
Appendix D:

Geology and Soils Supporting Information

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ANTON FONG RANCH
SACRAMENTO, CALIFORNIA

GEOTECHNICAL EXPLORATION

SUBMITTED TO
Ms. Le Anne Thomas
Acquisitions Manager
Anton Fong Ranch, LLC
1610 R Street, Suite 250
Sacramento, CA 95811

PREPARED BY
ENGEO Incorporated

April 4, 2022

PROJECT NO.
20044.000.001

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Ms. Le Anne Thomas
Acquisitions Manager
Anton Fong Ranch, LLC
1610 R Street, Suite 250
Sacramento, CA 95811

Subject: Anton Fong Ranch
3625 Fong Ranch Road
Sacramento, California

GEOTECHNICAL EXPLORATION

Dear Ms. Thomas:

ENGEO prepared this geotechnical report for Anton Fong Ranch, LLC as outlined in our agreement dated February 16, 2022. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations for design.

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. Please let us know when working drawings are nearing completion so that we can coordinate review of the plans and specifications and discuss geotechnical testing and observation services during construction.

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

Sincerely,

ENGEO Incorporated



Allison Hauger



Mark Gilbert, GE

ah/nb/mg/jf



Nicholas Broussard, GE

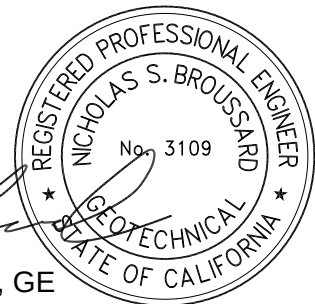


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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this geotechnical report for design of Anton Fong Ranch in Sacramento, California. As outlined in our agreement dated February 16, 2022, you authorized ENGEO to conduct the following scope of services.

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

For our use, we received the following.

- Withee Malcolm (2021); Site Study #3 for Fong Ranch Road, Sacramento, CA, October 4, 2021.
- Brusca Associates, Inc. (2022); Phase I Environmental Site Assessment, March 23, 2022.
- Morrow Surveying, Inc. (2022); Alta/NSPS Land Title Survey, 3625 Fong Ranch Road, Sacramento, California, March 16, 2022.

Additionally, we reviewed nearby geotechnical data associated with the levees along the edge of the Reclamation District (RD) 1000 B Drain that crosses through the site (Kleinfelder, 1996). The levees along the RD1000 B Drain were not studied as part of this scope of work. We understand these levees are under evaluation by the City of Sacramento and maintained by RD1000.

This report was prepared for the exclusive use of our client and their consultants for design of this project. If 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

Figure 1 displays a Site Vicinity Map. The approximately 22.8-acre site is located at 3625 Fong Ranch Road in Sacramento, California. The east-to-west-oriented RD1000 B Drain bisects the site, with approximately 4½ acres of the site north of the RD1000 B Drain.

Figure 2 shows site boundaries, proposed building and pavement areas, and our exploratory locations. The site is bordered by Interstate 80 to the north and northwest, Truxel Road to the west, and Fong Ranch Road to the east. Two schools, Discovery High School and Natomas High School, border the site to the south. Neighboring developments include residential subdivisions.

1.3 PROJECT DESCRIPTION

The preliminary site plans indicate the multi-family residential project will include construction of the following items on the portion of the site that is south of the RD1000 B Drain.

- Three- and four-story apartment buildings, that will consist of a total of 406 market rate apartments and 149 affordable units among multiple buildings
- One-story amenity and pool building
- Swimming pool
- Parking lots with carports
- Sidewalks
- A bike path
- Underground utilities
- Landscaping
- Bioretention swales

Structural loads and grading are yet to be determined; however, we assume that structural loads will be representative for this type of construction and that only minor grading will be required. We anticipate a perimeter soundwall and minor landscape walls may be incorporated into the project.

2.0 FINDINGS

2.1 SITE BACKGROUND

We reviewed the following historical topographic maps and aerial photographs that are included in the referenced Phase 1 Environmental Site Assessment (Brusca Associates, 2022).

TABLE 2.1-1: Historical Topographic Map and Aerial Photo Review Summary

HISTORICAL MAP/PHOTOGRAPH	YEARS
Topographic Maps	1891, 1892, 1893, 1902, 1911, 1950, 1951, 1954, 1967, 1980, 1992, 2012, 2015, 2018
Aerial Photographs	1937, 1947, 1953, 1964, 1966, 1972, 1984, 1993, 1998, 2006, 2009, 2012, 2016, 2021

In 1902 and 1911, the site was located within a low-lying area called Bush Lake, as shown in Exhibit 2.1-1. In 1937, Bush Lake was no longer visible and the site appeared to be under cultivation.

A drainage canal, now known as RD1000 B Drain, is first visible on the site in the 1937 aerial photograph and 1950 topographic map. We understand this drainage canal and associated levees were constructed in the early 1900s, improved in the 1970s, and then improved by flattening slopes and widening in the 1990s (Kleinfelder, 1996).

In 1937 through 1993 photographs, two unpaved agricultural roads were on the site. The unpaved agricultural roadways are no longer visible in the 1998 photograph.

Interstate 80 was visible in aerial photographs from 1972 to 2021, with the adjacent on-ramp to Interstate 80 constructed between 1993 and 1998. Minor soil stockpiles are visible in the aerial photographs from 2002 to 2021 along the southernmost site boundary. The site no longer appeared to be under cultivation by 2009.

EXHIBIT 2.1-1: Topographic Map (USGS, 1911 and 1915)



2.2 EXISTING GEOTECHNICAL DATA

The RD1000 B Drain levees were evaluated in 1996 to obtain FEMA certification. The geotechnical report associated with the evaluation included three soil borings drilled through the top of the levees that cross through the site, or are within close proximity; B-17, B-18, and B-26 (Kleinfelder, 1996). These soil boring logs are attached in Appendix D.

2.3 FIELD EXPLORATION

Our field exploration included drilling five borings and advancing five cone penetration test (CPT) soundings at various locations on the site. We performed our field exploration on March 11, 2022.

The location and elevations of our explorations are approximate and were estimated from features shown on the site plan; they should be considered accurate only to the degree implied by the method used.

2.3.1 Borings

We observed drilling of five borings at the locations shown on the Site Plan, Figure 2. An ENGEO representative observed the drilling and logged the subsurface conditions at each location. We retained a truck-mounted CME 55 drill rig and crew to advance the borings using 5-inch-diameter solid-flight auger and mud rotary methods. The borings were advanced to depths ranging from 13 to 51½ feet below existing grade. We permitted and backfilled the borings in accordance with the requirements of Sacramento County.

We obtained disturbed soil samples at various intervals in the borings using standard penetration tests. 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, the 3-inch O.D. Modified California Sampler was also used to obtain samples (driven 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.

The logs depict subsurface conditions at the exploration locations during the exploration; however, subsurface conditions may vary with time. The soil boring logs are included in Appendix A.

2.3.2 Cone Penetration Tests

We retained a 30-ton truck-mounted CPT rig to push the five cone penetrometers to a maximum depth of about 50 feet. The CPT has a 20-ton compression-type cone with a 10-square-centimeter (cm²) base area, an apex angle of 60 degrees, and a friction sleeve with a surface area of 225 cm². 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 D-5778. Measurements include the tip resistance to penetration of the cone (Qc), the resistance of the surface sleeve (Fs), and pore pressure (U) (Robertson and Campanella, 1988). The CPT operator also collected shear-wave velocity measurements in CPT 1-CPT1 and 1-CPT3. The CPT logs and shear wave velocity data interpretation are presented in Appendix C.

2.4 GEOLOGY

The site is located in the Great Valley Geomorphic Province. The Great Valley is an elongate, northwest-trending structural trough bound by the Coast Ranges on the west and the Sierra Nevada on the east. The Great Valley has been and is presently being filled with sediments primarily derived from surrounding mountain ranges.

As shown in Figure 3, Regional Geologic Map, the site is mapped as Holocene Alluvium (Qha), typically consisting of poorly to moderately sorted sand, gravel, and silt, and represents a higher energy deposit associated with modern day river systems (Gutierrez, 2011). However, Holocene Basin Deposits (Qhb), which are generally fine-grained silt and clay in topographically low areas,

are mapped nearby. This is consistent with the clay encountered in our borings. The site is mapped as having an average shear wave velocity in the upper 100 feet of 228 meters per second (748 feet per second) (Wills, 2015).

Older Pleistocene deposits such as Modesto Formation (11,700 to 42,000 years old) and/or Riverbank Formation (130,000 to 450,000 years old) underlie the Holocene basin and alluvial deposits in this area (Gutierrez, 2011; Helley and Harwood, 1985); this was confirmed by our exploration. These Pleistocene alluvial formations consist of gravel, sand, silt, and clay that generally show evidence of aging such as increased density, weathering, and cementation.

2.5 FAULTING AND SITE SEISMICITY

The Northern California Region contains numerous active earthquake faults. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone and no known surface expression of active faults is believed to exist within the site. An active fault is defined by the California Geologic Survey as one that has had surface displacement within Holocene time (about the last 11,700 years) (CGS, 2018).

Although fault rupture is not anticipated, an earthquake in the region could generate ground shaking at the site. Numerous small earthquakes occur every year in the Northern California region, and larger earthquakes have been recorded and can be expected to occur in the future. The Uniform California Earthquake Rupture Forecast (UCERF3) estimates the 30-year probability for a magnitude 6.7 or greater earthquake in Northern California Region at approximately 95 percent (Field et al., 2015).

The table below summarizes the distance to the fault rupture surface (R_{rup}) and the associated moment magnitude for nearby seismic sources used for the National Seismic Hazard Maps, which are incorporated into the California Building Code (CBC). We obtained the data using the USGS Unified Hazard Tool (Dynamic Conterminous U.S. 2014 (update) (v4.2.0)) and deaggregated the hazard at the peak ground acceleration (PGA) for 2,475-year return period, and Site Class D. These results represent fault sources contributing at least one percent to the seismic hazard at the site; gridded or areal sources are not presented.

TABLE 2.5-1: Nearby Seismic Sources (Latitude: 38.6343 Longitude: -121.4955)

SOURCE	R_{rup}		MOMENT MAGNITUDE M_w
	(KM)	(MILES)	
Great Valley 04a Trout Creek [2]	47.5	29.5	7.22
Hunting Creek – Berryessa [3]	65.4	40.7	7.46
Great Valley 03a Dunnigan Hills [0]	26.9	16.7	6.19
Great Valley 06 (Midland) alt1 [0]	37.6	23.4	6.86

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

Numerous small earthquakes occur every year in the Northern California Region, and larger earthquakes have been recorded and can be expected to occur in the future. Figure 4 shows the approximate locations of faults and significant historic earthquake epicenters recorded within the Northern California Region.

2.6 SURFACE CONDITIONS

The ALTA Survey indicates that site grades are relatively flat, with typical grades of approximately Elevation 11 to 13 feet (Datum: NAVD88) (Morrow, 2022). Along the northeast site boundary, site grades slope up to approximate Elevation 17 feet. The survey data indicates the RD 1000 B Drain is at least 4 feet deep below adjacent site grades, or at least 7 feet below the top of the adjacent levees.

We observed the following site features during our reconnaissance.

- Levees oriented along the north and south sides of the RD1000 B Drain were roughly 4 feet tall or less. The levees were densely vegetated by weeds and shrubs.
- The ground surface south of the RD1000 B Drain consisted of dried grasses, while the ground surface north of the RD1000 B Drain was densely vegetated with weeds. Mature trees were located along the southern border of the site.
- There was a chain link fence along the northwestern boundary along the on-ramp to Interstate 80, along the north side of the RD1000 B Drain, and along the southern site boundary. A transient camp and debris was in the northwest along the on-ramp to Interstate 80.
- Minor concrete debris and soil stockpiles along the southern boundary.
- Large soil stockpile with silt fencing located along the northeastern boundary.

PHOTO 2.6-1: Site Conditions South of RD1000 B Drain From East Looking West



PHOTO 2.6-2: Site Conditions North of RD1000 B Drain From West Looking East



Please refer to the Site Plan, Figure 2, for more information on site features.

2.7 SUBSURFACE CONDITIONS

We encountered fill in each of our five soil borings. The fill thickness varied from approximately 2 to 4 feet, as indicated on the boring logs and the Site Plan. The fill consisted of stiff to hard fat clay and lean clay with medium to high plasticity. Laboratory testing of the fill from Borings 1-B1 and 1-B4 resulted in plasticity index values of 32 and 34.

Below the fill, the borings generally encountered medium to high plasticity lean clay, fat clay, and elastic silt. The native clay and silt was very stiff to hard. Laboratory testing of the clay and silt from Borings 1-B4 and 1-B5 resulted in plasticity index values of 39 and 47. Below the native clays and silt, the borings encountered medium dense clayey sand and silty sand, and stiff to hard silt. The sand and silt was underlain by silt, silty sand, clayey sand, sand, and clay. We encountered loose to medium dense sand in Boring 1-B1 from a depth of 24 feet to the depth explored of 33 feet. We did not encounter any noticeably weak or compressible soil in our exploratory borings.

The CPT-interpreted stratigraphy appeared consistent with the soil borings.

The boring logs shown on Figure 2 by Kleinfelder (1996), encountered stiff to hard silt and fat clay in the upper 18 to 21½ feet, overlaying loose to medium dense sand. We include the Kleinfelder boring logs in Appendix C.

Consult the Site Plan (Figure 2) and boring logs (Appendix A) for specific subsurface conditions at each location. 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.

2.8 GROUNDWATER CONDITIONS

We encountered groundwater in Boring 1-B1 at a depth of 24 feet below the ground surface at the time of drilling; we measured the groundwater depth again 5 minutes later at a depth of 17 feet. We did not detect groundwater in the other borings.

We reviewed GeoTracker and SGMA Data Viewer, websites maintained by the State of California Water Resources Control Board for nearby facilities with records that include depth to groundwater measurements. The following information was publically available regarding local groundwater conditions.

- Monitoring well data collected from Well 386292N1214877W001 at Chuckwagon Park, located approximately ½ mile southeast of the site, indicated that groundwater between October 2011 and March 2018 was between approximately Elevation 0 to -8.6 feet (Datum NAVD88); this corresponds with depths that varied from approximately 13 feet to 21 feet below the surface grade at Elevation 12 feet.
- Monitoring well data collected from Well 386160N1215054W001 at Bannon Park, located approximately 1½ miles southwest of the site, indicated that groundwater between October 2011 and September 2018 was between approximately Elevation 0 to 6 feet (Datum NAVD88); this corresponds with depths that varied from approximately 8 to 14 feet below grade.

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.

2.9 LABORATORY TESTING

We performed laboratory tests on selected soil samples to evaluate their engineering properties. For this project, we performed moisture content, dry density, unconfined compression, compaction curve, remolded direct shear, plasticity index, hydrometer, and soil corrosion potential

testing. Moisture contents and dry densities are recorded on the boring logs in Appendix A; other laboratory data is included in Appendix B.

2.10 LIQUEFACTION ANALYSES

We performed a liquefaction evaluation using the data from the CPTs and mud rotary boring. Our analyses incorporated the 2019 CBC Site Class D peak ground acceleration of 0.32g, an earthquake magnitude of 6.6 obtained from the mean of the USGS disaggregation, and groundwater depth of 13 feet. We based the groundwater depth used in the analyses on groundwater data summarized in Section 2.8.

For our CPT-based liquefaction analysis, we utilized the commercially available software program CLiq (v3.4.1.2) and incorporated methodologies by Robertson (2009). Our analysis indicates some 1- to 6-foot-thick loose to medium-dense sand layers are potentially liquefiable at variable depths ranging from 17 to 30 feet.

Based on the findings published by Youd and Garris in 1995, sufficiently thick non-liquefiable soil that overlies liquefiable layers provide a capping effect, which has been observed to result in much less ground surface deformation than indicated by theoretical liquefaction analyses. At the CPT locations where potentially liquefiable sand layers up to approximately 1 to 6 feet thick were encountered, overlying layers of non-liquefiable soil were 17 to 25 feet thick. The Youd and Garris publication includes data compilations of past seismic events plotted with the thickness of liquefiable layer versus the thickness of non-liquefiable surface layer. According to the plot for a moment magnitude range of 5.9 to 7.0 and a peak horizontal ground acceleration of 0.26g to 0.35g, a minimum non-liquefiable surface layer thickness of at least 17 feet is considered appropriate to cap up to approximately 10 feet of liquefiable soil, to result in no observable surface effects.

Based on the information presented above, it is our opinion that the non-liquefiable surface soil layer should provide significant capping effect, and the overall ground surface deformation as a result of theoretical liquefaction-induced settlement will likely be less than the estimated theoretical values. In our opinion, based on engineering judgment, the capping effects will likely reduce the theoretical settlements by as much as one-half to two-thirds. This may result in liquefaction-induced total and differential ground settlements of approximately 1 inch and ½ inch, respectively.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, 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, shallow groundwater, and liquefaction. We summarize our conclusions below.

3.1 EXISTING FILL

We encountered existing fill in each of our soil borings that varied in thickness from approximately 2 to 4 feet. Based on the site background, we anticipate this fill is non-engineered. In addition, we

observed an area on the southern border of the site that had several soil stockpiles. There was a large soil stockpile in the northeast of 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 mitigation recommendations in Section 5.1.

3.2 EXPANSIVE SOIL

We observed potentially expansive lean clay and fat clay in the upper 4 to 9 feet of the soil surface in all of our borings, which include the existing fill and native soil below the fill. Our plasticity index tests on this soil ranges from 32 to 47. These laboratory tests correlate with a high to very high shrink/swell potential with variations in moisture content (Coduto, 1999).

Expansive soil changes in volume with changes in moisture and can shrink or swell and cause heaving and cracking of slabs-on-grade, surface improvements, pavements, and structures founded on shallow foundations.

Post-tensioned mat foundations are the preferred foundation system for the residential structures. Design criteria for this foundation type are presented in Section 6.1. Successful performance of structures on expansive soil requires special attention during construction. It is imperative that exposed soil be kept moist prior to placement of concrete for foundation construction. It can be difficult to remoisturize clayey soil without excavation, moisture conditioning, and recompaction.

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. Expansive soil mitigation recommendations are presented in Section 5.2 of this report.

3.3 GROUNDWATER

We encountered groundwater in Boring 1-B1 at a depth of 24 feet, which rose to a depth of 17 feet. However, our review of nearby groundwater data suggests groundwater in the vicinity of the site has been recorded as shallow as approximately 13 feet. Therefore, seasonal fluctuations of groundwater could impede deeper excavations and should be considered in the design of utilities that extend below the groundwater.

Groundwater may require temporary construction dewatering, shoring systems to deal with flowing sands, and buoyancy considerations for designing below-grade improvements.

Surface water can also perch on shallow fine-grained layers, impeding grading activities and transmitting moisture vapor through slabs-on-grade. Moisture transmission through slabs-on-grade can cause excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment. We provide recommendations to reduce the effects of groundwater in the sections addressing Over Optimum Soil Conditions, Underground Utility Backfill, Site Drainage, Temporary Dewatering, Slab Moisture Vapor Reduction, and Cutoff Curbs.

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. Common secondary seismic hazards include ground shaking, liquefaction, 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, or 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 Northern California 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 is clean, loose, saturated, uniformly graded, fine-grained sand. As described in Section 2.9, we evaluated liquefaction potential using the CPT data. Based on our analysis and engineering judgment, we recommend designing the proposed structures to accommodate liquefaction-induced settlements of up to 1 inch for total and ½ inch for differential. This should be added to the static settlement in Section 6.1.

Due to the thickness of non-liquefiable soils that cap the potentially liquefiable layers, we anticipate the risk of sand boils and surface manifestation of liquefaction to be low. Based on the depth of the potentially liquefiable soils relative to the free face of the RD1000 B Drain, we anticipate the risk of lateral spreading is low.

3.4.4 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 Northern California region, but based on the site location, it is our opinion that the offset is expected to be minor.

3.5 FLOODING

We reviewed the Federal Emergency Management Agency (FEMA) Flood Insurance Maps for Sacramento County, California and Incorporated Areas (Map 06067C0063J dated June 16, 2015). The site is mapped as Zone A99, an area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined. 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, as needed.

3.6 SOIL CORROSION POTENTIAL

As part of this study, we obtained two representative soil samples and submitted to an analytical lab for determination of pH, resistivity, sulfate, and chloride. We also performed one sulfate test per ASTM C1580 on a third sample. The results are included in Appendix B and summarized in the table below.

TABLE 3.6-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH	PH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
1-B2	6	7.29	620	4.5	83.7
1-B3	2	---	---	---	< 50
1-B4	2	7.02	960	7.3	39.7

The 2019 CBC references the 2014 American Concrete Institute Manual, ACI 318-14, Section 19.3.1 for concrete durability requirements. ACI Table 19.3.1.1 provides the following exposure categories and classes, and Table 19.3.2.1 provides requirements for concrete in contact with soil based upon the exposure class.

TABLE 3.6-2: ACI Table 19.3.1.1: Exposure Categories and Classes

CATEGORY	SEVERITY	CLASS	CONDITION
F Freezing and thawing	Not Applicable	F0	Concrete not exposed to freezing-and-thawing cycles
	Moderate	F1	Concrete exposed to freezing-and-thawing cycles and occasional exposure to moisture
	Severe	F2	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture
	Very Severe	F3	Concrete exposed to freezing-and-thawing cycles and in continuous contact with moisture and exposed to deicing chemicals
			WATER- SOLUBLE SULFATE IN SOIL % BY WEIGHT* DISSOLVED SULFATE IN WATER MG/KG (PPM)**
S Sulfate	Not applicable	S0	$SO_4 < 0.10$ $SO_4 < 150$
	Moderate	S1	$0.10 \leq SO_4 < 0.20$ $150 \leq SO_4 \leq 1,500$ seawater
	Severe	S2	$0.20 \leq SO_4 \leq 2.00$ $1,500 \leq SO_4 \leq 10,000$
	Very severe	S3	$SO_4 > 2.00$ $SO_4 > 10,000$
P	Not applicable	P0	In contact with water where low permeability is not required.

CATEGORY	SEVERITY	CLASS	CONDITION
Requiring low permeability	Required	P1	In contact with water where low permeability is required.
C Corrosion protection of reinforcement	Not applicable	C0	Concrete dry or protected from moisture
	Moderate	C1	Concrete exposed to moisture but not to external sources of chlorides
	Severe	C2	Concrete exposed to moisture and an external source of chlorides from deicing chemicals, salt, brackish water, seawater, or spray from these sources

* Percent sulfate by mass in soil determined by ASTM C1580

**Concentration of dissolved sulfates in water in ppm determined by ASTM D516 or ASTM D4130

In accordance with the criteria presented in the above table, this soil is categorized as F0 freeze-thaw class, S0 sulfate exposure class, P0 exposure class and C1 corrosion class. Cement type, water-cement ratio, and concrete strength are not specified for these ranges.

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.

Based on the resistivity measurements, the soil is considered extremely corrosive to buried metal piping (Roberge, 2006). Values tested for chloride do not pose a significant impact to metals or concrete.

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

3.7 2019 CBC SEISMIC DESIGN PARAMETERS

The 2019 CBC utilizes design criteria set forth in the 2010 ASCE 7 Standard. Using the CPT shear-wave velocity data obtained in 1-CPT1 and 1-CPT3, we estimated the Vs30 for the site to be 280 meters per second, or 917 feet per second. Based on the subsurface conditions and Vs30 data, we characterized the site as Site Class D in accordance with the 2019 CBC. We provide the 2019 CBC seismic design parameters in Table 3.7-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCER) spectral response acceleration parameters.

TABLE 3.7-1: 2019 CBC Seismic Design Parameters, Latitude: 38.634 Longitude: -121.496

PARAMETER	VALUE
Site Class	D
Mapped MCE _R Spectral Response Acceleration at Short Periods, S _S (g)	0.556
Mapped MCE _R Spectral Response Acceleration at 1-second Period, S ₁ (g)	0.25
Site Coefficient, F _A	1.356
Site Coefficient, F _V	Null*
MCE _R Spectral Response Acceleration at Short Periods, S _{MS} (g)	0.753
MCE _R Spectral Response Acceleration at 1-second Period, S _{M1} (g)	Null*

PARAMETER	VALUE
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	0.502
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	Null*
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.233
Site Coefficient, F_{PGA}	1.367
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.319
Long period transition-period, T_L	12 sec

* A site-specific seismic hazard analysis is required to obtain these values unless the exception discussed in ASCE 7-16 Section 11.4.8 is met. Under this exception, refer to ASCE 7-16 Table 11.4-2 to obtain the value for F_v for site Class D.

Considering the low-rise development, we estimate the fundamental periods of the proposed structures to be less than $1.5T_s$ (where T_s is 0.7 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 $1.5T_s < T \leq T_L$.”

We recommend that we collaborate with the structural engineer of record to further evaluate the effects of taking the exceptions on the structural design and identify the need for performing a site-specific seismic hazard analysis.

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. Figure 2 displays the approximate depths of the existing fill at the site and the location of the stockpiles that we observed. The lateral extent and depth of fill are expected to vary. Consult the soil boring logs in Appendix A for fill depths at specific locations.

5.2 EXPANSIVE SOIL MITIGATION

The presence of expansive soil may result in distress to site improvements, if these soils are not properly mitigated. To mitigate the risk of expansive soil for the buildings we recommend the use of post-tensioned (PT) mat foundations in Section 6.0. We recommend the pool deck be underlain by at least 18 inches of non-expansive import fill. Refer to Section 9.0 for additional recommendations to mitigating the risk of expansive soil for the pool and pool deck.

5.3 GENERAL SITE CLEARING

Areas to be developed should be cleared of surface and subsurface deleterious materials, including existing building foundations, slabs, buried utility and 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 5.6. ENGEO should be retained to observe and test backfilling.

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 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. 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. Wet soil should also be anticipate for deeper utilities.

5.5 ACCEPTABLE FILL

On-site soil material 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.

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.

5.6 FILL COMPACTION

5.6.1 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 the 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; compact the upper 6 inches of fill in pavement areas to at least 90 percent relative compaction prior to aggregate base placement.

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

5.6.2 Underground Utility Backfill

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. In general, if uniformly graded gravel is used for pipe or trench zone backfill it should be fully encapsulated in filter fabric to prevent settlement of surface improvements caused by a migration of fines into the gravel. As an alternative to a uniformly graded gravel, a well-graded gravel import is suitable from a geotechnical perspective.

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 at least 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.

5.6.3 Landscape Fill

Process, place, and compact fill in accordance with Sections 5.6.1 and 5.6.2, except compact to at least 85 percent relative compaction (ASTM D1557).

5.7 SLOPES GRADIENTS

For slopes shorter than 3 feet, construct final slope gradients to 2:1 (horizontal:vertical) or flatter. For slopes taller than 3 feet, such as that which may occur along the levees or northeast site boundary, construct final slope gradients to 3:1 or flatter. The contractor is responsible to construct temporary construction slopes in accordance with CALOSHA requirements.

5.8 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. 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. Do not allow water to pond near foundations, pavements, or exterior flatwork.

Refer to Section 9.0 for supplemental drainage recommendations related to the pool structure and pool deck.

5.9 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 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. Bioretention system internal slopes should follow the slope guidelines described in Section 5.7 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.

5.10 TEMPORARY CONSTRUCTION DEWATERING

Where groundwater is encountered in excavations or trenches, temporary construction dewatering should be performed, which may require using sumps, pumps or other methods. Depending upon environmental permitting requirements, it may or may not be possible to discharge collected groundwater to the City storm drain system. Additional sampling and testing of the groundwater may be required to obtain the appropriate discharge permit.

Dewatering groundwater levels should be maintained at least 2 feet below the bottom of trenches or excavations for pipeline and structure installations. The selection of equipment and method should be determined by the contractor. The dewatering system implemented should be selected so as to have minimal impact on the groundwater level surrounding the proposed excavations. In general, trenches and pits should be maintained a minimum of 50 feet from adjacent existing structures. In addition, the dewatering system should be designed to prevent pumping soil fines with the discharge water. Uncontrolled dewatering may cause settlement of the general area and may affect nearby existing structures.

5.11 LANDSCAPING CONSIDERATION

As the near-surface soil is highly 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

6.1 POST-TENSIONED MAT FOUNDATIONS

We recommend that the proposed buildings be supported on post-tensioned (PT) mat foundations.

We recommend that PT mats have a thickened edge at least 12 inches wide and 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. ENGEO should be retained to review the PT mat foundation design.

The PT mat design criteria are presented in Table 6.1-1 below. The values 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 Design Recommendations

CONDITION	CENTER LIFT	EDGE LIFT
Edge Moisture Variation Distance, e_m (feet)	5.5	3.2
Differential Soil Movement, y_m (inches)	0.6	2.2

PT mats may be designed for an average allowable bearing pressure of up to 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 reduction system as recommended in Section 6.2.

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 on top of the vapor retarder membrane to assist in concrete curing.

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 of the pads to achieve this moisture content.

6.4 TRENCH BACKFILL

Backfill and compact all trenches below building slabs-on-grade and to 5 feet laterally beyond any edge in accordance with Section 5.6.2.

7.0 RETAINING WALLS

We provide the following retaining wall recommendations for perimeter soundwalls and minor landscape walls retaining less than 4 feet of soil. The following recommendations are applicable for on-site soil used as retaining wall backfill.

7.1 LATERAL EARTH PRESSURES

Design proposed retaining walls to resist lateral earth pressures from adjoining natural materials and/or backfill and from any surcharge loads. Provided that adequate drainage is included as recommended below, design unrestrained retaining walls to resist an equivalent fluid pressure of 50 pounds per cubic foot (pcf) plus one-third of any surcharge loads.

Refer to Section 9.0 for lateral earth pressures for the swimming pool design.

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 both restrained and 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 Filter Material (Caltrans Specification 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 $\frac{3}{4}$ -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.

7.3 BACKFILL

Backfill behind retaining walls should be placed and compacted in accordance with Section 5.6. 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 FOUNDATIONS

Provided that the site is graded in accordance with the earthwork recommendations in this report, 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 wall footings be designed using an allowable bearing pressure of 2,500 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 2 feet below lowest adjacent grade. If footings are located within 5 feet from nearby tops of slopes or on sloping ground, the footing embedment should be increased to achieve at least 10 horizontal feet to the nearest free slope face. 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. Footing excavations should not be allowed to dry out prior to concrete placement.

Passive pressures acting on footings may be assumed as 250 pcf equivalent fluid pressure. 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 bioretention basins should neglect the passive resistance of the biotreatment soil media layer. 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

Soundwalls or retaining walls may also 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):	500 psf. This value may be increased by one-third when considering seismic or wind loads. Exclude the upper 1 foot of the pier shaft from pier load capacity computations	
Minimum pier spacing:	Three 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.	

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.

“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.

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.

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). Thicken flatwork edges to at least 8 inches to help control moisture variations in the subgrade and place wire mesh or rebar within the middle third of the slab to help control the width and offset of cracks. Construct control and construction joints in accordance with current American Concrete Institute guidelines.

9.0 SWIMMING POOL STRUCTURE AND DECKING

As a minimum, the following considerations should be included for pool design and adjacent deck.

- The pool walls should be designed by a Structural Engineer. The pool walls should be designed to resist at-rest lateral earth pressure of 75 pcf of the equivalent fluid density, since these walls will not be free to deflect at the top. This value does not include additional loads due to hydrostatic conditions or additional surcharge loads. If the wall height is greater than

6 feet, an additional equivalent fluid pressure for the dynamic increment of 15 pcf should be incorporated.

- The pool/spa structure should be provided with a hydrostatic pressure relief valve.
- We recommend that the pool include the installation of subdrainage facilities under the shells to protect adjacent improvements against any potential leakage. The subdrainage system should consist of a minimum 6-inch-thick layer of $\frac{3}{4}$ -inch clean crushed rock placed directly below the pool shell and enveloped on the bottom and sides with filter fabric. The subgrade should slope towards the center of the pool shell at 2 percent towards a 4-inch-diameter SDR35 subdrain. The subdrain should outlet to an appropriate location where leakage, if any, can be readily observed. This pipe should be completely encapsulated in crushed rock material. The perforated pipe should change to solid sealed pipe when it leaves the pool vicinity to its approved outlet, such as an area drain or standpipe. If unusual amounts of water are observed flowing from the outlet, the pool should be checked for leakage and repaired.
- The upper 18 inches of the pool deck subgrade (below baserock) should consist of low to non-expansive import fill that meet the acceptable fill recommendations in Section 5.6 or Class 2 Aggregate Base. The pool deck slabs-on-grade should be reinforced and designed by the structural engineer. It is our opinion that the pool deck concrete should be at least 5 inches thick and reinforced with No. 4 bars spaced at no greater than 12 inches on center each way. Control and expansion joints should be incorporated in accordance with guidelines by the American Concrete Institute. Exterior pool decks and flatwork should be constructed as units that are structurally independent of the pool shell. This allows the concrete to move without distress to the pool. The subgrade should be graded to slope away from the pool walls at a minimum slope of 2 percent.
- The pool excavations should be observed by ENGEO prior to concrete placement. The moisture of the soil exposed in the pool excavation should be maintained until concrete is placed by liberally sprinkling or other methods.
- All concrete expansion joints, including areas where pool coping abut the pool structure, should be filled with elastomeric sealant, which should be maintained regularly to close any openings that may develop.
- Where pool decking lies adjacent to landscape areas, a subdrain should be installed to collect excess irrigation water from the landscape areas. The subdrain should extend to a depth of at least 18 inches below the base of the deck and consist of a 4-inch-diameter perforated pipe (SDR 35 or approved equivalent) within a 12-inch-wide trench surrounded by Class 2 Permeable Material. A moisture barrier (i.e. 10-mil visqueen or approved equivalent) should be placed along the pool deck side of the subdrain trench. The upper 8 inches of the trench should be capped with clayey on-site native materials.
- All utility trenches entering the pool deck area must be provided with an impervious seal consisting of native materials or concrete slurry where the trenches pass under the perimeter of the pool deck. The impervious plug should extend at least 3 feet to either side of the crossing. This is to prevent surface water percolation into the sand where such water would remain trapped in a perched condition, allowing clay to develop to their full expansion potential.
- The improvement areas must be positively graded at all times to provide for rapid removal of surface water runoff from the pool. The finished concrete deck should have a minimum slope of at least to 2 percent away from the pool edge in all directions. All surface water should be

collected and discharged into the storm drain system. Sufficient area drains should be constructed around the swimming pool to remove excessive surface water.

10.0 PAVEMENT DESIGN

10.1 FLEXIBLE PAVEMENTS

We performed an R-value test on a bulk sample of the near-surface fill from Boring 1-B3, which resulted in an R-value of 8. Based on the presence of near-surface clay across the site and higher plasticity native clay below the fill, we judged an R-value of 5 to be appropriate for design. Using estimated traffic indexes (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.

TABLE 10.1-1: Recommended Asphalt Concrete Pavement Sections

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
5	3	10
6	3½	13
7	4	16

If the pavement sections are subject to the City of Sacramento minimum design guidelines, we provide the following summary of Table 15-6.2 of the City of Sacramento Street Design Standards (City of Sacramento, 2009).

TABLE 10.1-2: City of Sacramento Table 15-6.2 Structural Pavement Section Minimums

TRAFFIC INDEX	SECTION	
	ASPHALT CONCRETE (INCHES)	CLASS 2 AGGREGATE BASE (INCHES)
4.5	4	8
5	4	9
6	4	13
7	4	17

The civil engineer should determine the appropriate traffic indexes based on the estimated traffic loads and frequencies and if the City minimum sections apply. 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 Sacramento, and the fill compaction specifications in Section 5.6.

10.2 RIGID PAVEMENTS

Use concrete pavement sections to resist heavy loads and turning forces in areas such as fire lanes or 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.
- 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.

10.3 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with Section 5.6.1. 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.

10.4 CUTOFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered 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 barriers.

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

11.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the Anton Fong Ranch 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.

SELECTED REFERENCES

- Brusca Associates, Inc. (2022). Phase I Environmental Site Assessment, Fong Ranch Road Property, March 23, 2022.
- Bryant, W. and Hart, E. (2007). Special Publication 42, "Fault-Rupture Hazard Zones in California", Interim Revision 2007, California Department of Conservation.
- California Building Code (2019).
- California Geologic Survey (CGS) (2008). Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.
- California Geologic Survey (CGS). (2018). Special Publication 42, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California.
- City of Sacramento (2009). Design and Procedures Manual, Section 15 – Street Design Standards, City of Sacramento, July 2009.
- Coduto, Donald P. (1999). Geotechnical Engineering, Principles and Practices; ISBN 0-13-576380-0.
- Division of Mines and Geology. (1997). Special Publication 117, Guidelines for Evaluation and Mitigating Seismic Hazards in California, Adopted March 13.
- Field et al. (2015). Long-Term Time-Dependent Probabilities for the Third Uniform California Earthquake Rupture Forecast (UCERF3). Bulletin of the Seismological Society of America, Vol 105, No. 2A, pp. 511-543.
- Gutierrez, C.I.. (2011). Preliminary Geologic Map of the Sacramento Quadrangle, California, 1:100,000, California Geological Survey.
- Helley, E.J., and Harwood, D.S. (1985). Geologic map of the Late Cenozoic deposits of the Sacramento Valley and northern Sierran Foothills, California, U.S. Geologic Survey.
- Kleinfelder. (1996). Geotechnical Investigation Report, Natomas Levee Certification, Sacramento, California; October 29, 1996.
- Morrow Surveying, Inc. (2022). Alta/NSPS Land Title Survey, 3625 Fong Ranch Road, Sacramento, California; Project No. 0213-021; March 16, 2022.
- Post-Tensioning Institute (2012). DC10.5-12 Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soil, December 2012.
- Roberge, P.R. (2006). Corrosion Basics an Introduction, Second Edition, NACE International.
- United States Geologic Survey (USGS). (1911). Topographic Map of Arcade, California (1:31,680), May 1911.

SELECTED REFERENCES (Continued)

United States Geologic Survey (USGS). (1915), Topographic Map of Elkhorn Weir, California (1:31,680), June 1911.

Withee Malcolm (2021). Site Study #3 for Fong Ranch Road, Sacramento, CA, October 4, 2021.

Wills (2015). CGS Map Sheet 48: Shear-wave velocity in Upper 30m of Surficial Geology (Vs30), <https://maps-cnra-cadoc.opendata.arcgis.com/datasets/cadoc::cgs-map-sheet-48-shear-wave-velocity-in-upper-30m-of-surficial-geology-vs30/about>



FIGURES

FIGURE 1: Vicinity Map

FIGURE 2: Site Plan

FIGURE 3: Regional Geologic Map

FIGURE 4: Regional Faulting and Seismicity Map

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0 2000
FEET

BASE MAP SOURCE: BING MAPPING SERVICE



VICINITY MAP
ANTON FONG RANCH
SACRAMENTO, CALIFORNIA

PROJECT NO.: 20044.000.001

SCALE: AS SHOWN

DRAWN BY: SPPE

CHECKED BY: NSB

FIGURE NO.

1

ORIGINAL FIGURE PRINTED IN COLOR

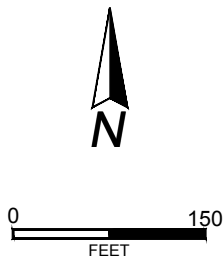
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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- 1-B5**  2' BORING (ENGEO, 2022), EXISTING FILL THICKNESS SHOWN IN FEET
- 1-CPT5**  CONE PENETRATION TEST (ENGEO, 2022)
- B-26**  BORING (KLEINFELDER, 1996)



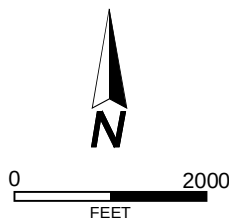
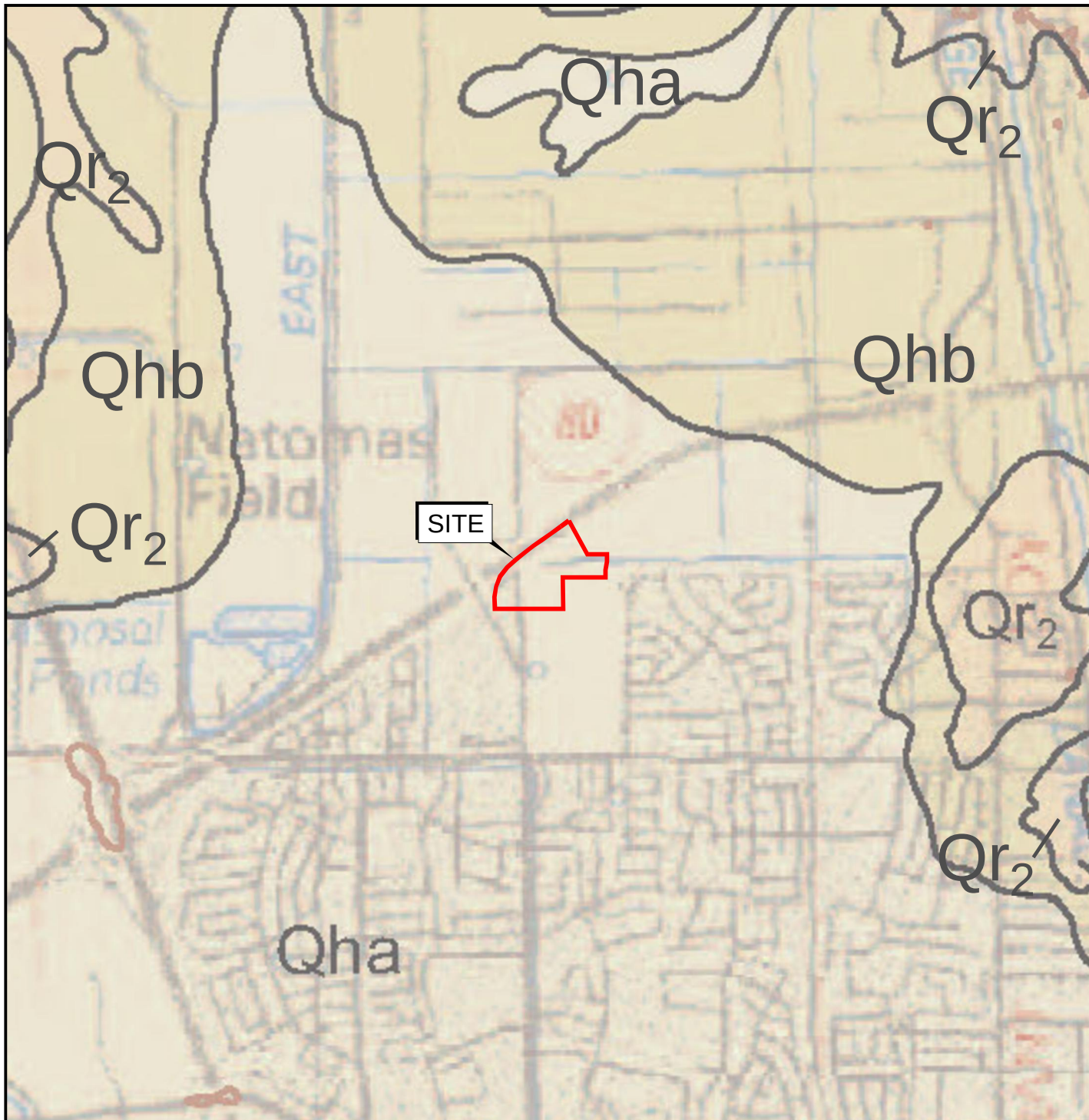
BASE MAP SOURCE: BING MAPPING SERVICE AND WITHEE MALCOM, 2021



SITE PLAN
ANTON FONG RANCH
SACRAMENTO, CALIFORNIA

PROJECT NO.: 20044.000.001
SCALE: AS SHOWN
DRAWN BY: SPPE CHECKED BY: NSB

FIGURE NO.
2



EXPLANATION

- Qha HOLOCENE ALLUVIUM
- Qhb HOLOCENE BASIN DEPOSITS
- Qr₂ RIVERBANK FORMATION

BASE MAP SOURCE: GUTIERREZ, 2011



REGIONAL GEOLOGIC MAP
ANTON FONG RANCH
SACRAMENTO, CALIFORNIA

PROJECT NO.: 20044.000.001

SCALE: AS SHOWN

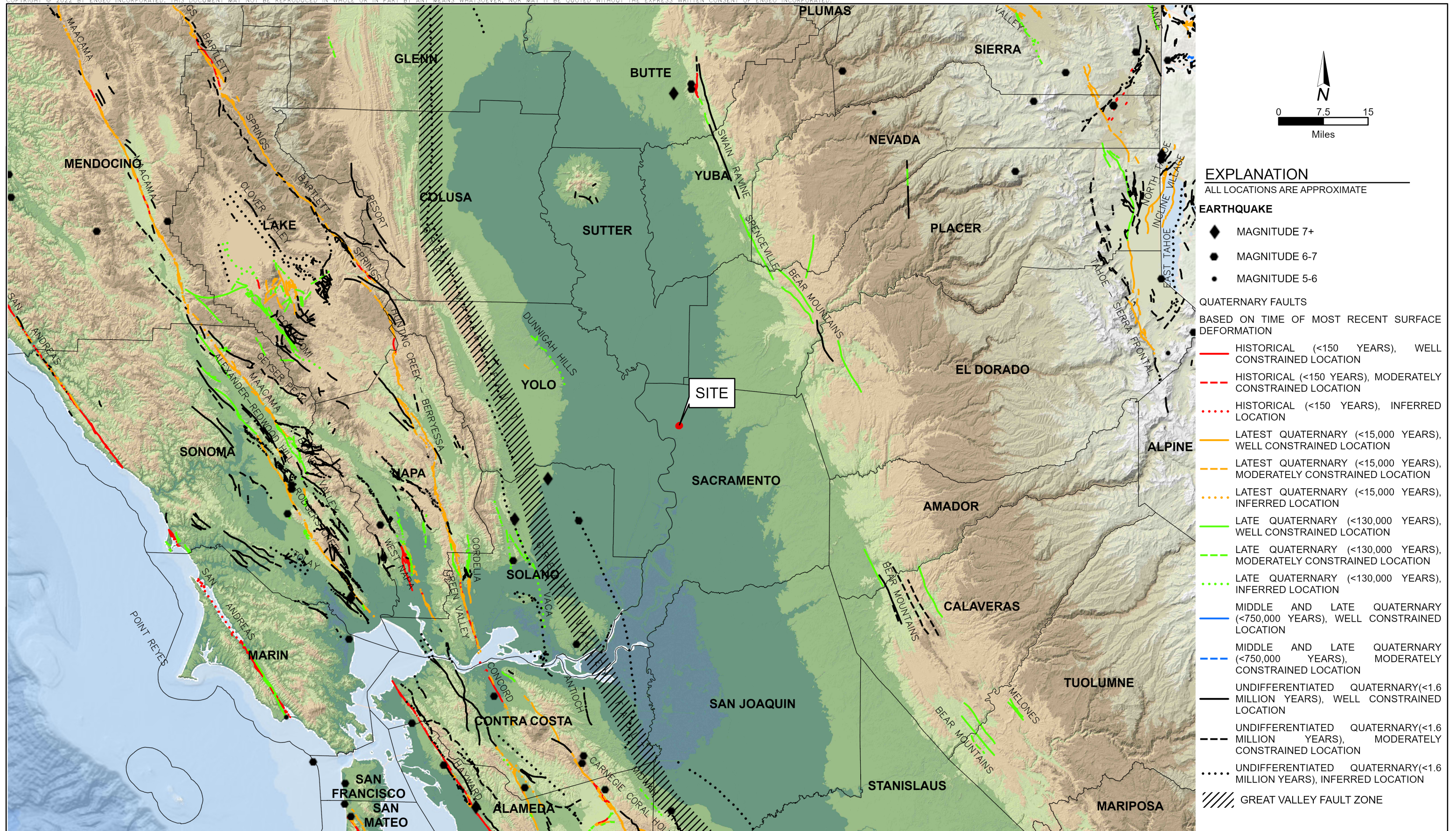
DRAWN BY: SPPE

CHECKED BY: NSB

FIGURE NO.

3

ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE	ESRI, GEBCO, DELORME, NATURALVUE COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION U.S.G.S. QUATERNARY FAULT DATABASE, 2018 U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800-PRESENT)
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REGIONAL FAULTING AND SEISMICITY

ANTON FONG RANCH
SACRAMENTO, CALIFORNIA

PROJECT NO. : 20044.000.001

SCALE: AS SHOWN

DRAWN BY: SPPE	CHECKED BY: NSB
----------------	-----------------

FIGURE NO.

4 |



APPENDIX A

**BORING LOG KEY
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				CLEAR SQUARE SIEVE OPENINGS			
200	40	10	4	3/4 "	3"	12"	
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS

VERY LOOSE
LOOSE
MEDIUM DENSE
DENSE
VERY DENSE

BLOWS/FOOT (S.P.T.)

0-4
4-10
10-30
30-50
OVER 50

CONSISTENCY

SILTS AND CLAYS

VERY SOFT
SOFT
MEDIUM STIFF
STIFF
VERY STIFF
HARD

STRENGTH*

0-1/4
1/4-1/2
1/2-1
1-2
2-4
OVER 4

MOISTURE CONDITION

DRY
MOIST
WET

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

LINE TYPES

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

GROUNDWATER SYMBOLS



Groundwater level during drilling



Stabilized groundwater level

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

(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

ENGEO
Expect Excellence

LOG OF BORING 1-B1

LATITUDE: 38.6345854

LONGITUDE: -121.4951736

Geotechnical Exploration
Anton Fong Ranch
Sacramento, CA
20044.000.001

DATE DRILLED: 3/11/2022
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAD88): Approx. 13 ft.

LOGGED / REVIEWED BY: A. Hauger / NB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			FAT CLAY (CH), dark brown, hard, moist, high plasticity, trace fine- to coarse-grained sand, trace dried grass [Fill]				60	26	34	98	32.1	89	4.5+*
	10		FAT CLAY (CH), dark brown, very stiff, moist, high plasticity, approximately 10% fine-grained sand, iron oxidation [Native] grades to gray			18							3.0*
	5					10							
						19							2.75*
	5		SANDY SILT (ML), olive brown, medium dense, moist, low plasticity, fine- to medium-grained sand, 46% fine- to medium-grained sand, iron oxidation										
	10					16				54			
	0		grades to dense										
	15												
			SANDY SILT (ML), yellowish brown, hard, moist, low plasticity, approximately 30% fine- to medium-grained sand, iron oxidation			42					27.7	95.6	4.5+*
	-5												
	20		SILTY SAND (SM), olive brown, medium dense, moist, fine- to medium-grained sand, 25% fines, mica, iron oxidation			32				25	20.2		
	-10												
	25		POORLY GRADED SAND WITH SILT (SP-SM), olive brown, loose, very moist, fine- to coarse-grained sand, 10% fines, mica										

LOG OF BORING 1-B1

LATITUDE: 38.6345854

LONGITUDE: -121.4951736

Geotechnical Exploration
Anton Fong Ranch
Sacramento, CA
20044.000.001

DATE DRILLED: 3/11/2022
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAD88): Approx. 13 ft.

LOGGED / REVIEWED BY: A. Hauger / NB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			POORLY GRADED SAND WITH SILT (SP-SM), olive brown, loose, very moist, fine- to coarse-grained sand, 10% fines, mica			4				10			
			[Switch to mud rotary]										
-15			POORLY GRADED SAND WITH GRAVEL (SP), olive brown, very loose, very moist, fine- to coarse-grained sand, 24% fine rounded gravel, 4% fines										
30						3							
			grades to medium dense			15				4	15.7		
-20			SANDY SILT (ML), olive brown, hard, low plasticity, 32% fine- to medium-grained sand										
35						33				67	26.1		
-25													
40			grades to decreasing fines, mica			28							
-30			SILT (ML), olive brown, hard, moist, low plasticity, approximately 5% fine-grained sand, mica, iron oxidation			50/6							
45													
			SILTY SAND (SM), olive brown, dense, moist, approximately 30% fines, mica, iron oxidation			49							
-35													
			SILT (ML), olive brown, hard, moist, low plasticity, approximately 5% fine-grained sand, mica, iron oxidation										
50													4.5+*



LOG OF BORING 1-B1

LATITUDE: 38.6345854

LONGITUDE: -121.4951736

Geotechnical Exploration
Anton Fong Ranch
Sacramento, CA
20044.000.001

DATE DRILLED: 3/11/2022
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAD88): Approx. 13 ft.

LOGGED / REVIEWED BY: A. Hauger / NB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: SFA, Switch to Mud
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
			SILT (ML), olive brown, hard, moist, low plasticity, approximately 5% fine-grained sand, mica, iron oxidation			64							4.5+*
			Bottom of boring at 51½ feet. Groundwater first encountered at a depth of 24 feet and rose to a depth of 17 feet after 5 minutes.										

LOG OF BORING 1-B2

LATITUDE: 38.6342873

LONGITUDE: -121.4937594

Geotechnical Exploration
Anton Fong Ranch
Sacramento, CA
20044.000.001

DATE DRILLED: 3/11/2022
HOLE DEPTH: Approx. 13 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAD88): Approx. 12 ft.

LOGGED / REVIEWED BY: A. Hauger / NB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
10			LEAN CLAY (CL), dark brown mottled with dark gray, very stiff, moist, approximately 10% fine-grained sand, trace rounded gravel [Fill]			19					38.6		3.5*
5			FAT CLAY (CH), dark gray, very stiff, moist, high plasticity, approximately 10% fine-grained sand [Native]			7							3.5*
5						19							3.5*
10			POORLY GRADED SAND WITH CLAY (SP-SC), yellowish brown, medium dense to medium dense, moist, fine- to coarse-grained sand			15					7.6		
0			LEAN CLAY (CL), dark brown, very stiff, moist, medium plasticity, approximately 15% fine-grained sand			9							3.0*
			Bottom of boring at 13 feet. No groundwater encountered.										

LOG OF BORING 1-B3

LATITUDE: 38.6331933

LONGITUDE: -121.4959831

Geotechnical Exploration
Anton Fong Ranch
Sacramento, CA
20044.000.001

DATE DRILLED: 3/11/2022
HOLE DEPTH: Approx. 21½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAD88): Approx. 11 ft.

LOGGED / REVIEWED BY: A. Hauger / NB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
10			LEAN CLAY (CL), yellowish brown, hard, slightly moist, medium plasticity, approximately 10% fine-grained sand, trace dried grass [Fill]										
			[Remolded Direct Shear, Phi=21.2 deg, Cohesion = 83 psf]			15					44	74	4.5+*
			FAT CLAY (CH), dark gray, very stiff, moist, high plasticity, trace fine-grained sand, iron oxidation			11					33.4		3.0*
5			grades to dark gray mottled with black, iron oxide			18							
5			grades to olive brown, manganese oxidation, trace fines rounded gravel			16							4.0*
10			SANDY FAT CLAY (CH), olive brown, hard, moist, high plasticity, approximately 30% fine- to coarse-grained sand, trace fine rounded gravel			46							4.5*
0			CLAYEY SAND (SC), olive brown, medium dense, fine-grained sand, 19% fines, iron and manganese oxidation										
15			grades to fine- to coarse-grained sand, trace fine rounded gravel			7				19	31.6		
-5			FAT CLAY WITH SAND (CH), dark brown, very stiff, moist, high plasticity, approximately 20% fine-grained sand										2.5*
20			SANDY FAT CLAY (CH), dark gray, medium dense, moist, high plasticity, fine- to medium-grained sand, 63% fines			10				63	25.3		
-10			Bottom of boring at 21½ feet. No groundwater encountered.										

LOG OF BORING 1-B4

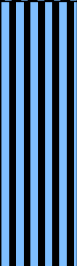
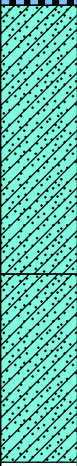
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LONGITUDE: 121.4978226

Geotechnical Exploration
Anton Fong Ranch
Sacramento, CA
20044.000.001

DATE DRILLED: 3/11/2022
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAD88): Approx. 12 ft.

LOGGED / REVIEWED BY: A. Hauger / NB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
10			FAT CLAY (CH), yellowish brown, very stiff, moist, medium plasticity, trace fine gravel [Fill] grades to dark gray			15	60	28	32	98	31.6		3.0*
5			ELASTIC SILT (MH), dark gray, very stiff, moist, high plasticity, trace fine-grained sand grades to iron oxide			8	76	37	39				3.5*
5						16					37.2	78.8	1.4
10			CLAYEY SAND (SC), yellowish brown, medium dense, moist, fine- to coarse-grained sand, approximately 30% fines. iron oxidation			16							3.5*
0													
15			CLAYEY SAND (SC), olive brown, medium dense, moist, coarse-grained sand, 26% fines, mica			10				26			
			Bottom of boring at 16½ feet. No groundwater encountered.										

LOG OF BORING 1-B5

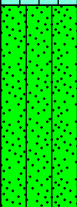
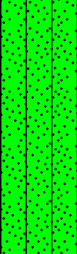
LATITUDE: 38.635673

LONGITUDE: -121.4953299

Geotechnical Exploration
Anton Fong Ranch
Sacramento, CA
20044.000.001

DATE DRILLED: 3/11/2022
HOLE DEPTH: Approx. 16½ ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (NAD88): Approx. 12 ft.

LOGGED / REVIEWED BY: A. Hauger / NB
DRILLING CONTRACTOR: Geo-Ex Subsurface
DRILLING METHOD: Solid Flight Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits			Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
							Liquid Limit	Plastic Limit	Plasticity Index				
		X	FAT CLAY (CH), dark brown to dark gray, hard, slightly moist, high plasticity, trace fine-grained sand, trace dry grass [Fill]			14	75	28	47	99	32.8		4.5+*
10			FAT CLAY (CH), dark gray, very stiff, moist, high plasticity, approximately 15% fine-grained sand, iron oxidation [Native]			9					32.1		4.0*
5			SANDY SILT (ML), dark brown, hard, moist, low plasticity, approximately 30% fine-grained sand, manganese oxide			29					28.4	94	2.1 4.5+*
5			SILTY SAND (SM), olive brown, medium dense, moist, fine- to medium-grained sand, approximately 30% fines, iron and manganese oxidation			22							
10													
0													
15			grades to fine- to coarse-grained sand, 38% fines, mica			18				38			
			Bottom of boring at 16½ feet. No groundwater encountered.										



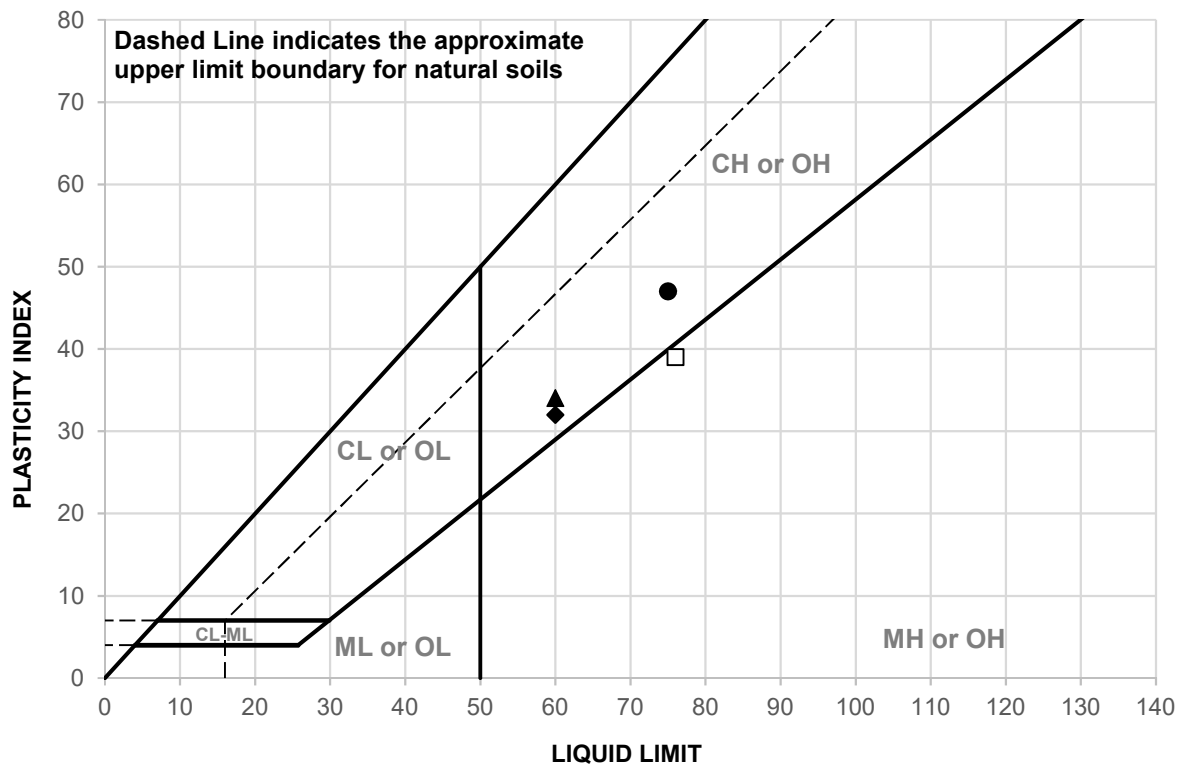
APPENDIX B

LABORATORY TEST DATA

**Liquid and Plastic Limits Test Report
Unconfined Compression Test
Particle Size Distribution Report
R-Value Test Report
Analytical Results of Soil Corrosion**

LIQUID AND PLASTIC LIMITS TEST REPORT

ASTM D4318



	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B1@2	2	See exploration logs	60	26	34
◆	1-B4@1.5	1.5	See exploration logs	60	28	32
□	1-B4@4	4	See exploration logs	76	37	39
●	1-B5@1.5	1.5	See exploration logs	75	28	47

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B1@2	PI: ASTM D4318, Wet Method	
◆	1-B4@1.5	PI: ASTM D4318, Wet Method	
□	1-B4@4	PI: ASTM D4318, Wet Method	
●	1-B5@1.5	PI: ASTM D4318, Wet Method	



CLIENT: Anton Fong Ranch, LLC

PROJECT NAME: Anton Fong Ranch

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

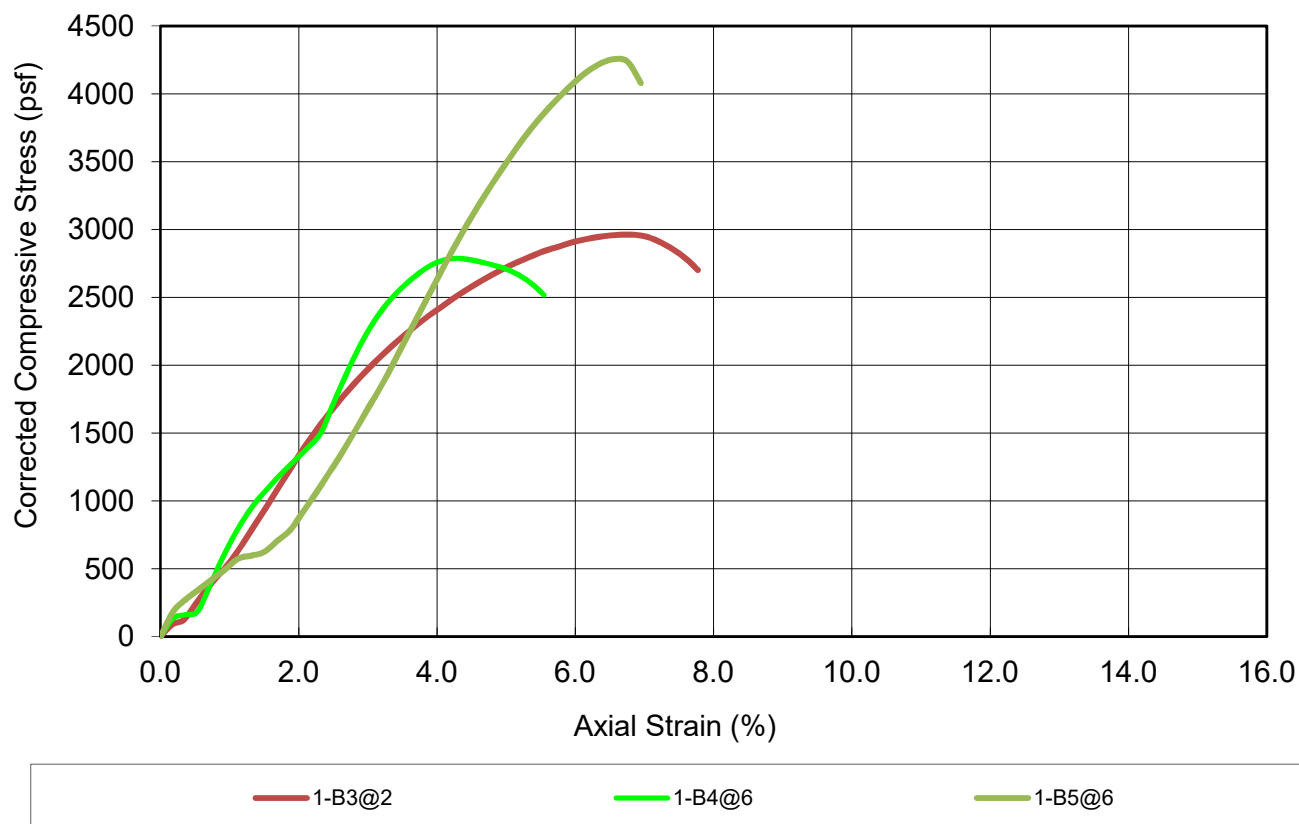
REPORT DATE: 3/29/2022

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard

UNCONFINED COMPRESSION TEST REPORT (ASTM D2166)

Compressive Stress vs. Axial Strain



BEFORE TEST			
	SPECIMEN 1-B3@2	SPECIMEN 1-B4@6	SPECIMEN 1-B5@6
Test Moisture Content (%)	43.97	34.65	28.37
Dry Density (pcf)	74.0	85.2	94.0
Saturation (%)	92.3	94.9	95.6
Void Ratio	1.30	0.99	0.81
Diameter (in)	2.370	2.383	2.374
Height (in)	5.920	5.590	5.327
Height-To-Diameter Ratio	2.50	2.35	2.24
TEST DATA			
Unconfined Compressive Strength (psf)	2963	2789	4257
Undrained Shear Strength (psf)	1481.6	1394.4	2128.4
Strain Rate (in/min)	0.050	0.050	0.050
Specific Gravity (ASSUMED)	2.720	2.720	2.720
Strain at Failure(%)	6.76	4.29	6.57
Test Remarks			
SPECIMEN	DESCRIPTION		
1-B3@2	See exploration logs		
1-B4@6	See exploration logs.		
1-B5@6	See exploration logs.		

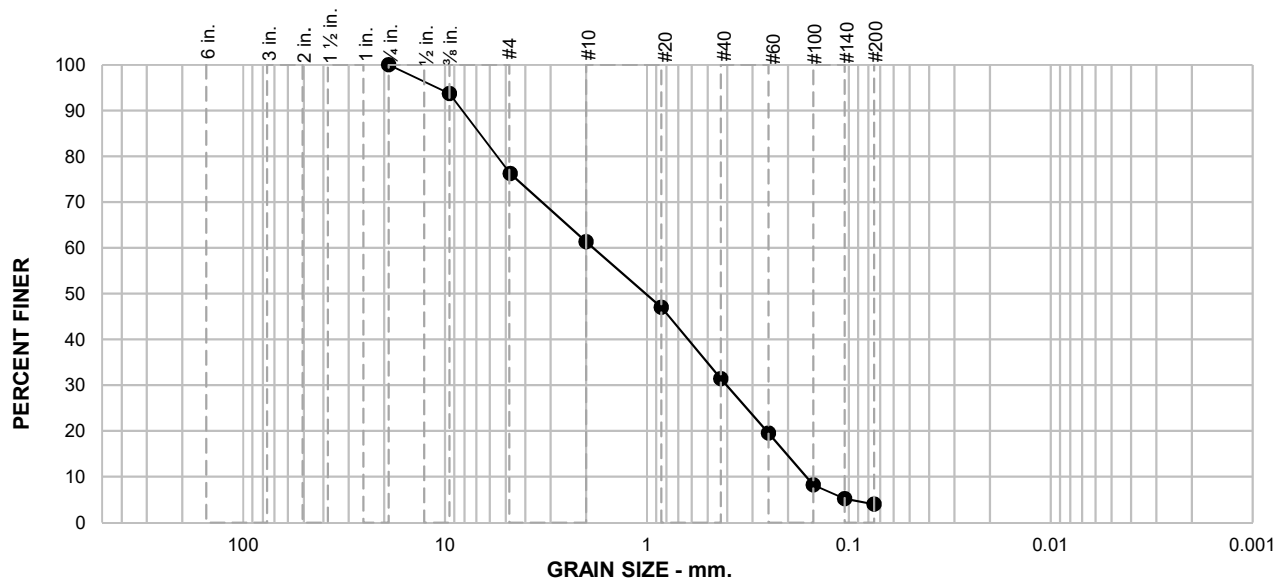


PROJECT NAME: Anton Fong Ranch
PROJECT NO: 20044.000.001 PH001
CLIENT: Anton Fong Ranch, LLC
LOCATION: Sacramento, CA

Report Date: 3/29/22
Tested By: R. Montalvo
Reviewed By: N. Broussard

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method A



SAMPLE ID: 1-B1@31.5

DEPTH (ft): 31.5

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
			24	15	30	27	4	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION				
				See exploration logs				
				ATTERBERG LIMITS				
				PL =		LL =		PI =
				COEFFICIENTS				
				D ₉₀ = 8.1605 mm		D ₈₅ = 6.7263 mm		D ₆₀ = 1.8814 mm
				D ₅₀ = 1.0211 mm		D ₃₀ = 0.4093 mm		D ₁₅ = 0.2021 mm
				D ₁₀ = 0.1633 mm		C _u = 11.52		C _c = 0.55
				CLASSIFICATION				
				USCS = SP				
				REMARKS				

* (no specification provided)

CLIENT: Anton Fong Ranch, LLC



PROJECT NAME: Anton Fong Ranch

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

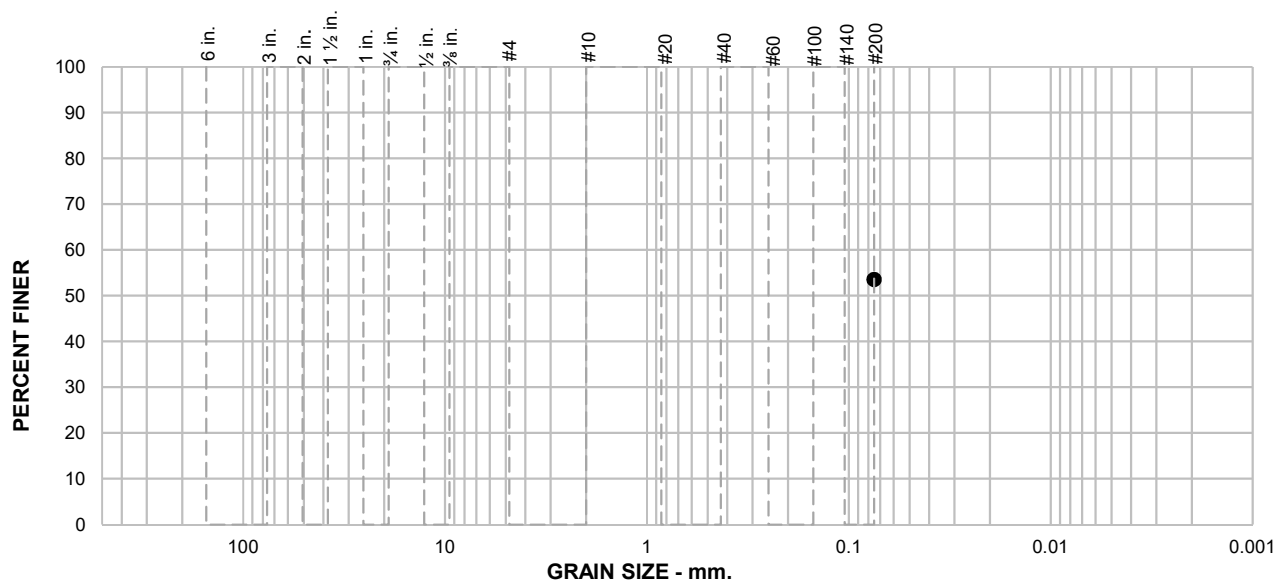
REPORT DATE: 3/29/2022

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B1@10

DEPTH (ft): 10

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
							53.5	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION				
#200	53.5			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 = 572.6 g Largest particle size < No. 4 Sieve				

* (no specification provided)

CLIENT: Anton Fong Ranch, LLC



PROJECT NAME: Anton Fong Ranch

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

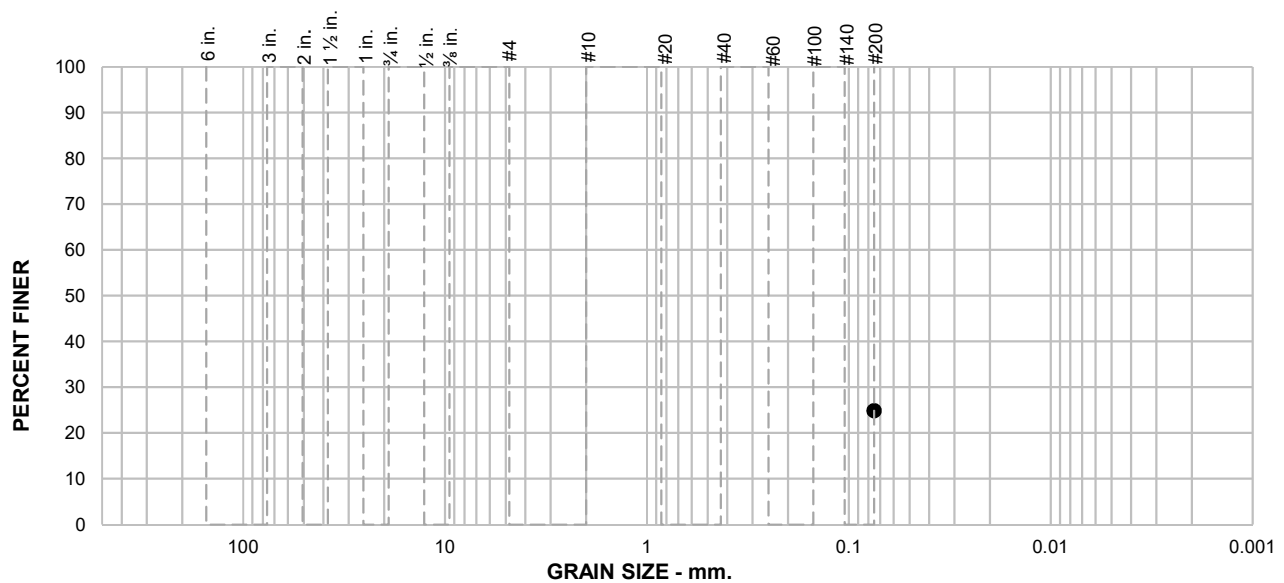
REPORT DATE: 3/29/2022

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method A



SAMPLE ID: 1-B1@21

DEPTH (ft): 21

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
							24.8	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION				
#200	24.8			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 = 757.2 g Largest particle size < No. 4 Sieve				

* (no specification provided)

CLIENT: Anton Fong Ranch, LLC



PROJECT NAME: Anton Fong Ranch

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

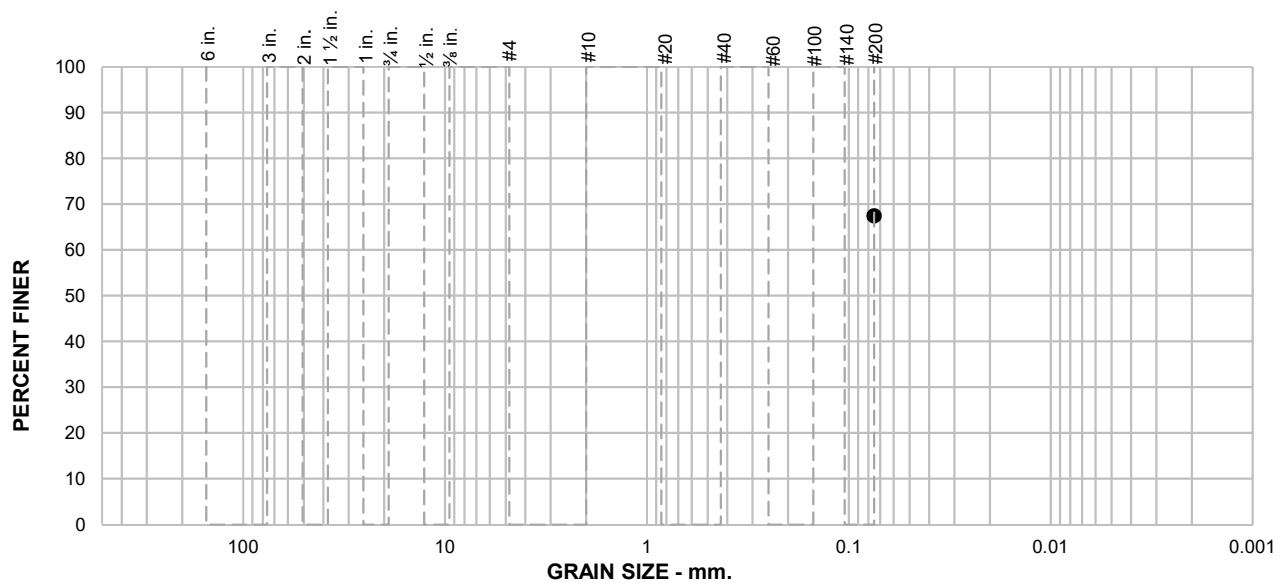
REPORT DATE: 3/29/2022

TESTED BY: R. Montalvo

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B1@35

DEPTH (ft): 35

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
							67.4	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION				
#200	67.4			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 = 412.6 g Largest particle size < No. 4 Sieve				

* (no specification provided)

CLIENT: Anton Venture Oaks Owners, LLC



PROJECT NAME: Anton Venture Oaks

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

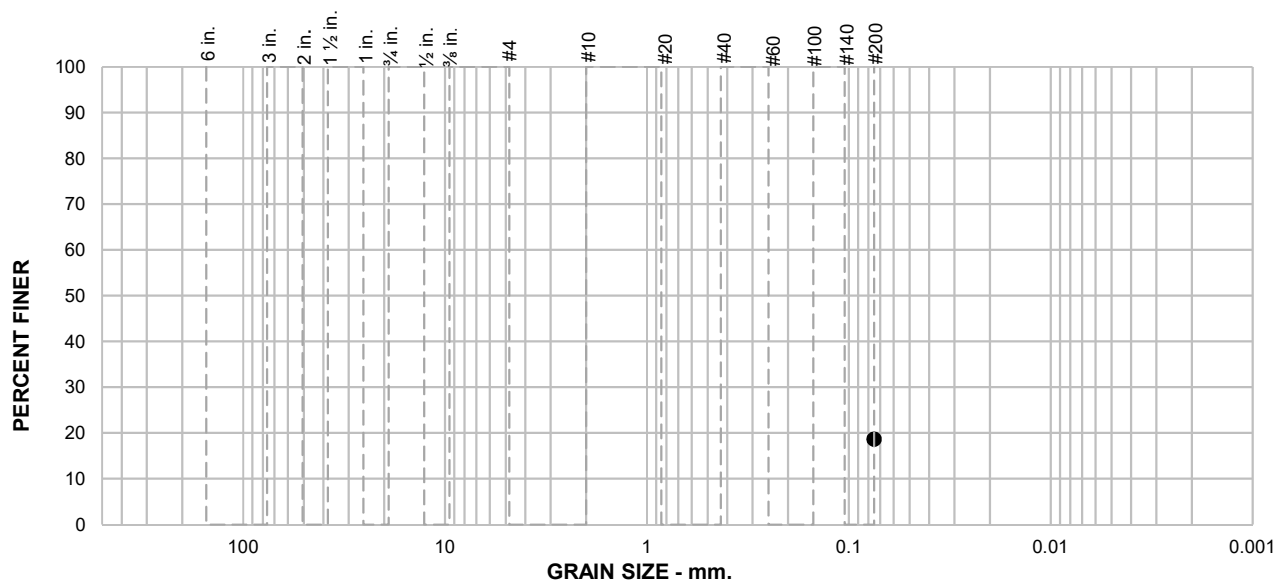
REPORT DATE: 3/28/2022

TESTED BY: R. Montalvo

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B3@15 (SAND)

DEPTH (ft): 15

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
							19	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION				
#200	19			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 = 229.9 g Largest particle size ≥ No. 4 Sieve				

* (no specification provided)

CLIENT: Anton Venture Oaks Owners, LLC



PROJECT NAME: Anton Venture Oaks

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

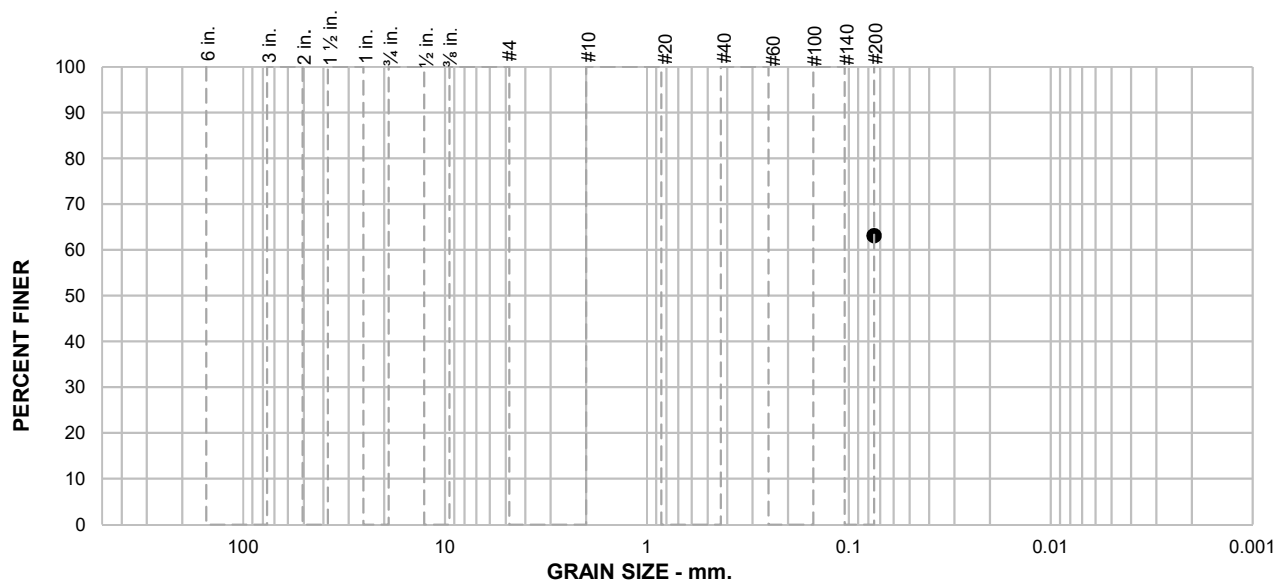
REPORT DATE: 3/25/2022

TESTED BY: R. Montalvo

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B3@20

DEPTH (ft): 20

% +75mm	% GRAVEL		% SAND			% FINES		
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY	
						63.1		
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION				
#200	63.1			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 = 189.8 g Largest particle size < No. 4 Sieve				

* (no specification provided)

CLIENT: Anton Venture Oaks Owners, LLC



PROJECT NAME: Anton Venture Oaks

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

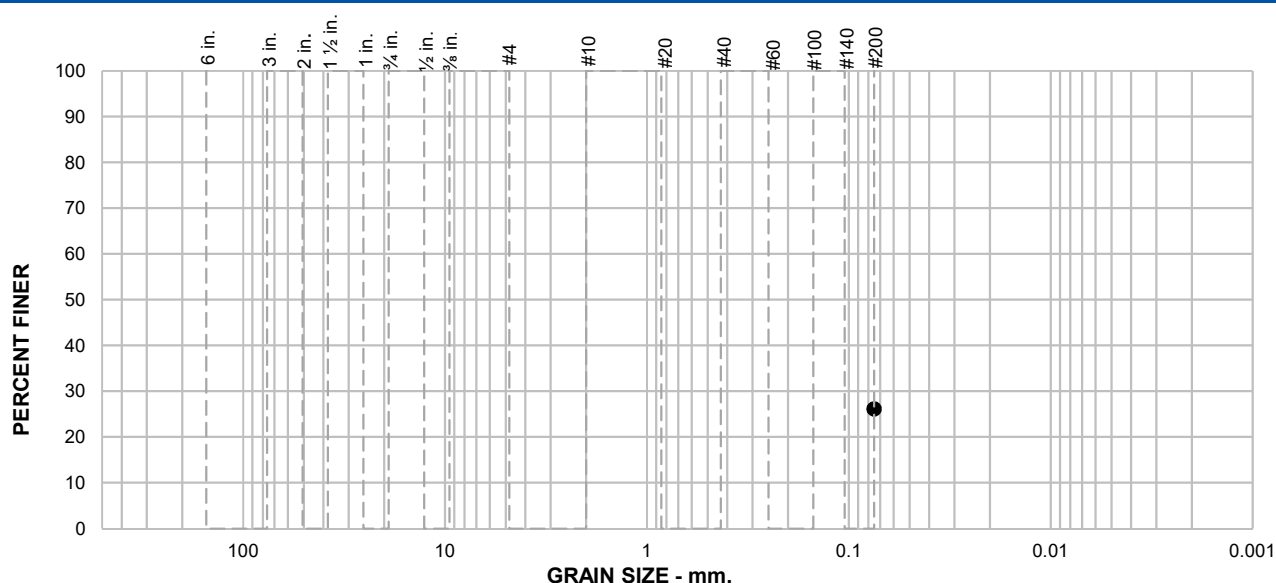
REPORT DATE: 3/28/2022

TESTED BY: R. Montalvo

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B4@15

DEPTH (ft): 15

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
						26.1	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION			
#200	26.1			See exploration logs			

* (no specification provided)

CLIENT: Anton Venture Oaks Owners, LLC



PROJECT NAME: Anton Venture Oaks

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

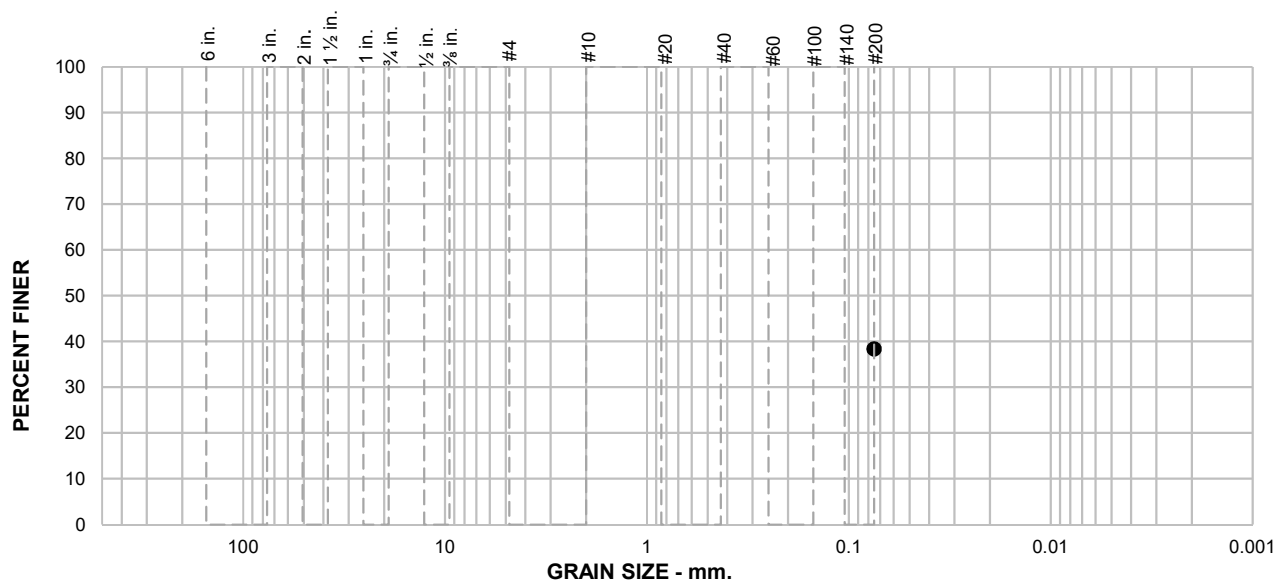
REPORT DATE: 3/28/2022

TESTED BY: R. Montalvo

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D1140, Method B



SAMPLE ID: 1-B5@16

DEPTH (ft): 16

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
							38.3	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION				
#200	38.3			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 = 326 g Largest particle size < No. 4 Sieve				

* (no specification provided)

CLIENT: Anton Venture Oaks Owners, LLC



PROJECT NAME: Anton Venture Oaks

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

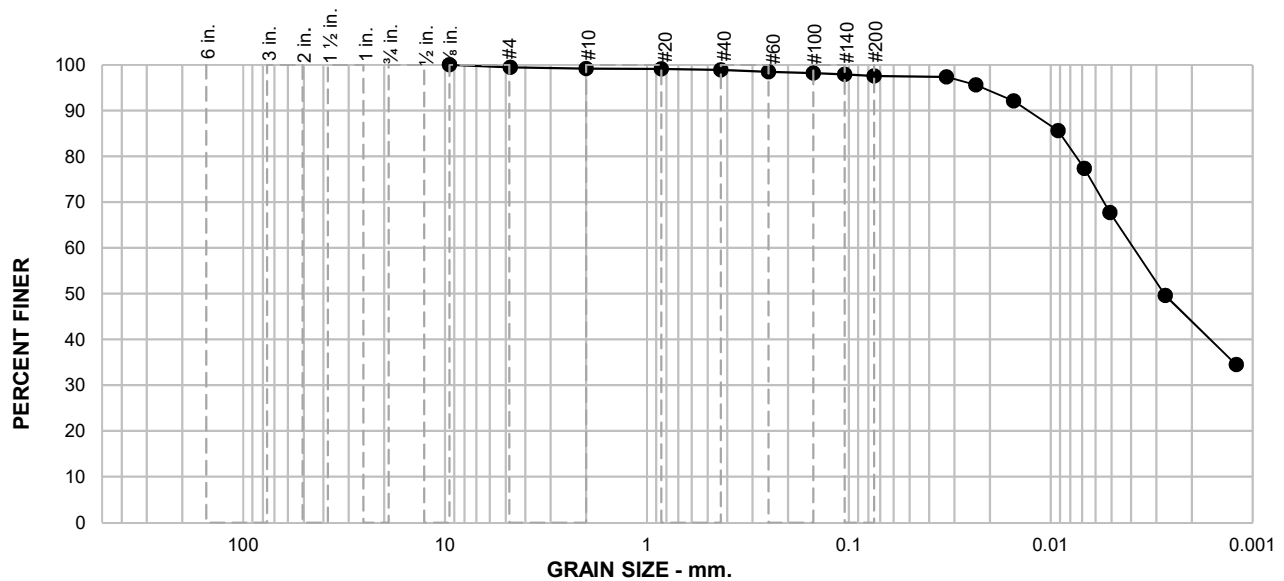
REPORT DATE: 3/28/2022

TESTED BY: R. Montalvo

REVIEWED BY: M. Gilbert

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D422



SAMPLE ID: 1-B4@1.5

DEPTH (ft): 1.5

% +75mm	% GRAVEL		% SAND			% FINES	
	COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
		1	0	0	1	53.7	43.9
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION See exploration logs			
3/4 in.	100			ATTERBERG LIMITS PL = 28 LL = 60 PI = 32			
#4	99						
#10	99			COEFFICIENTS D ₉₀ = 0.0129 mm D ₈₅ = 0.0090 mm D ₆₀ = 0.0039 mm D ₅₀ = 0.0027 mm D ₃₀ = D ₁₅ = D ₁₀ = C _u = C _c =			
#20	99						
#40	99			CLASSIFICATION USCS = CH			
#60	98						
#100	98			REMARKS Silt/clay division of 0.002mm used PI: ASTM D4318, Wet Method USCS: ASTM D2487			
#140	98						
#200	98						
0.0329 mm.	97.3						
0.0235 mm.	95.6						
0.0152 mm.	92.1						
0.0092 mm.	85.6						
0.0068 mm.	77.3						
0.0051 mm.	67.7						
0.0027 mm.	49.6						
0.0012 mm.	34.5						

* (no specification provided)

CLIENT: Anton Fong Ranch, LLC



PROJECT NAME: Anton Fong Ranch

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

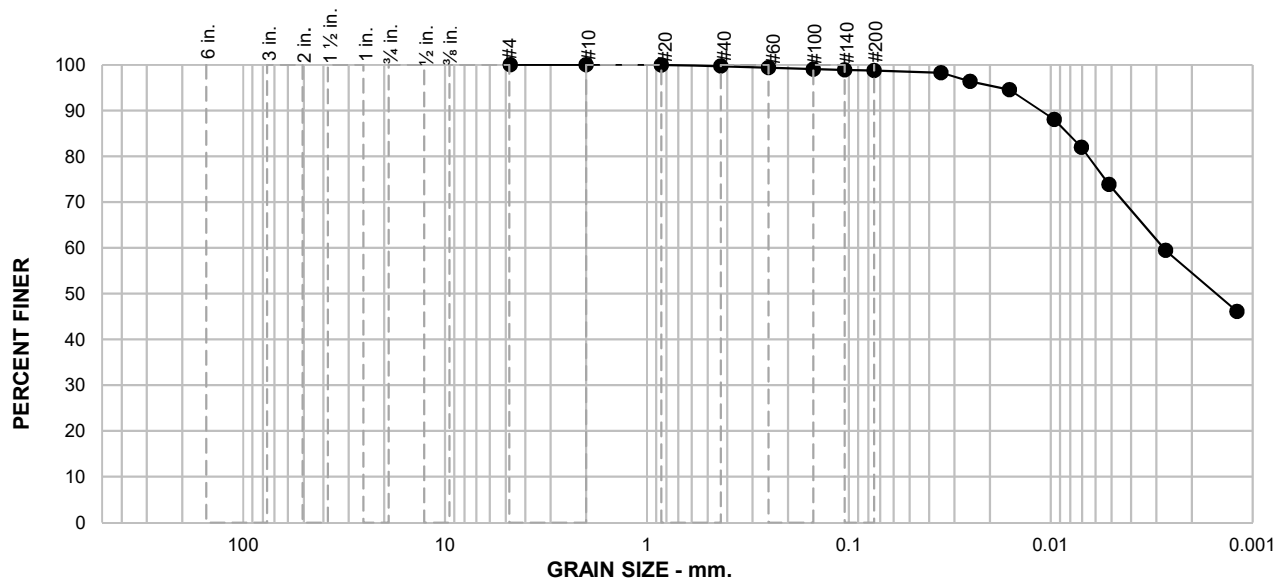
REPORT DATE: 3/29/2022

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D422



SAMPLE ID: 1-B5@1.5

DEPTH (ft): 1.5

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
					0	1	44.1	54.6
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION See exploration logs				
#4	100			ATTERBERG LIMITS PL = 28 LL = 75 PI = 47 COEFFICIENTS D ₉₀ = 0.0113 mm D ₈₅ = 0.0082 mm D ₆₀ = 0.0028 mm D ₅₀ = 0.0015 mm D ₃₀ = D ₁₅ = D ₁₀ = C _u = C _c =				
#10	100							
#20	100							
#40	100							
#60	99							
#100	99							
#140	99							
#200	99			CLASSIFICATION USCS = CH REMARKS Silt/clay division of 0.002mm used PI: ASTM D4318, Wet Method USCS: ASTM D2487				
0.0350 mm.	98.3							
0.0251 mm.	96.4							
0.0160 mm.	94.5							
0.0096 mm.	88.0							
0.0070 mm.	82.0							
0.0052 mm.	73.9							
0.0027 mm.	59.5							
0.0012 mm.	46.1							

* (no specification provided)

CLIENT: Anton Fong Ranch, LLC



PROJECT NAME: Anton Fong Ranch

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

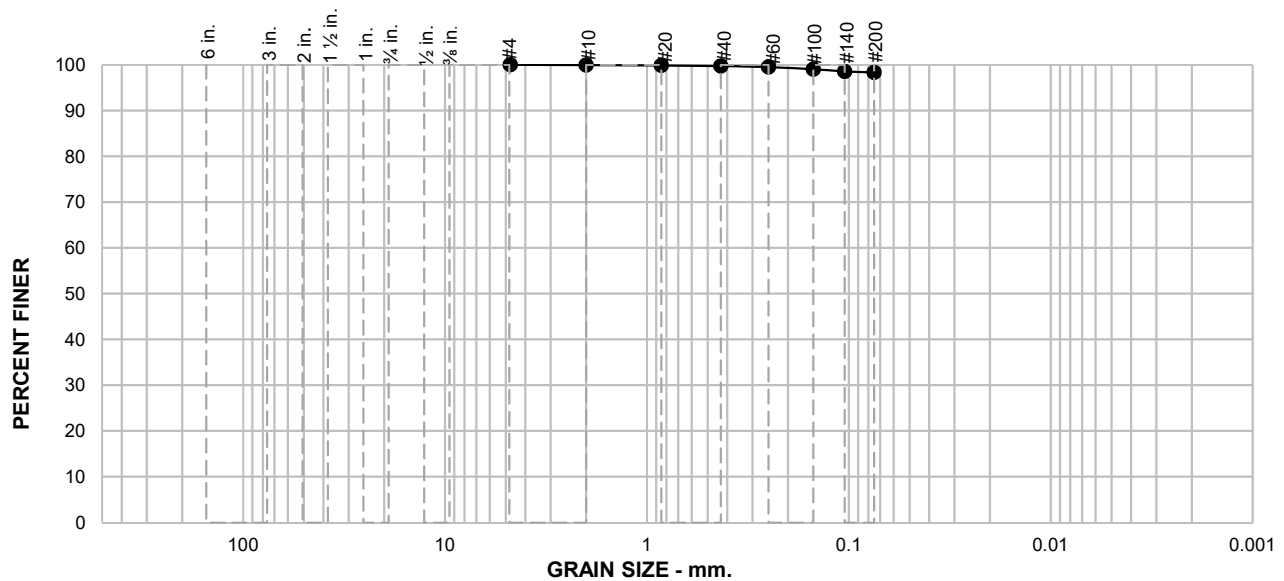
REPORT DATE: 3/29/2022

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B1@2

DEPTH (ft): 2

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
				0.1	0.1	1.5	98.3	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION				
				See exploration logs				
#4	100.0			ATTERBERG LIMITS PL = 26 LL = 60 PI = 34 COEFFICIENTS D₉₀ = D₈₅ = D₆₀ = D₅₀ = D₃₀ = D₁₅ = D₁₀ = C_u = C_c = CLASSIFICATION USCS = CH REMARKS PI: ASTM D4318, Wet Method USCS: ASTM D2487				
#10	99.9							
#20	99.9							
#40	99.8							
#60	99.5							
#100	99.0							
#140	98.6							
#200	98.3							

* (no specification provided)

CLIENT: Anton Fong Ranch, LLC



PROJECT NAME: Anton Fong Ranch

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

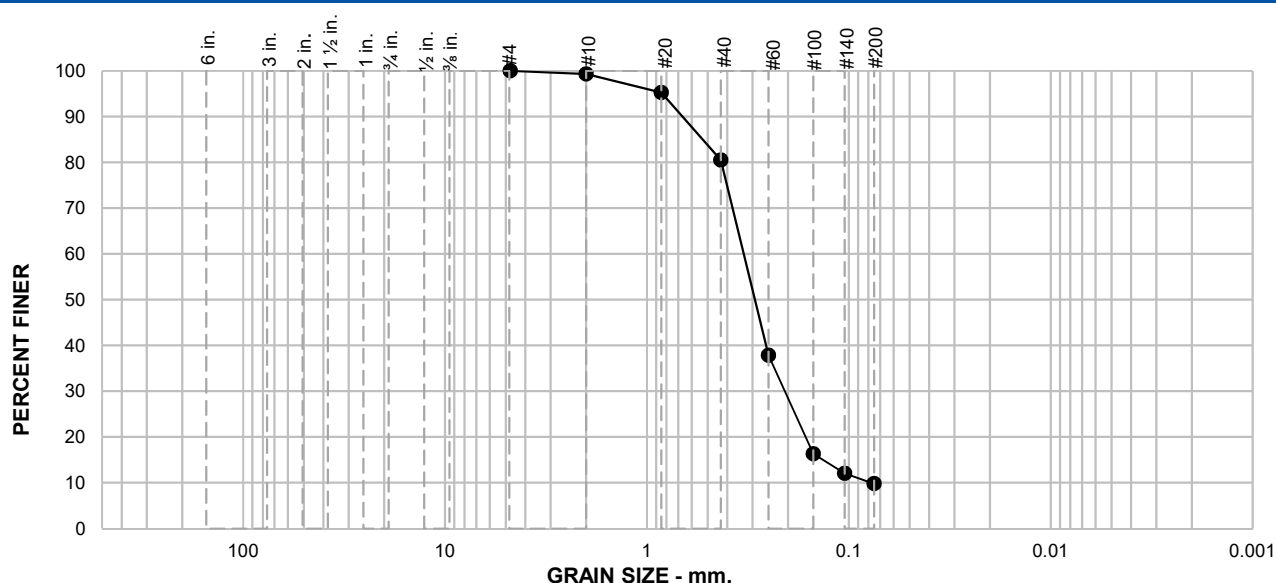
REPORT DATE: 3/29/2022

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B1@25

DEPTH (ft): 25

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
				0.7	18.8	70.7	9.8	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION See exploration logs				
#4	100.0			ATTERBERG LIMITS PL = LL = PI = COEFFICIENTS D₉₀ = 0.6679 mm D₈₅ = 0.5297 mm D₆₀ = 0.3312 mm D₅₀ = 0.2916 mm D₃₀ = 0.2074 mm D₁₅ = 0.1343 mm D₁₀ = 0.0772 mm C_u = 4.29 C_c = 1.68 CLASSIFICATION USCS = REMARKS				
#10	99.3							
#20	95.2							
#40	80.5							
#60	37.9							
#100	16.3							
#140	12.1							
#200	9.8							

* (no specification provided)

CLIENT: Anton Fong Ranch, LLC



PROJECT NAME: Anton Fong Ranch

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

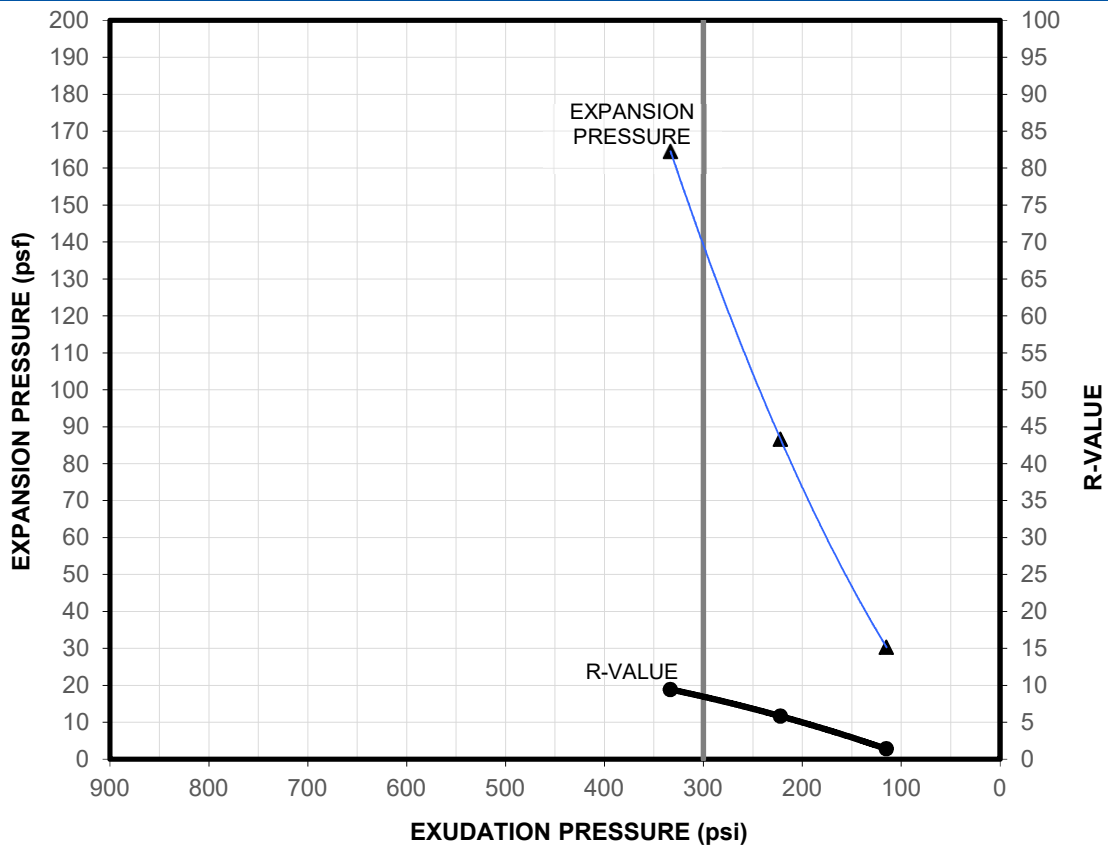
REPORT DATE: 3/29/2022

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard

R-VALUE TEST REPORT

CTM 301



SAMPLE ID	MATERIAL DESCRIPTION	SAMPLE LOCATION		
1-B3@0-1	See exploration logs	1-B3 at 0-1 feet		
SPECIMENS		1	2	3
EXUDATION PRESSURE (psi)		333	222	115
EXPANSION PRESSURE (psf)		165	87	30
R-VALUE		9	6	1
MOISTURE CONTENT (%)		28.2	30.6	33.4
DRY DENSITY (pcf)		90.4	88.2	85.1
EXPANSION PRESSURE (psf) AT EXUDATION PRESSURE OF 300 psi		140		
R-VALUE AT EXUDATION PRESSURE OF 300 psi		TEST RESULT		
		8		



CLIENT: Anton Fong Ranch, LLC

PROJECT NAME: Anton Fong Ranch

PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

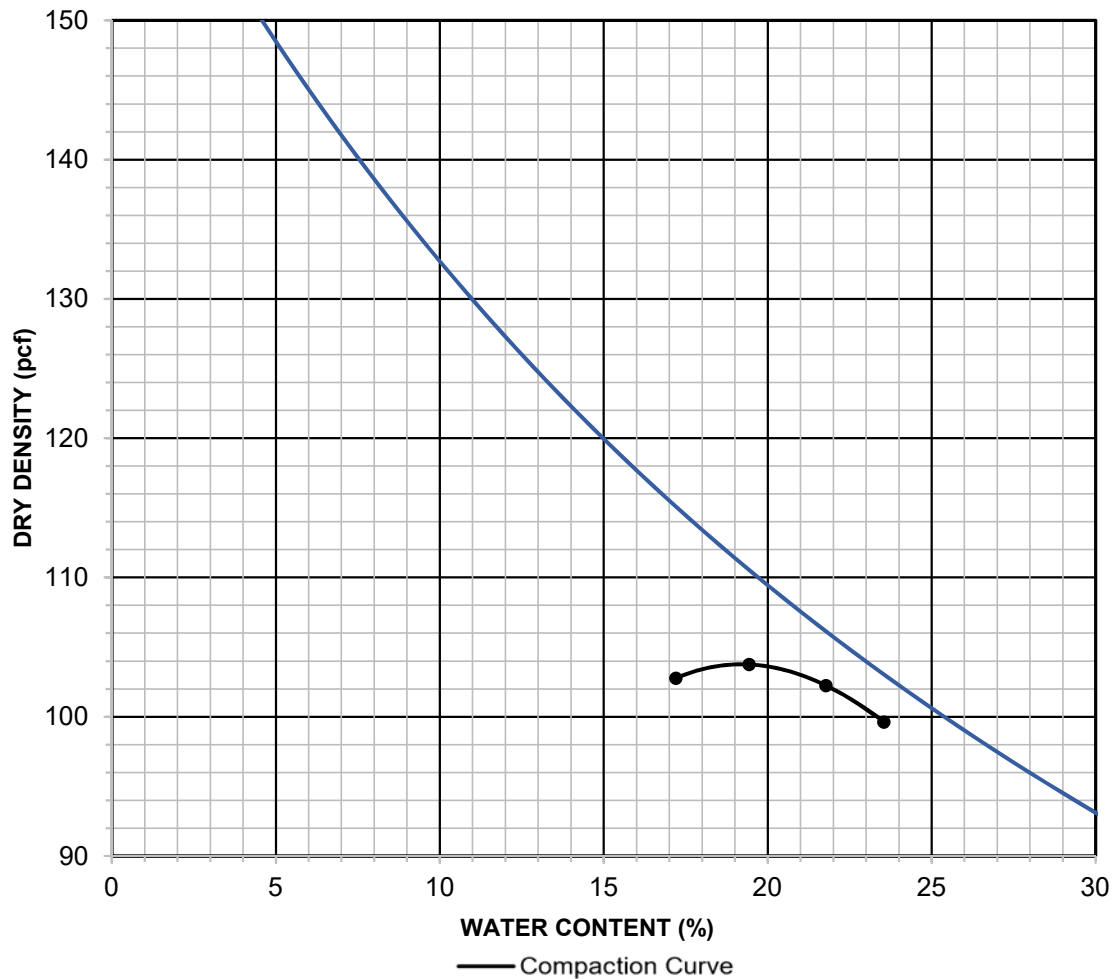
REPORT DATE: 3/29/2022

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard

COMPACTION CURVE REPORT

ASTM D1557



ZAV for
Specific
Gravity
2.70

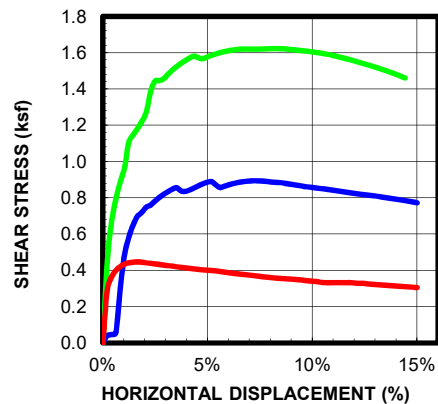
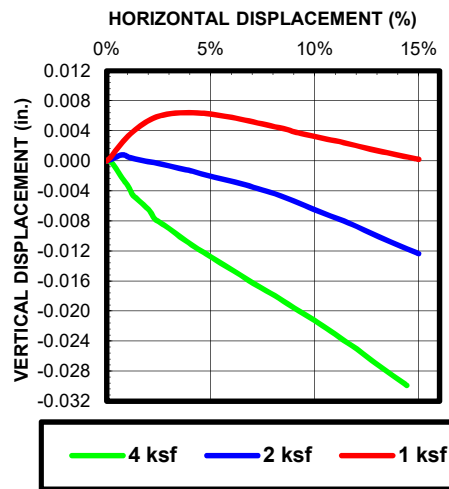
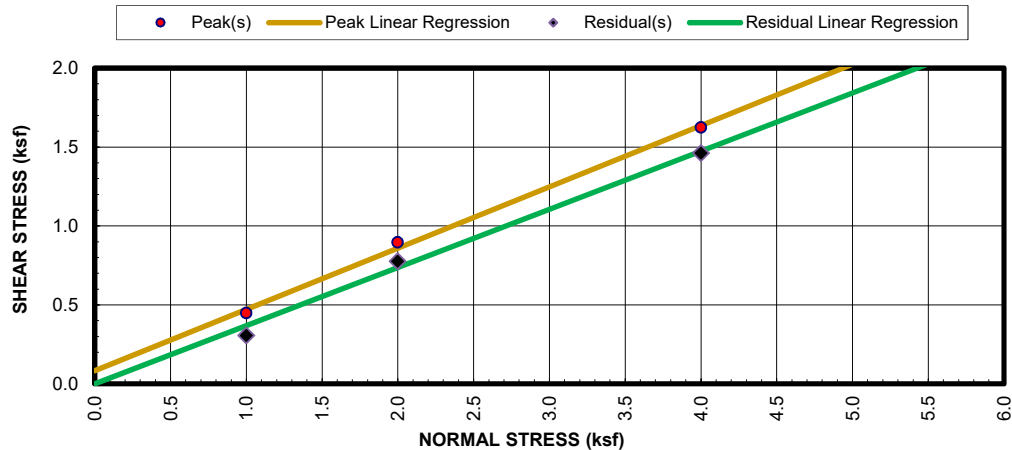
Curve Number: 1-B3@0-1
Test Specification: Method A
Sample Location: 1-B3 at 0-1 feet

RESULTS		MATERIAL DESCRIPTION
Maximum Dry Density, pcf	103.8	See exploration logs
Optimum Moisture Content, %	19.2	
Remarks		



CLIENT: Anton Fong Ranch, LLC
PROJECT NAME: Anton Fong Ranch
PROJECT NO: 20044.000.001 PH001
PROJECT LOCATION: Sacramento, CA
REPORT DATE: 3/29/2022
TESTED BY: R. Montalvo
REVIEWED BY: N. Broussard

CONSOLIDATED DRAINED DIRECT SHEAR ASTM D3080



SPECIMEN				
INITIAL PARAMETERS	4 ksf	2 ksf	1 ksf	
MOISTURE (%)	22.20	22.20	22.20	
DRY DENSITY (PCF)	93.42	93.28	93.36	
VOID RATIO	0.781	0.784	0.782	
SATURATION (%)	75.77	75.51	75.65	
DIAMETER (IN.)	2.412	2.412	2.412	
HEIGHT (IN.)	1.000	1.000	1.000	
DIAMETER-TO-HEIGHT RATIO	2.412	2.412	2.412	
SPECIFIC GRAVITY (ASTM D854)	2.665	2.665	2.665	
FINAL PARAMETERS	4 ksf	2 ksf	1 ksf	
MOISTURE (%)	27.90	29.25	29.74	
DRY DENSITY (PCF)	95.42	93.49	92.80	
VOID RATIO	0.744	0.780	0.793	
SATURATION (%)	100.0	100.0	100.0	
DIAMETER (IN.)	2.412	2.412	2.412	
HEIGHT (IN.)	0.979	0.998	1.006	
NORMAL STRESS (ksf)	4.00	2.00	1.00	
PEAK STRESS (ksf)	1.62	0.89	0.45	
PEAK STRAIN (%)	8.29	7.26	1.66	
RESIDUAL STRESS (ksf)	1.46	0.77	0.31	
RESIDUAL STRAIN (%)	14.43	15.00	15.00	
RATE (IN/MIN)	0.000359	0.000377	0.000503	
DIAMETER-TO-HEIGHT RATIO	2.463	2.417	2.398	
SPECIMEN INFORMATION		STRENGTH PARAMETERS		
SAMPLE ID:	1-B3@0-1		φ°	C(psf)
DEPTH (ft):	0-1 feet	PEAK:	21.2	83.0
SAMPLE TYPE:	In-situ	RESIDUAL:	20.2	0.0
DESCRIPTION:	See exploration logs	ASTM D4318		
		LIQUID LIMIT	n/a	
		PLASTIC LIMIT:	n/a	
REMARKS:	Specimens remolded to 90% of ASTM D1557 max. dry density of 103.8 pcf and +3% over optimum moisture content of 19.2%			



CLIENT: Anton Fong Ranch, LLC
PROJECT NAME: Anton Fong Ranch
PROJECT NO: 20044.000.001 PH001
PROJECT LOCATION: Sacramento, CA
REPORT DATE: 3/30/2022
TESTED BY: R. Montalvo
REVIEWED BY: N. Broussard

WATER-SOLUBLE SULFATE IN SOIL
ASTM C1580

SAMPLE NO.	SAMPLE ID.	DEPTH (Ft.)	MATERIAL DESCRIPTION	SAMPLE LOCATION	WATER SOLUBLE SULFATE % BY MASS
1	1-B3@2	2	See exploration logs	1-B3 at 2 feet	ND

¹ Per the test method, results are reported to the nearest 0.01% by weight or 100 ppm. Results less than 0.01% or 50 ppm will be reported as ND (Not Detectable)



CLIENT: Anton Fong Ranch, LLC

PROJECT NAME: Anton Fong Ranch

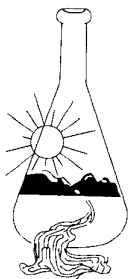
PROJECT NO: 20044.000.001 PH001

PROJECT LOCATION: Sacramento, CA

REPORT DATE: 3/30/2022

TESTED BY: R. Montalvo

REVIEWED BY: N. Broussard



Sunland Analytical

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

Date Reported 03/25/2022
Date Submitted 03/21/2022

To: Alli Hauger
Engeo, Inc.
2213 Plaza Dr.
Rocklin, CA 95765

From: Gene Oliphant, Ph.D. \ Randy Horney *RA*
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 20044.000.001 Site ID : 1-B4 @ 2.
Thank you for your business.

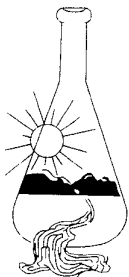
* For future reference to this analysis please use SUN # 86970-180942.

EVALUATION FOR SOIL CORROSION

Soil pH	7.02		
Minimum Resistivity	0.96 ohm-cm (x1000)		
Chloride	7.3 ppm	00.00073	%
Sulfate	39.7 ppm	00.00397	%

METHODS

pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m



Sunland Analytical

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

Date Reported 03/25/2022
Date Submitted 03/21/2022

To: Alli Hauger
Engeo, Inc.
2213 Plaza Dr.
Rocklin, CA 95765

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : 20044.000.001 Site ID : 1-B2 @ 6.
Thank you for your business.

* For future reference to this analysis please use SUN # 86970-180943.

EVALUATION FOR SOIL CORROSION

Soil pH	7.29	
Minimum Resistivity	0.62 ohm-cm (x1000)	
Chloride	4.5 ppm	00.00045 %
Sulfate	83.7 ppm	00.00837 %

METHODS

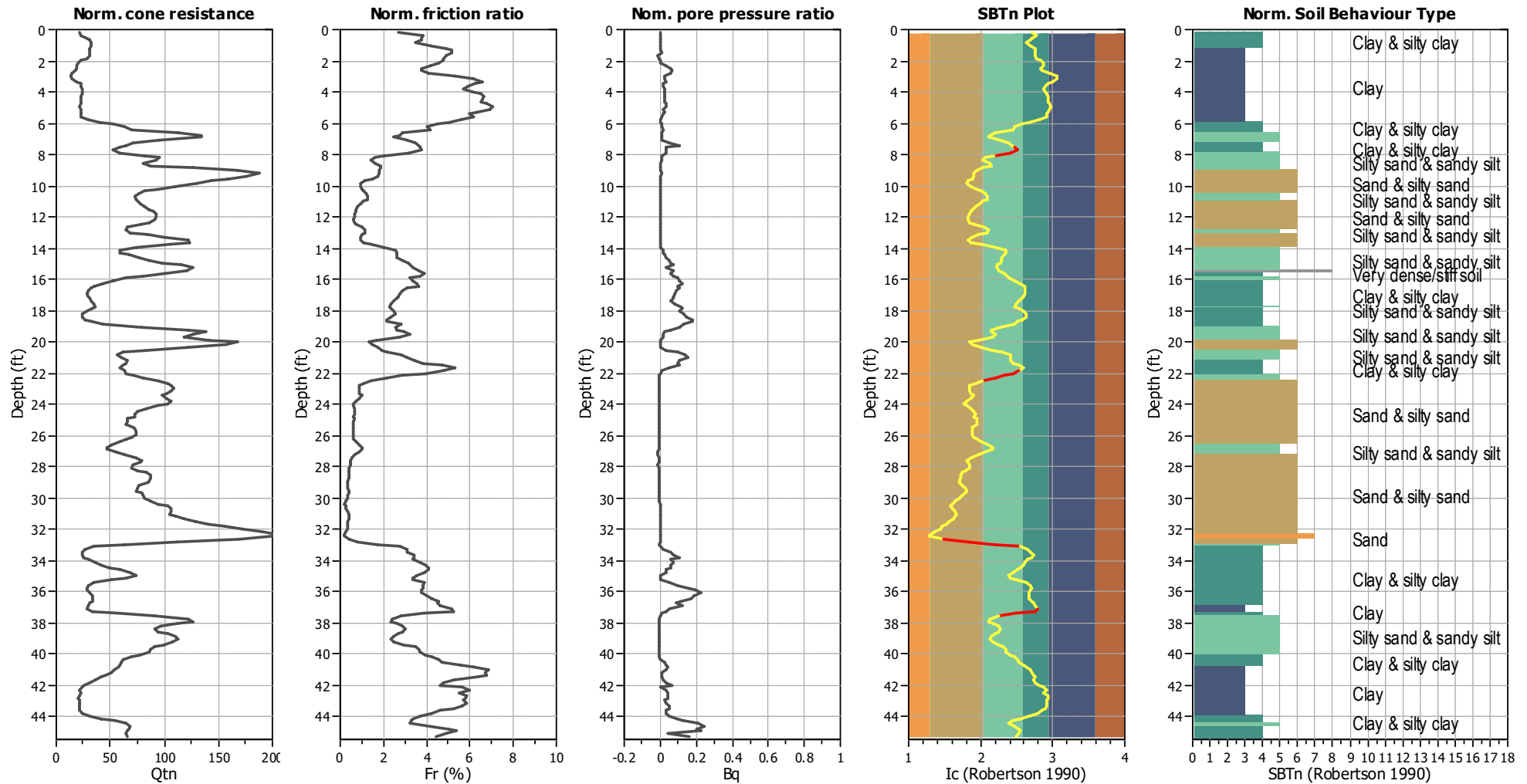
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m



APPENDIX C

CPT DATA

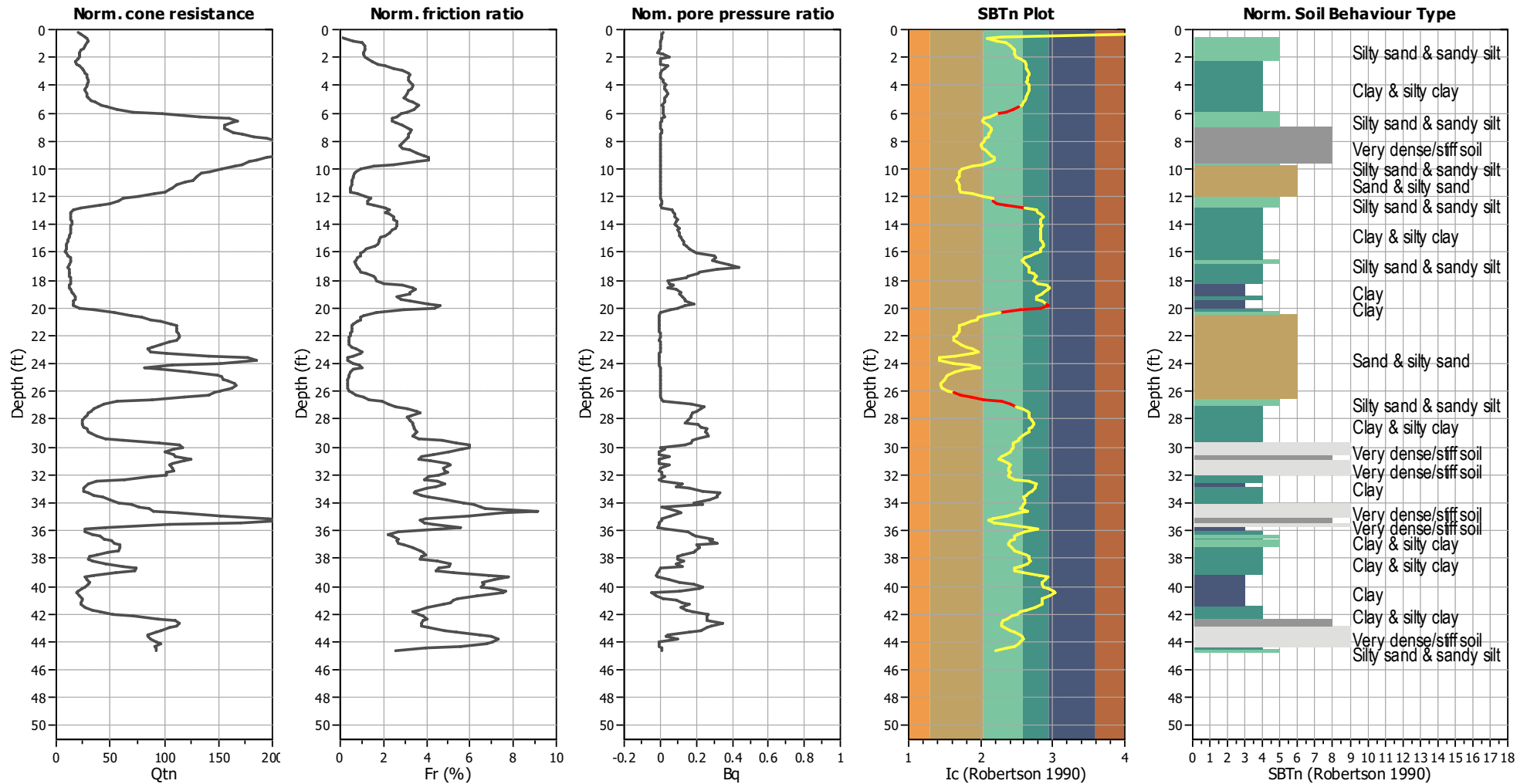
CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	No
Earthquake magnitude M _w :	6.60	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	17.00 ft	Fill height:	N/A	Limit depth:	N/A

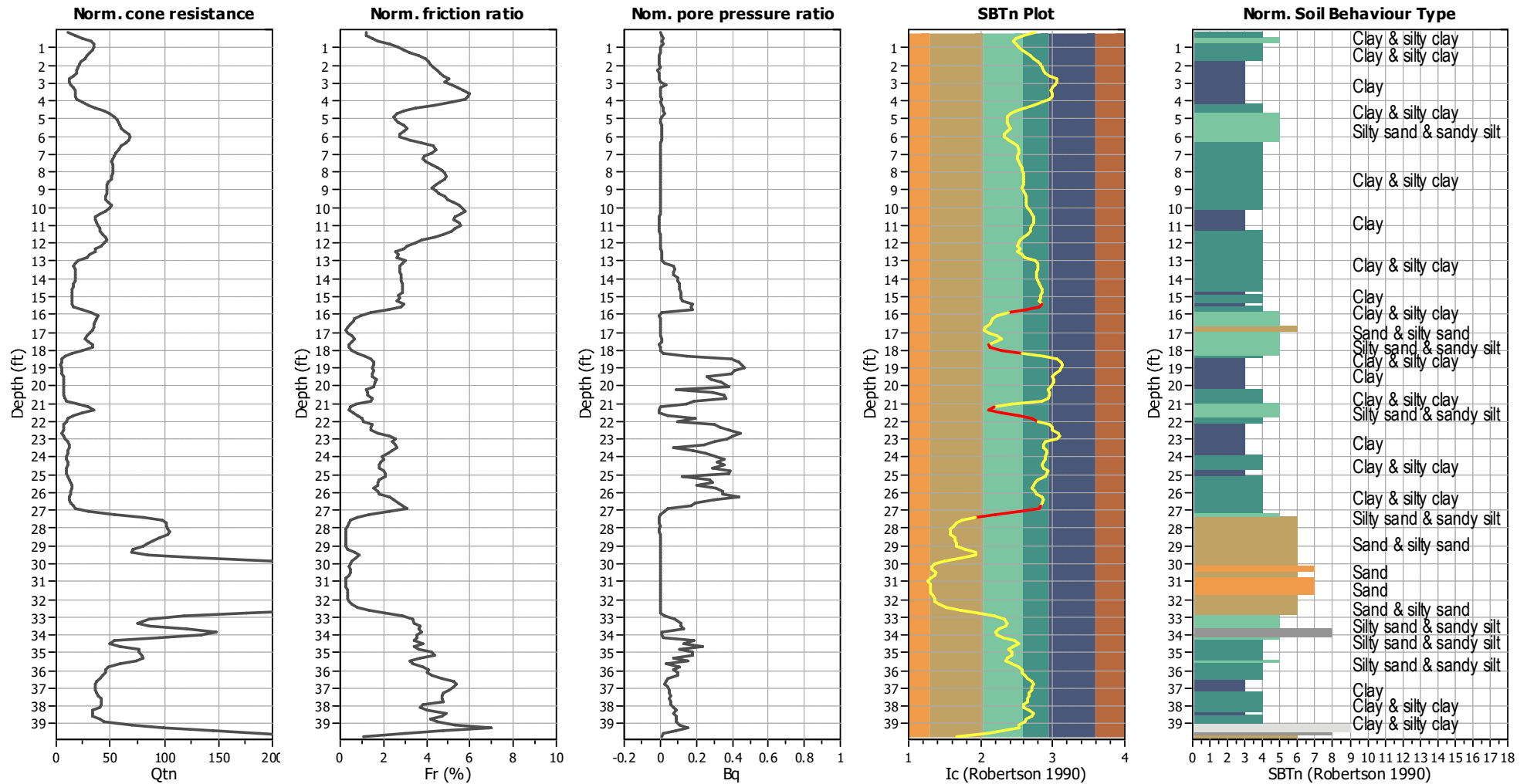
CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	No
Earthquake magnitude M _w :	6.60	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	17.00 ft	Fill height:	N/A	Limit depth:	N/A

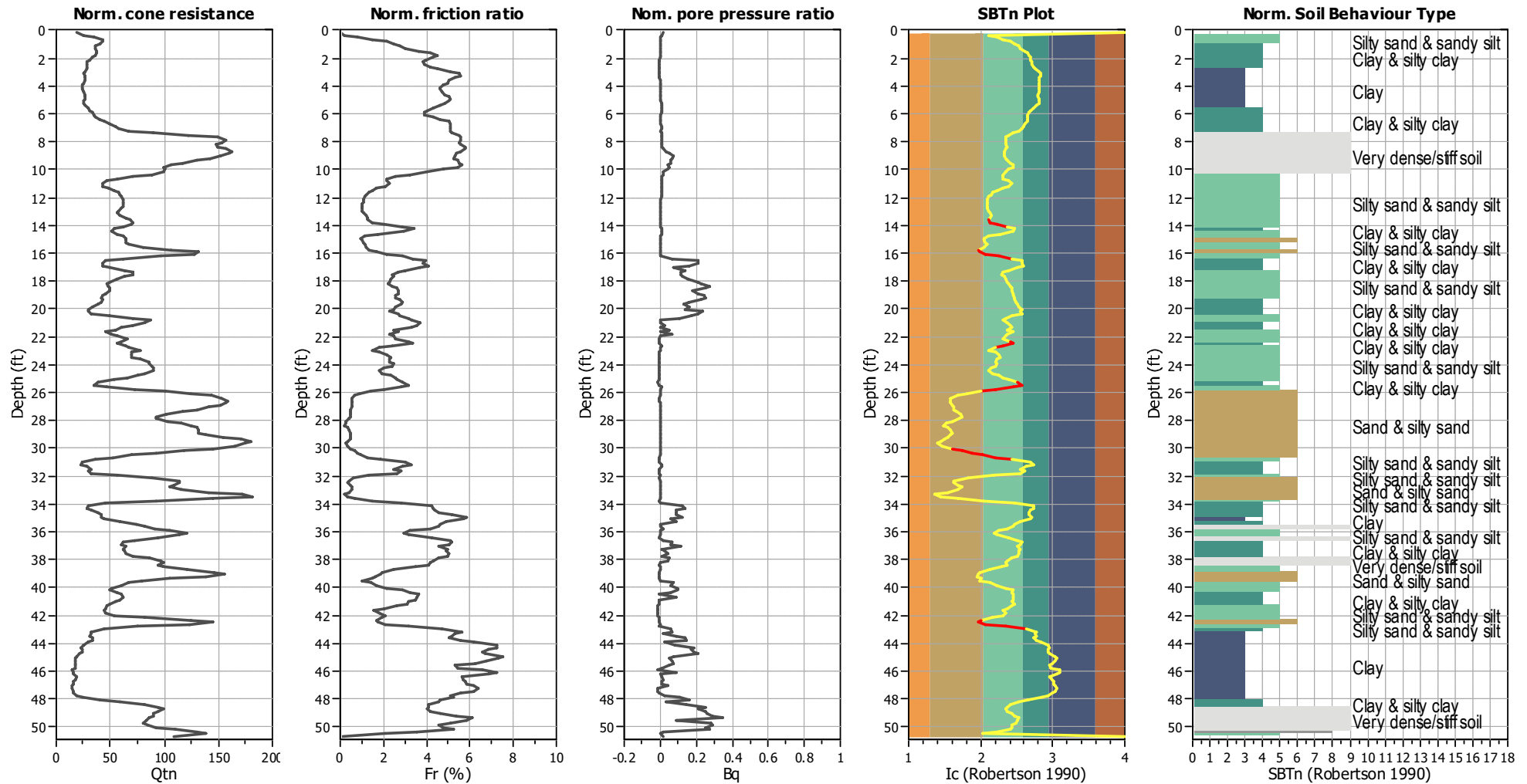
CPT basic interpretation plots (normalized)



Input parameters and analysis data

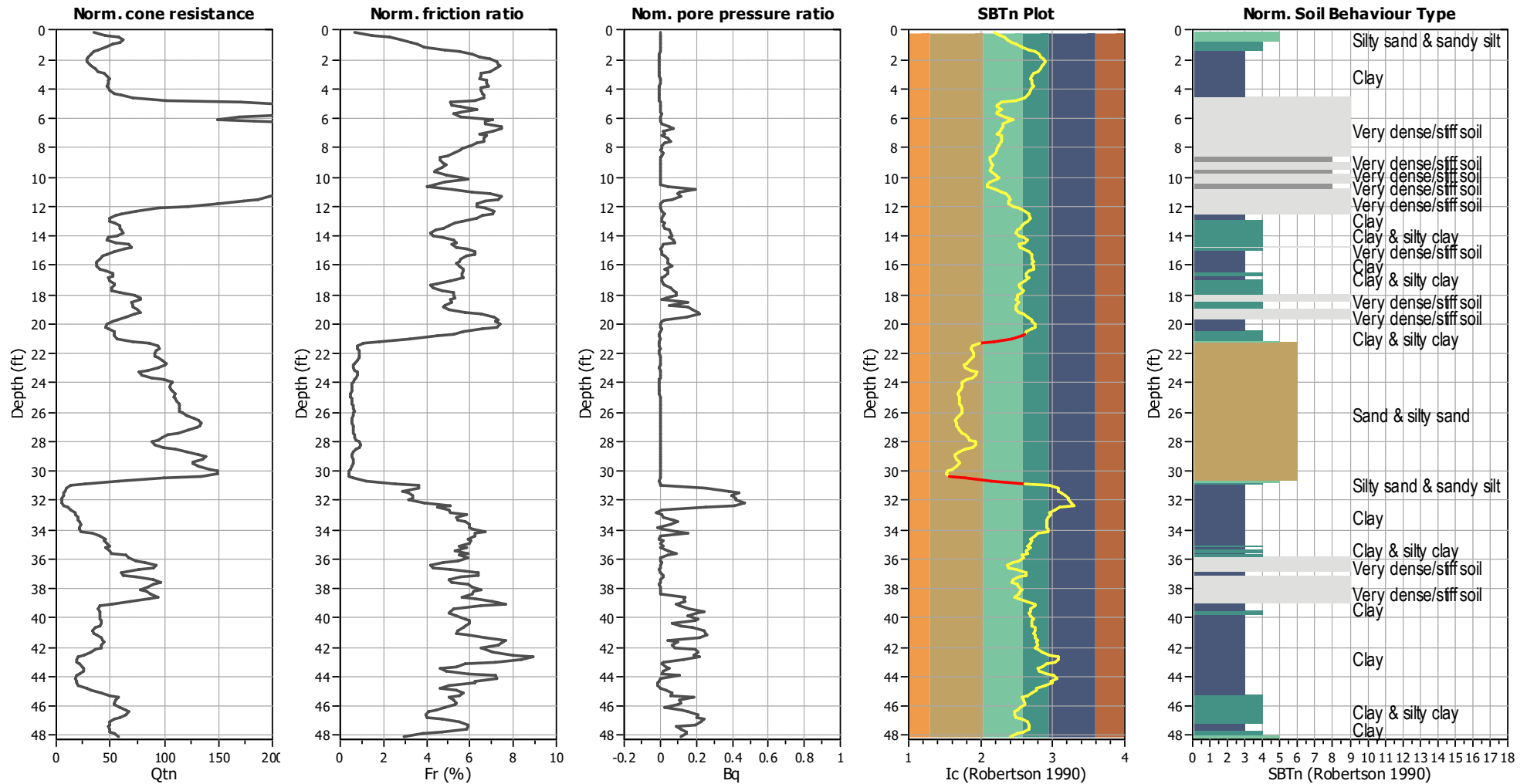
Analysis method:	Robertson (2009)	Depth to water table (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K _o applied:	No
Earthquake magnitude M _w :	6.60	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	17.00 ft	Fill height:	N/A	Limit depth:	N/A

CPT basic interpretation plots (normalized)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	No
Earthquake magnitude M _w :	6.60	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	17.00 ft	Fill height:	N/A	Limit depth:	N/A

CPT basic interpretation plots (normalized)**Input parameters and analysis data**

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	12.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K ₀ applied:	No
Earthquake magnitude M _w :	6.60	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.32	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	17.00 ft	Fill height:	N/A	Limit depth:	N/A

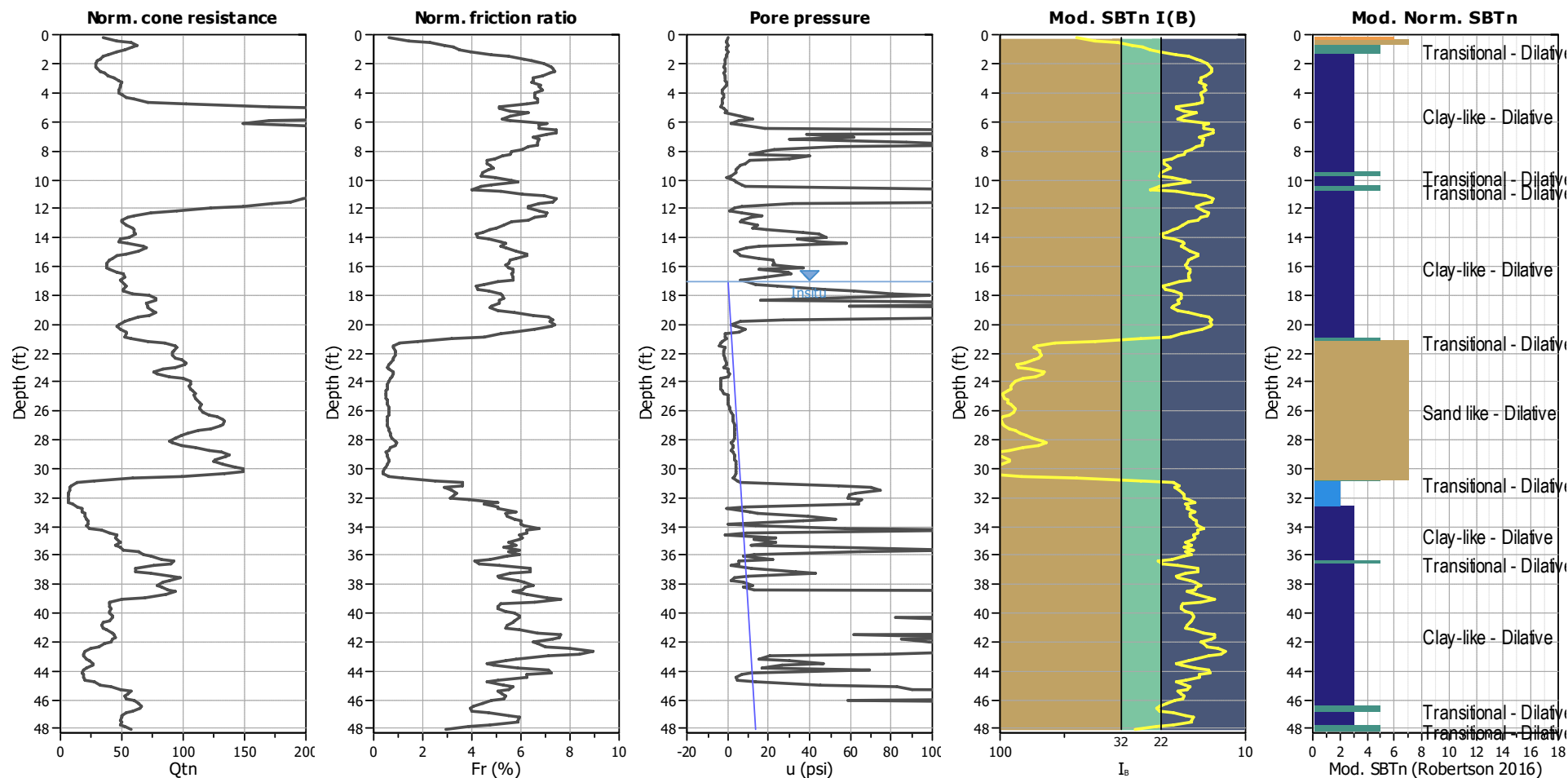
SBTn legend

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

Project: Fong Ranch
Location: Sacramento, CA

CPT: 1-CPT5

Total depth: 48.06 ft



Analysis method:	Robertson (2009)	G.W.T. (in-situ):	17.00 ft	Use fill:	No	Clay like behavior	
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	12.00 ft	Fill height:	N/A	applied:	All soils
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w :	6.60	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.32	Unit weight calculation:	Based on SBT	K_g applied:	No	MSF method:	Method based

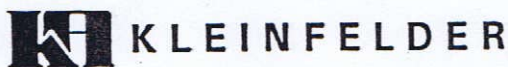


APPENDIX D

HISTORICAL BORING LOGS BY OTHERS

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	 GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			 GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	 GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			 GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	 SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			 SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	 SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			 SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)	 ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY	
		 CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
		 OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY	
	SILTS AND CLAYS (Liquid limit greater than 50)	 MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT	
		 CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
		 OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY	
HIGHLY ORGANIC SOILS			 PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS



UNIFIED SOIL CLASSIFICATION SYSTEM
NATOMAS LEVEE CERTIFICATION
SACRAMENTO, CALIFORNIA

PLATE

A-1

Drafted By: dwa Project No.: 23-482763-GEO
Date: 7/26/96 File Number:

LOG SYMBOLS

	BULK / BAG SAMPLE	-4	PERCENT FINER THAN THE NO. 4 SIEVE (ASTM Test Method C 136)
	MODIFIED CALIFORNIA SAMPLER (2-1/2 inch outside diameter)	-200	PERCENT FINER THAN THE NO. 200 SIEVE (ASTM Test Method C 117)
	CALIFORNIA SAMPLER (3 inch outside diameter)	LL	LIQUID LIMIT (ASTM Test Method D 4318)
	STANDARD PENETRATION SPLIT SPOON SAMPLER (2 inch outside diameter)	PI	PLASTICITY INDEX (ASTM Test Method D 4318)
	SHELBY TUBE (3 inch outside diameter)	EI	EXPANSION INDEX (UBC Standard 29-2)
	CONTINUOUS SAMPLER (3 inch outside diameter)	COL	COLLAPSE POTENTIAL
	WATER LEVEL (level after completion)	UC	UNCONFINED COMPRESSION
	WATER LEVEL (level where first encountered)	MC	MOISTURE CONTENT

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.



KLEINFELDER

LOG KEY

NATOMAS LEVEE CERTIFICATION

SACRAMENTO, CALIFORNIA

PLATE

A-2

Drafted By: dwa
Date: 7/26/96

Project No.: 23-482763-GEO
File Number:

Surface Conditions: Top of levee covered by loose soil and some broken organics.

Date Completed: 7/17/96

Groundwater: Encountered at a depth of approximately 18 feet below existing site grade.

Logged By: M. Rutz

Total Depth: 21-1/2 (feet)

Depth (feet)	FIELD				LABORATORY						Lithography	DESCRIPTION 0.0 to 21.5 feet
	Sample Type	Sample No.	Blows/ft	Pen (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)		
												Approximate Surface Elevation (feet): 15.4
		1A	20									Sandy SILT (ML): Red-gray, slightly moist, very stiff, low plasticity, with dark brown layer
		2A	17									Silty CLAY (CH): Red-gray, moist, very stiff, high plasticity, with dark brown lens
5		3A	17									dark brown with red-brown veins
										UC=4.2 tsf		
10		4B 4A	13		87	32						gray, stiff, moderate to high plasticity
15		5A	16									Sandy SILT (ML): Olive-brown and red-brown, moist, very stiff, low plasticity, with red-brown to black veins
20		6A	12									SAND (SP-SM): Dark brown, wet, loose to medium dense, fine to medium grained, with silt
25												Boring completed at a depth of approximately 21-1/2 feet below existing site grade.

KLEINFELDER

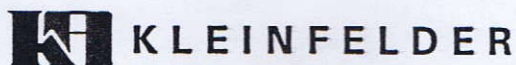
LOG OF BORING B-17
NATOMAS LEVEE CERTIFICATION
SACRAMENTO, CALIFORNIA

PLATE
1 of 1
A-19

Drafted By: dwa Project No.: 23-482763-GEO
Date: 7/26/96 File Number:

Surface Conditions: Top of levee covered by exposed soil and some organics.Date Completed: 7/17/96Groundwater: Encountered at a depth of approximately 20 feet below existing site grade.Logged By: M. RutzTotal Depth: 21-1/2 (feet)

Depth (feet)	FIELD				LABORATORY							Lithography	DESCRIPTION
	Sample Type	Sample No.	Blows/ft	Pen (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)	Other Tests		0.0 to 21.5 feet
													Approximate Surface Elevation (feet): 15.8
		1A	19										Sandy SILT (ML): Yellow-brown with dark brown, very stiff, low to moderate plasticity
		2A	20										cemented chunks
5		3A	21										Silty CLAY (CH): Dark brown, moist, very stiff, high plasticity, with cemented fragments
													with sand and scattered fine to coarse gravel
10		4A	15										dark brown with red-brown, sand and gravel fraction absent
													yellow-brown
15		5A	75										Sandy SILT (ML): Yellow-brown, moist, hard, low plasticity, with cemented grains
													Clayey SILT (ML): Yellow-brown, wet, stiff, low plasticity, with sand
20		6A	10										
25													Boring completed at a depth of approximately 21-1/2 feet below existing site grade.



LOG OF BORING B-18
NATOMAS LEVEE CERTIFICATION
SACRAMENTO, CALIFORNIA

PLATE
1 of 1

A-20

Drafted By: dwa
Date: 7/26/96
Project No.: 23-482763-GEO
File Number:

Surface Conditions: Top of levee covered by loose material.Date Completed: 7/18/96

Groundwater: _____

Logged By: M. RutzTotal Depth: 21-1/2 (feet)

Depth (feet)	FIELD				LABORATORY						Lithography	DESCRIPTION
	Sample Type	Sample No.	Blows/ft	Pen (tsf)	Dry Density (pcf)	Moisture Content (%)	Liquid Limit	Plasticity Index	Passing #4 Sieve (%)	Passing #200 Sieve (%)		0.0 to 21.5 feet
												Approximate Surface Elevation (feet): 15.7
												Recently Placed Dredge Fill
5		1A	12		91	25						Silty CLAY (CH): Dark brown, moist, stiff, high plasticity red-gray with trace sand
		2A	21									
		3A	12									
10												
		4A	15									slight sand fraction
15												
		5A	45/10*									red-gray, weakly cemented Sandy SILT (ML): Olive-brown, moist, hard, with some weakly cemented particles and trace clay
20												
			26									
25												Boring completed at a depth of approximately 21-1/2 feet below existing site grade.



KLEINFELDER

LOG OF BORING B-26

NATOMAS LEVEE CERTIFICATION

SACRAMENTO, CALIFORNIA

PLATE

1 of 1

A-28

Drafted By: dwa
Date: 7/26/96Project No.: 23-482763-GEO
File Number: _____



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