GEOTECHNICAL REPORT

Avila Beach CSD WWTP Improvements
2859 Avila Beach Drive
San Luis Obispo County, California

Yeh Project No.: 221-020

Revised June 10, 2021

Prepared for:

Avila Beach Community Services District
PO Box 309
Avila Beach, CA 93424
Attn: Mr. Brad Hagemann, P.E.

Prepared by:

Yeh and Associates, Inc.
391 Front Street, Suite D
Grover Beach, California 93433
Phone: 805-481-9590

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Geotechnical • Geological • Construction Services

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391 Front Street, Suite D
Grover Beach, CA 93433
(805) 481-9590
www.yeh-eng.com

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Avila Beach Community Services District
PO Box 309
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Attn: Mr. Brad Hagemann, P.E.

Subject: **Geotechnical Report for Avila Beach CSD WWTP Improvements, 2859 Avila Beach Drive, San Luis Obispo County, California**

Dear Mr. Hagemann:

Yeh and Associates, Inc. is pleased to submit this revision to the geotechnical report for the design of the Avila Beach Community Services District's wastewater treatment plant at 2859 Avila Beach Drive. This updated report was prepared in accordance with our proposal for geotechnical services dated January 15, 2021. This report summarizes, seismic data for use with the current building code or AWWA standards, and recommendations for the design of structure foundations, pipeline trenches, stormwater control basin, retaining walls, and pavement.

Per our proposal dated January 15, 2021, the previous version of this report dated August 11, 2020, has been updated to include results of infiltration testing at a proposed stormwater basin and recommendations for ground improvements and pavement design for the proposed project. Our understanding of the current project is based on meeting with the design team on June 1, 2021, attended by Mr. Bryan Childress with the Wallace Group, the civil engineer for the project and Mr. Nathan White with Taylor and Syfan Consulting Engineers, the structural engineer for the project. A summary of key geotechnical information and considerations for the project are as follows:

- The project generally consists of the design of a new membrane bioreactor (MBR) system, equalization tank (EQ tank) and associated site grading, piping, a retaining wall, and other site improvements. Yeh reviewed previous geotechnical studies the District provided for the site and performed four cone penetration test (CPT) soundings to depths ranging from 75 to 100 feet below the site to supplement subsurface information from the previous geotechnical studies.

- The site is in a low-lying coastal area along San Luis Obispo Creek and a 1/2-mile inland from the beach. The site was graded and covered with a variable thickness of fill material during its original construction in 1969. Approximately 40 feet of relatively soft alluvium was encountered in the CPT soundings below the fill. Interbedded layers of medium dense sand and gravel and medium to very stiff clay were encountered below 40 feet to maximum depths explored.

Groundwater was encountered about 10 feet below the ground surface and near the elevation of San Luis Obispo Creek.

- The design earthquake estimated for the site is a M6.7 event with a peak ground acceleration of 0.52g. The soft alluvium encountered below the existing fill was relatively compressible and contained layers of potentially liquefiable sand. An interpreted subsurface profile summarizing the liquefaction analyses for the site is presented on Plate 2. The average bearing pressures below foundations for the proposed MBR and EQ Tank will be on the order of 1,500 pounds per square foot. We estimate that static settlement from the foundation loads could be up to 2 to 3 inches and could occur differentially across the foundations. The estimated seismic settlement for the design earthquake is approximately 0.5 to 1 inch in the vicinity of the MBR and EQ Tank based on an analysis of the CPT data. The estimated settlements exceed what the design team considered tolerable for the proposed structures. Ground improvements using rigid inclusions are recommended to reduce the potential for static and seismic settlements to within tolerable limits of 1-inch or less of total settlement.

- The site is also within a tsunami hazard zone that the California Geologic Survey reports has a runup elevation of about 60 feet near the site. Tsunamis hazards can be extreme but infrequent events. Hazard warning systems to warn coastal communities of those events when they happen based on monitoring the oceans for tsunamis following near or far away earthquakes. The San Luis Obispo County Office of Emergency Services manages a tsunami hazard warning system for Avila Beach and other coastal communities.

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

Sincerely,
YEH AND ASSOCIATES, INC.

JUDD J. KING
No. 2903

Judd King, GE
Senior Project Manager

JAMIE LYNN CRAVENS
C 91504

Jamie L. Cravens, PE
Project Engineer

Reviewed by:

JONATHAN D. BLANCHARD
No. 2312

Jonathan D. Blanchard, GE
Principal Geotechnical Engineer

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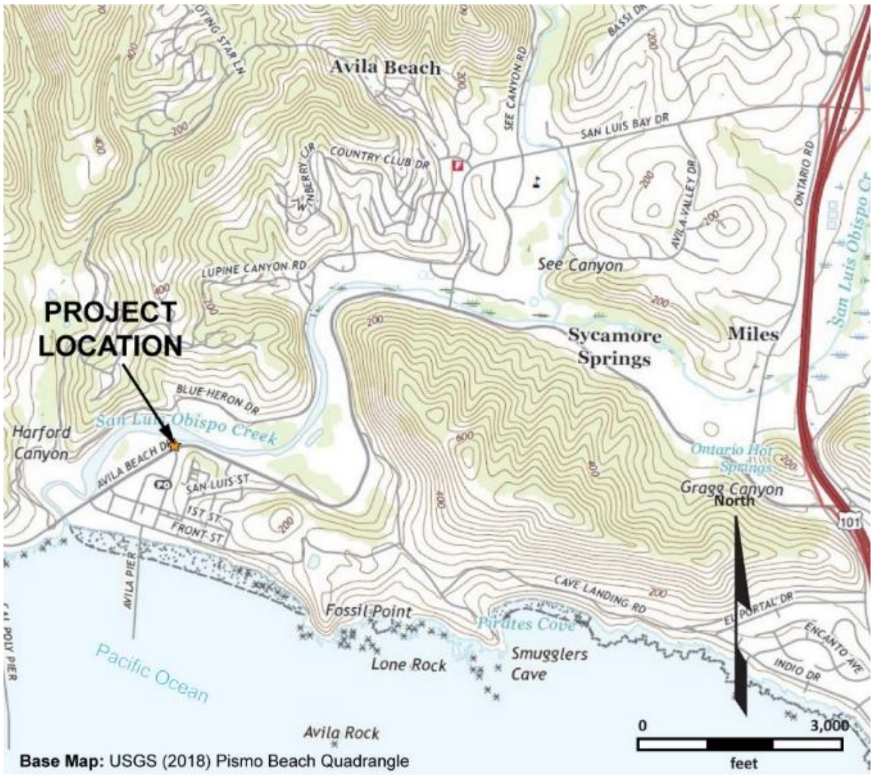
1. PURPOSE AND SCOPE OF STUDY

The Avila Beach Community Services District retained Yeh and Associates to provide geotechnical recommendations for the design of the improvements to the Avila Beach Community Services District's wastewater treatment plant at 2859 Avila Beach Drive in San Luis Obispo County, California. The location of the site is shown on Figure 1.



Base Map: USGS (2018) Pismo Beach Quadrangle

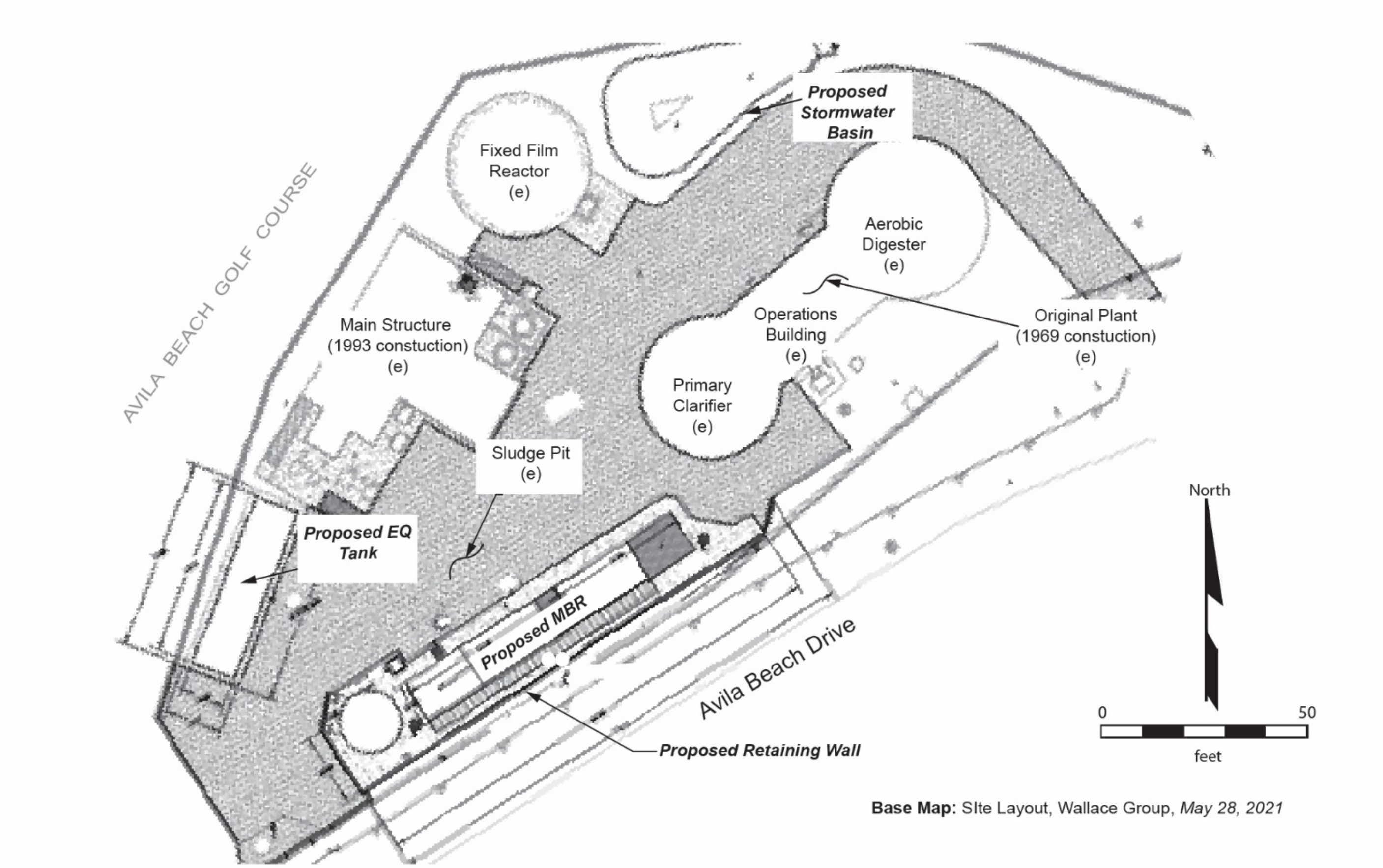
Figure 1: Vicinity Map

The geotechnical evaluation consisted of project coordination; review of existing historical photographs, previous geotechnical studies, and plans; field exploration; infiltration testing and engineering analyses as a basis for providing the recommendations in this report. This report provides seismic data for use with the current building code or AWWA standards, and recommendations for ground improvements, design of structure foundations, pipeline trenches, pavement and retaining walls. This report also updates a previous version prepared by Yeh and Associates (Yeh 2020) during the preliminary phase of the project.

2. PROJECT DESCRIPTION

2.1 EXISTING FACILITY

The District's existing wastewater treatment facility is in the coastal community of Avila Beach, California. The wastewater treatment plant is located on an approximately 0.6-acre site between Avila Beach Drive and San Luis Obispo Creek in Avila Beach, and services an average daily flow of about 0.06 million gallons per day (MGD). The proposed layout of the site and proposed improvements provided by Wallace Group (2021) is shown in Figure 2. The original plant was constructed in 1969. As-built plans (Pomeroy, Johnston & Bailey (PJB 1969)) show that the existing clarifier and digester are supported on driven timber piles that appear to extend to about 80 feet



Base Map: SIte Layout, Wallace Group, *May 28, 2021*

Figure 2: Site Plan

below the structures, and that the operations building is supported on spread footings bearing in compacted fill. The plant was expanded in 1993. The as-built plans (Kennedy Jenks 1993) show that the main structure and fixed film reactor are supported on driven precast concrete piles that appear to have been driven to about 60 feet below the structures (ESC 1992).

2.2 SITE DESCRIPTION

The site is on the north side of Avila Beach Drive, and approximately a ½ mile inland from the coastline and beach. The general vicinity is characterized as an alluvial valley bordered by steep ascending hillsides. The plant is bordered to the north by the Avila Beach Golf Course and the Bob Jones bike trail. The edge of San Luis Obispo Creek is approximately 50 feet northeast of the perimeter of the existing plant. The downtown area of Avila Beach is directly across Avila Beach Drive from the site. Grading for the existing plant (PJB 1969, KJ 1993) included placing approximately 3 to 4 feet of fill to raise the grade of the site to approximately elevation 10 feet above sea level. An approximately 5- to 8-foot-tall embankment along the westbound shoulder of Avila Beach Drive borders the southerly end of the existing plant. Site grades at the plant site vary from approximately 8 to 12 feet above sea level.

2.3 PROPOSED PROJECT

The proposed project generally includes the design of a new membrane bioreactor (MBR) unit, equalization tank, stormwater basin and associated grading, piping, and a new retaining wall. The locations of the proposed improvements are shown on the Site Plan in Figure 1. A summary of the proposed improvements follows:

- Two prefabricated MBR units and an aerated sludge tank will be mounted on a concrete mat slab foundation. The mat slab foundation will have a length of approximately 107 feet and a width of 20 feet. It will extend approximately 2.5 to 5 feet beyond the footprint of the MBR units and tank. These improvements will be in the area of the existing sludge pit in the southwest corner of the site. Approximately 3 feet of fill will be placed to fill the pit and match adjacent site grade. The MBR and sludge tank will range in weight from 158 to 220 kips when operational.
- An equalization tank (EQ Tank) will be designed above grade on a concrete mat slab foundation northwest of the proposed MBR unit. The mat foundation will be 43 feet long and 13 feet wide and will support the EQ Tank that has a capacity of 27,000 gallons and an operating weight of 240 kips.
- A concrete retaining wall will be provided along the southern side of the pad for the new MBR unit and tank to support a cut along the Avila Beach Drive embankment. The wall will be constructed upon the mat slab foundation. Yeh understands that the wall will be up to 6 feet high and extend up to 110 feet along the base of the existing slope.
- Various piping will be provided to convey wastewater to the new MBR unit and to discharge the filtered effluent and sludge. The pipes will be buried with a typical 3 or 4 feet of cover. Pipe diameters are expected to be less than 12 inches.
- Equipment pads used to for electrical controls, pumps, generators, or other light equipment will be designed. The pads are anticipated to be no more than 15 feet wide or long.
- A stormwater basin is planned along the north boundary of the site. The basin will have a depth up to 5 feet and will be used to capture and infiltrate stormwater from the site.
- The entry road and area around the existing plant facilities and new facilities will be paved with hot-mix asphalt concrete.

3. GEOTECHNICAL INVESTIGATION

The geotechnical investigation for this project included reviewing previous geotechnical data and as-built plans provided by the District, advancing four cone penetration test soundings at the site, hand excavating a hole in the bottom of the sludge pit, hand augers for infiltration testing and reviewing historical aerial photographs. The locations of various the explorations from the current and previous studies are shown on Figure 3. Data is included in Appendices A through C.

All boundaries and locations are approximate
Base Map: Google Earth (Imagery date September 7, 2018)

Figure 3: Field Exploration Plan

3.1 PREVIOUS STUDIES

Previous geotechnical data and as-built plans were provided by the District. The following provides a summary of the data that we reviewed.

- Pomeroy, Johnston, and Bailey (PJB 1969) prepared plans for the design of the original plant that included site grading, the existing digester and clarifier, the operation building between digester and clarifier, the outfall, and supporting geotechnical data. The plans show that the existing clarifier and digester were supported on driven timber piles. Sheet 16 of those plans (included in Appendix C) presented the results of three soil borings and one test pile. Two of the borings (B-1 and B-2 shown on Figure 3) were reportedly drilled near the digester to depths of 34 and 85 feet below the previous ground surface. Those borings were reportedly drilled by Central Coast Laboratories in 1967; however, details regarding the drilling and sampling were not included in the plan set. A third boring was drilled offsite along the outfall pipeline. The data for an 80-foot-deep driven test pile is also presented on Sheet 16 of the PJB plans, which is the basis for the assumption that the piles for the clarifier and digester are about 80 feet deep. The subsequent Earth Systems Consultants (ESC1992) report referenced a test pit, but those records were not included in the data reviewed.

- Kennedy Jenks (KJ 1993) prepared plans and Earth Systems Consultants (ESC 1992) prepared a Geotechnical Report for the design of the existing package plant and fixed film reactor. The plans show that these improvements were supported on 16-inch square precast concrete piles. The Earth Systems report recommended the piles be driven to depth of 60 feet (or elevation -51 feet). A log for one hollow stem auger boring and supporting laboratory data were provided in the Earth Systems report. A copy of those data is included in Appendix C.

3.2 CONE PENETRATION TEST (CPT) SOUNDINGS

The CPT subcontractor for this project was Gregg Drilling and Testing, Inc. of Signal Hill, California. Gregg advanced four soundings to depths ranging from 75 to 100 feet below the ground surface using a hydraulic ram mounted inside a 30-ton truck on July 24, 2019. CPT were performed in general accordance with ASTM D-5778 using an electric piezocone penetrometer. The piezocone had a diameter of approximately 1.7 inches, a tip area of 15 square centimeters (cm^2), and a sleeve area of 225 cm^2. Cone tip resistance (q_c), sleeve friction (f_s), and penetration pore water pressures measured from a transducer placed behind the tip (in the u2 location) were recorded at approximately 3-centimeter intervals during penetration using an on-board computer. The friction ratio (FR, the ratio of the sleeve friction to the tip resistance in percent) was computed for each value of q_c and f_s recorded. The data and soil behavior type classifications were used in subsequent geotechnical analyses and to evaluate soil types and boundaries for analyses. Upon removal of the CPT rod, the soil generally collapsed to near the groundwater level encountered. The void above that depth was filled with bentonite chips. A report from Gregg and logs of the CPT soundings are presented in Appendix A.

3.3 HAND EXCAVATIONS

A hand excavation was made in the bottom of the sludge pit using a post-hole digger and a ½-inch diameter t-probe on July 24, 2019. The excavation was advanced to a depth of four feet. Three holes were excavated to depths of 3.25 to 5 feet below the ground surface in the proposed stormwater basin area using a hand auger for infiltration tests on March 12, 2021. Logs for the borings are presented in Appendix B.

3.4 INFILTRATION TESTS

Yeh performed Shallow Quick Infiltration Tests in borings 21I-01, 21I-02, and 21I-03 in accordance with the Central Coast Low Impact Development Initiative (2013) testing methodology. Yeh performed the tests on March 15, 2021. Following excavation by hand using a hand auger, one to four inches of gravel (3/8-inch in dimension) was placed on the bottom of the borings, a 2-inch diameter perforated pipe was then placed into the 6-inch diameter hole, the annulus around the pipe was filled with gravel, and then the borings were presoaked for 24 hours prior to testing.

The initial phase of testing consisted of adding a measured volume of water to the borings to maintain a constant head for 30 minutes. The second phase of testing consisted of measuring the rate of water level fall (i.e., falling head) for a minimum of 2 hours or when all the water within the boring drained away at least twice so that up to three sets of readings were taken. A 100-gallon tank was used for the source of water for testing. The results of infiltration tests are presented in Appendix B.

3.5 HISTORIC AERIAL PHOTOGRAPHS

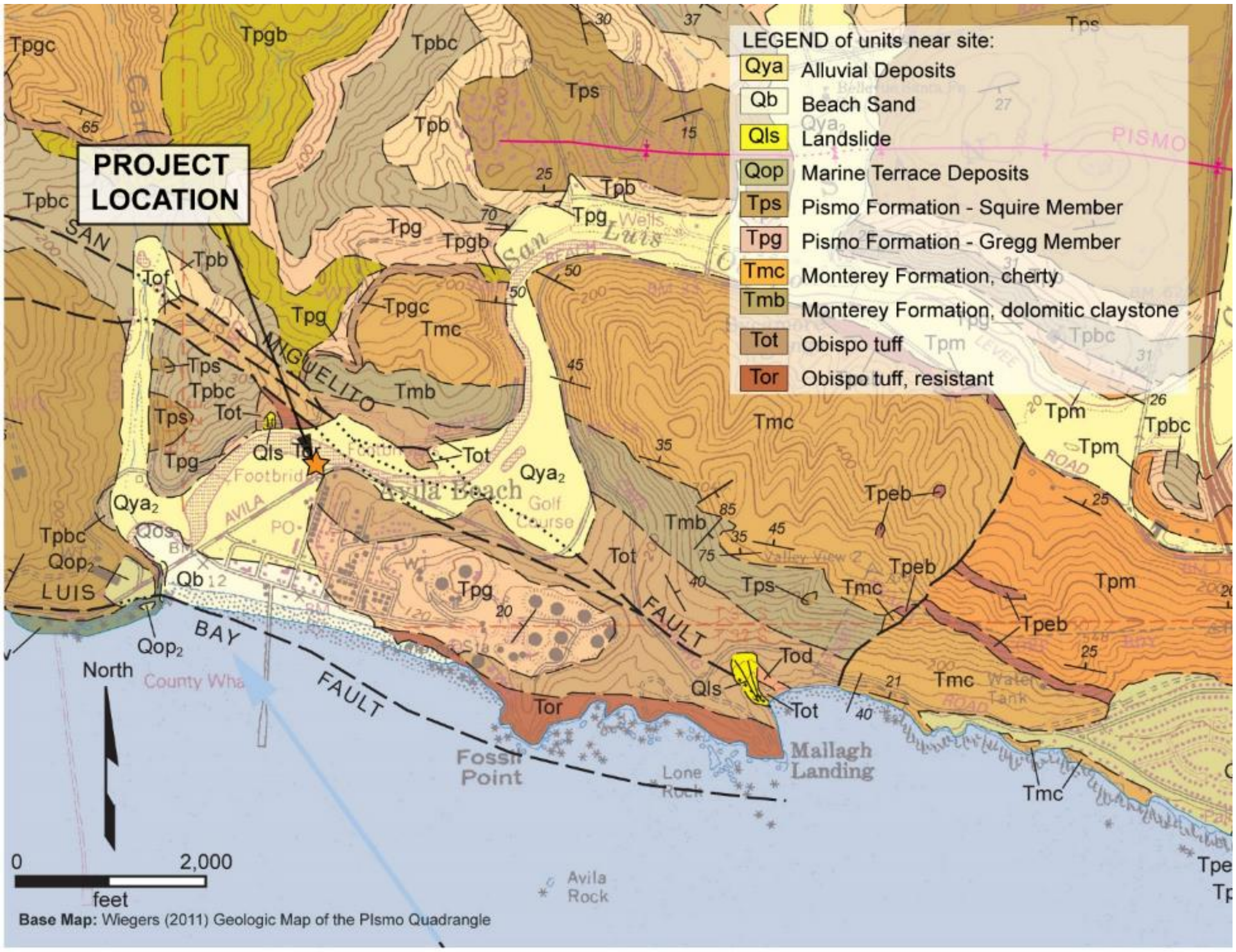
Historic aerial photos obtained from Environmental Data Resources (EDR 2019) and the University of California Santa Barbara (UCSB 2019) Map and Imagery Library were reviewed for the site. EDR provided photos for the years of 2016, 2012, 2009, 2006, 1994, 1981, 1976, 1963, 1960, 1956, and 1949. An additional photo from 1940 was obtained from UCSB Map and Imagery Library. A copy of the photos and EDR report are included in Appendix D. A summary of the site conditions observed in the photos follows:

- The site was occupied by a single small building in the 1940 photo. The railroad ran past the site near the existing alignment of Avila Beach Drive. The edge of an estuary that underlies the northern end of present-day community of Avila Beach was about 100 feet south of the treatment plant site. Avila Beach was developed along Front Street and generally east of San Miguel Drive. The south bank of San Luis Obispo Creek appears to be located near its current location, approximately 100 feet north of the plant site.
- The 1949 photo shows that the small building on the site was removed, and there were several new buildings located along the south bank of San Luis Obispo Creek just north of the site. Further development had occurred along front street and one block to the north. The estuary is partially filled from the south between the railroad and Avila Beach. The railroad bridge across San Luis Obispo Creek appeared to be removed, however, the bridge piers were evident.
- The 1956 photo shows that the estuary east of the railroad was filled in. Several new buildings are present along the north end of present-day San Miguel Street.
- The site vicinity looks similar until 1976 when the original plant (constructed in 1969) is evident. The estuary southwest of the site was channelized in the 1960 and 1963 photos and was filled in by the 1976 photo. Avila Beach Drive appears to have been constructed on its current day layout. The creek was bridged (by a pipeline likely) north of the plant. The structures along San Luis Obispo Creek north of the site had all been removed. The site vicinity looked similar through the 1994 photo (the KJ plant improvements were not observed in the 1994 photo).

- The 1981 photo showed that Avila Beach was built out west of San Miguel Street. The areas around the plant between Avila Beach Drive and San Luis Obispo Creek are undeveloped.
- The 2006 and subsequent photos showed the existing plant and surrounding golf course similar to the site conditions that are present today.

4. GEOLOGIC SETTING

The regional geology in the site vicinity as mapped by Wiegers (2011) is shown on Figure 4. The project is located within the Coast Ranges geologic and geomorphic province, which extends from the Transverse Ranges in southern California to the Klamath Mountains in northern California and into Oregon. The province is characterized by north-northwest trending mountain ranges composed of sedimentary, volcanic, and metamorphic formations. The formations are comprised of predominantly Jurassic and Cretaceous age rocks with Tertiary to Quaternary age rocks and soil commonly overlying the older formations along the flanks and foothills of those ranges.



Base Map: Wiegers (2011) Geologic Map of the Pismo Quadrangle

Figure 4: Geologic Map

Geologic structure mapped in the site vicinity consist of west to northwest trending faults and folds. The predominant structure includes the San Miguelito fault mapped just upstream of the site (see

Figure 4) at a bend in San Luis Obispo Creek, and the San Luis Bay fault mapped along the shoreline south of the site. These faults are included as a seismic source within the overall San Luis Range fault system.

Avila Beach is located on an alluvial plane near the mouth of San Luis Obispo Creek. The surface geology at the site is mapped as younger alluvium (Qya). The alluvium is comprised of sediments that are likely a combination of material deposited by San Luis Obispo Creek in the estuary areas along the creek. The creek cuts through hillsides immediately upstream of the site. The hills are predominantly mapped as bedrock units composed of Tertiary-age Pismo (Tp), volcanic tuff and mudstone of the Obispo Formation (To), and Monterey (Tm) formations. These rocks are exposed on hillsides above the north side of San Luis Obispo Creek opposite of the treatment plant and are exposed in outcrops along Avila Beach Drive only about 150 feet northeast of the existing treatment plant. The CPT soundings did not encounter bedrock to 100 feet below the site. Previous borings by Central Coast Labs (PJB 1969) reported encountering shale bedrock at approximately 70 feet below the site at the existing digester location.

5. SUBSURFACE CONDITIONS

5.1 GEOLOGIC UNITS

The subsurface conditions encountered in the CPT soundings and previous borings consisted of three predominant units: existing artificial fill (Af), young alluvium (Qya), and Obispo Formation (Tot) shale bedrock. An interpreted profile summarizing the subsurface conditions encountered is shown on Plate 1. The units shown on Plate 1 were differentiated based on geology, engineering properties relating to strength and compressibility, and classification. Descriptions of the units encountered and shown on the profile are summarized below. Logs of the CPT soundings and borings are provided in Appendices A, B, and C.

Artificial Fill (Af). As-built plans show that 3 to 4 feet of compacted fill was to be placed above the previous ground surface to construct the plant in 1969. Artificial fill also includes existing pavement, utility trench backfill, and structure backfill that were specifically encountered in the explorations. The artificial fill generally consisted of relatively dense silty to clayey sand and very stiff sandy clay with varying amounts of gravel. The artificial fill was underlain by alluvium.

Alluvium (Qya). Alluvium was encountered below the artificial fill at depths of about 3 to 5 feet below the existing ground surface. The alluvium was composed of three general units: Qya1, Qya2, and Qya3 (as shown on Plate 1). A description of those units follows:

Qya_1. This unit was encountered below the artificial fill to depths of approximately 10 feet below the existing ground surface. The unit is predominantly stiff clay and sandy clay that is above groundwater or within the level of fluctuating groundwater. The clay is relatively stiff and overconsolidated.

Qya_2. This unit was encountered immediately below Qya_1 to depths of approximately 40 feet below the existing ground surface. This unit is a bay mud type deposit interbedded within alluvial sediment that was developed in the back-bay areas of Avila Beach and deposited by San Luis Obispo Creek. The unit was predominantly composed of soft to medium stiff clay and silty clay with interbedded layers of sand. The clay was relatively soft, normally to lightly overconsolidated, and is considered compressible.

A layer of loose to medium dense sand running through the upper third of the Qya2 unit was encountered in each of the CPT soundings at depths of approximately 15 feet. The sand ranged in thickness from approximately 2 feet in C-1 and C-2 advanced near the proposed MBR to approximately 8 feet in C-3 and C-4 performed on the north side of the original plant. The sand unit is shown on Plate 1 and was interpreted to extend to the bed of San Luis Obispo Creek north of the site for the slope stability and liquefaction analyses presented in this report.

Qya_3. This unit was encountered below Qya_2 to the maximum depths of the CPT soundings, up to 100 feet below the existing ground surface. The unit consisted of relatively thick interbedded layers of medium dense to dense silty sand and stiff to very stiff clay. The layers encountered were both continuous and discontinuous between explorations, ranged in thickness from less than 1 foot to up to 8 feet, and varied in classification and consistency. Thinner lenses of sand and gravel were also encountered in some explorations. The unit was underlain by bedrock at a depth of 70 feet below the existing digester in a single boring (B-1 on Plate 1) drilled by Central Coast Laboratories in 1968 (PJB 1969).

Obispo Formation (Tot). The bedrock encountered in the Central Coast Laboratories in 1967 boring was encountered below the alluvium from a depth of 70 feet to the maximum depth of the boring, 85 feet below the ground surface. The rock was described as “dense gray SHALE” on the boring log. The rock is most likely associated with the Obispo Formation, based on an outcrop located about 150 feet northeast of the boring location and the geologic map (Figure 4). The outcrop exposed by a road cut on Avila Beach Drive is composed of relatively massive, moderately fractured tuff with blasting scars. Mapped units of the Obispo Formation locally contain mudstone (Wiegers 2011).

5.2 GROUNDWATER

The site is between San Luis Obispo Creek and the tidal estuary shown in 1940 aerial photos of the site. The area is known for shallow groundwater and to be prone to flooding. Groundwater was encountered in CPT soundings at depths of approximately 12 to 15 feet below the existing ground surface based on pore pressure response and dissipation tests. The groundwater level is near the water elevation in San Luis Obispo Creek. Groundwater was reportedly encountered at about 1.5 to 4 feet below previous site grades in 1967 borings drilled by Central Coast Laboratories (PJB 1969). Earth Systems Consultants (ESC 1992) encountered groundwater at a depth of approximately 10 feet below the ground surface near the main plant in a boring drilled in April 1992. Groundwater was not encountered in the recent hand auger borings excavated by Yeh in 2021. Groundwater and soil moisture conditions at the site are likely influenced by local and coastal flooding, stormwater runoff, and flows in San Luis Obispo Creek.

6. GEOTECHNICAL EVALUATION

6.1 SEISMICITY

Seismic data was estimated for the site as input to the liquefaction analysis and for design of the new plant structures. The recommended seismic data are presented for use with California Building Code or AWWA design standards in Section 7.2 of this report. The seismic data were estimated for a soft soil, Type E, site. The peak ground acceleration was estimated at 0.52g from ASCE 7-10 using the web tool application developed by SEAOC (USGS 2019a). The corresponding earthquake magnitude was estimated using the USGS online Unified Hazard Tool (USGS 2019b) to estimate the mean magnitude for a design earthquake having a 2 percent chance of being exceeded in 50 years. The design earthquake had an estimated mean magnitude (M) 6.7. The earthquake is mostly controlled by contributions from the Los Osos, Hosgri, and San Luis Range fault systems.

6.2 TSUNAMI HAZARD ZONE

The site and the town of Avila Beach are within the Tsunami Hazard Zone identified by the building code via the ASCE Tsunami Design Geodatabase Version 2016-1.0 (ASCE 2010). The County of San Luis Obispo Office of Emergency Services manages the tsunami warning and evacuation system for the area. Hazard recognition and participating in the emergency planning system should be considered by the District (if not previously considered).

6.3 LIQUEFACTION

The results of the CPT were used to evaluate the liquefaction potential of the soil encountered using the computer program CLiq by GeoLogismiki (Version 2.2.0.28). Liquefaction is the loss of soil strength due to an increase in soil porewater pressure resulting from seismic ground shaking. Liquefaction typically occurs in loose to medium dense granular soil that is below the groundwater table. The extent and severity of liquefaction is dependent upon the intensity and duration of the strong ground motion. Liquefaction can be manifested as sand boils, loss in soil strength and bearing capacity, seismically induced settlement, slope instability and lateral spreading.

The liquefaction potential of the foundation support soil was evaluated for the design earthquake using CPT data and NCEER screening criteria (Youd et al. 2001) processed within the CLiq program (GeoLogismiki 2006). Layers of sand encountered within units Qya2 and Qya3 and shown on Plate 1 are considered potentially liquefiable. Zones of subsurface material that have a potential for liquefaction are predominantly below depths of 30 to 50 feet and are presented on Plate 2.

Design Earthquake. Liquefaction is likely to be manifested as seismic settlement in response to the design earthquake and slope instability associated with lateral spreading (as discussed in the following section of this report). The settlement is estimated to range from approximately 0.5 to 1 inch at C-1 and 2 performed near the sludge pit, where the MBR unit and EQ Tank will be located. The thickness and frequency of potentially liquefiable sand layers was more common in C-3 and C-4 as shown on Plate 2. The seismic settlement is estimated to range from approximately 3 to 6 inches at C-3 and C-4 in the northern half of the existing facility. Recommendations to consider total and differential settlement in the design of the MBR unit and EQ Tank foundations are provided later in this report.

2003 San Simeon Earthquake. Plate 2 includes a comparison of the results of the liquefaction analyses for the design earthquake to liquefaction analyses for the 2003 M6.5 San Simeon Earthquake. Although the San Simeon Earthquake was a similar magnitude to the design earthquake, it occurred approximately 40 miles north of the site (Holzer et al 2004) and resulted in an estimated 0.15g peak ground acceleration at the site. The estimated ground acceleration for the San Simeon Earthquake was 3 to 4 times less than the peak ground acceleration estimated for the design earthquake. The analysis showed liquefaction potentially occurred in selected layers of the interpreted subsurface profile; however, the intensity of the shaking was likely not enough to result to the more widespread liquefaction estimated for the design earthquake. The District has not reported that there was evidence of liquefaction or lateral spreading observed at the plant following the 2003earthquake.

6.4 SLOPE STABILITY AND LATERAL SPREADING

Slope stability analyses were performed to evaluate the effects of post-liquefaction lateral spreading at the site and to check the feasibility of excavating a temporary slope between the edge of the new MBR and Avila Beach Drive. Liquefaction could result in slope instability or lateral spreading of the banks of San Luis Obispo Creek that could then impact the existing plant. The slopes were analyzed using the computer program SLIDE 2018 (Rocscience 2018).

6.4.1 INPUT AND ANALYSIS

For use with *SLIDE*, the user defines the surface and subsurface profile boundaries, groundwater conditions, the type of analysis to be performed, the layout and strength of any slope reinforcement, boundary loads, and the unit weight and strength of the soil and rock materials included in the analysis. The groundwater conditions modeled for existing slopes were based on groundwater levels encountered in the CPT soundings extrapolated to the water level in San Luis Obispo Creek. The cross-section geometry used for modeling the lateral spread condition was based on the interpreted subsurface profile shown on Plate 1. The cross-section geometry used for modeling the temporary slope was based on the MBR and retaining wall layout shown in Figure 2, and a section estimated from the topography and grading recommended in this report.

Lateral Spreading Criteria. Slope stability criteria used in the analysis were generally consistent with those defined by the California Geologic Survey Special Publication 117A (CGS 2008). The criteria consider that there is a potential for yielding and slope instability (lateral spreading) to occur when the estimated factor of safety for the slope being modeled is less than 1.0 considering the residual strength of the liquefied soil and the load for the design earthquake. The earthquake load is considered as an equivalent horizontal static force estimated using a pseudostatic coefficient (k_h) for the design earthquake. A value of k_h=0.17 was estimated from charts in SP117a for the design earthquake and considering a lateral displacement of 6 inches. The factor of safety of the slope was estimated using SLIDE and residual shear strength parameters estimated from CLiq. The analysis is considered complete if the estimated factor of safety from the pseudostatic analysis is 1.0 or greater. An estimate of the slope displacement was performed because the initial screening analysis produced an estimated factor of safety less than 1.0.

The estimate of slope displacement was made for the design seismic event using the Bray and Travasarou (2007) and Youd et al. (2002) procedures. The procedure used a simplified Newmark-type model and semiempirical predictive relationship to estimate the permanent slope displacement due to earthquake-induced shear. A yield coefficient (k_y) of 0.18 estimated for the slope from SLIDE represents the equivalent horizontal static force beyond which slope movement may occur. The coefficient is an input parameter for the calculation of estimated horizontal ground displacements.

Temporary Slope Criteria. The analysis was performed to check whether a temporary slope excavated below the property line along Avila Beach Drive would be feasible or would be vulnerable to slope instability associated with soft clay foundation soil encountered in the CPTs. The analysis used undrained shear strength parameters estimated from CPT data for soon-after-construction conditions and to consider the potential for the soft clay foundation to impact the stability of the temporary slope.

6.4.2 RESULTS

The results of the slope stability analyses for lateral spreading during the design seismic event and the temporary slope during construction are summarized below.

Lateral Spreading. Seismic displacement that could be associated with lateral spreading along the banks of San Luis Obispo Creek was estimated using a yield coefficient (k_y) of 0.18. The analysis from SLIDE showed that instability occurring from potential liquefaction of loose sand layers shown within the profile on Plate 2 could extend into or near the plant and is generally constrained by thinning of the upper sand layer encountered in unit Qya_2 (see Plate 1) away from the streambank. The estimated horizontal ground displacement is approximately 4 to 12 inches at the north end of the existing plant when considering the M6.7 design earthquake but did not extend into the area of the proposed improvements. The M6.5 San Simeon Earthquake had ground accelerations 3 to 4 times lower than the design earthquake and would not have been expected to cause yielding or lateral spreading that would impact the site. Potential impacts from lateral spreading could be addressed by soft fixes including emergency response plans for replacing pipes, shut off valves, and temporary bypasses for the plant.

Temporary Slope. The estimated factor of safety for the temporary slope conditions considered was approximately 1.2. The Caltrans (2011) Trenching and Shoring manual suggests a minimum factor of safety of 1.25 when considering the potential for deep-seated slope failures. Slope stability criteria for design of temporary slopes and shoring systems vary in practice and are the responsibility of the contractor per OSHA guidelines. The contractor should submit a shoring and excavation plan for the proposed excavation and or shoring of the embankment along Avila Beach Drive as part of the MBR construction.

6.5 SETTLEMENT

There is a potential for the weight of the MBR and EQ Tank to result in consolidation of an approximately 40-foot-thick layer of soft to medium stiff clay (Qya_2 unit on Plate 1). The estimated static settlement resulting from proposed project layout and loading configurations of the MBR and EQ Tank foundations is approximately 1.5 to 3 inches with a potential for half of the estimated settlement to occur differentially across those structures. The combined estimated static and seismic settlement exceeds the tolerable limits defined by the design team for the proposed structures that would allow them to function properly and maintain the needed hydraulic profile for efficient operation. A program of ground improvements is recommended in this report to reduce the estimated settlement to within the tolerable limits for the structures. Ground improvements including deep foundations, deep soil mixing, vibro-stone columns, and rigid inclusions were discussed with the design team. The team concluded the most suitable method for ground improvements would be to use rigid inclusions.

A rigid inclusion is defined as a column that is constructed within the ground by inserting a mandrel or auger to a specified depth, pumping concrete or grout into the ground via tremie as the mandrel or auger is removed, and repeating this process to form a grid pattern of rigid inclusions through the soft of liquefiable soil below the foundation areas. A 2- to 3-foot-thick load transfer platform (LTP) composed of a geosynthetic reinforced granular fill should then be provided over the tops of the inclusions to reduce the potential for concentrated pressures on the mat slab foundations that will support the new MBR and EQ Tanks. The recommended rigid inclusions coupled with the LTP should improve the stiffness of the ground and reduce estimated settlement to within tolerable limits and improve bearing capacity.

7. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on our understanding of the project as currently planned, and for use with the latest approved edition of the California Building Code (2019) and American Water Works Association Standards (2014).

7.1 EARTHWORK – GENERAL

7.1.1 SUGGESTED MATERIAL SPECIFICATIONS

The following specifications are suggested for materials referenced in various sections of this report. “Standard Specifications” refers to the 2018 edition of the *Standard Specifications* published by the California Department of Transportation (Caltrans 2018). Alternative specifications or materials should be reviewed by Yeh before being used on this project. Recommendations for material use are provided in subsequent sections of this report.

Aggregate Base. Aggregate base shall consist of imported aggregate that complies with the grading and quality requirements for ¾-inch Class 2 aggregate base per Section 26-1.02B of the Standard Specifications (minimum R-value = 78).

Compacted Fill. Compacted fill material to be placed in foundation areas shall consist of on-site soil or similar imported sandy material free of organic, oversize rock (greater than 3 inches), trash, debris, corrosive, and other deleterious materials. Imported fill shall have an expansion index less than 20 and at least 70 percent material passing the U.S. Standard No. 4 sieve. Imported fill placed with 3 feet of finished grade below pavement areas shall have an R-value of no less than 40.

Gravel Bedding Aggregate for gravel drains/gravel bedding for stabilization shall consist of imported gravel or crushed rock that is free of clay, organics, corrosive material, trash, debris, recycled or reclaimed material, and other deleterious substances. The gravel shall have a durability index of at least 40 when tested according to ASTM D3744. The gradation of the gravel shall conform to ASTM C-33 Number 4 aggregate (1 ½ inch x ¾ inch). Gravel shall be fully encased in geotextile fabric to provide separation.

Geocomposite Drains. Geocomposite drains for use in draining retaining walls shall conform to Section 96-1.02C of the Standard Specifications.

Geotextile for Separation (Filter Fabric). Geotextiles for filtration shall consist of Class C filter fabric conforming to Section 96-1.02B of the Standard Specifications.

Geotextile for Stabilization. Stabilization geotextile material shall consist of woven geosynthetic fabric. Geotextile for stabilization placed below crushed rock, on a soft subgrade or below rock fill shall comply with Subgrade Enhancement Geotextile in Section 96-1.02O of the Standard Specifications. Overlaps between adjacent rolls of geotextile shall be at least 2-feet wide or be spliced per the manufacture's recommendations. Geotextile shall be placed such that the fabric on the upstream or upslope side of the overlap is on top. Rocks, protrusions, or sharp objects that could potentially damage the geotextile shall be removed from the subgrade prior to placing the fabric. Depressions or holes left in the subgrade from the removal of obstructions shall be filled with sand. Geotextile shall be placed smooth without wrinkles and be secured by anchoring, pinning, placing aggregate, or anchoring in trenches as needed to maintain the integrity and location of the fabric when subsequent aggregates or fill is placed. Placement, anchorage, and construction methods shall comply with the manufacturer's recommendations.

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

Import. Material sources shall be approved by the Engineer before being brought to the site. Fill, backfill, and aggregates shall comply with all specified material requirements for the project.

Pipe Bedding/Pipe Zone Material. Pipe bedding and pipe zone material shall consist of imported sand free of clay, organics, corrosive material, trash, debris, and other deleterious materials. The sand shall have 100 percent material passing the 3/8-inch sieve, no less than 90 percent material passing the U.S. Standard No. Sieve, and no more than 3 percent passing the No. 200 sieve.

Structure Backfill. Imported structural backfill shall be non-expansive material having an Expansion Index of less than 20 when tested according to latest approved edition of ASTM D4829 and conform to Section 19-3.02C, "Structure Backfill" of the *Standard Specifications*. Material types include SC, SM, SP, SW per ASTM D2487. Fill and borrow sources shall be reviewed and approved by the geotechnical professional before being imported to the site.

Slurry Cement Backfill. Slurry cement backfill can be used as Trench Backfill or as Pipe Zone Material when approved by the Engineer. Slurry cement shall consist of 2-sack sand-cement slurry conforming to Section 19-3.02G of the Caltrans Standard Specifications. Aggregate shall be imported sand conforming to the gradation and quality requirements of the Standard Specifications. Slurry cement backfill shall be a stable flowable mix and shall be consolidated using vibration during placement. Subsequent backfill or compacted material shall not be placed above slurry cement backfill until the slurry cement can support foot-traffic without more than ¼-inch indentation. The Contractor shall provide ballast or stabilize the pipe as necessary to prevent movement or floating of the pipe during placement.

Trench Backfill. Trench backfill shall consist of onsite soil conforming to Compacted Fill or imported sand conforming to Pipe Bedding.

7.1.2 CLEARING AND GRUBBING

Clearing and grubbing should be performed to remove existing vegetation and objectionable material from improvement areas that will be graded, receive fill, or serve as borrow sources. Grubbing should include removing stumps, roots, vines, fencing and buried vegetation within the specified limits. Care should be taken not to injure trees, plants or existing improvements outside of the clearing limits or are designated to remain. Soil containing pavement, debris, organics, unsuitable, loose, or disturbed material should be removed prior to placing fill. Demolition areas should be cleared of old foundations, existing fill, pavement, abandoned utilities, and soil disturbed during clearing and grubbing. Depressions and excavations left from the removal or demolition of materials should be replaced with compacted fill.

7.1.3 SUBGRADE STABILIZATION

The geotechnical professional should review the subgrade conditions encountered at the time of construction to evaluate whether or not stabilization of the subgrade is needed, and to recommend the depth and limits of the subexcavation and stabilization.

Subgrade stabilization should be provided in areas where unsuitable materials or soft subgrade conditions are encountered that will not allow for proper compaction of the subgrade materials or consist of organic or other deleterious materials that will not provide suitable foundation support for the new roadway. Subgrade stabilization can consist of removing the existing soil to a depth at least 1 foot below the bottom of the structural section or bottom of the unsuitable material, whichever is deeper. If the subgrade is wet or yielding, subexcavation should be performed using backhoe type equipment such that construction equipment will not operate on the exposed subgrade during excavation.

A geotextile for stabilization should be placed over the undisturbed subgrade. The geotextile should be placed without gaps or wrinkles and comply with Caltrans Standard Specifications. Gravel for stabilization should consist of uniformly graded aggregate complying with Caltrans Standard Specifications. The aggregate should be fully encased in the geotextile to reduce the potential for the overlying base course to erode into the gravel.

7.1.4 COMPACTION AND GRADING

Table 1 provides a summary of the recommended minimum compaction for various locations where fill will be placed. Relative compaction should be assessed according to the latest approved edition of ASTM Standard Test Method D1557.

Table 1: Recommended Relative Compaction

Location of Fill Placement	Recommended Minimum Relative Compaction
General	90% U.O.N.
Pipe Bedding or Pipe Zone Material	90% U.O.N.
Trench Backfill	90% U.O.N.
Retaining wall backfill	90% U.O.N.
Fill or backfill placed within 3 feet of finished grade in pavement areas including Load Transfer Platform	95%
Foundation areas and within 5 feet horizontal of foundations	95%

U.O.N. = unless otherwise noted

7.1.5 FILL PLACEMENT

Jetting or ponding should not be permitted for placement or compaction of fill materials. Fill materials should be moisture conditioned and spread in lifts that are suitable for compaction with the equipment being used. Control of compaction layer thickness, moisture conditioning and selecting the proper size equipment will be necessary to achieve compaction throughout the material being placed. Fill should typically be placed in loose lifts of 8 inches or less, and within 2 percent of the optimum moisture content, to achieve the recommended compaction. The fill may need to be placed in thinner lifts to achieve the recommended compaction depending on the equipment being used.

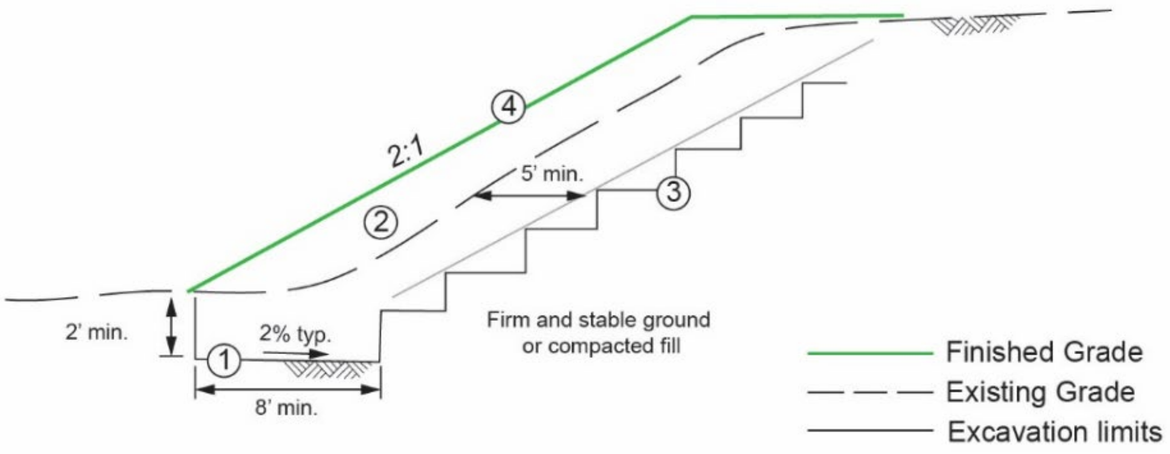
The moisture content of the material should be such that the specified compaction can be achieved in a firm and stable condition. Each layer should be spread evenly, bladed, and mixed to provide relative uniformity of material within each layer, and be moisture conditioned by adding water or drying the material to provide a moisture content suitable for compaction. Soft or yielding materials should be removed and replaced with properly compacted fill material prior to placing the next layer of fill.

Deleterious materials, such as concrete or pavement rubble, metal, glass or sharp objects should not be placed within the fill material being placed. Recycled or reused materials should only be used and placed within the fill when specifically permitted by the project specifications.

Rocks should not be nested, and voids should be filled with compacted fill material. Particles greater than half the compacted lift thickness can limit compactive effort. Rocks, cobbles, or other solid particles larger than 3 inches in the greatest dimension should be removed from the fill prior to compaction.

7.1.6 FILL PLACED ON OR AGAINST SLOPES

Fill to be placed against existing fill, natural slopes or on existing ground steeper than 20 percent grade should be keyed and benched into the existing slope as shown in Figure 5. The fill should be initiated from a base key that is excavated at the toe of the slope or the recommended depth of removal. The base key should be at least 8 feet wide at least 2 feet deep and sloped at 2 percent into the hillside. The new fill should be keyed into the existing slope such that at least the outer 5 feet of



1. Excavate base key into firm and competent material from toe of the proposed fill.
2. Place compacted fill per recommendations of the report.
3. Key and bench into existing embankment slope such that the outer 5 feet of the exitsting embankment is removed. Existing fill and excavated soil can be incorporated into the new fill.
4. Overbuild slope and cut back to expose compacted fill at finished grade.

Figure 5: Keying and Benching into Existing Slopes

the existing slope is removed. The excavated material can be incorporated into the fill being placed as the keying and benching progresses up the slope.

7.1.7 DESIGN OF GRADED SLOPES

Graded cut and fill slopes should be designed to an inclination of 2h:1v (horizontal to vertical) or flatter. Fill slopes should be constructed by placing compacted fill approximately 2 feet beyond the finished grade and then cutting the slope back to exposed compacted fill at the finished grade. The slope can then be track-walked and prepared for placement of landscaping and/or planting if needed.

7.1.8 EROSION AND DRAINAGE CONSIDERATIONS

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

7.1.9 REUSE OF EXCAVATED ONSITE MATERIAL

The excavation for the foundation areas, utilities or pipeline construction will likely encounter artificial fill and alluvium deposits consisting silty to clayey sand and sandy clay with varying amounts of silt (SC, CL, SM, SP-SM). Soil removed from these excavations that is free of organics or other deleterious material should be suitable for reuse as compacted fill placed below the MBR and EQ Tank foundations and as trench backfill. The soil may not be at a moisture content suitable for compaction as excavated and should be dried or wetted to a suitable moisture content prior to compaction. Select fill material needed for construction will include structure backfill, pipe bedding and pipe zone material. The excavated onsite soil should not be considered suitable for reuse as select material, such as pipe bedding, pipe zone material, structure (retaining wall) backfill, or aggregate base.

7.2 SEISMIC DATA

Structures should be designed to resist the lateral forces generated by earthquake shaking in accordance with the building code and standard design practice. Seismic data presented in Table 2 can be used for the design of structures with the building code and American Society of Civil Engineers (ASCE) 7-10. The seismic data were estimated for the site coordinates and using the United States Geological Survey (USGS) and SEAOC Seismic Design Maps (USGS 2019a) for ASCE Risk

Category III. The analysis assumed a Site Class E for a site underlain with more than 10 feet of soft clay. The USGS Unified Hazard Deaggregation Tool (USGS 2019b) was then used to estimate the mean magnitude and peak ground acceleration for the design earthquake that has a 2 percent probability of occurrence in 50 years.

Table 2: Seismic Data

Seismic Parameter	Value
Latitude, Degrees	35.1821
Longitude, Degrees	-120.7332
PGA_M, Peak ground acceleration	0.52g
Design Earthquake Magnitude	6.7
S_s, Seismic Factor, Site Class B at 0.2 second	1.341g
S_1, Seismic Factor, Site Class B at 1 second	0.485g
Site Class	S_E, Soft Soil
F_a, Site Coefficient for Site Class	0.9
F_v, Site Coefficient for Site Class	2.4
S_{MS}, Site-modified spectral acceleration for Site Class D at 0.2 seconds	1.207
S_{M1}, Site-modified spectral acceleration for Site Class D at 1 seconds	1.164
$S_{DS} = 2/3\ S_{MS}$	0.804g
$S_{D1} = 2/3\ S_{M1}$	0.776g
Long-Period Transition Period, T_L, seconds	8

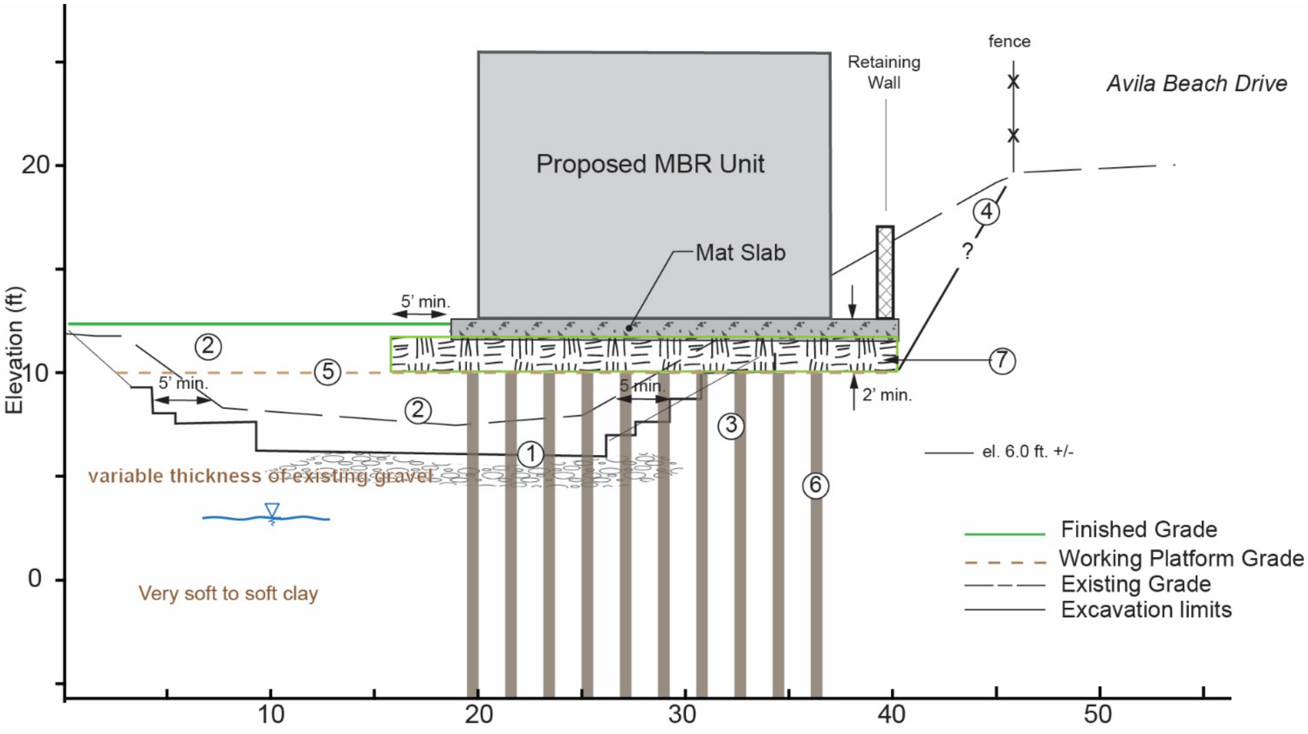
The potential for liquefaction, seismic settlement, lateral spreading, and tsunamis to proposed improvements were discussed in Section 6 of this report. The ground motions parameters provide for Type E soil are considered suitable for the site considering the soil profile and liquefaction potential. Lateral spreading along San Luis Obispo Creek could impact the northern portion of the site considering the design earthquake; however, is limited by thinning of the potentially liquefiable sand layers encountered near the proposed MBR. Rigid inclusions designed to limit static and seismic settlement to within tolerable limits are recommended in the following sections of this report.

7.3 MBR AND EQ TANK FOUNDATION DESIGN

The MBR units/sludge tank and the equalization tank (EQ Tank) will be supported on mat slab foundations bearing on ground improved with rigid inclusions and remedial grading as shown in Figure 6.

7.3.1 SLUDGE PIT

The existing soil within the sludge pit should be removed and replaced with compacted fill prior to ground improvement and grading for the MBR. A layer of gravel was encountered in a hand excavation made within the sludge pit that may have been placed as a stabilization layer during the previous site grading. The existing soil should be removed within the sludge pit area to elevation 6 feet, 1 foot below the bottom of the existing sludge pit, or to the top of the gravel layer if it is found to be uniform across the excavated area, whichever is deeper. The removal should then be stepped up the sides of the pit such that the outer 5 foot of soil is removed from the existing slopes.



1. Remove existing soil to approximately el. 6.0 feet
2. Place compacted fill to finished grades and key and bench into side slopes of sludge pit.
3. Extend the excavation as-needed to remove at least the outer 5 feet of the existing slopes.
4. Temporary slope and/or shoring by contractor.
5. Working platform grade for ground improvements
6. Rigid inclusions (typical)
7. Load Transfer Platform (per Contractor Design), 2' min

Figure 6: MBR Grading

The geotechnical professional should review the exposed subgrade conditions at the bottom of the excavation at the time of construction to evaluate if stabilization of the subgrade is needed, and to recommend the depth and limits of the subexcavation and stabilization.

7.3.2 GROUND IMPROVEMENTS

The foundation areas below the MBR slab and the EQ tank slab will be improved using a rigid inclusion type of ground improvement. Rigid inclusions should consist of concrete or cement grout columns placed on a grid pattern within the limits and depths recommended. This spacing and size of the inclusions should be designed by the contractor to meet specified performance criteria.

Limits and Depths of Ground Improvements. Ground improvements should be performed within the footprint of the MBR and EQ Tank foundations. The recommended typical configuration for ground improvement is shown on Figure 6. Ground improvements should extend from elevation -30 feet to a minimum of two feet below the bottom of the MBR and EQ Tank foundations: the bottom of the Load Transfer Platform (LTP).

Ground Improvement Plan. Prior to mobilizing to the site, the contractor should submit a plan for performing the ground improvement using rigid inclusions for review by the geotechnical professional. The plan should detail the layout and depths of the columns, the equipment that will be used, the methods of installation, proposed concrete or grout mix with supporting quality test data, methods for placing rigid inclusions, quality control, methods for verifying the submitted design, and the anticipated schedule to complete the work. Ground improvements should conform to at least the following recommendations:

- Rigid inclusions should be installed using the displacement auger method and have a minimum diameter of 18 inches. Concrete, sand/cement slurry neat cement grout should be used to construct the rigid inclusions.
- Ground improvement should provide a minimum allowable bearing capacity of 2,000 pounds per square foot and a combined subgrade modulus of 20 pounds per square inch per inch (pci) for the MBR and EQ Tank foundations.
- Rigid inclusions should limit differential settlement to ½ inch or less over 30 feet and total settlement to 1-inch static and seismic conditions with liquefaction. General design guidance for column spacing, strength and settlement checks should follow methods described in FHWA GEC-13: Ground Modification Methods Reference Manual (FHWA 2017) or similar references.
- Design should include a minimum 2-foot-thick load transfer platform constructed of geosynthetic reinforced aggregate base. The LTP should be designed to bridge between the rigid inclusions.
- Rigid inclusions should be installed from the bottom of the designed load transfer platform to an elevation of -30 feet for the MBR and EQ Tank foundations.
- Water, jetting or vibration should not be used to aid in insertion of the auger.
- Modulus testing of at least one column in each foundation area (MBR and EQ Tank) should be performed to verify the design. Modulus testing of columns should generally follow ASTM D1143 – *Standard Test Methods for Deep Foundations Under Static Axial Compressive Load*.
- The contractor should keep a daily record of what columns were constructed, the amount of concrete/grout placed in each column, and the time to install each column.

Site Conditions after Compaction. Installation of rigid inclusions typically will result in some heaving of the ground surface, muddy surface conditions as water and mud can be pushed up from below the ground. Site conditions may not be suitable for construction traffic immediately following ground improvements, and that wet soil from installing the rigid inclusions may need to be removed to restore the site to the previous grades in preparation for constructing the load transfer platform.

7.3.3 LOAD TRANSFER PLATFORM

A minimum 2-foot-thick load transfer platform should be provided below the MBR and EQ Tank foundations. The LTP should extend 5 feet beyond the edges of the foundations. The horizontal extent of the LTP can be reduced to the edge of the foundation along Avila Beach Drive at the MBR foundation. A Geotextile for Stabilization should be placed over the subgrade after removal of mud and spoils from ground improvements. The geotextile should be placed without gaps or wrinkles. Aggregate base should then be placed over the geotextile and compacted to 95 percent relative compaction. Additional layers or a thicker LTP may be required based upon the design of the ground improvement program.

7.3.4 MAT FOUNDATION DESIGN

The MBR and EQ Tank will be supported on concrete mat foundations. Mat foundations underlain by the recommended ground improvements can be designed using an allowable bearing capacity of 2,000 pounds per square foot and a modulus of subgrade reaction of 20 pounds per cubic inch. The modulus of subgrade reaction was estimated from the settlement analysis and foundation dimensions to limit the estimated total static settlement to less than 1-inch. The recommended modulus and bearing pressures can be increased by 1/3 when considering seismic or other transient loading conditions.

7.3.5 SETTLEMENT

The MBR and EQ Tank foundations should be designed to tolerate settlement and differential movement associated with static and seismic conditions. Foundations underlain by the recommended ground improvements with rigid inclusions should be designed to tolerate up to 1-inch of total settlement and 0.5 inches of differential settlement in 30 feet along the foundation. Flexible utility connections should be incorporated into the design of the project to allow for the estimated movement between the existing structures, the new structures and settlement of the ground outside of improvement areas.

7.3.6 LATERAL RESISTANCE

Resistance to lateral loading can be provided by sliding friction acting on the base of mat foundations combined with passive pressure acting on the sides of the foundations. A coefficient of friction of 0.4 should be used to estimate the sliding resistance along the bottom of foundations bearing on the LTP.

A net passive resistance of 380 pounds per cubic foot, equivalent fluid weight, should be used to estimate the lateral resistance acting on the sides of mat foundations. A 1/3 increase in the passive value can be used when considering short term wind or seismic loads. Passive resistance should not be used for the upper one foot of soil that is not constrained at the ground surface by slab-on-grade or pavement.

7.4 EQUIPMENT PADS

Equipment pads measuring up to 15 feet wide and long will be designed to support electrical control, generator, and other equipment. Recommendations for grading and foundations are provided. Refer to Section 7.3 for lateral resistance and settlement considerations of equipment pads.

7.4.1 GRADING

The existing soil within the foundation areas of equipment pads should be removed to a depth of 2 feet below the existing ground surface or 1 foot below the bottom of the foundation, whichever is deeper. The excavation should extend to a minimum of 1 foot beyond the edge of the equipment pad. The geotechnical professional should review the exposed subgrade conditions at the bottom of the excavation prior to fill placement to evaluate whether stabilization of the subgrade is needed. If the bottom of the excavation is not suitable for compaction, subgrade stabilization should be performed as recommended in the Earthwork section of this report prior to placing compacted fill. If the bottom of the excavation is firm and stable, the bottom should then be scarified to a depth of at least 9 inches, moisture conditioned, and compacted in-place to at least 95 percent relative compaction. Fill should then be placed to finish pad grade according to the recommendations of this report. The upper 4 inches of the fill below equipment pads should consist of aggregate base to provide a working platform.

7.4.2 FOOTING DESIGN

Foundations for equipment can be supported on the graded pads prepared according to the recommendations of this report. Footings should be designed to a minimum width of 1 foot and be embedded at least 12 inches below finished grade. A maximum allowable bearing pressure of 1,500 pounds per square foot (psf) may be used to design the foundations. The edge pressures on an eccentrically loaded footing can exceed the recommended allowable bearing pressure provided the average footing pressure is less than the allowable and the resultant acts within the middle third of the footing. The maximum allowable bearing pressure can be increased by one-third when considering short-term wind or seismic loads.

7.5 RETAINING WALL DESIGN

7.5.1 LATERAL EARTH PRESSURES

The retaining wall along Avila Beach Drive will be supported on the mat foundation for the MBR facility designed according to the recommendations presented in Section 7.3. The retaining wall and connection to the mat slab should be designed to resist lateral earth pressures from the wall and transfer those forces into the foundation. The proposed retaining wall will be a cantilever wall that is considered free to rotate or move within allowable building code tolerances and can be designed using active earth pressures. The existing slope behind the wall slopes at approximately 1.5h:1v above the proposed retaining wall location. The new graded slopes above the retaining wall will vary from flat to 1.5h:1v. Table 3 provides equivalent fluid weights that can be used to estimate the lateral earth pressure acting on the retaining wall for various backslopes.

Table 3: Recommended Lateral Earth Pressures

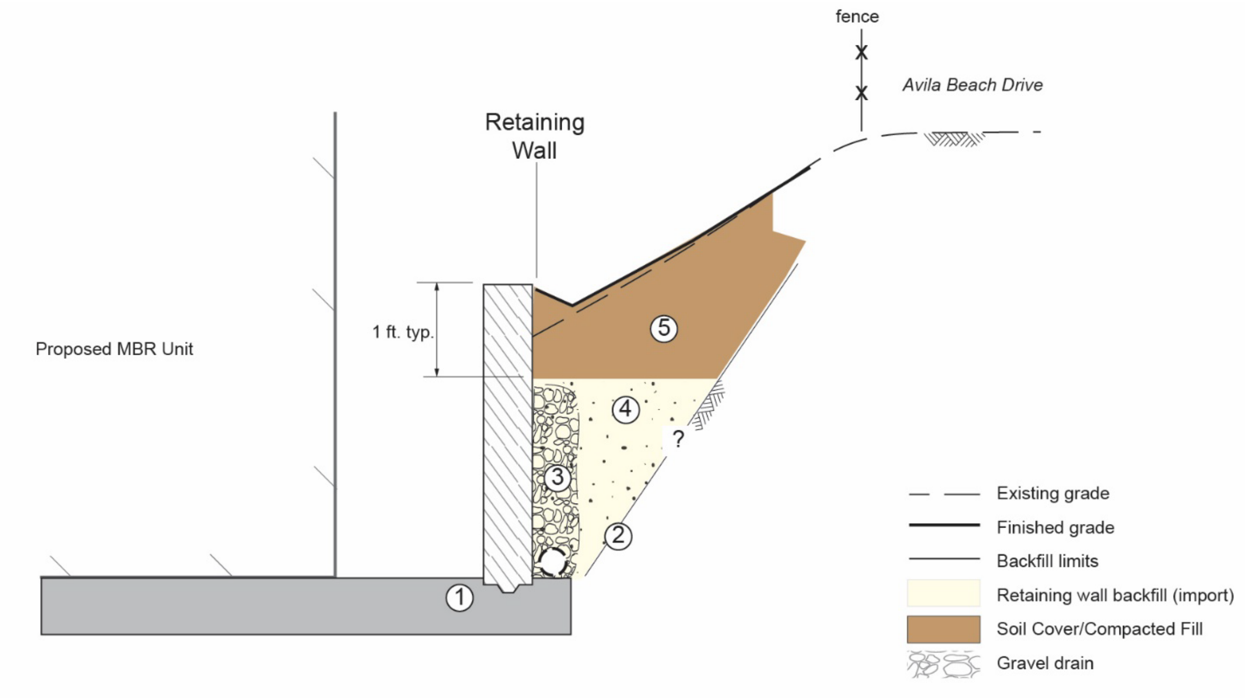
Earth Pressure Condition	Level Backslope	2h:1v Backslope	1.5h:1v Backslope
Active with drained backfill	35 pcf	52 pcf	70 pcf

The tabulated values are based on a soil unit weight of 125 pounds per cubic foot (pcf) and an effective friction angle of 34 degrees for drainage and backfill materials that comply with the recommendations of this report. The traffic surcharge along Avila Beach Drive is beyond the active zone of the retaining wall (location shown in Figure 2), and influence of traffic surcharges is not a design consideration for this segment of the retaining wall. The portion of the wall that will support the exit driveway should consider increases in the lateral earth pressure from vehicle surcharges. Traffic surcharges can be estimated as an additional 2 feet of soil cover, equal to a uniform lateral earth pressure of 72 pounds per square foot. The wall will be 6 feet or shorter and does not need to be designed for dynamic soil pressures per the California Building Code. Yeh should be contacted for additional recommendations if additional surcharges or other loading conditions need to be considered that could influence the lateral pressure on the wall.

7.5.2 WALL DRAINAGE AND BACKFILL

Figure 7 provides a typical detail showing the recommended wall drainage and backfill for the retaining wall. Retaining wall backfill material should consist of free-draining structure backfill materials compacted to at least 90 percent relative compaction and complying to the suggested material specifications of this report. Drainage should be provided behind the wall to reduce the potential for water to accumulate within the backfill. The back face of the wall should be covered with an appropriate sealant to reduce the potential for moisture to migrate through the wall. A at least 1-foot-thick gravel drain consisting of Gravel Bedding encased in Filter Fabric should be placed immediately behind the wall and extend vertical to 1 foot below finished grade at the top of the wall.

A Geocomposite Drainage Panel can be used in lieu of the gravel drain, if desired. Fill and backfill placed above the gravel drain or drainage panel can consist of compacted fill to reduce the potential for surface water to infiltrate behind the wall. The compacted fill should be placed to conform to adjacent grades above and beyond the wall.



1. Retaining wall footing to bear on mat foundation
2. Temporary slope by contractor
3. Place 1-foot wide gravel drain connected to collector pipe, geocomposite drainage panel or weep holes
4. Place Retaining Wall Backfill between wall and temporary slope to 1 foot below top of wall
5. Place compacted (native) fill to finished grades per recommendations of this report

Figure 7: Typical Retaining Wall Backfill

A collector pipe should be placed near the bottom of the backfill and connected to an outlet pipe or weep holes. Weep holes can consist of leaving a ½-inch wide gap in the head joint of the masonry wall. The open joint for the weep hole should be backed with a No. 4 galvanized wire mesh and approximately 1 cubic foot of gravel encased in a filter fabric or sack.

7.6 UTILITY TRENCHES AND PIPELINE DESIGN

A typical trench detail showing the cross-sectional limits of the Pipe Bedding or Gravel Bedding, Pipe Zone Material, and Trench Backfill are provided in Figure 8. The detail may be augmented per the Design Engineer. Material recommendations for Pipe Bedding or Gravel bedding, Pipe Zone Material, and Trench Backfill are described in this report.

7.6.1 FOUNDATION SUPPORT

The trench design should consider that trenches deeper than about 4 feet could encounter relatively soft and wet foundation soil. Prior to placing pipe bedding material, the foundation support soil exposed at the trench subgrade should be reviewed by the geotechnical professional. Stabilization of the subgrade should be provided prior to placing the pipe if the trench subgrade is soft or yielding. Yeh recommends that the trench design assume that the pipe will be bedded in at least 12 inches of Gravel Bedding encased in a geotextile for stabilization. If the foundation support soil is firm and unyielding the gravel can be replaced with sand bedding at the discretion of the Engineer. Pipe Bedding should be placed directly on the undisturbed subgrade at the bottom of the trench.

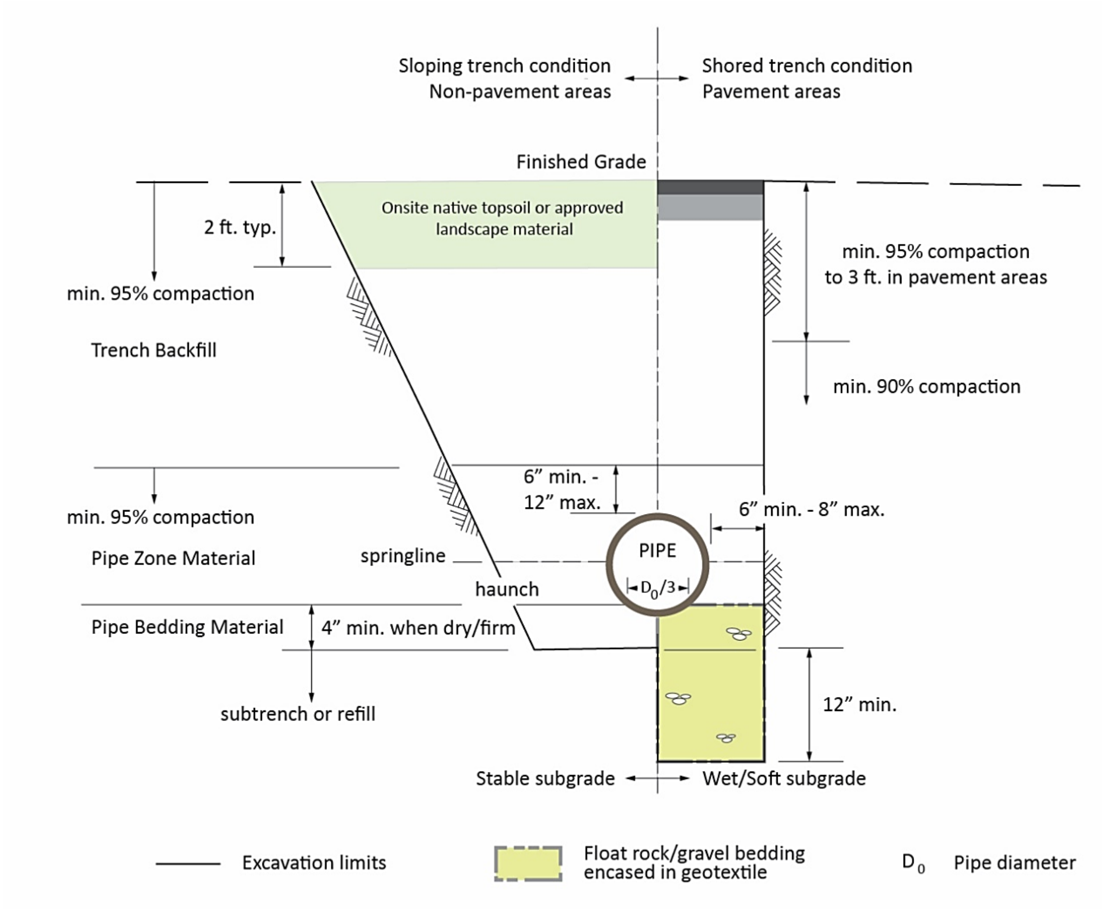


Figure 8: Typical Trench Detail

7.6.2 PIPE BEDDING

Pipe Bedding is initial backfill placed between the trench subgrade and the bottom of the pipe. The bedding should consist of at least 4 inches of Pipe Bedding placed on an undisturbed soil at the bottom of the trench if the foundation soil is firm and unyielding. The pipe should be placed on the bedding such that the middle third of the pipe is in contact with the bedding prior to placing initial backfill within the pipe zone (Do/3 as shown on Figure 8). The bedding may be loosened along the invert of the pipe if necessary to help form the cradle. Pipe Bedding should be compacted to at least 90 relative compaction. If the bottom of the trench is soft and yielding, the pipe bedding should consist of 12 inches of Gravel Bedding encased in a filter fabric as shown in Figure 8.

7.6.3 PIPE ZONE MATERIAL

Pipe Zone Material is material placed between the top of the Pipe Bedding to at least 6 inches (or maximum of 12 inches) above the crown of the pipe. Pipe Zone Material should be placed simultaneously on either side of the pipe to help support the pipe during placement and compaction. Compaction within the pipe zone should be performed such that the pipe is fully supported during compaction, that excessive deformation or damage to the pipe does not occur during compaction, and that the pipe is not moved off its alignment. Pipe Zone Material should not be placed above the springline until the Pipe Zone Material below the springline has been placed and compacted to properly support the haunches. The Pipe Zone Material should be compacted to at least 90 percent relative compaction prior to placing subsequent Trench Backfill.

7.6.4 TRENCH BACKFILL

Trench Backfill is material placed in the trench from the top of the pipe zone to finished grade. Trench backfill should consist of native or imported compacted fill material conforming to the recommendations described in this report. Material placed in trenches outside of building areas should be compacted to at least 95 percent relative compaction unless a higher degree of compaction is otherwise recommended.

7.7 PAVEMENT DESIGN RECOMMENDATIONS

7.7.1 STRUCTURAL SECTIONS

The structural section thickness was estimated using the methods in the Caltrans Highway Design Manual (2019) for Traffic Indices (TI) of 5, 5.5 and 6. An assumed design R-value of 10 was used for the subgrade conditions encountered. Structural section thicknesses were estimated considering full-depth and two-layer system of Hot-Mix Asphalt (HMA) over aggregate base (AB). Yeh should be contacted to provide additional structural section recommendation if traffic indices differ from those presented in Table 4.

Table 4: Recommended Pavement Structural Sections ($R_{subgrade}$ = 10)

Traffic Index (TI)	20-year Minimum Thicknesses (inches)	
	Full Depth HMA	HMA – 2 Layer
5.0	7 inches	3 inches HMA over 9 inches AB
5.5	8 inches	3 inches HMA over 11 inches AB
6.0	8.5 inche	3” HMA 12.5“ AB

HMA: Hot Mix Asphalt
AB: Class 2 Aggregate Base

7.7.2 PAVEMENT MAINTENANCE

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

7.8 STORMWATER BASIN

Infiltration testing was performed as input to the design of the proposed stormwater basin. Infiltration test data collected by Yeh are included in Appendix B. A summary of the estimated infiltration rates and averages for tests are summarized in Table 5. Initial (constant head) infiltration rates that ranged from less than 1 to 56 gallons per hour were measured in the borings. Equivalent twelve-inch diameter falling head percolation rates were estimated to range from approximately 0.33 to 52.25 inches per hour during infiltration testing. The results indicate a variable infiltration potential of subsurface materials.

Table 5: Estimated Infiltration Rates

Boring	Depth of Boring (ft)	Constant Head (gal/hr)	12-inch diameter Equivalent Falling Head (in/hr)
21I-01	3.25	23	52.25
21I-02	3.5	<1*	0.33
21I-03	5	56	4.40

* No additional water added after test head established. See Appendix B

Soil encountered in the upper 15 feet of the site in the borings and CPT soundings was predominantly clayey sand and sandy lean clay. The performance of stormwater control measures (SCM) can be affected by several properties. The designer of the SCM's should apply appropriate reduction factors to the reported infiltration rates. Compaction of soil below an SCM can lead to a reduction of infiltration rates such as where subgrade is compacted for roadway areas. The system designer should consider the impacts of the construction and long-term maintenance considerations for the SCM's planned at the site. Typical provisions for reducing the impact of SCM's on improvements include deepened curbs and impermeable liners along the bioswale margins. Bioswale plans and specifications should be reviewed by a geotechnical professional during the design process.

Any proposed bioswales should include provisions to reduce infiltration of stormwater into roadway and flatwork subgrade. Bioswales adjacent to or within the zone of influence[1] of foundations and at the top of slopes should also be avoided. Stormwater basins should be designed with slopes of 2h:1v or flatter.

8. CONSTRUCTION CONSIDERATIONS

8.1 GROUNDWATER CONSIDERATIONS

Groundwater conditions are discussed in this report. Groundwater was encountered at depths of approximately 10 feet (near elevation 1 foot) during the August 2019 CPT program. Soft and wet soil conditions were encountered in a hand excavation in the bottom of the existing sludge pit. Groundwater has been encountered at depths of 1.5 to 4 feet below the existing ground surface in previous borings drilled at the site.

Surface water or runoff that may enter the excavation during periods of rainfall should be removed prior to placing concrete. If needed, dewatering systems should be designed by a qualified engineer or hydrogeologist registered with the State of California. Sumps, well screens, and dewatering pits should be properly filtered such that fines and the surrounding soils are not removed by piping associated with dewatering.

8.2 TEMPORARY EXCAVATIONS

Temporary slopes should be braced or sloped according to the requirements of (Cal) OSHA. The design of temporary slopes or shoring systems needed for construction is the responsibility of the contractor. As input to design, the existing fill encountered above the bottom of the sludge pit generally consisted of stiff clay and clayey sand that which is classified as Type B soil by OSHA. The soil encountered below the sludge put is generally soft to medium stiff clay, which is classified as Type C soil by OSHA. Slopes should be inclined at 1h:1v (horizontal to vertical) and 1.5h:1v for Type B

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[1] Defined as the area within a 1h:1v plane projected down from the edge of a foundation or 1.5h:1v up from the bottom of a drainage swale.

and C soils, respectively. Slopes or shoring systems exceeding 20 feet in height are not addressed by OSHA and should be designed by a qualified professional engineer registered with the State of California.

Yeh analyzed a temporary cut slope along Avila Beach Drive based on several assumptions including slope angle, width of excavation, and time of year. These assumptions for the temporary slope could change depending on the contractor's approach to the project. Yeh recommends that the contractor submit an excavation and shoring plan for the project for review by the geotechnical professional prior to beginning earthwork at the site. The plan should consider the relatively complex subsurface conditions, soft soil and shallow groundwater, adjacent structures, as well as the proximity of excavations near Avila Beach Drive and utilities. The plan should include slope stability analyses, earth pressures, and monitoring plans for temporary excavation and shoring systems to support the proposed temporary excavation plan.

8.3 Adjacent Structures

Trenching to install utilities or pipelines could intersect or be parallel to existing utilities, pipelines and associated trench backfill, or be adjacent to existing structures. Bedding, shading, and possibly trench backfill of utilities or pipelines are likely composed of sandy soil that could become unstable or collapse into an adjacent excavation for this project. This could lead to lack of support for active utilities, pipelines, or structures and possibly damage existing infrastructure. Design of the shoring systems should consider adjacent structures and utilities or pipelines as well as loading of shoring systems. The contractor should submit an excavation plan for review by the geotechnical professional prior to beginning the excavation. The excavation plan should include shoring types, implementation, and response/contingency plans for addressing adjacent utility or pipeline trenches and/or infrastructure.

8.4 Subgrade Evaluation

The geotechnical professional should observe the bottom of excavations to evaluate if the exposed subgrade is suitable for fill placement. The project specifications should provide for review of the subgrade by the geotechnical professional, and for variations in the depth of excavation, if needed, to remove additional loose soil, undocumented fill, or unsuitable material.

8.5 Grading Observation

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

9. LIMITATIONS

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

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

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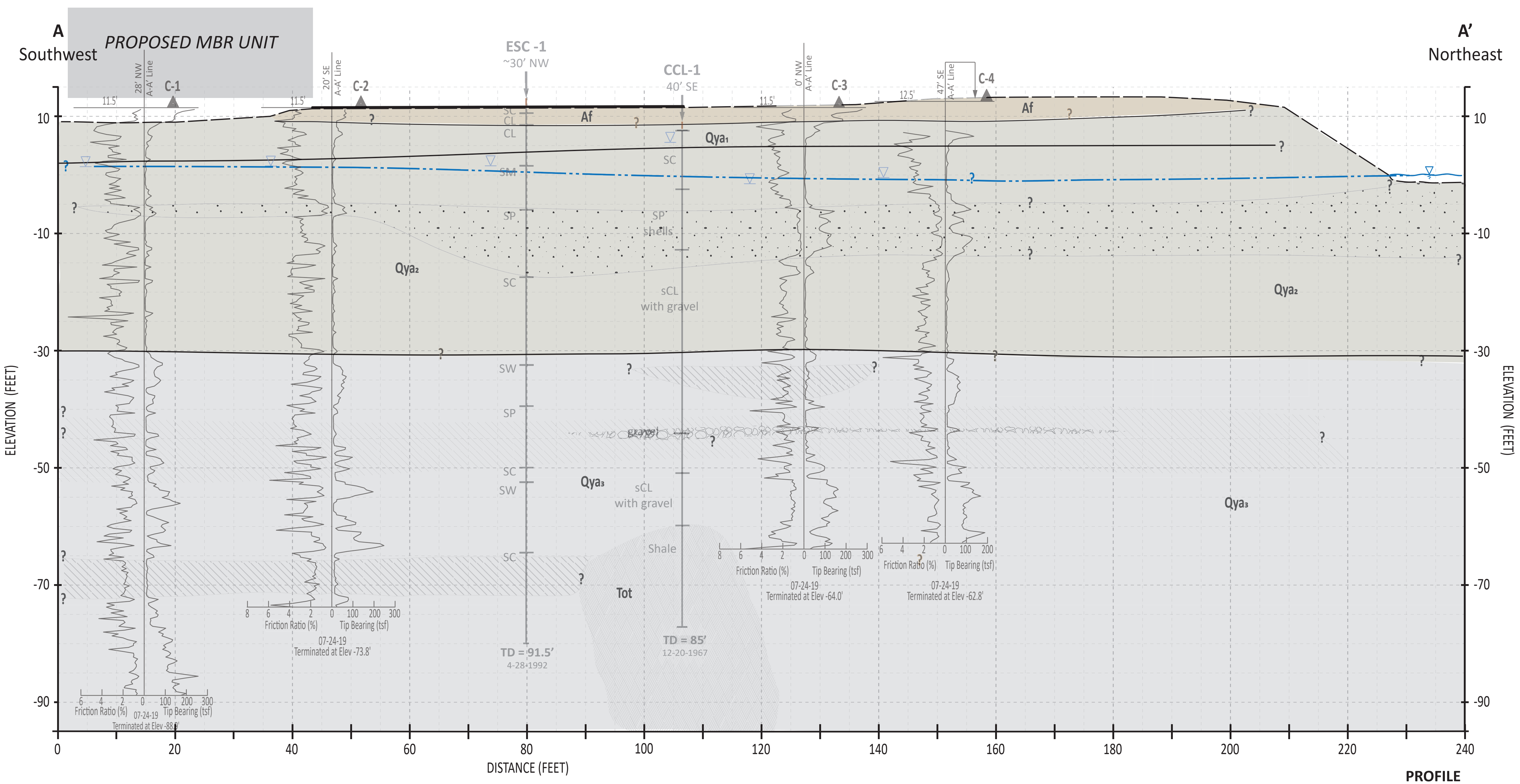
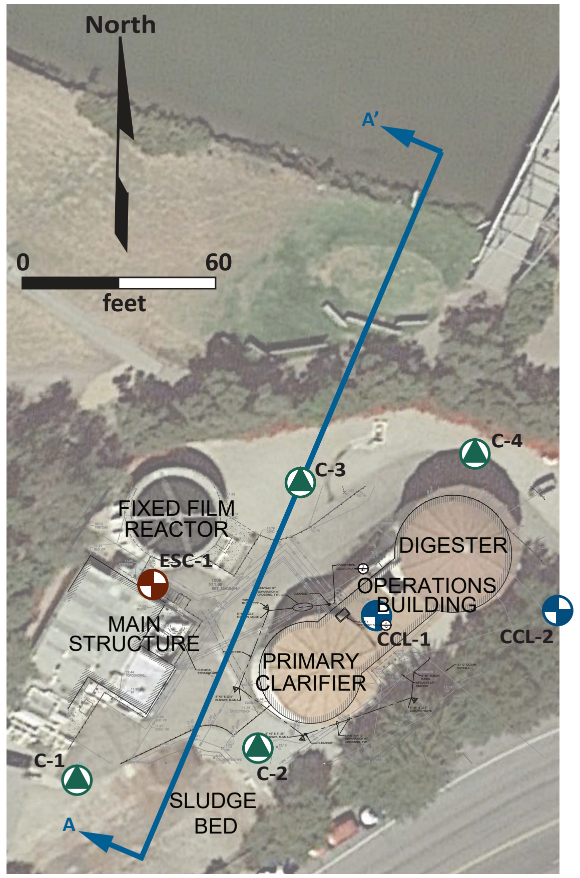




PROFILE

1 in. = 20 ft. vertical
1 in. = 20 ft. horizontal

LEGEND:

Af **Artificial Fill:** SAND and silty SAND

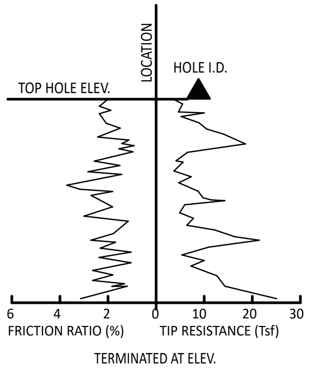
Qya **Younger Alluvium:**
1. Medium stiff to stiff CLAY with interbeds of sand and clayey sand

2. Soft to medium CLAY with interbeds of sand and clayey sand

3. Interbedded medium to stiff CLAY and medium to dense SAND with varying amounts of silt, sand, clay and gravel

Tot **Obispo Formation:** Clay SHALE

▨	Interbeds of GRAVEL with varying amounts of SILT
▨	Interbeds of SAND with varying amounts of SILT
▨	Interbeds of CLAY
▨	Shale bedrock
A A'	Cross section location
—?—	Geologic contact, queried where uncertain
—?·-—	Interpreted groundwater surface during cone penetration testing, queried where uncertain
▽	Groundwater level encountered during cone penetration testing
▲ No.	Cone penetration test (CPT) location (Yeh and Associates, 2019)
⊕ No.	Hollow stem auger boring location (Earth Systems Consultants, 1992)
⊕ No.	Test hole location (Central Coast Laboratories, 1967)



CONE PENETRATION TEST (CPT) DIAGRAM

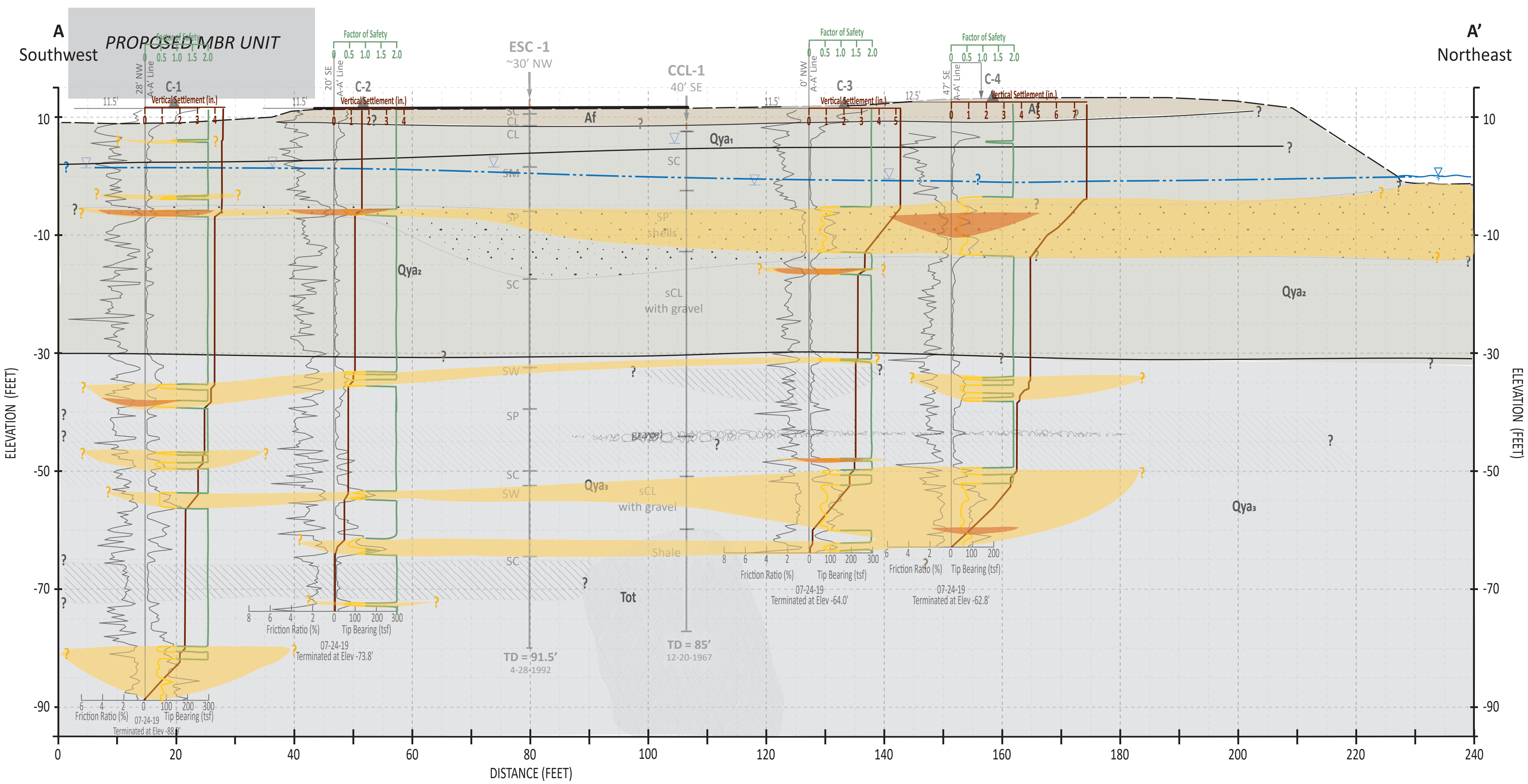
See text and logs of exploration for description of subsurface conditions.
All boundaries and locations are approximate.

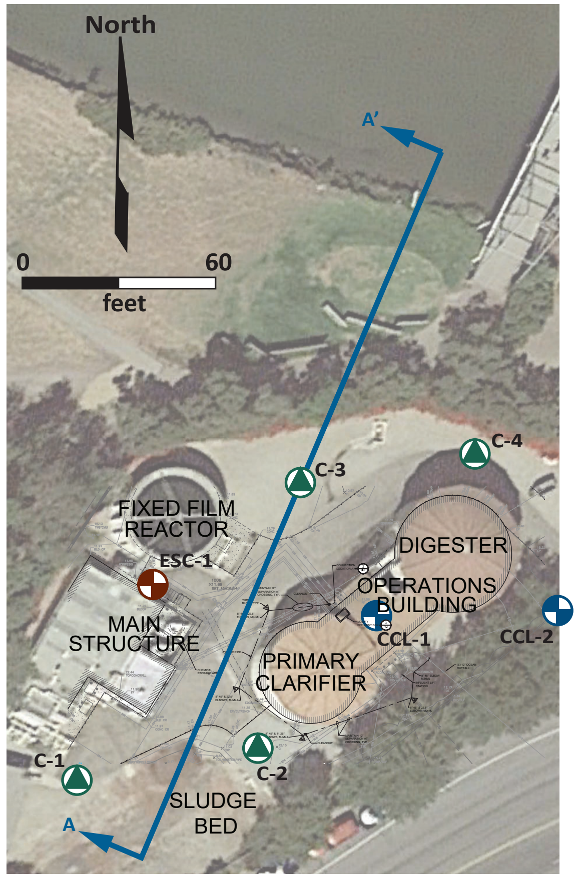
Yeh and Associates, Inc.
Geotechnical • Geological • Construction Services

INTERPRETED SUBSURFACE PROFILE

PROJECT NAME:	PLATE
AVILA BEACH COMMUNITY SERVICES DISTRICT WWTP UPGRADE Avila Beach, CA	1

PROJECT NUMBER:	REVISION DATE:
219-201	10.14.2019





PROFILE

1 in. = 20 ft. vertical
1 in. = 20 ft. horizontal

LEGEND:

Af **Artificial Fill:** SAND and silty SAND

Qya **Younger Alluvium:**
1. Medium stiff to stiff CLAY with interbeds of sand and clayey sand
2. Soft to medium CLAY with interbeds of sand and clayey sand
3. Interbedded medium to stiff CLAY and medium to dense SAND with varying amounts of silt, sand, clay and gravel

Tot **Obispo Formation:** Clay SHALE

▨	Interbeds of GRAVEL with varying amounts of SILT
⁘	Interbeds of SAND with varying amounts of SILT
▨	Interbeds of CLAY
▨	Shale bedrock

See Plate 1 for Plan Legend

🟨	Potentially liquefiable soil layers due to design earthquake (M6.7, 0.51g)
🟧	Potentially liquefiable soil layers due to 2003 San Simeon earthquake (M6.5, 0.15g)

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

Yeh and Associates, Inc.
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LIQUIFIED SUBSURFACE PROFILE

PROJECT NAME:
AVILA BEACH COMMUNITY SERVICES DISTRICT
WWTP UPGRADE
Avila Beach, CA

PLATE
2

PROJECT NUMBER: 219-201
REVISION DATE: 10.14.2019

APPENDIX A - CONE PENETRATION TESTS

GREGG DRILLING, LLC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

7/25/19

Yeh & Associates, Inc.
Attn: Jon Blanchard

Subject: CPT Site Investigation
Avila Beach CSD WWTF Improvements
Avila Beach, California
GREGG Project Number: 190588SH

Dear Mr. Blanchard:

The following report presents the results of GREGG Drilling Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	UVOST Laser Induced Fluorescence	(UVOST)	☐
5	Groundwater Sampling	(GWS)	☐
6	Soil Sampling	(SS)	☐
7	Vapor Sampling	(VS)	☐
8	Pressuremeter Testing	(PMT)	☐
9	Vane Shear Testing	(VST)	☐
10	Dilatometer Testing	(DMT)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact me at 714-863-0988.

Sincerely,
GREGG Drilling, LLC.

Frank Stolfi
HRSC Division Manager, Gregg Drilling, LLC.

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
C-1	7/24/2019	100.39	-	-	67.2
C-2	7/24/2019	85.3	-	-	75.2
C-3	7/24/2019	75.46	-	-	19.0
C-4	7/24/2019	75.3	-	-	-

Cone Penetration Test Coordinates

-Table 2-

CPT Sounding Identification	Date	Lat or Northing	Long or Easting	Elevation (Feet)
C-1	7/24/2019	35.18199	-120.73331	UNKNOWN
C-2	7/24/2019	35.18229	-120.733016	UNKNOWN
C-3	7/24/2019	35.18211	-118.73297	UNKNOWN
C-4	7/24/2019	35.18241	-120.732717	UNKNOWN

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Copies of ASTM Standards are available through www.astm.org

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The CPT parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (*PPDT*). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a “knock out” plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

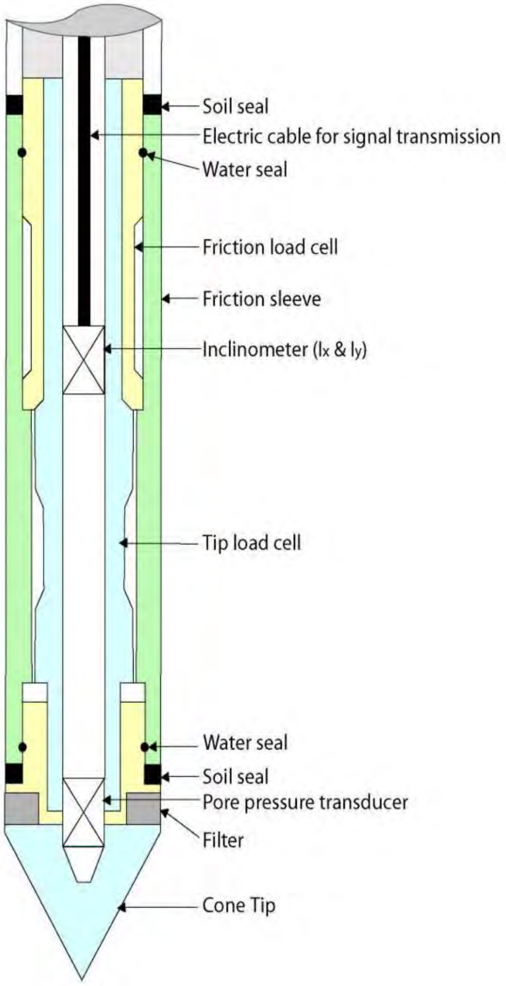


Figure CPT

Gregg 15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.85
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

Note: The repeatability on site will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (2009 & 2010). Typical plots display SBT based on the non-normalized charts of Robertson (2010). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (2009) which can be displayed as SBTn, upon request. The report can also include spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Robertson and Cabal (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface. Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.

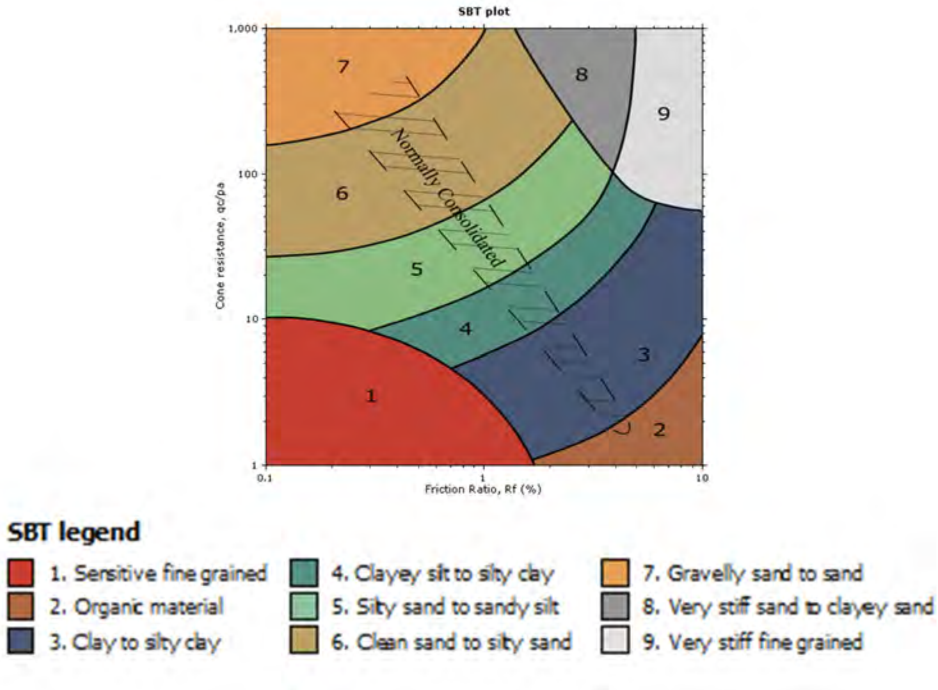


Figure SBT (After Robertson, 2010) – Note: Colors may vary slightly compared to plots

Cone Penetration Test (CPT) Interpretation

Gregg uses a commercial CPT interpretation and plotting software (CPeT-IT https://geologismiki.gr/products/cpet-it/). The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997) and updated by Robertson and Cabal (2015). The interpretation is presented in tabular format. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameter.

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236\right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a}\right) \cdot \frac{1}{10^{1.1268-0.2817 I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Drained Friction Angle, φ (°) ::

$$\varphi = \varphi'_{cv} + 15.94 \cdot \log(Q_{tn,cs}) - 26.88$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$
$\alpha = 14$ for $Q_{tn} > 14$
$\alpha = Q_{tn}$ for $Q_{tn} \leq 14$
$M_{CPT} = \alpha \cdot (q_t - \sigma_v)$

If $I_c \geq 2.20$

$$M_{CPT} = 0.03 \cdot (q_t - \sigma_v) \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho}\right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))}\right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin\varphi') \cdot OCR^{\sin\varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Peak Friction Angle, φ' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10 < B_q < 1.00$)

References

ASTM D5778-12, 2012, Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils. ASTM West Conshohocken, USA

Lunne, T., Robertson, P.K. and Powell, J.J.M., 1997. Cone Penetration Testing in Geotechnical Practice.

Robertson, P.K., 1990. Soil Classification using the Cone Penetration Test. Canadian Geotechnical Journal, Volume 27: 151-158

Robertson, P.K., 2009. Interpretation of Cone Penetration Tests – a unified approach. Canadian Geotechnical Journal, Volume 46: 1337-1355

Robertson, P.K., 2010, “Soil Behavior type from the CPT: an update”, 2nd International Symposium on Cone Penetration Testing, Huntington Beach, CA, Vol.2. pp 575-583

Robertson, P.K. and Cabal, K.L., “Guide to Cone Penetration Testing for Geotechnical Engineering”, 6th Edition, 2015, 145 p. Free online, http://www.greggdrilling.com/technical-guides.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, "Seismic CPT to Measure In-situ Shear Wave Velocity", Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 8, pp. 791-803, 1986.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

CPT LOGS

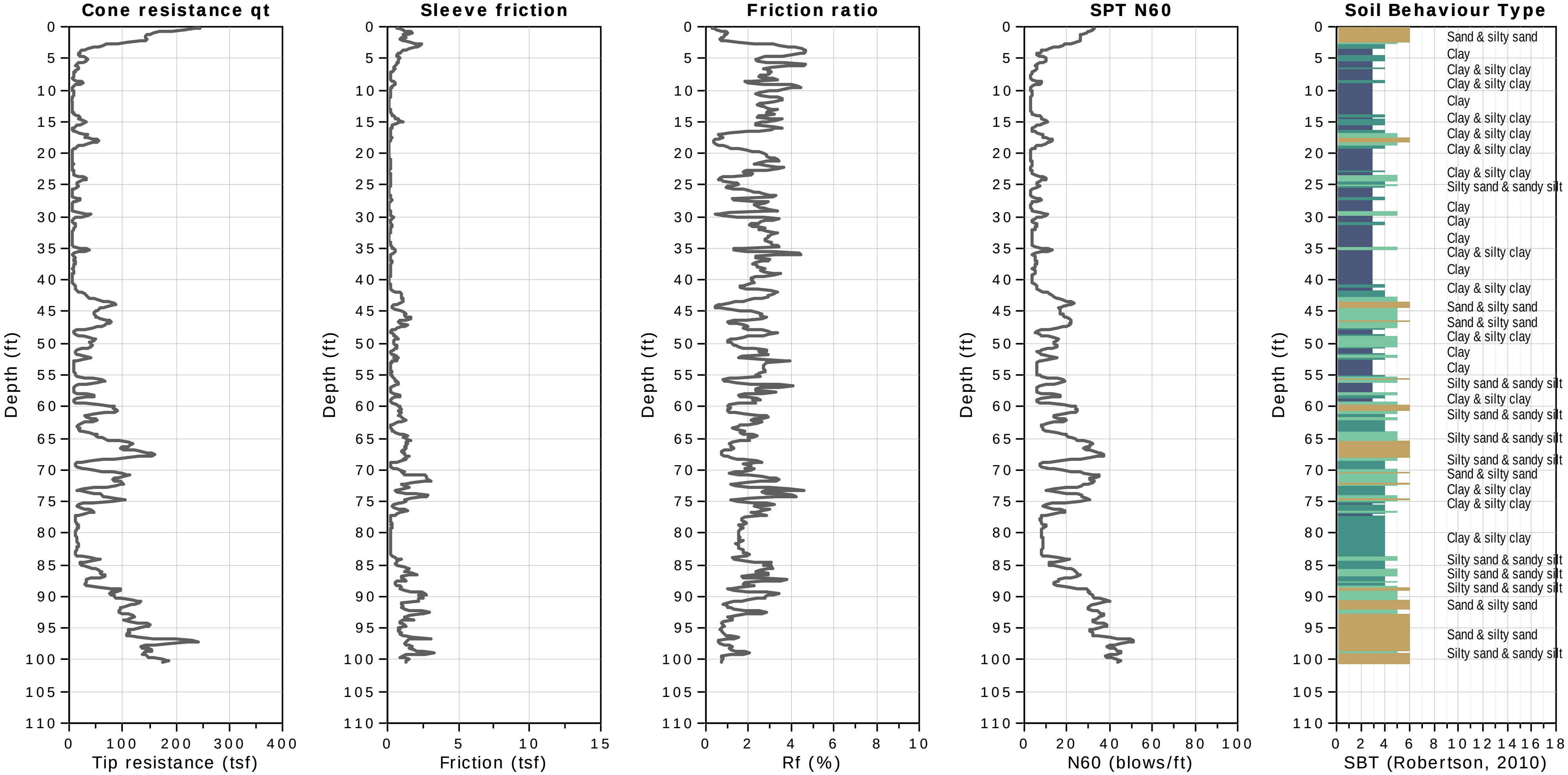
CPT: C-1

CLIENT: YEH & ASSOCIATES, INC.

SITE: AVILA BEACH CSD WWTF IMPROVEMENTS - 2850 AVILA BEACH DRIVE, AVILA BEACH, CA

FIELD REP: JAMIE C.

Total depth: 100.39 ft, Date: 7/24/2019



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

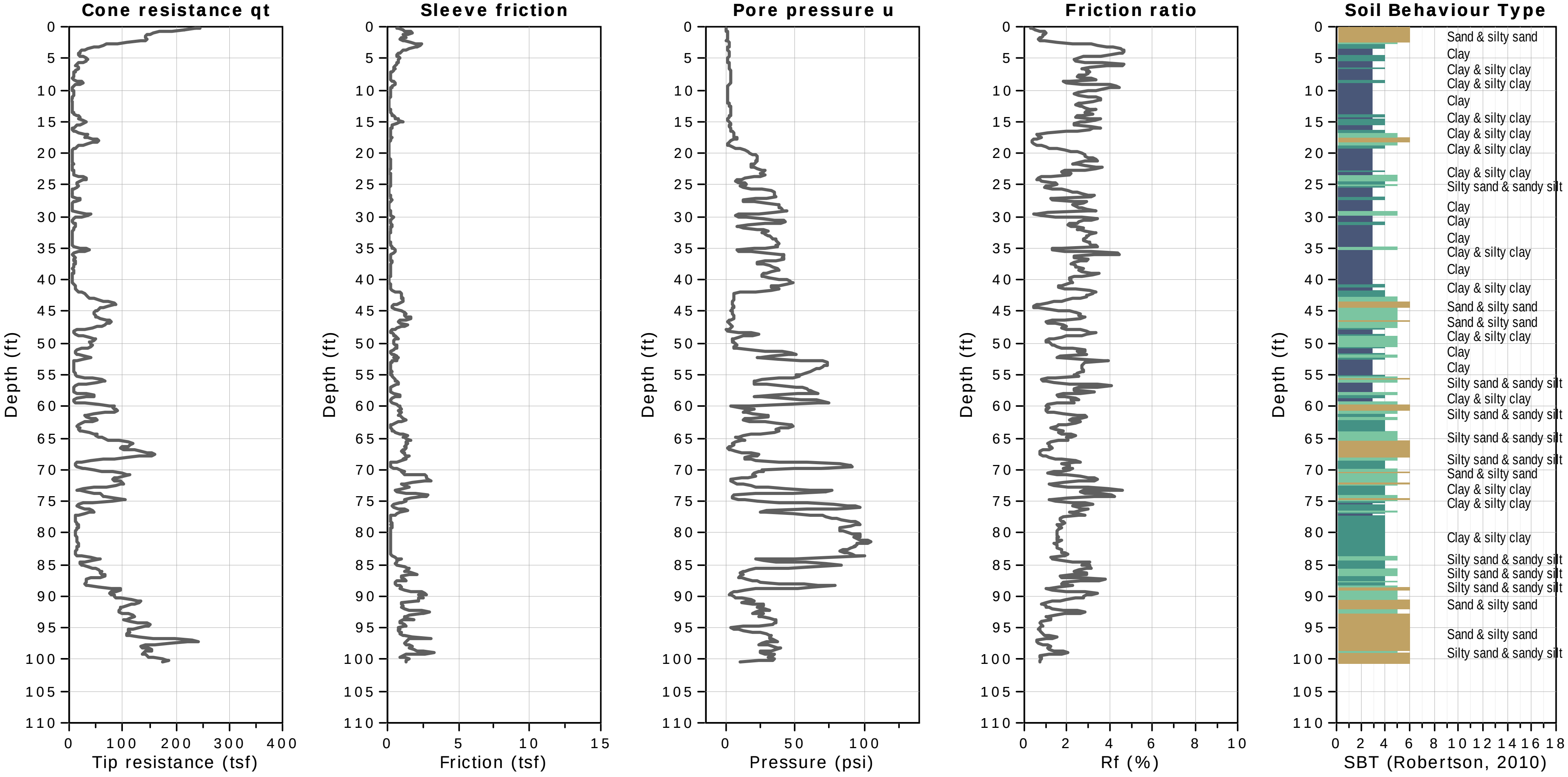
CPT: C-1

CLIENT: YEH & ASSOCIATES, INC.

SITE: AVILA BEACH CSD WWTF IMPROVEMENTS - 2850 AVILA BEACH DRIVE, AVILA BEACH, CA

Field Rep: JAMIE C.

Total depth: 100.39 ft, Date: 7/24/2019



SBTn legend

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

WATER TABLE FOR ESTIMATING PURPOSES ONLY

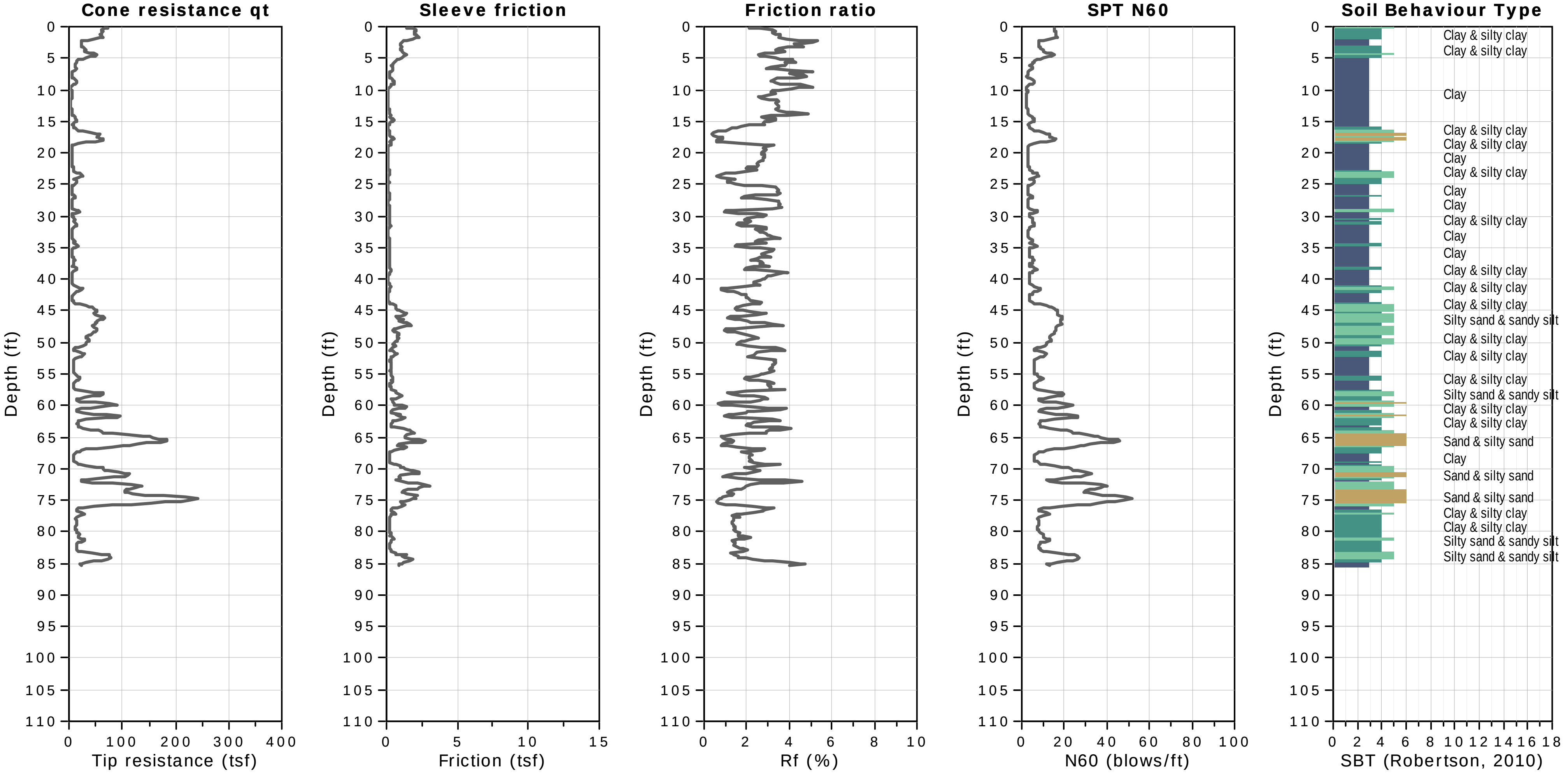
CPT: C-2

CLIENT: YEH & ASSOCIATES, INC.

FIELD REP: JAMIE C.

SITE: AVILA BEACH CSD WWTF IMPROVEMENTS - 2850 AVILA BEACH DRIVE, AVILA BEACH, CA

Total depth: 85.30 ft, Date: 7/24/2019



SBTn legend

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

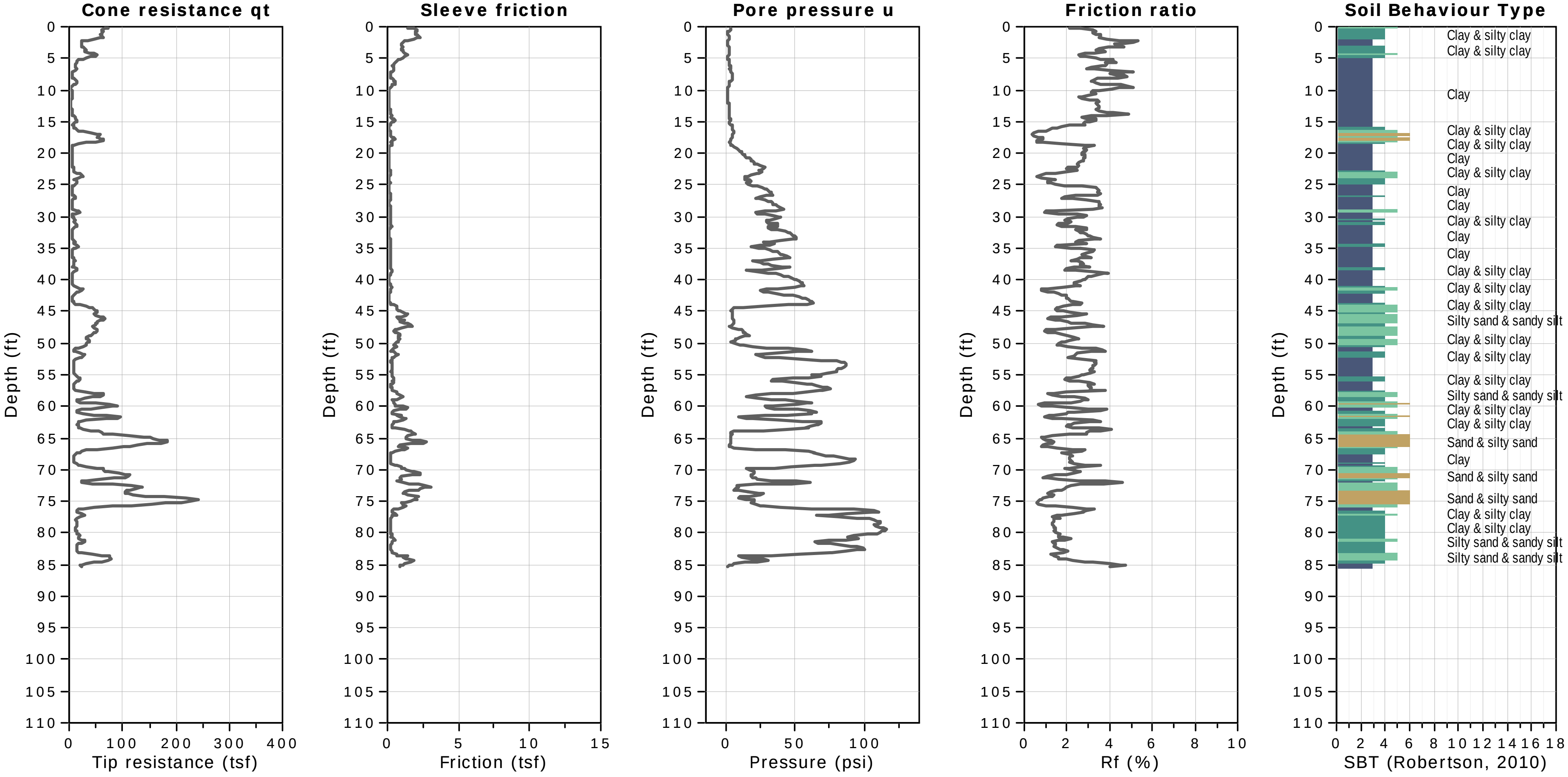
CPT: C-2

CLIENT: YEH & ASSOCIATES, INC.

Field Rep: JAMIE C.

SITE: AVILA BEACH CSD WWTF IMPROVEMENTS - 2850 AVILA BEACH DRIVE, AVILA BEACH, CA

Total depth: 85.30 ft, Date: 7/24/2019



SBTn legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty clay	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟨 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

WATER TABLE FOR ESTIMATING PURPOSES ONLY

GREGG DRILLING, INC.
www.greggdrilling.com

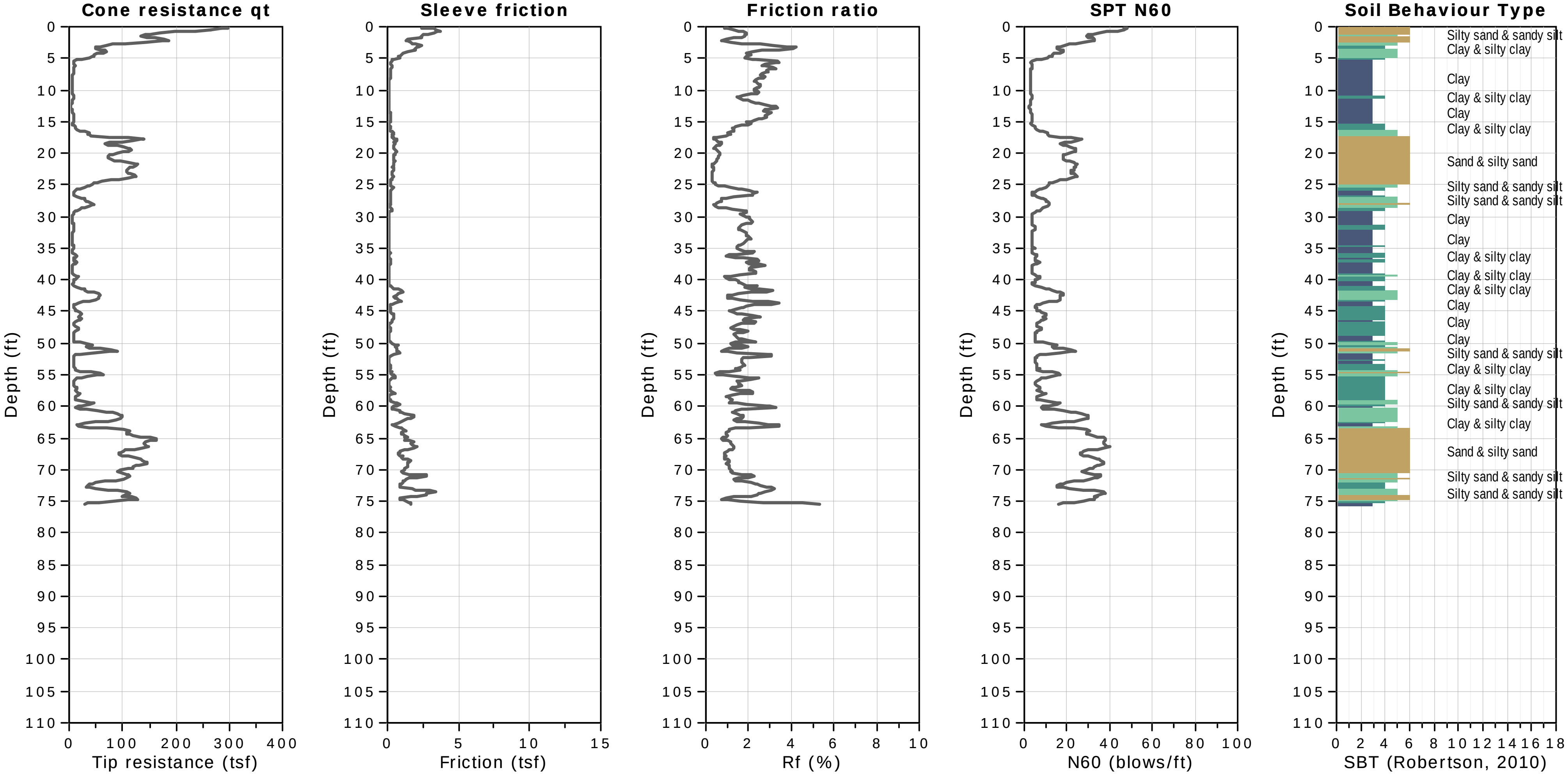
CPT: C-3

CLIENT: YEH & ASSOCIATES, INC.

FIELD REP: JAMIE C.

SITE: AVILA BEACH CSD WWTF IMPROVEMENTS - 2850 AVILA BEACH DRIVE, AVILA BEACH, CA

Total depth: 75.46 ft, Date: 7/24/2019



SBTn legend

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

GREGG DRILLING, INC.
www.greggdrilling.com

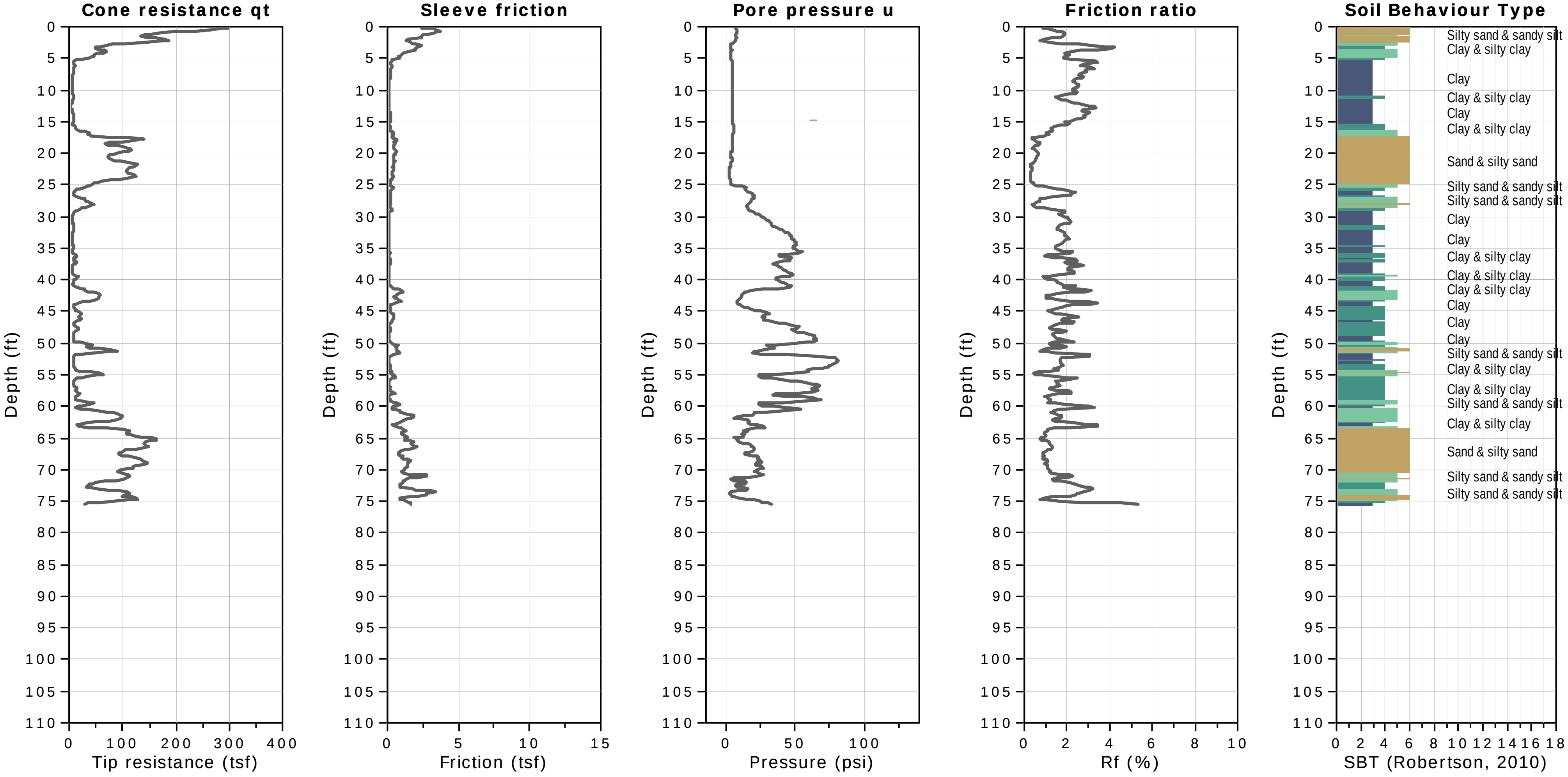
CPT: C-3

CLIENT: YEH & ASSOCIATES, INC.

SITE: AVILA BEACH CSD WWTF IMPROVEMENTS - 2850 AVILA BEACH DRIVE, AVILA BEACH, CA

Field Rep: JAMIE C.

Total depth: 75.46 ft, Date: 7/24/2019



SBTn legend

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

WATER TABLE FOR ESTIMATING PURPOSES ONLY

CPeT-IT v.19.0.1.19 - CPTU data presentation & interpretation software - Report created on: 7/25/2019, 9:44:05 AM
Project file: C:\Users\Frank Stolfi\OneDrive - Gregg Drilling\SH-2019\190588SH\REPORT\190588.cpt

GREGG DRILLING, INC.
www.greggdrilling.com

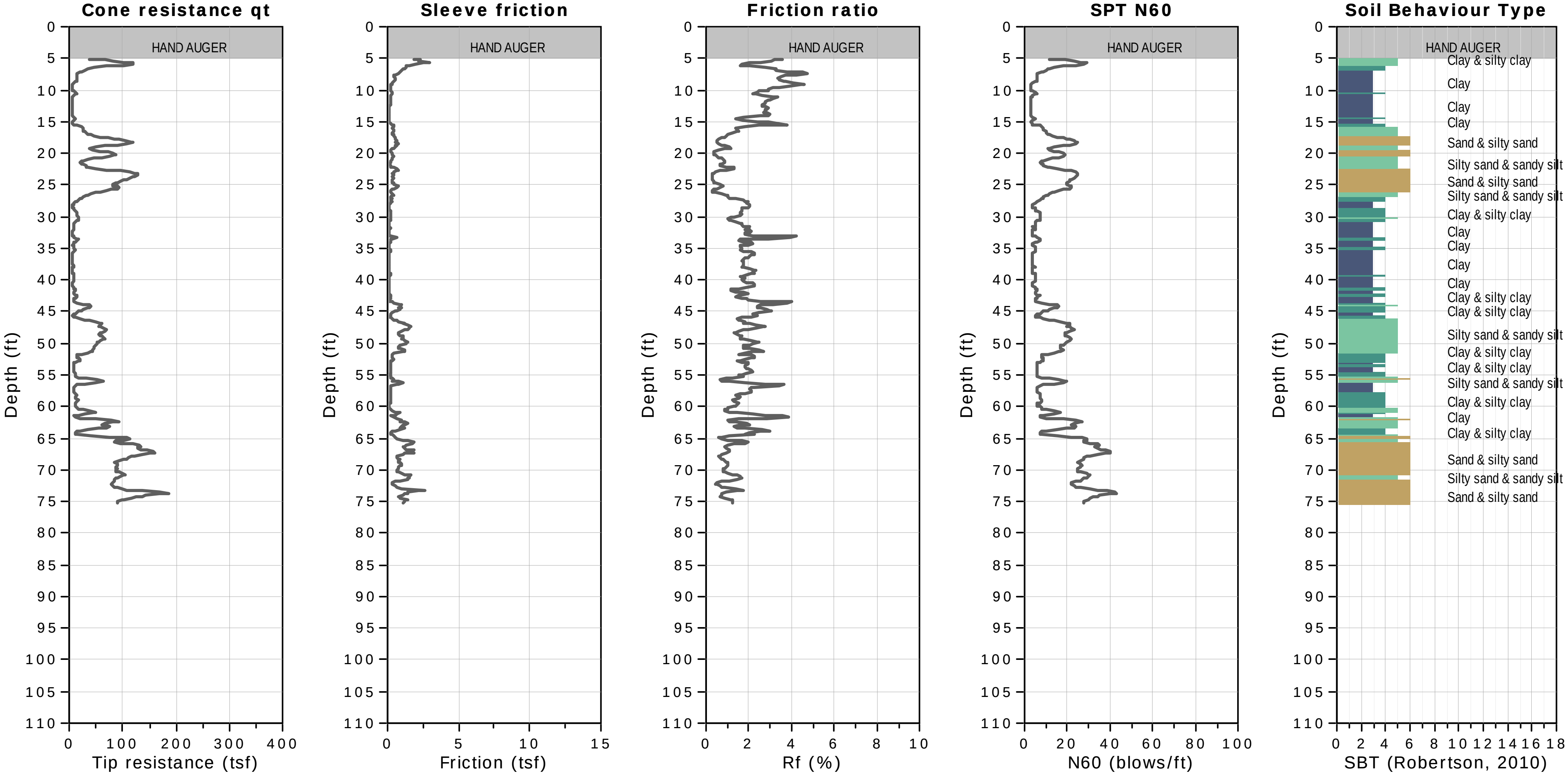
CPT: C-4

CLIENT: YEH & ASSOCIATES, INC.

SITE: AVILA BEACH CSD WWTF IMPROVEMENTS - 2850 AVILA BEACH DRIVE, AVILA BEACH, CA

FIELD REP: JAMIE C.

Total depth: 75.30 ft, Date: 7/24/2019



SBTn legend

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

CPeT-IT v.19.0.1.19 - CPTU data presentation & interpretation software - Report created on: 7/25/2019, 9:44:05 AM
Project file: C:\Users\Frank Stolfi\OneDrive - Gregg Drilling\SH-2019\190588SH\REPORT\190588.cpt

GREGG DRILLING, INC.
www.greggdrilling.com

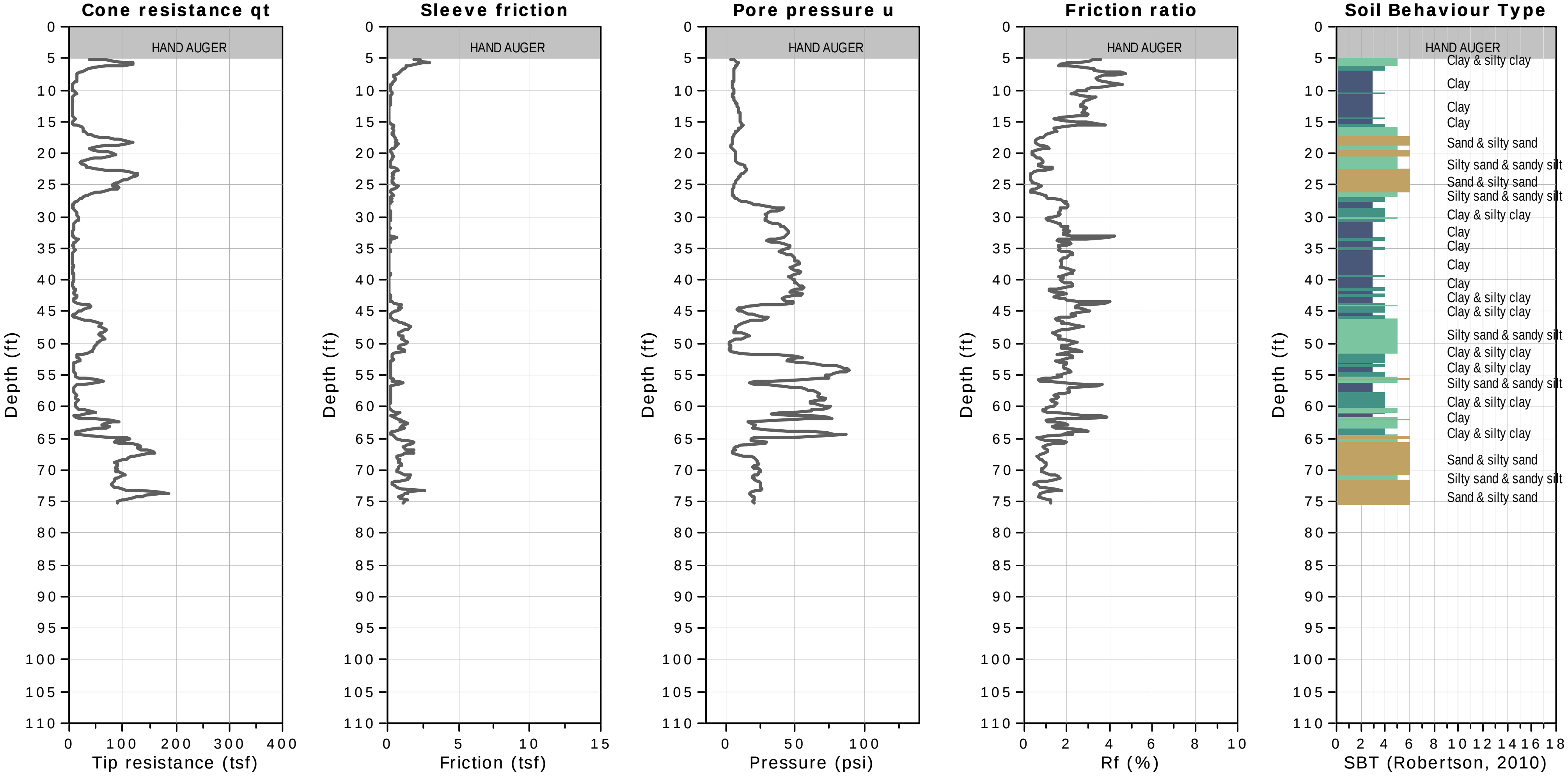
CPT: C-4

CLIENT: YEH & ASSOCIATES, INC.

SITE: AVILA BEACH CSD WWTF IMPROVEMENTS - 2850 AVILA BEACH DRIVE, AVILA BEACH, CA

Field Rep: JAMIE C.

Total depth: 75.30 ft, Date: 7/24/2019



SBTn legend

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

WATER TABLE FOR ESTIMATING PURPOSES ONLY

PORE PRESSURE DISSIPATION

Pore Pressure Dissipation Tests (PPDT

Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals can be used to measure equilibrium water pressure (at the time of the CPT). If conditions are hydrostatic, the equilibrium water pressure can be used to determine the approximate depth of the ground water table. A PPDT is conducted when penetration is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (*u)* with time is measured behind the tip of the cone and recorded.

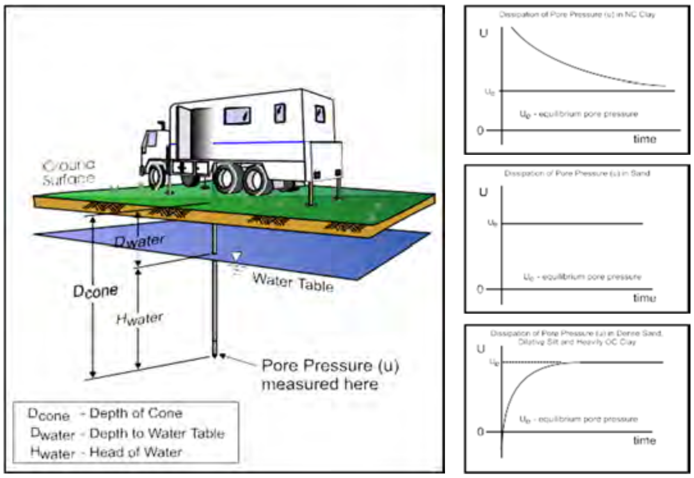
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In-situ horizontal coefficient of consolidation (c_h)
- In-situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until it reaches equilibrium, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992 and Lunne et al. 1997.

A summary of the pore pressure dissipation tests is summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

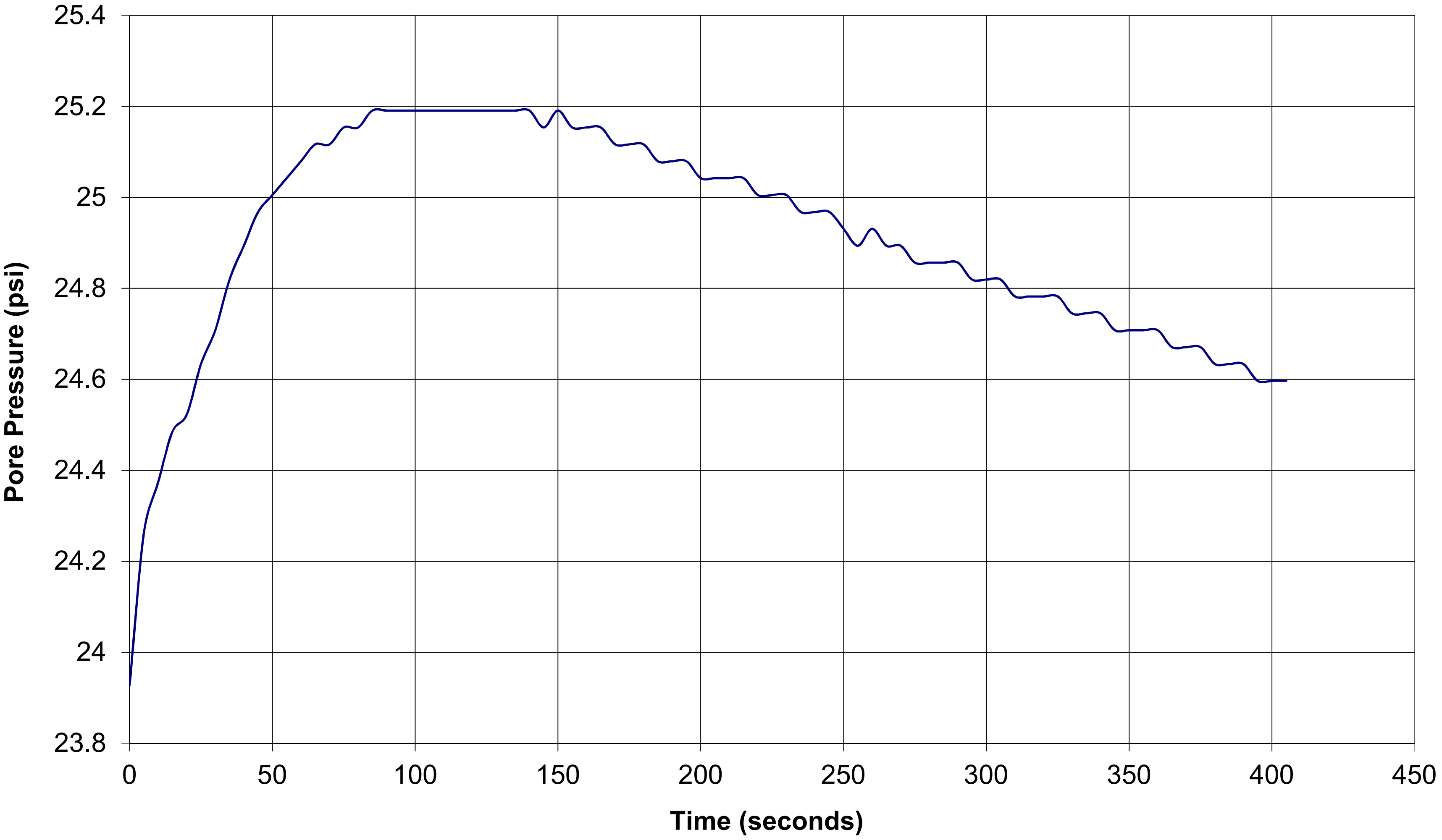
Useful Conversion Factors:
1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

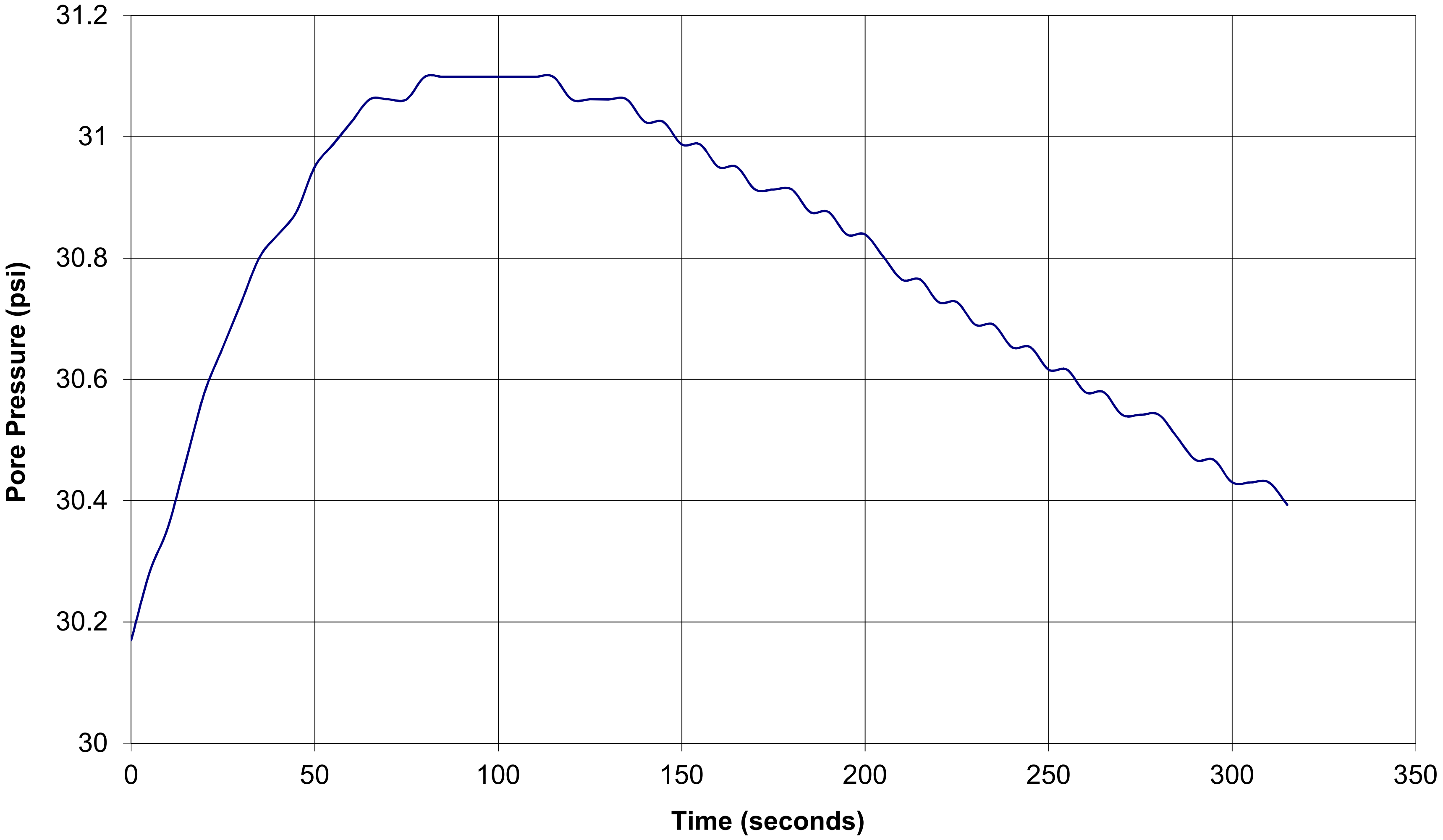
Sounding: C-1
Depth: 67.257015
Site: AVILA BEACH
Engineer: JAMIE C.



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

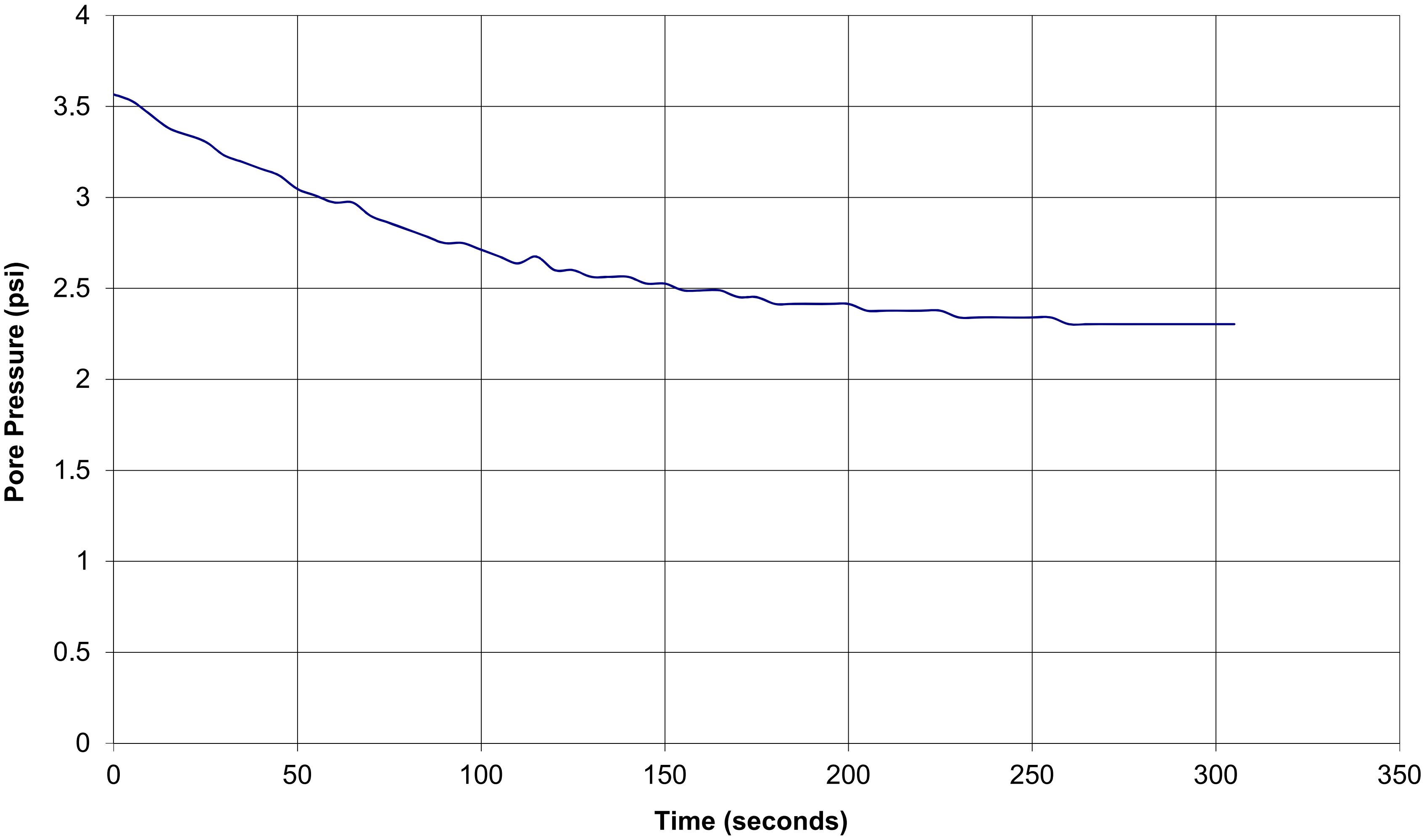
Sounding: C-2
Depth: 75.2950485
Site: AVILA BEACH
Engineer: JAMIE C.



GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: C-3
Depth: 19.028814
Site: AVILA BEACH
Engineer: JAMIE C.



APPENDIX B - HAND EXCAVATION AND INFILTRATION TEST LOGS AND RESULTS

GROUP SYMBOLS AND NAMES

Graphic / Symbol	Group Names	Graphic / Symbol	Group Names
GW	Well-graded GRAVEL Well-graded GRAVEL with SAND	CL	Lean CLAY Lean CLAY with SAND Lean CLAY with GRAVEL SANDY lean CLAY SANDY lean CLAY with GRAVEL GRAVELLY lean CLAY GRAVELLY lean CLAY with SAND
GP	Poorly graded GRAVEL Poorly graded GRAVEL with SAND		
GW-GM	Well-graded GRAVEL with SILT Well-graded GRAVEL with SILT and SAND	CL-ML	SILTY CLAY SILTY CLAY with SAND SILTY CLAY with GRAVEL SANDY SILTY CLAY SANDY SILTY CLAY with GRAVEL GRAVELLY SILTY CLAY GRAVELLY SILTY CLAY with SAND
GW-GC	Well-graded GRAVEL with CLAY (or SILTY CLAY) Well-graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GP-GM	Poorly graded GRAVEL with SILT Poorly graded GRAVEL with SILT and SAND	ML	SILT SILT with SAND SILT with GRAVEL SANDY SILT SANDY SILT with GRAVEL GRAVELLY SILT GRAVELLY SILT with SAND
GP-GC	Poorly graded GRAVEL with CLAY (or SILTY CLAY) Poorly graded GRAVEL with CLAY and SAND (or SILTY CLAY and SAND)		
GM	SILTY GRAVEL SILTY GRAVEL with SAND	OL	ORGANIC lean CLAY ORGANIC lean CLAY with SAND ORGANIC lean CLAY with GRAVEL SANDY ORGANIC lean CLAY SANDY ORGANIC lean CLAY with GRAVEL GRAVELLY ORGANIC lean CLAY GRAVELLY ORGANIC lean CLAY with SAND
GC	CLAYEY GRAVEL CLAYEY GRAVEL with SAND		
GC-GM	SILTY, CLAYEY GRAVEL SILTY, CLAYEY GRAVEL with SAND	OL	ORGANIC SILT ORGANIC SILT with SAND ORGANIC SILT with GRAVEL SANDY ORGANIC SILT SANDY ORGANIC SILT with GRAVEL GRAVELLY ORGANIC SILT GRAVELLY ORGANIC SILT with SAND
SW	Well-graded SAND Well-graded SAND with GRAVEL		
SP	Poorly graded SAND Poorly graded SAND with GRAVEL	CH	Fat CLAY Fat CLAY with SAND Fat CLAY with GRAVEL SANDY fat CLAY SANDY fat CLAY with GRAVEL GRAVELLY fat CLAY GRAVELLY fat CLAY with SAND
SW-SM	Well-graded SAND with SILT Well-graded SAND with SILT and GRAVEL		
SW-SC	Well-graded SAND with CLAY (or SILTY CLAY) Well-graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	MH	Elastic SILT Elastic SILT with SAND Elastic SILT with GRAVEL SANDY elastic SILT SANDY elastic SILT with GRAVEL GRAVELLY elastic SILT GRAVELLY elastic SILT with SAND
SP-SM	Poorly graded SAND with SILT Poorly graded SAND with SILT and GRAVEL		
SP-SC	Poorly graded SAND with CLAY (or SILTY CLAY) Poorly graded SAND with CLAY and GRAVEL (or SILTY CLAY and GRAVEL)	OH	ORGANIC fat CLAY ORGANIC fat CLAY with SAND ORGANIC fat CLAY with GRAVEL SANDY ORGANIC fat CLAY SANDY ORGANIC fat CLAY with GRAVEL GRAVELLY ORGANIC fat CLAY GRAVELLY ORGANIC fat CLAY with SAND
SM	SILTY SAND SILTY SAND with GRAVEL		
SC	CLAYEY SAND CLAYEY SAND with GRAVEL	OH	ORGANIC elastic SILT ORGANIC elastic SILT with SAND ORGANIC elastic SILT with GRAVEL SANDY elastic ELASTIC SILT SANDY ORGANIC elastic SILT with GRAVEL GRAVELLY ORGANIC elastic SILT GRAVELLY ORGANIC elastic SILT with SAND
SC-SM	SILTY, CLAYEY SAND SILTY, CLAYEY SAND with GRAVEL		
PT	PEAT	OL/OH	ORGANIC SOIL ORGANIC SOIL with SAND ORGANIC SOIL with GRAVEL SANDY ORGANIC SOIL SANDY ORGANIC SOIL with GRAVEL GRAVELLY ORGANIC SOIL GRAVELLY ORGANIC SOIL with SAND
	COBBLES COBBLES and BOULDERS BOULDERS		

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D2435)
CL	Collapse Potential (ASTM D5333)
CP	Compaction Curve (ASTM D1557)
CR	Corrosion, Sulfates, Chlorides (CTM 643; ASTM D4972, ASTM G187, ASTM D4327)
CU	Consolidated Undrained Triaxial (ASTM D4767)
DS	Direct Shear (ASTM D3080)
EI	Expansion Index (ASTM D4829)
M	Moisture Content (ASTM D2216)
OC	Organic Content (ASTM D2974)
P	Permeability (ASTM 5084)
PA	Particle Size Analysis (ASTM D422-63 [2007])
PI	Liquid Limit, Plastic Limit, Plasticity Index (ASTM D4318)
PL	Point Load Index (ASTM D5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (CTM 217)
SG	Specific Gravity (AASHTO T 100)
SL	Shrinkage Limit (ASTM D427)
SW	Swell Potential (ASTM D4546)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D2166) Unconfined Compression - Rock (ASTM D7012)
UU	Unconsolidated Undrained Triaxial (ASTM D2850)
UW	Unit Weight (ASTM D4767, ASTM D7263)
VS	Vane Shear (AASHTO T 223-96 [2004])
-200	200 Wash (ASTM D1140)

SAMPLER GRAPHIC SYMBOLS

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

DRILLING METHOD SYMBOLS

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

WATER LEVEL SYMBOLS

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

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

REPORT TITLE
BORING RECORD LEGEND

PROJECT NAME
Avila Beach CSD WWTF

DATE
10/14/2019

SHEET
1 of 1

LOGGED BY **J. Blanchard**	BEGIN DATE **7-24-19**	COMPLETION DATE **7-24-19**	HAMMER TYPE **--**	BORING NUMBER **19H-1**
FINAL BY **J. King**			BOREHOLE LOCATION (Lat/Long or North/East and Datum) **--/--**	SURFACE ELEVATION **--**
DRILLING METHOD **Hand Excavation**			BOREHOLE LOCATION (Offset, Station, Line) **--**	WEATHER NOTES **Sunny**
DRILLER **--**			LOCATION DESCRIPTION **W end of sludge bed~30 S and ~45' W of corner of Package Plant, 7' N of toe of slope**	BACKFILLED WITH **Native**
DRILL RIG **Posthole & T-Probe**			GROUNDWATER READINGS DURING DRILLING **3.5 ft** AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **4.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	0		ORGANIC SOIL (OL/OH); Snail shells and sludge.												
			Lean CLAY (CL); medium stiff; olive brown; moist.												
	1		COBBLES, GRAVEL, and snail shells common.												
			Fat CLAY (CH); dark gray; moist to wet.												
	2														
			Few GRAVEL; trace fines.												
	3		Wet.												
															▽
	4		Bottom of borehole at 4.0 ft bgs												
	5														
	6														
	7														
	8														
	9														
	10														

This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.

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Geotechnical • Geological • Construction Services

PROJECT NAME **Avila Beach CSD WWTF**	
PROJECT NUMBER **219-201**	
BORING NUMBER **19H-1**	
REVISION DATE **10/14/2019**	SHEET **1 of 1**

LOGGED BY	BEGIN DATE	COMPLETION DATE	HAMMER TYPE	BORING NUMBER
R. Hooke	**3-12-21**	**3-12-21**		**21IN-01**

FINAL BY **J. King**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **--/--**	SURFACE ELEVATION **--**
DRILLING METHOD **4" Hand Auger**	BOREHOLE LOCATION (Offset, Station, Line) **--**	WEATHER NOTES **sunny, cool**
DRILLER **John Madonna Construction**	LOCATION DESCRIPTION **15' west of water box, 3' south of Bob Jones Trail fence**	BACKFILLED WITH **Infiltration**
DRILL RIG **4" Hand Auger**	GROUNDWATER READINGS — DURING DRILLING: **Not Encountered**; AFTER DRILLING (DATE):	TOTAL DEPTH OF BORING **3.3 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	0	●	9" AGGREGATE BASE.										▽		
	1	▨	CLAYEY SAND with GRAVEL (SC); dark brown; moist; fine to coarse, subangular GRAVEL; fine to coarse SAND.										▽		
	2	▨											▽		
	3	▨											▽		
			Bottom of borehole at 3.3 ft bgs												
	4														
	5		This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.												
	6														
	7														
	8														
	9														
	10														
	11														
	12														
	13														
	14														
	15														
	16														
	17														
	18														
	19														
	20														
	21														
	22														
	23														
	24														
	25														

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PROJECT NAME **Avila Beach WWTP Improvements Final Design**	
PROJECT NUMBER **221-020**	
BORING NUMBER **21IN-01**	
REVISION DATE **3/18/2021**	SHEET **1 of 1**

LOGGED BY **R. Hooke**	BEGIN DATE **3-12-21**	COMPLETION DATE **3-12-21**	HAMMER TYPE	BORING NUMBER **21IN-02**
FINAL BY **J. King**			BOREHOLE LOCATION (Lat/Long or North/East and Datum) **--/--**	SURFACE ELEVATION **--**
DRILLING METHOD **4" Hand Auger**			BOREHOLE LOCATION (Offset, Station, Line) **--**	WEATHER NOTES **cool, cloudy**
DRILLER **John Madonna Construction**			LOCATION DESCRIPTION **5' north of driveway, 25' west of water box**	BACKFILLED WITH **Infiltration**
DRILL RIG **4" Hand Auger**			GROUNDWATER READINGS — DURING DRILLING **Not Encountered** — AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **3.5 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	0		1" AGGREGATE BASE.												
	1		CLAYEY SAND (SC); brown; moist; fine to medium, subangular GRAVEL.												
	2		Light yellowish brown.												
	3		SANDY lean CLAY (CL); black/green mottled; moist; fine GRAVEL.												
	4		Bottom of borehole at 3.5 ft bgs												
	5														
	6		This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.												
	7														
	8														
	9														
	10														
	11														
	12														
	13														
	14														
	15														
	16														
	17														
	18														
	19														
	20														
	21														
	22														
	23														
	24														
	25														

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PROJECT NAME **Avila Beach WWTP Improvements Final Design**	
PROJECT NUMBER **221-020**	
BORING NUMBER **21IN-02**	
REVISION DATE **3/18/2021**	SHEET **1 of 1**

LOGGED BY **R. Hooke** · BEGIN DATE **3-12-21** · COMPLETION DATE **3-12-21**	HAMMER TYPE	BORING NUMBER **21IN-03**
FINAL BY **J. King**	BOREHOLE LOCATION (Lat/Long or North/East and Datum) **--/--**	SURFACE ELEVATION **--**
DRILLING METHOD **4" Hand Auger**	BOREHOLE LOCATION (Offset, Station, Line) **--**	WEATHER NOTES **cool, cloudy**
DRILLER **John Madonna Construction**	LOCATION DESCRIPTION **10' south of Bob Jones Trail fence, 13' from reactor**	BACKFILLED WITH **Infiltration**
DRILL RIG **4" Hand Auger**	GROUNDWATER READINGS · DURING DRILLING **Not Encountered** · AFTER DRILLING (DATE)	TOTAL DEPTH OF BORING **5.0 ft**

ELEVATION (ft)	DEPTH (ft)	Material Graphics	DESCRIPTION	Sample Location	Sample Number	Blows per 6 in.	Blows per foot	Recovery (%)	RQD (%)	Moisture Content (%)	Dry Unit Weight (pcf)	Shear Strength (ksf)	Drilling Method	Casing Depth	Remarks
	0	▨	CLAYEY SAND (SC); brown; moist; fine to medium, subangular GRAVEL; fine to coarse SAND.										▽		
	1	▨											▽		
	2	▨											▽		
	3	▨	Roots.										▽		
	4	▨	Dark brown; organics. Brown.										▽		
	5		Bottom of borehole at 5.0 ft bgs												
	6														
	7		This Boring Record was developed in accordance with the Caltrans Soil & Rock Logging, Classification, and Presentation Manual (2010) except as noted on the Soil or Rock Legend or below.												
	8														
	9														
	10														
	11														
	12														
	13														
	14														
	15														
	16														
	17														
	18														
	19														
	20														
	21														
	22														
	23														
	24														
	25														

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PROJECT NAME **Avila Beach WWTP Improvements Final Design**	
PROJECT NUMBER **221-020**	
BORING NUMBER **21IN-03**	
REVISION DATE **3/18/2021**	SHEET **1 of 1**

Infiltration Test

References: Native Soil Assessment for Small Infiltration-Based Stormwater Control Measures, Central Coast LID Initiative, December 2013
Caltrans Test 749 and 750

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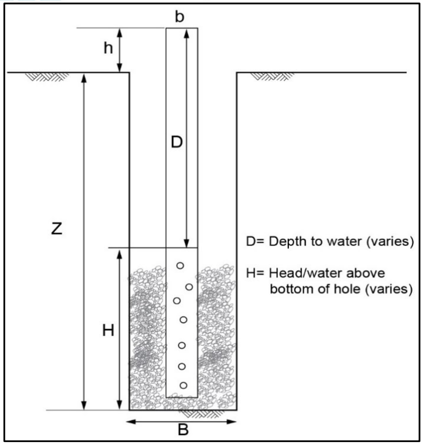
Project No.:	221-020	**Percolation Test No.:**	21I-01
Project Name:	Avila CSD - WWTP Improvements	**Surface Elevation (ft):**	--
Project MGR:	J. King	**Completion Depth, Z (ft):**	3.25
Tested By:	R. Hooke	**Pipe Above Grade, h (ft):**	0.5
Excavation Method:	4" Hand Auger	**Pipe Diameter, b (in):**	2
Weather:	cool, sunny	**Hole Diameter, B (in):**	6
Installation Date:	3/12/2021	**Backfill:**	Native
Test Date:	3/15/2021		



Constant Head Test Data

Time (min)	30
Volume of Water (gal)	11.3
Volume of Water(in^3)	2610
Rate (gal/hr)	23
Rate (in^3/hr)	5221

Test Notes

Maintained head of approximately 1 foot during test, 1.3 gallons to achieve head

Falling Head Percolation Test Data Table

Time	H (inches)	Δ (inches)	ΔT (minutes)	R (min/inch)
9:46	17.4	0.0	---	---
9:47	15.0	2.4	1	0.42
9:48	9.0	6.0	1	0.17
9:49	6.6	2.4	1	0.42
---	---	---	---	---
9:52	18.0	0.0	---	---
9:53	13.8	4.2	1	0.24
9:54	10.2	3.6	1	0.28
---	---	---	---	---
9:56	17.4	0.0	---	---
9:57	15.0	2.4	1	0.42
9:58	11.4	3.6	1	0.28

Average Percolation Rate, R:	0.32	Minutes/Inch
Average Percolation Rate, R:	190.02	Inches/Hour
Equivalent Unlined 12-inch diameter Test Rate:	1.15	Minutes/Inch
Equivalent Unlined 12-inch diameter Test Rate:	52.25	Inches/Hour

LEGEND:
H = Water head in test hole
Δ = Drop in water level between observations
T = Time interval between observations
R = Percolation Rate

Infiltration Test

References: Native Soil Assessment for Small Infiltration-Based Stormwater Control Measures, Central Coast LID Initiative, December 2013
Caltrans Test 749 and 750

Project No.:	221-020	**Percolation Test No.:**	21I-02
Project Name:	Avila CSD - WWTP Improvements	**Surface Elevation (ft):**	--
Project MGR:	J. King	**Completion Depth, Z (ft):**	3.5
Tested By:	R. Hooke	**Pipe Above Grade, h (ft)**	1.2
Excavation Method:	4" Hand Auger	**Pipe Diameter, b (in)**	2
Weather:	cool, sunny	**Hole Diameter, B (in):**	6
Installation Date:	3/12/2021	**Backfill**	Native
Test Date:	3/15/2021		

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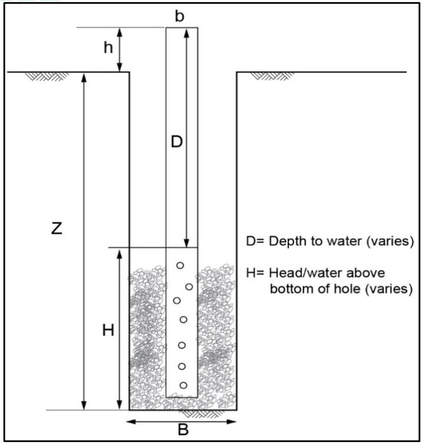

Constant Head Test Data

Time (min)	30
Volume of Water (gal)	0
Volume of Water(in^3)	0
Rate (gal/hr)	0
Rate (in^3/hr)	0

Test Notes

Maintained head of approximately 1.5 feet during test, 1.4 gallons to achieve head

Falling Head Percolation Test Data Table

Time	H (inches)	Δ (inches)	ΔT (minutes)	R (min/inch)
11:27	32.4	0.0	---	---
12:28	31.2	1.2	61	50.83
12:58	30.6	0.6	30	50.00
13:28	30.0	0.6	30	50.00
14:28	29.4	0.6	60	100.00

Average Percolation Rate, R:	50.28	Minutes/Inch
Average Percolation Rate, R:	1.19	Inches/Hour
Equivalent Unlined 12-inch diameter Test Rate:	182.86	Minutes/Inch
Equivalent Unlined 12-inch diameter Test Rate:	0.33	Inches/Hour

LEGEND:
H = Water head in test hole
Δ = Drop in water level between observations
T = Time interval between observations
R = Percolation Rate

Infiltration Test

References: Native Soil Assessment for Small Infiltration-Based Stormwater Control Measures, Central Coast LID Initiative, December 2013
Caltrans Test 749 and 750

Project No.:	221-020	**Percolation Test No.:**	21I-03
Project Name:	Avila CSD - WWTP Improvements	**Surface Elevation (ft):**	--
Project MGR:	J. King	**Completion Depth, Z (ft):**	5
Tested By:	R. Hooke	**Pipe Above Grade, h (ft)**	0.5
Excavation Method:	4" Hand Auger	**Pipe Diameter, b (in)**	2
Weather:	sunny, cool	**Hole Diameter, B (in):**	6
Installation Date:	3/12/2021	**Backfill**	Native
Test Date:	3/15/2021		

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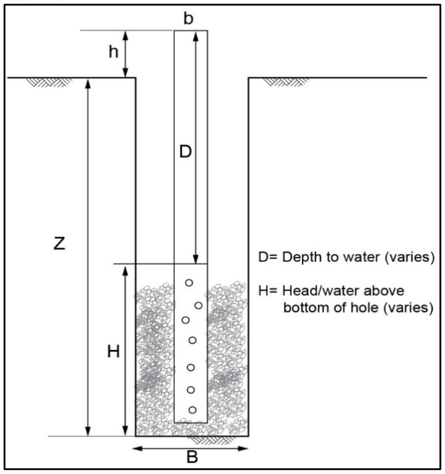


Constant Head Test Data

Time (min)	30
Volume of Water (gal)	28.2
Volume of Water(in^3)	6514
Rate (gal/hr)	56.40
Rate (in^3/hr)	13028

Test Notes

Maintained head of approximately 1.7 feet during test, 26 gallons to achieve head

Falling Head Percolation Test Data Table

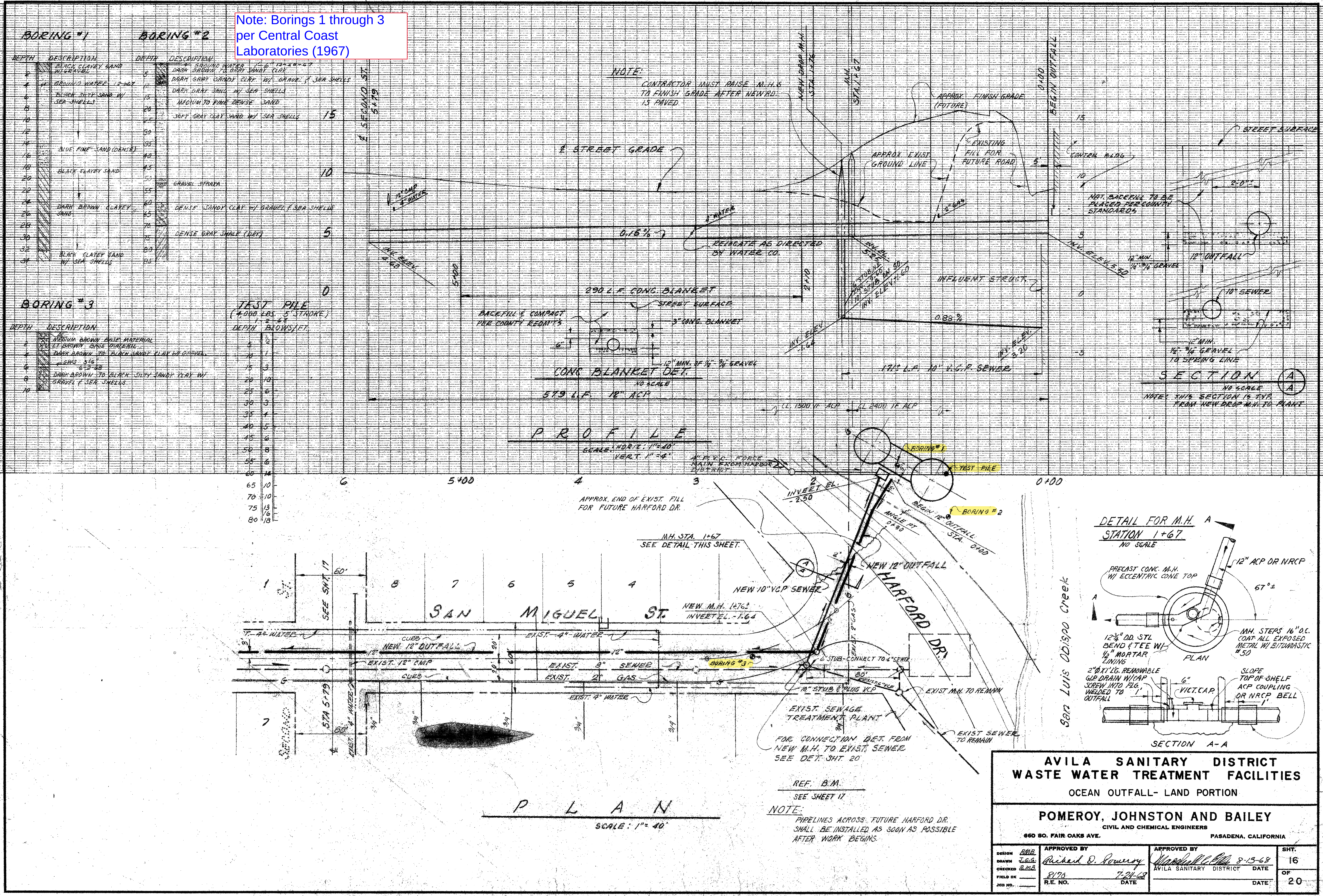
Time	H (inches)	Δ (inches)	ΔT (minutes)	R (min/inch)
12:27	26.4	0.0	---	---
12:28	21.6	4.8	1	0.21
12:29	18.0	3.6	1	0.28
12:30	17.4	0.6	1	1.67
12:33	16.2	1.2	3	2.50
12:35	15.6	0.6	2	3.33
12:40	14.4	1.2	5	4.17
12:45	13.2	1.2	5	4.17
12:50	11.4	1.8	5	2.78
12:55	10.2	1.2	5	4.17
13:00	8.4	1.8	5	2.78
---	---	---	---	---
13:09	30.0	0.0	---	---
13:14	17.4	12.6	5	0.40
13:15	16.2	1.2	1	0.83
13:17	15.6	0.6	2	3.33
13:22	14.4	1.2	5	4.17
13:27	12.6	1.8	5	2.78
13:33	10.8	1.8	6	3.33
---	---	---	---	---
13:43	25.2	0.0	---	---
13:44	18.0	7.2	1	0.14
13:45	17.4	0.6	1	1.67
13:50	15.6	1.8	5	2.78
13:55	13.7	1.9	5	2.60
14:00	12.0	1.7	5	2.98
14:05	11.4	0.6	5	8.33

Average Percolation Rate, R:	3.75	Minutes/Inch
Average Percolation Rate, R:	15.99	Inches/Hour
Equivalent Unlined 12-inch diameter Test Rate:	13.65	Minutes/Inch
Equivalent Unlined 12-inch diameter Test Rate:	4.40	Inches/Hour

LEGEND:
H = Water head in test hole
Δ = Drop in water level between observations
T = Time interval between observations
R = Percolation Rate

APPENDIX C - PREVIOUS STUDIES

Note: Borings 1 through 3 per Central Coast Laboratories (1967)



BORING #1

DEPTH	DESCRIPTION
2	BLACK CLAYEY SAND W/GRAVEL
4	GROUND WATER 12-1-67
6	BLACK SILTY SAND W/ SEA SHELLS
8	
10	
12	
14	
16	BLUE FINE SAND (DENSE)
18	BLACK CLAYEY SAND
20	
22	
24	
26	DARK BROWN CLAYEY SAND
28	
30	
32	
34	BLACK CLAYEY SAND W/ SEA SHELLS

BORING #2

DEPTH	DESCRIPTION
	GROUND WATER 1'-6" 12-28-67 DARK BROWN TO GRAY SANDY CLAY
5	DARK GRAY SANDY CLAY W/ GRAVEL & SEA SHELLS
10	DARK GRAY SAND W/ SEA SHELLS
15	MEDIUM TO FINE DENSE SAND
20	SOFT GRAY CLAY SAND W/ SEA SHELLS
25	
30	
35	
40	
45	
50	GRAVEL STRATA
55	
60	DENSE SANDY CLAY W/ GRAVEL & SEA SHELLS
65	
70	DENSE GRAY SHALE (DRY)
75	
80	
85	

BORING #3

DEPTH	DESCRIPTION
	AC MEDIUM BROWN BASE MATERIAL
2	LT BROWN BASE MATERIAL
4	DARK BROWN TO BLACK SANDY CLAY W/ GRAVEL.
	GWS 5'-6" 6-2-68
6	
8	DARK BROWN TO BLACK SILTY SANDY CLAY W/ GRAVEL & SEA SHELLS.
10	

TEST PILE

(4,000 LBS. 5' STROKE)
1-2-68

DEPTH	BLOWS/FT.
	½
5	1
10	3
15	10
20	6
25	3
30	4
35	5
40	6
45	8
50	6
55	14
60	10
65	10
70	13
75	16
80	18

NOTE: CONTRACTOR MUST RAISE M.H.S TO FINISH GRADE AFTER NEW RD. IS PAVED

NOTE: PIPELINES ACROSS FUTURE HARFORD DR. SHALL BE INSTALLED AS SOON AS POSSIBLE AFTER WORK BEGINS.

AVILA SANITARY DISTRICT
WASTE WATER TREATMENT FACILITIES
OCEAN OUTFALL- LAND PORTION

POMEROY, JOHNSTON AND BAILEY
CIVIL AND CHEMICAL ENGINEERS
660 SO. FAIR OAKS AVE. PASADENA, CALIFORNIA

DESIGN RMB
DRAWN J.G.G.
CHECKED RMB
FIELD BK
JOB NO.

APPROVED BY Richard D. Pomeroy
8170 7-24-68
R.E. NO. DATE

APPROVED BY 8-13-68
AVILA SANITARY DISTRICT DATE
DATE

SHT. 16
OF 20

EARTH SYSTEMS CONSULTANTS
Northern California, Pacific Geoscience Division

Boring No. 1

LOGGED BY: DB
DRILL RIG: Mobile B-53
AUGER TYPE: 6" Hollow Stem

PAGE 1 of 4
JOB NO.: NGS08167W01
DATE: 4/28/92

Avila Beach
Wastewater Treatment Plant
San Luis Obispo County, California

DEPTH (in feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA				
				INTERVAL	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
0	SC	▨	Moist, dense, brown Clayey SAND with gravel (fill)					
1	CL	▨	Moist, very stiff, dark grey Sandy Lean CLAY with gravel (fill)	2.0-3.5	■	104.1	18.8	12/19/25
3	CL	▨	Very moist, stiff, dark blue grey Sandy Lean CLAY, occasional gravel (native)	3.0-5.0	○			
				5.0-6.5	■	87.8	30.0	5/6/8
10	SC	▨	▼ Wet, loose, dark grey Clayey SAND, occasional gravel	10.0-11.5	■	No Return		2/3/3
			Silty SAND with gravel (SM) per lab tests	11.5-13.0	■	85.9	35.7	3/4/5
16			With gravel	15.0-16.5	■	No Return		3/3/3
16.5			Occasional gravel	16.5-18.0	■	91.7	33.5	3/6/6
18	SP	░	Wet, loose, grey Poorly-Graded SAND, fine to medium grained					
20			Medium dense	20.0-21.5	■	93.7	28.9	11/15/15
25				25.0-26.5	●			7/8/17

LEGEND: ■ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

Note: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

EARTH SYSTEMS CONSULTANTS
Northern California, Pacific Geoscience Division

Boring No. 1

LOGGED BY: DB
DRILL RIG: Mobile B-53
AUGER TYPE: 6" Hollow Stem

PAGE 2 of 4
JOB NO.: NGS08167W01
DATE: 4/28/92

Avila Beach
Wastewater Treatment Plant
San Luis Obispo County, California

DEPTH (in feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
25		⁙	Poorly-Graded SAND, as above					
26								
27								
28								
29	SC	▨	Wet, loose, grey Clayey SAND, abundant shell fragments					
30								
31				30.0-31.5	●			2/2/2
32								
33								
34								
35								
36			Dark grey brown	35.0-36.5	●			2/3/3
37								
38								
39								
40								
41			Grey, with gravel	40.0-41.5	●			5/10/11
42								
43								
44	SW	⁙	Wet, medium dense, grey Well-Graded SAND, occasional gravel					
45								
46								
47								
48								
49								
50								
				50.0-51.5	▬	91.4	31.3	12/9/14

LEGEND: ▬ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

Note: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

EARTH SYSTEMS CONSULTANTS
Northern California, Pacific Geoscience Division

Boring No. 1

LOGGED BY: DB
DRILL RIG: Mobile B-53
AUGER TYPE: 6" Hollow Stem

PAGE 3 of 4
JOB NO.: NGS08167W01
DATE: 4/28/92

Avila Beach
Wastewater Treatment Plant
San Luis Obispo County, California

DEPTH (in feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	SAMPLE DATA INTERVAL	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
50		░	Well-Graded SAND, as above					
51	SP	░	Wet, medium dense, grey Poorly-Graded SAND					
52		░						
53		░						
54		░						
55		░						
56		░						
57		░						
58		░						
59		░						
60		░						
61		░		60.0-61.5	●			9/11/8
62	SC	▨	Wet, medium dense, grey Clayey SAND with gravel					
63		▨						
64	SW	░	Wet, dense, grey Well-Graded SAND with gravels					
65		░						
66		░						
67		░						
68		░						
69		░						
70		░						
71		░		70.0-71.5	●			11/20/30
72		░						
73		░						
74		░						
75		░						

LEGEND: ■ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

Note: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

EARTH SYSTEMS CONSULTANTS
Northern California, Pacific Geoscience Division

Boring No. 1

LOGGED BY: DB
DRILL RIG: Mobile B-53
AUGER TYPE: 6" Hollow Stem

PAGE 4 of 4
JOB NO.: NGS08167W01
DATE: 4/28/92

Avila Beach
Wastewater Treatment Plant
San Luis Obispo County, California

DEPTH (in feet)	USCS CLASS	SYMBOL	SOIL DESCRIPTION	INTERVAL	SAMPLE TYPE	DRY DENSITY (pcf)	MOISTURE (%)	BLOWS PER 6 IN.
75		░	Well-Graded SAND, as above					
76	SC	▨	Wet, medium dense, grey Clayey SAND					
77		▨						
78		▨						
79		▨						
80		▨		80.0-81.5	●			6/8/15
81		▨	Thin interebedded sand lenses					
82		▨						
83		▨						
84		▨						
85		▨						
86		▨						
87		▨						
88		▨						
89		▨						
90		▨						
91		▨	Dense, with gravel	90.0-91.5	●			20/35/35
92			END OF BORING @ 91.5'. Subsurface water encountered @ 10'.					
93								
94								
95								
96								
97								
98								
99								
100								

LEGEND: ■ Ring Sample ○ Grab Sample ▭ Shelby Tube Sample ● SPT

Note: This log of subsurface conditions is a simplification of actual conditions encountered. It applies at the location and time of drilling. Subsurface conditions may differ at other locations and times.

PROJECT: AVILA BEACH WASTEWATER TREATMENT PLANT EXPANSION
JOB NO.: NGS08167W01
DATE: June 4, 1992

BULK DENSITY TEST RESULTS

BORING NO.	DEPTH feet	MOISTURE CONTENT, %	WET DENSITY, pcf	DRY DENSITY, pcf
1	3.0-3.5	18.8	123.6	104.1
1	6.0-6.5	30.0	114.1	87.8
1	12.5-13.0	35.7	116.6	85.9
1	17.5-18.0	33.5	122.5	91.7
1	21.0-21.5	28.9	120.8	93.7
1	51.0-51.5	31.3	120.0	91.4

EXPANSION INDEX TEST RESULTS

BORING NO.	DEPTH feet	EXPANSION INDEX
1	3-5	31

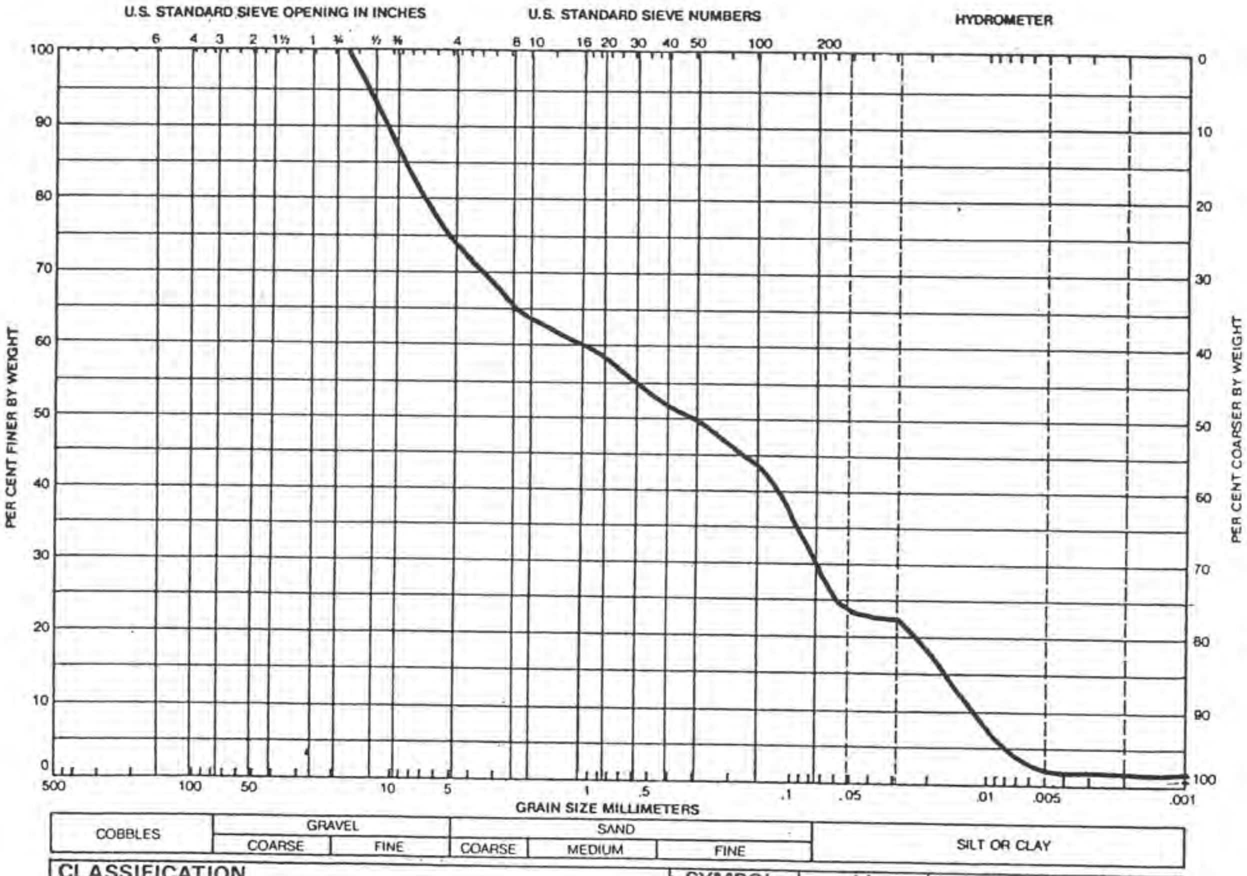
PLASTICITY INDEX TEST RESULTS

BORING NO.	DEPTH feet	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
1	6.0-6.5	37	20	17
1	12.5-13.0	45	31	14

PROJECT: AVILA BEACH WASTEWATER TREATMENT PLANT EXPANSION
JOB NUMBER: NGS08167W01
SAMPLE I.D.: Boring #1 @ 12.5-13'
DATE: 3-Jun-92

PARTICLE SIZE ANALYSIS

Sieve size	% Retained (Cumulative)	% Passing (Cumulative)
3/4"	0	100
1/2"	6	94
3/8"	13	87
#4	26	74
#8	35	65
#16	40	60
#30	45	55
#50	50	50
#100	56	44
#200	70	30
53 microns		24
28 microns		23
5 microns		1
2.5 microns		1
Colloids		1

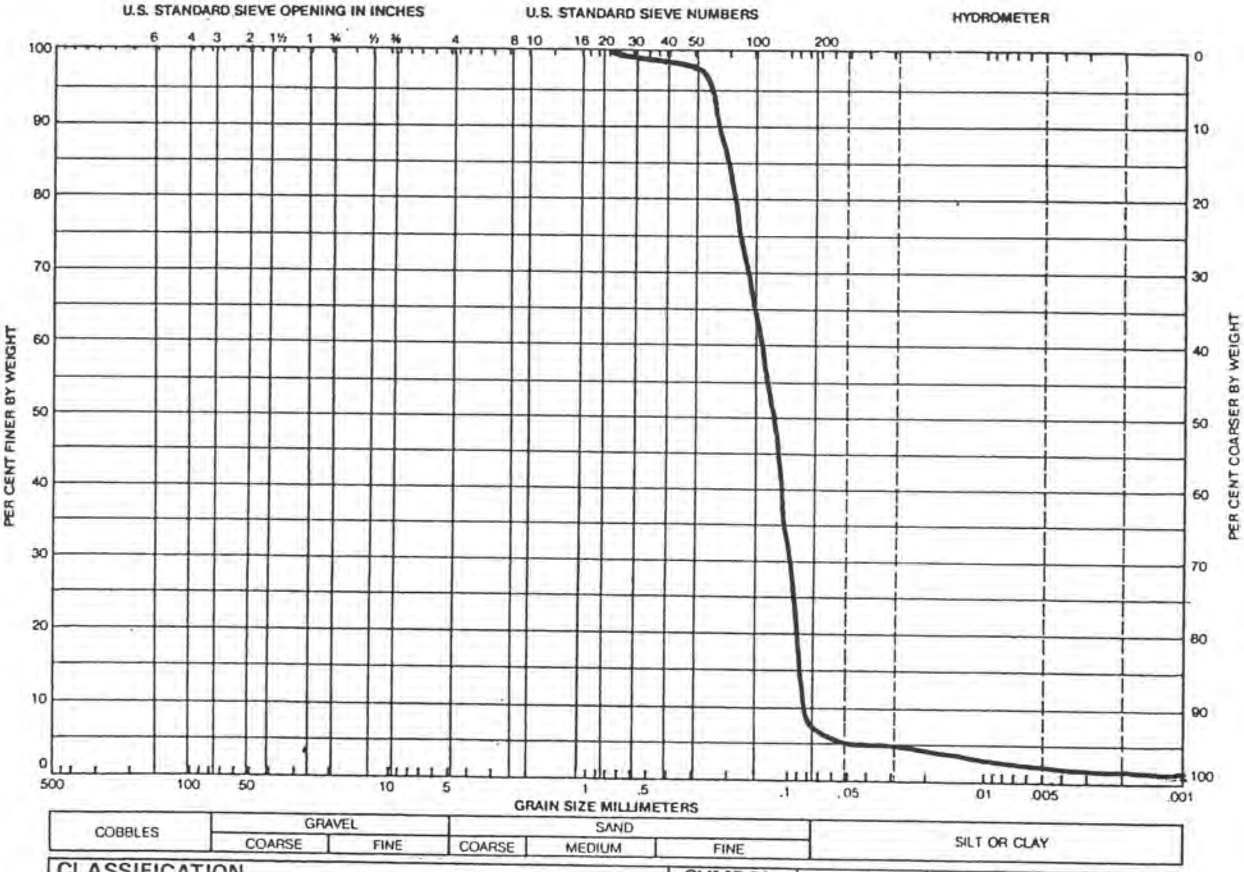


CLASSIFICATION	SYMBOL	LL.	PL	PI
SILTY SAND WITH GRAVEL	SM	45	31	14

PROJECT: AVILA BEACH WASTEWATER TREATMENT PLANT EXPANSION
JOB NUMBER: NGS08167W01
SAMPLE I.D.: Boring #1 @ 21-21.5'
DATE: 3-Jun-92

PARTICLE SIZE ANALYSIS

Sieve size	% Retained (Cumulative)	% Passing (Cumulative)
#16	0	100
#30	1	99
#50	2	98
#100	35	65
#200	93	7
53 microns		5
28 microns		5
5 microns		2
2.5 microns		1
Colloids		1

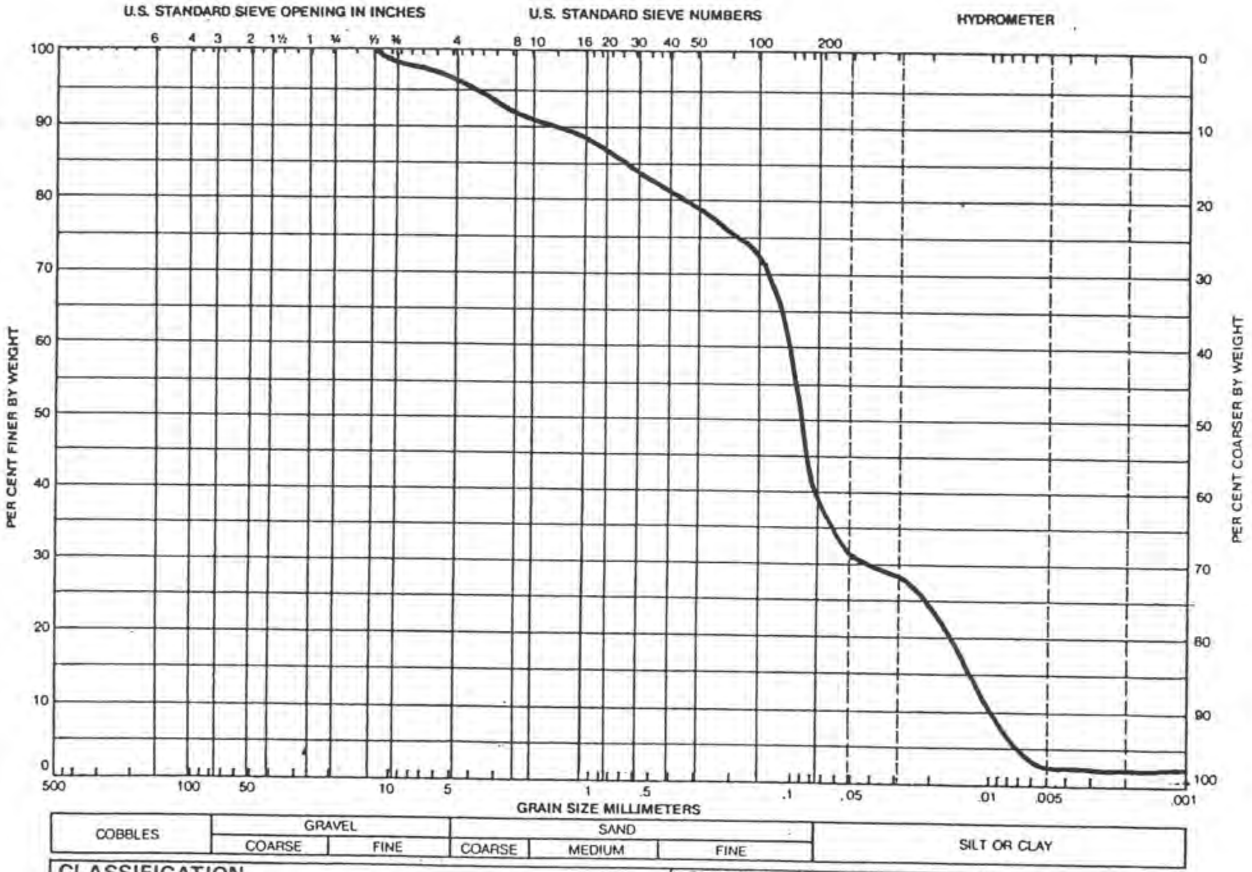


CLASSIFICATION	SYMBOL	LL	PL	PI
POORLY GRADED SAND WITH SILT	SP-SM			

PROJECT: AVILA BEACH WASTEWATER TREATMENT PLANT EXPANSION
JOB NUMBER: NGS08167W01
SAMPLE I.D.: Boring #1 @ 41-41.5'
DATE: 3-Jun-92

PARTICLE SIZE ANALYSIS

Sieve size	% Retained (Cumulative)	% Passing (Cumulative)
1/2"	0	100
3/8"	2	98
#4	4	96
#8	8	92
#16	11	89
#30	16	84
#50	21	79
#100	28	72
#200	61	39
53 microns		31
28 microns		27
5 microns		2
2.5 microns		2
Colloids		2



CLASSIFICATION	SYMBOL	LL.	PL	PI
CLAYEY SAND	SC			

PROJECT: AVILA BEACH WASTEWATER TREATMENT PLANT EXPANSION
JOB NUMBER: NGS08167W01
SAMPLE I.D.: Boring #1 @ 51-51.5'
DATE: 3-Jun-92

PARTICLE SIZE ANALYSIS

Sieve size	% Retained (Cumulative)	% Passing (Cumulative)
1/2"	0	100
3/8"	2	98
#4	20	80
#8	20	80
#16	28	72
#30	39	61
#50	62	38
#100	77	23
#200	86	14
53 microns		13
28 microns		11
5 microns		1
2.5 microns		1
Colloids		1

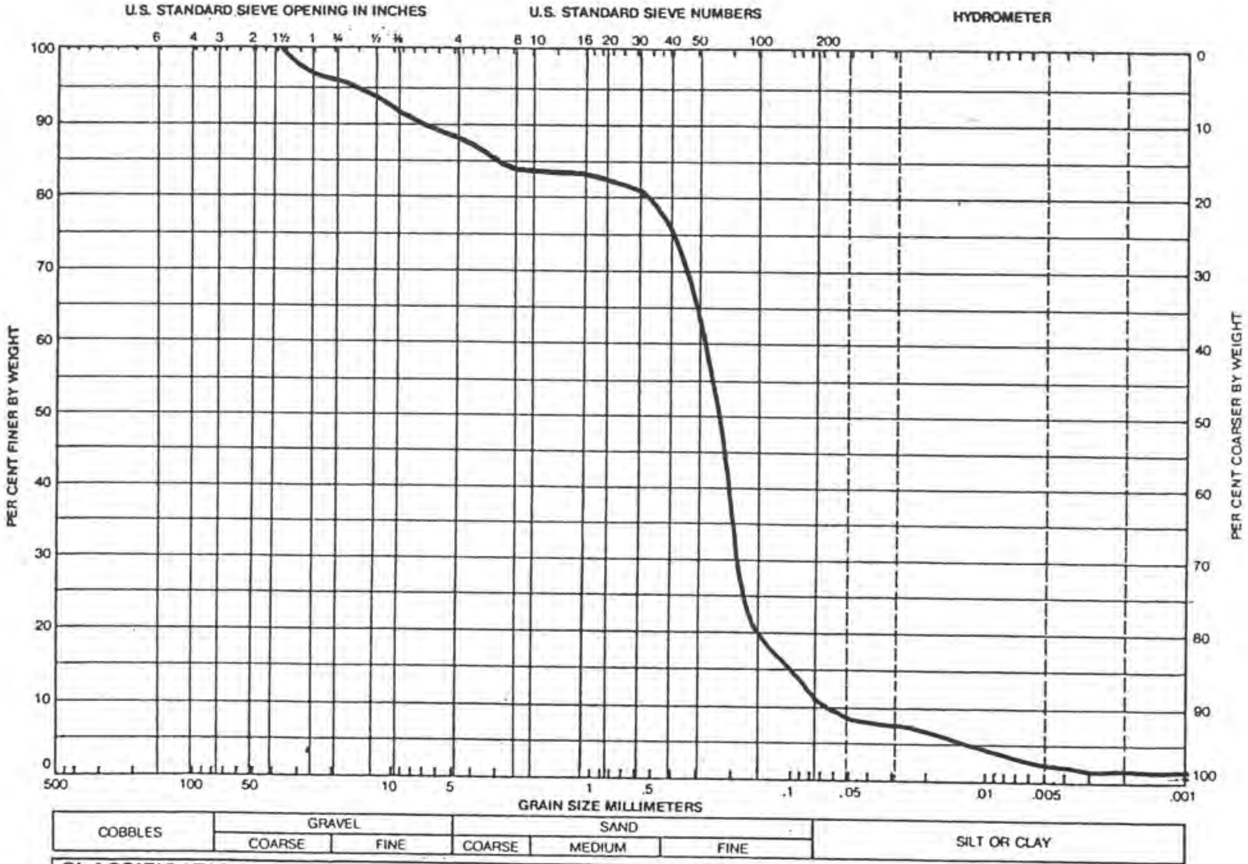
COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

CLASSIFICATION	SYMBOL	LL.	PL	PI
SILTY SAND WITH GRAVEL	SM			

PROJECT: AVILA BEACH WASTEWATER TREATMENT PLANT EXPANSION
JOB NUMBER: NGS08167W01
SAMPLE I.D.: Boring #1 @ 61-61.5'
DATE: 3-Jun-92

PARTICLE SIZE ANALYSIS

Sieve size	% Retained (Cumulative)	% Passing (Cumulative)
1.5"	0	100
1"	3	97
3/4"	4	96
1/2"	6	94
3/8"	8	92
#4	12	88
#8	16	84
#16	17	83
#30	19	81
#50	37	63
#100	80	20
#200	89	11
53 microns		9
28 microns		8
5 microns		2
2.5 microns		1
Colloids		1

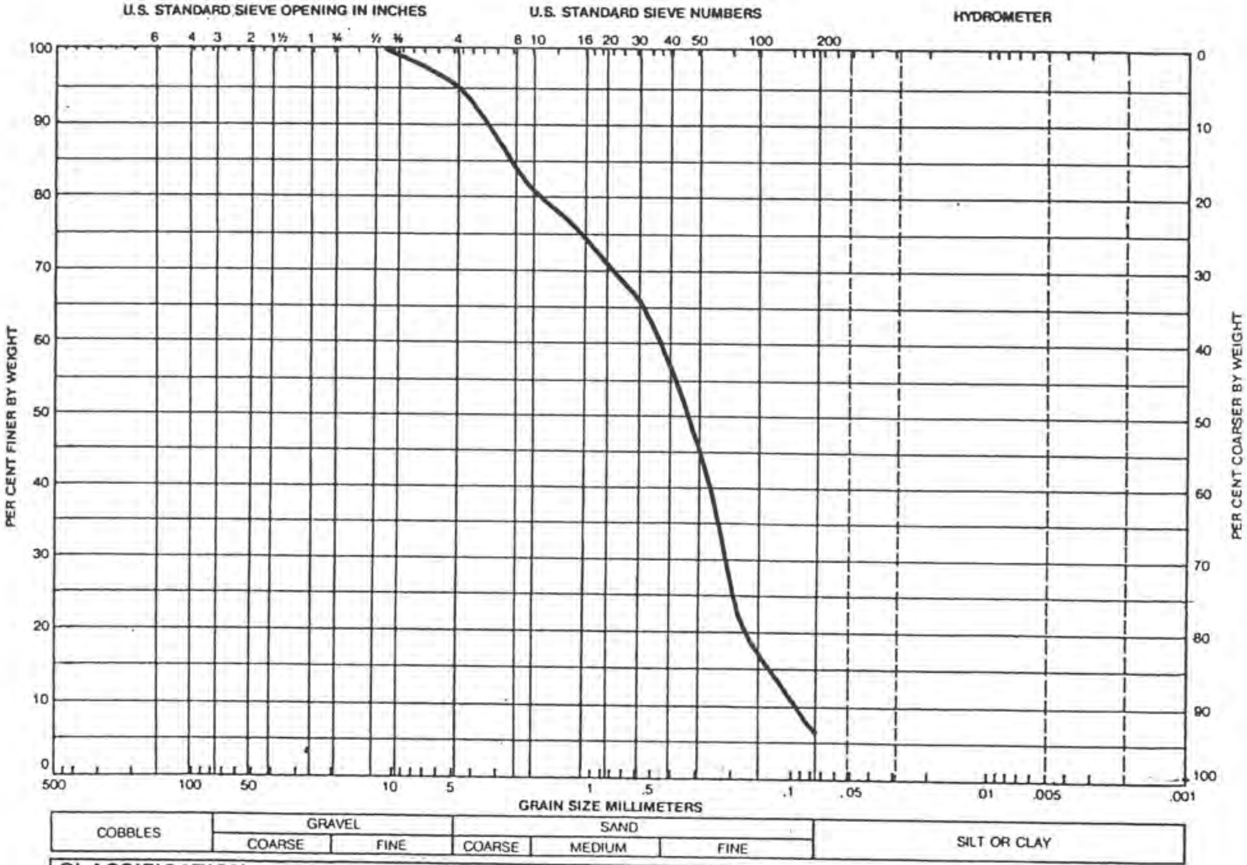


CLASSIFICATION	SYMBOL	LL	PL	PI
POORLY GRADED SAND WITH SILT	SP-SM			

PROJECT: AVILA BEACH WASTEWATER TREATMENT PLANT EXPANSION
JOB NUMBER: NGS08167W01
SAMPLE I.D.: Boring #1 @ 71-71.5'
DATE: 3-Jun-92

PARTICLE SIZE ANALYSIS

Sieve size	% Retained (Cumulative)	% Passing (Cumulative)
1/2"	0	100
3/8"	1	99
#4	5	95
#8	16	84
#16	25	75
#30	34	66
#50	53	47
#100	83	17
#200	93	7

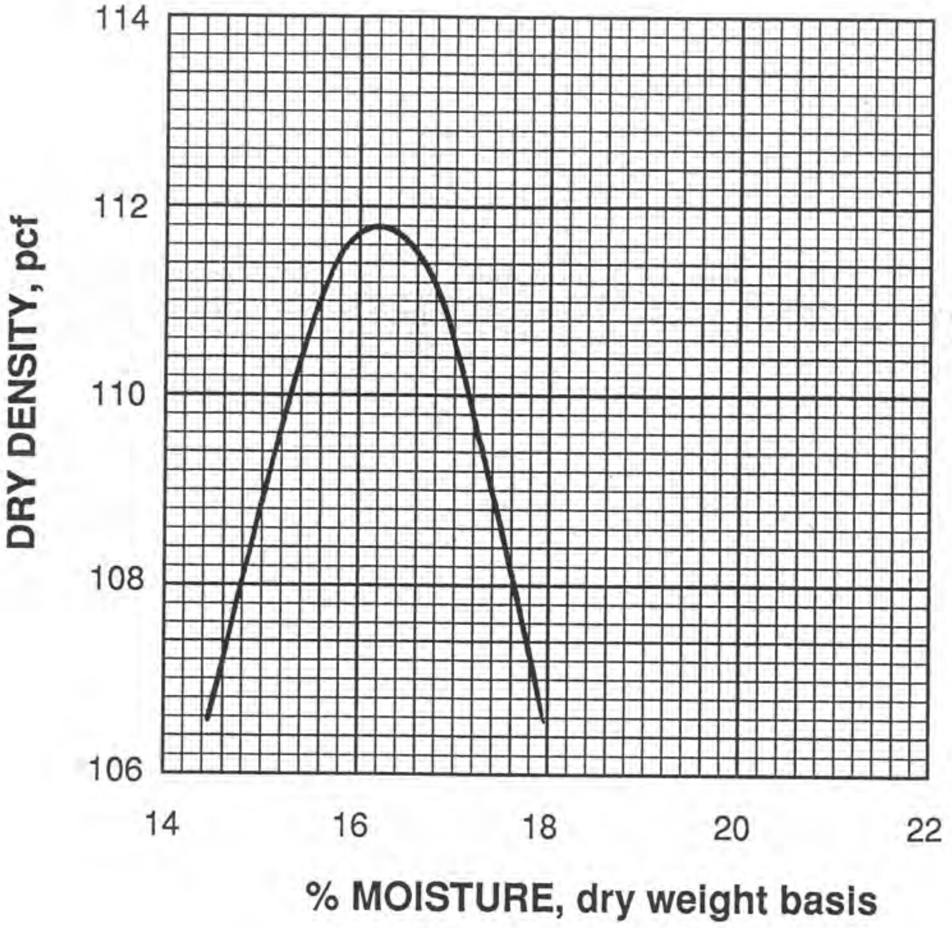


CLASSIFICATION	SYMBOL	LL	PL	PI
POORLY GRADED SAND WITH SILT	SP-SM			

MAXIMUM DENSITY/OPTIMUM MOISTURE CURVE

ASTM D-1557 METHOD: A

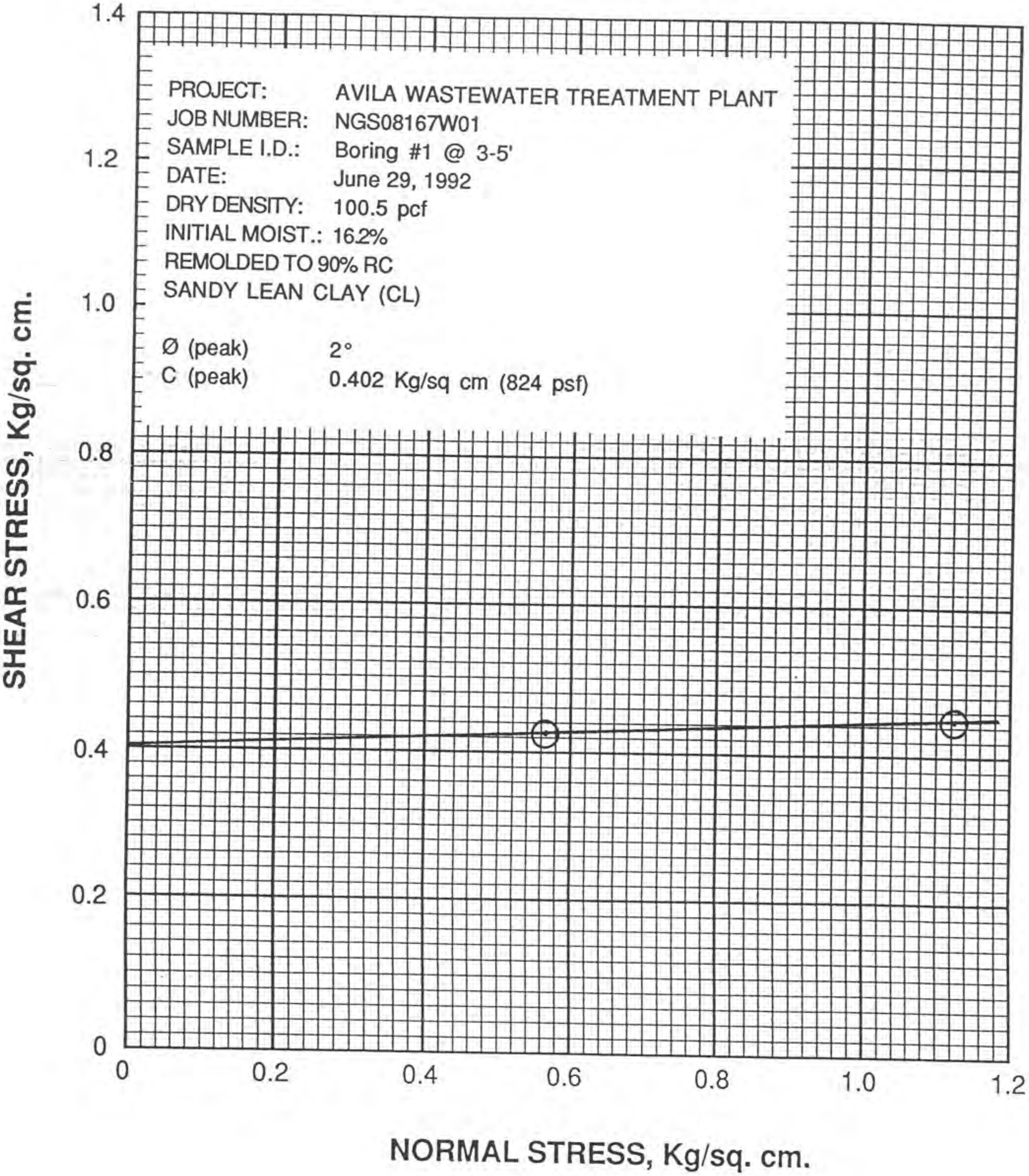
PROJECT: AVILA BEACH WASTEWATER TREATMENT PLANT EXPANSION
DATE: 6/1/92
JOB NUMBER: NGS08167W01



SAMPLE I. D.: Boring # 1 @ 3-5'
SOIL DESCRIPTION: DARK BROWN SANDY LEAN CLAY W/ GRAVEL(CL)
MAXIMUM DENSITY, pcf: 111.7
OPTIMUM MOISTURE, percent: 16.2

DIRECT SHEAR

SHEAR VS. NORMAL STRESS



PROJECT: AVILA WASTEWATER TREATMENT PLANT
JOB NUMBER: NGS08167W01
SAMPLE I.D.: Boring #1 @ 3-5'
DATE: June 29, 1992
DRY DENSITY: 100.5 pcf
INITIAL MOIST.: 16.2%
REMOLDED TO 90% RC
SANDY LEAN CLAY (CL)

Ø (peak) 2°
C (peak) 0.402 Kg/sq cm (824 psf)

UNCONFINED COMPRESSION ON COHESIVE SOIL

ASTM D2166

JOB NAME: AVILA BEACH WASTEWATER TREATMENT PLANT **JOB NO.:** NGS08167W01

SAMPLE I.D.: Boring #1 @ 6-6.5' **DATE:** 6/29/92

SOIL CLASSIFICATION: DARK BROWN SANDY LEAN CLAY (CL) **TESTED BY:** TR

UNDISTURBED: X REMOLDED: N / A RELATIVE COMPACTION: N / A

LENGTH, inches: 3.00 DIAMETER, inches: 2.375 HT/DIA RATIO: 1.26

WET DENSITY, pcf: 114.1 DRY DENSITY, pcf: 87.8

MOISTURE: 30.0% DEGREE OF SATURATION: 89.9%

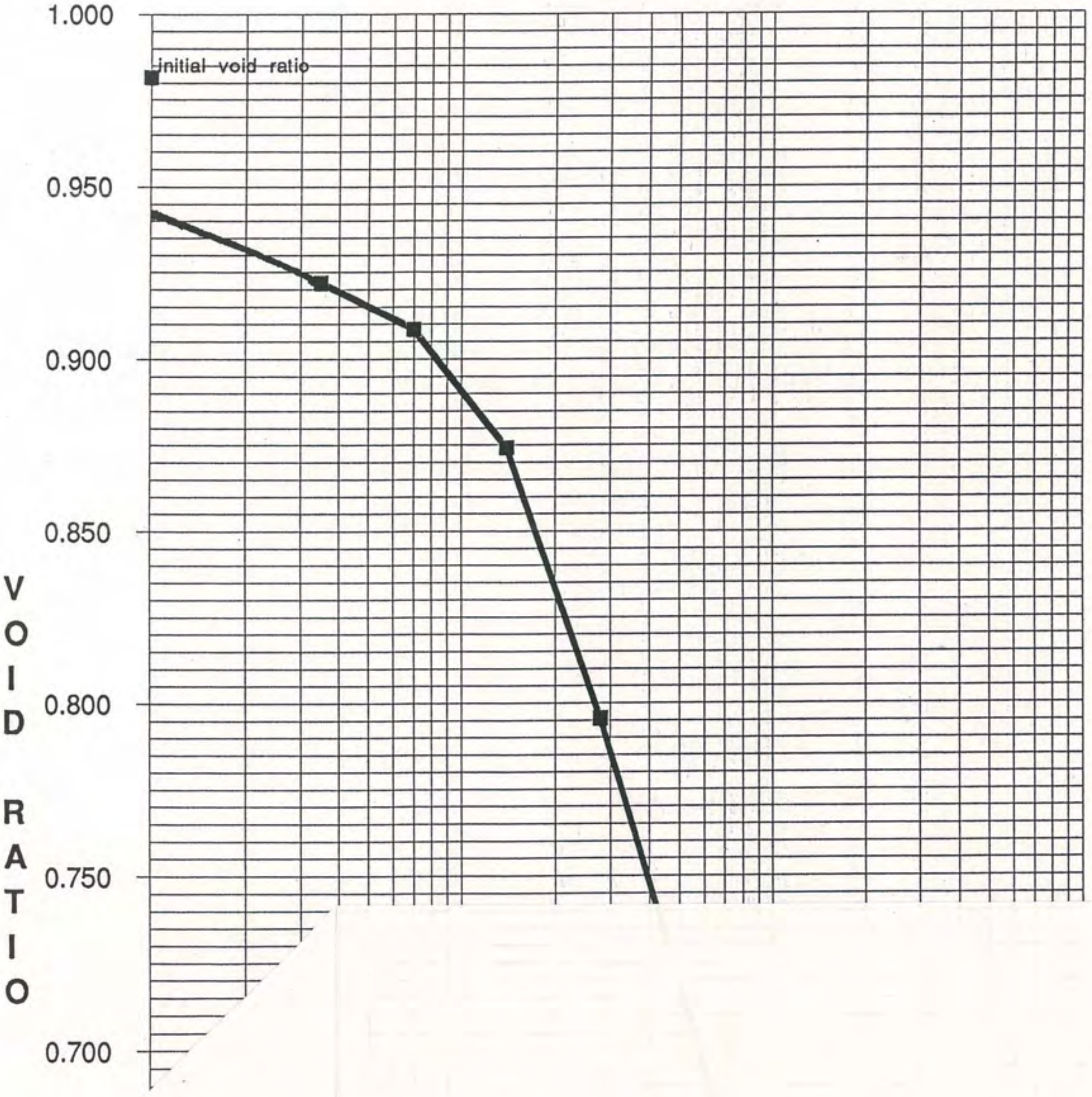
UNCONFINED COMPRESSIVE STRENGTH (psi): 12 (psf): 1,699

TIME (MINUTES)	DEFORM, in (X 1000)	AXIAL STRAIN	AREA (SQ. IN.)	APPLIED LOAD (LBS)	STRENGTH (PSI)	STRENGTH (PSF)
0.5	27	0.0090	4.42	10	2	326
1.0	52	0.0173	4.41	18	4	587
1.5	73	0.0243	4.41	25	6	817
2.0	95	0.0317	4.40	31	7	1,015
2.5	118	0.0393	4.39	35	8	1,148
3.0	142	0.0473	4.38	40	9	1,314
3.5	165	0.0550	4.38	41	9	1,349
4.0	188	0.0627	4.37	45	10	1,484
4.5	211	0.0703	4.36	46	11	1,519
5.0	235	0.0783	4.35	48	11	1,588
5.5	256	0.0853	4.34	50	12	1,657
6.0	279	0.0930	4.34	50	12	1,660
6.5	296	0.0987	4.33	50	12	1,662
7.0	324	0.1080	4.32	51	12	1,699
7.5	347	0.1157	4.31	50	12	1,669
8.0	371	0.1237	4.31	48	11	1,605
8.5	385	0.1283	4.30	50	12	1,674
9.0	417	0.1390	4.29	48	11	1,611
9.5	440	0.1467	4.28	46	11	1,546
10.0	462	0.1540	4.28	46	11	1,549
10.5	500	0.1667	4.26	46	11	1,554
11.0						
11.5						
12.0						
12.5						

VOID RATIO vs. NORMAL PRESSURE DIAGRAM

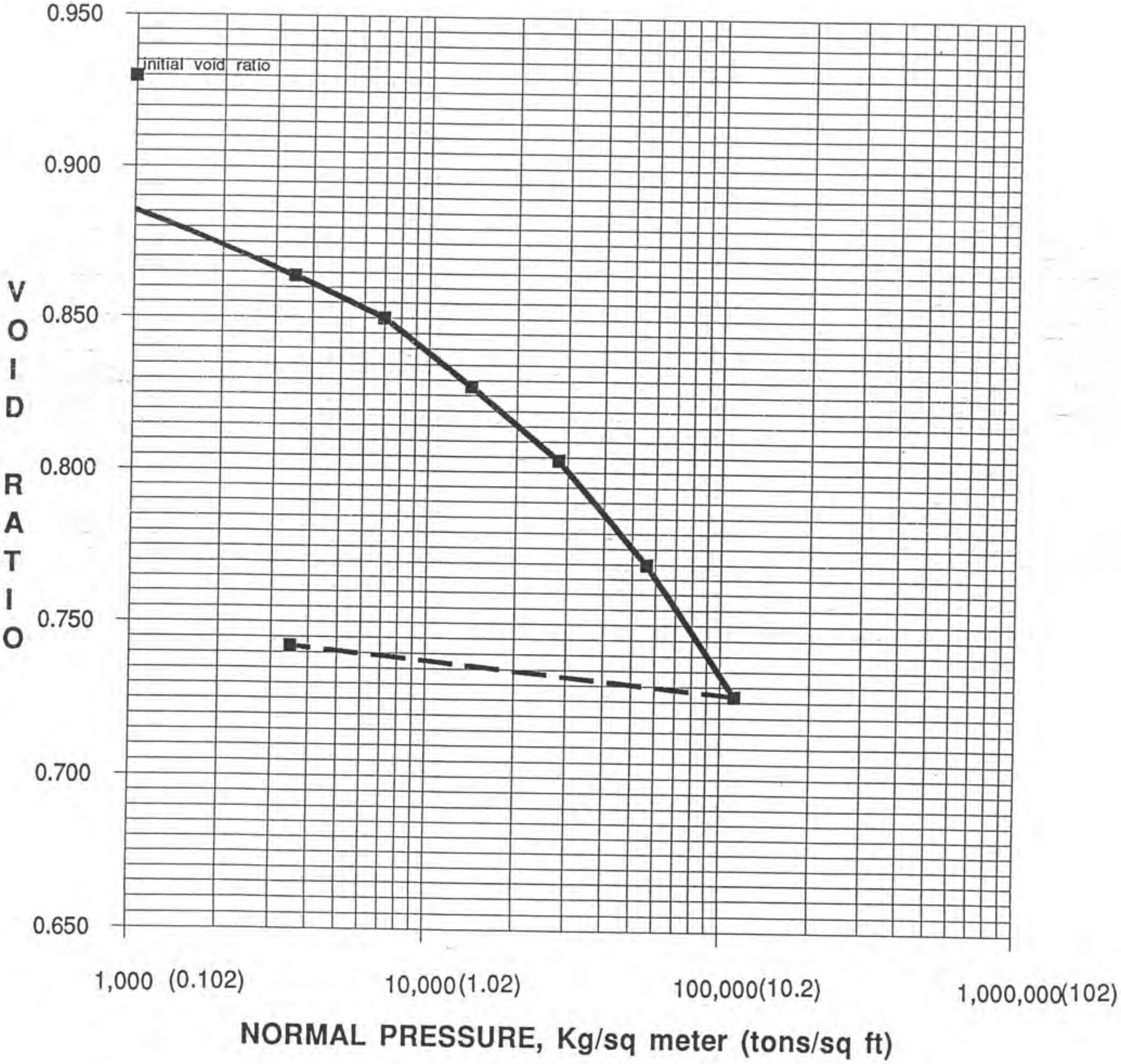
AVILA BEACH WASTEWATER TREATMENT PLANT

Boring #1 @ 6-6.5'



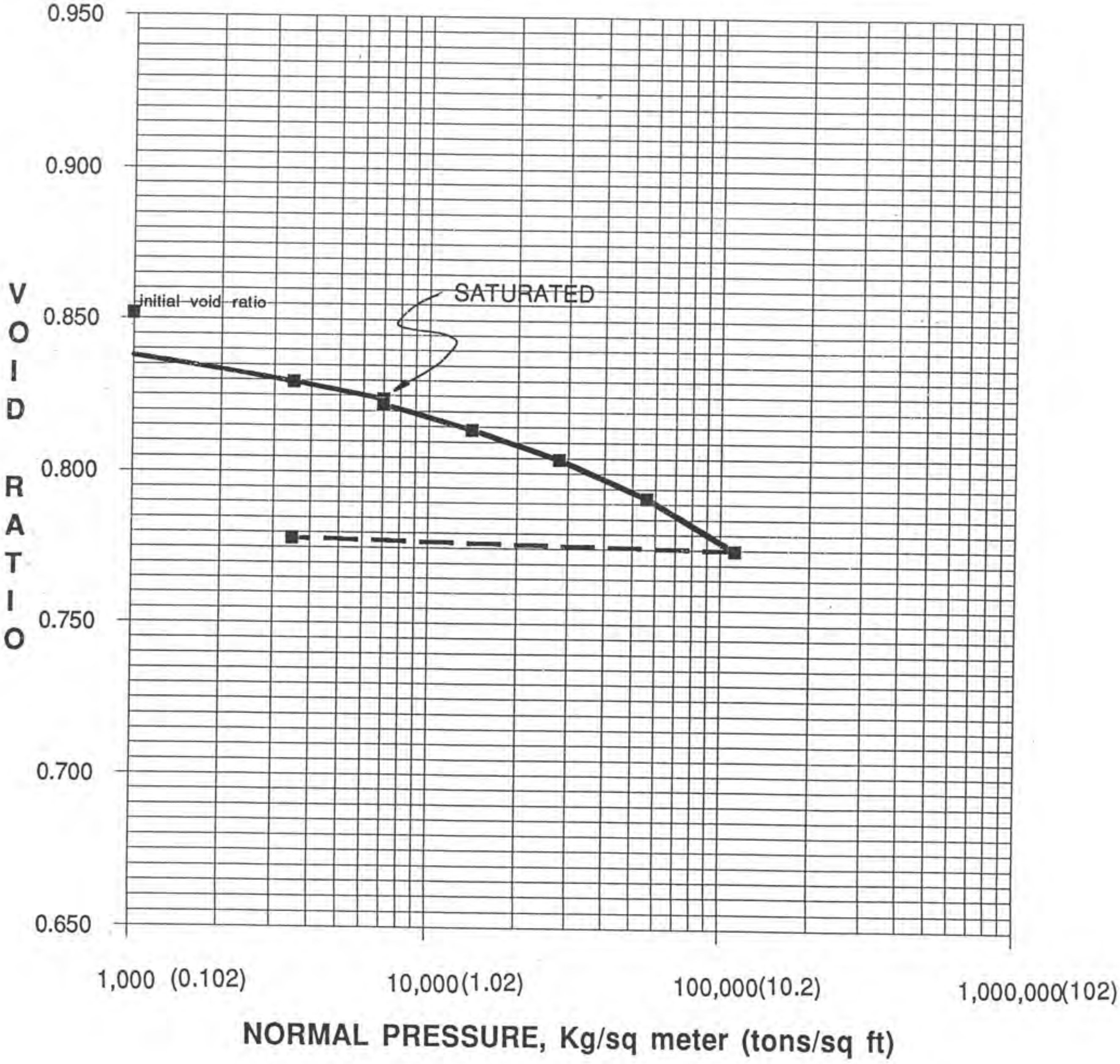
VOID RATIO vs. NORMAL PRESSURE DIAGRAM

AVILA BEACH WASTEWATER TREATMENT PLANT
Boring #1 @ 17.5-18'



VOID RATIO vs. NORMAL PRESSURE DIAGRAM

AVILA BEACH WASTEWATER TREATMENT PLANT
Boring #1 @ 21-21.5'



TL The Twining Laboratories, Inc.
Since 1898

Geotechnical and Environmental Engineering • Construction Inspection • Materials Testing • Analytical Chemistry

June 11, 1992

TL 492-0009-03

Earth Systems Consultants
4378 Santa Fe Road
San Luis Obispo, California 93401

[illegible] 15 1992

Attn: Mr. Terry Reyes

Subject: Corrosivity Analysis and Evaluation
Avila Beach, W.W.T.P.

Dear Mr. Reyes:

This letter presents the corrosivity evaluation based on the analytical results of the soil samples tested. The risk of corrosion of construction materials relates to the potential for soil induced chemical reaction. The rate of deterioration depends on soil resistivity, texture, acidity, and chemical concentration.

1. Based on the ASTM Special Technical Publication 741 and our test results, the soils are severely corrosive to ferrous alloy pipes as indicated by a resistivity value of 724. Buried metal objects should be protected in accordance with manufacturer's recommendations based on the severe corrosive potential of the soils. The evaluation was limited to the effects of soils to metal subjects, corrosion due to other potential sources, such as stray currents and groundwater, were not evaluated.

2. Corrosion of concrete due to sulfate attack by soil is not anticipated based on a low sulfate concentration of 0.092 percent by dry weight determined for the soils. The ACI Manual of Concrete Practice, Section 201.22-12, recommends using a Type II cement for foundations placed in these soils.

The analytical results of the tests are enclosed in the following pages.

We appreciate the opportunity to be of service to Earth Systems Consultants. If you have any questions regarding this letter or if we can be of further assistance, please contact us at your convenience.

Sincerely,

THE TWINING LABORATORIES, INC.

David R. Ansolabehere
Project Engineer
Geotechnical Engineering Division

David S. See, RCE
Division Manager
Geotechnical Engineering Division

REGISTERED PROFESSIONAL ENGINEER
DAVID S. SEE
No. 44940
Exp. Date 03/31/94
CIVIL
STATE OF CALIFORNIA

DRA/DSS/mcn
Enclosures

TL

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REPORT DATE : June 10, 1992
EXAMINATION NO.: 692-2854.1

PROJECT MGR: D. See

CLIENT : Earth Systems Consultants
4378 Santa Fe Rd.
San Luis Obispo, CA 93401

ATTENTION : Terry Reyes

PROJECT NAME : Avila Beach WWTP
Project # NGS08167W01

DATE SAMPLED : 05-16-92 by Client
DATE RECEIVED: 05-27-92 at 1512 hrs. via UPS

The Twining Laboratories is accredited by the State of California Department of Health Services for the analysis of Drinking Water, Wastewater and Hazardous Waste under Certificate No. 1371.

In accordance with your instructions, the samples submitted were analyzed for the components specified. All samples analyzed were submitted in good condition. The analytical results are enclosed on the following pages.

Please contact us if you have any questions concerning the analyses or results. Thank you for letting us serve you.

Debra Lehman
Authorized Signature
The Twining Laboratories, Inc.
Chemistry Division

JJK:dab
lc:herewith

2527 Fresno Street • P.O. Box 1472
Fresno, California 93716
(209) 268-7021 • Fax (209) 268-7126

4230 Kiernan Avenue, Suite 105
Modesto, California 95356

☐ 2521 East Valley Oaks Drive
Visalia, California 93291
(209) 625-1712 • Fax (209) 625-1714

☐ 3701 Pegasus Drive, Suite 124
Bakersfield, California 93308

REPORT DATE : June 10, 1992
EXAMINATION NO.: 692-2854.1

PROJECT MGR: D. See
PAGE 1 of 1

CLIENT : Earth Systems Consultants

PROJECT NAME : Avila Beach WWTP

DATE SAMPLED : 05-16-92 by Client
DATE RECEIVED: 05-27-92 at 1512 hrs. via UPS
DATE PREPARED: 06-04-92
DATE ANALYZED: 06-08-92

ANALYZED BY : D. Carlton, D. Spenhoff
REVIEWED BY : J. Strutzel

SAMPLE TYPE : Soil
SAMPLE IDENTIFICATION: B1 @ 3-5′

	RESULT	UNITS	MDL	METHOD
<u>CORROSIVITY</u>				
pH	8.3	pH	0-14	150.1
Resistivity	724	ohms/cm^3	N/A	DOT424
Chloride (Cl)	0.051	% by weight	0.001	A1000M
Sulfate (SO_4)	0.092	% by weight	0.01	A1000M

NOTES:

ohms/cm^3: ohms per cubic centimeter @ 25°C
DOT : Department of Transportation
N/A : Not Applicable
MDL : Method Detection Limit
ND : None Detected

APPENDIX D - HISTORIC AERIAL PHOTOGRAPHS

Avila CSD WWTP

2850 Avila Beach Drive

San Luis Obispo, CA 93405

Inquiry Number: 5706656.1

July 05, 2019

The EDR Aerial Photo Decade Package

6 Armstrong Road, 4th floor
Shelton, CT 06484
Toll Free: 800.352.0050
www.edrnet.com

EDR Aerial Photo Decade Package

07/05/19

Site Name:	**Client Name:**
Avila CSD WWTP	Yeh and Associates
2850 Avila Beach Drive	391 Front Street,Suite D
San Luis Obispo, CA 93405	Grover Beach, CA 93433
EDR Inquiry # 5706656.1	Contact: Jon Blanchard

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

Search Results:

Year	Scale	Details	Source
2016	1"=500'	Flight Year: 2016	USDA/NAIP
2012	1"=500'	Flight Year: 2012	USDA/NAIP
2009	1"=500'	Flight Year: 2009	USDA/NAIP
2006	1"=500'	Flight Year: 2006	USDA/NAIP
1994	1"=500'	Acquisition Date: May 13, 1994	USGS/DOQQ
1981	1"=500'	Flight Date: August 01, 1981	USDA
1976	1"=500'	Flight Date: June 28, 1976	USGS
1963	1"=500'	Flight Date: July 02, 1963	USGS
1960	1"=500'	Flight Date: April 02, 1960	USGS
1956	1"=500'	Flight Date: September 10, 1956	USDA
1949	1"=500'	Flight Date: April 03, 1949	USDA

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INQUIRY #: 5706656.1
YEAR: 2016
= 500'
N
EDR

INQUIRY #: 5706656.1

YEAR: 2012

= 500'

N

INQUIRY #: 5706656.1

YEAR: 2009

INQUIRY #: 5706656.1
YEAR: 2006
= 500'
N
EDR

INQUIRY #: 5706656.1

YEAR: 1994

= 500'

N

EDR

INQUIRY #: 5706656.1

YEAR: 1981

= 500'

N


INQUIRY #: 5706656.1

YEAR: 1976

= 500'

N

INQUIRY #: 5706656.1

YEAR: 1963

= 500'

N


INQUIRY #: 5706656.1
YEAR: 1960
= 500'
N

INQUIRY #: 5706656.1

YEAR: 1956

= 500'

N


INQUIRY #: 5706656.1

YEAR: 1949

= 500'

N


-27-40
6651 14