SUNSET AVENUE APARTMENTS
FAIRFIELD, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Ryan Keith
Red Tail Land Development, LLC
2082 Michelson Drive
4th Floor
Irvine, CA 92612

PREPARED BY
ENGEO Incorporated

April 16, 2021

PROJECT NO.
07912.001.000

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GEOTECHNICAL
ENVIRONMENTAL
COASTAL/MARITIME
WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
07912.001.000

April 16, 2021

Mr. Ryan Keith
Red Tail Land Development, LLC
2082 Michelson Drive, 4th Floor
Irvine, CA 92612

Subject: Sunset Avenue Apartments
1776 Sunset Avenue
Fairfield, California

GEOTECHNICAL EXPLORATION

Dear Mr. Keith:

ENGEO prepared this geotechnical report for Red Tail Land Development, LLC as outlined in our agreement dated February 9, 2021. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for the residential development design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated

REGISTERED PROFESSIONAL ENGINEER
NICHOLAS S. BROUSSARD
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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this geotechnical report for design of Sunset Avenue Apartments in Fairfield, California. We prepared this report as outlined in our agreement dated February 9, 2021. Red Tail Land Development, LLC authorized ENGEO to conduct the following scope of services.

- Subsurface field exploration
- Soil laboratory testing
- Data analysis and conclusions
- Report preparation

For our use, we received the following items from you:

- Conceptual Site Plan, Scheme 2 prepared by Angeleno Associates, Inc., dated March 8, 2021.
- Draft phase I environmental site assessment report prepared by Padre Associates, Inc., dated March 2021.
- Preliminary Base Map, prepared by DK Engineering, dated April 13, 2021.

We previously performed a subsurface exploration on the northern parcels of the site in December 2007. The geotechnical data from the 2007 field exploration were used in development of our recommendations in this report.

This report was prepared for the exclusive use of our client and their consultants for design of this project. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION

The site is located in Fairfield, California, as shown on the Vicinity Map, Figure 1. The site consists of three adjoining parcels located east of Sunset Avenue and south of East Tabor Avenue, as shown on the Site Plan – Existing Conditions, Figure 2A. The approximately 9-acre site is identified as Assessor's Parcel Numbers (APNs) 0037-030-200, 0037-030-210, and 0037-060-480. The site is bordered by East Tabor Avenue to the north, Sunset Avenue and apartments to the west, apartments to the north, a flood canal to the east, and single-family residences to the south.

1.3 PROJECT DESCRIPTION

The conceptual plan provided indicates the project will include construction of twenty-six apartment buildings for 130 units, a single-story recreation building with a swimming pool, parking lot, a park, open space, and associated utilities as shown on the Site Plan – Proposed Development, Figure 2B. The wood-framed apartment buildings will be two- to three-stories.

Based on our communication with you, we understand that minor cuts and fills are proposed with less than 3 feet thick over the majority of the site, and that the existing soil stockpile will be used to backfill the drainage channels within the site that are shown on Figures 2A and 2B.

Structural loads for the proposed buildings are yet to be determined; however, we assume that structural loads will be representative for this type of construction.

2.0 FINDINGS

2.1 SITE BACKGROUND

We reviewed the historical aerial photographs and topographic maps that were provided with the draft Phase 1 environmental site assessment report. The aerial photographs included those from 1937, 1947, 1952, 1957, 1968, 1974, 1982, 1993, 2006, 2009, 2012, and 2016.

The topographic maps and aerial images indicate that the northern two parcels have remained undeveloped since 1937, while the southern parcel had structures on the site in each of the photos listed above. Laurel Creek crossed through the northern two parcels, in what appears to be the same approximate location between 1937 and 1982. By 1993, Laurel Creek was rerouted and channelized along the east boundary of the site; however, the former channel remained incised across the site. In the 2006 photo, the northern and southern ends of Laurel Creek were backfilled. Earthwork activities were visible across much of the site in the 1974 photo, which appears to correspond with construction of the elementary school to the east of the site. Our comparison of the 1947 and 1974 images below suggests that the former Laurel Creek channel was widened at some point in time and the bend on the north boundary of the site widened to create a straighter alignment.

EXHIBIT 2.1-1: 1947 EDR Historic Aerial Photo

EXHIBIT 2.1-2: 1974 EDR Historical Aerial Photo

The draft Phase 1 environmental site assessment (ESA) described that two underground storage tanks (USTs) were removed from the southern end of the site in the 1980s. The depth of the tank removal was not detailed in the files attached to the Phase 1 ESA. Refer to the Site Plans, Figures 2A and 2B, for the location of these features.

2.2 GEOLOGY AND SEISMICITY

2.2.1 Geology

The site is located within the eastern portion of the Coast Range Geomorphic Province. The Coast Range province includes many separate ranges, coalescing mountain masses, and several major structural valleys. These mountain ranges and basement rock are largely made up of marine sedimentary rocks that have been highly faulted, folded, and altered by orogenic processes. The valleys are generally filled with quaternary age alluvial deposits that consist of gravel, sand, silt, and clay. The majority of the site is mapped as alluvial fan deposits (Qf), as shown on Figures 2A, 2B, and 3. The age of this unit is described as late Pleistocene (less than 30,000 years) to Holocene, and consists of sand, gravel, silt, and clay on gently sloping, fan-shaped, relatively undissected alluvial surfaces (Wiegers et al., 2006). The former Laurel Creek channel on the site is mapped as being underlain by modern stream channel deposits (Qhc), consisting of alluvial sand, gravel, and silt.

2.2.2 Seismicity

No active faults cross the property and the site is not located within an Alquist-Priolo Earthquake Fault Zone. However, numerous small earthquakes occur every year in the region, and larger earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of these faults and significant historic earthquakes recorded around the site.

The Uniform California Earthquake Rupture Forecast (UCERF 3) (Field et al., 2013) estimates the 30-year probability for a magnitude 6.7 or greater earthquake in the San Francisco Bay Area at approximately 72 percent, considering the known active seismic sources in the region.

To determine nearby active faults that are capable of generating strong seismic ground shaking at the site, we utilized the USGS Unified Hazard Tool* and deaggregated the hazard at the peak ground acceleration (PGA).These seismic sources are summarized in Table 2.2.2-1 below.

TABLE 2.2.2-1: USGS 2014 Seismic Sources Capable of Producing Significant Ground Shaking (Latitude: 38.2638 Longitude: 122.019263)

SOURCE	R_{RUP}		MOMENT MAGNITUDE M_W
	(KM)	(MILES)	
Great Valley 04b Gordon Valley [3]	10.8	6.7	6.7
Green Valley [2]	16.3	10.1	6.8
Great Valley 05 Pittsburg – Kirby Hills alt1 [3]	8.53	5.3	6.4
Green Valley [3]	18.0	11.2	6.6
Great Valley 06 (Midland) alt1 [1]	18.5	11.5	6.8
Great Valley 04b Gordon Valley [2]	13.0	8.1	7.1

*USGS Unified Hazard Tool - Edition: Dynamic Conterminous U.S. 2014 (update) (v4.2.0)

Other significant nearby active faults include the Hayward fault, approximately 25 miles to the west, Rodgers Creek – Healdsburg 2011, approximately 24 miles to the west, Calaveras fault approximately 29 miles to the southwest, and San Andreas fault approximately 44 miles to the west.

2.3 FIELD EXPLORATIONS

We previously conducted field explorations at the northern two parcels of the site, which included four borings and two cone penetration test (CPT) soundings in 2007. Our 2021 field exploration included drilling two borings and pushing a ⅜-inch diameter hand probe at two locations. These explorations were performed to supplement our 2007 field explorations, which were not previously published. Descriptions of our 2007 and 2021 field explorations are included below. The locations were roughly sited by pacing from existing features and should be considered accurately located only to the degree implied by the method used.

2.3.1 Borings (2007)

On August 29, 2007, we observed drilling of four borings at the locations shown on Figures 2A and 2B. An ENGEO representative observed the drilling and logged the subsurface conditions at each location. We retained a truck-mounted Mobile B-24 drill rig and crew to advance the borings using 6-inch-diameter hollow-stem auger methods. The borings were advanced to depths ranging from 19 to 42 feet below the existing grade. The borings were backfilled after the completion of the field exploration activities.

We retrieved soil samples at various intervals in the borings using standard penetration tests (SPT). The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration. In addition, 3-inch O.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows to drive the last 1 foot of penetration; the blow counts have not been converted using any correction factors. When sampler driving was difficult, penetration was recorded only as inches penetrated for 50 hammer blows.

We used the field logs to develop the report logs in Appendix A. The logs depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.3.2 Cone Penetration Tests (2007)

On August 27, 2007, we observed completion of the two cone penetration tests (CPT-01 and CPT-02) at the locations shown on Figures 2A and 2B. We retained a truck-mounted rig to advance the CPTs to depths of 68.4 feet at CPT-01 and 50 feet at CPT-02. The CPT rig had a 20-ton compression-type cone with a 15-square-centimeter (cm^2) base area, an apex angle of 60 degrees, and a friction sleeve with a surface area of 225 cm^2. The cone, connected with a series of rods, is pushed into the ground at a constant rate. Cone readings are taken at approximately 5-cm intervals with a penetration rate of 2 cm per second in accordance with ASTM D5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson, 2009). The CPT logs are presented in Appendix C.

We processed the CPT data using the commercially available program, CPeT-IT v2.3.1.6 to determine the soil behavior types and characteristics of the soil encountered; the results of this are also included in Appendix C.

2.3.3 Borings (2021)

On April 1, 2021, we observed drilling of two borings (2-B1 and 2-B2) at the locations shown on Figures 2A and 2B. An ENGEO representative observed the drilling and logged the subsurface conditions at each location. We retained a truck-mounted Soil Test Ranger drill rig and crew to advance the borings using 4-inch-diameter solid-flight auger methods. We permitted and backfilled the borings in accordance with the requirements of Solano County.

We obtained bulk soil samples from drill cuttings and retrieved soil samples at various intervals in the borings using standard penetration tests. We also collected a bulk sample of soil from the top of the stockpile by hand digging at the GS-1 location shown on Figure 2.

The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration. In addition, 3-inch O.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows to drive the last 1 foot of penetration; the blow counts have not been converted using any correction factors. When sampler driving was difficult, penetration was recorded only as inches penetrated for 50 hammer blows.

We used the field logs to develop the report logs in Appendix A. The logs depict subsurface conditions at the exploration locations for the date of exploration; however, subsurface conditions may vary with time.

2.3.4 ⅜-Inch-Diameter Hand Probes (2021)

On April 1, 2021, we also used a ⅜-inch-diameter hand probe to determine the depth of potentially compressible soil near the north and south ends of the former Laurel Creek drainage channel on the site. At each of the two locations, we were able to advance the probe relatively easy to a depth of approximately 2 feet.

2.4 SURFACE CONDITIONS

The topographic data indicates the majority of the site is gently sloping towards the south, with grades ranging from approximately Elevation 43 feet along the north boundary to 38 feet (Datum – NAVD88) near the southeast boundary. During our site visit on April 1, 2021, we observed the following features within the site:

- The surface of the site was generally covered by a medium growth of seasonal grasses and weeds, with some trees and shrubs. Debris and litter were scattered across the site.
- An approximately 12-foot-high stockpile of soil was located in the central portion of the site.
- The approximately 5- to 8-feet-deep drainage channel, associated with the former alignment of Laurel Creek, contained standing water and dense vegetation. The topographic plan indicates that the channel discharges into an existing storm drain at the southern end of the channel.
- An approximately 4-foot-deep drainage channel crosses along the center of the site from west to east. The topographic plan indicates that the channel discharges into an existing storm drain system.

- A flood canal that collects the water that formerly flowed through Laurel Creek is located along the eastern site boundary. Our review of the topographic data suggests the approximately 5-foot-high side slope of the flood channel is approximately 2:1 (horizontal:vertical) to 3:1. The top of the slope is approximately 10 feet from the eastern site boundary.
- A vacant single-story, wood-frame house was located on the southernmost portion of the site. A concrete slab foundation from a demolished storage building was observed east of the house. A hot-mix asphalt pavement area was located northeast of the house. There was a water supply well and surface tank east southeast of the house. The southern portion of the site is bordered on all four sides by a chain-link fence.

Surface conditions observed during our 2007 field exploration on the northern two parcels are generally consistent with our observations in 2021.

Please refer to the Site Plan, Figures 2A and 2B, for more information on site features.

PHOTO 2.4-1: View of Former Laurel Creek Drainage Channel Looking Southeast From Northwest Corner of Site

PHOTO 2.4-2: View From Existing House Looking East

2.5 SUBSURFACE CONDITIONS

With the exception to Boring 2-B2, the other five soil borings encountered existing fill on the site, as indicated on Figure 2B. The fill thickness generally varied from 2 to 4 feet, with up to 14 feet encountered in Boring B1 on top of the stockpile. The fill encountered in each of the five borings generally consisted of medium-stiff to stiff lean clay with varying amounts sand and some gravel. Laboratory tests of the fill and near-surface soil resulted in plasticity index values of 22 at GS-1, and 18 at Boring 2-B1; these results are an indication of the fill and near-surface soil as having a moderate- to high expansion potential.

The native soil encountered in Borings B1, B3, and 2-B2 generally consisted of very stiff to hard lean clay that considered as Pleistocene-aged deposits. Laboratory testing of the harder clay in Boring 2-B2 at a depth of 12 feet resulted in a plasticity index of 18. In Borings B2 and B4, we encountered stiff to hard silt and lean clay and loose to medium-dense clayey sand; these lower-density deposits are considered as younger alluvial fan deposits that overly the

Pleistocene-aged deposits and are encountered at a shallower depth on the southern end of the site. In Boring 2-B1, we encountered low-plasticity sandy silt to a depth of approximately 8 ½ feet, which we attribute to modern-day stream channel deposits. Based on a comparison of the CPT cone-tip resistances and Robertson (2016) modified soil behavior type index, the depth to the Pleistocene-aged deposits is approximately 20 feet on the north end of the site at CPT-02 and Boring 2-B1.

We processed the CPT data using the commercially available program, CPeT-IT v2.3.1.6 to determine the soil behavior types and characteristics of the soil encountered. Based on our matched pairs of CPT and boring, the CPT-predicted soil behavior types appear consistent with the boring logs. The Robertson (2016) modified soil behavior type interpretation indicates that the Pleistocene-aged sand and clay encountered in CPT-01 are generally classified as dilative (dense of critical state) starting at the ground surface. In CPT-02, the clay and sand are classified as contractive to a depth of approximately 20 feet, at which point the soil becomes dilative. This CPT data supports the Pleistocene-aged soil as having significant microstructure from cementation, bonding, and aging (Robertson, 2016).

We observed wet compressible material in portions of the former Laurel Creek drainage channel on the site. The thickness of the compressible material at the northern and southern ends of the drainage channel is approximately 2 feet thick, based on the depth at which we were able to easily penetrate a ⅜-inch-diameter hand probe.

Consult the Site Plan, boring logs, and CPT logs for specific subsurface conditions at each location. We include our exploration boring logs in Appendix A. The logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. The logs graphically depict the subsurface conditions encountered at the time of the exploration. The CPT logs are included in Appendix C.

2.6 GROUNDWATER CONDITIONS

We observed static groundwater in several of our subsurface explorations. We summarize our observations in the table below:

TABLE 2.6-1: Groundwater Observations

EXPLORATION LOCATION	APPROX. DEPTH TO GROUNDWATER (FEET)	APPROX. GROUNDWATER ELEVATION (FEET)
B1 (August 29, 2007)	31½ (17½ below stockpile)	18½
B2 (August 29, 2007)	12½	31
B4 (August 29, 2007)	11½	29½
2-B1 (April 1, 2021)	12	31
2-B2 (April 1, 2021)	19	21

Because of the Solano County backfill requirements, some of the borings may not have been left open a sufficient amount of time to allow water levels to stabilize.

Based on the measurements, the groundwater observed in the borings was at a deeper depth on the southern portion of the site.

Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

2.7 LABORATORY TESTING

We performed laboratory tests on selected soil samples to evaluate their engineering properties. For this project in 2021, we performed moisture content, dry density, unconfined compression, plasticity index, gradation, and soil corrosion potential testing. In 2007, we performed moisture content, dry density, unconfined compression, plasticity index, gradation, and sulfate testing. Moisture contents and dry densities are recorded on the boring logs in Appendix A; other laboratory test data is included in Appendix B.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the proposed project may be designed as planned, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications.

The primary geotechnical concerns that could affect development on the site are existing fill, expansive soil, liquefaction, and groundwater. We summarize our conclusions below.

3.1 EXISTING FILL

With the exception of Boring 2-B2, existing fill was found in the other five borings. Since the fill compaction is unknown, we consider this fill is non-engineered. The thickness of the fill varied from approximately 2 to 14 feet as depicted on Figure 2B. In addition to the fill that we encountered, our review of historic aerial photos suggest that a segment of Laurel Creek was backfilled between the existing southern terminus of the drainage channel and the site boundary. Furthermore, minor fill was placed at the northern end of the former Laurel Creek within the site near East Tabor Avenue. We also anticipate fill on the southern end of the site associated with the removal of the underground storage tanks, the single-family house, former storage building, and associated utilities. Due to the age of the structures on the southern portion of the site dating back to at least 1937, a below-grade septic system and leach field were likely in use on the site. Non-engineered fills can undergo excessive settlement, especially under new fill or building loads. Without proper documentation of existing fill placed on the site, we recommend complete removal and recompaction of the existing fill. We present fill removal recommendations in Section 5.1.

The stockpiled soil and fill encountered across the site appear generally similar to the native soil on site. Therefore, we suspect that the fill stockpile and fill on portions of the site could have been derived from the excavation for the adjacent drainage channel that borders the east side of the site or from a nearby location. Therefore, we judge the fill suitable for reuse on site as detailed in the Acceptable Fill section.

3.2 EXPANSIVE SOIL

We observed potentially expansive lean clay near the surface of the site in each of the borings, consisting of fill and native soil. Our laboratory test results indicate that this soil exhibits medium-to high shrink/swell potential with variations in moisture content (Coduto, 1998).

Expansive soil changes in volume with changes in moisture. They can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. To reduce the potential for damage to the planned structures, we recommend that buildings be supported on properly designed post-tensioned mat foundations bearing on competent native soil or compacted fill. In addition, to reduce expansion potential of compacted fills, we recommend that clay on site be compacted at a slightly lower relative compaction at a moisture content well over optimum.

We have also provided specific grading recommendations for compaction of clay soil at the site. The purpose of these recommendations is to reduce the swell potential of the clay by compacting the soil at a high moisture content and controlling the amount of compaction.

3.3 COMPRESSIBLE SOIL

During our site visit on April 1, 2021, we observed wet and soft recent channel deposits in the bottom of the former Laurel Creek channel that crosses the site. We were able to easily penetrate a ⅜-inch-diameter hand probe to a depth of approximately 2 feet at two locations; near the northern and southern ends of the channel. Compressible soil mitigation recommendations are presented in Section 5.2 of this report.

3.4 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking and ground lurching. The following sections present a discussion of these hazards as they apply to the site. Based on topographic and lithologic data, the risk of regional subsidence or uplift, landslides, tsunamis, and seiches is considered low to negligible at the site.

3.4.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, it is our opinion that ground rupture is unlikely at the subject property.

3.4.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the Bay Area region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, structures should be designed using sound engineering judgment and the 2019 California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

3.4.3 Liquefaction

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soil most susceptible to liquefaction are Holocene-aged, clean, loose, saturated, uniformly graded, fine-grained sand. Empirical evidence indicates that loose silty sand as well as lean silt and some clay are also potentially liquefiable. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective confining stress of the soil, it is said to have liquefied, and if the sand consolidates or vents to the surface during and following liquefaction, ground settlement and surface deformation may occur. The Association of Bay Area Governments (ABAG) liquefaction hazard map covering this site indicates that the majority of the site is considered to have a moderate susceptibility to liquefaction, with a high susceptibility along the former alignment of Laurel Creek.

We evaluated the liquefaction potential of the site soil using CPT-based liquefaction triggering procedures by Youd et al. (2001) and Robertson (2009) using the commercially available software program, Cliq v2.2.1.4. As discussed in the previous section, groundwater was measured at a depth of 11½ to 31 feet below the existing grade in five of our borings. We selected a groundwater depth of 10 feet, earthquake magnitude of 6.8, and a peak ground acceleration (PGA) of 0.70g as listed in Section 3.7. We judged a soil-behavior-type index (I_c) of 2.6 to be appropriate for analyzing the data.

The conventional liquefaction analysis methodologies referenced above were developed using empirical data from Holocene-aged soil deposits. As described in Sections 2.2.1 and 2.5, the site is underlain by Holocene- to Pleistocene-aged soil. At CPT-01, we interpret the Pleistocene-aged soil to be located below the fill at a depth of 3 feet based on the CPT tip resistance and blow counts in Boring B3 and dilative nature of the CPT data. At CPT-02, we interpreted the Pleistocene-aged soil to be at a depth of approximately 30 feet below existing grade based on CPT tip resistance and dilative state, and blow counts of nearby Borings B4 and 2-B1. Various researchers and practitioners have documented that the strength of sand increases with age, primarily as a result of chemical cementation and a more stable rearrangement of particles. Several researchers have found that this “aging” effect increases the cyclic shear resistance of the sand even though it may not be reflected in the conventional SPT or CPT penetration resistance (Leon et al., 2006, Arango et al., 2000). To capture the cyclic strength gain due to aging effects, an aging factor or strength gain factor was developed by Kulhawy and Mayne (1990), Skempton (1986), and Seed (1979). Arango et al. (2000) used cyclic simple shear laboratory testing on high-quality undisturbed samples of sand to demonstrate the cyclic shear strength gain and updated the strength gain relationship developed previously by others. Based on the age of the sand deposits and CPT interpretation indicating significant microstructure and dilative behavior, we used an aging factor for the CPT-based liquefaction evaluation on this site. Using a conservative age of 10,000 years (10^4) to represent the Pleistocene-aged soil, we applied an aging factor or strength gain factor of 2.0 in our analysis based on the Arango et al. (2000) updated relationship.

The analysis results suggest no liquefaction potential of the sandy soil found in CPT-01, which the soil consisted of entirely Pleistocene-aged soil. Based on our analysis results, the sandy layers within CPT-02 at a depth between approximately 20 feet and 30 feet are considered to be susceptible to liquefaction. Our analysis results estimate up to approximately 1 to 1½ inches of total liquefaction-induced settlement at CPT-02 on the northern end of the site. Therefore, we recommend designing the structures to accommodate up to 1½ inches of total seismic-induced settlement and ¾ inch of differential settlement over 50 feet.

3.4.4 Lateral Spreading

Lateral spreading is a failure within a nearly horizontal soil zone, which causes the overlying soil mass to move down a gentle slope or toward a free face such as a creek or open body of water. Lateral spreading is most often associated with strength loss due to liquefaction. As described above, the potentially liquefaction soil was encountered in a localized area of the site. However, due to the lack of significant slopes in proximity to this area of the site, the potential for lateral spreading to occur at the site during seismic shaking is also considered to be low. In addition, the site soil considered having a potential of liquefaction is located below the bottom of the existing creek channel that borders the east side of the site.

3.4.5 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soil. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the region, but based on the site location, it is our opinion that the offset is expected to be minor.

3.4.6 Flooding

According to the Flood Insurance Rate Map No. 06095C0269E (FEMA, 2009), the site is mapped as Zone X "areas of 0.2 percent annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1 percent annual chance flood." The Laurel Creek channel borders the east side of the site, which is mapped as a "special flood hazard area subject to inundation by the 1 percent annual chance flood." The Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if necessary.

3.5 SOIL CORROSION POTENTIAL

As part of this study, we obtained two representative near-surface soil samples that we submitted to a qualified analytical lab for determination of sulfate and chloride. In 2007, we also performed a sulfate test on a representative sample. The results are included in Appendix B and summarized in the table below.

TABLE 3.5-1: Sulfate and Chloride Test Results

SAMPLE LOCATION	DEPTH	CHLORIDE (PPM)	SULFATE (PPM / % BY WEIGHT)
2-B1	2 Feet	4.06	5.86 / 0.001%
GS-1	0 to 1 Foot	4.95	0.17 / 0.00%
B2	4½ Feet	-	113 / 0.011%

The 2019 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Section 19.3.1 for concrete durability requirements. In accordance with the criteria presented in ACI Table 19.3.1.1, we categorized the site soil as S0 sulfate exposure class. Considering a 'Not Applicable' sulfate exposure, there is no requirement for cement type or water-cement ratio; however, a minimum concrete compressive strength of 2,500 psi is specified by the building code. For this sulfate range, we recommend Type II cement and a concrete mix design for foundations and building slabs-on-grade that incorporates a maximum water-cement ratio of 0.50. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications. Values tested for chloride do not pose a significant impact to metals or concrete.

Although not performed as part of this study, we submitted soil samples for resistivity testing on the same geologic unit for the Grange Middle School located approximately 500 feet east of the site. The resistivity test results on two clay samples from depths of 3½ feet resulted were 910 and 1,150 oh-cm. These test results indicate the clay soil on site may be considered severely corrosive to buried metal piping.

If desired to investigate this further, we recommend a corrosion consultant be retained to evaluate if specific corrosion recommendations are advised for the project.

3.6 STATIC AND PERCHED GROUNDWATER

We encountered groundwater ranging from depths of approximately 11 to 19 feet below existing site grades. In addition, we observed some water in the bottom of the former Laurel Creek Drainage within the site. It does not appear that the static groundwater level beneath the site is likely to affect the proposed development; however, we anticipate groundwater may be encountered in deeper utility trenches and during backfill of the former drainage channel. Due to the hard clay soil, we also anticipate perched water may be encountered at various times of year. Perched groundwater can:

1. Impede grading activities.
2. Cause moisture damage to sensitive floor coverings.
3. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.
4. Cause premature pavement failure if hydrostatic pressures build up beneath the section.

We provide recommendations to reduce the effects of perched water in the later sections addressing Over Optimum Soil Conditions, Site Drainage, Landscaping Considerations, Slab Moisture Vapor Reduction, and Cut-off Curbs.

3.7 2019 CBC SEISMIC DESIGN PARAMETERS

The 2019 CBC utilizes design criteria set forth in the ASCE 7-16 Standard. Based on the subsurface conditions encountered, we characterized the site as Site Class F. However, since we anticipate the period of the structures will be less than 0.5 seconds, Site Class D is considered to be appropriate for the site. In addition, Site Class D-Stiff Soil is determined to be appropriate for the site, based on the use of published Vs_{30} maps in OpenSHA that suggested a Vs_{30} of approximately 290 meters per second or 950 feet per second for the site soil (Wills, 2015).

According to ASCE 7-16 Standard, stiff soil for Site Class D has Vs_{30} of 600 to 1,000 feet per second.

We provide the 2019 CBC seismic design parameters in Table 3 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted-Maximum-Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3: 2019 CBC Seismic Design Parameters, Latitude: 38.263800 **Longitude:** -122.019263°

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.558
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.545
Site Coefficient, F_A	1
Site Coefficient, F_V	Null*
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.558
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	Null*
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.038
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	Null*
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.639
Site Coefficient, F_{PGA}	1.1
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.70
Long period transition-period, T_L	8

*Requires site-specific ground motion hazard analysis per ASCE 7-16 Section 11.4.8

Considering the low-rise residential development, we estimate the fundamental periods of the proposed structures to be less than $1.5T_s$ (where T_s is 0.61 seconds for this project). Therefore, the structural engineer may consider exception(s) of Section 11.4.8 of ASCE 7-16 as follows:

> "A ground motion hazard analysis is not required for structures… where, structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) of ASCE 7-16 for values of $T \leq 1.5T_S$ and taken as equal to 1.5 times the value computed in accordance with Eq. (12.8-3) of ASCE 7-16 for $1.5T_s < T \leq T_L$."

4.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.

2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fills has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

5.0 EARTHWORK RECOMMENDATIONS

As used in this report, relative compaction refers to the in-place dry unit weight of soil expressed as a percentage of the maximum dry unit weight of the same soil, as determined by the ASTM D1557 laboratory compaction test procedure, latest edition. Compacted soil is not acceptable if it is unstable; it should exhibit only minimal flexing or pumping, as observed by an ENGEO representative. The term "moisture condition" refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry.

We define "structural areas" as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

5.1 EXISTING FILL REMOVAL

Remove existing fill to competent native soil, as evaluated by ENGEO. Figures 2A and 2B display the fill thickness encountered in five of the borings, the fill stockpile location, former underground storage tank (UST) locations, and backfilled portions of the former Laurel Creek channel. The lateral extent and depth of fill are expected to vary. Consult the exploration logs in Appendix A for fill depths at specific locations.

5.2 COMPRESSIBLE SOIL MITIGATION

We observed relatively soft soil within the bottom of the former Laurel Creek channel on the site, which was approximately 2 feet thick. We anticipate there also may be compressible soil within the drainage swale that crosses the site from west to east and below the fill that was used to backfill the southern end of the former Laurel Creek channel. We recommend removal of compressible soil during grading operations to competent native soil. The depth of soil removal should be evaluated and determined by ENGEO.

5.3 GENERAL SITE CLEARING

Areas to be developed should be cleared of surface and subsurface deleterious materials, including existing buildings and their foundations, slabs, buried utilities and their backfill, irrigation lines, pavements, debris, and designated trees, shrubs, and associated roots. Clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in Section 5.8. ENGEO should be retained to observe and test backfilling.

A water supply well is located at the southern parcel. This well should be abandoned in accordance with the requirements of the regulatory agency.

Following clearing, the site should be stripped to remove surface organic materials. Strip organics from the ground surface to a depth of at least 2 to 3 inches below the surface. Remove strippings from the site or, if considered suitable by the landscape architect and owner, use them in landscape fill.

5.4 CUT/FILL TRANSITION OR CUT LOTS

According to the boring data, the majority of the near-surface soil consist of silty clay. However, sandy silt was encountered in Boring 2-B1 at a depth of 2 feet below the existing grade at the northern end of the site, and clayey sand was found in Boring 2-B2 at the surface at the northern end of the site. Variable surface soil may exist on the site.

Building pads constructed in cuts may encounter variable subsurface conditions in the near-surface soil; these pads may therefore be subject to damaging differential soil movements. Building pads that transition from cut to fill within the building pad area also can experience differential soil movements.

We recommend such building pads be reconstructed to create uniform subgrade conditions in the upper 2 feet of the subgrade soil. This can be accomplished by subexcavating the soil on the building pads to a minimum depth of 2 feet below finished pad grade on cut lots or lots constructed over cut-and-fill transitions and replacing the subexcavated material with uniformly mixed compacted fill. The subexcavation should be performed over the entire flat pad area. Compacted fill used to replace subexcavated soil should be placed in accordance with Section 5.8. Our field representative will determine the necessary subexcavation and recompaction during site grading.

5.5 DIFFERENTIAL FILL THICKNESS

Differential building movements may result from conditions where building pads have significant differentials in fill thickness. We recommend that the differential fill thickness across any lot be no greater than 10 feet. Local subexcavation of soil material and replacement with compacted fill may be needed to achieve this recommendation.

The backfill depths of the former USTs are unknown. During the undocumented fill removal, we will determine if subexcavation is required to maintain the differential fill thickness requirement. In addition, as shown on the preliminary base map by dk Engineering, the depth of the former Laurel Creek within the site is approximately 7 to 8.5 feet deep. As discussed previously, soft soil within the creek bottom was approximately 2 feet thick. After the soft soil removal, we will determine if subexcavation is required to maintain the differential fill thickness requirement.

5.6 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. In addition, wet soil conditions may be found in the former Laurel Creek channel and drainage swale. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather,
2. Mixing with drier materials,
3. Mixing with a lime, lime-flyash, or cement product, or
4. Stabilizing with aggregate or geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated by ENGEO prior to implementation.

5.7 ACCEPTABLE FILL

On-site soil and fill material with less than 3 percent organics is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 8 inches in maximum dimension. ENGEO should observe the grading operation to check that the uniformity of the existing fill on the site.

Imported fill materials should meet the above requirements and have a plasticity index less than 12, and at least 20 percent passing the No. 200 sieve. Allow ENGEO to sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

5.8 FILL COMPACTION

5.8.1 Grading in Structural Areas

Perform subgrade compaction prior to fill placement, following cutting operations, and in areas left at grade as follows.

1. Scarify to a depth of at least 8 inches.
2. Moisture condition soil to at least 4 percentage points over the optimum moisture content.
3. Compact soil to between 87 and 92 percent relative compaction. Compact the upper 6 inches of finish pavement subgrade to at least 90 percent relative compaction prior to aggregate base placement.

After the subgrade has been compacted, place and compact acceptable fill as follows.

1. Spread fill in loose lifts that do not exceed 8 inches.
2. Moisture condition lifts to at least 4 percentage points over the optimum moisture content.
3. Compact fill to between 87 and 92 percent relative compaction (90 percent minimum relative compaction at depths of 3 feet or more below finish grades). Compact the upper 6 inches of finish pavement subgrade to at least 90 percent relative compaction prior to aggregate base placement.

Compact the pavement Caltrans Class 2 aggregate base to at least 95 percent relative compaction (ASTM D1557). Moisture condition aggregate base to or slightly above the optimum moisture content prior to compaction.

5.8.2 Underground Utility Backfill

5.8.2.1 General

The contractor is responsible for conducting trenching and shoring in accordance with CALOSHA requirements. Project consultants involved in utility design should specify pipe bedding materials.

5.8.2.2 Structural Areas

Place and compact trench backfill as follows.

1. Trench backfill should have a maximum particle size of 6 inches.
2. Moisture condition trench backfill to 4 percent above the optimum moisture content. Moisture condition backfill outside the trench.
3. Place fill in loose lifts not exceeding 12 inches.
4. Compact fill to between 87 and 92 percent relative compaction (90 percent minimum relative compaction at depths of 3 feet or more below finish grades).

Where utility trenches cross underneath buildings, we recommend that a plug be placed within the trench backfill to help prevent the normally granular bedding materials from acting as a conduit for water to enter beneath the building. The plug should be constructed using a sand cement slurry (minimum 28-day compressive strength of 500 psi) or relatively impermeable native soil for pipe bedding and backfill. We recommend that the plug extend for a distance of at least 3 feet in each direction from the point where the utility enters the building perimeter.

Jetting of backfill is not an acceptable means of compaction. We may allow thicker loose lift thicknesses based on acceptable density test results, where increased effort is applied to rocky fill, or for the first lift of fill over pipe bedding.

5.8.3 Landscape Fill

Process, place and compact fill in accordance with Sections 5.8.1, except compact to at least 85 percent relative compaction (ASTM D1557).

5.9 SLOPES

5.9.1 Gradients

Construct final slope gradients to 2:1 (horizontal:vertical) or flatter for slopes shorter than 5 feet tall. The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements. Refer to Section 7.2 for setbacks of foundations from slopes.

5.9.2 Fill Placed on Existing Slopes

We recommend benching where fills are placed on original grade with a gradient of 6:1 or steeper. Construct benches into original slope grade as filling proceeds every 2 feet vertically, to remove loose soil. Deeper bench depths may be recommended by ENGEO depending on actual conditions observed during construction. Bench widths may vary depending on the original slope grade and actual bench depth.

5.10 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical to reduce the potentially damaging effects of expansive soil. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations for a distance of 10 feet. Where lot lines or surface improvements restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following.

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.
2. Consider the use of rear lot surface drainage collection systems to reduce overland surface drainage from back to front of lot.
3. Do not allow water to pond near foundations, pavements, or exterior flatwork.

5.11 STORMWATER INFILTRATION AND SELECT PROJECT RISK LEVEL FACTORS

Due to the density of the site soil and fines content (percentage passing the No. 200 sieve) generally exceeding 30 percent, the near-surface site soil is expected to have a low permeability value for stormwater infiltration in grassy swales, unless subdrains are installed. Therefore, Best Management Practices should assume that limited stormwater infiltration will occur at the site.

5.12 STORMWATER BIORETENTION AREAS

Where bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or
2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

The retaining wall structures adjacent to the bioretention basins should be a cast-in-place or CMU wall system that would not allow water to freely pass through the wall.

We recommend that each of the bioretention basins and swales incorporate a waterproofing system lining the excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.

Site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document.

Given the nature of bioretention systems and possible proximity to improvements, we recommend ENGEO be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should reduce the exposure time such that the improvements are not detrimentally impacted.

5.13 LANDSCAPING CONSIDERATION

Since the near-surface soil is moderately expansive, we recommend greatly restricting the amount of surface water infiltration near structures, pavements, flatwork, and slabs-on-grade. This may be accomplished by:

- Selecting landscaping that requires little or no watering, especially within 3 feet of structures, slabs-on-grade, or pavements.
- Using low precipitation sprinkler heads.
- Regulating the amount of water distributed to lawn or planter areas by installing timers on the sprinkler system.
- Providing surface grades to drain rainfall or landscape watering to appropriate collection systems and away from structures, slabs-on-grade, or pavements.
- Preventing water from draining toward or ponding near building foundations, slabs-on-grade, or pavements.
- Avoiding open planting areas within 3 feet of the building perimeter.

We recommend that these items be incorporated into the landscaping plans.

6.0 FOUNDATION RECOMMENDATIONS

We developed foundation recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis. As previously mentioned, the alternatives proposed for addressing the effects of the native expansive soil on building foundations include post-tensioned mat foundations.

6.1 POST-TENSIONED MAT FOUNDATIONS

We recommend that the proposed residential structures be supported on post-tensioned (PT) mat foundations bearing on engineered fill. The following preliminary mat foundation recommendations are based on soil materials collected in the borings and hand sampling. Soil sampling should be conducted on the building pads once the site grading is complete to confirm the following mat foundation recommendations are valid for the site.

We recommend that PT mats have a thickened edge at least 2 inches greater than the mat thickness. The Structural Engineer should determine the actual PT mat thickness using the geotechnical recommendations in this report; we defer to the professional judgment of the Structural Engineer on the necessary mat thickness. We recommend that the thickened edge be at least 12 inches wide.

Preliminary post-tensioned mat design criteria are presented in Table 6.1-1 below. The values below are based on the procedure presented by the Post-Tensioning Institute DC10.5-12 “Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soil.”

TABLE 6.1-1: Post-Tensioned Mat Preliminary Design Recommendations

CONDITION	CENTER LIFT	EDGE LIFT
Edge Moisture Variation Distance, e_m (feet)	**8.5**	**4.5**
Differential Soil Movement, y_m (inches)	**0.9**	**1.3**

PT mats may be designed for an average allowable bearing pressure of 1,000 pounds per square foot (psf) for dead-plus-live loads with maximum localized bearing pressures of 1,500 psf at column or wall loads. Allowable bearing pressures can be increased by one-third for wind or seismic loads.

Underlay PT mats with a moisture vapor reduction system as recommended in Section 6.2.

We recommend designing the structures to accommodate up to 1½ inches of total seismic-induced settlement and ¾ inch of differential settlement over 50 feet.

We recommend that we review foundation plans to verify conformance with our recommendations and to provide supplemental recommendations as needed.

6.2 SLAB MOISTURE VAPOR REDUCTION

When buildings are constructed with concrete slab-on-grade, such as post-tensioned mats, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Install a vapor retarder membrane directly beneath the slab. Seal the vapor retarder at all seams and pipe penetrations. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E 1745, latest edition, “Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs.”
2. Concrete shall have a concrete water-cement ratio of no more than 0.50.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water cement ratio are used.
4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specific by the structural engineer.

The structural engineer should be consulted as to the use of a layer of clean sand or pea gravel (less than 5 percent passing the U.S. Standard No. 200 Sieve) placed below the vapor retarder membrane.

6.3 PAD MOISTURE CONDITIONING

Proper moisture conditioning of building pads immediately prior to foundation concrete placement is imperative. We recommend moisture conditioning building foundation subgrade to a moisture content of at least 4 percentage points above optimum to a depth of 12 inches immediately prior to post-tensioned foundation construction. The subgrade should not be allowed to dry below this specified moisture content prior to concrete placement. We also recommend that we be retained to observe the pre-pour moisture conditions to check that our design recommendations have been followed. During the drier parts of the year, it may require several days of soaking/flooding of the pads to achieve this moisture content.

6.4 FOUNDATION SETBACKS

According to the Conceptual Site Plan, the residential structures will be constructed along the eastern boundary that is located at the top of the flood channel. As shown on the plan, a setback of at least 15 feet will be provided between the eastern property line and the buildings. According to the Preliminary Base Map, the Laurel Creek bank is shown as up to approximately 10 feet high and at a gradient of 2:1 (horizontal:vertical).

The recommended slope setbacks for habitable structures are variable depending on slope height and soil conditions. Slope setbacks are intended to reduce the potential effects of long-term slope creep and possible earthquake-induced slope displacements on structures. Based on the site conditions, tor structures adjacent to downslopes, we recommend a minimum setback of at least 15 feet or one-third of the slope height, whichever is greater, from the top of slopes.

6.5 TRENCH BACKFILL

Backfill and compact all trenches below building slabs-on-grade and to 5 feet laterally beyond any edge in accordance with Section 5.8.

7.0 RETAINING WALLS AND SOUND WALLS

A perimeter wall is planned along the street sides of the property as indicated on the Conceptual Site Plan. We anticipate there may be some minor landscape walls that would retain less than 4 feet of soil. We anticipate the walls could include conventional retaining wall or mechanically stabilized earth (MSE) walls. The following recommendations are applicable for onsite soil used as retaining wall backfill.

7.1 LATERAL SOIL PRESSURES

Design proposed retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Design unrestrained retaining walls with adequate drainage to resist an equivalent fluid pressure of 45 pcf plus one-third of any surcharge loads.

The above lateral earth pressures assume level backfill conditions and sufficient drainage behind the walls to prevent any build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. If adequate drainage is not provided, we recommend that an additional equivalent fluid pressure of 40 pcf be added to the values recommended above for unrestrained walls. Damp-proofing of the walls should be included in areas where wall moisture would be problematic.

Construct a drainage system, as recommended below, to reduce hydrostatic forces behind the retaining wall.

7.2 RETAINING WALL DRAINAGE

Construct either graded rock drains or geosynthetic drainage composites behind the retaining walls to reduce hydrostatic lateral forces. For rock drain construction, we recommend two types of rock drain alternatives.

1. A minimum 12-inch-thick layer of Class 2 permeable material (Caltrans Standard Specifications, Section 68-2.02F) placed directly behind the wall, or
2. A minimum 12-inch-thick layer of washed, crushed rock with 100 percent passing the ¾-inch sieve and less than 5 percent passing the No. 4 sieve. Envelop rock in a minimum 6-ounce, nonwoven geotextile filter fabric.

For both types of rock drains:

1. Place the rock drain directly behind the walls of the structure.
2. Extend rock drains from the wall base to within 12 inches of the top of the wall.
3. Place a minimum of 4-inch-diameter perforated pipe (glued joints and end caps) at the base of the wall, inside the rock drain and fabric, with perforations placed down.
4. Place pipe at a gradient at least 1 percent to direct water away from the wall by gravity to a drainage facility.

ENGEO should review and approve geosynthetic composite drainage systems prior to use. If preapproved by the Geotechnical Engineer, prefabricated wall drain panels could be considered in lieu of the granular drain blanket above the pipe system. Drainage should be collected by solid pipes and directed to an outlet approved by the Civil Engineer.

7.3 BACKFILL

Backfill behind retaining walls should be placed and compacted in accordance with Section 5.8. Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement.

7.4 WALL FOUNDATIONS

Conventional site retaining walls and sound walls can be supported on continuous footings or drilled piers.

7.4.1 Shallow Continuous Footings

We recommend that retaining wall footings be designed using an allowable bearing pressure of 2,000 pounds per square foot (psf) for dead-plus-live-loading conditions. This value may be increased by one-third when evaluating the short-term effects of wind or seismic loading.

For a level foreground condition, the footing should be embedded at least 24 inches below lowest adjacent grade. If footings are located within 5 feet from nearby tops of downhill slopes or on sloping ground, the footing embedment should be increased to achieve at least 10 horizontal feet to the nearest free slope face. We recommend a minimum footing thickness of 12 inches. Actual footing design (sizing, reinforcement, etc.) should be determined by the structural engineer based on structural design considerations. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 plane projected upward from the bottom edge of the trench to the footing.

Passive pressures acting on footing foundations may be assumed as 250 pcf. Unless the surface directly in front of the wall is confined by a slab or pavement, we recommend starting passive pressure resistance at a depth of 1 foot below lowest adjacent grade, or that depth necessary to achieve a horizontal distance of 10 feet between the outer base edge of the footing and nearest free face, whichever is shallower. Retaining walls adjacent to bio-retention basins should neglect the passive resistance of the biotreatment soil media layer. Refer to Section 5.12 for additional recommendations associated with bio-retention basins. The friction factor for sliding resistance may be assumed as 0.25. Appropriate safety factors against overturning and sliding should be incorporated into the design calculations.

7.4.2 Drilled Pier Foundations

Perimeter walls/soundwalls and retaining walls may be supported on drilled piers. Drilled piers for these structures should be designed using the recommendations in Table 7.4.2-1 below.

TABLE 7.4.2-1: Design Parameters for Drilled Piers

PIER DESIGN ELEMENT	AUXILIARY STRUCTURE DESIGN PARAMETERS
Minimum pier diameter:	12 inches
Minimum pier depth:	8 feet
Downward load capacity (allowable skin friction):	350 psf. This value may be increased by one-third when considering seismic or wind loads. Exclude the upper 2 feet of the pier shaft from pier load capacity computations
Minimum pier spacing:	3 pier diameters, center-to-center
Passive Resistance Pressure:	250 pcf acting on two times the pier diameter. This value may be increased by one-third when considering seismic or wind loads. Passive resistance may start at the depth required to provide 10 feet of lateral confinement in front of the drilled piers. The passive resistance may be applied over two pier diameters

Appropriate safety factors against bending of wall elements and pier embedment should be incorporated into the design calculations. Actual pier depths and spacing should be determined by the structural engineer based on structural design considerations.

For piers located at the top of the downhill slope, the drilled piers should be designed to active pressures acting on one pier diameter to the depth of at least 10 horizontal feet to the nearest free slope face or toe of the downhill slope.

“Mushrooming” at the top of the piers should be avoided to prevent unnecessary uplift forces from being applied to the piers, and forming the upper portion of piers or other alternatives to removing excess concrete at the top of the piers may be necessary. Additionally, to further reduce panel movement, we recommend the panels be underlain with a degradable material such as “survoid,” or equivalent material, at least 2 inches thick between the bottom of the panels and the supporting soil. The use of a void forming material will reduce potential vertical panel movement.

Pier-drilling operations and concrete placement should be coordinated such that pier holes are left open a minimum amount of time. Pier holes should not be allowed to desiccate visibly before placing concrete. Depressions at the tops of the piers resulting from drilling operations or from any other cause should be backfilled to prevent ponding. In order to minimize potential future pier settlements, loose soil "slough" should be removed from the bottom of pier holes prior to placing concrete. If water collects in the pier shaft, it should be pumped out prior to the placement of concrete. Concrete should be placed by means of a tremie pipe or similar device to avoid concrete contamination by soil dislodging from the pier shaft.

We recommend that the excavation of piers be performed under our direct observation to establish that the piers are founded in suitable materials. Due to the potential for caving, each shaft may need to be cased. If groundwater is encountered, remove it from excavations prior to concrete placement. If groundwater cannot be removed from excavations prior to concrete placement, then we recommend that concrete be placed by tremie pipe. The concrete should be tremied to the bottom of the hole keeping the tremie pipe below the surface of the concrete to avoid entrapment of water in the concrete. As concrete is poured, water is displaced out of the hole.

7.5 MECHANICALLY STABILIZED EARTH (MSE) WALLS

If MSE walls are selected for the site, the following general assumptions and design guidelines should be incorporated into wall design:

- Blocks with positive mechanical connection (fiberglass pins) should be used.
- Site soil may be used as the reinforced soil, foundation soil, and retained soil.
- For level foreground, the base of the lowest block should be embedded at least 1 foot below lowest adjacent grade.
- For downsloping foreground and due to the presence of expansive site soil, the walls should be embedded to a minimum depth necessary to achieve a horizontal distance of 10 feet between the outer base edge of the footing and nearest free face; this is to provide sufficient depth to prevent future exposure of the bottom course of blocks due to slope creep, erosion, or other localized movement.
- The MSE walls should be provided with backdrainage as described in Section 7.2.

Considering the above assumptions and guidelines, the following soil criteria should be incorporated in the MSE wall design.

TABLE 7.5-1: Soil Material Parameters

	COHESION (C') (PCF)	FRICTION ANGLE (Ø') (DEGREES)	UNIT WEIGHT (γ) (PCF)
Reinforced Fill	0	27	125
Retained Soil	0	27	125
Foundation Fill	0	27	125

We recommend that the following minimum factors of safety be incorporated in the MSE wall design.

TABLE 7.5-2: External Stability

	SAFETY FACTOR (STATIC)
Sliding	1.5
Bearing Capacity	2.0
Overturning	2.0

TABLE 7.5-3: Internal Stability

	SAFETY FACTOR (STATIC)
Pull-out Resistance	1.5

We should be consulted if geogrid reinforcement is anticipated to extend beneath the building foundation or paved surface.

8.0 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum section of 4 inches of concrete over 4 inches of aggregate base. Compact the aggregate base to at least 90 percent relative compaction (ASTM D1557). Consideration should be given to thicken flatwork edges to at least 10 inches to help control moisture variations in the subgrade and placement of rebar within the center of the slab to help control the width and offset of cracks. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

9.0 PRELIMINARY PAVEMENT DESIGNS

9.1 FLEXIBLE PAVEMENTS

Based on the presence of lean clay across the site, we recommend the use of an R-value 5 for preliminary design purposes. Using estimated traffic indices (TI) for various pavement loading requirements, we developed the following recommended pavement sections using Chapter 630 of the Caltrans Highway Design Manual (including the asphalt factor of safety), presented in the table below. The table below presents aggregate base thickness that we calculated for private streets and the minimum thickness specified in Section 3.1 of the City of Fairfield Engineering Design Standards Section.

TABLE 9.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION		
	HOT MIX ASPHALT (INCHES)	CALCULATED CLASS 2 AGGREGATE BASE (INCHES)*	FAIRFIELD MINIMUM CLASS 2 AGGREGATE BASE (INCHES)*
5	3	10	Not Applicable
6	3½	12½	14
7	4	16	Not Applicable
8	5	18	20

* For City of Fairfield (COF) owned and maintained roadways the Fairfield minimum thickness of Class 2 Aggregate Base should be used.

The civil engineer should determine the appropriate traffic indices based on the estimated traffic loads and frequencies. The civil engineer should also confirm the design sections meet City minimums if appropriate. We understand the City of Fairfield also requires residential streets to be designed for a minimum traffic index of 6. The City of Fairfield Design Standards also state that geotextile subgrade fabric, as specified in the City of Fairfield Specific Provisions, shall be installed on all subgrade prior to placement of aggregate base (AB) or aggregate subbase (ASB) material. We defer to the civil engineer to determine if the City will require this of the pavement constructed interior to the development.

The above preliminary pavement sections are provided for estimating only. We recommend the actual subgrade material should be tested for R-value during roadway construction.

Pavement materials and construction should comply with the specifications and requirements of the Standard Specifications by the State of California Department of Transportation (Caltrans), City of Fairfield, and the fill compaction specifications in Section 5.8.

9.2 RIGID PAVEMENTS

Concrete pavement sections can be used to resist heavy loads and turning forces in areas such as fire lanes or concrete aprons adjacent to trash enclosures. Final design of rigid pavement sections and accompanying reinforcement should be performed based on estimated traffic loads and frequencies. We recommend the following minimum design sections for rigid pavements.

- Use a minimum section of 7½ inches of Portland Cement concrete over 8 inches of Caltrans Class 2 aggregate base (AB).
- Concrete pavement should have a minimum 28-day compressive strength of 3,500 psi.
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines.

9.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with Section 5.8.1. Aggregate base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02B of the latest Caltrans Standard Specifications.

9.4 CUT-OFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increase maintenance of pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. We recommend installation of pavement cutoff barriers where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture/water barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, then the cutoff barrier may be eliminated.

9.5 RESIDENTIAL DRIVEWAYS

We were not retained to provide design recommendations for residential driveways. They should be designed to resist the anticipated traffic and structural loads, and the effects of expansive soil movement.

11.0 GROUND HEAT EXCHANGE

Based on our findings and review of the proposed development, we consider the site to be *highly* suitable for using a Ground Heat-Exchange (GHX) system to achieve energy savings and to potentially eliminate the need for outdoor air conditioner units, if desired.

For the thermal properties of the soil and groundwater conditions at the site, either a closed-loop or open-loop GHX system would likely be well suited and could be implemented on select buildings or integrated into a project-wide system.

As project planning progresses into architectural design, we can meet with you, your architect, and your MEP designer to further assess and develop GHX energy saving opportunities and efficiencies.

12.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the Sunset Avenue Apartments project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; there is no warranty, express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

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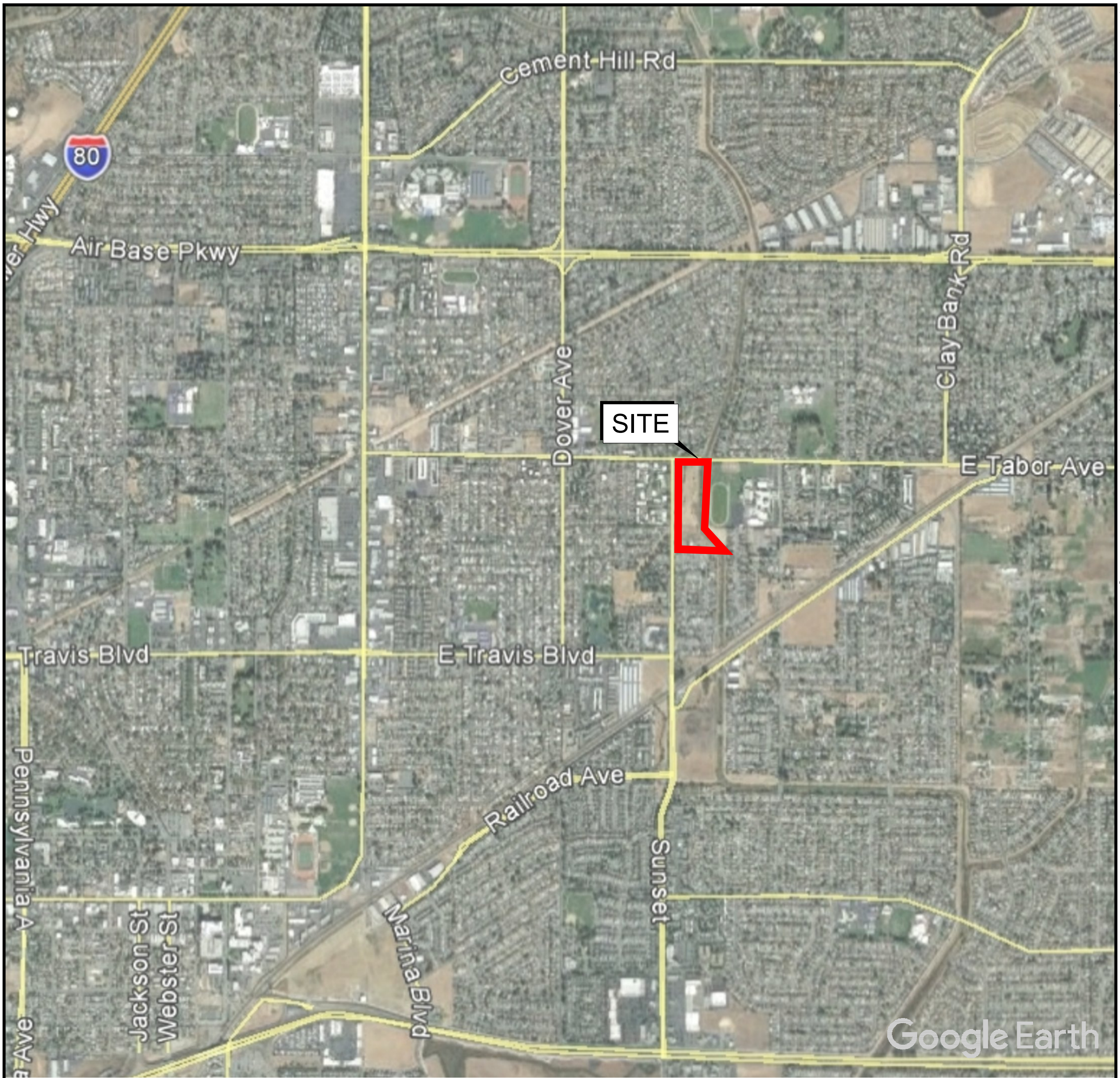
FIGURES



BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP SUNSET AVENUE APARTMENTS FAIRFIELD, CALIFORNIA	PROJECT NO.: 7912.001.000 SCALE: AS SHOWN DRAWN BY: LL / CHECKED BY: MT	FIGURE NO. 1

ORIGINAL FIGURE PRINTED IN COLOR

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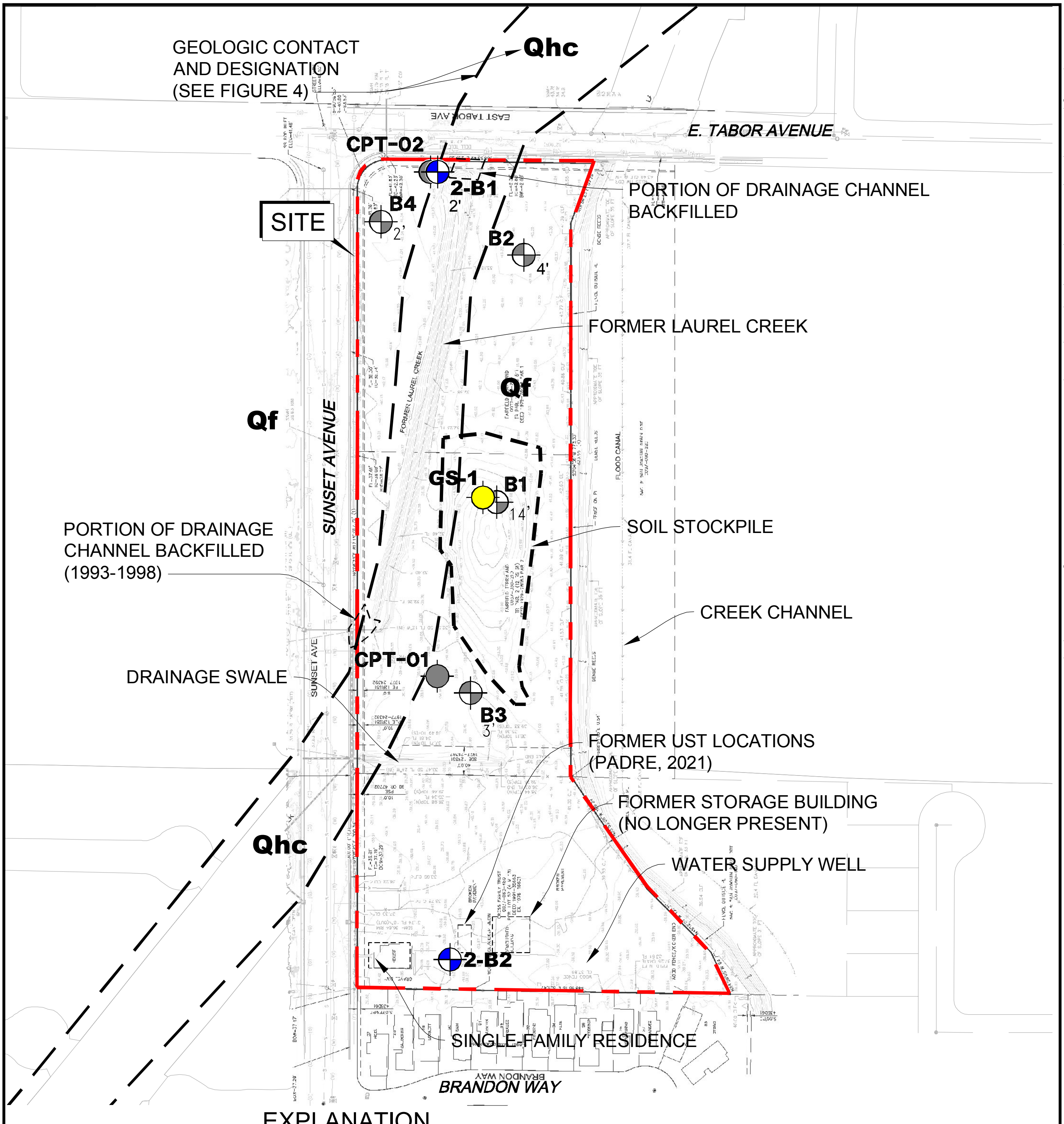

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

BORING, WITH FILL THICKNESS ENCOUNTERED SHOWN IN FEET (ENGEO, 2021)	BORING, WITH FILL THICKNESS ENCOUNTERED SHOWN IN FEET (ENGEO, 2007)
GRAB SAMPLE (ENGEO, 2021)	CONE PENETRATION TEST (ENGEO, 2007)

BASE MAP SOURCE: DK ENGINEERING, 2021

ENGEO
Expect Excellence

SITE PLAN - EXISTING CONDITIONS
SUNSET AVENUE APARTMENTS
FAIRFIELD, CALIFORNIA

PROJECT NO.: 7912.001.000		FIGURE NO.
SCALE: AS SHOWN		2A
DRAWN BY: LL	CHECKED BY: MT	

ORIGINAL FIGURE PRINTED IN COLOR

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- BORING, WITH FILL THICKNESS ENCOUNTERED SHOWN IN FEET (ENGEO, 2021)
- BORING, WITH FILL THICKNESS ENCOUNTERED SHOWN IN FEET (ENGEO, 2007)
- DRAINAGE CHANNEL
- SOIL STOCKPILE
- UNDOCUMENTED FILL
- GRAB SAMPLE (ENGEO, 2021)
- CONE PENETRATION TEST (ENGEO, 2007)

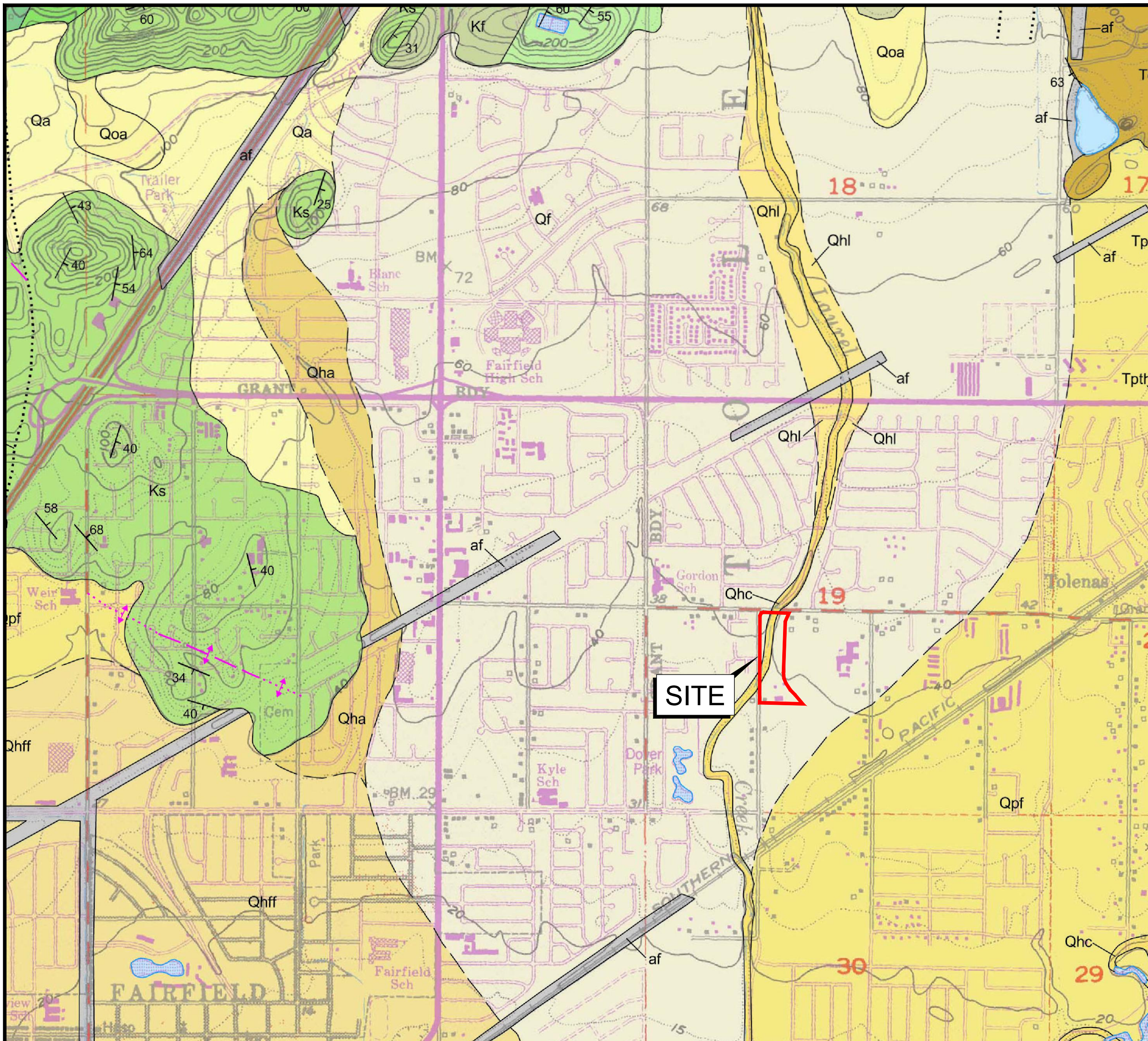
BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE AND ANGELENO AND ASSOCIATES, INC.

ENGEO
Expect Excellence

SITE PLAN - PROPOSED DEVELOPMENT	PROJECT NO.: 7912.001.000	FIGURE NO.
SUNSET AVENUE APARTMENTS	SCALE: AS SHOWN	2B
FAIRFIELD, CALIFORNIA	DRAWN BY: LL / CHECKED BY: MT	

ORIGINAL FIGURE PRINTED IN COLOR

FILE PATH: G:\Drafting\PROJECTS\7912\7912001000\Geotech\GEX\SunsetAprtmnts\7912001000-GEX-2B-SITEPLAN-0321.dwg SAVE DATE: 4/15/2021 2:51:51 PM SAVED BY: llee



EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

AXIS OF FOLD

ANTICLINE

STRIKE AND DIP OF STRATA

INCLINED

af	ARTIFICIAL FILL (HOLOCENE, HISTORIC)
Qhc	MODERN STREAM CHANNEL DEPOSITS
Qha	ALLUVIUM, UNDIVIDED (HOLOCENE)
Qhff	ALLUVIAL FAN DEPOSITS, FINE FACIES
Qhl	FAN LEVEE DEPOSITS
Qa	ALLUVIUM, UNDIVIDED
Qf	ALLUVIAL FAN DEPOSITS (LATE PLEISTOCENE <~ 30,000 YEARS TO HOLOCENE)
Qoa	ALLUVIAL DEPOSITS, UNDIVIDED
Ks	SITES FORMATION

N

0 2000
FEET

BASE MAP SOURCE: WIEGERS, 2006

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REGIONAL GEOLOGIC MAP
SUNSET AVENUE APARTMENTS
FAIRFIELD, CALIFORNIA

PROJECT NO.: 7912.001.000
SCALE: AS SHOWN
DRAWN BY: LL
CHECKED BY: MT
FIGURE NO. 3

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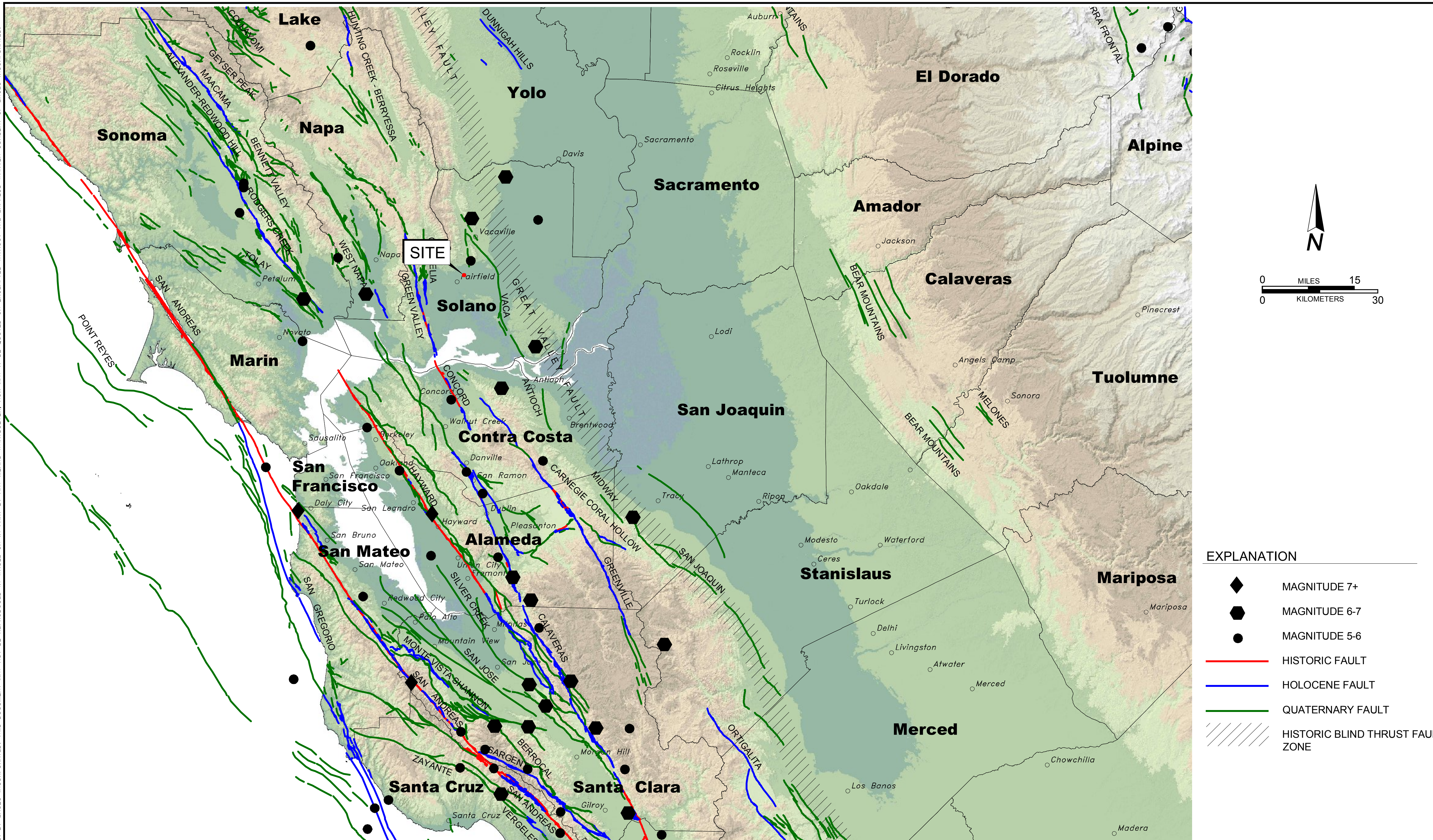
ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-2000)

ENGEO Expect Excellence

REGIONAL FAULTING AND SEISMICITY SUNSET AVENUE APARTMENTS FAIRFIELD, CALIFORNIA	PROJECT NO.: 7912.001.000		FIGURE NO. 4
	SCALE: AS SHOWN		
	DRAWN BY: LL	CHECKED BY: MT	

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ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

BORING LOG KEY
2021 BORING LOGS
2007 BORING LOGS

2021 BORING LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW - Well graded gravels or gravel-sand mixtures
			GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures
			GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW - Well graded sands, or gravelly sand mixtures
			SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures
			SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity
			CL - Inorganic clay with low to medium plasticity
			OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Elastic silt with high plasticity
			CH - Fat clay with high plasticity
			OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*
VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Dames and Moore Piston
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

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LOG OF BORING 2-B1

ENGEO Expect Excellence

LATITUDE: 38.265167 LONGITUDE: -122.019392

Geotechnical Exploration
Sunset Avenue Apartments
Fairfield, California
7912.001.000

DATE DRILLED: 4/1/2021
HOLE DEPTH: Approx. 26½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 43 ft.

LOGGED / REVIEWED BY: A. Hauger / MT
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			LEAN CLAY (CL), dark yellowish brown, hard, slightly moist, medium plasticity, trace concrete debris, rootlets [FILL]			22	37	19	18		12		4.5+*
	40		SANDY SILT (ML), dark yellowish brown, very stiff, slightly moist, low plasticity, fine-grained sand [NATIVE]			10					10		2.5*
5						21					9	105.9	4.5+*
	35		SANDY LEAN CLAY (CL), very dark yellowish brown, stiff, moist, medium plasticity, approximately 10% fine-grained sand			23					22	103.8	1.8
10			CLAYEY SAND (SC), yellowish red, loose, wet, low plasticity, poorly graded fine-grained sand, trace fine gravel			10				44	24.2		
	30				▽								
15			LEAN CLAY (CL), reddish yellow mottled with olive gray, stiff, wet, medium plasticity, trace fine gravel, iron oxidation			19					24		1.75*
	25												
20													

LOG OF BORING 2-B1

ENGEO — Expect Excellence

LATITUDE: 38.265167 | LONGITUDE: -122.019392

Geotechnical Exploration
Sunset Avenue Apartments
Fairfield, California
7912.001.000

DATE DRILLED: 4/1/2021
HOLE DEPTH: Approx. 26½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 43 ft.

LOGGED / REVIEWED BY: A. Hauger / MT
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			CLAYEY SAND (SC), yellowish brown mottled with olive gray, medium stiff, wet, medium plasticity, iron and manganese oxidation			9	27	19	8	47	27		0.5*
	20												
25			CLAYEY SAND (SC), dark yellowish brown mottled with olive gray, medium dense, wet, medium plasticity, poorly graded fine-grained sand, trace iron and manganese stained gravel			27					20		
			End of boring at 26 1/2 feet. Groundwater encountered at 12 feet at the time of drilling.										

LOG OF BORING 2-B2

LATITUDE: 38.262128 LONGITUDE: -122.019275

Geotechnical Exploration
Sunset Avenue Apartments
Fairfield, California
7912.001.000

DATE DRILLED: 4/1/2021
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 40 ft.

LOGGED / REVIEWED BY: A. Hauger / MT
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			CLAYEY SAND (SC), dark brown, very dense, moist, fine- to medium-grained sand, low plasticity fines, manganese oxidation, weak cementation [NATIVE]										
						62/10					19		
			LEAN CLAY WITH SAND (CL), reddish yellow, hard, slightly moist, low plasticity, approximately 10% fine-grained sand, moderately cemented			47					13.6		4.5+*
5	35					50/5							4.5+*
10	30		grades to hard, no cementation			50/6							4.5+*
			LEAN CLAY (CL), reddish yellow, very stiff, moist, medium plasticity, trace angular carbonates, iron and manganese oxidation			28	39	21	18		23.4		3.5*
15	25		grades to reddish brown mottled with olive brown			24							3.0*
					▽								
20	20												

LOG OF BORING 2-B2

ENGEO
Expect Excellence

LATITUDE: 38.262128 LONGITUDE: -122.019275

Geotechnical Exploration
Sunset Avenue Apartments
Fairfield, California
7912.001.000

DATE DRILLED: 4/1/2021
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAVD88): Approx. 40 ft.

LOGGED / REVIEWED BY: A. Hauger / MT
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SANDY LEAN CLAY (CL), reddish brown, stiff, moist, medium plasticity, approximately 20% fine-grained sand			26							1.5*
			End of boring ar 21 1/2 feet. Groundwater encountered at 19 feet at the time of drilling.										

2007 BORING LOGS

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LOG OF BORING B1

LATITUDE: 38.263911 LONGITUDE: -122.019056

Geotechnical Exploration
909 East Tabor Avenue
Fairfield, CA
7912.2.001.01

DATE DRILLED: 8/29/2007
HOLE DEPTH: Approx. 42 ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (NAVD88): Approx. 50 ft.

LOGGED / REVIEWED BY: J. White / TPB
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
0	50	Seasonal grasses, coarse gravel and cobbles at the surface								
		SILTY CLAY (CL), brown to reddish brown, medium stiff, moist, some medium grained sand. (FILL)	14				63			
5	45									
		Stiff, trace fine gravels	29					19	109	
10	40									
			32							
		SILTY CLAY (CL), dark gray, very stiff, moist, trace roots and organics.		41	14	27				
15	35									
		SILTY CLAY (CL), reddish brown, hard, moist, some fine grained sand, trace fine gravels, trace manganese staining.	73							
20	30									
		SANDY CLAY (CL), reddish brown, hard, moist, fine grained sand, trace fine gravels, some manganese staining.	50/4							
25	25									
			50/4					20	111	
30	20									

LOG OF BORING B1

LATITUDE: 38.263911 LONGITUDE: -122.019056

Geotechnical Exploration
909 East Tabor Avenue
Fairfield, CA
7912.2.001.01

DATE DRILLED:	8/29/2007	LOGGED / REVIEWED BY:	J. White / TPB
HOLE DEPTH:	Approx. 42 ft.	DRILLING CONTRACTOR:	West Coast Exploration
HOLE DIAMETER:	6.0 in.	DRILLING METHOD:	Hollow Stem Auger
SURF ELEV (NAVD88):	Approx. 50 ft.	HAMMER TYPE:	140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			SANDY CLAY (CL), reddish brown, hard, moist, fine grained sand, trace fine gravels, some manganese staining.		▼								
			SILTY CLAY (CL), red to reddish brown, hard, wet to moist, trace manganese staining.										
						50/4							
35	15												
						50/6					34	83	
40	10												
						50/4							

Bottom of boring at 42', groundwater encountered at 31.5'.

LOG OF BORING B2

LATITUDE: 38.264858 LONGITUDE: -122.018928

Geotechnical Exploration
909 East Tabor Avenue
Fairfield, CA
7912.2.001.01

DATE DRILLED: 8/29/2007
HOLE DEPTH: Approx. 20½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (NAVD88): Approx. 43½ ft.

LOGGED / REVIEWED BY: J. White / TPB
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
			Seasonal grasses, coarse gravel and cobbles at the surface										
			SILTY CLAY (CL), brown, stiff, moist, some coarse gravel. (FILL)										
	40					31							
5			SILTY CLAY (CL), brown, stiff to very stiff, moist, some coarse gravel.				40	19	21	84			
						22					23	100	1.3
	35												
10			Dark gray to dark brown.			21					21	107	
					▼								
	30		SILTY CLAY (CL), light olive gray, medium stiff, moist, trace carbonates, manganese staining.										
15						17							
			SANDY CLAY (CL), soft, saturated, trace fine gravels.										
	25												
20						15							
			Bottom of boring at 20.5', groundwater encountered at 12.5'.										

LOG - GEOTECHNICAL W/ELEV. 7912200101 BORELOGS FOR GEX.GPJ ENGEO INC.GDT 4/16/21

ENGEO
Expect Excellence

LOG OF BORING B3

LATITUDE: 38.263192 LONGITUDE: -122.019192

Geotechnical Exploration
909 East Tabor Avenue
Fairfield, CA
7912.2.001.01

DATE DRILLED:	8/29/2007	LOGGED / REVIEWED BY:	J. White / TPB
HOLE DEPTH:	Approx. 19½ ft.	DRILLING CONTRACTOR:	West Coast Exploration
HOLE DIAMETER:	6.0 in.	DRILLING METHOD:	Hollow Stem Auger
SURF ELEV (NAVD88):	Approx. 39 ft.	HAMMER TYPE:	140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	DESCRIPTION	Blow Count/Foot	Atterberg Limits: Liquid Limit	Plastic Limit	Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
		Seasonal grasses, coarse gravel and cobbles at the surface								
		SILTY CLAY (CL), brown, stiff, dry to moist, some fine-grained sand. (FILL)	35							
	35	SILTY CLAY (CL), brown, very stiff, moist.								
5		SANDY CLAY (CL), reddish brown, very stiff to hard, moist, fine to medium grained sand, trace fine gravels.	50/4					13	111	3.0
	30	SILTY CLAY (CL), reddish brown, hard, moist, trace fine gravels, trace carbonates.	50/4							
10										
	25	Hard, manganese staining	64				90	103	21	
15										
	20		50/4							
		Bottom of boring at 19.5', no groundwater encountered.								

ENGEO
Expect Excellence

LOG OF BORING B4

LATITUDE: 38.264978 LONGITUDE: -122.019631

Geotechnical Exploration
909 East Tabor Avenue
Fairfield, CA
7912.2.001.01

DATE DRILLED: 8/29/2007
HOLE DEPTH: Approx. 32½ ft.
HOLE DIAMETER: 6.0 in.
SURF ELEV (NAVD88): Approx. 41 ft.

LOGGED / REVIEWED BY: J. White / TPB
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Rope and Cathead

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
	40		SILTY CLAY (CL), brown, medium stiff, slightly moist, trace fine gravel. (FILL)										
			SILTY CLAY (CL), reddish brown, stiff to very stiff, moist, trace fine gravel, some manganese staining.			11							
5	35					30					25	106	2.0*
			Medium stiff.			12							0.5*
10	30				▼								
			SANDY CLAY (CL), light olive brown, very stiff, moist, trace carbonates, maganese staining, fine-grained sand.										
						43					21	106	
15	25												
						27				61			
20	20												
			SANDY CLAY (CL), light olive brown, stiff, saturated, course-grained sand.			16							
25	15												
						10				52			
30	10												
			CLAYEY SAND (SC), reddish brown, dense, saturated, coarse-grained.			59							
			Bottom of boring at approximately 32.5 feet. Groundwater encountered at the time of drilling at approximately 11.5 feet.										

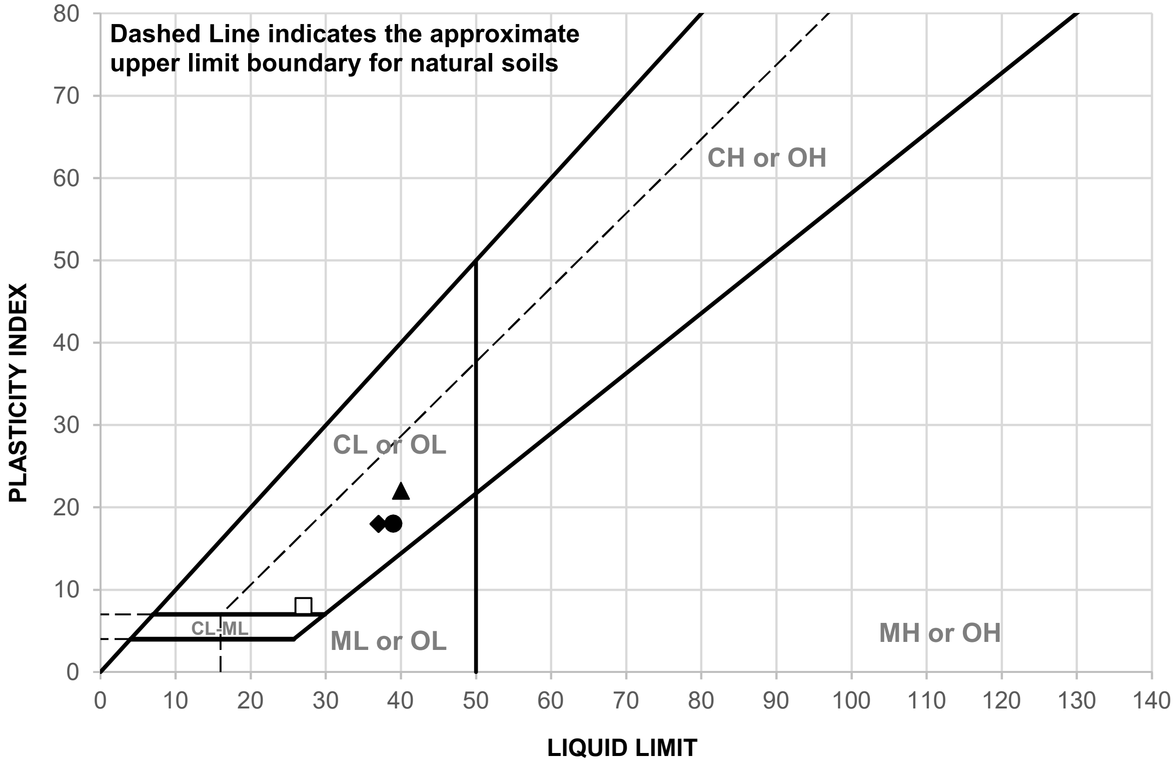
APPENDIX B

2021 LABORATORY TEST DATA
2007 LABORATORY TEST DATA

2021 LABORATORY TEST DATA

Liquid and Plastic Limits Test Report
Unconfined Compression Test
Particle Size Distribution Reports
Analytical Results of Soil Corrosion (2 pages)

LIQUID AND PLASTIC LIMITS TEST REPORT
ASTM D4318



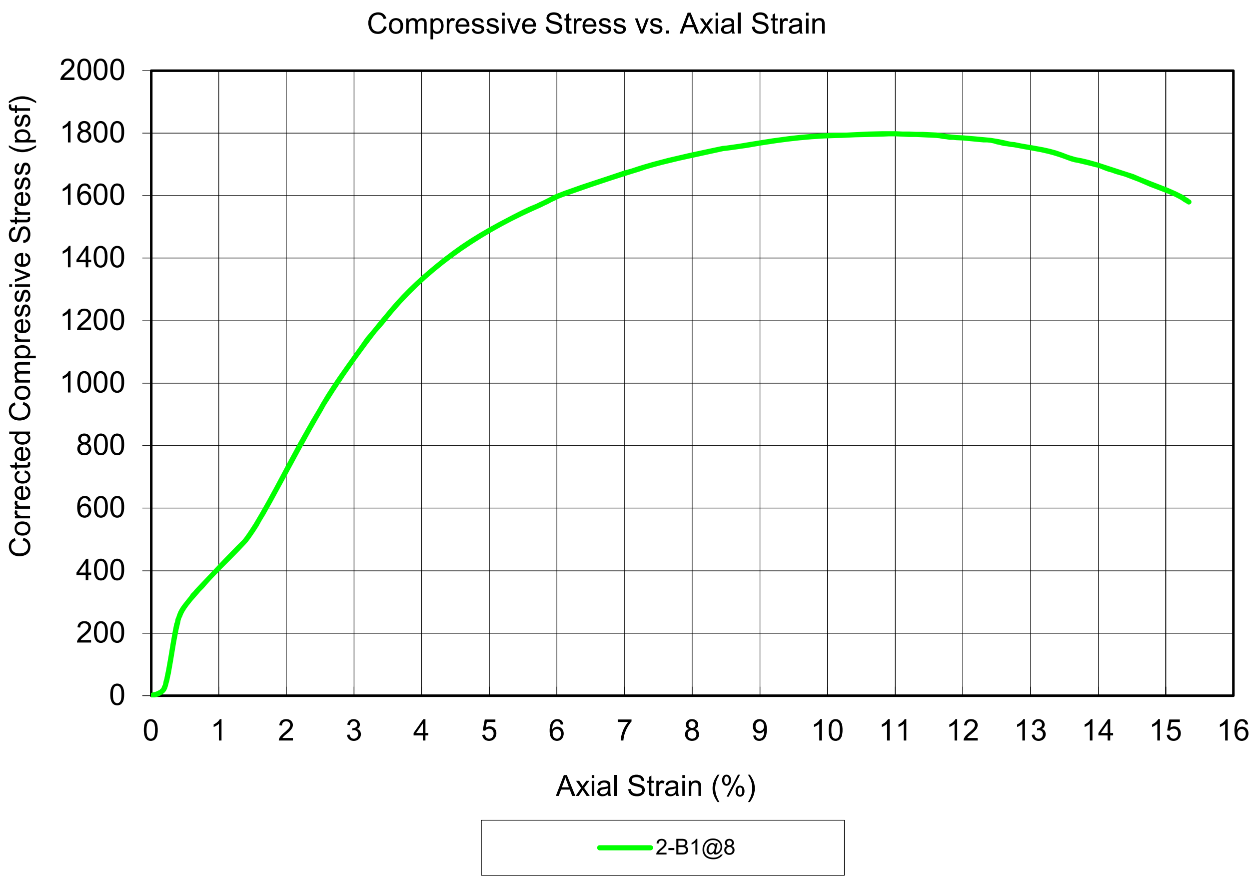
	SAMPLE ID	DEPTH	MATERIAL DESCRIPTION	LL	PL	PI
▲	GS-1	0.5	See exploration logs	40	18	22
◆	2-B1@1.5	1.5	See exploration logs	37	19	18
□	2-B1@20	20	See exploration logs	27	19	8
●	2-B2@12	12	See exploration logs	39	21	18

	SAMPLE ID	TEST METHOD	REMARKS
▲	GS-1	PI: ASTM D4318, Wet Method	
◆	2-B1@1.5	PI: ASTM D4318, Wet Method	
□	2-B1@20	PI: ASTM D4318, Wet Method	
●	2-B2@12	PI: ASTM D4318, Wet Method	

ENGEO
Expect Excellence

CLIENT: Red Tail Land Development, LLC
PROJECT NAME: Sunset Avenue Apartments
PROJECT NO: 7912.001.000 PH001
PROJECT LOCATION: Fairfield, CA
REPORT DATE: 4/9/2021
TESTED BY: R. Montalvo
REVIEWED BY: N. Broussard

UNCONFINED COMPRESSION TEST REPORT
ASTM D2166



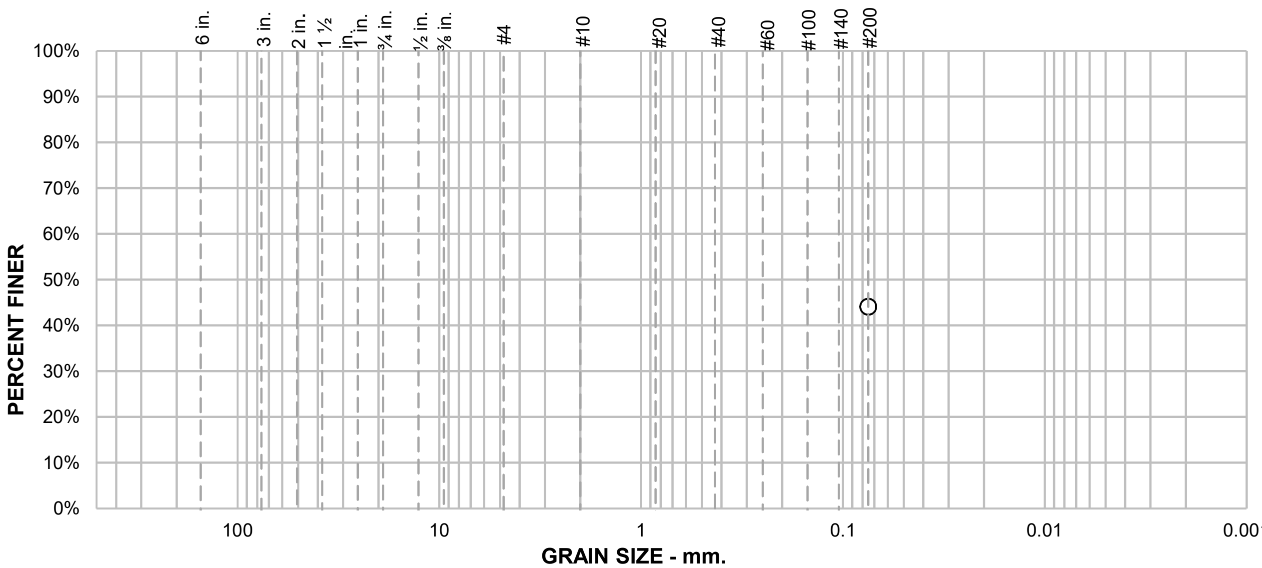
BEFORE TEST	SPECIMEN 2-B1@8
Test Moisture Content (%)	21.96
Dry Density (pcf)	103.8
Saturation (%)	93.8
Void Ratio	0.64
Diameter (in)	2.389
Height (in)	4.997
Height-To-Diameter Ratio	2.09
TEST DATA	
Unconfined Compressive Strength (psf)	1798
Undrained Shear Strength (psf)	899
Strain Rate (in/min)	0.050
Specific Gravity (ASSUMED)	2.720
Strain at Failure(%)	11.01
Test Remarks	

SPECIMEN	DESCRIPTION
2-B1@8	See exploration logs

PROJECT NAME:	Sunset Avenue Apartments	**Test Date:**	4/9/21
PROJECT NO:	7912.001.000 PH001	**Tested By:**	R. Montalvo
CLIENT:	Red Tail Land Development, LLC	**Reviewed By:**	N. Broussard
LOCATION:	Fairfield, CA		

ENGEO Expect Excellence

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D1140, Method B



SAMPLE ID: 2-B1@10

DEPTH (ft): 10

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						44.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	44.1		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = LL = PI =

COEFFICIENTS

D_{90} =	D_{85} =	D_{60} =
D_{50} =	D_{30} =	D_{15} =
D_{10} =	C_u =	C_c =

CLASSIFICATION

USCS =

REMARKS

Soak time = 180 min
Dry sample weight = 343.2 g

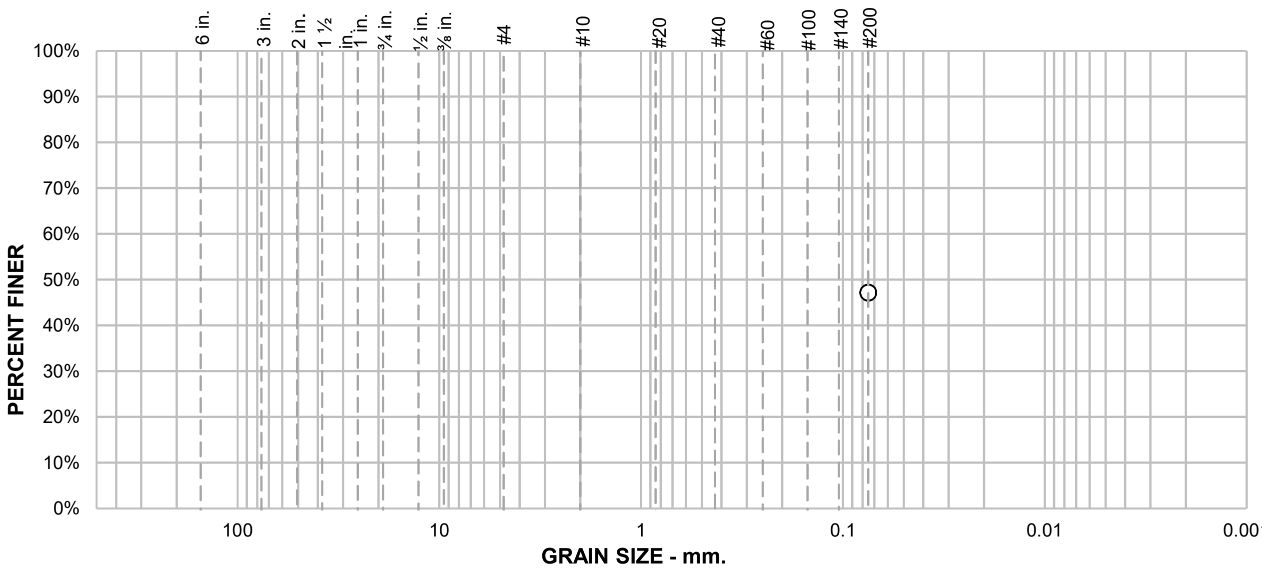
* (no specification provided)

ENGEO Expect Excellence

CLIENT: Red Tail Land Development, LLC
PROJECT NAME: Sunset Avenue Apartments
PROJECT NO: 7912.001.000 PH001
PROJECT LOCATION: Fairfield, CA
REPORT DATE: 4/9/2021
TESTED BY: R. Montalvo
REVIEWED BY: N Broussard

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 2-B1@20

DEPTH (ft): 20

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						47.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	47.2		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 19	LL = 27	PI = 8

COEFFICIENTS

D_{90} =	D_{85} =	D_{60} =
D_{50} =	D_{30} =	D_{15} =
D_{10} =	C_u =	C_c =

CLASSIFICATION

USCS =

REMARKS

PI: ASTM D4318, Wet Method
USCS: ASTM D2487

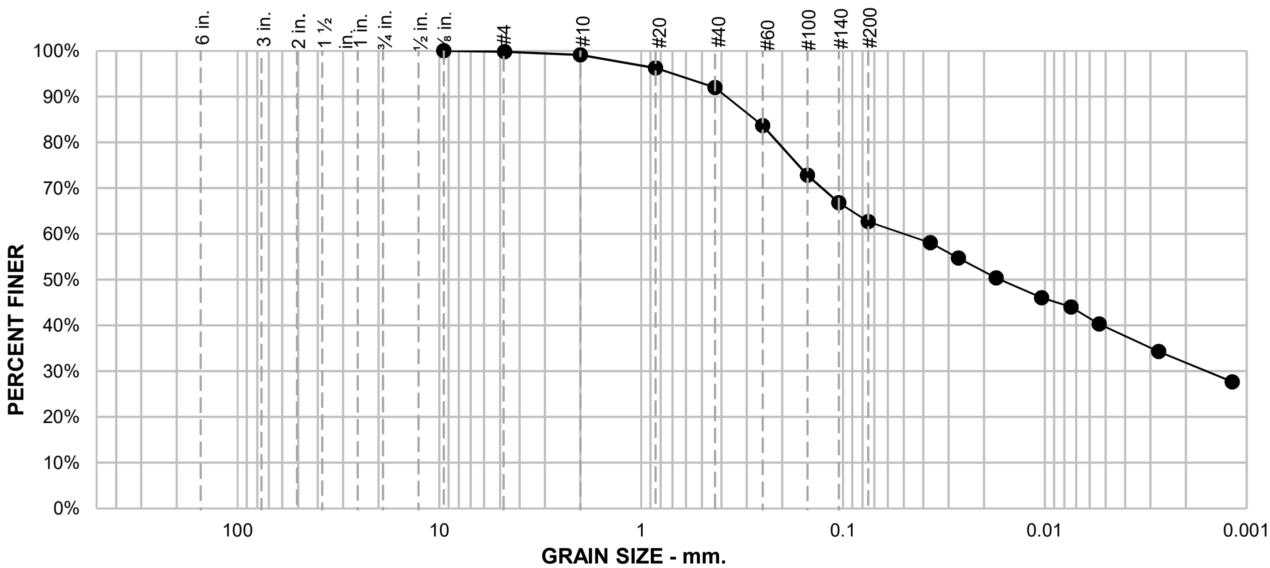
Soak time = 180 min
Dry sample weight = 378.2 g

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: Red Tail Land Development, LLC
PROJECT NAME: Sunset Avenue Apartments
PROJECT NO: 7912.001.000 PH001
PROJECT LOCATION: Fairfield, CA
REPORT DATE: 4/9/2021
TESTED BY: R. Montalvo
REVIEWED BY: N. Broussard

PARTICLE SIZE DISTRIBUTION REPORT
ASTM D422



SAMPLE ID: GS-1

DEPTH (ft): 0.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		0.2	0.7	7.1	29.3	30.8	31.9

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
⅜ in.	100.0		
#4	99.8		
#10	99.1		
#20	96.2		
#40	92.0		
#60	83.7		
#100	72.8		
#140	66.8		
#200	62.7		
0.0369 mm.	58.0		
0.0268 mm.	54.7		
0.0174 mm.	50.4		
0.0104 mm.	46.1		
0.0074 mm.	44.0		
0.0054 mm.	40.3		
0.0027 mm.	34.3		
0.0012 mm.	27.7		

SOIL DESCRIPTION

See exploration logs

ATTERBERG LIMITS

PL = 18 LL = 40 PI = 22

COEFFICIENTS

D_{90} = 0.3773 mm	D_{85} = 0.2722 mm	D_{60} = 0.0499 mm
D_{50} = 0.0166 mm	D_{30} = 0.0016 mm	D_{15} =
D_{10} =	C_u =	C_c =

CLASSIFICATION

USCS = CL

REMARKS

Silt/clay division of 0.002mm used
PI: ASTM D4318, Wet Method
USCS: ASTM D2487

* (no specification provided)

ENGEO
Expect Excellence

CLIENT: Red Tail Land Development, LLC

PROJECT NAME: Sunset Avenue Apartments

PROJECT NO: 7912.001.000 PH001

PROJECT LOCATION: Fairfield, CA

REPORT DATE: 4/13/2021

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 04/09/2021
Date Submitted 04/06/2021

To: Nick Broussard
Engeo, Inc.
2213 Plaza Dr.
Rockin, CA 95765

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 84489.
Your purchase order number is .
Thank you for your business.

SUN #	Sample Describ	Sample #	Chloride as ppm Cl /Dry Wt.	Sulfate as ppm SO4 /Dry Wt.
176155	7912.001.000 SUNSET	2-B1	4.06	5.86

Methods: Sulfate-Cal Trans #417, Chloride-Cal Trans #422m

Sunland Analytical

11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 04/14/2021
Date Submitted 04/07/2021

To: Nick Broussard
Engeo, Inc.
2213 Plaza Dr.
Rocklin, CA 95765

From: Gene Oliphant, Ph.D.
General Manager

The following is the report of analysis requested on SUN Order 84504.
Your purchase order number is .
Thank you for your business.

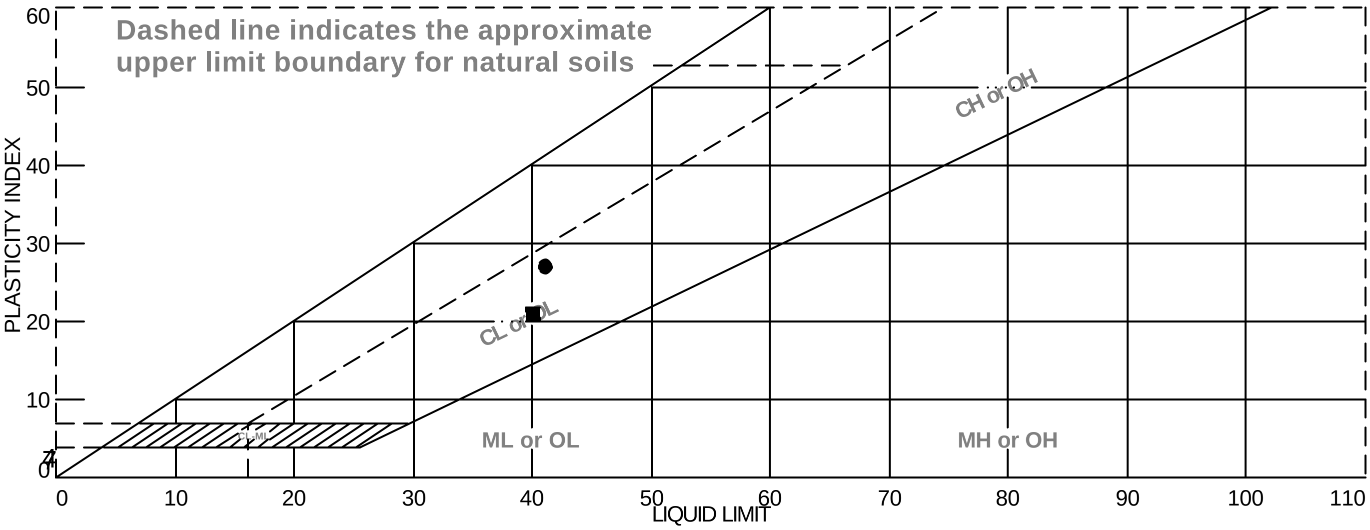
SUN #	Sample Describ	Sample #	Chloride as ppm Cl /Dry Wt.	Sulfate as ppm SO4 /Dry Wt.
176174	7912.001.000 SUNSET	GS-1	4.95	0.17

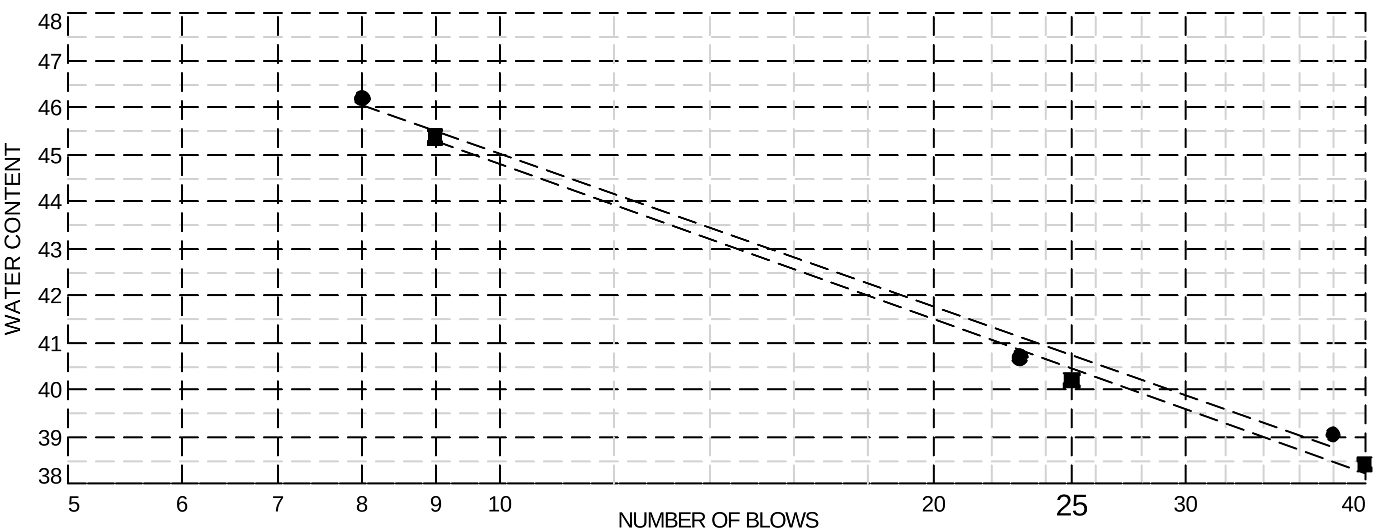
Methods: Sulfate-Cal Trans #417, Chloride-Cal Trans #422m

2007 LABORATORY TEST DATA

Liquid and Plastic Limits Test Report
Unconfined Compression Test
Particle Size Distribution Report
Sulfate Results

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Very dark grayish brown silty CLAY	41	14	27			CL
■	Very dark grayish brown silty CLAY with fine sand	40	19	21		84.4	CL

Project No. 7912.2.001.01 **Client:**

Project: 909 E. Tabor Avenue. Fairfield, CA

● **Depth:** 14.5' **Sample Number:** B1@14.5

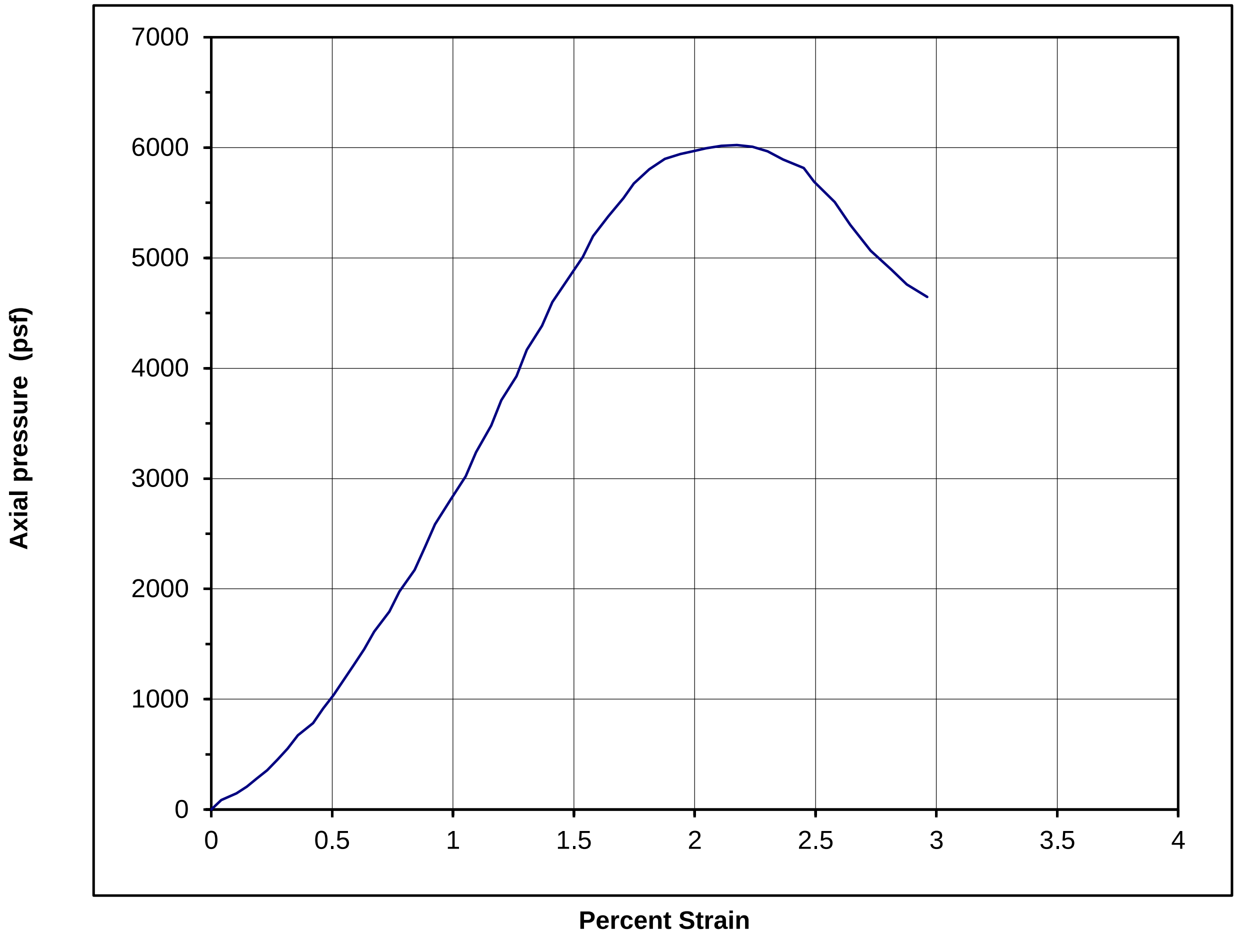
■ **Depth:** 4.5' **Sample Number:** B2@4.5

Remarks:

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Plate

Unconfined Compression Test
ASTM Test Method D2166



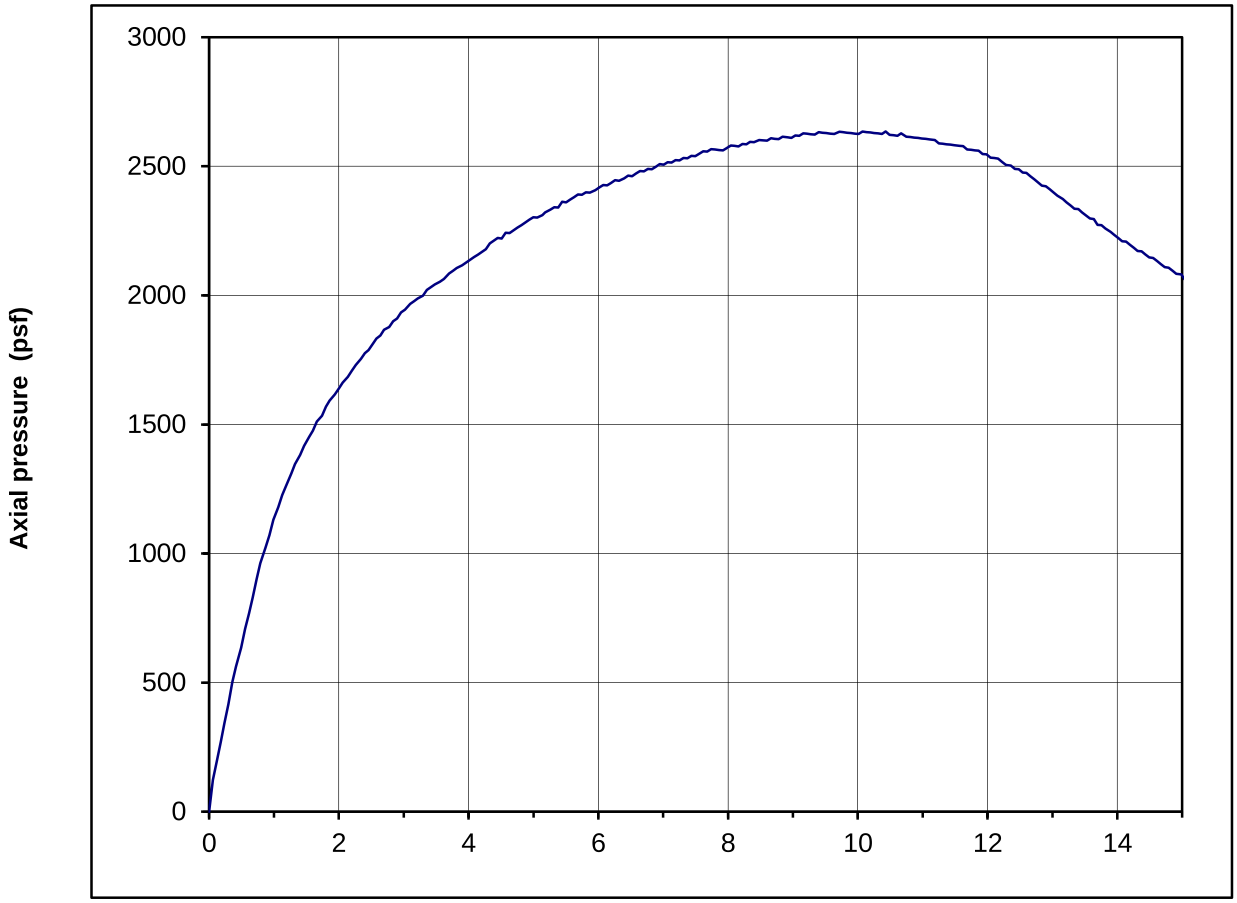
Unconfined Compressive Strength: **6020 psf** **3.0 tsf**

Sample Description: **Dark yellowish brown sandy CLAY**

Initial Diameter:	**2.420**	**in.**	**Sample Number:**	**B3**	
Initial Height:	**4.60**	**in.**	**Dry Unit Weight:**	**110.6**	**pcf**
Strain Rate:	**1.553**	**%/min**	**Moisture Content:**	**13.0**	**%**
Total Strain:	**2.96**	**%**	**Depth of Sample:**	**6.0**	**ft.**

***EN* GEO** INCORPORATED	**909 E. TABOR AVE.** **Fairfield, CA**	**Job No.:** 7912.2.001.01	**Figure No.**
		Sample Number: B3@6	
		Date: 9/12/2007	

Unconfined Compression Test
ASTM Test Method D2166



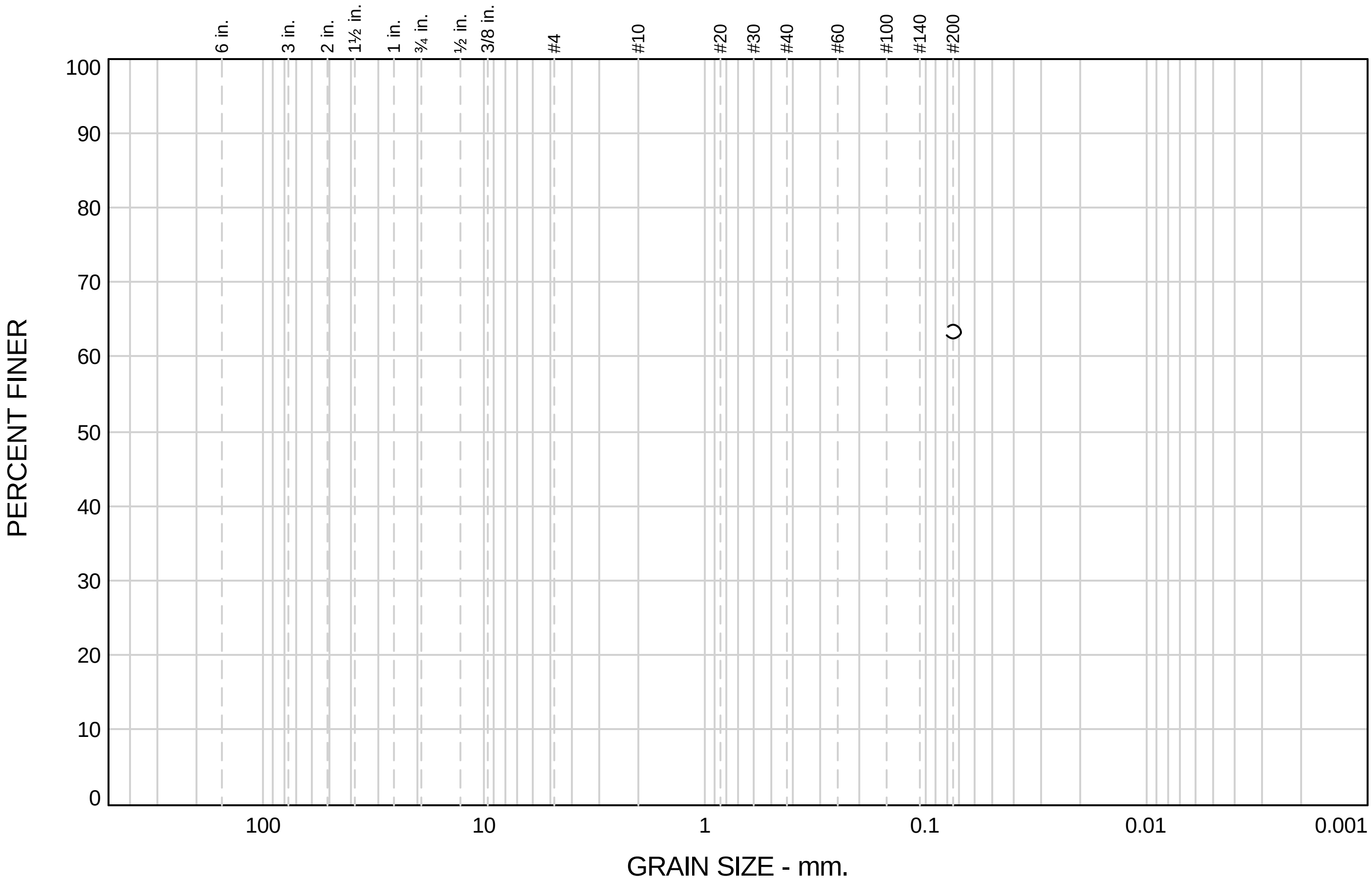
Unconfined Compressive Strength:	**2630 psf**	**1.3 tsf**

Sample Description: **Very dark grayish brown silty CLAY**

Initial Diameter:	**2.420**	**in.**	**Sample Number:**	**B2**	
Initial Height:	**4.90**	**in.**	**Dry Unit Weight:**	**100.2**	**pcf**
Strain Rate:	**1.879**	**%/min**	**Moisture Content:**	**22.6**	**%**
Total Strain:	**15.53**	**%**	**Depth of Sample:**	**7.5**	**ft.**

***EN* GEO** INCORPORATED	**909E. TABOR AVE.** Fairfield, CA	**Job No.:**	**7912.2.001.01**	**Figure No.**
		Sample Number:	**B2@7.5**	
		Date:	**9/12/2007**	

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						63.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	63.4		

* (no specification provided)

Material Description

Yellowish brown sandy silty CLAY

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

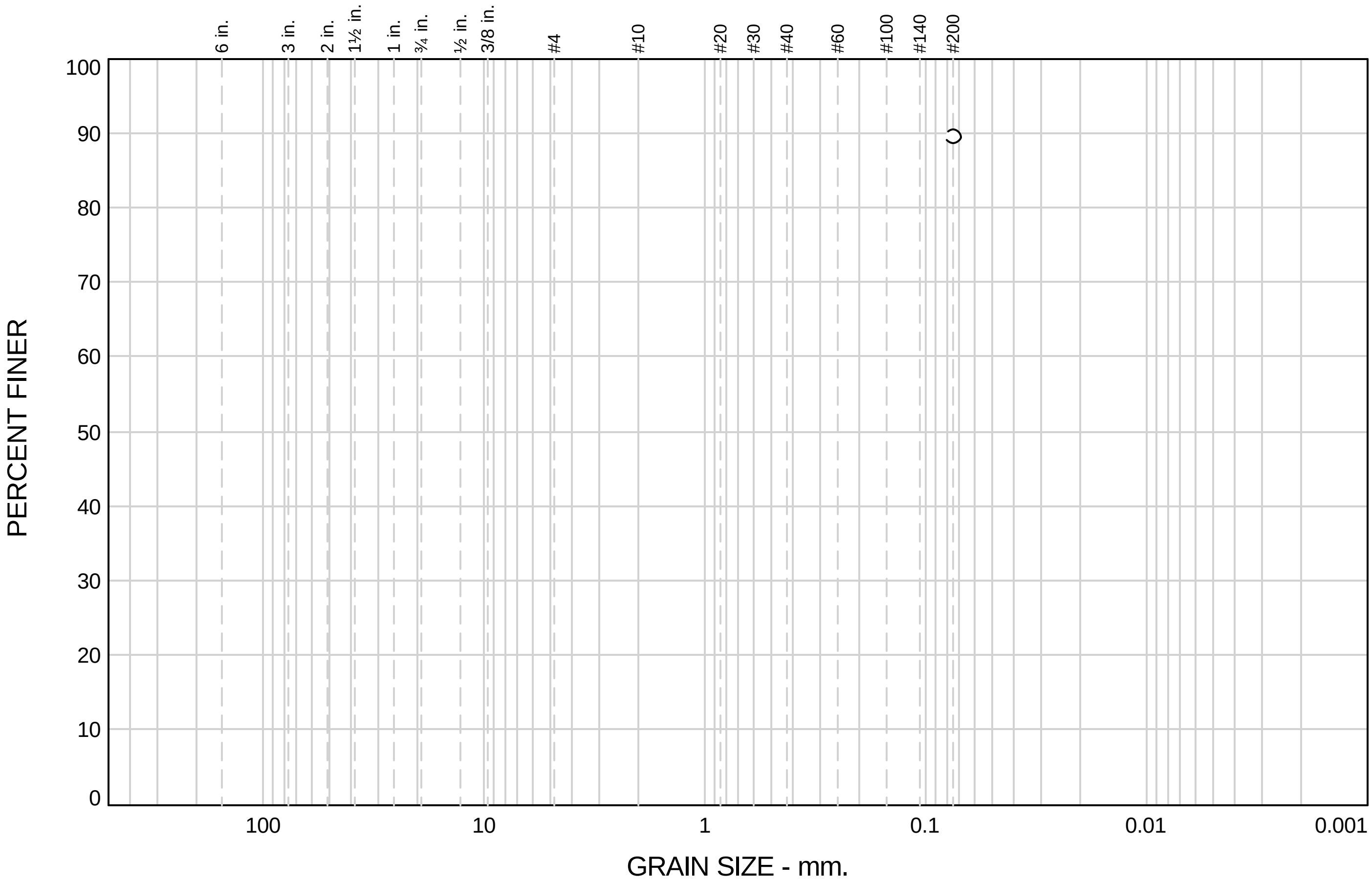
Sample Number: B1@4.5 **Depth:** 4.5' **Date:** 9/14/07

ENGEO INCORPORATED
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS
MATERIALS TESTING

Client:
Project: 909 E. Tabor Avenue. Fairfield, CA

Project No: 7912.2.001.01 **Plate**

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						89.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	89.7		

* (no specification provided)

Material Description

Yellowish brown silty CLAY with sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

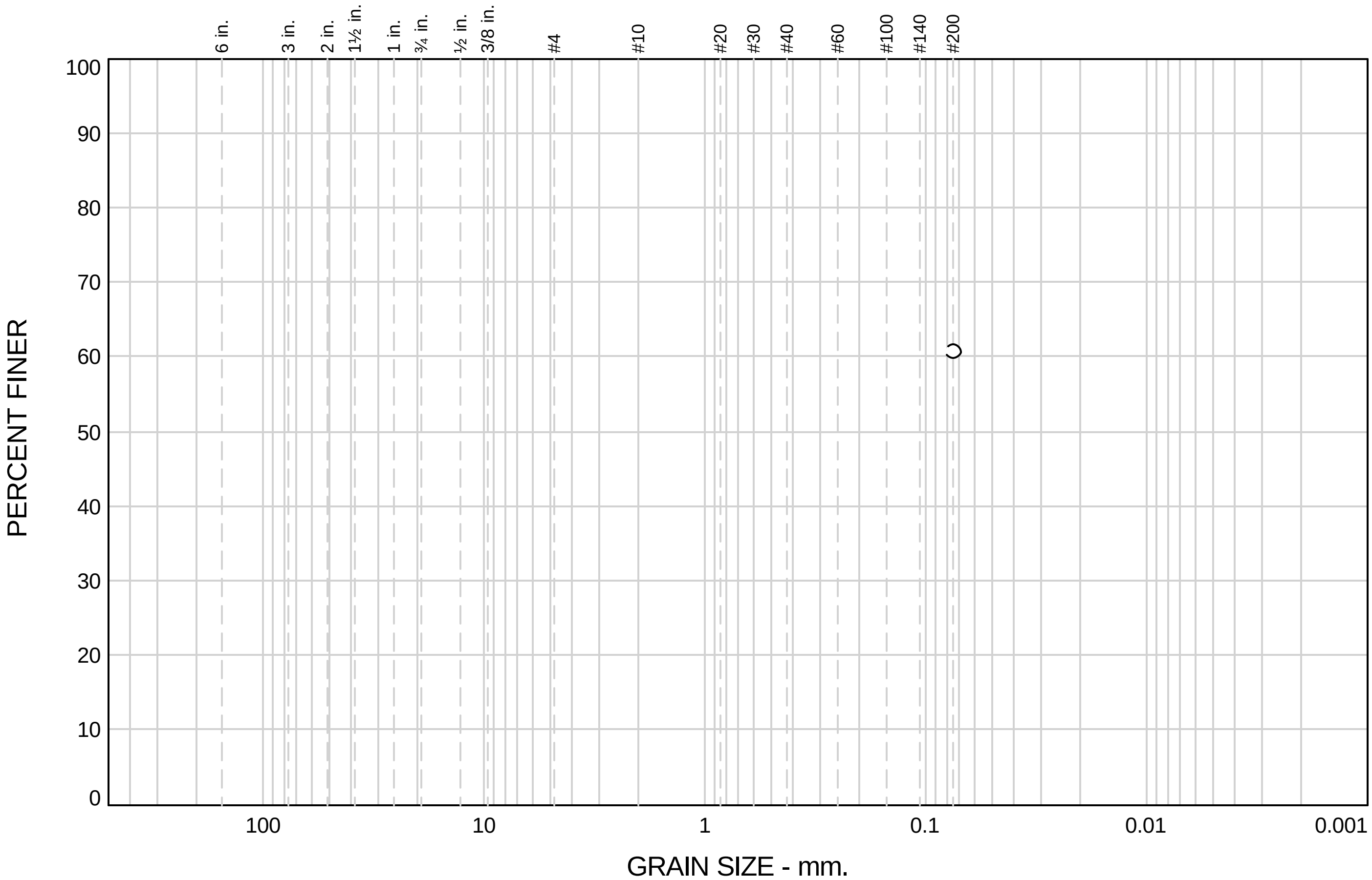
Sample Number: B3@14.5 **Depth:** 14.5' **Date:** 9/14/07

ENGEO INCORPORATED — GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: 909 E. Tabor Avenue. Fairfield, CA

Project No: 7912.2.001.01 **Plate**

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						60.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	60.9		

* (no specification provided)

Material Description

Yellowish brown sandy silty CLAY

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

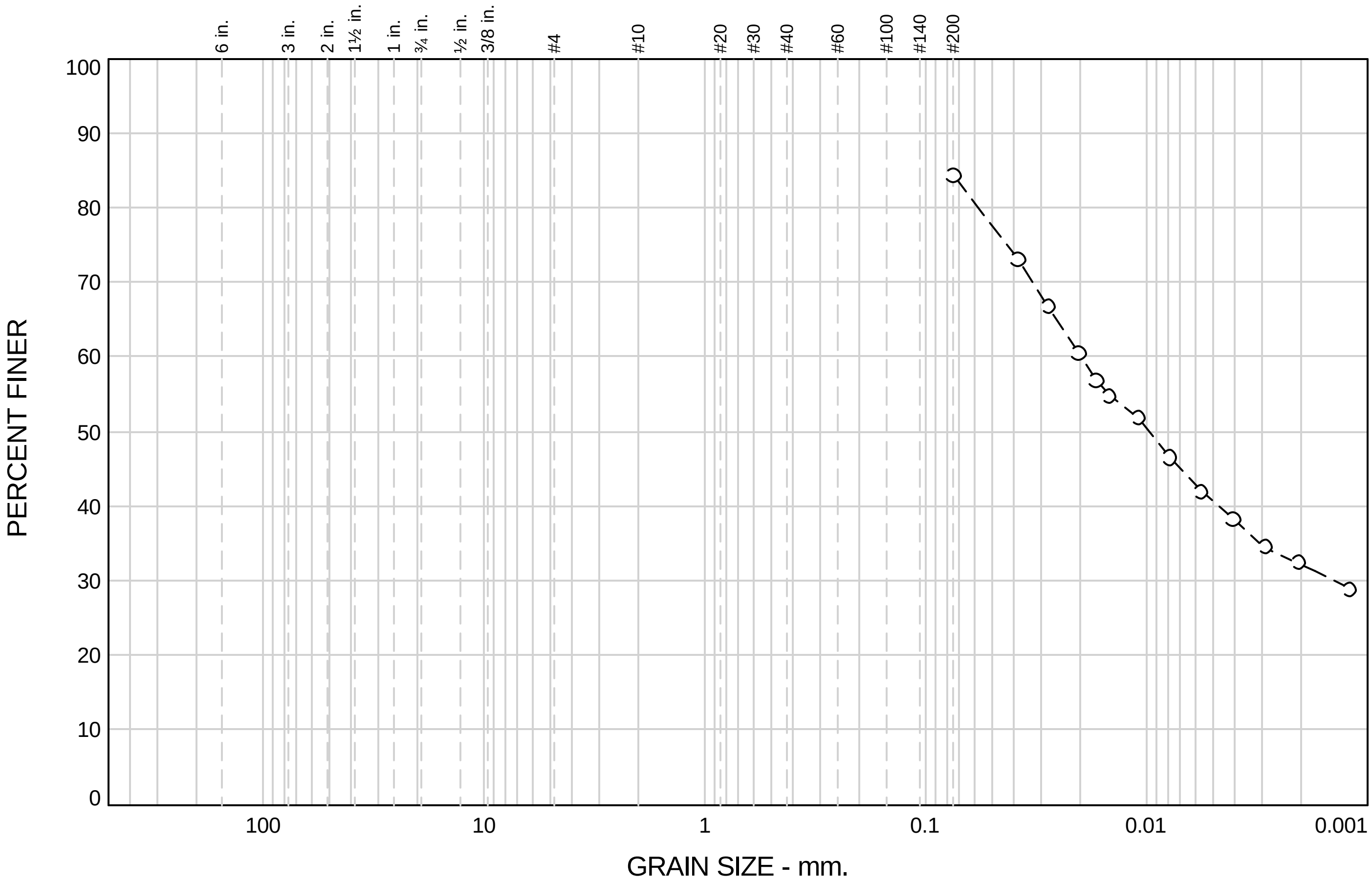
Sample Number: B4@19.5 **Depth:** 19.5' **Date:** 9/14/07

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: 909 E. Tabor Avenue. Fairfield, CA

Project No: 7912.2.001.01 **Plate**

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						52.2	32.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	84.4		

* (no specification provided)

Material Description

Very dark grayish brown silty CLAY with fine sand

Atterberg Limits

PL= 19 LL= 40 PI= 21

Coefficients

D_{85}= D_{60}= 0.0198 D_{50}= 0.0097
D_{30}= 0.0014 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

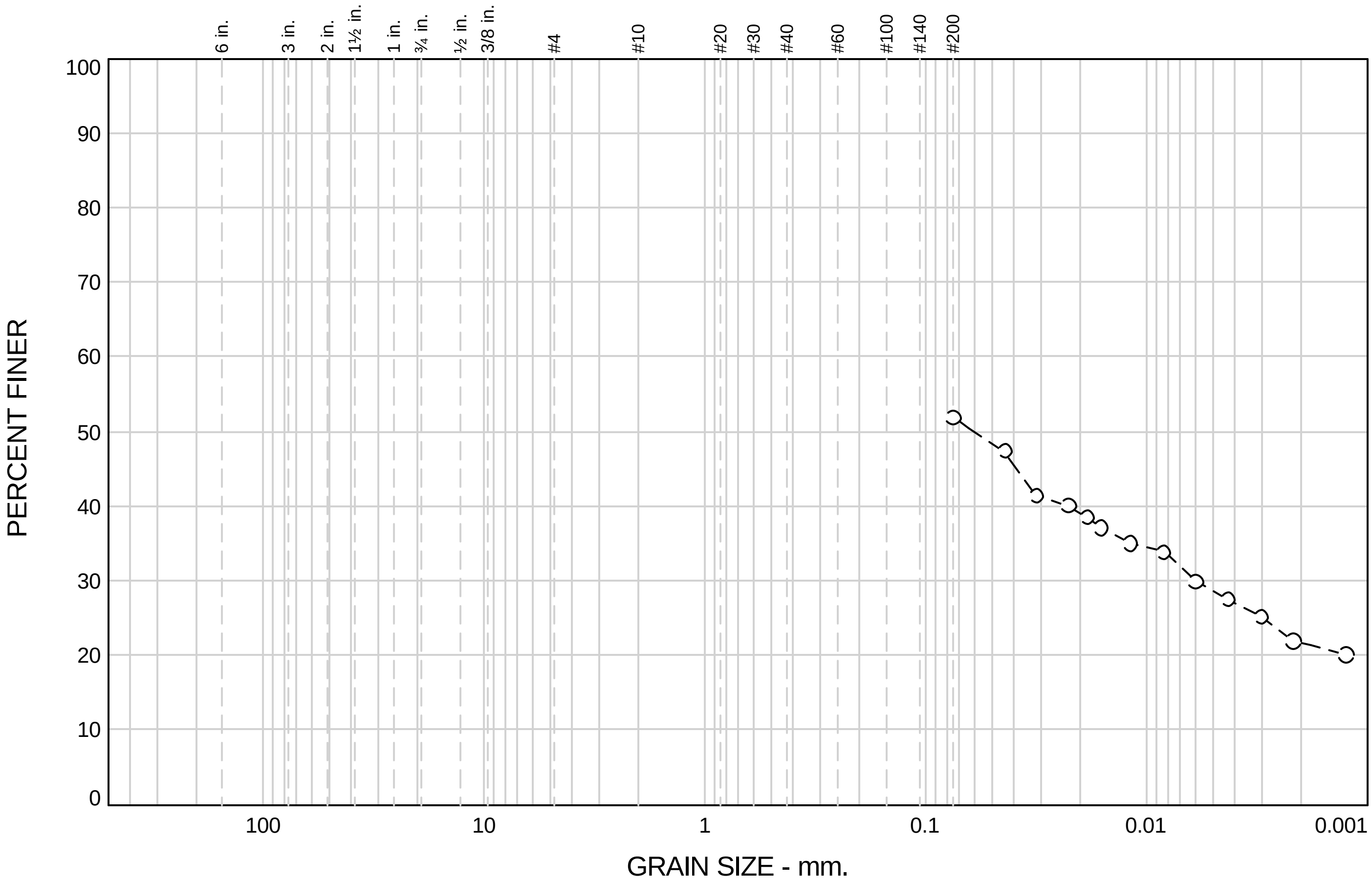
Sample Number: B2@4.5 **Depth:** 4.5' **Date:** 9/14/07

ENGEO INCORPORATED GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

Client:
Project: 909 E. Tabor Avenue. Fairfield, CA

Project No: 7912.2.001.01 **Plate**

Particle Size Distribution Report



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						30.3	21.6

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	51.9		

* (no specification provided)

Material Description

Yellowish brown sandy CLAY

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}= 0.0597
D_{30}= 0.0061 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO=

Remarks

Sample Number: B4@29.5 **Depth:** 29.5' **Date:** 9/14/07

ENGEO INCORPORATED — GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS — MATERIALS TESTING

Client:
Project: 909 E. Tabor Avenue. Fairfield, CA
Project No: 7912.2.001.01 **Plate**

EN GEO Incorporated

SULFATE TEST RESULTS

CALTRANS Test Method 417

Project Name: 909 E. Tabor Avenue   Project Number: 7912.2.001.01

Tested By: RC   Date: September 14, 2007

Measurements less than 15 mg/kg are reported as Not Detectable (ND)

Sample Number	Sample Location	Sample Description	Water Soluble Sulfate (SO_4) in Soil	
			mg/kg	% by Weight
1	B2@4.5'	Soil	113	0.011

APPENDIX C

CPT LOGS

August 29, 2007

Engeo
Attn: Jesus Espinoza
690 Walnut Ave., Suite 220
Mare Island, Vallejo, California 94592

Subject: CPT Site Investigation
909 E. Taber Ave.
Fairfield, California
GREGG Project Number: 07-258MA

Dear Mr. Espinoza:

The following report presents the results of GREGG Drilling & Testing's 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	Resistivity Cone Penetration Tests	(RCPTU)	☐
5	UVIF Cone Penetration Tests	(UVIFCPTU)	☐
6	Groundwater Sampling	(GWS)	☐
7	Soil Sampling	(SS)	☐
8	Vapor Sampling	(VS)	☐
9	Vane Shear Testing	(VST)	☐
10	SPT Energy Calibration	(SPTE)	☐

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 our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager

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)
CPT-01a	8/27/07	68.4	-	-	-
CPT-02	8/27/07	50	-	-	26.2

Bibliography

Lunne, T., Robertson, P.K. and Powell, J.J.M., “Cone Penetration Testing in Geotechnical Practice” E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., “Soil Classification using the Cone Penetration Test”, Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

Mayne, P.W., “NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization”, available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

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, 1986 pp. 791-803.

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.

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Campanella, R.G. and I. Weemees, “Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies”, Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegger, “Reliability of Soil Gas Sampling and Characterization Techniques”, International Site Characterization Conference - Atlanta, 1998.

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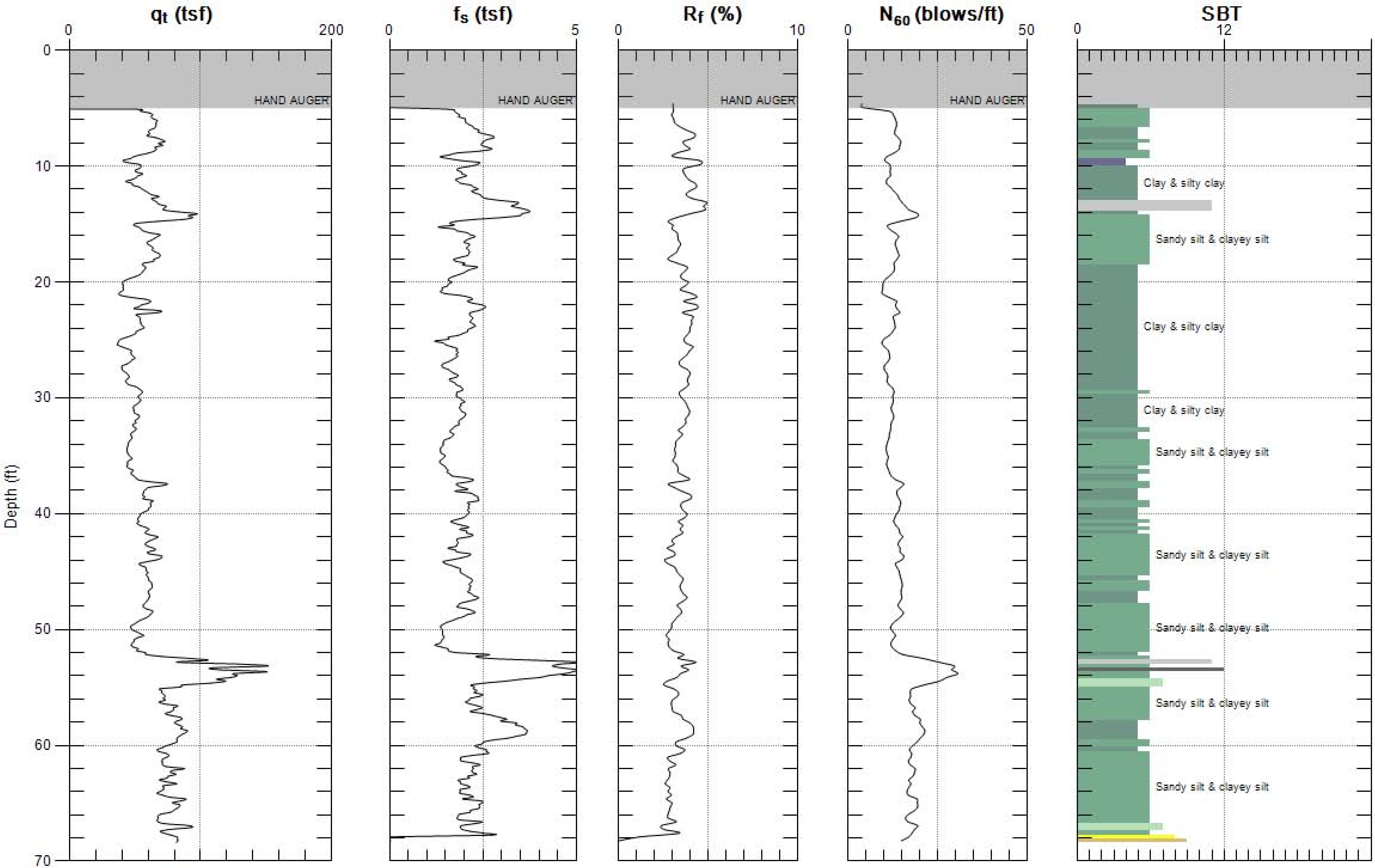
Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, “Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers” BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

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

GREGG ENGEO

Site: 909 E. TABER AVE.
Sounding: CPT-01A

Engineer: J.ESPINOZA
Date: 8/27/2007 10:17



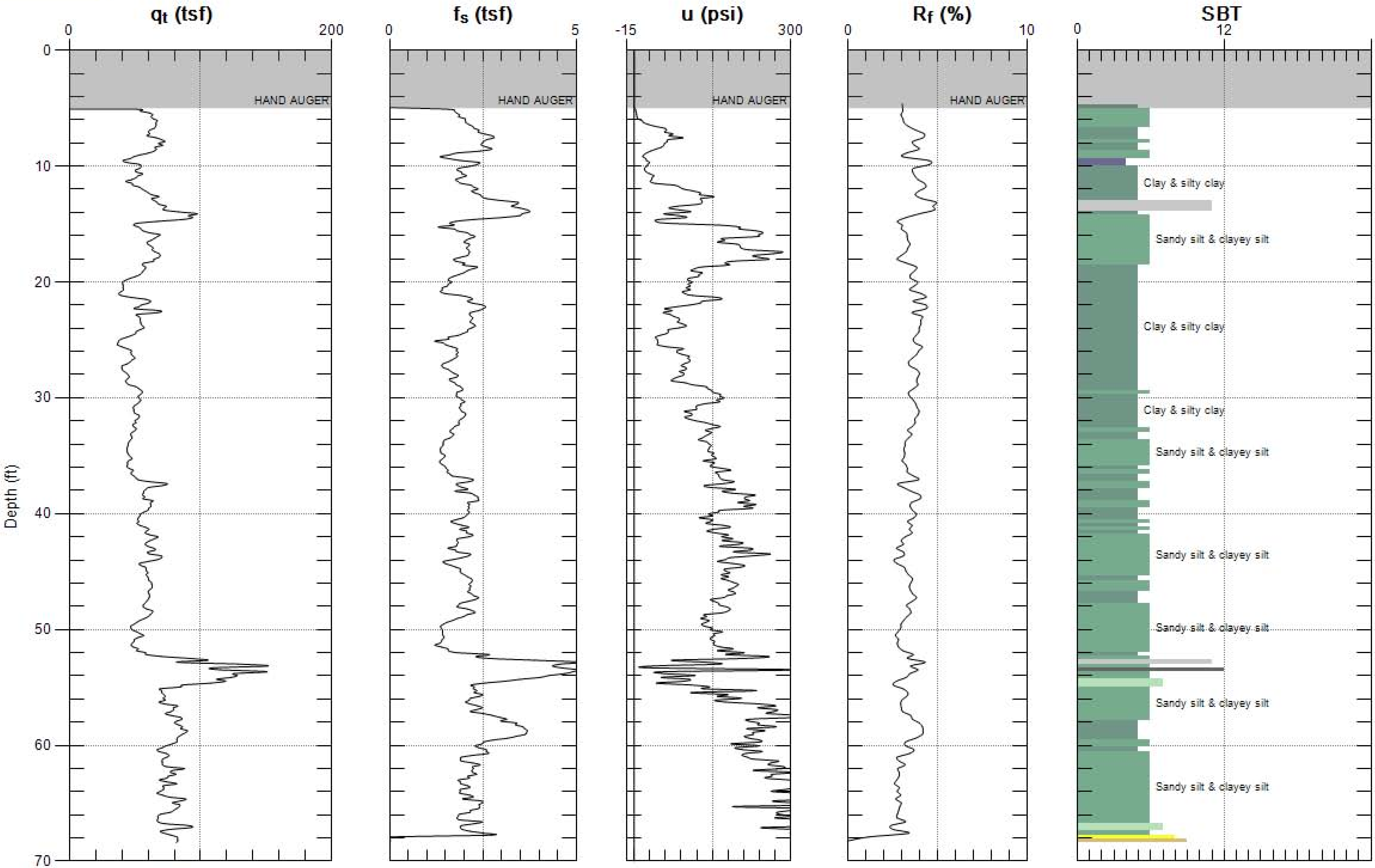
Max. Depth: 68.406 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **ENGEO**

Site: 909 E. TABER AVE. Engineer: J.ESPINOZA

Sounding: CPT-01A Date: 8/27/2007 10:17



Max. Depth: 68.406 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

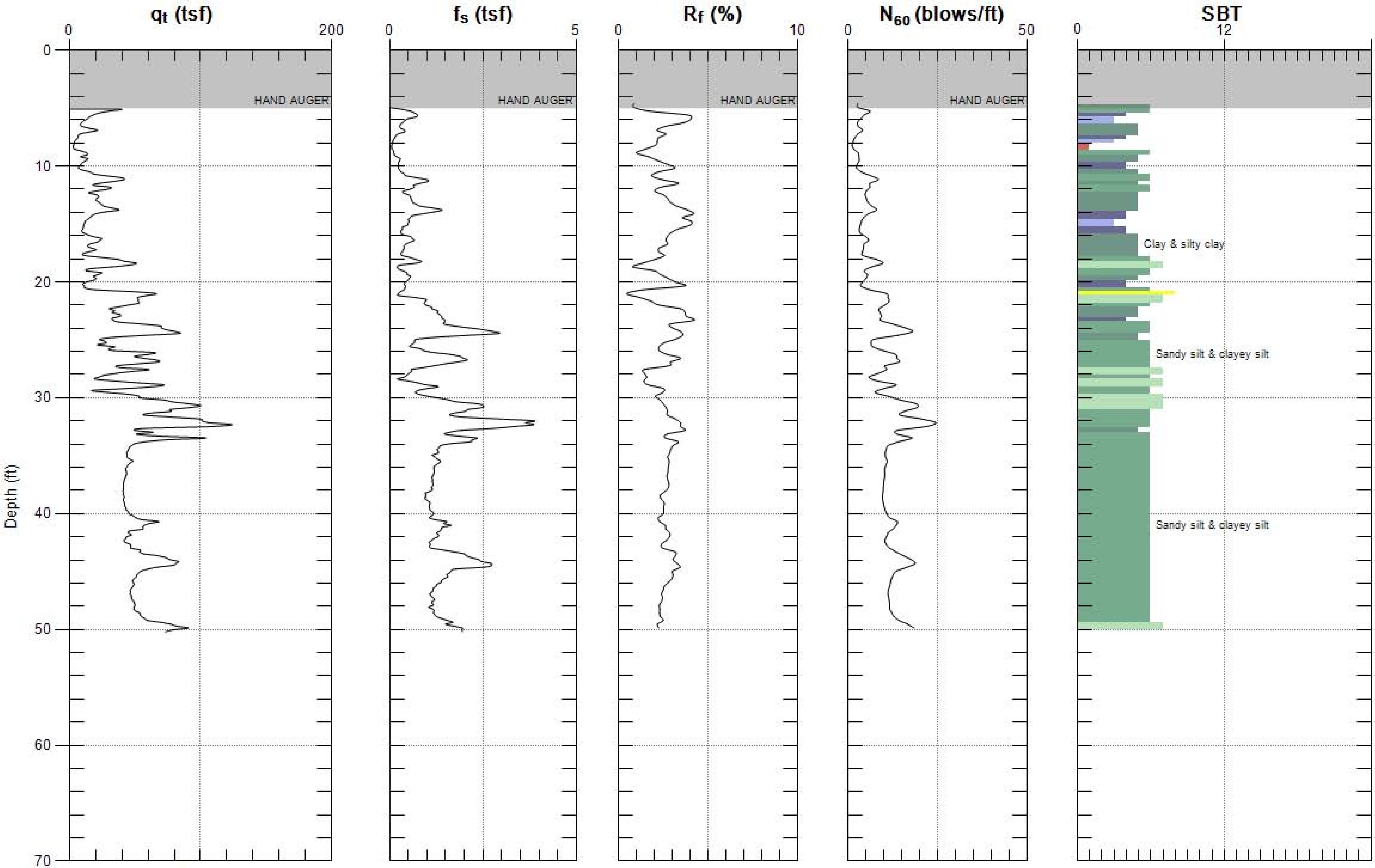
ENGEO

Site: 909 E. TABER AVE.

Sounding: CPT-02

Engineer: J.ESPINOZA

Date: 8/27/2007 01:03



Max. Depth: 50.197 (ft)

Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

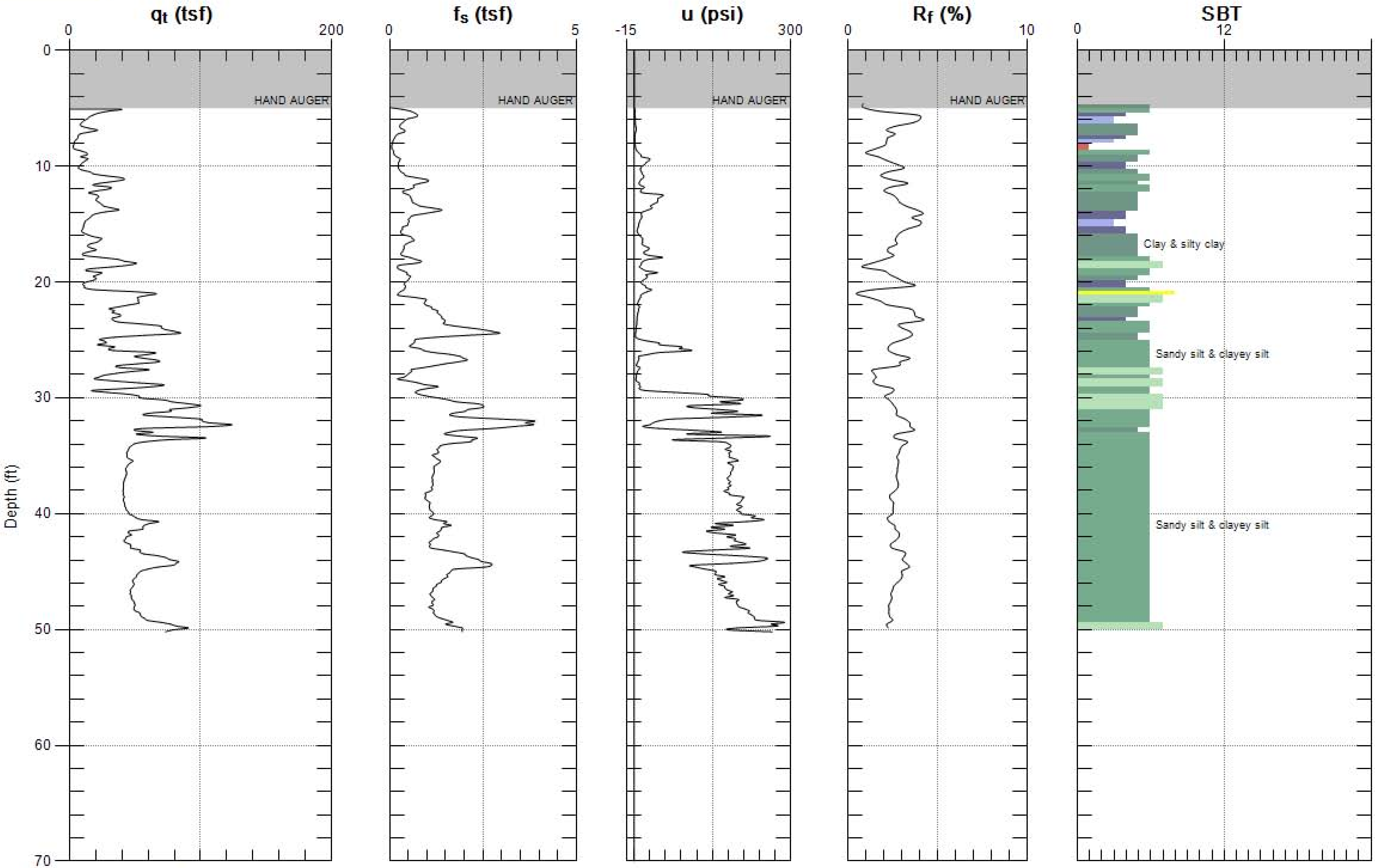
ENGEO

Site: 909 E. TABER AVE.

Sounding: CPT-02

Engineer: J.ESPINOZA

Date: 8/27/2007 01:03



Max. Depth: 50.197 (ft)

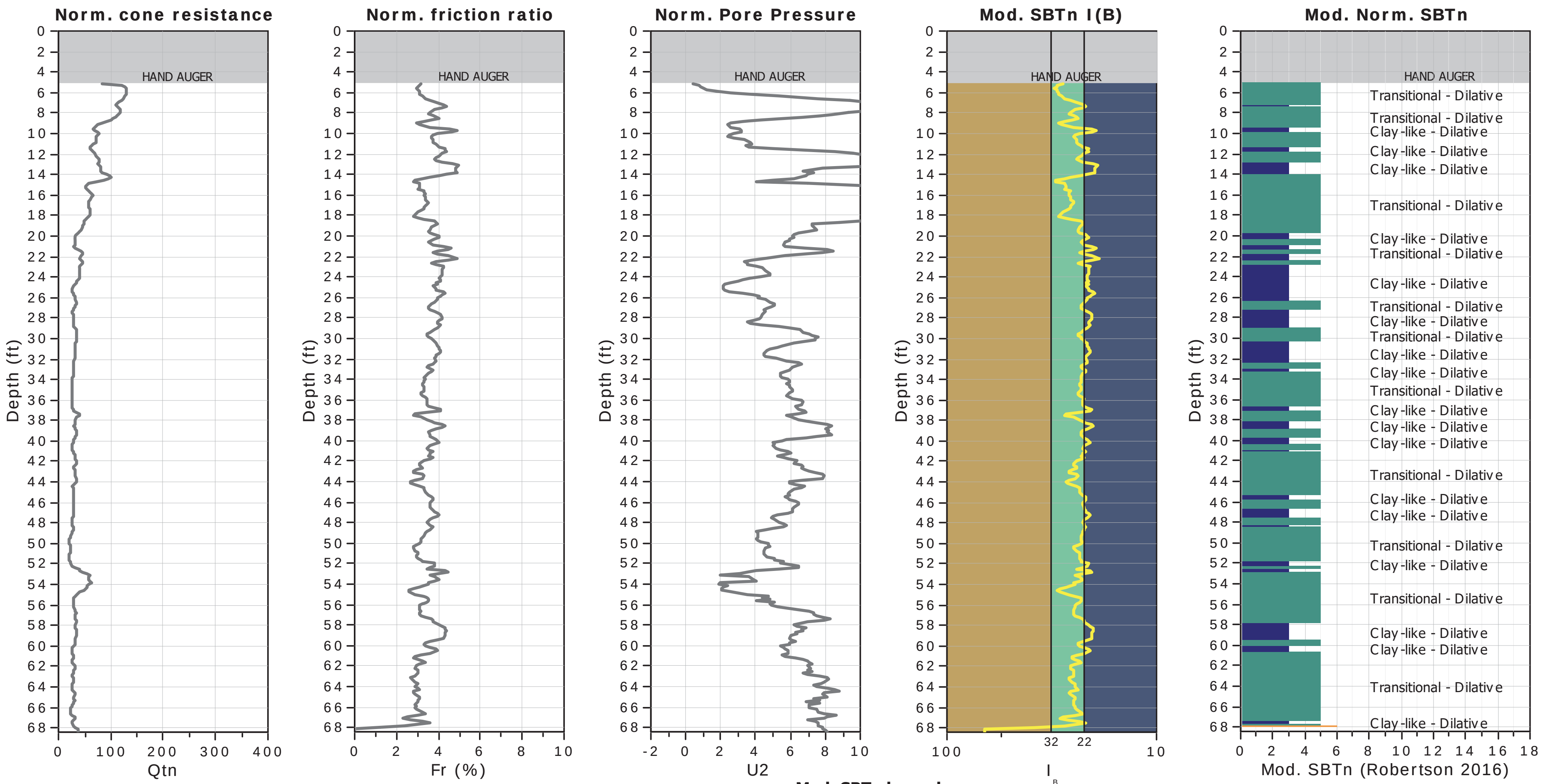
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

ENGEO, Incorporated
2213 Plaza Drive
Rocklin, CA 95765
engeo.com

Project: Sunset Ave Apartments
Location: Fairfield, CA

CPT: CPT-01
Total depth: 68.41 ft



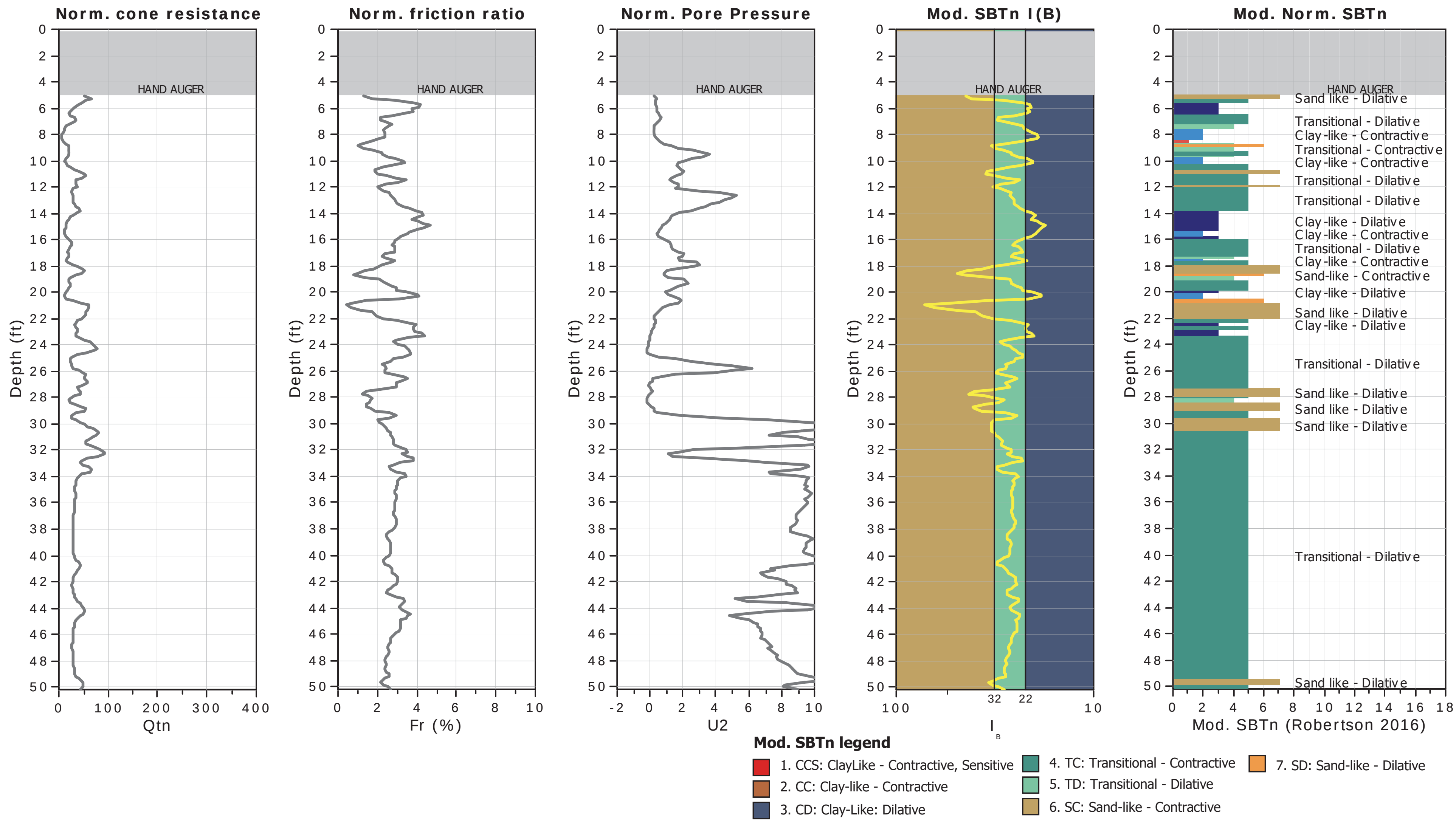
Mod. SBTn legend

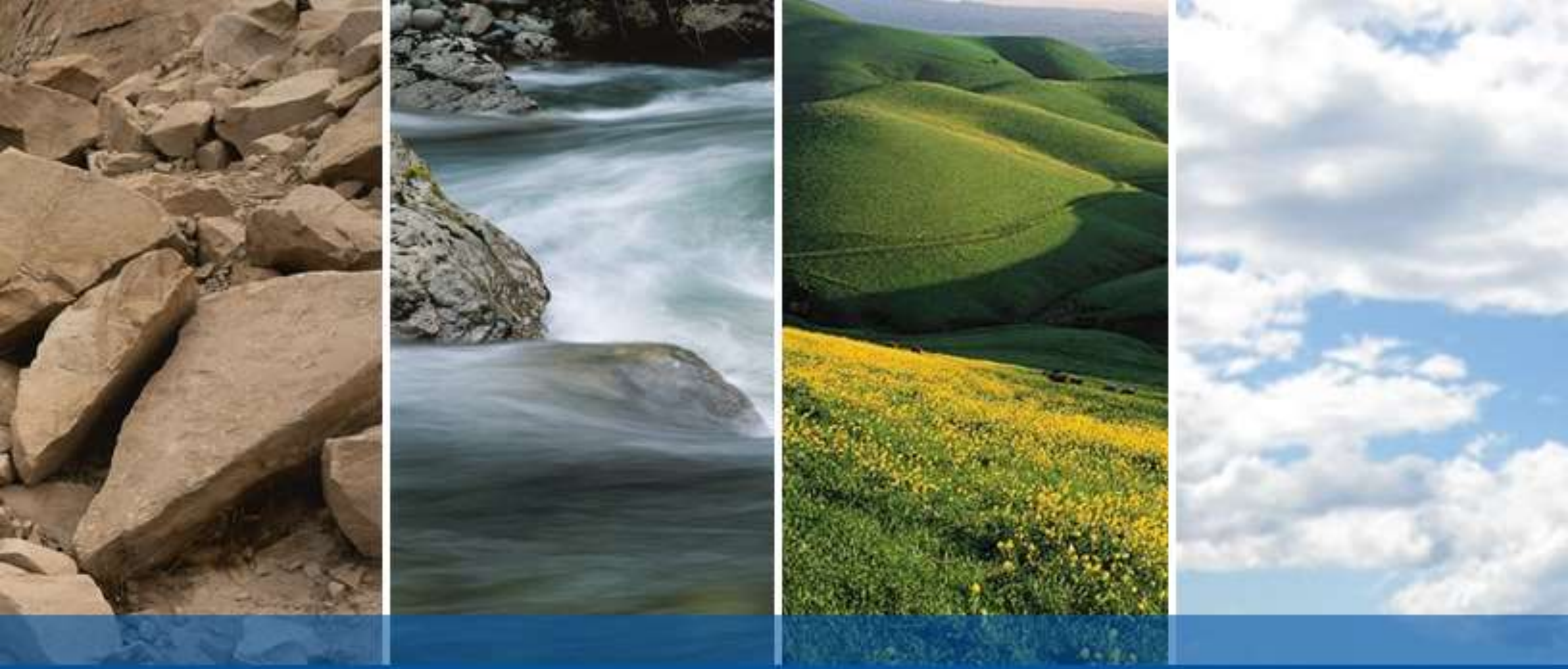
1. CCS: ClayLike - Contractive, Sensitive
2. CC: Clay-like - Contractive
3. CD: Clay-Like: Dilative
4. TC: Transitional - Contractive
5. TD: Transitional - Dilative
6. SC: Sand-like - Contractive
7. SD: Sand-like - Dilative

ENGEO, Incorporated
2213 Plaza Drive
Rocklin, CA 95765
engeo.com

Project: Sunset Ave Apartments
Location: Fairfield, CA

CPT: CPT-02
Total depth: 50.20 ft





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