GEOTECHNICAL REPORT

South San Luis Obispo County Sanitation District

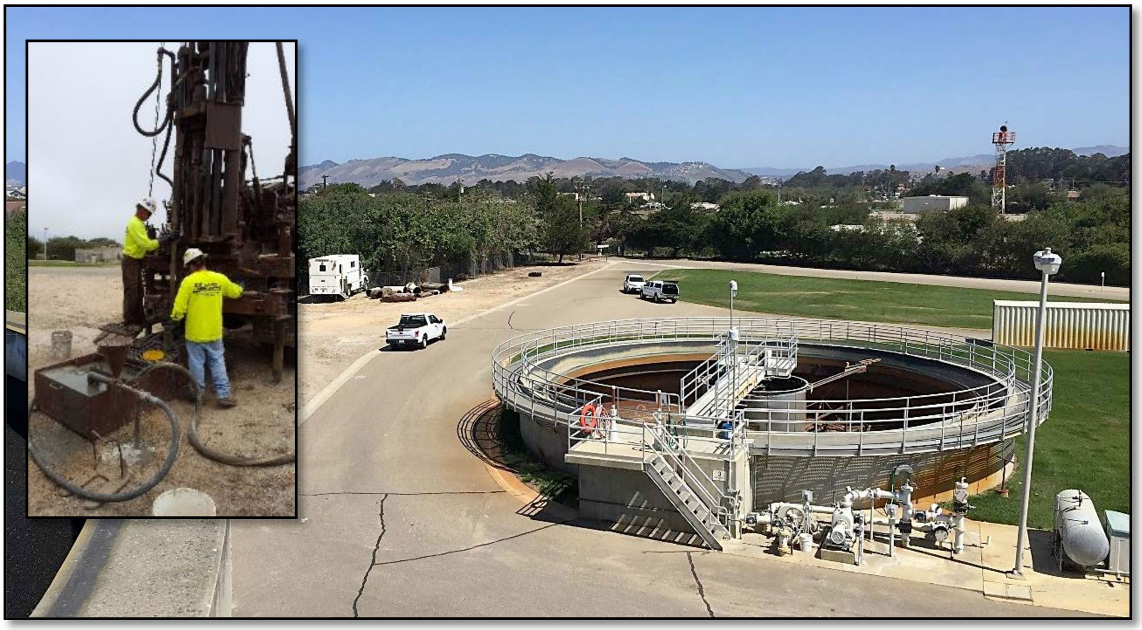
Wastewater Treatment Plant Redundancy Project

1600 Aloha Avenue

Oceano, California

Yeh Project No.: 216-193

January 15, 2019



Prepared for:
Kennedy/Jenks Consultants
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Attn: Mr. David Seymour, P.E.

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Attn: Mr. David Seymour, PE

Subject: **Geotechnical Report for South San Luis Obispo County Sanitation District – Wastewater Treatment Plant Redundancy Project, 1600 Aloha Avenue, Oceano, California**

Dear Mr. Seymour:

Yeh and Associates, Inc. is pleased to submit this geotechnical report for the design of the South San Luis Obispo County Sanitation District (SSLOCSD) Wastewater Treatment Plant (WWTP) Redundancy Project (Redundancy Project). The project includes a new secondary clarifier tank, aeration basin, blower building, equipment pads, site retaining walls and associated piping. This report was prepared in accordance with the Master Services Subcontract Agreement between Kennedy/Jenks Consultants and Yeh and Associates (Yeh), dated February 18, 2016 and Work Authorization Number 1668009.00-01.

The geotechnical evaluation consisted of a program of field exploration, laboratory testing, and analysis. Field and laboratory data collected for this study and previous studies are presented in the appendices to this report. Graphics showing the locations of the field explorations and interpreted subsurface profiles are also attached to this report. This report provides seismic data and geotechnical recommendations for the design of ground improvement, site preparation and grading, foundations for the structures, underground utilities; and construction considerations regarding excavations, groundwater and temporary slopes and shoring. A summary of key geotechnical considerations for the project are as follows:

- The field exploration program consisted of drilling three borings at the site to depths of 60.0 to 81.5 feet below the ground surface and advancing nine cone penetrometer test (CPT) soundings to depths of 70 to 105 feet. Plant personnel and the United States Geologic Survey have documented that liquefaction occurred at the site in response to the 2003 M6.5 San Simeon Earthquake. The design earthquake for the site is more severe than the San Simeon Earthquake: a M6.7 earthquake that could occur closer to the site and result in 3 to 4 times the ground motion than occurred at the site in response to the San Simeon Earthquake. Liquefaction was manifested at the site as sand boils following the San Simeon Earthquake; however, no visible damage or settlement of structures was reported by plant staff. Liquefaction can be manifested as a loss of bearing below foundations, deflection of buried pipes or pipe connections, changes to the hydraulic profile, lateral spreading or ground instability, and an increase in eccentric loads and lateral earth pressures.

- Recommendations for deep compaction using vibro-replacement with stone columns are provided in this report to reduce the potential for the plant improvements to be impacted by liquefaction. Potentially liquefiable soil conditions were encountered within the borings and CPT soundings within the upper approximate 25 to 36 feet. Seismic settlement of 3.5 to 8 inches were estimated as a result of the design earthquake. Mitigation options were presented and discussed in the draft *Preliminary Geotechnical Report* (Yeh 2016). Deep compaction with vibro-replacement (also known as stone columns) is recommended to mitigate liquefaction for design to limit estimated seismic settlement under the design earthquake to less than 1 inch.

- Excavations for the secondary clarifier and aeration basin are expected to range from 15 to 18 feet below the existing ground surface for the project. Our explorations encountered loose sandy soil below the groundwater table within the anticipated depths of these excavations. Excavations will need to be properly sloped, shored and dewatered when below the groundwater table (encountered as shallow as 4 feet below the existing ground surface) to maintain stable slopes and excavations. It is anticipated that excavations will likely involve installing dewatering wells to draw water below the anticipated depths of excavation and/or continuous shoring systems embedded below the depth of excavation that is designed to control and cutoff seepage.

- The proposed structures can be supported on conventional shallow spread and continuous foundations or mat slab foundations. The new foundations should bear on ground improved using deep compaction with stone columns to densify potentially liquefiable soil conditions encountered at the site to depths of approximately 36 feet below the existing ground surface.

We appreciate the opportunity to be of service. Please contact Judd King at 805-481-9590 x285 or jking@yeh-eng.com if you have questions or require additional information.

Sincerely,
YEH AND ASSOCIATES, INC.

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1. PURPOSE AND SCOPE OF STUDY

Figure 1: Vicinity Map

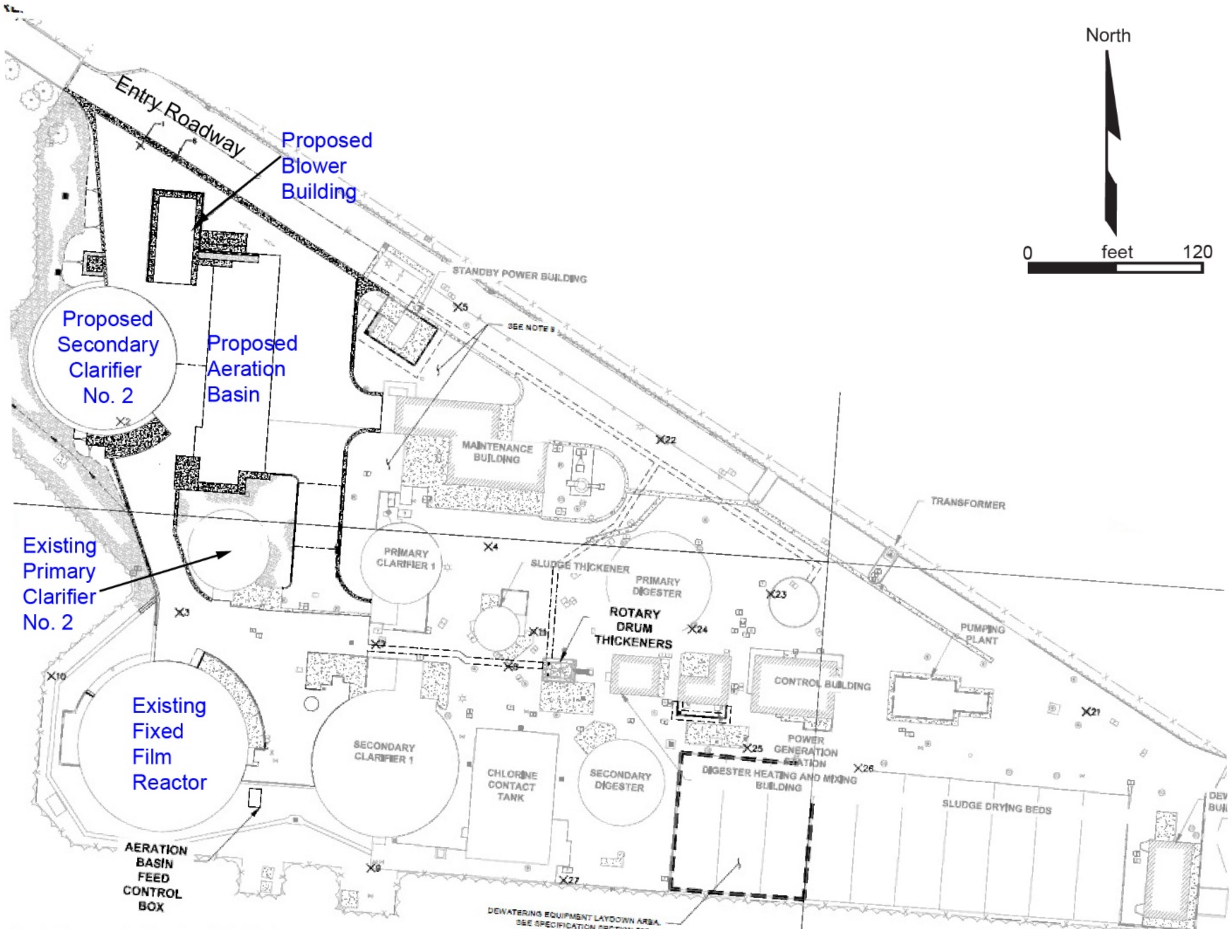
Yeh and Associates was retained by Kennedy/Jenks Consultants (K/J) to provide geotechnical recommendations for the design of a new clarifier, aeration basin, blower building, equipment pads and associated piping as part of the improvements to the South San Luis Obispo County Sanitation District (District) Wastewater Treatment Facility Redundancy Project at 1600 Aloha Avenue in Oceano, California. The location of the site is shown on Figure 1.

The geotechnical evaluation consisted of a program of project coordination, reviewing data, field exploration, laboratory testing, and engineering analyses as a basis for providing the recommendations in this report. This report provides recommendations for the seismic design, ground improvements, site preparation and grading, underground utilities, pavements, and foundation design for the clarifier, aeration basin, blower building, equipment pads and piping.

2. PROJECT UNDERSTANDING

2.1 PROPOSED PROJECT

The general layout of the existing plant and structures and the proposed structures (secondary clarifier, aeration basin, and blower building) are shown on Plate 1 – Field Exploration Plan and on the Site Plan in Figure 2. The proposed improvements will add redundancy to the existing wastewater treatment plant for operating and maintaining the plant; resiliency to the plant relative to flooding, seismic and coastal hazards; and provide modernization of controls and monitoring systems to assist with the operation of the plant.



Ref: Kennedy/Jenks (2018a)

Figure 2: Site Plan

A summary of the improvements structure dimensions, elevations, and contact pressures is provided in Table 1 based on the 90 percent design plans by Kennedy/Jenks (2018a) and data provided by Kennedy/Jenks (K/J) via email (K/J 2018b).

Table 1: Summary of Proposed Structures

Structure	Overall Structure Dimensions	Bottom of Foundation Elevation	Total Foundation Contact Pressures	Notes:
Secondary Clarifier	104 ft. diameter	Main Structure: -4.5 ft. Wet Well: -7.5 ft.	1.3 ksf (slab area) 1.9 ksf (center and edge of foundation)	Finish grades around the new structure will range from elevation (el.) 8 to 13 ft. The mat will be thickened to support additional load from the center column and tank walls.
Aeration Basin	161 ft. long by 50 ft. wide	Main Structure: 4.5 ft. Wet Well: -7.0 ft.	1.5 ksf (slab area) 1.9 ksf (continuous footing)	Finish grade around structure will be el.12.0 ft.
Blower Building	60 ft. long by 30 ft. wide	12.25 ft.	0.3 ksf (slab area) 2.4 ksf (continuous footing)	Slab-on-grade with finish floor el. 12.25 ft. Finish grade will be el. 12 ft. around building
Minor Equipment Slabs	20 ft. by 20 ft.	12 ft.	0.4 ksf	Slabs-on-grade. Finish grade will be el. 13 ft.
AB Feed Control Box	8 ft. by 14 ft.	1.5 ft.	0.9 ksf	Partially buried vessel. Finish grade will be el. 11 around structure
Generator Slab	10 ft. by 20 ft.	11 ft.	0.3 ksf	Mat Slab-on-grade. Finish grade will be el. 11.5 ft. around structure

ft. = feet
ksf = kips per square foot

The secondary clarifier and aeration basin will be reinforced concrete structures. Stairways, railings, and piping will be connected to the structures. Both structures will be partially buried below the existing grade to accommodate the hydraulic profile of the plant. We have estimated that excavations will be made 15 to 18 feet below the existing ground surface to construct the clarifier and aeration basin structures.

The blower building will be located north of the proposed secondary clarifier and aeration basin (see Figure 2). The building will be 15 to 20 feet tall with concrete masonry walls and a slab-on-grade floor to support the blowers and other equipment.

Conveyance pipes to connect and move effluent between existing structures and new structures at the plant will be up to 15 feet deep. Piping will also be installed below or near existing buried pipes. Other improvements will include hot-mix asphalt (HMA) concrete and Portland cement concrete (PCC) pavements, equipment slabs for generators, pumps and pipe stands, a fluid control box, landscaping, lighting, utilities and site drainage. Site walls (retaining and screen walls) up to 5 feet tall will be designed to protect various facilities from flooding and to retain soil at loading dock areas.

2.2 EXISTING FACILITY

The site topography on the K/J (2018a) plans show that the ground elevation within the existing facility is at approximately 8 to 12 feet above sea level (in reference to NGVD 29 per the project plans). The existing plant is about 1,500 feet east of the shoreline to the Pacific Ocean. The existing facility is bordered to the northeast by the Oceano Airport, along the south by the Arroyo Grande Creek levee, and to the west by residential lots and woodlands (see Plate 1).

Existing facility improvements include headworks, two primary clarifiers, one secondary clarifier, a fixed-film reactor, sludge drying ponds, buildings, piping, paved roadways, and landscape areas. The buildings were constructed near the previous site grades; however, as-built plans by Kennedy/Jenks/Chilton (1992) shows that primary clarifier no. 2 (the closest structure to the redundancy project) is partially buried to approximately 6 feet below the ground surface. We understand from K/J that other tanks (clarifiers and fixed-film reactor) are also partially buried to depths of approximately 6 to 15 feet below grade and are connected with buried piping. A pump-station located immediately south of the proposed secondary clarifier runs nearly continuously to remove subsurface water that infiltrates the plant's storm drain system.

The 90 percent (K/J 2018a) plans show that the outfall line from the Pismo Beach Wastewater Treatment Plant will be in close proximity to the proposed improvements. The outfall pipeline runs along the entry driveway off of Aloha Avenue to a valve box near the south boundary of the plan (see Plate 1). The proposed aeration basin/blower building and associated new piping will be in close proximity to the existing outfall pipe. We understand that the pipe is buried approximately 10 feet below the ground surface and will have approximately 5 to 25 feet of horizontal separation from the proposed structures. Both plants (Pismo Beach and SSLOCSD) share this same outfall line. The outfall extends approximately 4,400 feet off-shore into the Pacific Ocean.

2.3 SITE HISTORY AND RECORDS REVIEW

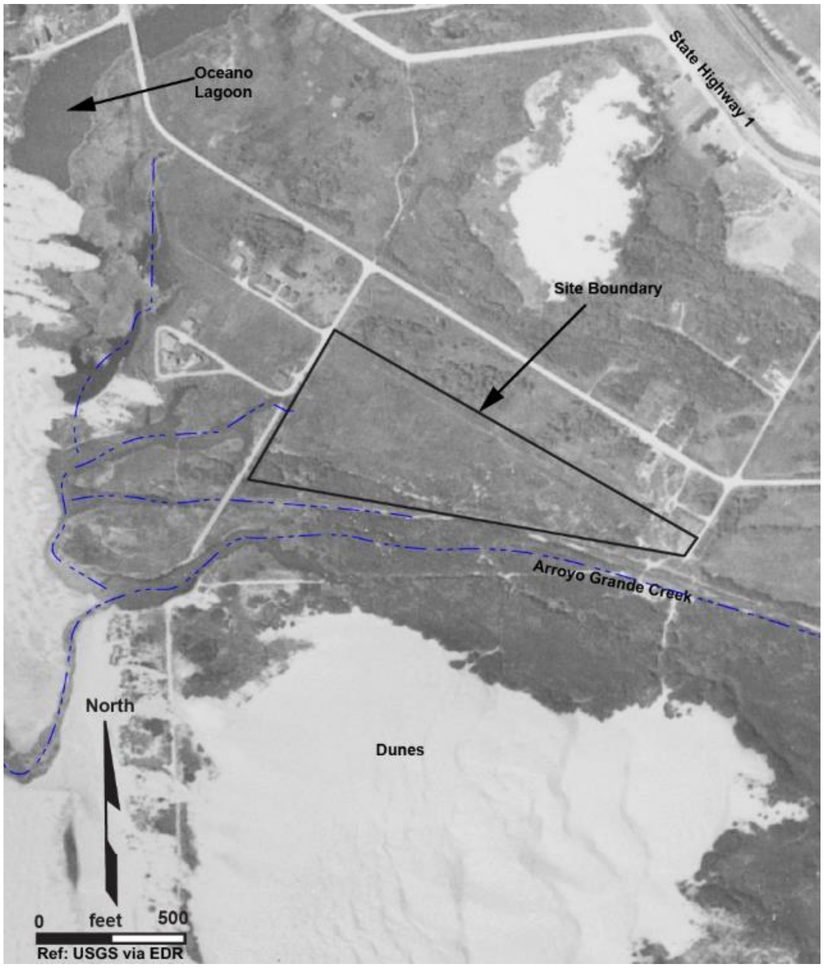


Figure 3: 1939 Aerial Photo – Pre WWTP Development

The EDR (2015) decade package provides historic aerial photographs of the site. Figure 3 shows the site relative to a 1939 aerial photo (from EDR 2015) and the channels of Arroyo Grande Creek and minor drainages converging southwest of the site before flowing into coastal dunes and toward the Pacific Ocean to the west. The photos from EDR show that previous drainage channels once converged near the site. Meadow Creek now flows into the north end of the Oceano Lagoon and confluences with Arroyo Grande Creek at the south end of the lagoon about 1,000 feet northwest of the project site. Arroyo Grande Creek lies at approximately the same elevation as the site but is separated by the existing levee that is present along the southern property line. The creek has been channelized and is heavily vegetated. Eventually the creek flows into the Pacific Ocean about 1,500 feet west of the site.

Figure 4 presents an overlay of the site imagery onto the historic map. The map shows the approximate location of an estero and the main thread of Arroyo Grande Creek. The overlay is T-sheet number 1393 from a mapping project performed in 1874 by the U.S. Coast Survey (NOAA 2018). The estero was eventually drained and the creeks were channelized to reduce flooding, create farmland and/or developable land. It is likely that some fill was placed across the site for development. The areas of the pre-development estero were those reported to be most impacted by liquefaction during the 2003 San Simeon Earthquake (Holzer, et. al. 2004).

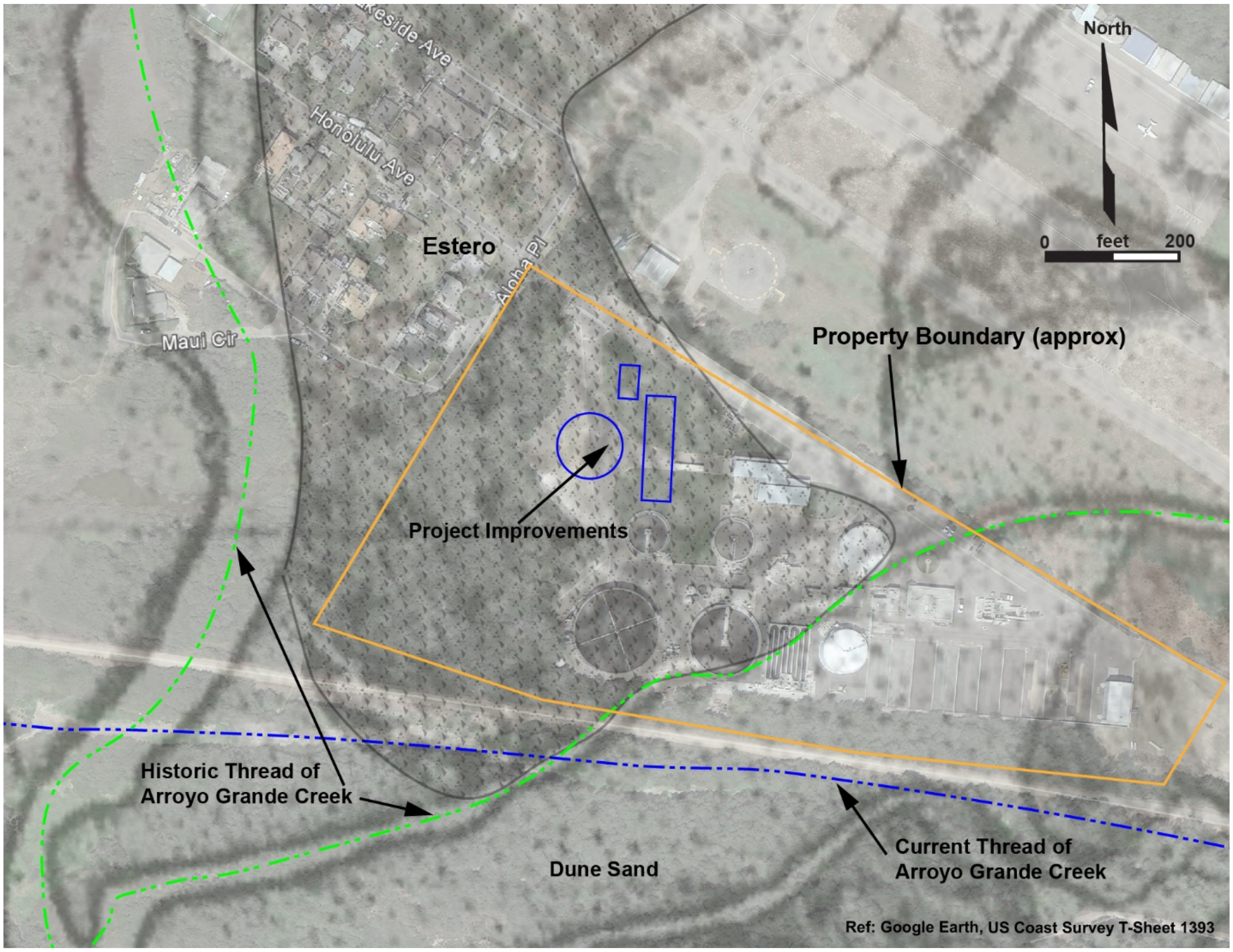


Figure 4: Historic Site Overlay

2.4 PREVIOUS STUDIES

Previous geotechnical reports prepared for the South San Luis Obispo County Sanitation District WWTP were reviewed as referenced in this report. The two most significant reports and data sets reviewed included:

- A geotechnical investigation was completed by Cooper, Clark and Associates in 1979. The report was prepared for Jenks & Harrison for the design of a new sludge thickener, standby power building, maintenance building, and sludge drying beds. This report provided five subsurface borings, B-1 to B-5. The locations of the Cooper, Clark and Associates (1979) borings are shown on Plate 1. Logs of the borings are included in Appendix C.

- A geotechnical investigation was completed by Subsurface Consultants, Inc. in 1984. The report was prepared for SSLOCSD and K/J for the design of the existing fixed-film reactor, generator building, and sludge drying facility. This report provided one boring, B-1, that was drilled between the existing fixed film reactor and secondary clarifier. The report also referenced two other previous investigations (one being the Cooper-Clark & Associates) and another unnamed company. Yeh was provided with the Cooper-Clark & Associates report referenced by Subsurface Consultants, Inc., but not the unnamed report. The location of the Subsurface Consultants (1984) boring is shown on Plate 1 and the boring log is included in Appendix C.

2.5 HISTORICAL AERIAL PHOTOGRAPH REVIEW

Historical aerial photographs from 1939, 1949, 1956, 1966, 1969, 1972, 1978, 1981, 1989, 1994, 2005, 2009, 2010, and 2012 were compiled by Environmental Data Resources (EDR) of Shelton, Connecticut and used to estimate the pre-existing estero and review the history of improvements to the project site. The photos were reviewed to evaluate changes in land use, topography, geomorphic features, and other characteristics pertinent to the site history, geology and geotechnical considerations discussed in this report. The photos from EDR are provided in Appendix D with the approximate site boundary noted on each photo. The following observations were made during the review:

- **1939:** No buildings or structures are visible on the project site and the only surrounding improvements are some roadways. Drainages are visible south and west of the project site. It appears that some of the drainages have been filled in or partially filled to accommodate the roadways in the area on the west side of the project site.
- **1949:** A strip of vegetation has been cleared across the north portion of the project site. Some channelization of Arroyo Grande Creek is visible south of the site.
- **1956:** The majority of the project site has been cleared of vegetation and the Oceano Airport runway has been constructed. Some trails and or roadways cross the project site boundaries. The alignment of Arroyo Grande Creek appears to be the same.
- **1966:** Components of the WWTP are visible. The primary clarifier, sludge digester, control building, sludge basin, and secondary clarifier are visible. The area of the subject project is sparsely vegetated. Arroyo Grande Creek has now been channelized with the levee that is present immediately south of the project site.
- No significant changes at the project site are visible in the 1969, 1972, 1978, and 1981 photographs.
- **1989:** The fixed-film reactor, maintenance building, and other structures at the plant are visible in the picture. The project site is sparsely vegetated. The levee along Arroyo Grande Creek is obscured by vegetation.

- **1994:** The second primary clarifier is visible, and the project area remains sparsely vegetated.
- No significant changes at the project site are visible except for developed roadways and vegetation in the 2005, 2010, and 2012 photographs.

3. FIELD INVESTIGATION AND LABORATORY TESTING

The field exploration and testing program consisted of drilling, cone penetration test (CPT) soundings, and laboratory testing. The boring and CPT locations for this and previous studies are shown on Plate 1.

3.1 DRILLING

The drilling subcontractor for this project was S/G Drilling Company of Lompoc, California. S/G used a truck-mounted CME-75 drill rig equipped for mud rotary and hollow-stem-augers to advance three borings at the site during the period of May 31 through June 2, 2016. Borings were advanced to depths ranging from approximately 60.0 to 81.5 feet below the ground surface. Boring 16E-01 was initiated using mud-rotary drilling. A 3.5-inch diameter, side-discharge bit was used to advance the boring to a depth of 35 feet. The hole lost circulation within a gravel layer at 35 feet, and the hole was completed below 35 feet using 8-inch outside diameter hollow-stem augers. Borings 16E-02 and E-03 were also drilled using hollow stem augers. Water and drilling mud were commonly added to the augers to help maintain a stable borehole during sampling. Yeh collected samples for subsequent laboratory testing, recorded blow counts (N-values) for the driven samples and prepared a field log of subsurface conditions encountered. The logs of the borings are presented in Appendix A.

Sampling within the borings was performed using driven modified California samplers and standard penetration test (SPT) split spoon samplers, and by pushing thin-walled tubes (Shelby tube). A driven sample was typically collected at a depth of 1 foot. Samples below 1 foot were taken at typical 5-foot depth intervals. The SPT sampler has a 2-inch outside diameter, 1-3/8-inch inside diameter and is equipped for but was used without liners. The modified California sampler has a 3-inch outside diameter, 2-3/8-inch inside diameter and was used with 1-inch high brass liners. Tube samplers were 30-inch long by 3-inch diameter steel tube with a 1/16-inch wall thickness and a slightly smaller diameter mandrelled cutting shoe to help reduce disturbance to the soil during sampling. Drive samples were collected using a 140-pound automatic trip hammer and sampling protocols in general accordance with ASTM 1586, the Standard Penetration Test. Tube samplers were pushed into the ground using the drill rig’s hydraulics. Bulk samples were collected from the augers as the borings were advanced.

Pocket penetrometer and torvane tests were performed in the field on the trimmed end of selected samples to measure the undrained shear strength of cohesive materials. The penetrometer was pushed to the designated penetration and the compressive strength was read from the spring scale on the device. The undrained shear was reported on the log as half of the measured compressive strength of the soil (noted by values of PP in ksf on the logs). The torvane was inserted into the end of the trimmed sample and rotated until the torque applied by the vane sheared the soil. The undrained shear strength from the torvane (noted by values of 'TV' in ksf on the logs) of cohesive soil was recorded. The undrained shear strength results from these field tests are noted on the logs in Appendix A.

Upon completion, Borings 16E-02 and 03 were backfilled with bentonite grout. The upper 22 feet of Boring 16E-01 was completed as a 2-inch diameter monitoring well (16MW-01). The details of the monitoring well are provided on the boring log in Appendix A.

3.2 CONE PENETRATION TEST (CPT) SOUNDINGS

The CPT subcontractor was Gregg Drilling and Testing, Inc. of Signal Hill, California. Gregg advanced nine soundings using a truck-mounted hydraulic ram on May 16, 2016. CPT soundings were advanced to depths ranging from approximately 70 to 105 feet below the ground surface. The soundings were terminated once refusal was encountered as indicated when the push load became excessive resulting in lifting of the rig or lateral deflection of the penetrometer. Logs of the CPT soundings are presented in Appendix A.

Soundings were performed in general accordance with ASTM D-5778 using an electric piezocone penetrometer. The piezocone penetrometer had a diameter of approximately 1.7 inches with a tip area of 15 square centimeters (cm^2) and a sleeve area of 225 cm^2. The cone tip resistance (q_c), sleeve friction (f_s), and penetration pore water pressures measured behind the tip (in the u2 location) were recorded at approximately 3-centimeter intervals during penetration using an on-board computer. The friction ratio (FR, the ratio of the sleeve friction to the tip resistance in percent) was computed for each value of q_c and f_s recorded. The data and soil behavior type classifications were used in subsequent geotechnical analyses and to evaluate soil types and boundaries for analyses. Upon removal of the CPT rod, the soil generally collapsed to near the ground surface. The void above that depth was filled with bentonite chips.

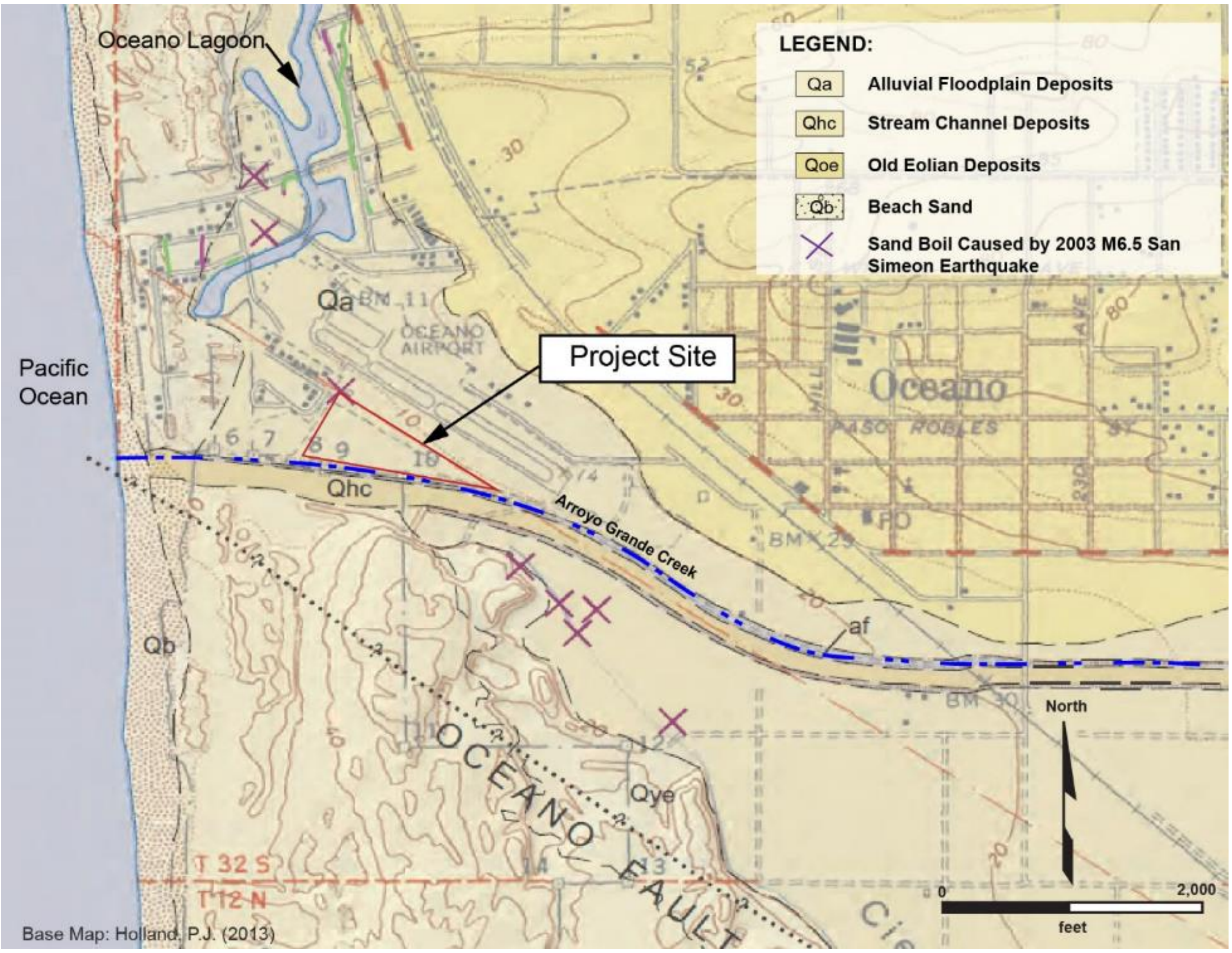
3.3 LABORATORY TESTING

Laboratory testing was performed on selected samples recovered from the field exploration program. Tests for moisture content, unit weight, gradation, Atterberg limits, compacted unit weight versus moisture content relation by the modified Proctor test, and pH and resistivity were performed at the Yeh office and laboratory in Grover Beach, California. Tests for soluble sulfates and chlorides were performed by Cooper Testing Laboratory in Palo Alto, California. An R-value test was performed by NV5 of Ventura, California. Tests for triaxial compressive strength using consolidated undrained (CU) loading, constant rate of strain (C) consolidation, and direct shear strength (DS) were performed at the Geo-E lab at the Cal Poly Civil Engineering Department in San Luis Obispo, California. Testing was performed in accordance with applicable ASTM standards. Laboratory test results are presented in Appendix B.

4. SUBSURFACE CONDITIONS

4.1 GEOLOGIC SETTING

The project site is within a coastal plain at the western base of the Santa Lucia Mountains within the Coast Ranges geologic and geomorphic province, which extends from the Transverse Ranges in southern California to the Klamath Mountains in northern California and into Oregon.



Base Map: Holland, P.J. (2013)

Figure 5: Geologic Map (Holland 2013)

The province is characterized by north-northwest trending mountain ranges (locally the Santa Lucia Mountains) composed of sedimentary, volcanic, and metamorphic rock formations. The rock units are predominately Jurassic and Cretaceous age with Tertiary to Quaternary age units commonly overlying the older rock along the flanks and foothills of those ranges. Recent sediments are found within intervening drainages and valleys, and coastal areas.

The site is located within the margins of a historic estuary and adjacent to coastal dunes and lagoons. The surficial geology at the site vicinity as mapped by Holland (2013) is shown on Figure 5. Holland maps the predominant geologic structure as late Holocene-aged unconsolidated sandy, silty, and clay soil (Qa), similar to surface geology mapped by Dibblee (2006). The Oceano Fault is located approximately 1,500 feet south of the site and concealed by the overlying soil. The Oceano Fault is a Late Quaternary age fault and is a part of the San Luis Range fault system and is mapped as a reverse fault (USGS 2017c).

The area is generally characterized as a low-lying area within the predevelopment estero. The estero was drained and filled within the past century to develop the site, roadways, the airport, levee and nearby residential areas.

4.2 SUBSURFACE CONDITIONS

The subsurface conditions encountered in Yeh's exploration programs consisted of a layer of artificial fill overlying alluvium. Profiles A-A' through C-C' showing the interpreted subsurface conditions relative to the proposed improvements are presented on Plates 2 through 4. A summary of the predominant geologic units encountered relative to the proposed improvements is presented below:

Artificial Fill (Af). Artificial fill was encountered from the surface to depths of approximately 4 to 6 feet in CPT soundings and borings. The fill consisted of medium dense silty and clayey sand with varying amounts gravel that was likely placed in association with phases of construction and grading of the existing facility, and is noted as (Af) on boring logs and the subsurface profiles.

Alluvium (Qa). Alluvium underlies the artificial fill and generally consists of interbedded layers of sand and clay soil likely associated with a shallow marine, estuarine or eolian[1] depositional environment. We have differentiated three predominant sub-units within the alluvium which are noted on the subsurface profiles and described below.

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[1] Eolian: Wind-blown sediment or dune sand

- Qa_1: Loose to medium dense poorly graded sand and silty sand interlayered with traces or lenses of gravel and 1- to 3-inch-thick lenses of soft silt and clay was found between depths from about 5 feet to 25 feet. The density of the sand increased below a depth of about 35 feet. Layers of shells and fibrous organic matter and were also found at various depths within this zone.
- Qa_2: An approximate 15 to 35 foot thick layer of soft to stiff lean clay, fat clay, and lean clay with sand was encountered to depths between 45 and 80 feet below the ground surface.
- Qa_3: Discontinuous lenses of loose to medium dense sand and gravel were encountered below depths of 45 to 80 feet below the ground surface. Shells and shell fragments were encountered at various depths and sample intervals within this zone.

A summary of the laboratory test results collected from the current study for the artificial fill and alluvium units is presented in Table 2.

Table 2: Laboratory Test Summary[2]

Geologic Unit	Boring Locations	Dry Unit Wt. (pcf)	Moisture Content (%)	Particle Size Analyses (%G, %S, %F)	Atterberg Limits LL, PI	Shear Strength (ksf)	Other
Artificial Fill (Af)	16E-01 to 03	88-116	15-34	4-11 G 57-68 S 28-32 F	28-31 LL 8-9 PI	Φ_{DS} = 37° c_{DS} = 0.0 ksf	pH = 8.39-8.97 ρ = 817-2008 Ω-cm $SO^{2-}{}_4$ = 122 mg/kg Cl- = 117 mg/kg UW_{MAX} =115 pcf $W\%_{OPT}$ = 11% R-value = 51
Alluvium (Qa_1, Qa_2, Qa_3)	16E-01 to 03	75-106	22-47	0-40 G 1-98 S 2-99 F	42-76 LL 16-44 PI	φ_{CU} = 37-40° c_{CU} = 0.0 ksf	pH = 8.09 ρ = 1507 Ω-cm

4.3 GROUNDWATER

Groundwater was encountered at depths of 4 to 6 feet below the ground surface during the May 2016 field exploration program. An open standpipe monitoring well with a depth of 22 feet was constructed within boring 16E-01 and the depth to water was measured in August 2018. Table 3 summarizes the groundwater depths and elevations (el.) encountered in borings.

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[2] *Geotechnical properties are noted for dry unit weight (γ_d) and moisture content (w_o); particle size as percent gravel (G), sand size (S) and fines content (F); electrical resistivity (ρ) in ohm-centimeters (Ω-cm), soluble sulfates (SO_{42-}) Atterberg liquid limit (LL) and plasticity index (PI); shear strength (S) in kips per square foot measured by pocket penetrometer (pp), torvane (tv) or unconsolidated undrained (uu) tests; friction angle (ϕ) or cohesion (c) in kips per square foot measured from direct shear (ds) or consolidated undrained (cu) tests.*

Table 3: Groundwater Data

Location	Approximate Surface Elevation[1], ft.	Depth to Groundwater and Corresponding Elevation (feet)		Notes
		5/31/2016	8/7/18	
16E-01	9	5.5 (el. 3.5)	3.9 (el. 5.1)	22-foot deep, 2-inch diameter monitoring well constructed in 16E-01
16E-02	10	6.0 (el. 3.0)	--	Boring backfilled with bentonite grout same day as drilling
16E-03	9	4.0 (el. 5.0)	---	Boring backfilled with bentonite grout same day as drilling
Elevations estimated from Kennedy/Jenks 90 percent design plans (2018a) which are based on a NGVD29 benchmark.				

We understand from verbal communication with Jimmy Tomac of Cannon (a subconsultant to K/J for this project) that groundwater ranging in depth from 5 to 8 feet below the ground surface was observed during utility potholing activities being performed for the design in July 2018. Soil moisture and groundwater conditions will vary seasonally and due to variations in storm runoff, irrigation and groundwater pumping in the site vicinity.

5. SEISMIC HAZARD EVALUATION

5.1 HISTORIC SEISMICITY

The site is within a seismically active area of California that has been impacted by earthquakes in the past. A significant and relatively recent earthquake to the site is the 2003 San Simeon Earthquake. The event and impacts near Oceano were documented by the United States Geologic Survey (Holzer, et. al. 2004). The San Simeon Earthquake was an estimated magnitude 6.5 event. The estimated peak ground accelerations (PGA) in the site vicinity was about 0.05 to 0.15g as presented by Holzer, et. al. (2004). Those accelerations may have been amplified by the underlying soft ground to as much as 0.25g in the site vicinity. The San Simeon Earthquake resulted in liquefaction at the site and adjacent areas that was manifested by sand boils at the site, instability and failure of the south levee of Arroyo Grande Creek just upstream of the site, and lateral spreading that resulted in cracking and displacement of homes, roadways, and sidewalks in Oceano; especially in the predevelopment estero areas.

SSLOCSD staff (verbal communication with the maintenance supervisor in 2016) stated that evidence of liquefaction at the site immediately following the 2003 event included sand boils that formed within the lawn areas where the aeration basin is planned. Sand boils heaved the ground in the lawn area resulting in an irregular surface following the earthquake. The area has since been releveled and the lawn reestablished.

5.2 LIQUEFACTION EVALUATION

Liquefaction typically occurs in young, loose to medium dense granular sand or sensitive clay and silt below the groundwater table that are subject to ground motions from an earthquake. The potential for liquefaction is dependent on site-specific properties such as the relative density, plasticity, and particle size of a soil; groundwater conditions; and geologic history. Potentially liquefiable soils may be vulnerable to loss of strength and foundation support, seismic settlement, slope instability or lateral spreading depending on the severity of the liquefaction hazard and site conditions.

Liquefaction resistance was calculated using Cliq and the NCEER procedures (Youd and Idriss 2001). The computer software Cliq V2.2.0.28 (GeoLogismiki) was used to evaluate the liquefaction potential of the site based on CPT sounding data. Loose to medium dense sand encountered to depths of approximately 20 to 36 feet beneath the site is considered vulnerable to liquefaction, seismic settlement and instability under the design earthquake. The sand between depths of 36 and 45 feet is relatively dense and is less prone to liquefaction based on our analyses.

Seismic Settlement. Cliq was used to process data from the CPT soundings to calculate the potential for liquefaction to occur at the site, and to estimate the amount of seismic settlement that could occur in association with liquefaction. Plots of the liquefaction analyses are provided with the logs of the CPT data in Appendix A. The liquefaction analyses considered both the design earthquake (M 6.7, PGA of 0.51g – See Section 6.5) and the 2003 San Simeon Earthquake (M 6.5, PGA of 0.25g) as summarized in Figure 6. Figure 6 shows a plot from Cliq showing the estimated seismic settlement resulting from the design earthquake and the estimated seismic settlement resulting from the 2003 San Simeon Earthquake for each CPT sounding. The amount of seismic settlement varies for each CPT sounding and increases with higher values of peak ground acceleration (PGA). Estimated settlements for the design earthquake are higher than those for the San Simeon event: on the order of 3.5 to 8 inches compared to less than 6 inches for the San Simeon earthquake. The estimated settlements for the design earthquake are higher because the ground motions (PGA) for the design earthquake are 2 to 4 times higher than those caused by the 2003 San Simeon Earthquake.

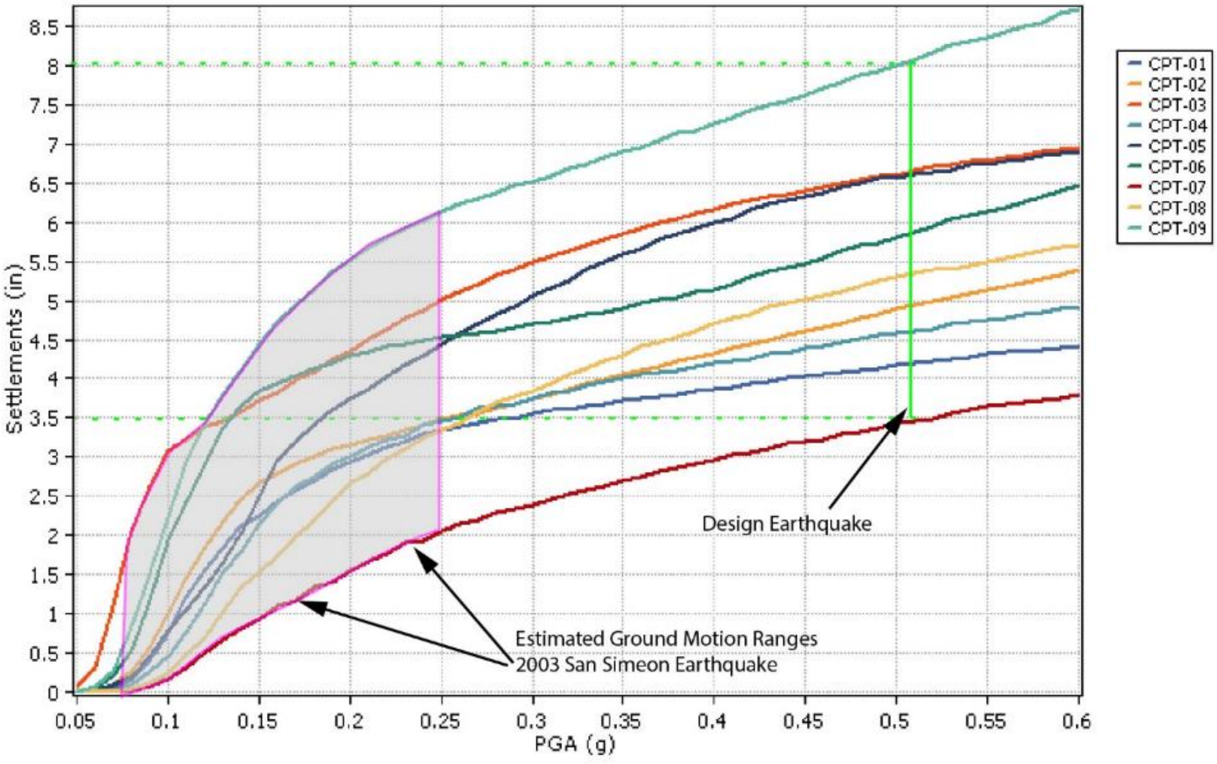

Figure 6: Estimated Seismic Settlements

Subsurface zones contributing to estimated settlements were also assessed to estimate the approximate depths of potentially liquefiable soil that would need to be improved to reduce the potential effect of liquefaction on the proposed structures and improvements. The goal of the recommended ground improvement is to limit the estimated total seismic settlement to less than 1-inch following deep compaction. The depth of improvement was estimated as the depth below which the estimated seismic settlement at the sounding location was less than 1-inch.

Liquefaction Potential vs. Depth. Figure 7 presents plots of the calculated seismic settlement for each of the CPT soundings versus depth. The estimated seismic settlement at CPT-08 is more than 1-inch above a depth of about 45 feet, while the remaining CPT soundings had more than 1-inch of seismic settlement calculated above a depth of about 36 feet (el. -26 feet). There is a significant increase in the estimated settlement (indicated by the abrupt bend in the plots) within the upper 20 to 30 feet of the site.

A predominant clay layer (that is not considered susceptible to liquefaction) was typically encountered below a depth of about 45 feet. This layer generally represents the bottom of the soil considered to be most-prone to liquefaction. With the exception of CPT-08, the estimated settlement below 36 feet was less than 1-inch. The estimated settlement in CPT-08 below 36 feet was mostly associated with relatively thin (1 to 2 feet thick) layers of silt, silty sand and sand within the clay.

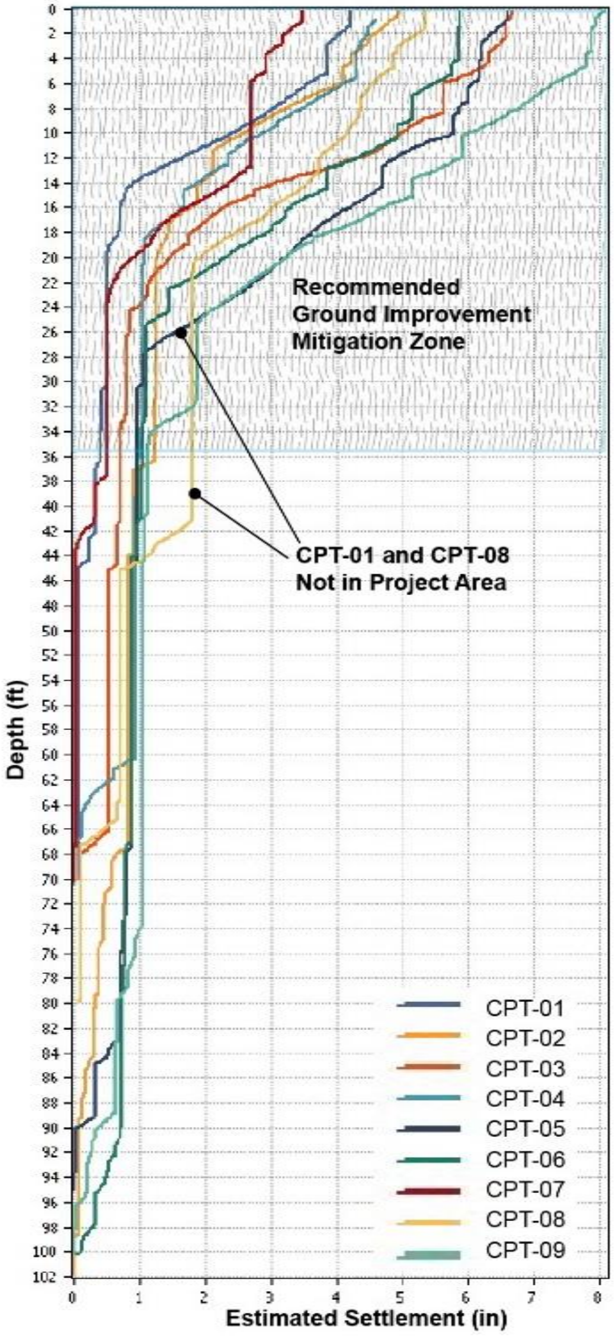

Figure 7: Plot of Seismic Settlement vs. Depth

The potential for liquefiable soil below a depth of about 60 feet is generally considered less likely to cause surface manifestation or settlement of the structures during a seismic event based on research of case histories presented by Ishihara (1985) and reiterated in Special

Publication 117 (CGS 1999). Ishihara's research on case histories indicate that where layers of liquefiable soils are overlain by a thick enough layer of non-liquefiable layers (such as the denser sand layers between 36 and 45 feet, the predominant clay layer between 45 and 60 feet, and the modified ground) of given thickness ratios, the potential for surface manifestation and damage is significantly reduced.

Lateral Spreads. There is a low potential for lateral spreading to occur on the immediate project site due to the relatively flat topography; however, the site could potentially be impacted if liquefaction resulted in instability of the adjacent Arroyo Grande Creek Levee. That hazard has not been specifically addressed in this study, although failure of the opposite levee occurred upstream of the site during the 2003 San Simeon Earthquake (Holzer et. al. 2004).

Mitigation Options. We presented options for liquefaction mitigation in our draft *Preliminary Geotechnical Report* (Yeh 2016). Those options included deep compaction (i.e. vibro-replacement), limited grading, deep soil mixing, deep foundations, or accepting the estimated settlement and associated risk and potential for damage to the structures. The proposed structures (blower building, secondary clarifier, and aeration basin) can be supported on conventional spread and/or continuous foundations or mat foundations and designed for typical seismic and static loads without consideration for liquefaction if ground improvement using vibro-replacement is implemented as recommended. Deep compaction using vibro-replacement (vibro-stone columns – VSC) is recommended as the method to reduce the effect of liquefaction and subsequent estimated seismic settlement at the site. A crushed rock stabilization layer is also recommended for below structures to help stabilize subgrade, provide a working platform for construction, and more evenly distribute the load between the bottoms of foundations to the underlying improved ground. The stabilization layer should be a geotextile wrapped and reinforced gravel mat. Considerations and recommendations for deep compaction are discussed later in this report.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 NOTES TO DESIGNER

6.1.1 EXCAVATIONS AND DEWATERING

Excavation and dewatering for the buried structures is a key geotechnical consideration for the construction of this project. Construction dewatering at this site could produce copious amounts of water that will need proper disposal relative to regulatory discharge requirements. Excavations will be made in loose, sandy soil below the groundwater table. The contractor should submit a detailed excavation and dewatering plan for review by the geotechnical professional before beginning the excavation. Excavation and dewatering plans should be designed by an engineer who is a qualified professional civil engineer registered in the State of California familiar with design of excavations, shoring, and dewatering in similar subsurface conditions. The plan should detail the dewatering plans, shoring, support of adjacent structures and adjacent utilities and a monitoring program appropriate for the anticipated subsurface conditions.

Dewatering should be provided prior to beginning the excavation for structures that will be below a depth of about 4 feet. Dewatering should lower the groundwater to at least 2 feet below the depth of the excavation and provide for a stable subgrade for construction.

6.1.2 IMPACTS TO EXISTING INFRASTRUCTURE

The Pismo Beach outfall line, existing primary clarifier no. 2, fixed film reactor, secondary clarifier no. 1, and any other structures within 25 feet of ground disturbance could be impacted by deep compaction, shoring installation, dewatering, or excavations. Project plans and specifications should indicate that these structures are sensitive to settlement, require a submittal for supporting these structures during construction, and set limits for settlement and for monitoring of those structures. The plan from the contractor should include detailed plans for supporting structures and pipelines, methods and implementation, a schedule, settlement monitoring, and allowance for additional support/stabilization procedures if movement beyond the tolerable limits are exceeded.

Vibrations from deep compaction is intended to consolidate the ground and can cause settlement or subsidence of the adjacent ground surface. Deep compaction could impact within an estimated zone of 0.5h:1v projected upward from the bottom of the stone column or downward from the edge of treatment, or to about 15 to 20 feet radially from the limits of the deep compaction. Installation of sheet piles for shoring will also increase the potential for settlement and damage to nearby structures and improvements. Other methods of shoring including deep soil mixing, slurry cut-off walls, secant or tangent pile walls, could also be used and would have less impact on adjacent structures and improvements due to vibration. Permeation grouting or compaction grouting could potentially be used to protect adjacent structures during deep compaction.

6.2 EARTHWORK - GENERAL

6.2.1 SUGGESTED MATERIAL SPECIFICATIONS

The following material specifications can be used for materials recommended in various sections of this report. Yeh should review changes to the specifications or alternative materials to evaluate whether they conform to the recommendations of this report. Caltrans Standard Specifications (2015) are referred to as *Standard Specifications* throughout the recommendations.

Aggregate Base. Aggregate base shall consist of imported material conforming to Section 26-1.02B of the *Standard Specifications* for Class 2 Aggregate Base.

Aggregate - Stone Columns. Aggregate shall consist of hard, durable, rock and shall be free from clay lumps, cementation, organic material, and other deleterious substances. Aggregate shall have a nominal size of 1-1/2 inches and no more than 2 percent material passing the No. 200 sieve. Aggregate should also have a durability index of no less than 40.

Aggregate - Stabilization. Aggregate for stabilization shall consist of hard, durable, angular, clan crushed rock and shall be free from clay lumps, cementation, organic material, and other deleterious substances. Aggregate shall meet the gradation and quality requirements for 1-1/2 inch x ¾ inch per Section 90-1.02C(4)(b) of the *Standard Specifications* and with no more than 2 percent material passing the No. 200 sieve.

Coarse Sand (placed below floor slabs) shall consist of imported material conforming to ASTM C-33 fine aggregate, and have no more than 3 percent of the material passing the U.S. Standard No. 200 sieve.

Geotextile – General. Geotextiles shall be placed per Section 19-10 of the *Standard Specifications*. Depressions or holes left in the subgrade from the removal of obstructions shall be filled with sand or material being placed within the geotextile. Placement, anchorage and construction methods shall also comply with the manufacturer's recommendations.

> ***Separation/Filter Fabric.*** Geotextile for separation such as used for drains, subdrains, and the load transfer platform shall comply with Class C Filter Fabric in Section 96-1.02B of the *Standard Specifications*.
>
> ***Stabilization.*** Geotextile for stabilization such as placed below crushed rock for subgrade stabilization, on a soft subgrade or below rock fill shall comply with Class B1 Subgrade Enhancement Geotextile in Section 96-1.02O of the *Standard Specifications.*
>
> ***Geogrid.*** Geogrid for the load transfer platform within crushed rock, shall comply with Geogrids in Section 96-1.02P of the *Standard Specifications.*

Hot-Mix Asphalt. Hot-mix asphalt pavement shall be Type A conforming to Section 39, "Asphalt Concrete," of the *Standard Specifications*. Asphalt binder shall be grade PG 64-10.

Compacted Fill. Site soil or borrow pits or sources that will be used to supply fill and aggregates to be used for compacted fill below buildings, pavement, slabs-on-grade or select material shall be reviewed by the geotechnical professional before being imported to the site. However, imported fill materials shall comply with all specifications for that material as-placed at the site. Imported fill shall have an expansion index of 20 or less, be free of organics, oversized rock (that is over 4 inches in diameter), trash, debris, and other deleterious materials. Imported materials shall comply with all specified material requirements for the area where the material is being placed.

Structure Backfill. Imported materials to be placed for retaining wall/buried structure backfill within a zone extending up from the bottom of the foundation at a 1:1 plane shall conform to Section 19-3.02C, "Structure Backfill" of the *Standard Specifications*.

Initial/Pipe Zone Material. Pipe zone material shall consist of imported material having a sand equivalent (SE per ASTM 2419) of at least 30 and conform to Section 19-3.02F(2) of the *Standard Specifications*.

Drainage Rock. Gravel bedding or rock refill, such as for stabilizing bottoms of pipeline or utility trenches and used for sub-drainage shall consist of angular crushed rock that is free of organics, corrosive material, clay, recycled or reclaimed materials or other deleterious substances and conforming to Class 1 Permeable Material, Type A, in Section 68-2.02F(2) of the *Standard Specifications* or No. 57 stone per ASTM C33. Gravel bedding shall be fully encased in geotextile filter fabric when in contact with in-situ or subsequent pipe zone or trench backfill material.

Slurry Cement Backfill. Slurry cement backfill can be used as Subsequent Trench Backfill or as Initial/Pipe Zone Material per project specifications when approved by the Engineer. Slurry cement shall consist of 2-sack sand-cement slurry conforming to Controlled Low Strength Material in Section 19-3.02G of the *Standard Specifications*. Slurry cement backfill shall be a stable flowable mix and shall be consolidated using vibration during placement. Subsequent backfill or compacted material shall not be placed above slurry cement backfill until the slurry cement can support foot-traffic without more than ¼ - inch indentation. The Contractor shall provide ballasts or stabilize the pipe as necessary to prevent movement during placement of the slurry.

Trench Backfill. Trench backfill shall consist of imported or onsite material that is free of organics, debris, oversized material (greater than 3 inches), and other deleterious materials. Trench backfill material shall have at least 50 percent of the material passing the U.S. Standard No. 4 sieve, and/or comply with the applicable requirements for the area where trench backfill is being placed (such as the pavement structural section or under buildings).

Vapor Barrier. Vapor barrier installation procedures, including over-laps, seams, and sealing at penetrations or service openings, shall conform to ASTM E 1643-11, modified as appropriate based on written specifications from the vapor barrier manufacturer.

6.2.2 EXISTING MONITORING WELL

The existing monitoring well constructed in boring 16E-01 can be used for monitoring dewatering during construction or can be abandoned during construction. Abandonment should be per local and state requirements (https://water.ca.gov/Programs/Groundwater-Management/Wells).

6.2.3 CLEARING AND GRUBBING

Clearing and grubbing should be performed to remove existing vegetation and deleterious material from improvement areas that will be graded, receive fill, or serve as borrow sources. Soil containing pavement, debris, organics, disturbed materials, or other unsuitable materials, should be excavated and removed prior to commencing fill placement. Demolition areas should be cleared of existing fill, pavement, abandoned utilities, and soil disturbed during the clearing and grubbing process. Depressions left from the removal and demolition of materials should be filled with compacted fill material. Fill placement and compaction can then be performed according to the recommendations of this report.

6.2.4 COMPACTION AND GRADING

Fill placement and grading operations should be performed according to the recommendations of this report. Site preparation for the various structures are presented in Section 6.4. Fill should be compacted to the minimum levels recommended for the location where the material is placed as shown in Table 5. Relative compaction should be assessed according to the latest approved edition of ASTM Standard Test Method D1557.

Table 4: Recommended Relative Compaction

Location of Fill Placement	Recommended Minimum Relative Compaction
General	90% U.O.N.[3]
Utility trench bedding, pipe zone or backfill	90% U.O.N.
Fill or backfill placed within 3 feet of finished grade in pavement areas	95%
Asphalt concrete, aggregate base, or subbase	95%
Building pad and foundation areas within 3 feet of finished rough grade	95%
Structure backfill	90% U.O.N.

A qualified, registered geotechnical professional should observe grading operations during construction to verify that fill placement and compaction is being performed according to the recommendations of this report. Field density testing should be performed to help evaluate the compaction and moisture content of the materials being placed. Fill and aggregates delivered to the site, and excavated onsite soil that will be reused as fill or backfill, should be sampled and tested for conformance with gradation and quality requirements for the project. The frequency and locations of the tests should be at the discretion of the geotechnical professional. The project specifications should include provisions for the contractor to allow for testing and to provide any shoring, ingress-egress, or traffic control needed to safely perform the testing at the locations and depths needed.

6.2.5 FILL PLACEMENT

Site preparation and the removal of existing soil should be performed according to the recommendations of this report prior to placing fill. Jetting or ponding should not be permitted for placement or compaction of fill material. Fill material should be suitable for the area where the material is being placed and comply with the suggested material recommendations of this report.

Fill materials should be moisture conditioned and spread in lifts that are suitable for compaction with the equipment being used. Control of compaction layer thickness will be necessary to achieve compaction throughout the material being placed. Fill will typically need to be spread in loose lifts of approximately 8 inches or less to achieve compaction. Each layer should then be spread evenly, moisture conditioned by adding water or drying the material to a moisture content suitable for compaction, and be thoroughly mixed during the spreading to provide relative uniformity of material within each layer. The moisture content of the material should be such that the specified compaction can be achieved in a firm and stable condition.

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[3] U.O.N. – unless otherwise noted

Soft or yielding materials should be removed and replaced with properly compacted fill material prior to placing the next layer of fill. Fill and backfill materials may need to be placed in thinner lifts to achieve the recommended compaction with the equipment being used.

The fill should not contain rocks, gravel or other solid particles larger than 4 inches in the greatest dimension. Deleterious materials, such as concrete or pavement rubble, metal, glass or sharp objects should not be placed within the fill material being placed. Recycled or reused materials should only be used and placed within the fill when specifically permitted by the project specifications and the geotechnical engineer. Rocks should not be nested, and voids should be filled with compacted fill material.

6.2.6 SUBGRADE STABILIZATION

Subgrade stabilization should be provided in areas where unsuitable materials or soft subgrade conditions are encountered that will not allow for proper compaction of the subgrade materials, or consist of organic or other deleterious materials that will not provide suitable foundation support for new pavement areas. The project specifications should include a quantity for stabilizing localized areas of the subgrade for foundation areas of the secondary clarifier, aeration basin, miscellaneous pads, blower building, trench bottoms that are 4 feet or deeper, and other areas where soil is unstable. Specifications should allow for increasing and/or decreasing the quantity based on the conditions encountered during construction.

Subgrade stabilization should consist of removing the existing soil to a depth at least 1-foot below the bottom of the excavation or bottom of the unsuitable material, whichever is deeper. If the subgrade is wet or yielding, subexcavation should be performed using backhoe type equipment such that construction equipment will not operate on the exposed subgrade during excavation.

A geotextile for stabilization should be placed over the undisturbed subgrade. The geotextile should be placed without gaps or wrinkles and overlapped a minimum of 1-foot. Gravel for stabilization should comply with the material specifications (Aggregate – Stabilization) provided in this report. The aggregate should be fully encased in the geotextile to reduce the potential for the overlying base course to erode into the gravel.

The geotechnical professional should review the subgrade conditions encountered at the time of construction to evaluate whether or not stabilization of the subgrade is needed, and to recommend the depth and limits of the subexcavation and stabilization. Project specifications should provide for specifying stabilization of subgrade and allow for addition or a reduction in the measurement and associated costs.

6.2.7 EROSION AND DRAINAGE CONSIDERATIONS

Drainage should be provided such that surface water does not run over slopes or pond on pavements, slabs, or adjacent to foundations. Downspouts should be provided to collect roof drainage and direct surface water to drainage pipes or areas away from foundation areas. Concentrated flows and runoff should not be permitted to discharge on slopes. Down drains, solid pipes, or lined ditches should be provided to carry water to the base of slopes. Energy dissipation and erosion control devices should be provided at the outlet of drain pipes and in areas of concentrated runoff to reduce the potential for erosion. Landscaping and maintenance of graded areas and slopes should be provided to assist the establishment of vegetation and reduce the potential for erosion.

6.2.8 REUSE OF EXCAVATED ONSITE MATERIAL

Material encountered within the anticipated depths of excavation consisted of clayey sand artificial fill overlying alluvium composed of layers of silt, clay and sand. The excavated material will likely consist of a heterogeneous mixture sand, clay and silt that should be suitable for reuse as trench backfill above the pipe zone or as compacted structure backfill around and below structures when cleansed of organics or other deleterious material. The excavated material will likely be wet and will need to be dried to near optimum moisture content prior to reusing the soil as compacted fill. The on-site soil encountered within the anticipated depths of excavation is not considered suitable for reuse as select material, such as for pipe bedding, pipe zone material, aggregate base, structure backfill, gravel or drainage material.

6.3 GROUND IMPROVEMENT

Ground improvement consisting of vibro-replacement with stone columns is recommended to mitigate liquefaction and reduce the estimated seismic settlement at the site to within limits that could be tolerated by the proposed improvements as previously discussed in this report. Deep compaction with stone columns is a common method used to densify granular soil that is considered prone to liquefaction. Vibro-replacement with stone columns involves the insertion of a vibratory probe to a targeted depth, filling the resulting void with gravel and the probe is removed, and compacting gravel in lifts using the vibration from the probe.

The probe is typically mounted on a hollow casing to allow aggregate to be placed from a hopper located at the top of the probe through the casing and be discharged through the bottom of the probe. Pneumatic air can be used to help discharge the aggregate. As the probe is withdrawn, the resulting void is backfilled with gravel that is compacted with the vibratory probe to build out the stone column and densify surrounding granular soil layers. The probe is repeatedly lowered into the gravel to increase densification of the surrounding soil and is reflected by the buildup of amperage on the generator used to power the probe. Predrilling may be used to help advance the probe through dense, hard or stiff cohesive material that may be encountered. The process is repeated on a typical grid pattern until the desired area of the site has been suitably compacted to a specified relative density.

6.3.1 DEEP COMPACTION

Deep compaction using vibro-replacement with stone columns should be provided to improve potentially liquefiable sand layers encountered from the bottom of the proposed structures to elevation -26 feet (to approximately 36 feet in depth below the existing ground surface). The recommended ground improvement limits the estimated seismic settlement of the soil encountered in the CPT soundings to 1 inch for the design earthquake.

Limits and Depths of Compaction. Deep compaction should be performed to at least the limits shown on Figures 8 and 9. The recommended deep compaction should be performed in the two areas shown as Zone A and B on Figure 8. The deep compaction within these zones should consist of the following:

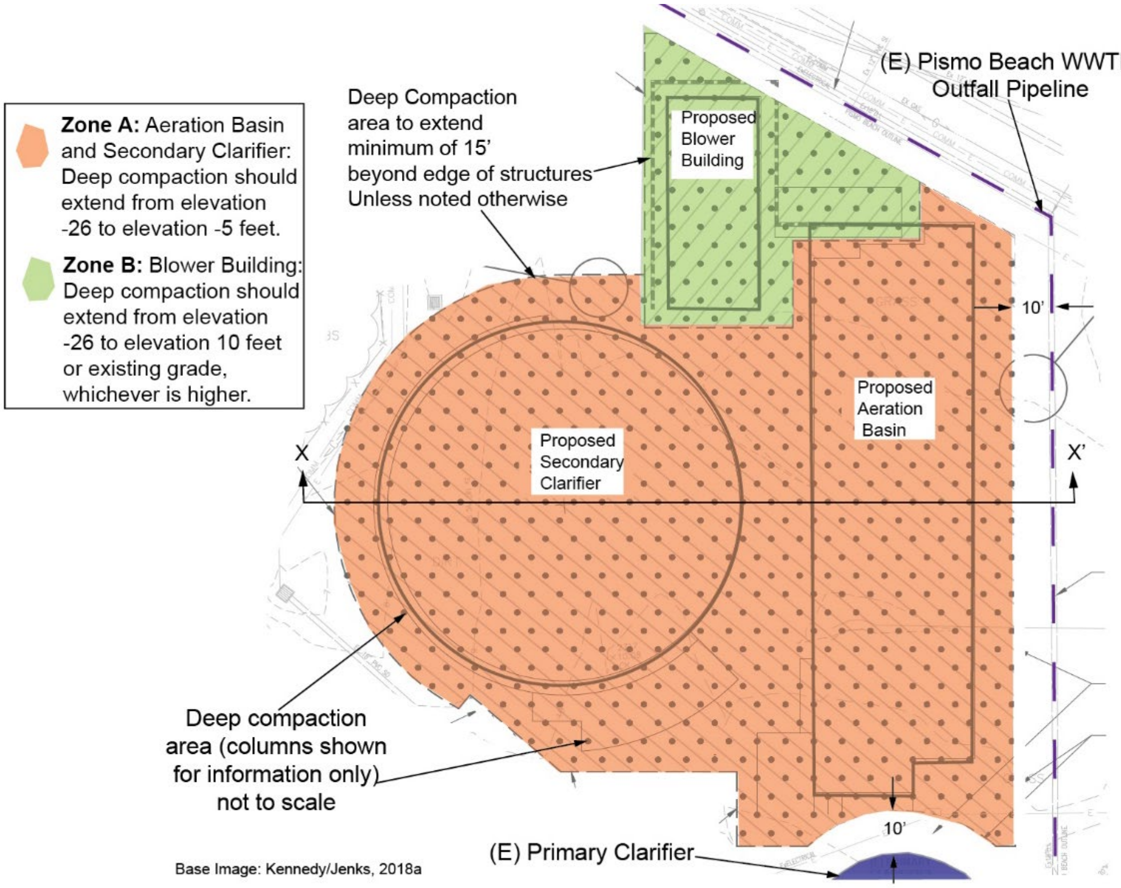

Figure 8: Recommended Ground Improvement - Plan View

- *Zone A: Aeration Basin and Secondary Clarifier*: Deep compaction within Zone A should extend from elevation -26 to elevation -5 feet. The void left by the probe above el. -5 feet to the ground surface can be filled with soil or gravel, since this material will be subsequently excavated to construct those structures and ground improvement above elevation -5 feet is not needed in this zone.
- *Zone B: Blower Building*: Deep compaction should extend from elevation -26 to elevation 10 feet (the finish floor elevation of the blower building) whichever is higher. The upper 5 feet of the area will be overexcavated and recompacted as a part of grading for the blower building.

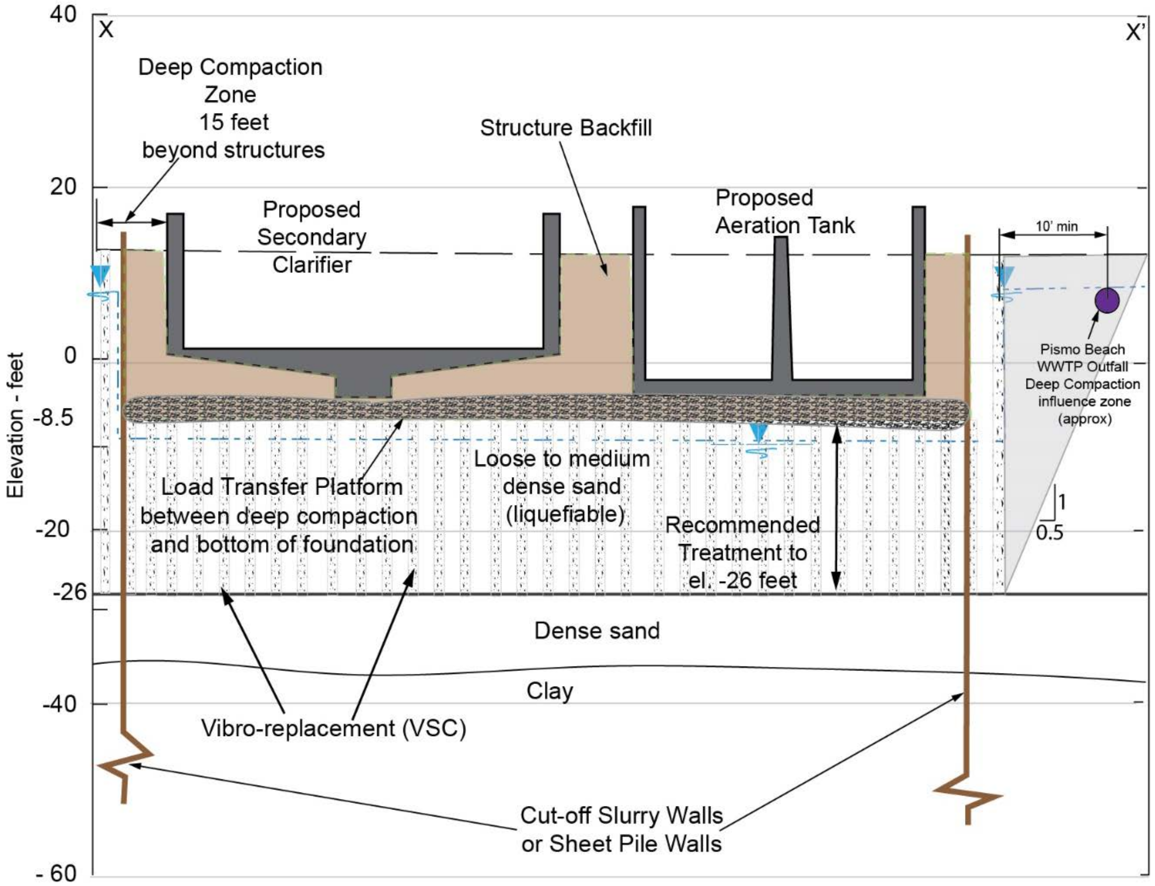

Figure 9: Recommended Ground Improvement – Section View X-X'

The horizontal limits of the treatment area should extend below the structure footprint within each zone to the recommended elevations and to a horizontal distance 15 feet beyond the edge of the proposed structures. The horizontal limits of deep compaction along the Pismo Beach outfall line, primary clarifier no. 2, stormwater lift station or other improvements may be adjusted to reduce the potential for damage to these structures from installation of deep compaction. Horizontal limits of the ground improvements (stone columns) may be reduced to provide an offset of a minimum of 10 feet from an existing structure.

Deep Compaction Plan. Prior to mobilizing to the site, the contractor should submit a plan for performing the deep compaction for review by the geotechnical professional. The plan should detail the layout and depths of the columns, the equipment that will be used, the methods of compaction, proposed aggregate with supporting quality and gradation test data, methods for placing stone columns, quality control, and the anticipated schedule to complete the work. The project specifications should allow for alternatives for ground improvement, such as deep soil mixing, to be considered provided they provide equivalent or better estimated performance outcomes than the recommended deep compaction.

Deep compaction should be confirmed by post-installation CPT testing (see Section 6.3.2) and conform to at least the following recommendations:

- Deep compaction should provide a normalized effective CPT tip resistance (q'_{cN}) of at least 150 tons per square for clean sand, as measured in sand, silty sand and sand with silt layers having a friction ratio less 1.5 percent and that are at least 2 feet thick, between 5 feet below the ground surface at the time of construction to elevation –26 feet.
- Predrilling can be provided as-needed to assist with installation of the probe though the existing fill or hard soil layers. Predrilling diameters should not exceed 60 percent of the diameter of the vibratory probe. The need for predrilling should be assessed and performed by the contractor as they deem necessary to achieve the recommended minimum CPT tip resistance.
- Stone columns should have a minimum area replacement ratio of 14 percent and be spaced at no more than 8 feet on center in a triangular grid pattern. Additional columns should be provided by the contractor at the midpoint of each grid where testing shows that the minimum CPT tip resistance (see Section 6.3.2) was not achieved during the initial column placement.
- Stone should consist of hard durable aggregate with a nominal size of 1.5 inches.
- Stone should be compacted by vibration in layers no greater than 4 feet until a build-up of at least 200 amps is achieved.
- The contractor should keep a daily record of what columns were constructed, the amount of stone placed in each column, the amperage build up during aggregate placement, noted depths where amperage buildup did not occur and where the amount of aggregate placed in the column exceeded the estimated amount.

Utilities and Lifelines. The recommended limits of deep compaction encompasses the area around the aeration basin, secondary clarifier, and blower building, but does not include deep compaction in areas where utilities are planned between the new and existing structures. Extending the estimated limits of deep compaction to include areas of utilities between structures could be provided to help reduce the potential for damage to those utilities that could result from differential movement between areas of ground improvement and unimproved ground. However, we understand from K/J that there are a number of existing buried utilities within these corridors that could potentially be damaged by subsidence or vibrations resulting from deep compaction. Other methods such as permeation grouting or compaction grouting could be used along utility corridors to reduce effect of liquefaction if needed.

Monitoring. Adjacent structures and infrastructure are present around the proposed ground improvement area. Construction of vibratory stone columns could result in ground subsidence beyond the specific limits of deep compaction recommended in Zones A and B and result in settlement of adjacent structures, pipelines, flatwork, pavements or other improvements. The zone of influence of stone columns can be estimated as the area within a line projected up from the bottom of the probe at 0.5h:1v (see Figure 9). The project plans and specifications should identify the existing Pismo Beach outfall line, primary clarifier no. 2, stormwater lift station and any other existing infrastructure that could be damaged and specify a monitoring program and the tolerable settlement of those facilities. The contractor should provide monitoring to help evaluate if deep compaction causes excessive movements of those facilities or if additional methods should be provided to reduce the potential for damage. The contractor should also submit a plan identifying methods that will be implemented to support infrastructure during construction (see Section 6.1.2).

Site Conditions after Compaction. Deep compaction typically will result in heaving of the ground surface, muddy surface conditions as water and mud are ejected from the hole during deep compaction, and excess gravel and spoils spilt over the site. The contractor should anticipate that the site conditions may not be suitable for construction traffic immediately following deep compaction, and that material may need to be removed to restore the site to the previous grades.

6.3.2 CONFIRMATION TEST PROGRAM

Testing should be provided during deep compaction to evaluate whether the recommended deep compaction has been achieved. At least one CPT sounding should be performed at the center of the grid for every 10 percent of the area where deep compaction is performed, or in areas where the contractor did not achieve proper amperage build up or gravel placement. A minimum of 5 CPT soundings are recommended for the ground improvement area in Zones A and B. The contractor should be responsible for procuring and scheduling the CPT rig to perform the testing to the top of the clay layer (approximately el. -35 feet) in accordance with ASTM D5778. CPT testing should be performed at locations selected and under the observation of the geotechnical professional.

Time should be allowed for porewater pressures to dissipate after the columns are constructed and prior to testing which is estimated to be between 3 and 5 days. The contractor can provide additional drainage (wick drains) if needed to speed the dissipation of porewater pressures or facilitate drainage. Additional compaction/stone columns should be provided where CPT test results (supported by daily construction records) show that the minimum level of compaction was not achieved or the work did not conform to the approved work plan.

The contractor should provide all raw electronic data from the CPT testing to the geotechnical professional. Data should be transmitted the day of data acquisition via USB drive or email in useable formats[4] for analyses. The geotechnical professional will review the data, perform calculations and check whether the deep compaction met the specification or additional deep compaction is recommended. Project specifications should allow for at least two weeks to review the data. Additional compaction, if needed, is typically provided by installing another stone column at the center of the grid in the areas where deep compaction was not achieved.

Construction quantities should be monitored to track the total volume gravel placed, and the ground surface should be surveyed before and after ground improvement to estimate the total amount of settlement or heave resulting from the compaction and gravel placement. Project specifications should provide for observation by the geotechnical professional of the deep compaction during construction for observing that the column spacing, diameters, and depth of treatment were provided.

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[4] File formats include .xls, .txt, or .cor.

6.4 SITE PREPARATION AND GRADING

6.4.1 BLOWER BUILDING

Following deep compaction, the existing soil within the blower building area should be excavated to a depth of 2 feet below bottom of the building foundation or 4 feet below the existing ground surface, whichever is deeper (see Figure 10). The excavation should extend horizontally to at least 5 feet outside the building footprint.

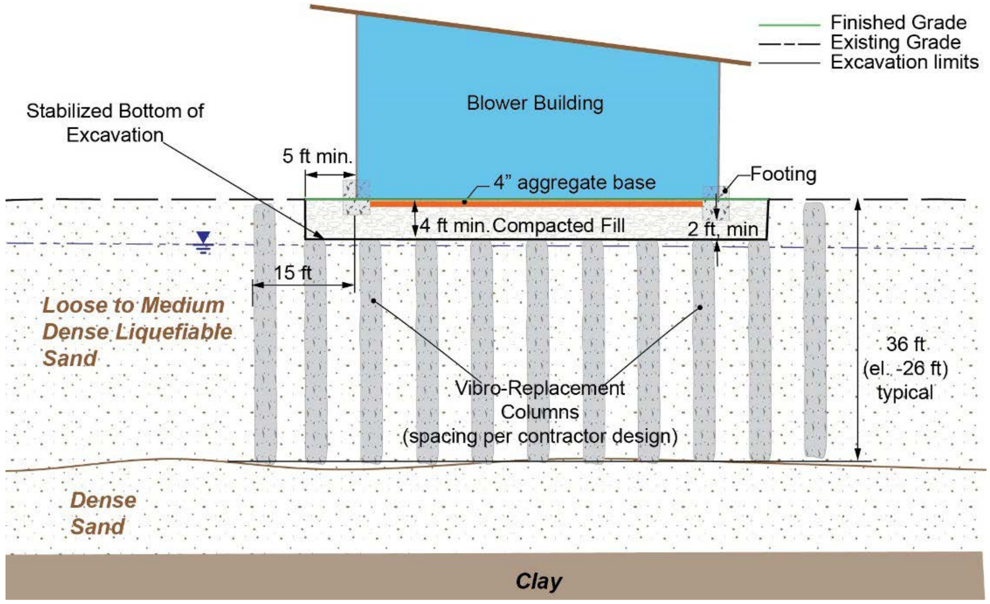


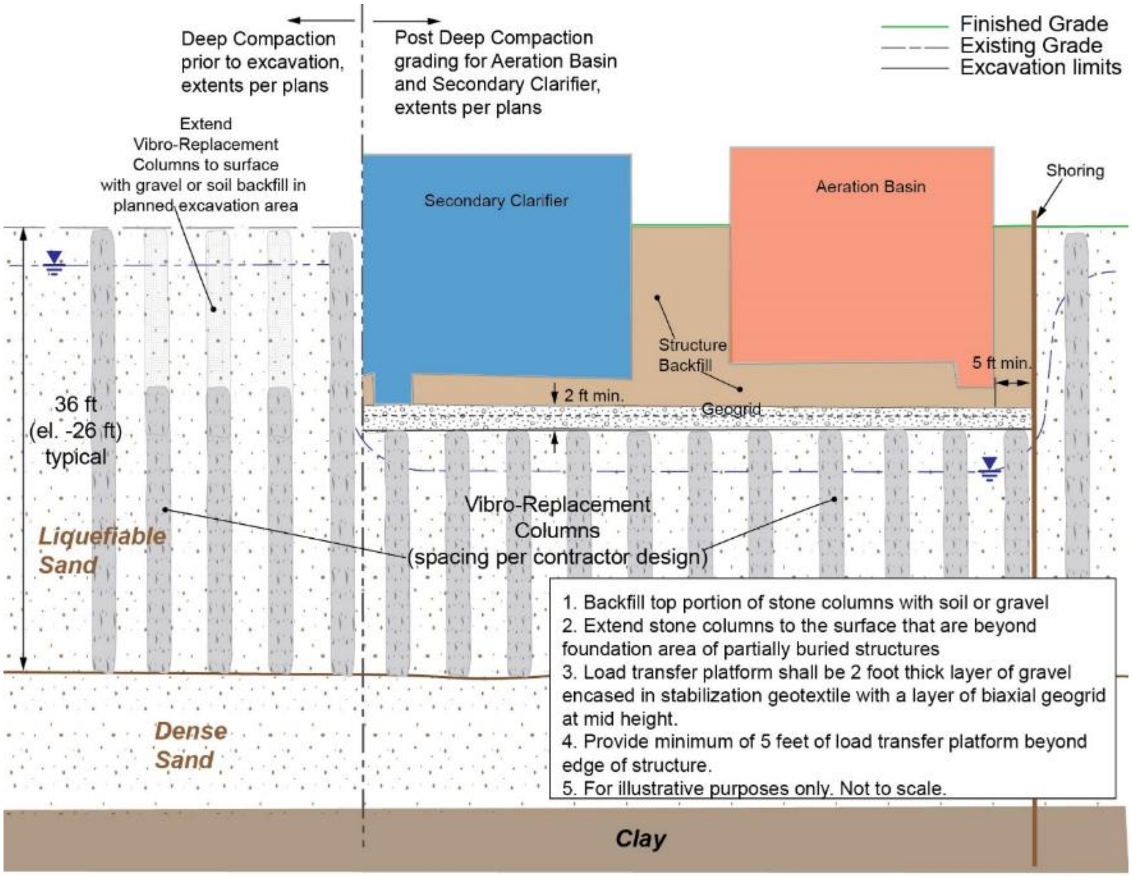
Figure 10: Earthwork - Blower Building

The bottom of the excavation should be reviewed by the geotechnical professional prior to placing fill to evaluate whether the subgrade is suitable for fill placement, or to provide additional recommendations if needed. The bottom of the excavation is expected to be saturated and unstable for the placement of fill. Stabilization should be performed across the bottom of the blower building pad area as recommended in Section 6.2.6. Subsequent lifts of fill should be placed above the stabilized bottom of excavation and compacted up to finish pad grade as recommended in this report.

6.4.2 AERATION BASIN AND SECONDARY CLARIFIER

Dewatering should be provided prior to beginning the excavation for the aeration basin and secondary clarifier. Dewatering should lower the groundwater to at least 2 feet below the depth of the excavation and provide for a stable subgrade for construction.

The subgrade below the aeration basin and secondary clarifier is expected to be relatively wet and vulnerable to disturbance. The lower 4 feet of the excavation should be performed with caution, using excavator type equipment, and such that construction equipment will not traverse across on the bottom of the excavation. To reduce the potential for further disturbance to the foundation support soil below the structures, at least 2 feet of crushed rock (aggregate for stabilization) encased in a geotextile should be placed on the undisturbed subgrade at the bottom of the excavation. The stabilization layer should be installed by placing a layer of geotextile for stabilization over the undisturbed subgrade. Crushed rock for stabilization should then be advanced over the subgrade in two 1-foot lifts. The first lift should be placed with an excavator or with light equipment such as a bob cat. The second lift can be placed using a wide-track dozer or similar equipment with low ground pressures. The rock should be reinforced with a single layer of geogrid (see Figure 11). The geogrid should be placed mid-height within the crushed rock layer as the aggregate is being paced. The excavation and stabilization layer should extend at least 5 feet beyond the footprint of the structures. The stabilization geotextile should completely encompass the gravel bedding. The resulting surface



1. Backfill top portion of stone columns with soil or gravel
2. Extend stone columns to the surface that are beyond foundation area of partially buried structures
3. Load transfer platform shall be 2 foot thick layer of gravel encased in stabilization geotextile with a layer of biaxial geogrid at mid height.
4. Provide minimum of 5 feet of load transfer platform beyond edge of structure.
5. For illustrative purposes only. Not to scale.

Figure 11: Earthwork - Secondary Clarifier and Aeration Basin

of gravel bedding should be consolidated with a minimum of 4 passes with a vibratory plate or by track-walking with a wide-tracked dozer.

The bottom of foundation elevations for the structures are stepped and vary by approximately 5 feet. We recommend that the stabilization layer be placed at a consistent elevation and that compacted fill be placed above the gravel blanket to bottom of foundation grade for the structures. The fill could be placed across the pad and then foundation excavations made into the compacted fill to facilitate constructability.

6.4.3 EQUIPMENT PADS AND SITE WALLS

Soil within the foundation areas of equipment pads and site walls should be removed to a depth of 2 feet below the existing ground surface or at least 1 foot below the bottom of the foundation, whichever is deeper. Equipment pad and site wall foundation areas are defined as the footprint of the equipment pad or site wall foundation and extending a minimum of 1 foot horizontally beyond the edge of the foundation. The bottom of the excavation should be reviewed by the geotechnical professional prior to placing fill to evaluate whether the subgrade is suitable for fill placement, or to provide additional recommendations if needed. If the bottom of the excavation is saturated and unstable for the placement of fill a stabilization should be performed as recommended in Section 6.2.6. If the bottom of the excavation is firm and stable, the bottom should then be scarified to a depth of at least 9 inches, moisture conditioned, and compacted in-place to at least 95 percent relative compaction. Subsequent lifts of fill should be placed and compacted up to finish pad grade.

6.5 SEISMIC DATA

Yeh estimated the design earthquake magnitude using the 2017 USGS Unified Hazard Tool (USGS 2017b) for an earthquake having a 2 percent exceedance probability in 50 years. The design earthquake is a M6.7 event with a corresponding estimated peak ground acceleration of approximately 0.51g at the site. The design earthquake is characterized as a potential near-field earthquake occurring on the San Luis Range fault system (a compilation of local faults mapped along the Highway 101 corridor that include the Wilmar Avenue fault, Oceano, Olson trace, and Santa Maria River faults). The design earthquake is estimated to result in ground motions two to four times stronger than those that occurred in the site vicinity during the San Simeon Earthquake.

Seismic data were estimated for the project site using the coordinates and the estimated shear wave velocity presented in Table 4 in conjunction with the United States Geological Survey (USGS) Seismic Design Maps (USGS 2017). The estimated average shear wave velocity for the upper 100 feet (V_{s100}) for the conditions encountered in the explorations is 802 feet per second (244 meters per second) based on CPT sounding data. While the estimated average velocity corresponds to a Site Class D, very dense soil site per Table 20.3-1 of ASCE 7-10 (2013), the site is classified as a Site Class E due to the having a soft clay layer and measured water content. The recommended ground improvement to mitigate for liquefaction potential is sand layers would not improve the clay layers. The seismic data were estimated for a Risk Category "I, II, or III." The USGS website provides spectral response accelerations estimated in accordance with ASCE 7-10 (2013).

Table 5: Seismic Data

Seismic Parameter	Value
Latitude, degrees	35.1009
Longitude, degrees	-120.6249
Site Class	"E" soft clay
Earthquake Magnitude	6.7
Peak ground acceleration (PGA) 2% in 50 years	0.51g
S_s, Seismic Factor for Site Class B at 0.2 seconds	1.222
S_1, Seismic Factor for Site Class B at 1 second	0.446
F_a, Site Specific Site Coefficient	0.900
F_v, Site Specific Site Coefficient	2.400
S_{MS}, Site Specific Response Parameter at 0.2 seconds	1.100
S_{M1}, Site Specific Response Parameter at 1 second	1.070
$S_{DS} = 2/3\ S_{MS}$	0.733
$S_{D1} = 2/3\ S_{M1}$	0.714

6.6 FOUNDATION DESIGN

6.6.1 SHALLOW FOUNDATION DESIGN

The blower building can be supported on continuous or spread footing foundations bearing in compacted fill prepared in accordance with the recommendations of this report. Foundations for the blower building, equipment pads or site walls can be designed using a maximum allowable bearing pressure of 3,000 pounds per square foot. Continuous and spread foundations should be designed with a minimum 1-foot width and be embedded a minimum of 2 feet below lowest adjacent grade, finished floor, existing slab elevation, or adjacent pavement elevation, whichever is lowest. The recommended bearing pressure for continuous and pad footings can be increased by 400 pounds per square foot for each additional foot of footing width or for each additional foot of embedment, to a maximum of 4,000 pounds per square foot. Designers should consider the depth to groundwater (approximately 4 feet below the existing ground surface) at the project site if the footings will be deepened for increased bearing pressure. Foundation excavations might need to be dewatered if footings extend to this depth.

The recommended allowable bearing pressure can be increased by 1/3 when considering seismic or other transient loading conditions. The edge pressure can exceed the maximum allowable bearing pressure when considering eccentric loads provided the resultant force acts within the middle third of the footing.

6.6.2 Mat Foundation Design

The aeration basin, secondary clarifier, and miscellaneous equipment pads can use mat foundations to support the structures. The modulus of subgrade reaction was estimated from settlement analysis of a mat foundation with dimensions and applied bearing pressures of the proposed structures provided by Kennedy/Jenks. Mat foundations can be designed using modulus of subgrade reaction values as shown in Table 6:

Table 6: Mat Foundation Design

Structure	Modulus of Subgrade Reaction (lb/in^3)
Aeration Basin	32
Secondary Clarifier	32
Equipment Pads	20

6.6.3 Settlement Considerations

Foundations should be designed to consider total static settlement of approximately 1-inch total and 3/4-inch differential in 30 feet for foundations designed according to the recommendations of this report. Ground improvement (see Section 6.3) should be used to limit seismic settlement and reduce the potential for liquefaction to impact the improvements.

We estimated that approximately 3.5 to 8 inches of seismic settlement could occur at the site when considering the design earthquake. The recommended deep compaction should reduce the estimated total seismic settlement to approximately 1-inch for the design earthquake. California Geological Survey Special Publication 117 (2008) suggests that the differential settlement across the structure should be estimated as ½ of the estimated total seismic settlement (approximately ½ to 1 inches). This estimated seismic settlement is in addition to the estimated static settlement.

6.6.4 Lateral Earth Pressures

Buried or partially buried structures such as the aeration basin and secondary clarifier should be designed to resist lateral earth pressures associated with both static and dynamic (earthquake) loads. Walls that are backfilled with structure backfill and are free to rotate or move can be designed using active earth equivalent fluid weights. Walls that are braced or are too stiff to allow movement should be designed using at-rest equivalent fluid weights. Design of the structures should account for the lateral earth and water pressures using the equivalent fluid weights provided below in Table 7 for the expected drained/undrained conditions.

Table 7: Lateral Earth Pressures

Retained Material	Unit Weight (pcf)	Active Equivalent Fluid Weight, (pcf)	At-rest Equivalent Fluid Weight, (pcf)
		Level	Level
Structure Backfill or Site Soil (Sand) – Drained (above flood elevation)	130	35	55
Structure Backfill or Site Soil (Sand) - Saturated	130	23 + 62.4	34 + 62.4
Backfill zone is the active wedge that includes the material within a plane projected up from the bottom of the wall at (1h:1v).			

Seismic Earth Pressures. The general limit equilibrium method and SLIDE software (Rocscience 2017) was used to estimate lateral earth pressures on buried structure walls ranging in depth from 5 to 15 feet needed to resist seismic soil loads. Yeh estimated a horizontal pseudostatic coefficient of 0.21 from the design peak ground acceleration and methods referenced in CGS (2008). The pseudostatic coefficient corresponds to a driving force causing a seismic wall displacement up to approximately 2 inches. If the secondary clarifier, aeration basin and other partially buried structures will be designed for active earth pressures they should be designed using an additional dynamic lateral earth pressure of 8H (where H is the retained height in feet) applied as a uniform pressure in pounds per square foot on the back of the wall. The seismic increment can be ignored if the structures are designed for at-rest equivalent fluid weights.

Surcharge Loads. The recommended earth pressure coefficients do not account for surcharge loads acting on the backfill. Footings bearing behind buried structures should be embedded below a 1:1 line projected upward from the top of the heel of the structure footing to reduce the potential for foundation pressures to act on the wall. Yeh should provide additional recommendations that are specific to the loads and configurations if surcharges from structures will be within this zone. Traffic surcharges can be estimated as an additional 2 feet of soil cover, equal to a uniform pressure of 72 pounds per square foot, and applied to the upper 10 feet of the wall. Traffic surcharges can be neglected if the load is applied beyond a 0.5:1 line projected upward from the heel of the structure footing or the bottom of the wall of the structure. Surcharges from construction equipment are also not included in the recommendations, but should be considered during design and construction of the structures.

Drainage. We have assumed that the walls for the aeration basin and secondary clarifier will be designed for undrained conditions. Design of below grade structures should account for undrained backfill conditions. We have assumed that site retaining walls around the blower building loading dock will be designed for drained conditions. Drainage should be provided behind the site retaining walls to reduce the potential for water to accumulate within the backfill and increase lateral pressures. Water proofing or drainage panels can be provided on the backside of the retaining walls if needed. A continuous layer of drainage material consisting of either 1-foot of drainage rock or geocomposite drainage panels should be provided along the backside of the wall. The drainage rock material should be terminated 1-foot below the finished grade of the wall backfill or at bottom of slab or pavement structural section and be topped with a cap of compacted on-site soil, topsoil to help reduce the potential for water to infiltrate directly behind the wall.

Weep holes, collector pipes, or other measures should be provided to assist in the removal of water from the backfill and to prevent the buildup of hydrostatic pressures behind site retaining walls. The collector pipe should be perforated pipe placed near the base of the wall, encased by at least 1 cubic foot of drainage rock per foot of pipe or wrapped in the geocomposite drainage panel if used to drain the wall, and directed to an outlet to a point downstream of the wall. A geotextile for separation should be provided around the gravel for the collector pipe, and between the wall drain and between the retaining wall backfill materials. Weep holes should be provided at 4-foot spacing along the base of the wall. The weep hole should be backed by a geotextile sack containing at least 1 cubic foot of drainage rock.

6.6.5 RESISTANCE TO LATERAL LOADS

Resistance to lateral loading can be provided by sliding friction acting on the base of foundations combined with passive pressure acting on the sides of the foundations. A coefficient of friction of 0.4 should be used to estimate the sliding resistance along the bottom of the foundation bearing in compacted fill. An ultimate passive resistance of 300 pounds per cubic foot, equivalent fluid weight, should be used to estimate the lateral resistance acting on the sides of the footings. A 1/3 increase in the passive value can be used when considering short-term wind or seismic loads. Passive resistance should not be used for the upper one foot of soil that is not constrained at the ground surface by slab-on-grade or pavement.

6.6.6 RESISTANCE TO UPLIFT LOADS

Groundwater was encountered at a depth of approximately 4 feet below the existing ground surface in the borings performed at the site. We also understand that the project site is in a

Federal Emergency Management Agency (FEMA) flood zone. Emptying a buried or partially buried structure reduces its total mass and decreases its ability to withstand buoyant forces acting upward on the structure. The design of below grade structures whose foundations will extend below the groundwater table or flood levels should consider buoyant forces that will act upward on the structure. The maximum expected flood elevation, relative to the elevation of the base of the structures, should be used in estimating uplift pressures for the structures. Pressure relief valves can be installed in the walls of below grade structures to help reduce uplift forces, if the structure does not need to be dewatered for maintenance. Valves should be designed to account for abrupt changes in pressures from lowering of water levels within the structures. Sequencing of filling, backfilling, and abandoning dewatering efforts should consider the possibility of the structures to float during construction. Project specifications should require that a sequencing plan be implemented to prevent the structures from moving upward due to buoyancy forces.

If needed, uplift forces due to buoyancy can be resisted by the buoyant dead weight of the structure, friction acting between the exterior walls of the structure and the surrounding soil, and foundations that extend beyond the walls and are buried with compacted fill. The maximum allowable frictional resistance between the soil and the buried concrete structure can be estimated as 0.2 times the effective overburden stress. The effective overburden stress, in psf, can be estimated using an effective buoyant unit weight of 60 pcf for submerged soil times the depth in feet. Uplift resistance due to wall friction should be neglected if filter fabric will be placed between the wall backfill and the ground or if the walls will be coated with a waterproofing membrane.

6.7 DESIGN OF SLABS-ON-GRADE

Recommendations for the design of interior slabs-on-grade are presented below for use with ACI 302.2R-06, “Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials,” published in 2006. It is anticipated that design of the blower building would incorporate moisture vapor barriers for the design of the slab. The performance of flooring depends on many factors including sub-slab relative humidity, concrete materials and water-cement ratio, internal relative humidity, and construction aspects, such as curing, length of drying, environmental conditions, pH, and other factors. The design engineer should review the ACI guide for background on moisture vapor penetration through concrete slabs relative to their design and issues regarding protection from delamination of flooring, blistering, staining, mold growth and other problems related to performance of moisture-sensitive flooring.

Slabs-on-grade should be supported on compacted fill placed according to the recommendations for grading within building areas. A 4-inch thick layer of coarse sand and a vapor barrier conforming to the Suggested Material Specifications (Section 6.2.1) of this report should underlie interior floor slabs with coverings. The vapor barrier may be placed directly over finish pad grade, in the middle of the 4-inch thick coarse sand layer, or directly under the concrete and omit the coarse sand. Compaction of the coarse sand layer should be performed with hand-operated equipment that will not damage the vapor barrier. Slabs to be overlain by floor coverings that are sensitive to moisture should be sealed and prepared according to the floor manufacturer’s recommendations.

Concrete slabs not subject to vehicular traffic should be at least 4 inches thick. The design engineer should design reinforcement for floor slabs based on the expected loading conditions and the current standards for control of cracking and structural design. The reinforcement should be placed above the mid-depth of the slab. The contractor should provide means to maintain the location of reinforcement during construction and concrete placement.

Exterior concrete flatwork and slabs-on-grade (such as walk ways and lightly loaded flatwork) should be at least 4 inches thick. Flatwork can be underlain with at least 4 inches of aggregate base to help limit erosion and mud-jacking from beneath joints or edges in the slab. For exterior slabs without vehicular traffic, at least the upper 1-foot of the subgrade below slab-on-grade should be compacted to at least 90 percent relative compaction. Exterior slabs that may be subject to vehicle traffic should be designed in accordance with the pavement design recommendations in this report. Expansion joints, control joints and/or reinforcement of slabs should be provided according to Portland Cement Association guidelines or other applicable design standards to control cracking such as ACI 318.

6.8 UTILITY TRENCHES AND PIPELINE DESIGN

A typical trench detail showing the cross-sectional limits of the bedding, initial backfill, and subsequent backfill material is provided in Figure 12. Pipe bedding, pipe zone, and trench backfill should comply with the suggested material specifications presented in this report.

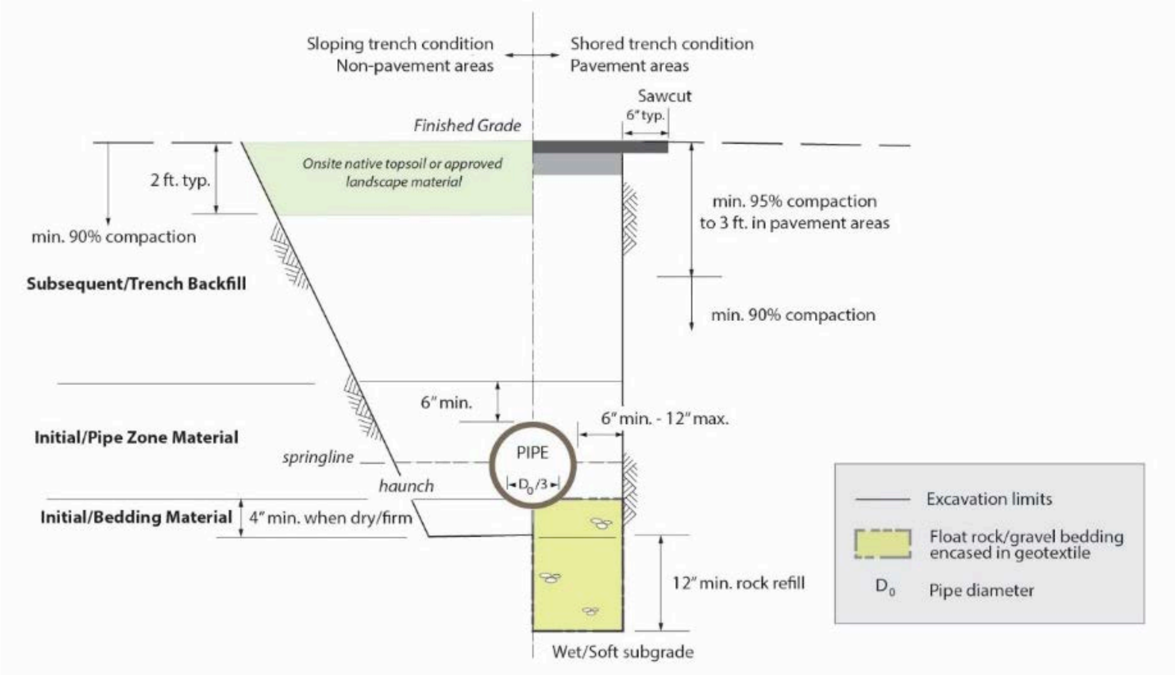


Figure 12: Typical Trench Detail

6.8.1 PIPE SETTLEMENT

Pipe connections to structures or equipment should be designed to accommodate estimated total settlement of 3.5 to 8 inches beyond the zone where deep compaction will be used. Flexible joints and accommodations for movement of pipes at structure connections should be incorporated into the design if needed.

6.8.2 FOUNDATION SUPPORT

Prior to placing bedding material, the foundation support soil exposed at the trench subgrade should be reviewed to allow for proper bedding and stabilization of the subgrade below the trench if needed. The trench bottom is expected to consist of loose to medium dense sand. Groundwater should be expected where excavations are 4 feet or deeper. We recommend that the project plans and specifications provide for stabilization of the trench subgrade with drainage gravel and stabilization geotextile for all pipeline trenches that extend 3 feet (or el. 7 feet) or deeper below the ground surface.

Subgrade stabilization should consist of removing the trench subgrade to 12 inches below the bottom of the trench, and replacing the material with drainage gravel wrapped in a stabilization geotextile conforming to the suggested materials specifications of this report. The gravel can be substituted for sand bedding material when subgrade stabilization is provided.

Trench stabilization may be omitted if the bottom of the trench is firm and stable. In such conditions, prior to placing bedding, the trench bottom should be scarified to a depth of 9 inches, moisture conditioned and compacted in place to at least 90 percent relative compaction.

6.8.3 PIPE BEDDING

Bedding material is initial backfill placed between the trench subgrade and the bottom of the pipe. The pipe should be placed on the bedding such that the middle third of the pipe (Do/3 on Figure 12) is in contact with the bedding prior to placing initial backfill within the pipe zone. The bedding maybe loosened along the invert of the pipe if necessary to help form the cradle.

Prior to placing bedding material, the foundation support soil exposed at the trench subgrade should be reviewed to evaluate whether the subgrade is suitable for placement of bedding. Pipe bedding should typically consist of at least 4 inches of select sand placed on an undisturbed subgrade that is firm and unsaturated. The subgrade at the trench bottom should be recompacted, if needed, to achieve the recommended compaction in the bedding material itself. Pipe bedding should be compacted to at least 90 percent relative compaction.

6.8.4 INITIAL BACKFILL/PIPE ZONE MATERIAL

Initial backfill is material placed in the pipe zone from the top of the bedding material to at least 6 inches above the crown of the pipe.

Granular. Granular initial backfill materials should be compacted to at least 90 percent relative compaction prior to placing the subsequent backfill. Compaction within the pipe zone should be performed such that the pipe is fully supported during compaction and excessive deformation or damage to the pipe does not occur. Backfill should be placed evenly on either side of the pipe to help support the pipe. Backfill should not be placed above the springline until the fill below springline has been placed and compacted to properly support the haunches.

Flowable Backfill. Sand/Cement slurry can be used for initial backfill in lieu of granular material when reviewed by the design engineer and permitted by the pipe manufacturer. The cement slurry will likely not provide additional support or limit deflection of the pipe, unless the slurry is allowed to cure prior to placing subsequent backfill. The slurry should be placed over the crown of the pipe to allow for the slurry to be delivered to either side of the pipe simultaneously during placement. The slurry should be vibrated during placement to help consolidate the material and assist in filling below the haunches of the pipe. The pipe should be ballasted and secured to reduce the potential for the pipe to be displaced or float during placement. Slurry should not be placed above the springline of the pipe until the slurry below the springline has adequately set or is specifically permitted by the design engineer and pipe manufacturer. Subsequent pipe zone and trench backfill material can be placed once the cement slurry has set such that foot traffic does not leave an imprint in slurry of more than ¼-inch. Longer curing times, 2 to 7 days, are needed if springline support that is superior to soil backfill is to be achieved.

6.8.5 SUBSEQUENT/TRENCH BACKFILL

Subsequent backfill is material placed in the trench from the top of the pipe zone to finished grade, and includes the pavement structural section in applicable areas. Subsequent backfill should consist of native or imported granular material conforming to the material recommendations of this report. Subsequent backfill placed within the upper 3 feet of the trench in pavement areas should be compacted to at least 95 percent relative compaction. Material placed deeper than 3 feet, and in trenches outside of pavement areas, should be compacted to at least 90 percent relative compaction unless a higher degree of compaction is otherwise recommended. In non-pavement areas the upper 2 feet of the backfill should consist of on-site native soil or appropriate landscape soil to help reduce the potential for surface water to infiltrate the trench.

6.8.6 TRENCH PATCH

In existing pavement areas, the trench should be saw cut along the trench line and 6 inches beyond prior to performing the excavation. The width of the trench will be determined by the contractor and should comply with the minimum dimensions shown on Figure 12. Any broken or loosened asphalt that results from the cutting operation or instability of sidewalls during excavation should also be removed. The trench patch should at a minimum match the thickness of the existing asphalt concrete plus an additional 2 inches. Aggregate base provided below the asphalt layer should have a thickness that will match the bottom of the existing structural section.

6.8.7 EXISTING UTILITIES

Existing buried utilities are present within the planned improvement areas. Existing utilities in structure areas should be removed, relocated, and/or abandoned as part of site clearing and grubbing. Utilities that will be abandoned should be removed and replaced with compacted fill. If the utilities are not entirely removed, they should be filled with sand-cement slurry or concrete and be exposed and backfilled with properly compacted fill. Otherwise, existing pipelines and utilities will need to be supported during installation of new pipeline.

Excavations parallel to and extending below existing utilities can reduce the support of pipes. The loss of support could lead to movement, distress, and possible breakage of the utility pipe. This potential is exacerbated due to the presence of shallow groundwater at this site. An excavation plan should be prepared by the contractor and reviewed by the design engineer prior to implementation. Methods of stabilization and protection of existing utilities should be included in the plan.

6.8.8 CONSIDERATION FOR FOUNDATIONS AND UTILITIES

The locations of existing and new utilities relative to foundations and earthwork should be considered in the foundation, structural, and grading plans for the project. The proximity of foundation and utility locations should be considered. Foundation support of structures could potentially be compromised by the presence of utility trench backfill within the foundation zone. Utilities should generally be located above a 1:1 line projects down from the edge of existing or proposed structures. *If the pipe or utility extends with that the 1:1 line, Yeh should review the location to evaluate if there are alternatives or potential impacts to the structure foundation.*

Additionally, foundation loads could potentially be exerted on pipelines, for which the pipes are not designed. Utility and service lines extending inside the footprint of structures should be placed above the bearing level of the foundation. Pipelines placed beneath foundations should be placed and backfilled according to the recommendations of this report. Trench backfill should be properly compacted to avoid settlement or additional impacts to adjacent structures.

6.9 CORROSION CONSIDERATIONS

Tests for pH and electrical resistivity were performed on selected samples from the site. The test results are presented in Appendix B. The pH for the four samples tested ranged from 8.09 to 8.97. The minimum soil resistivity of the tested samples ranged from 817 to 2008 ohm-centimeters. One sample having a soil resistivity less than 1,000 ohm-centimeters was also tested for soluble sulfate and chloride concentrations. Measured soluble sulfate concentrations were 122 mg/kg[5] and soluble chloride concentrations of the sample was 117 mg/kg. Design of the project should consider corrosivity test results using appropriate design standards.

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[5] Milligrams per kilogram or parts per million

6.10 PAVEMENT DESIGN

The project will include new hot-mix asphalt (HMA) and Portland cement concrete pavement around the structures. Minimum design thicknesses are recommended based on pavement design standards provided in the Caltrans (2017) *Highway Design Manual.*

6.10.1 STRUCTURAL SECTIONS

A subgrade sample recovered from the borings had a tested R-value of 51. Caltrans design standards limit subgrade R-values to 50, therefore, an R-Value of 50 was used in the design of flexible pavements for this site. The geotechnical professional should review subgrade during construction to review whether or not the material exposed at subgrade is generally consistent with the material used in the design of pavement. Additional R-value sampling and updated structural sections may be needed if the subgrade material is different than found during our investigation. Structural section thicknesses were estimated considering a two-layer system of HMA over aggregate base (AB) and Portland cement concrete (PCC) pavement over aggregate base. The following table provides pavement structural sections estimated for assumed Traffic Index (TI) values of less than 5 to 6 for the project. The geotechnical professional should be contacted if pavement sections for TI's not listed are needed for design.

Table 8: Recommended Pavement Structural Sections ($R_{subgrade}$ = 50)

Traffic Index (TI)	20-year Minimum Thicknesses (inches)	
	HMA	PCC
< 5	3 inches HMA over 4 inches AB	6 inches PCP over 4 inches AB
5.5	3 inches HMA over 4 inches AB	6 inches PCP over 4 inches AB
6.0	3 inches HMA over 5 inches AB	8 inches PCP over 6 inches AB
HMA: Hot-Mix Asphalt AB: Class 2 Aggregate Base PCC: Portland Cement Concrete		

6.10.2 PAVEMENT MAINTENANCE

The pavement condition should be periodically evaluated to help plan and scope the need for maintenance and rehabilitation following the initial construction of the pavement. Maintenance of asphalt concrete pavements should typically include periodic fog, chip or slurry seals to reduce the potential for weathering, as well as overlaying with additional HMA when needed to strengthen and further the life of the pavement.

6.11CONSTRUCTION CONSIDERATIONS

6.11.1 GROUNDWATER AND DEWATERING

Groundwater was encountered at a depth of 4 to 6 feet below the ground surface. Dewatering to lower groundwater levels for construction will be needed for excavation depths that are 4 feet or more. Dewatering should be performed in a controlled manner that includes the use of wells, well-points, gravel trenches, or other means of dewatering to lower the water surface elevation within the limits of the planned excavation as-needed to provide a stable subgrade for construction. Dewatering young alluvial sediments or porous soil types, such as those found at this site, are prone to consolidate or collapse when the groundwater level is lowered. Subsidence of the ground surface over the area where dewatering can occur as a result. Project specifications should indicate that dewatering should be concentrated to lower the groundwater elevation within the footprint of the excavation and only to the depth needed to facilitate construction as dewatering of soil below existing structures could result in settlement of those structures. The impact of dewatering on existing structures will be reduced if the contractor sufficiently provides support to those structures and infrastructure.

Dewatering facilities should be installed prior to beginning excavation, and time should be allowed for lowering of the groundwater table before beginning excavation. Secondary dewatering using sumps placed in the bottom of excavations and stabilization of the subgrade may be needed in addition to the initial dewatering. Well screens and sumps should be designed with properly designed filters such that sand and fine-grained materials are not removed from the soil during dewatering operations. Observation monitoring wells or points should be provided to check that groundwater has been lowered to a depth of at least 2 feet below the depth of excavation prior to beginning excavation.

6.11.2 TEMPORARY EXCAVATIONS AND SHORING

Soil within the proposed excavations are anticipated to be predominantly Type C sandy soils based on Cal OSHA guidelines for the design of temporary slopes and shoring systems. Type C soils above the groundwater level should be excavated no steeper than 1.5h:1v. Dewatering in advance of the excavation is needed to provide stable conditions within the excavation. Slopes should not be considered stable when excavated below the groundwater table or there is seepage daylighting on slopes. The excavation plan should consider and provide for support of the presence of adjacent structures, buried utilities, traffic or slopes.

Shoring systems to support temporary slopes typically consist of trench shields, sheet pilings, or braced excavations designed to support the anticipated soil and groundwater conditions and depth of excavation. “Dragging a shield” is a common method of providing worker safety during trenching and construction. However, unless specific provisions exist to emplace the shield tight against the sidewalls, a shield provides no support for the trench sidewalls and should not be considered an appropriate shoring system for this project due to the potential for shallow subsurface water and trench collapse. Shoring systems, such as sheet piling, slurry walls, or some other form of sealed shoring that actively supports the excavation, should be embedded adequately below the base of the excavation to cutoff groundwater and help stabilize the base of the excavation. Embedding the shoring into the underlying clay layer which we encountered at approximate el. -35 feet could aid in cutting off the excavations from the surrounding sandy soil and limiting the dewatering to within the confines of the shoring system. The contractor should provide a competent person to review the excavations and shoring requirements based on the conditions encountered in accordance with OSHA requirements.

According to OSHA, the lateral earth pressure acting on shoring can be estimated as a uniform soil pressure plus a surcharge for traffic loading. Active earth pressures acting on shoring can be estimated based on the following:

σ_a = 80H + 72 psf for soil being retained above the water table
σ_a = 40H + u + 72 psf for soil being retained below the water table
where:
"σ_a " is the uniform, active earth pressure acting on the shoring, in pounds per square foot (psf) with a level backslope
"H" is the height of the soil that is being retained in feet
"u" is the water pressure that increases at 62.4z, with z = depth in feet
"72 psf" is the temporary traffic surcharge

Excavated material and materials should generally be stockpiled or staged away from excavations, or the shoring systems should be designed for the additional surcharge from the materials. Shoring design should specifically address loads of equipment such as cranes, loaders, etc. if they are to be staged within a 1:1 line projected upward from the bottom interior edge of the shoring.

The contractor should be responsible for the design of shoring systems such that the construction will not result in settlement or instability of adjacent structures, private property, or existing improvements that will not be replaced as part of the project. In general, surcharge loads from existing structures can be neglected if the structure is behind a 1h:1v line projected upwards from the nearest bottom edge of a shored excavation. If excavations are made within the zone of influence of adjacent structures or foundations, the shoring design should account for the additional surcharge load.

The design of temporary slopes and shoring should also consider support of adjacent utilities and pipelines, and be constructed to prevent sloughing of trench backfill, pipe zone, and bedding materials from adjacent utilities into the new excavation. Particular attention should be directed to pressurized lines that may rely on the lateral support of the ground to constrain the pipe against movement.

7. LIMITATIONS

This study has been conducted in general accordance with currently accepted geotechnical practices in this area for use by the client for design purposes. The conclusions and recommendations submitted in this report are based upon the data obtained from field reconnaissance, drilling and sampling, and our understanding of the proposed project and type of construction described in this report. If there are any changes in the project or site conditions, Yeh should review those changes and provide additional recommendations, if needed. Any modifications to the recommendations of this report or approval of changes made to the project should not be considered valid unless they are made in writing. The report and drawings contained in this report are intended for design-input; and are not intended to act as construction drawings or specifications.

Site conditions will vary between points of observation or sampling, seasonally, and with time. The nature and extent of subsurface variations across the site may not become evident until excavation is performed. If during construction, fill, soil, or water conditions appear to be different from those described herein, Yeh should be advised and provided the opportunity to evaluate those conditions and provide additional recommendations, if necessary. The geotechnical professional should observe portions of the construction and site conditions, such as deep compaction, excavations, exposed subgrades and earthwork, to evaluate whether or not the conditions encountered are consistent with those assumed for design, and to provide additional recommendations during construction, if needed.

8. REFERENCES

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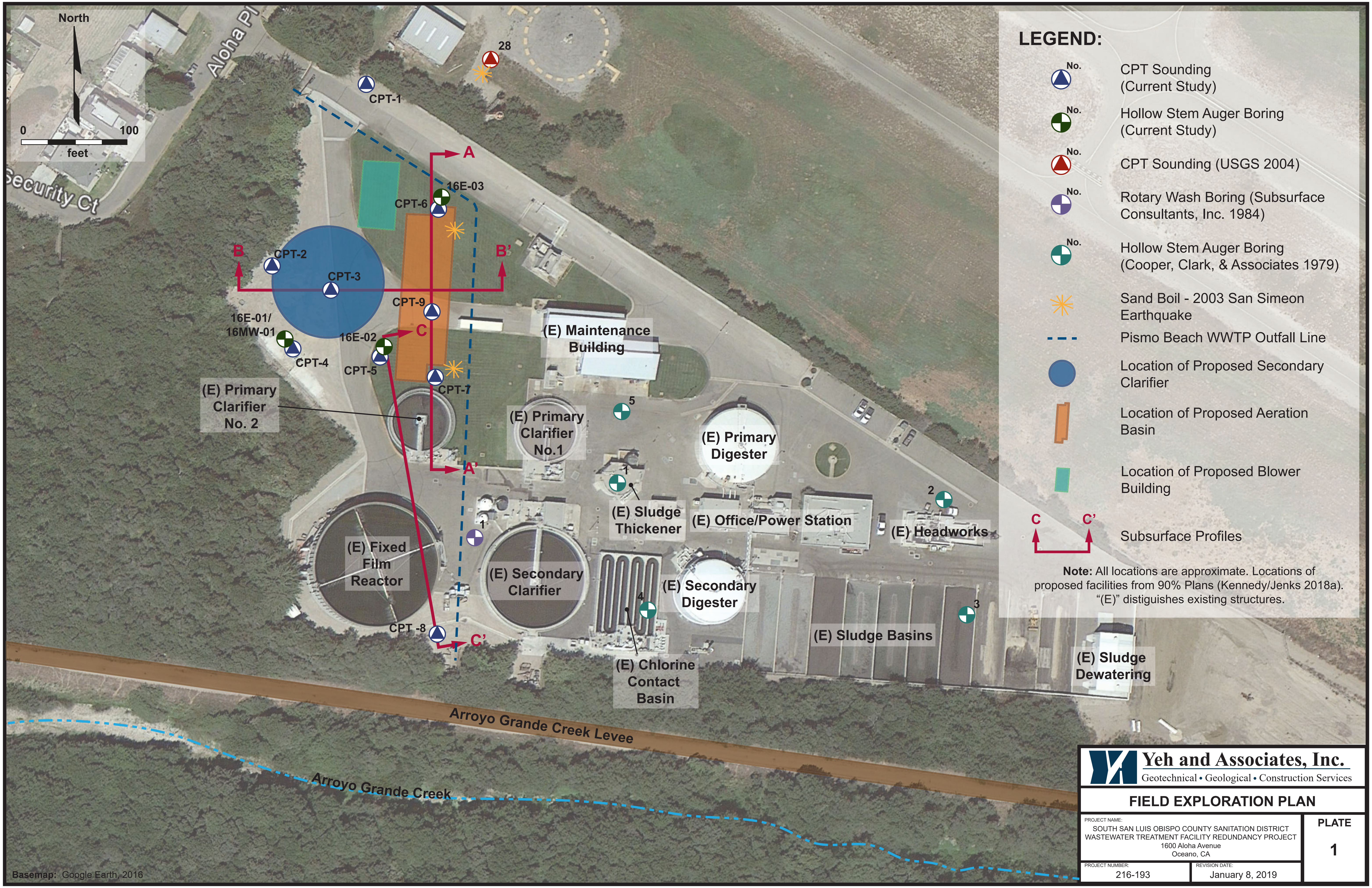

LEGEND:

No. ▲	CPT Sounding (Current Study)
No. ⊕	Hollow Stem Auger Boring (Current Study)
No. ▲	CPT Sounding (USGS 2004)
No. ⊕	Rotary Wash Boring (Subsurface Consultants, Inc. 1984)
No. ⊕	Hollow Stem Auger Boring (Cooper, Clark, & Associates 1979)
✳	Sand Boil - 2003 San Simeon Earthquake
- - -	Pismo Beach WWTP Outfall Line
●	Location of Proposed Secondary Clarifier
▮	Location of Proposed Aeration Basin
🟩	Location of Proposed Blower Building
C C'	Subsurface Profiles

Note: All locations are approximate. Locations of proposed facilities from 90% Plans (Kennedy/Jenks 2018a). "(E)" distiguishes existing structures.

Basemap: Google Earth, 2016

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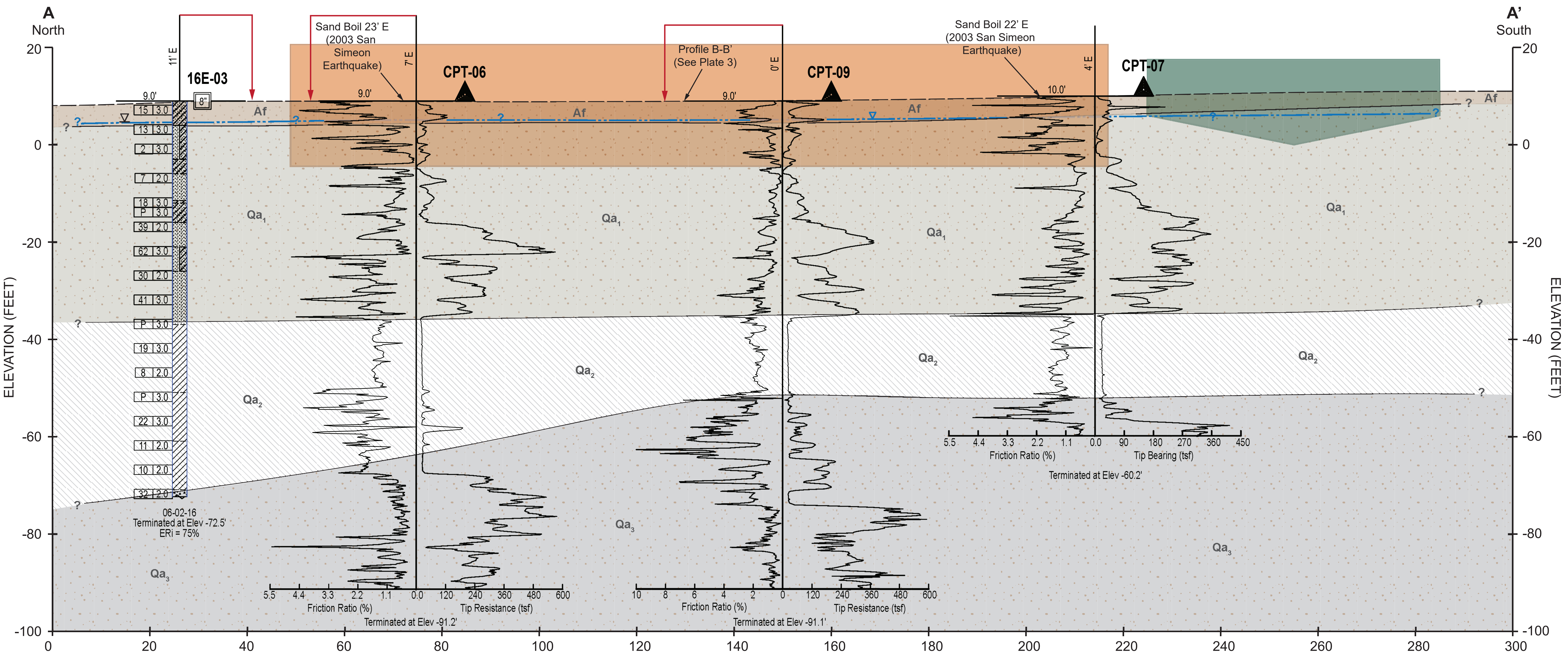
FIELD EXPLORATION PLAN

PROJECT NAME:
SOUTH SAN LUIS OBISPO COUNTY SANITATION DISTRICT
WASTEWATER TREATMENT FACILITY REDUNDANCY PROJECT
1600 Aloha Avenue
Oceano, CA

PROJECT NUMBER: 216-193
REVISION DATE: January 8, 2019

PLATE
1

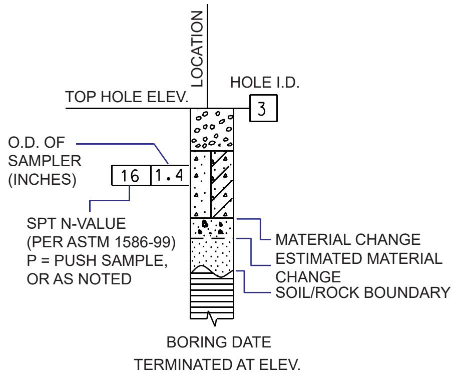


LEGEND:

Af	**Artificial Fill:** loose Clayey SAND (SC)
Qa_1	**Alluvium:** loose to medium dense SAND with varying amounts of SILT (SP, SM) with traces or lenses of gravel, lenses of soft silt and clay, and layers of shells and fibrous organic matter
Qa_2	**Alluvium:** medium stiff to stiff Lean to fat CLAY with varying amountsof SAND (CL, CH)
Qa_3	**Alluvium:** medium stiff and medium dense discontinuous lenses of CLAY, SAND, and GRAVEL with shells and shell fragments
—?—	Geologic contact, queried where uncertain

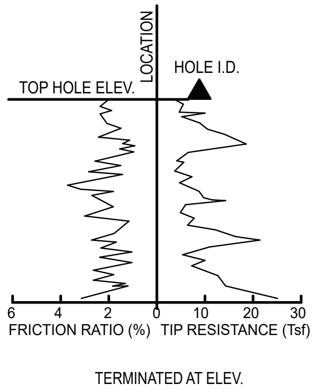
—?-·-	Interpreted groundwater surface during drilling, queried where uncertain
▽	Groundwater level encountered during drilling
🟧	Location of Proposed Aeration Basin (bottom of foundation el. -4.5 ft)
🟩	Location of Existing Primary Clarifier No. 2 (invert of foundation el. 0 ft)

See text and logs of exploration for description of subsurface conditions. All boundaries and locations are approximate.



ROTARY BORING

DRILL HOLE DIAGRAM



CONE PENETRATION TEST (CPT) DIAGRAM

Yeh and Associates, Inc.
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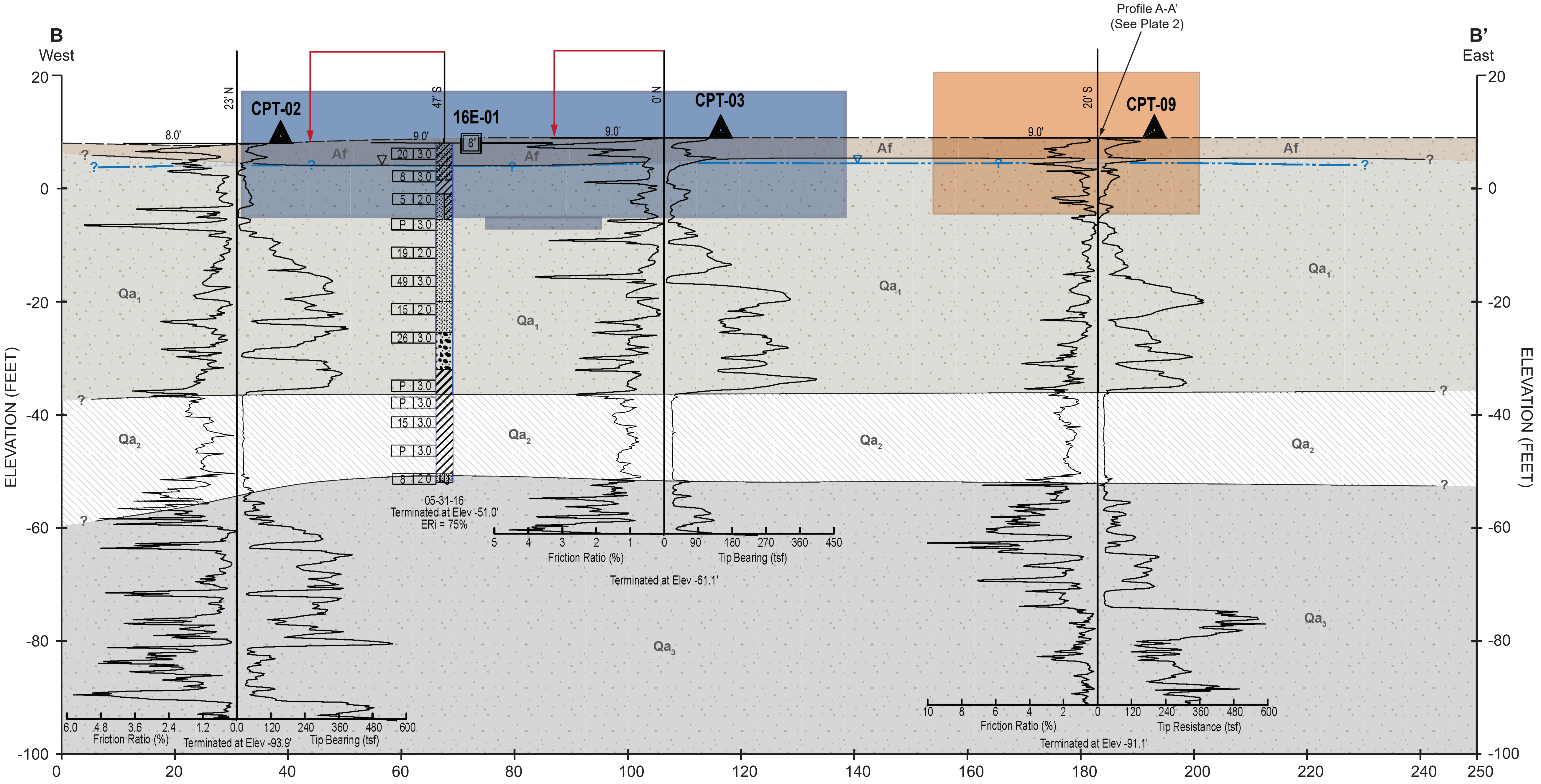
SUBSURFACE PROFILE A-A'

PROJECT NAME:
SOUTH SAN LUIS OBISPO COUNTY SANITATION DISTRICT
WASTEWATER TREATMENT FACILITY REDUNDANCY PROJECT
1600 Aloha Avenue
Oceano, CA

PLATE
2

PROJECT NUMBER:
216-193

REVISION DATE:
January 8, 2019

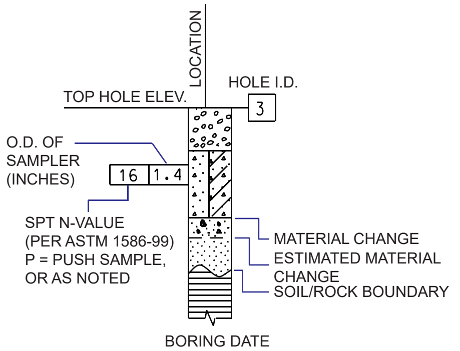


LEGEND:

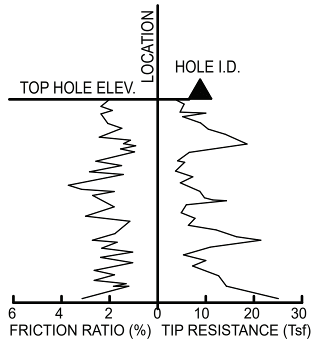
Af	**Artificial Fill:** medium dense Clayey SAND (SC)
Qa_1	**Alluvium:** loose to medium dense SAND with varying amounts of SILT (SP, SM) with traces or lenses of gravel, lenses of soft silt and clay, and layers of shells and fibrous organic matter
Qa_2	**Alluvium: stiff** Lean to fat CLAY with varying amounts of SAND (CL, CH)
Qa_3	**Alluvium:** loose discontinuous lenses of CLAY, SAND, and GRAVEL with shells and shell fragments
—?—	Geologic contact, queried where uncertain

—?·-·-	Interpreted groundwater surface during drilling, queried where uncertain
∇	Groundwater level encountered during drilling
🟧	Location of Proposed Aeration Basin (bottom of foundation el. -4.5 ft)
🟦	Location of Proposed Secondary Clarifier (bottom of foundation el. -5 ft (edge) and el. -7 ft (center))

See text and logs of exploration for description of subsurface conditions. All boundaries and locations are approximate.



BORING DATE
TERMINATED AT ELEV.
ROTARY BORING
DRILL HOLE DIAGRAM



TERMINATED AT ELEV.
CONE PENETRATION TEST (CPT) DIAGRAM

Yeh and Associates, Inc.
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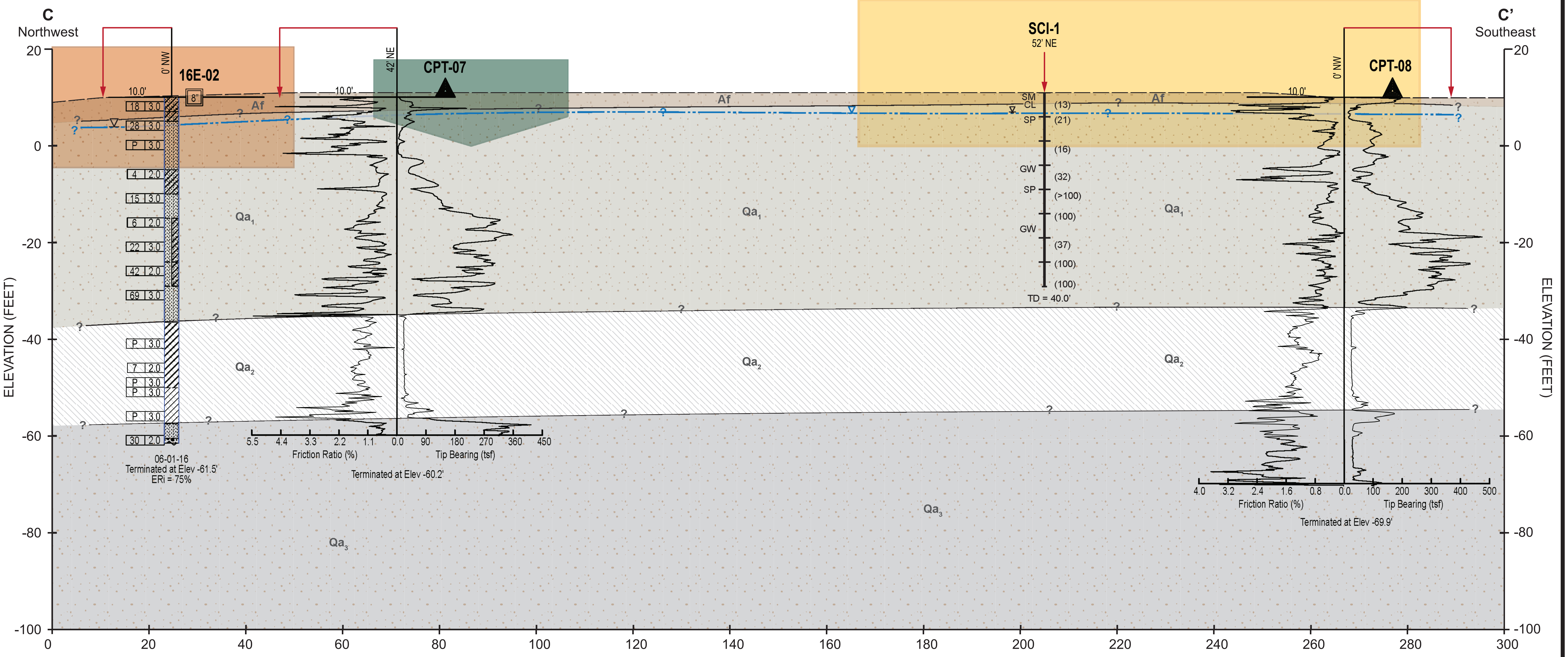
SUBSURFACE PROFILE B-B'

PROJECT NAME:
SOUTH SAN LUIS OBISPO COUNTY SANITATION DISTRICT
WASTEWATER TREATMENT FACILITY REDUNDANCY PROJECT
1600 Aloha Avenue
Oceano, CA

PROJECT NUMBER:
216-193

REVISION DATE:
January 8, 2019

PLATE
3

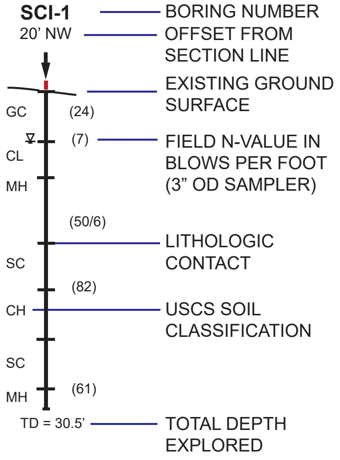


LEGEND:

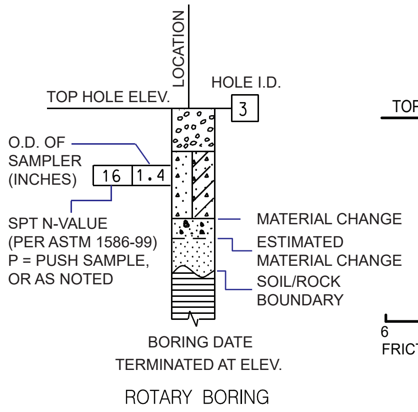
Af	**Artificial Fill:** medium dense Clayey SAND (SC)
Qa_1	**Alluvium:** loose to dense SAND with varying amounts of SILT (SP, SM) with traces or lenses of gravel, lenses of soft silt and clay, and layers of shells and fibrous organic matter
Qa_2	**Alluvium:** medium stiff Lean to fat CLAY with varying amounts of SAND (CL, CH)
Qa_3	**Alluvium:** medium dense discontinuous lenses of CLAY, SAND,and GRAVEL with shells and shell fragments
—?—	Geologic contact, queried where uncertain
—?·–	Interpreted groundwater surface during drilling, queried where uncertain

∇	Groundwater level encountered during drilling
🟧	Location of Proposed Aeration Basin (0-33' NE of C-C' line,bottom of foundation el. -4.5 ft)
🟩	Location of Existing Primary Clarifier No. 2 (invert of foundation el. 0 ft)
🟨	Location of Existing Fixed Film Reactor (centerline 47' SE of C-C' line, invert of foundation el. 0 ft)

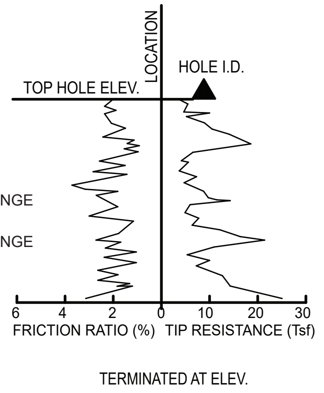
See text and logs of exploration for description of subsurface conditions. All boundaries and locations are approximate.



DRILL HOLE DIAGRAM (BY OTHERS)



DRILL HOLE DIAGRAM



CONE PENETRATION TEST (CPT) DIAGRAM

Notes:

1) 16E-02,CPT-07, CPT-08 by Yeh and Associates, May 2016 and June 2016

2) SCI-1 by Subsurface Consultants, Inc., June 1984

Yeh and Associates, Inc.
Geotechnical • Geological • Construction Services

SUBSURFACE PROFILE C-C'

PROJECT NAME:
SOUTH SAN LUIS OBISPO COUNTY SANITATION DISTRICT
WASTEWATER TREATMENT FACILITY REDUNDANCY PROJECT
1600 Aloha Avenue
Oceano, CA

PROJECT NUMBER: 216-193

REVISION DATE: January 8, 2019

PLATE
4

APPENDIX A - BORING LOGS AND CONE PENETROMETER SOUNDINGS

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)
GM	SILTY GRAVEL SILTY GRAVEL with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL
SP	Poorly graded SAND Poorly graded SAND with GRAVEL
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)
SM	SILTY SAND SILTY SAND with GRAVEL
SC	CLAYEY SAND CLAYEY SAND with GRAVEL
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL
PT	PEAT
	COBBLES COBBLES and BOULDERS BOULDERS

Graphic / Symbol	Group Names
CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435-04)
CL	Collapse Potential (ASTM D 5333-03)
CP	Compaction Curve (ASTM D1557)
CR	Corrosion, Sulfates, Chlorides (CTM 643 - 99; CTM 417 - 06; CTM 422 - 06)
CU	Consolidated Undrained Triaxial (ASTM D 4767-02)
DS	Direct Shear (ASTM D 3080-04)
EI	Expansion Index (ASTM D 4829-03)
M	Moisture Content (ASTM D 2216-05)
OC	Organic Content (ASTM D 2974-07)
P	Permeability (CTM 220 - 05)
PA	Particle Size Analysis (ASTM D 422-63 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (AASHTO T 89-02, AASHTO T 90-00)
PL	Point Load Index (ASTM D 5731-05)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301 - 00)
SE	Sand Equivalent (CTM 217 - 99)
SG	Specific Gravity (AASHTO T 100-06)
SL	Shrinkage Limit (ASTM D 427-04)
SW	Swell Potential (ASTM D 4546-03)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166-06) Unconfined Compression - Rock (ASTM D 2938-95)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850-03)
UW	Unit Weight (ASTM D 4767-04)
VS	Vane Shear (AASHTO T 223-96 [2004])
-200	200 Wash (ASTM D1140-14)

SAMPLER GRAPHIC SYMBOLS

- ⊠ Standard Penetration Test (SPT) (2" O.D.)
- ⧗ Standard California Sampler (2.5" O.D.)
- ⧗ Modified California Sampler (3" O.D.)
- ▯ Shelby Tube
- ▯ Piston Sampler
- ▯ Rock Core
- ⁞ Grab Sample
- ▩ Bulk Sample
- ◩ Other (see remarks)

DRILLING METHOD SYMBOLS

- ▯ Auger Drilling
- ▯ Rotary Drilling
- ⊠ Dynamic Cone or Hand Driven
- ⊠ Diamond Core

WATER LEVEL SYMBOLS

- ▽ First Water Level Reading (during drilling)
- ▼ Static Water Level Reading (short-term)
- ▼ Static Water Level Reading (long-term)

Yeh and Associates, Inc.
Consulting Engineers & Scientists

REPORT TITLE
BORING RECORD LEGEND

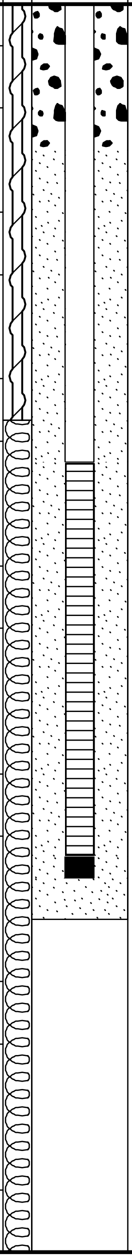
PROJECT NAME
SSLOCSD - Redundancy Project

DATE
9/25/2018

SHEET
1 of 1

LOGGED BY **J. King**	BEGIN DATE **5/31/16**	COMPLETION DATE **5/31/16**	HAMMER TYPE **140-lb Automatic Trip**	BORING NUMBER **16E-01**
FINAL BY **J. Cravens**			BOREHOLE LOCATION (Lat/Long or North/East and Datum) **--**	SURFACE ELEVATION **9.0 ft**
DRILLING METHOD **8" Hollow-Stem Auger/Mud Rotary**			BOREHOLE LOCATION (Offset, Station, Line) **--**	WEATHER NOTES **Cloudy**
DRILLER **S/G Drilling Company**			LOCATION DESCRIPTION **Gravel area on S edge of proposed secondary clarifier, 20' NW of SD pump station**	BACKFILLED WITH **Monitoring Well**
DRILL RIG **CME 75**			GROUNDWATER READINGS — DURING DRILLING **5.5 ft** — AFTER DRILLING (DATE) **3.9 ft on 8-7-18**	TOTAL DEPTH OF BORING **60.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Well Diagram	Well Description	Remarks
	0		CLAYEY SAND with GRAVEL (SC); medium dense; brown; moist; (ARTIFICIAL FILL).													
	1				1	6 6 14	20	89		15	94	PP =3.0				CR (pH = 8.97, r = 2,008 ohm-cm)
7	2														Bentonite Chips	
	3															
5	4		CLAYEY SAND (SC); loose; gray; moist; trace organic fibers; (ALLUVIUM).													▼
	5				2	1 5 3	8	89		34	88	PP =4.25				▽
3	6															
	7		Lean CLAY (CL); soft; gray; moist to wet; with shell fragments and fine GRAVEL.		A											CR (pH = 8.39, r = 817 ohm-cm, Chlorides = 117 mg/kg, Sulfates = 122 mg/kg)
1	8															
	9		Poorly graded SAND with CLAY (SP-SC); loose; gray; wet; fine SAND.		3	1 3 2	5	78		28						-200 (0% G, 92% S, 8% F)
-1	10															
	11															
-3	12															
	13															
-5	14		Poorly graded SAND (SP); grayish brown; wet; trace fine GRAVEL and shell layers to 3-inches thick.		4			89		27		PP =0.5				-200 (0% G, 98% S, 2% F)
	15														Cemex, #2/12 Sand	
-7	16															
	17															
-9	18														2-7/16" O.D. PVC Pipe 2" I.D., Slotted 0.01"	
	19		Medium dense; gray; trace CLAY.		5	6 10 9	19	56								
-11	20															
	21														2" PVC Cap	
-13	22															
	23															
-15	24				6	11 22 27	49	78		25	100					CU
	25															
-17	26															
	27															
-19	28		Poorly graded SAND with GRAVEL (SP); medium dense; gray; wet; abundant shell fragments, 0.25" to 1" in dimension.													
	29				7	6 9 6	15									
	30															



(continued)

Yeh and Associates, Inc.
Consulting Engineers & Scientists

PROJECT NAME **SSLOCSD - Redundancy Project**	
PROJECT NUMBER **216-193**	
BORING NUMBER **16E-01**	
REVISION DATE **9/25/2018**	SHEET **1 of 2**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Well Diagram	Well Description	Remarks
	30		Poorly graded SAND with GRAVEL (SP) *(continued)*.													
	31															
-23	32															
	33															
-25	34		Well-graded GRAVEL (GW); medium dense; gray; wet; sub-rounded.		8	5 10 16	26									Possible slough due to plugged hole at 33.0'
	35															Switched to 8-inch HSA
-27	36															
	37															
-29	38															
	39															
-31	40															
	41															
-33	42		Fat CLAY (CH); stiff; gray; very moist; mottled with dark gray and dark brown.		9			96		43	77	TV =0.75 tsf				C
	43															
-35	44															
	45		Trace shell fragments.		10			92		47	75	TV =0.75 tsf PP =2.0				-200 (0% G, 1% S, 99% F) PI (76 LL, 32 PL, 44 PI)
-37	46															
	47															
-39	48															
	49				11	4 7 8	15	100				TV =0.75 tsf				
-41	50															
	51															
-43	52															
	53															
-45	54				12			92				TV =0.8 tsf				
	55															
-47	56															
	57															
-49	58															
	59		SILTY SAND (SM); loose; dark grayish brown; very moist; trace fine, rounded GRAVEL and organic root clasts/charcoal fragments.		13	2 3 5	8	78								
-51	60		Bottom of borehole at 60.0 ft bgs													
	61															
-53	62															
	63		This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.													
-55	64															
	65															
	66															

BR - STANDARD WITH MONITORING WELL 216-193 SSLOCSD BORING LOGS.GPJ CALIFORNIA YEH LIBRARY (FEB 2013).GLB 9/25/18

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Consulting Engineers & Scientists

PROJECT NAME **SSLOCSD - Redundancy Project**	
PROJECT NUMBER **216-193**	
BORING NUMBER **16E-01**	
REVISION DATE **9/25/2018**	SHEET **2 of 2**

LOGGED BY **S. Boone**	BEGIN DATE **6-1-16**	COMPLETION DATE **6-1-16**	HAMMER TYPE **140-lb Automatic Trip**
BORING NUMBER **16E-02**			
FINAL BY **J. Cravens**			BOREHOLE LOCATION (Lat/Long or North/East and Datum) **°/°**
SURFACE ELEVATION **10.0 ft**			
DRILLING METHOD **8" Hollow-Stem Auger**			BOREHOLE LOCATION (Offset, Station, Line) **--**
WEATHER NOTES **Overcast**			
DRILLER **S/G Drilling Company**			LOCATION DESCRIPTION **Turf area between proposed aeration basin and clarifier, 47' NW of primary clarifier no. 2**
BACKFILLED WITH **Bentonite Grout**			
DRILL RIG **CME 75**			GROUNDWATER READINGS: DURING DRILLING **6.1 ft**; AFTER DRILLING (DATE)
TOTAL DEPTH OF BORING **71.5 ft**			

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	0		CLAYEY SAND (SC); medium dense; brown; moist; little angular GRAVEL to 1"; (ARTIFICIAL FILL).												
	1				B 14	4 7 11	18			15	116				-200 (11% G, 57% S, 32% F) PI (31 LL, 22 PL, 9 PI) CP (UW = 117 pcf, W = 12.5%) DS
8	2														
	3		CLAYEY SAND (SC); dark brown; moist; (ALLUVIUM).												
6	4														
	5		Poorly graded SAND (SP); medium dense; gray; wet; some angular GRAVEL to 1"; 1" to 2" layers of lean CLAY.		15	7 13 15	28			21	106				CR (pH = 8.50, r = 1,143 ohm-cm)
4	6														▽
	7														
2	8														
	9				16					37	82				
0	10														CU
	11														
-2	12														
	13														
-4	14														
	15		CLAYEY SAND (SC); loose; gray; wet; trace shell fragments to 0.25".		17	0 2 2	4	56		28					PA (3% G, 85% S, 12% F)
-6	16														
	17														
-8	18														
	19														
-10	20		Poorly graded SAND (SP); loose; gray; wet.		18	4 8 7	15	56		22	104				
	21														
-12	22														
	23														
-14	24														
	25		Poorly graded SAND with CLAY (SP-SC); loose; grayish brown; wet; with shell fragments to 0.75".		19	2 2 4	6			30					-200 (1% G, 90% S, 9% F)
-16	26														
	27														
-18	28														
	29														
	30														

(continued)

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PROJECT NAME
SSLOCSD - Redundancy Project
PROJECT NUMBER
216-193
BORING NUMBER
16E-02
REVISION DATE
9/25/2018
SHEET
1 of 3

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	30		Medium dense. Poorly graded SAND with CLAY (SP-SC) *(continued)*.		20	3 10 12	22	78		24	100				
	31														
-22	32														
	33														
-24	34		Poorly graded SAND with CLAY and GRAVEL (SP-SC); medium dense; tan; wet; tan, orange, and green angular to subangular GRAVEL to 1".												
	35				21	4 12 30	42	44		24					PA (34% G, 56% S, 10% F)
-26	36														
	37														
-28	38														
	39		Poorly graded SAND (SP); dense; brown to gray; wet; trace CLAY.												
-30	40				22	12 27 42	69	100		27	97				
	41														
-32	42														
	43														
-34	44														
	45		Fat CLAY (CH); medium stiff; gray; wet.		23 A/B										No Recovery of Shelby tube @ 45'; SPT driven
-36	46														
	47														
-38	48														
	49														
-40	50				24			42							
	51														
-42	52														
	53														
-44	54														
	55		Trace shell fragments to 0.25".		25	2 3 4	7			43					-200 (0% G, 8% S, 92% F) PI (67 LL, 29 PL, 38 PI)
-46	56														
	57														
-48	58				26			83				0.35 tsfTV			
	59														
-50	60		Lean CLAY (CL); medium stiff; gray; wet; trace fine SAND.		27			83		37	83	0.25 tsfTV			C
	61														
-52	62														
	63														
-54	64														
	65				28			100				0.175 tsfTV			
	66														

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PROJECT NAME
SSLOCSD - Redundancy Project
PROJECT NUMBER
216-193
BORING NUMBER
16E-02
REVISION DATE
9/25/2018
SHEET
2 of 3

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	66		Lean CLAY (CL) *(continued)*.		28			100				0.175 tsfTV			
	67														
-58	68		Poorly graded SAND (SP); medium dense; gray; wet; fine to medium SAND; with trace shell fragments to 0.25".												
	69														
-60	70				29 A/B	4 9 21	30	100							
	71		Well-graded GRAVEL with SAND (GW); medium dense; wet; subangular to subrounded GRAVEL to 1".												
-62	72		Bottom of borehole at 71.5 ft bgs												
	73														
-64	74		This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.												
	75														
-66	76														
	77														
-68	78														
	79														
-70	80														
	81														
-72	82														
	83														
-74	84														
	85														
-76	86														
	87														
-78	88														
	89														
-80	90														
	91														
-82	92														
	93														
-84	94														
	95														
-86	96														
	97														
-88	98														
	99														
-90	100														
	101														
	102														

5 BR - STANDARD 216-193 SSLOCSD BORING LOGS.GPJ CALIFORNIA YEH LIBRARY (FEB 2013).GLB 9/25/18

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PROJECT NAME
SSLOCSD - Redundancy Project
PROJECT NUMBER
216-193
BORING NUMBER
16E-02
REVISION DATE
9/25/2018
SHEET
3 of 3

LOGGED BY **S. Boone**
BEGIN DATE **6-2-16**
COMPLETION DATE **6-2-16**
HAMMER TYPE **140-lb Automatic Trip**
BORING NUMBER **16E-03**
FINAL BY **J. Cravens**
BOREHOLE LOCATION (Lat/Long or North/East and Datum) **°/°**
SURFACE ELEVATION **9.0 ft**
DRILLING METHOD **8" Hollow-Stem Auger**
BOREHOLE LOCATION (Offset, Station, Line) **--**
WEATHER NOTES **Overcast**
DRILLER **S/G Drilling Company**
LOCATION DESCRIPTION **Turf area at N end of proposed aeration basin, 225' N of primary clarifier no. 2**
BACKFILLED WITH **Bentonite Grout**
DRILL RIG **CME 75**
GROUNDWATER READINGS DURING DRILLING **4.0 ft** AFTER DRILLING (DATE)
TOTAL DEPTH OF BORING **81.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	0		CLAYEY SAND (SC); loose; gray; moist; trace angular GRAVEL to 1"; (ARTIFICIAL FILL).												
	1				30 C	3 6 9	15	89		25	97				PA (4% G, 65% S, 31% F) PI (28 LL, 20 PL, 8 PI) R (51) -200 (4% G, 68 % S, 28% F)
7	2														
	3														
5	4														▽
	5		Poorly graded SAND with CLAY (SP-SC); loose; gray; wet; interbedded CLAY beds to 2" thick; with organics (roots, decomposing vegetation); (ALLUVIUM).		31	2 7 6	13	100		29	91				DS
3	6														
	7														
1	8														
	9				32	0 1 1	2	89		36	84				
-1	10														CU
	11														
-3	12		CLAYEY SAND with GRAVEL (SC); very loose; gray; wet.		33			33		25					-200 (16% G, 64% S, 20% F) CR (pH = 8.09, r = 1,507 ohm-cm)
	13														
-5	14														
	15		Poorly graded SAND with GRAVEL (SP); loose; tan and green with red and black particles; wet; coarse SAND, fine GRAVEL to 1".		34	1 3 4	7	33		24					-200 (40% G, 56% S, 4% F)
-7	16														
	17														
-9	18														
	19														
-11	20				35	4 7 11	18	89							
	21		Lean CLAY (CL); medium stiff; gray; wet. CLAYEY SAND (SC); medium dense; dark gray; wet; with shell fragments.												
-13	22				36					37	86				-200 (6% G, 81% S, 13% F)
	23														
-15	24														
	25		Poorly graded SAND (SP); medium dense; gray; wet.		37	7 16 23	39	78							
-17	26														
	27														
-19	28														
	29														
	30														

(continued)

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PROJECT NAME **SSLOCSD - Redundancy Project**
PROJECT NUMBER **216-193**
BORING NUMBER **16E-03**
REVISION DATE **9/25/2018**
SHEET **1 of 3**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	30		Poorly graded SAND with CLAY and GRAVEL (SP-SC); dense; gray; wet; angular to subangular GRAVEL to 2".		38	8 23 39	62	89		24	107				-200 (17% G, 72% S, 10% F)
	31														
-23	32														
	33														
-25	34														
	35		Poorly graded SAND (SP); medium dense; grayish brown; wet.		39	7 11 19	30	100							
-27	36														
	37														
-29	38														
	39														
-31	40		Fine to very fine SAND.		40	9 17 24	41	100		25	100				
	41														
-33	42														
	43														
-35	44														
	45				41					30	90	0.35 tsfTV			
-37	46		Lean CLAY (CL); medium stiff; gray; wet.												
	47														
-39	48														
	49														
-41	50		Very stiff; trace shell fragments to 0.25".		42	4 9 10	19	89							
	51														
-43	52														
	53														
-45	54														
	55				43	2 4 4	8	100							
-47	56														
	57														
-49	58														
	59														
-51	60		Lean CLAY with SAND (CL); stiff; dark gray; wet.		44					33	87	0.5 tsfTV			PI (42 LL, 26 PL, 16 PI)
	61														
-53	62														
	63														
-55	64														
	65				45		22	100				0.6 tsfTV			
	66														

(continued)

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PROJECT NAME
SSLOCSD - Redundancy Project

PROJECT NUMBER
216-193

BORING NUMBER
16E-03

REVISION DATE
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ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	66	▨	Lean CLAY with SAND (CL) *(continued)*.	⧓		6 9 13									
	67	▨													
-59	68	▨													
	69	▨													
-61	70	▨	Lean CLAY (CL); medium stiff; dark gray; wet; trace fine SAND.	⧖	46	2 5 6	11	100							
	71	▨													
-63	72	▨													
	73	▨													
-65	74	▨													
	75	▨		⧖	47	2 4 6	10	100							
-67	76	▨													
	77	▨													
-69	78	▨													
	79	▨													
-71	80	∴	Well-graded SAND (SW); medium dense; gray with green and brown particles; wet; trace shell fragments to 0.125" and trace GRAVEL to 0.5".	⧖	48	4 12 20	32	78							
	81	∴													
-73	82		Bottom of borehole at 81.5 ft bgs												
	83														
-75	84		This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.												
	85														
-77	86														
	87														
-79	88														
	89														
-81	90														
	91														
-83	92														
	93														
-85	94														
	95														
-87	96														
	97														
-89	98														
	99														
-91	100														
	101														
	102														

PROJECT NAME
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PROJECT NUMBER
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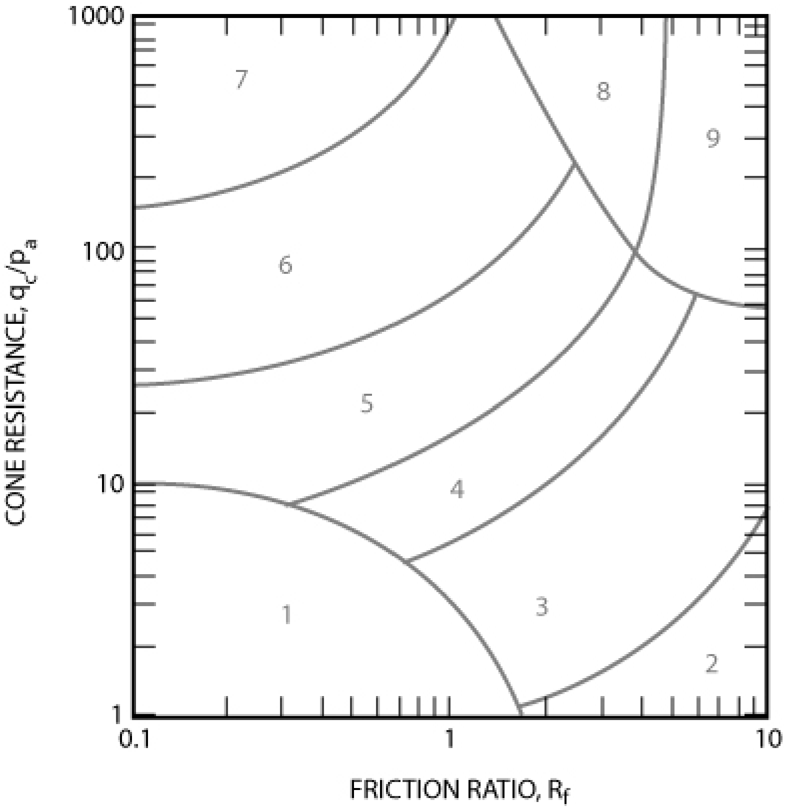

Zone	Soil Behavior Type	USCS
1	Sensitive, fine grained	OL-CH
2	Organic soils - clay	OL-OH, CH
3	Clay – silty clay to clay	CL-CH
4	Silt mixtures – clayey silt to silty clay	MH-CL
5	Sand mixtures – silty sand to sandy silt	SM-ML
6	Sands – clean sand to silty sand	SW-SP
7	Gravelly sand to dense sand	SW-GW
8	Very stiff sand to clayey sand*	SC-SM
9	Very stiff fine grained*	CH-CL

* Heavily overconsolidated or cemented

P_a = atmospheric pressure = 100 kPa = 1 tsf

Non-normalized CPT Soil Behavior Type (SBT) chart
(Robertson et al., 1986, updated by Robertson, 2010).

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REPORT TITLE
CPT SOIL BEHAVIOR CHART (SBT) LEGEND

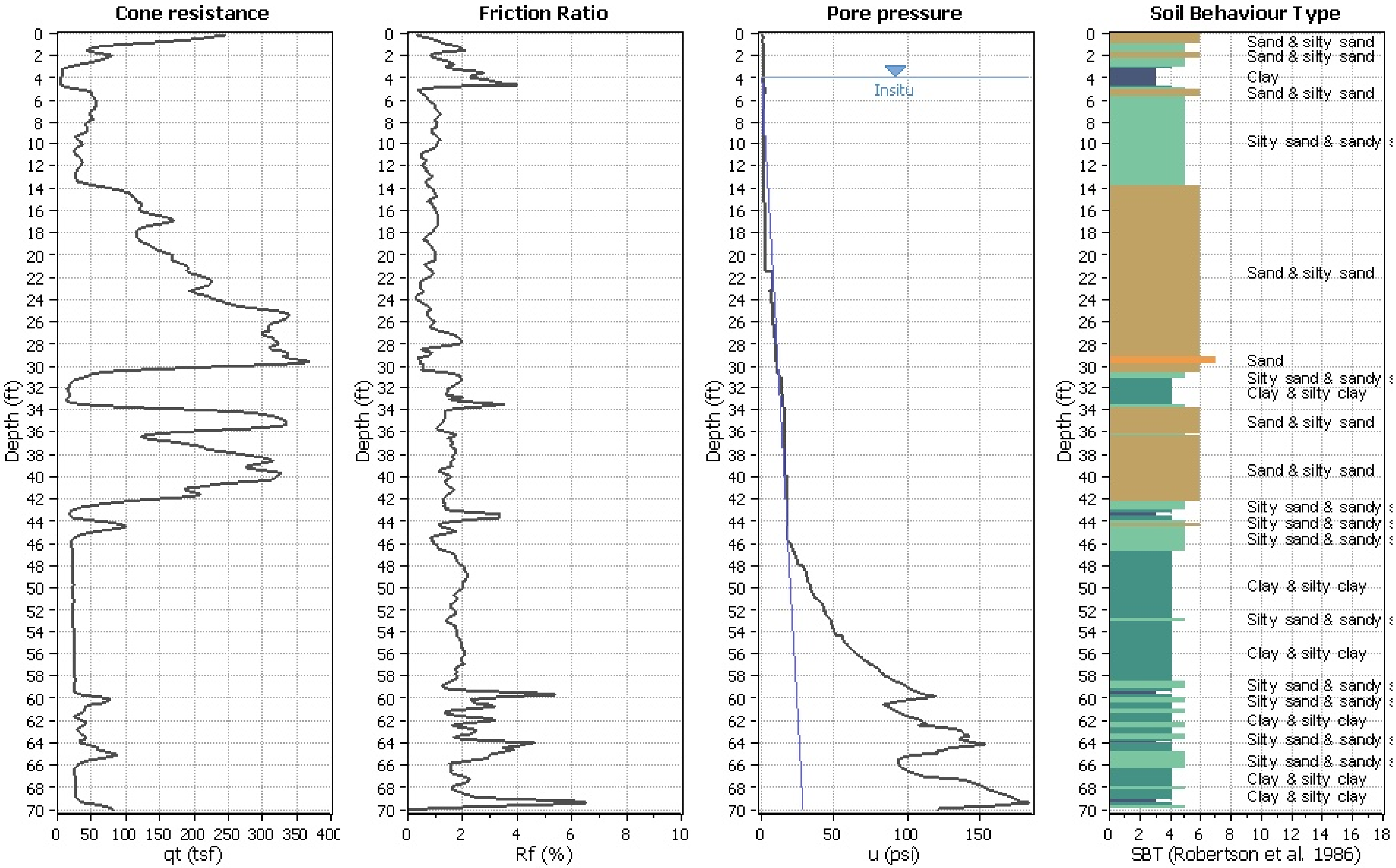
PROJECT NAME
SSLOCSD - Redundancy Project

DATE
9/25/2018

SHEET
1 of 1

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

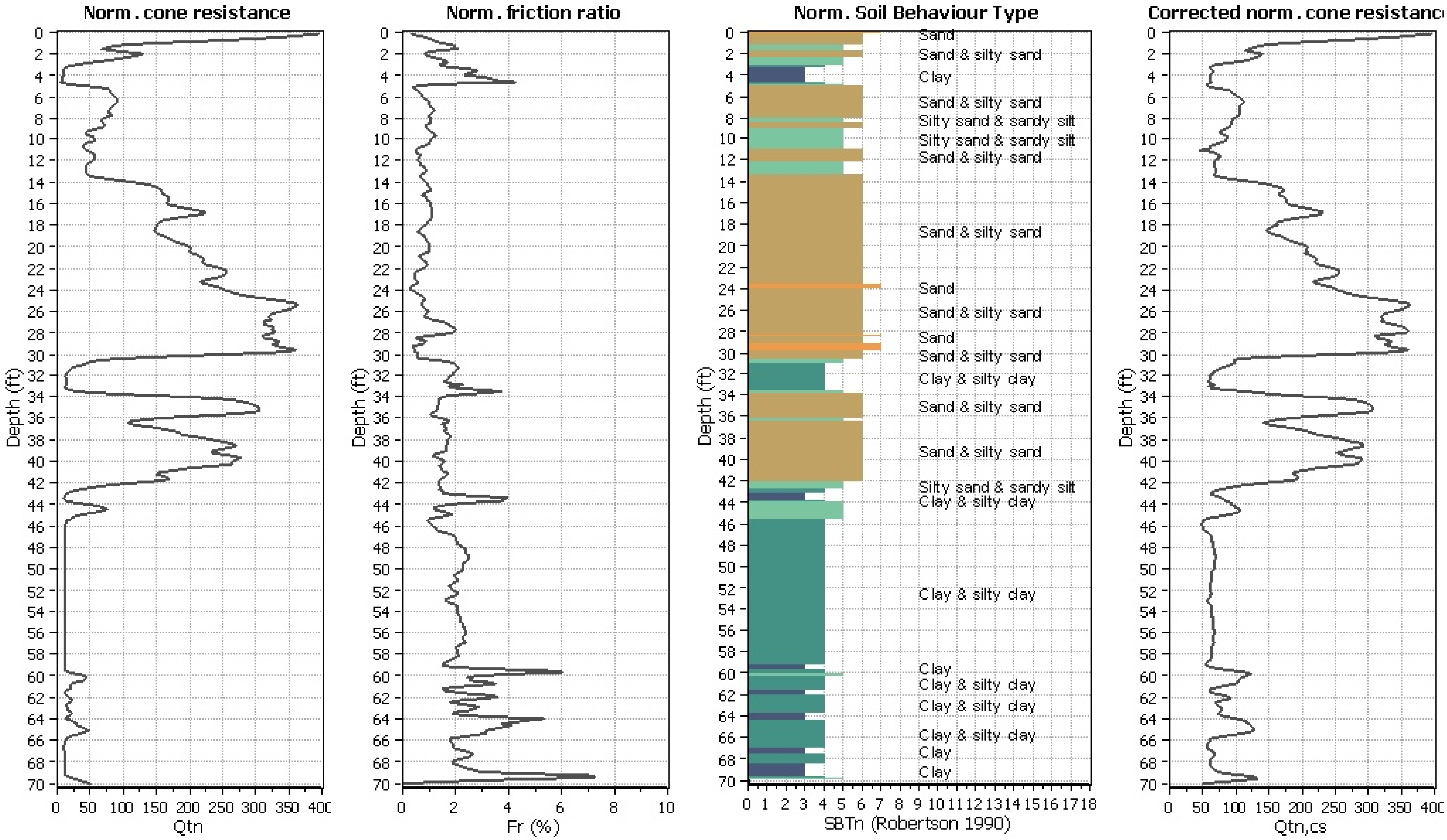
CPT: CPT-01
Total depth: 70.05 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-01
Total depth: 70.05 ft

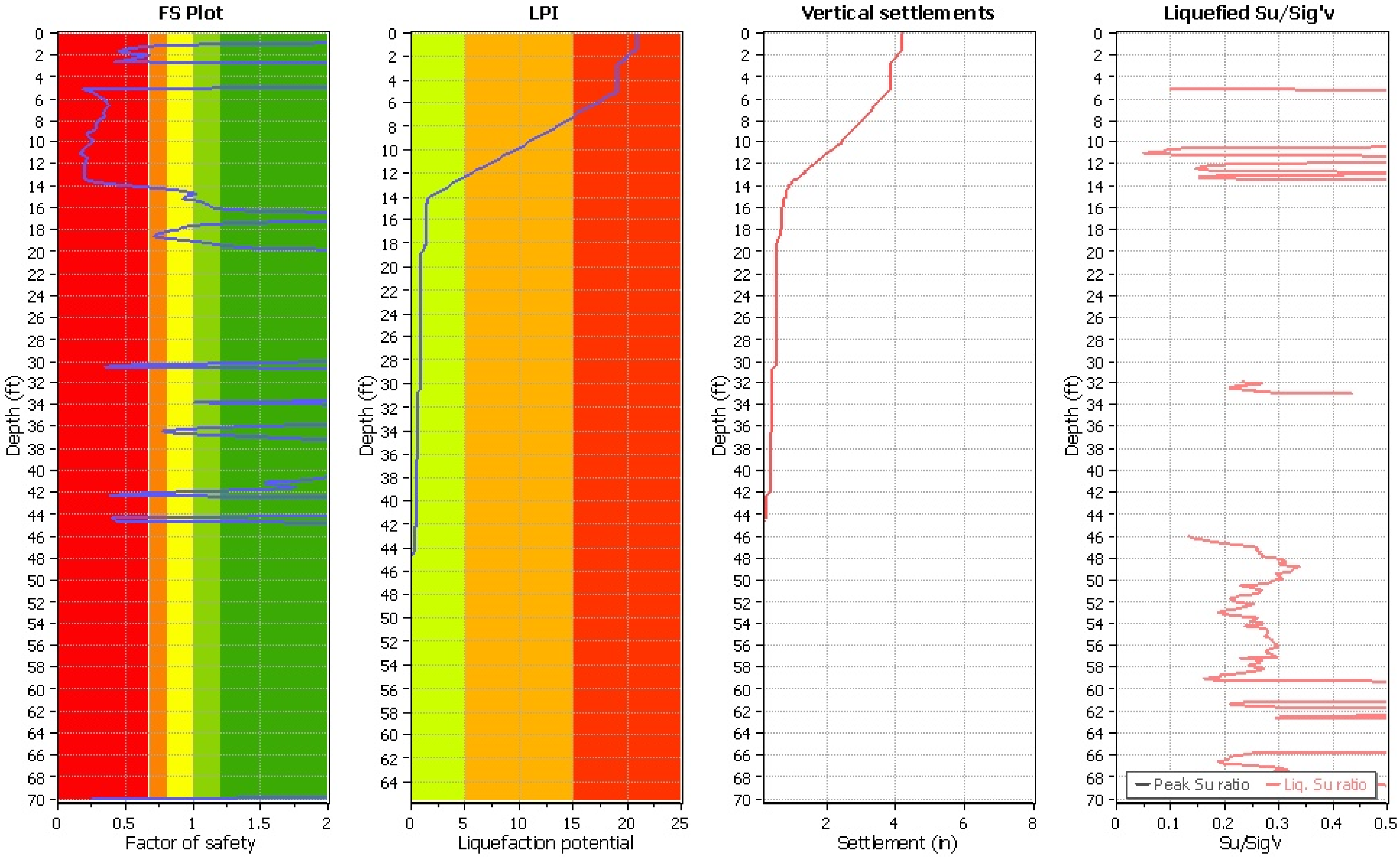


Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Yeh and Associates, Inc.
391 Front Street, Suite D
Grover Beach, CA 93433

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-01
Total depth: 70.05 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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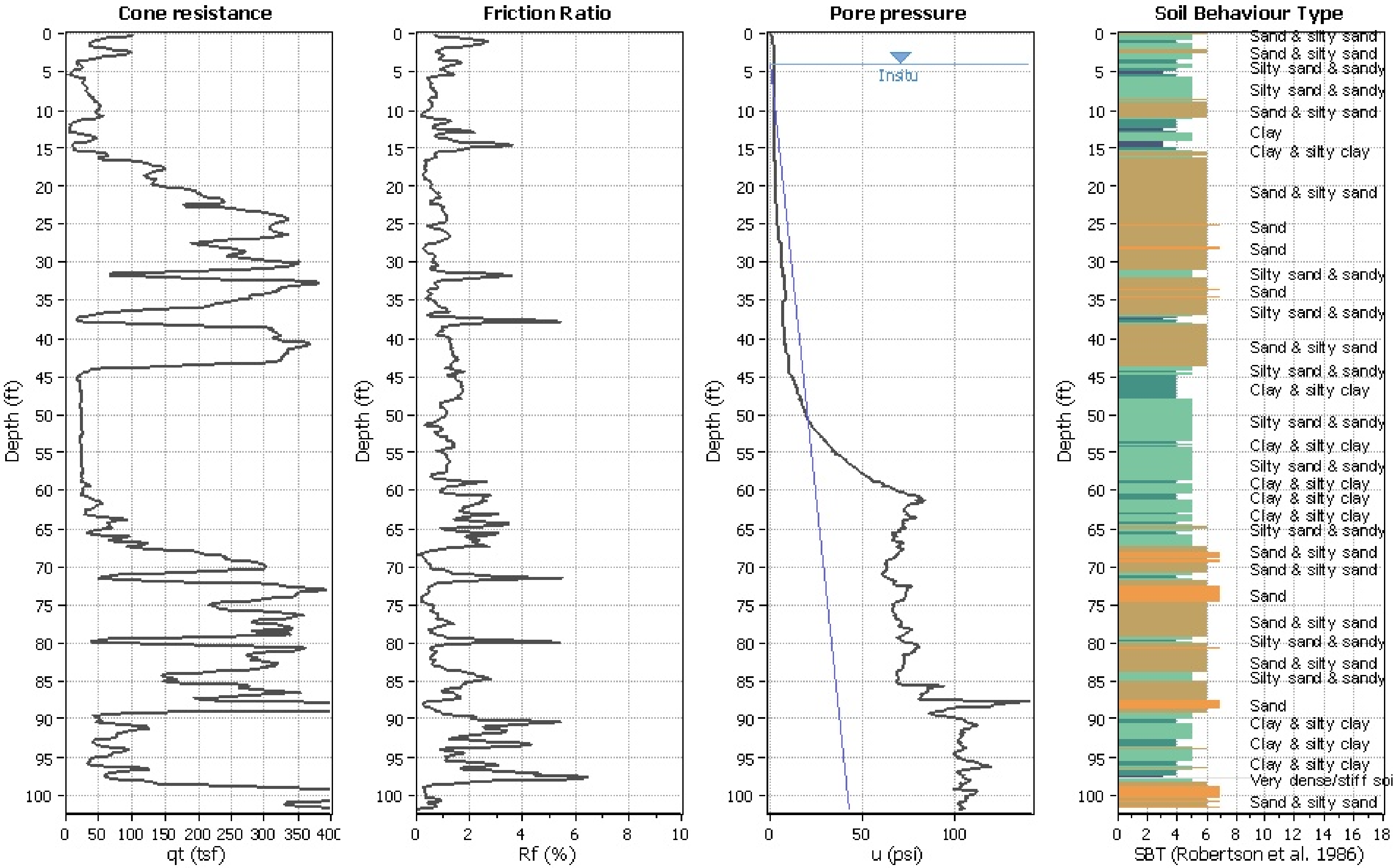
Grover Beach, CA 93433

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

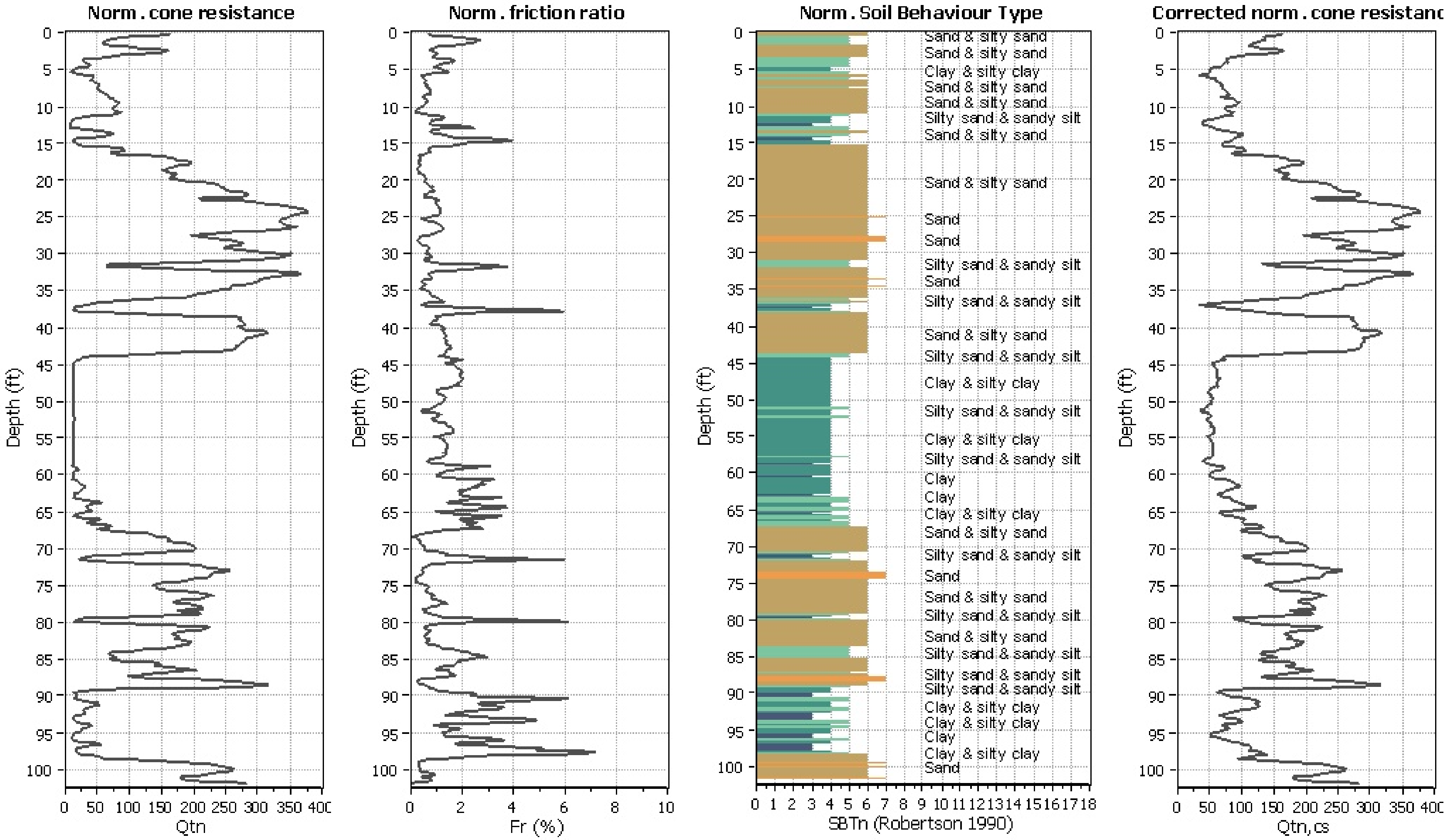
CPT: CPT-02
Total depth: 101.87 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

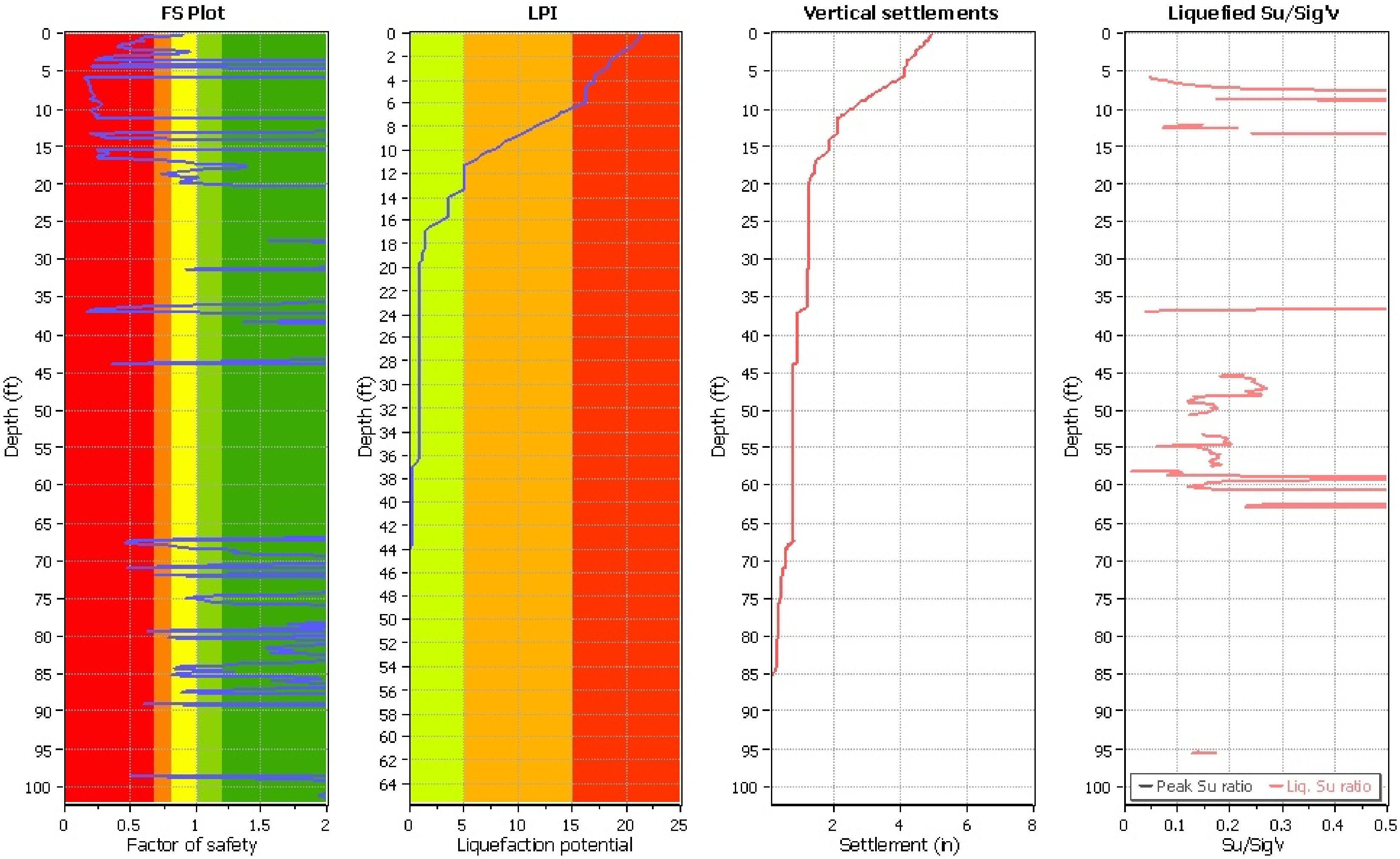
CPT: CPT-02
Total depth: 101.87 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-02
Total depth: 101.87 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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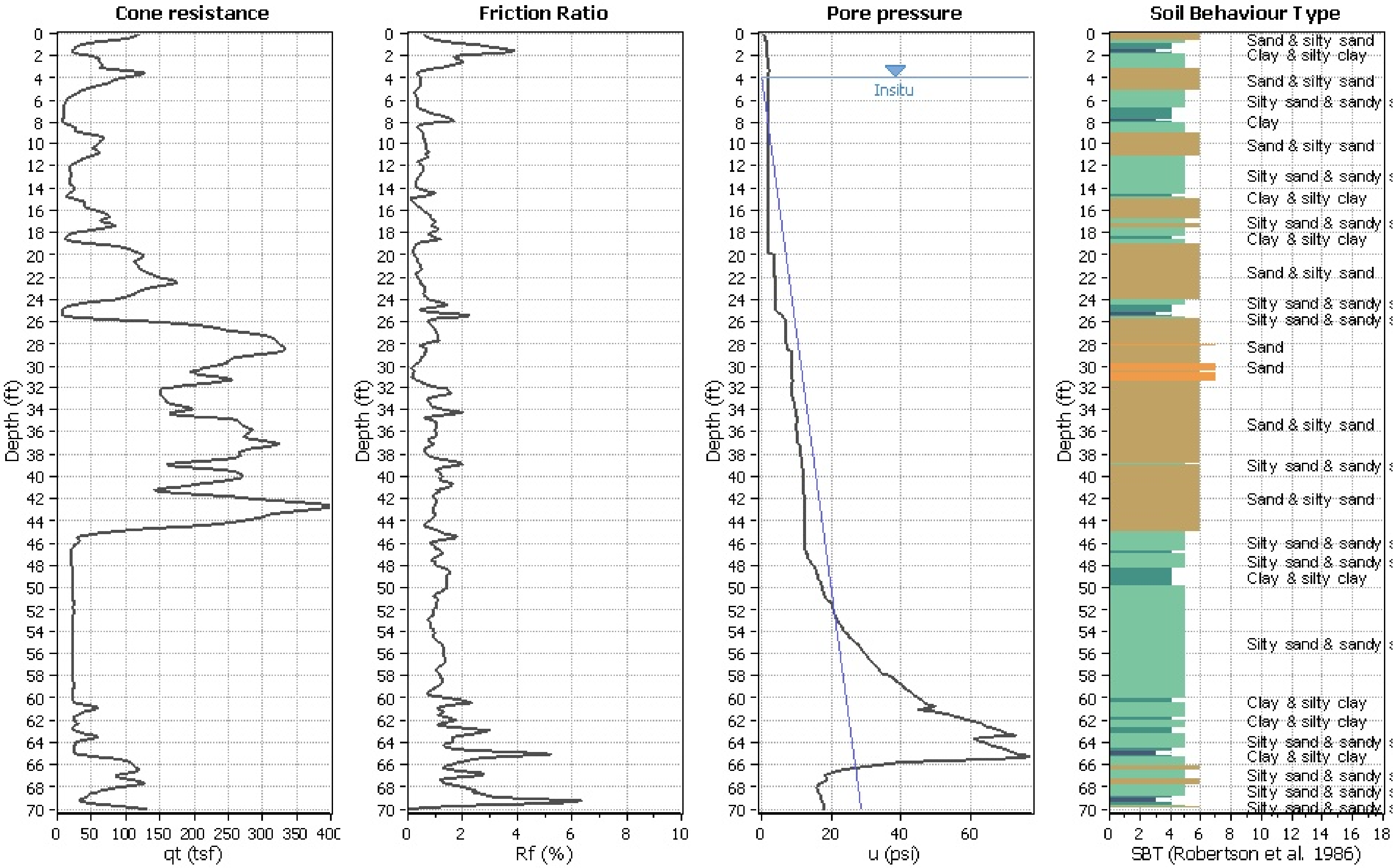
391 Front Street, Suite D
Grover Beach, CA 93433

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-03
Total depth: 70.05 ft



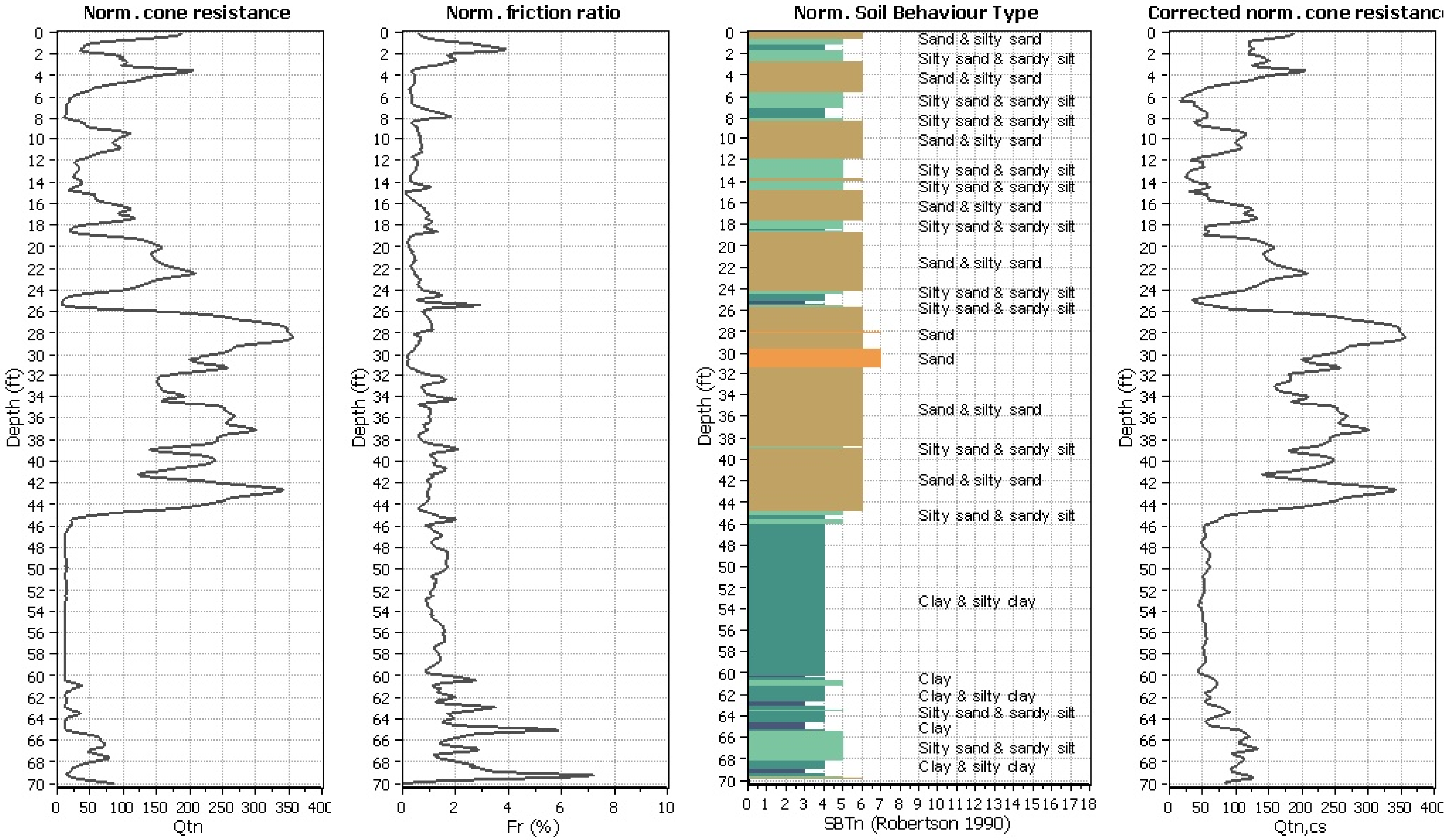
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

CLiq v.2.2.0.28 - CPTU data presentation & interpretation software - Report created on: 9/26/2018, 8:34:13 AM
Project file: Y:\Projects - Grover\216-193 SSLCSD - ES WWTP Redundancy KJ\4.0 CPT\216-193 SSLOCSD Redundancy Cliq updated 9-26-18 IC cutoff 2.4.clq

Yeh and Associates, Inc.
391 Front Street, Suite D
Grover Beach, CA 93433

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-03
Total depth: 70.05 ft



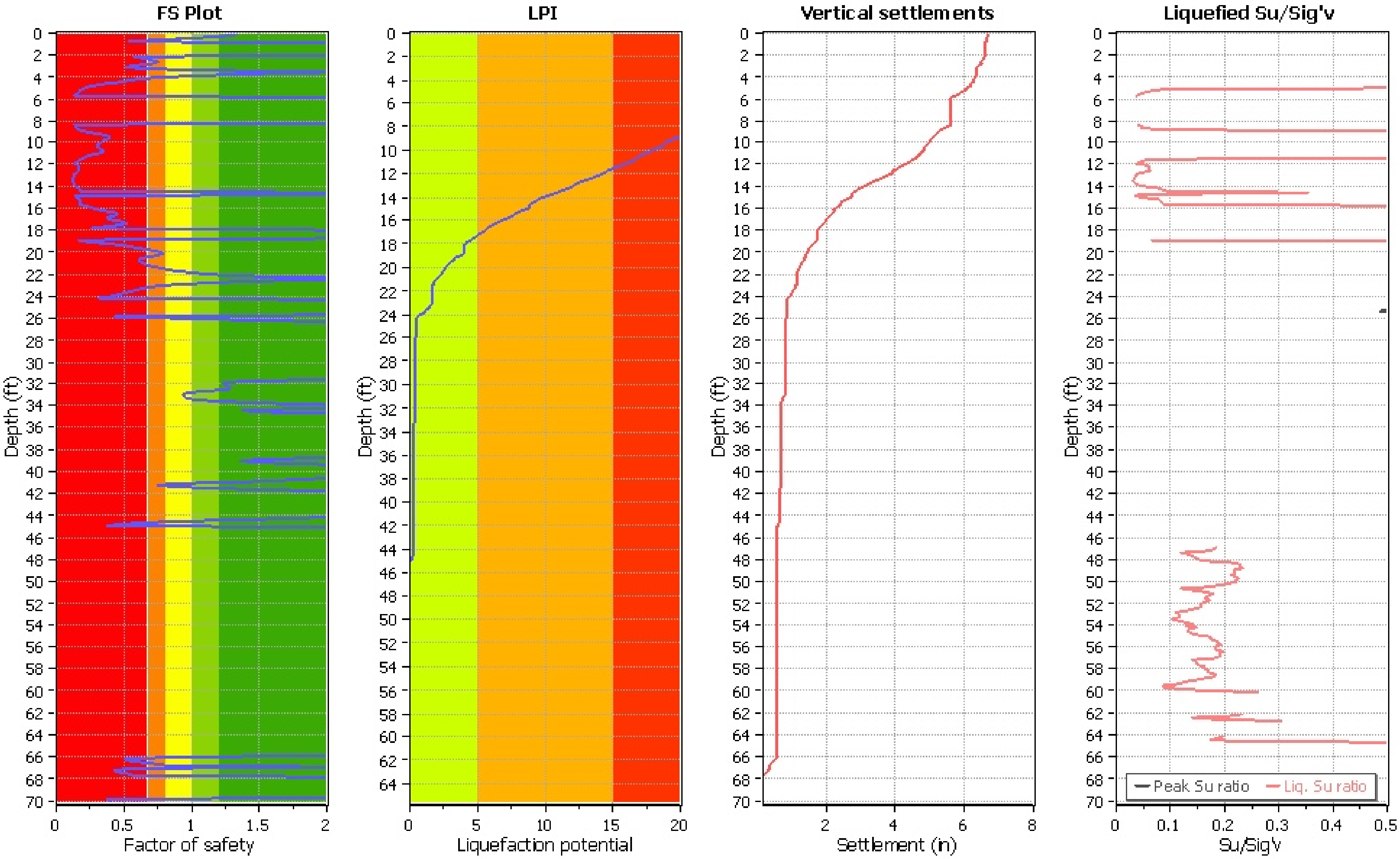
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Yeh and Associates, Inc.
Consulting Engineers & Scientists

Yeh and Associates, Inc.
391 Front Street, Suite D
Grover Beach, CA 93433

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-03
Total depth: 70.05 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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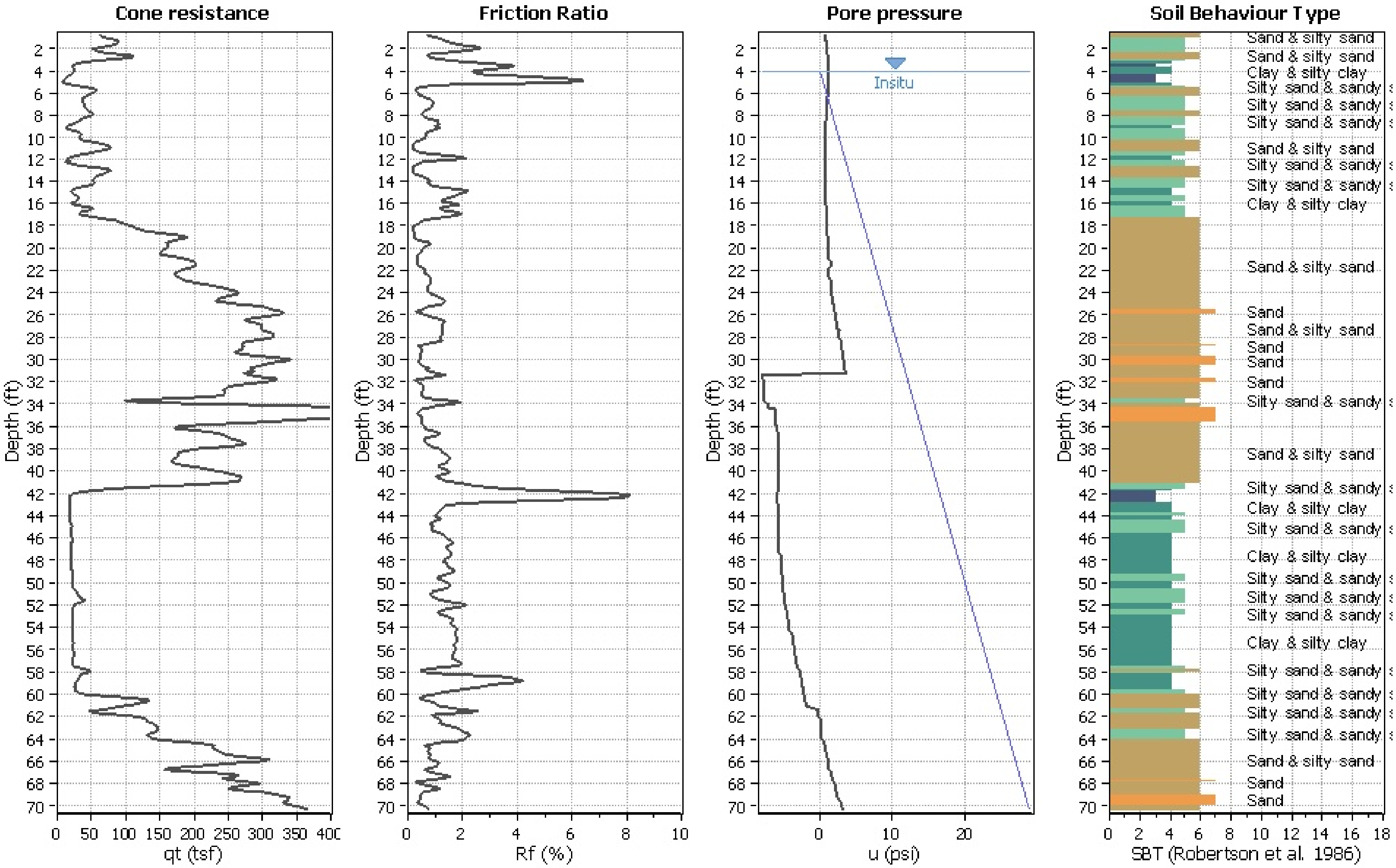
Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**

Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-04

Total depth: 70.37 ft



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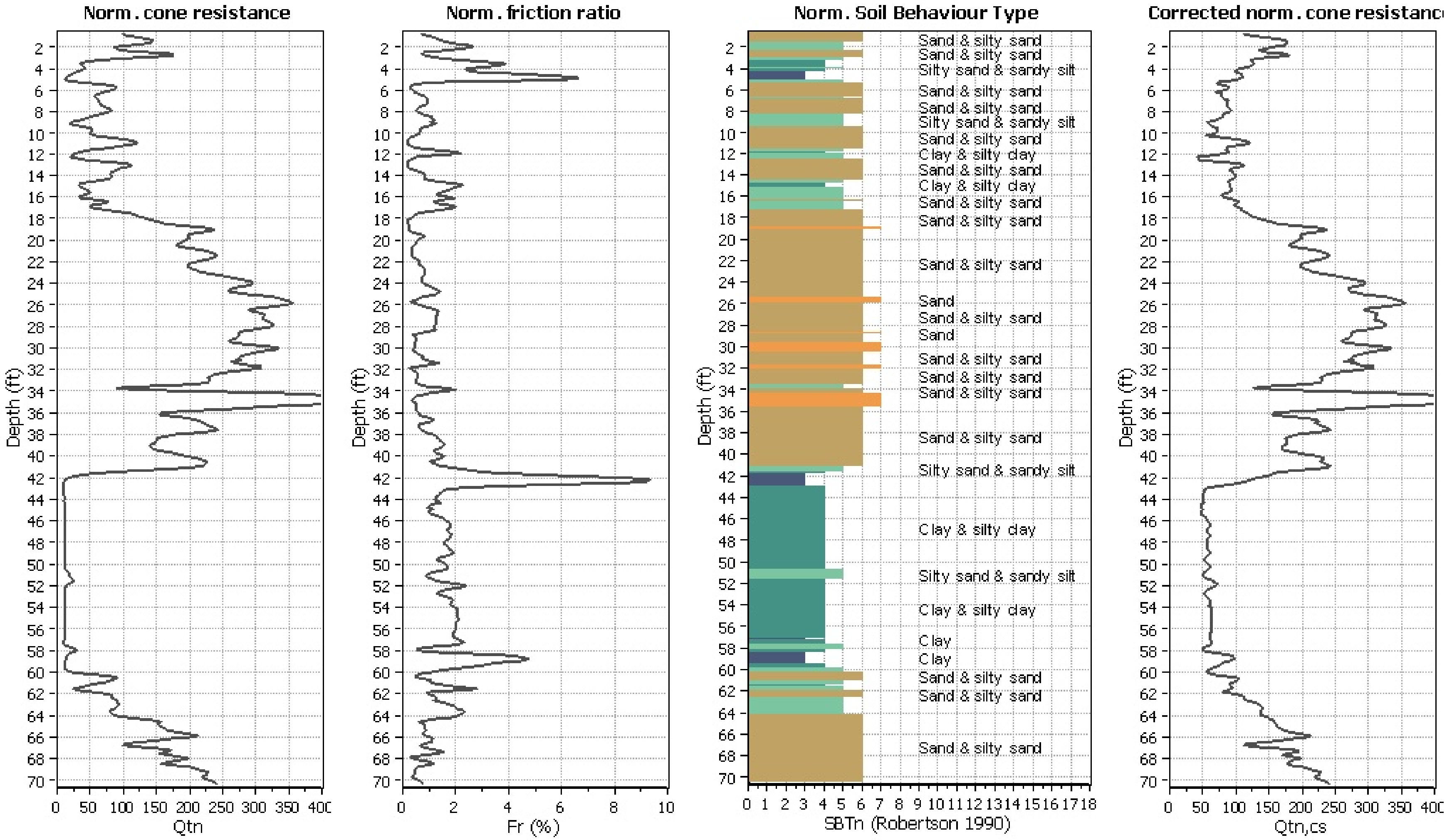
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-04
Total depth: 70.37 ft



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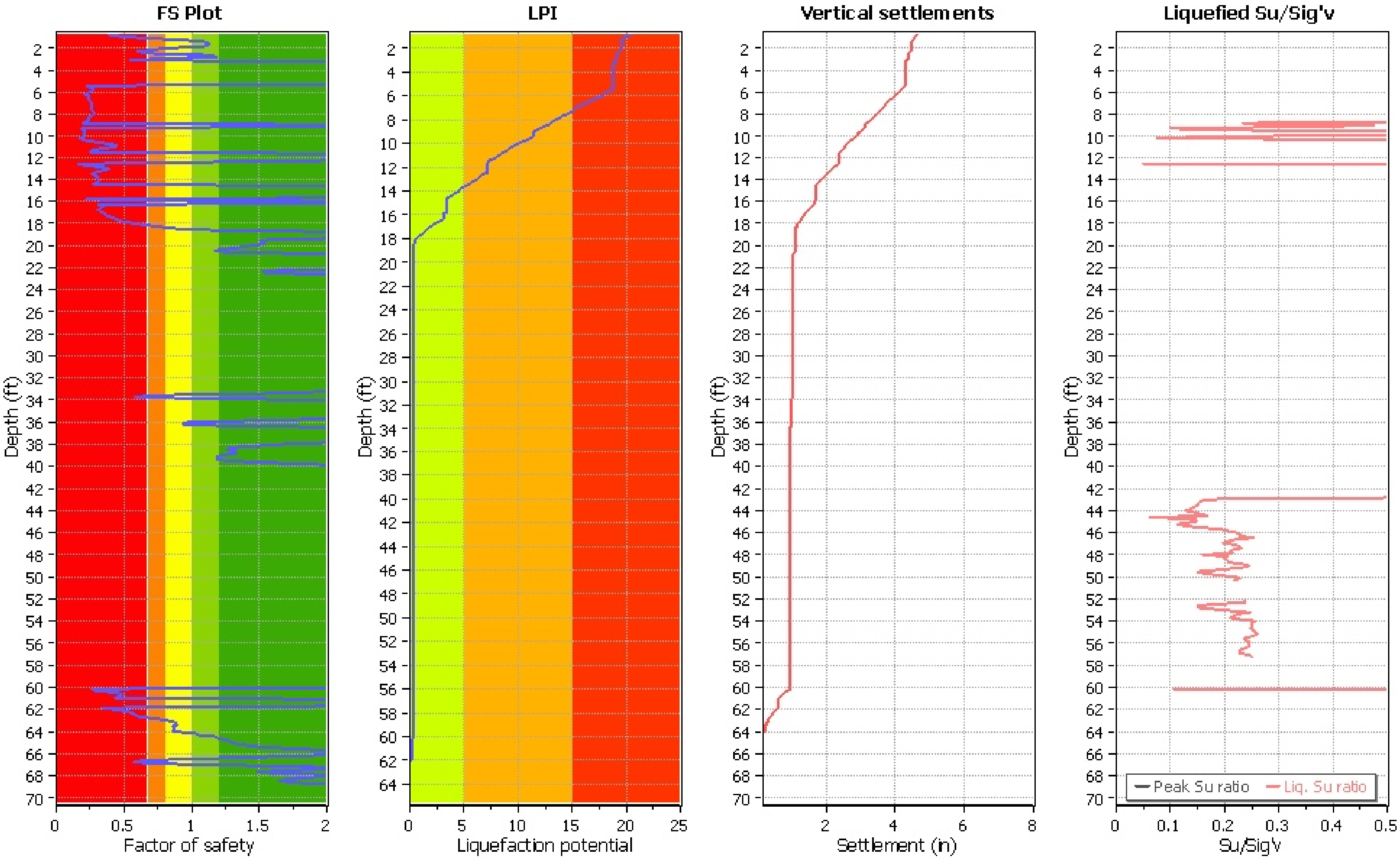
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-04
Total depth: 70.37 ft



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Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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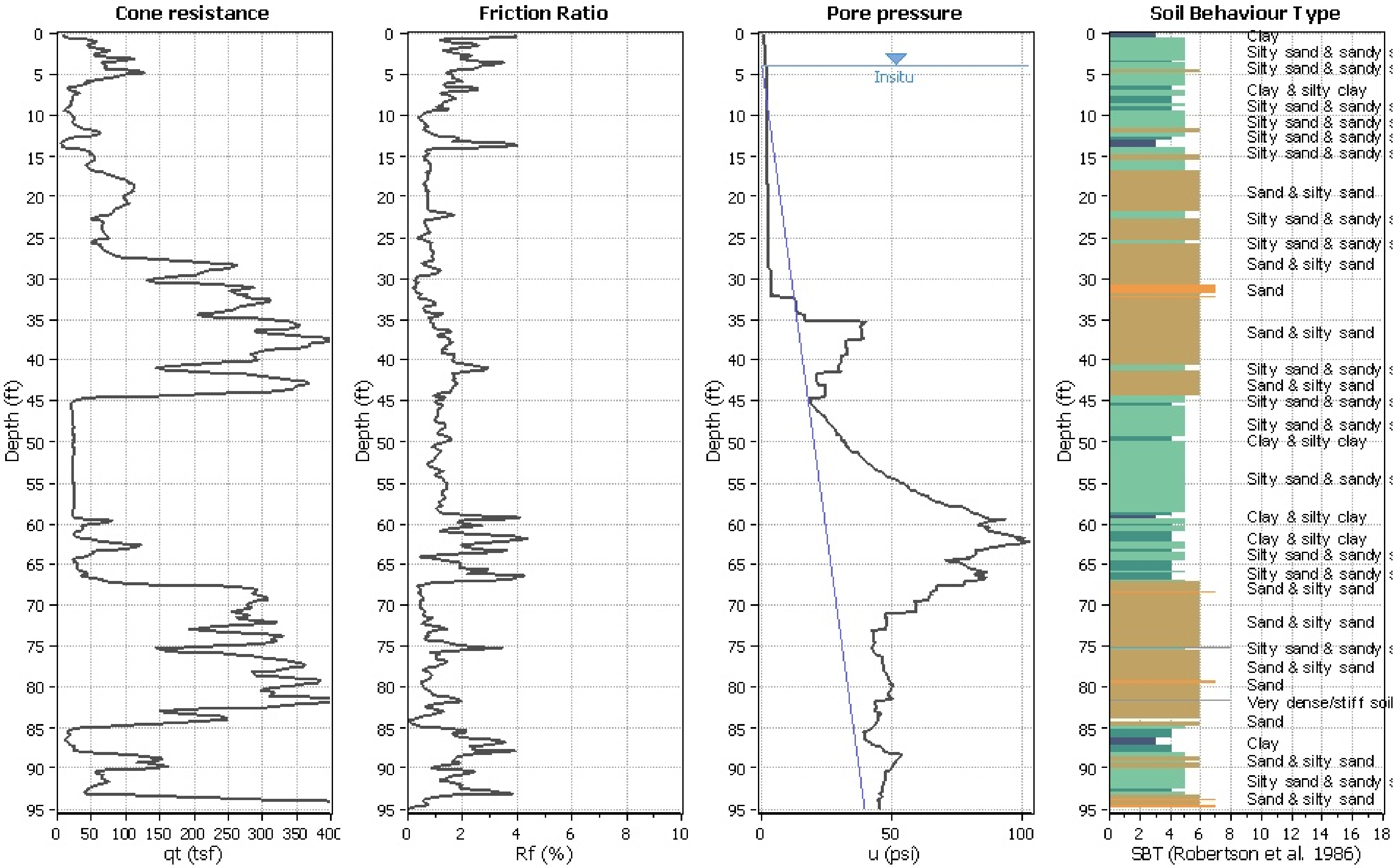
Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

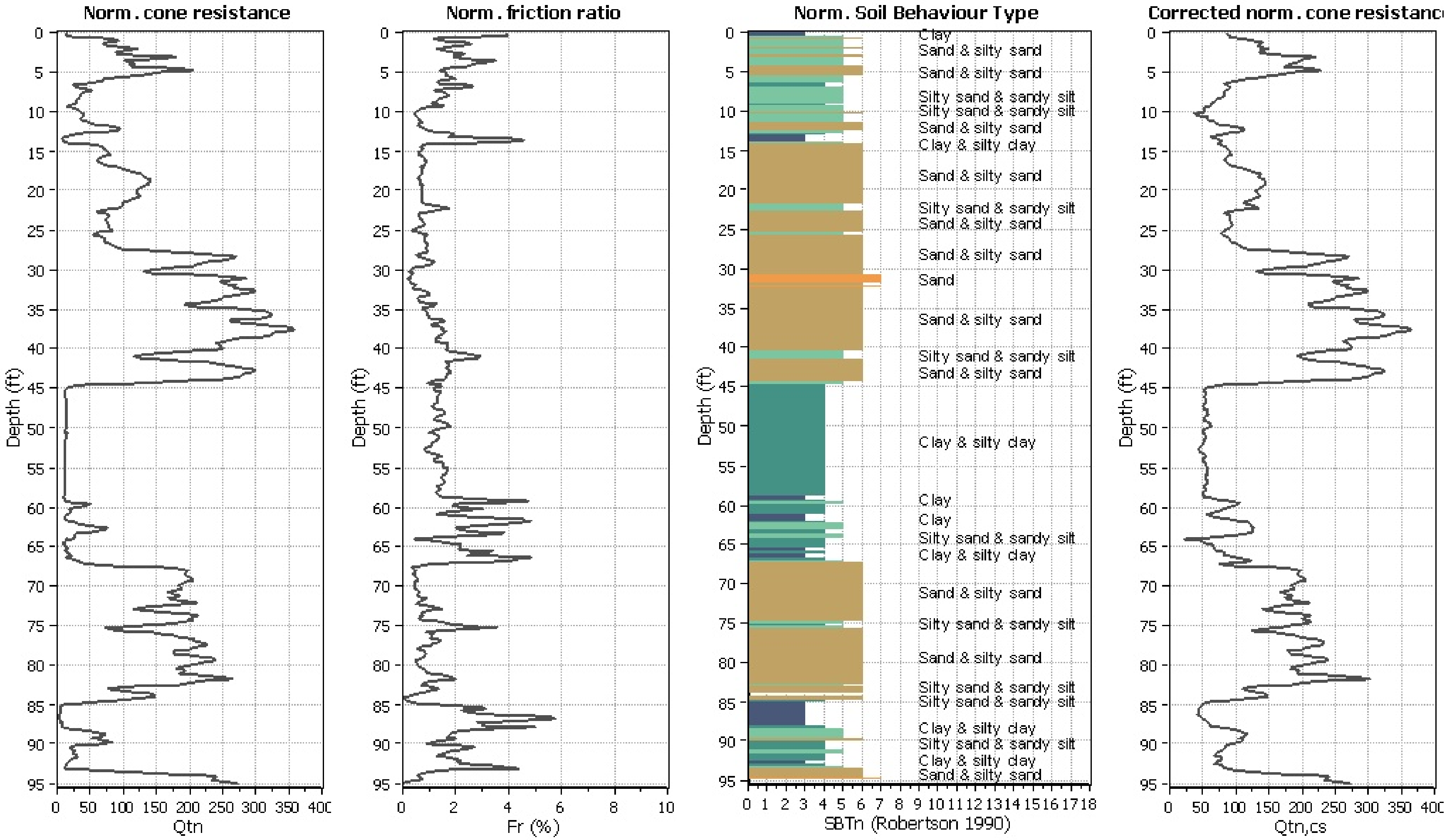
CPT: CPT-05
Total depth: 94.98 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

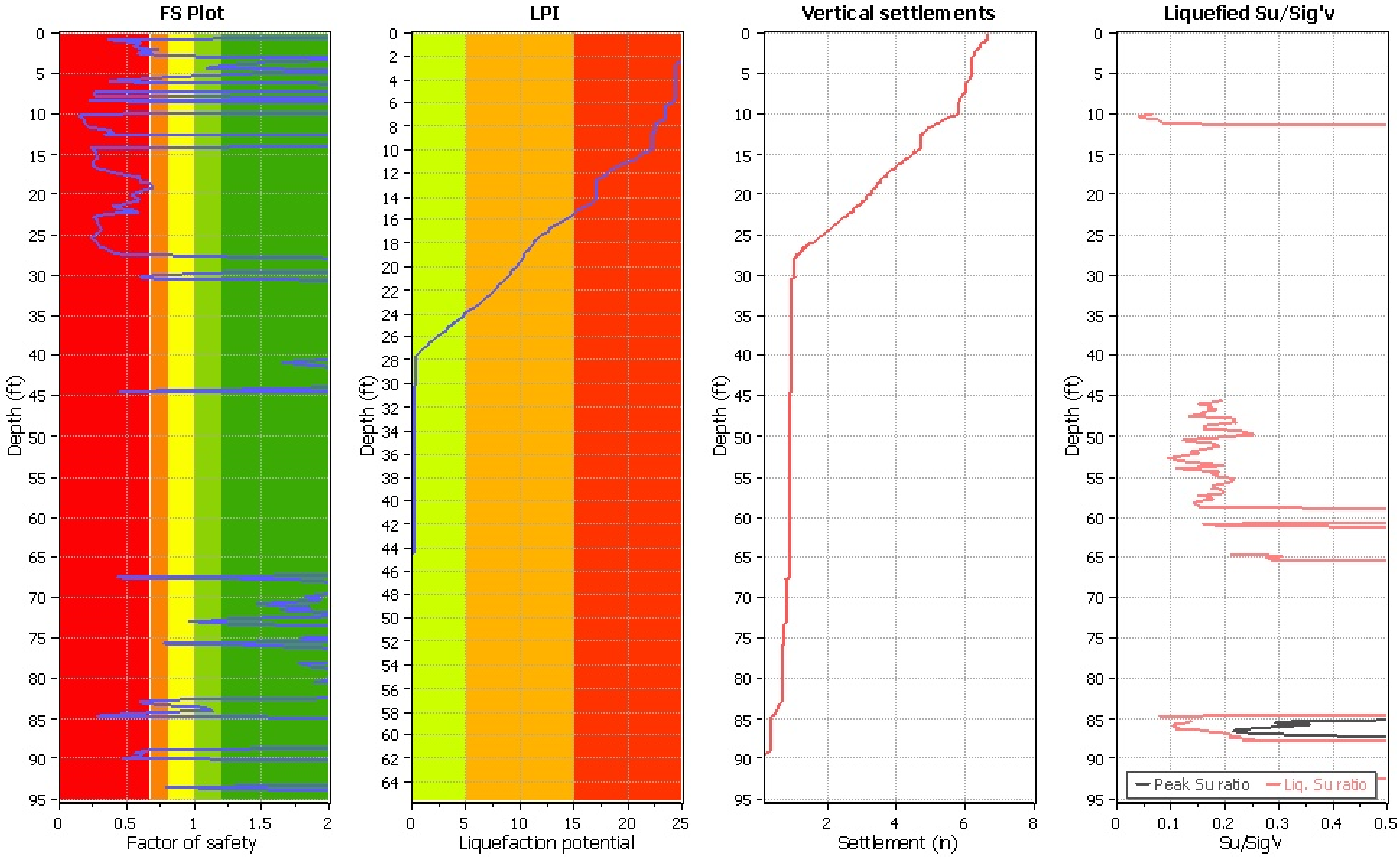
CPT: CPT-05
Total depth: 94.98 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-05
Total depth: 94.98 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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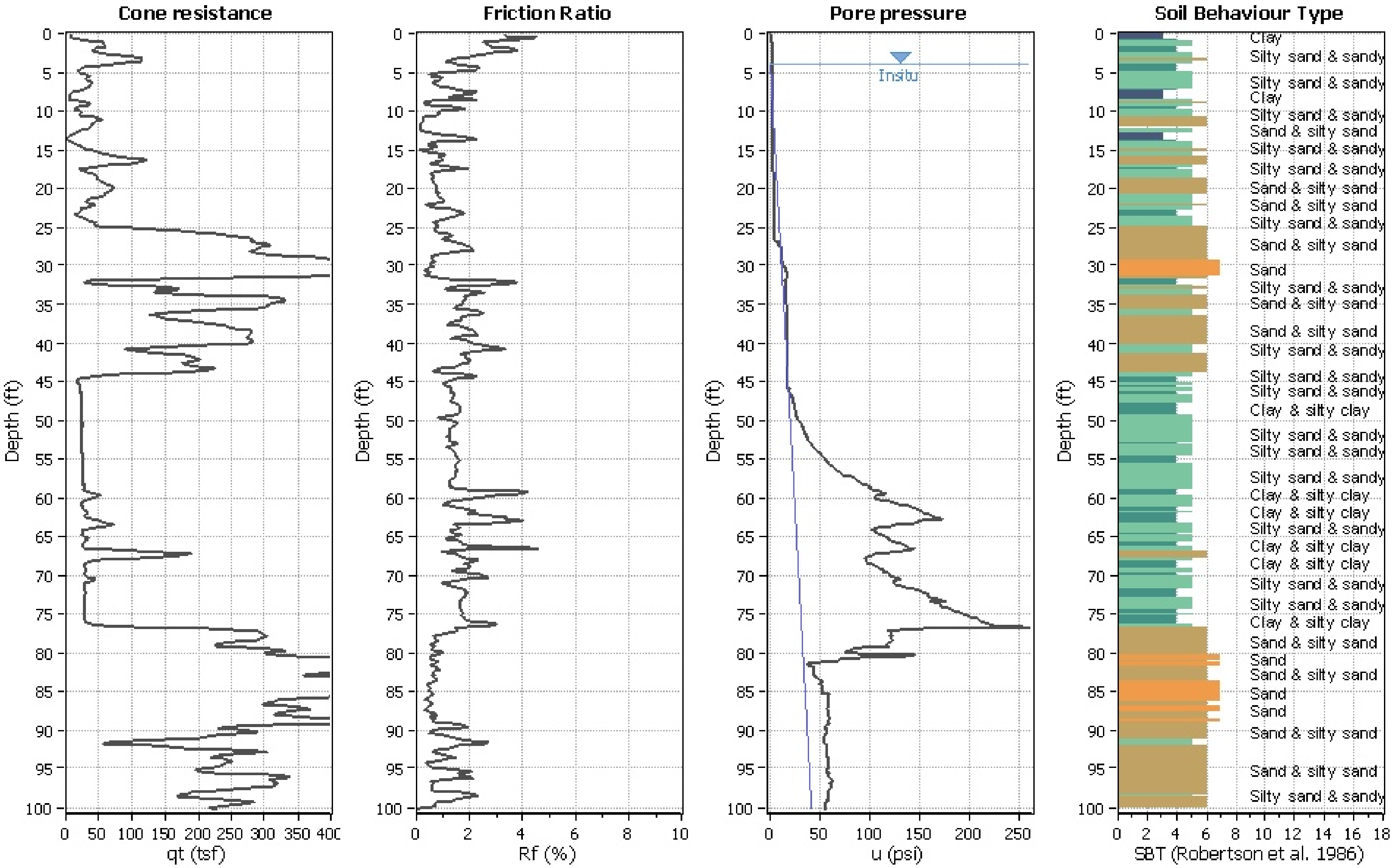
Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-06
Total depth: 100.23 ft

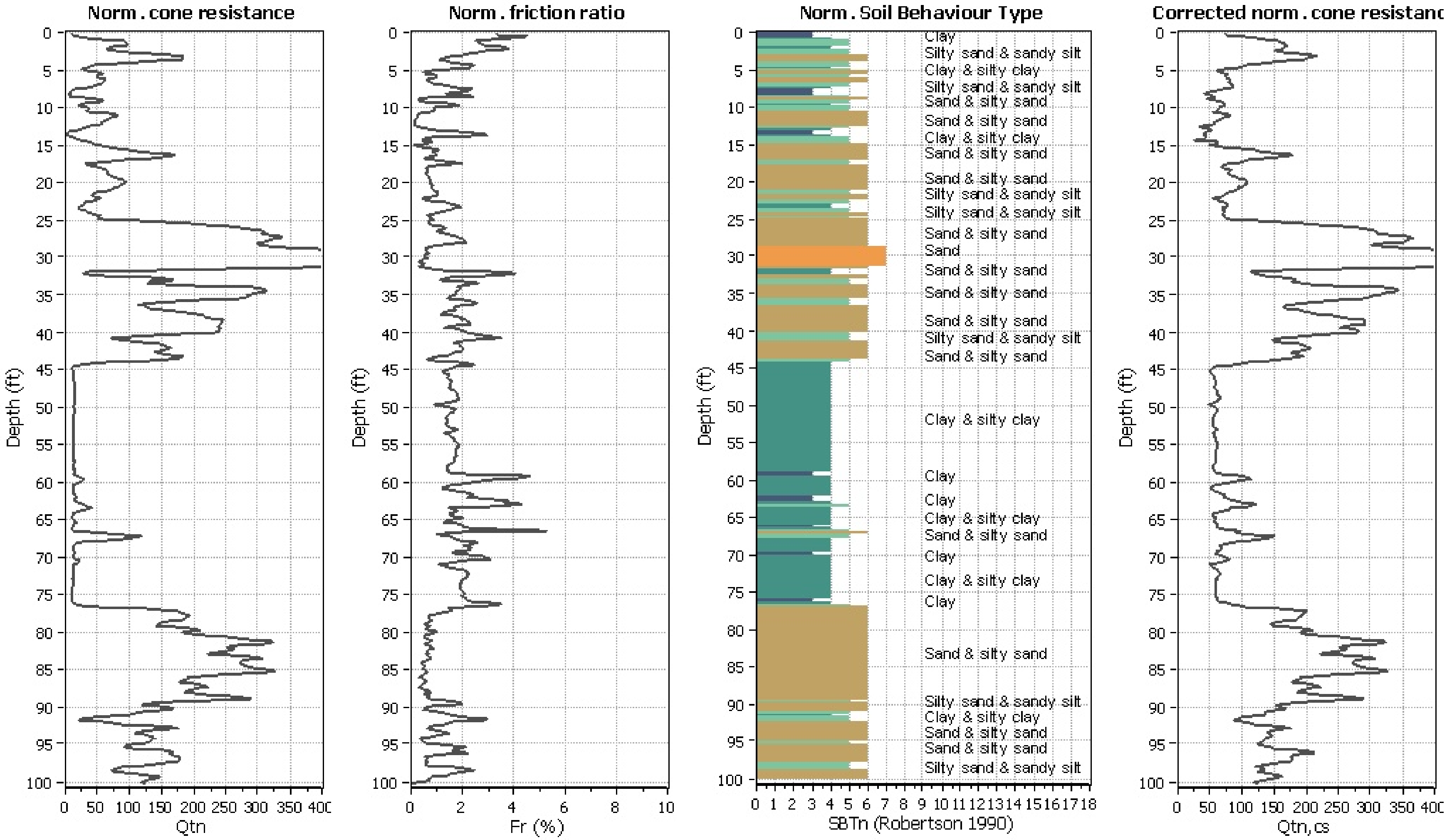


Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

CLiq v.2.2.0.28 - CPTU data presentation & interpretation software - Report created on: 9/26/2018, 8:34:17 AM
Project file: Y:\Projects - Grover\216-193 SSLCSD - ES WWTP Redundancy KJ\4.0 CPT\216-193 SSLOCSD Redundancy Cliq updated 9-26-18 IC cutoff 2.4.clq

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-06
Total depth: 100.23 ft



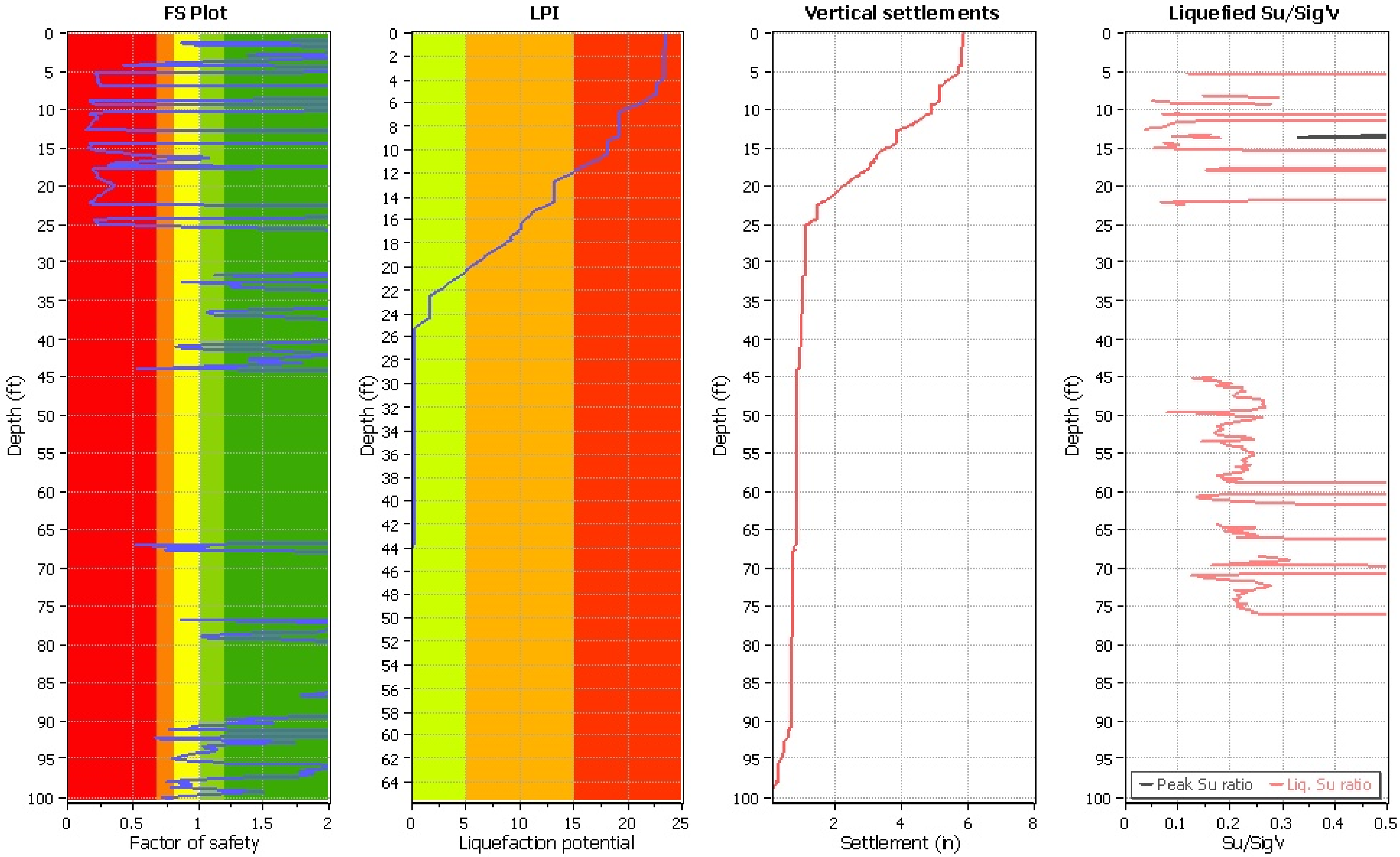
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**

Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-06

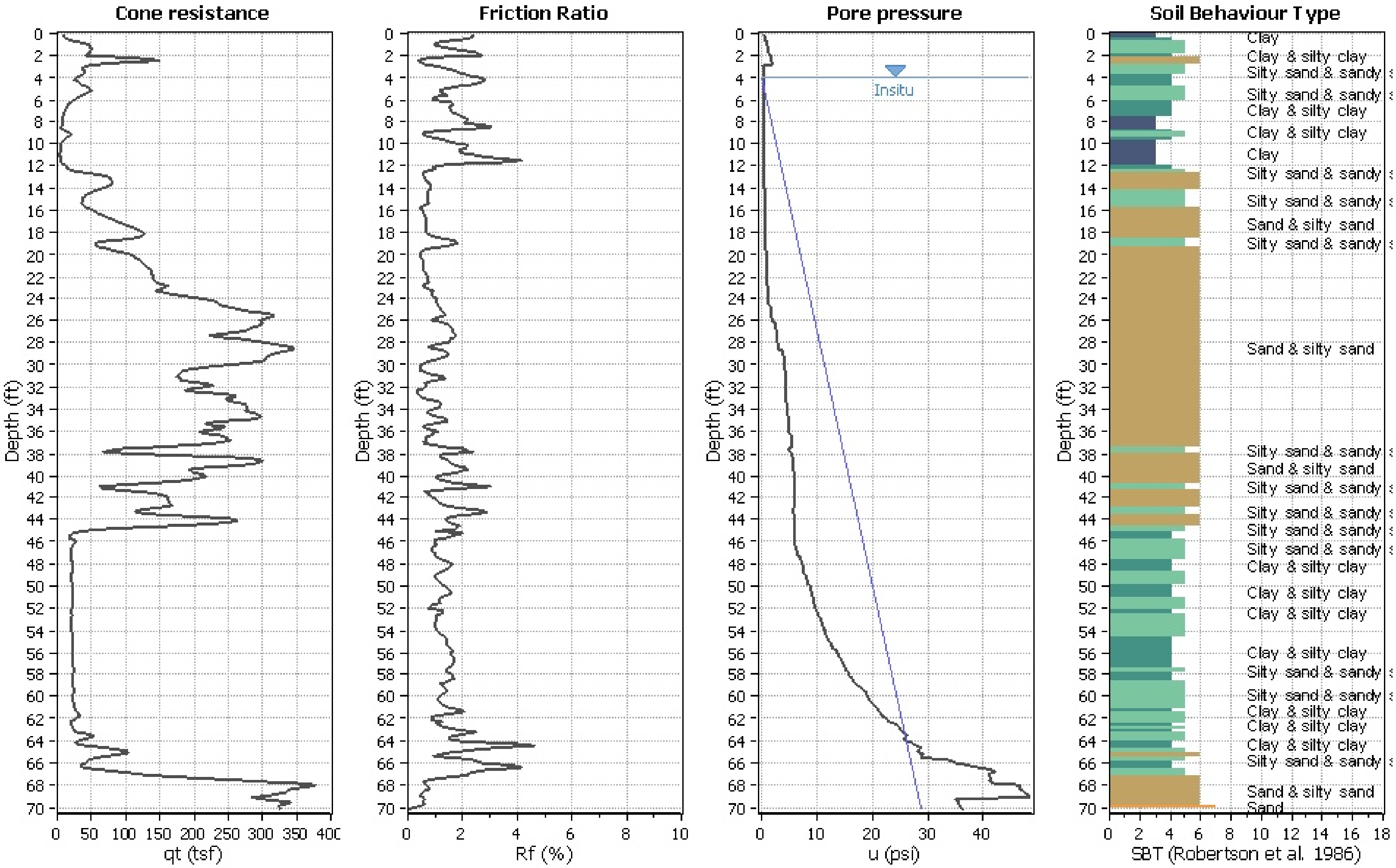
Total depth: 100.23 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

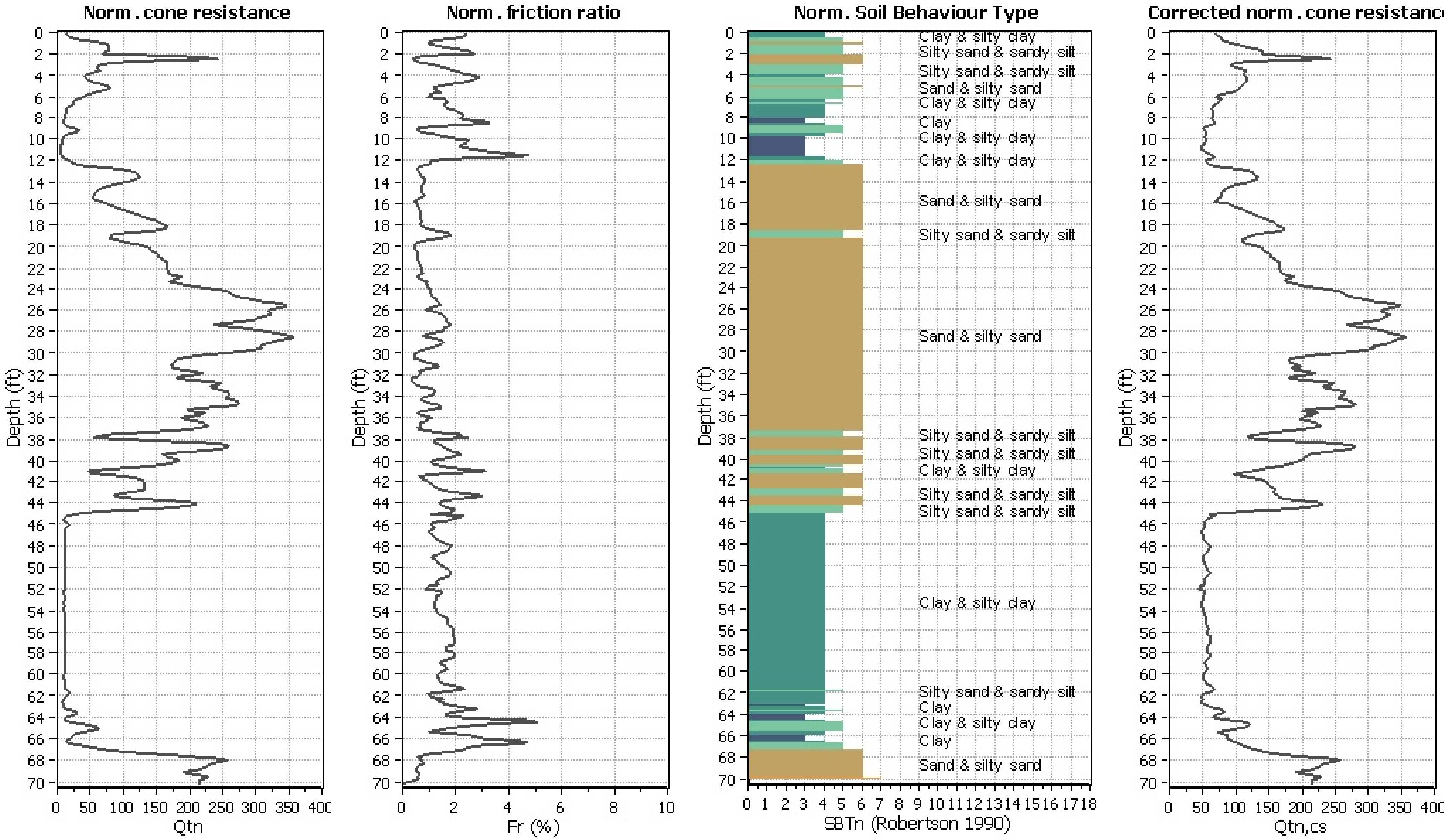
CPT: CPT-07
Total depth: 70.21 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

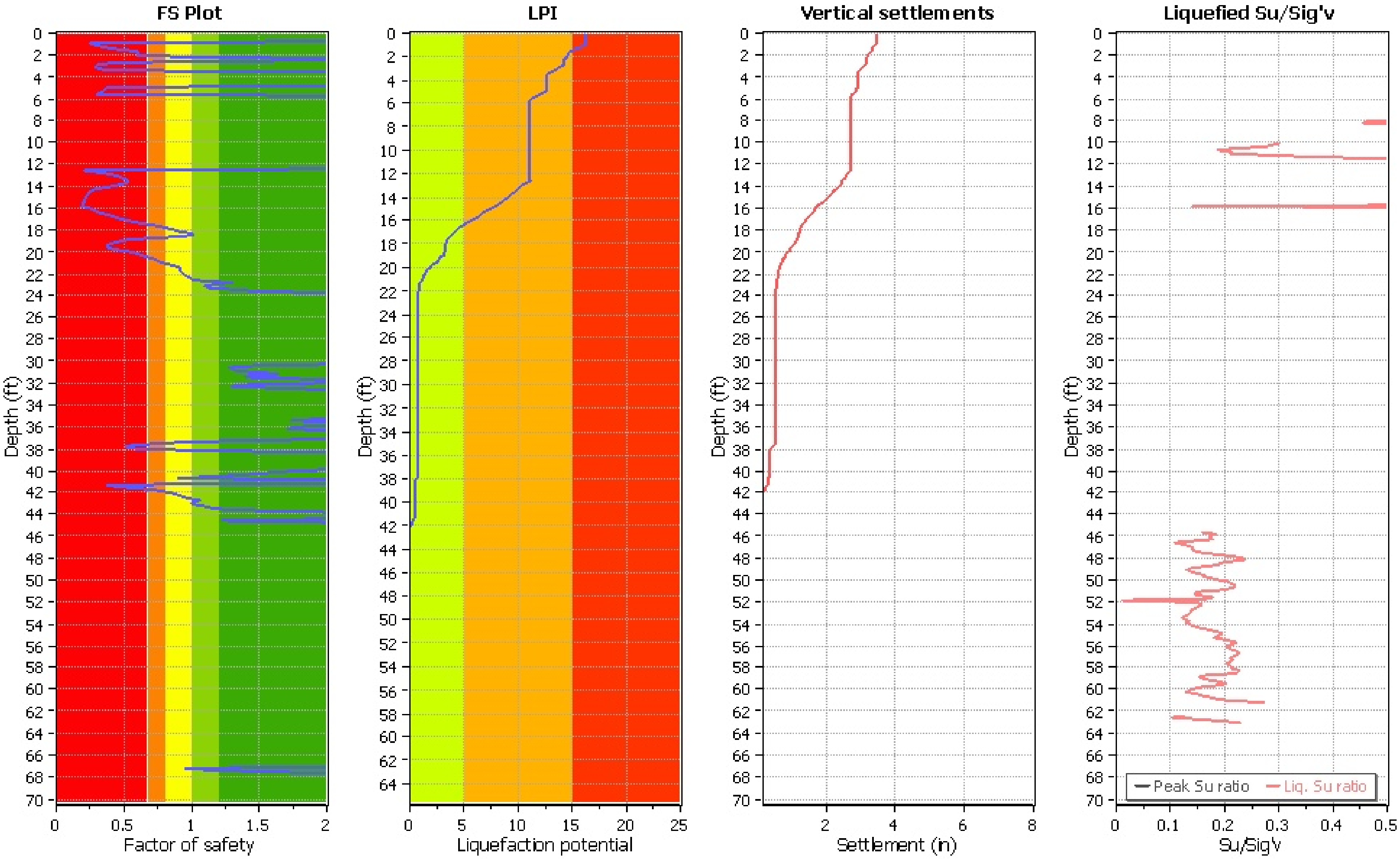
CPT: CPT-07
Total depth: 70.21 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

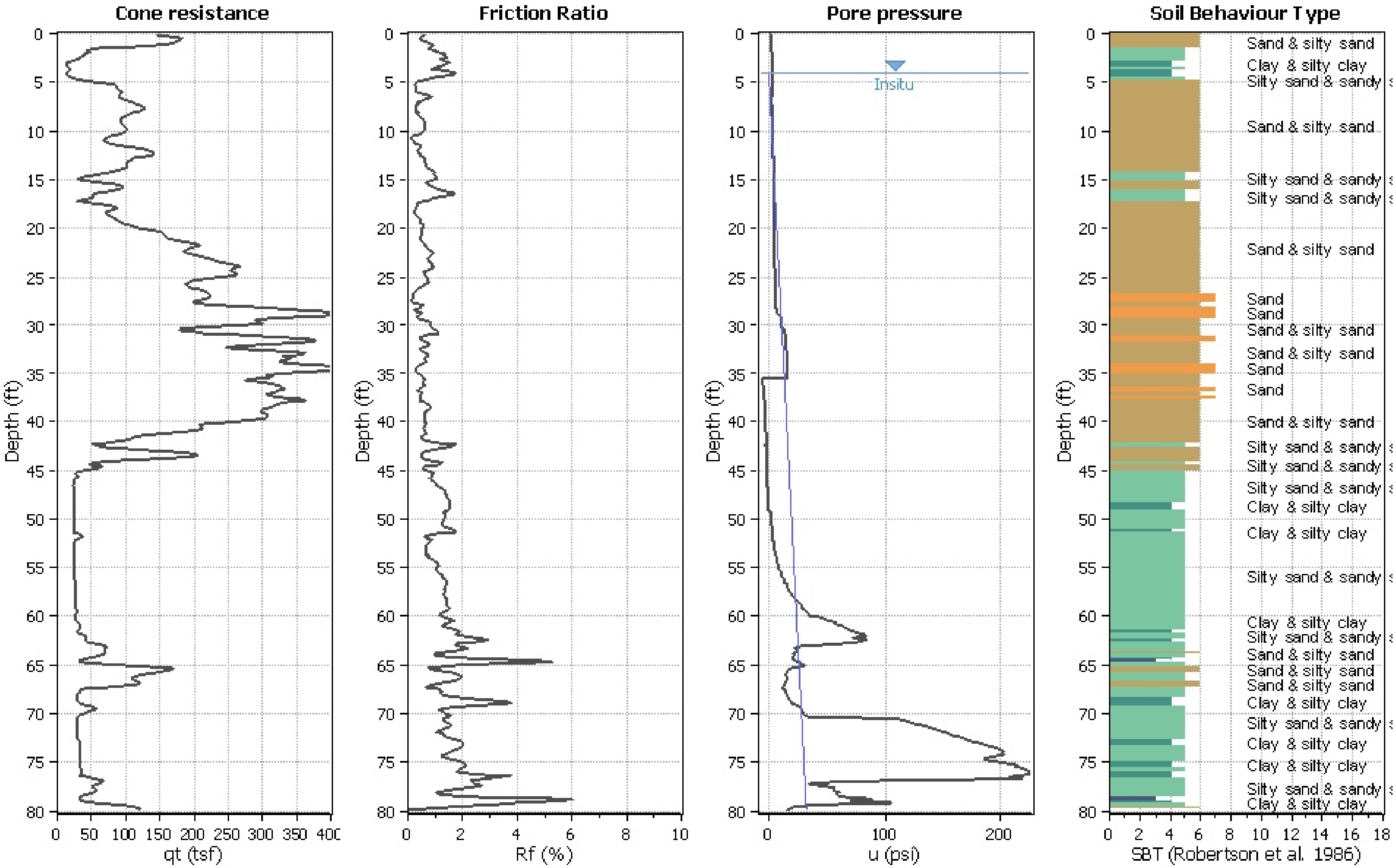
CPT: CPT-07
Total depth: 70.21 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-08
Total depth: 79.89 ft



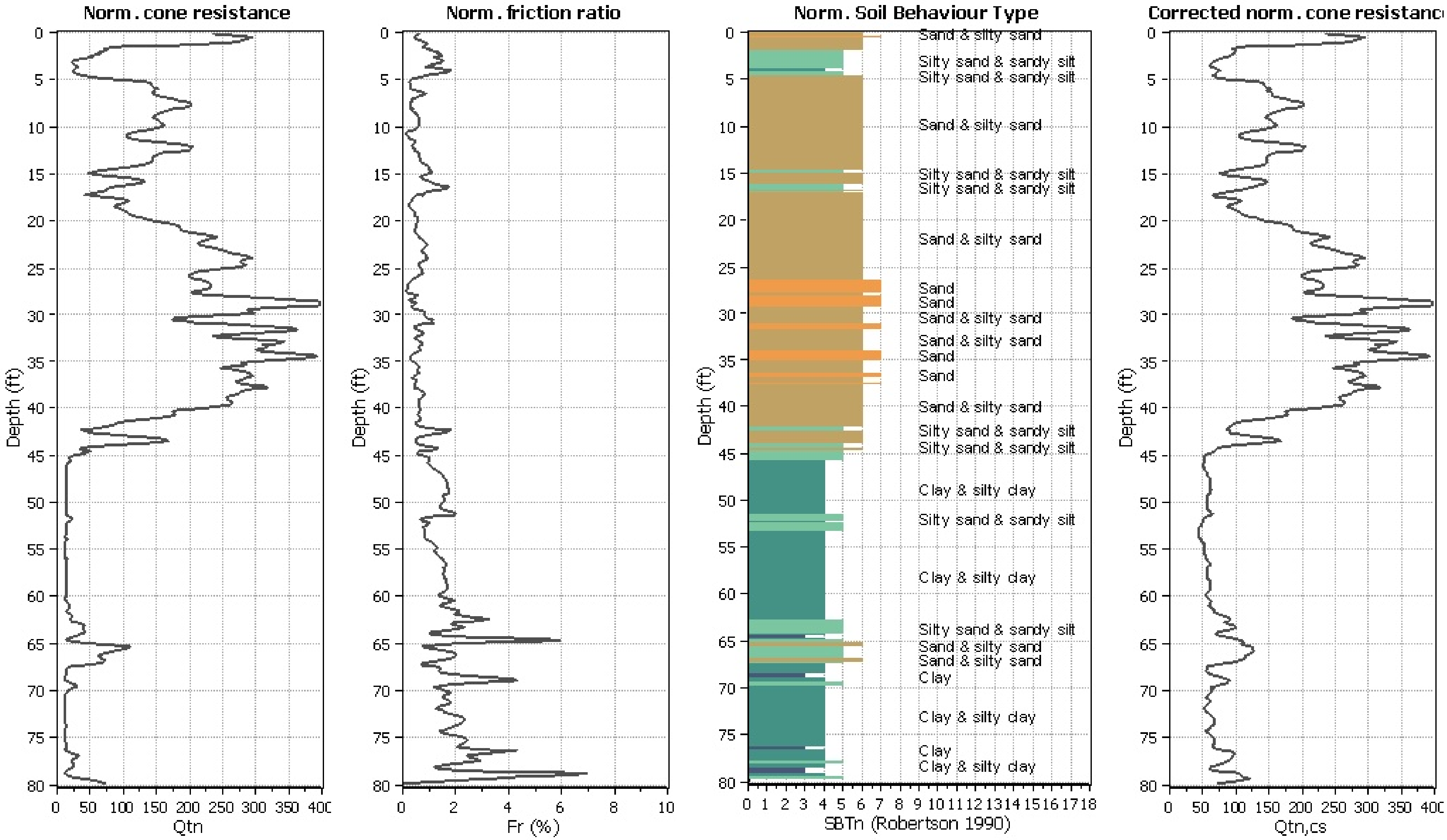
Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**

Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-08

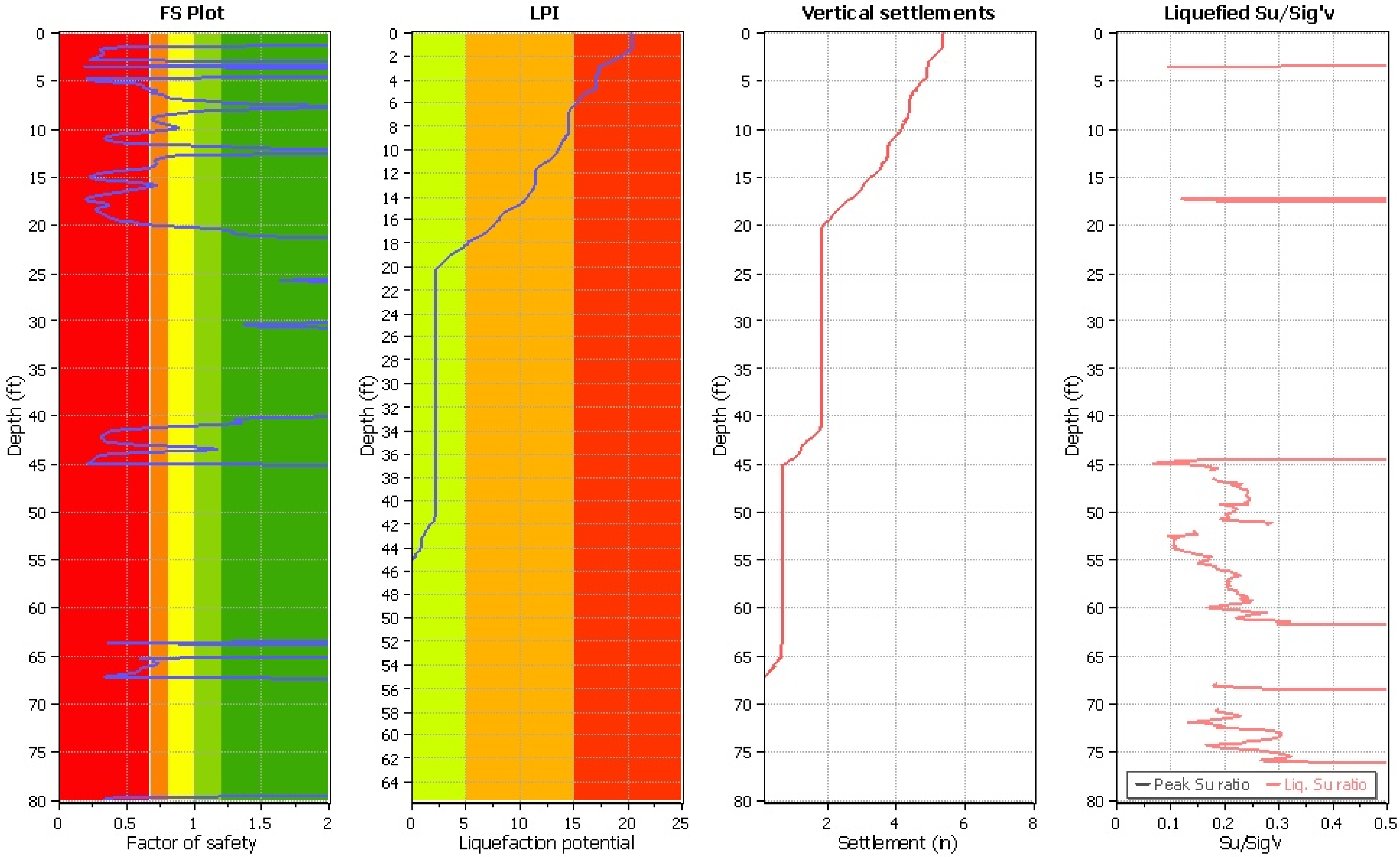
Total depth: 79.89 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No		
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Clay like behavior applied:	Sands only
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-08
Total depth: 79.89 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method based

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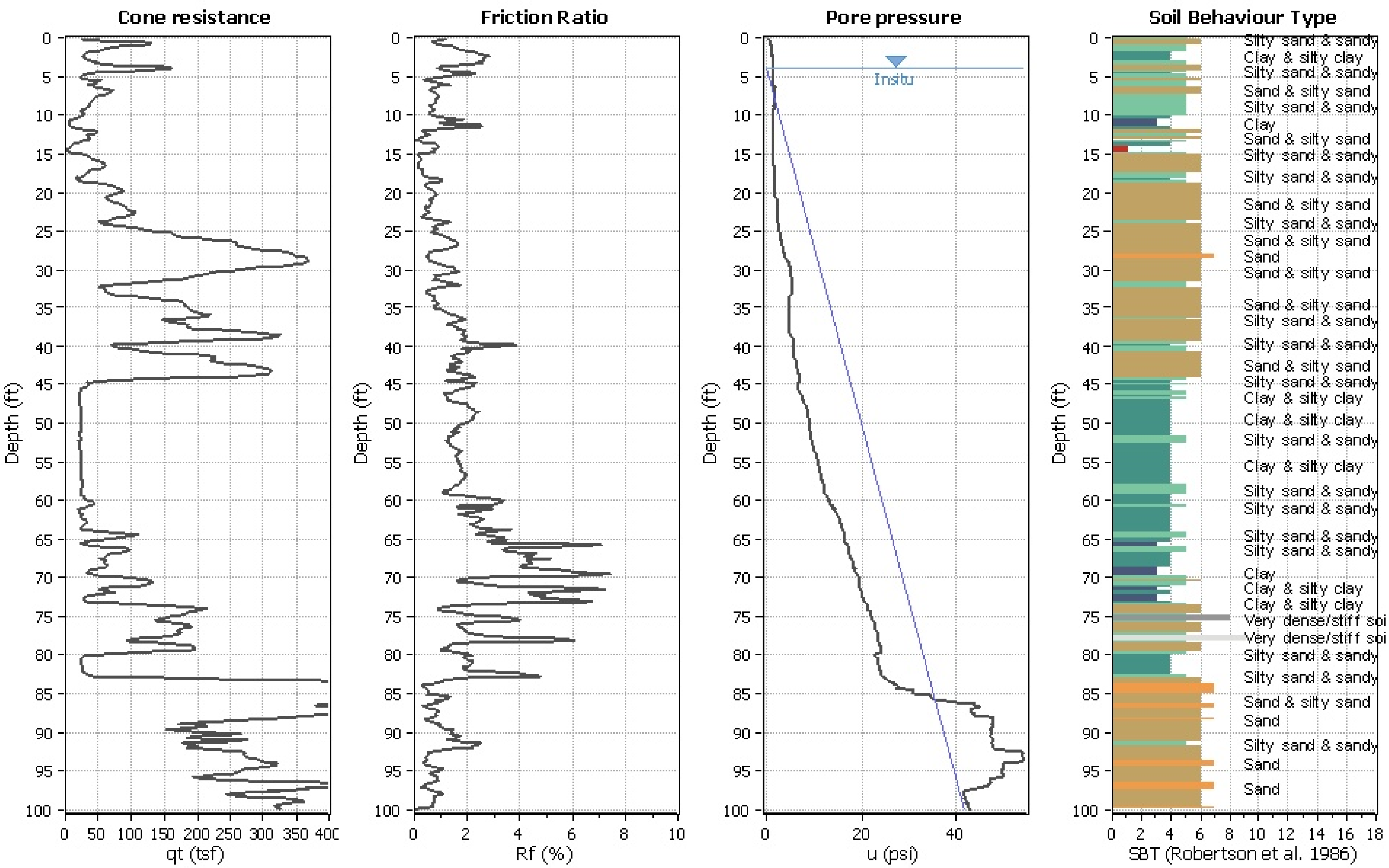
Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

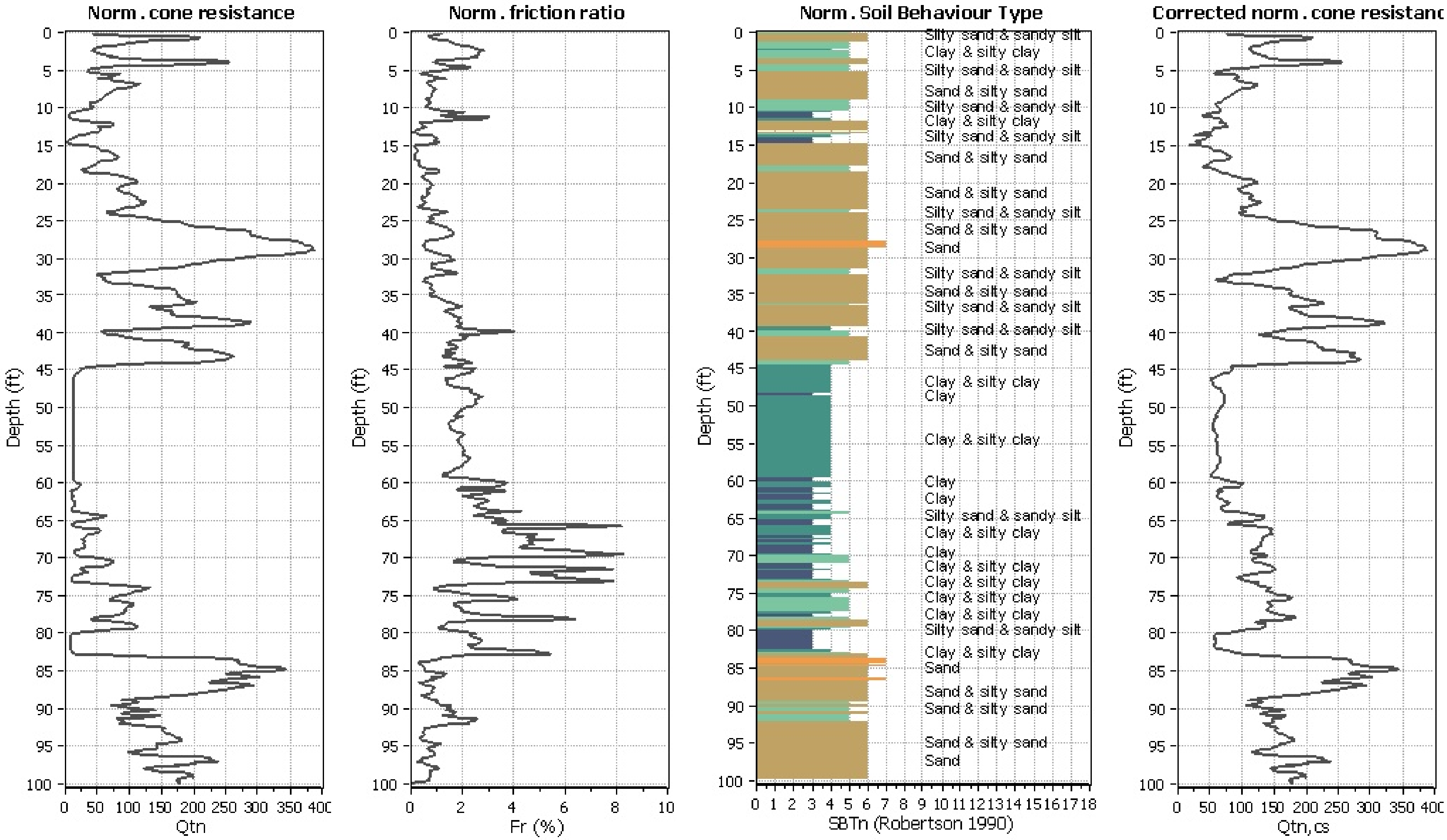
CPT: CPT-09
Total depth: 100.07 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-09
Total depth: 100.07 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

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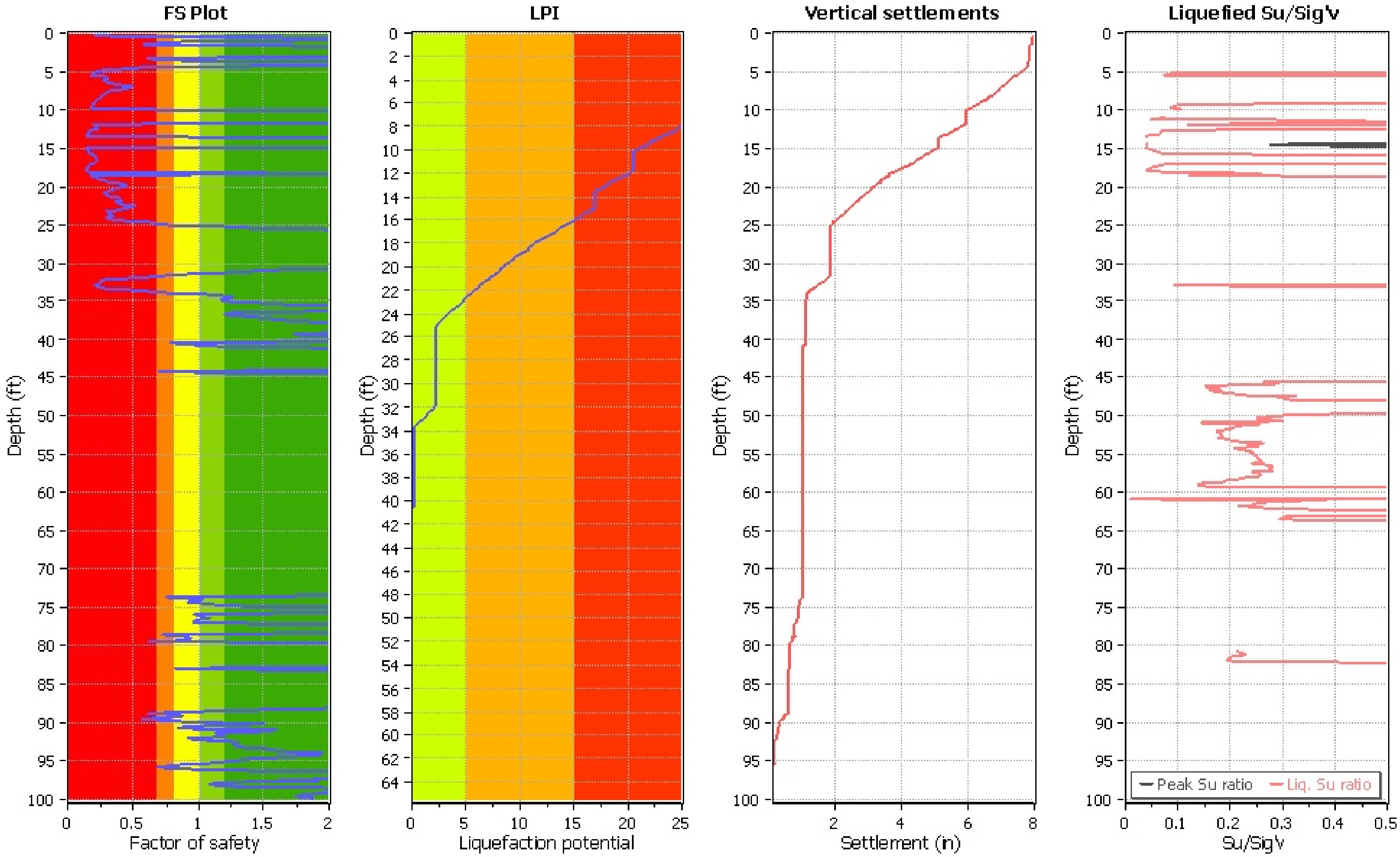
Project: **SSLOCSD - WWTP Redundancy Project - As-is Conditions**
Location: **1600 Aloha Ave, Oceano, CA**

CPT: CPT-09
Total depth: 100.07 ft



Analysis method:	NCEER (1998)	G.W.T. (in-situ):	4.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	0.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	6.70	Ic cut-off value:	2.40	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

APPENDIX B - RESULTS OF LABORATORY TESTING

Summary of Laboratory Test Results

Project No.	216-193	**Project Name**	SSLOCSD - Redundancy	**Date**	8/24/2018

Sample Location			Moisture Content (%)	Dry Unit Weight (pcf)	Gradation			Atterberg		Corrosion				Compaction		R-Value	USCS Classification	AASHTO Classification
Boring No.	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%) (s=silt, c= clay)	LL	PI	pH	R (ohm-cm)	SO_4^{2-} (mg/kg)	Cl^- (mg/kg)	Max. Unit Weight (pcf)	Optimum Water Content (%)			
16E-01	6.5-9	Bulk	--	--	--	--	--	--	--	8.39	817	122	117	--	--	--	Lean CLAY (CL)	--
16E-01	1	Cal	14.9	93.5	--	--	--	--	--	8.97	2008	--	--	--	--	--	Clayey SAND with Gravel (SC)	--
16E-01	5	Cal	33.9	88.1	--	--	--	--	--	--	--	--	--	--	--	--	Clayey SAND (SC)	--
16E-01	9	SPT	27.9	--	0	92	8	--	--	--	--	--	--	--	--	--	Poorly graded SAND with clay (SP-SC)	--
16E-01	13.5	Tube	27.0	--	0	98	2	--	--	--	--	--	--	--	--	--	Poorly graded SAND (SP)	--
16E-01	23.5	Cal	24.9	99.5	--	--	--	--	--	--	--	--	--	--	--	--	Poorly graded SAND (SP)	--
16E-01	42	Tube	42.9	76.6	--	--	--	--	--	--	--	--	--	--	--	--	Fat CLAY (CH)	--
16E-01	45	Tube	46.5	74.9	0	1	S: 39, C: 60	76	44	--	--	--	--	--	--	--	Fat CLAY (CH)	--
16E-02	1-8	Bulk	18.1	--	11	57	32	31	9	--	--	--	--	117	12.5	--	Clayey SAND (SC)	--
16E-02	1	Cal	15.1	116.3	--	--	--	--	--	--	--	--	--	--	--	--	Clayey SAND (SC)	--
16E-02	5	Cal	20.7	106.0	--	--	--	--	--	8.50	1143	--	--	--	--	--	Poorly graded SAND (SP)	--
16E-02	9	Tube	37.0	81.9	--	--	--	--	--	--	--	--	--	--	--	--	Poorly graded SAND (SP)	--
16E-02	15	SPT	27.9	--	3	84	12	--	--	--	--	--	--	--	--	--	Clayey SAND (SC)	--
16E-02	20	Cal	21.6	104.2	--	--	--	--	--	--	--	--	--	--	--	--	Poorly graded SAND (SP)	--
16E-02	25	SPT	30.2	--	1	90	9	--	--	--	--	--	--	--	--	--	Poorly graded SAND with clay (SP-SC)	--
16E-02	30	Cal	24.4	100.0	--	--	--	--	--	--	--	--	--	--	--	--	Poorly graded SAND with Clay (SP-SC)	--
16E-02	35	SPT	23.5	--	34	56	10	--	--	--	--	--	--	--	--	--	Poorly graded SAND with clay and gravel (SP-SC)	--
16E-02	40	Cal	26.7	96.9	--	--	--	--	--	--	--	--	--	--	--	--	Poorly graded SAND (SP)	--
16E-02	55	SPT	42.5	--	0	8	S: 45, C: 47	67	38	--	--	--	--	--	--	--	Fat CLAY (CH)	
16E-02	60	Tube	36.7	82.9	--	--	--	--	--	--	--	--	--	--	--	--	Lean CLAY (CL)	--
16E-03	1-7	Bulk	22.6	--	4	65	31	28	8	--	--	--	--	--	--	51	Clayey SAND (SC)	--
16E-03	1	Cal	25.1	97.3	4	68	28	--	--	--	--	--	--	--	--	--	Clayey SAND (SC)	--
16E-03	5	Cal	29.2	90.7	--	--	--	--	--	--	--	--	--	--	--	--	Poorly graded SAND with clay (SP-SC)	--
16E-03	9	Cal	35.8	83.8	--	--	--	--	--	--	--	--	--	--	--	--	Poorly graded SAND with Clay (SP-SC)	--
16E-03	12	Grab	25.4	--	16	64	20	--	--	8.09	1507	--	--	--	--	--	Clayey SAND with gravel (SC)	--
16E-03	15	SPT	23.6	--	40	56	4	--	--	--	--	--	--	--	--	--	Poorly graded SAND with gravel (SP)	--
16E-03	22	Tube	37.4	85.9	6	81	13	--	--	--	--	--	--	--	--	--	Clayey SAND (SC)	--
16E-03	30	Cal	24.4	106.8	17	72	10	--	--	--	--	--	--	--	--	--	Poorly graded SAND with clay and gravel (SP-SC)	--
16E-03	40	Cal	24.8	99.6	--	--	--	--	--	--	--	--	--	--	--	--	Poorly graded SAND (SP)	--

Summary of Laboratory Test Results

Project No.	216-193	**Project Name**	SSLOCSD - Redundancy	**Date**	6/17/2016

Sample Location			Moisture Content (%)	Dry Unit Weight (pcf)	Gradation			Atterberg		Corrosion				Compaction		R-Value	USCS Classification	AASHTO Classification
Boring No.	Depth (ft)	Sample Type			Gravel > #4 (%)	Sand (%)	Fines < #200 (%)	LL	PI	pH	R (ohm-cm)	SO_4^{2-} (mg/kg)	Cl^- (mg/kg)	Max. Unit Weight (pcf)	Optimum Water Content (%)			
16E-03	45	Tube	30.3	90.2	--	--	--	--	--	--	--	--	--	--	--	--	Poorly graded SAND (SP)	--
16E-03	60	Tube	32.6	87.3	--	--	--	42	16	--	--	--	--	--	--	--	Lean CLAY with sand (CL)	--

Mechnical Sieve Analysis

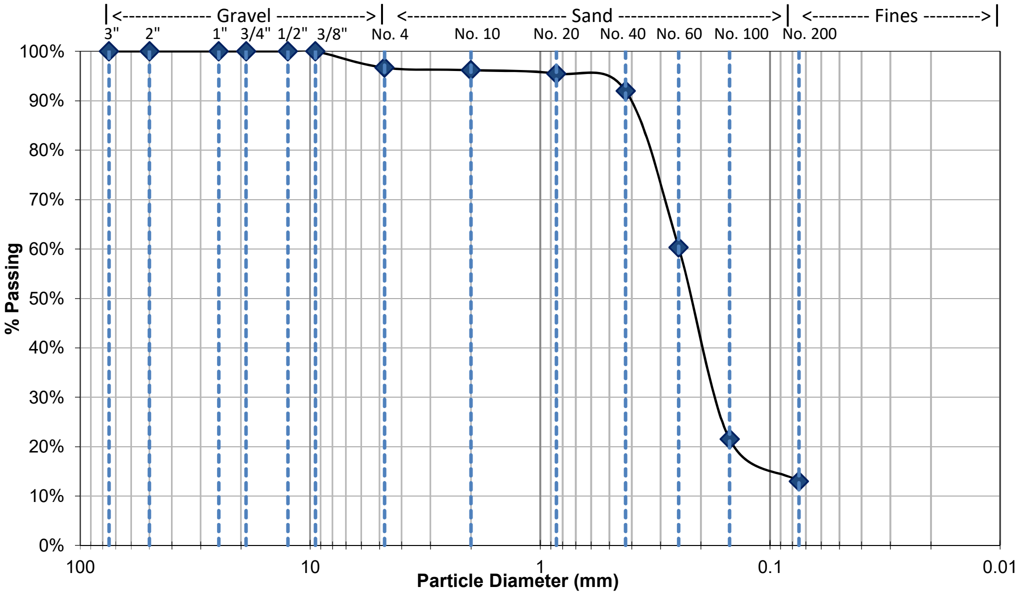
Test Method: ASTM D6913, D2487

Project Name	SSLOCSD - Redundancy		**Project No.**	216-193	
Tested By	J. Cravens	**Checked By**	J. King	**Testing Date**	6/10/2016

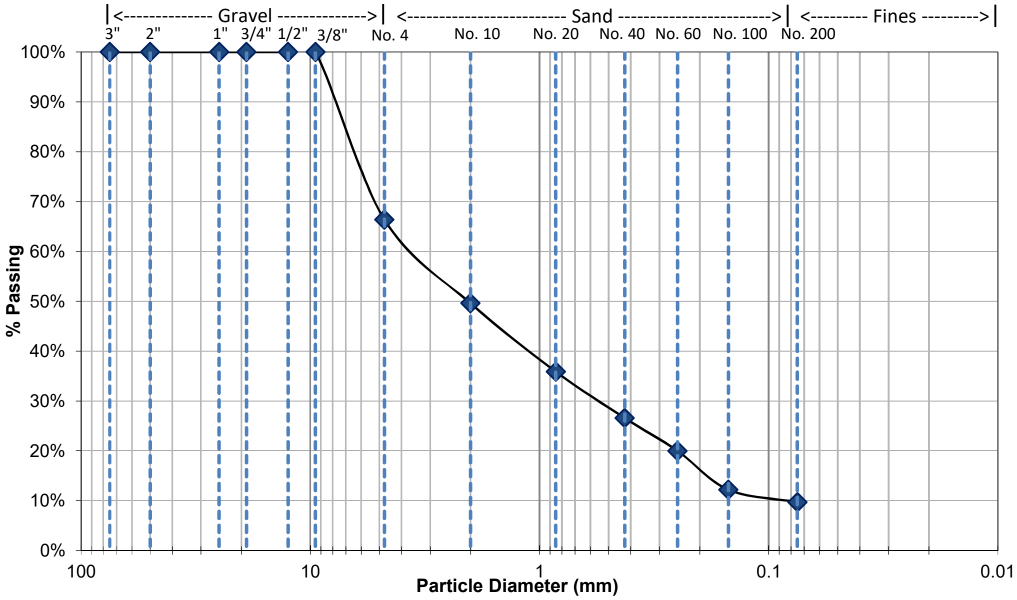
SUMMARY OF RESULTS													
Boring No.	**Sample No.**	**Depth (ft)**	**Water Content (%)**	**Gravel (%)**	**Sand (%)**	**Fines (%)**	D_{10} **(mm)**	D_{30} **(mm)**	D_{60} **(mm)**	C_u	C_c	**USCS Classification & Soil Description**	**AASHTO Classification**
16E-02	17	15	27.9%	3%	84%	12%	0.08	0.17	0.25	3.30	0.10	Clayey SAND (SC), grayish brown, wet	--
16E-02	21	35	23.5%	34%	56%	10%	0.08	0.58	3.70	44.10	14.80	Poorly graded SAND with clay and gravel (SP-SC), tan, wet, with shell fragments	--
16E-03	C	1-7	22.6%	4%	65%	31%	0.08	0.08	0.19	2.60	0.00	Clayey SAND (SC), dark brown, moist	--

GRADATION CHARTS

Boring No.: 16E-02
Sample No.: 17
USCS Classification: Clayey SAND (SC)

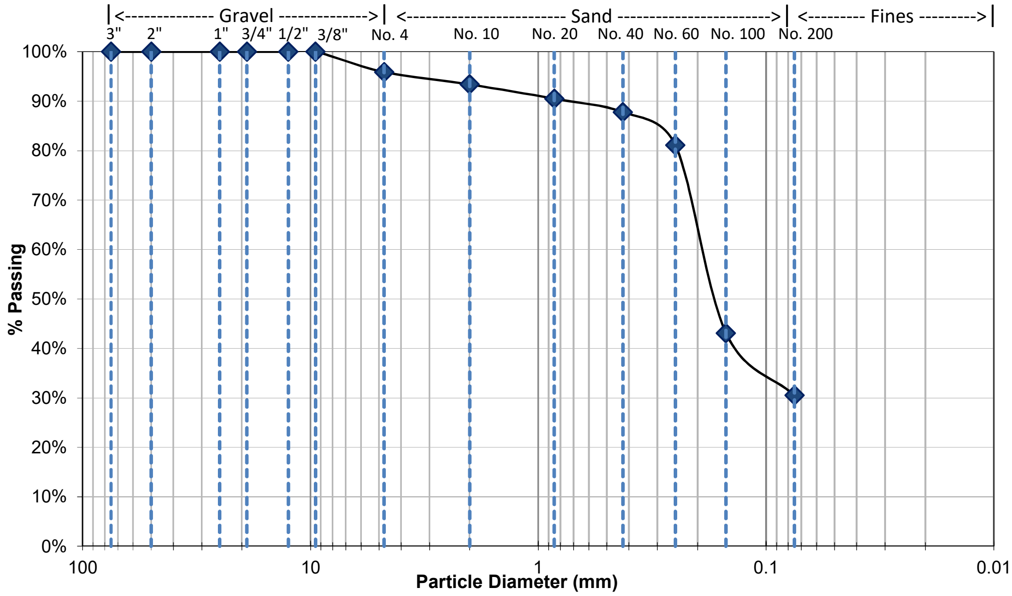


Boring No.: 16E-02
Sample No.: 21
USCS Classification: Poorly graded SAND with clay and gravel (SP-SC)



GRADATION CHARTS

Boring No.: 16E-03
Sample No.: C
USCS Classification: Clayey SAND (SC)



Yeh and Associates, Inc.
Consulting Engineers & Scientists

Mechnical Sieve Analysis with Hydrometer

Test Methods: ASTM D6913, D2487, D4718, D422

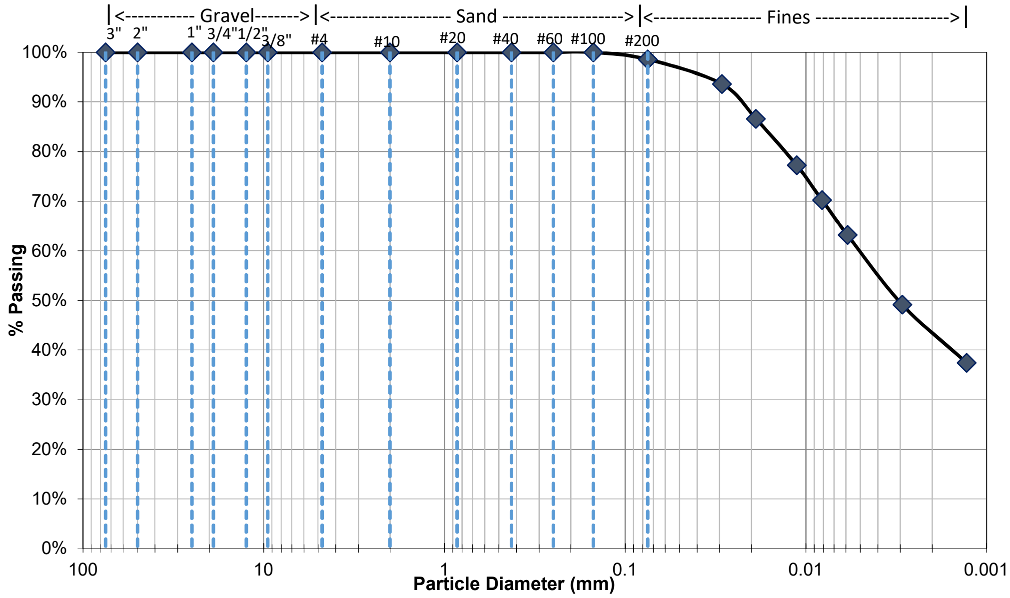
Project Name	SSLOCSD - Redundancy	**Project No.**	216-193
Project Manager	J. King	**Tested By**	J. Cravens
Checked By	J. King	**Testing Date**	6/15/2016

SUMMARY OF RESULTS

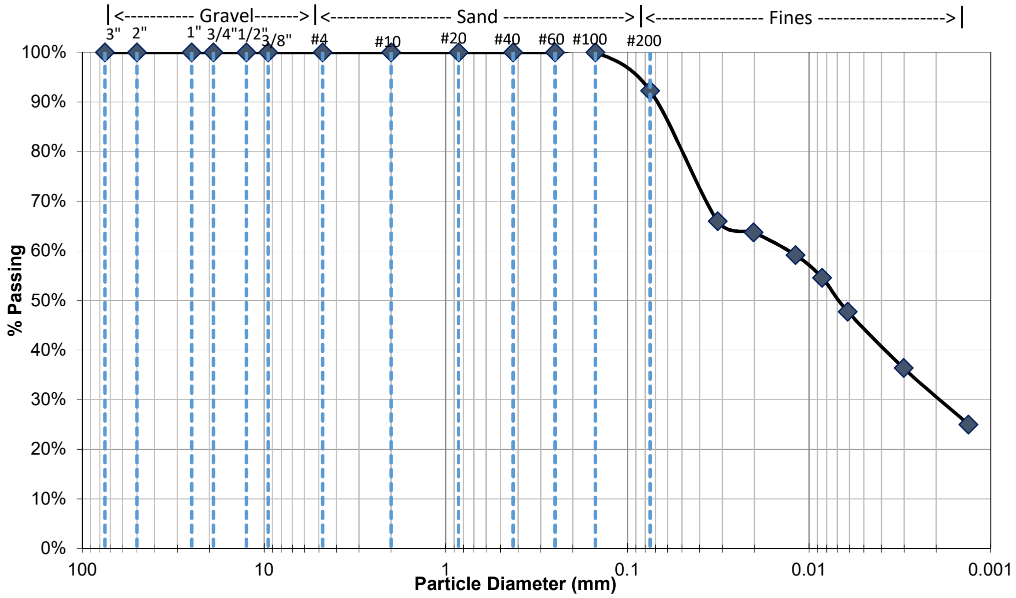
Boring No.	Sample No.	Depth (ft)	Water Content (%)	Gravel (%)	Sand (%)	Fines (%)	D_{10} (mm)	D_{30} (mm)	D_{50} (mm)	D_{60} (mm)	C_u	C_c	USCS Classification & Soil Description	AASHTO Classification
16E-01	10	45	47.0%	0%	1%	99%	0.001	0.001	0.003	0.01	--	--	Fat CLAY (CH) dark gray, moist, with shell fragments	--
16E-02	25	55	42.5%	0%	8%	92%	0.001	0.002	0.007	0.01	--	--	Fat CLAY (CH) dark gray, moist	--

GRADATION CHARTS

Boring No.: 16E-01
Sample No.: 10
USCS Classification: Fat CLAY (CH)



Boring No.: 16E-02
Sample No.: 25
USCS Classification: Fat CLAY (CH)



Percent Passing No. 200 Sieve

Test Method: ASTM D1140, D2487

Project Name	SPTLOCSD - Redundancy			**Project No.**	216-193
Tested By	J. Cravens	**Checked By**	J. King	**Testing Date**	6/10/2016

SPECIMEN ID AND MEASUREMENTS

							Prewash			Washed						
Boring	**Sample**	**Depth (ft)**	**USCS Classification & Soil Description**	**AASHTO Classification**	**Tin ID**	**Tin Mass (g)**	**Tin + Soil (g)**	**Tin + Dry Soil (g)**	**Tin + Dry Soil Minus No. 4**	**Tin + Dry Soil (g)**	**Passing No. 4**	**Passing No. 200**	**% Gravel**	**% Sand**	**% Fines**	**Water Content (%)**
16E-01	3	9	Poorly graded SAND with clay (SP-SC), dark grayish brown, wet	--	103A	134.7	544.4	455.1	455.1	428.7	100%	8%	0%	92%	8%	27.9%
16E-01	4	13.5	Poorly graded SAND (SP), gray, wet	--	107A	134.6	350.0	304.2	304.2	301.3	100%	2%	0%	98%	2%	27.0%
16E-01	10	45	Fat CLAY (CH), dark gray, moist, with shell fragments	--	108A	134.5	440.6	342.7	342.7	137.5	100%	99%	0%	1%	99%	47.0%
16E-02	B	1-8	Clayey SAND (SC), dark brown, moist	--	101R	135.1	383.1	346.8	324.2	262.8	89%	32%	11%	57%	32%	17.1%
16E-02	19	25	Poorly graded SAND with clay (SP-SC), grayish brown, wet	--	107C	134.9	392.7	332.9	330.5	313.7	99%	9%	1%	90%	9%	30.2%
16E-02	25	55	Fat CLAY (CH), dark gray, moist	--	109B	134.5	467.0	367.8	367.8	152.5	100%	92%	0%	8%	92%	42.5%
16E-03	30	1	Clayey SAND (SC), dark brown, moist	--	104A	134.7	348.5	303.5	296.7	250.8	96%	28%	4%	68%	28%	26.7%
16E-03	33	12	Clayey SAND with gravel (SC), gray, wet, with shell fragments	--	102R	134.7	351.7	307.8	280.9	251.2	84%	20%	16%	64%	20%	25.4%
16E-03	34	15	Poorly graded SAND with gravel (SP), brown, wet, with shell fragments	--	106B	134.6	305.9	273.2	218.0	214.9	60%	4%	40%	56%	4%	23.6%
16E-03	36	22	Clayey SAND (SC), dark gray, wet, with shell fragments	--	204A	135.0	377.4	335.1	322.2	297.9	94%	13%	6%	81%	13%	21.1%
16E-03	38	30	Poorly graded SAND with clay and gravel (SP-SC), tan to gray, wet, with shell fragments	--	105B	134.7	445.0	389.4	345.4	323.3	83%	10%	17%	72%	10%	21.8%

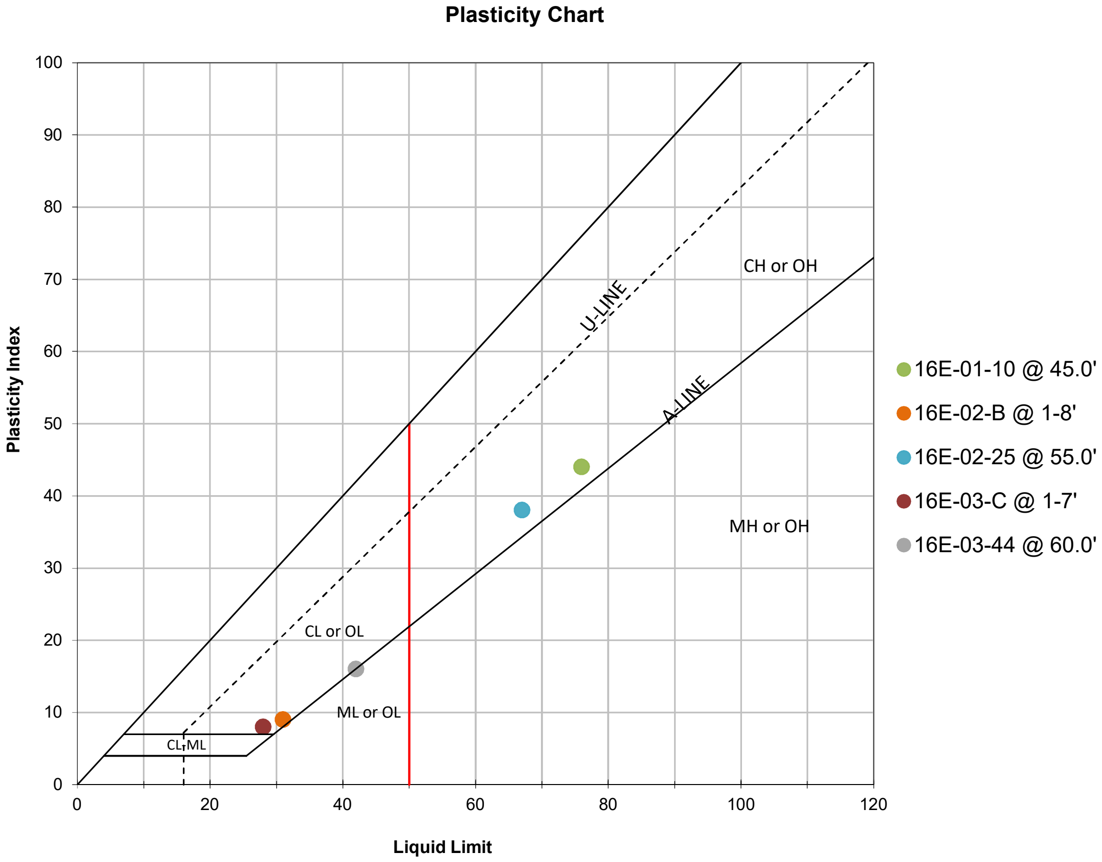
Atterberg Limits

Test Methods: ASTM D4318

Project Name	SPTLOCSD - Redundancy			**Project No.**	216-193
Tested By	J. Cravens	**Checked By**	J. King	**Testing Date**	6/10/2016

SUMMARY OF RESULTS

Boring No.	Sample No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	USCS Classification & Soil Description	AASHTO Classification
16E-01	10	45.0	76	32	44	Fat CLAY (CH), dark gray, moist, with shell fragments	--
16E-02	B	1-8	31	22	9	Clayey SAND (SC), dark brown, moist	--
16E-02	25	55.0	67	29	38	Fat CLAY (CH), dark gray, moist	--
16E-03	C	1-7	28	20	8	CLayey SAND (SC), dark brown, moist, trace gravel	--
16E-03	44	60.0	42	26	16	Lean CLAY with sand (CL), dark gray, moist	--



Proctor Compaction

Test Method: ASTM D698, D1557

Project Name	SSLOCSD - Redundancy			**Project No.**	216-193
Tested By	J. Cravens	**Checked By**	J. King	**Testing Date**	6/10/2016

SPECIMEN ID AND CLASSIFICATION

Boring No.	16E-02	**Sample No.**	B	**Depth (ft)**	1-8
USCS Classification & Soil Description	Clayey SAND (SC), dark brown, moist			**AASHTO Classification**	--

EQUIPMENT AND PROCEDURE

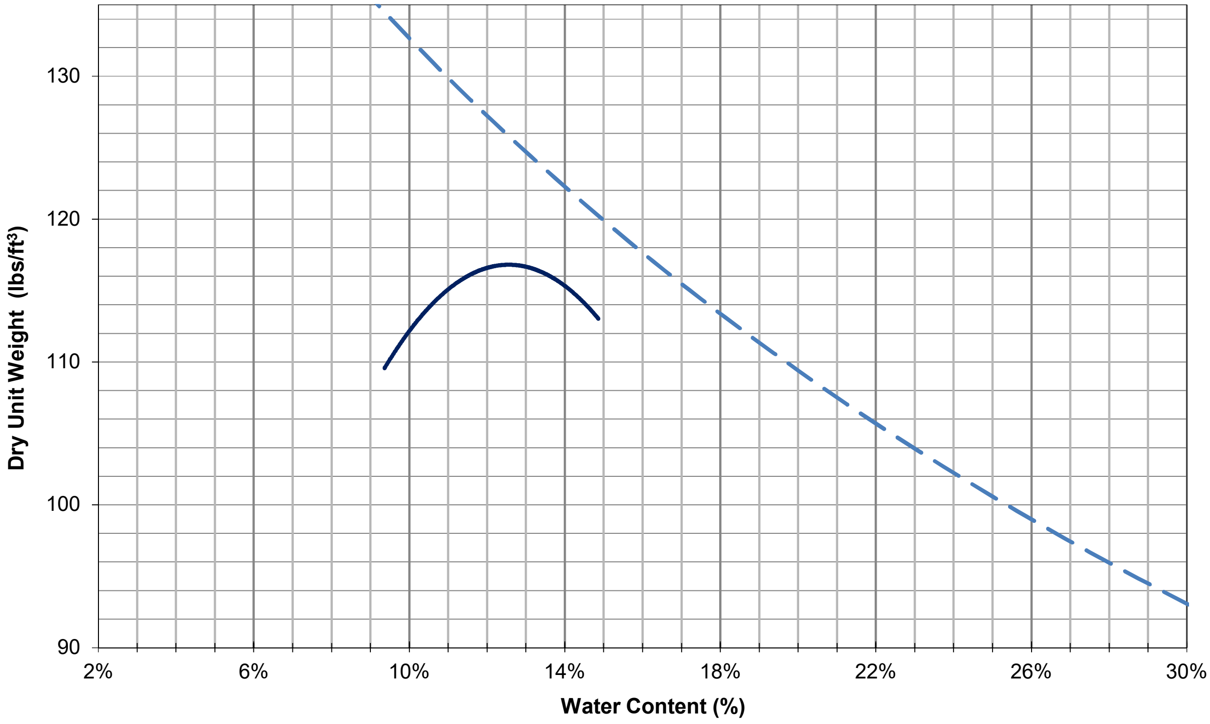
Test Method (D698 or D1557)	1557 B	**Ram. Mass (g)**	10 lb	**# of Lifts**	5
Mold Volume (cm^3)	949	**Mold Mass(g)**	2014	**Blows/ Lift**	25

DENSITY AND MOISTURE MEASUREMENTS

Mass of Soil + Mold (g)	3828.2	3919.3	3957.9	3989.2	
Dish ID	109C	101B	102A	201A	
Mass of Dish (g)	134.6	134.8	134.6	135.0	
Mass of Moist Soil + Dish (g)	550.4	306.1	312.3	567.0	
Mass of Dry Soil + Dish (g)	514.8	290.3	294.4	511.1	

RESULTS

Water Content	9.4%	10.2%	11.2%	14.9%	
Dry Density (Mg/m^3)	1.748	1.823	1.842	1.812	
Dry Unit Weight (lbs/ft^3)	109.1	113.7	114.9	113.1	
Lab Max. Dry Density (Mg/m^3)	1.875		**Optimum Water Content (%)**	12.5%	
Lab Max. Dry Unit Wt. (lbs/ft^3)	117.0				



pH and Resistivity

Test Methods: ASTM D4972, ASTM G187

Project Name	SSLOCSD - Redundancy			**Project Number**	216-193
Tested By	J. Cravens	**Checked By**	J. King		
Testing Date	6/16/2016	**Meter Used**	Miller 400A Analog	**Soil Box Factor**	0.693263

pH & RESISTIVITY MEASUREMENTS

Boring No.	Sample No.	Depth (ft)	USCS Classification & Soil Description	AASHTO Classification	pH in Water	Soil Temperature, T (°C)	Measured Resistance (Ω)	Corrected Resistance (Ω)	Calculated Resistivity (Ω-cm)
16E-01	A	6.5-9	Lean CLAY (CL), dark gray, wet, with shell fragments	--	8.39	22.2	1010	1179	817
16E-01	1	1	Clayey SAND with Gravel (SC), dark brown, moist	--	8.97	22.8	2450	2897	2008
16E-02	15	5	Clayey Sand (SCP), tan, moist	--	8.50	22.6	1400	1649	1143
16E-03	33	12	Clayey SAND with gravel (SC), gray, wet, with shell fragments	--	8.09	22.5	1850	2174	1507

Corrosivity Tests Summary

CTL # 687-017 **Date:** 6/23/2016 **Tested By:** PJ **Checked:** PJ

Client: Yeh and Associates, Inc. **Project:** SSLOCSD **Proj. No:** 216-193

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg	mg/kg	%		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	%	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
16E-01	A	6.5-9.0	-	-	-	117	122	0.0122	-	-	-	-	26.8	Lean CLAY (CL), gray

N|V|5

NV5 WEST, INC.
1868 Palma Drive, Suite A, Ventura, California 93003
Telephone: (805) 656-6074; Fax: (805) 650-6264

NV5 JOB No: **16-001890**
LAB No: 85790

Yeh & Associates, Inc.
391 Front St., Suite D
Grover Beach, CA 93433

Attention: Jonathan Blanchard

Project: Yeh & Associates, Inc. - Miscellaneous Testing
Project No. 216-193

The results of the requested laboratory tests are attached for your use.

This report includes the following test reports:

Test Description	Test Method	# of Tests
Resistance "R" Value Test	CTM 301/ ASTM D2844	1

NV5 WEST appreciates the opportunity to be of service. Please contact our office if you have any questions regarding this report.

Copies: 1-Yeh & Assoc./Jonathan Blanchard
1-File

Respectfully submitted,
NV5 WEST

Shaun Simon
Engineering Manager

NV5

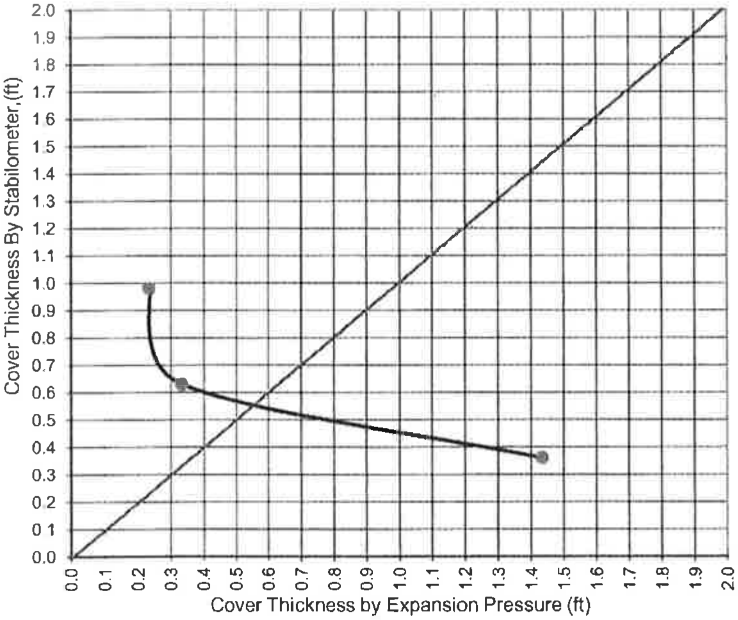
RESISTANCE "R" VALUE TEST

(CTM301 Caltrans / ASTM D2844)

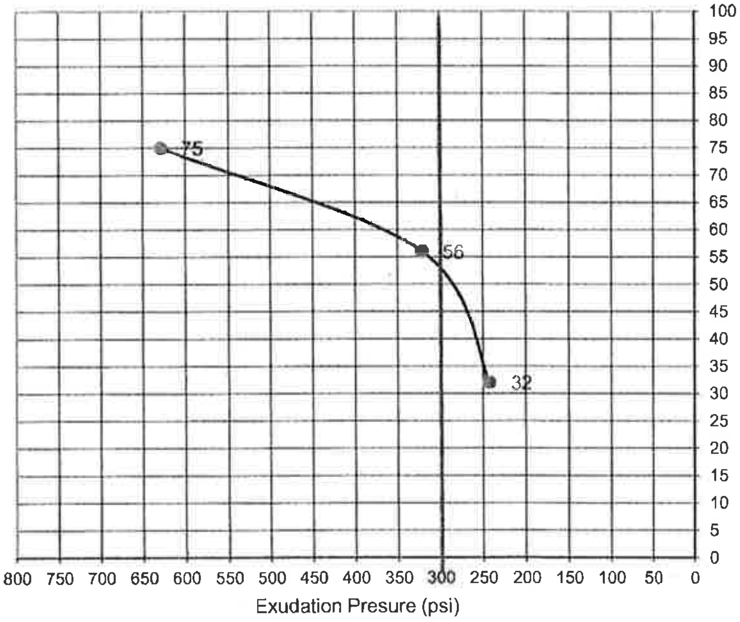
Date: 6/29/2016
Client: Yeh & Associates Inc.
Address: 391 Front Street, Suite D
Grover Beach, CA
Project : 216-193 SPTLOCSD
Project Address : NR

Job Number: 16-01890
Report Number: 4437
Lab Number: 113042

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART

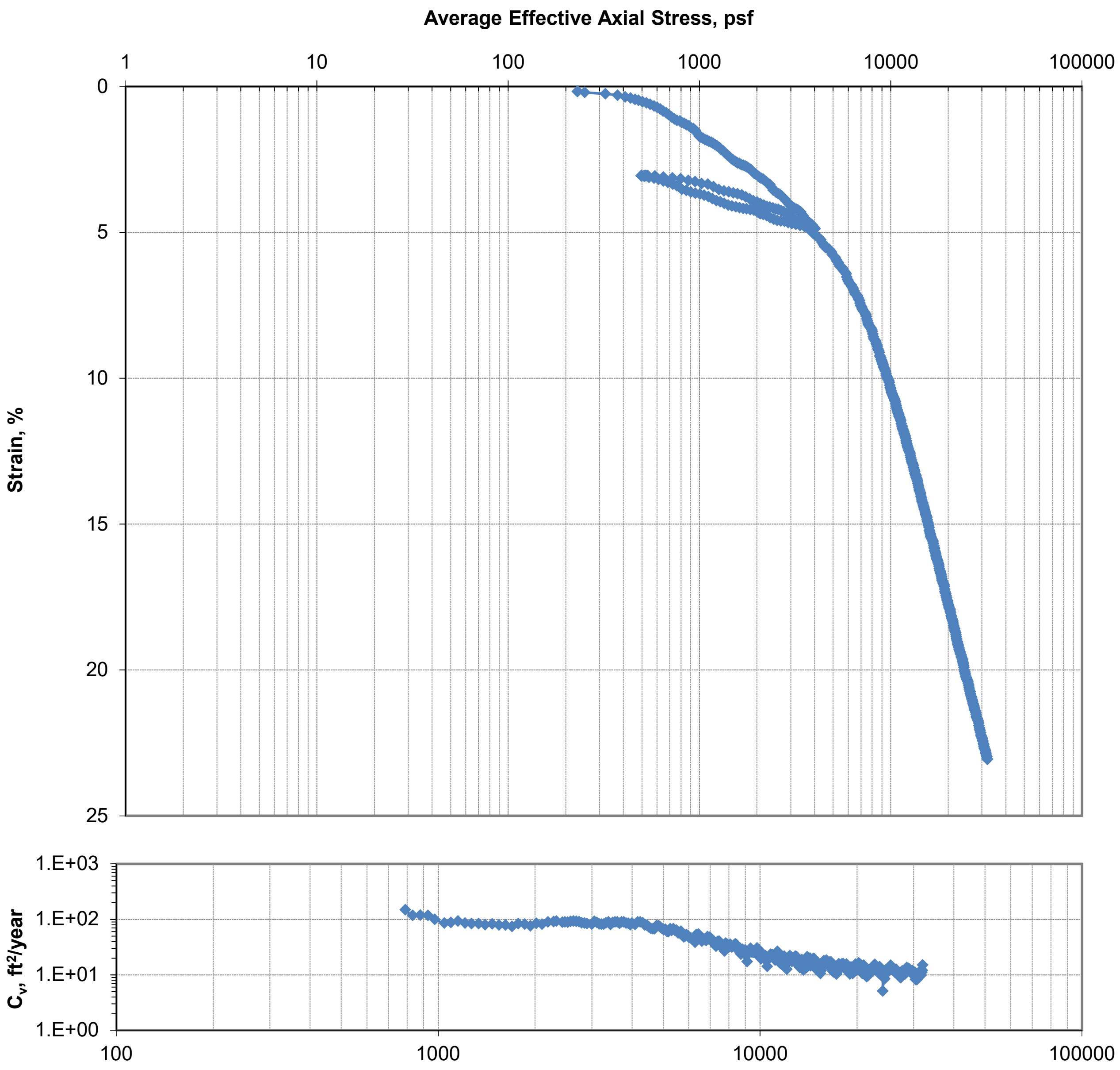


TEST SPECIMEN	A	B	C	D
COMP. FOOT PRESSURE, psi	350	350	300	
INITIAL MOISTURE %	2.8	2.8	2.8	
MOISTURE @ COMPACTION %	4.5	5.4	6.3	
DRY DENSITY, pcf	124.8	123.4	109.7	
EXUDATION PRESSURE, psi	629	322	243	
STABILOMETER VALUE 'R'	75	56	32	

R-VALUE BY EXUDATION	51
R-VALUE BY EXPANSION	63
R-VALUE AT EQUILIBRIUM	**51**

Respectfully Submitted,
NV5 West, Inc.

Sam Koohi, PE
Engineering Manager



SAMPLE ID	Boring, Sample #, Depth	16E-01 , #9 , 43.7 ft
	USCS Classification:	Fat CLAY (CH): gray, wet

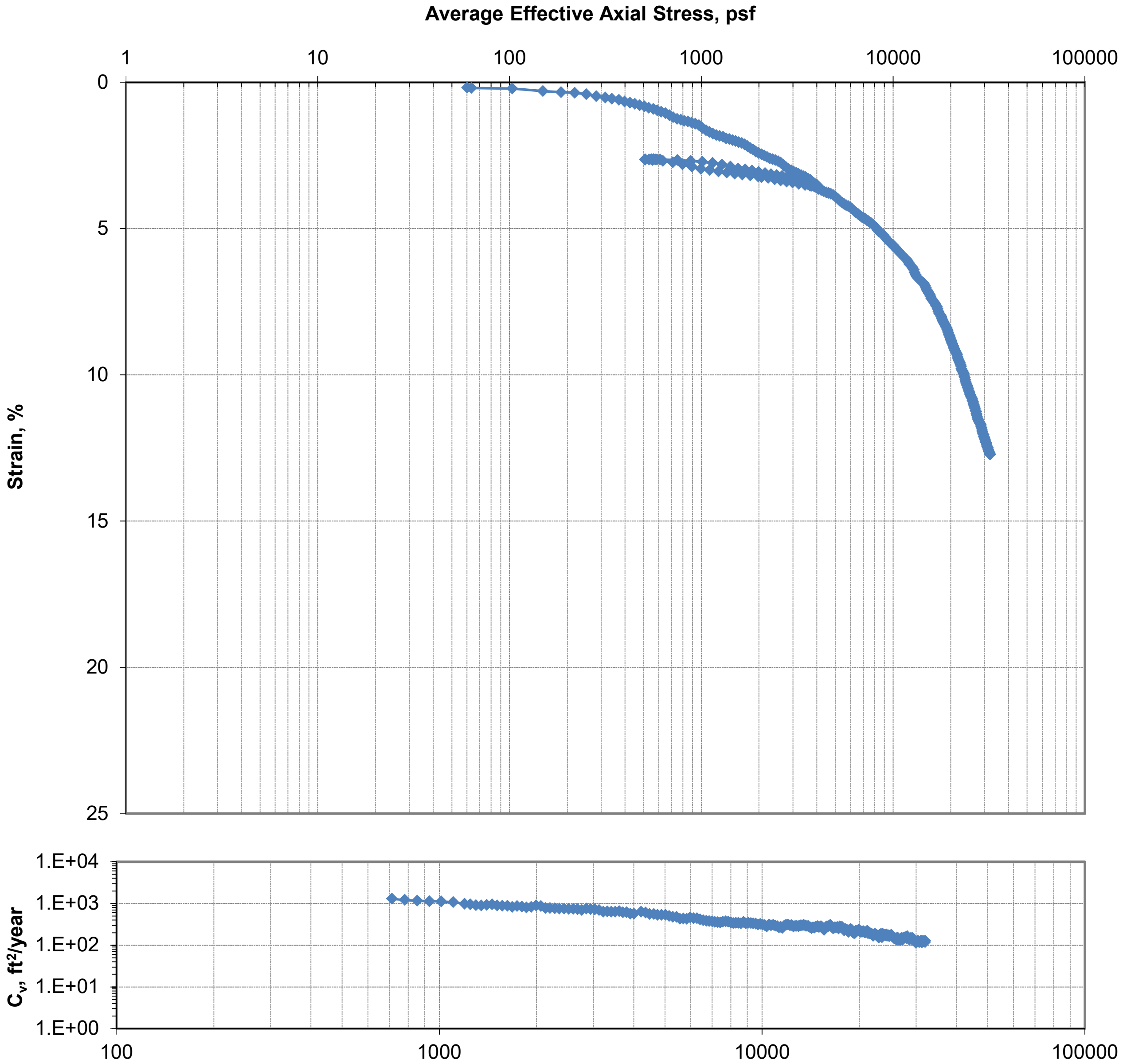
PROPERTIES		Initial	Final
	Water Content, %	42.9%	30.7%
	Dry Unit Weight, pcf	76.6	92.0
	Saturation, %	97%	100%
	Void Ratio	1.20	0.83
	Diameter, in	2.42	2.42
	Height, in	0.74	0.62

SUMMARY		
	Strain Rate, %/hr	2.0
	Liquid Limit, %	---
	Plastic Limit, %	---
	Plasticity Index, %	---
	Passing #200	---
	Estimated Gs	2.7

REMARKS

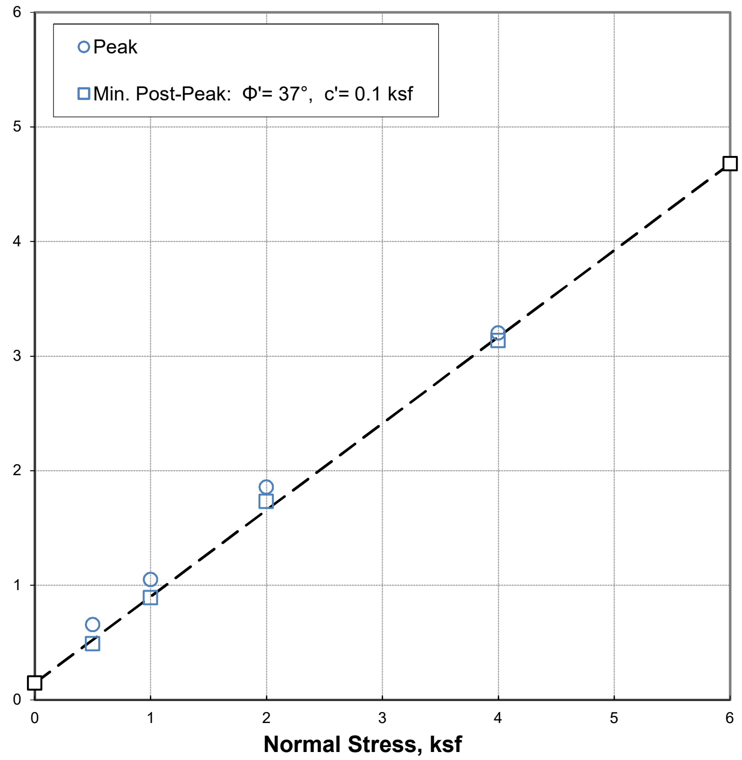
Test Method: D4186
Test by: N. Derbidge, CalPoly EGEO Lab
Checked by: J. King, Yeh & Associates

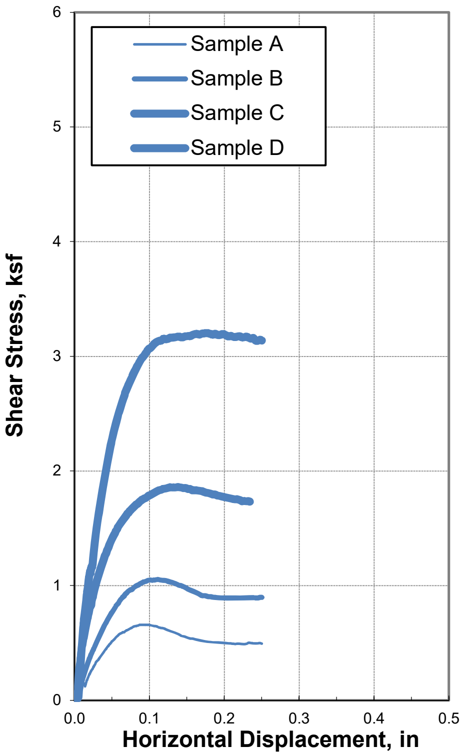
CONSOLIDATION TEST

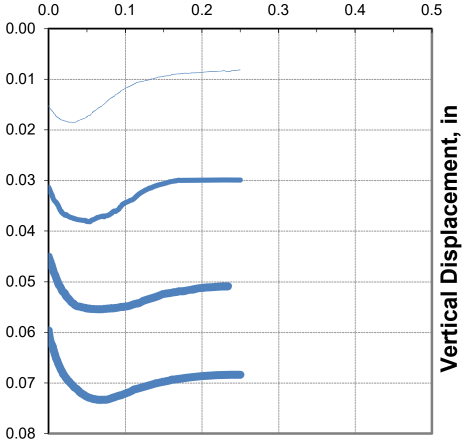


SAMPLE ID	Boring, Sample #, Depth	16E-02 , #27 , 61.0 ft		**SUMMARY**	Strain Rate, %/hr 2.0
	USCS Classification:	Lean CLAY (CL): gray, wet, with fine sand			Liquid Limit, % ---
PROPERTIES		Initial	Final		Plastic Limit, % ---
	Water Content, %	36.7%	32.7%		Plasticity Index, % ---
	Dry Unit Weight, pcf	82.9	89.5		Passing #200 ---
	Saturation, %	96%	100%		Estimated Gs 2.7
	Void Ratio	1.03	0.88	**REMARKS**	Test Method: D4186
	Diameter, in	2.42	2.42		Test by: N. Derbidge, CalPoly GEOE Lab
	Height, in	0.74	0.69		Checked by: J. King, Yeh & Associates

CONSOLIDATION TEST



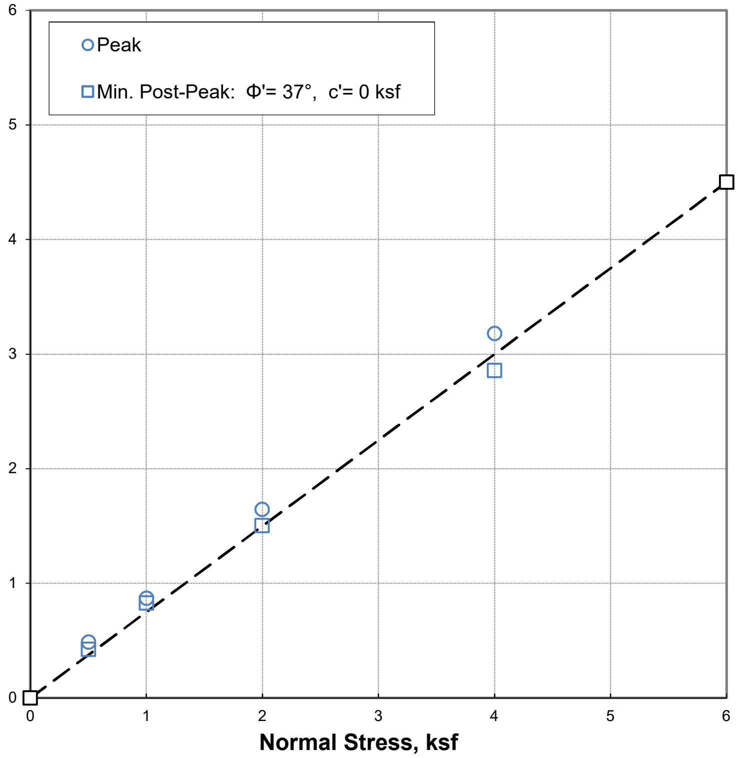


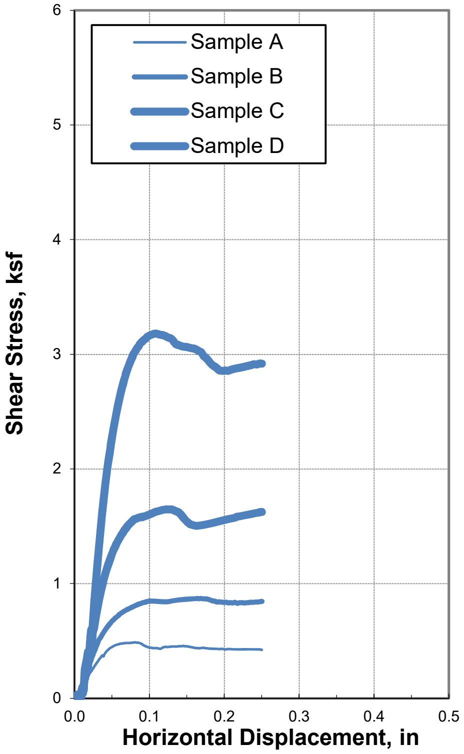


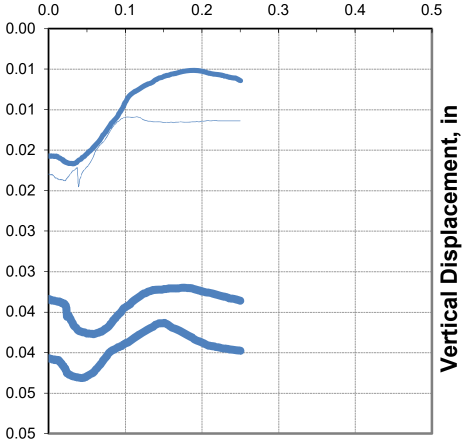
SAMPLE ID	Boring Number:	16E-02			
	Sample Number:	B			
	Sample Depth:	1-8 ft			
	USCS Classification:	Clayey SAND (SC): brown, moist			
INITIAL	Specimen	A	B	C	D
	Water Content, %	18.5%	18.5%	18.5%	18.5%
	Dry Unit Weight, pcf	109.2	108.9	108.7	110.0
	Saturation, %	89%	88%	88%	91%
	Void Ratio	0.57	0.58	0.58	0.56
	Diameter, in	2.42	2.42	2.42	2.42
	Height, in	1.00	1.00	1.00	1.00
FINAL	Water Content, %	19.9%	19.1%	18.7%	18.5%
	Dry Unit Weight, pcf	109.5	110.8	110.6	112.0
	Void Ratio	0.57	0.55	0.55	0.53
TEST SUMMARY	Displacement at Peak, in	0.09	0.11	0.14	0.18
	Displacement Rate, in/min	0.002	0.002	0.002	0.002
	Normal Stress, ksf	0.5	1.0	2.0	4.0
	Peak Shear Stress, ksf	0.66	1.05	1.86	3.20
	Min. Post-Peak Stress, ksf	0.49	0.89	1.73	3.14
REMARKS	Test Method: ASTM D3080				
	Specimens were remolded to 90% Relative Compaction				
	Tested by N. Derbidge, CalPoly GEOE lab				
	Checked by J. King, Yeh and Associates				

CLASSIFICATION	Sieve Size	% Passing
	3/8-in. (9.5mm)	---
	#4 (4.75mm)	---
	#16 (1.18mm)	---
	#30 (0.6mm)	---
	#100 (0.150mm)	---
	#200 (0.075mm)	---
	Atterberg Limits	
	Liquid Limit, %	---
	Plastic Limit, %	---
	Plasticity Index, %	---
	Estimated Gs	2.75
	k_{avg} 20ºC, cm/sec	---

DIRECT SHEAR TEST







SAMPLE ID		
	Boring Number:	16E-03
	Sample Number:	31
	Sample Depth:	5.0 ft
	USCS Classification:	Pootly-graded SAND with Clay (SP-SC): gray, wet, fine sand

	Specimen	A	B	C	D
INITIAL	Water Content, %	29.2%	29.2%	29.2%	29.2%
	Dry Unit Weight, pcf	90.7	90.0	89.3	91.8
	Saturation, %	94%	92%	91%	96%
	Void Ratio	0.82	0.84	0.85	0.80
	Diameter, in	2.42	2.42	2.42	2.42
	Height, in	1.00	1.00	1.00	1.00
FINAL	Water Content, %	35.9%	32.2%	34.4%	33.3%
	Dry Unit Weight, pcf	84.6	88.2	86.2	87.3
	Void Ratio	0.95	0.87	0.92	0.89
TEST SUMMARY	Displacement at Peak, in	0.08	0.16	0.13	0.11
	Displacement Rate, in/min	0.015	0.015	0.015	0.015
	Normal Stress, ksf	0.5	1.0	2.0	4.0
	Peak Shear Stress, ksf	0.49	0.87	1.65	3.18
	Min. Post-Peak Stress, ksf	0.42	0.83	1.50	2.86

REMARKS

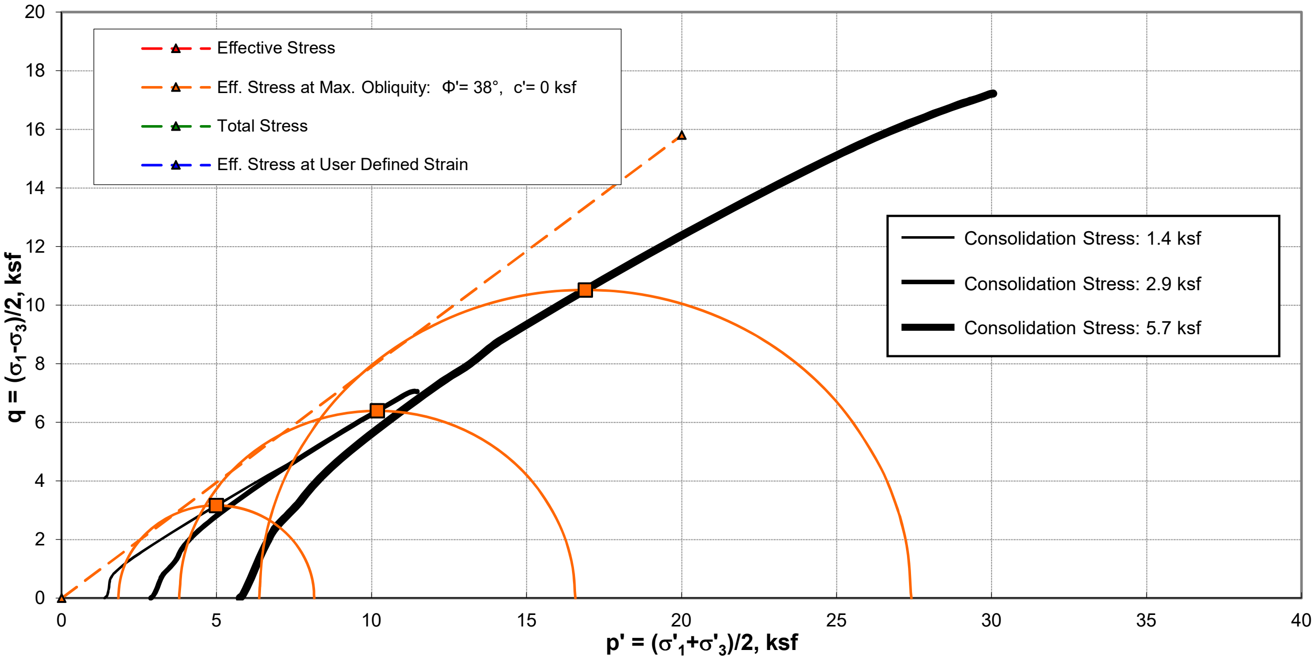
Test Method: ASTM D3080
Tested by: N. Derbidge, CalPoly GEOE Lab
Checked by: J. King, Yeh and Associates

CLASSIFICATION

Sieve Size	% Passing
3/8-in. (9.5mm)	---
#4 (4.75mm)	---
#16 (1.18mm)	---
#30 (0.6mm)	---
#100 (0.150mm)	---
#200 (0.075mm)	---

Atterberg Limits	
Liquid Limit, %	---
Plastic Limit, %	---
Plasticity Index, %	---
Estimated Gs	2.65
k_{avg} 20ºC, cm/sec	---

DIRECT SHEAR TEST



SAMPLE ID	Boring Number	16E-01
	Sample Number	6
	Specimen Depth	23.5 ft
	USCS Classification	Poorly-graded SAND (SP): gray, wet, fine sand

	Trial ID	A	B	C
INITIAL	Water Content, %	24.9%	24.9%	24.9%
	Dry Unit Weight, pcf	99.5	100.7	100.8
	Saturation, %	97%	100%	100%
	Void Ratio	0.69	0.67	0.67
	Diameter, in	2.42	2.47	2.47
	Height, in	5.00	4.76	4.72
PRE-SHEAR	Water Content, %	24.9%	24.9%	24.8%
	Dry Unit Weight, pcf	100.7	100.8	101.0
	Saturation, %	100%	100%	100%
	Void Ratio	0.67	0.67	0.67

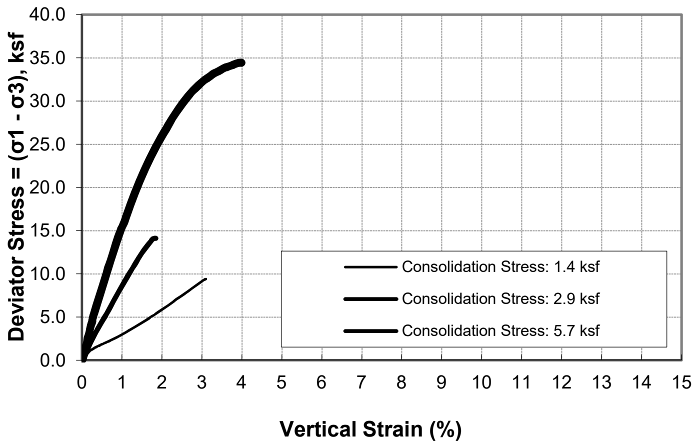
REMARKS

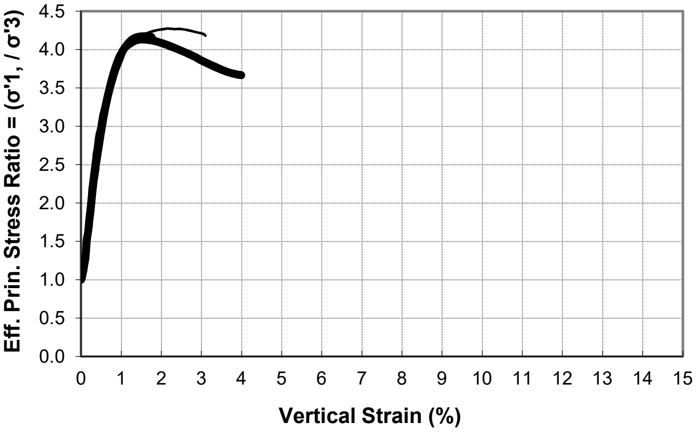
Test Method: ASTM 4767 (modified for staged testing)
Tested by: N. Derbidge, CalPoly GEOE lab
Checked by: J. King, Yeh and Associates

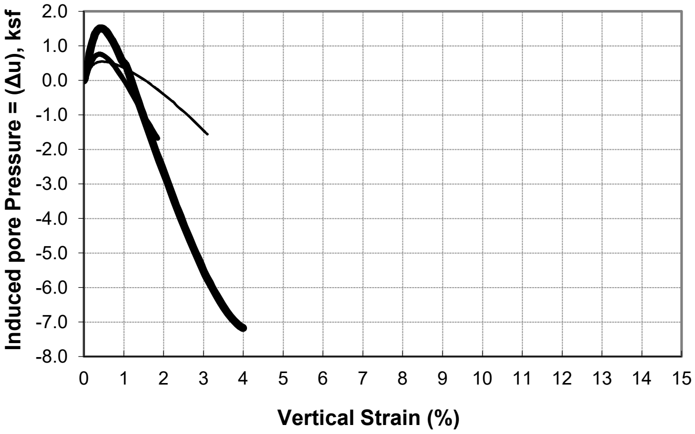
	Trial ID	A	B	C
CLASSIFICATION	Liquid Limit	---	---	---
	Plastic Limit	---	---	---
	Plastic Index	---	---	---
	Passing #4 (4.75 mm)	---	---	---
	Passing #200 (0.075 mm)	---	---	---
	Estimated Gs	2.70	2.70	2.70

	Trial ID	A	B	C
TEST SUMMARY	B-Parameter	0.98	0.98	0.98
	t_{50}, minutes	N/A	N/A	N/A
	Strain Rate, %/min	0.33	0.33	0.33
	Cell Pressure, ksf	7.2	8.7	11.5
	Back Pressure, ksf	5.8	5.8	5.8
	Consolidation Stress, ksf	1.4	2.9	5.7
	Deviator Stress $^{@}$ Failure, ksf	6.2	12.6	20.8
	Axial Strain $^{@}$ Failure, %	2.1	1.6	1.5
	σ'_{1F}, ksf	8.1	16.4	27.1
	σ'_{3F}, ksf	1.8	3.8	6.4
	Tested By:	ND	ND	ND
	Date Tested:	6/14/16	6/15/16	6/19/16

CONSOLIDATED UNDRAINED TRIAXIAL TEST

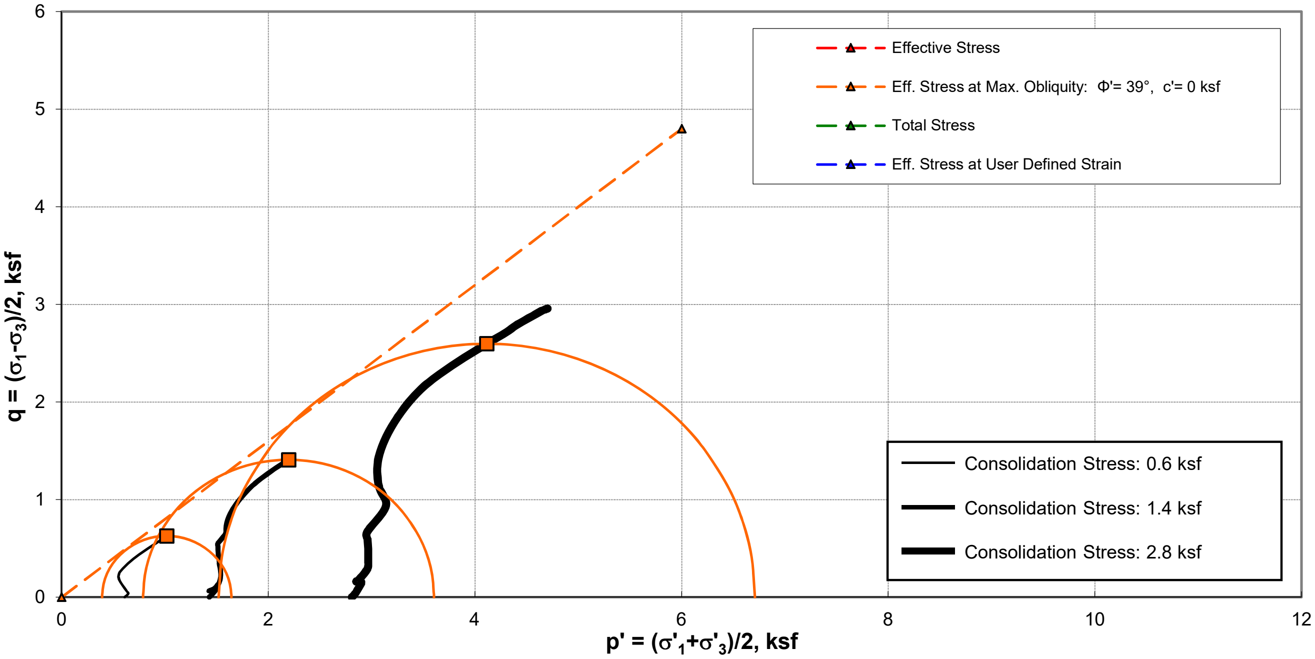






A	16E-01	#6	23.5 ft	Poorly-graded SAND (SP): gray, wet, fine sand
B	16E-01	#6	23.5 ft	Poorly-graded SAND (SP): gray, wet, fine sand
C	16E-01	#6	23.5 ft	Poorly-graded SAND (SP): gray, wet, fine sand

CONSOLIDATED UNDRAINED TRIAXIAL TEST



SAMPLE ID

Boring Number	16E-02
Sample Number	16
Specimen Depth	10.7 ft
USCS Classification	Poorly-graded SAND (SP): gray, wet, fine sand

INITIAL

Trial ID	A	B	C
Water Content, %	37.0%	36.4%	35.4%
Dry Unit Weight, pcf	81.9	85.0	86.2
Saturation, %	95%	100%	100%
Void Ratio	1.06	0.98	0.95
Diameter, in	2.88	2.87	2.88
Height, in	5.76	5.59	5.47

PRE-SHEAR

Water Content, %	36.4%	35.4%	34.1%
Dry Unit Weight, pcf	85.0	86.2	87.7
Saturation, %	100%	100%	100%
Void Ratio	0.98	0.95	0.92

REMARKS

Test Method: ASTM 4767 (modified for staged testing)
Tested by: N. Derbidge, CalPoly GEOE Lab
Checked by: J. King, Yeh and Associates

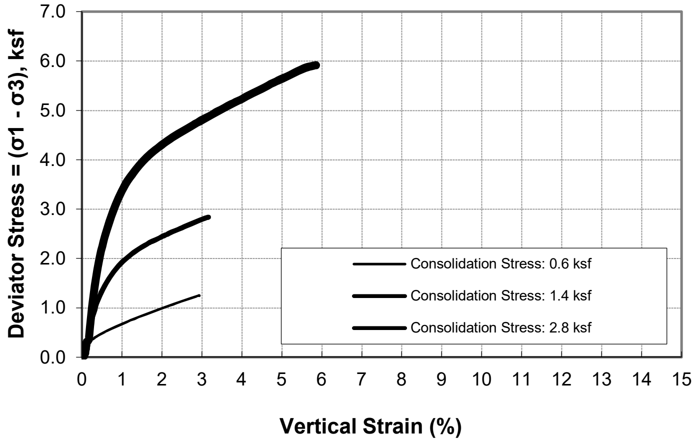
CLASSIFICATION

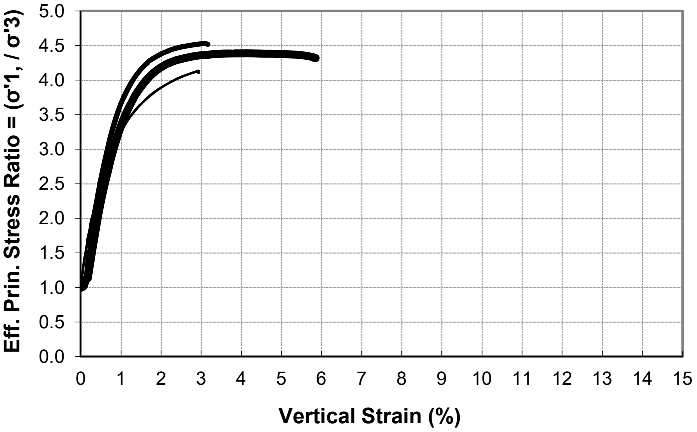
Trial ID	A	B	C
Liquid Limit	---	---	---
Plastic Limit	---	---	---
Plastic Index	---	---	---
Passing #4 (4.75 mm)	---	---	---
Passing #200 (0.075 mm)	---	---	---
Estimated Gs	2.70	2.70	2.70

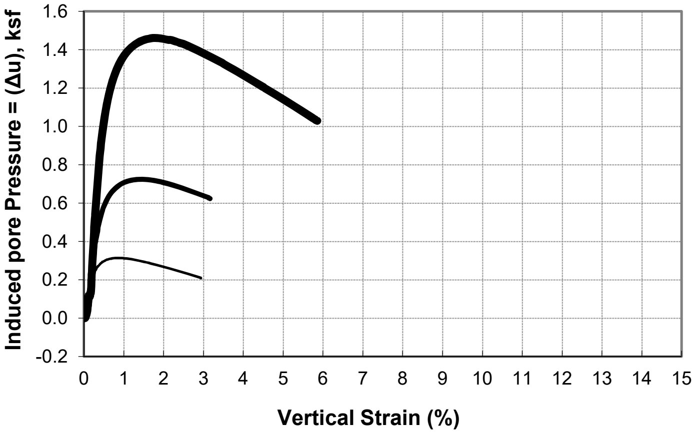
TEST SUMMARY

Trial ID	A	B	C
B-Parameter	0.98	0.98	0.98
t_{50}, minutes	N/A	N/A	N/A
Strain Rate, %/min	0.34	0.34	0.34
Cell Pressure, ksf	7.9	8.7	10.1
Back Pressure, ksf	7.3	7.3	7.3
Consolidation Stress, ksf	0.6	1.4	2.8
Deviator Stress @ Failure, ksf	1.2	2.8	5.1
Axial Strain @ Failure, %	2.9	3.1	3.9
σ'_{1F}, ksf	1.6	3.6	6.6
σ'_{3F}, ksf	0.4	0.8	1.5
Tested By:	ND	ND	ND
Date Tested:	6/14/16	6/15/16	6/19/16

CONSOLIDATED UNDRAINED TRIAXIAL TEST

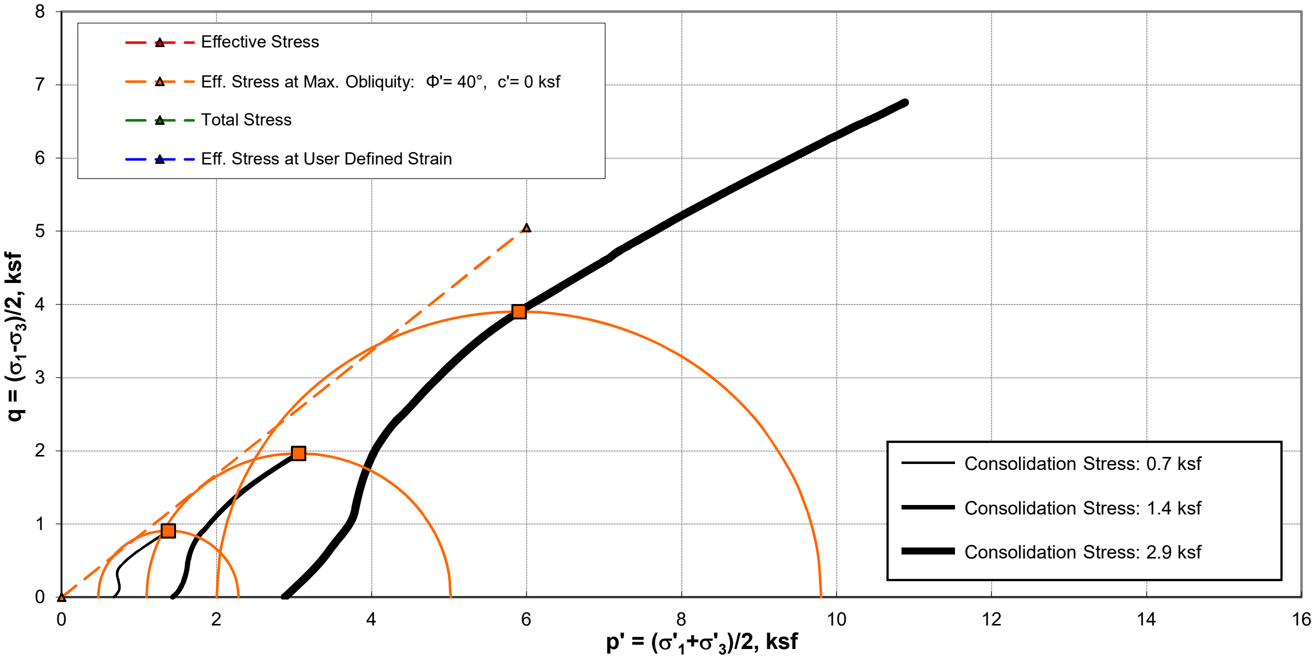






A	16E-02	#16	10.7 ft	Poorly-graded SAND (SP): gray, wet, fine sand
B	16E-02	#16	10.7 ft	Poorly-graded SAND (SP): gray, wet, fine sand
C	16E-02	#16	10.7 ft	Poorly-graded SAND (SP): gray, wet, fine sand

CONSOLIDATED UNDRAINED TRIAXIAL TEST



SAMPLE ID

Boring Number	16E-03
Sample Number	32
Specimen Depth	9.0 ft
USCS Classification	Poorly-graded SAND with Clay (SP-SC): gray, wet, with clay parting

CLASSIFICATION

Trial ID	A	B	C
Liquid Limit	---	---	---
Plastic Limit	---	---	---
Plastic Index	---	---	---
Passing #4 (4.75 mm)	---	---	---
Passing #200 (0.075 mm)	---	---	---
Estimated Gs	2.70	2.70	2.70

INITIAL

Trial ID	A	B	C
Water Content, %	35.8%	34.7%	33.8%
Dry Unit Weight, pcf	83.8	87.0	88.1
Saturation, %	96%	100%	100%
Void Ratio	1.01	0.94	0.91
Diameter, in	2.42	2.40	2.42
Height, in	5.00	4.88	4.76

PRE-SHEAR

Water Content, %	34.7%	33.8%	32.7%
Dry Unit Weight, pcf	87.0	88.1	89.5
Saturation, %	100%	100%	100%
Void Ratio	0.94	0.91	0.88

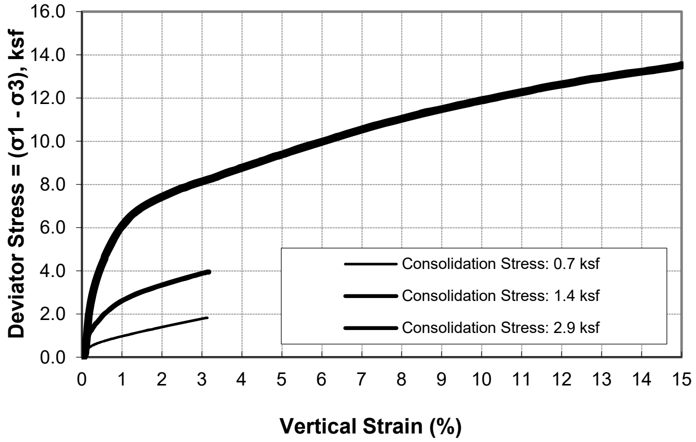
TEST SUMMARY

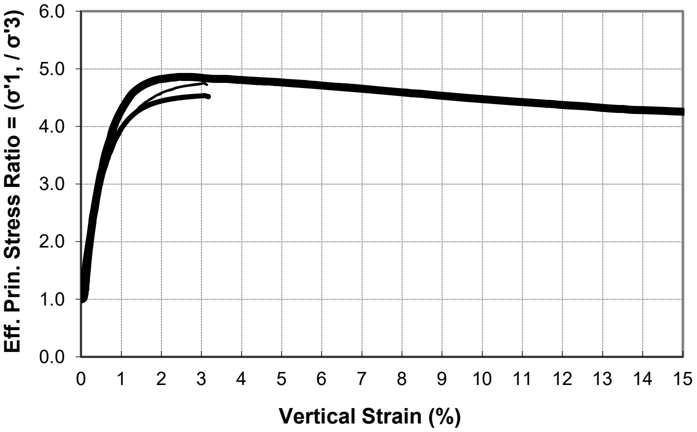
Trial ID	A	B	C
B-Parameter	0.99	0.99	0.99
t_{50}, minutes	N/A	N/A	N/A
Strain Rate, %/min	0.33	0.33	0.33
Cell Pressure, ksf	7.5	8.3	9.7
Back Pressure, ksf	6.9	6.9	6.8
Consolidation Stress, ksf	0.7	1.4	2.9
Deviator Stress @ Failure, ksf	1.8	3.9	7.7
Axial Strain @ Failure, %	3.1	3.1	2.5
σ'_{1F}, ksf	2.3	5.0	9.7
σ'_{3F}, ksf	0.5	1.1	2.0
Tested By:	ND	ND	ND
Date Tested:	6/14/16	6/15/16	6/19/16

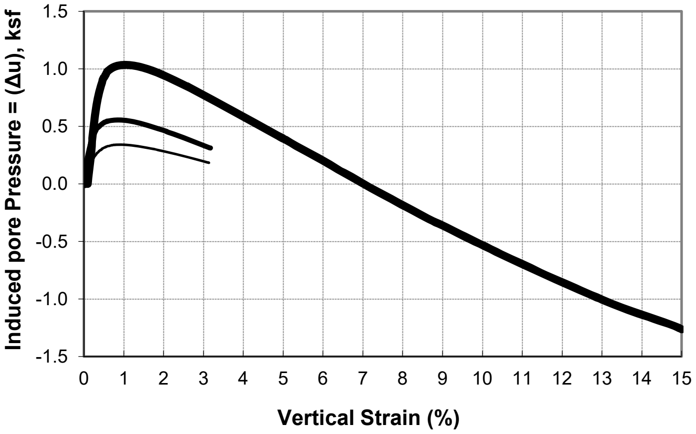
REMARKS

Test Method: ASTM 4767 (modified for staged testing)
Tested by: N. Derbidge, CalPoly GEOE Lab
Checked by: J. King, Yeh and Associates

CONSOLIDATED UNDRAINED TRIAXIAL TEST







A	16E-03	#32	9.0 ft	Poorly-graded SAND with Clay (SP-SC): gray, wet, with clay parting
B	16E-03	#32	9.0 ft	Poorly-graded SAND with Clay (SP-SC): gray, wet, with clay parting
C	16E-03	#32	9.0 ft	Poorly-graded SAND with Clay (SP-SC): gray, wet, with clay parting

CONSOLIDATED UNDRAINED TRIAXIAL TEST

APPENDIX C - BORING LOGS FROM COOPER-CLARK & ASSOCIATES (1979) AND SUBSURFACE CONSULTANTS, INC. (1984)

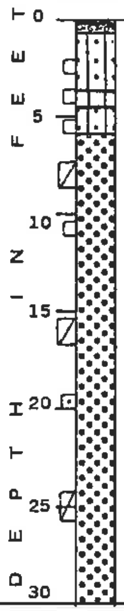
Revisions:
By______ Date______
By______ Date______
Location South San Luis Obispo County, CA

By BSK Date 2/16/78
Checked By BK 2/16/78
Job Number 2003-A Name South San Luis Obispo

BORING 1

ELEVATION 11.1 FEET

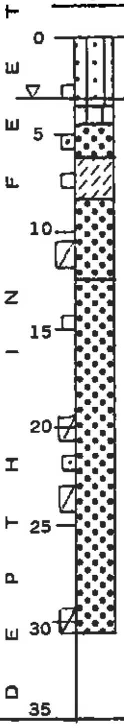
CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		
% FINES (-NO 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT	Blows/Ft.
			DS	1500	Natural	2150	11.4	109	19
							13.4	102	21
			DS	600	Natural	550	27.5	97	9
			DS	2500	Natural	1750	26.4	90	
									10
			DS	1200	Natural	900	22.9	97	
			DS	3000	Natural	2350	20.8	100	7
									12
									20
									27



1" asphaltic concrete on 5" sandy base
Brown silty sand (SM) (dense)
(grading gray)
Gray clayey silt (ML) (stiff)
Gray silty sand (SM) with rock fragments; trace of wood (medium dense)
FILL
Gray fine sand (SP) (medium dense)
(trace of organic)
(some small gravel)
(caving soil)
(increase in gravel; few broken shells)
(grading more dense)
(few thin bay mud lenses)
(grading more dense)

BORING 2

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT	Blows/Ft.
			DS	300	Natural	400	18.9	96	
			DS	2500	Natural	1900	23.8	90	14
									11
	74.7	39.5	DS	600	Natural	240	82.2	50	0
2.3									11
			DS	2500	Natural	2200	24.4	97	14
									3
									16
									26
									61



Brown silty sand (SM) (dense)
Water level 1/24/78
Gray clayey silt (ML) (moderately firm)
Gray sand (SP) (medium dense)
Gray organic clayey silt (OH) (soft bay mud)
Brown fine sand (SP) (medium dense)
Gray fine sand (SP) (medium dense)
(some broken shells and gravel)
(soft bay mud lens)
(grading dense)
(occasional thin gravelly layers)
(increase in gravel)

FIELD NOTES

1. The borings were drilled on January 24 and 25, 1978 with truck-mounted, 5-inch-diameter, rotary-wash equipment.
2. The following symbol, ☐, denotes an undisturbed sample taken in a 2½-inch-diameter, split-tube barrel, driven into the soil by 300-pound slip jars falling 18± inches inside the boring.
3. The following symbol, N ☒, denotes a standard penetration test. The number recorded for N is the penetration resistance: blows required to drive a standard 2-inch-diameter sampler for 12 inches (from 6 to 18 inches below the bottom of the boring) with a 140-pound hammer free-falling 30 inches.
4. The following symbol, ▣, denotes an attempted undisturbed sample with no recovery or with the sample partially disturbed.
5. Boring elevations were estimated by interpolation between available spot elevations.

LABORATORY NOTES AND ABBREVIATIONS

The tabulated shear strengths are yield point values.

DS = Strain controlled direct shear test at natural moisture content.

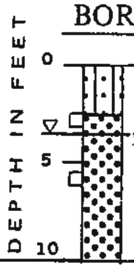
BORING LOGS

COOPER · CLARK & ASSOCIATES

PLATE 2-B

BORING 3

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT	Blows/Ft.
			DS	1000	Natural	1400	28.4	93	12
			DS DS	600 2500	Natural Natural	1250 3000	24.8 21.0	99 103	8



Brown silty sand (SM) (dense)

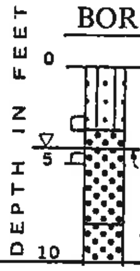
Brown sand (SP) (dense)
Water level 1/24/78

(some gravel)
(grading more coarse; caving soil)

(grading gray)

BORING 4

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT	Blows/Ft.
							52.2	71	8
			DS	1500	Natural	1300	21.9	100	7



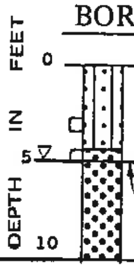
Brown silty sand (SM) (dense)

(thin clayey lenses)
Brown sand (SP) (medium dense)
Water level 1/24/78
(caving soil)

Gray sand (SP) with some gravel (medium dense)

BORING 5

CLASSIFICATION DATA			STRENGTH DATA				MOISTURE-DENSITY DATA		
% FINES (-NO. 200)	LIQUID LIMIT	PLASTICITY INDEX	TYPE OF STRENGTH TEST	TEST SURCHARGE PRESSURE, LBS/SQ FT	TEST MOISTURE CONTENT, %	SHEAR STRENGTH, LBS/SQ FT	NATURAL MOISTURE CONTENT, %	DRY DENSITY, LBS/CU FT	Blows/Ft.
			DS DS DS	400 2500 1500	Natural Natural Natural	650 2500 1550	17.5 17.9 22.6	100 103 96	19
									10



Brown silty sand (SM) (dense)

(grading gray)

Gray sand (SP) with some gravel (medium dense)
Water level 1/24/78

(grading less dense; mixed with some bay mud)

BORING LOGS

Revisions:
By ____ Date ____
By ____ Date ____
Location South San Luis Obispo County, CA
By BSK Date 2/16/78
Checked By FK 2/16/78
Job Number 2003-A Name South San Luis Obispo

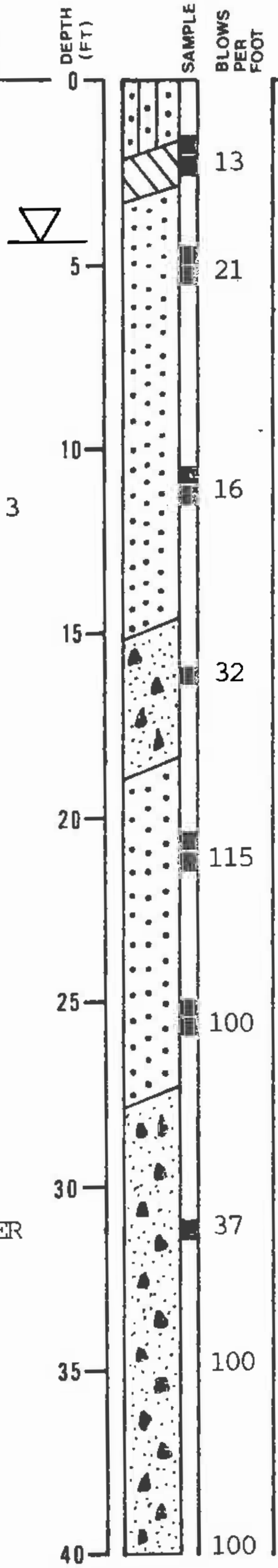
LOG OF TEST BORING 1

EQUIPMENT 8" ROTARY WASH

DATE DRILLED 5/8/84

ELEVATION *

LABORATORY TESTS | MOISTURE CONTENT % | DRY DENSITY (PCF) | DEPTH (FT) | SAMPLE | BLOWS PER FOOT



BROWN SILTY SAND (SM)
loose, moist

GRAY SANDY CLAY (CL)
soft to medium stiff, moist

GROUNDWATER LEVEL 5/8/84

GRAY SAND (SP)
loose, saturated, contains gravel to 3/4 inch

SIEVE ANALYSIS: SEE PLATE 3

significant increase in gravel content, contains numerous shell fragments below 9 feet

SIEVE ANALYSIS: SEE PLATE 3

GRAY SANDY GRAVEL (GW)
medium dense, saturated

GRAY SAND (SP)
dense, saturated, fine grained

numerous shell fragments below 24 feet

GRAY SANDY GRAVEL (GW)
medium dense, saturated

becomes dense

* TOP OF BORING IS 8 FEET BELOW RIM OF ADJACENT CLARIFIER

SAMPLER OD: 3.0 INCHES
SAMPLER ID: 2.5 INCHES

HAMMER WEIGHT: 325 LBS.
HAMMER DROP: 18 INCHES

Subsurface Consultants

TREATMENT PLANT -- SAN LUIS OBISPO CO.

JOB NUMBER	DATE	APPROVED	PLATE
103.003	6/6/84	JB	2

APPENDIX D - HISTORIC AERIAL PHOTOGRAPHS

Southern San Luis Obispo County WWTP
1600 Aloha
Oceano, CA 93445

Inquiry Number: 4496267.1
December 22, 2015

The EDR Aerial Photo Decade Package

6 Armstrong Road, 4th Floor
Shelton, Connecticut 06484
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EDR Aerial Photo Decade Package

Environmental Data Resources, Inc. (EDR) Aerial Photo Decade Package is a screening tool designed to assist environmental professionals in evaluating potential liability on a target property resulting from past activities. EDR's professional researchers provide digitally reproduced historical aerial photographs, and when available, provide one photo per decade.

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Date EDR Searched Historical Sources:

Aerial Photography December 22, 2015

Target Property:

1600 Aloha
Oceano, CA 93445

Year	***Scale***	***Details***	***Source***
1939	Aerial Photograph. Scale: 1"=500'	Flight Year: 1939	USGS
1949	Aerial Photograph. Scale: 1"=500'	Flight Year: 1949	USGS
1956	Aerial Photograph. Scale: 1"=500'	Flight Year: 1956	Hycon
1966	Aerial Photograph. Scale: 1"=500'	Flight Year: 1966	Mark Hurd
1969	Aerial Photograph. Scale: 1"=500'	Flight Year: 1969	Western
1972	Aerial Photograph. Scale: 1"=500'	Flight Year: 1972	Mark Hurd
1978	Aerial Photograph. Scale: 1"=500'	Flight Year: 1978	Pacific Air
1981	Aerial Photograph. Scale: 1"=500'	Flight Year: 1981	USGS
1989	Aerial Photograph. Scale: 1"=500'	Flight Year: 1989	USGS
1994	Aerial Photograph. Scale: 1"=500'	/DOQQ - acquisition dates: 1994	USGS/DOQQ
2005	Aerial Photograph. Scale: 1"=500'	Flight Year: 2005	USDA/NAIP
2009	Aerial Photograph. Scale: 1"=500'	Flight Year: 2009	USDA/NAIP
2010	Aerial Photograph. Scale: 1"=500'	Flight Year: 2010	USDA/NAIP
2012	Aerial Photograph. Scale: 1"=500'	Flight Year: 2012	USDA/NAIP

INQUIRY #: 4496267.1

YEAR: 1939

= 500'

N

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INQUIRY #: 4496267.1

YEAR: 1949

= 500'

N

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INQUIRY #: 4496267.1

YEAR: 1956

= 500'

N

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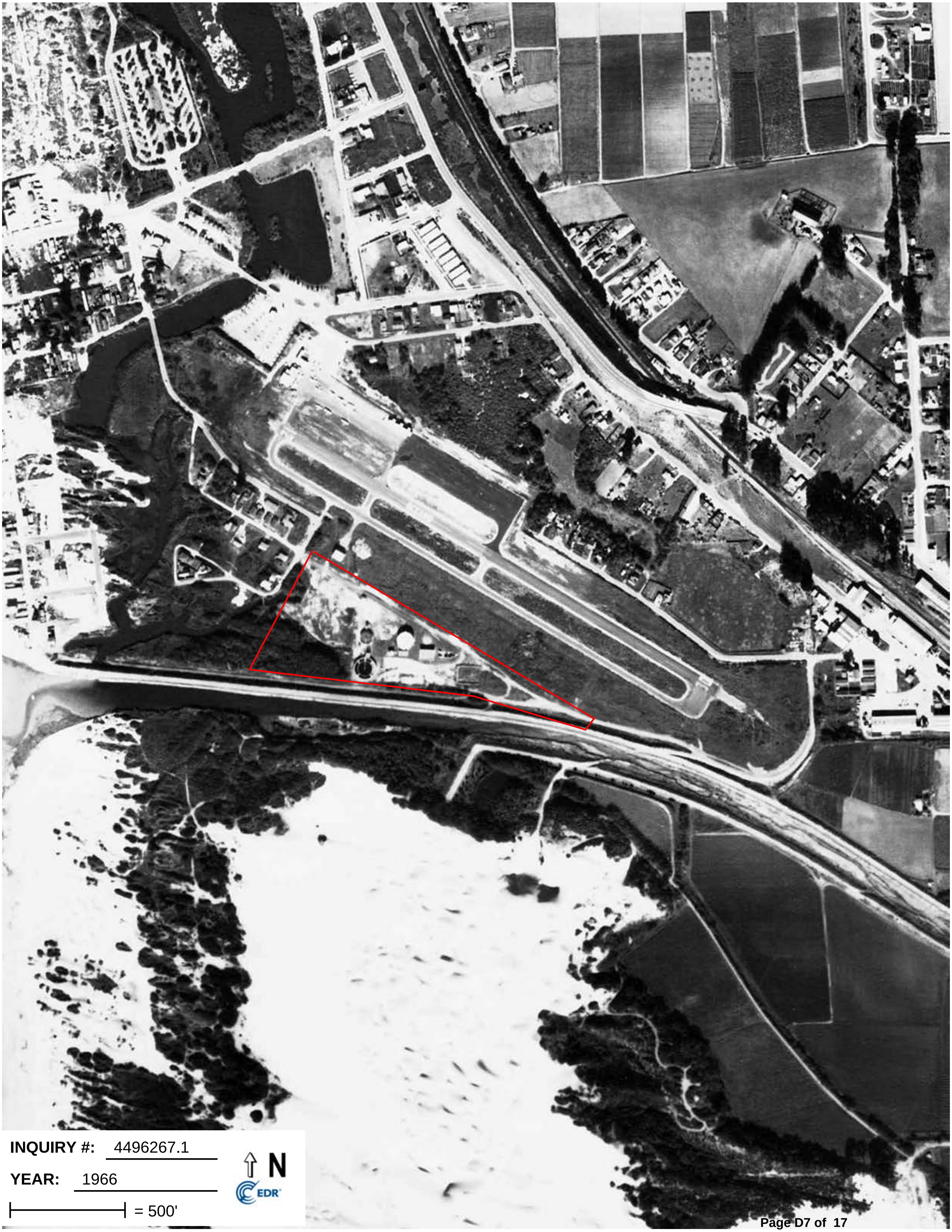


INQUIRY #: 4496267.1

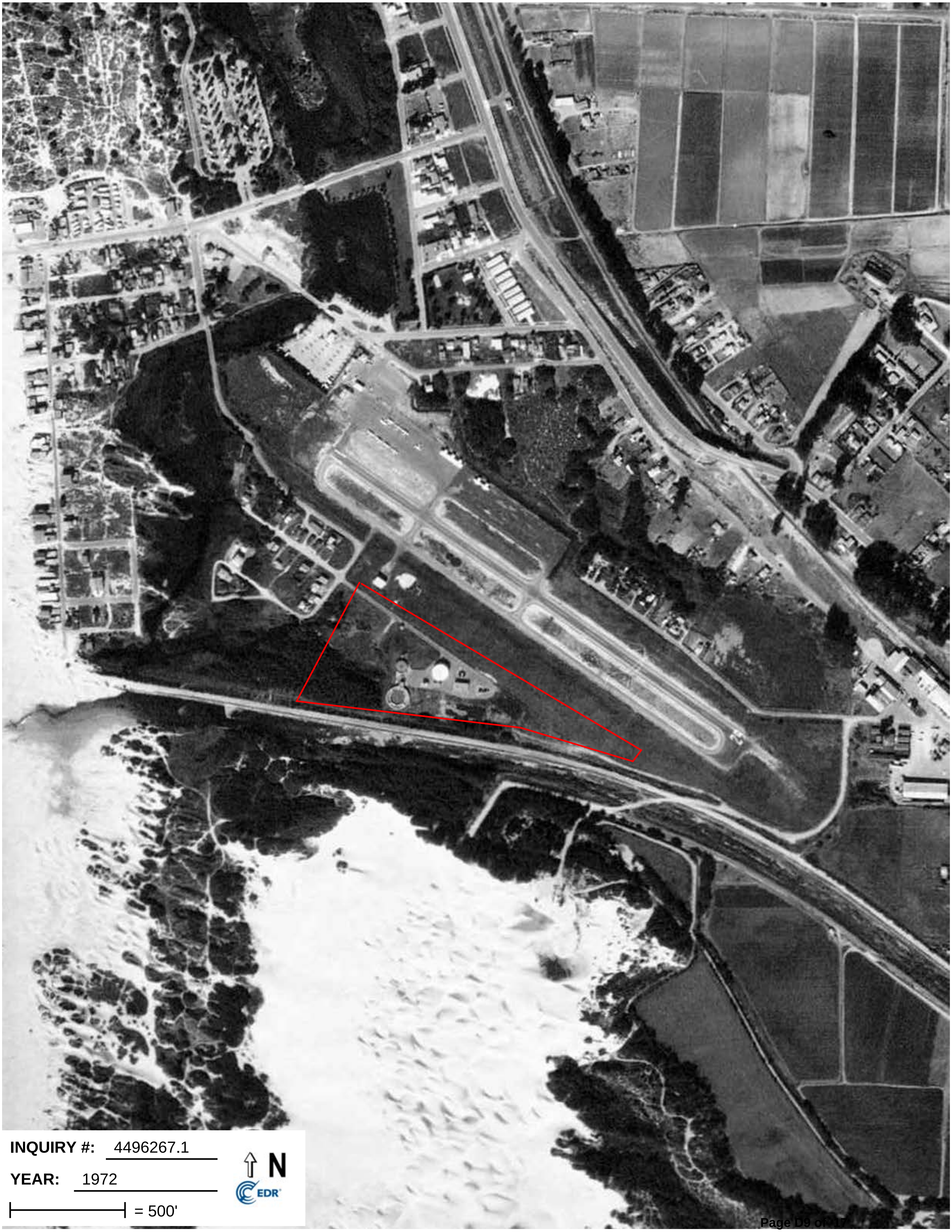
YEAR: 1966

= 500'

INQUIRY #: 4496267.1

YEAR: 1969

= 500'



INQUIRY #: 4496267.1

YEAR: 1972

= 500'

INQUIRY #: 4496267.1

YEAR: 1978

= 500'

N

EDR

INQUIRY #: 4496267.1

YEAR: 1981

= 500'

N

EDR

INQUIRY #: 4496267.1
YEAR: 1989
= 500'
N
EDR

INQUIRY #: 4496267.1

YEAR: 1994

= 500'

INQUIRY #: 4496267.1

YEAR: 2005

= 500'

N

EDR

INQUIRY #: 4496267.1

YEAR: 2009

= 500'

INQUIRY #: 4496267.1

YEAR: 2010

= 500'

N

EDR

INQUIRY #: 4496267.1

YEAR: 2012

= 500'

N

EDR