Appendix E:
Geotechnical Engineering Report

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Geotechnical Engineering Report

Langton Way Billboard
Hayward, Alameda County, California
May 7, 2021
Terracon Project No. ND215017

Prepared for:
Outfront Media
Berkeley, California

Prepared by:
Terracon Consultants, Inc.
Concord, California

May 7, 2021

Outfront Media
1695 Eastshore Highway
Berkeley, California 94710

Attn: Mr. Jeff McCuen
P: (510) 559 1114
E: jeff.mccuen@outfrontmedia.com

Re: Geotechnical Engineering Report
Langton Way Billboard
Langton Way
Hayward, Alameda County, California
Terracon Project No. ND215017

Dear Mr. McCuen:

We have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. PND215017 dated March 4, 2021. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning the foundation for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

Noah T. Smith, P.E., G.E.
Principal

Garret S.H. Hubbart, P.E., G.E.
Principal

REPORT TOPICS

Note: This report was originally delivered in a web-based format. **Orange Bold** text in the report indicates a referenced section heading. The PDF version also includes hyperlinks which direct the reader to that section and clicking on the ***GeoReport*** logo will bring you back to this page. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

Note: Refer to each individual Attachment for a listing of contents.

Geotechnical Engineering Report
Langton Way Billboard
Langton Way
Hayward, Alameda County, California
Terracon Project No. ND215017
May 7, 2021

INTRODUCTION

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed billboard to be located on Langton Way in Hayward, Alameda County, California. The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Seismic site classification per 2019 CBC
- Foundation design and construction
- Liquefaction

The geotechnical engineering Scope of Services for this project included the advancement of one Cone Penetration Test (CPT) sounding to a depth of 100 feet bgs.

Maps showing the site and CPT location are shown in the **Site Location** and **Exploration Plan** sections, respectively. The results of field testing are included on the CPT log in the **Exploration Results** section.

SITE CONDITIONS

The following description of site conditions was derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The project is located on Langton Way in Hayward, Alameda County, California. 37.6886°N 122.1054°W (approximate) See **Site Location**
Existing Improvements	The site is currently developed with a paved parking area.
Current Ground Cover	Asphalt pavement.
Existing Topography (from Google Earth Pro)	The site is relatively flat.

Item	Description
Geology	Geologic maps indicate subsurface conditions consist of Holocene age alluvial clays and sands of valley areas.[1] The subgrade conditions encountered in our CPT were generally consistent with the mapped geology.

PROJECT DESCRIPTION

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	The site address and following information was provided to Terracon by Outfront Media via email. ■ Preliminary Site Plan prepared by Chappell Surveying, Inc., dated February 11, 2021
Existing Report	Terracon prepared a geotechnical engineering report dated March 12, 2020 (Terracon project number: ND205015) for construction of a billboard at the end of the cul-de-sac on Langton Way. A CPT was advanced on February 26, 2020 at the end of the cul-de-sac approximately 170 feet northwest of the CPT advanced for this report.
Project Description	The project will consist of the construction of an 80-foot-tall single post billboard.
Proposed Billboard	80-foot-tall single-post metal frame billboard supported by an approximately 5-foot diameter pier.
Maximum Loads (assumed)	■ Shear: 27 kips ■ Moment: 1,793 kip-feet ■ Axial: 61 kips
Grading	We understand no grading is expected to take place as part of the billboard construction.

GEOTECHNICAL CHARACTERIZATION

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, geologic setting and our understanding of the project.

[1] *Dibblee, T.W., and Minch, J.A., 2005, Geologic map of the Hayward quadrangle, Contra Costa and Alameda Counties, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-163, scale 1:24,000*

Conditions encountered at the exploration point are indicated on the individual CPT log. The individual CPT log can be found in the **Exploration Results** section of this report.

Groundwater Conditions

A pore pressure dissipation test was performed in the CPT to help determine the groundwater level. The water level observed in the CPT can be found on the CPT log in **Exploration Results** and is summarized below.

CPT Number	Approximate Depth to Groundwater whileTesting[1] (feet)
CPT1	25

1. Below ground surface

Groundwater level fluctuations occur due to seasonal variations in the amount of rainfall, runoff and other factors not evident at the time the CPT was performed. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the CPT log. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

SEISMIC CONSIDERATIONS

The 2019 California Building Code (CBC) Seismic Design Parameters have been generated using the SEAOC/OSHPD Seismic Design Maps Tool. This web-based software application calculates seismic design parameters in accordance with ASCE 7-16 and 2019 CBC. The 2019 CBC requires that a site-specific ground motion study be performed in accordance with Section 11.4.8 of ASCE 7-16 for Site Class D sites with a mapped S_1 value greater than or equal 0.2.

However, Section 11.4.8 of ASCE 7-16 includes an exception from such analysis for specific structures on Site Class D sites. The commentary for Section 11 of ASCE 7-16 (Page 534 of Section C11 of ASCE 7-16) states that “In general, this exception effectively limits the requirements for site-specific hazard analysis to very tall and or flexible structures at Site Class D sites.” Based on our understanding of the proposed structure, it is our assumption that the exception in Section 11.4.8 applies to the proposed structure. However, the structural engineer should verify the applicability of this exception.

Description	Value
2019 California Building Code Site Classification (CBC) [1]	F[2,4]
Site Latitude	37.6886°N
Site Longitude	122.1054°W

Description	Value
S_s, Spectral Acceleration for a Short Period [3]	2.26
S_1, Spectral Acceleration for a 1-Second Period [3]	0.874
F_a, Site Coefficient [3]	1.0
F_v, Site Coefficient (1-second period) [3]	1.7
$S_{DS,}$ Spectral Acceleration for a Short Period [3]	1.507
$S_{D1,}$ Spectral Acceleration for a 1-Second Period [3]	0.991

1. Seismic site classification in general accordance with the *2019 California Building Code*.
2. The 2019 California Building Code (CBC) requires a site soil profile determination extending a depth of 100 feet for seismic site classification. Seismic shear wave velocity measurements were obtained from a CPT advanced to a depth of 100 feet bgs to determine the site classification.
3. These values were obtained using online seismic design maps and tools provided by the SEAOC/OSHPD (https://seismicmaps.org/).
4. The site qualifies as a site class F due to the presence of liquefiable soils. However, a site class D was used to develop the listed seismic design parameters based on shear wave velocity measurements that were obtained from a CPT advanced to a depth of 100 feet bgs. Based on the exception for liquefiable soils provided in ASCE 7-16 Section 20.3.1, structures may use the listed design parameters provided they have a period of 0.5s or less. Should the anticipated structures have a period greater than 0.5s, a site response analysis should be conducted to develop seismic design parameters. Terracon is qualified to perform such an analysis.

Faulting and Estimated Ground Motions

The site is located in the San Francisco Bay Area of California, which is a relatively high seismicity region. The type and magnitude of seismic hazards affecting the site are dependent on the distance to causative faults, the intensity, and the magnitude of the seismic event. The following table indicates the distance of the fault zones and the associated maximum credible earthquake that can be produced by nearby seismic events, as calculated using the USGS Unified Hazard Tool. Segments Hayward Fault, which is located approximately 3 kilometers from the site, are considered to have the most significant effect at the site from a design standpoint.

Characteristics and Estimated Earthquakes for Regional Faults			
Fault Name	**Approximate Contribution (%)**	**Approximate Distance to Site (kilometers)**	**Maximum Credible Earthquake (MCE) Magnitude**
Hayward (So) [6], UC33brAvg_FM31	34.38	2.94	7.05
Hayward (So) [6], UC33brAvg_FM32	34.35	2.94	7.05

Based on the ASCE 7-16 Standard, the peak ground acceleration (PGA_M) at the subject site is approximately 1.045g. Based on the USGS 2014 interactive deaggregations, the PGA at the subject site for a 2% probability of exceedance in 50 years (return period of 2475 years) is expected to be about 1.129g. The site is not located within an Alquist-Priolo Earthquake Fault Zone based on our review of the State Fault Hazard Maps.[1]

LIQUEFACTION

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils or low plasticity fine grained soils exist below groundwater. The California Geologic Survey (CGS) has designated certain areas within California as potential liquefaction hazard zones. These are areas considered at a risk of liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table. The project site is located in an area designated by the CGS as having a potential for earthquake-induced liquefaction. Therefore, a liquefaction analysis was performed to determine the liquefaction induced settlement.

Our liquefaction hazard evaluation was performed in general compliance with the California Geological Survey (CGS) Special Publication 117A (2008); Southern California Earthquake Center "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California," 1999 report; and the Seismic Hazard Zone Report for the Hayward 7.5-Minute Quadrangle, Alameda County, California (SHZR 091).

We performed a screening analysis to determine if there is a potential for liquefaction to occur at the site. We evaluated the soils encountered in our cone penetration test (CPT) which was advanced to a maximum depth of approximately 100 feet bgs. We evaluated these soils based on soil classification, similar water contents and Atterberg limits of soil samples collected in borings advanced near the site, groundwater elevation, peak ground acceleration, and CPT data. In our screening investigation, we looked at the Atterberg limits for clay soils collected from 22 to 50 feet in soil borings advanced in the vicinity of the site. The Atterberg limits for clay soils in the area exhibited liquid limits ranging from about 25 to 35 and plasticity indices varying from about 9 to 25. We also calculated the ratio of in-situ moisture contents to the liquid limits. This data was then compared to the criteria by Idriss and Boulanger (2006) and Bray and Sancio (2006) for potential liquefaction or cyclic softening of fine-grained soils. Based on the range of Atterberg

[1] California Department of Conservation Division of Mines and Geology (CDMG), *"Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Southern Region"*, CDMG Compact Disc 2000-003, 2000.

limits reviewed from nearby sites, the soils at the site likely classify as “clay-like” and non-liquefiable by Idriss/Boulanger. Due to the range of plasticity indexes of the clays and the ratios of the in-situ moisture contents to the liquid limits being 75% or less, the clay soils at the site would not be prone to cyclic softening according to Bray/Sancio. The data obtained from CPT1 indicates the cohesive soils at the site are over-consolidated. Based on our screening analysis, we believe the cohesive soils at the site will likely behave more like clay and have a low potential for cyclic softening/liquefaction.

We performed a quantitative evaluation of the potential for liquefaction to occur and the effects if liquefaction were to occur on this project. In performing our evaluation, we utilized data for the site obtained from the United States Geological Survey (USGS) website to determine the Peak Ground Acceleration (PGA_M) based on the ASCE 7-16 standard and the Peak Ground Acceleration (PGA) for the probabilistic exceedance of 2% in 50 years for the Maximum Considered Earthquake. The PGA_M was determined to be 1.05g based on the ASCE 7-16 criteria, while the PGA for the 2% exceedance in 50 years was determined to be 1.129g, as given using the USGS Unified Hazard Tool website based on the 2014 dynamic deaggregations. A mean magnitude of 7.05 for the project site was used. Based on groundwater level data we encountered in our CPT, and the historic high groundwater level provided in Seismic Hazard Zone Report 091 for the Hayward Quadrangle, a groundwater depth of 22 feet bgs was utilized in our analyses.

The liquefaction analysis was performed in general accordance with California Geologic Survey Special Publication 117. The liquefaction study utilized the software “CLiq” by GeoLogismiki Geotechnical Software. This analysis was based on the soil data obtained from the CPT sounding supplemented by laboratory data obtained from logs of borings advanced near the site. The analysis was performed on data obtained from CPT1. CPT calculations were assessed using the Robertson (NCEER 2001), Idriss & Boulanger (2008), and Boulanger & Idriss (2014) methods. A factor of safety of 1.3 was used against liquefaction. The liquefaction potential analyses were calculated from a depth of 22 to 50 feet bgs.

Based on our review of the calculations by the various methods, the anticipated potential total liquefaction-induced settlement at the proposed billboard location is about 1 to 1½ inches. Liquefaction is generally occurring between the depths of 32 to 42 feet bgs. Due to the presence of liquefiable soil, if the drilled pier is required to be deeper than 30 feet bgs the pier should extend through all potentially liquefiable soil to a depth of at least 47 feet and the pier design should incorporate a down drag load due to settlement of the liquefiable soil.

Due to the cohesive nature and thickness of non-liquefiable soils across the surface of the site as well as the lithology consisting predominantly of clayey soils, we believe the probability for liquefaction to manifest at the surface is low to moderate. Our review of the Seismic Hazard Zone Report 091 for the Hayward Quadrangle reported no historic evidence of ground failure due to liquefaction within at least 3/4 miles of the site.

With regards to the potential for lateral spreading, we note that the site and surrounding area is relatively level. Given the relative flatness of the local topography and distance to any open faces, it is our opinion that the potential for lateral spreading to affect this site is low.

DEEP FOUNDATIONS

We understand the proposed billboard is planned to be supported by a 5-foot diameter drilled pier. The following design parameters are applicable for the design of the billboard drilled pier foundation. The recommended soil parameters provided are based on our review of the CPT log and engineering properties have been estimated for the soil conditions presented in this report.

Drilled Pier Axial Design Parameters

Soil design parameters are provided below in the **Drilled Pier Design Summary** table for the axial design of the drilled pier foundation. The values presented for allowable side friction include a factor of safety of 2.5.

Drilled Pier Design Summary [1]		
Approximate Depth (feet)	Stratigraphy [2] Material	Allowable Skin Friction (psf) [3]
5[5] to 10	Silty Sand/Sand	120[4]
10 to 21	Clay	725[4]
21 to 25	Clay	325[4]
25 to 32	Clay	1,700[4]
32 to 42	Silty Sand/Sand	0
42 to 50	Clay	1,000
50 to 95	Clay	1,600

1. Design capacities are dependent upon the method of installation, and quality control parameters. The values provided are estimates and should be verified when installation protocol have been finalized.
2. See **Geotechnical Characterization** for more details on stratigraphy.
3. Applicable for compressive loading only. Reduce to 2/3 of values shown for uplift loading. Effective weight of the pier can be added to uplift load capacity.
4. Allowable skin friction values above 32 feet should be neglected if the drilled pier design requires the pier to extend deeper than 30 feet bgs.
5. Axial capacity based on skin friction should be neglected for the upper 5 feet of the pier.

Tensile reinforcement should extend to the bottom of the pier if subjected to uplift loading. Buoyant unit weights of the soil and concrete should be used in the calculations below the highest anticipated groundwater elevation. The drilled pier should be designed as a friction pier that derives its support from the underlying firm native soil. If the drilled pier design requires the pier to extend deeper than 30 feet bgs, the pier should extend through potentially liquefiable soils to a minimum depth of 47 feet bgs. If the pier will be deeper than 30 feet, the pier design should incorporate a down drag load of 475 kips which is based on the pier having the planned diameter of 5 feet. Terracon should be contacted to provide a revised drag load if the drilled pier will not have a diameter of 5 feet. We estimate a pier depth of about 67 feet would be required to accommodate the estimated drag load and planned axial structural loads if the pier is required to extend deeper than 30 feet. End bearing should not be used. Provided the pier is designed per the recommendations provided herein, we anticipate pier settlement to be less than ½ inch due to structural loads and an additional 1½ inches due to liquefaction for piers extending 30 feet bgs or shallower.

The structural capacity of the pier should be checked to assure it can safely accommodate the combined stresses induced by axial and lateral forces. The response of the drilled pier foundation to lateral loads is dependent upon the soil/structure interaction as well as the pier's diameter, length, stiffness and "fixed head" or "free head" condition.

Drilled Pier Lateral Design Parameters

The following table lists input values for use in LPILE analyses. LPILE will estimate values of k_h and ε_{50} based on strength; however, non-default values of k_h should be used where provided. Since deflection or a service limit criterion will most likely control lateral capacity design, no safety/resistance factor is included with the parameters.

Lateral Load Analyses						
Estimated Engineering Properties of Soils						
Top Depth / Bottom Depth	**Effective Unit Weight (pcf)**	**L-PILE/ GROUP Soil Type**	**Internal Angle of Friction (Degrees)**	**Cohesion (psf)**	**Coeff. of Static Subgrade Reaction K_s (pci)**	**ε_{50}**
3[3]	112	Sand	36	__	90	__
10			36	__	90	__
10	112	Stiff Clay without free water	--	1,800	--	0.007
21			--	1,800	--	0.007
21	115	Stiff Clay without free water	--	3,200	--	0.005
25			--	3,200	--	0.005
25	58	Stiff Clay without free water	--	4,400	--	0.005
32			--	4,400	--	0.005

32	63	Liquefied Sand	--	--	--	--
42			--	--	--	--
42	48	Stiff Clay without free water	--	2,400	--	0.006
50			--	2,400	--	0.006
50	55	Stiff Clay without free water	--	4,000	--	0.005
95			--	4,000	--	0.005

1. See **Subsurface Profile** in **Geotechnical Characterization** for details on Stratigraphy.
2. Parameters assume groundwater is located at a depth of 25 feet below the existing ground surface.
3. The upper 3 feet of the drilled pier should be neglected from design.

The parameters provided herein are based on the stresses induced in the supporting soil strata. The structural capacity of the pier should be checked to assure it can safely accommodate the combined stresses induced by axial and lateral forces. The response of the drilled pier foundation to lateral loads is dependent upon the soil/structure interaction as well as the pier's diameter, length, stiffness and "fixed head" or "free head" condition. The load-carrying capacity of pier may be increased by increasing the diameter and/or length.

Drilled Pier Construction Considerations

Groundwater was encountered in the CPT at a depth of 25 feet bgs at the time our field exploration. Groundwater levels and seepage can and will fluctuate. Caving may occur during the drilling and construction of the proposed pier. To prevent collapse of the sidewalls and/or to control groundwater seepage, the use of temporary steel casing and/or slurry drilling procedures may be required for construction of the drilled pier foundation. Significant seepage could occur in case of the excavation penetrating water-bearing sandy soil. The drilled pier contractor and foundation design engineer should be informed of these risks.

The use of temporary steel casing and/or slurry drilling procedures should be anticipated at this site during drilled pier construction to prevent collapse of the sidewalls within sand seams and layers and control groundwater seepage. If casing is removed during concrete placement, care should be exercised to maintain concrete inside the casing at a sufficient level to resist earth and hydrostatic pressures present on a casing exterior. Water or loose soil should be removed from the bottom of the drilled pier prior to placement of the concrete.

Use of a telescoping casing arrangement can be considered to avoid handling long casing lengths. If possible, excess water should be evacuated from the casing to place concrete in the "dry".

Care should be taken to not disturb the sides and bottom of the excavation during construction. The bottom of the pier excavation should be free of loose material before concrete placement. Concrete should be placed as soon as possible after the foundation excavation is completed, to reduce potential disturbance of the bearing surface.

A “wet” pier should be constructed by slurry displacement techniques. In this process, the pier excavation is filled with approved polymer-based slurry to counter-balance the hydraulic forces below the water level and stabilize the wall of the pier. Concrete would then be placed using a tremie extending to within 6 inches of the pier base of the slurry-filled excavation. The tremie remains inserted several feet into the fresh concrete as it displaces the slurry upward and until placement is complete. The slurry should have a sand content no greater than 1 percent at the time concrete placement commences. The maximum unit weight of the slurry should be established in consultation with Terracon.

Concrete for "dry" drilled pier construction should have a slump of about 5 to 7 inches. Concrete should be directed into the pier utilizing a centering chute. Concrete for "wet" pier construction would require higher slump concrete.

While withdrawing casing, care should be exercised to maintain concrete inside the casing at a sufficient level to resist earth and hydrostatic pressures acting on the casing exterior. Arching of the concrete, loss of seal and other problems can occur during casing removal and result in contamination of the drilled pier. These conditions should be considered during the design and construction phases. Placement of loose soil backfill should not be permitted around the casing prior to removal.

The formation of a mushroom or enlargement at the top of the pier should be avoided during pier drilling.

The drilled pier installation process should be performed under the direction of the Geotechnical Engineer. The Geotechnical Engineer should document the pier installation process including soil and groundwater conditions encountered, consistency with expected conditions, and details of the installed pier.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client, and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing. This report should not be used after 3 years without written authorization from Terracon.

ATTACHMENTS

EXPLORATION AND TESTING PROCEDURES

Field Exploration

Number of CPTs	CPT Depth (feet)	Planned Location
1	100	Planned billboard location

CPT Layout and Elevation: The CPT layout was performed by Terracon. Coordinates were obtained with a handheld GPS unit (estimated horizontal accuracy of about ±20 feet) and an approximate elevation was estimated using Google Earth Pro. If a more precise CPT location and elevation are desired, we recommend the CPT be surveyed.

For the cone penetrometer testing, the CPT rig hydraulically pushes an instrumented cone through the soil while nearly continuous readings are recorded to a portable computer. The cone is equipped with electronic load cells to measure tip resistance and sleeve resistance and a pressure transducer to measure the generated ambient pore pressure. The face of the cone has an apex angle of 60° and an area of 15 cm^2. Digital Data representing the tip resistance, friction resistance, pore water pressure, and probe inclination angle are recorded about every 2 centimeters while advancing through the ground at a rate between 1½ and 2½ centimeters per second. These measurements are correlated to various soil properties used for geotechnical design. No soil samples are gathered through this subsurface investigation technique. CPT testing was conducted in general accordance with ASTM D5778 “Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils.”

SITE LOCATION AND EXPLORATION PLANS

Contents:

Note: All attachments are one page unless noted above.

SITE LOCATION

Langton Way Billboard ■ Hayward, Alameda County, California
May 7, 2021 ■ Terracon Project No. ND215017

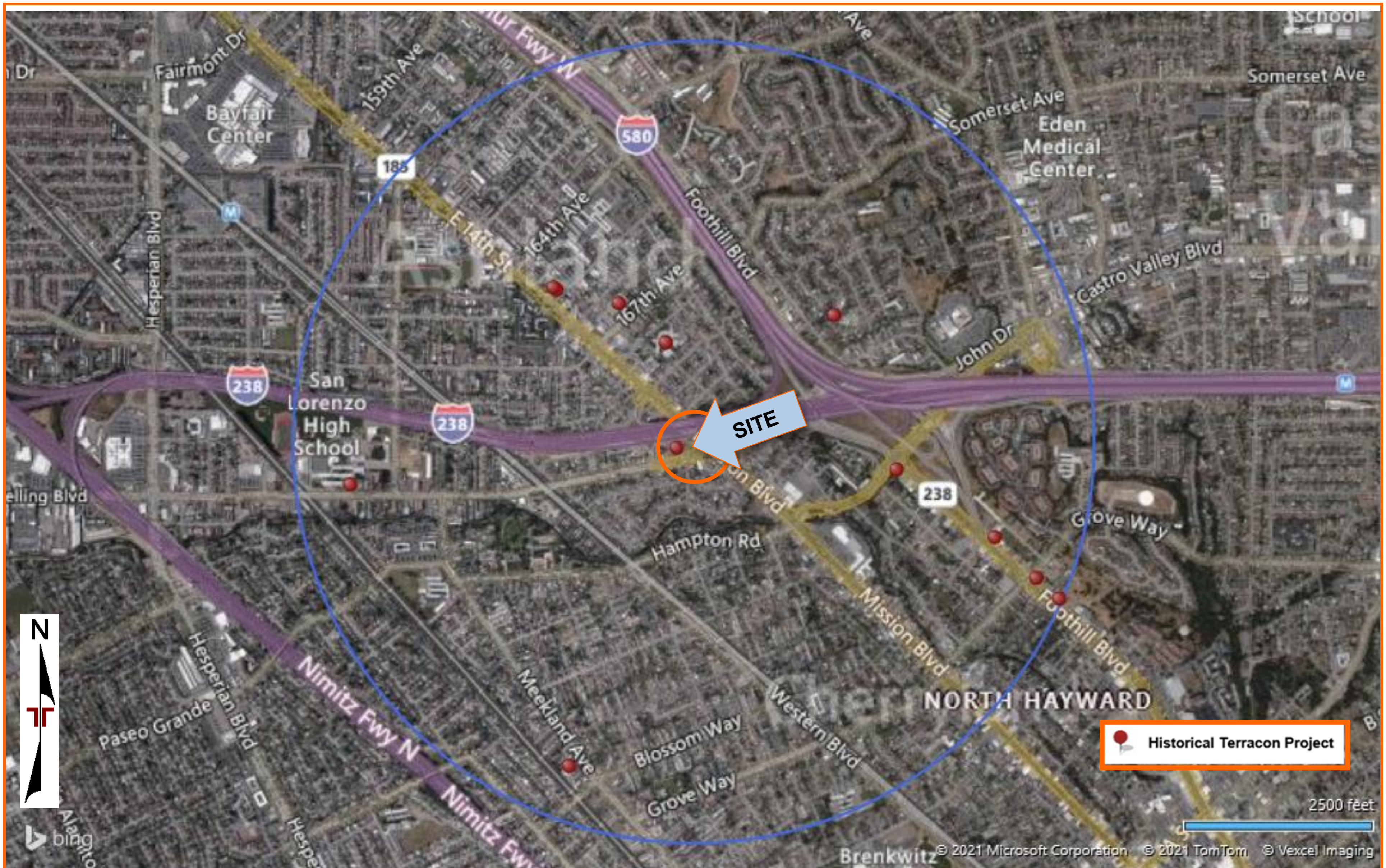


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION PLAN

Langton Way Billboard ■ Hayward, Alameda County, California

May 7, 2021 ■ Terracon Project No. ND215017

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION RESULTS

Contents:

Note: All attachments are one page unless noted above.

CPT LOG NO. CPT-01

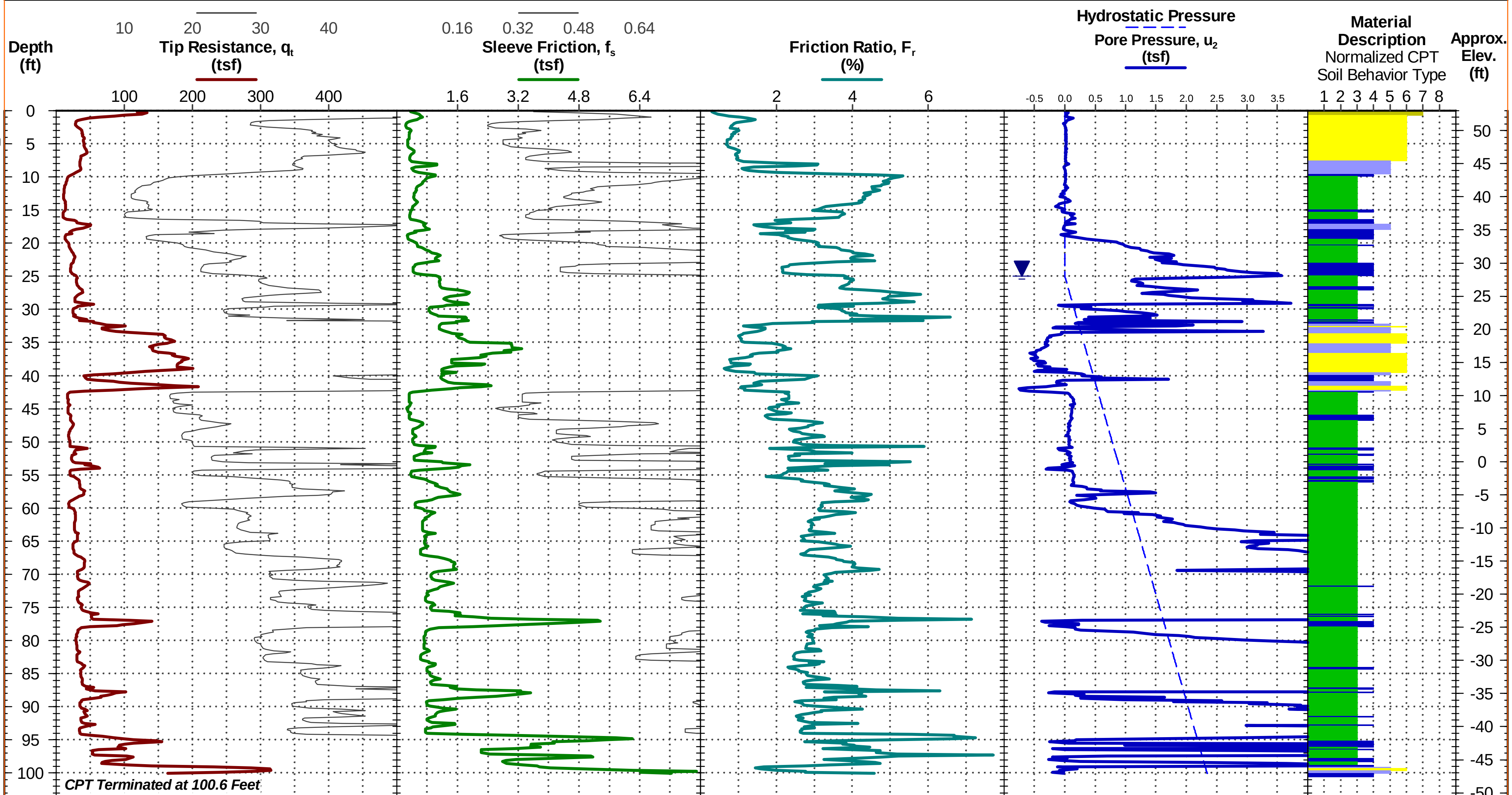
PROJECT: Langton Way Billboard

CLIENT: Outfront Media
Berkeley, CA

TEST LOCATION: See Exploration Plan

SITE: Langton Way
Hayward, CA

Approx. Surface Elev: 53 ft +/-
Latitude: 37.6886284°
Longitude: -122.1054036°



See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
Elevation was obtained from Google Earth Pro.

Dead weight of rig used as reaction force.
CPT sensor calibration reports available upon request.

1 Sensitive, fine grained
2 Organic soils - clay
3 Clay - silty clay to clay
4 Silt mixtures - clayey silt to silty clay
5 Sand mixtures - silty sand to sandy silt
6 Sands - clean sand to silty sand
7 Gravelly sand to dense sand
8 Very stiff sand to clayey sand
9 Very stiff fine grained

WATER LEVEL OBSERVATION	Probe no. DDG1587	Terracon 5075 Commercial Cir Ste E Concord, CA	CPT Started: 4/12/2021	CPT Completed: 4/12/2021
25 ft measured water depth (used in normalizations and correlations; See Supporting Information)			Rig: CPT	Operator: Middle Earth
			Project No.: ND215017	

THIS BORING LOG IS NOT VALID IF SEPARATED FROM ORIGINAL REPORT. CPT REPORT ND215017 LANGTON WAY BILLB.GPJ TERRACON_DATATEMPLATE.GDT 5/7/21

SUPPORTING INFORMATION

Contents:

Note: All attachments are one page unless noted above.

CPT GENERAL NOTES

DESCRIPTION OF MEASUREMENTS AND CALIBRATIONS

To be reported per ASTM D5778:

Uncorrected Tip Resistance, q_c
Measured force acting on the cone divided by the cone's projected area

Corrected Tip Resistance, q_t
Cone resistance corrected for porewater and net area ratio effects
$q_t = q_c + u_2(1 - a)$
Where a is the net area ratio, a lab calibration of the cone typically between 0.70 and 0.85

Pore Pressure, u
Pore pressure measured during penetration
u_1 - sensor on the face of the cone
u_2 - sensor on the shoulder (more common)

Sleeve Friction, f_s
Frictional force acting on the sleeve divided by its surface area

Normalized Friction Ratio, F_r
The ratio as a percentage of f_s to q_t, accounting for overburden pressure

To be reported per ASTM D7400, if collected:

Shear Wave Velocity, V_s
Measured in a Seismic CPT and provides direct measure of soil stiffness

DESCRIPTION OF GEOTECHNICAL CORRELATIONS

Normalized Tip Resistance, Q_{tn}
$Q_{tn} = ((q_t - \sigma_{V0})/P_a)(P_a/\sigma'_{V0})^n$
$n = 0.381(I_c) + 0.05(\sigma'_{V0}/P_a) - 0.15$

Over Consolidation Ratio, OCR
OCR (1) $= 0.25(Q_{tn})^{1.25}$
OCR (2) $= 0.33(Q_{tn})$

Undrained Shear Strength, S_u
$S_u = Q_{tn} \times \sigma'_{V0}/N_{kt}$
N_{kt} is a soil-specific factor (shown on S_u plot)

Sensitivity, S_t
$S_t = (q_t - \sigma_{V0}/N_{kt}) \times (1/f_s)$

Effective Friction Angle, ϕ'
ϕ' (1) $= \tan^{-1}(0.373[\log(q_t/\sigma'_{V0}) + 0.29])$
ϕ' (2) $= 17.6 + 11[\log(Q_{tn})]$

Unit Weight, γ
$\gamma = (0.27[\log(F_r)]+0.36[\log(q_t/atm)]+1.236) \times \gamma_{water}$
σ_{V0} is taken as the incremental sum of the unit weights

Small Strain Shear Modulus, G_0
G_0 (1) $= \rho V_s^2$
G_0 (2) $= 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$

Soil Behavior Type Index, I_c
$I_c = [(3.47 - \log(Q_{tn}))^2 + (\log(F_r) + 1.22)^2]^{0.5}$

SPT N_{60}
$N_{60} = (q_t/atm) / 10^{(1.1268 - 0.2817Ic)}$

Elastic Modulus, E_s (assumes $q/q_{ultimate}$ ~ 0.3, i.e. FS = 3)
E_s (1) $= 2.6\psi G_0$ where $\psi = 0.56 - 0.33\log Q_{tn,clean\ sand}$
E_s (2) $= G_0$
E_s (3) $= 0.015 \times 10^{(0.55Ic + 1.68)}(q_t - \sigma_{V0})$
E_s (4) $= 2.5q_t$

Constrained Modulus, M
$M = \alpha_M(q_t - \sigma_{V0})$
For $I_c > 2.2$ (fine-grained soils)
$\alpha_M = Q_{tn}$ with maximum of 14
For $I_c < 2.2$ (coarse-grained soils)
$\alpha_M = 0.0188 \times 10^{(0.55Ic + 1.68)}$

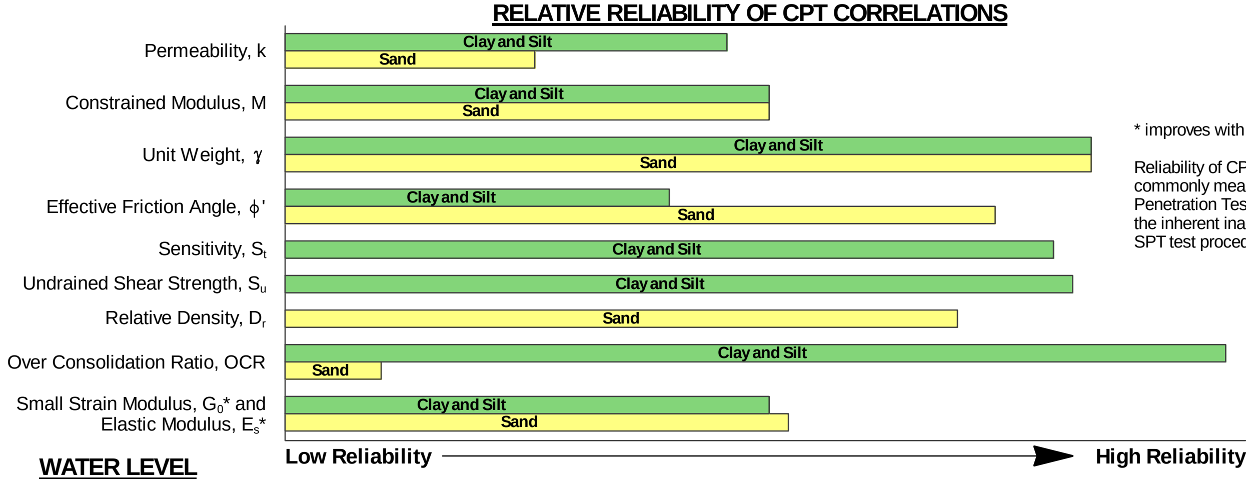
Hydraulic Conductivity, k
For $1.0 < I_c < 3.27$ $k = 10^{(0.952 - 3.04Ic)}$
For $3.27 < I_c < 4.0$ $k = 10^{(-4.52 - 1.37Ic)}$

Relative Density, D_r
$D_r = (Q_{tn} / 350)^{0.5} \times 100$

REPORTED PARAMETERS

CPT logs as provided, at a minimum, report the data as required by ASTM D5778 and ASTM D7400 (if applicable). This minimum data include q_t, f_s, and u. Other correlated parameters may also be provided. These other correlated parameters are interpretations of the measured data based upon published and reliable references, but they do not necessarily represent the actual values that would be derived from direct testing to determine the various parameters. To this end, more than one correlation to a given parameter may be provided. The following chart illustrates estimates of reliability associated with correlated parameters based upon the literature referenced below.

RELATIVE RELIABILITY OF CPT CORRELATIONS



* improves with seismic V_s measurements

Reliability of CPT-predicted N_{60} values as commonly measured by the Standard Penetration Test (SPT) is not provided due to the inherent inaccuracy associated with the SPT test procedure.

WATER LEVEL

The groundwater level at the CPT location is used to normalize the measurements for vertical overburden pressures and as a result influences the normalized soil behavior type classification and correlated soil parameters. The water level may either be "measured" or "estimated:"

Measured - Depth to water directly measured in the field

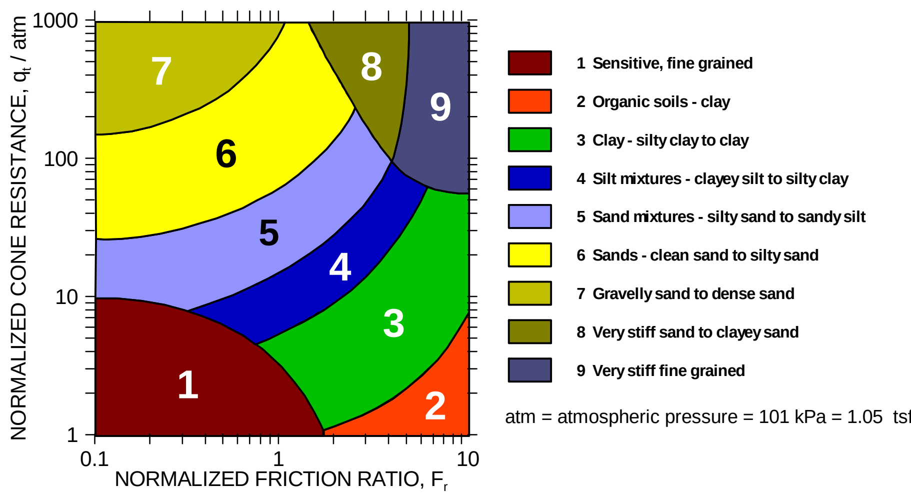
Estimated - Depth to water interpolated by the practitioner using pore pressure measurements in coarse grained soils and known site conditions

While groundwater levels displayed as "measured" more accurately represent site conditions at the time of testing than those "estimated," in either case the groundwater should be further defined prior to construction as groundwater level variations will occur over time.

CONE PENETRATION SOIL BEHAVIOR TYPE

The estimated stratigraphic profiles included in the CPT logs are based on relationships between corrected tip resistance (q_t), friction resistance (f_s), and porewater pressure (u_2). The normalized friction ratio (F_r) is used to classify the soil behavior type.

Typically, silts and clays have high F_r values and generate large excess penetration porewater pressures; sands have lower F_r's and do not generate excess penetration porewater pressures. The adjacent graph (Robertson *et al.*) presents the soil behavior type correlation used for the logs. This normalized SBT chart, generally considered the most reliable, does not use pore pressure to determine SBT due to its lack of repeatability in onshore CPTs.



atm = atmospheric pressure = 101 kPa = 1.05 tsf

REFERENCES

Kulhawy, F.H., Mayne, P.W., (1997). "Manual on Estimating Soil Properties for Foundation Design," Electric Power Research Institute, Palo Alto, CA.

Mayne, P.W., (2013). "Geotechnical Site Exploration in the Year 2013," Georgia Institue of Technology, Atlanta, GA.

Robertson, P.K., Cabal, K.L. (2012). "Guide to Cone Penetration Testing for Geotechnical Engineering," Signal Hill, CA.

Schmertmann, J.H., (1970). "Static Cone to Compute Static Settlement over Sand," *Journal of the Soil Mechanics and Foundations Division*, 96(SM3), 1011-1043.

Terracon

Exhibit: C-1

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Concord, CA 94520
https://www.terracon.com/

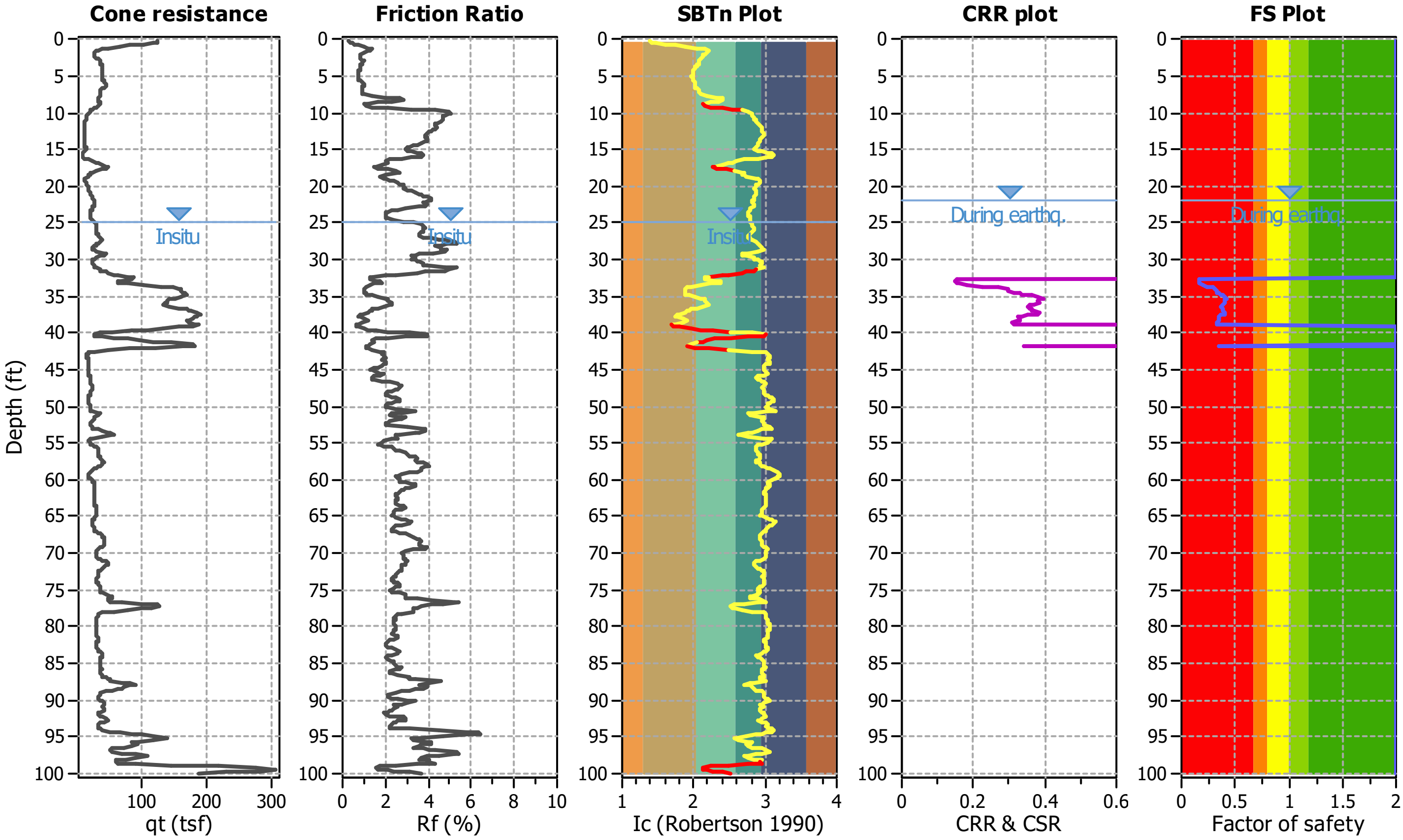
LIQUEFACTION ANALYSIS REPORT

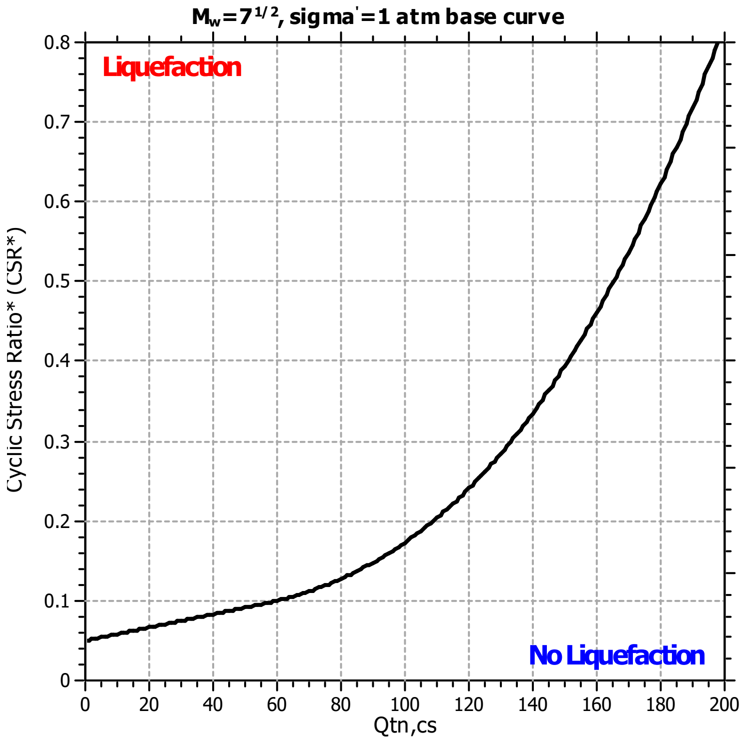
Project title : Langton Way **Location :**

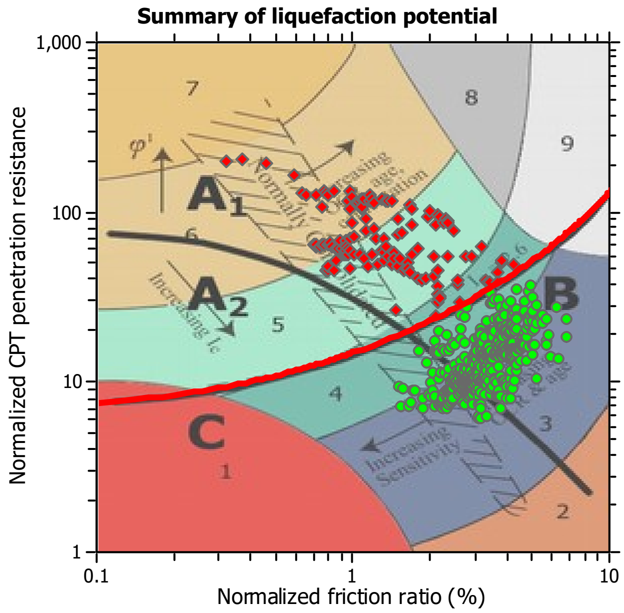
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	25.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	22.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.05	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	1.05	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

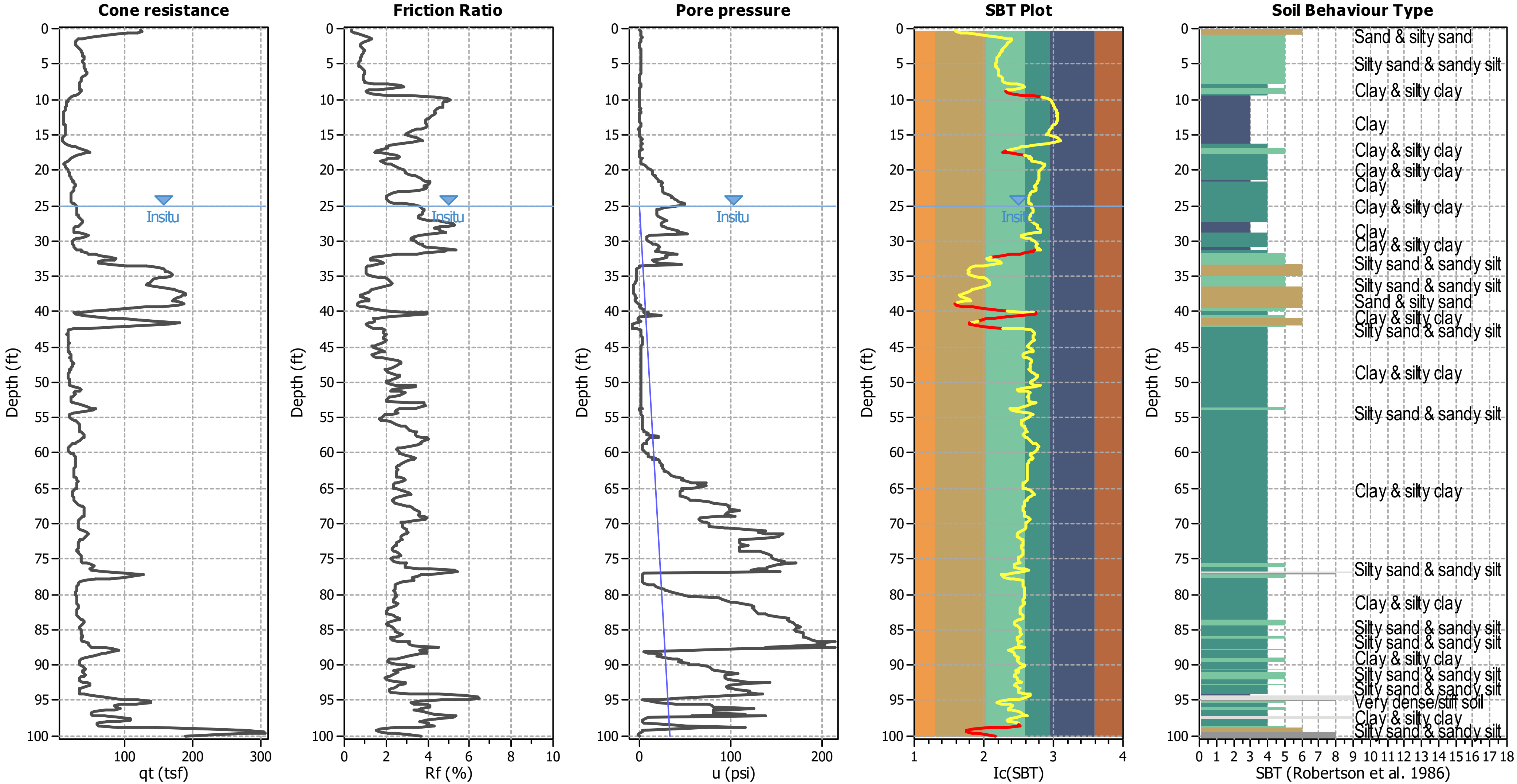






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



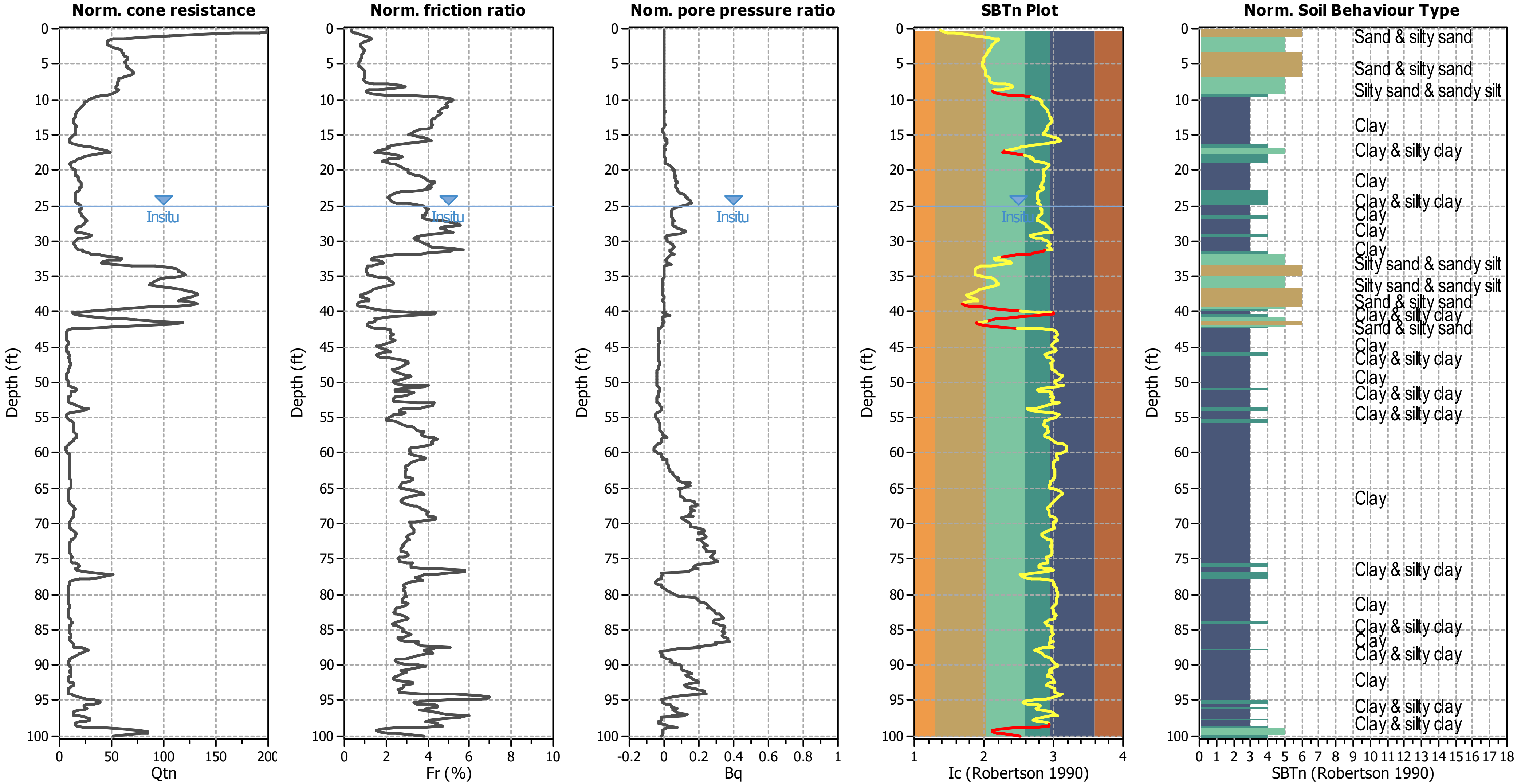
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	22.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.05	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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)



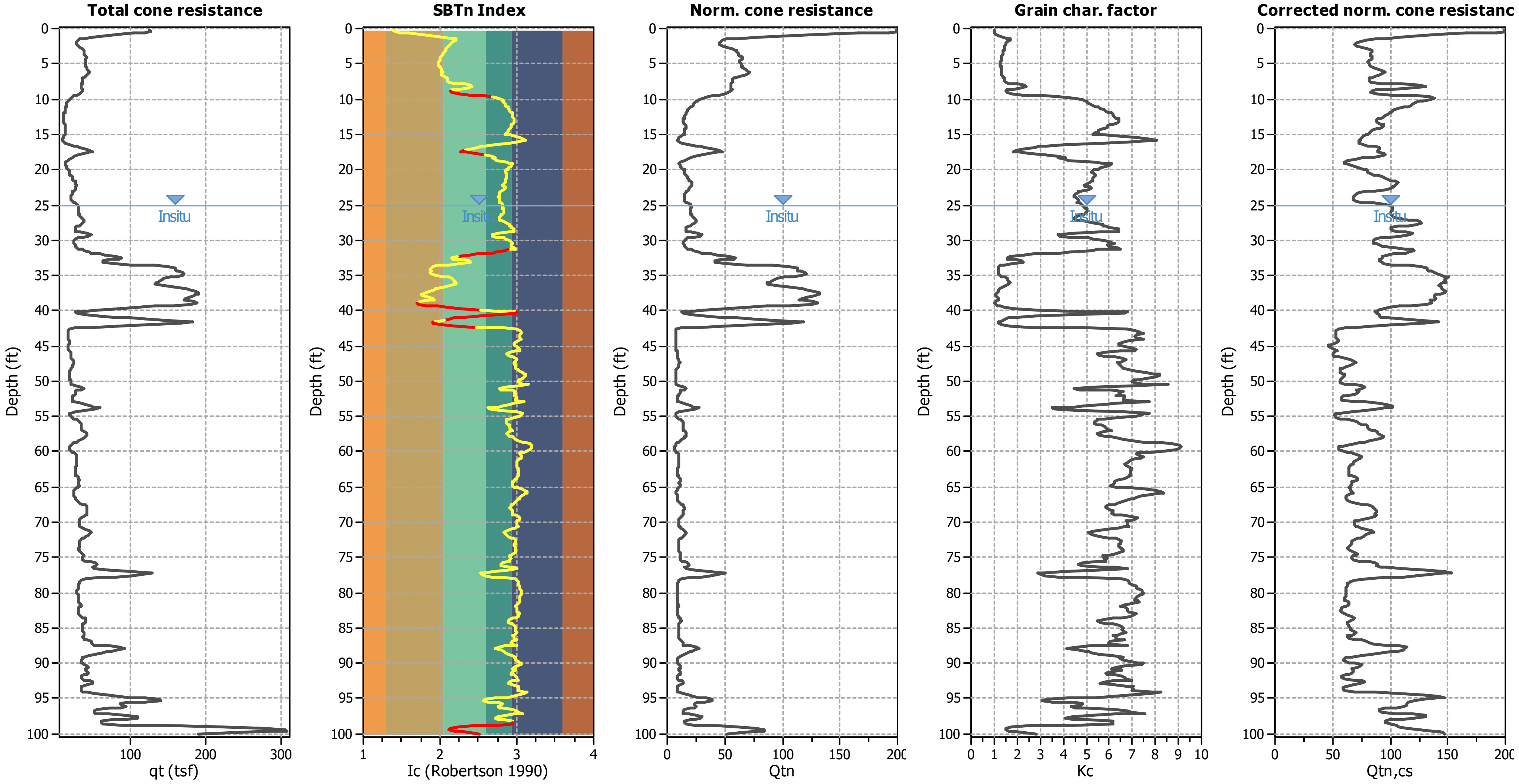
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	22.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.05	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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

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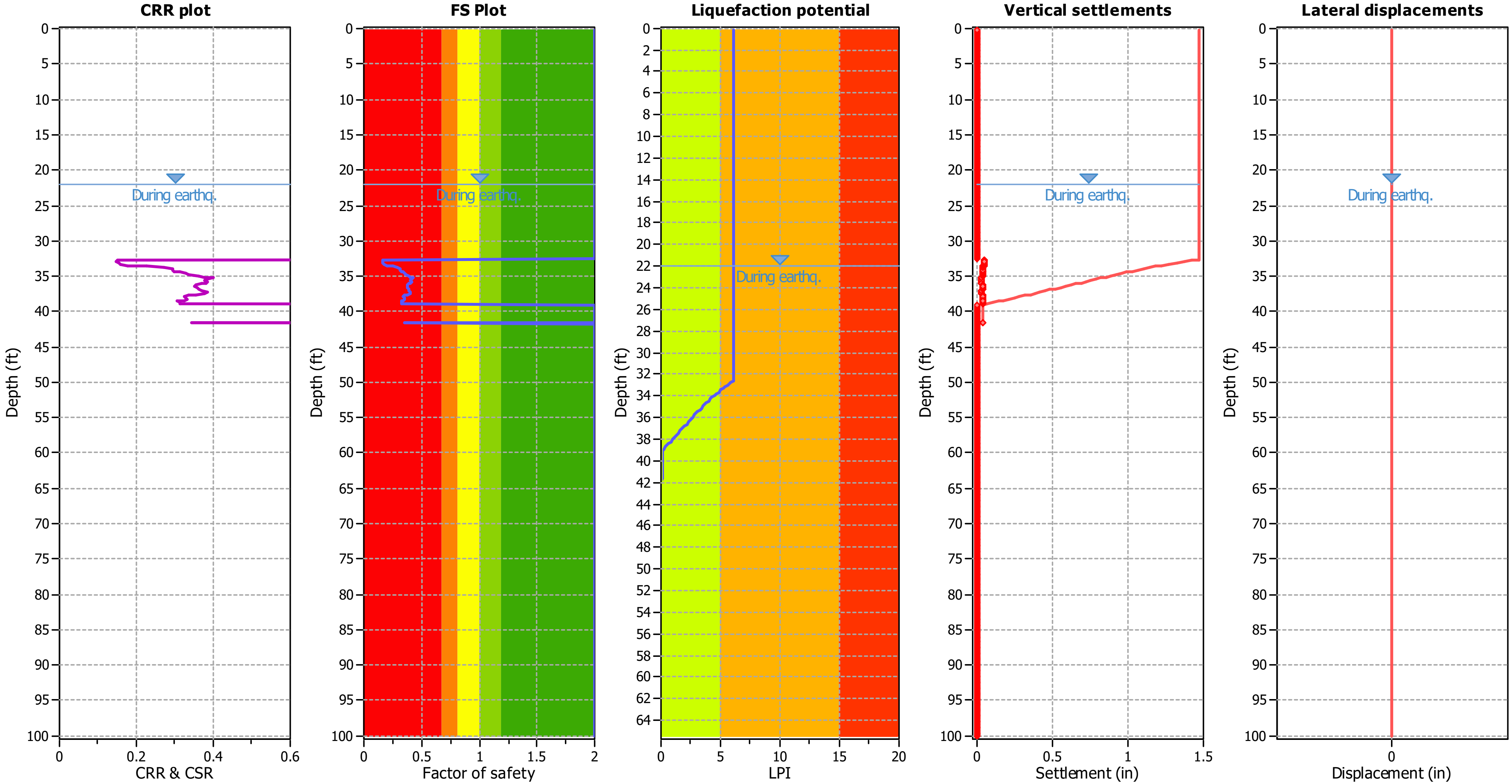
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.05	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	22.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
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Peak ground acceleration:	1.05	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

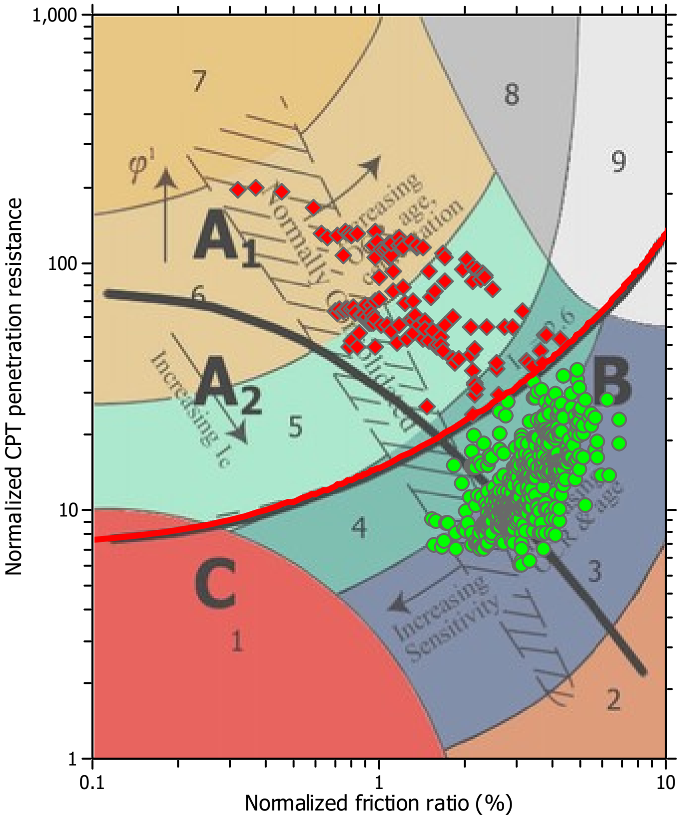
F.S. color scheme

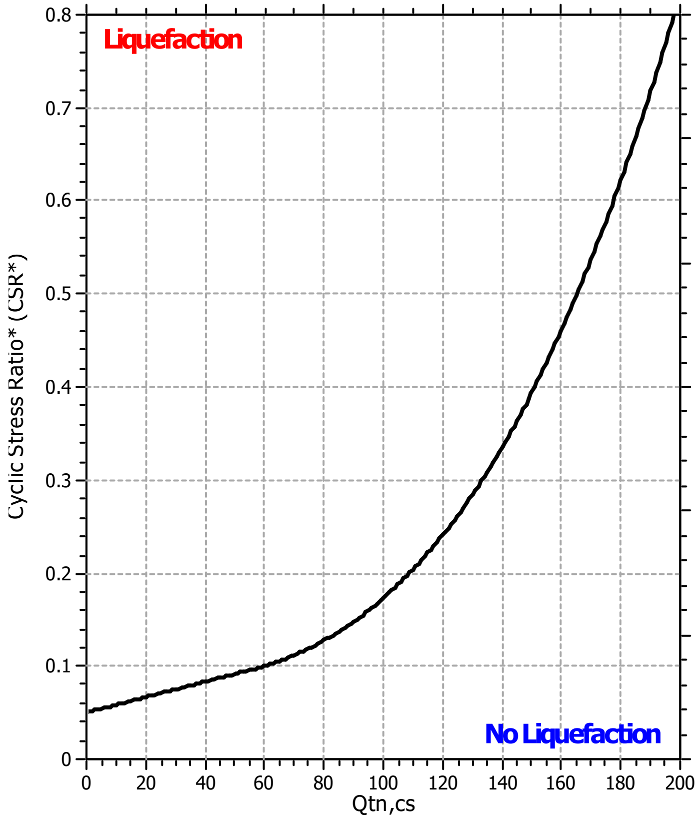
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

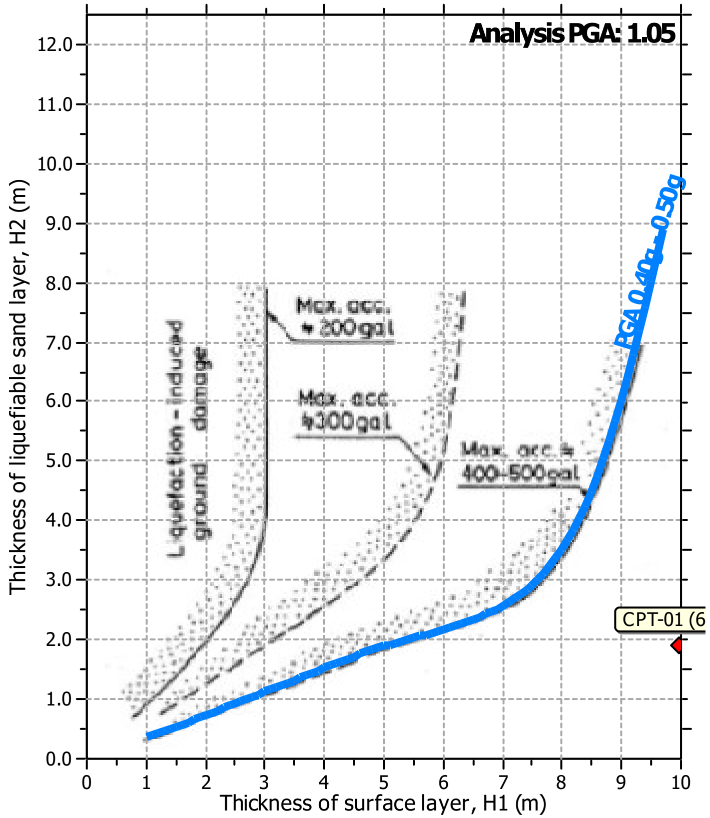
LPI color scheme

- 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.):	22.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
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Earthquake magnitude M_w:	7.05	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	1.05	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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