APPENDIX B

GEOTECHNICAL REPORT

Preliminary Geotechnical Investigation for Feasibility Purposes, 1400 Bristol Street N., Newport Beach, California, 92660.

PN 22029-00
November 4, 2022

KCG KLING Consulting Group, Inc.

November 4, 2022 **PN 22029-00**

Mr. Andrew Strohl
The Picerne Group
5000 Birch Street,
Newport Beach, CA 92660

Subject: **Preliminary Geotechnical Investigation for Feasibility Purposes, 1400 Bristol Street N., Newport Beach, California, 92660**

Dear Mr. Strohl:

At your request and authorization, Kling Consulting Group, Inc. (KCG) has performed a preliminary feasibility level geotechnical investigation for a proposed multi-level apartment complex located in Newport Beach, California (**see Figure 1 - Site Location Map**). The purpose of our evaluation has been to review site geologic/geotechnical conditions and assess potential constraints affecting development of the site. Subsurface field exploration consisting of three Cone Penetrometer (CPT) soundings, was completed to characterize the subsurface soils and determine selected engineering properties to develop preliminary geotechnical conclusions and recommendations for feasibility purposes. We expect our findings and opinions will assist in your decision-making process to develop the property and aid in development of preliminary costs and budgets for the project.

We appreciate this opportunity to be of continued service and to work with you on this project. Should you have any questions regarding this report, please do not hesitate to call.

Respectfully,

KLING CONSULTING GROUP

John Holder

John C. Holder
Staff Engineer

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Dist.: Pdf via email

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of our limited geotechnical investigation was to evaluate near-surface soil conditions to provide preliminary feasibility level geotechnical design recommendations for a proposed multi-level apartment building complex. The scope of work undertaken included the following tasks:

- Compilation and interpretation of available, previously documented geologic and geotechnical data for the property;

- Coordination with Underground Service Alert to mark and identify buried utilities;

- Subsurface exploration, including three (3) Cone-Penetrometer Soundings (CPTs) up to to maximum depths of approximately 50 feet. Continuous logs of the subsurface conditions, as encountered in the soundings, were recorded and are presented in Appendix B. The locations of the soundings are shown in **Figure 2 - Exploration Location Map**;

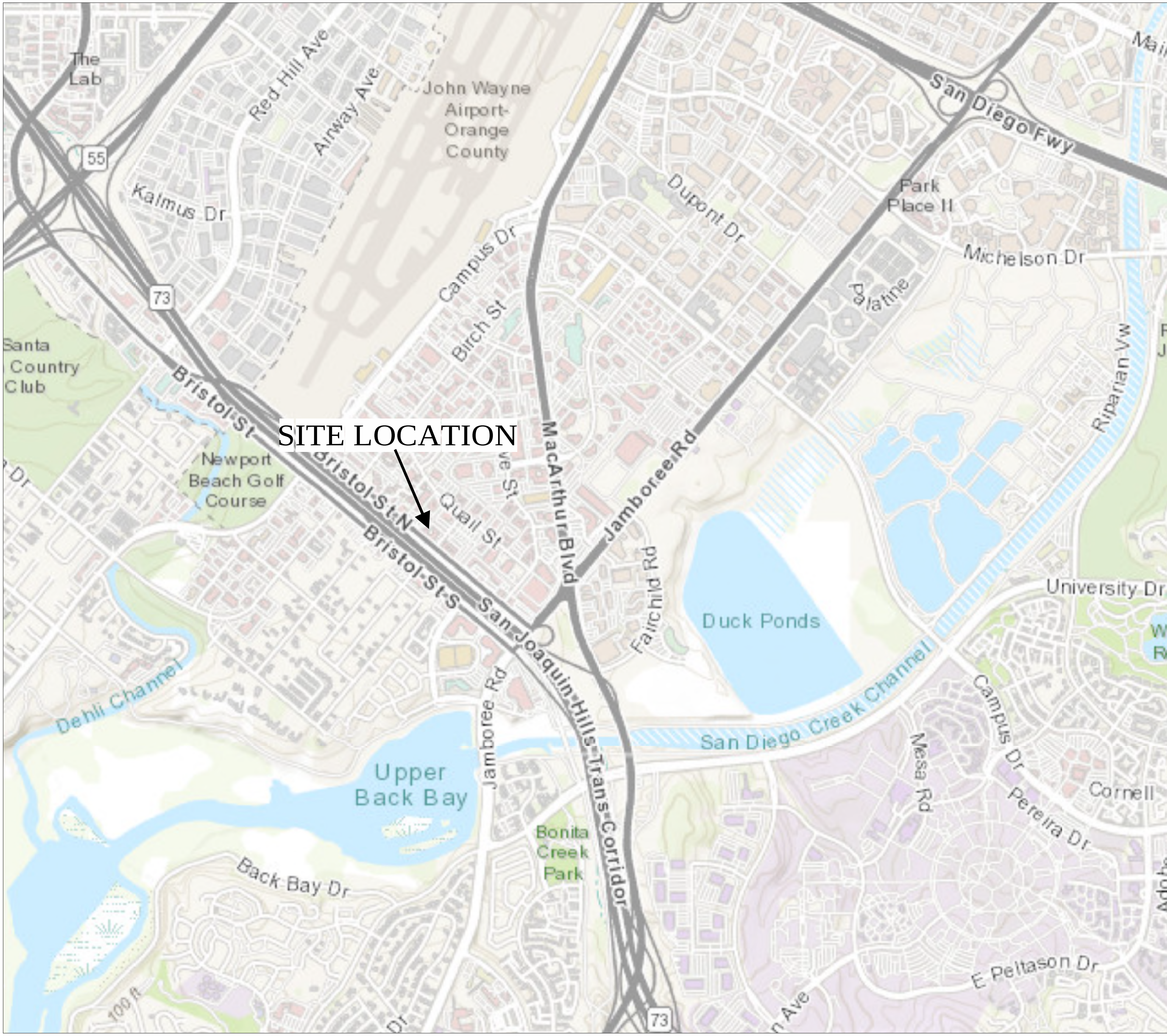
- Geotechnical engineering analysis and preliminary estimate of liquefaction settlement; and

- Preparation of this report along with accompanying maps and illustrations. This report presents our findings, conclusions, and feasibility level recommendations.

1.2 SITE AND PROJECT DESCRIPTION

The subject site is located along the north side of Bristol Street, north of the San Joaquin Hills Transportation Corridor, Highway 73, in Newport Beach, California. The subject site is identified as APN 427-332-02, and is addressed as 1400 Bristol Street N. The square-shaped site encompasses approximately 2.36-acres, and is currently occupied by two existing commercial office buildings along with paved drive and parking areas. The site is bordered by Bristol Street North to the south and west, Spruce Avenue to the east and existing commercial/retail buildings and paved parking to the north. The approximate location of the site is illustrated in **Figure 1 - Site Location Map**.

Through discussions with the client and a review of conceptual yield study plans provided by TCA Architects, dated September 16, 2022, it is understood the proposed development preliminarily consists of a multi-level podium III style building entailing multiple levels of studio, one-bedroom and two-bedroom apartments with both ground floor parking and two levels of subterranean parking. Access will be provided via a driveway that extends along the northern and eastern property boundaries.



Notes:

Scale: 1”: 1850’

0 925 ft 1850 ft

KCG KLING Consulting Group, Inc.

Client: The Picerne Group

Address: 1400 Bristol Street N., Newport Beach, CA 92660

Site Location Map

Drawn: J.H.	**Date:** 11/4/2022
P/N: 22029-00	**Figure:** 1

Specific grading plans are not available; however, grading is anticipated to include cut excavations of at least 24-feet below existing grades to achieve the proposed grades for subterranean parking.

2.0 GEOLOGIC CONDITIONS

2.1 Subsurface Investigation and Sampling

On October 10, 2022, three CPT soundings were advanced using a Cone Penetration Testing drill rig. The CPT soundings were completed to depths of 50 feet below the existing ground surface in the vicinity of the proposed development area. Records of the CPT soundings are included in Appendix B. The approximate location of the soundings is illustrated in **Figure 2 - Exploration Location Map**.

For this preliminary field exploration, no ring and bulk samples were obtained for laboratory testing.

2.2 Regional and Site Specific Geologic Setting

The subject site is located in the Peninsular Ranges Geomorphic Province, at the south-eastern edge of the Los Angeles Basin and within the nearly flat-lying area of the Tustin Plain. The site is primarily underlain by elevated Pleistocene and late Pliocene marine terrace deposits established by progressive and (or) episodic tectonic uplift of coastal southern California.

The National Geologic Map Database maps the site as being underlain by late to middle Pleistocene Old Paralic Deposits. The Old Paralic Deposits comprise a poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine, and colluvial deposits composed of silt, sand, and cobbles. These deposits rest on now emergent wave-cut abrasion platforms preserved by regional uplift.

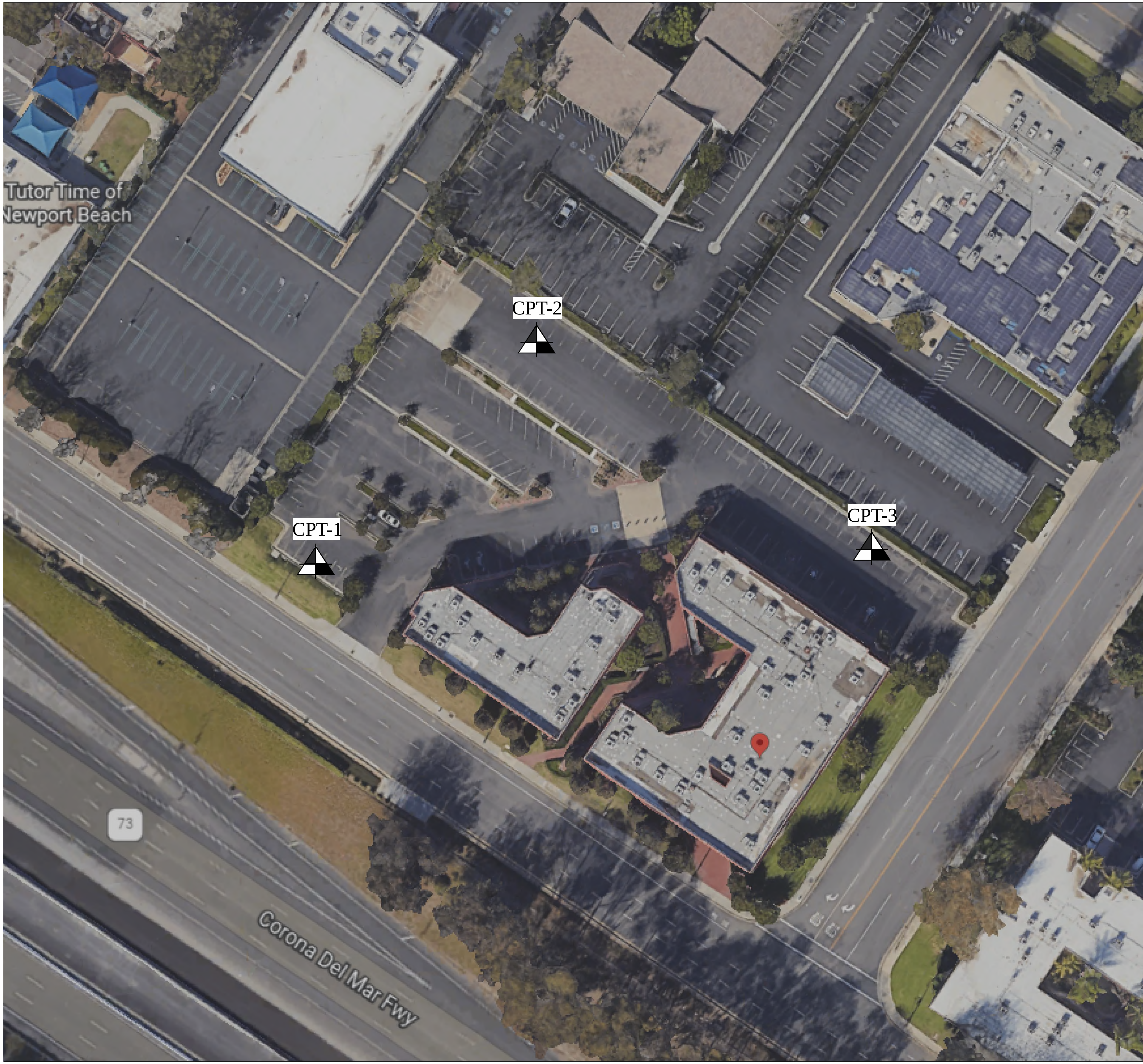
2.3 Subsurface Conditions

2.3.1 Asphalt

The site is mantled by a relatively thin veneer of asphalt to a depth of approximately 3.5-inches from the existing ground in the vicinity of the soundings.

2.3.2 Old Paralic Deposits ($Qopf_a$)

The site is underlain by sands, clay, and sandy-silt associated with the Old Paralic Deposits of Late to Middle Pleistocene age to an observed depth of up to 50.0 feet below the current ground level in the vicinity of the CPT soundings. The Old Paralic Deposits typically consist of light brown to dark gray, sandy clays and sandy to clayey silts, that are medium stiff to hard and moist to wet, with silty sands that are medium dense to very dense. Records of the CPT soundings are presented in Appendix B.



Notes:

- Cone Penetration Testing Location

Scale: 1'': 80'

0 40 ft 80 ft

KCG KLING Consulting Group, Inc.

Client:

The Picerne Group

Address:

1400 Bristol Street North,
Newport Beach, CA 92660

Exploration Location Map

Drawn: J.H.	**Date:** 11/4/2022
P/N: 22029-00	**Figure:** 2

2.4 Groundwater

Groundwater was encountered within all CPT soundings based on pore water dissipation readings at depths of approximately 40 feet below the existing ground surface. We anticipate that the groundwater levels would not significantly impact one level of subterranean parking founded at a depth of 12 feet below exsiting grades or two subterranean parking levels founded at a depth of 24 feet below existing grades. However, a previous Geotechncial Investigation (Reference 11) performed by KCG to the south of the subject site at 1300 N Bristol Street encountered perched groundwater levels at approximately 26 to 35 feet below the existing ground surface. These levels if present could have an impact on two subterrnanean parking levels founded at a depth of 24 feet.

It should be noted that groundwater variation may result from fluctuations in the ground surface topography, subsurface stratification, rainfall, irrigation and other factors that may not be evident at the time of our subsurface exploration. The depth to groundwater within the vicinity of the site should be confirmed as part of a design-level exploration of the site. Until the regional ground water table can be clearly established, the groundwater discussed above should be considered in construction planning and final design.

The nearest groundwater observation well, monitored by the California Department of Water Resources, is located west of the intersection of Irvine Ave and Bristol Street at a ground surface elevation of 28 feet above sea level. The highest recorded groundwater level was recorded at approximately 50 feet below the ground surface in February 1990 (Reference 4). The subject site is approximately 0.5 miles southeast from this observation well. According to the California Geologic Survey (CGS), Seismic Hazard Zone Maps and Report for the Tustin 7.5-Minute Quadrangle (References 6 and 7), the reported and mapped historical high groundwater level is approximately 10 feet below the current ground level in the vicinity of the site.

3.0 GEOTECHNICAL ENGINEERING CONSIDERATIONS

3.1 Expansive Soil Characteristics

We anticipate that subsurface soils will consist of interbedded sand, silt, and clay. While sandy soils are generally not susceptible to expansion, the potential exists that layers of expansive clay could be present at the foundation elevation. These layers should not be left in place or used as fill if any clay beds are encountered. Laboratory testing to evaluate expansion potential would be recommended as part of a design-level exploration of the site. Until future testing is performed, the soil should be considered as having moderate potential for expansion.

3.2 Sulfate Content

Sulfate testing was not performed as part of this investigation. Laboratory testing to evaluate sulfate content would be recommended as part of a design-level exploration of the site. Preliminarily, the soils can be considered "S0" sulfate per ACI-318 (Reference 2).

3.3 Faulting and Seismicity

The subject site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Zones, Jennings and Bryant, 2010; Hart and Bryant, 1997). The property is not located where a site-specific investigation to determine the locations of any active faults would be required. However, the Southern California region is seismically active. Active and potentially active faults within Southern California are capable of producing seismic shaking at the site. It is anticipated that the site will periodically experience ground acceleration due to exposure to moderate to large magnitude earthquakes occurring on distant faults.

However, no active faults are known to exist at the site, and the risk of surface fault rupture is considered low. The closest active fault zones to the subject site is the San Joaquin Hills fault located approximately 2.2 miles from the site and the Newport-Inglewood-Rose Canyon Fault Zone, located approximately 4.8 miles from the subject site.

3.4 Seismic Design Parameters

Presented below are the site seismic parameters utilizing generic geologic, seismic, and geotechnical data gathered for the site using the SEAC/OSHPD web based tool (Reference 12). All structures should be designed for earthquake-induced strong ground motions in accordance with the 2019 CBC procedures utilizing the following parameters:

Seismic Design Parameters

Site Class (Soil Profile)	**D**
Latitude	**33.659638**
Longitude	**-117.869390**
Short Period Spectral Acceleration, Ss:	**1.298**
1-Second Period Spectral Acceleration, S1:	**0.463**
Site Coefficient, Fa:	**1.0**
Site Coefficient, Fv:	**1.837**
Maximum Considered Earthquake Spectral Response Acceleration, SMS:	**1.298**
Maximum Considered Earthquake Spectral Response Acceleration, SM1:	**0.851**
Design Spectral Response Acceleration, SDS:	**0.865**
Design Spectral Response Acceleration, SD1:	**0.567**
Site modified peak ground	**0.612**

acceleration PGA_M	
Seismic Design Category	**D**

Note: A site specific ground motion analysis was not included in the scope of this investigation. Per ASCE 7-16, 11.4.8, structures on Site Class D with S_1 greater than or equal to 0.2 may require Site Specific Ground Motion Analysis. However, a site specific ground motion analysis may not be required based on exceptions listed in ASCE 7-16, 11.4.8. The project structural engineer should verify whether exceptions are valid for this site and if a Site Specific Ground Motion Analysis is required.

3.5 Seismic Hazards

3.5.1 Liquefaction Potential

Liquefaction occurs when ground water pressure in loose sandy soil becomes greater than overburden pressure due to seismic-induced cyclic shear stresses from earth quakes. The result is a near complete loss of soil shear strength and ground settlement. The California Geological Survey (CGS), Seismic Hazard Zone Map for the Tustin Quadrangle (Reference 6) indicates the site is not situated in a liquefaction zone. Our review of the Seismic Hazard Zone Report for the Tustin Quadrangle (Reference 7), indicates the historic groundwater is reported to be approximately 10 feet from existing grades in the vicinity of the property. Our liquefaction analysis conservatively incorporates the historic high groundwater depth of 10 feet. Our geotechnical evaluation indicated that localized and isolated sandy layers within the Old Paralic Deposits that underlie the site are susceptible to relatively minor amounts of liquefaction due to a design-level earthquake along a nearby fault. Overall seismic induced liquefaction settlement would be reduced with the removal of materials for the subterranean excavations. The portions of the site that appear to be susceptible to liquefaction and the magnitudes of seismic-induced settlement described above appear to be somewhat localized. The state of California has not established a seismic hazard zone for the area.

3.5.2 Liquefaction Settlement Analysis

The total earthquake-induced liquefaction settlement potential was calculated using the software program "CLiq v.1.7" by GeoLogismiki (Reference 9). Our evaluation was based on the site class and adjusted peak ground acceleration of 0.612g, as presented in the Seismic Design Parameters Table above, and a probabilistic 2,475-year modal magnitude of 6.89. Our analysis indicated the estimated settlement due to earthquake-induced liquefaction settlement is essentially negligible, approximately 0 to 0.35 inches. These settlement values are considered preliminary, and further geotechnical investigation would be required to provide refinement of the estimated differential settlement of the site. The results of our analysis are included herein in **Appendix C - Seismic Settlement Analysis.**

The liquefaction analysis was performed utilizing a groundwater level case presented below:

- 10-foot groundwater table based on the historic highest groundwater table as presented in *The Seismic Hazard Zone Report for the Tustin 7.5-Minute Quadrangle, Orange County, California* (Appendix A).

In addition, the analysis included the following parameters and assumptions:

- Factor of Safety = 1.0
- “Dry” seismic settlements calculated
- Soil Behavior Type Index (Ic) = 2.60[18].
- Weighting factor for volumetric strain applied[11].
- Cn limit value applied.

3.5.3 Sesimically-Induced Settlement

The liquefaction analyses results for seismically induced vertical ground settlement is presented below:

CPT	Vertical Settlements (Inches)	Liquefaction Potential Index (LPI)
1	0.35	4.596 (low risk)
2	0.26	4.473 (low risk)
3	0.31	5.004 (high risk)

The overall vertical settlement calculations include seismically induced “dry” settlements.

Based on our analysis, the seismic induced settlements range from approximately 0.26 inches to 0.35 inches. It should be noted the majority of the vertical ground settlement occurs in the upper 20 feet of the soil column. Vertical ground settlements between 24 and 50 feet are less than 0.2 inches. Additionally, seismically induced differential settlement is variable across the site, with a worst case differential of 0.09-inches over a horizontal distance of 150 feet.

3.5.4 Lateral Spreading

Lateral spreading, a phenomenon associated with seismically induced soil liquefaction, is the lateral displacement of soils due to inertial motion and lack of lateral support during or post liquefaction. Lateral spreading generally occurs on gently sloping ground or level ground with nearby free surface faces such as a drainage or stream channel. No open channels or free face surfaces are known to be located in close proximity to the site. According to studies undertaken by Zhang et al. (2004), Cubrinovski (2012), lateral displacements occur between 300 and 1000 feet from a "free face". As such, the potential for lateral spreading would be unlikely to occur within the project site.

4.0 CONCLUSIONS

The following conclusions are preliminary and based upon our analysis and data review obtained during our limited subsurface field investigation. It is our opinion that the proposed development concept is considered geotechnically feasible provided the recommendations presented herein are implemented during design and construction. Recommendations presented herein are subject to revision and refinement upon completion of the full geotechnical investigation.

- Based upon our review of the site and the proposed development plans, the underlying soils on-site are considered to have sufficient bearing capacity to support the proposed development, provided the recommendations herein are implemented.

- Our geotechnical evaluation indicates that the Old Paralic Deposits that underlie the site are not susceptible to siginificant liquefaction settlement due to a design-level earthquake incorporating a historical high groundwater level of 10 feet below existing grades (CGS/CDMG, 1998). The estimated settlements are in the range of 0.0 inches to 0.35 inches at the site during seismic events. Overall seismic induced liquefaction settlement would be reduced with the removal of the upper materials for the subterranean excavation, as summarized in Section 3.5. The liquefaction assessment is considered preliminary, and further study is required to refine the estimates and determine likely differential settlement. Lateral spreading is considered unlikely due to the lack of "free face" in the vicinity of the subject site.

- No active fault is known to exist at the site, and the risk of surface fault rupture is considered to be low. However, the project site lies within a region of historical seismicity and will likely be subject to seismic shaking in the future.

- KCG's professional opinion is that liquefaction-induced ground displacements are essentially negligible. As part of the supplemental investigation differential liquefaction settlement can be quantified and if needed, incorporated into the structural analysis. Should the final analysis determine differential settlement substantial enough to require mitigation, added stiffness from a mat foundation system or grade beams for spread footings, or similar could be considered.

- Soils underlying the subject site are not considered to be susceptible to hydrocollapse;

- Groundwater condition was encountered in our CPT soundings at depths of approximately 40 feet below the existing ground surface based on pore water dissipation. Although groundwater levels would not be expected to impact and pose a problem for for the proposed site construction of one subterranean level at or near a depth of 12 feet below exsiting grades, groundwater could potentially be an issue for two subterranean levels at or near a depth of 24 feet below existing grades based on our previous investigation completed south of the subject site at 1300 N Bristol Street. Further investigation that included piezometers could better define if the water near a depth of 24 feet is perched, or is connected to regional groundwater. For preliminary planning, temporary dewatering or other measures should be considered possible.

- Preliminarily, the soils underlying the site should be considered to have moderate expansion potential.

- The proposed development should not adversely affect neighboring properties, provided standard of practice excavation shoring methods are employed.

5.0 PRELIMINARY RECOMMENDATIONS

Preliminary recommendations presented below are based on plans obtained from the client and the limited geotechnical information gathered and analyzed to date. Based on our limited subsurface investigation, subsoils at one level deep consist of stiff clay/silt and two subterranean levels are dense sand. These soils should provide suitable soil support for the proposed structures. Foundations can be expected to bear directly on native soil, provided it has not been disturbed or found to be locally soft. Each foundation excavation should be evaluated and if loose disturbed or softened soil is found, it should be removed and replaced as engineered fill or processed in place and recompacted. The extent and depth of processing or recompaction should be as approved by the geotechnical consultant.

5.1 Supplemental Subsurface Exploration

During this limited feasibility level investigation, the subsurface exploration was limited to three sounding locations in readily accessible areas. We recommend that a supplemental geotechnical investigation be performed that includes additional CPT and soil borings (including installation of piezometers). The supplemental investigation should also include additional laboratory testing, foundation and settlement analysis; ground water measurements and to verify subsurface conditions. Recommendations would be updated as warranted.

5.2 Earthwork Specifications

All grading should be performed per the General Earthwork and Grading Specifications presented in Appendix F unless specifically revised or amended below. Grading should also conform to all applicable governing agency requirements. Prior to the commencement of grading operations, all vegetation, organic topsoil and human-made structure should be cleared and disposed of off-site. Any undocumented fill or back-fill encountered should be removed and re-compacted. All areas receiving fill should be scarified to 6 inches and/or over-excavated, moisture conditioned to between optimum moisture and two to four percent above optimum moisture content, and re-compacted to a minimum of 90 percent relative compaction as determined by ASTM D1557. Soil material excavated from the site should be adequate for re-use as compacted fill provided it is free of trash, vegetation and other deleterious material. All earthwork and grading operation should be performed under the observation and testing of the geotechnical consultant of record.

5.3 Remedial Earthwork

5.3.1 Conventional Foundations –One or Two Level Subterranean

For convential speard footings, the foundation excavations should be evaluated for suitability and any disturbed soil or localized softened soil be mitigated with removal and replacement, or processed in place and recompacted, as needed to create adequate support. The geotechnical consultant should perform the evaluation and approve mitigation measures, if needed.

5.3.2 Mat Slab Foundations –One or Two Level Subterranean

For Mat slab foundation systems, the exposed subgrade soil should be evaluated as recommended for spread footings. Any disturbed or locally soft soil encountered should be either removed and replaced with compacted fill, or processing (i.e. 12-inch scarification and recompaction) and proof rolling of the subgrade soils exposed at the subterranean level. Acceptance of exposed soil should be performed by the geotechnical consultant and should also approve any mitigation measures, if needed.

5.3.3 Proposed Pavement and Flatwork Areas

In areas outside of proposed structural areas that would support pavement and flatwork, the exposed sub-grade soils should be processed and re-compacted to a depth of 12-inches. If soils are disturbed during the removal of existing improvements, the disturbed soil should be removed and replaced with compacted fill. After removals are made, exposed soils should be scarified to a depth of 6-inches, brought to near optimum moisture content, and re-compacted.

5.4 Processing of Natural Soils and Fill Placement

Processing of in-place soils exposed after clearing, grubbing, and removal of unsuitable material and before placing fill should include the following items of work:

Scarification of the materials exposed after remedial removals should be accomplished to a depth of at least 6 inches or as dictated by actual soil conditions encountered;

The scarified soils should be brought to 2 to 4 percent above optimum moisture content by watering or drying, as required;

Compaction of the processed soils to at least 90 percent of the laboratory maximum dry density before placing fill.

Fill should be placed in relatively thin (6 to 8-inch) uniform lifts; moisture conditioned to 2 to 4 percent above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM D 1557. Actual lift thickness would depend on soil type and compaction equipment being used.

5.5 Preliminary Recommendations - Proposed Building Foundations

All foundation criteria are considered minimum requirements that may be superseded by more stringent requirements from the architect, structural engineer, or governing agencies; recommended preliminary geotechnical design parameters are being provided for conventional spread footing and reinforced mat slab foundation systems for the residential building with two subterranean parking levels.

5.5.1 Subterranean-Conventional Shallow Foundations

The following preliminary geotechnical design parameters are provided to design proposed conventional foundations for the proposed multi-level apartment building, with two levels of subterranean parking. The proposed foundations for the proposed building may be supported by square pad footings utilizing a maximum allowable bearing pressure of 4000 pounds per square foot with maximum width of 8-feet, and minimum depth of 2-feet below the lowest adjacent grade (including the top of the slab on grade). A coefficient of friction of 0.40 may be used, along with a passive lateral resistance of 250 pounds per square foot per foot of embedment. This bearing value could potentially be increased based on further subsurface exploration and laboratory analysis generated from supplemental investigation.

If normal code requirements are used for seismic design, the allowable bearing value and coefficient of friction may be increased by 1/3 for short duration loads, such as the effect of wind or seismic forces.

If any utility lines are within a 1:1 (horizontal: vertical) projection from the bottom of a footing, they may be within the influence zone of the proposed footing load; if this condition exists, the proposed footing should be deepened so that the utility is outside the zone of influence; the utility line could also be relocated or encased with concrete with concrete slurry. These conditions should be evaluated on a case by case basis.

5.5.2 Subterranean- Mat Slab

A rigid mat foundation may be used to support the structure, provided the recommendations above are implemented. The exposed soil in the excavation should be evaluated and if determined necessary, proof rolled or locally recompacted as needed, in accordance with the recommendations herein. When properly designed and constructed, a structural mat foundation system can be expected to support high structural loads and provide relatively uniform settlement and bridge over local areas of slightly less stiffness or density. Mat foundations should be properly reinforced to form a relatively rigid structural unit in accordance with the structural engineer's design. For designing a mat foundation, we preliminarily recommend a modulus of subgrade reaction of 120 pounds per square inch per inch (pci) with a maximum bearing value of 4000 psf. This value can be further refined as part of the supplemental investigation.

5.6 Settlement

Static settlement of proposed foundations is not expected to exceed one inch for total and one half inch differential over 50 horizontal feet, provided the recommendations presented above for the specific foundation system type is implemented. For preliminary design purposes, seismic induced liquefaction settlement for the apartment site ranges froms 0 to 0.35 inches. This is considered very minor settlement, however it should be refined and verified during the recommended supplemental investigation.

5.7 Slab-On-Grade

These recommendations are provided for planning purposes as the anticipated podium construction would not entail interior slab on grade floors. Additionally, the recommendations are considered minimum requirements that may be superseded by more stringent requirements from the architect, structural engineer, or governing agencies.

Concrete slabs should be at least 5-inches in thickness. Actual slab thickness and reinforcement should be determined by the structural engineer based on structural loads and soil interaction. Our recommendations should be superseded by the recommendations of the structural engineer or architect.

Subgrade soils should be placed wet of the optimum moisture content, and moisture should be maintained until placement of the concrete slab. Additional testing should be performed after precise grading to verify our recommendations.

The slab should be underlain by a minimum two-inch layer of sand, with a sand equivalent of 30 or greater. The sand layer should be underlain by a 15-mil Stego Wrap vapor retarder or equivalent product with a permeance rate of 0.012 perms and a puncture resistance of Class "A" or "B" per ASTM E 1745-97. As per the manufacturer's recommendations, all seams should overlap a minimum of 6 inches and should be sealed in accordance with the specifications provided by the vapor retarder manufacturer. All penetrations should be sealed using a combination of Stego Wrap, Stego Tape and/or Stego Mastic or approved equivalent product. The vapor retarder should be lapped downward a minimum of 12 inches where the vapor retarder encounters an interior footing or exterior thickened edge or footing. The vapor retarder should be placed on top of the sand layer if the sand is expected to become wet before pouring concrete. If the sand can be kept dry before pouring concrete, the vapor retarder should be placed under the sand layer. The water-cement ratio should be a minimum of 0.45 for all concrete within the structure that will contact the on-site soil.

If moisture sensitive floor coverings are utilized, interior concrete slabs should be designed and constructed in accordance with the applicable floor covering manufacturer's specifications.

Slab subgrade soil should be pre-saturated to at least optimum moisture content to a depth of at least 12 inches below the sand layer.

5.7.1 Basement Slab on Grade Floors

Parking garage basement slab in grade floors, other than a mat slab, should be a minimum of 5-inches in thickness and reinforced to resist shrinkage and temperature warping cracking. Actual slab thickness and steel reinforcement should be determined by the structural engineer based on environmental factors and concrete shrinkage considerations. An aggregate base layer may be required depending on the subgrade soils exposed during construction or determined from the supplemental investigation.

5.8 Permanent Subterranean Walls

We anticipate that where temporary shoring is installed, the permanent restrained retaining walls for the subterranean level will predominantly be placed directly against the temporary shoring. The design parameters provided below assume that granular non-expansive soils (Expansion Index <20 and SE≥30) are used to back-fill any retaining walls. Permanent subterranean walls should be designed to resist the pressure exerted by retained soils plus any additional lateral forces due to loads placed adjacent to or near the wall. Retaining walls that are free-draining, are situated above groundwater and are to be restrained from movement at the top, such as basement walls, should be designed for an equivalent fluid weight of 60 pcf for at-rest conditions (for a level surface of retained earth). If traffic loads are planned adjacent to the walls, the walls should be designed for an additional uniform horizontal pressure of 75 and 150 psf for passenger car and truck traffic, respectively. For other surcharge loads, we recommend the walls be designed to resist a uniform horizontal pressure equal to 30 percent of the uniform surcharge load.

If back-fill conditions (including the slope of the retained ground surface) differ from those assumed herein, Kling Consulting Group should be consulted to provide additional evaluation and/or recommendations as warranted. All retaining structures should be fully free draining. Building walls below grade should be waterproofed or damp-proofed, depending on the degree of moisture protection desired. The foundation system for the retaining walls should be designed in accordance with the recommendations presented in the preceding sections of this report, as appropriate. Footings should be embedded at a minimum of 18-inches below adjacent grade (excluding the 6-inch landscape layer).

For resistance to lateral loads, an allowable coefficient of friction of 0.35 between the base of the foundation elements and underlying material is recommended. In addition, an allowable passive resistance equal to an equivalent fluid weighing 250 pcf acting against the foundation may be used to resist lateral forces. Passive pressure in the upper 1.0-foot should be neglected unless confined by concrete slabs-on-grade or asphaltic pavement. These values may be increased by one-third for transient wind or seismic loads. A seismic surcharge of 19 H should be applied as an equivalent fluid pressure with the resultant acting at 1/3-height above the base of the wall, where H= the retained height of the wall greater than 6 feet.

The permanent subterranean wall should be provided with an adequate back drain system to reduce the potential for build-up of hydrostatic pressures.

Adequate drainage should be provided behind all retaining walls. The drainage system should consist of a minimum of four-inch diameter perforated PVC pipe (schedule 40 or approved equivalent) placed at the base of the retaining wall and surrounded by ¾-inch clean crushed rock wrapped in a Mirafi 140N filter fabric, or equivalent approved by the Geotechnical Engineer. The drain rock wrapped in fabric should be at least 12-inches wide and extend from the base of the wall to within two feet of the ground surface. The upper two feet of back-fill should consist of compacted native soil. The retaining wall drainage system should be sloped to outfall to the storm drain system or other appropriate facility.

For those portions of the wall not placed against shoring, the above values assume granular back-fill and free-draining conditions to prevent buildup of hydrostatic pressure in the back-fill. Back-fill materials should meet the recommendations described in the following section of this report. Import fill materials should be approved by the soils engineer prior to placement. Wall back-fill should be compacted by mechanical methods to at least 90 percent of the maximum dry density as determined by ASTM D 1557.

5.9 Temporary Excavations

We anticipate the on-site soils can be excavated using conventional heavy duty earthmoving equipment in good condition. Shoring systems, if used, may yield during excavation causing adjacent facilities and improvements to settle slightly. The magnitude of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill with installing the shoring system. Lateral deflections for a properly designed and constructed shoring system would likely be within ordinarily accepted limits of approximately 1-inch. A monitoring program should be established to evaluate the effects of shoring construction on other facilities.

Provided the excavations are above groundwater, temporary excavations and trench walls to a depth of four feet may be made vertically without shoring, subject to verification of safety by the contractor. Deeper excavations should be no steeper than 1.5:1 (horizontal to vertical) or braced or shored in accordance with CAL OSHA standards and guidelines. The contractor is assumed responsible for maintaining safety at the jobsite. All excavation work should be in compliance with current CAL OSHA standards. Under no circumstances should excavations be made deeper than four feet or below groundwater without shoring, bracing or laying-back, in accordance with CAL OSHA standards and guidelines. No surcharge loads should be allowed within five feet from the top of the cuts.

Existing utility lines, roadways and other easements/right-of-ways may be impacted by the temporary excavations may require shoring to obtain the full depth of the excavation.

5.10 Shoring

It is understood that a temporary or permanent shoring system may be warranted for areas of proposed subterranean basement excavation where space is not available for properly sloped backcuts. The shoring contractor should coordinate with the earthmoving contractor regarding sequence and requirements of installing the shoring system. The shoring contractor should also consider the potential for localized perched groundwater in the design and installation procedures of the shoring system.

We anticipate that the shoring system will be designed as a cantilever system and may consist of closely spaced steel H-Pile soldier piles and wooden lagging. Preliminary design considerations are presented in the following section for this anticipated shoring method. Please note that the method of temporary support can impact the design earth pressures. As such, Kling Consulting Group should perform a review of the shoring design and provide additional recommendations, as warranted.

Shoring systems, during excavation, may yield causing adjacent facilities and improvements to settle slightly. The magnitude of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill with installing the shoring system.

Lateral deflections for a properly designed and constructed shoring system would likely be within ordinarily accepted limits of approximately 1-inch. A monitoring program should be established to evaluate the effects of shoring construction on other facilities.

Horizontal and vertical movements of the shoring system should be monitored by a licensed surveyor. The construction monitoring and performance of the shoring system are ultimately the contractor's responsibility. At a minimum, we recommend that the tops of the soldier beams should be surveyed prior to excavation and that the top and bottom of the soldier beams be surveyed on a weekly basis until the foundation is completed. The surveyed soldier beam data points should be located at approximately 50 feet on-center. Surveying should consist of measuring movements in vertical and two perpendicular horizontal directions.

The shoring system should be designed to resist the pressure exerted by the retained soils plus any additional lateral forces due to loads applied near the top of the excavations. Cantilever shoring walls with a level back-fill surface should be designed for an equivalent fluid pressure of 40 pcf. For surcharge loads due to traffic, the shoring should be designed for an additional uniform horizontal pressure of 75 psf for passenger car traffic and 150 psf for heavy truck traffic. For other surcharge loads, the wall should be designed for a uniform horizontal pressure equal to one-third the anticipated surcharge pressure. These parameters all assume a level ground surface and that temporary shoring will not be subject to hydrostatic pressures. The shoring system should be properly embedded beneath the toe of the excavation to provide adequate structural stability.

It is recommended that the design of the shoring system incorporate a passive equivalent fluid weight of 250 pcf for the shoring embedded within relatively competent old paralic deposits material. The soldier piles should be spaced no closer than 3 diameters on center. The soldier piles should be drilled and back-filled with concrete to the full depth of the passive resistance zone. The area providing the passive resistance can be assumed to have a width equal to twice the concrete pile diameter.

The recommended passive pressure for the shoring assumes a horizontal surface for the soil mass extending at least 10 feet in front of the face of the shoring, or three times the height of the surface generating passive pressure, whichever is greater. The shoring system should be embedded a sufficient depth beneath the toe of the excavation so as to provide structural stability. We recommend that a factor of safety of at least 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 2,500 psf. The assumed geotechnical conditions should be verified as necessary during shoring construction by a representative of the geotechnical consultant.

Timber lagging may be used between the soldier piles to help support the exposed soils. If lagging is to remain after construction, treated lumber should be used. Lagging should be designed for the full lateral pressure recommended above. If possible, structural walls should be cast directly against the shoring, thus eliminating the need for placing back-fill within a narrow space. Voids between the soil and lagging should be properly grouted or slurried to reduce the potential for the voids to propagate to the surface.

Special provisions for wall drainage (such as the use of prefabricated composite drain) may be necessary above the groundwater table where this type of construction is used.

The performance of the proposed shoring system is highly dependent on the means and methods utilized by the contractors involved in the work and the judgment of the shoring design engineer. The shoring engineer and contractor shall be solely responsible for locating the existing improvements surrounding the site, controlling settlements of the surrounding structures and improvements within the structural and aesthetic limits. Load path and loading determination for underpinning design is the purview of the structural underpinning designer.

If the anticipated depth of excavation requires shoring that extends to depths where a cantilever shoring system is not feasible, we would be pleased to provide geotechnical recommendations for an anchored (tie-back) shoring system upon request. With deep excavations required to allow for the construction of subterranean levels that would normally require tie-back anchors, due to the proximity to the adjacent properties or structures tie-back systems may not be allowed and other options such as H-beam and lagging or rakers may be required.

5.11 Preliminary Pavement Design

Pavement section design is provided below based on near surface soil conditions encountered during our investigation and assumed traffic loading.

5.11.1 Asphalt Concrete Pavement

The upper on-site subgrade soils were classified as silty clays and clayey silts and sandy silts. To allow for soil variability, we are assuming an R-Value of 10 for preliminary design purposes.

Based on an R-value of 10, the parameters below are provided for preliminary design purposes. Pavement sections were calculated for traffic indices of 4.0 and 5.5, which are commonly used for parking stalls and drive aisles subject to passenger vehicles, respectively. However, the selection of actual traffic index should be the purview of the project civil or traffic engineer.

Pavement Section Design

Location	R-Value	Traffic Index	Multiple Layered	
			Asphalt Concrete (inches)	Aggregate Base* (inches)
Parking Stall	10	4.0	3.0	6.0
Drive Aisles	10	5.5	4.0	9.0

*Aggregate base material should consist of Class 2 aggregate base materials or Crushed Miscellaneous Base (CMB).

The upper 12 inches of the subgrade soils should be compacted to at least 90 percent of the laboratory maximum dry density (ASTM D1557). All base materials should be compacted to at least 95 percent of the laboratory maximum dry density (ASTM D1557).

5.11.2 Portland Cement Concrete Pavement

For preliminary design of concrete pavement, it is recommended that a concrete pavement section consisting of 6-inches of concrete underlain by at least 4-inches of either Class 2 or crushed miscellaneous base be used for preliminary design. Concrete Compressive strength should be 4000 psi or greater. Aggregate base material should be compacted to a minimum of 95 percent relative compaction as per ASTM D1557. Subgrade soil should be compacted to at least 90 percent of the laboratory maximum dry density in accordance with ASTM D1557. If concrete crack control is desired, the slabs should be minimally reinforced with No. 4 rebar, placed every 24 inches on center, both ways. A 10-foot square or less grid system should be used in the construction of continuous sections of concrete pavement or as recommended by the structural engineer.

For trash enclosures, concrete pavement should consist of a minimum 8-inch thick concrete slab placed over a minimum of 6-inches of either Class 2 or crushed miscellaneous base material, compacted to 95 percent relative compaction. Concrete should have a minimum strength of 4000 psi and be reinforced with a minimum of No. 4 bars placed at 24 inches on center, in each direction, positively supported (with concrete chairs or other devices) at mid-height in the slab. Crack control joints should be placed at a 10-foot maximum spacing in each direction in the slab or as recommended by the structural engineer. Concrete mix design should incorporate the recommendations presented in the slab on grade section of this report for improved geotechnical performance.

5.12 Exterior Flatwork

Laboratory testing of onsite soils by and our experience with similar soils in the site vicinity indicate that the upper on-site soil materials present possess a very low to high expansion potential.
Appendix D contains a table listing our hardscape recommendations for varying degrees of expansive soils. This table should be preliminarily followed for a low to high expansion potential for Expansion Index (E.I.) = 21 to 130. Additional testing should be performed during future supplemental investigation and subsequently during earthwork construction to confirm the as graded conditions.

The following general recommendations may be considered for concrete hardscape including expansive soils mitigation and may be superseded by the requirements of the City of Newport Beach. These recommendations are based on “medium” expansion potential and are preliminary.

5.12.1 Sidewalk, Pedestrian Walkways

Expansion Potential	Minimum Concrete Thickness (in)	Subgrade Pre-Soaking Depth	Reinforcement	Joint * Spacing
Medium (EI >51 & <90)	4 (Full)	120% of Optimum to 18" (or 5% over optimum,	#3 @ 16" OC, EW	4-5 Feet

* Joints at curves and angle points are recommended.

The above recommendations may be superseded by the project architect, structural engineer or the governing agency's requirements. These recommendations are not intended to mitigate cracking caused by shrinkage and temperature warping.

5.13 Drainage

Positive drainage should be maintained away from any building or graded slope face and directed to suitable areas via non-erosive devices, as designed by the project civil engineer. For drainage over soil and paved areas immediately adjacent to structures, please refer to Section 1804.4 of the 2019 CBC.

5.14 Geotechnical Observation and Testing

Geotechnical observation and testing should be conducted during the following stages of grading:

- During all phases of precise grading, footing excavations, etc.
- During slab subgrade pre-saturation and moisture conditioning.
- During utility trench excavation and compaction.
- During placement of retaining wall sub-drainage, back-fill, and compaction.
- For any unusual conditions encountered during grading.

6.0 PROFESSIONAL LIMITATIONS

Geotechnical services are provided by KCG in accordance with generally accepted professional engineering and geologic practice in the area where these services are to be rendered. Client acknowledges that the present standard in the engineering and geologic and environmental profession does not include a guarantee of perfection and, except as expressly set forth in the conditions above, no warranty, expressed or implied, is extended by KCG.

Geotechnical reports are based on the project description and proposed scope of work as described in the proposal. Our conclusions and recommendations are based on the results of the field, laboratory, and office studies, combined with an interpolation and extrapolation of soil conditions as described in the report. The results reflect our geotechnical interpretation of the limited direct evidence obtained. Our conclusions and recommendations are made contingent upon the opportunity for KCG to continue to provide geotechnical services beyond the scope in the proposal to include all geotechnical services. If parties other than KCG are engaged to provide such services, they must be notified that they will be required to assume complete responsibility for the geotechnical work of the project by concurring with the recommendations in our report or providing alternate recommendations.

It is the reader's responsibility to verify the correct interpretation and intention of the recommendations presented herein. KCG assumes no responsibility for misunderstandings or improper interpretations that result in unsatisfactory or unsafe work products. It is the reader's further responsibility to acquire copies of any supplemental reports, addenda, or responses to public agency reviews that may supersede recommendations in this report.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX A

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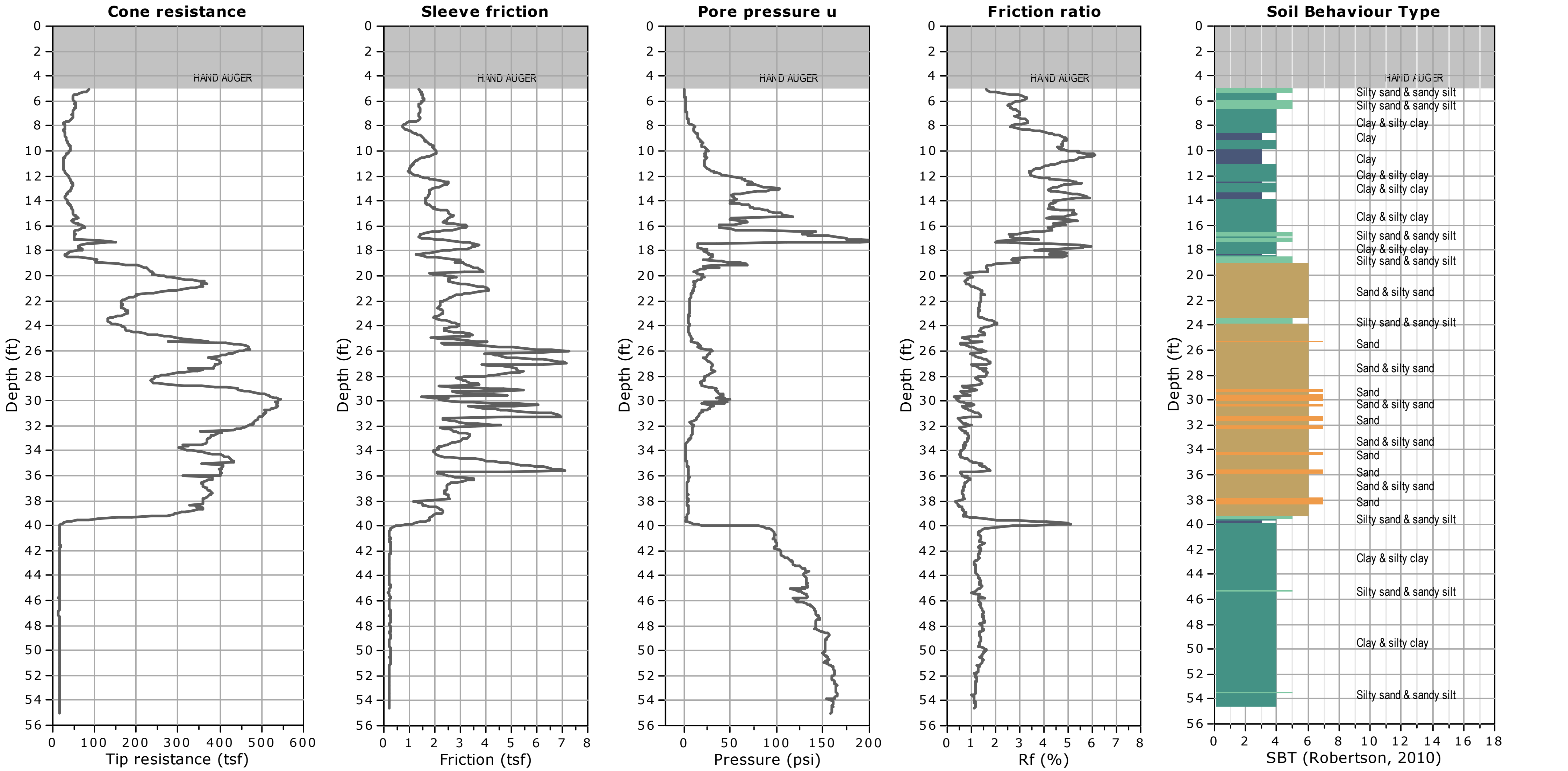
APPENDIX B

CPT SOUNDINGS

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Kling Consulting Group
Location: 1400 Bristol St. N, Newport Beach, CA

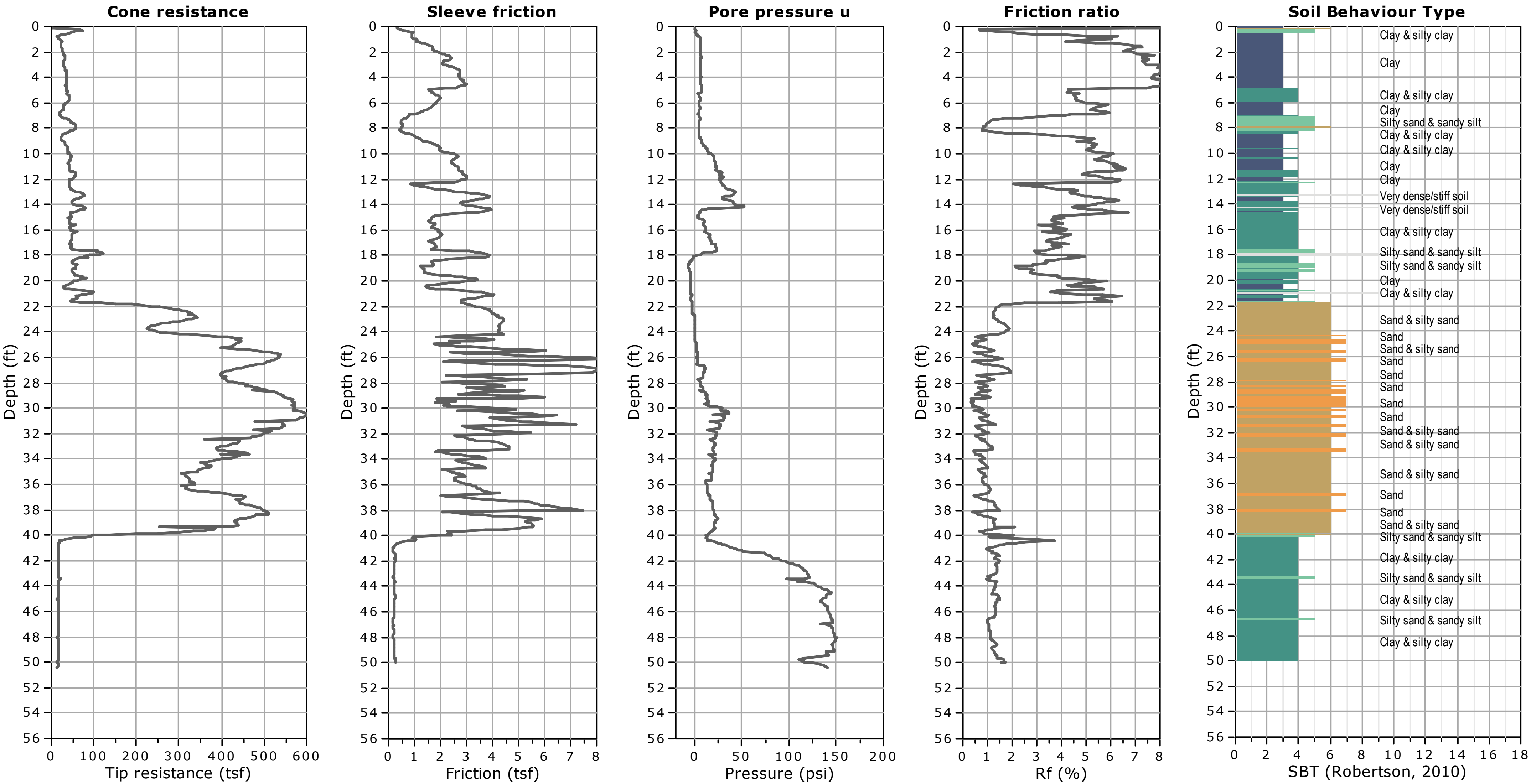
CPT-1
Total depth: 55.00 ft, Date: 10/10/2022



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Kling Consulting Group **CPT-2**

Location: 1400 Bristol St. N, Newport Beach, CA Total depth: 50.40 ft, Date: 10/10/2022



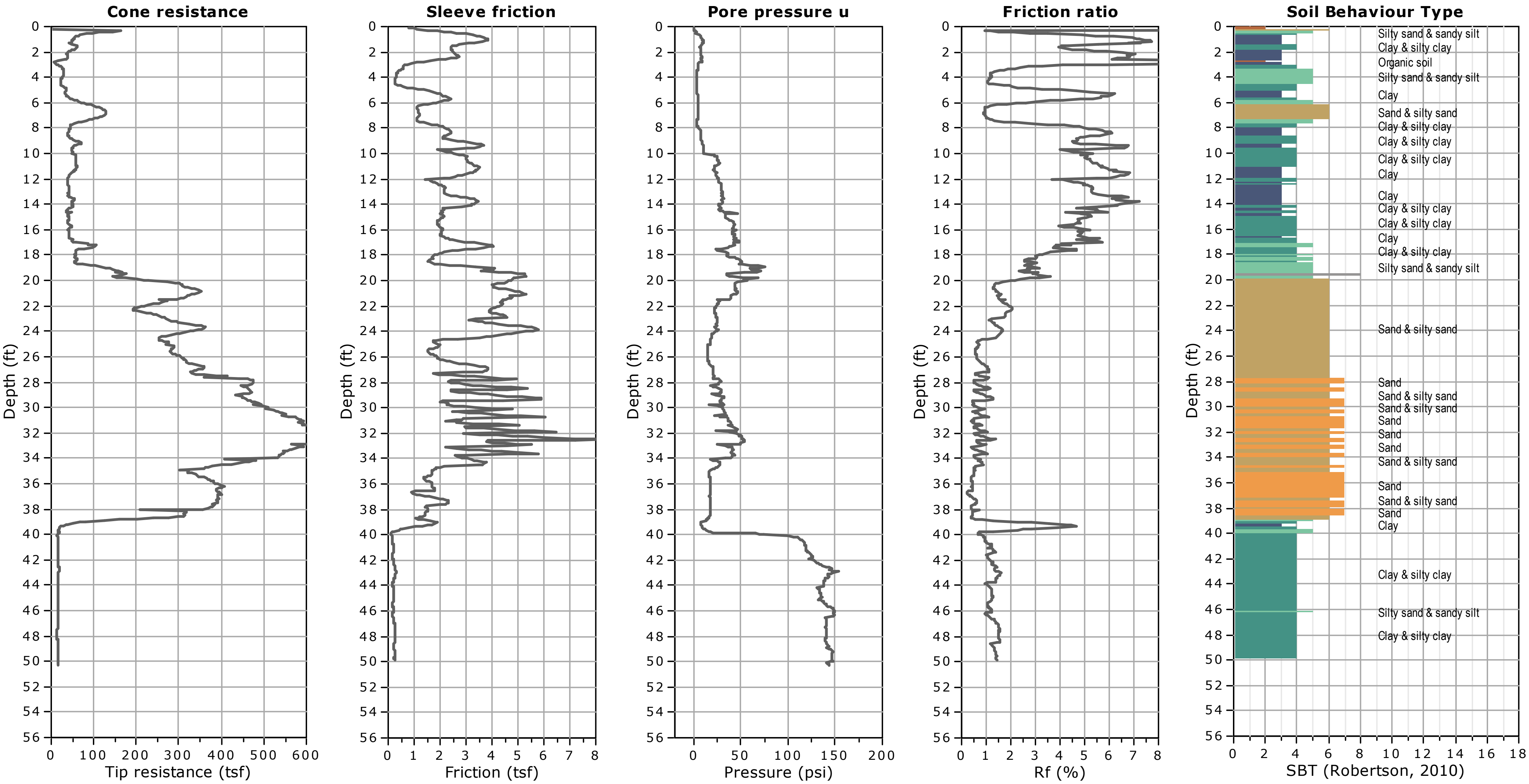
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **Kling Consulting Group**

Location: **1400 Bristol St. N, Newport Beach, CA**

CPT-3

Total depth: 50.27 ft, Date: 10/10/2022



APPENDIX C

LIQUEFACTION AND SEISMIC SETTLEMENT ASSESSMENT

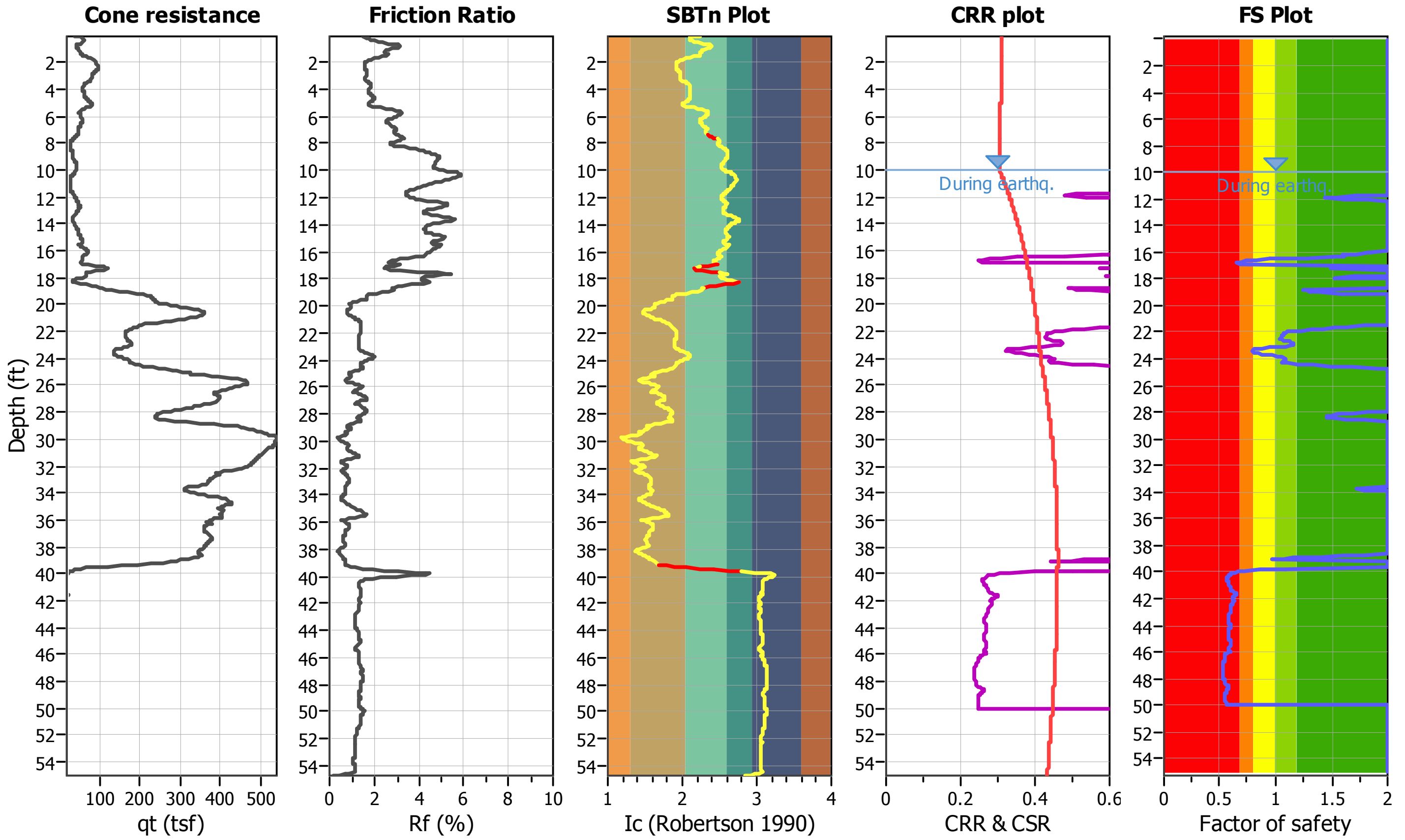
Kling Consulting Group, Inc.
18008 Sky Park Circle, Suite 250
Irvine, CA 92614
www.klingconsultinggroup.com

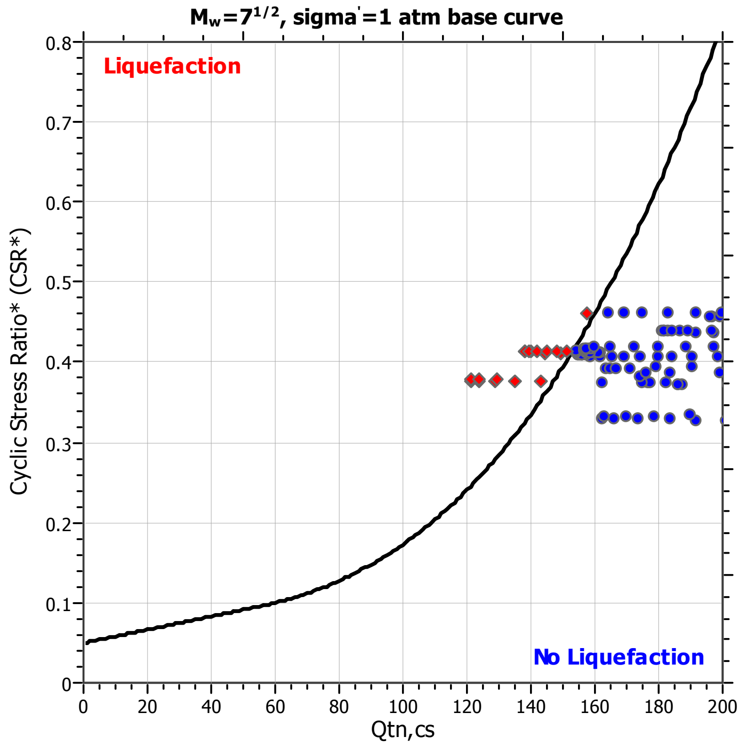
Project title : **Location :**

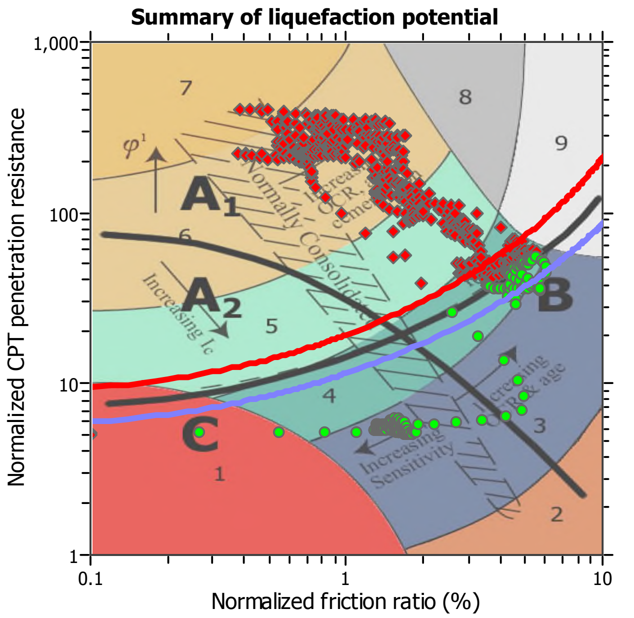
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	40.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.82	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		

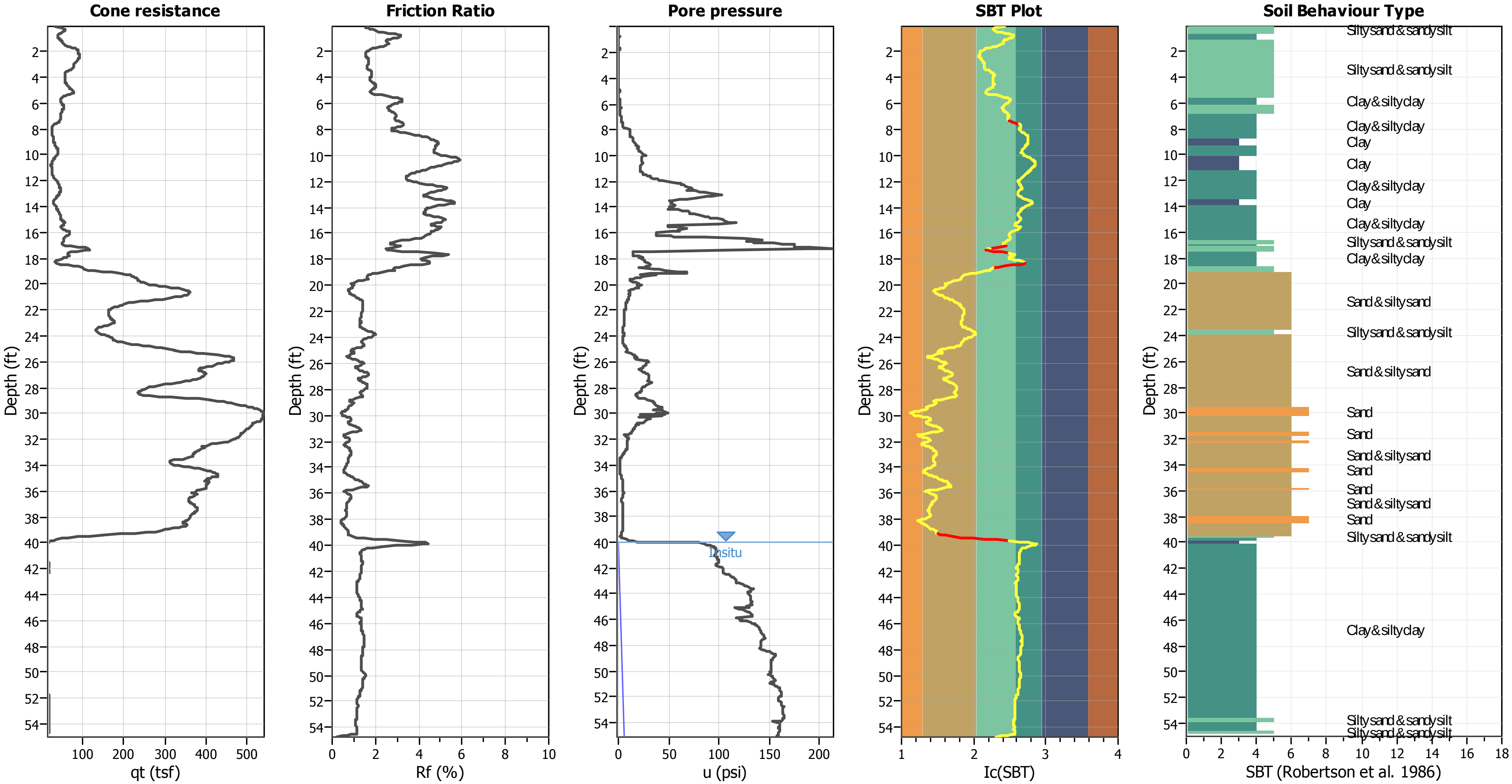






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



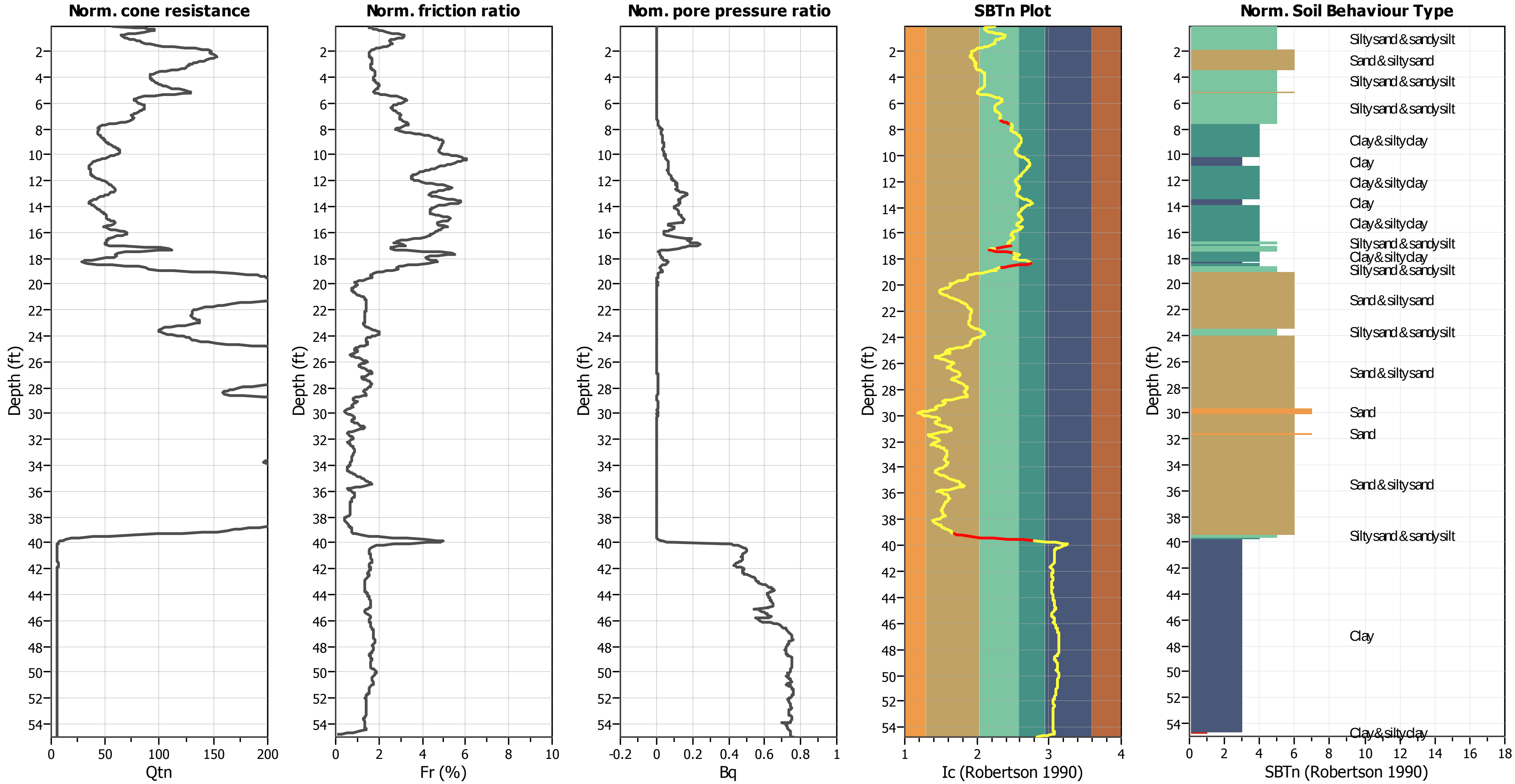
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



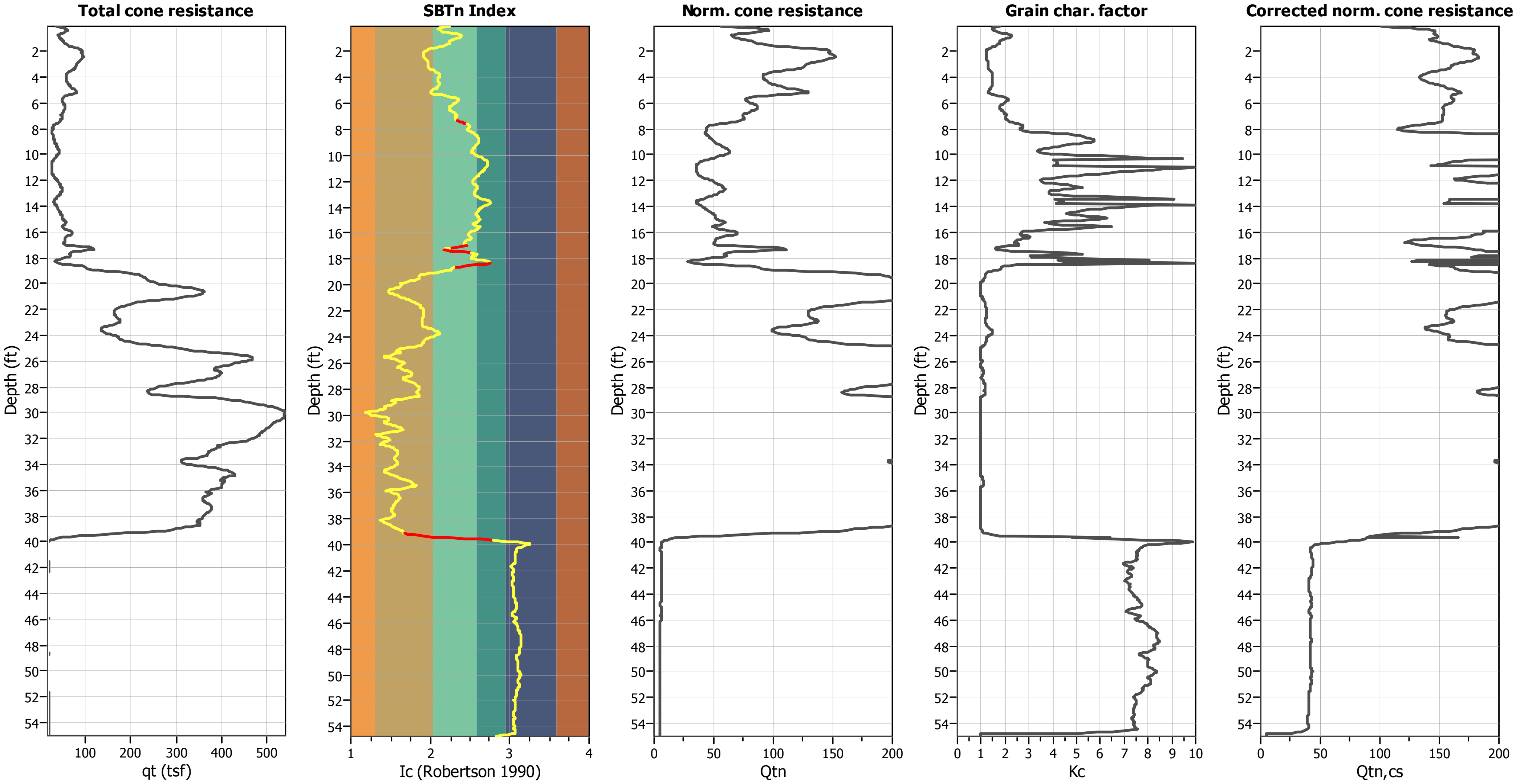
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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

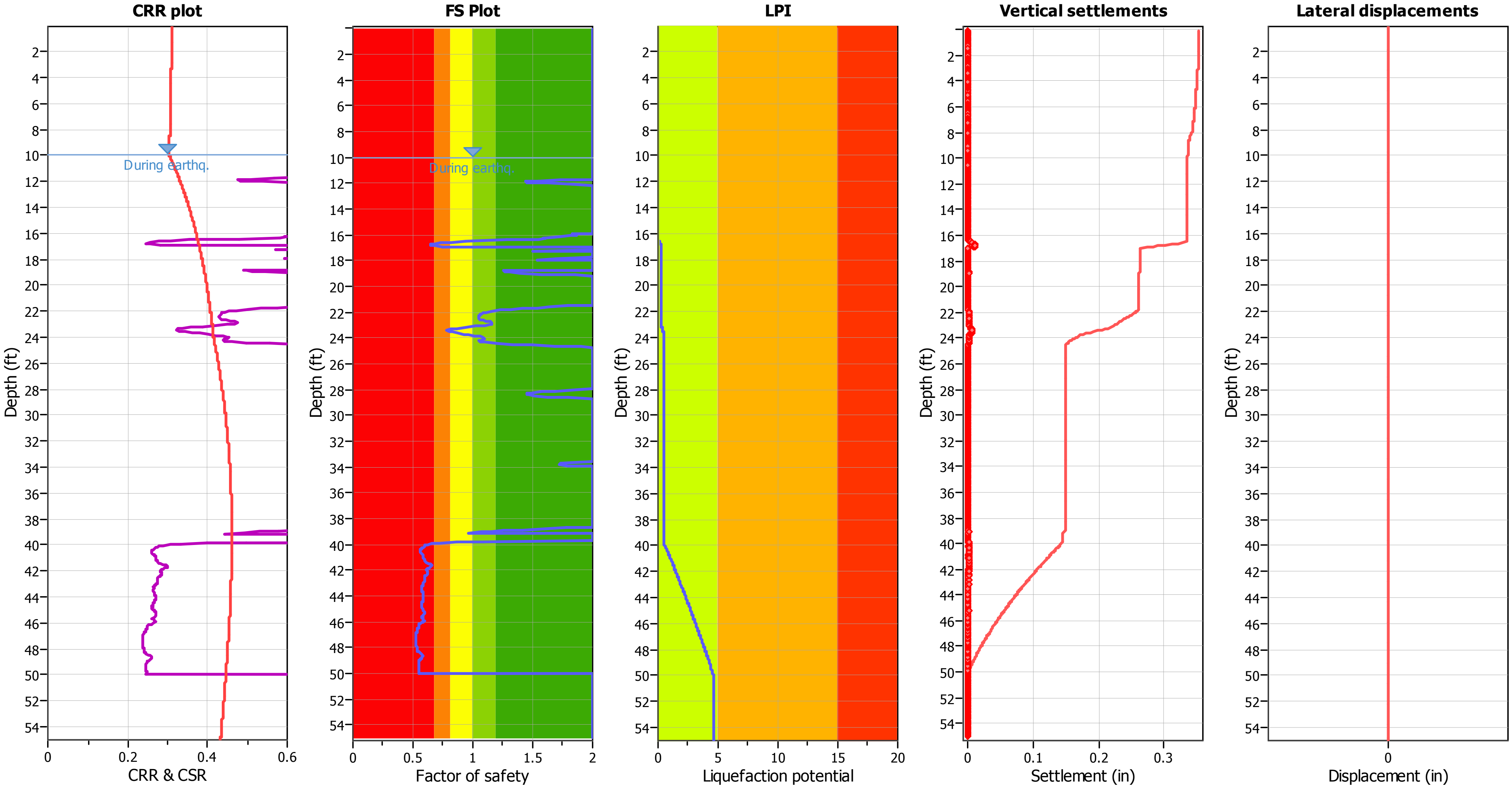
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

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Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 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

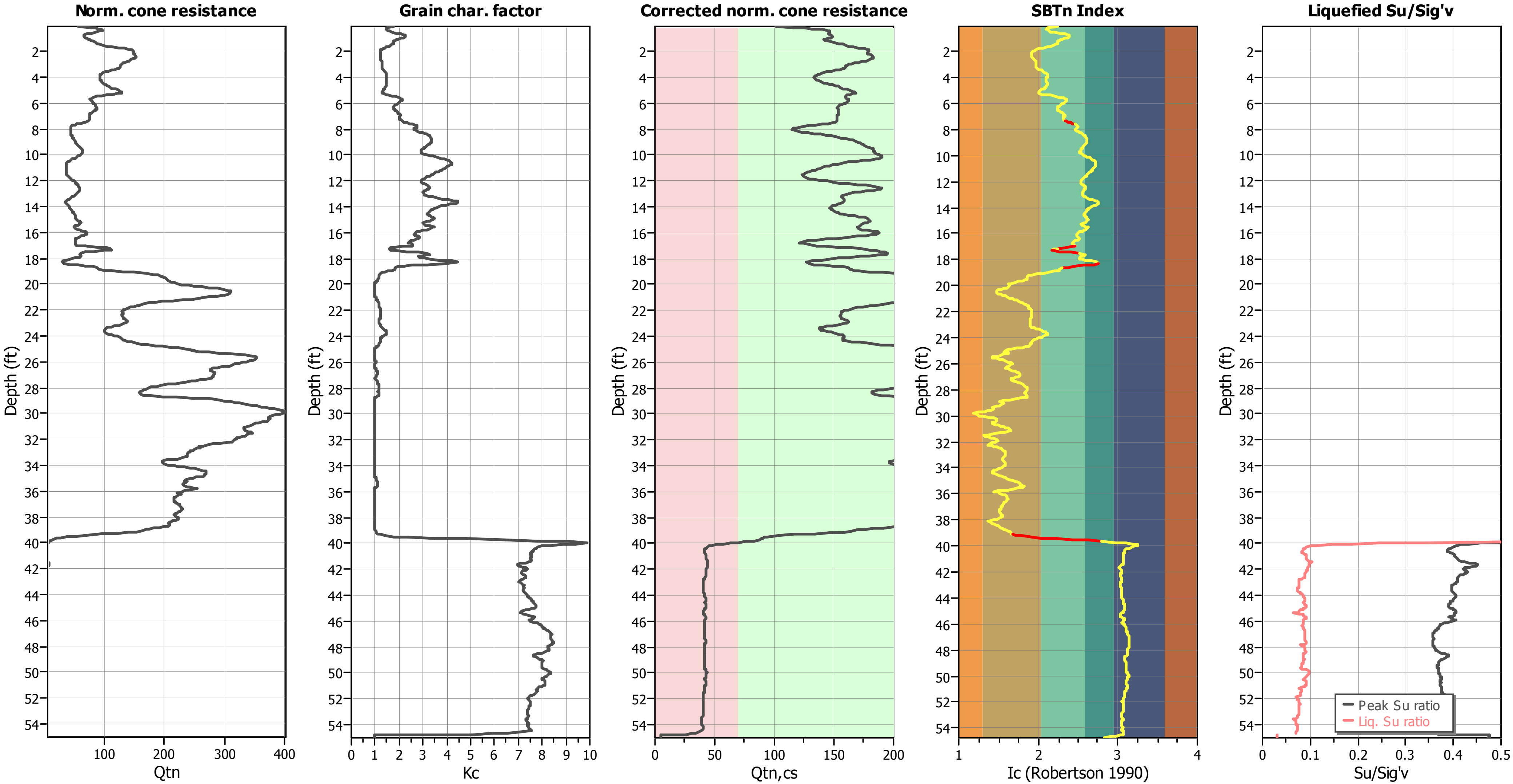
- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots

Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

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Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Transition layer No	Number of points	Depth	SBT_n number	SBT_n description

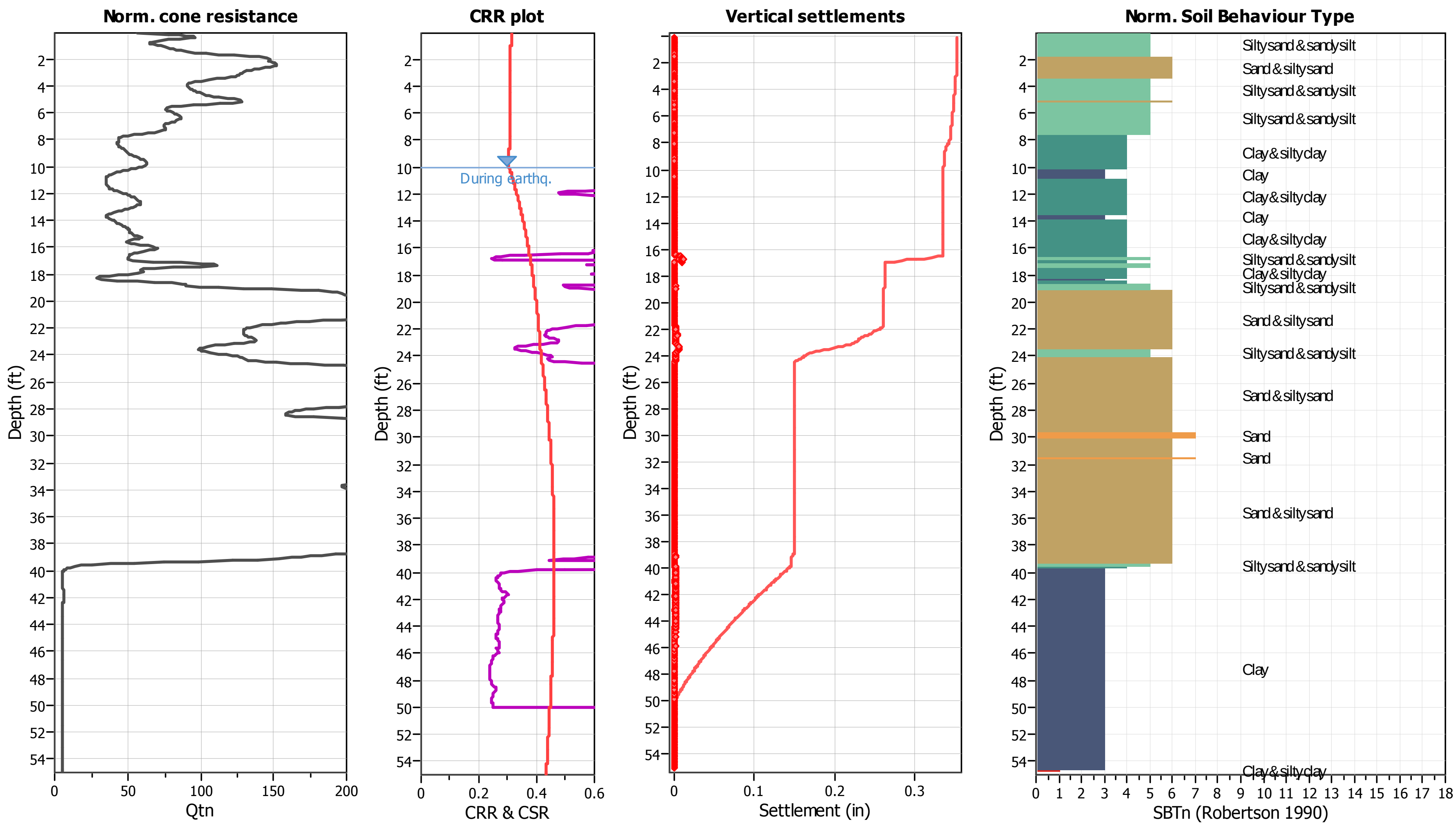
Kling Consulting Group, Inc.
18008 Sky Park Circle, Suite 250
Irvine, CA 92614
www.klingconsultinggroup.com

Project:

Location:

CPT: CPT-1

Total depth: 55.00 ft



Analysis method:	Robertson (2009)	G.W.T. (in-situ):	40.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.82	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		

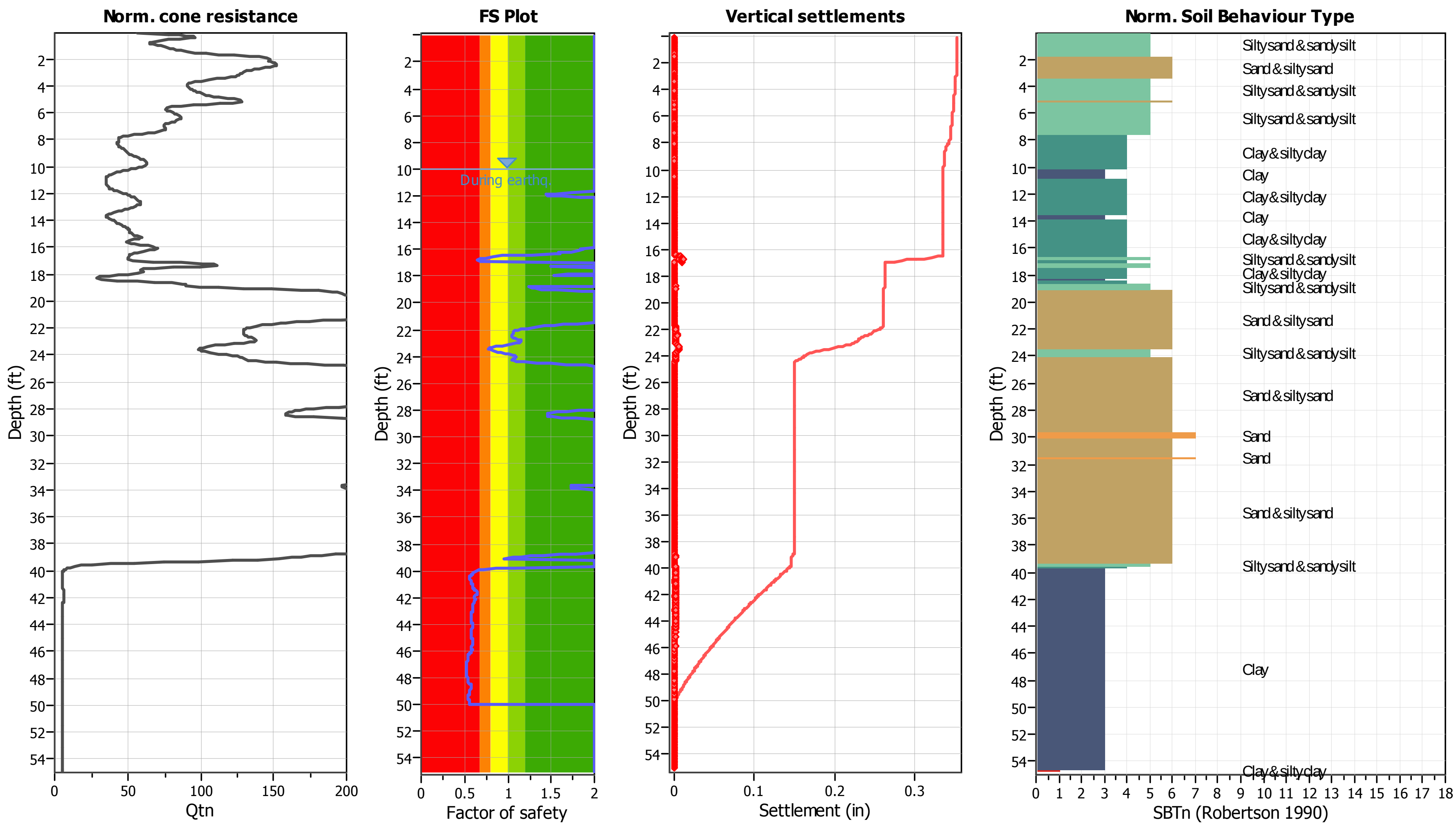
Kling Consulting Group, Inc.
18008 Sky Park Circle, Suite 250
Irvine, CA 92614
www.klingconsultinggroup.com

Project:

Location:

CPT: CPT-1

Total depth: 55.00 ft



Analysis method:	Robertson (2009)	G.W.T. (in-situ):	40.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.82	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		

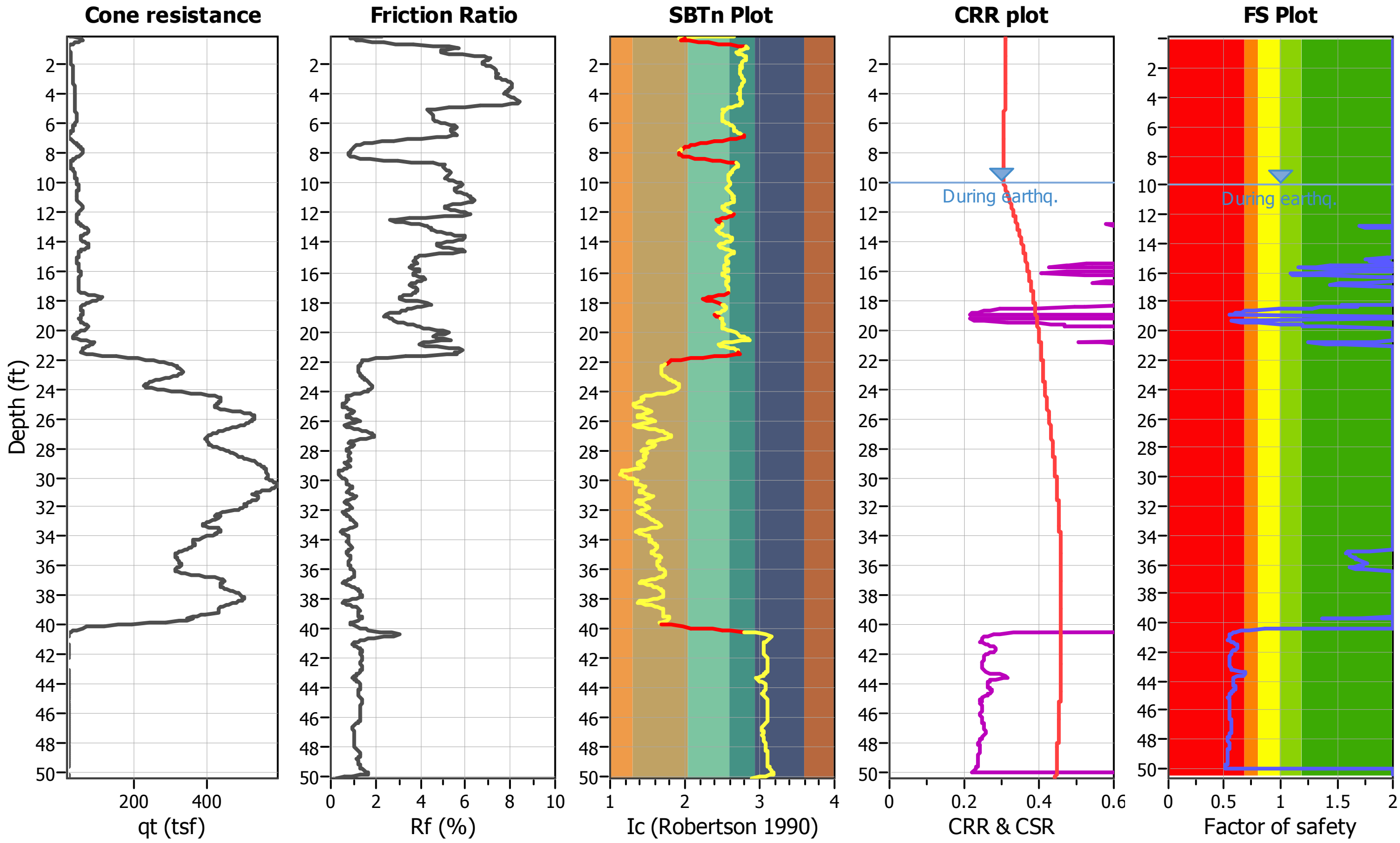
Kling Consulting Group, Inc.
18008 Sky Park Circle, Suite 250
Irvine, CA 92614
www.klingconsultinggroup.com

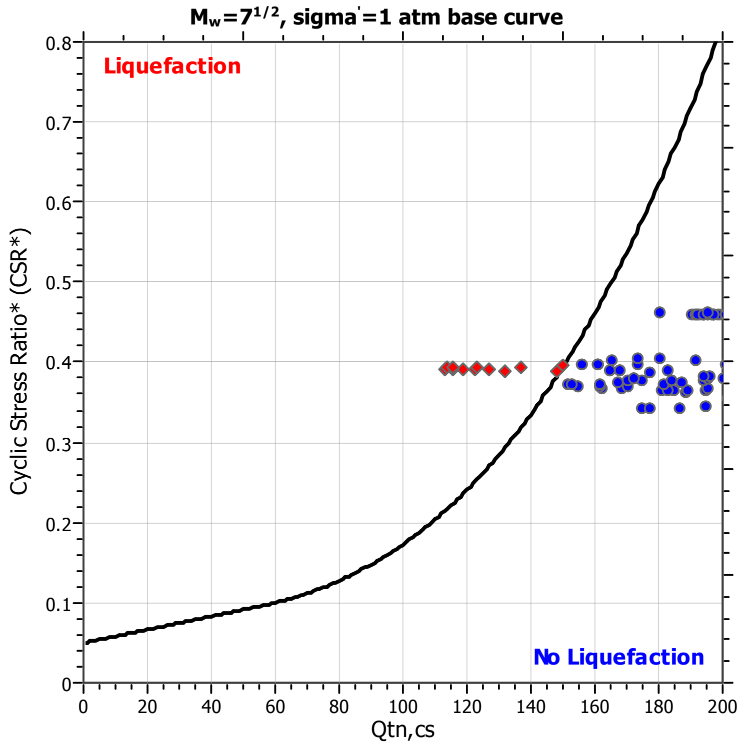
Project title : **Location :**

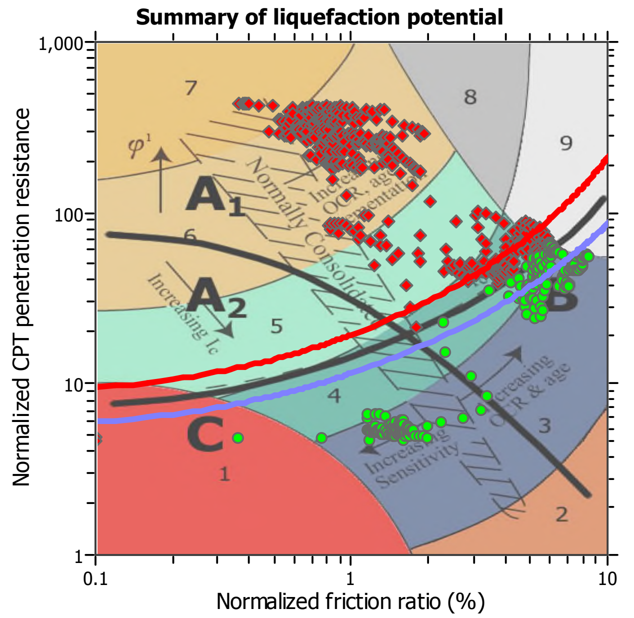
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	40.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.82	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		

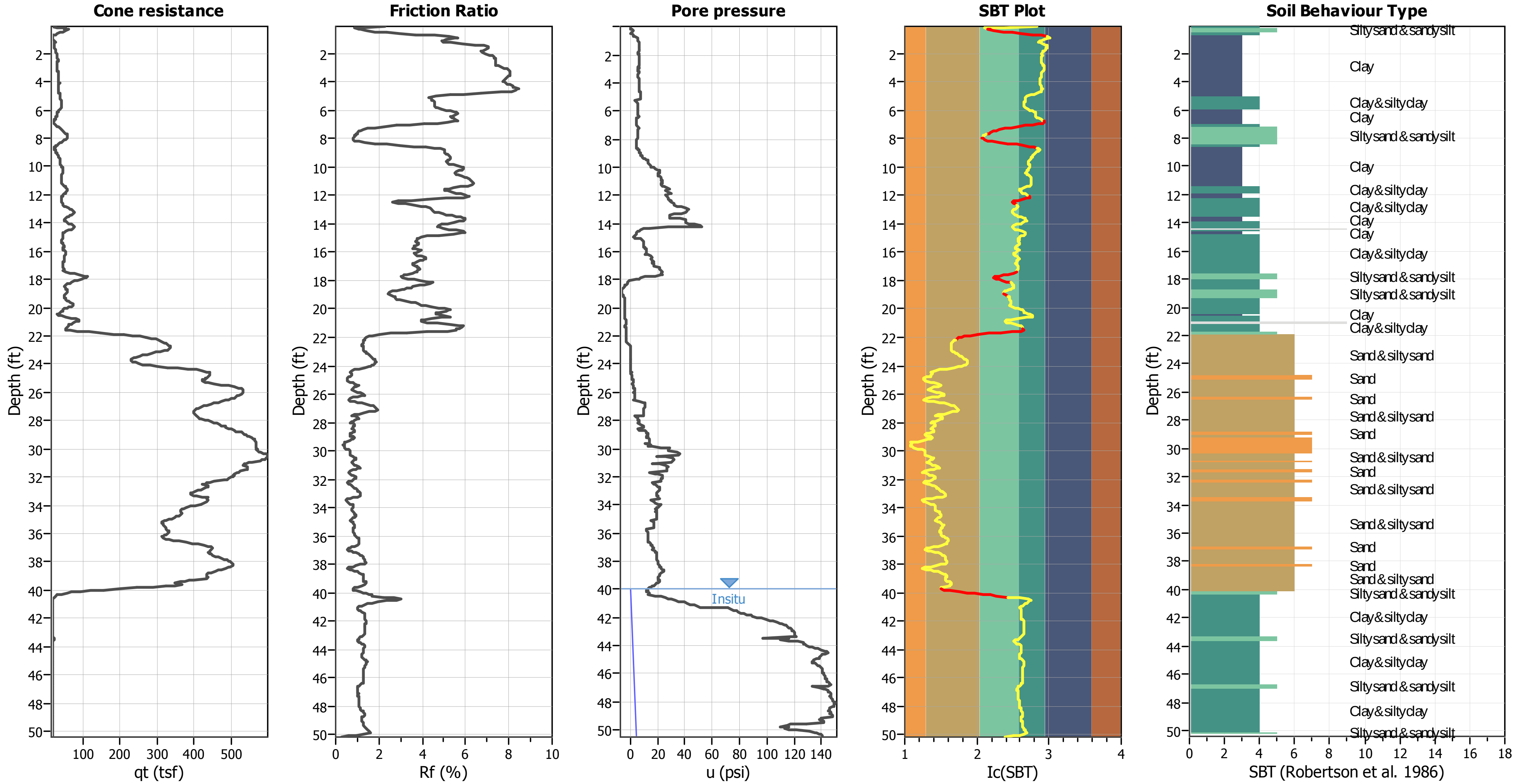






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



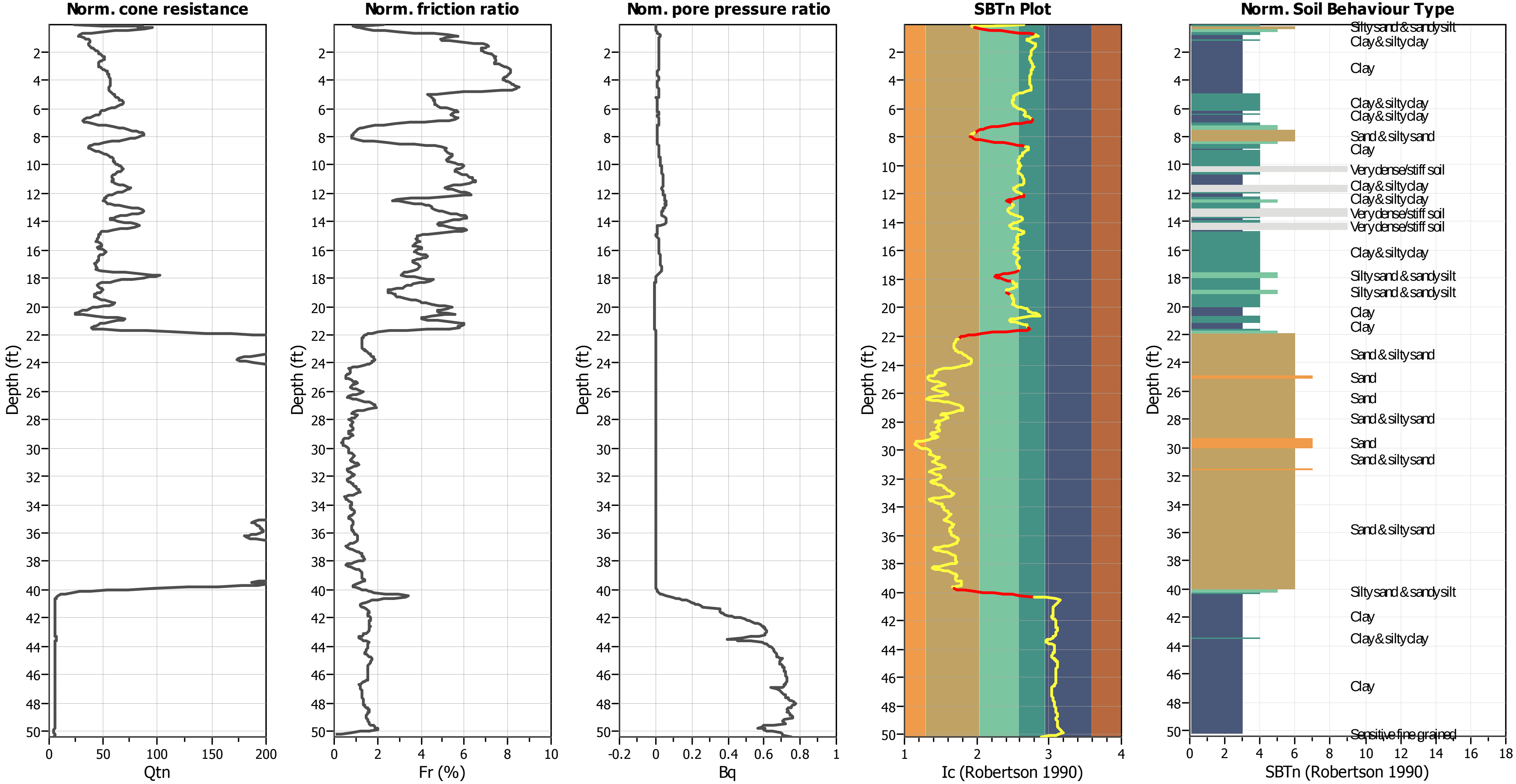
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



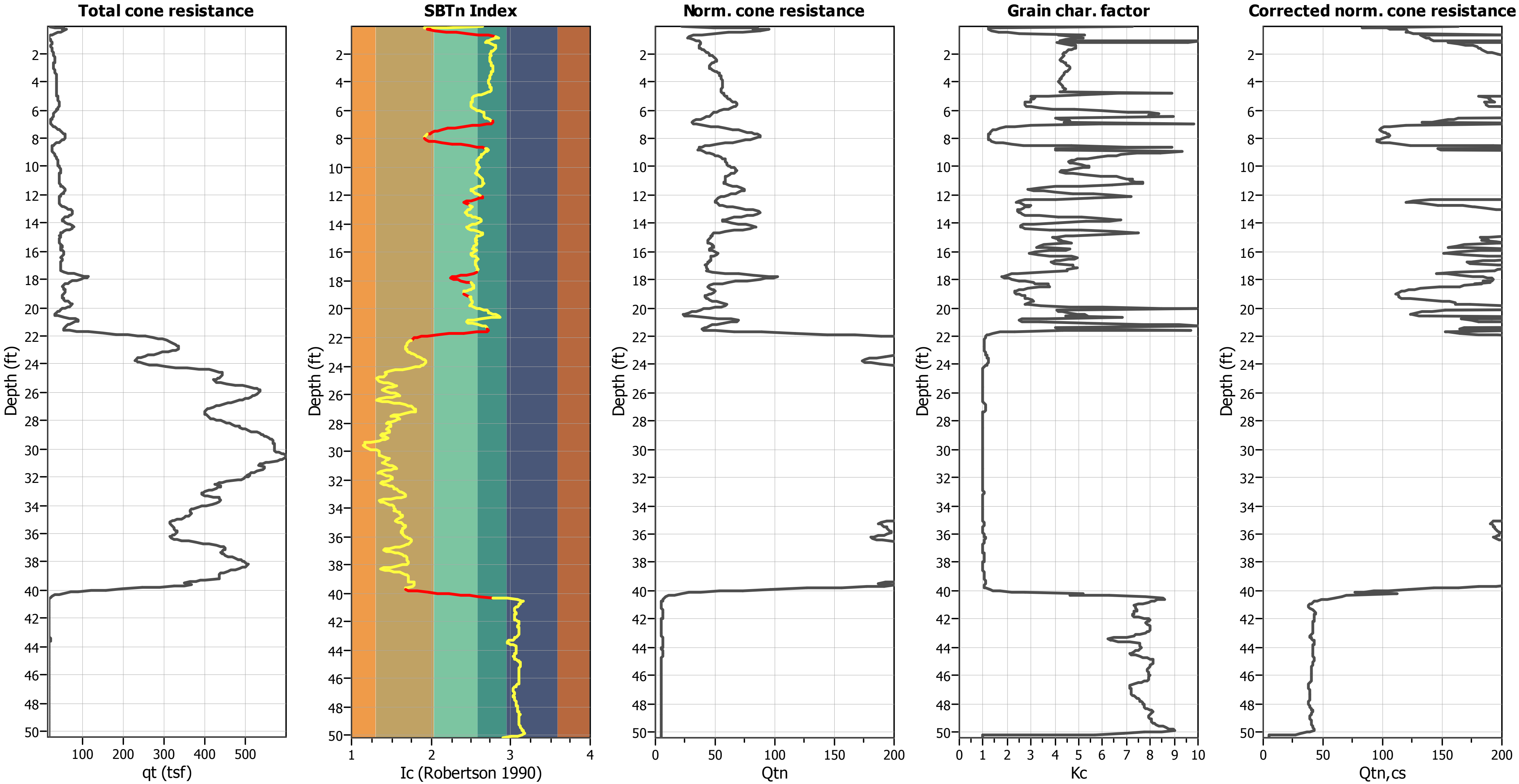
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 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

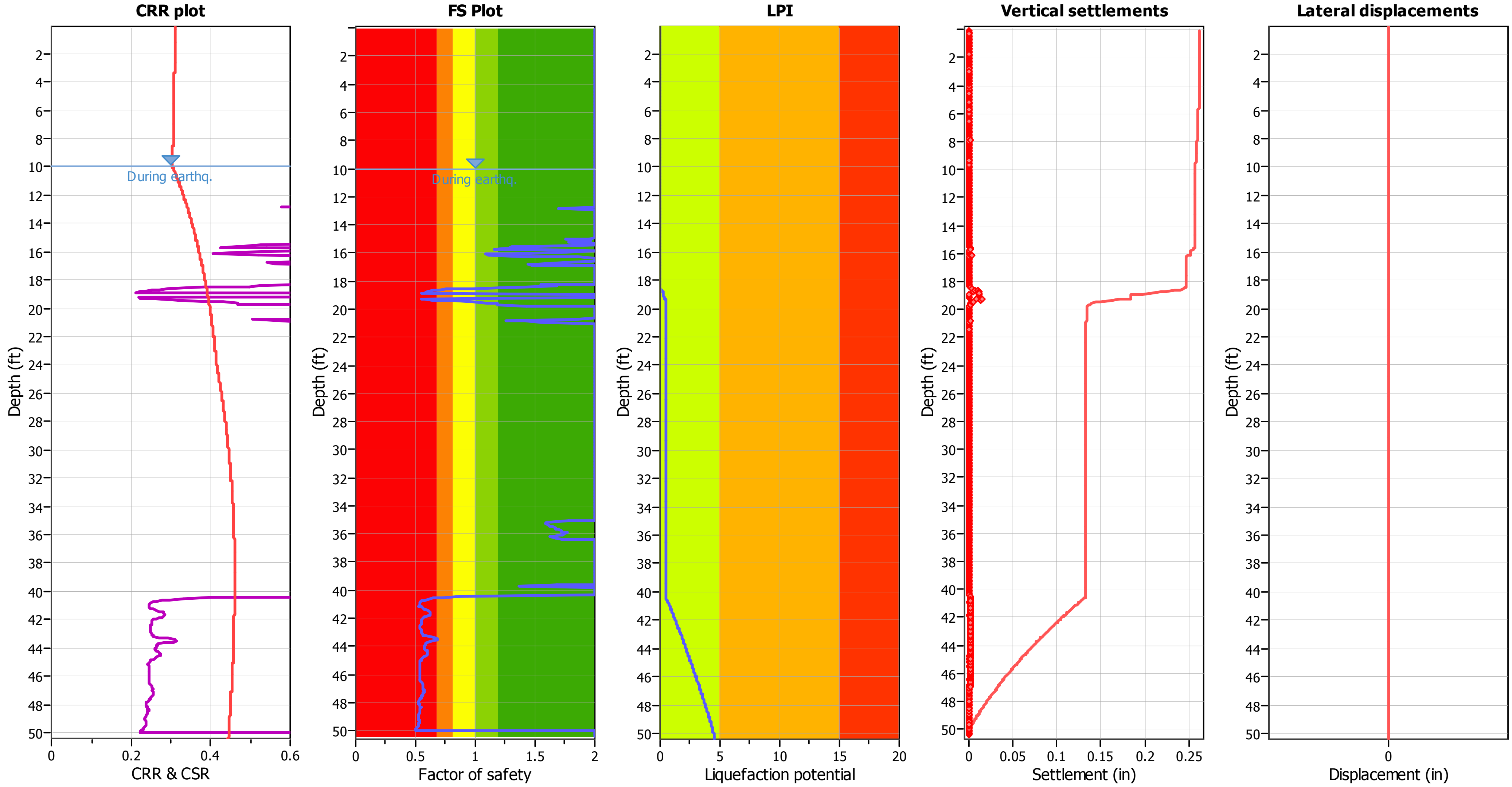
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 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

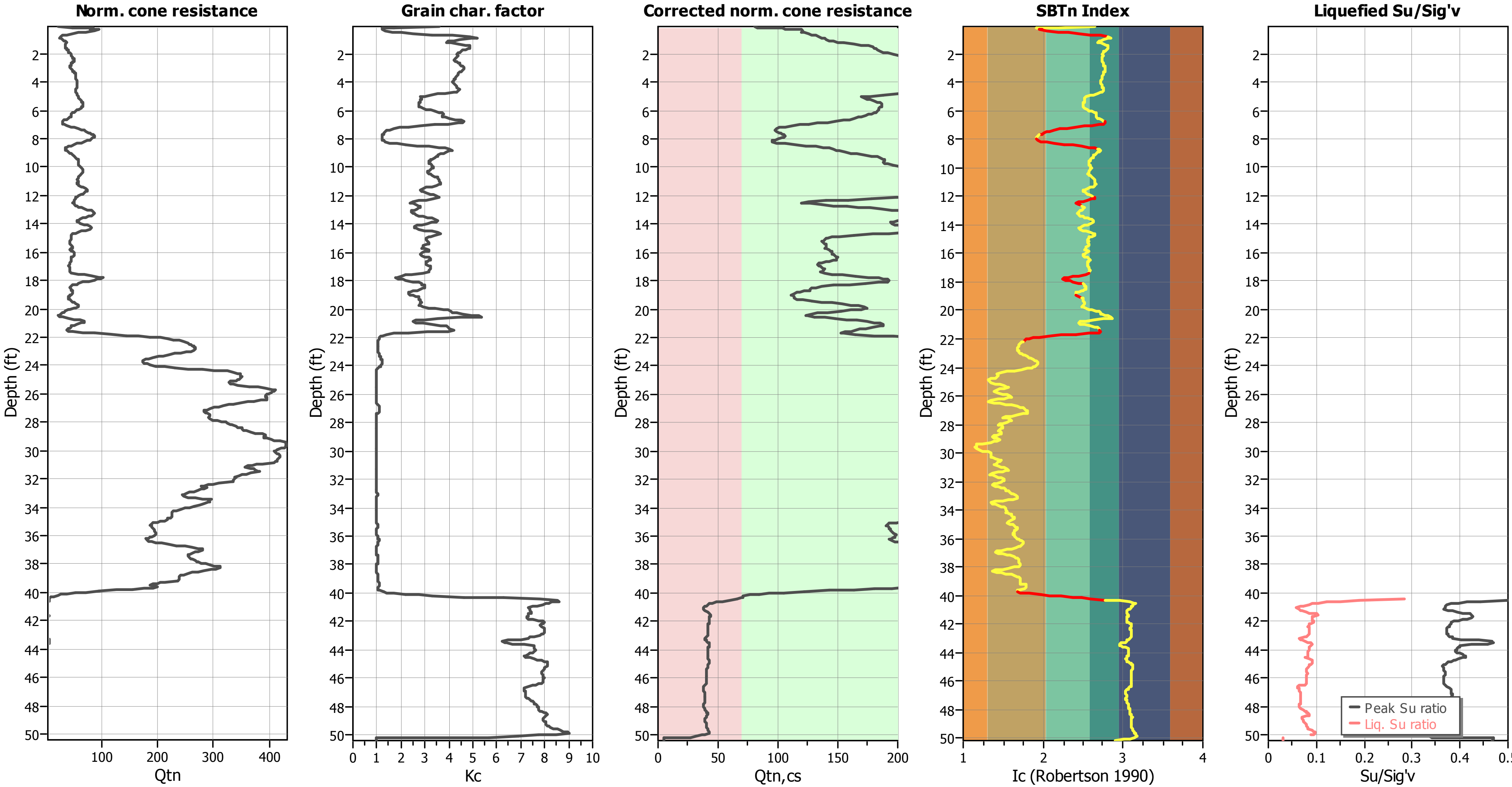
- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots

Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Transition layer No	Number of points	Depth	SBT_n number	SBT_n description

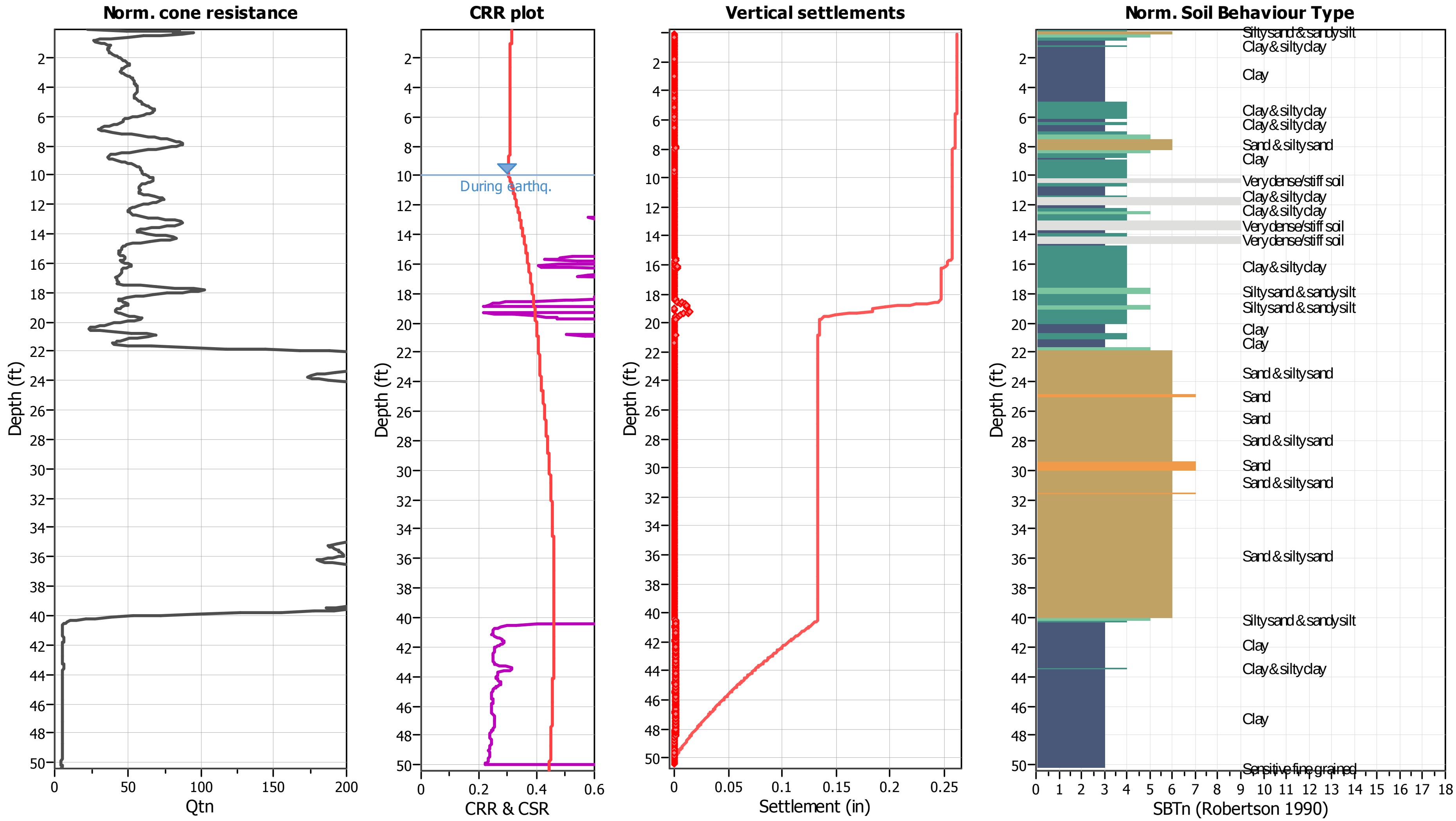
Kling Consulting Group, Inc.
18008 Sky Park Circle, Suite 250
Irvine, CA 92614
www.klingconsultinggroup.com

Project:

Location:

CPT: CPT-2

Total depth: 50.40 ft



Analysis method:	Robertson (2009)	G.W.T. (in-situ):	40.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.82	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		

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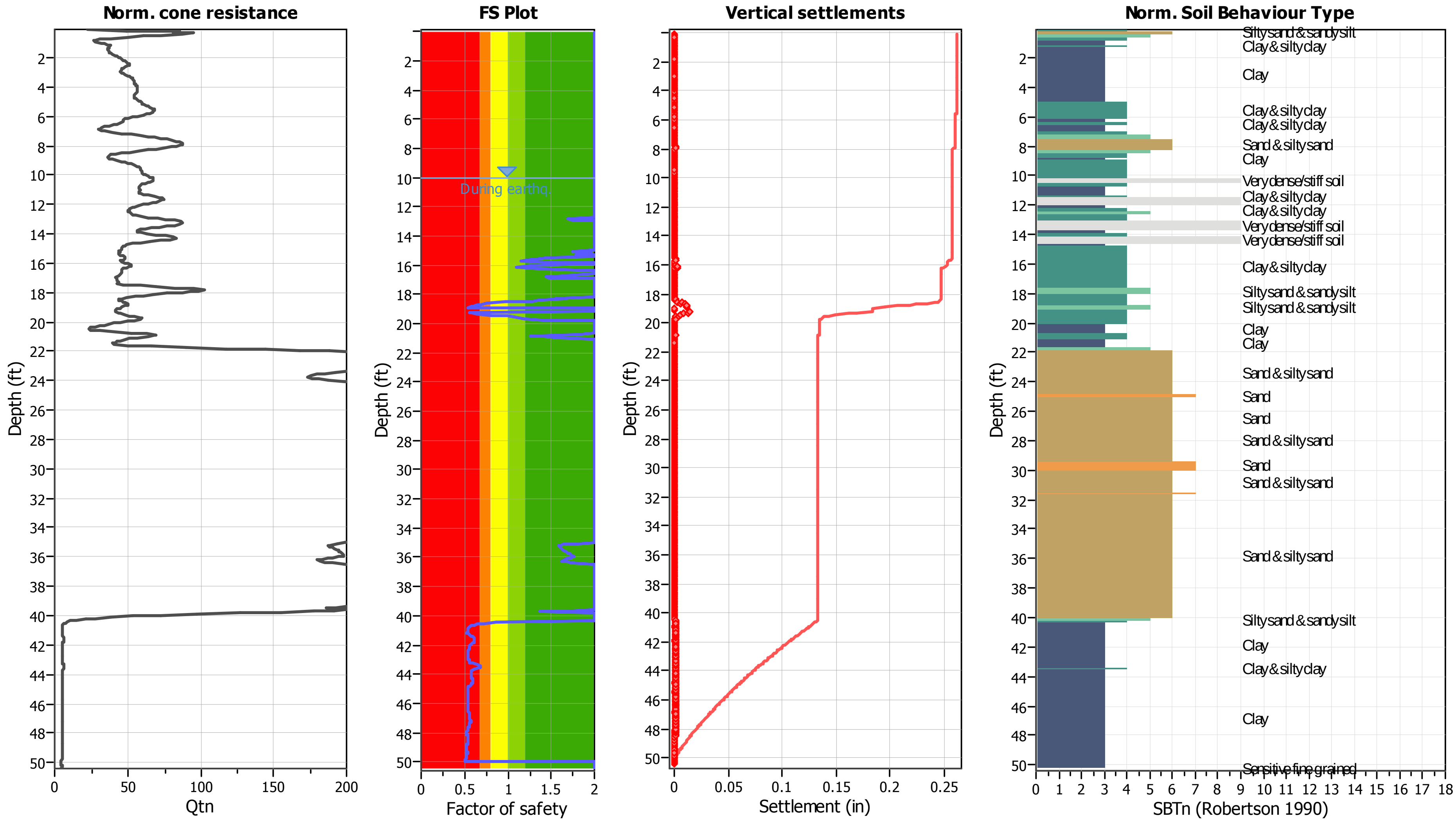
Irvine, CA 92614
www.klingconsultinggroup.com

Project:
Location:

CPT: CPT-2
Total depth: 50.40 ft



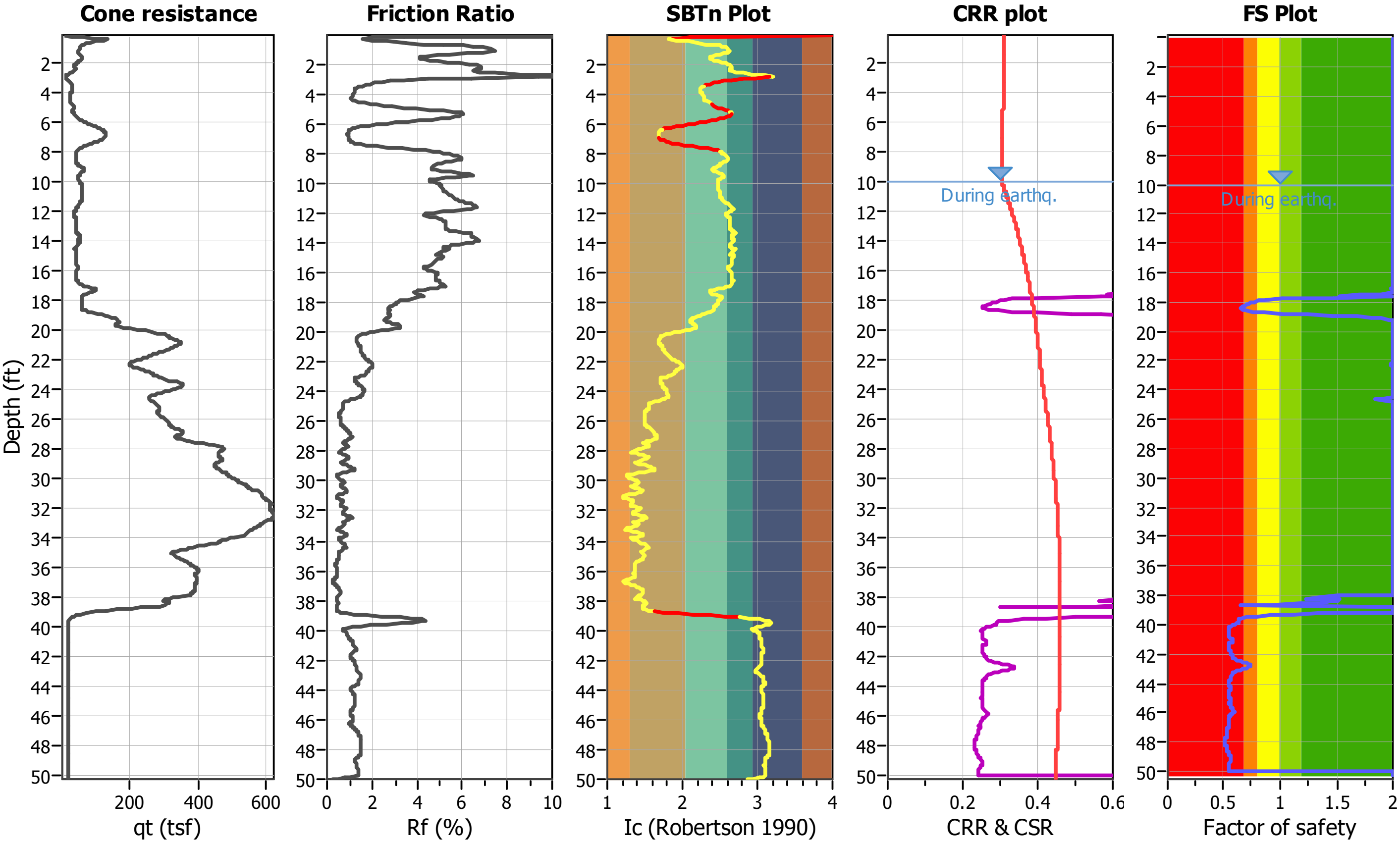
Analysis method:	Robertson (2009)	G.W.T. (in-situ):	40.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.82	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		

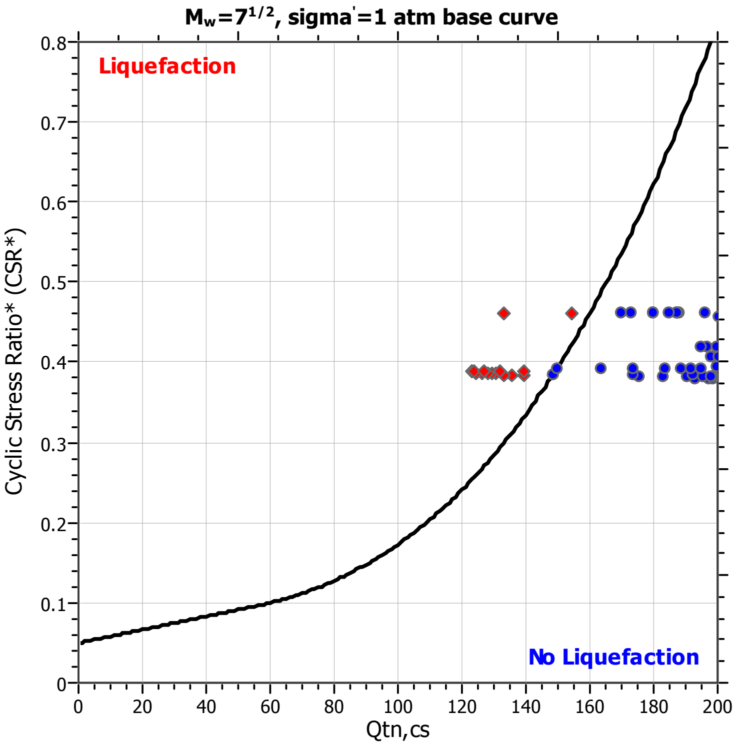
Project title : **Location :**

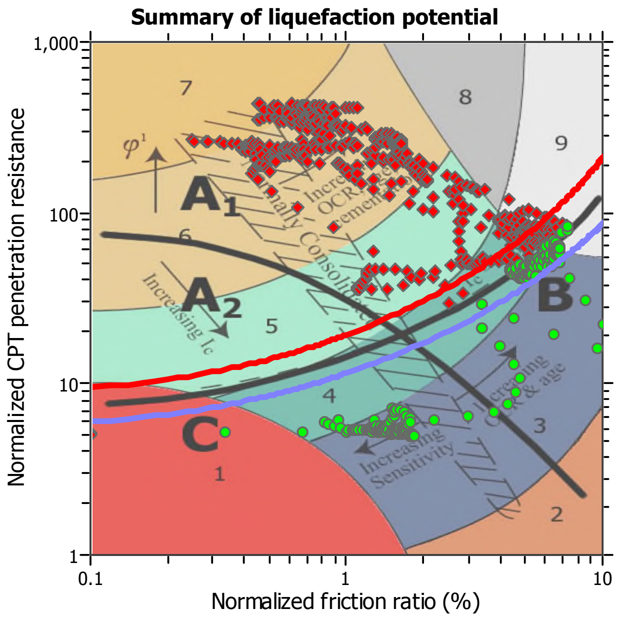
CPT file : CPT-3

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	40.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.82	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		

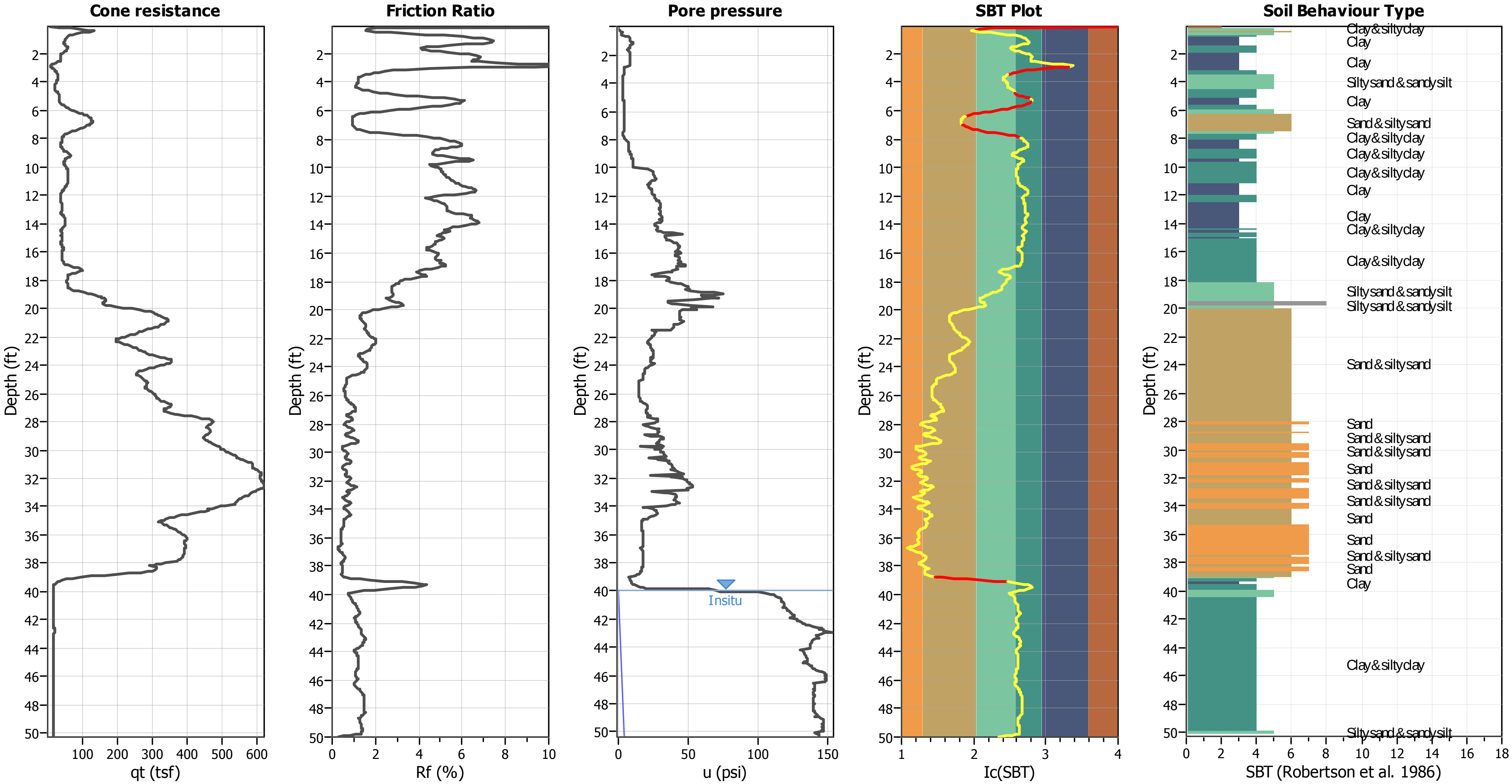






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



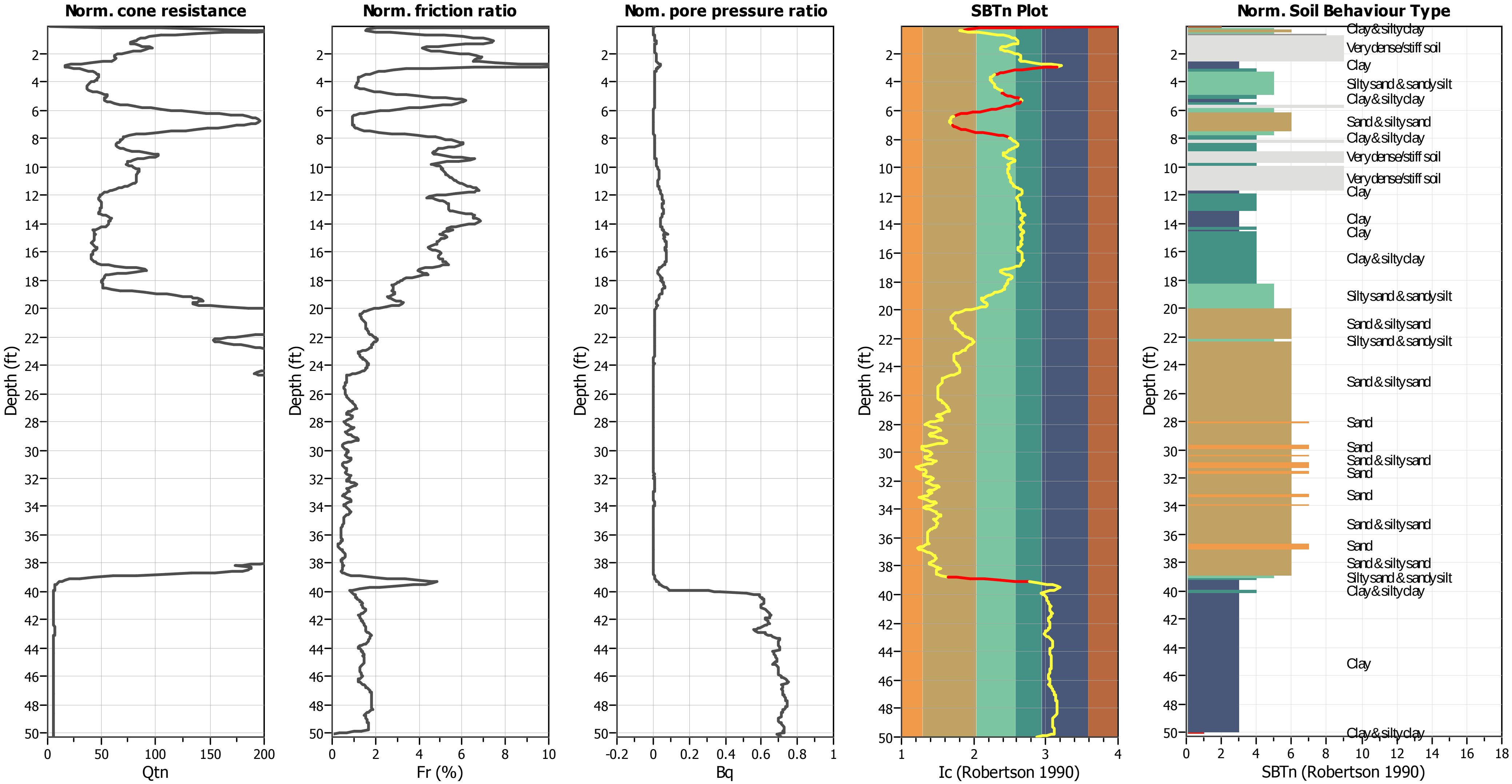
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



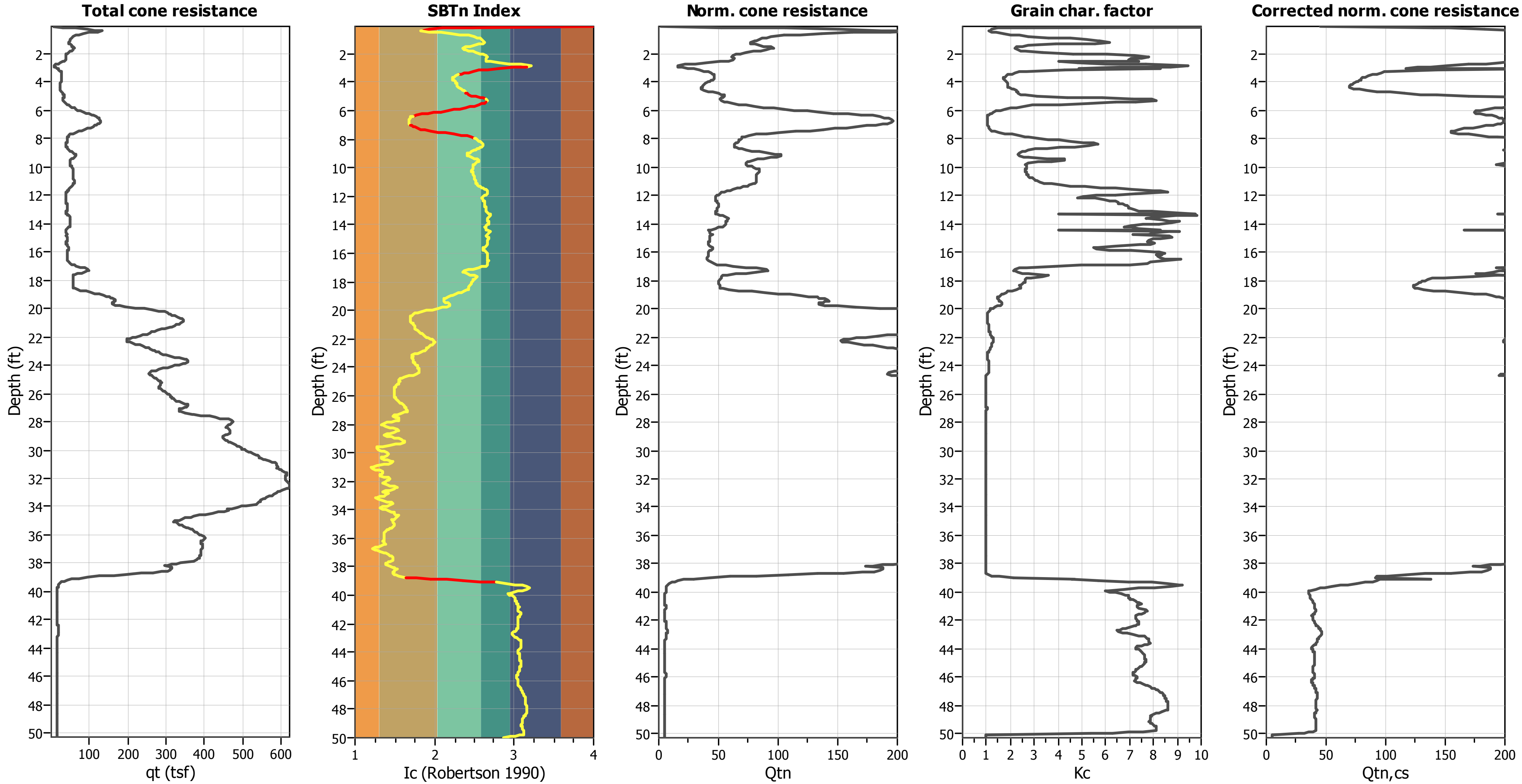
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

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

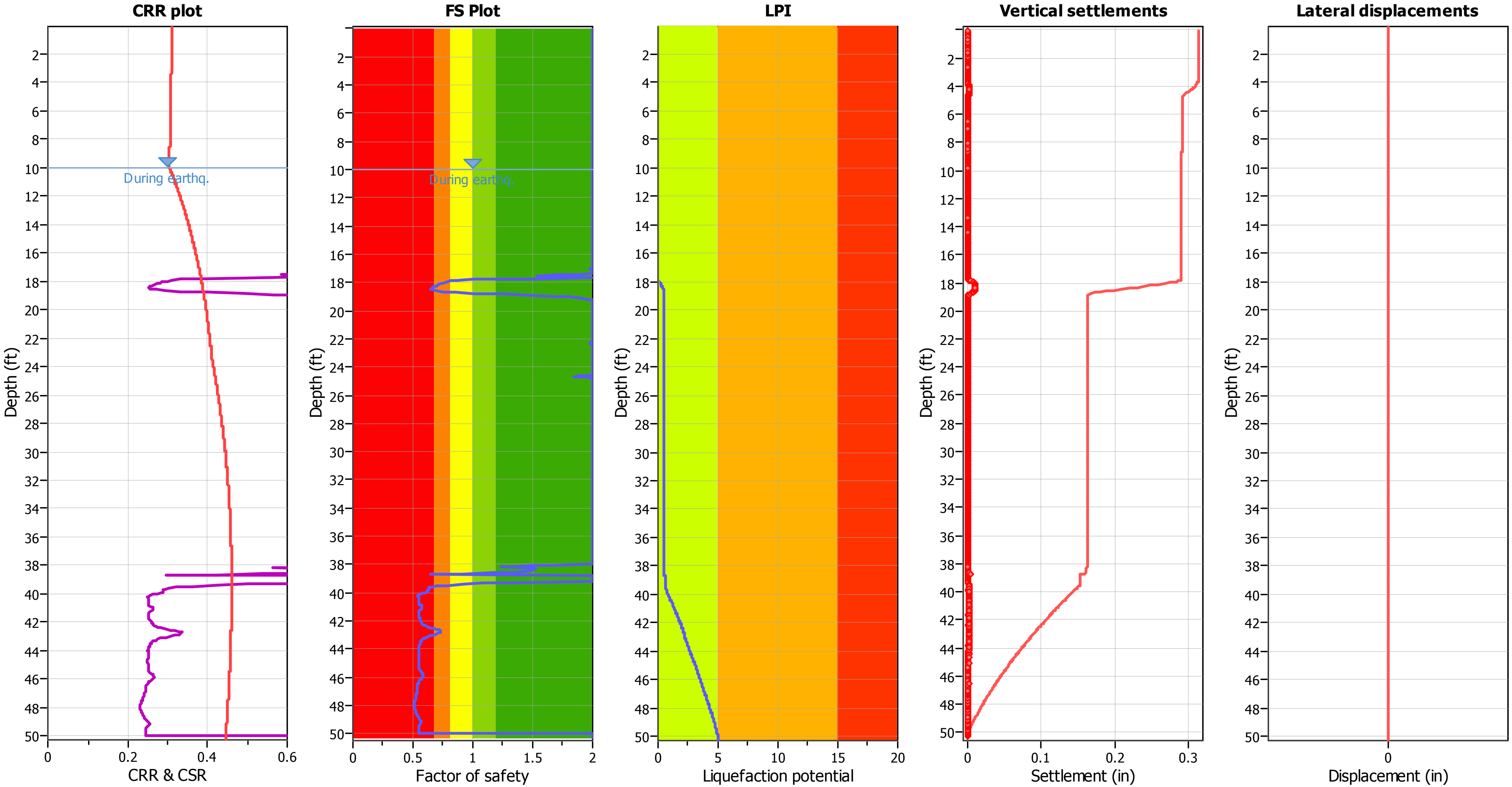
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 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

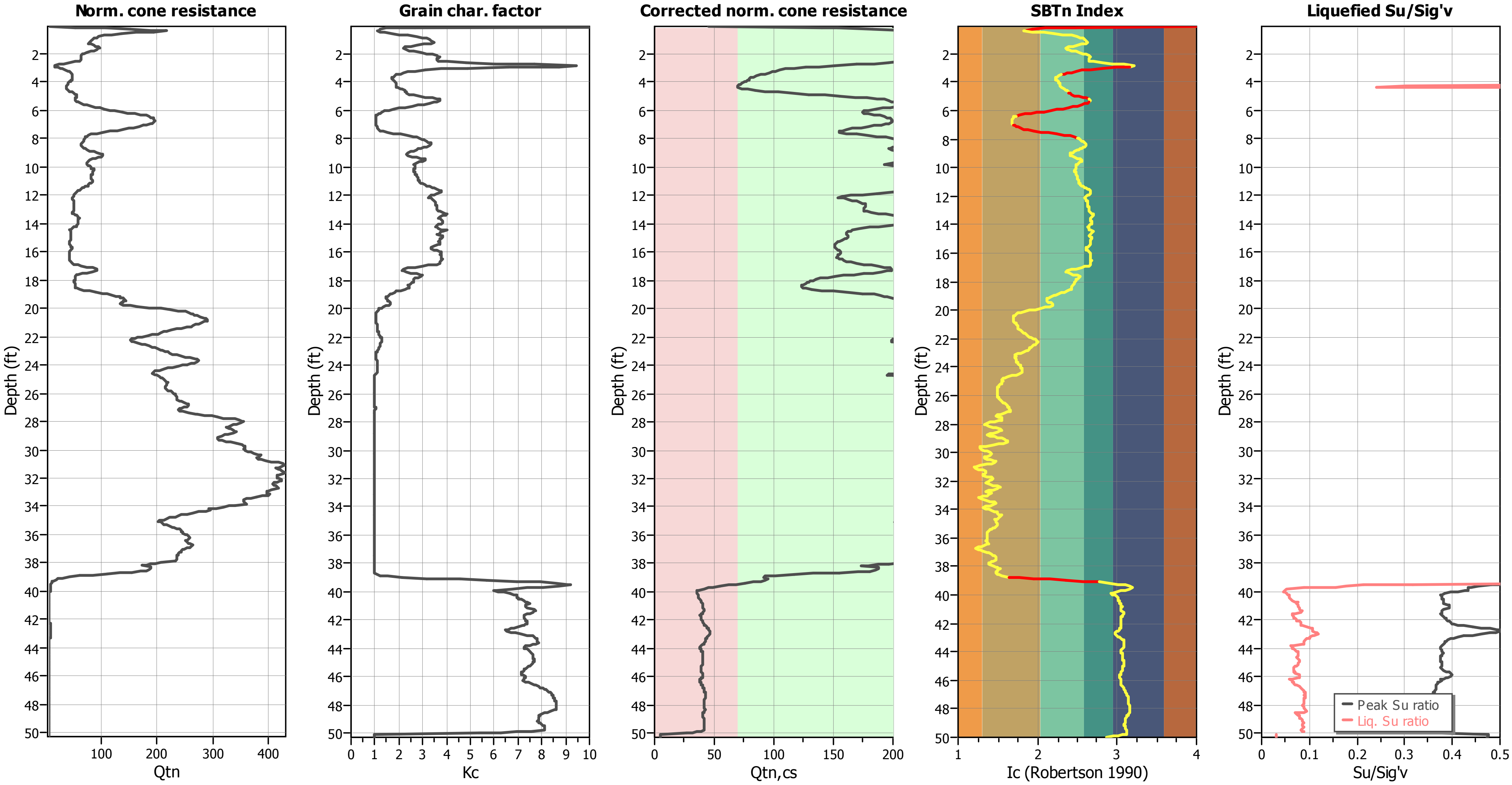
- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots

Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	5	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:	6.82	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.61	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	40.00 ft	Fill height:	N/A	Limit depth:	50.00 ft

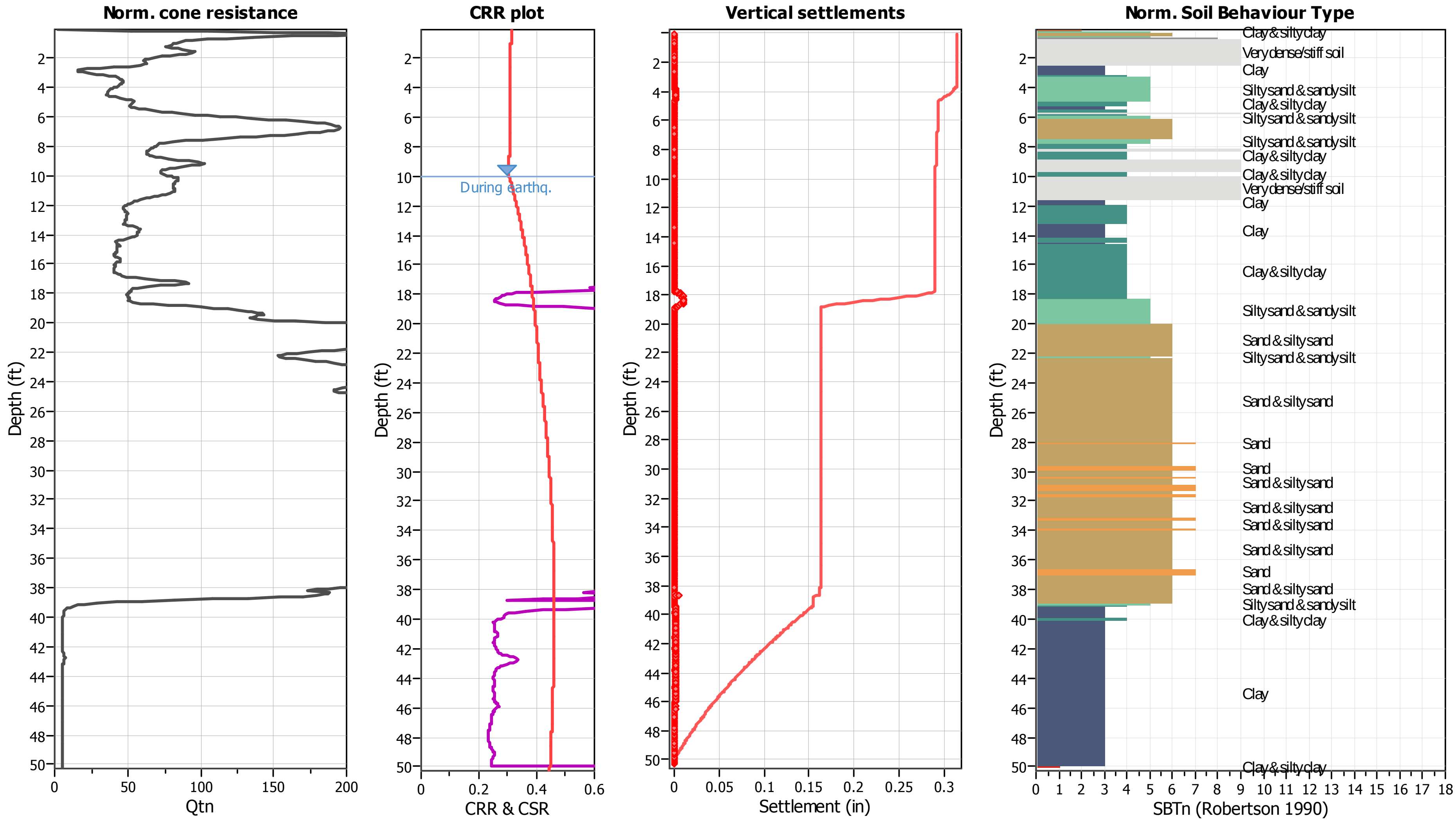
Transition layer No	Number of points	Depth	SBT_n number	SBT_n description

Kling Consulting Group, Inc.
18008 Sky Park Circle, Suite 250
Irvine, CA 92614
www.klingconsultinggroup.com

Project:

Location:

CPT: CPT-3
Total depth: 50.27 ft



Analysis method:	Robertson (2009)	G.W.T. (in-situ):	40.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.82	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		

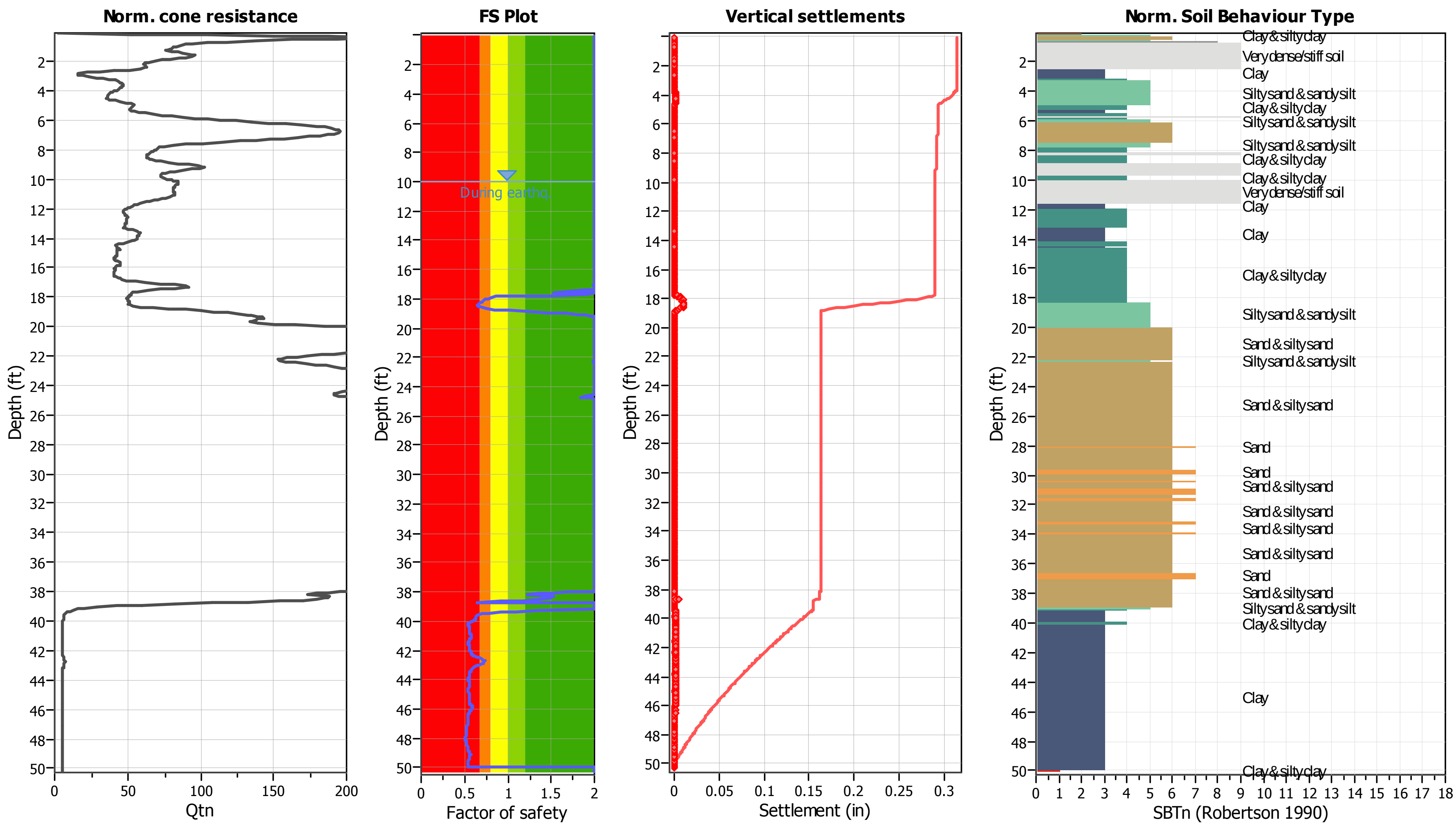
Kling Consulting Group, Inc.
18008 Sky Park Circle, Suite 250
Irvine, CA 92614
www.klingconsultinggroup.com

Project:

Location:

CPT: CPT-3

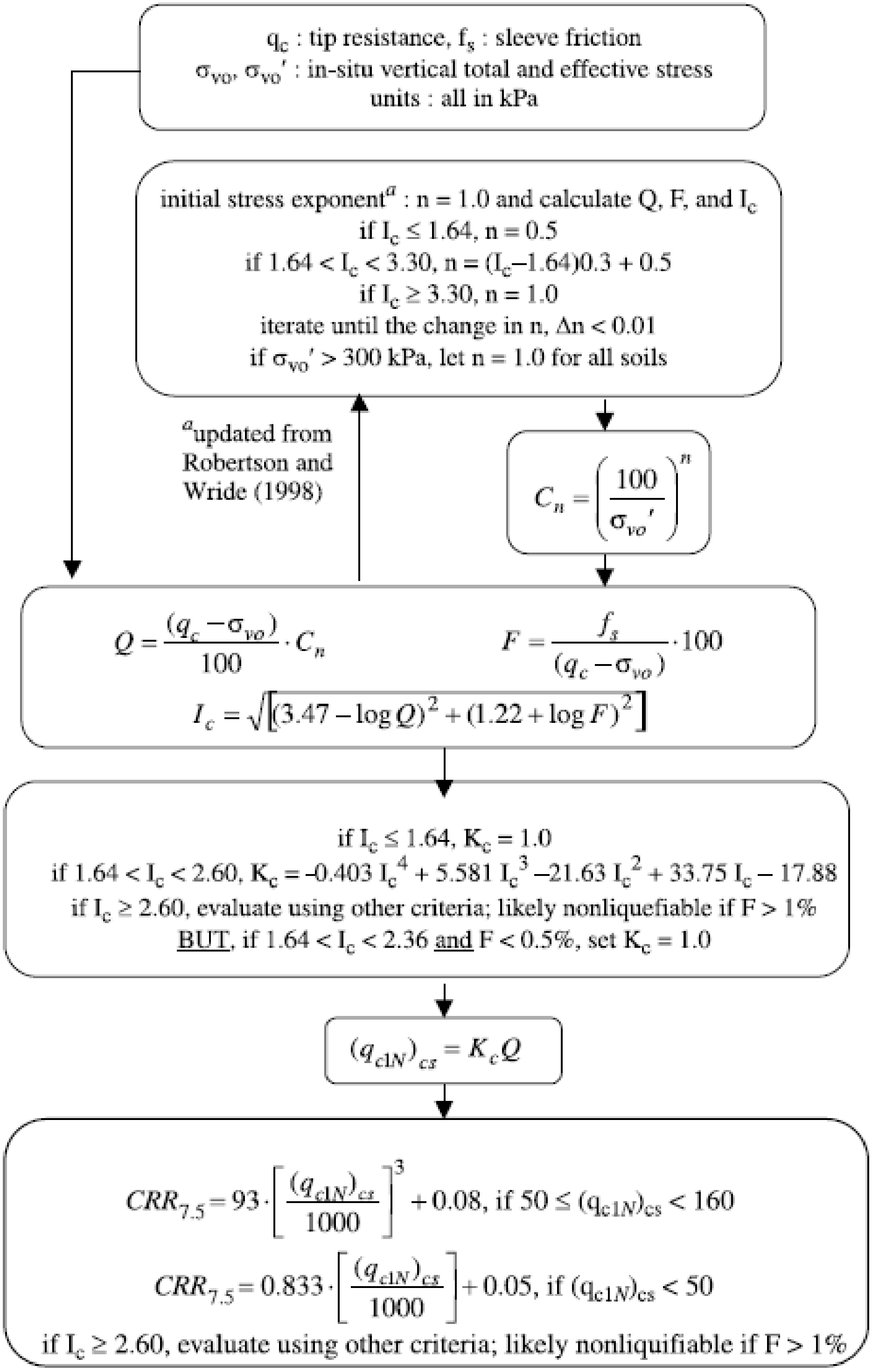
Total depth: 50.27 ft



Analysis method:	Robertson (2009)	G.W.T. (in-situ):	40.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	5	Fill weight:	N/A	Limit depth:	50.00 ft
Earthquake magnitude M_w:	6.82	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		

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

APPENDIX D

HARDSCAPE RECOMMENDATIONS

KCG KLING Consulting Group, Inc.

HARDSCAPE RECOMMENDATIONS FOR EXPANSIVE SOILS (COMMERCIAL/INDUSTRIAL BUILDING)[4]

Description	Minimum Concrete Thickness (Inches)	Subgrade Pre-Soaking Depth	Reinforcement [(1)]	Cutoff Barrier or Edge Thickness	Joint [(2)] Spacing (Max)	Base
Common Sidewalks - Isolated EI<21 EI 21-50 EI 51-90 EI 91-130 EI>130	 4 4 4 5 5	 Optimum to 12" 120% of/or 5% over optimum (whichever is greater) to 12" 120% of/or 5% over optimum (whichever is greater) to 18" 120% of/or 5% over optimum (whichever is greater) to 24" 130% of/or 5% over optimum (whichever is greater) to 24"	N.R.	N.R.	5-10 Feet 5-10 Feet 5-10 Feet 6 feet 6 feet	N.R.
Common Sidewalks - Not Isolated (adjacent to curbs or structures) EI<21 EI 21-50 EI 51-90 EI 91-130 EI>130	 4 4 4 5 5	 Optimum to 12" 120% of/or 5% over optimum (whichever is greater) to 12" 120% of/or 5% over optimum (whichever is greater) to 18" 120% of/or 5% over optimum (whichever is greater) to 24" 120% of/or 5% over optimum (whichever is greater) to 24"	Dowel into curbs and entries with #4 Re-bar at 24" O.C.	N.R.	5-10 Feet 5-10 Feet 5-10 Feet 6 feet 6 feet	N.R.
Enhanced or Decorative Concrete (where higher degree of crack control is desired) E<21 EI 21-50 EI 51-90 EI 91-130 EI>130	 5 5 5 6 6	 Optimum to 12" 120% of/or 5% over optimum (whichever is greater) to 12" 120% of/or 5% over optimum (whichever is greater) to 18" 120% of/or 5% over optimum (whichever is greater) to 24" 120% of/or 5% over optimum (whichever is greater) to 24"	6x6 – W1.4xW1.4 Mesh 6x6 – W2.9xW2.9 Mesh #3 re-bar @ 18" O.C., E.W. #3 re-bar @ 12" O.C., E.W. #4 re-bar @ 12" O.C., E.W.	12" thick x 12" wide 12" thick x 12" wide 12" thick x 12" wide 12" thick x 12" wide 12" thick x 12" wide	5-10 Feet 5-10 Feet 5-10 Feet 6 feet 6 feet	N.R.
Curb and Gutter	C.S.	Scarify 6"/Pre-Moisten	N.R.	N.R.	10 Feet	N.R.
General Concrete Paving [3]	7	N.R.	N.R.	12"x12" where adjacent to landscape	10 Feet	6"
Trash Enclosure/Loading Bay [3]	8	N.R.	N.R.	12"x12" where adjacent to landscape	10 Feet	6"

N.R. = Not Recommended
C.S. = City/County Standard
O.C. = On Center
E.W. = Each Way

General Notes:
(A) All concrete thickness should be "full"
(B) Square concrete panels when possible
(C) Maintain positive drainage from concrete flatwork
(D) All slab reinforcement should be placed at mid-height of slab
(E) The above recommendations are intended to mitigate expansive soils independent of other design considerations. The recommendations of the structural engineer and/or architect should also be incorporated into the final design.

Footnotes:
(1) Reinforcement to extend into cutoff barrier in thickened edge.
(2) Joint at curves or angle points.
(3) The above concrete paving recommendations are for planning purposes only.
An actual pavement design should be generated based on concrete strength, and frequency and magnitude of anticipated axle loads.
(4) The above recommendations are intended to mitigate expansive soils independent of other design considerations.
The recommendations of the structural engineer and/or architect should also be incorporated into the final design.

APPENDIX E

ASFE INSERT

Important Information About Your
Geotechnical Engineering Report

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

The following information is provided to help you manage your risks.

Geotechnical 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 civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one - not even you* - should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from alight industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- 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. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ-sometimes significantly from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of 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 equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in-this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely on Your ASFE-Member Geotechnical Engineer For Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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