Adventure Park
Response to GMED Review
Addendum No.1

TO:	COMPANY:	DATE:
Jennifer Walker **Lindsay Maldonado** **Aric Torreyson**	Watearth Watearth Tetra Tech	January 18, 2021
FROM:	PROJECT NUMBER:	PHONE NUMBER:
Fernando Cuenca, Ph.D., G.E.	TET 20-179E	(909) 860-7777

RE:

Adventure Park
Response to GMED Review
Whittier, California

References: *Geotechnical Investigation Report. Adventure Park. Stormwater Underground Storage Facilities. 10130 Gunn Avenue. South Whittier, California. Prepared by Tetra Tech and **dated November 2, 2020**.*

*100% Design Plan Check Review. Adventure Park Stormwater Project. GMED Review 0000289. County of Los Angeles. Reviewed by Karen Mendez. **Dated December 15, 2020**.*

INTRODUCTION

GMED has performed a review dated December 15, 2020 of the Geotechnical Report referenced above. Two comments refer to this report and they are addressed in this addendum. **Section 9.1.1** regarding lateral earth pressures on underground walls has been modified as indicated in this document and supersede this section in the original report.

GMED COMMENTS

For ease of reference GMED comments regarding the report are included below:

Comment 1. *Revise liquefaction analysis input parameters provided in Tetra Tech's November 2, 2020 report, a weighted PGA and magnitude of 7.5 or design PGA and design magnitude should be used. Revise as necessary.*

Tetra Tech contacted the County reviewer regarding this comment. The County reviewer (Karen Mendez) in phone conversation with Tetra Tech (Fernando Cuenca) that took place on January 18, 2021 indicated that this comment was incorrect and did not apply to our report. Therefore, there is no need to address it.

Comment 2. *Verify whether the seismic seismic earth pressure for walls over 6 feet in height were obtained per Administrative manual County of Los Angeles Department of Public Works*

Geotechnical and Materials Engineering Division memo for Seismic Earth Pressures on Retaining Walls. Revise recommended seismic earth pressure in Tetra Tech's November 2, 2020 report, as needed.

The seismic earth pressure increments had been originally evaluated the methodology established by Mikola and Sitar (2013) for cohesionless backfill which is typically used behind most earth retaining structures with selected backfill materials. To comply with the County request these seismic pressure increments have been reevaluated using the procedures recommended by the reviewer and published by the County of Los Angeles in publication S004.0 dated January 6, 2020 using the pressures for cohesive backfill which is conservative. Section 9.1.1. of the report has been modified accordingly. For ease of reference the modified values of the seismic pressure increments have been highlighted in the revised Table 12 provided herein.

Revised Section 9.1.1. Lateral Earth Pressures on Underground Walls and Storage Facility

Based on the 2020 for the County of Los Angeles Building Code, which is based on the 2019 California Building Code (CBC) the design of retaining walls higher than 6 feet, as measured from the top of the footing, requires the inclusion of not only static lateral pressures but also of additional seismically induced lateral earth pressures.

The static lateral pressures acting on the proposed on-site underground structures storage and infiltration structures should be calculated based on the recommendations provided in Table 12.

According to the 2019 CBC the dynamic seismic lateral earth pressures on foundation walls and retaining walls should be determined using the design earthquake ground motions. Based on the USGS U.S. Seismic Design Maps website application (http://earthquake.usgs.gov/designmaps/us/application.php), the PGA from the Design Response Spectrum at the site is approximately 0.47g where the design PGA is calculated as $0.4*S_{DS,}$ where S_{DS} is the risk-targeted, maximum rotated acceleration direction, design response spectrum parameter for short periods. The seismic induced earth pressure increments were estimated using the method recommended by Agusti and Sitar (2013) and as required by the County of Los Angeles publication S004 dated January 6, 2020 regarding Seismic Earth Pressures on Retaining Walls. These recommendations are provided in Table 12. Lateral earth pressures presented in this table are for a level backfill.

Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls. In walls with stiff clay backfill that are free to rotate at least 0.01 radians (deflection at the top of the wall of at least 0.01 x H) may be designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) located within a 1(H):1(V) plane drawn upward from the heel of the wall footing should be added to the lateral earth pressures.

Suitable backfill materials within a zone immediately the behind the subterranean walls, including the underground storage facility walls, should have a Sand Equivalent of about 30, an Expansion Index of less than 20, and fines content (passing #200 sieve) of less than 15 percent. It is expected that due to the clayey nature of most of the on-site material, the on-site materials will not be generally suitable as a backfill immediately behind. Consequently, a select on-site or import material with an Expansion Index less than 20 or approved non-expansive import material should be used for the backfill within at least 5 feet behind the back of the underground walls. It is expected that additional laboratory testing will be necessary to determine the suitability of the selected on-site or import materials. The select on-site or import materials that are approved as backfill materials should be moisture-conditioned 110 percent of the optimum moisture content, and placed in horizontal lifts not more than 8 inches in uncompacted thickness, and compacted to at least 90 percent of the maximum dry density, as evaluated by the latest version of ASTM D1557.

Table 12
Geotechnical Design Parameters for Subterranean Walls
Lateral Pressures due to Static and Seismic Loads

Active Pressure for Yielding Walls		
Static *active* pressure (psf)	above groundwater	$51z + 0.42Q$
	below groundwater (at depth $z > z_w$)	$51z_w + 89(z - z_w) + 0.42Q$
Active seismic pressure increment (psf)		$25z$
At rest Pressure for Non-yielding Walls		
Static *at-rest* pressure (psf)	above groundwater	$71z + 0.59Q$
	below groundwater (at depth $z > z_w$)	$71z_w + 99(z - z_w) + 0.59Q$
At-rest seismic pressure increment (psf)		$40z$
Lateral Passive Resistance		
Allowable static lateral *passive* pressure (psf) Includes a Factor of Safety of 2	above groundwater	$140z_1$
	below groundwater at depth z_w	$140z_w + 74(z_1 - z_w)$
Ultimate total passive resistance for seismic conditions (psf)	above groundwater	$145\ z_1$
	below groundwater at depth z_w	$145z_w + 75(z_1 - z_w)$

Notes:
- Lateral Pressures due to Seismic Loading are based on a PGA=0.47g for a design response spectrum taken as 2/3 MCER response spectrum. The appropriate total seismic force (active plus seismic increment for yielding walls and at rest plus seismic increment for non-yielding walls) should be calculated be assuming a downward increasing triangular equivalent fluid pressure distribution. The resulting force should be assumed to act at 0.4 of the height of the wall above the bottom of the wall.
- Pressure based on soil with $\phi = 24^{o}$, c = 0 psf, $\gamma_t = 120$ pcf (above groundwater), $\gamma_t = 125$ pcf (below groundwater)
- The 2019 CBC requires that basement walls be designed for at rest earth pressures for static conditions.

Legend:
z … Depth (ft) below the grade behind the wall – depth measured from the ground surface to the depth where the soil lateral pressure is being evaluated;
z_1 … Depth (ft) below the grade where passive conditions apply, i.e., usually in front of the wall – depth measured from the ground surface to the depth where the soil lateral pressure is being evaluated;
z_w … Depth to groundwater (ft) – depth measured from the ground surface to the groundwater;
Q … Uniform surcharge (psf) within a 1(H):1(V) plane drawn upward from the heel of the wall footing

CLOSURE

We appreciate the opportunity to assist Watearth and the County of Los Angeles with this GMED review of the Adventure Park Stormwater Capture project. If you have any questions, please feel free to contact our office at (909) 860-7777.

Respectfully submitted,

Tetra Tech

Fernando A. Cuenca M

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Filename: 2021-01-18 Addendum No.1.docx

Geotechnical Engineering ● Engineering Geology

Geotechnical Investigation Report

ADVENTURE PARK
STORMWATER UNDERGROUND STORAGE FACILITIES
10130 Gunn Avenue
South Whittier, California

Prepared for: Watearth, Inc.
3371 Glendale Blvd, Suite 208
Los Angeles CA 90039

Prepared by: Tetra Tech
21700 Copley Drive, Suite 200
Diamond Bar, California 91765

Rev.1 November 2, 2020
(Supersedes April 29, 2020 Report)
Project No. TET 20-179E

TETRA TECH

Project No. TET 20-179E
November 2, 2020
(Supersedes April 29, 2020 Report)

Ms. Jennifer Lundberg
Watearth, Inc.
3371 Glendale Blvd, Suite 208
Los Angeles CA 90039

Subject: **GEOTECHNICAL INVESTIGATION REPORT Rev.1**
ADVENTURE PARK
STORMWATER UNDERGROUND STORAGE FACILITIES
10130 Gunn Avenue
South Whittier, California

Dear Ms. Lundberg:

Presented herein is Tetra Tech's geotechnical investigation report for the proposed stormwater underground storage facilities at Adventure Park located at 10130 Gunn Avenue, in the City of Whittier, California. This report summarizes the results of our geotechnical investigation to characterize the soils at the site and provides recommendations for the geotechnical design and construction of the proposed facilities including the stormwater underground storage facility, diversion structures, pumping structures, pretreatment units, pipelines, and temporary shoring. The appendices of the report include logs of borings from previous investigations, Cone Penetration Testing (CPT) logs from previous and current investigations, agronomic testing results, analytical testing results, soil laboratory tests, seismic demand, and liquefaction analyses.

We appreciate the opportunity to provide our professional services on this project. If you have any questions regarding this report or if we can be of further service, please do not hesitate to contact the undersigned.

Respectfully submitted,
Tetra Tech

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Filename: 2020-04-29 Adventure Geotechnical Report RPT.docx

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1. INTRODUCTION

This report presents the results of Tetra Tech's geotechnical engineering evaluation for the proposed stormwater capture, underground storage, and conveyance facilities at the Adventure Park Project (see Figure 1 – Project Location Map) located at 10130 Gunn Avenue, just outside of the City of Whittier in unincorporated South Whittier, California.

Adventure Park is a 15.5-acre parcel owned by the County of Los Angeles. The park includes ballfields, playground equipment, splashpad, picnic areas, multiple parking lots, and several building structures including a community center. The park is divided into 2 areas by the Sorensen Drain, a 34-foot wide by 13-foot high concrete channel, which traverses the site from the northwest to the southeast. The proposed project footprint and auxiliary components will be located entirely on the area northeast of Sorensen Drain with no disruption to the area to the southwest (see Figure 2 – Site Layout, Boring and CPT Location Map). The proposed facilities include:

- a 19.6 acre-foot (6.4 MG) underground storage facility at a depth between 25 and 30 feet;
- a diversion structure from Sorensen Drain;
- a pump wet well at a depth of about 28 feet;
- a pre-treatment unit at a depth of about 26 feet; and
- conveyance pipelines.

The proposed diversion structure within the channel at Sorensen Drain will consist of an inflatable rubber dam to impound runoff. The diversion structure will convey water from the channel via gravity along a 36-inch diameter reinforced concrete pipe (RCP) to a pretreatment device. Pretreatment will be an integral component of the strategies to extend the life of the stormwater capture system. After pretreatment the stormwater flow will be conveyed to the underground storage facilities. A pump and filter system will lift the water from the underground storage facility invert and provide final pollutant removal prior to discharge back into the storm drain channel or into the existing Los Angeles County Sanitation District (LACSD) north plan outfall relief trunk sewer running along Light Street.

The purpose of this study was to evaluate the subsurface conditions at the site and to provide recommendations for the design and construction of the proposed improvements. This report summarizes the collected data and presents our findings, conclusions, and geotechnical design recommendations.

2. SCOPE OF WORK

Tetra Tech's scope of services for this project consisted of the following tasks:

- Review readily available background data, including existing geotechnical reports prepared for the site by Ninyo and Moore and the County of Los Angeles.
- Perform a reconnaissance site visit to observe ground conditions and mark boring locations.
- Obtain drilling permits from the Los Angeles County Department of Public Health (LACDPH).
- Coordinate with LACDPH engineering staff, park personnel, and Underground Service Alert (USA) for clearance of buried on-site utilities prior to drilling.
- Advance 4 Cone Penetration Tests (CPTs) to a maximum depth of 50 feet to characterize the subsurface conditions at the site and take one soil sample at a depth of 15 feet for analytical testing.
- Conduct an evaluation of the geotechnical data to develop geotechnical recommendations for the design and construction of the proposed structures including the following items:
    - An evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
    - An evaluation of the liquefaction potential and dynamic settlement of the on-site granular materials.
    - An evaluation of the suitability of on-site soils for the support of structures.
    - A presentation of the results of agronomic testing.
    - A presentation of the results of analytical testing for a soil sample.
    - Recommendations for design of foundation systems including allowable bearing capacity, lateral resistance, and settlement estimates.
    - Determination of seismic design parameters in accordance with the 2019 California Building Code (CBC).
    - Evaluation of lateral earth pressure parameters for the design of the underground storage facility and for the design of temporary shoring during construction.
    - An evaluation of the corrosion potential of the on-site soils to buried concrete.
- Prepare this written report documenting the work performed, physical data acquired, and geotechnical design recommendations.

3. PROJECT BACKGROUND AND DESCRIPTION

The Upper San Gabriel River Watershed Management Area Group (USGR Group) is comprised of the County of Los Angeles (County), Los Angeles County Flood Control District (LACFCD), and the cities of Baldwin Park, Covina, Glendora, Industry, La Puente, and West Covina. The USGR Group was formed in accordance with the National Pollutant Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System (MS4) Permit Order No. R4-2012-0175 (Permit). The USGR Group, through a cooperative and collaborative process, developed an Enhanced Watershed Management Program (EWMP). The Final USGR Group EWMP was subsequently approved by the Los Angeles Regional Water Quality Control Board on April 11, 2016.

The USGR Group EWMP identified a suite of watershed control measures and structural Best Management Practices (BMPs) to address the water quality objectives within the San Gabriel River watershed. The Adventure Park site was identified as one of the top ranked projects for implementation and is owned by the County of Los Angeles. Through coordination with the County of Los Angeles, the Adventure Park site was included within the EWMP and has the potential to provide significant water quality benefits for multiple jurisdictions due to the large drainage area, location of the adjacent storm drains, and available development space for one of the largest stormwater capture facilities within a park.

The primary design goal of the Adventure Park project is to reduce long-term annual loading of pollutants to Coyote Creek and the San Gabriel River using zinc as the limiting pollutant in the analysis as established by the EWMP for this Watershed Group.

The objective of this report is to provide the County of Los Angeles final design recommendations that will ultimately guide the development of design and construction documents. The following configuration is being considered:

- Diversion of up to 50 cubic feet per second (cfs) of runoff from Sorensen Drain,
- Overall stormwater storage volume of 19.6 acre-foot (6.4 MG) with underground units about 12 feet high (including 10 feet of storage depth and 1 foot of freeboard),
- Pump station with a capacity of 5.76 cfs,
- Discharge via either sewer discharges during non-peak hours (with 2.88 cfs filtration) or via filtration only (5.76 cfs), and
- Partitioned irrigation storage with pump/filter infrastructure for non-potable onsite reuse.

At this moment no final information regarding the depth, and size of the diversion structures, pre-treatment unit, and pump station is available. Therefore, the designer should verify with the Geotechnical Engineer the applicability of the recommendations contained herein once the final layout and preliminary design is completed.

4. PREVIOUS SUBSURFACE INVESTIGATIONS

Two previous soil investigation were conducted at Adventure Park:

- Exploration by Ninyo and Moore (2015) on March 2015 on the southwest area of Sorensen Drain (outside of the project area) which consisted of drilling one borehole to a depth of 46.5 feet. The investigation encountered fill materials to a depth of 1 foot. Alluvium was observed underlying the fill material extending to the total depth explored, and consisted of well graded sands with silts, silty sands, clayey sands and sandy silts. Groundwater was encountered at a depth of 31 feet. Boring logs from this exploration are presented in Appendix A. Laboratory tests results from the Ninyo and Moore exploration are included in Appendix C.

- Exploration by the Los Angeles County Department of Public Works (2018) on December 2016 and June 2017 on the northeast area of Sorensen Drain within the current project area consisted of 7 Cone Penetration Tests (CPTs) to a maximum depth of 100 feet, and 3 soil borings drilled to a maximum depth of 51.5 feet. The CPTs indicated that the subsurface materials consisted of silty sand and sandy silt, clay and silty clay. CPT logs from this exploration are presented in Appendix B. In the borings, fill materials were encountered in the upper 2.5 feet underlain by alluvium consisting of silty sand, clayey silt, silty clay and fat clay with the consistency of the clay ranging from stiff to hard. Boring logs from this exploration are presented in Appendix A.

  Groundwater was encountered in borings B-1 through B-3 at a depth of 22, 27.5, and 28.5 feet respectively. Selected soil samples were retrieved to characterize the soils and their engineering properties. Laboratory tests results from the LACDPW exploration are included in Appendix C.

The locations of these 2 previous explorations are shown on Figure 2.

5. CURRENT SUBSURFACE EXPLORATION

The current subsurface soil conditions beneath the site were explored by Tetra Tech on March 31, 2020 and included advancing 4 CPTs denoted CPT-1 through CPT-4.

Prior to starting the field exploration program, a field reconnaissance was conducted to observe surface conditions and to mark the locations of the planned CPTs in agreement with County engineering staff and park staff. A drilling permit was obtained from the Los Angeles County Department of Public Health (LACDPH) for all the subsurface explorations. Underground Service Alert, the LACDPH inspector, and park staff were also notified of the subsurface exploration schedule at least 48 hours prior to drilling.

The CPTs were advanced to a maximum depth of approximately 50 feet using a standard electronic piezocone with a 15 cm^2 area and a 60-degree apex angle. The piezocone was pushed utilizing a 30-ton truck. The piezocone was pushed at a rate of 2 cm/sec and the soil tip resistance, soil-sleeve friction, and immediate dynamic pore water pressure response were recorded at 1-inch intervals. CPT testing was carried out in accordance with ASTM D5778. A copy of the Cone Penetration Test Data report is included in Appendix D.

A soil sample for environmental screening was retrieved using the CPT push-in discrete depth soil sampler positioned at a depth of 15 feet next to CPT-4. At the completion of drilling, the CPT holes were backfilled with a bentonite cement grout in accordance with LACDPH requirements.

Watearth performed infiltration testing at the site using the double infiltrometer. The results of the infiltration testing will be reported separately by Watearth (2020). Two surficial soil samples were retrieved by Watearth at the approximate locations indicated in Figure 2 and then transferred to Tetra Tech for agronomic testing and sieve analysis.

The approximate latitude and longitude of the current soil explorations, the approximate elevations, and depths of the current explorations are included in Table 1. These locations are also shown on Figure 2.

Table 1
Tetra Tech CPT Information and Watearth Testing/Sample Locations

Exploration Number	Approximate Latitude	Approximate Longitude	Approximate Ground Elevation (ft msl)*	Approximate Depth (ft)
CPT-1	33.943038	-118.035171	142	50.2
CPT-2	33.942669	-118.035030	140	50.2
CPT-3	33.942341	-118.034263	141	50.2
CPT-4	33.942771	-118.033986	144	50.3
DRI-01/ (TP-1)	33.942940	-118.035584	136	1.5
DRI-02/ (TP-2)	33.942279	-118.035070	136	1.5

*Estimated from Google Earth

6. LABORATORY TESTING

Laboratory tests were performed by Tetra Tech on a soil sample recovered from the CPT sampling for analytical testing. As requested by Watearth the following tests were performed:

- EPA 8015M – Extended Range Hydrocarbons
- EPA 8260B by 5035 – Volatile Organics
- EPA 6010B by 350B and EPA 7471A – CAM 17 Metals

The results of the analytical testing and interpretation are included in Appendix E.

Watearth also retrieved 2 surficial soil samples taken during their percolation testing. These samples were provided to Tetra Tech for agronomic testing and for sieve analysis.

The agronomic testing included:

- pH and electroconductivity (salinity) measurement,
- saturated extract paste nutrients/toxic elements measurement of DTPA extract for measurement of sodicity (Sodium Adsorption Ratio),
- saturation extract: nitrate, chloride, sulfate, sodium, calcium, magnesium, potassium, soluble phosphate and boron,
- estimate of soil texture and soil organic matter presence of lime.

The results of the agronomic testing and interpretation are provided in Appendix F.

Sieve analyses were performed in general accordance with ASTM D6913, and the test results are provided in Appendix G.

7. SUBSURFACE CONDITIONS

7.1. Regional Geology

The project site is located in the northeastern part of the greater Los Angeles Basin (basin). The Los Angeles Basin is located within Peninsular Ranges geomorphic province which is characterized by generally northwest trending elevated ground, hills and mountain ranges with intervening valleys. Topographic relief across the basin includes a low-lying plain that rises gently from the Pacific Ocean inland to the surrounding Santa Monica and San Gabriel Mountains to the north, Puente Hills to the northeast, the Santa Ana Mountains to the Southeast, and the San Joaquin hills and Palos Verdes Peninsula to the south.. Structurally, the Peninsular Ranges province includes northwest-southeast trending structural blocks separated by northwest-southeast trending strike-slip faults. The coastal portion of the basin is filled with several thousand feet of recent to later Tertiary age sediments.

7.2. Site Geology

Regional geologic mapping published by Dibblee and Ehrenspeck (2001) (Figure 3 – Geologic Map), shows that the site to be underlain by valley and floodplain alluvial deposits of Holocene to late Pleistocene age consisting of gravel, sand, and silt. Generalized descriptions of subsurface materials reported in previous site-specific exploration investigations by Ninyo & Moore (2015) and LACDPW (2018) included fill and native alluvial soils. Reported subsurface conditions are described in the following sections.

7.2.1. Artificial Fill (af)

Artificial fill soil was reportedly encountered in borings within the park grass areas to depths up to 2.5 feet below the existing site grades. As reported, the artificial fill soils were composed of medium dense, dark brown, moist, fine to coarse grained silty sand. Artificial fill soils were not encountered in the northeast parking lot (Figure 3 – Site Layout, Boring and CPT Location Map).

7.2.2. Native Alluvium (Qa)

Native alluvial (Qa) soil was reportedly encountered below the fill soils to the maximum explored depth of 51.5 feet below the ground surface and included fine- and coarse-grained soils. The fine-grained materials reportedly consisted of dark brown, brown to yellowish brown and reddish-brown fat clay, silty clay, silt, and sandy silt. The coarse-grained materials reportedly consisted of light brown and grayish brown to brown silty sand to poorly and well-graded sand. The coarse-grained soils were generally found at a depth ranging from about 20 to 30 feet below the ground surface across the explored area.

Uncorrected SPT blowcounts in the native alluvium for the fine-grained (clay, silt) soils ranged from 4 to 65 indicating firm to very hard consistency. Uncorrected SPT blowcounts in the native alluvial coarse-grained (sandy) soils ranged from 20 to 75 indicating medium dense to very dense materials.

7.3. Groundwater

According to the State of California (CDMG, 1998), the historic high groundwater level near the site has been mapped at a depth of about 10 feet (Figure 4 – Historic High Groundwater Map). Groundwater was encountered in the Ninyo and Moore (2015) exploratory boring at a depth of approximately 31 feet, although this boring is outside the planned project area, southwest of Sorensen Drain. Groundwater was encountered in the 3 LACDPW exploratory borings within the project area at a depth of 22, 27.5, and 28.5 feet. Table 2 presents the inferred groundwater elevations for each LACDPW boring at the time of their exploration (June 2017).

Table 2
LACDPW Borings and Groundwater Depths/Elevations (June 2017)

Exploration Number	Approximate Ground Elevation (ft msl)*	Approximate Groundwater Depth	Approximate Groundwater Elevation (ft msl)
B-1/LACDPW	139	22	117
B-2/LACDPW	146	27.5	118.5
B-3/LACDPW	145	28.5	116.5

LACDPW also performed an evaluation of nearby wells within a 1-mile radius which indicated groundwater depths ranging from 17 to 145 feet over the period 1950 to 1989.

Based on the assessment of the local stratigraphy and local topography, it is our opinion that the Ninyo and Moore data as well as the LACDPW data can be utilized for interpretation of the project groundwater conditions. Therefore, it is our conclusion that the high groundwater at the site has been deeper than 15 feet within the last 50 years.

Fluctuations of the groundwater level, localized zones of perched water, and increased soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or adjacent to the site can also cause a fluctuation of local groundwater levels. Evaluation of such factors is beyond the scope of our services.

Based on the research and observed conditions, groundwater is expected to impact the construction of the proposed development and the historic high groundwater depth of about 10 feet should be considered for the design. Therefore, any proposed structures embedded or buried deeper than 10 feet should consider hydrostatic lateral and hydraulic uplift forces. For construction purposes the Contractor should consider the possibility of a groundwater depth of about 20 feet i.e., elevation 115 feet. However, it is acknowledged that depending on the year and season when construction actually takes place, the groundwater depth could be different and in some cases even deeper, therefore an evaluation of the actual groundwater depth before construction is recommended by installing a monitoring well at the site.

8. ENGINEERING SEISMOLOGY AND GEOLOGIC HAZARDS

8.1. General Seismic Setting

The Southern California region is known to be seismically active. Earthquakes occurring within approximately 60 miles of the site are considered capable of generating ground shaking of engineering significance to the proposed construction. The project area is located in the general proximity of several active and potentially active faults, as shown on Figure 5 – Regional Faults and Seismicity Map. Active faults are defined as those that demonstrated evidence of surface displacement within the Holocene period (approximately the last 11,000 years).

Active faults within approximately 10 miles of the subject site include the Puente Hills Blind Thrust located 1.2 miles south of the site, the Elsinore - Whittier fault located approximately 2.9 miles northeast of the site, and the Los Alamitos fault located approximately 8.8 miles southwest of the site. The San Andreas Fault is located about 35.5 miles to the northeast of the site.

Table 3 lists selected principal known active faults that may affect the subject site and the maximum moment magnitude (M_{max}) as published by Cao et al. (2003) for the California Geological Survey (CGS). The approximate distances to the site were calculated from Jennings (2010).

Superimposed on the area map in Figure 5 are earthquake epicenters of magnitude M5.0 or more as recorded by the USGS between 1900 to present day. Significant seismic activity for the period has been recorded surrounding the project site. However, relatively few earthquake epicenters have been recorded in the immediate area of the subject site. Notable historic earthquakes in Southern California of significance to the project are listed in Table 4. The most significant historic earthquake near the project site was the 1933 Long Beach earthquake.

Table 3
Principal Active Faults

Fault Name	Approximate Fault Distance to Site[1] (miles)	Maximum Moment Magnitude[2] (Mmax)
Puente Hills Blind Thrust	1.2	7.1
Elsinore - Whittier	2.9	6.8
Los Alamitos	8.8	6.2
Newport-Inglewood	12.1	7.1
Raymond	13.2	6.5
Sierra Madre	14.6	7.2
Hollywood	14.7	6.4
THUMS-Huntington Beach	17.3	7.0
Verdugo	17.6	6.9
Palos Verdes	18.6	7.3
Charnock	19.6	6.5
Cabrillo	21.3	6.8
Redondo Canyon	22.8	6.5
Santa Monica	23.5	6.6
San Andreas	35.5	7.8
Malibu Coast	36.1	6.7
Anacapa-Dume	42.5	7.5

Notes:
[1] per Jennings, 2010
[2] per Cao, et al., 2003

Table 4
Historic Earthquakes in Southern California

Earthquake Name	Year	Fault and Fault Type	Earthquake Magnitude*	Epicenter	
				Latitude	Longitude
La Habra	2014	Shallow previously unknown fault	5.1 M_w	33.933°N	117.916°W
Chino Hills	2008	Yorba Linda Trend	5.4 M_w	33.949°N	117.766°W
Northridge	1994	Northridge Thrust (Blind Thrust) (a.k.a. Pico Thrust)	6.7 M_w	34.21°N	118.54°W
Sierra Madre	1991	Clamshell-Sawpit Canyon Fault (Reverse)	5.8 M_L	34.20°N	118.14°W
Pasadena	1988	Raymond Fault (left lateral strike-slip)	5.0 M_w	34.14°N	118.13°W
Whittier Narrows	1987	Puente Hills Fault (Blind Thrust Fault)	5.9 M_L	34.06°N	118.08°W
San Fernando	1971	San Fernando Fault (thrust)	6.5-6.7 M_w	34.42°N	118.37°W
Torrance-Gardena	1941	Palos Verdes Fault (right-reverse)	4.8 M_L	33.82°N 33.78°N	118.22°W 118.25°W
Long Beach	1933	Newport-Inglewood Fault (right- lateral strike-slip)	6.4 M_w	33.63°N	118.00°W
San Jacinto	1923	San Jacinto Fault (right- lateral strike-slip)	6.3 M_L	34.00°N	117.24°W
San Jacinto	1918	San Jacinto Fault (right- lateral strike-slip)	6.7 M_w	33.65°N	117.43°W
Elsinore	1910	Elsinore Fault (right- lateral strike-slip)	6 M_L	33.75°N	117.45°W
Fort Tejon	1857	South Central Segment of the San Andreas Fault (right- lateral strike-slip)	7.9 M_w	35.43°N	120.19°W

*M_w refers to Moment Magnitude scale
M_L refers to Local Magnitude scale

Potential seismic sources of significance to the project include active faults previously described and faults that are not known to break the ground surface but are considered active. This latter group of faults includes buried or "blind" thrust faults. Current tectonic models for the Los Angeles basin include buried thrust faults, several of which are considered partly responsible for the north-to-south compression of the basin. Although these faults are not currently zoned by the State of California for surface rupture hazards (Earthquake Fault Zones), many are considered capable of generating seismic shaking of significance to structures.

Of these buried active faults the closest to the site is the Puente Hills Trust Fault (PHTF). The PHTF is currently defined as 3 separate but juxtaposed, generally east-west trending and north-dipping, fault surfaces that combined extend from Downtown Los Angeles to Brea. From west to east these include the Los Angeles, Santa Fe Springs, and Coyote Hills segments. Based upon recent studies by several researchers, including: Shaw et al., (2002), Olsen and Cooke (2005), and Leon et al. (2007), the three fault surfaces are interpreted to extend from depths in excess of 9 miles on the north side of the Los Angeles basin to less than 1.2 miles at the southerly limits of the fault surfaces in the central portion of the basin. Fault surface geometries are interpreted from historical petroleum exploration data, geotechnical subsurface exploration data, and limited seismicity (e.g., the 1987 magnitude 5.9 Whittier Narrows earthquake).

Leon et al. (2007) estimates that upwards of 60 percent of the total Los Angeles basin compression may be attributed to strain along the PHTF. Although ground rupture has not been officially attributed to the fault, the presence of youthful hills (e.g., Coyote Hills) and shallow folding at depth in the upper portion of the interpreted thrust ramp suggests recent activity. The PHTF is considered capable of generating earthquake magnitudes up to about M_w 7.1.

8.2. Surface Fault Rupture

Official Maps of Earthquake Fault Zones were reviewed to evaluate the location of the project site relative to active fault zones. Earthquake Fault Zones (known as Special Studies Zones prior to 1994) have been established in accordance with the Alquist-Priolo Special Studies Zones Act enacted in 1972. The Act directs the State Geologist to delineate the regulatory zones that encompass surface traces of active faults that have a potential for future surface fault rupture. The purpose of the Alquist-Priolo Act is to regulate development near active faults in order to mitigate the hazard of surface fault rupture.

The site is not located within a designated Earthquake Fault Zone for fault surface rupture hazard. Based on a review of State of California Earthquake Fault Zone maps, the closest fault zone for surface rupture is the Elsinore-Whittier Fault Zone approximately 2.9 miles northeast of the site according to the CGS website application (https://maps.conservation.ca.gov/cgs/EQZApp/app/).

No surface traces of any active or potentially active faults are known to pass directly through or project towards the site. Neither our field exploration nor literature review disclosed an active fault trace projecting to the ground surface in the project area. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low.

8.3. Seismic Hazard Zones

Maps of seismic hazard zones are issued by the California Geological Survey (CGS, formerly California Department of Conservation, Division of Mines and Geology (CDMG)) in accordance with the Seismic Hazards Mapping Act enacted in April 1997. The intent of the Seismic Hazards Mapping Act is to provide for a statewide seismic hazard mapping and technical advisory program to assist cities and counties in developing compliance requirements to protect the public health and safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure and other seismic hazards caused by earthquakes.

Based on the review of the Whittier Quadrangle Official Map of Seismic Hazard Zones issued March 25, 1998 (see Figure 6 – Seismic Hazard Zones Map), the proposed development is located within an area identified by the State of California as subject to the hazard of liquefaction. Because the site is located in a mapped area where the potential for liquefaction exists and due to the increase in the code-prescribed seismic demand since the Seismic Hazard Map was generated, liquefaction analyses were performed per the 2019 CBC to evaluate the site liquefaction potential.

8.4. Liquefaction Potential and Dynamic Settlement

Liquefaction of soils can be caused by ground shaking during earthquakes. Research and historical data indicate that loose, relatively clean granular soils and low plasticity silts are susceptible to liquefaction and dynamic settlement, whereas the stability of the majority of clayey silts, silty clays and clays are not typically adversely affected by ground shaking. Liquefaction is generally known to occur in saturated or near-saturated cohesionless soils at depths shallower than about 50 feet.

8.4.1. Soil Description

Evaluation of liquefaction potential for the on-site materials was performed based on soil stratigraphy encountered in the field explorations. The encountered soil materials generally consisted of alluvial deposits made up of layers of medium dense to dense silty and clayey sands interspersed with layers of stiff to hard lean and fat clays and silts.

Materials that are above the groundwater table are not considered susceptible to liquefaction. Thus, the focus of this investigation was aimed at evaluating the liquefaction potential of the soils encountered at a depth between 10 and 50 feet. Fine grained soils as described in the following sections can undergo severe strength loss during ground shaking, and thus an evaluation of their sensitivity was performed.

8.4.2. Groundwater Level for Liquefaction Analysis

Groundwater was encountered during the Ninyo and Moore (2015) and the LACDPW (2018) field explorations at depths ranging between 22 to 31 feet. However, the historic high groundwater at the site was mapped by CDMG (Whittier Quadrangle) at a depth of about 10 feet. Therefore, a groundwater depth of 10 feet was assumed for evaluation of liquefaction potential at the site.

8.4.3. Liquefaction Seismic Demand

Based on the Structural Engineers Association of California (SEAOC) and the Office of Statewide Health Planning and Development (OSHPD) website application (https://www.seaoc.org/page/seismicdesignmaptool), for a site with latitude and longitude 33.942746°, -118.034187°, respectively, the mapped Geometric Mean Peak Ground Acceleration (PGA_M) was estimated to be approximately 0.836g for a site class D (assumed v_s = 259 m/s), for a ground motion corresponding to the Maximum Considered Earthquake (MCE). From the Seismic

Hazard Interactive Deaggregation website (https://earthquake.usgs.gov/hazards/interactive/) this ground motion approximately corresponds to a predominant earthquake magnitude of M_w 6.85. The largest contributor to the seismic hazard at the site (about 20 percent) is the Puente Hills fault located about 1.2 miles southeast from the site. These ground motion parameters were used in the liquefaction analyses. A summary of the seismic demand parameters is presented in Appendix H.

8.4.4. Evaluation of Liquefaction Potential and Sensitivity Analyses

The liquefaction potential at the site was evaluated by the LACDPW (2018) in accordance with the County of Los Angeles GMED Manual for Geotechnical Reports GS 045.0 (LACDPW, 2014) and the CGS Publication 117A (2008). The analysis was based on data obtained from boring B-1 drilled in 2017, which yielded a total seismically induced settlement at the site of about 3.4 inches and within the allowable limits stated in GS 045.0 so that structural mitigation is feasible.

The liquefaction potential of cohesionless (sandy) soils was re-evaluated based on the SPT blowcounts from boring B-3 from the LACDPW (2018) exploration and laboratory test results utilizing the procedure published by Boulanger and Idriss and (2014) and as recommended in GS 045.0 (2014). The SPT blowcounts from soil boring B-1 from the Ninyo and Moore (2015) exploration were used to re-evaluate the liquefaction potential for comparison purposes with the acknowledgment that the deposits southeast of Sorensen Drain are older alluvial deposits with a different liquefaction susceptibility than those at the site.

The analyses based on standard penetration test (SPT) considered the energy ratio correction factor C_E of 1.5, and sampler factor C_S of 1.2 as provided by LACDPW (2018). The borehole diameter factor C_B of 1.15 used by the County was modified to 1.0 based on the internal diameter of the hollow stem auger system used during drilling per SP 117. The blowcounts recorded for soils driven with the 3-inch O.D. California Sampler with brass rings were converted to equivalent SPT blowcounts using a reduction factor of 0.67 as recommended by SP 117.

Results of liquefaction analyses of granular soils are summarized in Table 5 in the next section of this report and presented in Appendix I. The analyses based on SPT data from the soil borings indicates that the on-site sandy silts and silty sands found at a depth interval between 23 and 30 feet and between 35 and 45 feet are susceptible to liquefaction.

The liquefaction potential of the subsurface materials was also evaluated from the CPT data using the computer software CLiq v.2.0.6.97 by Geologismiki. The liquefaction susceptibility and the liquefaction induced settlements were evaluated using the Boulanger and Idriss and (2014) method. The CPT analyses indicate that that the on-site sandy silts and silty sands found at a depth interval between approximately 21 and 26 feet are susceptible to liquefaction. Results of the liquefaction analysis using CPT data are summarized in Table 5 and presented in Appendix I.

Seismic sensitivity of fine-grained soils was further evaluated per County of Los Angeles Administrative Manual GS045.0 where the fine-grained soils are classified in the following 3 categories:

1. Soils with Plasticity Index < 12 and moisture content > 85 percent of the liquid limit are classified as fine-grained soils susceptible to liquefaction (typically includes silts);

2. Soils with Plasticity Index > 18 and a degree of sensitivity $S_t > 6$ are classified as fine-grained soils potentially susceptible to significant loss of strength during seismic shaking and require additional evaluation. The sensitivity of the on-site fine-grained soils is evaluated based on the water content, Atterberg limits, and effective vertical stresses using the procedures suggested by Holtz and Kovacs (1981) and Terzaghi, Peck and Mesri (1996).

3. Fine-grained soils falling outside the two categories described above are considered to behave like clays and are not considered susceptible to liquefaction or seismic sensitivity.

Analyses of the sensitivity of the saturated fine-grained soils to ascertain the potential for cyclic softening was not performed for the fine-grained materials based on the boring logs because the PI of 15 and 4 representative of these soils fell below the threshold level of 18 indicating that the soils are not susceptible to cyclic softening although they are susceptible to liquefaction. To further confirm this, the sensitivity was estimated from the CPT data based on published correlations which indicated that the fine-grained soils at the site ranged between 1 and 3.5 with most values in the order of 2, i.e., significantly less than the accepted sensitivity threshold value of 6. Therefore, these soils are not considered to be susceptible to undergo seismically induced cyclic softening and associated deformations. Consequently, the potential for significant loss of strength of fine-grained materials and ensuing bearing failure during seismic shaking is considered low. The results of the sensitivity analyses for the soils based on the CPT data are included in Appendix I.

8.4.5. Dynamic Settlement

Dynamic settlement can occur in both dry and saturated sands when loose to medium-dense granular soils undergo volumetric changes during ground shaking. Dynamic settlement can occur in saturated sands due to liquefaction or in dry sands due to densification of the soil matrix. The anticipated dynamic settlement of the saturated soils at the site was evaluated using SPT data from the Ninyo and Moore (2015) exploration and the SPT data from the LACDPW (2018) exploration using procedures outlined by Yoshimine et al (2006). The estimated settlements by this procedure were further adjusted by a calibration factor of 0.9 as recommended by Cetin (2009).

The potential for dry dynamic settlement using SPT data was calculated according to the procedure outlined in Pradel (1998a and 1998b). The potential for dry dynamic settlement using CPT data was evaluated using the computer software CLiq v.2.0.6.97 according to the procedure outlined in Robertson and Shao (2010).

Table 5 presents the results of liquefaction analyses and dry dynamic settlement. The details of dynamic settlement analyses are presented in Appendix I.

As shown in Table 5, the combined dynamic settlement of the on-site soils estimated from boring SPT data and CPT data is less than 3.5 inches at the ground surface. The differential seismic settlement is estimated to be no greater than 1.8 inches over a horizontal distance of 30 feet.

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Furthermore, for most of the structures that are embedded or buried at least 25 feet the total seismic settlement is on the order of 2.0 inches with a differential settlement of about 1.0 inch over 30 feet. Therefore, structural mitigation of the total and differential seismic settlement is acceptable at this site.

It is noted that although the magnitude of the estimated dynamic settlements corresponds to an mean estimated settlement which can vary in the order of +- 50 percent, the standard of practice uses mean estimated values in developing guidelines and evaluating potential damage to structures.

Table 5
Results of Liquefaction and Dry Dynamic Settlement Analyses

Exploration No.	Assumed Groundwater Depth (feet)	Liquefiable Zone Depth Interval (feet)	FS_{liq}	Liquefaction Settlement (inches)[1]	Settlement of Dry Sands (inches)[1]	Combined Dynamic Settlement (inches)[1]
B-1 (old alluvium) (Ninyo and Moore, 2015)	10	36-45	0.5	1.7	0.1	1.8
B-3 (LACDPW, 2018)		23-30 35-45	0.8 0.5	2.7	negligible	2.7
CPT-1 (Tetra Tech 2020)		20-32	0.3	2.3	negligible	2.3
CPT-2 (Tetra Tech 2020)		21-27	0.25	1.8	1.7	3.5
CPT-3 (Tetra Tech 2020)		22-23	0.2	1.0	1.9	2.9
CPT-4 (Tetra Tech 2020)		10-15 20-26	0.2	2.5	negligible	2.5

[1]Estimated settlements are mean values which can vary within +-50 percent.

8.5. Earthquake-Induced Landslides

The site is not located in an Earthquake-induced Landslide Hazard Zone on the State of California Seismic Hazard Zones Map (see Figure 6). No evidence of landsliding was observed on or in the immediate vicinity of the site. Therefore, the occurrence of an earthquake-induced landslide at the site is not considered to be hazard to the site.

8.6. Lateral Spreading

Since the only open face at the site is at Sorensen drain and is well above the historic high groundwater and since the overall site is generally flat, the hazard of lateral spreading is considered low.

8.7. Subsidence

Land subsidence is the lowering of the ground surface due to extraction or lowering of water levels or other stored fluids within the subsurface soil pores, or due to seismic activity. Groundwater withdrawal causes the alluvial sediments in the basin to compact. Fine-grained materials such as clays and silts that comprise the aquitard that separates the Upper and Lower aquifers in the east valley are more susceptible to compaction and subsidence than coarse-grained sediments, such as sands when groundwater is removed. Damage caused by subsidence can be visible cracks, fissures, or surface depression.

The site is not mapped within an area of land subsidence in California (https://ca.water.usgs.gov/land_subsidence/california-subsidence-areas.html). Therefore, subsidence is not considered a hazard at this site.

8.8. Gaseous Hazards

Methane is a naturally occurring gas associated with the decomposition of organic materials. In high concentrations, methane is considered to be an explosion hazard. According to the City of The site is not within 300 feet of an oil or gas well or 1,000 feet of a methane producing site. A methane mitigation system may not be required according to the LA County website (https://pw.lacounty.gov/epd/swims/OnlineServices/search-methane-hazards-esri.aspx). The site is not located within an oil or gas field according to the State of California Department of Conservation's Division of Oil, Gas and Geothermal Resources Map (https://maps.conservation.ca.gov/doggr/wellfinder/).

8.9. Expansive/Collapsible Soils

Based on our field investigation, the soil materials below the artificial fill are alluvial deposits consisting of stiff to hard silts and clays and medium dense to dense sands. Such deposits are generally not susceptible to collapse; therefore, the potential of collapse is expected to be low.

The Expansion Index of two selected samples as tested by the LACDPW ranges between 39 and 42, which indicates that the soils at the site have a low expansion potential. However, it is noted that only a limited and small number of samples were tested to represent the overall site.

9. DESIGN RECOMMENDATIONS

9.1. General

Based on the results of the field exploration and engineering analyses, it is Tetra Tech's opinion that the proposed construction is feasible from a geotechnical standpoint, provided that the recommendations contained in this report are incorporated into the design plans and implemented during construction. It is expected that conventional foundation and construction methods will be suitable for the proposed improvements.

Observations and laboratory tests indicate that the on-site soils have negligible levels of water-soluble sulfates, therefore, the soils are not expected to cause injurious sulfate attack on concrete with a minimum 28-day compressive strength of 2,500 psi.

The key geotechnical design focus will be on:

- Excavation and shoring design;
- Dewatering measures;
- Foundation design of the subterranean structures.

The design recommendations presented below reflect these considerations.

The design recommendations presented below are based on Tetra Tech's current understanding of the project. Once the project configuration is finalized and the design is complete, Tetra Tech should review the plans and specifications to evaluate if the geotechnical design recommendations have been incorporated as intended.

9.2. Clearing and Grubbing

The construction area should be cleared of any pavement, structures, vegetation, trash and debris, prior to commencement of the earthwork. Any subterranean installations not to be preserved, such as pipes, utility collectors, tanks, older foundations, etc., should be abandoned and removed per Geotechnical Engineer's recommendations and in accordance with applicable regulations. All undocumented fills including the existing landscape fill mounds and other unsuitable materials within the construction areas should be removed.

9.3. Site Preparation

In order to create uniform bearing conditions for the proposed improvements the following is recommended:

- <u>Underground storage facility, pump well, pretreatment unit, deep actuated valve wells,</u> should be founded on competent native soils. No need for overexcavation is expected for the foundations located at anticipated invert depths between 10 and 30 feet, unless loose/soft unsuitable conditions are encountered as discussed below. As a minimum, a 6-inch-thick layer of gravel should be placed on top of the approved subgrade below the bottom of all

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underground units. This gravel thickness can be increased based on manufacturer's specifications. This layer should extend wherever possible 5 feet beyond the outer edge of underground units. The gravel layer should be separated from the approved subgrade with a woven geotextile i.e., Mirafi RS380i or equivalent.

- Lightly loaded ancillary structures areas should be overexcavated to a depth of at least 2 feet below the bottom of the proposed footing or floor slab or to competent native soils, whichever is deeper. The excavation should extend a horizontal distance of at least 2 feet beyond the outside perimeter of the structure.

- Pavement areas and flatwork areas should be overexcavated and recompacted to a depth of at least 1 foot below the proposed subgrade elevation, or to uniform acceptable soils, whichever is deeper. To the extent practicable, the zone of overexcavation should extend a horizontal distance of at least 2 feet beyond the outside perimeter of the pavement.

- In non-structural/landscaped areas, any existing fill may remain in place. However, depending on the future use of the area, existing fill may need to be excavated and replaced as compacted fill. This can be evaluated during grading.

- Disturbed soils at structural and non-structural areas will likely occur after demolition of existing site improvements. These soils should be overexcavated and recompacted to the total depth of the disturbed material.

The exposed overexcavation subgrade for all structures and slabs, should be probed and accepted by the Geotechnical Engineer. The soils should be scarified to a depth of 4 inches. Fine-grained soils should be moisture conditioned to a minimum of 125 percent of optimum moisture content and compacted to at least 90 percent of the maximum dry density as evaluated by the latest version of ASTM D1557, sandy soils should be moisture conditioned to a minimum of 110 percent of optimum moisture content and compacted to at least 90 percent of the maximum dry density.

Localized zones of loose and/or unstable soils may be encountered during the grading operations at the subgrade level and should be overexcavated and recompacted. If loose/soft/wet areas are encountered that are not practical to be excavated and processed, Table 6 below provides options for stabilizing the subgrade. The objective is to produce at least 3 feet for foundations and 2 feet for pavements of competent fill to bridge over the impacted area. The specific type of remediation and associated area limits will need to be evaluated in the field by a representative of Tetra Tech.

All fill placement associated with fine-grained soils used to replace overexcavated soils, fill placed to achieve finish grade or subgrade, or utility trench backfill should be moisture conditioned to a minimum of 125 percent of optimum moisture content and compacted to at least 90 percent of the maximum dry density as evaluated by the latest version of ASTM D1557; sandy soils should be moisture conditioned to a minimum of 110 percent of optimum moisture content and compacted to at least 90 percent of the maximum dry density. The upper 1 foot of soils below pavements and any flatwork should be processed and compacted to at least 95 percent of the maximum dry density (per ASTM D1557).

Excavated on-site soils may be re-used as compacted fill provided they are free of organics, deleterious materials, debris and particles over 3 inches in largest dimension. Locally, particles up to 6 inches in largest dimension may be incorporated in the fill soils based on specific approval and placement recommendations provided by the Geotechnical Engineer of Record during grading.

Table 6
Conceptual Options for Handling Unstable Materials at the Excavated Subgrade

Areas where the soils are soft and/or unstable at the excavation subgrade	• Overexcavate at least 3 feet for foundations, 2 feet for pavement areas • Stabilize the soft subgrade by working open-graded aggregate material (typically 3/4" or 1.5" crushed rock, coarser for softer subgrade) at least 4 to 6 inches into the soil. • Place woven geotextile, Mirafi RS580i or approved equivalent, over the stabilized subgrade. • Place and compact well-graded fill (e.g., AB, CMB) or general approved backfill material to specified compaction over the geotextile.
<u>Larger</u> areas where the soils are <u>excessively</u> soft and/or unstable	• Overexcavate at least 3 feet for foundations, 2 feet for pavement areas • Improve the soft subgrade by working in open-graded aggregate material as much as possible/practical into the subgrade. • Place woven geotextile, Mirafi RS580i or approved equivalent, over the exposed soil. • Place at least 8 inches (12-18 inches preferred) of well graded aggregate material (e.g., AB, CMB); only reasonably achievable compaction is required. • Place woven geotextile, Mirafi RS580i or approved equivalent, over the aggregate layer. • Place and compact fill to specified compaction over the geotextile.

In the event that any soil materials (including backfill or base course materials) are imported to the site, such soils should be sampled, tested, and approved by Tetra Tech <u>prior</u> to arrival on-site. In general, any soils imported to the site for use as fill should be predominantly granular and have an Expansion Index less than 30. Additional recommendations for site grading are provided in the "General Site Grading Recommendations" section of this report.

9.4. Temporary Slopes and Trench Excavations

The on-site soils are not expected to pose unusual excavation difficulties, and therefore, conventional earth-moving equipment may be used. Localized sloughing/raveling of exposed soil intervals should be anticipated. All trench excavations should be performed in accordance with Cal-OSHA regulations. The on-site soils may be considered Type B soils to a depth of 20 feet as defined by the current Cal-OSHA soil classification.

Unsurcharged excavations: Sides of temporary, unsurcharged excavations less than 20 feet deep should be sloped back at an inclination of 1(H):1(V) or flatter according to Cal-OSHA. For Type B soils benching could be used as long as the overall slope is kept at an inclination of 1(H):1(V) or flatter, however the bottom vertical height of the trench or excavation must not exceed 4 feet and the subsequent benches cannot be higher than 4 feet. Where space for sloped sides is not available, shoring will be necessary. If benching is selected, then the Geotechnical Engineer must verify that the bottom of the trench or excavation exposes only cohesive materials.

Surcharge setback recommendations: Stockpiled (excavated) materials should be placed no closer than 4 feet from the top of the trench or 8 feet from the edge of the excavation. Spoils should be placed so that they do not slide or fall back into the excavation. A greater setback may be necessary when considering surcharge loads such as heavy vehicles, concrete trucks and cranes. Tetra Tech should be advised of such heavy vehicle loadings so that specific setback requirements can be established for the used equipment. Alternatively, a shoring system may be designed to allow reduction in the setback distance.

Personnel from Tetra Tech should observe the excavation progress so that appropriate modifications to the excavation design may be recommended, if necessary, due to encountered conditions differing from the design assumptions.

9.5. Temporary Shored Excavations

Significant excavation is required for the construction for the proposed 19.5 acre-foot (6.4 MG) stormwater underground storage facility with foundations and associated piping anticipated at a depth between 25 to 30 feet. If sloping back the excavation is not feasible then shoring will be required. The groundwater depth is expected to be at about 15 to 25 feet; therefore, dewatering measures will be required. At these depths the use of a simple cantilevered shoring is not likely to be feasible and a shoring system assisted by tiebacks and/or soil nail walls with shotcrete facing may be necessary for the temporary support of the excavation in areas where not enough space is available for slope cuts at the inclinations indicated above. Presented herein are preliminary design recommendations for the recommended shoring systems, including a cantilevered system, based on the information available at this time. We can furnish specific design recommendations as the design progresses, if requested. The designer will need to take into account the likelihood of encroaching outside the property limits and the need to account for the presence of utilities, conduits, and other underground structures that may affect the design and installation of the shoring system.

All components of the shoring system, including the penetration depth, should be designed by a specialist Registered Civil Engineer in the State of California and should further satisfy requirements of Cal-OSHA. It is recommended that all shoring designs be reviewed by the Geotechnical Engineer of Record. The following recommendations are based on the assumption that groundwater remains below the excavation bottom, and the face of the shoring is not subject to groundwater pressure within the retained soils.

9.5.1. Soldier Pile and Lagging Wall System

Temporary soldier pile and lagging shoring system may be used to facilitate the proposed excavation. Tiebacks are usually required for excavation depths greater than about 15 feet. Alternate measures may be considered that would allow for elimination of the tiebacks such as installation of rakers, partial lowering of the grade just outside the excavation, or use of oversized soldier pile beams. If there is not sufficient space to install the tieback anchors to the desired lengths on any side of the excavation, the soldier piles of the shoring system may require internal bracing.

The soldier pile and lagging system would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and restrained with tiebacks. Continuous timber lagging or steel plates may be used between the soldier piles. Because groundwater fluctuations outside the stormwater underground facility are possible, the timber lagging should be removed at the time of backfilling.

9.5.1.1. Soldier Pile Wall Design

Table 7 below summarizes the governing geotechnical design parameters and loading diagrams for a cantilevered and tieback-supported soldier pile wall shoring system. These values are based on the following assumptions:

- the shored soil grade is level at the ground surface,
- there are no hydrostatic pressures behind the wall (dewatering system in place), and
- the shoring is temporary.

If the contractor provides a dewatering system within the excavation footprint only, then hydrostatic pressures need to be added to those provide in Table 7. Any surcharge (live or dead load) located within a 1(H):1(V) plane drawn up from the base of the shoring should be added to the lateral earth pressures. For the soldier pile wall systems, the lateral contribution of a uniform surcharge load beginning immediately behind the wall and extending a horizontal distance equal to at least the retained height, may be calculated by multiplying the surcharge by a factor of 0.42. This uniform lateral load, i.e., independent of depth, should be applied as a minimum throughout the whole exposed height of the soldier pile wall. As a minimum, a 2 feet of equivalent uniform soil surcharge, i.e., 240 psf, is recommended to be included to account for nominal construction surcharge. This office can provide recommendations for other surcharge configurations, if requested.

Table 7
Temporary Soldier Pile Wall with No Hydrostatic Pressure
(Dewatering behind the wall)
Cantilevered and Shoring with Tieback Anchors
Geotechnical Design Parameters

<table>
<tr><td>Excavation bottom depth</td><td colspan="4">Up to ~30 feet</td></tr>
<tr><td>Subsurface materials</td><td colspan="4">Alluvial Soils
Mostly very stiff lean and fat clays to a depth of 20 feet with some layers of silty sand between 22 and 30 feet</td></tr>
<tr><td>SHORING SYSTEM</td><td colspan="2">For cantilevered shoring systems</td><td colspan="2">For restrained shoring systems
Soldier pile tieback wall
– single level of tiebacks
– multiple levels of tiebacks</td></tr>
<tr><td>Soil unit weight, γ</td><td colspan="4">125 pcf</td></tr>
<tr><td>Design friction angle, ϕ</td><td colspan="2">24^{o}</td><td colspan="2">0^{o}</td></tr>
<tr><td>Design cohesion, c</td><td colspan="2">200 psf</td><td colspan="2">1,800 psf</td></tr>
<tr><td>Stability number, Ns = $\frac{\gamma \cdot H}{c}$</td><td colspan="2">n/a</td><td colspan="2">2</td></tr>
<tr><td colspan="5">LOADING PRESSURE</td></tr>
<tr><td>Ka … coefficient of active lateral pressure</td><td colspan="2">0.42</td><td colspan="2">n/a</td></tr>
<tr><td>Equivalent fluid density, EFD</td><td colspan="2">53 pcf</td><td colspan="2">n/a</td></tr>
<tr><td>Loading Diagram behind the shoring</td><td colspan="2">53 pcf EFD
(triangular distribution)</td><td colspan="2">Trapezoidal load distribution (see Diagram 1 below) based on stability number Ns = 1.3</td></tr>
<tr><td colspan="5">ALLOWABLE PASSIVE PRESSURE FOR CANTILEVER AND RESTRAINED SHORING</td></tr>
<tr><td>For shoring passive system extending to depth interval in feet</td><td>15 to 30</td><td>30 to 40</td><td>40 to 50</td><td>50 to 70</td></tr>
<tr><td>Design friction angle, ϕ</td><td>24^{o}</td><td>34^{o}</td><td>29^{o}</td><td>34^{o}</td></tr>
<tr><td>Arching capability *</td><td>1.9</td><td>2.7</td><td>2.0</td><td>2.7</td></tr>
<tr><td>Kp … coefficient of passive lateral pressure</td><td>2.4</td><td>3.5</td><td>2.9</td><td>3.5</td></tr>
<tr><td>Equivalent fluid density (pcf EFD) **
(triangular distribution)
– includes Safety Factor of 1.5
– considers arching
– ignore resistance within the upper 12 inches</td><td>190</td><td>390</td><td>240</td><td>390</td></tr>
</table>

* Per Caltrans Trenching and Shoring Manual (2011)
** Valid without reduction for soldier pile spacing > arching capability times the effective pile width. This office can provide recommendations for reduction of the allowable passive pressure for more closely spaced soldier piles

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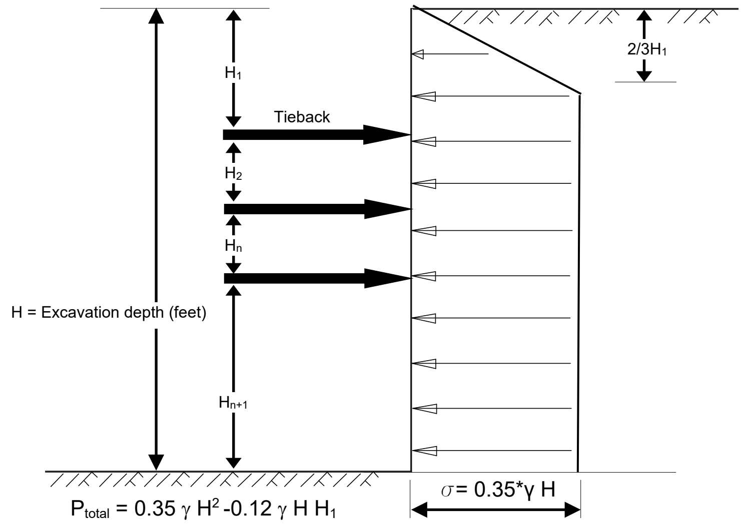

Diagram 1. Trapezoidal lateral pressures loading diagram for cohesive soils for a shoring wall with tiebacks or a braced excavation

To resist the lateral loading on shoring, the necessary depth of penetration of isolated soldier piles below the excavation bottom can be calculated based on the passive soil resistance provided in Table 7. Passive resistance should be ignored for the upper 12 inches below the excavation bottom line elevation to account for potential near-surface soil disturbance. The passive resistance of individual soldier piles in Table 7 was increased to account for soil arching and factored by a Factor of Safety of 1.5. The provided value is applicable for soldier piles that are spaced no closer than the arching capability times the pile width/diameter. For closer spacing the passive resistance would need to be reduced. The passive pressures provided in Table 7 do account for submerged conditions below the excavation bottom, however, the pressures are also based on the assumption that groundwater surrounding the excavation is lowered to at least the excavation depth. If dewatering is not performed outside of the excavation (internal dewatering only), then the effective passive pressure must be reduced to account for developed seepage forces. In addition, the impact of potential basal heave must be evaluated and, if required, mitigated.

The soldier pile beams below the excavation bottom should be backfilled with concrete, and pea gravel can be used to backfill the hole from the excavation bottom to the top. If the soldier pile beams are to be retrieved after construction, the soldier pile beams below the excavation bottom should be backfilled with a weaker cementitious slurry mix. If the contractor chooses to use a well-rounded uniform pea gravel material to fill the hole below the excavation bottom, then a reduction of 33 percent should be applied to the passive resistance values provided in Table 7 to account for the yielding of the pea gravel backfill.

Continuous timber lagging or steel plates may be used between the soldier piles. The lagging should be installed behind the front flange (closest to the excavation) and not behind the back flange (closest to the retained soil). Lagging should be removed at the time of backfilling. For the design of the lagging, earth pressures may be reduced by a factor of 0.6 to account for soil arching. The design earth pressure diagram will be the sum of the lateral pressure due to soil loading as defined in Table 7 reduced for soil arching (not need to exceed 400 psf) and the lateral pressure due to surcharge loads as defined above. Lagging should be designed to allow for drainage of any incidental seepage that could cause a temporary buildup of hydrostatic pressures

Dewatering will be required to perform the excavation and installation of shoring, and then for subsequent construction. Dewatering should be designed to keep the groundwater elevation at least 4 feet below the bottom of the excavation.

9.5.1.2. Tieback Design

Friction tieback anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 33 degrees with the vertical through the bottom of the excavation. The tieback bonded zone must not encroach inside the active zone. The unbonded length of the anchor should extend either a minimum of distance of H/5, where H is the height of the wall, or 5 feet behind the surface defined by the active wedge. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If the anchors are spaced at least 6 feet on centers, no reduction due to group action in the capacity of the anchors needs to be considered.

It should be specified that all or at least the upper row of anchors be de-tensioned after completion of the underground storage facility construction.

Tiebacks are considered to assist with the lateral restraint of the shoring and to reduce soil movement behind the shoring wall. Straight shaft, <u>pressure-grouted</u> tiebacks may be initially designed for an allowable bond stress of 1,500 psf. The allowable bond stress includes a factor of Safety of 2. The allowable bond stress should be verified by pre-production testing at the beginning of the construction.

The center of the anchor bond zone should be a minimum of 15 feet below the ground surface. The tieback bond stress may need to be adjusted depending on the tieback depth and grouting method.

To evaluate the global stability of the tieback system the soil strength parameters provided in Table 8 can be used. The analysis should incorporate the proper groundwater conditions developed by the dewatering system.

9.5.1.3. Tieback Testing

The bond stress and capacities of anchors should be verified by testing during construction. The tieback proof and performance testing program should be in compliance with the latest (4^{th} edition) Post-Tensioning Institute (PTI) guidelines "Recommendations For Prestressed Rock And Soil Anchors". This office should review and approve the actual testing program and observe and interpret the execution of the testing program.

9.5.1.4. Tieback Installation

The anchors should be installed at angles of 15 to 30 degrees below the horizontal. Caving of the anchor holes at certain locations should be anticipated and provisions should be made available to minimize such caving. The anchors should be filled with grout placed by pumping from the tip out, and the grout should extend from the tip of the anchor to the active wedge. To minimize the potential for caving, we suggest that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill may contain a small amount of cement to allow the sand to be placed by pumping. For post-grouted anchors the anchor may be filled with grout to the face of the shoring provided the tieback strands are enclosed in plastic sheathing.

9.5.1.5. Deflection

It is difficult to accurately predict the amount of deflection of a shored excavation as it largely depends on the quality of construction. It should be realized, however, that some deflection will likely occur. We estimate that this deflection for the constrained wall could be on the order of 1 inch at the top of the shored excavation. If greater deflection occurs during construction, additional bracing or restraint may be necessary to minimize settlement of the nearby improvements. If it is desired to reduce the deflection of the shoring, a greater lateral earth pressure could be used in the shoring design.

9.5.1.6. Construction Staging

The shoring should be constructed utilizing a top-down method of construction whereas the soil is first partially excavated to produce a bench for installation of the topmost row of tiebacks. Following the installation of the tiebacks, the excavation will proceed so that each row of tiebacks can be installed from the excavated bench.

Lagging should be installed simultaneously as the excavation proceeds. In order to facilitate a tight connection between the lagging and the soils and to minimize settlement, any voids left behind the lagging should be filled with cement grout as the excavation advances. In order to continuously support the excavation, tieback installation bench should not be excavated more than 4 feet below the elevation of the centerline of the tieback row. The shoring designer should analyze each stage of tieback installation to ensure that the excavated bench level has an adequate factor of safety.

9.5.1.7. Internal Bracing

Locally, where tiebacks cannot be used, raker bracing may be used to internally brace the soldier pile wall. If used, raker bracing could be supported laterally by temporary concrete footings (deadmen). For design of such temporary footings, poured with the bearing surface normal to the rakers inclined at 45 to 60 degrees from the vertical, a bearing value of 2,000 psf may be used, provided the shallowest point of the footing is at least 1 foot below the lowest adjacent grade. To reduce the movement of the shoring, the rakers should be tightly wedged against the footings and/or shoring system.

Internal bracing can also be provided with struts. The struts can be designed for the earth pressures provided in Table 7 for restrained shoring and shown schematically in Diagram 1.

9.5.2. Soil Nail Wall

A soil nail wall may be considered an option for support of the proposed excavation for the installation of the underground storage facility. The soil nail wall alternative may be more economical for the 25 to 30 feet deep excavations than a soldier pile wall shoring system with tiebacks depending on the actual shoring configuration. Dewatering will be required to perform the excavation and installation of the soil nail wall, and then for subsequent construction. Dewatering should be designed to keep the groundwater elevation at least 4 feet below the bottom of the excavation.

9.5.2.1. Soil Nail Wall Design

Preliminary parameters to be used for the initial soil nail wall design are summarized in Table 8 for the prevailing on-site stiff silty clays for solid bar nails. The design of the soil nail wall should incorporate the proper groundwater conditions developed by the dewatering system.

Table 8
Summary of Soil Nail Design Parameters

Design Parameter	Design Value
Ultimate Bond Stress (for rotary drilled nails)	5 psi
Ultimate Bond Stress (for augered nails)	6 psi
Yield Strength of Reinforcement Steel	60 ksi
Minimum Soil Nail Diameter	6 inches
Design Parameters for Global Stability of Soil Nail Wall	**Design Value**
Design effective friction angle, ϕ	24^{o}
Design effective cohesion, c	200 psf

This office can provide the design of the soil nail wall, if requested.

9.5.2.2. Soil Nail Testing

Soil nail testing should be performed in accordance with the testing guidelines described in Chapter 9 of the FHWA Geotechnical Engineering Circular No.7 – Soil Nail Walls (FHWA-NHI-14-007) under the oversight of the Geotechnical Engineer. The designer must design the appropriate drainage behind the shotcrete so that no hydrostatic pressures develop during construction.

9.5.2.3. Construction Staging

The soil nail wall construction should be performed using top-down method in multiple stages. In the first stage, the vertical excavation will be cut to allow for construction of the top row of soil nails and the shotcrete facing within the highest section of the wall. In the following stages, the soil will be excavated and soil nails installed and the shotcrete facing applied one row of soil nails at a time. In order to continuously support the excavation, the soil nail installation bench should not be excavated more than 4 feet below the elevation of the centerline of the soil nail row.

In general, it is expected that the conditions of the cut face encountered during construction will be favorable, i.e., no large scale or continuous caving will be encountered. However, this does not eliminate the potential for localized problems in cohesionless zones. If localized caving is encountered, it could be handled by reducing the unsupported height at that installation level and by flash coating the surface with shotcrete. If caving is encountered the contractor should not expose more than 10 feet horizontally at a given time. The contractor should install the mesh and apply the shotcrete as soon as possible after the excavation is performed so that the fine-grained soils are not allowed to swell or fall off into the excavation. All voids behind the wall must be filled with shotcrete.

9.5.3. Shoring Performance Monitoring

Some means of monitoring the performance of the shoring system are recommended. The monitoring should consist of periodic visual inspections and lateral and vertical surveying of the tops of the soldier piles or survey monuments installed on top or behind the soil nail wall. This office can provide further recommendations of the monitoring when the design of the shoring system is being finalized. It is recommended that a survey be performed before construction begins and then as the excavation proceeds the monitoring should be performed daily or whenever excavation activities are taking place. In addition, the Contractor should inspect daily the shoring and actively search for presence of cracks or excessive movements and report immediately to the shoring designer and the Geotechnical Engineer.

9.5.4. Irrigation and Drainage Control

It is recommended that while the shoring system is being installed and during its temporary operation no irrigation at the park be allowed within a horizontal distance of 80 feet measured from the top of the excavation to minimize possible buildup of pore water pressures. All excavations should be protected by the Contractor during rain events from overflow at the top. If cracks appear on the ground at the top of the excavation, the contractor must not only monitor the cracks and their extent and inform the Geotechnical Engineer, but the contractor must also provide protection against flow of runoff water into the cracks. The contractor must also protect the toe of the excavation so that no water accumulates at the toe creating a potential for localized softening. It is recommended that drainboard be installed behind the shoring to mitigate any potential pore pressure build up behind the shoring. In addition, a compatible weep drain system should be attached to the drainboard and extended outwards through the shotcrete or the lagging.

9.6. Temporary Dewatering

Depending on the timing and season of the construction of the project, groundwater could potentially be above the excavation bottom. Dewatering will likely be required for the construction of the project. The dewatering system must be designed by the Contractor to lower the groundwater to a depth of at least 4 feet below the bottom of the excavation at all places in order to provide a suitable subgrade that is not wet and workable for installation of the different structures, specially the underground storage facility. If the subgrade is not considered suitable by the Geotechnical Engineer, the Contractor must be ready to implement a subgrade stabilization program including mechanical stabilization, chemical, etc, which must be approved by the Geotechnical Engineer. Dewatering should comply with local regulations and with the appropriate Storm Water Pollution Plan (SWPP). The design of the dewatering system needs to consider the soil variability at the site, specially the likely transition from the upper fine-grained soils to the lower coarse-grained soils at a depth anywhere between approximately 20 to 32 feet. It is important that a Contractor with a wide experience in dewatering be selected.

9.7. Uplift of Buried Structures

Buried structures at a depth greater than 10 feet should be designed to resist uplift forces due to potential buoyancy exerted by a high groundwater depth of about 10 feet. These buoyant forces created by the groundwater need to be accounted to prevent buried structures including pipelines from floating or shifting upward. The designer must consider all the downward and upward forces on the structures and design for the worst-case scenario.

9.8. Foundations

We anticipate that the proposed underground storage reservoir and pump station vault will be supported on either on mat foundations, or on pad footings with concrete slab on-grade established on subgrade prepared in accordance with recommendations provided in "Site Preparation" section of this report. Recommendations for the design and construction of shallow foundations are presented below.

9.8.1. Design Parameters for At-Depth Foundations

Foundations for the underground storage facility, pretreatment unit, and the pump well vault located about 20 to 30 feet below the existing grade should be designed for the anticipated at-depth soil conditions using the geotechnical design parameters presented in Table 9. Footings should be designed and reinforced in accordance with the recommendations of the Structural Engineer and should conform to the 2019 California Building Code.

9.8.2. Design Parameters for At Grade Shallow Foundations

Shallow foundations for at-grade structures should be designed for the anticipated near surface soil conditions using the geotechnical design parameters presented in Table 10. Footings should be designed and reinforced in accordance with the recommendations of the Structural Engineer and should conform to the 2019 California Building Code.

9.8.3. Footings Adjacent to Trenches

The bottom of any trenches that are required for any buried utilities and piping should be kept outside a zone defined by a plane inclined at a gradient of 1(H): 1(V) and projected from the outside bottom edge of any existing or proposed footings. Backfill materials and procedures shall conform to the recommendations provided in the "Site Preparation" and "General Site Grading" sections of this report. If any piping needs to be placed within the zone of influence, the pipes should be designed to account for the increased surcharge from the applied footing pressures and to withstand potential differential settlement between the surcharged and unsurcharged segments of the pipe. Generally, the pipes within the impacted zone should be protected with concrete encasement, utilidors, or other suitable form of protection. This office should be contacted to review any specific utility interaction configurations and their proposed mitigation.

9.8.4. Foundation Construction Observations

To evaluate the presence of satisfactory materials at foundation subgrade, foundation excavations should be observed by a representative of Tetra Tech and be clean of loosened soil and debris before placing steel or concrete. If soft or loose soils or other unsatisfactory materials are encountered, such materials should be removed and replaced with compacted fill prior to pouring the footing.

Table 9
Geotechnical Design Parameters
At-Depth Footing Foundations

Continuous Strip Footings			
Dimensions	• At least 1 foot wide but less than 4 feet wide		
Depth of Embedment **Allowable Bearing Capacity**	• Embedded at least 10 and less than 25 feet below the lowest adjacent grade. • 2,800 psf		
Depth of Embedment **Allowable Bearing Capacity**	• Embedded at least 25 and less than 30 feet below the lowest adjacent grade. • 3,800 psf		
Spread Footings or Pads			
Dimensions (feet)	• Up to 4 feet x 4 feet	• Up to 8 x 8 feet	• Up to 15 x 15 feet
Depth of Embedment **Allowable Bearing Pressure**	• At least 10 feet but less than 25		
	• 4,000 psf	• 2,800 psf	• 1,800 psf
Depth of Embedment **Allowable Bearing Pressure**	• At least 25 feet		
	• 4,500 psf	• 4,500 psf	• 4,500 psf
All Foundations			
Allowable Bearing for Transient Live Loads	• The allowable bearing pressure value may be increased by one-third for transient live loads from wind or seismicity.		
Estimated Settlement	• Approximately 1-inch total settlement. • Approximately 0.5-inch differential settlement between supports or over a distance of 30 feet.		
Allowable **Adhesion at the base** (incorporates Factor of Safety of 1.5)	• 800 psf • Adhesion to be multiplied by the contact area as limited per 2019 CBC Section 1806.3.2.		
Allowable **Lateral Passive Resistance** (incorporates Factor of Safety of 2)	• 75 pcf (EFD) (assumes submerged condition) • The passive resistance derived of the upper 12 inches should be neglected.		
Allowable Combined Lateral Resistance	• The total allowable resistance to lateral loads can be calculated by combining the lateral resistance due to adhesion at the base and the lateral passive resistance. • The passive resistance values may be increased by one-third when considering transient wind or seismic loading		
Uplift Capacity	• The weight of the soil that contributes to the uplift capacity can be estimated as a zone defined by an angle of 30 degrees from the vertical projected from the top edge of the footing to the adjacent grade. • A total unit weight of 120 pcf may be used for the soil. • The lowest depth of embedment from the adjacent grade shall be used in the estimations		

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Table 10
Geotechnical Design Parameters
Shallow Footing Foundations

Continuous Strip Footings		
Dimensions	• At least 1 foot wide but less than 4 feet wide • Minimize footing dimensions by maximizing the bearing pressure to confine and reduce the post-construction swelling of any expansive soils. • Embedded at least 2 feet below the lowest adjacent grade.	
Allowable Bearing Capacity	• 2,200 psf	
Spread Footings or Pads		
Dimensions (feet)	• Up to 4 feet x 4 feet	• Up to 8 x 8 feet
Depth of Embedment	• At least 2 feet	• At least 2 feet
Allowable Bearing Pressure	• 3,500 psf	• 2,400 psf
All Foundations		
Allowable Bearing for Transient Live Loads	• The allowable bearing pressure value may be increased by one-third for transient live loads from wind or seismicity.	
Estimated Settlement	• Approximately 1-inch total settlement. • Approximately 0.5-inch differential settlement between supports or over a distance of 30 feet.	
Allowable Adhesion along concrete – soil interface (incorporates Factor of Safety of 1.5)	• 800 psf • Adhesion to be multiplied by the contact area as limited per 2019 CBC Section 1806.3.2.	
Allowable Lateral Passive Resistance (incorporates Factor of Safety of 2)	• 140 pcf (EFD) • The passive resistance derived of the upper 12 inches should be neglected.	
Allowable Combined Lateral Resistance	• The total allowable resistance to lateral loads can be calculated by combining the lateral resistance due to adhesion at the base and the lateral passive resistance. • The passive resistance values may be increased by one-third when considering transient wind or seismic loading	
Uplift Capacity	• The weight of the soil that contributes to the uplift capacity can be estimated as a zone defined by an angle of 30 degrees from the vertical projected from the top edge of the footing to the adjacent grade. • A total unit weight of 120 pcf may be used for the soil. • The lowest depth of embedment from the adjacent grade shall be used in the estimations	

9.9. Concrete Slab-On-Grade or Mats

The recommendations provided in the "Site Preparation" section of this report and in this section are intended to provide a firm bearing subgrade to help reduce the occurrence of cracks in concrete and associated horizontal separation and vertical offset. However, it should be understood that concrete slabs may still crack due to structural design or detailing, curing, or construction execution even when these recommendations are implemented. If cracking of the concrete is desired to be minimized, the reinforcement, concrete mix, and curing specifications should be designed by the Structural Engineer and Concrete Specialist.

9.9.1. Structure Floor Slab-On-Grade or Mats

Structure floor slab-on-grade and mat foundations for the pump station vault, pretreatment units, or the underground storage facility, if considered, may be designed based on the reference modulus of subgrade reaction k_1 for a 1-foot by 1-foot square plate of 80 pounds per cubic inch. For the on-site silty and clayey soils, the design modulus of subgrade reaction *k* in pci for a concrete rectangular element can be determined as:

$$k = k_1 \frac{1 + 0.5 * \frac{B}{L}}{1.5 * B}$$

Where *B* and *L* are the width and length of the element in feet, respectively, while *B* is no more than 14 times the thickness of the element, i.e., floor slab, and k_1 is as defined above.

In order to assist with initiation of the floor slab design, the slab-on-ground should have a minimum thickness of 5 inches. The minimum reinforcement to reduce separation and offset of potential concrete cracks should consist of No. 4 reinforcing bars spaced at 18 inches on-center, each way, placed in the middle one-third of the section. The slab should be doweled into the perimeter building footings to reduce the potential for differential movement. Reinforcement should be properly placed and supported on blocks or “chairs.” Welded wire mesh reinforcement is not recommended.

Control joints should be constructed in accordance with recommendations from the Structural Engineer and the Architect. For preliminary design considerations, control joints should be provided in all concrete slabs-on-grade as recommended by American Concrete Institute (ACI) guidelines and at a maximum spacing (in feet) of 2 to 3 times of the slab thickness (in inches), but generally no more than 10 feet,. All joints should form approximately square patterns to reduce potential for randomly oriented shrinkage cracks. The control joints should be tooled at the time of the pour or sawcut to ¼ of slab depth within 6 to 8 hours of concrete placement. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this semi-arid region (refer to ACI guidelines).

An allowable adhesion of 600 psf (to be multiplied by the contact area) may be used to account for the lateral resistance generated in the contact between the slabs and the supporting soils. In no case, the lateral resistance can exceed 50 percent of the dead load.

9.9.2. Exterior Slabs

Exterior slabs should be placed on subgrade prepared in accordance with the recommendations provided in the "Site Preparation" section of this report. As indicated above, a Structural Engineer or an Engineer specialized in concrete design should be consulted if cracking of the exterior slabs is to be minimized. As a minimum for exterior walkways, it is recommended that narrow strip concrete slabs, such as sidewalks, be reinforced with at least No. 4 reinforcing bars placed longitudinally at 18 inches on center. Wide exterior slabs should be reinforced with at least No. 4 reinforcing bars placed 18 inches on center, each way. Placement of control joints within exterior slabs should follow the recommendations presented for floor slabs. Reinforcement should extend through the control joints to reduce the potential for differential movement. Control joints should be constructed in accordance with recommendations from the Structural Engineer and Architect.

9.10. Seismic Design Parameters

The seismic design coefficients provided below in Table 11 are based on Chapter 16 Section 1613 of the 2019 CBC. According to ASCE 7-16 Section 11.4.8, a ground motion hazard analysis shall be performed if structures on Site Class D have an S_1 greater than or equal to 0.2 unless the seismic coefficient *Cs* determined by Equation (12.8-2) is used for values of $T \leq 1.5\ T_s$ and taken as equal to 1.5 times the value computed in accordance with either Equation (12..8-3) for $T_L \geq T > 1.5\ T_s$ or Equation (12.8-4) for $T > T_L$.

If a site-specific ground motion hazard analysis is required, Tetra Tech can provide such an analysis. The seismic design coefficients provided below in Table 11 are based on Chapter 16 of the 2019 CBC, and on the information provided by the Structural Engineers Association of California (SEAOC) and the Office of Statewide Health Planning and Development (OSHPD) website application (https://www.seaoc.org/page/seismicdesignmaptool).

Table 11
Site Categorization and 2019 CBC Site Coefficients
Site Latitude 33.942746° and Longitude -118.034187°

Parameter	Design Value
Site Class (Table 20.3-1 ASCE 7)	D*
Short Period Spectral Acceleration Parameter S_s	1.755**
1-sec. Period Spectral Acceleration Parameter S_1	0.626**
Short Period Design Spectral Acceleration Parameter S_{DS}	1.17**
1-sec. Period Design Spectral Acceleration Parameter S_{D1}	0.710***

* Soil profile based on estimated v_{s30} of 300 m/s
** Values obtained from Structural Engineers Association of California (SEAOC) and the Office of Statewide Health Planning and Development (OSHPD) website application, https://www.seaoc.org/page/seismicdesignmaptool based on ASCE7-16 and 2018 International Building Code.
*** See requirements for site-specific ground motions in Section 11.4.8 of ASCE 7-16.

9.11. Lateral Earth Pressures on Underground Walls and Storage Facility

Based on the 2020 for the County of Los Angeles Building Code, which is based on the 2018 California Building Code (CBC) the design of retaining walls higher than 6 feet, as measured from the top of the footing, requires the inclusion of not only static lateral pressures but also of additional seismically induced lateral earth pressures.

The static lateral pressures acting on the proposed on-site underground structures storage and infiltration structures should be calculated based on the recommendations provided in Table 12.

According to the 2019 CBC the dynamic seismic lateral earth pressures on foundation walls and retaining walls should be determined using the design earthquake ground motions. Based on the USGS U.S. Seismic Design Maps website application (http://earthquake.usgs.gov/designmaps/us/application.php), the PGA from the Design Response Spectrum at the site is approximately 0.47g where the design PGA is calculated as 0.4*S_{DS}, where S_{DS} is the risk-targeted, maximum rotated acceleration direction, design response spectrum parameter for short periods. The seismic induced earth pressure increments were estimated using the method recommended by Mikola and Sitar (2013). These recommendations are provided in Table 12. Lateral earth pressures presented in this table are for a level backfill.

Determination of whether the active or at-rest condition is appropriate for design will depend on the flexibility of the walls. In walls with stiff clay backfill that are free to rotate at least 0.01 radians (deflection at the top of the wall of at least 0.01 x H) may be designed for the active condition. Walls that are not capable of this movement should be assumed rigid and designed for the at-rest condition. The effect of any surcharge (dead or live load) located within a 1(H):1(V) plane drawn upward from the heel of the wall footing should be added to the lateral earth pressures.

Suitable backfill materials within a zone immediately the behind the subterranean walls, including the underground storage facility walls, should have a Sand Equivalent of about 30, an Expansion Index of less than 20, and fines content (passing #200 sieve) of less than 15 percent. It is expected that due to the clayey nature of most of the on-site material, the on-site materials will not be generally suitable as a backfill immediately behind. Consequently, a select on-site or import material with an Expansion Index less than 20 or approved non-expansive import material should be used for the backfill within at least 5 feet behind the back of the underground walls. It is expected that additional laboratory testing will be necessary to determine the suitability of the selected on-site or import materials. The select on-site or import materials that are approved as backfill materials should be moisture-conditioned 110 percent of the optimum moisture content, and placed in horizontal lifts not more than 8 inches in uncompacted thickness, and compacted to at least 90 percent of the maximum dry density, as evaluated by the latest version of ASTM D1557.

Table 12
Geotechnical Design Parameters for Subterranean Walls
Lateral Pressures due to Static and Seismic Loads

Active Pressure for Yielding Walls		
Static *active* pressure (psf)	above groundwater	$51z + 0.42Q$
	below groundwater (at depth $z > z_w$)	$51z_w + 89(z - z_w) + 0.42Q$
Active seismic pressure increment (psf)		$15z$
At rest Pressure for Non-yielding Walls		
Static *at-rest* pressure (psf)	above groundwater	$71z + 0.59Q$
	below groundwater (at depth $z > z_w$)	$71z_w + 99(z - z_w) + 0.59Q$
At-rest seismic pressure increment (psf)		$33z$
Lateral Passive Resistance		
Allowable static lateral *passive* pressure (psf) Includes a Factor of Safety of 2	above groundwater	$140z_1$
	below groundwater at depth z_w	$140z_w + 74(z_1 - z_w)$
Ultimate total passive resistance for seismic conditions (psf)	above groundwater	$145\ z_1$
	below groundwater at depth z_w	$145z_w + 75(z_1 - z_w)$

Notes:
- Lateral Pressures due to Seismic Loading are based on a PGA=0.47g for a design response spectrum taken as 2/3 MCER response spectrum. The appropriate total seismic force (active plus seismic increment for yielding walls and at rest plus seismic increment for non-yielding walls) should be calculated be assuming a downward increasing trianglular equivalent fluid pressure distribution. The resulting force should be assumed to act at 1/3 of the height of the wall above the bottom of the wall.
- Pressure based on soil with $\phi = 24^o$, $c = 0$ psf, $\gamma_t = 120$ pcf (above groundwater), $\gamma_t = 125$ pcf (below groundwater)
- The 2019 CBC requires that basement walls be designed for at rest earth pressures for static conditions.

Legend:
z … Depth (ft) below the grade behind the wall – depth measured from the ground surface to the depth where the soil lateral pressure is being evaluated;
z_1 … Depth (ft) below the grade where passive conditions apply, i.e., usually in front of the wall – depth measured from the ground surface to the depth where the soil lateral pressure is being evaluated;
z_w … Depth to groundwater (ft) – depth measured from the ground surface to the groundwater;
Q … Uniform surcharge (psf) within a 1(H):1(V) plane drawn upward from the heel of the wall footing

9.12. Embedded Posts and Poles at Grade

The allowable static lateral soil bearing pressure can be assumed to be at least 140 pcf EFD. The earth pressure value incorporates a Factor of Safety of 2. Isolated poles for uses such as flagpoles or signs and poles used to support buildings that are not adversely affected by a 1/2-inch (12.7 mm) motion at the ground surface due to short-term lateral loads can be designed using lateral soil bearing pressure of 200 pcf EFD. Vertical compressive loading can be resisted utilizing an allowable end bearing pressure of 2,800 psf.

9.12.1. Non-Constrained Case

For the non-constrained case where the pole is not restricted to move at the ground level, the minimum depth of embedment required to resist lateral loads should be determined in accordance with the 2019 CBC Section 1807.3.2.1. Where bare ground (without concrete or asphalt cover) is present adjacent to the foundation, the lateral resistance should be ignored for the upper 12 inches below grade. Therefore, a trapezoidal pressure distribution should be used starting at 12 inches below grade.

9.12.2. Constrained Case

For the constrained case where the pole is restricted from movement at the ground level by encasement in surrounding concrete or similar, the minimum depth of embedment required to resist lateral loads should be determined in accordance with the 2019 CBC, Section 1807.3.2.2.

9.13. Pipeline Design and Construction

Design recommendations for the proposed pipeline trenches and backfill are provided below.

9.13.1. Trench Excavation

Recommendations provided in the "Temporary Slopes and Trench Excavations" section of this report should be followed for design and construction of trenches for the proposed pipelines.

9.13.2. Trench Bottom Preparation

Trench bottom preparation should produce a uniform, firm, and unyielding subgrade. The exposed trench bottom should be probed and accepted by the Geotechnical Engineer. Any particle size greater than 3 inches should be removed. The soils should be scarified to a depth of 6 inches and compacted at a minimum of 110 percent of optimum moisture content to at least 90 percent of the maximum dry density, as evaluated by the latest version of ASTM D1557.

Localized zones of loose and/or unstable soils may be encountered during the grading operations at the trench bottom level and should be overexcavated and recompacted. If loose/soft/wet areas are encountered that are not practical to be excavated and processed, woven geotextile material such as Mirafi RS580i or equivalent should be placed along the trench bottom prior to placement of bedding material.

9.13.3. Trench Backfill

Bedding and pipe zone backfill. The bedding is the material placed in the bottom of the trench on which the pipe is laid. Bedding material should extend at least 6 inches below the bottom of pipe and up to the pipe springline level. The pipe-zone backfill is defined as the area placed above the bedding, around the pipe, and up to at least 12 inches over the pipe. Common types of bedding and pipe-zone backfill material range from native soils to imported sand and gravel, to soil-cement slurry (or flowable fill / controlled low strength material (CLSM)).

Bedding and pipe zone backfill material for the pipelines should consist of clean sand or gravel. The actual selection and suitability of the material should be determined based on the pipe design loading and requirements. The excavated artificial fill materials are not considered suitable bedding and pipe zone backfill material whereas the native alluvial sands are considered to be appropriate. The pipe bedding material should be placed over the full width of the trench. The bedding should be placed upon firm and unyielding subgrade soils approved by the Geotechnical Engineer. After placement of the pipe, the pipe-zone backfill should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the pipe haunches.

The bedding / pipe zone backfill should be placed in horizontal lifts no greater than 6 inches. The bedding and pipe zone backfill should be moisture-conditioned to at least 110 percent of optimum moisture and hand tamped to achieve a density of at least 90 percent of maximum density per ASTM D1557. The use of mechanized compaction equipment within the pipe zone should be carefully controlled/minimized to avoid overstressing or damaging the pipe. As a general guideline, any soils imported to the site for use as bedding and pipe zone fill should be predominantly granular (i.e., fines content less than 5 percent) and the maximum size should not exceed ¾ inches.

General trench backfill. This zone extends from the top of the pipe zone backfill to the finished grade. Approved excavated soil may be used for general trench backfill. If the excavated on-site material is used as the trench backfill, it should be moisture-conditioned to at least 125 percent of optimum moisture content and compacted to at least 90 percent of maximum dry density per the latest version of ASTM D1557. Lift thickness for backfill will be dependent on the type of compaction equipment utilized but should generally be placed in lifts no greater than 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the trench backfill. The upper 12 inches of backfill should be compacted to at least 95 percent of maximum dry density.

Due to the clayey nature of most of the soil above a depth of 20 feet, and the presence of some clay layers below this depth, ponding or jetting of the trench backfill materials will likely not be feasible at this site.

9.13.4. Construction Observations

The Geotechnical Engineer should observe all temporary excavations and construction slopes, as well as the backfill operations so that appropriate modifications to the design criteria presented herein may be recommended, if necessary, due to encountered conditions differing from the design assumptions.

9.13.5. Drainage Control

Surface water should be controlled so that the subgrade of the pipeline is protected even during periods of heavy rainfall. Furthermore, flooding of the trench excavations should be prevented at all times.

9.14. Pavement Sections

New pavements for driveways and parking lots are anticipated to be constructed on the native soils. The recommendations presented below are for pavements constructed on native soils subgrade or on at least 2 feet of backfill soil. For different conditions this office should be contacted. If pavements are to be constructed directly on top of the underground storage facility, the pavement sections should be incorporated into the structural design.

9.14.1. Subgrade Preparation

The subgrade preparation and fill placement in the areas to be paved should conform to the recommendations provided in the "Site Preparation" and "General Site Grading" sections of this report.

9.14.2. Asphalt Concrete Pavement Design

Flexible pavement sections have been evaluated in general accordance with the Caltrans Highway Design Manual method for flexible pavement design using a 20-year design life period. It is estimated that parking lots may be designed for a Traffic Index of 5. If fire access is required, a Traffic Index of 5 or 6 is typically considered acceptable by regulatory agencies. These assumed TI values will need to be confirmed during design. Based on the prevailing on-site subgrade clayey sand soils R-value of 10 was assumed. The resulting recommended pavement sections are presented in Table 13.

Table 13
Flexible Pavement Sections

Location	R-Value	Assumed Traffic Index	Asphalt Concrete (inches)	Aggregate Base (inches)	Full Depth Asphalt Concrete Alternative
Parking / drive aisles	10	5 or less	3.0	9.0	7.5
Light / moderate traffic		6	3.5	11.5	9.0
Residential Streets		7	4.0	14.5	10.5

Asphalt concrete and aggregate base should conform to the Specifications for Public Works Construction (Green Book) Sections 203-6 and 200-2, respectively. The aggregate base course should be compacted to 95 percent or more of the maximum dry density, as evaluated by the latest version of ASTM D1557.

9.14.3. Pavement Construction Observations

The preparation of the pavement subgrade and the placement of base course and pavement sections should be observed by Tetra Tech personnel. Careful observation is recommended to evaluate that the pavement subgrade is consistent with the design assumptions and that it is uniform and uniformly compacted and that the recommended pavement and base course thickness are achieved. Paved areas should be properly sloped, and surface drainage facilities should be established to reduce water infiltration into the pavement subgrade. Curbing located adjacent to paved areas should be founded in the soil subgrade in order to provide a cutoff to reduce water infiltration into the base course.

9.15. Soil Corrosion

The corrosion potential of the on-site materials to buried steel and concrete was evaluated based on laboratory testing on 3 representative soil samples from previous explorations. Table 14 below presents the results of the corrosivity testing.

Table 14
Corrosivity Test Results

Boring	Sample ID	Depth (feet)	pH	Resistivity (ohm-cm)	Chlorides (ppm/%)	Soluble Sulfate Content in Soil (ppm/%)
B-1 (Ninyo and Moore, 2015)	n/a	6-10	7.0	950	155/0.0155	220/0.022 Category S0 per 2019 CBC
B1 (LACDPW, 2018)	2B	7.5-9	5.77	500	52/0.0052	55/0.0055 Category S0 per 2019 CBC
B1 (LACDPW, 2018)	6B	21.5-23.5	5.83	500	21/0.0021	346/0.0346 Category S0 per 2019 CBC

Per 2019 CBC/ 2018 IBC, Section 1904.1, concrete subject to exposure to sulfates shall comply with the requirements set forth in ACI 318. Based on the measured water-soluble sulfate results the exposure of buried concrete to sulfate attack should be considered "not a concern", i.e., exposure class S0 per ACI 318, Table 19.3.1.1. Consequently, injurious sulfate attack is not anticipated for concrete with a minimum 28-day compressive strength of 2,500 psi. Per ACI 318, Section 19.3.1.1 the maximum permitted amount of water-soluble chloride ions incorporated into the concrete depends on the degree of exposure to an anticipated external source of moisture and chlorides. Additional information on the effects of chlorides on the corrosion of steel reinforcement is provided in ACI 201.2R which provides guidance on concrete durability and ACI 222R which provides guidance on factors that impact corrosion of metals in concrete.

The evaluation of potential for corrosion of buried metals was based on the minimum resistivity per NACE (1984) and our experience with similar soils. The on-site soils are anticipated to likely have a "corrosive" potential to buried ferrous metals. A corrosion specialist should be consulted regarding suitable types of piping and necessary protection for underground metal conduits. The corrosion potential of the on-site soils should be verified during construction for each encountered soil type. Imported fill materials should be tested prior to placement to confirm that their corrosion potential is not more severe than the one assumed for the project.

9.16. Drainage Control

The intent of this section is to provide general information regarding the control of surface water. The control of surface water is essential to the satisfactory performance of the building construction and site improvements. Surface water should be controlled so that conditions of uniform moisture are maintained beneath and adjacent to the structure, even during periods of heavy rainfall. The following recommendations should be considered as minimal.

- Ponding and areas of low flow gradients should be avoided.
- Paved surfaces within 10 feet from the building foundation should be provided with a gradient of at least 2 percent sloping away from improvements.
- Bare soil, e.g., planters, within 10 feet of the structure should be sloped away from the improvement at a gradient of 5 percent.
- Positive drainage devices, such as graded swales, paved ditches, and/or catch basins should be employed to accumulate and convey water to appropriate discharge points.
- Concrete walks and flatwork should not obstruct the free flow of surface water.
- Area drains should be recessed below grade to allow free flow of water into the basin.
- Enclosed raised planters should be sealed at the bottom and provided with an ample flow gradient to a drainage device. Recessed planters and landscaped areas should be provided with area inlet and subsurface drain pipes.
- Planters should not be located immediately adjacent to structures. If planters are to be located adjacent to a structure, they should be positively sealed, should incorporate a subdrain, and should be provided with free discharge capacity to a drainage device.
- Planting areas at grade should be provided with positive drainage. Wherever possible, the grade of exposed soil areas should be established above adjacent paved grades. Drainage devices and curbing should be provided to prevent runoff from adjacent pavement or walks into planted areas.
- Gutter and downspout systems should be provided to capture discharge from roof areas. The accumulated roof water should be conveyed to an appropriate disposal or stormwater storage area by a pipe or concrete swale system.
- Landscape watering should be performed judiciously to preclude either soaking or desiccation of soils. The watering should be such that it just sustains plant growth without excessive infiltration. Sprinkler systems should be checked periodically to detect leakage and irrigation efforts should be reduced or halted during the rainy season.

10. GENERAL SITE GRADING RECOMMENDATIONS

The intent of this section is to provide general information regarding the site grading. Site grading operations should conform with applicable local building and safety codes and to the rules and regulations of those governmental agencies having jurisdiction over the subject construction.

The grading contractor is responsible for notifying governmental agencies, as required, and a representative of the Geotechnical Engineer at the start of site cleanup, at the initiation of grading, and any time that grading operations are resumed after an interruption. Each step of the grading should be accepted in a specific area by a representative of the Geotechnical Engineer, and where required, should be approved by the applicable governmental agencies prior to proceeding with subsequent work.

The following site grading recommendations should be regarded as minimal. The site grading recommendations should be incorporated into the project plans and specifications.

1. Prior to grading, existing vegetation, trash, surface structures and debris should be removed and disposed off-site at a legal dumpsite. Any existing utility lines, or other subsurface structures which are not to be utilized, should be removed, destroyed, or abandoned in compliance with current governmental regulations.

2. Subsequent to cleanup operations, and prior to initial grading, a reasonable search should be made for subsurface obstructions and/or possible loose fill or detrimental soil types. This search should be conducted by the contractor, with advice from and under the observation of a representative of the Geotechnical Engineer.

3. Prior to the placement of fill or foundations within the building area, the site should be prepared in accordance with the recommendations presented in the section “Site Preparation” of this report. All undocumented fill or disturbed soils within the building areas should be removed and processed as recommended by the representative of the . Geotechnical Engineer

4. The exposed subgrade and/or excavation bottom should be observed and approved by a representative of the Geotechnical Engineer for conformance with the intent of the recommendations presented in this report and prior to any further processing or fill placement. <u>It should be understood that the actual encountered conditions may warrant excavation and/or subgrade preparation beyond the extent recommended and/or anticipated in this report.</u>

5. On-site inorganic granular soils that are free of debris or contamination are considered suitable for placement as compacted fill. Any rock or other soil fragments greater than 6 inches in size should not be placed within 5 feet of the foundation subgrade.

6. Any imported fill material required for backfill or grading should be tested and approved prior to delivery to the site.

7. Visual observations and field tests should be performed during grading by a representative of the Geotechnical Engineer. This is necessary to assist the contractor in obtaining the proper

moisture content and required degree of compaction. Wherever, in the opinion of a representative of the Geotechnical Engineer, an unsatisfactory condition is being created in any area, whether by cutting or filling, the work should not proceed in that area until the condition has been corrected.

11. DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical review of plans and specifications is of paramount importance in engineering practice. The poor performance of many structures has been attributed to inadequate geotechnical review of construction documents. Additionally, observation and testing of the subgrade will be important to the performance of the proposed development. The following sections present our recommendations relative to the review of construction documents and the monitoring of construction activities.

11.1. Plans and Specifications

The design plans and specifications should be reviewed and approved by the Geotechnical Engineer prior to bidding and construction, as the geotechnical recommendations may need to be re-evaluated in the light of the actual design configuration and loads. This review is necessary to evaluate whether the recommendations contained in this report have been incorporated into the project plans and specifications as intended.

11.2. Construction Monitoring

Site preparation, pile installation, assessment of imported fill materials, fill placement, and other site grading operations should be observed and tested. The subgrade soils exposed during the construction may differ from those anticipated in the preparation of this report. Continuous observation by a representative of the Geotechnical Engineer should be implemented during construction to allow for evaluation of the soil conditions as they are encountered, and to provide the opportunity to recommend appropriate revisions as needed.

12. STATEMENT 111

Based on the data and evaluations presented in the report, it is the opinion of Tetra Tech that the subject project site for the proposed Adventure Park facilities will be safe against hazards from future landsliding, settlement or slippage and that the proposed grading construction as long as the recommendations provided herein are implemented will have no adverse impact on the geologic stability of property outside of the project site.

13. LIMITATIONS

The recommendations and opinions expressed in this report are based on Tetra Tech's review of background documents and on information obtained from the Ninyo and Moore (2015) investigation, the LACDPW (2018) investigation and the current geotechnical investigation. It should be noted that this study did not evaluate the presence of hazardous materials although the results of analytical testing for one soil sample are presented herein.

Due to the limited nature of the field explorations, conditions not observed and described in this report may be present on the site. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation and laboratory testing can be performed upon request. It should be understood that conditions different from those anticipated in this report may be encountered during grading operations, for example, the extent of unsuitable soil and the associated additional effort required to mitigate them.

Site conditions can change with time as a result of natural processes or the activities of man. Changes to the applicable laws, regulations, codes, and standards of practice may occur as a result of government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Tetra Tech has no control. Therefore, this report should be reviewed and recertified by Tetra Tech if it were to be used for a project design commencing more than one year after the date of issuance of this report.

Tetra Tech's recommendations for this site are dependent upon verification of the actual encountered field conditions, appropriate quality control of grading operations including shoring installation, overexcavation, processing and replacement of the on-site materials, and foundation construction. Accordingly, the recommendations are made contingent upon the opportunity for Tetra Tech to observe all aspects of subgrade preparation for the proposed construction. If parties other than Tetra Tech are engaged to provide such services, such parties are assuming complete responsibility as the Geotechnical Engineer of Record for the project and implicitly concur with the recommendations provided in this report or may provide alternative recommendations.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Tetra Tech should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. Reliance by others on the data presented herein or for purposes other than those stated in the text is authorized only if so permitted in writing by Tetra Tech. It should be understood that such an authorization may incur additional expenses and charges.

Tetra Tech has endeavored to perform its evaluation using the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical professionals with experience in this area in similar soil conditions. No other warranty, either expressed or implied, is made as to the conclusions and recommendations contained in this report.

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Figures

TETRA TECH BAS GEOSCIENCE

21700 Copley Drive, Diamond Bar, CA 91765
TEL 909.860.7777 www.tetratech.com

ADVENTURE PARK - WHITTIER, CA

PROJECT LOCATION MAP

JOB NO.	TET 20-179E
DATE	MAR 2020
DRAWN BY	TAC
CHECKED BY	FC

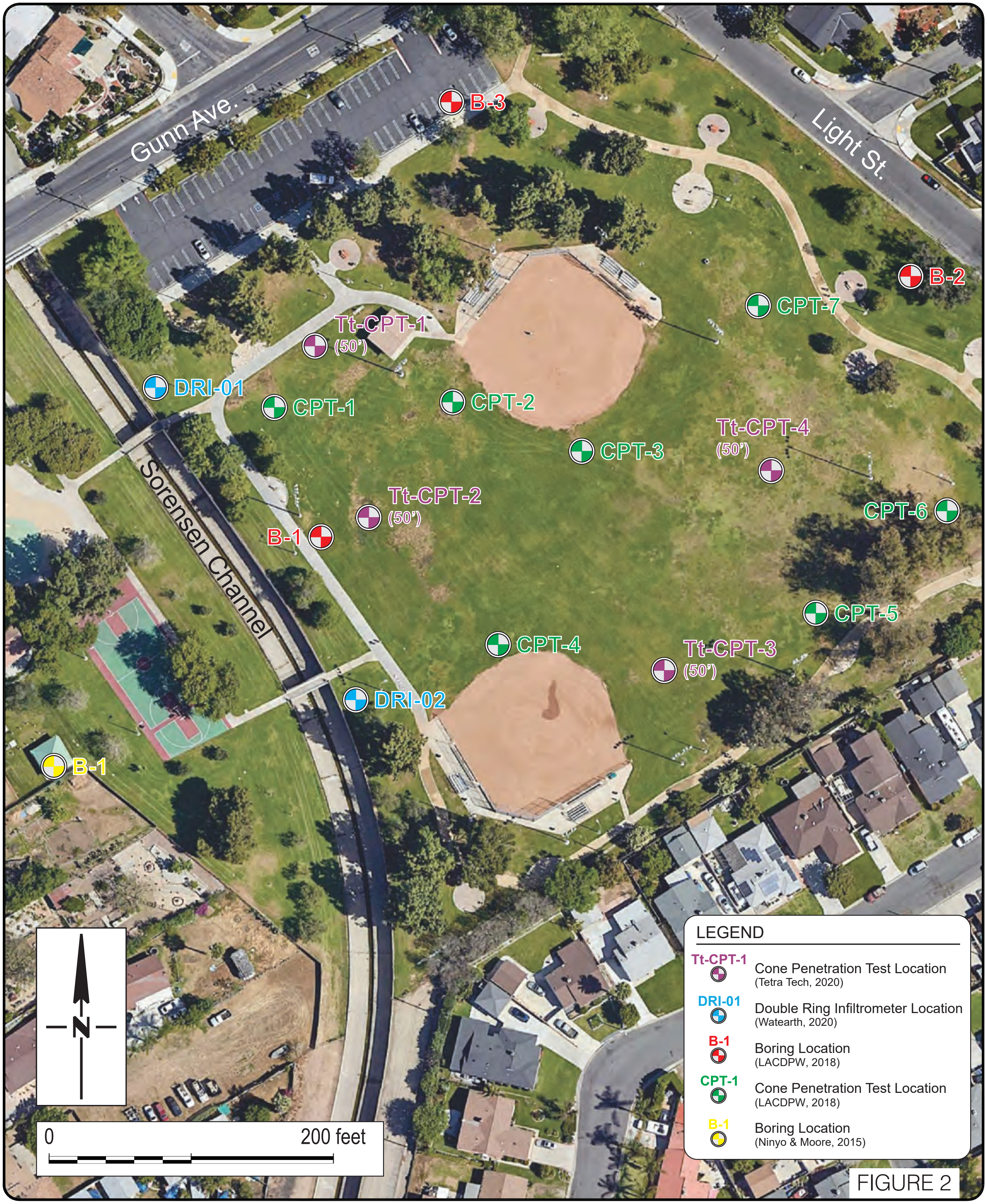

FIGURE 2

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ADVENTURE PARK - WHITTIER, CA

SITE LAYOUT,
BORING AND CPT LOCATION MAP

JOB NO.	TET 20-179E
DATE	MAR 2020
DRAWN BY	SCM
CHECKED BY	FC

Image referenced from: Geologic Map of the Whittier and La Habra quadrangles (western Puente Hills), Los Angeles and Orange Counties, California, Dibblee, T.W. and Ehrenspeck, H.E., 2001

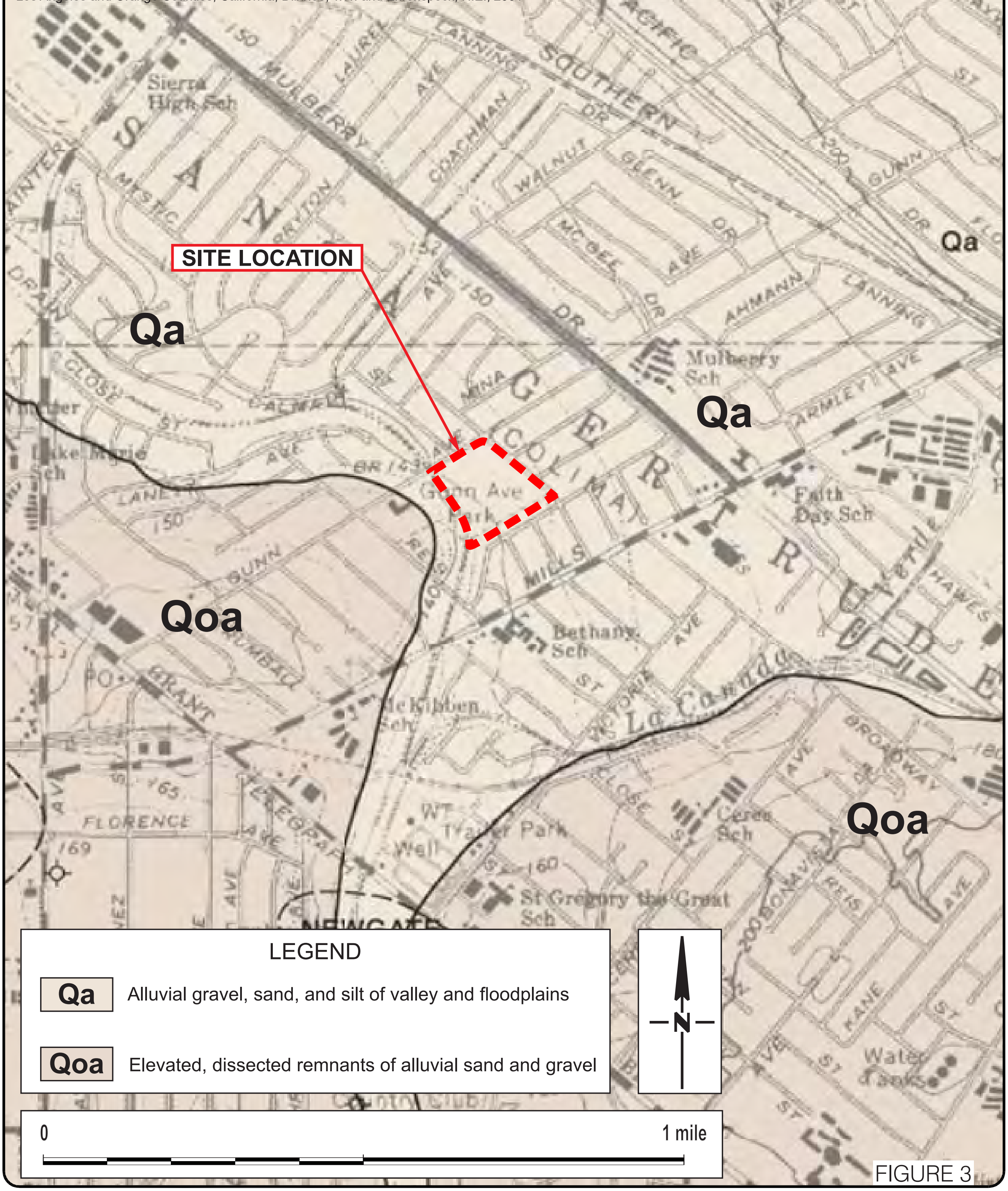

FIGURE 3

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ADVENTURE PARK - WHITTIER, CA

GEOLOGIC MAP

JOB NO.	TET 20-179E
DATE	MAR 2020
DRAWN BY	TAC
CHECKED BY	FC

Image referenced from: California Geological Survey, Seismic Hazard Zone Report
for the Whittier 7.5-Minute Quadrangle, Plate 1.2, 1998

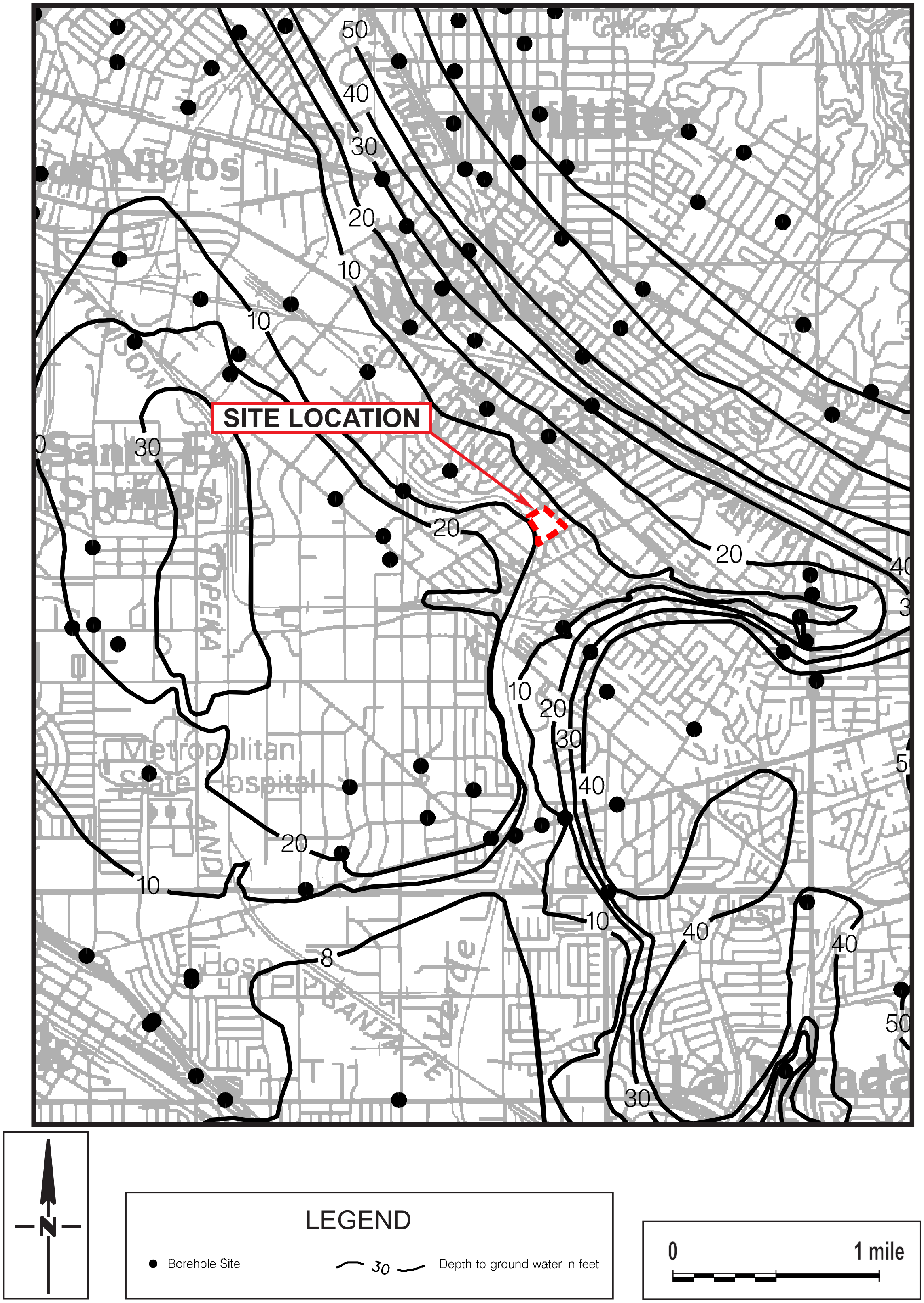

FIGURE 4

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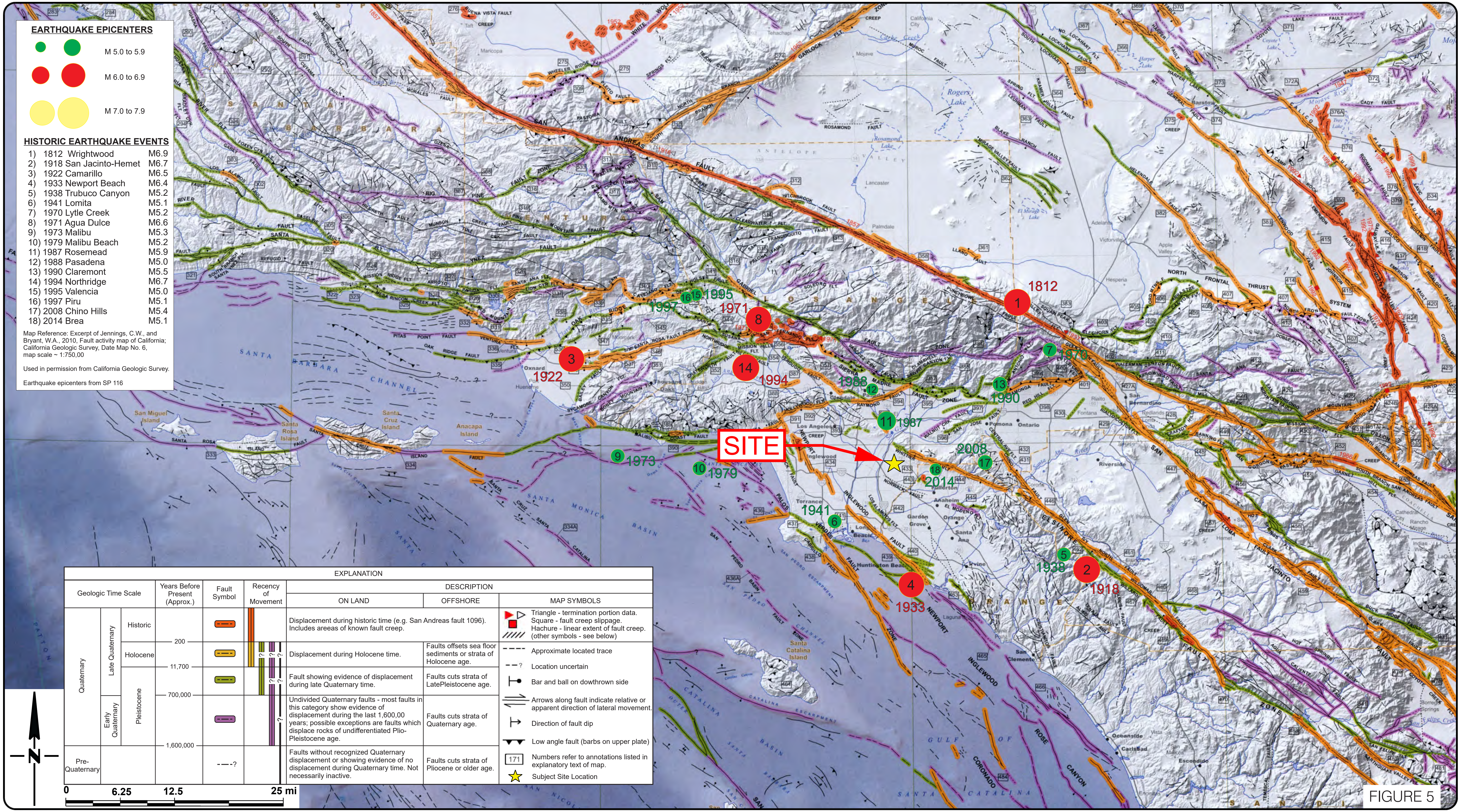
ADVENTURE PARK - WHITTIER, CA	JOB NO. TET 20-179E
HISTORIC HIGH GROUNDWATER MAP	DATE MAR 2020
	DRAWN BY TAC
	CHECKED BY FC



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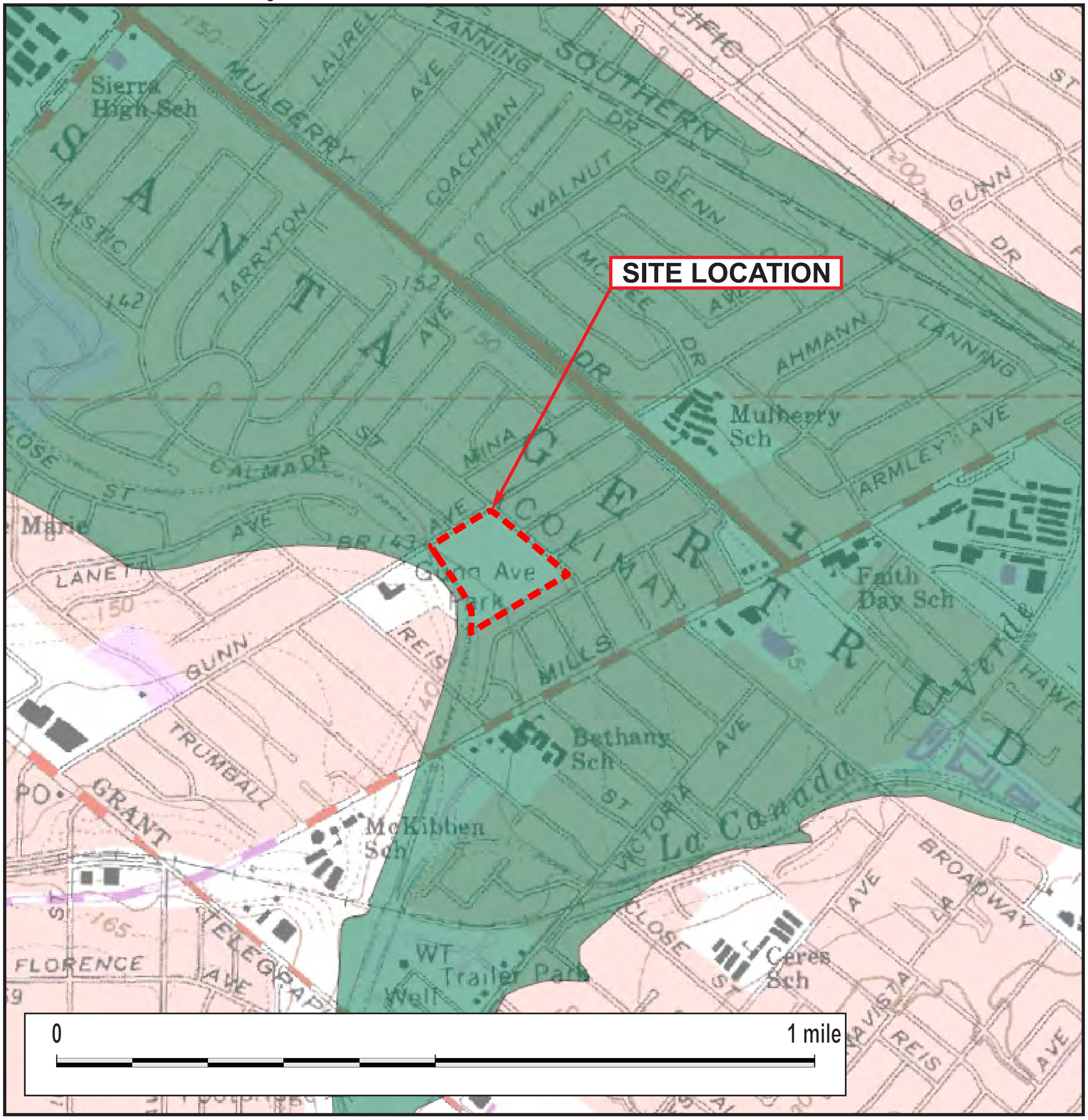
ADVENTURE PARK - WHITTIER, CA

REGIONAL FAULTS AND SEISMICITY MAP

JOB NO.	TET 20-179E
DATE	MAR 2020
DRAWN BY	TAC
CHECKED BY	FC

Image referenced from: California Geological Survey, Earthquake Fault and Seismic Hazard Zones, Whittier Quadrangle, March 25, 1999



SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

N

FIGURE 6

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ADVENTURE PARK - WHITTIER, CA

SEISMIC HAZARD ZONES MAP

JOB NO.	TET 20-179E
DATE	MAR 2020
DRAWN BY	TAC
CHECKED BY	FC

Appendix A

Logs of Previous Exploratory Borings

Los Angeles County Department of Public Works
Geotechnical and Materials Engineering Division
900 South Fremont Avenue, Alhambra, CA 91803

BORING NUMBER B1

CLIENT Stormwater Compliance Division
PROJECT NAME Adventure Park Multi-Benefit Project
PROJECT NUMBER F21816i07
PROJECT LOCATION 10130 S. Gunn Ave, Whittier

Dates(s) Drilled	**6/20/2017**	Boring Location	**33°56'33.44" N, 118°02'06.62" W**	Logged By	**KM**	Checked By	**WM**
Drilling Contractor	**LACDPW SWMD**	Drill Bit Size	**6**	Approx. Surf. Elevation (ft)	**139**	Drilled Depth (ft)	**31.5**
Drilling Method	**Hollow Stem Auger**	Boring Diameter	**6**	Depth to Groundwater	**22**	Inclination/ Bearing (°)	**90**
Drill Rig Type	**CME 75**	Hammer Description	**Autohammer**	Sample Type(s)			
Notes/ Comments							

DEPTH (ft)	GRAPHIC	MATERIAL DESCRIPTION 1) USCS Class. 2) Density/Stiff 3) Color 4) Moisture 5) Percentage (Gravels,Sands,Fines) 6) Grain Size/Percent 7) Other (Mineral Content, Discoloration, Odor, etc.)	SAMPLE TYPE NUMBER	RECOVERY (in)	BLOW COUNTS (N VALUE)	COMMENTS 1) Rig Behavior 2) Air Monitoring 3) Pocket Pen/ Torvane	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**GRASS** **FILL: SILTY SAND (FILL)** medium dense, dark brown, moist, medium to coarse grained, scattered roots										
		ALLUVIUM: FAT CLAY (CH) medium dense, dark brown, moist, some silts										
5		**SILTY CLAY** medium dense, brown, moist pocket of SILTY SAND (SM), alternating dark and light brown	MC 1R	100	2-3-5 (8)							
		@8': layer of SILTY CLAY, medium dense, brown	AU 2B									
10		**CLAYEY SILT (ML)** medium dense, dark brown, moist, interbedded brown to light brown SILTY SAND (SM)	SPT 3S		1-2-2 (4)							
		@13': layer of LEAN CLAY (CL) with some SILT (ML), dark brown										
15		**SILTY CLAY (CL-ML)** medium stiff, brown, moist pocket of very fine grained SILTY SAND (SM), dark grey	MC 4R	100	3-3-4 (7)		94.9	23.9				50
20		@20': very stiff	SPT 5S		1-4-15 (19)							
		SILTY SAND (SM) medium dense, brown, wet	AU 6B									76.2
25		**SILTY CLAY (CL-ML)** **CLEAN SAND (SP)** medium dense, brown, wet	MC 7R	0	2-13-19 (32)		100.1	15.9				
30		**FAT CLAY (CH)** stiff, light brown to yellowish orange, very moist	SPT 8S		2-3-5 (8)							92
		End of boring at 31.5 feet.										

Los Angeles County Department of Public Works
Geotechnical and Materials Engineering Division
900 South Fremont Avenue, Alhambra, CA 91803

BORING NUMBER B2

CLIENT Stormwater Compliance Division

PROJECT NAME Adventure Park Multi-Benefit Project

PROJECT NUMBER F21816i07

PROJECT LOCATION 10130 S. Gunn Ave, Whittier

Dates(s) Drilled	**6/20/2017**	Boring Location	**33°56'33.44" N, 118°02'01.10" W**	Logged By	**KM**	Checked By	**WM**
Drilling Contractor	**LACDPW SWMD**	Drill Bit Size	**6**	Approx. Surf. Elevation (ft)	**146**	Drilled Depth (ft)	**31.5**
Drilling Method	**Hollow Stem Auger**	Boring Diameter	**6**	Depth to Groundwater	**27.5**	Inclination/ Bearing (°)	**90**
Drill Rig Type	**CME 75**	Hammer Description	**Autohammer**	Sample Type(s)			
Notes/ Comments							

DEPTH (ft)	GRAPHIC	MATERIAL DESCRIPTION 1) USCS Class. 2) Density/Stiff 3) Color 4) Moisture 5) Percentage (Gravels,Sands,Fines) 6) Grain Size/Percent 7) Other (Mineral Content, Discoloration, Odor, etc.)	SAMPLE TYPE NUMBER	RECOVERY (in)	BLOW COUNTS (N VALUE)	COMMENTS 1) Rig Behavior 2) Air Monitoring 3) Pocket Pen/ Torvane	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		**GRASS** **FILL: SILTY SAND (SM)** medium dense, brown, moist, scattered roots										
		ALLUVIUM: SILTY CLAY (CL-ML) medium dense, brown to dark brown, moist										
5		**FAT CLAY (CH)** very stiff, brown to dark brown, moist	MC 1R	100	4-10-15 (25)							
10		**SILT (ML)** stiff, red-orange brown, slightly moist, trace of sand	SPT 2S		3-8-12 (20)							
15		**SILTY CLAY (CL-ML)** stiff, reddish brown, slightly moist	MC 3R	100	3-7-10 (17)							
20		@20': stiff, light brown	SPT 4S		2-4-7 (11)		83.6	19.9				71.9
25		**SILTY SAND** dense, brown, grey, moist	MC 5R	100	7-13-25 (38)							
		@27.5': wet										
30		**WELL-GRADED SAND (SW)** medium dense, greyish brown, wet	SPT 6S		5-14-16 (30)							1.4
		End of boring a 31.5 feet.										

Los Angeles County Department of Public Works
Geotechnical and Materials Engineering Division
900 South Fremont Avenue, Alhambra, CA 91803

BORING NUMBER B3

CLIENT Stormwater Compliance Division
PROJECT NAME Adventure Park Multi-Benefit Project
PROJECT NUMBER F21816i07
PROJECT LOCATION 10130 S. Gunn Ave, Whittier

Dates(s) Drilled	6/21/2017	Boring Location	33°56'36.84" N, 118°02'05.34" W	Logged By	KM	Checked By	WM
Drilling Contractor	LACDPW SWMD	Drill Bit Size	6	Approx. Surf. Elevation (ft)	145	Drilled Depth (ft)	51.5
Drilling Method	Hollow Stem Auger	Boring Diameter	6	Depth to Groundwater	28.5	Inclination/ Bearing (°)	90
Drill Rig Type	CME 75	Hammer Description	Autohammer	Sample Type(s)			
Notes/ Comments							

DEPTH (ft)	GRAPHIC	MATERIAL DESCRIPTION 1) USCS Class. 2) Density/Stiff 3) Color 4) Moisture 5) Percentage (Gravels,Sands,Fines) 6) Grain Size/Percent 7) Other (Mineral Content, Discoloration, Odor, etc.)	SAMPLE TYPE NUMBER	RECOVERY (in)	BLOW COUNTS (N VALUE)	COMMENTS 1) Rig Behavior 2) Air Monitoring 3) Pocket Pen/ Torvane	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
0		**3" ASPHALT, 7" BASE (Northeast Parking Lot)**										
		ALLUVIUM: SILT (ML) medium dense, black with orange veins, moist, little clay, little plasticity										
5		@5': dark brown @5.5': black to dark brown, some plasticity	MC 1R	100	1-6-10 (16)		103	18.1				84.5
10		**FAT CLAY (CH)** stiff, brown, moist	SPT 2S		3-5-7 (12)		97.6	17.9				83.5
15		@15': pocket of black FAT CLAY (CH)	MC 4R	100	4-7-9 (16)							
20		@20': hard, light brown, dry, brittle	SPT 5S		2-36-29 (65)							
25		**SANDY SILT (ML)** dense, greyish brown, moist, very fine sand	MC 6R	100	3-10-14 (24)		105.1	18.1				89.8
30		**CLEAN SAND (SW)** medium dense, light brown to brown and greyish brown, wet	SPT 7S		2-7-19 (26)							
35												

(Continued Next Page)

CLIENT Stormwater Compliance Division
PROJECT NAME Adventure Park Multi-Benefit Project
PROJECT NUMBER F21816i07
PROJECT LOCATION 10130 S. Gunn Ave, Whittier

DEPTH (ft)	GRAPHIC	MATERIAL DESCRIPTION 1) USCS Class. 2) Density/Stiff 3) Color 4) Moisture 5) Percentage (Gravels,Sands,Fines) 6) Grain Size/Percent 7) Other (Mineral Content, Discoloration, Odor, etc.)	SAMPLE TYPE NUMBER	RECOVERY (in)	BLOW COUNTS (N VALUE)	COMMENTS 1) Rig Behavior 2) Air Monitoring 3) Pocket Pen/ Torvane	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
35		**CLEAN SAND (SW)** medium dense, light brown to brown and greyish brown, wet *(continued)* @35': some pea-sized gravel @36.5': small pocket of clay at tip of sampler	MC 8R	100	2-7-19 (26)		121.9	4.9				1.4
40		@40': brown to greyish brown	SPT 9S		6-11-9 (20)							0.2
												1
45		@45': fine to coarse grained sand	MC 10R	100	12-33-38 (71)							0.2
50			SPT 11S		7-10-14 (24)							0.2
		FAT CLAY (CH) stiff, brown, wet End of boring at 51.5 feet.										

DATE DRILLED 3/10/15 BORING NO. B-1

GROUND ELEVATION 150' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Diameter Hollow-Stem Auger (CME 75) (Geoboden)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY GTF

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Dark brown, moist, medium dense, silty fine to medium SAND; scattered roots and grass.
							SM	ALLUVIUM: Light brown, moist, medium dense, silty, fine to medium SAND.
							SC	Dark brown, moist, medium dense, clayey fine to medium SAND; scattered roots.
			28				ML	Dark yellowish brown, moist, medium dense, fine sandy SILT; trace clay.
								Trace medium sand; scattered caliche.
							CL-SC	Grayish brown, moist, very stiff, silty CLAY with pockets of light brown, moist, medium dense, clayey fine to coarse SAND with gravel.
10			15	19.2				
			20	10.5			ML	Dark yellowish brown, moist, medium dense, fine sandy SILT; trace medium and coarse sand; some grayish brown mottling.
20			35	1.7			SW-SM	Light brown, moist, dense to very dense, well graded SAND with silt; some fine gravel.
			75	2.2	120.7			Scattered gravel up 1-inch in diameter.
30			50	9.5				Scattered gravel up to 1-1/2-inch in diameter.
								Wet.
			23	15.8				Less gravel.
				16.3			ML	Dark reddish brown, wet, hard, clayey SILT; some fine sand; micaceous.
40								

Ninyo & Moore

BORING LOG

ADVENTURE PARK - UPPER SAN GABRIEL RIVER EWMP
LOS ANGELES COUNTY, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107900001	6/15	A-1

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			18	21.3			ML	ALLUVIUM: (Continued) Reddish brown and grayish brown (mottled), wet, very stiff, clayey SILT; some fine sand.
			26	32.8				Dense; silt with fine SAND; finely laminated; trace medium to coarse sand; no clay; gravel in shoe.
								Total Depth = 46.5 feet. Groundwater encountered at approximately 32 feet during drilling and measured at approximately 31 feet 30 minutes after drilling Backfilled shortly after drilling on 3/10/15. Notes: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
50								
60								
70								
80								

DATE DRILLED 3/10/15 BORING NO. B-1

GROUND ELEVATION 150' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow-Stem Auger (CME 75) (Geoboden)

DRIVE WEIGHT 140 lbs. (Auto-Trip Hammer) DROP 30"

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY GTF

Ninyo & Moore

BORING LOG

ADVENTURE PARK - UPPER SAN GABRIEL RIVER EWMP
LOS ANGELES COUNTY, CALIFORNIA

PROJECT NO.	DATE	FIGURE
107900001	6/15	A-2

Appendix B

Logs of Previous Cone Penetration Tests (CPTs)

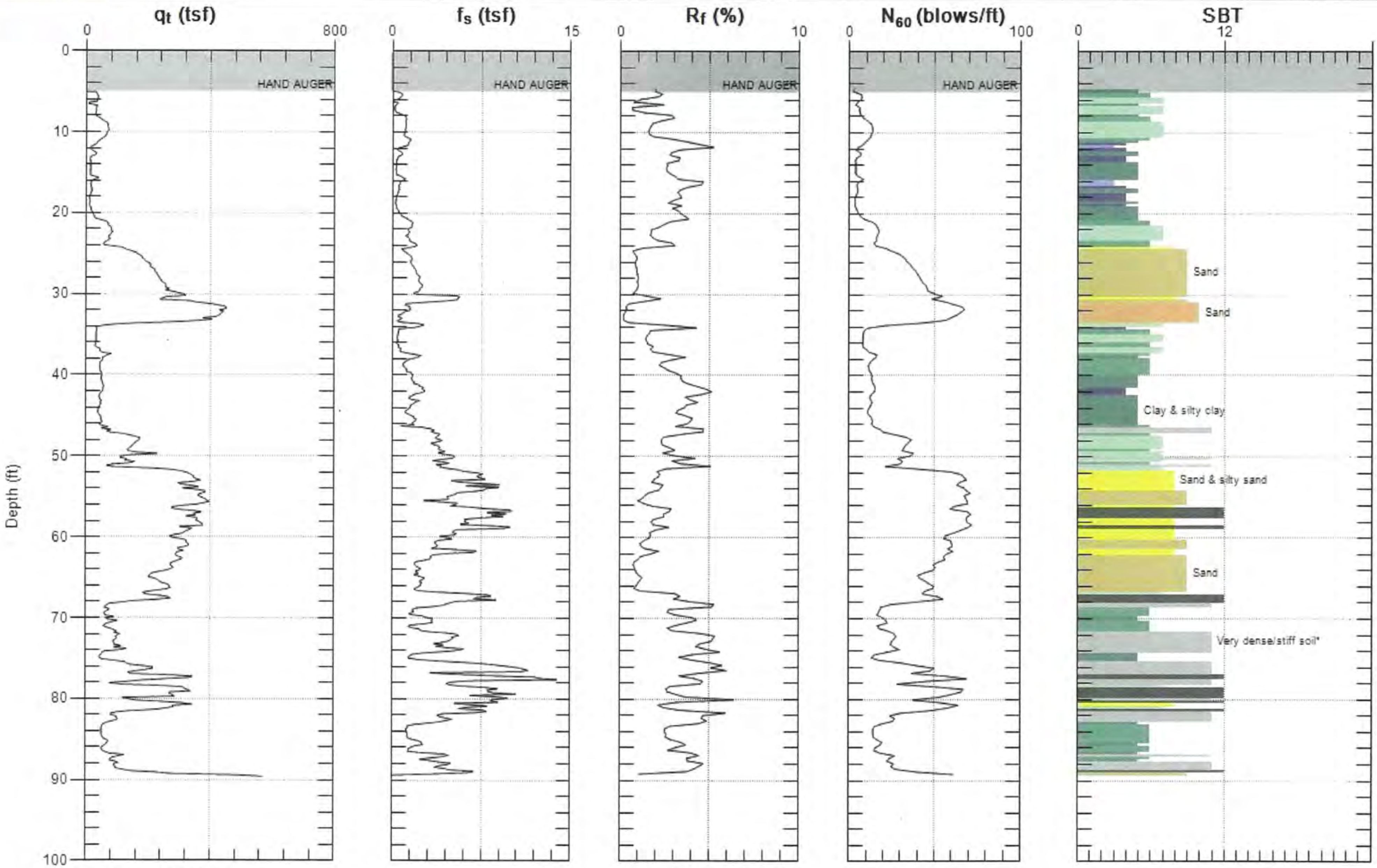
LACDPW

Site: ADVENTURE PARK

Sounding: CPT-1

Engineer: K.MENDEZ

Date: 12/27/16 07:22



Max. Depth: 89.567 (ft)
Avg. Interval: 0.328 (ft)

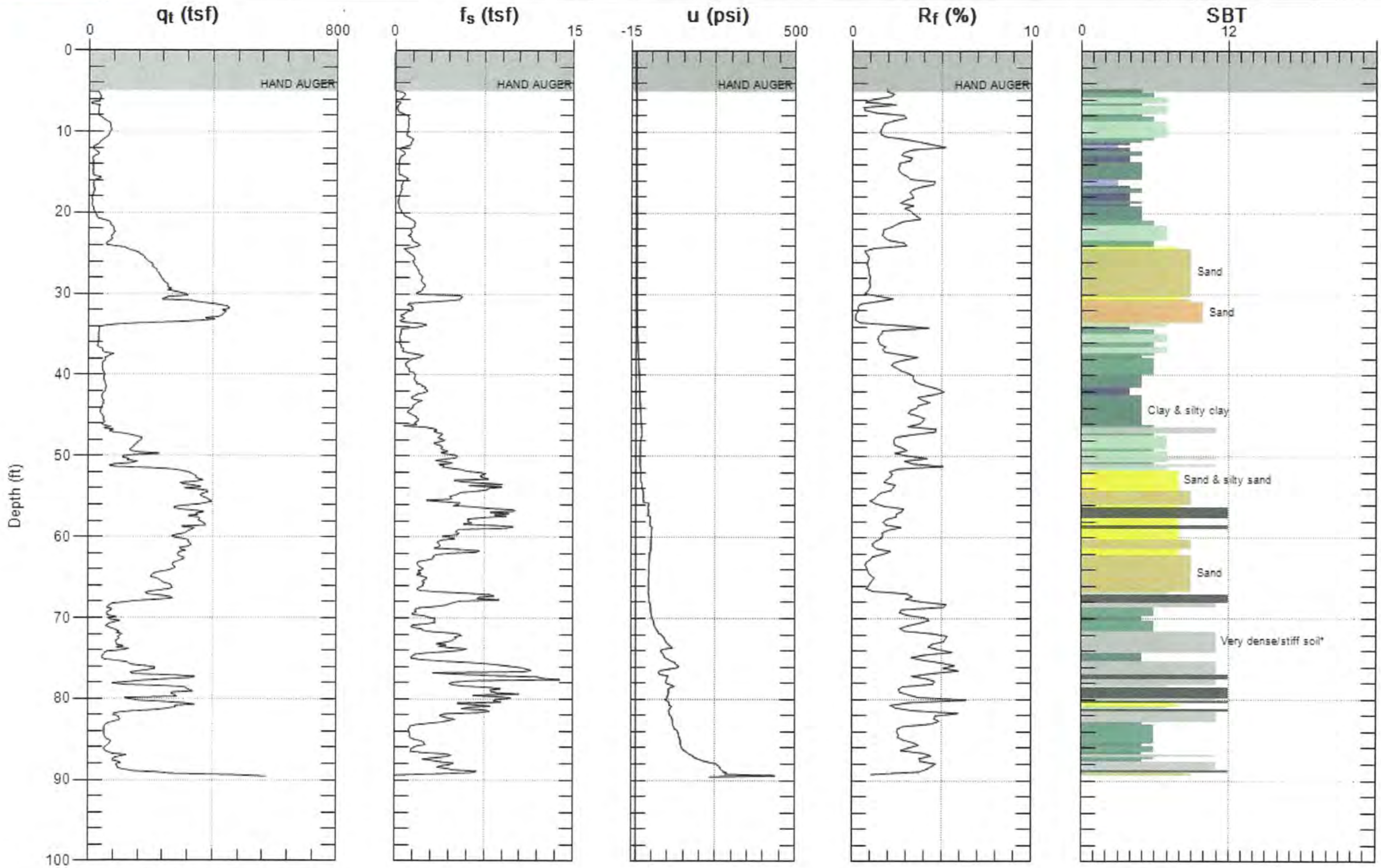
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK
Sounding: CPT-1

Engineer: K.MENDEZ
Date: 12/27/16 07:22



Max. Depth: 89.567 (ft)
Avg. Interval: 0.328 (ft)

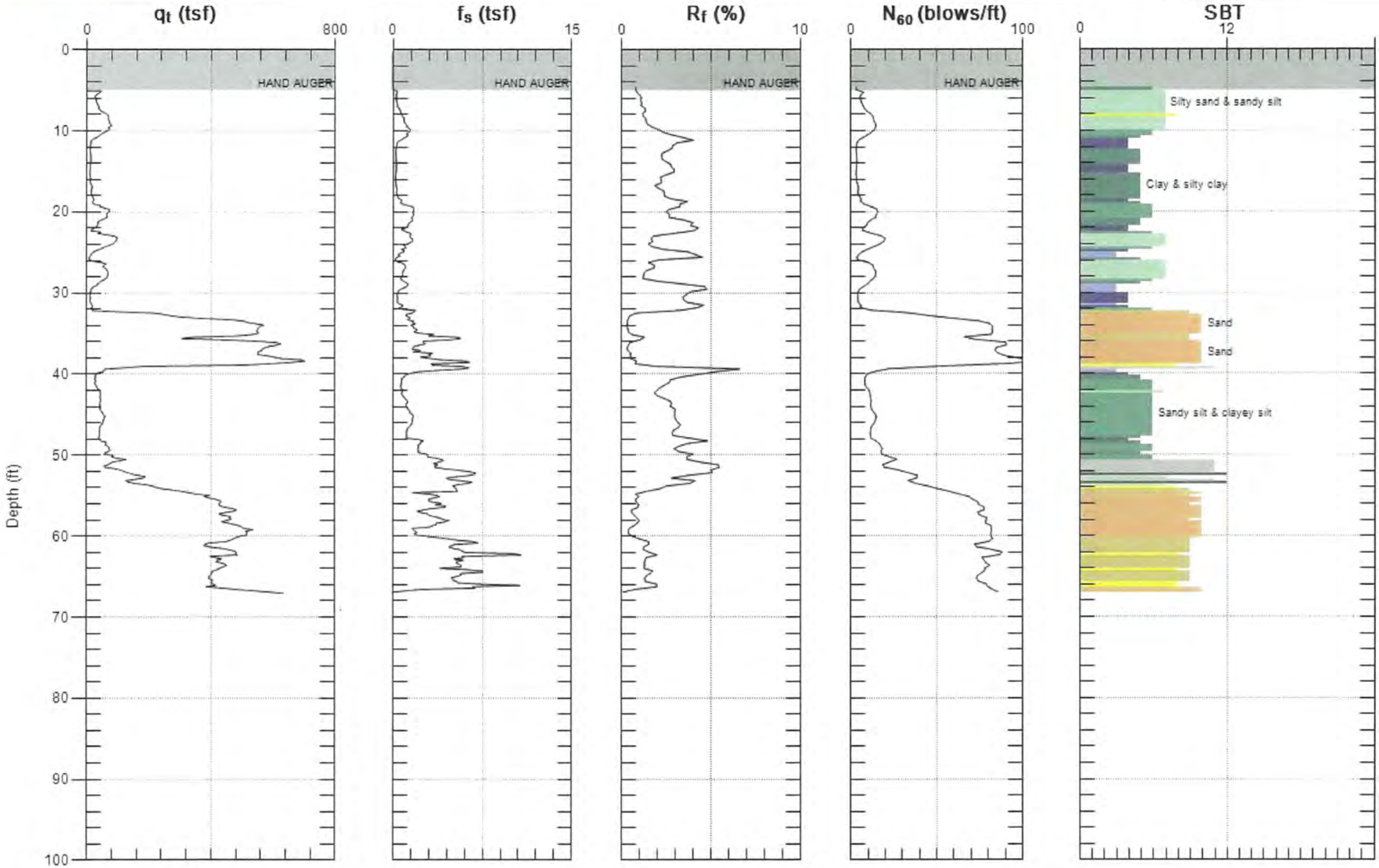
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK **Engineer: K.MENDEZ**

Sounding: CPT-2 **Date: 12/27/16 08:53**



Max. Depth: 67.093 (ft)
Avg. Interval: 0.328 (ft)

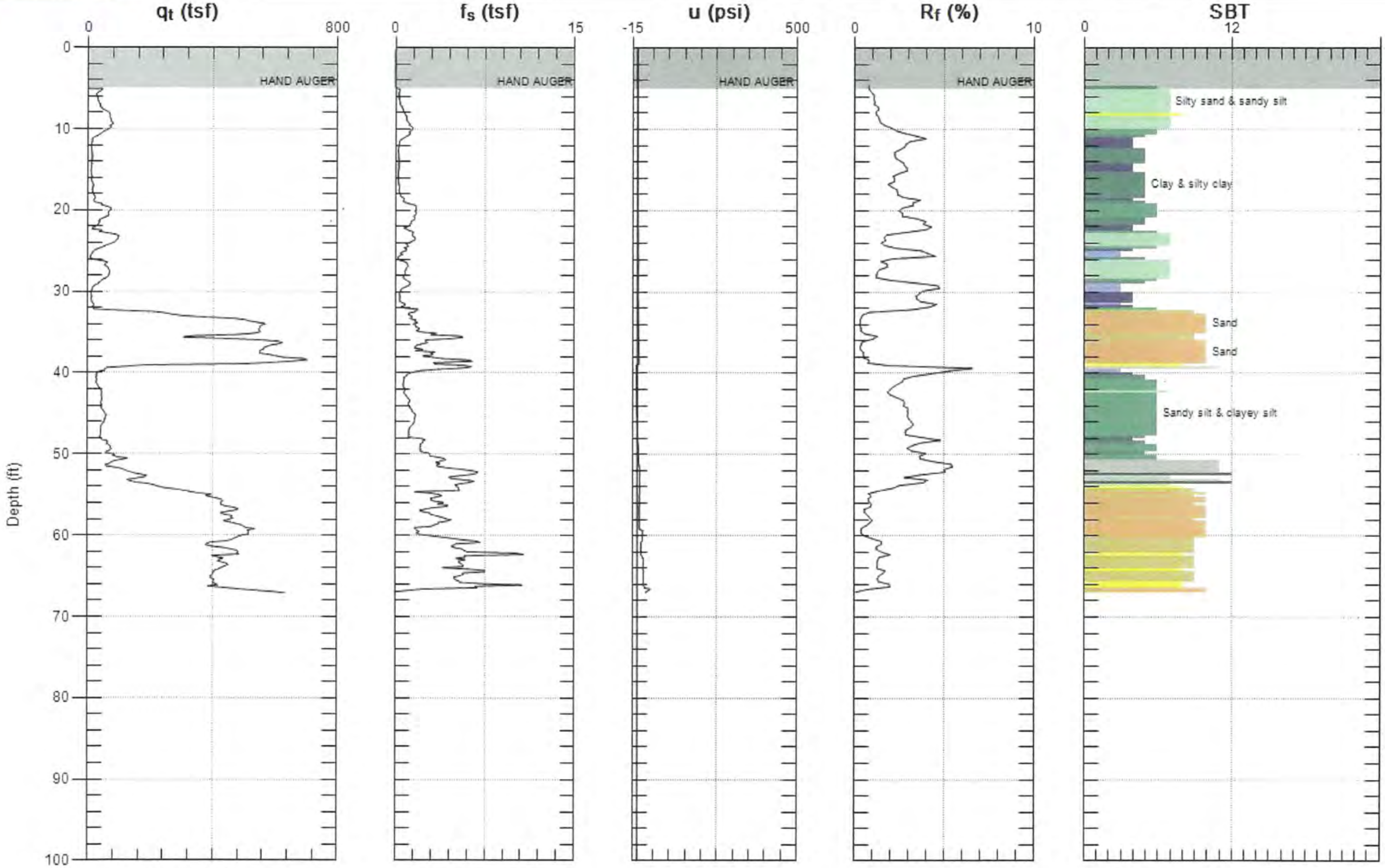
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK
Sounding: CPT-2

Engineer: K.MENDEZ
Date: 12/27/16 08:53



Max. Depth: 67.093 (ft)
Avg. Interval: 0.328 (ft)

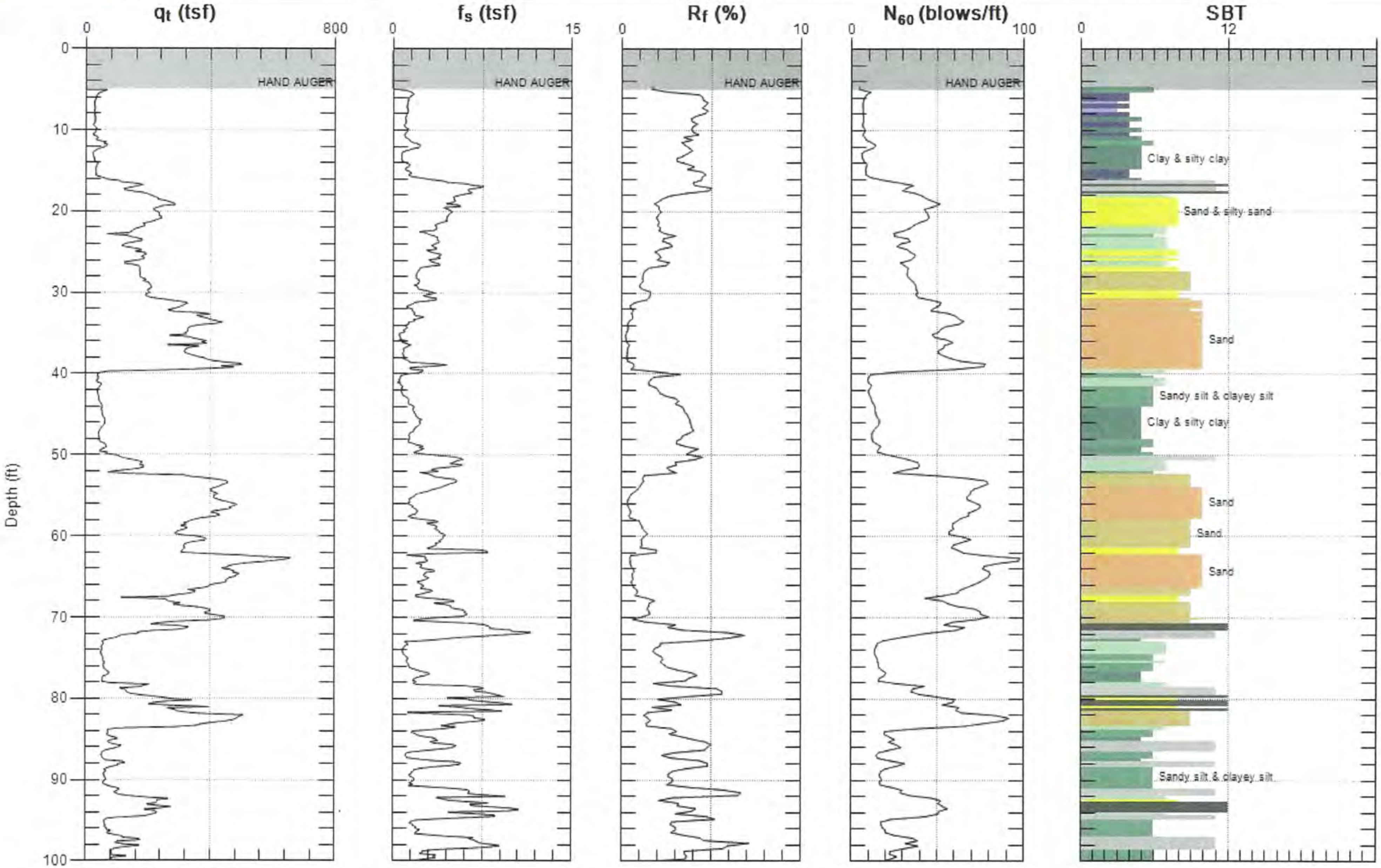
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK
Sounding: CPT-3

Engineer: K.MENDEZ
Date: 12/27/16 10:05



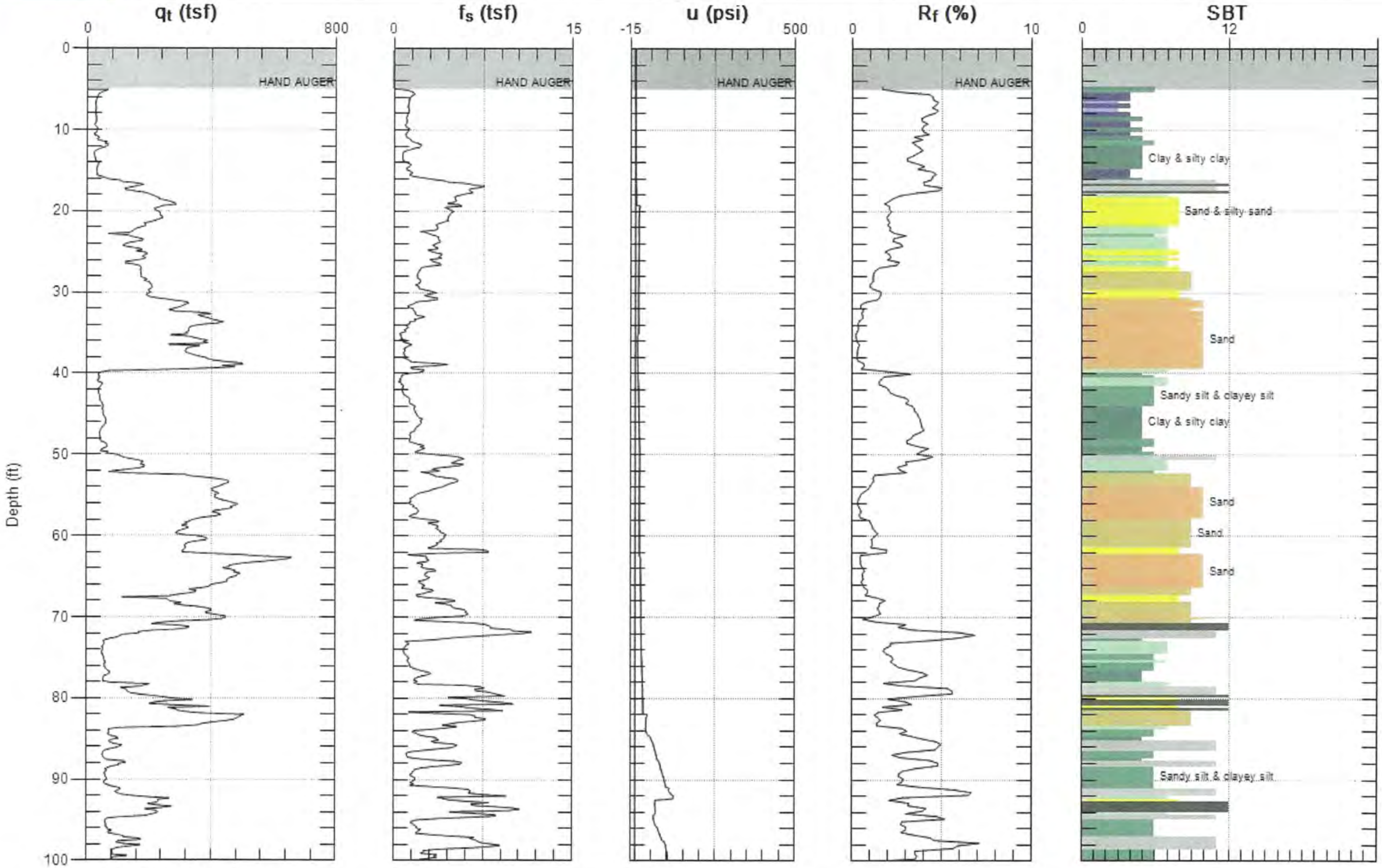
Max. Depth: 100.230 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK | Engineer: K.MENDEZ
Sounding: CPT-3 | Date: 12/27/16 10:05



Max. Depth: 100.230 (ft)
Avg. Interval: 0.328 (ft)

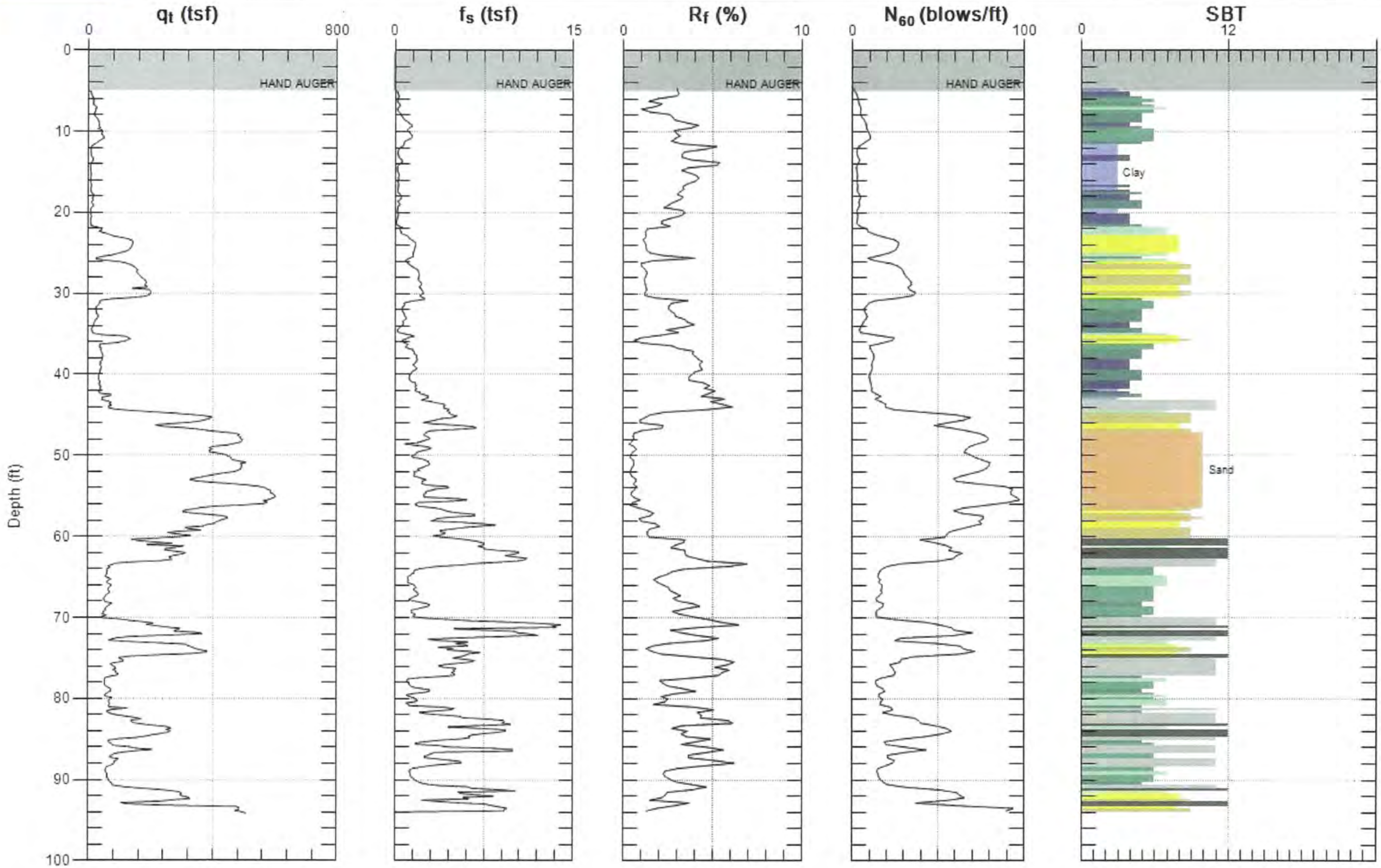
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK **Engineer: K.MENDEZ**

Sounding: CPT-4 **Date: 12/27/16 12:03**



Max. Depth: 94.160 (ft)
Avg. Interval: 0.328 (ft)

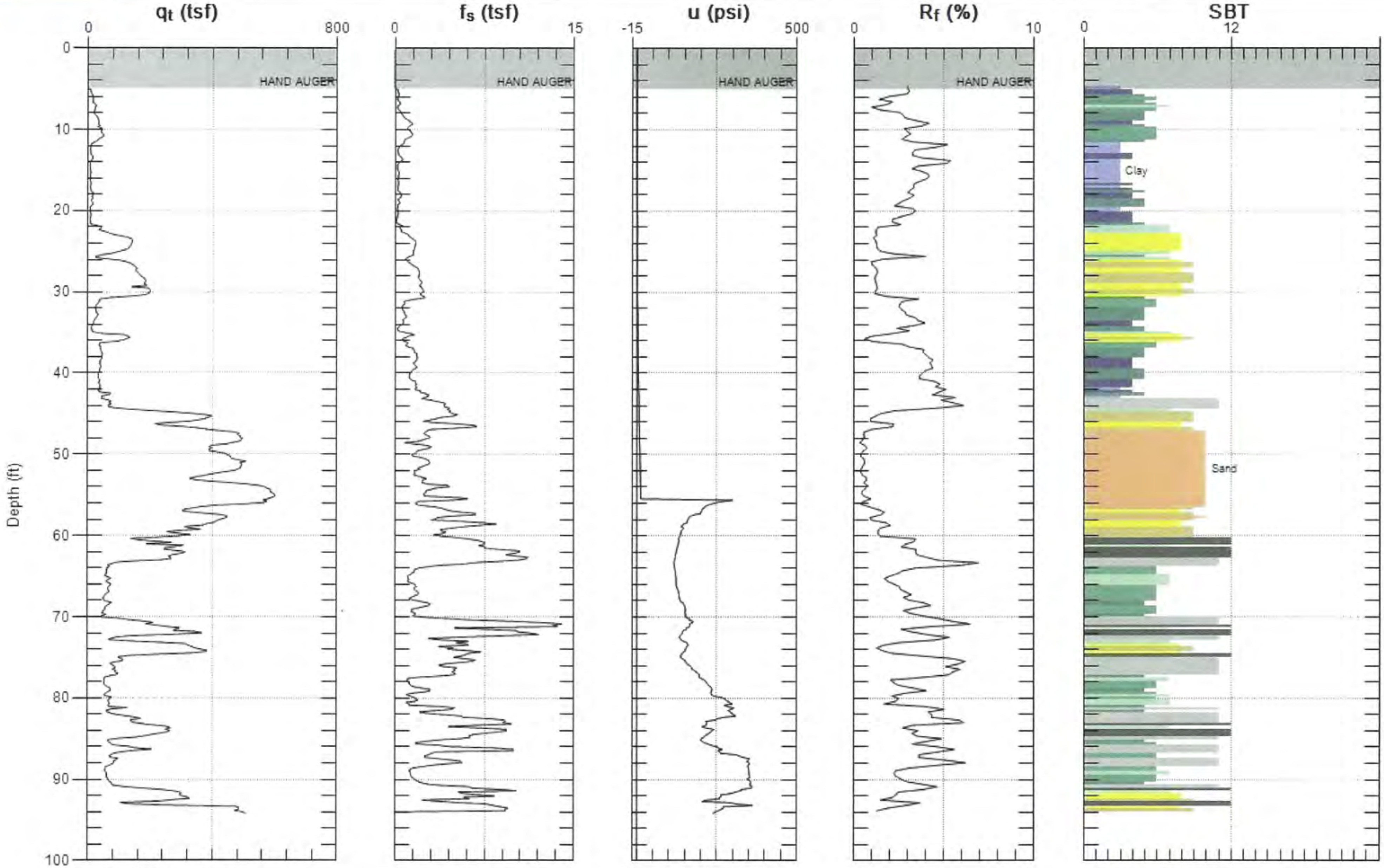
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK
Sounding: CPT-4

Engineer: K.MENDEZ
Date: 12/27/16 12:03



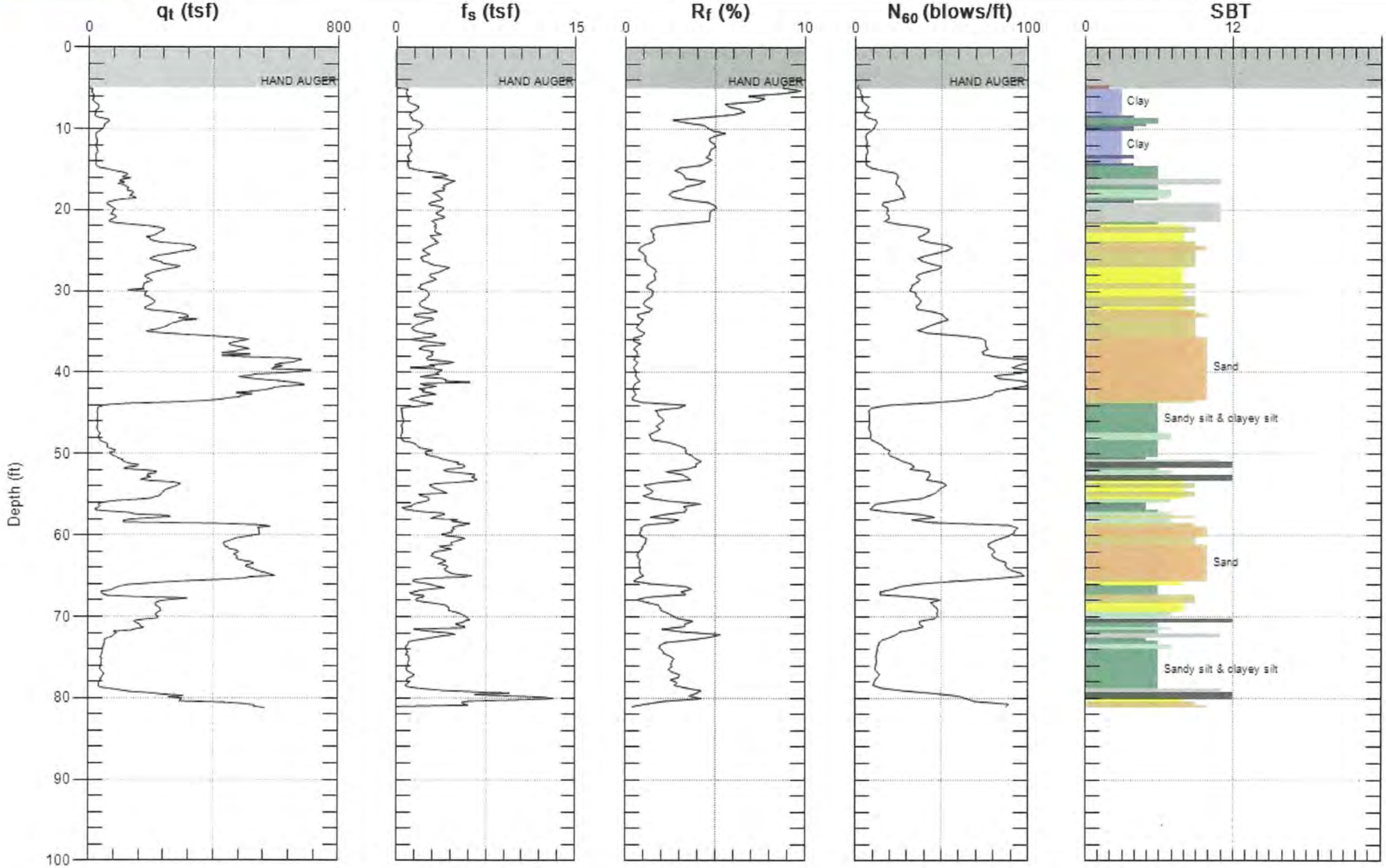
Max. Depth: 94.160 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **LACDPW**

Site: ADVENTURE PARK
Sounding: CPT-5

Engineer: K.MENDEZ
Date: 12/27/16 01:41



Max. Depth: 81.201 (ft)
Avg. Interval: 0.328 (ft)

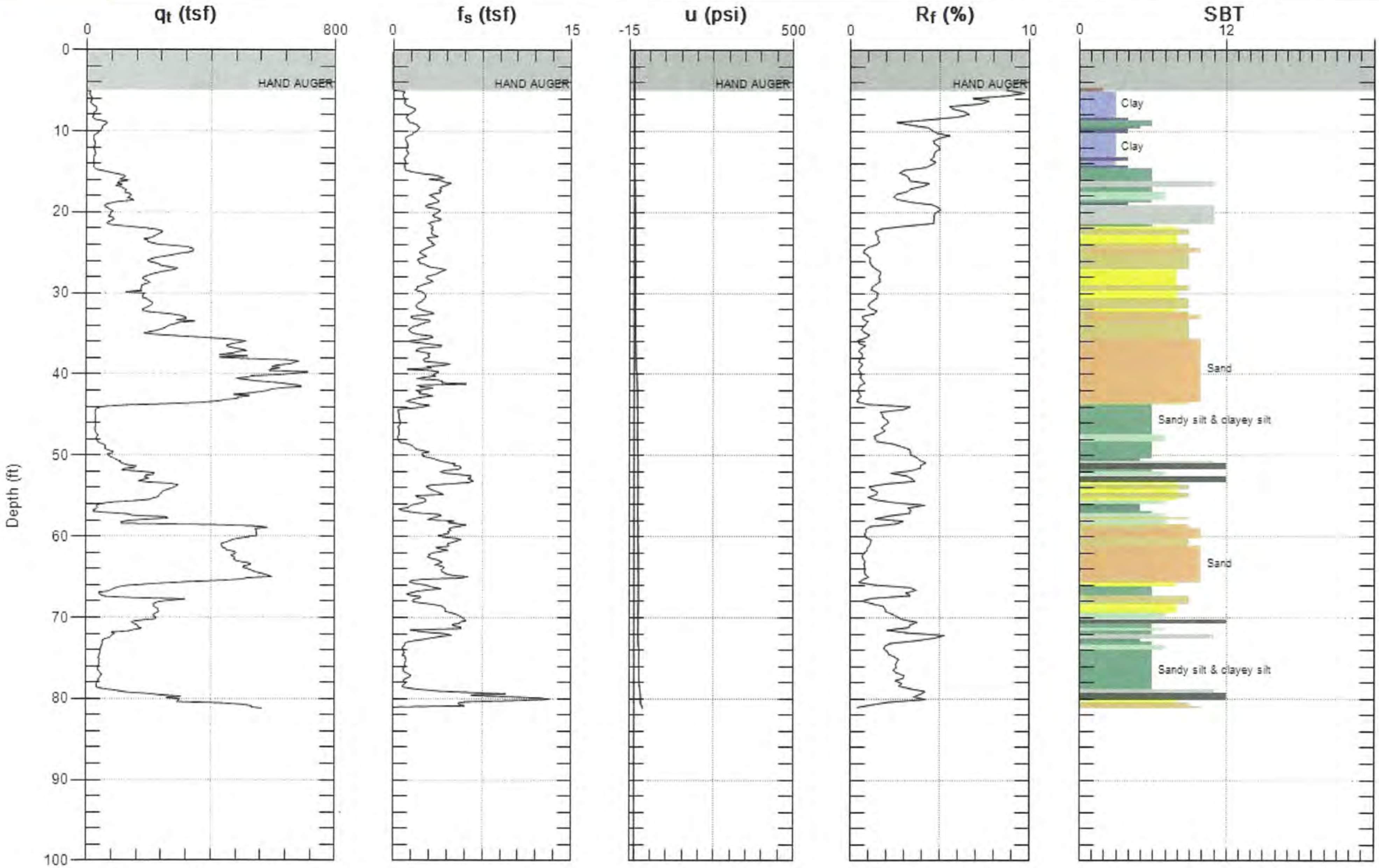
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK
Sounding: CPT-5

Engineer: K.MENDEZ
Date: 12/27/16 01:41



Max. Depth: 81.201 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

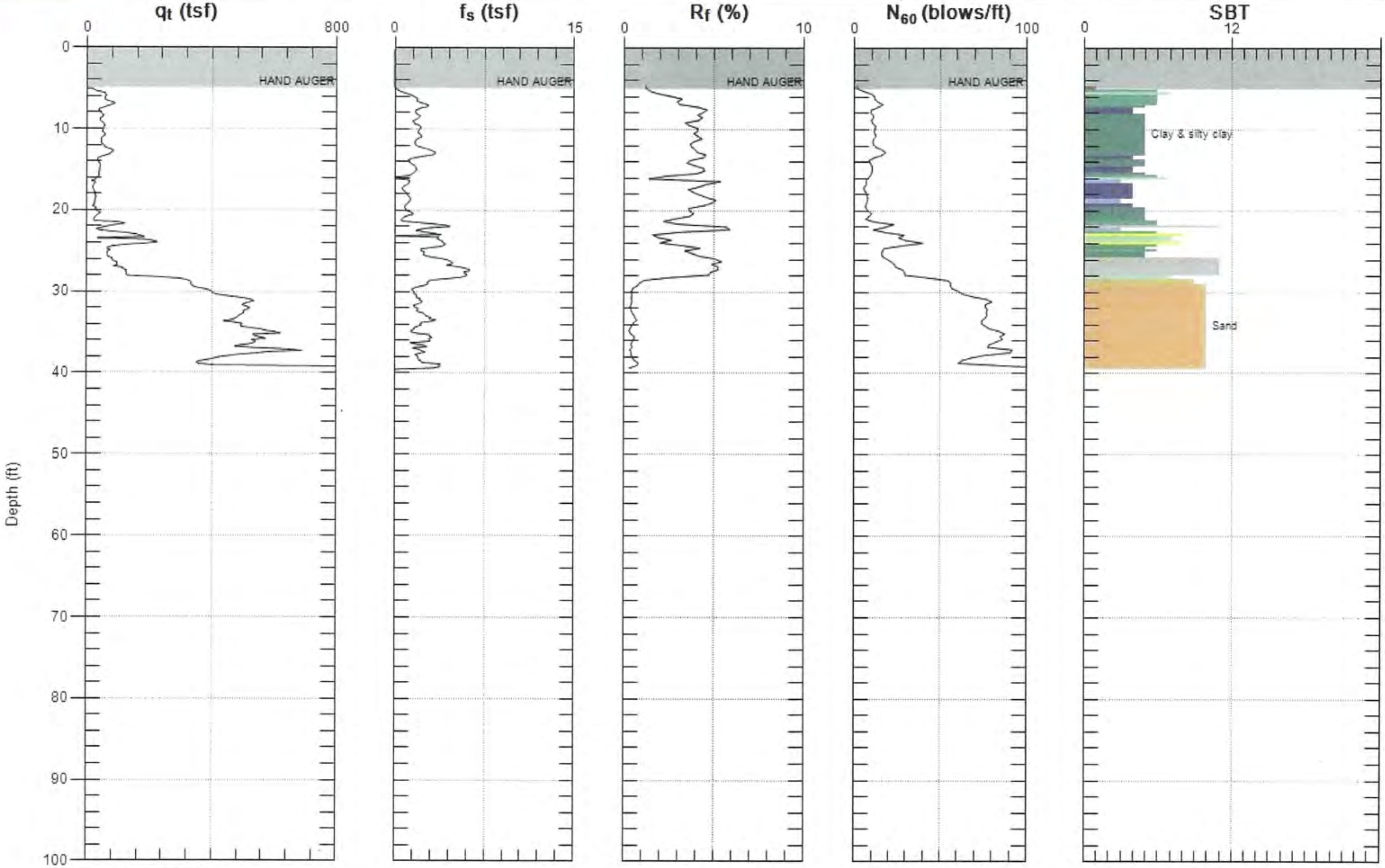
LACDPW

Site: ADVENTURE PARK

Sounding: CPT-6

Engineer: K.MENDEZ

Date: 12/27/16 02:47



Max. Depth: 39.698 (ft)
Avg. Interval: 0.328 (ft)

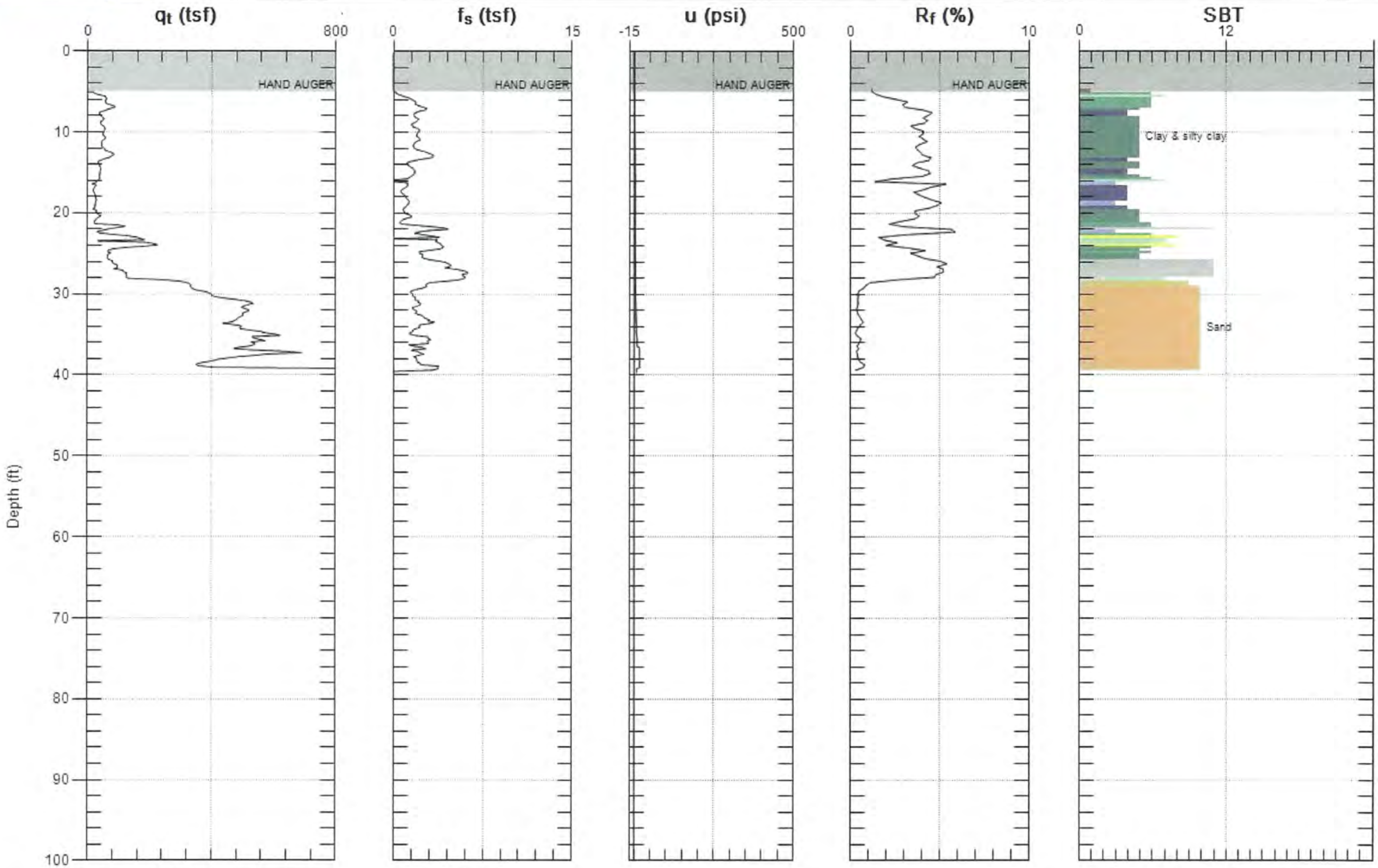
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK
Sounding: CPT-6

Engineer: K.MENDEZ
Date: 12/27/16 02:47



Max. Depth: 39.698 (ft)
Avg. Interval: 0.328 (ft)

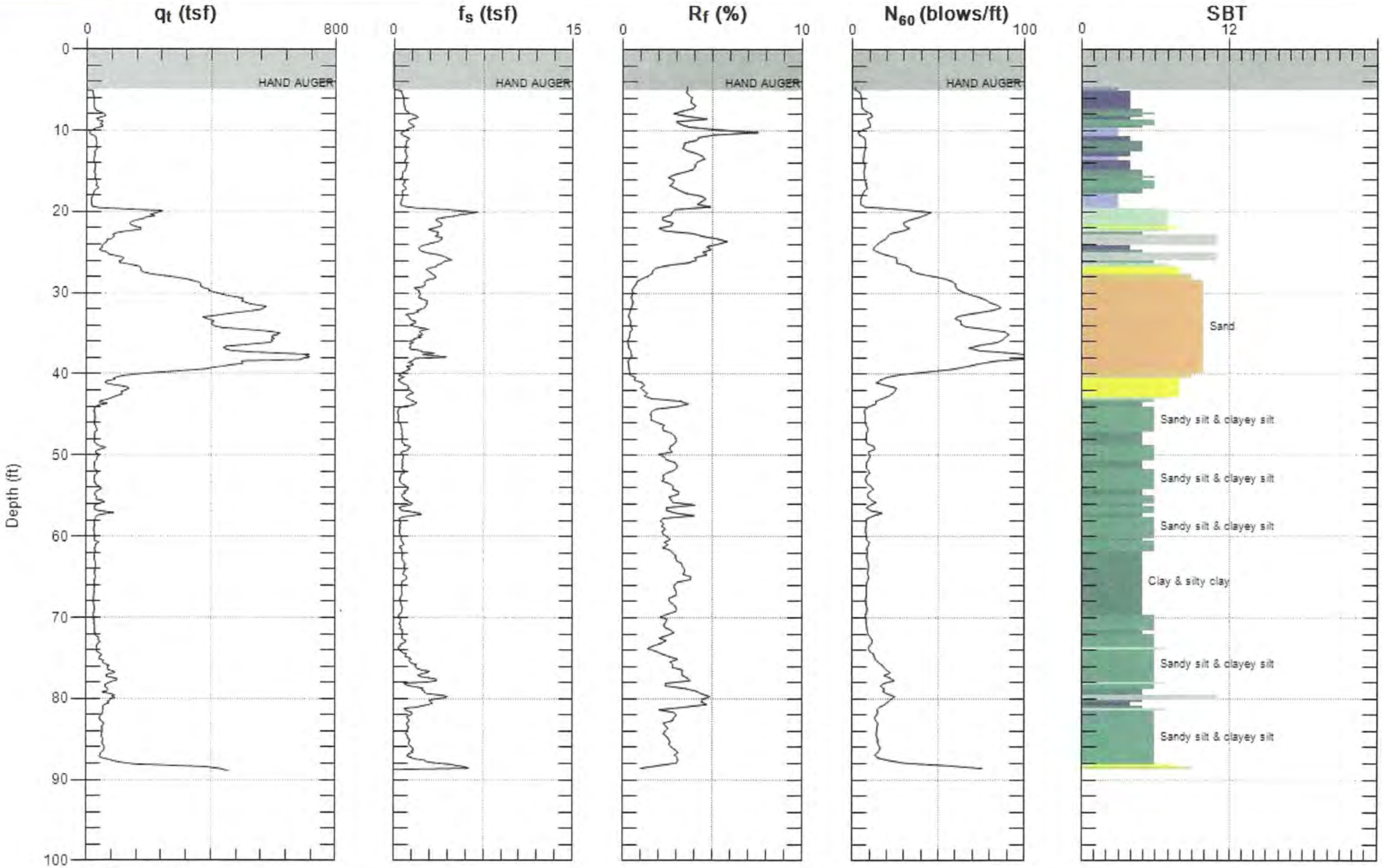
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK
Sounding: CPT-7

Engineer: K.MENDEZ
Date: 12/27/16 03:43



Max. Depth: 88.911 (ft)
Avg. Interval: 0.328 (ft)

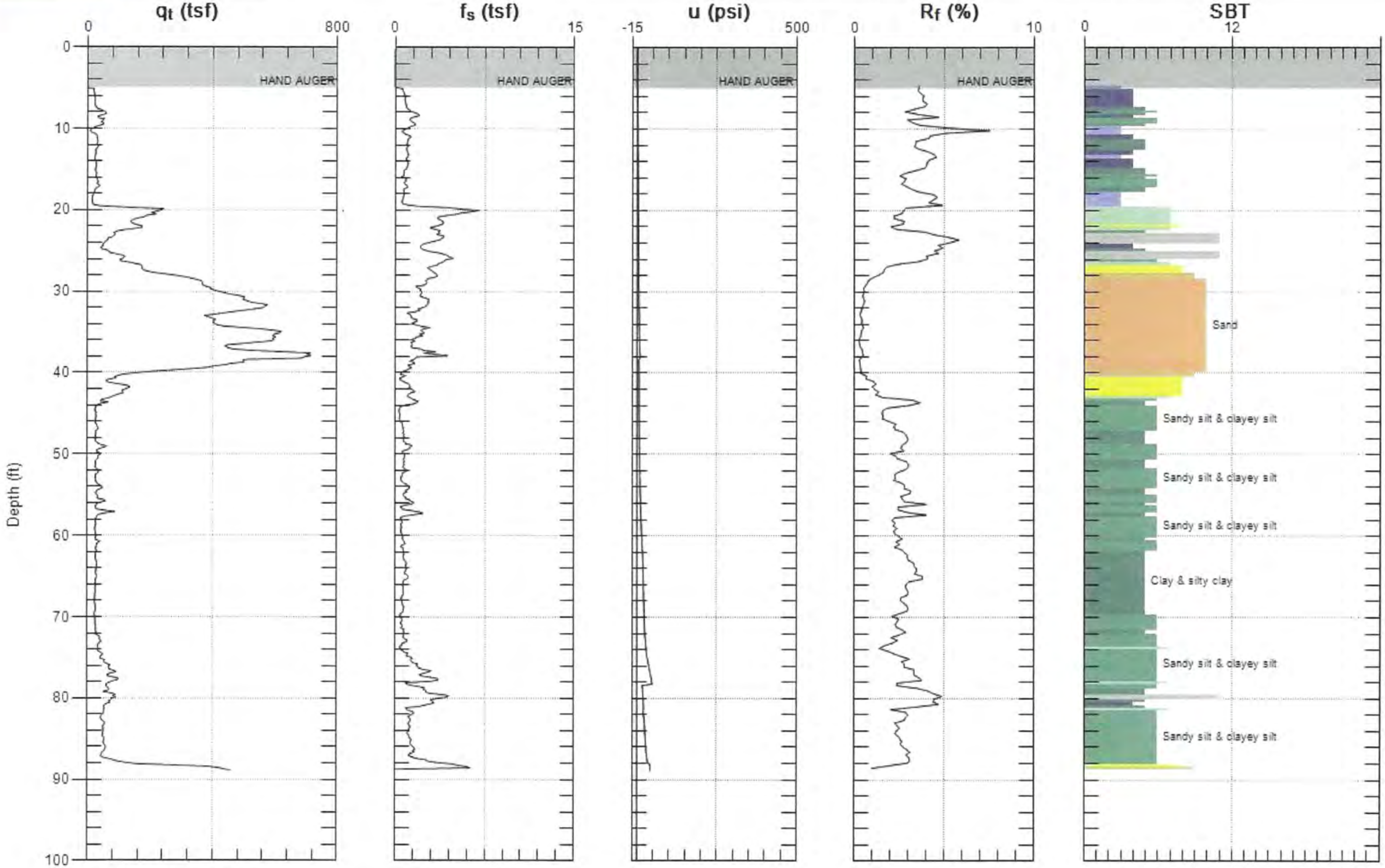
SBT: Soil Behavior Type (Robertson 1990)

GREGG

LACDPW

Site: ADVENTURE PARK
Sounding: CPT-7

Engineer: K.MENDEZ
Date: 12/27/16 03:43



Max. Depth: 88.911 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Appendix C

Previous Laboratory Testing

PROJECT NAME: Adventure Park
TECHNICIAN: GP, EH
PCA: F21816i07

ENGINEER: K. Mendez
DATE: 9/13/2017
PAGE: 1 **OF** 1

updated 3/27/2018

BORING/ SAMPLE B - S	DEPTH (ft)	UNIFIED SOIL CLASSIFICATION Class.	ATTERBERG LIMITS LL	PI	#4 % Pass	#200 % Pass	MOISTURE AND DRY DENSITY γ field pcf	m.c. field %	γ bag pcf	m.c. "as recv."* %	DIRECT SHEAR Φ ult Degree	c ult psf	Φ maxi. Degree	c maxi. psf	CHEMICAL pH	Min. Resistivity (K ohm-cm)	Cl (ppm)	SO_4 (ppm)	Expansion Index (EI / Potential)
B1-2B	7.5-9														5.77	0.5	52	55	
B1-4R	15-16.5	ML	non plastic		100.0	50.0	94.9	23.9											
B1-6B	21.5-23.5	CL	42	20	100.0	76.2									5.83	0.5	21	346	
B1-7R	25-26.5						100.1	15.9			29	94	31	182					
B1-8S	30-31.5				99.1	92.0													39 / low
B2-1R	5-6.5																		
B2-4S	20-21.5				99.7	71.9			* 83.6	19.9									
B2-5R	25-26.5																		
B2-6S	30-31.5				99.9	1.4													
B3-1R	5-6.5	CL	44	25	98.8	84..5	103.0	18.1											
B3-2S	10-11.5				99.8	83.5			* 97.6	17.9									
B3-4R	15-16.5						106.2				24	353	25	700					42 / low
B3-5R	20-21.5																		
B3-6R	25-26.5				100.0	89.8	105.1	18.1			24	383	26	421					
B3-7R	30-31.5				95.0	1.0													
B3-8R	35-36.5	SP			95.2	1.4	121.9	4.9			31	0	38	0					
B3-9S	40-41.5	SW			96.8	0.2													
B3-10R	45-46.5				96.2	0.2	121.8	10.0											
B3-11S	50-51.5				99.9	0.2			* 102.4	15.5									

* Density - ASTM D1188

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

Material Engineering Division

Geotechnical Laboratory

Chemical / Resistivity Report

PROJECT NAME: **Adventure Park**

PCA: **F21816I07**

ENGINEER: **K. Mendez**

BORING-SAMPLE:	B1-2B	B1-6B		
DEPTH (ft):	7.5-9	21.5-23.5		
MINIMUM RESISTIVITY (K ohms-cm):	0.5	0.5		
PH :	5.77	5.83		
CHLORIDE CONTENT (ppm):	52	21		
SO4 (ppm):	55	346		

Remarks:

DEPARTMENT OF PUBLIC WORKS, LOS ANGELES
GEOTECHNICAL & MATERIALS ENGINEERING
Density Experiments (ASTM D1188)

Project: Adventure Park
PCA: F21816i07
Sample: B2-4S
Tech: G.P
Date: 8/23/17

Ring Density (moderate pounding)

#	ring wgt (lbs)	ring & wet sample (lbs)	field sample weight (lbs)	ring diameter (inch)	ring volume (ft3)	Wet Density (pcf)	Dry Density (pcf)	void ratio	deg. Satur.	Avg Dry Density (PCF)
1	0.1435	0.386	0.2425	2.375	0.003	94.6	79.3	1.0808	0.4740	83.6
2	0.1445	0.408	0.2635	2.375	0.003	102.8	86.1	0.9150	0.5599	
3	0.144	0.405	0.261	2.375	0.003	101.8	85.3	0.9333	0.5489	
4										
5										

Moisture Content (%)	
Tare:	74
Tare & Wet (gms):	124
Tare & Dry (gms):	115.9
	19.3%

NOTES:

DEPARTMENT OF PUBLIC WORKS, LOS ANGELES
GEOTECHNICAL & MATERIALS ENGINEERING
Density Experiments (ASTM D1188)

Project: Adventure Park
PCA: F21816i07
Sample: B3-2S
Tech: G.P
Date: 8/24/17

Ring Density (moderate pounding)

#	ring wgt (lbs)	ring & wet sample (lbs)	field sample weight (lbs)	ring diameter (inch)	ring volume (ft3)	Wet Density (pcf)	Dry Density (pcf)	void ratio	deg. Satur.	Avg Dry Density (PCF)
1	0.1435	0.423	0.2795	2.375	0.003	109.0	92.4	0.7841	0.6058	
2	0.1435	0.447	0.3035	2.375	0.003	118.4	100.4	0.6430	0.7387	
3	0.1435	0.446	0.3025	2.375	0.003	118.0	100.1	0.6484	0.7326	97.6
4										
5										

Moisture Content (%)	
Tare:	73.9
Tare & Wet (gms):	123.9
Tare & Dry (gms):	116.3
	17.9%

NOTES:

DEPARTMENT OF PUBLIC WORKS, LOS ANGELES
GEOTECHNICAL & MATERIALS ENGINEERING
Density Experiments (ASTM D1188)

Project: Adventure Park
PCA: F21816i07
Sample: B3-11S
Tech: G.P
Date: 8/24/17

Ring Density (moderate pounding)

#	ring wgt (lbs)	ring & wet sample (lbs)	field sample weight (lbs)	ring diameter (inch)	ring volume (ft3)	Wet Density (pcf)	Dry Density (pcf)	void ratio	deg. Satur.	Avg Dry Density (PCF)
1	0.1435	0.443	0.2995	2.375	0.003	116.8	101.2	0.6303	0.6505	
2	0.1435	0.44	0.2965	2.375	0.003	115.7	100.2	0.6468	0.6339	
3	0.1435	0.457	0.3135	2.375	0.003	122.3	105.9	0.5575	0.7355	102.4
4										
5										

Moisture Content (%)	
Tare:	73.8
Tare & Wet (gms):	123.8
Tare & Dry (gms):	117.1
	15.5%

NOTES:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
GEOTECHNICAL & MATERIALS ENGINEERING DIVISION

Expansion Index - ASTM D4829

PROJECT NAME:	Adventure Park	LAB #:	00122
PCA:	F21816i07	BORING/SAMPLE:	B1-8S
TESTED BY:	GP	DATE TESTED:	8/23/2017
CHECKED BY:	EH	DATE CHECKED:	9/11/2017

Field Moisture (%):		**Dry Density (pcf):**	n/a	**Init. Ring HGT (inch):**	1.0
Optimum Moisture (%):	n/a	**Specific Gravity (G):**	2.65		
Ring Wt. (Lbs):	0.443	**Final Saturated Moist. (%):**	18.2	**LL / PI:**	n/a

Dry Den. of Compacted Soil: $d_c = 100W_s/[(100+M)*0.00727]$

Void Ratio Value: $e = (G*W/d_c)-1$ W=62.4 pcf and d_c=Dry Den. of Soil

Degree of Saturation: $S = (G*M/e)$

Expansion Index = (Final thickness-Initial Thickness/Initial thickness)*1000

Test Number:	1	2	3	4	**
Molding Moisture Content (M) %:	12.1				
Wt. of Compacted Soil+Ring:	1.262				
Wt. of Compacted Soil (Ws):	0.82				
Dry Density (dc) pcf:	100.5				
Void Ratio Value (e):	0.645				
Degree of Saturation (S_{Meas}) %:	49.7				
Indicator Zero Reading:	0				
Indicator Intital Reading:	0.0001				
Indicator Final Reading:	0.0392				
EI_{Meas*}	**39.1**				

** If the degree of saturation is not between 49 to 51, then the above E.I. $_{Meas}$ is calculated for E.I.$_{50}$.

EI_{50} =

POTENTIAL EL:	low	**39**
POTENTIAL EI_{50}:		

NOTES: f. sandy silt w/ *trace* clays

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

Depth of Interval	Weight Factor
0-1	0.4
1-2	0.3
2-3	0.2
3-4	0.1
Below 4	0

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
GEOTECHNICAL & MATERIALS ENGINEERING DIVISION

Expansion Index - ASTM D4829

PROJECT NAME:	Adventure Park	LAB #:	00131
PCA:	F21816i07	BORING/SAMPLE:	B3-4R
TESTED BY:	GP	DATE TESTED:	8/23/2017
CHECKED BY:	EH	DATE CHECKED:	9/11/2017

Field Moisture (%):		**Dry Density (pcf):**	n/a	**Init. Ring HGT (inch):**	1.0
Optimum Moisture (%):	n/a	**Specific Gravity (G):**	2.65		
Ring Wt. (Lbs):	0.805	**Final Saturated Moist. (%):**	14.3	**LL / PI:**	n/a

Dry Den. of Compacted Soil: $d_c = 100W_s/[(100+M)*0.00727]$

Void Ratio Value: $e = (G*W/d_c)-1$ W=62.4 pcf and d_c=Dry Den. of Soil

Degree of Saturation: $S = (G*M/e)$

Expansion Index = (Final thickness-Initial Thickness/Initial thickness)*1000

Test Number:	1	2	3	4	**
Molding Moisture Content (M) %:	12.4				
Wt. of Compacted Soil+Ring:	1.6095				
Wt. of Compacted Soil (Ws):	0.80				
Dry Density (dc) pcf:	98.5				
Void Ratio Value (e):	0.680				
Degree of Saturation (S_{Meas}) %:	48.4				
Indicator Zero Reading:	0				
Indicator Intital Reading:	0.0002				
Indicator Final Reading:	0.0419				
EI_{Meas*}	**41.7**				

** If the degree of saturation is not between 49 to 51, then the above $E.I._{Meas}$ is calculated for $E.I._{50}$.

EI_{50} =

POTENTIAL EL:	low	**42**
POTENTIAL EI_{50}:		

NOTES: f. sandy silt w/ *trace* clays

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

Depth of Interval	Weight Factor
0-1	0.4
1-2	0.3
2-3	0.2
3-4	0.1
Below 4	0

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
GEOTECHNICAL AND MATERIALS ENGINEERING DIVISION

Field Moisture and Density Data Sheet / ASTM D2216 & CTM 226

PROJECT NAME: Adventure Park
PCA: F21816i07
PROJECT ENGINEER: K. Mendez

DATE TESTED: 9/2/2017
TECHNICIAN: GP
CHECKED BY: EH

	1	2	3	4	5	6	7	8
BORING NO./SAMPLE NO.	**B3-1R**	**B3-6R**	**B3-8R**	**B3-10R**				
LABORATORY NO.	129	133	135	136				
DEPTH (ft.)	5-6.5	25-26.5	35-36.5	45-46.5				
FIELD CLASSIFICATION	n/a	n/a	n/a	n/a				
SAMPLE SIZE (in.)	2.375	2.375	2.375	2.375				
NO. OF RINGS SAMPLED	10	12	9	10				
NO. OF RINGS TESTED	6	12	7	9				
VOLUME OF SOIL TESTED (ft^3)	0.01538	0.03076	0.01795	0.02307				
TARE + WET SOIL (lbs.)	2.77	5.62	3.34	4.44				
TARE (lbs.)	0.90	1.80	1.05	1.35				
WET SOIL (lbs.)	1.87	3.82	2.29	3.09				
WEIGHT OF #4 ROCK (lbs.)	0.01	0.01	0.93	0.11				
WEIGHT OF 3/4 ROCK (lbs.)	0.00	0.00	0.00	0.00				
WET FINES	1.86	3.81	1.37	2.98				
WET WEIGHT (gms.)FOR MOIST. CONTENT	50.0	163.0	166.1	50.0				
DRY WEIGHT FOR MOISTURE CONTENT (GMS)	42.3	137.9	153.1	45.3				
MOISTURE CONTENT OF FINES (%)	18.2	18.2	8.5	10.4				
DRY FINES	1.57	3.22	1.26	2.70				
TOTAL DRY SOIL (lbs.)	1.58	3.23	2.19	2.81				
TOTAL WATER (lbs.)	0.29	0.59	0.11	0.28				
COMPOSITE MOISTURE (%)	**18.1**	**18.1**	**4.9**	**10.0**				
% OF #4 ROCK	0.8	0.3	42.4	3.8				
% OF 3/4 ROCK	0.0	0.0	0.0	0.0				
COMPOSITE DRY DENSITY (pcf)	**103.0**	**105.1**	**121.9**	**121.8**				
Void Ratio:	0.61	0.57	0.36	0.36				
Degree of Saturation (%):	79.00	83.86	36.32	73.89				

DEPARTMENT OF PUBLIC WORKS, LOS ANGELES
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DIRECT SHEAR ASTM D3080

Project: Adventure Park

PCA: F21816i07 **USC:** n/a **% (-200):** n/a

Boring/Sample: B1-7R **LL:** n/a **PI:** n/a

Depth (ft): 25-26.5 **% ret. 3/4":** 0.0 **% ret. #4:** 0.0

Sample Condition: med. Den. Composite **Dry Density (pcf):** 100.1

Field Class.: n/a Composite **Moisture (%):** 15.9

Number of Rings: 5 **Initial** (Field) **Void Ratio:** 0.65

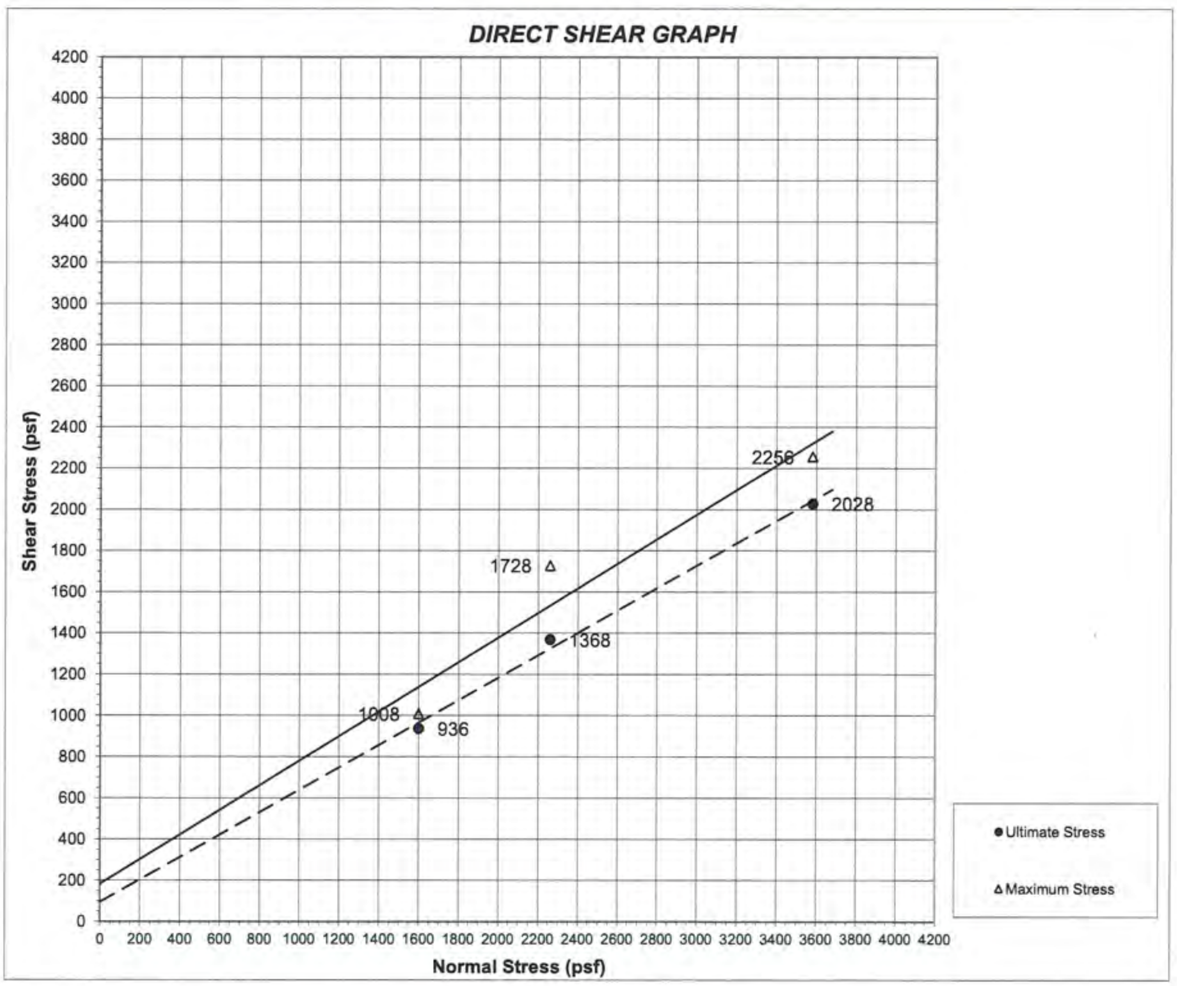
App. Soaking Time: 24 hrs **Initial** (Field) **Saturation (%):** 64.6

Notes: Lean clay, *trace* silt stone, v. moist, brown, homogenous

Ring Dia.:	2.375		
Normal Stress	**Ultimate Stress**	**Maximum Stress**	**RATE**
(psf)	(psf)	(psf)	IN./MIN
0	----	----	----
1600	936	1008	0.0015
2260	1368	1728	
3580	2028	2256	

ϕ Max	31
ϕ Ult	29
C_{max}	182
C_{ult}	94

Max (-tan)	0.5974
Ult (-tan)	0.5442



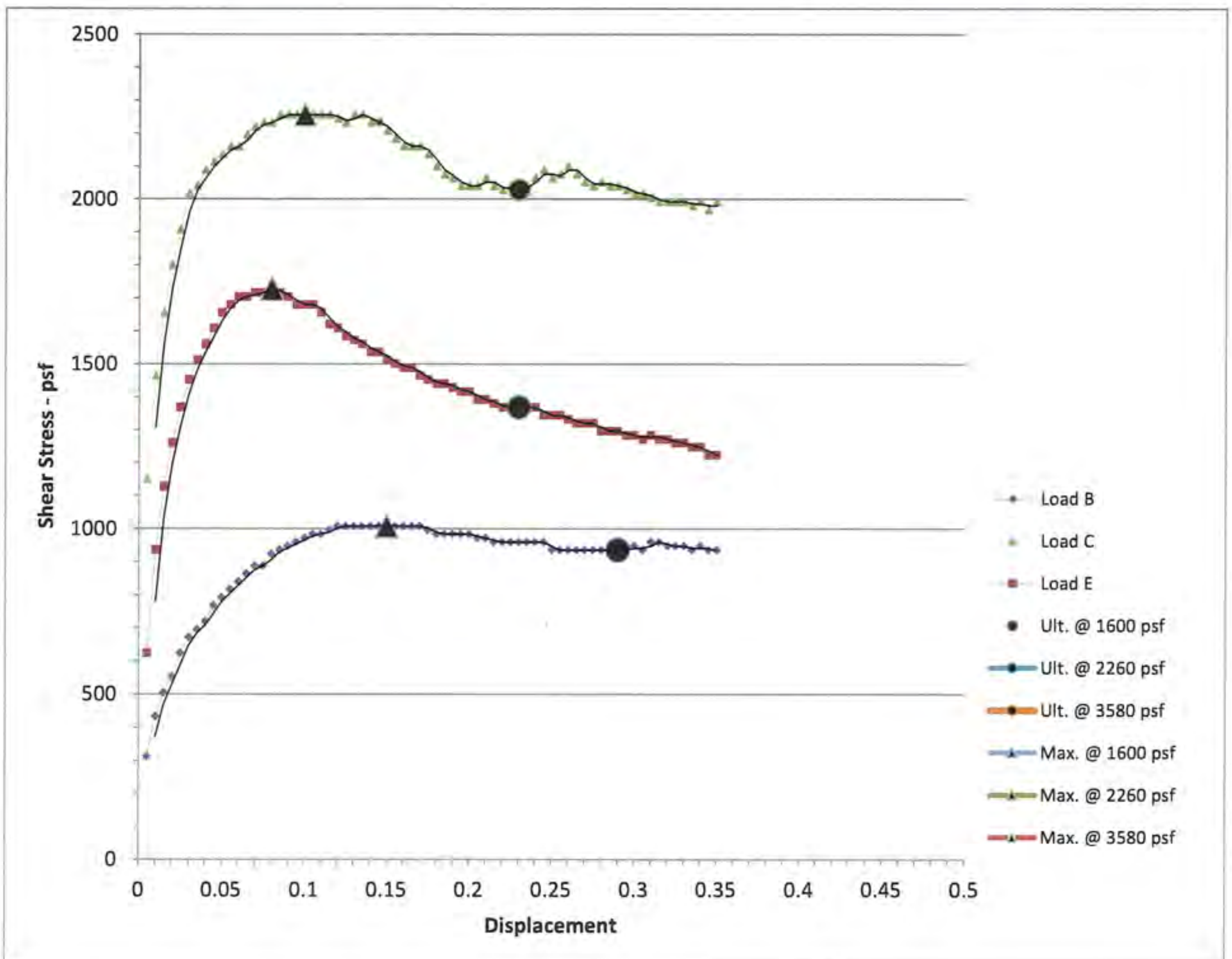
DEPARTMENT OF PUBLIC WORKS, LOS ANGELES
GEOTECHNICAL & MATERIALS ENGINEERING
DIRECT SHEAR ASTM D3080

Multi Graph

Project Name: **Adventure Park**
PCA: F21816i07
Boring/Sample: B1-7R

	Normal Stress (psf)	Ult. Stress (psf)	Dist. (inch)	Max. Stress (psf)	Dist. (inch)
Load B	1600	936	0.29	1008	0.15
Load C	2260	1368	0.23	1728	0.08
Load E	3580	2028	0.23	2256	0.1

	Ring WGT + Wet Soil (lb.)	Approx. Field Density (psf)
Load B	0.3935	82.0
Load C	0.3975	83.3
Load E	0.425	92.6



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GEOTECHNICAL & MATERIALS ENGINEERING
DIRECT SHEAR ASTM D3080

Project: Adventure Park

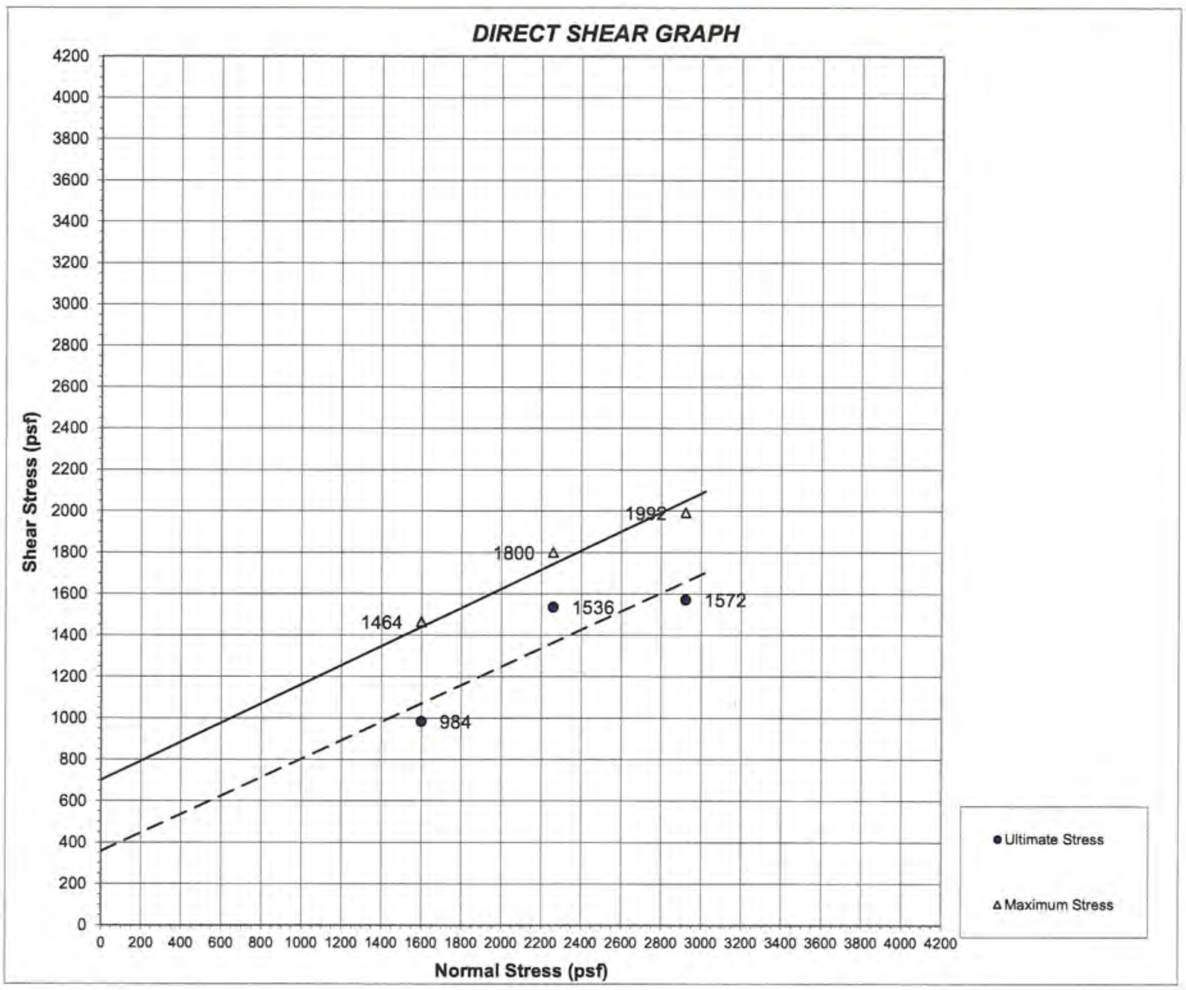
PCA: F21816i07	**USC:** n/a	**% (-200):** n/a	
Boring/Sample: B3-4R	**LL:** n/a	**PI:** n/a	
Depth (ft): 15-16.5	**% ret. 3/4":** 0.0	**% ret. #4:** 0.0	
Sample Condition: v. dense	**Composite Dry Density (pcf):** 106.2		
Field Class.: N/A	**Composite Moisture (%):** 18.2		
Number of Rings: 11	**Initial (Field) Void Ratio:** 0.56		
App. Soaking Time: 72 hrs	**Initial (Field) Saturation (%):** 86.8		

Notes: wet, clay, top (light brown with dark spot), bottom (light brown with few med. coarse sand)

Ring Dia.:	2.375		
Normal Stress	**Ultimate Stress**	**Maximum Stress**	**RATE**
(psf)	(psf)	(psf)	IN./MIN
0	----	----	----
			0.003
1600	984	1464	
2260	1536	1800	
2920	1572	1992	

ϕ Max	25
ϕ Ult	24
C_{max}	700
C_{ult}	357

Max (-tan)	0.462
Ult (-tan)	0.4455



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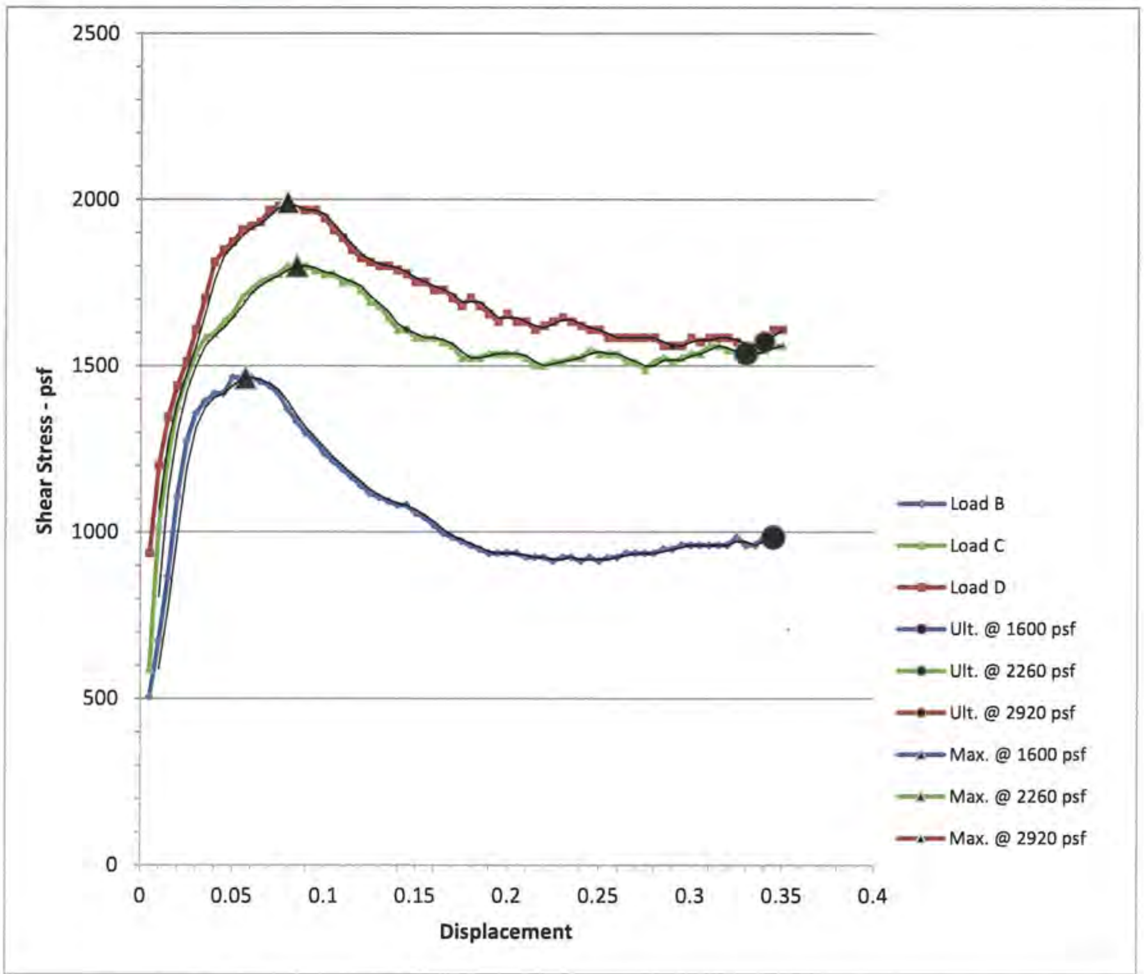
DIRECT SHEAR ASTM D3080

Multi Graph

Project Name: **Adventure Park**
PCA: F21816i07
Boring/Sample: B3-4R

	Normal Stress psf	Ult. Stress psf	Dist. inch	Max. Stress psf	Dist. inch	Ring WGT + Wet Soil lb.	Approx. Field Density psf
Load B	1600	984	0.345	1464	0.057	0.4680	104.9
Load C	2260	1536	0.33	1800	0.085	0.4700	105.6
Load D	2920	1572	0.34	1992	0.08	0.4575	101.5



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GEOTECHNICAL & MATERIALS ENGINEERING
DIRECT SHEAR ASTM D3080

Project: Adventure Park

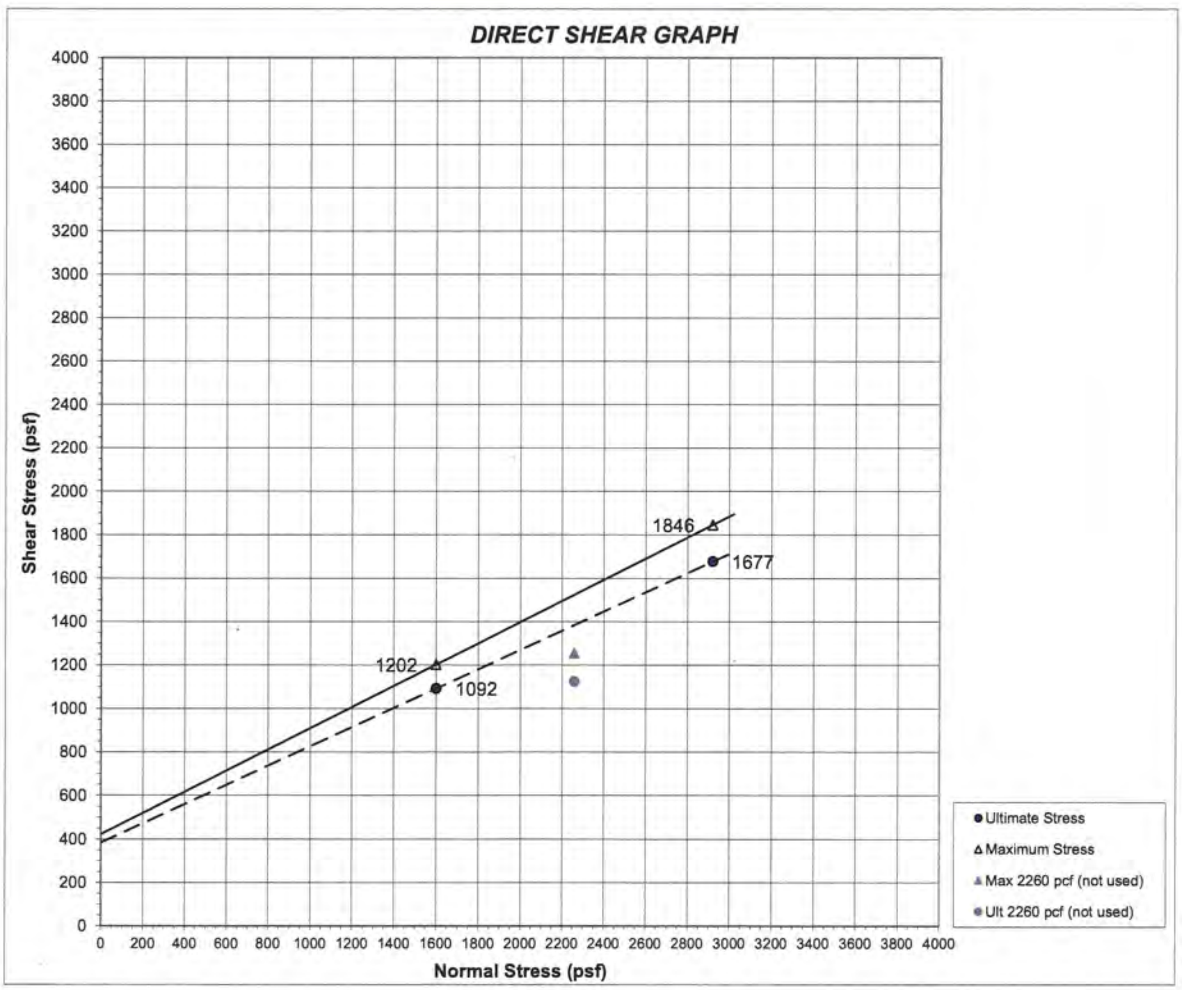
PCA: F21816i07 **USC:** n/a **% (-200):** 89.8
Boring/Sample: B3-6R **LL:** n/a **PI:** n/a
Depth (ft): 25-56.5 **% ret. 3/4":** 0.0 **% ret. #4:** 0.0
Sample Condition: v. hard **Composite Dry Density (pcf):** 105.0
Field Class.: N/A **Composite Moisture (%):** 18.2
Number of Rings: 12 **Initial (Field) Void Ratio:** 0.57
App. Soaking Time: 72 hrs **Initial (Field) Saturation (%):** 84.0

Notes: wet, light grey/brown, layers/pockets of f. sandy silt - silty clay

Ring Dia.:	2.375		
Normal Stress	**Ultimate Stress**	**Maximum Stress**	**RATE**
(psf)	(psf)	(psf)	IN./MIN
0	----	----	----
1600	1092	1202	0.008
2260	1124	1254	
2920	1677	1846	

ϕ Max	26
ϕ Ult	24
C_{max}	421
C_{ult}	383

Max (-tan)	0.4879
Ult (-tan)	0.4432



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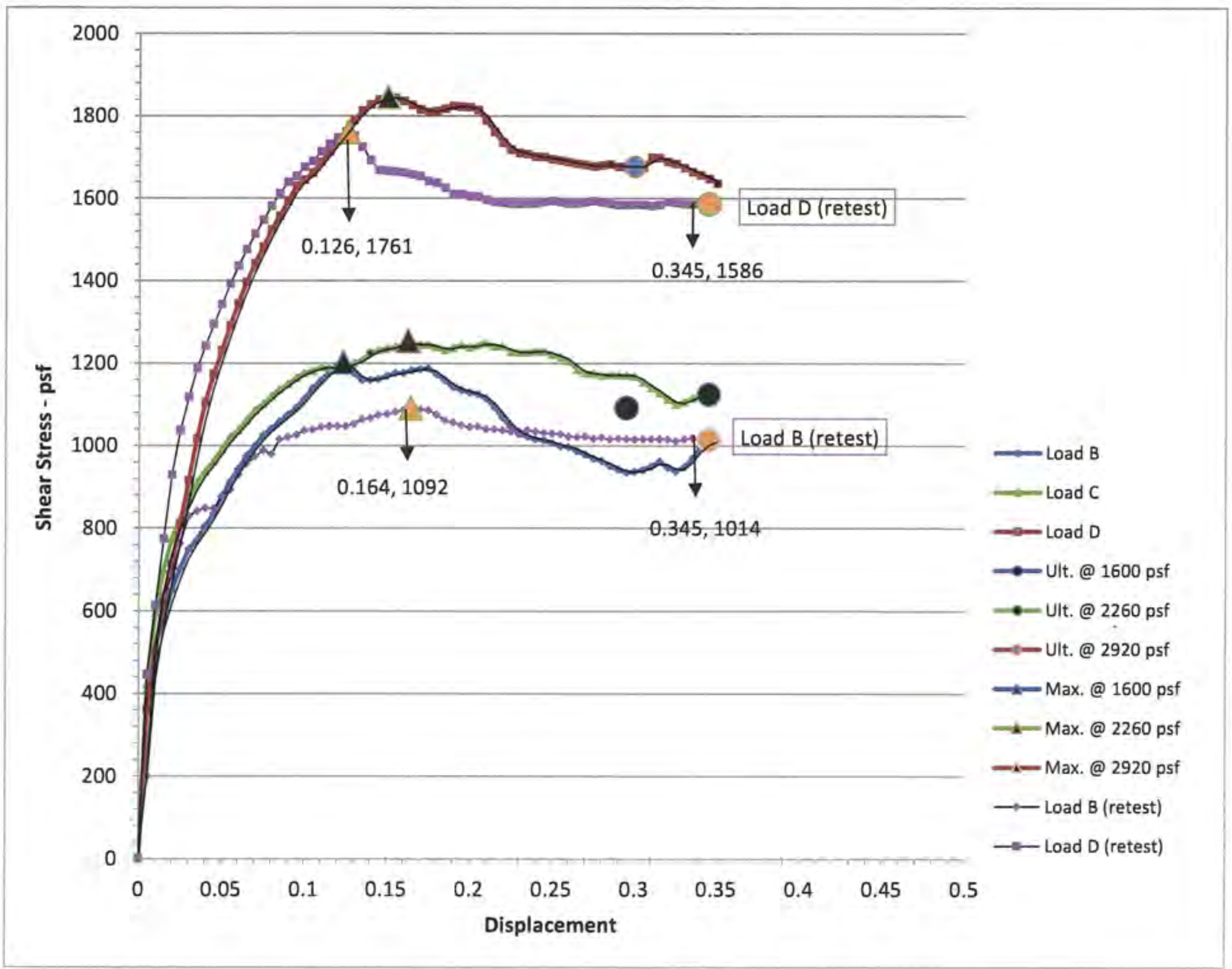
GEOTECHNICAL & MATERIALS ENGINEERING
DIRECT SHEAR ASTM D3080

Multi Graph

Project Name: **Adventure Park**
PCA: F21816i07
Boring/Sample: B3-6R

	Normal Stress psf	Ult. Stress psf	Dist. inch	Max. Stress psf	Dist. inch	Ring WGT + Wet Soil lb.	Approx. Field Density psf
Load B	1600	1092	0.295	1202	0.1236	0.4670	104.6
Load C	2260	1124	0.345	1254	0.1627	0.4690	105.3
Load D	2920	1677	0.3	1846	0.1505	0.4700	105.6



DEPARTMENT OF PUBLIC WORKS, LOS ANGELES
GEOTECHNICAL & MATERIALS ENGINEERING
DIRECT SHEAR ASTM D3080

Project: Adventure Park

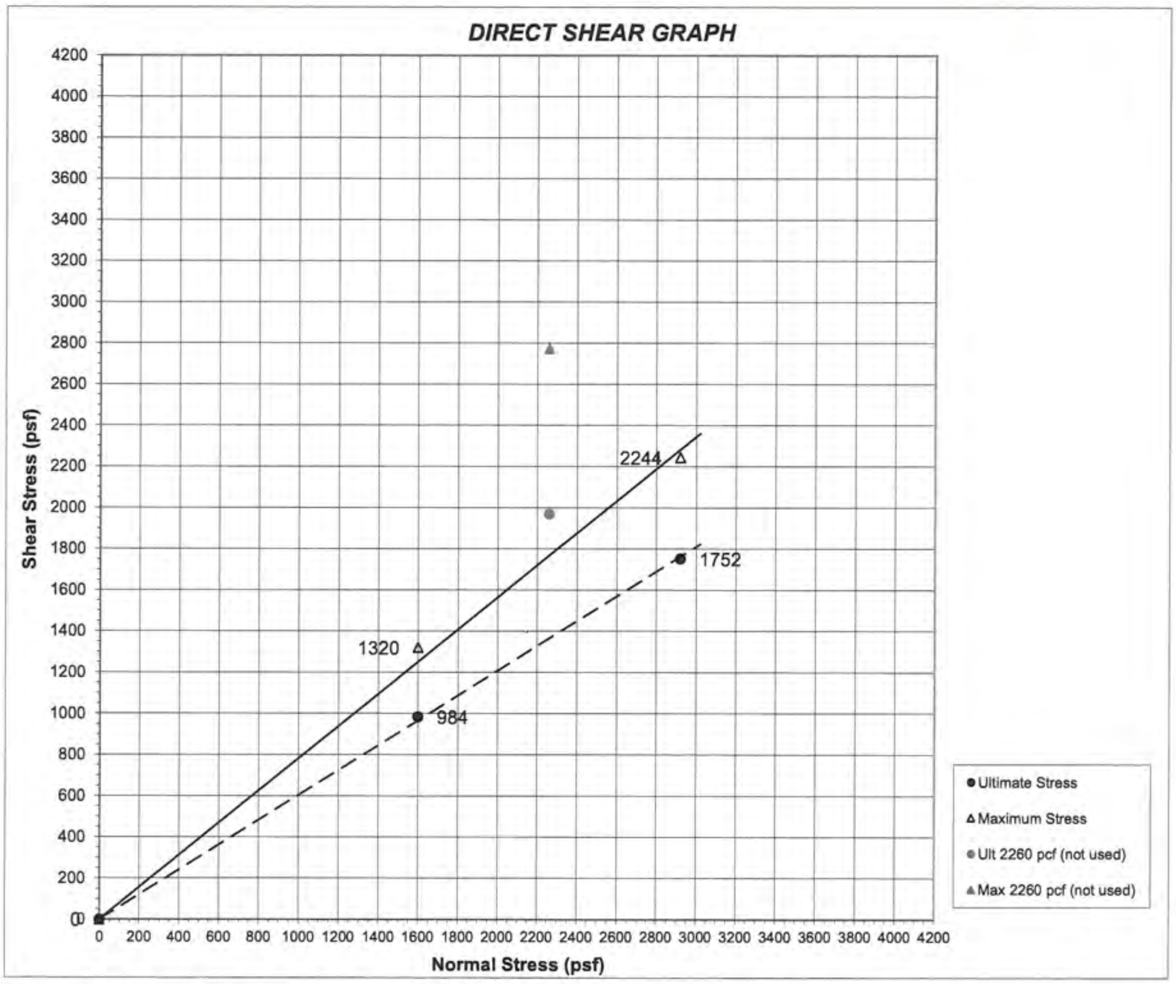
PCA: F21816i07 **USC:** SP **% (-200):** 1.4
Boring/Sample: B3-8R **LL:** n/a **PI:** n/a
Depth (ft): 35-36.5 **% ret. 3/4":** 0.0 **% ret. #4:** 0.9
Sample Condition: loose Composite **Dry Density (pcf):** 117.9
Field Class.: N/A Composite **Moisture (%):** 8.4
Number of Rings: 9 **Initial (Field) Void Ratio:** 0.40
App. Soaking Time: 24 hrs **Initial (Field) Saturation (%):** 55.4

Notes: Moist, loose sand, light brown, coarse sand, *few* gravels, classification is SP but close to SW

Ring Dia.:	2.375		
Normal Stress	**Ultimate Stress**	**Maximum Stress**	**RATE**
(psf)	(psf)	(psf)	IN./MIN
0	----	----	----
1600	984	1320	0.008
2260	1968	2771	
2920	1752	2244	

ϕ Max	38
ϕ Ult	31
C_{max}	0
C_{ult}	0

Max (-tan)	0.7815
Ult (-tan)	0.6035

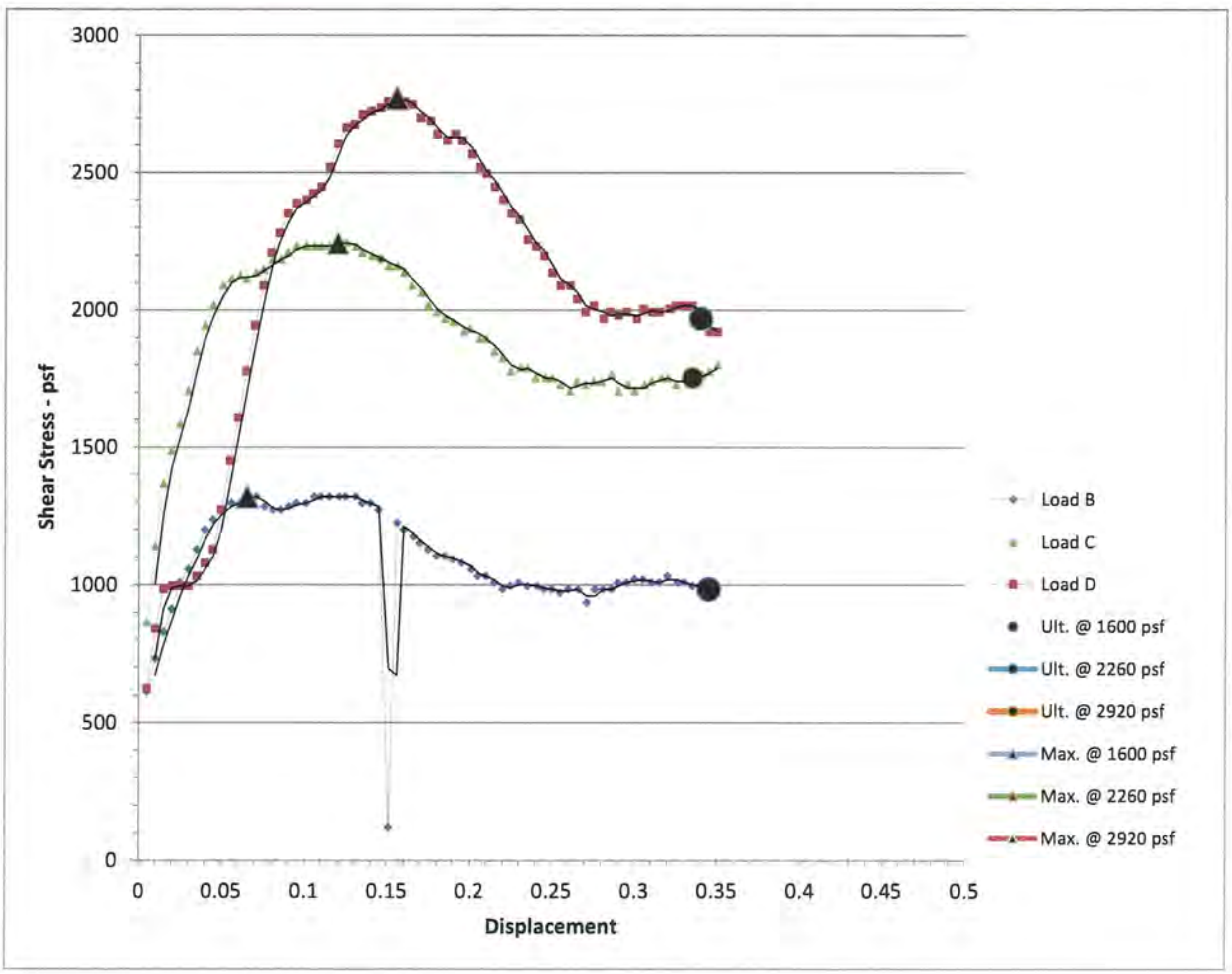


DEPARTMENT OF PUBLIC WORKS, LOS ANGELES
GEOTECHNICAL & MATERIALS ENGINEERING
DIRECT SHEAR ASTM D3080

Multi Graph

Project Name:	**Adventure Park**
PCA:	F21816i07
Boring/Sample:	B3-8R

	Normal Stress psf	Ult. Stress psf	Dist. inch	Max. Stress psf	Dist. inch	Ring WGT + Wet Soil lb.	Approx. Field Density psf
Load B	1600	984	0.345	1320	0.0655	0.469	114.8
Load C	2260	1968	0.34	2771	0.1555	0.471	115.5
Load D	2920	1752	0.335	2244	0.12	0.48	118.7



LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: n/a
CLASSIFICATION: ML (SM)
TESTED BY: EH/TA
CHECKED BY: EH

PCA: F21816i07
BORING / SAMPLE: B1-R4
DEPTH (FT): 15-16.5
DATE TESTED: 9/28/17
DATE CHECKED: 10/4/17

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 > 50%, CLAY or SILT

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1					
3/8"	9.52				100.0	
No. 4	4.76	0.00	0.0	0.0	100.0	
PAN	0	0.27				
TOTAL FRACTIONS		0.27				
OVEN-DRY FINES		0.22				
* TOTAL OVEN-DRY		0.22				

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	1.25
DRY WEIGHT (gm)	1.01
MOISTURE (%)	23.9

* Cobbles not included in total oven-dry weight
• If moisture was not taken from Course material a 1% moisture content will be assumed.

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	363.20
CALCULATED OVEN-DRY WEIGHT (gms)	293.06
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	293.19

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
10	2	0.61	0.2	0.3	99.7	
20	0.85	1.61	0.5	0.8	99.2	
40	0.425	2.27	0.8	1.6	98.4	
60	.2.5	3.44	1.2	2.8	97.2	
140	0.106	23.71	8.1	10.8	89.2	
200	0.074	114.82	39.2	50.0	50.0	
PAN	0	14.62	5.0			
TOTAL FRACTIONS		161.08	54.9			
TOTAL DRY WEIGHT AFTER WET SEIVING		161.00	54.9			
SIEVE LOSS-GAIN		-0.08	0.0			

Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS: non plastic

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME:	Adventure Park	PCA:	F21816i07
LAB. ID:	00120	BORING / SAMPLE:	B1-6B
CLASSIFICATION:	CL	DEPTH (FT):	21.5-23.5
TESTED BY:	GP	DATE TESTED:	8/21/17
CHECKED BY:	EH	DATE CHECKED:	3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel

If % Passing #200 > 50%, CLAY or SILT

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1					
3/8"	9.52				100.0	
No. 4	4.76	0.01	0.0	0.0	100.0	
PAN	0	26.59				
TOTAL FRACTIONS		26.60				
OVEN-DRY FINES		24.55				
* TOTAL OVEN-DRY		24.56				

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	300.00
DRY WEIGHT (gm)	277.00
MOISTURE (%)	8.3

* Cobbles not included in total oven-dry weight

• If moisture was not taken from Course material a 1% moisture content will be assumed.

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	600.00
CALCULATED OVEN-DRY WEIGHT (gms)	554.00
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	554.23

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
10	2					
20	0.85					
40	0.425					
60	.2.5					
140	0.106					
200	0.074			23.8	76.2	
PAN	0					
TOTAL FRACTIONS		0.00	0.0			
TOTAL DRY WEIGHT AFTER WET SEIVING		131.90	23.8			
SIEVE LOSS-GAIN		131.90				

Atterberg Test

Liquid Limit	42
Plastic Limit	22
Plastic Index	20

SOIL DESCRIP. / REMARKS: Field Moist.- 45.1%

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

GEOTECHNICAL & MATERIALS ENGINEERING DIVISION / Geotechnical Laboratory

LIQUID LIMIT AND PLASTICITY INDEX TESTS

ASTM D4318 / CTM 204

PROJECT NAME:	Adventure Park	**PCA:**	F21816i07
LABORATORY ID:	00120	**BOR./SAMP.:**	B1-6B
TESTED BY:	GP	**DATE TESTED:**	8/10/2017
CHECKED BY:	EH	**DATE CHECKED:**	3/15/2018
CLASSIFICATION:	CL	**- #(200):**	**76.2**

LIQUID LIMIT

Container Number	G-21
Number of Blows (N)	26
Wet Sample + Tare (gms.)	17.6110
Dry Sample + Tare (gms.)	16.9290
Wt. of Water (gms.)	0.682
Wt. of Tare (gms.)	15.2990
Wt. of Dry Soil (gms.)	1.630
Moisture Content (%, W_n)	41.8
Liquid Limit	**42**

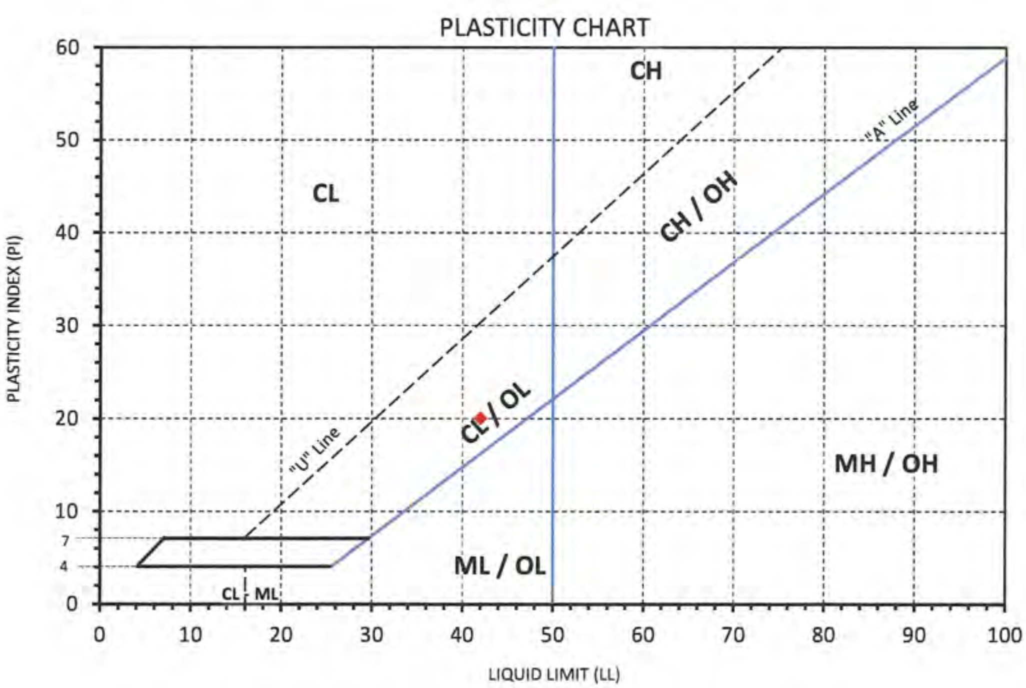
$LL = (W_n)(N/25)\ 0.121$

PLASTICITY INDEX

(LL-PL) = **20**

PLASTIC LIMIT

No. of Samples Tested	3		
Run Number	**1**	**2**	**3**
Container Number	G-30	G-37	G-38
Wet Sample + Tare (gms.)	17.2430	17.0200	17.0590
Dry Sample + Tare (gms.)	16.8980	16.6900	16.7030
Wt. of Water (gms.)	0.345	0.330	0.356
Wt. of Tare (gms.)	15.3640	15.2300	15.1000
Wt. of Dry Soil (gms.)	1.534	1.460	1.603
Moisture Content (%)	22.5	22.6	22.2
Plastic Limit (Avg. Value)		**22**	



LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00122
CLASSIFICATION: n/a
TESTED BY: GP
CHECKED BY: EH

PCA: F21816i07
BORING / SAMPLE: B1-8S
DEPTH (FT): 30-31.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 > 50%, CLAY or SILT

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1					
3/8"	9.52				100.0	
No. 4	4.76	0.01	0.9	0.9	99.1	
PAN	0	1.02				
TOTAL FRACTIONS		1.03				
OVEN-DRY FINES		0.91				
* TOTAL OVEN-DRY		0.92				

* Cobbles not included in total oven-dry weight
• If moisture was not taken from Course material a 1% moisture content will be assumed.

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	50.00
DRY WEIGHT (gm)	44.59
MOISTURE (%)	12.1

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	463.60
CALCULATED OVEN-DRY WEIGHT (gms)	413.44
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	417.07

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
10	2					
20	0.85					
40	0.425					
60	.2.5					
140	0.106					
200	0.074			8.0	92.0	
PAN	0					
TOTAL FRACTIONS		0.00	0.0			
TOTAL DRY WEIGHT AFTER WET SEIVING		29.70	7.1			
SIEVE LOSS-GAIN		29.70				

Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00123
CLASSIFICATION: n/a
TESTED BY: GP
CHECKED BY: EH

PCA: F21816i07
BORING / SAMPLE: B2-4S
DEPTH (FT): 20-21.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 > 50%, CLAY or SILT

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1				100.0	
3/8"	9.52	0.00	0.3	0.3	99.7	
No. 4	4.76	0.00	0.1	0.3	99.7	
PAN	0	1.87				
TOTAL FRACTIONS		1.88				
OVEN-DRY FINES		1.59				
* TOTAL OVEN-DRY		1.59				

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	50.00
DRY WEIGHT (gm)	42.30
MOISTURE (%)	18.2

* Cobbles not included in total oven-dry weight
- If moisture was not taken from Course material a 1% moisture content will be assumed.

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	200.00
CALCULATED OVEN-DRY WEIGHT (gms)	169.20
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	169.75

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
8	2.38					
16	1.19					
30	0.59					
50	0.297					
100	0.149					
200	0.074			28.1	71.9	
PAN	0					
TOTAL FRACTIONS		0.00	0.0			
TOTAL DRY WEIGHT AFTER WET SEIVING		47.20	27.8			
SIEVE LOSS-GAIN		47.20				

Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136
SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00125
CLASSIFICATION: n/a
TESTED BY: GP
CHECKED BY: EH
Cu / Cc: 8.3 0.3

PCA: F21816i07
BORING / SAMPLE: B2-6S
DEPTH (FT): 30-31.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 < 50%, SILT, SAND or DUAL

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1					
3/8"	9.52				100.0	
No. 4	4.76	0.00	0.1	0.1	99.9	
PAN	0	2.06				
TOTAL FRACTIONS		2.07				
OVEN-DRY FINES		1.69				
* TOTAL OVEN-DRY		1.70				

* Cobbles not included in total oven-dry weight

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	916.90
DRY WEIGHT (gm)	752.60
MOISTURE (%)	21.8

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	610.00
CALCULATED OVEN-DRY WEIGHT (gms)	500.69
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	500.99

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
8	2.38	0.02	0.0	0.1	99.9	
16	1.19	0.29	0.1	0.1	99.9	
30	0.59	15.09	3.0	3.1	96.9	
50	0.297	172.23	34.4	37.5	62.5	
100	0.149	194.33	38.8	76.3	23.7	
200	0.074	111.69	22.3	98.6	1.4	
PAN	0	6.36	1.3			
TOTAL FRACTIONS		500.01	99.8			
TOTAL DRY WEIGHT AFTER WET SEIVING		500.20	99.8			
SIEVE LOSS-GAIN		0.19	0.0			

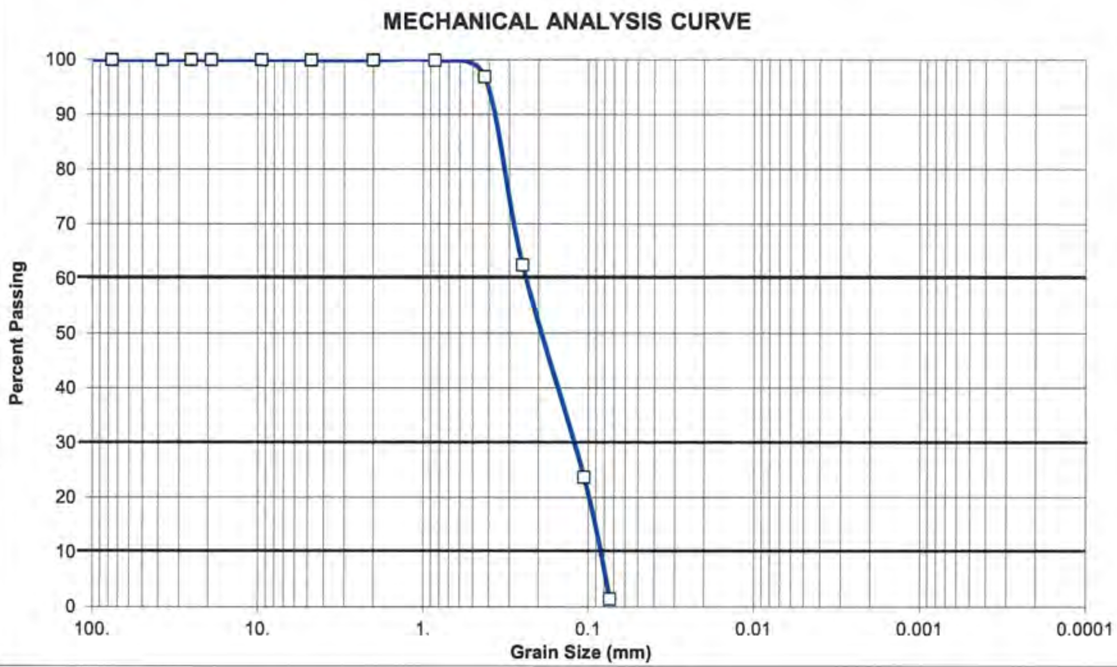
Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
MATERIALS ENGINEERING DIVISION
Geotechnical Laboratory
PARTICLE SIZE DISTRIBUTION REPORT

PROJECT NAME: Adventure Park **PCA:** F21816i07
CLASSIFICATION: n/a **BORING/SAMPLE:** B2-6S



	% COBBLES	% GRAVEL		% SAND			% FINES	
		coarse	fine	coarse	medium	fine	silt +	clay
(mm)*	(300-75)	(75-19)	(19-4.75)	(4.75-2)	(2-.425)	(.425-.075)	(-.075)	(-.005)
(%)	0.0	0.0	0.1	0.0	3.1	95.5	1.4	
TOTAL	-	= 0.1				= 98.5	#N/A	#N/A
*sieve#	(12" - 3")	(3" - 3/4")	(3/4"- #4)	(#4 - #10)	(#10 - #40)	(#40 - #200)	pass#200	pass#270

V	V
1st# passing	*2nd# retaining*

Avg. Organic Content
n/a %

SAND EQUIVALENT / ASTM D2419

	Cylind. 1	Cylind. 2	VALUE
Sand			n/a
Clay			

			(mm)		
% Retained #200 =	*98.6*	D_{10} =	*0.09*	$C_u = D_{60} / D_{10}$ =	*8.33*
% Retained # 4 =	*0.1*	D_{30} =	*0.13*	$C_c = D_{30}^2 / (D_{10}{*}D_{60})$ =	*0.25037*
% #4 / % #200 =	*0.1*	D_{60} =	*0.75*		

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division

Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00129
CLASSIFICATION: CL
TESTED BY: GP
CHECKED BY: EH

PCA: F21816i07
BORING / SAMPLE: B3-1R
DEPTH (FT): 5-6.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 > 50%, CLAY or SILT

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1					
3/8"	9.52				100.0	
No. 4	4.76	0.03	1.2	1.2	98.8	
PAN	0	2.94				
TOTAL FRACTIONS		2.97				
OVEN-DRY FINES		2.46				
* TOTAL OVEN-DRY		2.49				

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	300.00
DRY WEIGHT (gm)	251.20
MOISTURE (%)	19.4

* Cobbles not included in total oven-dry weight
• If moisture was not taken from Course material a 1% moisture content will be assumed.

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	600.00
CALCULATED OVEN-DRY WEIGHT (gms)	502.40
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	508.52

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
10	2					
20	0.85					
40	0.425					
60	.2.5					
140	0.106					
200	0.074			15.5	84.5	
PAN	0					
TOTAL FRACTIONS		0.00	0.0			
TOTAL DRY WEIGHT AFTER WET SEIVING		72.80	14.3			
SIEVE LOSS-GAIN		72.80				

Atterberg Test

Liquid Limit	44
Plastic Limit	19
Plastic Index	25

SOIL DESCRIP. / REMARKS:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

GEOTECHNICAL & MATERIALS ENGINEERING DIVISION / Geotechnical Laboratory

LIQUID LIMIT AND PLASTICITY INDEX TESTS

ASTM D4318 / CTM 204

PROJECT NAME:	Adventure Park	**PCA:**	F21816i07
LABORATORY ID:	00129	**BOR./SAMP.:**	B3-1R
TESTED BY:	GP	**DATE TESTED:**	8/17/2017
CHECKED BY:	EH	**DATE CHECKED:**	3/15/2018
CLASSIFICATION:	CL	**- #(200):**	**84.5**

LIQUID LIMIT

Container Number	G-21
Number of Blows (N)	29
Wet Sample + Tare (gms.)	18.6100
Dry Sample + Tare (gms.)	17.6190
Wt. of Water (gms.)	0.991
Wt. of Tare (gms.)	15.3010
Wt. of Dry Soil (gms.)	2.318
Moisture Content (%, W_n)	42.8
Liquid Limit	**44**

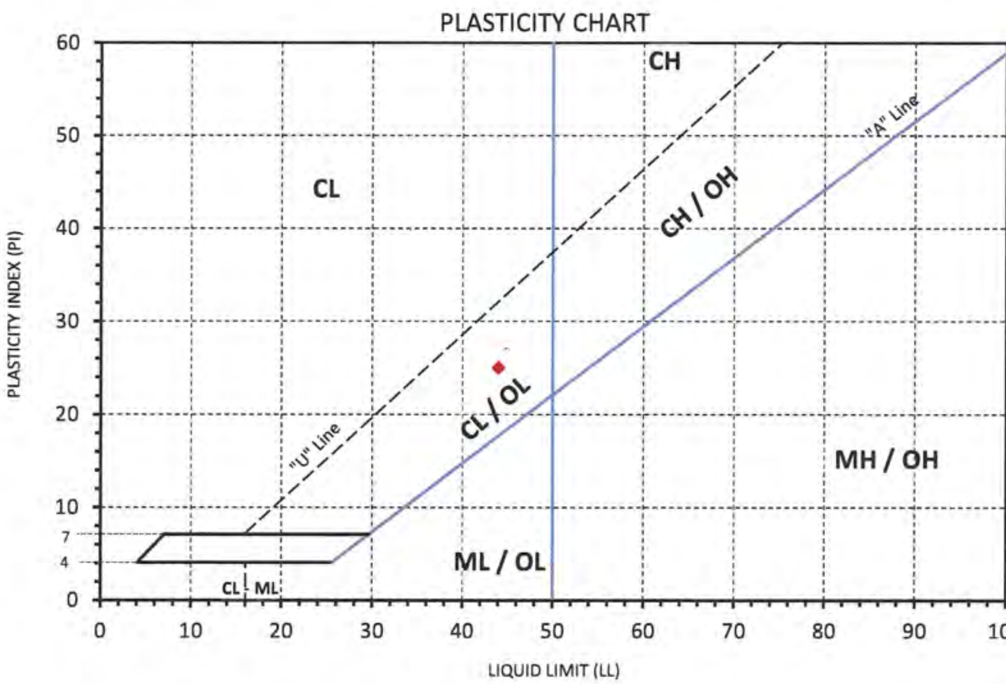
$LL = (W_n)(N/25)^{0.121}$

PLASTICITY INDEX

(LL-PL) = **25**

PLASTIC LIMIT

No. of Samples Tested	3		
Run Number	**1**	**2**	**3**
Container Number	G-30	G-37	G-38
Wet Sample + Tare (gms.)	17.1340	17.2960	17.0420
Dry Sample + Tare (gms.)	16.8560	16.9730	16.7340
Wt. of Water (gms.)	0.278	0.323	0.308
Wt. of Tare (gms.)	15.3650	15.2300	15.0990
Wt. of Dry Soil (gms.)	1.491	1.743	1.635
Moisture Content (%)	18.6	18.5	18.8
Plastic Limit (Avg. Value)		19	



LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00130
CLASSIFICATION: n/a
TESTED BY: GP
CHECKED BY: EH

PCA: F21816i07
BORING / SAMPLE: B3-2S
DEPTH (FT): 10-11.5
DATE TESTED: 8/21/17
DATE CHECKED: 9/11/17

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 > 50%, CLAY or SILT

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1					
3/8"	9.52				100.0	
No. 4	4.76	0.00	0.2	0.2	99.8	
PAN	0	1.86				
TOTAL FRACTIONS		1.86				
OVEN-DRY FINES		1.57				
* TOTAL OVEN-DRY		1.58				

* Cobbles not included in total oven-dry weight

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	50.00
DRY WEIGHT (gm)	42.40
MOISTURE (%)	17.9

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	600.00
CALCULATED OVEN-DRY WEIGHT (gms)	508.80
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	509.77

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
10	2					
20	0.85					
40	0.425					
60	.2.5					
140	0.106					
200	0.074			16.5	83.5	
PAN	0					
TOTAL FRACTIONS		0.00	0.0			
TOTAL DRY WEIGHT AFTER WET SEIVING		83.30	16.3			
SIEVE LOSS-GAIN		83.30				

Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00133
CLASSIFICATION: n/a
TESTED BY: GP
CHECKED BY: EH

PCA: F21816i07
BORING / SAMPLE: B3-6R
DEPTH (FT): 25-26.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 > 50%, CLAY or SILT

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1					
3/8"	9.52				100.0	
No. 4	4.76	0.00	0.0	0.0	100.0	
PAN	0	5.87				
TOTAL FRACTIONS		5.87				
OVEN-DRY FINES		5.57				
* TOTAL OVEN-DRY		5.57				

* Cobbles not included in total oven-dry weight

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	137.10
DRY WEIGHT (gm)	130.20
MOISTURE (%)	5.3

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	600.00
CALCULATED OVEN-DRY WEIGHT (gms)	569.80
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	570.01

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
8	2.38					
16	1.19					
30	0.59					
50	0.297					
100	0.149					
200	0.074			10.2	89.8	
PAN	0					
TOTAL FRACTIONS		0.00	0.0			
TOTAL DRY WEIGHT AFTER WET SEIVING		57.70	10.1			
SIEVE LOSS-GAIN		57.70				

Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00134
CLASSIFICATION: n/a
TESTED BY: GP
CHECKED BY: EH
Cu / Cc: 2.6 0.8

PCA: F21816i07
BORING / SAMPLE: B3-7R
DEPTH (FT): 30-31.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 < 50%, SILT, SAND or DUAL

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1				100.0	
3/8"	9.52	0.03	1.7	1.7	98.3	
No. 4	4.76	0.06	3.3	5.0	95.0	
PAN	0	1.95				
TOTAL FRACTIONS		2.04				
OVEN-DRY FINES		1.71				
* TOTAL OVEN-DRY		1.80				

* Cobbles not included in total oven-dry weight
• If moisture was not taken from Course material a 1% moisture content will be assumed.

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	919.80
DRY WEIGHT (gm)	806.90
MOISTURE (%)	14.0

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	675.00
CALCULATED OVEN-DRY WEIGHT (gms)	592.15
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	623.32

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
8	2.38	37.45	6.0	11.0	89.0	
16	1.19	95.85	15.4	26.4	73.6	
30	0.59	210.51	33.8	60.2	39.8	
50	0.297	192.69	30.9	91.1	8.9	
100	0.149	36.36	5.8	96.9	3.1	
200	0.074	12.98	2.1	99.0	1.0	
PAN	0	0.08	0.0			
TOTAL FRACTIONS		585.92	94.0			
TOTAL DRY WEIGHT AFTER WET SEIVING		585.90	94.0			
SIEVE LOSS-GAIN		-0.02	0.0			

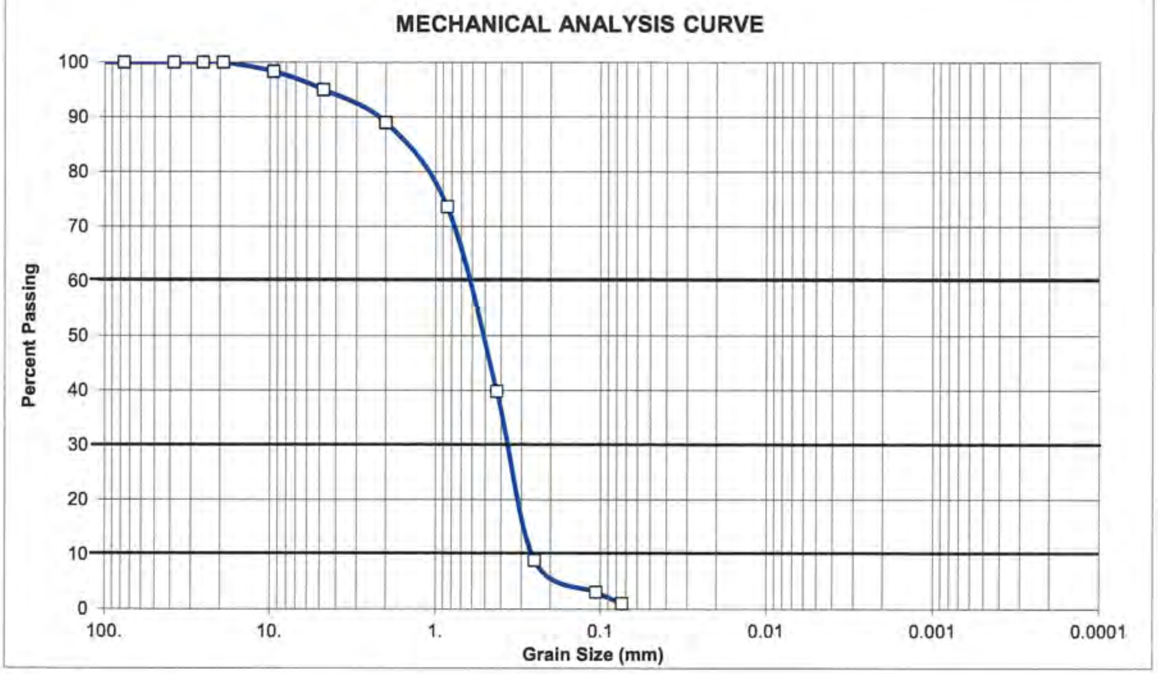
Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
MATERIALS ENGINEERING DIVISION
Geotechnical Laboratory
PARTICLE SIZE DISTRIBUTION REPORT

PROJECT NAME: Adventure Park **PCA:** F21816i07
CLASSIFICATION: n/a **BORING/SAMPLE:** B3-7R



	% COBBLES	% GRAVEL		% SAND			% FINES	
		coarse	fine	coarse	medium	fine	silt +	clay
(mm)*	(300-75)	(75-19)	(19-4.75)	(4.75-2)	(2-.425)	(.425-.075)	(-.075)	(-.005)
(%)	0.0	0.0	5.0	6.0	49.1	38.8	1.0	
TOTAL	-	= 5.0				= 94.0	#N/A	#N/A
*sieve#	(12" - 3")	(3" - 3/4")	(3/4"- #4)	(#4 - #10)	(#10 - #40)	(#40 - #200)	pass#200	pass#270

| V V
1st# passing *2nd# retaining*

Avg. Organic Content
n/a %

SAND EQUIVALENT / ASTM D2419

	Cylind. 1	Cylind. 2	VALUE
Sand			n/a
Clay			

			(mm)		
% Retained #200 =	***99.0***	D_{10} =	***0.25***	$C_u = D_{60} / D_{10}$ =	***2.60***
% Retained # 4 =	***5.0***	D_{30} =	***0.36***	$C_c = D_{30}^2 / (D_{10} * D_{60})$ =	***0.79754***
% #4 / % #200 =	*5.1*	D_{60} =	***0.65***		

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division

Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00135
CLASSIFICATION: SP (close to SW)
TESTED BY: GP
CHECKED BY: EH
Cu / Cc: 22.5 3.1

PCA: F21816i07
BORING / SAMPLE: B3-8R
DEPTH (FT): 35-36.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 < 50%, SILT, SAND or DUAL

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1				100.0	
3/8"	9.52	0.09	3.3	3.3	96.7	
No. 4	4.76	0.04	1.5	4.8	95.2	
PAN	0	2.67				
TOTAL FRACTIONS		2.79				
OVEN-DRY FINES		2.46				
* TOTAL OVEN-DRY		2.59				

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	549.60
DRY WEIGHT (gm)	507.80
MOISTURE (%)	8.2

* Cobbles not included in total oven-dry weight
• If moisture was not taken from Course material a 1% moisture content will be assumed.

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	507.80
CALCULATED OVEN-DRY WEIGHT (gms)	469.18
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	492.81

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
10	2					
20	0.85					
40	0.425					
60	.2.5					
140	0.106					
200	0.074			98.6	1.4	
PAN	0					
TOTAL FRACTIONS		0.00	0.0			
TOTAL DRY WEIGHT AFTER WET SEIVING		462.40	93.8			
SIEVE LOSS-GAIN		462.40				

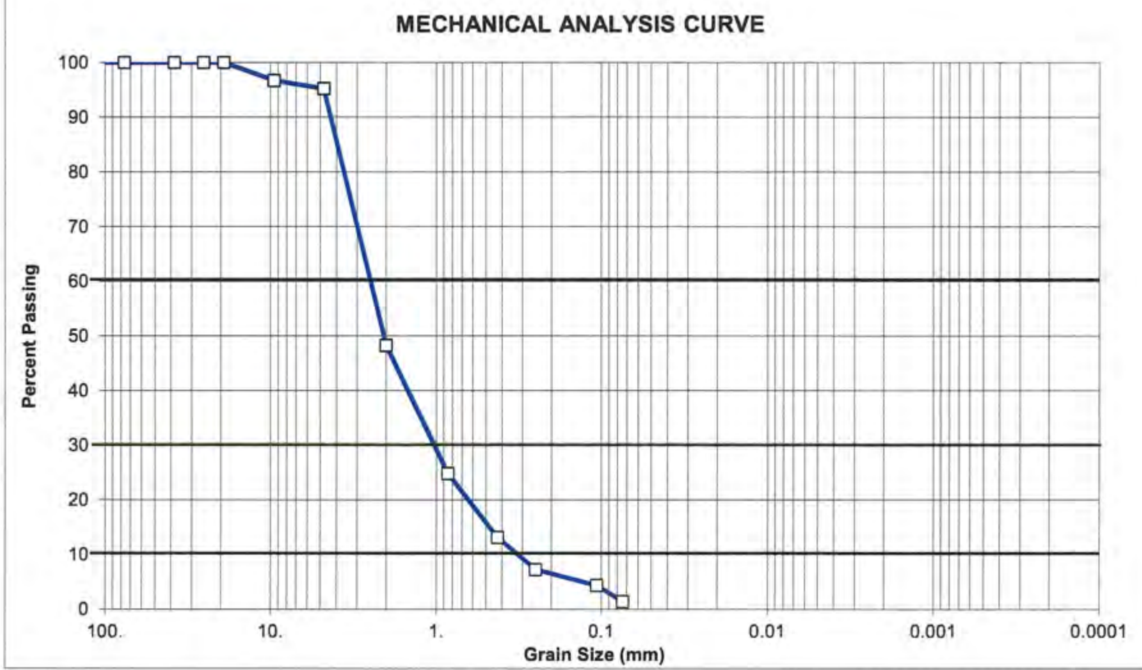
Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS: non plastic

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
MATERIALS ENGINEERING DIVISION
Geotechnical Laboratory
PARTICLE SIZE DISTRIBUTION REPORT

PROJECT NAME: Adventure Park **PCA:** F21816i07
CLASSIFICATION: SP (close to SW) **BORING/SAMPLE:** B3-8R



	% COBBLES	% GRAVEL		% SAND			% FINES	
		coarse	fine	coarse	medium	fine	silt +	clay
(mm)*	(300-75)	(75-19)	(19-4.75)	(4.75-2)	(2-.425)	(.425-.075)	(-.075)	(-.005)
(%)	0.0	0.0	4.8	0.0	0.0	0.0	0.0	
TOTAL	-	= 4.8			=	0.0	#N/A	#N/A
*sieve#	(12" - 3")	(3" - 3/4")	(3/4"- #4)	(#4 - #10)	(#10 - #40)	(#40 - #200)	pass#200	pass#270

V V
1st# passing *2nd# retaining*

Avg. Organic Content
n/a %

SAND EQUIVALENT / ASTM D2419

	Cylind. 1	Cylind. 2	VALUE
Sand / Clay			n/a

			(mm)	
% Retained #200 =	98.6	D_{10} =	0.12	$C_u = D_{60} / D_{10}$ = 22.50
% Retained # 4 =	4.8	D_{30} =	1.00	$C_c = D_{30}^2 / (D_{10} \cdot D_{60})$ = 3.08642
% #4 / % #200 =	4.9	D_{60} =	2.70	

No wash grading was performed points on the distribution curve from the #8 - #100 sieves were averaged using data from the % Accu. Pass. #4 & #200 sieves -- Data should not be used as actual test results but only referred to as a reference

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00138
CLASSIFICATION: SW
TESTED BY: GP
CHECKED BY: EH
Cu / Cc: 3.4 1.6

PCA: F21816i07
BORING / SAMPLE: B3-9S
DEPTH (FT): 40-41.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 < 50%, SILT, SAND or DUAL

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1					
3/8"	9.52				100.0	
No. 4	4.76	0.05	3.2	3.2	96.8	
PAN	0	1.79				
TOTAL FRACTIONS		1.84				
OVEN-DRY FINES		1.54				
* TOTAL OVEN-DRY		1.58				

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	838.70
DRY WEIGHT (gm)	719.30
MOISTURE (%)	16.6

* Cobbles not included in total oven-dry weight
- If moisture was not taken from Course material a 1% moisture content will be assumed.

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	605.00
CALCULATED OVEN-DRY WEIGHT (gms)	518.87
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	535.78

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
8	2.38	16.10	3.0	6.2	93.8	
16	1.19	48.83	9.1	15.3	84.7	
30	0.59	122.62	22.9	38.2	61.8	
50	0.297	180.24	33.6	71.8	28.2	
100	0.149	99.77	18.6	90.4	9.6	
200	0.074	50.09	9.3	99.8	0.2	
PAN	0	1.23	0.2			
TOTAL FRACTIONS		518.88	96.8			
TOTAL DRY WEIGHT AFTER WET SEIVING		518.80	96.8			
SIEVE LOSS-GAIN		-0.08	0.0			

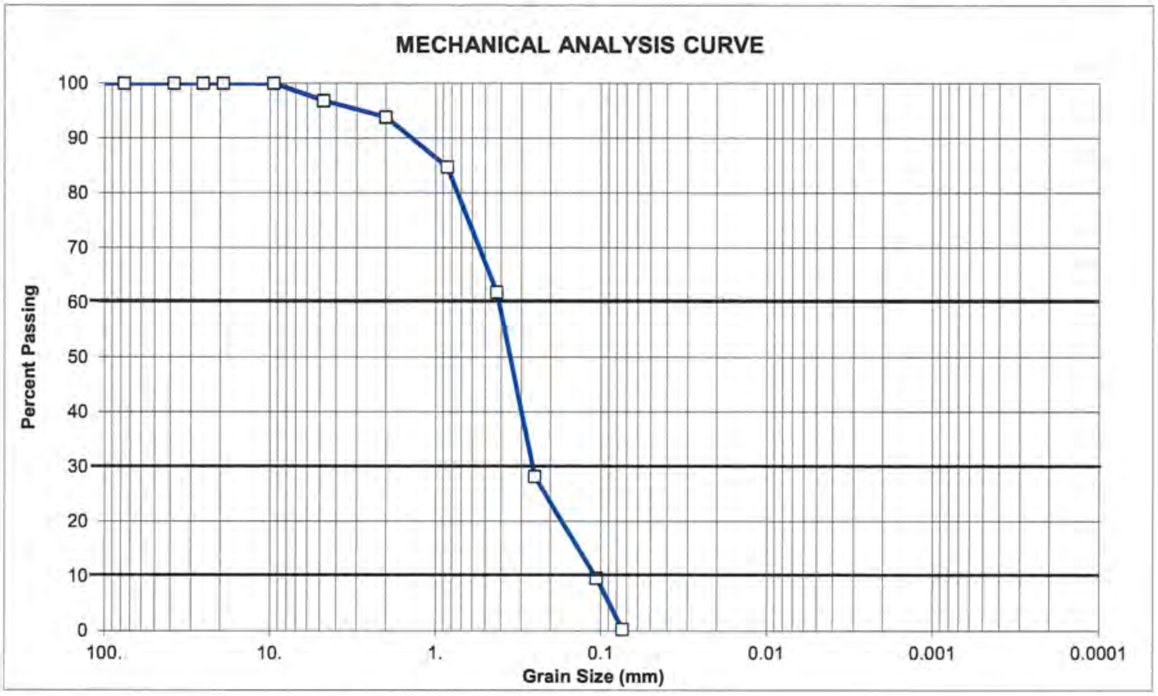
Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
MATERIALS ENGINEERING DIVISION
Geotechnical Laboratory
PARTICLE SIZE DISTRIBUTION REPORT

PROJECT NAME: Adventure Park **PCA:** F21816i07
CLASSIFICATION: SW **BORING/SAMPLE:** B3-9S



	% COBBLES	% GRAVEL		% SAND			% FINES	
		coarse	fine	coarse	medium	fine	silt +	clay
(mm)*	(300-75)	(75-19)	(19-4.75)	(4.75-2)	(2-.425)	(.425-.075)	(-.075)	(-.005)
(%)	0.0	0.0	3.2	3.0	32.0	61.6	0.2	
TOTAL	-	= 3.2			=	96.6	#N/A	#N/A
*sieve#	(12" - 3")	(3" - 3/4")	(3/4"- #4)	(#4 - #10)	(#10 - #40)	(#40 - #200)	pass#200	pass#270

1st# passing | V   *2nd# retaining* | V

Avg. Organic Content
n/a %

SAND EQUIVALENT / ASTM D2419

	Cylind. 1	Cylind. 2	VALUE
Sand / Clay			n/a

			(mm)	
% Retained #200 =	***99.8***	D_{10} =	***0.12***	$C_u = D_{60} / D_{10}$ = ***3.42***
% Retained # 4 =	***3.2***	D_{30} =	***0.28***	$C_c = D_{30}^2 / (D_{10} * D_{60})$ = ***1.5935***
% #4 / % #200 =	*3.2*	D_{60} =	***0.41***	

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS

Geotechnical and Materials Engineering Division

Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00136
CLASSIFICATION: n/a
TESTED BY: GP
CHECKED BY: EH
Cu / Cc: 4.2 1.3

PCA: F21816i07
BORING / SAMPLE: B3-10R
DEPTH (FT): 45-46.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 < 50%, SILT, SAND or DUAL

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	152.4					
3"	76.2					
1 1/2"	38.1					
1"	25.4					
3/4"	19.1					
3/8"	9.52				100.0	
No. 4	4.76	0.11	3.8	3.8	96.2	
PAN	0	3.11				
TOTAL FRACTIONS		3.22				
OVEN-DRY FINES		2.78				
* TOTAL OVEN-DRY		2.89				

* Cobbles not included in total oven-dry weight
• If moisture was not taken from Course material a 1% moisture content will be assumed.

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	1407.60
DRY WEIGHT (gm)	1260.70
MOISTURE (%)	11.7

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	600.00
CALCULATED OVEN-DRY WEIGHT (gms)	537.38
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	558.62

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
10	2	25.27	4.5	8.3	91.7	
20	0.85	82.07	14.7	23.0	77.0	
40	0.425	143.32	25.7	48.7	51.3	
60	.2.5	165.40	29.6	78.3	21.7	
140	0.106	79.94	14.3	92.6	7.4	
200	0.074	40.21	7.2	99.8	0.2	
PAN	0	0.99	0.2			
TOTAL FRACTIONS		537.20	96.2			
TOTAL DRY WEIGHT AFTER WET SEIVING		537.19	96.2			
SIEVE LOSS-GAIN		-0.01	0.0			

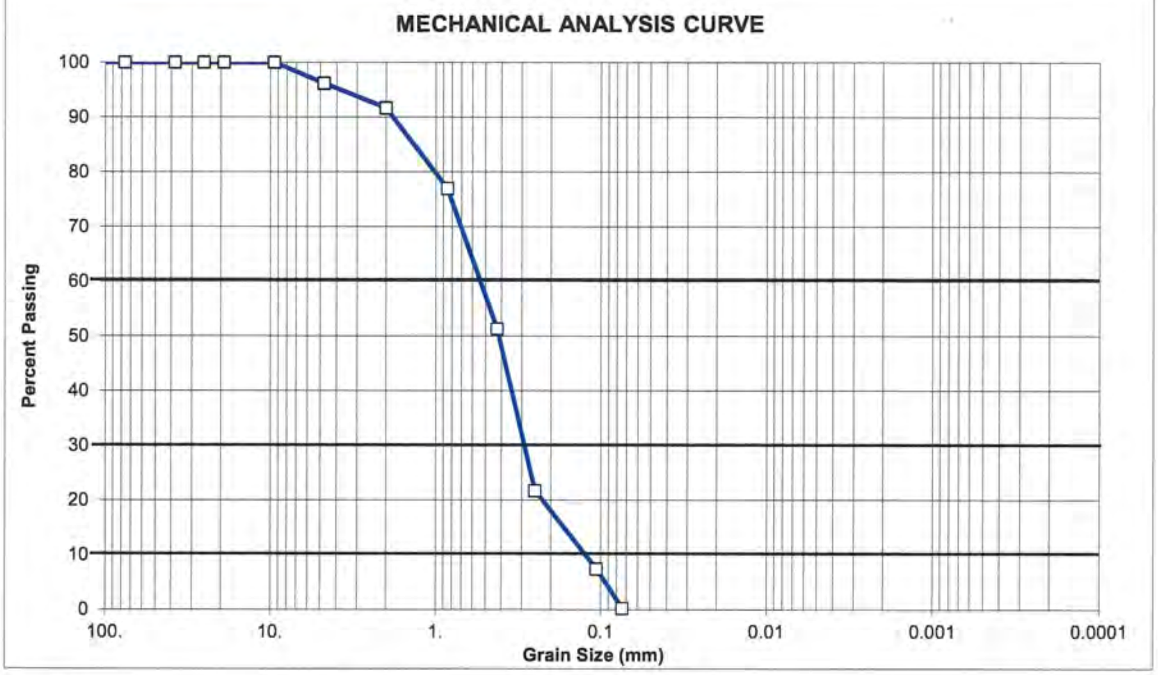
Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS:

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
MATERIALS ENGINEERING DIVISION
Geotechnical Laboratory
PARTICLE SIZE DISTRIBUTION REPORT

PROJECT NAME: Adventure Park **PCA:** F21816i07
CLASSIFICATION: n/a **BORING/SAMPLE:** B3-10R



	% COBBLES	% GRAVEL		% SAND			% FINES	
		coarse	fine	coarse	medium	fine	silt +	clay
(mm)*	(300-75)	(75-19)	(19-4.75)	(4.75-2)	(2-.425)	(.425-.075)	(-.075)	(-.005)
(%)	0.0	0.0	3.8	4.5	40.3	51.1	0.2	
TOTAL	-	= 3.8				= 96.0	#N/A	#N/A
*sieve#	(12" - 3")	(3" - 3/4")	(3/4"- #4)	(#4 - #10)	(#10 - #40)	(#40 - #200)	pass#200	pass#270

V	V
1st#	*2nd#*
passing	*retaining*

Avg. Organic Content
n/a %

SAND EQUIVALENT / ASTM D2419

	Cylind. 1	Cylind. 2	VALUE
Sand			n/a
Clay			

			(mm)		
% Retained #200 =	***99.8***	D_{10} =	***0.13***	$C_u = D_{60} / D_{10}$ =	***4.23***
% Retained # 4 =	***3.8***	D_{30} =	***0.30***	$C_c = D_{30}^2 / (D_{10} * D_{60})$ =	***1.25874***
% #4 / % #200 =	*3.8*	D_{60} =	***0.55***		

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
Geotechnical and Materials Engineering Division
Geotechnical Laboratory - ASTM D2487, D6913, C117, C136

SIEVE ANALYSIS WORKSHEET

PROJECT NAME: Adventure Park
LAB. ID: 00137
CLASSIFICATION: n/a
TESTED BY: GP
CHECKED BY: EH
Cu / Cc: 2.5 1.1

PCA: F21816i07
BORING / SAMPLE: B3-11S
DEPTH (FT): 50-51.5
DATE TESTED: 8/21/17
DATE CHECKED: 3/15/18

If % Accum. Ret. #4 / % Accum. Ret. #200 > 50%, then Gravel
If % Passing #200 < 50%, SILT, SAND or DUAL

COARSE (Plus no. 4)

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (lb)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
6"	*152.4*					
3"	*76.2*					
1 1/2"	*38.1*					
1"	*25.4*					
3/4"	*19.1*					
3/8"	*9.52*				100.0	
No. 4	*4.76*	0.00	0.1	0.1	99.9	
PAN	*0*	1.68				
TOTAL FRACTIONS		1.68				
OVEN-DRY FINES		1.47				
* TOTAL OVEN-DRY		1.47				

* Cobbles not included in total oven-dry weight

MOISTURE CONTENT OF FINES	
WET WEIGHT (gm)	664.80
DRY WEIGHT (gm)	583.30
MOISTURE (%)	14.0

MOISTURE CONTENT OF COURSE	
Wet WGT. (gm)	0.00
Dry WGT. (gm)	0.00
MOISTURE (%)	0.01

FINES (Minus no. 4)

WET WEIGHT OF FINES USED FOR WASHING (gms)	618.00
CALCULATED OVEN-DRY WEIGHT (gms)	542.24
WT. OF TOTAL SAMPLE REPRESENTED BY FINES, OVEN-DRY (gms):	542.61

ASTM SIEVE NUMBER	SIZE (mm)	RETAINED (gms)	% OF TOTAL OVEN DRY RETAINED	ACCUM. % RETAINED	ACCUM. % PASSING ACTUAL	ACCUM. % PASSING SPEC. REQ.
10	*2*	11.41	2.1	2.2	97.8	
20	*0.85*	34.89	6.4	8.6	91.4	
40	*0.425*	113.69	21.0	29.6	70.4	
60	*.2.5*	215.76	39.8	69.3	30.7	
140	*0.106*	128.88	23.8	93.1	6.9	
200	*0.074*	36.54	6.7	99.8	0.2	
PAN	*0*	1.46	0.3			
TOTAL FRACTIONS		542.63	100.0			
TOTAL DRY WEIGHT AFTER WET SEIVING		542.60	100.0			
SIEVE LOSS-GAIN		-0.03	0.0			

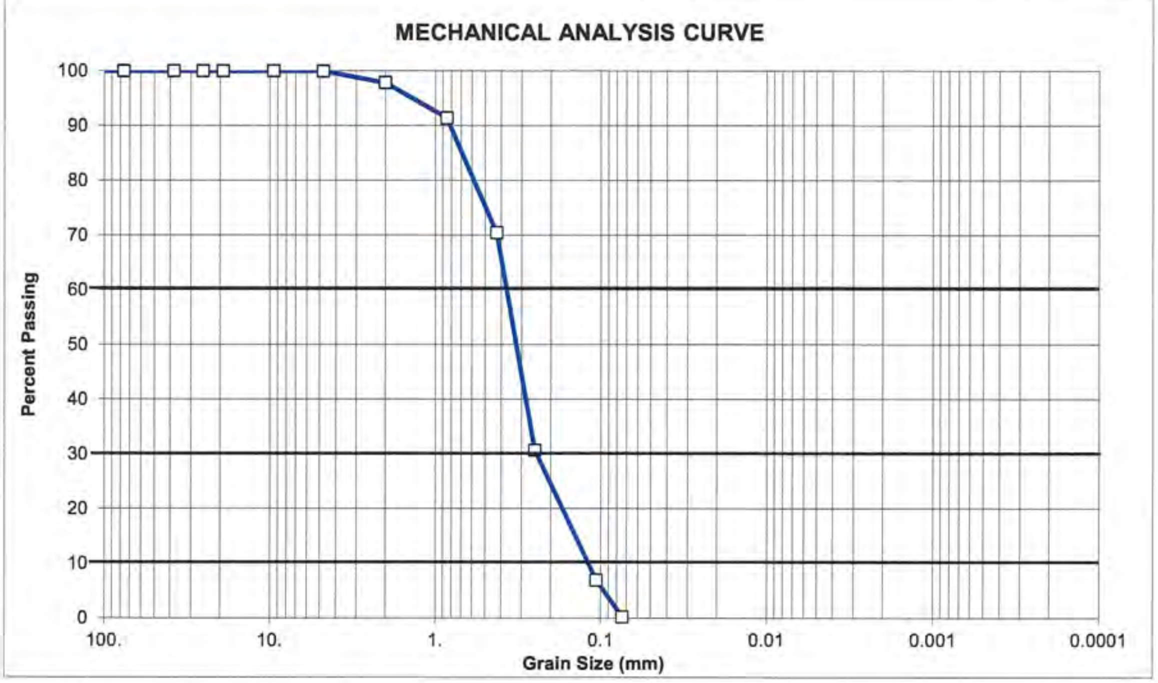
Atterberg Test

Liquid Limit	n/a
Plastic Limit	n/a
Plastic Index	n/a

SOIL DESCRIP. / REMARKS: Field Moist. - 15.5%

LOS ANGELES COUNTY DEPARTMENT OF PUBLIC WORKS
MATERIALS ENGINEERING DIVISION
Geotechnical Laboratory
PARTICLE SIZE DISTRIBUTION REPORT

PROJECT NAME: Adventure Park **PCA:** F21816i07
CLASSIFICATION: n/a **BORING/SAMPLE:** B3-11S



	% COBBLES	% GRAVEL		% SAND			% FINES	
		coarse	fine	coarse	medium	fine	silt +	clay
(mm)*	(300-75)	(75-19)	(19-4.75)	(4.75-2)	(2-.425)	(.425-.075)	(-.075)	(-.005)
(%)	0.0	0.0	0.1	2.1	27.4	70.2	0.2	
TOTAL	-	= 0.1			=	99.7	#N/A	#N/A
*sieve#	(12" - 3")	(3" - 3/4")	(3/4"- #4)	(#4 - #10)	(#10 - #40)	(#40 - #200)	pass#200	pass#270

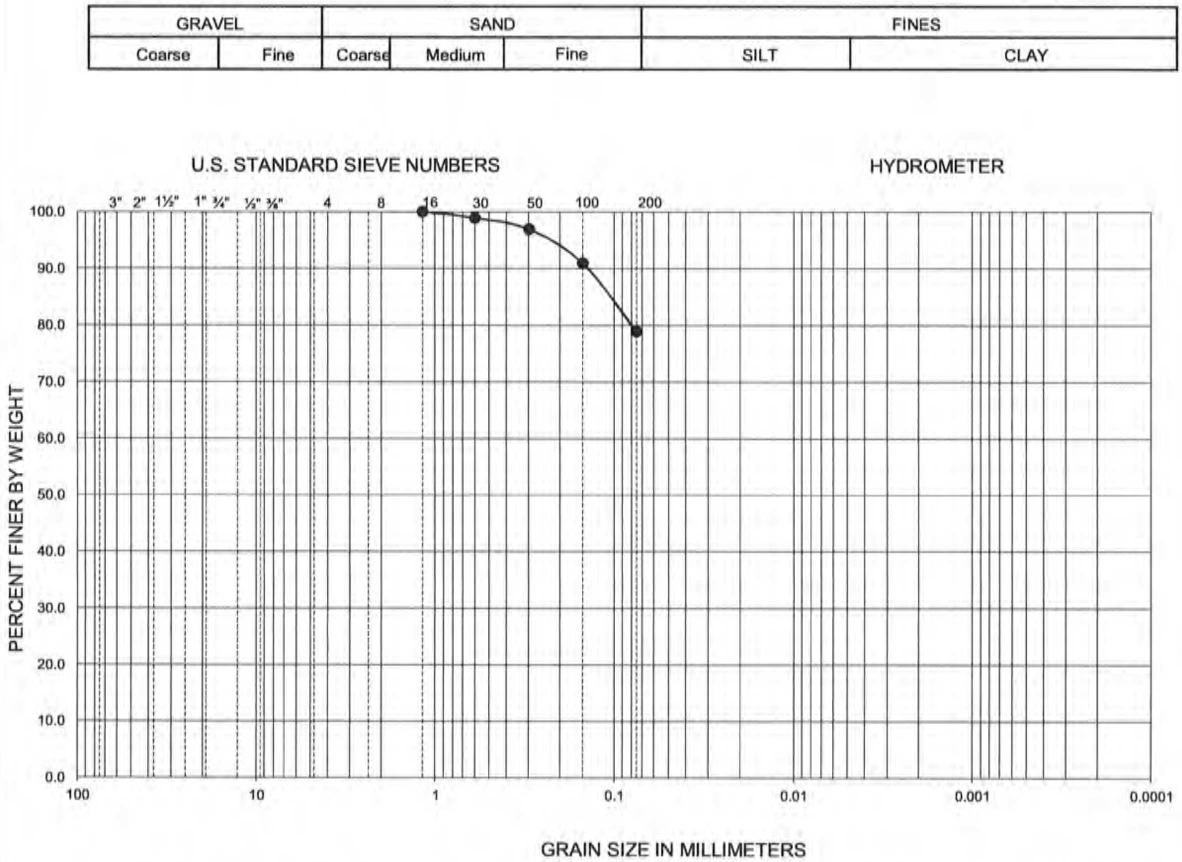
(12" - 3"): 1st# passing, 2nd# retaining

Avg. Organic Content
n/a %

SAND EQUIVALENT / ASTM D2419

	Cylind. 1	Cylind. 2	VALUE
Sand			n/a
Clay			

			(mm)	
% Retained #200 =	99.8	D_{10} =	0.16	$C_u = D_{60} / D_{10}$ = 2.50
% Retained # 4 =	0.1	D_{30} =	0.27	$C_c = D_{30}^2 / (D_{10} * D_{60})$ = 1.13906
% #4 / % #200 =	0.1	D_{60} =	0.40	

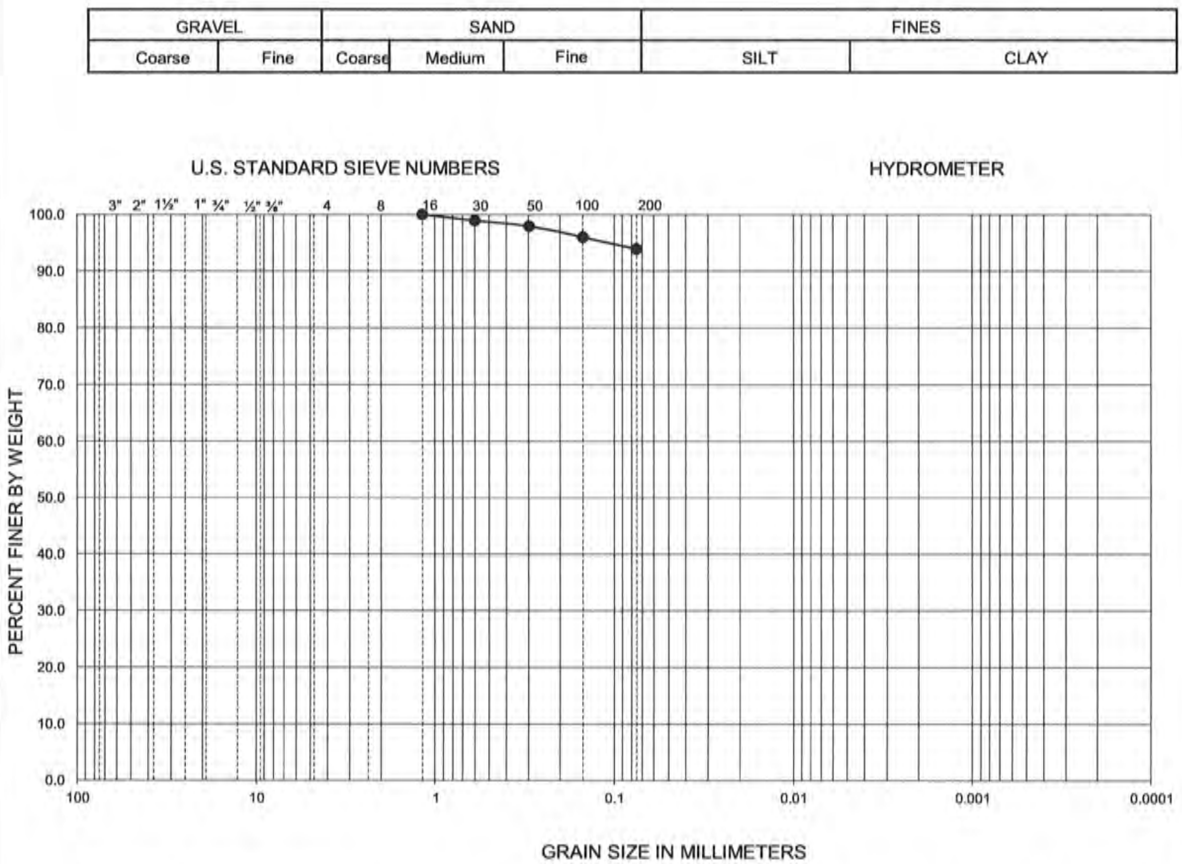


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-1	5.0-6.5	--	--	--	--	--	--	--	--	79	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	ADVENTURE PARK UPPER SAN GABRIEL RIVER EWMP LOS ANGLES COUNTY, CALIFORNIA	B-1
107900001	6/15		

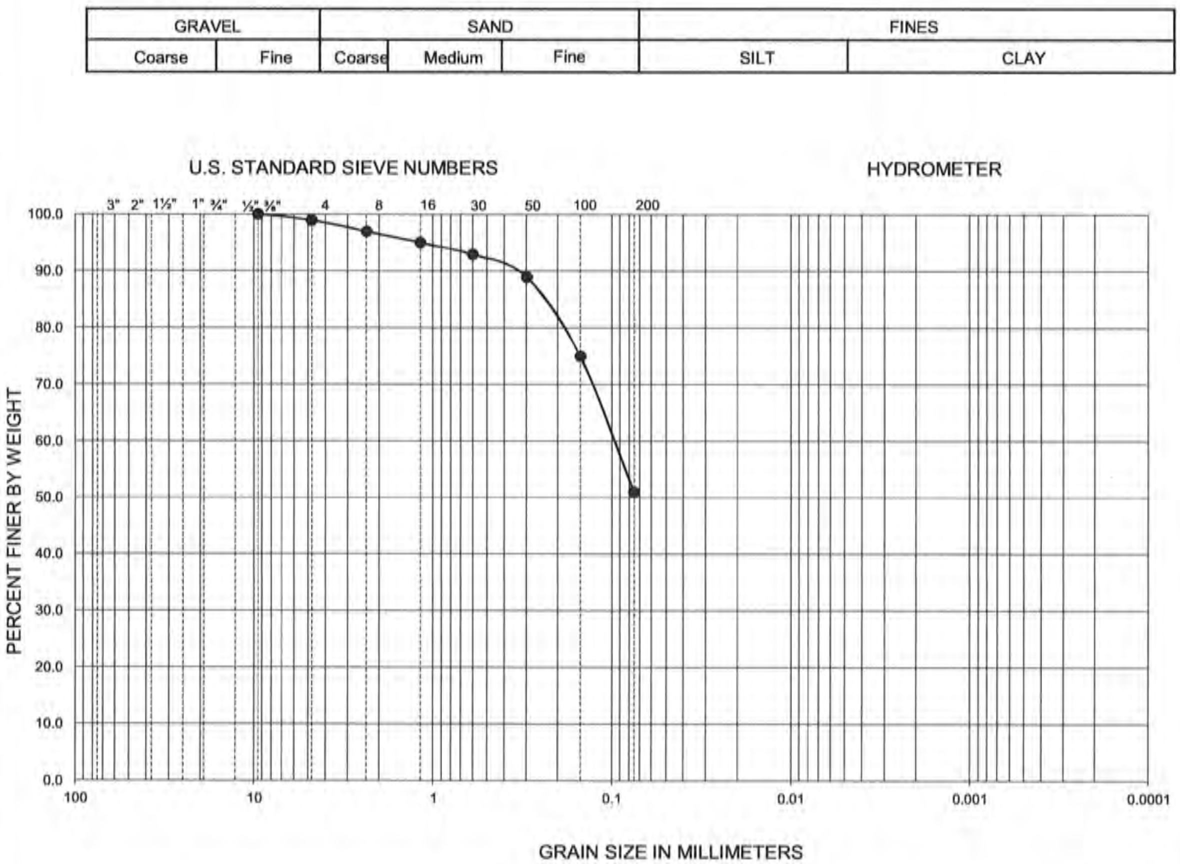
107900001_SIEVE B-1 @ 5.0-6.5.xls



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-1	10.0-11.5	--	--	--	--	--	--	--	--	94	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore	GRADATION TEST RESULTS	FIGURE
PROJECT NO. / 107900001	DATE / 6/15	ADVENTURE PARK UPPER SAN GABRIEL RIVER EWMP LOS ANGLES COUNTY, CALIFORNIA
		B-2



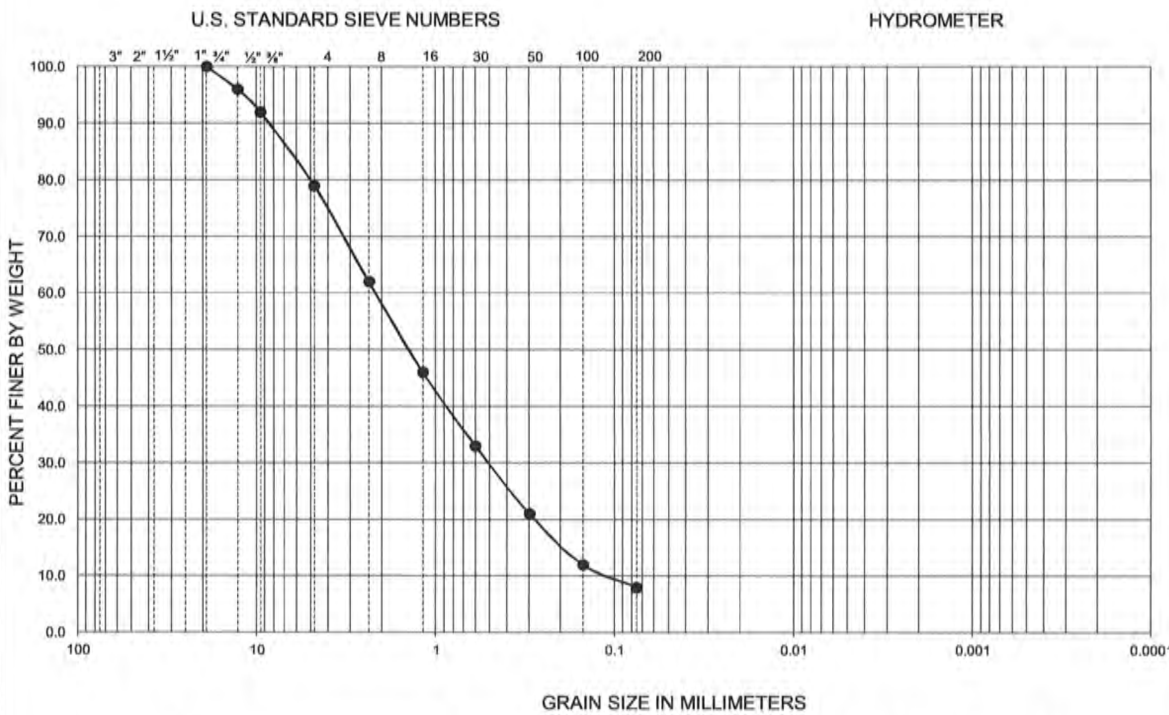
Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-1	15.0-16.5	--	--	--	--	--	--	--	--	51	ML

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	ADVENTURE PARK UPPER SAN GABRIEL RIVER EWMP LOS ANGLES COUNTY, CALIFORNIA	B-3
107900001	6/15		

107900001_SIEVE B-1 @ 15.0-16.5.xls

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-1	30.0-31.5	--	--	--	0.12	0.51	2.15	17.9	1.0	8	SW-SM

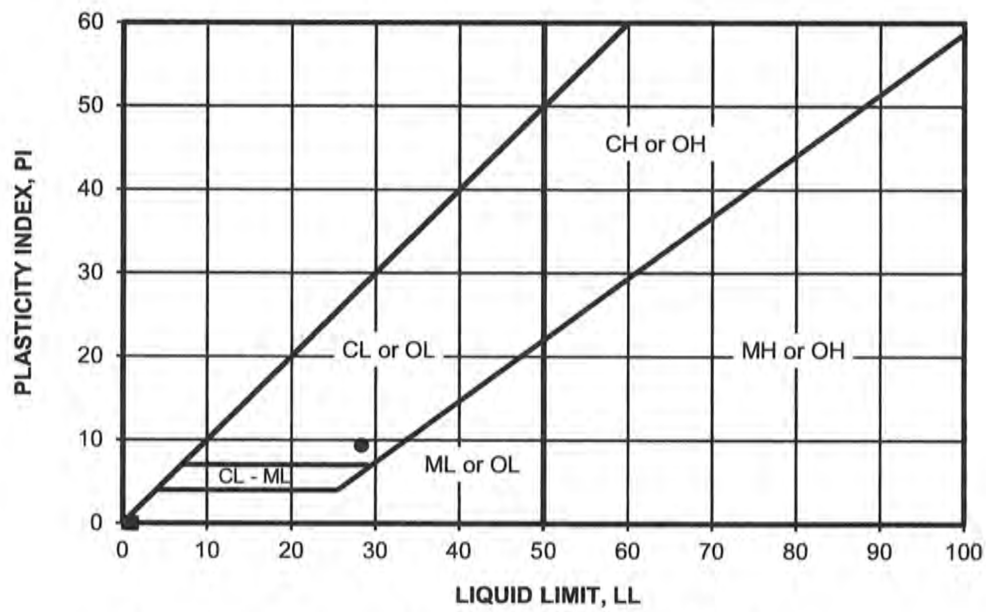
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	ADVENTURE PARK UPPER SAN GABRIEL RIVER EWMP LOS ANGLES COUNTY, CALIFORNIA	B-4
107900001	6/15		

107900001_SIEVE B-1 @ 30.0-31.5.xls

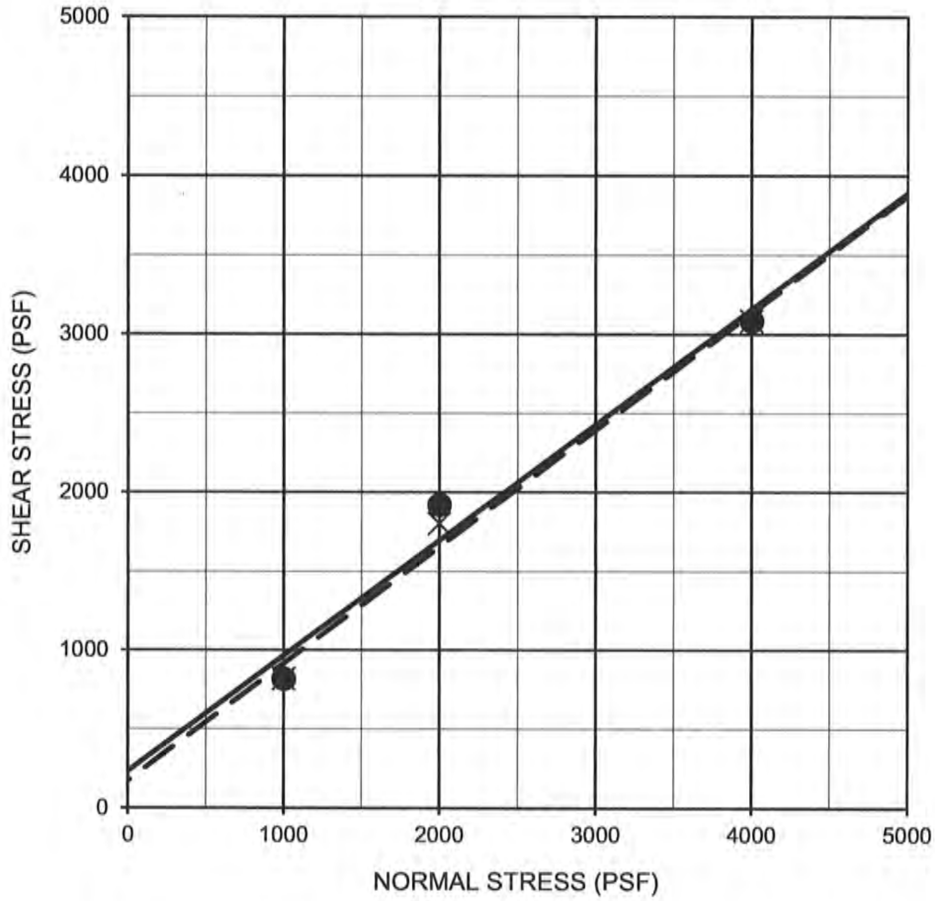
SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
●	B-1	10.0-11.5	28	19	9	CL	CL

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

Ninyo & Moore	ATTERBERG LIMITS TEST RESULTS	FIGURE
PROJECT NO. 107900001 / DATE 6/15	ADVENTURE PARK UPPER SAN GABRIEL RIVER EWMP LOS ANGLES COUNTY, CALIFORNIA	B-5



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SAND	—●—	B-1	25.0-26.5	Peak	230	36	SM
Silty SAND	- - x - -	B-1	25.0-26.5	Ultimate	170	36	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	ADVENTURE PARK UPPER SAN GABRIEL RIVER EWMP LOS ANGELES COUNTY, CALIFORNIA	B-6
107900001	6/15		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH [1]	RESISTIVITY [1] (Ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	6.0-10.0	7.0	950	220	0.022	155

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore

CORROSIVITY TEST RESULTS

PROJECT NO.	DATE
107900001	6/15

ADVENTURE PARK
UPPER SAN GABRIEL RIVER EWMP
LOS ANGELES COUNTY, CALIFORNIA

FIGURE
B-7

107900001 CORROSIVITY1.xlsx

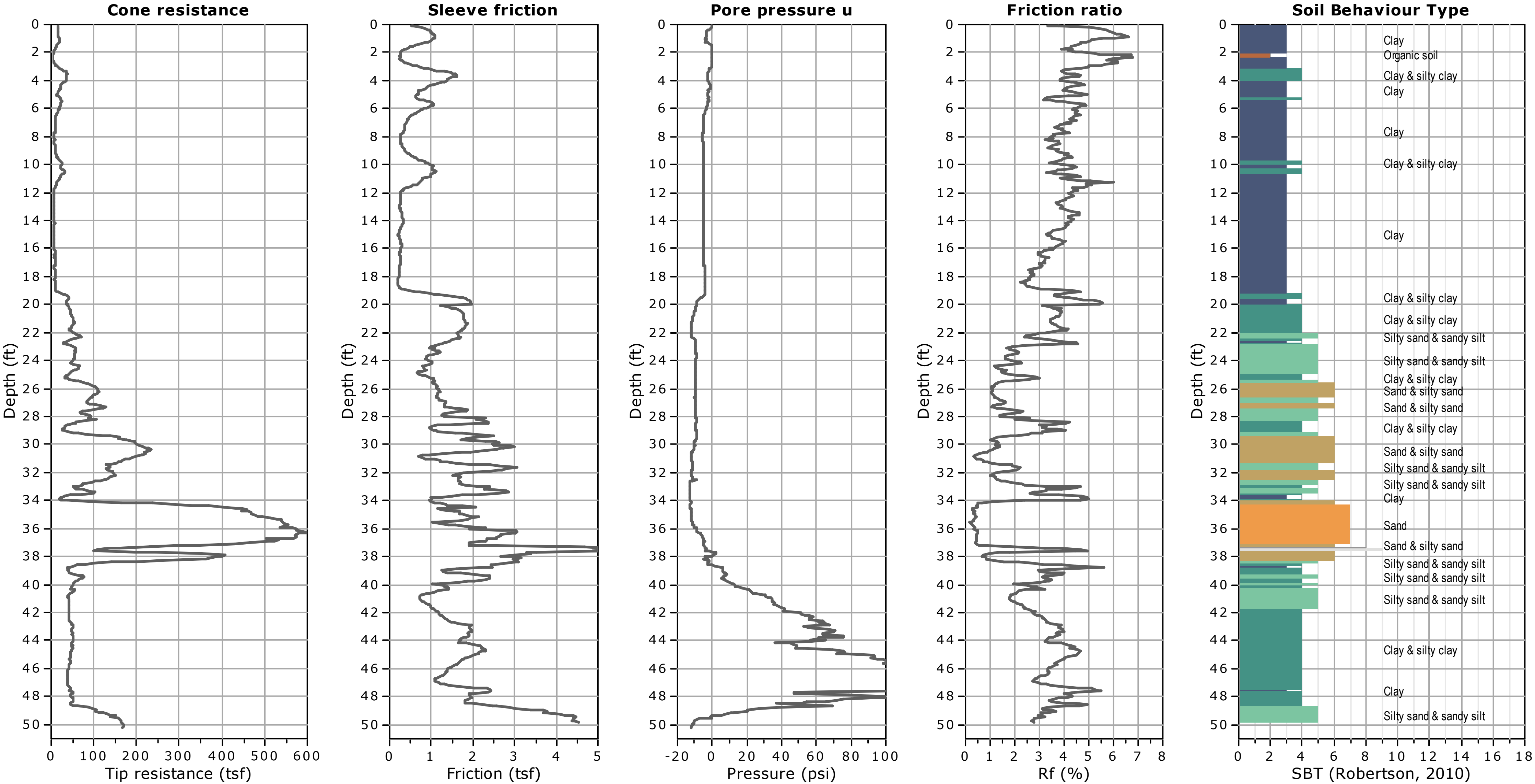
Appendix D

Logs of Tetra Tech Cone Penetration Tests (CPTs)

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

Project: Tetra Tech BAS / Adventure Park Stormwater
Location: Whittier, CA

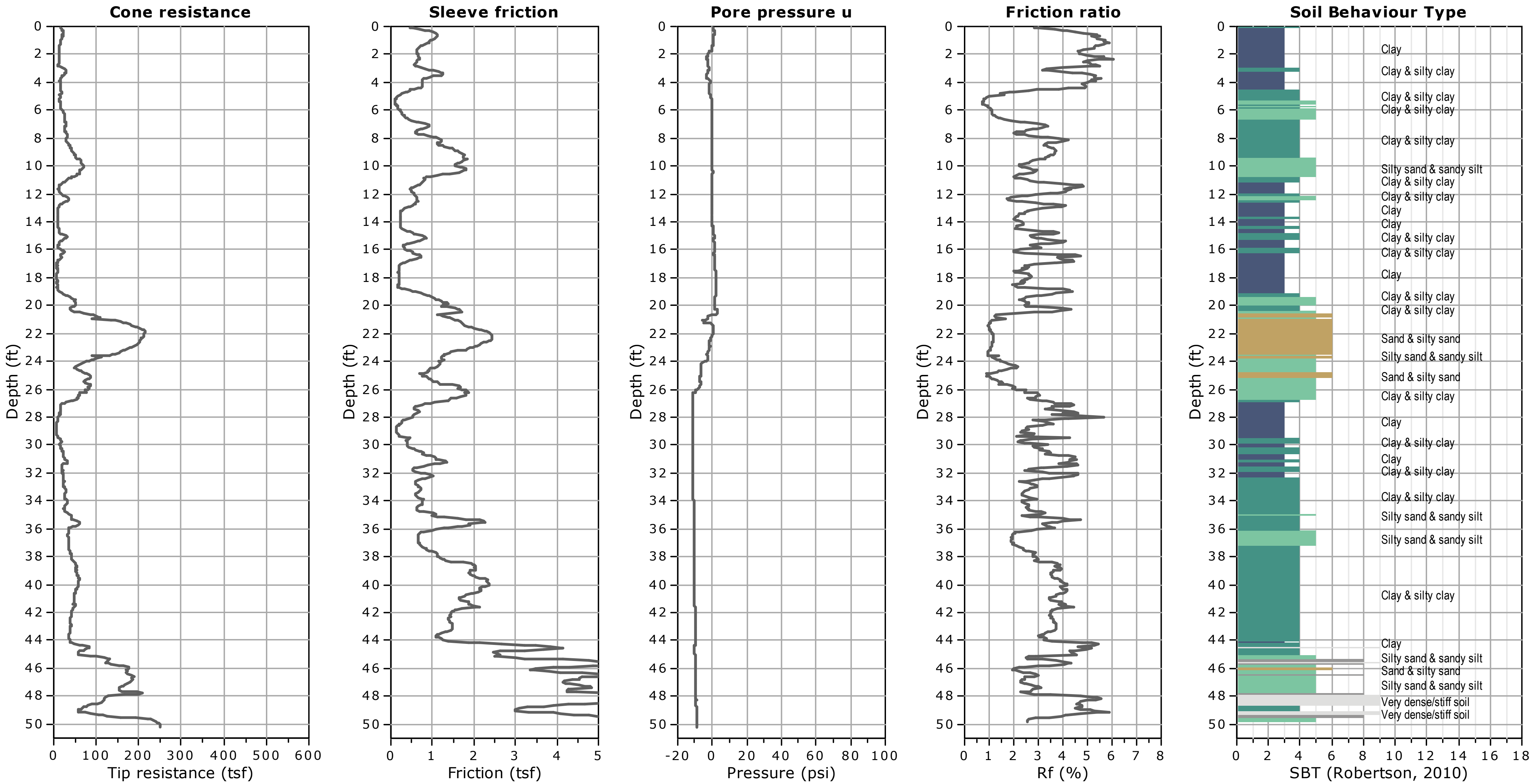
CPT-1
Total depth: 50.23 ft, Date: 3/31/2020



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

Project: **Tetra Tech BAS / Adventure Park Stormwater**
Location: **Whittier, CA**

CPT-2
Total depth: 50.22 ft, Date: 3/31/2020



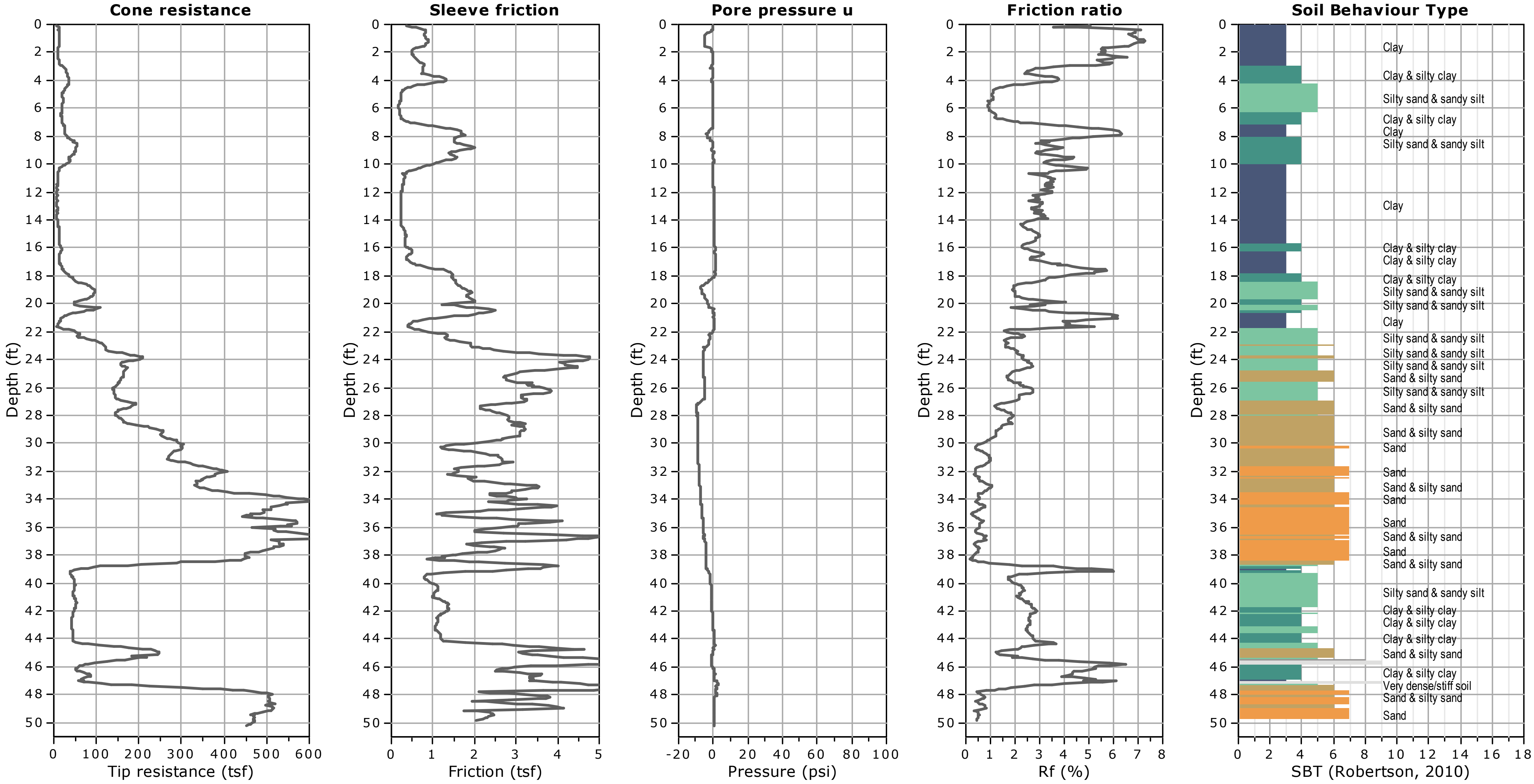
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Tetra Tech BAS / Adventure Park Stormwater

Location: Whittier, CA

CPT-3

Total depth: 50.22 ft, Date: 3/31/2020



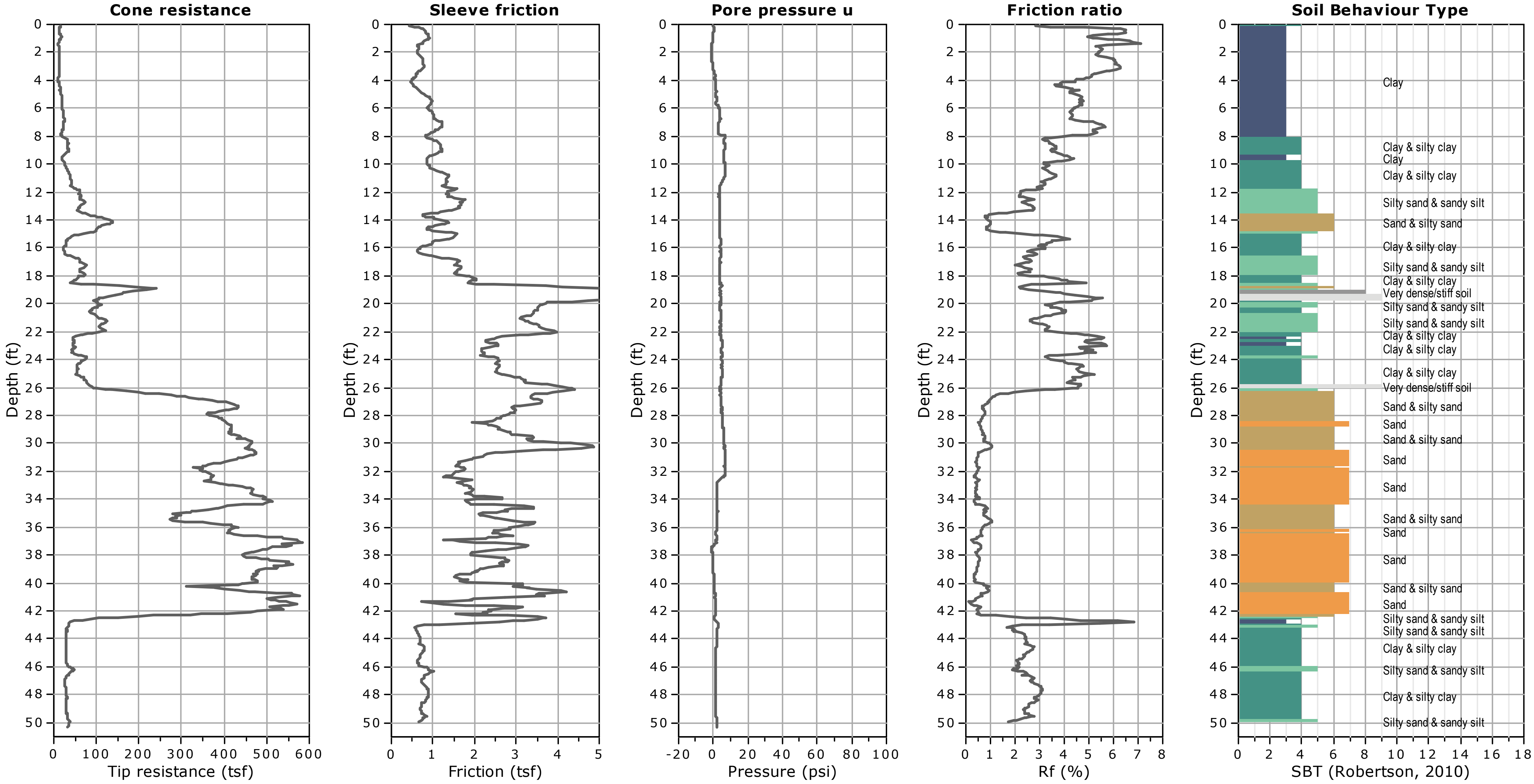
Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: Tetra Tech BAS / Adventure Park Stormwater

Location: Whittier, CA

CPT-4

Total depth: 50.34 ft, Date: 3/31/2020



Appendix E

Results of Analytical Laboratory Testing

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JONES ENVIRONMENTAL
LABORATORY RESULTS

Client:	Tetra Tech	**Report date:**	04/07/20
Client Address:	21700 Copley Dr, Suite 200	**JEL Ref. No.:**	ST-15296
	Diamond Bar, Ca	**Client Ref. No.:**	TET20-179e
Attn:	Shawn Morrish	**Date Sampled:**	03/31/2020
		Date Received:	03/31/2020
		Date Analyzed:	04/02/2020-
Project:	Adventure Park Stormwater		04/06/2020
Project Address:	10130 Gunn Ave.	**Physical State:**	Soil
	Whittier, Ca		

ANALYSES REQUESTED

Soil:

1. EPA 8015M – Extended Range Hydrocarbons
2. EPA 8260B by 5035 – Volatile Organics by GC/MS + Oxygenates/Gasoline Range Organics
3. EPA 6010B by 3050B and EPA 7471A – CAM 17 Metals

Approval: A. O'Toole

Annalise O'Toole
Mobile Lab Manager

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JONES ENVIRONMENTAL
LABORATORY RESULTS

Client:	Tetra Tech	**Report date:**	4/2/2020
Client Address:	21700 Copley Dr, Suite 200 Diamond Bar, CA	**Jones Ref. No.:**	ST-15296
		Client Ref. No.:	TET20-179E
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/2/2020
Project Address:	10130 Gunn Ave. Whittier, CA	**Physical State:**	Soil

EPA 8015M - Extended Range Hydrocarbons

Sample ID:	**CPt #3 15-17'**		
Jones ID:	**ST-15296-01**	**Reporting Limit**	**Units**
Carbon Chain Range			
C10 - C11	ND	1.0	mg/kg
C12 - C13	ND	1.0	mg/kg
C14 - C15	ND	1.0	mg/kg
C16 - C17	ND	1.0	mg/kg
C18 - C19	ND	1.0	mg/kg
C20 - C23	ND	1.0	mg/kg
C24 - C27	ND	1.0	mg/kg
C28 - C31	ND	1.0	mg/kg
C32 - C35	ND	1.0	mg/kg
C36 - C39	ND	1.0	mg/kg
C40 - C43	ND	1.0	mg/kg
C13 - C22	ND	10.0	mg/kg
C23 - C40	ND	10.0	mg/kg
C10 - C28	ND	10.0	mg/kg
C29 - C40	ND	10.0	mg/kg
Dilution Factor	1		
Surrogate Recovery:		**QC Limits**	
Hexacosane	84%	30 - 120	
Batch:	8015 _040120_01		

ND = Value less than reporting limit

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JONES ENVIRONMENTAL

LABORATORY RESULTS

Client:	Tetra Tech	**Report date:**	4/2/2020
Client Address:	21700 Copley Dr, Suite 200 Diamond Bar, CA	**Jones Ref. No.:**	ST-15296
		Client Ref. No.:	TET20-179E
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/2/2020
Project Address:	10130 Gunn Ave. Whittier, CA	**Physical State:**	Soil

EPA 8015M - Extended Range Hydrocarbons

Sample ID:	**METHOD BLANK**		
Jones ID:	**MB-040120_01**	**Reporting Limit**	**Units**
Carbon Chain Range			
C10 - C11	ND	1.0	mg/kg
C12 - C13	ND	1.0	mg/kg
C14 - C15	ND	1.0	mg/kg
C16 - C17	ND	1.0	mg/kg
C18 - C19	ND	1.0	mg/kg
C20 - C23	ND	1.0	mg/kg
C24 - C27	ND	1.0	mg/kg
C28 - C31	ND	1.0	mg/kg
C32 - C35	ND	1.0	mg/kg
C36 - C39	ND	1.0	mg/kg
C40 - C43	ND	1.0	mg/kg
C13 - C22	ND	10.0	mg/kg
C23 - C40	ND	10.0	mg/kg
C10 - C28	ND	10.0	mg/kg
C29 - C40	ND	10.0	mg/kg
Dilution Factor	1		
Surrogate Recovery:		**QC Limits**	
Hexacosane	73%	30 - 120	
Batch:	8015 _040120_01		

ND = Value less than reporting limit

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JONES ENVIRONMENTAL QUALITY CONTROL INFORMATION

Client:	Tetra Tech	**Report date:**	4/2/2020
Client Address:	21700 Copley Dr, Suite 200	**Jones Ref. No.:**	ST-15296
	Diamond Bar, CA	**Client Ref. No.:**	TET20-179E
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/2/2020
Project Address:	10130 Gunn Ave.	**Physical State:**	Soil
	Whittier, CA		

BATCH: 8015 _040120_01 **Prepared:** 4/1/2020 **Analyzed:** 4/2/2020

EPA 8015M - Extended Range Hydrocarbons

	Result	Spike Level	% Recovery	% RPD	% Recovery Limits	Units
LCS:	LCS-040120_01	**SAMPLE SPIKED:**		CLEAN SOIL		
Analyte:						
Diesel	**391**	500	78%		60 - 140	mg/kg
Surrogate Recovery:						
Hexacosane			72%		30 - 120	
LCSD:	LCSD-040120_01	**SAMPLE SPIKED:**		CLEAN SOIL		
Analyte:						
Diesel	**374**	500	75%	4.4%	60 - 140	mg/kg
Surrogate Recoveries:						
Hexacosane			70%		30 - 120	
CCV:	CCV-040220_02					
Analyte:						
Diesel	**1010**	1000	101%		80 - 120	mg/kg

LCS = Laboratory Control Sample
LCSD= Laboratory Control Sample Duplicate
CCV = Continuing Calibration Verification
RPD = Relative Percent Difference

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JONES ENVIRONMENTAL LABORATORY RESULTS

Client:	Tetra Tech	**Report date:**	4/7/2020
Client Address:	21700 Copley Dr, Suite 200 Diamond Bar, CA	**Jones Ref. No.:**	ST-15296
		Client Ref. No.:	TET20-179E
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/3/2020
Project Address:	10130 Gunn Ave. Whittier, CA	**Physical State:**	Soil

EPA 8260B by 5035 – Volatile Organics by GC/MS + Oxygenates/Gasoline Range Organics

Sample ID:	**CPt #3 15-17'**		
Jones ID:	**ST-15296-01**	**Reporting Limit**	**Units**
Analytes:			
Benzene	ND	1.0	μg/kg
Bromobenzene	ND	1.0	μg/kg
Bromodichloromethane	ND	1.0	μg/kg
Bromoform	ND	1.0	μg/kg
n-Butylbenzene	ND	1.0	μg/kg
sec-Butylbenzene	ND	1.0	μg/kg
tert-Butylbenzene	ND	1.0	μg/kg
Carbon tetrachloride	ND	1.0	μg/kg
Chlorobenzene	ND	1.0	μg/kg
Chloroform	ND	1.0	μg/kg
2-Chlorotoluene	ND	1.0	μg/kg
4-Chlorotoluene	ND	1.0	μg/kg
Dibromochloromethane	ND	1.0	μg/kg
1,2-Dibromo-3-chloropropane	ND	1.0	μg/kg
1,2-Dibromoethane (EDB)	ND	1.0	μg/kg
Dibromomethane	ND	1.0	μg/kg
1,2- Dichlorobenzene	ND	1.0	μg/kg
1,3-Dichlorobenzene	ND	1.0	μg/kg
1,4-Dichlorobenzene	ND	1.0	μg/kg
1,1-Dichloroethane	ND	1.0	μg/kg
1,2-Dichloroethane	ND	1.0	μg/kg
1,1-Dichloroethene	ND	1.0	μg/kg
cis-1,2-Dichloroethene	ND	1.0	μg/kg
trans-1,2-Dichloroethene	ND	1.0	μg/kg
1,2-Dichloropropane	ND	1.0	μg/kg
1,3-Dichloropropane	ND	1.0	μg/kg
2,2-Dichloropropane	ND	1.0	μg/kg
1,1-Dichloropropene	ND	1.0	μg/kg
cis-1,3-Dichloropropene	ND	1.0	μg/kg

JONES ENVIRONMENTAL LABORATORY RESULTS

EPA 8260B by 5035 – Volatile Organics by GC/MS + Oxygenates/Gasoline Range Organics

Sample ID:	**CPt #3 15-17'**	**Reporting Limit**	**Units**
Jones ID:	**ST-15296-01**		
Analytes:			
trans-1,3-Dichloropropene	ND	1.0	µg/kg
Ethylbenzene	ND	1.0	µg/kg
Freon 11	ND	5.0	µg/kg
Freon 12	ND	5.0	µg/kg
Freon 113	ND	5.0	µg/kg
Hexachlorobutadiene	ND	1.0	µg/kg
Isopropylbenzene	ND	1.0	µg/kg
4-Isopropyltoluene	ND	1.0	µg/kg
Methylene chloride	ND	1.0	µg/kg
Naphthalene	ND	1.0	µg/kg
n-Propylbenzene	ND	1.0	µg/kg
Styrene	ND	1.0	µg/kg
1,1,1,2-Tetrachloroethane	ND	1.0	µg/kg
1,1,2,2-Tetrachloroethane	ND	1.0	µg/kg
Tetrachloroethene	ND	1.0	µg/kg
Toluene	ND	1.0	µg/kg
1,2,3-Trichlorobenzene	ND	1.0	µg/kg
1,2,4-Trichlorobenzene	ND	1.0	µg/kg
1,1,1-Trichloroethane	ND	1.0	µg/kg
1,1,2-Trichloroethane	ND	1.0	µg/kg
Trichloroethene	ND	1.0	µg/kg
1,2,3-Trichloropropane	ND	1.0	µg/kg
1,2,4-Trimethylbenzene	ND	1.0	µg/kg
1,3,5-Trimethylbenzene	ND	1.0	µg/kg
Vinyl chloride	ND	1.0	µg/kg
m,p-Xylene	ND	2.0	µg/kg
o-Xylene	ND	1.0	µg/kg
Methyl-tert-butylether	ND	5.0	µg/kg
Ethyl-tert-butylether	ND	5.0	µg/kg
Di-isopropylether	ND	5.0	µg/kg
tert-amylmethylether	ND	5.0	µg/kg
tert-Butylalcohol	ND	50.0	µg/kg
Gasoline Range Organics (C4-C12)	ND	0.20	mg/kg
TIC:			
Ethanol	ND	50.0	µg/kg
Dilution Factor	1		

Surrogate Recoveries:		**QC Limits**
Dibromofluoromethane	101%	60 - 140
Toluene-d_8	99%	60 - 140
4-Bromofluorobenzene	98%	60 - 140
	VOC3-040320-01	

ND= Value less than reporting limit

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JONES ENVIRONMENTAL QUALITY CONTROL INFORMATION

Client:	Tetra Tech	**Report date:**	4/7/2020
Client Address:	21700 Copley Dr, Suite 200	**Jones Ref. No.:**	ST-15296
	Diamond Bar, CA	**Client Ref. No.:**	TET20-179E
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/3/2020
Project Address:	10130 Gunn Ave.	**Physical State:**	Soil
	Whittier, CA		

EPA 8260B by 5035 – Volatile Organics by GC/MS + Oxygenates/Gasoline Range Organics

Sample ID:	**METHOD BLANK**		
Jones ID:	**040320-V3MB1**	**Reporting Limit**	**Units**
Analytes:			
Benzene	ND	1.0	µg/kg
Bromobenzene	ND	1.0	µg/kg
Bromodichloromethane	ND	1.0	µg/kg
Bromoform	ND	1.0	µg/kg
n-Butylbenzene	ND	1.0	µg/kg
sec-Butylbenzene	ND	1.0	µg/kg
tert-Butylbenzene	ND	1.0	µg/kg
Carbon tetrachloride	ND	1.0	µg/kg
Chlorobenzene	ND	1.0	µg/kg
Chloroform	ND	1.0	µg/kg
2-Chlorotoluene	ND	1.0	µg/kg
4-Chlorotoluene	ND	1.0	µg/kg
Dibromochloromethane	ND	1.0	µg/kg
1,2-Dibromo-3-chloropropane	ND	1.0	µg/kg
1,2-Dibromoethane (EDB)	ND	1.0	µg/kg
Dibromomethane	ND	1.0	µg/kg
1,2- Dichlorobenzene	ND	1.0	µg/kg
1,3-Dichlorobenzene	ND	1.0	µg/kg
1,4-Dichlorobenzene	ND	1.0	µg/kg
1,1-Dichloroethane	ND	1.0	µg/kg
1,2-Dichloroethane	ND	1.0	µg/kg
1,1-Dichloroethene	ND	1.0	µg/kg
cis-1,2-Dichloroethene	ND	1.0	µg/kg
trans-1,2-Dichloroethene	ND	1.0	µg/kg
1,2-Dichloropropane	ND	1.0	µg/kg
1,3-Dichloropropane	ND	1.0	µg/kg
2,2-Dichloropropane	ND	1.0	µg/kg
1,1-Dichloropropene	ND	1.0	µg/kg
cis-1,3-Dichloropropene	ND	1.0	µg/kg

JONES ENVIRONMENTAL QUALITY CONTROL INFORMATION

EPA 8260B by 5035 – Volatile Organics by GC/MS + Oxygenates/Gasoline Range Organics

Sample ID:	**METHOD BLANK**		
Jones ID:	**040320-V3MB1**	**Reporting Limit**	**Units**
Analytes:			
trans-1,3-Dichloropropene	ND	1.0	μg/kg
Ethylbenzene	ND	1.0	μg/kg
Freon 11	ND	5.0	μg/kg
Freon 12	ND	5.0	μg/kg
Freon 113	ND	5.0	μg/kg
Hexachlorobutadiene	ND	1.0	μg/kg
Isopropylbenzene	ND	1.0	μg/kg
4-Isopropyltoluene	ND	1.0	μg/kg
Methylene chloride	ND	1.0	μg/kg
Naphthalene	ND	1.0	μg/kg
n-Propylbenzene	ND	1.0	μg/kg
Styrene	ND	1.0	μg/kg
1,1,1,2-Tetrachloroethane	ND	1.0	μg/kg
1,1,2,2-Tetrachloroethane	ND	1.0	μg/kg
Tetrachloroethene	ND	1.0	μg/kg
Toluene	ND	1.0	μg/kg
1,2,3-Trichlorobenzene	ND	1.0	μg/kg
1,2,4-Trichlorobenzene	ND	1.0	μg/kg
1,1,1-Trichloroethane	ND	1.0	μg/kg
1,1,2-Trichloroethane	ND	1.0	μg/kg
Trichloroethene	ND	1.0	μg/kg
1,2,3-Trichloropropane	ND	1.0	μg/kg
1,2,4-Trimethylbenzene	ND	1.0	μg/kg
1,3,5-Trimethylbenzene	ND	1.0	μg/kg
Vinyl chloride	ND	1.0	μg/kg
m,p-Xylene	ND	2.0	μg/kg
o-Xylene	ND	1.0	μg/kg
Methyl-tert-butylether	ND	5.0	μg/kg
Ethyl-tert-butylether	ND	5.0	μg/kg
Di-isopropylether	ND	5.0	μg/kg
tert-amylmethylether	ND	5.0	μg/kg
tert-Butylalcohol	ND	50.0	μg/kg
Gasoline Range Organics (C4-C12)	ND	0.20	mg/kg
TIC:			
Ethanol	ND	50.0	μg/kg
Dilution Factor	1		

Surrogate Recoveries:		**QC Limits**
Dibromofluoromethane	101%	60 - 140
Toluene-d_8	101%	60 - 140
4-Bromofluorobenzene	96%	60 - 140
	VOC3-040320-01	

ND= Value less than reporting limit

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JONES ENVIRONMENTAL QUALITY CONTROL INFORMATION

Client:	Tetra Tech	**Report date:**	4/7/2020
Client Address:	21700 Copley Dr, Suite 200	**Jones Ref. No.:**	ST-15296
	Diamond Bar, CA	**Client Ref. No.:**	TET20-179E
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/3/2020
Project Address:	10130 Gunn Ave.	**Physical State:**	Soil
	Whittier, CA		

EPA 8260B by 5035 – Volatile Organics by GC/MS + Oxygenates/Gasoline Range Organics

Sample Spiked: CLEAN SOIL **GC#:** VOC3-040320-01

Jones ID: **040320-V3MS1** **040320-V3MSD1** **040320-V3CCV1**

Parameter	MS Recovery (%)	MSD Recovery (%)	RPD	Acceptability Range (%)	CCV	Acceptability Range (%)
Vinyl chloride	125%	113%	10.8%	60 - 140	104%	80 - 120
1,1-Dichloroethene	39%[1]	34%[1]	13.3%	60 - 140	81%	80 - 120
Cis-1,2-Dichloroethene	107%	105%	1.9%	70 - 130	112%	80 - 120
1,1,1-Trichloroethane	99%	95%	4.3%	70 - 130	113%	80 - 120
Benzene	106%	104%	1.7%	70 - 130	114%	80 - 120
Trichloroethene	100%	96%	4.1%	70 - 130	112%	80 - 120
Toluene	91%	88%	3.8%	70 - 130	101%	80 - 120
Tetrachloroethene	90%	87%	3.4%	70 - 130	102%	80 - 120
Chlorobenzene	86%	83%	2.6%	70 - 130	94%	80 - 120
Ethylbenzene	101%	96%	5.6%	70 - 130	108%	80 - 120
1,2,4 Trimethylbenzene	98%	92%	5.5%	70 - 130	109%	80 - 120
Gasoline Range Organics (C4-C12)	99%	95%	4.1%	70 - 130		
Surrogate Recovery:						
Dibromofluoromethane	104%	106%		60 - 140	109%	60 - 140
Toluene-d_8	100%	98%		60 - 140	111%	60 - 140
4-Bromofluorobenzene	96%	95%		60 - 140	108%	60 - 140

[1]= Recovery outside acceptable limits. CCV recovery and LCS/LCSD PRD were within acceptable QC limits, therefore data was accepted
MS = Matrix Spike
MSD = Matrix Spike Duplicate
CCV = Continuing Calibration Verification
RPD = Relative Percent Difference; Acceptability range for RPD is ≤ 20%

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JONES ENVIRONMENTAL LABORATORY RESULTS

Client:	Tetra Tech	**Report date:**	4/7/2020
Client Address:	21700 Copley Dr, Suite 200	**Jones Ref. No.:**	ST-15296
	Daimond Bar, CA	**Client Ref. No.:**	TET20-179e
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/3&6/2020
Project Address:	10130 Gunn Avenue	**Physical State:**	Soil
	Whittier, CA		

Sample ID: **CPt #3 15-17'** **Jones ID:** **ST-15296-01**

EPA 6010B by 3050 - Title 22 CAM 17 Trace Metals by ICP-OES

Analytes:	Result	Dilution	Batch	Prepared	Analyzed	Reporting Limit	Units
Silver, Ag	ND	1	I20040301	4/3/2020	4/6/2020	0.5	mg/kg
Arsenic, As	ND	1	"	"	"	5.0	mg/kg
Barium, Ba	**35.9**	1	"	"	"	0.5	mg/kg
Beryllium, Be	ND	1	"	"	"	0.5	mg/kg
Cadmium, Cd	**1.2**	1	"	"	"	0.5	mg/kg
Cobalt, Co	**6.4**	1	"	"	"	0.5	mg/kg
Chromium, Cr	**12.1**	1	"	"	"	0.5	mg/kg
Copper, Cu	**8.2**	1	"	"	"	0.5	mg/kg
Molybdenum, Mo	**0.7**	1	"	"	"	0.5	mg/kg
Nickel, Ni	**11.5**	1	"	"	"	0.5	mg/kg
Lead, Pb	**2.8**	1	"	"	"	1.0	mg/kg
Antimony, Sb	ND	1	"	"	"	5.0	mg/kg
Selenium, Se	ND	1	"	"	"	5.0	mg/kg
Thallium, Tl	ND	1	"	"	"	5.0	mg/kg
Vanadium, V	**22.8**	1	"	"	"	0.5	mg/kg
Zinc, Zn	**29.0**	1	"	"	"	1.5	mg/kg

EPA 7471A - Mercury by Cold Vapor Atomic Absorption

	Result	Dilution	Batch	Prepared	Analyzed	Reporting Limit	Units
Mercury, Hg	**0.026**	1	H20040301	4/3/2020	4/3/2020	0.020	mg/kg

ND= Not Detected

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JONES ENVIRONMENTAL QUALITY CONTROL INFORMATION

Client:	Tetra Tech	**Report date:**	4/7/2020
Client Address:	21700 Copley Dr, Suite 200	**Jones Ref. No.:**	ST-15296
	Daimond Bar, CA	**Client Ref. No.:**	TET20-179e
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/3&6/2020
Project Address:	10130 Gunn Avenue	**Physical State:**	Soil
	Whittier, CA		

BATCH: I20040301 **Prepared:** 4/3/2020 **Analyzed:** 4/6/2020

EPA 6010B by 3050 - Title 22 CAM 17 Trace Metals by ICP-OES

Analytes:	Result	Spike Level	% REC	% REC Limits	% RPD	Reporting Limit	Units
METHOD BLANK:	**I200403-MB1**						
Silver, Ag	ND					0.5	mg/kg
Arsenic, As	ND					5.0	mg/kg
Barium, Ba	ND					0.5	mg/kg
Beryllium, Be	ND					0.5	mg/kg
Cadmium, Cd	ND					0.5	mg/kg
Cobalt, Co	ND					0.5	mg/kg
Chromium, Cr	ND					0.5	mg/kg
Copper, Cu	ND					0.5	mg/kg
Molybdenum, Mo	ND					0.5	mg/kg
Nickel, Ni	ND					0.5	mg/kg
Lead, Pb	ND					1.0	mg/kg
Antimony, Sb	ND					5.0	mg/kg
Selenium, Se	ND					5.0	mg/kg
Thallium, Tl	ND					5.0	mg/kg
Vanadium, V	ND					0.5	mg/kg
Zinc, Zn	ND					1.5	mg/kg

ND= Not Detected

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JONES ENVIRONMENTAL QUALITY CONTROL INFORMATION

Client:	Tetra Tech	**Report date:**	4/7/2020
Client Address:	21700 Copley Dr, Suite 200	**Jones Ref. No.:**	ST-15296
	Daimond Bar, CA	**Client Ref. No.:**	TET20-179e
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/3&6/2020
Project Address:	10130 Gunn Avenue	**Physical State:**	Soil
	Whittier, CA		

BATCH: **I20040301** **Prepared:** 4/3/2020 **Analyzed:** 4/6/2020

EPA 6010B by 3050 - Title 22 CAM 17 Trace Metals by ICP-OES

Analytes:	Result	Spike Level	% REC	% RPD	% REC Limits	Units
LCS:	**I200403-LCS1**					
Barium, Ba	**206**	200	103%		80 - 120	mg/kg
Cobalt, Co	**51.2**	50.0	102%		80 - 120	mg/kg
Lead, Pb	**53.4**	50.0	107%		80 - 120	mg/kg
Selenium, Se	**192**	200	96%		80 - 120	mg/kg
Zinc, Zn	**46.4**	50.0	93%		80 - 120	mg/kg
LCSD:	**I200403-LCSD1**					
Barium, Ba	**207**	200	104%	0.5%	80 - 120	mg/kg
Cobalt, Co	**49.5**	50.0	99%	3.4%	80 - 120	mg/kg
Lead, Pb	**53.2**	50.0	106%	0.4%	80 - 120	mg/kg
Selenium, Se	**190**	200	95%	1.0%	80 - 120	mg/kg
Zinc, Zn	**45.1**	50.0	90%	2.8%	80 - 120	mg/kg
CCV:	**I200403-CCV1**					
Barium, Ba	**1.05**	1.00	105%		90-110	mg/L
Cobalt, Co	**1.06**	1.00	106%		90-110	mg/L
Lead, Pb	**1.04**	1.00	104%		90-110	mg/L
Selenium, Se	**1.05**	1.00	105%		90-110	mg/L
Zinc, Zn	**1.01**	1.00	101%		90-110	mg/L

ND= Not Detected

RPD = Relative Percent Difference; Acceptability range for RPD is ≤ 15%

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JONES ENVIRONMENTAL QUALITY CONTROL INFORMATION

Client:	Tetra Tech	**Report date:**	4/7/2020
Client Address:	21700 Copley Dr, Suite 200	**Jones Ref. No.:**	ST-15296
	Daimond Bar, CA	**Client Ref. No.:**	TET20-179e
Attn:	Shawn Morrish	**Date Sampled:**	3/31/2020
		Date Received:	3/31/2020
Project:	Adventure Park Stormwater	**Date Analyzed:**	4/3&6/2020
Project Address:	10130 Gunn Avenue	**Physical State:**	Soil
	Whittier, CA		

BATCH: **H20040301** **Prepared:** 4/3/2020 **Analyzed:** 4/3/2020

EPA 7471A - Mercury by Cold Vapor Atomic Absorption

Analytes:	Result	Spike Level	% REC	% RPD	% REC Limits	Reporting Limit	Units
METHOD BLANK:	**H200403-MB1**						
Mercury, Hg	ND					0.020	mg/kg
LCS:	**H200403-LCS1**						
Mercury, Hg	**1.04**	1.00	104%		80 - 120		mg/kg
LCSD:	**H200403-LCSD1**						
Mercury, Hg	**1.04**	1.00	104%	**0.1%**	80 - 120		mg/kg
CCV:	**H200403-CCV1**						
Mercury, Hg	**5.03**	5.00	101%		90-110		µg/L

ND= Not Detected

RPD = Relative Percent Difference; Acceptability range for RPD is ≤ 15%

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Chain-of-Custody Record

LAB USE ONLY
Jones Project # ST-15296
Page 1 of 1

Sample Condition as Received:
Chilled ☐ yes ☐ no
Sealed ☐ yes ☐ no

Client Tetratech
Date 3/31/20
Project Name Adventure Park Stormwater
Client Project # ~~[illegible]~~ TET20-17T6
Project Address 10130 Gunn Avenue
Whittier, CA
Email Shawn.Morrish@tetratech.com
Phone 909-860-7777 / Cell 626-807-6051
Report To Shawn Morrish **Sampler** SCM

Turn Around Requested:
☐ Immediate Attention
☐ Rush 24 Hours
☐ Rush 48 Hours
☐ Rush 72 Hours
☑ Normal

Report Options
EDD______
EDF* - 10% Surcharge______
*Global ID______

Sample Container / Preservative Abbreviations
V - VOAS
AS - Acetate Sleeve
SS - Stainless Steel Sleeve
BS - Brass Sleeve
G - Glass Jar
AB - Amber Bottle
P - Plastic
SOBI - Sodium Bisulfate
MeOH - Methanol
HCl - Hydrochloric Acid
HNO3 - Nitric Acid
O - Other (See Notes)

Analysis Requested

Sample ID	Date	Sample Collection Time	Jones ID Lab Use Only	Preservative	Sample Container	Sample Matrix: Soil (S), Sludge (SL), Aqueous (A), Free Product (FP)	VOC's	TPH	Priority Metals								Hold	Number of Containers	Notes & Special Instructions
CPT#3 15-17'	3/31	11:15a	ST-15296-01		6	S	X	X	X										

Relinquished By (Signature)	Printed Name	Date	Time	Received By (Signature)	Printed Name	Date	Time
[signature]	Shawn Morrish	3/31					
Company: TetraTech				Company:			
Relinquished By (Signature)	Printed Name	Date	Time	Received By Laboratory (Signature)	Printed Name	Date	Time
				[signature]	Doug Fowler	03/31/20	1148
Company:				Company: JEL			

Total Number of Containers

Client signature on this Chain of Custody form constitutes acknowledgment that the above analyses have been requested, and the information provided herein is correct and accurate

Appendix F

Results of Agronomic Laboratory Testing

WALLACE LABS
365 Coral Circle
El Segundo, CA 90245
(310) 615-0116

SOILS REPORT

Print Date	Apr. 6, 2020	Receive Date	4/3/20
Location	TET 20-179E		
Requester	Shawn Morrish, Tetra Tech		

graphic interpretation: * very low, ** low, *** moderate
* * * * high, * * * * * very high

ammonium bicarbonate/DTPA
extractable - mg/kg soil

Interpretation of data (low / medium / high)	Sample ID Number	20-97-19				20-97-20			
	Sample Description	TP-1, 1-1.5'				TP-2, 1-1.5'			
	elements		graphic				graphic		
0 - 7 / 8-15 / over 15	**phosphorus**	10.32	***			11.23	***		
0-60 / 60 -120 / 121-180	**potassium**	246.72	*****			686.07	*****		
0 - 4 / 4 - 10 / over 10	**iron**	8.21	***			11.77	****		
0- 0.5 / 0.6- 1 / over 1	**manganese**	0.22	*			0.22	*		
0 - 1 / 1 - 1.5 / over 1.5	**zinc**	3.08	****			1.76	****		
0- 0.2 / 0.3- 0.5 / over 0.5	**copper**	6.02	*****			6.13	*****		
0- 0.2 / 0.2- 0.5 / over 1	**boron**	0.20	***			0.34	***		
	calcium	375.63	***			416.95	****		
	magnesium	227.42	*****			280.64	*****		
	sodium	54.77	**			174.94	***		
	sulfur	4.34	*			12.54	*		
	molybdenum	0.03	***			0.15	****		
	nickel	0.66	*			0.98	*		
The following trace	**aluminum**	n d	*			n d	*		
elements may be toxic	**arsenic**	0.26	*			0.25	*		
The degree of toxicity	**barium**	0.40	*			0.36	*		
depends upon the pH of	**cadmium**	0.25	*			0.36	*		
the soil, soil texture,	**chromium**	n d	*			n d	*		
organic matter, and the	**cobalt**	n d	*			n d	*		
concentrations of the	**lead**	1.59	**			1.48	**		
individual elements as well	**lithium**	0.02	*			0.04	*		
as to their interactions	**mercury**	n d	*			n d	*		
	selenium	0.07	*			0.14	*		
The pH optimum depends	**silver**	n d	*			n d	*		
upon soil organic	**strontium**	1.78	*			2.47	*		
matter and clay content-	**tin**	0.03	*			0.14	*		
for clay and loam soils:	**vanadium**	0.41	*			0.52	*		
under 5.2 is too acidic									
6.5 to 7 is ideal	**Saturation Extract**								
over 8.0 is too alkaline	**pH value**	7.19	***			7.23	***		
The ECe is a measure of	**ECe (milli-**	0.21	*			0.40	**		
the soil salinity:	**mho/cm)**				millieq/l				millieq/l
1-2 affects a few plants	**calcium**	14.2			0.7	17.7			0.9
2-4 affects some plants,	**magnesium**	1.8			0.2	2.4			0.2
> 4 affects many plants.	**sodium**	14.0			0.6	49.3			2.1
	potassium	4.5			0.1	15.7			0.4
	cation sum				1.6				3.6
problems over 150 ppm	**chloride**	23			0.6	36			1.0
good 20 - 30 ppm	**nitrate as N**	8			0.5	4			0.3
	phosphorus as P	0.1			0.0	0.1			0.0
toxic over 800	**sulfate as S**	2.9			0.2	15.8			1.0
	anion sum				1.4				2.3
toxic over 1 for many plants	**boron as B**	0.07	*			0.13	*		
increasing problems start at 3	**SAR**	0.9	*			2.9	**		
est. gypsum requirement-lbs./1000 sq. ft.		9				40			
	relative infiltration rate	slow				slow			
	estimated soil texture	clay				clay			
	lime (calcium carbonate)	yes				slight			
	organic matter	fair				fair			
	moisture content of soil	17.3%				28.0%			
	half saturation percentage	28.3%				32.3%			

Elements are expressed as mg/kg dry soil or mg/l for saturation extract.
pH and ECe are measured in a saturation paste extract. nd means not detected.
Analytical data determined on soil fraction passing a 2 mm sieve.

WALLACE LABORATORIES, LLC
365 Coral Circle
El Segundo, CA 90245
phone (310) 615-0116 fax (310) 640-6863

April 7, 2020

Shawn Morrish, Shawn.Morrish@tetratech.com
Tetra Tech
21700 Copley Drive
Diamond Bar, CA 91765

RE: TET 20-179E
Two samples received April 3, 2020

Dear Shawn,

TP-1, 1-1.5' – The pH is slightly alkaline at 7.18. Limestone is present. It induces iron deficiency in acid-loving plants. Salinity is low at 0.21 millimho/cm.

Nitrogen, manganese and sulfur are low. Phosphorus and boron are moderate. Potassium, zinc, copper and magnesium are high. Sodium is modest. SAR (sodium adsorption ratio) is 0.9. The concentrations of common non-essential heavy metals are low.

TP-2, 1-1.5' – The pH is slightly alkaline at 7.23. Salinity is modest at 0.40 millimho/cm.

Nitrogen, manganese and sulfur are low. Phosphorus and boron are moderate. Potassium, iron, zinc, copper and magnesium are high. Sodium is moderate. SAR is 2.9. The concentrations of common non-essential heavy metals are low.

Recommendations

Manganese is expected to be supplied by the organic amendment.

General soil preparation on a square foot basis. Broadcast the following uniformly. The rates are per 1,000 square feet for a 6-inch lift. Incorporate them homogeneously 6 inches deep:

Simplot or Yara calcium ammonium nitrate (27-0-0) – 4 pounds for both
Triple superphosphate (0-45-0) – 3 pounds for both
Agricultural gypsum - 10 pounds for TP-1, 30 pounds for TP-2
Organic soil amendment - about 4 cubic yards or as needed, sufficient for 4% to 6% soil organic matter on a dry weight basis

For soil preparation on a volume basis, incorporate homogeneously the following materials into clean soil. Rates are expressed on a cubic yard basis:

Simplot or Yara calcium ammonium nitrate (27-0-0) – 1/4 pound for both
Triple superphosphate (0-45-0) – 1/4 pound for both
Agricultural gypsum – 0.5 pound for TP-1, 1.5 pounds for TP-2
Organic soil amendment - about 15% by volume or as needed, sufficient for 4% to 6% soil organic matter on a dry weight basis

Soil organic amendment suggestions

1. Humus material shall have an acid-soluble ash content of no less than 6% and no more than 20%. The organic matter content shall be 50% or more on a dry weight basis.
2. The pH of the material shall be between 6 and 7.5.
3. The salt content shall be less than 10 millimho/cm @ 25° C. in a saturated paste extract.
4. Boron content of the saturated extract shall be less than 1.0 parts per million.
5. Silicon content (acid-insoluble ash) shall be less than 50%.
6. Calcium carbonate shall not be present if to be applied on alkaline soils.
7. Types of acceptable products are composts, manures, mushroom composts, straw, alfalfa, peat mosses etc. low in salts, low in heavy metals, free from weed seeds, free of pathogens and other deleterious materials.
8. Composted wood products are conditionally acceptable [stable humus must be present]. Wood based products are not acceptable which are based on red wood or cedar.
9. Sludge-based materials are not acceptable.
10. Carbon:nitrogen ratio is less than 25:1.
11. The compost shall be aerobic without malodorous presence of decomposition products.
12. The maximum particle size shall be 0.5 inch, 80% or more shall pass a No. 4 screen for soil amending.

Maximum total permissible pollutant concentrations in amendment in parts per million on a dry weight basis:

arsenic	12	copper	100	selenium	30
cadmium	15	lead	200	silver	10
chromium	200	mercury	10	vanadium	50
cobalt	50	molybdenum	20	zinc	200
		nickel	100		

For site maintenance, apply Simplot or Yara calcium ammonium nitrate (27-0-0) at 4 pounds per 1,000 square feet about once per quarter.

Correct iron deficiency as needed with BASF Sprint 138 Fe or other FeEDDHA chelated iron.

Monitor the site with periodic soil testing. Adjust the maintenance program as needed.

If manganese is needed, dissolve manganese EDTA (12.0% manganese) at 1/2 tablespoonful in five gallons of water. Drench the soil when it is partially dry. Be careful not to stain masonry surfaces.

Sincerely,

Garn A. Wallace, Ph. D.
GAW:n

Appendix G

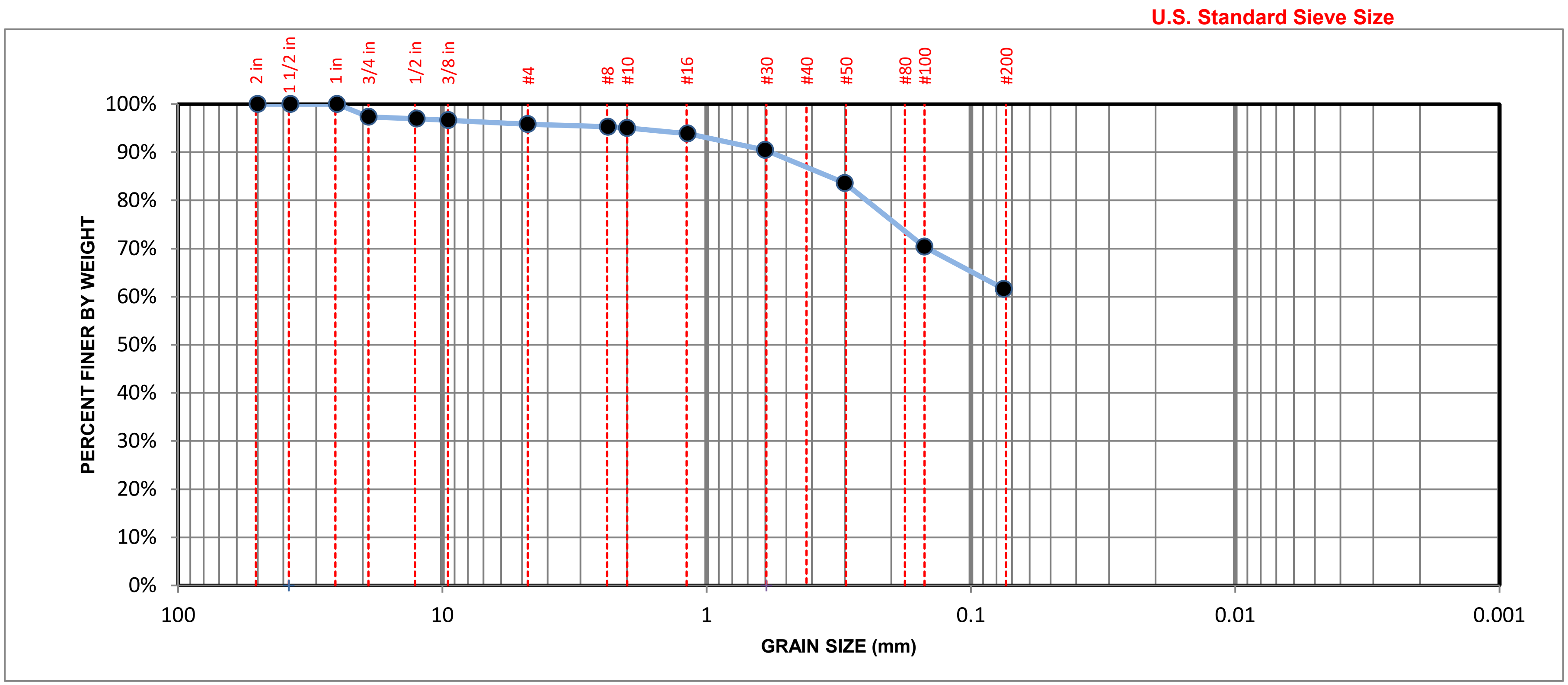
Results of Tetra Tech Laboratory Testing

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D6913

TETRA TECH BAS GEOSCIENCE

Job Name:	Adventure Park, Whittier	**Tested By :**	MG
Job Number:	TET-20-179E	**Date Completed:**	April 8, 2020
Address:			
Date Sampled:	March 31, 2020		



Symbol	Boring No.	Sample #	Deph (feet)	LL	PI	USCS	Gravel	Sand	Fines	2μ
●		DRI-1/TP-1	1-1.5	-	-	**CL**	**4%**	**34%**	**62%**	

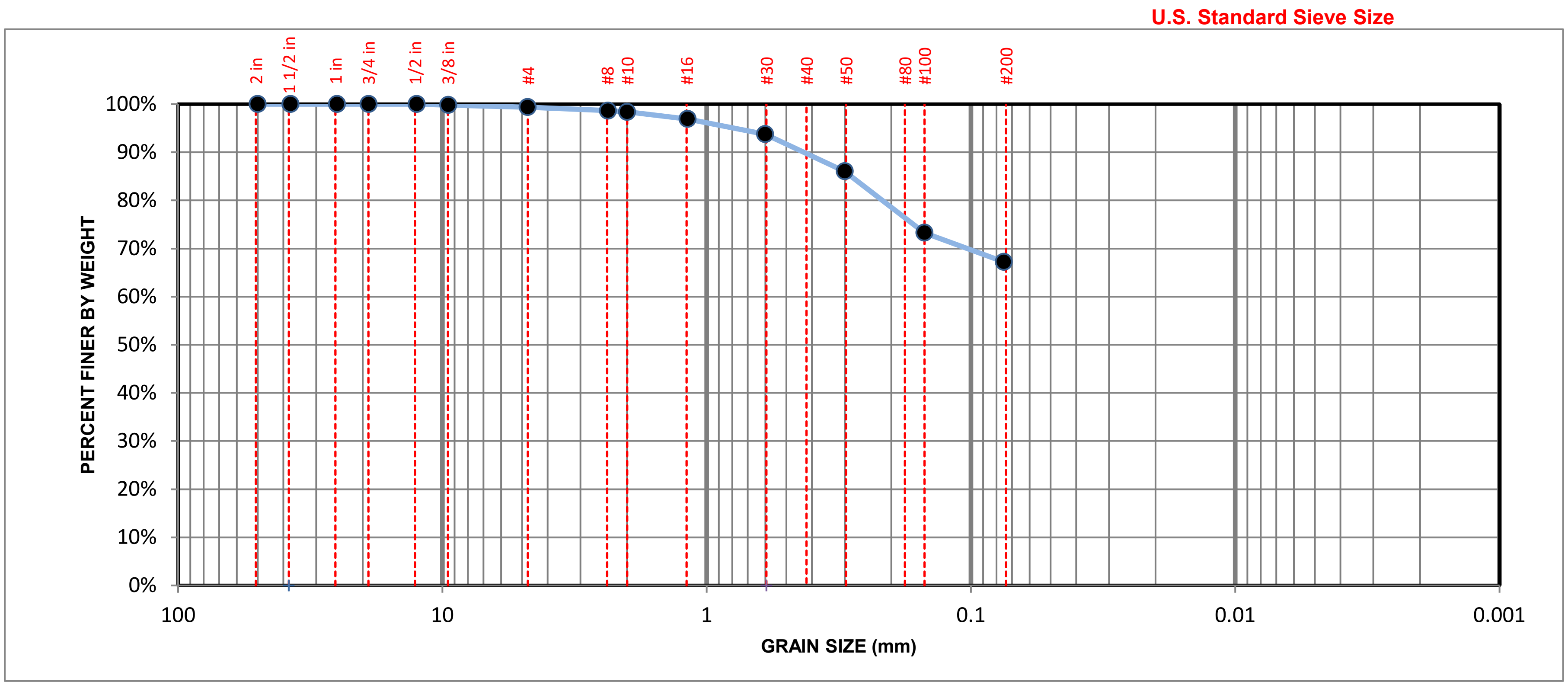
1360 Valley Vista Drive * Diamond Bar, CA 91765 * Tel.: (909) 860-7777

GRAIN SIZE DISTRIBUTION ANALYSIS

ASTM C136/C117/D6913

TETRA TECH BAS GEOSCIENCE

Job Name:	Adventure Park, Whittier	**Tested By :**	MG
Job Number:	TET-20-179E	**Date Completed:**	April 8, 2020
Address:			
Date Sampled:	March 31, 2020		



Symbol	Boring No.	Sample #	Deph (feet)	LL	PI	USCS	Gravel	Sand	Fines	2μ
●		DRI-2/TP-2	1-1.5	-	-	**CL**	**1%**	**32%**	**67%**	

Appendix H

Seismic Demand

Adventure Park

Latitude, Longitude: 33.942746, -118.034187

Date	4/14/2020, 2:48:38 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.755	MCE_R ground motion. (for 0.2 second period)
S_1	0.626	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.755	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.17	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.76	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.836	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.755	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.947	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.342	Factored deterministic acceleration value. (0.2 second)
S1RT	0.626	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.695	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.78	Factored deterministic acceleration value. (1.0 second)
PGAd	0.944	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.901	Mapped value of the risk coefficient at short periods
C_{R1}	0.901	Mapped value of the risk coefficient at a period of 1 s

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U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (v4.2.0)

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.942746

Time Horizon

Return period in years

2475

Longitude

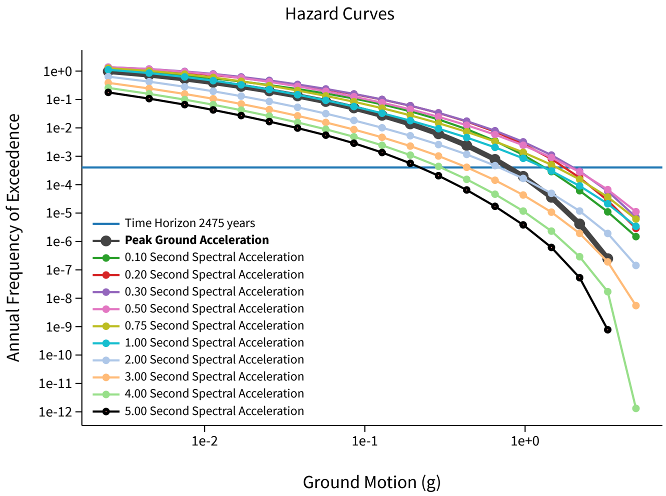
Decimal degrees, negative values for western longitudes

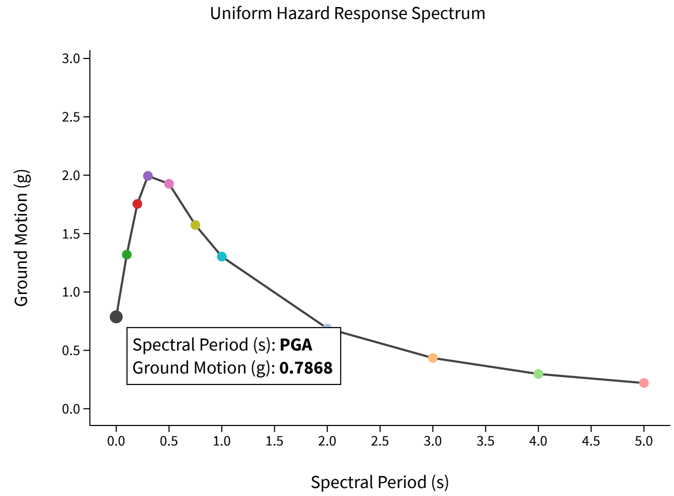
-118.034187

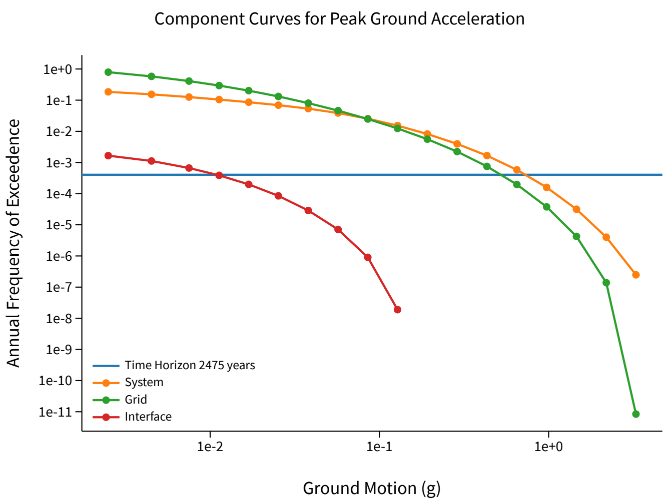
Site Class

259 m/s (Site class D)

Hazard Curve





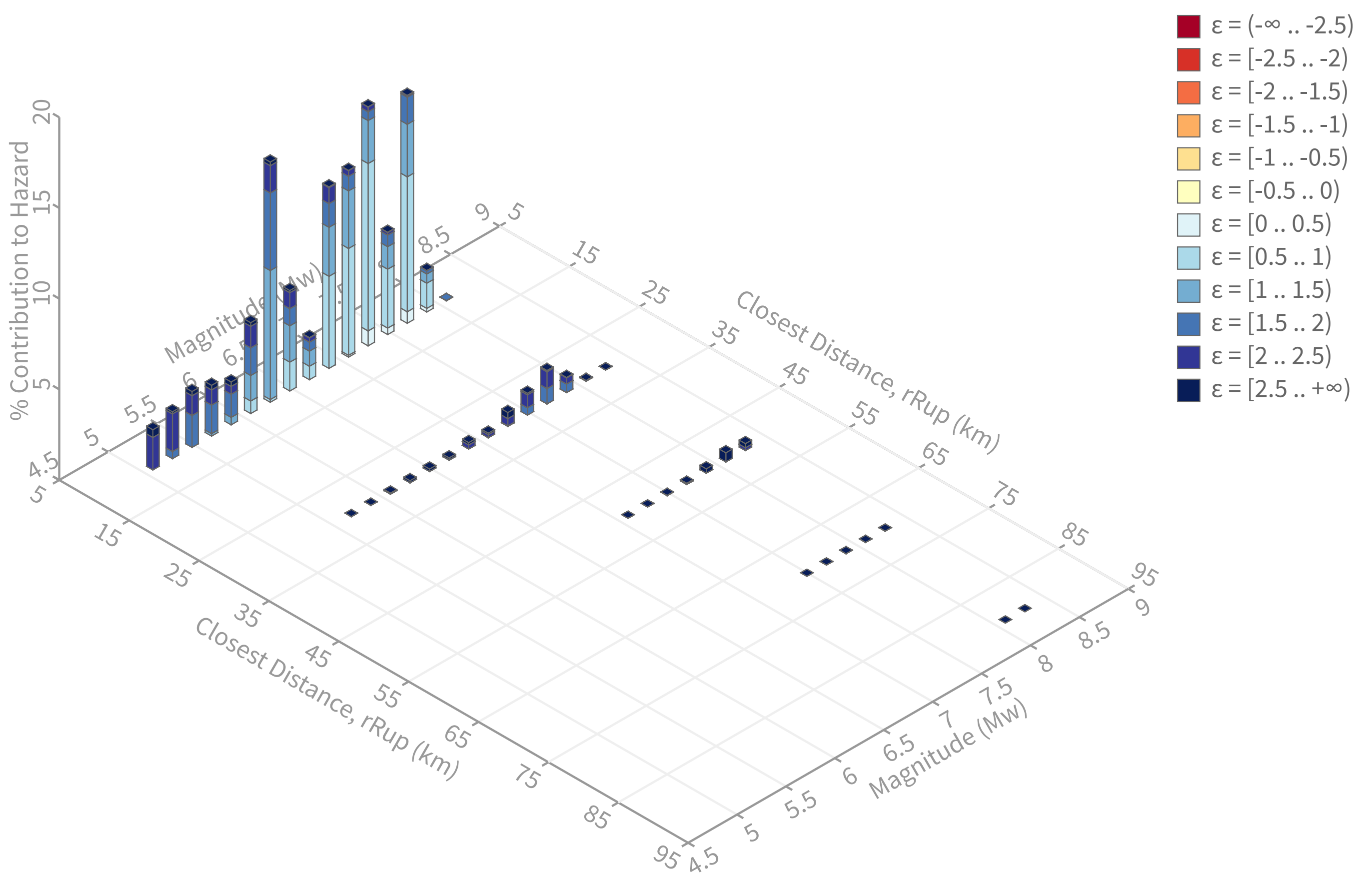


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.78684708 g

Recovered targets

Return period: 2906.7795 yrs
Exceedance rate: 0.00034402334 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.06 %

Mean (over all sources)

m: 6.85
r: 10.05 km
ε_0: 1.34 σ

Mode (largest m-r bin)

m: 6.27
r: 6.67 km
ε_0: 1.54 σ
Contribution: 13.24 %

Mode (largest m-r-ε_0 bin)

m: 7.29
r: 9.88 km
ε_0: 0.72 σ
Contribution: 9.23 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

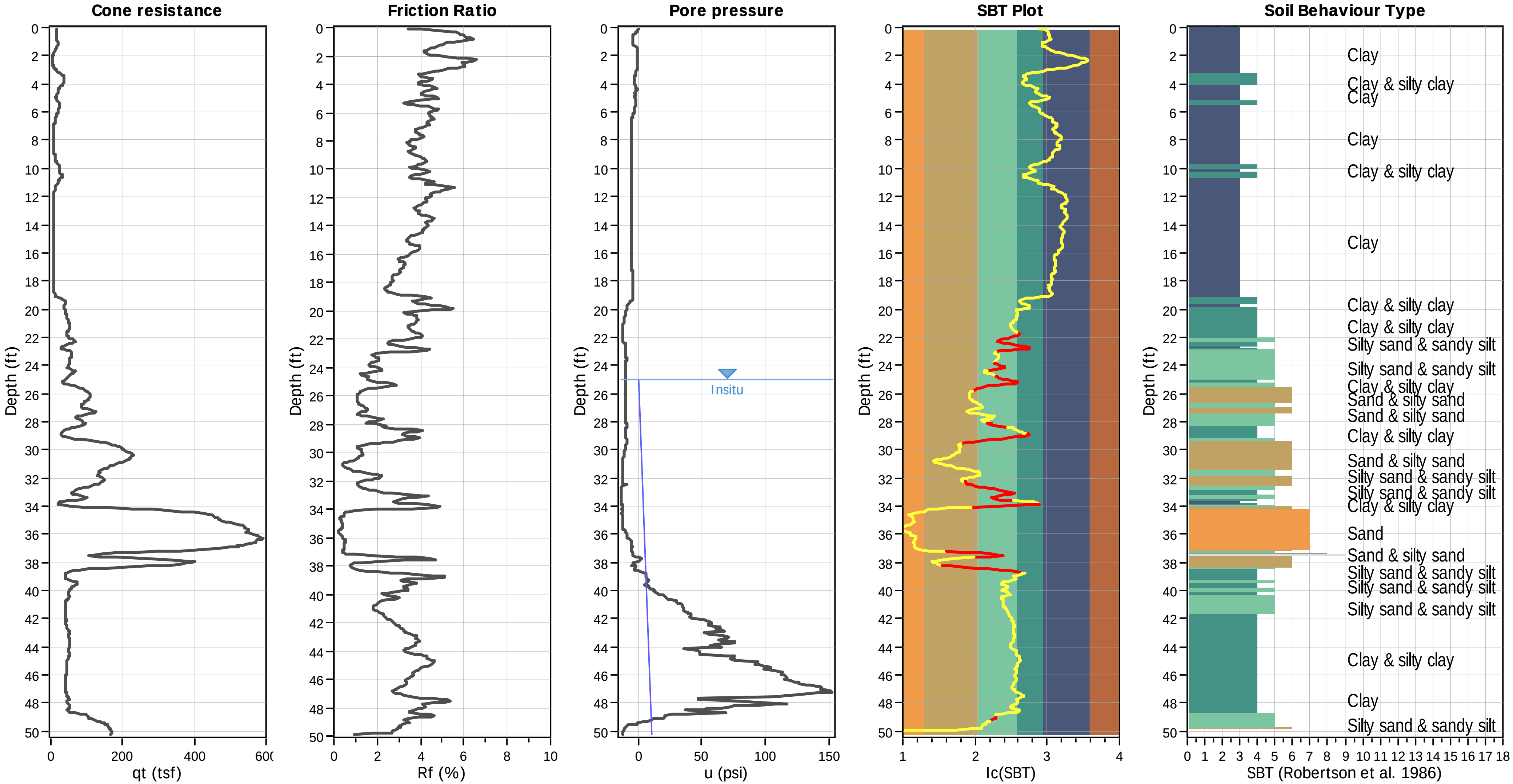
Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							41.01
Puente Hills (Santa Fe Springs) [0]		4.06	7.17	0.85	118.033°W	33.948°N	8.41	12.23
Compton [1]		13.82	7.29	0.81	118.161°W	33.764°N	210.57	6.62
Puente Hills (Coyote Hills) [1]		5.05	7.28	0.77	118.044°W	33.915°N	196.70	5.15
Whittier alt 2 [6]		5.16	6.96	1.21	118.010°W	33.983°N	26.80	4.06
Puente Hills (LA) [0]		9.85	7.16	1.43	118.116°W	33.990°N	304.91	2.55
Anaheim [2]		8.84	7.19	0.86	118.063°W	33.881°N	200.71	1.86
Whittier alt 2 [5]		5.96	6.71	1.39	117.982°W	33.972°N	56.00	1.45
Compton [2]		18.19	7.50	1.50	118.286°W	33.817°N	239.03	1.04
UC33brAvg_FM31	System							38.24
Whittier alt 1 [7]		5.19	6.73	1.29	118.009°W	33.983°N	27.65	8.68
Puente Hills [1]		5.82	7.33	0.88	118.031°W	33.946°N	40.91	8.37
Compton [1]		13.82	7.25	0.81	118.161°W	33.764°N	210.57	6.38
Whittier alt 1 [6]		5.55	6.42	1.45	117.990°W	33.975°N	48.51	2.63
Anaheim [2]		8.84	7.13	0.90	118.063°W	33.881°N	200.71	1.78
Compton [2]		18.19	7.53	1.50	118.286°W	33.817°N	239.03	1.22
Puente Hills [2]		6.07	7.03	0.96	118.052°W	33.949°N	294.41	1.05
Elysian Park (Upper) [0]		16.56	6.55	2.32	118.097°W	34.077°N	338.86	1.01
Puente Hills [3]		12.03	6.86	1.61	118.143°W	33.972°N	288.28	1.00
UC33brAvg_FM32 (opt)	Grid							10.92
PointSourceFinite: -118.034, 33.992		7.33	5.71	1.71	118.034°W	33.992°N	0.00	1.94
PointSourceFinite: -118.034, 33.992		7.33	5.71	1.71	118.034°W	33.992°N	0.00	1.94
PointSourceFinite: -118.034, 34.010		8.62	5.77	1.87	118.034°W	34.010°N	0.00	1.89
PointSourceFinite: -118.034, 34.010		8.62	5.77	1.87	118.034°W	34.010°N	0.00	1.89
UC33brAvg_FM31 (opt)	Grid							9.83
PointSourceFinite: -118.034, 34.010		8.67	5.74	1.88	118.034°W	34.010°N	0.00	1.73
PointSourceFinite: -118.034, 34.010		8.67	5.74	1.88	118.034°W	34.010°N	0.00	1.73
PointSourceFinite: -118.034, 33.992		7.35	5.69	1.72	118.034°W	33.992°N	0.00	1.61
PointSourceFinite: -118.034, 33.992		7.35	5.69	1.72	118.034°W	33.992°N	0.00	1.61

Appendix I

Liquefaction Analyses

CPT basic interpretation plots



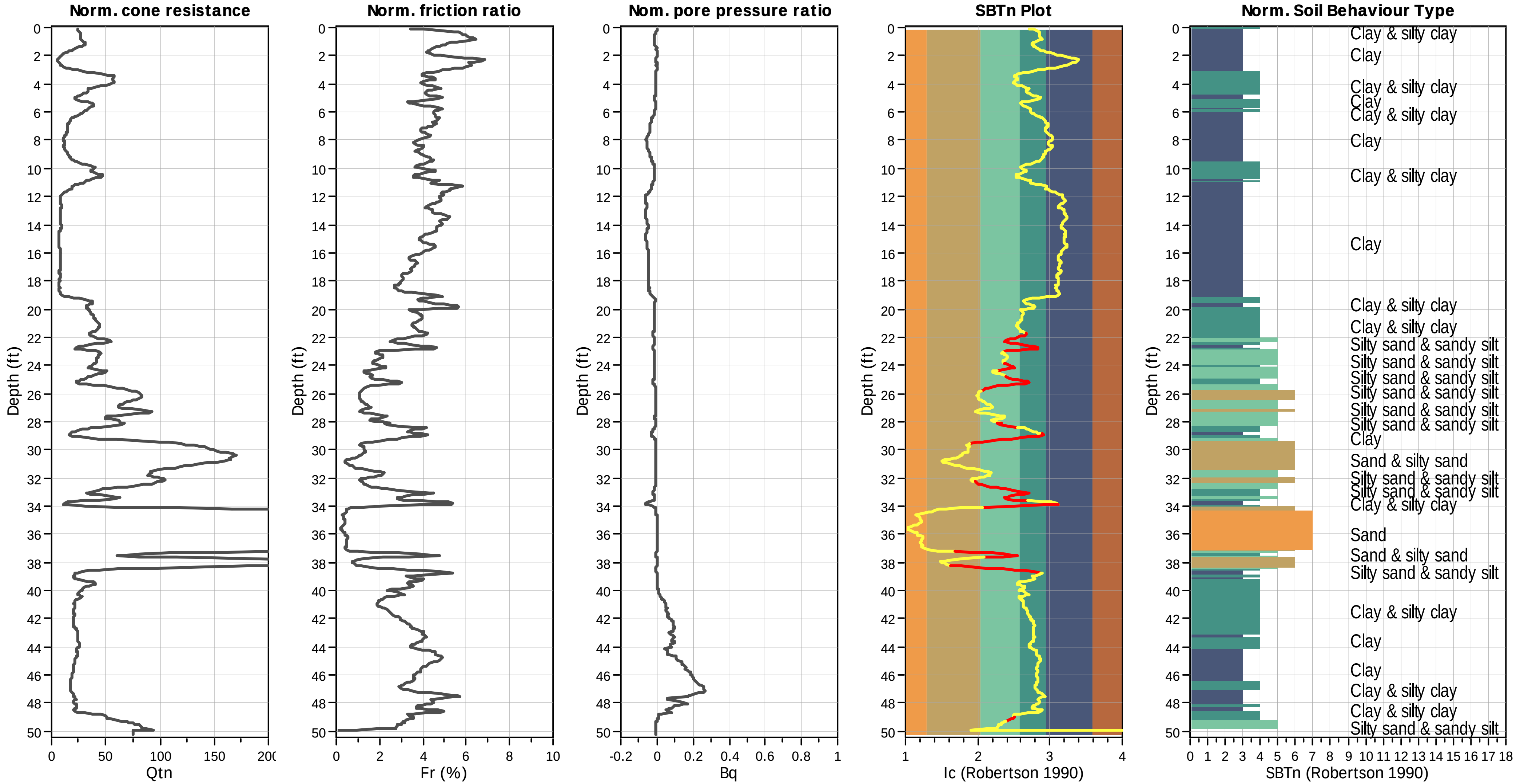
Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.85	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

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

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
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Earthquake magnitude M_w:	6.85	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
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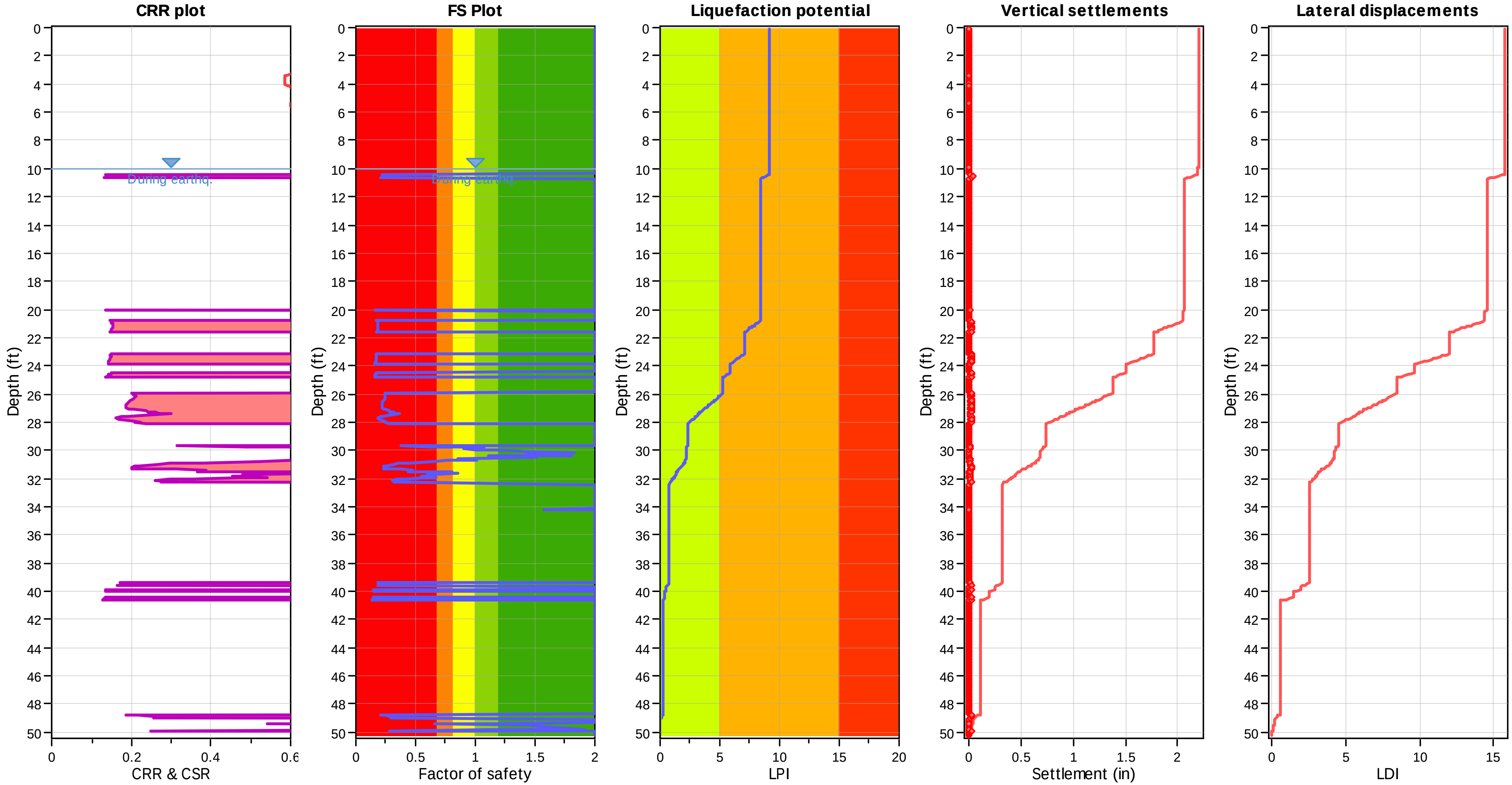
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.85	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

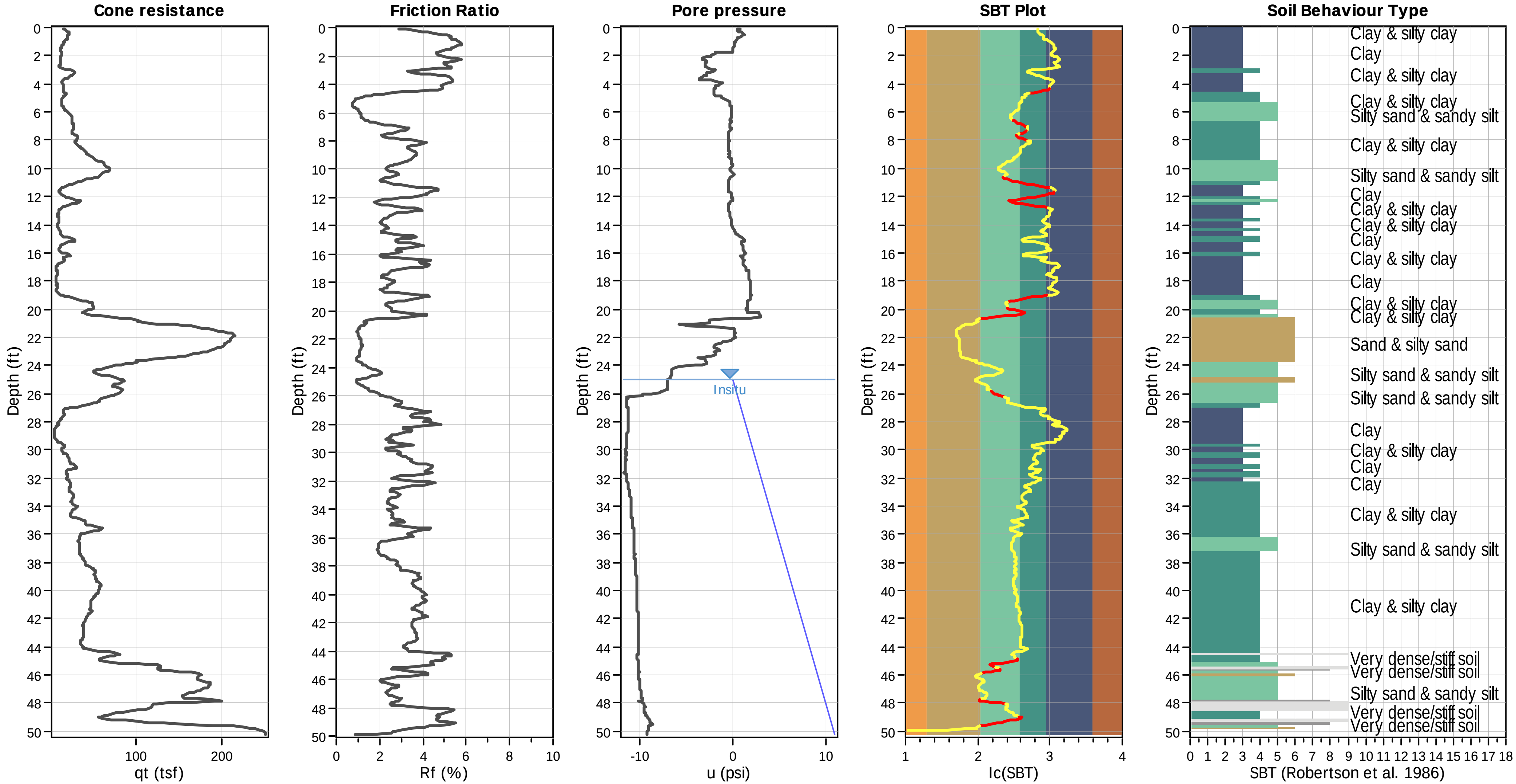
F.S. color scheme

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

LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
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SBT legend

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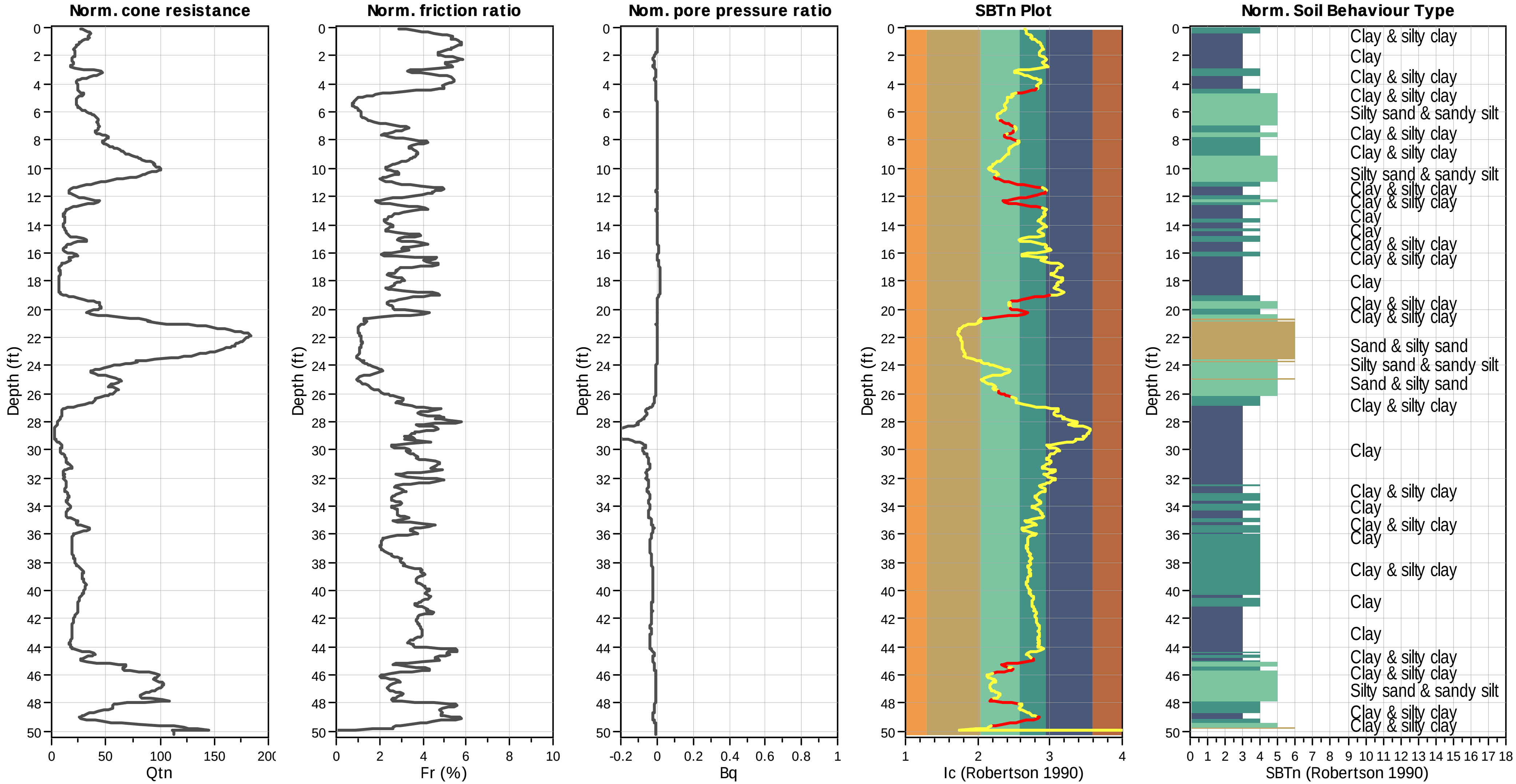
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
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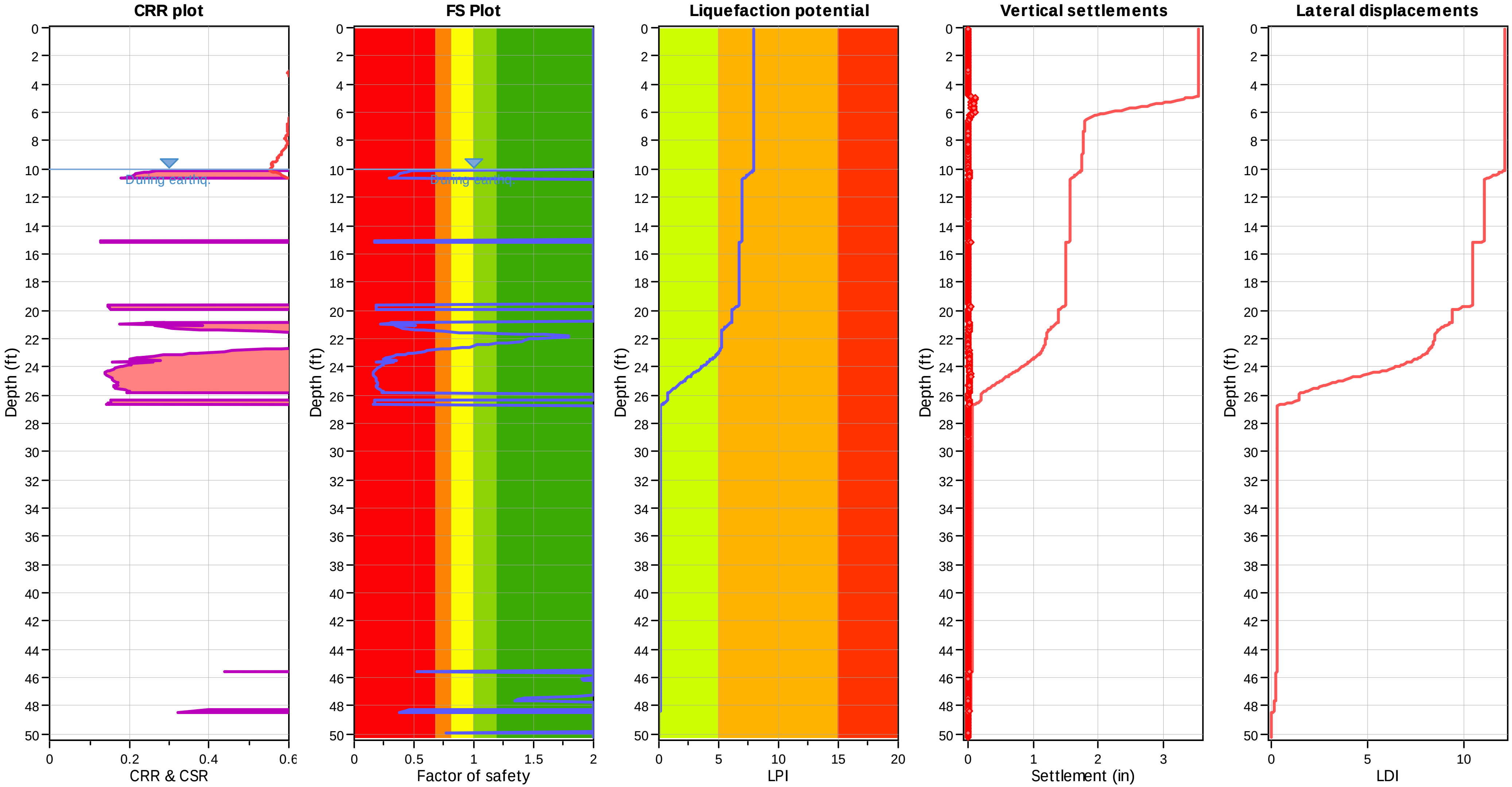
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

Liquefaction analysis overall plots



Input parameters and analysis data

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Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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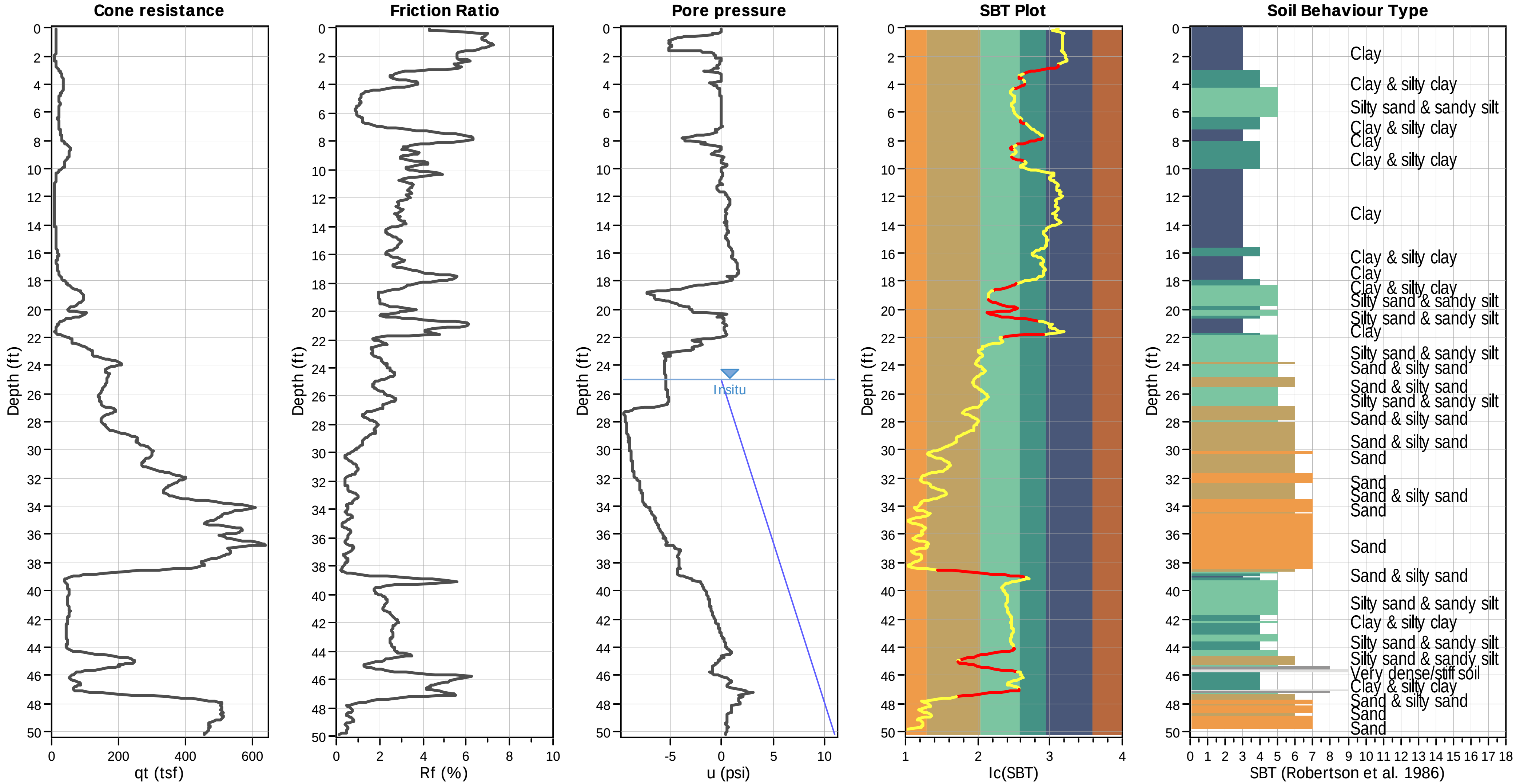
LPI color scheme

- Very high risk
- High risk
- Low risk

CPT basic interpretation plots



Input parameters and analysis data

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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
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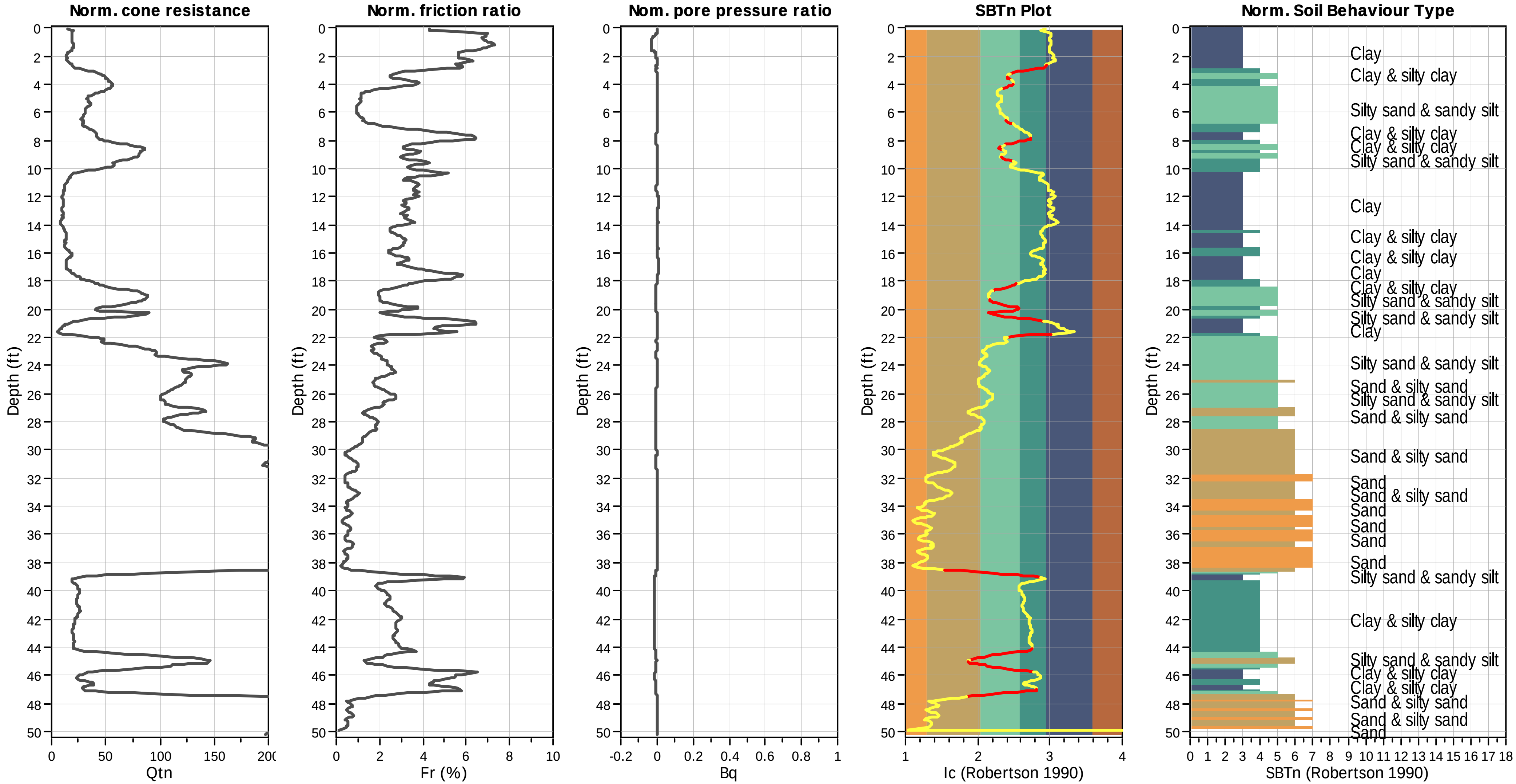
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to
9. Very stiff fine grained

CPT basic interpretation plots (normalized)



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SBTn legend

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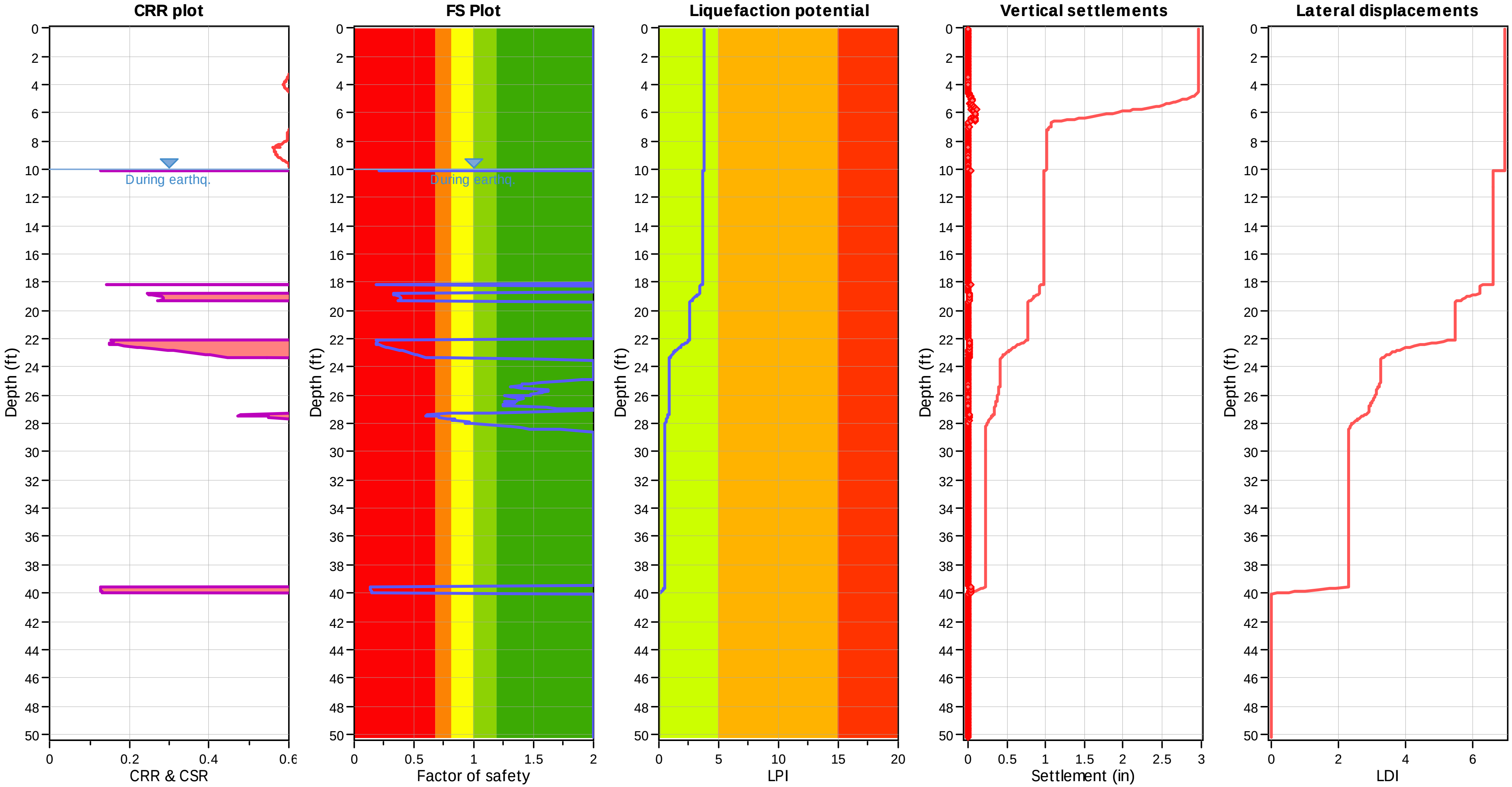
Liquefaction analysis overall plots



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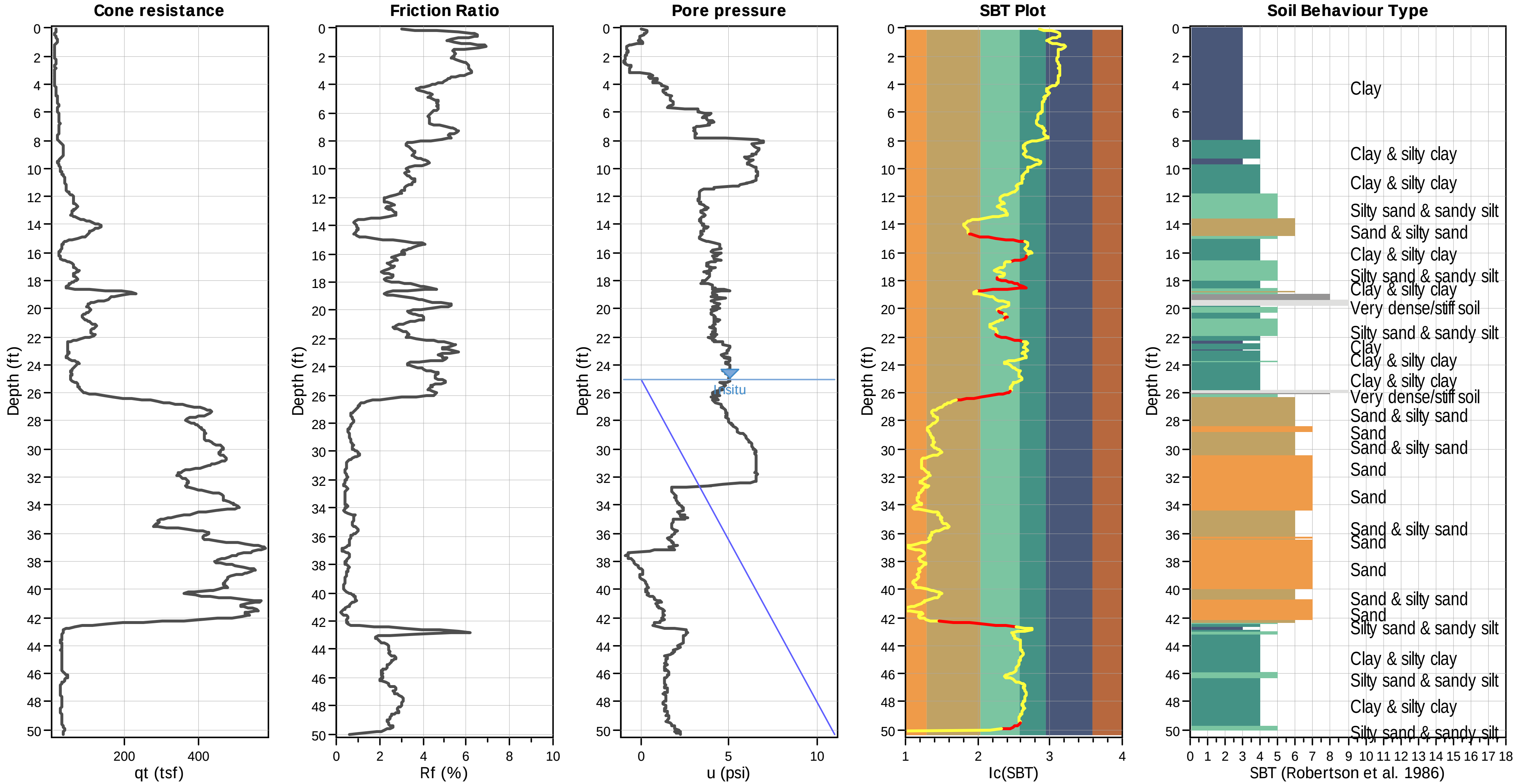
CPT basic interpretation plots



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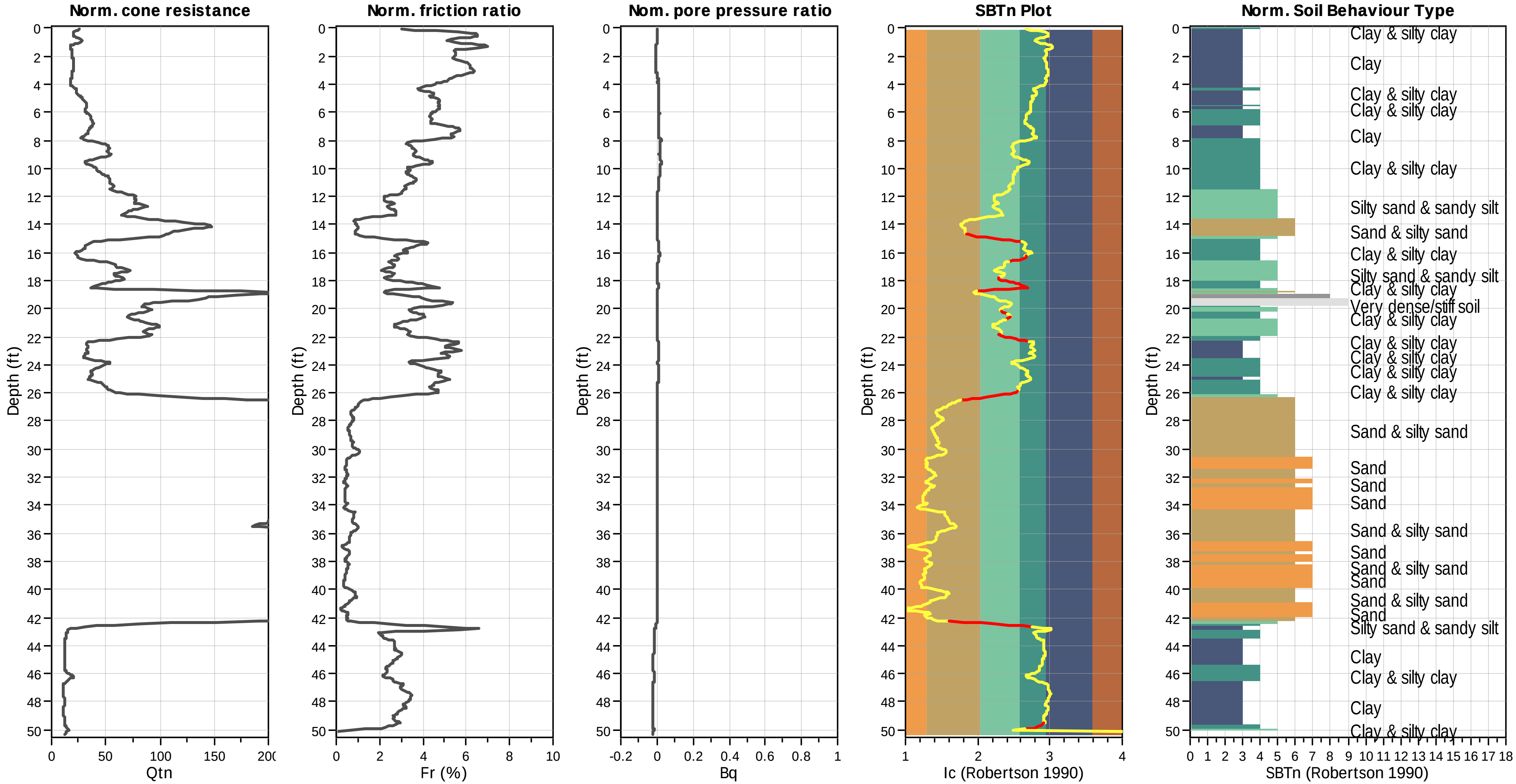
3. Clay to silty clay	6. Clean sand to silty sand	9. Very stiff fine grained

CPT basic interpretation plots (normalized)



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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.85	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	Yes
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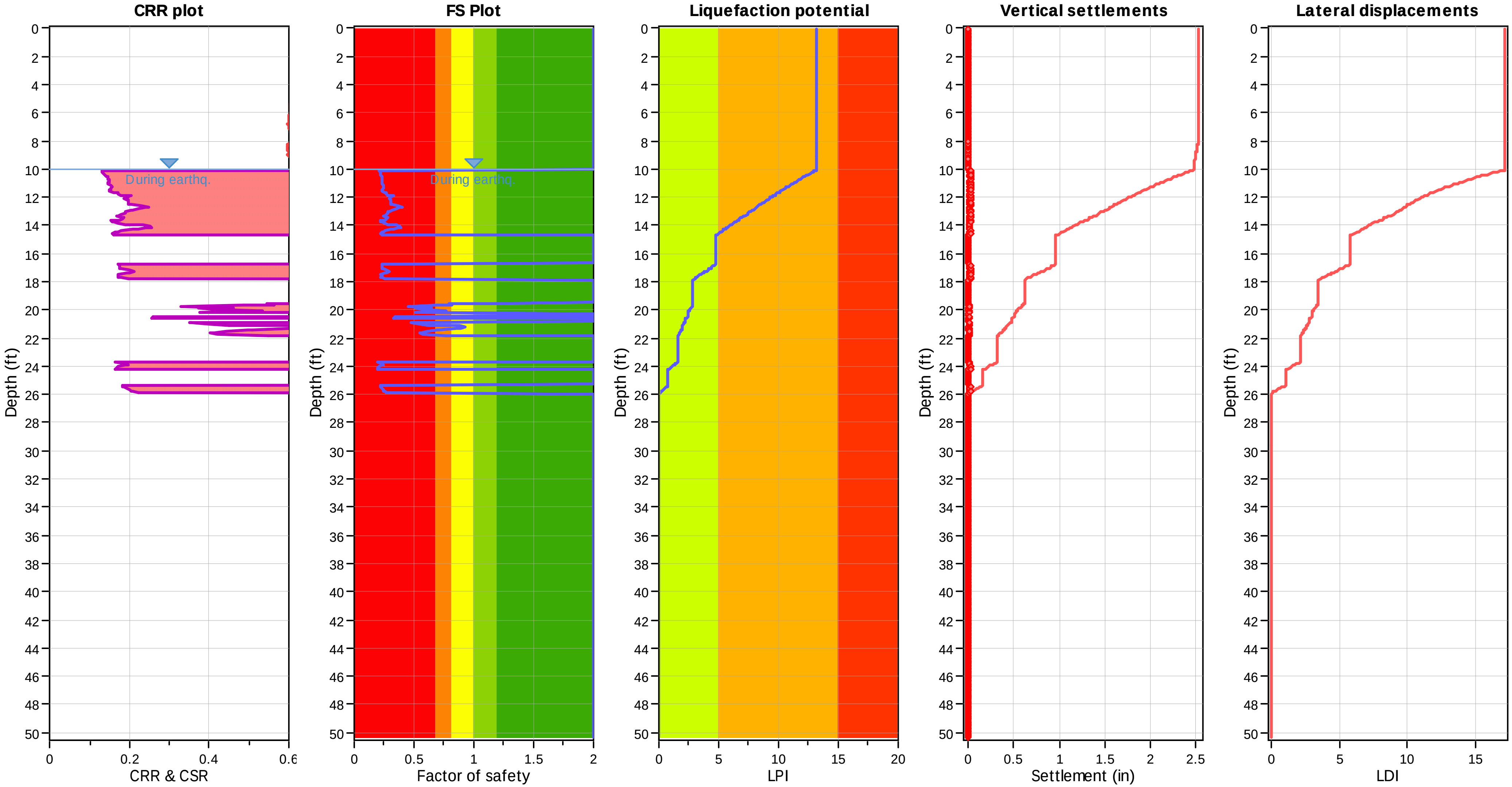
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	10.00 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	6.85	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.84	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	25.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

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LPI color scheme

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- Low risk

Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	TET 20-179E Adventure Park - Whittier	Boring:	**B-1**	Engineer:		Date:	4/17/2020

Liquefaction Evaluation Method		
Correction for fines content	Idriss & Boulang. 2008, 2014	
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs	
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014	
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014	
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014	
Magnitude scaling factor MSF	Idriss & Boulang. 2014	
Dry settlement	Pradel, 1998a,b	
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration	

Liquefaction behavior Plasticity Index threshold	less or equal to 12
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Liquefaction Analysis Statistics		
Total thickness of evaluated profile	46.5	feet
Profile thickness susceptible to liquefaction	20.5	feet
Number of evaluated intervals	11	
Number of potentially liquefiable intervals	9	
Average Factor of Safety of sandy intervals	**2.06**	
Dry sand settlement	0.13	inches
Liquefaction settlement	1.71	inches
Total earthquake-induced settlement	**1.84**	**inches**

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Total Unit Weight Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	Considered Blowcounts N1,60	Considered Blowcounts N1,60,cs	Factor of Safety Liquefaction	Factor of Safety Cyclic softening	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	5	132.25	135.0	20	n/plastic	18.8	34.4	38.9	–	–	- no groundwater	0.13	1.84
5	5	132.25	135.0	70	4	18.8	26.3	31.9	–	–	- no groundwater	0.00	1.71
10	5	136.85	135.0	94	9	15.0	20.1	25.6	0.65	–	- liquefieable - FS < 1.3	0.00	1.71
15	5	127.65	135.0	70	4	20.0	26.3	31.9	1.29	–	- liquefieable - FS < 1.3	0.20	1.71
20	4	123.42	135.0	8	n/plastic	35.0	43.8	44.2	3.79	–	- too dense – (N1)60,CS > 32	0.00	1.51
24	4	123.42	135.0	8	n/plastic	50.3	63.3	63.7	3.58	–	- too dense – (N1)60,CS > 32	0.00	1.51
28	4	133.1	135.0	8	n/plastic	50.0	65.0	65.4	3.43	–	- too dense – (N1)60,CS > 32	0.00	1.51
32	4	140.36	145.0	8	n/plastic	50.0	63.7	64.0	3.32	–	- too dense – (N1)60,CS > 32	0.00	1.51
36	4	133.4	135.0	70	4	23.0	24.9	30.5	0.85	–	- liquefieable - FS < 1.3	0.37	1.51
40	5	139.15	135.0	70	4	18.0	18.2	23.8	0.42	–	- liquefieable - FS < 1.3	1.07	1.14
45	1.5	152.95	151.0	70	4	26.0	27.4	32.9	1.23	–	- too dense – (N1)60,CS > 32	0.07	0.07
255.00	46.50	1474.80	1511.00	496.00	29.00	324.77	413.39	452.72	18.56	0.00		1.84	15.71

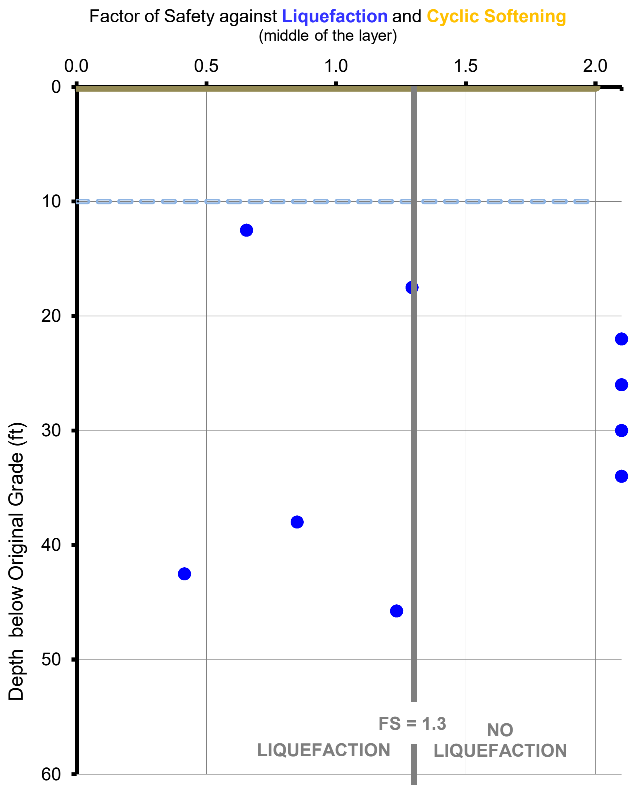
Profile		
In-Situ Groundwater depth	32.00	feet
DESIGN Groundwater depth	10.00	feet
DESIGN Excavation depth	0.00	feet
DESIGN Surcharge (fill)	0.00	feet

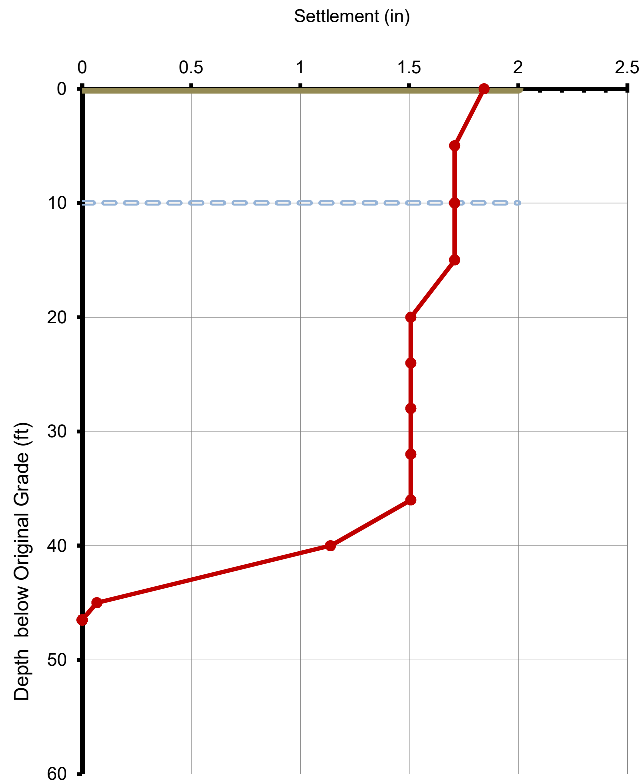
Earthquake loading	
M	6.85
PGA	0.836

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

Version v2 2018-07

B-1 Design excavation depth 0.00 feet
Design groundwater depth 10.00 feet





Summary of Liquefaction and Earthquale-Induced Settlement Analysis

Project:	TET 20-179E Adventure Park - Whittier	Boring:	B-3	Engineer:		Date:	4/17/2020

Liquefaction Evaluation Method	
Correction for fines content	Idriss & Boulang. 2008, 2014
Correction for overburden C_N	Idriss & Boulang. 2014 (N1)60cs
Cyclic resistance ratio of soil CRR_{CS}	Idriss & Boulang. 2004, 2014
Correction for overburden Kσ	Idriss & Boulang. 2008, 2014
Stress reduction factor r_D	Idriss 1999, I&B 2008,2014
Magnitude scaling factor MSF	Idriss & Boulang. 2014
Dry settlement	Pradel, 1998a,b
Liquefaction settlement	Yoshimine et al., 2006 – w/ calibration

Liquefaction behavior Plasticity Index threshold	less or equal to 12
Saturated settlement threshold	less or equal to 70% fines
Dry settlement threshold	less or equal to 50% fines
Cyclic softening Plasticity Index threshold	greater or equal to 18

Liquefaction Analysis Statistics	
Total thickness of evaluated profile	51.5 feet
Profile thickness susceptible to liquefaction	17 feet
Number of evaluated intervals	12
Number of potentially liquefiable intervals	6
Average Factor of Safety of sandy intervals	**1.49**
Dry sand settlement	0.00 inches
Liquefaction settlement	2.66 inches
Total earthquake-induced settlement	**2.66 inches**

Depth to Layer Top	Layer Thickness	Total Unit Weight In-situ	Total Unit Weight Design	Fines %	Plasticity Index	Considered Blowcounts SPT-N	N1,60	N1,60,cs	Factor of Safety Liquefaction	Cyclic softening	Liquefaction potential rationale	Layer Settlement	Cumulative Settlement
feet	feet	pcf	pcf	%	–	bpf	bpf	bpf			–	in	in
0	4	121.54	130.0	70	25	10.7	24.6	30.2	–	–	- no groundwater	0.00	2.66
4	6	121.54	130.0	70	25	10.7	20.6	26.1	–	–	- no groundwater	0.00	2.66
10	3	115.64	130.0	84	15	12.0	21.5	27.0	–	–	- clay-like behaviour	0.00	2.66
13	7	115.64	130.0	84	15	10.7	16.8	22.4	–	–	- clay-like behaviour	0.00	2.66
20	3	115.64	130.0	84	15	65.0	112.2	117.8	–	–	- clay-like behaviour	0.00	2.66
23	5.5	123.9	130.0	70	4	16.1	23.9	29.5	0.79	–	- liquefieable - FS < 1.3	0.60	2.66
28.5	1.5	123.9	130.0	70	4	16.1	22.8	28.4	0.67	–	- liquefieable - FS < 1.3	0.20	2.06
30	5	128.1	135.0	1	n/plastic	26.0	39.5	39.5	3.36	–	- too dense – (N1)60,CS > 32	0.00	1.86
35	5	133.4	135.0	1	n/plastic	17.4	24.1	24.1	0.42	–	- liquefieable - FS < 1.3	1.06	1.86
40	5	133.4	135.0	1	n/plastic	20.0	27.2	27.2	0.56	–	- liquefieable - FS < 1.3	0.80	0.80
45	5	133.4	135.0	1	n/plastic	47.6	78.8	78.8	3.13	–	- too dense – (N1)60,CS > 32	0.00	0.00
50	1.5	115.64	130.0	84	15	24.0	33.2	38.8	–	–	- clay-like behaviour	0.00	0.00
298.50	51.50	1481.74	1580.00	620.00	118.00	276.31	445.19	489.62	8.93	0.00		2.66	22.55

Profile	
In-Situ Groundwater depth	28.50 feet
DESIGN Groundwater depth	10.00 feet
DESIGN Excavation depth	0.00 feet
DESIGN Surcharge (fill)	0.00 feet

Earthquake loading	
M	6.85
PGA	0.836

Checks	
Groundwater depth check	OK
Design groundwater/excavation depth check	OK
Fines correction method compatibility	OK
Idris & Boulanger, 2004 method for C_N	not used
Cetin 2009 settlement method	not used

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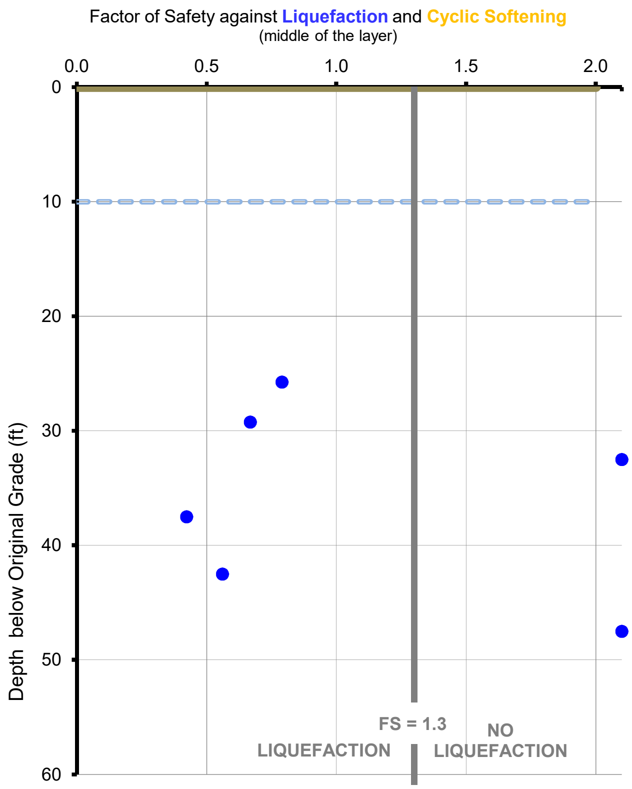
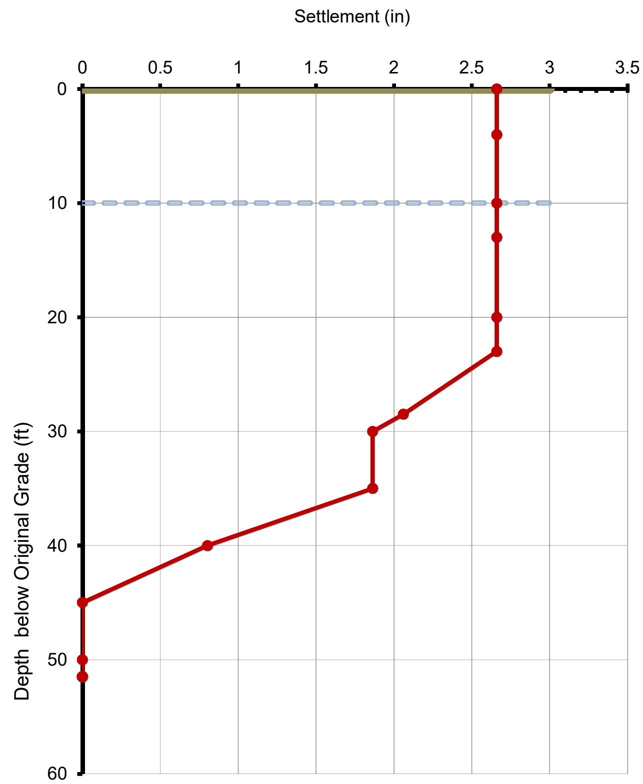
B-3 Design excavation depth 0.00 feet
Design groundwater depth 10.00 feet





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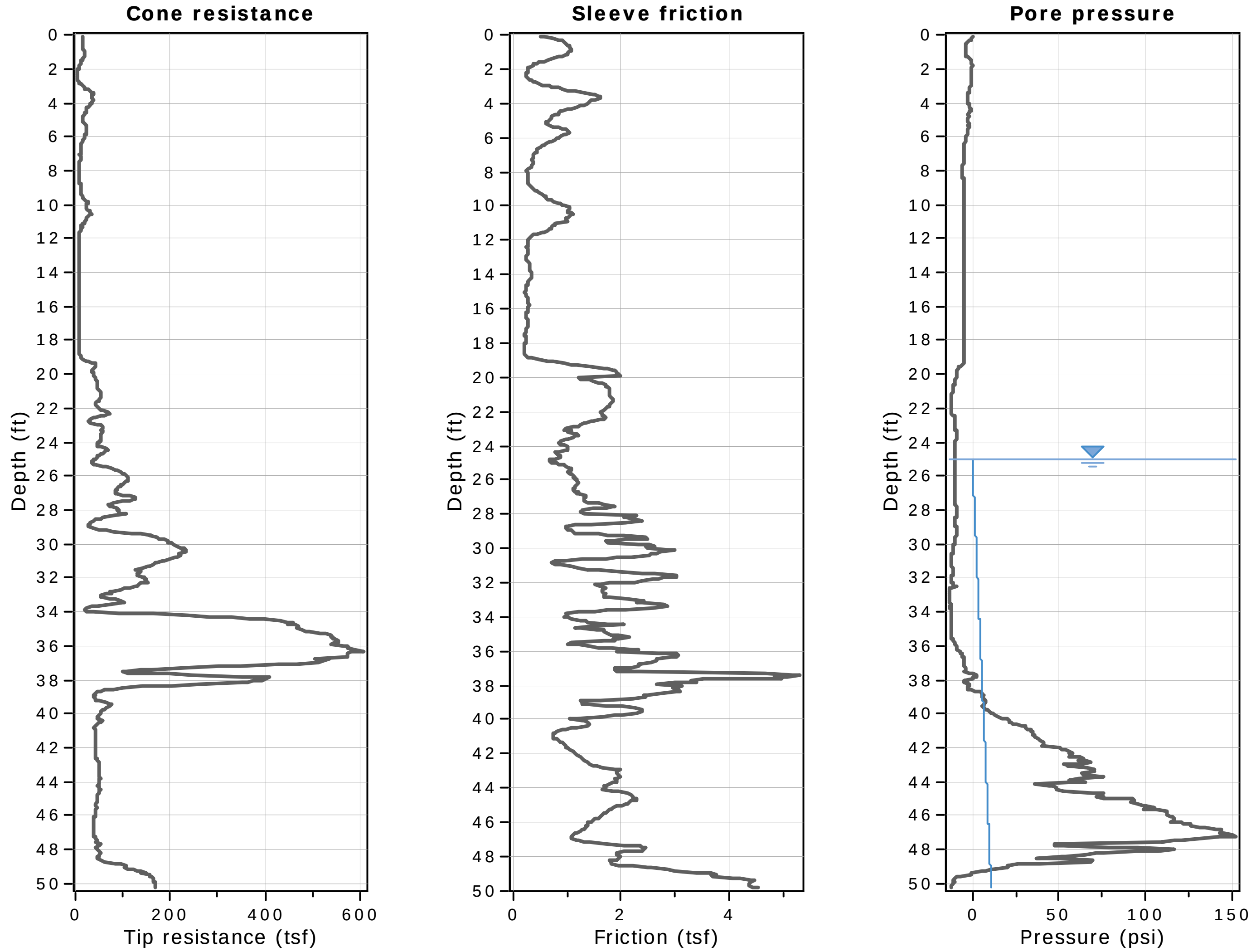
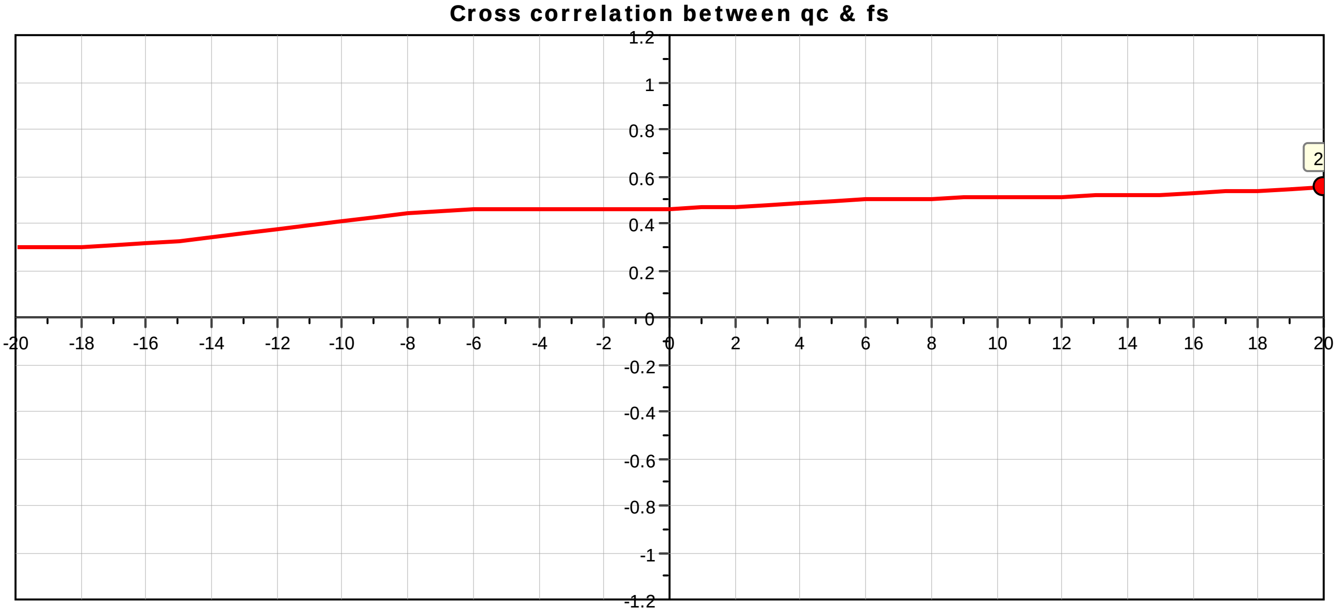
CPT: CPT-1
Total depth: 50.23 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).

Cross correlation between qc & fs



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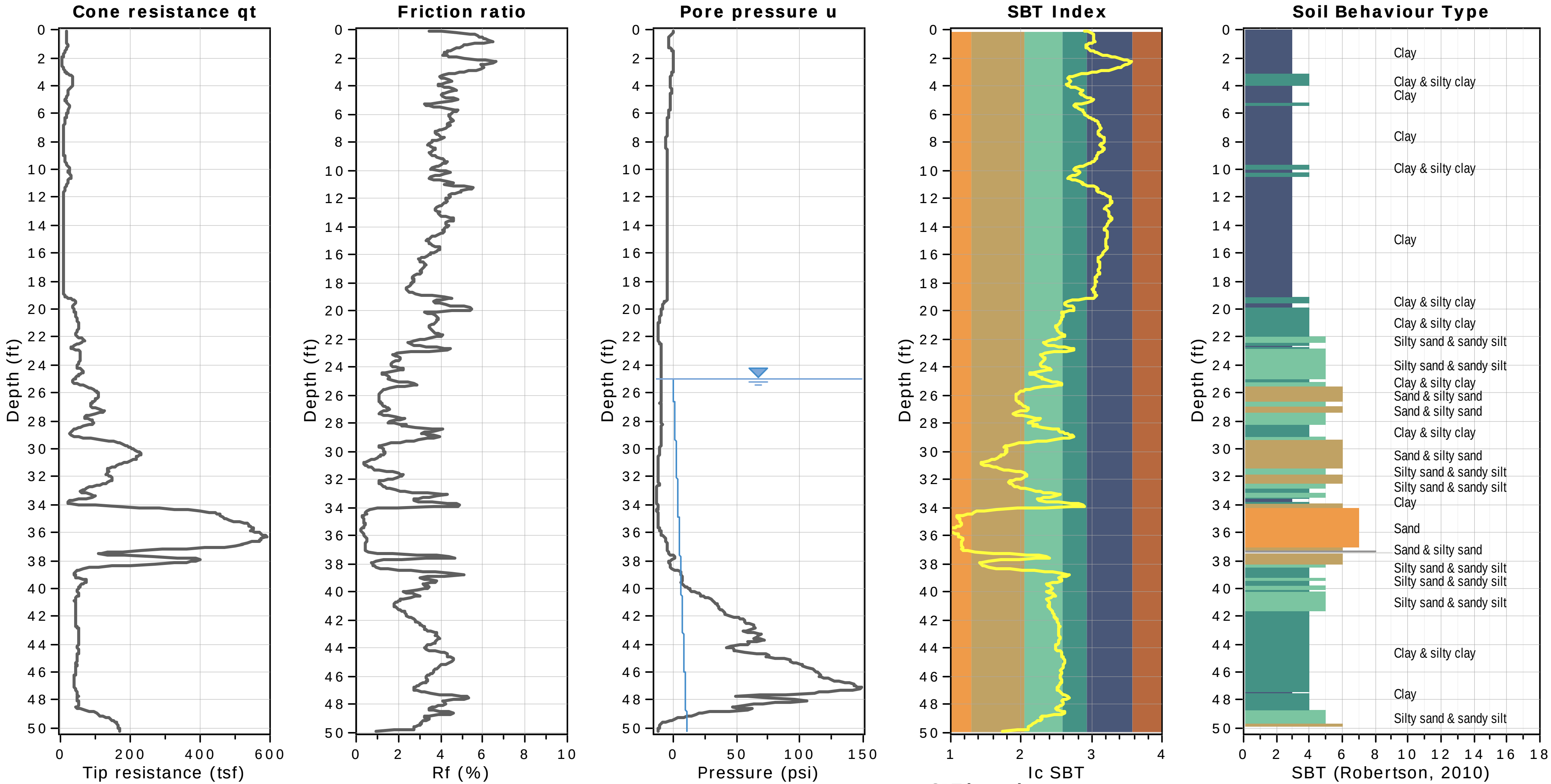
CPT: CPT-1
Total depth: 50.23 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



SBT legend

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

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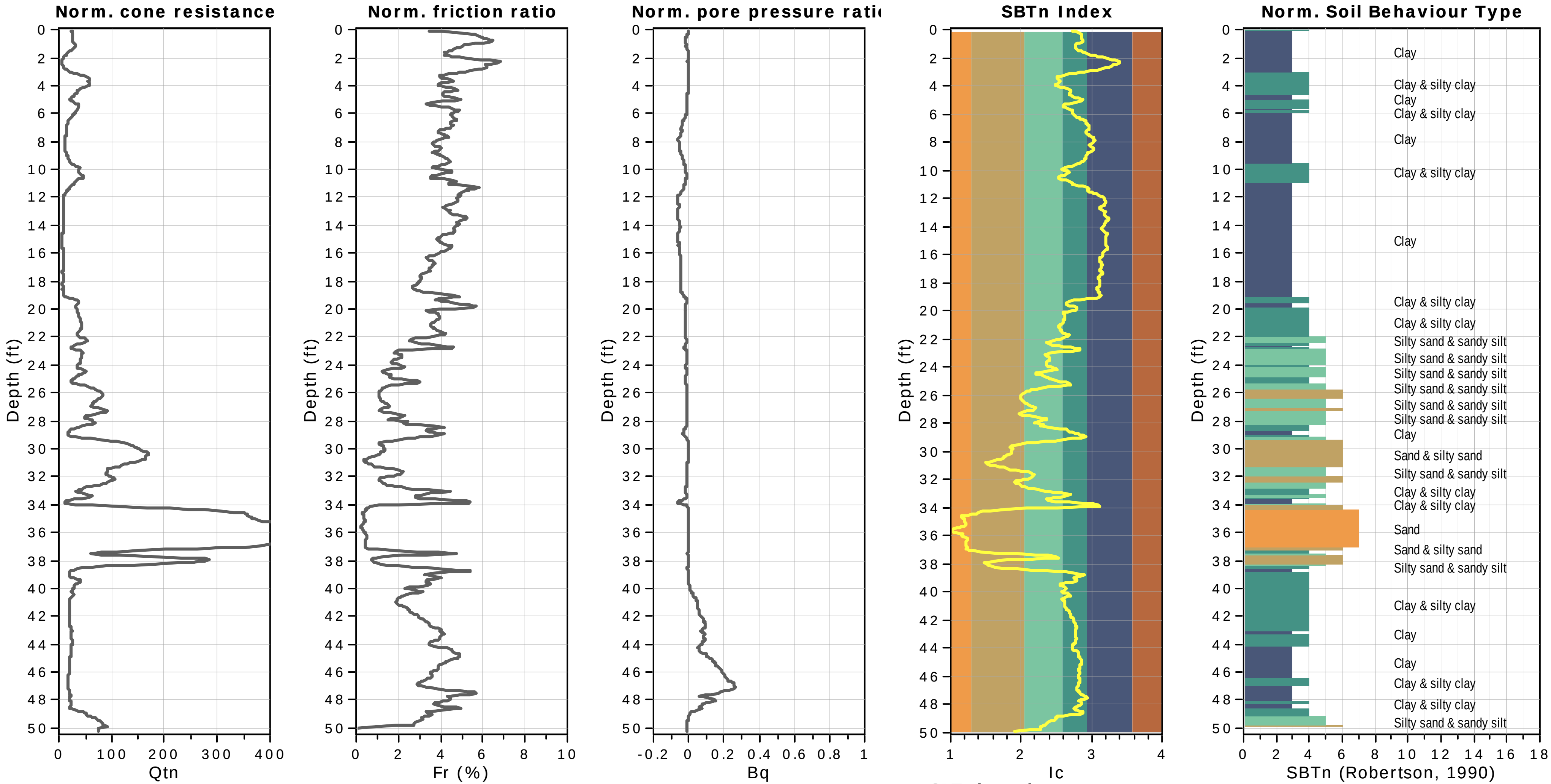
CPT: CPT-1
Total depth: 50.23 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



SBTn legend

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

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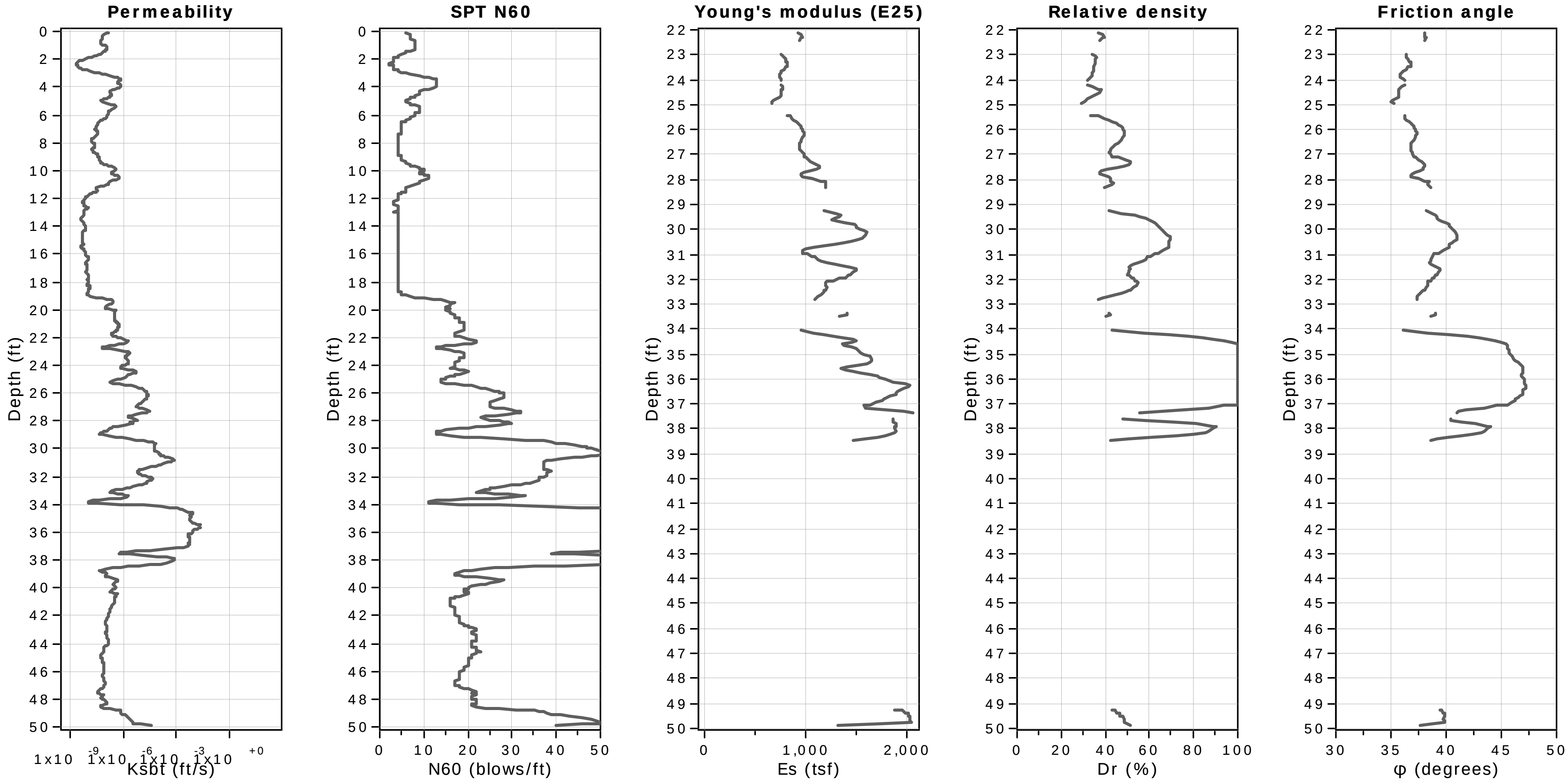
CPT: CPT-1
Total depth: 50.23 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



Calculation parameters

Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

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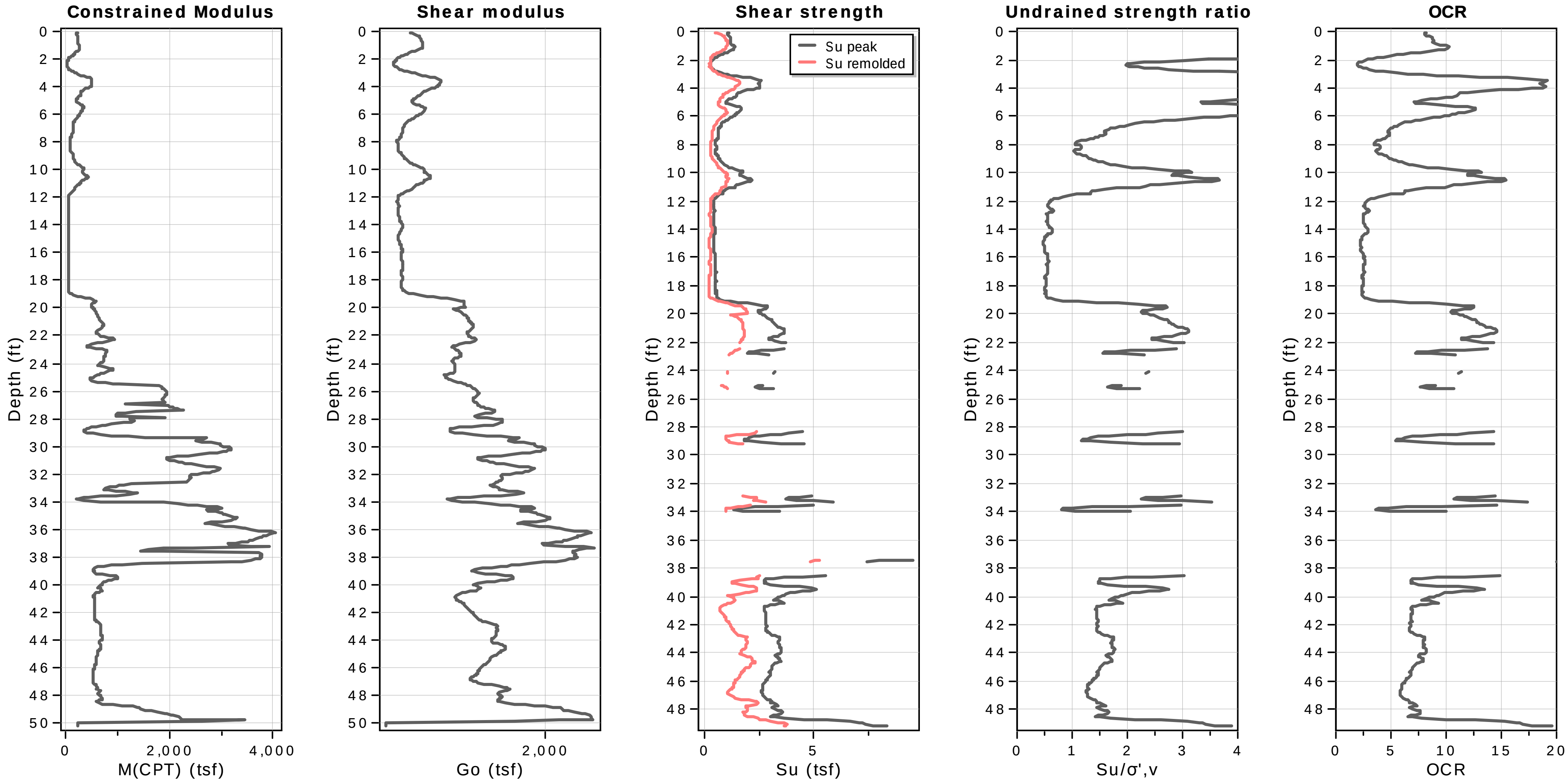
CPT: CPT-1
Total depth: 50.23 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



Calculation parameters
Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14
OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data

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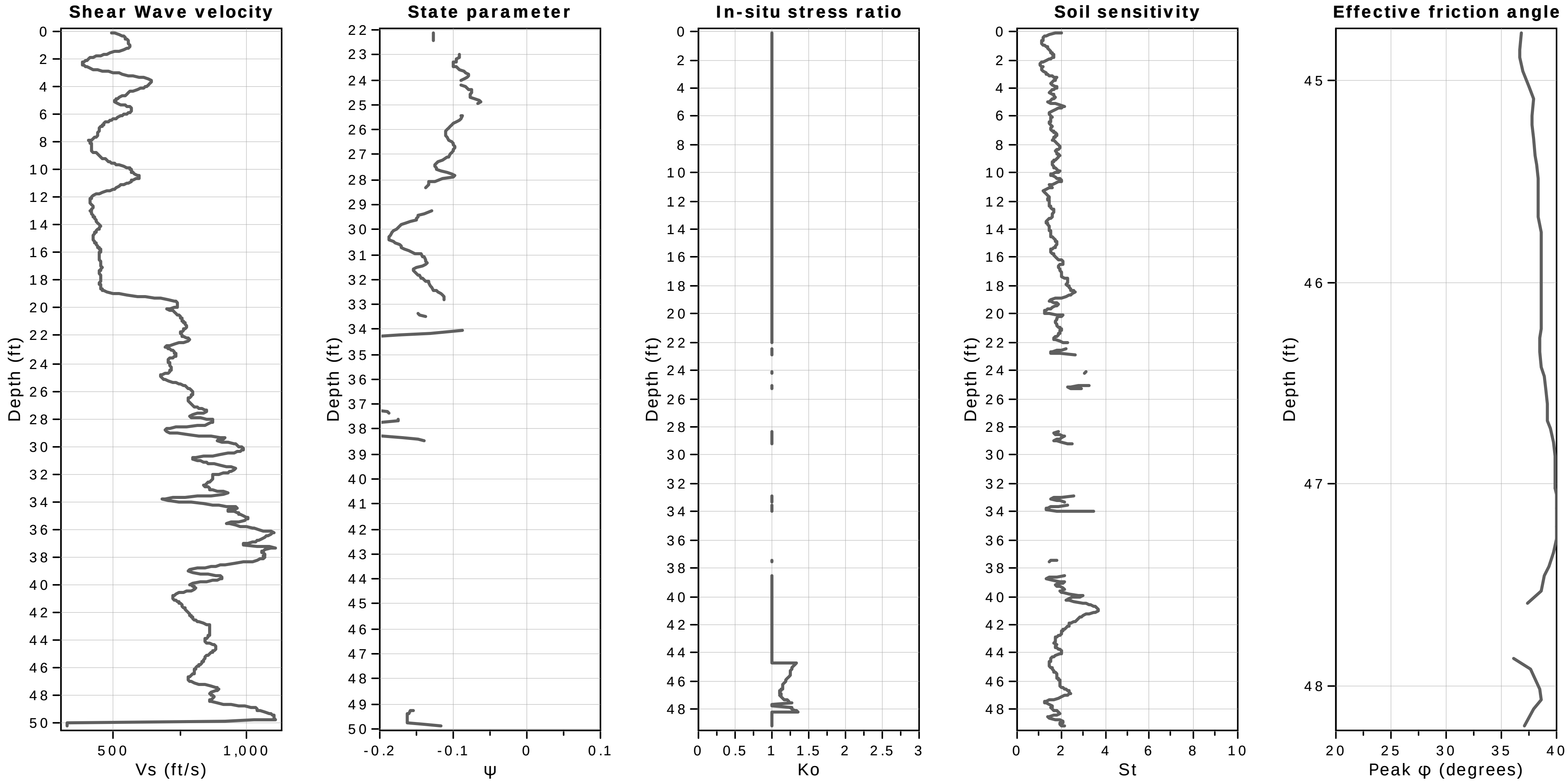
CPT: CPT-1
Total depth: 50.23 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



Calculation parameters

Soil Sensitivity factor, N_S: 7.00
User defined estimation data

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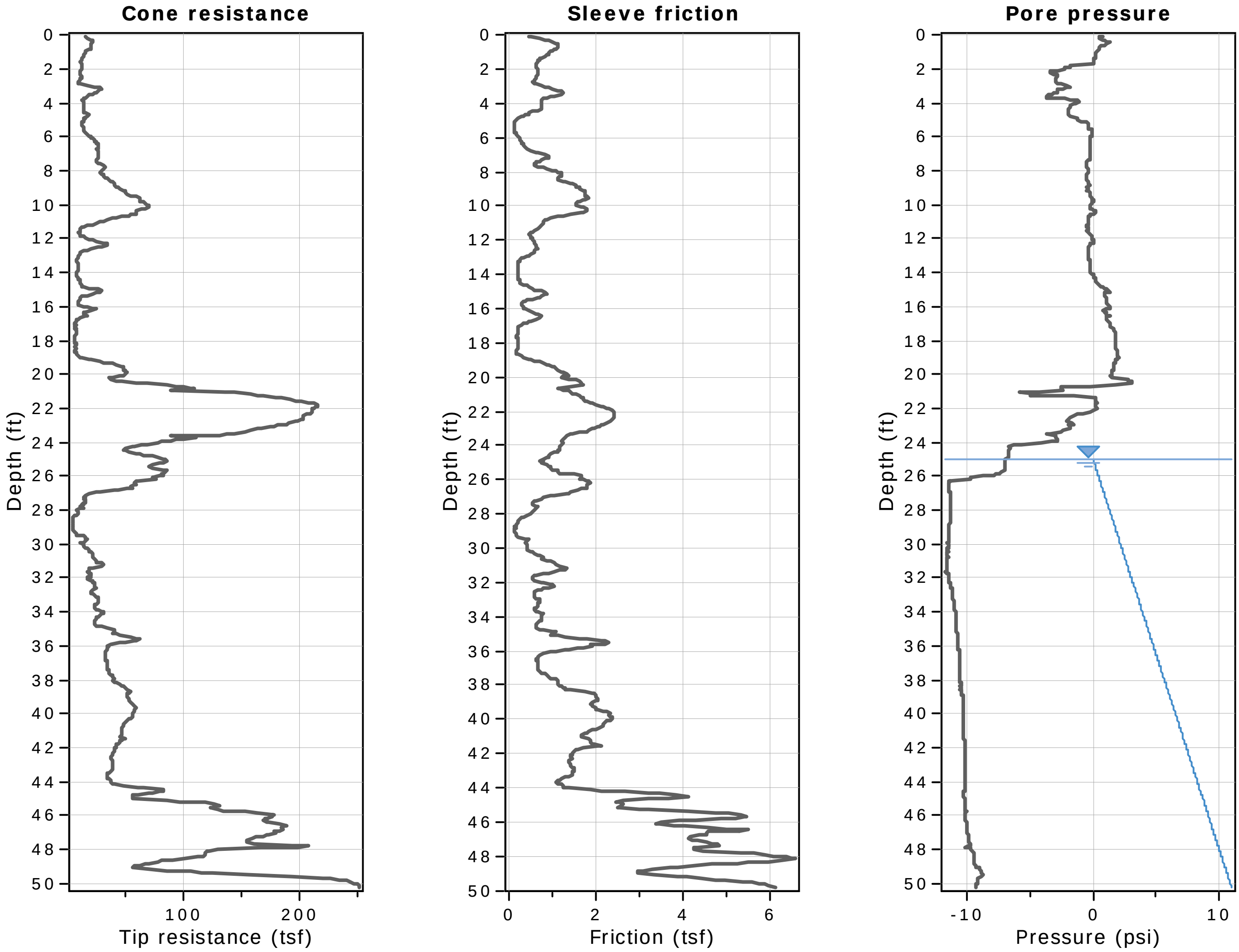
CPT: CPT-2
Total depth: 50.22 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: Adventure Park
Location: Whittier, California



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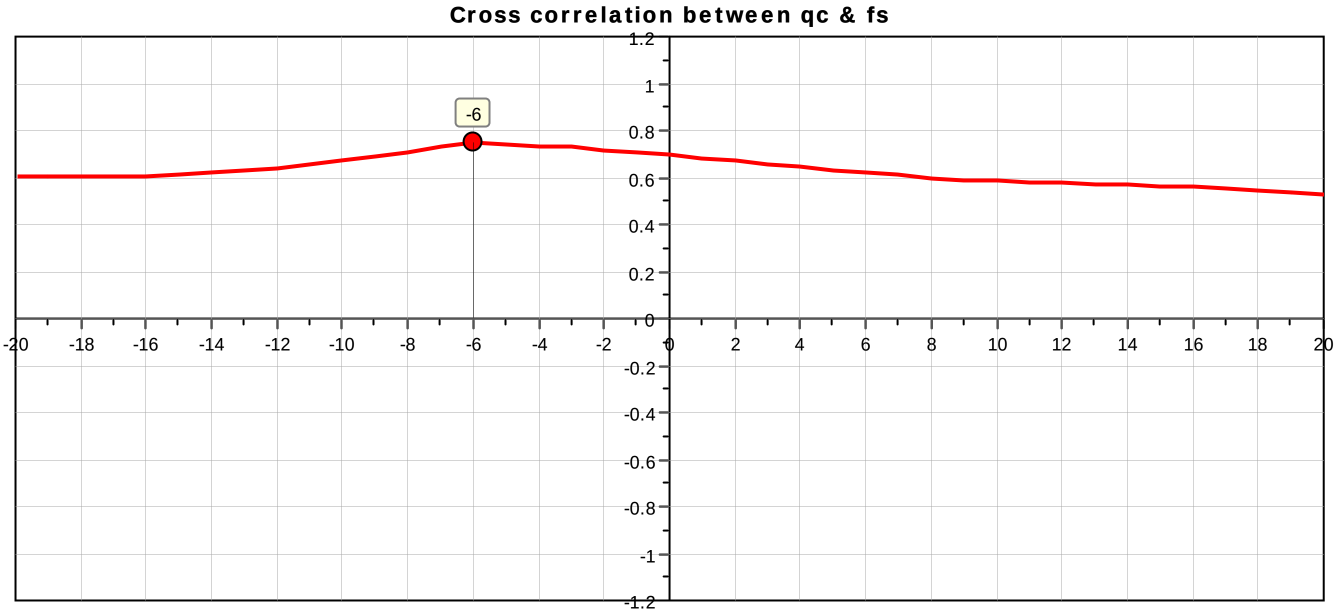

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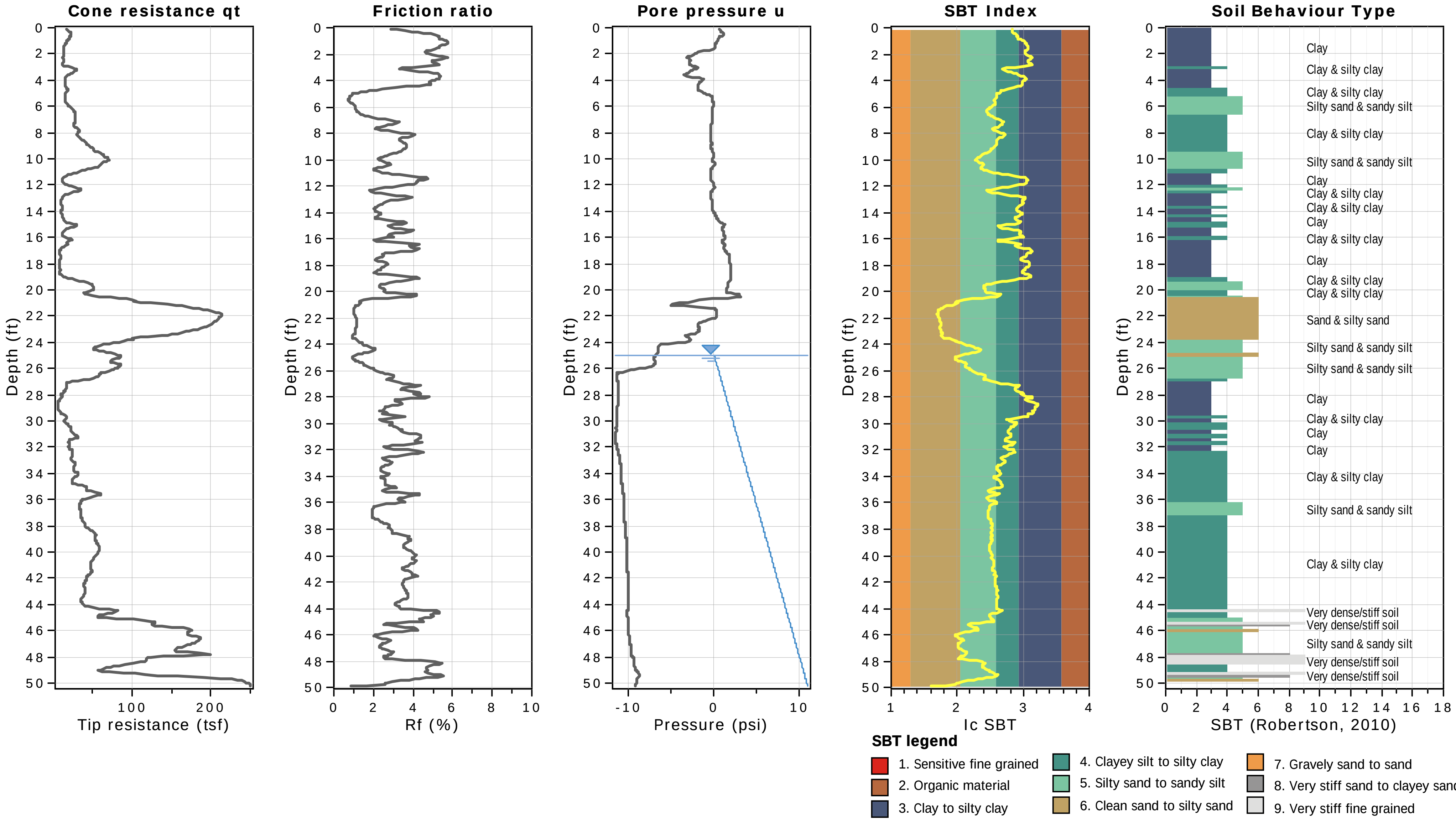
CPT: CPT-2
Total depth: 50.22 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: Adventure Park
Location: Whittier, California



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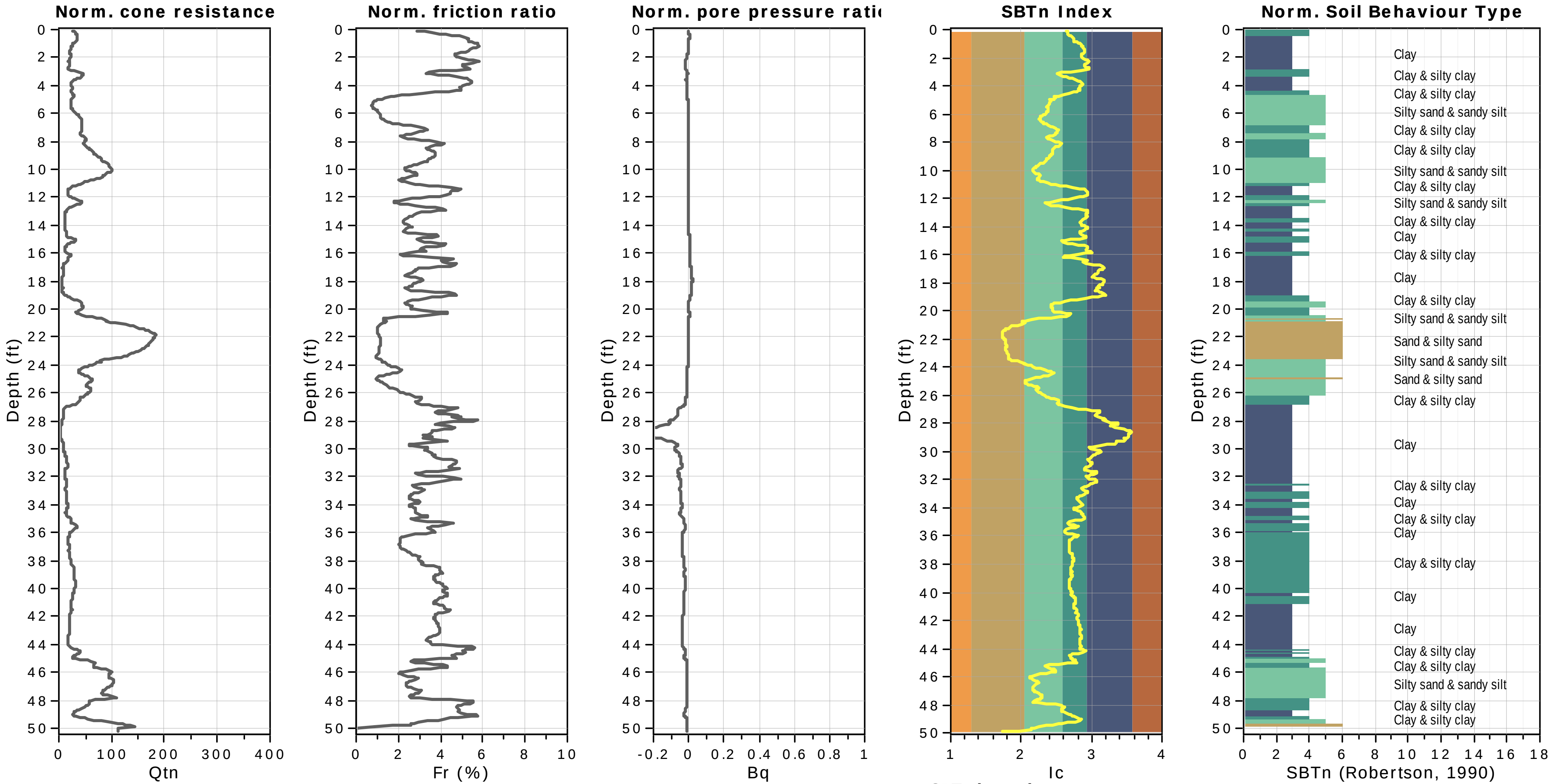
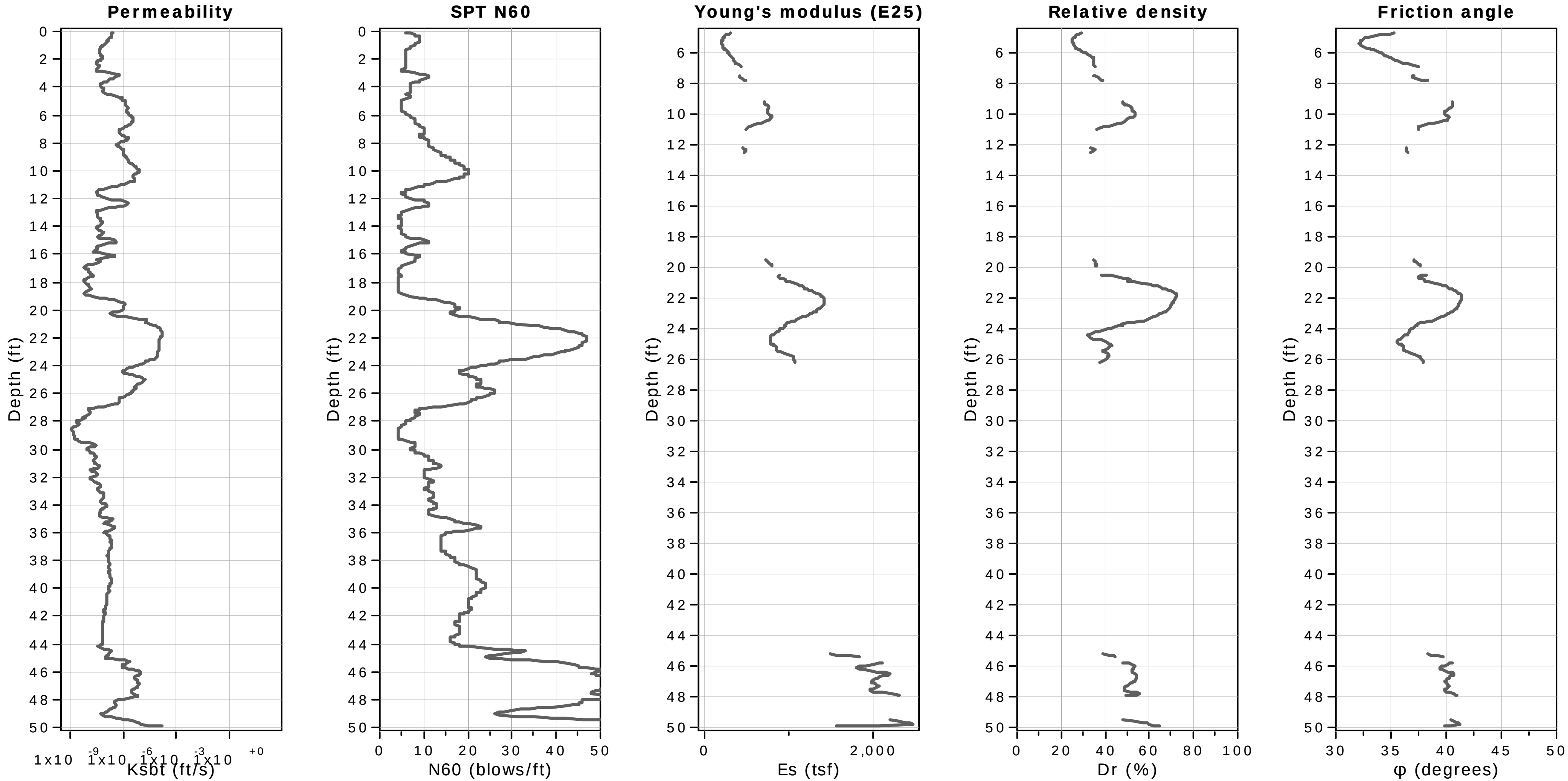
CPT: CPT-2
Total depth: 50.22 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: Adventure Park
Location: Whittier, California



SBTn legend

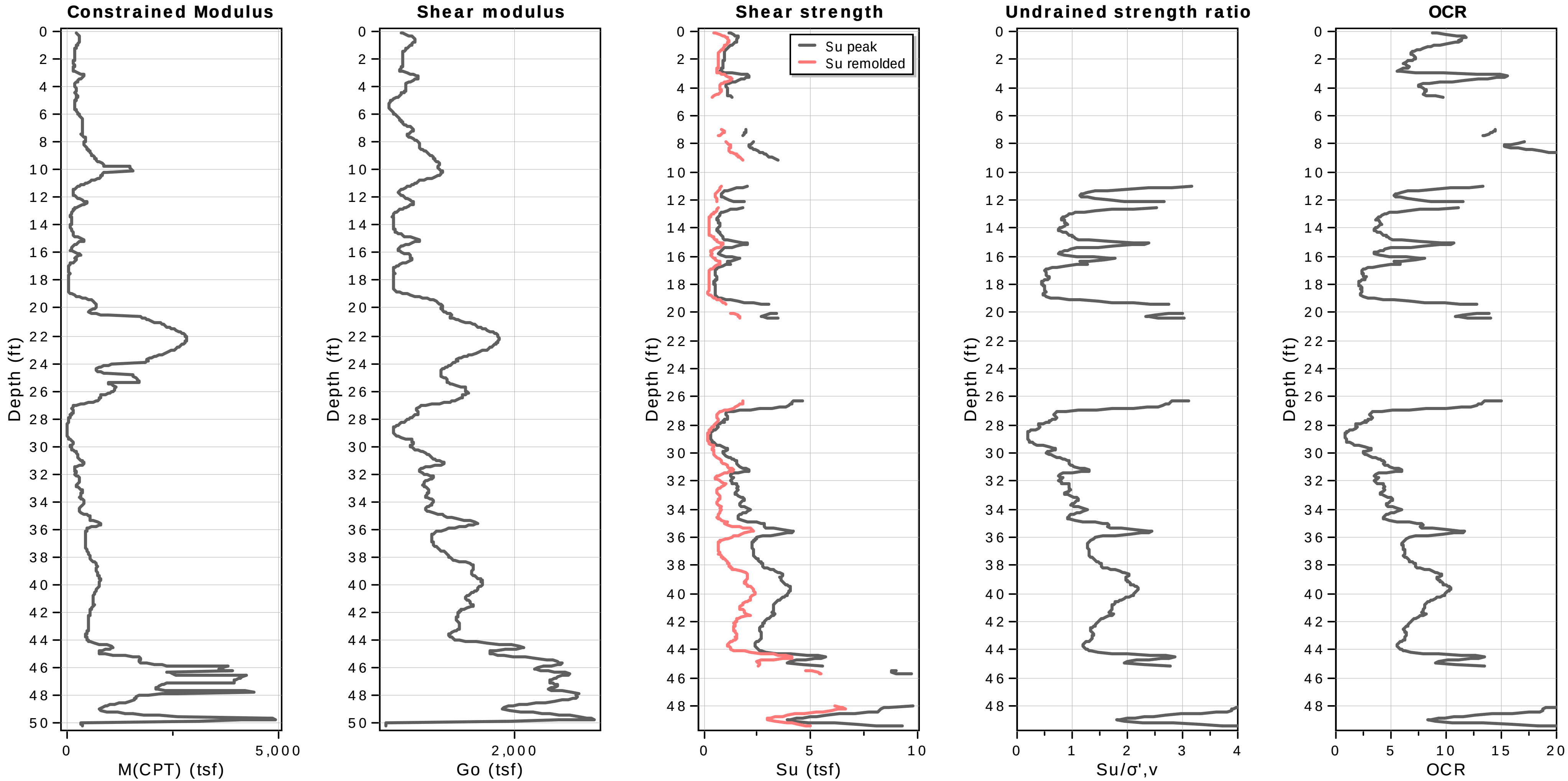
- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained



Calculation parameters

Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data



Calculation parameters

Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data

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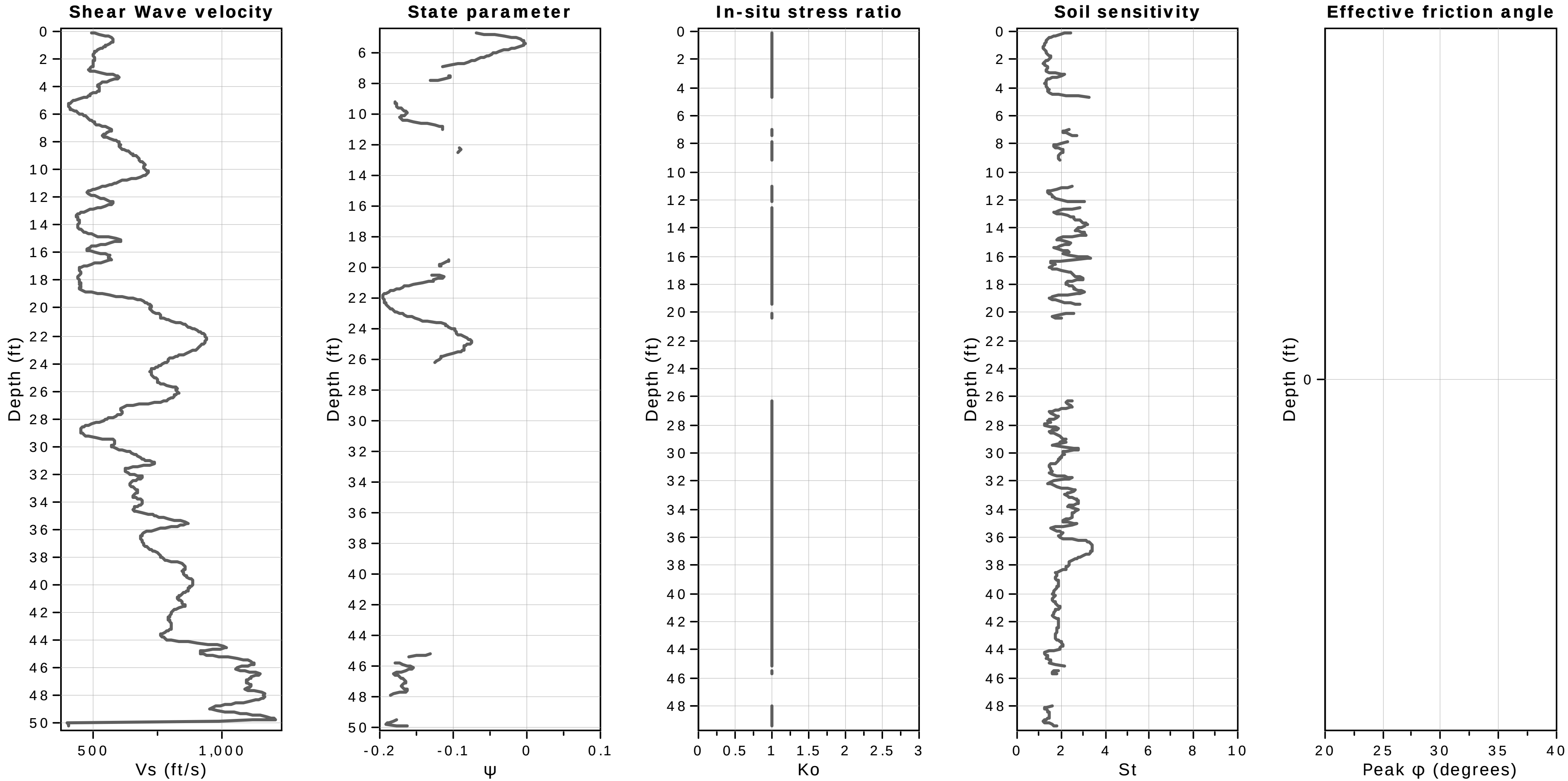
CPT: CPT-2
Total depth: 50.22 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



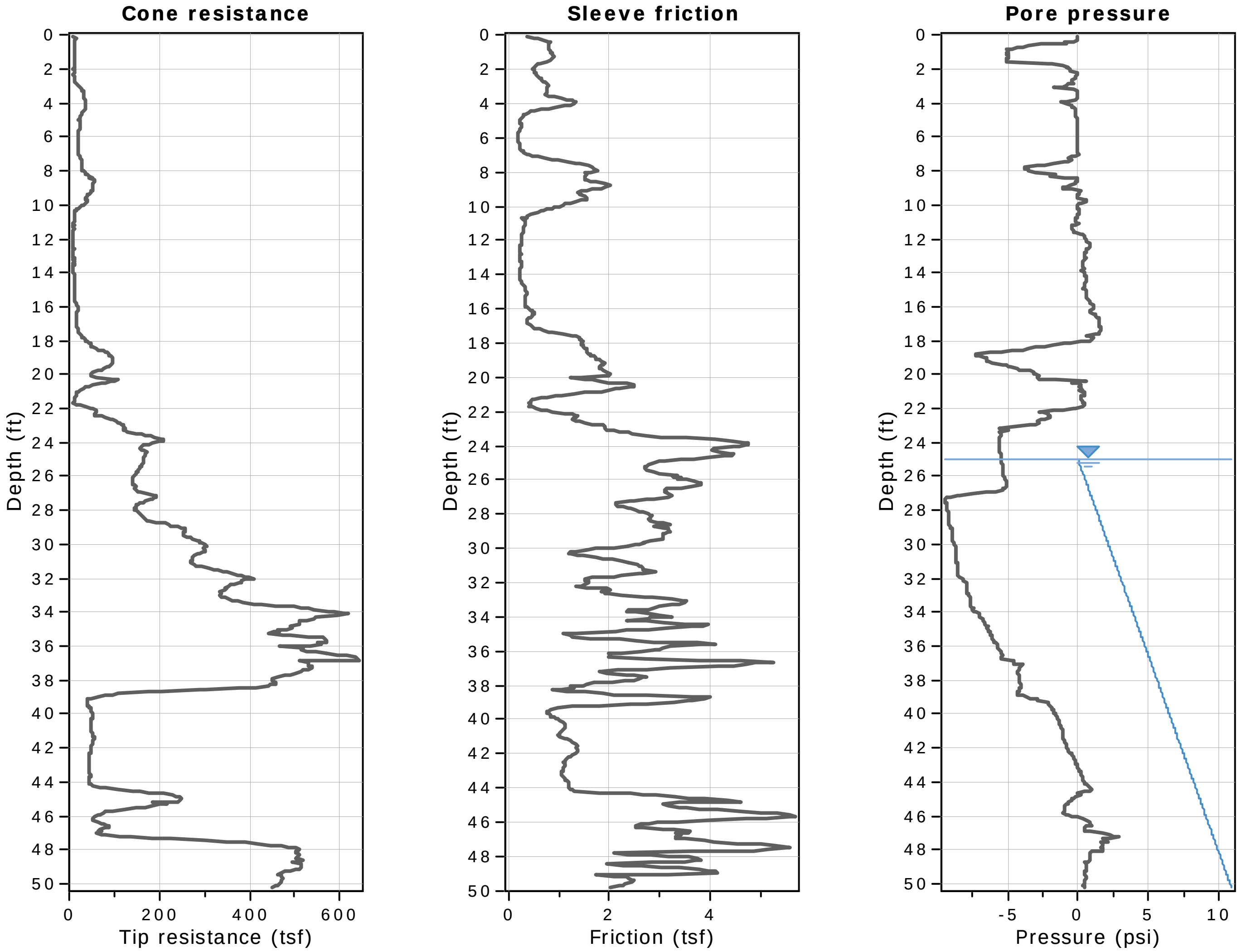
Calculation parameters

Soil Sensitivity factor, N_S: 7.00

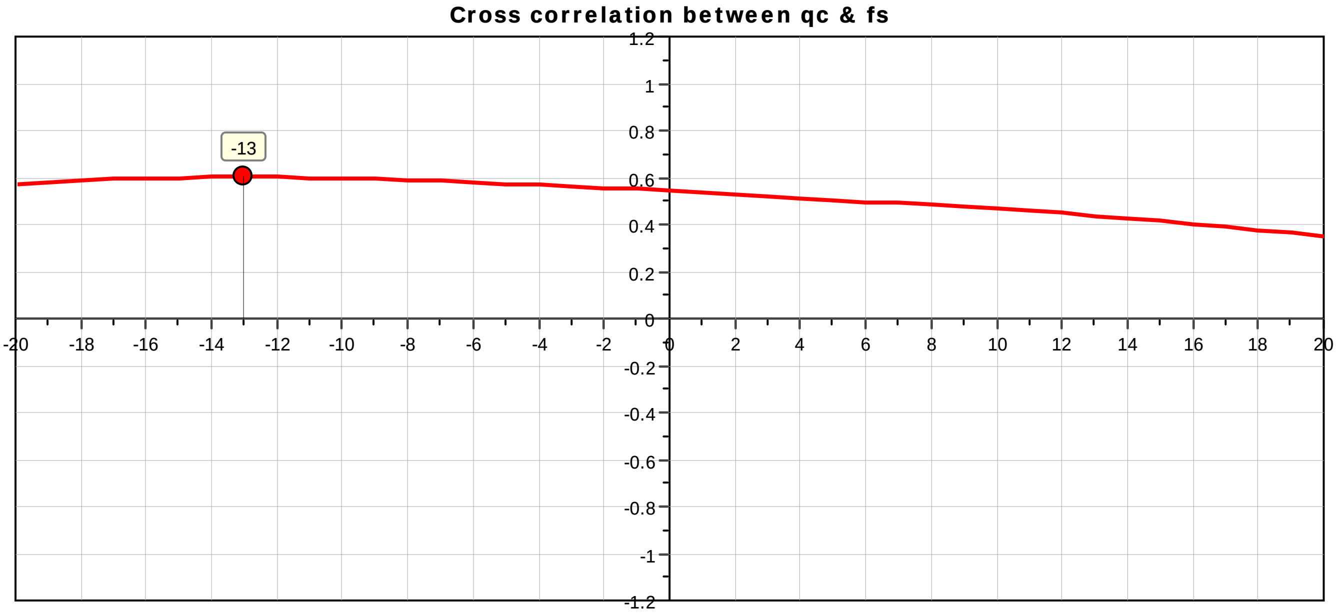
User defined estimation data

CPT: CPT-3
Total depth: 50.22 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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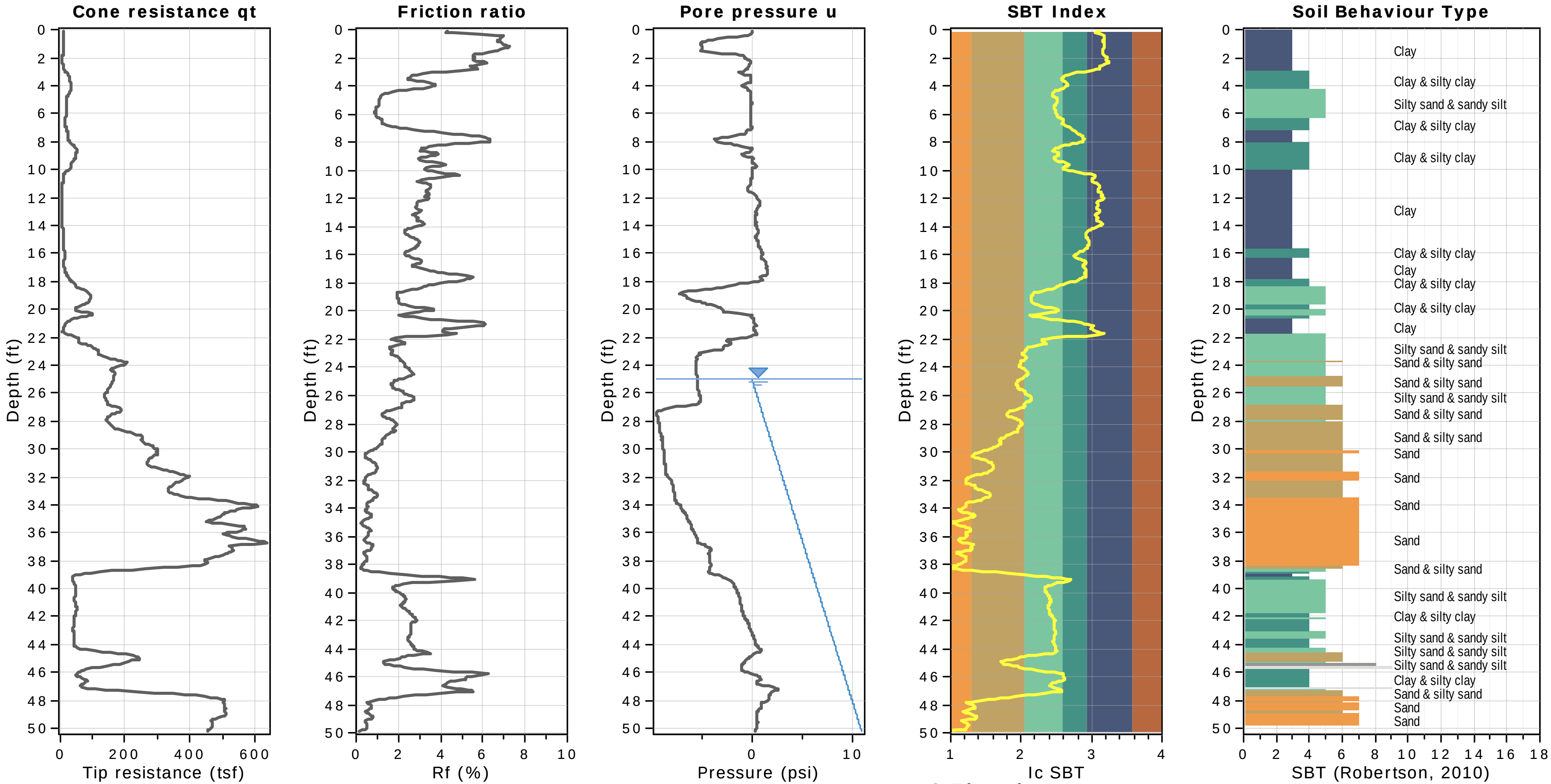
CPT: CPT-3
Total depth: 50.22 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: Adventure Park
Location: Whittier, California



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

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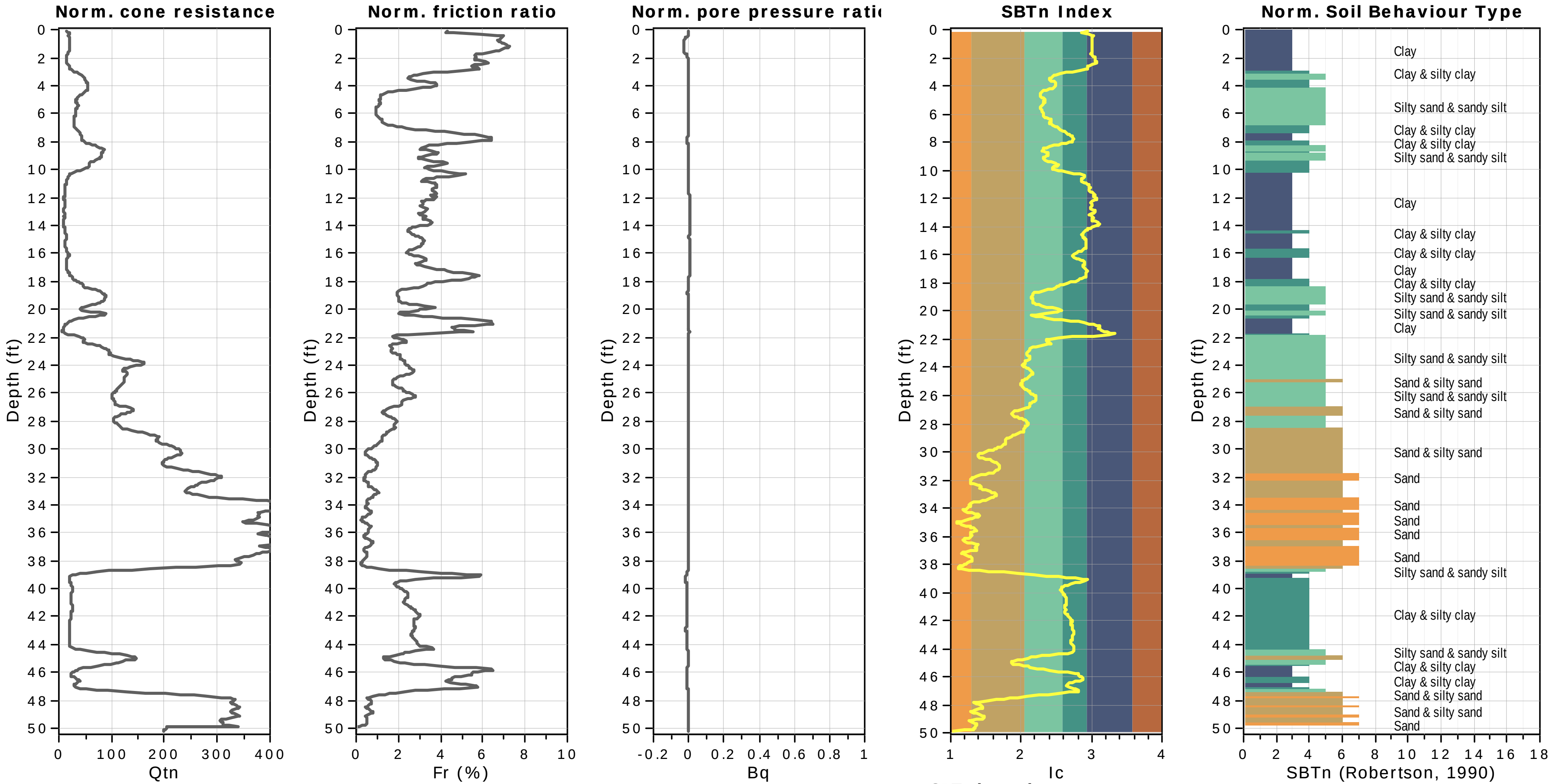
CPT: CPT-3
Total depth: 50.22 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: Adventure Park
Location: Whittier, California



SBTn legend

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6. Clean sand to silty sand
7. Gravely sand to sand
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9. Very stiff fine grained

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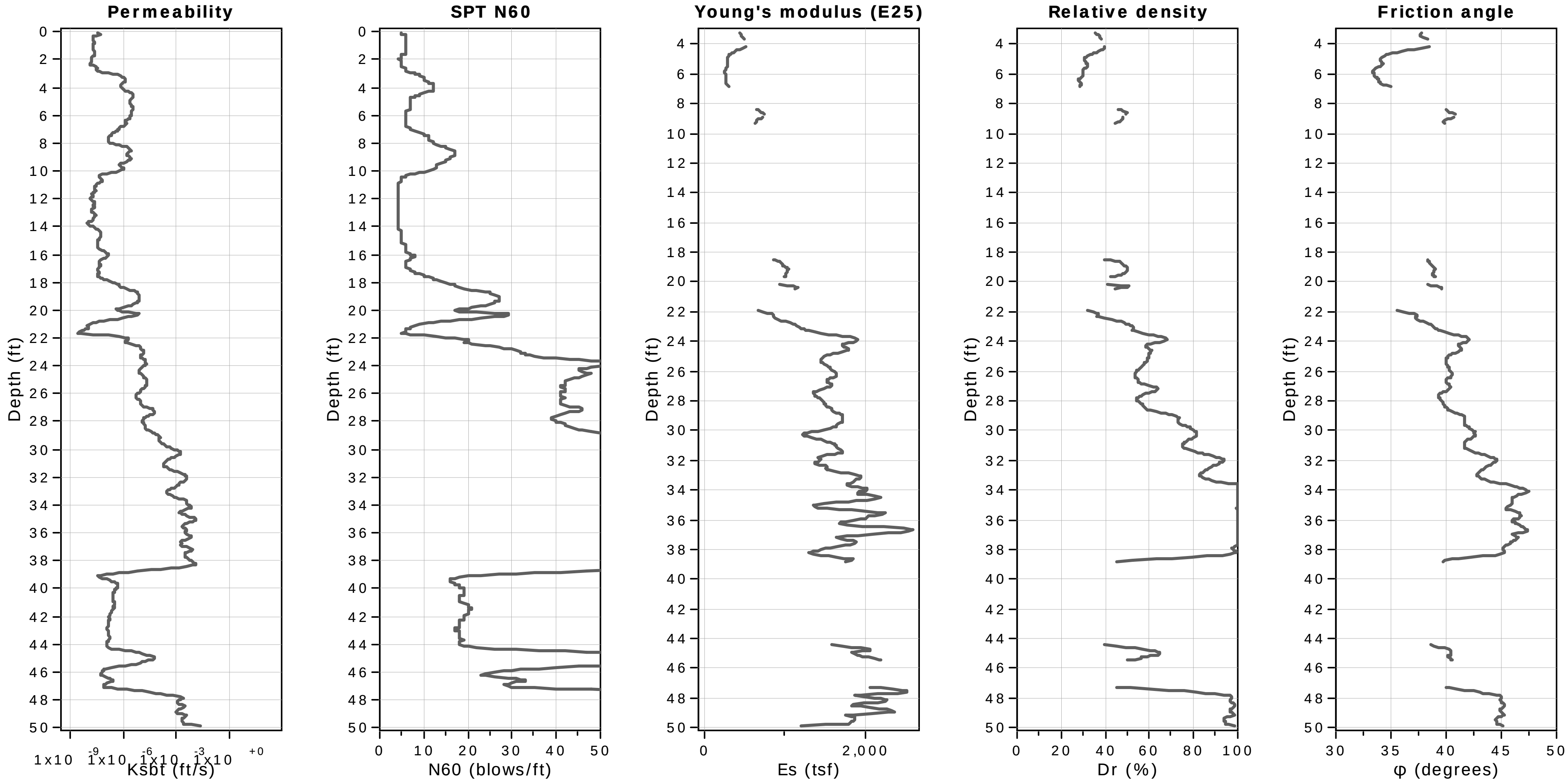
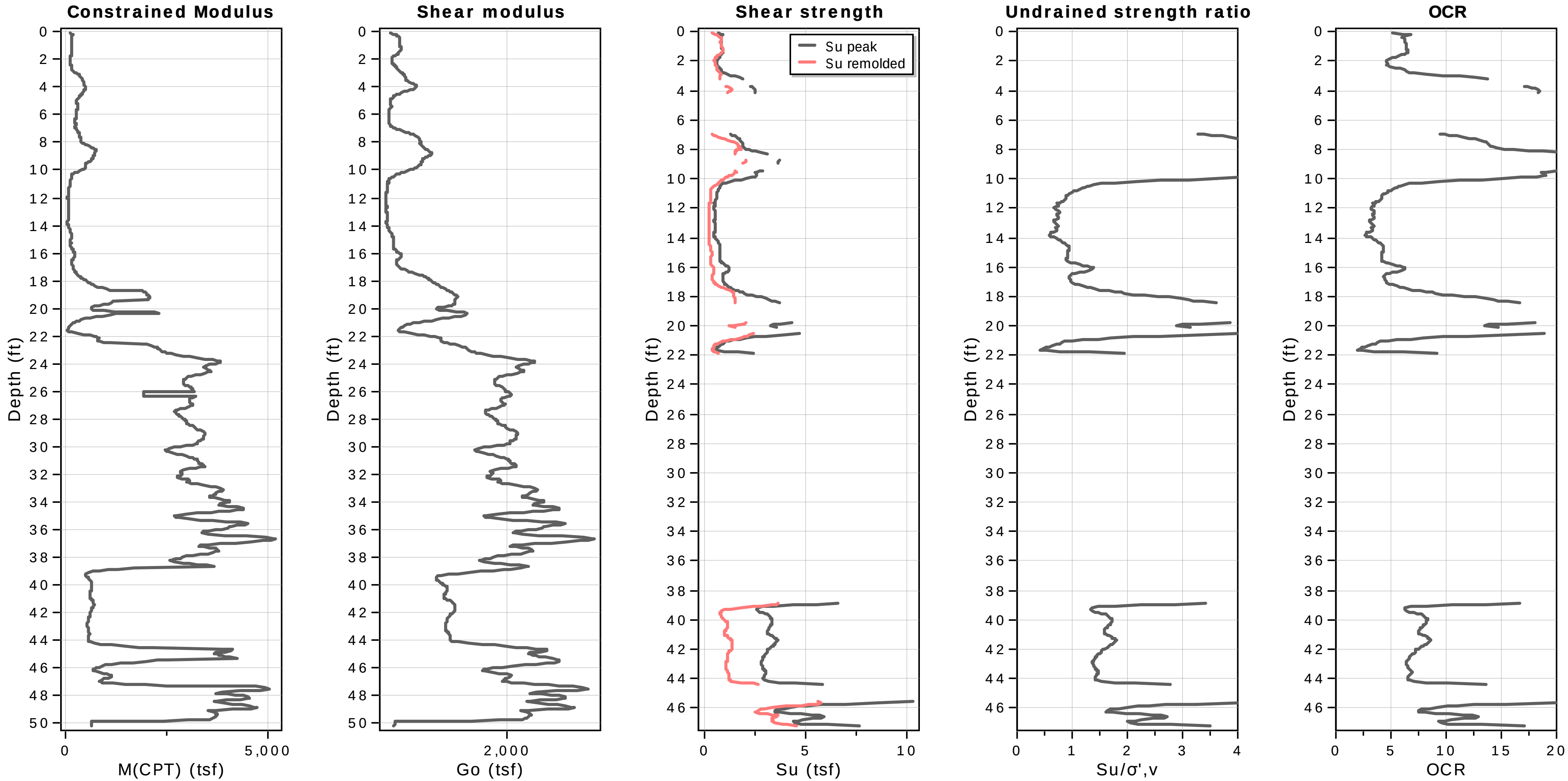
CPT: CPT-3
Total depth: 50.22 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



Calculation parameters
Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)
Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

Project: Adventure Park
Location: Whittier, California



Calculation parameters

Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data

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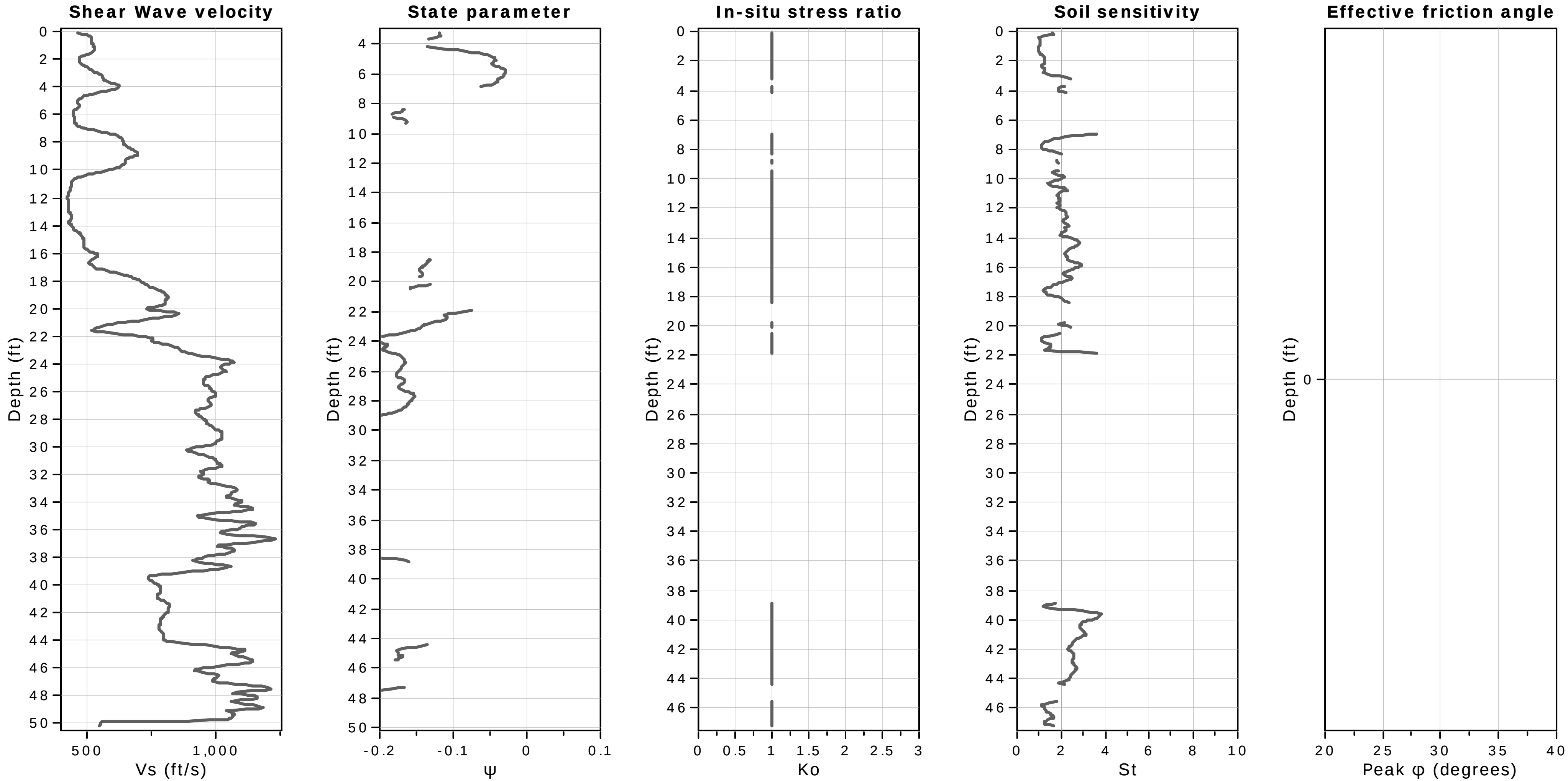
CPT: CPT-3
Total depth: 50.22 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



Calculation parameters

Soil Sensitivity factor, N_S: 7.00

User defined estimation data

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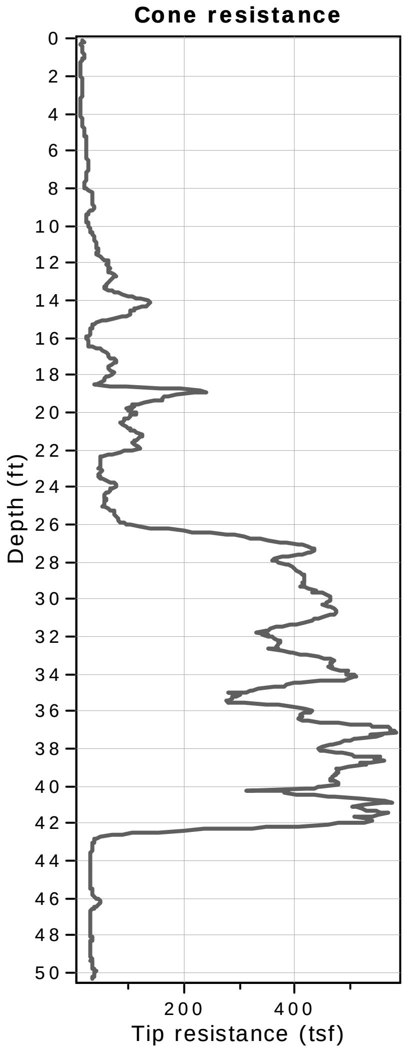
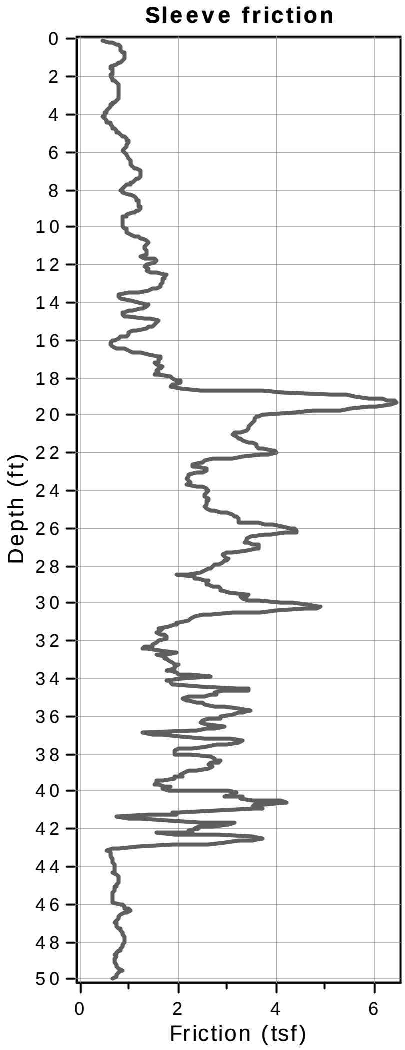
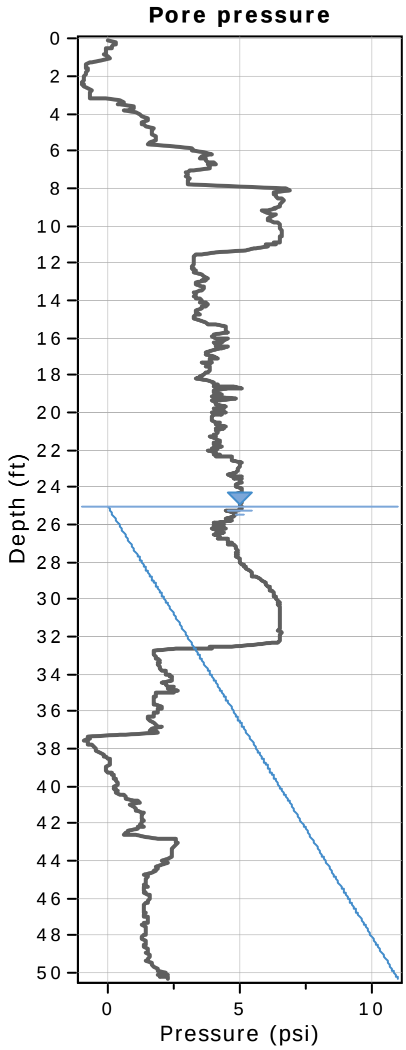
CPT: CPT-4
Total depth: 50.34 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

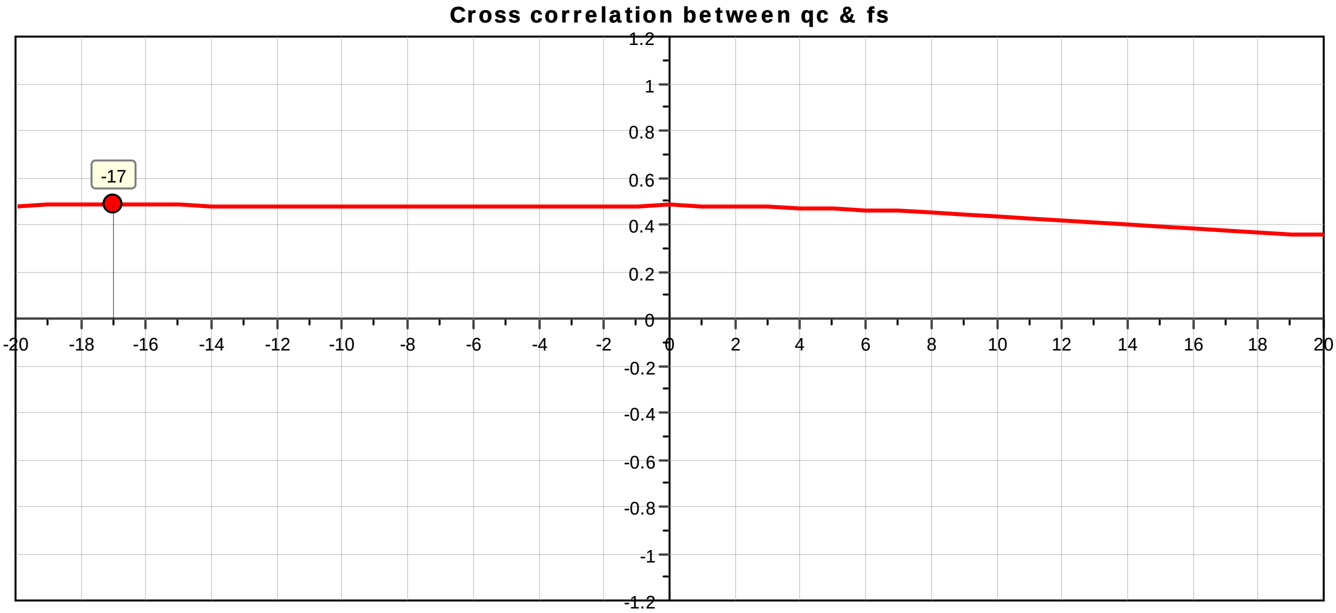
Project: Adventure Park
Location: Whittier, California







The plot below presents the cross correlation coeficient between the raw qc and fs values (as measured on the field). X axes presents the lag distance (one lag is the distance between two sucessive CPT measurements).



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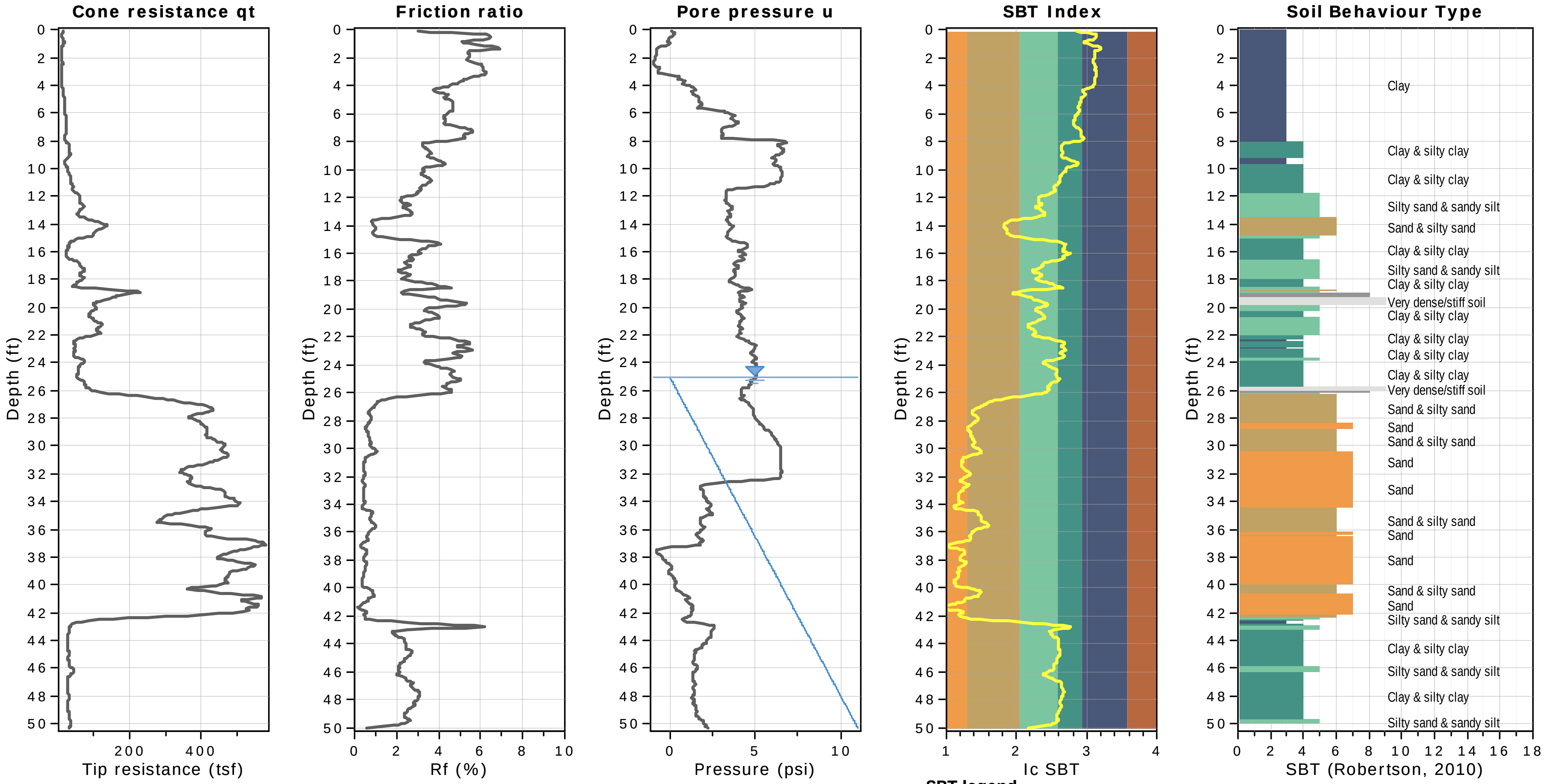
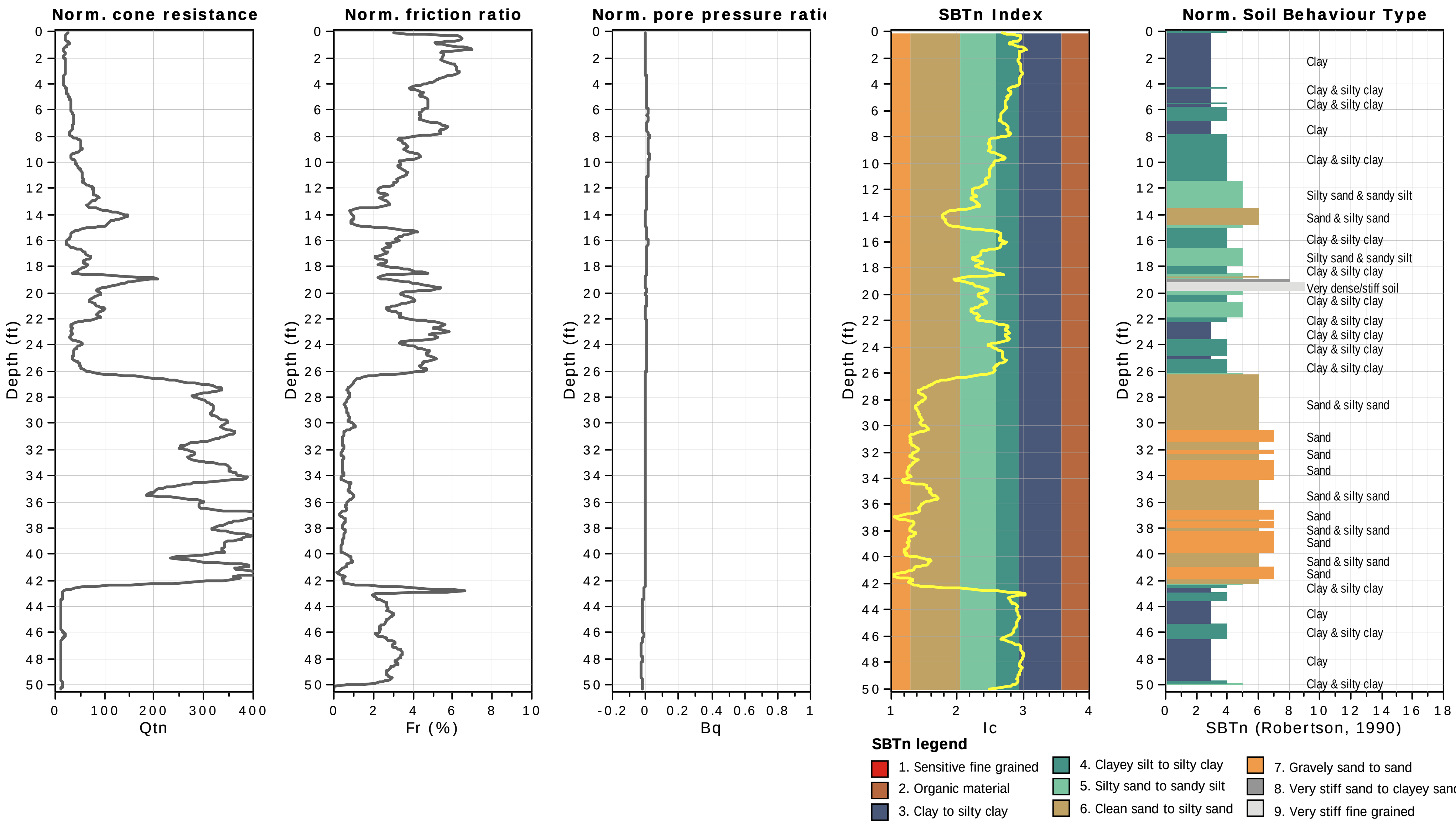
CPT: CPT-4
Total depth: 50.34 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: Adventure Park
Location: Whittier, California



SBT legend

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



Norm. cone resistance
Depth (ft)
Qtn
Norm. friction ratio
Fr (%)
Norm. pore pressure ratio
Bq
SBTn Index
Ic
Norm. Soil Behaviour Type
SBTn (Robertson, 1990)
Clay
Clay & silty clay
Silty sand & sandy silt
Sand & silty sand
Very dense/stiff soil
Sand
SBTn legend
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

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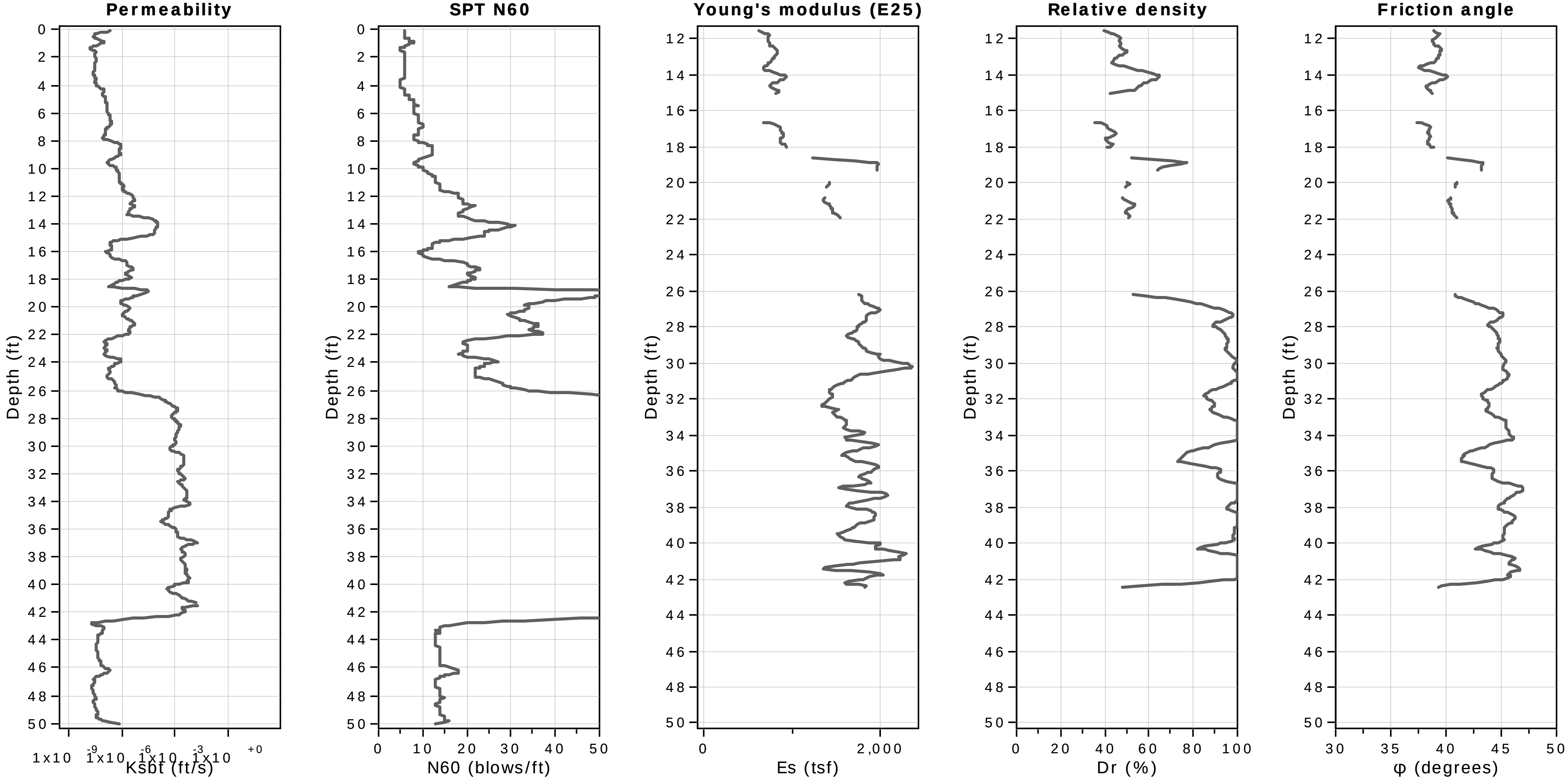
CPT: CPT-4
Total depth: 50.34 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



Calculation parameters

Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
User defined estimation data

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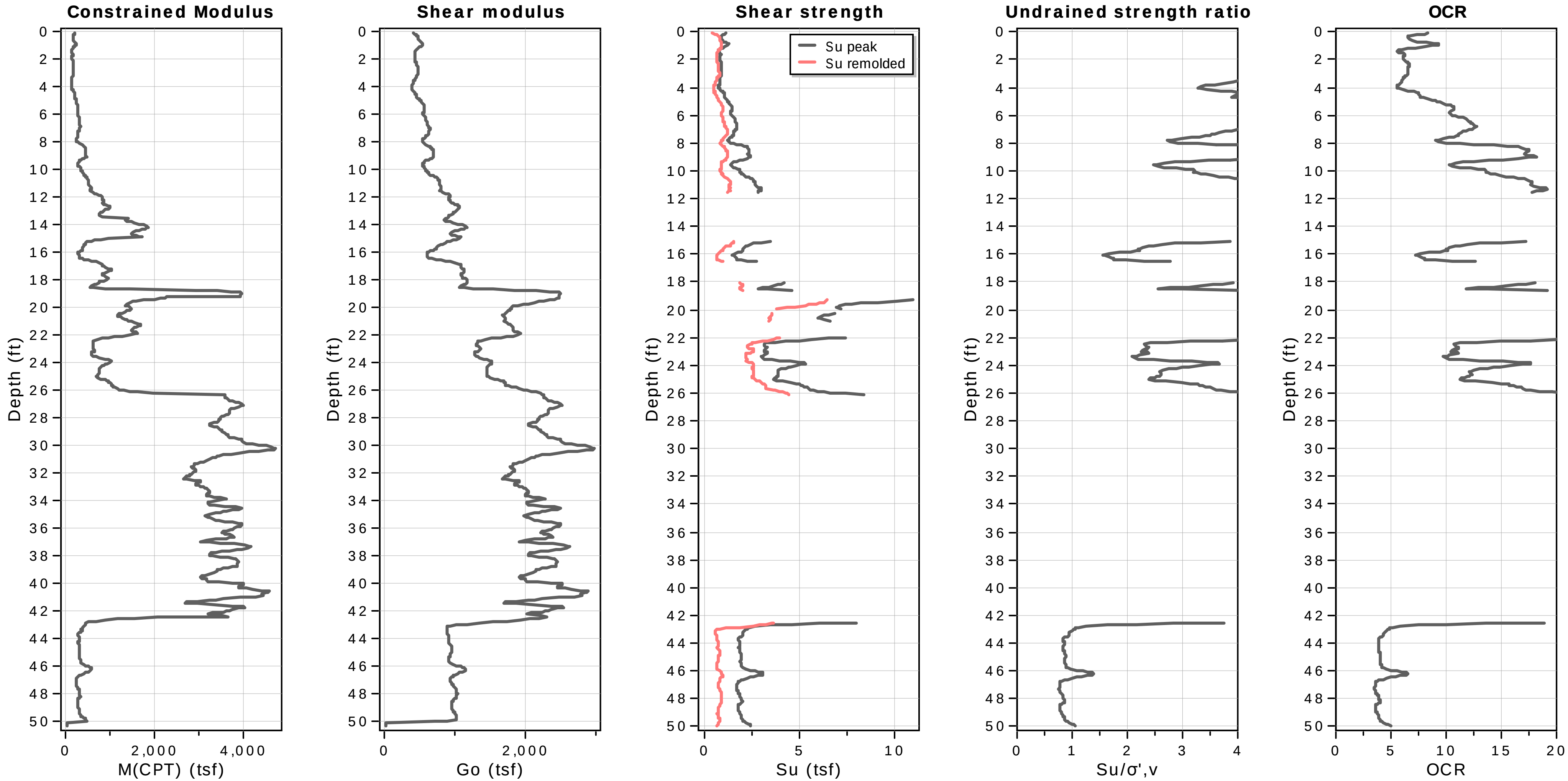
CPT: CPT-4
Total depth: 50.34 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: Adventure Park
Location: Whittier, California



Calculation parameters

Constrained modulus: Based on variable *alpha* using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable *alpha* using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

OCR factor for clays, N_{kt}: 0.33
User defined estimation data
Flat Dilatometer Test data

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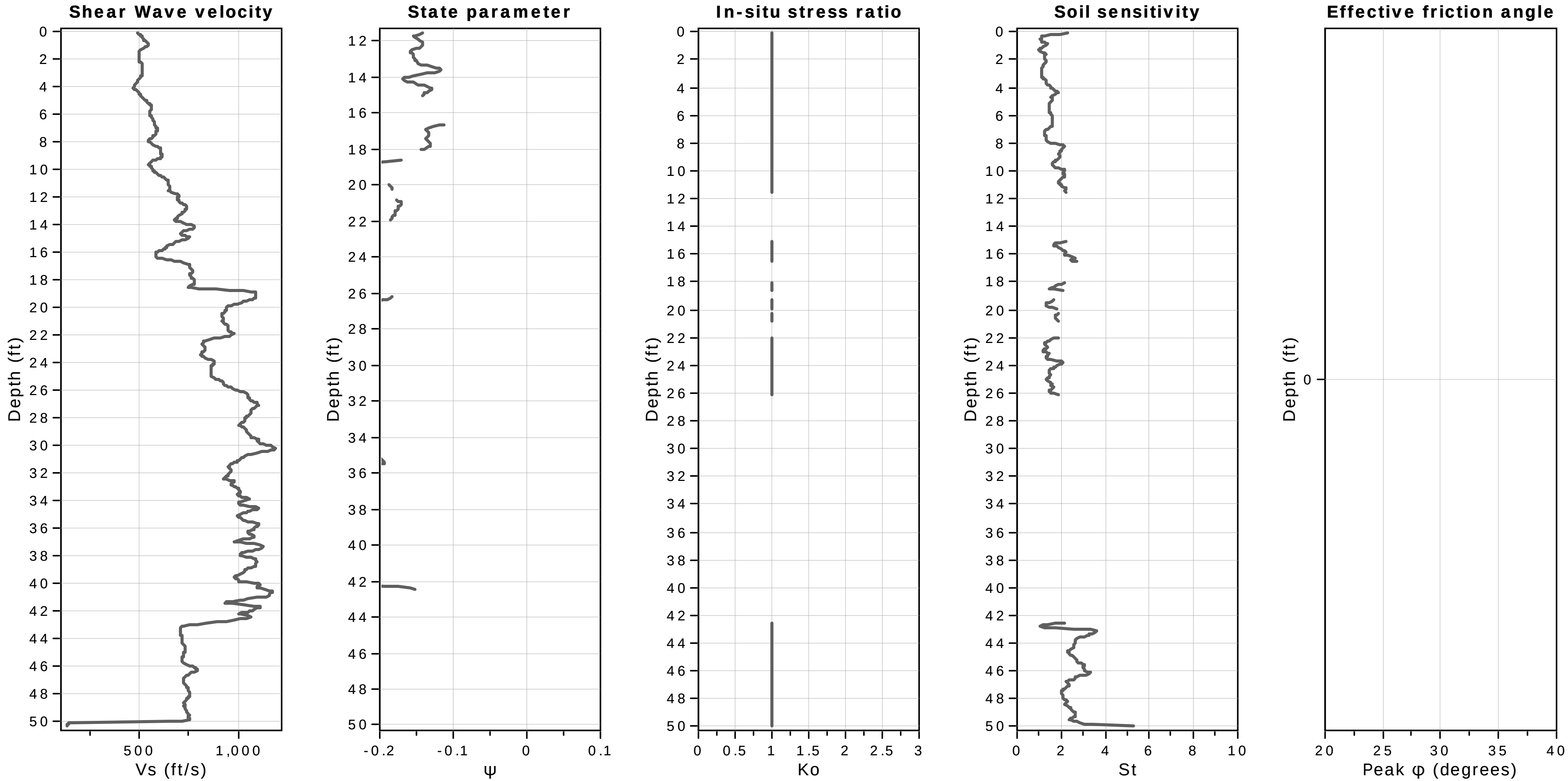
CPT: CPT-4
Total depth: 50.34 ft, Date: 4/15/2020
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **Adventure Park**
Location: **Whittier, California**



Calculation parameters

Soil Sensitivity factor, N_S: 7.00
User defined estimation data