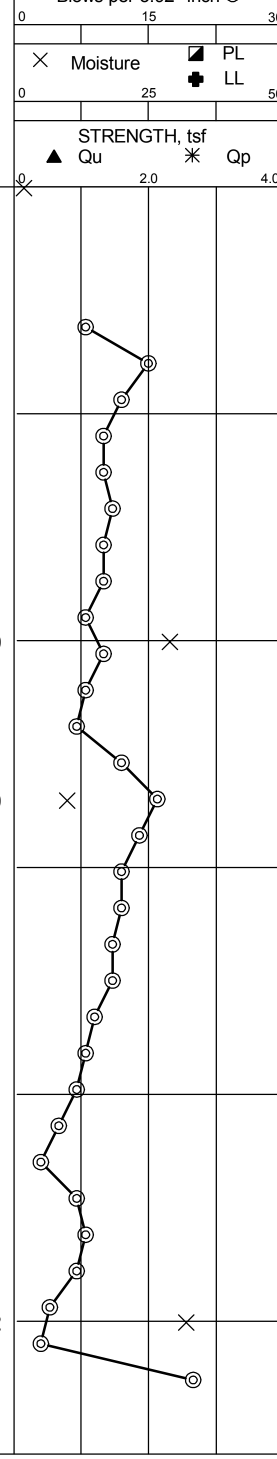
Sheet 1 of 1

PSI Job No.:	07121398	Excavation Method:	Hand Auger
Project:	Harbor Freight	Sampling Method:	Continous
Location:	2400 Riverside Drive	DCP Type:	Wild Cat
	Mount Vernon, WA	Boring Location:	

WATER LEVELS

Depth, (feet)	MATERIAL DESCRIPTION	USCS Classification	Dynamic Cone (DCP) Blows per 3.92"-inch	Moisture, %
0	**Poorly graded GRAVEL with silt & sand:** with organics (Topsoil)			2
	Silty SAND: brown, moist, loose to medium dense, fine to medium sand [Alluvium]	SM	8, 15, 12, 10, 10, 11, 10, 10, 8, 10, 8, 7, 12	29 (at 4 ft)
	SILT with Sand: light brown, moist, medium stiff to stiff. [Alluvium]	ML	16, 14, 12, 12, 11, 11, 9, 8, 7, 5, 3, 7, 8, 7, 4	10 (at 5.4 ft)
10	**Poorly graded SAND:** grayish brown, moist, medium dense, fine sand, trace fines [Alluvium]	SP	3, 20	32
	Bottom of boring at 10 feet 2 inches, DCP to 9 feet 10 inches. No groundwater observed.			

DYNAMIC CONE PENETRATION TEST DATA



Additional Remarks

Completion Depth:	10.0 ft
Date Boring Started:	6/7/16
Date Boring Completed:	6/7/16
Logged By:	SM
Excavation Contractor:	PSI, Inc.

Sample Types:
- Shelby Tube
- Dynamic Cone (DCP)
- Grab Sample

Latitude: 48.4422°
Longitude: -122.33388°
Excavation Equipment:
Remarks:

The stratification lines represent approximate boundaries. The transition may be gradual.

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LOG OF HAB-2

PSI Job No.: 07121398
Project: Harbor Freight
Location: 2400 Riverside Drive
Mount Vernon, WA

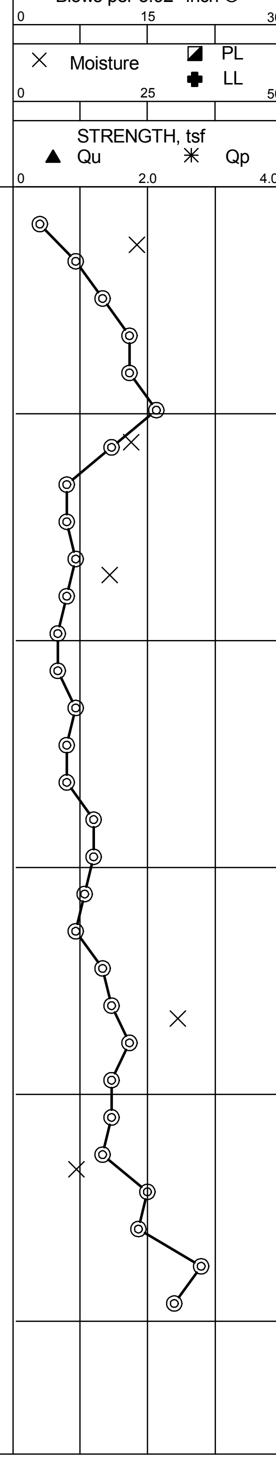
Excavation Method: Hand Auger
Sampling Method: Continous
DCP Type: Wild Cat
Boring Location:

WATER LEVELS

Depth (feet)	Material Description	USCS Classification	Dynamic Cone (DCP) Blows per 3.92"-inch	Moisture, %
0 – 0.5	**TOPSOIL:** dark brown, moist, loose, silty SAND with organics.	SM	3	23
0.5 – 7.3	**Silty SAND:** brown, moist, loose to medium dense, fine to medium sand [Alluvium]	SM	7, 10, 13, 13, 16, 11, 6, 6, 7, 6, 5, 5, 7, 6, 6, 9, 9, 8, 7, 10	22, 18
7.3 – 8.7	**SILT with Sand:** light brown, moist, medium stiff to stiff. [Alluvium]	ML	11, 13, 11, 11, 10	31
8.7 – 10.6	**Poorly graded SAND:** grayish brown, moist, medium dense, fine sand, trace fines [Alluvium]	SP	15, 14, 21, 18	12

Bottom of boring at 10 feet 2 inches, DCP to 9 feet 10 inches. No groundwater observed.

DYNAMIC CONE PENETRATION TEST DATA
Blows per 3.92"-inch



Additional Remarks

Completion Depth: 10.6 ft
Date Boring Started: 6/7/16
Date Boring Completed: 6/7/16
Logged By: SM
Excavation Contractor: PSI, Inc.

Sample Types:
Shelby Tube
Dynamic Cone (DCP)
Grab Sample

Latitude: 48.44219°
Longitude: -122.33457°
Excavation Equipment:
Remarks:

The stratification lines represent approximate boundaries. The transition may be gradual.

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LOG OF HAB-3

Sheet 1 of 1

PSI Job No.: 07121398
Project: Harbor Freight
Location: 2400 Riverside Drive
Mount Vernon, WA

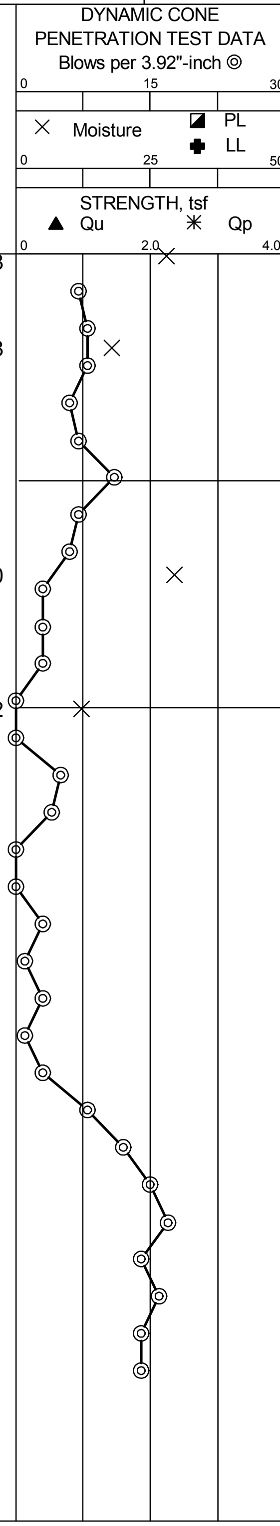
Excavation Method: Hand Auger
Sampling Method: Continous
DCP Type: Wild Cat
Boring Location:

WATER LEVELS

Depth, (feet)	MATERIAL DESCRIPTION	USCS Classification
0–0.8	**TOPSOIL:** dark brown, moist, loose, silty SAND with organics.	
0.8–3.5	**Silty SAND:** brown, moist, very loose to medium dense, fine to medium sand [Alluvium]	SM
3.5–5.5	**SILT with Sand:** light brown with orange mottling, moist, very soft to soft. [Alluvium]	ML
5.5–5.8	**Poorly graded SAND:** grayish brown, moist, medium dense, fine sand, trace fines [Alluvium]	SP
	Bottom of boring and refusal at 5 feet 10 inches, DCP to 9 feet 10 inches. No groundwater observed.	

Dynamic Cone (DCP) Blows per 3.92"-inch: 7, 8, 8, 6, 7, 11, 7, 6, 3, 3, 3, 0, 0, 5, 4, 0, 0, 3, 1, 3, 1, 3, 8, 12, 15, 17, 14, 16, 14, 14

Moisture, %: 28, 18, 30, 12



Additional Remarks

Completion Depth:	5.8 ft
Date Boring Started:	6/7/16
Date Boring Completed:	6/7/16
Logged By:	SM
Excavation Contractor:	PSI, Inc.

Sample Types:
Shelby Tube
Dynamic Cone (DCP)
Grab Sample

Latitude: 48.44238°
Longitude: -122.33457°
Excavation Equipment:
Remarks:

The stratification lines represent approximate boundaries. The transition may be gradual.

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LOG OF HAB-4

Sheet 1 of 1

PSI Job No.:	07121398	Excavation Method:	Hand Auger
Project:	Harbor Freight	Sampling Method:	Continous
Location:	2400 Riverside Drive	DCP Type:	Wild Cat
	Mount Vernon, WA	Boring Location:	

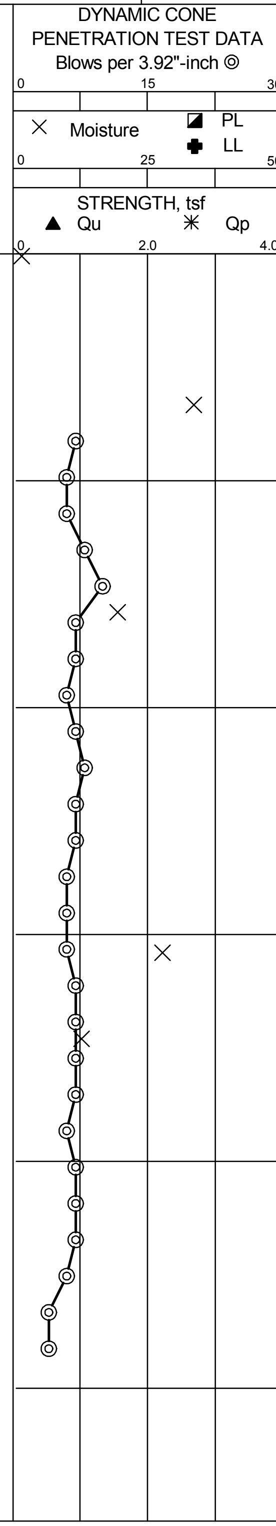
WATER LEVELS

Depth, (feet)	MATERIAL DESCRIPTION	USCS Classification	Moisture, %
0 – 1	**Crushed Rock Surfaceing:**		2
1 – 6.2	**Silty SAND:** brown, moist, loose, fine to medium sand [Alluvium]	SM	34, 20, 28
6.2 – 6.9	**SILT with Sand:** light brown, moist, medium stiff. [Alluvium]	ML	13
6.9 – 10	**Poorly graded SAND:** grayish brown, moist, loose to very loose, fine sand, trace fines [Alluvium]	SP	
10 – 10.2	**SILT with Sand:** grayish brown, moist, soft. [Alluvium]	ML	

Bottom of boring at 10 feet 2 inches, DCP to 8 feet 4 inches . No groundwater observed.

Dynamic Cone (DCP) Blows per 3.92"-inch: 7, 6, 6, 8, 10, 7, 7, 6, 7, 8, 7, 7, 6, 6, 6, 7, 7, 7, 7, 6, 7, 7, 7, 6, 4, 4

DYNAMIC CONE PENETRATION TEST DATA



Additional Remarks

Completion Depth:	10.2 ft	Sample Types:	Latitude: 48.44257°
Date Boring Started:	6/7/16	Shelby Tube	Longitude: -122.3339°
Date Boring Completed:	6/7/16	Dynamic Cone (DCP)	Excavation Equipment:
Logged By:	SM	Grab Sample	Remarks:
Excavation Contractor:	PSI, Inc.		

The stratification lines represent approximate boundaries. The transition may be gradual.

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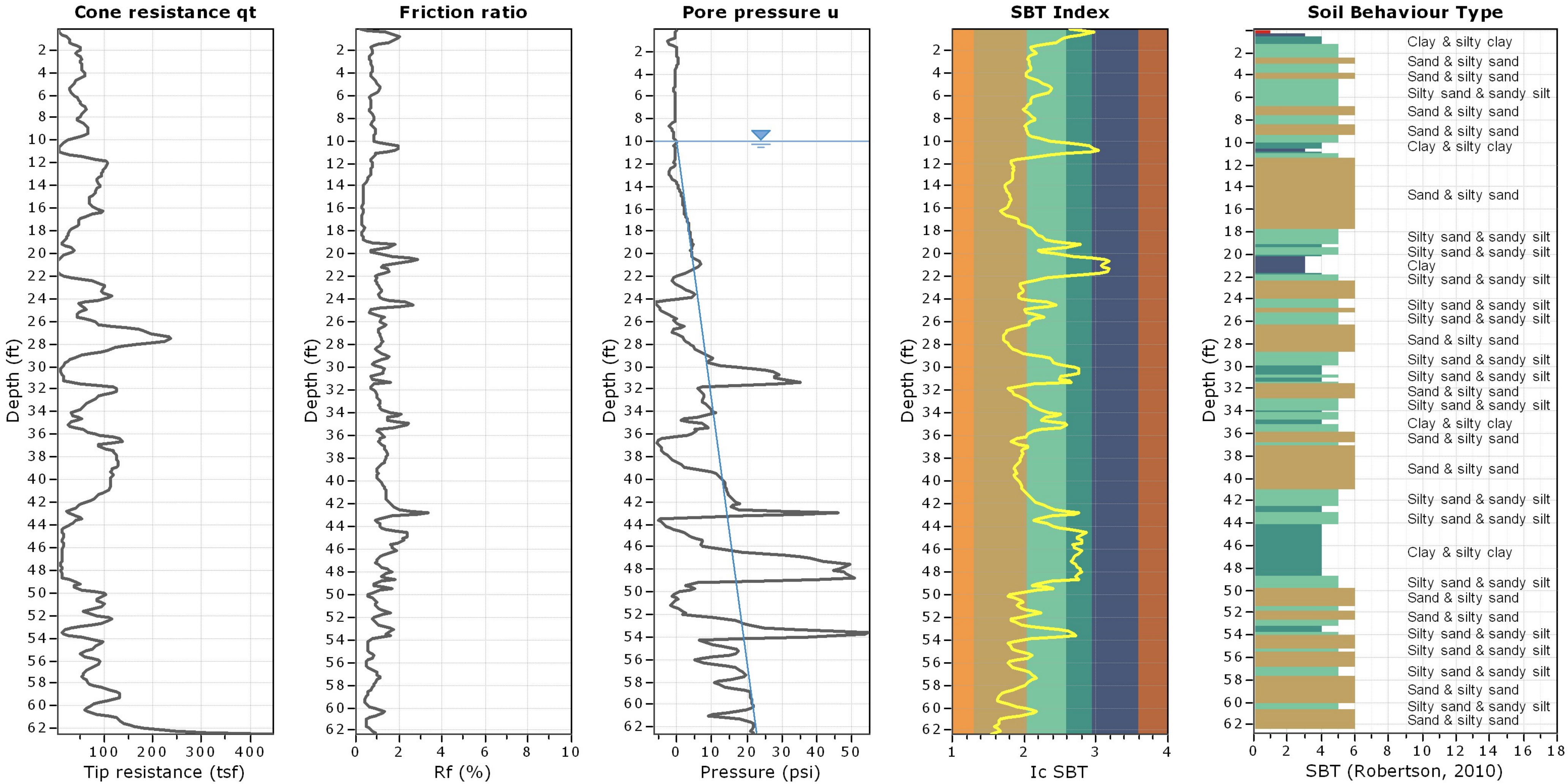
CPT: CPT-01

Total depth: 62.50 ft, Date: 6/10/2016

Surface Elevation: 0.00 ft

Project:

Location:



SBT legend

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 sand
9. Very stiff fine grained

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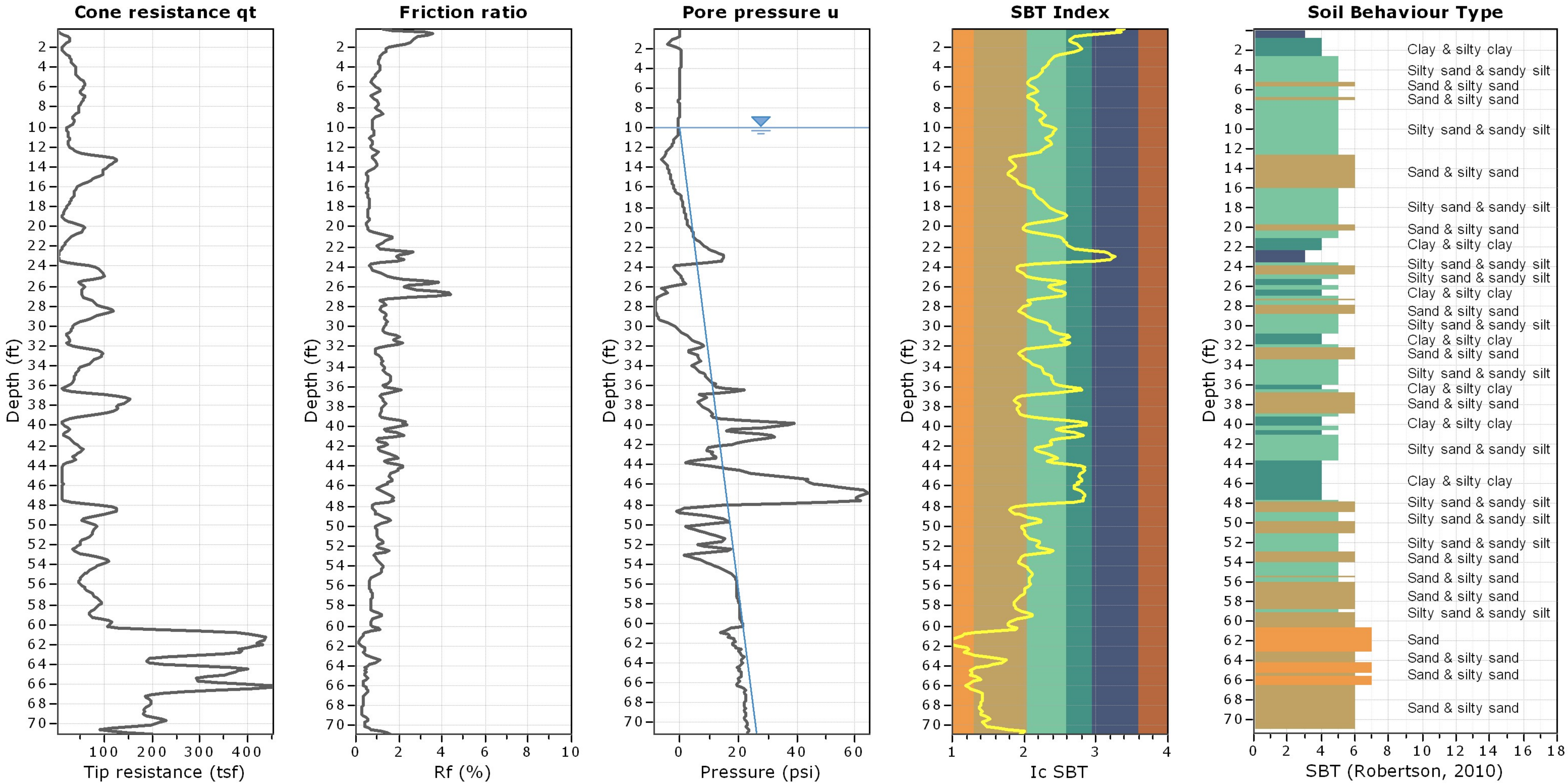
CPT: CPT-02

Project:

Total depth: 71.03 ft, Date: 6/10/2016

Location:

Surface Elevation: 0.00 ft



SBT legend

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 sand
9. Very stiff fine grained

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Phone: 425-409-2504

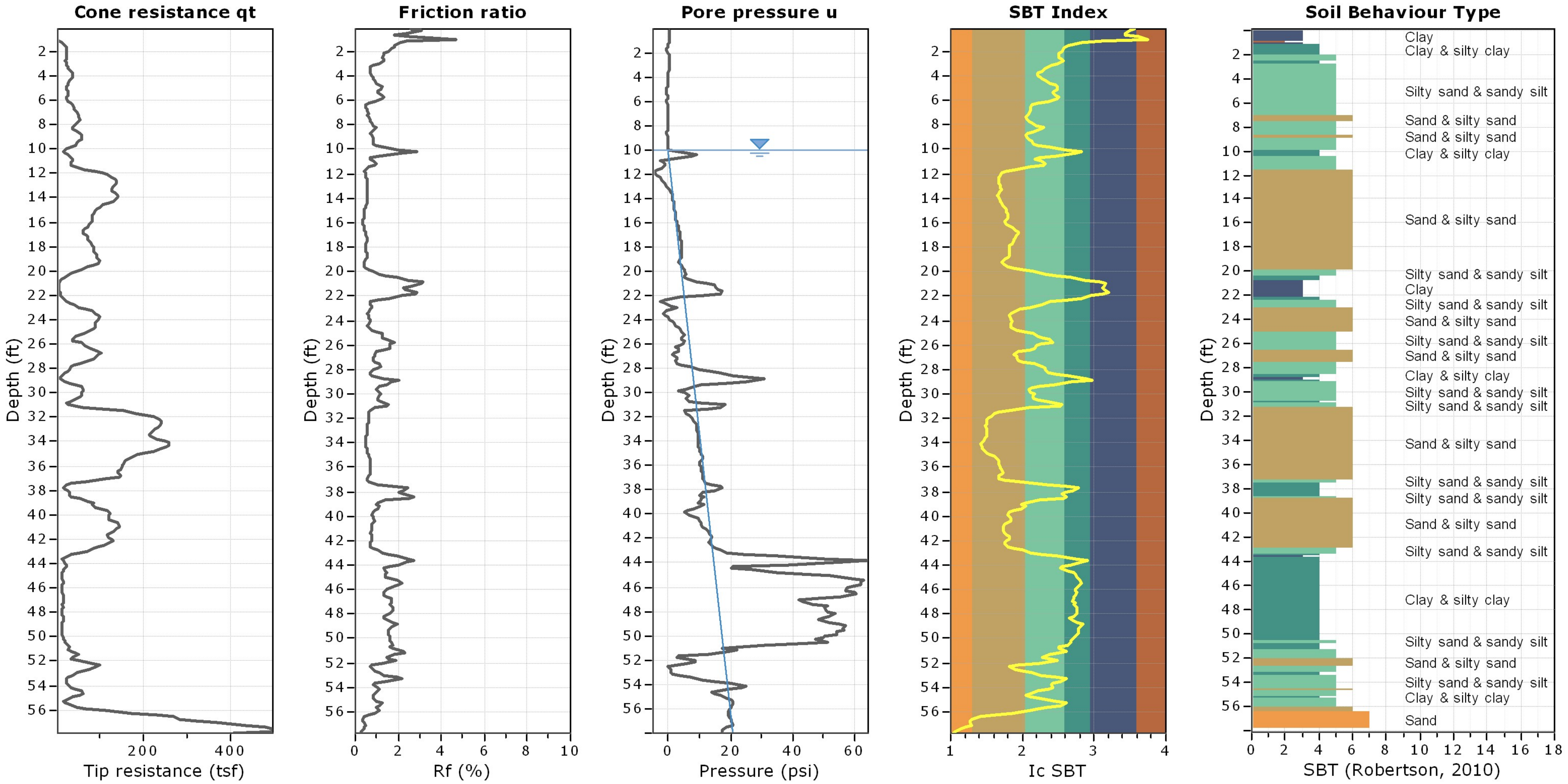
CPT: CPT-03

Total depth: 57.91 ft, Date: 6/10/2016

Surface Elevation: 0.00 ft

Project:

Location:



SBT legend

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 sand
9. Very stiff fine grained

APPENDIX B

LABORATORY TEST RESULTS

Laboratory Testing Program and Procedures

General

Soil samples obtained during the field explorations were examined in our laboratory. The physical characteristics of the samples were noted and the field classifications were modified where necessary in accordance with terminology presented the General Notes included in this appendix.

Representative samples were selected during the course of the examination for further testing. The testing procedures and results of the tests are summarized below. The phrase “In general accordance with guidelines presented in…” means that certain local and common descriptive practices and methodologies have been followed.

Visual-Manual Classification

The soil samples were classified in general accordance with guidelines presented in ASTM D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. Certain terminology incorporating current local engineering practice, as provided in the Soil Classification Chart included with or in lieu of ASTM terminology. The term which best described the major portion of the sample was used in determining the soil type (that is, gravel, sand, silt or clay).

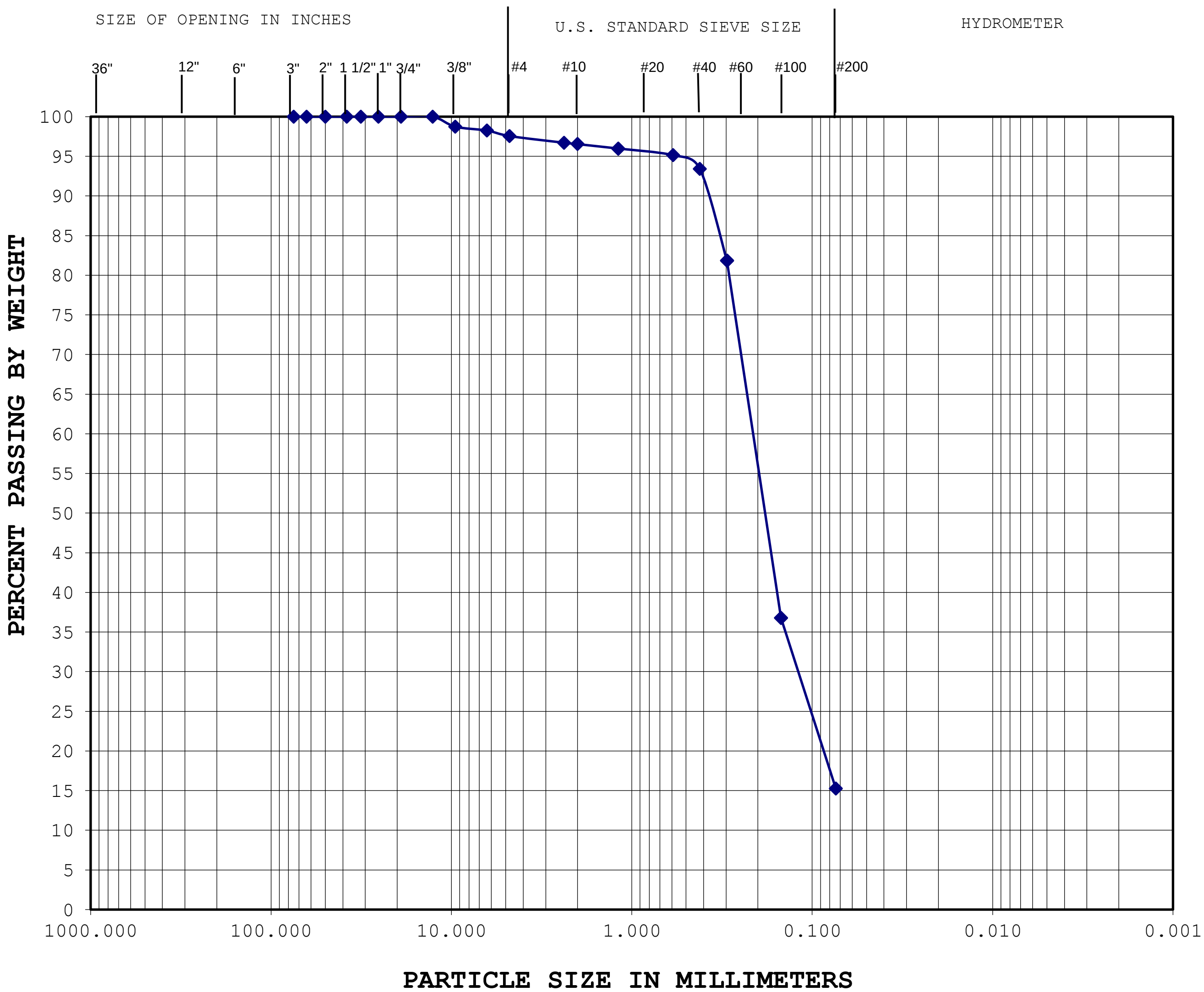
Moisture Content

Natural moisture content determinations were made on all samples. The natural moisture content is defined as the ratio of the weight of water to dry weight of soil, expressed as a percentage. The results of the moisture content determinations are presented on the boring logs in this appendix.

Grain Size Analysis

Select samples from the borings were analyzed for grain size in general conformance with ASTM C 136 and ASTM C117. In general, samples were oven dried, weighed then washed over a #200 sieve to remove silt and clay sized particles and then dried again. The samples were separated through a series of sieves of progressively smaller openings for determination of particle size distribution. The material passing and/or retained on each sieve was recorded as a percent of the total sample weight. The results of the sieve analysis are depicted in this appendix.

PARTICLE SIZE ANALYSIS - ASTM C136/C117



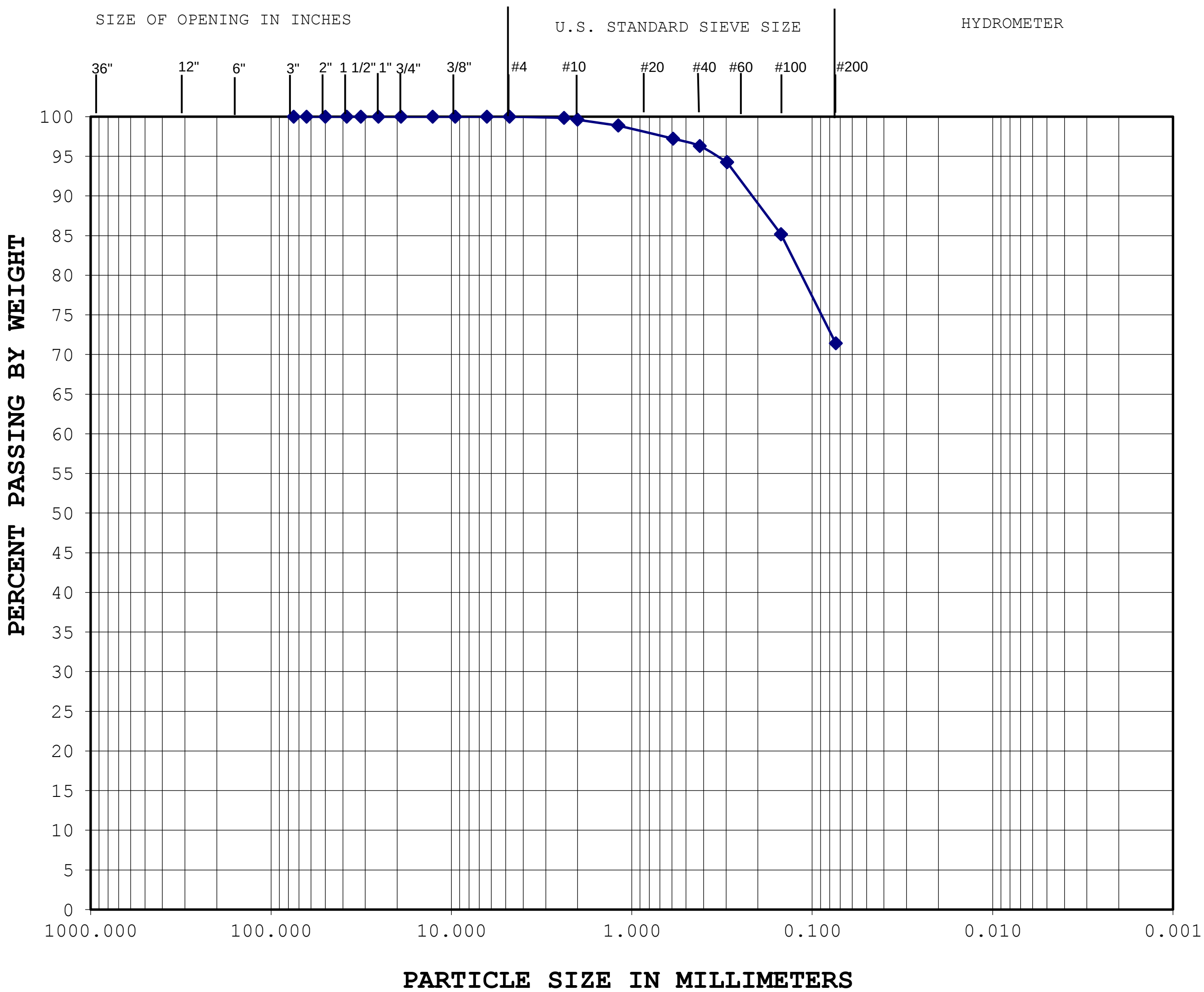
BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

% Gravel	% Sand	% Fines	PL = -
2.4%	82.3%	15.3%	LL = -
Soil Classification Silty SAND			PI = -

Exploration	Sample	Depth (feet)	Moisture	Reviewed	USCS Symbol
HAB-1		1/2 foot	10.3	MSP	SM

psi Information To Build On Engineering • Consulting • Testing	PROJECT NO:	PROJECT NAME:
	7121398	Harbor Freight, Riverside DR., Mount Vernon

PARTICLE SIZE ANALYSIS - ASTM C136/C117



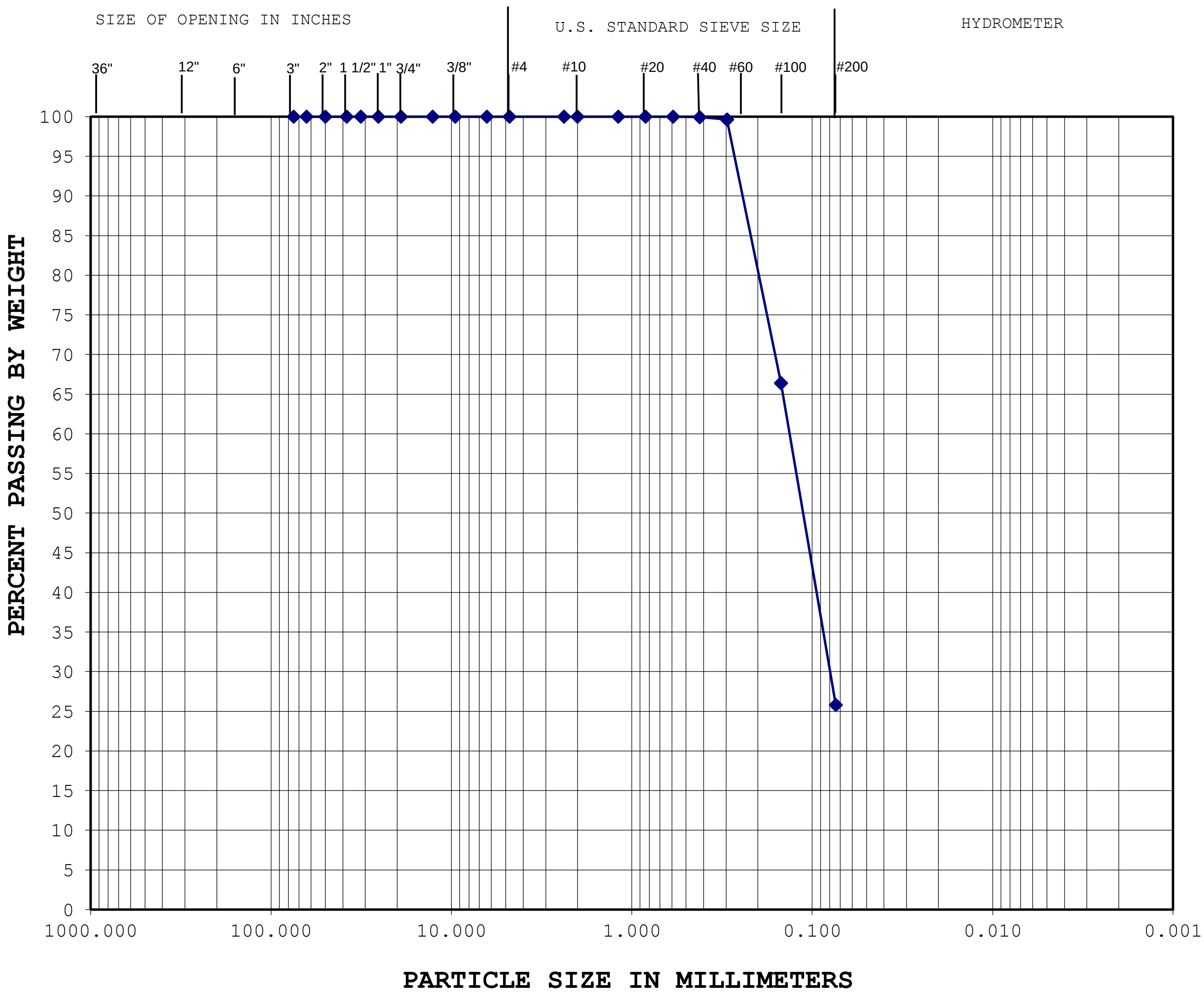
BOULDERS	COBBLES	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
		GRAVEL		SAND			FINE GRAINED	

% Gravel	% Sand	% Fines	
0.0%	28.6%	71.4%	PL = -
			LL = -
Soil Classification SILT with Sand			PI = -

Exploration	Sample	Depth(feet)	Moisture	Reviewed	USCS Symbol
HAB-2		8 feet 3 inches	30.6	MSP	ML

psi Information To Build On Engineering • Consulting • Testing	PROJECT NO: 7121398	PROJECT NAME: Harbor Freight, Riverside DR., Mount Vernon

PARTICLE SIZE ANALYSIS - ASTM C136/C117



BOULDERS	COBBLES	GRAVEL		SAND			FINE GRAINED	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

% Gravel	% Sand	% Fines	PL = -
0.0%	74.2%	25.8%	LL = -
Soil Classification Silty SAND			PI = -

Exploration	Sample	Depth(feet)	Moisture	Reviewed	USCS Symbol
HAB-4		10 feet	31.4	MSP	SM

psi Information To Build On Engineering • Consulting • Testing	PROJECT NO:	PROJECT NAME:
	7121398	Harbor Freight, Riverside DR., Mount Vernon

APPENDIX C

LIQUEFACTION ANALYSIS

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

Professional Service Industries, Inc.
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Lynnwood, WA 98036
Phone: 425-409-2504

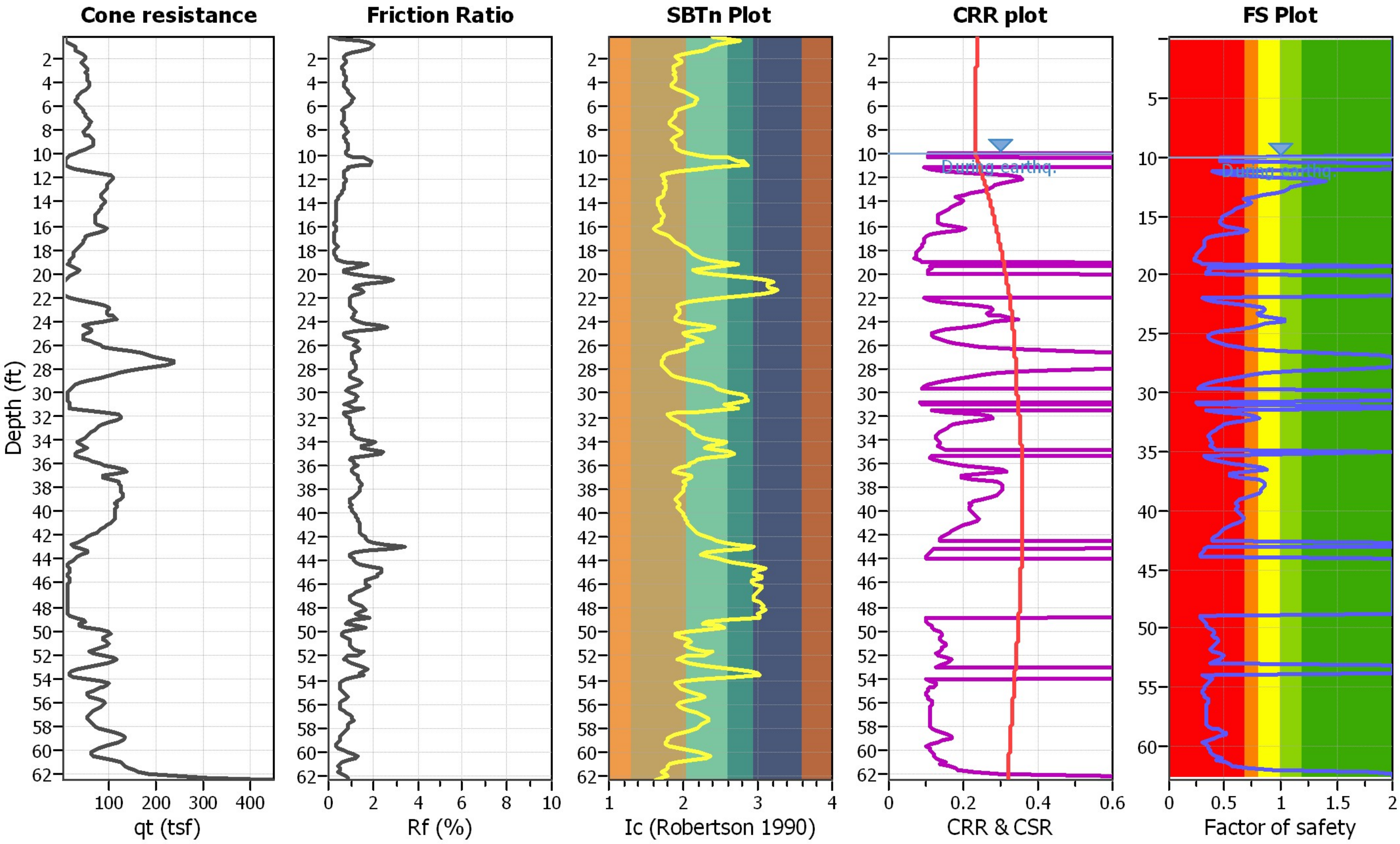
LIQUEFACTION ANALYSIS REPORT

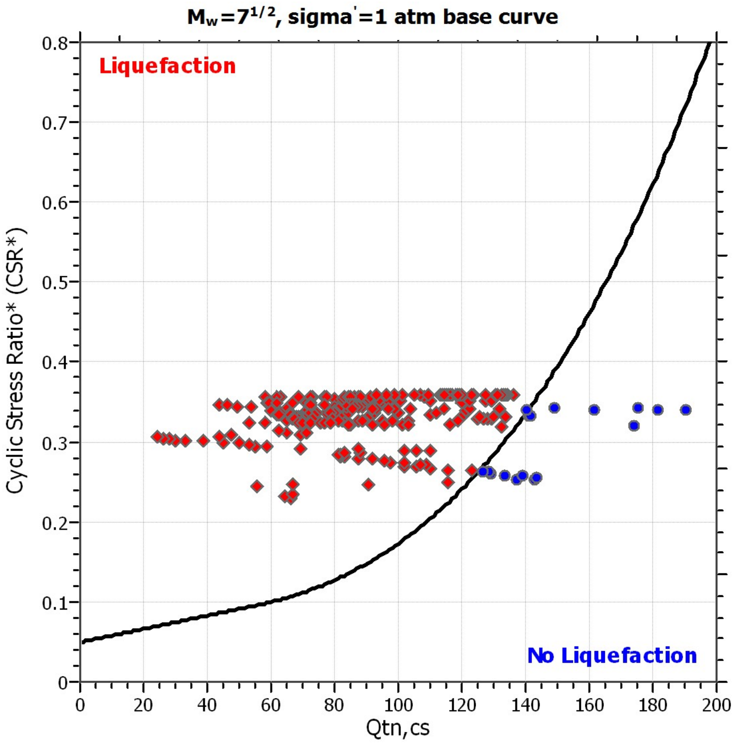
Project title : **Location :**

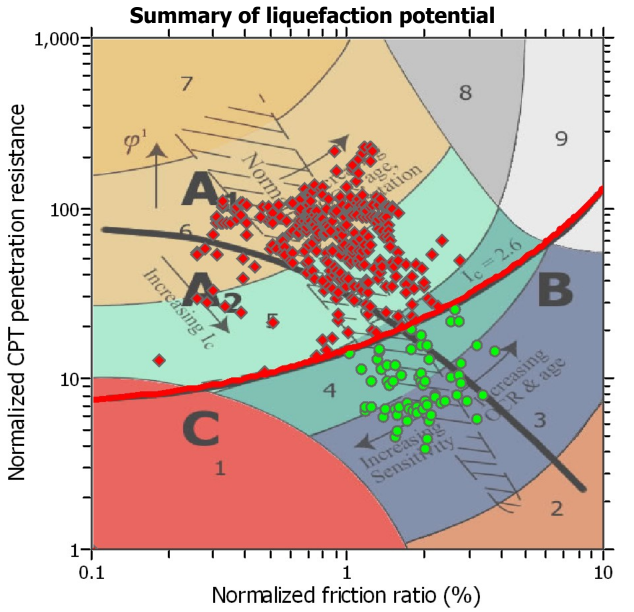
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.01	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.43	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

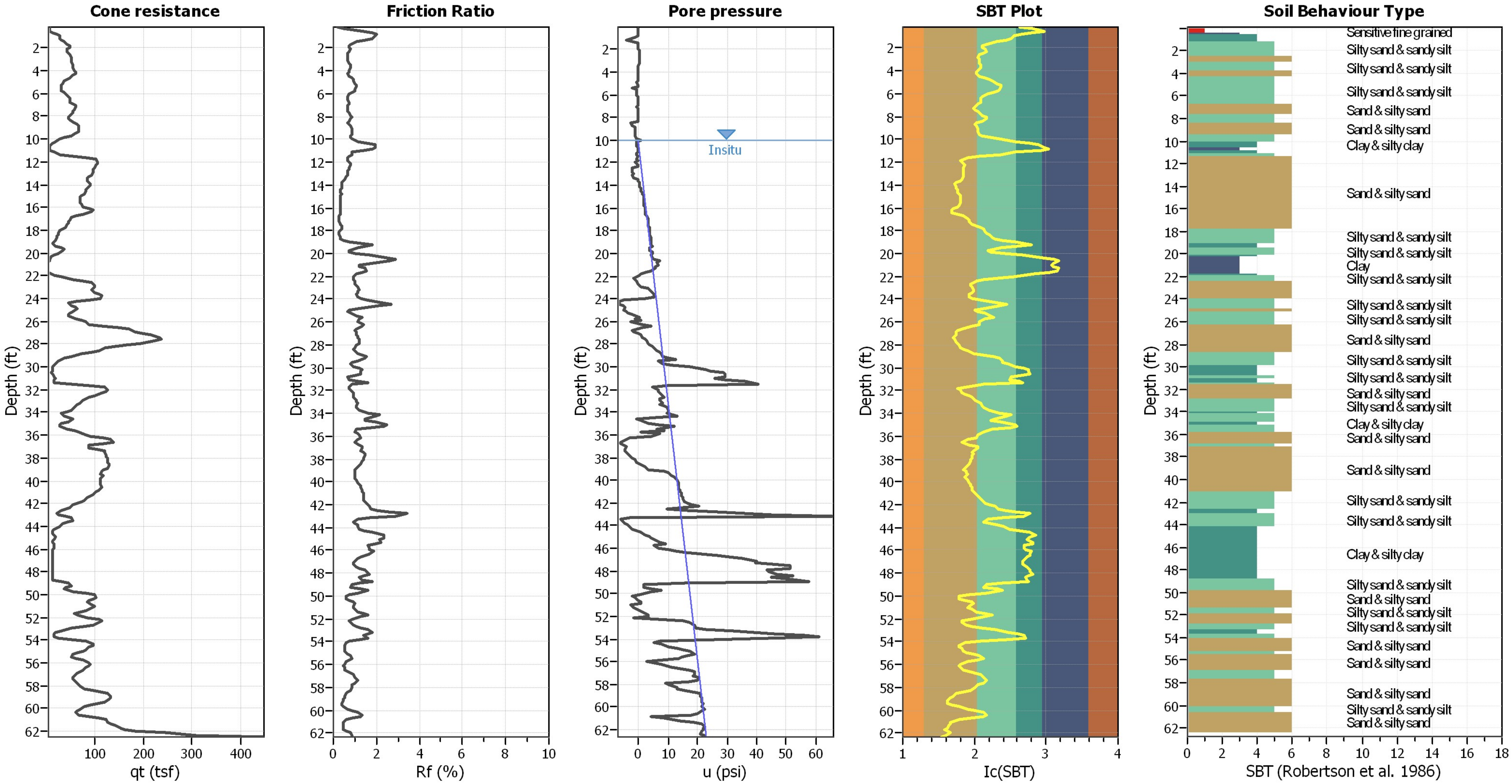






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



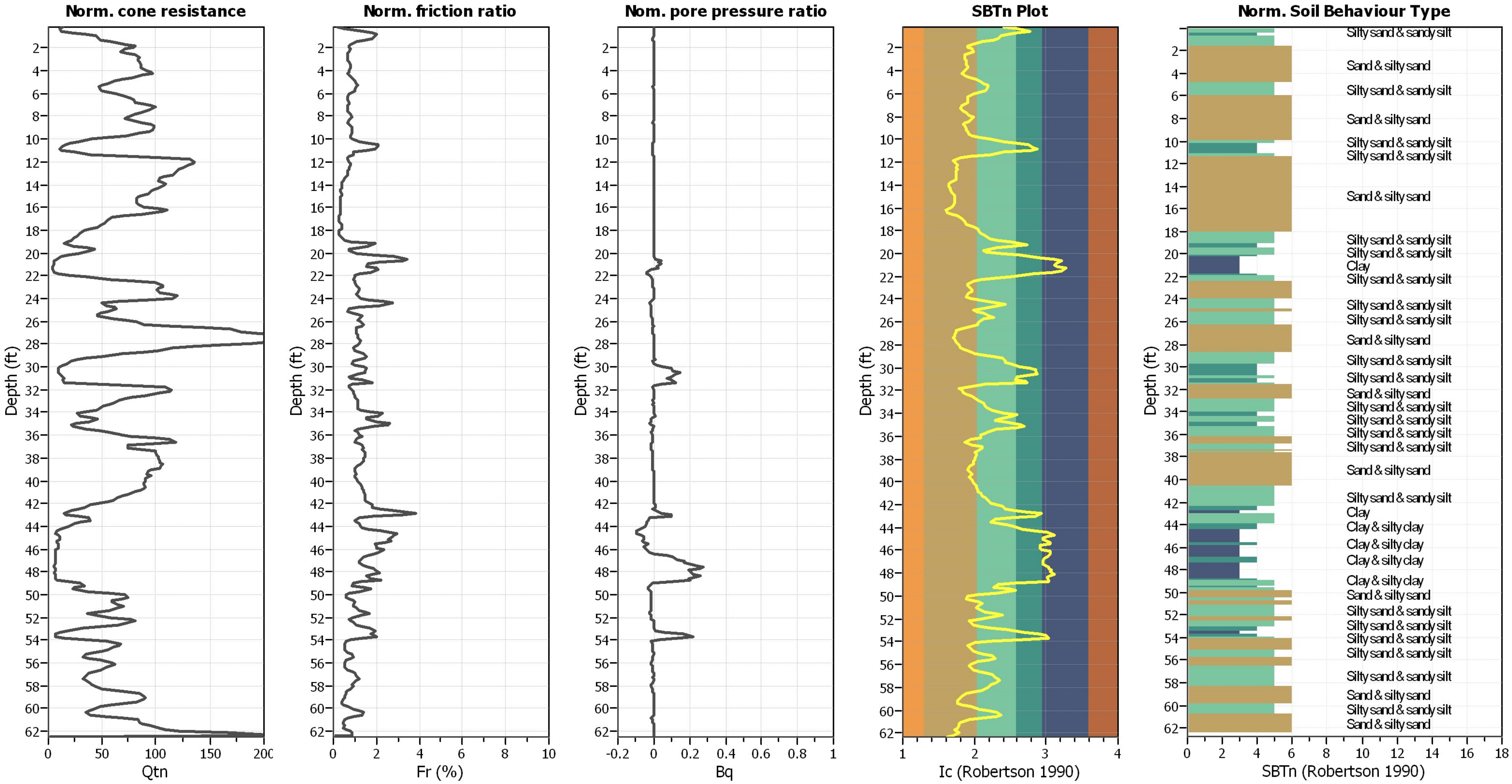
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A		
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes		
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A		

SBT legend

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

CPT basic interpretation plots (normalized)



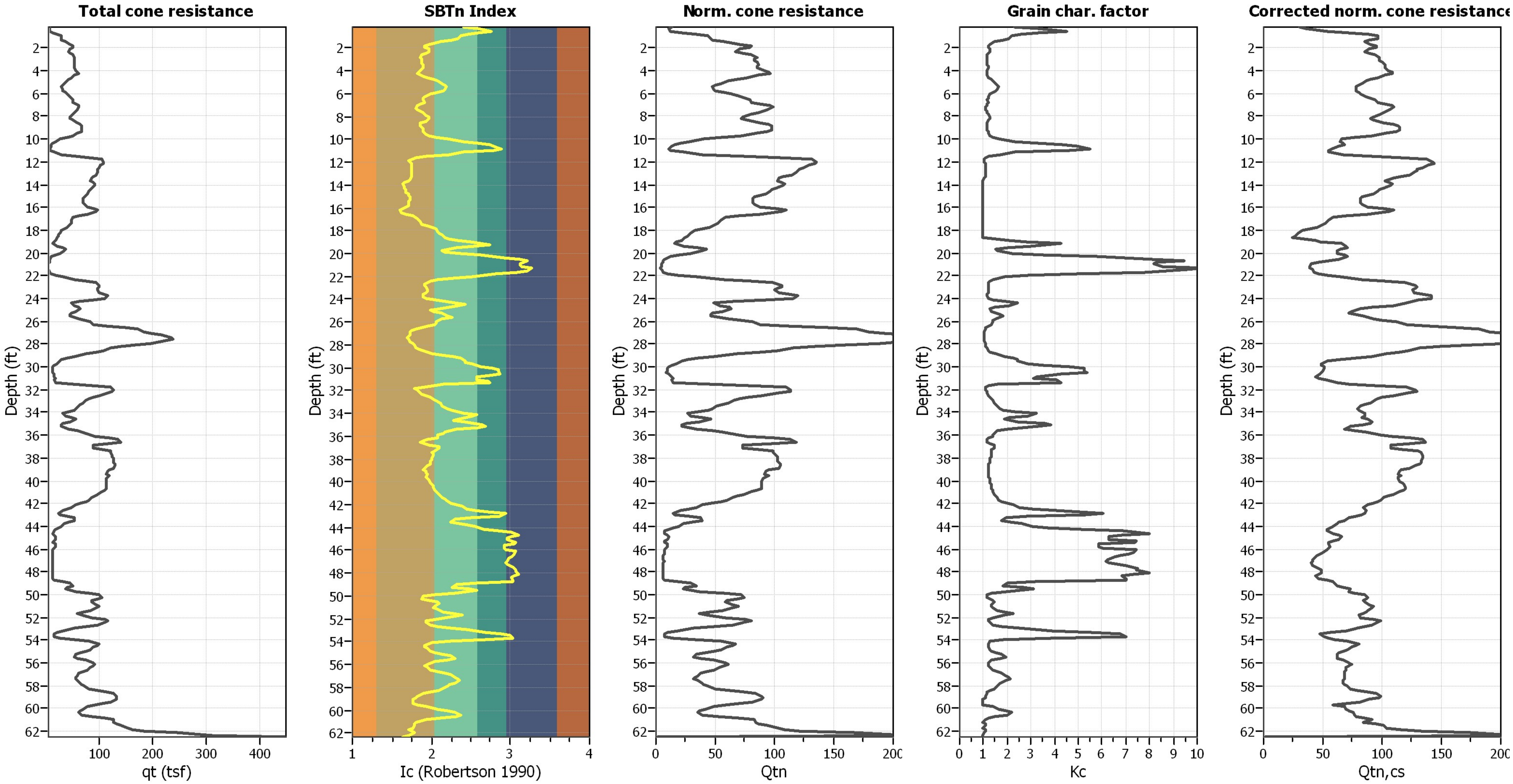
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

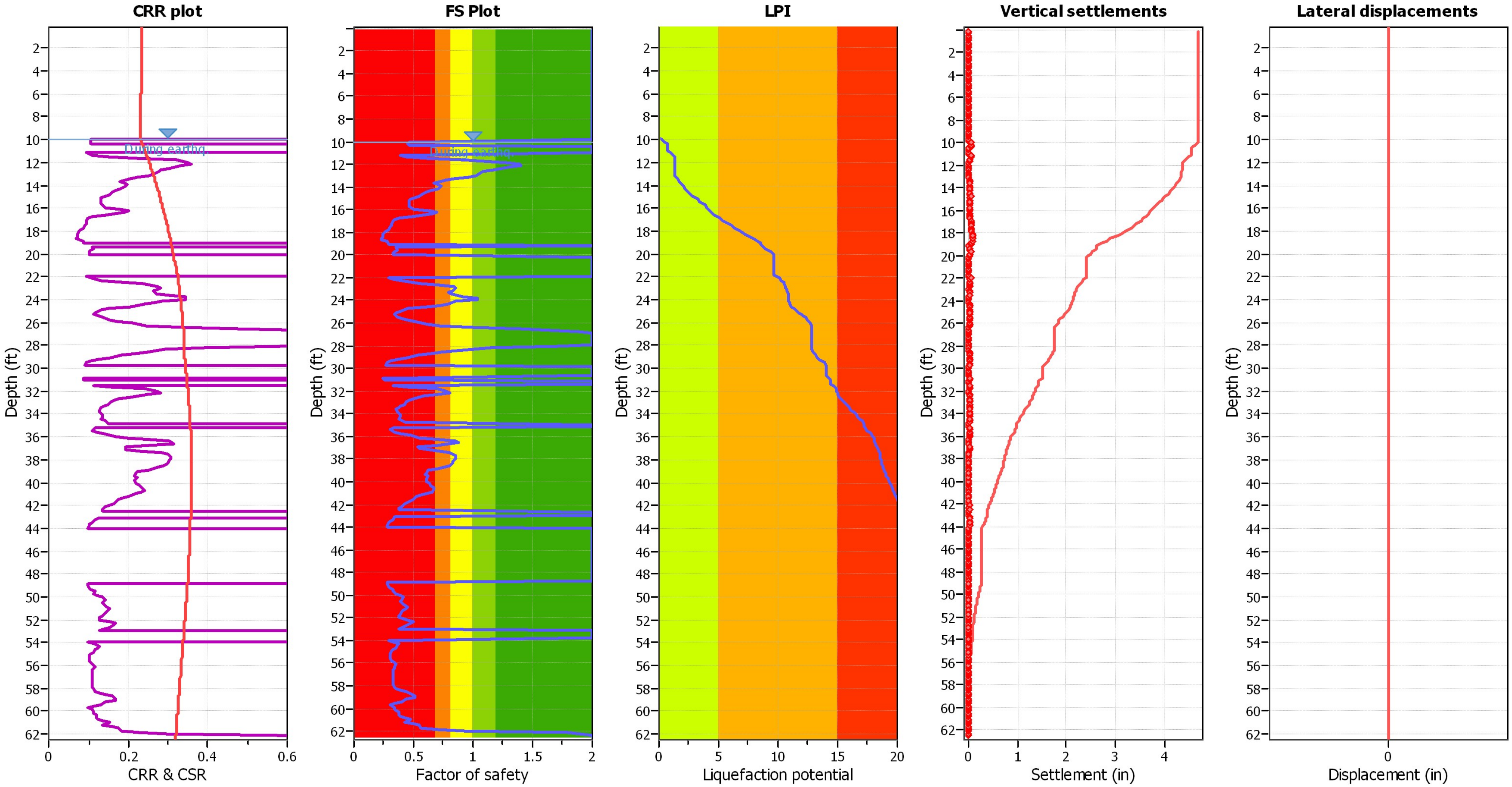
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

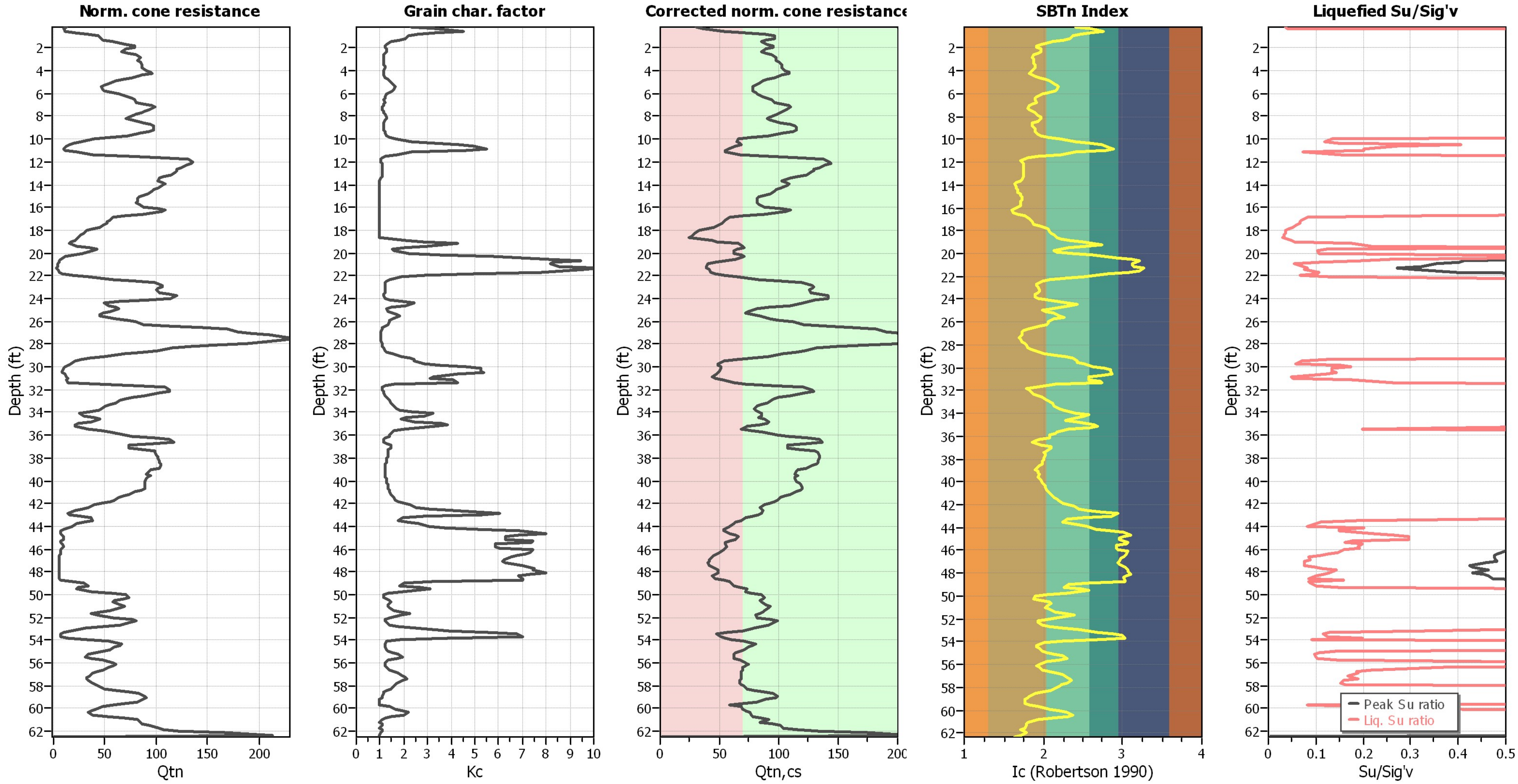
F.S. color scheme

- 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

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

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Phone: 425-409-2504

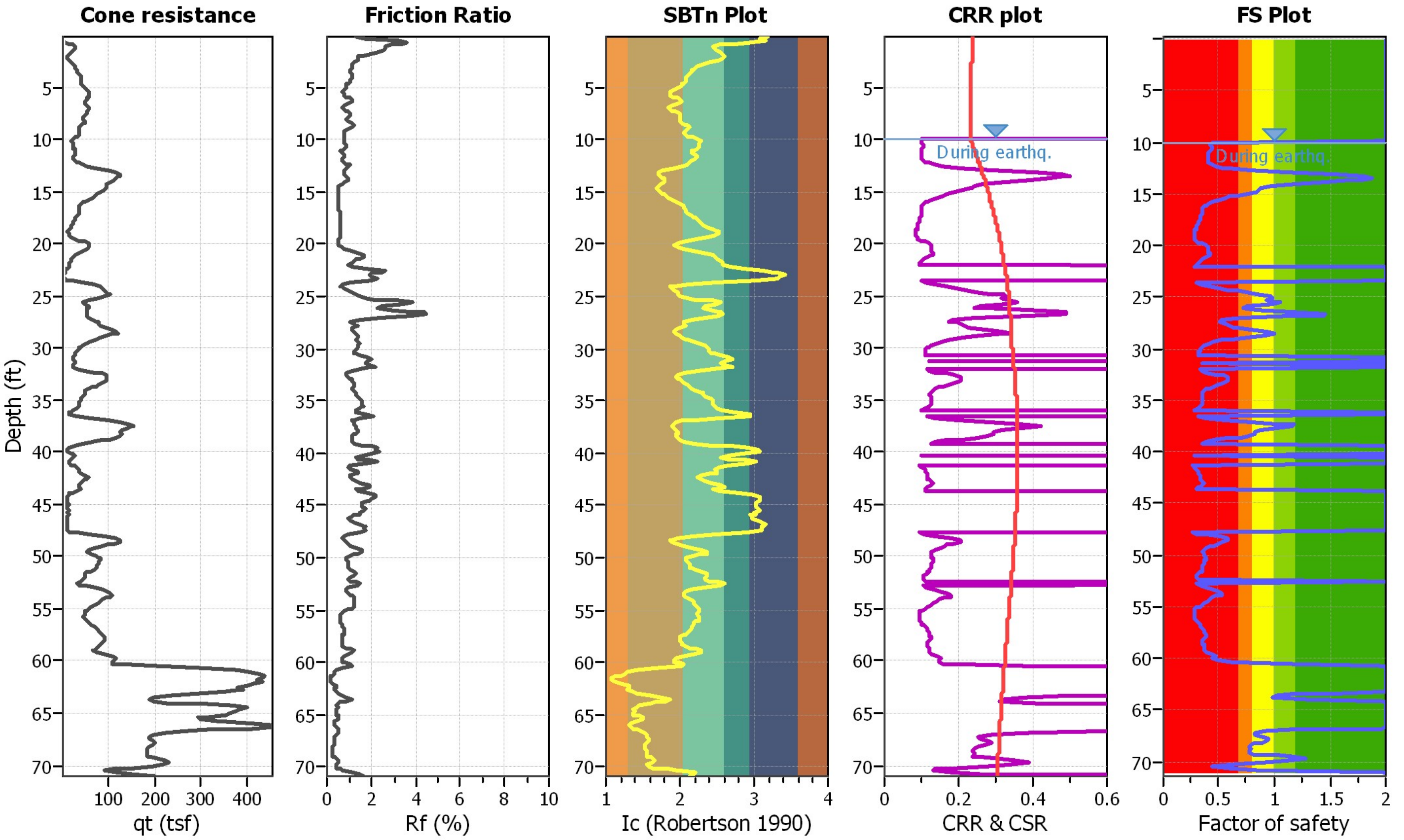
LIQUEFACTION ANALYSIS REPORT

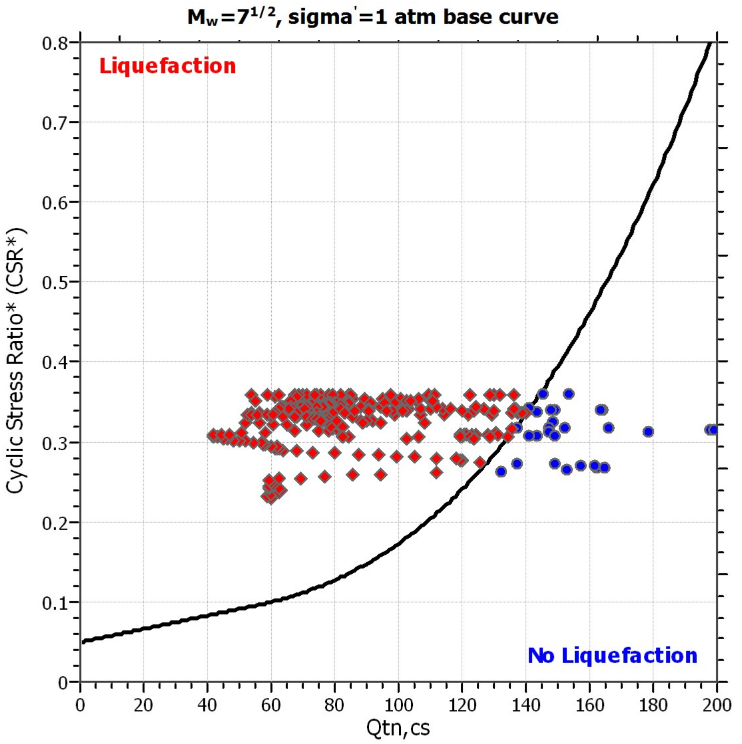
Project title : **Location :**

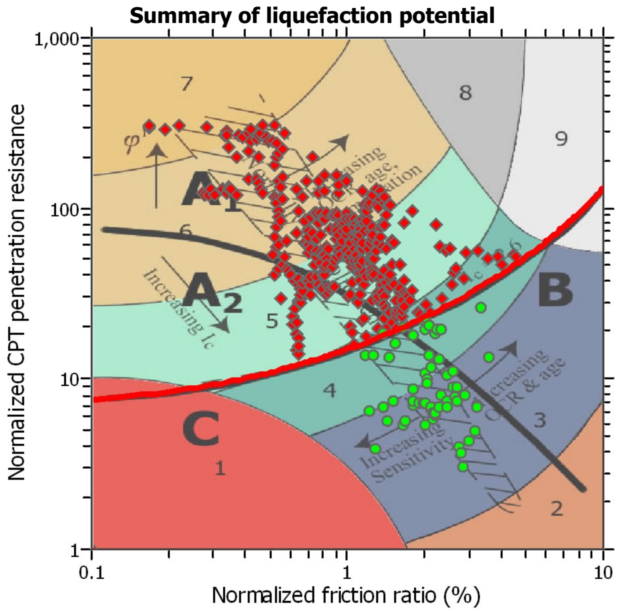
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.01	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.43	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

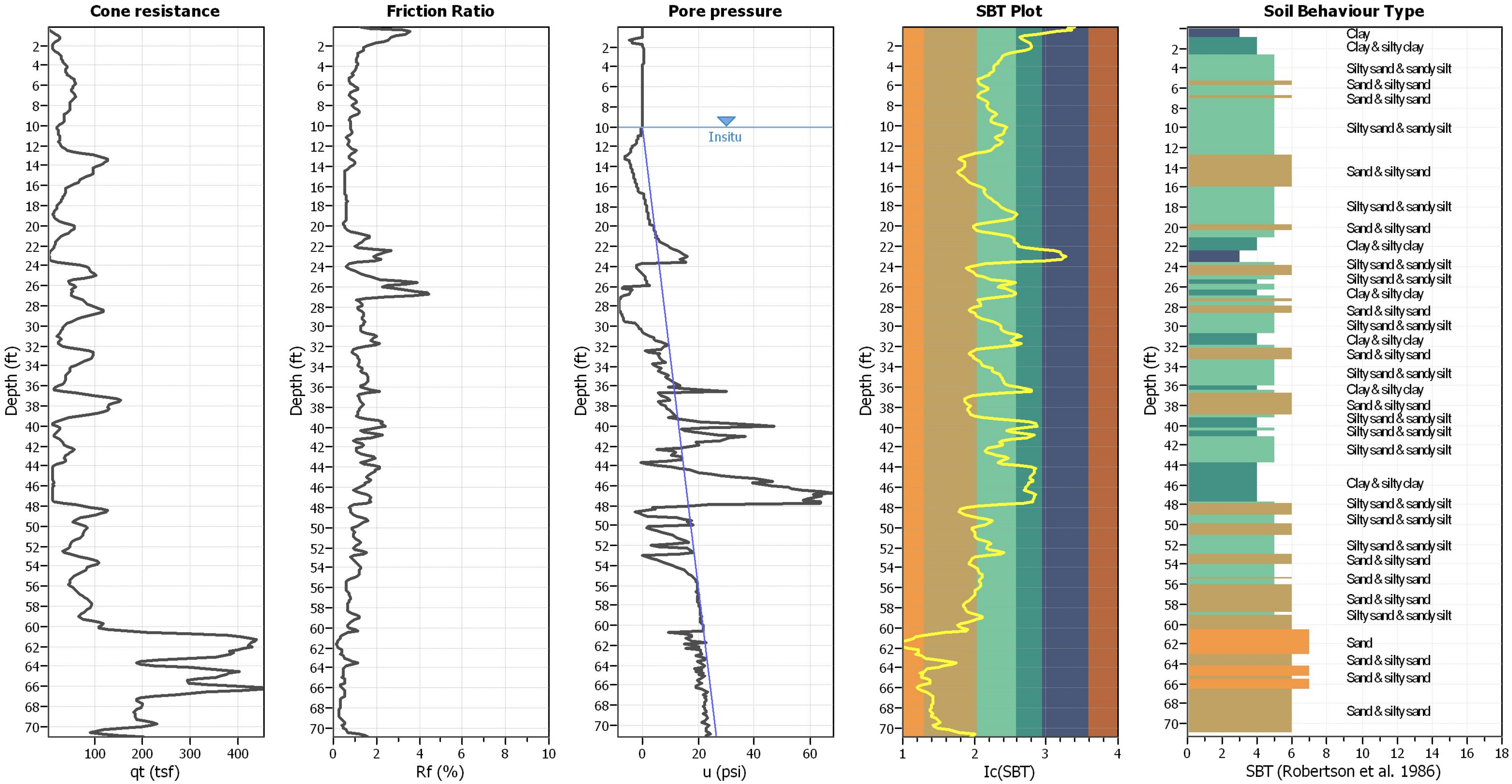






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



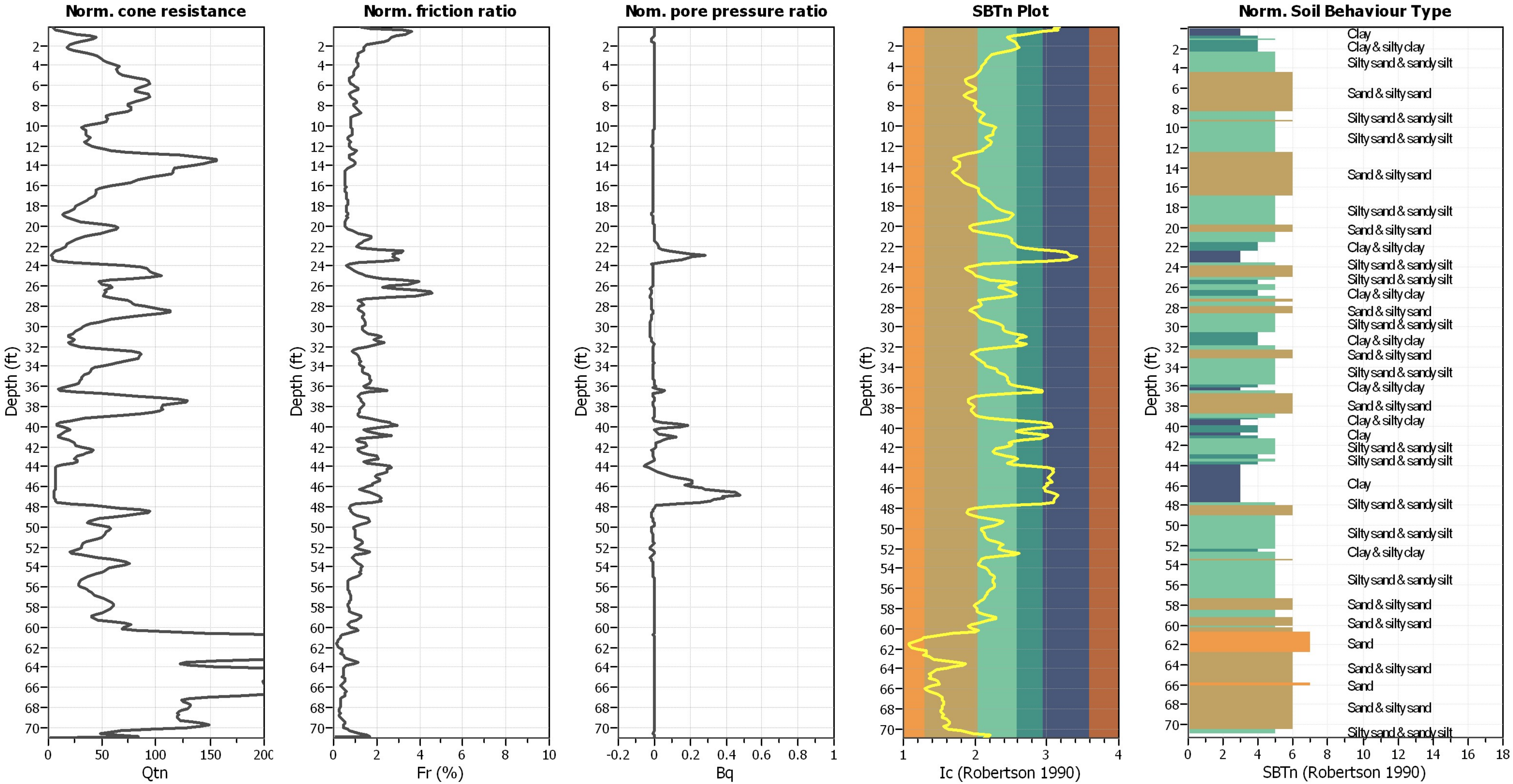
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



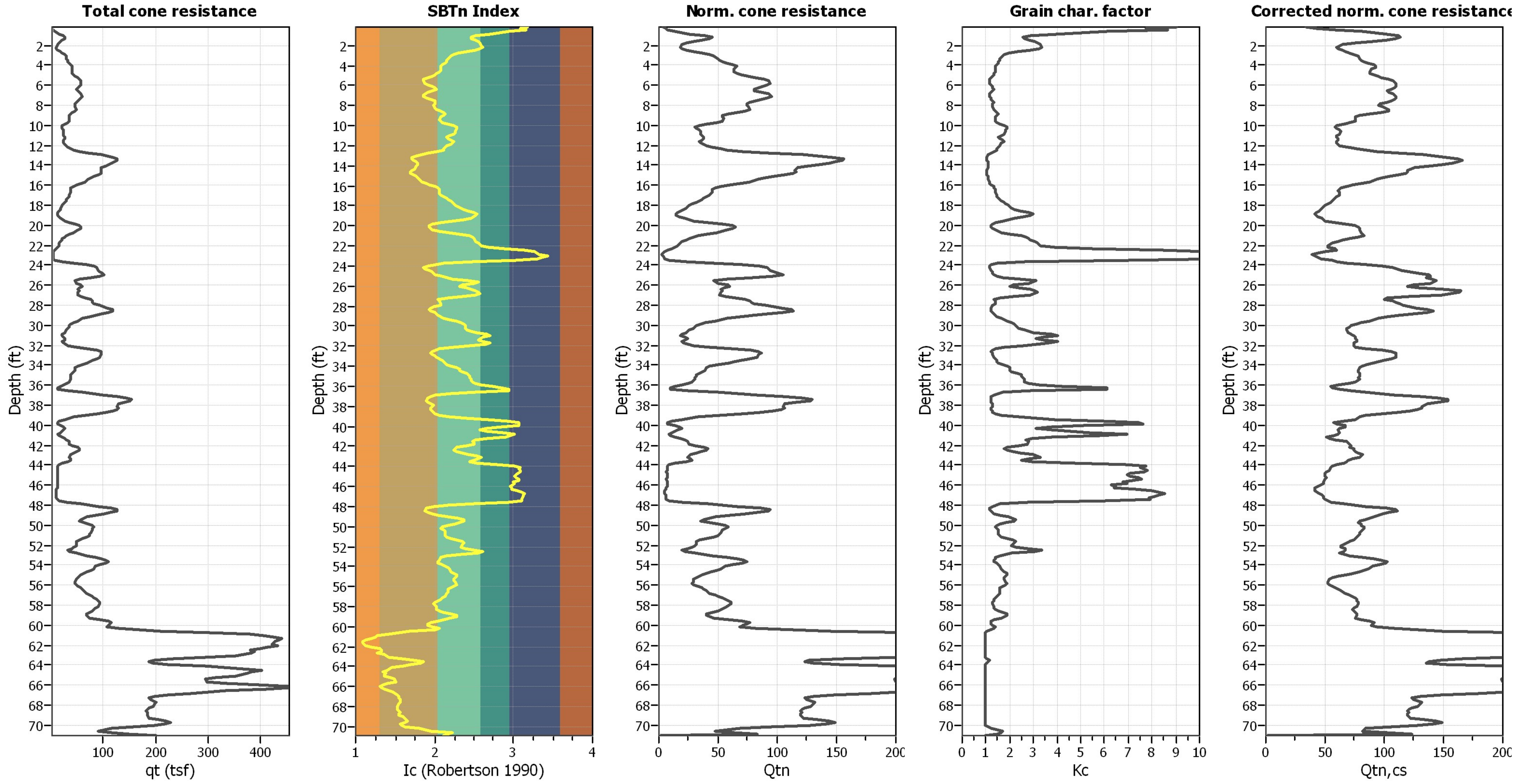
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
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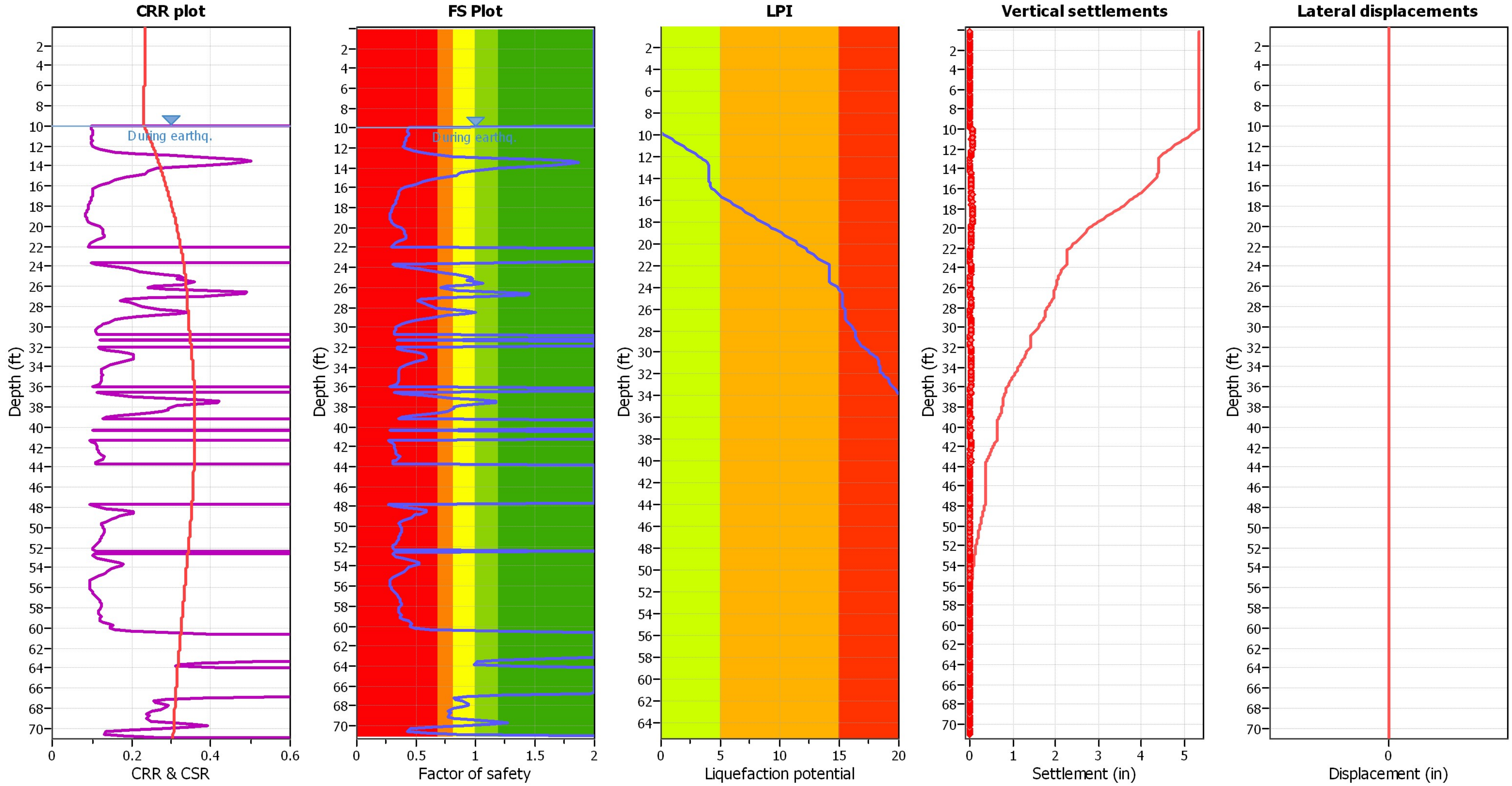
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

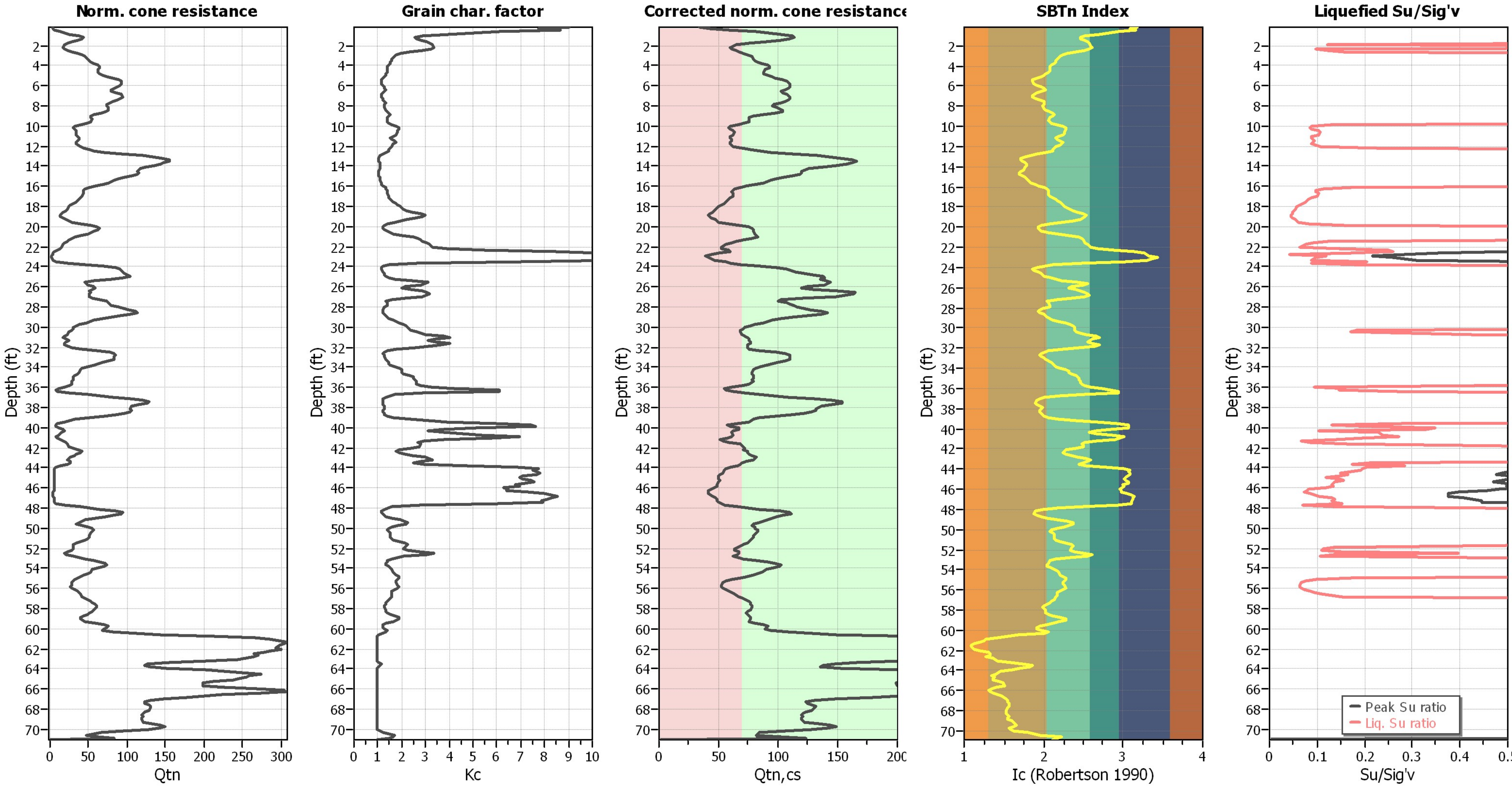
F.S. color scheme

- 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

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

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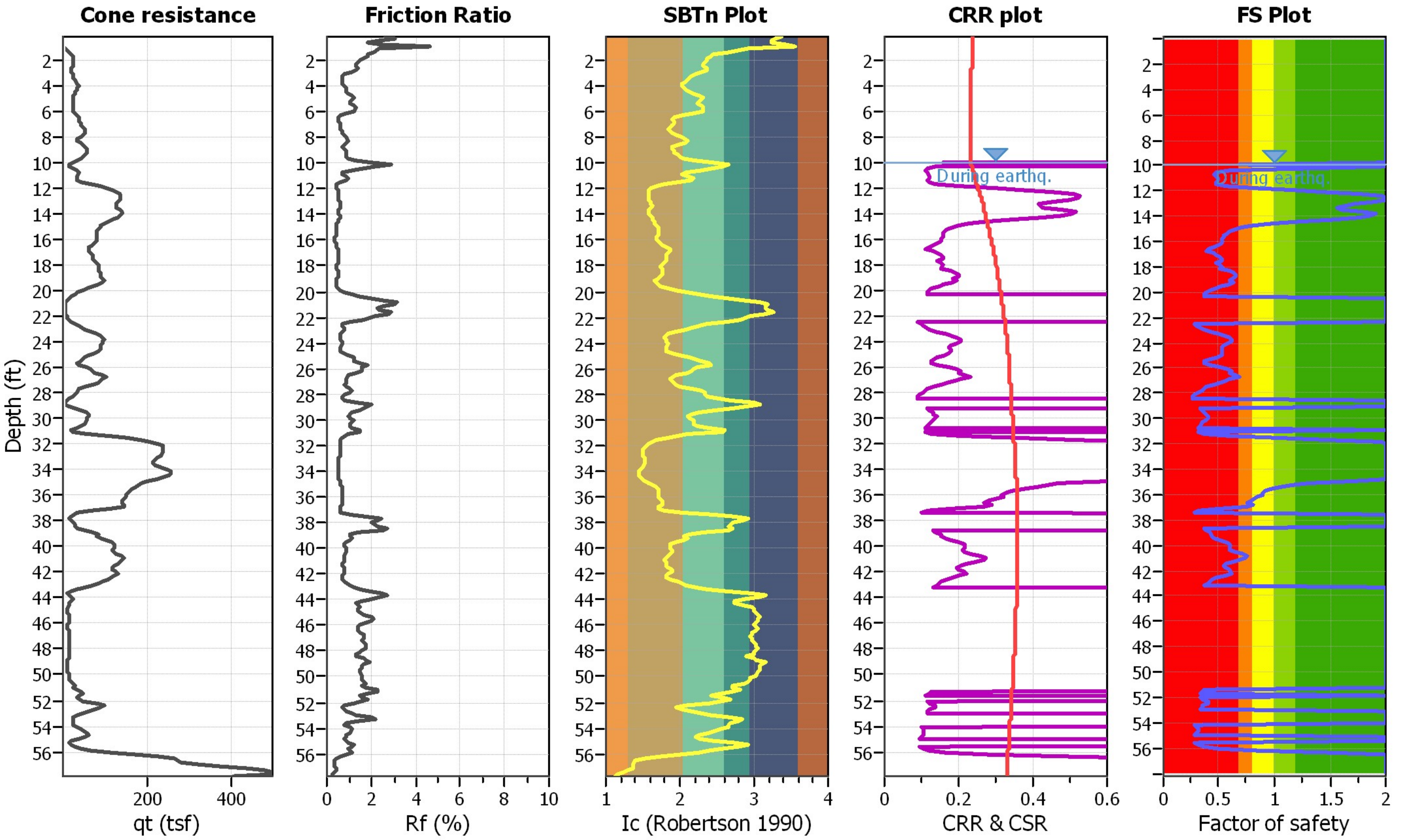
LIQUEFACTION ANALYSIS REPORT

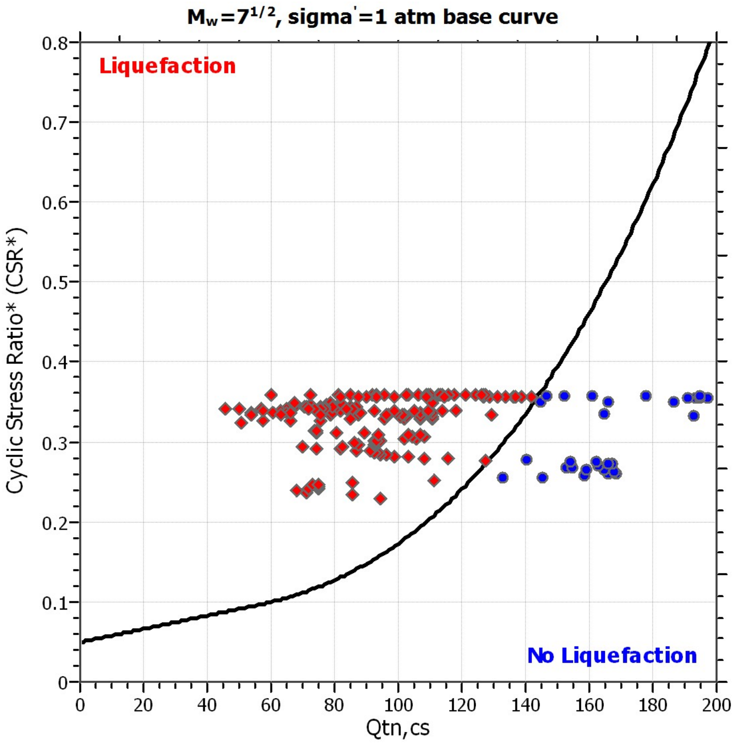
Project title : **Location :**

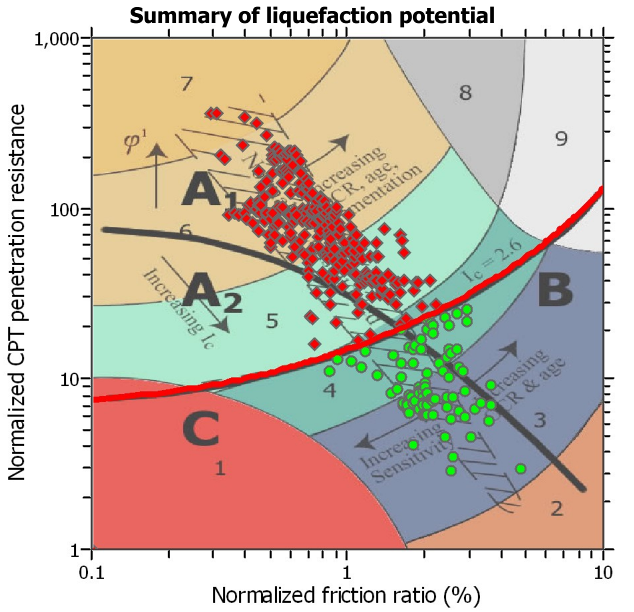
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.01	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.43	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

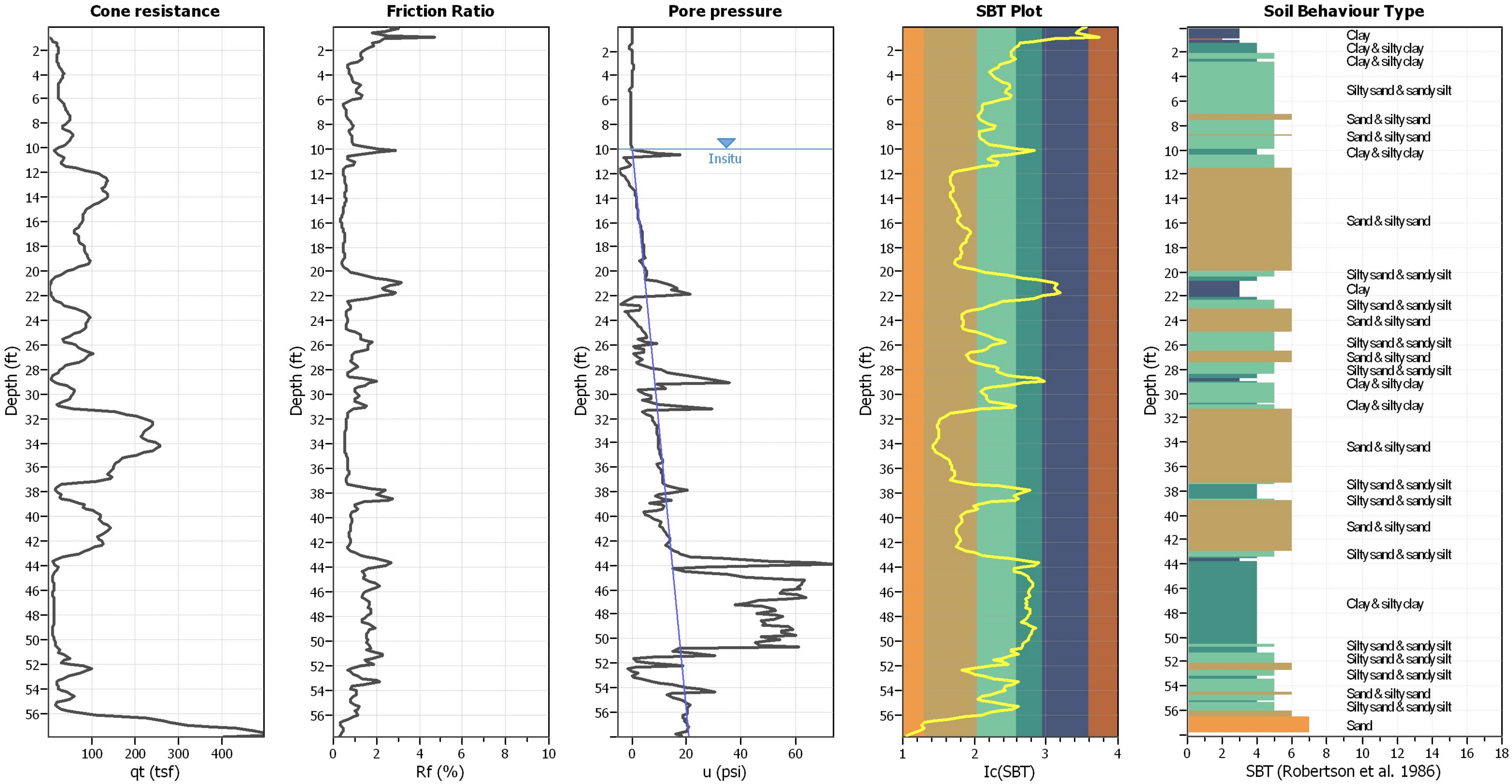






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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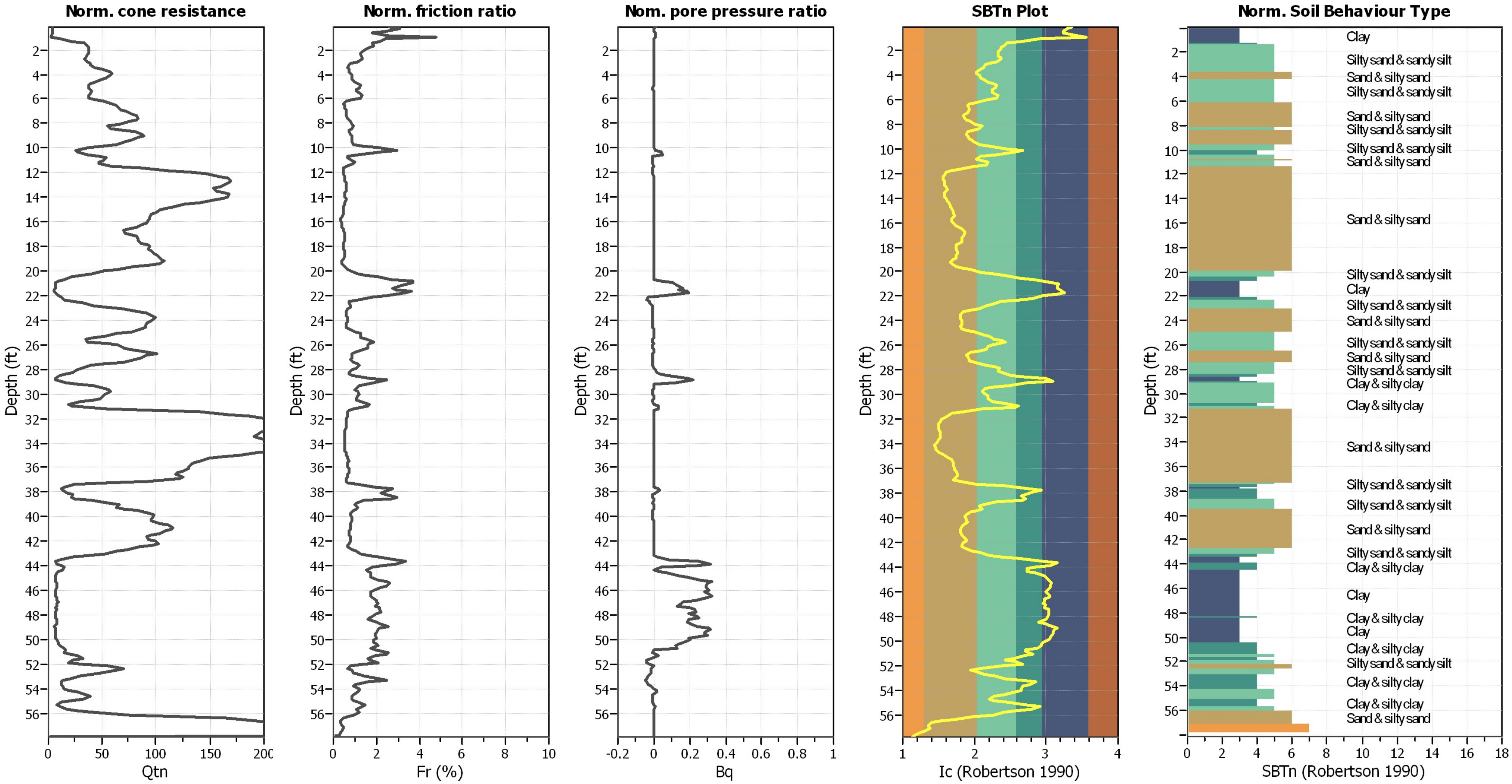
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A		
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SBTn legend

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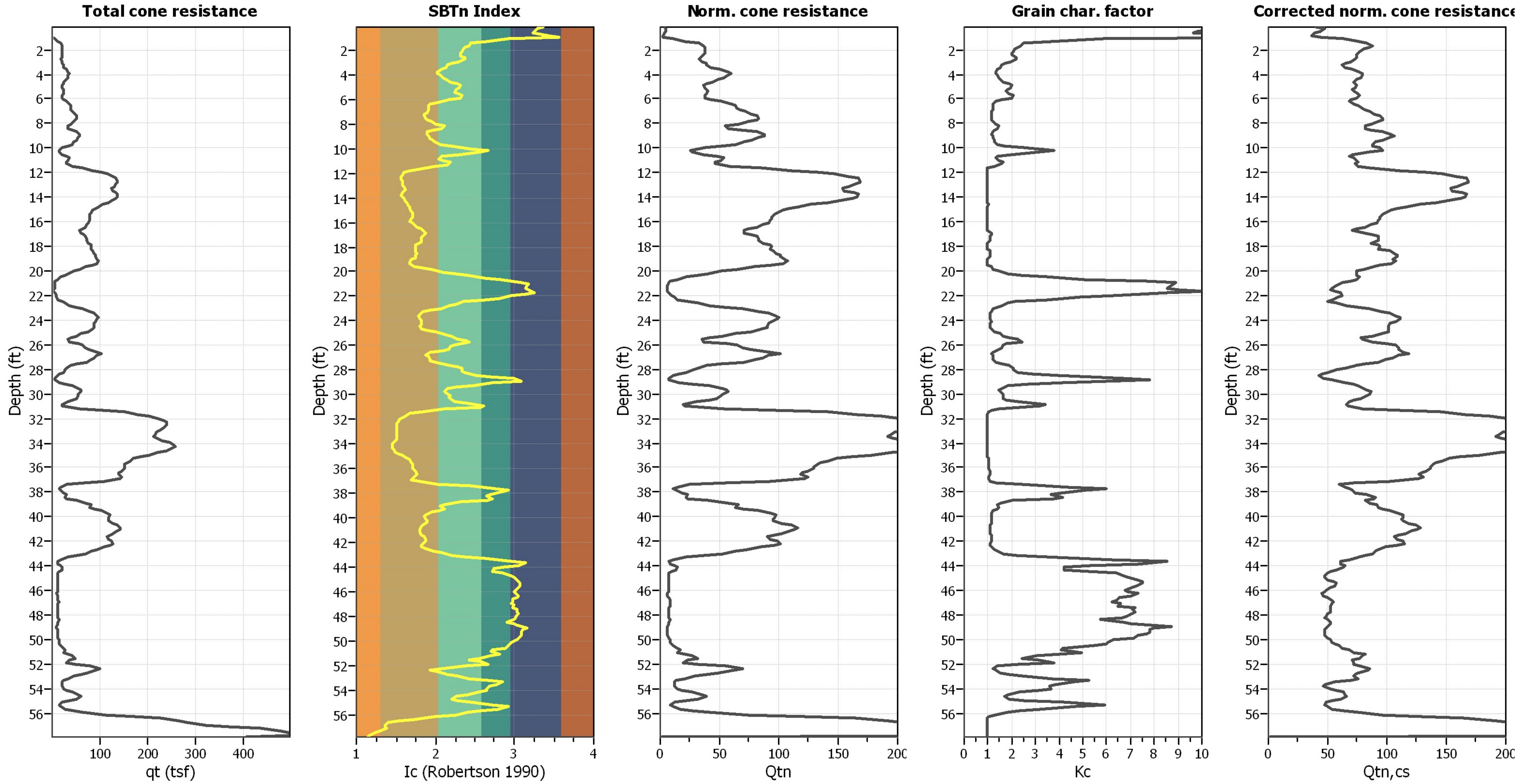
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

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Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
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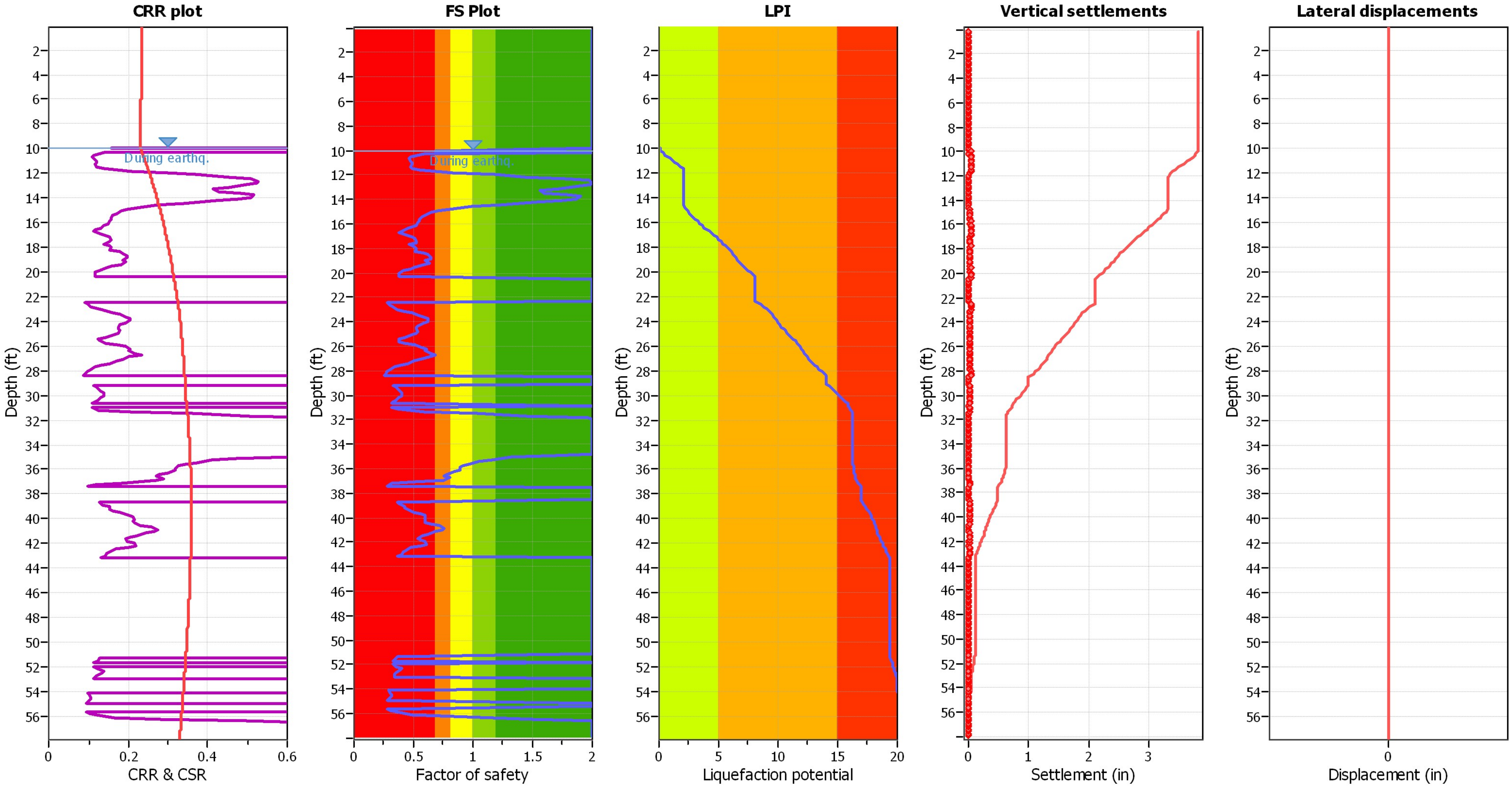
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
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Earthquake magnitude M_w:	7.01	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

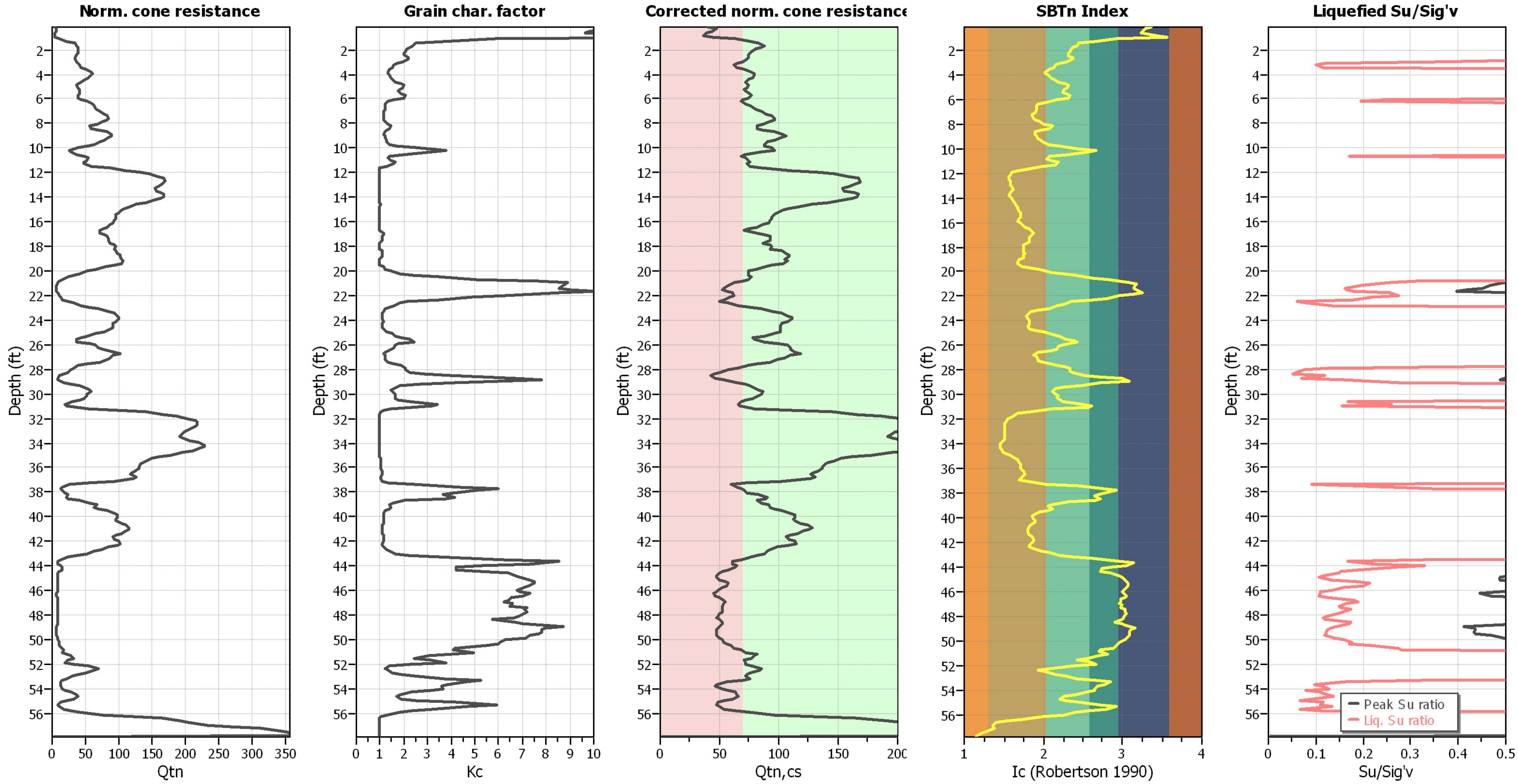
F.S. color scheme

- 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

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
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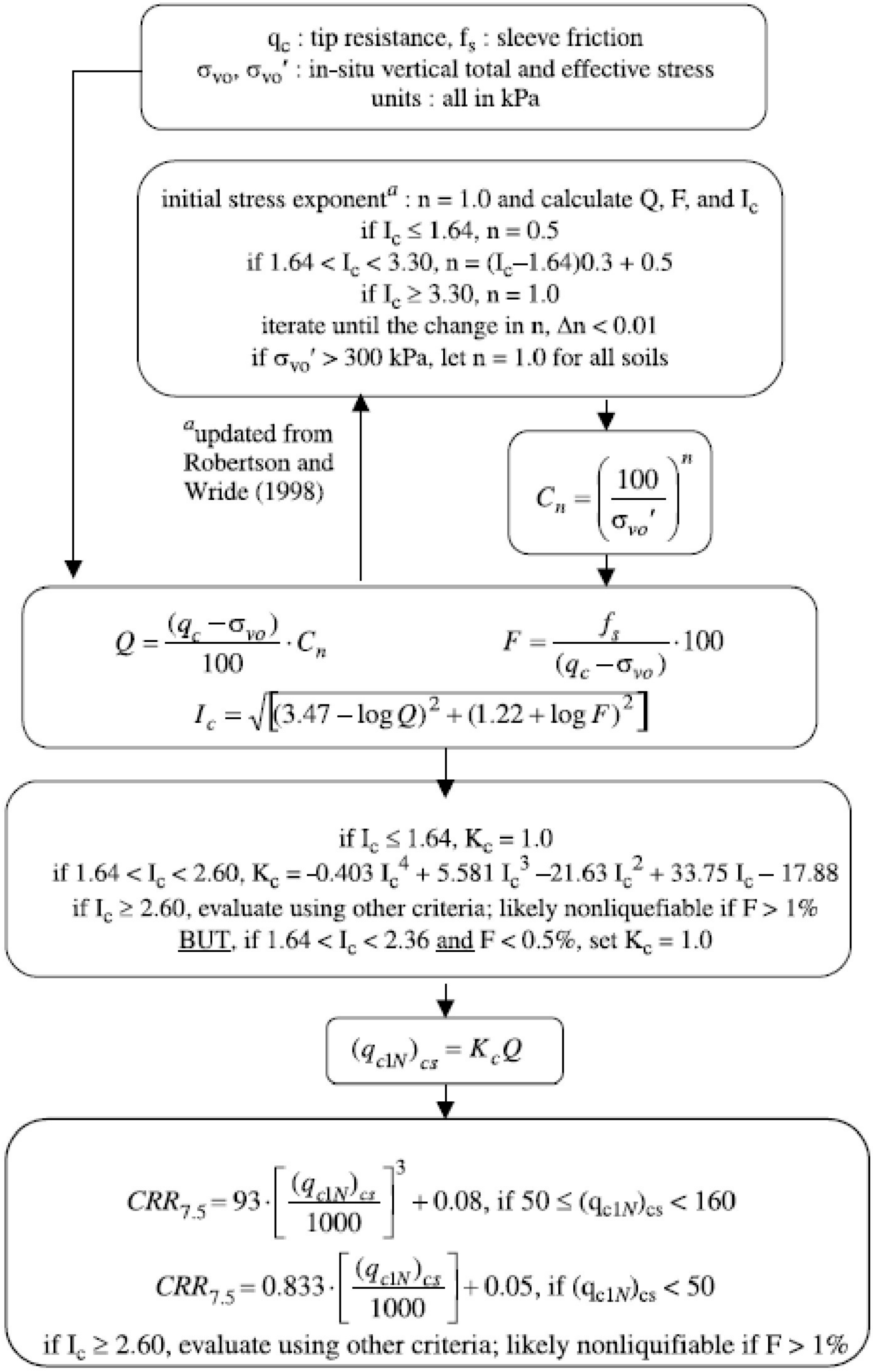
Peak ground acceleration:	0.43	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	N/A

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

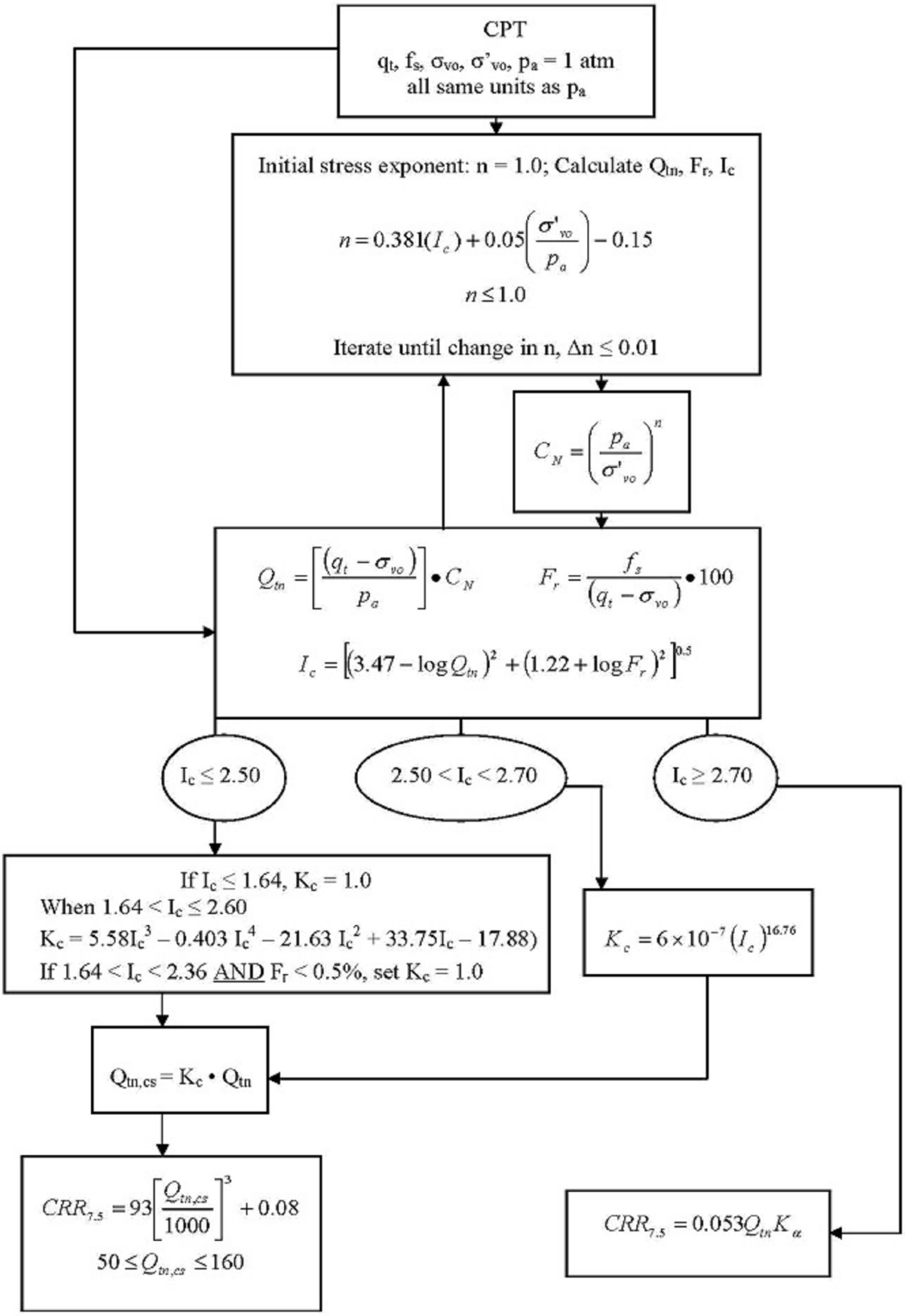
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of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



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

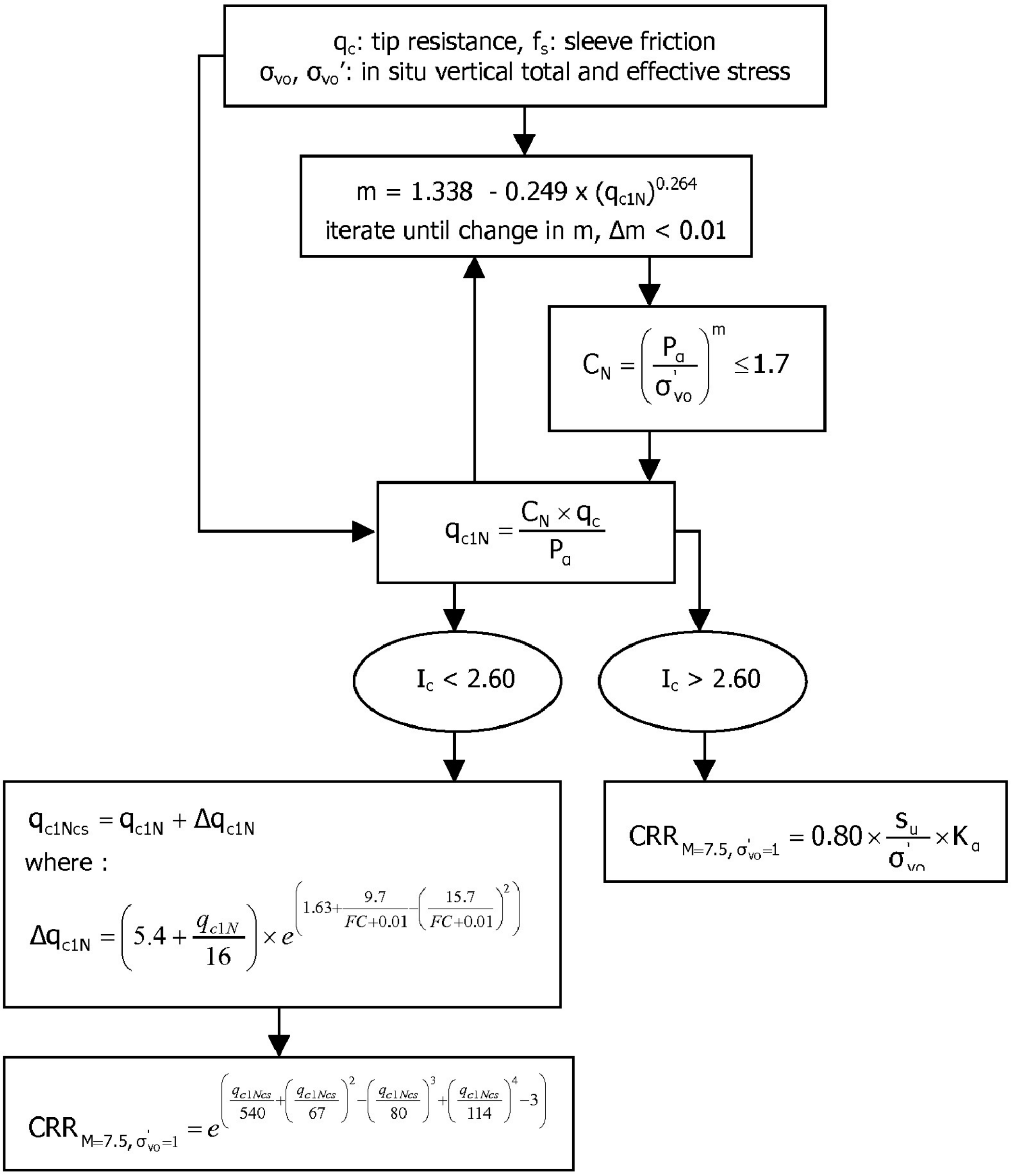
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

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



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

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



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

CPT

q_t, f_s, I_c

$I_c < I_c$ cut-off

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

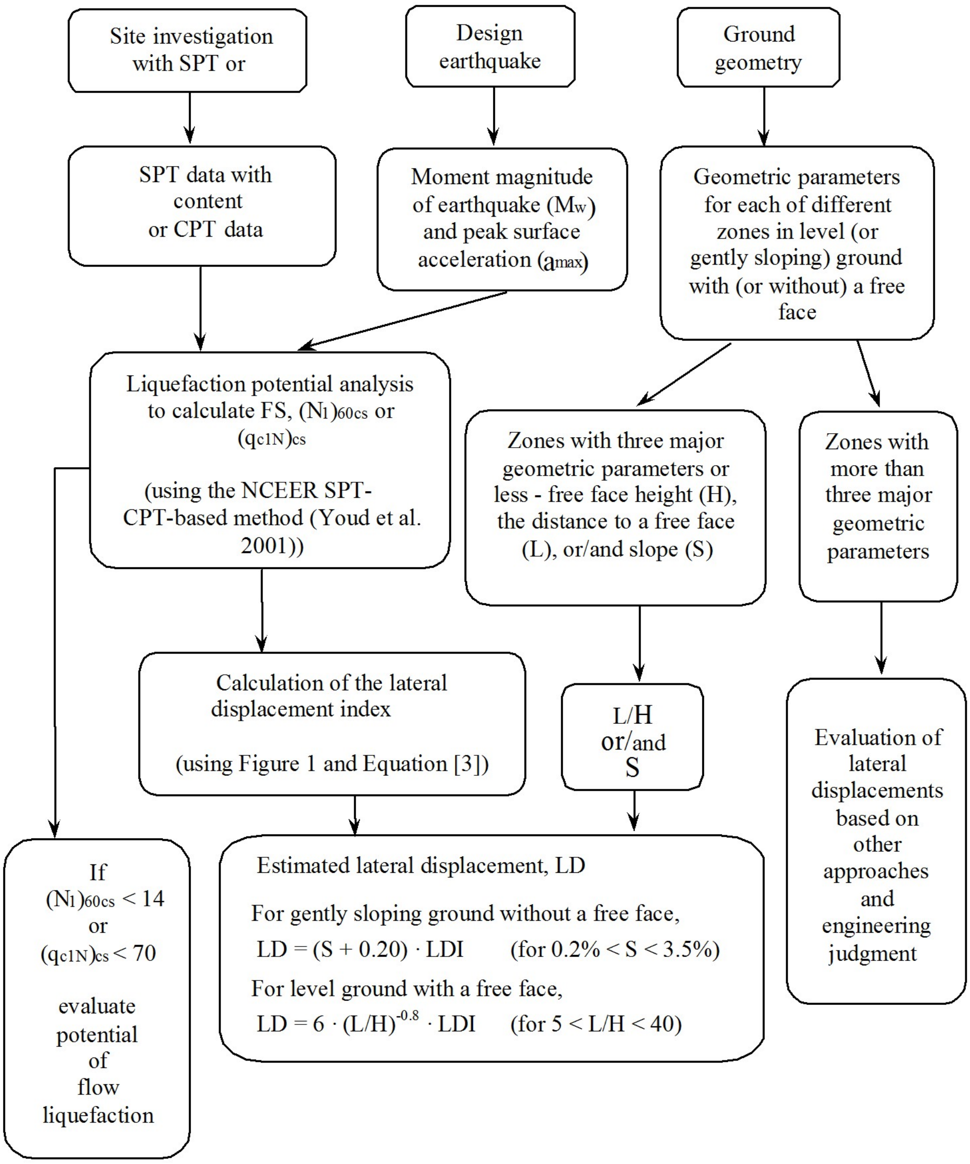
$$c = f_1 \cdot \left(\frac{R_f}{f_3}\right)^{f_2}$$

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

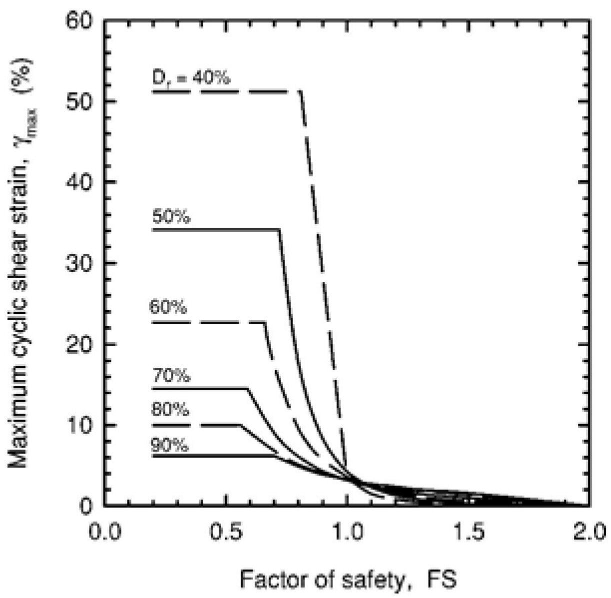
$$q_{t,1} = C_q \cdot q_t$$

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

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



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



[1] **Figure 1**

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

[1] **Equation [3]**

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

Procedure for the estimation of seismic induced settlements in dry sands

Average shear stress, τ_{av}

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

↓

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

↓

Estimate shear strain amplitude, γ

(based on Pradel (1998))

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

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

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

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

↓

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

↓

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

↓

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

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

Liquefaction Potential Index (LPI) calculation procedure

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

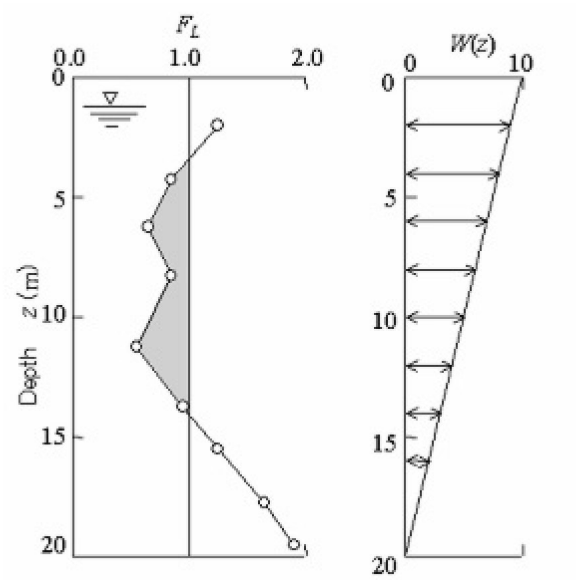
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

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

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

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

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



Graphical presentation of the LPI calculation procedure

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