Project No.
12165.000.002

April 5, 2019
Latest Revision July 29, 2019

Mr. Tim Steele
The Sobrato Organization
10600 N. De Anza Boulevard, Suite 200
Cupertino, CA 95014

Subject: Sobrato Property - Area 4
7200 Stevenson Boulevard
Newark, California

SUMMARY OF PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Steele:

As requested, we prepared this letter to evaluate whether any changes to the project or the circumstances, or new information identified in the technical reports listed below, would result in a changed evaluation of impacts or mitigation measures listed in the 2015 *Newark Areas 3 and 4 Specific Plan Project Recirculated EIR (2015 REIR)*, for the Sobrato Property Area 4 project. As discussed below, the conclusions and recommendations of the following reports are consistent with the conclusions and recommendations in the REIR:

- ENGEO; Preliminary Geotechnical Exploration, Sobrato Property - Area 4, Newark, California; Project No. 12165.000.002; April 5, 2018.
- ENGEO; Preliminary Geotechnical Exploration, Stevenson Boulevard Bridge – East Abutment, Sobrato Property – Area 4, Newark, California; Project No. 12165.000.002; December 4, 2018.
- ENGEO; Surcharge Recommendations, Sobrato Property – Area 4, Newark, California; Project No. 12165.000.002; April 8, 2019.

We did not identify any new impacts and no new mitigation measures are required.

The Site is bounded by Union Pacific Railroad (UPRR) tracks on the northeast, an Alameda County Flood Control canal on the northwest and southeast, and Mowry Slough on the southwest. The Vesting Tentative Map (VTM) prepared by Carlson, Barbee, & Gibson, Inc. (CBG) dated July 11, 2019, shows that the property will be subdivided into 469 residential lots, three park parcels, and four boardwalk overlooks. Overall, the density of the development will be 2.6 dwelling units per acre. No development is planned within the existing wetlands areas. Four bridges are proposed as part of the street system, including a bridge extending Stevenson Boulevard over the existing Union Pacific Railroad on the north portion of the site. Two bridges are proposed for the emergency vehicle access (EVA)/trail along the UPRR tracks. We understand that bridges will be designed to cross existing wetland areas and abutments for the bridges will be located outside of the wetland areas, avoiding the need for wetland fill.

GEOTECHNICAL EXPLORATION MITIGATION MEASURES

REIR mitigation measures GEO-1.1, GEO-2.1, GEO-3.1, GEO-5.1, and GEO-6.1 require design-level geotechnical investigations for proposed project elements, including residences, bridges, and other improvements. REIR mitigation measure GEO-4.1 requires design-level geotechnical investigations for the proposed Stevenson Bridge.

We performed preliminary geotechnical investigation and engineering analyses for the project. As discussed in the referenced reports and based on those preliminary investigations, laboratory test results and analyses, we conclude that the proposed development at Area 4 and the proposed Stevenson Boulevard Bridge are feasible from a geotechnical and geologic standpoint, provided that the preliminary recommendations included in the referenced reports, which are consistent with the mitigation measures identified in the 2015 REIR, are incorporated in the design and construction of the project. No new mitigation measures are required.

Liquefaction

Consistent with REIR mitigation measures GEO-1.1 and GEO-2.1, a design-level geotechnical study will be required to be prepared to characterize and mitigate potential liquefaction-induced settlement and lateral deformation. Additional subsurface exploration and collection of soil samples during the design-level study will better delineate the areas with a potential for liquefaction and estimates of liquefaction-induced settlement magnitude. Specifically, the design-level study will analyze the lateral extent, thickness and volumetric strain of potentially liquefiable soils. The design level geotechnical report will consider the current California Building Standards Code guidelines and the requirements of American Society of Civil Engineers (ASCE) 7-16 Section 12.13.9 when evaluating liquefaction induced settlement and lateral deformation.

Based on the requirements of ASCE 7-16 Section 12.13.9, the maximum allowable differential settlement from liquefaction should not exceed 5½ inches, which is 0.01L (Table 12.13-3 of ASCE 7-16). Our estimates of differential settlement presented in the Preliminary Geotechnical Exploration Report are less than the threshold in ASCE 7-16, as such post-tensioned mat foundations are suitable to mitigate liquefaction-induced ground settlement without soil mitigation methods (or rigid foundations). If liquefaction-induced settlement concluded after completion of the design-level geotechnical exploration exceeds 5½ inches, REIR mitigation measures GEO-1.1 and GEO-2.1, which contemplate additional methods such as deep foundations or ground improvement methods, will be implemented and will mitigate any potential impacts.

REIR mitigation measures GEO-1.1 and GEO-2.1 will reduce impacts from seismically induced liquefaction and lateral spreading risks to a less than significant level. The proposed project will not result in any new or substantially more significant impacts or require new mitigation measures with respect to liquefaction induced settlement and lateral deformation compared to those identified in the REIR.

Compressible Bay Mud

Consistent with REIR mitigation measure GEO-3.1, ground improvement consisting of a surcharge program is recommended in the Surcharge Recommendation Letter dated April 8, 2019. The surcharge program will reduce load-induce settlement by placement of fill and building loads over compressible Bay Mud. The surcharge program is designed to achieve ½ inch or less of

post-construction residual consolidation settlement. As discussed in the Preliminary Geotechnical Report, properly designed post-tensioned foundation mats will be able to resist the residual differential settlement induced by consolidation settlement after surcharge mitigation is performed. If larger amounts of consolidation induced settlement are concluded after completion of the design-level geotechnical exploration, additional methods such as deep foundations or ground improvement methods, consistent with REIR mitigation measure GEO-3.1, will be implemented.

REIR mitigation measure GEO-3.1 will reduce impacts from load-induced consolidation settlement risks to a less than significant level. The proposed project will not result in any new or substantially more significant impacts or require new mitigation measures with respect to load-induced consolidation settlement compared to those identified in the REIR.

Existing Fill

It should be noted that the auto dismantler site is no longer part of the Area 4 development area. We did not encounter significant amounts of existing non-engineered fill within Area 4 in our preliminary geotechnical exploration study. If encountered, non-engineered fill, which may be poorly compacted, will be over-excavated and recompacted or removed and replaced, consistent with REIR mitigation measure GEO-5.1.

REIR mitigation measure GEO-5.1 will reduce impacts from undocumented existing fill to a less than significant level. The proposed project will not result in any new or substantially more significant impacts or require new mitigation measures with respect to undocumented fill compared to those identified in the REIR.

Expansive Fill

The Preliminary Geotechnical Exploration Report for Area 4 identified low to moderately expansive soil within the upper 5 feet of the site. However, since site grades will be raised significantly to achieve finished grades, we recommended imported soil consist of low to moderately expansive clay (PI of 20 or less and more than 70 percent passing No. 200 sieve). REIR mitigation measure GEO-6.1 will reduce impacts from expansive fill to a less than significant level. The proposed project will not result in any new or substantially more significant impacts or require new mitigation measures with respect to expansive fill compared to those identified in the REIR.

Groundwater

As presented in the Preliminary Geotechnical Exploration Report, groundwater within Area 4 ranged from 0 to 5 feet below ground surface. However, the project ground surface will be raised between 5 to 16 feet by engineered fill. When site improvement plans are available, we should perform a plan review to evaluate whether underground improvements will be subject to hydrostatic uplift pressures, consistent with REIR mitigation measure GEO-7.1. Supplemental recommendations consistent with REIR mitigation measure GEO-7.1 will be implemented, if needed, to anchor pipelines subject to hydrostatic uplift if needed. Mitigation measure GEO-7.1 will reduce impacts from shallow ground water to a less than significant level. The proposed project will not result in any new or substantially more significant impacts or require new mitigation measures with respect to shallow ground water compared to those identified in the REIR.

Soil Corrosivity

The project ground surface will be raised by engineered fill. Only soil that is not considered corrosive to structural elements will be accepted as engineered fill for the site. Nonetheless, the Earthwork and Import Fill Memo (ENGEO, October 2018) recommend soil corrosion testing during soil import and grading activities to determine corrosion levels for concrete and steel protection, which is required by mitigation measure GEO-8.1 in the REIR. With implementation of mitigation, impacts will be less than significant, which is consistent with the conclusion for corrosive soil in the REIR.

Slope Stability

The design level geotechnical report conducted pursuant to REIR mitigation measure GEO-3.1 will include a study to evaluate the risk of slope deformation and instability along the perimeter slopes of the project during grading, surcharge program and post-construction static and seismic conditions. Pursuant to REIR mitigation measure GEO-3.1, the Project applicant engaged ENGEO to undertake a site-specific investigation resulting in the Preliminary Geotechnical Report, which states graded slopes up to 6 feet may be constructed at an inclination of 2:1 (horizontal:vertical) and recommends measures for reinforcement as anticipated by the REIR, including a geogrid. If slope instability conditions, such as a “mud wave”, are identified in the additional design-level geotechnical exploration, the required design-level geotechnical exploration would ensure that appropriate design details, such as properly placed retaining walls or staging of fill placement, will be provided to prevent a potential “mud wave” from forming at the toe of the fill slope.

Stevenson Boulevard Bridge

Consistent with REIR mitigation measure GEO-4.1, additional design-level geotechnical investigations will be prepared to characterize and recommend measures to minimize potential load-induced settlement as well as a liquefaction-induced settlement within the abutments of the Stevenson Boulevard Bridge. Additional subsurface exploration and collection of soil samples during the design-level investigations will better delineate the areas with liquefiable and compressible soils. Specifically, the design-level study will analyze the lateral extent, thickness, compressibility and volumetric strain of compressible and potentially liquefiable soils. The design level geotechnical report will consider the current California Transportation Department Bridge Design Manual and Seismic Design Criteria.

Pursuant to REIR mitigation measure GEO-4.1, the Project applicant engaged ENGEO to undertake a site-specific investigation for the proposed Stevenson Boulevard Bridge, resulting in the Preliminary Geotechnical Report prepared for the Stevenson Boulevard Bridge – East Abutment. Based on this report, 66-inch-diameter cast-in-steel-shell (CISS) piles are recommended for supporting the bridge. As anticipated by the REIR, this report recommends measures to address load-induced settlement under the embankment:

- Ground improvements such as deep soil mixing or drilled displacement columns
- Placement of lightweight fill
- Surcharge program along with wick drains
- Long-term maintenance program to raise the settled embankment

Additional site-specific geotechnical explorations for the Stevenson Boulevard Bridge, which will include supplemental explorations within the east and west abutments, will be prepared for the Stevenson Boulevard Bridge consistent with REIR mitigation measure GEO-4.1. REIR mitigation measure GEO-4.1 will reduce impacts from load-induced consolidation settlement risks to a less than significant level. The proposed project will not result in any new or substantially more significant impacts or require new mitigation measures with respect to load-induced consolidation settlement compared to those identified in the REIR.

CLOSING

If you have any questions or comments regarding this letter, please call.

Sincerely,

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SOBRATO PROPERTY - AREA 4

NEWARK, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO

Mr. Tim Steele
The Sobrato Organization
10600 N. De Anza Boulevard, Suite 200
Cupertino, CA 95014

PREPARED BY

ENGEO Incorporated

April 5, 2018

PROJECT NO.

12165.000.002

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ENGEO
Expect Excellence

Project No.
12165.000.002

April 5, 2018

Mr. Tim Steele
The Sobrato Organization
10600 N. De Anza Boulevard, Suite 200
Cupertino, CA 95014

Subject: Sobrato Property - Area 4
7200 Stevenson Boulevard
Newark, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Steele:

We prepared this preliminary geotechnical report for The Sobrato Organization for the proposed residential development in Newark, California, as outlined in our agreement dated February 2, 2018. We characterized the subsurface conditions at the site to provide the enclosed geotechnical recommendations.

Based upon our initial assessment, the site is suitable for the planned development from a geotechnical standpoint provided the conclusions and preliminary recommendations presented in this report are incorporated into preliminary design. We recommend that a design-level study be performed and we hope that you will retain us for these services.

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

Sincerely,

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF SERVICES

We prepared this preliminary geotechnical report for the proposed residential development within Area 4 of the Sobrato property in Newark, California. We prepared this report as outlined in our agreement dated February 2, 2018. We are authorized to conduct the following scope of services:

- Review available literature and geologic maps.
- Review available geotechnical explorations and geophysical data.
- Perform subsurface field exploration.
- Perform soil laboratory testing.
- Analyze preliminary data.
- Provide preliminary recommendations.

We received the conceptual plans prepared by Carlson, Barbee & Gibson, Inc., dated February 8, 2018, which form our basis of understanding of the proposed development at the site.

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

1.2 PROJECT LOCATION

The roughly 450-acre site is bounded by Union Pacific railroad tracks on the northeast, an Alameda County Flood Control (ACFC) canal on the northwest and southeast, and Mowry slough on the southwest. The site is currently undeveloped and is vacant. The site is covered with seasonal grass and some low-lying areas are covered with water due to the recent rain.

1.3 PROJECT DESCRIPTION

Based on our discussions with you and review of the information provided, we understand that approximately half of the project site will be developed with residential structures and the remaining areas will remain as seasonal wetlands. The residential area will provide approximately 490 single-family houses, as well as several parks, recreational trails, new streets and utilities. The residential development area is further divided into Residential Areas B and C, as shown on Figure 1.

We understand that a bridge is proposed at the northeast boundary of the site; the bridge will extend Stevenson Boulevard over the existing Union Pacific Railroad. Additional smaller bridges are also planned within the project site.

As shown on the tentative map, site elevations currently range from -1 to 5 feet (NGVD29). We understand that the ground surface elevations within Residential Area B will be raised by 8 to 12 feet (Elevations 8 to 12 feet) and Residential C will be raised by approximately 5 feet (El. 10). The perimeter slopes of the project site is currently planned to be 2:1 (horizontal:vertical).

1.4 SITE HISTORY AND BACKGROUND

We reviewed historical aerial photographs dating back to 1946. The site appears to be used for agricultural purposes from 1946 to early 1990s. Additional Road and the northern ACFC canal appeared to be under construction in 1958 and completed by 1959. It appears that agricultural activities ceased in the 1990. The site remain vacant and undeveloped since the 1990s.

1.5 PREVIOUS STUDIES

We reviewed the geotechnical feasibility evaluation report for Newark, Areas 3 and 4, prepared by Cornerstone Earth Group (CEG), dated February 4, 2009, as part of our study. CEG advanced six boreholes and seven CPTs within the development area of Area 4. The CPT Logs from CEG's study are included in Appendix B. The approximate locations of the CEG explorations are shown on Figure 2.

2.0 SITE GEOLOGY AND SEISMICITY

2.1 GEOLOGY

The project site is located in the Coast Ranges geologic province of California, which is dominated by a series of northwest-trending valleys and ridges. The Coast Ranges are typified by a system of northwest-trending, fault-bounded mountain ranges and intervening alluviated valleys. Bedrock in the Coast Ranges consists of igneous, metamorphic and sedimentary rocks that range in age from Jurassic to Pleistocene. The present physiography and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate boundary fault movements are largely concentrated along the well-known fault zones, which in the area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-order faults.

According to Graymer (1997), the site is situated in an area mapped as Quaternary basin deposits (Qhb), Quaternary salt affected basin deposits (Qhbs), and Quaternary bay mud deposits (Qhbm), as shown on Figure 3.

2.2 REGIONAL SEISMICITY

The Bay Area contains numerous active faults. The major active faults in the area include the San Andreas to the west and the Hayward and Calaveras to the east. An active fault is defined by the State Mining and Geology Board as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997).

Numerous small earthquakes occur every year in the San Francisco Bay Region, and larger earthquakes have been recorded in the past and can be expected to occur in the future. Figure 4 shows the approximate locations of mapped active faults and significant historic earthquakes recorded within the San Francisco Bay Region. The following table lists several of the mapped active faults and their proximity to the site.

TABLE 2.2-1: Summarized Nearest Active Faults

FAULT NAME	APPROXIMATE DISTANCE FROM PROJECT SITE (MILES)	DIRECTION FROM SITE	MAXIMUM MOMENT MAGNITUDE
Hayward	4.4	Northeast	7.3
Calaveras	8.7	Northeast	7.0
Monte Vista Shannon	12.3	Southwest	6.5
San Andreas	14.6	Southwest	8.1
Mount Diablo Thrust	18.6	Northeast	6.7
Greenville Connected	22.0	Northeast	7.0

The Uniform California Earthquake Rupture Forecast (UCERF3, 2015) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area. The UCERF3 estimates an overall probability of 72 percent for the San Francisco Region as a whole.

The site is not located within a currently designated Earthquake Fault Special Study Zone and no known surface expression of active faults is believed to exist within the site. Fault rupture through the site, therefore, is not anticipated. There are no earthquake-induced site effects known to have occurred at the project site in historic earthquake events.

The site is mapped on the current seismic hazard zonation with potential permanent ground displacements due to liquefaction based on the Department of Conservation, California Geologic Survey Seismic Hazard Zone Maps. This liquefaction susceptibility mapping is based on regional geologic mapping of soil and rock deposits and is not based on site-specific exploration or analyses. We performed detailed analysis of the liquefaction-induced settlement and have provided the results in the subsequent sections of this report.

2.3 2016 CBC SEISMIC DESIGN PARAMETERS

Based on the subsurface conditions encountered, we characterized the site as Site Class D - Stiff Soil in accordance with the 2016 California Building Code (CBC). We provide the 2016 CBC seismic design parameters below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 2.3-1: 2016 CBC Seismic Design Parameters
Latitude: 37.50192°N **Longitude:** -122.00468°W

PARAMETER	VALUE
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	1.540
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.607
Site Coefficient, F_A	1.0
Site Coefficient, F_V	1.5
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	1.540
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	0.910
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.026

PARAMETER	VALUE
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.607
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.594 g
Long period transition-period, T_L	12 sec

3.0 FIELD EXPLORATION

Our field exploration included drilling four borings at various locations on the site. We performed our field exploration between February 15 and 16, 2018. The location of our explorations are based on handheld GPS with an accuracy of +/- 15 feet, as shown on Figure 2. The boring elevation is based on topographic elevations presented on the CBC tentative map dated February 8, 2018.

3.1 BORINGS

We observed drilling of our 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 the services of a subcontractor with a track-mounted Mobile B53 drill rig and crew to advance the borings using 6-inch-diameter mud rotary methods. We advanced the borings to depths ranging from 51½ to 54 feet below existing grade. We permitted and backfilled the borings in accordance with the requirements of Alameda County Water District.

We obtained soil samples at various intervals using standard penetration test (SPT) samplers and California Modified samplers. We obtained the blow counts shown on our bore logs with an automatic trip, 140-pound hammer with a 30-inch free fall. We drove the sampler 18 inches and recorded the number of blows for each 6 inches of penetration. The blow counts presented on the borelogs have not been converted using any correction factors. When sampler driving was difficult, we recorded the penetration only as inches penetrated for 50 hammer blows. We also obtained pushed Shelby tube samples of the Young Bay Mud soil.

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

3.2 LABORATORY TESTING

We tested select samples recovered during drilling activities to determine the following soil characteristics.

TABLE 3.2-1: Laboratory Testing Methods

SOIL CHARACTERISTIC	TESTING METHOD	LOCATION OF RESULTS
Natural Unit Weight and Moisture Content	ASTM D-2216	Appendix A
Plasticity Index	ASTM D-4318	Appendix C
Grain Size Distribution	ASTM D-1140	Appendix C
Constant Rate of Strain Consolidation	ASTM D-4186	Appendix C
Laboratory Miniature Vane Shear	ASTM D-4648	Appendix C
Triaxial Compression – Undrained, Unconsolidated (TXUU)	ASTM-D2850	Appendix C

SOIL CHARACTERISTIC	TESTING METHOD	LOCATION OF RESULTS
Unconfined Compression	ASTM-D2166	Appendix C
Organic Matter of Peat and Other Soils	ASTM D-2974	Appendix C
Sulfate Content	ASTM G-200	Appendix C

The laboratory test results are included in Appendix C. Additional corrosivity tests performed by CERCO Analytical are also included in Appendix C.

3.3 SURFACE CONDITIONS

We performed a brief site reconnaissance on January 29, 2018. We observed that both Residential Areas B and C are relatively level, with surface conditions consisting of seasonal vegetation. We observed a few abandoned piers and small concrete structures within Residential Area C. The southern area of Residential Area C was inundated with water at the time of our site reconnaissance. We also observed several dirt roads and abandoned equipment within Residential Area B. An Alameda County Flood Control (ACFC) canal is present along the boundary of Residential Areas B and C. A wood bridge is also present at the north end of Residential Area C over another ACFC canal.

3.4 SUBSURFACE STRATIGRAPHY

We generally encountered approximately 10 feet of medium stiff to hard lean clay over Young Bay Mud at both Residential Areas B and C. Variable subsurface conditions were encountered at each residential areas and are discussed further in the following sections. We included our exploration logs in Appendix A. The logs contain the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. The logs graphically depict the subsurface conditions encountered at the time of the exploration. Appendix A also provides additional exploratory information in the general notes to the logs.

3.4.1 Residential Area B

Explorations encountered approximately 6 to 7 feet of Young Bay Mud is encountered below the surficial lean clay within Residential Area B where 1-B3 and 1-B4 are drilled. The soil below the Young Bay Mud deposits in the east and west sides of Residential Area B vary. Within the east side where 1-B3 was drilled, Below the Young Bay Mud the explorations encountered lean clay extending to the terminal depth of the boring, with a sandy silt layer seam from approximately 41 to 47 feet below the ground surface (bgs). CEG explorations EB-3, CPT1 and CPT10 encountered a similar subsurface profile within the east side of Residential Area B.

In the west side of Residential Area B, where 1-B4 was drilled, the explorations encountered Young Bay Mud underlain by 15 feet of silt, clayey sand and silty sand extending to a depth of approximately 30 feet bgs. The silty and sandy layer was underlain by stiff lean clay extending to the terminal depth of the boring, with a silt layer between 32 and 40 feet bgs. CEG explorations EB-4, EB-5, EB-6, CPT2, CPT8 and CPT9 encountered a similar subsurface profile within the west side of Residential Area B.

We present an idealized subsurface profile below for design purposes only; variations in the layering and thicknesses should be anticipated and these tables are for design purposes only and are not a basis for differing conditions during construction.

TABLE 3.4.1-1A: Generalized Subsurface Profile for Residential Area B (EAST)

DEPTH (FEET)	USCS SOIL TYPE	DESCRIPTION
0 to 10	CL	Medium stiff to hard, lean Clay
10 to 17	CH	Young Bay Mud – soft, fat Clay
17 to 41	CL	Very stiff, lean clay
41 to 47	SM	Medium Dense, silty sand
Below 47	CL	Very stiff, lean clay

TABLE 3.4.1-1B: Generalized Subsurface Profile for Residential Area B (WEST)

DEPTH (FEET)	USCS SOIL TYPE	DESCRIPTION
0 to 10	CL	Medium stiff to hard, lean Clay
10 to 17	CH	Young Bay Mud – soft, fat Clay
17 to 32	ML, SM, SC	Medium stiff Silt and medium dense silty, clayey sand
32 to 40	ML	Medium stiff Silt
Below 47	CL	Very stiff, lean clay

3.4.2 Residential Area C

The explorations encountered approximately 8 to 9 feet of Young Bay Mud within Residential Area C where 1-B1 and 1-B2 were drilled. A 2 to 4-inche-thick layer of organic silt was encountered at the bottom of the Young Bay Mud layer. Below the organic silt deposits, the explorations encountered approximately 5 to 9 feet of stiff silt, followed by 8 feet of poorly graded, medium dense sand and poorly graded gravel extending to a depth of approximately 46 to 50 feet bgs. Hard lean clay is encountered at depth and extended to the terminal depth of the borings in Residential Area C, at 51.5 feet. CEG explorations EB-1 and CPT3 encountered a similar subsurface profile within Residential Area C

We present a generalized subsurface profile below for design purposes only; variations in the layering and thicknesses should be anticipated and these tables are for design purposes only and are not a basis for differing conditions during construction.

TABLE 3.4.2-1: Generalized Subsurface Profile for Residential Area C

DEPTH (FEET)	USCS SOIL TYPE	DESCRIPTION
0 to 10	CL	Medium stiff to hard, lean Clay
10 to 19	CH	Young Bay Mud – soft, fat Clay
19 to 27	ML to SM	Stiff Silt to Silty Sand
27 to 35	SP	Dense, poorly graded Sand
35 to 46	GP	Dense, poorly graded Gravel
Below 46	CL	Very stiff, lean clay

3.5 GROUNDWATER

We measured the groundwater during drilling in the 1-B1 and 1-B2 borings. Due to the mud rotary drilling method, we did not measure the groundwater depth in borings 1-B3 and 1-B4. We present

the approximate groundwater levels encountered in the recent borings and 2007 CEG explorations in Table 3.5-1.

TABLE 3.5-1: Groundwater Observations

EXPLORATION LOCATION	APPROX. DEPTH TO GROUNDWATER (FEET)
1-B1	7
1-B2	8.5
EB-1	20
EB-2	15
EB-3	4
EB-4	15
EB-5	10
EB-6	11

Fluctuations in groundwater levels should be expected during tidal changes, seasonal changes, or over a period of years because of precipitation changes, perched zones, and changes in drainage patterns, irrigation and other conditions. Groundwater levels at sites close to the San Francisco Bay are usually influenced by tidal changes and may fluctuate throughout the day.

Historical high groundwater level in the area is mapped by the California Geological Survey at approximately 5 feet bgs within Residential Area C and between 0 and 5 feet bgs within Residential Area B. For the purposes of this preliminary report, we assumed groundwater at Elevation 0 feet (NGVD29). Groundwater depths should be confirmed during design-level geotechnical studies.

4.0 GEOLOGIC AND GEOTECHNICAL HAZARDS

Based on the exploration and laboratory test results, the proposed residential development is feasible as planned, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications. Primary geotechnical considerations for the planned improvements are as follows:

- Seismic hazards, including ground shaking and liquefaction
- Consolidation settlement of Young Bay Mud deposits
- Presence of corrosive soils

The design-level report should include additional borings, CPTs and vibrating wire piezometers to provide additional information for preparation of specific recommendations regarding grading, foundation design, and drainage for the proposed development. The design-level exploration will also allow for more detailed evaluation of the geotechnical issues discussed below and afford the opportunity to provide recommendations regarding techniques and procedures to be implemented during construction to mitigate potential geotechnical and geological hazards.

4.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, 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 is considered low to negligible at the site.

4.1.1 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the current CBC requirements, at 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 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 well-designed and well-constructed structures will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

4.1.2 Soil Liquefaction

Liquefaction is a phenomenon in which saturated, cohesionless soil is subject to a temporary, but essentially total, loss of shear strength because of pore pressure buildup under the reversing cyclic shear stresses associated with earthquakes. Maps prepared by the Association of Bay Area Governments (ABAG, 2001) indicate that the site has a high potential for liquefaction.

We performed preliminary liquefaction analyses on the borings based on guidelines provided in the California Geological Survey's Special Publication 117A (2008), as well as analysis methods by Bray and Sancio and Idriss and Boulanger (2008). We used SPT blow counts in the saturated silt, silty sand, and clayey sand layers to calculate the factor of safety against liquefaction. Our analyses consider a peak ground acceleration of 0.594g (PGA_M), Mw of 7.3 and groundwater levels at Elevations 0 feet (NGVD29), which is approximately 5 feet bgs.

We include results of our liquefaction assessment in Appendix D. We list potentially liquefiable layers in Table 4.1.2-1 below.

TABLE 4.1.2-1: Potentially Liquefiable Layers

DEVELOPMENT AREA	EXPLORATION ID	POTENTIALLY LIQUEFIABLE LAYERS
Residential Area B (EAST)	1-B3	Silty deposits between 18 and 23 feet bgs
Residential Area B (WEST)	1-B4	Clayey Sand between 17 and 19 feet bgs Silty Sand between 20 and 30 feet bgs Silty between 35 and 40 feet bgs

DEVELOPMENT AREA	EXPLORATION ID	POTENTIALLY LIQUEFIABLE LAYERS
Residential Area C	1-B1	Silt between 20 and 28 feet bgs
	1-B2	Silt between 18 and 23 feet bgs

Our analysis suggests that interbedded layers of silt, silty sand, and clayey sand below 16 feet bgs are susceptible to liquefaction and cyclic softening. The liquefiable layers do not appear to be in continuous layers throughout the site.

4.1.3 Liquefaction-Induced Ground Settlement

We calculated potential liquefaction-induced settlement estimates based on methods published by Idriss and Boulanger (2008) and soils encountered in Borings 1-B1, 1-B2, 1-B3 and 1-B4. In addition, we summarize liquefaction-induced settlement presented in the 2007 Cornerstone report in Table 4.1.3-1 below. Cornerstone considered a PGA of 0.53g, Mw of 7.3 and groundwater at 5 feet below ground surface.

TABLE 4.1.3-1: Preliminary Liquefaction Induced Settlement

DEVELOPMENT AREA	EXPLORATION ID	PRELIMINARY LIQUEFACTION INDUCED SETTLEMENT (INCHES)
Residential Area B (EAST)	1-B3	1.1
	CPT1*	1.5
Residential Area B (WEST)	1-B4	5.2
	CPT2*	2
	CPT8*	1
	CPT9*	0.5
	CPT10*	1
Residential Area C	1-B1	1.4
	1-B2	0.8
	CPT3*	4

*Results per Cornerstone calculation in their report from 2007

4.1.4 Earthquake-Induced Densification

Densification of granular soil above the groundwater level can cause settlement during an earthquake. Since the deposits encountered above groundwater at the site are generally cohesive, the potential for densification of granular soil above groundwater due to an earthquake is low.

4.1.5 Surface manifestation

In addition to the above preliminary liquefaction analysis, we also evaluated the capping effect of any overlying non-liquefiable soil. We identified potentially liquefiable soil as shallow as 17 feet bgs and the layers ranged from 5 to 17-feet thick. We understand that site grade will be raised by 5 to 12 feet and the thickest liquefiable layers are located in the west side of Residential Area B where the thickest engineered fill will be placed to raise site grade.

In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a force sufficient to break through the overlying soil and vent to the surface resulting in sand boils or fissures. We based our analyses and review on guidelines provided by Ishihara (1985) and Youd and Garris (1995). We performed our assessment considering planned final grade conditions. Based on our preliminary analyses and review, the potentially liquefiable soil at the project site have marginally adequate cap of non-liquefiable soil when site grades are raised to prevent venting of liquefiable soil. Soil conditions indicate a potential risk for liquefaction-induced surface rupture or sand boils during a strong seismic event, particularly in the vicinity of Boring 1-B4.

Additional subsurface exploration and collection of soil samples during the design-level study will better delineate the areas with a potential for liquefaction and estimates of liquefaction-induced settlement magnitude. Soil samples of the fine-grained soil should be selected for cyclic simple shear testing during the design-level study to further evaluate liquefaction potential and volumetric strain.

4.1.6 Lateral Spreading

Lateral spreading involves lateral ground movement caused by earthquake vibrations. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied sand or weak soil.

As discussed above, potentially liquefiable deposits are present at a depth of 17 feet bgs. The Mowry Slough and ACFC canal within the project site generally ranged from 5 to 10-feet deep. Lateral spreading of potentially liquefiable layers into free faces along channels is unlikely.

4.2 COMPRESSIBLE BAY MUD

The site is underlain by highly compressible normally consolidated Young Bay Mud ranging from approximately 6 to 9 feet thick, which extends from approximately 10 feet to 16 feet bgs. Based on laboratory consolidation test results, Young Bay Mud within the project site is generally over consolidated by 50 to 100 percent.

We anticipate the Young Bay Mud will experience consolidation settlement from the weight of new fill and new building loads supported shallow foundations. The design of surface grades, underground utilities, and all structures and improvements must accommodate or resist potential differential settlement. We performed preliminary consolidation settlement calculations considering up to 12 feet of new fill. Our analysis considered an over consolidation ratio (OCR) of 1.5.

TABLE 4.2-1: Consolidation Settlement in 30 years

PROPOSED IMPROVEMENTS	LOAD (PSF)	AREA B TOTAL SETTLEMENT IN 30 YEARS (INCHES)	AREA C TOTAL SETTLEMENT IN 30 YEARS (INCHES)
1 foot of Fill	120	0.75	0.6
2 feet of Fill	240	1	0.8
4 feet of Fill	480	2	1.5
6 feet of Fill	720	3	2.0
8 feet of Fill	960	4	3

PROPOSED IMPROVEMENTS	LOAD (PSF)	AREA B TOTAL SETTLEMENT IN 30 YEARS (INCHES)	AREA C TOTAL SETTLEMENT IN 30 YEARS (INCHES)
10 feet of Fill	1080	4.5	3.5
12 feet of Fill	1500	5.5	4

To mitigate this potential settlement, we recommend implementation of a surcharge program for the single-family structures and deep foundations for heavy structures such as bridges and tall retaining walls. We provide preliminary recommendations for these mitigations in Section 5.

4.3 EXPANSIVE SOIL

We collected near-surface soil samples during the field exploration that consist of low to moderately expansive clay. We understand that site grades will be raised to achieve finished grade. Moderately expansive native and imported soil is susceptible to shrink and swell that could affect structures and other improvements constructed on the ground (such as pavement and flatwork).

Some of the mitigation measures typically used to reduce adverse effects of expansive soil for projects such as this may include one or several of the following:

- Using a rigid mat foundation, such as post-tensioned mat foundations, designed to resist potential movement;
- Deepening foundations to below the zone of moisture fluctuation, such as using deep embedded footings, drilled piers, or piles; and
- Importing low expansive import fill or use of lime treatment of the soil below the building combined with the use of shallow foundations.

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 is extremely difficult to re-moisturize clayey soil without excavation, moisture conditioning, and re-compaction.

4.4 SOIL CORROSION

As part of this study, we obtained a representative soil sample and submitted to CERCO analytical laboratory for determination of pH, resistivity, sulfate, and chloride. The test results are included in Appendix C and summarized in the table below.

TABLE 4.4-1: Soil Corrosivity Test Results

SAMPLE NUMBER	SOIL TYPE	SULFATE (MG/KG)	CHLORIDE (MG/KG)	RESISTIVITY (OHMS-CM)	PH	REDOX (MV)
1-B4 @ 3’	Clay	190	1,100	150	8.01	220

* ASTM D4327

Severely corrosive soil is also present in the nearby Area 3 project area. Additional corrosivity tests should be performed during design level exploration to determine levels for concrete and steel protection.

5.0 PRELIMINARY RECOMMENDATIONS

5.1 PRELIMINARY GRADING CONSIDERATIONS

5.1.1 Demolition and Site Clearing

The site should be cleared of vegetation, obstructions, including existing abandoned concrete structures and buried utilities. Existing underground utilities at the site should be identified and either properly abandoned or relocated. If loose and deleterious materials are present, the materials should be removed and the area should be backfilled with engineered fill.

5.1.2 Selection of Materials

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, high organic content soil (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils (if any), we anticipate the site soil is suitable for use as engineered fill. Other material and debris, including trees with their root balls, should be removed from the project site. Imported soil should have a PI less than 20, with more than 70 percent passing the No. 200 sieve and less corrosive than site soils.

5.1.3 Fill Compaction

For land planning and cost estimating purposes, the following compaction control requirements should be anticipated for general fill areas and utility trench backfill:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 3 percentage points above optimum moisture content.
Minimum Relative Compaction:	Relative compaction 90 percent.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material. In the event that imported fill material is characterized and following the design level geotechnical report, the recommendations may change with respect to the soil type.

5.1.4 Graded Slope

Graded slopes along the perimeter of the site are currently planned with an inclination of 2:1 (horizontal:vertical). For preliminary planning, graded slope up to 6 feet high can constructed at an inclination of 2:1. For higher slope, geogrid reinforcement may be needed to construct a stable 2:1 slope. Improvements located at the tops of slopes could be subject to minor lateral movements associated with soil creep.

5.2 PRELIMINARY FOUNDATION CONSIDERATIONS

Major considerations in foundation design for this project are:

- Long-term settlement of Young Bay Mud deposits.
- Liquefaction induced settlement.

In our opinion, the most effective foundation system for the proposed residential structures is shallow foundations with consideration of specific ground improvement measures. Deep foundation (driven piles or drilled piers) system should be considered for heavily loaded structures such as the bridges and retaining walls above 6 feet high.

TABLE 5.2-1: Foundation Design Options

FOUNDATION OPTION	FOUNDATION DESCRIPTION	GROUND IMPROVEMENT OPTION	SUITABLE BUILDING TYPE
A	Shallow Foundation (mat foundation slab)	Surcharge Program	Single family buildings
B	Deep Foundation (driven piles)	Not required	Bridges and retaining wall exceeding 6 feet high

5.3 SHALLOW FOUNDATIONS

Shallow foundations such as post-tensioned mats may be considered for supporting the two to three story single-family homes after completion of a surcharge program. For preliminary planning purposes, an allowable foundation bearing capacity of 1,000 psf can be used for shallow foundations design against dead plus live load combinations. The shallow foundation systems may be required to withstand differential settlement ranging from ½ to 2½ inches cause by liquefaction. The total amount of liquefaction-induced settlement will be refined in the design-level report.

5.3.1 Surcharge and Wick Drains

Preconsolidation of the Young Bay Mud deposits prior to site development with a surcharge program can reduce post-construction settlement due to structural load and new fill placed to raise site grades, Surcharging, if properly performed, will also reduce any down-drag loading on deep foundations.

A surcharge program would involve the placement of temporary fill, uniformly blanketing development areas until the Young Bay Mud deposits has been further consolidated as determined by a site-specific settlement monitoring program. The height of the required surcharge fill is dependent on the total anticipated civil fill loads and the construction schedule. Wick drains can be used to increase the rate of consolidation if the durations shown in Table 5.3.1-1 are not consistent with project schedule.

Estimated surcharge heights, duration and total settlement for the project are presented in Table 5.3.1-1 for preliminary project planning and budgeting purposes. We assume site grades will be raise by a maximum of 12 feet to achieve finished grades.

TABLE 5.3.1-1: Preliminary Load-induced Settlement

HEIGHT OF CIVIL FILL	LOAD FROM PROPOSED FILL	TOTAL CONSOLIDATION SETTLEMENT IN 30 YEARS WITHOUT SURCHARGE *	TIME NEEDED TO REDUCE RESIDUAL SETTLEMENT TO 1/2 INCH OR LESS
1	125 psf	0.75 inches	3 months
2	250 psf	1 inches	3 months
4	500 psf	2 inches	12 months
6	750 psf	3 inches	12 months
8	1000 psf	4 inches	18 months

HEIGHT OF CIVIL FILL	LOAD FROM PROPOSED FILL	TOTAL CONSOLIDATION SETTLEMENT IN 30 YEARS WITHOUT SURCHARGE *	TIME NEEDED TO REDUCE RESIDUAL SETTLEMENT TO 1/2 INCH OR LESS
10	1250 psf	4.5 inches	18 months
12	1500 psf	5.5 inches	24 months

*Total settlement presented include 350 psf of building load.

The surcharge height, surcharge duration and wick drain spacing presented in Table 5.3.1-2 may be considered to shorten the time needed to reduce residual settlement to less than ½ inch.

TABLE 5.3.1-2: Preliminary Surcharge Duration

HEIGHT OF CIVIL FILL	TOTAL CONSOLIDATION SETTLEMENT WITHOUT SURCHARGE PROGRAM (INCHES)	SURCHARGE PROGRAM*		
		SURCHARGE HEIGHT (FEET)	SURCHARGE TIME (MONTHS)	WICK DRAINS SPACING (FEET ON CENTER)
4	2	10	3	No wick drains
		8	2	5
6	3	10	4	No wick drains
		8	6	5
8	4	12	6	No wick drains
		12	3	5
10	4.5	15	6	No wicks
		12	6	5
12	5.5	15	6	5
		12	6	3

*Surcharge program is intended to reduce post-construction settlement to less than ½ inch.

5.3.1.1 Lightweight Fill

In areas where a surcharge program and associated settlement may conflict with off-site improvement, such as along the northeast boundary where the railroad is located, consolidation settlement can be mitigated by removing portions of the existing site soil and replacing them with lightweight fill. The most cost-effective type of lightweight fill for this site is likely cellular concrete. Cellular concrete is a mixture of cement, water, and a proprietary foaming agent. The typical total unit weight of cellular concrete ranges from 20 to 40 pounds per cubic foot (pcf).

5.3.2 Deep Foundations

A foundation system bearing on competent soil below the Young Bay Mud and medium dense sand can be considered for bridges and retaining walls within the project site. Based on our experience, driven precast pre-stressed concrete piles, cast-in-drill-hole (CIDH) piles or steel H-piles may be used to support these heavier structures. Steel piles may not be economical due to the current cost of steel and cathodic protection or sacrificial thickness necessary due to the corrosive soil environment. Pile lengths are highly dependent on the structural loads, the thickness of the Young Bay Mud and variation of driving resistance in the underlying supporting alluvium soil. The piles will derive their vertical capacity primarily from skin friction within the stiff soil layers below the existing fill, Young Bay Mud, and medium dense sand. Typically pile embedment depths in similar subsurface condition ranges from 50 to 70 feet deep for a 50-ton

capacity. A separate geotechnical exploration is currently underway to provide design-level foundation recommendations for the Stevenson Boulevard extension bridge.

5.4 FLEXIBLE PAVEMENT DESIGN

We developed the following preliminary pavement sections for Traffic Indices ranging from 5 to 7, an assumed R-value of 5, and in accordance with the design methods contained in Topic 630 of Caltrans Highway Design Manual.

TABLE 5.4-1: Preliminary Pavement Sections

TRAFFIC INDEX	AC (INCHES)	AB (INCHES)
5.0	3.0	10.0
6.0	3.5	12.5
7.0	4.0	16.0

Note: AC – Asphalt Concrete
AB – Caltrans Class 2 aggregate base (R-value of 78 or greater)

We provide the pavement sections above for estimating only. Once the source of fill has been identified, R-value testing can be performed to optimize the pavement design. We recommend the Traffic Index and minimum pavement section(s) be confirmed by the Civil Engineer and the City of San Mateo.

6.0 FUTURE GEOTECHNICAL STUDIES

This report presents preliminary geotechnical findings, conclusions, and recommendations intended for preliminary planning purposes only. A design-level geotechnical exploration and assessment should be performed when development plans are finalized. The design-level exploration should further evaluate the following:

- The potential for liquefaction and volumetric strain.
- The extent of the shallow silty soil within Residential Area B.
- The thickness of compressible soil in proposed development area not explored in this study.
- Specific recommendations for site grading, ground improvement, and the design and construction of foundations and utilities.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements for Residential Areas B and C within Sobrato Property Area 4 in Newark, California. 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 geotechnical engineering principles and practices currently employed in the area; no warranty is expressed 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 or provide insurance; 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 is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, stockpiled material, 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, notify ENGEO 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, notify the proper regulatory officials immediately.

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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 onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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

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FIGURES

FIGURE 1: Vicinity Map
FIGURE 2: Site Plan
FIGURE 3: Regional Geologic Map
FIGURE 4: Regional Faulting and Seismicity Map

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N

0 FEET 2000

0 METERS 1000

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

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VICINITY MAP SOBRANTO PROPERTY - AREA 4, PHASE I NEWARK, CALIFORNIA	PROJECT NO.: 12165.000.001	FIGURE NO.
	SCALE: AS SHOWN	1
	DRAWN BY: LL / CHECKED BY: JK	

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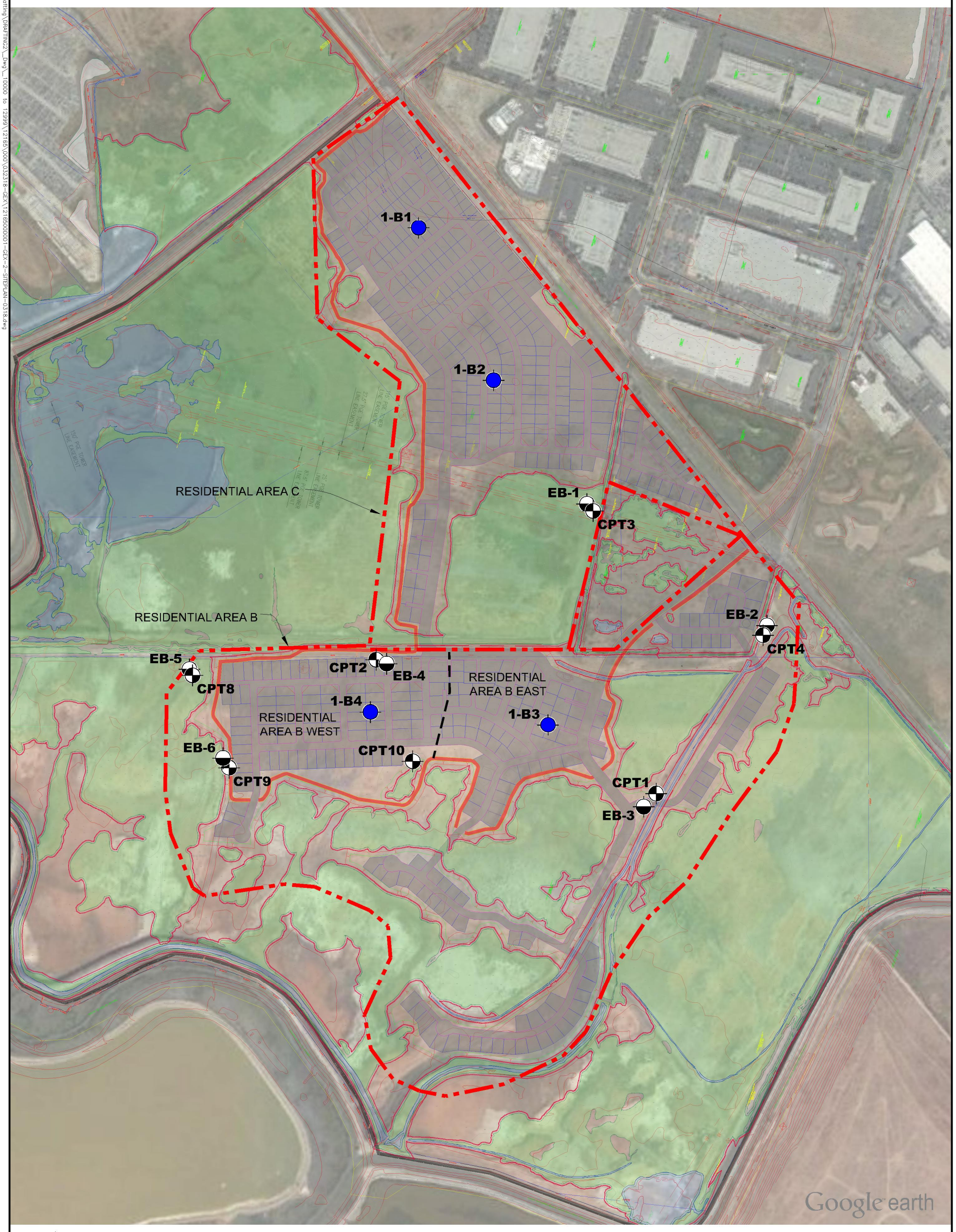

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EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

- 1-B4 — BORING (ENGEO, 2018)
- EB-6 — BORING (CORNERSTONE, 2007-2008)
- CPT10 — CONE PENETRATION TEST (CORNERSTONE, 2007-2008)

BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE AND CBG, 2018

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SITE PLAN
SOBRATO PROPERTY - AREA 4, PHASE I
NEWARK, CALIFORNIA

PROJECT NO.: 12165.000.001	FIGURE NO. 2
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: JK

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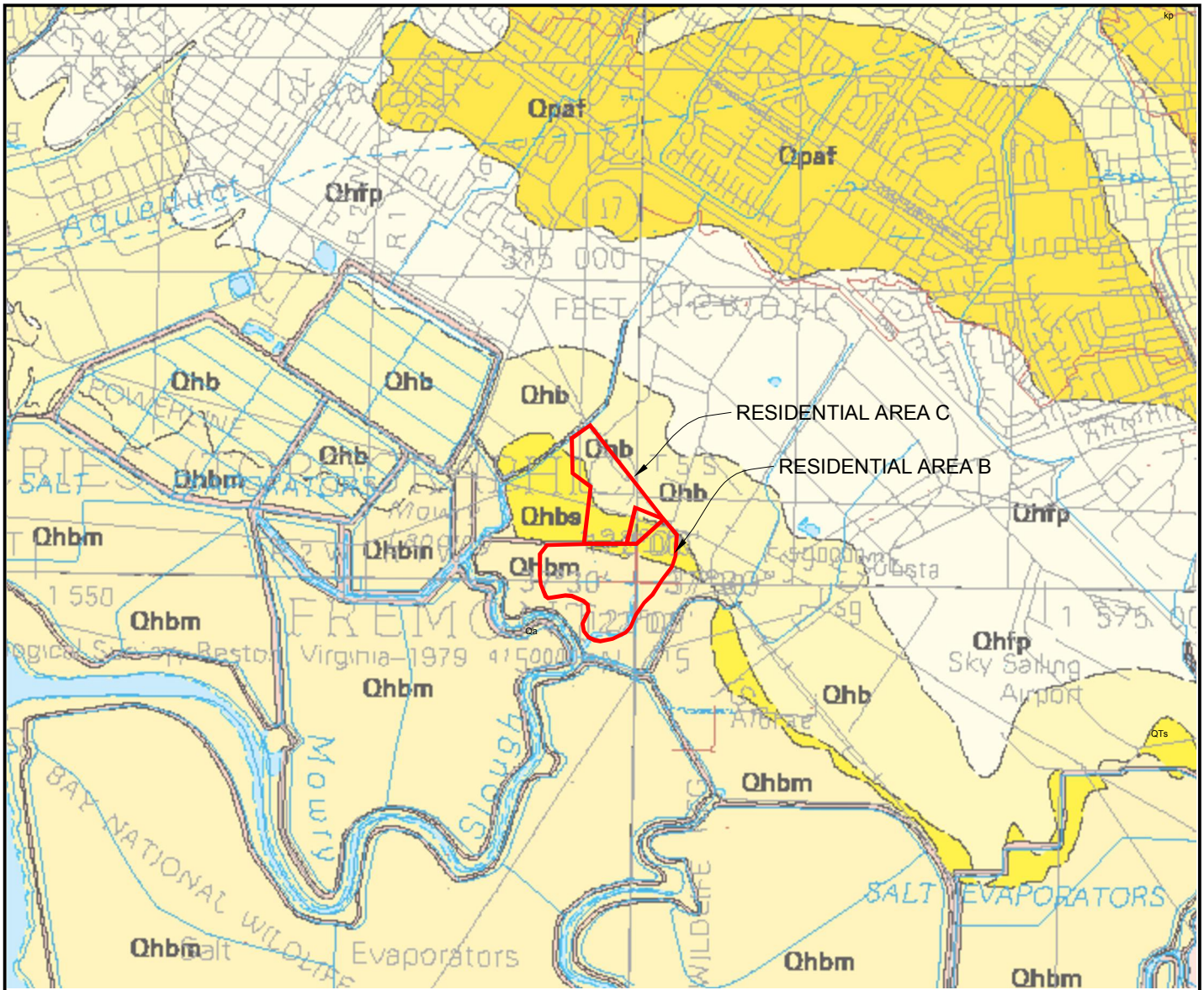

EXPLANATION

GEOLOGIC CONTACT-DASHED WHERE GRADATIONAL OR APPROXIMATELY LOCATED

FAULT-DASHED WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE EXISTENCE IS DOUBTFUL. SAWTEETH ARE ON UPPER PLATE OF LOW ANGLE THRUST FAULT.

STRIKE AND DIP OF STRATA

INCLINED VERTICAL OVERTURNED

Qhb BASIN DEPOSITS (HOLOCENE)

Qhbs SALT AFFECTED BASIN DEPOSITS (HOLOCENE)

Qhbm BAY MUD DEPOSITS (HOLOCENE)

Qhfp FLOODPLAIN DEPOSITS (HOLOCENE)

Qpaf ALLUVIAL FAN DEPOSITS (PLEISTOCENE)

N

0 FEET 4000

0 METERS 2000

BASE MAP SOURCE: GRAYMER, 1997

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REGIONAL GEOLOGIC MAP

SOBRATO PROPERTY - AREA 4, PHASE I

NEWARK, CALIFORNIA

PROJECT NO.: 12165.000.001	FIGURE NO. 3
SCALE: AS SHOWN	
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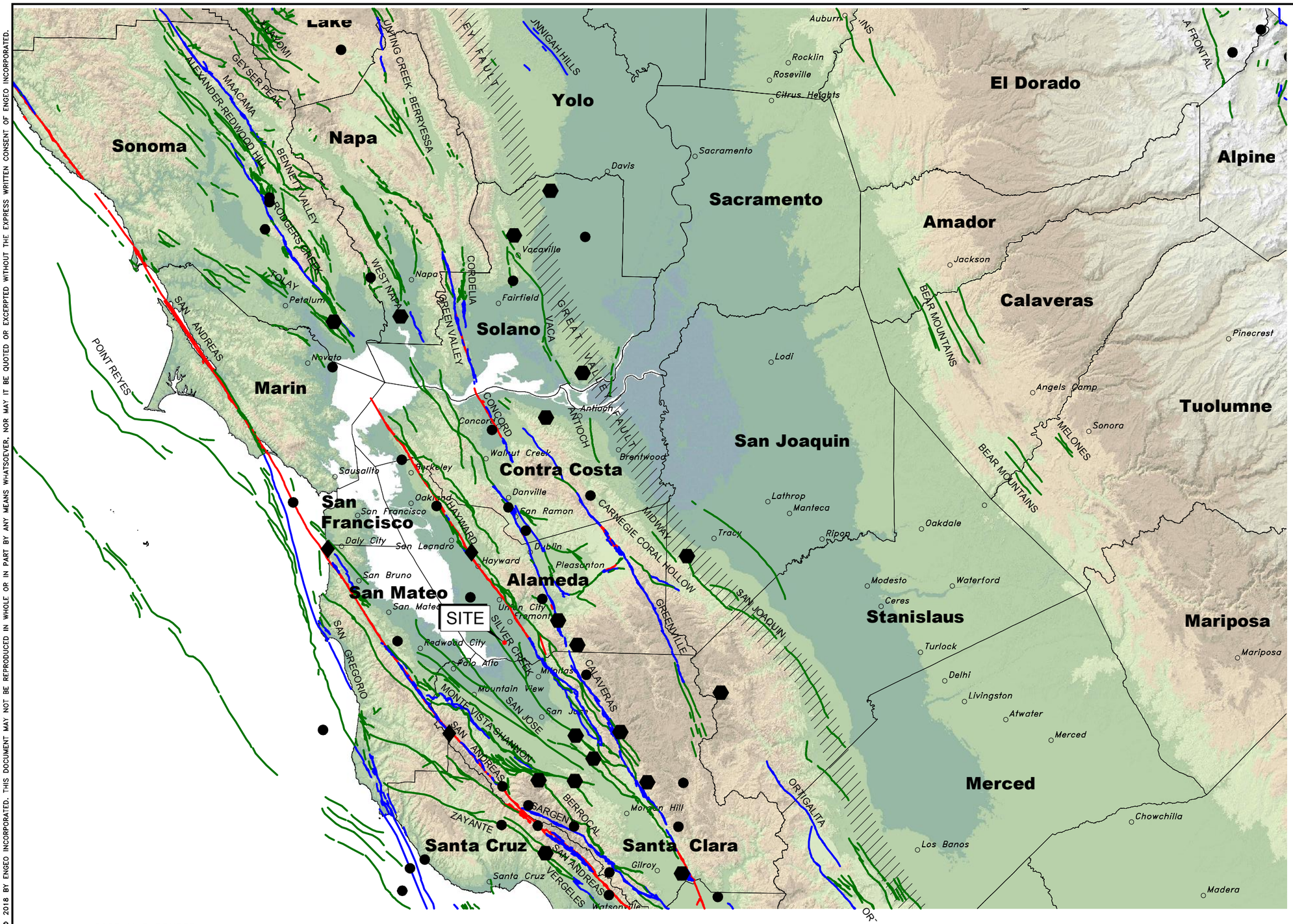

BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

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REGIONAL FAULTING AND SEISMICITY
SOBRATO PROPERTY - AREA 4, PHASE I
NEWARK, CALIFORNIA

PROJECT NO.: 12165.000.001	FIGURE NO. 4
SCALE: AS SHOWN	
DRAWN BY: LL	CHECKED BY: JK

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APPENDIX A

EXPLORATION LOGS
(ENGEO, 2018)

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

CONSISTENCY

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

SAMPLER SYMBOLS

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

MOISTURE CONDITION

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

LINE TYPES

Solid - Layer Break

Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

Groundwater level during drilling

Stabilized groundwater level

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

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

ENGEO
Expect Excellence

LOG OF BORING 1-B1

ENGEO Expect Excellence

LATITUDE: 37.50871939 LONGITUDE: -122.00490827

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

DATE DRILLED: 2/15/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 5 ft.

LOGGED / REVIEWED BY: B. Hassett / JAF
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), dark brown, stiff, moist, rootlets and organics												
			LEAN CLAY (CL), gray, hard, moist, 5-15% coarse grained sand, trace angular gravel, no organics			32								+4.5*	PP
			Gray mottled with yellowish brown, increase in silt content, trace sand												
5	0					15								2*	PP
			Gray mottled with dark brown, lense of clayey fine grained sand		▽										
			Medium stiff, grades siltier			10					34	91	926	0.75*	UU PP
10	-5		FAT CLAY (CH), grayish green, soft, wet [YOUNG BAY MUD]			7					34	89		0.5*	PP
			Soft, medium to high plasticity												
15	-10										48	71			
													340*		TV
			SILT (ML), grayish green, stiff, wet, some fine grained sand								35	90			
20	-15														

LOG OF BORING 1-B1

ENGEO — Expect Excellence

LATITUDE: 37.50871939 LONGITUDE: -122.00490827

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

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HAMMER TYPE: 140 lb. Auto Trip

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			SILT (ML), grayish green, stiff, wet, some fine grained sand										863		LVS
														1*	PP
25	-20						26	23	3	94	27	100			
			POORLY GRADED SAND (SP), olive brown, medium dense, wet, medium grained sand, less than 5% fines, 5-15% subrounded gravel												
30	-25					30									
						22									
			Increase in gravel content, taking of some rotary wash fluid												
35	-30		CLAYEY GRAVEL (GC), medium dense, wet, fine to coarse grained sand, 15-20% fines content, 5-10% sand			20									
			POORLY GRADED GRAVEL (GP), medium dense, wet, subangular to subrouned, with medium to coarse grained sand												
40	-35					28									

LOG OF BORING 1-B1

ENGEO Expect Excellence

LATITUDE: 37.50871939 LONGITUDE: -122.00490827

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

DATE DRILLED: 2/15/2018
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HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			POORLY GRADED GRAVEL (GP), medium dense, wet, subangular to subrounded, with medium to coarse grained sand 30-40% sand, 5-10% fines			28				6					
45	-40		Dense			32									
50	-45		Coarse gravel			24									
			CLAYEY SAND (SC), olive brown, medium dense, wet, fine to medium grained sands, 10-15% fines content												
			Boring terminated at an approximate depth of 51½ feet below existing grade. Groundwater was encountered at the time of drilling at approximately 7 feet below existing grade.												

LOG OF BORING 1-B2

ENGEO — Expect Excellence

LATITUDE: 37.50656875 LONGITUDE: -122.0033853

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

DATE DRILLED: 2/15/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 5 ft.

LOGGED / REVIEWED BY: B. Hassett / JAF
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), dark brown, stiff, moist, rootlets and organics, trace sand												
			LEAN CLAY (CL), gray mottled with yellowish brown, medium stiff, moist, 30-35% fine grained sand, less than 5% gravel			26	38	20	18		29	94		0.64 3.5*	UC PP
5	0					14									
			Stiff												
					▽	11									
			Gray mottled with dark brown, wet, 5-10% sand												
10	-5		FAT CLAY (CH), grayish green, soft, wet [YOUNG BAY MUD]			8								1*	PP
			Grayish green mottled with yellowish brown, medium plasticity								39	83	1015		UU
15	-10										54	69			
			ORGANIC SILT (OH), dark brownish black, soft, wet				329	249	80		202				
			SILT (ML), grayish green, medium stiff, wet, some fine grained sand												
20	-15										23	105			

LOG OF BORING 1-B2

ENGEO
Expect Excellence

LATITUDE: 37.50656875 LONGITUDE: -122.0033853

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

DATE DRILLED: 2/15/2018
HOLE DEPTH: Approx. 51½ ft.
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DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SILT (ML), grayish green, medium stiff, wet, some fine grained sand							90					
			POORLY GRADED SAND (SP), brown, medium dense, wet, fine to medium grained sand, 5-10% clay fines, some clayey sand seams												
25	-20		with 30-40% subrounded gravel, dense			51									
						40									
30	-25					25									
			POORLY GRADED GRAVEL (GP), brown, dense, wet, subrounded, fine to coarse grained sand												
35	-30					42									
40	-35					30									

LOG OF BORING 1-B2

ENGEO Expect Excellence

LATITUDE: 37.50656875 LONGITUDE: -122.0033853

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

DATE DRILLED: 2/15/2018
HOLE DEPTH: Approx. 51½ ft.
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DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			POORLY GRADED GRAVEL (GP), brown, dense, wet, subrounded, fine to coarse grained sand Sand seam, medium grained sand												
45	-40					34									
			LEAN CLAY (CL), pale olive mottled with grayish brown, very stiff, wet												
50	-45					30									
			Boring terminated at an approximate depth of 51½ feet below existing grade. Groundwater was encountered at the time of drilling at approximately 8½ feet below existing grade.												

LOG OF BORING 1-B3

ENGEO Expect Excellence

LATITUDE: 37.501384 LONGITUDE: -122.002189

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

DATE DRILLED: 2/28/2018
HOLE DEPTH: Approx. 54 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 1 ft.

LOGGED / REVIEWED BY: B. Hassett / JAF
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), dark brown, moist, with organics and rootlets, <5% sand												
	0		LEAN CLAY (CL), pale olive mottled with gray, stiff, moist, <5% gravel												
5						12					27	99		1.4	UC
	-5														
10			FAT CLAY (CH), grayish green, medium stiff, moist, <5% sand [YOUNG BAY MUD]			6									
	-10		Soft										500*		TV
											30	93	601		UU
											36	87			
			Dark gray organic layer from 14.5 to 15 feet										800*		TV
15															
	-15														
			LEAN CLAY (CL), pale olive, medium stiff, moist, 5 to 10% sand, <5% gravel												
			No recovery												
20															

LOG OF BORING 1-B3

ENGEO Expect Excellence

LATITUDE: 37.501384 LONGITUDE: -122.002189

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

DATE DRILLED: 2/28/2018
HOLE DEPTH: Approx. 54 ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 1 ft.

LOGGED / REVIEWED BY: B. Hassett / JAF
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), pale olive, medium stiff, moist, 5 to 10% sand, <5% gravel												
	-20												2085		LVS
										87	22	102			
													700*		TV
25															
	-25														
			Mottled with light yellowish brown, stiff			22					20	112	1600*		TV
30															
	-30														
			Medium stiff			13					27	100	480*		TV
35															
	-35														
40															

LOG OF BORING 1-B3

ENGEO Expect Excellence

LATITUDE: 37.501384 LONGITUDE: -122.002189

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

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DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), pale olive, medium stiff, moist, 5 to 10% sand, <5% gravel												
	-40		SANDY SILT (ML), pale olive, medium stiff, moist												
			Silty, with fine-grained sand			12					28	98		1*	PP
45															
	-45														
			LEAN CLAY (CL), pale olive, medium stiff, moist, 5 to 10% sand												
50															
	-50		Brown, very stiff			38								3*	PP
						25									
			Boring terminated at an approximate depth of 54 feet below existing grade. No groundwater data was obtained.												

LOG OF BORING 1-B4

ENGEO — Expect Excellence

LATITUDE: 37.501511 LONGITUDE: -122.005466

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

DATE DRILLED: 2/27/2018
HOLE DEPTH: Approx. 51½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 1 ft.

LOGGED / REVIEWED BY: B. Hassett / JAF
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), dark brown, moist, with organics and seasonal grass												
	0		LEAN CLAY (CL), light brown, stiff, moist, <5% sand												
						14								1.5*	PP
5															
	-5		Grades to gray mottled with light yellowish brown			12					28	97		2*	PP
						7					32	92		0.67	UC
10			Medium stiff FAT CLAY (CH), grayish green, soft, wet, <5% sand [YOUNG BAY MUD]												
	-10					6					37	87		0.5*	PP
15															
	-15		SILT WITH SAND (ML), grayish green, medium stiff, wet, 20-25% sand				28	20	8	80	27	100			
			CLAYEY SAND (SC), grayish green, wet												
20															

LOG OF BORING 1-B4

ENGEO
Expect Excellence

LATITUDE: 37.501511 LONGITUDE: -122.005466

Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
12165.000.002

DATE DRILLED: 2/27/2018
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Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-20		SILTY SAND (SM), grayish green mottled with brown, medium dense, wet, 40-45% fines								19	111			
25	-25					21				40					
						15									
30			Medium-grained subrounded gravel from 30 to 30.5 feet												
	-30		LEAN CLAY (CL), light yellowish brown, very stiff, moist, 10-15% sand, 5-10% subrounded gravel			26									
35	-35		SILT (ML), light yellowish brown, stiff, wet, 10-15% sand			18	30	21	9		25	103		1.5*	PP
40															

LOG OF BORING 1-B4

ENGEO Expect Excellence

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Geotechnical Exploration
Sobrato Area 4 - Phase 1
Newark, CA
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			LEAN CLAY (CL), light yellowish brown mottled with yellowish red, stiff, moist, <5% sand												
	-40					19								1*	PP
45															
	-45					24					27	100		1.5*	PP
50															
	-50		Grades to gray, very stiff			35								2.5*	PP
			Boring terminated at an approximate depth of 51½ feet below existing grade. No groundwater data was obtained.												

APPENDIX B

Previous Exploration Logs (Cornerstone Earth Group, 2009)

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Areas 3 and 4 EIR Study
PROJECT NUMBER 118-3-3
PROJECT LOCATION Newark, Stevenson Boulevard
DATE STARTED 7/23/07 DATE COMPLETED 7/23/07
GROUND ELEVATION 6 FT +/- BORING DEPTH 45 ft.
DRILLING CONTRACTOR EGI
LATITUDE 37°30'17.838" LONGITUDE -123°59'54.564"
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY JRD
AT TIME OF DRILLING 20 ft.
NOTES
AT END OF DRILLING Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
6.0	0	**Silty Clay (CH)** very stiff, moist, dark gray, trace fine sand, some light gray mottling	21	MC-1	101	23		
			21	MC-2B	104	21		
		color change to mottled brown and gray						
1.0	5	**Silty Clay (CL)** stiff, moist, blue-gray and brown, trace roots, moderate plasticity	13	MC-3B	93	29		
	10		7	MC-4	80	40		
	15		9	MC-5A	85	33		
-14.0	20		13	MC-6A	95	27		
		Silty Sand (SM) loose to medium dense, wet, gray, fine to medium sand	8	SPT-7		25		48
	25		26	SPT-8		25		31
			16	NR				
	30	with occasional sandy silt seams, approximately 1 to 2 inches thick	11	SPT-9		23		25
-26.0		**Silty Gravel (GM)** dense, wet, gray, subrounded	36	SPT-10		8		8
-29.0	35							

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

Continued Next Page

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:14 - P:\DRAFTING\GINT FILES\118-3-3 NEWARK AREAS 3 AND 4 EIR STUDY.GPJ

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) 1.0 2.0 3.0 4.0 5.0
-29.0	35		**Silty Gravel (GM)** dense, wet, gray, subrounded							
				17	SPT-11		9		9	
	40			39	SPT-12					
-36.5			**Silty Clay (CL)** very stiff, moist, brown and gray mottled, moderate plasticity	41	MC-13					
-39.0	45		Bottom of Boring at 45.0 feet.							
	50									
	55									
	60									
	65									
	70									
	75									

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:14 - P:\DRAFTING\GINT FILES\118-3-3 NEWARK AREAS 3 AND 4 EIR STUDY.GPJ

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Areas 3 and 4 EIR Study
PROJECT NUMBER 118-3-3
PROJECT LOCATION Newark, Stevenson Boulevard
DATE STARTED 7/24/07 **DATE COMPLETED** 7/24/07
GROUND ELEVATION 5 FT +/- **BORING DEPTH** 40 ft.
DRILLING CONTRACTOR EGI
LATITUDE 37°30'11.8584" **LONGITUDE** -122°0'6.9012"
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY JRD
AT TIME OF DRILLING 15 ft.
NOTES
AT END OF DRILLING Not Encountered

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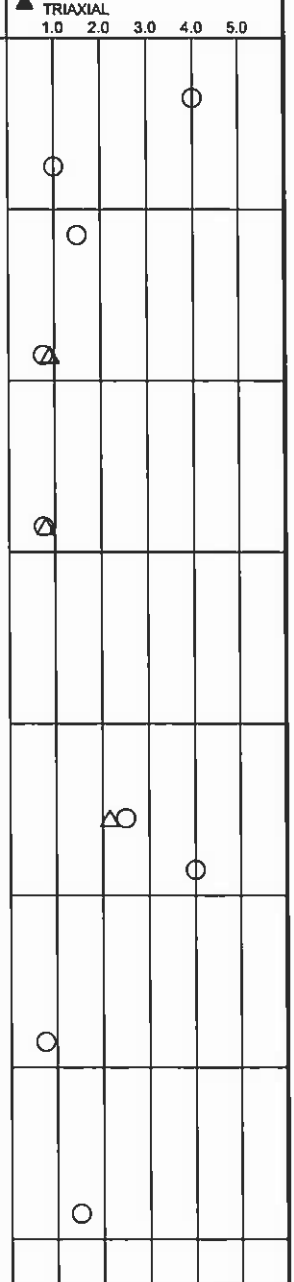
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
5.0	0	**Silty Clay (CH)** stiff, moist, black, trace fine sand, high plasticity, upper 12 inches loose	25	MC-1				
2.5		**Silty Clay (CH)** stiff, moist, dark gray , moderate to high plasticity, some carbonate nodules	14	MC-2A	100	25		
0.0	5	**Silty Clay (CL)** medium stiff, moist, olive-brown, with gray mottles, trace fine sand, moderate plasticity	9	MC-3B	64	87		
			8	MC-4A	87	32		
	10	color change to olive-gray, some brown mottles						
		Silty Clay (CH) soft to medium stiff, moist, blue-gray, trace roots	9	MC-5	88	34		
-10.0	15	**Sandy Silt (ML)** soft, wet, blue-gray, fine to medium sand	2	MC-6A	101	24	9	72
			4	MC-7A	109	21	8	63
	20							
-16.0		**Silty Clay (CH)** very stiff, wet, blue-gray with brown mottles, some fine sand, moderate plasticity	27	MC-8A	111	19		
			25	MC-9A				
	25							
-22.0		**Silty Clay (CL)** medium stiff to stiff, wet, brown, some fine sand	11	MC-10A	109	20		
	30							
			19	MC-11				
-30.0	35							

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



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BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) 1.0 2.0 3.0 4.0 5.0
-30.0	35		**Silty Clay (CL)** medium stiff to stiff, wet, brown, some fine sand							
			(no recovery with MC, sampled with SPT to recover disturbed sample)	19	NR SPT-12					
-35.0	40		Bottom of Boring at 40.0 feet.							
	45									
	50									
	55									
	60									
	65									
	70									
	75									

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BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Areas 3 and 4 EIR Study
PROJECT NUMBER 118-3-3
PROJECT LOCATION Newark, Stevenson Boulevard
DATE STARTED 7/23/07 DATE COMPLETED 7/23/07
GROUND ELEVATION 2 FT +/- BORING DEPTH 40 ft.
DRILLING CONTRACTOR EGI
LATITUDE 37°29'58.7364" LONGITUDE -123°59'57.3108"
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY JRD
AT TIME OF DRILLING 15 ft.
NOTES
AT END OF DRILLING 4 ft.

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
2.0	0	**Silty Clay (CL) [Fill]** stiff, moist, brown-gray, trace sand	22	MC-1				
-1.0		**Silty Clay (CL)** stiff, moist, dark gray, high plasticity, some carbonate nodules	14	MC-2A	83	30		
	5		10	MC-3A	122	25		
		color change to light gray with brown mottles						
-6.0		**Silty Clay (CH)** medium stiff to stiff, moist, blue-gray, some brown mottling, trace organics	6	MC-4A	94	29		
	10			ST-5				
		clayey sand seam from 12 1/2 feet to 13 feet	5	MC-6B	92	28		
	15			ST-7				
	20		15	MC-8A	105	23	18	
-19.5		**Silty Clay (CL)** stiff, moist, brown, trace fine sand, moderate plasticity	22	MC-9A				
	25							
	30		16	MC-10A	108	20		
		color change to olive-brown with some black staining	25	MC-11A	101	25		
-33.0	35							

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

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BORING NUMBER EB-3

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL) 1.0 2.0 3.0 4.0 5.0
-33.0	35		**Silty Clay (CL)** stiff, moist, brown, trace fine sand, moderate plasticity							
			thin clayey sand seam at 38.5 feet to 38.8 feet	23	MC-12					○
-38.0	40		Bottom of Boring at 40.0 feet.							
	45									
	50									
	55									
	60									
	65									
	70									
	75									

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BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Areas 3 and 4 EIR Study
PROJECT NUMBER 118-3-3
PROJECT LOCATION Newark, Stevenson Boulevard
DATE STARTED 7/23/07 **DATE COMPLETED** 7/23/07
GROUND ELEVATION 3 FT +/- **BORING DEPTH** 46.5 ft.
DRILLING CONTRACTOR EGI
LATITUDE 37°30'9.108" **LONGITUDE** -123°59'41.2836"
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY JRD
AT TIME OF DRILLING 15 ft.
NOTES
AT END OF DRILLING Not Encountered

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
3.0	0	**Clayey Sand (SC) [Fill]** medium dense, moist, brown, fine to medium sand, some gravel						
1.7		**Silty Clay (CH)** very stiff to hard, moist, black, trace fine sand, high plasticity	29	MC-1A	103	10		
-1.5	5	**Silty Clay (CL)** stiff, moist, olive brown and gray mottled, moderate plasticity, some carbonate nodules	22	MC-2A	108	13		
			12	MC-3A	103	24		
	10	becomes soft, some organics	6	MC-4A	83	37		
				ST-5		37		
-11.0				ST-6	92	31		
	15	**Silty Clay (CH)** soft, wet, dark gray to olive gray, high plasticity, some sand	4	MC-7	96	28		
-15.0	20	**Sandy Clay (CL)** medium stiff to stiff, wet, olive-brown, fine to medium sand	6	MC-8	103	23		
	25		19	MC-9A	108	20	14	
	30		13	MC-10B	108	21		
-32.0	35							

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

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BORING NUMBER EB-4

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Areas 3 and 4 EIR Study

PROJECT NUMBER 118-3-3

PROJECT LOCATION Newark, Stevenson Boulevard

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf (○ HAND PENETROMETER, △ TORVANE, ● UNCONFINED COMPRESSION, ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL; 1.0 2.0 3.0 4.0 5.0)
-32.0	35									
-32.5			**Silty Sand (SM)** loose, wet, brown, fine to medium sand	11	MC-11A	103	22		38	
-36.0			**Silty Clay (CL)** medium stiff to stiff, wet, brown, trace fine sand	13	MC-12					
	40			9	MC-13					
	45		3 inch clayey sand seam at 44 1/2 feet	8	SPT-14					○
-43.5			Bottom of Boring at 46.5 feet.							
	50									
	55									
	60									
	65									
	70									
	75									

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:14 - P:\DRAFTING\GINT FILES\118-3-3 NEWARK AREAS 3 AND 4 EIR STUDY.GPJ

BORING NUMBER EB-5

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Feasibility Area 3 and 4
PROJECT NUMBER 102-4-1
PROJECT LOCATION Newark, CA

DATE STARTED 4/25/08 DATE COMPLETED 4/25/08
GROUND ELEVATION 6 FT +/- BORING DEPTH 35 ft.
DRILLING CONTRACTOR EGI
LATITUDE 37°30'8.5608" LONGITUDE -123°59'29.3676"
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY Jacob Fink
AT TIME OF DRILLING 13 ft.
NOTES
AT END OF DRILLING 10 ft.

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
6.0	0	**Silty Clay (CL) [Fill]** stiff, moist, dark gray, moderate to high plasticity	20	MC-1	92	23		
4.0		**Silty Clay** (CL) medium stiff to soft, wet, olive gray, moderate to high plasticity	6	MC-2B	65	62		
	5		16	MC-3B	98	27		
0.0		**Silty Clay with Fine Sand (CH)** medium stiff, wet, olive gray, calcite nodules, fine to medium sand, moderate to high plasticity	10	MC-4B	88	31		
	10			ST-5				
		color change to dark gray	10	MC-6B	102	24		
	15			ST-7				
	20	very stiff, color change to olive gray	37	MC-8B	114	18		
-18.5	25	**Sandy Clay** (CL) stiff, wet, light brown, calcite nodules, fine sand, low to moderate plasticity	47	MC-9B	118	17		
			30	MC-10B	102	24		
	30							
-25.0		**Clayey Silt with Sand (ML)** soft, wet, olive gray with light brown, increase sand with depth	9	MC-11B				78
-29.0	35	Bottom of Boring at 35.0 feet.						

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:08 - P:\DRAFTING\GINT FILES\102-4-1 NEWARK AREA 3 AND 4.GPJ

BORING NUMBER EB-6

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Feasibility Area 3 and 4
PROJECT NUMBER 102-4-1
PROJECT LOCATION Newark, CA

DATE STARTED 4/25/08 DATE COMPLETED 4/25/08
GROUND ELEVATION 5 FT +/- BORING DEPTH 44 ft.
DRILLING CONTRACTOR EGI
LATITUDE 37°30'3.3984" LONGITUDE -123°59'29.9724"
DRILLING METHOD 8" HSA
GROUND WATER LEVELS:
LOGGED BY Jacob Fink
AT TIME OF DRILLING 13 ft.
NOTES
AT END OF DRILLING 11.33 ft.

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ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
5.0	0	**Silty Clay (CL) [Fill]** stiff, moist, dark gray, moderate to high plasticity	27	MC-1	81	41		
2.0		**Silty Clay (CH)** medium stiff, wet, olive gray to light gray, moderate to high plasticity brown mottles, roots	12	MC-2B	86	35		
	5		15	MC-3B	95	30		
			9	MC-4	89	33		
	10			ST-5				
		color change to dark gray	6	MC-6B	102	24		
	15			ST-7				
		stiff, moist	33	MC-8	107	22		
	20							
-19.5		color change to olive gray	48	MC-9B	118	17		
	25	**Sandy Clay (CL)** very stiff, moist, light brown, fine sand, low to moderate plasticity						
			14	MC-10B	106	22		
	30							
-26.0		**Clayey Silt with Sand (ML)** soft, wet, olive gray with brown mottles, minor amounts of coarse sand						
			9	SPT-11		28		88
-30.5	35							

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0 5.0

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CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:09 - P:\DRAFTING\GINT FILES\102-4-1 NEWARK AREA 3 AND 4.GPJ

BORING NUMBER EB-6

CORNERSTONE EARTH GROUP

PROJECT NAME Newark Feasibility Area 3 and 4

PROJECT NUMBER 102-4-1

PROJECT LOCATION Newark, CA

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ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX, %	PERCENT PASSING NO. 200 SIEVE
-30.5									
-31.0			**Sandy Clay** (CL) stiff, moist, olive gray with brown mottles, fine sand, moderate plasticity	22	SPT-12A		27	9	89
				31	SPT-13		24		
	40			30	MC-14B	109	21		
-37.0			**Silty Sand (SM)** medium dense, wet, light brown, fine grained rounded sand, decrease silt content with depth	19	SPT-15A				77
-38.5				26	SPT-16A 16B		22		28
-39.0	45		**Silty Clay (CL)** very stiff, moist, olive gray with minor amounts of brown mottles, high plasticity						
			Bottom of Boring at 44.0 feet.						
	50								
	55								
	60								
	65								
	70								
	75								

UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

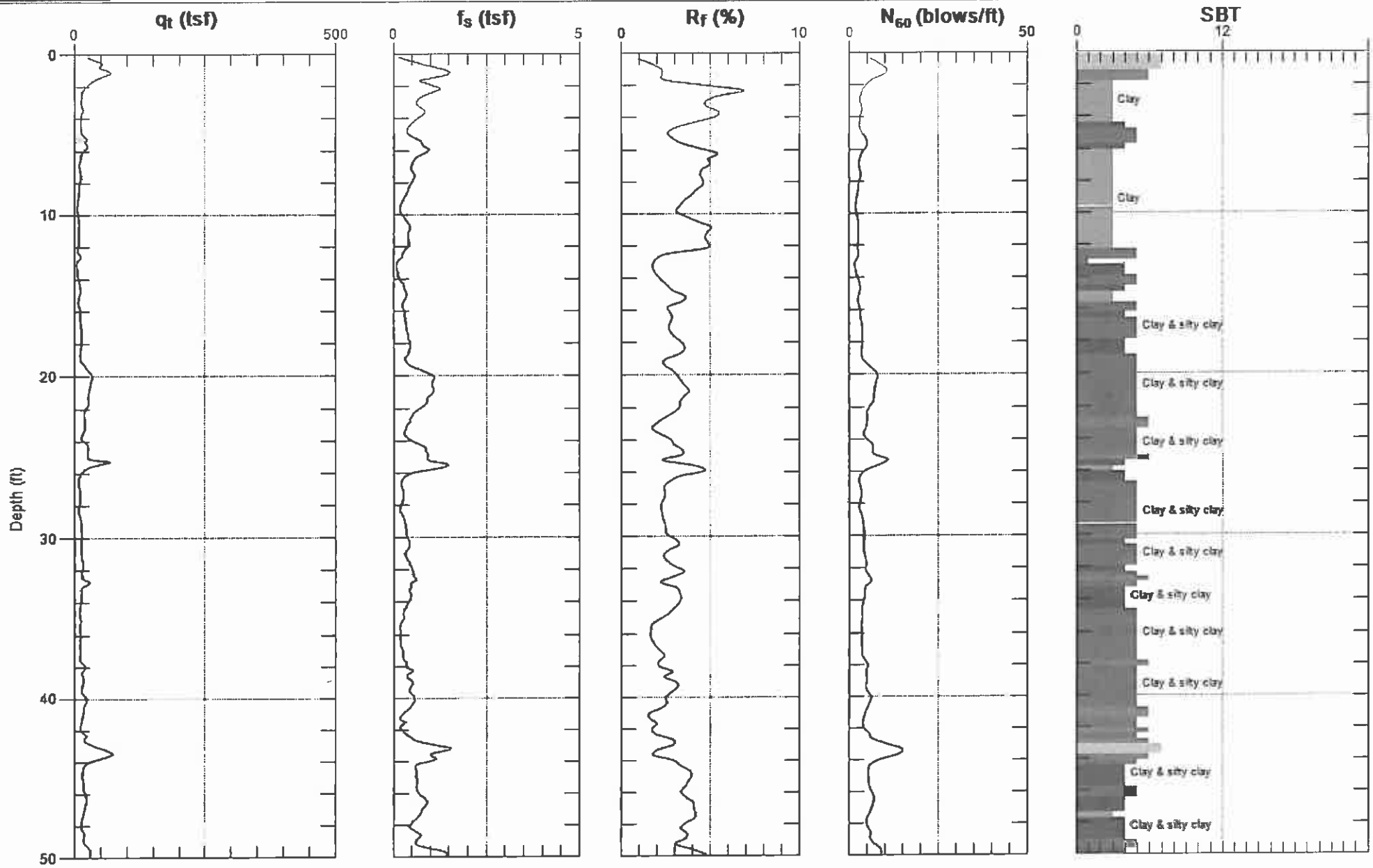
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0 5.0

CORNERSTONE EARTH GROUP - CORNERSTONE.GDT - 6/9/08 11:09 - P:\DRAFTING\GINT FILES\102-4-1 NEWARK AREA 3 AND 4.GPJ

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-01
Engineer: J.DYE
Date: 5/9/2007 07:33



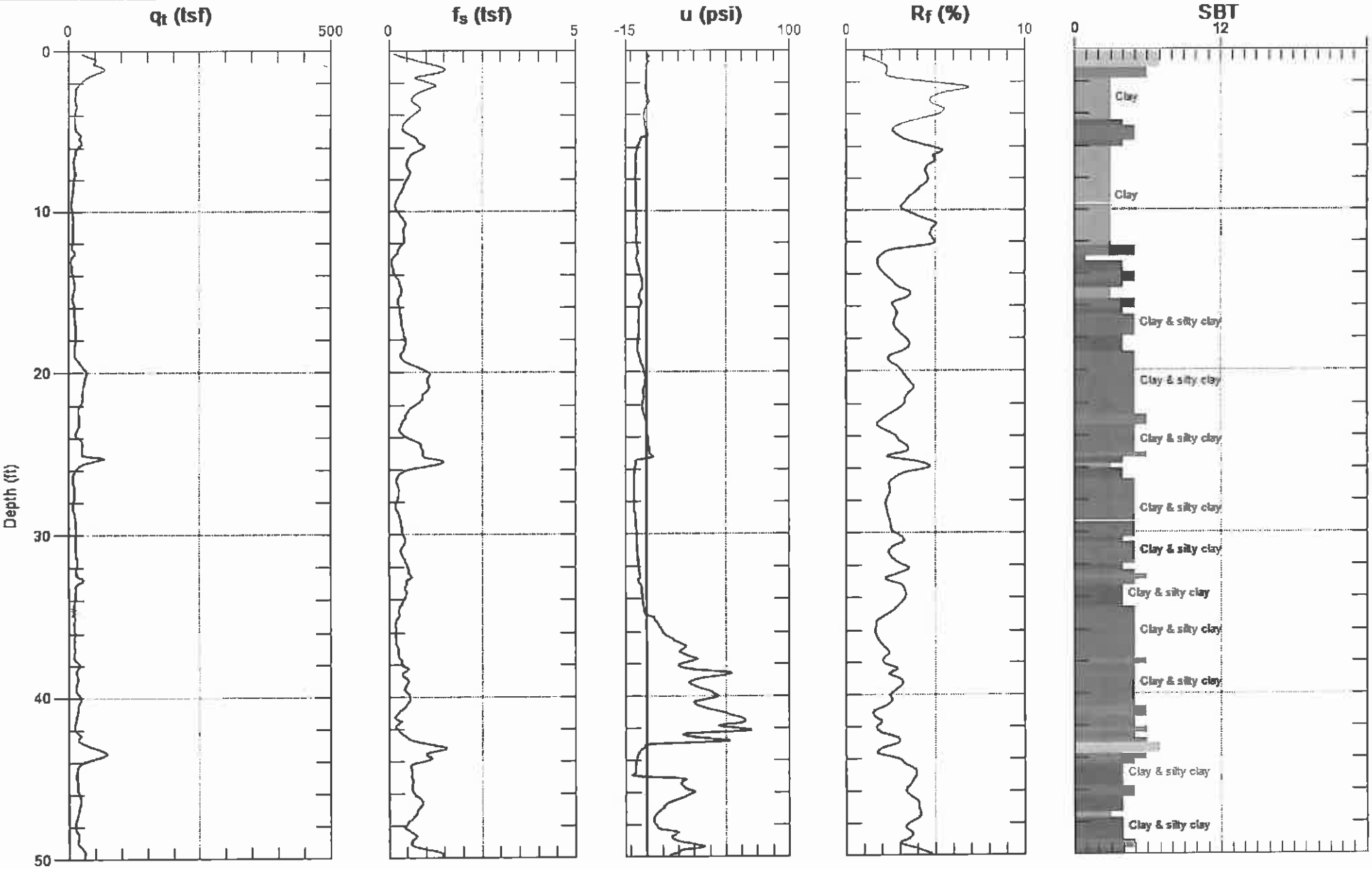
Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-01

Engineer: J.DYE
Date: 5/9/2007 07:33



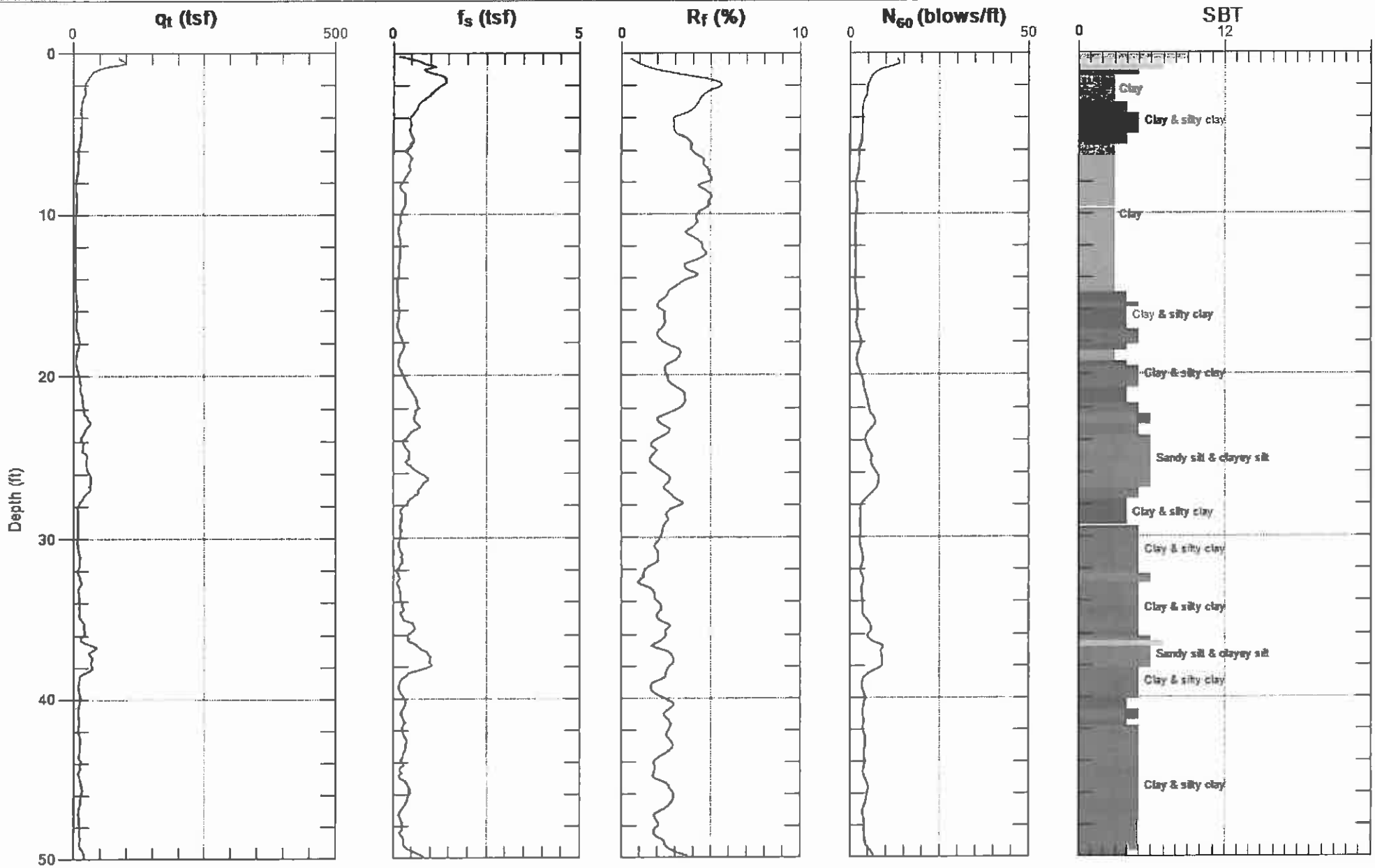
Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-02

Engineer: J.DYE
Date: 5/9/2007 08:52



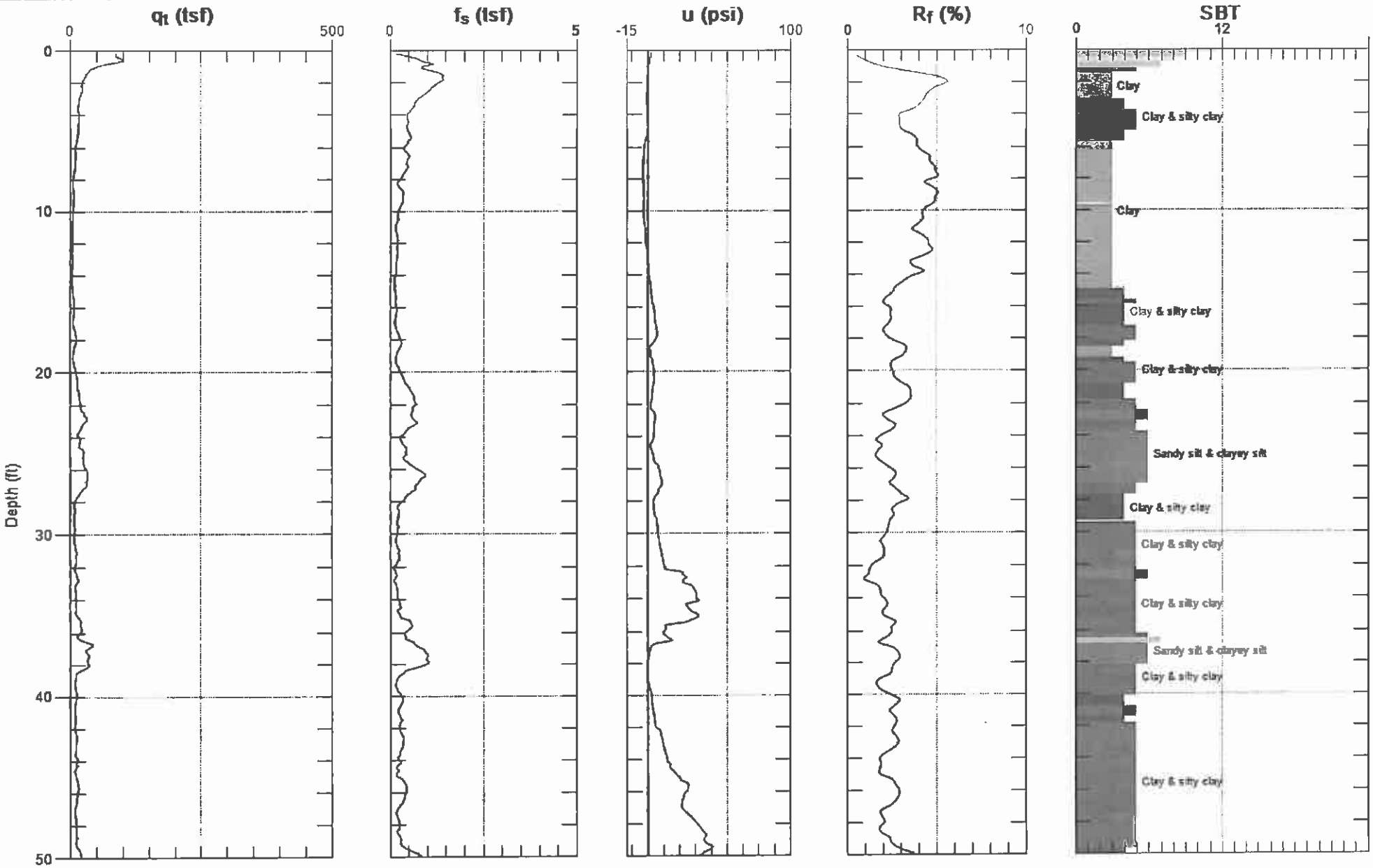
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-02

Engineer: J.DYE
Date: 5/9/2007 08:52

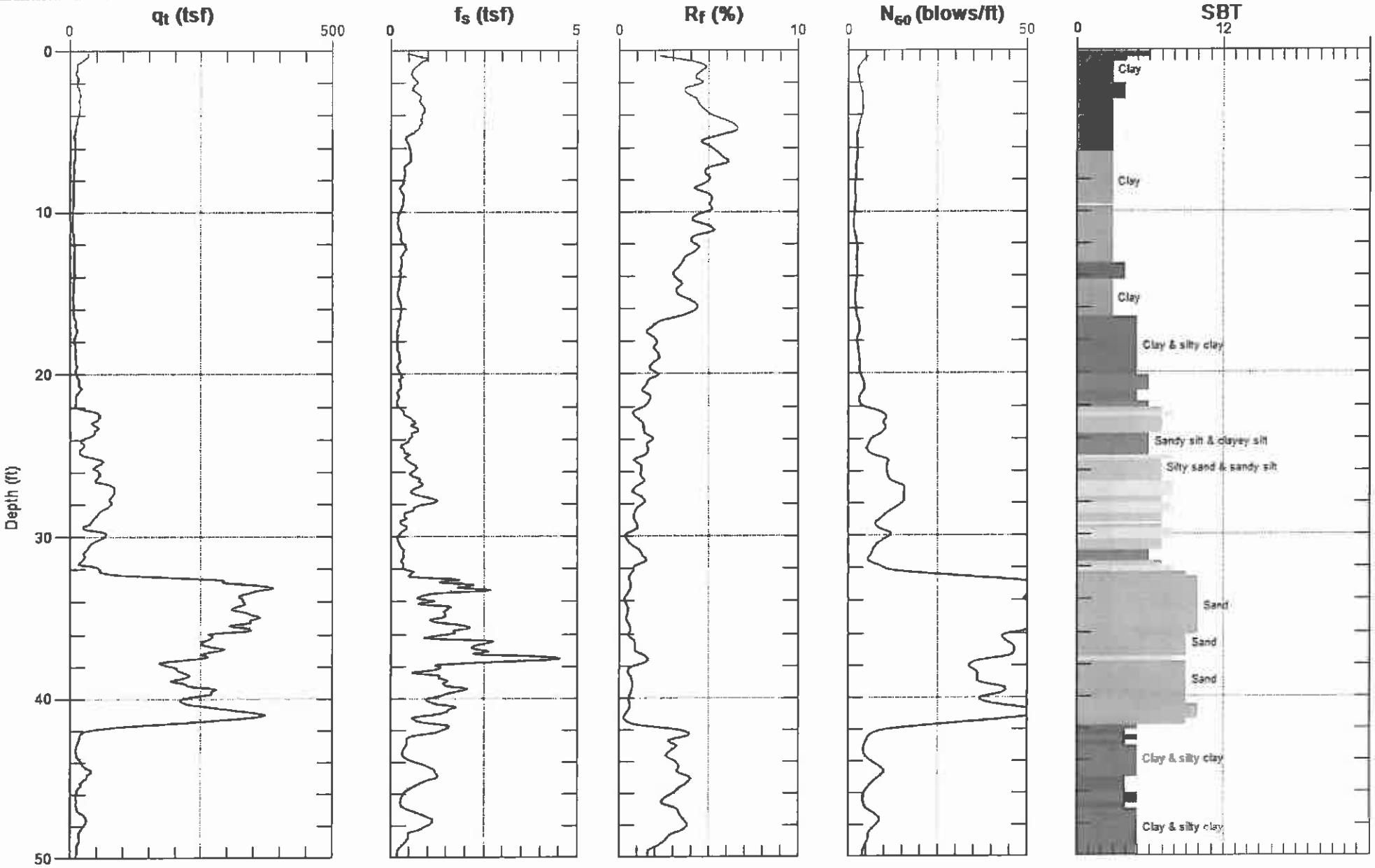


Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-03
Engineer: J.DYE
Date: 5/9/2007 10:42



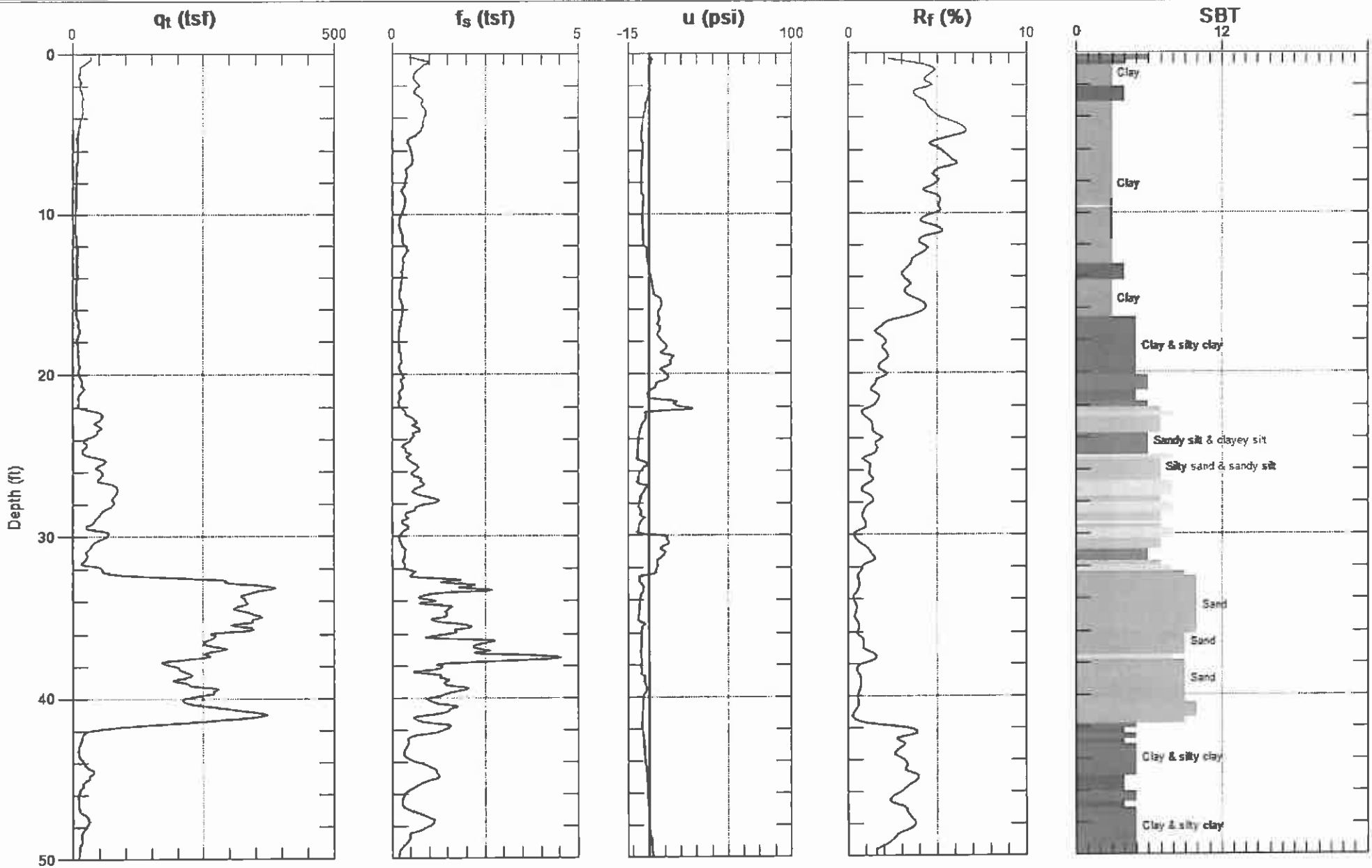
Max Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-03

Engineer: J.DYE
Date: 5/9/2007 10:42



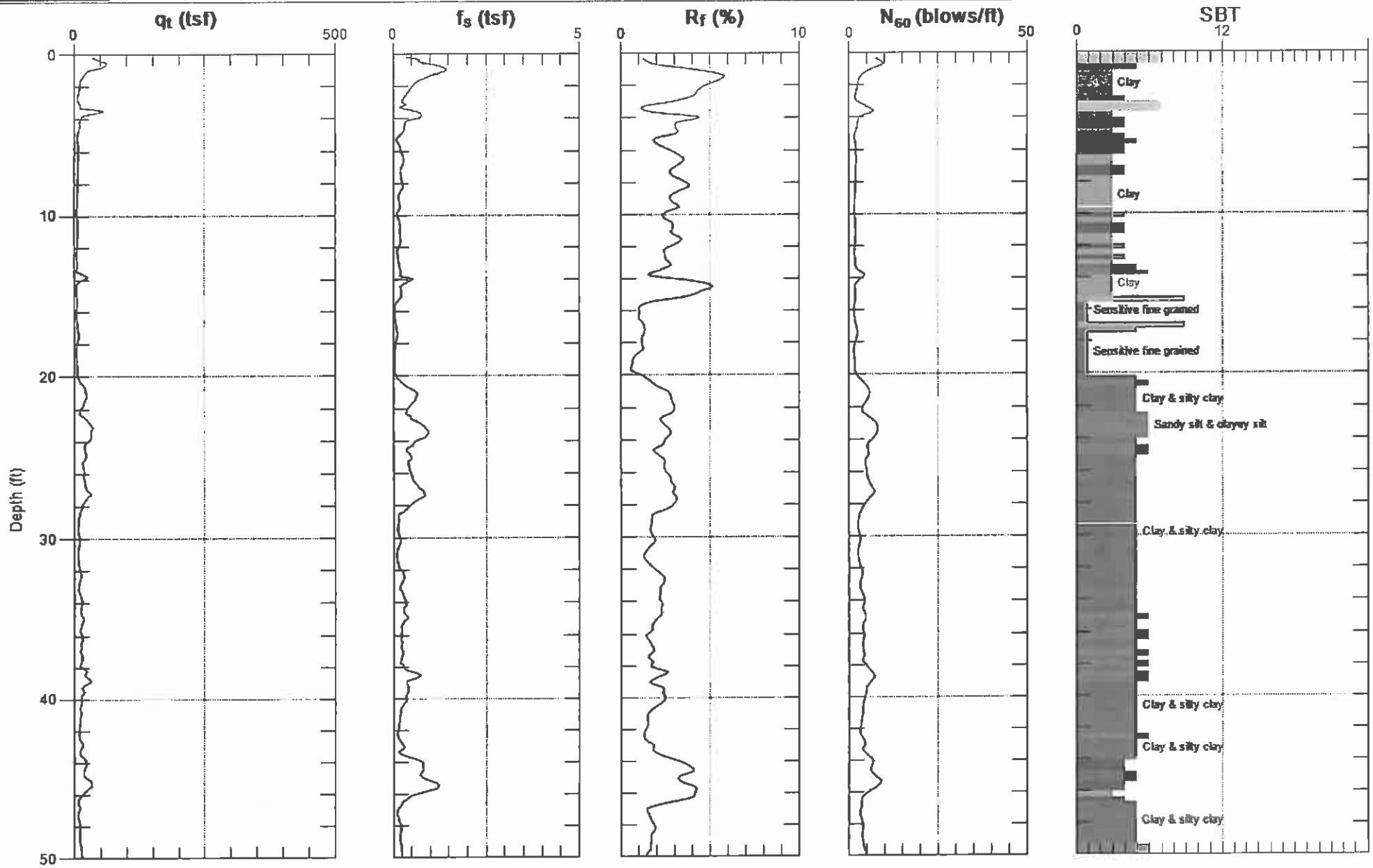
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-04

Engineer: J.DYE
Date: 5/9/2007 06:12



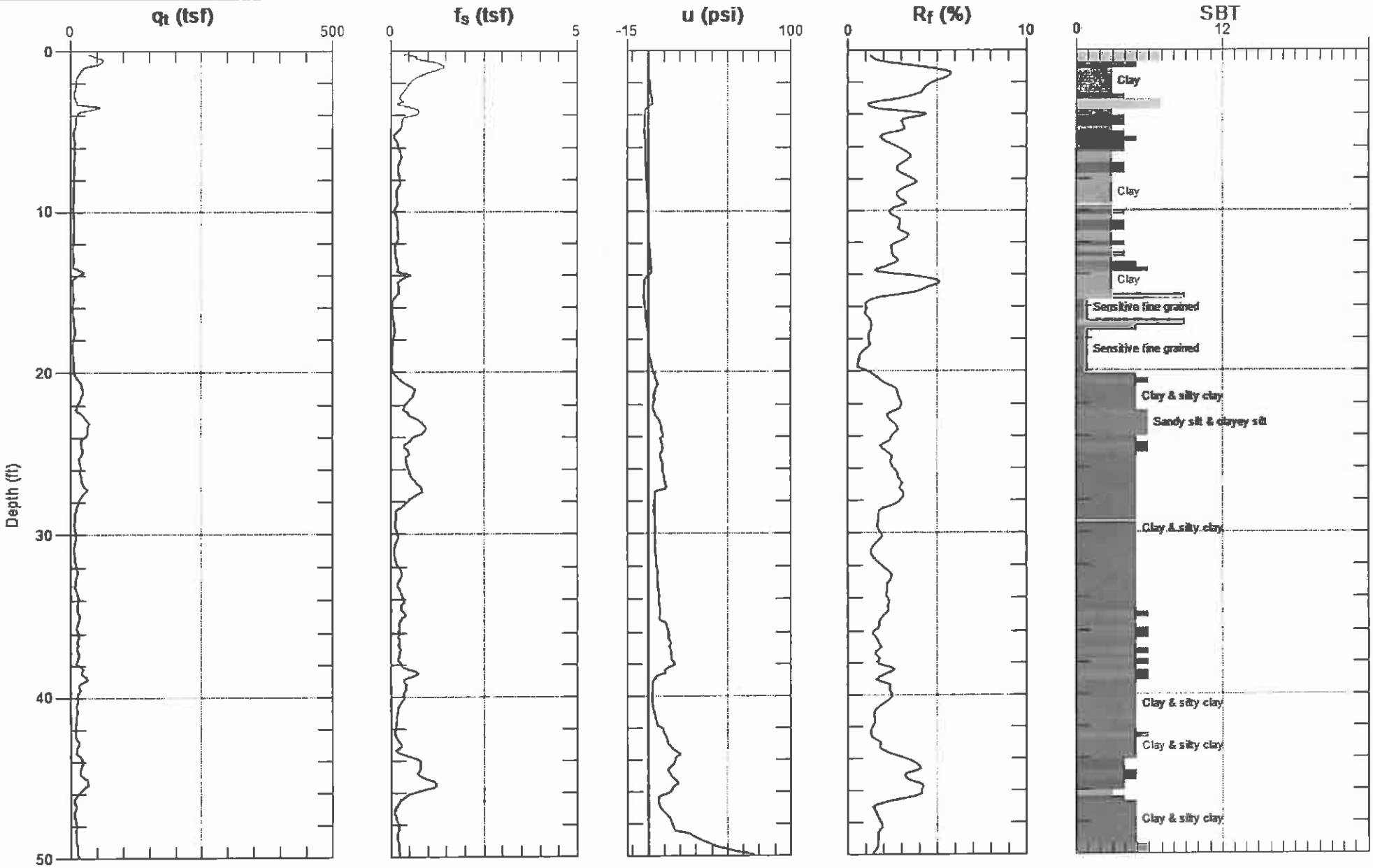
Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-04

Engineer: J.DYE
Date: 5/9/2007 06:12



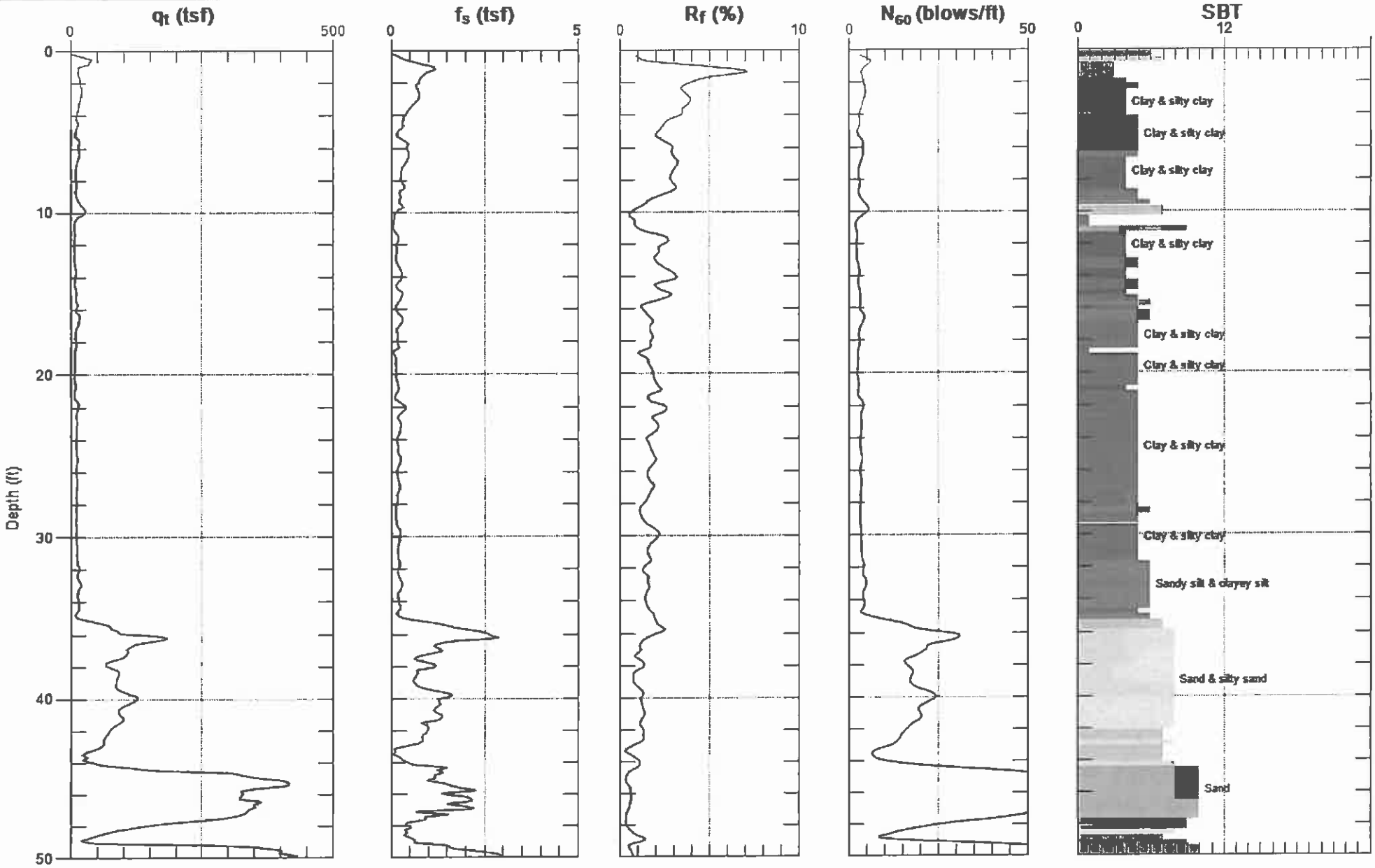
Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-05

Engineer: J.DYE
Date: 5/10/2007 05:37



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

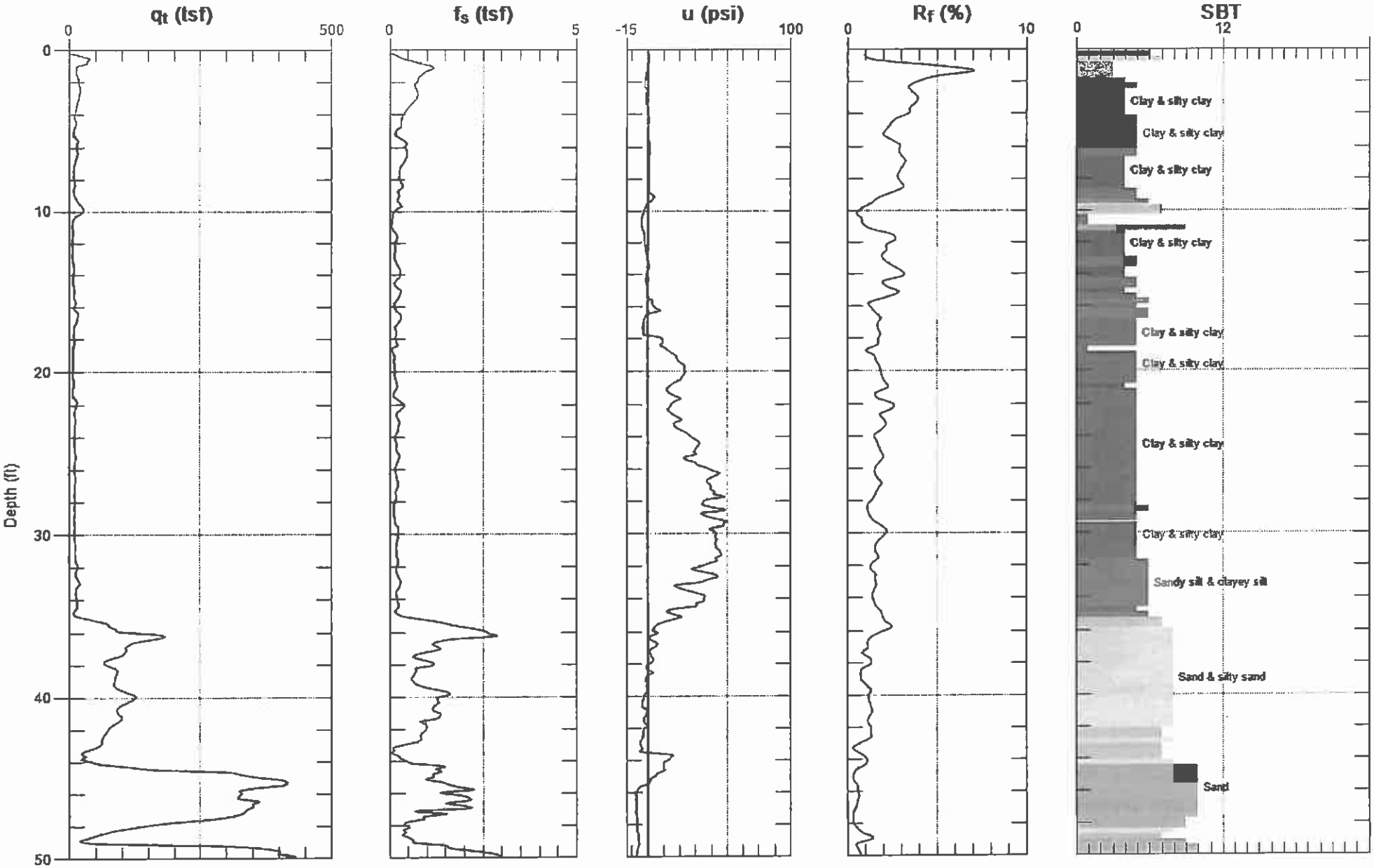
SBT: Soil Behavior Type (Robertson 1990)

GREGG

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-05

Engineer: J.DYE
Date: 5/10/2007 05:37



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

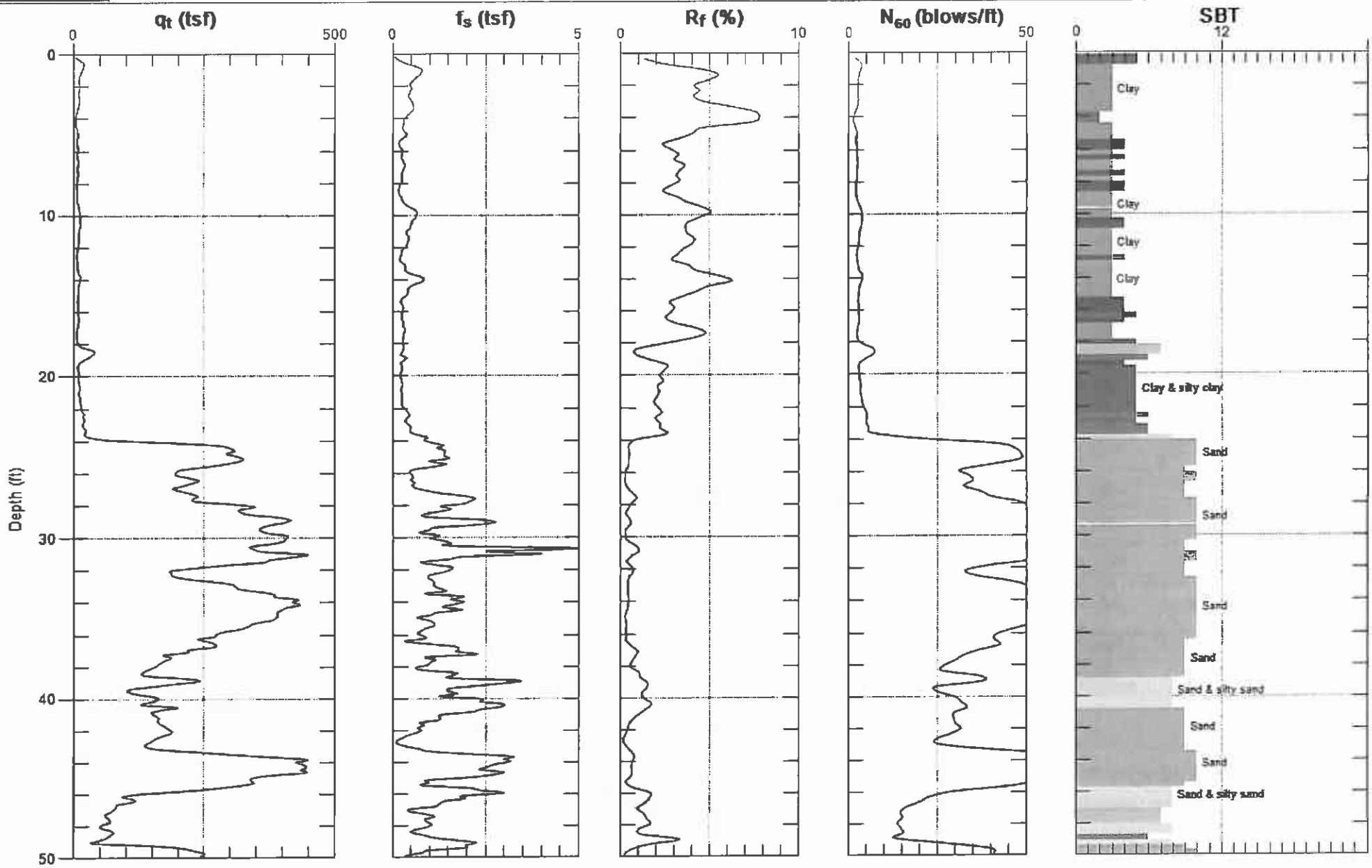
SBT: Soil Behavior Type (Robertson 1990)

GREGG

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-06

Engineer: J.DYE
Date: 5/9/2007 01:40



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

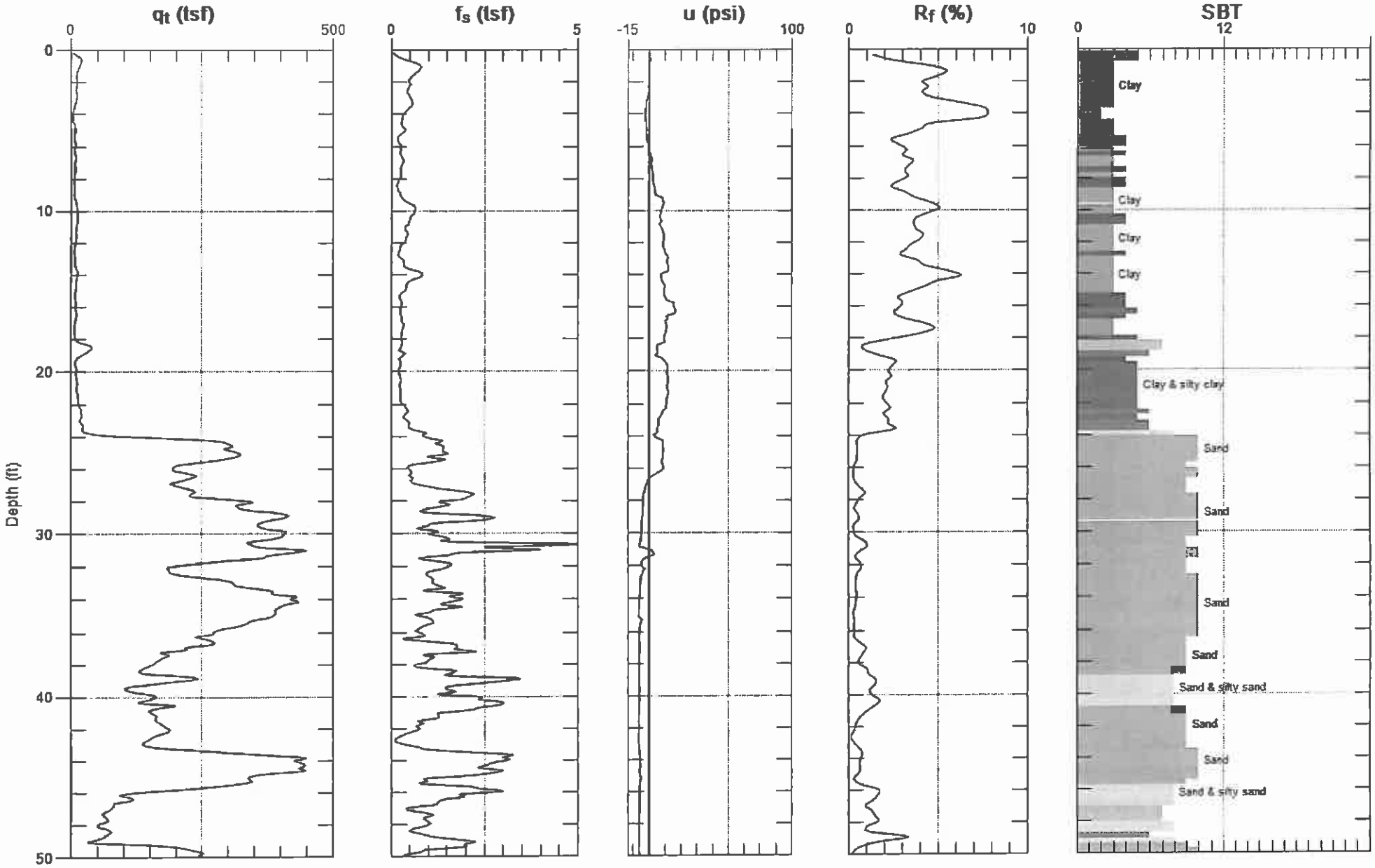
SBT: Soil Behavior Type (Robertson 1990)

GREGG

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-06

Engineer: J.DYE
Date: 5/9/2007 01:40



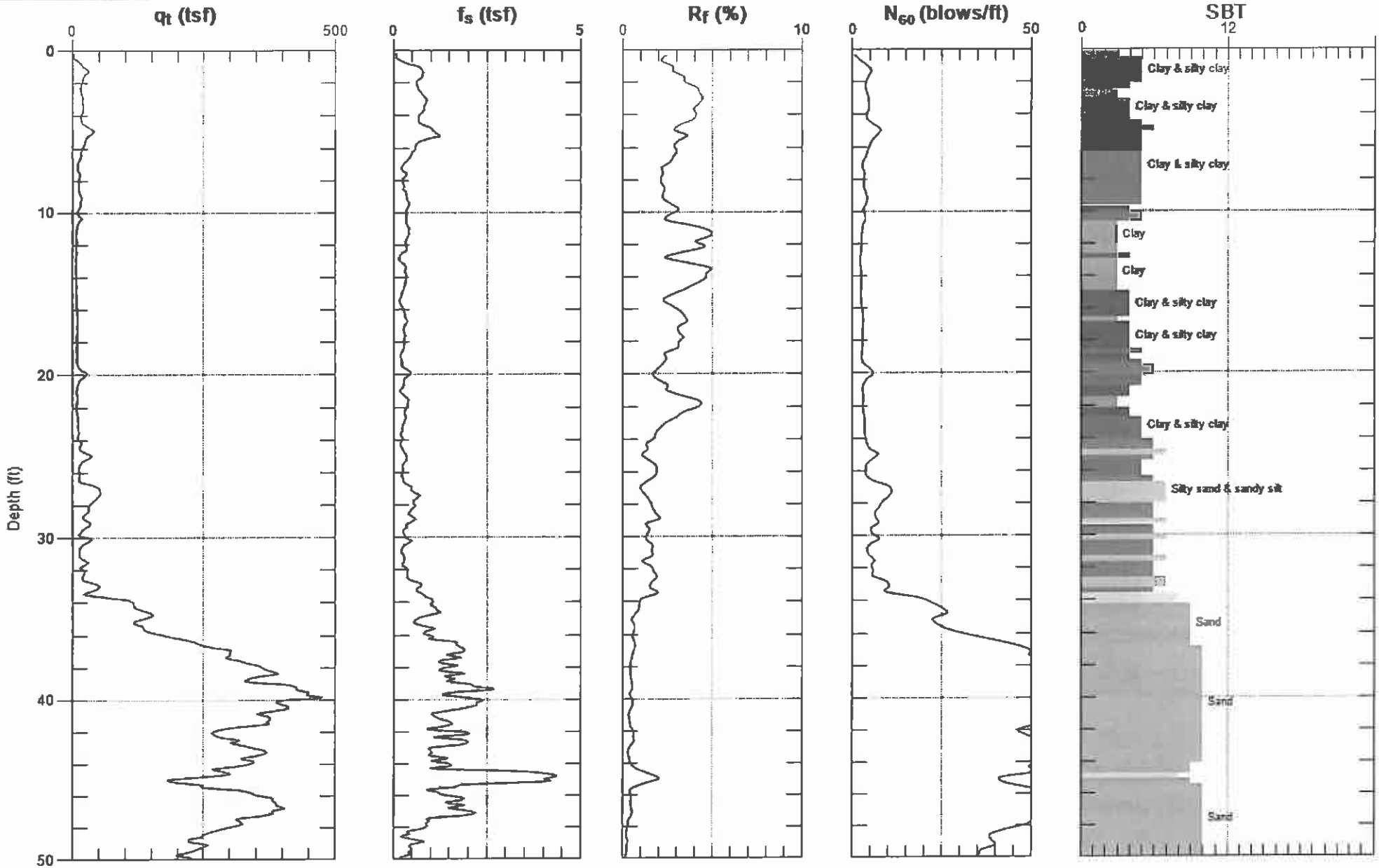
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

Site: NEWARK AREAS 3&4
Sounding: CPT-07

Engineer: J.DYE
Date: 5/9/2007 12:18



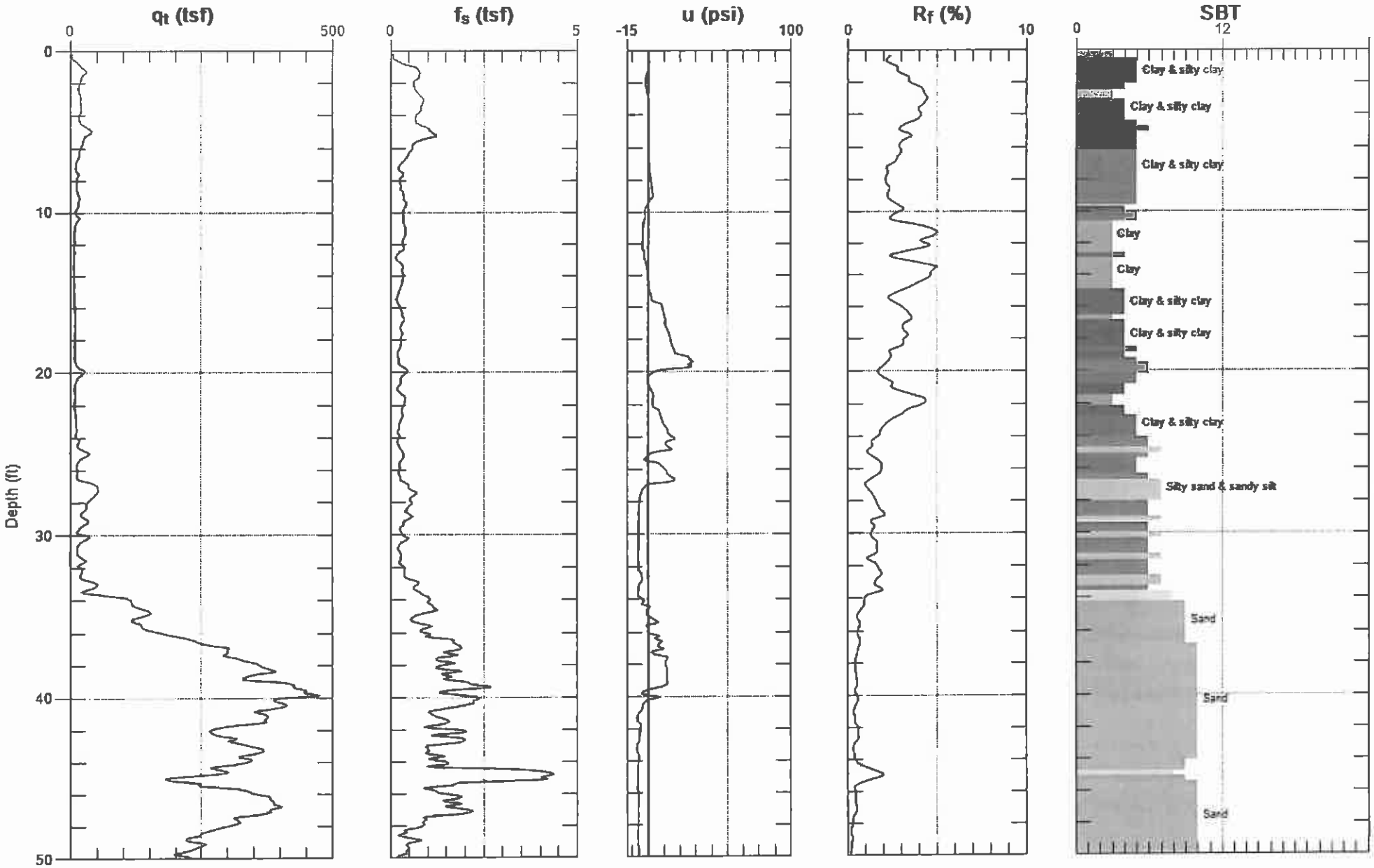
Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

CORNERSTONE EARTH GROUP

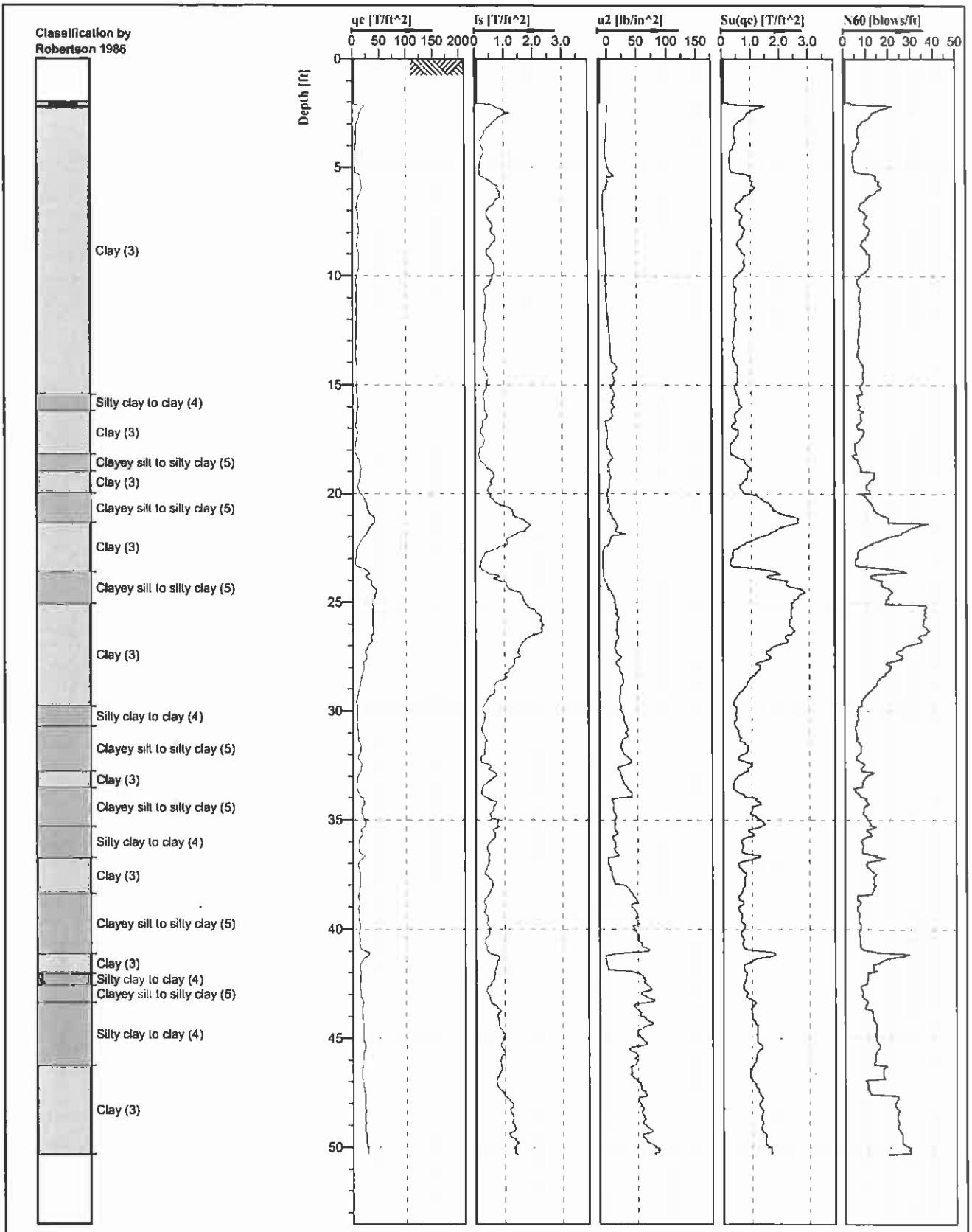
Site: NEWARK AREAS 3&4
Sounding: CPT-07

Engineer: J.DYE
Date: 5/9/2007 12:18



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

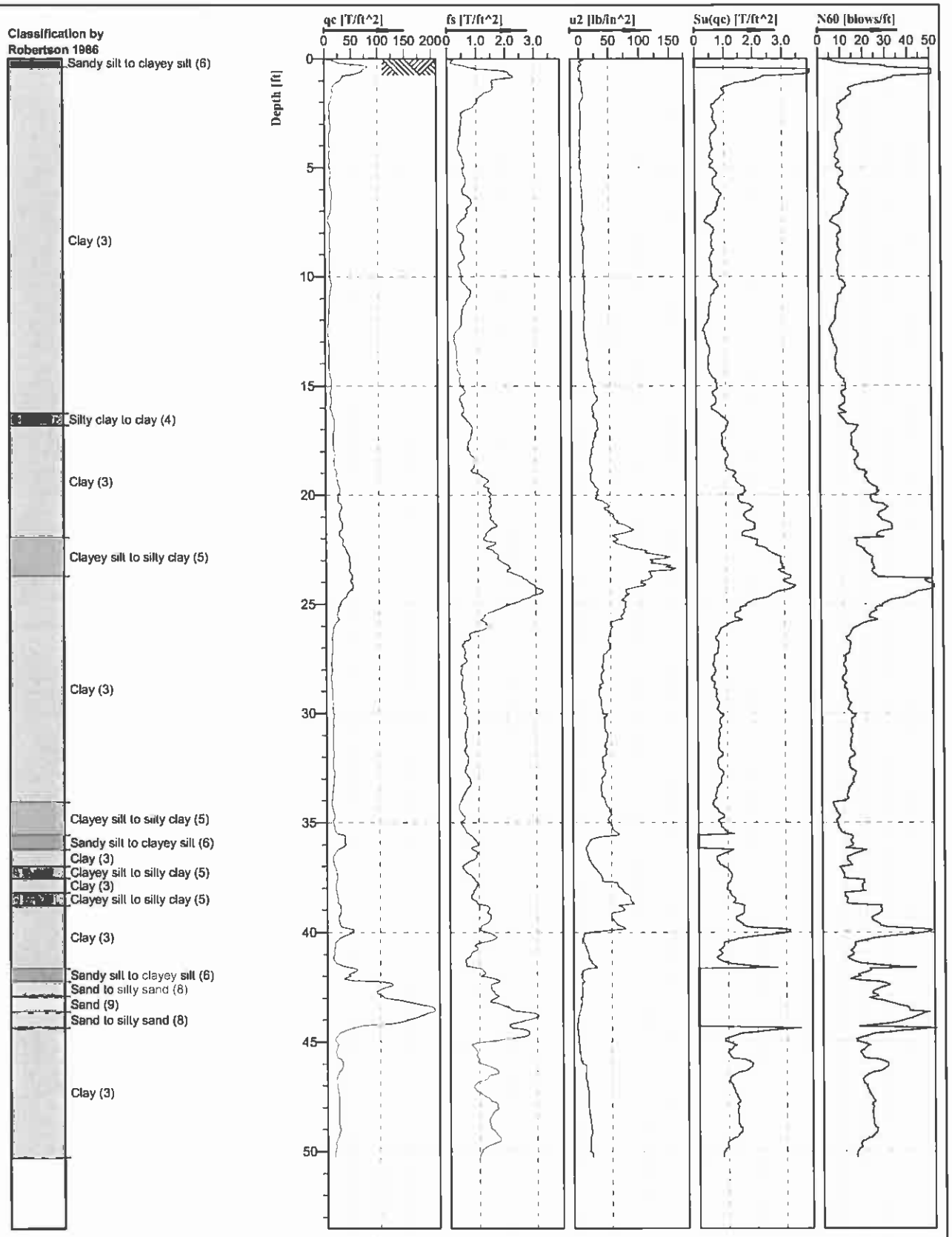
SBT: Soil Behavior Type (Robertson 1990)



CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

Cone No 3794
Tip area [cm2] 10
Sleeve area [cm2] 150

Location: Newark, California	Position:	Ground level:	Test no: CPT-8
Project ID: 102-4-1	Client: Cornerstone Earth Group	Date: 4/24/2008	Scale: 1 : 75
Project: Areas 3 and 4		Page: 1/1	Fig:
		File: CPT-8.cpd	

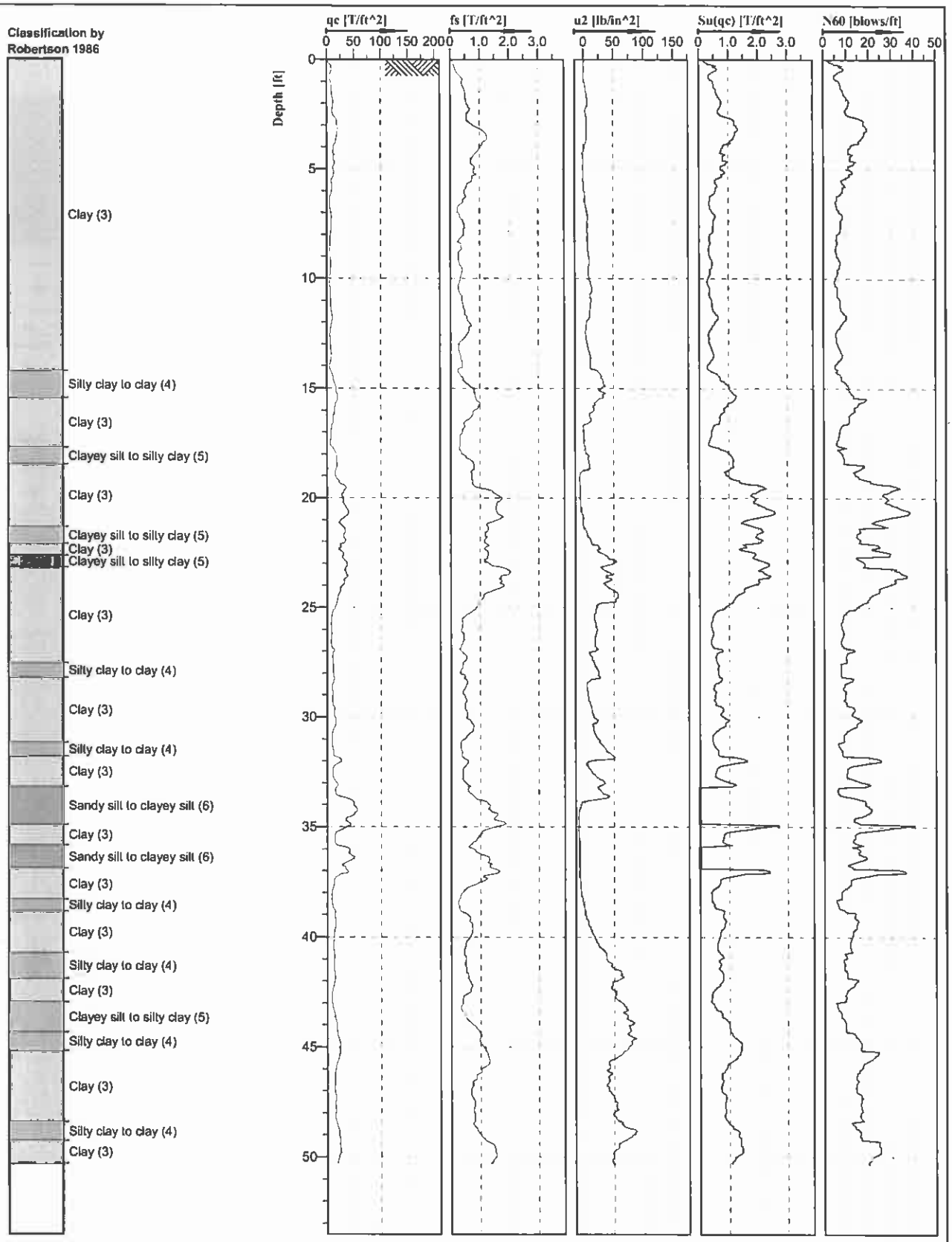


CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

u2

Cone No 3794
Tip area [cm2] 10
Sleeve area [cm2] 150

Location: Newark, California	Position:	Ground level:	Test no CPT-9
Project ID 102-4-1	Client: Cornerstone Earth Group	Date: 4/24/2008	Scale: 1 : 75
Project: Areas 3 and 4		Page 1/1	Fig:
		File: CPT-9.cpd	



CALIFORNIA PUSH TECHNOLOGIES INCORPORATED

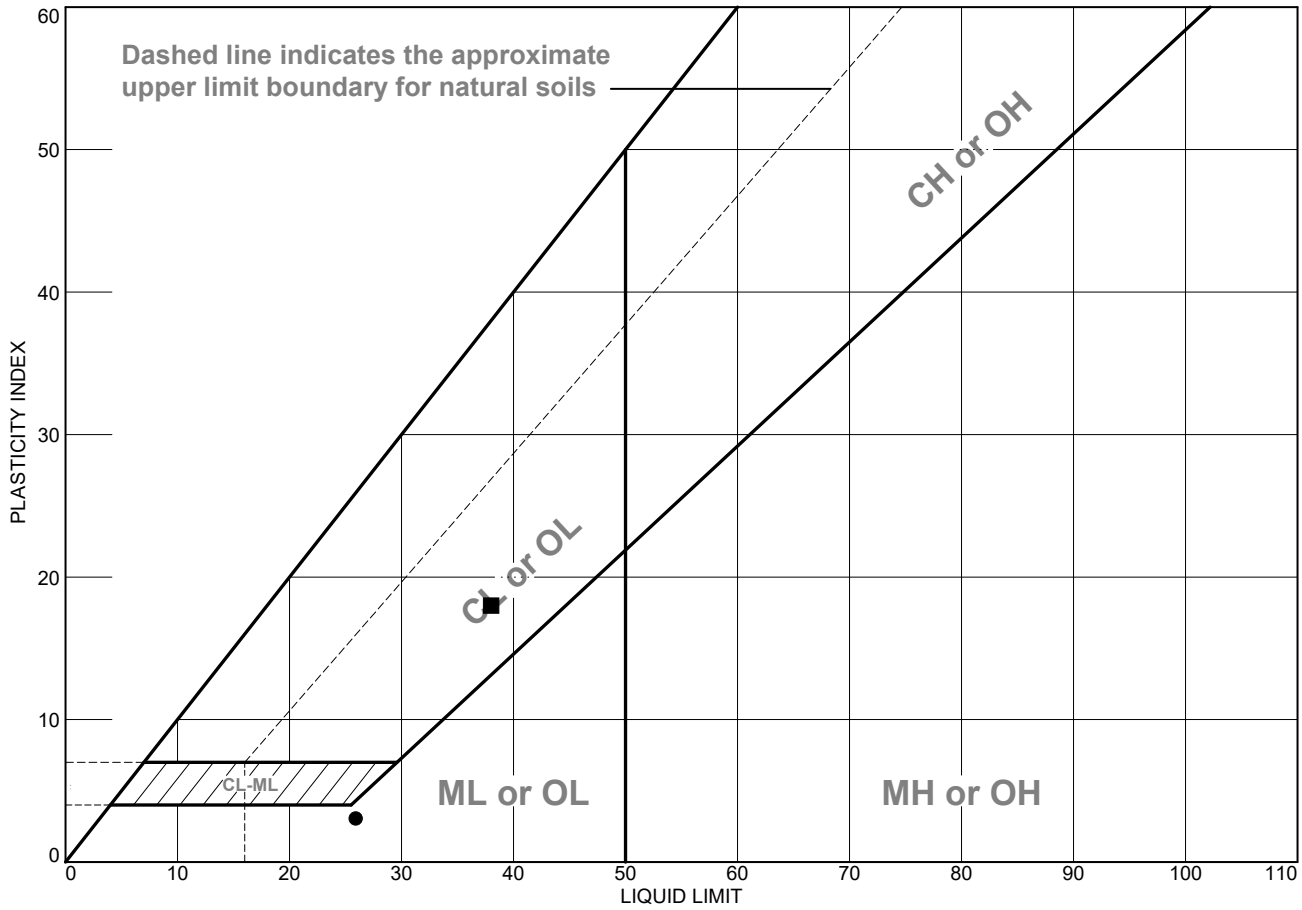
Cone No. 3794
Tip area [cm2] 10
Sleeve area [cm2] 150

Location: Newark, California	Position:	Ground level:	Test no: CPT-10
Project ID: 102-4-1	Client: Cornerstone Earth Group	Date: 4/24/2008	Scale: 1 : 75
Project: Areas 3 and 4		Page: 1/1	Fig:
		File: CPT-10.cpd	

APPENDIX C

LABORATORY TEST DATA
(ENGEO, 2018)

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	26	23	3		94.3	
■	See exploration logs	38	20	18			

Project No. 12165.000.002 **Client:** The Sobrato Organization

Project: Sobrato Property - Area 4

●**Depth:** 26.0 feet **Sample Number:** 1-B1 @ 26

■**Depth:** 3.0 feet **Sample Number:** 1-B2 @ 3

Remarks:

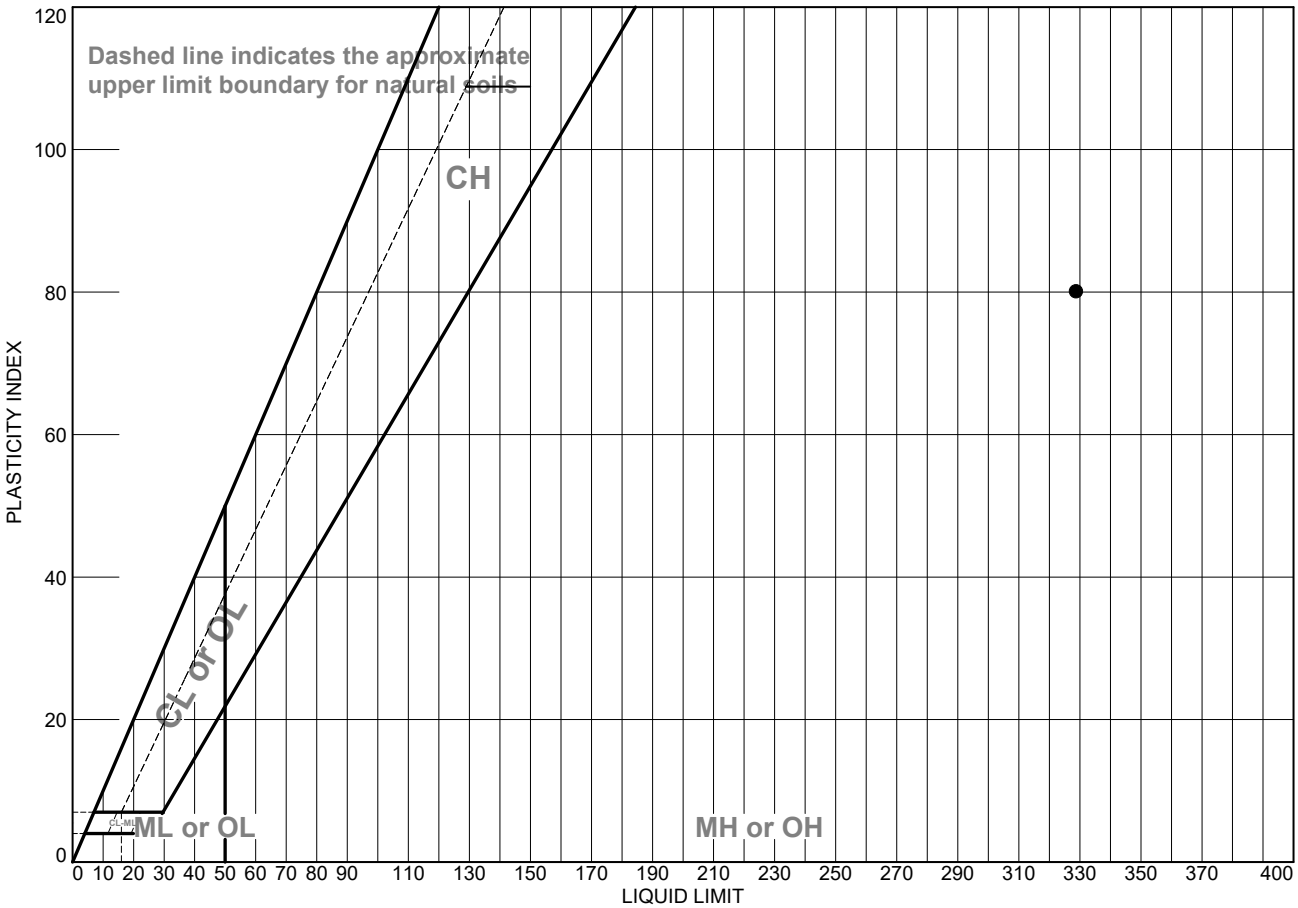
●PI: ASTM D4318, Wet method
GS: ASTM D1140

■ASTM D4318, Wet method

ENGEO INCORPORATED

Tested By: M. Bromfield **Checked By:** M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	329	249	80			

Project No. 12165.000.002 **Client:** The Sobrato Organization

Project: Sobrato Property - Area 4

●**Depth:** 16.5 feet **Sample Number:** 1-B2 @ 16.5

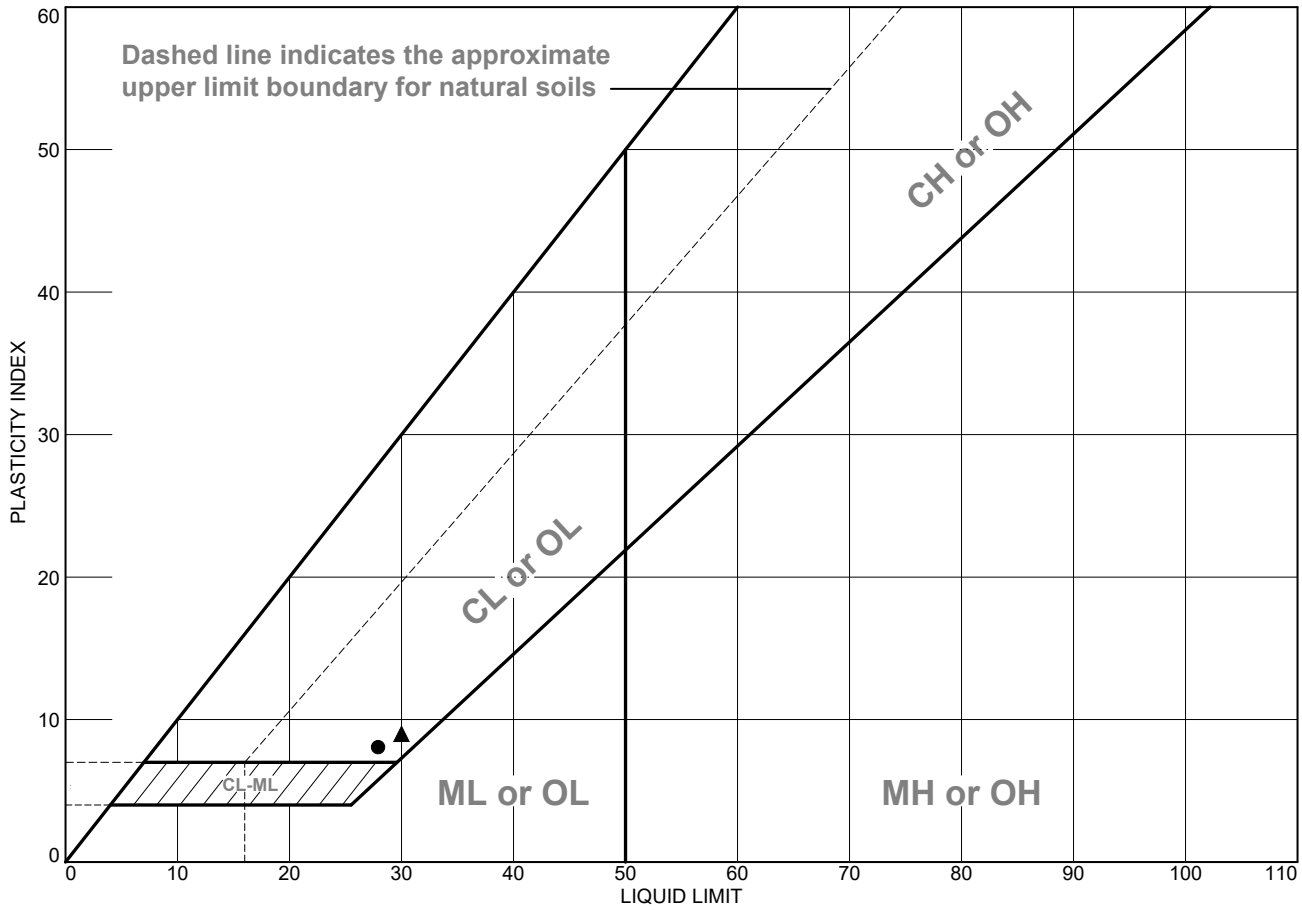
Remarks:

●ASTM D4318, Wet method

ENGEO INCORPORATED

Tested By: M. Bromfield **Checked By:** M. Quasem

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	28	20	8		79.8	
■	See exploration logs	NV	NP	NP		40.2	
▲	See exploration logs	30	21	9			

Project No. 12165.000.002 **Client:** The Sobrato Organization

Project: Sobrato Property - Area 4

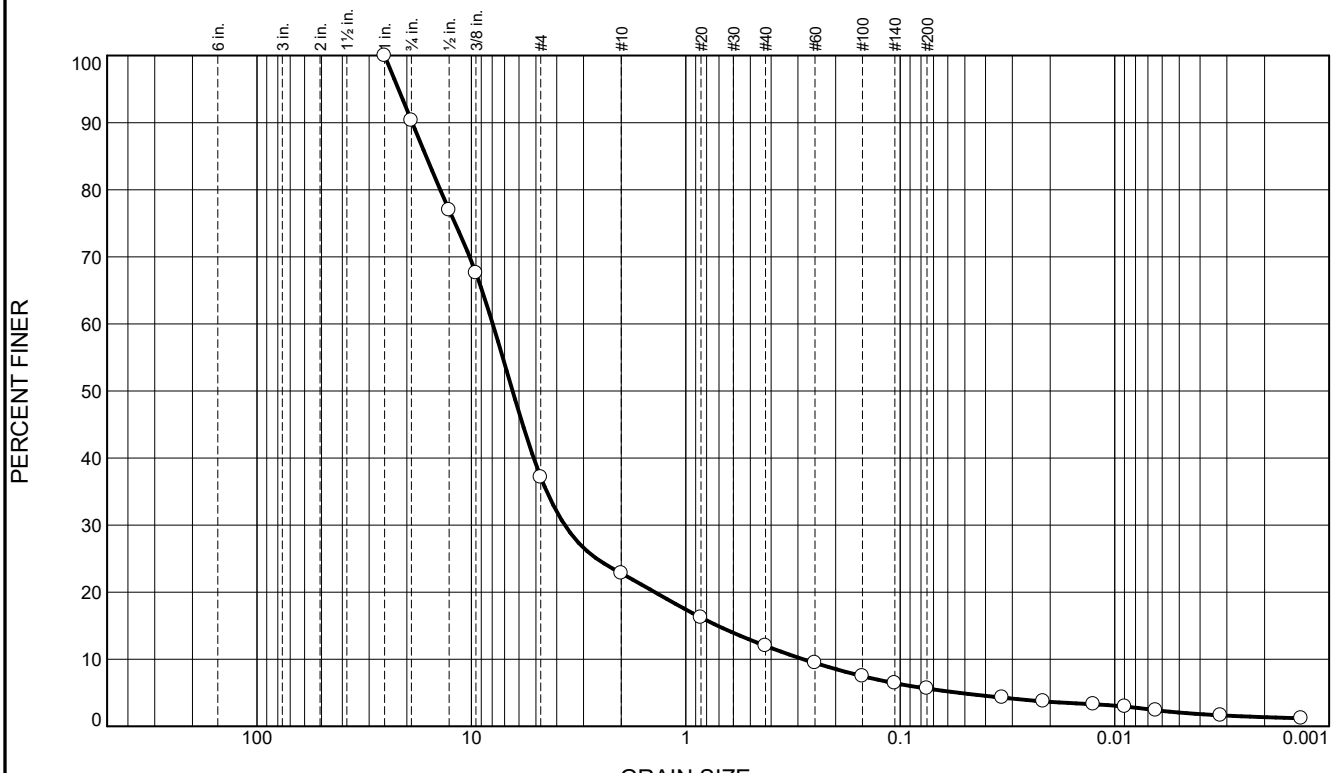
●**Sample Number:** 1-B4 @ 16.5
■**Sample Number:** 1-B4 @ 26.5
▲**Sample Number:** 1-B4 @ 36

Remarks:
●PI: ASTM D4318, Wet method
GS: ASTM D1140
■PI: ASTM D4318, Wet method
GS: ASTM D1140
▲ASTM D4318, Wet method

ENGEO INCORPORATED

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	9.7	53.2	14.3	10.8	6.4	4.2	1.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	90.3		
1/2	77.0		
3/8	67.6		
#4	37.1		
#10	22.8		
#20	16.2		
#40	12.0		
#60	9.4		
#100	7.5		
#140	6.4		
#200	5.6		
0.0335 mm.	4.3		
0.0215 mm.	3.7		
0.0126 mm.	3.3		
0.0090 mm.	2.9		
0.0064 mm.	2.4		
0.0032 mm.	1.6		
0.0013 mm.	1.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 18.8638 D_{85}= 16.2616 D_{60}= 7.9493
D_{50}= 6.4241 D_{30}= 3.6692 D_{15}= 0.7105
D_{10}= 0.2834 C_u= 28.05 C_c= 5.98

Classification

USCS= AASHTO=

Remarks

GS: ASTM D422; Specific gravity = 2.681, ASTM D854
Silt/clay division of 0.002mm was usd

Sample Number: 1-B1 @ 40 and 45 **Depth:** 40.0 and 45.0 feet

Date: 3/01/18

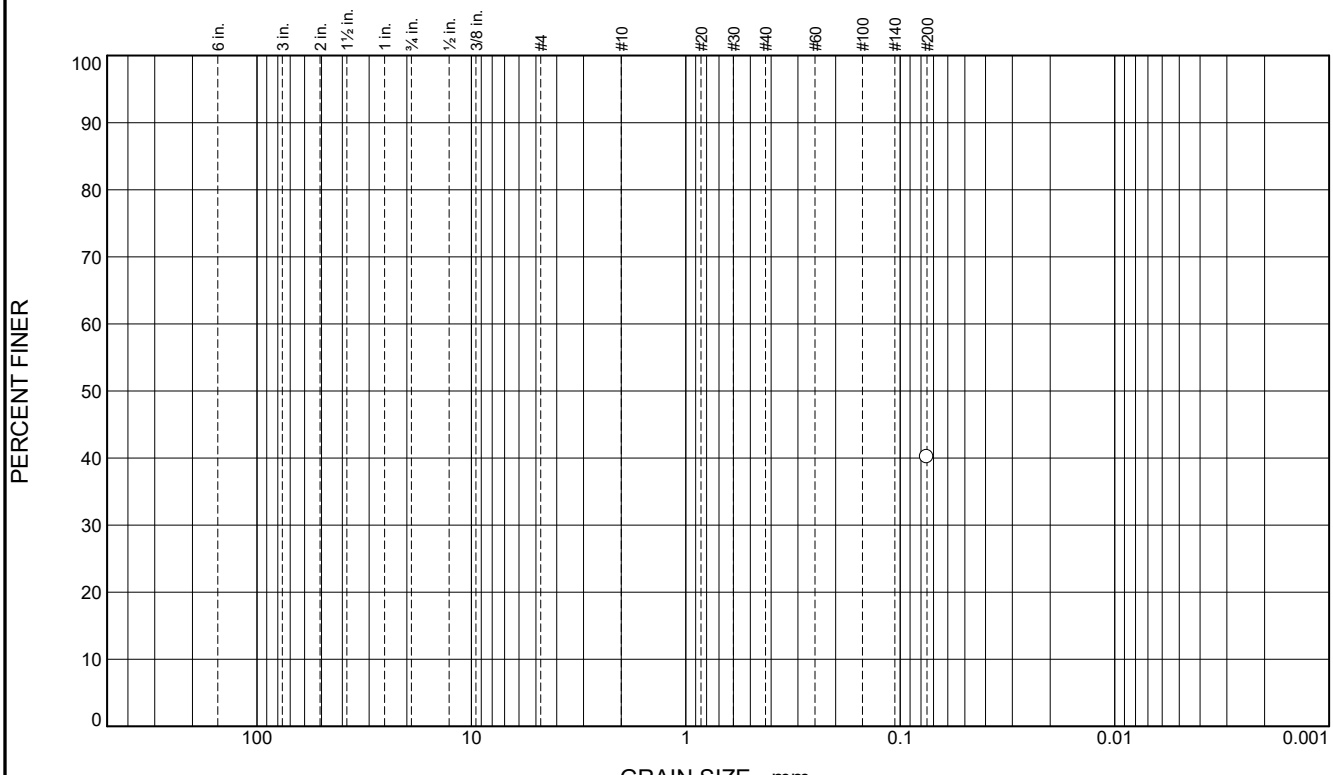
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Quasem **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						40.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	40.2		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140
PI: ASTM D4318, Wet method

Sample Number: 1-B4 @ 26.5

Date: 3/13/2018

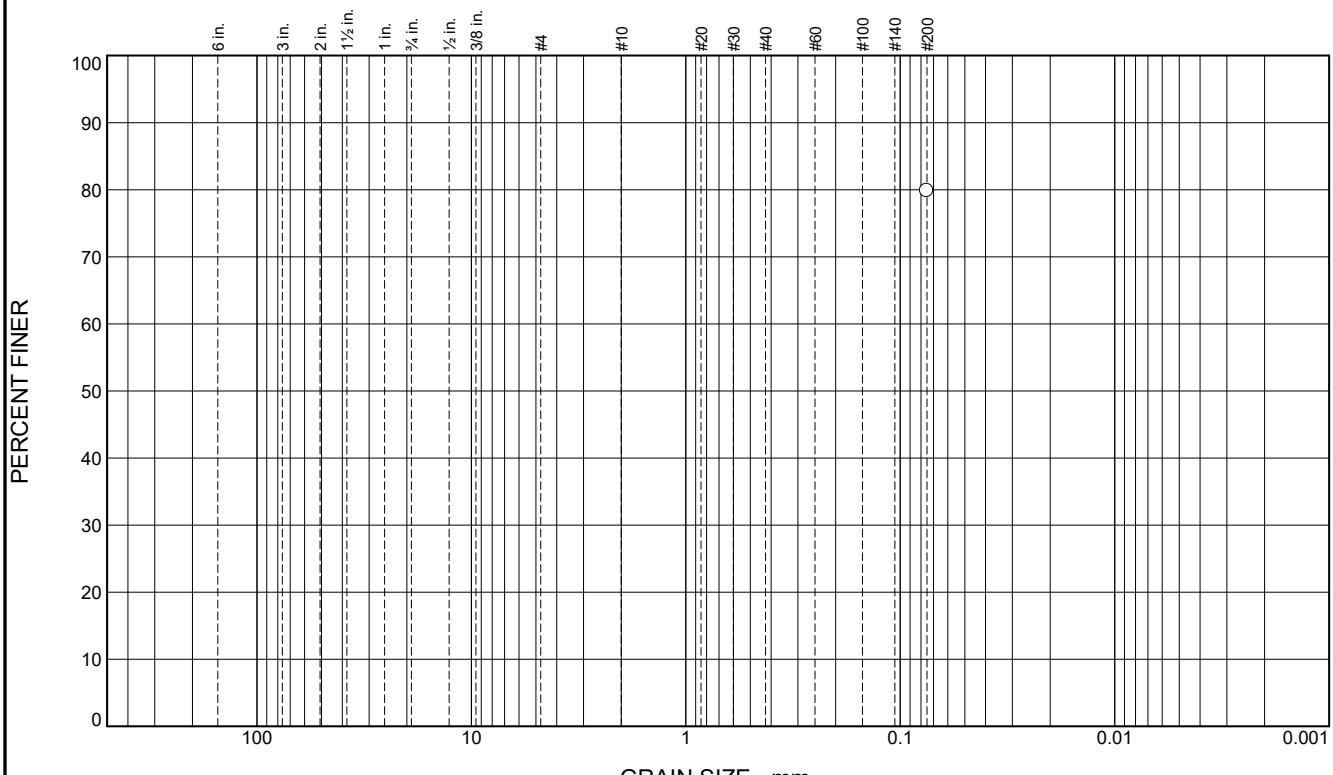
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						79.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	79.8		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 20 LL= 28 PI= 8

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140
PI: ASTM D4318, Wet method

Sample Number: 1-B4 @ 16.5

Date: 3/13/2018

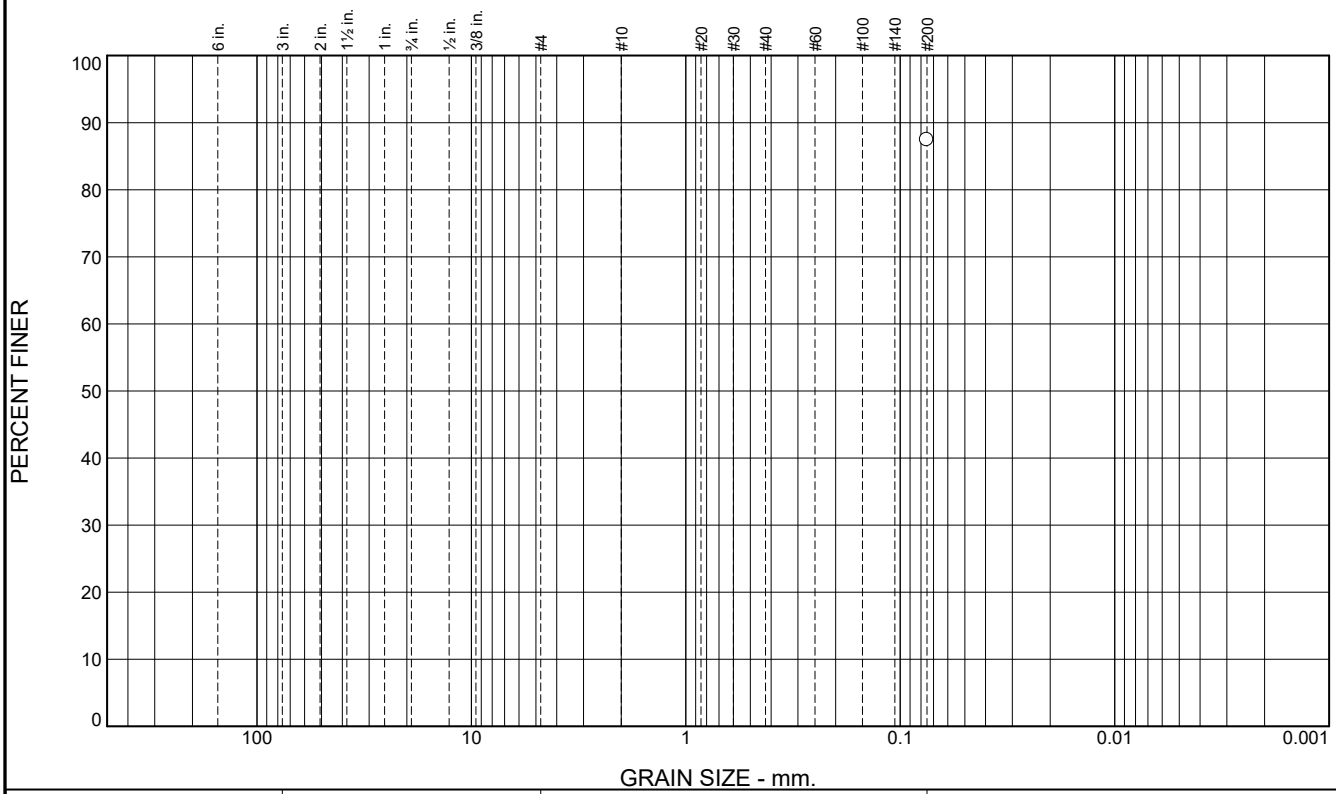
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Bromfield **Checked By:** G. Criste

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						87.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	87.4		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Sample Number: 1-B3 @ 23

Date: 3/21/18

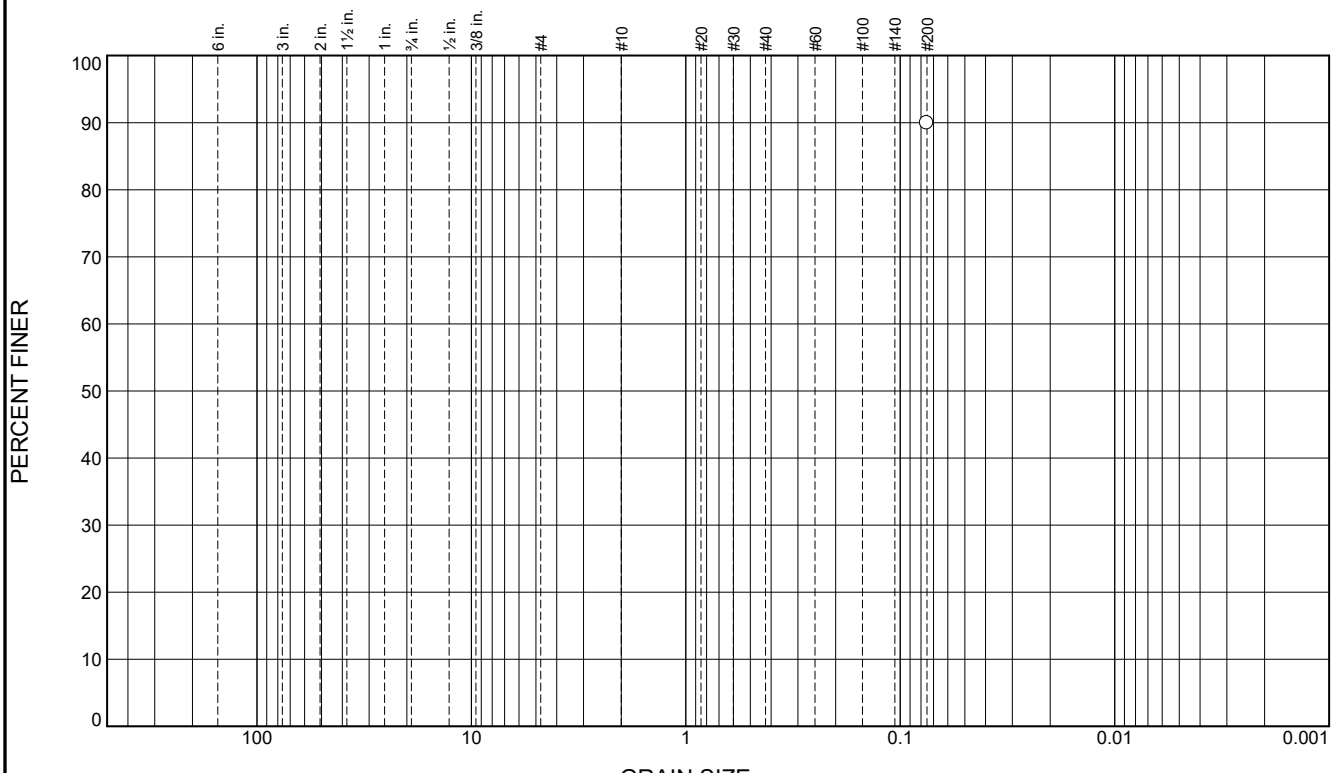
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Quasem **Checked By:** M. Bromfield

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						89.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	89.9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

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

Classification

USCS= AASHTO=

Remarks

ASTM D1140

Sample Number: 1-B2 @ 21 **Depth:** 21.0 feet

Date: 3/5/18

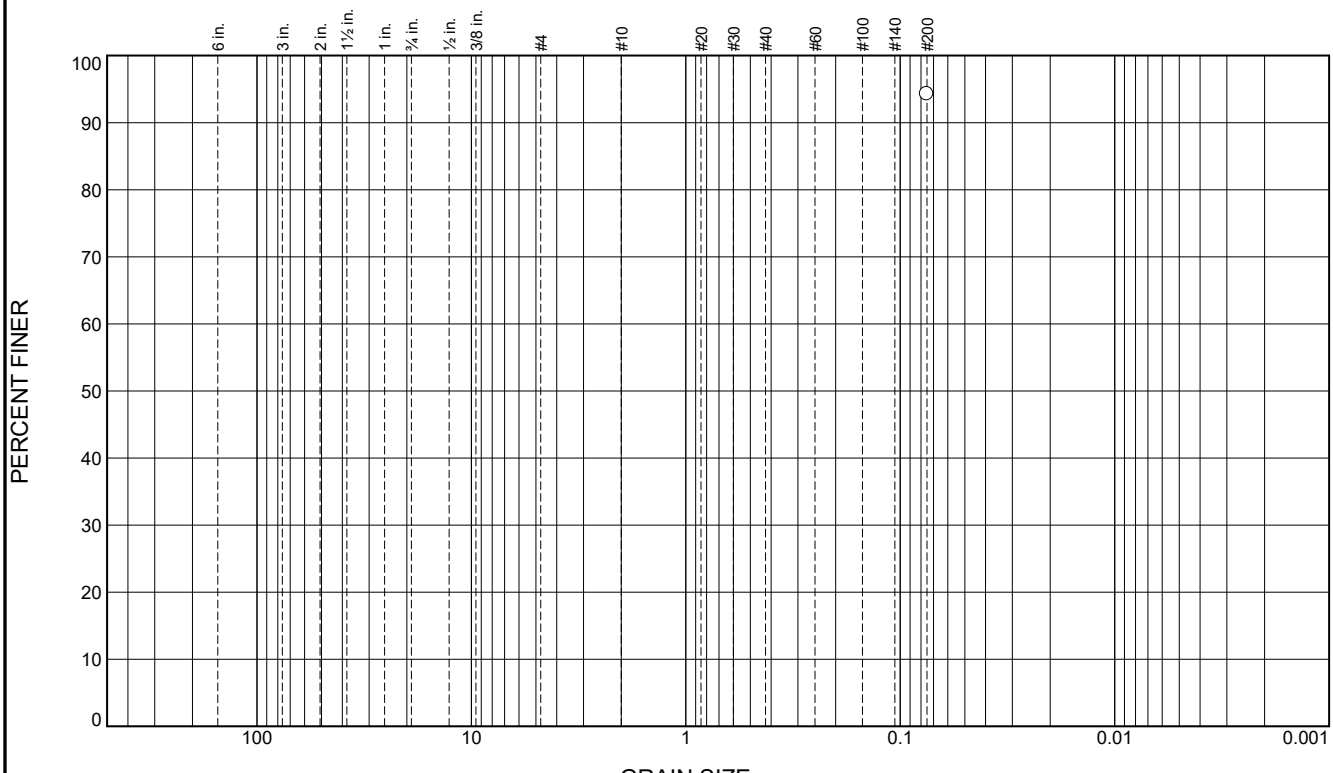
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Quasem **Checked By:** D. Seibold

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						94.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	94.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 23 LL= 26 PI= 3

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140
PI: ASTM D4318, Wet method

Sample Number: 1-B1 @ 26 **Depth:** 26.0 feet

Date: 3/1/18

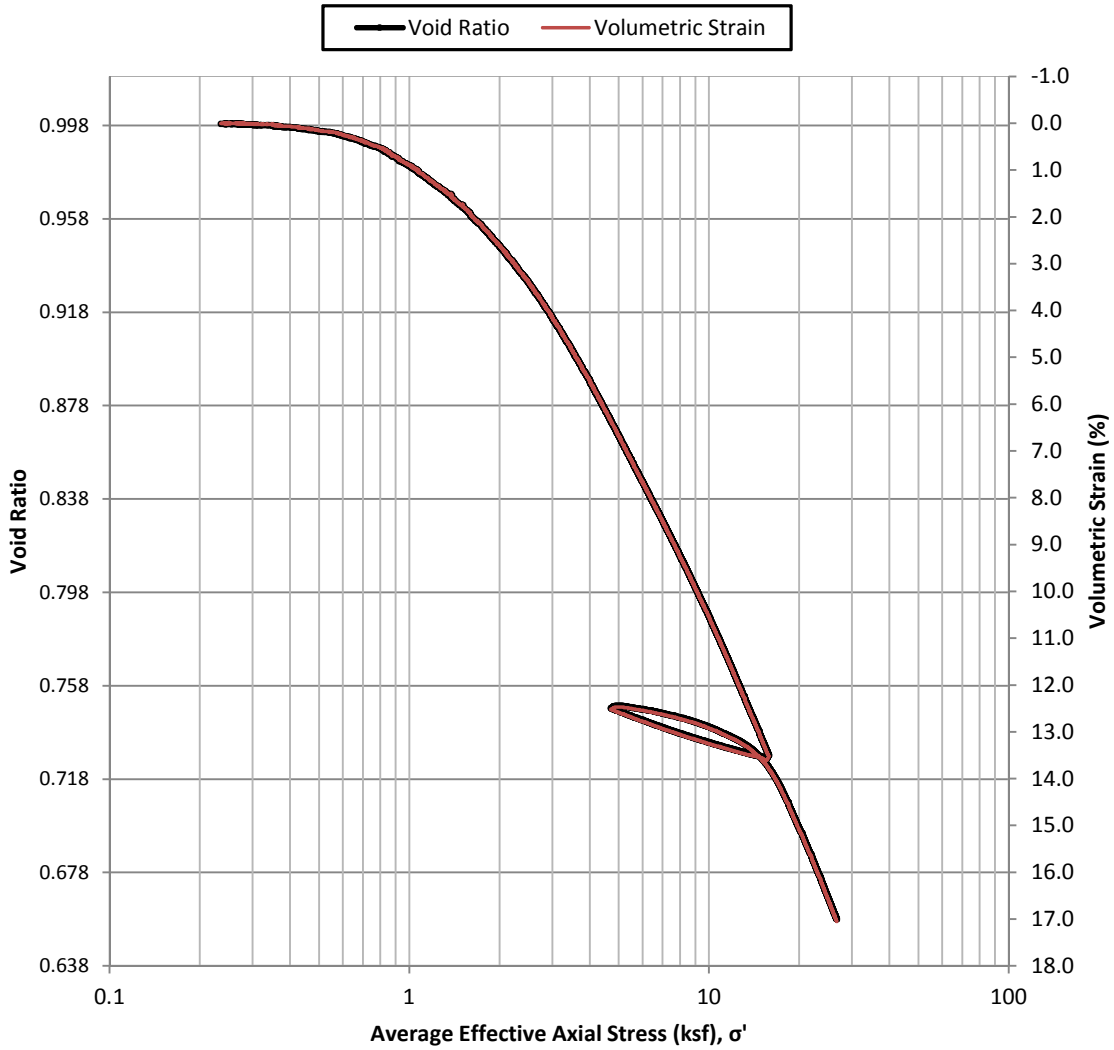
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Quasem **Checked By:** D. Seibold

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	35.59%	28.01%	Liquid Limit:	
Dry Density (pcf):	86.90	104.76	Plastic Limit:	
Saturation (%):	99.33%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.9987	0.6579	Specific Gravity:	2.787
			Soil Description:	See exploration logs

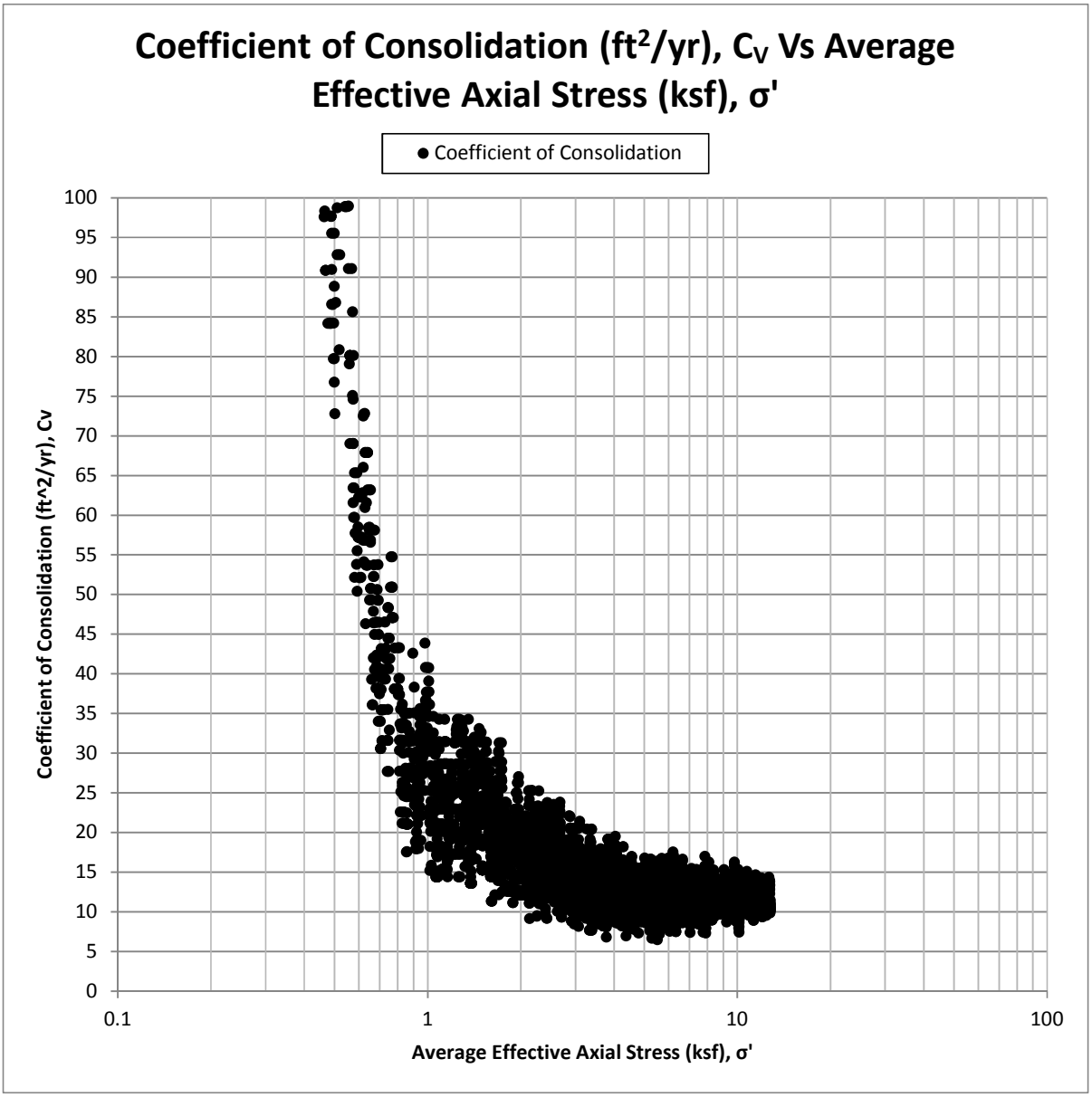
Test Date: 3/14/2018

Project Number:	13553.001.000	**Depth:**	14-14.5 ft
Sample Number:	1-B3 @ 12-15	**Boring #:**	1-B3
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks:

ENGEO
Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - 2974 Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	35.59%	28.01%	Liquid Limit:	
Dry Density (pcf):	86.90	104.76	Plastic Limit:	
Saturation (%):	99.33%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.9987	0.6579	Specific Gravity:	2.787
			Soil Description:	See exploration logs

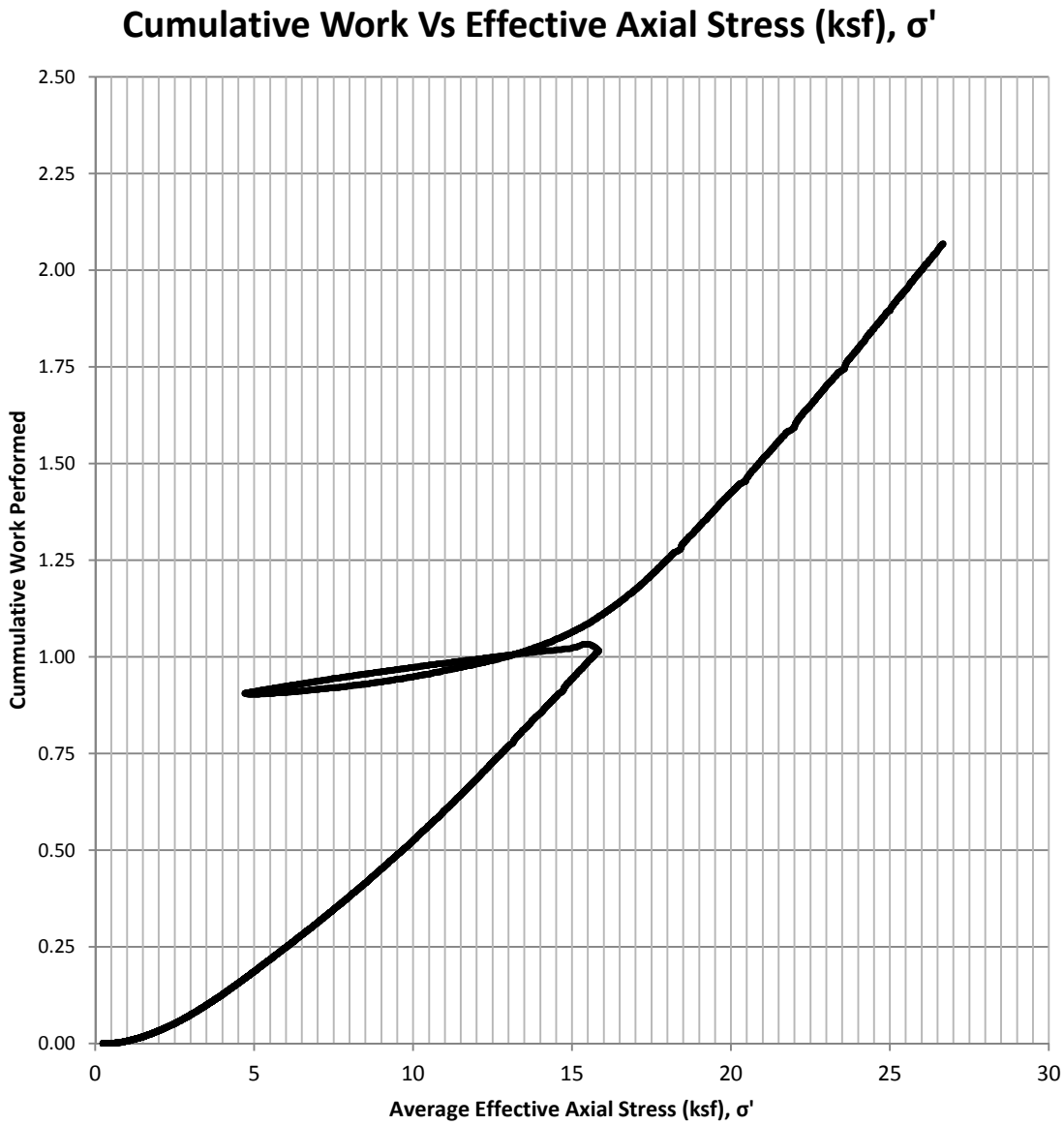
Test Date: 3/14/2018

Project Number:	13553.001.000	**Depth:**	14-14.5 ft
Sample Number:	1-B3 @ 12-15	**Boring #:**	1-B3
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks:

ENGEO Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	35.59%	28.01%	Liquid Limit:	
Dry Density (pcf):	86.90	104.76	Plastic Limit:	
Saturation (%):	99.33%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.9987	0.6579	Specific Gravity:	2.787
			Soil Description:	See exploration logs

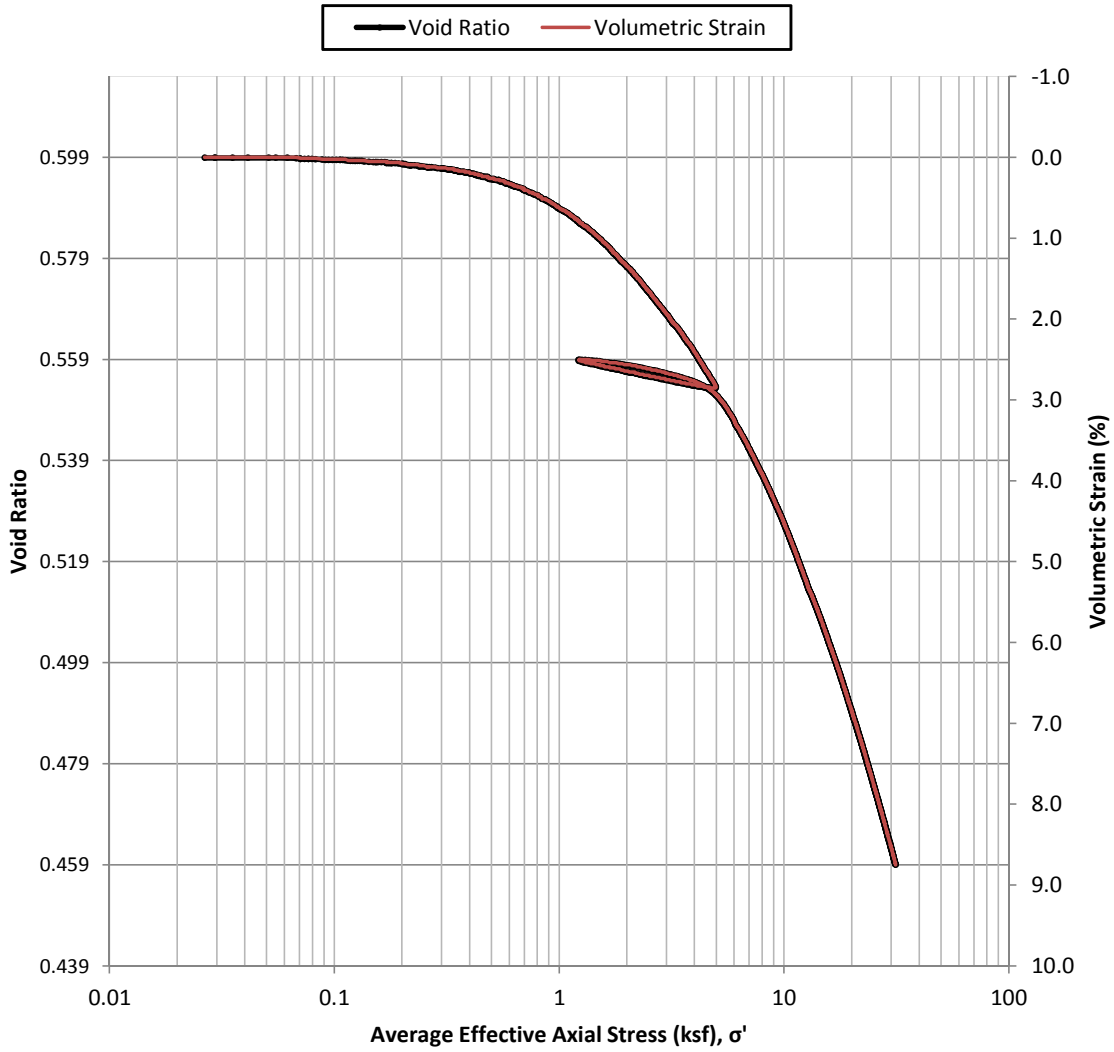
Test Date: 3/14/2018

Project Number:	13553.001.000	**Depth:**	14-14.5 ft
Sample Number:	1-B3 @ 12-15	**Boring #:**	1-B3
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks:

ENGEO Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	23.28%	21.37%	Liquid Limit:	
Dry Density (pcf):	104.73	114.77	Plastic Limit:	
Saturation (%):	104.43%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.5989	0.4590	Specific Gravity:	2.687
			Soil Description:	See exploration logs

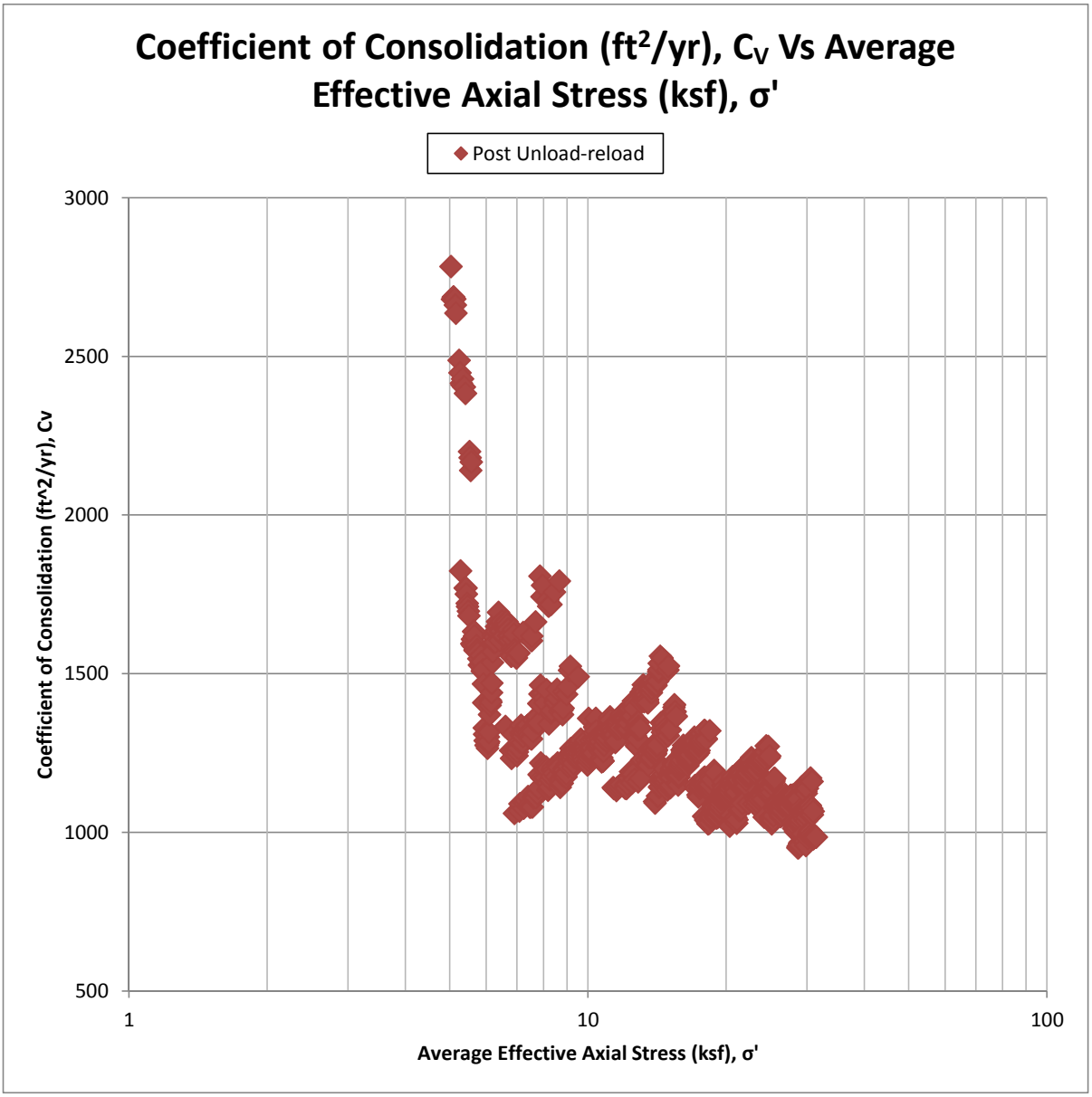
Test Date: 3/1/2018

Project Number:	12165.000.002	**Depth:**	21-21.25 ft
Sample Number:	1-B2 @ 19-22	**Boring #:**	1-B2
Project Name:	Sobrato Property - Area 4		
Client:	The Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

ENGEO
Expect Excellence

Remarks: Residual (free-standing) water was present during specimen preparation. 11.8% was retained on the #200 sieve.

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - 2974 Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	23.28%	21.37%	Liquid Limit:	
Dry Density (pcf):	104.73	114.77	Plastic Limit:	
Saturation (%):	104.43%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.5989	0.4590	Specific Gravity:	2.687
			Soil Description:	See exploration logs

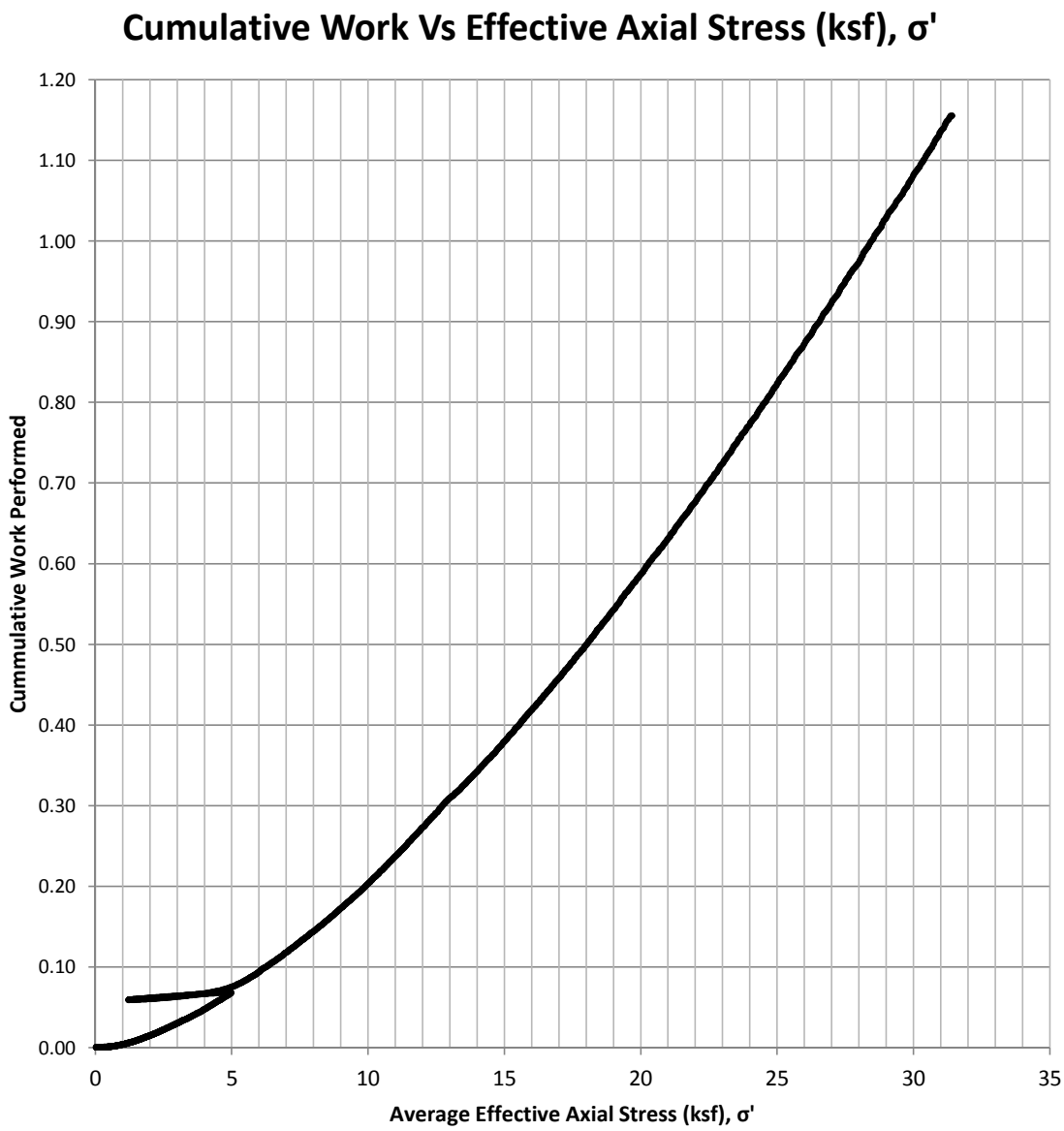
Test Date: 3/1/2018

Project Number:	12165.000.002	**Depth:**	21-21.25 ft
Sample Number:	1-B2 @ 19-22	**Boring #:**	1-B2
Project Name:	Sobrato Property - Area 4		
Client:	The Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks: Residual (free-standing) water was present during specimen preparation. 11.8% was retained on the #200 sieve.

ENGEO Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	23.28%	21.37%	Liquid Limit:	
Dry Density (pcf):	104.73	114.77	Plastic Limit:	
Saturation (%):	104.43%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.5989	0.4590	Specific Gravity:	2.687
			Soil Description:	See exploration logs

Test Date: 3/1/2018

Project Number:	12165.000.002	**Depth:**	21-21.25 ft
Sample Number:	1-B2 @ 19-22	**Boring #:**	1-B2
Project Name:	Sobrato Property - Area 4		
Client:	The Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks: Residual (free-standing) water was present during specimen preparation. 11.8% was retained on the #200 sieve.

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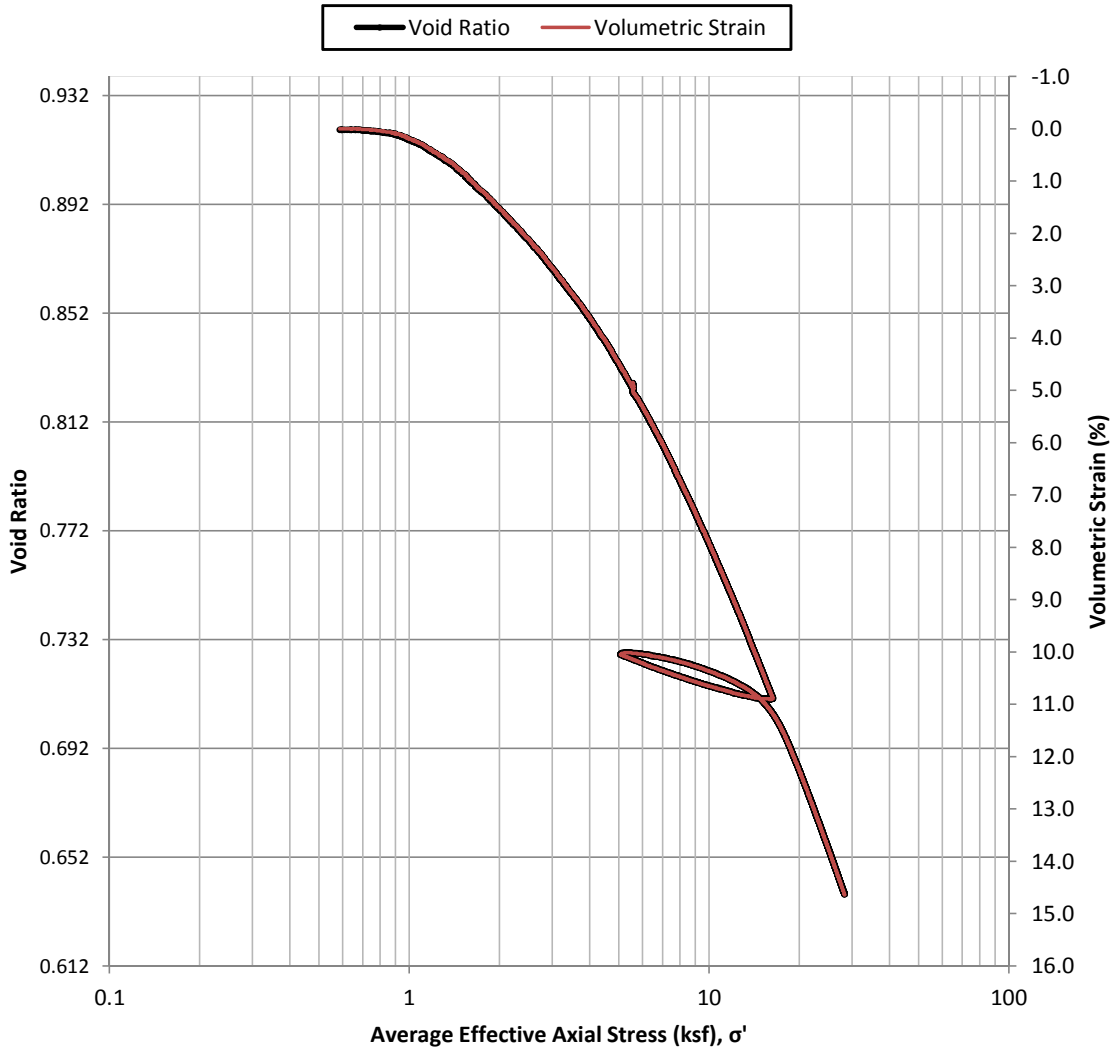
Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	35.54%	27.40%	Liquid Limit:	
Dry Density (pcf):	90.06	105.51	Plastic Limit:	
Saturation (%):	107.21%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.9195	0.6384	Specific Gravity:	2.774
			Soil Description:	See exploration logs

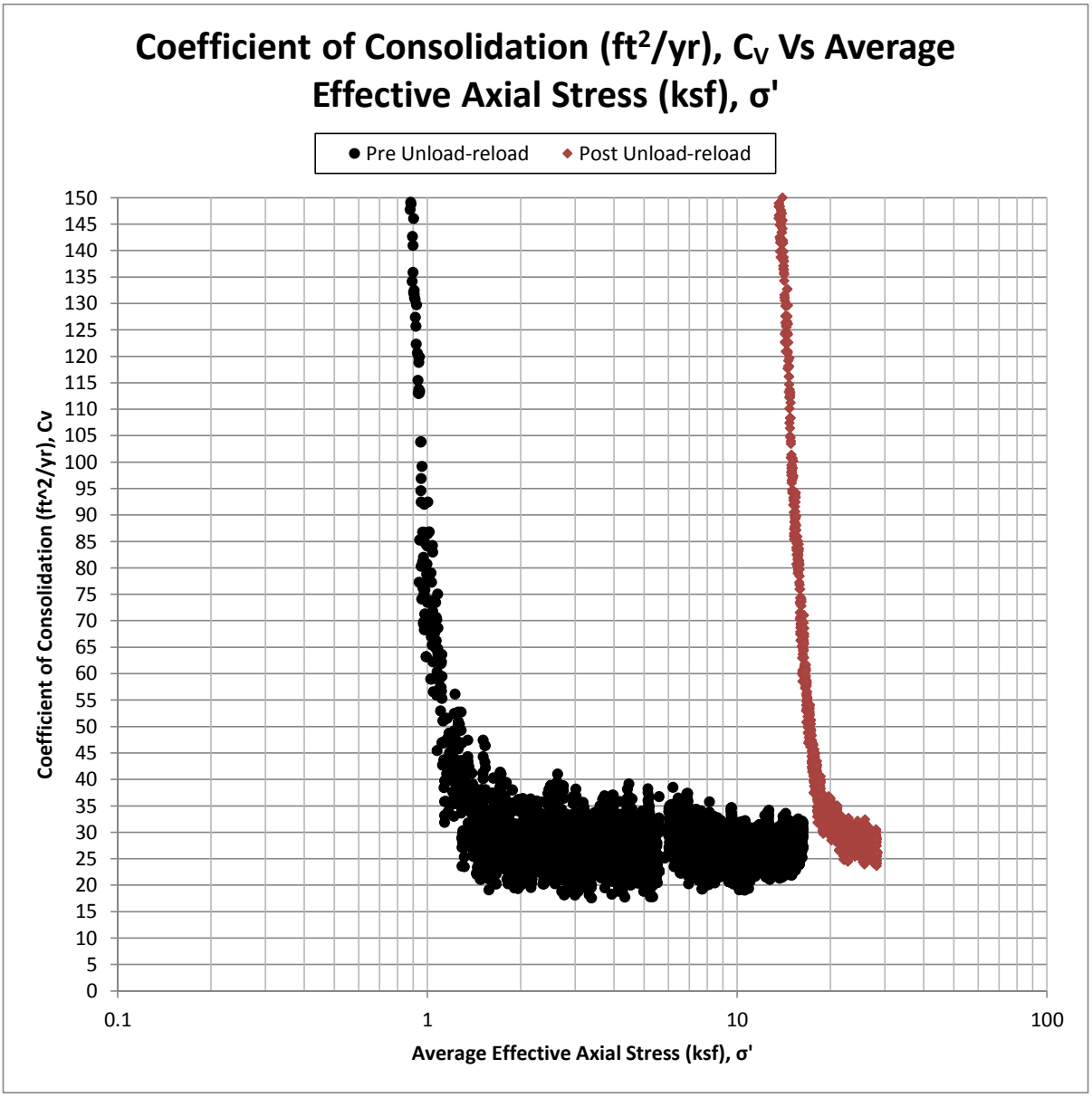
Test Date: 3/8/2018

Project Number:	12165.000.002	**Depth:**	20-20.25 ft
Sample Number:	1-B1 @ 19-22	**Boring #:**	1-B1
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

ENGEO Expect Excellence

Remarks: Initial specimen saturation may be greater than 100% due to the presence of organic material.

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - 2974 Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	35.54%	27.40%	Liquid Limit:	
Dry Density (pcf):	90.06	105.51	Plastic Limit:	
Saturation (%):	107.21%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.9195	0.6384	Specific Gravity:	2.774
			Soil Description:	See exploration logs

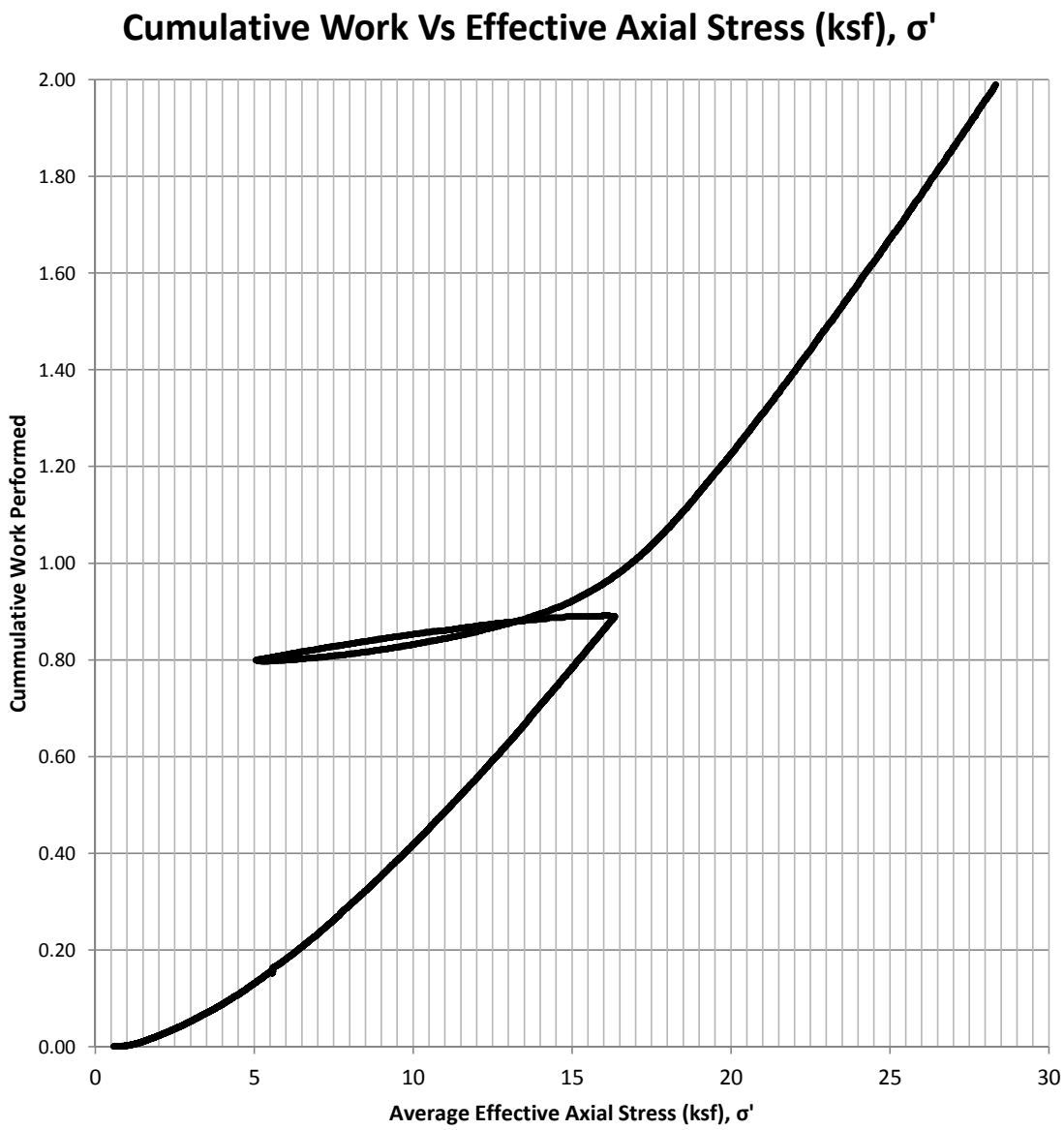
Test Date: 3/8/2018

Project Number:	12165.000.002	**Depth:**	20-20.25 ft
Sample Number:	1-B1 @ 19-22	**Boring #:**	1-B1
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks: Initial specimen saturation may be greater than 100% due to the presence of organic material.

ENGEO Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	35.54%	27.40%	Liquid Limit:	
Dry Density (pcf):	90.06	105.51	Plastic Limit:	
Saturation (%):	107.21%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.9195	0.6384	Specific Gravity:	2.774
			Soil Description:	See exploration logs

Test Date: 3/8/2018

Project Number:	12165.000.002	**Depth:**	20-20.25 ft
Sample Number:	1-B1 @ 19-22	**Boring #:**	1-B1
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks: Initial specimen saturation may be greater than 100% due to the presence of organic material.

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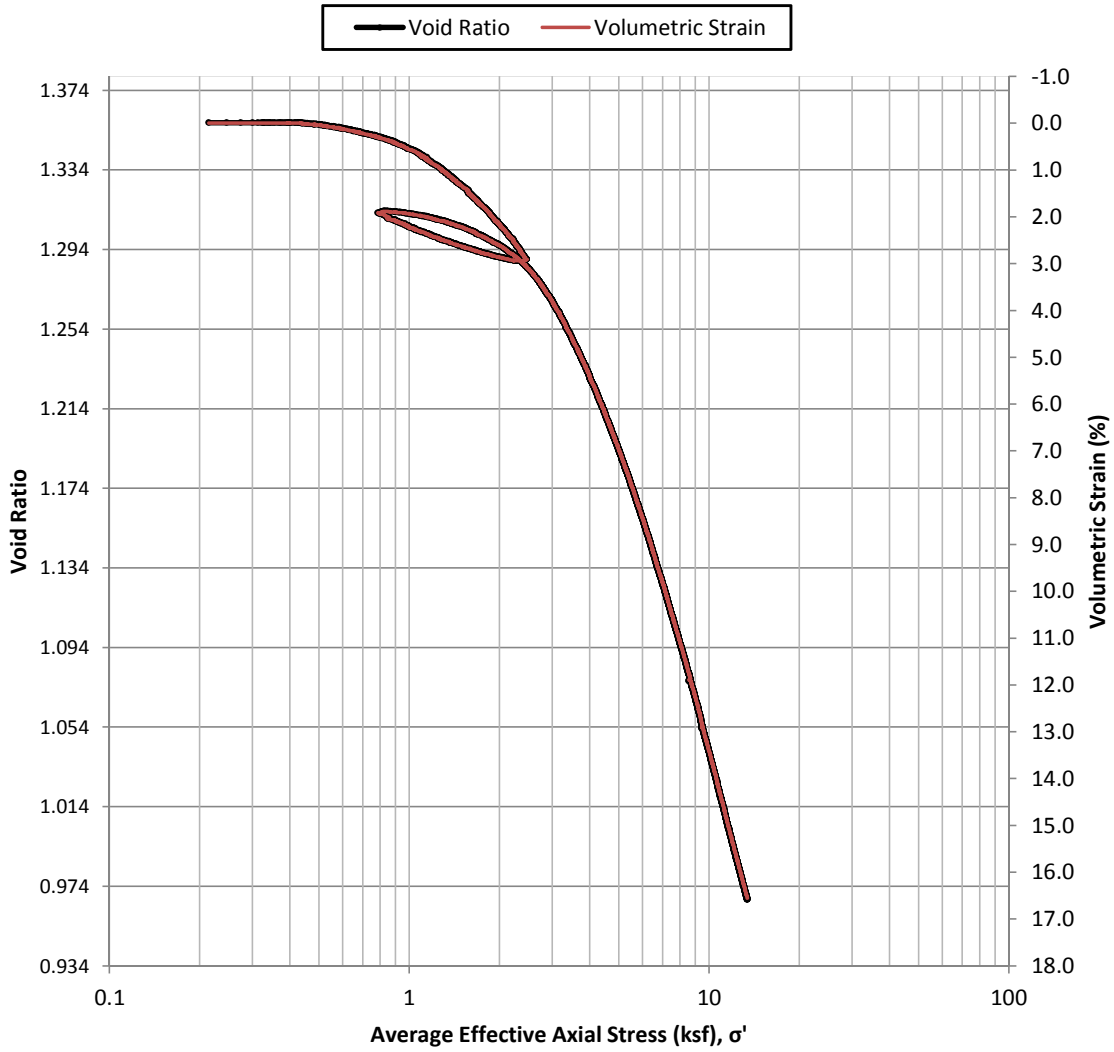
Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	48.11%	43.84%	Liquid Limit:	
Dry Density (pcf):	71.02	85.11	Plastic Limit:	
Saturation (%):	95.22%	100.00%	ASTM D854 - Measured	
Void Ratio:	1.3576	0.9674	Specific Gravity:	2.687
			Soil Description:	See exploration logs

Test Date: 2/28/2018

Project Number:	12165.000.002	**Depth:**	16-16.5 ft
Sample Number:	1-B1 @ 14-17	**Boring #:**	1-B1
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks:

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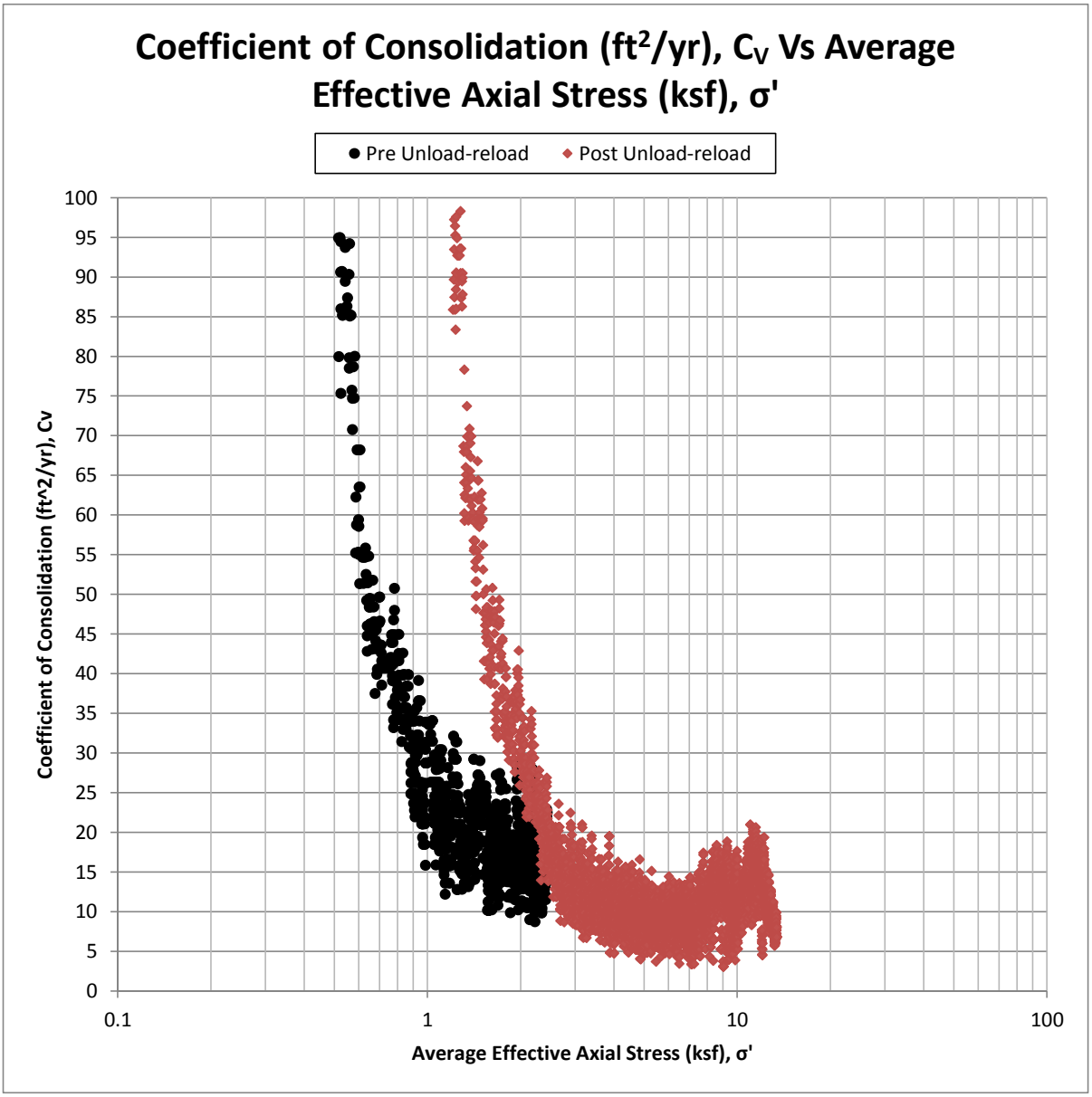
Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - 2974 Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	48.11%	43.84%	Liquid Limit:	
Dry Density (pcf):	71.02	85.11	Plastic Limit:	
Saturation (%):	95.22%	100.00%	ASTM D854 - Measured	
Void Ratio:	1.3576	0.9674	Specific Gravity:	2.687
			Soil Description:	See exploration logs

Test Date: 2/28/2018

Project Number:	12165.000.002	**Depth:**	16-16.5 ft
Sample Number:	1-B1 @ 14-17	**Boring #:**	1-B1
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks:

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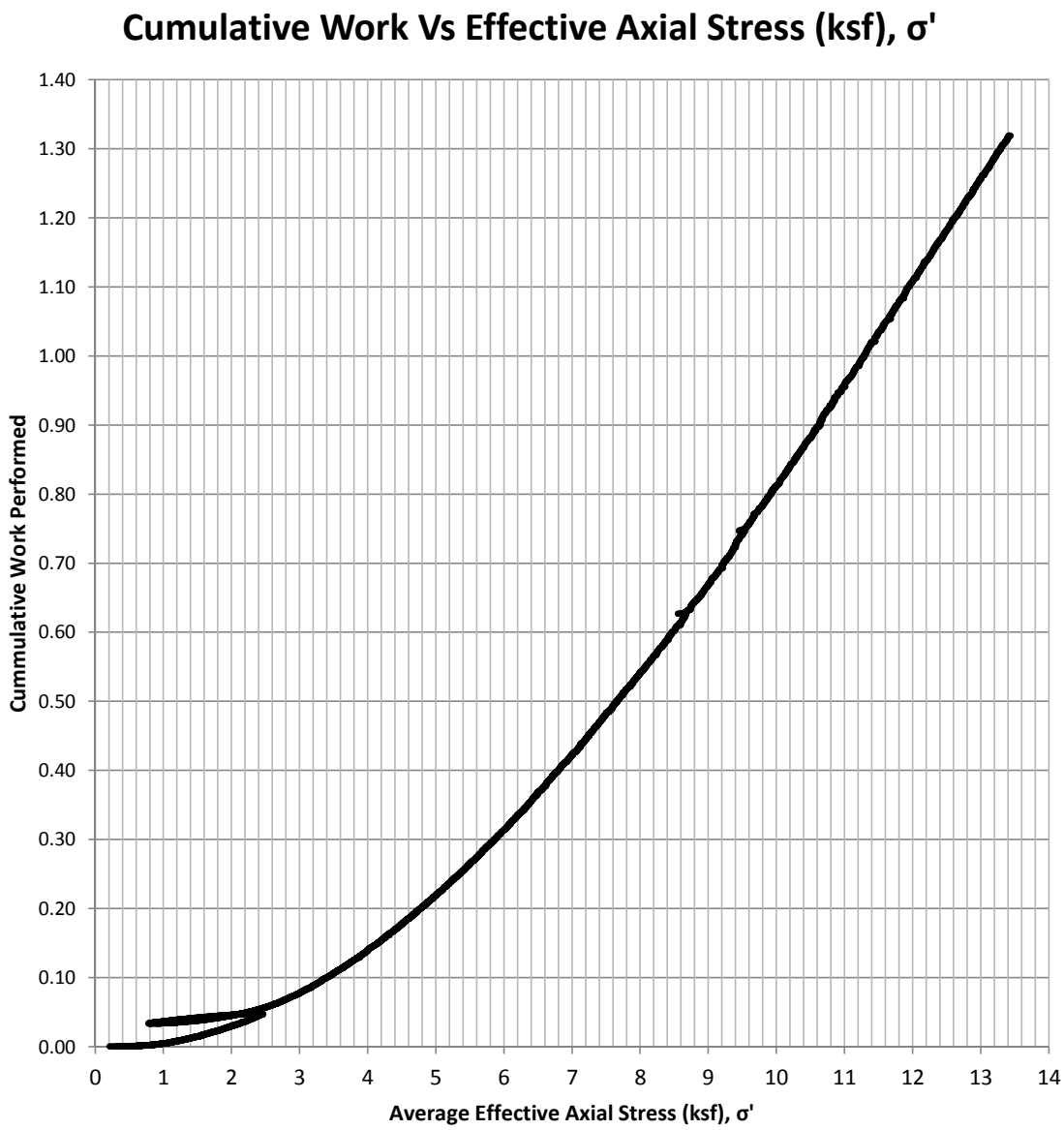
Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	48.11%	43.84%	Liquid Limit:	
Dry Density (pcf):	71.02	85.11	Plastic Limit:	
Saturation (%):	95.22%	100.00%	ASTM D854 - Measured	
Void Ratio:	1.3576	0.9674	Specific Gravity:	2.687
			Soil Description:	See exploration logs

Test Date: 2/28/2018

Project Number:	12165.000.002	**Depth:**	16-16.5 ft
Sample Number:	1-B1 @ 14-17	**Boring #:**	1-B1
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks:

ENGEO Expect Excellence

LABORATORY MINIATURE VANE SHEAR

ASTM D4648

APPARATUS USED: Wykeham Farrance, Model 27-WF1730/4

Sample #	Sample ID	Remold? (Y/N)	Test depth (ft)	Spring number	Shear strength (psf)
1	1-B3 @ 21	N	21.0-21.5	4	2085

Testing remarks:

PROJECT NAME: Sobrato Property - Area 4

DATE: 03/12/18

PROJECT NUMBER: 12165.000.002

CLIENT: The Sobrato Organization

PHASE NUMBER: 001

ENGEO Expect Excellence

Tested by: M. Quasem

Reviewed by: G. Criste

LABORATORY MINIATURE VANE SHEAR

ASTM D4648

APPARATUS USED: Wykeham Farrance, Model 27-WF1730/4

Sample #	Sample ID	Remold? (Y/N)	Test depth (ft)	Spring number	Shear strength (psf)
1	1-B1 @ 21	N	21.0-21.5	3	863

Testing remarks:

PROJECT NAME: Sobrato Property - Area 4
PROJECT NUMBER: 12165.000.002
CLIENT: The Sobrato Organization
PHASE NUMBER: 001

DATE: 03/02/18

ENGEO
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Tested by: M. Quasem

Reviewed by: G. Criste

EN GEO

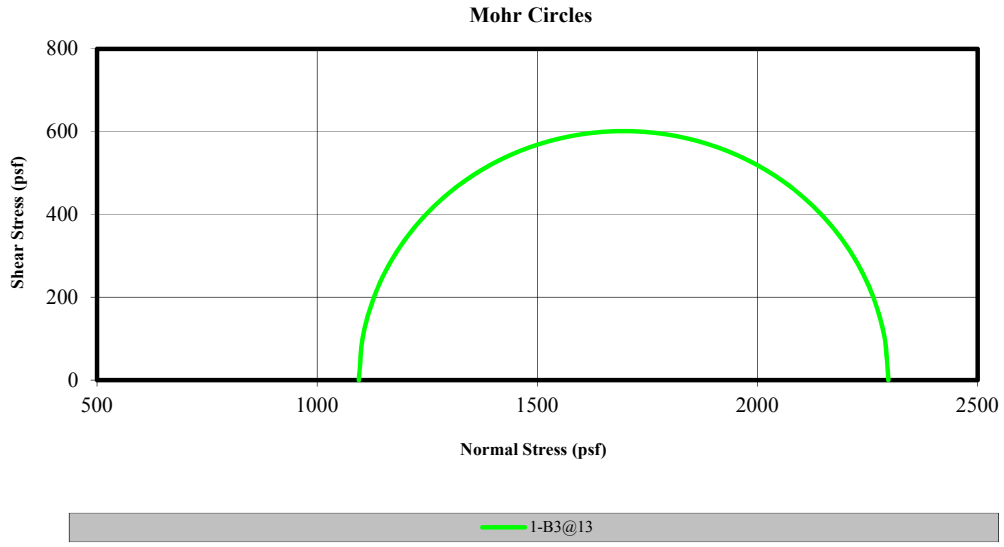
Unconsolidated Undrained Triaxial Test (ASTM D2850)

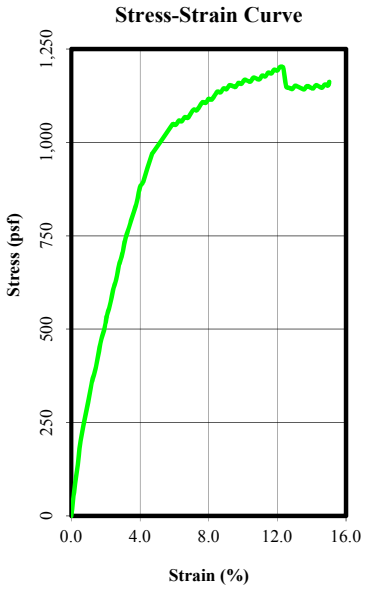
Date: 03/20/18

Checked By: G. Criste

Date: 03/20/18

Tested By: M. Quasem





Before Test	Specimen 1-B3@13			
Water Content (%)	29.88			
Dry Density (pcf)	93.03			
Saturation (%)	100.00			
Void Ratio	0.78			
Diameter (in)	2.855			
Height (in)	5.790			
Liquid Limit	-			
Plastic Limit	-			
Specific Gravity (Assumed)	2.650			
Height-to-Diameter Ratio	2.028			
After Test	**1-B3@13**			
Water Content (%)	29.88			
Saturation (%)	100.00			
Strain Rate (in/min)	0.05			
Peak Deviator Stress (psf)	1202.6			
Axial Strain @ Failure (%)	12.171			
	Cell Pressure			
Cell (psf)	1094.4			
Back (psf)	n/a			
	Principle Stresses at Failure			
σ1 (psf)	2297.0			
σ3 (psf)	1094.4			

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	0.0		601.3			
Friction Angle Ø	0.00		n/a			

Project Information			
Project Name:	Sobrato Property - Area 4		
Project Number:	12165.000.002	Job Number:	12165.000.002
Location:	Newark, CA	Boring Number:	1-B3
Client:	The Sobrato Organization	Sample Number:	1-B3 @ 13
Description:	See exploration logs		

EN GEO

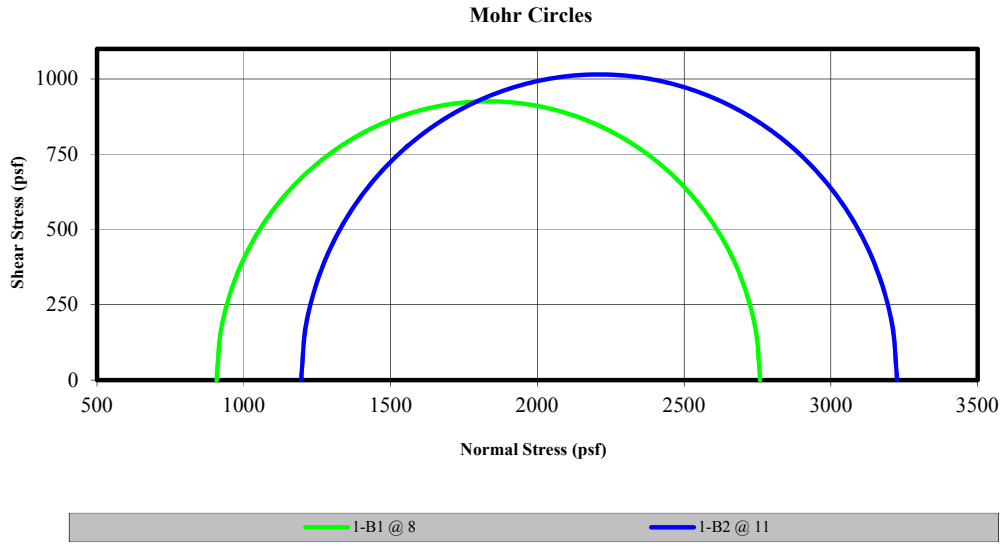
Unconsolidated Undrained Triaxial Test (ASTM D2850)

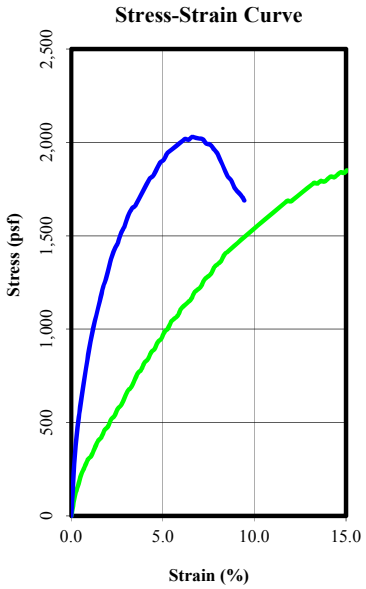
Date: 03/02/18

Checked By: D. Seibold

Date: 03/02/18

Tested By: M. Quasem



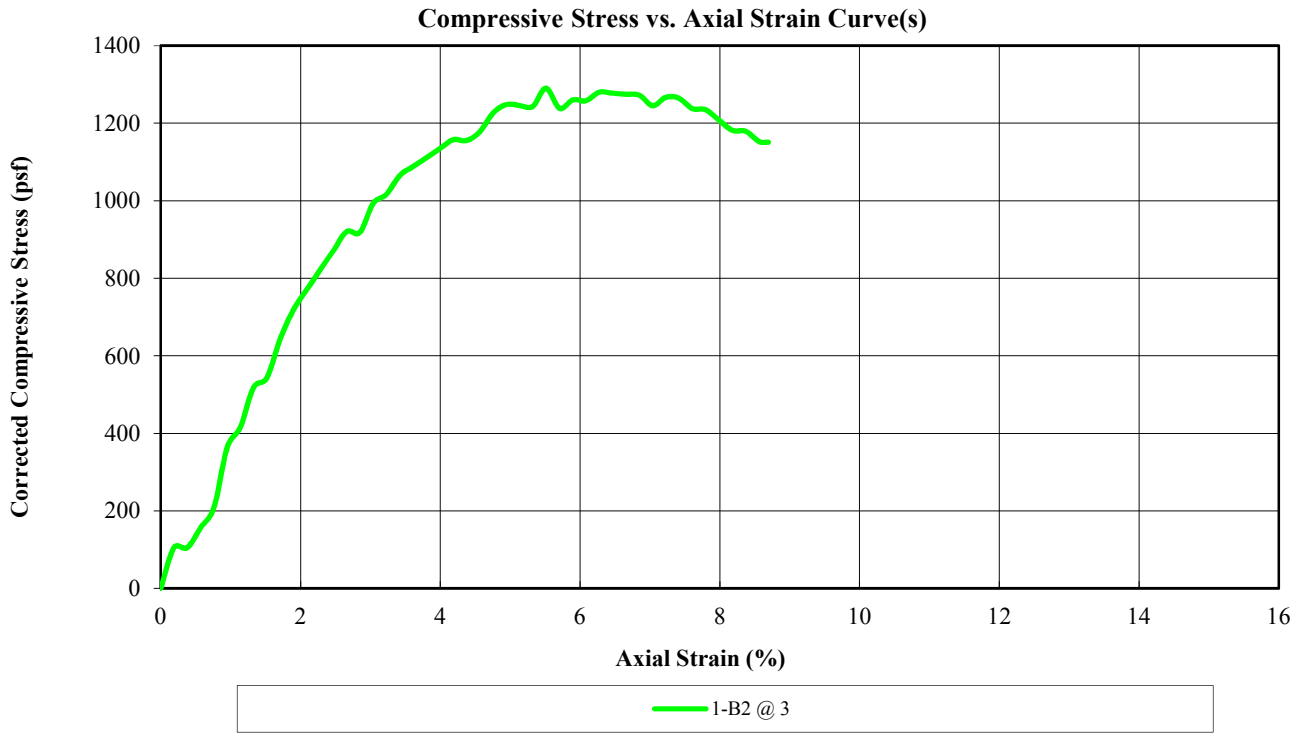


Before Test	Specimen 1-B1 @ 8	1-B2 @ 11		
Water Content (%)	33.60	39.29		
Dry Density (pcf)	90.51	83.37		
Saturation (%)	100.00	100.00		
Void Ratio	0.83	0.98		
Diameter (in)	2.377	2.369		
Height (in)	5.540	5.185		
Liquid Limit	-	-		
Plastic Limit	-	-		
Specific Gravity	2.650	2.650		
Height-to-Diameter Ratio	2.331	2.189		
After Test	**1-B1 @ 8**	**1-B2 @ 11**		
Water Content (%)	33.60	39.29		
Saturation (%)	100.00	100.00		
Strain Rate (in/min)	0.05	0.05		
Peak Deviator Stress (psf)	1851.2	2029.7		
Axial Strain @ Failure (%)	15.042	6.601		
	Cell Pressure			
Cell (psf)	907.2	1195.2		
Back (psf)	n/a	n/a		
	Principle Stresses at Failure			
σ1 (psf)	2758.4	3224.9		
σ3 (psf)	907.2	1195.2		

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	0.0		925.6	1014.9		
Friction Angle Ø	0.00		n/a	n/a		

Project Information			
Project Name:	Sobrato Property - Area 4		
Project Number:	12165.000.002	Job Number:	12165.000.002
Location:	Newark, CA	Boring Number:	Multiple
Client:	The Sobrato Organization	Sample Number:	Multiple
Description:	See exploration logs		

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B2 @ 3
Moisture Content (%)	20.2
Dry Density (pcf)	106.6
Saturation (%)	97.0
Void Ratio	0.55
Diameter (in)	2.399
Height (in)	5.30
Height-To-Diameter Ratio	2.21
TEST DATA	
Unconfined Compressive Strength (psf)	1290
Undrained Shear Strength (psf)	645
Strain Rate (in./min.)	0.05
Specific Gravity	2.650
Strain at Failure (%)	5.51
Liquid Limit	-
Plastic Limit	-
Test Remarks	

SPECIMEN	DESCRIPTION
1-B2 @ 8.5	See exploration logs

PROJECT NAME: Sobrato Property - Area 4
PROJECT NO: 12165.000.002
CLIENT: The Sobrato Organization
LOCATION: Newark, CA
PHASE NO: 001

Test Date: 02/28/18
Tested By: M. Quasem
Reviewed By: D. Seibold

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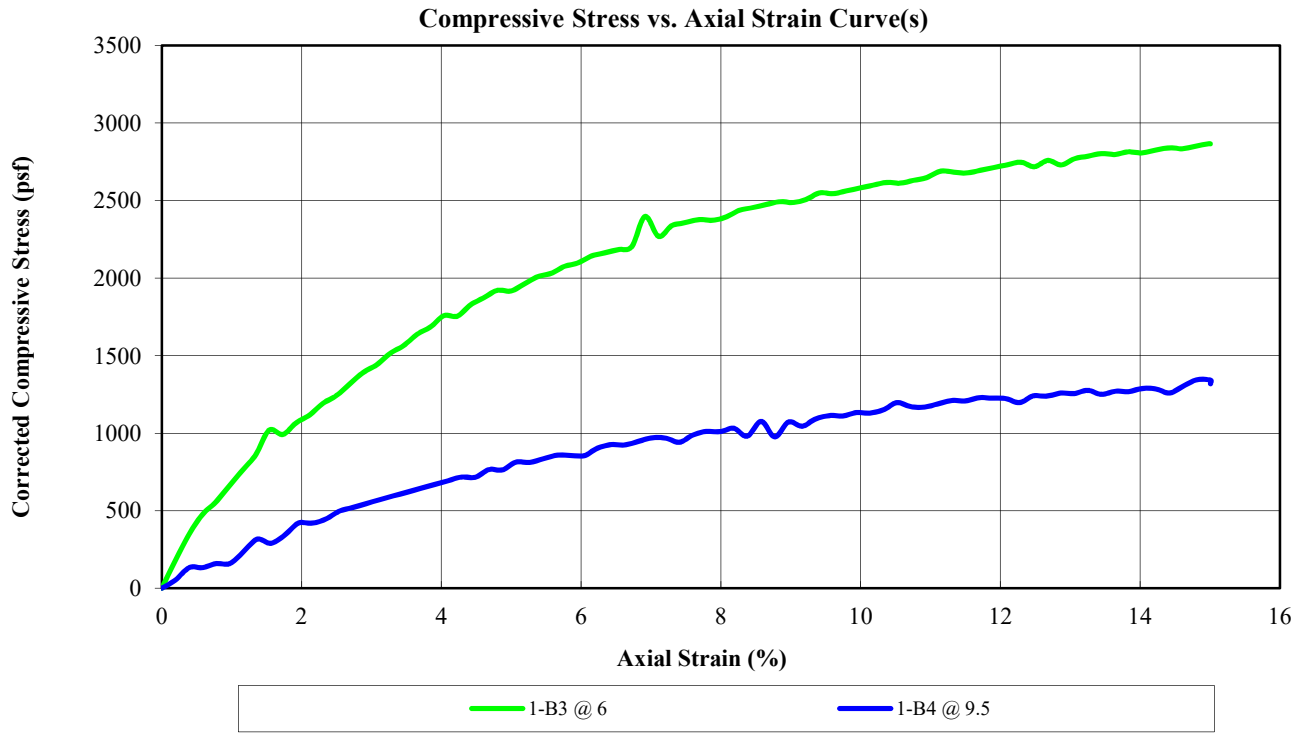
UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



BEFORE TEST	SPECIMEN 1-B3 @ 6	1-B4 @ 9.5
Moisture Content (%)	27.1	31.6
Dry Density (pcf)	99.1	92.3
Saturation (%)	99.9	98.6
Void Ratio	0.76	0.89
Diameter (in)	2.386	2.377
Height (in)	5.25	5.18
Height-To-Diameter Ratio	2.20	2.18
TEST DATA		
Unconfined Compressive Strength (psf)	2867	1345
Undrained Shear Strength (psf)	1433	672
Strain Rate (in./min.)	0.05	0.05
Specific Gravity (Assumed)	2.790	2.790
Strain at Failure (%)	14.98	14.81
Liquid Limit		
Plastic Limit		
Test Remarks		

SPECIMEN	DESCRIPTION
1-B3 @ 6	See exploration logs
1-B4 @ 9.5	See exploration logs

PROJECT NAME: Sobrato Property - Area 4
PROJECT NO: 12165.000.002
CLIENT: The Sobrato Organization
LOCATION: Newark, California
PHASE NO: 001

Test Date: 03/09/18
Tested By: M. Quasem
Reviewed By: G. Criste

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Organic Matter of Peat & Other Organic Soils

ASTM D2974

SAMPLE NO.	LOCATION/BORING ID	DEPTH (ft)	Moisture Content	ASH CONTENT %	ORGANIC MATTER %
1	1-B2	16.5	202.1	57.9	42.1

Testing remarks: Moisture Content (Method A); Ash Content (Method C)

PROJECT NAME: Sobrato Property - Area 4
PROJECT NUMBER: 12165.000.002
CLIENT: The Sobrato Organization
PHASE NUMBER: 001

DATE: 03/01/18

ENGEO
Expect Excellence

Tested by: M. Quasem

Reviewed by: G. Criste

Lab Address: 3420 Fostoria Way, Suite E, San Ramon, CA 94583 Phone No. (925) 355-9047.

WATER SOLUBLE SULFATES IN SOILS

ASTM C1580

Sample number	Sample Location / ID	Matrix	Water Soluble Sulfate % by mass
1	1-B1 @ 3.5	soil	ND

Remarks: Results are reported to the nearest 100mg/kg. Anything less than 50mg/kg will be reported as 'ND' for Not-Detectable.

PROJECT NAME: Sobrato Property - Area 4
PROJECT NUMBER: 12165.000.002
CLIENT: The Sobrato Organization
PHASE NUMBER: 001

DATE: 02/28/18

ENGEO
Expect Excellence

Tested by: M. Quasem

Reviewed by: D. Seibold

Lab Address: 3420 Fostoria Way Suite E, San Ramon, CA 94583. Phone No. (925) 355-9047

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: ENGEO Incorporated
Client's Project No.: 12165.000.002
Client's Project Name: Sobrato-Area 4
Date Sampled: 8-Mar-18
Date Received: 9-Mar-18
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 19-Mar-2018

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1803074-001	1-B4 @ 3'	220	8.01	-	150	N.D.	1,100	190

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	13-Mar-2018	13-Mar-2018	-	13-Mar-2018	19-Mar-2018	13-Mar-2018	13-Mar-2018

* Results Reported on "As Received" Basis

N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX D

LIQUEFACTION ANALYSIS

Liquefaction Evaluation - Youd 2001, Seed 2003, I&B 2008 Methods -

Water Table depth at time of Exploration	Water Table depth at time of Liquefaction	amax/g	Mw	Estimated Unit Weight
5	5	0.59	7.3	115.0

Dry	Liq	Total Settlement
0.00	5.22	5.22

						At time of Exploration		At time of Liquefaction	
Boring Designation	Depth [ft]	Layer Thickness [ft]	Soil Type	N_m [Blows/ft]	FC	Total Stress [psf]	Effective Stress [psf]	Total Stress [psf]	Effective Stress [psf]
1-B4	5	10	CL	8	100	575	575	575	575
1-B4	13	6	CH	4	100	1495	995.8	1495	995.8
1-B4	16.5	1	ML	4	80	1897.5	1179.9	1897.5	1179.9
1-B4	18.5	3	SC	4	20	2127.5	1285.1	2127.5	1285.1
1-B4	25.25	10.5	SM	15	40	2903.75	1640	2903.75	1640
1-B4	33	5	CL	26	75	3795	2047.8	3795	2047.8
1-B4	37.5	4	ML	12	85	4312.5	2284.5	4312.5	2284.5

N_m = Measured SPT Blow Count

YOUD 2001 Methodology Results

Boring Designation	Depth	CRR	CSR	FS
1-B4	5	0.24	0.38	0.62
1-B4	13	0.14	0.56	0.25
1-B4	16.5	0.14	0.60	0.23
1-B4	18.5	0.12	0.61	0.19
1-B4	25.25	0.44	0.64	0.68
1-B4	33	TDL	0.64	TDL
1-B4	37.5	0.25	0.63	0.40

TDL = Too Dense to Liquefy based on blowcount criteria

SEED 2003 Methodology Results

			CSR			Calculated FS		
Boring Designation	Depth	CRR	mean rd	rd + sigma	rd - sigma	mean rd	rd + sigma	rd - sigma
1-B4	5	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A	#N/A
1-B4	13	0.12	0.18	0.21	0.15	0.67	0.58	0.80
1-B4	16.5	0.10	0.16	0.20	0.12	0.61	0.49	0.79
1-B4	18.5	0.07	0.15	0.20	0.11	0.45	0.35	0.63
1-B4	25.25	0.26	0.13	0.20	0.06	1.97	1.29	FS>2.5
1-B4	33	1.17	0.11	0.21	0.02	FS>2.5	FS>2.5	FS>2.5
1-B4	37.5	0.20	0.11	0.22	-0.01	1.85	0.90	-36.96

THC = CRR capped at 4, in high seismicity cases, verify

Idriss & Boulanger 2008 Methodology Results

Boring Designation	Depth	CRR	CSR	FS
1-B4	5	0.19	0.33	0.57
1-B4	13	0.13	0.51	0.26
1-B4	16.5	0.13	0.54	0.24
1-B4	18.5	0.12	0.56	0.22
1-B4	25.25	0.30	0.58	0.52
1-B4	33	1.80	0.60	FS>2.5
1-B4	37.5	0.20	0.60	0.33

THC = CRR capped at 4, in high seismicity cases, verify

Liquefaction Evaluation - Idriss and Boulanger (2008)* *Limited to depths of 40ft.

Input

Water Table depth at time of Exploration	Water Table depth at time of Liquefaction	amax/g	Mw
8.5	7	0.594	7.33

						At time of Exploration		At time of Liquefaction	
Boring Designation	Depth [m]	Depth [ft]	Soil Type	N_m [Blows/ft]	FC	Total Stress [psf]	Effective Stress [psf]	Total Stress [psf]	Effective Stress [psf]
1-B1	23.5	8	ML	4	90	2702.5	1766.5	2702.5	1672.9
1-B2	20.25	4.5	ML	4	90	2328.75	1595.55	2328.75	1501.95
1-B3	44	6	ML	8	60	5060	2844.8	5060	2751.2
1-B4	16.5	1	ML	4	80	1897.5	1398.3	1897.5	1304.7
1-B4	18.5	3	SC	4	20	2127.5	1503.5	2127.5	1409.9
1-B4	25.25	10.5	SM	15	40	2903.75	1858.55	2903.75	1764.95
1-B4	37.5	4	ML	12	85	4312.5	2502.9	4312.5	2409.3

N_m = Measured SPT Blow Count

Results

Boring Designation	Depth [ft]	CRR	CSR	FS	Ht. of Layer (ft)	(N1)60cs	Soil Type (USCS)	Liquefiable?	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔLDI	Volumetric Strain Used ε_v	ΔS_i
1-B1	23.5	0.12	0.54	0.21	8.00	10	ML	yes	48%	0.92	48%	3.83	1.50%	1.44
1-B2	20.3	0.12	0.52	0.23	4.50	10	ML	yes	45%	0.90	45%	2.02	1.50%	0.81
1-B3	44.0	0.14	0.57	0.24	6.00	14	ML	yes	32%	0.80	32%	1.89	1.50%	1.08
1-B4	16.5	0.13	0.54	0.24	1.00	12	ML	yes	39%	0.87	39%	0.39	1.50%	0.18
1-B4	18.5	0.12	0.56	0.22	3.00	10	SC	Yes	48%	0.92	48%	1.45	1.50%	0.54
1-B4	25.3	0.30	0.58	0.52	10.50	29	SM	Yes	5%	-0.03	5%	0.55	3.00%	3.78
1-B4	37.5	0.20	0.60	0.33	4.00	22	ML	yes	13%	0.43	13%	0.53	1.50%	0.72

TDL = Too Dense to Liquefy based on blowcount criteria
Volumetric Strains based on Figure 103

APPENDIX E

SUPPLEMENTAL RECOMMENDATIONS

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GENERAL INFORMATION

PREFACE

These supplemental recommendations are intended as a guide for earthwork and are in addition to any previous earthwork recommendations made by the Geotechnical Engineer. If there is a conflict between these supplemental recommendations and any previous recommendations, it should be immediately brought to the attention of ENGEO. Testing standards identified in this document shall be the most current revision (unless stated otherwise).

DEFINITIONS

BACKFILL	Soil, rock or soil-rock material used to fill excavations and trenches.
DRAWINGS	Documents approved for construction which describe the work.
THE GEOTECHNICAL ENGINEER	The project geotechnical engineering consulting firm, its employees, or its designated representatives.
ENGINEERED FILL	Fill upon which the Geotechnical Engineer has made sufficient observations and tests to confirm that the fill has been placed and compacted in accordance with geotechnical engineering recommendations.
FILL	Soil, rock, or soil-rock materials placed to raise the grades of the site or to backfill excavations.
IMPORTED MATERIAL	Soil and/or rock material which is brought to the site from offsite areas.
ONSITE MATERIAL	Soil and/or rock material which is obtained from the site.
OPTIMUM MOISTURE	Water content, percentage by dry weight, corresponding to the maximum dry density as determined by ASTM D-1557.
RELATIVE COMPACTION	The ratio, expressed as a percentage, of the in-place dry density of the fill or backfill material as compacted in the field to the maximum dry density of the same material as determined by ASTM D-1557.
SELECT MATERIAL	Onsite and/or imported material which is approved by the Geotechnical Engineer as a specific-purpose fill.

PART I - EARTHWORK

1.0 GENERAL

1.1 WORK COVERED

Supplemental recommendations for performing earthwork and grading. Activities include:

- ✓ Site Preparation and Demolition
- ✓ Excavation
- ✓ Grading
- ✓ Backfill of Excavations and Trenches
- ✓ Engineered Fill Placement, Moisture Conditioning, and Compaction

1.2 CODES AND STANDARDS

The contractor should perform their work complying with applicable occupational safety and health standards, rules, regulations, and orders. The Occupational Safety and Health Standards (OSHA) Board is the only agency authorized in the State to adopt and enforce occupational safety and health standards (Labor Code § 142 et seq.). The owner, their representative and contractor are responsible for site safety; ENGEO representatives are not responsible for site safety.

Excavating, trenching, filling, backfilling, shoring and grading work should meet the minimum requirements of the applicable Building Code, and the standards and ordinances of state and local governing authorities.

1.3 TESTING AND OBSERVATION

Site preparation, cutting and shaping, excavating, filling, and backfilling should be carried out under the testing and observation of ENGEO. ENGEO shall be retained to perform appropriate field and laboratory tests to check compliance with the recommendations. Any fill or backfill that does not meet the supplemental recommendations shall be removed and/or reworked, until the supplemental recommendations are satisfied.

Tests for compaction shall be made in accordance with test procedures outlined in ASTM D-1557, as applicable, unless other testing methods are deemed appropriate by ENGEO. These and other tests shall be performed in accordance with accepted testing procedures, subject to the engineering discretion of ENGEO.

2.0 MATERIALS

2.1 STANDARD

Materials, tools, equipment, facilities, and services as required for performing the required excavating, trenching, filling and backfilling should be furnished by the Contractor.

2.2 ENGINEERED FILL AND BACKFILL

Material to be used for engineered fill and backfill should be free from organic matter and other deleterious substances, and of such quality that it will compact thoroughly without excessive voids when watered and rolled.

Unless specified elsewhere by ENGEO, engineered fill and backfill shall be free of significant organics, or any other unsatisfactory material. In addition, engineered fill and backfill shall comply with the grading requirements shown in the following table:

TABLE 2.2-1: Engineered Fill and Backfill Requirements

US STANDARD SIEVE	PERCENTAGE PASSING
3"	100
No. 4	35–100
No. 30	20–100

Earth materials to be used as engineered fill and backfill shall be cleared of debris, rubble and deleterious matter. Rocks and aggregate exceeding the maximum allowable size shall be removed from the site. Rocks of maximum dimension in excess of two-thirds of the lift thickness shall be removed from any fill material to the satisfaction of ENGEO.

ENGEO shall be immediately notified if potential hazardous materials or suspect soils exhibiting staining or odor are encountered. Work activities shall be discontinued within the area of potentially hazardous materials. ENGEO shall be notified at least 72 hours prior to the start of filling and backfilling operations. Materials to be used for filling and backfilling shall be submitted to ENGEO no less than 10 days prior to intended delivery to the site. Unless specified elsewhere by ENGEO, where conditions require the importation of low expansive fill material, the material shall be an inert, low to non-expansive soil, or soil-rock material, free of organic matter and meeting the following requirements:

TABLE 2.2-2: Imported Fill Material Requirements

	SIEVE SIZE	PERCENT PASSING
GRADATION (ASTM D-421)	2-inch	100
	#200	15 - 70
PLASTICITY (ASTM D-4318)	Plasticity Index < 12	
ORGANIC CONTENT (ASTM D-2974)	Less than 2 percent	

A sample of the proposed import material should be submitted to ENGEO no less than 10 days prior to intended delivery to the site.

2.3 SUBDRAINS

A subdrain system is an underground network of piping used to remove water from areas that collect or retain surface water or subsurface water. Subsurface water is collected by allowing

water into the pipe through perforations. Subdrain systems may drain and discharge to an appropriate outlet such as storm drain, natural swales or drainage, etc.. Details for subdrain systems may vary depending on many items, including but not limited to site conditions, soil types, subdrain spacing, depth of the pipe and pervious medium, as well as pipe diameter.

2.4 PIPE

Subdrain pipe shall conform with these supplemental recommendations unless specified elsewhere by ENGEO. Perforated pipe for various depths shall be manufactured in accordance with the following requirements:

TABLE 2.4-1: Perforated Pipe Requirements

PIPE TYPE	STANDARD	TYPICAL SIZES (INCHES)	PIPE STIFFNESS (PSI)
PIPE STIFFNESS ABOVE 200 PSI (BELOW 50 FEET OF FINISHED GRADE)			
ABS SDR 15.3		4 to 6	450
PVC Schedule 80	ASTM D1785	3 to 10	530
PIPE STIFFNESS BETWEEN 100 PSI AND 150 PSI (BETWEEN 15 AND 50 FEET OF FINISHED GRADE)			
ABS SDR 23.5	ASTM D2751	4 to 6	150
PVC SDR 23.5	ASTM D3034	4 to 6	153
PVC Schedule 40	ASTM D1785	3 to 10	135
ABS Schedule 40/DWV	ASTM D1527 & D2661	3 to 10	
PIPE STIFFNESS BETWEEN 45 PSI AND 50 PSI* (BETWEEN 0 TO 15 FEET OF FINISHED GRADE)			
PVC A-2000	ASTM F949	4 to 10	50
PVC SDR 35	ASTM D3034	4 to 8	46
ABS SDR 35	ASTM D2751	4 to 8	45
Corrugated PE	AASHTO M294 Type S	4 to 10	45

***Pipe with a stiffness less than 45 psi should not be used.**

Other pipes not listed in the table above shall be submitted for review by the Geotechnical Engineer not less 72 hours before proposed use.

2.5 OUTLETS AND RISERS

Subdrain outlets and risers must be fabricated from the same material as the subdrain pipe. Outlet and riser pipe and fittings must not be perforated. Covers must be fitted and bolted into the riser pipe or elbow. Covers must seat uniformly and not be subject to rocking.

2.6 PERMEABLE MATERIAL

Permeable material shall generally conform to Caltrans Standard Specification unless specified otherwise by ENGEO. Class 2 permeable material shall comply with the gradation requirements shown in the following table.

TABLE 2.6-1: Class 2 Permeable Material Grading Requirements

SIEVE SIZES	PERCENTAGE PASSING
1"	100
3/4"	90 to 100
3/8"	40 to 100
No. 4	25 to 40
No. 8	18 to 33
No. 30	5 to 15
No. 50	0 to 7
No. 200	0 to 3

2.7 FILTER FABRIC

Filter fabric shall meet the following Minimum Average Roll Values unless specified elsewhere by ENGEO.

Grab Strength (ASTM D-4632) ..... 180 lbs
Mass per Unit Area (ASTM D-4751) ..... 6 oz/yd^2
Apparent Opening Size (ASTM D-4751) .....70-100 U.S. Std. Sieve
Flow Rate (ASTM D-4491)..... 80 gal/min/ft^2
Puncture Strength (ASTM D-4833) ..... 80 lbs

Areas to receive filter fabric must comply with the compaction and elevation tolerance specified for the material involved. Handle and place filter fabric under the manufacturer's instructions. Align and place filter fabric without wrinkles.

Overlap adjacent roll ends of filter fabric in accordance with manufacturer’s recommendations. The preceding roll must overlap the following roll in the direction that the permeable material is being spread. Completely replace torn or punctured sections damaged during placement or repair by placing a piece of filter fabric that is large enough to cover the damaged area and comply with the overlap specified. Cover filter fabric with the thickness of overlying material shown within 72 hours of placing the fabric.

2.8 GEOCOMPOSITE DRAINAGE

Geocomposite drainage is a prefabricated material that includes filter fabric and plastic pipe. Filter fabric must be Class A. The drain shall be of composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three-dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile.

A geotextile flap shall be provided along drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the

core. The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes. If the fabric on the geocomposite drain is torn or punctured, replace the damaged section completely. The specific drainage composite material and supplier shall be preapproved by ENGEO.

The Contractor shall submit a manufacturer's certification that the geocomposite meets the design properties and respective index criteria measured in full accordance with applicable test methods. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Contractor will supply design property test data from a laboratory approved by ENGEO, to support the certified values submitted.

Geocomposite material suppliers shall provide a qualified and experienced representative onsite to assist the Contractor and ENGEO at the start of construction with directions on the use of drainage composite. If there is more than one application on a project, this criterion will apply to construction of the initial application only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining applications. The soil surface against which the geocomposite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

Edge seams shall be formed by utilizing the flap of the geotextile extending from the geocomposite's edge and lapping over the top of the fabric of the adjacent course. The fabric flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. To prevent soil intrusion, exposed edges of the geocomposite drainage core edge must be covered.

Approved backfill shall be placed immediately over the geocomposite drain. Backfill operations should be performed to not damage the geotextile surface of the drain. Also during operations, avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than 7 days prior to backfilling.

PART II - GEOGRID SOIL REINFORCEMENT

Geogrid soil reinforcement (geogrid) shall be submitted to ENGEO and should be approved before use. The geogrid shall be a regular network of integrally connected polymer tensile elements with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction to ultraviolet degradation and to chemical and biological degradation encountered in the soil being reinforced. The geogrids shall have an Allowable Tensile Strength (T_a) and Pullout Resistance, for the soil type(s) as specified on design plans.

The contractor shall submit a manufacturer's certification that the geogrids supplied meet plans and project specifications. The contractor shall check the geogrid upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geogrid shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geogrid will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geogrid damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geogrid material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project, for a minimum of three days, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall also be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). Geogrid reinforcement may be joined with mechanical connections or overlaps as recommended and approved by the manufacturer. Joints shall not be placed within 6 feet of the slope face, within 4 feet below top of slope, nor horizontally or vertically adjacent to another joint.

The geogrid reinforcement shall be installed in accordance with the manufacturer's recommendations. The geogrid reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed. The geogrid reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. However, if the Contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the manufacturer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geogrid reinforcement shall be overlapped or mechanically connected where exposed in a wrap around face system, as applicable.

The Contractor may place only that amount of geogrid reinforcement required for immediately pending work to prevent undue damage. After a layer of geogrid reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geogrid reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geogrid reinforcement and soil. Geogrid reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geogrid reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geogrid reinforcement in position until the subsequent soil layer can be placed.

Under no circumstances shall a track-type vehicle be allowed on the geogrid reinforcement before at least 6 inches of soil have been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geogrid reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the geosynthetic reinforcement at slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided. During construction, the surface of the fill should be kept approximately horizontal. Geogrid reinforcement shall be placed directly on the compacted horizontal fill surface. Geogrid reinforcements are to be placed as shown on plans, and oriented correctly.

PART III - GEOTEXTILE SOIL REINFORCEMENT

The specific geotextile material and supplier shall be preapproved by ENGEO. The contractor shall submit a manufacturer's certification that the geotextiles supplied meet the respective index criteria set when geotextile was approved by ENGEO, measured in full accordance with specified test methods and standards.

The contractor shall check the geotextile upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the geotextile shall be protected from temperatures greater than 140°F, mud, dirt, dust, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the geotextile will be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be repaired by placing a patch over the damaged area. Any geotextile damaged during storage or installation shall be replaced by the Contractor at no additional cost to the owner.

Geotextile material suppliers shall provide a qualified and experienced representative onsite at the initiation of the project to assist the Contractor and ENGEO personnel at the start of construction. The geotextile reinforcement shall be installed in accordance with the manufacturer's recommendations. The geotextile reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed, secured with staples, pins, or small piles of backfill, placed without wrinkles, and aligned with the primary strength direction perpendicular to slope contours. Cover geotextile reinforcement with backfill within the same work shift. Place at least 6 inches of backfill on the geotextile reinforcement before operating or driving equipment or vehicles over it, except those used under the conditions specified below for spreading backfill.

Adjacent strips, in the case of 100 percent coverage in plan view, need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacing between reinforcement no greater than 40 inches. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. Adjacent rolls of geotextile reinforcement shall be overlapped or mechanically connected where exposed in a wraparound face system, as applicable.

The contractor may place only that amount of geotextile reinforcement required for immediately pending work to prevent undue damage. After a layer of geotextile reinforcement has been placed, the succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geotextile reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geotextile reinforcement and soil.

Geotextile reinforcement shall be placed to lay flat and be pulled tight prior to backfilling. After a layer of geotextile reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geotextile reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geotextile reinforcement before at least six inches of soil has been placed. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geotextile reinforcement. If approved by the Manufacturer, rubber-tired equipment may pass over the

geotextile reinforcement as slow speeds, less than 10 mph. Sudden braking and sharp turning shall be avoided.

During construction, the surface of the fill should be kept approximately horizontal. Geotextile reinforcement shall be placed directly on the compacted horizontal fill surface. Geotextile reinforcements are to be placed within three inches of the design elevations and extend the length as shown on the elevation view unless otherwise directed by ENGEO.

Replace or repair any geotextile reinforcement damaged during construction. Grade and compact backfill to ensure the reinforcement remains taut. Geotextile soil reinforcement must be tested to the required design values using the following ASTM test methods.

TABLE III-1: Geotextile Soil Reinforcements

PROPERTY	TEST
Elongation at break, percent	ASTM D 4632
Grab breaking load, lb, 1-inch grip (min) in each direction	ASTM D 4632
Wide width tensile strength at 5 percent strain, lb/ft (min)	ASTM D 4595
Wide width tensile strength at ultimate strength, lb/ft (min)	ASTM D 4595
Tear strength, lb (min)	ASTM D 4533
Puncture strength, lb (min)	ASTM D 6241
Permittivity, sec^{-1} (min)	ASTM D 4491
Apparent opening size, inches (max)	ASTM D 4751
Ultraviolet resistance, percent (min) retained grab break load, 500 hours	ASTM D 4355

PART IV - EROSION CONTROL MAT

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels. The specific erosion control material and supplier shall be pre-approved by ENGEO.

The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the criteria specified when the material was approved by ENGEO. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. Jute mesh shall consist of processed natural jute yarns woven into a matrix, and netting shall consist of coconut fiber woven into a matrix. Erosion control blankets shall be made of processed natural fibers that are mechanically, structurally, or chemically bound together to form a continuous matrix that is surrounded by two natural nets.

The Contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 140°F, mud, dirt, and debris. Manufacturer's recommendations in regard to protection from direct sunlight must also be followed. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by ENGEO, torn or punctured sections may be removed by cutting out a section of the mat. The remaining ends should be overlapped and secured with ground anchors. Any erosion mat/blanket damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

Erosion control material suppliers shall provide a qualified and experienced representative onsite, to assist the Contractor and ENGEO personnel at the start of construction. If there is more than one slope on a project, this criterion will apply to construction of the initial slope only. The representative shall be available on an as-needed basis, as requested by ENGEO, during construction of the remaining slope(s). The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil on maximum 1½ foot centers. Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 3 inches.

Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Anchors shall be as designated on the construction drawings, with a minimum of 12-inch length, and shall be spaced as designated on the construction drawings, with a maximum spacing of 4 feet.

SAN RAMON

SAN FRANCISCO

SAN JOSE

OAKLAND

LATHROP

ROCKLIN

SANTA CLARITA

IRVINE

CHRISTCHURCH

WELLINGTON

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Expect Excellence

STEVENSON BOULEVARD BRIDGE – EAST ABUTMENT
SOBRATO PROPERTY- AREA 4
NEWARK, CALIFORNIA

PRELIMINARY GEOTECHNICAL EXPLORATION

SUBMITTED TO
Mr. Tim Steele
The Sobrato Organization
10600 N. De Anza Boulevard, Suite 200
Cupertino, CA 95014

PREPARED BY
ENGEO Incorporated

December 4, 2018

PROJECT NO.
12165.000.002

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Project No.
12165.000.002

December 4, 2018

Mr. Tim Steele
The Sobrato Organization
10600 N. De Anza Boulevard, Suite 200
Cupertino, CA 95014

Subject: Stevenson Boulevard Bridge - East Abutment
Sobrato Property – Area 4
7200 Stevenson Boulevard
Newark, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Steele:

We prepared this preliminary geotechnical exploration report for the Sobrato Organization for design of the proposed bridge over the existing Union Pacific railroad in Newark, California. We characterized the subsurface conditions at the location of the proposed Stevenson Boulevard Bridge to provide the enclosed preliminary geotechnical recommendations for design.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical observation and testing services during construction. We understand the bridge design is at a conceptual stage, once the bridge type has been selected and structural demands are determined, we will be glad to discuss further detail needed for design level of this project.

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

Sincerely,

ENGEO Incorporated

REGISTERED PROFESSIONAL ENGINEER
LEROY CHAN
No. 3001
GEOTECHNICAL
STATE OF CALIFORNIA

Leroy Chan, GE

Jeff Fippin, GE

Janet Kan, CEG, GE

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this report to provide preliminary recommendations for the eastern embankment of the proposed Stevenson Boulevard Bridge planned for the Sobrato Property, Area 4, in Newark, California. We plan to perform additional explorations at the western abutment area at a later date when a drill rig can access the area.

For this report, we conducted a subsurface exploration, performed laboratory testing and preliminary engineering analysis to evaluate foundation options and geotechnical risks. We previously provided geotechnical recommendations regarding development within Area 4 of the Sobrato property on April 5, 2018 (Reference 14).

For our use, we received the following:

- Advance Planning Study, Stevenson Boulevard Overhead, Newark, California, Biggs Cardosa Associates Incorporated, dated November 13, 2018.

- DRAFT Plan Line Study, Stevenson Boulevard, Area 4 Tract 8495, Newark, California, Carlson, Barbee and Gibson (CBG), November 26, 2018.

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

1.2 PROJECT LOCATION

The proposed Stevenson Boulevard Bridge is located in the western portion of Newark, California, as shown on the Vicinity Map, Figure 1. The Stevenson Boulevard Extension Bridge will be located to the east of the Area 4 property and will span the existing Union Pacific Railroad that runs northwest to southeast; the bridge will provide access from Stevenson Boulevard to the planned residential development in Area 4, as shown on Figure 2.

1.3 PROJECT DESCRIPTION

The Stevenson Boulevard extension bridge is currently planned to be approximately 60 to 61.5 feet wide, with one vehicular travel lane and a bike lane for each direction and a pedestrian walkway (part of the bay trail) along the west bound travel lane. Based on the plans provided by Biggs Cardosa Associates, the bridge span will be approximately 227.5 feet long. Two bents, one on each side of the Union Pacific Railroad tracks, are planned. The bents and abutment walls are proposed to be supported by Cast-In-Steel-Shell (CISS) pile foundations with diameters in the range of 5 to 6 feet.

2.0 FINDINGS

2.1 GEOLOGY AND SEISMICITY

2.1.1 Geology

The site is located within the Coast Ranges geologic province of California. The Coast Ranges province is typified by a system of northwest-trending, fault-bounded mountain ranges and intervening alluviated valleys. More specifically, the site is located west of the Hayward fault and south of the Coyote Hills on the eastern side of the San Francisco Bay.

According to published geologic mapping by Helley and Graymer (1997), the project site is underlain by Quaternary basin deposits (Qhb), that typically consist of very fine silty clay to clay deposits occupying flat-floored basins at the distal edge alluvial fans adjacent to the bay mud. A stockpile of fill was placed within the proposed footprint of the east abutment, as shown in Figure 2.

2.1.2 Seismicity

Because of the presence of nearby active faults[1], the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (greater than Moment Magnitude, M_w, 7) earthquakes have been recorded and can be expected to occur in the future. The site is not located within a State of California Earthquake Fault Zone. Figure 3 shows the approximate location of known active and potentially active faults and significant historic earthquakes mapped within the San Francisco Bay Region.

TABLE 2.1.2-1: Regional Faults

FAULT NAME	APPROXIMATE DISTANCE (miles)	DIRECTION FROM SITE	ESTIMATE OF MAXIMUM MAGNITUDE (Ellsworth)
Hayward/Rogers Creek	4.0	Northeast	7.3
Calaveras	8.4	Northeast	7.0
Monte Vista-Shannon	12.7	Southwest	6.5
San Andreas	15.1	Southwest	8.0

Site: Latitude = 37.5044; Longitude = -121.9979

The Silver Creek Fault is mapped west of the site concealed beneath Quaternary alluvium. Recent work by the USGS on the Silver Creek Fault concluded that no surface trace is evident and convincing evidence of Holocene offset is lacking (OFR 2010-1010).

The Working Group on California Earthquake Probabilities (WGCEP, 2007) evaluated the 30-year probability of an M_w 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area, including the Hayward fault. The WGCEP generated an overall probability of 63 percent for the Bay Area as whole, and a probability of 31 percent for the Hayward fault, 7 percent for the Calaveras fault, and 21 percent for the San Andreas Fault.

[1] An active fault is defined by the California Geological Survey as one that has had surface displacement within Holocene time (about the last 11,000 years) (Hart, 1997).

2.2 SURFACE CONDITIONS

According to the plan line study provided to us by CBG, existing site grades are currently at approximately Elevation 7 to Elevation 27 feet (Datum: NGVD29) in the vicinity of the eastern abutment. Much of the eastern abutment footprint is occupied by an existing stockpile of fill approximately 20 feet high. From review of aerial photographs, it appears that the stockpile was placed between 1998 and 2002 when excavation of the neighboring detention basin was observed. No document associated with placement of this stockpile can be found at the City of Newark archives.

2.3 FIELD EXPLORATION

Our field exploration included drilling three exploratory borings (2-B1, 2-B2, and 2-B3) between April 2 and April 4, 2018 at the proposed eastern bridge abutment. The locations of the borings are shown on the Site Plan, Figure 2. We retained the services of a drilling crew operating a truck-mounted drill rig to advance Borings 2-B1 through 2-B3 using 5-inch-diameter mud rotary methods to a depths of approximately 41½ to 81½ feet below the ground surface (bgs).

A representative of our firm logged the borings in the field and collected disturbed and relatively undisturbed soil samples using a 2½-inch-inside-diameter (I.D.) California-type split-spoon sampler fitted with 6-inch-long stainless steel liners and a 2-inch-outside-diameter (O.D.) Standard Penetration Test (SPT) split-spoon sampler. In addition, 3-inch O.D. Shelby tube samplers were utilized.

We obtained the standard blow counts by dropping a 140-pound hammer through a 30-inch free fall. We drove the sampler 18 inches and the number of blows was recorded for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 1 foot of penetration; the blow counts reported on the logs have not been converted using any correction factors.

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

2.4 SUBSURFACE CONDITIONS

Within the limits of the existing stockpile, we generally encountered fill consisting of stiff to hard lean clay with varying amounts of gravel from the top of the stockpile to depths ranging from Elevations 3 to -4 feet (NGVD29). Below the stockpile fill, we generally encountered medium stiff, high plasticity and potentially compressible fat clay extending to approximately Elevation -11 feet. Below the fat clay, we encountered stiff lean clayey soil that extended to approximately Elevation -47 feet. Below the lean clayey soil, we encountered a layer of loose clayey sand, approximately 10 to 15 feet thick. This layer of loose clayey sand may be potentially liquefiable.

We encountered stiff/dense silty, sandy and gravelly deposits below Elevation -61 feet that extends to the termination depths of our borings. The specific stratigraphy for Borings 2-B1 through 2-B3 are depicted on the exploration logs in Appendix A. A generalized subsurface profile is summarized in Table 2.4-1 below.

Table 2.4-1: Generalized Subsurface Profile

ELEVATION (NGVD29)	APPROXIMATE THICKNESS (FT)	DESCRIPTION
Above -4 feet	10 to 25	Stockpile Fill: Stiff to hard lean clay, gravelly clay and sandy clay.
-4 to -11 feet	7	Fat Clay: Medium stiff, high plasticity clay. **Potentially compressible.**
-11 to -16 feet	5	Clayey Sand/Silty Sand/Sandy Silt/Sandy Clay: Medium stiff, low to medium plasticity clay and silts. **Potentially liquefiable.**
-16 to -47 feet	36	Lean Clay: Stiff, low to medium plasticity clay.
-47 to -61 feet	14	Clayey Sand/Silty Sand: loose, clayey and silty sand. **Potentially liquefiable.**
-61 to -75 feet	14	Sand and Gravel: Dense, contains some fines.

2.5 GROUNDWATER CONDITIONS

We encountered groundwater at approximately Elevation 0 feet in Boring 2-B1. In addition, as described in Reference 14, we encountered groundwater between approximately Elevation -2 and -3.5 feet in our previous exploration. Based on the above-mentioned information regarding depth to groundwater, we used a design groundwater level of Elevation 0 feet for our analysis.

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

2.6 LABORATORY TESTING

The lab tests performed for this report include moisture content, dry density, plasticity index, gradation, shear strength, R-value, and sulfate content on selected samples. The laboratory test results are included on the borelogs in Appendix A. Individual test results are presented in Appendix B. We also submitted select samples for analytical corrosion testing including pH, redox, resistivity, sulfate and chloride content and results are provided in Appendix C.

3.0 DISCUSSION AND CONCLUSIONS

Based on our preliminary explorations, laboratory test results and analyses, we believe that the proposed bridge is feasible from a geotechnical and geologic standpoint, provided that the preliminary recommendations included in this report, along with other sound engineering practices, are incorporated in the design and construction of the project. We recommend that additional exploration and laboratory testing to be performed in the vicinity of the western abutment in order to assess the subsurface conditions across the proposed bridge.

3.1 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking and ground lurching. The following sections present a discussion of these hazards as they apply to the site.

3.1.1 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, ground rupture is unlikely at the subject property.

3.1.2 Ground Shaking and Seismic Design Criteria

As discussed previously, the project site is located in a seismically active region. An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause significant ground shaking at the project site. The degree of shaking is dependent on the magnitude of the event, the distance to its zone of rupture and local geologic conditions. The seismic design criteria are discussed below.

We developed the Acceleration Response Spectra (ARS) for the project using the Caltrans ARS Online (v2.3.06) online web tool; this application follows Version 1.7 of the Caltrans Seismic Design Criteria (Caltrans SDC, 2013). To perform our analysis, we estimated the average shear wave velocity for the top 30 meters ($V_{s,30}$) to be 200 meters per second based on the explorations previously discussed.

The Caltrans ARS curve is calculated from the following:

- Deterministic Criteria based on late-Quaternary faults in the 2012 fault database published by Caltrans referencing latest updates from the CGS and United States Geological Survey (USGS).

- Probabilistic Criteria based on 5% in 50 years probability of exceedance ground motion (975-year return period). The probabilistic analyses are based on source data from the 2008 USGS National Seismic Hazard Map.

- Minimum Spectrum based on a $M_w = 6.5$ strike-slip event occurring at a distance of 7.5 miles (12 km) from the site.

The resultant 2013 Caltrans ARS includes near-fault directivity effects. The ARS curve developed based on these criteria is presented in Appendix D.

3.1.3 Liquefaction Evaluation

We evaluated the potential for liquefaction utilizing subsurface data from Borings 2-B1 through 2-B3.

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. The soil considered most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sands below the groundwater table. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop and liquefaction of susceptible soil to occur.

Historically, standard geotechnical engineering practices for liquefaction assessment have included layers of loose to medium dense and saturated sandy deposits as being potentially liquefiable, however, empirical evidence from recent major earthquakes and published research projects at major universities indicate that some fine-grained soil (including low plasticity silt and

clay) can also liquefy. Bray and Sancio (2006) discussed the susceptibility of fine-grained soil to liquefaction as a function of liquid limit (LL), plasticity index (PI) and water content (w_c); loose soils with PI less than 12 and w_c/LL greater than 0.85 are considered susceptible to liquefaction, and loose soils with a plasticity index between 12 and 18 and w_c/LL greater than 0.8 are more resistant to liquefaction.

To assess liquefaction potential at the bridge location, we used a design groundwater level of Elevation 0 feet and a peak ground acceleration (PGA) of 0.63g based on the Caltrans ARS Curve (Appendix D). Based on the 2008 United States Geologic Survey probabilistic seismic hazard deaggregation analysis, we used the Hayward fault contribution to hazard with a moment magnitude (M_w) of 7.3.

We performed preliminary liquefaction analyses on the borings based on guidelines provided in the California Geological Survey's Special Publication 117A (2008), as well as analysis methods by Bray and Sancio and Idriss and Boulanger (2008). We used SPT blow counts in the saturated silt, silty sand, and clayey sand layers to calculate the factor of safety against liquefaction. We include the results of our liquefaction assessment in Appendix E. We list potentially liquefiable layers in Table 3.1.3-1 below.

TABLE 3.1.3-1: Potentially Liquefiable or Cyclically Soften Layers

EXPLORATION ID	ELEVATION (NGVD29)	POTENTIALLY LIQUEFIABLE LAYERS
2-B1 Bottom of Boring at El. -75.5 feet	-0.5 to -4 feet	Poorly Graded Gravel (GP) [Fill]
	-11.5 to -16.5 feet	Silty Clay to Clayey Silt (CL-ML) [Native]
	-47 to -61 feet	Clayey Sand to Silty Sand (SC-SM) [Native]
2-B2 Bottom of Boring at El. -55.5 feet	-11.5 to -14 feet	Clayey Sand (SC) [Native]
	-32 to -35 feet	Poorly Graded Sand (SP) [Native]
	-49 to -52 feet	Clayey Sand to Silty Sand (SC-SM) [Native]
2-B3 Bottom of boring at El. -24.5 feet	-13 to -16 feet	Clayey Sand to Silty Sand (SC-SM) [Native]

Our analysis suggests that native soil between Elevations -11 and -16 feet, -32 to -35 feet, and -47 to -61 feet are susceptible to liquefaction and cyclic softening. In addition, a localized layer of gravelly fill at Elevation -0.5 to -4 feet is potentially liquefiable and should be mitigated.

Additional subsurface exploration and collection of soil samples during the design-level study will better delineate the areas with a potential for liquefaction and estimates of liquefaction-induced settlement magnitude. Soil samples of the fine-grained soil should be selected for cyclic simple shear testing during the design-level study to further evaluate liquefaction potential and volumetric strain.

3.1.4 Liquefaction-Induced Ground Settlement

We calculated potential liquefaction-induced settlement estimates based on methods published by Idriss and Boulanger (2008) and soil encountered in Borings 2-B1, 2-B2, and 2-B3.

TABLE 3.1.4-1: Preliminary Liquefaction or Cyclic Softening Induced Settlement

EXPLORATION ID	PRELIMINARY LIQUEFACTION INDUCED SETTLEMENT (INCHES)
2-B1	6.2
2-B2	3.9
2-B3	1.7

The deep foundations recommended in Section 4.2 will, therefore, be subject to downdrag forces due to liquefaction. There is a potential of differential settlement between the abutment fill and the pile-supported bridge. This differential settlement could be as large as 6 inches if the maximum estimated settlement occurs. On-going maintenance such as grading or pavement overlay of the bridge approach may be required to address differential settlement if it occurs. Alternatively, ground improvement can be performed to mitigate liquefaction and load induced settlement within the bridge approach. Downdrag and differential settlement due to liquefaction should be further evaluated in the design-level report.

3.1.5 Surface manifestation

In addition to the above preliminary liquefaction analysis, we also evaluated the capping effect of any overlying non-liquefiable soil. We understand that the new bridge abutment will raise site grades by 5 to 15 feet. The additional fill will increase the non-liquefiable cap above the potentially liquefiable layers identified in Section 3.1.3. In order for liquefaction-induced ground failure to occur, the pore water pressure generated within the liquefied strata must exert a force sufficient to break through the overlying soil and vent to the surface resulting in sand boils or fissures. We based our analyses and review on guidelines provided by Ishihara (1985) and Youd and Garris (1995).

When considering planned final grade, the potentially liquefiable soil at the project site have adequate caps of non-liquefiable soil when site grades are raised to prevent venting of liquefiable soil.

3.1.6 Lateral Spreading

Lateral spreading involves lateral ground movement caused by earthquake vibrations. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied sand or weak soil.

The shallowest liquefiable layer is located at 2-B1 at Elevation -0.5 to -4 feet; however, the liquefiable material identified is loose gravelly fill previously placed to construct the embankment and does not appear to be continuous across the site. As discussed above, we encountered the next shallowest and continuous liquefiable layer consisting of clayey silt, silty clay and sandy clay between Elevation -11 and -16.5 feet. The neighboring detention basin creates a free face extending from the ground surface to Elevation 5 feet. As such, the continuous liquefiable layer at Elevation -11 to -16.5 feet does not intercept the free face created by the neighboring detention basin. Thus, the potential for lateral spreading is low.

3.2 EXISTING FILL

Existing fill is present within the footprint of the eastern bridge abutment in the form of a stockpile. No placement records are available for the existing stockpile fill. The existing stockpile fill is

predominately stiff to hard clay with a layer of medium dense gravelly clayey sand and poorly graded gravel at the base of the stockpile. A layer of soft to medium stiff fat clay deposits are present below the sandy and gravelly layer below the base of the stockpile. We did not encounter debris or deleterious material within the stockpile fill in our exploratory borings.

3.3 COMPRESSIBLE BAY MUD

The eastern abutment is underlain with potentially compressible fat clay ranging from approximately 7 to 10 feet thick. The top of this compressible fat clay layer was encountered at approximately Elevation 0 to -4 feet bgs. Based on laboratory consolidation test result, this fat clay layer below the existing fill is over consolidated by 50 percent.

We anticipate this fat clay layer may experience consolidation settlement from the weight of new fill. The design of surface grades and underground utilities must accommodate or resist potential differential settlement.

We performed preliminary consolidation settlement calculations considering up to 25 feet of new fill over the existing 25 feet of stockpile fill and a traffic surcharge load of 120 psf. Our analysis considered an over consolidation ratio (OCR) of 1.5 and a compressive index of approximately 0.15.

TABLE 3.3-1: Consolidation Settlement in 30 years

PROPOSED IMPROVEMENTS OVER EXISTING STOCKPILE FILL	LOAD (PSF)	TOTAL SETTLEMENT IN 30 YEARS (INCHES)
5 foot of Fill	745	0.5
10 feet of Fill	1,370	1.5
15 feet of Fill	1,995	2.5
20 feet of Fill	2,620	3.5
25 feet of Fill	3,245	4

The above load-induced settlement is in addition to the estimated liquefaction-induced settlement. The following measures may be considered to mitigate potential load-induced settlement:

- Ground improvement such as deep soil mixing or drilled displacement columns.
- Placement of lightweight fill.
- Surcharge program along with wick drains.
- Long-term maintenance program to raise the settled embankment.

The above mitigation measures should be evaluated in the design-level geotechnical exploration.

3.4 SOIL CORROSIVITY

As a part of our previous study, we collected one sample from Boring 2-B1 at 10 feet bgs and transported the sample under chain-of-custody to CERCO Analytical for corrosion testing. At CERCO Analytical, the samples were tested for pH, resistivity, chloride and sulfate concentration. These tests provide an indication of the corrosion potential of the soil environment on buried concrete structures and metal pipes. The results are summarized in the table below. A detailed

description of the laboratory results is contained in the report prepared by CERCO Analytical, Inc. (Appendix C).

TABLE 3.4-1: Soil Corrosivity Test Results

SAMPLE NUMBER	SOIL TYPE	SULFATE (mg/kg)	CHLORIDE ION (mg/kg)	RESISTIVITY (Ohm-cm)	pH
2-B1@10'	CLAY	97	440	510	7.57

According to Caltrans Corrosion Guidelines Version 2.0 (2012), soil is considered corrosive if chloride concentration is 500 ppm or above, sulfate concentration is 2000 ppm or above, or pH is 5.5 or less. Based on these criteria, the site soil is not considered corrosive to structural elements. Additional corrosivity tests should be performed during design-level exploration to determine corrosion levels for concrete and steel protection. We recommend a corrosion consultant be retained to evaluate specific corrosion recommendations for the project.

4.0 PRELIMINARY RECOMMENDATIONS

Based on the results of the geotechnical exploration presented in this report, it is our opinion that the proposed bridge project is feasible from a geotechnical standpoint when preliminary recommendations provided in the following sections are considered in the bridge design.

4.1 PRELIMINARY GRADING CONSIDERATIONS

4.1.1 Demolition and Site Clearing

The existing stockpile fill should be considered non-engineered based on its age and lack of documentation on placement. We recommend removing the existing stockpile fill and reconstructing the bridge abutment with engineered fill. In general, the footprint of the eastern abutment should be cleared of vegetation and deleterious material. Existing underground utilities at the site should be identified and either properly abandoned or relocated. If loose and deleterious materials are present, the materials should be removed and the area should be backfilled with engineered fill.

4.1.2 Selection of Materials

With the exception of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, high organic content soil (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils (if any), we anticipate the existing stockpile fill is suitable for use as engineered fill for the bridge approach.

In accordance with Caltrans requirements, expansive soil should not be placed as part of the embankment within the limits of a bridge abutment. Materials placed behind abutment wall should be low-expansive soil with an Expansion Index (EI) less than 50 and Sand Equivalent (SE) of more than 20.

Other materials and debris, including trees and shrubs with their root balls, should be removed from the project site. Imported soil for the bridge abutment zone should meet Caltrans requirements. Imported soil for the approach should have a PI less than 20, with more than 70 percent passing the No. 200 sieve and less corrosive than site soil.

4.1.3 Fill Compaction

For cost estimating purposes, the following compaction control requirements should be anticipated for the new embankment fill:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 3 percentage points above optimum moisture content.
Minimum Relative Compaction:	Relative compaction 90 percent.

The upper 12 to 24 inches of the new embankment may require higher relative compaction to support intended traffic load. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material. In the event that imported fill material is characterized and following the design-level geotechnical report, the recommendations may change with respect to the soil type.

4.1.4 Graded Slope

Graded slopes along the embankment should be planned with an inclination of 2:1 (horizontal:vertical) or flatter for heights up to 10 feet. For higher slopes, geogrid reinforcement may be needed to construct a stable 2:1 slope. Improvements located at the tops of embankment slopes could be subject to minor lateral movements associated with soil creep.

4.2 PRELIMINARY FOUNDATION CONSIDERATIONS

Major considerations in the bridge and abutment foundation design for this project are:

- Long-term settlement of potentially compressible fat clay deposits.
- Liquefaction induced settlement.

Based on the plans provided by Biggs Cardosa Associates (BCA), we understand that the eastern abutment is planned to be supported by 12-inch square piles and the bents will be supported on 66-inch-diameter Cast-in-steel-shell (CISS) piles. The piles on the downslope side of the abutment may be designed as battered piles. The tables below show the loading and geometry data provided by BCA.

TABLE 4.2-1: Conceptual Pile Foundation Design By BCA

SUPPORT NO.	PILE DIMENSIONS, SHAPE AND TYPE	NO. OF PILES PER SUPPORT	TOP OF PILE ELEVATION (NGVD29)
Bent 3	5.5-foot Round CISS Pile	2	6
Abutment 2	1-foot Square Concrete Pile	4	20

4.2.1 Pile Vertical Capacity

We performed preliminary analysis of the proposed CISS and concrete piles based on the soil profile presented in Table 4.2.1-1 below. Our analysis only includes side friction for the pile support; no end bearing resistance is used. For the strength limit state, we used a reduction factor of 0.7 for friction resistance per applicable Caltrans amendments to the AASHTO LRFD Bridge Design

Specifications. We performed our analyses in accordance with the guidelines provided in the FHWA Publication FHWA NHI-05-042, using the method published by Randolph and Murphy (1985) for the skin friction. The results of our analyses are shown on the pile capacity chart included in Appendix F.

Due to presence of compressible soil and potentially liquefiable soil, piles will experience downdrag forces. Preliminary downdrag forces are considered in the extreme limit state. Design tip elevations for the piles are controlled by compression under Strength Limit Conditions. Settlement does not control design. The following table provides the design tip elevations for 5.5-foot-diameter CISS piles at Bent 3 and 12-inch concrete piles at Abutment 2.

TABLE 4.2.1-1: Preliminary Pile Design Table

LOCATION	PILE DIMENSIONS (feet) AND SHAPE	STRENGTH LIMIT STATE LOAD PER PILE (kips) COMPRESSION	NOMINAL RESISTANCE PER PILE (kips) COMPRESSION	DESIGN PILE LENGTH (feet)	DESIGN TIP ELEVATION* (feet)
Bent 3	5.5-foot Round	500	775	55	-45
Abutment 2	1.0-foot Square	75	125	50	-30

* The structural designer shall calculate the lateral lengths.

Our liquefaction analysis indicates that potentially liquefiable clayey sand and silt layers are present from Elevations -47 to -61 feet. Additional exploration should be performed in order to further assess the soil conditions 20 to 30 feet below the pile tip elevations presented in Table 4.2.1-1.

4.2.2 Lateral Load Design

Lateral load resistance for pile-supported structures is developed through pile bending/soil interaction. The magnitude of the lateral load resistance is dependent upon several factors, including axial load on the pile, pile stiffness, pile embedment length, conditions of fixity at the pile cap, the physical properties of surrounding soil and the magnitude of allowable lateral deflections. We will provide lateral load resistance parameters when bridge design drawings are available. Preliminary L-pile parameters are provided in Table 4.2.2-1 to support pile type selection.

TABLE 4.2.2-1: Preliminary L-Pile Parameters

ELEVATION (FEET, NGVD29)	L-PILE SOIL TYPE	EFFECTIVE UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	UNDRAINED COHESION, C (PSF)	STRAIN FACTOR E50	MODULUS OF SOIL REACTION K (PCI)
Above 0	STIFF CLAY WITHOUT FREE WATER	125	--	1000	0.007	150
0 to -4	STIFF CLAY WITHOUT FREE WATER	58	--	1000	0.007	150
-4 to -11	STIFF CLAY WITHOUT FREE WATER	38	--	500	0.015	30

ELEVATION (FEET, NGVD29)	L-PILE SOIL TYPE	EFFECTIVE UNIT WEIGHT (PCF)	FRICTION ANGLE (DEGREES)	UNDRAINED COHESION, C (PSF)	STRAIN FACTOR E50	MODULUS OF SOIL REACTION K (PCI)
-11 to -16	SILT	48	18	800	0.020	30
-16 to -47	STIFF CLAY WITHOUT FREE WATER	58	--	2000	0.005	500
-47 to -61	API SAND	48	25	--	--	20
-61 to -75	API SAND	73	45	--	--	60

4.3 ABUTMENT RETAINING WALLS

The abutments of the proposed bridge will be constructed to retain engineered fill. Abutment walls should be designed to resist both lateral loads imposed by earth pressures and any additional lateral loads caused by surcharging from construction equipment and vehicles operating above the walls. For long-term lateral earth pressure against the abutment walls, the walls should be designed to be free standing (i.e. not braced at the top by the bridge deck).

Assuming existing onsite material is used for the backfill of the abutment walls, the following soil design parameters can be used.

TABLE 4.3-1: Soil Design Parameters for Abutment Walls

SOIL PROPERTIES	ENGINEERING DESIGN PARAMETERS
Density of Retained Engineered Fill	120 pcf
Friction Angle	28 degrees
Cohesion	500 psf
Active Pressure	55 pcf
At-rest Pressure	65 pcf
Passive Resistance	Section 7.8 of Caltrans SDC (2013)
Seismic Active Increment Pressure*	23 pcf

*The Seismic Active Increment shall be modeled with a triangle distribution

The above parameters assume a level backfill at the top of the wall. If lateral movement at the top of abutment walls is restrained, the at-rest pressure should be used for design. If applicable, a uniform lateral pressure of at least 72 psf due to vehicle loads should be added to the above lateral earth pressure.

Backdrains should be installed behind the abutment walls to relieve hydrostatic pressure. Backdrains should be constructed in accordance with Bridge Detail 3-1 on Sheet BO-3 per Caltrans Standard Plans (2010). If backdrains are not installed, lateral earth pressure values should be increased by 40 pcf to account for hydrostatic pressure.

4.4 PAVEMENT DESIGN

The existing stockpile fill consist of soil with a tested R-value of 6. Based on this result, pavement design for the bridge approach should conform to the recommendations provided in our April 5, 2018, Preliminary Geotechnical Exploration report (Reference 14).

5.0 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 review the project plans and specifications and provide construction monitoring. For this project, we recommend that we be retained to:

1. Review the final 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. All 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 have been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is essential.

3. Provide observation during pile driving. The piles should be installed within the alignment and depth tolerances specified by the Structural Engineer. We should be notified at least 48 hours in advance of inspections required. If difficulties in pile driving occur, the Geotechnical Engineer, in consultation with the Structural Engineer, should determine what measures, if any, need to be taken.

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).

6.0 FUTURE GEOTECHNICAL STUDIES

This report presents preliminary geotechnical findings, conclusions, and recommendations intended for bridge type selection and preliminary engineering design only. A design-level geotechnical exploration and assessment should be performed when development plans are finalized. The design-level exploration should further evaluate the following:

- Soil conditions below 20 to 30 feet of the design pile tip elevation.
- The thickness and volumetric strain of liquefiable soils.
- Mitigation measures for load-induced and liquefaction-induced settlement.
- The soil conditions in the western abutment area.
- Specific recommendations for site grading, ground improvement, and the design and construction of foundations and utilities.

7.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.0. 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 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 geotechnical engineering principles and practices currently employed in the area; no warranty is expressed 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 or provide insurance; 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 is representative of the actual subsurface conditions across the site. Considering possible underground variability of soil, rock, stockpiled material, 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, notify us 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, notify the proper regulatory officials immediately.

This document must not be subject to unauthorized reuse, that is, reusing without our written authorization. Such authorization is essential because it requires us 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 our documents. Therefore, we must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If our scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, we cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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

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FIGURES

FIGURE 1: Vicinity Map
FIGURE 2: Site Exploration Plan
FIGURE 3: Regional Faulting and Seismicity Map

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FEET

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VICINITY MAP

STEVENSON BOULEVARD BRIDGE - EAST ABUTMENT

NEWARK, CALIFORNIA

PROJECT NO. : 12165.000.002	FIGURE NO.
SCALE: AS SHOWN	1
DRAWN BY: QRL	CHECKED BY: JAF

PATH: G:\DRAFTING\DRAFTING2_DWG_10000 TO 12999\12165\000\002\GIS\STEVENSON BOULEVARD BRIDGE – EAST ABUTMENT.APRX

ORIGINAL FIGURE PRINTED IN COLOR

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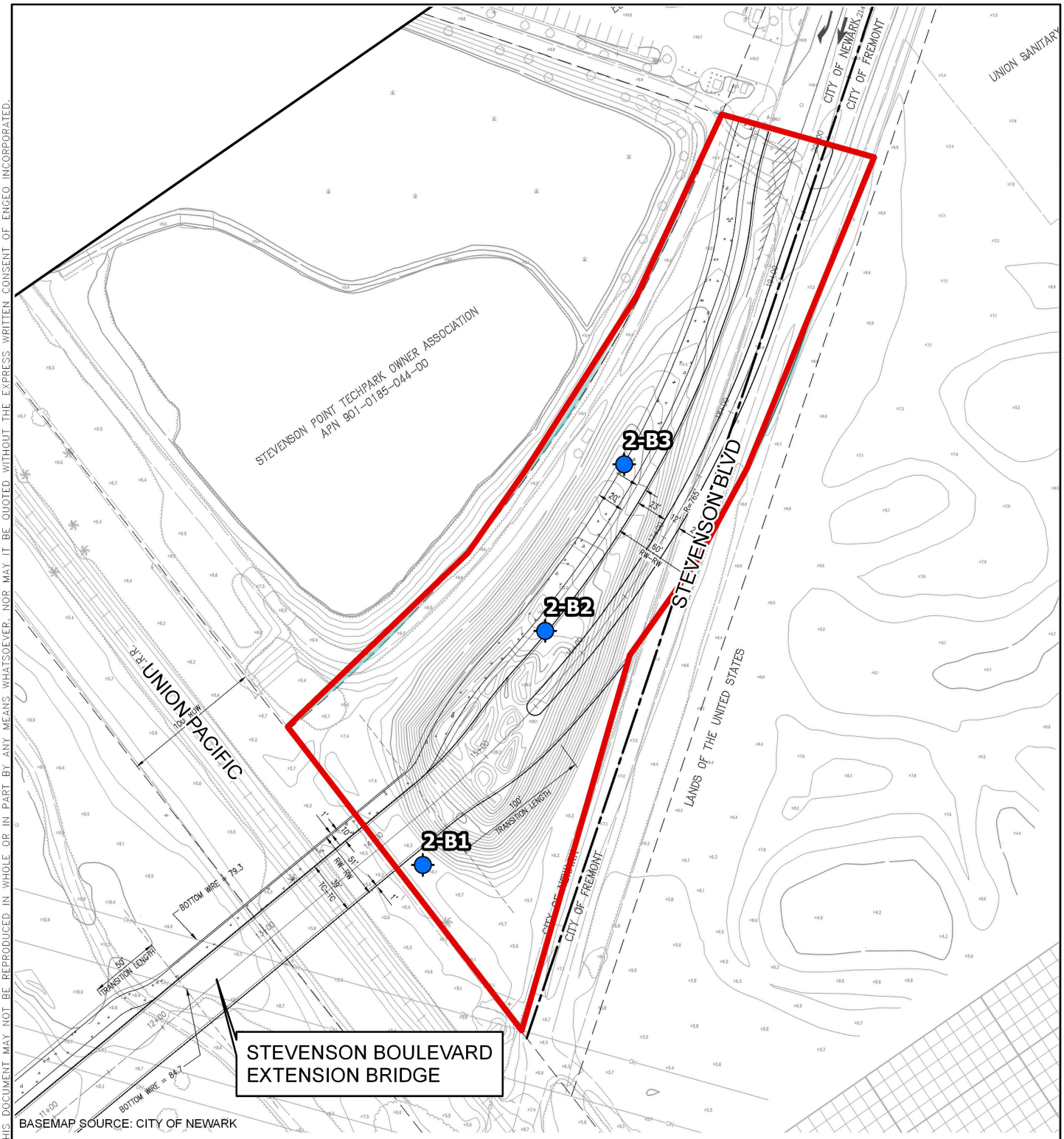

BASEMAP SOURCE: CITY OF NEWARK

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FEET

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

PROJECT SITE

2-B3 BORINGS (ENGEO, 2018)

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SITE PLAN	PROJECT NO. : 12165.000.002	FIGURE NO.
STEVENSON BOULEVARD BRIDGE - EAST ABUTMENT	SCALE: AS SHOWN	2
NEWARK, CALIFORNIA	DRAWN BY: QRL CHECKED BY: JAF	

PATH: G:\DRAFTING\DRAFTING2_DWG_10000 TO 12999\12165\000\002\GIS\STEVENSON BOULEVARD BRIDGE – EAST ABUTMENT.APRX

ORIGINAL FIGURE PRINTED IN COLOR

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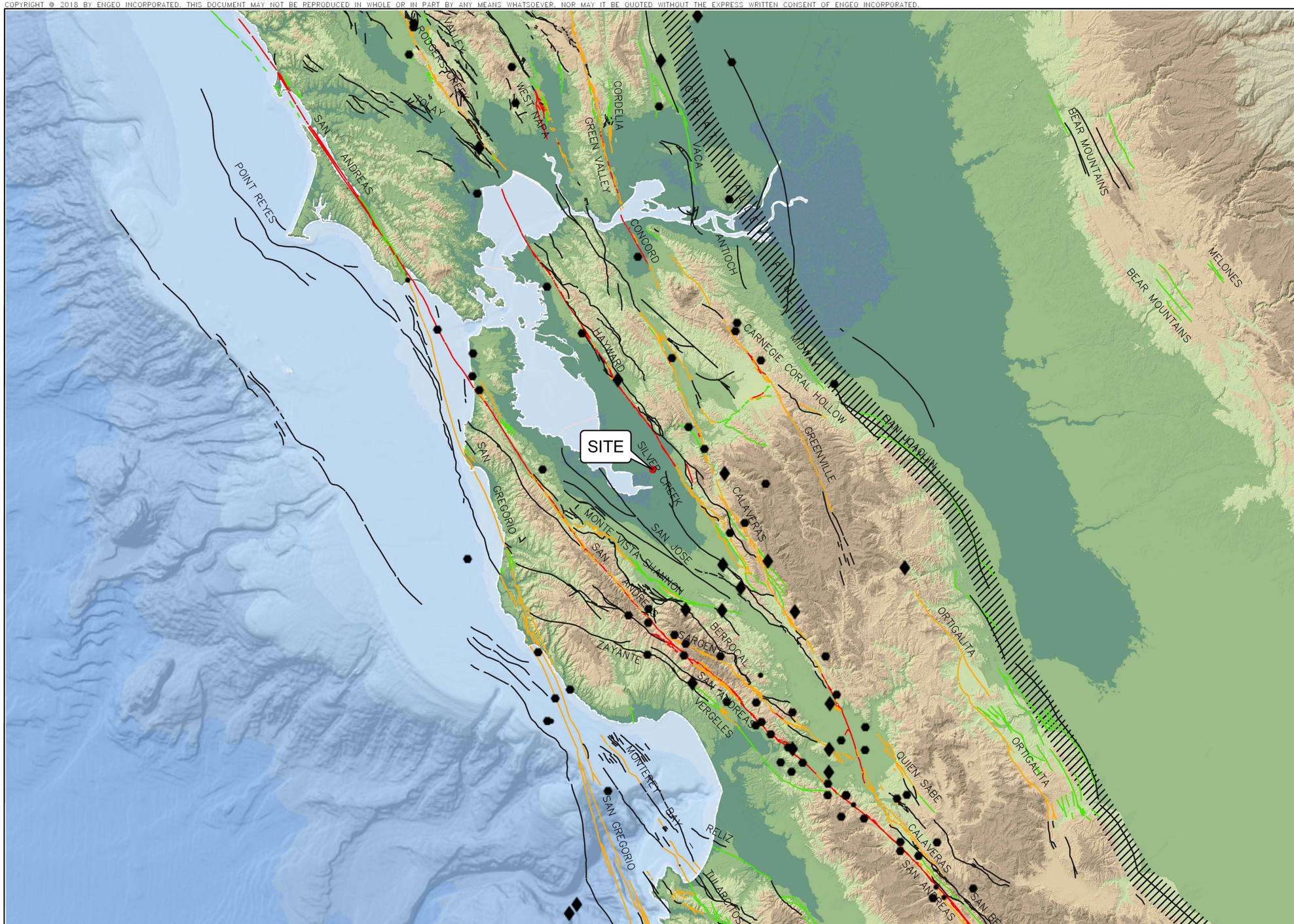

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE
EARTHQUAKE

- MAGNITUDE 7+
- MAGNITUDE 6-7
- MAGNITUDE 5-6

USGS QUATERNARY FAULTS

- HISTORICAL
- LATEST QUATERNARY
- LATE QUATERNARY
- UNDIFFERENTIATED QUATERNARY

HISTORIC BLIND THRUST FAULT ZONE

BASE MAP SOURCE
ESRI, GARMIN, GEBCO, NOAA NGDC, AND OTHER CONTRIBUTORS
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
STEVENSON BOULEVARD BRIDGE - EAST ABUTMENT
NEWARK, CALIFORNIA

PROJECT NO. : 12165.000.002	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: QRL	CHECKED BY: JAF

PATH: G:\DRAFTING\DRAFTING2_DWG_10000 TO 12999\12165\000\002\GIS\STEVENSON BOULEVARD BRIDGE - EAST ABUTMENT.APRX

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

BORING LOG KEY
EXPLORATION LOGS

KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION	
COARSE-GRAINED SOIL (≥50% LARGER THAN NO. 200 SIEVE)	GRAVELS (MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE)	≤ 5% FINES	**GW**	Well-graded gravel
			GP	Poorly-graded gravel
		5 < FINES ≤ 12%	**GW-GM**	Well-graded gravel with silt
			GW-GC	Well-graded gravel with clay
			GP-GM	Poorly-graded gravel with silt
			GP-GC	Poorly-graded gravel with clay
		> 12% FINES	**GM**	Silty gravel
			GC	Clayey gravel
	SANDS (MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE)	≤ 5% FINES	**SW**	Well-graded sand
			SP	Poorly-graded sand
		5 < FINES ≤12%	**SW-SM**	Well-graded sand with silt
			SW-SC	Well-graded sand with clay
			SP-SM	Poorly-graded sand with silt
			SP-SC	Poorly-graded sand with clay
		> 12% FINES	**SM**	Silty sand
			SC	Clayey sand
FINE-GRAINED SOIL (≥ 50% SMALLER THAN NO. 200 SIEVE)	SILTS AND CLAYS (LIQUID LIMIT < 50%)		**ML**	Silt
			CL	Lean clay
			OL	Low plasticity organic silt and clay
	SILTS AND CLAYS (LIQUID LIMIT ≥ 50%)		**MH**	Elastic silt
			CH	Fat clay
			OH	High plasticity organic silt and clay

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

CONSISTENCY

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

SAMPLER SYMBOLS

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

MOISTURE CONDITION

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

LINE TYPES

———— Solid - Layer Break

– – – – – – Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

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

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

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LOG OF BORING 2-B1

ENGEO — Expect Excellence

LATITUDE: 37.5044 LONGITUDE: -121.9982

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

DATE DRILLED: 4/2/2018
HOLE DEPTH: Approx. 81½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 6 ft.

LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), dark brown, medium stiff, moist, <5% rootlets [FILL]												
	5														
			Brown mottled with dark brown and yellowish brown												
5															
	0					7							700*		TV
			POORLY GRADED GRAVEL (GP), dark reddish brown, medium dense, wet, 5 to 10% clay [FILL]			14									
10			FAT CLAY (CH), gray, medium stiff, wet, <5% sand [NATIVE]												
	-5					5								0.5*	PP
15											29	95			
	-10		Consolidation Test at 16'												
			<5% organics												
			SILTY CLAY TO CLAYEY SILT WITH SAND (CL-ML), grayish green, medium stiff, 5 to 10% gravel											1.0*	PP
20															

LOG OF BORING 2-B1

ENGEO Expect Excellence

LATITUDE: 37.5044 LONGITUDE: -121.9982

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

DATE DRILLED: 4/2/2018
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HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			SILTY CLAY TO CLAYEY SILT WITH SAND (CL-ML), grayish green, medium stiff, 5 to 10% gravel												
	-15						24	18	6	63	21	110	2096		UU
			Grades to sandy clay												
			LEAN CLAY (CL), pale olive mottled with yellowish brown, stiff, wet, 10 to 20% sand												
25															
	-20					35					18	116	1,000*		TV
30			10 to 20% sand												
	-25					32					20	112	2,096		UU
													2,000*		TV
35															
	-30														
40															

LOG OF BORING 2-B1

ENGEO Expect Excellence

LATITUDE: 37.5044 LONGITUDE: -121.9982

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
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DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), pale olive mottled with yellowish brown, stiff, wet, 10 to 20% sand												
	-35					47							3,000*		TV
45															
	-40														
50															
	-45														
			CLAYEY SAND TO SILTY SAND (SC-SM), brown, loose, wet												
55															
	-50					15	23	17	6	21	23				
						17									
60															

LOG OF BORING 2-B1

ENGEO
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LATITUDE: 37.5044 LONGITUDE: -121.9982

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

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DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			CLAYEY SAND TO SILTY SAND (SC-SM), brown, loose, wet												
	-55														
65															
	-60														
			SILT (ML), grayish green, stiff, wet, 20 to 30% sand, <5% organics												
70															
	-65														
						46									
75															
	-70														
			POORLY GRADED GRAVEL (GP), brown, dense, wet, subangular, 30 to 40% sand, 5 to 10% fines												
80															

LOG OF BORING 2-B1

ENGEO Expect Excellence

LATITUDE: 37.5044 LONGITUDE: -121.9982

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

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HOLE DIAMETER: 5.0 in.
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LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-75		POORLY GRADED GRAVEL (GP), brown, dense, wet, subangular, 30 to 40% sand, 5 to 10% fines			36									
			Boring terminated at an approximate depth of 81.5 feet below existing grade. Groundwater was encountered at approximatley 6.5 feet below existing grade at the time of drilling.												

LOG OF BORING 2-B2

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LATITUDE: 37.5048 LONGITUDE: -121.9979

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

DATE DRILLED: 4/3/2018
HOLE DEPTH: Approx. 81½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 26 ft.

LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	25		LEAN CLAY (CL), brown, stiff, moist, 5 to 10% sand, <5% gravel and organics, with asphalt fragments [FILL]												
			20 to 30% gravel			21								4.5+*	PP
5	20					19					14	121	4654		UU
			CLAYEY GRAVEL (GC), dark reddish brown, medium dense, moist, 30 to 40% sand, 20 to 30% clay [FILL]			23					12	122		2.2	UC
10	15		GRAVELLY LEAN CLAY (CL), brown, very stiff, moist, 30 to 40% gravel, 10 to 20% sand [FILL]			41					13	125		4.5*	PP
			GRAVELLY SANDY CLAY (GC), brown, medium dense, moist, 25% gravel [FILL]			21				42					
15	10					20									
			LEAN CLAY (CL), brown, medium stiff, moist, <5% gravel, <5% sand [FILL]			9								1.5*	PP
20															

LOG OF BORING 2-B2

ENGEO Expect Excellence

LATITUDE: 37.5048 LONGITUDE: -121.9979

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

DATE DRILLED: 4/3/2018
HOLE DEPTH: Approx. 81½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 26 ft.

LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), pale olive mottled with yellowish brown, stiff, <5% sand [FILL]												
	5					14					26	101	1212		UU
25															
	0		Medium stiff			10					31	96	1,000*		TV
30			FAT CLAY (CH), grayish green, soft to medium stiff, moist [NATIVE]												
	-5					11					33	95	1,300*		TV
35															
	-10														
						50psi									
			Grades to sandy clay with trace gravel			100psi									
			SANDY CLAY AND SANDY SILT (CL-ML), grayish green, wet			150psi									
40															

LOG - GEOTECHNICAL_SU+QU W/ ELEV GINT 2-B1 THROUGH 2-B3.GPJ ENGEO INC.GDT 12/2/18

LOG OF BORING 2-B2

ENGEO Expect Excellence

LATITUDE: 37.5048 LONGITUDE: -121.9979

Geotechnical Exploration
Sobrato Area 4 - City Parcel
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12165.000.002

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SURF ELEV (NGVD29): Approx. 26 ft.

LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-15		LEAN CLAY (CL), grayish green, stiff, moist, <5% sand, <5% gravel, with wood fragments			18					20	112	1,300*		TV
45	-20		Pale olive mottled with yellowish brown, very stiff			53							3,250*		TV
50	-25														
55	-30		Grayish green mottled with yellowish brown, stiff			17					27	100	1417		UU
60			POORLY GRADED SAND (SP), brown, medium dense, wet												

LOG OF BORING 2-B2

ENGEO Expect Excellence

LATITUDE: 37.5048 LONGITUDE: -121.9979

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

DATE DRILLED: 4/3/2018
HOLE DEPTH: Approx. 81½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 26 ft.

LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			POORLY GRADED SAND (SP), brown, medium dense, wet			29	21	17	4	7	18				
	-35		LEAN CLAY (CL), brown, stiff, wet, 10 to 20% sand										1,000*		TV
65															
	-40		Grayish green, stiff, <5% angular gravel			17					28	99	1,000*		TV
70															
	-45		Mottled with yellowish brown			11							1,000*		TV
			SILT (ML), grayish green, medium stiff, wet, 20 to 30% fine grained sand												
75															
	-50		Grades to 5 to 10% sand			12							375*		TV
			POORLY GRADED SAND WITH CLAY (SP-SC), brown, medium dense, wet, 5 to 10% clay												
80															

LOG OF BORING 2-B2

ENGEO
Expect Excellence

LATITUDE: 37.5048 LONGITUDE: -121.9979

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

DATE DRILLED: 4/3/2018
HOLE DEPTH: Approx. 81½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 26 ft.

LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
	-55		POORLY GRADED SAND WITH CLAY (SP-SC), brown, medium dense, wet, 5 to 10% clay			33									
			Boring terminated at an approximate depth of 81.5 feet below existing grade. No groundwater data was obtained.												

LOG OF BORING 2-B3

ENGEO — Expect Excellence

LATITUDE: 37.5051 LONGITUDE: -121.9977

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

DATE DRILLED: 4/4/2018
HOLE DEPTH: Approx. 41½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 17 ft.

LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			GRAVELLY LEAN CLAY (CL), brown, hard, moist, 40 to 50% gravel, 10 to 20% sand, <5% organics [FILL]												
	15					44					8	124		4.3	UC
5						56					8	134		4.5+*	PP
	10		dark brown			26									
10			LEAN CLAY (CL), dark brown, very stiff, moist, 5 to 10% gravel, <5% sand [FILL]			29					21	106	2806		UU
	5					20							3,000*		TV
			LEAN CLAY (CL), light brown, medium stiff, moist, <5% sand [FILL]												
15			LEAN CLAY (CL), pale olive, medium stiff to stiff, moist [NATIVE]			8					23	104	1545		UU
	0														
20															

LOG OF BORING 2-B3

ENGEO — Expect Excellence

LATITUDE: 37.5051 LONGITUDE: -121.9977

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

DATE DRILLED: 4/4/2018
HOLE DEPTH: Approx. 41½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 17 ft.

LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			FAT CLAY (CH), grayish green mottled with light yellowish brown, stiff, wet			9					35	89	1,000*		TV
	-5														
25			<5% gravel and organics												
	-10														
30			CLAYEY SAND TO SILTY SAND (SC-SM), grayish green, loose, wet			8	21	15	6	34	18				
	-15														
			LEAN CLAY (CL), grayish green, very stiff, wet Increasing clay content at 33'												
35						33							2,500*		TV
	-20														
40															

LOG OF BORING 2-B3

ENGEO Expect Excellence

LATITUDE: 37.5051 LONGITUDE: -121.9977

Geotechnical Exploration
Sobrato Area 4 - City Parcel
Newark, CA
12165.000.002

DATE DRILLED: 4/4/2018
HOLE DEPTH: Approx. 41½ ft.
HOLE DIAMETER: 5.0 in.
SURF ELEV (NGVD29): Approx. 17 ft.

LOGGED / REVIEWED BY: B. Hassett / JK
DRILLING CONTRACTOR: Pitcher Drilling
DRILLING METHOD: Mud Rotary
HAMMER TYPE: 140 lb. Auto Trip

Depth in Feet	Elevation in Feet	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count/Foot	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Limit	Atterberg Limits: Plasticity Index	Fines Content (% passing #200 sieve)	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Shear Strength (psf) *field approximation	Unconfined Strength (tsf) *field approximation	Strength Test Type
			LEAN CLAY (CL), grayish green, very stiff, wet Increasing clay content at 33'			11							500*		TV
			Boring terminated at an approximate depth of 41.5 feet below existing grade. No groundwater data was obtained.												

APPENDIX B

LABORATORY TEST RESULTS

MOISTURE-DENSITY DETERMINATION
ASTM D7263

BORING ID:	2-B1	2-B1	2-B1	2-B2	2-B2	2-B2	2-B2	2-B2
DEPTH (ft.):	20-23	26	56	11	26	31	41	60.5
MOISTURE CONTENT (%):	21.3	18.2	22.6	12.8	30.9	33.4	20.1	18.2
DRY DENSITY (lbs/ft^3):	109.9	115.5		124.5	95.5	94.9	111.7	

BORING ID:	2-B2	2-B3	2-B3	2-B3				
DEPTH (ft.):	66	6	21	31				
MOISTURE CONTENT (%):	27.9	8.4	34.8	18.1				
DRY DENSITY (lbs/ft^3):	98.8	133.9	89.0					

PROJECT NAME: Sobrato Property - Area 4
PROJECT NUMBER: 12165.000.002
CLIENT: The Sobrato Organization
PHASE NUMBER: 2

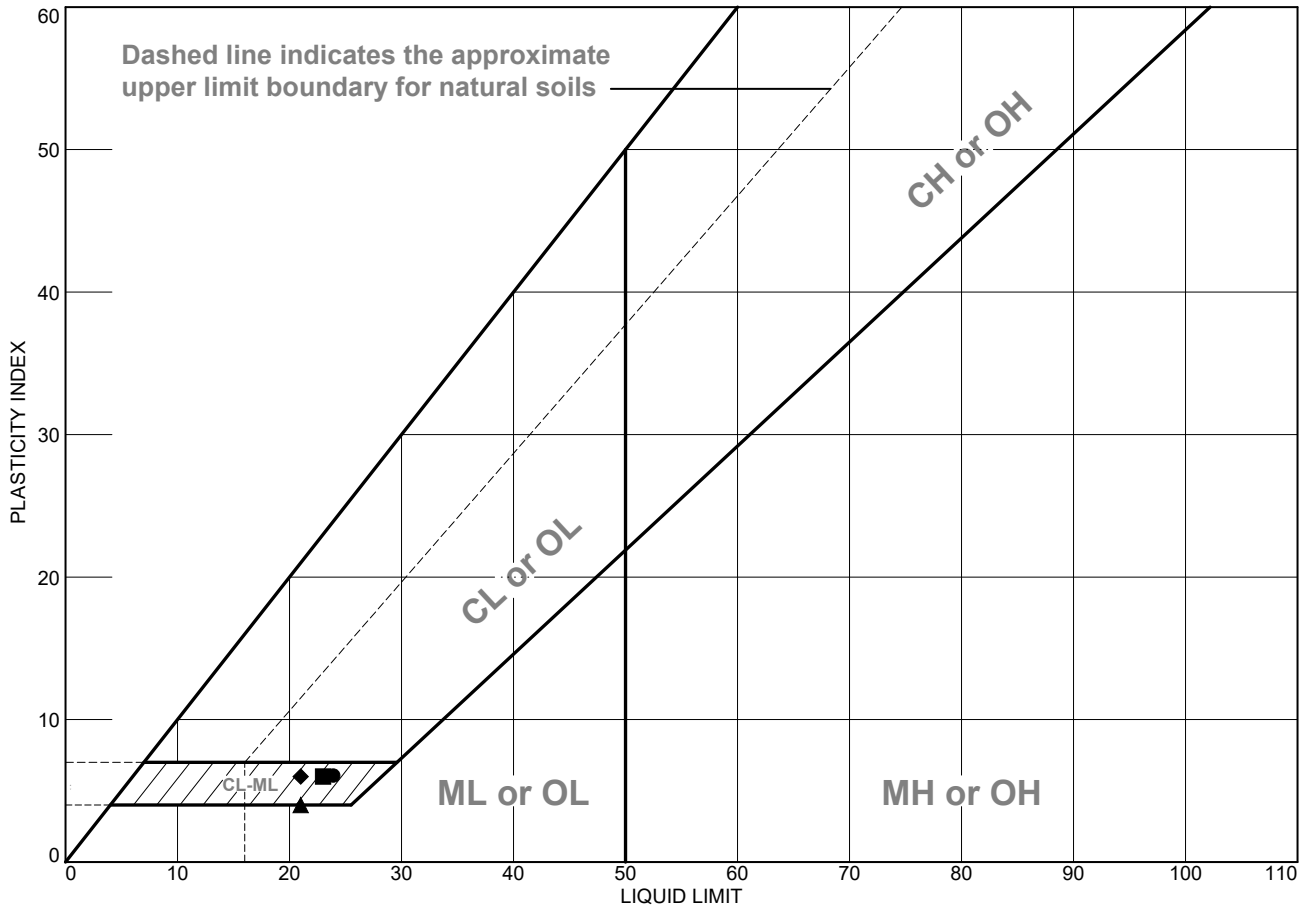
DATE: 04/17/18

ENGEO
Expect Excellence

Tested by: M. Bromfield Reviewed by: M. Quasem

ENGEO Incorporated 2010 Crow Canyon Place Suite 250 San Ramon, CA 94583 Laboratory address: 3420 Fostoria Way San Ramon, CA 94583

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	See exploration logs	24	18	6		63.1	
■	See exploration logs	23	17	6		21.3	
▲	See exploration logs	21	17	4		6.9	
◆	See exploration logs	21	15	6		34.3	

Project No. 12165.000.002 **Client:** The Sobrato Organization

Project: Sobrato Property - Area 4

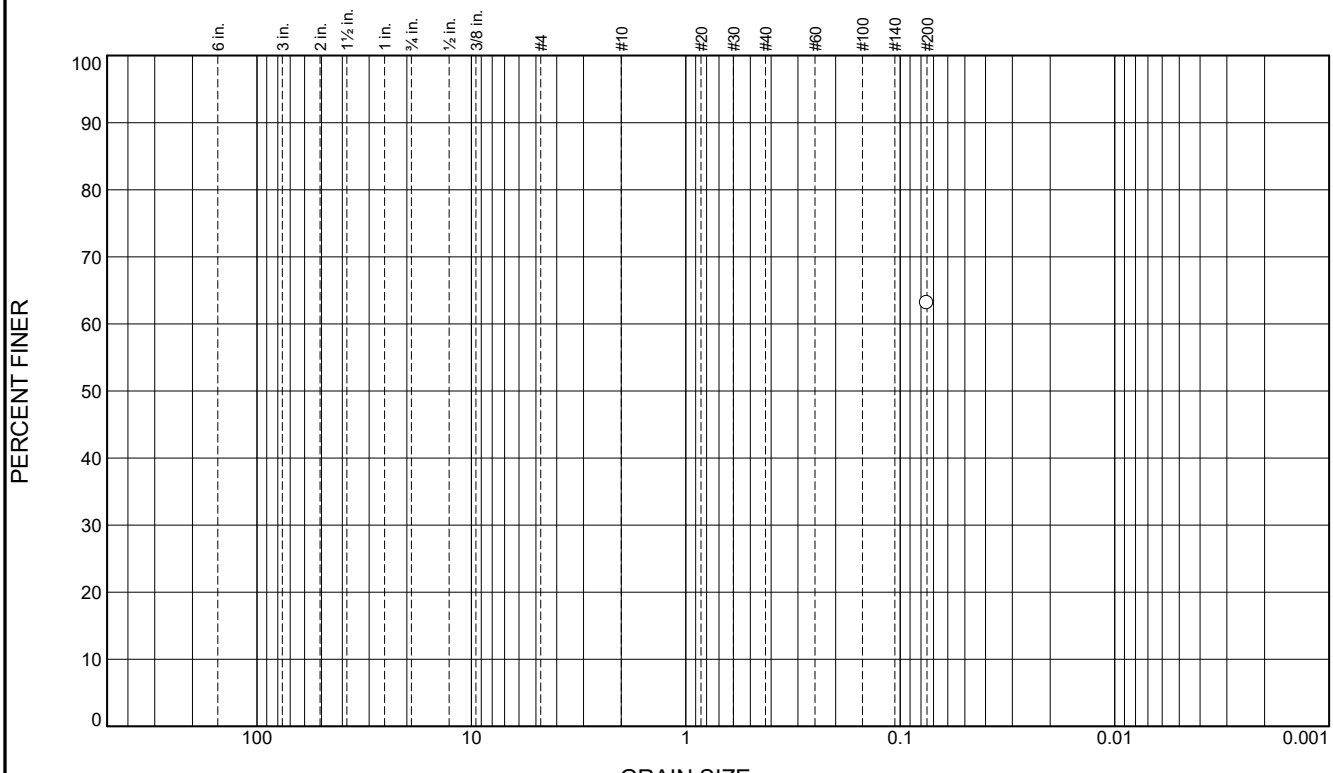
●**Sample Number:** 2-B1 @ 20-23
■**Sample Number:** 2-B1 @ 56
▲**Sample Number:** 2-B2 @ 60.5
◆**Sample Number:** 2-B3 @ 31

Remarks:
●PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
■PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
▲PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B
◆PI: ASTM D4318, Wet method
GS: ASTM D1140, Method B

ENGEO INCORPORATED

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						63.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	63.1		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 18 LL= 24 PI= 6

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 51.24; Soak Time = 4 hrs 10 min
PI: ASTM D4318, Wet method

Sample Number: 2-B1 @ 20-23

Date: 4/17/2018

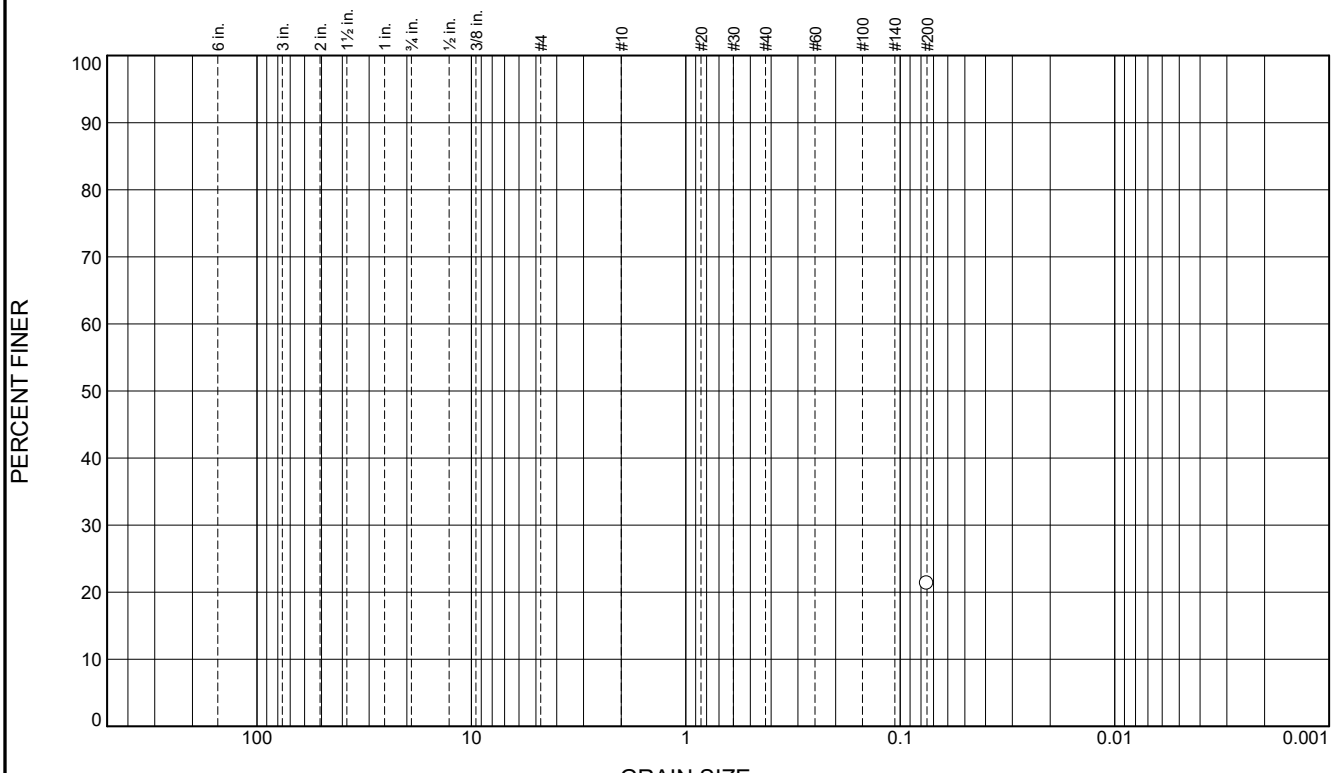
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						21.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	21.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 17 LL= 23 PI= 6

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 338.19; Soak Time = 4 hrs 10 min
PI: ASTM D4318, Wet method

Sample Number: 2-B1 @ 56

Date: 4/16/2018

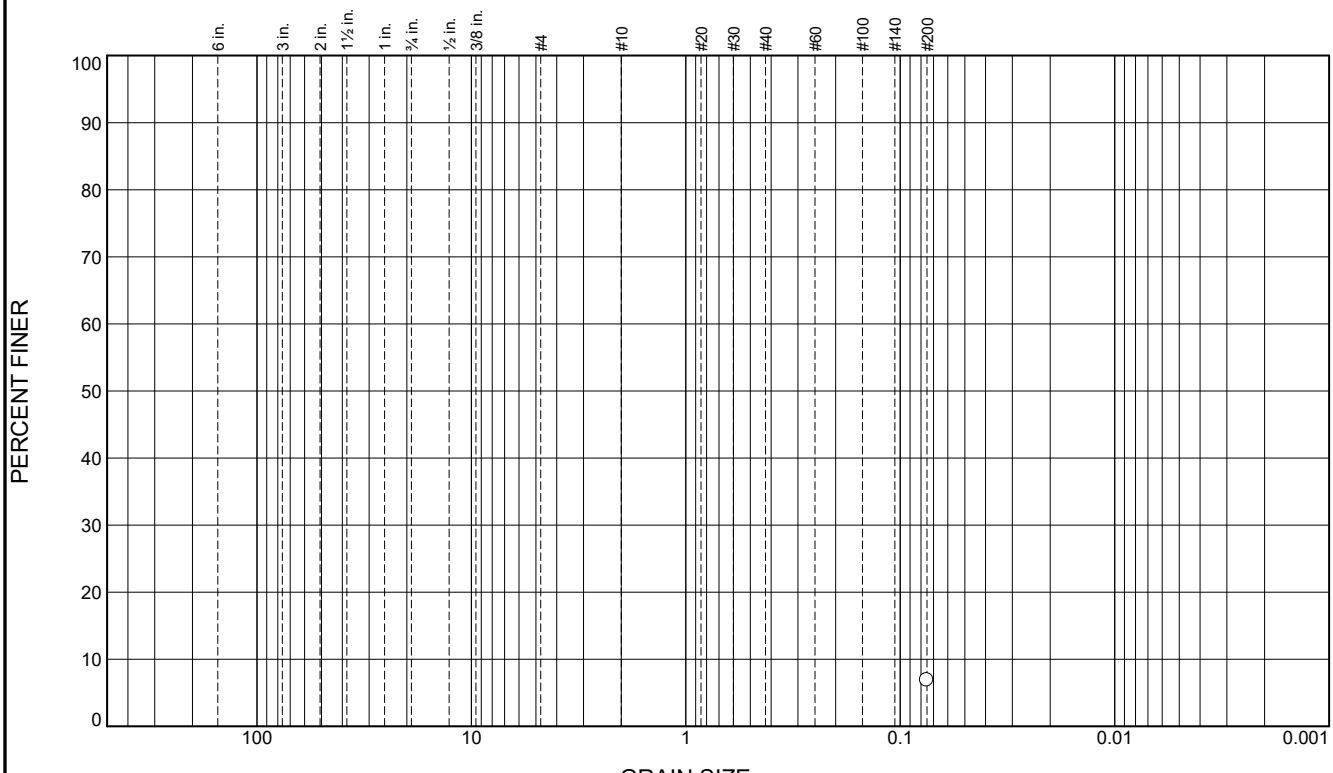
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						6.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	6.9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 17 LL= 21 PI= 4

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 376.04; Soak Time = 4 hrs 10 min

Sample Number: 2-B2 @ 60.5

Date: 4/16/2018

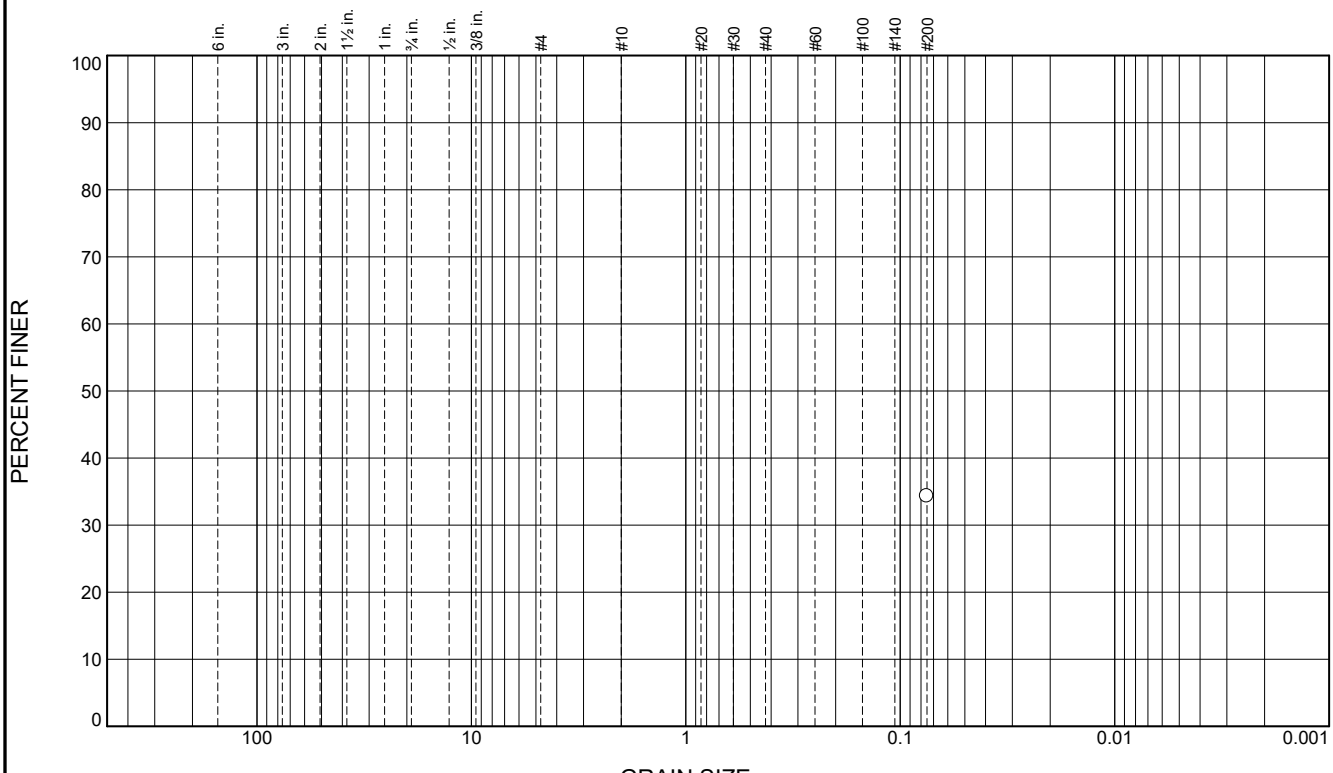
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						34.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	34.3		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= 15 LL= 21 PI= 6

Coefficients

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

Classification

USCS= AASHTO=

Remarks

GS: ASTM D1140, Method B
Dry Sample Weight = 413.49; Soak Time = 4 hrs 10 min

Sample Number: 2-B3 @ 31

Date: 4/16/2018

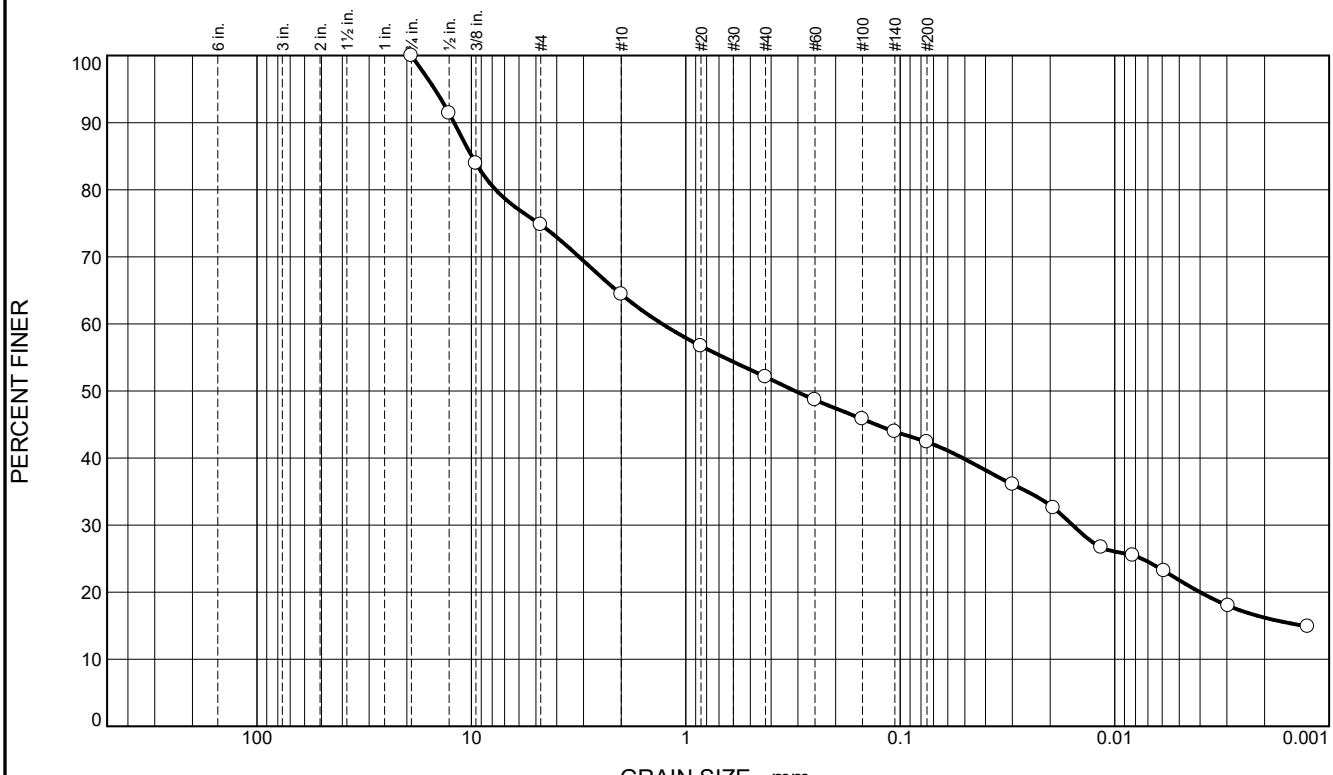
ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4

Project No: 12165.000.002

Tested By: M. Bromfield **Checked By:** M. Quasem

Particle Size Distribution Report



% +75mm	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	25.2	10.4	12.3	9.7	26.2	16.2

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	91.4		
3/8	83.9		
#4	74.8		
#10	64.4		
#20	56.7		
#40	52.1		
#60	48.7		
#100	45.8		
#140	43.9		
#200	42.4		
0.0299 mm.	36.1		
0.0193 mm.	32.6		
0.0115 mm.	26.7		
0.0082 mm.	25.5		
0.0059 mm.	23.2		
0.0030 mm.	18.0		
0.0013 mm.	14.9		

* (no specification provided)

Soil Description

See exploration logs

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 12.0296 D_{85}= 9.9621 D_{60}= 1.2890
D_{50}= 0.3097 D_{30}= 0.0157 D_{15}= 0.0013
D_{10}= C_u= C_c=

Classification

USCS= AASHTO=

Remarks

ASTM D422
Silt/clay division of .002 mm was used.

Sample Number: 2-B2 @ 12.5 & 15

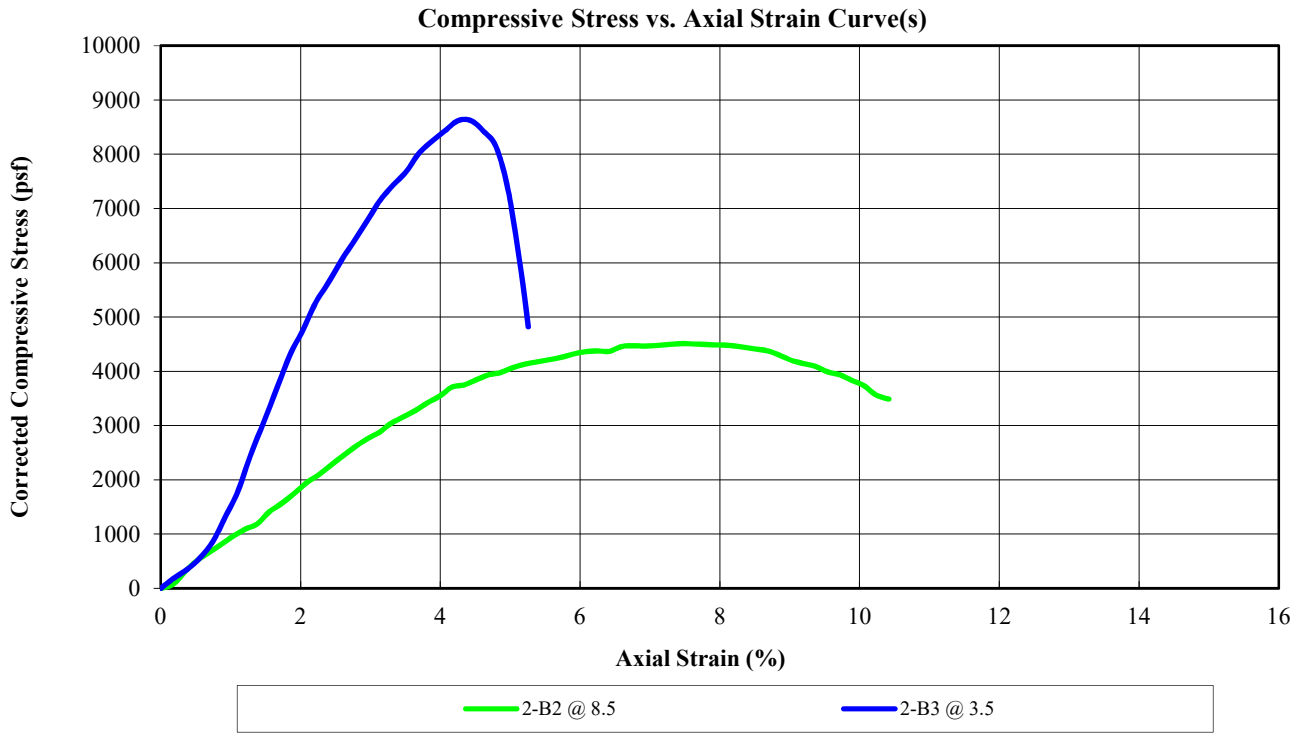
Date: 4/17/2018

ENGEO INCORPORATED

Client: The Sobrato Organization
Project: Sobrato Property - Area 4
Project No: 12165.000.002

Tested By: M. Bromfield **Checked By:** M. Quasem

UNCONFINED COMPRESSION TEST REPORT
(ASTM D2166)



	SPECIMEN			
BEFORE TEST	**2-B2 @ 8.5**	**2-B3 @ 3.5**		
Moisture Content (%)	12.0	8.1	0.0	0.0
Dry Density (pcf)	121.5	124.3	0.0	0.0
Saturation (%)	92.6	83.9		
Void Ratio	0.34	0.26		
Diameter (in)	2.421	2.376		
Height (in)	5.81	5.46		
Height-To-Diameter Ratio	2.40	2.30		
TEST DATA				
Unconfined Compressive Strength (psf)	4510	8623		
Undrained Shear Strength (psf)	2255	4311		
Strain Rate (in./min.)	0.05	0.05		
Specific Gravity	2.600	2.650		
Strain at Failure (%)	7.46	4.43		
Liquid Limit				
Plastic Limit				
Test Remarks				

SPECIMEN	DESCRIPTION
2-B2 @ 8.5	See exploration logs
2-B3 @ 3.5	See exploration logs

PROJECT NAME: Sobrato Property - Area 4
PROJECT NO: 12165.000.002
CLIENT: The Sobrato Organization
LOCATION: Newark, CA
PHASE NO: 002

Test Date: 04/16/18
Tested By: M. Bromfield
Reviewed By: M. Quasem

ENGEO
Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

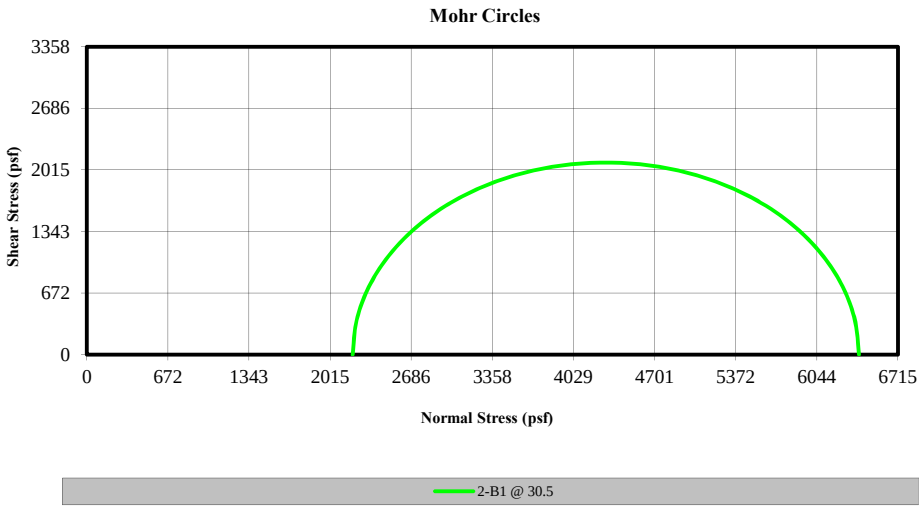
(ASTM D2850)

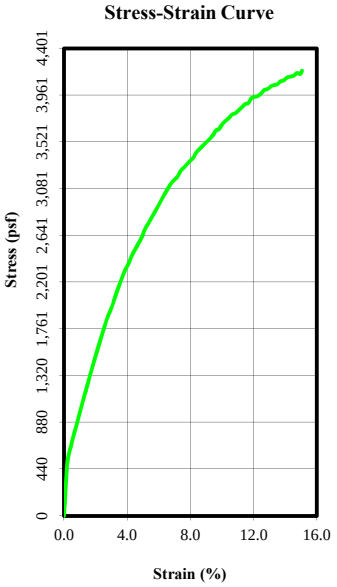
Date: 04/26/18

Checked By: C. Crawford

Date: 04/25/18

Tested By: K. Lecce





Before Test	Specimen 2-B1 @ 30.5			
Water Content (%)	20.17			
Dry Density (pcf)	111.74			
Saturation (%)	99.14			
Void Ratio	0.57			
Diameter (in)	2.380			
Height (in)	5.065			
Height-to-Diameter Ratio	2.128			
ASTM D4318 - Wet Method				
Liquid Limit	-			
Plastic Limit	-			
ASTM D854 - Measured				
Specific Gravity	2.815			
After Test	**2-B1 @ 30.5**			
Water Content (%)	20.17			
Saturation (%)	99.14			
Strain Rate (in/min)	0.05			
Peak Deviator Stress (psf)	4191.9			
Axial Strain @ Failure (%)	15.065			
	Cell Pressure			
Cell (psf)	2200.3			
Back (psf)	n/a			
	Principle Stresses at Failure			
σ1 (psf)	6392.2			
σ3 (psf)	2200.3			
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	0.0		2095.9			
Friction Angle Ø	0.00		n/a			

Project Information	
Project Name:	Sobrato Property - Area 4
Project Number:	12165.000.002
Project Location:	Newark, CA
Client:	Sobrato Organization
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

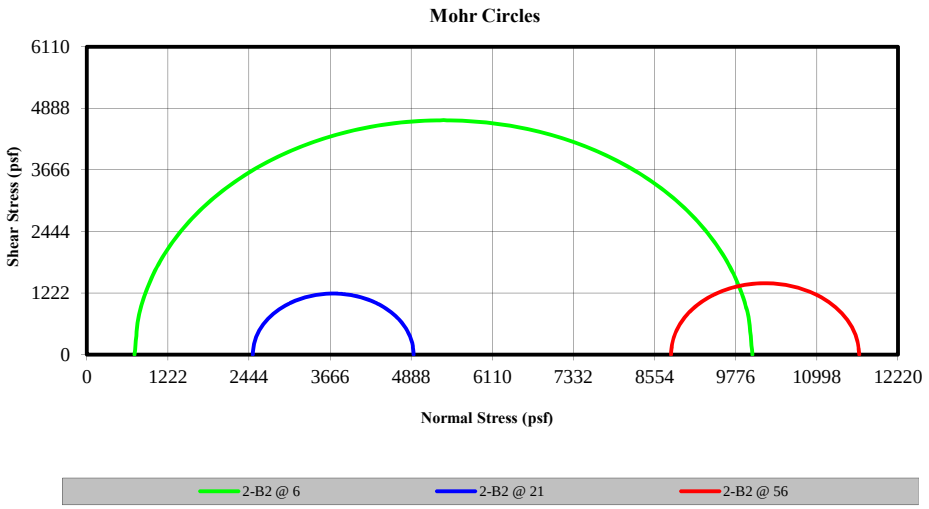
(ASTM D2850)

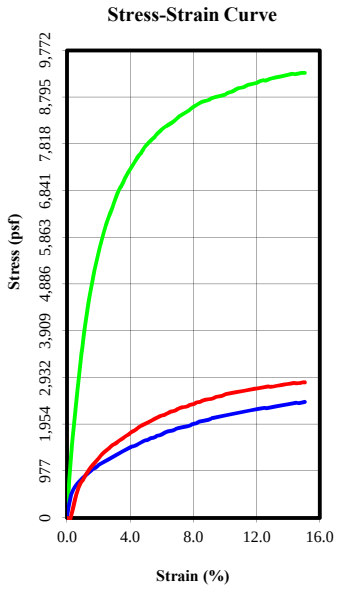
Date: 04/23/18

Checked By: C. Crawford

Date: 04/20/18

Tested By: K. Lecce





Before Test	Specimen 2-B2 @ 6	2-B2 @ 21	2-B2 @ 56	
Water Content (%)	14.20	25.70	27.31	
Dry Density (pcf)	120.58	101.14	99.99	
Saturation (%)	99.35	99.55	99.86	
Void Ratio	0.38	0.72	0.78	
Diameter (in)	2.390	2.419	2.410	
Height (in)	5.090	5.082	5.055	
Height-to-Diameter Ratio	2.130	2.101	2.098	
ASTM D4318 - Wet Method				
Liquid Limit	-	-	-	
Plastic Limit	-	-	-	
ASTM D854 - Assumed				
Specific Gravity	2.668	2.785	2.850	
After Test	**2-B2 @ 6**	**2-B2 @ 21**	**2-B2 @ 56**	
Water Content (%)	14.20	25.70	27.31	
Saturation (%)	99.35	99.55	99.86	
Strain Rate (in/min)	0.05	0.05	0.05	
Peak Deviator Stress (psf)	9307.1	2424.3	2834.8	
Axial Strain @ Failure (%)	14.866	15.060	14.969	
	Cell Pressure			
Cell (psf)	720.0	2499.8	8799.8	
Back (psf)	n/a	n/a	n/a	
	Principle Stresses at Failure			
σ1 (psf)	10027.1	4924.1	11634.6	
σ3 (psf)	720.0	2499.8	8799.8	
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	0.0		4653.6	1212.1	1417.4	0.0
Friction Angle Ø	0.00		n/a	n/a	n/a	n/a

Project Information	
Project Name:	Sobrato Property - Area 4
Project Number:	12165.000.002
Project Location:	Newark, CA
Client:	Sobrato Organization
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Isotropic Unconsolidated Undrained Triaxial Test

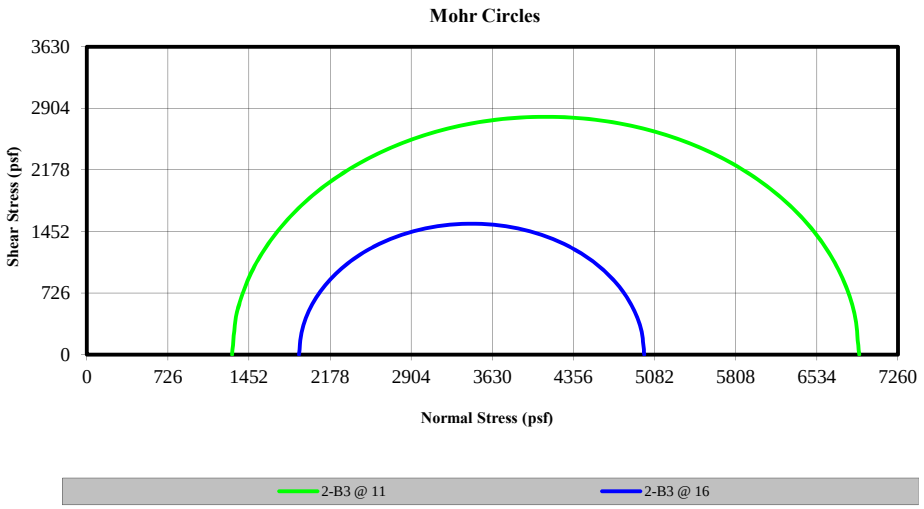
(ASTM D2850)

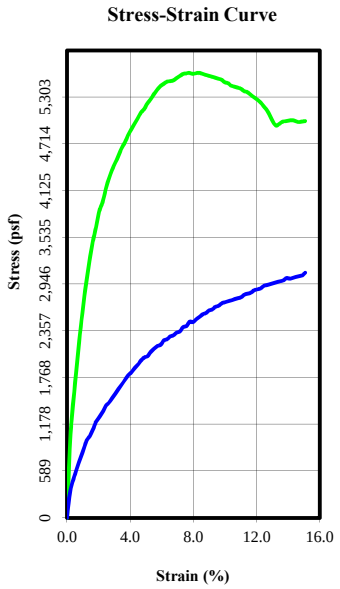
Date: 04/23/18

Checked By: C. Crawford

Date: 04/20/18

Tested By: K. Lecce





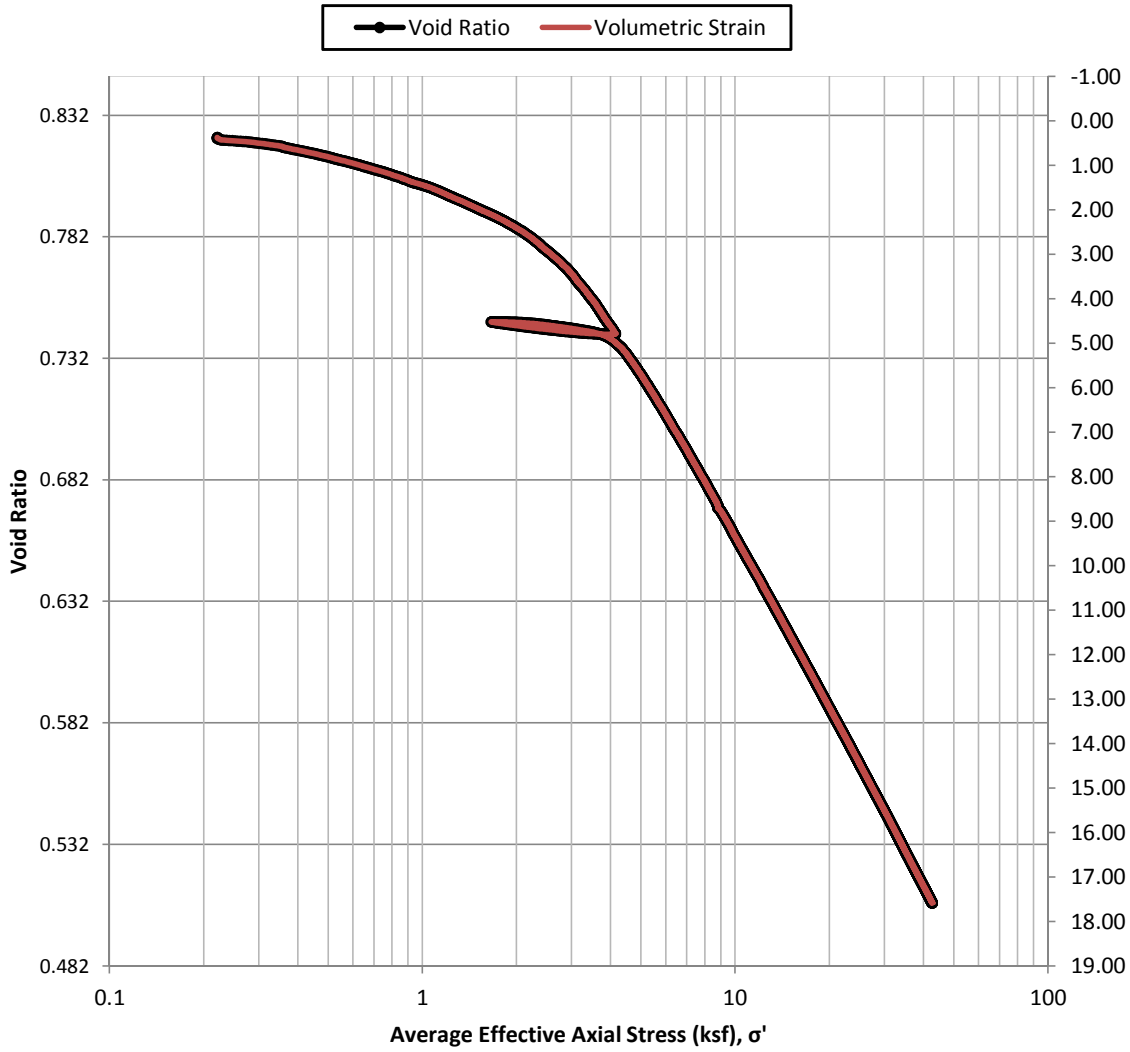
Before Test	Specimen 2-B3 @ 11	2-B3 @ 16		
Water Content (%)	20.89	23.30		
Dry Density (pcf)	106.00	104.41		
Saturation (%)	98.74	100.00		
Void Ratio	0.56	0.58		
Diameter (in)	2.390	2.385		
Height (in)	5.090	5.075		
Height-to-Diameter Ratio	2.130	2.128		
ASTM D4318 - Wet Method				
Liquid Limit	-	-		
Plastic Limit	-	-		
ASTM D854 - Measured				
Specific Gravity	2.650	2.650		
After Test	**2-B3 @ 11**	**2-B3 @ 16**		
Water Content (%)	20.89	23.30		
Saturation (%)	98.74	100.00		
Strain Rate (in/min)	0.05	0.05		
Peak Deviator Stress (psf)	5611.7	3089.9		
Axial Strain @ Failure (%)	8.350	15.069		
	Cell Pressure			
Cell (psf)	1300.3	1899.4		
Back (psf)	n/a	n/a		
	Principle Stresses at Failure			
σ1 (psf)	6912.0	4989.3		
σ3 (psf)	1300.3	1899.4		
Corrected Peak Deviator Stress				

Mohr-Coulomb Parameters with a Non-zero Friction Angle (Ø≠0)			Cohesion at Failure with a Zero Friction Angle (Ø=0)			
Cohesion, c (psf)	0.0		2805.9	1545.0		
Friction Angle Ø	0.00		n/a	n/a		

Project Information	
Project Name:	Sobrato Property - Area 4
Project Number:	12165.000.002
Project Location:	Newark, CA
Client:	Sobrato Organization
Description:	See exploration logs
Test Remarks:	

ENGEO
Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	28.57%	19.18%	Liquid Limit:	
Dry Density (pcf):	95.06	115.36	Plastic Limit:	
Saturation (%):	96.09%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.8297	0.5077	Specific Gravity:	2.791
			Soil Description:	See exploration logs

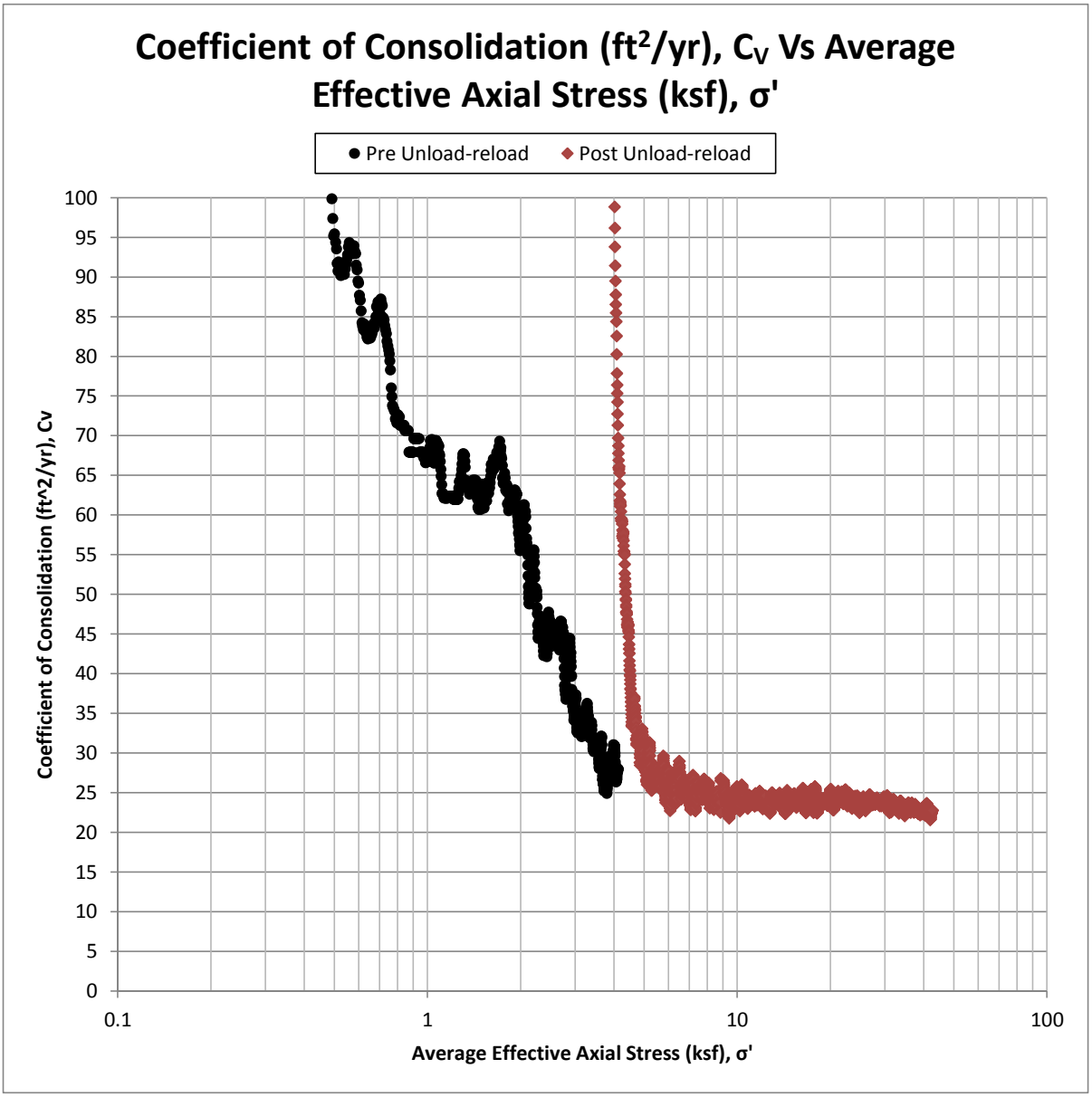
Test Date: 4/13/2018

Project Number:	12165.000.002	**Depth:**	15.5-16.0 ft
Sample Number:	2-B1 @ 16	**Boring #:**	2-B1
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks:

ENGEO Expect Excellence

Constant Rate of Strain Consolidation
ASTM D4186

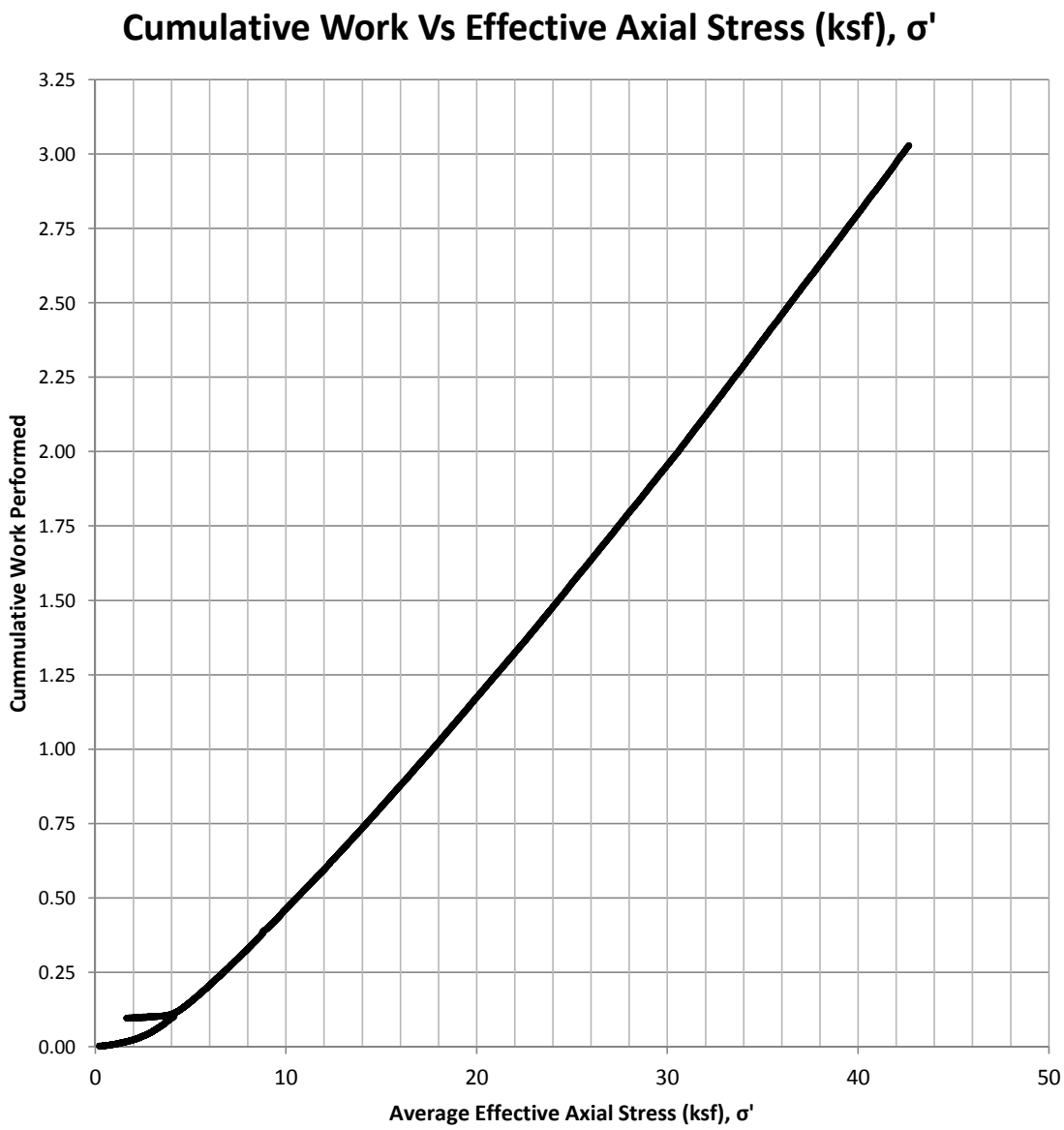


ASTM D2974 - 2974 Method A (OD mass)					
	Initial	Final			Test Date: 4/13/2018
Moisture (%):	28.57%	19.18%	ASTM D4318 - Wet Method		
Dry Density (pcf):	95.06	115.36	Liquid Limit:		
Saturation (%):	96.09%	100.00%	Plastic Limit:		
Void Ratio:	0.8297	0.5077	ASTM D854 - Measured		
			Specific Gravity:	2.791	
			Soil Description:	See exploration logs	

Project Number:	12165.000.002	Depth:	15.5-16.0 ft
Sample Number:	2-B1 @ 16	Boring #:	2-B1
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	Reviewed By:	J. Kan

Remarks:

Constant Rate of Strain Consolidation
ASTM D4186



ASTM D2974 - Method A (OD mass)				
	Initial	Final	ASTM D4318 - Wet Method	
Moisture (%):	28.57%	19.18%	Liquid Limit:	
Dry Density (pcf):	95.06	115.36	Plastic Limit:	
Saturation (%):	96.09%	100.00%	ASTM D854 - Measured	
Void Ratio:	0.8297	0.5077	Specific Gravity:	2.791
			Soil Description:	See exploration logs

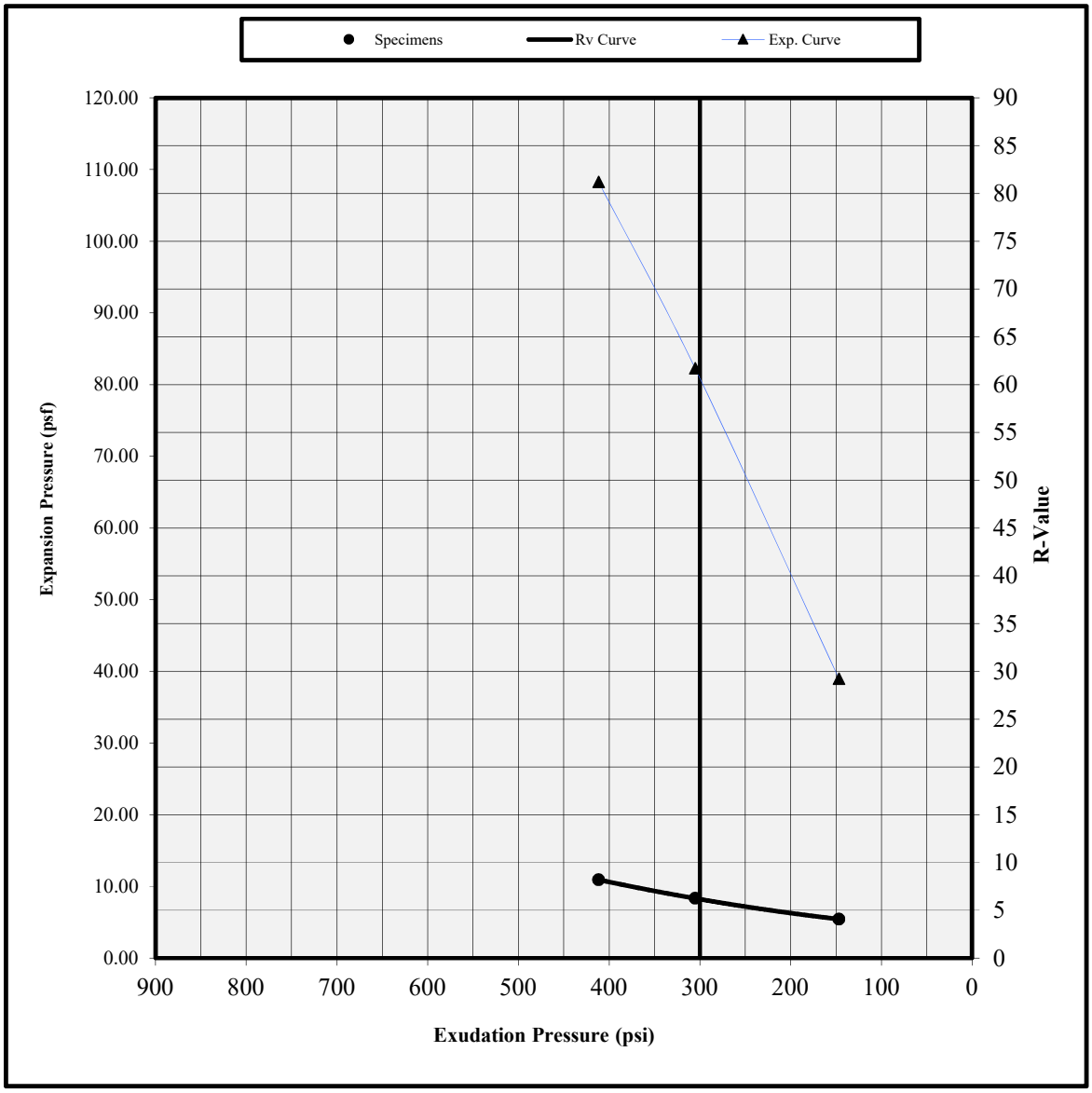
Test Date: 4/13/2018

Project Number:	12165.000.002	**Depth:**	15.5-16.0 ft
Sample Number:	2-B1 @ 16	**Boring #:**	2-B1
Project Name:	Sobrato - Area 4		
Client:	Sobrato Organization		
Location:	Newark, California		
Tested By:	D. Seibold	**Reviewed By:**	J. Kan

Remarks:

ENGEO Expect Excellence

R VALUE TEST REPORT
CTM-301



Sample ID/Location: Embankment 0'-2'
Description: Brown sandy CLAY
Test remarks:

Specimen	Specimen 1	Specimen 2	Specimen 3
Exudation Pressure (p.s.i.)	412	305	147
Expansion dial (0.0001")	25	19	9
Expansion Pressure (p.s.f.)	108	82	39
Resistance Value, "R"	8	6	4
% Moisture at Test	18.5	19.4	21.2
Dry Density at Test, p.c.f.	108.7	107.5	104.0
"R" Value at Exudation Pressure of 300 psi.	6		
Expansion Pressure (psf) at Exudation Pressure of 300 psi.	81		

PROJECT NAME: Sobrato Property - Area 4
PROJECT NUMBER: 12165.000.002
CLIENT: The Sobrato Organization
PHASE NUMBER: 002

DATE: 04/13/18

ENGEO
Expect Excellence

Tested by: W. Miller
Reviewed by: G. Criste

Lab Address : 3420 Fostoria Way, Suite E, San Ramon, CA 94583

WATER SOLUBLE SULFATES IN SOILS

ASTM C1580

Sample number	Sample Location / ID	Matrix	Water Soluble Sulfate % by mass
1	2-B1 @ 6.5	soil	0.01
2	2-B2 @ 46	soil	ND
3	2-B3 @ 25-28	soil	ND

Remarks: Results are reported to the nearest 100mg/kg. Anything less than 50mg/kg will be reported as 'ND' for Not-Detectable.

PROJECT NAME: Sobrato Property - Area 4
PROJECT NUMBER: 12165.000.002
CLIENT: The Sobrato Organization
PHASE NUMBER: 002

DATE: 04/13/18

ENGEO
Expect Excellence

Tested by: M. Bromfield

Reviewed by: M. Quasem

Lab Address: 3420 Fostoria Way Suite E, San Ramon, CA 94583. Phone No. (925) 355-9047

APPENDIX C

CERCO TEST RESULTS

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: ENGEO Incorporated
Client's Project No.: 12165.000.002
Client's Project Name: Sobrato Area 4 - City Parcel
Date Sampled: 9-Apr-18
Date Received: 11-Apr-18
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 19-Apr-2018

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1804086-001	2-B1 @ 10'	440	7.57	-	510	N.D.	440	97

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	19-Apr-2018	19-Apr-2018	-	19-Apr-2018	16-Apr-2018	18-Apr-2018	18-Apr-2018

* Results Reported on "As Received" Basis
N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

1804086 12398

CHAIN OF CUSTODY RECORD

PROJECT NUMBER: 12165.000.002

PROJECT NAME: Sobrato Area 4 - City Parcel

SAMPLED BY: (SIGNATURE/PRINT) B. Hassett

PROJECT MANAGER: J. Kan

ROUTING: E-MAIL bhassett@engeo.com jkan@engeo.com

Hard Copy None

	SAMPLE NUMBER	DATE	TIME	MATRIX	NUMBER OF CONTAINERS	CONTAINER SIZE	PRESERVATIVE	TPH - Gas	TPH - Diesel/Oil	CAM 17 Metals	SVOCs EPA 8270	VOCs EPA 8260b	Pesticides EPA 8081	PCBs EPA 8082	PNAs	Cyanide	pH	Dissolved Gases (Explosive)	ASTM Corrosivity Package					REMARKS REQUIRED DETECTION LIMITS
001	2-B1 @ 10'	4/9/2018		S	1	baggie	none												X					

RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
	4/10/18 4 PM			4/11/18 2:26pm	
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)	RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED BY: (SIGNATURE)
	4/11/18 1525				
RELINQUISHED BY: (SIGNATURE)	DATE/TIME	RECEIVED FOR LABORATORY BY: (SIGNATURE)	DATE/TIME		

REMARKS

ASTM Redox, pH, Sulfate, resistivity, chloride, sulfide for sample

DISTRIBUTION: ORIGINAL ACCOMPANIES SHIPMENT; COPY TO PROJECT FIELD FILES

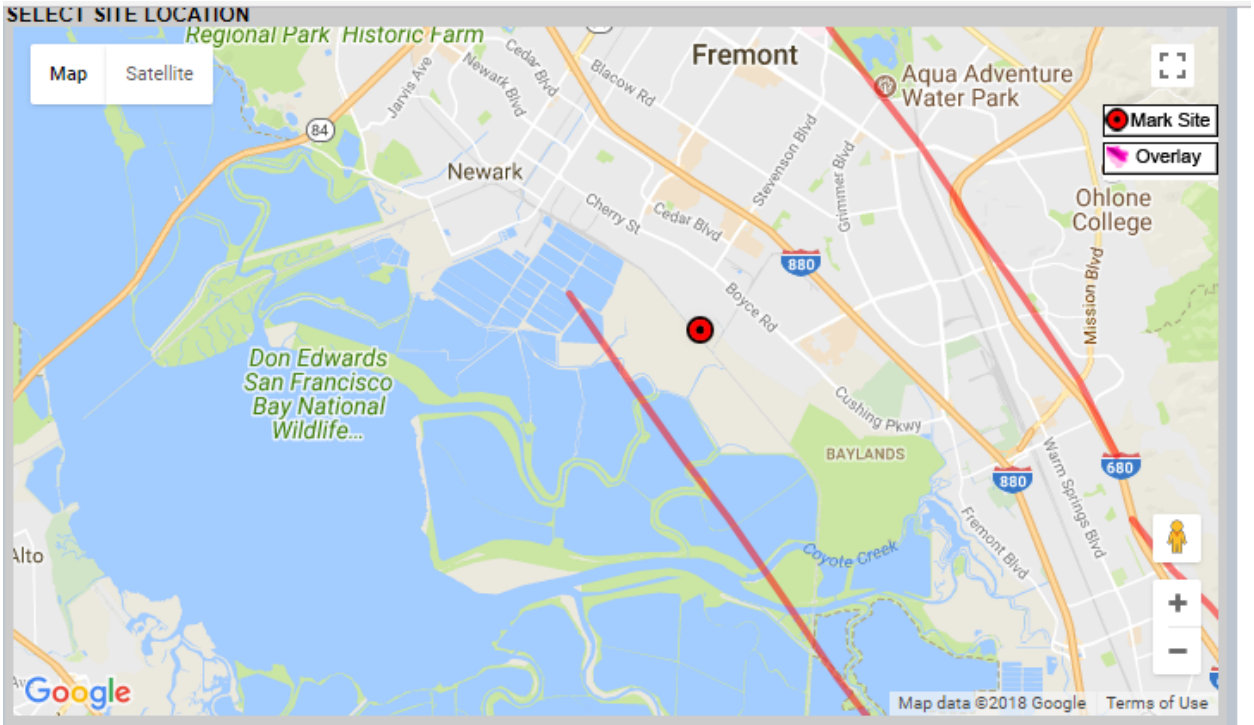
ENGEO INCORPORATED

101 California Street, Suite 875, San Francisco, CA 94111
(925) 866-9000 FAX (925) 866-0199
WWW.ENGEO.COM

APPENDIX D

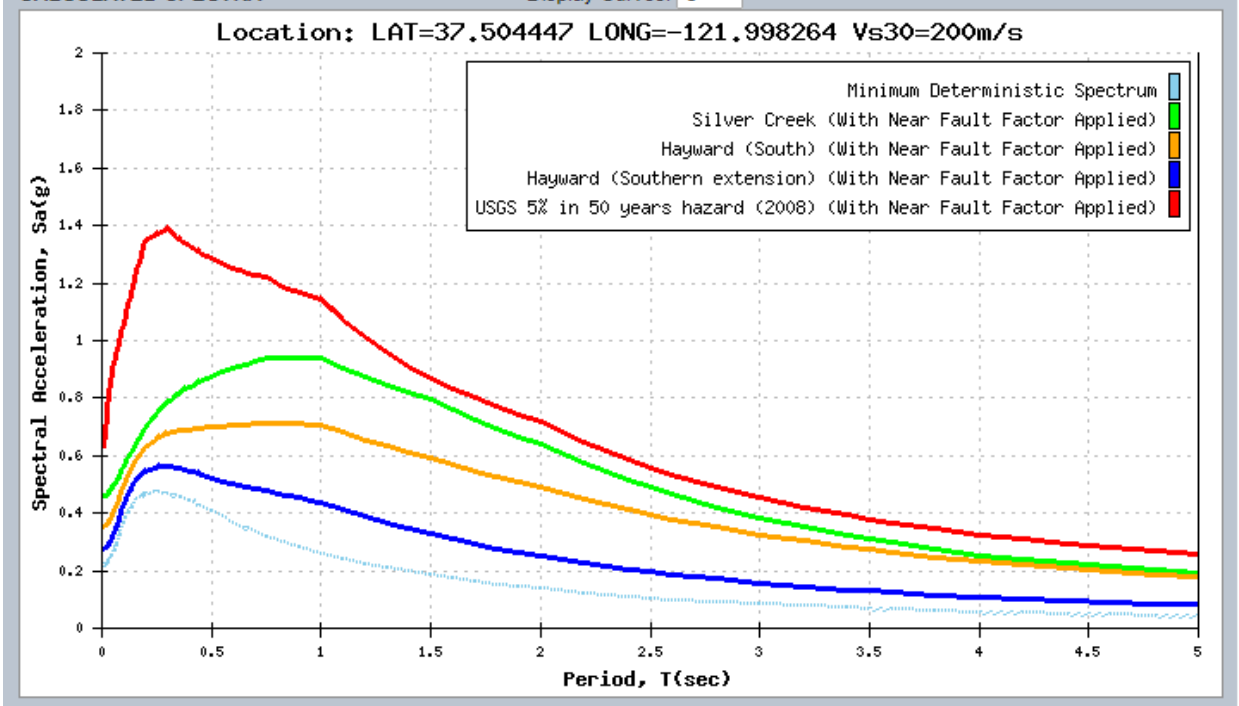
ARS CURVES

SELECT SITE LOCATION



Latitude: 37.504447 **Longitude:** -121.998264 **Vs30:** 200 m/s Calculate

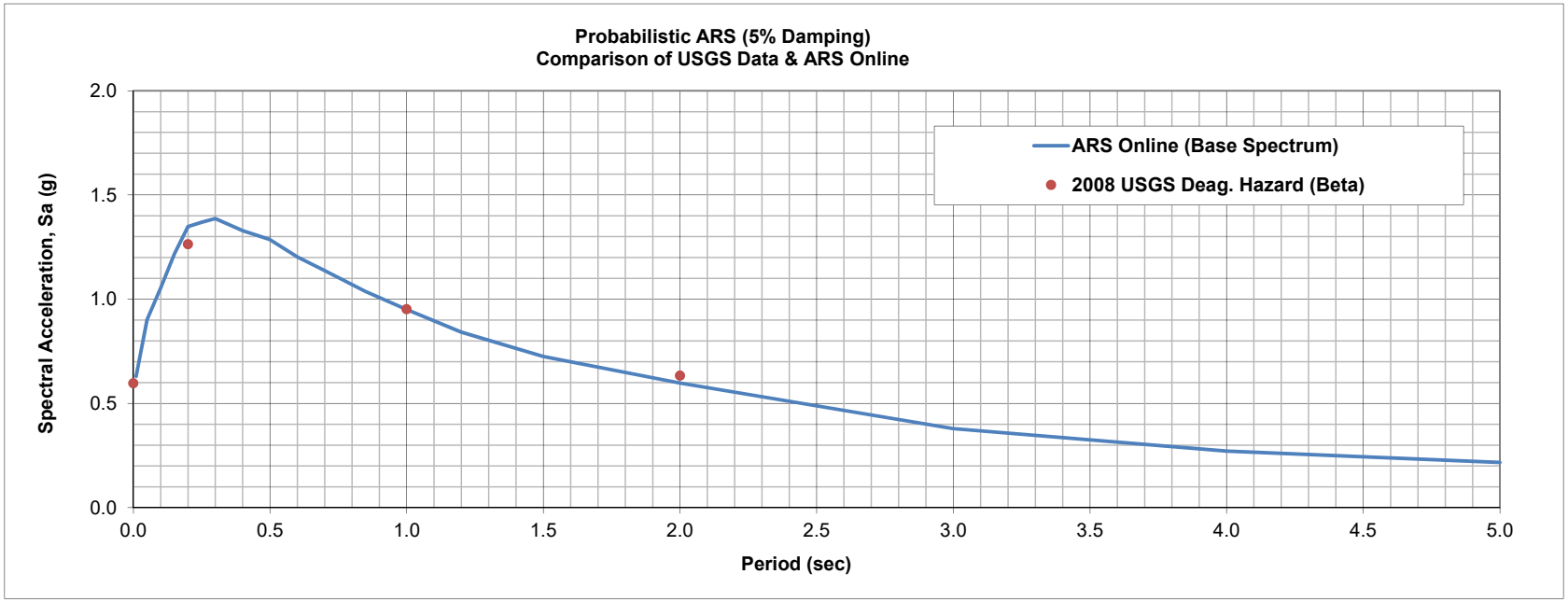
CALCULATED SPECTRA Display Curves: 3



Tabular Data | Envelope Only | Hide Near Fault | Axis Scale | Show Basin

Comparison spreadsheet of the 2008 USGS Probabilistic Seismic Hazard Data and ARS Online Probabilistic Data

- This spreadsheet facilitates a data check of the ARS Online base spectrum vs the USGS Data. This spreadsheet does not perform a deaggregation.
- Spectral acceleration points may be obtained from USGS Website at https://geohazards.usgs.gov/deaggint/2008/



Place ARS Online Probabilistic Data Here (Use 19 Period Data Option in ARS Online)				
T (sec)	Base Spectrum S(a)	Basin Factor	Near Fault Factor	Final Adj. Spectrum S(a)
0.01	0.63	1	1	0.63
0.05	0.902	1	1	0.902
0.1	1.054	1	1	1.054
0.15	1.217	1	1	1.217
0.2	1.348	1	1	1.348
0.25	1.369	1	1	1.369
0.3	1.387	1	1	1.387
0.4	1.329	1	1	1.329
0.5	1.286	1	1	1.286
0.6	1.203	1	1.04	1.251
0.7	1.137	1	1.08	1.228
0.85	1.037	1	1.14	1.182
1	0.951	1	1.2	1.141
1.2	0.842	1	1.2	1.01
1.5	0.725	1	1.2	0.87
2	0.598	1	1.2	0.718
3	0.379	1	1.2	0.455
4	0.271	1	1.2	0.325
5	0.217	1	1.2	0.26

Input USGS Deaggregation Hazard Data for a Exceedance Probability of 5% in 50yr			
Period (sec)	INPUT USGS Deagg. Spec Accel	ARS Online Base Sa(g)	% Difference (bet. USGS & ARS Online)
0	0.5967	0.63	5.3%
0.2	1.2636	1.348	6.3%
1	0.9518	0.951	0.1%
2	0.6335	0.598	5.9%

Max % Difference = 6.3%

APPENDIX E

LIQUEFACTION ANALYSIS

Liquefaction Evaluation - Idriss and Boulanger (2008)

Borehole: 2-B1

Input

Water Table depth at time of Exploration	Water Table depth at time of Liquefaction	amax/g	Mw
6	6	0.63	7.33

						At time of Exploration		At time of Liquefaction	
Boring Designation	Depth [m]	Depth [ft]	Soil Type	N_m [Blows/ft]	FC	Total Stress [psf]	Effective Stress [psf]	Total Stress [psf]	Effective Stress [psf]
2-B1	3.25	6.5	CL	4	100	390	390	390	390
2-B1	8.25	3.5	GP	14	5	990	849.6	990	849.6
2-B1	13.75	7.5	CH	5	100	1650	1166.4	1650	1166.4
2-B1	20	5	CL-ML	6	63	2400	1526.4	2400	1526.4
2-B1	37.75	30.5	CL	21	80	4530	2548.8	4530	2548.8
2-B1	53	14	SC-SM	17	21	6360	3427.2	6360	3427.2
2-B1	69.25	4.5	ML	8	70	8310	4363.2	8310	4363.2
2-B1	74.75	6.5	ML	46	70	8970	4680	8970	4680
2-B1	79.75	3.5	GP	36	5	9570	4968	9570	4968
0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0
0	0	0	0	0	0	0	0	0	0

N_m = Measured SPT Blow Count

Results

Boring Designation	Depth [ft]	CRR	CSR	FS	Ht. of Layer (ft)	(N1)60cs	Soil Type (USCS)	Liquefiable?	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔLDI	Calculated Volumetric Strain ε_v	Volumetric Strain Used ε_v	ΔS_i
2-B1	3.3	0.13	0.36	0.37	6.50	13	CL	Unsaturated	0%	0.00	0%	0.00	0.00%	0.00%	Unsaturated
2-B1	8.3	0.21	0.41	0.53	3.50	22	GP	Yes	13%	0.41	13%	0.45	2.13%	1.50%	0.63
2-B1	13.8	0.14	0.51	0.27	7.50	14	CH	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B1	20.0	0.14	0.56	0.26	5.00	14	CL-ML	Yes	29%	0.78	29%	1.46	2.95%	1.50%	0.90
2-B1	37.8	0.41	0.61	0.66	30.50	32	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B1	53.0	0.22	0.60	0.36	14.00	21	SC-SM	Yes	14%	0.46	14%	1.98	2.21%	2.21%	3.71
2-B1	69.3	0.13	0.56	0.23	4.50	13	ML	Yes	34%	0.83	34%	1.53	3.17%	1.50%	0.81
2-B1	74.8	THC	0.68	THC	6.50	50	ML	Yes	0%	-1.60	0%	0.00	0.00%	0.00%	0.00
2-B1	79.8	0.81	0.63	1.27	3.50	29	GP	Yes	5%	-0.01	2%	0.07	0.42%	0.42%	0.17
											LDI	5.49	Settlement (in)		6.22

TDL = Too Dense to Liquefy based on blowcount criteria
Volumetric Strains based on Figure 103

Liquefaction Evaluation - Idriss and Boulanger (2008)

Borehole: 2-B2

Input

Water Table depth at time of Exploration	Water Table depth at time of Liquefaction	amax/g	Mw
23	23	0.63	7.33

Boring Designation	Depth [m]	Depth [ft]	Soil Type	N_m [Blows/ft]	FC	At time of Exploration: Total Stress [psf]	At time of Exploration: Effective Stress [psf]	At time of Liquefaction: Total Stress [psf]	At time of Liquefaction: Effective Stress [psf]
2-B2	3.5	7	CL	12	90	420	420	420	420
2-B2	8.25	2.5	GC	15	20	990	990	990	990
2-B2	10.75	2.5	CL	27	60	1290	1290	1290	1290
2-B2	14.75	5.5	SM	20	42	1770	1770	1770	1770
2-B2	27.5	20	CL	9	90	3300	3019.2	3300	3019.2
2-B2	38.25	1.5	SC	6	12	4590	3638.4	4590	3638.4
2-B2	41	4	CL	12	90	4920	3796.8	4920	3796.8
2-B2	45.5	5	CL	35	90	5460	4056	5460	4056
2-B2	53	10	CL	12	90	6360	4488	6360	4488
2-B2	59.5	3	SP-SM	20	7	7140	4862.4	7140	4862.4
2-B2	67	12	CL	7	80	8040	5294.4	8040	5294.4
2-B2	75.5	5	ML	8	70	9060	5784	9060	5784
2-B2	79.75	3.5	SP-SC	22	5	9570	6028.8	9570	6028.8

N_m = Measured SPT Blow Count

Results

Boring Designation	Depth [ft]	CRR	CSR	FS	Ht. of Layer (ft)	(N1)60cs	Soil Type (USCS)	Liquefiable?	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔLDI	Calculated Volumetric Strain ε_v	Volumetric Strain Used ε_v	ΔS_i
2-B2	3.5	0.29	0.36	0.82	7.00	28	CL	Unsaturated	0%	0.00	0%	0.00	0.00%	0.00%	Unsaturated
2-B2	8.3	0.30	0.35	0.84	2.50	27	GC	Unsaturated	0%	0.00	0%	0.00	0.00%	0.00%	Unsaturated
2-B2	10.8	3.55	0.35	FS>2.5	2.50	49	CL	Unsaturated	0%	0.00	0%	0.00	0.00%	0.00%	Unsaturated
2-B2	14.8	0.47	0.37	1.28	5.50	34	SM	Unsaturated	0%	0.00	0%	0.00	0.00%	0.00%	Unsaturated
2-B2	27.5	0.15	0.40	0.37	20.00	16	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B2	38.3	0.10	0.44	0.23	1.50	7	SC	Yes	50%	0.95	50%	0.75	4.47%	4.47%	0.80
2-B2	41.0	0.17	0.45	0.37	4.00	18	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B2	45.5	THC	0.52	THC	5.00	42	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B2	53.0	0.16	0.47	0.34	10.00	17	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B2	59.5	0.17	0.48	0.36	3.00	16	SP-SM	Yes	24%	0.69	24%	0.71	2.69%	2.69%	0.97
2-B2	67.0	0.12	0.46	0.26	12.00	11	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B2	75.5	0.12	0.45	0.27	5.00	12	ML	Yes	39%	0.87	39%	1.94	3.37%	1.50%	0.90
2-B2	79.8	0.16	0.47	0.34	3.50	15	SP-SC	Yes	26%	0.74	26%	0.93	2.83%	2.83%	1.19
											LDI	4.32	Settlement (in)		3.86

TDL = Too Dense to Liquefy based on blowcount criteria
Volumetric Strains based on Figure 103

Liquefaction Evaluation - Idriss and Boulanger (2008)

Borehole: 2-B3

Input

Water Table depth at time of Exploration	Water Table depth at time of Liquefaction	amax/g	Mw
11	11	0.63	7.33

						At time of Exploration		At time of Liquefaction	
Boring Designation	Depth [m]	Depth [ft]	Soil Type	N_m [Blows/ft]	FC	Total Stress [psf]	Effective Stress [psf]	Total Stress [psf]	Effective Stress [psf]
2-B3	4.75	9.5	CL	26	50	570	570	570	570
2-B3	11.5	4	CL	19	90	1380	1348.8	1380	1348.8
2-B3	20.5	14	CL	5	95	2460	1867.2	2460	1867.2
2-B3	28.25	1.5	CL	5	70	3390	2313.6	3390	2313.6
2-B3	31	4	SC-SM	5	34	3720	2472	3720	2472
2-B3	40.75	1.5	CL	7	80	4890	3033.6	4890	3033.6

N_m = Measured SPT Blow Count

Results

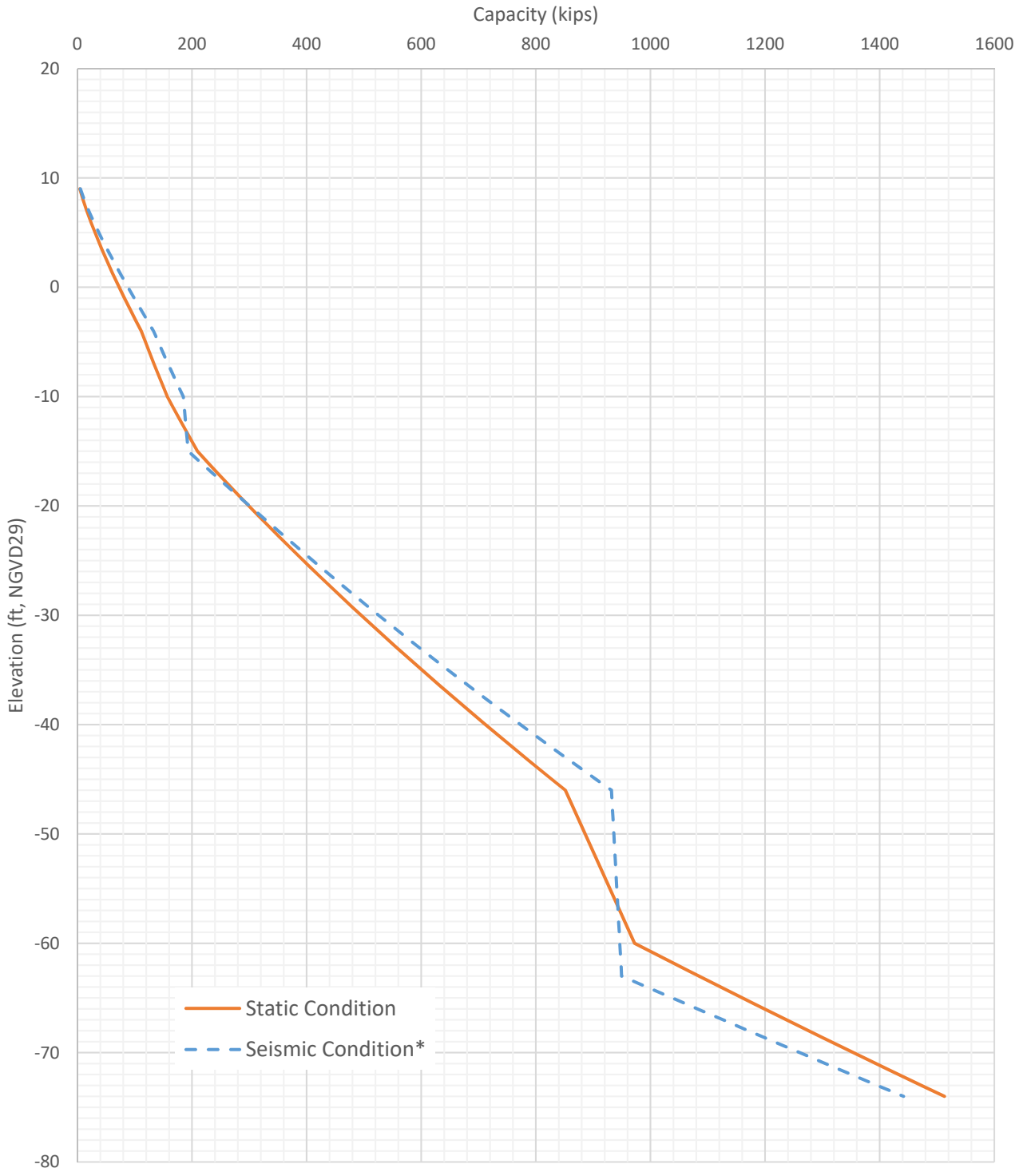
Boring Designation	Depth [ft]	CRR	CSR	FS	Ht. of Layer (ft)	(N1)60cs	Soil Type (USCS)	Liquefiable?	Limiting Shear Strain γ_{lim}	Parameter F_α	Maximum Shear Strain γ_{max}	ΔLDI	Calculated Volumetric Strain ε_v	Volumetric Strain Used ε_v	ΔS_i
2-B3	4.8	THC	0.35	THC	9.50	56	CL	Unsaturated	0%	0.00	0%	0.00	0.00%	0.00%	Unsaturated
2-B3	11.5	0.40	0.37	1.09	4.00	33	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B3	20.5	0.13	0.48	0.26	14.00	12	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B3	28.3	0.12	0.52	0.24	1.50	11	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
2-B3	31.0	0.12	0.53	0.23	4.00	11	SC-SM	Yes	41%	0.88	41%	1.62	3.45%	3.45%	1.66
2-B3	40.8	0.13	0.55	0.25	1.50	13	CL	No	0%	0.00	0%	0.00	0.00%	0.00%	0.00
											LDI	1.62	Settlement (in)		1.66

TDL = Too Dense to Liquefy based on blowcount criteria
Volumetric Strains based on Figure 103

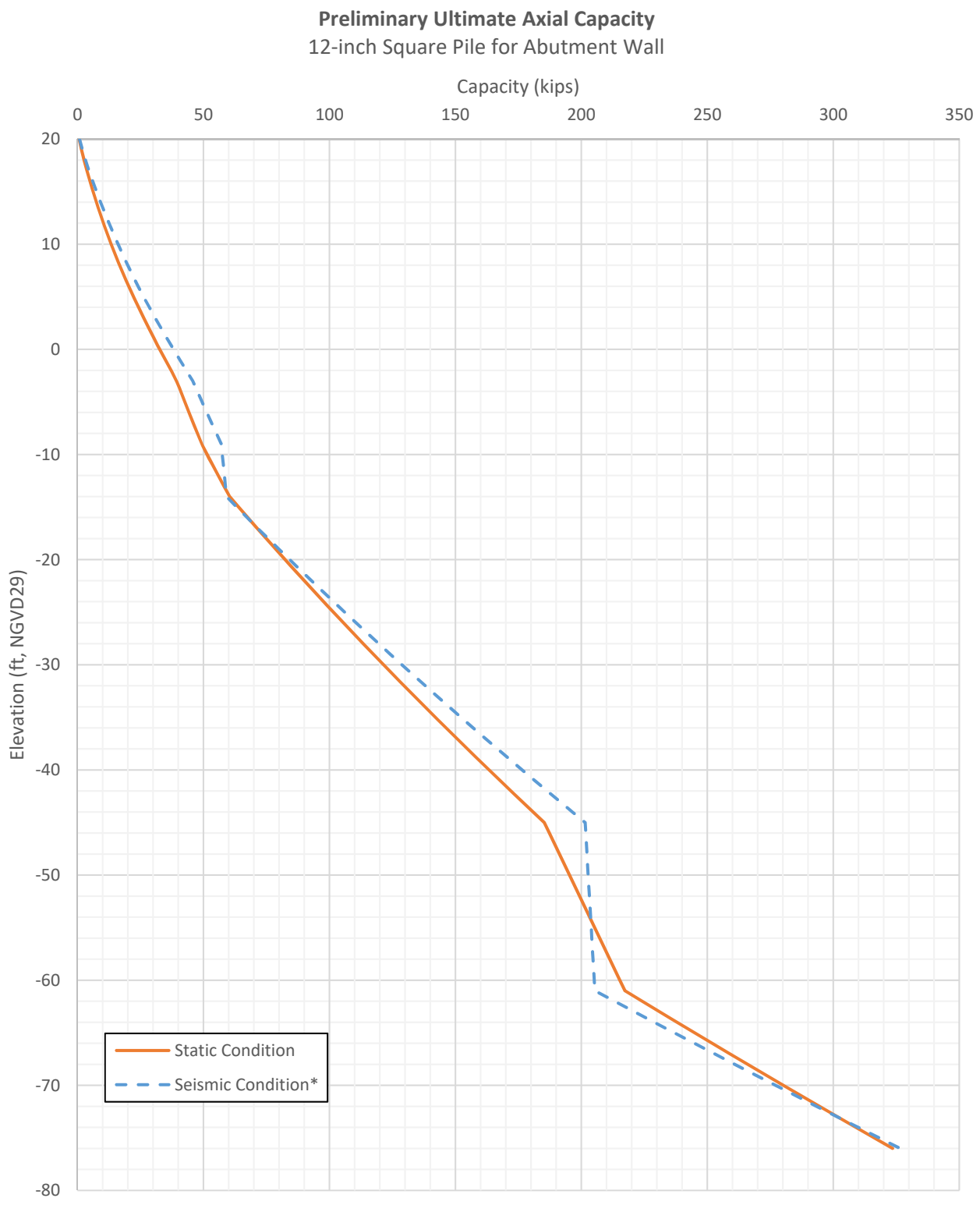
APPENDIX F

PRELIMINARY PILE CAPACITIES

Preliminary Ultimate Axial Capacity
66-inch Diameter CISS for Bridge Bent



*For seismic conditions, additional downdrag loads should be consider when evaluating the structual capacity of the pile. Drawdrag loads will be provided upon completion of design-level geotechnical studies.



*For seismic conditions, additional downdrag loads should be consider when evaluating the structual capacity of the pile. Drawdrag loads will be provided upon completion of design-level geotechnical studies.

ENGEO

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Project No.
12165.000.002

April 8, 2019

Mr. Tim Steele
The Sobrato Organization
10600 N. DeAnza Boulevard, Suite 200
Cupertino, CA 95014

Subject: Sobrato Property – Area 4
Newark, California

SURCHARGE RECOMMENDATIONS

Dear Mr. Steele:

As requested, we prepared this letter with surcharge recommendations for the proposed residential development within Area 4 of the Sobrato Property in Newark, California. The project site is roughly 450 acres, bounded by Union Pacific railroad tracks on the northeast, an Alameda County Flood Control (ACFC) canal on the northwest and southeast, and Mowry slough on the southwest. The site is currently undeveloped and is vacant.

PROJECT BACKGROUND

The Vesting Tentative Map (VTM) prepared by Carlson, Barbee & Gibson, Inc. (CBG) dated September 17, 2018, shows that approximately half of the project site will be developed with residential structures and the remaining areas will remain as seasonal wetlands. The residential area will provide approximately 469 single-family lots, as well as several parks, recreational trails, new streets and utilities. We divided the larger residential development into four development areas (Figure 1).

The current ground surface at the site ranges from Elevation 0 to 6 feet (NGVD29) and we understand ground surface elevations will be raised by 7 to 17 feet (Elevations 13 to 18 feet). The perimeter slopes of the project site are currently planned to be either at a 3:1 (horizontal:vertical) or 2:1 slope. Based on discussions with you and CBG, we understand the project will utilize a rolling surcharge program to mitigate settlement induced by consolidation of compressible soil.

SURCHARGE DESIGN CONSIDERATIONS

Our surcharge program is designed to achieve ½ inch or less of post-construction residual consolidation settlement. We completed our analysis using the software Settle3D. We assumed an equivalent building pressure of 0.5 thousand pounds per square foot (ksf), a civil and surcharge fill unit weight of 125 pounds per cubic foot (pcf), wick drain spacing of 3 and 5 feet, and a minimum surcharge duration of 6 months. Additional surcharge duration was considered for Development Area 4 (DA-4) because this area is the last area to be developed and allows a longer surcharge time. The limits of each development area are shown on Figure 1.

Consistent monitoring of settlement instruments should be conducted to evaluate that the desired settlement has occurred. When the desired settlement has occurred, ENGEO will provide written notification to remove the surcharge.

EXISTING SUBSURFACE CONDITIONS

Development Area 1 (DA-1)

We drilled Borings 2-B4, 2-B5 and 2-B15 in DA-1 and encountered approximately 5½ to 10 feet of compressible fat clay below the surficial lean clay. Towards the western edge of DA-1, the borings generally encountered fat clay deposits at 13 feet below existing ground surface. The fat clay ranges in thickness from 8 to 10 feet and is deposited over 5 to 6 feet of lean clay which are above layers of sand and gravel with varying amounts of clay to the terminal depth of the borings.

Moving west to east through DA-1, we consistently encountered the fat clay deposits 13 to 14 feet bgs. In Boring 2-B4, the fat clay deposits are underlain by poorly graded sand with varying amounts of gravel and silt to a depth of 44 feet. Below the poorly graded sand, we encountered very stiff lean clay extending to the terminal depth of 51½ feet. Past explorations that we performed, including Borings 1-B1 and 1-B2 and previous Conerstone Earth Group (CEG) explorations EB-1 and CPT3, encountered similar subsurface soil within DA-1.

Based on laboratory consolidation test results, the compressive fat clay deposits at DA-1 yield a compressive index of approximately 0.18 and an over-consolidation ratio of 1.5 or above.

We present an idealized subsurface profile for DA-1 for design purposes only; variations in the layering and thicknesses should be anticipated and these tables are for design purposes only and are not a basis for differing conditions during construction.

TABLE 1: Generalized Subsurface Profile for Development Area 1 (DA-1)

ELEVATION (NGVD29)	USCS SOIL TYPE	DESCRIPTION
Above -5	CL	Medium stiff to very stiff, Lean Clay
-5 to -15	CH	Soft to medium stiff, compressible, Fat Clay
-15 to -23	ML/SM	Stiff Silt, Silty Sand
-23 to -28	SP	Medium dense, Poorly graded sand
-28 to -40	GC	Dense Clayey Gravel
Below -40	CL [SE Portion]	Stiff to very stiff, Lean Clay
	SP/GP/SC [NW Portion]	Dense, Clayey gravelly Sand

Development Area 2 (DA-2)

We advanced Borings 2-B6 and 2-B7 within DA-2 and encountered approximately 5 to 10 feet of compressible fat clay below the surficial lean clay. The fat clay deposits thickens toward the south. Below the fat clay, Borings 2-B6 and 2-B7 encountered lean clay with varying amounts of sand and silt that extend to the terminal depths of the borings.

Based on laboratory consolidation test results, the compressive fat clay deposits at DA-2 yield a compressive index of approximately 0.16 and an over-consolidation ratio of 1.5 or above.

We present an idealized subsurface profile for DA-2 for design purposes only; variations in the layering and thicknesses should be anticipated and these tables are for design purposes only and are not a basis for differing conditions during construction.

TABLE 2: Generalized Subsurface Profile for Development Area 2 (DA-2)

ELEVATION (NGVD29)	USCS SOIL TYPE	DESCRIPTION
Above -10	CL	Medium stiff to very stiff, Lean Clay
-10 to -20	CH	Soft to medium stiff, compressible, Fat Clay
-20 to -25	CL	Medium stiff to very stiff, Lean Clay
-25 to -50	CL/SC	Medium dense clayey sand and stiff Lean Clay

Development Area 3 (DA-3)

We advanced Borings 2-B8, 2-B10, 2-B11, as well as CPTs 2-CPT5 and 2-CPT7 within Development Area 3 (DA-3) and encountered surficial lean clay extending to 5 to 9 feet bgs underlain by fat clay. The fat clay varied in thickness from approximately 5 to 9 feet and appears to increase in thickness from east to west across DA-3. The fat clay is deposited over elastic silt and silty clay, which ranges in thickness from 4 to 10 feet. Following the elastic silt and silty clay, the borings generally encountered poorly graded sand layers with varying amounts of silt and clay intermixed with layers of lean clay. We terminated all of our borings in lean clay. The conditions encountered during drilling are consistent with the soil profiles presented in the CPT soundings.

Past explorations that we performed in DA-3 encountered fat clay below 12 to 14 feet of surficial lean clay. The fat clay ranges in thickness from 10 to 13 feet and is above a thin layer of silt in Boring 1-B3 and a thick layer of lean clay, extending from 17 to 40 feet bgs, in Boring 1-B4. Previous CEG explorations EB-4, EB-5, EB-6, CPT2, CPT8, and CPT9 also encountered a similar subsurface profiles within DA-3.

Based on laboratory consolidation test results, the compressive fat clay deposits at DA-3 yield a compressive index of approximately 0.14 and an over consolidation ratio of 1.5 or above.

We present an idealized subsurface profile for DA-3 for design purposes only; variations in the layering and thicknesses should be anticipated and these tables are for design purposes only and are not a basis for differing conditions during construction.

TABLE 3: Generalized Subsurface Profile for Development Area 3 (DA-3)

ELEVATION (NGVD29)	USCS SOIL TYPE	DESCRIPTION
Above -6	CL	Medium stiff to very stiff, Lean Clay
-6 to -13	CH	Medium stiff, compressible, Fat Clay
-13 to -20	CL/CL-ML	Stiff Lean Clay and Silty Clay
-20 to -38	SP/SC/SM	Medium dense, Clayey and Silty poorly-graded Sand
-38 to -63	CL	Very stiff, Lean Clay
-63 to -78	SC	Very dense, Clayey Sand

Development Area 4 (DA-4)

We advanced explorations within DA-4 including Borings 2-B12, 2-B13 and CPTs 2-CPT9, 2-CPT10 and 2-CPT11. Near the northern edge of DA-4, 2-B12 encountered compressible fat clay deposits below surficial lean clay at a depth of approximately 2 feet. The compressible fat clay deposits at this location are roughly 5 feet thick and deposited over moderately compressible lean clay and fat clay deposits. The moderately compressible lean clay and fat clay deposits extend to a depth of approximately 18 feet bgs followed by deposits of silt with varying sand content to a depth of roughly 42 feet.

Based on laboratory consolidation test results, the upper compressive fat clay deposits at DA-4 yield a compressive index of approximately 0.26 and is normally consolidated. The lower moderately compressive lean clay and fat clay deposits yield a compressive index of approximately 0.15 with an over consolidation ratio of 2 or above.

The CPT soundings performed in DA-4 encountered similar subsurface profiles as the borings. We present an idealized subsurface profile for DA-4 for design purposes only; variations in the layering and thicknesses should be anticipated and these tables are for design purposes only and are not a basis for differing conditions during construction.

TABLE 4: Generalized Subsurface Profile for Development Area 4

ELEVATION (NGVD29)	USCS SOIL TYPE	DESCRIPTION
0 to -2	CL	Medium stiff to hard, Lean Clay
-2 to -7	CH	Soft to medium stiff, compressible, Fat Clay
-7 to -18	CL/CH	Medium stiff to stiff, moderately compressible, Lean Clay and Fat Clay
-18 to -33	SM	Medium dense Silty Sand
-33 to -40	CL [SE Portion]	Stiff to very stiff, Lean Clay
	SP [NW Portion]	Very dense, Poorly graded Sand

SURCHARGE RECOMMENDATIONS

Development Area 1

Approximately 8 to 11 feet of civil fill is planned within DA-1. We developed the recommended surcharge heights and wick drains spacing in Table 5 to achieve ½ inch or less residual consolidation settlement at the end of the surcharge period.

TABLE 5: Preliminary Surcharge and Wick Drain Dimensions for DA-1

HEIGHT OF CIVIL FILL (FEET)	TOTAL CONSOLIDATION SETTLEMENT WITHOUT SURCHARGE PROGRAM (INCHES)	SURCHARGE FILL HEIGHT* (FEET)	SURCHARGE INDUCED SETTLEMENT (INCHES)	TOTAL FILL HEIGHT, SURCHARGE + CIVIL (FEET)	WICK DRAIN SPACING (FEET ON CENTER)	SURCHARGE TIME (MONTHS)
8	2.5	4	2.0	12	5	6
10	3.0	3	2.5	13	3	6
12	3.5	3	3.0	15	3	6

*Surcharge fill is the height of fill to be placed above the civil fill elevation.

Development Area 2

Approximately 9 to 11 feet of civil fill is planned within DA-2. We developed the surcharge heights and wick drains spacing in Table 6 to achieve ½ inch or less residual consolidation settlement at the end of the surcharge period.

TABLE 6: Preliminary Surcharge and Wick Drain Dimensions for DA-2

HEIGHT OF CIVIL FILL (FEET)	TOTAL CONSOLIDATION SETTLEMENT WITHOUT SURCHARGE PROGRAM (INCHES)	SURCHARGE FILL HEIGHT* (FEET)	SURCHARGE INDUCED SETTLEMENT (INCHES)	TOTAL FILL HEIGHT, SURCHARGE + CIVIL (FEET)	WICK DRAIN SPACING (FEET ON CENTER)	SURCHARGE TIME (MONTHS)
8	2.0	4	1.5	12	5	6
10	2.3	2	1.8	12	3	6
12	2.5	3	2.0	15	3	6

*Surcharge fill is the height of fill to be placed above the civil fill elevation.

Development Area 3

Approximately 11 to 15 feet of civil fill is planned within DA-3. We developed the surcharge heights and wick drains spacing in Table 7 to achieve ½ inch or less residual consolidation settlement at the end of the surcharge period.

TABLE 7: Preliminary Surcharge and Wick Drain Dimensions for DA-3

HEIGHT OF CIVIL FILL (FEET)	TOTAL CONSOLIDATION SETTLEMENT WITHOUT SURCHARGE PROGRAM (INCHES)	SURCHARGE FILL HEIGHT* (FEET)	SURCHARGE INDUCED SETTLEMENT (INCHES)	TOTAL FILL HEIGHT, SURCHARGE + CIVIL (FEET)	WICK DRAIN SPACING (FEET ON CENTER)	SURCHARGE TIME (MONTHS)
10	2.0	2	1.7	12	5	6
12	2.2	2	1.9	14	5	6
14	2.5	2	2.1	16	5	6
16	2.6	2	2.3	18	5	6
18	2.8	2	2.5	20	5	6

*Surcharge fill is the height of fill to be placed above the civil fill elevation.

Development Area 4

Approximately 14 to 17 feet of civil fill is planned within DA-4. We developed the surcharge heights and wick drains spacing in Table 8 to achieve ½ inch or less residual consolidation settlement at the end of the surcharge period.

TABLE 8: Preliminary Surcharge and Wick Drain Dimensions for DA-4

HEIGHT OF CIVIL FILL (FEET)	TOTAL CONSOLIDATION SETTLEMENT WITHOUT SURCHARGE PROGRAM (INCHES)	SURCHARGE FILL HEIGHT* (FEET)	SURCHARGE INDUCED SETTLEMENT (INCHES)	TOTAL FILL HEIGHT, SURCHARGE + CIVIL (FEET)	WICK DRAIN SPACING (FEET ON CENTER)	SURCHARGE TIME (MONTHS)
12	9.0	4	8.5	16	5	18
14	9.5	4	9.0	18	5	18
16	10.0	4	9.5	20	5	18
18	11.0	4	10.5	20	5	18

*Surcharge fill is the height of fill to be placed above the civil fill elevation.

EARTHWORK RECOMMENDATIONS

Upon completion of environmental remediation, all development areas should be cleared of vegetation and obstructions, including existing abandoned concrete structures and buried utilities. Existing underground utilities at the site should be identified and either properly abandoned or relocated. If loose and deleterious materials are present, the materials should be removed and the area should be backfilled with engineered fill.

Wick Drain Installation

Wick drains should be installed vertically in a triangular grid pattern at the spacings indicated in the surcharge monitoring plans (Figures 2 through 5). All Wick drains should extend to sandy alluvial deposits, situated at a depth of approximately 30 feet.

Settlement Monitoring Instruments Installation

All settlement monitoring plates and surface markers should be placed at designated locations as shown on the surcharge monitoring plans (Figures 2 through 5). The settlement monitoring plates and surface settlement markers should be protected from construction equipment and be periodically surveyed by the Civil Engineer to determine elevations. The monitoring results should be provided to us within three days of collection. When the target settlement amount is achieved, we will notify the project team to remove the surcharge. We anticipate surcharge can be removed approximately six months after placement. As a minimum, settlement monitoring instruments should be surveyed every week during fill placement, every two weeks for the first two months after surcharge fill completion, and monthly thereafter. All settlement monument elevations should be tied to benchmarks established well beyond the zone of surcharge influence.

Civil Fill Placement

Upon installation of wick drains and settlement monuments, placement of civil fill to the planned finished grade elevation may commence. The civil fill should be compacted to 90 percent relative compaction and no less than 3 percentage points above optimum moisture content (per ASTM D-1557).

Surcharge Fill Placement

Additional surcharge fill will be placed over the civil fill to further accelerate settlement. Surcharge fill should be overbuilt a minimum of 5 feet beyond the proposed building pads and improvements.

The lower 12 inches of the surcharge fill should be compacted to at least 90 percent relative compaction. The remaining surcharge fill should be compacted to at least 85 percent relative compaction. The exact duration of the surcharge program for each area will depend on the results of the settlement monitoring, but we anticipate the duration to be approximately 6 months. To reduce the amount of export at the end of the surcharge program, the surcharge can be divided into phases and the surcharge fill can be moved from phase to phase. This approach will increase the total amount of time required for surcharging, so the phasing should consider infrastructure construction and potential neighborhood phasing.

Selection of Materials

Site soil is suitable for use as civil and surcharge fill. Import soil should have a plastic index (PI) less than 25, with more than 70 percent passing the No. 200 sieve, less than 5 percent larger than 4 inches, and be less corrosive than site soil. The target wet density of the import fill is 120 to 130 pounds per cubic foot (pcf) when compacted to 90 percent relative compaction. Soil placed as civil fill or surcharge fill should be free of construction debris (wood, brick, asphalt, concrete, metal, etc.), trees, and high organic content soil (soil that contains more than 3 percent organic content by weight).

Import fill should also be tested in accordance with California Department of Toxic Substance Control (DTSC) fill import guidelines, prior to acceptance. Imported fill materials should be approved by our staff prior to importing, so we should be provided at least 5 to 7 business days prior to site delivery to sample and test proposed imported fill materials.

Graded Slope

Graded slopes (civil fill) along the perimeter of the site are currently planned with an inclination of either 3:1 (horizontal:vertical) or 2:1. The temporary slope of the surcharge fill placed up to 5 feet in height over the civil fill can be constructed at an inclination of 2:1. For steeper surcharge fill gradients, we recommend the placement of geogrid extending at least 25 feet from the face of the slope. The contractor should wrap the geotextile reinforcing fabric tightly around the face of the wall using methods determined to be suitable for temporary retention. Sloughing along the face of the slope is anticipated, the use of form boards or other suitable methods as determined by the contractor may be implemented to minimize impacts to adjacent areas.

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

Sincerely,

ENGEO Incorporated

Jonas F. Bauer

Jeff Fippin, GE

REGISTERED PROFESSIONAL ENGINEER
JEFFREY A. FIPPIN
No. 2631
GEOTECHNICAL
STATE OF CALIFORNIA

Janet Kan, GE
jfb/jf/jk/jf

Attachments: Selected References
Figure 1 – Site Plan
Figure 2 – Surcharge Plan for DA-1
Figure 3 – Surcharge Plan for DA-2
Figure 4 – Surcharge Plan for DA-3
Figure 5 – Surcharge Plan for DA-4
Figure 6 – Wick Drain Layout and Standard Details

SELECTED REFERENCES

1. Carlson, Barbee & Gibson, Inc.; Vesting Tentative Map, Area 4, City of Newark, Alameda County, California; September 18, 2018.

2. Carlson, Barbee & Gibson, Inc.; Cut/Fill Exhibit, Sanctuary West Tract 8495, City of Newark, Alameda County, California; February 27, 2018.

3. ENGEO; Preliminary Geotechnical Exploration; Sobrato Property, Area 4, Newark, California; April 5, 2018; Project No. 12165.000.002.

4. ENGEO; DRAFT Preliminary Earthwork and Import Fill Recommendations, Sobrato Property, Area 4, Newark, California; October 19, 2018; Project No. 12165.000.002.

5. ENGEO; Preliminary Geotechnical Exploration; Stevenson Bridge Extension- East Abutment, Sobrato Property - Area 4, Newark, California; December 4, 2018; Project No. 12165.000.002

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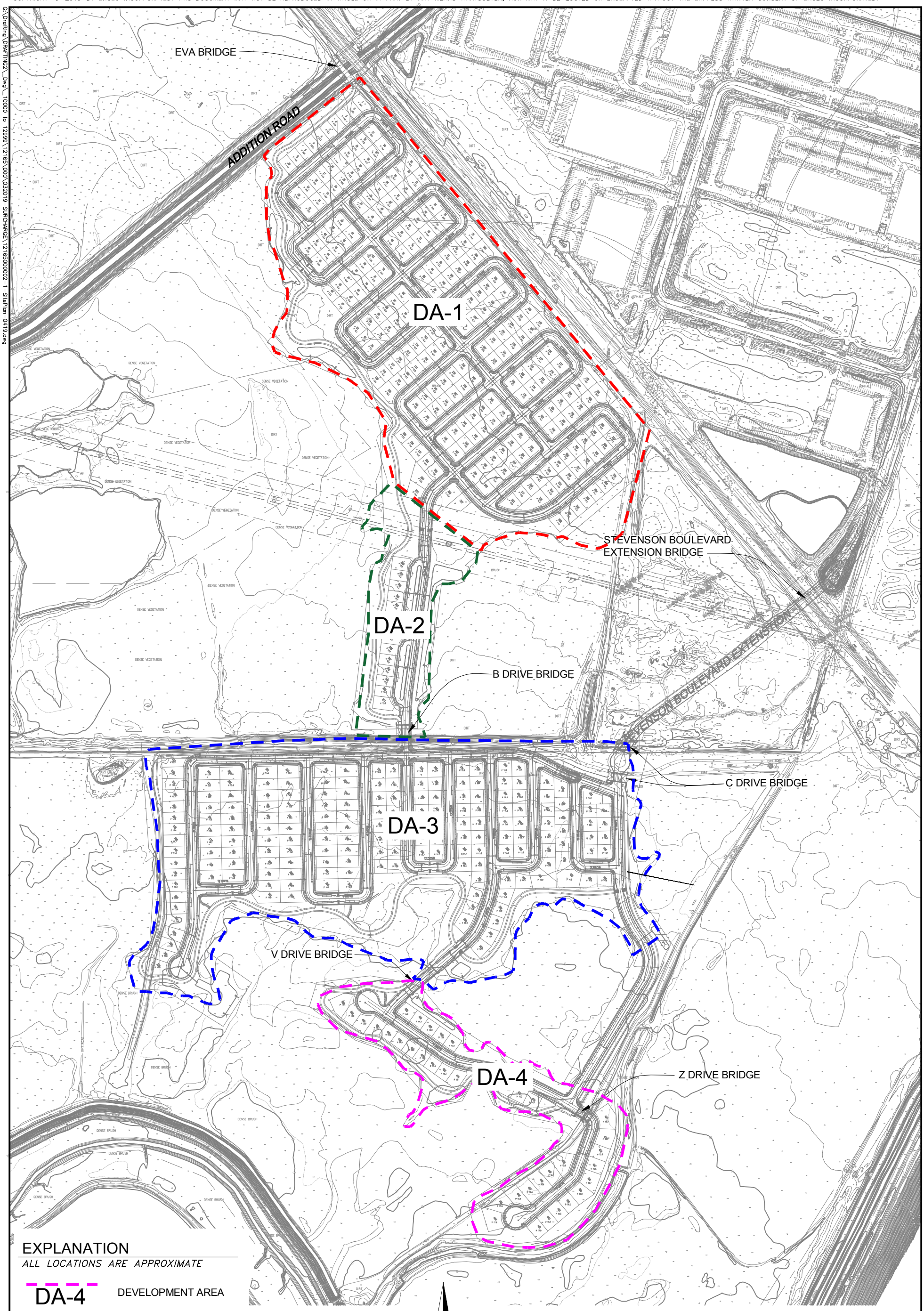

BASE MAP SOURCE: cbg

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SITE PLAN
SURCHARGE PLAN
NEWARK, CALIFORNIA

PROJECT NO.: 12165.000.002	FIGURE NO. 1
SCALE: AS SHOWN	
DRAWN BY: JV / CHECKED BY: JK	

ORIGINAL FIGURE PRINTED IN COLOR

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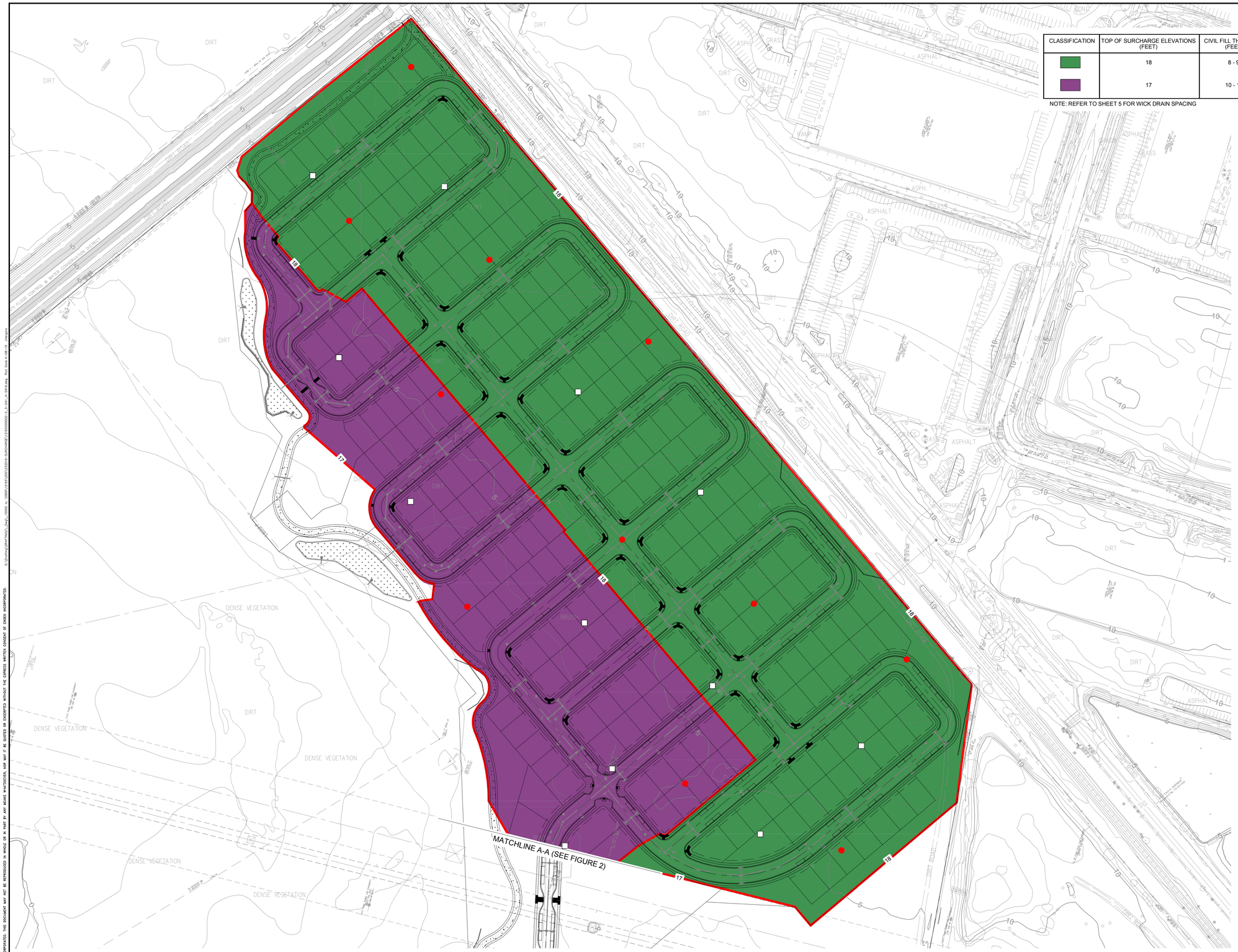


CLASSIFICATION	TOP OF SURCHARGE ELEVATIONS (FEET)	CIVIL FILL THICKNESS (FEET)	SURCHARGE FILL HEIGHT (FEET)	ESTIMATED SURCHARGE DURATION (MONTHS)
	18	8 - 9	3 - 4	6
	17	10 - 11	3 - 4	6

NOTE: REFER TO SHEET 5 FOR WICK DRAIN SPACING

EXPLANATION

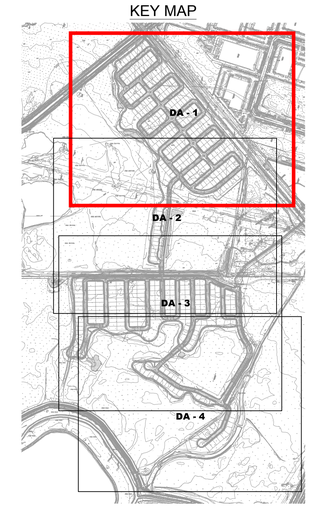
ALL LOCATIONS ARE APPROXIMATE

- 18 — TOP OF DESIGN SURCHARGE ELEVATION (FEET)
- SURFACE PLATES
- DEEP MONUMENT

SHEET INDEX

SHEET NUMBER	TITLE
1	DEVELOPMENT AREA 1
2	DEVELOPMENT AREA 2
3	DEVELOPMENT AREA 3
4	DEVELOPMENT AREA 4
5	DETAILS

KEY MAP



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CALIFORNIA - NEVADA - NEW ZEALAND - AUSTRALIA

DEVELOPMENT AREA 1

SURCHARGE PLAN

NEWARK, CALIFORNIA

DESIGNED BY: JFB
DRAWN BY: JV
CHECKED BY: JK
DATE: 03 MAR 19
SCALE: AS SHOWN

REV.	DATE	DESCRIPTION	BY
-			

SHEET NUMBER
2
OF 6 SHEETS

PROJECT NO.
12165.000.002

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CLASSIFICATION	TOP OF SURCHARGE ELEVATIONS (FEET)	CIVIL FILL THICKNESS (FEET)	SURCHARGE FILL HEIGHT (FEET)	ESTIMATED SURCHARGE DURATION (MONTHS)
	16	9 - 11	3 - 4	6
	15	10	2	6

NOTE: REFER TO SHEET 5 FOR WICK DRAIN SPACING

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

16 — TOP OF DESIGN SURCHARGE ELEVATION (FEET)

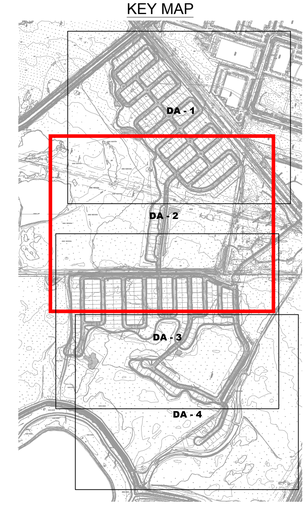
SURFACE PLATES

DEEP MONUMENT

SHEET INDEX

SHEET NUMBER	TITLE
1	DEVELOPMENT AREA 1
2	DEVELOPMENT AREA 2
3	DEVELOPMENT AREA 3
4	DEVELOPMENT AREA 4
5	DETAILS

KEY MAP



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DEVELOPMENT AREA 2

SURCHARGE PLAN

NEWARK, CALIFORNIA

DESIGNED BY: JFB
DRAWN BY: JV
CHECKED BY: JK
DATE: 03 MAR 19
SCALE: AS SHOWN

REV.	DATE	DESCRIPTION	BY

SHEET NUMBER 3 OF 6 SHEETS

PROJECT NO. 12165.000.002

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CLASSIFICATION	TOP OF SURCHARGE ELEVATIONS (FEET)	CIVIL FILL THICKNESS (FEET)	SURCHARGE FILL HEIGHT (FEET)	ESTIMATED SURCHARGE DURATION (MONTHS)
	17	13 - 15	2	6
	16	11 - 12	2	6

NOTE: REFER TO SHEET 5 FOR WICK DRAIN SPACING

EXPLANATION

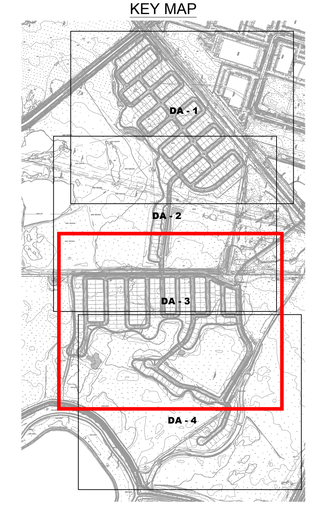
ALL LOCATIONS ARE APPROXIMATE

- 17 — TOP OF DESIGN SURCHARGE ELEVATION (FEET)
- SURFACE PLATES
- DEEP MONUMENT

SHEET INDEX

SHEET NUMBER	TITLE
1	DEVELOPMENT AREA 1
2	DEVELOPMENT AREA 2
3	DEVELOPMENT AREA 3
4	DEVELOPMENT AREA 4
5	DETAILS

KEY MAP



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DEVELOPMENT AREA 3

SURCHARGE PLAN

NEWARK, CALIFORNIA

DESIGNED BY: JFB
DRAWN BY: JV
CHECKED BY: JK
DATE: 03 MAR 19
SCALE: AS SHOWN

REV.	DATE	DESCRIPTION	BY

SHEET NUMBER
4
OF 6 SHEETS

PROJECT NO.
12165.000.002

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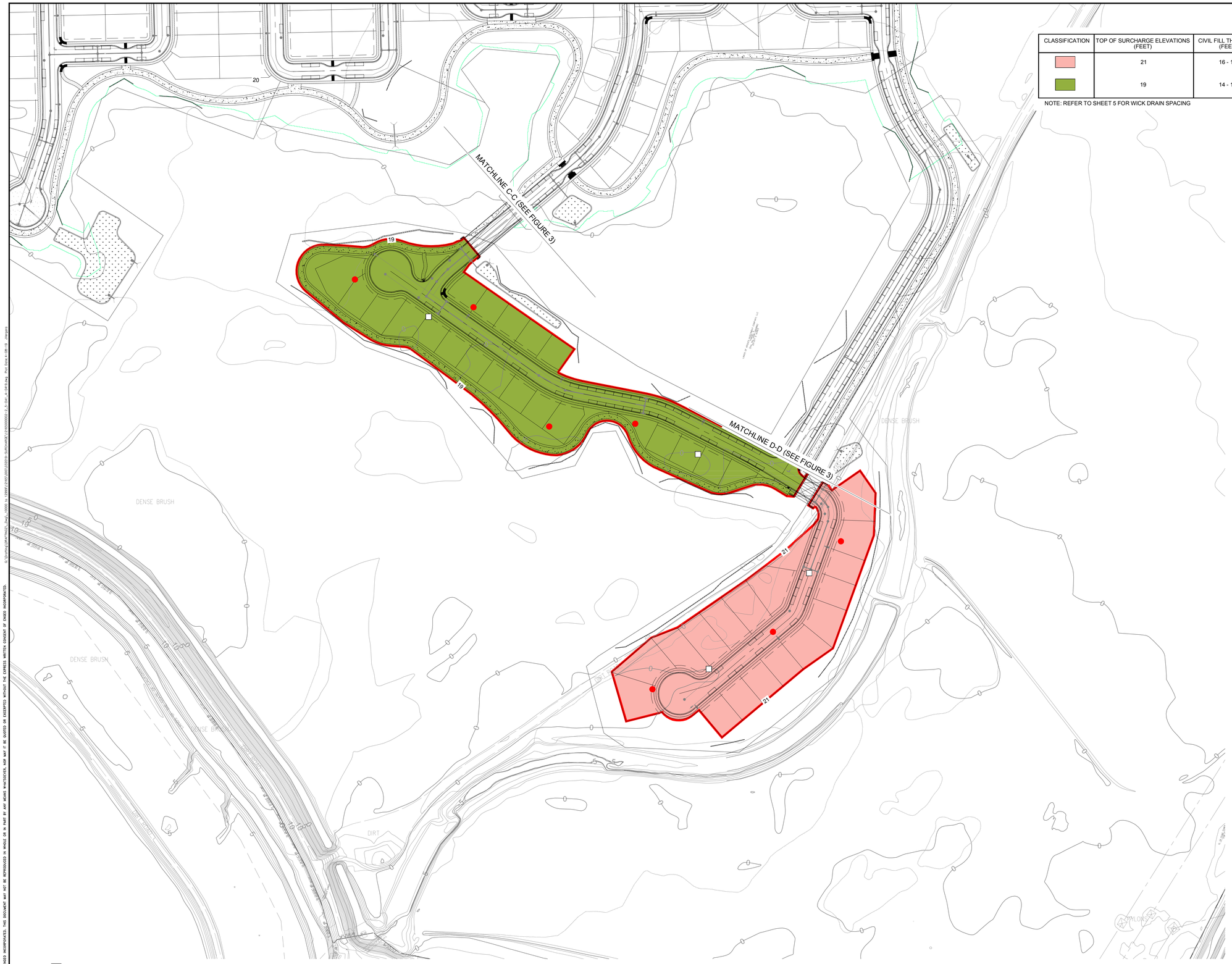
CLASSIFICATION	TOP OF SURCHARGE ELEVATIONS (FEET)	CIVIL FILL THICKNESS (FEET)	SURCHARGE FILL HEIGHT (FEET)	ESTIMATED SURCHARGE DURATION (MONTHS)
	21	16 - 17	4	12 - 18
	19	14 - 15	4	12 - 18

NOTE: REFER TO SHEET 5 FOR WICK DRAIN SPACING



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

21 — TOP OF DESIGN SURCHARGE ELEVATION (FEET)

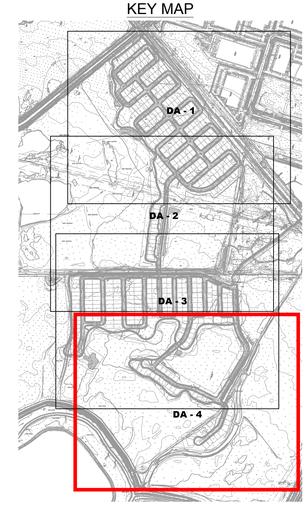
□ SURFACE PLATES

● DEEP MONUMENT

SHEET INDEX

SHEET NUMBER	TITLE
1	DEVELOPMENT AREA 1
2	DEVELOPMENT AREA 2
3	DEVELOPMENT AREA 3
4	DEVELOPMENT AREA 4
5	DETAILS

KEY MAP



N

−100 −50 0 100

SCALE IN FEET

USA NORTH 811

Call 811 Before You Dig.

2010 CROW CANYON PLACE
SUITE 250
SAN RAMON, CALIFORNIA 94583-4634
(925) 688-9000
(888) 279-2698

ENGEO

Expect Excellence

CALIFORNIA - NEVADA - NEW ZEALAND - AUSTRALIA

DEVELOPMENT AREA 4

SURCHARGE PLAN

NEWARK, CALIFORNIA

DESIGNED BY:	JFB
DRAWN BY:	JV
CHECKED BY:	JK
DATE:	03 MAR 19
SCALE:	AS SHOWN

REV.	DATE	DESCRIPTION	BY

SHEET NUMBER

5

OF 6 SHEETS

PROJECT NO.

12165.000.002

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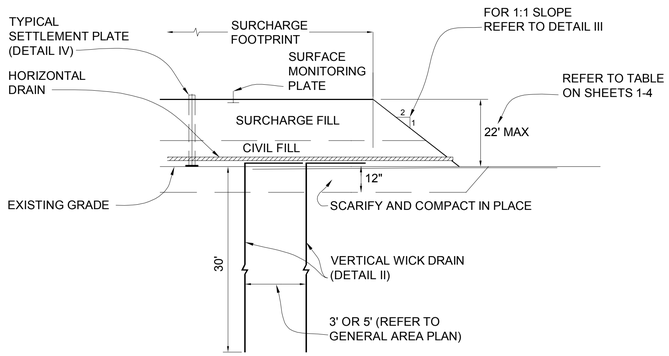
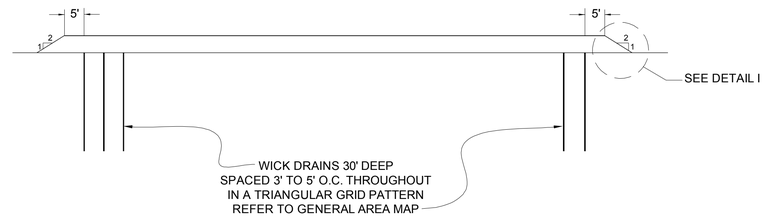


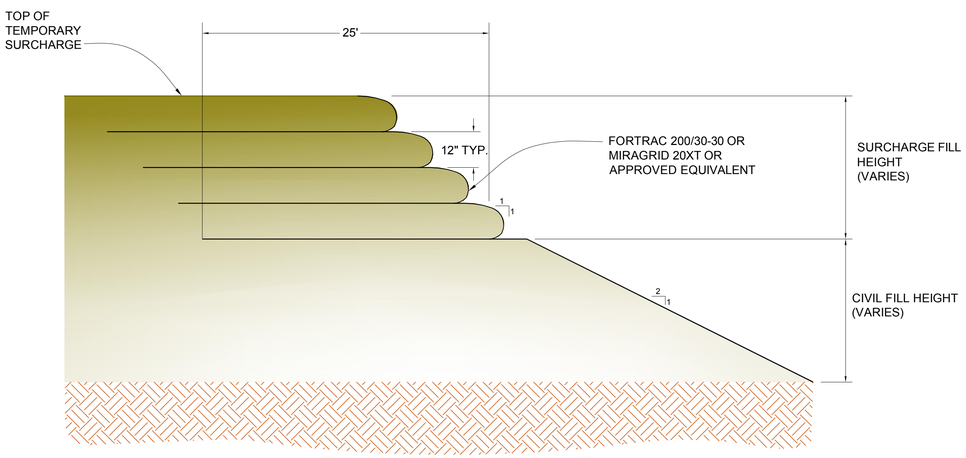
NOTE:
CIVIL FILL = 18 FEET MAX
SURCHARGE FILL = 4 FEET MAX

FILL PLACED SHOULD BE COMPACTED IN ACCORDANCE WITH PROJECT SPECIFICATIONS.

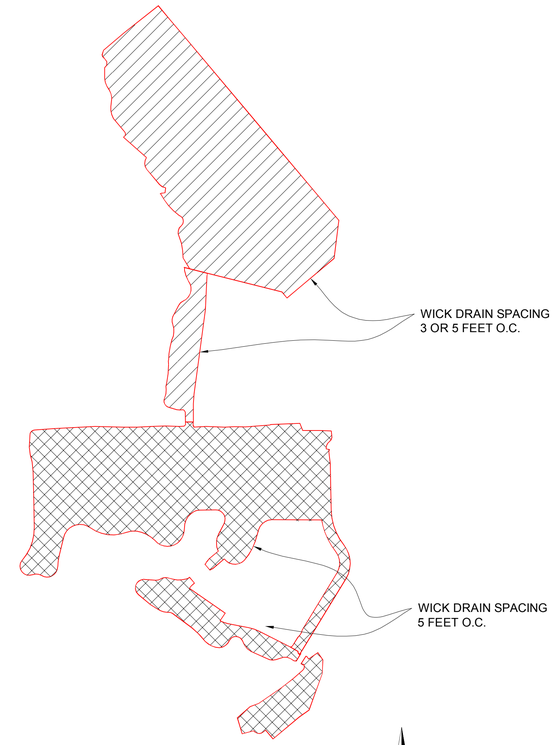
TYPICAL DETAIL I
NO SCALE



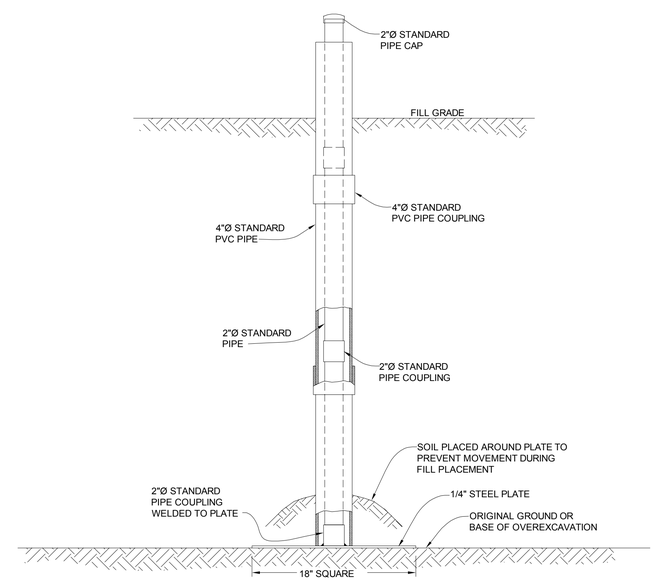
DETAIL II
NO SCALE



DETAIL III
NO SCALE



GENERAL AREA MAP
1" = 500'



DETAIL IV
NO SCALE

NOTES:

1. IT IS THE CONTRACTORS RESPONSIBILITY TO VERIFY THE LOCATIONS OF ALL EXISTING UNDERGROUND UTILITIES. CALL U.S.A. (UNDERGROUND SERVICE ALERT) 48 HOURS BEFORE DIGGING AT (800) 642-2444. LOCATIONS SHOWN ON THE PLANS WERE PROVIDED BY VTA AND ARE APPROXIMATE AND SHOWN FOR GENERAL INFORMATION ONLY, AND MAY BE INCOMPLETE. RELOCATION OR REPAIR OF ANY DAMAGE TO UTILITIES OR PIPELINES AND PLUGGING OR REMOVAL OF ABANDONED LINES SHALL BE THE CONTRACTORS RESPONSIBILITY.

2. THE CONTRACTOR SHALL TAKE ALL NECESSARY PRECAUTIONS TO PROTECT SETTLEMENT MONUMENT INSTALLED WITHIN THE TEST AREA.

3. THIS PROJECT SHALL BE CONSTRUCTED IN ACCORDANCE WITH PROJECT SPECIFICATIONS

4. SUGGESTED CONSTRUCTION SEQUENCE:
(REFER TO PROJECT SPECIFICATIONS FOR DETAILS)

A. CLEARING AND GRUBBING

B. STAKING OF SURCHARGE AREA

C. INSTALL SETTLEMENT PLATE

D. INSTALL WICK DRAINS AND HORIZONTAL DRAINS

E. PLACE AND COMPACT FILL

i) PLACE ONE 12 INCH THICK LIFT OF BASELINE FILL. COMPACT TO NO LESS THAN 90% RELATIVE COMPACTION AT 3% ABOVE OPTIMUM MOISTURE CONTENT

ii) OBTAIN ELEVATION AT ALL SETTLEMENT MONUMENT

iii) PLACE SURCHARGE FILL IN LIFTS LESS THAN 18 INCHES. COMPACT LOWER 12 INCHES TO AT LEAST 90% AND REMAINING SURCHARGE TO 85%

F. INSTALL SURFACE PLATES

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SUITE 250
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(925) 688-9000
(888) 279-2698
CALIFORNIA - NEVADA - NEW ZEALAND - AUSTRALIA

WICK DRAIN LAYOUT AND STANDARD DETAILS
SURCHARGE PLAN
NEWARK, CALIFORNIA

DESIGNED BY: JFB
DRAWN BY: JV
CHECKED BY: JK
DATE: 03 MAR 19
SCALE: AS SHOWN

REV.	DATE	DESCRIPTION	BY
-			

SHEET NUMBER
6
OF 6 SHEETS

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