APPENDIX F

Geotechnical Report

GEOTECHNICAL INVESTIGATION, CONCORDIA AT LOS ARBOLITOS APN: 158-301-46-00 OCEANSIDE, CALIFORNIA

Prepared for:

Concordia Homes
380 Stevens Avenue, Suite 307
Solana Beach, California 92075

Project No. 12807.002

October 16, 2020

Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY

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Concordia Homes
380 Stevens Avenue, Suite 307
Solana Beach, California 92075

Attention: Mr. Jeb Hall

Subject: Geotechnical Investigation
Concordia at Los Arbolitos, APN: 158-301-46-00
Aspen Street, Oceanside, California

In accordance with your request and authorization, we have conducted a geotechnical investigation of the property for the design and construction of the proposed residential development project.

Based on the results of our study, it is our professional opinion that the site is suitable to receive the proposed improvements. The accompanying report presents a summary of our investigation and provides geotechnical conclusions and recommendations relative to the proposed site development.

If you have any questions regarding our report, please do not hesitate to contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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1.0 INTRODUCTION

We recommend that all individuals utilizing this report read the preceding information sheet prepared by ASFE (the Association of Engineering Firms Practicing in the Geosciences) and the Limitations, Section 7.0, located at the end of this report.

1.1 Purpose and Scope

This report presents the results of our geotechnical investigation for the site located in the City of Oceanside, California (Figure 1). The intent of this report is to provide specific geotechnical conclusions and recommendations for the currently proposed project.

1.2 Site Location and Description

The site is located east of San Luis Rey drainage in Oceanside, California (Figure 1). The site is currently largely undeveloped, with isolated culverts and dirt pedestrian pathways throughout.

The site is roughly rectangular shaped with the long axis oriented north-south and encompassing a footprint of approximately 7.4 acres. Specifically, the property is bounded on the north and west by the San Luis Rey River, and on the south and east by existing residential properties. Site elevations vary between 48 feet above mean sea level (msl) and 50 feet msl with topography across the site gently sloping from the northeast to the southwest.

Site Latitude and Longitude
33.236379º N
117.339162º W

1.3 Proposed Development

While precise grading plans were not available for our review, we understand that the project will consist of construction of a single familymulti-building residential project. Specifically, construction is currently proposed to consist of a 53 single family units, associated utilities, roadways, landscape and hardscape. We also understand Pala Road will be extended west ward up to San Luis Rey River as part of the development.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Current Site Investigation

Our subsurface exploration of the site was performed on July 21 and September 18, 2020, and consisting of excavating twelve (12) exploratory test pits and (4) cone penetration tests (CTPs). The exploratory test pits (TP-1 through TP-12) were advanced with rubber tire backhoe to characterize the onsite soils, including those likely to be encountered at and below the proposed foundation elevations for this project. The four Cone Penetration Tests (CPT's) were also advanced to further characterize the onsite soils for the purpose of evaluating liquefaction potential. A geologist from our firm visually logged the soil types encountered in accordance to ASTM D2488. Select soil samples were obtained for laboratory testing. The approximate locations of the test pits and CPTs are presented on the Geotechnical Exploration Map (Figure 2) and the test pit logs and CPT profiles are presented in Appendix B.

2.2 San Luis Rey Project

As part of this study, we performed a limited review of the various As-built Plans related to the San Luis Rey River flood control project by the United States Army Corps of Engineers (1994, 1999). Improvements related to the project consisted of construction of a grouted stone lined levee embankment, including placement of completed fill, aggregate base and asphaltic concrete pavement. The levee construction consisted of removing upper 5 feet of alluvial material and placing compacted fill at 92% relative compaction for levee 2:1 fill slopes.

2.3 Laboratory Testing

Laboratory testing was performed on selected soil samples to evaluate particle size and distribution, maximum bulk density and optimum moisture content, and expansion index. A discussion of the laboratory tests performed and a summary of the laboratory test results are presented in Appendix C.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Regional Geologic Setting

The project area is situated in the Peninsular Ranges Geomorphic Province of California. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California, and varies in width from approximately 30 to 100 miles (Norris and Webb, 1990). The province is characterized by mountainous terrain on the east composed mostly of Mesozoic igneous and metamorphic rocks, and relatively low-lying coastal terraces to the west underlain by late Cretaceous-age, Tertiary-age, and Quaternary-age sedimentary units. Most of the coastal region of the County of San Diego, including the site, occur within this coastal region and are underlain by sedimentary units. More locally, the site generally consists of subdued landforms underlain by sedimentary bedrock.

3.2 Site-Specific Geology

Based on our subsurface exploration and review of geologic literature and maps (Appendix A), the geologic units underlying the site consist of localized undocumented artificial fill overlying surficial alluvial floodplain deposits (Quaternary-aged Young Alluvial Floodplain Deposits) (Figure 3). A brief description of the geologic units encountered on the site is presented below.

3.2.1 Undocumented Fill – (Afu)

During our subsurface exploration, undocumented artificial fill soil on the order of up to approximately 3 feet was encountered at the exploration locations. The fill was apparently placed during the site's initial construction (possibly in association with levee construction) and deeper fills may exist that were not observed during our exploration. An as-graded report was not available for our review, and it is assumed that no engineering observations of these fill soils were provided at the time of grading. As encountered, the fill soils generally consisted of light gray, dry to moist, loose to medium dense, silty sand with gravels. Older fill to the west of the site were placed during construction of the San Luis Rey River Flood Control project. Based on our review, these fills were properly compacted up to the top of the levee.

3.2.2 Quaternary Young Alluvial Deposits (Qya)

Quaternary-aged Young Alluvial Deposits were observed to underlie the site. As encountered, young alluvial flood-plain deposits underlay the fill and consist of materials that range from silts and clays to sands and gravels. The materials are generally unconsolidated, loose to medium dense and soft to firm. The young alluvium generally consists of interbedded layers of medium to dark gray, friable, loose to medium dense, sandy silts to silty sands and silty clays

3.3 Surface and Ground Water

No indication of surface water or evidence of surface ponding was encountered within the limits of the proposed development during our geotechnical investigation performed at the site. In addition, surface water may drain as sheet flow across the site during rainy periods.

Ground water was not observed in the recent test pit explorations at the site. It should however be noted that perched ground water levels may develop and fluctuate during periods of precipitation.

Based on our experience in the site area, the recent and previous subsurface investigations along with measurements of two previously completed piezometers at the site, we anticipate the static ground water to be at a depth of roughly 17 feet below the existing ground surface (bgs), or an elevation of 31 feet msl. Therefore, we anticipate the lowest site foundations and utilities will be above the existing static ground water table at the site.

3.4 Engineering Characteristics of On-site Soils

Based on the results of our laboratory testing of representative on-site soils, and our professional experience on similar sites with similar soils conditions, the engineering characteristics of the on-site soils are discussed below.

3.4.1 Expansion Potential

Based on our visual observations performed during our site reconnaissance, subsurface investigation, laboratory testing, and similar projects in the site vicinity, we anticipate the near surface soils to have a generally very low to low expansion potential. However, soils with greater expansion potential may be encountered during grading and additional testing may be warranted. Nevertheless, expansive soils are not anticipated to impact the proposed site development.

3.4.2 Compressible Soils

Based on the results of our subsurface explorations at the site, and review of other projects in the area, we expect that the upper 8 feet of the site is underlain by undocumented fill or alluvial deposits which are considered compressible. These soils are not considered suitable for support of foundation loads in their present condition. Recommendations for remedial grading of these soils are provided in Section 6 of this report.

3.4.3 Soil Corrosivity

A preliminary corrosive soil screening for the on-site materials was completed to evaluate their potential effect on concrete and ferrous metals. The corrosion potential was evaluated using the results of laboratory testing on one representative soil sample obtained during our subsurface evaluation.

Laboratory testing was performed to evaluate pH, minimum electrical resistivity, and chloride and soluble sulfate content. The sample tested had measured pH value of 7.7, and a measured minimum electrical resistivity of 4,400 ohm-cm. Test results also indicated that the sample had a chloride content of zero parts per million (ppm), and a soluble sulfate content of less than 0.0150 percent by weight in soil.

3.4.4 Excavation Characteristics

The site is underlain by undocumented fill and Quaternary Young Alluvial Deposits generally consisting of silty sands to sandy silts with trace gravels. With regards to the proposed project, it is anticipated these on-site soils can be excavated with conventional heavy-duty construction equipment. Oversize cobble material, if encountered, should be placed in non-structural areas or hauled off-site. Friable sands should be anticipated within the alluvial material and may require special consideration during utility excavations.

4.0 SEISMIC AND GEOLOGIC HAZARDS

4.1 Regional Tectonic Setting and Seismicity

The site is considered to lie within a seismically active region, as can all of Southern California. During the late Pliocene, several new faults developed in Southern California, creating a new tectonic regime superposed on the flat-lying section of Tertiary and late Cretaceous rocks in the San Diego region.

The principal known onshore faults which collectively account for the majority of seismic hazard in southernmost California are the San Andreas, San Jacinto, Elsinore, Imperial and Rose Canyon faults. The balance of seismic hazard is taken by the offshore zone of faults which include the Coronado Bank, San Diego Trough, and San Clemente faults off of the San Diego. Most of the offshore faults coalesce south of the international border, where they come onshore as the Agua Blanca fault which transects the Baja California peninsula south of Ensenada (Jennings, 2010).

The primary seismic hazard for San Diego is the Rose Canyon fault zone which is located approximately 7.5 miles west of the site and is the 'active' seismogenic fault considered having the most significant effect at the site from a design standpoint.

4.2 Local Faulting

Our review of available geologic literature (Appendix A) indicates that there are no known active or potentially active faults transecting, or projecting toward the site. The nearest active fault is the Rose Canyon fault zone located approximately 7.5 miles west of the site.

4.3 Seismic Hazards

Severe ground shaking is most likely to occur during an earthquake on one of the regional active faults in Southern California that are mentioned above. The effect of seismic shaking may be mitigated by adhering to the California Building Code or state-of-the-art seismic design parameters of the Structural Engineers Association of California.

4.3.1 Shallow Ground Rupture

As previously discussed, no faults are mapped transecting or projecting toward the site. Therefore, surface rupture hazard due to faulting is considered very low. Ground cracking due to shaking from a seismic event is not considered a significant hazard either, since the site is not located near slopes.

4.3.2 Mapped Seismic Hazard Zones

The site is not located within a State mapped Earthquake Fault Zone (EFZ). However, the site is mapped within a County of San Diego liquefaction zone. The results of our analysis regarding secondary seismic hazards at the site are summarized in Section 4.4 below.

4.3.3 Site Class

The onsite soils are considered to be liquefiable under a California Building Code design level earthquake. Liquefiable sites are to be classified as Site Class F, requiring a site-specific response analysis. However, per Section 20.3.1 of ASCE 7-16, for structures having fundamental periods of vibration less than 0.5s, Site Class may be determined in accordance to Section 20.3. It is understood that the proposed structures will have a fundamental period less than 0.5 s; therefore, we have utilized a Site Class D for determining spectral acceleration parameters. If it is determined by the structural engineer that the proposed structure has a fundamental period of vibration greater than 0.5 s, a site-specific response analysis will be required.

4.3.4 Building Code Mapped Spectral Acceleration Parameters

The effect of seismic shaking may be mitigated by adhering to the California Building Code and state-of-the-art seismic design practices of the Structural Engineers Association of California. Provided below in Table 1 are the spectral acceleration parameters for the project determined in accordance with the 2019 CBC (CBSC, 2019) and the SEA/OSHPD Web Application. Since the site has an S_1 value greater than 0.2g a ground motion hazard analysis was also performed according to ASCE 7-16 Section 11.4.8.

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Table 1 2019 CBC Mapped Spectral Acceleration Parameters			
Site Class	D		
Site Coefficients	F_a	=	1.122
	F_v	=	null
Mapped MCE Spectral Accelerations	S_S	=	0.946g
	S_1	=	0.35g
Site Modified MCE Spectral Accelerations	S_{MS}	=	1.061g
	S_{M1}	=	null
Design Spectral Accelerations	S_{DS}	=	0.707g
	S_{D1}	=	null
Transitional Period	F_v	=	1.950g
	S_{M1*}	=	0.683g
	S_{D1*}	=	0.455g
	$T_s = S_{D1}/S_{DS}$	=	0.628s

*Site-specific ground motion hazard analysis is required for determination of S_{M1} and S_{D1} for use in seismic design. Values of S_{M1} and S_{D1} presented are only for the purposes of determining T_S as per Supplement 1 to ASCE 7-16 (ASCE, 2018).

Utilizing ASCE Standard 7-16, in accordance with Sections 11.8.2 and 11.8.3, the following additional parameters for the peak horizontal ground acceleration are associated with the Geometric Mean Maximum Considered Earthquake (MCE_G). The mapped MCE_G peak ground acceleration (PGA) is 0.41g for the site. For a Site Class D, the F_{pga} is 1.19 and the mapped peak ground acceleration adjusted for Site Class effects (PGA_M) is 0.488g for the site.

Since the mapped spectral response at 1-second period is less than 0.75g, then all structures subject to the criteria in Section 1613.2.5 of the 2019 CBC are assigned Seismic Design Category D.

4.4 Secondary Seismic Hazards

In general, secondary seismic hazards can include soil liquefaction, seismically-induced settlement, lateral displacement, surface manifestations of liquefaction, landsliding, seiches, and tsunamis. The potential for secondary seismic hazards at the subject site is discussed below.

4.4.1 Liquefaction and Dynamic Settlement

Liquefaction and dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Granular soils tend to densify when subjected to shear strains induced by ground shaking during earthquakes. Research and historical data indicate that loose granular soils underlain by a near surface ground water table are most susceptible to liquefaction, while the clay-rich materials are not susceptible to liquefaction. Liquefaction is characterized by a loss of shear strength in the affected soil layer, thereby causing the soil to behave as a viscous liquid. This effect may be manifested at the ground surface by settlement and, possibly, sand boils where insufficient confining overburden is present over liquefied layers. Where sloping ground conditions are present, liquefaction-induced lateral instability can result.

In our preliminary liquefaction analysis utilizing the computer program CLiq Version 3.0.3.2, we used a deaggregation of the Maximum Considered Earthquake event with a magnitude M6.9 (i.e., associated with the Design Earthquake Ground Motion). The peak horizontal ground acceleration associated with the Maximum Considered Earthquake (MCE) Ground Motion is 0.49g. The MCE was obtained utilizing USGS Unified Hazard Tool. Based on the results of the liquefaction analysis, several discontinuous and variable thickness layers of saturated alluvial materials are located between a depth of approximately 17 to 52 feet bgs. As encountered in the CPT explorations, these layers are considered susceptible to liquefaction at the design earthquake ground motion.

Total dynamic settlement at the site as a result of the Design Earthquake Ground Motion is roughly estimated at between approximately 1.3 to 3.1 inches. Differential dynamic settlement at the site is anticipated to be on the order of 1.5 inches or less within 50 feet considering the depth and discontinuous nature of the liquefied zones. Summary plots showing idealized profile, relevant CPT data, calculated cyclic stress and resistance ratio, factor of safety, and liquefaction-induced settlement are provided in Appendix D.

A summary plot showing idealized profile, relevant CPT data, calculated cyclic stress and resistance ratio, factor of safety, and liquefaction-induced settlement is provided in Appendix D.

4.4.2 Lateral Spread

Empirical relationships have been derived (Youd et al., 1999) to estimate the magnitude of lateral spread due to liquefaction. These relationships include parameters such as earthquake magnitude, distance of the earthquake from the site, slope height and angle, the thickness of liquefiable soil, and gradation characteristics of the soil.

The susceptibility to earthquake-induced lateral spread is considered to be low for the site because of the generally discontinuous nature of the underlying liquefiable layers, construction method of the fortified levee at the San Luis Rey River, and the nearest distance to an open slope face (approximately 150 feet to the San Luis Rey river).

4.4.3 Tsunamis and Seiches

Based on a site elevation of approximately 50 feet msl, the distance of the site from the Pacific coastline, and the CGS Tsunami Inundation Map of the area (CalEMA, 2009) the potential for flood damage to occur at the site from a tsunami or seiche is considered nil.

4.5 Landslides

Several formations within the San Diego region are particularly prone to landsliding. These formations generally have high clay content and mobilize when they become saturated with water. Other factors, such as steeply dipping bedding that project out of the face of the slope and/or the presence of fracture planes, will also increase the potential for landsliding.

No landslides or indications of deep-seated landsliding were indicated at the site during our field exploration or our review of available geologic literature, topographic maps, and stereoscopic aerial photographs. Furthermore, our field reconnaissance and the local geologic maps indicate the site is generally underlain by favorable oriented geologic structure, consisting of massively bedded silty to clayey sands and sandy to silty clays, and flat lying topographic conditions. Therefore, the potential for significant landslides or large-scale slope instability at the site is considered nil.

4.6 Flood Hazard

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2012); the majority of the site is located within a Zone X floodplain, and the southwestern portion of the site is located in Zone AO (100-year) floodplain, see Figure 5. However, based on this review and our site reconnaissance, the potential for flooding of the site is considered low since the adjacent portion of the San Luis Rey River has been channelized.

5.0 CONCLUSIONS

Based on the results of our geotechnical investigation of the site, it is our opinion that the proposed development is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications.

- Generally loose surficial soils consisting of fill and alluvium having depths of up to approximately 8 feet locally underlie the site and are considered compressible. Therefore, in their present condition, these soils are not considered suitable for the support of structural loads or the support of engineered fill soils and site improvements. Section 6.1.2 of this report provides specific recommendations regarding mitigation of these soil materials.
- Based on the results of our subsurface explorations and available geologic references, ground water is not anticipated to be a constraint during site construction, and we do not anticipate that temporary dewatering will be necessary. Ground water was encountered at an elevation of approximately 17 feet below the ground surface across the site (elevation of 31 feet msl).
- The underlying alluvial deposits are subject to localized liquefaction or seismic settlement. Differential dynamic settlement at the site is anticipated to be on the order of 1.5 inches or less across 50 feet considering the depth and discontinuous nature of the liquefied zones.
- Based on the results of our subsurface investigation, we anticipate that the onsite materials should be generally rippable with conventional heavy-duty earthwork equipment. Although, localized areas of gravels were encountered during our exploration, the existing onsite soils are suitable for reuse as engineered fill provided they are relatively free of organic material, debris, and rock fragments larger than 6 inches in maximum dimension. Loose caving friable sand should be anticipated during site excavations. In addition, unknown items such as buried concrete and debris left from previous fill placement should be anticipated.
- Based on visual classification, materials derived from the on-site soil materials possess a very low to medium expansion potential, although locally more expansive materials may be encountered.

- Although Leighton does not practice corrosion engineering, laboratory test results indicate the soils present on the site have a negligible potential for sulfate attack on normal concrete. The onsite soils are considered to be moderately corrosive to buried uncoated ferrous metals.

6.0 RECOMMENDATIONS

6.1 Earthwork

We anticipate that earthwork at the site will consist of site preparation, shallow excavation and fill operations. We recommend that earthwork on the site be performed in accordance with the following recommendations and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations supersede those in Appendix E.

6.1.1 Site Preparation

Prior to grading, all areas to receive structural fill, engineered structures, or hardscape should be stripped of vegetation and cleared of surface and subsurface obstructions, including any existing debris and undocumented fill, loose, compressible, or unsuitable soils. Removed vegetation and debris should be properly disposed off site. All areas to receive fill and/or other surface improvements should be scarified to a minimum depth of 8 inches, brought to optimum or above-optimum moisture conditions, and recompacted to at least 90 percent relative compaction based on ASTM Test Method D1557.

6.1.2 Removal of Compressible Soils

Potentially compressible undocumented fill and alluvial soils at the site may settle as a result of wetting or settle under the surcharge of engineered fill and/or structural loads supported on shallow foundations. These soils should be removed to undisturbed medium dense alluvium and replaced as moisture conditioned engineered fill. In general, removal depths will extend to 8 feet below the existing ground surface across the site. Additionally, removal depths should extend to a minimum of 3 feet below bottom of foundation footings or a depth equal to 2 times the foundation width, whichever is greater. The lateral limits of the removal bottom should extend at least 10 feet beyond the foundation limits where possible. The bottom of all removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

In areas of proposed pavements, hardscape and landscaping features, removals should be performed to a depth of 4 feet below proposed subgrade elevation and extend at least 4 feet beyond the limits of the proposed improvements. The bottom of all removals should be evaluated by a Certified Engineering Geologist to confirm conditions are as anticipated.

In general, the soil that is removed may be reused and placed as engineered fill provided the material is moisture conditioned to above optimum moisture content, and then recompacted prior to additional fill placement or construction. Soil with an expansion index greater than 50 should not be used within 5 feet of finish grade in the building pad. The actual depth and extent of the required removals should be confirmed during grading operations by the geotechnical consultant.

6.1.3 Cut/Fill Transition Mitigation

Although final grading plans were not available at the time of drafting this report, the proposed site is situated in an area where generally flat topography is present. Therefore, we do not anticipate mitigation for cut/fill transitions will be necessary. However, should such conditions occur, to mitigate the impact of the underlying cut/fill transition condition beneath possible structures that are planned across existing or future cut/fill transitions, the cut portion should be over-excavated to at least 3 feet below the bottoms of proposed building foundations. The over-excavated material should be replaced with properly compacted fill. The overexcavation should laterally extend at least 5 feet beyond the building pad area and all associated settlement-sensitive structures. As an alternative to overexcavation of the cut portions, the pad grade may be raised following surficial soil preparation, to achieve similar results.

6.1.4 Excavations and Oversize Material

Excavations of the onsite materials may generally be accomplished with conventional heavy-duty earthwork equipment. Due to the generally friable nature of the fill and alluvium, temporary excavations, such as utility trenches with vertical sides, may slough over time.

In accordance with OSHA requirements, excavations deeper than 5 feet should be shored or be laid back if workers are to enter such excavations. Temporary sloping gradients should be determined in the field by a "competent person" as defined by OSHA. For preliminary planning, sloping of fill soils at 1:1 (horizontal to vertical) may be assumed. Excavations supporting structures or greater than 20 feet in height will require an alternative sloping plan or shoring plan prepared by a California registered civil engineer.

6.1.5 Engineered Fill

In areas proposed to receive engineered fill, the existing upper 8 inches of subgrade soils should be scarified then moisture conditioned to moisture content at or above the optimum content and compacted to 90 percent or more relative to the maximum laboratory dry density, as evaluated by ASTM D 1557. Soil materials utilized as fill should be free of oversized rock, organic materials, and deleterious debris. Rocks greater than 6 inches in diameter should not be placed within 2 feet of finished grade. Fill should be moisture conditioned to at least 2 percent above the optimum moisture content and compacted to 90 percent or more relative to the maximum laboratory dry density, as evaluated by ASTM D 1557. Although the optimum lift thickness for fill soils will be dependent on the type of compaction equipment utilized, fill should generally be placed in uniform lifts not exceeding approximately 8 inches in loose thickness.

In vehicle pavement and trash enclosure areas the upper 12 inches of subgrade soils should be scarified then moisture conditioned to a moisture content above optimum content and compacted to 95 percent or more relative to the maximum laboratory dry density, as evaluated by ASTM D 1557.

Placement and compaction of fill should be performed in general accordance with current City of Oceanside grading ordinances, California Building Code, sound construction practice, these recommendations and the General Earthwork and Grading Specifications for Rough Grading presented in Appendix E.

6.1.6 Earthwork Shrinkage/Bulking

The volume change of excavated onsite materials upon recompaction as fill is expected to vary with material and location. Typically, the surficial soils vary significantly in natural and compacted density, and therefore, accurate earthwork shrinkage/bulking estimates cannot be determined. However, based on our experience, a 5 to 7 percent shrinkage factor is considered appropriate for the artificial fill and surficial alluvium at the site.

6.1.7 Import Soils

If import soils are necessary to bring the site up to the proposed grades, these soils should be granular in nature, environmentally clean, have an expansion index less than 50 (per ASTM Test Method D4829) and have a low corrosion impact to the proposed improvements. Import soils and/or the borrow site location should be evaluated by the geotechnical consultant prior to import.

6.1.8 Expansive Soils and Selective Grading

Based on our visual observations, we anticipate the onsite soil materials possess a very low to medium expansion potential. Although not anticipated, should an abundance of highly expansive materials be encountered, selective grading may need to be performed. In addition, to accommodate conventional foundation design, the upper 5 feet of materials within the building pad and 5 feet outside the limits of the building foundation should have a very low to low expansion potential (EI<50).

6.2 Foundation and Slab Considerations

At the time of drafting this report, building loads for were not known. However, based on our understanding of the project, the proposed buildings should be constructed with post-tension foundation due to the liquefaction potential. Foundations and slabs should be designed in accordance with structural considerations and the following recommendations. These recommendations assume that the soils encountered within 5 feet of pad grade have a low potential for expansion (EI<50). If more expansive materials are encountered and selective grading cannot be accomplished, revised foundation recommendations may be necessary. The foundation recommendations below assume that the all building foundations will be underlain by properly compacted engineered fill in accordance to Section 6.1.5 of this report.

6.2.1 Post-Tension Foundation Recommendations

Due to liquefaction potential at the site we recommended post-tensioned foundations. We recommend that post-tensioned foundations be designed using the geotechnical parameters presented in table below and criteria of the 2019 California Building Code and the Third Edition of Post-Tension Institute Manual. A post-tensioned foundation system designed and constructed in accordance with these recommendations is expected to be structurally adequate for the support of the buildings planned at the site provided our recommendations for surface drainage and landscaping are carried out and maintained through the design life of the project. Based on an evaluation of the depths of fill beneath the building pads, the attached Table 2 presents the recommended post-tension foundation category for residential buildings on subject site.

Table 2
Post-Tensioned Foundation Design Recommendations

Design Criteria		
Edge Moisture Variation, e_m	Center Lift:	7.0 feet
	Edge Lift:	3.7 feet
Differential Swell, y_m	Center Lift:	1.09 inches
	Edge Lift:	1.99 inches
Perimeter Footing Depth:		30 inches
Allowable Bearing Capacity		2,000 psf

The post-tensioned (PT) foundation and slab should also be designed in accordance with structural considerations. For a ribbed PT foundation, the concrete slabs section should be at least 5 inches thick. Continuous footings (ribs or thickened edges) with a minimum width of 12 inches and a minimum depth of 12 inches below lowest adjacent soil grade may be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot. For a uniform thickness “mat” PT foundation, the perimeter cut off wall should be at least 8 inches below the lowest adjacent grade. However, note that where a foundation footing or perimeter cut off wall is within 3 feet (horizontally) of adjacent drainage swales, the adjacent footing should be embedded a minimum depth of 12 inches below the swale flow line. The allowable bearing capacity may be increased by one-third for short-term loading. The slab subgrade soils should be presoaked in accordance with the recommendation presented in Table 4 prior to placement of the moisture barrier.

The slab should be underlain by a moisture barrier as discussed in Section 6.2.3 above. Note that moisture barriers can retard, but not eliminate moisture vapor movement from the underlying soils up through the slabs. We recommend that the floor covering installer test the moisture vapor flux rate prior to attempting applications of the flooring. "Breathable" floor coverings should be considered if the vapor flux rates are high. A slip-sheet or equivalent should be utilized above the concrete slab if crack-sensitive floor coverings (such as ceramic tiles, etc.) are to be placed directly on the concrete slab. Additional guidance is provided in ACI Publications 302.1R-04 Guide for Concrete Floor and Slab Construction and 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Floor Materials.

6.2.2 Foundation Setback

We recommend a minimum horizontal setback distance from retaining walls or slopes for all structural foundations, footings, and other settlement-sensitive structures as indicated on the Table 3 below. The minimum recommended setback distance from the most proximal foundation of retaining wall is equal to the height of the retaining wall. This distance is measured from the outside bottom edge of the structural footing, horizontally to the slope or retaining wall rear face, and is based on the slope or wall height. However, the foundation setback distance may be revised by the geotechnical consultant on a case-by-case basis if the geotechnical conditions are different than anticipated.

Table 3 Minimum Foundation Setback from Retaining walls	
Slope Height	Setback
less than 5 feet	5 feet
5 to 15 feet	7 feet

Please note that the soils within the structural setback area possess poor lateral stability, and improvements (such as retaining walls, sidewalks, fences, pavements, etc.) constructed within this setback area may be subject to lateral movement and/or differential settlement. Potential distress to such improvements may be mitigated by providing a deepened footing or a grade beam foundation system to support the improvement.

In addition, open or backfilled utility trenches that parallel or nearly parallel structure footings should not encroach within an imaginary 1:1 (horizontal to vertical) downward sloping line starting from the bottom edge of the footing and should also not be located closer than 18 inches from the face of the footing. Deepened footings should meet the setbacks as described above. Also, over-excavation should be accomplished such that deepening of footings to accomplish the setback will not introduce a cut/fill transition bearing condition.

Where pipes cross under footings, the footings should be specially designed. Pipe sleeves should be provided where pipes cross through footings or footing walls and sleeve clearances should provide for possible footing settlement, but not less than 1 inch around the pipe.

6.2.3 Settlement

The foundation the recommended allowable-bearing capacity is based on a maximum total and differential static settlement of 1-inch and 3/4-inch, respectively. Since settlements are a function of footing size and contact bearing pressures, some differential settlement can be expected where a large differential loading condition exists.

Differential dynamic settlement at the site is anticipated to be on the order of 1.5 inch or less within 50 feet considering the depth and discontinuous nature of the liquefied zones.

6.2.4 Moisture Conditioning

The slab subgrade soils underlying the foundation systems should be presoaked in accordance with the recommendations presented in Table 3 prior to placement of the moisture barrier and slab concrete. The subgrade soil moisture content should be checked by a representative of Leighton prior to slab construction.

Presoaking or moisture conditioning may be achieved in a number of ways. But based on our professional experience, we have found that minimizing the moisture loss on pads that has been completed (by periodic wetting to keep the upper portion of the pad from drying out) and/or berming the lot and flooding for a short period of time (days to a few weeks) are some of the more efficient ways to meet the presoaking recommendations. If flooding is performed, a couple of days to let the upper portion of the pad dry out and form a crust so equipment can be utilized should be anticipated.

Table 4
Presoaking Recommendations Based on Finish Grade Soil Expansion Potential

Expansion Potential	Presoaking Recommendations
Very Low	Near-optimum moisture content to a minimum depth of 6 inches
Low	120 percent of the optimum moisture content to a minimum depth of 12 inches below slab subgrade
Medium	130 percent of the optimum moisture content to a minimum depth of 18 inches below slab subgrade
High	130 percent of the optimum moisture content to a minimum depth of 24 inches below slab subgrade

6.3 Lateral Earth Pressures and Retaining Wall Design

Should retaining walls be added to the project, Table 5 presents the lateral earth pressure values for level or sloping backfill for walls backfilled with and bearing against fully drained soils of very low to low expansion potential (less than 50 per ASTM D4829). oils used to backfill retaining walls should be classified as one of the following types according to ASTM D 2487: GW, GP, GM, GC, SW, SP, or SM. These backfill soils should be used within horizontal distance behind the wall equal to one-half the wall height. Retaining wall footings should extend a minimum of 18 inches beneath the lowest adjacent soil grade. At these depths, footings may be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot (psf).

Table 5
Static Equivalent Fluid Weight (pcf)

Conditions	Level	2:1 Slope
Active	35	55
At-Rest	55	85
Passive	350 (Maximum of 3 ksf)	150 (sloping down)

Walls up to 10 feet in height should be designed for the applicable pressure values provided above. If conditions other than those covered herein are anticipated, the equivalent fluid pressure values should be provided on an individual case-by-case basis by the geotechnical engineer. A surcharge load for a restrained or unrestrained wall resulting from automobile traffic may be assumed to be equivalent to a uniform lateral pressure of 75 psf which is in addition to the equivalent fluid pressure given above. For other uniform surcharge loads, a uniform pressure equal to 0.35q should be applied to the wall. The wall pressures assume walls are backfilled with free draining materials and water is not allowed to accumulate behind walls. A typical drainage design is contained in Appendix E. Wall backfill should be compacted by mechanical methods to at least 90 percent relative compaction (based on ASTM D1557). If foundations are planned over the backfill, the backfill should be compacted to 95 percent. Wall footings should be designed in accordance with the foundation design recommendations and reinforced in accordance with structural considerations. For all retaining walls, we recommend a minimum horizontal distance from the outside base of the footing to daylight as outlined in Section 6.2.2.

Lateral soil resistance developed against lateral structural movement can be obtained from the passive pressure value provided above. Further, for sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. These values may be increased by one-third when considering loads of short duration including wind or seismic loads. The total resistance may be taken as the sum of the frictional and passive resistance provided that the passive portion does not exceed two-thirds of the total resistance.

To account for potential redistribution of forces during a seismic event, retaining walls providing lateral support where exterior grades on opposites sides differ by more than 6 feet fall under the requirements of 2019 CBC Section 1803.5.12 and/or ASCE 7-16 Section 15.6.1 and should also be analyzed for seismic loading. For that analysis, an additional uniform lateral seismic force of 8H should be considered for the design of the retaining walls with level backfill, where H is the height of the wall. This value should be increased by 150% for restrained walls.

6.4 Geochemical Considerations

Concrete in direct contact with soil or water that contains a high concentration of soluble sulfates can be subject to chemical deterioration commonly known as “sulfate attack.” Soluble sulfate results (Appendix C) indicated a negligible soluble sulfate content. We recommend that concrete in contact with earth materials be designed in accordance with Section 4 of ACI 318-14 (ACI, 2014). In addition, the electrical resistivity characteristics of the tested soil sample indicate a moderately corrosive site environment to ferrous materials in contact with earth materials. We recommend measures to mitigate corrosion be implemented during design and construction.

6.5 Concrete Flatwork

Concrete sidewalks and other flatwork (including construction joints) should be designed by the project civil engineer and should have a minimum thickness of 4 inches. For all concrete flatwork, the upper 12 inches of subgrade soils should be moisture conditioned to at least 2 percent above optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM Test Method D1557 prior to the concrete placement.

6.6 Preliminary Pavement Design

The pavement section design below is based on an assumed Traffic Index (TI), our visual classification of the subject site soils, and our limited laboratory testing (we have estimated an R-value of 15). The TI values were chosen based on our experience with similar projects. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the mass grading operations. Flexible pavement sections have been evaluated in general accordance with the Caltrans method for flexible pavement design. The recommended flexible pavement section for this condition is given in Table 6 below:

Table 6
Preliminary Pavement Sections

Assumed Traffic Index (TI)	Asphalt Concrete (inches)	Aggregate Base (inches)
4.5	3.0	7.0
5.0	4.0	6.0
6.0	4.0	10.0

Flexible pavements should be constructed in accordance with current Caltrans Standard Specifications. Aggregate base should comply with the Caltrans Standard Specifications of Section 26. Aggregate base should be compacted to a minimum of 95 percent relative compaction based on ASTM Method D 1557.

For areas subject to regular truck loading (i.e., trash truck apron), we recommend a full depth of Portland Cement Concrete (P.C.C.) section of 8 inches with appropriate steel reinforcement and crack-control joints as designed by the project structural engineer. We recommend that sections be as nearly square as possible. A 3,500-psi mix that produces a 550-psi modulus of rupture should be utilized.

All pavement section materials conform to and be placed in accordance with the latest revision of the California Department of Transportation Standard Specifications (Caltrans) and American Concrete Institute (ACI) codes. The upper

12 inches of subgrade soil and all aggregate base should be compacted to a relative compaction of at least 95 percent (based on ASTM Test Method D1557).

If pavement areas are adjacent to heavily watered landscape areas, we recommend some measure of moisture control be taken to prevent the subgrade soils from becoming saturated. It is recommended that the concrete curing separating the landscaping area from the pavement extend below the aggregate base to help seal the ends of the sections where heavy landscape watering may have access to the aggregate base. Concrete swales should be designed in roadway or parking areas subject to concentrated surface runoff.

6.7 Infiltration Best Management Practices

Regarding Best Management Practices (BMP) and Low Impact Development (LID) measures, we are of the opinion that infiltration basins, and other on-site storm water retention and infiltration systems can potentially create adverse perched groundwater conditions, both on-site and off-site, when not installed using proper design recommendations (such as the use of liners) and infiltration design parameters. Due to the compressible nature of the underlying artificial fill and alluvium we anticipate infiltration across the site could cause significant settlement to the proposed residential buildings, the existing residences adjacent to the site, and existing onsite sewer and gas utilities. In addition, infiltration could create groundwater mounding due to geologic variability of the alluvial material. Lateral migration of stormwater infiltration could create seepage conditions of the existing levee fill slope west and north of the site. Therefore, infiltration at the site is **not** recommended due to the reason stated above.

6.8 Control of Ground Water and Surface Waters

Surface drainage should be controlled at all times and carefully taken into consideration during precise grading, landscaping, and construction of site improvements. Positive drainage (e.g., roof gutters, downspouts, area drains, etc.) should be provided to direct surface water away from structures and improvements and towards the street or suitable drainage devices. Ponding of water adjacent to structures or pavements should be avoided. Roof gutters, downspouts, and area drains should be aligned so as to transport surface water to a minimum distance of 5 feet away from structures. The performance of structural foundations is dependent upon maintaining adequate surface drainage away from structures.

Water should be transported off the site in approved drainage devices or unobstructed swales. We recommend a minimum flow gradient for unpaved drainage within 5 feet of structures of 2 percent sloping away.

The impact of heavy irrigation or inadequate runoff gradient can create perched water conditions, resulting in seepage or shallow ground water conditions where previously none existed. Maintaining adequate surface drainage and controlled irrigation will significantly reduce the potential for nuisance-type moisture problems. To reduce differential earth movements such as heaving and shrinkage due to the change in moisture content of foundation soils, which may cause distress to a structure and improvements, moisture content of the soils surrounding the structure should be kept as relatively constant as possible. Below grade planters should not be situated adjacent to structures or pavements unless provisions for drainage such as catch basins and drains are made.

All area drain inlets should be maintained and kept clear of debris in order to function properly. In addition, landscaping should not cause any obstruction to site drainage. Rerouting of drainage patterns and/or installation of area drains should be performed, if necessary, by a qualified civil engineer or a landscape architect.

6.9 Construction Observation

The recommendations provided in this report are based on preliminary design information and subsurface conditions disclosed by widely spaced excavations. The interpolated subsurface conditions should be checked by Leighton and Associates, Inc. in the field during construction. Construction observation of all onsite excavations and field density testing of all compacted fill should be performed by a representative of this office. We recommend that all excavations be mapped by the geotechnical consultant during grading to determine if any potentially adverse geologic conditions exist at the site.

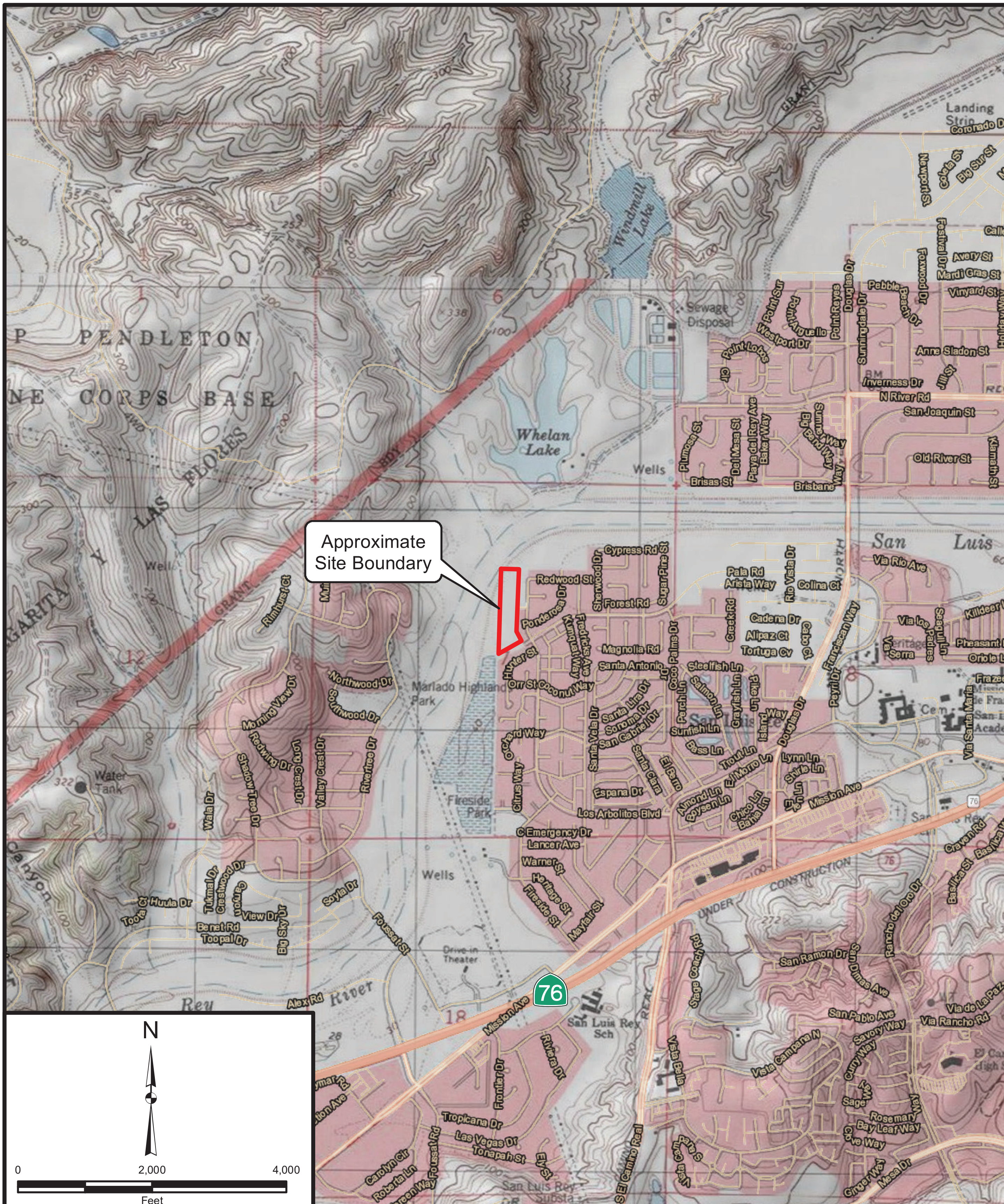
6.10 Plan Review

Final project grading and foundation plans should be reviewed by Leighton as part of the design development process to ensure that recommendations in this report are incorporated in project plans.

7.0 LIMITATIONS

The conclusions and recommendations presented in this report are based in part upon data that were obtained from a limited number of observations, site visits, excavations, samples, and tests. Such information is by necessity incomplete. The nature of many sites is such that differing geotechnical or geological conditions can occur within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if Leighton has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

Figures



Project: 12807.002	Eng/Geol: WDO/MDJ	
Scale: 1 " = 2,000 '	Date: September 2020	
Base Map: Bing Maps 2020		
Author: (mmurphy)		

SITE LOCATION MAP

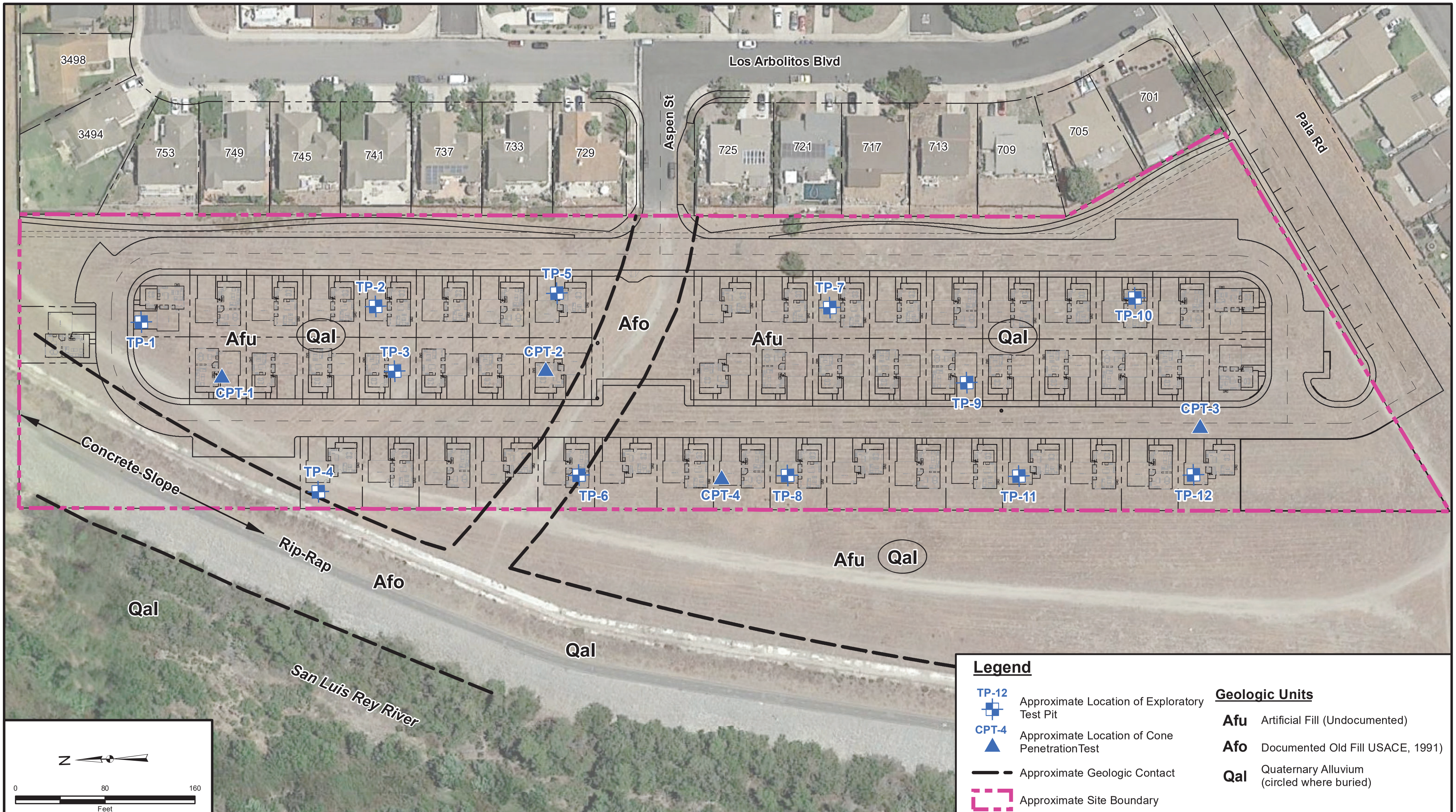
Concordia Geotechnical Investigation

Oceanside, California

Figure 1

Leighton

Map Saved as V:\Drafting\12807\002\Maps\12807-002_F01_SLM_2020-09-17.mxd on 9/18/2020 8:39:57 AM



Project: 12807.002	Eng/Geol: WDO/MDJ
Scale: 1 " = 81 '	Date: October 2020

Base Map: Google Earth and WHAT??

Author: (mmurphy)

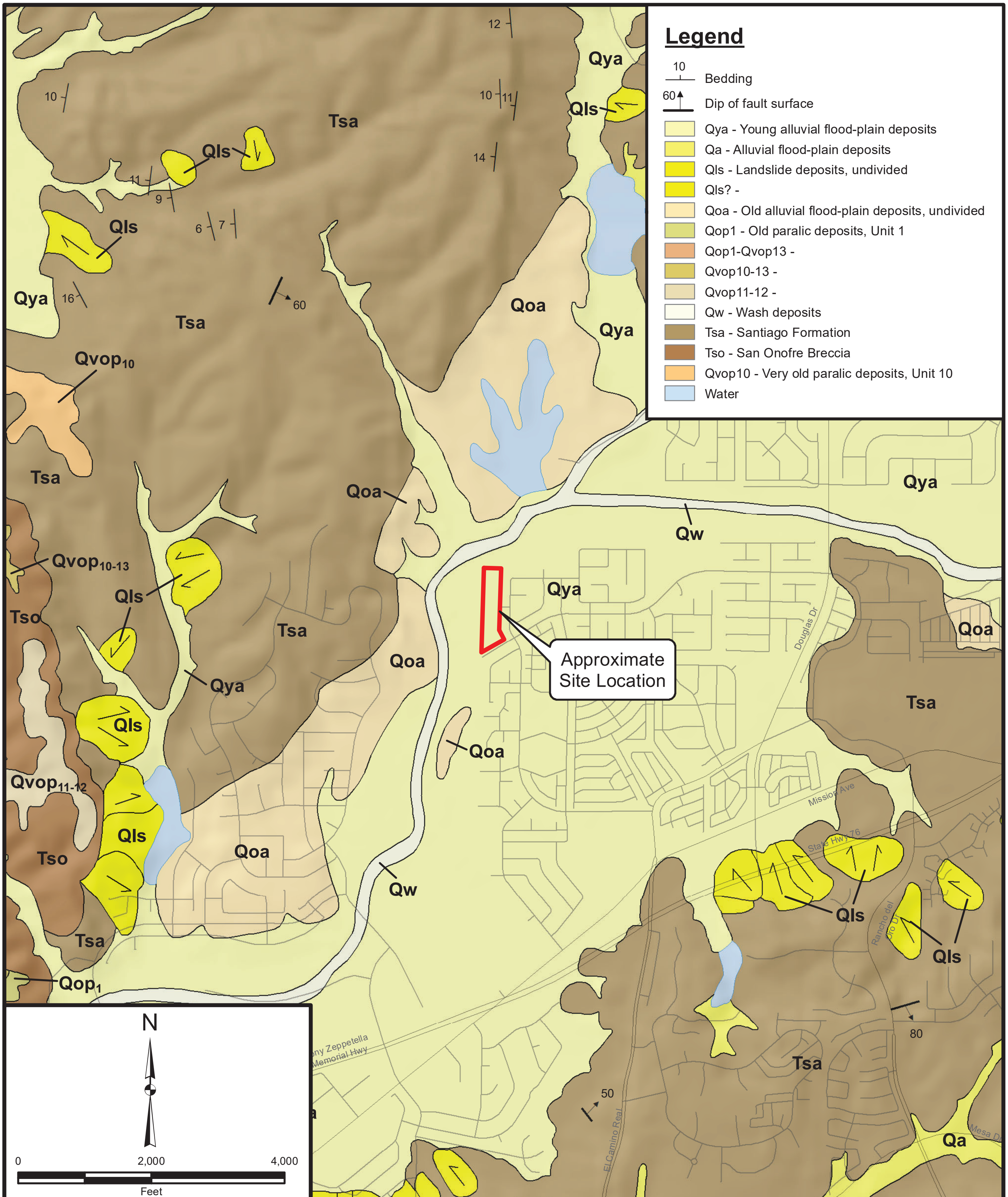
GEOTECHNICAL EXPLORATION MAP

Concordia Geotechnical Investigation
Oceanside, California

Figure 2

Leighton

Map Saved as V:\Drafting\12807\002\Maps\12807-002_F02_GEM_2020-09-17.mxd on 10/15/2020 3:42:59 PM



Project: 12807.002 | Eng/Geol: WDO/MDJ

Scale: 1 " = 2,000 ' | Date: September 2020

Geoogy map: Map of the Oceanside 30'x60' quadrangle, California, compiled by Michael P. Kennedy and Siang S. Tan, 2008

Author: mmurphy (mmurphy)

GEOLOGY MAP

Concordia Geotechnical Investigation

Oceanside, California

Figure 3

Leighton

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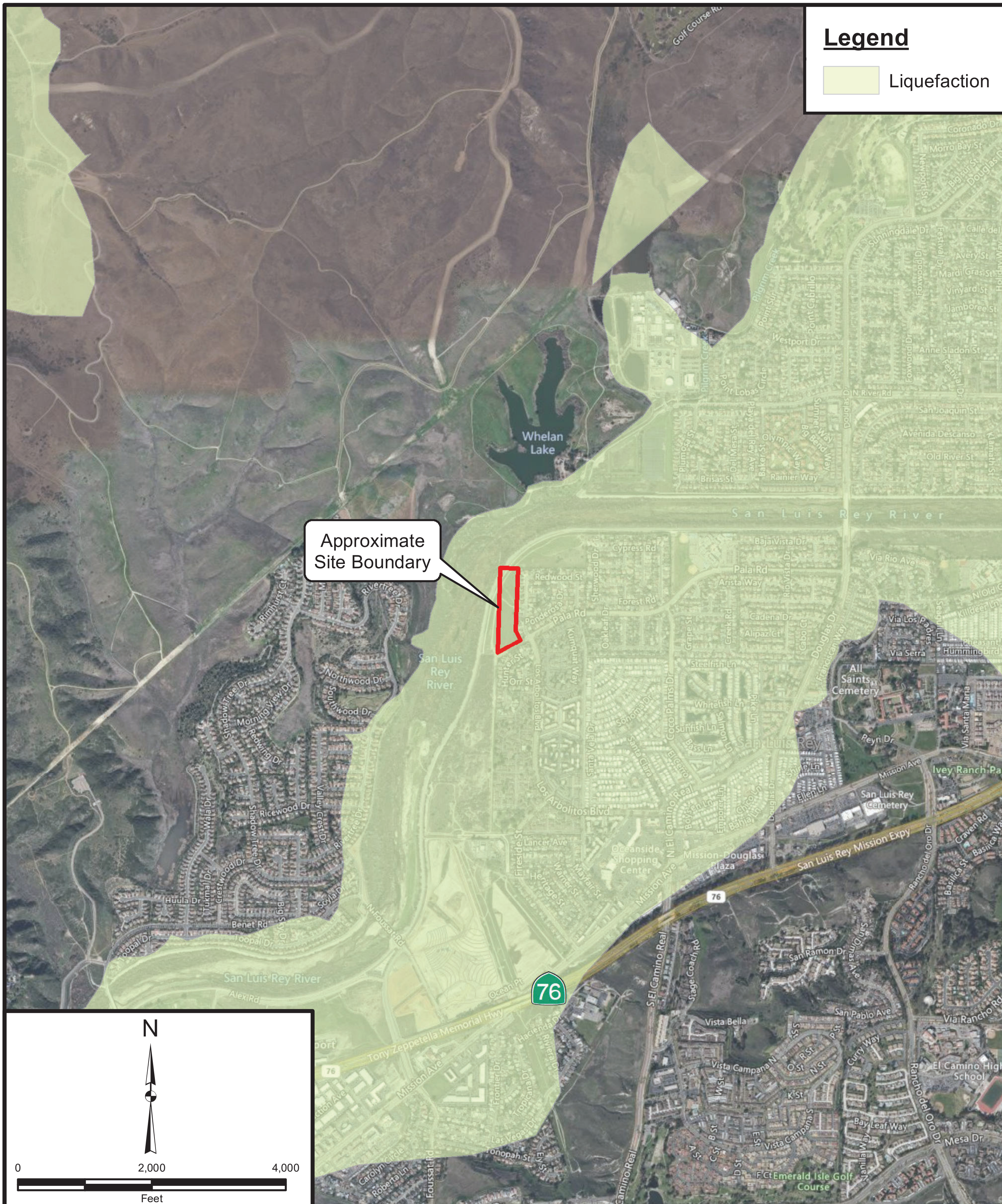


Project: 12807.002	Eng/Geol: WDO/MDJ
Scale: 1 " = 2,000 '	Date: September 2020
Base Map: Bing Maps 2020	
Author: (mmurphy)	

LIQUEFACTION ZONE MAP

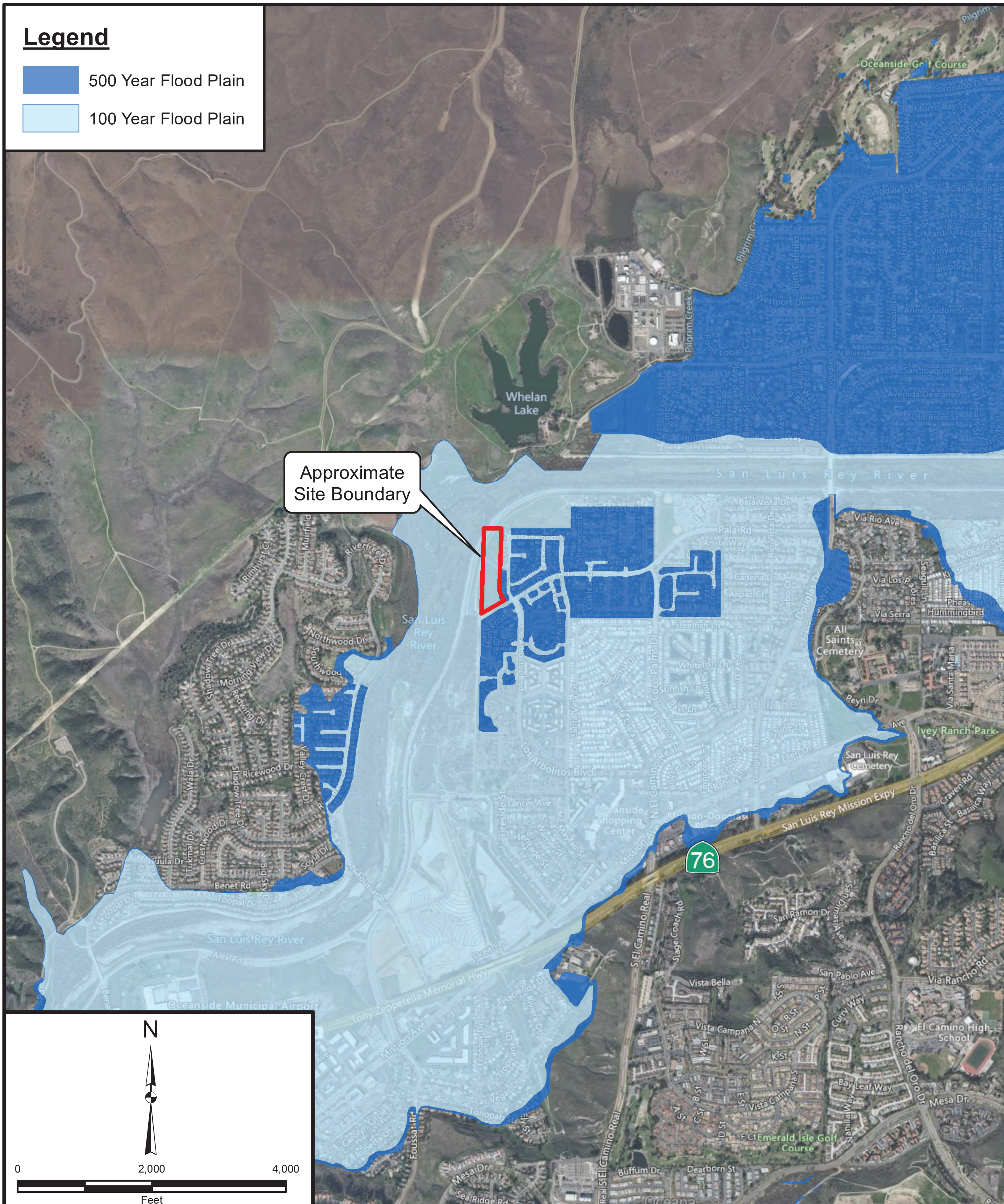
Concordia Geotechnical Investigation

Oceanside, California

Figure 4

Leighton

Map Saved as P:\Drafting\12807\002\Maps\12807-002_F04_LZM_2020-09-17.mxd on 9/18/2020 9:52:10 AM



Project: 12807.002	Eng/Geol: WDO/MDJ
Scale: 1 " = 2,000 '	Date: October 2020

Base Map: Bing Maps 2020
Flood data: SanGIS.

Author: (mmurphy)

FLOOD HAZARD ZONE MAP

Concordia Geotechnical Investigation
Oceanside, California

Figure 5

Leighton

Map Saved as V:\Drafting\12807\002\Maps\12807-002_F05_FHZM_2020-09-17.mxd on 10/13/2020 3:08:23 PM

Appendix A
References

APPENDIX A
REFERENCES

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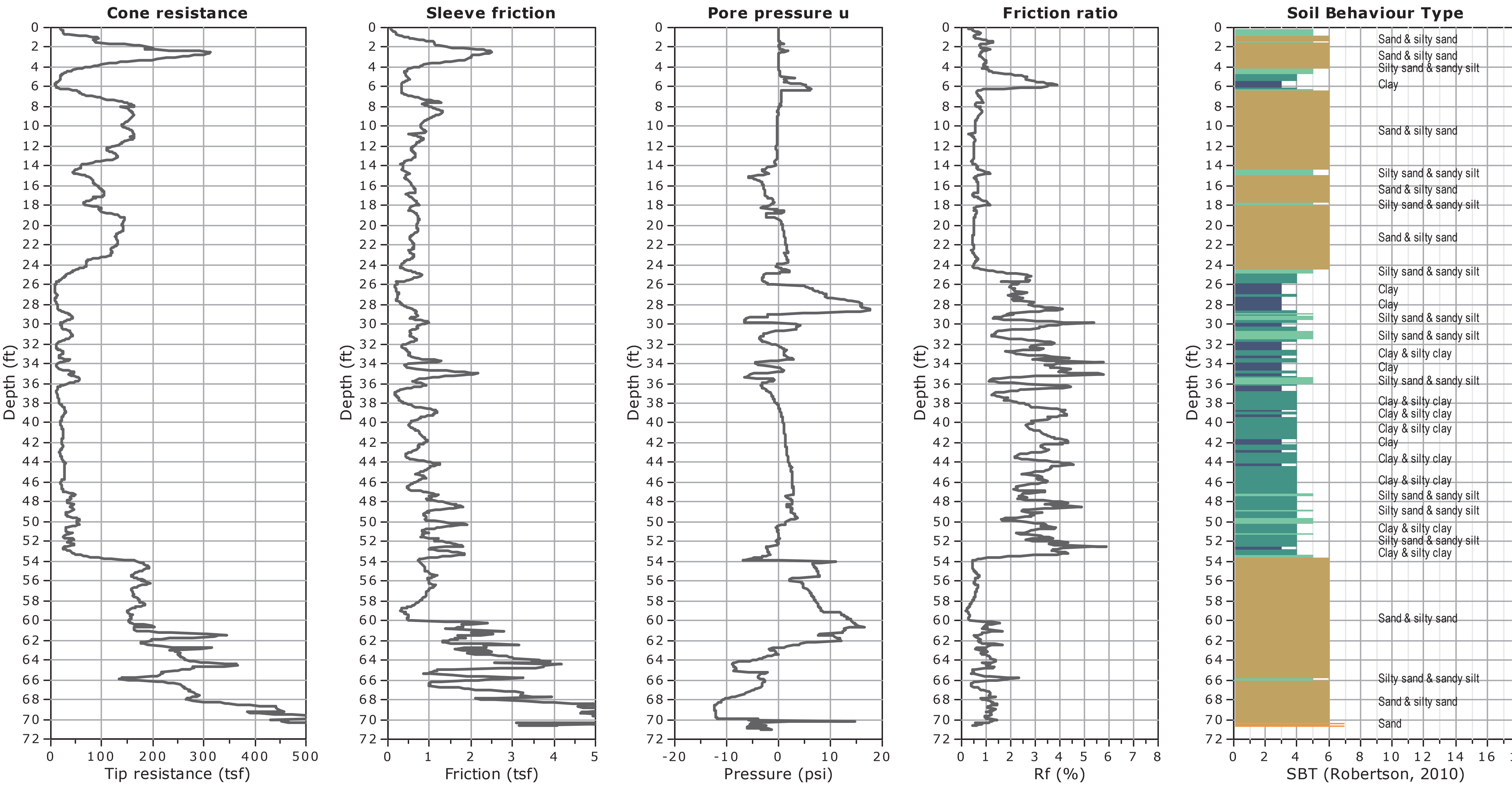
Appendix B
Trench and CPT Logs

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

Project: Leighton & Associates / Concordia
Location: Oceanside, CA

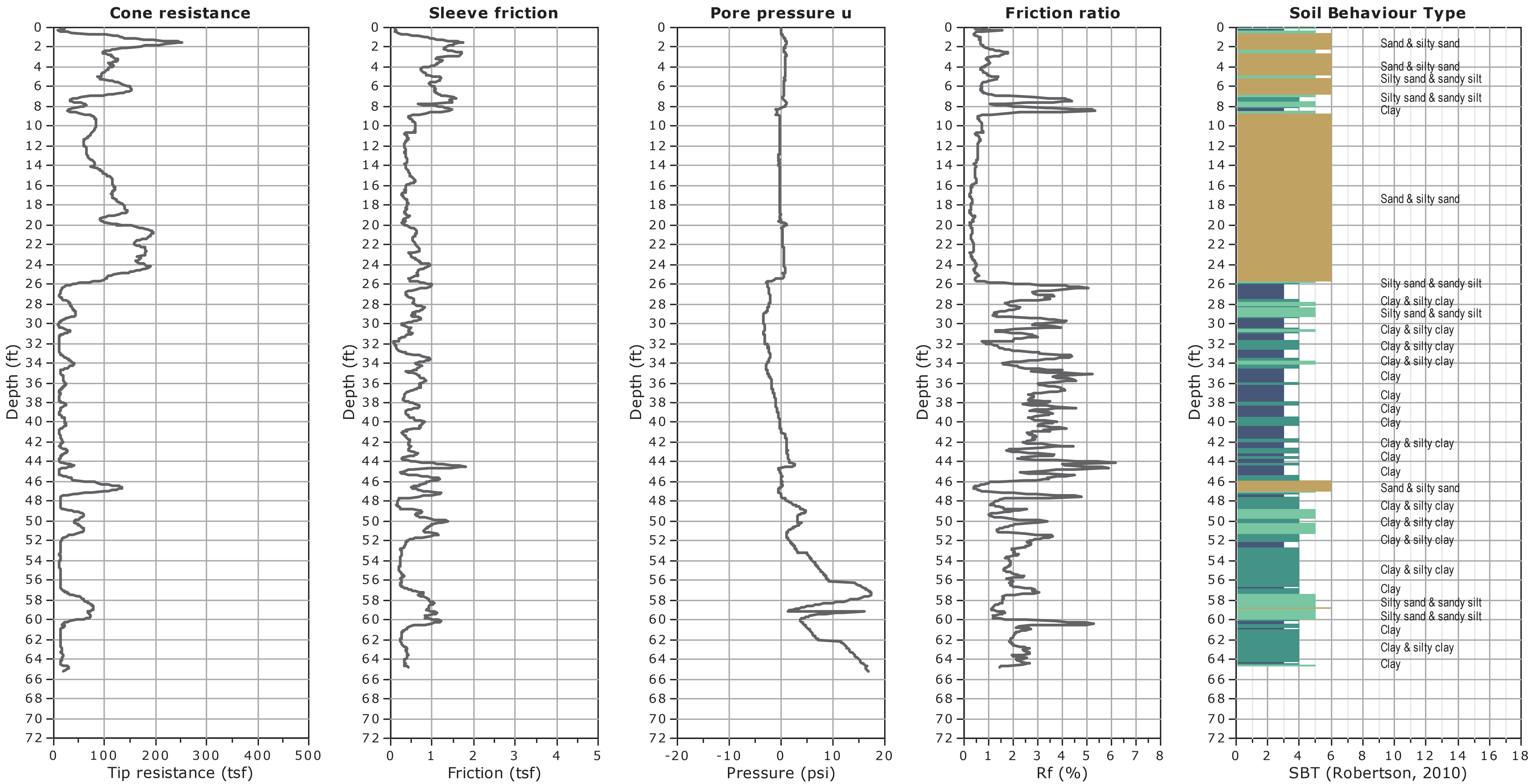
CPT-1
Total depth: 71.06 ft, Date: 7/21/2020



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

Project: Leighton & Associates / Concordia
Location: Oceanside, CA

CPT-2
Total depth: 65.17 ft, Date: 7/21/2020

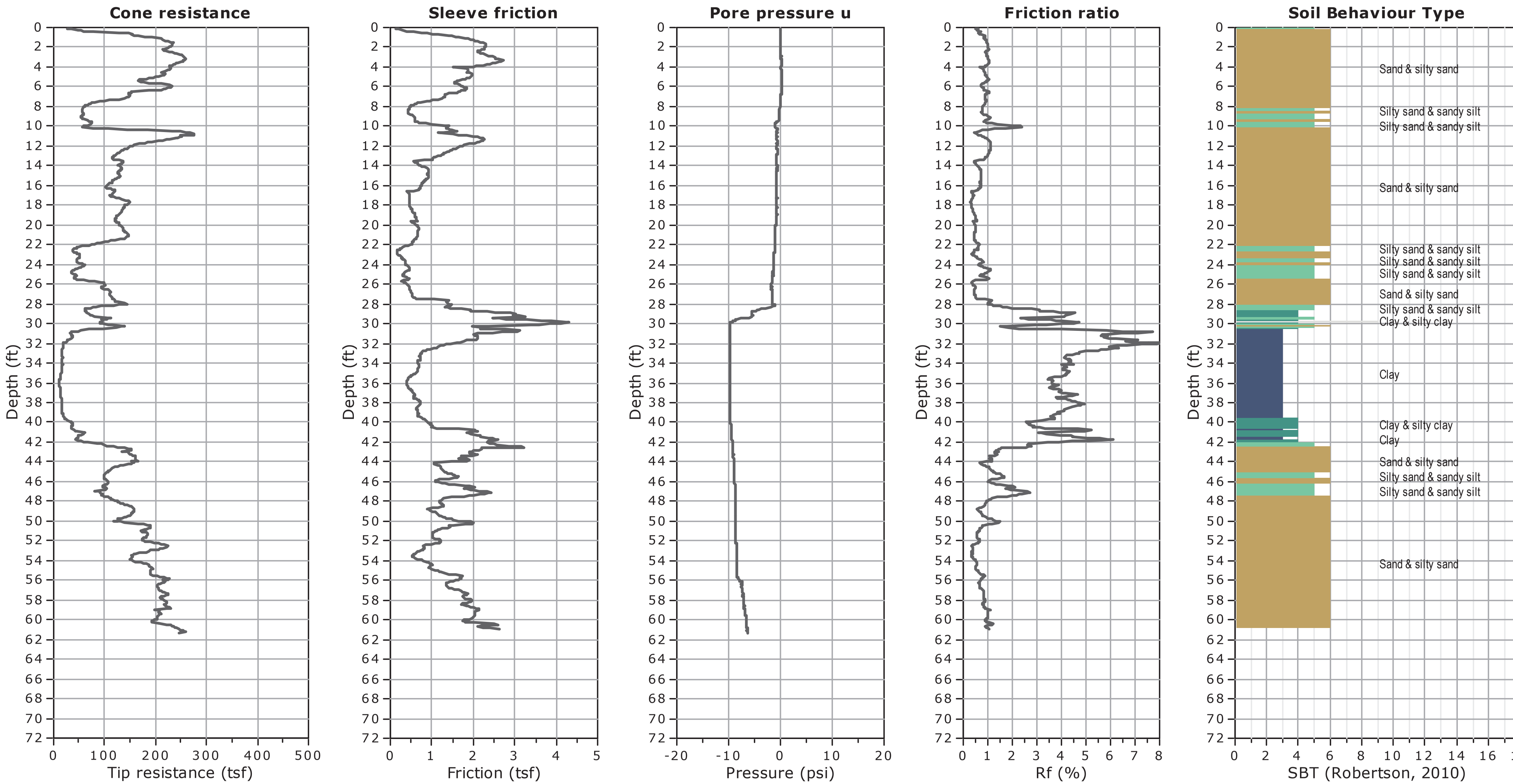


CPeT-IT v.2.3.1.9 - CPTU data presentation & interpretation software - Report created on: 7/22/2020, 11:13:53 AM
Project file:

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

Project: Leighton & Associates / Concordia
Location: Oceanside, CA

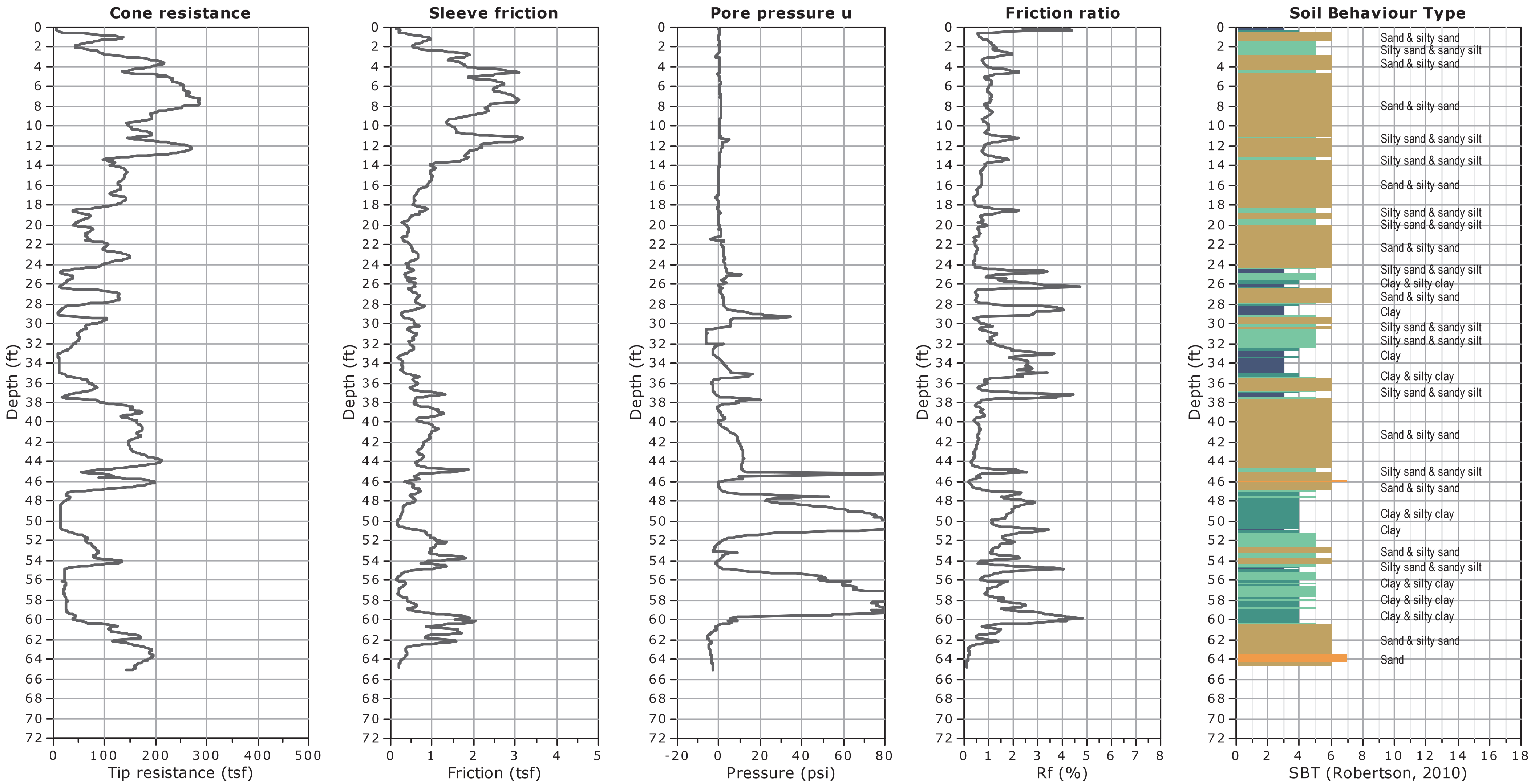
CPT-3
Total depth: 61.30 ft, Date: 7/21/2020



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

Project: Leighton & Associates / Concordia
Location: Oceanside, CA

CPT-4
Total depth: 65.16 ft, Date: 7/21/2020



LOG OF TRENCH: T-1

Project Name: Concordia/Oceanside
Logged by: ERB
Project Number: 12807.002
Elevation: 50 Feet
Equipment: Rubber Tire Backhoe
Location/Grid:

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	ENGINEERING PROPERTIES: USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-3': Silty SAND, loose, light gray, dry, fine-grained, trace debris		SM	B-1 @ 0-3'		
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 3'-8': Silty SAND, loose, dark gray, moist fine SAND, micaceous, friable, denser at depth					

GRAPHICAL REPRESENTATION: South SCALE: 1"-5' SURFACE SLOPE: TREND:

Total Depth = 8 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-2

Project Name: Concordia/Oceanside
Logged by: ERB
Project Number: 12807.002
Elevation: 49.5 Feet
Equipment: Rubber Tire Backhoe
Location/Grid:

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-2’: Silty SAND, loose, light grayish-brown, dry, medium SAND, trace micas, trace debris		SM			
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 2’-7’: Silty SAND, loose, medium gray, moist, fine to medium SAND, trace micas, friable, caving		SM			
	@ 7’-10’: Becomes fine, micaceous, dark gray to black, medium dense		SM			

ENGINEERING PROPERTIES

GRAPHICAL REPRESENTATION: North SCALE: 1”-5’ SURFACE SLOPE: TREND:

Total Depth = 10 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-3

Project Name: Concordia/Oceanside | Logged by: ERB

Project Number: 12807.002 | Elevation: 49.5 Feet

Equipment: Rubber Tire Backhoe | Location/Grid:

ENGINEERING PROPERTIES

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-2': Silty SAND, loose, light gray, dry, fine-grained, trace micas		SM	B-1 @ 2'-7'		
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 2'-7': Poorly-graded SAND, loose, medium gray, moist, fine-grained, micaceous, friable		SP	B-2 @ 7'-9'		
	@ 7'-9': Silty SAND, loose to medium dense, dark gray, fine SAND, moist, micaceous, friable		SM			

GRAPHICAL REPRESENTATION: Southwest SCALE: 1"-5' SURFACE SLOPE: TREND:

Total Depth = 9 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-4

Project Name: Concordia/Oceanside
Logged by: ERB
Project Number: 12807.002
Elevation: 49 Feet
Equipment: Rubber Tire Backhoe
Location/Grid:

ENGINEERING PROPERTIES

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-2': Silty SAND, loose, light gray, dry, fine to medium SAND, friable		SM			
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 2'-6.5': Silty SAND, loose to medium dense, dark gray, moist, fine SAND, micaceous		SM			

GRAPHICAL REPRESENTATION: West SCALE: 1"-5' SURFACE SLOPE: TREND:

Total Depth = 6.5 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-5

Project Name: Concordia/Oceanside | Logged by: ERB

Project Number: 12807.002 | Elevation: 49 Feet

Equipment: Rubber Tire Backhoe | Location/Grid:

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-1.5': Silty SAND, loose, light gray, dry, fine-grained, trace micas, trace debris		SM	B-1 @ 0-6'		
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 1.5'-7': Silty SAND, medium dense, medium gray, moist, trace rounded 1"-2" gravel, fine-grained, micaceous, interbedded, light gray, poorly-graded SAND, loose, friable (approximately 6" layers)		SP-SM			
	@ 7'-9.5': Silty SAND, loose to medium dense, dark gray, moist, fine-grained, micaceous		SM			

GRAPHICAL REPRESENTATION: East | SCALE: 1"-5' | SURFACE SLOPE: | TREND: N/A

Total Depth = 9.5 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-6

Project Name: Concordia/Oceanside

Logged by: ERB

Project Number: 12807.002

Elevation: 48 Feet

Equipment: Rubber Tire Backhoe

Location/Grid:

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-2.5': Silty SAND, loose, light gray, dry, fine- to medium-grained, friable, rootlets and debris throughout		SM			
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 2.5'-7': Silty SAND, loose to medium dense, medium to dark gray, fine SAND, moist, micaceous, interbedded with poorly-graded SAND, loose, light gray, medium to coarse SAND, friable, medium dense at 7'		SP-SM	B-1 @ 2.5'-7'		

GRAPHICAL REPRESENTATION: South SCALE: 1"-5' SURFACE SLOPE: TREND: N/A

Total Depth = 7 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-7

Project Name: Concordia/Oceanside
Logged by: ERB
Project Number: 12807.002
Elevation: 47 Feet
Equipment: Rubber Tire Backhoe
Location/Grid:

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-2.5': Silty SAND, loose, light gray, dry, fine to medium SAND, friable		SM			
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 2.5'-9.5': Silty SAND, medium dense, dark gray, moist, fine SAND, interbedded with poorly graded SAND, loose, light gray, dry, friable, denser at 8'		SP-SM			

ENGINEERING PROPERTIES

GRAPHICAL REPRESENTATION: Southwest SCALE: 1"-5' SURFACE SLOPE: TREND:

Total Depth = 9.5 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-8

Project Name: Concordia/Oceanside

Logged by: ERB

Project Number: 12807.002

Elevation: 47 Feet

Equipment: Rubber Tire Backhoe

Location/Grid:

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-2.5': Silty SAND, loose, light gray, fine to medium SAND, friable		SM	B-1 @ 0-5'		
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 2.5'-4': Silty SAND, medium dense, dark gray, moist, fine SAND, micaceous		SM-SP	B-2 @ 5'-9.5'		
	@ 4'-6.5': Poorly-graded SAND, loose, light gray, dry, friable		SM			
	@ 6.5'-9.5': Silty SAND, medium dense, dark gray, moist, fine SAND, micaceous					

GRAPHICAL REPRESENTATION: SCALE: 1"-5' SURFACE SLOPE: TREND:

Total Depth = 9.5 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-9

Project Name: Concordia/Oceanside
Logged by: ERB
Project Number: 12807.002
Elevation: 47 Feet
Equipment: Rubber Tire Backhoe
Location/Grid:

ENGINEERING PROPERTIES

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	<u>ARTIFICIAL FILL (Afu)</u>	Afu				
	@ 0-2.5': Silty SAND, loose, light gray, dry, fine to medium SAND		SM			
	<u>QUATERNARY YOUNG ALLUVIUM (Qya)</u>	Qya				
	@ 2.5'-9.5': Silty SAND, loose to medium dense, dark gray, moist, fine SAND, micaceous, denser at 8 Feet		SM			

GRAPHICAL REPRESENTATION: West SCALE: 1"-5' SURFACE SLOPE: TREND:

Total Depth = 9.5 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-10

Project Name: Concordia/Oceanside   Logged by: ERB

Project Number: 12807.002   Elevation: 46.5 Feet

Equipment: Rubber Tire Backhoe   Location/Grid:

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-2.5': Silty SAND, loose, light gray, dry, fine to medium SAND		SM	B-1 @ 5-12'		
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 2.5'-6': Silty SAND, loose, medium gray, moist, fine to medium SAND, friable, interbedded poorly-graded SAND, loose, light gray, friable		SP-SM			
	@ 6'-12': Silty SAND, medium dense, dark gray-black, moist, fine SAND, micaceous		SM			

GRAPHICAL REPRESENTATION: Southwest   SCALE: 1"-5'   SURFACE SLOPE: TRENCH:

Total Depth = 12 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-11

Project Name: Concordia/Oceanside Logged by: ERB

Project Number: 12807.002 Elevation: 47 Feet

Equipment: Rubber Tire Backhoe Location/Grid:

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-2.5': Silty SAND, light gray, loose, dry, trace micas, rootlets, debris		SM	B-1 @ 0-9'		
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 2.5'-9.5': Silty SAND, medium dense, gray, moist, fine SAND, micaceous, interbedded with poorly-graded SAND, loose, light gray, medium to coarse SAND, friable		SM			

GRAPHICAL REPRESENTATION: East SCALE: 1"-5' SURFACE SLOPE: TREND:

Total Depth = 9.5 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

LOG OF TRENCH: T-12

Project Name: Concordia/Oceanside | Logged by: ERB

Project Number: 12807.002 | Elevation: 46.5 Feet

Equipment: Rubber Tire Backhoe | Location/Grid:

GEOLOGIC ATTITUDES	DATE: 9/18/2020 DESCRIPTION:	GEOLOGIC UNIT	USCS	Sample No.	Moisture (%)	Density (pcf)
	ARTIFICIAL FILL (Afu)	Afu				
	@ 0-2.5’: Silty SAND, loose, light gray, dry, medium to coarse SAND, friable		SM			
	QUATERNARY YOUNG ALLUVIUM (Qya)	Qya				
	@ 2.5’-11’: Silty SAND, medium dense, dark gray, moist, fine SAND, interbedded with poorly-graded SAND, loose to medium dense, light gray, dry, medium SAND, friable		SP-SM			

ENGINEERING PROPERTIES

GRAPHICAL REPRESENTATION: `West | SCALE: 1”-5’ | SURFACE SLOPE: | TREND:

Total Depth = 11 Feet
No Ground Water Encountered
Backfilled: 9/18/2020

Appendix C
Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

Moisture and Density Determination Tests: Moisture content and dry density determinations were performed on relatively undisturbed samples obtained from the soil borings. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from disturbed samples.

Maximum Dry Density and Optimum Moisture Content Tests: The maximum dry density and optimum moisture content of a selected representative soil sample was evaluated in general accordance with ASTM D 1557. The test results are presented on the attached figures.

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with Caltrans Test Method CT643. The results are presented in the table below:

Sample Location	Sample Description	pH	Minimum Resistivity (ohms-cm)
TP-10 @ 5’-10’	Brown Silty SAND	7.7	4,400

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method CT422. The results are presented below:

Sample Location	Sample Description	Chloride Content, ppm
TP-10 @ 5’-10’	Brown Silty SAND	0

APPENDIX C (continued)

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (Caltrans Test Method CT417). The test results are presented in the table below:

Sample Location	Sample Description	Sulfate Content (%)	Potential Degree of Sulfate Attack*
TP-10 @ 5'-10'	Brown Silty SAND	Less than 0.0150	Not Applicable

* Based on the 2011 edition of American Concrete Institute (ACI) Committee 318R, Table No. 4.2.1.

Particle/Grain Size Analysis: Particle size analysis was performed by mechanical sieving, wash sieving, and hydrometer methods according to ASTM D422, D 1140, and D6913. The percent fine particles from these analyses are summarized below. Plots of the sieve and hydrometer results are provided on the figures at the end of this Appendix.

Appendix D
Liquefaction Analysis

Leighton
San Diego
Ocean

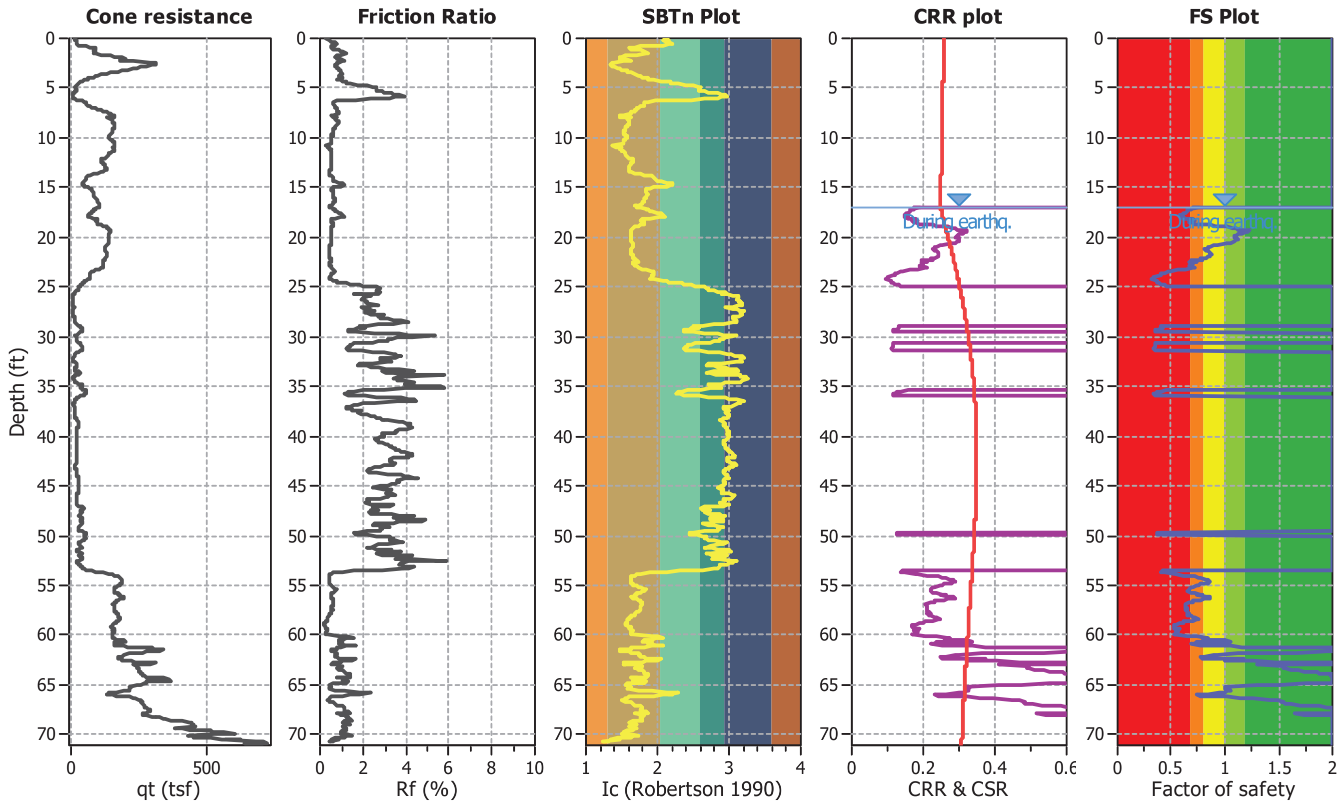
LIQUEFACTION ANALYSIS REPORT

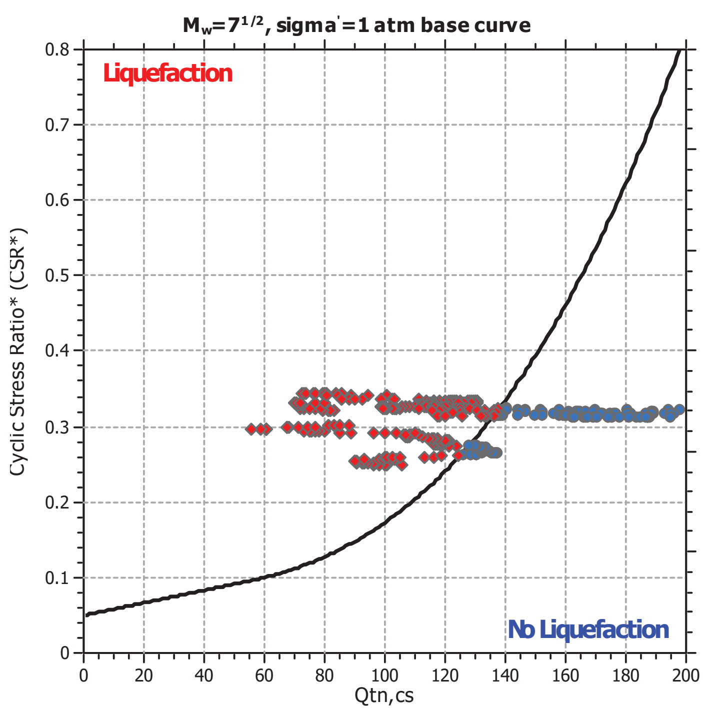
Project title : 12807.002 Concordia at Los Arbolitos/Geotech **Location : Oceanside, CA**

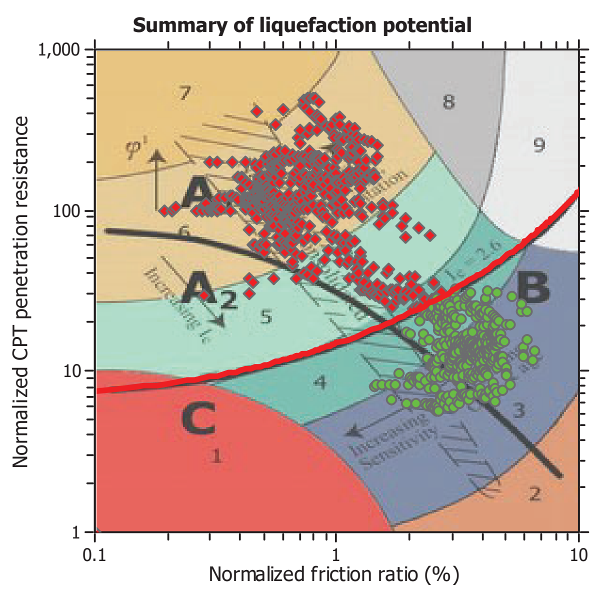
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	16.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	17.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.49	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

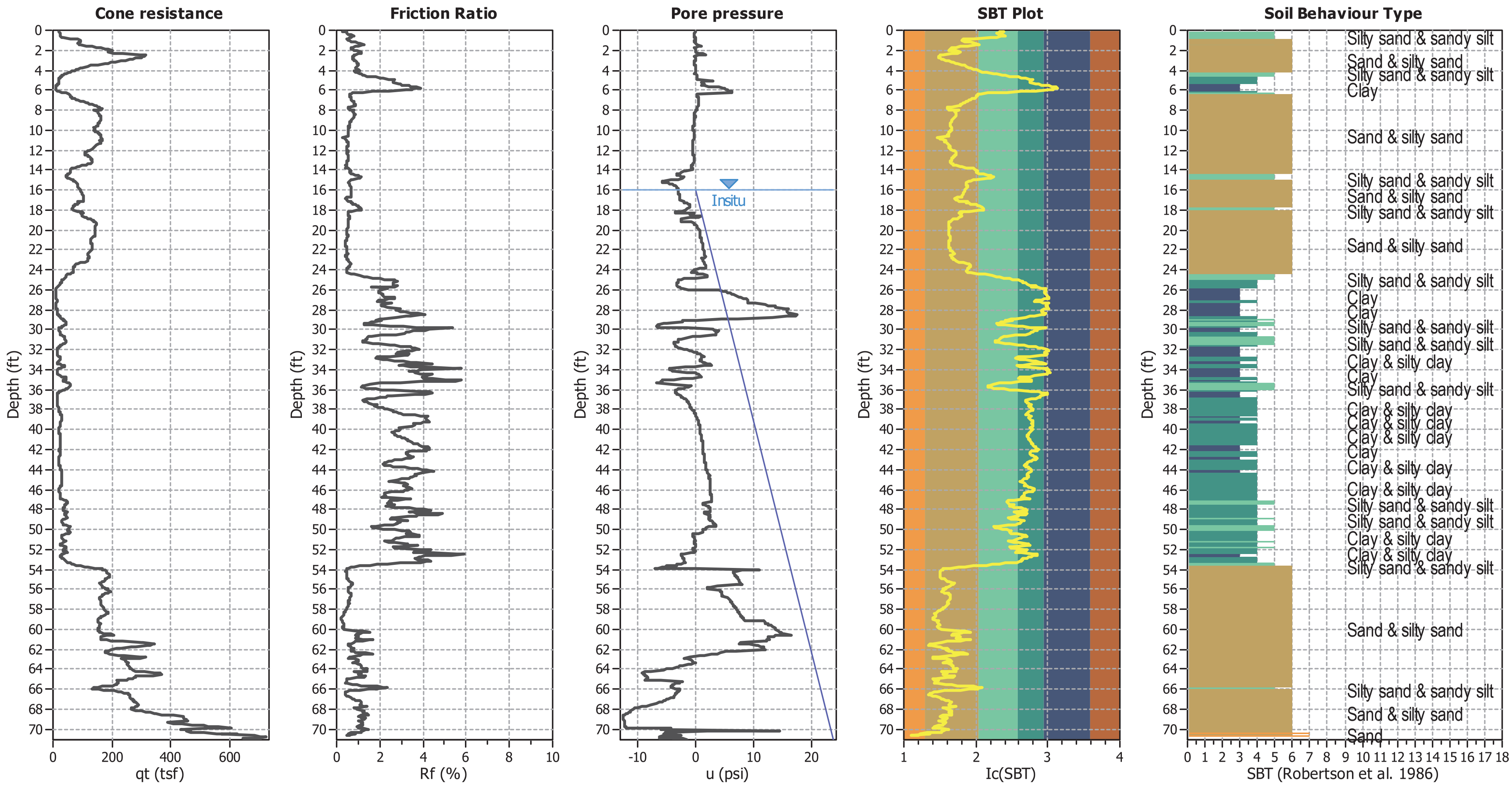






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



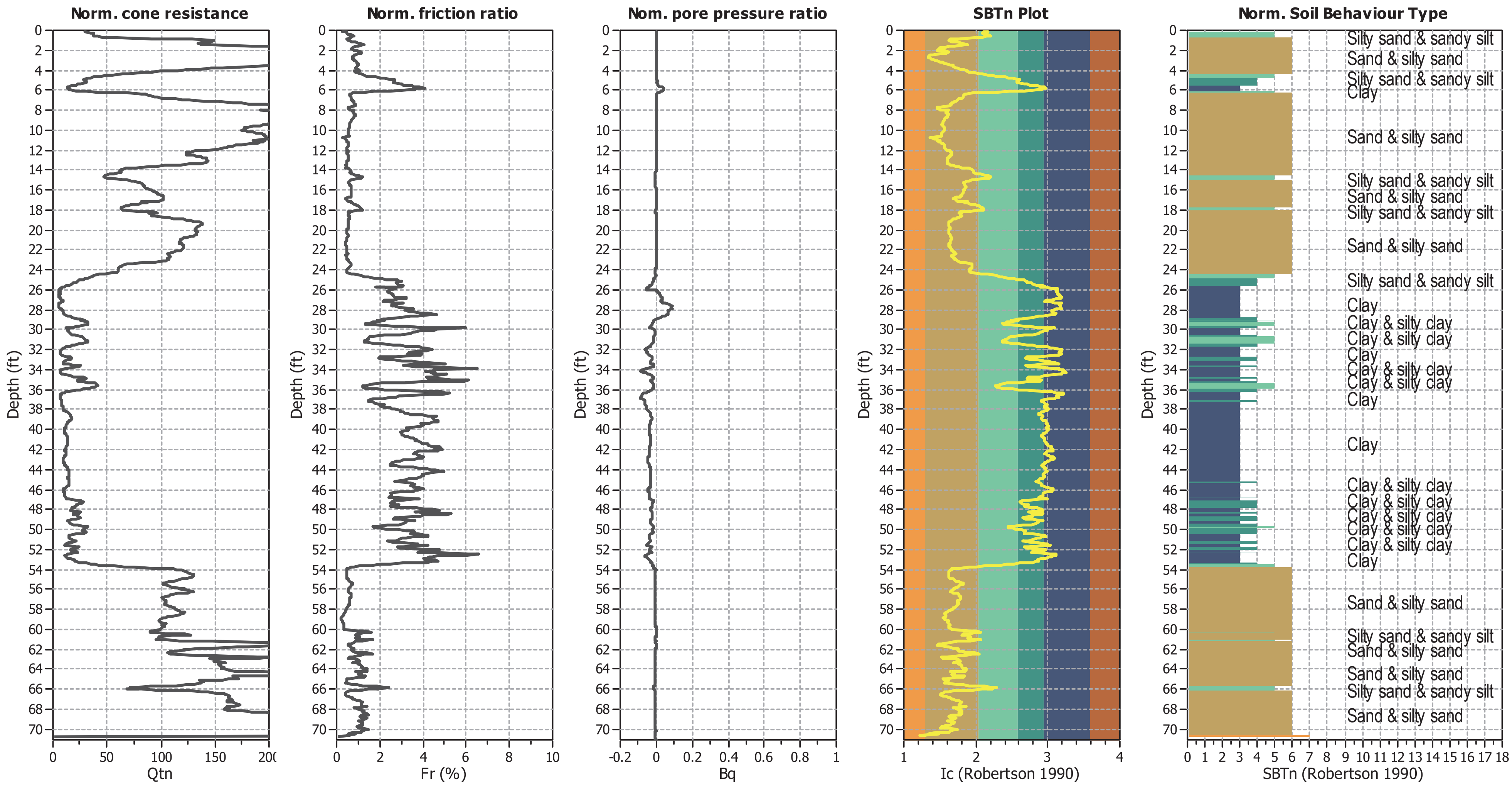
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



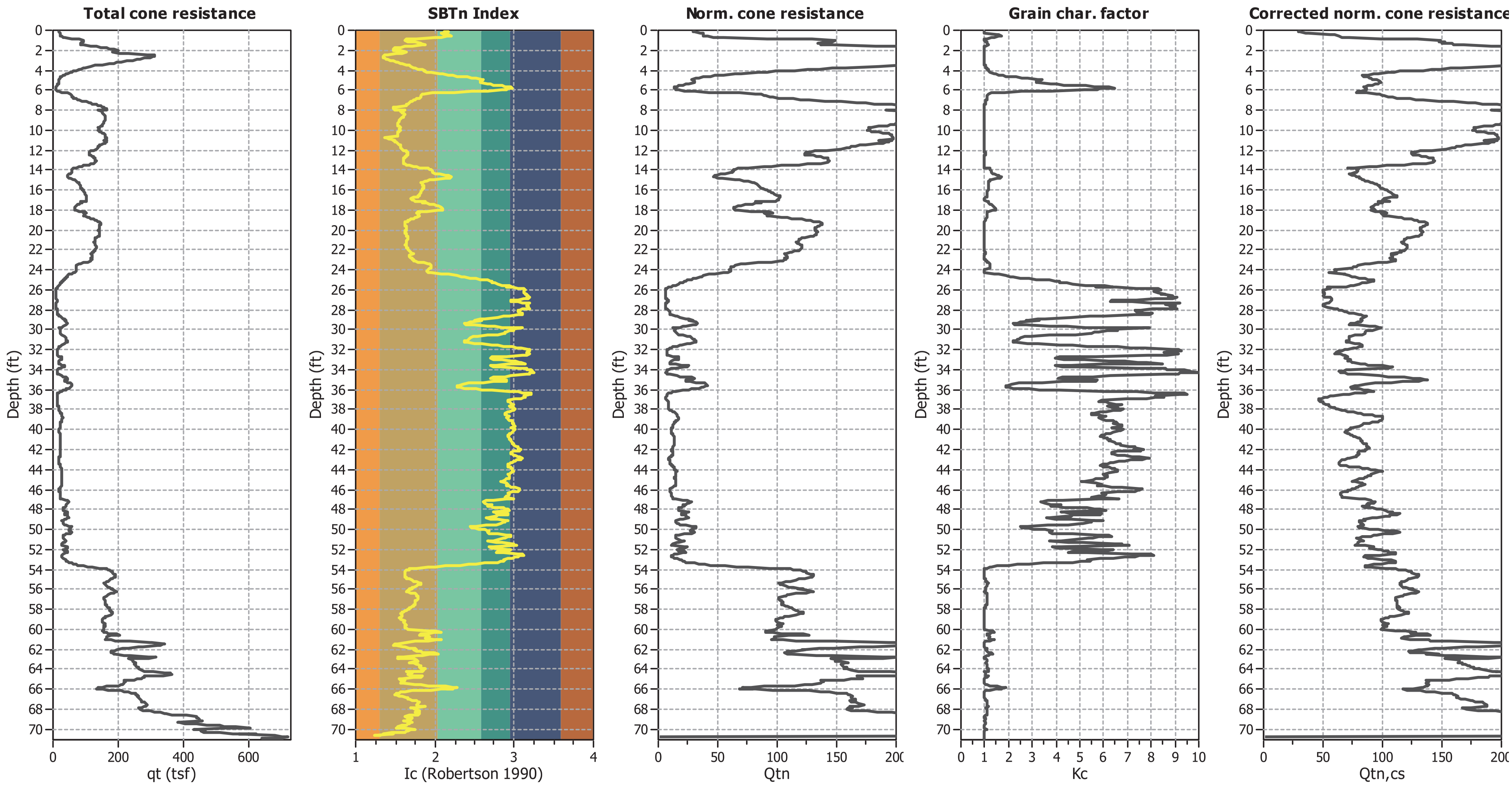
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A		
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes		
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A		

SBTn legend

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

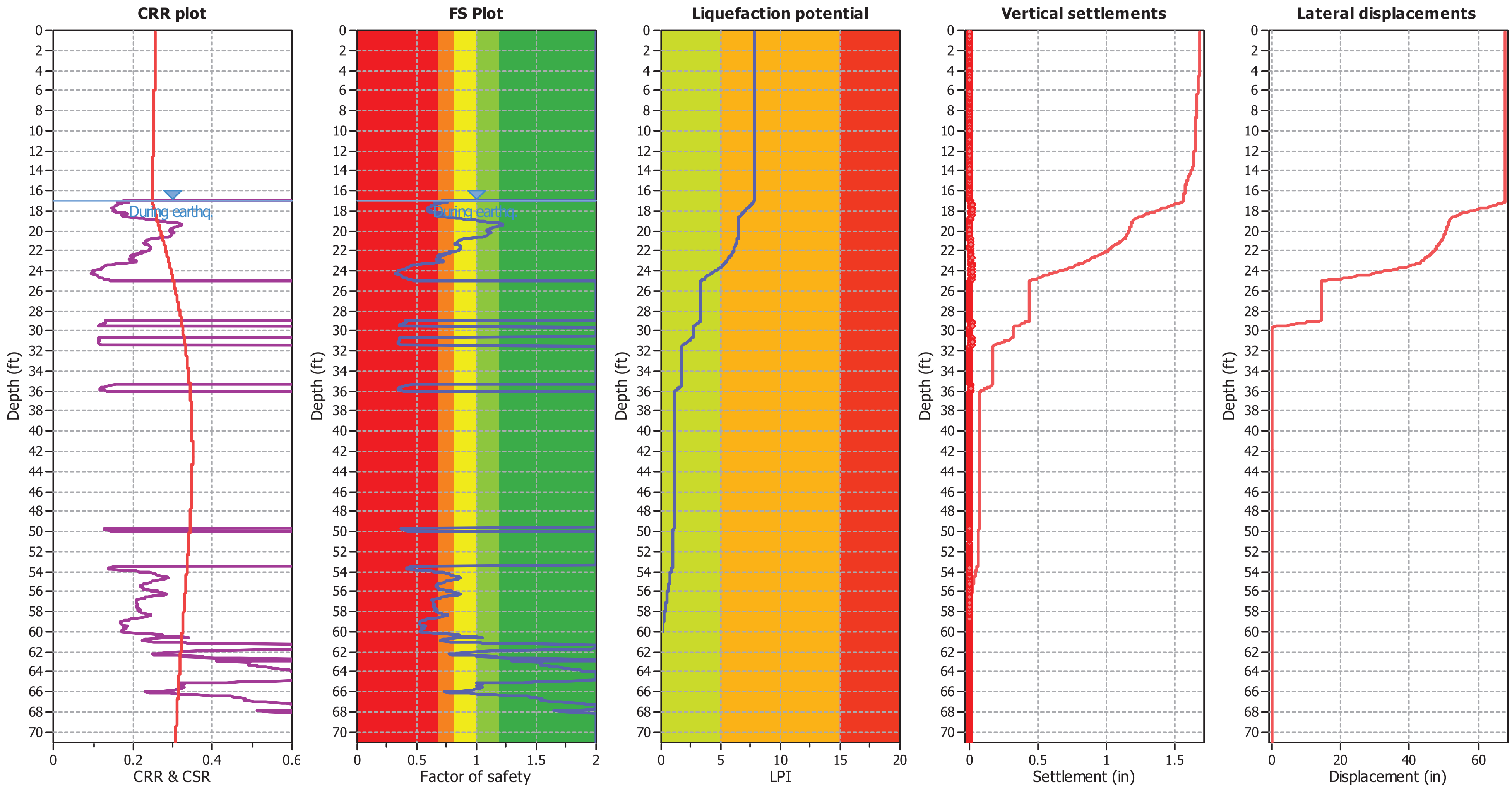
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

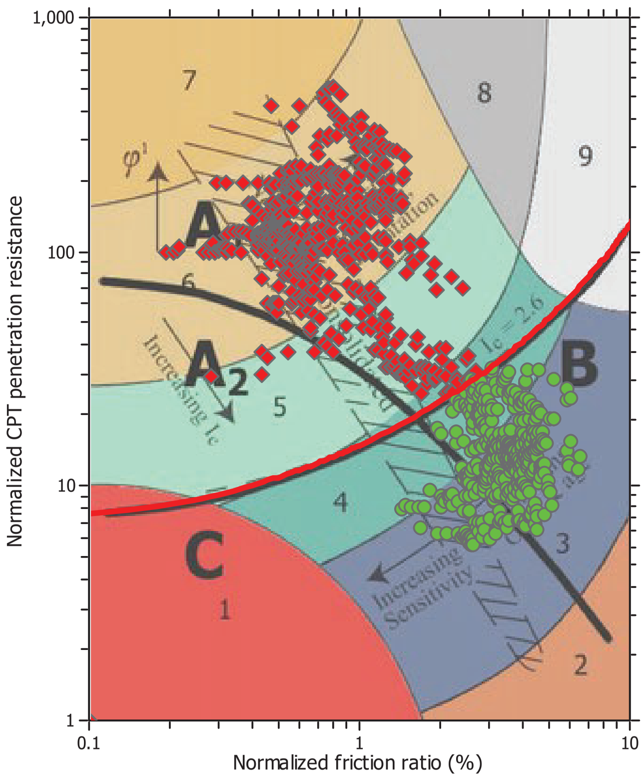
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

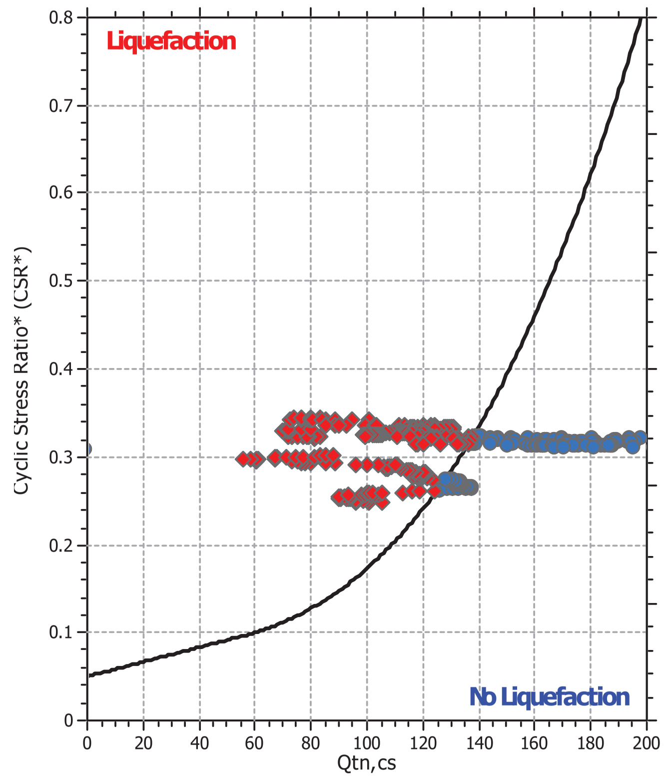
LPI color scheme

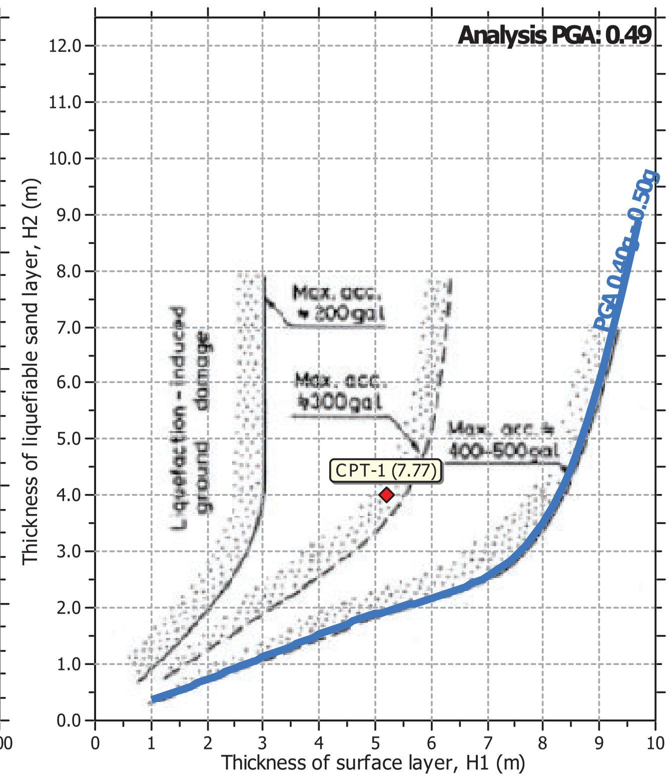
- Very high risk
- High risk
- Low risk

Project file: P:\`InFocus PROJECTS\12501-13000\12807-Concordia Homes-Los Arbolitos-Phase 1 ESA\002 Concordia at Los Arbolitos Geotech\Analyses\Liquefaction Analysis\Liquefaction Analysis.clq

Liquefaction analysis summary plots



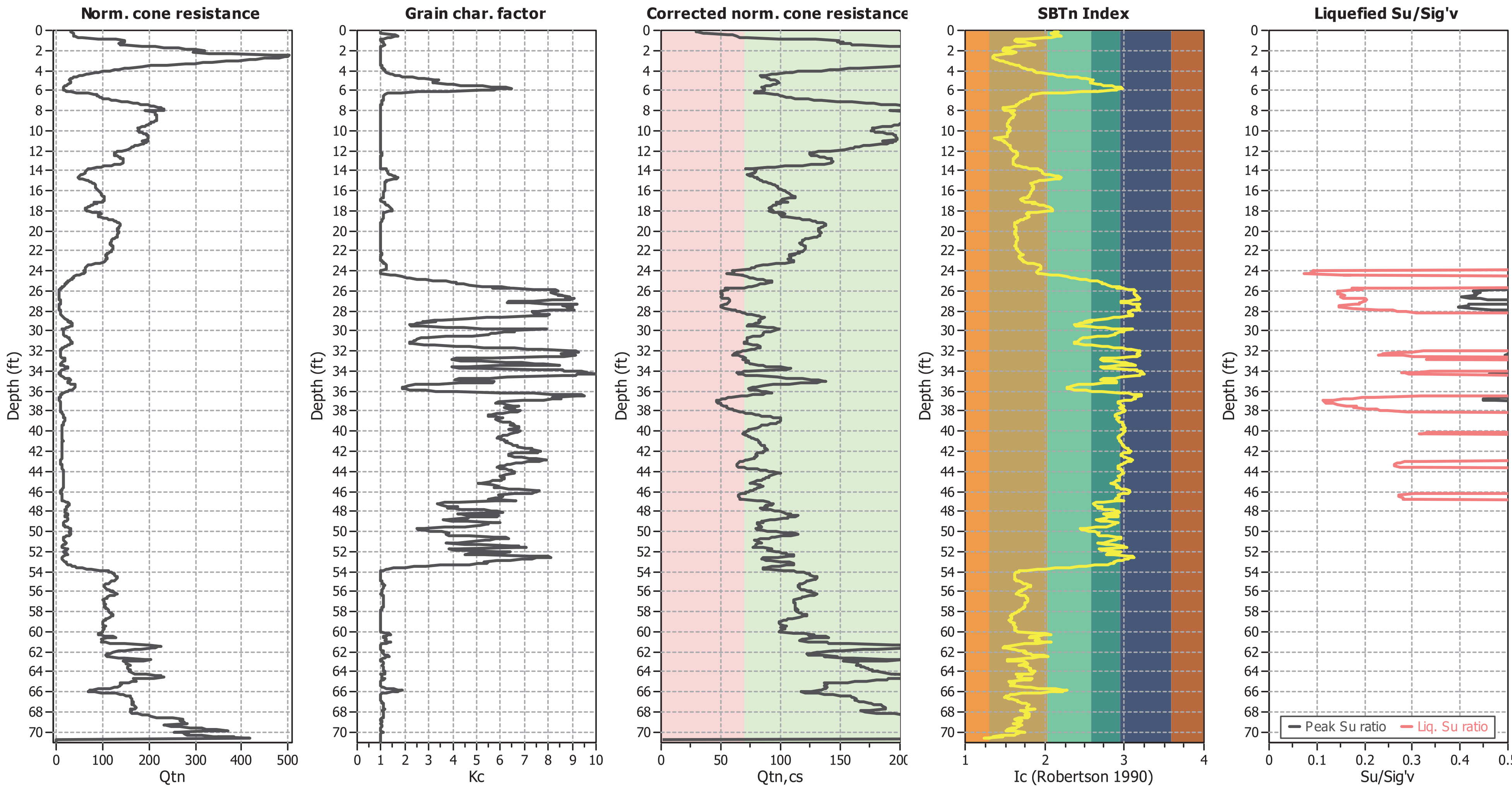




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

Leighton
San Diego
Ocean

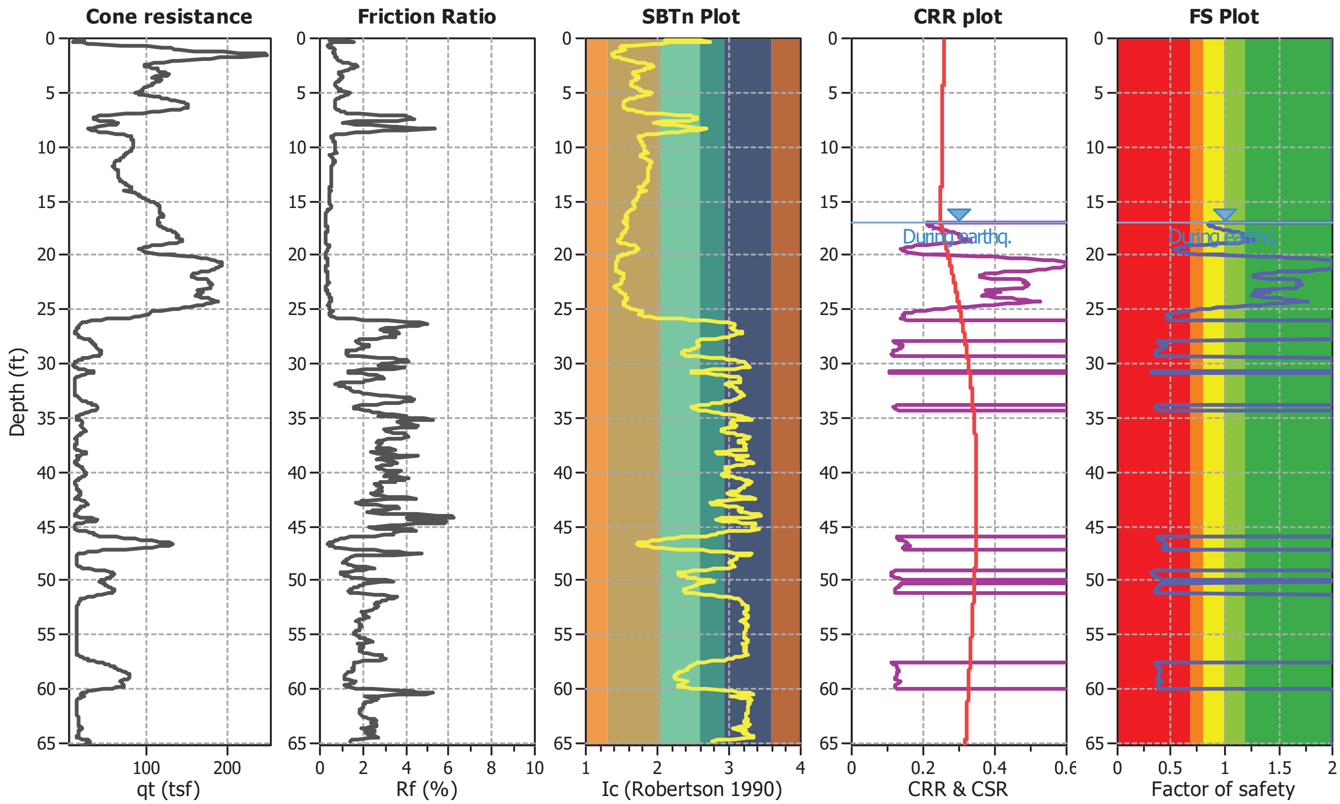
LIQUEFACTION ANALYSIS REPORT

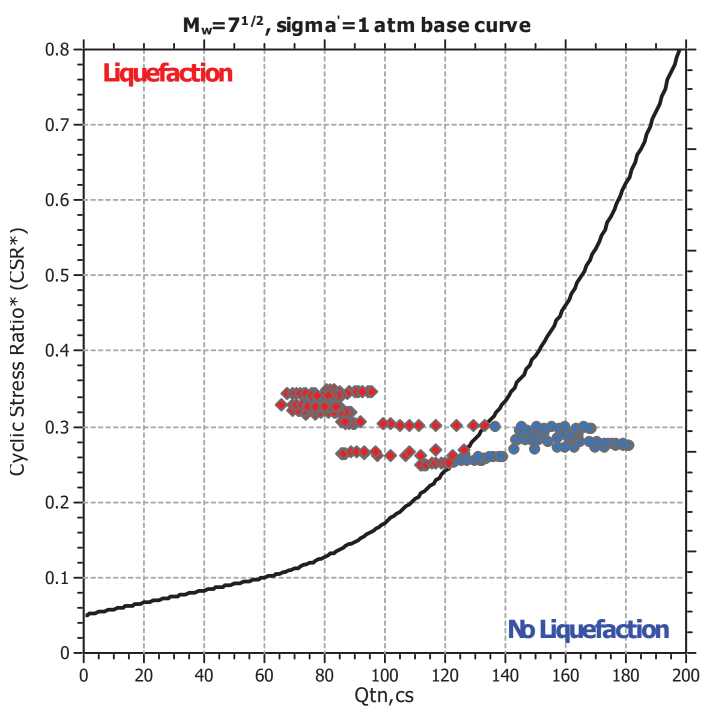
Project title : 12807.002 Concordia at Los Arbolitos/Geotech **Location : Oceanside, CA**

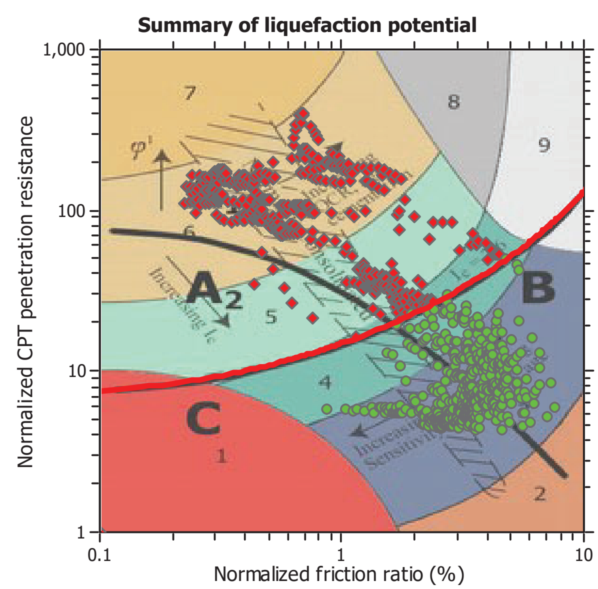
CPT file : CPT-2

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	16.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	17.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.49	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

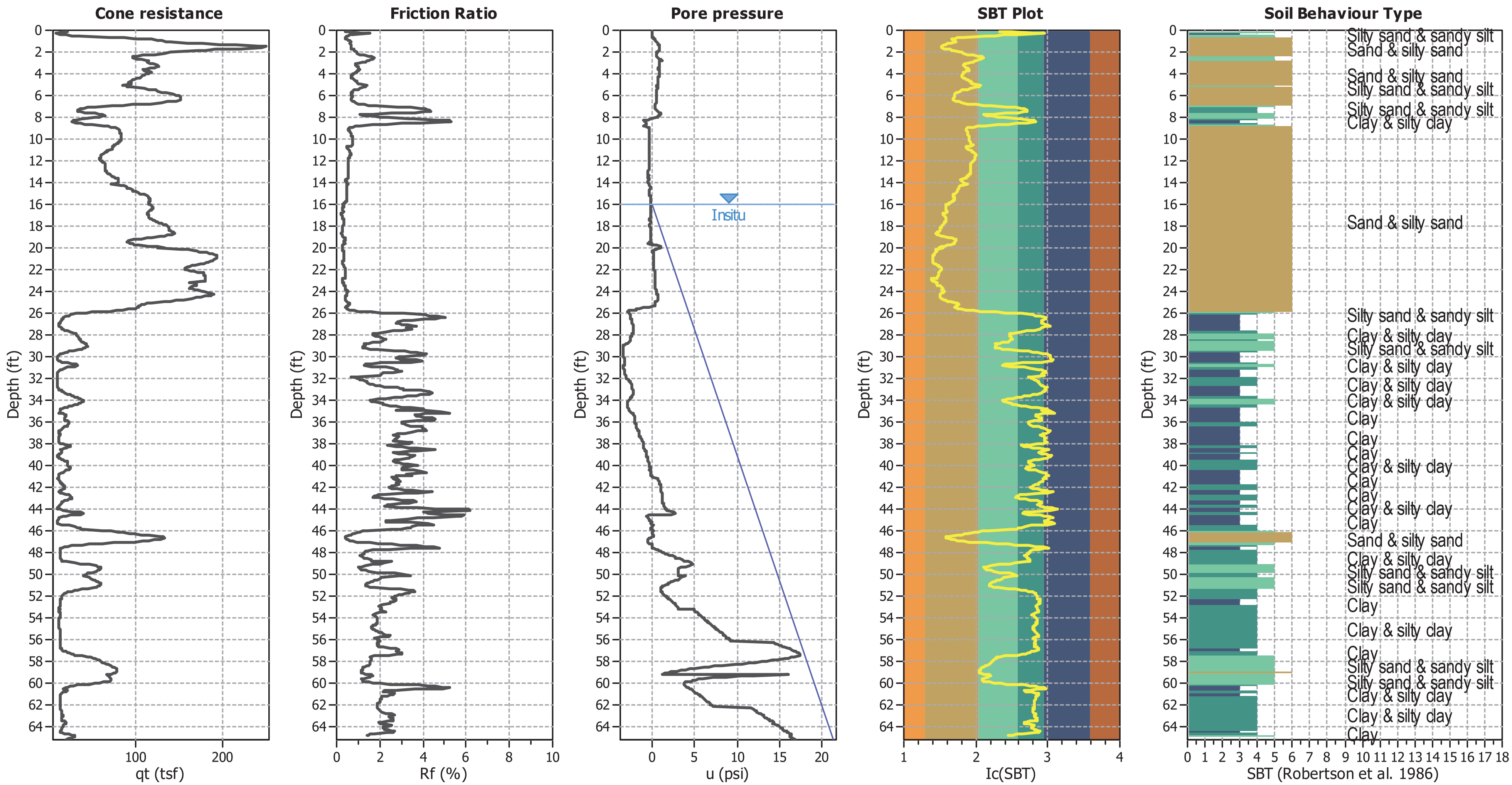






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

CPT basic interpretation plots



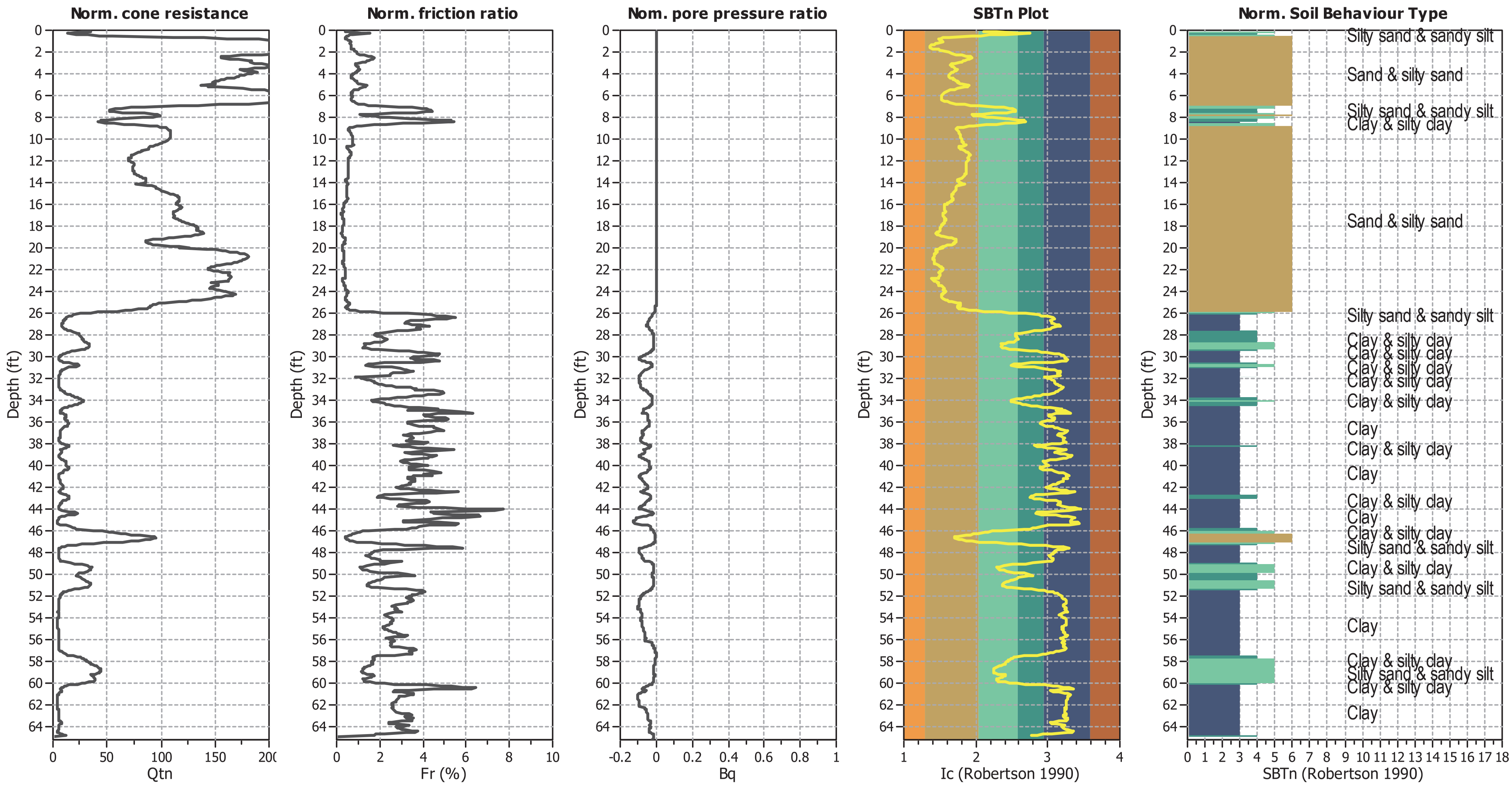
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



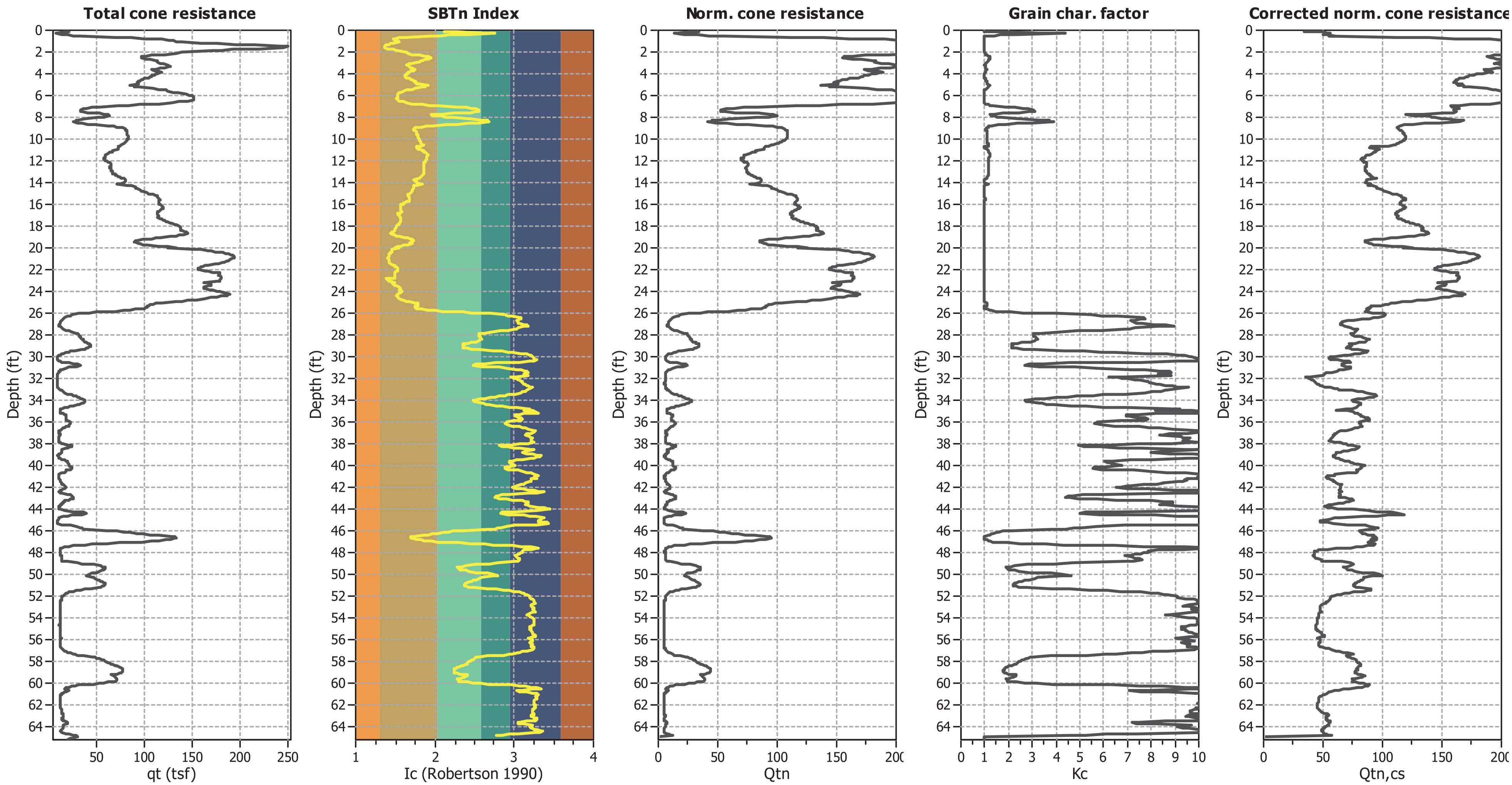
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A		
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes		
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A		

SBTn legend

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

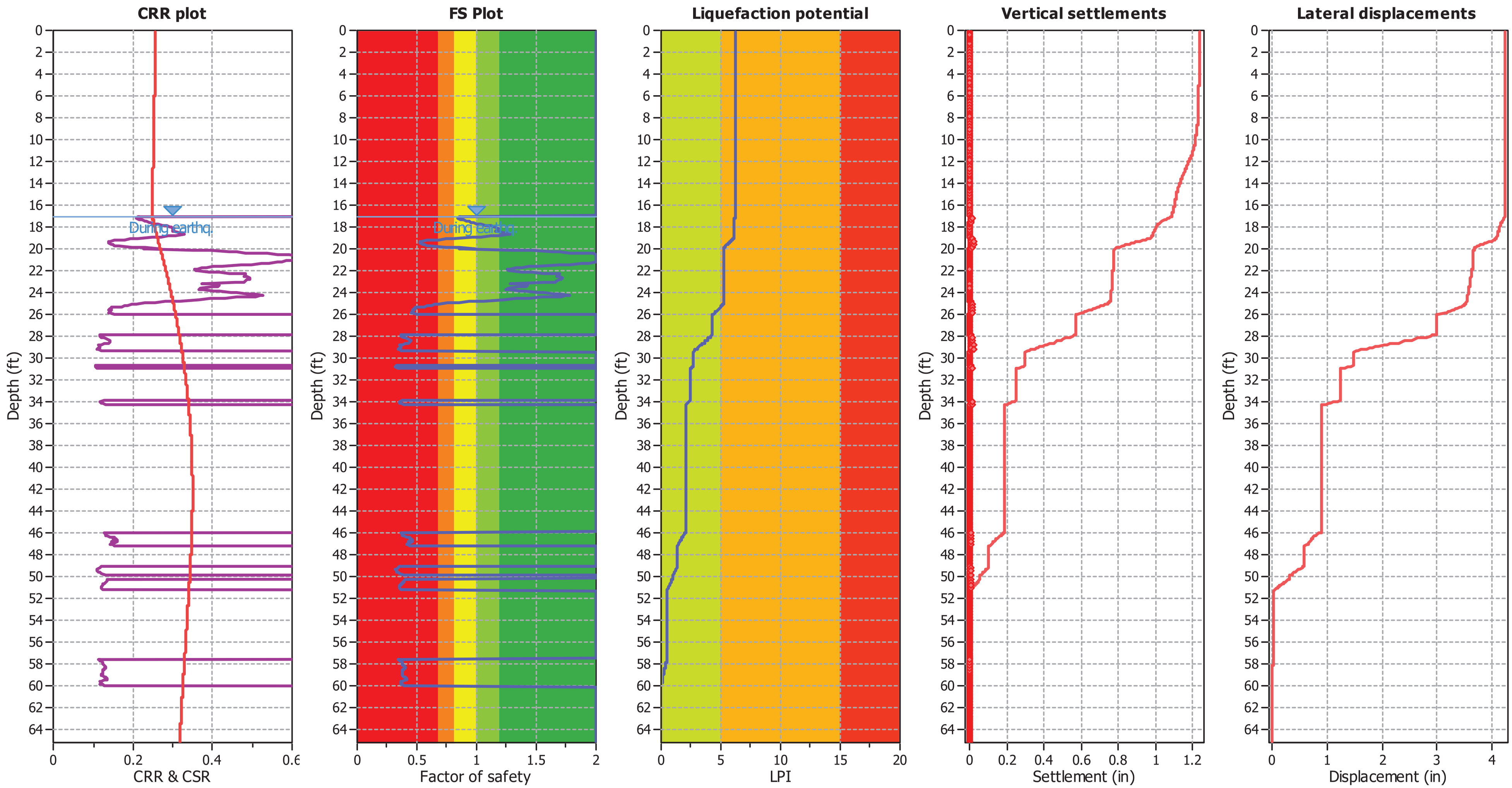
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

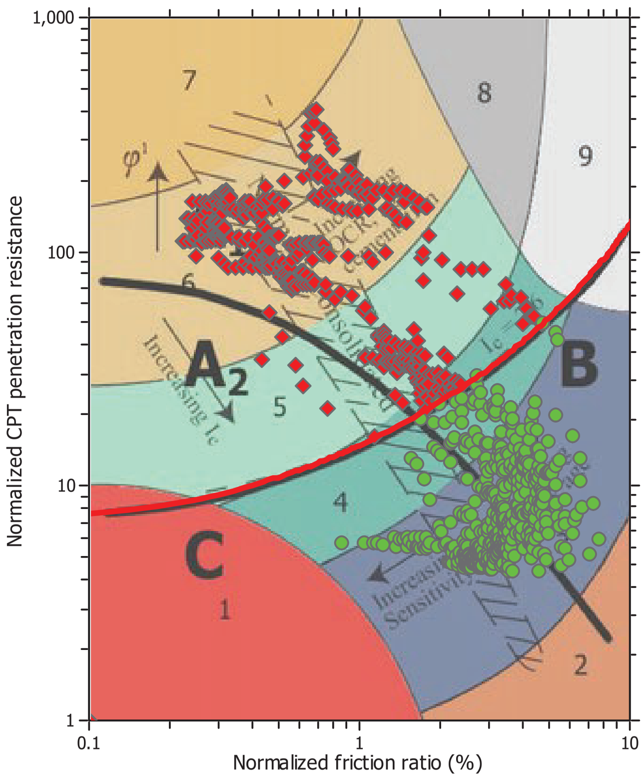
F.S. color scheme

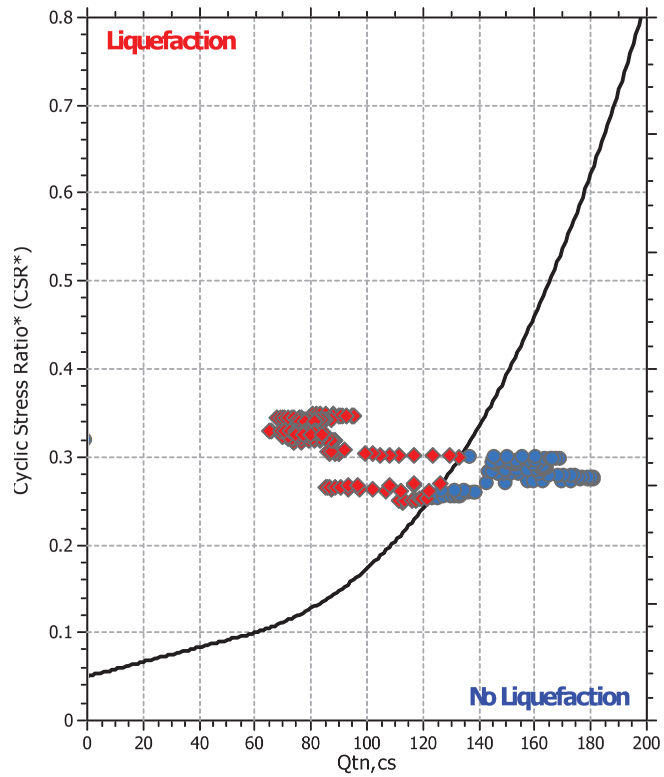
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

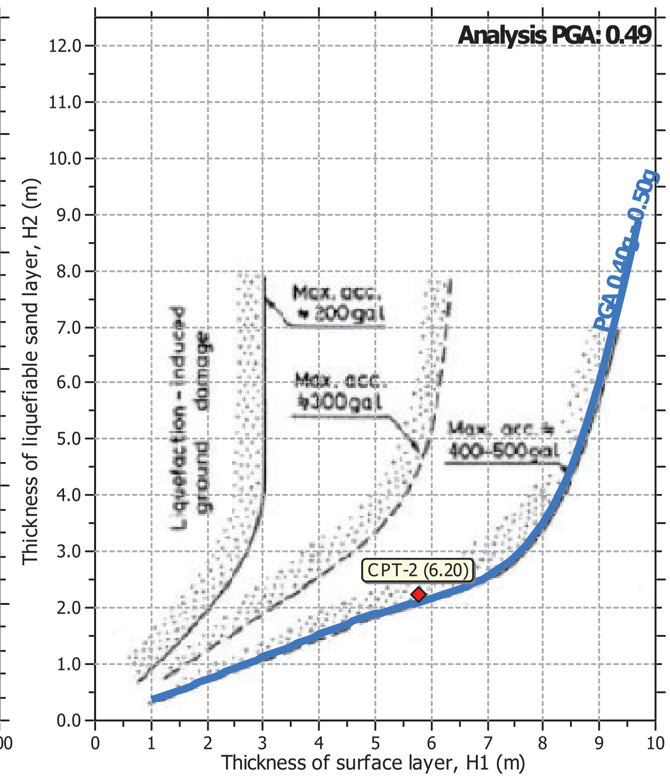
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



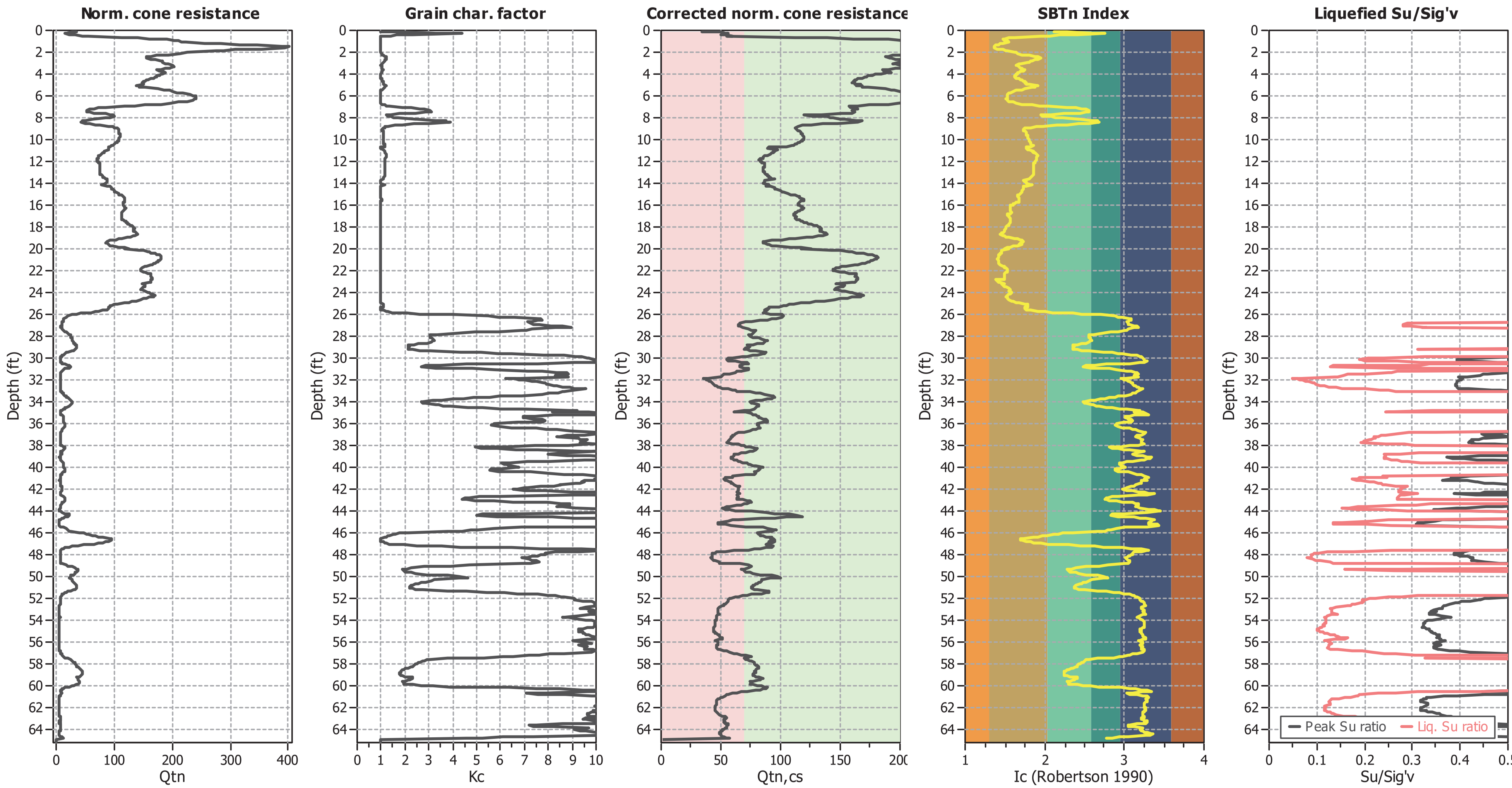




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	16.00 ft	Fill height:	N/A	Limit depth:	N/A

LIQUEFACTION ANALYSIS REPORT

Project title : 12807.002 Concordia at Los Arbolitos/Geotech **Location : Oceanside, CA**

CPT file : CPT-3

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
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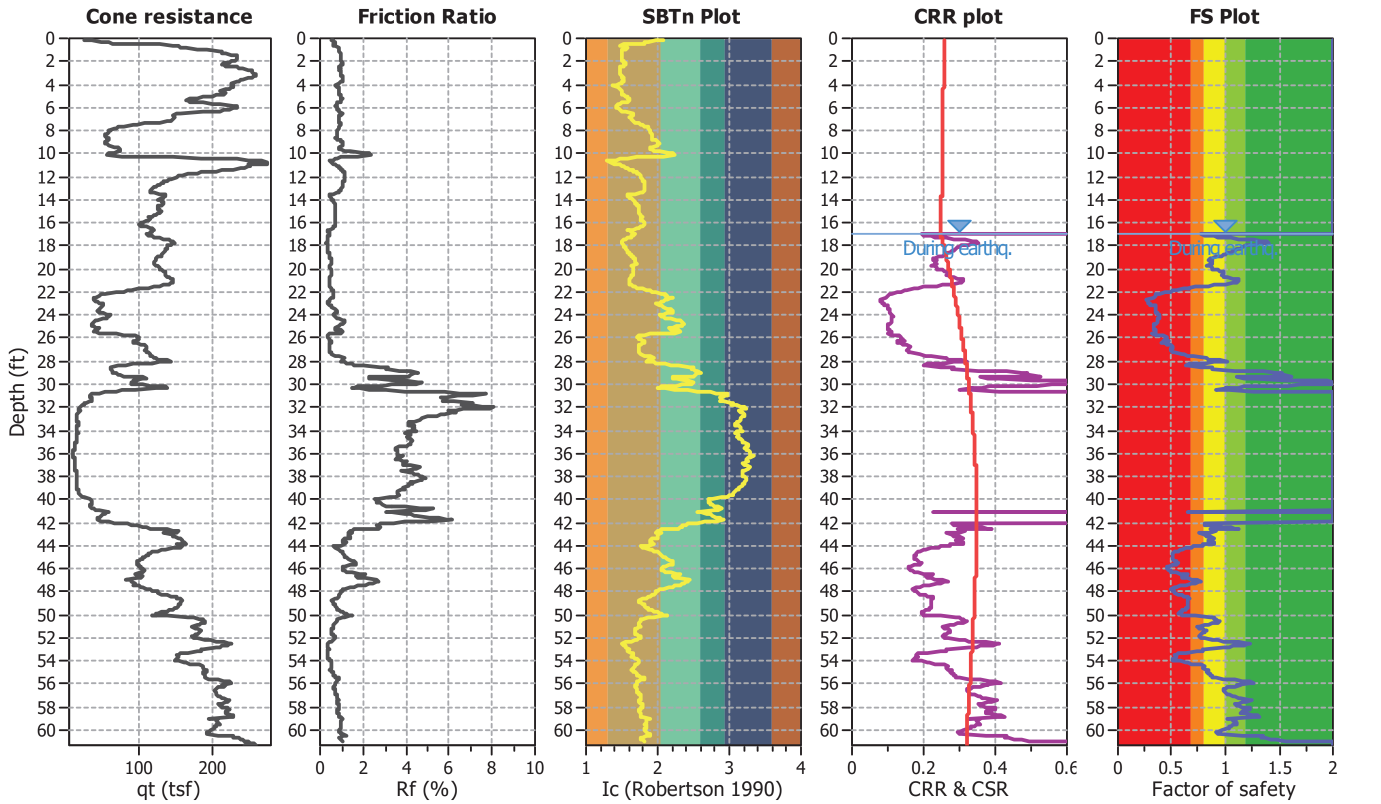
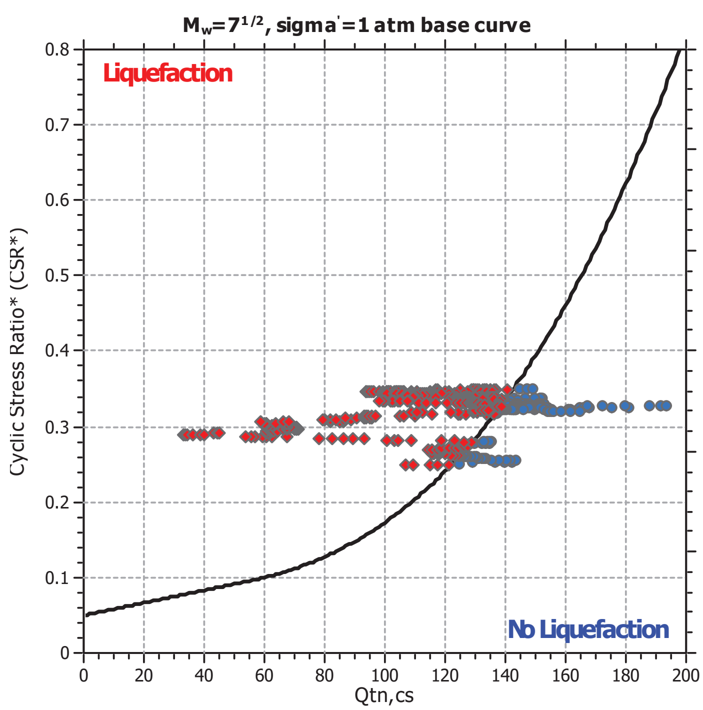
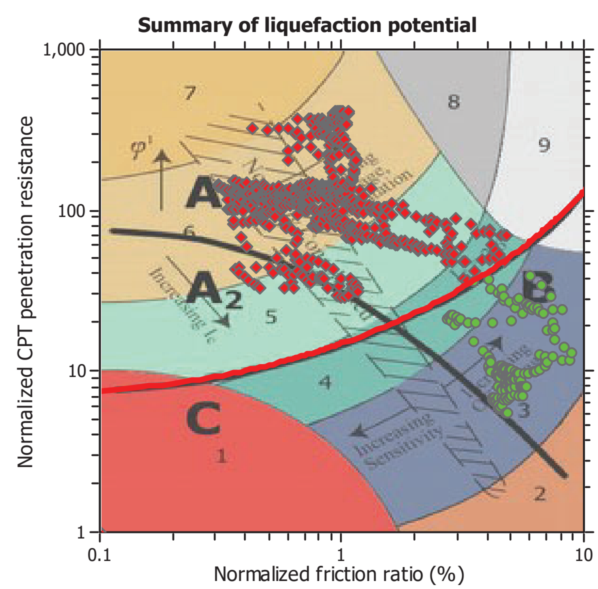
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.49	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

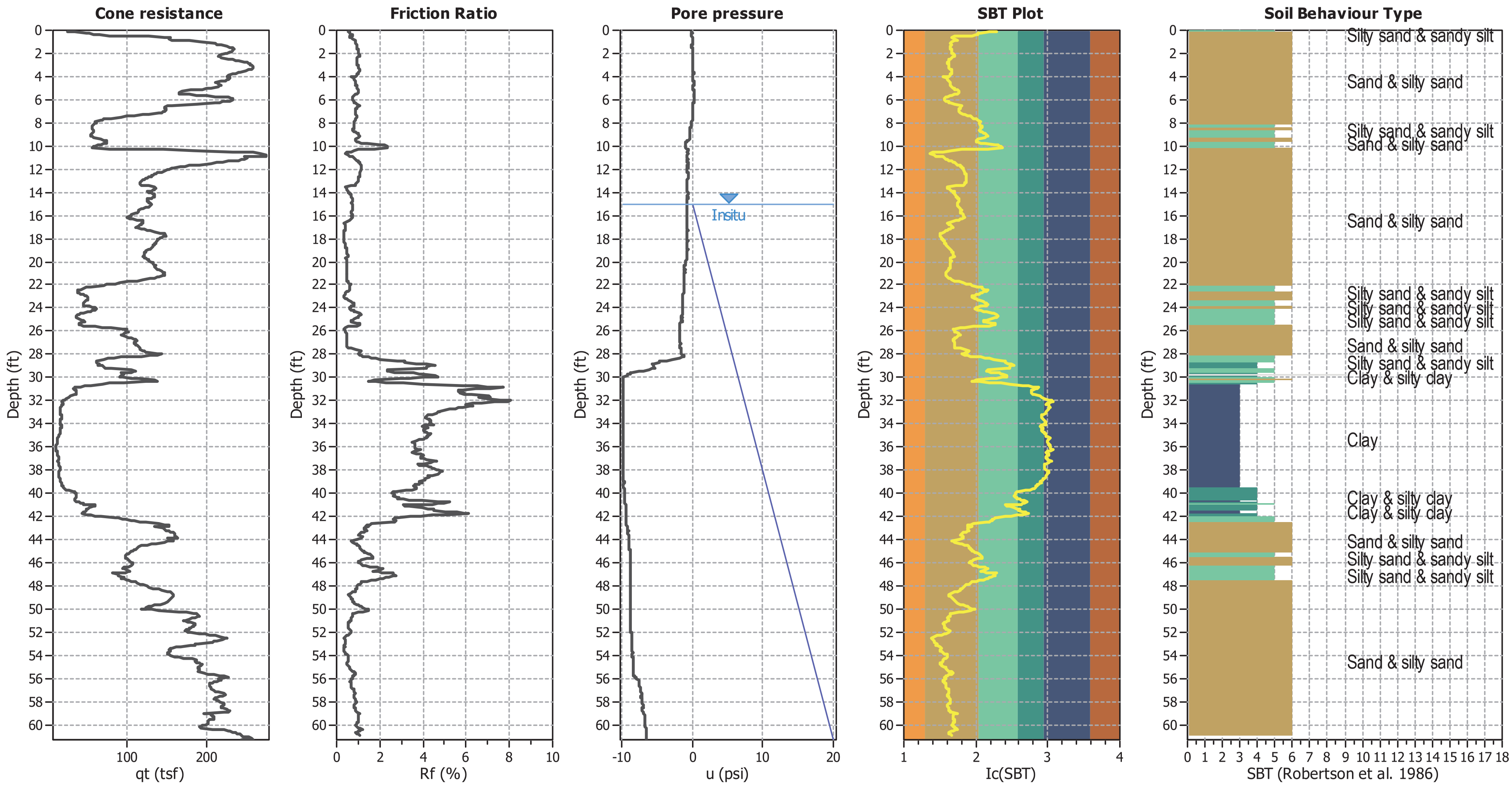






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and 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

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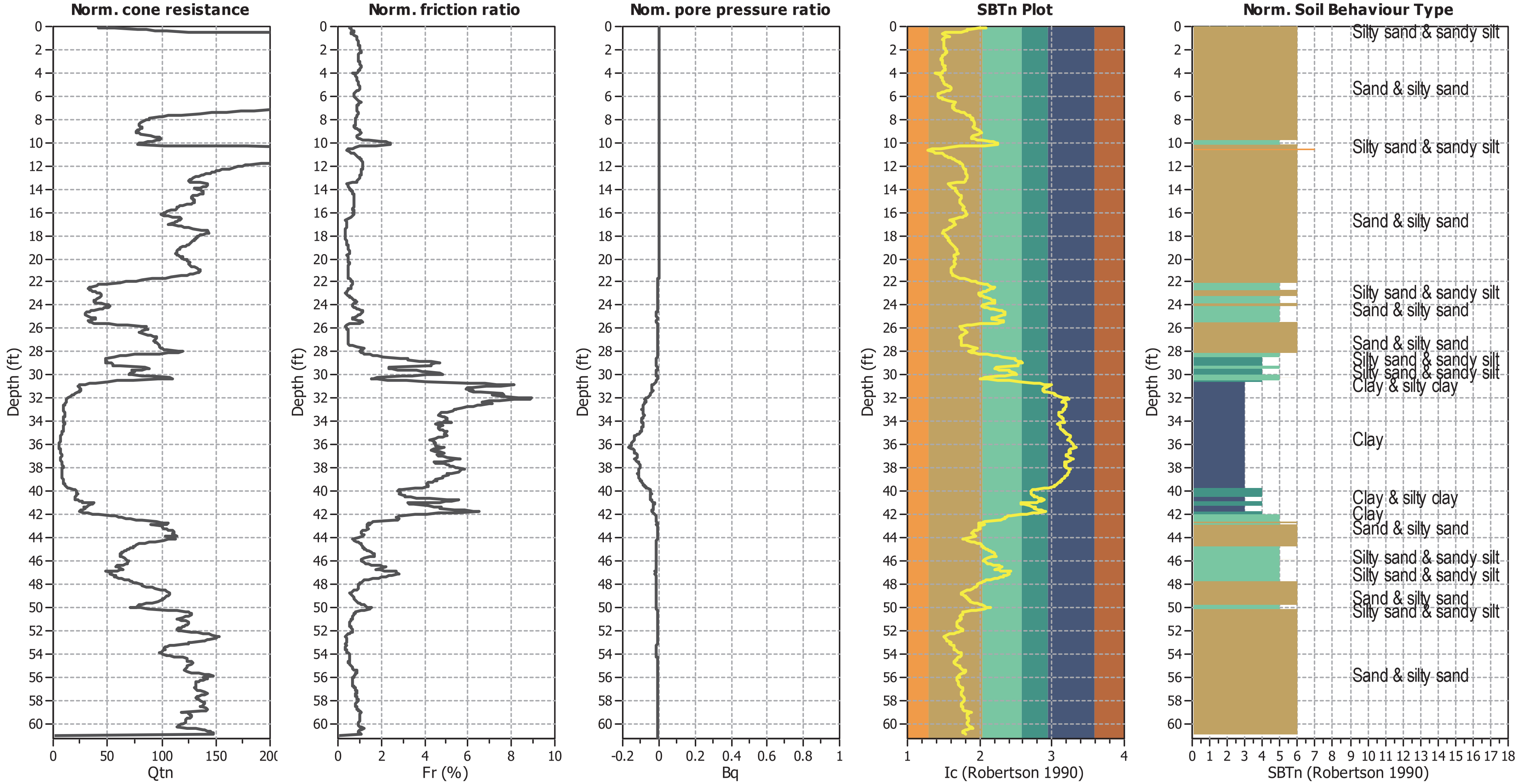
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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3. Clay to silty clay
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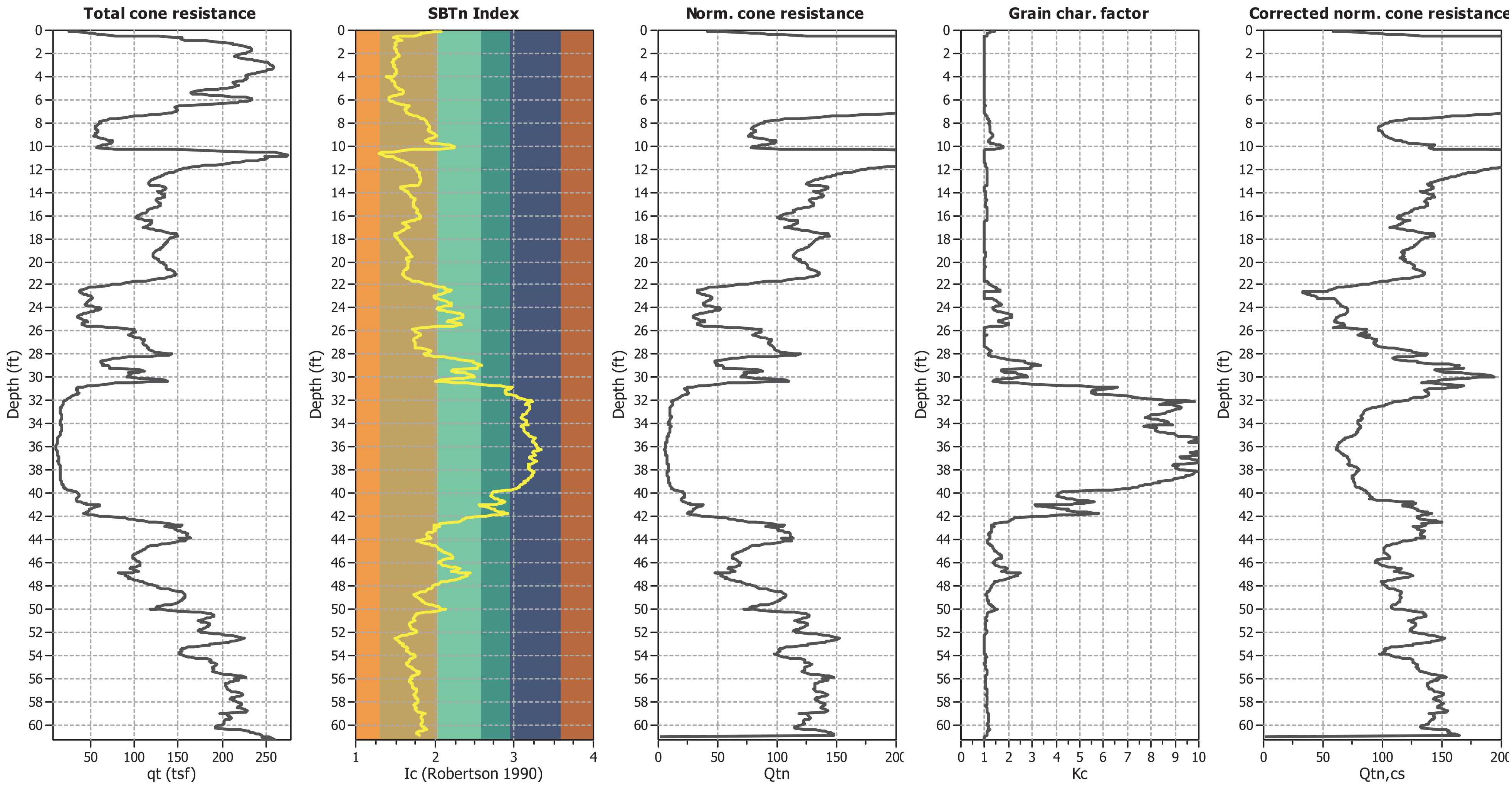
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

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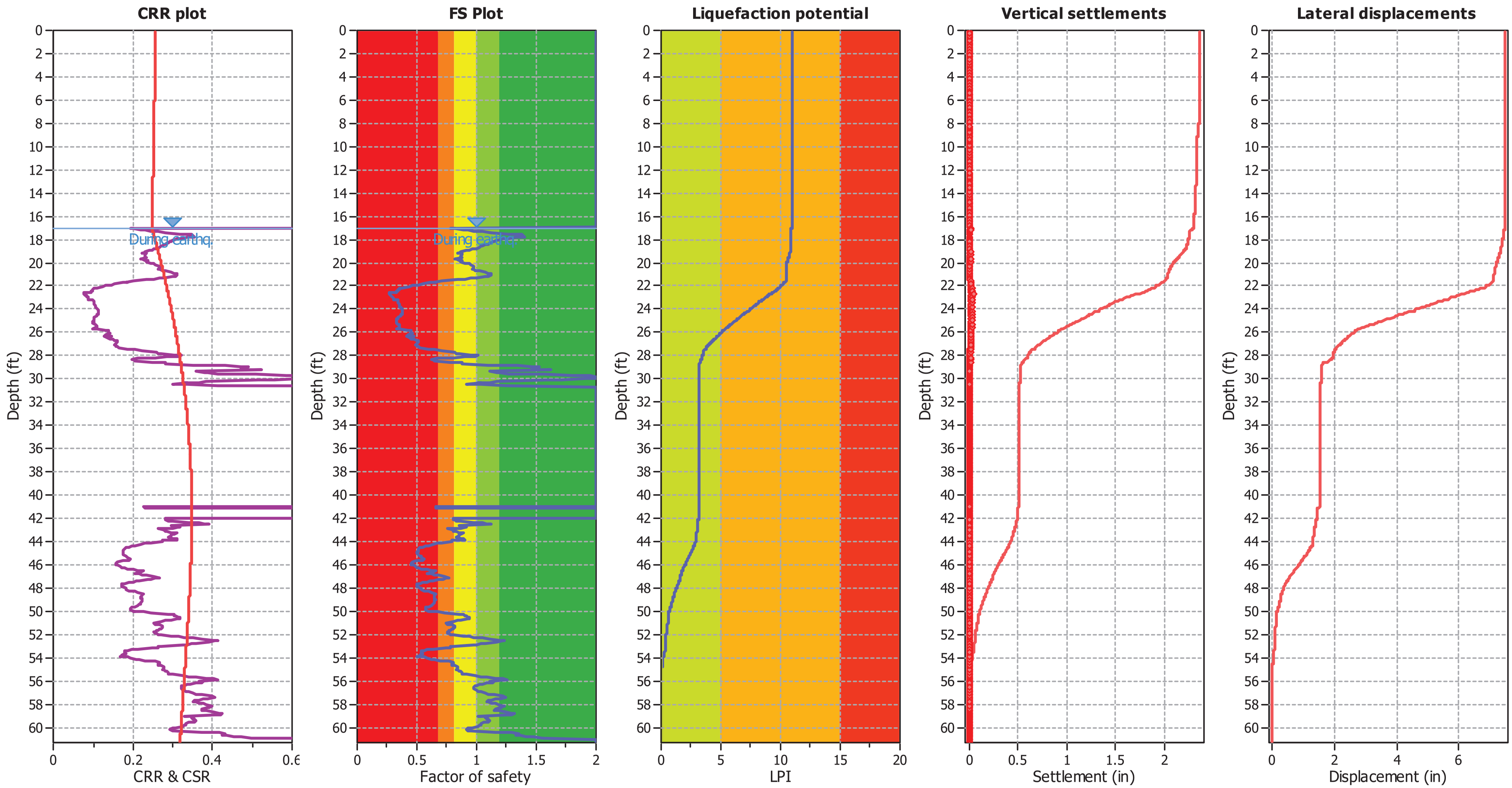
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

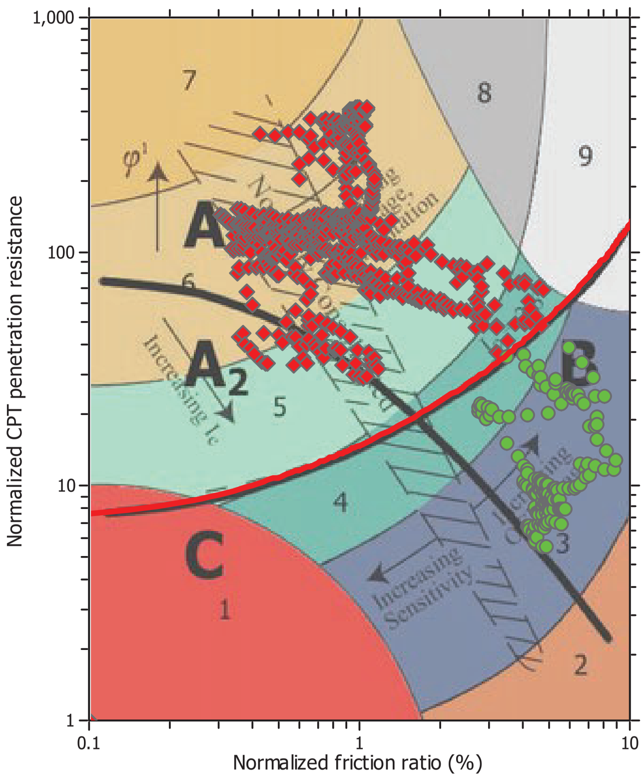
F.S. color scheme

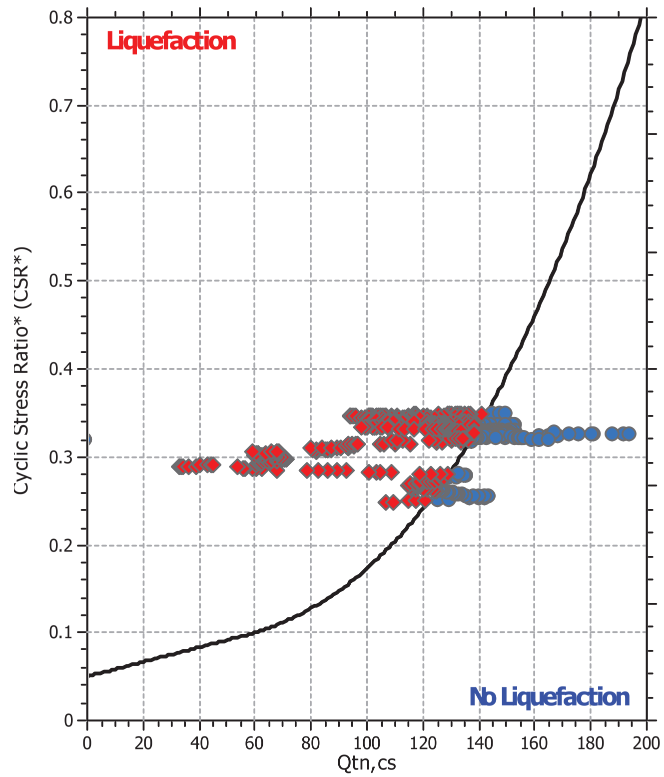
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

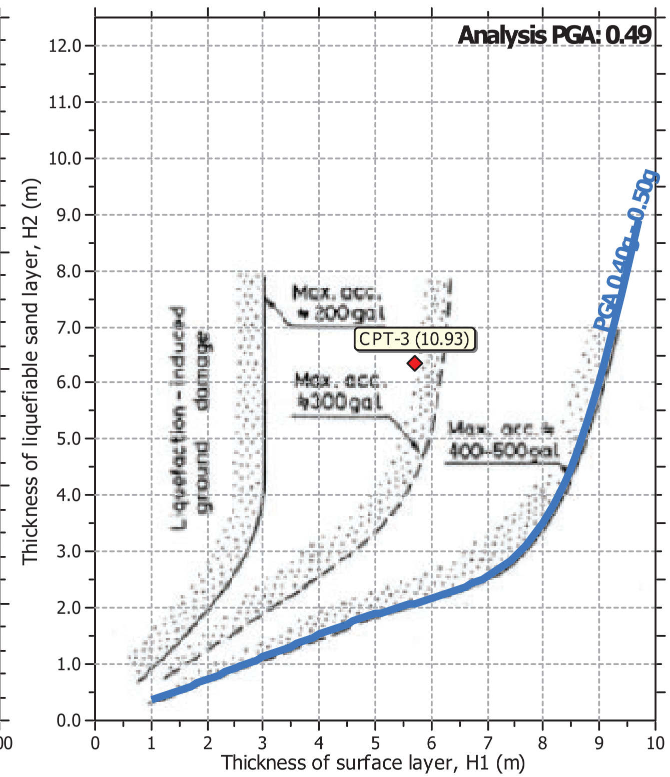
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



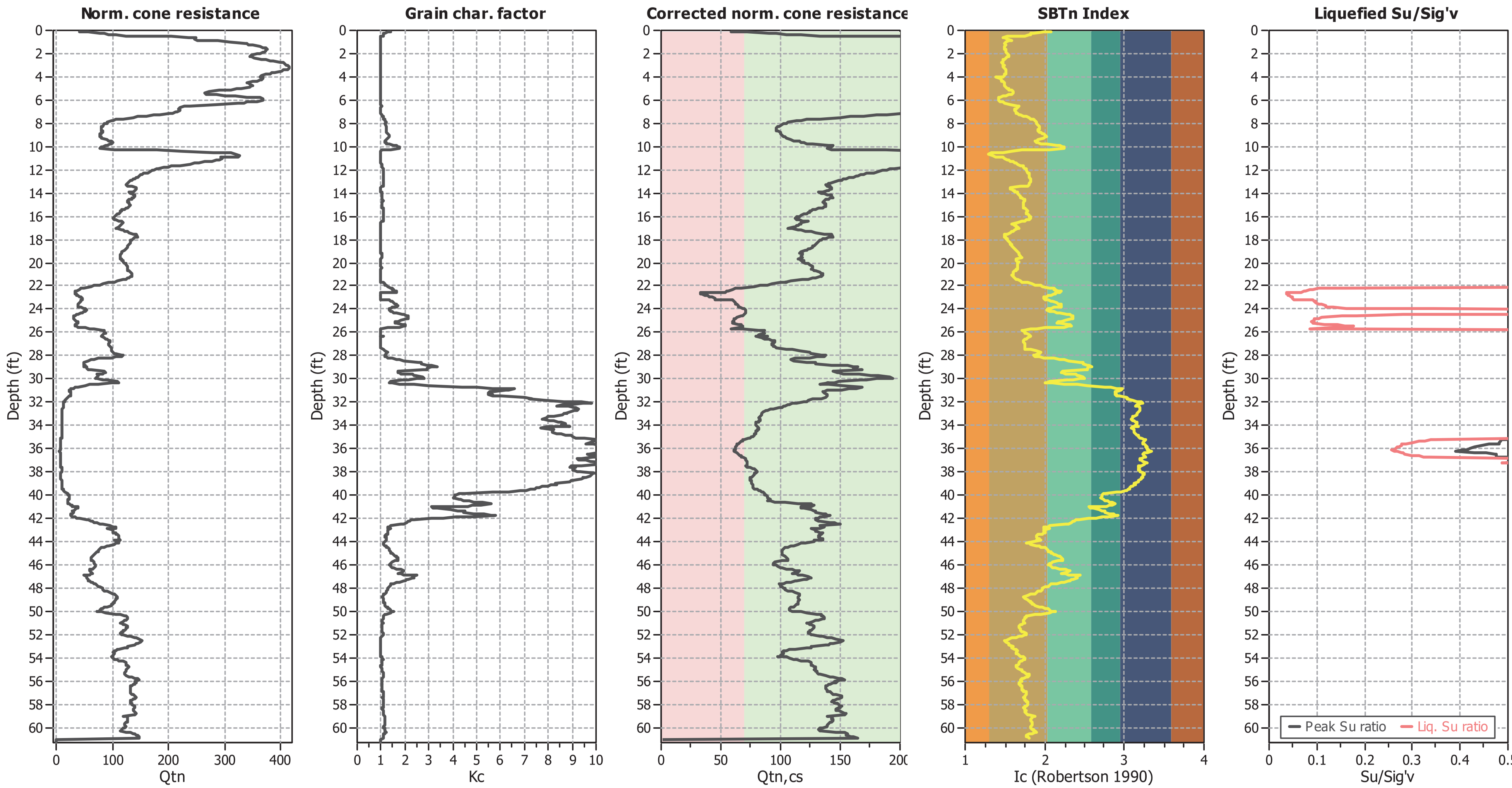




Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

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Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

LIQUEFACTION ANALYSIS REPORT

Project title : 12807.002 Concordia at Los Arbolitos/Geotech **Location : Oceanside, CA**

CPT file : CPT-4

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	15.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
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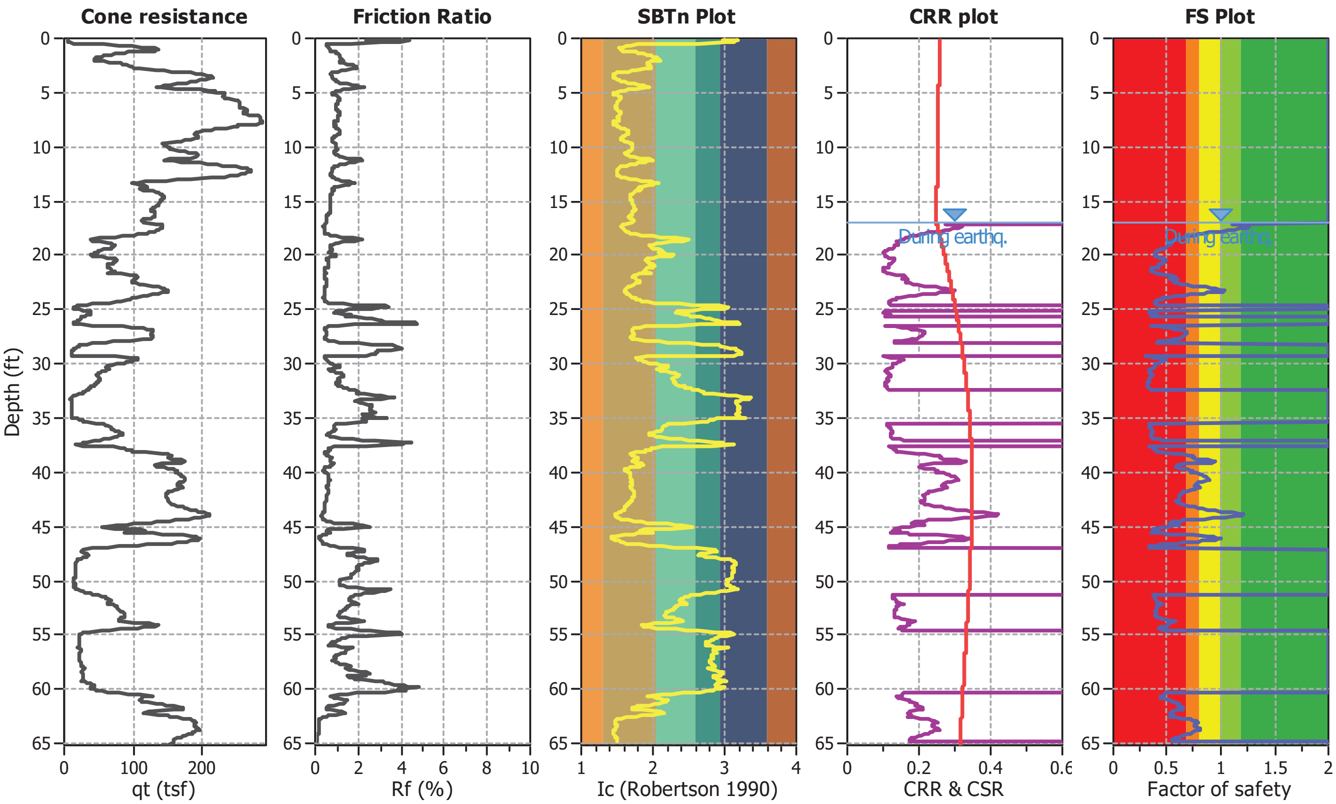
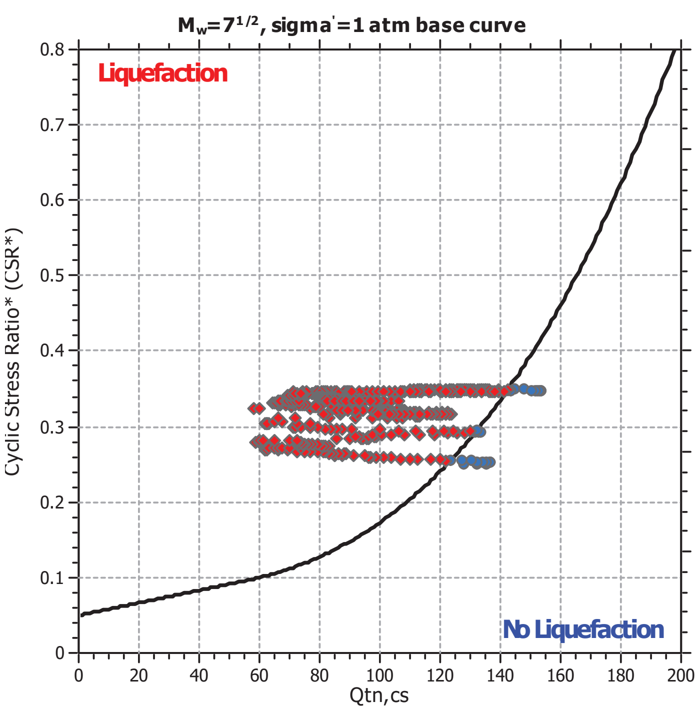
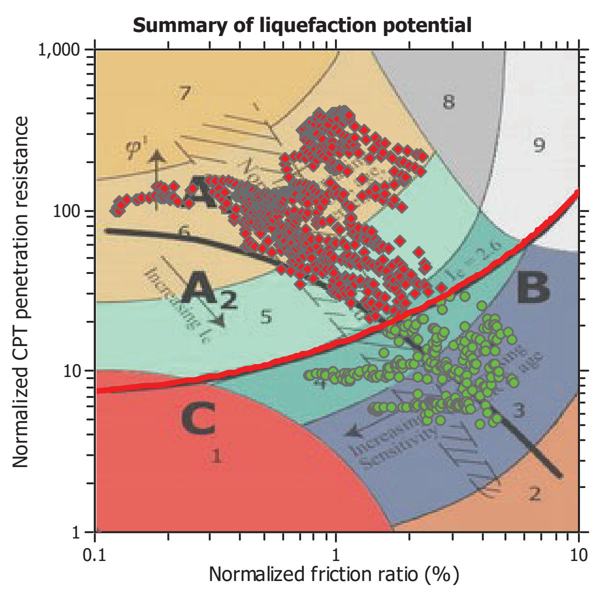
Points to test:	Based on Ic value	Average results interval:	1	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.49	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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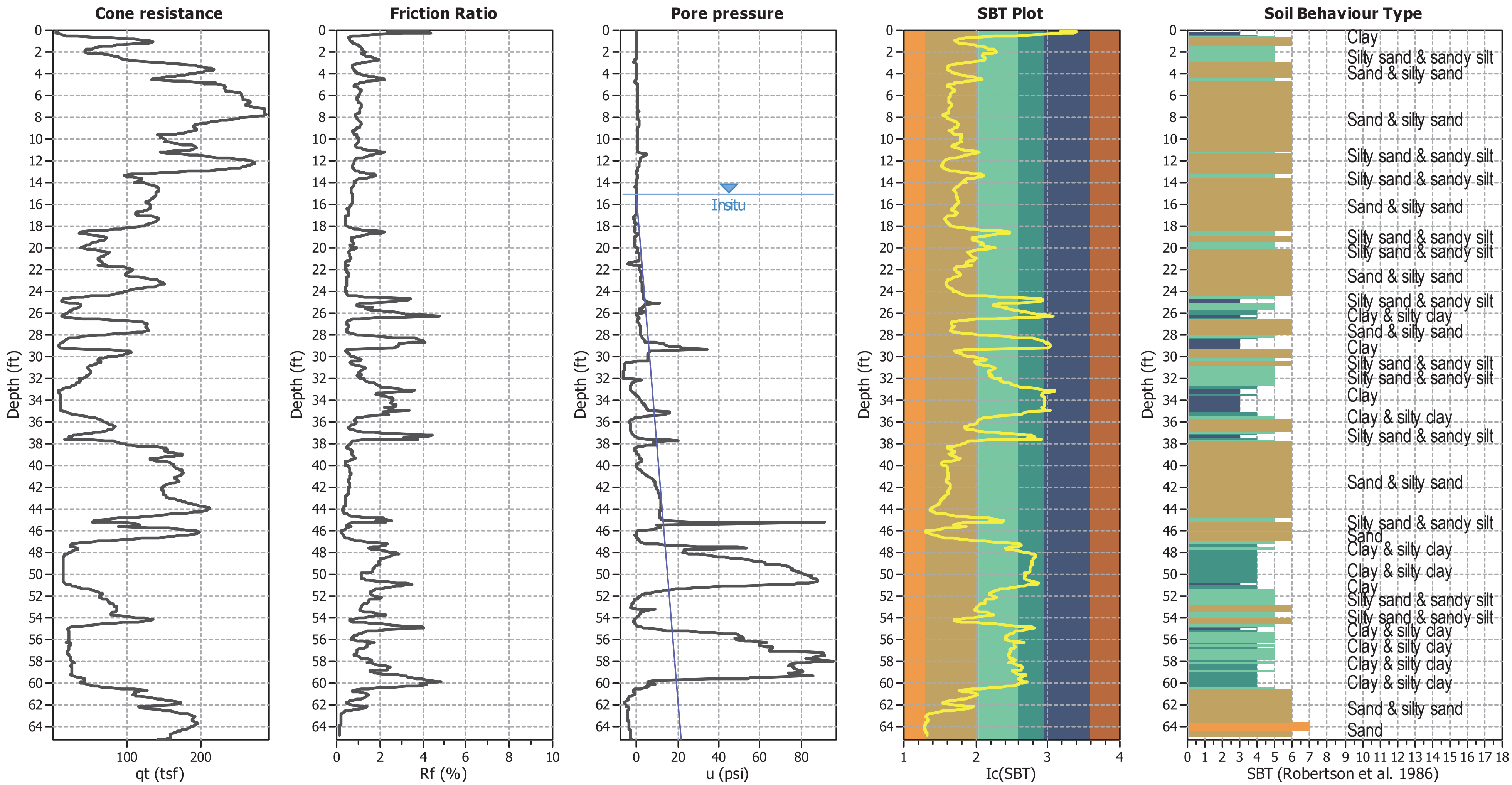
CPT basic interpretation plots



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Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
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Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

SBT legend

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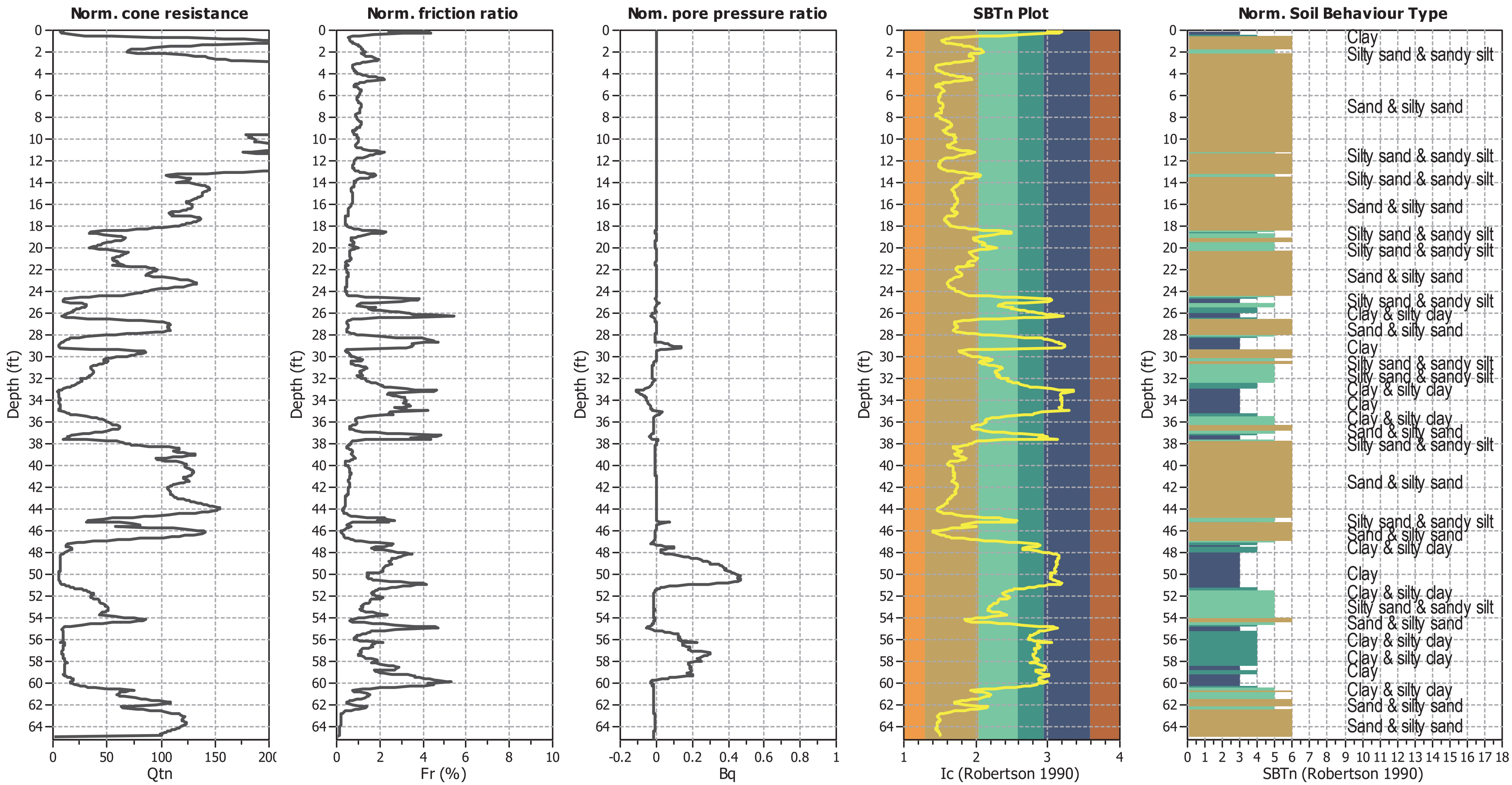
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



Input parameters and analysis data

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Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
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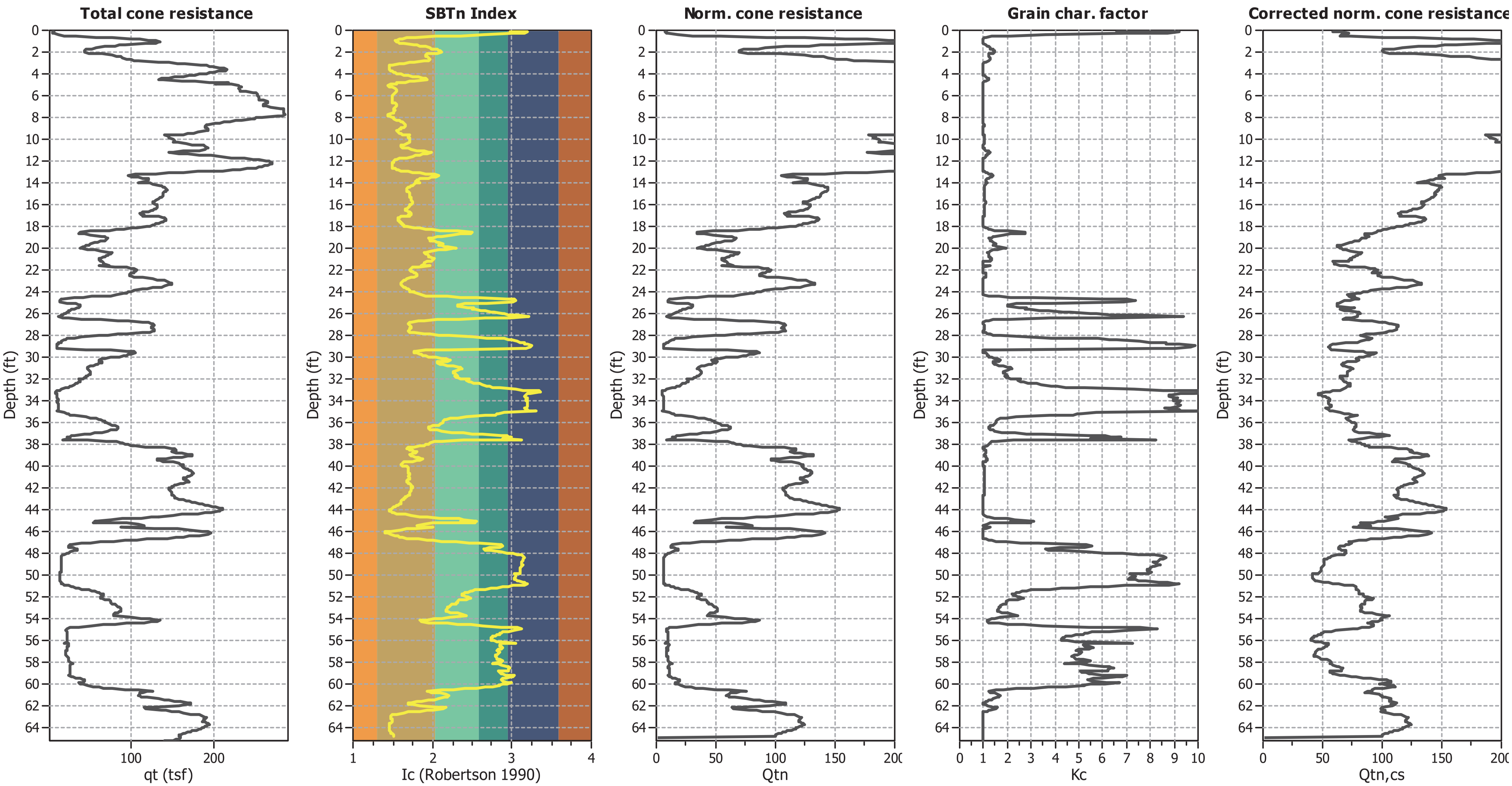
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

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Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
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Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
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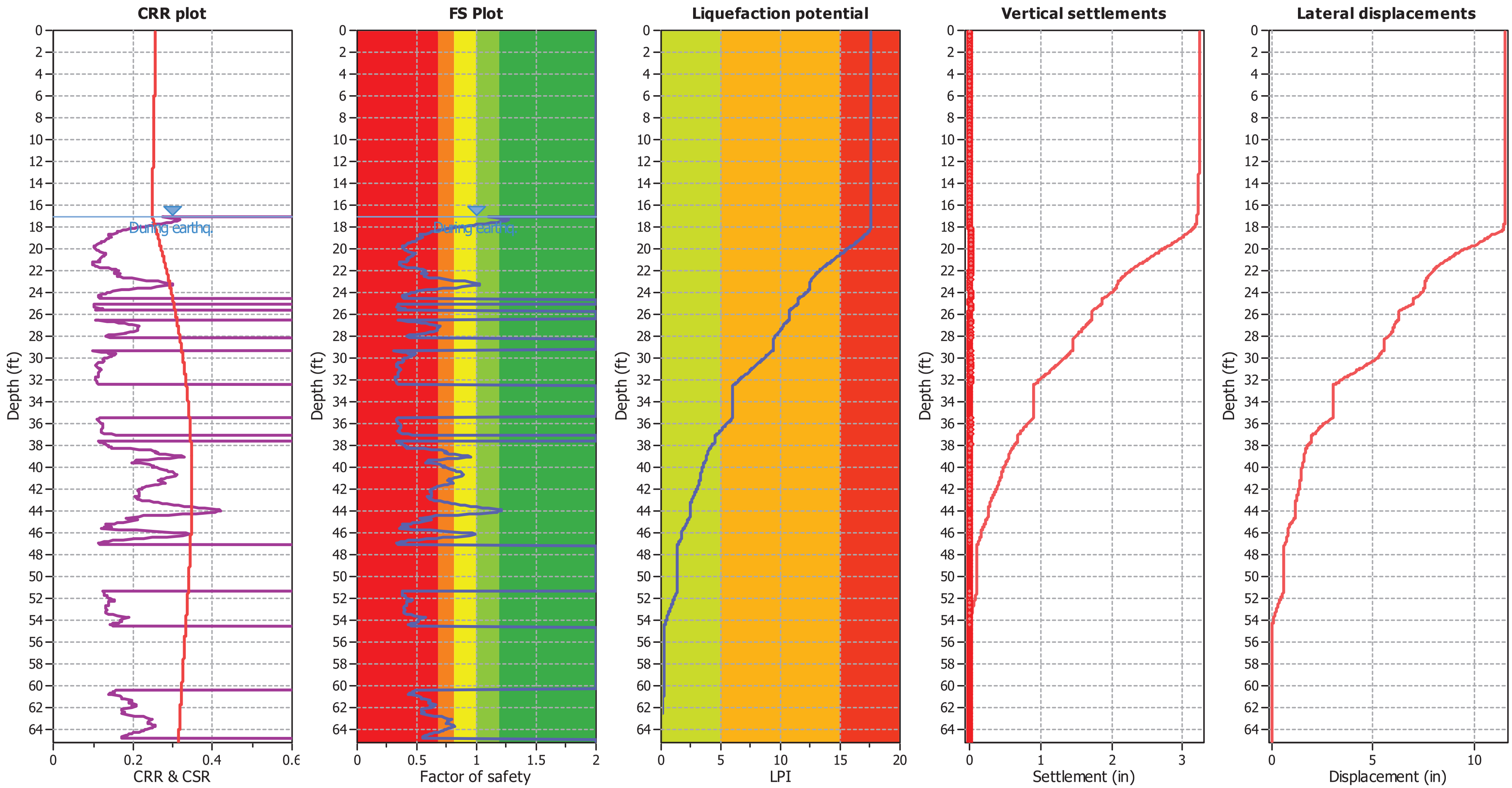
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
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LPI color scheme

- Very high risk
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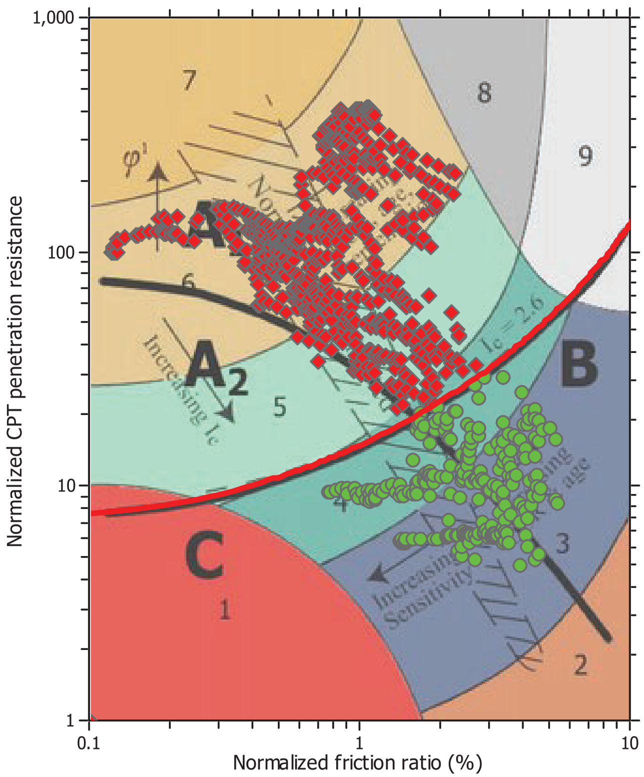
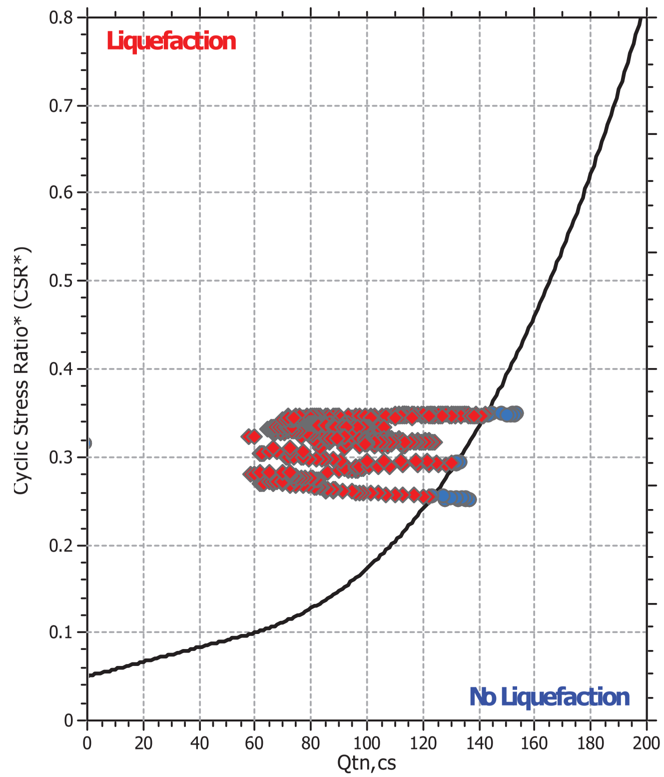
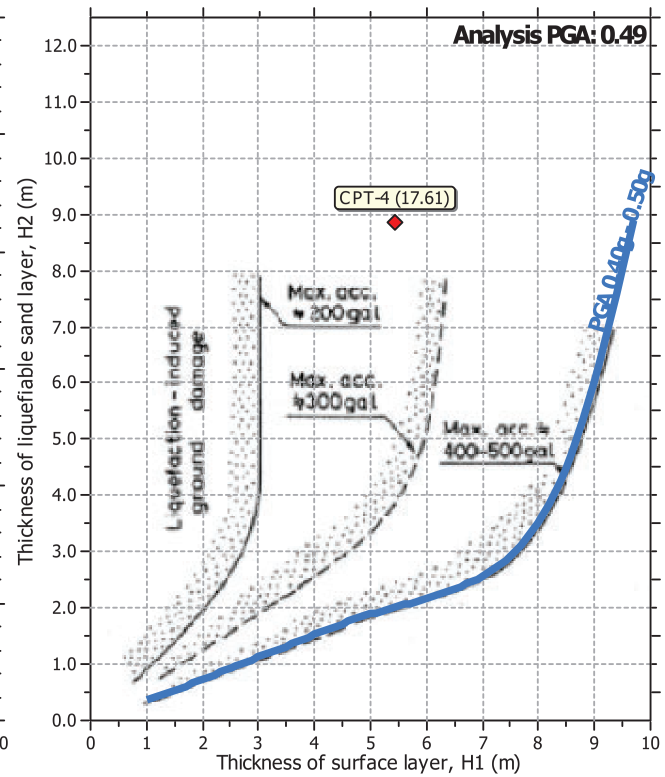
- Low risk

Liquefaction analysis summary plots







Input parameters and analysis data

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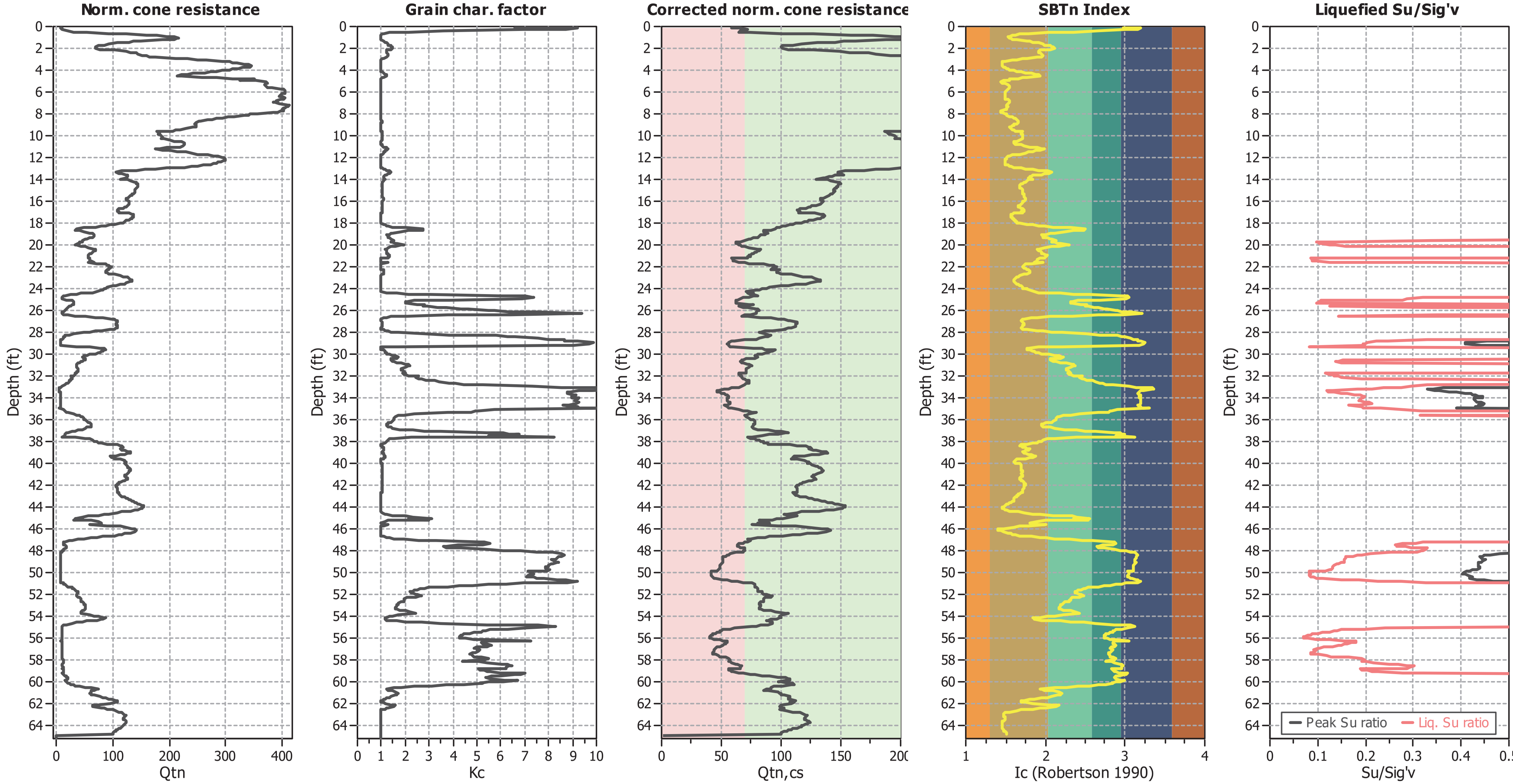


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	17.00 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	1	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.49	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	15.00 ft	Fill height:	N/A	Limit depth:	N/A

Appendix E
General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

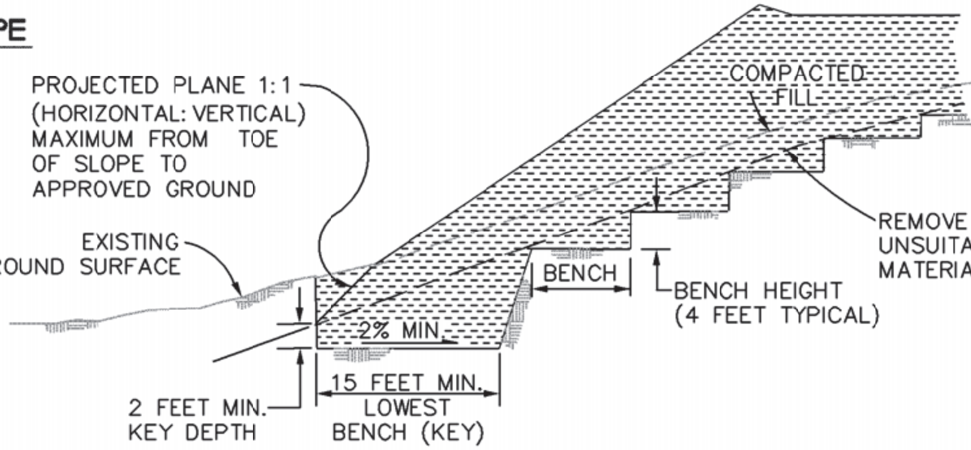
7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

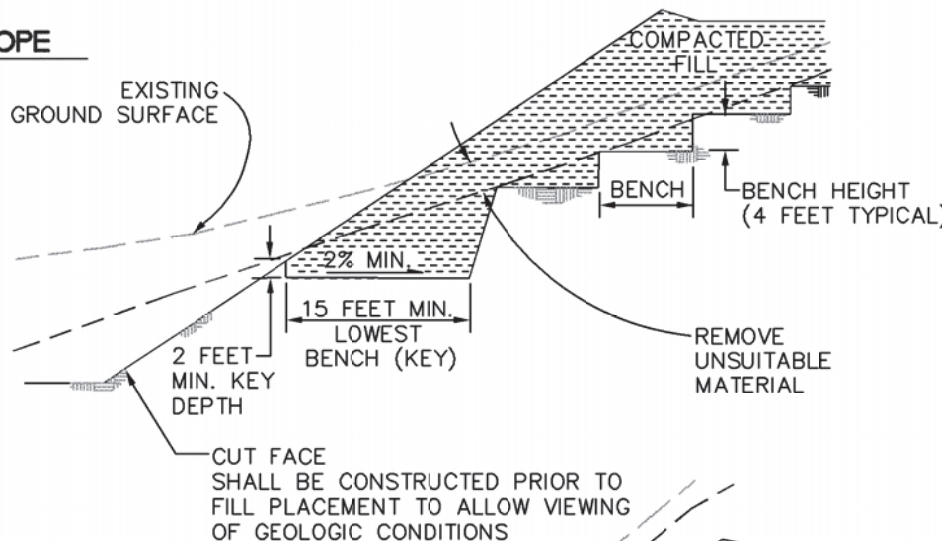
7.4 Observation and Testing

The densification of the bedding around the conduits shall be observed by the Geotechnical Consultant.

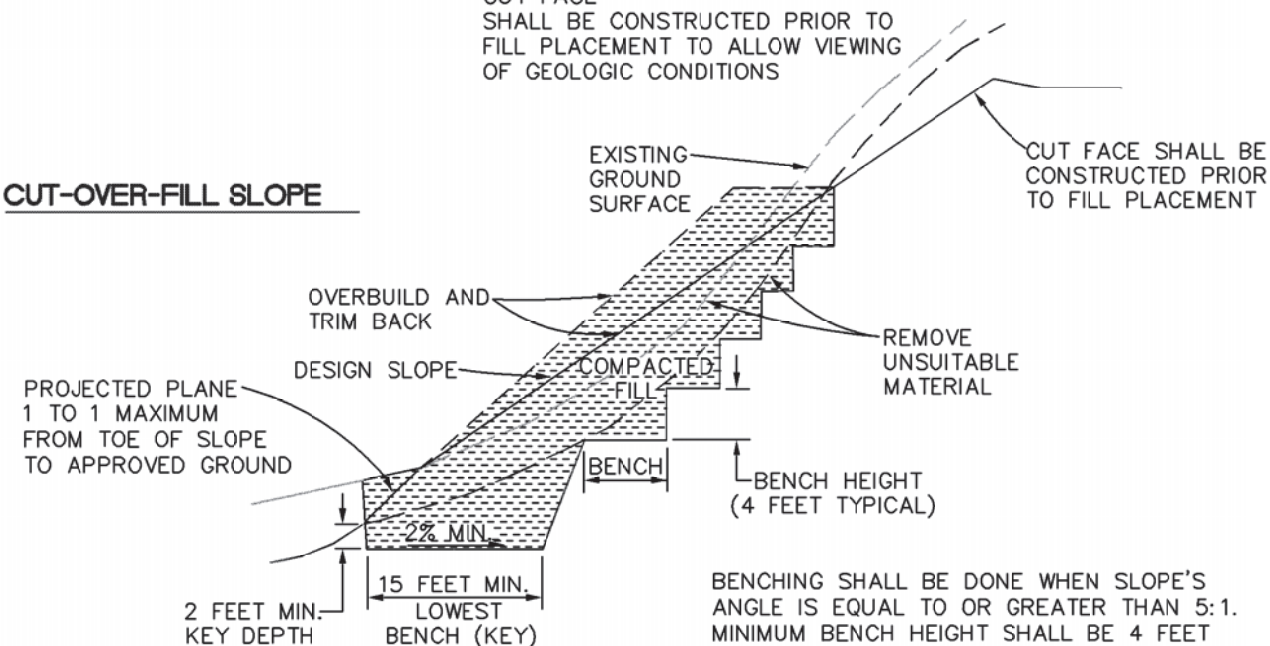
FILL SLOPE



FILL-OVER-CUT SLOPE



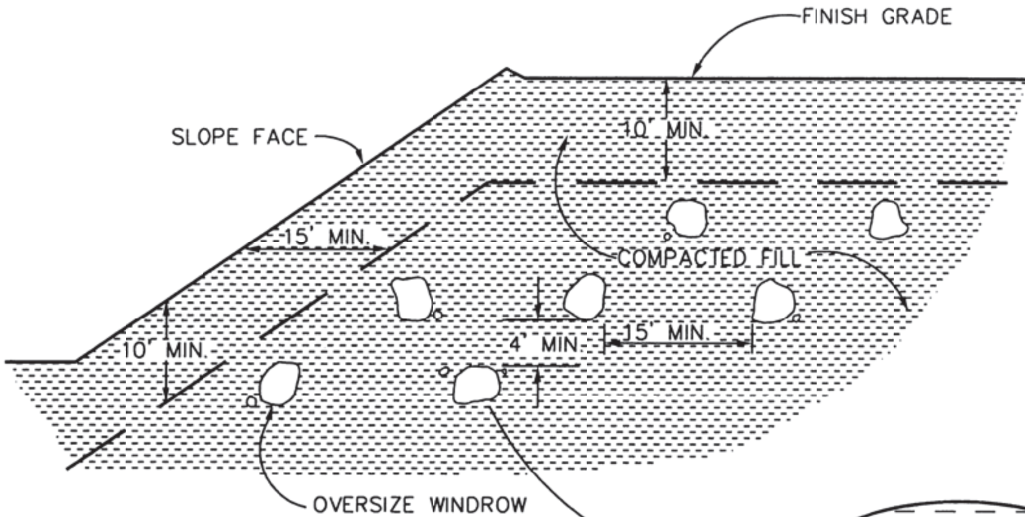
CUT-OVER-FILL SLOPE

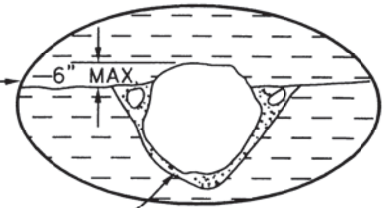


BENCHING SHALL BE DONE WHEN SLOPE'S ANGLE IS EQUAL TO OR GREATER THAN 5:1. MINIMUM BENCH HEIGHT SHALL BE 4 FEET AND MINIMUM FILL WIDTH SHALL BE 9 FEET.

KEYING AND BENCHING

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL A

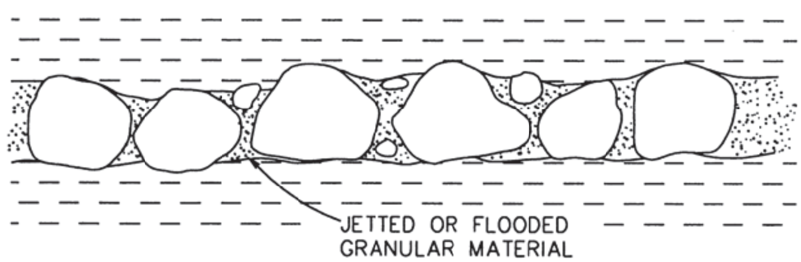




GRANULAR MATERIAL TO BE DENSIFIED IN PLACE BY FLOODING OR JETTING.

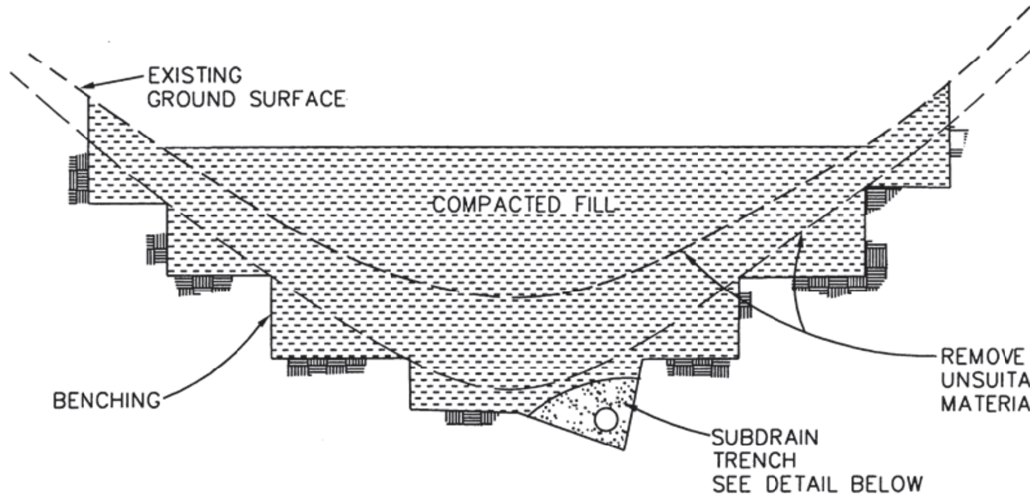
DETAIL

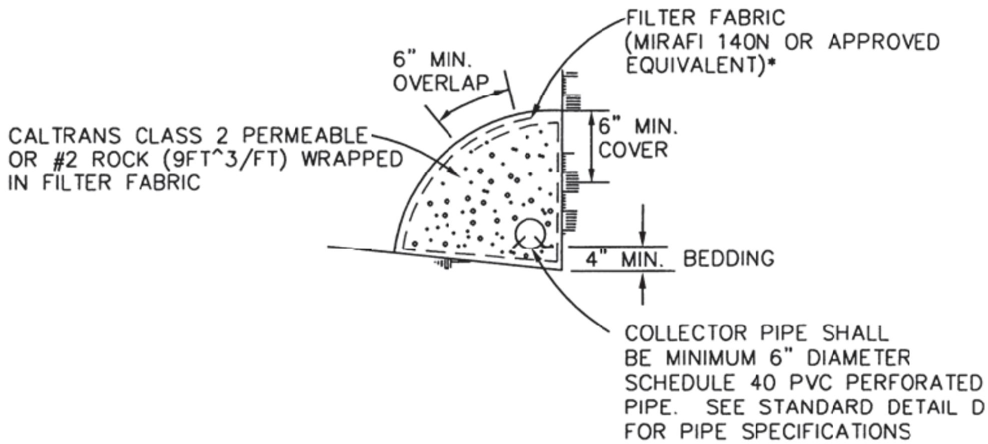
* OVERSIZE ROCK IS LARGER THAN 8 INCHES IN LARGEST DIMENSION.
* EXCAVATE A TRENCH IN THE COMPACTED FILL DEEP ENOUGH TO BURY ALL THE ROCK.
* BACKFILL WITH GRANULAR SOIL JETTED OR FLOODED IN PLACE TO FILL ALL THE VOIDS.
* DO NOT BURY ROCK WITHIN 10 FEET OF FINISH GRADE.
* WINDROW OF BURIED ROCK SHALL BE PARALLEL TO THE FINISHED SLOPE.



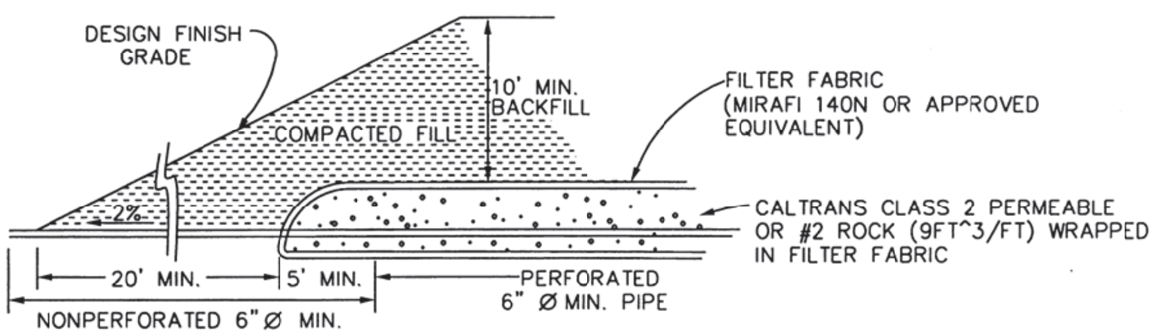
TYPICAL PROFILE ALONG WINDROW

OVERSIZE ROCK DISPOSAL	GENERAL EARTHWORK AND GRADING SPECIFICATIONS STANDARD DETAIL B





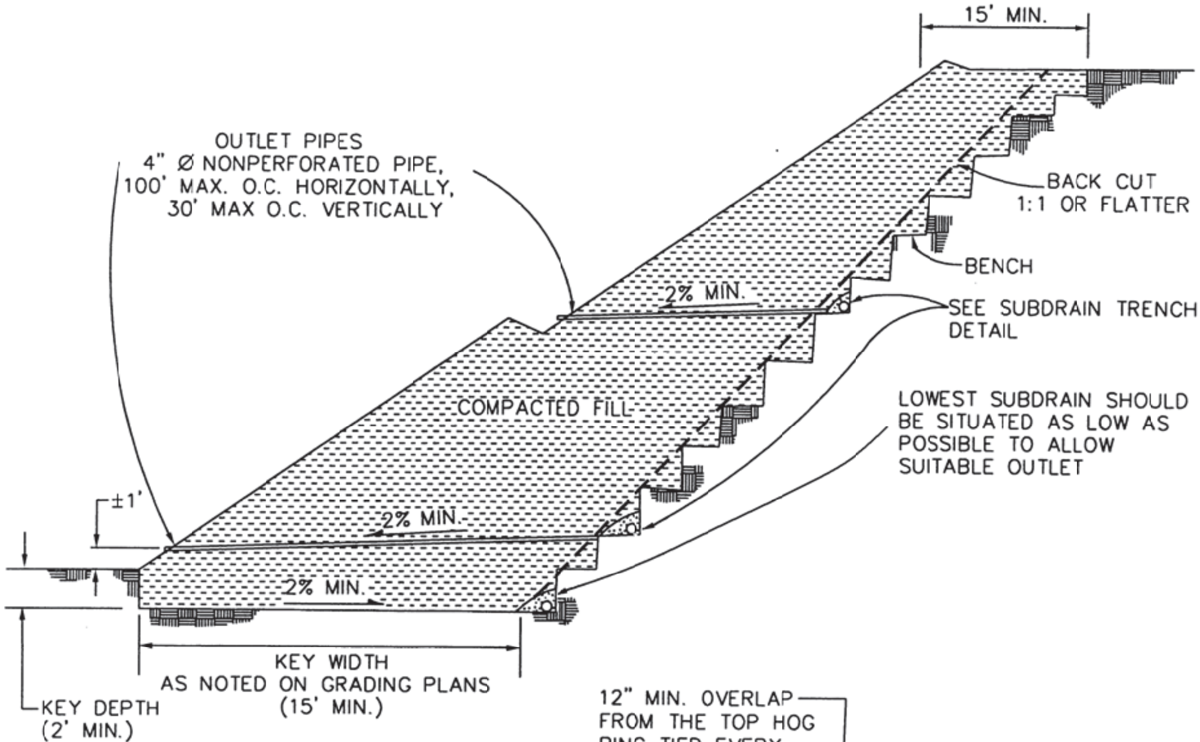
SUBDRAIN DETAIL

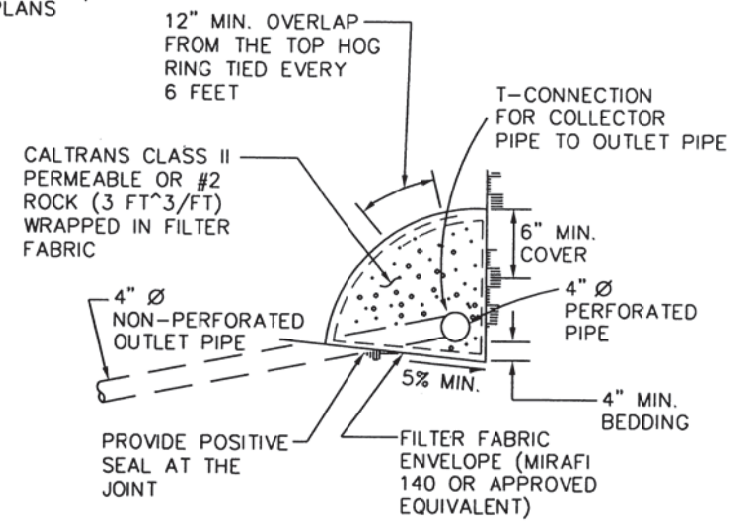


DETAIL OF CANYON SUBDRAIN OUTLET

CANYON SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL C





SUBDRAIN TRENCH DETAIL

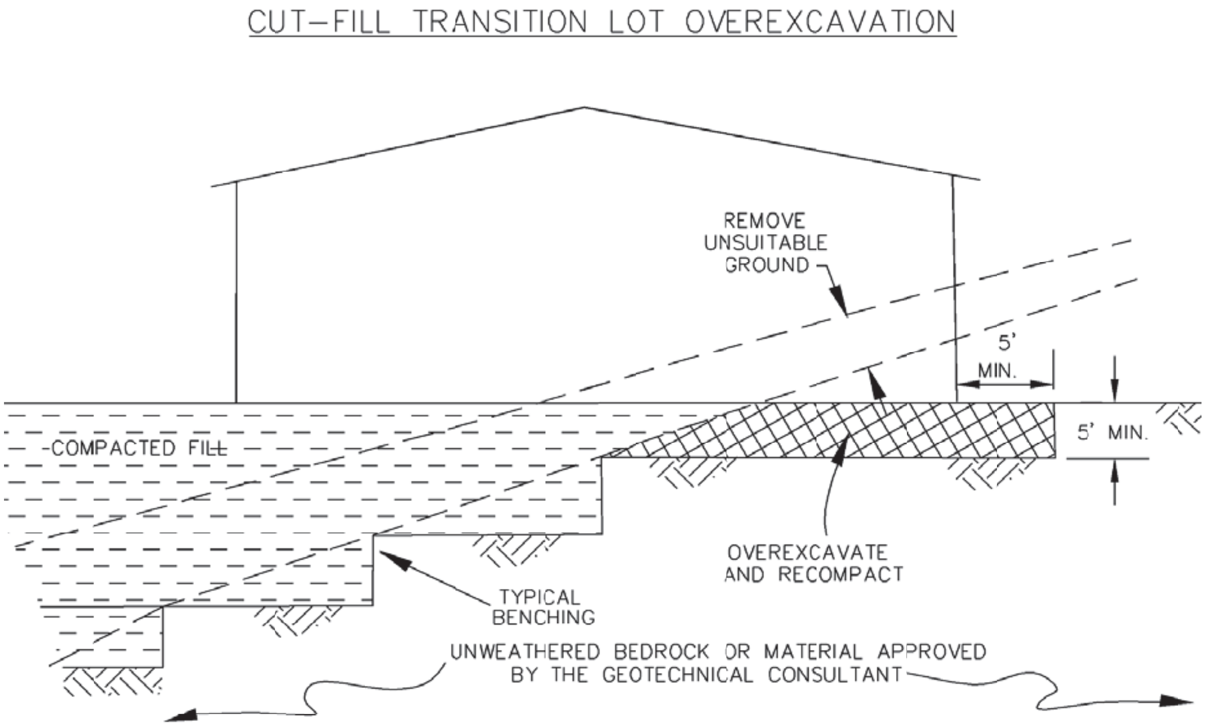
SUBDRAIN INSTALLATION – subdrain collector pipe shall be installed with perforation down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drill holes are used. All subdrain pipes shall have a gradient of at least 2% towards the outlet.

SUBDRAIN PIPE – Subdrain pipe shall be ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40, or ASTM D3034, SDR 23.5, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe.

All outlet pipe shall be placed in a trench no wider than twice the subdrain pipe.

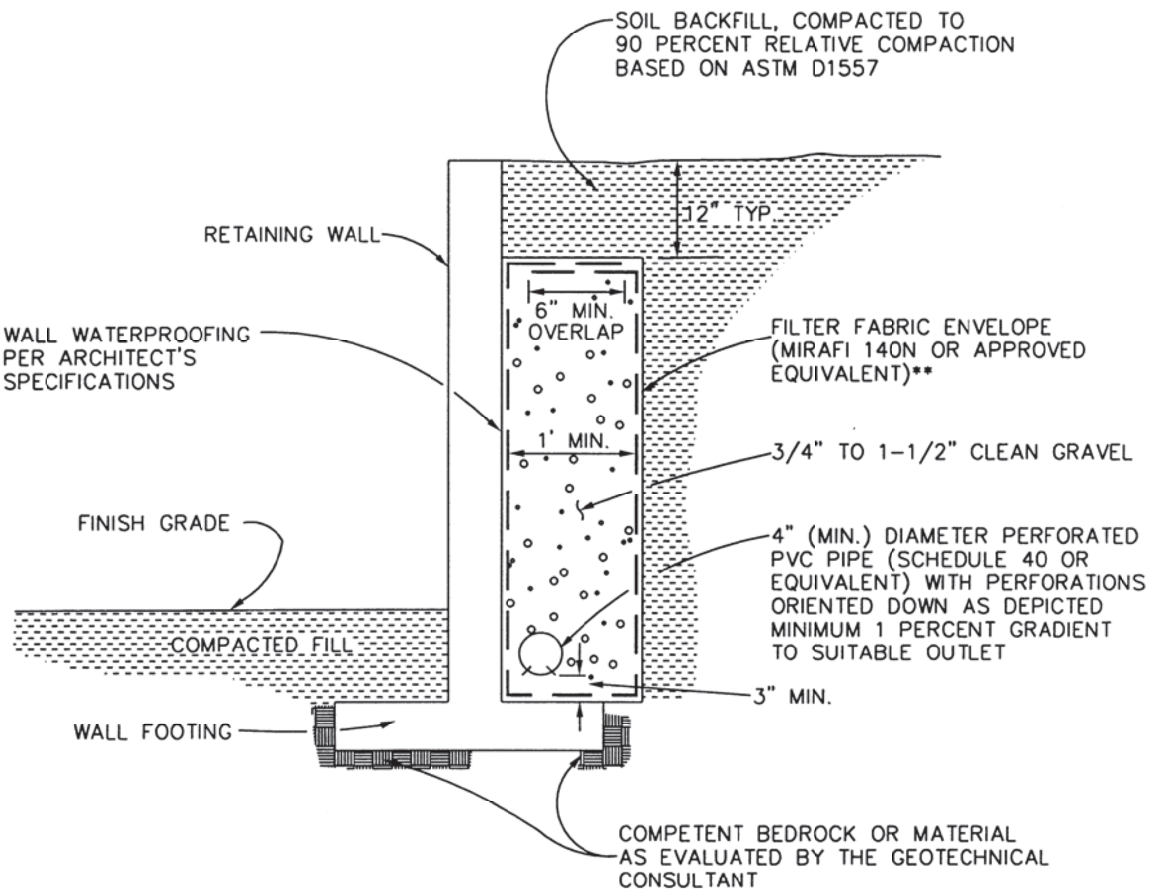
BUTTRESS OR REPLACEMENT FILL SUBDRAINS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL D



TRANSITION LOT FILLS

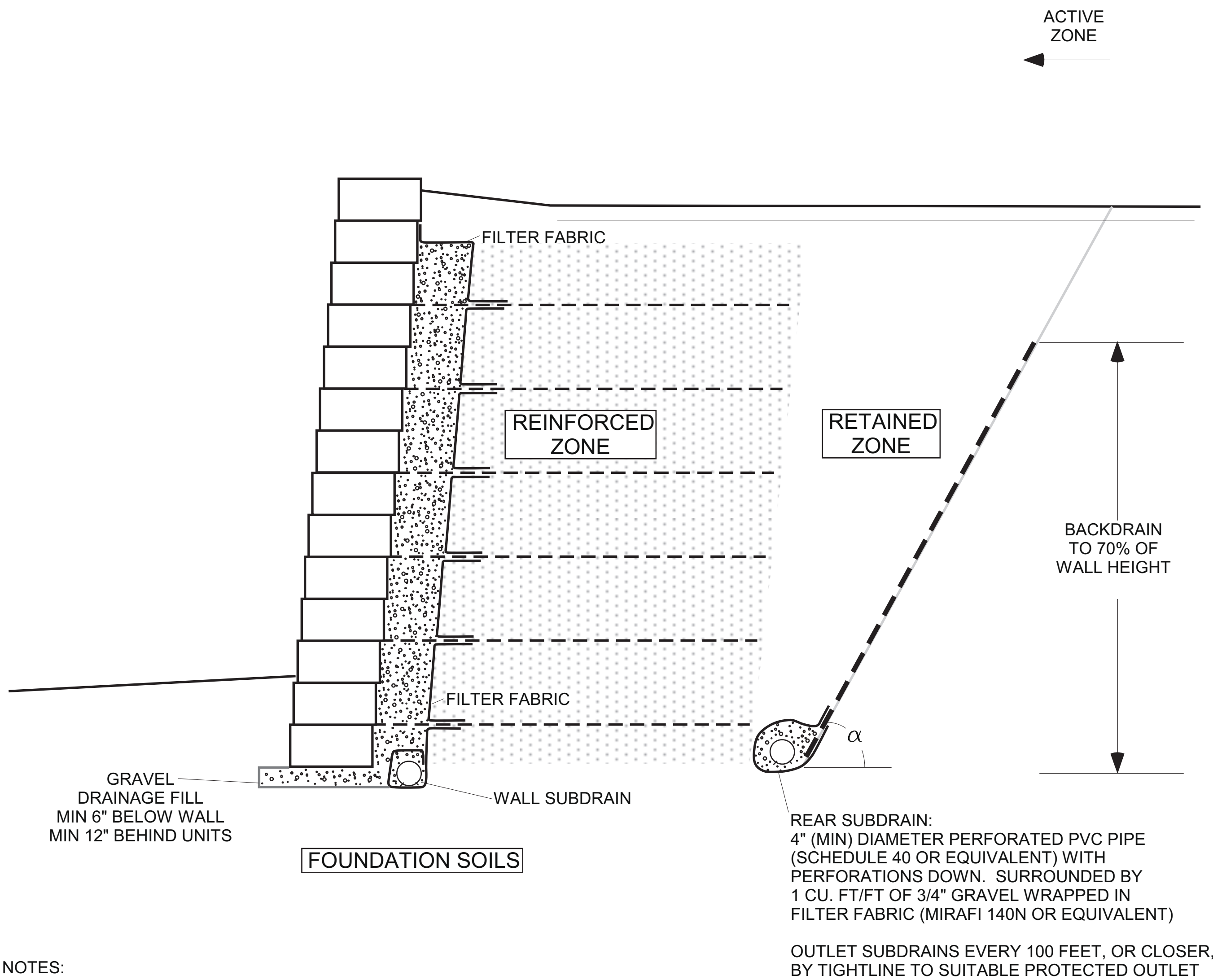
GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL E



NOTE: UPON REVIEW BY THE GEOTECHNICAL CONSULTANT, COMPOSITE DRAINAGE PRODUCTS SUCH AS MIRADRAIN OR J-DRAIN MAY BE USED AS AN ALTERNATIVE TO GRAVEL OR CLASS 2 PERMEABLE MATERIAL. INSTALLATION SHOULD BE PERFORMED IN ACCORDANCE WITH MANUFACTURER'S SPECIFICATIONS.

RETAINING WALL DRAINAGE

GENERAL EARTHWORK AND GRADING SPECIFICATIONS
STANDARD DETAIL F



NOTES:

1) MATERIAL GRADATION AND PLASTICITY

REINFORCED ZONE:

SIEVE SIZE	% PASSING
1 INCH	100
NO. 4	20-100
NO. 40	0-60
NO. 200	0-35

FOR WALL HEIGHT < 10 FEET, PLASTICITY INDEX < 20
FOR WALL HEIGHT 10 TO 20 FEET, PLASTICITY INDEX < 10
FOR TIERED WALLS, USE COMBINED WALL HEIGHTS
WALL DESIGNER TO REQUEST SITE-SPECIFIC CRITERIA FOR WALL HEIGHT > 20 FEET

GRAVEL DRAINAGE FILL:

SIEVE SIZE	% PASSING
1 INCH	100
3/4 INCH	75-100
NO. 4	0-60
NO. 40	0-50
NO. 200	0-5

2) CONTRACTOR TO USE SOILS WITHIN THE RETAINED AND REINFORCED ZONES THAT MEET THE STRENGTH REQUIREMENTS OF WALL DESIGN.

3) GEOGRID REINFORCEMENT TO BE DESIGNED BY WALL DESIGNER CONSIDERING INTERNAL, EXTERNAL, AND COMPOUND STABILITY.

3) GEOGRID TO BE PRETENSIONED DURING INSTALLATION.

4) IMPROVEMENTS WITHIN THE ACTIVE ZONE ARE SUSCEPTIBLE TO POST-CONSTRUCTION SETTLEMENT. ANGLE $\alpha = 45 + \phi/2$, WHERE ϕ IS THE FRICTION ANGLE OF THE MATERIAL IN THE RETAINED ZONE.

5) BACKDRAIN SHOULD CONSIST OF J-DRAIN 302 (OR EQUIVALENT) OR 6-INCH THICK DRAINAGE FILL WRAPPED IN FILTER FABRIC. PERCENT COVERAGE OF BACKDRAIN TO BE PER GEOTECHNICAL REVIEW.

SEGMENTAL RETAINING WALLS

GENERAL EARTHWORK AND
GRADING SPECIFICATIONS
STANDARD DETAIL G