ROCKRIDGE
GEOTECHNICAL

Prepared for **Eden Housing**

**FINAL GEOTECHNICAL INVESTIGATION**
**PROPOSED AFFORDABLE HOUSING DEVELOPMENT**
**RUBY & A STREETS**
**Castro Valley, California**

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February 6, 2019
Project No. 18-1444

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ROCKRIDGE GEOTECHNICAL

February 6, 2019
Project No. 18-1444

Ms. Ellen Morris
Project Developer
Eden Housing
22645 Grand Street
Hayward, CA 94541

Subject: Final Geotechnical Investigation Report
Proposed Affordable Housing Development
Ruby & A Streets
Castro Valley, California

Dear Ms. Morris:

We are pleased to present our final geotechnical investigation report for the proposed affordable housing development of the properties located at Ruby & A Streets in Castro Valley, California. Our geotechnical investigation was performed in accordance with our proposal dated July 24, 2018. We previously performed a preliminary geotechnical investigation at the site and presented our results in a report dated March 9, 2018.

The site encompasses an area of approximately 5.6 acres along the southwestern side of Ruby Street northwest of its intersection with A Street. The site, which has roughly 2.6 acres of buildable area, is a vacant lot bordered by San Lorenzo Creek to the southwest, a commercial building and A Street to the southeast, and single-family residences and Crescent Street to the north/northwest.

We understand Eden Housing is proposing to construct a four-story wood-framed residential building in the northwest portion of the site. The building will have approximately 70 units and will be constructed at-grade. Other proposed site improvements include at-grade parking lots, landscaping areas, and a creekside trail.

Based on the results of our engineering analyses using the data from our field investigation, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concern is the presence of loose/weak alluvial soils blanketing the site which are compressible under static and seismic loading.

Therefore, we conclude the proposed building should be supported on a shallow foundation system bearing on recompacted soil (engineered fill). Alternatively, the building may be supported on a shallow foundation system bearing on a ground improvement system. Soil improvement serves to stiffen the overall soil matrix by densifying loose soil layers and/or transferring the foundation loads to more competent material below the compressible layers, thus reducing settlements and providing increased bearing capacity. We estimate total settlement of a building supported on shallow foundations bearing on soil improved using a properly designed and implemented ground improvement system will be less than about 1 inch under static conditions and differential settlement will be less than 3/4 inch over a horizontal distance of 30 feet. In addition to the static settlement, we estimate the building will experience seismically induced total settlement of about 1 inch and differential settlement of 1/2 inch during a major earthquake.

The recommendations contained in our report are based on a limited subsurface exploration. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe grading and foundation installation curing which time we may make changes in our recommendations, if deemed necessary.

We appreciate the opportunity to provide our services to you on this project. If you have any questions, please call.

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC.

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Enclosure

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ROCKRIDGE GEOTECHNICAL

**FINAL GEOTECHNICAL INVESTIGATION**
**PROPOSED AFFORDABLE HOUSING DEVELOPMENT**
**RUBY & A STREETS**
**Castro Valley, California**

## 1.0 INTRODUCTION

This report presents the results of the final geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed affordable housing development to be constructed at the property on Ruby Street in Castro Valley, California. We previously performed a preliminary geotechnical investigation at the site and presented our results in a report dated March 9, 2018.

The site encompasses an area of approximately 5.6 acres along the southwestern side of Ruby Street northwest of its intersection with A Street, as shown on the Site Location Map, Figure 1. The site, which has roughly 2.6 acres of buildable area, is a vacant lot bordered by San Lorenzo Creek to the southwest, a commercial building and A Street to the southeast, and single-family residences and Crescent Street to the north/northwest.

Based on our review of preliminary plans[1], we understand the project consists of constructing a four-story wood-framed residential building in the northwest portion of the site, as shown on the attached Site Plan, Figure 2. The building will have approximately 70 units and will be constructed at-grade. Other proposed site improvements include at-grade parking lots, landscaping areas, and a creekside trail.

## 2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal dated July 24, 2018. The objective of our investigation was to evaluate subsurface conditions at the site and develop conclusions and recommendations regarding the geotechnical aspects of the proposed project. Our scope of work consisted of reviewing existing subsurface data available for the subject site and site vicinity, further evaluating subsurface conditions at the site by advancing

[1] *Ruby Street Entitlement v4.9* by Pyatok Architecture, dated November 19, 2018

five CPTs, drilling three exploratory borings, and performing engineering analyses to develop conclusions and recommendations regarding:

- site seismicity and seismic hazards, including the potential for cyclic densification, liquefaction and seismically induced lateral and vertical ground movements
- measures to mitigate seismic hazards, where appropriate
- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of static and seismically-induced foundation settlement
- subgrade preparation for pavements and exterior concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- flexible and rigid pavement design
- soil corrosivity
- 2016 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations

## 3.0 FIELD INVESTIGATION

During our preliminary investigation, we explored subsurface conditions at the site by advancing five cone penetration tests (CPTs) and advancing three hand-auger borings. For the current (final) subsurface investigation we advanced an additional five CPTs and drilled three borings. Prior to each investigation, we obtained a drilling permit from the Alameda County Public Works Agency (ACPWA), contacted Underground Service Alert (USA) to notify them of our work, and retained a private utility locator to reduce the potential for encountering an underground utility while performing our work. Additional details are presented in the following subsections.

### 3.1 Cone Penetration Tests

Middle Earth Geo Testing, Inc. of Orange, California advanced 5 CPTs on January 31, 2018 (CPT-1 through CPT-5) and 5 additional CPTs on December 20, 2018 (CPT-6 through CPT-10).

The CPTs were performed with a truck-mounted rig hydraulically pushing a 1.7-inch-diameter cone-tipped probe into the ground. The cone-tipped probe measured tip resistance, pore water pressure, and frictional resistance on a sleeve behind the cone tip. Electrical sensors within the cone continuously measured these parameters for the entire depth advanced, and the readings were digitized and recorded by a computer. Accumulated data were processed by computer to provide engineering information such as soil behavior types, estimated liquefaction resistance, and correlated strength characteristics of the soil encountered.

The approximate locations of the CPTs are shown on the Site Plan, Figure 2. The CPTs were advanced to depths between 30 and 50 feet below ground surface (bgs). The CPT logs, showing tip resistance, friction ratio, pore water pressure, and soil behavior type, are attached in Appendix A. Upon completion, the CPT holes were backfilled with cement grout in accordance with ACPWA requirements.

### 3.2 Exploratory Borings

Three exploratory borings were drilled on December 20, 2018 by Pitcher Services of East Palo Alto, California. The borings, designated B-1 through B-3, were each drilled to a depth of about 30 to 33 feet bgs using a track-mounted drill rig equipped with six-inch-diameter hollow-stem augers. During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The approximate locations of the exploratory borings are shown on the Site Plan, Figure 2. The logs of the borings are presented on Figures B-1 through B-3. The soil encountered in the borings was classified in accordance with the Classification Chart shown on Figure B-7.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter stainless steel tubes
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners
- Shelby Tube (ST) thin-walled stainless steel tube with an outside diameter of 3.0 inches.

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil, the SPT sampler was used to evaluate the relative density of granular soils, and the Shelby Tubes were used to collect relatively undisturbed samples of soft to medium stiff fine-grained samples for laboratory testing.

The S&H and SPT samplers were driven with a 140-pound, above-ground automatic safety hammer falling about 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every six inches and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.8 and 1.4, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was designed to accommodate liners, but liners were not used. The blow counts used for this conversion were the last two blow counts. The converted SPT N-values are presented on the boring logs. The Shelby Tubes were statically pushed into the ground using hydraulic pressure from the drill rig; the pressure required to advance the samplers are included on the boring logs.

Upon completion, the borings were backfilled with cement grout in accordance with ACPWA standards. Soil cuttings generated during drilling were spread onsite.

### 3.3 Hand Auger Borings

During our preliminary investigation, we explored the subsurface conditions by advancing three hand-auger borings, designated as HA-1 through HA-3 at the approximate locations shown on the Site Plan, Figure 2.

The hand-auger borings were advanced to obtain samples of the soil for visual classification and laboratory testing. The borings were each advanced using a three-inch-diameter hand auger to depths of about 5 to 10 feet bgs. The subsurface conditions encountered in the borings are presented on Figures B-4 through B-6 in Appendix B. The soil encountered is classified in accordance with the charts presented on Figure B-7.

### 3.4 Laboratory Testing

We re-examined each soil sample obtained from our borings to confirm the field classifications and selected representative samples for laboratory testing. Soil samples were tested to measure moisture content, dry density, Atterberg limits (plasticity index), grain size distribution, collapse potential, resistance (R-Value), compaction characteristics, and corrosivity. The results of the geotechnical laboratory tests are presented on the boring logs and in Appendix C.

## 4.0 SUBSURFACE CONDITIONS

The geologic units in the site vicinity, as shown on the regional geologic map prepared by Graymer et al. (2006) for the U.S. Geological Survey, are presented on Figure 3 (Regional Geologic Map). This map indicates the site is primarily underlain by Holocene alluvium (Qha). The results of our field investigation indicate the site is underlain by heterogeneous alluvium that extends to the maximum depth explored of 50 feet bgs.

The upper 10 to 15 feet of alluvium generally consist of material with between about 35 and 65 percent fines and is non-plastic to low-plasticity. This material spans multiple soil-type transition within the United Soil Classification System (USCS), and could classify as silty sand (SM), clayey sand (SC), clayey-silty sand (SC-SM), sandy silt (ML), sandy clay (CL), or sandy clayey silt (CL-ML), depending on the exact properties of the layer under consideration. The upper 5 to 10 feet of this material is loose to medium dense (where sandy) and soft to medium stiff (where clayey).

Below a depth of 15 to 20 feet, the alluvium generally consists of dense sand with occasional medium dense sand layers and interbedded stiff clay layers. The clay layers encountered were a maximum of about eight feet thick.

During our preliminary investigation (January 2018), the groundwater level was estimated to be at a depth of about 18 feet bgs based on a pore-pressure dissipation test performed in CPT-3.

During our final investigation (December 2018) we encountered groundwater at depths between of about 22 and 27 feet bgs. To better estimate the highest potential groundwater level at the site, we also reviewed information on the State of California Water Resources Control Board GeoTracker website (http://geotracker.waterboards.ca.gov/). The closest site with groundwater information on the GeoTracker website is a carwash at 1367 A Street, which is about 500 feet southwest of the subject property and at an elevation roughly 5 feet higher. Groundwater was measured at the 1367 A Street site intermittently between 2007 and 2016. The groundwater depths typically ranged from about 25 to 27 feet bgs. The depth to groundwater is expected to vary several feet seasonally, depending on the amount of rainfall. Groundwater maps prepared by the California Geological Survey (CGS) for the Hayward Quadrangle indicate historic high groundwater in the site vicinity is between 10 and 20 feet bgs. Based on the available groundwater data at the site, we conclude a high groundwater level of 15 feet bgs should be used for design.

## 5.0 SEISMIC CONSIDERATIONS

The San Francisco Bay Area is considered to be one of the more seismically active regions in the world. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

### 5.1 Regional Seismicity

The site is located in the Coast Ranges geomorphic province of California that is characterized by northwest-trending valleys and ridges. These topographic features are controlled by folds and faults that resulted from the collision of the Farallon plate and North American plate and subsequent strike-slip faulting along the San Andreas fault system. The San Andreas fault is more than 600 miles long from Point Arena in the north to the Gulf of California in the south. The Coast Ranges province is bounded on the east by the Great Valley and on the west by the Pacific Ocean.

The major active faults in the area are the Hayward, San Andreas, and Calaveras faults. These and other faults in the region are shown on Figure 4. The fault systems in the Bay Area consist of several major right-lateral strike-slip faults that define the boundary zone between the Pacific and the North American tectonic plates. Numerous damaging earthquakes have occurred along these fault systems in recorded time. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated mean characteristic moment magnitude[2] [Working Group on California Earthquake Probabilities (USGS 2008) and Cao et al. (2003)] are summarized in Table 1.

**TABLE 1**
**Regional Faults and Seismicity**

| Fault Segment | Approximate Distance from Site (km) | Direction from Site | Mean Characteristic Moment Magnitude |
|---|---|---|---|
| Total Hayward | ~0.9 | West | 7.0 |
| Total Hayward-Rodgers Creek | ~0.9 | West | 7.3 |
| Total Calaveras | 13 | East | 7.0 |
| Mount Diablo Thrust | 19 | Northeast | 6.7 |
| Green Valley Connected | 26 | Northeast | 6.8 |
| N. San Andreas - Peninsula | 30 | West | 7.2 |
| N. San Andreas (1906 event) | 30 | West | 8.0 |
| Greenville Connected | 30 | East | 7.0 |
| Monte Vista-Shannon | 31 | Southwest | 6.5 |
| San Gregorio Connected | 41 | West | 7.5 |
| Great Valley 5, Pittsburg Kirby Hills | 43 | Northeast | 6.7 |
| N. San Andreas - North Coast | 45 | West | 7.5 |
| Great Valley 7 | 49 | East | 6.9 |

2 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

In the past 200 years, four major earthquakes (i.e., Magnitude > 6) have been recorded on the San Andreas fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) Intensity Scale occurred east of Monterey Bay on the San Andreas fault (Toppozada and Borchardt, 1998). The estimated moment magnitude, $M_w$, for this earthquake is about 6.25. In 1838, an earthquake occurred on the Peninsula segment of the San Andreas fault. Severe shaking occurred with an MM of about VIII-IX, corresponding to an $M_w$ of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an $M_w$ of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of October 17, 1989 had an $M_w$ of 6.9 and occurred about 74 kilometers south of the site. On August 24, 2014 an earthquake with an estimated maximum intensity of VIII (severe) on the MM scale occurred on the West Napa fault. This earthquake was the largest earthquake event in the San Francisco Bay Area since the Loma Prieta Earthquake. The $M_w$ of the 2014 South Napa Earthquake was 6.0.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward fault. The estimated $M_w$ for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably an $M_w$ of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

The U.S. Geological Survey's 2014 Working Group on California Earthquake Probabilities has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Region during the next 30 years (starting from 2014) is 72 percent. The highest probabilities are assigned to the Hayward fault, Calaveras fault, and the northern segment of the San Andreas fault. These probabilities are 14.3, 7.4, and 6.4 percent, respectively.

### 5.2 Local Faulting

The site is mapped approximately 3,000 feet northeast of a "well-defined" trace of the Hayward fault and approximately 1,000 feet southwest of a "inferred trace" of the Chabot fault, as mapped in the *Quaternary Fault and Fold Database of the United States* prepared by the US Geological Survey (USGS, 2015). We are not aware of specific fault studies in the near vicinity of the subject site; therefore, some uncertainty exists regarding the accuracy of the inferred trace of the Chabot fault. There is no evidence of Holocene (last 11,000 years) movement on the Chabot fault (Phelps et al., 2008) and the fault is not considered "active" by the California Geological Survey (CGS).

### 5.3 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to violent shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[3] lateral spreading,[4] and cyclic densification[5]. We used the results of our field investigation to evaluate the potential of these phenomena occurring at the project site. The results of our analyses and evaluation are presented in the following sections.

#### 5.3.1 Ground Shaking

The seismicity of the site is governed by the activity of the Hayward fault, although ground shaking from future earthquakes on other faults will also be felt at the site. The intensity of earthquake ground motion at the site will primarily depend upon the characteristics of the generating fault, distance to the earthquake epicenter, and magnitude and duration of the earthquake. We judge that strong to violent ground shaking could occur at the site during a large earthquake on one of the nearby faults.

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3 Liquefaction is a phenomenon where loose, saturated, cohesionless soil experiences temporary reduction in strength during cyclic loading such as that produced by earthquakes.

4 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces.

5 Cyclic densification is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

### 5.3.2 Ground Surface Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act. Although an inferred trace of the Chabot fault runs within approximately 1,000 feet of the site, as discussed in Section 5.2, we are not aware of data that suggests the fault is “active”, and therefore, we conclude the likelihood of surface faulting and consequent secondary ground failure during the life of the proposed project is relatively low.

### 5.3.3 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

The site is located within a mapped zone of liquefaction potential as shown on the map titled *State of California Seismic Hazard Zones, Hayward Quadrangle, Official Map*, prepared by the California Geological Survey (CGS), dated July 2, 2003 (Figure 5).

Liquefaction susceptibility was assessed using the software CLiq v2.2.0.37 (GeoLogismiki, 2018). CLiq uses measured field CPT data and determines liquefaction potential, including post-earthquake settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). We performed a liquefaction triggering analysis using our CPT data in accordance with the methodology by Boulanger and Idriss (2014). Settlements from post-liquefaction reconsolidation were computed using the methodology by Zhang et al. (2002). Calculated settlements were then modified using the methodology proposed by Çetin et al. (2009) to account for the depth of the liquefiable layers.

Our analysis indicates there are potentially liquefiable soil layers underlying the site. These layers are primarily present between depths of about 15 and 25 feet bgs, with thinner potentially liquefiable layers present to a depth of about 40 feet bgs. A six-foot-thick potentially liquefiable soil layer was encountered in CPT-1 and CPT-2 at depths of about 18 and 15 feet, respectively. Based on the available subsurface information, this layer may be continuous beneath the northern portion of the site along the creek channel. The potentially liquefiable layers in the other CPTs range in thickness from a few inches to a maximum of about two feet. We estimate total “free-field” ground surface settlements associated with liquefaction (referred to as post-liquefaction reconsolidation) within these layers after a MCE event on a nearby fault may vary from about 1/2 to 1-1/2 inches across the site and 1/2 to 1 inch within the proposed building footprint. Liquefaction-induced differential settlements of up to 1/2 inch over a horizontal distance of 30 feet are anticipated. A summary of the estimated liquefaction-induced “free-field” ground surface settlements for each CPT are presented below in Table 2.

**TABLE 2**

**Summary of Estimated Liquefaction-Induced Free-Field Settlements**

| Investigation Location | Estimated Ground Settlement (inches) |
|---|---|
| CPT-1 | 1 |
| CPT-2 | 1-1/2 |
| CPT-3 | 3/4 |
| CPT-4* | 1/2 |
| CPT-5* | 1/2 |
| CPT-6* | 1 |
| CPT-7* | 1/2 |
| CPT-8* | 3/4 |
| CPT-9* | 1/2 |
| CPT-10* | 1/2 |

*denotes CPT in immediate vicinity of proposed building

Ishihara (1985) presented empirical relationship that provides criteria that can be used to evaluate whether liquefaction-induced ground failure, such as sand boils and loss of bearing, would be expected to occur under a given level of shaking for a liquefiable layer of given thickness overlain by a resistant, or protective, surficial layer. Our analysis indicate at the locations in the vicinity of the proposed building, (CPT-4 through CPT-10), the non-liquefiable soil overlying the potentially liquefiable soil layers is sufficiently thick and the uppermost potentially liquefiable layers are sufficiently thin such that the potential for surface manifestations of liquefaction are low. At the CPT-1 and CPT-2 locations, the relative thickness of the non-liquefiable and potentially liquefiable layers indicate the potential for surface manifestations is moderate.

Lateral Spreading

Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported downslope or in the direction of a free face by earthquake and gravitational forces. Because CPT-1 and CPT-2 encountered what may be a continuous, potentially liquefiable soil layer, and there is free-face (creek channel) immediately adjacent to the site, we conclude there is potential for localized occurrences of lateral spreading to occur in that portion of the site. A similar layer was not encountered in the other CPTs.

The amount of lateral ground movement is difficult to predict, as there are limited analytical tools available to predict displacements. Using the methodology proposed by Zhang et al. (2004) we estimate lateral spread displacements on the order of 1 to 3 feet may occur in the northwest portion of the site following a large earthquake on a nearby fault.

### 5.3.4 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The results of our investigation indicate the soil encountered above the groundwater table includes loose to medium dense silty sand which may be susceptible to cyclic densification. Existing methods to evaluate cyclic densification settlements cannot be directly implemented due to the large magnitude of anticipated MCE ground shaking; however, we estimate cyclic densification on the order of 4 inches may occur, with differential settlements of about 2 inches over a horizontal distance of 30 feet.

## 6.0 DISCUSSIONS AND CONCLUSIONS

Based on the results of our geotechnical investigation, we conclude the proposed project can be developed as planned. The primary geotechnical concern is the presence of loose/weak alluvial soils blanketing the site which are compressible under static and seismic loading. This and other geotechnical concerns are discussed in the following sections.

### 6.1 Foundations and Settlement

Building loads were not available at the time this report was prepared. Based on our experience with similar structures, we assumed an average (dead plus live) bearing pressure of about 400 pounds per square foot (psf) throughout most of the building footprint, with concentrated loads in certain locations.

We considered conventional spread footings for the wood-framed apartment building; however, considering the lack of large concentrated loads associated with this building construction type, and the multiple angle points and jogs typically used in the building layout, conventional footings are generally not an efficient foundation system from a constructability standpoint. We believe a mat foundation would be a more economical and better-performing foundation type for the proposed building.

The results of our investigation indicate the site is blanketed by 5 to 10 feet of loose to medium dense sand and/or soft to medium stiff clay. This material has low to moderate strength and is moderately to highly compressible under static loads (where clayey) and susceptible to cyclic densification (where sandy) during a major earthquake. If the proposed building is supported on a shallow foundation system bearing on native soil, we judge the anticipated differential settlement due to both static and seismic load conditions would exceed typical structural tolerances.

Therefore, we conclude the proposed building should be supported on a shallow foundation system bearing on recompacted soil (engineered fill). Alternatively, the building may be supported on a shallow foundation system bearing on a ground improvement system. These options are further discussed in the following subsections.

### 6.1.1 Overexcavation and Recompaction

We conclude the upper loose alluvial soils can be over-excavated and recompacted to provide uniform support for the proposed building. After site clearing is completed, the proposed building pad should be overexcavated down approximately 10 feet below the current grades (to Elevation 119 feet[6]) to allow for moisture-conditioning and recompaction of the loose alluvial deposits blanketing the site. The subgrade exposed at the bottom of the overexcavation should be scarified to a depth of at least eight inches, moisture-conditioned, and compacted. The base of the overexcavation should extend at least five feet beyond the perimeter of the proposed building, except where constrained by the property line. Recommendations for engineered fill quality and compaction are presented in Section 7.1.

Our settlement analyses indicate total settlement of a shallow foundation system bearing on engineered fill, designed using the allowable bearing pressures presented in Section 7.2 of this report, will be on the order of 3/4 inch and differential settlement will be less than 1/2 inch over a 30-foot horizontal distance. An additional 1 inch of total settlement and 1/2 inch of differential settlement over a horizontal distance of 30 feet may occur due to post-liquefaction reconsolidation during following a large earthquake. The structural engineer should confirm these settlement estimates are tolerable.

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[6]Based on *Topographic and Boundary Survey for Eden Housing Joint Facility*, Job No. 10004A10 prepared by Luk & Associates and dated March 30, 2010. No datum indicated

### 6.1.2 Ground Improvement

Shallow foundations supported on a ground improvement system improved ground would be an acceptable alternative to overexcavation, provided the soil improvement was extended to a depth that would reduce differential settlement of the structure under both static and seismic conditions to a tolerable amount. There are several types of ground improvement that may be utilized to mitigate the weak and compressible alluvial deposits underlying the site. We consider drill displacement sand-cement (DDSC) columns or compacted aggregate piers (CAPs) to be the most economical ground improvement methods for this project. DDSC columns and CAPs are typically installed under design-build contracts by specialty contractors.

The actual design allowable bearing pressures and estimated settlements should also be evaluated by the design-build ground improvement contractor, as they will be based on the diameter, depth, and spacing of the ground improvement elements. If soil improvement is to be considered, we recommend a preliminary design, including calculations of static and seismic settlement, be prepared by the ground improvement contractor and submitted for our review. For planning purposes, ground improvement elements can be assumed to extend 10 to 15 feet below foundation level.

Compacted Aggregate Piers

CAPs are typically constructed by drilling a 30-inch-diameter shaft and replacing the excavated soil with compacted aggregate. The aggregate generally consists of clean, open-graded crushed rock below the water table and Class 2 aggregate base above the water table. The aggregate is compacted in approximately 12-inch-thick lifts using a modified hydraulic hammer mounted on an excavator. Due to the potential for vibrations to cause settlement under existing improvements, CAPs should not be installed within 15 feet of existing structures or sensitive underground utilities.

CAPs develop vertical support through a combination of frictional resistance along the shaft of the pier and improvement of the surrounding soil matrix, allowing use of significantly larger bearing capacities than feasible in unimproved soil. CAPs can also be designed to resist transient uplift loads by installing steel rods in the center of the pier; the rods are attached to a flat steel plate at the base of the of the footings. Lateral loads are resisted through a combination of passive pressure on the face of the footings and friction along the base of the footings. The frictional resistance is larger for a CAP-supported footing than for a footing supported on unimproved ground because of the presence of the compacted aggregate. The required size, spacing, length, and strength of piers should be determined by the contractor, based on the desired level of improvement.

Drilled Displacement Sand Columns

DDSC columns are installed by advancing a continuous flight, hollow-stem auger that mostly displaces the soil and then pumping a sand-cement mixture into the hole under pressure as the auger is withdrawn. DDSC columns result in low vibration during installation and generate little to no drilling spoils for off-haul. DDSC columns are installed under design-build contracts by specialty contractors.

### 6.2 Temporary Slopes

We anticipate excavations up to about 10 feet deep could be required to construct the building foundation. Excavations that will be deeper than five feet and will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). We preliminarily recommend the soil at the site be classified as “Type C” (1.5H:1V). The contractor should be responsible for the construction and safety of temporary slopes.

### 6.3 Soil Corrosivity

Corrosivity analyses were performed by Project X Corrosion on a sample of native soil from Boring HA-1 at a depth of 3 feet bgs. The results of the tests are presented on Figure C-6.

Based on the results of the corrosivity analyses performed on the samples, we conclude the soil at this site is “mildly corrosive” with respect to resistivity. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron should be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection. The chloride ion concentrations are moderately corrosive to steel reinforcement in concrete structures below ground. The test results indicate that sulfate ion concentrations are sufficiently low to not pose a threat to buried concrete.

### 6.4 Construction Considerations

The soil to be excavated consists of sand, silt, and clay, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. The soil is expected to have a high silt content and will likely be highly susceptible to "pumping" (i.e., excessive deflection under wheel loads) when compacted at a high moisture content. If site grading is performed during the rainy season, the material will likely have to be dried or cement treated before compaction can be achieved. Heavy rubber-tired equipment could cause pumping of the material and, therefore, should be avoided if grading occurs during the rainy season.

There are existing buildings adjacent to the site. Because the adjacent structures may experience some settlement during construction of the proposed building, a crack survey should be performed on each adjacent property prior to the start of construction. Heavy vibratory equipment should not be used within 15 feet of adjacent buildings at the site due to the potential to cause settlement of adjacent buildings. Compaction of fill in these areas should be performed with a static roller or hand-operated equipment such as a jumping jack or large vibratory plate ("Turtle").

## 7.0 RECOMMENDATIONS

Recommendations regarding site grading, foundation support, and other geotechnical aspects of the project are presented in this section.

### 7.1 Site Preparation and Grading

Site demolition should include the removal of existing structures, pavements, underground utilities, and foundations. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside of the proposed building footprint and will not interfere with the proposed construction, they may be abandoned in-place if allowed by the utility company. Voids resulting from demolition activities should be properly backfilled with compacted fill following the recommendations provided later in this section and under the observation of our field engineer.

Fill should consist of on-site soil or imported soil (select fill) that is free of organic matter, contains no rocks or lumps larger than four inches in greatest dimension, has a liquid limit of less than 40 and a plasticity index lower than 12, and is approved by the Geotechnical Engineer. Samples of proposed imported fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed imported material.

In areas to receive fill or improvements the soil subgrade should be scarified to a depth of at least eight inches, moisture-conditioned, and compacted to at least 90 percent relative compaction[7]. Fill should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. Fill consisting of clean sand or gravel (defined as soil with less than 10 percent fines by weight) should be compacted to at least 95 percent relative compaction. Fill greater than five feet in thickness or placed within the upper foot of pavement soil subgrade should also be compacted to at least 95 percent relative compaction, and be non-yielding.

#### 7.1.1 Building Pad

If the building will be supported on engineered fill, the proposed building pad should be overexcavated to Elevation 119 feet (approximately 10 feet below existing grades). The

[7] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.

excavations should extend at least five feet beyond the perimeter of the proposed building, except where constrained by property lines.

### 7.1.2 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped.

Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted as according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the improvements above the fill.

Foundations for the proposed structure should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of the utility trenches running parallel to the foundation. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with controlled low-strength material (CLSM). If utility trenches are to be excavated below this zone-of-influence line after the construction of the building foundations, the trench walls need to be fully supported with shoring until CLSM is placed.

### 7.1.3 Exterior Flatwork Subgrade Preparation

Any areas to receive pavements and concrete flatwork, including sidewalks, should be overexcavated at least eight inches, moisture-conditioned, and compacted. The excavations should extend at least two feet beyond the edges of pavement and flatwork, except where constrained by property lines or existing utilities.

We recommend a minimum of four inches of Class 2 aggregate base be placed below exterior concrete flatwork, including sidewalks; the aggregate base should extend at least six inches beyond the slab edges where adjacent to landscaping. Class 2 aggregate base beneath exterior slabs-on-grade should be compacted in accordance with the recommendations presented above in Section 7.1.

## 7.2 Foundations

Recommendations for design and construction of foundations for the proposed building are presented in the following sections. We conclude the most appropriate foundation type for the proposed wood-framed residential structure is a mat foundation bearing on engineered fill or ground improvement.

Recommendations for design and construction of foundations for the proposed development are presented in the following section. Based on the settlement evaluation discussed in Section 6.1 and our experience with similar projects, we conclude the most appropriate foundation system consists of a mat foundation bearing on engineered fill or a ground improvement system to reduce settlements to tolerable levels. We recommend the mat perimeter be deepened to bottomed at least one foot below the outside adjacent grade.

### 7.2.1 Mat Foundation on Engineered Fill

If the building will be supported on engineered fill, the mat foundation may be designed using allowable bearing pressures of 4,000 pounds per square foot (psf) for dead-plus-live loads and 5,300 psf for total design loads, which include wind or seismic forces. The allowable bearing pressures include factors of safety of at least 2.0 and 1.5 for static and transient loading conditions, respectively. We anticipate the average bearing pressure across the building footprint will be significantly lower.

For structural design of the mat foundation, we recommend using a coefficient of vertical subgrade reaction (dead-plus-live-load conditions) of 40 pounds per cubic inch (pci). This value was estimated considering 3/4 inch of static settlement at the most highly loaded locations (assumed to be 4,000 psf). We should review the structural engineer's computed bearing pressure and settlement profiles to refine these values, as needed.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the foundations and friction between the bottoms of the mat and the supporting soil. To compute passive resistance, we recommend using an equivalent fluid weight of 280 pounds per cubic foot (pcf). The upper foot of soil should be ignored unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30 where the mat foundation is in contact with soil. Where a vapor retarder is placed beneath the mat foundation, a base friction coefficient of 0.20 should be used. These values include a factor of safety of at least 1.5 and may be used in combination without reduction.

The subgrade should be free of standing water, debris, and disturbed materials and be checked by the project geotechnical engineer prior to placing the mud slab or reinforcing steel if no mud slab will be used.

### 7.2.2 Foundations on Ground Improvement

Viable ground improvement systems include drilled displacement sand-cement (DDSCs) columns or compacted aggregate piers (CAPs), which are typically installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of the ground improvement elements should be determined by the design-build contractor based on the proposed structural loads and the desired level of improvement (tolerable settlement and/or desired bearing capacity). The allowable bearing pressures and subgrade moduli under static and seismic conditions should be provided to the structural engineer by the design-build contractor and reviewed by the geotechnical engineer.

For preliminary design purposes, we recommend assuming ground improvement elements will extend about 10 to 15 feet below foundation level and be spaced at seven feet center-to-center. We anticipate the ground improvement systems described later in this section, if properly designed, should be capable of increasing the average allowable bearing pressures to approximately 4,000 psf for dead-plus-live-loads and limiting static differential settlement to less than 1/2 inch and seismically induced differential settlement to less than 1/2 inch. The actual design allowable bearing pressures and estimated settlements should be evaluated by the design-build ground improvement contractor, as they will be based on the diameter, depth, and spacing of the ground improvement elements. Our geotechnical report should be provided to potential design-build ground improvement contractors and we should be retained to provide technical input and review the geotechnical aspects of their final design prior to construction.

We recommend the ground improvement design be verified in the field by performing at least one full-scale load test in compression and one load test in tension (if applicable). Details regarding the proposed load testing program should be included in the design-build submittal for our review prior to mobilization to the site. The load tests should be performed on pre-production elements, under our observation, constructed using the same equipment, means-and-methods, area replacement ratio, and grout factor proposed for the production elements. The results of the load testing program should be evaluated by the design-build contractor's engineer, as well as our engineer, to confirm the columns provide adequate factor of safety with respect to axial failure and allowable axial deflection at the design load prior to commencing with production installation.

If DDSC columns are used, we recommend the interface between the DDSC elements and bottoms of foundations be separated by a 12-inch-thick compacted aggregate cushion, consisting of Class 2 aggregate base (AB) or crushed rock. The purpose of the aggregate cushion is to provide some degree of isolation between the two elements, which will help prevent excessive moments from being induced in the ground improvement columns during lateral loading, as the elements do not typically contain reinforcing steel to resist bending stresses.

To compute passive resistance, we recommend using an equivalent fluid weight of 280 pounds per cubic foot (pcf). The upper foot of soil should be ignored unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30 where foundations are in contact with soil. Where a vapor retarder is placed beneath the mat foundation, a base friction coefficient of 0.20 should be used. These values include a factor of safety of at least 1.5 and may be used in combination without reduction.

Foundation excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The geotechnical engineer should check foundation subgrade prior to placement of reinforcing steel to check for proper bearing and cleanout.

## 7.3 Vapor Barrier

To reduce water vapor transmission through the mat, we recommend installing a capillary moisture break and water vapor retarder beneath the mat. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. For the mat foundation option, the four inches of capillary break material may be eliminated provided the vapor retarder meets the requirements for Class A vapor retarders. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The particle size of the capillary break material and sand (if used) should meet the gradation requirements presented in Table 3.

**TABLE 3**
**Gradation Requirements for Capillary Moisture Break**

| Sieve Size | Percentage Passing Sieve |
|---|---|
| *Gravel or Crushed Rock* | |
| 1 inch | 90 – 100 |
| 3/4 inch | 30 – 100 |
| 1/2 inch | 5 – 25 |
| 3/8 inch | 0 – 6 |
| *Sand* | |
| No. 4 | 100 |
| No. 200 | 0 – 5 |

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If the concrete is poured directly over the vapor retarder, we recommend the w/c ratio of the concrete not exceed 0.45. In either case, water should not be added to the concrete mix in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

## 7.4 Pavement Design

### 7.4.1 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. The final soil subgrade in pavement areas will likely consist of a relatively low-plasticity soil with variable amounts of sand, silt, and clay. We obtained near-surface soil samples and performed laboratory tests to determine the R-value for pavement design. Laboratory test results indicate the soil tested has an R-value of 13. We used a reduced R-value of 10 for our pavement design to account for soil variability across the site.

If the proposed pavement will experience little or no truck traffic (including garbage trucks), we recommend a traffic index (TI) of 4.5 be used for asphalt concrete pavement design. Pavement areas that will be subject to garbage truck traffic should be designed for a TI of 5.5. The project civil engineer should check that the TI's presented in this report are appropriate for the intended use. Recommended pavement sections for these traffic indices are presented in Table 4.

**TABLE 4**
**Recommended AC Pavement Sections**

| TI | Asphaltic Concrete (inches) | Class 2 Aggregate Base R = 78 (inches) |
|---|---|---|
| 4.5 | 2.5 | 8.5 |
| 5.0 | 3.0 | 9.0 |
| 5.5 | 3.0 | 11.0 |

The upper eight inches of the subgrade should overexcavated, moisture-conditioned, and compacted in accordance with requirements presented in Section 7.1. The aggregate base should be moisture-conditioned to near optimum and compacted to at least 95 percent relative compaction to provide a firm and non-yielding surface.

To prevent irrigation water from entering the pavement section, curbs adjacent to landscaped areas should extend through the aggregate base and at least three inches into the underlying soil subgrade.

### 7.4.2 Rigid (Portland Cement Concrete) Pavement

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle load of 32,000 pounds and light truck traffic (i.e., a few trucks per week). The recommended rigid pavement section for these axle loads is 6.5 inches of Portland cement concrete over six inches of Class 2 aggregate base.

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt concrete pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. For areas that will receive weekly garbage truck traffic, we recommend the slab be reinforced with a minimum of No. 4 bars at 16-inch spacing in both directions. Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those we have described above for asphalt concrete pavement.

### 7.5 Seismic Design

We anticipate the proposed building will be designed using the seismic provisions in the 2016 California Building Code (CBC). Although the 2016 CBC calls for a Site Class F designation for sites underlain by potentially liquefiable soil, we conclude the thin, discontinuous liquefiable soil layers beneath the proposed building site will not incur significant nonlinear behavior during strong ground shaking. Therefore, for seismic design we recommend Site Class D be used.

The latitude and longitude of the site are 37.6804° and -122.0778°, respectively. Hence, in accordance with the 2016 CBC, we recommend the following seismic design parameters:

- $S_S$ = 2.453g, $S_1$ = 1.021g
- $S_{MS}$ = 2.453g, $S_{M1}$ = 1.531g
- $S_{DS}$ = 1.636g, $S_{D1}$ = 1.021g
- $PGA_M$ = 0.949g
- Seismic Design Category E for Risk Categories I, II, and III.

## 8.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, Rockridge Geotechnical should review the project plans and specifications to verify that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during subgrade preparation, ground improvement, installation of new foundations, and fill placement and compaction. These observations will allow us to compare actual with anticipated subsurface conditions and to verify that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

## 9.0 LIMITATIONS

This geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the exploratory borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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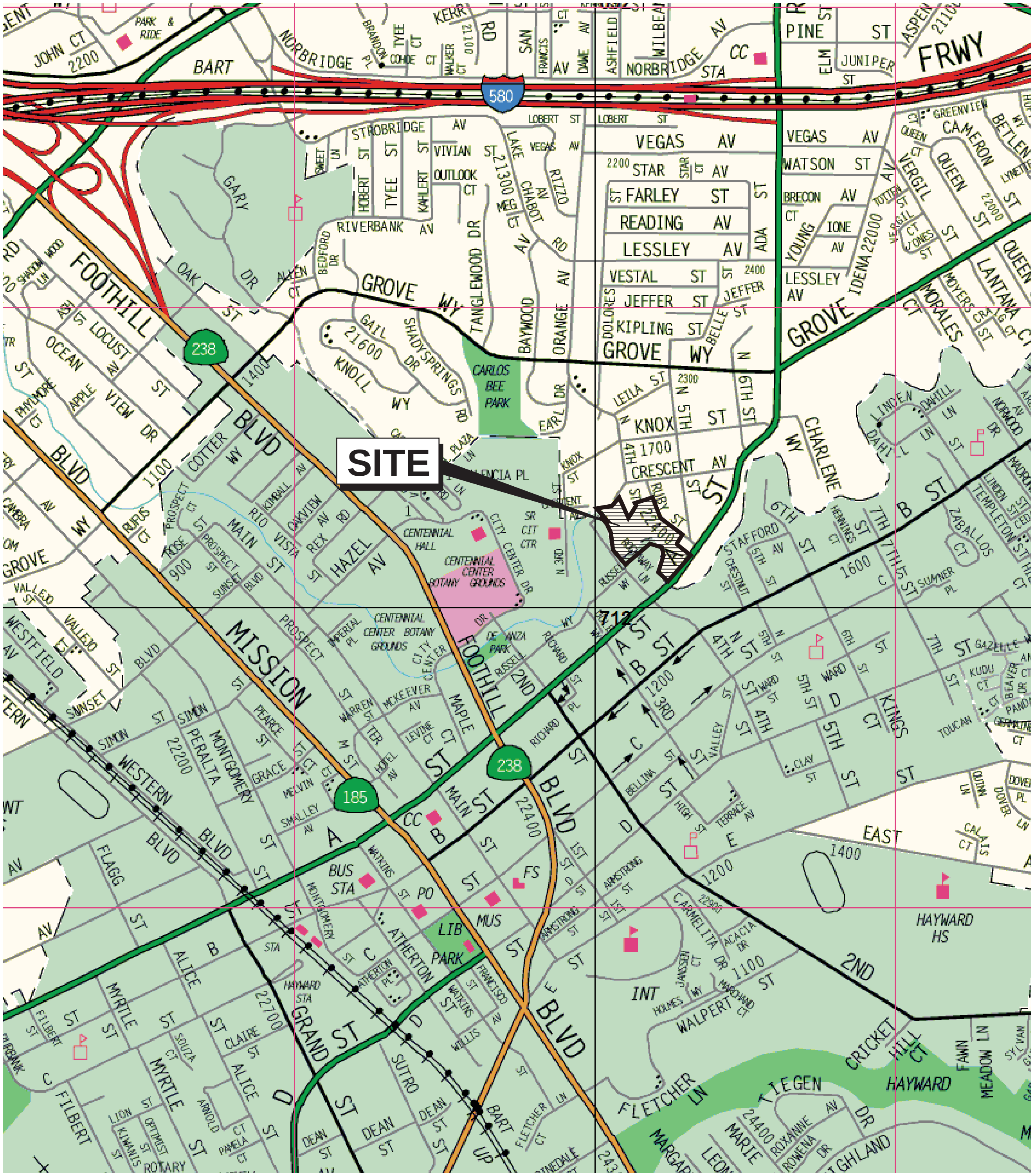
# FIGURES




Base map: The Thomas Guide
Alameda County
2002

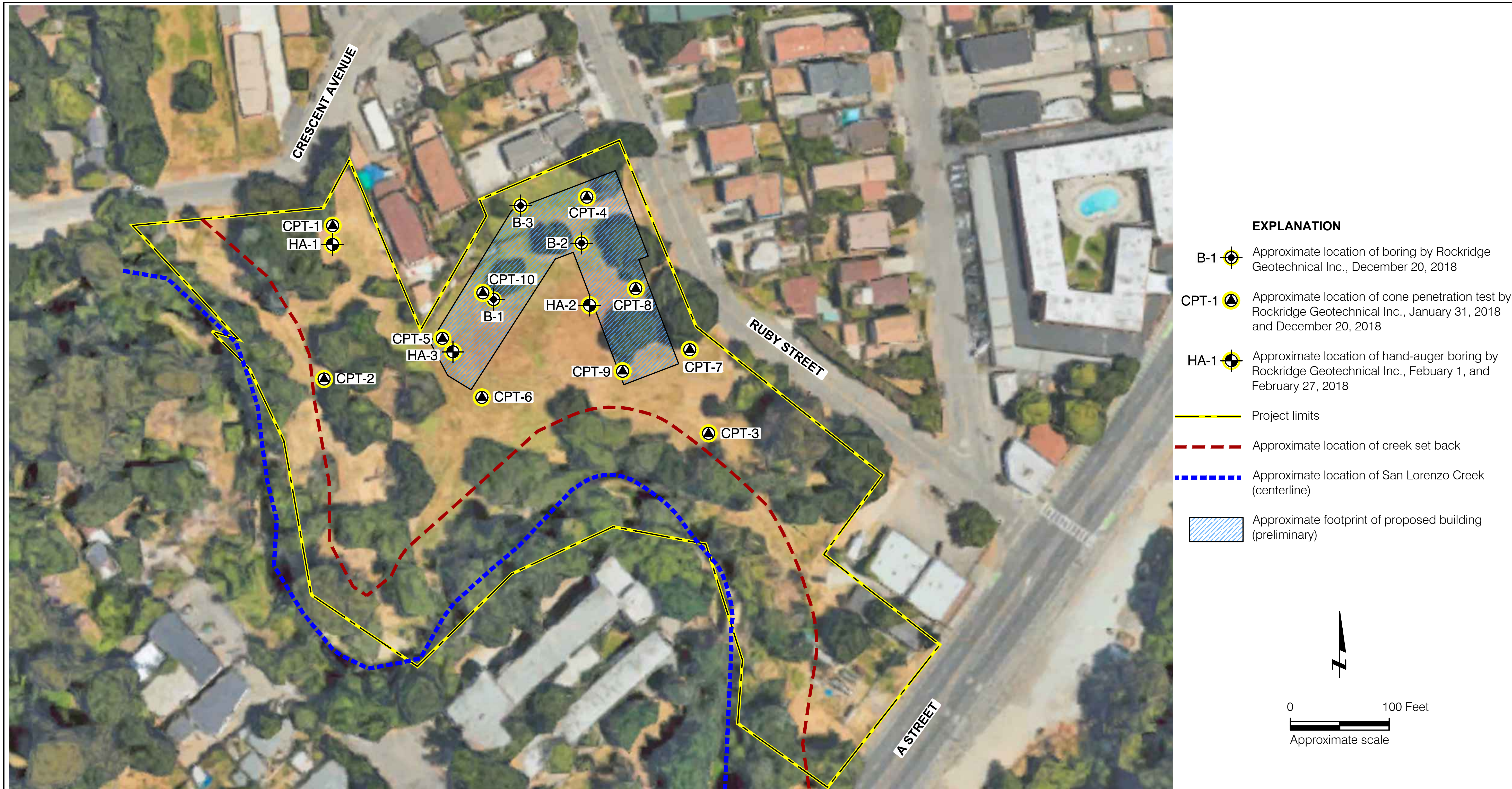
0 1/4 1/2 Mile
Approximate scale

**RUBY & A STREETS**
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

**SITE LOCATION MAP**

| Date 02/01/19 | Project No. 18-1444 | Figure 1 |
|---|---|---|



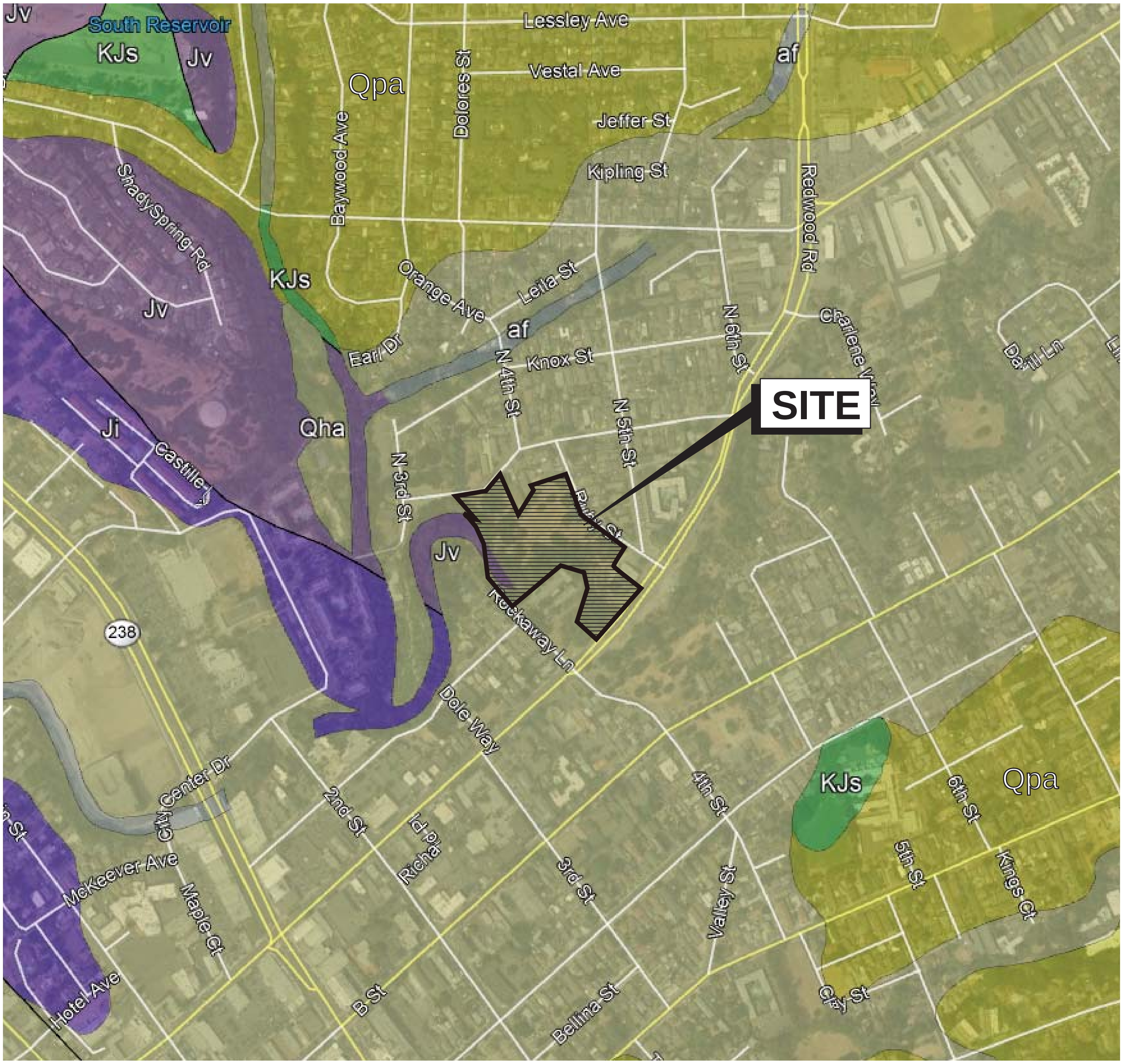

Base map: Google Earth, 2017.

**RUBY & A STREET**
Castro Valley, California

**SITE PLAN**

| Date 02/01/19 | Project No. 18-1444 | Figure 2 |
|---|---|---|

ROCKRIDGE GEOTECHNICAL




Base map: Goolge Earth with U.S. Geological Survey (USGS), Alameda County, 2017.

**EXPLANATION**

| | |
|---|---|
| af | Artificial Fill |
| Qha | Alluvium (Holocene) |
| Qpa | Alluvium (Pleistocene) |
| KJs | Great Valley complex sedimentary rocks (Early Cretaceous and (or) Late Jurassic) |
| Jv | Great Valley Complex volcanic rocks (Jurassic) |
| Ji | Great Valley Complex plutonic rocks (Jurassic) |

**Geologic contact:** dashed where approximate and dotted where concealed, queried where uncertin

0 750 1,500 Feet

Approximate Scale

**RUBY & A STREETS**
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

## REGIONAL GEOLOGIC MAP

Date 02/01/19 | Project No. 18-1444 | Figure 3


Base Map: U.S. Geological Survey (USGS), National Seismic Hazards Maps - Fault Sources, 2014.

**EXPLANATION**

Strike slip

Thrust (Reverse)

Normal

0 5 10 Miles

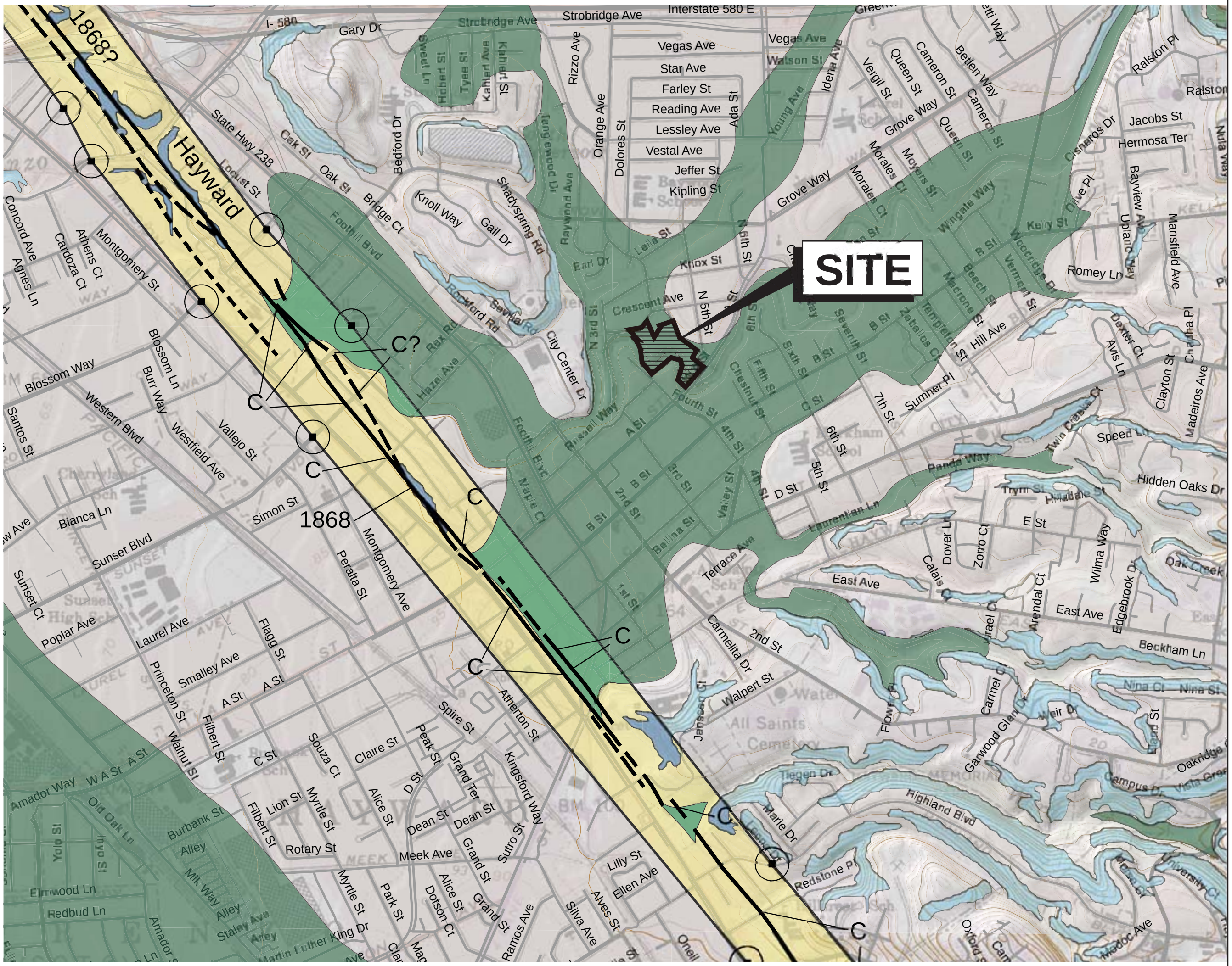
Approximate scale

**RUBY & A STREETS**
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

**REGIONAL FAULT MAP**

| Date 02/01/19 | Project No. 18-1444 | Figure 4 |
|---|---|---|




**EARTHQUAKE FAULT ZONES**

1906 C

**Active Fault Traces**
Faults considered to have been active during Holocene time and to have potential for surface rupture; solid line where accurately located, long dash where approximately located, short dash where inferred, dotted where concealed; query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

**ZONES OF REQUIRED INVESTIGATION**

**Earthquake Fault Zones**
Zones are areas delineated as straight-line segments that connect encircled turning points encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as defined in Public Resources Code Section 2621.5(a) would be required.

**SEISMIC HAZARD ZONES**

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

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

**OVERLAPPING ZONES**

**Overlap of Earthquake Fault Zone and Liquefaction Zone**
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone. Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

**Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone**
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone. Note: Mitigation methods differ for each zone – AP Act only allows avoidance; Seismic Hazard Mapping Act allows mitigation by engineering/geotechnical design as well as avoidance.

Reference:
State of California "California Geological Survey" Hayward Quadrangle, 2012.

0 2000 4000 Feet

Approximate scale

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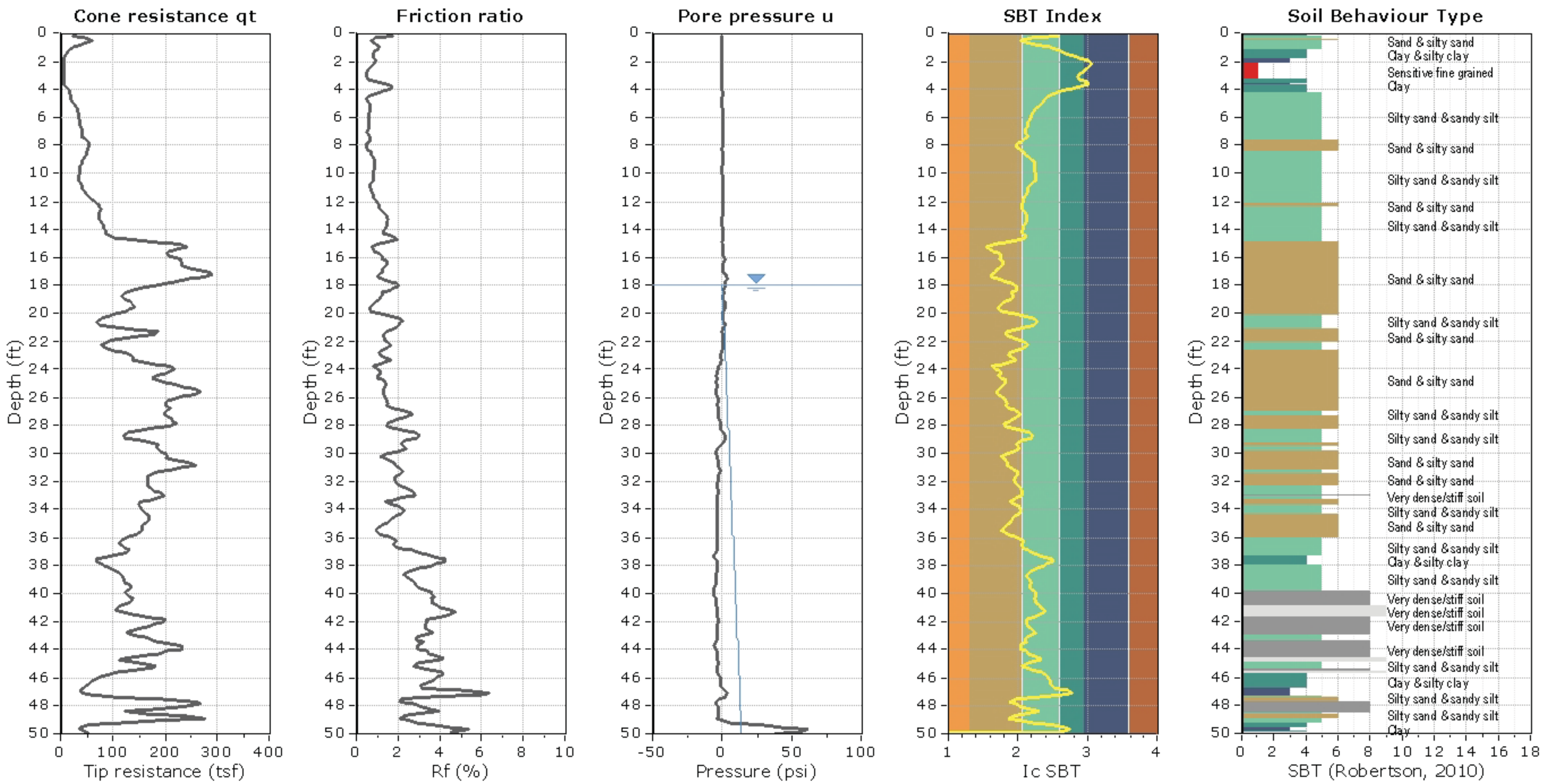
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

**SEISMIC HAZARDS ZONE MAP**

| Date 02/01/19 | Project No. 18-1444 | Figure 5 |
|---|---|---|

# APPENDIX A
## Cone Penetration Test Results




**SBT legend**

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

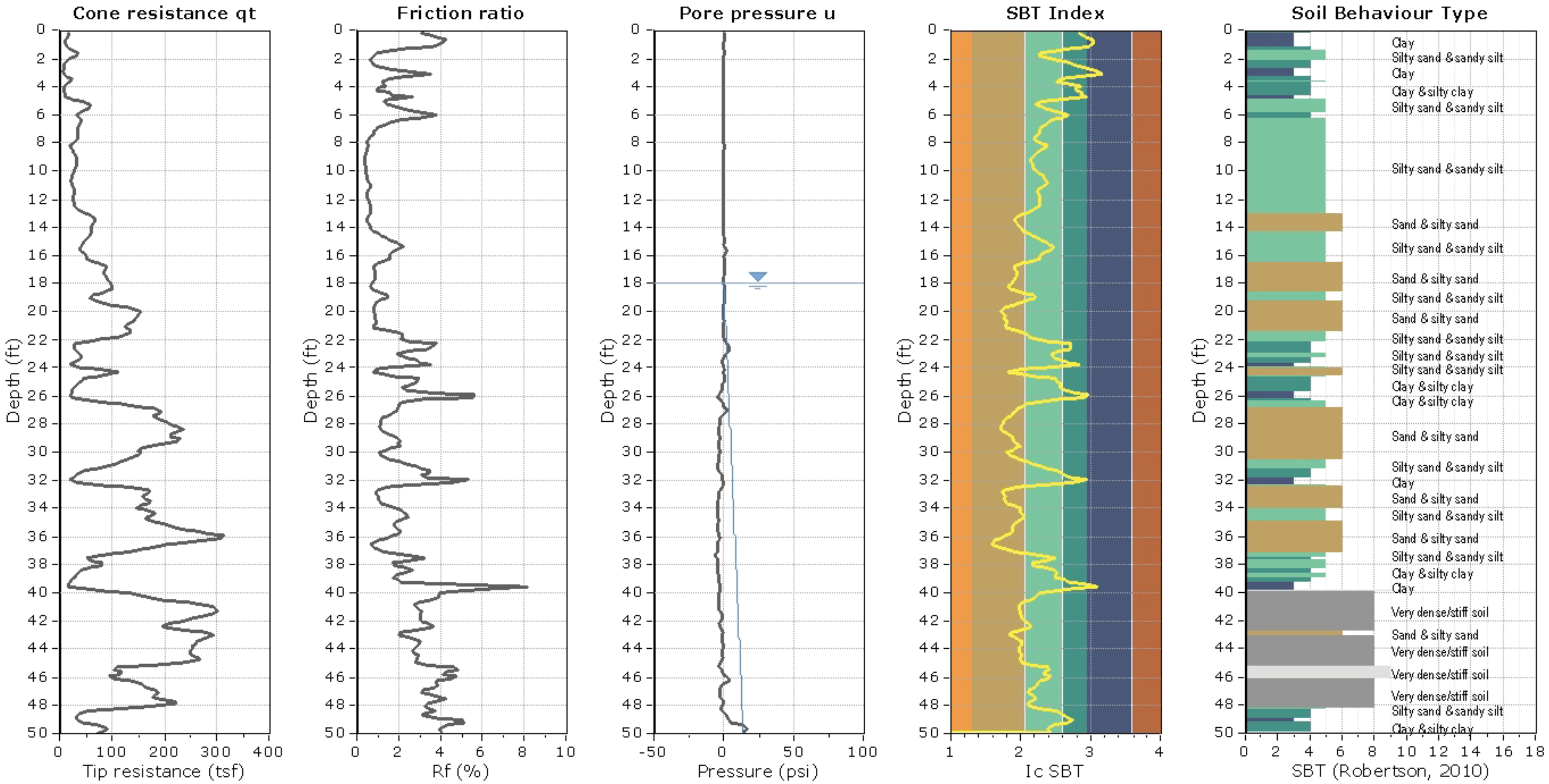
Total depth: 50.52 ft, Date: 1/31/2018
Estimated Groundwater Depth: 18 feet bgs
Cone Operator: Middle Earth Geo Testing, Inc.

**RUBY & A STREETS**
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

**CONE PENETRATION TEST RESULTS**
**CPT-1**

| Date 02/01/19 | Project No. 18-1444 | Figure A-1 |
|---|---|---|




**SBT legend**

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

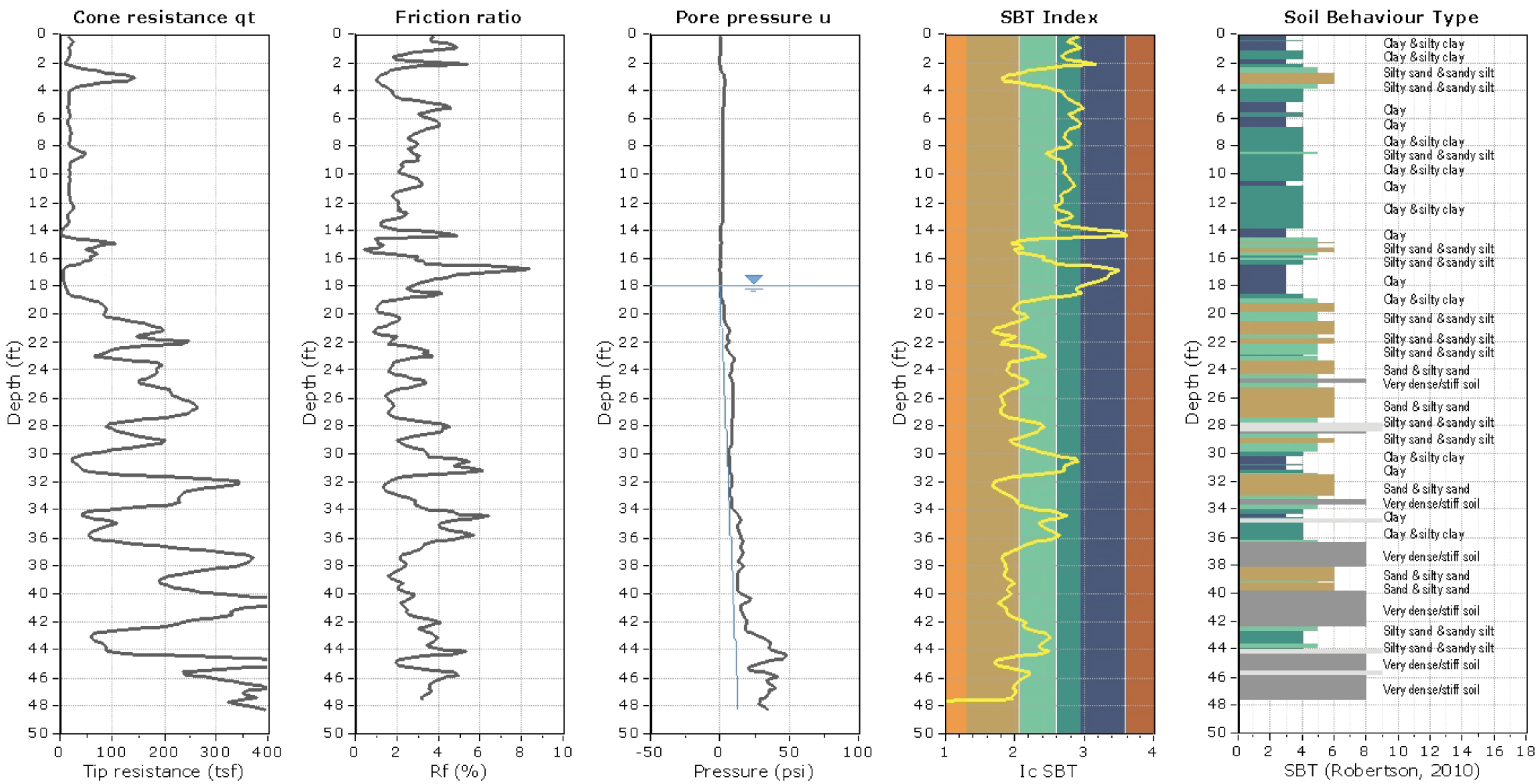
Total depth: 50.52 ft, Date: 1/31/2018
Estimated Groundwater Depth: 18 feet bgs
Cone Operator: Middle Earth Geo Testing, Inc.

**RUBY & A STREETS**
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

**CONE PENETRATION TEST RESULTS**
**CPT-2**

Date 02/01/19 | Project No. 18-1444 | Figure A-2

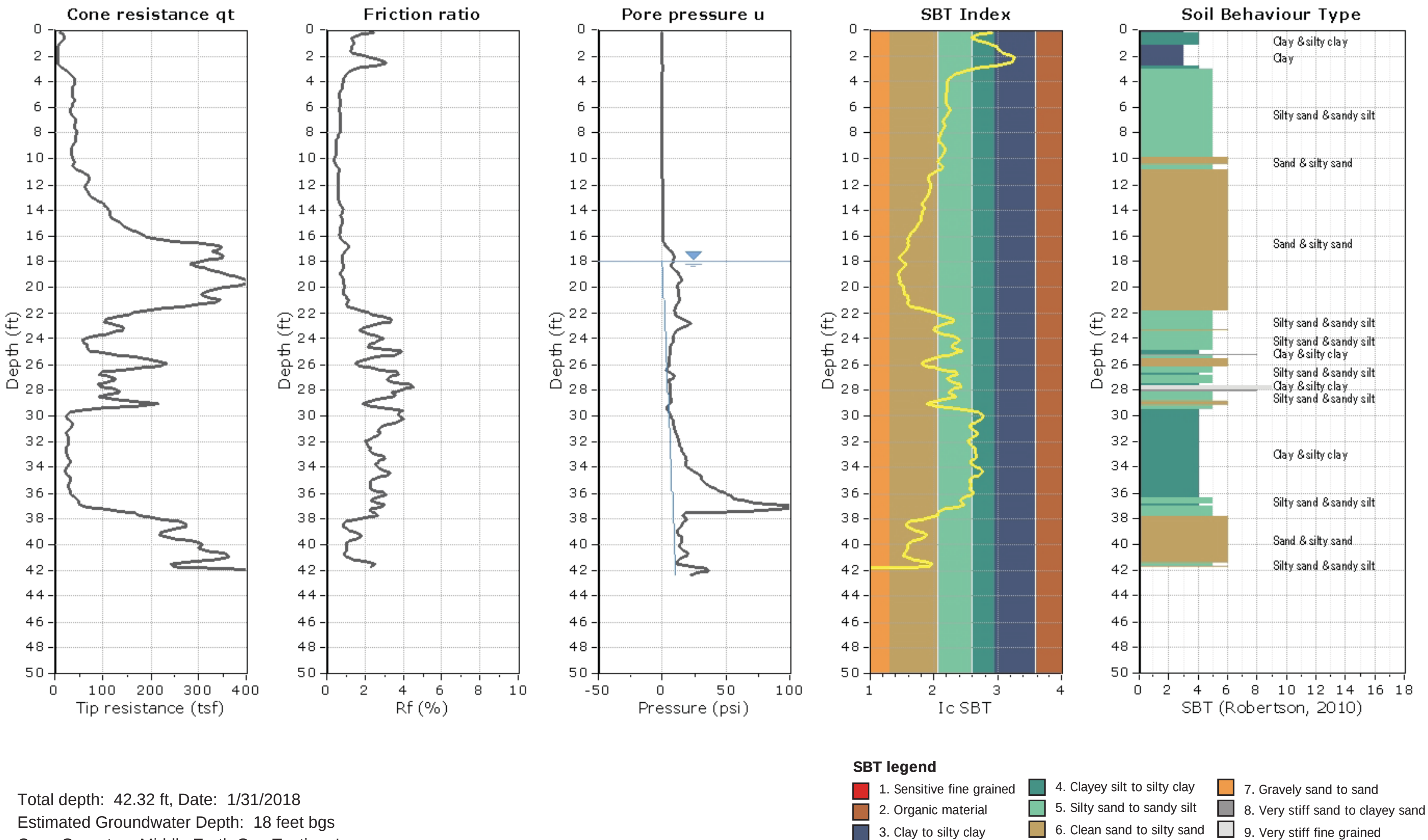



**SBT legend**

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

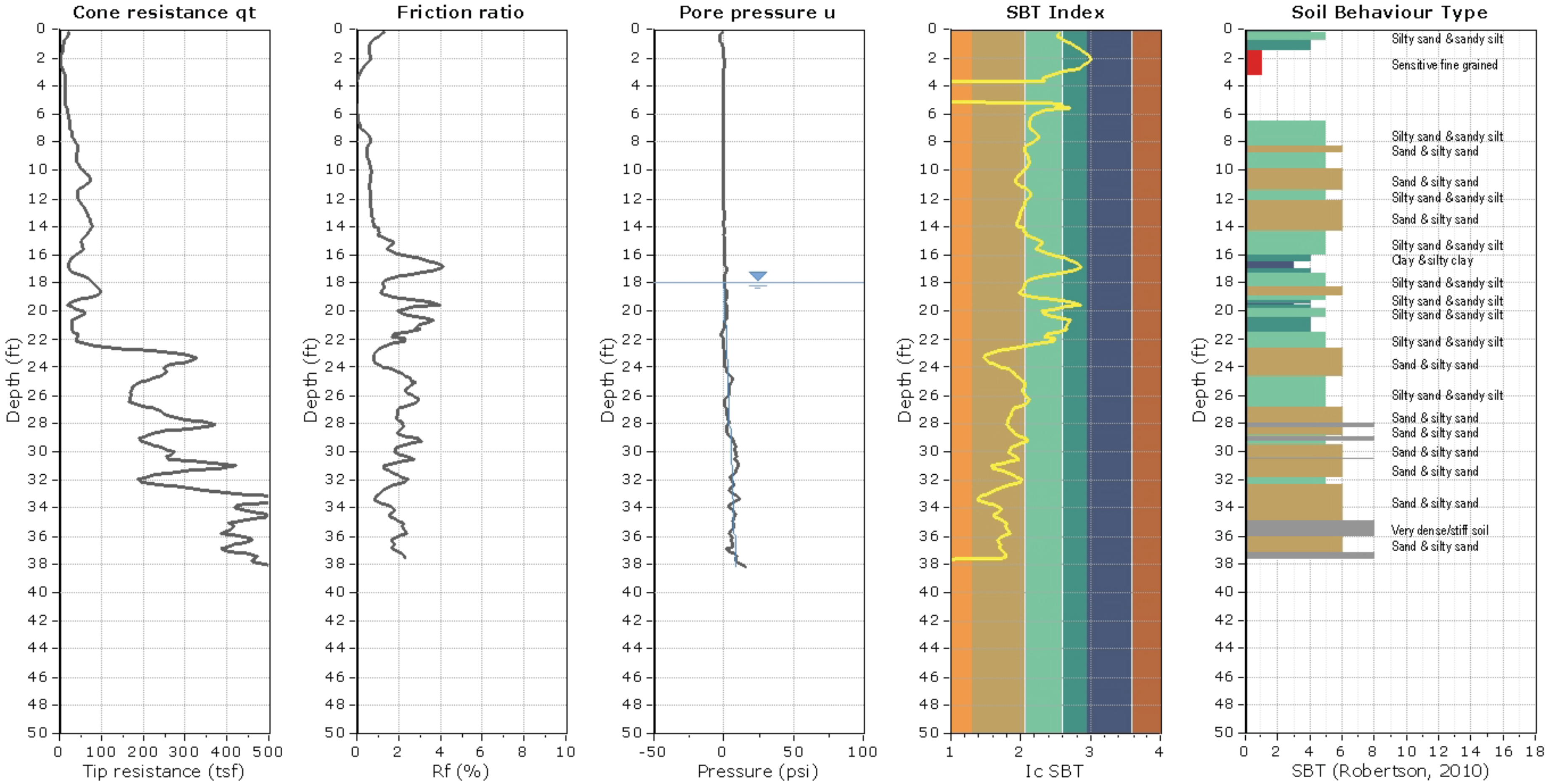
Total depth: 48.23 ft, Date: 1/31/2018
Estimated Groundwater Depth: 18 feet bgs (pore pressure dissapation)
Cone Operator: Middle Earth Geo Testing, Inc.

| | |
|---|---|
| **RUBY & A STREETS**<br>Castro Valley, California | **CONE PENETRATION TEST RESULTS**<br>**CPT-3** |
| ROCKRIDGE GEOTECHNICAL | Date 02/01/19 · Project No. 18-1444 · Figure A-3 |




Total depth: 42.32 ft, Date: 1/31/2018
Estimated Groundwater Depth: 18 feet bgs
Cone Operator: Middle Earth Geo Testing, Inc.

| | |
|---|---|
| **RUBY & A STREETS**<br>Castro Valley, California | **CONE PENETRATION TEST RESULTS**<br>**CPT-4** |
| ROCKRIDGE GEOTECHNICAL | Date 02/01/19 \| Project No. 18-1444 \| Figure A-4 |




**SBT legend**

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

Total depth: 38.22 ft, Date: 1/31/2018
Estimated Groundwater Depth: 18 feet bgs
Cone Operator: Middle Earth Geo Testing, Inc.

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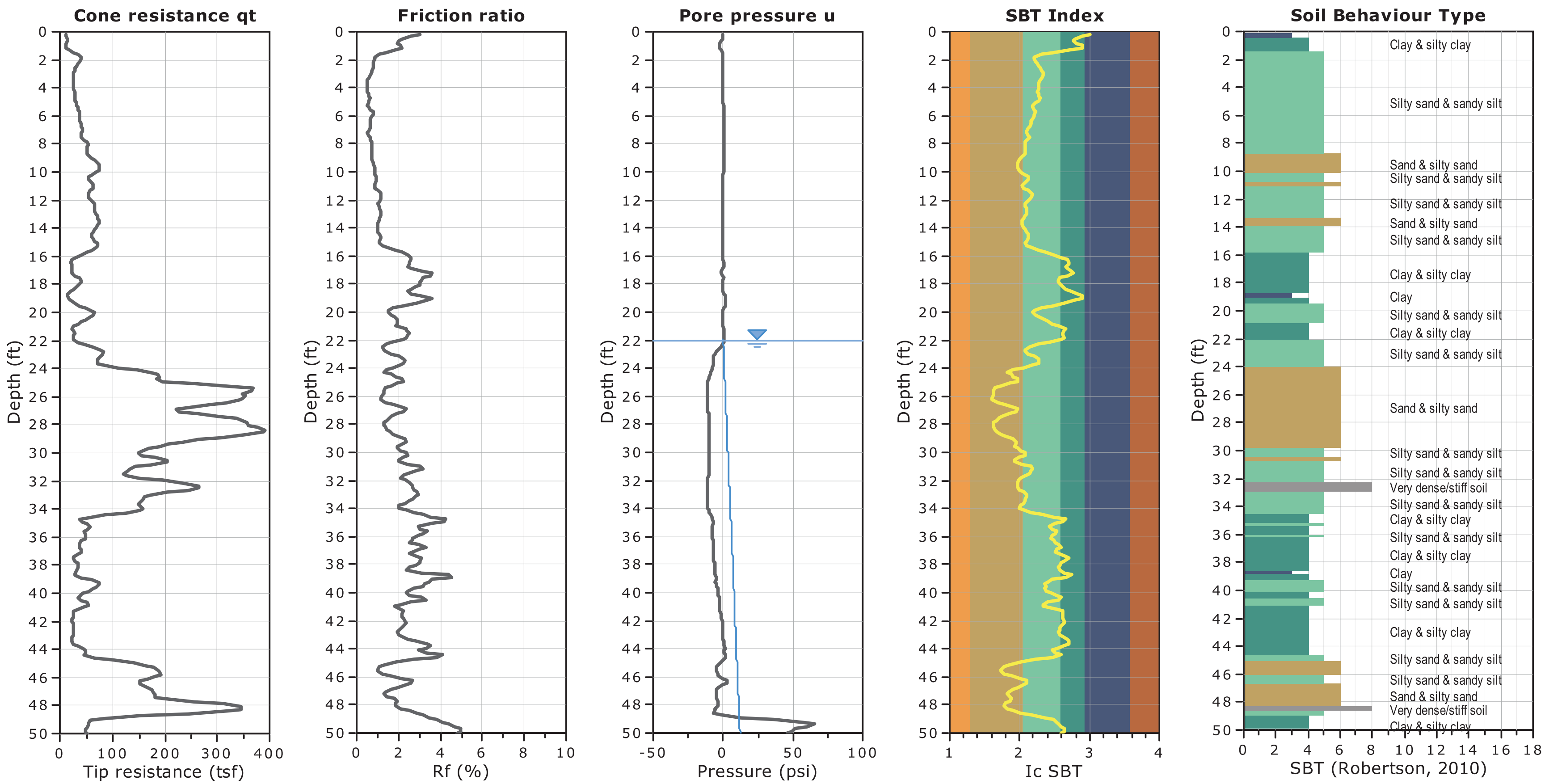
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

**CONE PENETRATION TEST RESULTS**

**CPT-5**

Date 02/01/19 | Project No. 18-1444 | Figure A-5




**SBT legend**

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

Total depth: 50.36 ft, Date: 12/20/2018
Estimated Groundwater Depth: 22 feet bgs
Cone Operator: Middle Earth Geo Testing, Inc.

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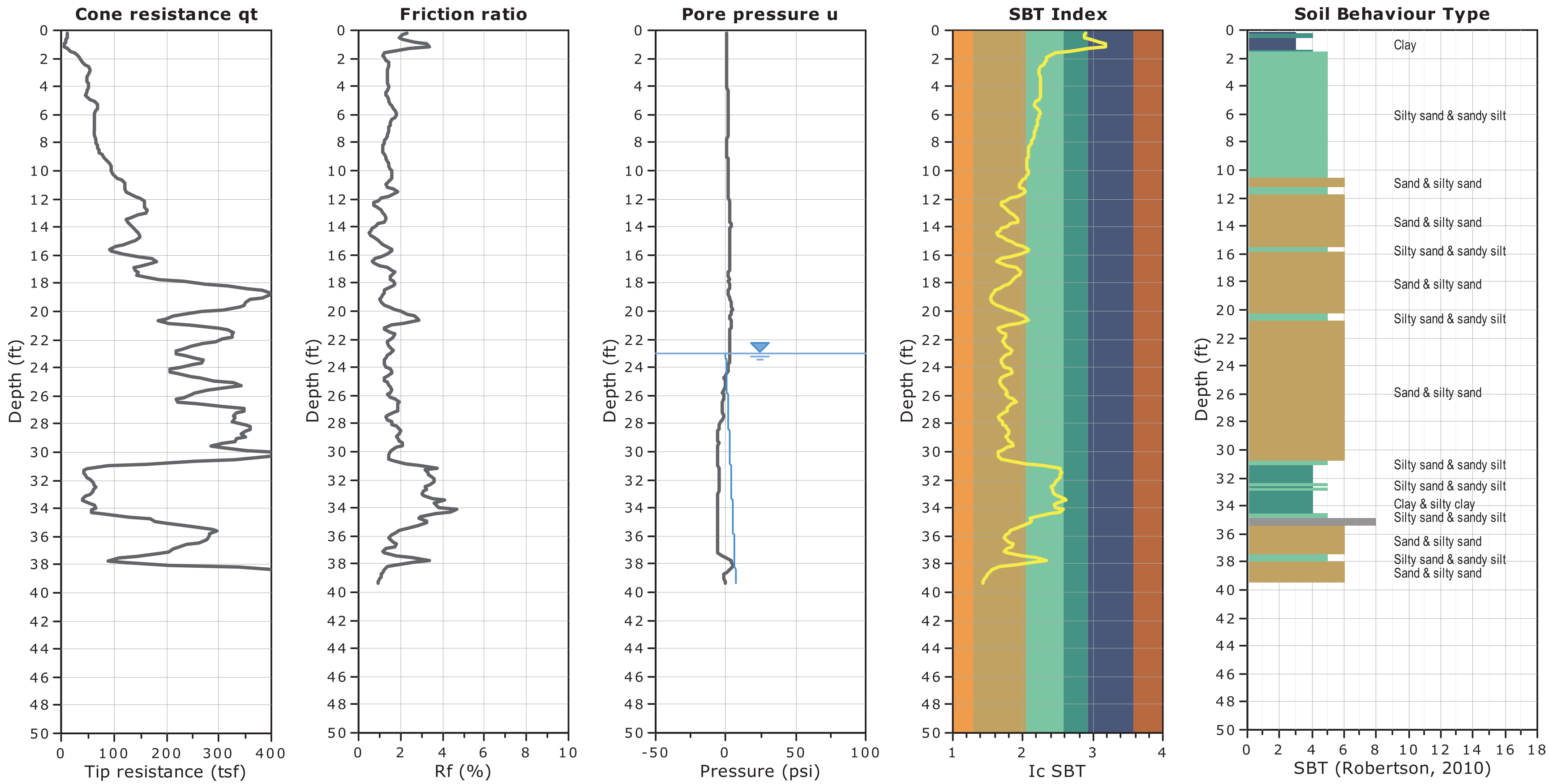
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

# CONE PENETRATION TEST RESULTS
# CPT-6

| Date 02/01/19 | Project No. 18-1444 | Figure A-6 |
|---|---|---|




**SBT legend**

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

Total depth: 39.37 ft, Date: 12/20/2018
Estimated Groundwater Depth: 23 feet bgs
Cone Operator: Middle Earth Geo Testing, Inc.

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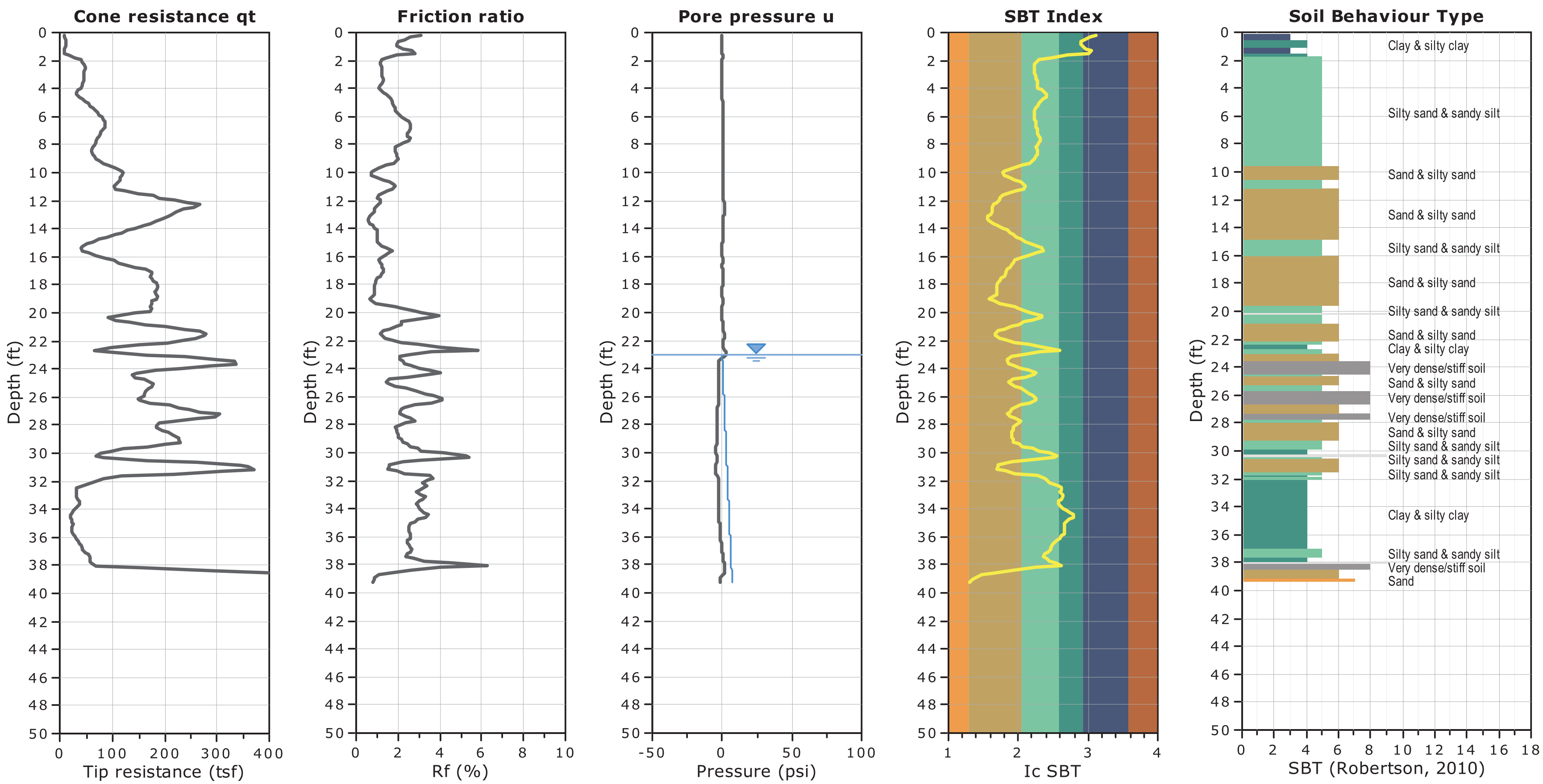
ROCKRIDGE GEOTECHNICAL

**CONE PENETRATION TEST RESULTS**
**CPT-7**

Date 02/01/19 | Project No. 18-1444 | Figure A-7

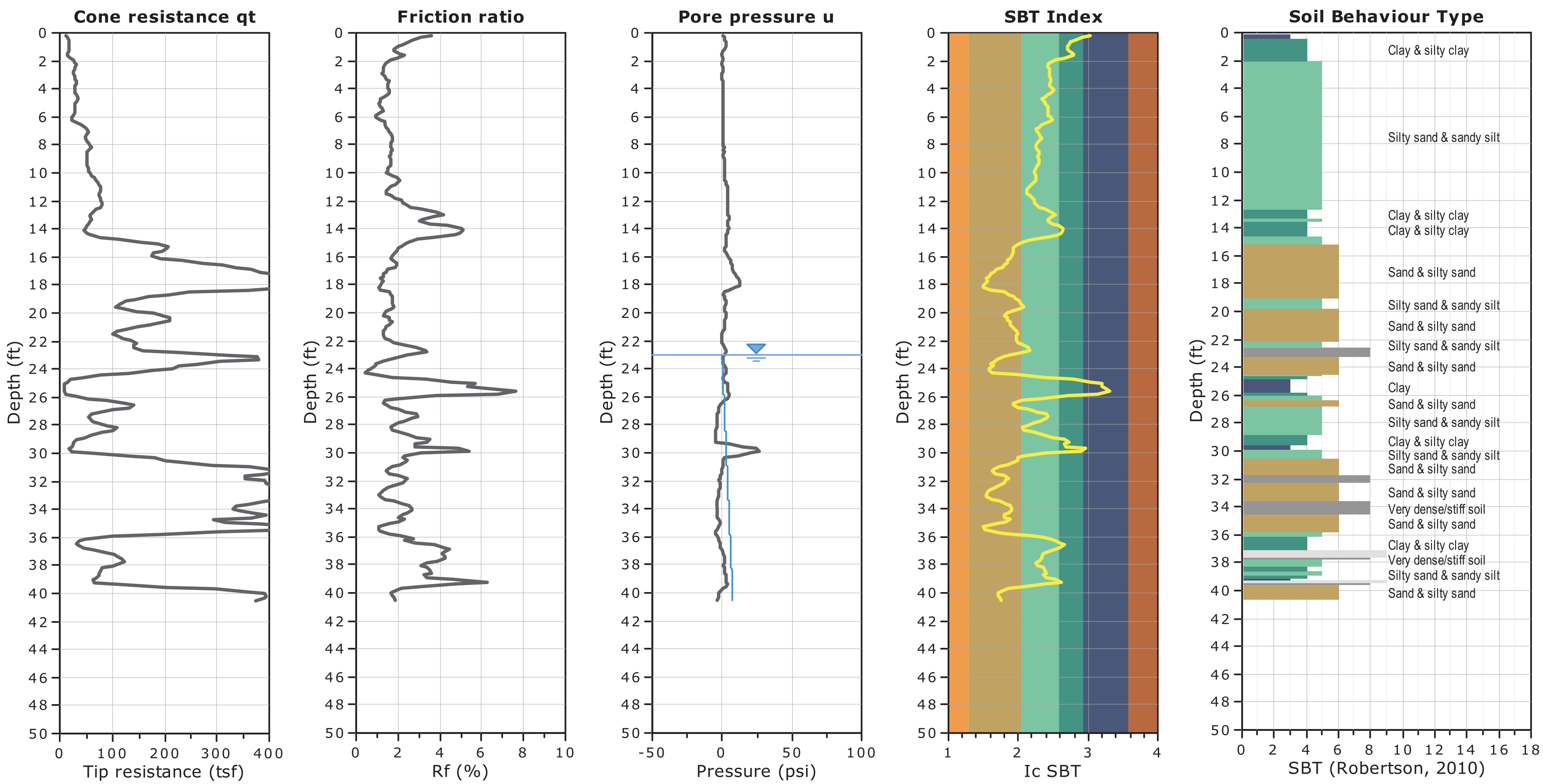



Total depth: 39.21 ft, Date: 12/20/2018
Estimated Groundwater Depth: 23 feet bgs
Cone Operator: Middle Earth Geo Testing, Inc.

**SBT legend**

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

| | |
|---|---|
| **RUBY & A STREETS**<br>Castro Valley, California | **CONE PENETRATION TEST RESULTS**<br>**CPT-8** |
| ROCKRIDGE GEOTECHNICAL | Date 02/01/19 \| Project No. 18-1444 \| Figure A-8 |




**SBT legend**

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

Total depth: 40.52 ft, Date: 12/20/2018
Estimated Groundwater Depth: 23 feet bgs
Cone Operator: Middle Earth Geo Testing, Inc.

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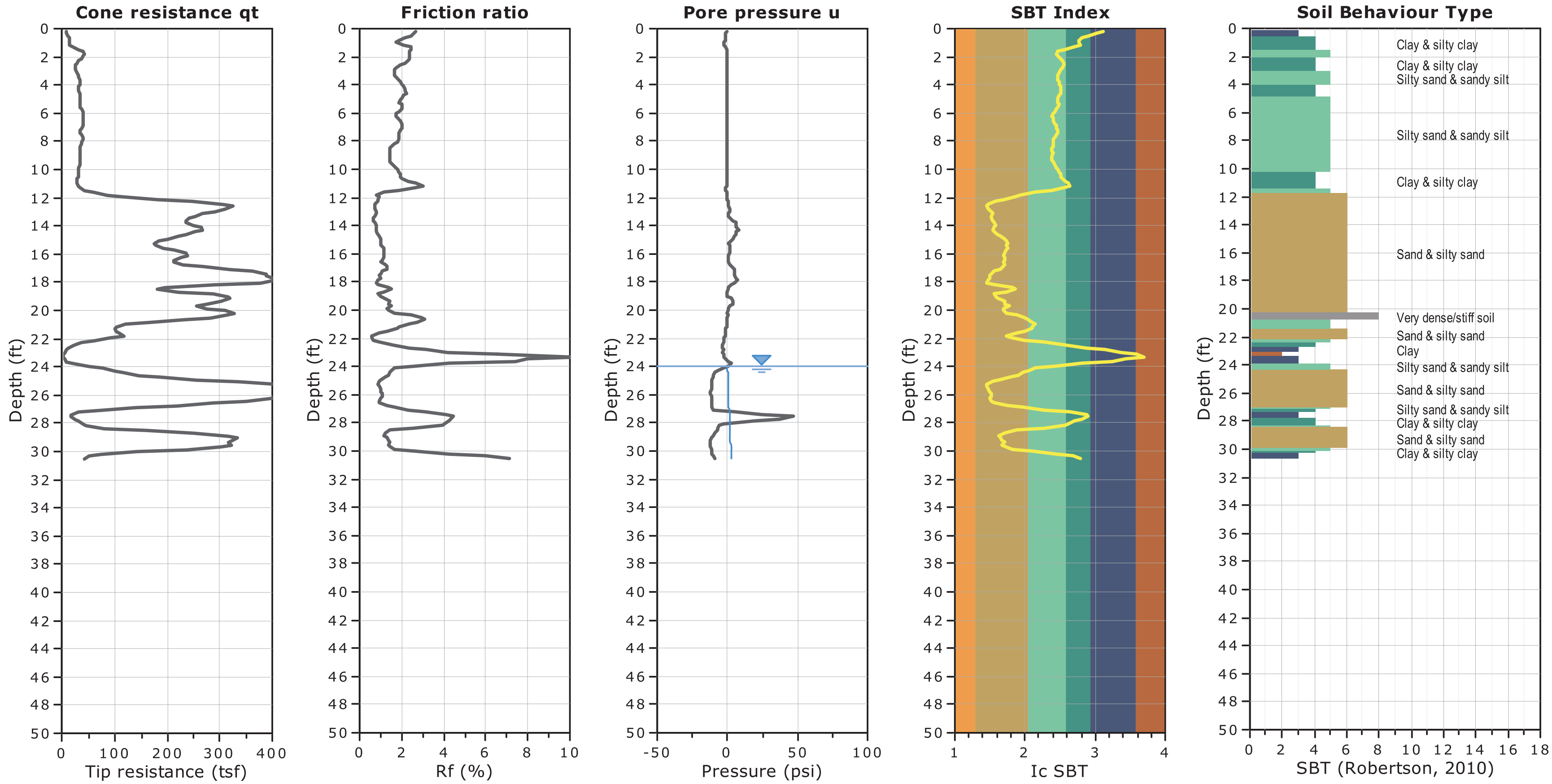
ROCKRIDGE GEOTECHNICAL

**CONE PENETRATION TEST RESULTS**
**CPT-9**

| Date 02/01/19 | Project No. 18-1444 | Figure A-9 |
|---|---|---|




**SBT legend**

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

Total depth: 30.51 ft, Date: 12/20/2018
Estimated Groundwater Depth: 24 feet bgs
Cone Operator: Middle Earth Geo Testing, Inc.

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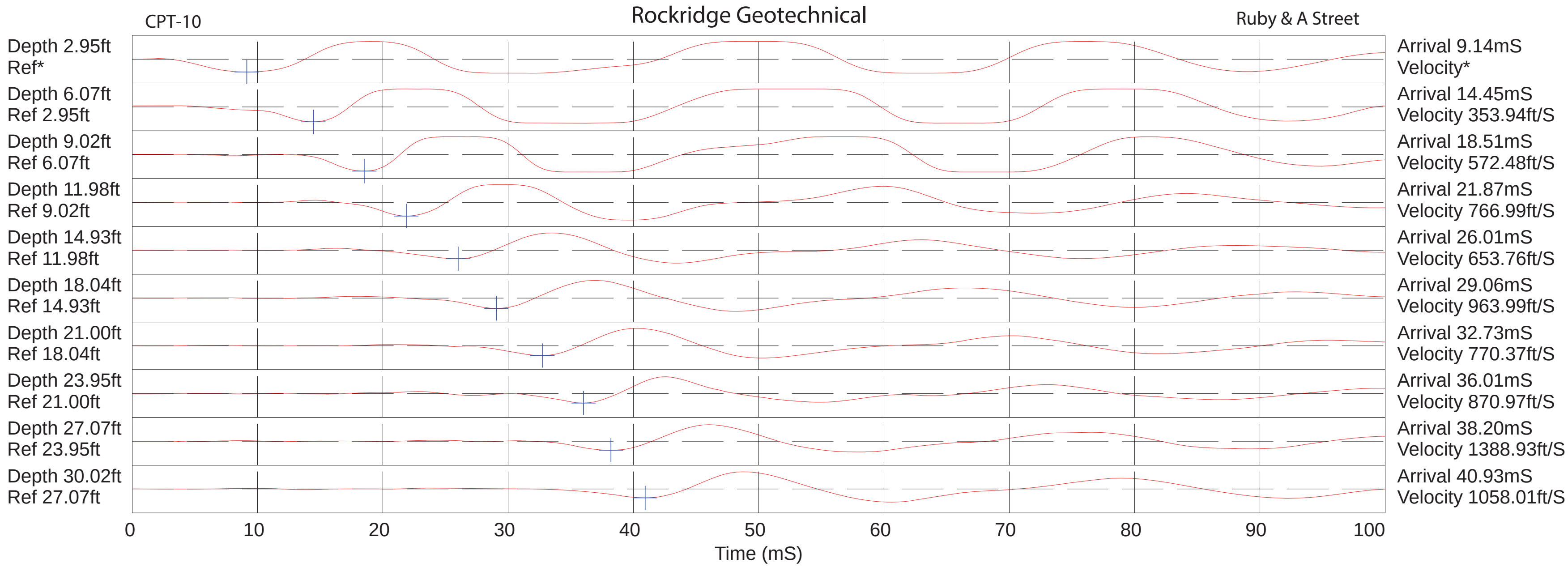
ROCKRIDGE GEOTECHNICAL

**CONE PENETRATION TEST RESULTS**
**CPT-10**

| Date 02/01/19 | Project No. 18-1444 | Figure A-10 |
|---|---|---|

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Castro Valley, California

ROCKRIDGE GEOTECHNICAL

**SHEAR WAVE VELOCITY**
**CPT-10**

| Date 02/01/19 | Project No. 18-1444 | Figure A-10b |
|---|---|---|

ROCKRIDGE GEOTECHNICAL

**APPENDIX B**
**Logs of Borings and Hand Augers**

| PROJECT: | RUBY & A STREETS<br>Castro Valley, California | Log of Boring B-1<br>PAGE 1 OF 1 |
|---|---|---|

Boring location: See Site Plan, Figure 2
Date started: 12/20/18 | Date finished: 12/20/18
Drilling method: Hollow Stem Auger (6" OD)
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic Safety
Sampler: Sprague & Henwood (S&H), 1.5" Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: C. Proto
Drilled by: Pitcher Drilling Company
Rig: CME 850/PD-80 Track

| DEPTH (feet) | Sampler Type | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2 | | | | CL | SANDY CLAY (CL) gray-brown, soft to medium stiff, moist to wet, fine-grained sand, with rootlets | | | | | | |
| 3 | S&H | 3 3 2 | 4 | CL | SANDY CLAY (CL) yellow-brown, soft to medium stiff, dry to moist, fine-grained sand, trace rootlets | | | | | 9.4 | 82 |
| 1–4 | | | | | BULK SAMPLE | | | | | | |
| 6 | ST | 50-100 psi | | CL | LL = 28, PI = 10; see Figure C-1<br>Collapse Test; see Figure C-2 | | | | 52 | 15.3 | 87 |
| 8 | SPT | 4 4 5 | 13 | ML | SANDY SILT (ML) yellow to yellow-brown, medium dense, dry to moist, fine-grained sand | | | | 57 | 12.3 | |
| 11 | ST | 50-200 psi | | ML | | | | | | | |
| 12–20 | | | | SP-SM | SAND with GRAVEL and SILT (SP-SM) yellow-brown, very dense, dry to moist, fine- to coarse-grained sand | | | | | | |
| 16 | SPT | 15 20 18 | 53 | SP-SM | | | | | | | |
| 21 | SPT | 10 6 18 | 34 | CL | SANDY CLAY (CL) olive-gray with orange-brown mottling, very stiff to hard, moist to wet, fine-grained sand | | | | | | |
| 22–24 | | | | SM | SILTY SAND with GRAVEL (SM) olive-gray, dense, moist to wet, fine-grained sand | | | | | | |
| 24–29 | | | | GP-GM | SANDY GRAVEL with SILT (GP-GM) olive-gray with yellow and orange mottling, very dense, wet | | | | | | |
| 26 | SPT | 18 35 36 | 99 | GP-GM | | | | | | | |
| 29–31 | | | | SC | CLAYEY SAND (SC) gray, medium dense to dense, wet, fine-grained sand | | | | | | |
| 31 | SPT | 5 9 15 | 34 | SC | | | | | | | |
| 31–31.5 | | | | SM | SILTY SAND (SM) gray, dense, wet, fine- to coarse-grained sand, trace fine gravel | | | | | | |

Boring terminated at a depth of 31.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 24 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.4, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1444 | Figure: B-1

# Log of Boring B-2



PROJECT: **RUBY & A STREETS**
Castro Valley, California

Boring location: See Site Plan, Figure 2
Date started: 12/20/18 | Date finished: 12/20/18
Drilling method: Hollow Stem Auger (6" OD)
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Automatic Safety
Sampler: Sprague & Henwood (S&H), 1.5" Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: C. Proto
Drilled by: Pitcher Drilling Company
Rig: CME 850/PD-80 Track

| DEPTH (feet) | SAMPLES: Sampler Type | SAMPLES: Blows/ 6" | SAMPLES: SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2 | | | | CL | SANDY CLAY (CL) gray-brown, moist, fine-grained sand, with rootlets | | | | | | |
| 2–3.5 | S&H | 3, 4, 4 | 6 | ML | SANDY SILT (ML) brown to yellow-brown, medium stiff, dry, fine-grained sand, with rootlets | | | | 66 | 8.8 | 78 |
| 5–6.5 | S&H | 6, 7, 7 | 11 | ML | stiff | | | | 60 | 8.8 | 91 |
| 7.5–10 | ST | 25 - 200 psi | | SM | SILTY SAND (SM) yellow to yellow-brown, medium dense, dry, fine-grained sand<br>Collapse Test; see Figure C-3 | | | | 36 | 10.1 | 93 |
| 11–15.5 | S&H (12–13.5) | 13, 16, 18 | 27 | SP | SAND with GRAVEL (SP) yellow-brown, medium dense, dry, fine-grained sand, fine to coarse subrounded gravel, trace silt | | | | | | |
| 15.5–20 | SPT (17–18.5) | 9, 11, 10 | 29 | GP-GC | GRAVEL with CLAY and SAND (GP-GC) yellow-brown and orange-brown, medium dense, moist, coarse to fine gravel | | | | | | |
| 20–23 | SPT (22–23.5) | 9, 13, 11 | 34 | SP | SAND (SP) olive-gray, dense, wet, fine to medium grained sand, trace silt | | | | | | |
| 23–26 | | | | CL | CLAY (CL) yellow-brown with horizontal orange-brown mottling, very stiff to hard, wet, trace fine-grained sand | | | | | | |
| 26–29.5 | S&H (27.5–29) | 10, 12, 14 | 21 | SC | CLAYEY SAND (SC) gray, medium dense, wet, fine-grained sand, trace rootlets | | | | | | |
| 29–30 | SPT (28.5–30) | 7, 12, 20 | 45 | SM | SILTY SAND (SM) yellow-brown with horizontal orange-brown mottling, dense, wet, fine-grained sand | | | | | | |

Boring terminated at a depth of 30 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 22.7 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.4, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1444 | Figure: B-2

# Log of Boring B-3



PROJECT: **RUBY & A STREETS**
Castro Valley, California

| | |
|---|---|
| Boring location: | See Site Plan, Figure 2 |
| Date started: | 12/20/18 |
| Date finished: | 12/20/18 |
| Drilling method: | Hollow Stem Auger (6" OD) |
| Hammer weight/drop: | 140 lbs./30 inches |
| Hammer type: | Automatic Safety |
| Sampler: | Sprague & Henwood (S&H), 1.5" Standard Penetration Test (SPT), Shelby Tube (ST) |
| Logged by: | F. Zamani |
| Drilled by: | Pitcher Drilling Company |
| Rig: | CME 850/PD-80 Track |

LABORATORY TEST DATA

| DEPTH (feet) | Sampler Type | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–1 | | | | CL | Topsoil/SANDY CLAY (CL) | | | | | | |
| 1–3.5 | S&H (1.5–3) | 2, 2, 4 | 5 | SM | SILTY SAND (SM) yellow-brown, loose, dry, fine-grained sand, trace rootlets; BULK SAMPLE (1–4) | | | | | | |
| 3.5–8.5 | S&H (4–5.5) | 3, 4, 5 | 7 | CL | SANDY CLAY (CL) light brown, medium stiff, dry, fine-grained sand, trace rootlets; LL = 27, PI = 9; see Figure C-1; Grain Size Distribution; see Figure C-4 | | | | 67 | 8.4 | 81 |
| 7–9 | ST | 50-75 psi | | | | | | | 58 | | |
| 8.5–15 | SPT (9.5–11) | 3, 4, 5 | 13 | CL | SANDY CLAY (CL) gray-brown, stiff, moist, fine-grained sand | | | | | | |
| 13 | SPT (12–13.5) | 4, 7, 7 | 20 | | very stiff | | | | | | |
| 15–20 | SPT (17–18.5) | 7, 7, 14 | 29 | SM | SILTY SAND (SM) olive-gray, medium dense, moist, fine-grained sand, trace clay | | | | | | |
| 20–28.5 | SPT (22–23.5) | 18, 19, 20 | 55 | GP-GM | SANDY GRAVEL with SILT (GP-GM) yellow and orange-brown, very dense, wet, fine- to coarse sand and gravel | | | | | | |
| | SPT (27–28.5) | 12, 20, 20 | 56 | | | | | | | | |
| 28.5–33 | S&H (32–33) | 30, 50/6" | 64 | GP-GM | SANDY GRAVEL with SILT (GP-GM) (continued) | | | | | | |

Boring terminated at a depth of 33 feet below ground surface.
Boring backfilled with cement grout.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.8 and 1.4, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1444

Figure: B-3

PROJECT: **RUBY & A STREETS**
Castro Valley, California

# Log of Boring HA-1



| | | | |
|---|---|---|---|
| Boring location: | See Site Plan, Figure 2 | Logged by: | J. Sarmiento |
| Date started: 2/1/18 | Date finished: 2/1/18 | | |
| Drilling method: | Hand Auger | | |
| Hammer weight/drop: N/A | Hammer type: N/A | | |
| Sampler: | | | |

| DEPTH (feet) | SAMPLES: Sampler Type | Sample | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | LABORATORY TEST DATA: Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | CL | SANDY CLAY (CL) dark brown, moist, fine-grained sand, with silt, trace rootlets | | | | | | |
| 1 | GRAB | X | | | | LL = 28, PI = 8; see Figure C-1 | | | | | 23.1 | |
| 2 | | | | | | | | | | | | |
| 3 | GRAB | X | | | | brown, increased silt content, low plasticity Corrosion Test; see Figure C-6 | | | | | | |
| 4 | | | | | | | | | | | | |
| | | | | | SM | SILTY SAND (SM) brown, dry, fine-grained sand | | | | | | |
| 5 | GRAB | X | | | | | | | | | | |
| 6 | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | |

Boring terminated at a depth of 5.5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1444

Figure: B-4

| PROJECT: | **RUBY & A STREETS** Castro Valley, California | **Log of Boring HA-2** PAGE 1 OF 1 |
|---|---|---|

| | | |
|---|---|---|
| Boring location: See Site Plan, Figure 2 | | Logged by: J. Sarmiento |
| Date started: 2/1/18 | Date finished: 2/1/18 | |
| Drilling method: Hand Auger | | |
| Hammer weight/drop: N/A | Hammer type: N/A | |
| Sampler: | | |

| DEPTH (feet) | Sampler Type | Sample | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | SANDY CLAY (CL) dark brown, moist, fine-grained sand, with silt, trace rootlets | | | | | | |
| 1 | GRAB | X | | | CL | | | | | | | |
| 2 | | | | | | | | | | | | |
| | | | | | | SILTY SAND (SM) brown, dry to moist, fine-grained sand, trace clay | | | | | | |
| 3 | GRAB | X | | | | | | | | | | |
| 4 | | | | | SM | | | | | | | |
| 5 | GRAB | X | | | | dry | | | | | | |
| 6 | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | |

LABORATORY TEST DATA applies to the last six columns.

Boring terminated at a depth of 5.5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1444

Figure: B-5

| PROJECT: | RUBY & A STREETS<br>Castro Valley, California | Log of Boring HA-3<br>PAGE 1 OF 1 |
|---|---|---|

| | |
|---|---|
| Boring location: See Site Plan, Figure 2 | Logged by: M. Hachey |
| Date started: 2/27/18 / Date finished: 2/27/18 | |
| Drilling method: Hand Auger | |
| Hammer weight/drop: N/A / Hammer type: N/A | LABORATORY TEST DATA |
| Sampler: | |

| DEPTH (feet) | SAMPLES: Sampler Type | Sample | Blows/ 6" | SPT N-Value[1] | LITHOLOGY | MATERIAL DESCRIPTION | Type of Strength Test | Confining Pressure Lbs/Sq Ft | Shear Strength Lbs/Sq Ft | Fines % | Natural Moisture Content, % | Dry Density Lbs/Cu Ft |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | SANDY CLAY (CL)<br>dark brown, moist, fine-grained sand, with silt, trace rootlets | | | | | | |
| 1 | GRAB | X | | | | | | | | | | |
| 2 | | | | | | | | | | | | |
| 3 | | | | | | SILTY SAND (SM)<br>brown, dry to moist, fine-grained sand, trace rootlets | | | | | | |
| 4 | GRAB | X | | | | | | | | | | |
| 5 | | | | | | | | | | | | |
| 6 | | | | | SM | | | | | | | |
| 7 | GRAB | X | | | | | | | | | | |
| 8 | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | |
| | GRAB | X | | | | LL = 14, PI = 2; see Figure C-1 | | | | 34 | 8.8 | |
| 10 | | | | | | | | | | | | |

Boring terminated at a depth of 10 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1444

Figure: B-6

| UNIFIED SOIL CLASSIFICATION SYSTEM | | | |
|---|---|---|---|
| **Major Divisions** | | **Symbols** | **Typical Names** |
| **Coarse-Grained Soils** (more than half of soil > no. 200 sieve size) | Gravels (More than half of coarse fraction > no. 4 sieve size) | **GW** | Well-graded gravels or gravel-sand mixtures, little or no fines |
| | | **GP** | Poorly-graded gravels or gravel-sand mixtures, little or no fines |
| | | **GM** | Silty gravels, gravel-sand-silt mixtures |
| | | **GC** | Clayey gravels, gravel-sand-clay mixtures |
| | Sands (More than half of coarse fraction < no. 4 sieve size) | **SW** | Well-graded sands or gravelly sands, little or no fines |
| | | **SP** | Poorly-graded sands or gravelly sands, little or no fines |
| | | **SM** | Silty sands, sand-silt mixtures |
| | | **SC** | Clayey sands, sand-clay mixtures |
| **Fine -Grained Soils** (more than half of soil < no. 200 sieve size) | Silts and Clays LL = < 50 | **ML** | Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts |
| | | **CL** | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays |
| | | **OL** | Organic silts and organic silt-clays of low plasticity |
| | Silts and Clays LL = > 50 | **MH** | Inorganic silts of high plasticity |
| | | **CH** | Inorganic clays of high plasticity, fat clays |
| | | **OH** | Organic silts and clays of high plasticity |
| **Highly Organic Soils** | | **PT** | Peat and other highly organic soils |

| GRAIN SIZE CHART | | |
|---|---|---|
| **Classification** | **Range of Grain Sizes** | |
| | **U.S. Standard Sieve Size** | **Grain Size in Millimeters** |
| Boulders | Above 12" | Above 305 |
| Cobbles | 12" to 3" | 305 to 76.2 |
| Gravel | 3" to No. 4 | 76.2 to 4.76 |
| coarse | 3" to 3/4" | 76.2 to 19.1 |
| fine | 3/4" to No. 4 | 19.1 to 4.76 |
| Sand | No. 4 to No. 200 | 4.76 to 0.075 |
| coarse | No. 4 to No. 10 | 4.76 to 2.00 |
| medium | No. 10 to No. 40 | 2.00 to 0.420 |
| fine | No. 40 to No. 200 | 0.420 to 0.075 |
| Silt and Clay | Below No. 200 | Below 0.075 |

Unstabilized groundwater level

Stabilized groundwater level

## SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

Sonic

## SAMPLER TYPE

| | |
|---|---|
| C | Core barrel |
| CA | California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter |
| D&M | Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube |
| O | Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube |
| PT | Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube |
| S&H | Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter |
| SPT | Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter |
| ST | Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure |

**RUBY & A STREETS**
Castro Valley, California

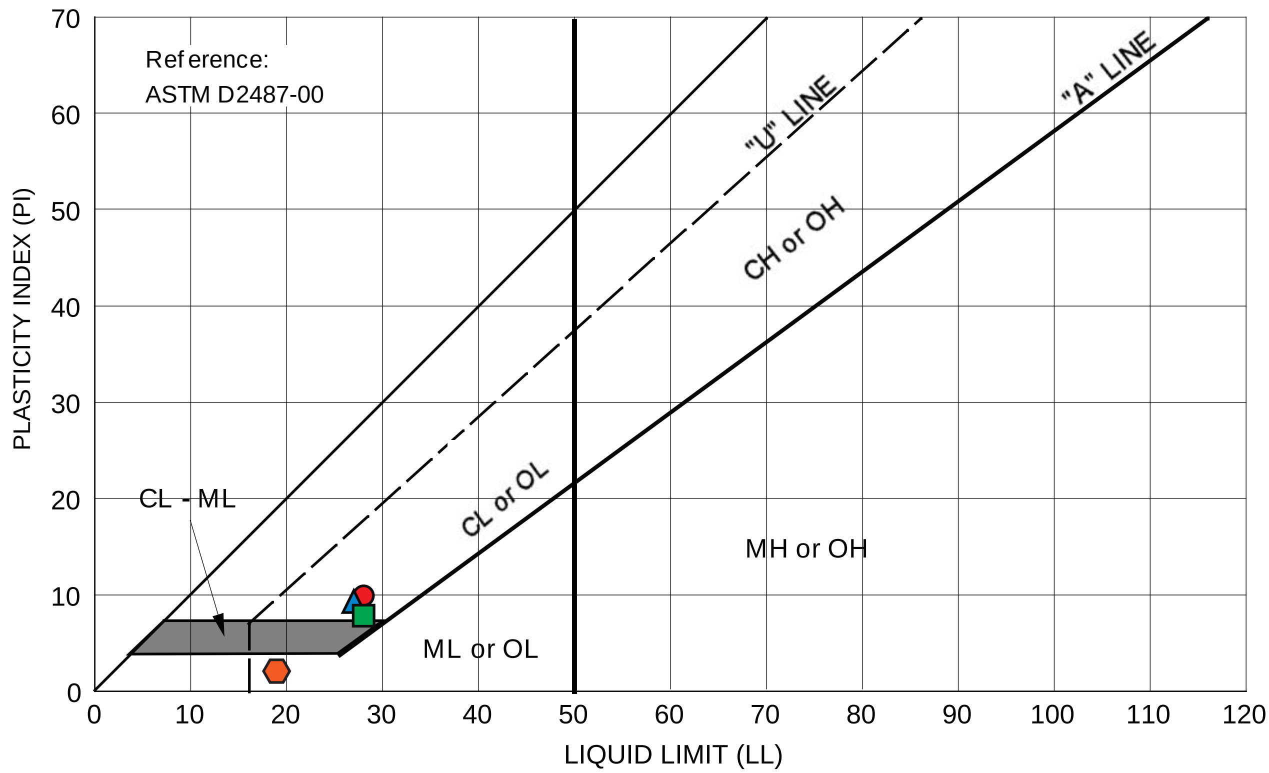
ROCKRIDGE GEOTECHNICAL

## CLASSIFICATION CHART

Date 02/01/19 | Project No. 18-1444 | Figure B-7

ROCKRIDGE GEOTECHNICAL

# APPENDIX C
# Laboratory Test Results



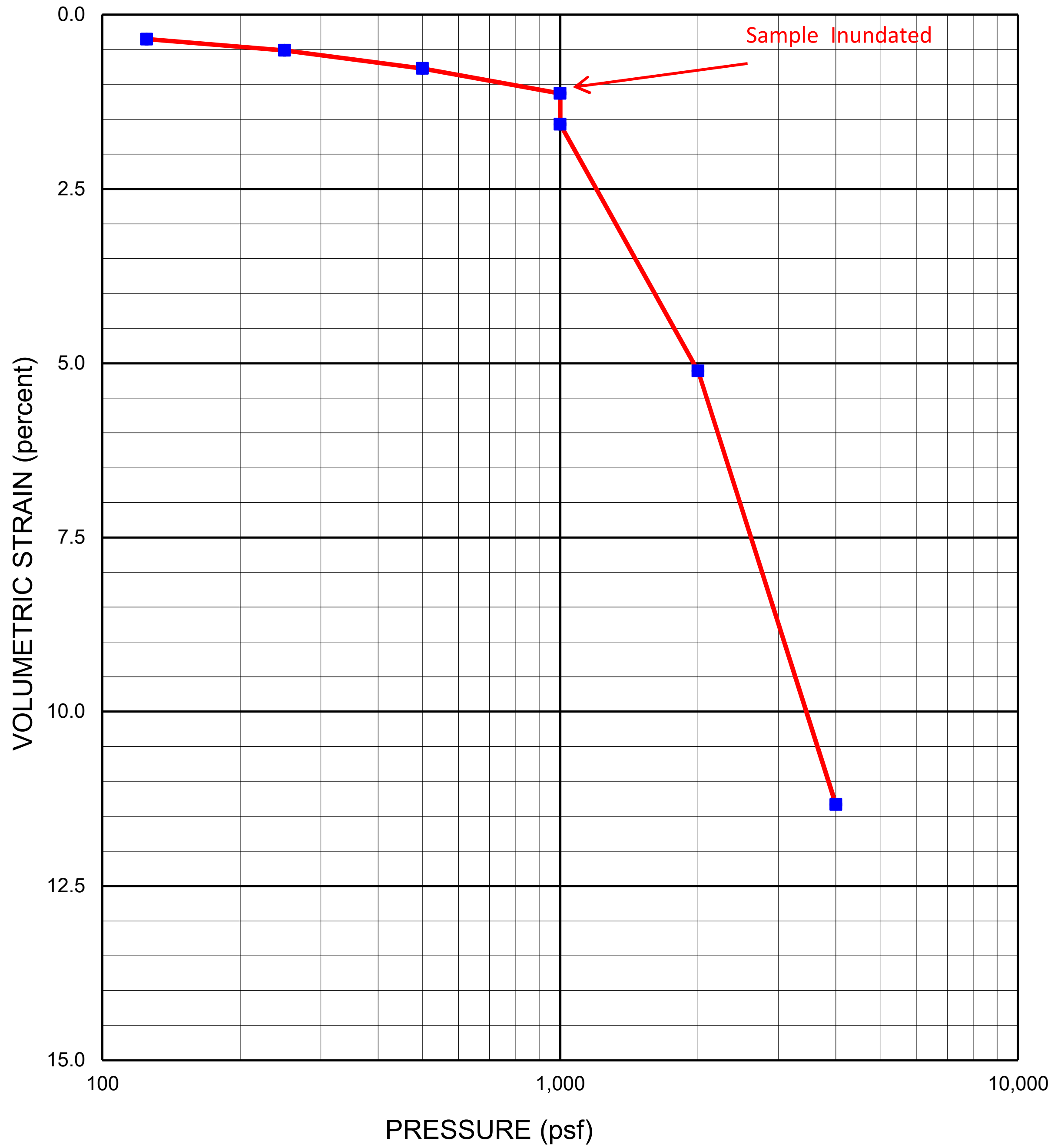

| Symbol | Source | Description and Classification | Natural M.C. (%) | Liquid Limit (%) | Plasticity Index (%) | % Passing #200 Sieve |
|---|---|---|---|---|---|---|
| ● | B-1 at 5.0 feet | SANDY CLAY (CL), yellow-brown | 15.6 | 28 | 10 | 52 |
| ▲ | B-3 at 5.5 feet | SANDY CLAY (CL), light brown | 8.4 | 27 | 9 | 67 |
| ■ | HA-1 at 1.0 feet | SANDY CLAY (CL), dark brown | 23.1 | 28 | 8 | -- |
| ⬢ | HA-3 at 9.5 feet | SILTY SAND (SM), brown | 8.8 | 19 | 2 | 34 |

**RUBY & A STREETS**
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

**PLASTICITY CHART**

Date 02/01/19 | Project No. 18-1444 | Figure C-1




| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Sampler Type: | Shelby Tube | | | Condition | Before Test | | | After Test | | |
| Diameter (in) | 2.00 | Height (in) | 0.75 | Water Content | $w_o$ | 15.3 | % | $w_f$ | 25.2 | % |
| Boring #: | B-1 | | | Void Ratio | $e_o$ | 0.91 | | $e_f$ | 0.69 | |
| Sample #: | 3 | | | Saturation | $S_o$ | 44.7 | % | $S_f$ | 96 | % |
| Depth: | 5.0' | | | Dry Density | $\gamma_d$ | 87 | pcf | $\gamma_d$ | 98 | pcf |
| | | LL | 28 | PL 18 | PI | 10 | | $G_s$ | 2.70 | (assumed) |
| Description: | SANDY CLAY (CL) yellow-brown | | | | | | | | | |

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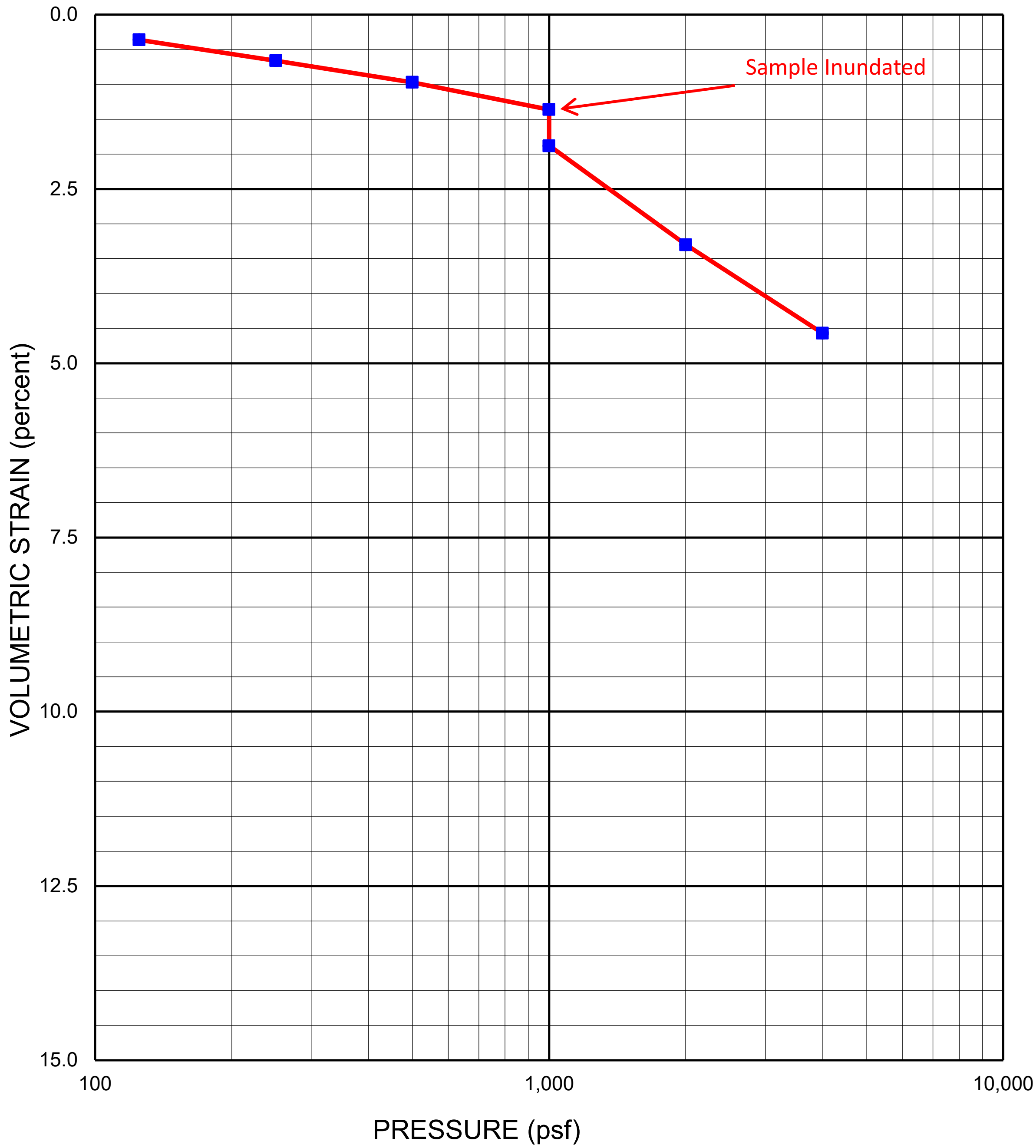
## COLLAPSE TEST REPORT

Date 01/23/19 Project No. 18-1444 Figure C-2




| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Sampler Type: | Shelby Tube | | | Condition | Before Test | | | After Test | |
| Diameter (in) | 2.00 | Height (in) | 0.75 | Water Content | $w_o$ | 10.1 % | $w_f$ | 22.6 % | |
| Boring #: | B-2 | | | Void Ratio | $e_o$ | 0.78 | $e_f$ | 0.70 | |
| Sample #: | 3 | | | Saturation | $S_o$ | 34.2 % | $S_f$ | 86 % | |
| Depth: | 7.5' | | | Dry Density | $\gamma_d$ | 93 pcf | $\gamma_d$ | 97 pcf | |
| | | LL | | PL | PI | | $G_s$ | 2.65 | (assumed) |
| Description: | SILTY SAND (SM) yellow to yellow-brown | | | | | | | | |

**RUBY & A STREETS**
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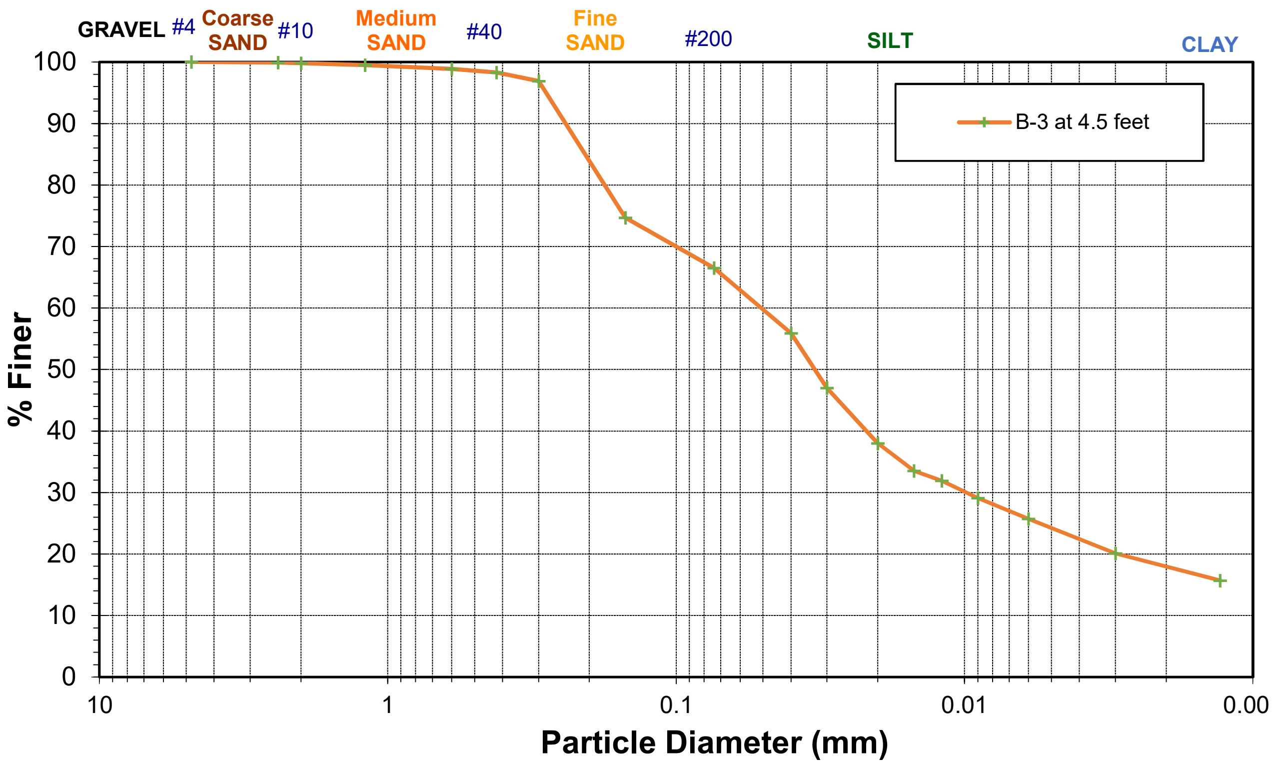
ROCKRIDGE GEOTECHNICAL

## COLLAPSE TEST REPORT

Date 01/23/19 Project No. 18-1444 Figure C-3




| Boring | | | B-3 |
|---|---|---|---|
| Depth (feet) | | | 4.5 |
| | | Size (mm) | SANDY CLAY (CL) light |
| Percent Finer | #4 | 4.8 | 100 |
| | #8 | 2.4 | 99.9 |
| | #10 | 2.0 | 99.8 |
| | #16 | 1.2 | 99.5 |
| | #30 | 0.60 | 98.9 |
| | #40 | 0.42 | 98.3 |
| | #50 | 0.30 | 96.9 |
| | #100 | 0.15 | 74.7 |
| | #200 | 0.074 | 66.5 |
| | Hydrometer Analysis | 0.040 | 55.9 |
| | | 0.030 | 47.0 |
| | | 0.020 | 38.0 |
| | | 0.015 | 33.5 |
| | | 0.012 | 31.9 |
| | | 0.009 | 29.1 |
| | | 0.006 | 25.7 |
| | | 0.003 | 20.1 |
| | | 0.001 | 15.7 |

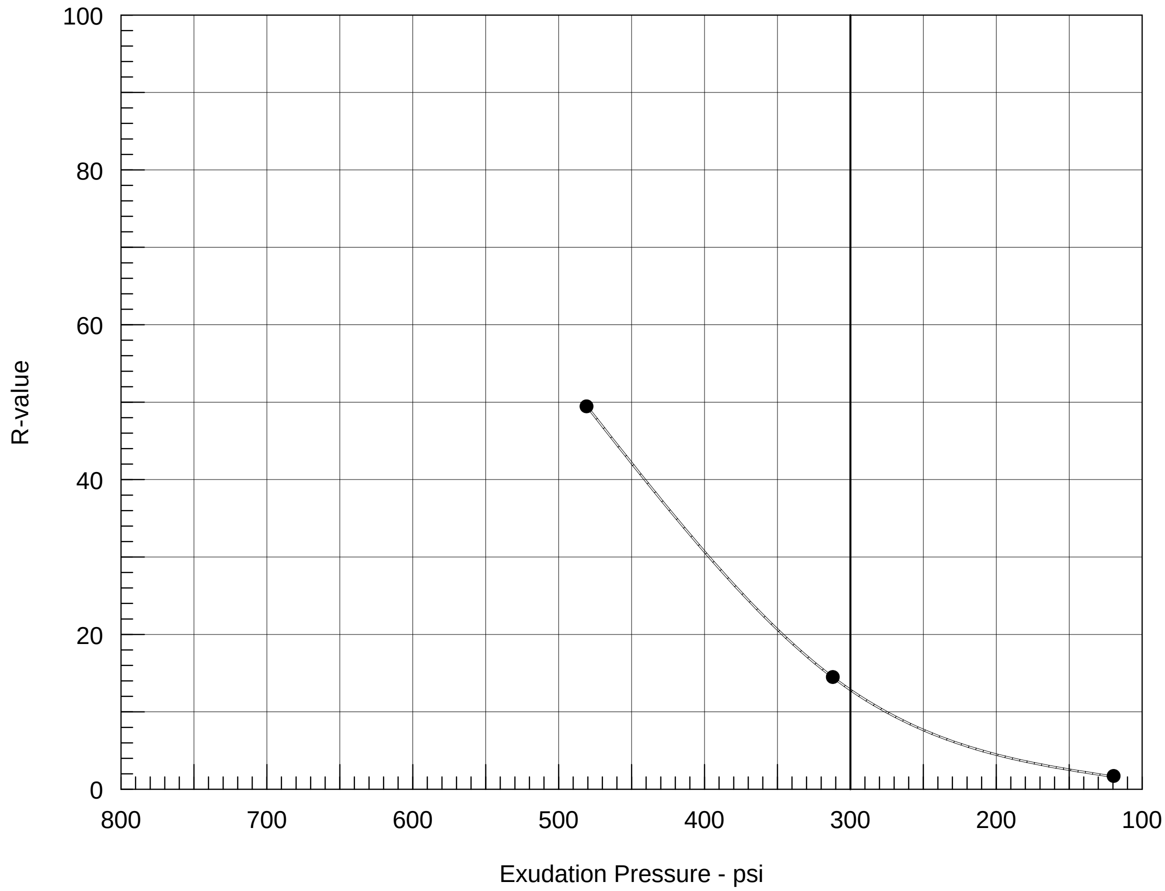
**RUBY & A STREETS**
Castro Valley, California

ROCKRIDGE GEOTECHNICAL

**Grain Size Analysis**

Date: 1/31/19 | Project No. 18-1444 | Figure C-4

# R-VALUE TEST REPORT




**Resistance R-Value and Expansion Pressure - Cal Test 301**

| No. | Compact. Pressure psi | Density pcf | Moist. % | Expansion Pressure psf | Horizontal Press. psi @ 160 psi | Sample Height in. | Exud. Pressure psi | R Value | R Value Corr. |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 120 | 115.7 | 13.4 | 0 | 53 | 2.30 | 480 | 55 | 49 |
| 2 | 100 | 112.7 | 15.6 | 0 | 119 | 2.46 | 311 | 14 | 14 |
| 3 | 50 | 108.7 | 17.2 | 0 | 155 | 2.70 | 119 | 1 | 2 |
| | | | | | | | | | |

| Test Results | Material Description |
|---|---|
| **R-value at 300 psi exudation pressure =** 13 | SILTY SAND (SM)<br>SANDY CLAY (CL) |

**Project No.:** 1190053C

**Project:** 18-1444 - Ruby and A Street

**Source of Sample:** B-1 and B-3, 1-4 foot composite

**Date:** 1/29/2019

**Tested by:** S. Poulos

**Checked by:** S. Poulos

**Remarks:**
Sampled by client on 12/20/18 and received on
1/17/19

**R-VALUE TEST REPORT**

**Applied Materials & Engineering, Inc.**

**Figure C-5**

# Soil Analysis Lab Results

Client: Rockridge Geotechnical
Job Name: Ruby and A Streets
Client Job Number: 18-1444
Project X Job Number: S180215D
February 19, 2018

| | Method | ASTM G187 | | ASTM D516 | | ASTM D512B | | SM 4500-NO3-E | SM 4500-NH3-C | SM 4500-S2-D | ASTM G200 | ASTM G51 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Bore# / Description** | **Depth** | **Resistivity** As Rec'd | Minimum | **Sulfates** | | **Chlorides** | | **Nitrate** | **Ammonia** | **Sulfide** | **Redox** | **pH** |
| | (ft) | (Ohm-cm) | (Ohm-cm) | (mg/kg) | (wt%) | (mg/kg) | (wt%) | (mg/kg) | (mg/kg) | (mg/kg) | (mV) | |
| HA-1-2 | 3.0-3.5 | 5,561 | 4,623 | 60 | 0.0060 | 660 | 0.0660 | 60 | 84.0 | 18.60 | 173 | 8.40 |

Unk = Unknown
NT = Not Tested
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract

Please call if you have any questions.

Prepared by,

Ernesto Padilla, BSME
Field Engineer

Respectfully Submitted,

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NACE Corrosion Technologist #16592
Professional Engineer
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