Appendix E

Geotechnical Report

GEOTECHNICAL STUDY | INVESTIGATION

ARISTA NETWORKS

PROPOSED NEW 4-STORY ABOVE-GRADE OFFICE BUILDING WITH ONE-LEVEL BELOW-GRADE PARKING

ASSESSOR PARCEL NO.: 104-50-011
5200 PATRICK HENRY DRIVE
SANTA CLARA, CALIFORNIA

SUBMITTED TO:

MR. ANDREAS BECHTOLSCHEIM | Chief Development Officer

C/O ARISTA NETWORKS

5453 GREAT AMERICA PARKWAY
SANTA CLARA | California

REGISTERED PROFESSIONAL ENGINEER
ALEX A KASSAI
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Exp. 9/30/23
CIVIL
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AST FILE NO: 21164-S
Date: JANUARY 2022

Signed: January 07, 2022

Al Mirza
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Project Engineer

Alex Kassai
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Principal Engineer
PE #34882 | Exp: 09/30/2023

This report was prepared by the staff of Advance Soil Technology, Inc. under the supervision of the Engineer(s) and or/ Geologist(s) whose seal(s) and signature(s) appear hereon. The findings, recommendations, specifications, and professional opinions are presented within the limits described by the client, in accordance with the generally accepted professional engineering and geologic practice. No Warranty is expressed or/ implied.

AST

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File No. 21164-S
January 07, 2022

Mr. Andreas Bechtolsheim | Chief Development Officer
C/O ARISTA Networks
5453 Great America Parkway
Santa Clara, California 95054

Subject: Proposed New Four-Story Office Building
One-Level Below-Grade Parking Garage
Assessor Parcel No: #104-50-011
5200 Patrick Henry Drive
Santa Clara, California
Geotechnical Study | Investigation Summary

Dear Mr. BechtolSheim -

Advance Soil Technology, Inc. ("AST") is pleased to present herein the results of our geotechnical investigation for the Proposed New Four-Story Office Building with One-Level Below-Grade Parking to be located at 5200 Patrick Henry Drive in Santa Clara, California.

In the proceeding sections of this report, enclosed please find the results of our geotechnical investigation/evaluation of the subsurface soil conditions, which formed the basis of our conclusions, considerations and recommendations related to the geotechnical and foundation design aspects of this project.

Based on the results of our investigation and preliminary analysis, it is our professional opinion that the site is suitable for the proposed development and construction of the above-mentioned structure(s), provided the recommendations presented in our report are incorporated in the design and during the construction phase of the project.

From a geotechnical standpoint, we conclude the site is suitable for the intended development. Our investigation indicates the site is underlain alluvial soil deposits with soft to moderately compressible soils in the upper (±18) to (±21)-feet with interbedded liquefiable layers and very thin layers of organic peats (less than six-inch thick). The estimated static settlements are anticipated from the weight of the proposed structure and from potentially liquefiable soil below the subsurface.

The project will probably involve installing temporary shoring, excavation of the basement to a depth of about (±12) to (±14)-feet below the existing ground surface and constructing of the new above-grade structure above it. From a geotechnical standpoint, we judge the project can be constructed as planned. The primary geotechnical issues for this site are follows: the presence of shallow groundwater, compressible clays with interbedded organic clay and sand layers beneath the basement level, seismically-induced settlement, the support of adjacent streets, utilities, effects of hydrostatic pressures on the basement floor and walls due to shallow groundwater table. Our conclusions and recommendations regarding these issues are discussed in the remainder sections of this report.

Please note that the conclusions and recommendations presented in this report are based on the subsurface soil investigation, variations between the anticipated and the actual subsurface soil conditions may occur in localized areas during the construction phase of the project. It is recommended that Advance Soil Technology, Inc. (AST) be retained during construction phase of the project to observe earthwork operations, and installation of foundations/verification to make changes and provide additional recommendations as deemed necessary, due to varying subsurface soil conditions. Furthermore, it is also recommended that AST review the plans and specifications pertaining to the grading and foundation aspects of the project, prior to completion of the final construction documents to assure compliance to the recommendations presented in this report.

Very truly yours,
ADVANCE SOIL TECHNOLOGY, INC.

REGISTERED PROFESSIONAL ENGINEER
ALEX A KASSAI
C34882
Exp. 9/30/23
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Kier & Wright Inc. | Mr. Mark Knudsen

TABLE OF CONTENTS

TABLE OF CONTENTS (cont)

TABLE OF CONTENTS (cont)

GEOTECHNICAL STUDY | INVESTIGATION
ARISTA NETWORKS
PROPOSED FOUR-STORY ABOVE-GRADE STRUCTURE
ONE-LEVEL BELOW GRADE PARKING
ASSESSORS PARCEL NO.: 104-50-011
5200 PATRICK HENRY DRIVE
SANTA CLARA, CALIFORNIA

1.0 INTRODUCTION

Advance Soil Technology, Inc. ("AST") is pleased to present herein the results of our subsurface geotechnical investigation/study for a proposed new four-story structure with one-level below-grade parking for Arista Networks, New Office Building to be located at 5200 Patrick Henry Drive in Santa Clara, California.

The site of proposed development consists of single-parcel of land located to the west of Patrick Henry Drive in Santa Clara, California. It is an irregular shaped parcel of land with plan dimensions approximately ranging between (±424.86) to (±487.45)-feet along the northern and southern boundary and (±527.01) to (±572.36)-feet along the eastern and the western boundary respectively. As mentioned above, the site is located in the Santa Clara with Patrick Henry Drive bordering to the East, Calabazas Creek bordering to the west and other commercial commercial/office properties bordering to the north and south of the subject property.

At the time of this geotechnical investigation, the site of proposed development was occupied by a single-story structure with walkways and elevated landscape areas with drive thru and parking areas on all four sides of the building. The north-western side of the building had an existing loading dock, trash compactor, possibly a generator and other mechanical equipment located along the northwest corner of the building.

The purpose of this geotechnical investigation was to evaluate the existing subsurface soil conditions and provide recommendations for the proposed development and for surrounding on and off-site improvements associated with it.

Based on the results of our sub-surface investigation and analysis, it is our professional opinion that the site is suitable for the proposed development and construction of a four-story above-grade structure with one-level of below-grade subterranean parking, provided the recommendations presented in the following sections of our report are incorporated in the design and during the construction phase of the project.

The subsurface investigation at the site included exploration extending to depth (±75) to (±80)-feet below the existing ground surface. The subsurface materials encountered at the site generally consists of alluvial deposits with highly expansive and soft to moderately compressible clays, which are underlain with interbedded layers of organic peats, medium dense to dense sandy silts/silty sands along with some layers of clayey silts and stiff clays that extended to the depth of our cone penetration tests and exploratory borings. Some of these sandy silts and sandy layers that are medium dense could potentially liquefy during a seismic event. Details regarding these subsurface conditions and their effect on foundation design are presented in the following sections of this report; therefore, anyone relying on this report should read it in its entirety.

In the proceeding sections of this report, please find recommendations for the earthwork operation/grading and foundation design, based on the results of this investigation that are to be incorporated in the design and during the construction phase of the project.

Please refer to the site plan enclosed in Appendix “A” of this report for information pertaining to the site of proposed development and the locations of the cone penetration test(s) and exploratory boring(s).

2.0 PROJECT DESCRIPTION

As mentioned above, the site of proposed development is a single-parcel property owned by 5200 Patrick Henry Drive, LLC (Arista Networks) located in a commercial part of City and County of Santa Clara. This parcel that is currently under consideration for development borders Patrick Henry Drive with APN Parcel No. 104-50-011 with an approximate square footage of (±5.64)-acres. The project is being developed as follows as per the conceptual exhibit that was submitted to City of Santa Clara along with other on and off-site improvements at the site, such as underground utilities, exterior flatwork, rigid and flexible parking areas, landscape and low impact development areas etc.

- Proposed Office Space: ± 182,500 SF
- Proposed Below-Grade Parking: ± One-level SF
- Building Footprint: ± 365X130-Feet
- Basement Footprint: ± 365X195-Feet

TOPOGRAPHIC INFOMATION: Based on the site reconnaissance, the site of proposed development is a flat parcel of land and so is the natural topography of the area in general with the exception of the elevated area along the western boundary bordering the Calabazas Creek. Review of the existing available geological maps, aerial photographs, Google Earth Maps and Site Topographic Plans from Kier & Wright, Land Surveyors/Civil Engineer, it is our understanding that the elevation at the site based on the existing site conditions has been determined to be approximately in the range of (±10) to (±11)-feet above mean sea level (msl).

STRUCTURAL INFORMATION: The structural loads and aerial pressures provided by the structural engineer are presented in the table below. Based on the conceptual information received, it is our understanding that the proposed structure is a braced frame office building with possible precast panels and walls. The anticipated maximum structural loads (dead plus live loads) for the structures are as stated and reflected in the table below:

- Total Building Dead Loads (DL): 43,000 Kips
- Total Building Live Loads (LL): 26,000 Kips
- Areal Pressure Dead Loads (DL): 800 psf
- Areal Pressure Live Loads (DL): 450 psf

TABLE I – BUILDING LOADS & INFORMATION

Building No.	No. of Stories	Max. Column Loads (DL+LL) kips	Areal Pressure (DL+LL) psf	Lateral Load kips
Building	4	±1700 kips (Excluding the weight of Mat)	1250 psf (Excluding the weight of Mat)	1500 kips
Below Grade Parking	1			

3.0 PURPOSE AND SCOPE OF SERVICES

The purpose of our investigation was to evaluate the existing subsurface soil conditions at the site as necessary to characterize subsurface strata, geologic hazards and develop geotechnical recommendations for the structural design and construction of the proposed development. The scope of our services for this study included the following:

- Reconnaissance of site of proposed development, location of the existing underground utility lines and subsurface storage units (if any) with respect to the Exploratory Boring and CPT locations, prior to commencement of the drilling operation.
- Mark boring/CPT locations and notify Underground Service Alert at least (72)-hours prior to the planned exploration activity.
- Exploration of subsurface soil conditions at the site of proposed development included advancing (5)-Cone Penetration Tests with Middle Earth Geo-Testing (MEGT) extending to a depth of approximately (±80)-feet below existing ground surface (bgs) below the existing ground surface and Drilling (2)-Exploratory Soil Boring with Exploration Geo-Services, Inc. (EGI) using hollow-stem auger extending to a depth of approximately (±75)-feet below the existing ground surface (bgs) for evaluating the subsurface soil conditions.
- Research and review of pertinent geotechnical information, report(s) and geological maps relevant to the site regarding seismic and geologic history of the site and the immediate vicinity.
- Review of Existing Geotechnical Reports by other consultants for various projects in the area and historic aerial photographs for topographic changes and information pertaining to tonal variations (if any) at the subject property.
- Evaluation of the potential local and regional geologic hazards at the site, including liquefaction and resulting seismic settlements as per the requirement of the California Geological Survey (CGS)
- Perform laboratory testing and analysis of the soil samples to evaluate the engineering characteristics of the subsurface materials. Laboratory tests included Soil Classification, Atterberg Limits Tests; In-situ Moisture Density, Consolidation, Direct Shear, Corrosivity Analysis and other tests as deemed necessary.

Engineering analyses based on the results of laboratory testing and strength characteristics of the subsurface soils included the following:

- Review of published geology and seismology reports and fault maps pertinent to the site area regarding the existing conditions in the vicinity of the site.
- Soil classification and seismic design parameters based on ASCE 7-16 and CBC 2019 (California Building Code). Define specific site conditions and subsurface soil profile encountered in the cone penetrometer test and exploratory borings.
- Geology and seismicity of the project, including the appropriate soil profile type and other seismic parameters per 2019 Edition of CBC.
- Impact of groundwater or/ seepage on below grade structures and retaining walls, etc. based on the depth, design and construction of the proposed improvements.
- Seismic evaluation of the site including seismic compaction, ground shaking, distance from the earthquake faults, seismic coefficients per the requirements of 2019 CBC (Reference), site specific response spectra and future earthquake probability.
- Overall assessment of the general surface and subsurface soil conditions and impact due to static settlement under the anticipated structural loads including potential for settlement due to liquefaction during a seismic event, which is considered to be in addition to the static settlement and mitigation measures.
- Recommendations for the type of foundation system for the structures are based on anticipated structural loads and the type of structure, total and differential settlements etc.
- Design criteria for structurally supported concrete slabs, concrete floors, non-structural slabs and modulus of subgrade reaction.
- Recommendations for retaining walls including lateral earth pressures (active and at-rest), seismic increments, drainage behind walls etc.
- Impact of soil corrosion on buried elements, including below grade foundation, walls and other concrete structures, buried steel and underground utilities in general.
- Recommendations for overall site grading, engineered fill materials, lime treatment or/ chemical treatment of subgrade soils, surface and subsurface drainage.
- Recommendations for Flexible, Rigid and Permeable Pavement Design Sections for Low Impact Development (LID) areas.

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The site of proposed development under this investigation is located in a commercial part of the City and County of Santa Clara, California. It is a relatively flat parcel of land and so is the natural topographic of the area with the exception of the elevated slope/landscape along the western boundary. The site of proposed development was identified in the United States Geological Survey (USGS) Milpitas Quadrangle Map 7.5-minute Series, which was reviewed for this geotechnical study. It is located approximately at the following co-ordinates: Latitude 37.40641º North and -121.985366º West Longitude respectively.

As mentioned above, the site of proposed development consists of a single parcel with an approximate square footage of (±5.64)-acres. It is an irregular shaped parcel located on the west side of Patrick Henry Drive in the City and County of Santa Clara, California. It is bound by adjacent commercial parcels to the north and south with Patrick Henry Drive bordering to the east and lands of Santa Clara Valley Water District (SCVWD) to the West respectively.

At the time of this geotechnical investigation, the site was occupied by an existing single-story structure surrounded with walkways, elevated landscape areas, loading dock and asphalt paved parking and drive thru areas with driveway access from Patrick Henry Drive.

Based on the contour lines reflected on the preliminary draft copy of the site topographic survey provided by Kier & Wright Civil Engineers and Land Surveyors, it is our understanding that the ground surface elevations at the site range approximately between (±10) to (±11)-feet above mean sea level (msl), based on the NAVD 1988 Datum.

We also reviewed FEMA electronic files/floodplain maps/data (Federal Emergency Management Agency) Map for the Santa Clara County, Map No. 06085C0061H dated May 18, 2019, the site is located in a flood zone area “Shaded X”. According to FEMA the definition of “Shaded X” indicates an area with 0.2% annual chance of flood hazard, areas of 1% chance flood with an average depth of one-foot or/ with drainage areas of less than one square mile. We recommend that the Project Civil Engineer be retained to confirm this information and verify the base flood elevation (if appropriate).

4.2 FIELD INVESTIGATION

Middle Earth Geo-Testing and Exploration Geo-Services were subcontracted to provide CPT sounding and exploratory drilling services respectively to evaluate the subsurface soil conditions and the potential for liquefaction at the site. The subsurface soil investigation at the site was performed on November 29 and December 09, 2021. Prior to commencement of the drilling operation, the approximate exploratory boring and CPT locations were identified and marked as selected by AST based on the conceptual architectural/civil topographic site plan and the proposed location of the structure(s). All CPT and exploratory boring locations were cleared by a private underground utility service locators and USA, prior to commencement of the drilling operation.

4.2.1 CONE PENETROMETER TESTS

A total of (5)-CPT Soundings (Cone Penetration Tests) were advanced at the site of proposed development. The CPT soundings were advanced using a truck mounted integrated electronic cone system to a depth of approximately (±80)-feet below the existing ground surface. The CPTs were advanced/performed in general accordance with ASTM D3441 and ASTM D5778. The information gathered from CPTs was used for identifying the potential for liquefaction, soft compressible soils and to evaluate the foundation support design criteria.

As mentioned above, the cone penetrometer tests (CPTs) were advanced using truck mounted rig within the areas of proposed development. The log for the CPTs showing tip resistance and friction ratio by depth, as well as interpreted SPT N-values, friction angle, soil shear strength parameters and an interpreted soil classification as presented in the following sections of this report. The stratigraphic interpretation of the collected CPT data was performed based on relationships between cone bearing and sleeve friction versus depth of penetration. The friction ratio (Rf), which is sleeve friction divided by cone bearing is a calculated parameter which is used to infer the type of soil behavior.

Generally, cohesive soils (clays) have high friction ratios and low cone bearing and generate large pore pressures. Cohesionless soils (sands) have lower friction ratios, high cone bearing and generate small excess pore water pressures. The interpretation of the soil properties from the cone data has been carried out using recent correlations developed by Robertson et al, 1986 and Olsen, 1988. It should be noted that it is not always possible to clearly identify a soil type based on cone bearing (Qc) and sleeve friction (Fs). In these situations, experience and judgment and an assessment of the pore pressure dissipation data should be used to infer the soil behavior type.

The CPT data collected from the sounding (cone bearing, sleeve friction, friction ratio and equivalent standard penetration test blow counts (N) versus penetration depth below the subsurface is presented in the attached report in Appendix "B" of the final geotechnical report.

4.2.2 EXPLORATORY BORINGS:

A total of (2)-exploratory borings were also drilled at the site to a depth of approximately (±75)-feet below the existing ground surface. The exploratory borings were advanced using a truck mounted drill-rig utilizing an eight-inch diameter hollow stem auger. During drilling, the soil was logged and samples of the material encountered were obtained for visual classification and laboratory testing. The soil was logged in accordance with the soil classification system described and the data will be presented upon completion of the laboratory analysis.

Soil samples were collected using two different types of samplers: two driven split-barrel samplers. The sampler types are as follows: 1) Sprague & Henwood (S&H) split-barrel sampler with a (3.0)-inch outside diameter and (2.5)-inch inside diameter, lined with steel or brass tubes with an inside diameter of (2.43)-inches and 2) Standard Penetration Test (SPT) split-barrel sampler with a (2.0)-inch outside diameter and (1.5)-inch inside diameter, without liners. The sampler types were chosen on the basis of soil type being sampled and desired sample quality for laboratory testing.

In general, the S&H sampler was used to obtain samples in stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of sandy soils. The SPT and S&H samplers were driven with a 140-pound, above-ground, automatic safety hammer falling (30)-inches. The samplers were driven up to (18)-inches and the hammer blows required to drive the samplers for every six-inches of penetration were recorded and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors to account for sampler type and hammer energy and are shown on the boring log. The blow counts used for this conversion were the last two blow counts for bottom (12)-inch penetration of the sampler. The soil cuttings generated from the borings were left on-site in the (55)-gallon drums and the boreholes was backfilled with cement grouted under the supervision of the inspector in accordance with the requirements of the Santa Clara valley Water District (SCVWD).

The location of the CPT(s), Exploratory Borings and Elevations were estimated by AST based on rough measurements from the existing features at the site and the preliminary topographic survey provided by the Civil Engineer Kier & Wright Civil Engineers & Land Surveyors, Inc.

4.3 SUBSURFACE SOIL CONDITIONS AT THE SITE

The subsurface soil conditions encountered at the location of the CPTs (cone penetrometer tests) and the exploratory boring that were advanced or/ drilled at the location of the proposed development were as follows:

A total of (5)-CPTs and two (2) exploratory borings were advanced/drilled within or/ in close proximity to the building footprint with designation CPT-01, 02, 03, 04, 05, EB-01 and EB-02 were advanced to a depth of (±75) to (±80)-feet below the existing ground surface (bgs) for evaluating the subsurface soil conditions. At the location of these above-mentioned CPTs and Borings, the following existing pavement section (asphalt and aggregate base) was encountered and they area s follows:

TABLE 2 – EXISITNG PAVEMENT SECTIONS

CPT/Boring Location	Asphalt/Concrete (± inches)	Silty Sandy Gravel (± inches)
CPT No. 1	4½	9 to 10
CPT No. 2	2¼	6 to 7
CPT No. 3	6½ (concrete)	6½
CPT No. 4	2.0	6 ½
CPT No. 5	2½	6 to 7
EB No. 1	2½	6 to 7
EB No. 2	1½	8 to 9

Below these above-mentioned pavement sections, mostly organic peat, fine grained and sensitive soils were encountered on the surface (appears to be mostly fill material) and extended to a depth of approximately (±5.0) to (±6.0)-feet below the existing ground surface. At this depth mostly medium stiff to stiff clays with interbedded layers of medium dense to dense clean sands to silty sandy layers that were encountered at a depth of (±36.0)-feet in CPT No. 1 and extended to a depth of (±39.0)-feet below the existing grade and in CPT No. 3, sandy layers were encountered at a depth (±21.0)-feet and extended to a depth of (±32.0)-feet below the existing grade. Below these depths mostly lean clays to silty clays were encountered and extended to the bottom of the CPTs and the exploratory borings. The exploratory borings were terminated at a depth of (±75)-feet below the existing ground surface (bgs) and the CPTs were terminated at a depth of (±80-feet) below the existing ground surface (bgs) respectively. In general, CPTs and borings indicated mostly alluvial soils with behavior types including lean clay, silty clay to clay, clayey silt to silty clay, sandy silt to clayey silt, silty sand to sandy silt, sand to silty sand, and sand etc.

Where tested, the undrained shear strengths of the clay range from 487 to 1,900 psf. Laboratory test results and data from the CPTs indicate the clay is moderately compressible and is normally consolidated to overconsolidated. Laboratory test results indicate the clays have compression ratios of 0.11 to 0.12 and over consolidation ratios (OCRs) ranging approximately between 1.5 to 2.0. Where tested, the sandy layers contain about 5 to 48 percent fines (passing Sieve #200).

Please note that the above information depicts the existing subsurface soil conditions at the specific CPT and boring locations as reflected on the site plan. Stratification lines represent the approximate boundaries between the material types. The actual transitions between the materials may be gradual. Subsurface soil and groundwater conditions may vary at other locations from the conditions that were encountered at the CPT and boring locations with the passage of time. For subsurface soil information, please refer to the boring and CPT logs on Plates (8) thru (33) enclosed in Appendix "B" of this report.

4.4 GROUNDWATER ELEVATION

Groundwater elevation at the site could fluctuate and eventually stabilize to the required depth over a period of time. However, based on the review of the CGS Seismic Hazard Zone Map Report (051) for the Milpitas Quadrangle, it is our understanding that the historic groundwater elevation in the vicinity of this site is being considered to be at a depth of approximately (±3.5) to (±4.0)-feet below the existing ground surface. Based on the groundwater elevations measured in the exploratory borings and the pore pressure data that was collected from the CPTs, free groundwater elevations at the site were as follows:

TABLE 3 - GROUNDWATER ELEVATIONS

Exploratory Boring/CPT	Date	Groundwater Depth Below Existing Ground Surface (±) feet	Existing Ground Elevation (msl) feet	Groundwater Elevation (msl) feet
CPT-01 (PPDT)	11/29/2021	4.2	10.00	5.8
CPT-02 (PPDT)	11/29/2021	9.4	10.00	0.6
CPT-03 (PPDT)	11/29/2021	5.9	10.00	4.1
CPT-04 (PPDT)	11/29/2021	6.5	10.00	3.5
CPT-05 (PPDT)	11/29/2021	8.6	10.00	1.4
EB-01	12/09/2021	4.9	10.00	5.1
EB-02	12/09/2021	5.0	10.00	5.0

Notes:
1. Existing Ground Elevation Based on Contours from Site Topographic Map by Kier & Wright Land Surveyors
2. Boring groundwater depths may not represent stabilized levels and levels may fluctuate due to seasonal rainfall.
3. Depth of Existing Groundwater (bgs) in feet
4. bgs - Below Existing Ground Surface
5. msl - Mean Sea Level (NAVD88)
6. PPDT = Pore Pressure Dissipation Test.

Please note that seasonal groundwater studies were beyond the scope of this investigation. It shall be noted that groundwater level and elevation may fluctuate due to variations in rainfall, fluctuating groundwater elevation in the Calabasas Creek, geological changes, temperature, natural springs, pumping water from the wells and other factors that were not evident at the time of this investigation. For information pertaining to the depth of groundwater elevation, please refer to the boring logs enclosed in Appendix "B" of this report.

5.0 LABORATORY TESTING

Laboratory testing program performed on the soil samples collected from the site was directed towards a quantitative determination of the physical and engineering properties of the soils underlying the site. To evaluate the strength characteristics of the soil for the foundation engineering design, tests were performed on relatively undisturbed soil samples collected from various depths and from different boring locations at the site.

In addition to the above, Unit Weight, Moisture Content, Atterberg Limits Tests, to determine the Liquid Limit and Plasticity Index of the soil samples, Washed Particle Size Distribution, Consolidated Undrained Direct Shear and Consolidation Tests were performed on selected samples.

- **Moisture Content:** The natural water content was determined (ASTM D2216) on all samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.
- **Dry Densities:** In place dry density determinations (ASTM D2937) were performed on all samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the sample depths.
- **Washed Sieve Analyses:** The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on three samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.
- **Plasticity Index:** Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which these materials, exhibit plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential.
- **Consolidated Un-drained Direct Shear Strength:** The undrained shear strength was determined on two relatively undisturbed sample(s) by consolidated-undrained direct shear strength testing (ASTM D3080M).
- **Consolidation:** Two consolidation tests (ASTM D2435) were performed on selected relatively undisturbed samples of the subsurface clayey soils to assist in evaluating the compressibility property of this soil.
- **Corrosion:** Soil samples were tested for pH (ASTM G51), resistivity (ASTM G57), chloride (ASTM D4327), and sulfate (ASTM D4327).

Results of these tests are shown on the boring logs at the appropriate sample depths and are attached in the appendix. These laboratory analyses were conducted in accordance with the criteria and guidelines set-forth by CGS Special Publication 117A - Guidelines for Evaluating and Mitigating Seismic Hazards in California, SCEC (1999) Southern California Earthquake Center Report. The results of these laboratory testing are presented on Plates (34) thru (42) in Appendix "C" of this report.

5.1 PLASTICITY & EXPANSION POTENTIAL

Atterberg Limits test performed on the selected samples from the upper layers of the soil revealed moderate to high plasticity index (PI) and expansion potential, when subjected to fluctuations in the moisture content with plasticity indexes (PI) ranging between 21 to 25. For information, please refer to Plate No. (39) In Appendix "B" of this report.

5.2 CORROSIVITY ANALYSIS

Many factors can affect the corrosion potential of soil including soil moisture content, resistivity, permeability and pH, as well as chloride and sulfate concentration. In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor. Based on classification developed by William J. Ellis (1978), the approximate relationship between soil corrosiveness was developed as shown in Table below.

TABLE 4 – RESULTS OF CORROSIVITY ANALYSIS

Chemical Analysis (ASTM Method)	Sample Results (CPT No. 1)	Sample Results (CPT No. 4)	Corrosion Classification
Chlorides (ASTM D4327)	36 mg/kg	88 mg/kg	Non-corrosive
Sulfates (ASTM D4327)	51 mg/kg	100 mg/kg	Non-Corrosive
pH (ASTM G51)	8.4	8.5	Non-Corrosive
Saturated Resistivity (ASTM G57)	848 ohm-cm	1317 ohm-cm	Severely Corrosive
Sulfide (ASTM D1498)	---	---	PG&E
Redox (ASTM G200)	454	413	Non-Corrosive
Moisture (%)	22.7	18.5	Not-applicable

Notes:
1. With respect to bare steel or ductile iron
2. With respect to mortar coated steel

Chloride and sulfate ion concentrations and pH appear to play secondary roles in affecting corrosion potential. High chloride levels tend to reduce soil resistivity and break down otherwise protective surface deposits, which can result in corrosion of buried metallic improvements or/ reinforced concrete structures. Although the chloride iron concentration in the soil is negligible, the sulfate ion concentration is determined to be sufficient and potentially be detrimental to reinforced concrete structures and cement mortar-coated steel. Concrete in contact with the soil should use sulfate resistant cement such as Type II, with a maximum water-cement ratio of 0.55.

Underground Metallic Pipelines - The soils at the project site are generally considered to be “very severely corrosive” to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at this site depending upon the critical nature of the piping. Pressure piping systems such as domestic and fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

TABLE 5 - RELATIONSHIP BETWEEN SOIL RESISTIVITY & SOIL CORROSIVITY

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very Severely Corrosive
900 to 2,300	Severely Corrosive
2,300 to 5,000	Moderately Corrosive
5,000 to 10,000	Mildly Corrosive
10,000 to >100,000	Very Mildly Corrosive

Sulfate ions in the soil can lower the soil resistivity and can be highly aggressive to Portland cement concrete (PCC) by combining chemically with certain constituents of the concrete, principally tricalcium aluminate. This reaction is accompanied by expansion and eventual disruption of the concrete matrix. Soils containing high sulfate content could also cause corrosion of the reinforcing steel in concrete.

TABLE 6 - RELATIONSHIP BETWEEN SULFATE CONCENTRATION & EXPOSURE

Water-Soluble Sulfate (SO4) in soil (ppm)	Sulfate Exposure
0 to 1,000	Negligible
1,000 to 2,000	Moderate[1]
2,000 to 20,000	Severe
over 20,000	Very Severe

[1]= seawater

Reinforced Concrete Foundations - Due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The type of cement used should be in accordance with California Building Code (CBC) for soils which have less than 0.10 percent by weight of water-soluble sulfate (SO4) in soil and the minimum depth of cover for the reinforcing steel should be as specified in CBC as well.

Acidity is an important factor of soil corrosivity. The lower the pH (the more acidic the environment), the higher will the soil corrosivity be with respect to buried metallic structures. As soil pH increases above 7 (the neutral value), the soil is increasingly more alkaline and less corrosive to buried steel structures due to protective surface films which form on steel in high pH environments. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint.

Furthermore, we recommend retaining a corrosion consultant (if needed) to provide specific design recommendations for corrosion protection for buried metals and concrete elements. The design team should also consider City of Santa Clara’s specific requirements for underground improvements constructed in such environment.

5.3 ENVIRONMENTAL SERVICES FOR SOIL OFFHAUL

Environmental services were not part of this geotechnical investigation or/ part of this project scope. Therefore, AST has no environmental responsibility associated with this project. All existing and future environmental concerns associated with this site should be addressed by the project environmental consultant. In addition to the above, the environmental consultant should review and incorporate, any and all geotechnical recommendations provided in this report for compatibility, as part of the mitigation measures for any existing environmental concerns as they may exist at the present time due to its usage in the past.

6.0 GEOLOGIC SETTING

6.1 REGIONAL GEOLOGY

Geologically, the site under evaluation is located within the physiographic region known as the San Francisco Bay Area, which itself lies within the Coast Range geomorphic province of California, which consists of a series of northwest trending mountains and valleys along the western edge of the North American Continent. The San Francisco Bay Area itself lies within the Coast Range Geomorphic Province, a more or less discontinuous series of northwest trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting.

Geologic and Geomorphic structures within the San Francisco Bay Area are dominated by tectonic deformation and along the San Andreas Fault system. This right-lateral strike-slip fault extends on land from the Gulf of California in Mexico, to Cape Mendocino, on the Coast of Humboldt County in northern California. It forms a portion of the boundary between two independent tectonic plates on the surface of the earth. To the west of the San Andreas Fault, the Pacific plate moves north relative to the North American plate, located east of the fault.

In the San Francisco Bay Area, movement across this plate boundary is distributed across the San Andreas Fault and a number of other faults including the Hayward, Calaveras, and San Gregorio. Together, these faults are referred to as the San Andreas Fault system. The general trend of the faults within this system is responsible for the strong northwest-southeast structural grain of geologic and geomorphic features in the San Francisco Bay Area.

For most of the length of the San Andreas Fault, basement rock on the east generally consists of a chaotic mixture of highly deformed marine sedimentary, submarine volcanic and metamorphic rocks of the Franciscan Complex. The Franciscan rocks are generally considered Jurassic and Cretaceous age (about 65 to 205 million years old). Overlying the basement rocks are Cretaceous marine, as well as Tertiary (about 65 to 1.6 million years old) marine and non-marine sedimentary rocks with some continental volcanic rock. These Cretaceous and Tertiary rocks typically have been extensively folded and faulted largely as a result of movement along the San Andreas Fault System over the last 25 million years.

6.2 LOCAL GEOLOGY

In this Quadrangle there are 16 Quaternary units mapped by Knudsen and others (2000a). Coalescing late Pleistocene and Holocene alluvial fans form a northeastward sloping bajada that covers much of the western and southern parts of the quadrangle. From west to east, the creeks that supply sediment to the alluvial fans are Calabazas, Saratoga, Tomas

Aquinas, Los Gatos, and Ross. Near the heads of the fans, creeks have incised large, latest Pleistocene alluvial fan deposits (Qpf) consisting of coarse sand and gravel. Farther upstream, a few small upland valleys containing undifferentiated late Pleistocene to Holocene alluvium (Qa) are mapped in the foothills of the Santa Cruz Mountains.

The site is in the Santa Clara Valley Basin, which is bound on the north by San Francisco Bay to the west and south by the Santa Cruz Mountains, and to the east by the Mount Diablo Foothills. The valley basin is a down-thrown block structure filled in with thousands of feet of marine and continental sediments derived from the surrounding mountains. These sediments consist primarily of marine, alluvial fan, and stream deposits that extend to great depths. They typically consist of discontinuous, interbedded layers of silt, clay, sand, and gravel.

In the vicinity of the site, fine grained basin and marine deposits are predominant and consist of silt and clay. Thin interbedded, discontinuous and buried channel deposits of sand and gravel are within these fine-grained units. The silt and clay layers, interbedded with sand and gravel, extend to a depth greater than 100-feet beneath the ground surface.

Review of the above-mentioned report and Geologic Map and the quaternary units mapped by Witter and others in 2006 indicates that the surrounding areas in the Milpitas Quadrangle are underlain with Holocene alluvial fan deposits (Qhff/Qhfy), Holocene alluvial fan deposits, fine grained facies (Qhff/Qhfy), and Holocene alluvial fan Levee deposits. The soils layers encountered at the site in general exhibited moderate to high plasticity clays, sandy clays and lean clays with interbedded layers of clayey sands and silts with occasional gravel and varying degree of moisture content. These layers were very moist, medium stiff to very stiff and medium dense to dense. For additional information, please refer to Plate (4) in Appendix "A" of this report.

7.0 SEISMIC HAZARD ASSESSMENT

7.1 SEISMICITY/GROUND SHAKING

The San Francisco Bay Area is recognized by geologists and seismologists as one of the most seismically active regions in the United States. A broad system of inter-related northwest-southeast trending strike slip faults represents a segment boundary between the pacific and North American crustal plates. For 15 million years, the Pacific Plate has been slipping northwest ward with respect to the North American Plate (Atwater, 1970; Graham, 1978).

The majority of the movement has been along the San Andreas Fault System; however, there are other faults within this broad system that have experienced movement at one time or/ another. The faults of Santa Clara County are mostly characterized by both strike-slip and dip-slip components of displacement. There are three major fault systems that display large right-lateral offsets, the San Andreas, the Pilarcitos and the San Gregorio fault zones. These fault systems trend roughly N30W. Movement on the many splays of the San Andreas Fault system has produced the dominant northwest-oriented structural and topographic trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates: The North American Plate to the East and the Pacific Plate to the West. The San Andreas Fault system and its major branching faults is about 40 miles wide in the Bay area and extends from the San Gregorio

Fault near the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the Great Central Valley as shown on the historical Fault Map. The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Within the region, the San Andreas Fault system, which distributes shearing across a complex assemblage of primarily right lateral, strike-slip, parallel and sub-parallel faults that includes the Hayward, San Andreas, Calaveras and Monte-Vista Shannon Faults.

In general, the site will experience strong to severe seismic shaking during the lifetime of the proposed structure(s) from the above-mentioned and other faults reflected in the following sections of this report. These faults that are capable of generating significant earthquakes are generally associated with well-defined areas of crustal movement, which generally trend in the northwesterly direction. Table 6 reflected below presents the considered active faults within a 100 km (62 mile) of the site of proposed development and they are as follows:

TABLE 7 - NEAREST SEISMIC SOURCE WITHIN 100 KILOMETERS

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude	Mean Slip Rate (mm/yr)	Approx. Fault Length (km)
Hayward-Rodgers Creek; HN+HS	10.7	North	7.00	9	87
Hayward-Rodgers Creek; HS	10.7	North	6.78	9	52
Hayward-Rodgers Creek; RC+HN+HS	10.7	North	7.33	9	150
Monte Vista-Shannon	13.1	Southwest	6.50	0.4	45
Calaveras; CC	16.2	Northeast	6.39	15	59
Calaveras; CC+CS	16.2	Northeast	6.50	15	78
Calaveras; CN+CC	16.2	Northeast	7.00	11	104
Calaveras; CN+CC+CS	16.2	Northeast	7.03	12	123
Calaveras; CN	16.2	Northeast	6.87	6	45
N. San Andreas; SAN+SAP	18.2	Southwest	7.73	22	274
N. San Andreas; SAN+SAP+SAS	18.2	Southwest	7.87	21	336
N. San Andreas; SAO+SAN+SAP	18.2	Southwest	7.95	22	410
N. San Andreas; SAO+SAN+SAP+SAS	18.2	Southwest	8.05	22	472
N. San Andreas; SAP	18.2	Southwest	7.23	17	85
N. San Andreas; SAP+SAS	18.2	Southwest	7.48	17	147
N. San Andreas; SAS	25.4	Southwest	7.12	17	62
Zayante-Vergeles	35.1	Southwest	7.00	0.1	58
San Gregorio Connected	38.2	West	7.50	5.5	176

Fault Segment	Approx. Distance from fault (km)	Direction from Site	Mean Characteristic Moment Magnitude	Mean Slip Rate (mm/yr)	Approx. Fault Length (km)
Mount Diablo Thrust	38.4	North	6.70	2	25
Greenville Connected	39.7	East	7.00	2	50
Hayward-Rodgers Creek; HN	50.6	Northwest	6.60	9	35
Hayward-Rodgers Creek; RC+HN	50.6	Northwest	7.19	9	97
Great Valley 7	54.2	Northeast	6.90	1.5	45
Green Valley Connected	54.5	North	6.80	4.7	56
Monterey Bay-Tularcitos	54.9	Southwest	7.30	0.5	83
Ortigalita	63.8	East	7.10	1	70
Calaveras; CS	64.8	Southeast	5.83	15	19
N. San Andreas; SAO+SAN	66.6	Northwest	8.00	24	326
N. San Andreas; SAN	66.6	Northwest	7.51	24	189
Great Valley 5, Pittsburg Kirby Hills	68.3	North	6.70	1	32
Great Valley 8	72.8	East	6.80	1.5	41
Quien Sabe	76.2	Southeast	6.60	1	23
SAF - creeping segment	77.0	Southeast	6.70	34	125
Rinconada	83.2	South	7.50	1	191
Hayward-Rodgers Creek; RC	84.8	Northwest	7.07	9	62
West Napa	86.7	North	6.70	1	30
Great Valley 9	94.1	East	6.80	1.5	39

7.2 HISTORICAL EARTHQUAKES

The earthquake epicenters for events with magnitude greater than 5.0 from January 1800 through August 2015 are enclosed in Appendix "C" of this report. Since 1800, four major earthquakes have been recorded on the San Andreas Fault. In 1836 an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, Mw, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an Mw of about 7.5.

The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an Mw of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma

Prieta Earthquake occurred on 17 October 1989, in the Santa Cruz Mountains with a magnitude of Mw of 6.9. In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated Mw for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably Mw of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (Mw= 6.2). The most recent earthquake to affect the Bay Area occurred on 24 August 2014 and was located on the West Napa fault, with an approximate magnitude MW of 6.0.

7.3 FUTURE EARTHQUAKE PROBABILITY

The presence of faults in the San Francisco Bay Area Region and the seismic activity in the recent past has led USGS to constantly upgrade the predictions for the possibility of the next major earthquake in the Bay Area. Per the information received from the review of the documents, it is our understanding that The Working Group on California Earthquake Probabilities (2008) has concluded that the probability of a magnitude 7.0+ earthquake in the San Francisco Bay Area over the next 30 years is 63 percent. This probability is a low estimate since only three active faults in the area; the Hayward Fault, San Andreas Fault and Rodgers Creek Fault were included in the study.

Schwartz (1994) concludes that the probability of occurrence of one or more magnitude 6.7+ earthquakes in the Bay Area is substantially higher than 63 percent, possibly as high as 99.7 percent. It shall be noted that significant earthquakes could occur on an active fault or a potentially active fault for which probabilities might not have been estimated.

Even though research on earthquake predictions has increased tremendously in recent years, seismologists still cannot predict when or where an earthquake of that magnitude will occur. Based on the information that is available, it is our understanding that the site will likely be subjected to at least one moderate to severe earthquake within or/ less than 50 years of the proposed construction. During such an earthquake, the possibility of fault offset could be perceived to be low. However, strong to severe ground shaking will be experienced at the site.

The probabilities maximum moment magnitude earthquake occurring during the next (30)-year period are presented in the table below and they are as follows:

TABLE 8 – EARTHQUAKE PROBABILITIES

Fault Segment	(30)–Year Probability (%) Earthquake Magnitude M = 6.7
Hayward (North & South)	32
San Andreas (Peninsula Segment)	33
Calaveras	25
San Gregorio	6
Concord-Green Valley	3
Greenville	3
Mount Diablo Thrust	1

8.0 GEOLOGICAL HAZARDS

This section presents our geologic hazards review per the requirements of the California Geological Survey (CGS) for the proposed development located to the west of Patrick Henry Drive in Santa Clara, California. The site is located approximately at Latitude 37.4063986º North and -121.9853448º West Longitude respectively. Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, liquefaction, and ground lurching. In the proceeding sections, potential geologic hazards related to the site are addressed as below:

8.1 SURFACE FAULT RUPTURE

Earthquakes generally are caused by a shift or/ displacement along a discrete zone of weakness, termed as fault in Earth's crust. Surface fault rupture, which is a manifestation of the fault displacement at the ground surface, usually is associated with moderate- to large-magnitude earthquakes (magnitudes of about 6 or larger). Generally, primary surface fault rupture occurs on active faults having mapped traces or/ zones at the ground surface. In other words, major faults tend to rupture on pre-existing planes of weakness. The amount of surface fault displacement can be as much as (20)-feet, depending on the earthquake magnitude and other factors. Large earthquakes also can "trigger" slip on adjacent faults, which may or may not be mapped at the surface, causing co-seismic ground deformation (*e.g.,* Lawson 1908; Schmidt and Others, 1995). Potential surface fault rupture hazards exist along the known active faults in the greater San Francisco Bay Region.

The faults which have been identified as potential surface rupture hazards by the California Geologic Survey in close proximity to the site include the Hayward, Calaveras, San Andreas, San Gregorio and Monte Vista-Shannon Faults, which are located in close proximity to the site. These faults show historic (last 200 years) displacement associated with mapped surface rupture or/ surface creep. The site is not located within a currently designated California Alquist-Priolo Earthquake Fault Zone (CDMG, 1982), formerly known as a Special Studies Zone, City of San Jose Fault Hazard Zone (CSJ, 1983), or/ a Santa Clara County Fault Rupture Hazard Zone (SCC, 2002). No indications of active faulting were observed in aerial photographs or/ in the field, nor have any surface fault expressions been mapped across the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

8.2 HISTORIC GROUND FAILURES

Many historical earthquakes have occurred on active faults and fault branches throughout coastal California. Hayward and San Andreas Fault are considered to be some of the major active faults of the region generating damaging earthquakes in 1836 and 1868, as well as the great San Francisco Earthquake of 1906, which had an approximate Richter Magnitude of 8.3, and the Loma Prieta Earthquake of 1989.

Lawson (1908) reported considerable damage from the 1906 earthquake in the Bay Area. Very few observations of the 1868 Hayward earthquake record specific evidence for liquefaction in the region. However, Lawson (1908) reports that water spurted up in the streets of San Jose, and out in the road between Milpitas and San Jose, to the height of several feet. Lawson (1908) described damage resulting from the 1906 earthquake in the downtown area of San Jose as; severe structural damage too many brick and mortar buildings and many chimneys were toppled. The main part of San Jose and surrounding areas shook at Rossi-Forel intensity VIII (many chimneys fall) to IX (partial or complete collapse of some buildings). Witter et. al., (2006) indicate four historical occurrences of ground deformation within the "Qhf" mapping unit within the region. These ground failures were concentrated close to the bay several miles north of the site.

Youd and Hoose (1978) report that following the 1906 earthquake numerous cracks were observed on both sides of Coyote Creek from Milpitas "all the way to San Jose." They also report that within the valley no cracking was observed along Alum Rock Road east of Coyote Creek. No observations of liquefaction were recorded in the San Jose West Quadrangle following the 1989 Loma Prieta earthquake (Tinsley and others, 1998).

Co-seismic contractional deformation occurred within the south bay area during the 1989 Loma Prieta earthquake in an area traversed by several northwest-trending reverse faults, including the Monte Vista, Shannon, and Berrocal faults (Haugerud and Ellen, 1990). These breaks coincide with the projections of vegetation lineaments and linear depressions on both the northwestern and southeastern sides of Los Gatos Creek. Schmidt et al., (1995) documented cases of co-seismic damage in the region however they did not document any evidence of such damage at or/ near the site of proposed development.

8.3 LIQUEFACTION POTENTIAL

Liquefaction is a soil behavior phenomenon in which a soil loses a substantial amount of strength due to high excess pore-water pressure generated by strong earthquake ground shaking. Recently-deposited (i.e., within about the past 11,000 years) and relatively unconsolidated soils and artificial fills located below the ground water surface are considered susceptible to liquefaction (Youd and Perkins, 1978).

Typically, the soils that are most susceptible to liquefaction include relatively clean, loose, uniformly graded sand, silty sand, and non-plastic silt deposits (e.g., National Research Council, 1985). Liquefaction is the transformation of clean, loose saturated sand and silt (cohesionless soil) and some sensitive clayey silt soil from a solid state to a semi-liquid state. This transformation occurs during a seismic event/ground shaking, when soil is subjected to cyclic stresses that cause increase in hydrostatic pressure that induces liquefaction. The resulting upward flow of water will often turn cohesionless soil into a liquefied condition (loss of density).

At the ground surface, liquefaction is manifested by the formation of sand boils, ground cracking, and lateral spreading and in some cases development of quicksand like conditions, which results in the settlement or movement of the structures. Cohesive soils are generally not considered susceptible to soil liquefaction. However, the soils that are most susceptible to liquefaction are loose to moderately dense, saturated granular soils. Sensitive clays are vulnerable to significant strength loss under relatively minor strains during seismic event.

The site of proposed development is located within the liquefaction zone as outlined by CGS Seismic Hazard Zone Report (051) for the Milpitas Quadrangle and could experience vertical settlement during a seismic event.

8.3.1 LIQUEFACTION EVALUATION AND ANALYSIS CRITERIA

To evaluate liquefaction potential at this site, we performed our liquefaction analysis in accordance with the State of California Special Publication 117A, Guidelines for Evaluation and Mitigation of Seismic Hazards in California and following the procedures presented in the 1996 NCEER and the 1998 NCEER/NSF workshops on the Evaluation of Liquefaction Resistance of Soils (Youd and Idriss 2001). The NCEER methods are updates of the simplified procedures developed by Seed et al. (1971).

To estimate volumetric strain and associated liquefaction-induced settlement, we used the procedure developed by Tokimatsu and Seed (1987). These methods are used to estimate a factor of safety against liquefaction triggering by taking the ratio of soil strength (resistance of the soil to cyclic shaking) to the seismic demand that can be expected from a design level seismic event. Specifically, two distinct terms are used in the liquefaction triggering analyses.

- Cyclic Resistance Ratio (CRR), which quantifies the soil's resistance to cyclic shaking; a function of soil depth, density, depth of groundwater, earthquake magnitude, and overall soil behavior

- Cyclic Stress Ratio (CSR), which quantifies the stresses that are anticipated to develop during cyclic shaking and is dependent on the duration, magnitude, and peak ground acceleration associated with a design seismic event

The factor of safety (FS) against liquefaction triggering can be expressed as the ratio of CRR over CSR. For our analyses, if the FS for a soil layer is less than 1.3, it is considered possible that the soil layer may liquefy during a large seismic event. For our calculations of estimated liquefaction-induced settlement, we assumed layers with a FS equal to or greater than 1.3 will not experience liquefaction-induced settlement.

Our CRR calculations are based on the CPT tip resistance. The CPT tip pressures were normalized and corrected for overburden pressure; fines content, CPTs tip resistance was also modified to account for thin layers, where appropriate. The CPT method also utilizes the soil behavior type index (IC) and the exponential factor "n" applied to the Normalized Cone Resistance "q" to evaluate the cohesive nature of the soil. According to Bray and Sancio (2006) and Seed et al. (1982), soil with these properties is not susceptible to liquefaction. The lower gravelly deposits have less than 5 percent fines and low SPT blow counts (N). CRR calculations are based on Standard Penetration Test (SPT) blow counts and CPT tip resistance. The SPT blow counts and CPT tip pressures were normalized and corrected for overburden pressure, fines content, CPTs tip resistance was also modified to account for thin layers, where appropriate. The CPT method also utilizes the soil behavior type index (IC) and the exponential factor "n" applied to the Normalized Cone Resistance "q" to evaluate the cohesive nature of the soil.

All of the above-mentioned calculations are included in these analyses. The CSR is obtained using the equations presented in NCEER paper and is based on the density of the soil, the depth to design groundwater, the estimated peak horizontal acceleration at the ground surface (PGA_M), and a stress reduction coefficient (rd). However, for this liquefaction study, the data collected from CPTs (CPT-01 thru CPT-05) were utilized for the liquefaction settlement and lateral spreading analyses. We have used the computer software program CLiq (Version 2.2.1.11, Geologismiki) and the in-situ soil parameters measured in the CPT soundings. The software utilized the 1998 National Center for Earthquake Engineering Research (NCEER) method of analysis which was developed with the broad consensus of national geotechnical earthquake engineering experts. We have evaluated the potential for liquefaction and resultant settlements at the site using the CPT data and the methodology of Youd et. al. (2001).

In this liquefaction analysis, we have incorporated an earthquake moment magnitude (Mw) of 7.9 to 8.05 and a historic groundwater depth of (±3.5) to (±4.0)-feet (with in-situ groundwater elevation varying from (±4.2) to (±9.4)-feet (bgs) below the existing ground surface). The groundwater elevation used in our analysis was obtained from an historic high groundwater exhibit presented within the CGS Seismic Hazard Report (051) for the Milpitas Quadrangle. Our evaluation was performed in accordance with the methodology of Idriss and Boulanger (2008) and considered cyclic softening in clayey soils. The 2019 CBC indicates a ground motion/Peak Ground Acceleration (PGA_M) of 0.61g is acceptable for design, which was obtained from the USGS Design Maps Summary Report.

In estimating post-liquefaction settlement at the site using CLIQ Software Program, we have implemented both with and without auto transition layer detection and depth *weighting factor proposed by Cetin (2009)*. Following evaluation of 49-high quality cyclically induced ground settlement case histories from seven different earthquakes, Cetin proposed the use of a weighting factor based on the depth of layers. The weighting procedure was used to tune the surface observations at liquefaction sites to produce a better model fit with measured data.

Aside from the better model fit it produced, the rationale behind the use of a depth weighting factor is based on the following: 1) upward seepage, triggering void ratio redistribution, and resulting in unfavorably higher void ratios for the shallower sub-layers of soil layers; 2) reduced induced shear stresses and number of shear stress cycles transmitted to deeper soil layers due to initial liquefaction of surficial layers; and 3) possible arching effects due to non-liquefied soil layers. All these may significantly reduce the contribution of volumetric settlement of deeper soil layers to the overall ground surface settlement (Cetin, 2009).

Based on the results of our analyses, we conclude several of these sandy soil layers could potentially liquefy during a major seismic event/earthquake and may experience liquefaction induced settlement. The primary design parameters used in our liquefaction triggering calculations are summarized in Appendix "E' of this report. In our analyses, soil that has significant amount of plastic fines, Ic greater than 2.6, was considered too cohesive to liquefy; a corrected cone tip resistance (qc1N) greater than 160 tons per square foot (tsf), was considered too dense to liquefy. In addition, a corrected shear wave velocity (Vs1) greater than 200 meters per second (m/s), was considered too dense to liquefy. Because the predominant earthquake is a moment magnitude 7.3, the cyclic resistance ratio (CRR) has been scaled to a moment magnitude of 7.5/8.05 using magnitude scaling factors developed by Idriss (Youd and Idriss, 2001).

As discussed in sections above, the proposed development will have a one-level basement. Assuming the basement floor slab is (±2.0)-thick, the excavation for the basement is estimated to extend to a depth of approximately (14)-feet below the ground surface (bgs), corresponding to approximate Elevation (-2.0) to (-4.0)-feet. State guidelines (SCEC, 1999) recommend a minimum depth of 50 feet below lowest proposed bottom of excavation grade for evaluation of liquefaction potential. During our investigation, exploratory borings EB-1 and EB-2 were drilled to a depth of about (±75)-feet below the existing ground surface (bgs) and CPT-1 through CPT-5 were advanced to depths of about (±80)-feet below the existing ground surface (bgs), which satisfies the guidelines.

Layers of medium dense sands with varying amounts of clay and silt, varying in thickness were encountered below the groundwater level in the borings and at the CPT locations. On the basis of the results of our analyses, we conclude some of these layers could potentially liquefy during a major earthquake and experience liquefaction-induced settlement. A summary of the data regarding liquefaction triggering and associated settlement from existing ground surface are presented in table below. The potential for sand boils and lateral spreading are also discussed in the following sections.

A summary of our CPT subsurface data for the exploration points, as well as other pertinent parameters regarding liquefaction triggering and associated settlement are presented in our liquefaction analysis enclosed in Appendix E of this report.

TABLE 9 – SETTLEMENT DUE TO LIQUEFACTION

Location	Total Settlement with Layer Transition & Weighted Average (±inches)	Differential Settlement (±inches)
CPT-01	0.320	0.213
CPT-02	0.048	0.032
CPT-03	0.967	0.645
CPT-04	0.303	0.202
CPT-05	0.173	0.115

Therefore, we conclude several layers are potentially liquefiable during a major earthquake. The excavation for the basement of the proposed development will not remove these layers. We conclude up to (±1¼)-inches of total seismically induced-settlement could occur with up to one-inch of differential settlement over a horizontal distance of (30)-feet could occur within the basement footprint.

8.3.2 LATERAL SPREADING

Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported down slope or in the direction of a free face by earthquake and gravitational forces. As the failure tends to propagate as block failures, it is difficult to analyze and estimate, where the first tension crack will form. The nearest open face channel (Calabazas Creek) is located approximately more than (±≥175-feet) to the west of the site of proposed development with upslope towards the creek bank. Since there are no open faces along the western boundary of the site and the nature of the potentially liquefiable layers are discontinuous with no historical evidence of lateral spreading that has been recorded in the vicinity of the site. Considering these conditions, we judge the potential for lateral spreading is low.

8.4 SAND BOILS

Ishihara (1985) has shown that the presence of a sufficient thickness of a non-liquefiable surface layer may prevent the observable effect of at-depth liquefaction from reaching the surface. A more recent study by Youd and Garris (1995) expanded on the work of Ishihara to include data from over 300 exploratory borings, 15 different earthquakes, and several ranges of recorded peak ground acceleration. Considering the capping effects from overlying non-liquefiable layers and additional engineered fill to be placed to raise site grades, the soils above the potentially liquefiable soils are thick enough to resist upward pressure and the liquefiable lenses are thin enough to provide only a limited reservoir of water. Therefore, we do not anticipate the occurrence of sand boils at the site should liquefaction occur.

8.5 SEISMIC SETTLEMENT/CYCLIC DENSIFICATION

Seismic densification of non-saturated, cohesionless soil following a major earthquake was analyzed using the procedure outlined by Tokimatsu and Seed (1987) and the Pradel (1998) method. The CPTs and boring indicate the soil above and below the groundwater is generally clayey alluvial soils, however there are several interbedded layers of medium dense sand with varying amounts of clay and silt encountered above the groundwater level. Using the Pradel (1998) method for evaluating seismically-induced settlement in dry sand, we estimate seismic densification settlements in these layers of up to approximately ¼-inch.

8.6 DIFFERENTIAL COMPACTION

If near surface soil varies in composition both vertically and laterally, strong earthquake shaking can cause non-uniform compaction of soil strata, resulting in movement of near surface soils. Based on the nature of the soils encountered at the site, we judge the probability of differential compaction at the site to be low.

8.7 GROUND LURCHING & LANDSLIDING

Seismically induced ground lurching and landsliding is a lateral movement of portions of the ground normally accompanied by fissuring perpendicular to the direction of lurching. It usually occurs along steep slopes and unconsolidated and unsupported stream banks. However, there are no open channels or/ banks located in immediate vicinity of the site. Our interpretation of ground lurching and landslide potential is based on interpretation of aerial photographs, geologic maps, site reconnaissance and review of the previous geotechnical investigation, research of published maps and reports and our subsurface exploration. Since the site is located in a flat area, we judge the possibility of Ground Lurching/Landsliding to be low.

8.8 FLOODING AND RESERVOIR INUNDATION

Based on our internet search of the Federal Emergency Management Agency (FEMA) and the review of the available FEMA Flood Zone Maps, it is our understanding that the site is not located in a flood zone. However, we recommend that the Project Civil Engineer be retained to confirm this information and verify the base flood elevation (if appropriate).

8.9 SEISMICALLY INDUCED WAVES – TSUNAMIS & SEICHES

Tsunamis are seismic sea waves that are typically, an open ocean phenomenon caused by underwater landslides, volcanic eruptions and seismic events, which primarily impact low-lying coastal areas. Seiches are earthquake-generated waves or oscillations (sloshing) of the water surface in restricted bodies of water, such as the San Francisco Bay. The 1868 earthquake on the Hayward Fault is reported to have generated Seiches activity in the bay. Seiches are extremely rare in the Bay, which generally attenuates such activity due to its irregular shape and shallow shoreline.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 4 miles inland from the San Francisco Bay shoreline, and is approximately 10-feet above mean sea level. Therefore, the potential for inundation due to tsunami or seiche is considered low.

Recent published maps (California Emergency Management Agency, 2009) indicate the site is not within the tsunami inundation zone; therefore, we conclude the potential risk by inundation from tsunami to be low for the site. However, the Project Civil Engineer should evaluate the impact of sea level rise on the potential risk of inundation from a tsunami over the life of the proposed new structure/building.

9.0 CBC - CODE BASED SEISMIC DESIGN CRITERIA -

9.1 SITE LOCATION AND PROVIDED DATA FOR 2019 CBC SEISMIC DESIGN

The project is located at longitude 37.4063986° and latitude -121.9853448°, which is based on Google Earth (WGS84) coordinates of the site located at 5200 Patrick Henry Drive in Santa Clara, California. We have assumed that a Seismic Importance Factor (I_e) of 1.0 (needs to be confirmed by the structural Engineer) has been assigned to the structure(s) in accordance with Table 1.5-2 of ASCE 7-16 for structures classified as Risk Category II.

9.2 SITE CLASSIFICATION – CHAPTER 20 OF ASCE 7-16

Code-based site classification and ground motion attenuation relationships are based on the time-weighted average shear wave velocity of the top approximately 100 feet (30 meters) of the soil profile (V_{S30}). Shear wave velocity (V_S) measurements were performed while advancing CPTs, resulting in a time-averaged shear wave velocity taken at four different locations for the top 30 meters (V_{S30}) ranging approximately 247-meters per second to 250-meters per second respectively. Our site-specific ground motion hazard analysis considered an average shear velocity V_{S30} of 248 m/s for evaluating the seismic design parameters.

9.3 CODE-BASED SEISMIC DESIGN PARAMETERS

Code-based spectral acceleration parameters were determined based on mapped acceleration response parameters adjusted for the specific site conditions. Mapped Risk-Adjusted Maximum Considered Earthquake (MCE_R) spectral acceleration parameters (S_S and S_1) were determined using the ATC Hazards by Location (Latitude & Longitude) from website (https://hazards.atcouncil.org). The mapped acceleration parameters were adjusted for local site conditions based on the average soil conditions for the upper 100 feet (30 meters) of the soil profile. Code-based MCE_R spectral response acceleration parameters adjusted for site effects (S_{MS} and S_{M1}) and design spectral response acceleration parameters (S_{DS} and S_{D1}) are presented in Table 10 below.

In accordance with Section 11.4.8 of ASCE 7-16, structures on Site Class D sites with mapped 1-second period spectral acceleration (S_1) values greater than or equal to 0.2 require a site-specific ground motion hazard analysis be performed in accordance with Section 21.2 of ASCE 7-16. Design site-specific seismic parameters are presented in Appendix D of this report. Code values presented in table below are for reference only and should not be used for design. Values summarized in Table 10 are only used to determine Seismic Design Category and comparison with minimum code requirements in our site-specific ground motion hazard analysis.

TABLE 10 - CBC 2019 CODE SEISMIC DESIGN PARAMETERS (Reference Only)

Parameters	Values
Site Class	D
Site Latitude	37.406399° N
Site Longitude	-121.9853° W
Risk Category	II
0.2-Second Period Mapped Spectral Acceleration, Class B (short), Ss	1.500g
1-Second Period Mapped Spectral Acceleration, Class B, S1	0.600g
Short Period Site Coefficient - Fa	1.0
Long Period Site Coefficient - Fv	2.5[1,2]
0.2-second Period Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects (short), SMS	1.500g
1.0-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for the Site Effects, SM1	1.500g[1]
0.2-second Period, Design Earthquake Spectral Response Acceleration, SDS	1.000g
1.0-second Period, Design Earthquake Spectral Response Acceleration, SD1	1.000g[1]
Long Period Transition - TL	12-secs
Peak Ground Acceleration MCEG (PGA)	0.553g
Site Coefficient FPGA	1.1
Site Modified Peak Ground Acceleration (PGA_M)	0.608g
Mapped Value of the Risk Coefficient at 0.2-sec Period (CRS)	0.950
Mapped Value of the Risk Coefficient at 1.0-sec Period (CR1)	0.929

Notes:
1. The 2019 parameters are based on the assumption that the buildings will conform to ASCE 7-16 11.4.8 - Exception No. 2
2. The 2019 CBC Fv parameter shall only be used for calculation of Ts. (ASCE Table 11.4-2, Supplement 1, Note (a)

9.4 SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS

This section presents the details of our estimation of the level of ground shaking at the site during future earthquakes. To develop site-specific response spectra in accordance with 2019 California Building Code (CBC) criteria, and by reference ASCE 7-16, we performed probabilistic seismic hazard analysis (PSHA) and deterministic analysis to develop smooth, site-specific horizontal spectra for two levels of shaking, namely:

- Risk-Targeted Maximum Considered Earthquake (MCE_R), which corresponds to the lesser of two percent probability of exceedance in 50 years (2,475-year return period) or/ 84th percentile of the controlling deterministic event both considering the maximum direction as described in ASCE 7-16.

- Design Earthquake (DE) which corresponds to 2/3 of the MCE_R.

In accordance with Section 11.4.8 of ASCE 7-16, we performed a ground motion hazards analysis (GMHA) in accordance with Chapter 21, Section 21.2 of ASCE 7. Following the methodology outlined in Section 21.2, we evaluated both Probabilistic MCE_R Ground Motions in accordance with Method 1 and Deterministic MCE_R Ground Motions to generate our recommended design response spectrum for the project. We performed a site-specific GMHA in accordance with ASCE 7-16 Chapter 21.2 and 2019 CBC Section 1803.6. Our analyses were performed using the USGS interface Unified Hazard Tool (UHT) based on the UCERF 3 Data Set, Business Seismic Safety Council (BSSC) Scenario Catalog 2014 event set (BSSC 2014), and the 2014 National Seismic Hazard Maps – Source Parameters (NSHMP deterministic event set).

Additionally, we utilized the computer program EZ Frisk, version 8.07 (Risk Engineering) and USGS program Response Spectra Plotter with combined models (Combined: WUS 2014 (4.1)). Our analysis utilized the mean ground motions predicted by four of the Next Generation Attenuation West 2 (NGA-West 2) relationships: Boore-Atkinson (2014), Campbell-Bozorgnia (2014), Chiou-Youngs (2014), and Abrahamson-Silva (2014). Rotation factors (scale factors) were determined as specified in ASCE 7-16 Chapter 21, Section 21.2, to calculate the maximum rotated component of ground motions (ASCE, 2016).

9.4.1 PROBABILISTIC HAZARD ANALYSIS

Because of the site location, recurrence interval, and magnitude of future earthquakes are uncertain, we performed a PSHA, which systematically accounts for these uncertainties. The results of a PSHA define a uniform hazard for a site in terms of a probability that a particular level of shaking will be exceeded during the given life of the structure. To perform a PSHA, information regarding the seismicity, location, geometry of each source, along with empirical relationships that describe the rate of attenuation of strong ground motion with increasing distance from the source is needed. The assumptions that are necessary to perform the PSHA are as follows:

- the geology and seismic tectonic history of the region are sufficiently known, such that the rate of occurrence of earthquakes can be modeled by historic or geologic data

- the level of ground motion at a particular site can be expressed by an attenuation relationship that is primarily dependent upon earthquake magnitude and distance from the source of the earthquake

- the earthquake occurrence can be modeled as a Poisson process with a constant mean occurrence rate

As part of the development of the site-specific spectra, we performed a PSHA to develop a site-specific response spectrum for 2 percent probability of exceedance in 50 years. The ground surface spectrum was developed using the computer code EZFRISK 8.07 (Fugro Consultants, Inc. 2021). The approach used in EZFRISK is based on the probabilistic seismic hazard model developed by Cornell (1968) and McGuire (1976). Our analysis modeled the faults in the Bay Area as linear sources, and earthquake activities were assigned to the faults based on historical and geologic data. The levels of shaking were estimated using Next Generation Attenuation (NGA) relationships that are primarily dependent upon the magnitude of the earthquake, the distance from the site to the fault and the shear wave velocity of the upper 30 meters, V_{S30}.

9.4.1.1 PROBABILISTIC MODEL

In probabilistic models, the occurrence of earthquake epicenters on a given fault is assumed to be uniformly distributed along the fault. This model considers ground motions arising from the portion of the fault rupture closest to the site rather than from the epicenter. Fault rupture lengths were modeled using fault rupture length-magnitude relationships given by Wells and Coppersmith (1994).

9.4.1.2 SOURCE MODELING AND CHARACTERIZATION

The segmentation of faults, mean characteristic magnitudes, and recurrence rates were modeled using the data presented in the WGCEP (2008) and Cao et al. (2003) reports. We also included the combination of fault segments and their associated magnitudes and recurrence rates as described in the WGCEP (2008) in our seismic hazard model. Table 7 in the sections above, presents the distance and direction from the site to the fault, mean characteristic magnitude, mean slip rate, and fault length for individual fault segments. We used the California fault database identified as "USGS08 California - 2014 Rates Excluded" in EZFRISK 8.07. We understand EZFRISK obtained this database directly from USGS and models the faults with multiple segments. Each segment is characterized with multiple magnitudes, occurrence or slip rates and weights. This approach takes into account the epistemic uncertainty associated with the various seismic sources in our model.

9.4.2 ATTENUATION RELATIONSHIPS

Based on the subsurface conditions, the site is classified as a stiff soil profile, site class D. Using the subsurface information including shear wave velocity measurements, we estimate the shear wave velocity of the upper 100 feet (30 meters), V_{S30}, is approximately 248-meters per second (813-feet per second). The NGA-2 database indicates that Z_1 and $Z_{2.5}$ are about 570 meters and 0.87 kilometer, respectively. These values were used in the development of site-specific spectra.

The Pacific Earthquake Engineering Research Center (PEER) embarked on the NGA-West 2 project to update the previously developed ground motion prediction equations (attenuation relationships), which were mostly published in 2014. We used the relationships by Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014) and Chiou and Youngs (2014). These attenuation relationships include the average shear wave velocity in the upper (100)-feet. Furthermore, these relationships were developed the same earthquake database, therefore, the average of the relationships (using equal weights for each attenuation relationship) is appropriate and was used to develop the recommended spectra. The NGA relationships were developed for the orientation-independent geometric mean of the data. Geometric mean is defined as the square root of the product of the two recorded components.

9.4.3 PSHA RESULTS

Plate No. 43 in Appendix "D" presents the geometric mean results of the PSHA for soil for the 2 percent probability of exceedance in 50 years hazard level (2,475-year return period) using the four relationships discussed above as well as the average of these relationships. For comparison we also included on the figure, the geomean for 2 percent probability of exceedance in 50 years hazard level PSHA results from OpenSHA using the same site parameters and average of four attenuations relationships. As can been seen, the results based on EZFRISK (UCERF2 model) and OPENSHA Hazard Spectrum Application 1.5.0 (UCERF3 model) are similar. The OpenSHA results are about 4 to 16 percent higher depending on the period; however, as discussed in section below, the deterministic results are lower than the PSHA results and thus will govern.

ASCE 7-16 specifies the development of MCE_R site-specific response spectra in the maximum direction. Shahi and Baker (2014) provide scaling factors that modify the geometric mean spectra to provide spectral values for the maximum response (maximum direction). We used the scaling factors presented in Table 1 of Shahi and Baker (2014) for ratios of $Sa_{RotD100}$/ $Sa_{GMRotI50}$ to modify the average of the PSHA results for two percent probability of exceedance in 50 years. The maximum direction spectrum is also shown on Plate No. 43 in Appendix D of this report. Plate No. 44 presents the deaggregation plots of the PSHA results for the 2 percent probability of exceedance in 50 years hazard level. From the examination of these results, it can be seen that the Hayward and San Andreas faults dominate the hazard at the project site at different periods of interest.

9.4.4 DETERMINISTIC HAZARD ANALYSIS

We performed a deterministic analysis to develop the MCE_R spectrum at the site. In a deterministic analysis, a given magnitude earthquake occurring at a certain distance from the source is considered as input into an appropriate ground motion attenuation relationship. On the basis of the deaggregation results we developed deterministic spectra for both scenario earthquakes:

- a Moment Magnitude of 7.33 on the Hayward fault at a distance of 12.5 kilometers from the site, and;
- a Moment Magnitude of 8.05 on the San Andreas fault at a distance of 18.3 kilometers from the site

The deterministic MCE_R spectrum was defined as an envelop for the both scenario earthquakes. This appears to be consistent with the deaggregation results discussed in section above. The same attenuation relationships and weighting factors as discussed in section above were used in our deterministic analysis. Plate No. 44, 45 and 46 in Appendix D present the results of 84th percentile and deterministic results for both the Hayward and San Andreas scenarios, respectively. The average of the four attenuation relationships were used for the geometric mean are also presented on those plates. Similarly, to the PSHA results, we developed the 84th percentile deterministic spectrum in the maximum direction using the Shahi and Baker (2014) ratios. Plate No. 45 presents the average of the 84th percentile deterministic results in the maximum direction for both scenarios as well as the recommended envelop of both scenarios.

9.4.5 RECOMMENDED SPECTRA

The MCE_R as defined in ASCE 7-16 is the lesser of the maximum direction PSHA spectrum having a two percent probability of exceedance in 50 years (2,475-year return period) or/ the maximum direction 84th percentile deterministic spectrum of the governing earthquake scenario and the DE spectrum is defined as 2/3 times the MCE_R spectrum. Furthermore, the MCE_R spectrum is defined as a risk targeted response spectrum, which corresponds to a targeted collapse probability of one percent in 50 years. The USGS Risk-Targeted Ground Motion calculator was used to determine the risk coefficients for each period of interest for the probabilistic spectrum. We used these risk coefficients to develop the risk-targeted PSHA spectrum.

Furthermore, we followed the procedures outlined in Chapter 21 of ASCE 7-16 and Supplement No. 1 to develop the site-specific spectra for MCE_R and DE. Chapter 21 of ASCE 7-16 requires the following checks:

- the largest spectral response acceleration of the resulting 84th percentile deterministic ground motion response spectra shall not be less than $1.5 \times F_a$ where F_a is equal to 1.0.
- the DE spectrum shall not fall below 80 percent of Sa determined in accordance with Section 11.4.6, where F_a is determined using Table 11.4-1 and F_v is taken as 2.5 for $S_1 \geq 0.2$ (Section 21.3 of Chapter 21 ASCE 7-16). The site-specific MCE_R spectral response acceleration at any period shall not be taken as less than 150 percent of the site-specific design response spectrum determined in accordance with Section 21.3.

Table 11 presents digitized values of the site-specific spectra for the PSHA 2,475-year return period (max. dir.) and the 84th percentile deterministic (max. dir.). The largest spectral response acceleration of the 84th percentile deterministic response spectrum is 2.281g and is greater than $1.5 \times F_a$ (where F_a=1.0 for Site Class D); therefore, no further scaling of the 84th percentile deterministic spectra was needed.

Plate No. 49 and Table 11 present a comparison of the site-specific spectra for the risk-targeted 2,475-year return period PSHA and the 84th percentile deterministic spectra, both in the maximum direction. In this case, the 84th percentile deterministic spectrum is less than the risk-targeted PSHA spectrum for a 2 percent probability of exceedance in 50 years (2,475-year return period) for periods less than 4 seconds and therefore, the deterministic spectrum should be used as the basis for the development of the MCE_R spectrum for periods less than 4 seconds. For periods greater than 4 seconds the results of the PSHA are less than the 84th percentile deterministic spectrum and therefore, the results of the PSHA should be used as the basis for development of the MCE_R spectrum. The DE spectrum is defined as 2/3 times the MCE_R; however, the DE spectrum should not be less than 80 percent of the DE code spectrum as determined using F_a equal to 1.0 and F_v equal to 2.5 (per Section 21.3 of ASCE 7-16). As shown on Plate No. 49 and Table 11 the DE spectrum is greater than 80 percent of the of the DE code spectrum for all periods.

TABLE 11 – DEVELOPMENT OF SITE-SPECIFIC MCER SPECTRUM

Period (sec)	Risk Targeted Probabilistic MCE_R Max. Direction (g)	Largest 84^{th} Percentile Deterministic Max. Direction (g)	Lesser of Two Spectra Initial MCE_R (g)	2/3 Site Specific MCE_R Initial DE (g)	ASCE 7-16 80% DE Section 21.3 Site Class D; Fv=2.50 (g)	Recommended Spectra	
						MCE_R (g)	DE (g)
0.000	0.930	0.957	0.930	0.620	0.320	0.930	0.620
0.100	1.624	1.420	1.420	0.947	0.800	1.420	0.947
0.150	1.946	1.743	1.743	1.162	0.800	1.743	1.162
0.200	2.143	1.960	1.960	1.307	0.800	1.960	1.307
0.250	2.242	2.120	2.120	1.413	0.800	2.120	1.413
0.300	2.292	2.239	2.239	1.493	0.800	2.239	1.493
0.400	2.281	2.334	2.281	1.521	0.800	2.281	1.521
0.500	2.161	2.266	2.161	1.441	0.800	2.161	1.441
0.750	1.754	1.884	1.754	1.169	0.800	1.754	1.169
1.000	1.487	1.631	1.487	0.991	0.800	1.487	0.991
1.500	1.085	1.192	1.085	0.723	0.534	1.085	0.723
2.000	0.844	0.919	0.844	0.563	0.400	0.844	0.563
3.000	0.874	0.585	0.574	0.383	0.266	0.574	0.383
4.000	0.424	0.423	0.423	0.282	0.200	0.423	0.282
5.000	0.327	0.338	0.327	0.218	0.160	0.327	0.218

9.4.6 DESIGN RESPONSE SPECTRUM

The site-specific Recommended Design Spectrum (DRS) is defined in ASCE 7-16 Section 21.3 as two-thirds of the site-specific MCE_R, but not less than 80% of the general design response spectrum is presented above:

TABLE 12 – DEVELOPMENT OF SITE-SPECIFIC DESIGN RESPONSE SPECTRUM

Period (seconds)	Site Specific MCE_R (g)	Design Response Spectrum (g)
0.000	0.930	0.620
0.100	1.420	0.947
0.150	1.743	1.162
0.200	1.960	1.307
0.250	2.120	1.413

Period (seconds)	Site Specific MCE_R (g)	Design Response Spectrum (g)
0.300	2.239	1.493
0.400	2.281	1.521
0.500	2.161	1.441
0.750	1.754	1.169
1.000	1.487	0.991
1.500	1.085	0.723
2.000	0.844	0.563
3.000	0.574	0.383
4.000	0.423	0.282
5.000	0.327	0.218

Since site-specific procedure was used to determine the recommended response spectra, the corresponding values of SMS, SM1, SDS and SD1 per Section 21.4 of ASCE 7-16 should be used as shown in Table 13.

TABLE 13 – SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Revised Parameters	Values
0.2-second Period Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects (short), SMS	2.054g
1.0-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for the Site Effects, SM1	1.724g
0.2-second Period, Design Earthquake Spectral Response Acceleration, SDS	1.369g
1.0-second Period, Design Earthquake Spectral Response Acceleration, SD1	1.149g

10.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface investigation, we conclude that from a geotechnical engineering viewpoint, the proposed structure may be constructed as planned, provided the design and construction are performed in accordance with the recommendations presented in this report.

PRIMARY GEOTECHNICAL CONCERNS

The primary geotechnical and geologic concerns at the site are as follows:

- Presence of highly expansive soils
- Shallow Groundwater & Considerations
- Differential Movement at On-grade to On-Structure Transitions
- Potential for Corrosion for buried steel
- Differential Settlement for Utility Tie-ins
- Potential for liquefaction-induced total and differential settlement
- Compressible soils and Consolidation
- Shoring Considerations/Excavation and Monitoring
- Demolition of the existing buildings and pavements, prior to site development

We have prepared a brief description of the issues and present typical approaches to manage potential concerns associated with the long-term performance of the development.

10.1 PRESENCE OF HIGH EXPANSIVE SOILS

Moderately to high expansive surficial soil generally blankets the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage, the planned flatwork should have sufficient reinforcement and be supported on a (12)-inch layer of non-expansive fill material. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

10.2 SHALLOW GROUNDWATER & CONSIDERATIONS

As discussed in section above, groundwater was encountered in our CPTs and exploratory borings at depths ranging from approximately (±4.2) to (±9.4)-feet below the existing ground surface. Published information by CGS indicates historical high groundwater levels have mapped to be approximately at a depth of (±4.0)-feet below existing ground surface. Based on the anticipated depth of the proposed building excavation and our experience with similar sites in the vicinity, shallow groundwater could significantly impact grading and underground construction. These impacts typically consist of potentially wet, over-saturated and unstable subgrade, difficulty achieving compaction, and difficult underground utility installation. Dewatering and shoring of utility trenches may be required in some isolated areas of the site. Detailed recommendations addressing this concern are presented in the following sections of this report.

CONSIDERATIONS: We recommend that a groundwater elevation (±6.5)-feet (above msl) be considered as the design groundwater elevation. The basement walls and floor should be designed to resist lateral and uplift hydrostatic pressures, respectively, using the same design groundwater elevation of (±6.5)-feet (above msl). Tiedown anchors may be required to resist the anticipated uplift pressures. The basement walls and floors should be waterproofed and water-stops should be provided across all below grade construction joints.

To construct the basement of the building, the groundwater will need to be lowered to a depth of at least three feet below the bottom of the planned excavation and maintained at that level until sufficient weight and/or tiedown capacity is available to resist the hydrostatic uplift forces on the bottom of the foundation.

Variables which significantly influence the performance of the dewatering system and the quantity of water produced include the number of wells, the depth and positioning of the wells, the interval over which each well is screened, and the rate at which each well is pumped. Different combinations of these variables can be used to successfully dewater the site. The site dewatering should be designed and implemented by an experienced dewatering contractor. However, we should review the dewatering system proposed by the contractor prior to installation. Additional wells may be needed where deep local excavations are planned (such as elevator pits for example).

Dewatering the site should remain as localized as possible. Widespread dewatering could result in subsidence of the area around the site due to increases in effective stress in the soil. Nearby streets and structures should be monitored on a regular basis for vertical movement and groundwater levels, outside the area of excavation monitored through wells while dewatering is in progress. Should excessive settlement or/ groundwater drawdown be measured, the contractor should be prepared to recharge the groundwater outside the excavation through recharge wells. A recharge program should be submitted as part of the dewatering plan.

10.3 DIFFERENTIAL MOVEMENT AT ON-GRADE TO ON-STRUCTURE TRANSITIONS

Some of the at-grade areas and other improvements will transition from on-grade support to overlying the below-grade parking levels. Where the depth of soil cover overlying the roof of the parking levels is thin or/ where the walls of the parking levels extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of backfill closest to the wall. We recommend consideration be given where engineered fill is placed behind retaining walls extending to near finished grade, and that subslabs be included beneath flatwork or/ pavers that can cantilever at least (3.0)-feet beyond the wall. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to a minimum of 95 percent relative compaction as per ASTM D1557 where surface improvements are planned.

10.4 POTENTIAL FOR CORROSION

As discussed above, the corrosion potential to buried metallic improvements constructed within the native soils may be characterized as very severely corrosive. A qualified corrosion engineer should be engaged to provide site-specific recommendations regarding corrosion protection for concrete, steel, buried metal pipe or buried metal pipe-fittings etc.

10.5 DIFFERENTIAL SETTLEMENT FOR UTILITIES TIE-INS

The utilities entering the structure could experience differential settlement at the tie-in locations. We recommend emergency shut-off valves and flexible utility and piping connections that can accommodate at least two inches of movement be considered in the design.

10.6 LIQUEFACTION-INDUCED TOTAL AND DIFFERENTIAL SETTLEMENT

As mentioned in the sections above, potentially liquefiable soils were encountered in the upper (50)-feet of subsurface soils and total/differential settlements reflected in the liquefaction section of this report should be considered in the design and are considered appropriate for the design of shallow foundation.

10.7 COMPRESSIBLE SOILS & CONSOLIDATION

As discussed in the subsurface soil conditions section above, we encountered layers of moderate to medium stiff clays with interbedded thin layers of organic peats. Based on our discussion, it is our understanding that an excavation of about (±12) to (±14)-feet is anticipated for the basement level. As mentioned, above, exploratory borings and CPTs indicate that the basement level will be underlain by a wet, soft, compressible layer of clay with interbedded, thin, very compressible peat layers. We judge these layers will compress under the weight of the building. Furthermore, the medium dense sandy layers underlying the subsurface may be prone to consolidation and liquefaction induced settlement, as discussed in section above. This settlement may be random and erratic.

Laboratory test results also indicate these soils are over-consolidated with an over-consolidation ratio (OCR) of greater than 2. These layers will consolidate somewhat under light foundation loads, however if heavy loads are applied to these strata from either the weight of new structures or/ new fill, then settlement will occur ranging between (±2.0) to (±2¼)-inches in addition to the (±1¼)-inch settlement anticipated due to liquefaction.

10.8 SHORING CONSIDERATIONS

During excavation of the basement and construction of the building, the adjacent property and streets should be supported by temporary shoring and underpinning. There are several key considerations in selecting a suitable shoring system. Those we consider to be primary concerns are:

- protection of surrounding improvements, including roadways, underground utilities and adjacent structures
- penetration of shoring supports into the clay below the bottom of the excavation
- proper construction of the shoring system to minimize the potential for ground movement
- cost

We recommend a soldier pile and lagging system be selected as the shoring system for this project. Soldier piles should be placed in predrilled holes which will be backfilled with concrete or installed with a soil-cement mixing drill rig. Wood lagging should be placed between the soldier beams as the excavation proceeds. Drilling of the boreholes for the soldier piles may require casing and/or the use of drilling mud to prevent caving of any sand layers below the subsurface. Furthermore, the depth of the excavation will be on the order of (±14)-feet and the shoring system may require a system of lateral restraint. Either grouted tiebacks or/ internal bracing that is considered acceptable.

Tiebacks will require encroachment permits from adjacent property owners. Tiebacks on the street sides of the excavation should avoid underground utilities in the street. Minor deflections of the ground surface and adjacent structures should be expected with a soldier pile and lagging system. The amount of movement and distress to adjacent improvements will depend on the rigidity of the shoring and the workmanship of the contractor. If cohesionless layers are encountered, some caving may occur while lagging boards are installed. To reduce movements and caving, it may be necessary to limit the unsupported height of the excavation to the height of the lagging boards.

During excavation, the shoring and underpinning systems are expected to yield and deform, which could cause surrounding improvements to settle and move. The magnitude of shoring movements and resulting settlements are difficult to estimate because they depend on many factors, including the method and the shoring contractor's skill in the installation. We estimate a properly installed system will limit settlements to adjacent improvements to less than one inch. The settlement should decrease linearly with distance from the excavation, and should be relatively insignificant at a distance twice the depth of excavation.

The selection, design, construction, and performance of the shoring and underpinning system should be the responsibility of the shoring contractor. A structural/civil engineer knowledgeable in this type of construction should be retained to design the shoring. We should review the final shoring and underpinning plans to check that they are consistent with the recommendations presented in this report.

10.9 EXCAVATION AND MONITORING

Below-grade excavations may be constructed with temporary slopes in accordance with the “Temporary Cut and Fill Slopes” section above if space allows. Alternatively, temporary shoring may support the planned cuts. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor’s judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor’s responsibility.

A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor’s scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

10.10 WET, UNSTABLE EXCAVATION SUBGRADE SOIL

The proposed building excavation will extend into saturated clay and possibly into sandy layers with varying strength. Due to the high moisture content and relatively low strength of this material, it may become unstable under the weight of track-mounted or/ rubber-tired construction equipment. To provide a firm base for construction of the foundation, it may be necessary to remove and an additional approximately (18)-inches of native soil below the foundation level and replace it with a bridging layer, such as crushed rock and stabilization fabric (Mirafi 700X). Otherwise, a layer of lean cement-sand slurry layer ("rat slab") can be considered or a combination of the two. Temporary dewatering to a depth of at least three to five-feet below the bottom of the building excavation will be required during construction.

10.11 DEMOLITION DEBRIS

Construction debris both above and below grade is anticipated as a result of the site demolition required prior to site grading. The debris should be either: 1) collected and off-hauled to an appropriate facility prior to beginning the earthwork for the project, or 2) the concrete crushed and re-used as fill at the site. Recycled materials containing asphalt concrete (AC) should not be used below interior floor slabs, therefore if recycled materials are proposed to be re-used beneath interior floor slabs, AC pavements should be segregated from the debris. It has been our experience that some debris will remain in the soil on-site after the demolition contractor has completed their work. Therefore, it should be anticipated that some debris would be encountered in excavations for underground utilities and foundations. It has been our experience that some coordination between the demolition contractor, grading contractor and geotechnical engineer is needed to identify the scope of the excavation backfill and other similar work items. Recommendations for re-use of recycled materials are presented in the Earthwork section of this report.

10.12 PLANS, SPECIFICATIONS AND CONSTRUCTION REVIEW

We recommend that our firm perform a plan review of the geotechnical aspects of the project design for general conformance with our recommendations. In addition, subsurface materials encountered in the relatively small diameter, widely spaced borings and CPTs may vary significantly from other subsurface materials on the site. Therefore, we also recommend that a representative of our firm observe and confirm the geotechnical specifications of the project construction. This will allow us to form an opinion about the general conformance of the project plans and construction with our recommendations. In addition, our observations during construction will enable us to note subsurface conditions that may vary from the conditions encountered during our investigation and, if needed, provide supplemental recommendations. For the above reasons, our geotechnical recommendations are contingent upon our firm providing geotechnical observation and testing services during construction.

11.0 RECOMMENDATIONS -

11.1 EXISITNG UTILITIES

Existing utilities located within the areas of proposed improvements should be removed in their entirety ten-feet beyond the exterior face of the proposed footings. Utilities within the proposed areas of improvements (outside the building pad area) could be considered for in-place abandonment, provided they do not conflict with new improvements, that the ends and all laterals are located and completely pressure grouted and the previous fills associated with the utility trenches do not pose a risk to the proposed improvements.

Utilities outside the areas of proposed improvements should be removed or/ abandoned in-place by grouting or plugging ends with concrete. Fills associated with utilities abandoned in place could pose some risk of settlement; utilities that are plugged could also pose some risk of future collapse or erosion should they leak or become damaged. The potential risks are relatively low for smaller diameter pipes abandoned in place and increasingly higher with increase in diameter.

11.2 EARTHWORK SECTION – DEMOLITION, CLEARING & SITE PREPARATION

Any areas to be graded should initially be cleared of all obstructions, including the buried foundation system (if any), footings, underground utility pipes, including drain lines, landscape areas, brush, trees not designated to remain, debris, stumps, root balls, existing pavements, rubble and debris should be removed completely and hauled off from the site. Roots greater than ½-inches diameter shall be removed completely. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about three to six- inches below existing grade in vegetated areas.

All active or/ inactive underground utilities within the construction/building pad area and ten-feet beyond should be relocated or/ removed completely from the areas of proposed improvements without exception. Any pipes that are abandoned in place beyond the areas of proposed improvements should be filled/pressure grouted with non-shrink grouts/cement slurry. Depressions/Holes/ excavated areas resulting from the removal of the existing foundation or/ underground utilities, drain-inlets, subsurface obstructions, trees/root balls etc. below the existing or/ proposed finished subgrade levels should be cleared for engineered fill (as per the engineered fill requirements reflected in section 11.5 below).

After the excavation/removal of the structures, slabs, foundation elements, utility lines, root balls etc., the excavations shall be backfilled (if needed) and compacted to a minimum of 95% relative compaction as per ASTM D1557. Upon backfill of the trenches, the entire excavated bottom shall be scarified to a minimum of (12)-inches, and compacted to 95% relative compaction as per ASTM D1557, prior to placement of any additional fill material. The fill material shall be placed in lifts and each lift shall also be compacted to 95% relative compaction as per ASTM D1557.

Over-excavation and re-compaction should extend laterally a minimum of (10)-feet beyond the limits of structures/face of the footings/Pile caps (where achievable), and (2)-feet beyond for all flatwork including patios, sidewalk, walkways, stress pads, vertical curbs/curb and gutter areas and other miscellaneous improvements. Please note that all earthwork operations at the site including demolition and fill placement shall be observed by Advance Soil Technology, Inc. ("AST") or/ by its representative. It is important that during the demolition, removal of buried footings, buried structures, underground utilities, drain-inlets, below grade structures, stripping and scarification process our representative be present to observe whether any undesirable material is encountered in the construction area and whether exposed soils are similar to those encountered during our geotechnical/field investigation.

11.2.1 CONSTRUCTION DEWATERING

Historic groundwater elevations are mapped to be approximately (±4'-0")-feet below the existing ground surface; therefore, temporary dewatering may be necessary during construction of the new foundations associated with the new retaining walls. Design, selection of the equipment and dewatering method, and construction of temporary dewatering should be the responsibility of the contractor. Modifications to the dewatering system are often required in layered alluvial soils and should be anticipated by the contractor. The dewatering plan, including planned dewatering well filter pack materials, should be forwarded to our office for review prior to implementation.

The dewatering design should maintain ground water at least three-feet below the bottom of the mass excavation, and at least to the bottom of foundation excavations and elevator shafts. If the dewatering system was to shut down for an extended period of time, destabilization and/or heave of the excavation bottom requiring over-excavation and stabilization, flooding and softening, and/or shoring failures could occur; therefore, we recommend that a backup power source be considered. Depending on the ground water quality and previous environmental impacts to the site and surrounding area, settlement and storage tanks, particulate filtration, and environmental testing may be required prior to discharge, either into storm drain or/ sanitary sewer or/ trucked to an off-site facility under an appropriate permit from the agencies.

11.3 PAD PREPARATION

Following the clearing and grubbing operation, demolition, over-excavation, removal of the existing fills and disturbed soils areas/backfill of the existing subsurface structures, underground utilities and any obstructions that interfere with installation of proposed foundation system, the existing exposed at-grade subgrade shall be scarified to a minimum depth of (12)-inches, moisture conditioned as needed and compacted to a minimum of 95% relative compaction as per ASTM D1557, prior to any placement of any fill material. Any additional fill placed on the pads shall be compacted to 95% relative compaction as per ASTM D1557.

After the completion of the above, the entire building pad area and ten-feet beyond the building footprint or/ the exterior face of the foundation element shall be graded to a depth of the subgrade (native soil) to accommodate structural under-slab section under the structural mat slab (thickness as designed and reflected by structural engineer) or/ the Civil Engineer. The bottom of the below-grade excavation/subgrade shall then be uniformly graded to the required subgrade elevation and lime treated to a minimum depth of (18)-inches below the subgrade elevation or/ utilize other stabilization measures (for information, please refer to the lime treatment section of this report). Upon completion, the subgrade shall be compacted to a minimum of 95% relative compaction as per afore-mentioned procedure, smooth drum-rolled sealed off, prior receiving other improvements.

11.4 SUBGRADE STABILIZATION MEASURES

Soil exposed at the bottom of the proposed basement excavation (subgrade) that predominantly consists of clayey silt to silty clays and sands with varying amounts of fines will be below the groundwater level. The soil at the basement grade will be near or/ over-saturation even after dewatering. Since the moisture content is already over the laboratory optimum moisture, the soil will be subjected to softening and yielding (pumping, especially from construction loading or/ become unworkable during placement and compaction. Additionally, the exposed subgrade at the foundation level will be susceptible to disturbance movement of the construction equipment and the loads induced by them. In addition, repetitive rubber-tire loading will destabilize the subgrade soils.

To help protect the soil subgrade a working pad (please refer to options below) should be constructed during the initial phase of construction, prior to installation of piles and this will require stabilization measures, over-excavation, lime/cement treatment or/ drying of subgrade soils, prior to placement of a minimum (3.0) to (4.0)-inch thick mud/rat slab or/ as designed to provide much needed support for the waterproofing and to help support the rebar/structural steel until the mat is constructed.

There are several alternate methods to address unstable soil conditions and facilitate subgrade preparations, fill placement and trench backfill etc. Implementation of appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals that are particular to the site. We have provided these options below that could be considered or/ selected based on anticipated cost impact and encountered subsurface soil conditions.

11.4.1 SCARIFICATION AND DRYING

The subgrade may be scarified to a depth of (12)-inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of or/ constant scarification may be needed to break up the soil clods. The amount of time required to dry the soil will depend on the weather conditions and therefore should be determined by the general contractor, based on the project schedule.

11.4.2 REMOVAL AND REPLACMENT

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or/ import materials. AST representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthetic is required and what import materials could be utilized for backfill/replacement. As mentioned above, some of the planned excavations will extend below the design groundwater level, we recommend that the contractor plan to excavate an additional (±18)-inches below the required subgrade elevation, place a layer of stabilization fabric (Mirafi 700X, or/ equivalent) at the bottom, and backfill with clean crushed rock. The crushed rock should be consolidated in place with light vibratory equipment. Rubber-tire equipment should not be allowed to operate on the exposed subgrade; the crushed rock should be stockpiled and pushed out over the stabilization fabric using a track equipment. For budgeting purposes, an allowance for this working pad should be included as an option.

11.4.3 CHEMICAL TREATMENT

The unstable exposed subgrade soils could also be chemically treated in place to a depth of (18)-inches or/more in place with HiCal Quicklime or/ Cement to achieve the desired stability required for the construction phase of the project. This process may be more cost effective than removal/replacement of unstable soils and off-haul from the site. Recommended chemical treatment depths will typically range depending on the magnitude of the instability, however the recommendation reflected in section (11.6) could be utilized for budgeting purposes.

Please note that the need for the working pad and the thickness of rock section should be evaluated by the general contractor and the pile sub-contractor for stability of equipment support, when the bottom of the excavation or/ over-excavation is reached; AST will provide the recommendations as needed for stabilizing the subgrade. However, this is considered as a Construction Issue, "Means & Methods" and is usually needs to be addressed by the general contractor.

11.5 FILL MATRIALS

11.5.1 ON-SITE SOILS

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 3-inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding (12)-inches. All fills shall be compacted to a minimum of 95 percent relative compaction as per ASTM D1557.

11.5.2 MATERIALS FOR ON-SITE IMPROVEMENTS

Based on the existing site conditions, there would be significant quantities of asphalt concrete (AC) grindings, aggregate base (AB), and Portland cement concrete (PCC) will be generated during site demolition. If the AC grindings are mixed with the underlying AB to meet Class 2 AB specifications, they may be utilized as recycled aggregate base under the new pavement and flatwork structural sections, provided the PCC is pulverized meets the "Material for Fill" requirements of this report or/ meets Class 2 AB specifications, the recycled PCC may likely be used within the pavement structural sections. PCC grindings also make good winter construction access roads. AC/AB grindings may not be reused within the building areas. Laboratory testing will be required to confirm the grindings meet project specifications.

11.5.3 GEOTECHNICAL REQUIREMENTS FOR IMPORT MATERIALS

Any and all imported soil required at the site of proposed development from an off-site source shall consist of non-expansive fill material or/ lime treated base rock and shall comply with the following geotechnical criteria for evaluation and acceptability of the material. This is to include but not limit the materials in the pad areas, under concrete slab-on-construction and backfill behind walls or/ retained structures shall be primarily granular material with low plasticity and expansion potential:

- Resistance R-Value Not less than 25

➢ Plasticity Index	12 or/ less
➢ Liquid Limit	30% or/ less
➢ Expansion Index	20% or/ less
➢ Passing Sieve #200	Between 10 and 20%
➢ Maximum rock size	≤ (3)-inches

11.6 LIME/CEMENT TREATMENT

Due to the presence of high moisture content and expansion potential in the soil at the site, the subgrade soils under the building pad and five-feet beyond footprint and areas with excessive moisture could be lime (HiCal Lime) treated to lower the moisture content and expansion potential. The lime treatment shall penetrate the proposed subgrade/the bottom of the excavation to a minimum depth of (18)-inches, below the exposed ground surface.

Lime/Cement treatment shall be conducted with appropriate equipment, such that a uniform mix (5 to 6% by weight of the soil approximately (120.0)-pounds per cubic feet) is achieved over the entire area. Upon achieving the uniform mix, the lime treated material shall then be compacted to a minimum of 95% relative compaction as per ASTM D1557. After the initial mix and hydration, the treated material shall be allowed to mellow a minimum of (36)-hours, prior to the re-mixing the material.

Upon completion of the mellowing time, the material shall be moisture conditioned as needed, remixed and re-hydrated to the full depth of treatment, prior to achieving the desired degree of 95% relative compaction as per ASTM D1557. The lime treatment contractor shall discuss the treatment procedure with the Soil Engineer and submit the process for review and recommendations for the type of equipment usage.

11.7 WEATHER/MOISTURE CONSIDERATIONS

All imported soil/fill material shall be approved by the Geotechnical Engineer, prior to hauling the material to the site. Based on our experience in the area, grading during the rainy season may be difficult due to the type of soil at the site. If earthwork operations and construction for this project are scheduled to be performed during the rainy season or in areas containing saturated soils, provisions may be required for drying of soil or providing admixtures to the soil prior to compaction. If desired, we can provide supplemental recommendations for wet weather earthwork and alternatives for drying the soil prior to compaction. Conversely, additional moisture may be required during dry months. Water trucks should be made available in sufficient numbers to provided adequate water during earthwork operations. If site grading is performed during the rainy months, the site soils could become very wet and difficult to compact without undergoing significant drying. This may not be feasible without delaying the construction schedule. For this reason, drier import soils could be required or lime treating may be needed if construction takes place during winter months.

11.8 TEMPORARY SLOPES & TRENCH EXCAVATIONS

The contractor should be responsible for all temporary excavations, slopes and trenches excavated at the site and the design of any required shoring system. Shoring, bracing and benching shall be performed by the contractor in accordance with the strict governing safety standards. Temporary shoring is usually considered as a construction issue (means and methods) and has to be addressed by the contractor. Shoring and bracing should be provided in accordance with all applicable local, state and federal safety regulations, including but not limited to the current OSHA excavation and trench safety standards. Surface water inflows into excavations must be prevented from causing caving and running ground conditions. Field conditions must be carefully assessed before excavations are made so that appropriate measures can be taken to prevent sloughing, caving and excessive ground movement during the construction phase.

11.9 ON-SITE UNDERGROUND UTILITY TRENCHING

Differential Settlement for Utility Tie-ins The utilities entering the structure could experience differential settlement specifically at the tie-in locations. We recommend emergency shut-off valves and flexible utility and piping connections that could accommodate at least two inches of vertical and horizontal movement.

Bedding and embedment materials around the underground utility lines should be well-graded sand or gravel and should be placed and compacted in accordance with the project specifications, local requirements and governing jurisdictions. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or/ fine gravel. After pipes and conduits are tested, inspected (if required), and approved, they should be covered with a minimum of six-inches with sand or fine gravel above the pipe, which should then be mechanically tamped or/ in accordance with the requirements of City of San Jose. Where trenches extend below the groundwater level, it will be necessary to temporarily dewater them to allow for placement of the pipe and/or conduits and backfill.

All underground utility trenches on-site must be compacted to a minimum of 95% relative compaction per requirements of the local governing agency or/ as recommended by the Soils Engineer and in accordance with the test procedure ASTM D1557. Utility trenches located adjacent to the existing or proposed structures shall be no closer than the required slope criteria 2:1(horizontal to vertical). This means that no trenches should be located within an area, which would intercept the hypothetical slope line drawn from the bottom edge of the footing at a 2:1 (horizontal to vertical) slope.

The trenches could be backfilled with base rock, quarry fines, cement slurry or/ with concrete densified fill all the way up to the required subgrade elevation. The material shall be moisture conditioned (±2 to 3% over optimum) and shall be placed in (8) inch un-compacted lifts and each lift shall be compacted to a minimum of 95% relative compaction as per ASTM D1557.

Utility Trenches that are crossing the foundation shall be backfilled with concrete/ cement slurry, a minimum of four (±4.0)-feet on either side of the footing. Trenches located in the landscape areas should be compacted to a minimum of 90% relative compaction with the exception of the top-foot, compacted to a minimum of 85% relative compaction as per ASTM D1557.

Please note that jetting of the trenches will not be permitted (without any exception) at any time during the backfill operation.

12.0 FLOOR SLABS

12.1 STRUCTURAL SLABS

As discussed below in the foundation section, a structural reinforced mat may be utilized as a foundation element to support the proposed structure. Based on the depth of the excavation for the below grade parking/basement, it is our understanding that the excavation extends below the design groundwater level. Therefore, we recommend the design ground water Elevation of (±6.5)-feet (above msl) be considered/used to evaluate hydrostatic uplift. A structural mat system is anticipated to be a very cost-effective foundation system for this project since the pad can remain relatively flat which allows for more efficient construction of waterproofing system, thereby saving a significant amount of time and labor. In addition to the above, the floor slab should be waterproofed and a waterproofing consultant should be retained to provide recommendations for the type of waterproofing and its installation.

Please also note that the on-site soils are very severely corrosive and contains moderate levels of the sulfate ion concentration. The floor slabs should be designed to mitigate the effects of corrosion and sulfate ion concentration.

12.2 UNDERSLAB DRAINAGE

In addition to the perimeter building retaining wall subdrain system; and if it is planned to drain all walls and floors, a network of subdrain trenches embedded in a layer of gravel beneath the underlying capillary break also can be used for water drainage. Trenches should be a minimum of (12.0)-inches-square and should consist of a (4.0)-inch minimum diameter perforated pipe surrounded with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition.

Alternatively, ½-inch to ¾-inch diameter crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. Trenches should be spaced approximately (20) to (25)-feet apart. The trench/rock may need to be thickened to allow the pipe to gravity feed to the drainage pump system.

As an alternative to subdrain trenches, TenCate Mirafi G200N drainage composite, Contech C-100 strip-drain (filter geotextile bonded to both sides), or an approved equivalent drainage system also can be used. The drainage system should be placed directly beneath the capillary break and spaced approximately (15) to (20) feet apart. The drainage system (drainage panels) should be connected to a drainage pump system.

However, if the design team or/ the waterproofing consultant considers the designing the project as a bathtub, then AST does not foresee the need for the under-slab drainage, provided it is designed for uplift, inspected and approved by the waterproofing consultant.

12.3 HYDROSTATIC UPLIFT AND WATERPROOFING

Because of the proximity of the ground water table to the proposed basement floor and/or floor slab should be designed to resist potential hydrostatic uplift pressures. Retaining walls extending below design ground water should be waterproofed and designed to resist hydrostatic pressure for the full wall height. Where portions of the walls extend above the design ground water level, a drainage system may be added as discussed in the "Retaining Wall" section of this report; otherwise, the walls should be designed as undrained for the full height.

In addition, the portions of the structures extending below design ground water should be waterproofed to limit moisture infiltration, including thickened slab areas, all construction joints, and any retaining walls. Water stops should be placed across all construction joints. The waterproofing should be placed directly against the backside of the basement walls. Substrate preparation should be as per the manufacturer's recommendation for blind-side application.

Waterproofing should be designed by an experienced waterproofing consultant and placed and inspected in accordance with the manufacturer's specifications. The use of a mud slab below the waterproofing will reduce the potential for subgrade disturbance and protect the waterproofing from damage during foundation construction. The mud slab should also provide a firm, smooth working surface for placement of reinforcing steel. Because of the various waterproofing products available, the specifications for installing the waterproofing or/ the need for a mud slab or/ protection slab should be considered or/ provided, if required by the waterproofing consultant.

Quality control is a critical element to a successful waterproofing project. The waterproofing installation should be inspected daily basis, especially during placement of reinforcement for the floor slabs and perimeter walls. Any holes or/ tears should be repaired in accordance with the manufacturer's recommendations and utility penetrations should be carefully sealed. All seams, including separations between wall and slab membranes should be checked for tightness. We recommend that the waterproofing manufacturer inspect the waterproofing operations during construction and approve all work prior to placement of concrete. We also suggest discussing waterproofing detailing with the selected product manufacturer.

12.4 EXTERIOR FLATWORK

Due to the anticipated settlement surrounding pile-supported structure, hardscape areas surrounding the building (including vertical curbs, curb and gutter areas) and flatwork connecting to pile-supported building entrance areas, should be designed as a hinged slab to prevent separation at the joint with the building. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least (5.0)-inches thick, reinforced with a minimum of #3 rebar spaced (12)-inches on center both ways and supported on at least (4.0)-inches of Class 2 aggregate base overlying (12)-inches of non-expansive material overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below.

To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included in the design. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

13.0 FOUNDATION

As discussed above, the proposed four-story above-grade structure will be supported on a one-level below-grade parking. Based on the planned depth of excavation, the proposed structure may be supported on a reinforced concrete mat foundation, bearing on (12)-inches of gravel and engineered fill or/ pad prepared in accordance with the "Earthwork" section of this report, and designed in accordance with the recommendations below.

13.1 ALLOWABLE MAT BEARING PRESSURE

Based on the anticipated structural loads and the preliminary estimated areal pressures provided by the structural engineer of about 1,250 pounds per square foot (Dead Load: 800 psf and Live Load: 450 psf). To reduce potential differential movement, the mat should be designed for a maximum average areal bearing pressure of 1000 pounds per square foot (psf) for dead plus live loads; we recommend the allowable bearing pressure at heavier loaded portions of the mat slab be limited to 1,500 psf for dead plus live loads. The maximum bearing pressure may be increased by one-third for all loads, including wind or/ seismic. This pressure is a net value; the mat weight may be neglected for the portion of the mat extending below grade. Top and bottom reinforcing steel should be included as required to help span irregularities and differential settlement. It is essential that we observe the mat foundation pad, prior to placement of reinforcing steel.

13.2 MAT FOUNDATION SETTLEMENT

We estimate the total settlement due to static loading would be about (2.0) to (2¼)-inches and total post-construction differential movement of up to (1¼)-inches across the mat area. In addition, the mat should be designed to accommodate up to an additional one-inch of seismic differential movement between the center and the edge of the mat. Accounting for liquefaction-induced and static differential settlement, we recommend the mat be designed to tolerate total differential movement of (2¼)-inches from the center to the edge of the mat for a (2.0)-foot thick mat. If foundations designed in accordance with the above recommendations are not capable of resisting such differential movement, additional reinforcement or/ increased mat thickness may be required.

13.3 MAT FOUNDATION LATERAL LOADING

Lateral loads may be resisted by friction between the bottom of mat foundation and the supporting subgrade, and also by passive pressures generated against deepened mat edges. An ultimate frictional resistance of 0.45 (0.30 allowable) applied to the mat dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf (300 pcf allowable) may be used in design.

13.4 MODULUS OF SOIL SUBGRADE REACTION

The modulus value of soil subgrade reaction is a model element that represents the response to a specific loading condition, including the magnitude, rate, and shape of loading, given the subsurface soil conditions at that location. We recommend using a variable modulus of subgrade reaction to provide a more accurate soil response and prediction of shears and moments in the mat. This will require at least one iteration between our soil model and the structural SAFE (or/ equivalent) analysis for the mat. A preliminary modulus of subgrade reaction for the initial analysis is provided below for the average areal pressure reflected above. On the basis on this pressure, we calculated a preliminary modulus of soil subgrade reaction for the mat foundation. Based on the anticipated loads for the mat slabs and existing subsurface soil conditions, we recommend an initial modulus of soil subgrade reaction of 5 pci be used for preliminary SAFE runs. As discussed above, this modulus of soil subgrade reaction is intended for use in the first iteration of the structural SAFE analysis for the mat design. Following the output from initial SAFE runs indicating bearing pressures, due to dead plus live loading across the mat, we could provide a revised value for the modulus of subgrade reaction.

13.5 HYDROSTATIC UPLIFT AND WATERPROOFING

As previously discussed in the proceeding section, ground water was encountered at depths of (±4.2) to (±9.4)-feet below the existing ground surface and the design groundwater elevation are considered to be at a depth of (±4.0)-feet below the existing ground surface, based on the historical groundwater contours mapped by CGS. Furthermore, since the planned bottom of the garage levels is (±14)-feet below the existing grade, mat foundations should be designed to resist at least (±12) feet of hydrostatic uplift. To mitigate potential impacts to the structure due to perched water, we recommend the full height of the garage level walls be designed for hydrostatic pressure (an additional 40 pcf of fluid pressure) and be waterproofed.

13.6 DEEP FOUNDATION OR/ GROUND IMPROVEMENT

As an alternative to the mat foundation, if the estimated settlements for mat foundation exceeds the structural requirements, then the proposed structure could also be supported on deep foundation or/ ground improvements such as “Drilled Displacement Columns” below the proposed mat foundation. If these options are desired, then additional recommendations could be provided to the structural engineer.

14.0 SOIL RETAINING STRUCTURES

14.1 TEMPORARY SHORING

Based on the site conditions encountered, the cuts may be supported by soldier beams and tiebacks, braced excavations, or potentially other methods. Where shoring will extend more than about (10)-feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring.

In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, staging areas, etc.) and material stockpiles be kept at least (15)-feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the anticipated loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum recommended geotechnical parameters for shoring design are provided below.

TABLE 16 - DESIGN PARAMETRS FOR TEMPORARY SHORING

Parameter	Design Values
Earth Pressure – Cantilever Wall	40 pcf
Earth Pressure – Restrained Wall (From Ground Surface to H/4 Feet)	0 to 25H psf
Earth Pressure – Restrained Wall (Below H/4 Feet, H is the height of the wall)	25H psf (Uniform Pressure)
Minimum Lateral Wall Surcharge	120 psf
Passive Pressure Starting below/bottom of the adjacent excavation	300 pcf to a Maximum of 1100 psf
Maximum Deflection (Designed Shoring System)	Less than ½-inch

Notes:
1. H equals the height of the excavation; passive pressures are assumed to act over 2.5 times the soldier pile diameter
2. The cantilever and restrained pressures are for drained designs with dewatering. If undrained shoring is designed, an additional 40 pcf should be added for hydrostatic pressures.
3. Bottom of adjacent excavation is bottom of mass excavation or bottom of footing excavation, whichever is deeper directly adjacent to the shoring element.

If shotcrete lagging is used for the shoring facing, the permanent retaining wall drainage materials, as discussed in the "Wall Drainage" section of this report, will likely need to be installed during temporary shoring construction. At a minimum, 2-foot-wide vertical panels should be placed between soil nails or tiebacks that are spaced at 6-foot centers. For 8-foot centers, 4-foot-wide vertical panels should be provided. A horizontal strip drain connecting the vertical panels should be provided, or/ pass-through connections should be included for each vertical panel.

We performed our borings with hollow-stem auger drilling equipment and as such were not able to evaluate the potential for caving soils, which can create difficult conditions during soldier beam, tie-back, or soil nail installation; caving soils can also be problematic during excavation and lagging placement. The contractor is responsible for evaluating excavation difficulties prior to construction. Where relatively clean sands (especially encountered below ground water) or/ difficult drilling or/ gravel/cobble conditions were encountered during our exploration, pilot holes performed by the contractor may be desired to further evaluate these conditions prior to the finalization of the shoring budget.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created, then they should be backfilled as soon as possible with sand, gravel, or/ grout. As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements.

All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. The monitoring frequency should be established and agreed to by the project team prior to start of shoring construction. The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system.

A California-licensed civil or/ structural engineer must design and be responsible for the temporary shoring design. The shoring contractor is responsible for means and methods of construction, as well as site safety.

14.2 RETAINING WALLS

Retaining structures that are free to rotate or translate laterally (Cantilevered Retaining Walls) through a horizontal distance to wall height ratio of no less than 0.004 are referred to as unrestrained or yielding retaining structures. Such walls can generally move enough to develop active earth pressure conditions.

Retaining structures that are unable to rotate or deflect laterally (Restrained Basement Walls) are referred to as restrained or/ non-yielding walls and subject to at-rest earth pressure conditions. Backfill materials behind the wall and within a 1h: 1v projection up from the foundation should consist of free draining gravels or/ granular soils as per Section (11.5.3) depending upon the type of drainage.

14.3 STATIC EARTH PRESSURES

Cantilevered walls with granular soil backfill can be designed for active earth pressures using an equivalent fluid weight of 45 pcf for horizontal backfill and drained conditions (no hydrostatic loading). Restrained walls with granular soil backfill should be designed for at-rest earth pressures estimated using an equivalent fluid weight of 45 pcf assuming drained and horizontal backfill conditions plus an additional uniform lateral pressure of 8H psf, where H is the height of the backfill above the top of the wall footing in feet. Wherever, walls are subjected to surcharge loads, they should be designed for an additional uniform lateral pressure equal to ½ or 1/3 the anticipated surcharge loads for restrained or unrestrained walls, respectively.

Retaining walls with sloping backfill should be designed for an additional uniform lateral pressure of 1 pcf for 3 degrees of slope inclination. The lateral earth pressure distributions should be applied along a vertical line through the heel of the wall between the intersection of the vertical line with the ground surface above the wall and a point defined by the elevation of the lowest structural member of the wall. Surcharge loads induce additional pressures on earth retaining structures. Uniform area surcharge pressures for retaining walls may be assumed equal to 0.5 of the applied surcharge pressures. Please refer to the table below.

TABLE 17 – RETAINING WALL EARTH PRESSURES (DRAINED CONDITION)

Backfill Condition (Horizontal to Vertical)	Lateral Earth Pressure	
	Unrestrained – Cantilever Walls	Restrained – Braced Walls
Level	45 pcf	45 pcf + 8H psf
3:1	50 pcf	50 pcf + 8H psf

Backfill Condition (Horizontal to Vertical)	Lateral Earth Pressure	
	Unrestrained – Cantilever Walls	Restrained – Braced Walls
2½:1	55 pcf	55 pcf + 8H psf
2:1	60 pcf	60 pcf + 8H psf
Additional Surcharge	1/3 Vertical Loads - Top of Wall	1/2 Vertical Loads - top of Wall

Notes:

1. H is the distance in feet between the bottom of footing and top of retained soil
2. Lateral earth pressures are based on an equivalent fluid pressure (EFP)
3. The more critical condition of either at-rest pressure for static conditions or active pressure plus a seismic pressure increment for seismic conditions should be checked. The seismic pressure increment has been calculated based on the design earthquake
4. Where traffic will pass within 10 feet of basement walls, temporary traffic loads should be considered in the design of the walls. Traffic loads may be modeled by a uniform pressure of 100 pounds per square foot applied in the upper 10 feet of the walls
5. If the walls are designed for an undrained condition or/ if adequate drainage cannot be provided behind the wall, then an additional equivalent fluid pressure of 40 pcf should be added to the above-mentioned values in
6. the table for both restrained and unrestrained walls
7. If surcharge loads occur above an imaginary 45-degree line projected up from the bottom of a basement wall, a surcharge pressure should be included in the wall design. For the sections of the shoring adjacent to the neighboring structures, the wall should be designed for the surcharge loads imposed by neighboring structures

14.4 DYNAMIC EARTH PRESSURE FOR RETAINING WALLS

The increase in lateral earth pressure on walls from earthquake loading can be estimated using the Mononobe-Okabe theory, as described by Seed and Whitman (1970). That theory is based on the assumption that sufficient wall movement occurs during seismic shaking to allow active earth pressure conditions to develop. For restrained walls, the increase in lateral earth pressure resulting from earthquake loading can also be estimated using the Mononobe-Okabe theory. Because that theory is based on the assumption that sufficient movement occurs such that active earth pressure conditions develop during seismic shaking, the applicability of the theory to restrained or/ basement walls is not direct. However, Nadim and Whitman (1992) suggest the theory that can be used for such walls.

If the active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures, then an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above. Based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment may exceed the restrained (i.e., at-rest) wall earth pressures. Therefore, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the SEAOC (2010) paper and the 2019 CBC. With respect to the load from lateral earth pressure and ground water pressure, the basic combinations as shown in CBC equations 16-2 and 16-7.

[Eq. 16-2]: 1.2(D + F) + 1.6(L + H) + 0.5(Lr or S or R)

In Eq. 16-2: H - should represent the total static lateral earth pressure for the restrained site wall (use 45 pcf + 8H psf)

[Eq. 16-7]: 0.9(D + F) + 1.0E + 1.6H

In Eq. 16-7: H - should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf)

E – should represent the seismic increment component in Eq. 16-7, a triangular load with a resultant force of $24H^2$, which should be applied one third of the height up from the base of the wall.

The recommendations reflected in the SEAOC paper more appropriately split out "active" earth pressure (and not the restrained "at-rest" pressure) from the report and provide the total seismic increment so that different load factors can be applied in accordance with different risk levels.

14.5 SITE WALLS

We are not aware of any exterior landscape retaining walls for the project. However, minor site walls or/ landscaping walls (i.e., walls less than 6-feet in height) are proposed, then the design of these walls for seismic lateral earth pressures in addition to static earth pressures recommended above is not warranted. These walls could be designed for an allowable bearing capacity of 1000 psf for dead plus live loads and could be increase by one-third for short term seismic and wind loads. The footings shall be a minimum of (18)-inches wide and extend a minimum of (18)-inches below the lowest adjacent grade. The bottom of all footings shall be compacted to a minimum of 95% relative compaction as per ASTM D1557.

14.6 DRAINAGE PROVISIONS

Drainage measures should be provided behind the walls to help collect groundwater seepage and prevent the buildup of hydrostatic pressures. Drainage measures can consist of constructing a vertical drainage system behind the wall by placing free-draining backfill (meeting the requirements of Section 11.5) directly behind the wall. The free-draining material should be at least (1.0)-foot wide and a perforated pipe should be placed at the base of the material to collect and convey water to an outlet.

Depending on the type of free-draining material that is used, filter fabric may be required to separate the drainage material from the adjacent soils or backfill. The backside of retaining walls should be waterproofed to prevent potential effervescence (salt buildup) from forming on the front side of the wall. In lieu of using a (1.0)-foot-wide zone of free-draining backfill material to provide drainage behind the wall, geo-composite drains (for example, Miradrain, manufactured by Mirafi, Inc., or/ similar) can be used to control groundwater and prevent the impact of hydrostatic pressures on the walls. However, if drainage panel products are used, they should be appropriate for the proposed usage and installed in accordance with the manufacturer’s recommendations. In either case, a perforated pipe should be placed at the base of the material to collect and convey water to an outlet location.

14.7 COMPACTION ADJACENT TO WALLS

Backfill placed within (5.0)-feet of the retaining structures (measured horizontally behind the wall) should be compacted with lightweight compactor or/ a hand-operated compaction equipment to reduce the potential for developing compaction-induced stresses. If large or/ heavy compaction equipment is used, lateral earth pressures could exceed those presented previously. If larger or/ a heavier compaction equipment is to be used, further evaluation of the potential for compaction-induced stresses in the walls is recommended. Backfill material should be brought up uniformly behind below-grade walls (i.e., the backfill should be at about the same elevation all around the wall as the backfill is placed). That is, the elevation difference of the backfill surface around the wall should not be greater than about (2.0)-feet, unless the wall is designed for the potential for differential backfill heights. The backfill material shall be placed and compacted to a minimum of 90% relative compaction as per ASTM D1557. However, if the backfills are deeper than (5.0)-feet then they should be compacted to 95% relative compaction as per ASTM D1557.

15.0 PAVEMENT DESIGN

15.1 SUBGRADE PREPARATION FOR PARKING LOT AREAS

The proposed areas of improvements including parking lot, vertical curbs and curb and gutter areas shall be graded uniformly, scarified to a depth of (12)-inches (ripped and cross ripped); moisture conditioned, mixed thoroughly to achieve a uniform mix, prior to being compacted. Upon achieving a uniform mix, the soil shall then be compacted to a minimum of 95% relative compaction according to ASTM D1557 test procedure, prior to placement of any additional fill material.

Additional fill material, if required shall be placed in lifts and each lift shall then be compacted to a minimum of 95% relative compaction as per ASTM D1557 all the way up to the required/proposed subgrade elevation. The subgrade preparation for the proposed pavement areas shall extend a minimum of two (2)-feet beyond the curb line and shall also be compacted to not less than 95% relative compaction, using the aforementioned procedure. The material shall be moisture conditioned slightly over the optimum moisture content and shall be spread in lifts not exceeding (8) inches (un-compacted thickness) and compacted to not less than 95% relative compaction using the ASTM D1557 test procedure.

Upon achieving the desired subgrade elevation and relative compaction, the required base rock section shall be placed in lifts and each lift shall be compacted to 95% relative compaction as per ASTM D1557.

15.2 PAVEMENT CUT-OFF/SEEPAGE CONTROL

Concrete slabs around the landscaping areas should be protected from water seepage. The water seepage from these areas usually creates over-saturation of the base rock and the subgrade, thereby causing over-saturated unstable conditions. Henceforth, we recommend the following:

- Provide vertical cut-off or a deep vertical curb section all along the proposed pavement section and the landscape areas. The vertical cut-off should extend through the base rock and a minimum of four inches into the subgrade. The vertical cut-off will limit the moisture intrusion/water seepage around the foundation, into the pavement section and thereby extending the life of the pavement.
- All the utility trenches in the concrete slabs shall be capped with at least one foot of native material or concrete or cement slurry. We recommend that the utility lines located close to the foundations and along the side of the buildings be inspected to make sure they are installed correctly and compacted properly.
- Utility trenches (irrigation lines, electrical conduits, plumbing, etc.) shall not be placed

close to the foundation especially, parallel to the building. This means that no trenches should be located within an area, which would intercept the hypothetical slope line drawn from the bottom edge of the footing at a 2:1 (horizontal to vertical) slope. If the trenches are excavated close to the foundation, then 2:1(horizontal to vertical) slope criterion shall be achieved at all times. If the above-mentioned criteria are not honored or utilized, then the trenches become a pathway for water intrusion into the footing and slab areas, resulting in soil distress and settlement problems.

- In landscape areas, to minimize moisture changes in the natural soils and fills, we recommend the usage of drought resistant plants with a drip irrigation watering system.

15.3 PERMEABLE PAVEMENT

Permeable pavements may be considered as a potential component of the site's surface water management system. Our evaluation of potential site constraints such as subsurface infiltration rate at the site suggests low permeability of the subsurface soils. According to Jackson (2003), the best locations for permeable pavement are parking lots and low-volume roads. Areas with high frequency and/or heavy truck loading should not be considered.

Cahill et al. (2004) recommend permeable pavements be designed to a ratio of 5:1 impervious area to infiltration area and be laid on flat slopes with inclinations of 6 percent or less. A typical permeable pavement section, from top to bottom, includes a porous asphalt course, a top filter course, a reservoir course, an optional bottom filter course, and filter fabric overlying a level base of native un-compacted soil. The thickness of the reservoir course is typically designed to allow complete drainage within 72 hours; however, capacity should be designed by an engineer proficient in hydrology and storm water design and should comply with local regulations.

Permeable pavements should be maintained to promote unobstructed drainage and prevent the accumulation of fines within the system. Pavement edges are usually lined with unpaved stone or catch basins to provide additional drainage pathways to the reservoir course if the asphalt course is repaved or becomes impermeable.

Additionally, the section bottom is typically designed with positive overflow elements to prevent saturation of the pavement if the native soil subgrade becomes impermeable. Our review of relevant literature suggests that the long-term performance of permeable pavement systems is generally unknown and may require periodic maintenance. Pavers, if being proposed for the site could be designed and supported as follows:

- (2)-inch of No. 8 clean washed gravel, compacted to achieve locking action
- (6)-inches of No. 57 clean washed gravel, compacted to achieve locking action
- (10)-inches of No. 2 clean washed gravel, compacted to achieve locking action
- Four-inch Perforated Pipe in the middle
- Mirafi Fabric RS580i or Equal
- (12)-inches of subgrade compacted (min. of 90% compaction per ASTM D1557)

15.4 RIGID CONCRETE PAVEMENT

Rigid Concrete Pavement (Portland Cement Concrete Pavement section) will be required at truck loading dock ramps, stress pads at the trash enclosure areas and where movement of heavy traffic is anticipated. Portland cement concrete pavements are typically better able to resist the intense stresses induced in pavements by the turning motions of vehicles, particularly delivery and garbage trucks. Concrete pavements should be used in areas frequented by such vehicles as well as in driveway and entry aprons. Concrete pavement sections presented in the table below are based on current Portland Cement Association (PCA) design procedures and the assumptions reflected below:

- Modulus of subgrade reaction = 50 psi/in
- Modulus of rupture of concrete = 550 psi
- Aggregate Interlock Joints
- No concrete shoulders
- 20-year design life
- Load Safety Factor = 1.0

TABLE 18 – RECOMMENDED PORTLAND CEMENT

Proposed Usage	Average Daily Truck Traffic	Portland Cement Concrete		Aggregate Base		Subgrade Scarification (inches)
		Inches	Feet	Inches	Feet	
Light Duty	15	7.0	0.58	8.0	0.67	12
Heavy Duty	30	7.5	0.63	8.0	0.67	12

Portland cement concrete pavement sections provided above are contingent on the following recommendations being implemented during construction phase of the project as follows:

- Pavement areas shall be supported on a subgrade, scarified to a depth of (12)-inches; moisture conditioned to 2 to 3% over optimum moisture content and compacted to a minimum of 95% relative compaction as per testing procedure ASTM D1557, prior to placement of the base rock section. The base rock shall be placed and compacted to a minimum of 95% relative compaction.
- Adequate drainage (both surface and subsurface) should be provided such that the subgrade soils are not allowed to become wet.
- Concrete pavement should have a minimum 28-day compressive strength of 4,000 psi. Concrete slumps should be from 3 to 4 inches. The concrete should be properly cured in accordance with PCA recommended procedures and vehicular traffic should not be allowed for 3 days (automobile traffic) or a minimum of 7 days (truck traffic).
- Construction and/or control joint spacing should not exceed (12)-feet.
- Thickened edges should be used along outside edges of concrete pavements. Edge thickness should be at least (4)-inches greater than the concrete pavement thickness and taper to the actual concrete pavement thickness (36)-inches inward from the edge. Integral curbs may be used in lieu of thickened edges.
- To help offset plastic shrinkage, concrete pavement may be reinforced with at least No. 4 bars, 16 inches on-center, both ways (located 1/3 of the slab thickness from the top of the slab).
- Over-finishing of concrete pavements should be avoided. Typically, a broom or burlap drag finish should be used.

The above pavement recommendations should be incorporated into project plans and specifications by the project architect and/or engineer.

15.5 FLEXIBLE PAVEMENT

The following pavement section design is based on an estimated laboratory resistance "R" value of 5 for the near surface soil samples and for the assumed traffic indices ranging between 4.5, 5.5 and 6.5 for parking areas and automobile drive thru areas have been presented in the following sections of this report. Usually, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will be using the pavements or/ heavy traffic is anticipated.

TABLE 19 - RECOMMENDED ASPHALT PAVEMENT SECTIONS

General Traffic Condition	Min. Traffic Index	Asphalt (inches)	Class II Aggregate Base R = 78 min. (inches)	Total Pavement Thickness (inches)	Native Subgrade (inches)	% Relative Compaction
Parking Areas	4.5	3.0	9.0	12.0	12	95%
Driveway Aisles	5.5	3.0	12.0	15.0	12	95%
Truck Traffic & Heavy Traffic Access Areas	6.5	4.0	14.0	18.0	12	95%

- 1) Resistance Value of imported Aggregate base shall be a minimum of 78
- 2) A sample of the material shall be submitted to the geotechnical engineer for approval

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier extending below the subgrade, behind the pavement curb.

16.0 SITE DRAINAGE

16.1 SURFACE DRAINAGE

Bio-swales if proposed for the site, then they shall be located at a minimum of (10.0)-foot offset from the exterior face of the building foundation / footing to the top of slope of the bio-swale and a minimum of (5.0)-feet from any concrete slabs on grade or/ any pavement areas. Positive surface drainage should be provided around the building to direct surface water away from the foundations. To reduce the potential for water ponding adjacent to the building, we recommend the ground surface within a horizontal distance of five feet from the building slope down away from the building with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. As mentioned above, Infiltration basins or/ bio-swales should not be placed within (10)-feet of the foundations. Because the subgrade soil consists predominantly of clay, it will have a relatively low permeability. If infiltration basins or bio-swales are planned, drains should be provided that direct the water away to an appropriate outlet.

16.2 STORMWATER TREATMENT DESIGN CONSIDERATIONS

If storm water treatment improvements, such as shallow bio-retention swales, basins or/ pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction. General Bioswale Design Guidelines are as follows:

- If possible, avoid placing bioswales or/ basins within (10.0) feet of the building perimeter foundation or/ within (5.0)-feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with a heavy-duty liner to reduce water infiltration into the surrounding expansive clays.
- Bioswales constructed within (5.0)-feet of proposed buildings may be within the foundation or/ from the exterior face of the footings zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the "foundation plane of influence" an imaginary 2:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.
- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

16.3 BIOSWALE CONSTRUCTIONS ADJACENT TO PAVEMENTS

If bioswales or/ bio-detention areas are located adjacent to proposed parking lots or/ exterior flatwork, we recommend that mitigation measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should have an offset from the vertical edge of a bioswale such that there is a greater than or/ a minimum of (2.0)-foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every one-foot of vertical bioswale depth, or/

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the “Retaining Walls” section of this report, or/ concrete curbs or edge restraint should be adequately keyed into the native soil or/ engineered fill material to reduce the potential for lateral movement of the curbs.

16.4 IRRIGATION AND LANDSCAPING LIMITATIONS

The use of water-intensive landscaping around the perimeter of the buildings should be avoided to reduce the amount of water introduced to the expansive clay subgrade. In addition, irrigation of landscaping around the buildings should be limited to drip or bubbler-type systems. The purpose of these recommendations is to avoid large differential moisture changes adjacent to the foundations, which has been known to cause large differential settlement over short horizontal distances in expansive soil, resulting in cracking of slabs and architectural damage.

Moderately to highly expansive native clay is expected to be present at or/ at the subgrade levels. For this condition, prior experience and industry literature indicate some species of high water-demand trees can induce ground surface settlement by drawing water from the expansive soil and causing it to shrink. Where these types of trees are planted adjacent to structures, the ground-surface settlement may result in damage to structure. This problem usually occurs ten or more years after project completion as the trees reach mature height.

To reduce the risk of tree-induced, ground-surface settlement, we recommend trees of the following genera shall not be planted within a horizontal distance from the building equal to the mature height of the tree such as Eucalyptus, Populus etc. This limited list does not include all genera that could induce ground-surface settlement. Therefore, the project landscape architect should exercise proper judgment in limiting other types or trees with similar properties in the vicinity or close proximity to the building foundation.

17.0 ADDITIONAL SERVICES

17.1 PLAN REVIEW, CONSTRUCTION OBSERVATION AND TESTING

All conclusions and recommendations presented in this report are contingent upon Advance Soil Technology, Inc. (AST) being retained to review the grading plans, prior to construction. The general contractor/grading contractor/sub-contractors shall comply with the recommendations of the geotechnical engineer at all times. Appropriate field adjustments will be made as deemed necessary during the construction phase of the project. Any unforeseen soil conditions encountered during the grading operation shall be immediately brought to the attention of the geotechnical engineer for recommendations and to minimize the chance of the grading work not being approved by the engineer. In addition to the above, we shall observe and perform compaction test as deemed necessary during the grading (earthwork) operation at the site. It is the responsibility of the owner or his representative to schedule the inspections for the purpose of documentation. Site preparation and grading, excavation, cutting and backfilling shall be carried out under the observation of the Geotechnical Engineer.

AST will perform appropriate field and laboratory tests to evaluate the suitability of the fill material, proper moisture content for compaction and the degree of compaction as needed per the requirements of this report. Any fill material that does not meet the specification requirements shall be removed or replaced and reworked until the requirements are completely satisfied. Grading, shaping, excavating, conditioning, backfilling and compacting procedures require approval of AST as they are performed in the field.

17.2 CONSTRUCTION ACCEPTANCE

A representative from AST shall be present during the entire grading operation, so that he can provide recommendations as deemed necessary during the construction phase of the project. Unobserved and unapproved grading work will not be accepted under any circumstances. The grading operation shall be performed under the supervision of the soil engineer and in accordance with the requirements of this report.

17.3 SEASONAL LIMITS

No fill material shall be placed, spread or rolled during unfavorable weather conditions. If the grading operation is interrupted due to heavy rain, fill operations shall not be resumed until field density/moisture test have been taken and indicate that the moisture content of the fill is as previously specified or approved/directed by the soil engineer.

17.4 UNUSUAL/WET CONDITIONS

In the event that any unusual conditions, not covered by the special provisions, are encountered during grading operations, the soil engineer shall be immediately notified for supplemental recommendations.

18.0 SUMMARY OF COMPACTION RECOMMENDATIONS

Compaction at the site of proposed improvements for earthwork activities shall be in conformance with the ASTM D1557 Standard and in accordance with the requirements reflected table below and they are as follows:

AREAS	COMPACTION REQUIREMENTS
GENERAL ENGINEERED FILL	Compact to a minimum of 95 percent compaction at a minimum of 2 percent over the optimum moisture content. Where fills are deeper than (5.0)-feet, the portion below (5)-feet should be compacted as per ASTM D1557 to a minimum of 95 percent relative compaction.
NON-EXPANSIVE FILL	Compact to a minimum of 95 percent compaction at near the optimum moisture content.
UNDERGROUND UTILITY TRENCHES	Compact to a minimum of 90 percent compaction at a minimum of 2 percent over the optimum moisture content with the exception of upper two (2.0)-feet, compacted to a minimum of 95% relative compaction.

AREAS	COMPACTION REQUIREMENTS
EXTERIOR FLATWORK	Compact to a minimum of 90 percent compaction at a minimum of 2 percent over the optimum moisture content. Where exterior flatwork is subjected to vehicular traffic; compact upper 12 inches of subgrade to a minimum of 95 percent relative compaction at a minimum of ±2 percent over the optimum moisture content. Compact base rock to a minimum of 95 percent relative compaction at or/ near the optimum moisture content.
PARKING & ACCESS DRIVEWAYS	Compact upper (12)-inches of subgrade to a minimum of 95 percent relative compaction at a minimum of ±2 percent over the optimum moisture content, Compact base-rock to a minimum of 95 percent relative compaction at or/ near optimum moisture content.
LANDSCAPE AREAS	Compact to 90 percent relative compaction with the exception of upper (12)-inches to 85% relative compaction as per ASTM D1557
GENERAL NOTES	➢ Depths are below finished subgrade elevation. ➢ All compacted surfaces should be firm, stable, and unyielding under compaction equipment. ➢ All compaction requirements refer to relative compaction as a percentage of the laboratory standard described by ASTM D1557. All lifts to be compacted shall be a maximum of (8)-inch loose thickness, unless otherwise recommended.

19.0 LIMITATIONS

The recommendations made in this report are based on the assumptions that subsurface soil and groundwater conditions do not deviate from those disclosed at the location of the cone penetration tests and exploratory borings drilled at this site. If any variations or/ undesirable conditions are encountered during construction, the effects of these conditions on the recommendations presented herein should be evaluated again and if necessary, provide supplemental recommendations as deemed necessary. The recommendations of this report have been provided for the sole use of our client Arista Networks/5200 Patrick Henry Drive, LLC for the proposed development to be located 5200 Patrick Henry Drive in Santa Clara, California as described above in this report.

In the performance of our professional services, AST, its employees, and its agents will comply with the standards of care and skill ordinarily exercised by members of our profession practicing in the same or similar localities. This report may not provide all of the subsurface information that may be needed by a contractor to construct the project. No warranty, either expressed or/ implied, is made or intended in connection with the work performed by us, or/ by the proposal for consulting or/ other services, or/ by the furnishing of oral or written reports or/ findings. We are responsible for this geotechnical report and the conclusions and recommendations presented within this report, which are based on data related only to the specific project and locations discussed herein.

This report is for the sole use of the Client and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than one year from the date of issuance of this report. The work performed was based on project information provided by Client. If Client does not retain AST to review any plans and specifications, including any revisions or modifications to the plans and specifications, AST assumes or/ has no responsibility for the suitability of our recommendations. In addition, if there are any changes in the field conditions or/ to the plans and specifications, Client must obtain written approval from AST that such changes do not affect our recommendations. Failure to do so will invalid the report and its recommendations in its entirety.

Do not over-rely on the construction recommendations included in this report. These recommendations are not final, because they were developed principally from AST's professional judgment and opinions. AST's recommendations can be finalized only by observing actual subsurface conditions revealed during the construction phase of the project.

AST cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation. Sufficient monitoring, testing and consultation by AST should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether the earthwork activities were completed in accordance with the recommendation reflected in the geotechnical report. Retaining AST for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. In the event conclusions or/ recommendations are made or/ provided by others, based on these data reflected in this report, then such conclusions and recommendations are not our responsibility unless we have been given an opportunity to review and concur with such conclusions and recommendation in writing.

20.0 REFERENCES

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Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

Appendix "A"

Plate 1	Site Location Map
Plate 2	Site Location Map – Aerial View
Plate 3	Topographic Map
Plate 4	Geologic Map
Plate 5	Historical Earthquake Map
Plate 6	Site Plan (CPT & Exploratory Boring Locations)
Plate 7	Geological Cross Section A-A'

Site Location Map

Advance Soil Technology, Inc.

Geological , Geotechnical, Environmental
Consulting & Construction Services

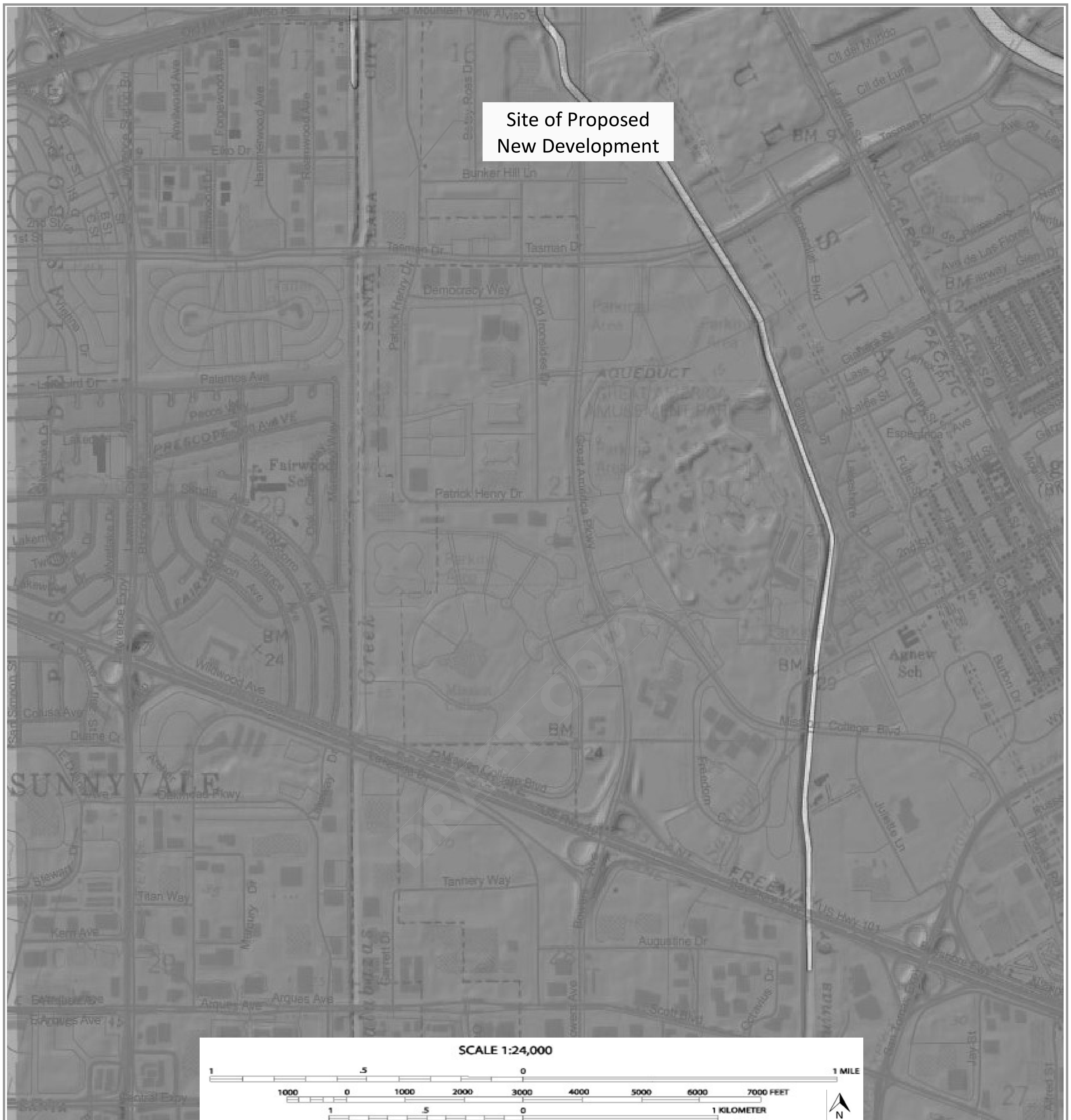
Geotechnical Study/Investigation

5200 PATRICK HENRY DRIVE

Four-Story Building | One-Level Below Grade Parking

343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 1



CGS Official Map - Milpitas | Dated September 2001

Seismic Hazard Zone Report 051 by Kevin B. Clahan, Elise Mattison, Anne Rosinski & Jacqueline Bott

Topographic | Liquefaction Map

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 2

Qhff - Holocene Alluvial Fan Deposits w/ fine grained facies
Qhfy - Latest Holocene Alluvial Fan Deposits
Qhly - Latest Holocene Alluvial Fan Levee Deposits
alf/ac - Artificial Stream Channel Deposits

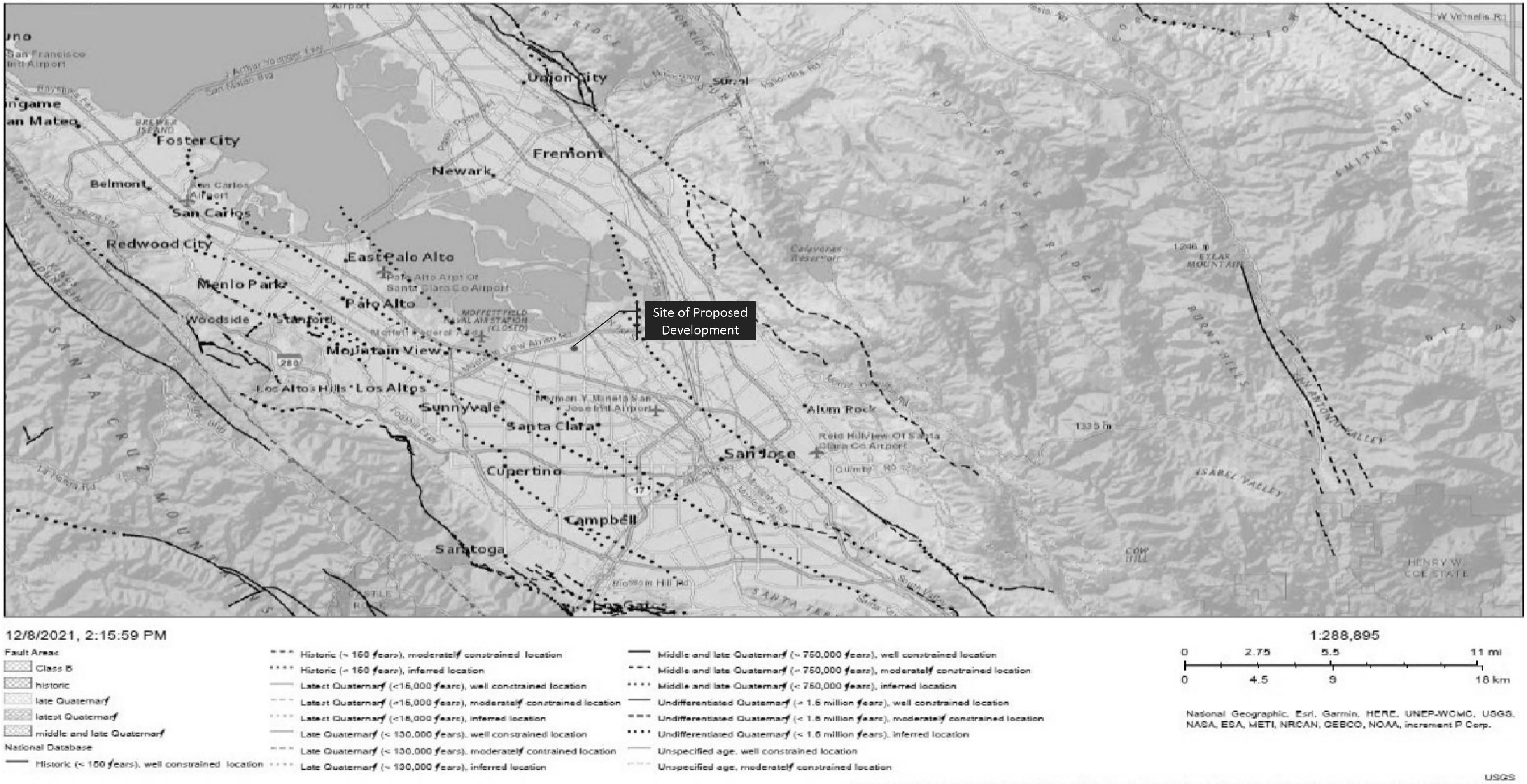
CGS official Map - Milpitas Quadrangle Map - Dated February 2001

Seismic Hazard Zone Report 051 by Kevin B. Clahan, Elise Mattison, Anne M. Rosinski & Jacqueline D.J. Bott

Geologic Map

Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue - San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 4



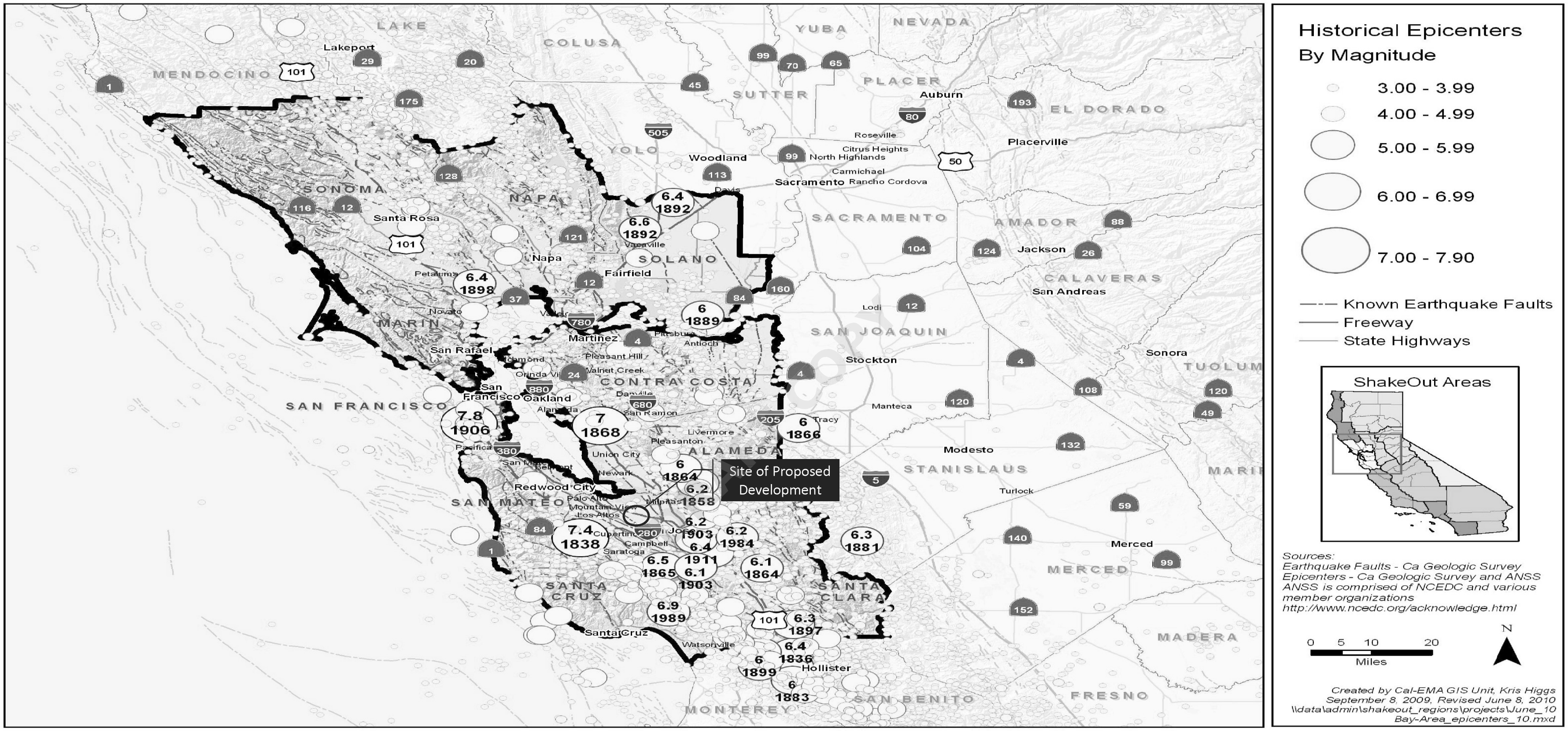
California Fault Map

AST	Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Investigation **5200 PATRICK HENRY DRIVE**	**5200 Patrick Henry Drive, LLC C/O ARISTA NETWORKS**
343 So. Baywood Avenue | San Jose, California		5200 Patrick Henry Drive | Santa Clara, California	5453 Great America Parkway, Suite 810 | Santa Clara, California
AST File No. 21164-S		Date: December 2021	Plate No. 4

Bay Area ~ ShakeOut Area
Historical Epicenters

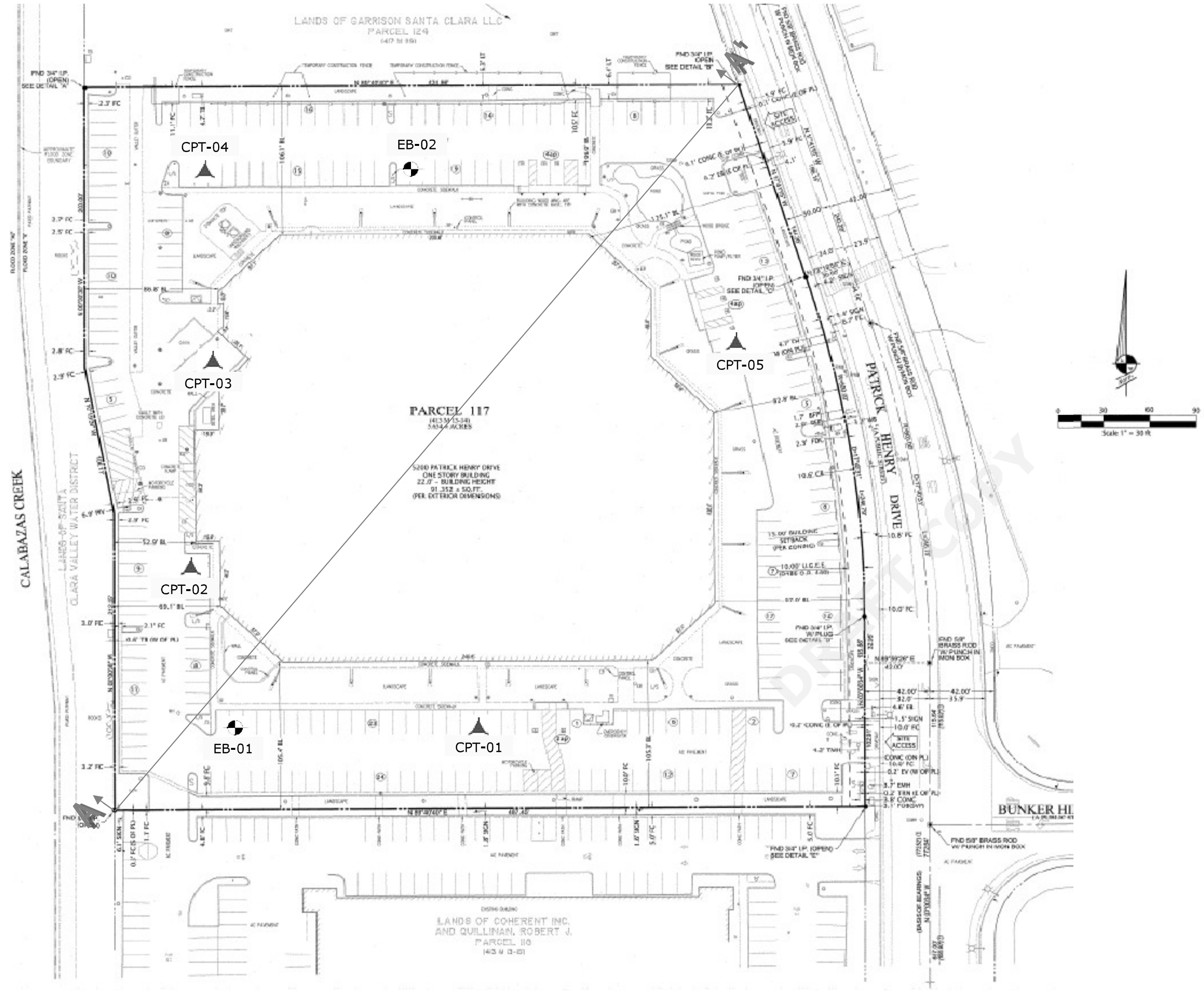
Earthquake Country Alliance
We're all in this together.

Cal EMA
California Emergency Management Agency



Historical Seismicity

AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Investigation **5200 PATRICK HENRY DRIVE**	**5200 Patrick Henry Drive, LLC C/O ARISTA NETWORKS**
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California	5453 Great America Parkway, Suite 810 | Santa Clara, California
AST File No. 21164-S	Date: December 2021	Plate No. 5



Project Notes:

- Indicates Approximate CPT Locations
- Indicates Approximate Boring Locations

Exploratory Boring Data

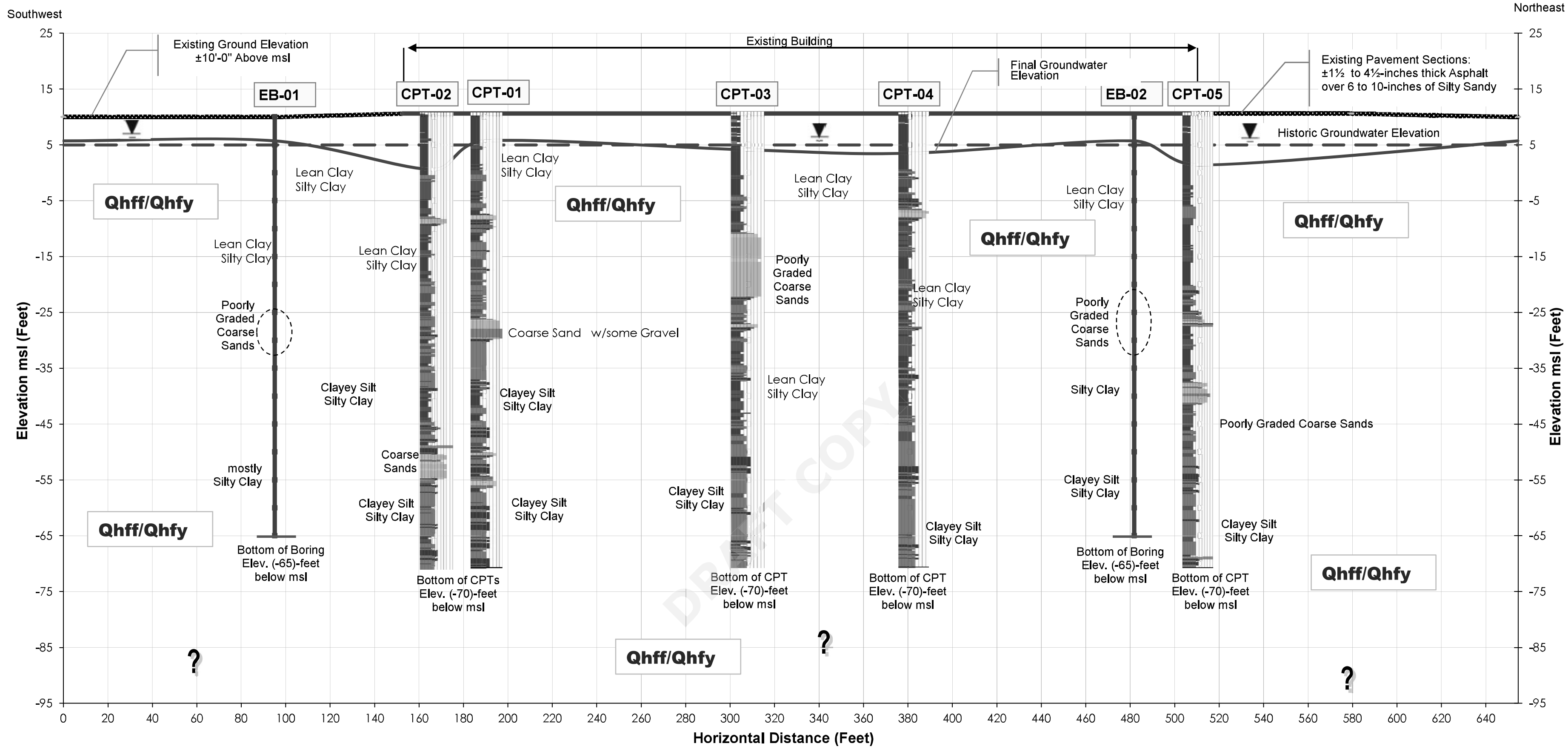
Boring No.	Depth	Type of Boring
EB-01	±75.0-feet	Hollow Stem | Geotechnical Exploratory Boring
EB-02	±75.0-feet	Hollow Stem | Geotechnical Exploratory Boring

Cone Penetration Data

CPT No.	Depth	Type of Cone Penetration Test
CPT No. 1	±80.22-feet	Geotechnical, Piezometric & Seismic
CPT No. 2	±80.22-feet	Geotechnical, Piezometric & Seismic
CPT No. 3	±80.38-feet	Geotechnical, Piezometric & Seismic
CPT No. 4	±80.22-feet	Geotechnical, Piezometric & Seismic
CPT No. 5	±80.22-feet	Geotechnical, Piezometric & Seismic

SITE PLAN (CPT & Boring Locations)

AST **Advance Soil Technology, Inc.** Geological, Geotechnical, Environmental Consulting & Construction Services	**Geotechnical Study/Investigation** Proposed New Four-Story Office Building | One-Level Below Grade Parking	**ARISTA NETWORKS**
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California	5453 Great America Parkway | Santa Clara, California
AST File No. 21164-S	Date: December 2021	Plate No. 6



Legend:

1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

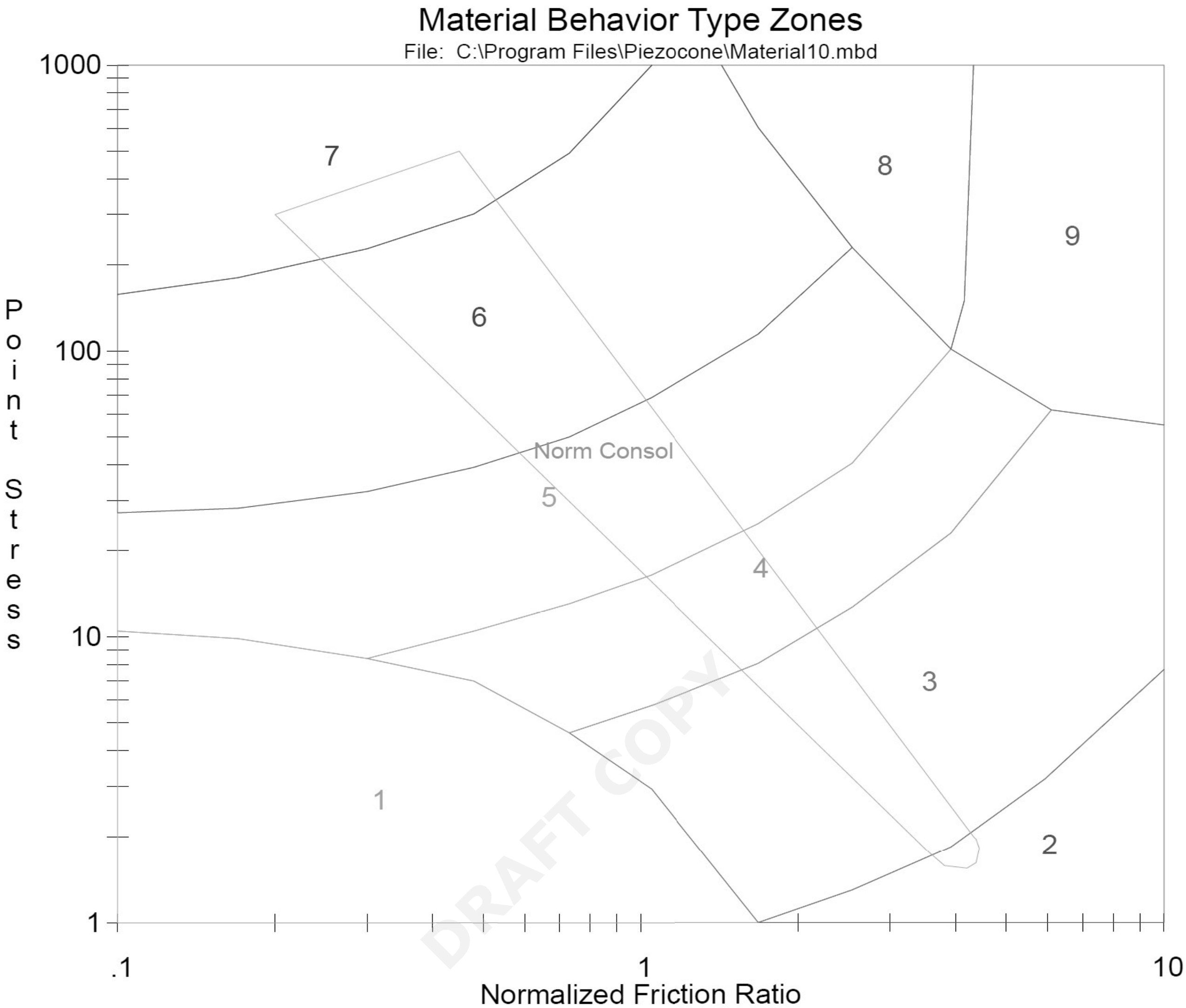
CPT-01	Proj. 147.6-Feet West-northwest	CPT-03	Proj. 146.4-Feet East-southeast	CPT-05	Proj. 112.2-Feet West-northwest	EB-02	Proj. 127.5-Feet East-southeast
CPT-02	Proj. 59.7-Feet East-southeast	CPT-04	Proj. 237-Feet East-southeast	EB-01	Proj. 27-Feet West-northwest	Qhc	Holocene Alluvial Fan Deposits

Qhff	Holocene Alluvial Fan Deposits -Fine Grained Facies
Qhfy	Latest Holocene Alluvial Fan Deposits
Qhff	Holocene Alluvial Fan Deposits -Fine Grained Facies
Qhf	Modern Stream Channel Deposits
alf/ac	Artificial Fill/Artificial Stream Channel Deposits

Idealized Geological Cross Section A-A'

AST **Advance Soil Technology, Inc.** Geological, Geotechnical, Environmental Consulting & Construction Services	**Geotechnical Study/Investigation** Proposed New Four-Story Office Building | One-Level Below Grade Parking	**ARISTA NETWORKS**
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California	5453 Great America Parkway | Santa Clara, California
AST File No. 21164-S	Date: December 2021	Plate No. 7

Appendix "B"



1. sensitive fine SOIL
2. Organic SOILS - Peats
3. silty CLAY to CLAY
4. clayy SILT to silty CLAY
5. silty SAND to sandy SILT
6. clean SAND to silty SAND
7. grvly SAND to dense SAND
8. stiff SAND to clayy SAND
9. very stiff fine SOIL

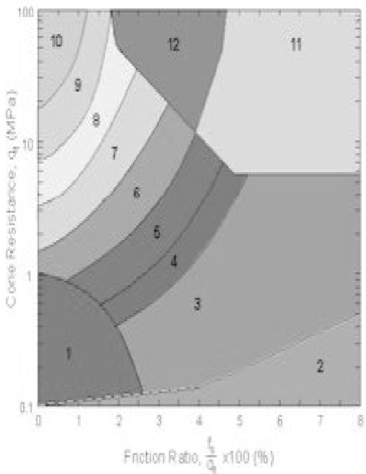
Material Behavior Zones

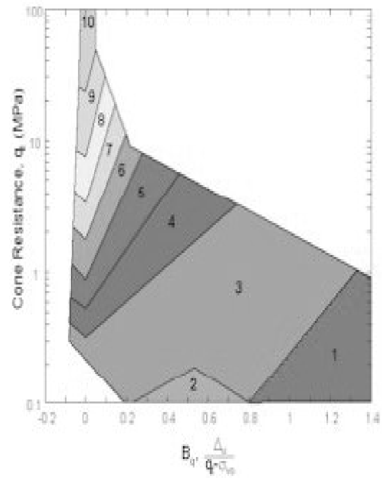
Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 8

CPT Soil Behavior Type Legend

Robertson et al. 1986

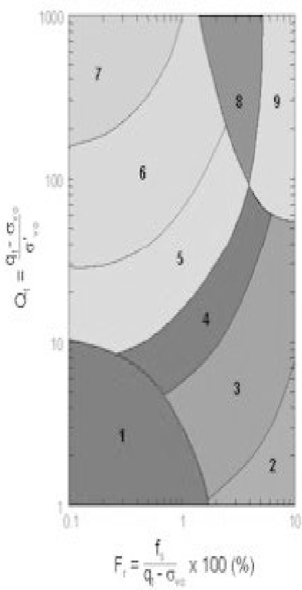


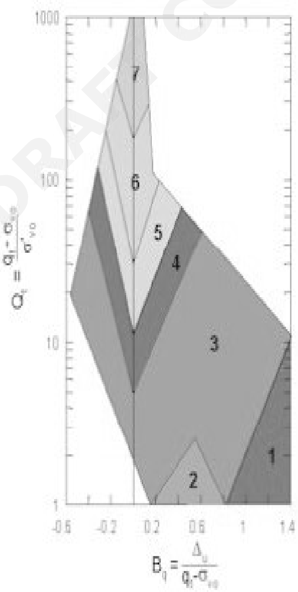


Zone	Soil Behavior Type
1	Sensitive, Fine Grained
2	Organic Material
3	Clay
4	Silty Clay to Clay
5	Clayey Silt to Silty Clay
6	Sandy Silt to Clayey Silt
7	Silty Sand to Sandy Silt
8	Sand to Silty Sand
9	Sand
10	Gravelly Sand to Sand
11	Very Stiff Fine Grained*
12	Sand to Clayey Sand*

*Overconsolidated or Cemented

Robertson et al. 1990





Zone	Soil Behavior Type
1	Sensitive, Fine Grained
2	Organic Soils-Peats
3	Clays; Clay to Silty Clay
4	Silt Mixtures; Clayey Silt to Silty Clay
5	Sand Mixtures; Silty Sand to Sandy Silt
6	Sands; Clean Sands to Silty Sands
7	Gravelly Sand to Sand
8	Very Stiff Sand to Clayey Sand*
9	Very Stiff Fine Grained*

*Overconsolidated or Cemented

Material Behavior Zones

AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 9

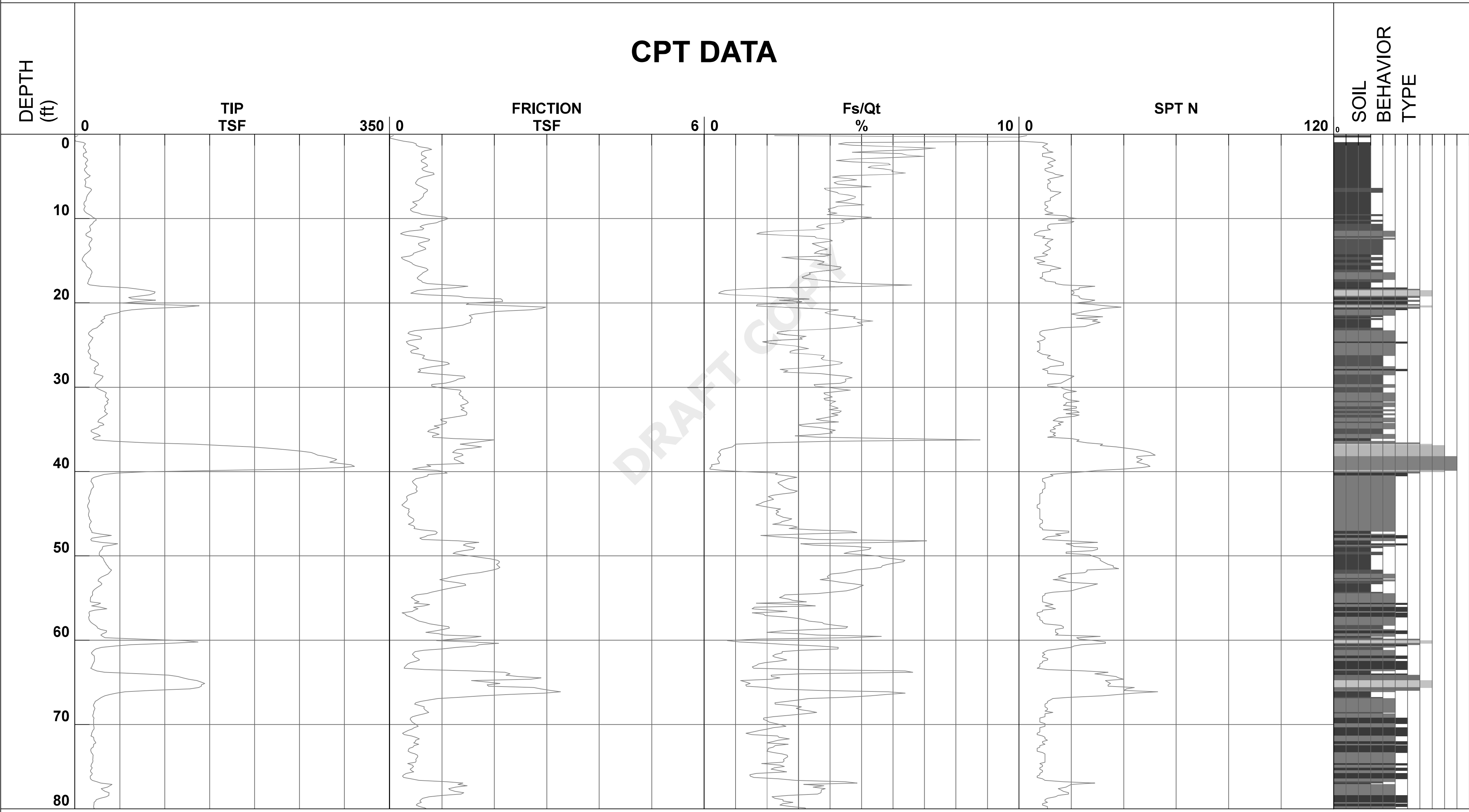
Advance Soil Technologies

Project	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY	Filename	SDF(554).cpt
Job Number	21164-S	Cone Number	DDG1589	GPS	
Hole Number	CPT-01	Date and Time	11/29/2021 8:43:09 AM	Maximum Depth	80.71 ft
EST GW Depth During Test	8.00 ft				

PLATE NO.: 10

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

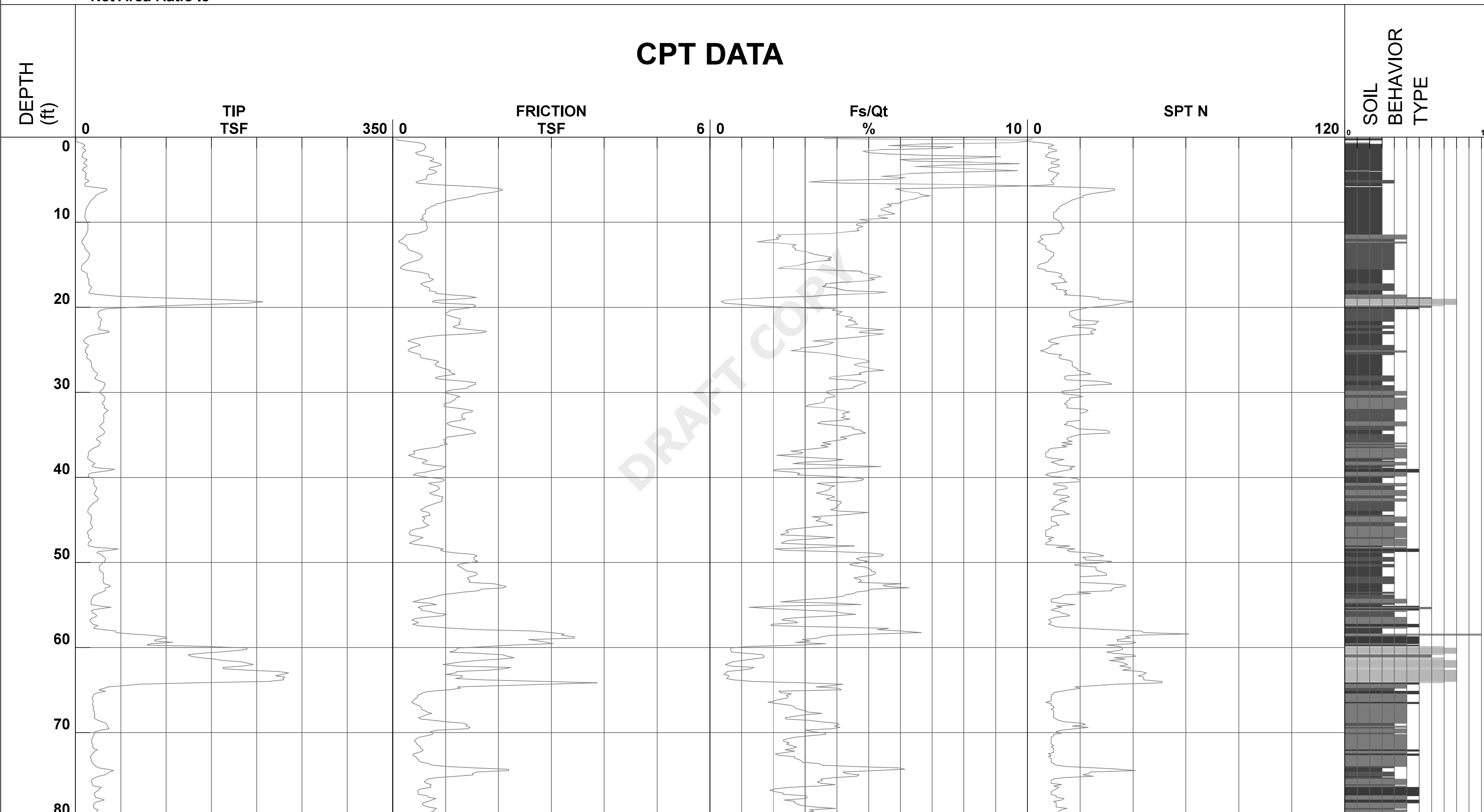
Advance Soil Technologies

Project	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY	Filename	SDF(555).cpt
Job Number	21164-S	Cone Number	DDG1589	GPS	
Hole Number	CPT-02	Date and Time	11/29/2021 11:25:32 AM	Maximum Depth	80.71 ft
EST GW Depth During Test			8.00 ft		

PLATE NO. 11

Net Area Ratio .8

CPT DATA



■ 1 - sensitive fine grained
■ 2 - organic material
■ 3 - clay
■ 4 - silty clay to clay
■ 5 - clayey silt to silty clay
■ 6 - sandy silt to clayey silt
■ 7 - silty sand to sandy silt
■ 8 - sand to silty sand
■ 9 - sand
■ 10 - gravelly sand to sand
■ 11 - very stiff fine grained (*)
■ 12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

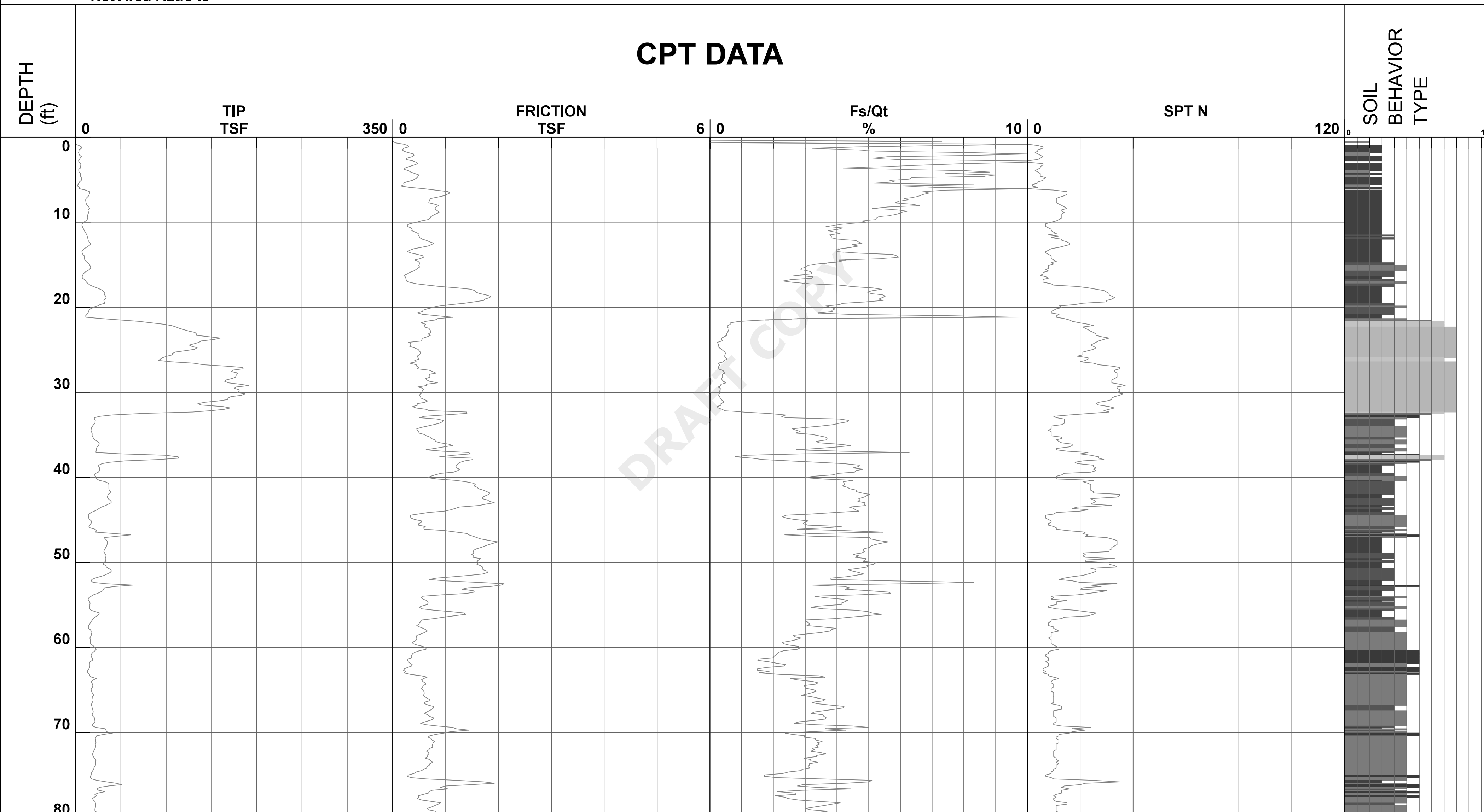
Advance Soil Technologies

Project	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY	Filename	SDF(556).cpt
Job Number	21164-S	Cone Number	DDG1589	GPS	
Hole Number	CPT-03	Date and Time	11/29/2021 1:27:50 PM	Maximum Depth	80.87 ft
EST GW Depth During Test	9.00 ft				

PLATE NO.: 12

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

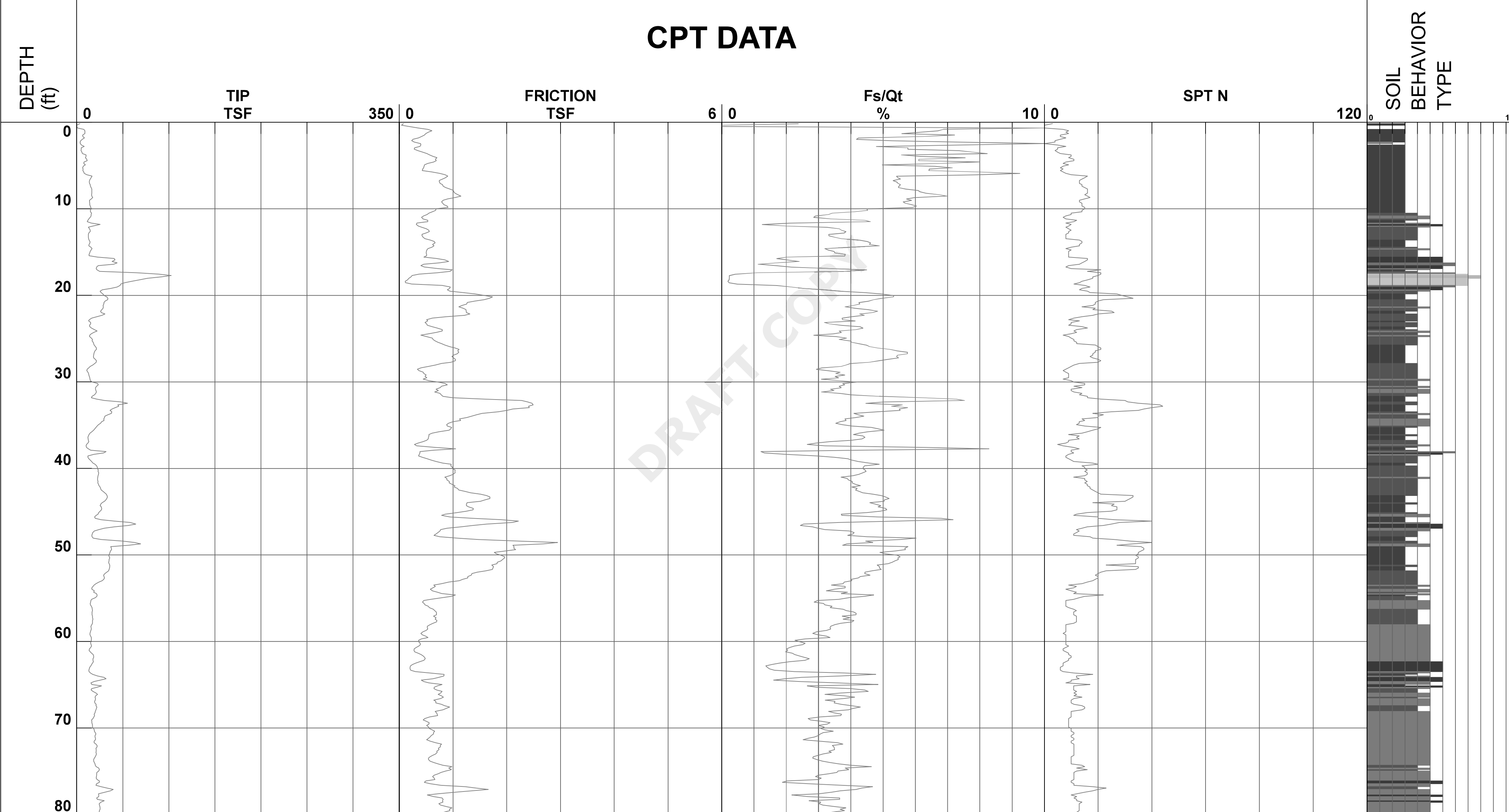
Advance Soil Technologies

Project	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY	Filename	SDF(557).cpt
Job Number	21164-S	Cone Number	DDG1589	GPS	
Hole Number	CPT-04	Date and Time	11/29/2021 3:00:27 PM	Maximum Depth	80.71 ft
EST GW Depth During Test	7.00 ft				

PLATE NO.: 13

Net Area Ratio .8

CPT DATA



1 - sensitive fine grained	4 - silty clay to clay	7 - silty sand to sandy silt	10 - gravelly sand to sand
2 - organic material	5 - clayey silt to silty clay	8 - sand to silty sand	11 - very stiff fine grained (*)
3 - clay	6 - sandy silt to clayey silt	9 - sand	12 - sand to clayey sand (*)

Cone Size 15cm squared

S*Soil behavior type and SPT based on data from UBC-1983

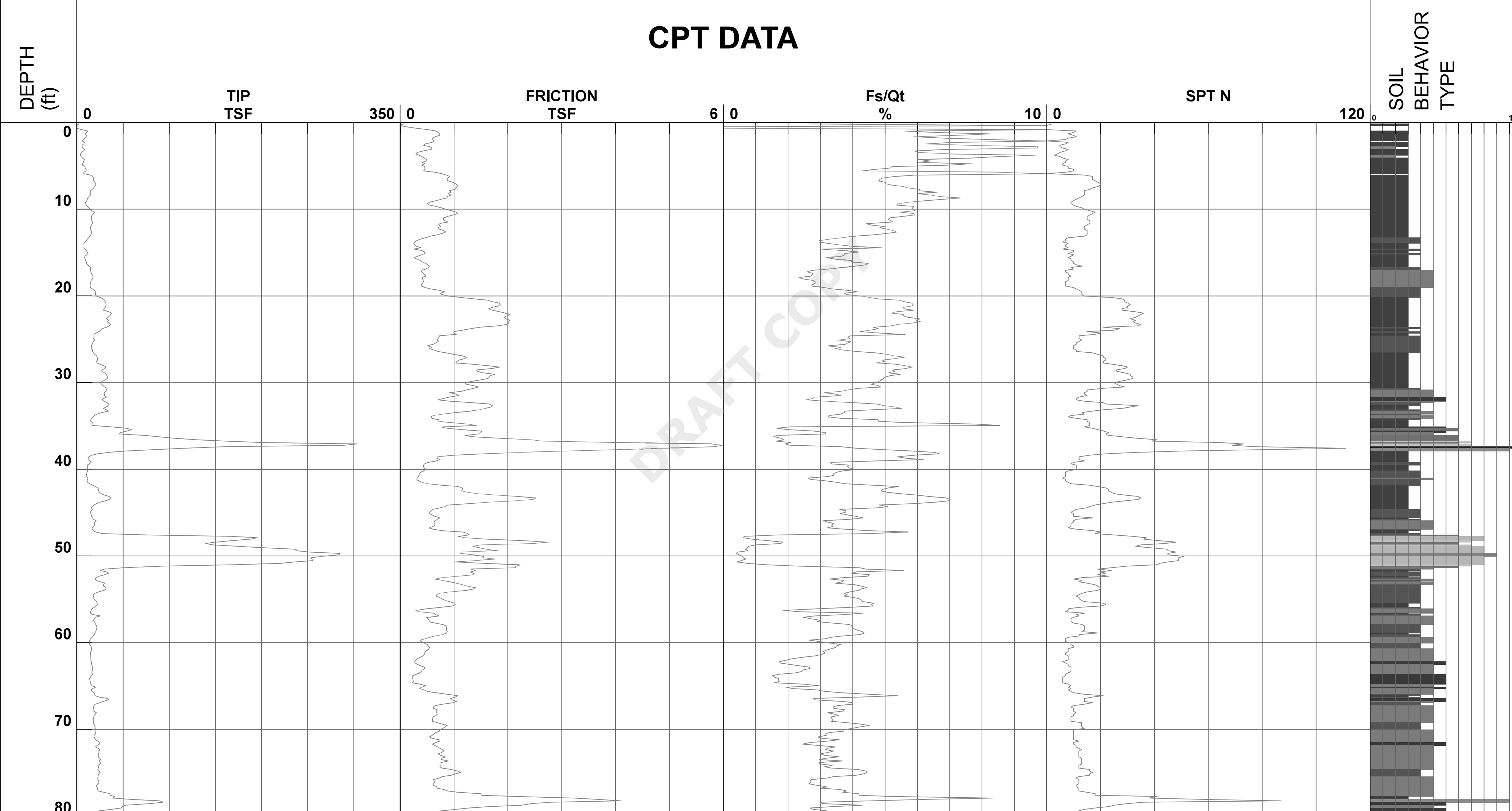
Advance Soil Technologies

Project	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY	Filename	SDF(558).cpt
Job Number	21164-S	Cone Number	DDG1589	GPS	
Hole Number	CPT-05	Date and Time	11/29/2021 4:29:43 PM	Maximum Depth	80.71 ft
EST GW Depth During Test		8.00 ft			

PLATE NO.: 14

Net Area Ratio .8

CPT DATA



- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey sand (*)

Cone Size 15cm squared

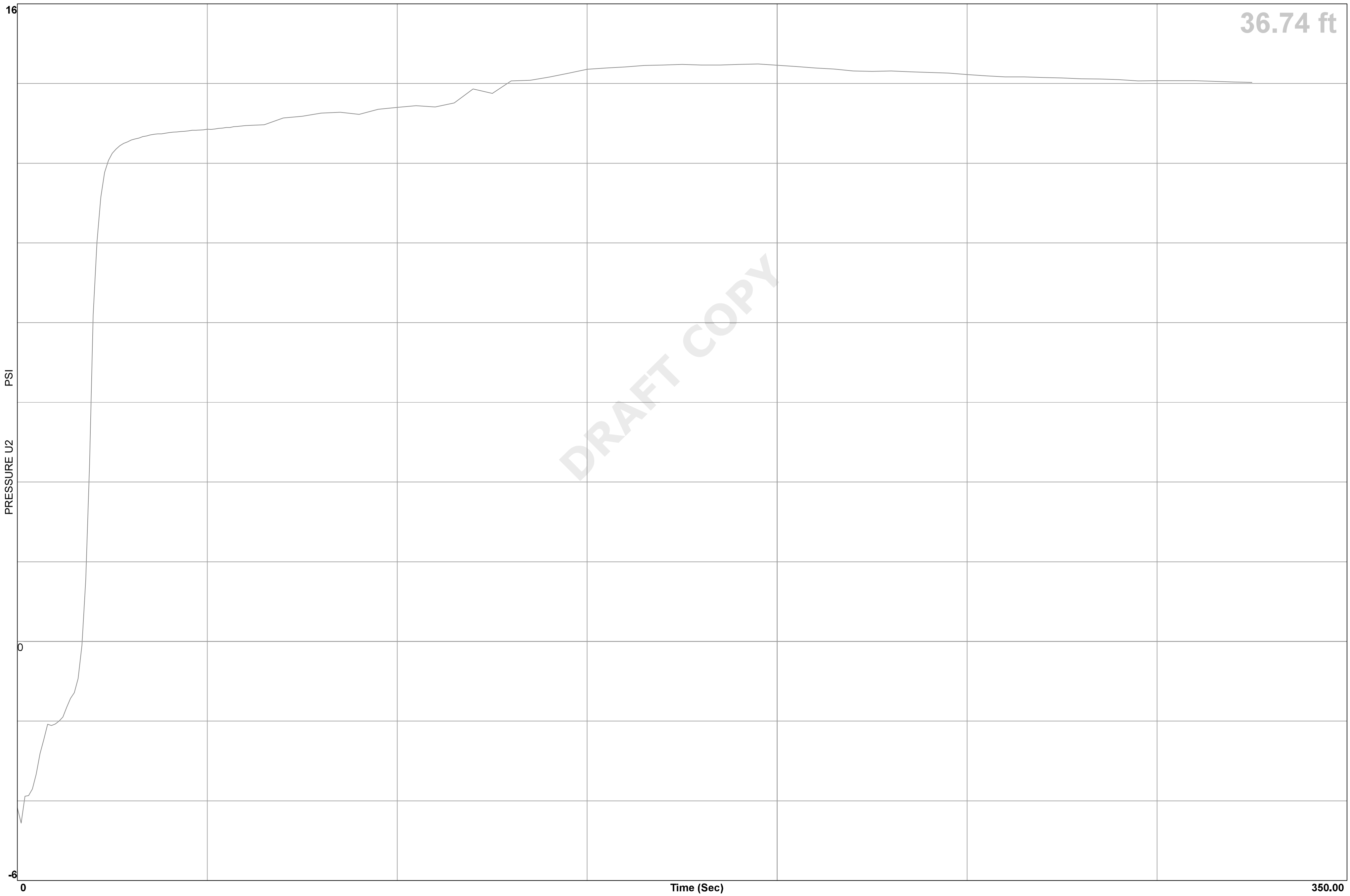
S*Soil behavior type and SPT based on data from UBC-1983

Advance Soil Technologies

Location	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY
Job Number	21164-S	Cone Number	DDG1589
Hole Number	CPT-01	Date and Time	11/29/2021 8:43:09 AM
Equilized Pressure	14.0	EST GW Depth During Test	4.3

GPS

PLATE NO.: 15

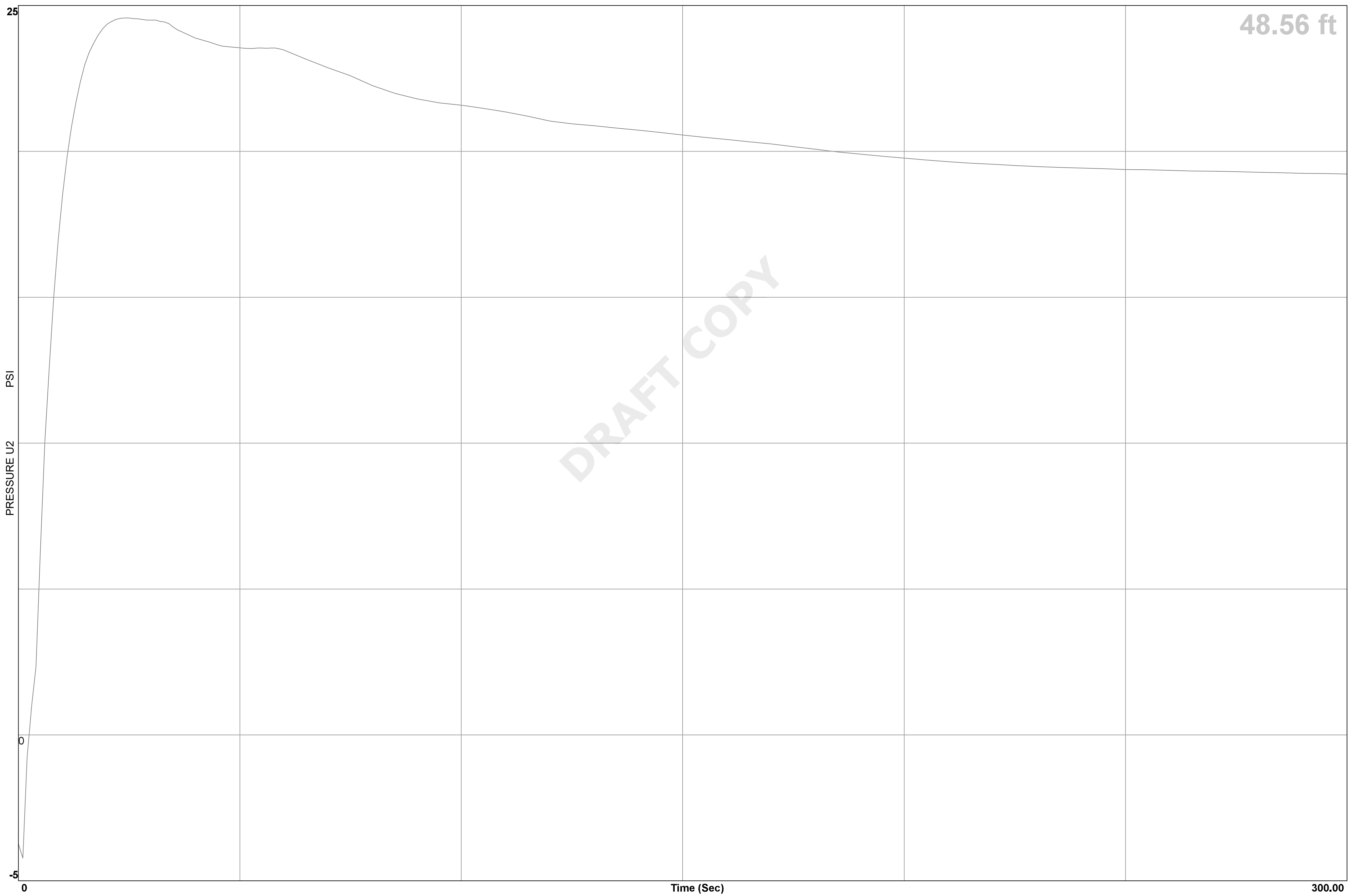


Advance Soil Technologies

Location	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY
Job Number	21164-S	Cone Number	DDG1589
Hole Number	CPT-01	Date and Time	11/29/2021 8:43:09 AM
Equilized Pressure	19.1	EST GW Depth During Test	4.2

GPS

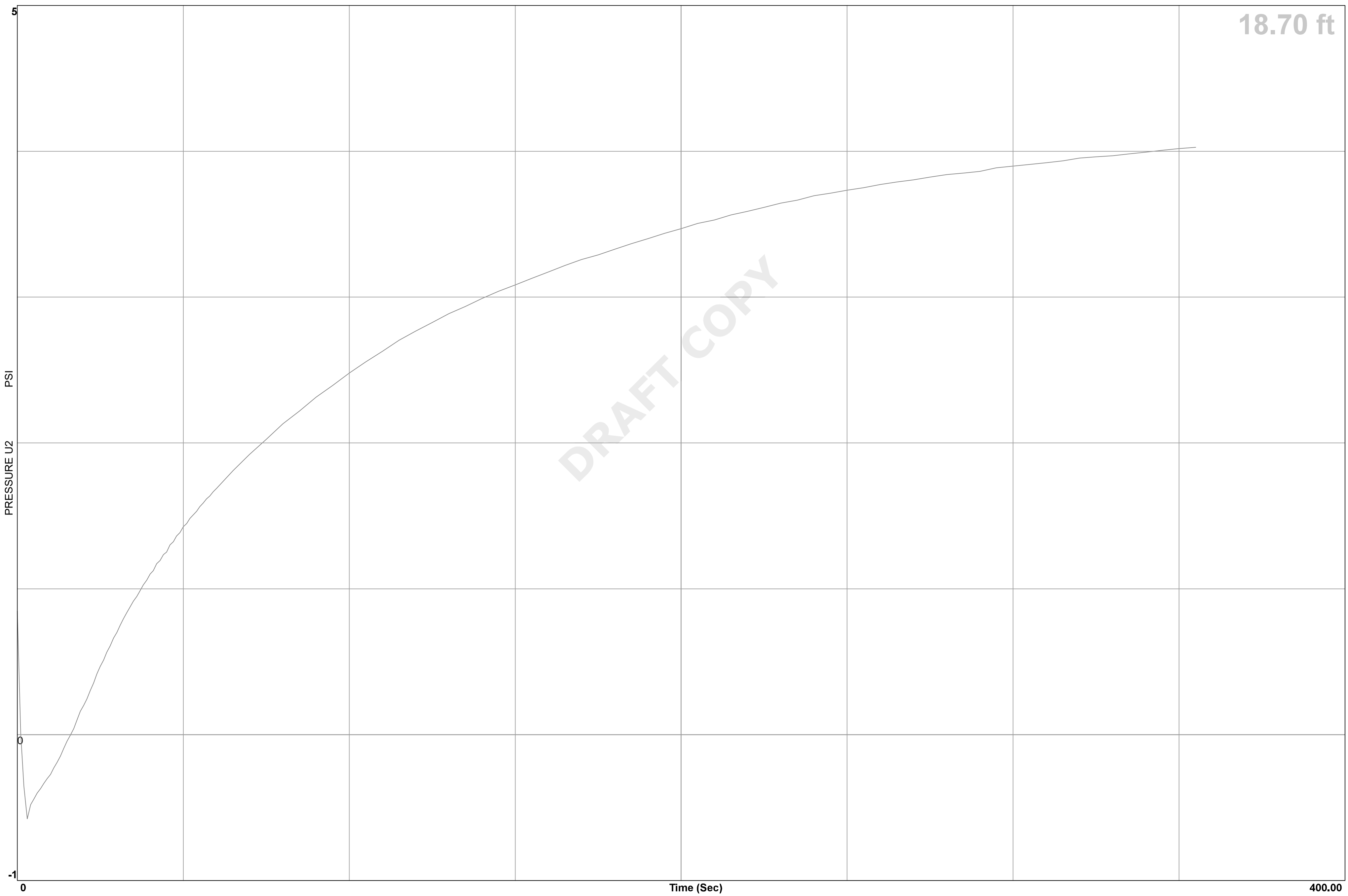
PLATE NO.: 16



Advance Soil Technologies

Location	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY		
Job Number	21164-S	Cone Number	DDG1589	GPS	
Hole Number	CPT-02	Date and Time	11/29/2021 11:25:32 AM		
Equilized Pressure	4.0	EST GW Depth During Test	9.4		

PLATE NO.: 17

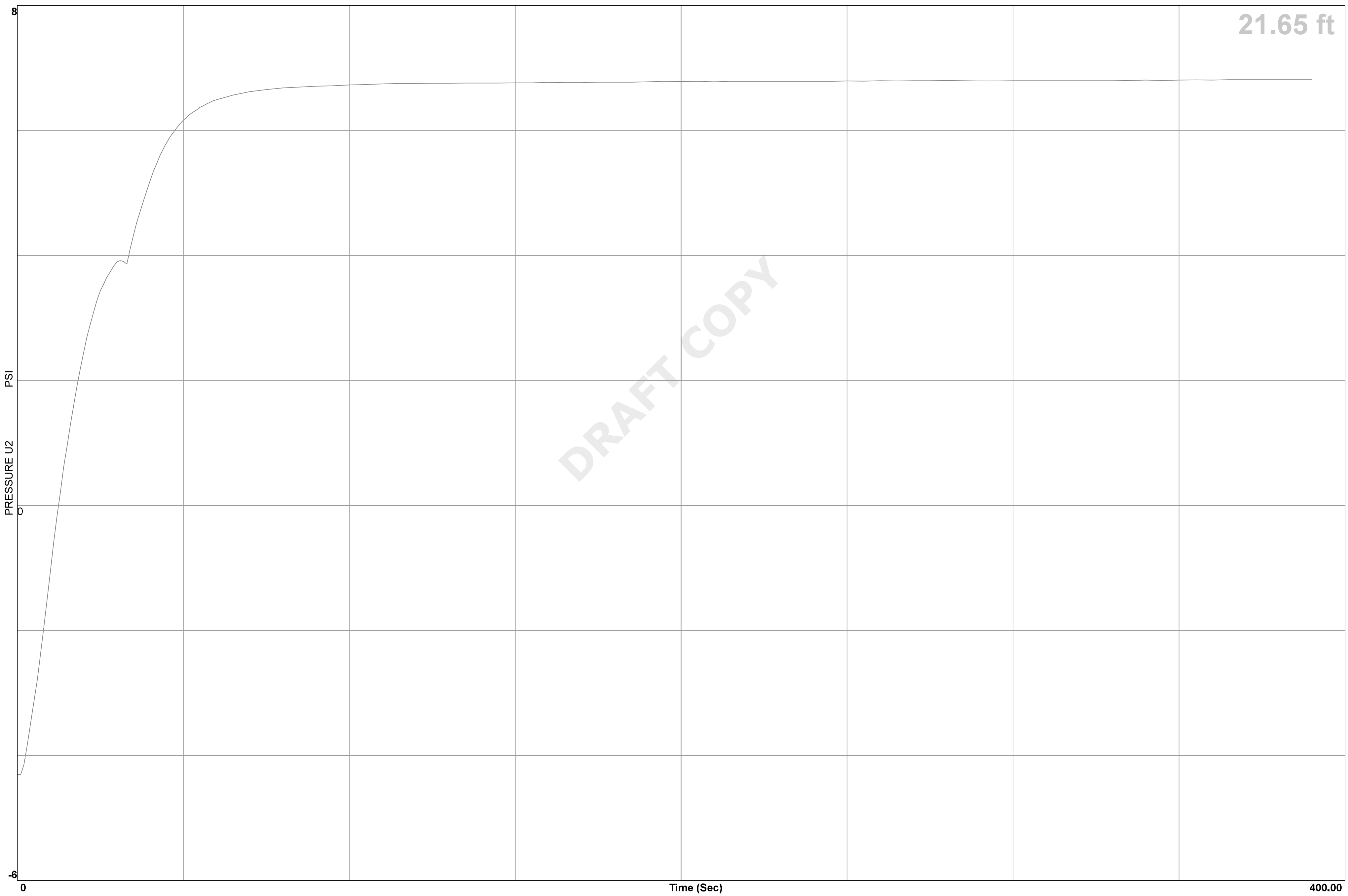


Advance Soil Technologies

Location	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY
Job Number	21164-S	Cone Number	DDG1589
Hole Number	CPT-03	Date and Time	11/29/2021 1:27:50 PM
Equilized Pressure	6.7	EST GW Depth During Test	5.9

GPS

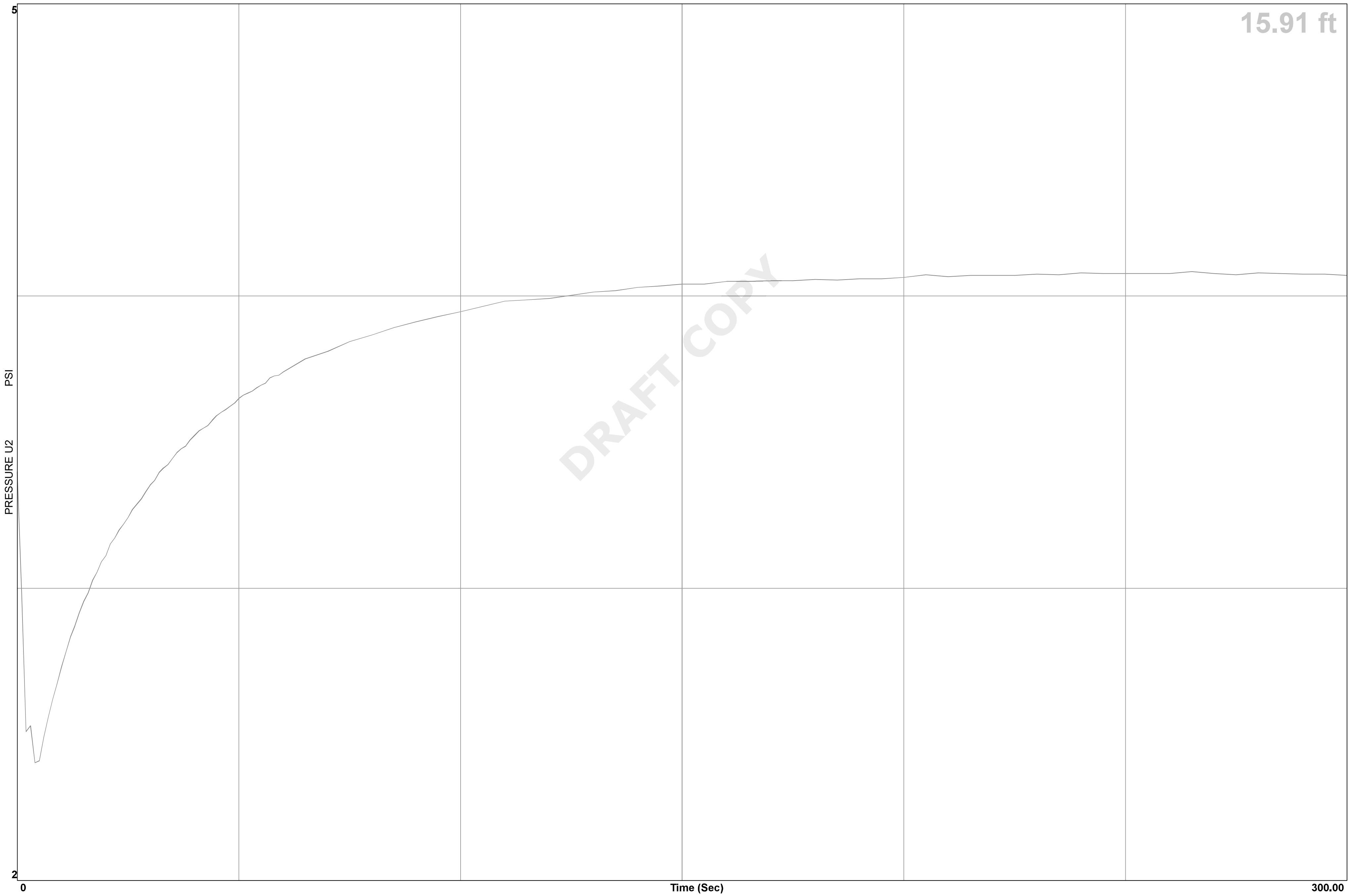
PLATE NO.: 18



Advance Soil Technologies

Location	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY		
Job Number	21164-S	Cone Number	DDG1589	GPS	
Hole Number	CPT-04	Date and Time	11/29/2021 3:00:27 PM		
Equilized Pressure	4.0	EST GW Depth During Test	6.5		

PLATE NO.: 19



Advance Soil Technologies

Location	5200 PATRICK HENRY DRIVE LLC	Operator	JM-IY	GPS	
Job Number	21164-S	Cone Number	DDG1589		
Hole Number	CPT-05	Date and Time	11/29/2021 4:29:43 PM		
Equilized Pressure	11.6	EST GW Depth During Test	8.6		

PLATE NO.: 20

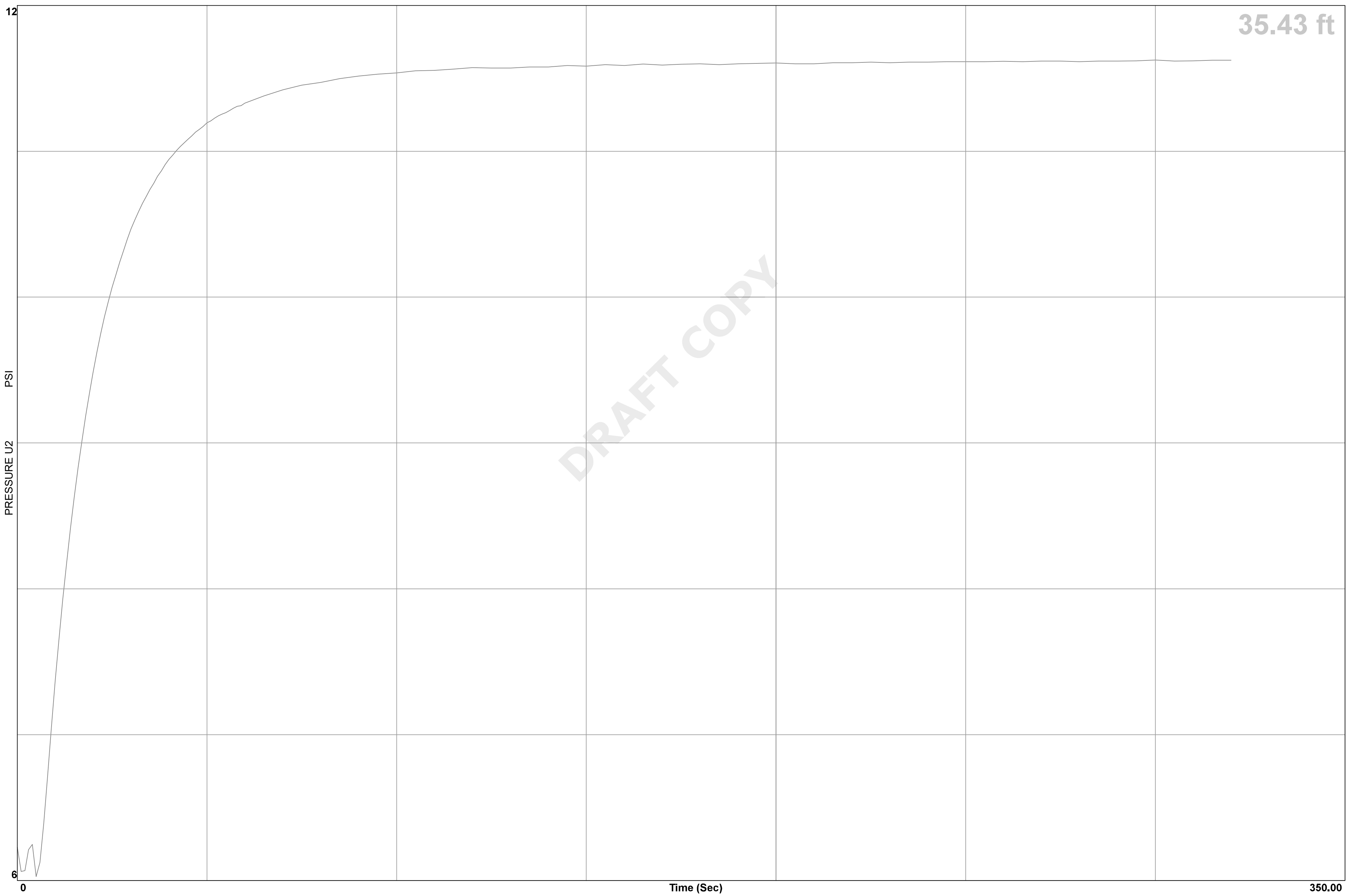
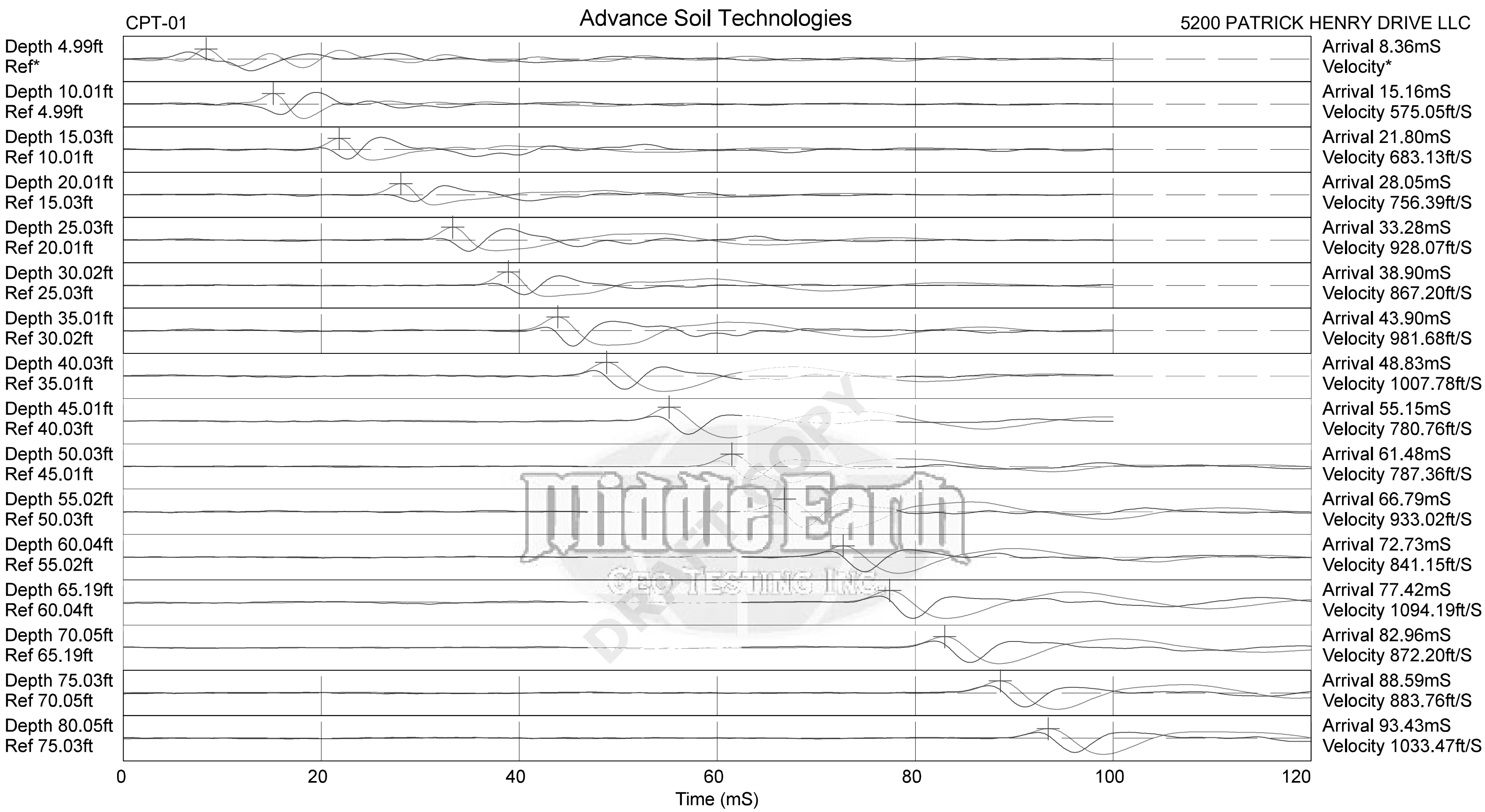


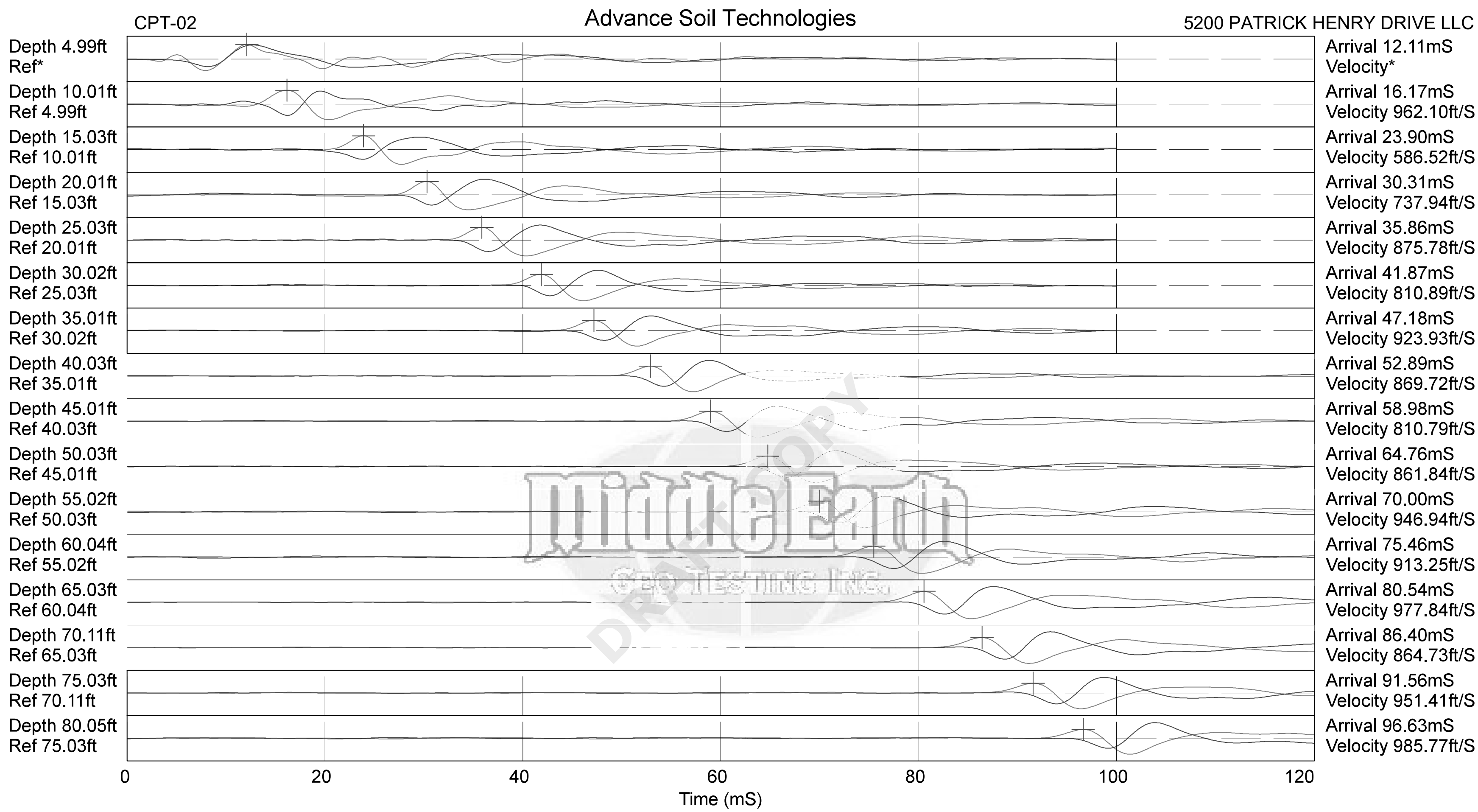
PLATE NO.: 21



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

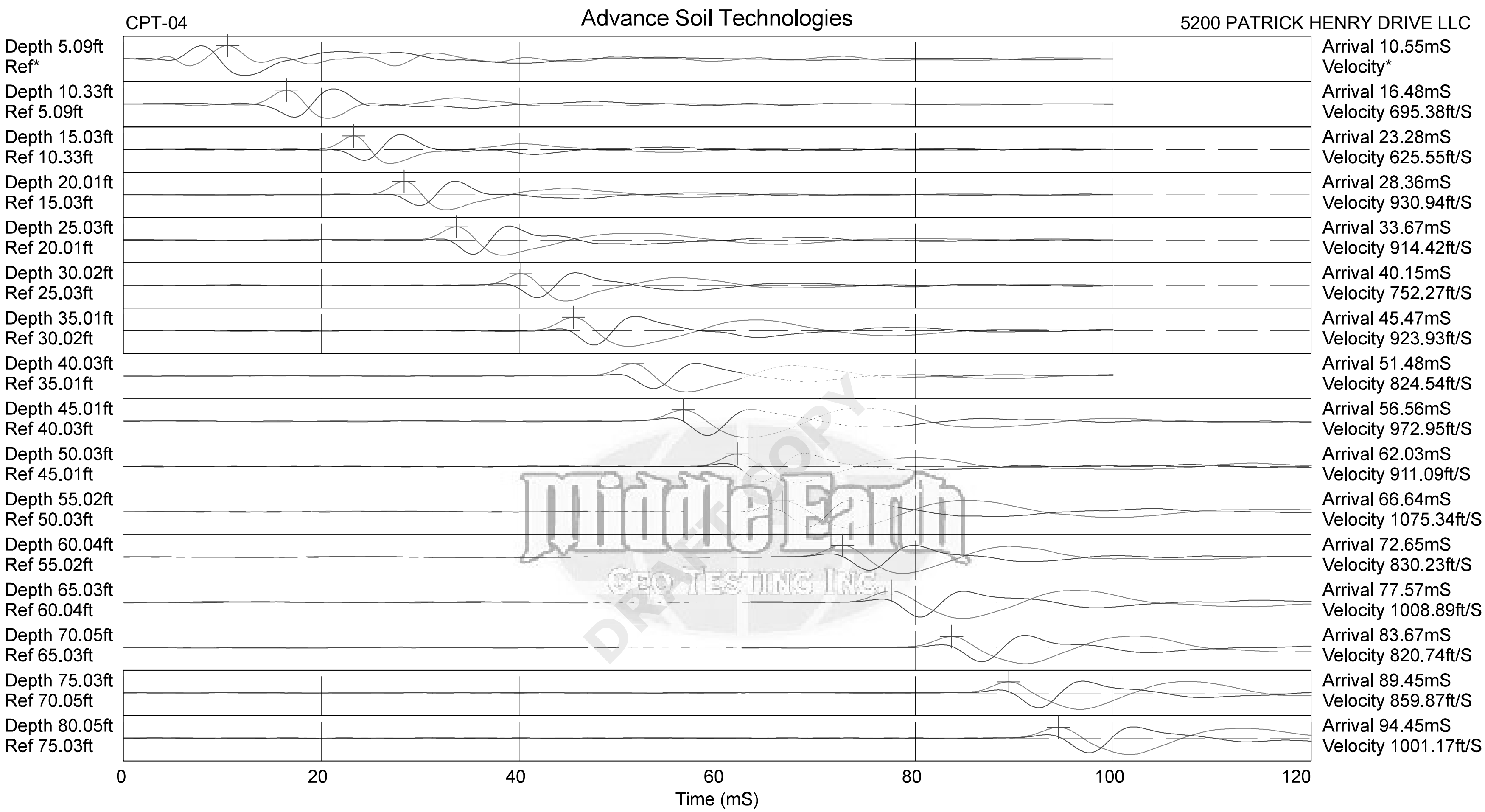
PLATE NO.: 22



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

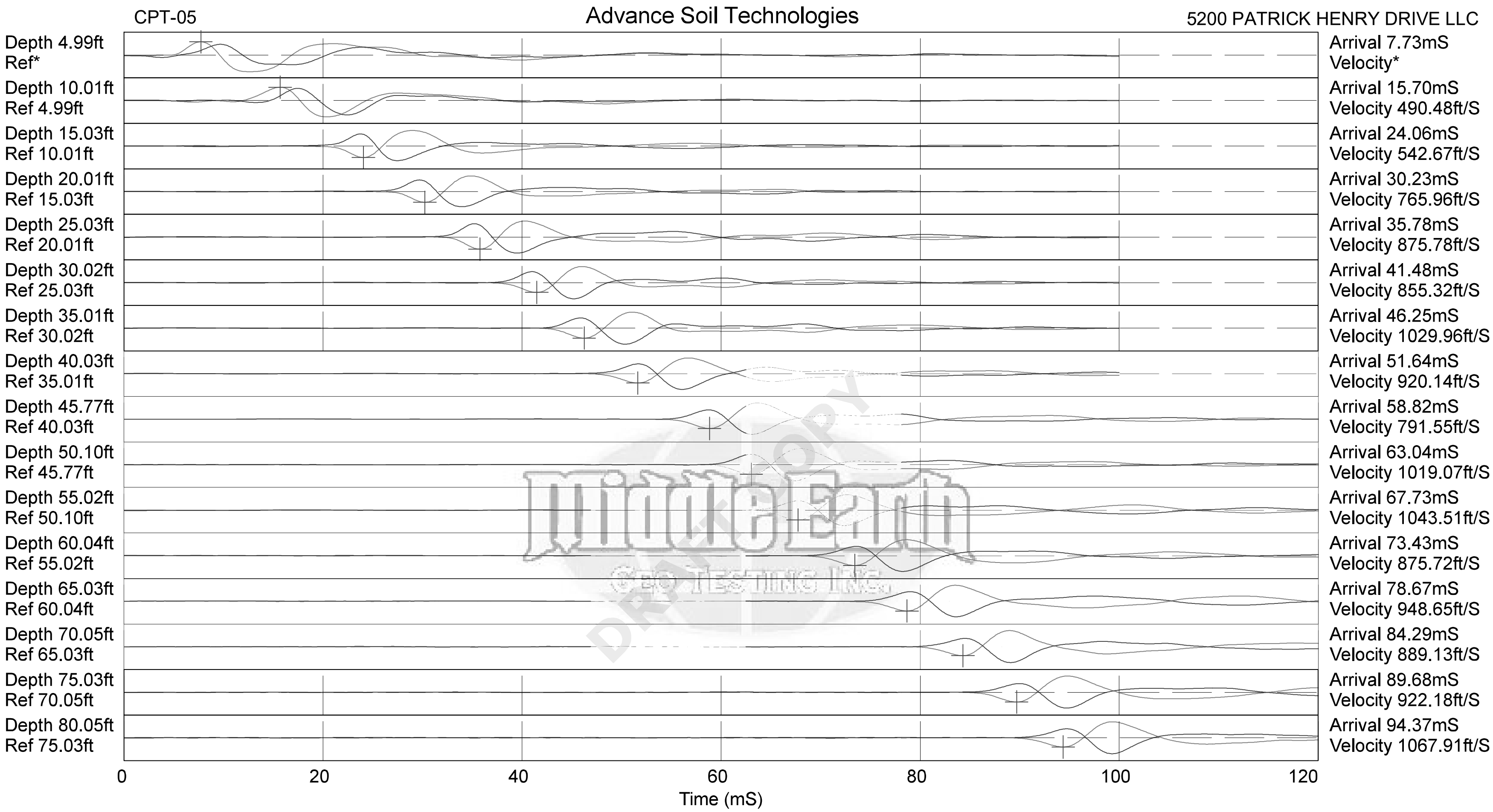
PLATE NO.: 23



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

PLATE NO.: 24



Hammer to Rod String Distance (ft): 5.83
* = Not Determined

COMMENT:

COARSE-GRAINED SOILS

LESS THAN 50% Fines

GROUP SYMBOLS	ILLUSTRATED GROUP NAMES	MAJOR DIVISIONS
GW	Well graded gravel Well graded gravel with sand	GRAVELS More than Half of coarse fraction is larger than No. 4 sieve size
GP	Poorly graded gravel Poorly graded gravel with sand	
GM	Silty gravel Silty gravel with sand	
GC	Clayey gravel Clayey gravel with sand	
SW	Well graded Sand Well graded sand with gravel	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
SP	Poorly graded sand Poorly graded sand with gravel	
SM	Silty Sand Silty Sand with gravel	
SC	Clayey sand Clayey sand with gravel	

Note: Coarse grained soils receive dual symbols if:
* The fines are CL-ML or GC-GM or SC-SM
* If they contain 5-12% fines

FINE-GRAINED SOILS

MORE THAN 50% FINES*

GROUP SYMBOLS	ILLUSTRATED GROUP NAMES	MAJOR DIVISIONS
CL	Lean clay Sandy clay with gravel	SILTS & CLAYS liquid limit less than 50
ML	Silt Sandy, Clayey silt with fine sand	
OL	Organic clay Sandy organic clay with gravel	
CH	Clayey gravel Clayey gravel with sand	SILTS & CLAYS liquid limit more than 50
MH	Elastic Silt Sandy elastic silt with gravel	
OH	Organic clay Sandy organic clay with gravel	
PT	Peat Highly organic silt	HIGHLY ORGANIC SOILS

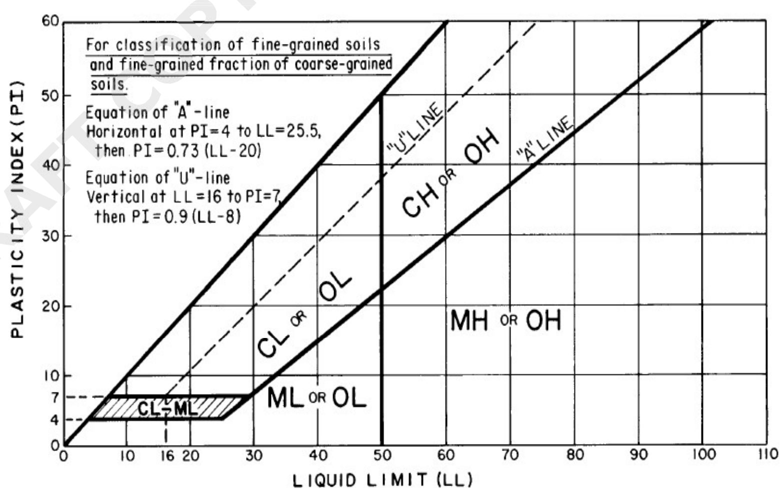
Note: Fine Grained soils will receive dual symbols if their limits in the hatches zone on the Plasticity Chart

SOIL SIZES

COMPONENT	SIZE RANGE
BOULDERS	Above 12 in.
COBBLES	3 in. to 12 in.
GRAVEL	No. 4 to 3 in.
Coarse	3/4 in. to 3 in.
Fine	No. 4 to 3/4 in.
SAND	No. 200 to No. 4
Coarse	No. 10 to No. 4
Medium	No. 40 to No. 10
Fine	No. 200 to No. 40
FINES	Below No. 200

Note: Classification is based on the portion of a sample that passes the 3-inch sieve

PLASTICITY CHART



Reference: ASTM D2487-00. Standard Classification of Soils for Engineering purposes (Unified Soil Classification System)

General Notes: The tables list 30 out of a possible 110 group names, all of which are assigned to unique proportions of constituent soils. Flow Charts in ASTM D2487 aid assignment of the group names. Some general rules for fine grained soils are: less than 15% sand or gravel is not mentioned; 15% to 25% sand or gravel is termed "with sand or with gravel, and 30 to 49% sand or gravel is termed as sandy or gravelly. Some general rules for coarse-grained soils are uniformly-graded or gap-graded soils are poorly graded (SP or GP); 15% or more sand or gravel is termed "with sand" or "with gravel". 15% to 25% clay and silt is termed clayey and silty and any cobbles or boulders are termed "with cobbles" or "with boulders".

Unified Soil Classification System

Soil Types

Boulders	Particles of rock that will not pass a 12-inch screen
Cobbles	Particles of rock that will pass a 12-inch screen; but not a 3-inch sieve
Gravel	Particles of rock that will pass a 3-inch sieve; but not a #4 sieve
Sand	Particles of rock that will pass a #4 sieve; but not a #200 sieve
Silt	soil that will pass a #200 sieve; that is non-plastic or very slightly plastic, and that exhibits little or no strength when dry
Clay	soil that will pass a #200 sieve; that can be made to exhibit plasticity (putty-like properties) within a range of water contents, and that exhibits considerable strength, when dry

Measures of Consistency of Cohesive Soils (Clays)

Very Soft	N=0-1*	C=0-250 psf	Squeezes between fingers
Soft	N=2-4	C=250-500 psf	Easily molded by finger pressure
Medium Stiff	N=5-8	C=500-1000 psf	Molded by strong finger pressure
Stiff	N=9-15	C=1000-2000 psf	Dented by Strong finger pressure
Very Stiff	N=16-30	C=2000-4000 psf	Dented by slight finger pressure
Hard	N=30	C=4000 psf	Dented slightly by pencil point

*N=blows per foot in the Standard Penetration Test. In cohesive soils, with the 3-inch diameter ring sampler. 140-pound weight, divide the blow by 1.2 get N

Moisture & Density

Moisture Condition	an observation term, dry, moist, wet or saturated
Moisture Content	the weight of water in a sample divided by the weight of dry soil in the soil sample, expressed as a percentage
Dry Density	pounds of dry soil in a cubic foot

Measures of Relative Density of Granular Soils (Gravels, Sands and Silts)

Very Loose	N=0-4*	RD=0-30	Easily push a 1/2-inch Reinf Bar by hand
Loose	N=5-10	RD=30-50	Push a 1/2-inch Reinf Bar by hand
Medium Dense	N=11-30	RD=50-70	Easily drive a 1/2-inch Reinf Bar
Dense	N=31-50	RD=70-90	Drive a 1/2-inch Reinf Bar one-foot
Very Dense	N=50	RD=90-100	Drive a 1/2-inch Reinf Bar a few inches

*N=blows per foot in the Standard Penetration Test. In granular soils, with the 3-inch diameter ring sampler, 140-pound weight, divide the blow by 2 get N

Reference

*ASTM Designation D2487. Standard Classification of soils for Engineering Purposes
*Means R.E. and Parcher J.V. Physical Properties of Soils (1963)
*Terzaghi, Karl and Peck Ralph B. Soil Mechanics Engineering Practice (1967)
*Das B.M. (1994) Principles of Geotechnical Engineering
*Sivakugan N. Soil Classification (2000)

Soil Terminology

Sampler Symbols

- Split Spoon Sampler 3.0-inch O.D.
- Shelby Tube
- Grab Sample
- Standard Penetration Test Sampler (Terzaghi Sampler)
- Rock Core

Groundwater Symbols

- Indicates historic groundwater elevation
- Indicates final groundwater elevation
- Indicates initial groundwater elevation

msl - mean sea level
bgs - below ground surface

Geotechnical Abbreviations

*	- Stratifications shown on borings are approximate, transition between materials may be gradual
ppt	- Pocket Penetrometer Test (tsf)
UC	- Unconfined compressive strength ASTM D2166
TXUU	- Triaxial Compression test Unconsolidated Undrained ASTM D2850
TXCU	- Triaxial Compression test Consolidated Undrained ASTM D4767
TXCD	- Triaxial Compression test Consolidated Drained ASTM (USACE)
consol	- Consolidation Test ASTM D2435
% Fines Content	- Passing Sieve #200
PI	- Plasticity Index
LL	- Liquid Limit
PL	- Plastic Limit
Blows/foot	- No. of blows per foot of the driven sampler
SPT	- N value, Standard Penetration Test
C	- Cohesion
Ø	- Friction Angle

Notes & Abbreviations

AST
AST
Advance Soil Technology, Inc.
Geological , Geotechnical, Environmental
Consulting & Construction Services
343 So. Baywood Avenue I San Jose, California

Geotechnical Study/Investigation
5200 PATRICK HENRY DRIVE
Four-Story Building | One-Level Below Grade Parking
5200 Patrick Henry Drive | Santa Clara, California

	Date: December 09, 2021	Boring No.: EB-01
	AST Project No. 21164-S	Plate No. 28
	Client: Arista Networks | 5200 Patrick Henry Drive, LLC	Driller: Exploration Geo-Services Inc.
	Location: 5200 Patrick Henry Drive, Santa Clara | California	Drilling Method: Hollow Stem Auger
	Indicates Historic Groundwater Elevation	Sampler: Modified California & SPT
	▼ Indicates Final Groundwater Elevation	Boring Diameter (in): 8.0
	▽ Indicates Initial Groundwater Elevation	Boring Depth: (75)-feet below existing grade

Elevation (feet)	Depth (feet)	Sample Description & Soil Type	Sample No.	Sample	Penetration Resistance	SPT N Blows/Foot	Liquid Limit	Plasticity Index	% Fines (Passing # 200)	Dry Density (pcf)	(%) Water Content	Ultimate Friction Angle	Ultimate Cohesion (psf)	Remarks
±10	0	Pavement: (±3.0)-inch of AC over (±9.0) to (±10.0)-inches of silty sandy gravel, moist, damp ….												
	1													
	2	Lean Fat Clay (CL) Black to Dark Brown with traces of sand, trace organics, moist, stiff high plasticity: Liquid Limit=53, Plastic Limit=24	1-1		GB-1		53	24		----	22.1			PI
	3													
	4													
▼ ±5	5	Lean Clay (CL) Medium to Tan Brown with grayish mottling, trace organics, rust stains, moist, medium stiff high plasticity: Liquid Limit=47, Plastic Limit=22	1-2		4 6 7	8	47	22	26.8	108.0	18.6			PI
	6													
	7													
	8													
	9													
±0 ▽	10	Lean Clay (CL) Medium to Tan Brown with grayish mottling, rust stains moist, stiff	1-3		7 8 9	10				104.8	24.9			
	11													
	12													
	13													
	14													
-5	15	Lean Clay (CL) Medium to Tan Brown with grayish mottling, organics, rust stains, moist, stiff	1-4		7 8 8	10				108.3	17.8	24.4	150.0	DS/CU
	16													
	17													
	18													
	19	Sandy Clay (CL) Medium Brown with fine sand, moist, stiff												
-10	20		1-5		7 8 11	11				109.6	20.5			Consol
	21													
	22													
	23													
	24	Lean Clay with Sand (CL) Medium to Tan/Olive Brown with fine sand, moist, stiff												
-15	25		1-6		8 10 10	12				104.7	23.3			Consol
	26													
	27													

Date: December 09, 2021	Boring No.:	EB-01
AST Project No. 21164-S	Plate No.	29
Client: Arista Networks | 5200 Patrick Henry Drive, LLC	Driller: Exploration Geo-Services Inc.	
Location: 5200 Patrick Henry Drive, Santa Clara | California	Drilling Method: Hollow Stem Auger	
Indicates Historic Groundwater Elevation	Sampler: Modified California & SPT	
Indicates Final Groundwater Elevation	Boring Diameter (in): 8.0	
Indicates Initial Groundwater Elevation	Boring Depth: (75)-feet below existing grade	

Elevation (feet)	Depth (feet)	Sample Description & Soil Type	Sample No.	Sample	Penetration Resistance	SPT N Blows/Foot	Liquid Limit	Plasticity Index	% Fines (Passsing # 200)	Dry Density (pcf)	(%) Water Content	Ultimate Friction Angle	Ultimate Cohesion (psf)	Remarks
	28	Lean Clay with Sand (CL) Medium to Tan/Olive Brown with fine sand, moist, stiff												
	29	Changes to Lean Clay (CL) Olive Brown, moist, very stiff												
-20	30		1-7		14 24 24	29				114.9	19.4			
	31													
	32													
	33													
	34	Sandy Silty Clay to Silty Sand (CL-SM) Medium to Tan Brown, moist, very stiff												
-25	35		1-8		12 14 26	24				113.5	19.2			
	36	Poorly Graded Sand with Silt (SP-SM) medium dense, wet, gray, fine to coarse sand												
	37													
	38													
	39													
-30	40	medium dense (SP-SM)	1-9		12 14 14	28			5.5	SPT	17.7			
	41													
	42													
	43	Lean Clay with Sand (CL) Medium to Tan Brown, moist, very stiff, hard, gray, fine sand												
	44													
-35	45		1-10		24 24 30	32				105.1	18.3			
	46													
	47													
	48													
	49	Lean Clay (CL) Medium Brown with grayish mottling, moist, very stiff												
-40	50	moist, very stiff	1-11		14 15 18	20				104.3	25.3			
	51													
	52													
	53													
	54													
-45	55	moist, very stiff, hard	1-12		24 28 30	34				102.9	25.7			

AST		
	Date: December 09, 2021	Boring No.: EB-01
	AST Project No. 21164-S	Plate No. 30
	Client: Arista Networks | 5200 Patrick Henry Drive, LLC	Driller: Exploration Geo-Services Inc.
	Location: 5200 Patrick Henry Drive, Santa Clara | California	Drilling Method: Hollow Stem Auger
	Indicates Historic Groundwater Elevation	Sampler: Modified California & SPT
	Indicates Final Groundwater Elevation	Boring Diameter (in): 8.0
	Indicates Initial Groundwater Elevation	Boring Depth: (75)-feet below existing grade

Elevation (feet)	Depth (feet)	Sample Description & Soil Type	Sample No.	Sample	Penetration Resistance	SPT N Blows/Foot	Liquid Limit	Plasticity Index	% Fines (Passing # 200)	Dry Density (pcf)	(%) Water Content	Ultimate Friction Angle	Ultimate Cohesion (psf)	Remarks
	56													
	57													
	58													
	59				12									
			1-13		18	23				100.6	25			
-50	60	Lean Clay (CL) Medium to Tan Brown, moist, very stiff, gray, some fine sand,			20									
	61													
	62													
	63													
	64				16									
			1-14		19	24				98.7	28			
-55	65	Lean Clay (CL) Medium to Tan Brown, moist, very stiff, gray, some fine sand,			21									
	66													
	67													
	68													
	69				12									
			1-15		16	22				97.8	29			
-60	70	moist, very stiff (CL)			21									
	71													
	72													
	73													
	74				14									
			1-16		16	24				112.6	20			
-65	75	moist, very stiff (CL)			24									
		Exploratory Boring Terminated at a depth of (75)-feet below the												
	76	existing ground surface (bgs)												
	77													
	78													
	79													
	80													
	81													
	82													
	83													

AST	Date: December 09, 2021	Boring No.: EB-02
	AST Project No. 21164-S	Plate No. 31
	Client: Arista Networks | 5200 Patrick Henry Drive, LLC	Driller: Exploration Geo-Services Inc.
	Location: 5200 Patrick Henry Drive, Santa Clara | Californi	Drilling Method: Hollow Stem Auger
	▽ Indicates Historic Groundwater Elevation	Sampler: Modified California & SPT
	▼ Indicates Final Groundwater Elevation	Boring Diameter (in): 8.0
	▽ Indicates Initial Groundwater Elevation	Boring Depth: (75)-feet below existing grade

Elevation (feet)	Depth (feet)	Sample Description & Soil Type	Sample No.	Sample	Penetration Resistance	SPT N Blows/Foot	Liquid Limit	Plasticity Index	% Fines (Passing # 200)	Dry Density (pcf)	(%) Water Content	Ultimate Friction Angle	Ultimate Cohesion (psf)	Remarks
±10	0	Pavement: (±1½)-inch of AC over (±9.0) to (±10.0)-inches of silty sandy gravel, moist, damp ….												
	1													
	2	Lean Fat Clay (CL) Black to Dark Brown with traces of sand, trace organics, moist, stif high plasticity: Liquid Limit=51, Plastic Limit=25	2-1		GB-2		51	25		----	20.4			PI
	3													
	4													
▼	5	Lean Clay (CL) Medium to Tan Brown with grayish mottling, trace organics, rust stains, moist, stiff high plasticity: Liquid Limit=48, Plastic Limit=21	2-2		7 9 16	15	48	21		96.6	29.5			PI
	6													
	7													
	8													
	9		2-3		8 12 14	16				95.4	29.9			
±0	10	Lean Clay (CL) Medium to Tan Brown with grayish mottling, rust stains moist, stiff												
▽	11													
	12													
	13													
	14													
-5	15	Lean Clay (CL) Medium to Tan Brown with grayish mottling, organics, rust stains, moist, stiff	2-4		9 10 13	14				94.6	30.7	12.6	500.0	DS/CU
	16													
	17													
	18													
	19	Lean Clay/Silty Clay (CL) Medium to Tan/Olive Brown, moist, stiff	2-5		8 10 12	11				109.0	20.7			
-10	20													
	21													
	22													
	23													
	24	Lean Clay with Sand (CL) Medium to Tan/Olive Brown with fine sand, moist, very stiff	2-6		16 18 20	23				113.6	19.0			
-15	25													
	26													
	27													

AST		
	Date: December 09, 2021	Boring No.: EB-02
	AST Project No. 21164-S	Plate No. 32
	Client: Arista Networks | 5200 Patrick Henry Drive, LLC	Driller: Exploration Geo-Services Inc.
	Location: 5200 Patrick Henry Drive, Santa Clara | California	Drilling Method: Hollow Stem Auger
	Indicates Historic Groundwater Elevation	Sampler: Modified California & SPT
	Indicates Final Groundwater Elevation	Boring Diameter (in): 8.0
	Indicates Initial Groundwater Elevation	Boring Depth: (75)-feet below existing grade

Elevation (feet)	Depth (feet)	Sample Description & Soil Type	Sample No.	Sample	Penetration Resistance	SPT N Blows/Foot	Liquid Limit	Plasticity Index	% Fines (Passing # 200)	Dry Density (pcf)	(%) Water Content	Ultimate Friction Angle	Ultimate Cohesion (psf)	Remarks
	28													
	29	Clayey Sandy Silt/Sandy Silt (ML)			6									
		Medium brown with grayish mottling, rust stains, moist, stiff	2-7		8	11				107.8	23			
-20	30				10									
	31													
	32													
	33													
	34				11									
			2-8		17	22			48.4	112.4	24			
-25	35	moist, Very Stiff (ML)			20									
	36													
	37													
	38													
	39				10									
		Lean Clay with Sand (CL)	2-9		12	15				104.7	24			
-30	40	Medium to Tan Brown, moist, stiff, gray, fine sand			13									
	41													
	42													
	43													
	44				20									
			2-10		22	29				112.6	20			
-35	45	Lean Clay (CL)			26									
		Medium to Tan/Olive Brown, moist, very Stiff												
	46													
	47													
	48													
	49	Lean Clay (CL)			10									
		Medium Brown with grayish mottling, moist, very stiff	2-11		13	20				106.2	23			
-40	50				20									
	51													
	52													
	53													
	54				20									
			2-12		24	27				111.9	22			
-45	55	moist, very stiff			21									

AST		
Date: December 09, 2021	Boring No.:	EB-02
AST Project No. 21164-S	Plate No.	33
Client: Arista Networks | 5200 Patrick Henry Drive, LLC	Driller: Exploration Geo-Services Inc.	
Location: 5200 Patrick Henry Drive, Santa Clara | Californi	Drilling Method: Hollow Stem Auger	
▽ Indicates Historic Groundwater Elevation	Sampler: Modified California & SPT	
▼ Indicates Final Groundwater Elevation	Boring Diameter (in): 8.0	
▽ Indicates Initial Groundwater Elevation	Boring Depth: (75)-feet below existing grade	

Elevation (feet)	Depth (feet)	Sample Description & Soil Type	Sample No.	Sample	Penetration Resistance	SPT N Blows/Foot	Liquid Limit	Plasticity Index	% Fines (Passing # 200)	Dry Density (pcf)	(%) Water Content	Ultimate Friction Angle	Ultimate Cohesion (psf)	Remarks
	56													
	57													
	58													
	59				16									
-50	60	Lean Clay/Sandy Silty Clay (CL) Medium to Tan Brown, moist, very stiff, gray, some fine sand,	2-13		19 20	23				100.6	25			
	61													
	62													
	63													
	64				16									
-55	65	Lean Clay/Sandy Silty Clay (CL) Medium to Tan Brown, moist, very stiff, hard, grayish mottling, with some fine sand	2-14		26 28	32				98.7	28			
	66													
	67													
	68													
	69				12									
-60	70	moist, very stiff (CL)	2-15		15 20	21				97.8	29			
	71													
	72													
	73													
	74				16									
-65	75	moist, very stiff (CL)	2-16		19 23	25				112.6	20			
	76	Exploratory Boring Terminated at a depth of (75)-feet below the existing ground surface (bgs)												
	77													
	78													
	79													
	80													
	81													
	82													
	83													

Appendix "C"

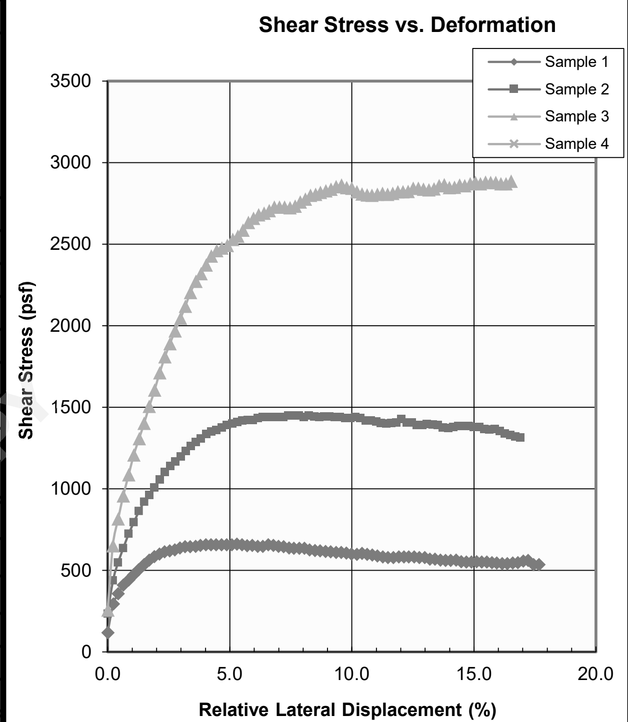
Consolidated Undrained Direct Shear (ASTM D3080M)

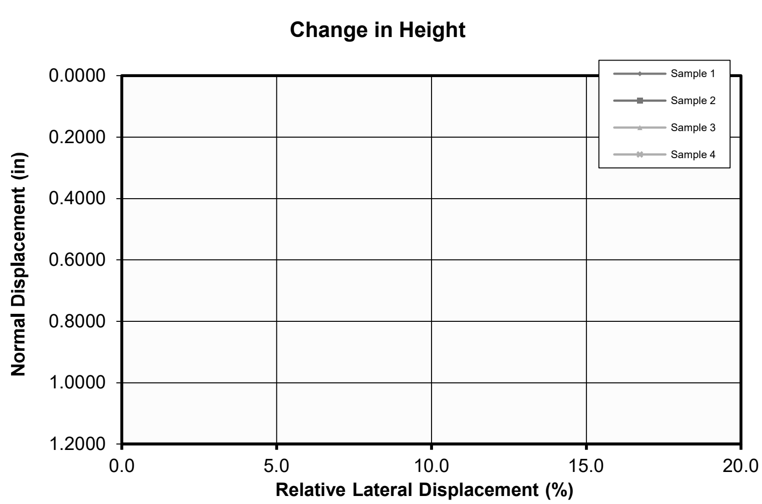
COOPER TESTING LABORATORY

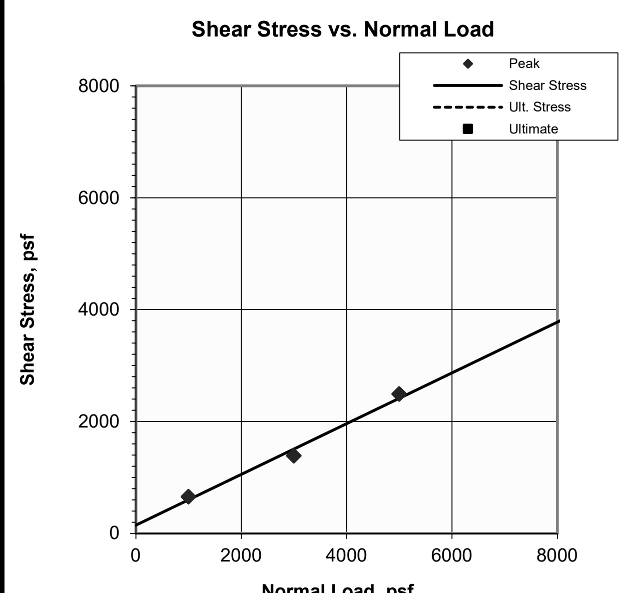
CTL Job #:	132-121	Project #:	21164-S	By:	MD
Client:	Advance Soil Technology	Date:	12/20/2021	Checked:	PJ
Project Name:	5200 Patrick Henry Drive, LLC	Remolding Info:			

Phi (deg)	24.4	Ult. Phi (deg)	
Cohesion (psf)	150	Ult. Cohesion (psf)	

Specimen Data	1	2	3	4
Boring:	EB-1	EB-1	EB-1	
Sample:	1-4	1-4	1-4	
Depth (ft):	15	15	15	
Visual Description:	Olive Brown CLAY w/ Sand	Olive Brown CLAY w/ Sand	Olive Brown CLAY w/ Sand	
Normal Load (psf)	1000	3000	5000	
Dry Mass of Specimen (g)	113.0	115.7	113.9	
Initial Height (in)	1.00	1.01	1.00	
Initial Diameter (in)	2.42	2.42	2.42	
Initial Void Ratio	0.874	0.844	0.857	
Initial Moisture (%)	29.6	29.3	28.9	
Initial Wet Density (pcf)	120.9	122.5	121.4	
Initial Dry Density (pcf)	93.3	94.8	94.1	
Initial Saturation (%)	94.9	97.1	94.5	
ΔHeight Consol (in)	0.0287	0.0552	0.0839	
At Test Void Ratio	0.820	0.743	0.702	
At Test Moisture (%)	28.5	26.5	24.8	
At Test Wet Density (pcf)	123.4	126.8	128.2	
At Test Dry Density (pcf)	96.0	100.3	102.7	
At Test Saturation (%)	97.2	99.7	99.1	
Strain Rate (%/min)	1.2	1.1	1.2	
Strengths Picked at	5%	5%	5%	
Shear Stress (psf)	657	1390	2492	
ΔHeight (in) at 5%				
Ultimate Stress (psf)				







Remarks: *DS-CU* A fully undrained condition may not be attained in this test. ΔH is not measured during undrained direct shear tests.

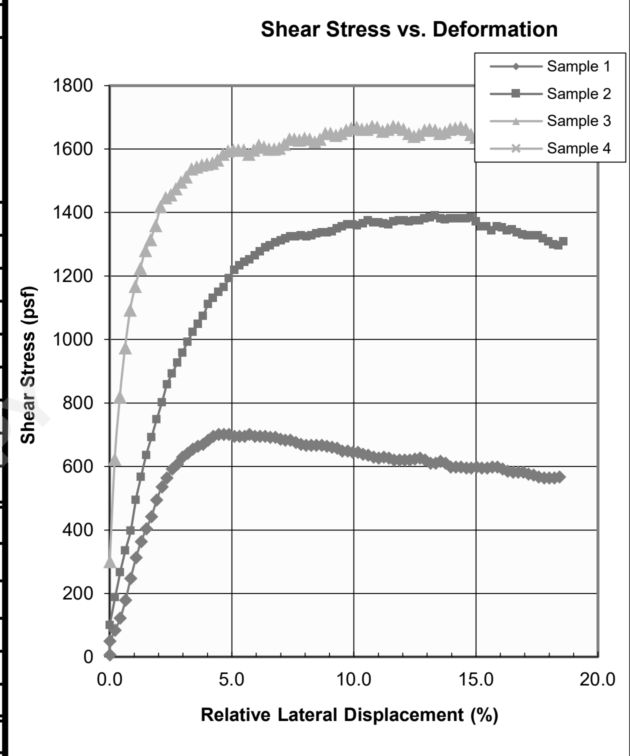
PLATE NO.: 34

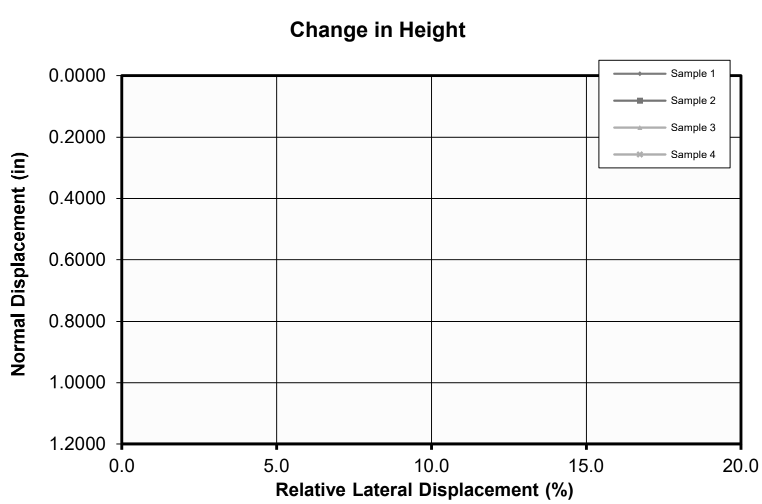
Consolidated Undrained Direct Shear (ASTM D3080M)

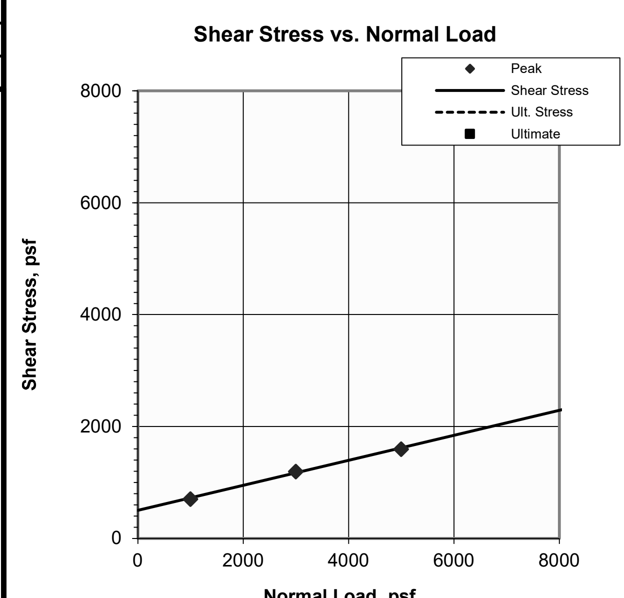
CTL Job #:	132-121	Project #:	21164-S	By:	MD
Client:	Advance Soil Technology	Date:	12/20/2021	Checked:	PJ
Project Name:	5200 Patrick Henry Drive, LLC	Remolding Info:			

Phi (deg)	12.6	Ult. Phi (deg)	
Cohesion (psf)	500	Ult. Cohesion (psf)	

Specimen Data	1	2	3	4
Boring:	EB-2	EB-2	EB-2	
Sample:	2-4	2-4	2-4	
Depth (ft):	15	15	15	
Visual Description:	Olive Gray CLAY w/ Sand	Olive Gray CLAY w/ Sand	Olive Gray CLAY w/ Sand	
Normal Load (psf)	1000	3000	5000	
Dry Mass of Specimen (g)	109.3	112.7	118.9	
Initial Height (in)	1.01	1.00	1.01	
Initial Diameter (in)	2.42	2.42	2.42	
Initial Void Ratio	0.948	0.878	0.789	
Initial Moisture (%)	32.4	31.1	27.2	
Initial Wet Density (pcf)	118.9	122.0	124.2	
Initial Dry Density (pcf)	89.7	93.1	97.7	
Initial Saturation (%)	95.8	99.0	96.4	
ΔHeight Consol (in)	0.0256	0.0576	0.0548	
At Test Void Ratio	0.898	0.771	0.692	
At Test Moisture (%)	31.8	27.3	24.3	
At Test Wet Density (pcf)	121.4	125.7	128.4	
At Test Dry Density (pcf)	92.1	98.7	103.3	
At Test Saturation (%)	99.2	99.4	98.2	
Strain Rate (%/min)	1.2	1.1	1.1	
Strengths Picked at	5%	5%	5%	
Shear Stress (psf)	701	1193	1594	
ΔHeight (in) at 5%				
Ultimate Stress (psf)				





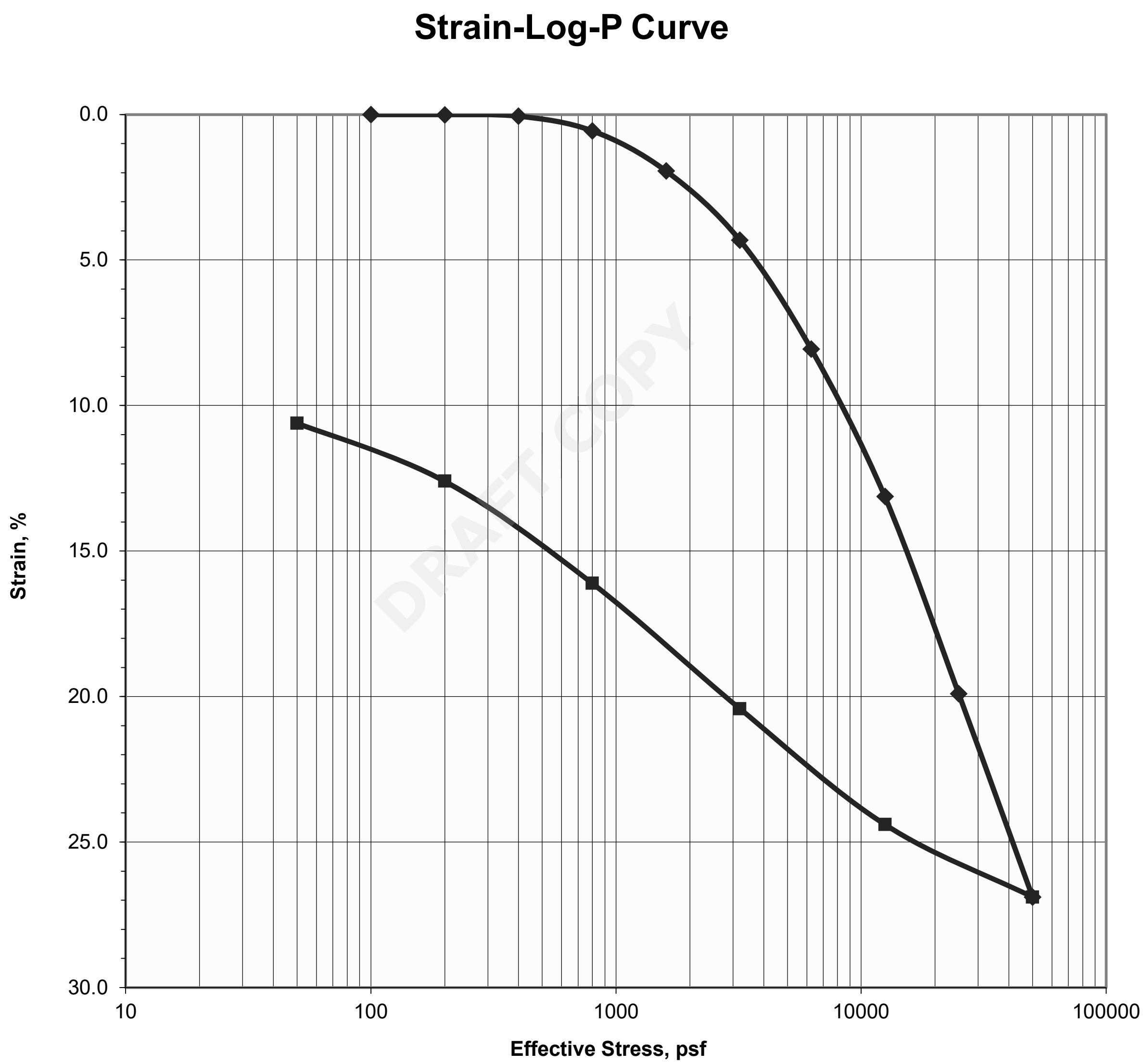


Remarks: *DS-CU* A fully undrained condition may not be attained in this test. ΔH is not measured during undrained direct shear tests.

PLATE NO.: 35

Consolidation Test
ASTM D2435

Job No.:	132-113	**Boring:**	EB-01	**Run By:**	MD
Client:	AST Inc	**Sample:**	1-4	**Reduced:**	PJ
Project:	20144-S	**Depth, ft.:**	15	**Checked:**	PJ/DC
Soil Type:	Grayish Brown CLAY			**Date:**	7/23/2020



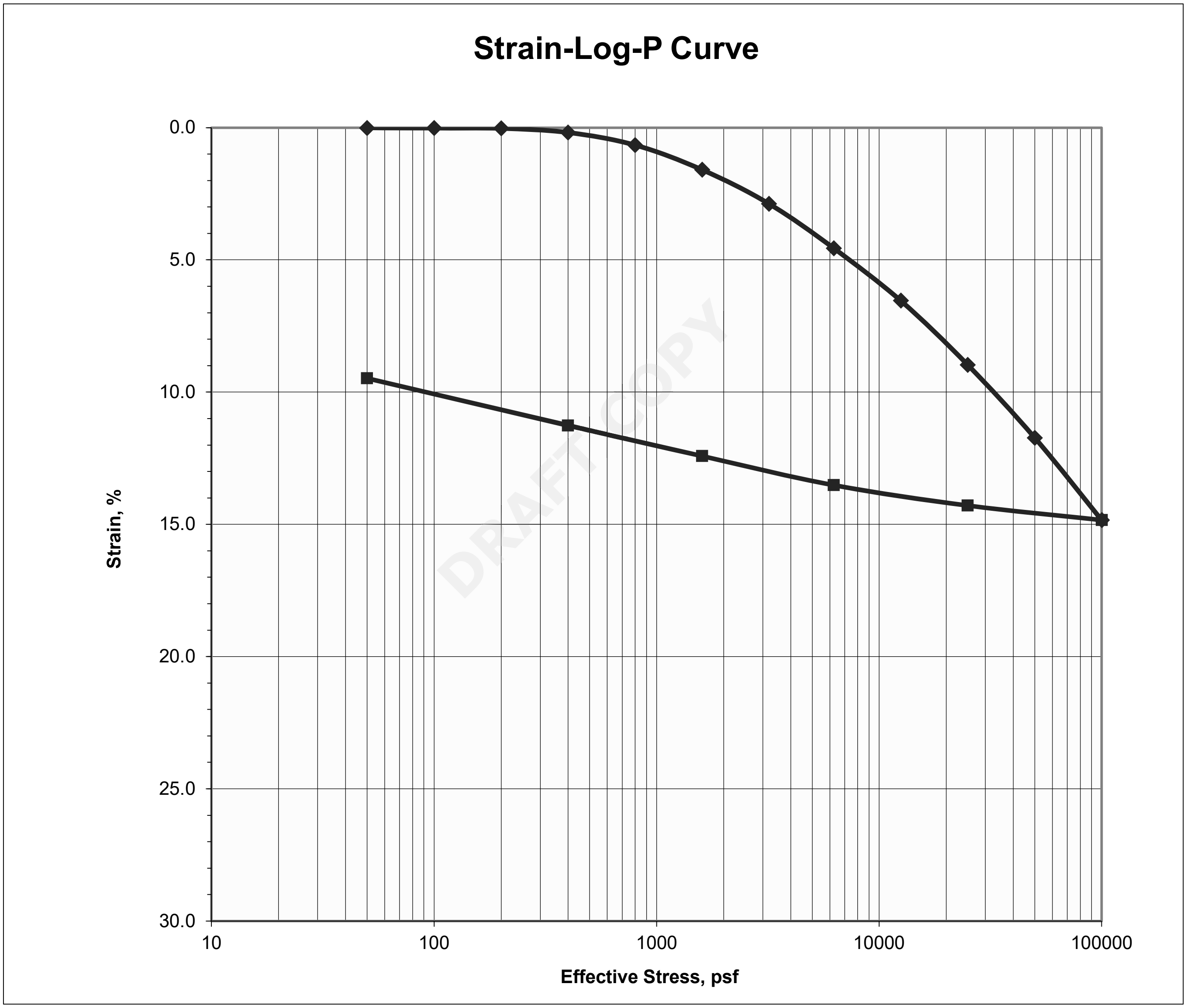
Assumed Gs 2.7	Initial	Final
Moisture %:	48.8	41.5
Dry Density, pcf:	71.1	79.5
Void Ratio:	1.370	1.120
% Saturation:	96.2	100.0

Remarks:

PLATE NO.: 36

Consolidation Test
ASTM D2435

Job No.:	132-113	**Boring:**	EB-01	**Run By:**	MD
Client:	AST Inc.	**Sample:**	1-5	**Reduced:**	PJ
Project:	20144-S	**Depth, ft.:**	20	**Checked:**	PJ/DC
Soil Type:	Gray Sandy CLAY			**Date:**	1/20/1900



Assumed Gs 2.7	Initial	Final
Moisture %:	23.4	19.4
Dry Density, pcf:	100.5	110.7
Void Ratio:	0.678	0.523
% Saturation:	93.3	100.0

Remarks:

PLATE NO.: 37

Corrosivity Tests Summary

CTL #: 132-120
Date: 12/13/2021
Tested By: PJ
Checked: PJ
Client: ADVANCE SOIL TECH, INC
Project: 5200 Patrick Henry Drive, LLC
Proj. No: 21164-S
Remarks:

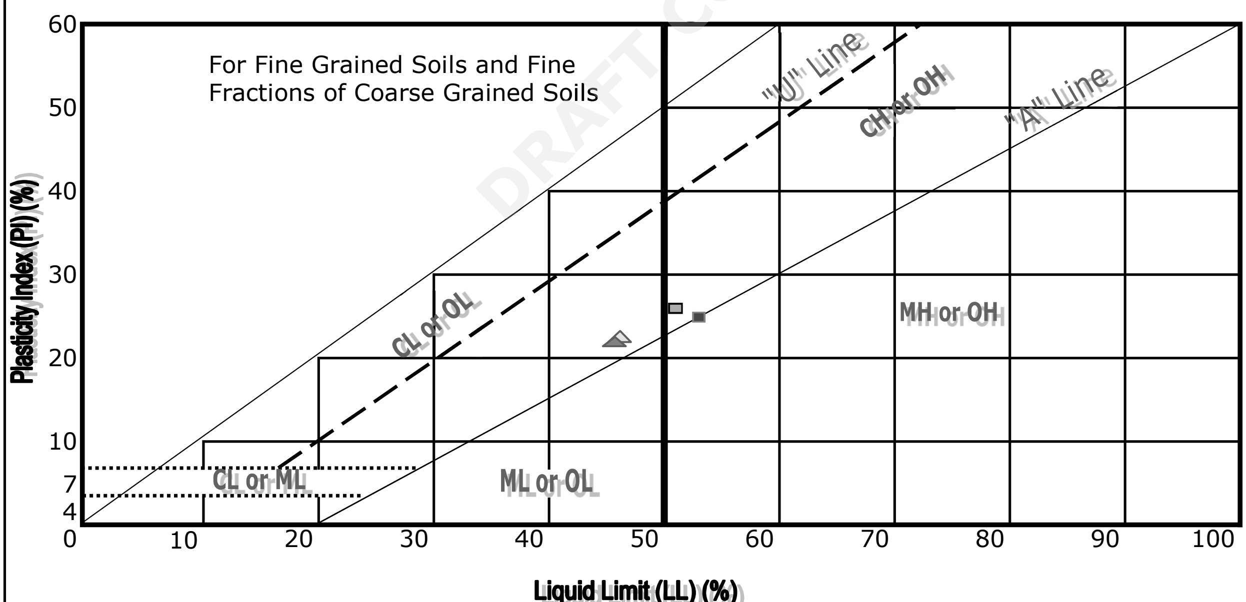
PLATE NO.: 38

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
CPT-01	comp	2-6	-	-	848	36	51	0.0051	8.4	454	18	-	22.7	Dark Grayish Brown CLAY w/ Sand
CPY-04	comp	2-6	-	-	1,317	88	100	0.0100	8.5	439	18	-	18.5	Dark Yellowish Brown CLAY w/ Sand

Plasticity Data

Key Symbol	Boring No.	Depth (feet)	Natural Water Content (Wc) %	Wc/LL	Liquid Limit %	Plasticity Index %	Unified Soil Classification & Description of Soil	
■	EB-01	2	22.1	0.42	53	24	CL	Clay
▲	EB-01	5	18.6	0.40	47	22	CL	Clay
△	EB-02	2	20.4	0.43	48	21	CL	Clay
□	EB-02	15	29.5	0.58	51	25	CL	Clay

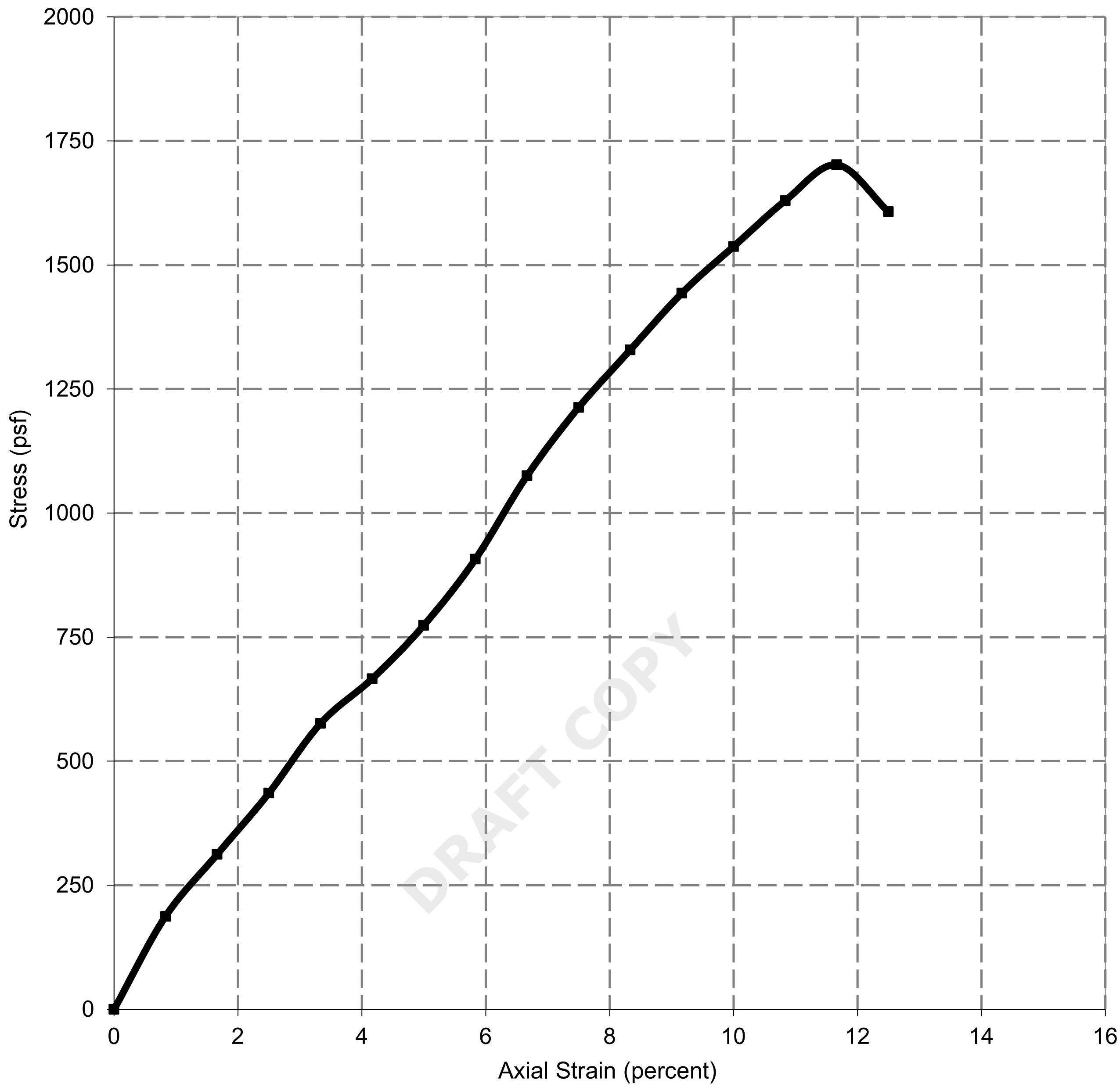
Plasticity Chart



Plasticity Index

AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 40

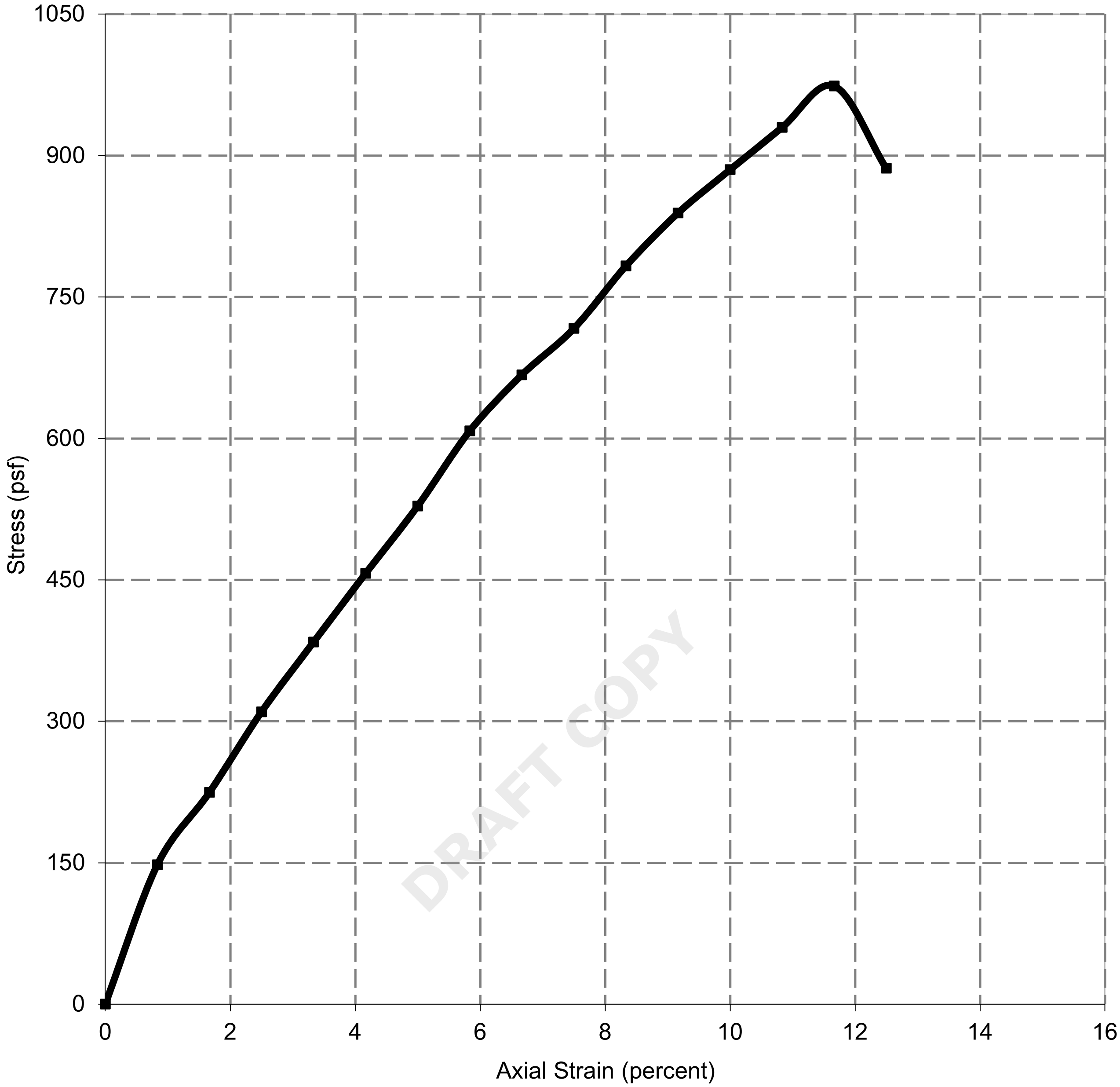


Sampler Type: Split Spoon Sampler			Shear Strength:	851	psf
Diameter (in):	2.43	Height (in): 6.0	Strain At Failure:	11.6	%
Moisture Content:	19.3	%	Stress:	1702	psf
Dry Density:	109.2	pcf	Source: EB-1, 1-4 @ 15.0-feet		
Soil Description: Lean Clay, Medium to Tan Brown with Grayish Mottling (CL)					

Unconfined Compression Test ASTM D2166

AST AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 40

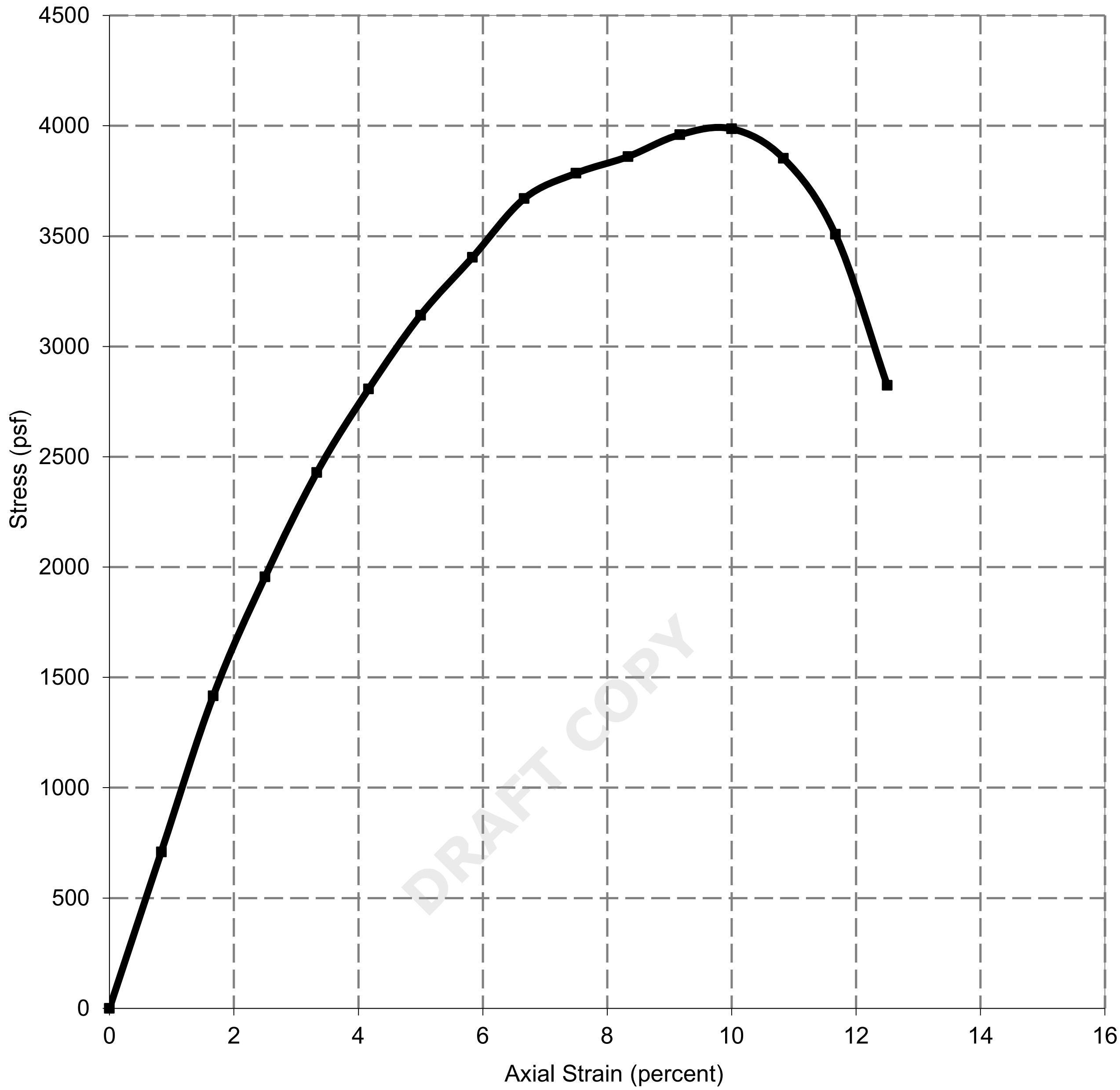


Sampler Type: Split Spoon Sampler				Shear Strength:	487	psf
Diameter (in):	2.43	Height (in): 6.0		Strain At Failure:	11.6	%
Moisture Content:		20.1	%	Stress:	974	psf
Dry Density:		108.3	pcf	Source: EB-1, 1-5 @ 20.0-feet		
Soil Description: Lean Clay, Medium to Tan Brown with Grayish Mottling (CL)						

Unconfined Compression Test ASTM D2166

AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 41



Sampler Type: Split Spoon Sampler			Shear Strength:	1993	psf
Diameter (in):	2.43	Height (in): 6.0	Strain At Failure:	10	%
Moisture Content:		22.1 %	Stress:	3986	psf
Dry Density:		108.2 pcf	Source: EB-1, 2-5 @ 20.0-feet		
Soil Description: Lean Clay, Medium to Tan Brown with Grayish Mottling & Rust Stains (CL)					

Unconfined Compression Test ASTM D2166

AST
AST
Advance Soil Technology, Inc.
Geological , Geotechnical, Environmental
Consulting & Construction Services

343 So. Baywood Avenue | San Jose, California

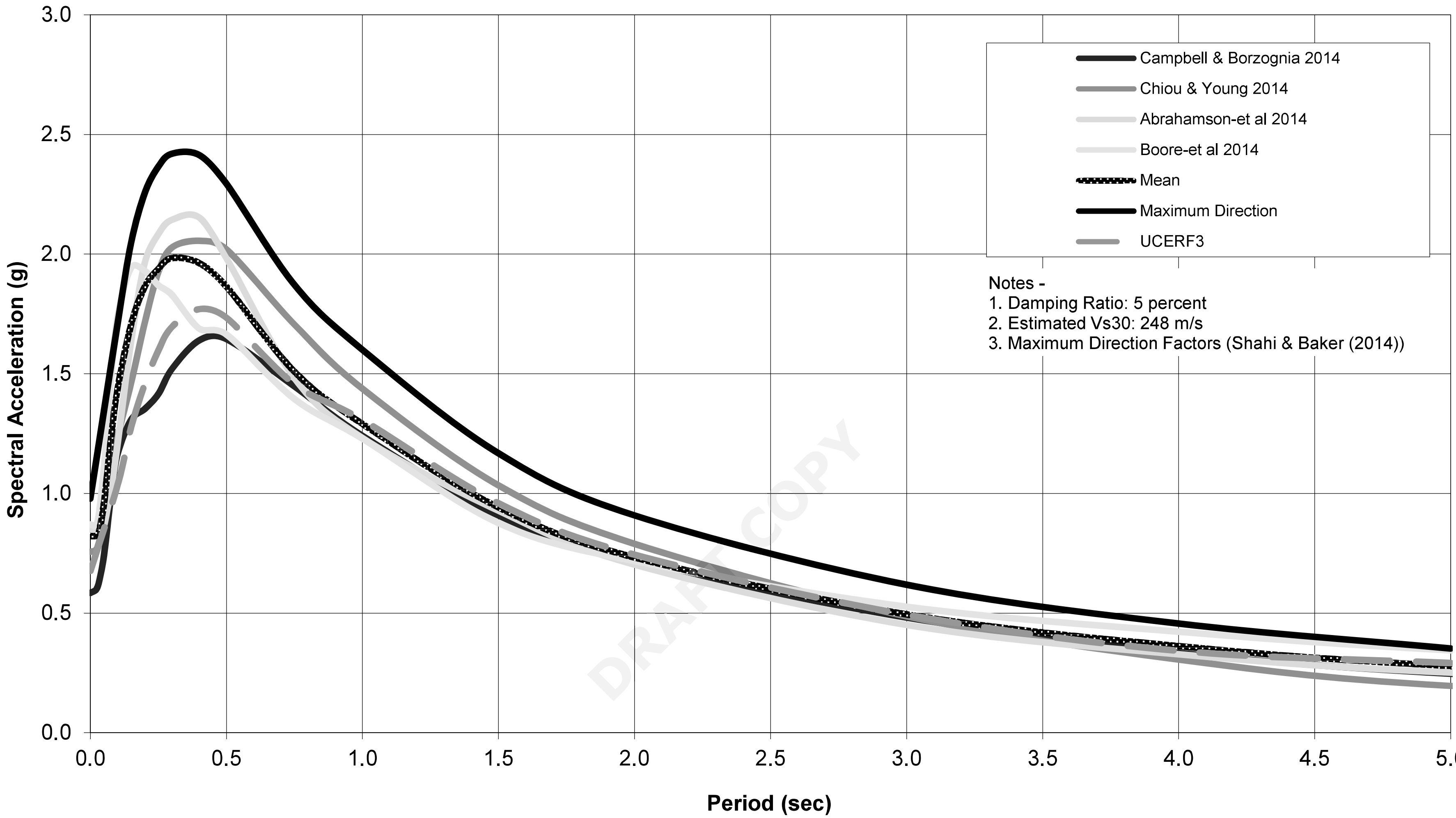
Geotechnical Study/Investigation

5200 PATRICK HENRY DRIVE

Four-Story Building | One-Level Below Grade Parking

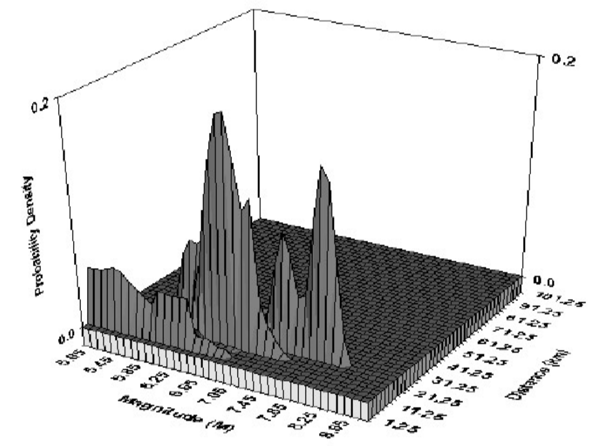
5200 Patrick Henry Drive | Santa Clara, California

Appendix "D"

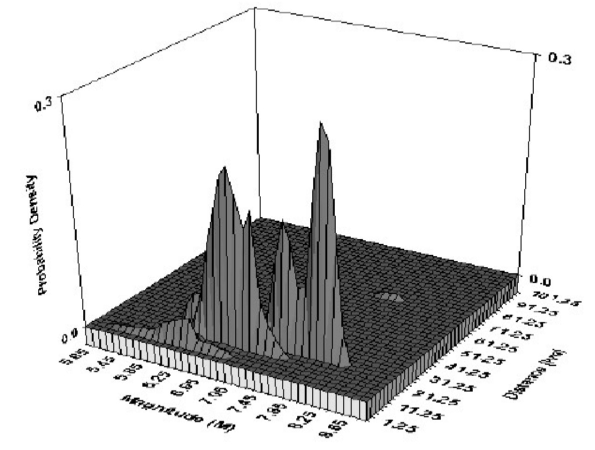


Probabilistic Hazard Analysis (2% Probability of Exceedance in 50 Years)

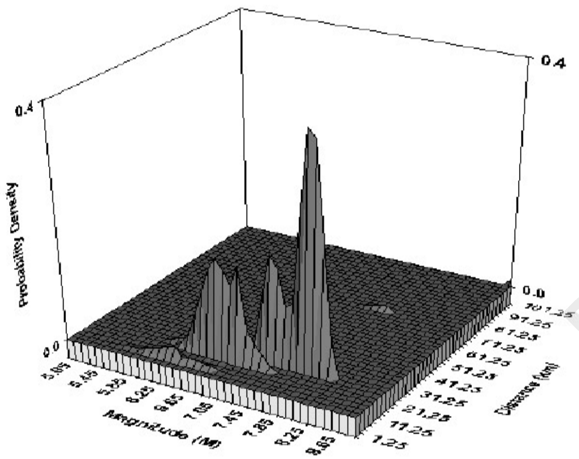
AST	Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California		5200 Patrick Henry Drive | Santa Clara, California
AST File No. 21164-S	Date: December 2021	Plate No. 43



PGA



Sa, T= 1.0-sec



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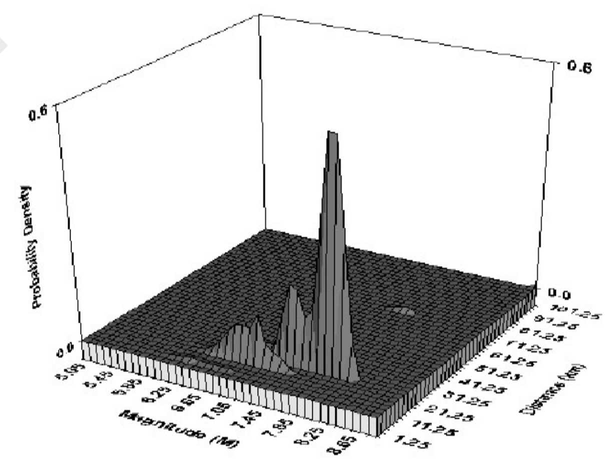
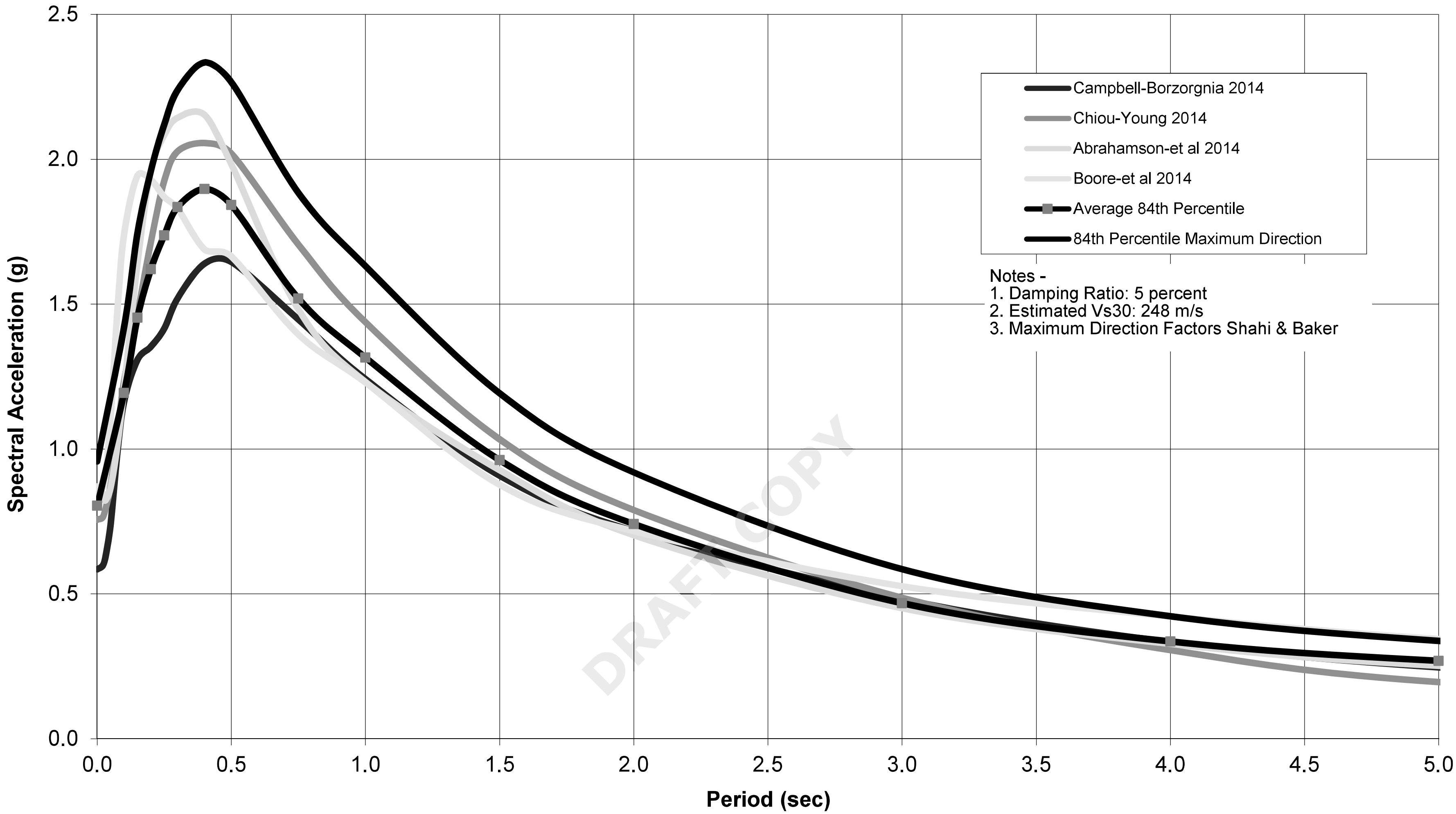


Sa, T= 4.0-sec

Deaggregation

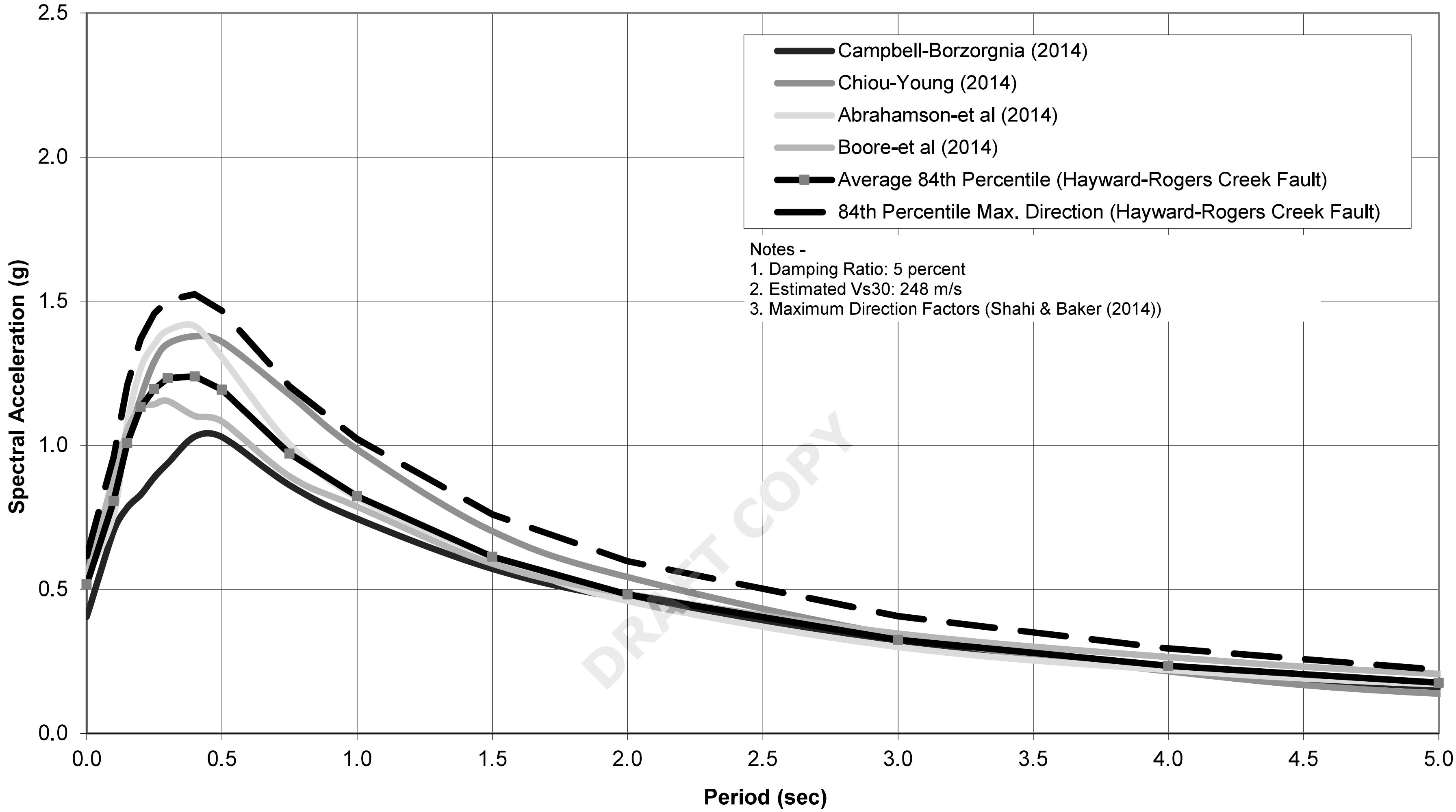
AST **Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 44



Deterministic Hazard Analysis

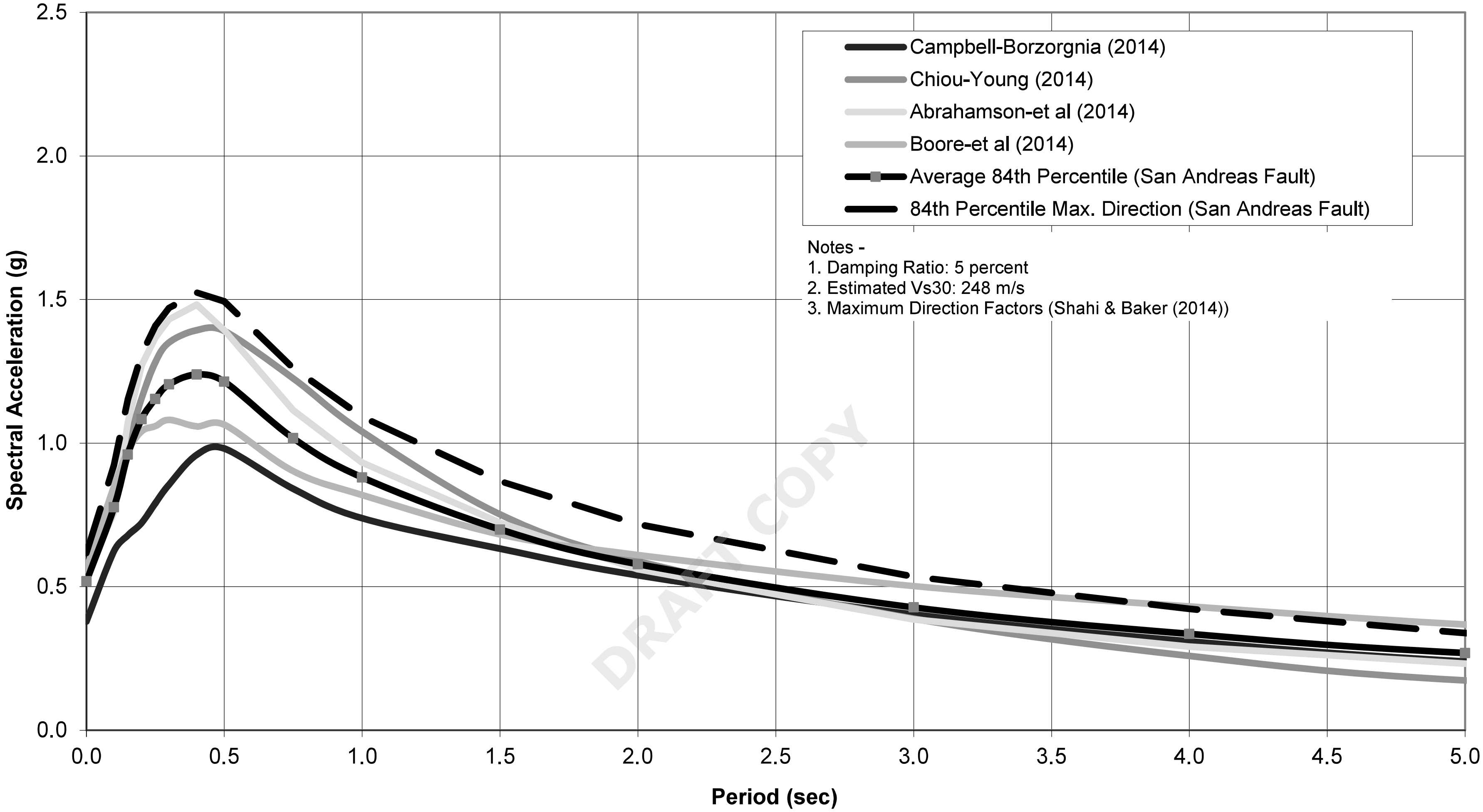
AST	Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California		5200 Patrick Henry Drive | Santa Clara, California
AST File No. 21164-S	Date: December 2021	Plate No. 45



Results of 84th Percentile Deterministic Analysis for Hayward-Rogers Creek Fault

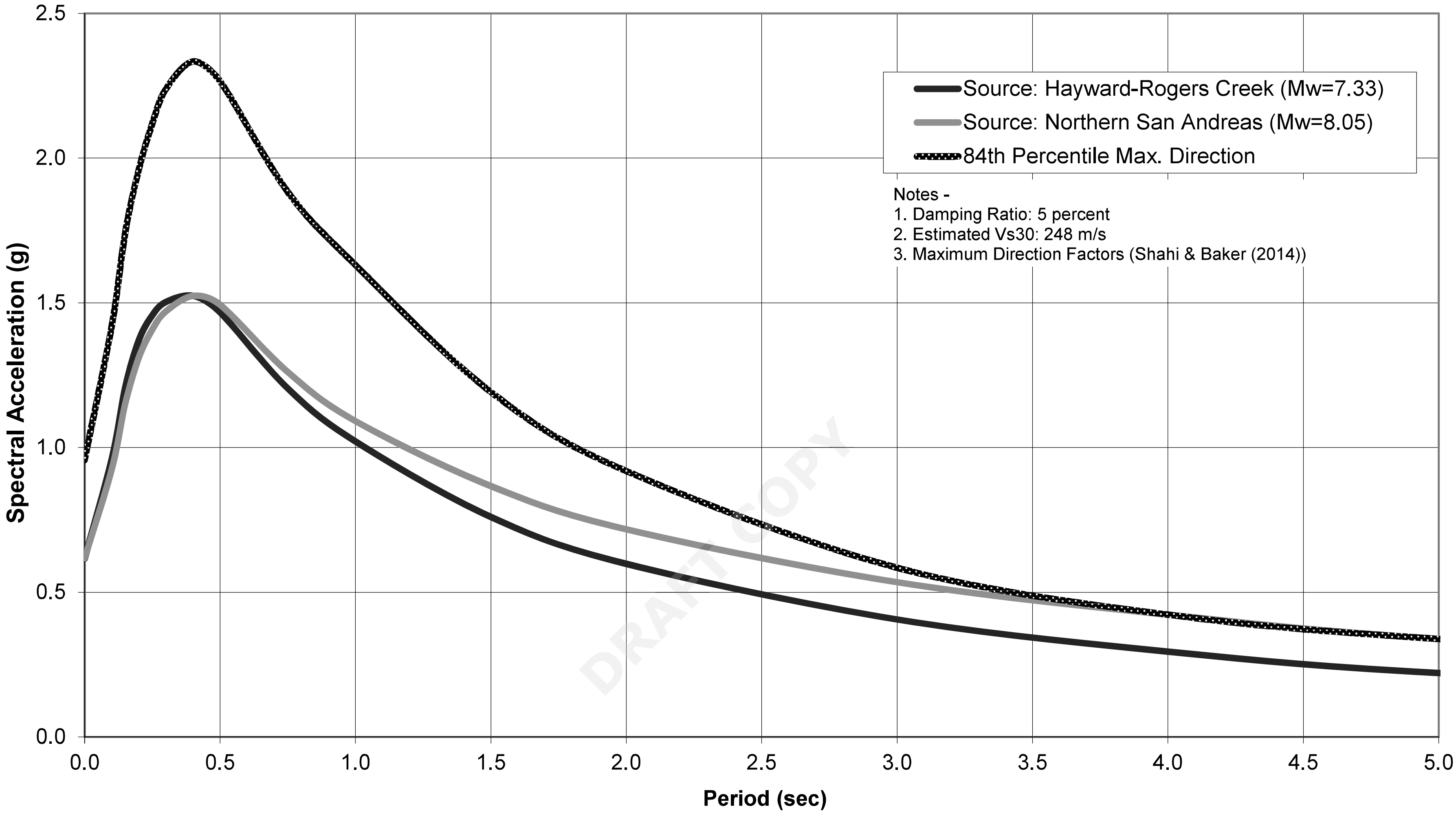
AST	Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
	343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 46



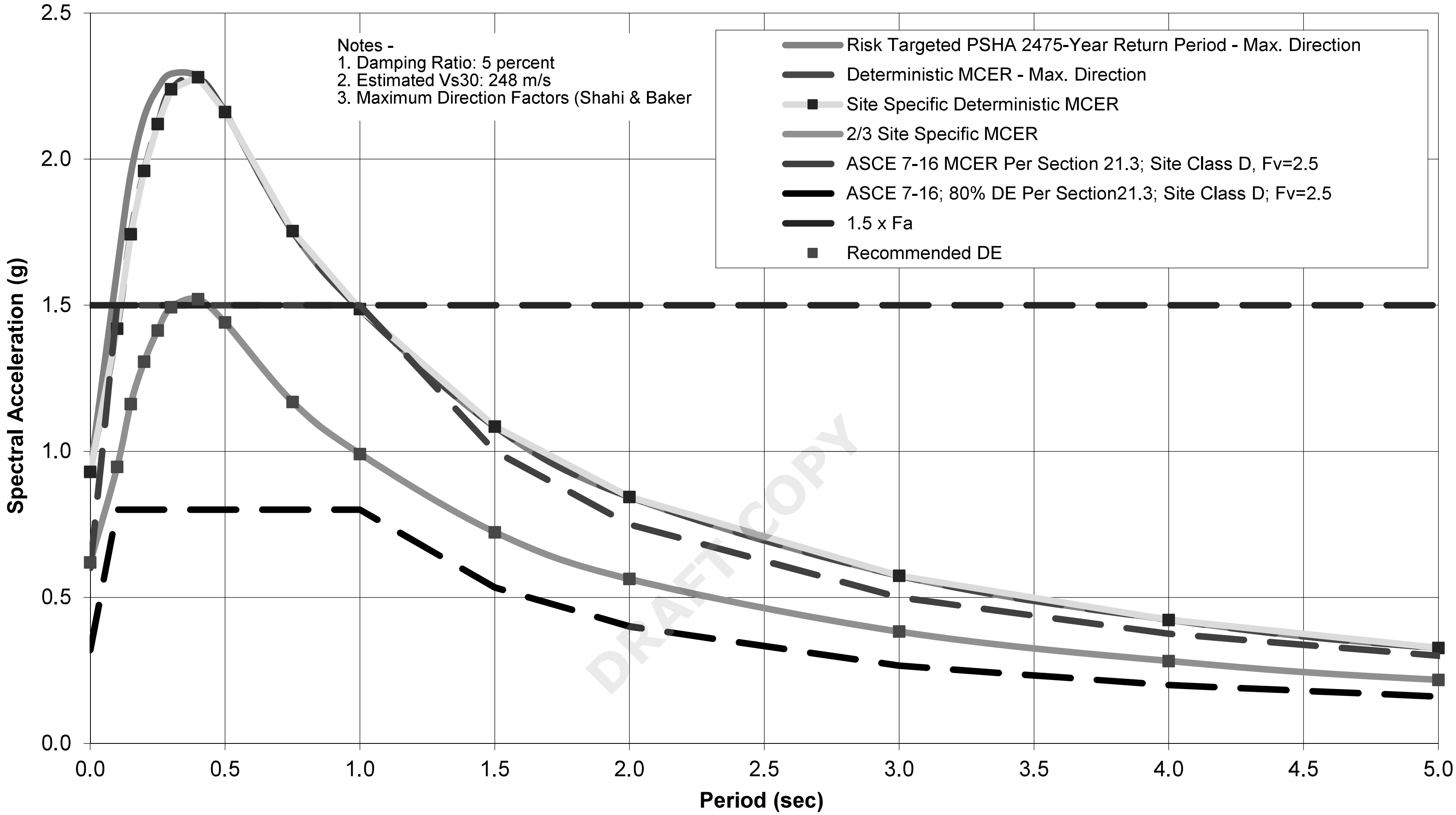
Results of 84th Percentile Deterministic Analysis for San Andreas Fault

AST	Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California		5200 Patrick Henry Drive | Santa Clara, California
AST File No. 21164-S	Date: December 2021	Plate No. 47



Comparison of Results 84th Percentile - Deterministic Analysis (Hayward-Rogers Creek & San Andreas Faults)

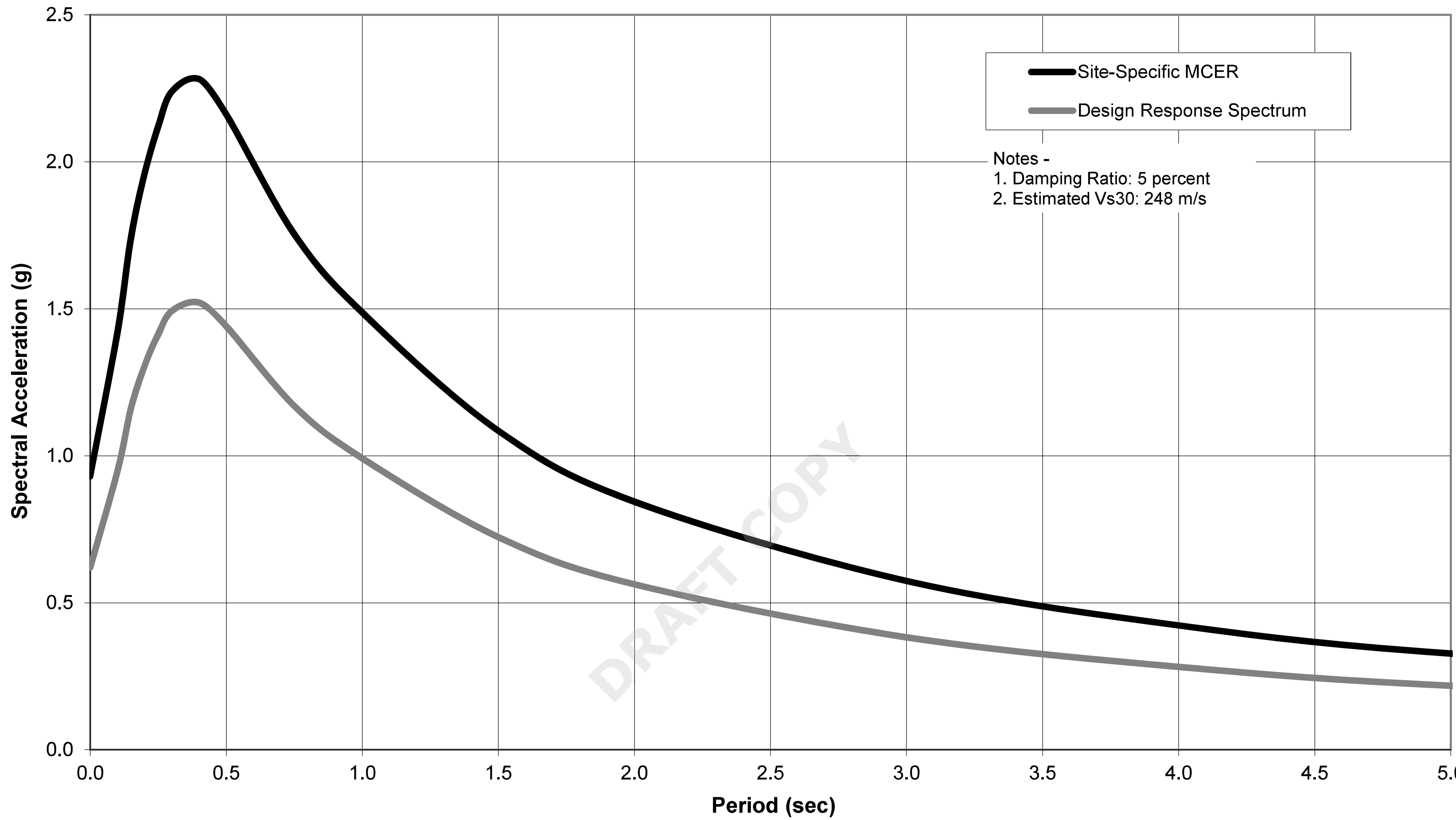
AST	Advance Soil Technology, Inc. Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
	343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California
AST File No. 21164-S	Date: December 2021	Plate No. 48



Comparison of Results of Probabilistic, Deterministic & Code Spectra

AST	**Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
	343 So. Baywood Avenue | San Jose, California	5200 Patrick Henry Drive | Santa Clara, California

AST File No. 21164-S	Date: December 2021	Plate No. 49



Recommended Spectra

AST	**Advance Soil Technology, Inc.** Geological , Geotechnical, Environmental Consulting & Construction Services	Geotechnical Study/Investigation **5200 PATRICK HENRY DRIVE** Four-Story Building | One-Level Below Grade Parking
343 So. Baywood Avenue | San Jose, California		5200 Patrick Henry Drive | Santa Clara, California
AST File No. 21164-S	Date: December 2021	Plate No. 50

Appendix "E"

Plate 51 Liquefaction Analysis

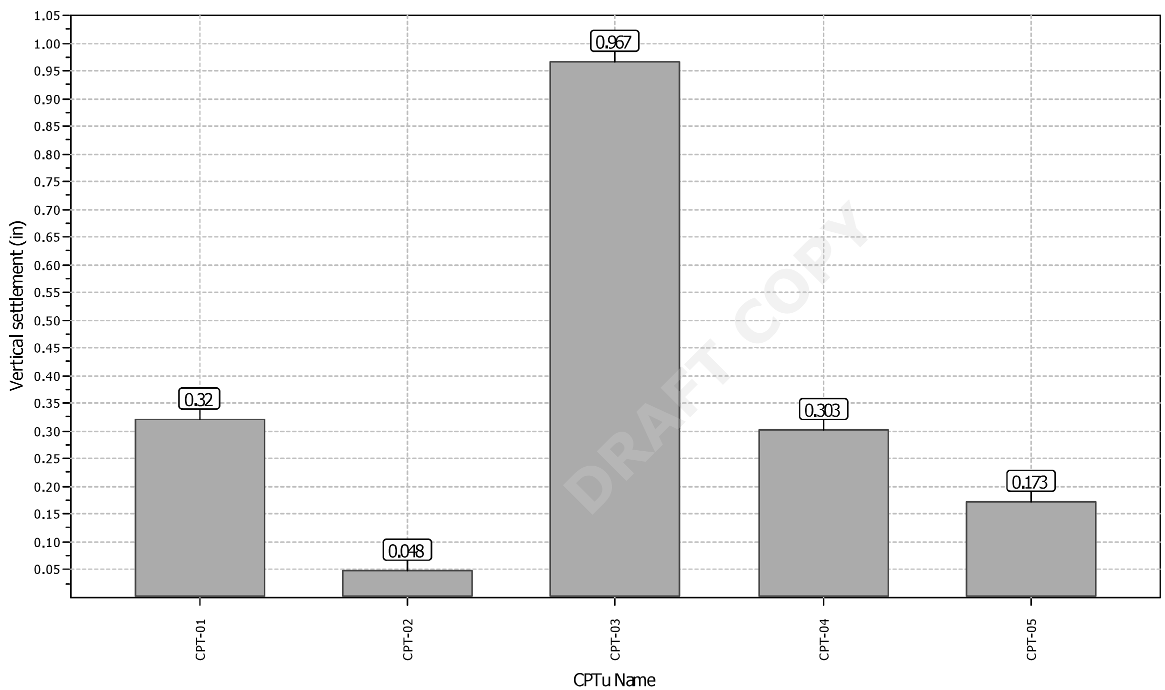
TABLE OF CONTENTS

Advance Soil Technology, Inc. (AST)
Geotechnical | Environmental Engineers
343 So. Baywood Avenue, San Jose CA 95128
http://www.advancesoil.com

Project title : 5200 Patrick Henry Drive, LLC

Location : 5200 Patrick Henry Drive, Santa Clara

Overall vertical settlements report



Advance Soil Technology, Inc. (AST)
Geotechnical | Environmental Engineers
343 So. Baywood Avenue, San Jose CA 95128
http://www.advancesoil.com

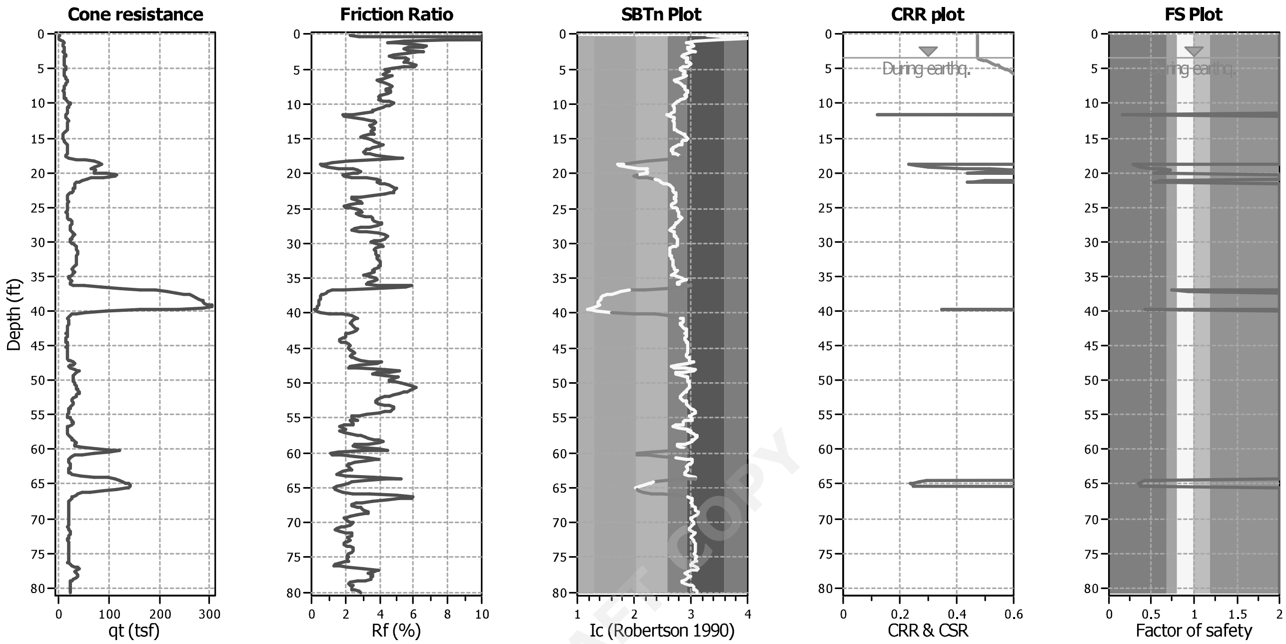
LIQUEFACTION ANALYSIS REPORT

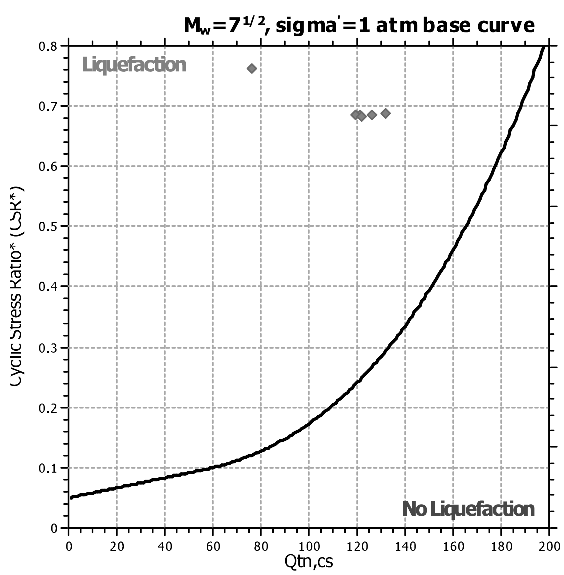
Project title : 5200 Patrick Henry Drive, LLC **Location : 5200 Patrick Henry Drive, Santa Clara**

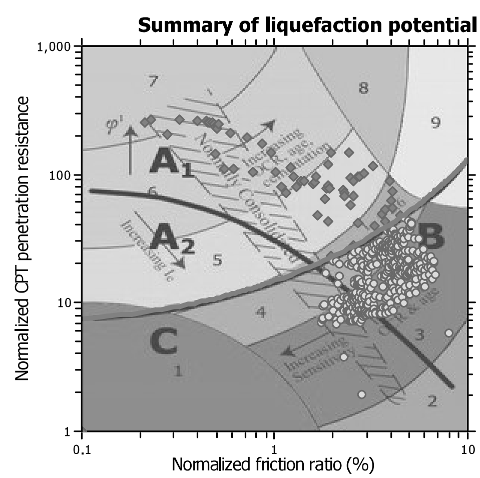
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.30 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.50 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:	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.61	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

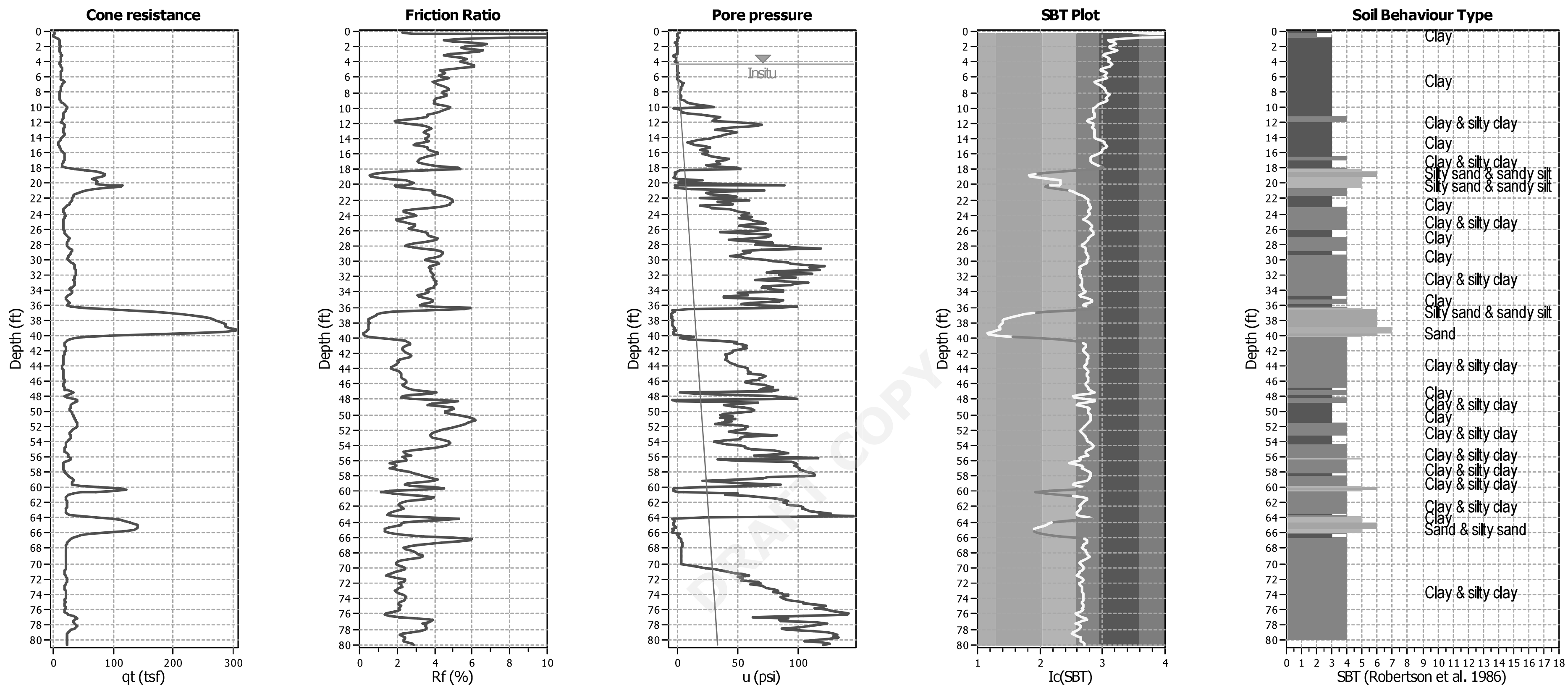






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



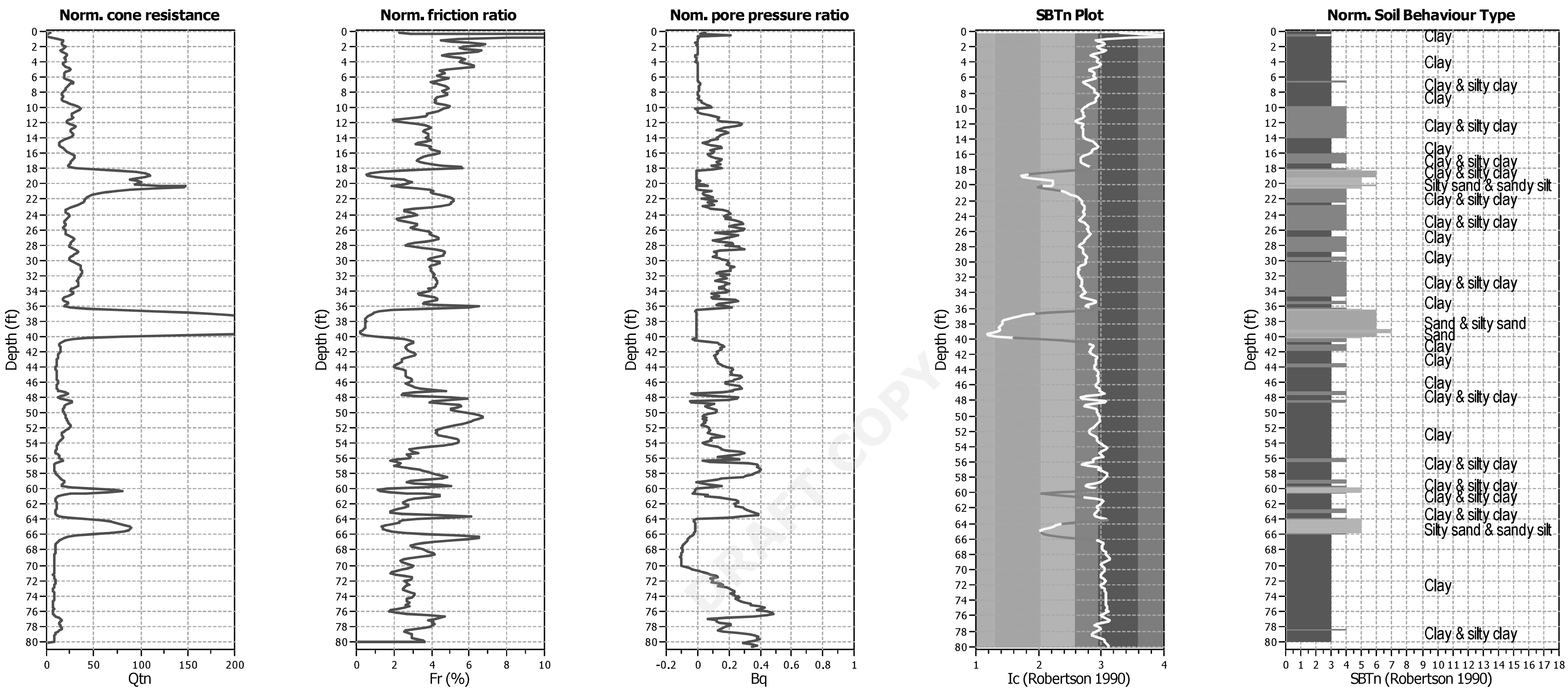
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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)



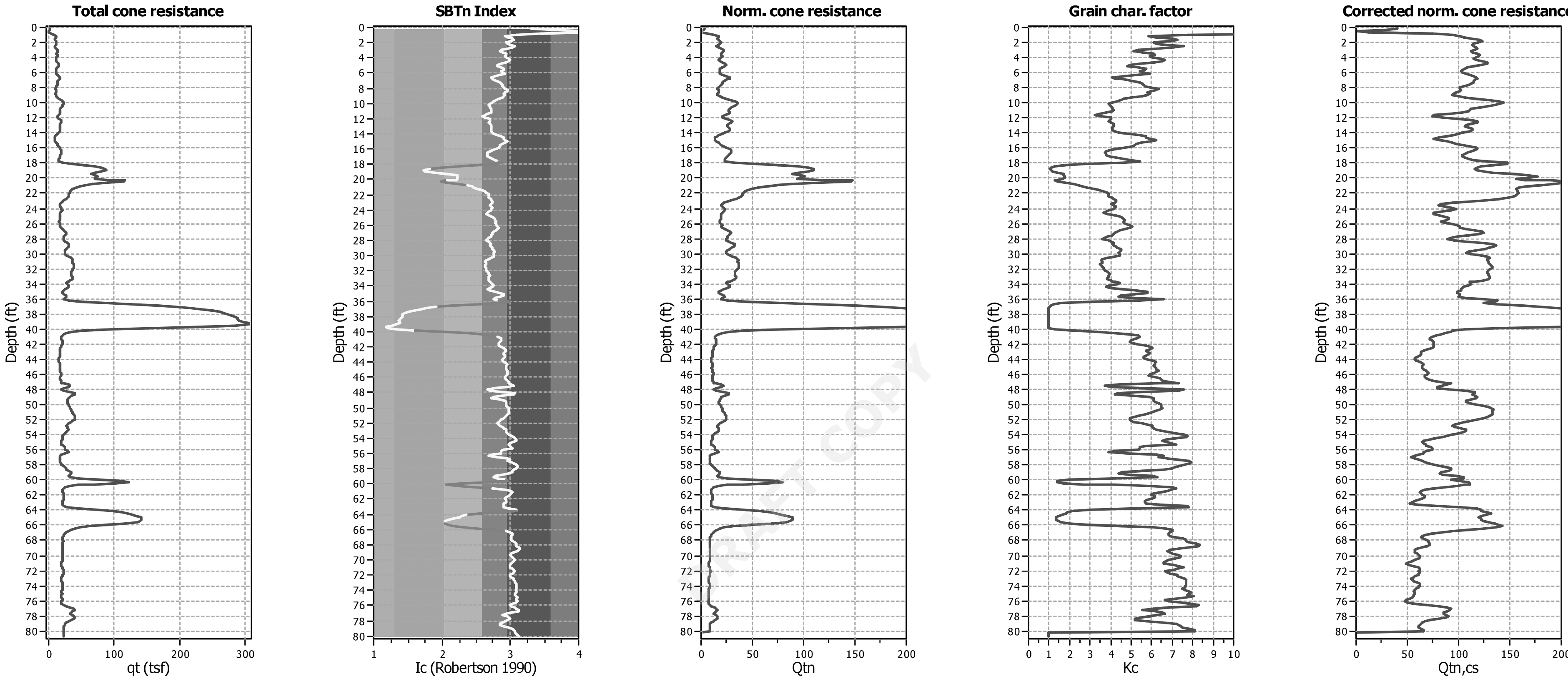
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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

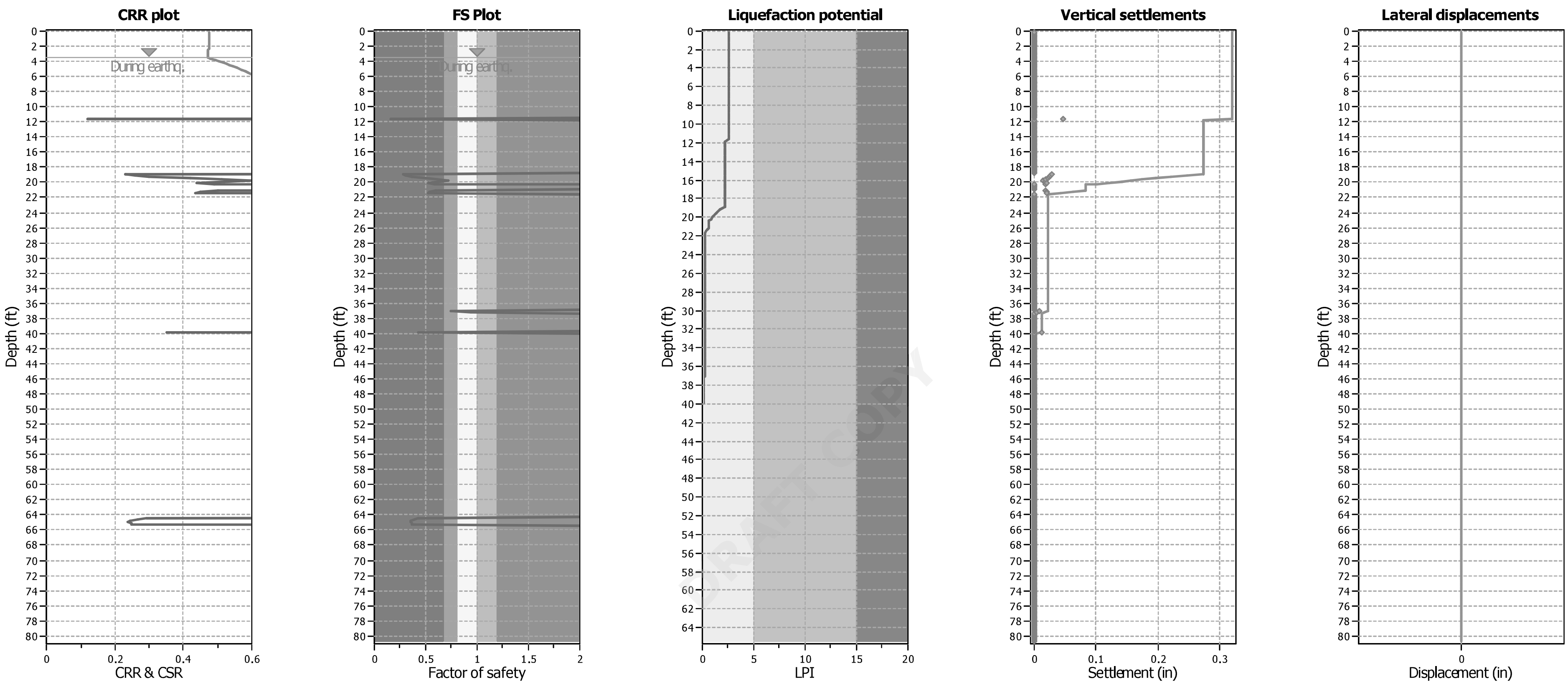
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

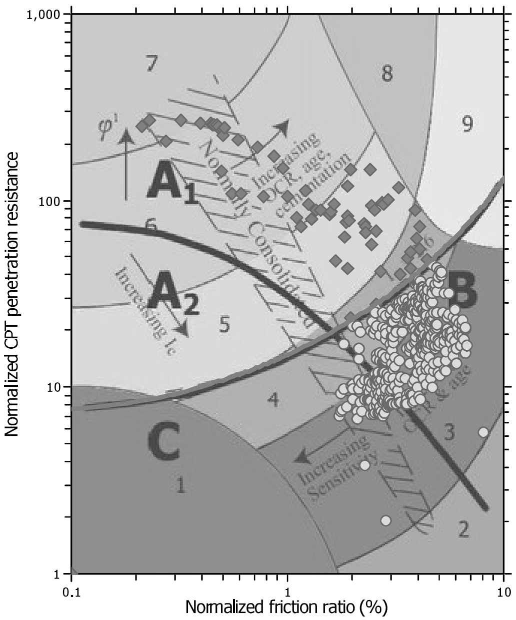
F.S. color scheme

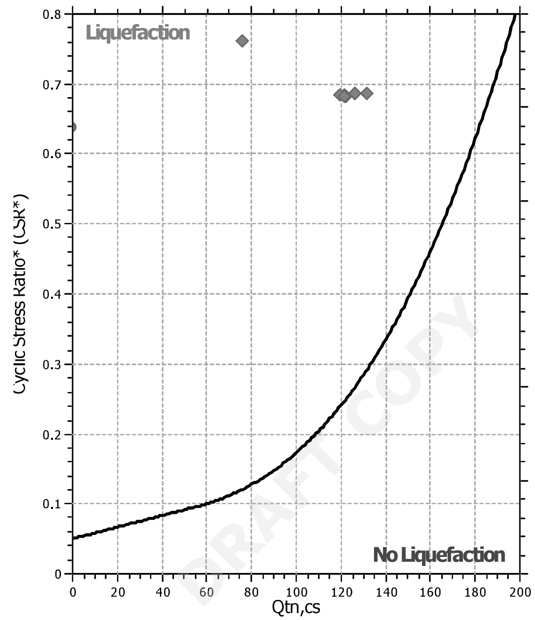
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

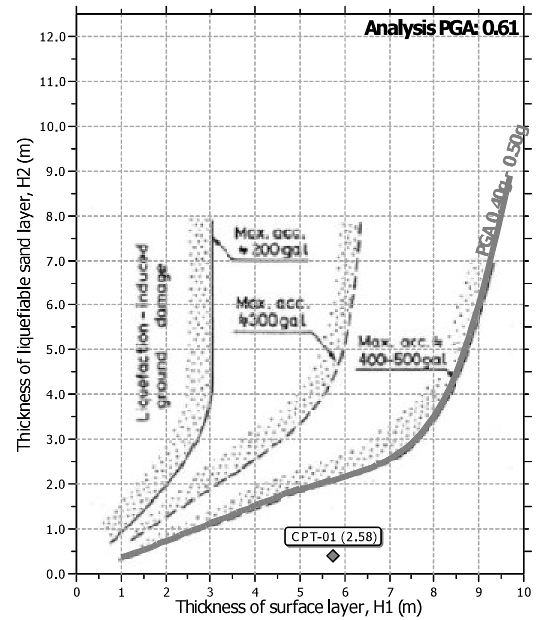
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



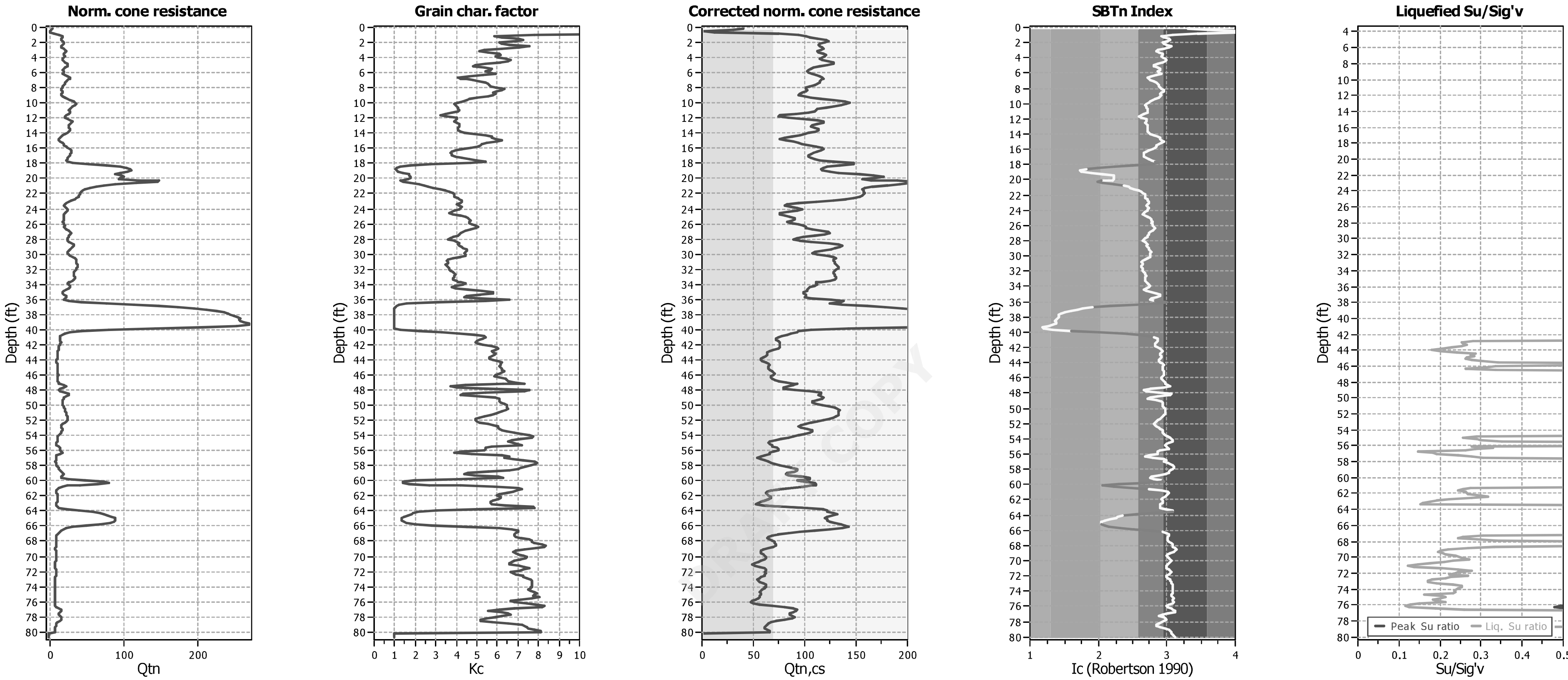




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

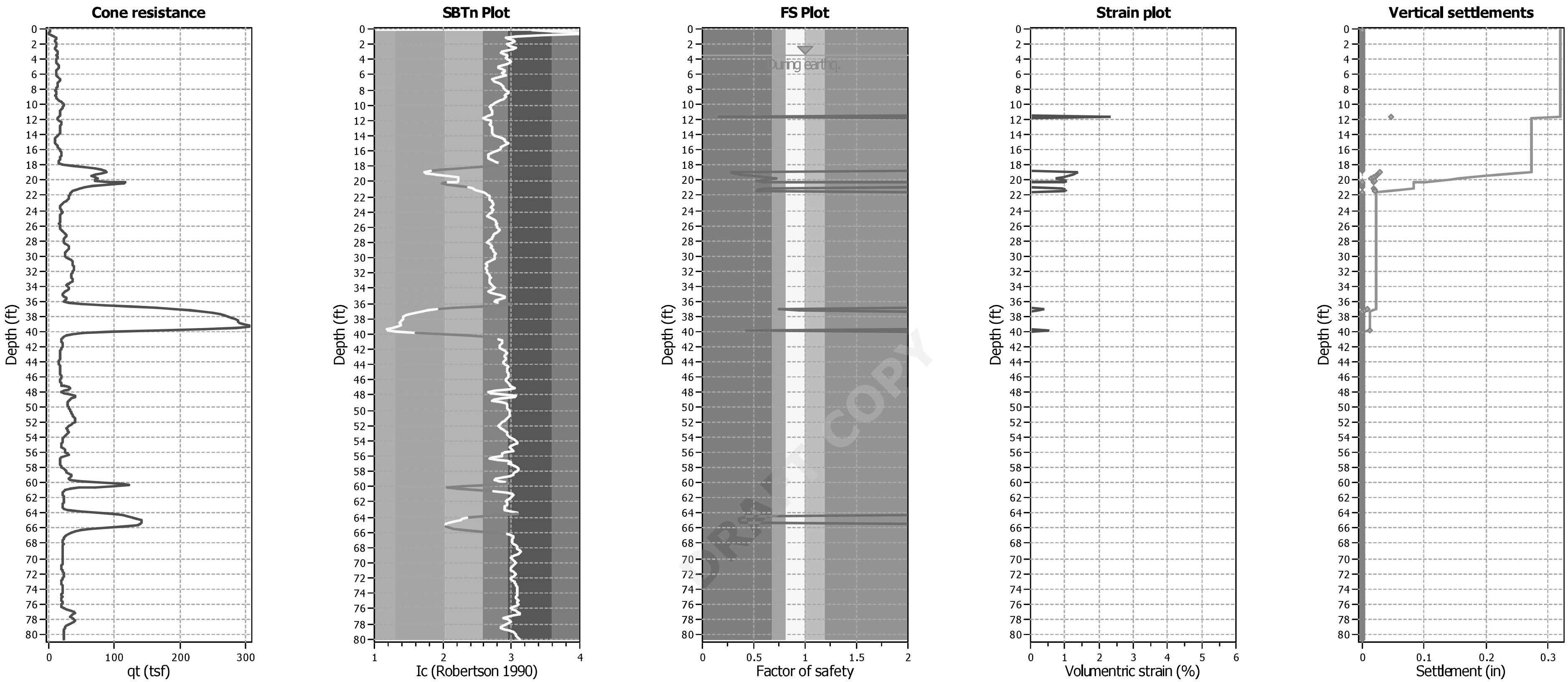
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

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

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
3.61	120.83	2.00	0.00	0.94	0.00	3.77	120.01	2.00	0.00	0.94	0.00
3.94	117.24	2.00	0.00	0.93	0.00	4.10	115.16	2.00	0.00	0.93	0.00
4.27	116.81	2.00	0.00	0.93	0.00	4.43	122.83	2.00	0.00	0.92	0.00
4.59	128.20	2.00	0.00	0.92	0.00	4.76	128.40	2.00	0.00	0.92	0.00
4.92	122.35	2.00	0.00	0.92	0.00	5.09	114.36	2.00	0.00	0.91	0.00
5.25	109.07	2.00	0.00	0.91	0.00	5.41	105.96	2.00	0.00	0.91	0.00
5.58	103.85	2.00	0.00	0.91	0.00	5.74	102.76	2.00	0.00	0.90	0.00
5.91	103.76	2.00	0.00	0.90	0.00	6.07	107.53	2.00	0.00	0.90	0.00
6.23	111.39	2.00	0.00	0.89	0.00	6.40	115.15	2.00	0.00	0.89	0.00
6.56	116.87	2.00	0.00	0.89	0.00	6.73	117.92	2.00	0.00	0.89	0.00
6.89	116.72	2.00	0.00	0.88	0.00	7.05	115.49	2.00	0.00	0.88	0.00
7.22	114.73	2.00	0.00	0.88	0.00	7.38	114.18	2.00	0.00	0.87	0.00
7.55	112.45	2.00	0.00	0.87	0.00	7.71	109.99	2.00	0.00	0.87	0.00
7.87	104.81	2.00	0.00	0.87	0.00	8.04	101.70	2.00	0.00	0.86	0.00
8.20	100.95	2.00	0.00	0.86	0.00	8.37	102.58	2.00	0.00	0.86	0.00
8.53	101.79	2.00	0.00	0.86	0.00	8.69	97.73	2.00	0.00	0.85	0.00
8.86	94.73	2.00	0.00	0.85	0.00	9.02	94.50	2.00	0.00	0.85	0.00
9.19	99.44	2.00	0.00	0.84	0.00	9.35	105.49	2.00	0.00	0.84	0.00
9.51	115.47	2.00	0.00	0.84	0.00	9.68	128.01	2.00	0.00	0.84	0.00
9.84	139.38	2.00	0.00	0.83	0.00	10.01	143.57	2.00	0.00	0.83	0.00
10.17	140.66	2.00	0.00	0.83	0.00	10.33	135.60	2.00	0.00	0.82	0.00
10.50	128.51	2.00	0.00	0.82	0.00	10.66	120.46	2.00	0.00	0.82	0.00
10.83	112.32	2.00	0.00	0.82	0.00	10.99	110.41	2.00	0.00	0.81	0.00
11.15	110.23	2.00	0.00	0.81	0.00	11.32	103.60	2.00	0.00	0.81	0.00
11.48	90.94	2.00	0.00	0.81	0.00	11.65	76.00	0.16	2.35	0.80	0.05
11.81	74.40	2.00	0.00	0.80	0.00	11.98	84.08	2.00	0.00	0.80	0.00
12.14	99.19	2.00	0.00	0.79	0.00	12.30	111.70	2.00	0.00	0.79	0.00
12.47	118.13	2.00	0.00	0.79	0.00	12.63	117.83	2.00	0.00	0.79	0.00
12.80	112.47	2.00	0.00	0.78	0.00	12.96	107.25	2.00	0.00	0.78	0.00
13.12	106.26	2.00	0.00	0.78	0.00	13.29	109.56	2.00	0.00	0.77	0.00
13.45	113.28	2.00	0.00	0.77	0.00	13.62	113.84	2.00	0.00	0.77	0.00
13.78	110.53	2.00	0.00	0.77	0.00	13.94	104.17	2.00	0.00	0.76	0.00
14.11	99.76	2.00	0.00	0.76	0.00	14.27	95.52	2.00	0.00	0.76	0.00
14.44	87.88	2.00	0.00	0.76	0.00	14.60	80.04	2.00	0.00	0.75	0.00
14.76	76.07	2.00	0.00	0.75	0.00	14.93	82.05	2.00	0.00	0.75	0.00
15.09	88.81	2.00	0.00	0.74	0.00	15.26	94.02	2.00	0.00	0.74	0.00
15.42	98.14	2.00	0.00	0.74	0.00	15.58	103.64	2.00	0.00	0.74	0.00
15.75	110.77	2.00	0.00	0.73	0.00	15.91	116.57	2.00	0.00	0.73	0.00
16.08	118.09	2.00	0.00	0.73	0.00	16.24	115.63	2.00	0.00	0.72	0.00
16.40	111.94	2.00	0.00	0.72	0.00	16.57	108.59	2.00	0.00	0.72	0.00
16.73	106.00	2.00	0.00	0.72	0.00	16.90	103.48	2.00	0.00	0.71	0.00
17.06	103.12	2.00	0.00	0.71	0.00	17.22	105.60	2.00	0.00	0.71	0.00
17.39	109.39	2.00	0.00	0.71	0.00	17.55	114.88	2.00	0.00	0.70	0.00
17.72	128.60	2.00	0.00	0.70	0.00	17.88	147.50	2.00	0.00	0.70	0.00
18.04	146.98	2.00	0.00	0.69	0.00	18.21	134.17	2.00	0.00	0.69	0.00
18.37	122.45	2.00	0.00	0.69	0.00	18.54	117.71	2.00	0.00	0.69	0.00
18.70	115.26	2.00	0.00	0.68	0.00	18.86	117.55	0.28	1.39	0.68	0.03
19.03	124.50	0.31	1.32	0.68	0.03	19.19	133.48	0.36	1.24	0.67	0.02

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
19.36	154.21	0.50	1.10	0.67	0.02	19.52	167.04	0.61	0.97	0.67	0.02
19.69	177.30	0.72	0.73	0.67	0.01	19.85	167.70	0.62	0.95	0.66	0.02
20.01	156.65	0.52	1.07	0.66	0.02	20.18	164.21	0.59	0.97	0.66	0.02
20.34	190.74	2.00	0.00	0.66	0.00	20.51	204.57	2.00	0.00	0.65	0.00
20.67	203.27	2.00	0.00	0.65	0.00	20.83	192.95	2.00	0.00	0.65	0.00
21.00	180.42	2.00	0.00	0.64	0.00	21.16	165.36	0.60	0.94	0.64	0.02
21.33	158.71	0.54	1.02	0.64	0.02	21.49	156.30	0.52	1.03	0.64	0.02
21.65	156.78	2.00	0.00	0.63	0.00	21.82	157.88	2.00	0.00	0.63	0.00
21.98	158.26	2.00	0.00	0.63	0.00	22.15	157.60	2.00	0.00	0.62	0.00
22.31	156.07	2.00	0.00	0.62	0.00	22.47	153.06	2.00	0.00	0.62	0.00
22.64	147.88	2.00	0.00	0.62	0.00	22.80	140.67	2.00	0.00	0.61	0.00
22.97	127.67	2.00	0.00	0.61	0.00	23.13	111.32	2.00	0.00	0.61	0.00
23.29	93.82	2.00	0.00	0.61	0.00	23.46	82.68	2.00	0.00	0.60	0.00
23.62	80.58	2.00	0.00	0.60	0.00	23.79	86.46	2.00	0.00	0.60	0.00
23.95	93.09	2.00	0.00	0.59	0.00	24.11	97.53	2.00	0.00	0.59	0.00
24.28	92.43	2.00	0.00	0.59	0.00	24.44	84.19	2.00	0.00	0.59	0.00
24.61	76.04	2.00	0.00	0.58	0.00	24.77	75.34	2.00	0.00	0.58	0.00
24.93	79.94	2.00	0.00	0.58	0.00	25.10	85.37	2.00	0.00	0.57	0.00
25.26	90.59	2.00	0.00	0.57	0.00	25.43	90.35	2.00	0.00	0.57	0.00
25.59	87.27	2.00	0.00	0.57	0.00	25.75	83.13	2.00	0.00	0.56	0.00
25.92	86.51	2.00	0.00	0.56	0.00	26.08	93.48	2.00	0.00	0.56	0.00
26.25	99.34	2.00	0.00	0.56	0.00	26.41	100.68	2.00	0.00	0.55	0.00
26.57	101.63	2.00	0.00	0.55	0.00	26.74	107.23	2.00	0.00	0.55	0.00
26.90	116.28	2.00	0.00	0.54	0.00	27.07	123.46	2.00	0.00	0.54	0.00
27.23	124.61	2.00	0.00	0.54	0.00	27.40	118.94	2.00	0.00	0.54	0.00
27.56	109.19	2.00	0.00	0.53	0.00	27.72	98.23	2.00	0.00	0.53	0.00
27.89	92.32	2.00	0.00	0.53	0.00	28.05	88.86	2.00	0.00	0.52	0.00
28.22	95.09	2.00	0.00	0.52	0.00	28.38	106.08	2.00	0.00	0.52	0.00
28.54	122.69	2.00	0.00	0.52	0.00	28.71	133.12	2.00	0.00	0.51	0.00
28.87	136.23	2.00	0.00	0.51	0.00	29.04	133.79	2.00	0.00	0.51	0.00
29.20	129.43	2.00	0.00	0.51	0.00	29.36	125.57	2.00	0.00	0.50	0.00
29.53	117.23	2.00	0.00	0.50	0.00	29.69	109.92	2.00	0.00	0.50	0.00
29.86	106.77	2.00	0.00	0.49	0.00	30.02	112.37	2.00	0.00	0.49	0.00
30.18	121.82	2.00	0.00	0.49	0.00	30.35	128.15	2.00	0.00	0.49	0.00
30.51	130.28	2.00	0.00	0.48	0.00	30.68	128.55	2.00	0.00	0.48	0.00
30.84	127.90	2.00	0.00	0.48	0.00	31.00	128.90	2.00	0.00	0.47	0.00
31.17	129.46	2.00	0.00	0.47	0.00	31.33	130.11	2.00	0.00	0.47	0.00
31.50	131.21	2.00	0.00	0.47	0.00	31.66	132.76	2.00	0.00	0.46	0.00
31.82	132.84	2.00	0.00	0.46	0.00	31.99	130.61	2.00	0.00	0.46	0.00
32.15	128.77	2.00	0.00	0.46	0.00	32.32	128.58	2.00	0.00	0.45	0.00
32.48	128.48	2.00	0.00	0.45	0.00	32.64	129.42	2.00	0.00	0.45	0.00
32.81	129.75	2.00	0.00	0.44	0.00	32.97	130.97	2.00	0.00	0.44	0.00
33.14	131.00	2.00	0.00	0.44	0.00	33.30	128.72	2.00	0.00	0.44	0.00
33.46	123.93	2.00	0.00	0.43	0.00	33.63	116.35	2.00	0.00	0.43	0.00
33.79	110.99	2.00	0.00	0.43	0.00	33.96	110.60	2.00	0.00	0.42	0.00
34.12	111.67	2.00	0.00	0.42	0.00	34.28	110.30	2.00	0.00	0.42	0.00
34.45	105.43	2.00	0.00	0.42	0.00	34.61	103.69	2.00	0.00	0.41	0.00
34.78	102.82	2.00	0.00	0.41	0.00	34.94	102.04	2.00	0.00	0.41	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
35.10	98.27	2.00	0.00	0.41	0.00	35.27	98.22	2.00	0.00	0.40	0.00
35.43	100.94	2.00	0.00	0.40	0.00	35.60	101.34	2.00	0.00	0.40	0.00
35.76	100.29	2.00	0.00	0.39	0.00	35.93	105.95	2.00	0.00	0.39	0.00
36.09	125.65	2.00	0.00	0.39	0.00	36.25	138.16	2.00	0.00	0.39	0.00
36.42	135.24	2.00	0.00	0.38	0.00	36.58	123.84	2.00	0.00	0.38	0.00
36.75	137.16	2.00	0.00	0.38	0.00	36.91	161.11	2.00	0.00	0.37	0.00
37.07	178.60	0.74	0.40	0.37	0.01	37.24	194.85	0.94	0.22	0.37	0.00
37.40	211.40	2.00	0.00	0.37	0.00	37.57	225.24	2.00	0.00	0.36	0.00
37.73	234.94	2.00	0.00	0.36	0.00	37.89	240.12	2.00	0.00	0.36	0.00
38.06	244.41	2.00	0.00	0.35	0.00	38.22	249.20	2.00	0.00	0.35	0.00
38.39	255.03	2.00	0.00	0.35	0.00	38.55	257.45	2.00	0.00	0.35	0.00
38.71	256.52	2.00	0.00	0.34	0.00	38.88	257.00	2.00	0.00	0.34	0.00
39.04	261.98	2.00	0.00	0.34	0.00	39.21	269.58	2.00	0.00	0.34	0.00
39.37	269.57	2.00	0.00	0.33	0.00	39.53	252.47	2.00	0.00	0.33	0.00
39.70	207.07	2.00	0.00	0.33	0.00	39.86	142.59	0.43	0.57	0.32	0.01
40.03	105.57	2.00	0.00	0.32	0.00	40.19	94.27	2.00	0.00	0.32	0.00
40.35	92.51	2.00	0.00	0.32	0.00	40.52	86.38	2.00	0.00	0.31	0.00
40.68	82.93	2.00	0.00	0.31	0.00	40.85	78.02	2.00	0.00	0.31	0.00
41.01	73.79	2.00	0.00	0.30	0.00	41.17	72.38	2.00	0.00	0.30	0.00
41.34	72.43	2.00	0.00	0.30	0.00	41.50	73.81	2.00	0.00	0.30	0.00
41.67	75.32	2.00	0.00	0.29	0.00	41.83	75.91	2.00	0.00	0.29	0.00
41.99	75.81	2.00	0.00	0.29	0.00	42.16	75.97	2.00	0.00	0.29	0.00
42.32	75.70	2.00	0.00	0.28	0.00	42.49	73.89	2.00	0.00	0.28	0.00
42.65	70.02	2.00	0.00	0.28	0.00	42.81	65.88	2.00	0.00	0.27	0.00
42.98	63.59	2.00	0.00	0.27	0.00	43.14	63.13	2.00	0.00	0.27	0.00
43.31	63.18	2.00	0.00	0.27	0.00	43.47	62.23	2.00	0.00	0.26	0.00
43.64	60.24	2.00	0.00	0.26	0.00	43.80	57.69	2.00	0.00	0.26	0.00
43.96	56.79	2.00	0.00	0.25	0.00	44.13	58.08	2.00	0.00	0.25	0.00
44.29	61.70	2.00	0.00	0.25	0.00	44.46	64.07	2.00	0.00	0.25	0.00
44.62	65.43	2.00	0.00	0.24	0.00	44.78	64.79	2.00	0.00	0.24	0.00
44.95	64.33	2.00	0.00	0.24	0.00	45.11	64.11	2.00	0.00	0.24	0.00
45.28	65.31	2.00	0.00	0.23	0.00	45.44	67.72	2.00	0.00	0.23	0.00
45.60	69.66	2.00	0.00	0.23	0.00	45.77	70.53	2.00	0.00	0.22	0.00
45.93	69.65	2.00	0.00	0.22	0.00	46.10	67.27	2.00	0.00	0.22	0.00
46.26	67.30	2.00	0.00	0.22	0.00	46.42	68.57	2.00	0.00	0.21	0.00
46.59	70.85	2.00	0.00	0.21	0.00	46.75	73.85	2.00	0.00	0.21	0.00
46.92	81.22	2.00	0.00	0.20	0.00	47.08	88.60	2.00	0.00	0.20	0.00
47.24	92.73	2.00	0.00	0.20	0.00	47.41	89.22	2.00	0.00	0.20	0.00
47.57	84.35	2.00	0.00	0.19	0.00	47.74	79.61	2.00	0.00	0.19	0.00
47.90	78.96	2.00	0.00	0.19	0.00	48.06	89.68	2.00	0.00	0.19	0.00
48.23	106.01	2.00	0.00	0.18	0.00	48.39	116.10	2.00	0.00	0.18	0.00
48.56	115.82	2.00	0.00	0.18	0.00	48.72	112.91	2.00	0.00	0.17	0.00
48.88	116.01	2.00	0.00	0.17	0.00	49.05	118.48	2.00	0.00	0.17	0.00
49.21	117.25	2.00	0.00	0.17	0.00	49.38	112.79	2.00	0.00	0.16	0.00
49.54	107.80	2.00	0.00	0.16	0.00	49.70	107.11	2.00	0.00	0.16	0.00
49.87	110.54	2.00	0.00	0.15	0.00	50.03	117.09	2.00	0.00	0.15	0.00
50.20	123.31	2.00	0.00	0.15	0.00	50.36	128.72	2.00	0.00	0.15	0.00
50.52	132.12	2.00	0.00	0.14	0.00	50.69	133.56	2.00	0.00	0.14	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
50.85	132.99	2.00	0.00	0.14	0.00	51.02	132.46	2.00	0.00	0.14	0.00
51.18	132.43	2.00	0.00	0.13	0.00	51.35	132.44	2.00	0.00	0.13	0.00
51.51	131.09	2.00	0.00	0.13	0.00	51.67	128.05	2.00	0.00	0.12	0.00
51.84	123.99	2.00	0.00	0.12	0.00	52.00	118.47	2.00	0.00	0.12	0.00
52.17	112.20	2.00	0.00	0.12	0.00	52.33	105.67	2.00	0.00	0.11	0.00
52.49	100.74	2.00	0.00	0.11	0.00	52.66	96.10	2.00	0.00	0.11	0.00
52.82	94.47	2.00	0.00	0.10	0.00	52.99	96.46	2.00	0.00	0.10	0.00
53.15	102.46	2.00	0.00	0.10	0.00	53.31	107.11	2.00	0.00	0.10	0.00
53.48	107.89	2.00	0.00	0.09	0.00	53.64	104.74	2.00	0.00	0.09	0.00
53.81	99.57	2.00	0.00	0.09	0.00	53.97	94.19	2.00	0.00	0.09	0.00
54.13	88.75	2.00	0.00	0.08	0.00	54.30	83.73	2.00	0.00	0.08	0.00
54.46	77.28	2.00	0.00	0.08	0.00	54.63	71.55	2.00	0.00	0.07	0.00
54.79	65.97	2.00	0.00	0.07	0.00	54.95	65.13	2.00	0.00	0.07	0.00
55.12	65.46	2.00	0.00	0.07	0.00	55.28	68.25	2.00	0.00	0.06	0.00
55.45	68.36	2.00	0.00	0.06	0.00	55.61	73.17	2.00	0.00	0.06	0.00
55.77	74.60	2.00	0.00	0.05	0.00	55.94	74.22	2.00	0.00	0.05	0.00
56.10	69.95	2.00	0.00	0.05	0.00	56.27	66.30	2.00	0.00	0.05	0.00
56.43	66.36	2.00	0.00	0.04	0.00	56.59	60.47	2.00	0.00	0.04	0.00
56.76	56.78	2.00	0.00	0.04	0.00	56.92	54.16	2.00	0.00	0.04	0.00
57.09	57.88	2.00	0.00	0.03	0.00	57.25	61.09	2.00	0.00	0.03	0.00
57.41	64.58	2.00	0.00	0.03	0.00	57.58	67.35	2.00	0.00	0.02	0.00
57.74	71.08	2.00	0.00	0.02	0.00	57.91	75.28	2.00	0.00	0.02	0.00
58.07	81.25	2.00	0.00	0.02	0.00	58.23	87.49	2.00	0.00	0.01	0.00
58.40	92.37	2.00	0.00	0.01	0.00	58.56	93.06	2.00	0.00	0.01	0.00
58.73	89.87	2.00	0.00	0.00	0.00	58.89	83.36	2.00	0.00	0.00	0.00
59.06	81.94	2.00	0.00	0.00	0.00	59.22	84.29	2.00	0.00	0.00	0.00
59.38	97.69	2.00	0.00	0.00	0.00	59.55	104.53	2.00	0.00	0.00	0.00
59.71	105.32	2.00	0.00	0.00	0.00	59.88	92.82	2.00	0.00	0.00	0.00
60.04	100.29	2.00	0.00	0.00	0.00	60.20	110.17	2.00	0.00	0.00	0.00
60.37	111.27	2.00	0.00	0.00	0.00	60.53	111.33	2.00	0.00	0.00	0.00
60.70	106.08	2.00	0.00	0.00	0.00	60.86	98.88	2.00	0.00	0.00	0.00
61.02	86.71	2.00	0.00	0.00	0.00	61.19	75.63	2.00	0.00	0.00	0.00
61.35	66.91	2.00	0.00	0.00	0.00	61.52	63.17	2.00	0.00	0.00	0.00
61.68	62.76	2.00	0.00	0.00	0.00	61.84	63.23	2.00	0.00	0.00	0.00
62.01	65.01	2.00	0.00	0.00	0.00	62.17	66.69	2.00	0.00	0.00	0.00
62.34	66.96	2.00	0.00	0.00	0.00	62.50	64.63	2.00	0.00	0.00	0.00
62.66	60.54	2.00	0.00	0.00	0.00	62.83	56.94	2.00	0.00	0.00	0.00
62.99	54.55	2.00	0.00	0.00	0.00	63.16	52.97	2.00	0.00	0.00	0.00
63.32	59.81	2.00	0.00	0.00	0.00	63.48	79.60	2.00	0.00	0.00	0.00
63.65	102.51	2.00	0.00	0.00	0.00	63.81	118.64	2.00	0.00	0.00	0.00
63.98	121.64	2.00	0.00	0.00	0.00	64.14	122.21	2.00	0.00	0.00	0.00
64.30	127.63	2.00	0.00	0.00	0.00	64.47	131.78	0.43	0.00	0.00	0.00
64.63	126.45	0.39	0.00	0.00	0.00	64.80	121.35	0.36	0.00	0.00	0.00
64.96	119.62	0.35	0.00	0.00	0.00	65.12	121.82	0.36	0.00	0.00	0.00
65.29	121.74	0.36	0.00	0.00	0.00	65.45	124.68	2.00	0.00	0.00	0.00
65.62	129.78	2.00	0.00	0.00	0.00	65.78	135.07	2.00	0.00	0.00	0.00
65.94	139.47	2.00	0.00	0.00	0.00	66.11	143.00	2.00	0.00	0.00	0.00
66.27	136.67	2.00	0.00	0.00	0.00	66.44	124.36	2.00	0.00	0.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
66.60	109.40	2.00	0.00	0.00	0.00	66.77	95.67	2.00	0.00	0.00	0.00
66.93	83.88	2.00	0.00	0.00	0.00	67.09	74.55	2.00	0.00	0.00	0.00
67.26	68.37	2.00	0.00	0.00	0.00	67.42	64.09	2.00	0.00	0.00	0.00
67.59	63.52	2.00	0.00	0.00	0.00	67.75	66.01	2.00	0.00	0.00	0.00
67.91	68.63	2.00	0.00	0.00	0.00	68.08	70.15	2.00	0.00	0.00	0.00
68.24	70.69	2.00	0.00	0.00	0.00	68.41	71.85	2.00	0.00	0.00	0.00
68.57	71.52	2.00	0.00	0.00	0.00	68.73	68.98	2.00	0.00	0.00	0.00
68.90	64.39	2.00	0.00	0.00	0.00	69.06	60.20	2.00	0.00	0.00	0.00
69.23	57.71	2.00	0.00	0.00	0.00	69.39	57.25	2.00	0.00	0.00	0.00
69.55	57.72	2.00	0.00	0.00	0.00	69.72	59.19	2.00	0.00	0.00	0.00
69.88	60.83	2.00	0.00	0.00	0.00	70.05	62.64	2.00	0.00	0.00	0.00
70.21	62.52	2.00	0.00	0.00	0.00	70.37	61.08	2.00	0.00	0.00	0.00
70.54	57.85	2.00	0.00	0.00	0.00	70.70	54.67	2.00	0.00	0.00	0.00
70.87	51.05	2.00	0.00	0.00	0.00	71.03	49.35	2.00	0.00	0.00	0.00
71.19	51.75	2.00	0.00	0.00	0.00	71.36	56.45	2.00	0.00	0.00	0.00
71.52	61.31	2.00	0.00	0.00	0.00	71.69	62.65	2.00	0.00	0.00	0.00
71.85	62.56	2.00	0.00	0.00	0.00	72.01	60.83	2.00	0.00	0.00	0.00
72.18	61.63	2.00	0.00	0.00	0.00	72.34	61.18	2.00	0.00	0.00	0.00
72.51	60.79	2.00	0.00	0.00	0.00	72.67	57.76	2.00	0.00	0.00	0.00
72.83	55.72	2.00	0.00	0.00	0.00	73.00	53.95	2.00	0.00	0.00	0.00
73.16	54.35	2.00	0.00	0.00	0.00	73.33	56.57	2.00	0.00	0.00	0.00
73.49	59.61	2.00	0.00	0.00	0.00	73.65	61.75	2.00	0.00	0.00	0.00
73.82	62.20	2.00	0.00	0.00	0.00	73.98	61.61	2.00	0.00	0.00	0.00
74.15	61.13	2.00	0.00	0.00	0.00	74.31	60.71	2.00	0.00	0.00	0.00
74.48	58.18	2.00	0.00	0.00	0.00	74.64	56.85	2.00	0.00	0.00	0.00
74.80	56.07	2.00	0.00	0.00	0.00	74.97	57.45	2.00	0.00	0.00	0.00
75.13	56.74	2.00	0.00	0.00	0.00	75.30	55.99	2.00	0.00	0.00	0.00
75.46	56.38	2.00	0.00	0.00	0.00	75.62	55.20	2.00	0.00	0.00	0.00
75.79	52.80	2.00	0.00	0.00	0.00	75.95	48.64	2.00	0.00	0.00	0.00
76.12	47.50	2.00	0.00	0.00	0.00	76.28	50.07	2.00	0.00	0.00	0.00
76.44	55.73	2.00	0.00	0.00	0.00	76.61	67.97	2.00	0.00	0.00	0.00
76.77	80.22	2.00	0.00	0.00	0.00	76.94	88.48	2.00	0.00	0.00	0.00
77.10	92.28	2.00	0.00	0.00	0.00	77.26	91.13	2.00	0.00	0.00	0.00
77.43	88.93	2.00	0.00	0.00	0.00	77.59	85.54	2.00	0.00	0.00	0.00
77.76	85.37	2.00	0.00	0.00	0.00	77.92	88.16	2.00	0.00	0.00	0.00
78.08	90.39	2.00	0.00	0.00	0.00	78.25	88.06	2.00	0.00	0.00	0.00
78.41	81.05	2.00	0.00	0.00	0.00	78.58	72.37	2.00	0.00	0.00	0.00
78.74	66.81	2.00	0.00	0.00	0.00	78.90	64.10	2.00	0.00	0.00	0.00
79.07	63.62	2.00	0.00	0.00	0.00	79.23	62.66	2.00	0.00	0.00	0.00
79.40	61.54	2.00	0.00	0.00	0.00	79.56	61.02	2.00	0.00	0.00	0.00
79.72	62.92	2.00	0.00	0.00	0.00	79.89	65.49	2.00	0.00	0.00	0.00
80.05	65.95	2.00	0.00	0.00	0.00	80.22	-1.00	2.00	0.00	0.00	0.00
80.38	-1.00	2.00	0.00	0.00	0.00	80.54	-1.00	2.00	0.00	0.00	0.00
80.71	-1.00	2.00	0.00	0.00	0.00						

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)

Total estimated settlement: 0.32

Abbreviations

$Q_{tn,cs}$:	Equivalent clean sand normalized cone resistance
FS:	Factor of safety against liquefaction
e_v (%):	Post-liquefaction volumentric strain
DF:	e_v depth weighting factor
Settlement:	Calculated settlement

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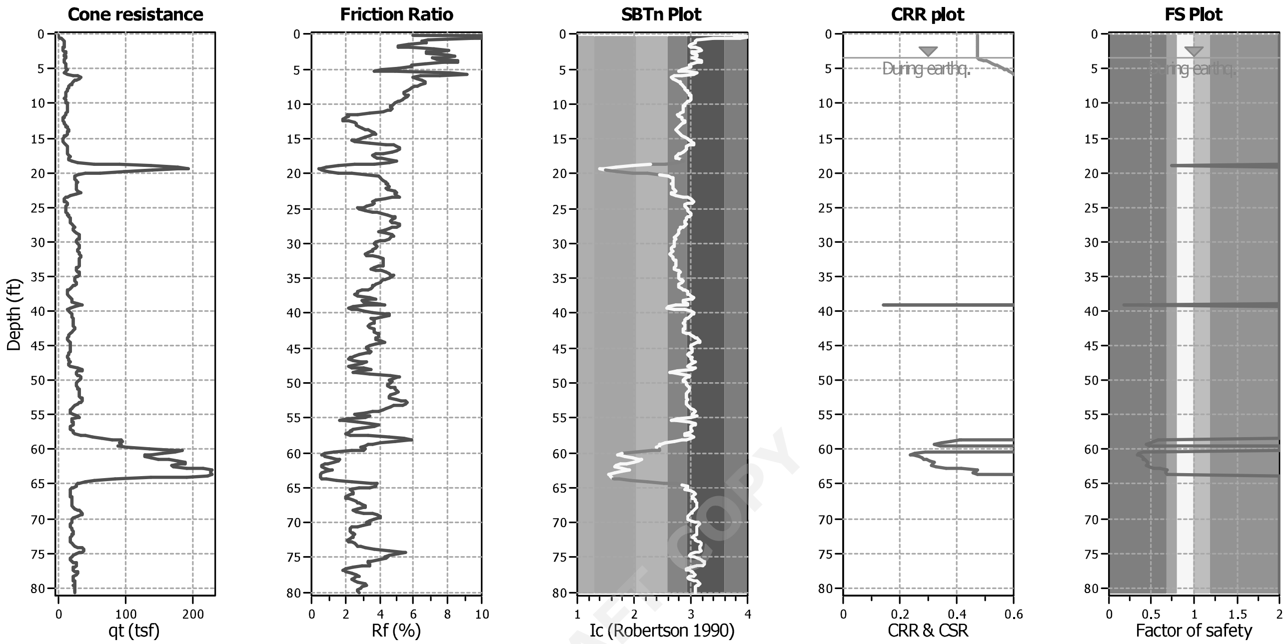
LIQUEFACTION ANALYSIS REPORT

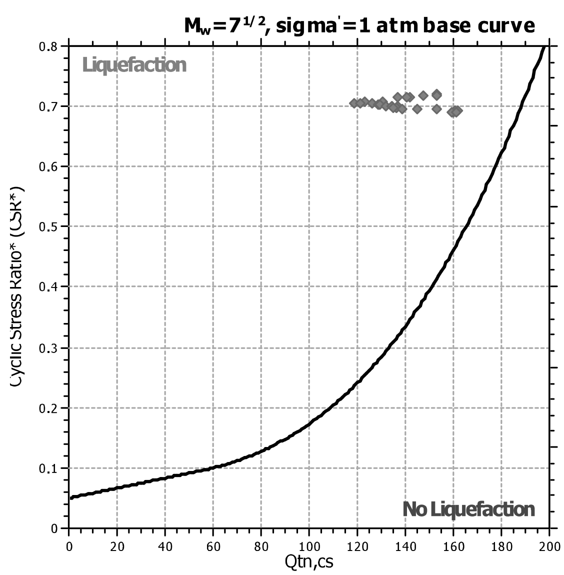
Project title : 5200 Patrick Henry Drive, LLC **Location : 5200 Patrick Henry Drive, Santa Clara**

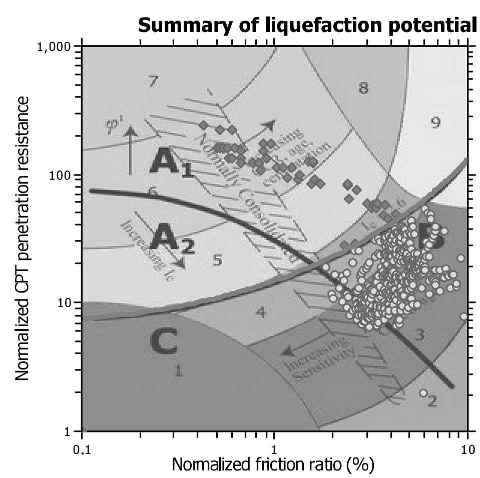
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.30 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.50 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:	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.61	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

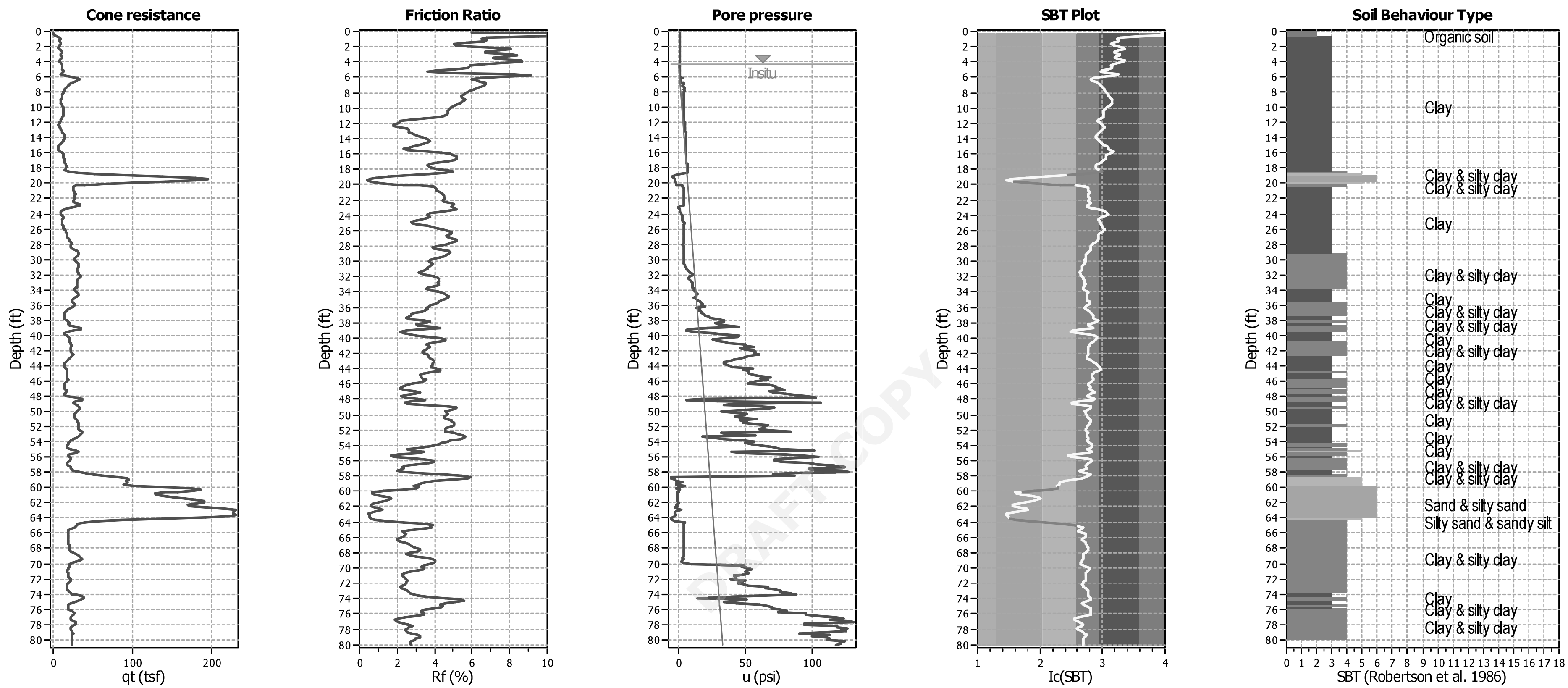






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



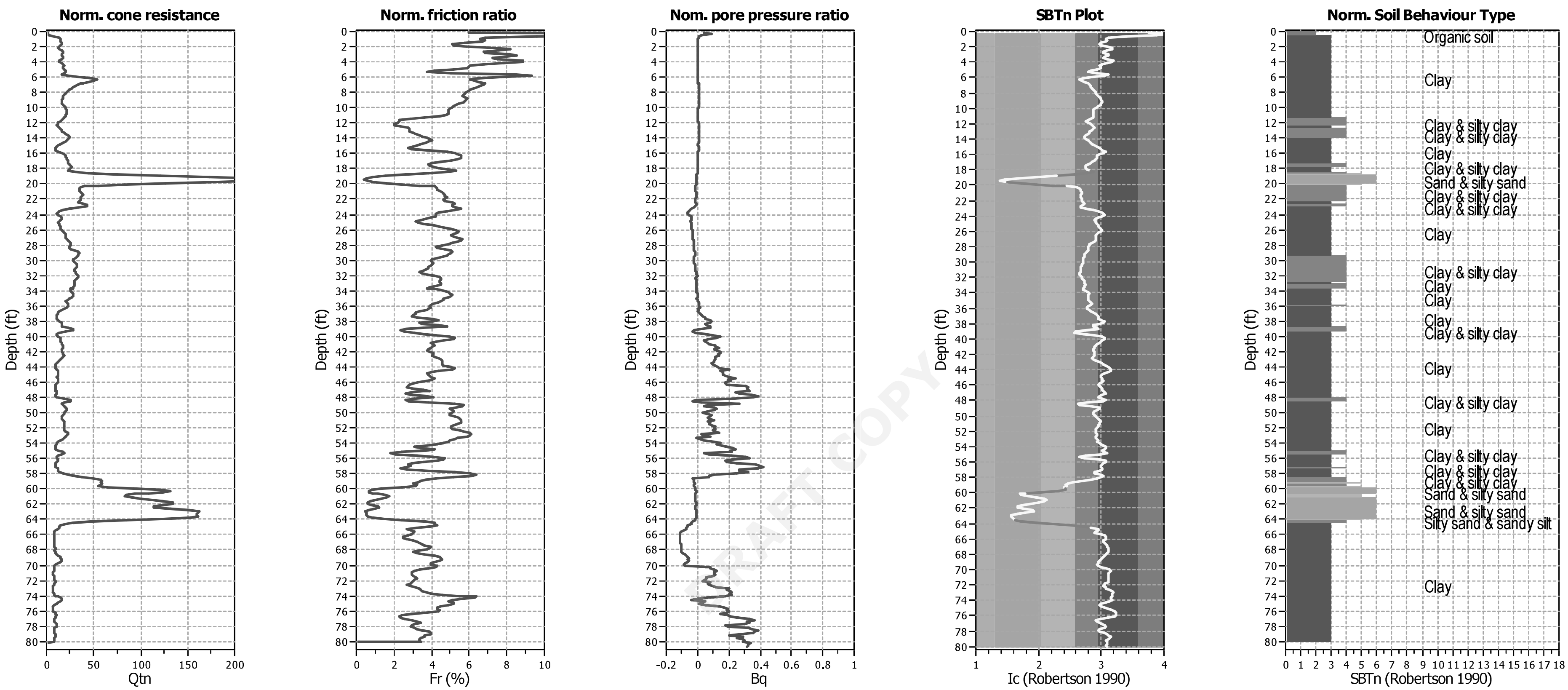
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A		
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes		
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	4.30 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)



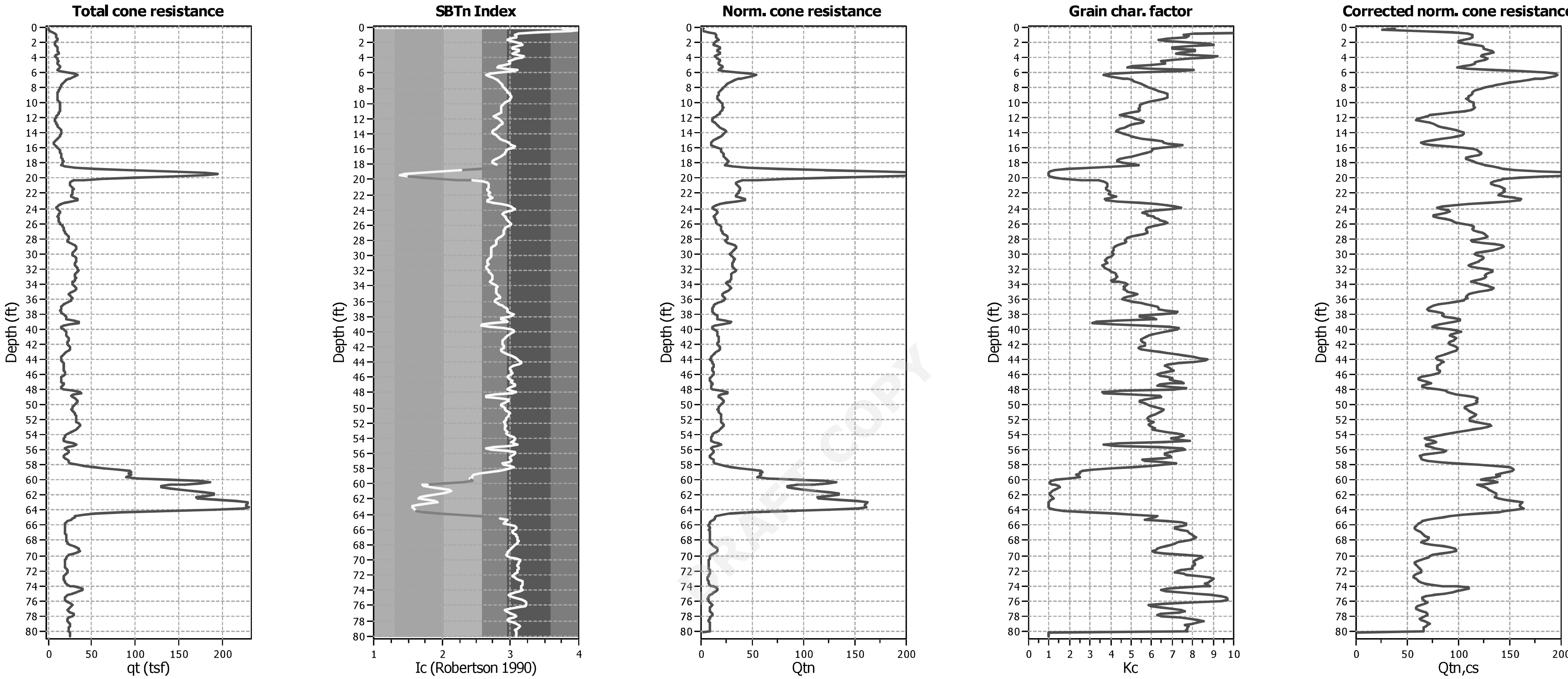
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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

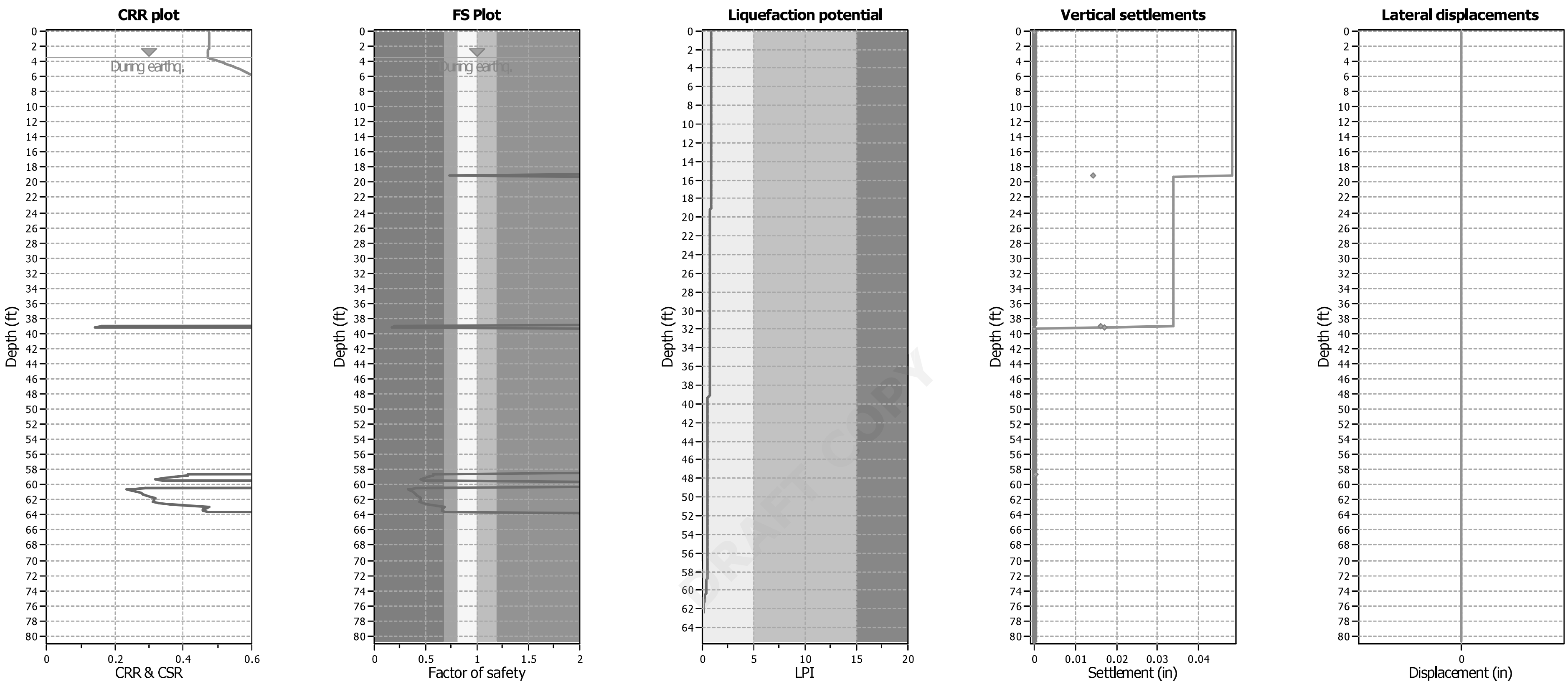
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

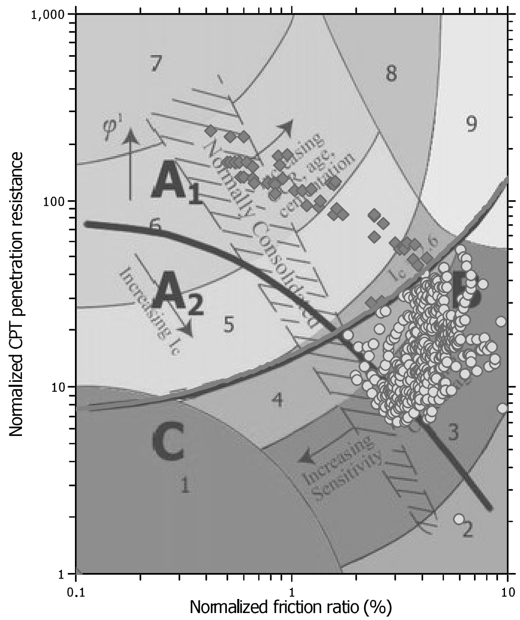
F.S. color scheme

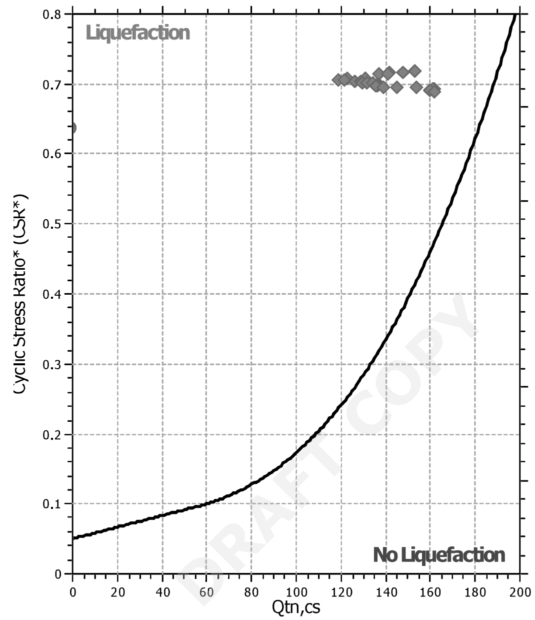
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

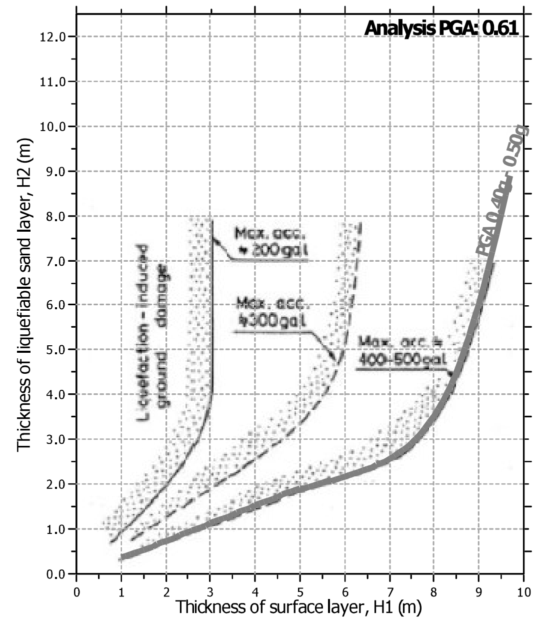
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



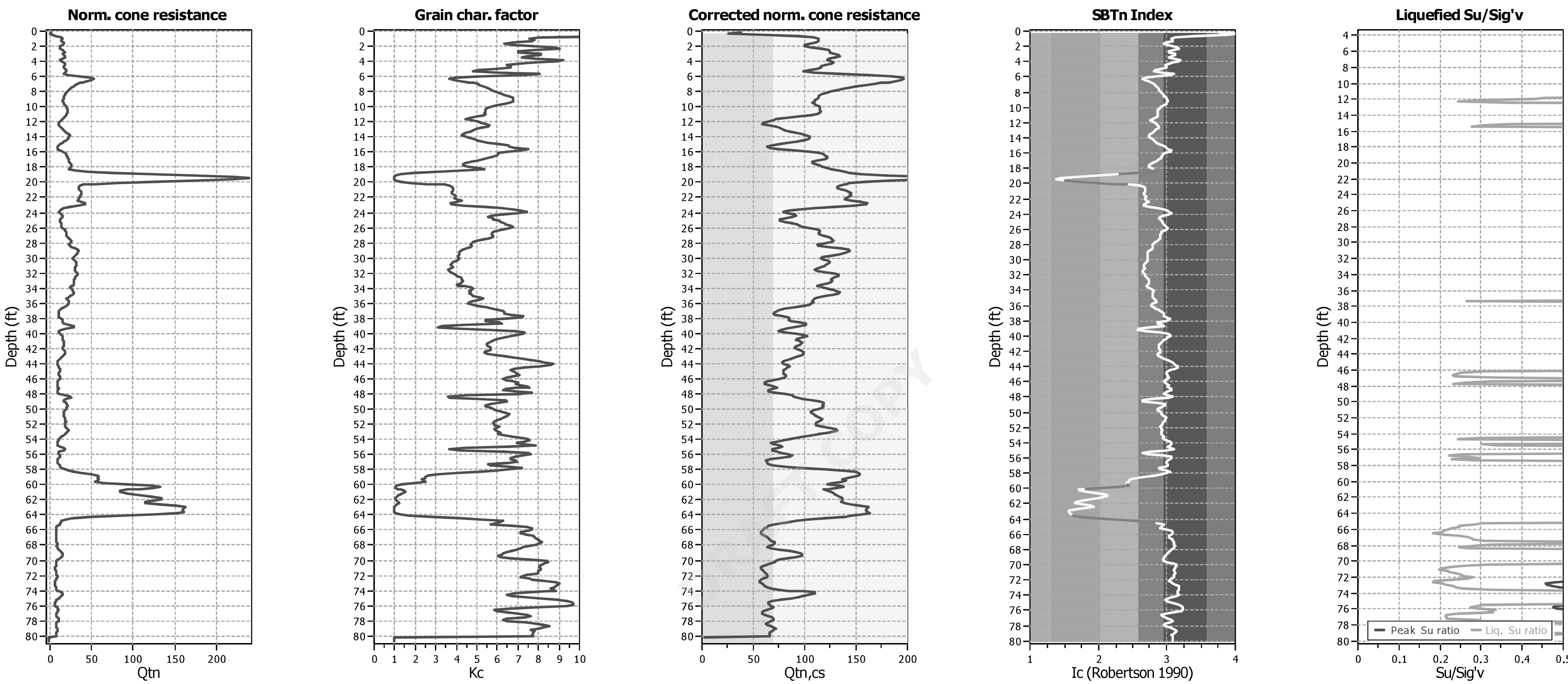




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))

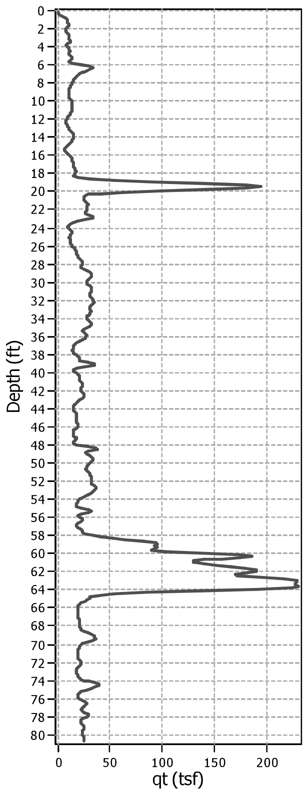


Input parameters and analysis data

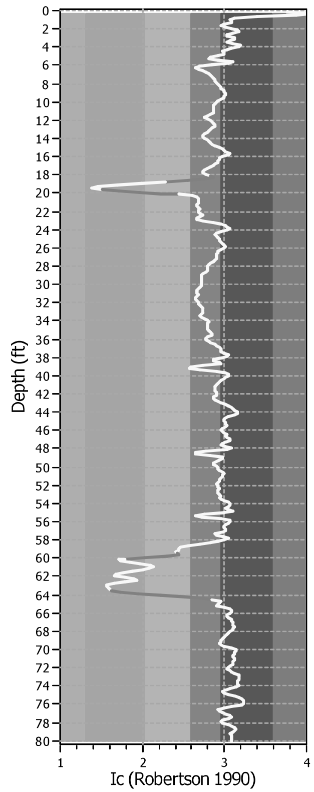
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

Estimation of post-earthquake settlements

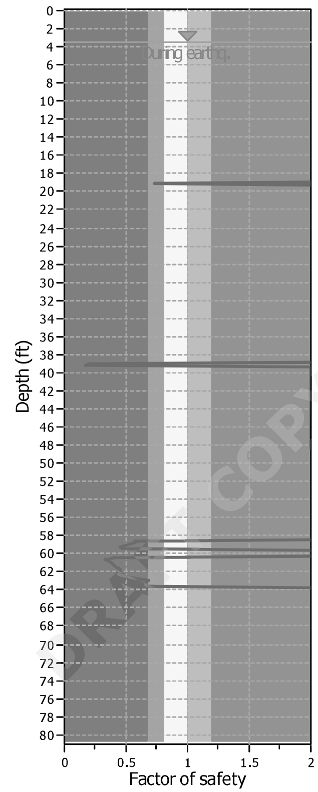
Cone resistance



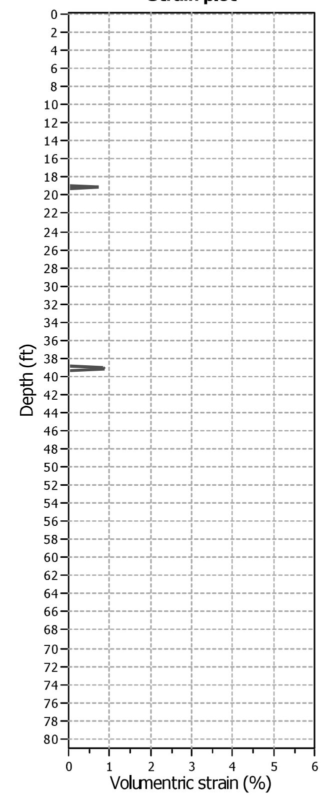
SBTn Plot



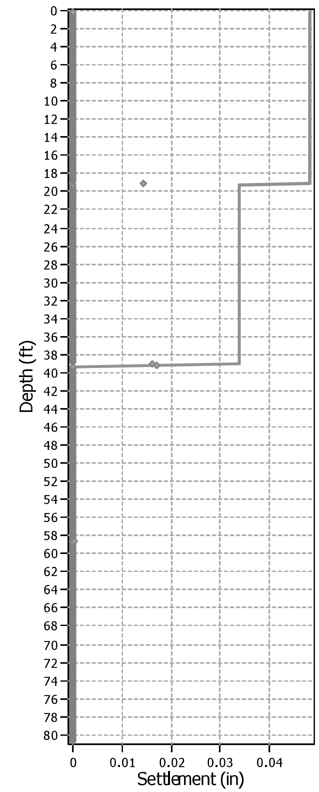
FS Plot



Strain plot



Vertical settlements



Abbreviations

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

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
3.61	122.88	2.00	0.00	0.94	0.00	3.77	121.47	2.00	0.00	0.94	0.00
3.94	125.43	2.00	0.00	0.93	0.00	4.10	128.44	2.00	0.00	0.93	0.00
4.27	125.03	2.00	0.00	0.93	0.00	4.43	119.71	2.00	0.00	0.92	0.00
4.59	116.30	2.00	0.00	0.92	0.00	4.76	115.46	2.00	0.00	0.92	0.00
4.92	112.25	2.00	0.00	0.92	0.00	5.09	105.07	2.00	0.00	0.91	0.00
5.25	99.30	2.00	0.00	0.91	0.00	5.41	108.58	2.00	0.00	0.91	0.00
5.58	132.66	2.00	0.00	0.91	0.00	5.74	162.74	2.00	0.00	0.90	0.00
5.91	185.44	2.00	0.00	0.90	0.00	6.07	195.25	2.00	0.00	0.90	0.00
6.23	196.78	2.00	0.00	0.89	0.00	6.40	194.31	2.00	0.00	0.89	0.00
6.56	187.71	2.00	0.00	0.89	0.00	6.73	181.47	2.00	0.00	0.89	0.00
6.89	174.52	2.00	0.00	0.88	0.00	7.05	167.79	2.00	0.00	0.88	0.00
7.22	159.24	2.00	0.00	0.88	0.00	7.38	150.54	2.00	0.00	0.87	0.00
7.55	143.40	2.00	0.00	0.87	0.00	7.71	136.22	2.00	0.00	0.87	0.00
7.87	131.32	2.00	0.00	0.87	0.00	8.04	125.93	2.00	0.00	0.86	0.00
8.20	122.38	2.00	0.00	0.86	0.00	8.37	117.63	2.00	0.00	0.86	0.00
8.53	115.04	2.00	0.00	0.86	0.00	8.69	113.56	2.00	0.00	0.85	0.00
8.86	113.63	2.00	0.00	0.85	0.00	9.02	111.99	2.00	0.00	0.85	0.00
9.19	110.17	2.00	0.00	0.84	0.00	9.35	109.25	2.00	0.00	0.84	0.00
9.51	107.86	2.00	0.00	0.84	0.00	9.68	109.55	2.00	0.00	0.84	0.00
9.84	111.18	2.00	0.00	0.83	0.00	10.01	114.11	2.00	0.00	0.83	0.00
10.17	114.52	2.00	0.00	0.83	0.00	10.33	114.63	2.00	0.00	0.82	0.00
10.50	115.22	2.00	0.00	0.82	0.00	10.66	115.52	2.00	0.00	0.82	0.00
10.83	114.86	2.00	0.00	0.82	0.00	10.99	111.80	2.00	0.00	0.81	0.00
11.15	106.60	2.00	0.00	0.81	0.00	11.32	94.03	2.00	0.00	0.81	0.00
11.48	82.87	2.00	0.00	0.81	0.00	11.65	71.63	2.00	0.00	0.80	0.00
11.81	69.39	2.00	0.00	0.80	0.00	11.98	64.60	2.00	0.00	0.80	0.00
12.14	59.98	2.00	0.00	0.79	0.00	12.30	58.78	2.00	0.00	0.79	0.00
12.47	64.25	2.00	0.00	0.79	0.00	12.63	71.71	2.00	0.00	0.79	0.00
12.80	76.15	2.00	0.00	0.78	0.00	12.96	78.16	2.00	0.00	0.78	0.00
13.12	80.07	2.00	0.00	0.78	0.00	13.29	84.86	2.00	0.00	0.77	0.00
13.45	91.00	2.00	0.00	0.77	0.00	13.62	97.48	2.00	0.00	0.77	0.00
13.78	101.80	2.00	0.00	0.77	0.00	13.94	104.69	2.00	0.00	0.76	0.00
14.11	105.07	2.00	0.00	0.76	0.00	14.27	103.92	2.00	0.00	0.76	0.00
14.44	98.92	2.00	0.00	0.76	0.00	14.60	92.13	2.00	0.00	0.75	0.00
14.76	83.37	2.00	0.00	0.75	0.00	14.93	75.50	2.00	0.00	0.75	0.00
15.09	68.62	2.00	0.00	0.74	0.00	15.26	63.46	2.00	0.00	0.74	0.00
15.42	65.34	2.00	0.00	0.74	0.00	15.58	74.86	2.00	0.00	0.74	0.00
15.75	90.67	2.00	0.00	0.73	0.00	15.91	105.35	2.00	0.00	0.73	0.00
16.08	114.66	2.00	0.00	0.73	0.00	16.24	118.56	2.00	0.00	0.72	0.00
16.40	119.41	2.00	0.00	0.72	0.00	16.57	121.53	2.00	0.00	0.72	0.00
16.73	122.09	2.00	0.00	0.72	0.00	16.90	119.52	2.00	0.00	0.71	0.00
17.06	112.43	2.00	0.00	0.71	0.00	17.22	107.44	2.00	0.00	0.71	0.00
17.39	106.72	2.00	0.00	0.71	0.00	17.55	111.22	2.00	0.00	0.70	0.00
17.72	114.83	2.00	0.00	0.70	0.00	17.88	117.37	2.00	0.00	0.70	0.00
18.04	121.78	2.00	0.00	0.69	0.00	18.21	125.51	2.00	0.00	0.69	0.00
18.37	133.33	2.00	0.00	0.69	0.00	18.54	139.33	2.00	0.00	0.69	0.00
18.70	143.66	2.00	0.00	0.68	0.00	18.86	155.00	2.00	0.00	0.68	0.00
19.03	178.75	0.73	0.73	0.68	0.01	19.19	221.03	2.00	0.00	0.67	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
19.36	238.96	2.00	0.00	0.67	0.00	19.52	219.30	2.00	0.00	0.67	0.00
19.69	183.51	2.00	0.00	0.67	0.00	19.85	159.84	2.00	0.00	0.66	0.00
20.01	144.32	2.00	0.00	0.66	0.00	20.18	140.53	2.00	0.00	0.66	0.00
20.34	138.73	2.00	0.00	0.66	0.00	20.51	133.59	2.00	0.00	0.65	0.00
20.67	132.04	2.00	0.00	0.65	0.00	20.83	132.10	2.00	0.00	0.65	0.00
21.00	134.94	2.00	0.00	0.64	0.00	21.16	140.11	2.00	0.00	0.64	0.00
21.33	143.03	2.00	0.00	0.64	0.00	21.49	144.89	2.00	0.00	0.64	0.00
21.65	144.24	2.00	0.00	0.63	0.00	21.82	144.54	2.00	0.00	0.63	0.00
21.98	142.27	2.00	0.00	0.63	0.00	22.15	139.73	2.00	0.00	0.62	0.00
22.31	139.23	2.00	0.00	0.62	0.00	22.47	145.33	2.00	0.00	0.62	0.00
22.64	155.14	2.00	0.00	0.62	0.00	22.80	161.26	2.00	0.00	0.61	0.00
22.97	159.92	2.00	0.00	0.61	0.00	23.13	147.58	2.00	0.00	0.61	0.00
23.29	129.51	2.00	0.00	0.61	0.00	23.46	109.77	2.00	0.00	0.60	0.00
23.62	95.45	2.00	0.00	0.60	0.00	23.79	84.35	2.00	0.00	0.60	0.00
23.95	79.87	2.00	0.00	0.59	0.00	24.11	83.32	2.00	0.00	0.59	0.00
24.28	90.20	2.00	0.00	0.59	0.00	24.44	91.99	2.00	0.00	0.59	0.00
24.61	88.28	2.00	0.00	0.58	0.00	24.77	81.87	2.00	0.00	0.58	0.00
24.93	76.17	2.00	0.00	0.58	0.00	25.10	75.90	2.00	0.00	0.57	0.00
25.26	80.55	2.00	0.00	0.57	0.00	25.43	87.50	2.00	0.00	0.57	0.00
25.59	92.18	2.00	0.00	0.57	0.00	25.75	94.54	2.00	0.00	0.56	0.00
25.92	98.02	2.00	0.00	0.56	0.00	26.08	104.79	2.00	0.00	0.56	0.00
26.25	111.94	2.00	0.00	0.56	0.00	26.41	114.95	2.00	0.00	0.55	0.00
26.57	114.28	2.00	0.00	0.55	0.00	26.74	113.08	2.00	0.00	0.55	0.00
26.90	114.58	2.00	0.00	0.54	0.00	27.07	118.35	2.00	0.00	0.54	0.00
27.23	123.26	2.00	0.00	0.54	0.00	27.40	125.72	2.00	0.00	0.54	0.00
27.56	127.32	2.00	0.00	0.53	0.00	27.72	127.93	2.00	0.00	0.53	0.00
27.89	125.41	2.00	0.00	0.53	0.00	28.05	119.34	2.00	0.00	0.52	0.00
28.22	111.71	2.00	0.00	0.52	0.00	28.38	113.03	2.00	0.00	0.52	0.00
28.54	122.81	2.00	0.00	0.52	0.00	28.71	135.26	2.00	0.00	0.51	0.00
28.87	142.64	2.00	0.00	0.51	0.00	29.04	143.83	2.00	0.00	0.51	0.00
29.20	141.06	2.00	0.00	0.51	0.00	29.36	138.25	2.00	0.00	0.50	0.00
29.53	132.45	2.00	0.00	0.50	0.00	29.69	125.46	2.00	0.00	0.50	0.00
29.86	117.66	2.00	0.00	0.49	0.00	30.02	115.92	2.00	0.00	0.49	0.00
30.18	118.40	2.00	0.00	0.49	0.00	30.35	122.81	2.00	0.00	0.49	0.00
30.51	124.08	2.00	0.00	0.48	0.00	30.68	123.31	2.00	0.00	0.48	0.00
30.84	120.61	2.00	0.00	0.48	0.00	31.00	118.99	2.00	0.00	0.47	0.00
31.17	115.60	2.00	0.00	0.47	0.00	31.33	112.26	2.00	0.00	0.47	0.00
31.50	109.30	2.00	0.00	0.47	0.00	31.66	112.76	2.00	0.00	0.46	0.00
31.82	119.59	2.00	0.00	0.46	0.00	31.99	128.26	2.00	0.00	0.46	0.00
32.15	132.81	2.00	0.00	0.46	0.00	32.32	132.40	2.00	0.00	0.45	0.00
32.48	129.58	2.00	0.00	0.45	0.00	32.64	126.77	2.00	0.00	0.45	0.00
32.81	126.14	2.00	0.00	0.44	0.00	32.97	127.04	2.00	0.00	0.44	0.00
33.14	126.12	2.00	0.00	0.44	0.00	33.30	122.37	2.00	0.00	0.44	0.00
33.46	116.06	2.00	0.00	0.43	0.00	33.63	112.26	2.00	0.00	0.43	0.00
33.79	113.02	2.00	0.00	0.43	0.00	33.96	117.19	2.00	0.00	0.42	0.00
34.12	121.85	2.00	0.00	0.42	0.00	34.28	127.27	2.00	0.00	0.42	0.00
34.45	131.09	2.00	0.00	0.42	0.00	34.61	134.04	2.00	0.00	0.41	0.00
34.78	132.55	2.00	0.00	0.41	0.00	34.94	128.05	2.00	0.00	0.41	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
35.10	119.98	2.00	0.00	0.41	0.00	35.27	113.80	2.00	0.00	0.40	0.00
35.43	109.44	2.00	0.00	0.40	0.00	35.60	108.64	2.00	0.00	0.40	0.00
35.76	107.36	2.00	0.00	0.39	0.00	35.93	107.99	2.00	0.00	0.39	0.00
36.09	105.99	2.00	0.00	0.39	0.00	36.25	104.43	2.00	0.00	0.39	0.00
36.42	98.00	2.00	0.00	0.38	0.00	36.58	90.49	2.00	0.00	0.38	0.00
36.75	80.55	2.00	0.00	0.38	0.00	36.91	75.05	2.00	0.00	0.37	0.00
37.07	72.95	2.00	0.00	0.37	0.00	37.24	70.42	2.00	0.00	0.37	0.00
37.40	69.08	2.00	0.00	0.37	0.00	37.57	70.64	2.00	0.00	0.36	0.00
37.73	78.11	2.00	0.00	0.36	0.00	37.89	84.14	2.00	0.00	0.36	0.00
38.06	85.85	2.00	0.00	0.35	0.00	38.22	84.35	2.00	0.00	0.35	0.00
38.39	86.62	2.00	0.00	0.35	0.00	38.55	95.01	2.00	0.00	0.35	0.00
38.71	101.79	2.00	0.00	0.34	0.00	38.88	101.76	2.00	0.00	0.34	0.00
39.04	94.87	0.19	0.83	0.34	0.02	39.21	88.44	0.18	0.87	0.34	0.02
39.37	82.20	2.00	0.00	0.33	0.00	39.53	76.33	2.00	0.00	0.33	0.00
39.70	74.68	2.00	0.00	0.33	0.00	39.86	80.62	2.00	0.00	0.32	0.00
40.03	91.65	2.00	0.00	0.32	0.00	40.19	99.35	2.00	0.00	0.32	0.00
40.35	102.31	2.00	0.00	0.32	0.00	40.52	97.35	2.00	0.00	0.31	0.00
40.68	92.97	2.00	0.00	0.31	0.00	40.85	91.51	2.00	0.00	0.31	0.00
41.01	94.81	2.00	0.00	0.30	0.00	41.17	97.06	2.00	0.00	0.30	0.00
41.34	96.72	2.00	0.00	0.30	0.00	41.50	95.06	2.00	0.00	0.30	0.00
41.67	91.72	2.00	0.00	0.29	0.00	41.83	89.78	2.00	0.00	0.29	0.00
41.99	90.93	2.00	0.00	0.29	0.00	42.16	95.15	2.00	0.00	0.29	0.00
42.32	97.95	2.00	0.00	0.28	0.00	42.49	98.92	2.00	0.00	0.28	0.00
42.65	98.50	2.00	0.00	0.28	0.00	42.81	97.23	2.00	0.00	0.27	0.00
42.98	94.51	2.00	0.00	0.27	0.00	43.14	90.40	2.00	0.00	0.27	0.00
43.31	86.28	2.00	0.00	0.27	0.00	43.47	82.60	2.00	0.00	0.26	0.00
43.64	79.07	2.00	0.00	0.26	0.00	43.80	77.61	2.00	0.00	0.26	0.00
43.96	79.33	2.00	0.00	0.25	0.00	44.13	82.47	2.00	0.00	0.25	0.00
44.29	85.41	2.00	0.00	0.25	0.00	44.46	83.30	2.00	0.00	0.25	0.00
44.62	81.82	2.00	0.00	0.24	0.00	44.78	79.54	2.00	0.00	0.24	0.00
44.95	79.59	2.00	0.00	0.24	0.00	45.11	79.14	2.00	0.00	0.24	0.00
45.28	79.45	2.00	0.00	0.23	0.00	45.44	81.53	2.00	0.00	0.23	0.00
45.60	82.09	2.00	0.00	0.23	0.00	45.77	80.69	2.00	0.00	0.22	0.00
45.93	75.35	2.00	0.00	0.22	0.00	46.10	69.14	2.00	0.00	0.22	0.00
46.26	64.02	2.00	0.00	0.22	0.00	46.42	61.47	2.00	0.00	0.21	0.00
46.59	60.90	2.00	0.00	0.21	0.00	46.75	64.05	2.00	0.00	0.21	0.00
46.92	69.71	2.00	0.00	0.20	0.00	47.08	73.48	2.00	0.00	0.20	0.00
47.24	73.12	2.00	0.00	0.20	0.00	47.41	68.56	2.00	0.00	0.20	0.00
47.57	64.27	2.00	0.00	0.19	0.00	47.74	64.25	2.00	0.00	0.19	0.00
47.90	70.87	2.00	0.00	0.19	0.00	48.06	80.09	2.00	0.00	0.19	0.00
48.23	87.83	2.00	0.00	0.18	0.00	48.39	89.63	2.00	0.00	0.18	0.00
48.56	92.40	2.00	0.00	0.18	0.00	48.72	97.61	2.00	0.00	0.17	0.00
48.88	106.75	2.00	0.00	0.17	0.00	49.05	114.48	2.00	0.00	0.17	0.00
49.21	118.30	2.00	0.00	0.17	0.00	49.38	118.08	2.00	0.00	0.16	0.00
49.54	117.15	2.00	0.00	0.16	0.00	49.70	117.96	2.00	0.00	0.16	0.00
49.87	116.83	2.00	0.00	0.15	0.00	50.03	113.09	2.00	0.00	0.15	0.00
50.20	108.01	2.00	0.00	0.15	0.00	50.36	105.64	2.00	0.00	0.15	0.00
50.52	106.51	2.00	0.00	0.14	0.00	50.69	108.16	2.00	0.00	0.14	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
50.85	109.57	2.00	0.00	0.14	0.00	51.02	112.43	2.00	0.00	0.14	0.00
51.18	115.11	2.00	0.00	0.13	0.00	51.35	116.87	2.00	0.00	0.13	0.00
51.51	115.49	2.00	0.00	0.13	0.00	51.67	113.09	2.00	0.00	0.12	0.00
51.84	111.24	2.00	0.00	0.12	0.00	52.00	110.97	2.00	0.00	0.12	0.00
52.17	111.26	2.00	0.00	0.12	0.00	52.33	116.75	2.00	0.00	0.11	0.00
52.49	122.91	2.00	0.00	0.11	0.00	52.66	130.10	2.00	0.00	0.11	0.00
52.82	131.81	2.00	0.00	0.10	0.00	52.99	127.92	2.00	0.00	0.10	0.00
53.15	122.22	2.00	0.00	0.10	0.00	53.31	113.60	2.00	0.00	0.10	0.00
53.48	107.20	2.00	0.00	0.09	0.00	53.64	98.41	2.00	0.00	0.09	0.00
53.81	91.26	2.00	0.00	0.09	0.00	53.97	85.07	2.00	0.00	0.09	0.00
54.13	79.37	2.00	0.00	0.08	0.00	54.30	73.49	2.00	0.00	0.08	0.00
54.46	67.26	2.00	0.00	0.08	0.00	54.63	69.24	2.00	0.00	0.07	0.00
54.79	75.21	2.00	0.00	0.07	0.00	54.95	78.41	2.00	0.00	0.07	0.00
55.12	74.49	2.00	0.00	0.07	0.00	55.28	68.85	2.00	0.00	0.06	0.00
55.45	68.73	2.00	0.00	0.06	0.00	55.61	73.09	2.00	0.00	0.06	0.00
55.77	77.31	2.00	0.00	0.05	0.00	55.94	84.57	2.00	0.00	0.05	0.00
56.10	87.71	2.00	0.00	0.05	0.00	56.27	85.26	2.00	0.00	0.05	0.00
56.43	76.00	2.00	0.00	0.04	0.00	56.59	66.17	2.00	0.00	0.04	0.00
56.76	62.16	2.00	0.00	0.04	0.00	56.92	63.49	2.00	0.00	0.04	0.00
57.09	63.73	2.00	0.00	0.03	0.00	57.25	64.46	2.00	0.00	0.03	0.00
57.41	69.65	2.00	0.00	0.03	0.00	57.58	81.69	2.00	0.00	0.02	0.00
57.74	96.86	2.00	0.00	0.02	0.00	57.91	118.04	2.00	0.00	0.02	0.00
58.07	135.74	2.00	0.00	0.02	0.00	58.23	149.68	2.00	0.00	0.01	0.00
58.40	152.33	2.00	0.00	0.01	0.00	58.56	153.05	0.57	0.01	0.01	0.00
58.73	152.95	0.57	0.01	0.00	0.00	58.89	147.47	0.53	0.00	0.00	0.00
59.06	141.89	0.48	0.00	0.00	0.00	59.22	136.94	0.45	0.00	0.00	0.00
59.38	140.80	0.48	0.00	0.00	0.00	59.55	136.56	2.00	0.00	0.00	0.00
59.71	127.01	2.00	0.00	0.00	0.00	59.88	121.91	2.00	0.00	0.00	0.00
60.04	134.69	2.00	0.00	0.00	0.00	60.20	137.38	2.00	0.00	0.00	0.00
60.37	130.93	0.41	0.00	0.00	0.00	60.53	123.02	0.36	0.00	0.00	0.00
60.70	118.75	0.33	0.00	0.00	0.00	60.86	121.54	0.35	0.00	0.00	0.00
61.02	126.02	0.38	0.00	0.00	0.00	61.19	128.64	0.40	0.00	0.00	0.00
61.35	129.52	0.40	0.00	0.00	0.00	61.52	131.66	0.42	0.00	0.00	0.00
61.68	134.22	0.44	0.00	0.00	0.00	61.84	136.97	0.46	0.00	0.00	0.00
62.01	136.36	0.45	0.00	0.00	0.00	62.17	136.38	0.45	0.00	0.00	0.00
62.34	135.25	0.44	0.00	0.00	0.00	62.50	138.71	0.47	0.00	0.00	0.00
62.66	144.88	0.52	0.00	0.00	0.00	62.83	153.43	0.60	0.00	0.00	0.00
62.99	161.91	0.68	0.00	0.00	0.00	63.16	160.90	0.68	0.00	0.00	0.00
63.32	159.71	0.66	0.00	0.00	0.00	63.48	159.56	0.66	0.00	0.00	0.00
63.65	161.51	0.68	0.00	0.00	0.00	63.81	162.83	2.00	0.00	0.00	0.00
63.98	155.95	2.00	0.00	0.00	0.00	64.14	145.63	2.00	0.00	0.00	0.00
64.30	138.70	2.00	0.00	0.00	0.00	64.47	119.14	2.00	0.00	0.00	0.00
64.63	103.49	2.00	0.00	0.00	0.00	64.80	91.99	2.00	0.00	0.00	0.00
64.96	85.89	2.00	0.00	0.00	0.00	65.12	76.83	2.00	0.00	0.00	0.00
65.29	69.15	2.00	0.00	0.00	0.00	65.45	65.06	2.00	0.00	0.00	0.00
65.62	63.11	2.00	0.00	0.00	0.00	65.78	61.29	2.00	0.00	0.00	0.00
65.94	60.03	2.00	0.00	0.00	0.00	66.11	58.56	2.00	0.00	0.00	0.00
66.27	57.01	2.00	0.00	0.00	0.00	66.44	56.93	2.00	0.00	0.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
66.60	58.39	2.00	0.00	0.00	0.00	66.77	61.23	2.00	0.00	0.00	0.00
66.93	63.52	2.00	0.00	0.00	0.00	67.09	64.57	2.00	0.00	0.00	0.00
67.26	65.75	2.00	0.00	0.00	0.00	67.42	67.38	2.00	0.00	0.00	0.00
67.59	70.17	2.00	0.00	0.00	0.00	67.75	70.67	2.00	0.00	0.00	0.00
67.91	69.46	2.00	0.00	0.00	0.00	68.08	65.68	2.00	0.00	0.00	0.00
68.24	63.96	2.00	0.00	0.00	0.00	68.41	67.29	2.00	0.00	0.00	0.00
68.57	74.73	2.00	0.00	0.00	0.00	68.73	85.02	2.00	0.00	0.00	0.00
68.90	92.00	2.00	0.00	0.00	0.00	69.06	95.94	2.00	0.00	0.00	0.00
69.23	96.99	2.00	0.00	0.00	0.00	69.39	97.13	2.00	0.00	0.00	0.00
69.55	91.16	2.00	0.00	0.00	0.00	69.72	82.56	2.00	0.00	0.00	0.00
69.88	73.91	2.00	0.00	0.00	0.00	70.05	72.25	2.00	0.00	0.00	0.00
70.21	71.12	2.00	0.00	0.00	0.00	70.37	67.68	2.00	0.00	0.00	0.00
70.54	63.21	2.00	0.00	0.00	0.00	70.70	59.65	2.00	0.00	0.00	0.00
70.87	57.80	2.00	0.00	0.00	0.00	71.03	57.60	2.00	0.00	0.00	0.00
71.19	58.22	2.00	0.00	0.00	0.00	71.36	59.74	2.00	0.00	0.00	0.00
71.52	60.86	2.00	0.00	0.00	0.00	71.69	61.94	2.00	0.00	0.00	0.00
71.85	63.37	2.00	0.00	0.00	0.00	72.01	63.90	2.00	0.00	0.00	0.00
72.18	63.24	2.00	0.00	0.00	0.00	72.34	60.05	2.00	0.00	0.00	0.00
72.51	56.97	2.00	0.00	0.00	0.00	72.67	55.79	2.00	0.00	0.00	0.00
72.83	56.51	2.00	0.00	0.00	0.00	73.00	58.20	2.00	0.00	0.00	0.00
73.16	59.27	2.00	0.00	0.00	0.00	73.33	60.76	2.00	0.00	0.00	0.00
73.49	63.65	2.00	0.00	0.00	0.00	73.65	67.28	2.00	0.00	0.00	0.00
73.82	73.90	2.00	0.00	0.00	0.00	73.98	83.97	2.00	0.00	0.00	0.00
74.15	98.94	2.00	0.00	0.00	0.00	74.31	109.65	2.00	0.00	0.00	0.00
74.48	109.91	2.00	0.00	0.00	0.00	74.64	103.68	2.00	0.00	0.00	0.00
74.80	97.43	2.00	0.00	0.00	0.00	74.97	93.56	2.00	0.00	0.00	0.00
75.13	86.21	2.00	0.00	0.00	0.00	75.30	75.96	2.00	0.00	0.00	0.00
75.46	68.34	2.00	0.00	0.00	0.00	75.62	64.87	2.00	0.00	0.00	0.00
75.79	64.42	2.00	0.00	0.00	0.00	75.95	65.82	2.00	0.00	0.00	0.00
76.12	67.47	2.00	0.00	0.00	0.00	76.28	69.36	2.00	0.00	0.00	0.00
76.44	66.87	2.00	0.00	0.00	0.00	76.61	63.80	2.00	0.00	0.00	0.00
76.77	59.87	2.00	0.00	0.00	0.00	76.94	58.85	2.00	0.00	0.00	0.00
77.10	58.84	2.00	0.00	0.00	0.00	77.26	61.44	2.00	0.00	0.00	0.00
77.43	64.19	2.00	0.00	0.00	0.00	77.59	68.84	2.00	0.00	0.00	0.00
77.76	70.06	2.00	0.00	0.00	0.00	77.92	69.95	2.00	0.00	0.00	0.00
78.08	66.73	2.00	0.00	0.00	0.00	78.25	64.06	2.00	0.00	0.00	0.00
78.41	62.60	2.00	0.00	0.00	0.00	78.58	63.77	2.00	0.00	0.00	0.00
78.74	67.30	2.00	0.00	0.00	0.00	78.90	69.97	2.00	0.00	0.00	0.00
79.07	71.35	2.00	0.00	0.00	0.00	79.23	69.72	2.00	0.00	0.00	0.00
79.40	68.30	2.00	0.00	0.00	0.00	79.56	66.21	2.00	0.00	0.00	0.00
79.72	65.47	2.00	0.00	0.00	0.00	79.89	65.41	2.00	0.00	0.00	0.00
80.05	66.32	2.00	0.00	0.00	0.00	80.22	-1.00	2.00	0.00	0.00	0.00
80.38	-1.00	2.00	0.00	0.00	0.00	80.54	-1.00	2.00	0.00	0.00	0.00
80.71	-1.00	2.00	0.00	0.00	0.00						

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)

Total estimated settlement: 0.05

Abbreviations

$Q_{tn,cs}$: Equivalent clean sand normalized cone resistance
FS: Factor of safety against liquefaction
e_v (%): Post-liquefaction volumentric strain
DF: e_v depth weighting factor
Settlement: Calculated settlement

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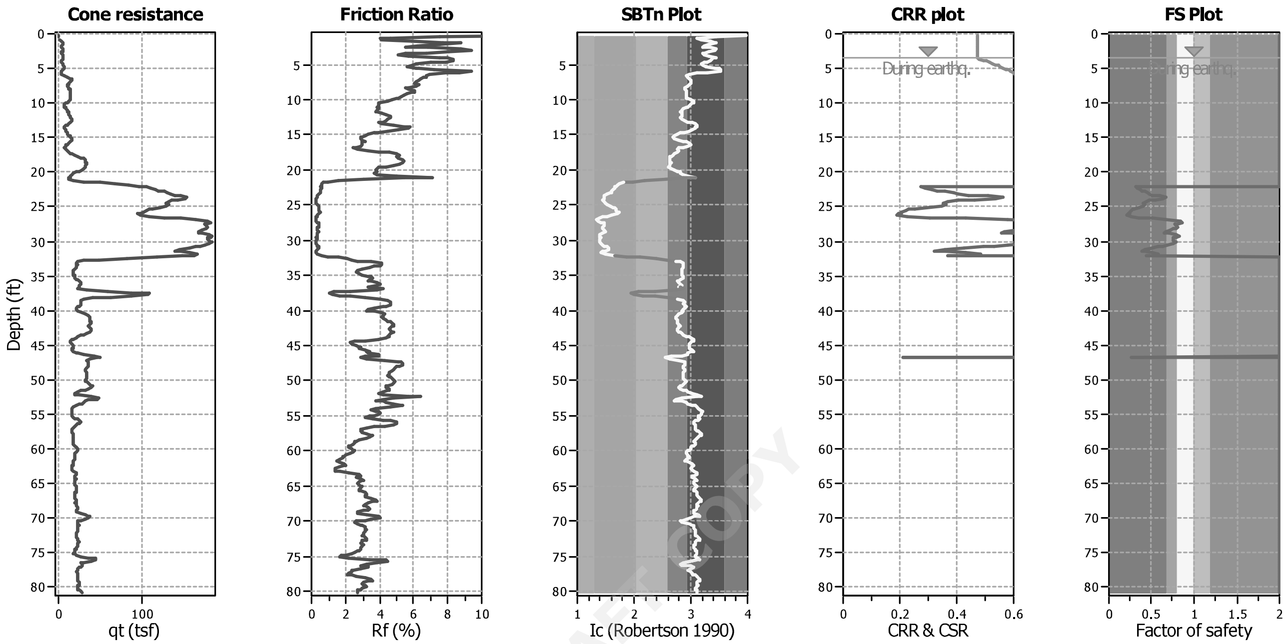
LIQUEFACTION ANALYSIS REPORT

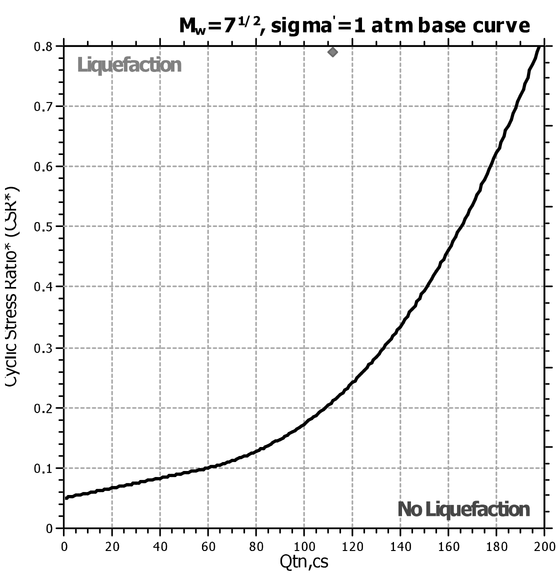
Project title : 5200 Patrick Henry Drive, LLC **Location : 5200 Patrick Henry Drive, Santa Clara**

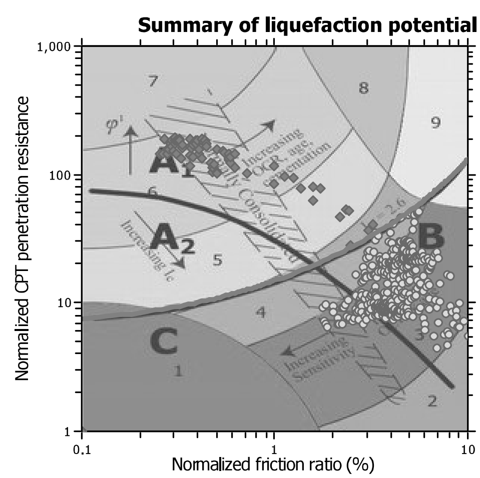
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.30 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.50 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:	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.61	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

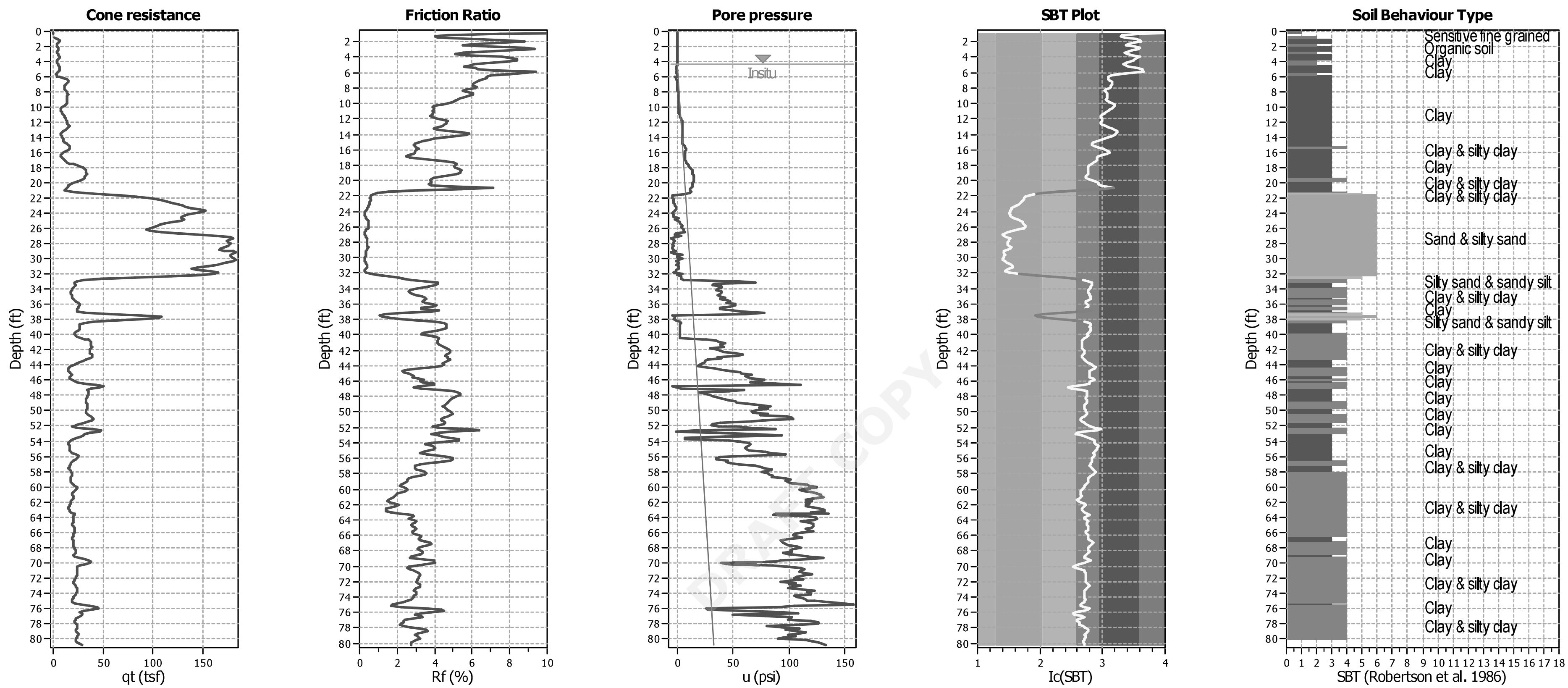






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



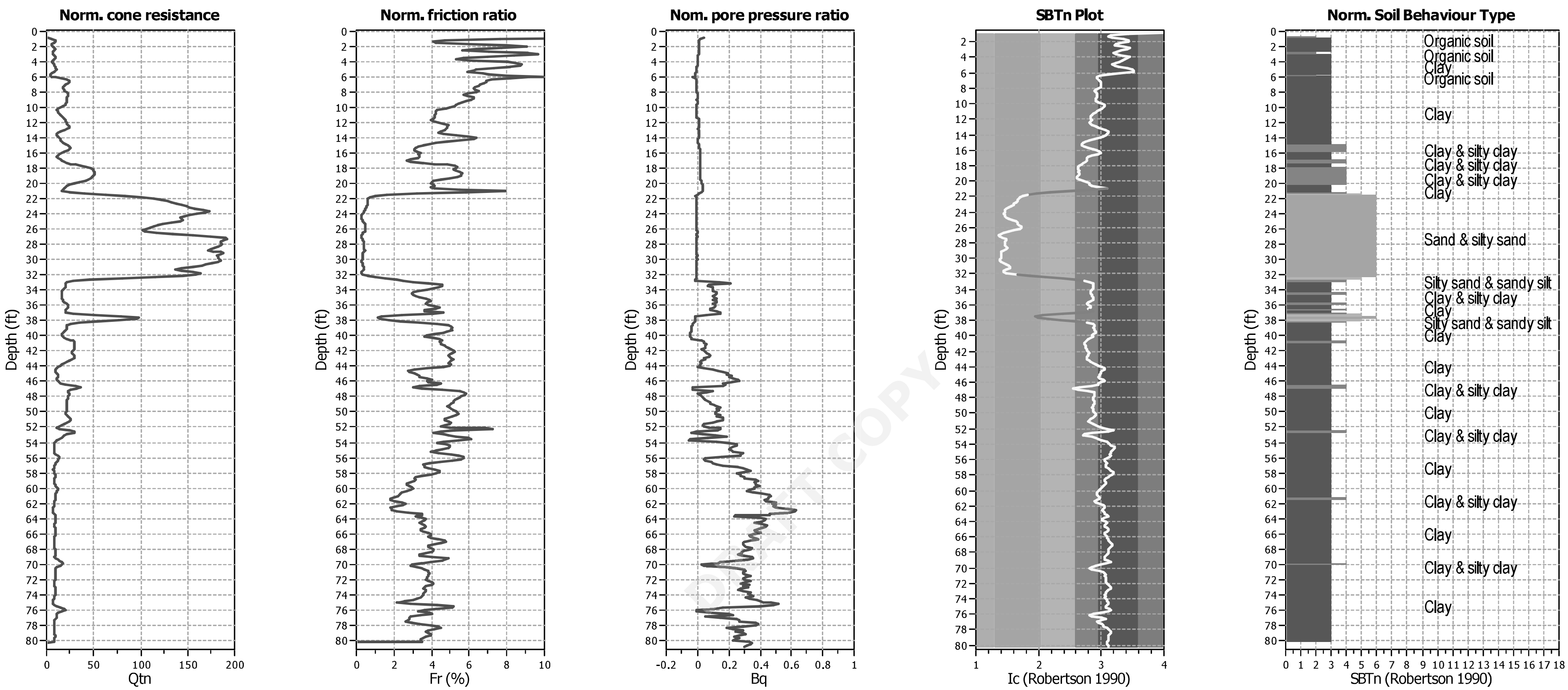
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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)



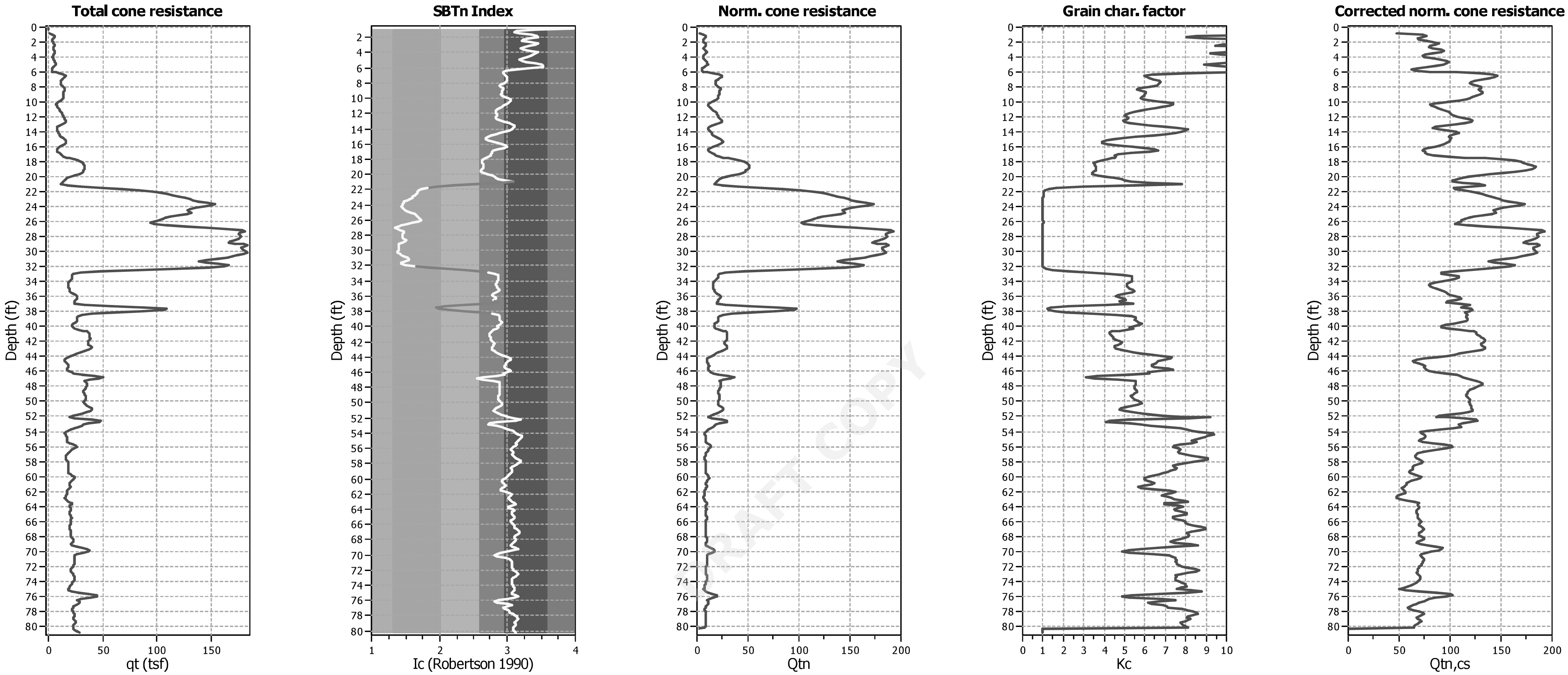
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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

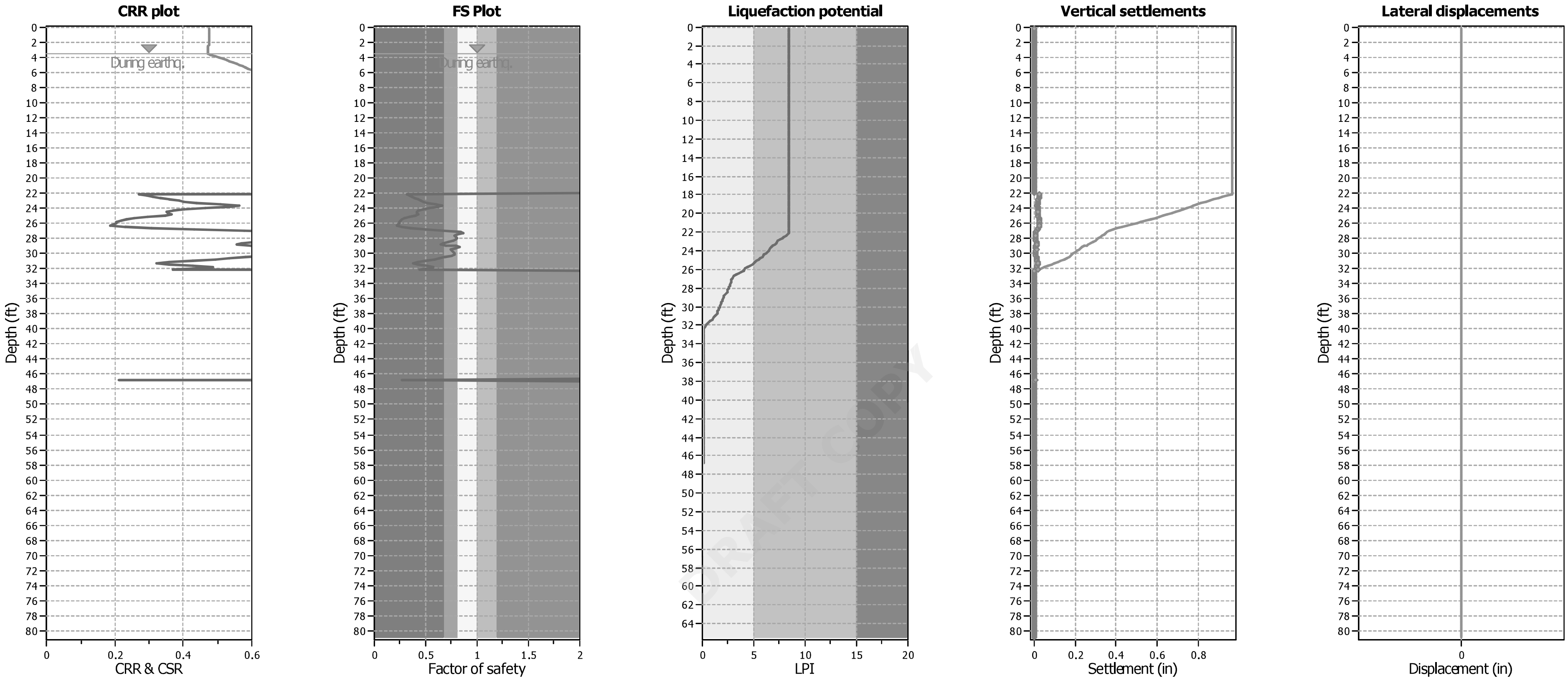
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

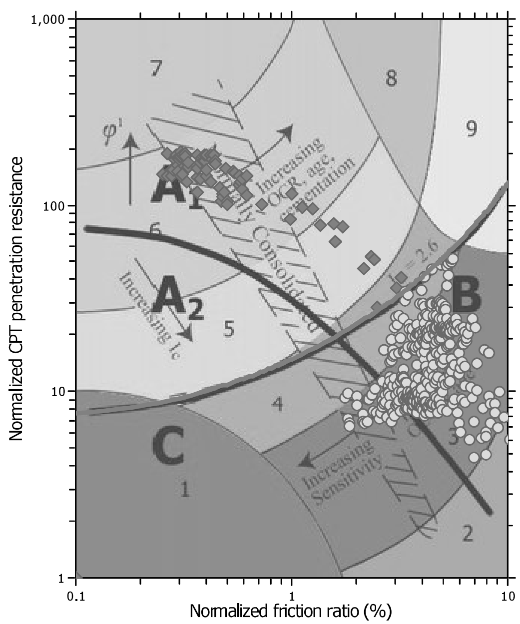
F.S. color scheme

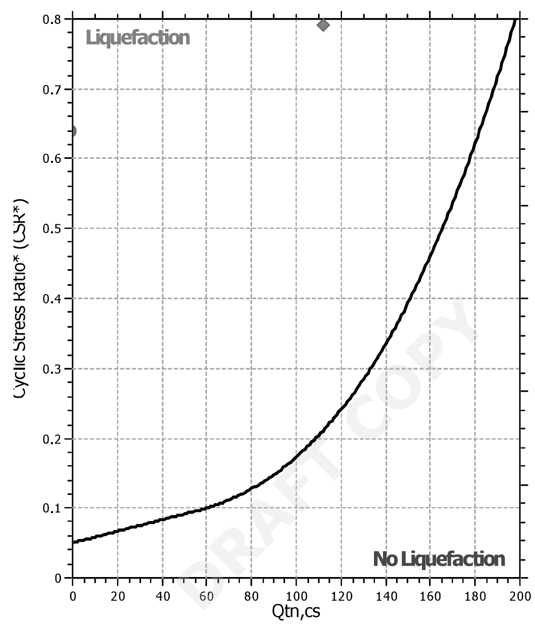
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

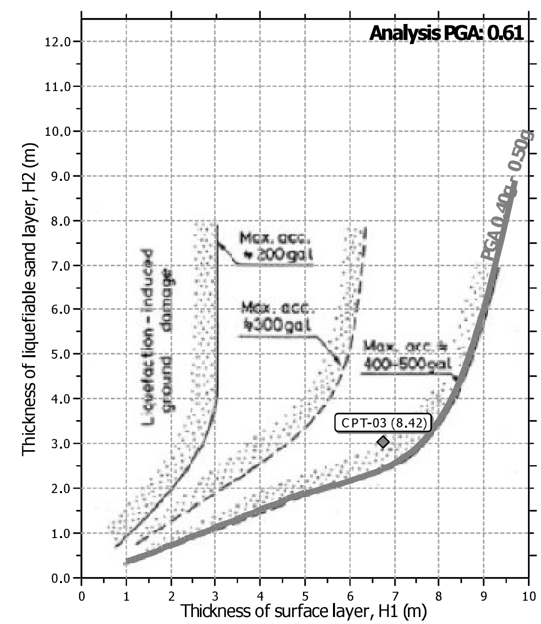
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



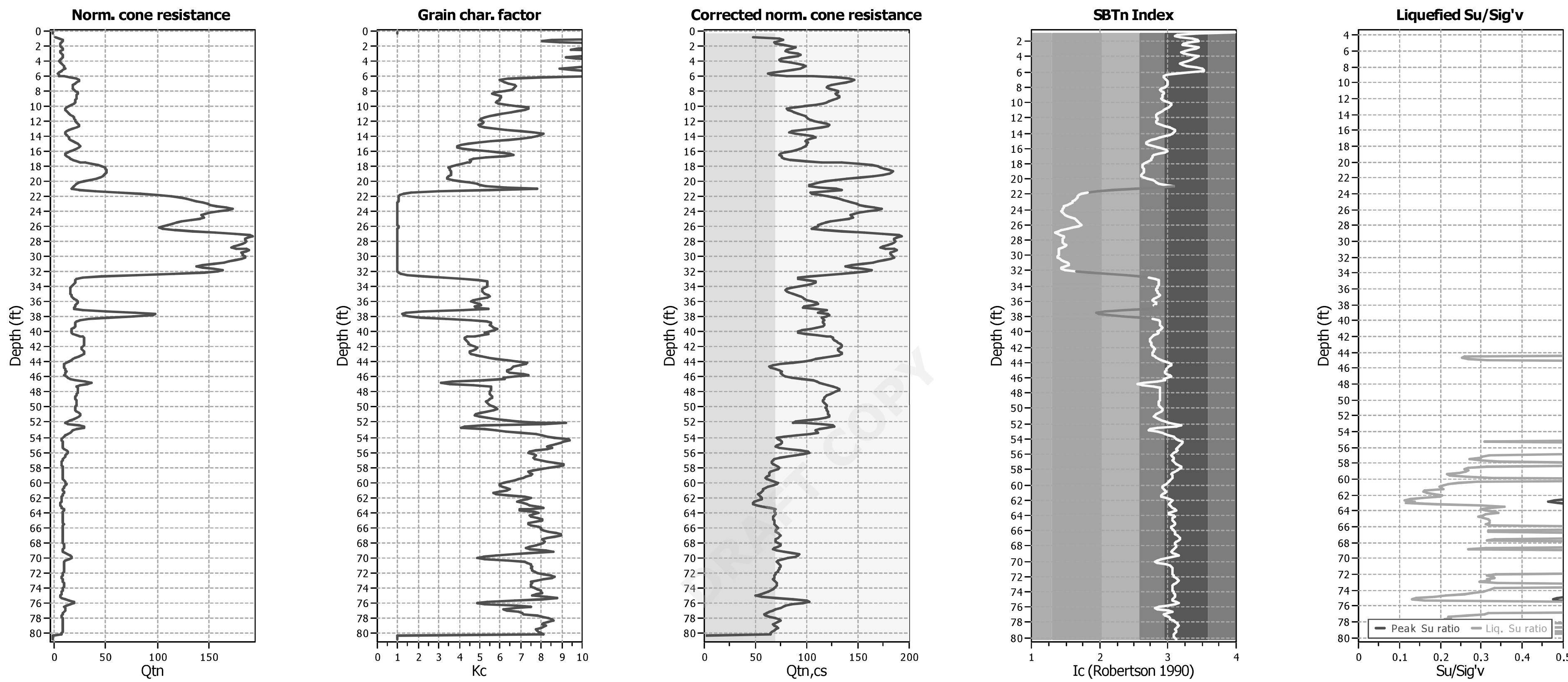




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

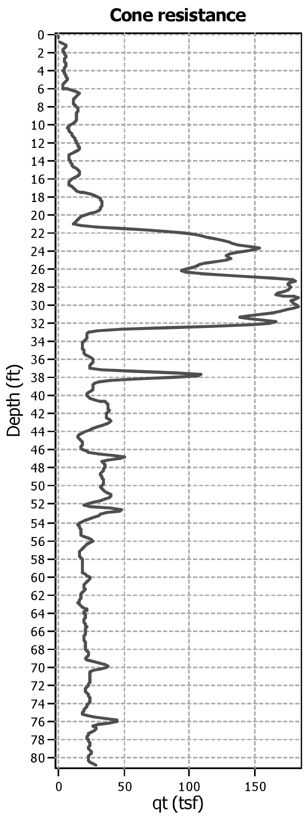
Check for strength loss plots (Robertson (2010))

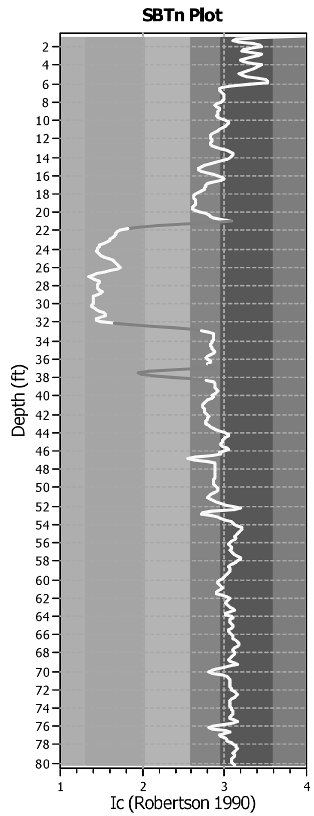


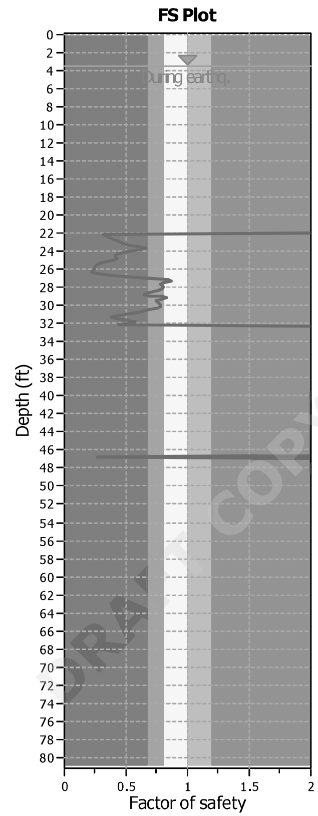
Input parameters and analysis data

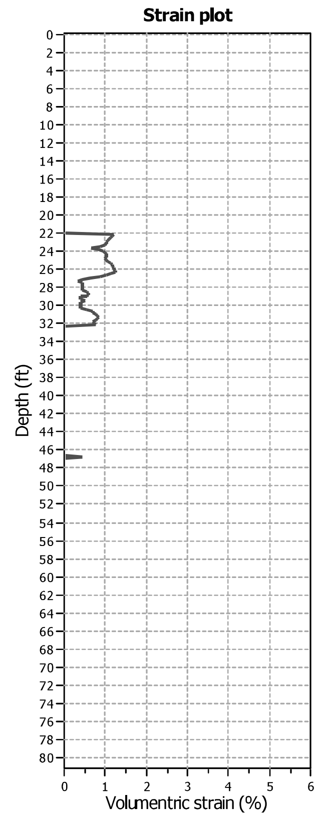
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

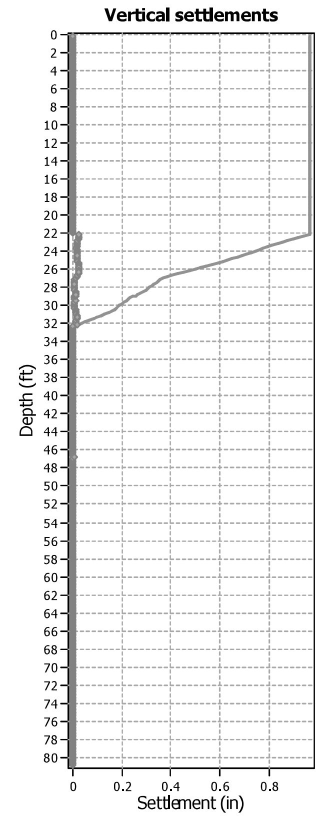
Estimation of post-earthquake settlements











Abbreviations

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

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
3.61	75.02	2.00	0.00	0.94	0.00	3.77	73.11	2.00	0.00	0.94	0.00
3.94	74.41	2.00	0.00	0.93	0.00	4.10	83.05	2.00	0.00	0.93	0.00
4.27	90.51	2.00	0.00	0.93	0.00	4.43	95.97	2.00	0.00	0.92	0.00
4.59	98.27	2.00	0.00	0.92	0.00	4.76	96.91	2.00	0.00	0.92	0.00
4.92	93.99	2.00	0.00	0.92	0.00	5.09	86.44	2.00	0.00	0.91	0.00
5.25	77.54	2.00	0.00	0.91	0.00	5.41	68.42	2.00	0.00	0.91	0.00
5.58	61.74	2.00	0.00	0.91	0.00	5.74	66.23	2.00	0.00	0.90	0.00
5.91	80.07	2.00	0.00	0.90	0.00	6.07	106.22	2.00	0.00	0.90	0.00
6.23	129.08	2.00	0.00	0.89	0.00	6.40	141.44	2.00	0.00	0.89	0.00
6.56	146.39	2.00	0.00	0.89	0.00	6.73	144.49	2.00	0.00	0.89	0.00
6.89	138.70	2.00	0.00	0.88	0.00	7.05	131.20	2.00	0.00	0.88	0.00
7.22	124.37	2.00	0.00	0.88	0.00	7.38	120.54	2.00	0.00	0.87	0.00
7.55	119.47	2.00	0.00	0.87	0.00	7.71	121.84	2.00	0.00	0.87	0.00
7.87	126.73	2.00	0.00	0.87	0.00	8.04	130.19	2.00	0.00	0.86	0.00
8.20	130.08	2.00	0.00	0.86	0.00	8.37	128.48	2.00	0.00	0.86	0.00
8.53	129.59	2.00	0.00	0.86	0.00	8.69	131.81	2.00	0.00	0.85	0.00
8.86	131.80	2.00	0.00	0.85	0.00	9.02	129.18	2.00	0.00	0.85	0.00
9.19	125.08	2.00	0.00	0.84	0.00	9.35	122.32	2.00	0.00	0.84	0.00
9.51	120.09	2.00	0.00	0.84	0.00	9.68	115.35	2.00	0.00	0.84	0.00
9.84	108.03	2.00	0.00	0.83	0.00	10.01	97.04	2.00	0.00	0.83	0.00
10.17	87.68	2.00	0.00	0.83	0.00	10.33	81.07	2.00	0.00	0.82	0.00
10.50	82.01	2.00	0.00	0.82	0.00	10.66	85.53	2.00	0.00	0.82	0.00
10.83	88.38	2.00	0.00	0.82	0.00	10.99	91.24	2.00	0.00	0.81	0.00
11.15	95.05	2.00	0.00	0.81	0.00	11.32	98.70	2.00	0.00	0.81	0.00
11.48	100.55	2.00	0.00	0.81	0.00	11.65	101.10	2.00	0.00	0.80	0.00
11.81	103.51	2.00	0.00	0.80	0.00	11.98	107.82	2.00	0.00	0.80	0.00
12.14	113.48	2.00	0.00	0.79	0.00	12.30	119.54	2.00	0.00	0.79	0.00
12.47	121.78	2.00	0.00	0.79	0.00	12.63	121.00	2.00	0.00	0.79	0.00
12.80	114.91	2.00	0.00	0.78	0.00	12.96	105.95	2.00	0.00	0.78	0.00
13.12	95.11	2.00	0.00	0.78	0.00	13.29	85.87	2.00	0.00	0.77	0.00
13.45	83.52	2.00	0.00	0.77	0.00	13.62	88.90	2.00	0.00	0.77	0.00
13.78	98.69	2.00	0.00	0.77	0.00	13.94	106.31	2.00	0.00	0.76	0.00
14.11	107.98	2.00	0.00	0.76	0.00	14.27	103.69	2.00	0.00	0.76	0.00
14.44	100.34	2.00	0.00	0.76	0.00	14.60	99.33	2.00	0.00	0.75	0.00
14.76	101.15	2.00	0.00	0.75	0.00	14.93	100.56	2.00	0.00	0.75	0.00
15.09	99.89	2.00	0.00	0.74	0.00	15.26	98.53	2.00	0.00	0.74	0.00
15.42	96.83	2.00	0.00	0.74	0.00	15.58	93.83	2.00	0.00	0.74	0.00
15.75	89.84	2.00	0.00	0.73	0.00	15.91	84.94	2.00	0.00	0.73	0.00
16.08	78.00	2.00	0.00	0.73	0.00	16.24	74.50	2.00	0.00	0.72	0.00
16.40	73.38	2.00	0.00	0.72	0.00	16.57	75.15	2.00	0.00	0.72	0.00
16.73	75.71	2.00	0.00	0.72	0.00	16.90	77.71	2.00	0.00	0.71	0.00
17.06	84.75	2.00	0.00	0.71	0.00	17.22	98.34	2.00	0.00	0.71	0.00
17.39	114.90	2.00	0.00	0.71	0.00	17.55	133.97	2.00	0.00	0.70	0.00
17.72	151.33	2.00	0.00	0.70	0.00	17.88	163.90	2.00	0.00	0.70	0.00
18.04	169.73	2.00	0.00	0.69	0.00	18.21	172.14	2.00	0.00	0.69	0.00
18.37	175.81	2.00	0.00	0.69	0.00	18.54	180.71	2.00	0.00	0.69	0.00
18.70	183.81	2.00	0.00	0.68	0.00	18.86	182.66	2.00	0.00	0.68	0.00
19.03	179.95	2.00	0.00	0.68	0.00	19.19	173.87	2.00	0.00	0.67	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
19.36	165.20	2.00	0.00	0.67	0.00	19.52	152.51	2.00	0.00	0.67	0.00
19.69	139.14	2.00	0.00	0.67	0.00	19.85	127.57	2.00	0.00	0.66	0.00
20.01	118.37	2.00	0.00	0.66	0.00	20.18	111.91	2.00	0.00	0.66	0.00
20.34	107.31	2.00	0.00	0.66	0.00	20.51	102.25	2.00	0.00	0.65	0.00
20.67	102.27	2.00	0.00	0.65	0.00	20.83	112.45	2.00	0.00	0.65	0.00
21.00	129.29	2.00	0.00	0.64	0.00	21.16	134.07	2.00	0.00	0.64	0.00
21.33	120.94	2.00	0.00	0.64	0.00	21.49	104.03	2.00	0.00	0.64	0.00
21.65	105.26	2.00	0.00	0.63	0.00	21.82	112.98	2.00	0.00	0.63	0.00
21.98	120.72	2.00	0.00	0.63	0.00	22.15	127.28	0.32	1.20	0.62	0.02
22.31	131.43	0.34	1.16	0.62	0.02	22.47	135.39	0.37	1.13	0.62	0.02
22.64	139.69	0.39	1.09	0.62	0.02	22.80	144.13	0.42	1.06	0.61	0.02
22.97	149.08	0.46	1.03	0.61	0.02	23.13	151.58	0.47	1.01	0.61	0.02
23.29	156.95	0.52	0.98	0.61	0.02	23.46	166.45	0.60	0.87	0.60	0.02
23.62	173.40	0.66	0.67	0.60	0.01	23.79	169.71	0.63	0.84	0.60	0.02
23.95	160.27	0.54	0.94	0.59	0.02	24.11	151.50	0.47	0.98	0.59	0.02
24.28	146.02	0.43	1.01	0.59	0.02	24.44	142.74	0.41	1.02	0.59	0.02
24.61	143.45	0.41	1.01	0.58	0.02	24.77	145.14	0.43	1.00	0.58	0.02
24.93	141.93	0.40	1.01	0.58	0.02	25.10	132.52	0.35	1.07	0.57	0.02
25.26	123.29	0.30	1.13	0.57	0.02	25.43	118.20	0.27	1.16	0.57	0.02
25.59	114.67	0.26	1.18	0.57	0.02	25.75	110.50	0.24	1.21	0.56	0.02
25.92	110.69	0.24	1.21	0.56	0.02	26.08	107.78	0.23	1.23	0.56	0.02
26.25	104.94	0.22	1.25	0.56	0.02	26.41	116.94	0.27	1.14	0.55	0.02
26.57	134.48	0.36	1.01	0.55	0.02	26.74	155.63	0.50	0.89	0.55	0.02
26.90	174.88	0.68	0.60	0.54	0.01	27.07	190.01	0.84	0.43	0.54	0.01
27.23	192.31	0.87	0.32	0.54	0.01	27.40	187.27	0.81	0.44	0.54	0.01
27.56	184.76	0.78	0.44	0.53	0.01	27.72	185.52	0.79	0.44	0.53	0.01
27.89	186.78	0.80	0.43	0.53	0.01	28.05	185.31	0.79	0.43	0.52	0.01
28.22	183.07	0.76	0.44	0.52	0.01	28.38	178.78	0.72	0.56	0.52	0.01
28.54	174.05	0.67	0.58	0.52	0.01	28.71	172.32	0.65	0.58	0.51	0.01
28.87	177.05	0.70	0.56	0.51	0.01	29.04	185.72	0.79	0.42	0.51	0.01
29.20	188.33	0.82	0.41	0.51	0.01	29.36	185.68	0.79	0.41	0.50	0.01
29.53	181.54	0.75	0.53	0.50	0.01	29.69	182.10	0.75	0.42	0.50	0.01
29.86	182.81	0.76	0.42	0.49	0.01	30.02	184.52	0.78	0.41	0.49	0.01
30.18	184.87	0.79	0.40	0.49	0.01	30.35	181.96	0.75	0.41	0.49	0.01
30.51	175.29	0.68	0.54	0.48	0.01	30.68	169.60	0.63	0.68	0.48	0.01
30.84	164.47	0.58	0.70	0.48	0.01	31.00	155.34	0.51	0.77	0.47	0.02
31.17	144.03	0.42	0.82	0.47	0.02	31.33	137.43	0.38	0.84	0.47	0.02
31.50	143.92	0.42	0.81	0.47	0.02	31.66	155.43	0.51	0.75	0.46	0.01
31.82	163.48	0.58	0.69	0.46	0.01	31.99	158.71	0.53	0.73	0.46	0.01
32.15	145.85	0.44	0.78	0.46	0.02	32.32	133.89	2.00	0.00	0.45	0.00
32.48	118.35	2.00	0.00	0.45	0.00	32.64	103.67	2.00	0.00	0.45	0.00
32.81	91.38	2.00	0.00	0.44	0.00	32.97	91.04	2.00	0.00	0.44	0.00
33.14	99.45	2.00	0.00	0.44	0.00	33.30	108.27	2.00	0.00	0.44	0.00
33.46	108.78	2.00	0.00	0.43	0.00	33.63	105.21	2.00	0.00	0.43	0.00
33.79	99.61	2.00	0.00	0.43	0.00	33.96	92.60	2.00	0.00	0.42	0.00
34.12	85.38	2.00	0.00	0.42	0.00	34.28	80.64	2.00	0.00	0.42	0.00
34.45	79.43	2.00	0.00	0.42	0.00	34.61	80.33	2.00	0.00	0.41	0.00
34.78	83.12	2.00	0.00	0.41	0.00	34.94	86.26	2.00	0.00	0.41	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
35.10	90.74	2.00	0.00	0.41	0.00	35.27	93.41	2.00	0.00	0.40	0.00
35.43	96.22	2.00	0.00	0.40	0.00	35.60	97.66	2.00	0.00	0.40	0.00
35.76	100.08	2.00	0.00	0.39	0.00	35.93	104.39	2.00	0.00	0.39	0.00
36.09	109.96	2.00	0.00	0.39	0.00	36.25	111.34	2.00	0.00	0.39	0.00
36.42	106.65	2.00	0.00	0.38	0.00	36.58	97.20	2.00	0.00	0.38	0.00
36.75	96.51	2.00	0.00	0.38	0.00	36.91	108.47	2.00	0.00	0.37	0.00
37.07	119.07	2.00	0.00	0.37	0.00	37.24	114.63	2.00	0.00	0.37	0.00
37.40	111.38	2.00	0.00	0.37	0.00	37.57	120.90	2.00	0.00	0.36	0.00
37.73	122.55	2.00	0.00	0.36	0.00	37.89	119.79	2.00	0.00	0.36	0.00
38.06	114.76	2.00	0.00	0.35	0.00	38.22	115.48	2.00	0.00	0.35	0.00
38.39	116.57	2.00	0.00	0.35	0.00	38.55	116.38	2.00	0.00	0.35	0.00
38.71	115.90	2.00	0.00	0.34	0.00	38.88	116.55	2.00	0.00	0.34	0.00
39.04	116.76	2.00	0.00	0.34	0.00	39.21	115.81	2.00	0.00	0.34	0.00
39.37	110.22	2.00	0.00	0.33	0.00	39.53	103.29	2.00	0.00	0.33	0.00
39.70	95.68	2.00	0.00	0.33	0.00	39.86	90.87	2.00	0.00	0.32	0.00
40.03	91.16	2.00	0.00	0.32	0.00	40.19	99.15	2.00	0.00	0.32	0.00
40.35	110.11	2.00	0.00	0.32	0.00	40.52	119.46	2.00	0.00	0.31	0.00
40.68	123.72	2.00	0.00	0.31	0.00	40.85	125.37	2.00	0.00	0.31	0.00
41.01	126.01	2.00	0.00	0.30	0.00	41.17	126.69	2.00	0.00	0.30	0.00
41.34	128.94	2.00	0.00	0.30	0.00	41.50	131.03	2.00	0.00	0.30	0.00
41.67	133.50	2.00	0.00	0.29	0.00	41.83	134.71	2.00	0.00	0.29	0.00
41.99	134.47	2.00	0.00	0.29	0.00	42.16	132.81	2.00	0.00	0.29	0.00
42.32	130.93	2.00	0.00	0.28	0.00	42.49	130.94	2.00	0.00	0.28	0.00
42.65	131.96	2.00	0.00	0.28	0.00	42.81	134.35	2.00	0.00	0.27	0.00
42.98	133.99	2.00	0.00	0.27	0.00	43.14	131.87	2.00	0.00	0.27	0.00
43.31	123.82	2.00	0.00	0.27	0.00	43.47	116.32	2.00	0.00	0.26	0.00
43.64	108.87	2.00	0.00	0.26	0.00	43.80	104.79	2.00	0.00	0.26	0.00
43.96	96.99	2.00	0.00	0.25	0.00	44.13	86.41	2.00	0.00	0.25	0.00
44.29	74.10	2.00	0.00	0.25	0.00	44.46	66.49	2.00	0.00	0.25	0.00
44.62	63.38	2.00	0.00	0.24	0.00	44.78	65.12	2.00	0.00	0.24	0.00
44.95	69.77	2.00	0.00	0.24	0.00	45.11	73.67	2.00	0.00	0.24	0.00
45.28	75.13	2.00	0.00	0.23	0.00	45.44	74.03	2.00	0.00	0.23	0.00
45.60	75.48	2.00	0.00	0.23	0.00	45.77	77.22	2.00	0.00	0.22	0.00
45.93	79.15	2.00	0.00	0.22	0.00	46.10	85.46	2.00	0.00	0.22	0.00
46.26	95.46	2.00	0.00	0.22	0.00	46.42	106.60	2.00	0.00	0.21	0.00
46.59	109.88	2.00	0.00	0.21	0.00	46.75	111.94	0.27	0.44	0.21	0.01
46.92	115.37	2.00	0.00	0.20	0.00	47.08	120.53	2.00	0.00	0.20	0.00
47.24	125.00	2.00	0.00	0.20	0.00	47.41	129.33	2.00	0.00	0.20	0.00
47.57	131.85	2.00	0.00	0.19	0.00	47.74	131.41	2.00	0.00	0.19	0.00
47.90	128.43	2.00	0.00	0.19	0.00	48.06	125.67	2.00	0.00	0.19	0.00
48.23	122.67	2.00	0.00	0.18	0.00	48.39	119.97	2.00	0.00	0.18	0.00
48.56	117.93	2.00	0.00	0.18	0.00	48.72	117.25	2.00	0.00	0.17	0.00
48.88	116.54	2.00	0.00	0.17	0.00	49.05	116.35	2.00	0.00	0.17	0.00
49.21	115.80	2.00	0.00	0.17	0.00	49.38	117.65	2.00	0.00	0.16	0.00
49.54	118.59	2.00	0.00	0.16	0.00	49.70	119.42	2.00	0.00	0.16	0.00
49.87	118.95	2.00	0.00	0.15	0.00	50.03	118.47	2.00	0.00	0.15	0.00
50.20	118.98	2.00	0.00	0.15	0.00	50.36	119.55	2.00	0.00	0.15	0.00
50.52	120.35	2.00	0.00	0.14	0.00	50.69	120.59	2.00	0.00	0.14	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
50.85	121.07	2.00	0.00	0.14	0.00	51.02	121.95	2.00	0.00	0.14	0.00
51.18	121.91	2.00	0.00	0.13	0.00	51.35	118.25	2.00	0.00	0.13	0.00
51.51	110.73	2.00	0.00	0.13	0.00	51.67	99.59	2.00	0.00	0.12	0.00
51.84	89.09	2.00	0.00	0.12	0.00	52.00	86.21	2.00	0.00	0.12	0.00
52.17	96.55	2.00	0.00	0.12	0.00	52.33	114.61	2.00	0.00	0.11	0.00
52.49	126.07	2.00	0.00	0.11	0.00	52.66	127.20	2.00	0.00	0.11	0.00
52.82	119.54	2.00	0.00	0.10	0.00	52.99	112.53	2.00	0.00	0.10	0.00
53.15	108.72	2.00	0.00	0.10	0.00	53.31	110.03	2.00	0.00	0.10	0.00
53.48	110.88	2.00	0.00	0.09	0.00	53.64	103.49	2.00	0.00	0.09	0.00
53.81	90.56	2.00	0.00	0.09	0.00	53.97	77.66	2.00	0.00	0.09	0.00
54.13	71.28	2.00	0.00	0.08	0.00	54.30	71.87	2.00	0.00	0.08	0.00
54.46	73.88	2.00	0.00	0.08	0.00	54.63	75.61	2.00	0.00	0.07	0.00
54.79	75.87	2.00	0.00	0.07	0.00	54.95	73.87	2.00	0.00	0.07	0.00
55.12	71.03	2.00	0.00	0.07	0.00	55.28	69.29	2.00	0.00	0.06	0.00
55.45	73.38	2.00	0.00	0.06	0.00	55.61	82.74	2.00	0.00	0.06	0.00
55.77	93.83	2.00	0.00	0.05	0.00	55.94	101.31	2.00	0.00	0.05	0.00
56.10	101.92	2.00	0.00	0.05	0.00	56.27	96.93	2.00	0.00	0.05	0.00
56.43	88.97	2.00	0.00	0.04	0.00	56.59	80.41	2.00	0.00	0.04	0.00
56.76	73.70	2.00	0.00	0.04	0.00	56.92	68.78	2.00	0.00	0.04	0.00
57.09	67.44	2.00	0.00	0.03	0.00	57.25	65.74	2.00	0.00	0.03	0.00
57.41	65.49	2.00	0.00	0.03	0.00	57.58	66.55	2.00	0.00	0.02	0.00
57.74	69.48	2.00	0.00	0.02	0.00	57.91	72.15	2.00	0.00	0.02	0.00
58.07	72.75	2.00	0.00	0.02	0.00	58.23	71.39	2.00	0.00	0.01	0.00
58.40	68.04	2.00	0.00	0.01	0.00	58.56	65.24	2.00	0.00	0.01	0.00
58.73	63.74	2.00	0.00	0.00	0.00	58.89	63.53	2.00	0.00	0.00	0.00
59.06	63.09	2.00	0.00	0.00	0.00	59.22	61.44	2.00	0.00	0.00	0.00
59.38	60.15	2.00	0.00	0.00	0.00	59.55	60.79	2.00	0.00	0.00	0.00
59.71	64.43	2.00	0.00	0.00	0.00	59.88	68.68	2.00	0.00	0.00	0.00
60.04	71.46	2.00	0.00	0.00	0.00	60.20	70.43	2.00	0.00	0.00	0.00
60.37	66.94	2.00	0.00	0.00	0.00	60.53	62.44	2.00	0.00	0.00	0.00
60.70	59.60	2.00	0.00	0.00	0.00	60.86	57.93	2.00	0.00	0.00	0.00
61.02	57.49	2.00	0.00	0.00	0.00	61.19	55.90	2.00	0.00	0.00	0.00
61.35	54.51	2.00	0.00	0.00	0.00	61.52	52.92	2.00	0.00	0.00	0.00
61.68	53.72	2.00	0.00	0.00	0.00	61.84	55.44	2.00	0.00	0.00	0.00
62.01	56.56	2.00	0.00	0.00	0.00	62.17	55.54	2.00	0.00	0.00	0.00
62.34	52.46	2.00	0.00	0.00	0.00	62.50	48.77	2.00	0.00	0.00	0.00
62.66	47.75	2.00	0.00	0.00	0.00	62.83	47.34	2.00	0.00	0.00	0.00
62.99	50.69	2.00	0.00	0.00	0.00	63.16	56.96	2.00	0.00	0.00	0.00
63.32	65.01	2.00	0.00	0.00	0.00	63.48	69.26	2.00	0.00	0.00	0.00
63.65	69.36	2.00	0.00	0.00	0.00	63.81	67.99	2.00	0.00	0.00	0.00
63.98	68.17	2.00	0.00	0.00	0.00	64.14	69.32	2.00	0.00	0.00	0.00
64.30	69.36	2.00	0.00	0.00	0.00	64.47	68.62	2.00	0.00	0.00	0.00
64.63	67.21	2.00	0.00	0.00	0.00	64.80	66.94	2.00	0.00	0.00	0.00
64.96	67.19	2.00	0.00	0.00	0.00	65.12	67.96	2.00	0.00	0.00	0.00
65.29	68.52	2.00	0.00	0.00	0.00	65.45	68.46	2.00	0.00	0.00	0.00
65.62	68.54	2.00	0.00	0.00	0.00	65.78	69.30	2.00	0.00	0.00	0.00
65.94	71.00	2.00	0.00	0.00	0.00	66.11	72.12	2.00	0.00	0.00	0.00
66.27	71.27	2.00	0.00	0.00	0.00	66.44	69.62	2.00	0.00	0.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
66.60	69.37	2.00	0.00	0.00	0.00	66.77	71.58	2.00	0.00	0.00	0.00
66.93	73.76	2.00	0.00	0.00	0.00	67.09	74.53	2.00	0.00	0.00	0.00
67.26	73.61	2.00	0.00	0.00	0.00	67.42	71.91	2.00	0.00	0.00	0.00
67.59	69.94	2.00	0.00	0.00	0.00	67.75	69.31	2.00	0.00	0.00	0.00
67.91	70.06	2.00	0.00	0.00	0.00	68.08	72.39	2.00	0.00	0.00	0.00
68.24	73.98	2.00	0.00	0.00	0.00	68.41	74.12	2.00	0.00	0.00	0.00
68.57	71.92	2.00	0.00	0.00	0.00	68.73	68.31	2.00	0.00	0.00	0.00
68.90	66.88	2.00	0.00	0.00	0.00	69.06	70.17	2.00	0.00	0.00	0.00
69.23	78.28	2.00	0.00	0.00	0.00	69.39	85.78	2.00	0.00	0.00	0.00
69.55	92.44	2.00	0.00	0.00	0.00	69.72	91.86	2.00	0.00	0.00	0.00
69.88	89.30	2.00	0.00	0.00	0.00	70.05	81.74	2.00	0.00	0.00	0.00
70.21	76.78	2.00	0.00	0.00	0.00	70.37	72.48	2.00	0.00	0.00	0.00
70.54	71.22	2.00	0.00	0.00	0.00	70.70	71.78	2.00	0.00	0.00	0.00
70.87	73.43	2.00	0.00	0.00	0.00	71.03	73.88	2.00	0.00	0.00	0.00
71.19	73.95	2.00	0.00	0.00	0.00	71.36	73.19	2.00	0.00	0.00	0.00
71.52	73.11	2.00	0.00	0.00	0.00	71.69	72.26	2.00	0.00	0.00	0.00
71.85	71.36	2.00	0.00	0.00	0.00	72.01	69.57	2.00	0.00	0.00	0.00
72.18	69.02	2.00	0.00	0.00	0.00	72.34	68.82	2.00	0.00	0.00	0.00
72.51	68.75	2.00	0.00	0.00	0.00	72.67	68.59	2.00	0.00	0.00	0.00
72.83	67.66	2.00	0.00	0.00	0.00	73.00	68.52	2.00	0.00	0.00	0.00
73.16	69.40	2.00	0.00	0.00	0.00	73.33	71.30	2.00	0.00	0.00	0.00
73.49	71.29	2.00	0.00	0.00	0.00	73.65	70.81	2.00	0.00	0.00	0.00
73.82	69.41	2.00	0.00	0.00	0.00	73.98	68.44	2.00	0.00	0.00	0.00
74.15	66.74	2.00	0.00	0.00	0.00	74.31	65.07	2.00	0.00	0.00	0.00
74.48	62.05	2.00	0.00	0.00	0.00	74.64	58.97	2.00	0.00	0.00	0.00
74.80	54.70	2.00	0.00	0.00	0.00	74.97	51.48	2.00	0.00	0.00	0.00
75.13	50.41	2.00	0.00	0.00	0.00	75.30	56.46	2.00	0.00	0.00	0.00
75.46	70.12	2.00	0.00	0.00	0.00	75.62	86.99	2.00	0.00	0.00	0.00
75.79	99.98	2.00	0.00	0.00	0.00	75.95	102.81	2.00	0.00	0.00	0.00
76.12	95.80	2.00	0.00	0.00	0.00	76.28	85.13	2.00	0.00	0.00	0.00
76.44	79.76	2.00	0.00	0.00	0.00	76.61	76.57	2.00	0.00	0.00	0.00
76.77	74.07	2.00	0.00	0.00	0.00	76.94	69.18	2.00	0.00	0.00	0.00
77.10	66.78	2.00	0.00	0.00	0.00	77.26	63.79	2.00	0.00	0.00	0.00
77.43	60.95	2.00	0.00	0.00	0.00	77.59	59.04	2.00	0.00	0.00	0.00
77.76	59.88	2.00	0.00	0.00	0.00	77.92	63.37	2.00	0.00	0.00	0.00
78.08	69.30	2.00	0.00	0.00	0.00	78.25	73.11	2.00	0.00	0.00	0.00
78.41	73.83	2.00	0.00	0.00	0.00	78.58	71.79	2.00	0.00	0.00	0.00
78.74	69.21	2.00	0.00	0.00	0.00	78.90	67.55	2.00	0.00	0.00	0.00
79.07	68.39	2.00	0.00	0.00	0.00	79.23	70.42	2.00	0.00	0.00	0.00
79.40	71.60	2.00	0.00	0.00	0.00	79.56	69.90	2.00	0.00	0.00	0.00
79.72	67.75	2.00	0.00	0.00	0.00	79.89	65.64	2.00	0.00	0.00	0.00
80.05	64.44	2.00	0.00	0.00	0.00	80.22	64.34	2.00	0.00	0.00	0.00
80.38	-1.00	2.00	0.00	0.00	0.00	80.54	-1.00	2.00	0.00	0.00	0.00
80.71	-1.00	2.00	0.00	0.00	0.00	80.87	-1.00	2.00	0.00	0.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)

Total estimated settlement: 0.97

Abbreviations

$Q_{tn,cs}$: Equivalent clean sand normalized cone resistance
FS: Factor of safety against liquefaction
e_v (%): Post-liquefaction volumentric strain
DF: e_v depth weighting factor
Settlement: Calculated settlement

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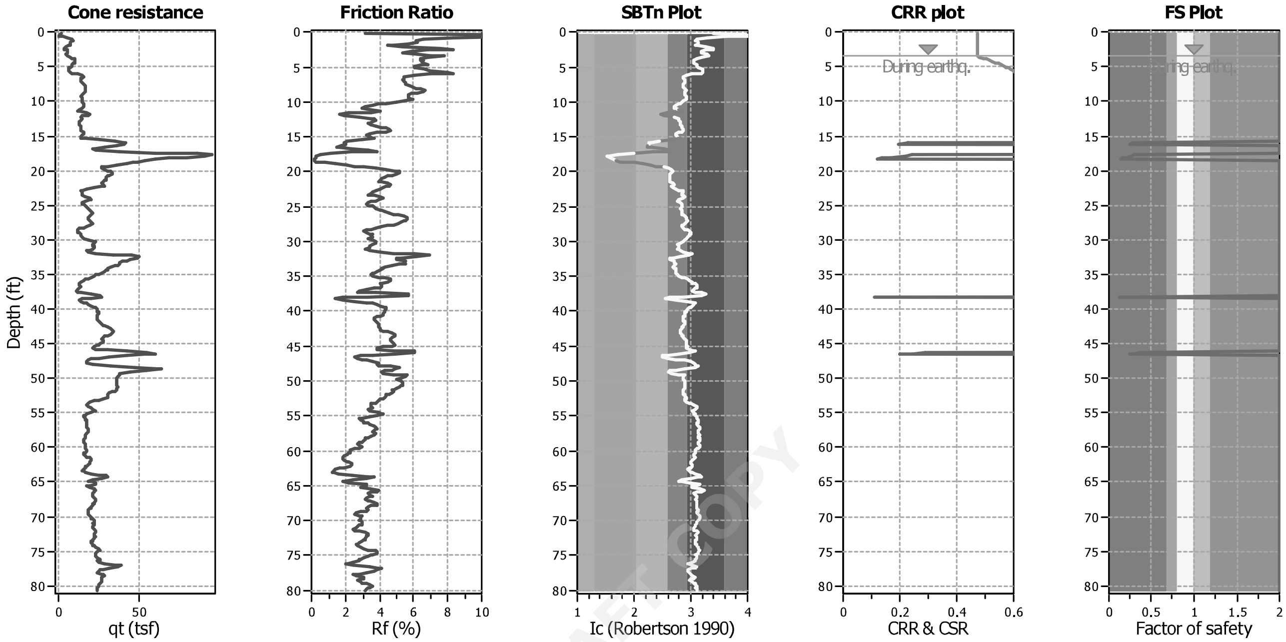
LIQUEFACTION ANALYSIS REPORT

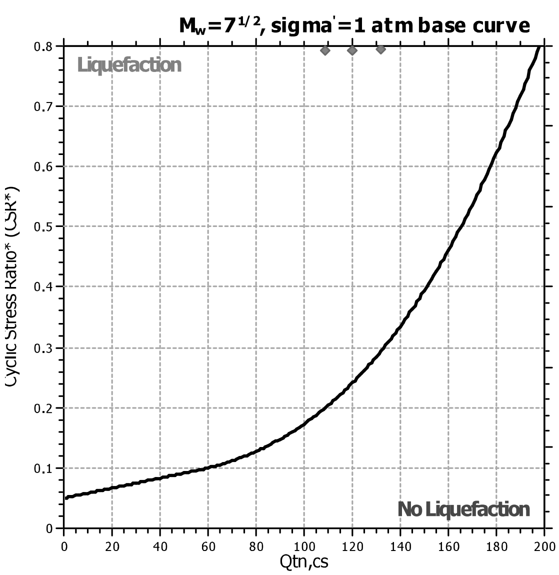
Project title : 5200 Patrick Henry Drive, LLC **Location : 5200 Patrick Henry Drive, Santa Clara**

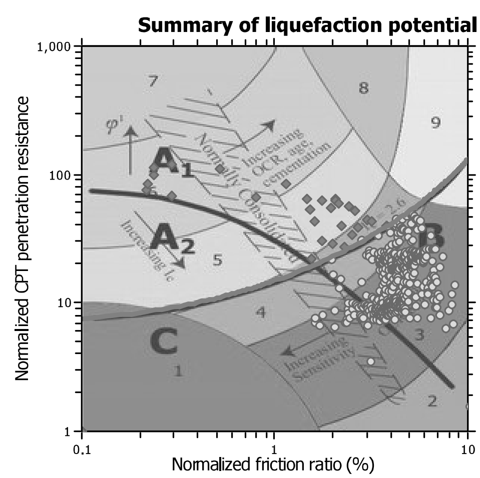
CPT file : CPT-04

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.30 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.50 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:	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.61	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

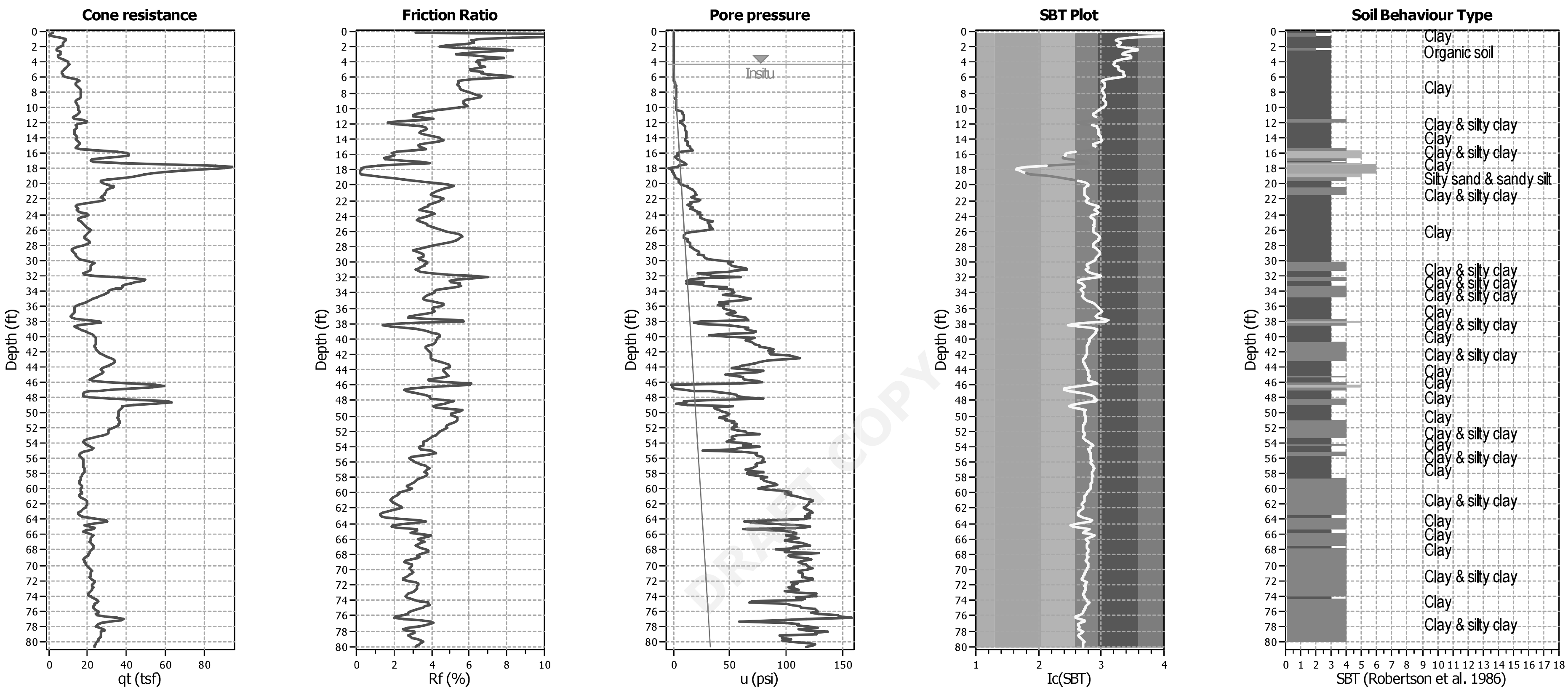






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



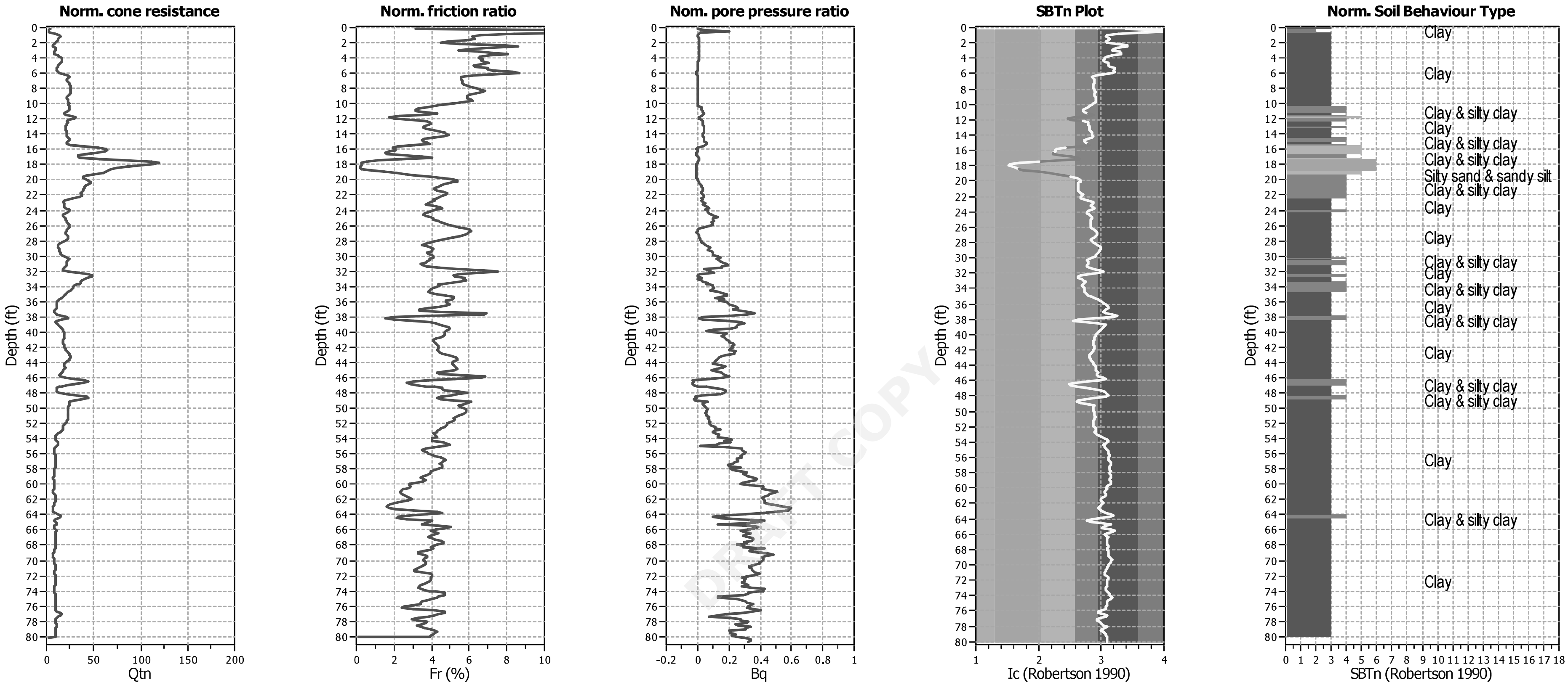
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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)



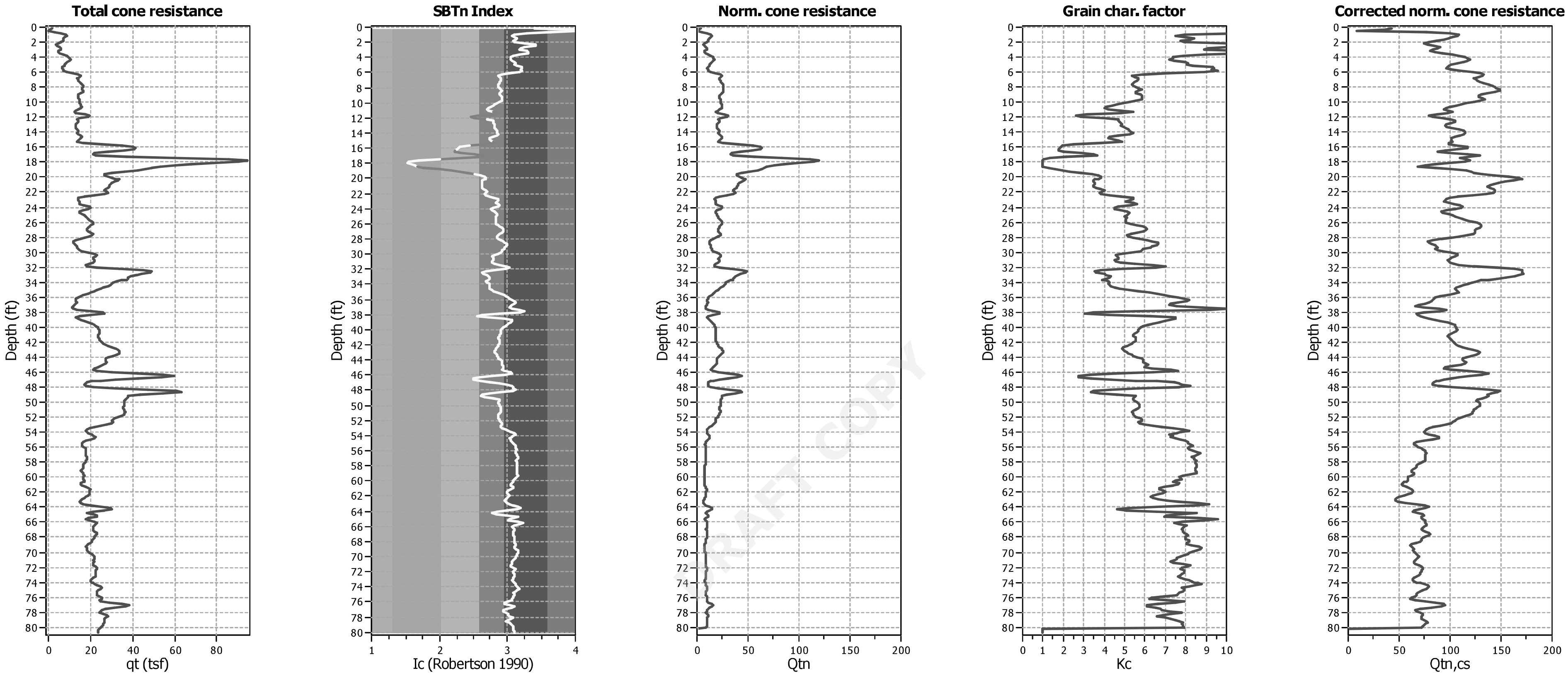
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A		
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes		
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	4.30 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

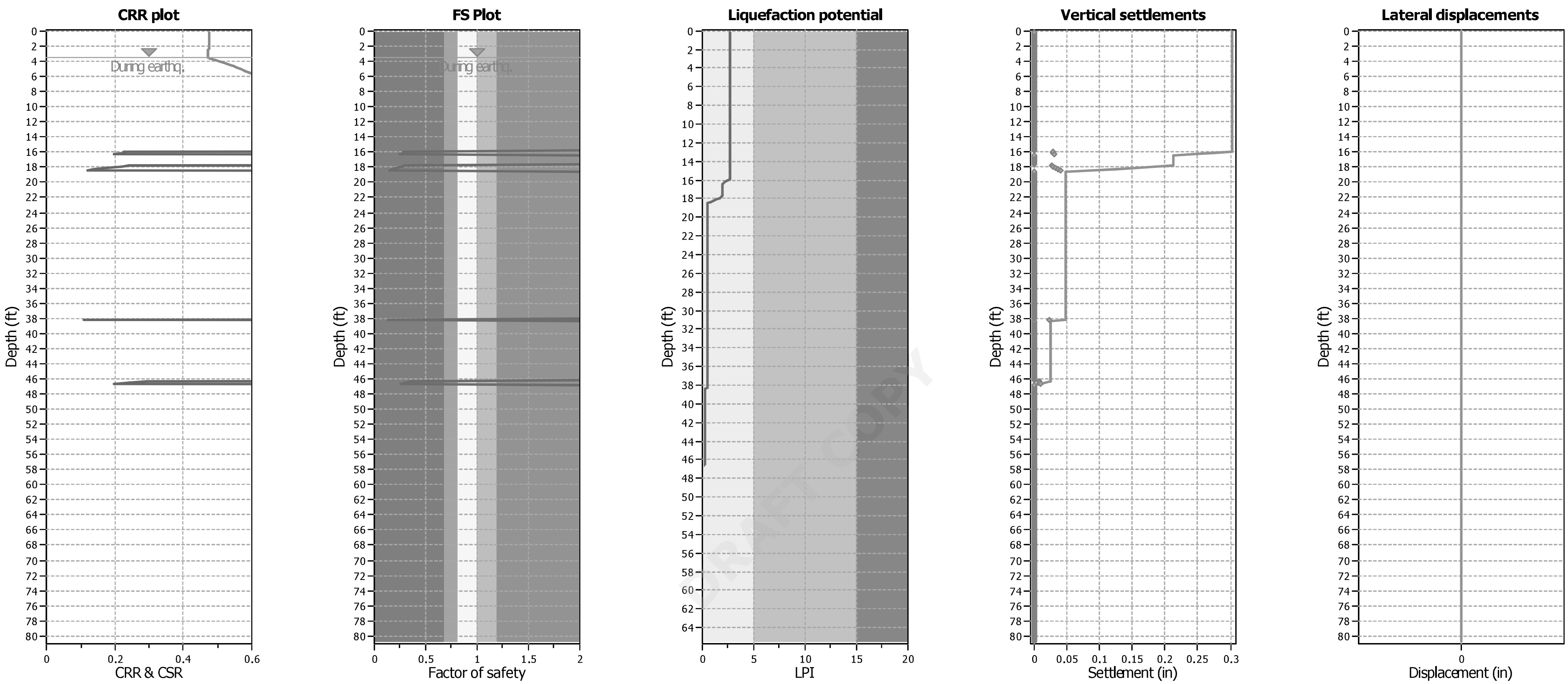
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

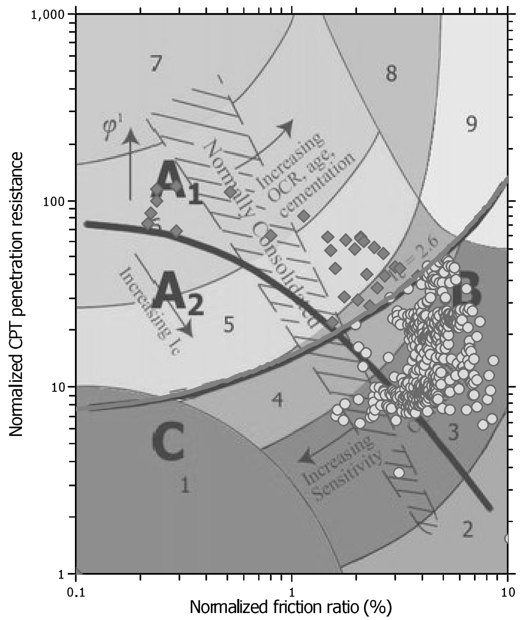
F.S. color scheme

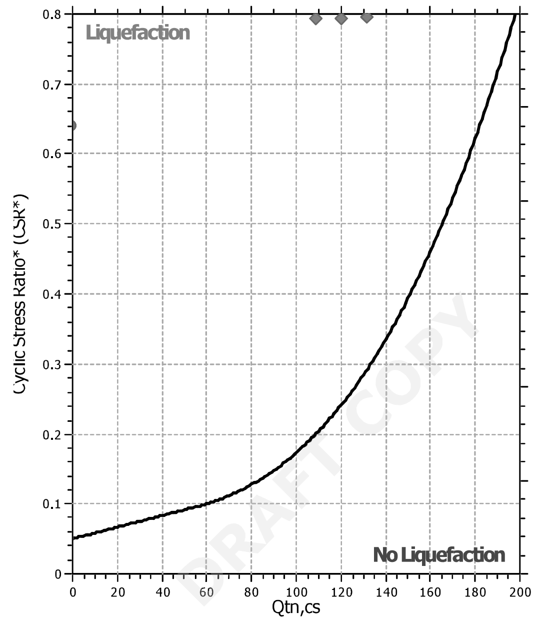
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

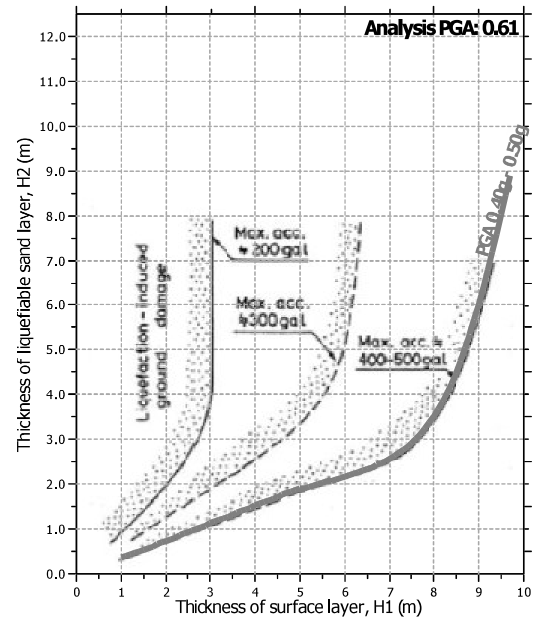
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



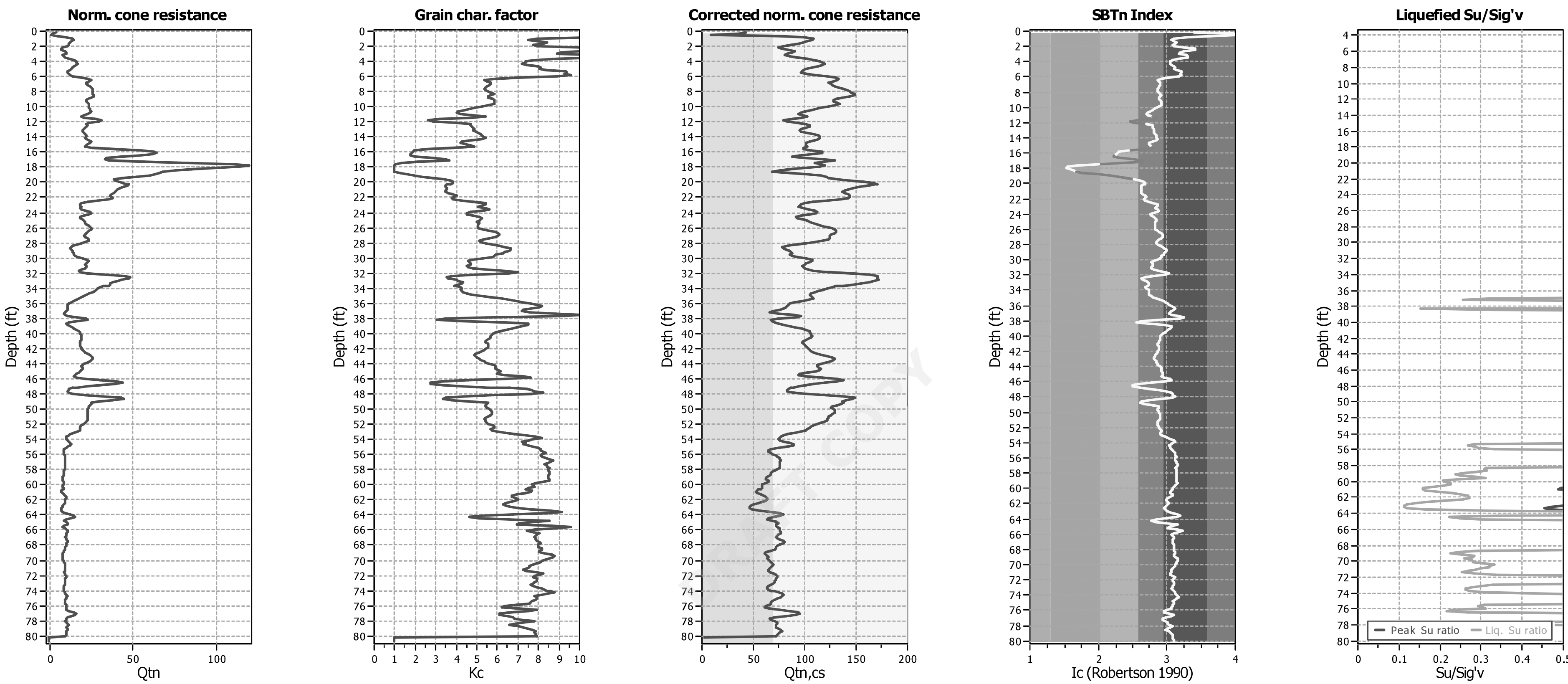




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

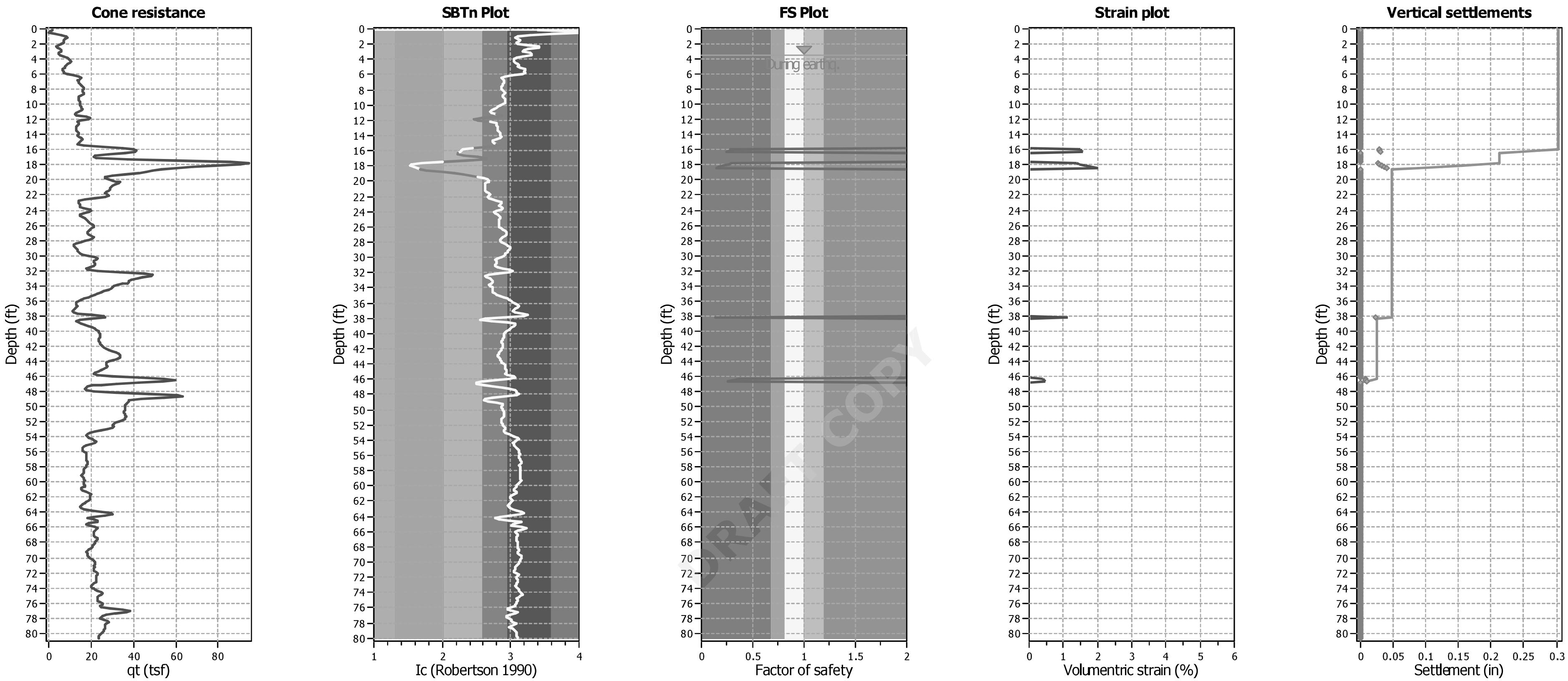
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

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

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
3.61	102.44	2.00	0.00	0.94	0.00	3.77	108.83	2.00	0.00	0.94	0.00
3.94	114.52	2.00	0.00	0.93	0.00	4.10	117.54	2.00	0.00	0.93	0.00
4.27	119.39	2.00	0.00	0.93	0.00	4.43	117.50	2.00	0.00	0.92	0.00
4.59	113.29	2.00	0.00	0.92	0.00	4.76	106.74	2.00	0.00	0.92	0.00
4.92	101.40	2.00	0.00	0.92	0.00	5.09	98.81	2.00	0.00	0.91	0.00
5.25	97.05	2.00	0.00	0.91	0.00	5.41	95.88	2.00	0.00	0.91	0.00
5.58	100.96	2.00	0.00	0.91	0.00	5.74	110.03	2.00	0.00	0.90	0.00
5.91	122.39	2.00	0.00	0.90	0.00	6.07	131.07	2.00	0.00	0.90	0.00
6.23	133.31	2.00	0.00	0.89	0.00	6.40	131.45	2.00	0.00	0.89	0.00
6.56	127.66	2.00	0.00	0.89	0.00	6.73	124.90	2.00	0.00	0.89	0.00
6.89	123.73	2.00	0.00	0.88	0.00	7.05	125.31	2.00	0.00	0.88	0.00
7.22	127.07	2.00	0.00	0.88	0.00	7.38	129.50	2.00	0.00	0.87	0.00
7.55	132.63	2.00	0.00	0.87	0.00	7.71	137.25	2.00	0.00	0.87	0.00
7.87	140.90	2.00	0.00	0.87	0.00	8.04	143.37	2.00	0.00	0.86	0.00
8.20	145.49	2.00	0.00	0.86	0.00	8.37	148.77	2.00	0.00	0.86	0.00
8.53	148.87	2.00	0.00	0.86	0.00	8.69	144.41	2.00	0.00	0.85	0.00
8.86	136.57	2.00	0.00	0.85	0.00	9.02	130.35	2.00	0.00	0.85	0.00
9.19	128.15	2.00	0.00	0.84	0.00	9.35	128.50	2.00	0.00	0.84	0.00
9.51	131.20	2.00	0.00	0.84	0.00	9.68	133.58	2.00	0.00	0.84	0.00
9.84	129.54	2.00	0.00	0.83	0.00	10.01	123.61	2.00	0.00	0.83	0.00
10.17	115.30	2.00	0.00	0.83	0.00	10.33	111.08	2.00	0.00	0.82	0.00
10.50	105.94	2.00	0.00	0.82	0.00	10.66	100.37	2.00	0.00	0.82	0.00
10.83	96.14	2.00	0.00	0.82	0.00	10.99	94.10	2.00	0.00	0.81	0.00
11.15	97.80	2.00	0.00	0.81	0.00	11.32	102.26	2.00	0.00	0.81	0.00
11.48	97.91	2.00	0.00	0.81	0.00	11.65	87.21	2.00	0.00	0.80	0.00
11.81	79.66	2.00	0.00	0.80	0.00	11.98	83.38	2.00	0.00	0.80	0.00
12.14	92.66	2.00	0.00	0.79	0.00	12.30	100.11	2.00	0.00	0.79	0.00
12.47	104.44	2.00	0.00	0.79	0.00	12.63	104.49	2.00	0.00	0.79	0.00
12.80	100.88	2.00	0.00	0.78	0.00	12.96	96.62	2.00	0.00	0.78	0.00
13.12	94.74	2.00	0.00	0.78	0.00	13.29	96.65	2.00	0.00	0.77	0.00
13.45	101.71	2.00	0.00	0.77	0.00	13.62	108.00	2.00	0.00	0.77	0.00
13.78	113.45	2.00	0.00	0.77	0.00	13.94	114.74	2.00	0.00	0.76	0.00
14.11	114.83	2.00	0.00	0.76	0.00	14.27	112.88	2.00	0.00	0.76	0.00
14.44	109.54	2.00	0.00	0.76	0.00	14.60	104.79	2.00	0.00	0.75	0.00
14.76	101.65	2.00	0.00	0.75	0.00	14.93	101.10	2.00	0.00	0.75	0.00
15.09	101.54	2.00	0.00	0.74	0.00	15.26	101.96	2.00	0.00	0.74	0.00
15.42	98.53	2.00	0.00	0.74	0.00	15.58	98.97	2.00	0.00	0.74	0.00
15.75	106.37	2.00	0.00	0.73	0.00	15.91	116.80	0.28	1.50	0.73	0.03
16.08	117.60	0.28	1.49	0.73	0.03	16.24	107.90	0.24	1.59	0.72	0.03
16.40	94.65	2.00	0.00	0.72	0.00	16.57	87.42	2.00	0.00	0.72	0.00
16.73	98.67	2.00	0.00	0.72	0.00	16.90	116.54	2.00	0.00	0.71	0.00
17.06	128.89	2.00	0.00	0.71	0.00	17.22	119.36	2.00	0.00	0.71	0.00
17.39	109.20	2.00	0.00	0.71	0.00	17.55	115.34	2.00	0.00	0.70	0.00
17.72	120.02	0.29	1.41	0.70	0.03	17.88	114.28	0.26	1.46	0.70	0.03
18.04	99.13	0.21	1.63	0.69	0.03	18.21	85.38	0.17	1.84	0.69	0.04
18.37	75.55	0.14	2.03	0.69	0.04	18.54	68.12	2.00	0.00	0.69	0.00
18.70	84.96	2.00	0.00	0.68	0.00	18.86	101.12	2.00	0.00	0.68	0.00
19.03	116.20	2.00	0.00	0.68	0.00	19.19	120.13	2.00	0.00	0.67	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
19.36	123.40	2.00	0.00	0.67	0.00	19.52	132.19	2.00	0.00	0.67	0.00
19.69	144.37	2.00	0.00	0.67	0.00	19.85	157.60	2.00	0.00	0.66	0.00
20.01	167.21	2.00	0.00	0.66	0.00	20.18	170.50	2.00	0.00	0.66	0.00
20.34	168.56	2.00	0.00	0.66	0.00	20.51	161.84	2.00	0.00	0.65	0.00
20.67	153.83	2.00	0.00	0.65	0.00	20.83	147.48	2.00	0.00	0.65	0.00
21.00	142.37	2.00	0.00	0.64	0.00	21.16	138.72	2.00	0.00	0.64	0.00
21.33	137.00	2.00	0.00	0.64	0.00	21.49	138.74	2.00	0.00	0.64	0.00
21.65	142.44	2.00	0.00	0.63	0.00	21.82	144.05	2.00	0.00	0.63	0.00
21.98	144.17	2.00	0.00	0.63	0.00	22.15	142.58	2.00	0.00	0.62	0.00
22.31	135.27	2.00	0.00	0.62	0.00	22.47	122.56	2.00	0.00	0.62	0.00
22.64	107.69	2.00	0.00	0.62	0.00	22.80	99.96	2.00	0.00	0.61	0.00
22.97	95.88	2.00	0.00	0.61	0.00	23.13	95.25	2.00	0.00	0.61	0.00
23.29	94.41	2.00	0.00	0.61	0.00	23.46	98.34	2.00	0.00	0.60	0.00
23.62	103.47	2.00	0.00	0.60	0.00	23.79	109.61	2.00	0.00	0.60	0.00
23.95	112.44	2.00	0.00	0.59	0.00	24.11	111.30	2.00	0.00	0.59	0.00
24.28	105.46	2.00	0.00	0.59	0.00	24.44	96.12	2.00	0.00	0.59	0.00
24.61	91.24	2.00	0.00	0.58	0.00	24.77	92.97	2.00	0.00	0.58	0.00
24.93	98.55	2.00	0.00	0.58	0.00	25.10	102.38	2.00	0.00	0.57	0.00
25.26	104.48	2.00	0.00	0.57	0.00	25.43	108.16	2.00	0.00	0.57	0.00
25.59	111.91	2.00	0.00	0.57	0.00	25.75	116.31	2.00	0.00	0.56	0.00
25.92	121.14	2.00	0.00	0.56	0.00	26.08	126.27	2.00	0.00	0.56	0.00
26.25	129.37	2.00	0.00	0.56	0.00	26.41	131.01	2.00	0.00	0.55	0.00
26.57	130.47	2.00	0.00	0.55	0.00	26.74	128.79	2.00	0.00	0.55	0.00
26.90	126.03	2.00	0.00	0.54	0.00	27.07	124.33	2.00	0.00	0.54	0.00
27.23	124.50	2.00	0.00	0.54	0.00	27.40	124.94	2.00	0.00	0.54	0.00
27.56	123.18	2.00	0.00	0.53	0.00	27.72	117.54	2.00	0.00	0.53	0.00
27.89	109.77	2.00	0.00	0.53	0.00	28.05	100.76	2.00	0.00	0.52	0.00
28.22	91.84	2.00	0.00	0.52	0.00	28.38	83.10	2.00	0.00	0.52	0.00
28.54	77.87	2.00	0.00	0.52	0.00	28.71	78.83	2.00	0.00	0.51	0.00
28.87	82.11	2.00	0.00	0.51	0.00	29.04	85.68	2.00	0.00	0.51	0.00
29.20	86.34	2.00	0.00	0.51	0.00	29.36	87.62	2.00	0.00	0.50	0.00
29.53	85.83	2.00	0.00	0.50	0.00	29.69	87.51	2.00	0.00	0.50	0.00
29.86	92.37	2.00	0.00	0.49	0.00	30.02	100.74	2.00	0.00	0.49	0.00
30.18	106.84	2.00	0.00	0.49	0.00	30.35	107.20	2.00	0.00	0.49	0.00
30.51	105.42	2.00	0.00	0.48	0.00	30.68	101.09	2.00	0.00	0.48	0.00
30.84	99.63	2.00	0.00	0.48	0.00	31.00	96.96	2.00	0.00	0.47	0.00
31.17	98.47	2.00	0.00	0.47	0.00	31.33	99.75	2.00	0.00	0.47	0.00
31.50	103.80	2.00	0.00	0.47	0.00	31.66	108.94	2.00	0.00	0.46	0.00
31.82	123.74	2.00	0.00	0.46	0.00	31.99	146.92	2.00	0.00	0.46	0.00
32.15	163.58	2.00	0.00	0.46	0.00	32.32	170.14	2.00	0.00	0.45	0.00
32.48	170.50	2.00	0.00	0.45	0.00	32.64	170.76	2.00	0.00	0.45	0.00
32.81	171.94	2.00	0.00	0.44	0.00	32.97	167.27	2.00	0.00	0.44	0.00
33.14	162.87	2.00	0.00	0.44	0.00	33.30	154.82	2.00	0.00	0.44	0.00
33.46	146.11	2.00	0.00	0.43	0.00	33.63	137.35	2.00	0.00	0.43	0.00
33.79	130.28	2.00	0.00	0.43	0.00	33.96	126.53	2.00	0.00	0.42	0.00
34.12	121.66	2.00	0.00	0.42	0.00	34.28	117.98	2.00	0.00	0.42	0.00
34.45	113.62	2.00	0.00	0.42	0.00	34.61	110.13	2.00	0.00	0.41	0.00
34.78	105.91	2.00	0.00	0.41	0.00	34.94	104.34	2.00	0.00	0.41	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
35.10	105.93	2.00	0.00	0.41	0.00
35.43	108.40	2.00	0.00	0.40	0.00
35.76	97.45	2.00	0.00	0.39	0.00
36.09	85.46	2.00	0.00	0.39	0.00
36.42	83.22	2.00	0.00	0.38	0.00
36.75	79.86	2.00	0.00	0.38	0.00
37.07	69.48	2.00	0.00	0.37	0.00
37.40	73.89	2.00	0.00	0.37	0.00
37.73	96.17	2.00	0.00	0.36	0.00
38.06	74.31	2.00	0.00	0.35	0.00
38.39	68.41	2.00	0.00	0.35	0.00
38.71	74.18	2.00	0.00	0.34	0.00
39.04	84.86	2.00	0.00	0.34	0.00
39.37	97.32	2.00	0.00	0.33	0.00
39.70	104.49	2.00	0.00	0.33	0.00
40.03	105.59	2.00	0.00	0.32	0.00
40.35	106.79	2.00	0.00	0.32	0.00
40.68	104.16	2.00	0.00	0.31	0.00
41.01	98.65	2.00	0.00	0.30	0.00
41.34	98.94	2.00	0.00	0.30	0.00
41.67	100.83	2.00	0.00	0.29	0.00
41.99	103.54	2.00	0.00	0.29	0.00
42.32	105.74	2.00	0.00	0.28	0.00
42.65	111.63	2.00	0.00	0.28	0.00
42.98	122.06	2.00	0.00	0.27	0.00
43.31	128.76	2.00	0.00	0.27	0.00
43.64	125.97	2.00	0.00	0.26	0.00
43.96	115.77	2.00	0.00	0.25	0.00
44.29	112.28	2.00	0.00	0.25	0.00
44.62	115.39	2.00	0.00	0.24	0.00
44.95	110.40	2.00	0.00	0.24	0.00
45.28	95.58	2.00	0.00	0.23	0.00
45.60	101.23	2.00	0.00	0.23	0.00
45.93	131.99	2.00	0.00	0.22	0.00
46.26	131.59	0.37	0.40	0.22	0.01
46.59	108.59	0.25	0.46	0.21	0.01
46.92	94.15	2.00	0.00	0.20	0.00
47.24	86.31	2.00	0.00	0.20	0.00
47.57	82.54	2.00	0.00	0.19	0.00
47.90	89.17	2.00	0.00	0.19	0.00
48.23	120.41	2.00	0.00	0.18	0.00
48.56	148.85	2.00	0.00	0.18	0.00
48.88	140.13	2.00	0.00	0.17	0.00
49.21	137.57	2.00	0.00	0.17	0.00
49.54	131.41	2.00	0.00	0.16	0.00
49.87	126.16	2.00	0.00	0.15	0.00
50.20	129.69	2.00	0.00	0.15	0.00
50.52	128.90	2.00	0.00	0.14	0.00
35.27	107.64	2.00	0.00	0.40	0.00
35.60	104.08	2.00	0.00	0.40	0.00
35.93	89.68	2.00	0.00	0.39	0.00
36.25	83.71	2.00	0.00	0.39	0.00
36.58	82.20	2.00	0.00	0.38	0.00
36.91	75.34	2.00	0.00	0.37	0.00
37.24	66.14	2.00	0.00	0.37	0.00
37.57	89.53	2.00	0.00	0.36	0.00
37.89	90.99	2.00	0.00	0.36	0.00
38.22	67.14	0.13	1.14	0.35	0.02
38.55	70.78	2.00	0.00	0.35	0.00
38.88	79.51	2.00	0.00	0.34	0.00
39.21	90.60	2.00	0.00	0.34	0.00
39.53	101.81	2.00	0.00	0.33	0.00
39.86	104.50	2.00	0.00	0.32	0.00
40.19	106.17	2.00	0.00	0.32	0.00
40.52	105.95	2.00	0.00	0.31	0.00
40.85	100.77	2.00	0.00	0.31	0.00
41.17	97.59	2.00	0.00	0.30	0.00
41.50	99.49	2.00	0.00	0.30	0.00
41.83	102.27	2.00	0.00	0.29	0.00
42.16	104.93	2.00	0.00	0.29	0.00
42.49	108.47	2.00	0.00	0.28	0.00
42.81	116.94	2.00	0.00	0.27	0.00
43.14	126.61	2.00	0.00	0.27	0.00
43.47	128.27	2.00	0.00	0.26	0.00
43.80	120.40	2.00	0.00	0.26	0.00
44.13	112.10	2.00	0.00	0.25	0.00
44.46	113.92	2.00	0.00	0.25	0.00
44.78	114.70	2.00	0.00	0.24	0.00
45.11	102.80	2.00	0.00	0.24	0.00
45.44	93.30	2.00	0.00	0.23	0.00
45.77	116.52	2.00	0.00	0.22	0.00
46.10	138.41	2.00	0.00	0.22	0.00
46.42	119.92	0.30	0.43	0.21	0.01
46.75	100.73	2.00	0.00	0.21	0.00
47.08	89.04	2.00	0.00	0.20	0.00
47.41	84.67	2.00	0.00	0.20	0.00
47.74	82.95	2.00	0.00	0.19	0.00
48.06	101.53	2.00	0.00	0.19	0.00
48.39	139.34	2.00	0.00	0.18	0.00
48.72	146.88	2.00	0.00	0.17	0.00
49.05	136.26	2.00	0.00	0.17	0.00
49.38	135.55	2.00	0.00	0.16	0.00
49.70	127.22	2.00	0.00	0.16	0.00
50.03	128.16	2.00	0.00	0.15	0.00
50.36	129.87	2.00	0.00	0.15	0.00
50.69	127.19	2.00	0.00	0.14	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
50.85	125.82	2.00	0.00	0.14	0.00	51.02	123.73	2.00	0.00	0.14	0.00
51.18	122.74	2.00	0.00	0.13	0.00	51.35	121.65	2.00	0.00	0.13	0.00
51.51	121.61	2.00	0.00	0.13	0.00	51.67	119.84	2.00	0.00	0.12	0.00
51.84	116.38	2.00	0.00	0.12	0.00	52.00	111.83	2.00	0.00	0.12	0.00
52.17	108.45	2.00	0.00	0.12	0.00	52.33	106.19	2.00	0.00	0.11	0.00
52.49	105.06	2.00	0.00	0.11	0.00	52.66	102.91	2.00	0.00	0.11	0.00
52.82	100.73	2.00	0.00	0.10	0.00	52.99	95.94	2.00	0.00	0.10	0.00
53.15	90.41	2.00	0.00	0.10	0.00	53.31	83.35	2.00	0.00	0.10	0.00
53.48	79.30	2.00	0.00	0.09	0.00	53.64	76.41	2.00	0.00	0.09	0.00
53.81	75.13	2.00	0.00	0.09	0.00	53.97	74.25	2.00	0.00	0.09	0.00
54.13	75.70	2.00	0.00	0.08	0.00	54.30	79.22	2.00	0.00	0.08	0.00
54.46	85.67	2.00	0.00	0.08	0.00	54.63	89.34	2.00	0.00	0.07	0.00
54.79	89.19	2.00	0.00	0.07	0.00	54.95	82.32	2.00	0.00	0.07	0.00
55.12	73.89	2.00	0.00	0.07	0.00	55.28	67.33	2.00	0.00	0.06	0.00
55.45	64.88	2.00	0.00	0.06	0.00	55.61	65.11	2.00	0.00	0.06	0.00
55.77	67.05	2.00	0.00	0.05	0.00	55.94	68.86	2.00	0.00	0.05	0.00
56.10	71.58	2.00	0.00	0.05	0.00	56.27	73.47	2.00	0.00	0.05	0.00
56.43	75.33	2.00	0.00	0.04	0.00	56.59	76.12	2.00	0.00	0.04	0.00
56.76	76.48	2.00	0.00	0.04	0.00	56.92	75.70	2.00	0.00	0.04	0.00
57.09	75.57	2.00	0.00	0.03	0.00	57.25	75.22	2.00	0.00	0.03	0.00
57.41	76.12	2.00	0.00	0.03	0.00	57.58	76.12	2.00	0.00	0.02	0.00
57.74	75.38	2.00	0.00	0.02	0.00	57.91	73.28	2.00	0.00	0.02	0.00
58.07	70.35	2.00	0.00	0.02	0.00	58.23	68.40	2.00	0.00	0.01	0.00
58.40	67.56	2.00	0.00	0.01	0.00	58.56	67.07	2.00	0.00	0.01	0.00
58.73	65.57	2.00	0.00	0.00	0.00	58.89	63.09	2.00	0.00	0.00	0.00
59.06	61.67	2.00	0.00	0.00	0.00	59.22	62.60	2.00	0.00	0.00	0.00
59.38	65.01	2.00	0.00	0.00	0.00	59.55	64.81	2.00	0.00	0.00	0.00
59.71	62.38	2.00	0.00	0.00	0.00	59.88	59.04	2.00	0.00	0.00	0.00
60.04	58.47	2.00	0.00	0.00	0.00	60.20	59.09	2.00	0.00	0.00	0.00
60.37	59.11	2.00	0.00	0.00	0.00	60.53	57.67	2.00	0.00	0.00	0.00
60.70	55.19	2.00	0.00	0.00	0.00	60.86	53.09	2.00	0.00	0.00	0.00
61.02	52.31	2.00	0.00	0.00	0.00	61.19	54.04	2.00	0.00	0.00	0.00
61.35	56.93	2.00	0.00	0.00	0.00	61.52	60.08	2.00	0.00	0.00	0.00
61.68	62.34	2.00	0.00	0.00	0.00	61.84	63.65	2.00	0.00	0.00	0.00
62.01	63.81	2.00	0.00	0.00	0.00	62.17	62.06	2.00	0.00	0.00	0.00
62.34	58.64	2.00	0.00	0.00	0.00	62.50	54.30	2.00	0.00	0.00	0.00
62.66	50.41	2.00	0.00	0.00	0.00	62.83	47.74	2.00	0.00	0.00	0.00
62.99	46.40	2.00	0.00	0.00	0.00	63.16	46.38	2.00	0.00	0.00	0.00
63.32	48.74	2.00	0.00	0.00	0.00	63.48	56.49	2.00	0.00	0.00	0.00
63.65	67.34	2.00	0.00	0.00	0.00	63.81	75.91	2.00	0.00	0.00	0.00
63.98	78.68	2.00	0.00	0.00	0.00	64.14	75.69	2.00	0.00	0.00	0.00
64.30	69.08	2.00	0.00	0.00	0.00	64.47	64.48	2.00	0.00	0.00	0.00
64.63	62.99	2.00	0.00	0.00	0.00	64.80	68.82	2.00	0.00	0.00	0.00
64.96	73.12	2.00	0.00	0.00	0.00	65.12	74.66	2.00	0.00	0.00	0.00
65.29	72.39	2.00	0.00	0.00	0.00	65.45	71.58	2.00	0.00	0.00	0.00
65.62	72.95	2.00	0.00	0.00	0.00	65.78	75.20	2.00	0.00	0.00	0.00
65.94	76.04	2.00	0.00	0.00	0.00	66.11	75.87	2.00	0.00	0.00	0.00
66.27	76.43	2.00	0.00	0.00	0.00	66.44	76.88	2.00	0.00	0.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
66.60	74.92	2.00	0.00	0.00	0.00	66.77	72.84	2.00	0.00	0.00	0.00
66.93	71.44	2.00	0.00	0.00	0.00	67.09	73.41	2.00	0.00	0.00	0.00
67.26	75.64	2.00	0.00	0.00	0.00	67.42	79.00	2.00	0.00	0.00	0.00
67.59	80.02	2.00	0.00	0.00	0.00	67.75	79.56	2.00	0.00	0.00	0.00
67.91	75.94	2.00	0.00	0.00	0.00	68.08	73.41	2.00	0.00	0.00	0.00
68.24	71.77	2.00	0.00	0.00	0.00	68.41	72.31	2.00	0.00	0.00	0.00
68.57	70.36	2.00	0.00	0.00	0.00	68.73	66.83	2.00	0.00	0.00	0.00
68.90	62.35	2.00	0.00	0.00	0.00	69.06	61.12	2.00	0.00	0.00	0.00
69.23	62.27	2.00	0.00	0.00	0.00	69.39	63.98	2.00	0.00	0.00	0.00
69.55	64.09	2.00	0.00	0.00	0.00	69.72	64.00	2.00	0.00	0.00	0.00
69.88	64.09	2.00	0.00	0.00	0.00	70.05	65.69	2.00	0.00	0.00	0.00
70.21	67.40	2.00	0.00	0.00	0.00	70.37	68.63	2.00	0.00	0.00	0.00
70.54	69.03	2.00	0.00	0.00	0.00	70.70	68.05	2.00	0.00	0.00	0.00
70.87	67.02	2.00	0.00	0.00	0.00	71.03	65.51	2.00	0.00	0.00	0.00
71.19	64.20	2.00	0.00	0.00	0.00	71.36	64.13	2.00	0.00	0.00	0.00
71.52	66.05	2.00	0.00	0.00	0.00	71.69	69.86	2.00	0.00	0.00	0.00
71.85	72.53	2.00	0.00	0.00	0.00	72.01	73.74	2.00	0.00	0.00	0.00
72.18	72.92	2.00	0.00	0.00	0.00	72.34	72.47	2.00	0.00	0.00	0.00
72.51	71.50	2.00	0.00	0.00	0.00	72.67	71.14	2.00	0.00	0.00	0.00
72.83	70.24	2.00	0.00	0.00	0.00	73.00	69.12	2.00	0.00	0.00	0.00
73.16	67.16	2.00	0.00	0.00	0.00	73.33	65.08	2.00	0.00	0.00	0.00
73.49	63.74	2.00	0.00	0.00	0.00	73.65	63.59	2.00	0.00	0.00	0.00
73.82	64.97	2.00	0.00	0.00	0.00	73.98	67.47	2.00	0.00	0.00	0.00
74.15	70.68	2.00	0.00	0.00	0.00	74.31	74.91	2.00	0.00	0.00	0.00
74.48	77.50	2.00	0.00	0.00	0.00	74.64	79.54	2.00	0.00	0.00	0.00
74.80	78.27	2.00	0.00	0.00	0.00	74.97	76.42	2.00	0.00	0.00	0.00
75.13	73.15	2.00	0.00	0.00	0.00	75.30	70.25	2.00	0.00	0.00	0.00
75.46	68.09	2.00	0.00	0.00	0.00	75.62	67.07	2.00	0.00	0.00	0.00
75.79	67.20	2.00	0.00	0.00	0.00	75.95	65.45	2.00	0.00	0.00	0.00
76.12	62.63	2.00	0.00	0.00	0.00	76.28	61.24	2.00	0.00	0.00	0.00
76.44	65.37	2.00	0.00	0.00	0.00	76.61	75.33	2.00	0.00	0.00	0.00
76.77	85.29	2.00	0.00	0.00	0.00	76.94	94.37	2.00	0.00	0.00	0.00
77.10	95.38	2.00	0.00	0.00	0.00	77.26	91.44	2.00	0.00	0.00	0.00
77.43	81.34	2.00	0.00	0.00	0.00	77.59	71.25	2.00	0.00	0.00	0.00
77.76	65.72	2.00	0.00	0.00	0.00	77.92	66.76	2.00	0.00	0.00	0.00
78.08	70.56	2.00	0.00	0.00	0.00	78.25	72.78	2.00	0.00	0.00	0.00
78.41	73.03	2.00	0.00	0.00	0.00	78.58	71.96	2.00	0.00	0.00	0.00
78.74	71.43	2.00	0.00	0.00	0.00	78.90	72.05	2.00	0.00	0.00	0.00
79.07	74.93	2.00	0.00	0.00	0.00	79.23	76.76	2.00	0.00	0.00	0.00
79.40	77.46	2.00	0.00	0.00	0.00	79.56	75.64	2.00	0.00	0.00	0.00
79.72	74.44	2.00	0.00	0.00	0.00	79.89	72.76	2.00	0.00	0.00	0.00
80.05	71.42	2.00	0.00	0.00	0.00	80.22	-1.00	2.00	0.00	0.00	0.00
80.38	-1.00	2.00	0.00	0.00	0.00	80.54	-1.00	2.00	0.00	0.00	0.00
80.71	-1.00	2.00	0.00	0.00	0.00						

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)

Total estimated settlement: 0.30

Abbreviations

$Q_{tn,cs}$: Equivalent clean sand normalized cone resistance
FS: Factor of safety against liquefaction
e_v (%): Post-liquefaction volumentric strain
DF: e_v depth weighting factor
Settlement: Calculated settlement

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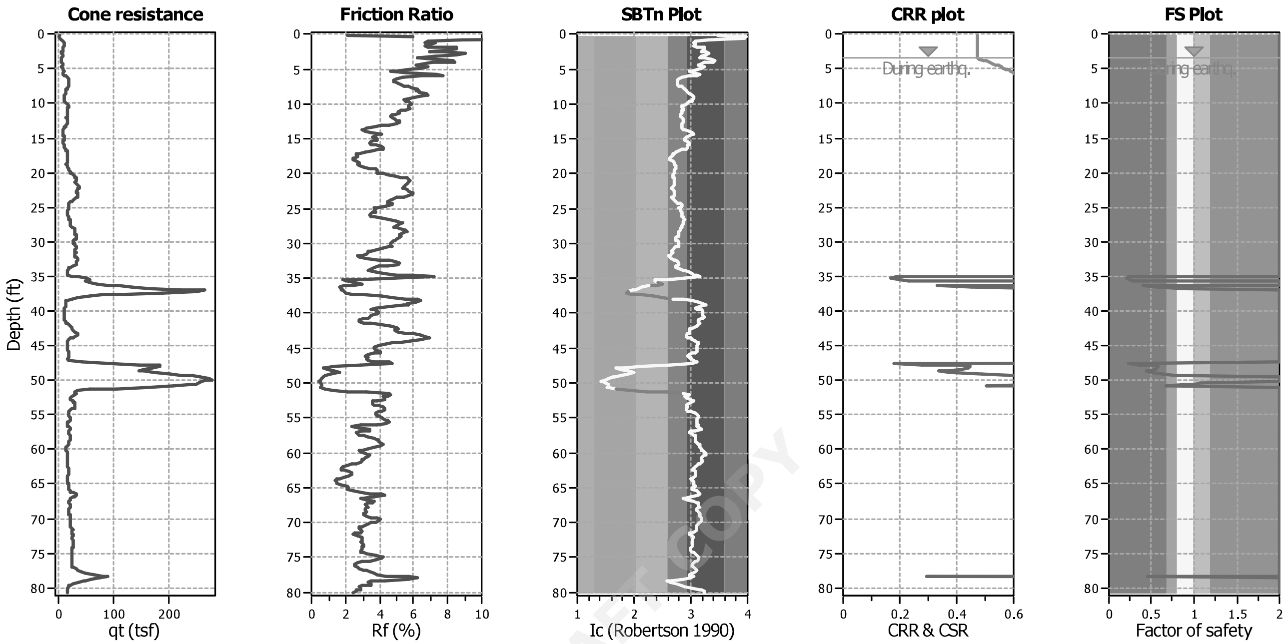
LIQUEFACTION ANALYSIS REPORT

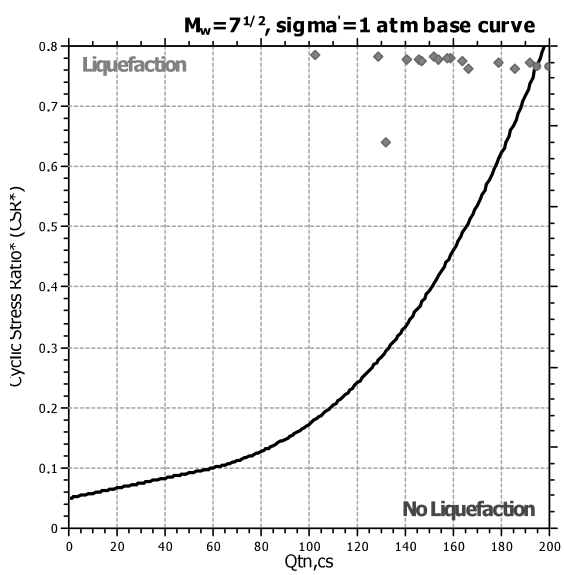
Project title : 5200 Patrick Henry Drive, LLC **Location : 5200 Patrick Henry Drive, Santa Clara**

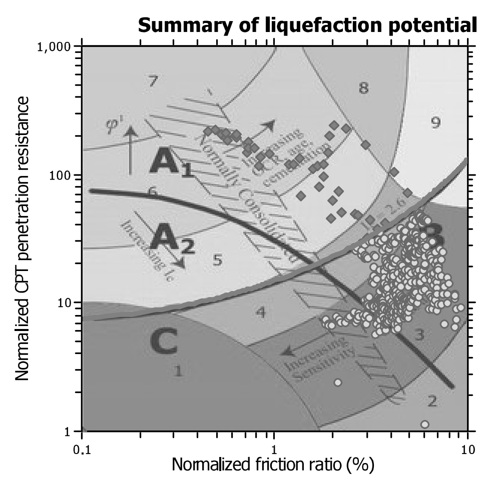
CPT file : CPT-05

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.30 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	3.50 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:	8.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.61	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

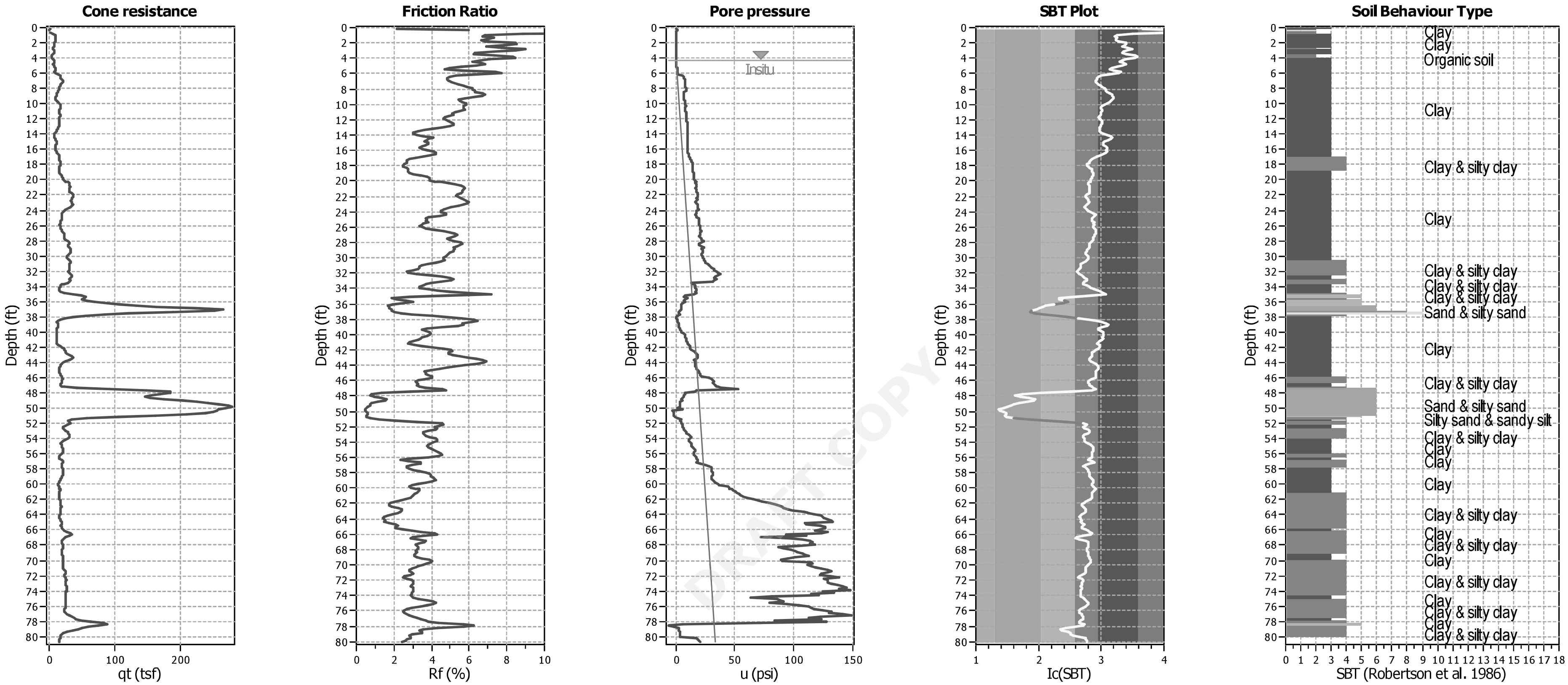






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



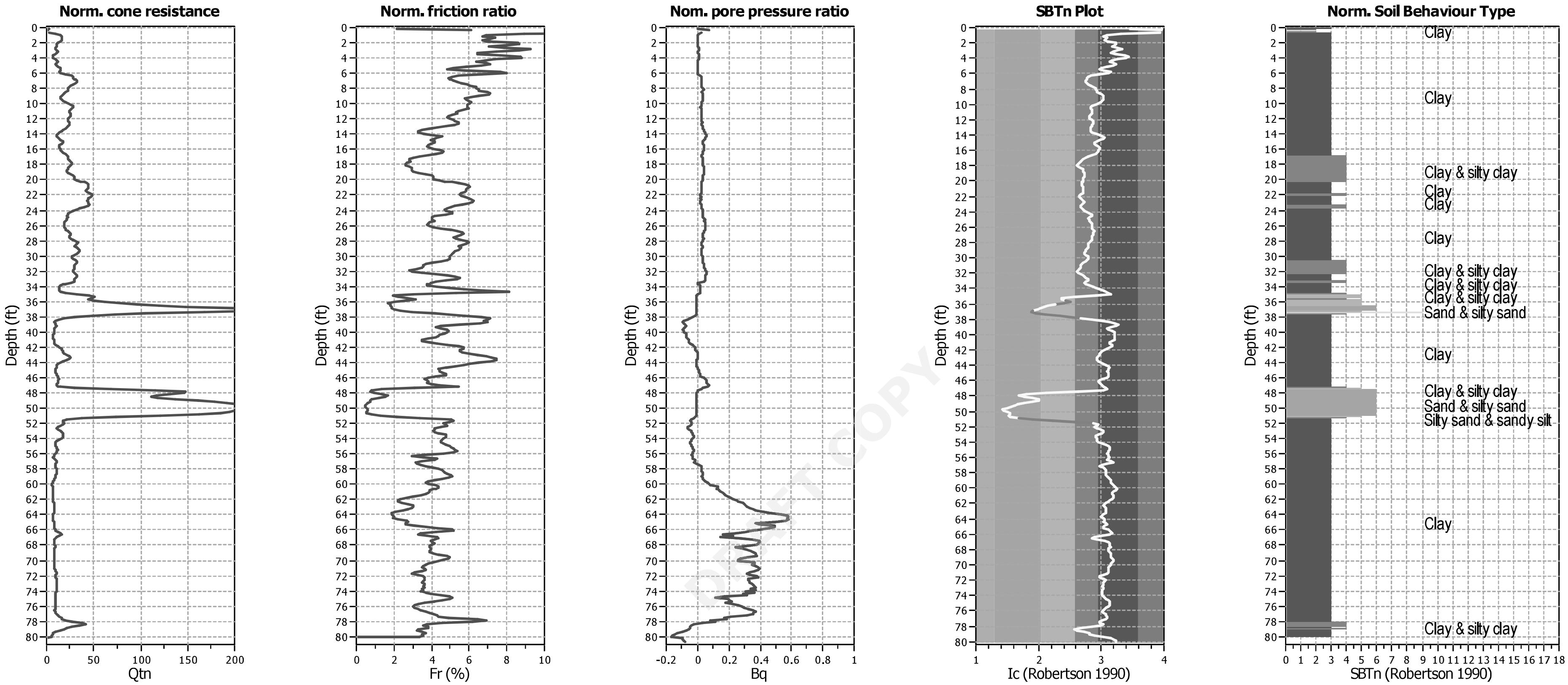
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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

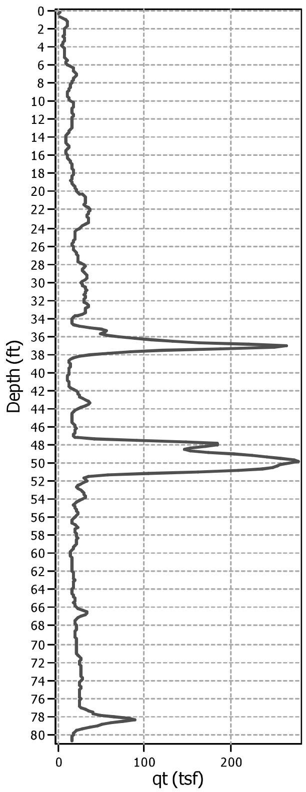
Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A		
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_o applied:	Yes		
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A		

SBTn legend

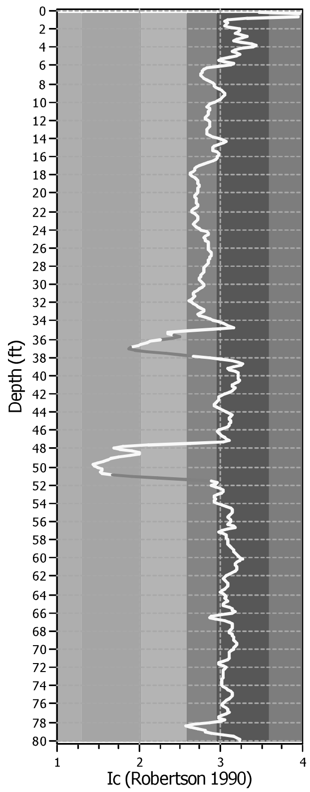
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)

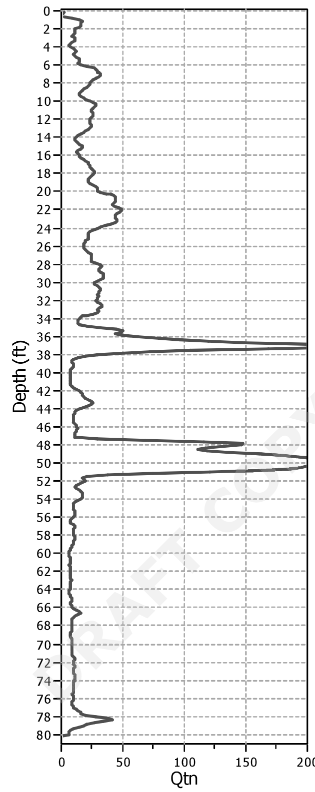
Total cone resistance



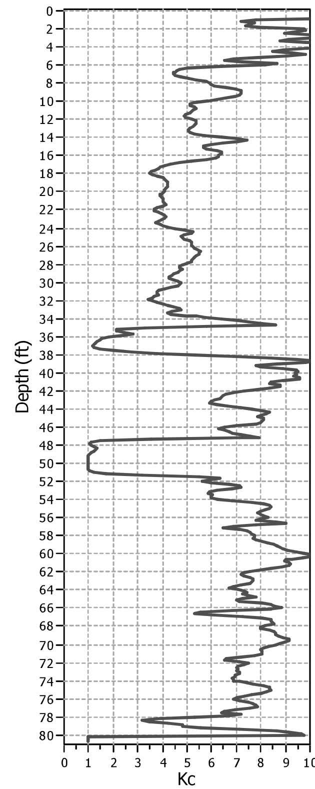
SBTn Index



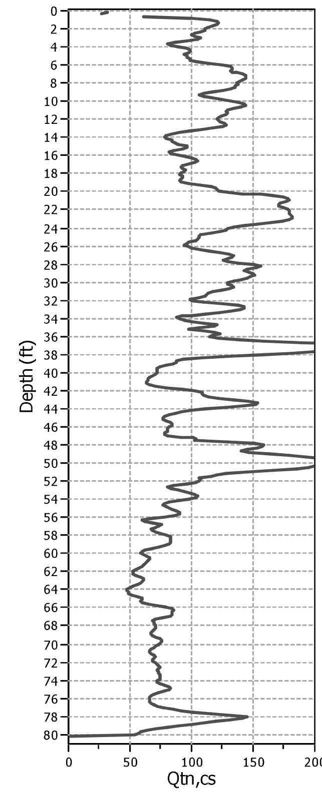
Norm. cone resistance



Grain char. factor



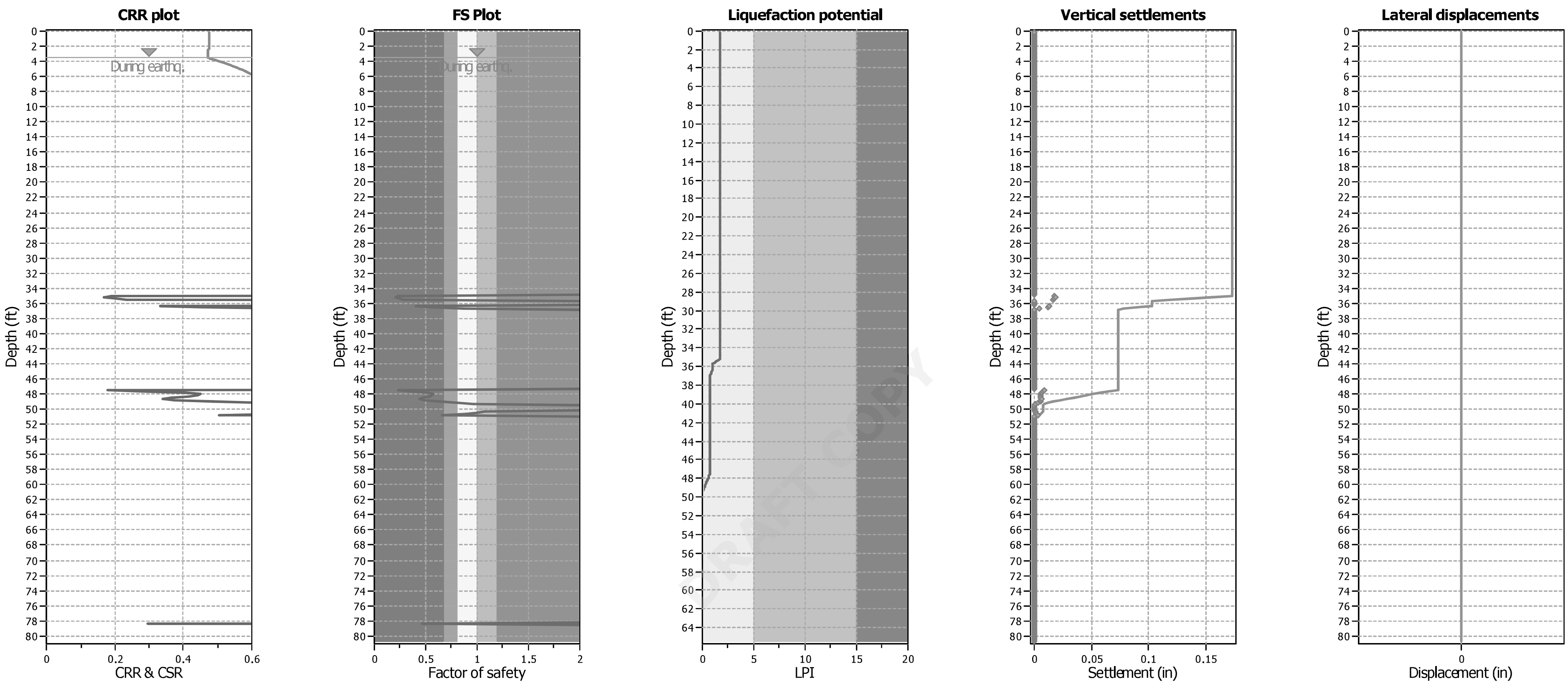
Corrected norm. cone resistance



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 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.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

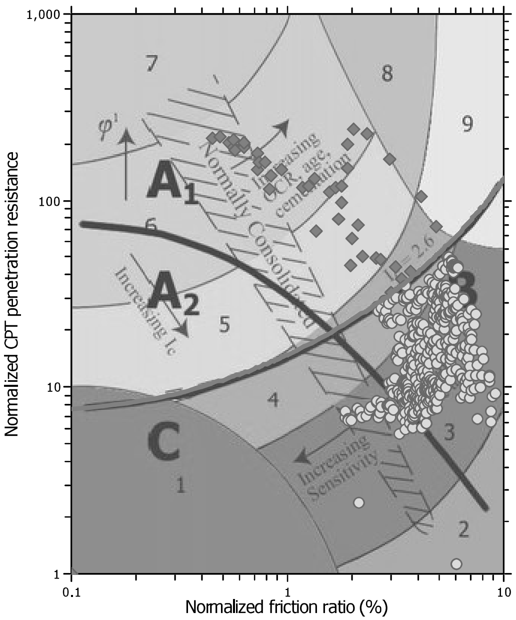
F.S. color scheme

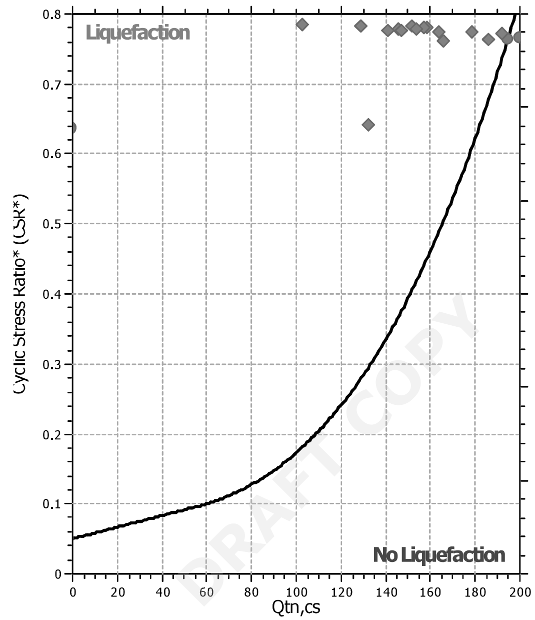
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

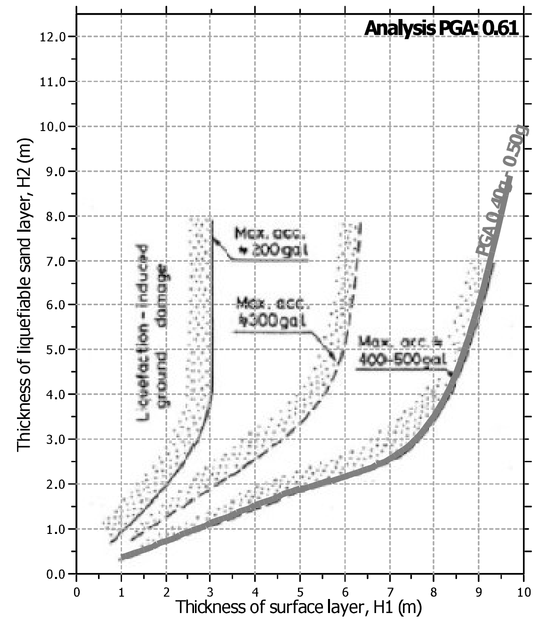
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



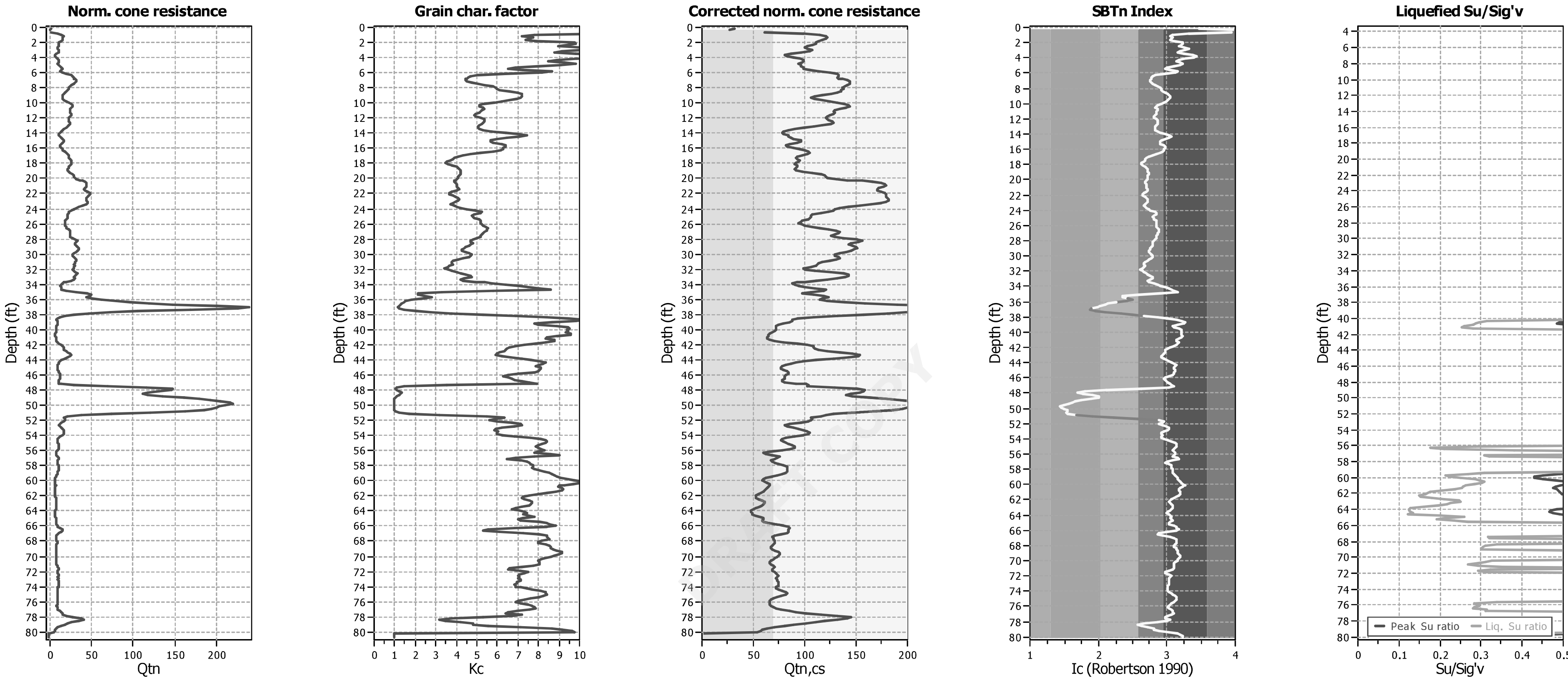




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

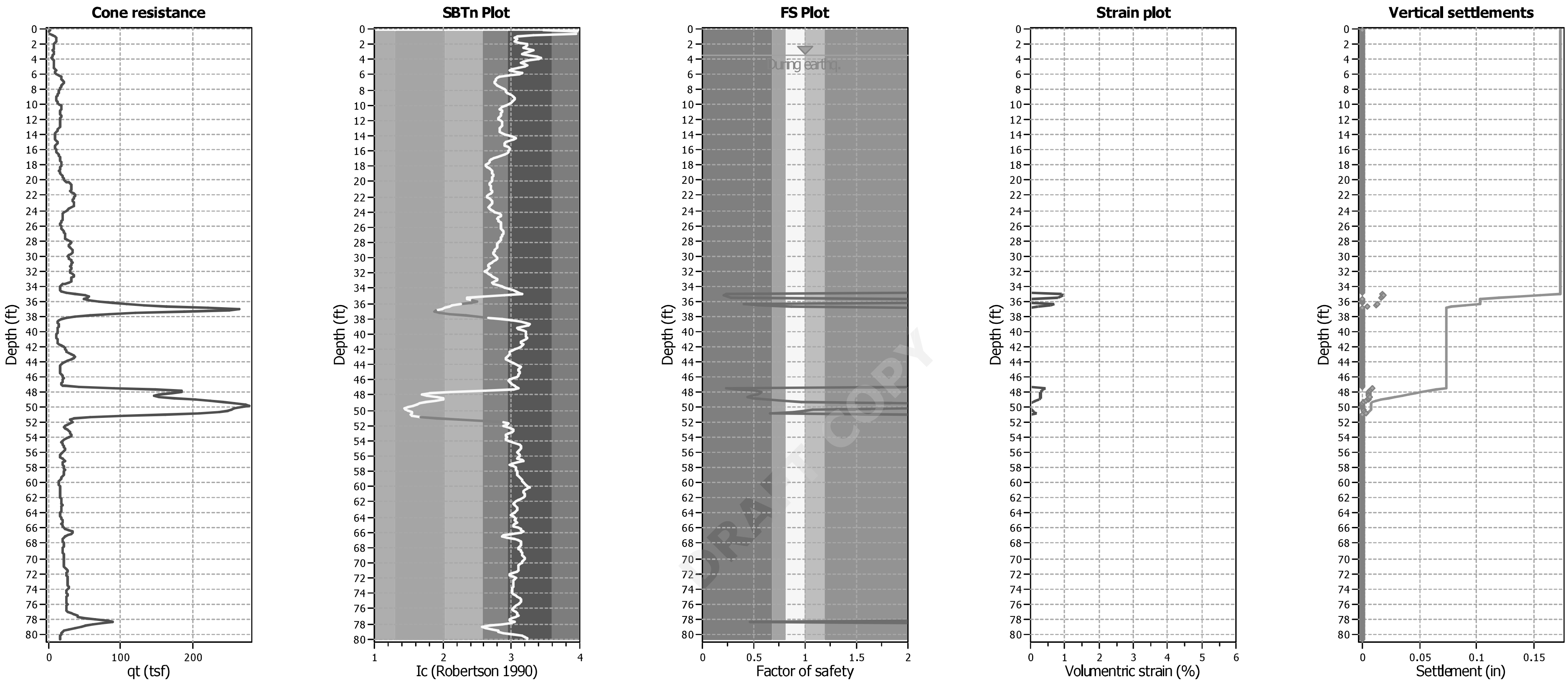
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	3.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	8.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	4.30 ft	Fill height:	N/A	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

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

:: Post-earthquake settlement due to soil liquefaction ::

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
3.61	80.39	2.00	0.00	0.94	0.00	3.77	82.61	2.00	0.00	0.94	0.00
3.94	88.69	2.00	0.00	0.93	0.00	4.10	95.96	2.00	0.00	0.93	0.00
4.27	99.30	2.00	0.00	0.93	0.00	4.43	98.57	2.00	0.00	0.92	0.00
4.59	97.08	2.00	0.00	0.92	0.00	4.76	93.77	2.00	0.00	0.92	0.00
4.92	96.05	2.00	0.00	0.92	0.00	5.09	96.62	2.00	0.00	0.91	0.00
5.25	99.09	2.00	0.00	0.91	0.00	5.41	98.34	2.00	0.00	0.91	0.00
5.58	104.61	2.00	0.00	0.91	0.00	5.74	113.08	2.00	0.00	0.90	0.00
5.91	124.94	2.00	0.00	0.90	0.00	6.07	132.06	2.00	0.00	0.90	0.00
6.23	133.29	2.00	0.00	0.89	0.00	6.40	132.86	2.00	0.00	0.89	0.00
6.56	131.61	2.00	0.00	0.89	0.00	6.73	133.50	2.00	0.00	0.89	0.00
6.89	137.07	2.00	0.00	0.88	0.00	7.05	141.12	2.00	0.00	0.88	0.00
7.22	143.76	2.00	0.00	0.88	0.00	7.38	144.34	2.00	0.00	0.87	0.00
7.55	144.01	2.00	0.00	0.87	0.00	7.71	140.86	2.00	0.00	0.87	0.00
7.87	139.46	2.00	0.00	0.87	0.00	8.04	136.61	2.00	0.00	0.86	0.00
8.20	137.40	2.00	0.00	0.86	0.00	8.37	135.70	2.00	0.00	0.86	0.00
8.53	135.64	2.00	0.00	0.86	0.00	8.69	131.27	2.00	0.00	0.85	0.00
8.86	126.09	2.00	0.00	0.85	0.00	9.02	117.03	2.00	0.00	0.85	0.00
9.19	109.68	2.00	0.00	0.84	0.00	9.35	105.76	2.00	0.00	0.84	0.00
9.51	109.39	2.00	0.00	0.84	0.00	9.68	116.71	2.00	0.00	0.84	0.00
9.84	124.67	2.00	0.00	0.83	0.00	10.01	131.97	2.00	0.00	0.83	0.00
10.17	138.09	2.00	0.00	0.83	0.00	10.33	142.98	2.00	0.00	0.82	0.00
10.50	143.60	2.00	0.00	0.82	0.00	10.66	140.44	2.00	0.00	0.82	0.00
10.83	134.48	2.00	0.00	0.82	0.00	10.99	129.60	2.00	0.00	0.81	0.00
11.15	128.10	2.00	0.00	0.81	0.00	11.32	129.71	2.00	0.00	0.81	0.00
11.48	127.97	2.00	0.00	0.81	0.00	11.65	124.83	2.00	0.00	0.80	0.00
11.81	121.45	2.00	0.00	0.80	0.00	11.98	120.90	2.00	0.00	0.80	0.00
12.14	121.57	2.00	0.00	0.79	0.00	12.30	123.22	2.00	0.00	0.79	0.00
12.47	126.96	2.00	0.00	0.79	0.00	12.63	128.12	2.00	0.00	0.79	0.00
12.80	125.06	2.00	0.00	0.78	0.00	12.96	117.37	2.00	0.00	0.78	0.00
13.12	108.50	2.00	0.00	0.78	0.00	13.29	98.88	2.00	0.00	0.77	0.00
13.45	90.62	2.00	0.00	0.77	0.00	13.62	83.73	2.00	0.00	0.77	0.00
13.78	79.15	2.00	0.00	0.77	0.00	13.94	77.80	2.00	0.00	0.76	0.00
14.11	79.15	2.00	0.00	0.76	0.00	14.27	84.26	2.00	0.00	0.76	0.00
14.44	83.62	2.00	0.00	0.76	0.00	14.60	86.91	2.00	0.00	0.75	0.00
14.76	89.14	2.00	0.00	0.75	0.00	14.93	96.04	2.00	0.00	0.75	0.00
15.09	96.33	2.00	0.00	0.74	0.00	15.26	92.29	2.00	0.00	0.74	0.00
15.42	85.19	2.00	0.00	0.74	0.00	15.58	81.70	2.00	0.00	0.74	0.00
15.75	83.42	2.00	0.00	0.73	0.00	15.91	89.17	2.00	0.00	0.73	0.00
16.08	94.84	2.00	0.00	0.73	0.00	16.24	99.42	2.00	0.00	0.72	0.00
16.40	103.36	2.00	0.00	0.72	0.00	16.57	104.58	2.00	0.00	0.72	0.00
16.73	102.56	2.00	0.00	0.72	0.00	16.90	97.57	2.00	0.00	0.71	0.00
17.06	92.59	2.00	0.00	0.71	0.00	17.22	90.92	2.00	0.00	0.71	0.00
17.39	92.22	2.00	0.00	0.71	0.00	17.55	94.58	2.00	0.00	0.70	0.00
17.72	93.31	2.00	0.00	0.70	0.00	17.88	91.94	2.00	0.00	0.70	0.00
18.04	90.74	2.00	0.00	0.69	0.00	18.21	92.14	2.00	0.00	0.69	0.00
18.37	92.69	2.00	0.00	0.69	0.00	18.54	91.72	2.00	0.00	0.69	0.00
18.70	90.40	2.00	0.00	0.68	0.00	18.86	91.86	2.00	0.00	0.68	0.00
19.03	97.23	2.00	0.00	0.68	0.00	19.19	106.80	2.00	0.00	0.67	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
19.36	116.12	2.00	0.00	0.67	0.00	19.52	119.93	2.00	0.00	0.67	0.00
19.69	120.27	2.00	0.00	0.67	0.00	19.85	121.61	2.00	0.00	0.66	0.00
20.01	130.00	2.00	0.00	0.66	0.00	20.18	142.02	2.00	0.00	0.66	0.00
20.34	155.86	2.00	0.00	0.66	0.00	20.51	166.13	2.00	0.00	0.65	0.00
20.67	174.32	2.00	0.00	0.65	0.00	20.83	177.68	2.00	0.00	0.65	0.00
21.00	179.23	2.00	0.00	0.64	0.00	21.16	176.40	2.00	0.00	0.64	0.00
21.33	172.35	2.00	0.00	0.64	0.00	21.49	170.22	2.00	0.00	0.64	0.00
21.65	171.33	2.00	0.00	0.63	0.00	21.82	174.39	2.00	0.00	0.63	0.00
21.98	178.00	2.00	0.00	0.63	0.00	22.15	179.66	2.00	0.00	0.62	0.00
22.31	179.87	2.00	0.00	0.62	0.00	22.47	179.08	2.00	0.00	0.62	0.00
22.64	180.13	2.00	0.00	0.62	0.00	22.80	181.30	2.00	0.00	0.61	0.00
22.97	181.27	2.00	0.00	0.61	0.00	23.13	178.82	2.00	0.00	0.61	0.00
23.29	174.20	2.00	0.00	0.61	0.00	23.46	164.58	2.00	0.00	0.60	0.00
23.62	154.07	2.00	0.00	0.60	0.00	23.79	142.96	2.00	0.00	0.60	0.00
23.95	134.60	2.00	0.00	0.59	0.00	24.11	129.02	2.00	0.00	0.59	0.00
24.28	127.98	2.00	0.00	0.59	0.00	24.44	122.68	2.00	0.00	0.59	0.00
24.61	116.27	2.00	0.00	0.58	0.00	24.77	107.57	2.00	0.00	0.58	0.00
24.93	106.03	2.00	0.00	0.58	0.00	25.10	105.71	2.00	0.00	0.57	0.00
25.26	104.66	2.00	0.00	0.57	0.00	25.43	103.26	2.00	0.00	0.57	0.00
25.59	98.15	2.00	0.00	0.57	0.00	25.75	96.12	2.00	0.00	0.56	0.00
25.92	94.10	2.00	0.00	0.56	0.00	26.08	97.01	2.00	0.00	0.56	0.00
26.25	99.80	2.00	0.00	0.56	0.00	26.41	106.87	2.00	0.00	0.55	0.00
26.57	114.85	2.00	0.00	0.55	0.00	26.74	123.57	2.00	0.00	0.55	0.00
26.90	130.56	2.00	0.00	0.54	0.00	27.07	133.82	2.00	0.00	0.54	0.00
27.23	132.41	2.00	0.00	0.54	0.00	27.40	128.70	2.00	0.00	0.54	0.00
27.56	125.07	2.00	0.00	0.53	0.00	27.72	128.89	2.00	0.00	0.53	0.00
27.89	138.31	2.00	0.00	0.53	0.00	28.05	150.70	2.00	0.00	0.52	0.00
28.22	155.79	2.00	0.00	0.52	0.00	28.38	152.11	2.00	0.00	0.52	0.00
28.54	145.44	2.00	0.00	0.52	0.00	28.71	142.18	2.00	0.00	0.51	0.00
28.87	146.86	2.00	0.00	0.51	0.00	29.04	149.67	2.00	0.00	0.51	0.00
29.20	150.64	2.00	0.00	0.51	0.00	29.36	147.98	2.00	0.00	0.50	0.00
29.53	145.15	2.00	0.00	0.50	0.00	29.69	140.23	2.00	0.00	0.50	0.00
29.86	133.85	2.00	0.00	0.49	0.00	30.02	129.16	2.00	0.00	0.49	0.00
30.18	129.37	2.00	0.00	0.49	0.00	30.35	132.77	2.00	0.00	0.49	0.00
30.51	133.85	2.00	0.00	0.48	0.00	30.68	130.65	2.00	0.00	0.48	0.00
30.84	124.45	2.00	0.00	0.48	0.00	31.00	116.49	2.00	0.00	0.47	0.00
31.17	113.02	2.00	0.00	0.47	0.00	31.33	112.07	2.00	0.00	0.47	0.00
31.50	111.31	2.00	0.00	0.47	0.00	31.66	106.55	2.00	0.00	0.46	0.00
31.82	98.85	2.00	0.00	0.46	0.00	31.99	99.56	2.00	0.00	0.46	0.00
32.15	109.46	2.00	0.00	0.46	0.00	32.32	125.64	2.00	0.00	0.45	0.00
32.48	137.61	2.00	0.00	0.45	0.00	32.64	143.20	2.00	0.00	0.45	0.00
32.81	142.69	2.00	0.00	0.44	0.00	32.97	139.01	2.00	0.00	0.44	0.00
33.14	131.79	2.00	0.00	0.44	0.00	33.30	122.32	2.00	0.00	0.44	0.00
33.46	111.49	2.00	0.00	0.43	0.00	33.63	100.58	2.00	0.00	0.43	0.00
33.79	92.08	2.00	0.00	0.43	0.00	33.96	88.09	2.00	0.00	0.42	0.00
34.12	90.50	2.00	0.00	0.42	0.00	34.28	93.64	2.00	0.00	0.42	0.00
34.45	100.29	2.00	0.00	0.42	0.00	34.61	109.80	2.00	0.00	0.41	0.00
34.78	120.93	2.00	0.00	0.41	0.00	34.94	117.97	2.00	0.00	0.41	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
35.10	106.13	0.23	0.90	0.41	0.02	35.27	97.73	0.20	0.96	0.40	0.02
35.43	108.79	0.24	0.87	0.40	0.02	35.60	118.04	0.28	0.81	0.40	0.02
35.76	123.34	2.00	0.00	0.39	0.00	35.93	116.91	2.00	0.00	0.39	0.00
36.09	114.24	2.00	0.00	0.39	0.00	36.25	121.74	2.00	0.00	0.39	0.00
36.42	139.35	0.40	0.68	0.38	0.01	36.58	159.01	0.55	0.59	0.38	0.01
36.75	189.73	0.87	0.23	0.38	0.00	36.91	239.25	2.00	0.00	0.37	0.00
37.07	279.34	2.00	0.00	0.37	0.00	37.24	278.98	2.00	0.00	0.37	0.00
37.40	244.57	2.00	0.00	0.37	0.00	37.57	219.53	2.00	0.00	0.36	0.00
37.73	199.27	2.00	0.00	0.36	0.00	37.89	180.89	2.00	0.00	0.36	0.00
38.06	156.74	2.00	0.00	0.35	0.00	38.22	131.40	2.00	0.00	0.35	0.00
38.39	108.21	2.00	0.00	0.35	0.00	38.55	93.26	2.00	0.00	0.35	0.00
38.71	88.18	2.00	0.00	0.34	0.00	38.88	87.87	2.00	0.00	0.34	0.00
39.04	85.51	2.00	0.00	0.34	0.00	39.21	79.86	2.00	0.00	0.34	0.00
39.37	74.72	2.00	0.00	0.33	0.00	39.53	72.36	2.00	0.00	0.33	0.00
39.70	71.89	2.00	0.00	0.33	0.00	39.86	72.27	2.00	0.00	0.32	0.00
40.03	72.19	2.00	0.00	0.32	0.00	40.19	70.46	2.00	0.00	0.32	0.00
40.35	68.04	2.00	0.00	0.32	0.00	40.52	65.78	2.00	0.00	0.31	0.00
40.68	64.74	2.00	0.00	0.31	0.00	40.85	64.08	2.00	0.00	0.31	0.00
41.01	63.21	2.00	0.00	0.30	0.00	41.17	63.30	2.00	0.00	0.30	0.00
41.34	65.33	2.00	0.00	0.30	0.00	41.50	70.31	2.00	0.00	0.30	0.00
41.67	79.77	2.00	0.00	0.29	0.00	41.83	92.42	2.00	0.00	0.29	0.00
41.99	102.81	2.00	0.00	0.29	0.00	42.16	108.46	2.00	0.00	0.29	0.00
42.32	109.04	2.00	0.00	0.28	0.00	42.49	110.24	2.00	0.00	0.28	0.00
42.65	114.00	2.00	0.00	0.28	0.00	42.81	122.51	2.00	0.00	0.27	0.00
42.98	134.30	2.00	0.00	0.27	0.00	43.14	146.40	2.00	0.00	0.27	0.00
43.31	153.46	2.00	0.00	0.27	0.00	43.47	152.23	2.00	0.00	0.26	0.00
43.64	142.46	2.00	0.00	0.26	0.00	43.80	128.00	2.00	0.00	0.26	0.00
43.96	111.49	2.00	0.00	0.25	0.00	44.13	99.57	2.00	0.00	0.25	0.00
44.29	91.02	2.00	0.00	0.25	0.00	44.46	86.24	2.00	0.00	0.25	0.00
44.62	81.66	2.00	0.00	0.24	0.00	44.78	77.91	2.00	0.00	0.24	0.00
44.95	76.62	2.00	0.00	0.24	0.00	45.11	76.92	2.00	0.00	0.24	0.00
45.28	78.96	2.00	0.00	0.23	0.00	45.44	82.41	2.00	0.00	0.23	0.00
45.60	83.92	2.00	0.00	0.23	0.00	45.77	83.96	2.00	0.00	0.22	0.00
45.93	81.78	2.00	0.00	0.22	0.00	46.10	80.82	2.00	0.00	0.22	0.00
46.26	80.67	2.00	0.00	0.22	0.00	46.42	80.37	2.00	0.00	0.21	0.00
46.59	78.27	2.00	0.00	0.21	0.00	46.75	77.73	2.00	0.00	0.21	0.00
46.92	80.73	2.00	0.00	0.20	0.00	47.08	91.11	2.00	0.00	0.20	0.00
47.24	100.64	2.00	0.00	0.20	0.00	47.41	104.06	2.00	0.00	0.20	0.00
47.57	102.81	0.23	0.44	0.19	0.01	47.74	128.90	0.36	0.36	0.19	0.01
47.90	151.82	0.52	0.31	0.19	0.01	48.06	158.45	0.58	0.29	0.19	0.01
48.23	157.30	0.57	0.29	0.18	0.01	48.39	153.74	0.54	0.30	0.18	0.01
48.56	145.41	0.47	0.30	0.18	0.01	48.72	140.66	0.44	0.31	0.17	0.01
48.88	146.79	0.48	0.29	0.17	0.01	49.05	163.76	0.63	0.25	0.17	0.00
49.21	178.83	0.79	0.14	0.17	0.00	49.38	192.11	0.96	0.08	0.16	0.00
49.54	204.40	2.00	0.00	0.16	0.00	49.70	216.51	2.00	0.00	0.16	0.00
49.87	220.10	2.00	0.00	0.15	0.00	50.03	211.45	2.00	0.00	0.15	0.00
50.20	203.24	2.00	0.00	0.15	0.00	50.36	199.93	1.08	0.05	0.15	0.00
50.52	194.94	1.01	0.07	0.14	0.00	50.69	185.67	0.88	0.09	0.14	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
50.85	166.08	0.66	0.17	0.14	0.00	51.02	145.08	2.00	0.00	0.14	0.00
51.18	128.45	2.00	0.00	0.13	0.00	51.35	120.84	2.00	0.00	0.13	0.00
51.51	116.37	2.00	0.00	0.13	0.00	51.67	106.53	2.00	0.00	0.12	0.00
51.84	106.98	2.00	0.00	0.12	0.00	52.00	106.46	2.00	0.00	0.12	0.00
52.17	103.59	2.00	0.00	0.12	0.00	52.33	96.07	2.00	0.00	0.11	0.00
52.49	85.81	2.00	0.00	0.11	0.00	52.66	81.08	2.00	0.00	0.11	0.00
52.82	83.40	2.00	0.00	0.10	0.00	52.99	89.08	2.00	0.00	0.10	0.00
53.15	94.30	2.00	0.00	0.10	0.00	53.31	98.31	2.00	0.00	0.10	0.00
53.48	102.50	2.00	0.00	0.09	0.00	53.64	104.65	2.00	0.00	0.09	0.00
53.81	104.26	2.00	0.00	0.09	0.00	53.97	98.20	2.00	0.00	0.09	0.00
54.13	90.02	2.00	0.00	0.08	0.00	54.30	81.89	2.00	0.00	0.08	0.00
54.46	77.59	2.00	0.00	0.08	0.00	54.63	77.38	2.00	0.00	0.07	0.00
54.79	78.73	2.00	0.00	0.07	0.00	54.95	81.47	2.00	0.00	0.07	0.00
55.12	84.24	2.00	0.00	0.07	0.00	55.28	87.60	2.00	0.00	0.06	0.00
55.45	89.96	2.00	0.00	0.06	0.00	55.61	90.36	2.00	0.00	0.06	0.00
55.77	86.27	2.00	0.00	0.05	0.00	55.94	78.05	2.00	0.00	0.05	0.00
56.10	66.89	2.00	0.00	0.05	0.00	56.27	59.51	2.00	0.00	0.05	0.00
56.43	62.38	2.00	0.00	0.04	0.00	56.59	68.88	2.00	0.00	0.04	0.00
56.76	75.61	2.00	0.00	0.04	0.00	56.92	73.42	2.00	0.00	0.04	0.00
57.09	70.95	2.00	0.00	0.03	0.00	57.25	67.68	2.00	0.00	0.03	0.00
57.41	68.44	2.00	0.00	0.03	0.00	57.58	70.79	2.00	0.00	0.02	0.00
57.74	74.86	2.00	0.00	0.02	0.00	57.91	79.53	2.00	0.00	0.02	0.00
58.07	82.37	2.00	0.00	0.02	0.00	58.23	83.20	2.00	0.00	0.01	0.00
58.40	83.14	2.00	0.00	0.01	0.00	58.56	83.13	2.00	0.00	0.01	0.00
58.73	82.90	2.00	0.00	0.00	0.00	58.89	81.73	2.00	0.00	0.00	0.00
59.06	78.26	2.00	0.00	0.00	0.00	59.22	73.29	2.00	0.00	0.00	0.00
59.38	67.83	2.00	0.00	0.00	0.00	59.55	62.75	2.00	0.00	0.00	0.00
59.71	59.78	2.00	0.00	0.00	0.00	59.88	58.89	2.00	0.00	0.00	0.00
60.04	61.05	2.00	0.00	0.00	0.00	60.20	63.34	2.00	0.00	0.00	0.00
60.37	65.46	2.00	0.00	0.00	0.00	60.53	66.21	2.00	0.00	0.00	0.00
60.70	66.03	2.00	0.00	0.00	0.00	60.86	64.61	2.00	0.00	0.00	0.00
61.02	63.11	2.00	0.00	0.00	0.00	61.19	62.02	2.00	0.00	0.00	0.00
61.35	61.21	2.00	0.00	0.00	0.00	61.52	59.76	2.00	0.00	0.00	0.00
61.68	57.35	2.00	0.00	0.00	0.00	61.84	54.86	2.00	0.00	0.00	0.00
62.01	52.80	2.00	0.00	0.00	0.00	62.17	51.92	2.00	0.00	0.00	0.00
62.34	52.52	2.00	0.00	0.00	0.00	62.50	54.88	2.00	0.00	0.00	0.00
62.66	58.29	2.00	0.00	0.00	0.00	62.83	60.74	2.00	0.00	0.00	0.00
62.99	61.21	2.00	0.00	0.00	0.00	63.16	60.14	2.00	0.00	0.00	0.00
63.32	58.26	2.00	0.00	0.00	0.00	63.48	56.00	2.00	0.00	0.00	0.00
63.65	52.60	2.00	0.00	0.00	0.00	63.81	49.70	2.00	0.00	0.00	0.00
63.98	48.09	2.00	0.00	0.00	0.00	64.14	48.14	2.00	0.00	0.00	0.00
64.30	48.46	2.00	0.00	0.00	0.00	64.47	48.34	2.00	0.00	0.00	0.00
64.63	51.47	2.00	0.00	0.00	0.00	64.80	56.25	2.00	0.00	0.00	0.00
64.96	59.63	2.00	0.00	0.00	0.00	65.12	59.31	2.00	0.00	0.00	0.00
65.29	58.49	2.00	0.00	0.00	0.00	65.45	60.20	2.00	0.00	0.00	0.00
65.62	65.15	2.00	0.00	0.00	0.00	65.78	71.88	2.00	0.00	0.00	0.00
65.94	79.34	2.00	0.00	0.00	0.00	66.11	84.16	2.00	0.00	0.00	0.00
66.27	85.19	2.00	0.00	0.00	0.00	66.44	83.96	2.00	0.00	0.00	0.00

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)
66.60	84.18	2.00	0.00	0.00	0.00	66.77	84.68	2.00	0.00	0.00	0.00
66.93	82.44	2.00	0.00	0.00	0.00	67.09	76.87	2.00	0.00	0.00	0.00
67.26	71.27	2.00	0.00	0.00	0.00	67.42	68.70	2.00	0.00	0.00	0.00
67.59	69.24	2.00	0.00	0.00	0.00	67.75	70.06	2.00	0.00	0.00	0.00
67.91	71.09	2.00	0.00	0.00	0.00	68.08	71.12	2.00	0.00	0.00	0.00
68.24	70.88	2.00	0.00	0.00	0.00	68.41	69.64	2.00	0.00	0.00	0.00
68.57	68.19	2.00	0.00	0.00	0.00	68.73	67.10	2.00	0.00	0.00	0.00
68.90	66.81	2.00	0.00	0.00	0.00	69.06	67.93	2.00	0.00	0.00	0.00
69.23	70.39	2.00	0.00	0.00	0.00	69.39	74.00	2.00	0.00	0.00	0.00
69.55	75.90	2.00	0.00	0.00	0.00	69.72	76.09	2.00	0.00	0.00	0.00
69.88	74.31	2.00	0.00	0.00	0.00	70.05	72.90	2.00	0.00	0.00	0.00
70.21	71.12	2.00	0.00	0.00	0.00	70.37	69.63	2.00	0.00	0.00	0.00
70.54	67.64	2.00	0.00	0.00	0.00	70.70	65.89	2.00	0.00	0.00	0.00
70.87	65.35	2.00	0.00	0.00	0.00	71.03	67.15	2.00	0.00	0.00	0.00
71.19	69.90	2.00	0.00	0.00	0.00	71.36	70.96	2.00	0.00	0.00	0.00
71.52	69.25	2.00	0.00	0.00	0.00	71.69	67.98	2.00	0.00	0.00	0.00
71.85	68.31	2.00	0.00	0.00	0.00	72.01	70.17	2.00	0.00	0.00	0.00
72.18	72.02	2.00	0.00	0.00	0.00	72.34	73.54	2.00	0.00	0.00	0.00
72.51	73.82	2.00	0.00	0.00	0.00	72.67	72.70	2.00	0.00	0.00	0.00
72.83	71.92	2.00	0.00	0.00	0.00	73.00	72.86	2.00	0.00	0.00	0.00
73.16	73.70	2.00	0.00	0.00	0.00	73.33	73.79	2.00	0.00	0.00	0.00
73.49	74.40	2.00	0.00	0.00	0.00	73.65	74.14	2.00	0.00	0.00	0.00
73.82	73.97	2.00	0.00	0.00	0.00	73.98	72.20	2.00	0.00	0.00	0.00
74.15	71.83	2.00	0.00	0.00	0.00	74.31	71.97	2.00	0.00	0.00	0.00
74.48	74.26	2.00	0.00	0.00	0.00	74.64	78.14	2.00	0.00	0.00	0.00
74.80	81.98	2.00	0.00	0.00	0.00	74.97	82.79	2.00	0.00	0.00	0.00
75.13	81.33	2.00	0.00	0.00	0.00	75.30	77.32	2.00	0.00	0.00	0.00
75.46	73.72	2.00	0.00	0.00	0.00	75.62	69.46	2.00	0.00	0.00	0.00
75.79	67.44	2.00	0.00	0.00	0.00	75.95	66.18	2.00	0.00	0.00	0.00
76.12	66.21	2.00	0.00	0.00	0.00	76.28	65.83	2.00	0.00	0.00	0.00
76.44	66.27	2.00	0.00	0.00	0.00	76.61	66.98	2.00	0.00	0.00	0.00
76.77	69.62	2.00	0.00	0.00	0.00	76.94	72.07	2.00	0.00	0.00	0.00
77.10	76.96	2.00	0.00	0.00	0.00	77.26	84.76	2.00	0.00	0.00	0.00
77.43	91.08	2.00	0.00	0.00	0.00	77.59	101.08	2.00	0.00	0.00	0.00
77.76	115.61	2.00	0.00	0.00	0.00	77.92	131.74	2.00	0.00	0.00	0.00
78.08	145.28	2.00	0.00	0.00	0.00	78.25	141.64	2.00	0.00	0.00	0.00
78.41	132.18	0.46	0.00	0.00	0.00	78.58	119.06	2.00	0.00	0.00	0.00
78.74	110.65	2.00	0.00	0.00	0.00	78.90	100.98	2.00	0.00	0.00	0.00
79.07	89.42	2.00	0.00	0.00	0.00	79.23	80.13	2.00	0.00	0.00	0.00
79.40	71.22	2.00	0.00	0.00	0.00	79.56	63.98	2.00	0.00	0.00	0.00
79.72	59.05	2.00	0.00	0.00	0.00	79.89	56.89	2.00	0.00	0.00	0.00
80.05	53.95	2.00	0.00	0.00	0.00	80.22	-1.00	2.00	0.00	0.00	0.00
80.38	-1.00	2.00	0.00	0.00	0.00	80.54	-1.00	2.00	0.00	0.00	0.00
80.71	-1.00	2.00	0.00	0.00	0.00						

:: Post-earthquake settlement due to soil liquefaction :: (continued)

Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)	Depth (ft)	$Q_{tn,cs}$	FS	e_v (%)	DF	Settlement (in)

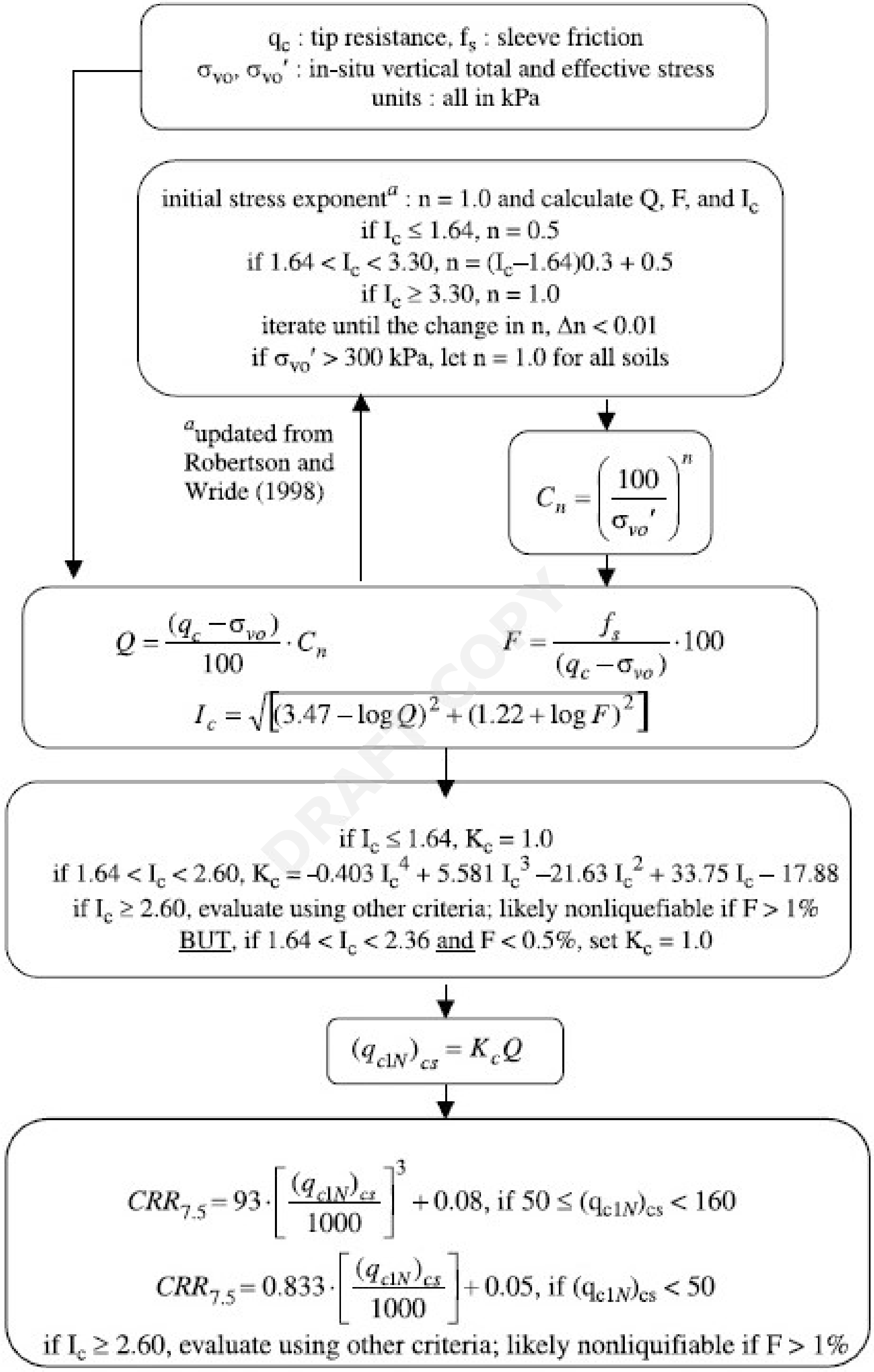
Total estimated settlement: 0.17

Abbreviations

$Q_{tn,cs}$:	Equivalent clean sand normalized cone resistance
FS:	Factor of safety against liquefaction
e_v (%):	Post-liquefaction volumentric strain
DF:	e_v depth weighting factor
Settlement:	Calculated settlement

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

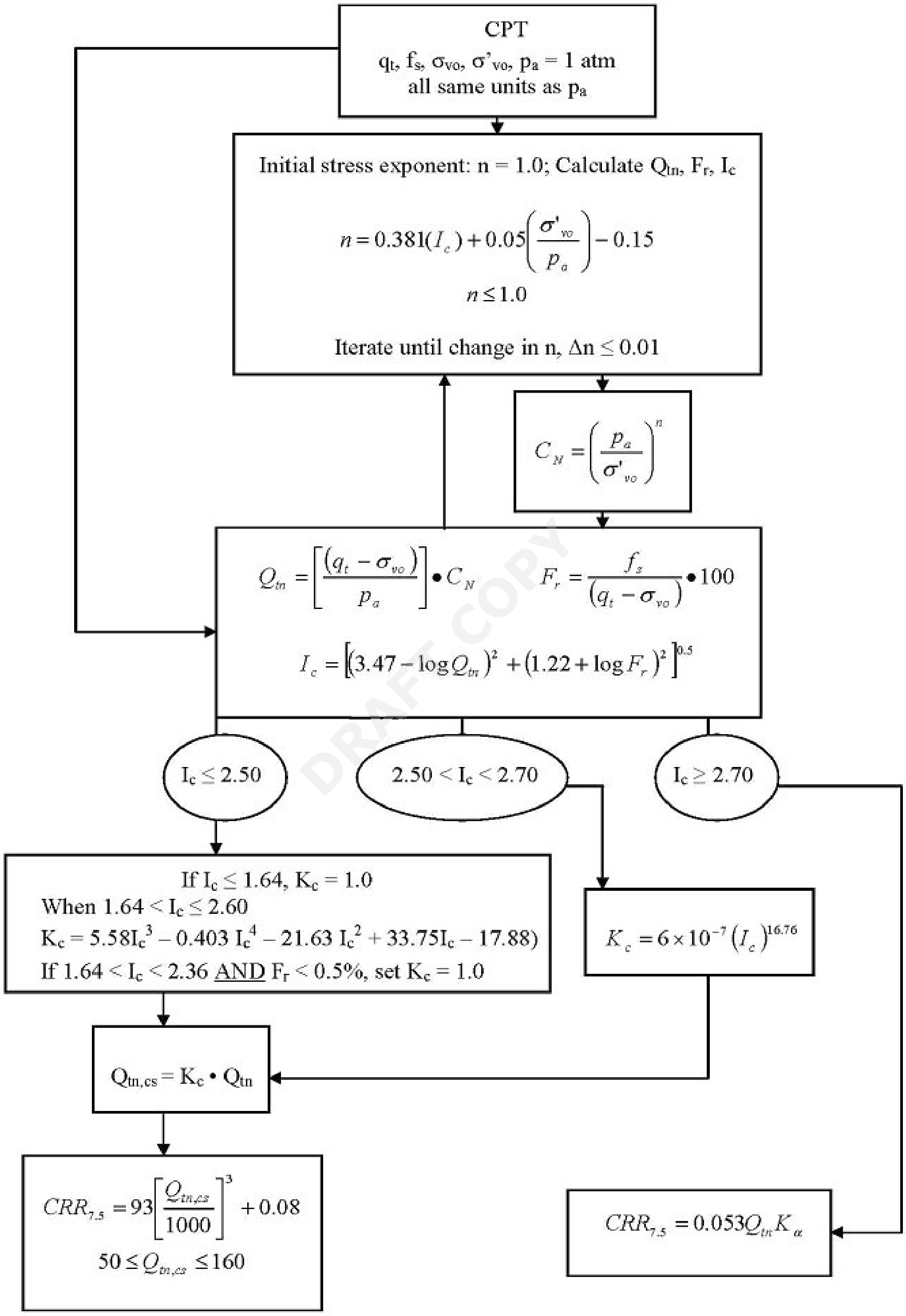
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of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

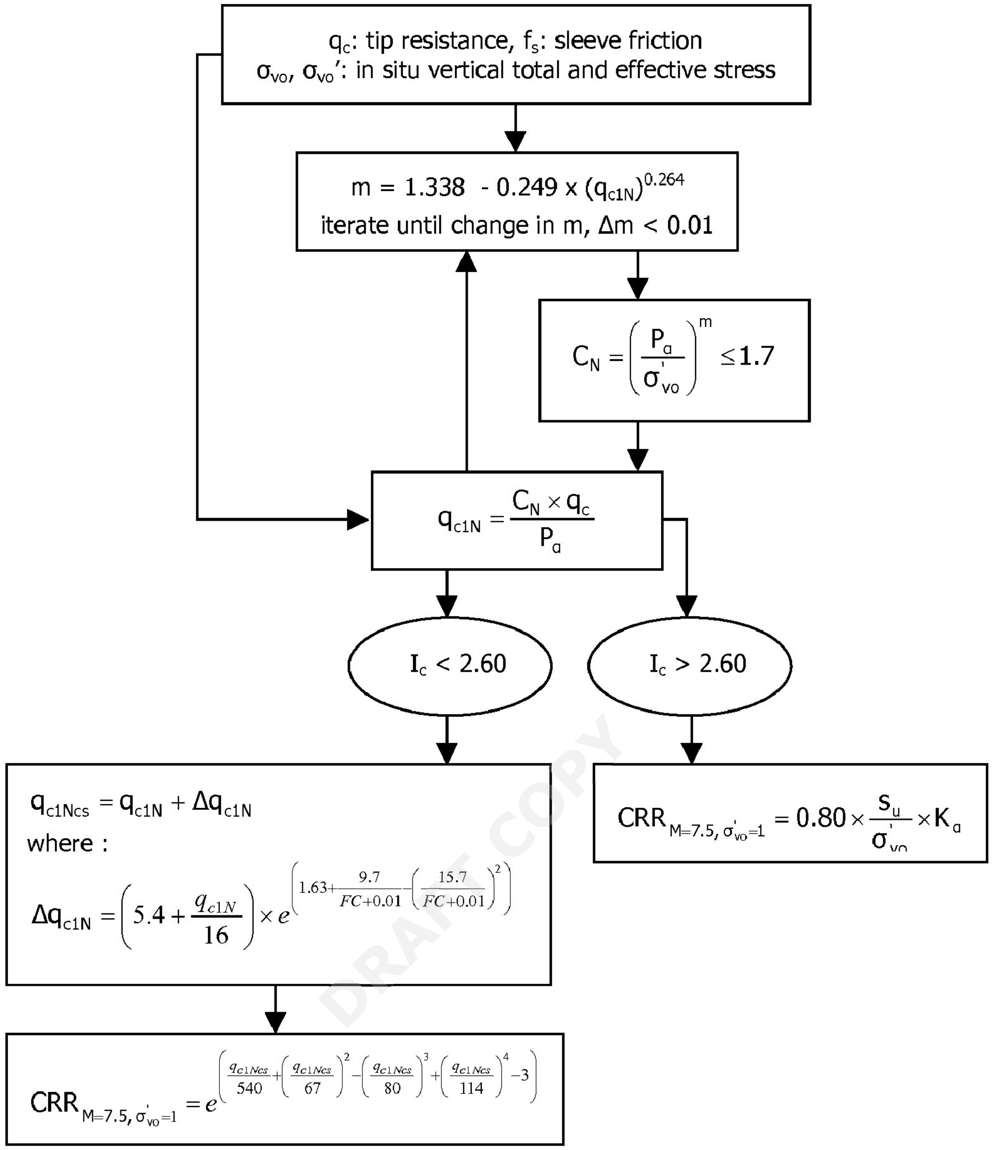
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

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.

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

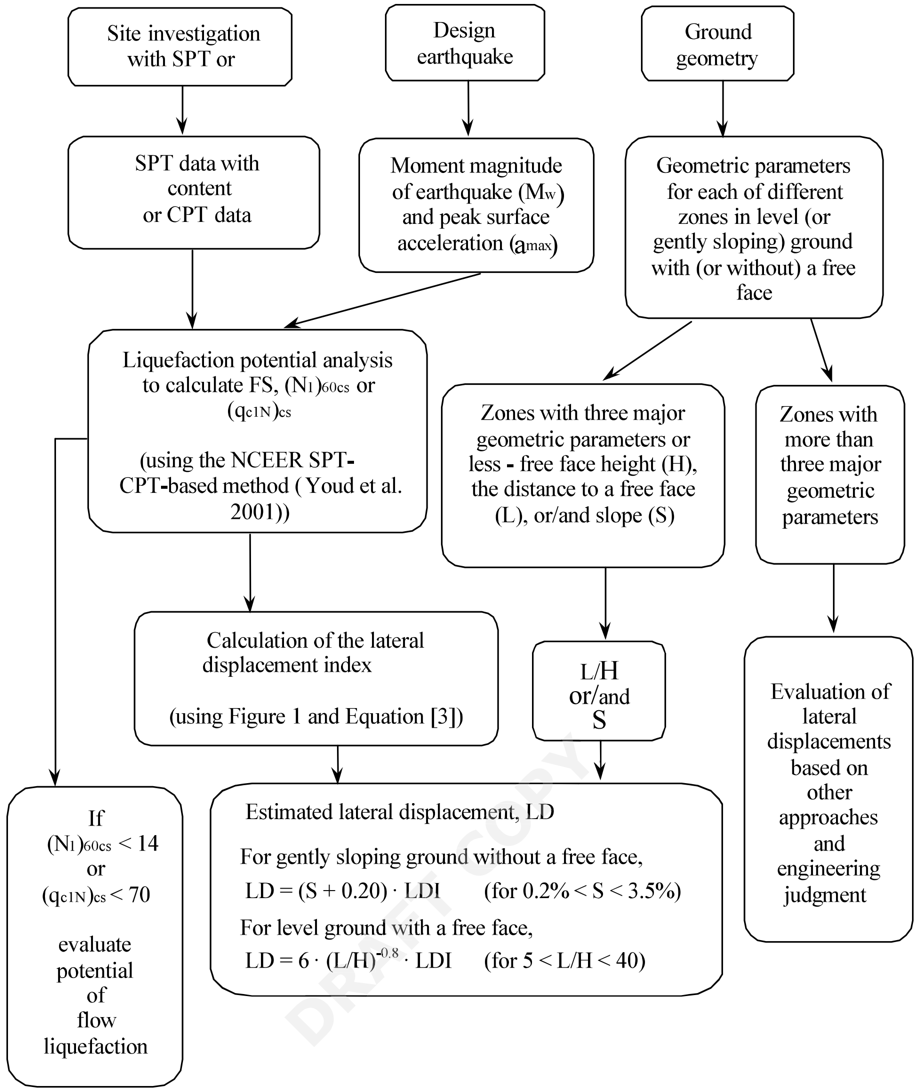
$$I_c = \left[\left(3.47-\log(Q)\right)^2+\left(1.22+\log(F)\right)^2\right]^{0.5}$$

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

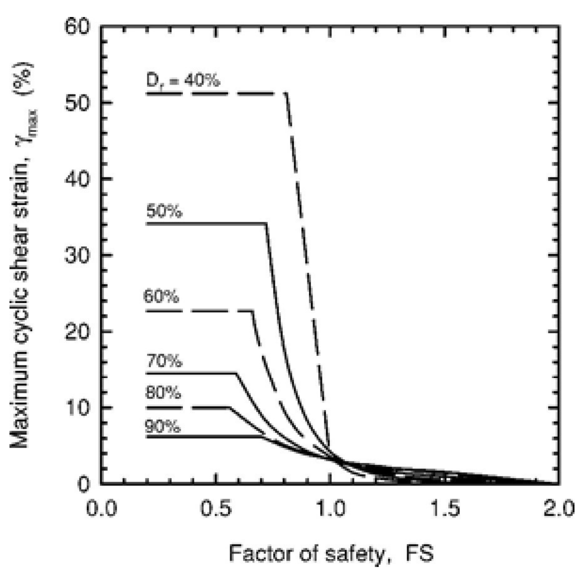
$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

σ'_v *at time of CPT sounding*

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



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



[1] **Figure 1**

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

[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.

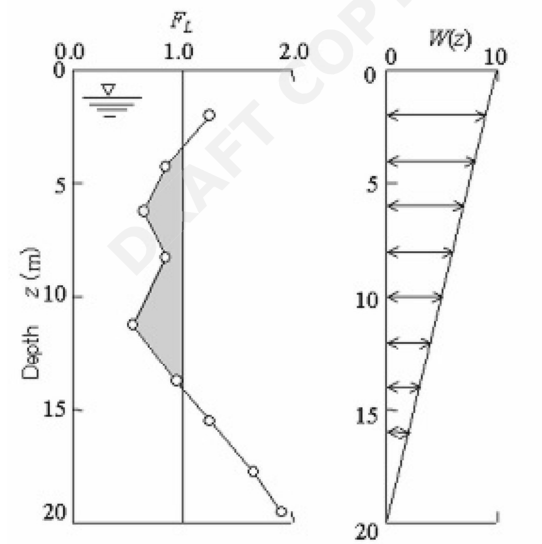
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

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

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

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

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



Graphical presentation of the LPI calculation procedure

Shear-Induced Building Settlement (Ds) calculation procedure

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

$$Ln(Ds) = c1 + c2 * LBS + 0.58 * Ln\left(Tanh\left(\frac{HL}{6}\right)\right) +$$

$$4.59 * Ln(Q) - 0.42 * Ln(Q)^2 - 0.02 * B +$$
$$0.84 * Ln(CAVdp) + 0.41 * Ln(Sa1) + \varepsilon$$

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

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

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

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