

Prepared for **Santa Clara County Housing Authority**

**GEOTECHNICAL INVESTIGATION
PROPOSED BUENA VISTA MOBILE HOME PARK
REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California**

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May 22, 2023
Project No. 23-2398

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Mary Jo Lee
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Subject: Geotechnical Investigation
Proposed Buena Vista Mobile Home Park – Redevelopment
3980 El Camino Real
Palo Alto, California

Dear Ms. Lee,

We are pleased to present our geotechnical investigation report for the proposed Buena Vista Mobile Home Park Redevelopment project to be constructed at 3980 El Camino Real in Palo Alto, California. Our geotechnical investigation was performed in accordance with our proposal dated March 22, 2023.

The project site is a relatively level, rectangular-shaped lot with plan dimensions of approximately 350 feet by 720 feet. It is bordered by commercial buildings to the northeast, Los Robles Avenue to the southeast, and single-family residences to the northwest and southwest. The site is currently occupied by the Buena Vista Mobile Home Park, which consists of at least 117 mobile home spaces, as well as various RVs and a single-family home. Other improvements at the site include internal streets and parking areas. A motel that previously occupied a portion of the property was recently demolished.

Plans are to demolish the existing improvements and redevelop the site in two phases to maintain at least 117 units. Approximately the northeastern two-thirds of the site will be redeveloped into a mobile home park and the southwestern one-third of the site will be redeveloped into a multi-family apartment building. As currently envisions, the multi-family apartment building will be three to four stories and constructed at grade. Other improvements will include new utilities, internal street improvements, and an asphalt-paved parking lot with 79 spaces for the multi-family apartment building.

From a geotechnical standpoint, 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

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issues to be addressed are: 1) the potential for seismically induced settlement due to post-liquefaction reconsolidation and cyclic densification following a major earthquake, and 2) providing adequate foundation support for the proposed structures. We conclude the proposed apartment building may be supported on a mat foundation bearing on engineered fill and the mobile homes may be supported on jacks bearing on Portland-cement concrete pavement.

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 site grading and foundation installations during 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.



Alex D. Limpert, P.E.
Project Engineer



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Enclosure

QUALITY CONTROL REVIEWER:



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**GEOTECHNICAL INVESTIGATION
BUENA VISTA MOBILE HOME PARK REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California**

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the Buena Vista Mobile Home Park redevelopment project at 3980 El Camino Real in Palo Alto, California. The site is located on the southwestern side of El Camino Real, just northwest of its intersection with Los Robles Avenue, as shown on the Site Location Map (Figure 1).

The project site is a relatively level, rectangular-shaped lot with plan dimensions of approximately 350 feet by 720 feet, as shown on the Site Plan (Figure 2). It is bordered by commercial buildings to the northeast, Los Robles Avenue to the southeast, and single-family residences to the northwest and southwest. The site is currently occupied by the Buena Vista Mobile Home Park, which consists of at least 117 mobile home spaces, as well as various RVs and a single-family home. Other improvements at the site include internal streets and parking areas. A motel that previously occupied a portion of the property was recently demolished.

Plans are to demolish the existing improvements and redevelop the site in two phases to maintain at least 117 units. Approximately the northeastern two-thirds of the site will be redeveloped into a mobile home park and the southwestern one-third of the site will be redeveloped into a multi-family apartment building. As currently envisions, the multi-family apartment building will be three to four stories and constructed at grade. Other improvements will include new utilities, internal street improvements, and an asphalt-paved parking lot with 79 spaces for the multi-family apartment building.

2.0 SCOPE OF SERVICES

Our investigation was performed in accordance with our proposal dated March 22, 2023. Our geotechnical investigation included exploring subsurface conditions at the site by performing

five cone penetration tests (CPTs), drilling eight test borings, and performing laboratory testing on selected soil samples. We used the data from our subsurface investigation to perform engineering analyses to develop conclusions and recommendations regarding:

- subsurface and groundwater conditions
- site seismicity and seismic hazards, including the potential for liquefaction and lateral spreading, and total and differential settlement resulting from liquefaction and/or cyclic densification
- the most appropriate foundation type(s) for the proposed apartment building and mobile homes
- design criteria for the recommended foundation type(s)
- estimates of settlement of the proposed buildings
- slab-on-grade floors, if appropriate
- lateral earth pressures for design of site retaining walls and below-grade walls (i.e., elevator pit walls)
- site grading and fill placement, including fill quality and compaction requirements
- subgrade preparation for interior and exterior concrete slabs-on-grade
- surface drainage and bioswales
- flexible and rigid pavement design
- 2022 California Building Code (CBC) site class and design spectral response acceleration parameters
- corrosivity of the near-surface soil and the potential effects on buried concrete and metal structures and foundations
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

We investigated the subsurface conditions beneath the site by performing five CPTs, drilling eight test borings, and performing laboratory tests on selected soil samples. The approximate locations of the CPTs and borings are shown on Figure 2. Prior to our field investigation, we obtained a drilling permit from Santa Clara Valley Water District (SCVWD). We also contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained

C. Cruz Sub-Surface Locators, a private utility locator, to check for buried utilities at the CPT and boring locations to reduce the potential of encountering buried utilities during our field investigation. Details of the field investigation and laboratory testing are described below.

3.1 Cone Penetration Tests

Five CPTs, designated as CPT-1 through CPT-5, were performed on April 11, 2023, by Middle Earth Geo Testing, Inc. (Middle Earth) of Hayward, California. The CPTs were advanced to target depths of 20 to 100 feet below the ground surface (bgs). Middle Earth performed the CPTs by hydraulically pushing an 1.7-inch-diameter cone-tipped probe with a projected area of 15 square centimeters into the ground using a truck rig with 25-ton pushing capacity. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone measured soil parameters for the entire depth advanced. Soil data, including tip resistance, frictional resistance, and pore water pressure, were recorded by a computer while the tests were conducted. A computer processed accumulated data to provide engineering information, such as the soil behavior types (Robertson, 2010) and approximate strength characteristics of the soil encountered. The CPT logs, showing tip resistance, friction ratio, pore pressure, and correlated soil behavior type with depth are presented in Appendix A on Figures A-1 through A-5. Groundwater was measured in the CPTs and the depth of the groundwater and the measurement method are noted on the CPT logs. Upon completion, the CPTs were backfilled with cement grout in accordance with SCVWD requirements.

3.2 Test Borings

Eight test borings, designated as Boring B-1 through B-8, were drilled on April 1 and 8, 2023, by Exploration Geoservices, Inc. of San Jose, California, at the approximate locations shown on Figure 2. The borings drilled on April 1 (B-1 and B-2) were drilled to depths of 40 feet bgs using a Mobile B-53 truck-mounted drill rig. The borings drilled on April 8 were drilled to depths of 15 to 20 feet bgs using a Mobile B-40 truck-mounted drill rig. Both drill rigs were equipped with 8-inch-outside-diameter hollow-stem flight augers. During drilling, our field engineer logged the

soil encountered and obtained representative samples for visual classification and laboratory testing. Our field engineer noted the date and time when groundwater was encountered during drilling. The logs of the borings are presented on Figures A-6 through A-13 in Appendix A. The soil encountered in the borings was classified in accordance with the classification system presented in Figure A-13.

Soil samples were obtained using the following samplers:

- Modified California (MC) 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; the sampler was designed to accommodate liners, but liners were not used.

The samplers were driven with a 140-pound downhole 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 6 inches and are presented on the boring logs. A “blow count” is defined as the number of hammer blows per 6 inches of penetration or 50 blows for 6 inches or less of penetration. The blow counts required to drive the MC and SPT samplers were converted to approximate SPT N-values using factors of 0.63 and 1.08, 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: (1) the last two blow counts if the sampler was driven more than 12 inches, (2) the last one blow count if the sampler was driven more than 6 inches but less than 12 inches, and (3) the only blow count if the sampler was driven 6 inches or less. The converted SPT N-values are presented on the boring logs.

Upon completion of drilling, the boreholes were backfilled with neat cement grout in accordance with SCVWD requirements and patched with concrete. The soil cuttings generated by the borings were placed in 55-gallon drums and temporarily stored on-site. A sample of the drum cuttings was submitted for analytical testing, and the drums were subsequently disposed of at an appropriate landfill facility.

3.3 Laboratory Testing

We re-examined each soil sample from our borings to confirm the field classification and selected representative samples for laboratory testing. Laboratory tests were performed by ISI Inspection Services, Inc. of Berkeley, California or B. Hillebrandt Soils Testing, Inc. of Alamo, California to measure moisture content, dry density, plasticity (Atterberg limits), and fines content. Laboratory tests were performed by Construction Materials Testing, Inc. of Livermore, California to measure the resistance (R-value) of the near-surface soil. Laboratory tests were performed by Project X Corrosion Engineering of Murrieta, California on two near-surface soil samples to provide data for evaluating the soil corrosivity. The results of the laboratory tests are presented on the boring logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS

Regional geologic information (Figure 3) indicates the site is underlain by Pleistocene-age alluvial deposits (Qpa). Alluvial deposits generally consist of a mixture of fine-grained and coarse-grained deposits and are deposited by rivers and streams. The results of our borings and CPTs indicate the site is underlain by alluvium that extends to the maximum depth explored of 100 feet bgs. The upper approximately 13 to 20 feet of the alluvium consists of medium stiff to very stiff clay with variable sand and gravel content and loose to medium dense clayey sand. Atterberg limits tests indicate the near-surface clay is moderately expansive¹ with plasticity indices (PIs) of 17 to 21.

Below depths of approximately 13 to 20 feet bgs, the alluvium is generally stronger and highly variable, and consists of interbedded layers of very stiff to hard clay with variable sand content and dense to very dense sand with variable clay content that extend to the maximum depth explored of 100 feet bgs.

¹ Expansive soil undergoes volume changes with changes in moisture content (i.e., swells when wetted and shrinks when dried).

4.1 Groundwater

Groundwater was measured in the CPTs between 9 and 19 feet bgs using a weighted tape prior to grouting. Groundwater was encountered in Borings B-1, B-2, B-4, B-5, and B-8, and was measured between 13-1/2 and 18-1/2 feet bgs. Groundwater was not encountered in the other borings. It should be noted the groundwater level was likely not given adequate time to stabilize in the boreholes and CPTs at the time the measurements were taken.

To further evaluate the groundwater level at the site, we reviewed information on the State of California Water Resources Control Board GeoTracker website. From the GeoTracker website, we obtained information from monitoring wells installed for the former Shell service station at 3972 El Camino Real, which was at the northeastern border of the mobile home park along El Camino Real. A summary of groundwater level measurements presented in the *Groundwater Monitoring Results – First Quarter 2004* prepared by RRM, Inc. indicates the groundwater level was measured periodically between June 1988 and February 2004. Measured groundwater levels ranged from 10.6 to 25.8 feet bgs.

The groundwater level at the site is expected to vary several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall. Based on our review of available historic groundwater information within the site vicinity, we estimate the historic high groundwater level at the site is about 10 feet bgs.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity

The site is located within the Coast Ranges Geomorphic Province of California, which 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 and North American plates and subsequent strike-slip faulting along the San Andreas Fault system. The San Andreas Fault is more than 600 miles long and extends from Point Arena in the north to the Gulf of California in the south. The Coast Ranges Geomorphic 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 Monte Vista - Shannon faults. These and other faults in the region are shown in Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude² [Petersen et al. (2014) & Thompson et al. (2016)] are summarized in Table 1. These references are based on the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013).

² Moment magnitude (M_w) 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.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Characteristic Moment Magnitude
Monte Vista - Shannon	4.1	Southwest	7.14
Total North San Andreas (SAO+SAN+SAP+SAS)	9.3	Southwest	8.04
North San Andreas (Peninsula, SAP)	9.3	Southwest	7.38
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	20	East	7.58
Hayward (South, HS)	20	East	7.00
Butano	20	Southwest	6.93
San Gregorio (North)	26	West	7.44
Total Calaveras (CN+CC+CS+CE)	28	East	7.43
Calaveras (North, CN)	28	East	6.86
Calaveras (Central, CC)	28	East	6.85
North San Andreas (Santa Cruz Mts, SAS)	29	Southeast	7.15
Zayante-Vergeles (2011 CFM)	29	Southwest	7.48
Hayward (Extension, HE)	29	East	6.18
Sargent	35	Southeast	6.71
Las Positas	35	Northeast	6.50
Zayante-Vergeles	39	Southeast	7.00
Hayward (North, HN)	41	North	6.90
Mount Diablo Thrust South	44	Northeast	6.50
Mount Diablo Thrust	44	Northeast	6.67
Mount Diablo Thrust North CFM	44	Northeast	6.72
Reliz	49	South	7.44

Damaging earthquakes have occurred along many of these faults in recorded history, as depicted on Figure 4 (USGS, 2021). Notable historic earthquakes which have impacted the Bay Area in recorded history include:

- 1838 San Andreas Earthquake, $M_w = 7.4$ (estimated)
- 1865 San Andreas Earthquake, $M_w = 6.5$ (estimated)
- 1868 Hayward Earthquake, $M_w = 7.0$ (estimated)
- 1906 Great San Francisco Earthquake (San Andreas Fault), $M_w = 7.9$ (estimated)
- 1989 Loma Prieta Earthquake (San Andreas Fault), $M_w = 6.9$

- 2014 West Napa Earthquake, $M_w = 6.0$

As a part of the UCERF3 project, researchers estimated that the probability of at least one $M_w \geq 6.7$ earthquake occurring in the greater San Francisco Bay Area during a 30-year period (starting in 2014) is 72 percent. The highest probabilities are assigned to sections of the Hayward (South), Calaveras (Central), and San Andreas (Santa Cruz Mountains) faults. The respective probabilities are approximately 25, 21, and 17 percent.

5.2 Geologic Hazards

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, 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.

5.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the Monte Vista-Shannon, San Andreas, and Hayward faults, although ground shaking from future earthquakes on other faults will also be felt at the site. The ground shaking intensity felt at the project site will 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 very strong ground shaking could occur at the site during a large earthquake on one of the nearby faults.

5.2.2 Ground Surface Fault 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

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.

Fault Zoning Act, and no known active or potentially active faults exist on the site. We therefore conclude there is no risk of fault offset at the site from a known active fault. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is very low.

5.2.3 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.

As shown on Figure 5, the site has been mapped adjacent to a zone of liquefaction potential on the map titled *Earthquake Zones of Required Investigation, Palo Alto Quadrangle, Official Map*, prepared by the California Geological Survey (CGS), dated October 18, 2006. CGS has provided recommendations for procedures and report content for site investigations performed within seismic hazard zones in Special Publication 117 (SP-117), titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California*, dated September 11, 2008. SP-117 recommends that subsurface investigations in mapped liquefaction hazard zones be performed using rotary-wash borings and/or cone penetration tests. We evaluated liquefaction potential at the site using the data collected in our CPTs.

Our liquefaction analyses were performed using the software CLiq v3.4.1.4 (GeoLogismiki, 2022). CLiq uses measured field CPT data and assesses liquefaction potential 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

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

developed by Boulanger and Idriss (2014). Our analyses were performed using in-situ groundwater depths as measured at the termination of each CPT and a “during earthquake” groundwater depth of 10 feet bgs. In accordance with the 2022 CBC, we used a peak ground acceleration of 0.82 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PG_{AM}). We also used a moment magnitude 8.04 earthquake, which is consistent with the characteristic moment magnitude for the Total San Andreas Fault, as presented in Table 1.

Our liquefaction analyses indicate there are thin layers of potentially liquefiable soil between depths of approximately 10 and 42 feet bgs. The potentially liquefiable layers beneath the site are generally less than 3 feet thick. We estimate total ground-surface settlement resulting from post-earthquake reconsolidation following an MCE event with PG_{AM} of 0.82g will be will range from about 1/4 to 3/4 inch. Due to the variability of the thickness and lateral extent of the potentially liquefiable soil layers across the site, we estimate differential settlement resulting from post-liquefaction reconsolidation could be up to 1/2 inch across a horizontal distance of 30 feet.

Our analysis and laboratory test results indicate the potentially liquefiable layers are sufficiently thin and/or have a sufficient amount of plastic fines such that the potential for surface manifestations from liquefaction, such as sand boils, and loss of bearing capacity for shallow foundations is low.

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as a shoreline slope, or in the direction of a regional slope or gradient. Based on the lack of controlling boundary conditions and the discontinuous nature and thickness of the potential liquefiable soil layers, we conclude the potential for lateral spreading to occur at the project site is nil.

5.2.4 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. We judge that the loose to medium dense sand and clayey sand above the groundwater table encountered in the borings in the eastern portion of the site is susceptible to cyclic densification. We evaluated the cyclic densification potential of soil encountered at the site using data collected from our borings using the methodology developed by Pradel (1998).

The upper 10 feet of soil encountered beneath the proposed apartment building footprint in the southern one-third of the site consists mostly clay that is not susceptible to cyclic densification because of its cohesion. Layers of loose to medium dense clayey sand were encountered in most of the borings drilled in the northern two-thirds of the site. Although the clay content should limit the potential for cyclic densification of the clayey sand layers, we judge it would be prudent to assume up to 1 inch of total settlement due to cyclic densification could occur during the MCE event described in the preceding section. Due to the variability of the upper 10 feet of soil, we judge differential settlement could be up to 1 inch over a horizontal distance of 30 feet.

6.0 DISCUSSION AND CONCLUSIONS

Based on the results of our engineering analyses using the subsurface data collected from our investigation, we conclude the site may be redeveloped as proposed, provided the geotechnical issues discussed in this report are properly addressed. The primary geotechnical issues to be addressed are: 1) the potential for seismically induced settlement due to post-liquefaction reconsolidation and cyclic densification following a major earthquake, and 2) providing adequate foundation support for the proposed structures. These issues are discussed in more detail below.

6.1 Foundation Support and Settlement

The selection of a suitable foundation system for the proposed apartment building is governed by the estimated differential settlement under a combination of static and seismic loading and the

presence of relatively weak soil above a depth of 4 feet bgs. For our settlement analysis of the proposed apartment building, we assumed the building will impose an average pressure of 450 pounds per square foot (psf) over the building footprint under dead plus sustained live load conditions. For this loading, we estimate total and differential settlement of the proposed apartment building supported on a shallow foundation system will be approximately 1-1/2 inches and 3/4 inch over a horizontal distance of 30 feet, respectively. As discussed in Section 5.2.2, we estimate post-liquefaction reconsolidation following a major earthquake could result in additional differential settlement of up to 1/2 inch over a horizontal distance of 30 feet.

To reduce differential settlement of a shallow foundation system and increase the allowable bearing capacity of the near-surface soil, we conclude the upper 3 feet of soil should be overexcavated and recompacted. Although we estimate the overexcavation and recompaction would reduce differential settlement under static conditions to less than 1/2 inch over a horizontal distance of 30 horizontal feet, the combined static plus seismically induced differential settlement of 1 inch would still be greater than is typically acceptable for a conventional spread footing system. Further, considering the presence of a potentially liquefiable soil layer at a depth of 10 feet bgs, we believe a stiffer foundation system, such as a reinforced concrete mat or a post-tensioned slab-on-grade (P-T Slab), would perform better during a major earthquake. Therefore, we conclude the proposed apartment building should be supported on a mat foundation or P-T slab.

The portion of the site that will be occupied by mobile homes is underlain by medium stiff to stiff clay with varying amounts of sand and loose to medium dense clayey sand that extend to depths of 15 to 20 feet bgs. This upper soil is relatively weak and compressible; however, the loads imposed on the soil by mobile homes are generally light. Provided low design bearing pressures are used, we conclude the mobile homes can be supported by jacks bottomed Portland cement concrete (PCC) pavement. We estimate total and differential settlements of shallow foundations would be less than 1/2 inch and 1/4 inch over a horizontal distance of 30 feet, respectively. Because shallow foundations supporting the mobile homes may experience up to 1-1/2 inches of differential settlement over a horizontal distance of 30 feet due to a combination of

cyclic densification and post-liquefaction reconsolidation during a major earthquake, it may be necessary to relevel the homes after the seismic event.

6.2 Construction Considerations

The soil to be excavated generally consists of clay with varying amounts of sand which can be excavated with conventional earth-moving equipment such as loaders and backhoes. If site grading is performed during the rainy season, the near-surface clay will likely be wet and will have to be dried before compaction can be achieved. Heavy rubber-tired equipment, such as scrapers and vibratory rollers, could cause excessive deflection (pumping) of the wet clay and, therefore, should be avoided. If the project schedule or weather conditions do not permit sufficient time for drying of the soil by aeration, the subgrade can be treated with lime prior to compaction or imported granular fill can be used. The appropriate amount of lime should be determined during construction based on a visual examination and, if necessary, laboratory testing of the soil to be treated. It is also important that the moisture content of the subgrade soil is sufficiently high to reduce the expansion potential. If the grading work is performed during the dry season, moisture-conditioning may be required.

Excavations that will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes.

6.3 Soil Corrosivity

Corrosivity tests were performed by Project X Corrosion Engineering of Murrieta, California on soil samples obtained from Borings B-1 and B-4 at 2-1/2 and 1-1/2 feet bgs, respectively. The corrosivity test results are presented in Appendix B.

Many factors can affect the corrosion potential of soil including, but not limited to, resistivity, pH, and chloride and sulfate concentrations. Based on the minimum soil resistivity measurements of 1,742 and 1,474 ohm-cm, we conclude the soil is “highly corrosive” to buried metal (Roberge, 2018). Accordingly, all buried iron, steel, cast 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 results of the pH tests (7.4) indicate the near-surface soil is “negligibly corrosive” to buried metallic and concrete structures. The chloride ion concentrations (11.2 and 24.3 mg/kg) indicate the chlorides in the near-surface soil are “negligibly corrosive” to buried metallic structures and reinforcing steel in concrete structures below ground. The results also indicate the sulfate ion concentrations (226.9 and 668.4 mg/kg) are sufficiently low such that sulfates do not pose a threat to buried concrete and mortars.

7.0 RECOMMENDATIONS

Recommendations for site grading, foundation design, ground improvement, and seismic design are presented in this section of the report.

7.1 Site Preparation and Grading

Site demolition should include the removal of all existing 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/or will not interfere with the proposed construction, they may be abandoned in place provided the lines are filled with lean concrete or cement grout to the property line. 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. If zones of existing undocumented fill or weak/unstable soil are encountered during site grading, the fill should be overexcavated under the observation and replaced as properly compacted fill.

If grading work is performed during the rainy season, the contractor may find the subgrade material too wet to compact to the recommended relative compaction and will have to be scarified and aerated to lower its moisture content so the specified compaction can be achieved.

Material to be dried by aeration should be scarified to a depth of at least 8 inches; the scarified soil should be turned at least twice a day to promote uniform drying. Once the moisture content of the aerated soil has been reduced to an acceptable level, the soil should be compacted in accordance with our recommendations. Aeration typically is the least costly method used to stabilize the subgrade soil; however, it generally takes the most time to complete. Other soil stabilization alternatives include overexcavating the wet soil and replacing it or mixing it with drier soil, and lime treatment.

7.1.1 Subgrade Preparation

In areas that will receive fill or improvements (i.e., building pad subgrade, pavement, or flatwork), the soil subgrade exposed should be scarified to a depth of at least 8 inches and moisture-conditioned and compacted to the recommendations presented in Table 2 (Section 7.1.2). The soil subgrade should be kept moist until it is covered by fill or improvements. Additional subgrade preparation recommendations for the apartment building pad and exterior concrete flatwork are presented in the following paragraphs.

Apartment Building Pad

After site clearing is completed, the building pad for the proposed apartment building should be overexcavated to a depth of 3 feet below existing site grades. The excavation should extend at least 5 feet beyond the perimeter of the proposed building except where constrained by property lines or existing utility lines. The excavation subgrade should be scarified to a depth of at least 8 inches, moisture-conditioned, and compacted in accordance with the recommendations presented in Table 2. The excavated material should then be placed in lifts not exceeding 8 inches in loose thickness, moisture-conditioned, and compacted in accordance with the recommendations presented in Table 2.

Exterior Concrete Flatwork

We recommend a minimum of 6 inches of Class 2 aggregate base (AB) be placed beneath proposed exterior concrete flatwork. The AB should extend at least 6 inches beyond the slab edges where the flatwork is adjacent to landscaping. Class 2 AB placed beneath exterior slabs-

on-grade, such as patios and sidewalks, should be moisture-conditioned and compacted in accordance with the recommendations presented in Table 2.

Even with 6 inches of AB, exterior slabs may experience some cracking due to shrinking and swelling of the underlying moderately expansive soil. Thickening the slab edges and adding additional reinforcement will control this cracking to some degree. Where slabs are adjacent to landscaped areas, thickening the concrete edge will help control water infiltration beneath the slabs. In addition, where slabs provide access to the proposed building, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries.

7.1.2 Fill Quality and Compaction

Fill should consist of on-site soil or imported soil (select fill) that is free of organic matter, contain no rocks or lumps larger than 3 inches in greatest dimension, and be approved by the Geotechnical Engineer. Imported soil (select fill) should also have a liquid limit of less than 40 and a plasticity index lower than 12. Samples of proposed imported fill 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.

Fill should be placed in horizontal lifts not exceeding 8 inches in uncompacted thickness, moisture-conditioned, and compacted according to the recommendations presented in Table 2.

TABLE 2
Summary of Compaction Recommendations

Location	Relative Compaction (percent)	Moisture Content
Building pad – expansive clay	90+	2+% above optimum
Building pad – low-plasticity soil	95+	Above optimum
Exterior slabs – expansive clay	90+	2+% above optimum
Exterior slabs – low-plasticity soil	90+	Above optimum
Pavements – expansive clay	92+	Above optimum
Pavements – low-plasticity soil	95+	Above optimum
Pavements - aggregate base	95+	Near optimum
General fill – expansive clay	90+	2+% above optimum
General fill – low-plasticity soil	90+	Above optimum
General fill – clean sand or gravel	95+	Near optimum
General fill – low-plasticity more than 5 feet thick	95+	Above optimum

Notes: 1) Select fill is considered low-plasticity.

2) Backfill for utility trenches is considered fill and should be compacted following recommendations presented in Table 2.

Where the above-recommended compaction requirements are in conflict with the City of Palo Alto standard details for pavements and sidewalks within the public right-of-way, the City Engineer or inspector should determine which compaction requirements should take precedence.

7.1.3 Utility Trench Excavation and 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 4 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 6 inches

with sand or fine gravel, which should be mechanically tamped. The pipe bedding and cover should be eliminated where an impermeable plug is required as described below.

Backfill for utility trenches and other excavations is also considered fill, and it should be placed and compacted in accordance with the recommendations previously presented in Section 7.1.2. If imported clean sand or gravel (defined as soil with less than 5 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Pea gravel, drain rock, and rod mill should be mechanically tamped in 12-inch-thick lifts where placed beneath pavements. 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 pavement section.

Foundations for the proposed apartment building should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of utility trenches that are 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) with a 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).

Where utility trenches enter the building pad for the proposed apartment building, an impermeable plug consisting of CLSM, at least 3 feet in length, should be installed where the trenches enter the building footprint. Furthermore, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

7.1.4 Surface Drainage and Bioswales

Positive surface drainage should be provided around the building and mobile homes to direct surface water away from foundations. Grades around the buildings and mobile homes should be determined by the Civil Engineer and conform to the requirements of the 2022 CBC, which will help minimize stormwater accumulation adjacent to foundations. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. The use of water-intensive landscaping around the perimeter of the residential building should be avoided to reduce the amount of water introduced to the expansive clay subgrade.

Care should be taken to minimize the potential for subsurface water to collect beneath pavements and pedestrian walkways. Where landscape beds and tree wells are immediately adjacent to pavements and flatwork that are not designed as permeable systems, we recommend vertical cutoff barriers be incorporated into the design to prevent irrigation water from saturating the subgrade and AB. These barriers may consist of either flexible impermeable membranes or deepened concrete curbs.

Where bioswales will be part of the project, we recommend that bioswales be constructed at least 5 feet from the proposed apartment building and provided with underdrains and/or drain inlets. The subdrain pipes should be installed 8 inches above the bottom of the infiltration area for treatment areas that are at least 5 feet away from the buildings and pavements. The intent of this recommendation is to allow infiltration into the underlying soil, but to reduce the potential for bio-retention areas to flood during periods of heavy rainfall.

Where it is necessary for a bioswale to be constructed within 5 feet of the proposed building and pavements because of site constraints, the bottom of the bioswale should be lined with an impermeable liner. Where a vertical curb or foundation is constructed near a bioswale, the curb and the edge of the foundation should be founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal to vertical) from the base of the bioswale.

7.2 Foundation Design

The proposed multi-family apartment building should be supported on a conventionally reinforced mat foundation or a post-tensioned slab-on-grade (P-T slab) bearing on 3 feet of engineered fill as described in Section 7.1.1. The mobile homes should be supported on timber foundations bearing on asphalt concrete or Portland-cement concrete pavement.

Recommendations for design of a mat foundation for the apartment building and timber footings for the mobile homes are presented below. Recommendations for a P-T slab foundation for the apartment building can be provided upon request.

7.2.1 Mat Foundation – Apartment Building

The mat foundation subgrade should be prepared following the recommendations presented in Section 7.1.1. The edges of the mat should be thickened such that the mat edge is bottomed at least 9 inches below the adjacent soil subgrade. In addition, we recommend the mat be founded below an imaginary plane extending up at an inclination of 1.5:1 (horizontal to vertical) from the base of any vault, utility trench, bioswale/stormwater treatment area, etc. If the design bottom-of-mat elevation is above this plane, the edge of mat can either be deepened, or it can be over-excavated below the zone-of-influence line and replaced with CLSM with a 28-day unconfined compressive strength of at least 100 psi.

For structural design of the mat foundation, we recommend using a coefficient of vertical subgrade reaction of 25 pounds per cubic inch (pci) for dead-plus-live loads; this value has already been scaled to take into account the plan dimensions of the mat foundation (therefore, this is not k_{v1} for 1-foot-square plate) and may be increased by 50 percent for total load conditions. Once the Structural Engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of subgrade reaction, if appropriate.

We recommend the mat foundation be designed to limit bearing pressures to 3,000 pounds per square foot (psf) for dead-plus-live loads; this pressure may be increased by one-third for total

load conditions. The allowable bearing pressures for dead-plus-live and total loads include factors of safety of at least 2.0 and 1.5, respectively.

Lateral forces can be resisted by friction along the base of the mat and by passive pressure against the sides of the mat foundation. To compute lateral resistance, we recommend using an allowable uniform pressure of 2,000 psf (rectangular distribution) for transient load conditions and an equivalent fluid weight (triangular distribution) of 250 pounds per cubic foot (pcf) for sustained load conditions; 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 a vapor retarder is placed beneath the mat, a base friction coefficient of 0.20 should be used. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

The upper 3 feet of the mat subgrade should consist of engineered fill prepared following the recommendations presented in Section 7.1.1. We should check the mat subgrade prior to placing the vapor retarder or rebar to confirm it is free of standing water, debris, and disturbed materials.

Vapor Retarder

To reduce water vapor transmission through the mat foundation for the apartment building, we recommend a vapor retarder be placed between the bottom of the mat and the underlying subgrade soil. The vapor retarder should be at least 15 mils thick and meet the requirements for Class A vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by 6 inches, taping seams, and sealing penetrations in the vapor retarder.

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 mat foundation. Therefore, concrete for the mat should have a w/c ratio of less than 0.45. 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.

7.2.2 Foundations – Mobile Homes

We understand the mobile homes will be supported on metal jacks bearing on Portland cement concrete (PCC) pavement. We recommend the PCC pavement supporting the jacks consist of 5 inches of PCC over 6 inches of Class 2 AB. The upper 8 inches of the soil subgrade and the AB should be compacted to at least 95 percent relative compaction. The PCC should have a 28-day unconfined compressive strength of at least 3,000 psi. For evaluation of the pavement to support the jacks, we recommend using a modulus of vertical subgrade reaction (k_{v1}) of 150 pci and allowable bearing pressures of 1,500 psf for dead-plus-live loads and 2,000 psf for total loads.

7.3 Retaining Walls

Retaining walls should be designed to resist lateral earth pressure imposed by the retained soil, as well as surcharge pressure from nearby foundations and vehicular loading, where appropriate. For static conditions, we recommend restrained and unrestrained walls be designed for the following lateral earth pressures:

- Restrained Wall - At-rest earth pressure using an equivalent fluid weight of 63 pcf for drained conditions and 94 pcf for undrained conditions.
- Unrestrained Wall - Active earth pressure using an equivalent fluid weight of 42 pcf for drained conditions and 83 pcf for undrained conditions.

We anticipate that any walls at the site will retain less than 6 feet of soil, and therefore, do not need to be checked for seismic loading. Where traffic loads are expected within a horizontal distance equal to the height of the walls, a uniform vehicular surcharge pressure of 50 psf should be applied to the upper 10 feet of wall or the entire wall height, whichever is less. Below-grade walls adjacent to existing foundations should be designed for surcharge pressures if the foundations are founded above the zone-of-influence for the basement walls. This zone is defined as an imaginary line extending up from the bottom of the wall at an inclination of 1.5:1 (horizontal to vertical). The influence on a wall from a foundation that is founded within this

zone-of-influence should be analyzed on an individual basis after the geometry has been determined.

Although the permanent walls will likely be above the groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines. All retaining walls designed using the recommended “drained” earth pressures presented above should be constructed with a backdrain. One acceptable method for backdraining a retaining wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the wall. The pipe should be surrounded by at least 4 inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140NC or equivalent). The perforated pipe should be sloped to drain by gravity to a suitable outlet.

Retaining Wall Foundations

Site retaining walls may be supported on spread footings bottomed at least 18 inches below the lowest adjacent exterior finished grade. Spread footings may be designed using an allowable bearing pressure of 2,000 psf. We estimate total settlement of spread footings will be less than 3/4 inch and differential settlement will less than 1/2 inch over a horizontal distance of 30 feet.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute lateral resistance, we recommend using an equivalent fluid weight of 220 pcf; the upper foot of soil should be ignored for lateral resistance unless confined by pavement. This passive pressure value assumes level ground in front of the footing. Frictional resistance should be computed using a base friction coefficient of 0.30. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without further reduction.

Footing excavations should bottom on firm native soil or engineered fill and be free of standing water, debris, and weak or disturbed material prior to placing concrete. The bottoms and sides of the footing excavations should be maintained in a moist condition until concrete is placed. We

should check footing excavations prior to placement of reinforcing steel. Where fill or weak native soil is encountered at the bottom of a footing excavation, the excavation should be deepened to bottom on suitable bearing material, as determined by our field engineer. The portion of the footing excavation that extends below the design bottom-of-footing elevation may be filled with engineered fill, structural concrete or controlled low-strength material with a 28-day compression strength of at least 100 psi.

7.4 Pavement Design

Design recommendations for asphalt concrete and Portland-cement concrete pavements are presented in the following sections.

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. We performed a resistance value (R-value) test of a near-surface bulk sample from Boring B-2. The results indicated that the soil has an R-value of 7 (Figure B-3). Recommended pavement sections for traffic indices (TIs) ranging from 4.0 to 7.0 are presented in Table 3. The Civil Engineer for the project should check that the TIs presented are appropriate for the intended use. We can provide additional pavement sections for different TIs upon request.

TABLE 3
Asphalt Concrete Pavement Sections

TI	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.5	2.5	9.0
5.0	3.0	9.5
5.5	3.0	11.5
6.0	3.5	12.5
6.5	4.0	13.5
7.0	4.0	15.0

The upper 12 inches of the subgrade should be moisture-conditioned and compacted in accordance with the recommendations presented in Section 7.1.1 and should be non-yielding. The Class 2 AB should be moisture-conditioned to near optimum and compacted to at least 95 percent relative compaction and be non-yielding.

If pavements are adjacent to irrigated landscaped areas (including infiltration basins), curbs adjacent to those areas should extend through the aggregate base and at least 3 inches into the underlying soil to reduce the potential for irrigation water to infiltrate into the pavement section. If drip irrigation is used in the landscaping adjacent to the pavement, however, the deepened curb is not required.

7.4.2 Rigid (Portland-Cement Concrete) Pavement Design

The minimum thickness for concrete pavements should be based on the anticipated traffic loading, the modulus of rupture of the concrete used, and the supporting characteristics of the subgrade below the pavement section. Pavements should be designed and constructed in accordance with the American Concrete Institute (ACI) Commercial Concrete Parking Lots and Site Paving Design and Construction Guide (ACI PRC-330-21). The compressive strength of the concrete should be at least 3,750 psi with a modulus of rupture of the concrete of 550 psi at 28 days. Reinforcing steel may be used for shrinkage crack control. The recommended minimum rigid pavement section and maximum spacing between joints are presented in Table 4 below.

TABLE 4
Rigid Concrete Pavement Design

Traffic Categories	Maximum ADTT⁶	Concrete Thickness (inches)	Class 2 Aggregate Base Thickness (inches)	Maximum Spacing Between Joints (feet)
Car parking areas and Access Lanes (Category A)	10	5.5	6	12.5
Entrance and Truck Service Lanes (Category B)	25	6.0	6	15
Garbage or Fire Truck lane (Category E)	1	7.0	6	15

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. Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those described above for asphalt concrete pavement.

7.5 Seismic Design

The latitude and longitude of the site are 37.4149° and -122.1297°, respectively. For design in accordance with the 2022 CBC, we recommend the following:

- Site Class D (stiff soil, non-default)
- $S_s = 1.803g$, $S_1 = 0.638g$

The 2022 CBC is based on the guidelines contained within ASCE 7-16 (Supplement 3 revision), which stipulates that where S_1 is greater than 0.2 times gravity (g) for Site Class D, a ground motion hazard analysis is required unless the long-period spectral design parameters (S_{M1} , S_{D1}) are increased by 50%. Therefore, we recommend the following seismic design parameters, which include the 50% increase as designated by an asterisk:

⁶ ADDT is the Average Daily Truck Traffic in both directions (excludes panel trucks, pickup trucks, and other four-wheel vehicles).

- $F_a = 1.0, F_v = 1.7$
- $S_{MS} = 1.803g, S_{M1}^* = 1.627g$
- $S_{DS} = 1.202g, S_{D1}^* = 1.085g$
- Seismic Design Category D for Risk Factors I, II, and III

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Prior to construction, we should review the project plans and specifications to check that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during placement and compaction of fill, grading, and installation of foundations. These observations will allow us to compare actual with anticipated soil conditions and to check 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 field investigation. 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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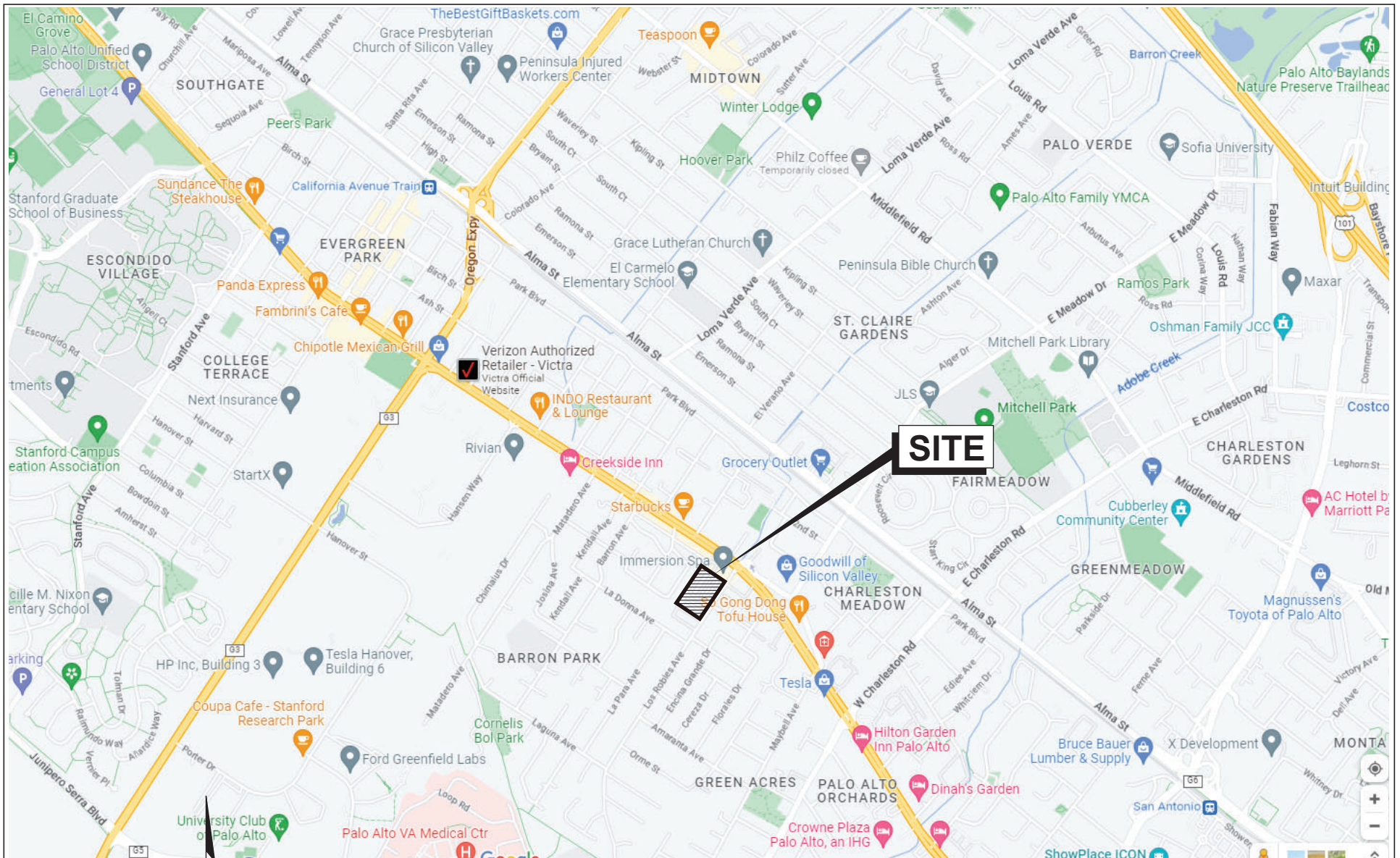
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FIGURES



Base map: Google Maps, 2023

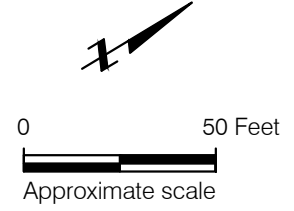
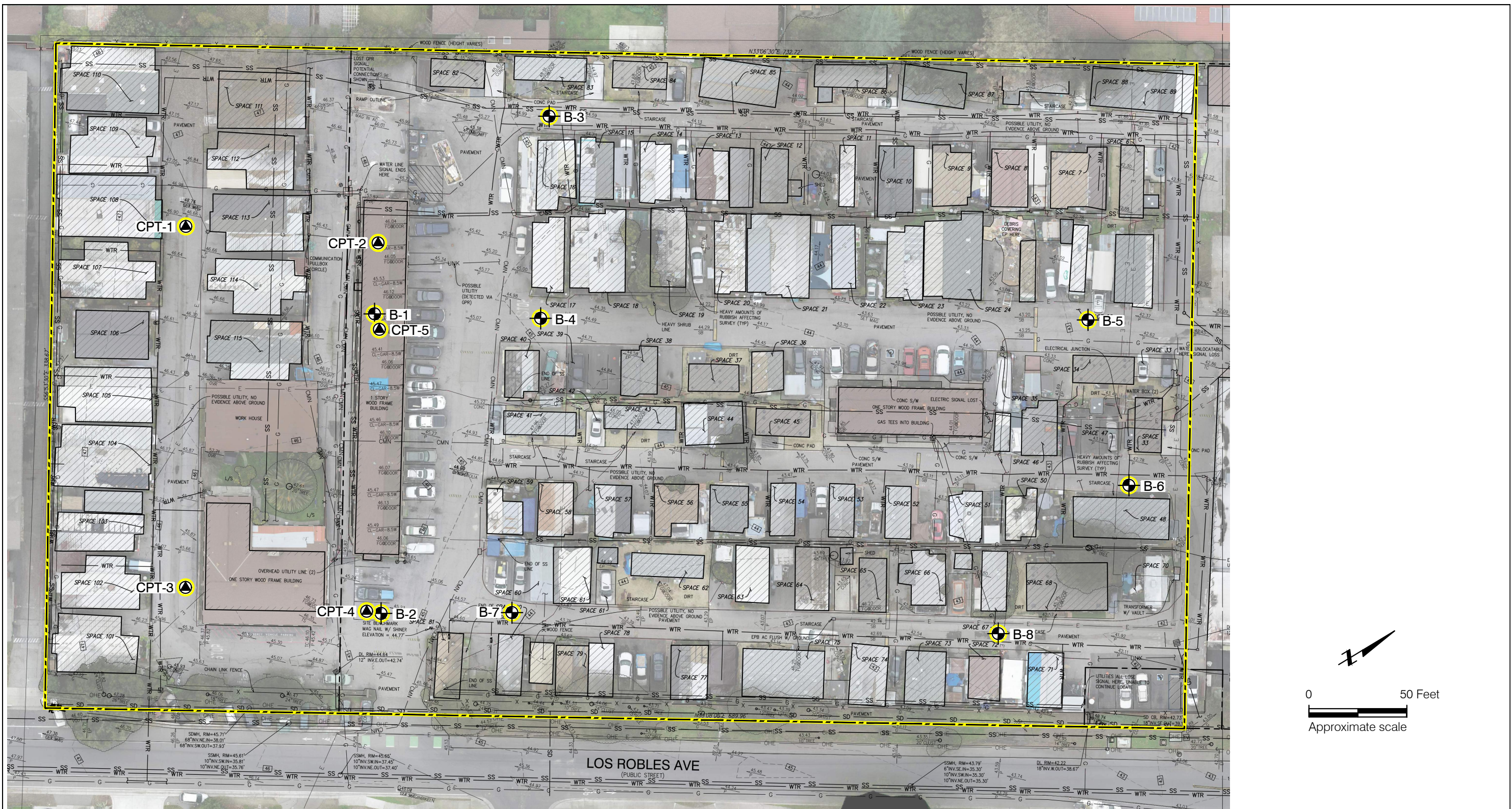
0 1,800 3,600 Feet
Approximate scale

**BUENA VISTA MOBILE
HOME PARK REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California**

**ROCKRIDGE
GEOTECHNICAL**

SITE LOCATION MAP

Date 04/10/23	Project No. 23-2398	Figure 1
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EXPLANATION

- CPT-1  Approximate location of cone penetration test by Rockridge Geotechnical, Inc., April 11, 2023
- B-1  Approximate location of boring by Rockridge Geotechnical, Inc., April 1 & 8, 2023
-  Project limits

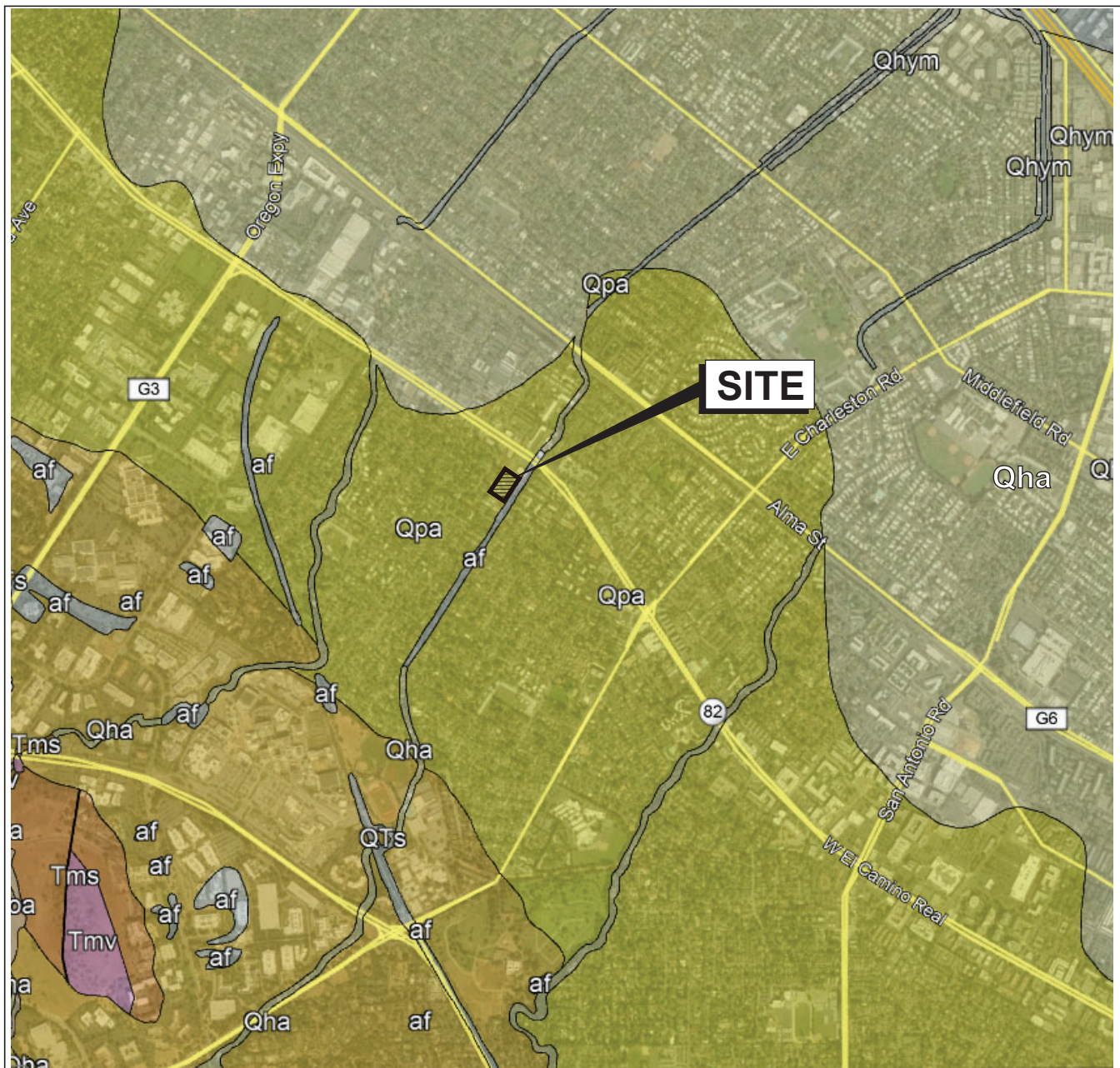
Reference: Base map from a drawing titled "Topographic Survey", by Sandis, dated February 15, 2022.

**BUENA VISTA MOBILE
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Palo Alto, California**

SITE PLAN

Date 04/14/23 Project No. 23-2398 Figure 2



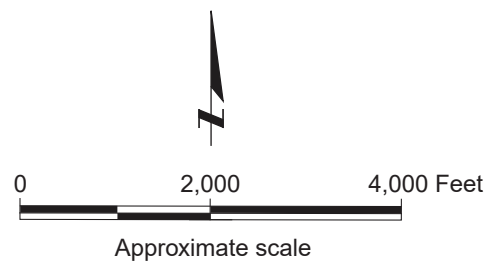


Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2021.

EXPLANATION

af	Artificial Fill
Qhym	Mud deposits (late Holocene)
Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)
QTs	Sediments (early Pleistocene and (or) Pliocene)
Tms	Sedimentary rocks (Miocene)
Tmv	Volcanic rocks (Miocene)

Geologic contact:
dashed where approximate and dotted
where concealed, queried where uncertain



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3980 EL CAMINO REAL
Palo Alto, California**

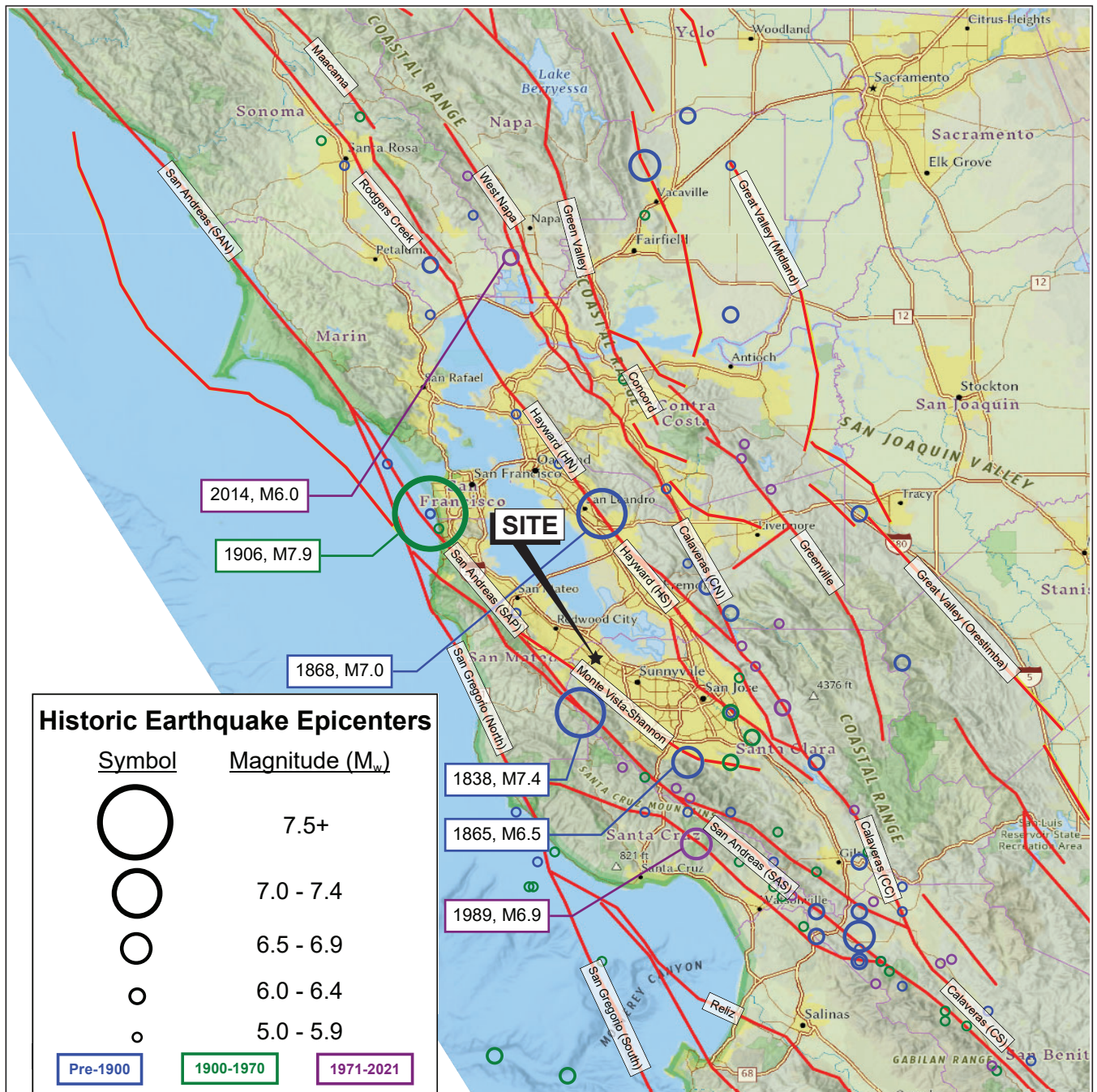
**ROCKRIDGE
GEOTECHNICAL**

REGIONAL GEOLOGIC MAP

Date 04/10/23

Project No. 23-2398

Figure 3



EXPLANATION

— Faults (National Seismic Hazard Model)



0 10 20 Miles
0 16 32 Kilometers
Approximate scale

**BUENA VISTA MOBILE
HOME PARK REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California**

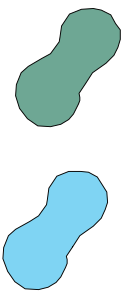
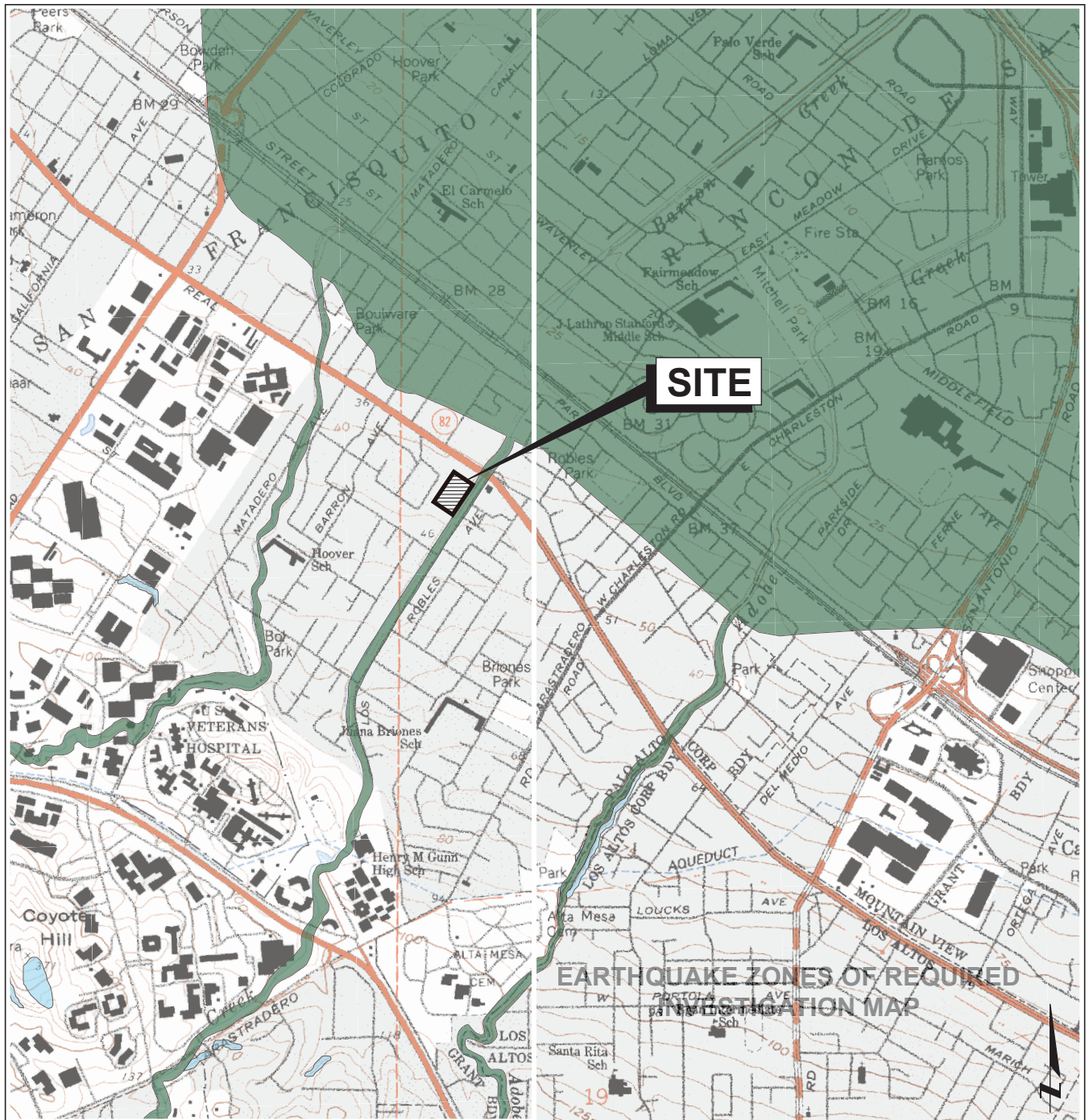
**ROCKRIDGE
GEOTECHNICAL**

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Date 04/10/23

Project No. 23-2398

Figure 4



Liquefaction Zones

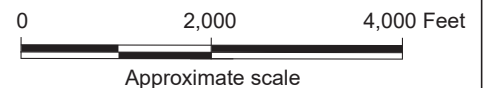
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones

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.

Reference:

Earthquake Zones of Required Investigation
California Geological Survey
Palo Alto Quadrangle
Mountain View Quadrangle
Released October 18, 2006



**BUENA VISTA MOBILE
HOME PARK REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California**

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**EARTHQUAKE ZONES OF REQUIRED
INVESTIGATION MAP**

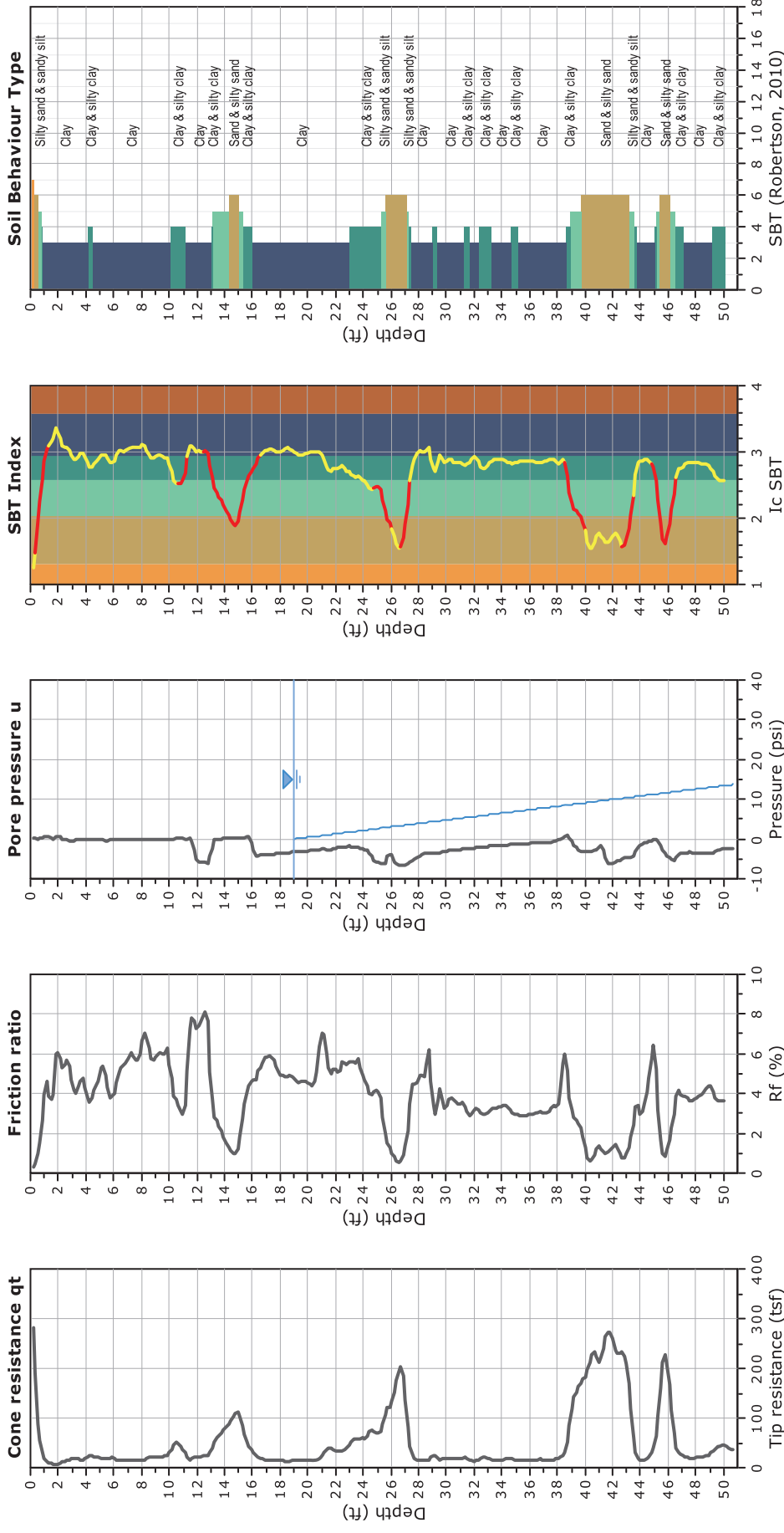
Date 04/10/23

Project No. 23-2398

Figure 5

APPENDIX A

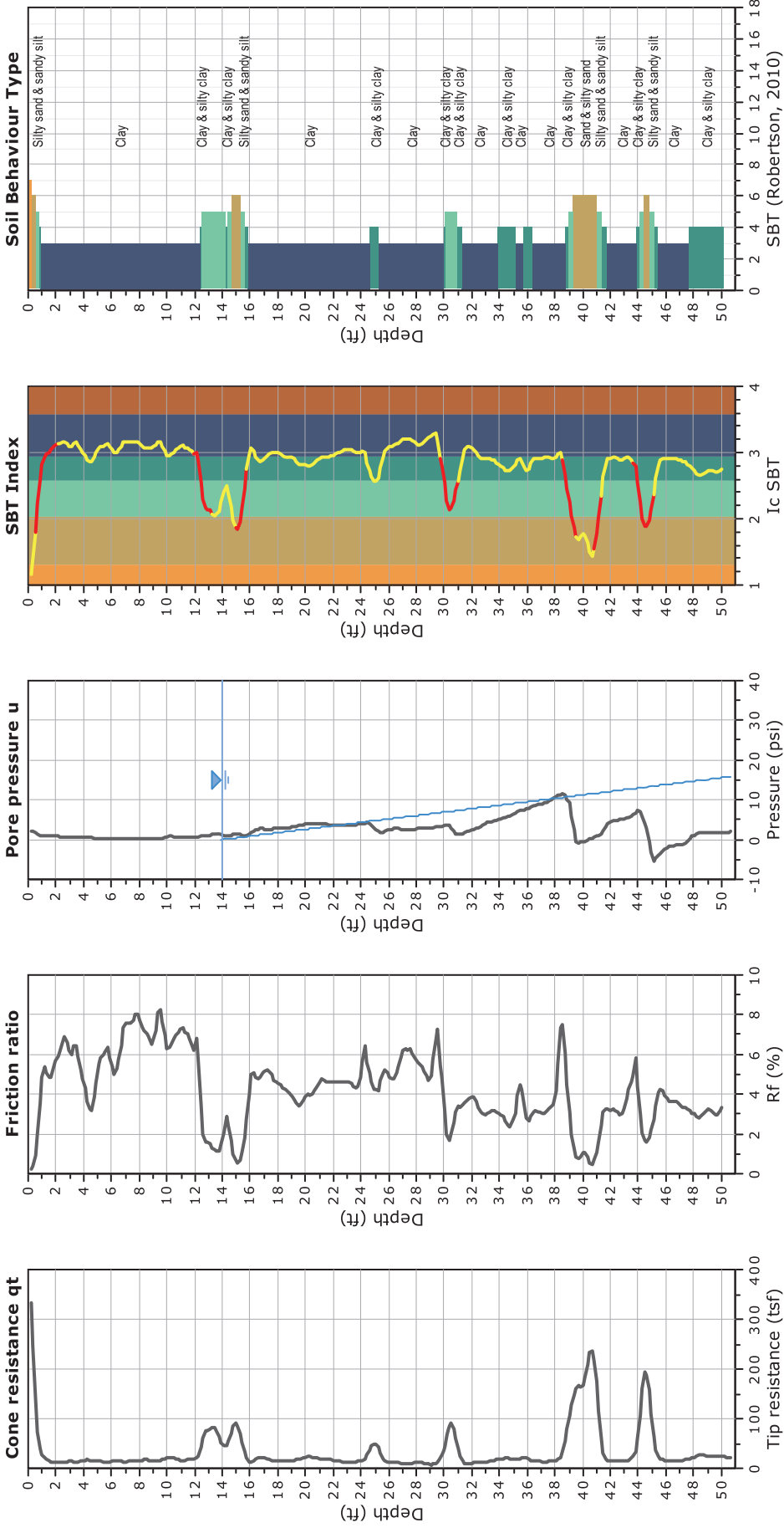
Cone Penetration Test Results and Boring Logs



Total Depth: 50.7 ft, Date: April 11, 2023
Depth to Groundwater: 19 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

BUENA VISTA MOBILE
HOME PARK REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California
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CONE PENETRATION TEST RESULTS
CPT-1



Total Depth: 50.7 ft, Date: April 11, 2023
Depth to Groundwater: 14 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

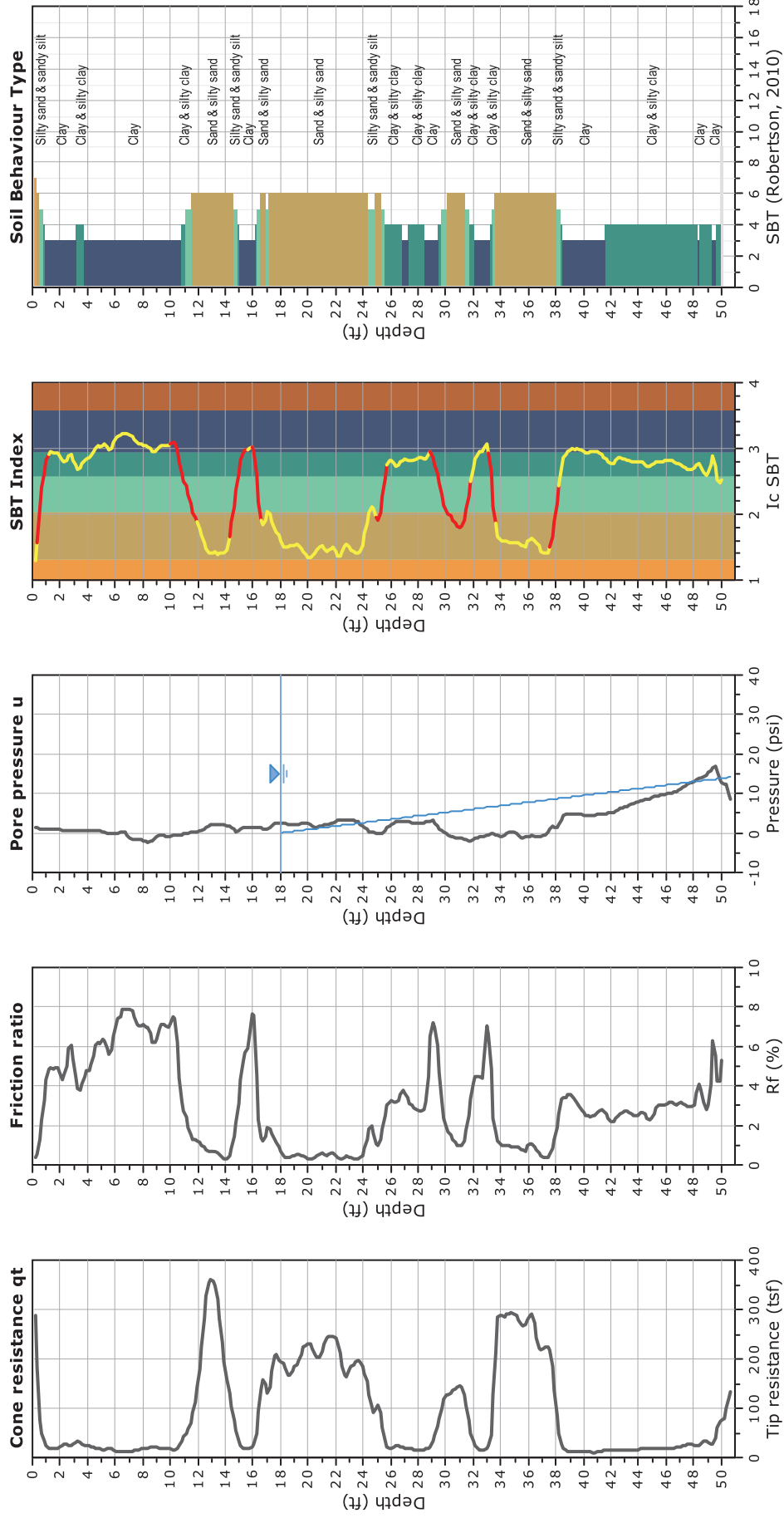
BUENA VISTA MOBILE
HOME PARK REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California

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CONE PENETRATION TEST RESULTS

CPT-2

Date 04/21/23 Project No. 23-2398 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. Gravelly sand to sand
 - 8. Very stiff sand to clayey sand
 - 9. Very stiff fine grained

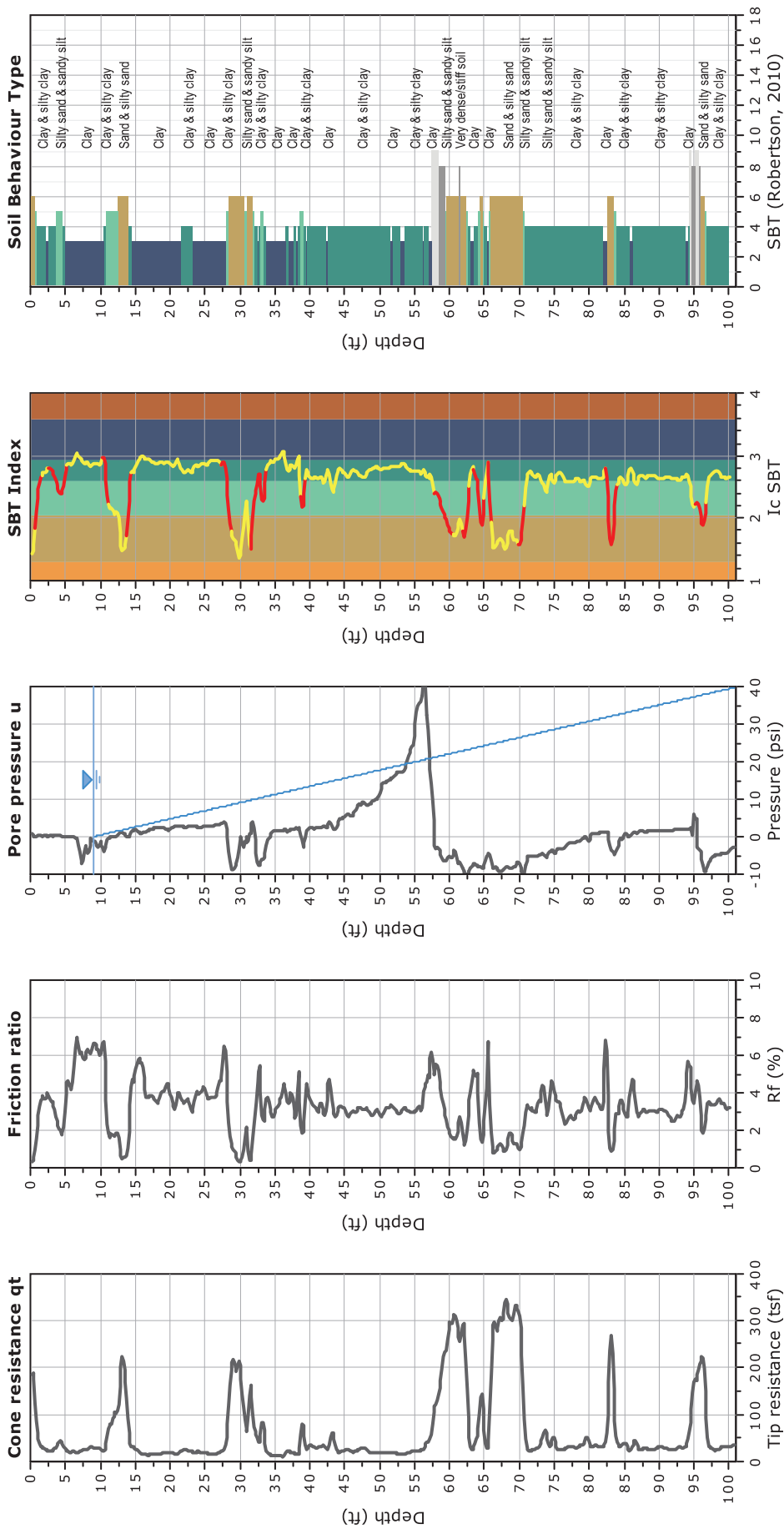
Total Depth: 50.7 ft, Date: April 11, 2023
Depth to Groundwater: 18 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

BUENA VISTA MOBILE
HOME PARK REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California

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CONE PENETRATION TEST RESULTS

CPT-3



- 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. Gravelly sand to sand
 - 8. Very stiff sand to clayey sand
 - 9. Very stiff fine grained

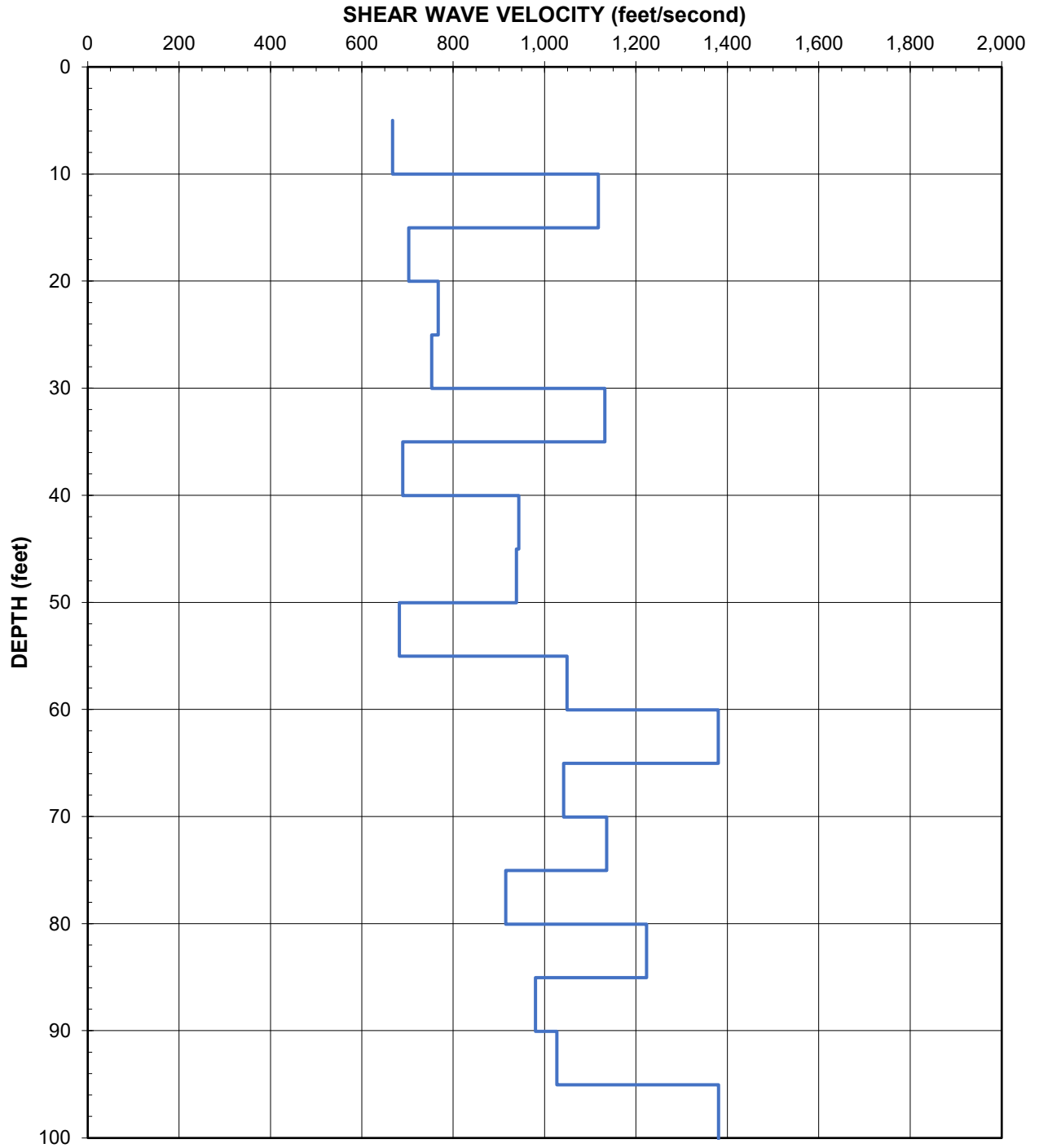
Total Depth: 100.7 ft, Date: April 11, 2023
Depth to Groundwater: 9 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

BUENA VISTA MOBILE
HOME PARK REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California

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CONE PENETRATION TEST RESULTS

CPT-4



**SHEAR WAVE VELOCITY PROFILE
CPT-4**

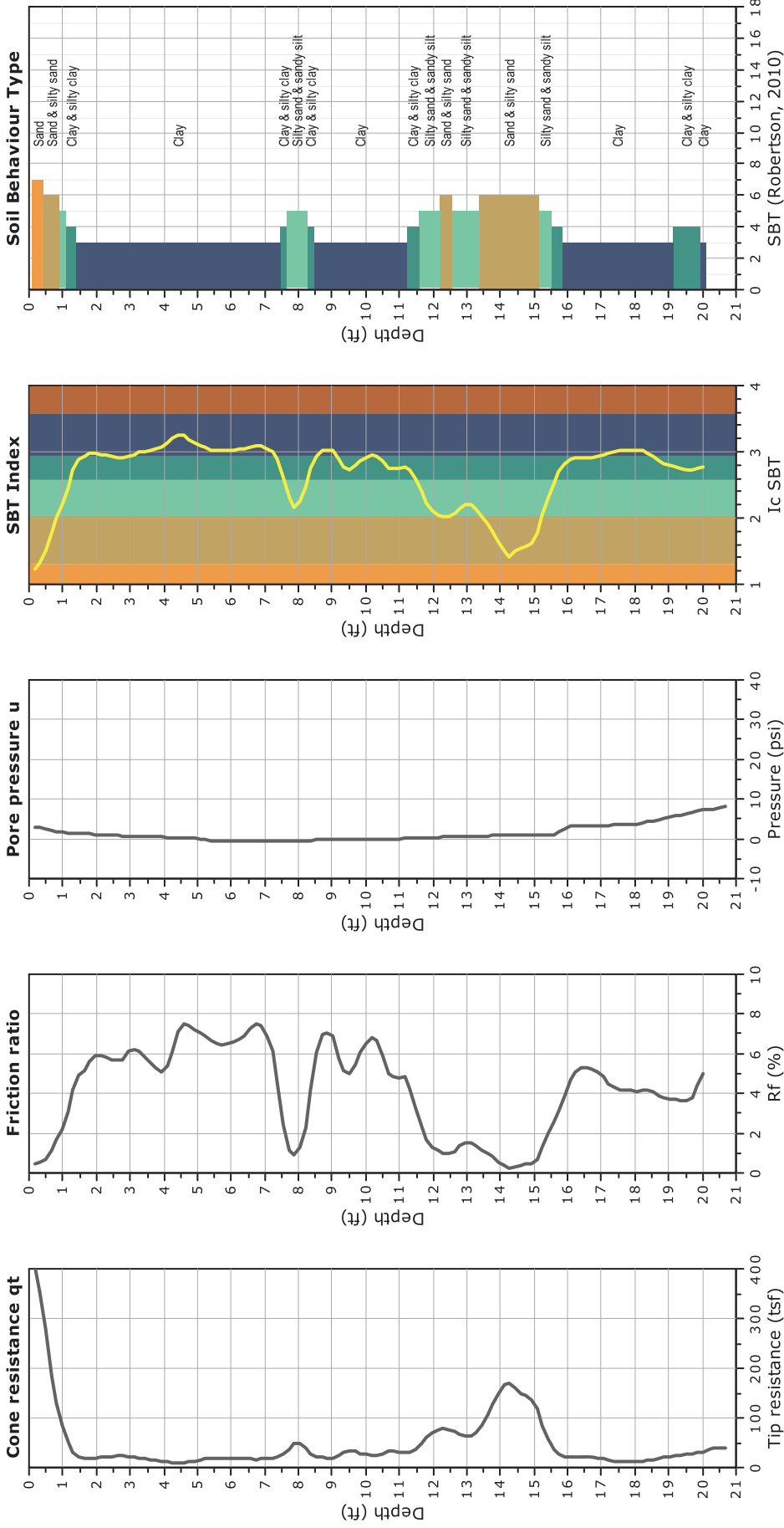
BUENA VISTA MOBILE HOME PARK REDEVELOPMENT
Palo Alto, California

Date: 5/10/2023

Project No. 23-2398

Figure A-4b

 **ROCKRIDGE
GEOTECHNICAL**



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. Gravelly sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

Total Depth: 20.7 ft, Date: April 11, 2023
Groundwater not measured
Cone Operator: Middle Earth Geo Testing, Inc.

BUENA VISTA MOBILE
HOME PARK REDEVELOPMENT
3980 EL CAMINO REAL
Palo Alto, California

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CONE PENETRATION TEST RESULTS
CPT-5

Date 04/21/23

Project No. 23-2398

Figure A-5

PROJECT:		BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California				Log of Boring B-1 PAGE 1 OF 2						
Boring location: See Site Plan, Figure 2						Logged by: J. Lei Drilled by: Exploration Geoservices, Inc. Rig: Mobile B-53R						
Date started: 04/01/2023		Date finished: 04/01/2023										
Drilling method: 8-inch-diameter hollow-stem auger												
Hammer weight/drop: 140 lbs./30 inches		Hammer type: Downhole Safety Hammer										
Sampler: Modified California (MC), Standard Penetration Test (SPT)												
DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cut Ft
1						8 inches of asphalt concrete						
2						SANDY CLAY (CL) brown to dark brown, stiff, moist, fine to medium sand						
3	MC		10	11		Soil Corrosivity Test; see Appendix B LL = 39, PI = 21; see Appendix B				14.2	121	
4			9									
5	MC		7	11		brown, fine sand, trace fine rounded gravel LL = 36, PI = 19; see Appendix B				15.5	115	
6			8									
7	MC		9	13		brown to dark brown, fine to medium sand, no gravel						
8			9									
9			12									
10	MC		11	24		brown grades to dark brown, very stiff, trace fine gravel						
11			17									
12			21									
13												
14	MC		23	31	SP-SC	SAND with CLAY (SP-SC) gray, dense, moist, medium to coarse sand, trace fine gravel						
15			24									
16			25		SC	CLAYEY SAND (SC) gray, dense, moist, fine to coarse sand, trace fine subangular gravel						
17												
18						CLAY with SAND (CL) brown, very stiff, wet, fine sand, trace fine gravel (04/01/2023; 1:47 PM)						
19	MC		10	23								
20			17									
21			20									
22												
23												
24	MC		11	21	CL	decreasing sand content with depth, no gravel						
25			14									
26			19									
27												
28												
29	MC		14	32		hard, increase in sand content						
30			24		SC	CLAYEY SAND (SC)						
			27									


**ROCKRIDGE
GEOTECHNICAL**

Project No.: 23-2398	Figure: A-6a
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PROJECT: BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California						Log of Boring B-1 PAGE 2 OF 2						
DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31					SC	CLAYEY SAND (SC) (continued) brown, dense, wet, fine sand						
32												
33						CLAY with SAND (CL) light brown, very stiff to hard, wet						
34	SPT		12	30								
35			12		CL							
36			16									
37												
38						SANDY CLAY (CL) brown, hard, wet						
39	SPT		10	38	CL							
40			13									
41			22									
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60	Boring terminated at a depth of 40 feet below ground surface. Boring backfilled with cement grout. Groundwater encountered at a depth of 18.5 feet during drilling. ¹MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63 and 1.08, respectively, to account for sampler type and hammer energy.  Project No.: 23-2398 Figure: A-6b											

PROJECT:		BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California				Log of Boring B-2 PAGE 1 OF 2							
Boring location: See Site Plan, Figure 2						Logged by: J. Lei Drilled by: Exploration Geoservices, Inc. Rig: Mobile B-53R							
Date started: 04/01/2023		Date finished: 04/01/2023											
Drilling method: 8-inch-diameter hollow-stem auger													
Hammer weight/drop: 140 lbs./30 inches		Hammer type: Downhole Safety Hammer											
Sampler: Modified California (MC), Standard Penetration Test (SPT)													
DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA						
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cut Ft	
1						8 inches of asphalt concrete							
2						SANDY CLAY (CL) dark brown, very stiff, moist, fine to trace coarse sand, trace fine subrounded gravel							
3	MC		14	22		LL = 36, PI = 20; see Appendix B				9.8	120		
4			16										
5	MC		11	25	CL	brown, no gravel							
6			15										
7	MC		11	21		increase in sand content with depth, trace fine sub-angular gravel							
8			16										
9	MC		11	22		CLAYEY SAND (SC) brown to light brown, medium dense, moist, fine to medium sand, trace fine gravel				24	9.3	120	
10			13										
11					SC								
12													
13						gray-brown, wet, coarse sand							
14	MC		18	13						44	22.6	107	
15			9							57	22.1	108	
16			12		CL	SANDY CLAY (CL) yellow-brown, stiff, wet, medium sand LL = 39, PI = 23; see Appendix B							
17													
18						CLAY with SAND (CL) light brown, very stiff, wet, fine sand, increasing sand content with depth (04/01/2023; 10:44 AM)							
19	MC		14	26									
20			19		CL								
21			22										
22													
23						SANDY CLAY (CL) yellow-brown, very stiff, wet							
24	MC		12	26									
25			19		CL								
26			22										
27													
28						light brown, hard							
29	MC		26	32/6"	SP-SC	SAND with CLAY (SP-SC) yellow-brown and gray, very dense, wet, coarse sand							
30			50/6"										
						 ROCKRIDGE GEOTECHNICAL							
						Project No.: 23-2398		Figure: A-7a					

PROJECT: BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California						Log of Boring B-2 PAGE 2 OF 2						
DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		11 19 24	46	SP-SC	SAND with CLAY (SP-SC) (continued)						
32												
33												
34												
35	SPT		17 23 24	51	CL	CLAY with SAND (CL) brown, hard, wet						
36												
37												
38												
39	SPT		17 23 24	51	SC	CLAYEY SAND (SC) gray, very dense, wet, fine to coarse sand						
40												
41												
42												
43	SPT		17 23 24	51	CL	CLAY (CL) gray, hard, wet, trace fine sand						
44												
45												
46												
47	SPT		17 23 24	51								
48												
49												
50												
51	SPT		17 23 24	51								
52												
53												
54												
55	SPT		17 23 24	51								
56												
57												
58												
59	SPT		17 23 24	51								
60												
Boring terminated at a depth of 40 feet below ground surface. Boring backfilled with cement grout. Groundwater encountered at a depth of 18.5 feet during drilling.						¹ MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63 and 1.08, respectively, to account for sampler type and hammer energy.						
						ROCKRIDGE GEOTECHNICAL						
						Project No.: 23-2398			Figure: A-7b			

PROJECT: BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California		Log of Boring B-3 PAGE 1 OF 1	
Boring location: See Site Plan, Figure 2		Logged by: J. Lei Drilled by: Exploration Geoservices, Inc. Rig: Mobile B-40	
Date started: 04/08/2023	Date finished: 04/08/2023		
Drilling method: 8-inch-diameter hollow-stem auger			
Hammer weight/drop: 140 lbs./30 inches		Hammer type: Downhole Safety Hammer	
Sampler: Hand Auger (HA), Modified California (MC)			

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA						
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cut Ft	
1	HA	☒			SC	2 inches of asphalt concrete 4 inches of aggregate base CLAYEY SAND (SC) gray to black, moist, fine to trace coarse sand, trace fine subrounded to subangular gravel							
2													
3	HA	☒											
4													
5	HA	☒											
6													
7													
8					CL	SANDY CLAY with GRAVEL (CL) gray with brown, medium stiff to stiff, moist, fine to trace coarse sand, fine subrounded to subangular gravel							
9	MC	■	5	8									
10		■	6										
11													
12													
13					SC	CLAYEY SAND with GRAVEL (SC) olive-gray, loose to medium dense, moist, medium to coarse sand, fine to coarse gravel							
14	MC	■	5	10									
15		■	6										
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 15 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

¹ MC blow counts for the last two increments were converted to SPT N-Values using a factor of 0.63 to account for sampler type and hammer energy.

**ROCKRIDGE
GEOTECHNICAL**

Project No.: **23-2398**

Figure: **A-8**

BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California						Log of Boring B-4 PAGE 1 OF 1						
Boring location: See Site Plan, Figure 2						Logged by: J. Lei						
Date started: 04/08/2023			Date finished: 04/08/2023			Drilled by: Exploration Geoservices, Inc.						
Drilling method: 8-inch-diameter hollow-stem auger						Rig: Mobile B-40						
Hammer weight/drop: 140 lbs./30 inches			Hammer type: Downhole Safety Hammer									
Sampler: Modified California (MC)												
DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cut Ft
1	MC		6	8	CL	2 inches of asphalt concrete						
2			6			6 inches of aggregate base						
3			7			SANDY CLAY (CL) dark brown with red, medium stiff to stiff, moist, trace debris Soil Corrosivity Test; see Appendix B LL = 35, PI = 17; see Appendix B						
4	MC		7	8		yellow-brown, no debris, fine sand, increasing sand content with depth						
5			7									
6			5									
7	MC		5	8	SC	CLAYEY SAND (SC) brown, loose, moist, fine to medium sand						
8			6									
9			4									
10	MC		5	9		GRAVEL with SAND (GP) brown, loose, wet, fine to coarse sand, fine subrounded to subangular gravel Particle Size Distribution; see Appendix B (04/08/2023; 12:37 PM)						
11	9											
12	9											
13	MC		4	8	GP	GRAVEL with SAND (GP) brown, loose, wet, fine to coarse sand, fine subrounded to subangular gravel Particle Size Distribution; see Appendix B (04/08/2023; 12:37 PM)						
14			6									
15			7									
16	MC		4	6	CL	CLAY with SAND (CL) red-yellow, medium stiff to stiff, wet						
17			4									
18			4									
19	MC		4	6		medium stiff						
20			4									
21			6									
22												
23												
24												
25												
26												
27												
28												
29												
30												

Boring terminated at a depth of 20 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 13.5 feet during drilling.

¹ MC blow counts for the last two increments were converted to SPT N-Values using a factor of 0.63 to account for sampler type and hammer energy.


**ROCKRIDGE
GEOTECHNICAL**

Project No.: 23-2398

Figure: A-9

PROJECT: BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California		Log of Boring B-5 PAGE 1 OF 1	
Boring location: See Site Plan, Figure 2		Logged by: J. Lei	
Date started: 04/08/2023	Date finished: 04/08/2023	Drilled by: Exploration Geoservices, Inc.	
Drilling method: 8-inch-diameter hollow-stem auger		Rig: Mobile B-40	
Hammer weight/drop: 140 lbs./30 inches		Hammer type: Downhole Safety Hammer	
Sampler: Modified California (MC)			

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA						
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cut Ft	
1	MC		4	6	SC	2 inches of asphalt concrete							
2			3			6 inches of aggregate base							
3			6			CLAYEY SAND (SC) brown and yellow, loose, moist, fine to trace coarse sand							
4	MC		6	6	CL	SANDY CLAY (CL) brown to yellow-brown, medium stiff, moist, fine to medium sand increasing sand content with depth							
5			4										
6			5										
7	MC		6	8		medium stiff to stiff							
8	5												
9	7												
10	MC		4	6		CLAYEY SAND (SC) red-yellow, loose, moist LL = 28, PI = 10; see Appendix B Particle Size Distribution; see Appendix B				39	12.3	117	
11	4												
12	5												
13	MC		5	11	SC	medium dense, increasing medium to coarse sand with depth							
14	9												
15	9												
16	MC		5	8		loose, increasing clay content with depth							
17	6												
18	6												
19	MC		5	9	CL	(04/08/2023; 11:27 AM) SANDY CLAY (CL) red-yellow, stiff, wet, fine sand							
20			6										
21			8										
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 20 feet below ground surface.
 Boring backfilled with cement grout.
 Groundwater encountered at a depth of 18 feet during drilling.

¹ MC blow counts for the last two increments were converted to SPT N-Values using a factor of 0.63 to account for sampler type and hammer energy.


ROCKRIDGE
GEOTECHNICAL
 Project No.: **23-2398**
 Figure: **A-10**

PROJECT: BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California		Log of Boring B-6 PAGE 1 OF 1	
Boring location: See Site Plan, Figure 2		Logged by: J. Lei	
Date started: 04/08/2023	Date finished: 04/08/2023	Drilled by: Exploration Geoservices, Inc.	
Drilling method: 8-inch-diameter hollow-stem auger		Rig: Mobile B-40	
Hammer weight/drop: 140 lbs./30 inches		Hammer type: Downhole Safety Hammer	
Sampler: Hand Auger (HA), Modified California (MC)			

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA								
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cut Ft			
1						2 inches of asphalt concrete									
2						4 inches of aggregate base									
3	HA					CL	SANDY CLAY (CL) gray-brown, moist, fine to trace coarse sand								
4	HA														
5															
6															
7	MC		7	8		medium stiff to stiff, trace coarse sand, increasing sand content with depth									
8			6												
9	MC		10												
10			12	14	SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist, fine to coarse sand, trace fine gravel									
11			10												
12	MC		5												
13			9	11		SANDY CLAY (CL) light brown, medium stiff to stiff, moist, fine to medium sand									
14	MC		9												
15			5												
16			6	8	CL										
17															
18															
19															
20															
21															
22															
23															
24															
25															
26															
27															
28															
29															
30															

Boring terminated at a depth of 15 feet below ground surface.
 Boring backfilled with cement grout.
 Groundwater not encountered during drilling.

¹ MC blow counts for the last two increments were converted to SPT N-Values using a factor of 0.63 to account for sampler type and hammer energy.


ROCKRIDGE
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Project No.: 23-2398

Figure: A-11

PROJECT: BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California		Log of Boring B-7 PAGE 1 OF 1	
Boring location: See Site Plan, Figure 2		Logged by: J. Lei	
Date started: 04/08/2023	Date finished: 04/08/2023	Drilled by: Exploration Geoservices, Inc.	
Drilling method: 8-inch-diameter hollow-stem auger		Rig: Mobile B-40	
Hammer weight/drop: 140 lbs./30 inches		Hammer type: Downhole Safety Hammer	
Sampler: Modified California (MC)			

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA								
	Sampler Type	Sample	Blows/6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cut Ft			
1						2 inches of asphalt concrete									
2	MC		5	4	CL	3 inches of aggregate base									
3			4			SANDY CLAY (CL)									
4	MC		4	6		red-brown and brown, soft to medium stiff, moist, trace debris									
5			5			CLAYEY SAND (SC)									
6	MC		4	6		brown, loose, moist, fine to medium sand, trace debris, increasing clay content with depth									
7			5												
8			5		SC										
9															
10															
11	MC		5	6		yellow-brown, increasing sand content with depth									
12			4			SANDY CLAY (CL)									
13			5			light brown, medium stiff, moist									
14					CL										
15	MC		4	9		stiff, increase in medium to coarse sand content									
16			6												
17			9												
18															
19															
20															
21															
22															
23															
24															
25															
26															
27															
28															
29															
30															

Boring terminated at a depth of 15 feet below ground surface.
 Boring backfilled with cement grout.
 Groundwater not encountered during drilling.

¹ MC blow counts for the last two increments were converted to SPT N-Values using a factor of 0.63 to account for sampler type and hammer energy.

ROCKRIDGE
GEOTECHNICAL

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Figure: A-12

PROJECT:		BUENA VISTA MOBILE HOME PARK REDEVELOPMENT 3980 EL CAMINO REAL Palo Alto, California			Log of Boring B-8 PAGE 1 OF 1				
Boring location: See Site Plan, Figure 2					Logged by: J. Lei				
Date started: 04/08/2023		Date finished: 04/08/2023			Drilled by: Exploration Geoservices, Inc.				
Drilling method: 8-inch-diameter hollow-stem auger					Rig: Mobile B-40				
Hammer weight/drop: 140 lbs./30 inches		Hammer type: Downhole Safety Hammer and Hand Auger							
Sampler: Hand Auger (HA), Modified California (MC)									

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA						
	Sampler Type	Sample	Blows/ 6"	SPT N-Value ¹			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cut Ft	
1	HA					3 inches of asphalt concrete							
2						2 inches of aggregate base							
3	HA					SANDY CLAY (CL) yellow-brown, moist, fine to medium sand, trace coarse sand							
4						trace subrounded gravel							
5													
6	MC		10	8	CL	medium stiff to stiff, increasing coarse sand with depth							
7			7										
8			6										
9													
10						CLAYEY SAND (SC) yellow, loose to medium dense, moist, fine sand							
11	MC		10	10	SC								
12			8										
13			8		SP-SC	SAND with CLAY and GRAVEL (SP-SC) gray and yellow-brown, loose to medium dense, moist, fine to coarse sand							
14	MC		5	13	SC	CLAYEY SAND (SC) light brown, loose to medium dense, moist, fine to medium sand							
15			8			(04/08/2023; 3:24 PM) medium dense, wet							
16			12										
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 15 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 14 feet during drilling.

¹ MC blow counts for the last two increments were converted to SPT N-Values using a factor of 0.63 to account for sampler type and hammer energy.

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Figure: **A-13**

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 coarse fine	3" to No. 4	76.2 to 4.76
	3" to 3/4"	76.2 to 19.1
	3/4" to No. 4	19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200	4.76 to 0.075
	No. 4 to No. 10	4.76 to 2.00
	No. 10 to No. 40	2.00 to 0.420
	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 taken with California or Modified California split-barrel sampler. 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	PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter	MC	Modified California sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube	SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.38- or 1.5-inch inside diameter (refer to text)
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube	ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

BUENA VISTA MOBILE
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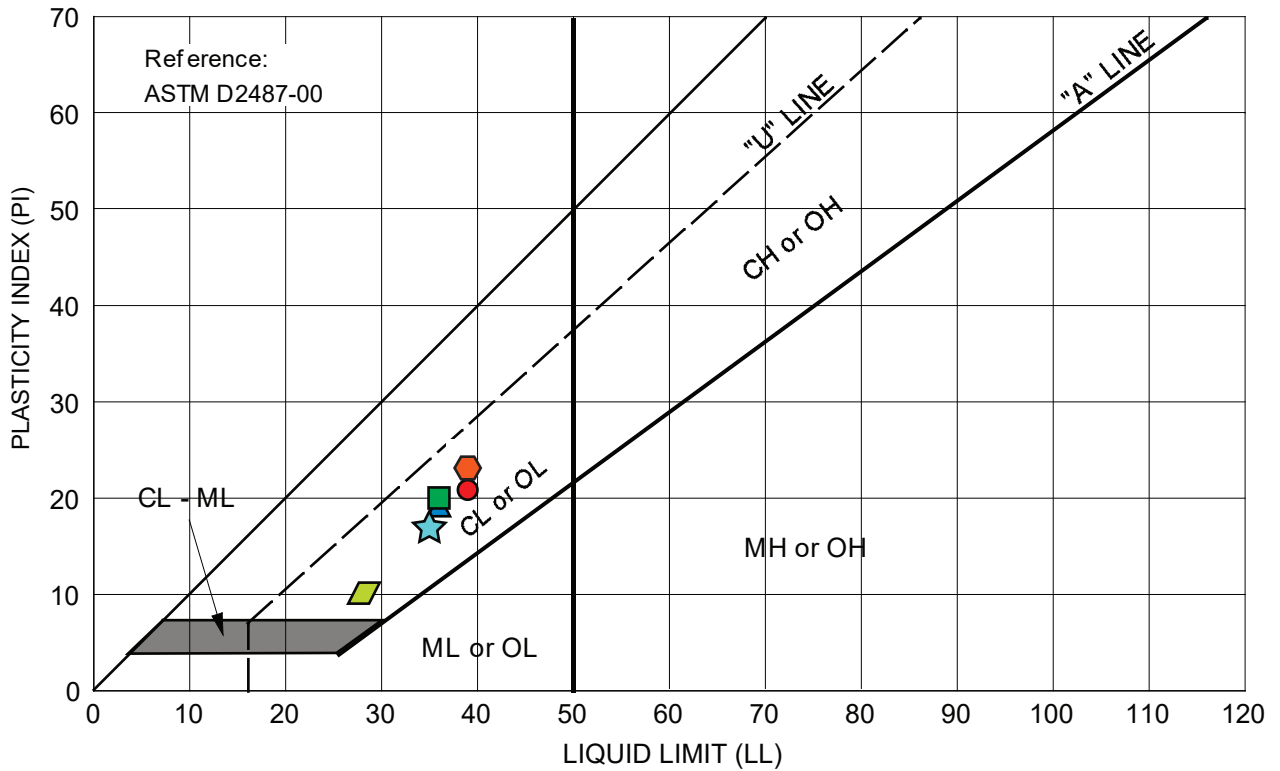
CLASSIFICATION CHART

Date 04/14/23

Project No. 23-2398

Figure A-14

APPENDIX B
Laboratory Test Results



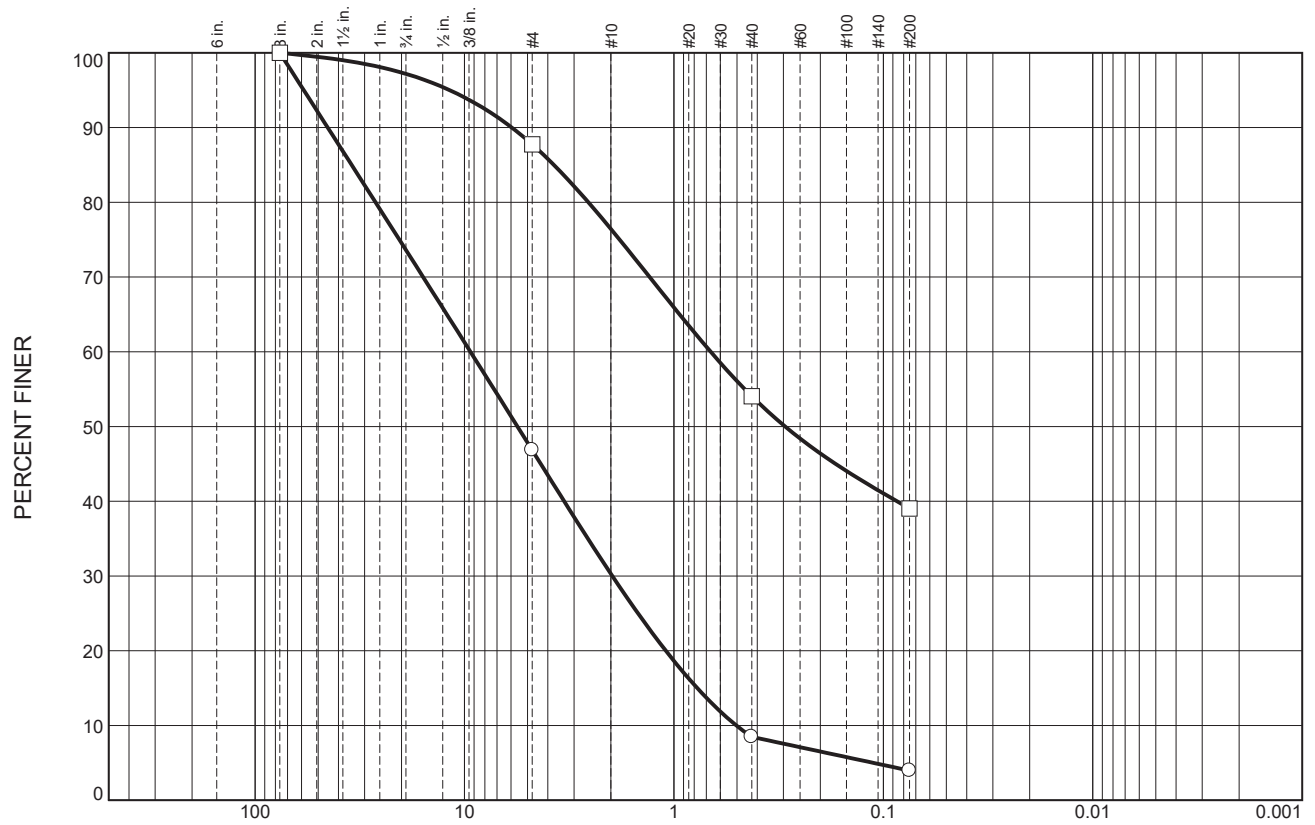
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 2.8 feet	SANDY CLAY (CL), brown to dark brown	14.2	39	21	--
▲	B-1 at 5.5 feet	SANDY CLAY (CL), brown	15.5	36	19	--
■	B-2 at 3.0 feet	SANDY CLAY (CL), dark brown	9.8	36	20	--
⬡	B-2 at 14.5 feet	SANDY CLAY (CL), yellow-brown	21.1	39	23	--
★	B-4 at 2.0 feet	SANDY CLAY (CL), dark brown with red	12.0	35	17	--
▭	B-5 at 9.0 feet	CLAYEY SAND (SC), red-yellow	12.3	28	10	39

**BUENA VISTA MOBILE
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PLASTICITY CHART

Date 05/02/23 Project No. 23-2398 Figure B-1



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	26.4	26.8	16.5	21.8	4.5	4.0	
□	0.0	2.8	9.4	11.4	22.3	15.1	39.0	

SOIL DATA				
SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-4	14.0'	GRAVEL with SAND, brown	GP
□	B-5	9.0'	CLAYEY SAND, red-yellow	SC

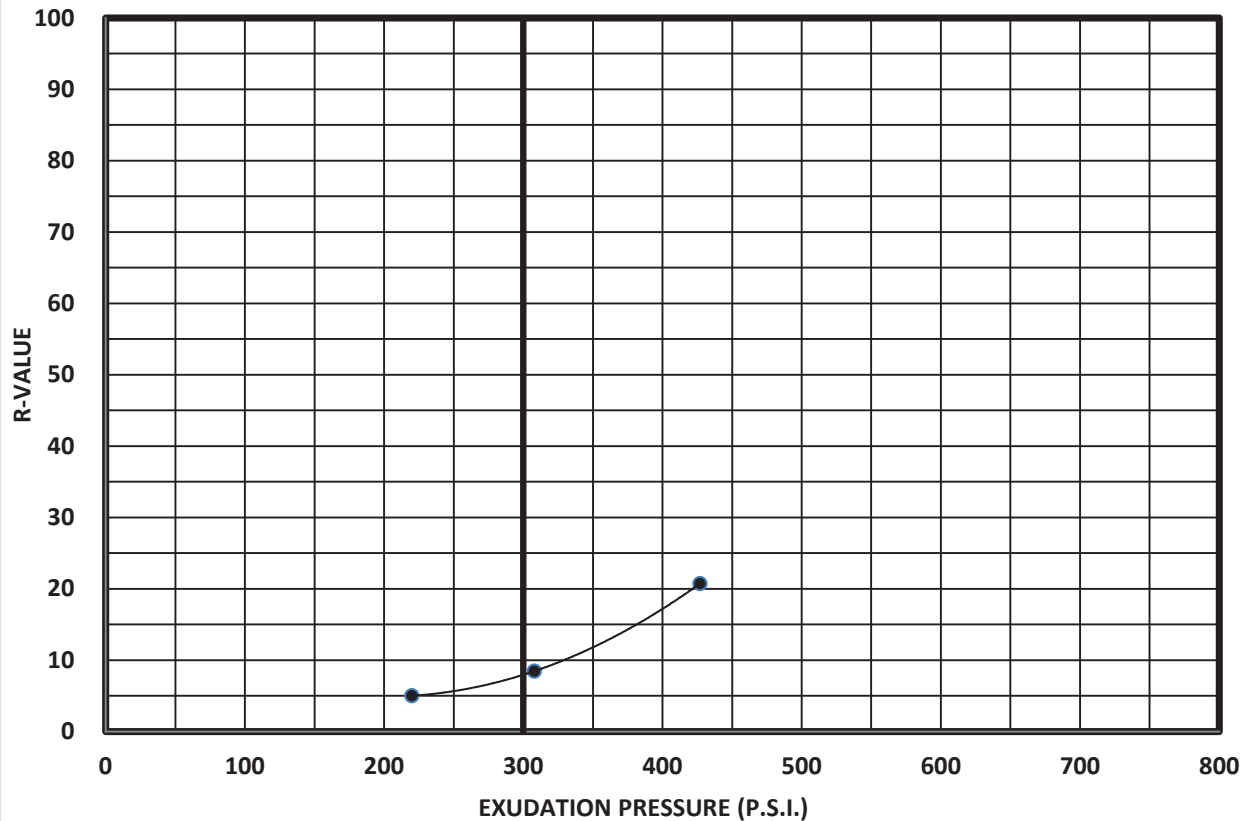
Source: 1'-4'

Sample No: B-2

Client Name & Job No.: Rockridge Geotechnical #23-2398

Sample Description: SANDY CLAY (CL) dark brown

R-VALUE CAL-TEST 301



Exudation (psi)	Compaction (psi)	Expansion (0.0001")	Expansion (psf)	Moisture %	Dry Density	Resistance Value
427	185	11	48	13.7	124.1	21
308	147	0	0	15.4	120.5	8
220	137	0	0	17.6	114.6	5

Resistance Value

7

**BUENA VISTA MOBILE
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**ROCKRIDGE
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RESISTANCE VALUE TEST REPORT

Date 04/21/23

Project No. 23-2398

Figure B-3



Project X Corrosion Engineering

Corrosion Control – Soil, Water, Metallurgy Testing Lab

REPORT S230406B

Bore # / Description	Method	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G187	ASTM G51	ASTM G200	SM 4500-D	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
		Sulfates SO ₄ ²⁻ (mg/kg)	Chlorides Cl ⁻ (mg/kg)	Resistivity As Rec'd / Minimum (Ohm-cm) / (Ohm-cm)	pH	Redox (mV)	Sulfide S ₂ ⁻ (mg/kg)	Nitrate NO ₃ ⁻ (mg/kg)	Ammonium NH ₄ ⁺ (mg/kg)	Lithium Li ⁺ (mg/kg)	Sodium Na ⁺ (mg/kg)	Potassium K ⁺ (mg/kg)	Magnesium Mg ²⁺ (mg/kg)	Calcium Ca ²⁺ (mg/kg)	Fluoride F ₂ ⁻ (mg/kg)	Phosphate PO ₄ ³⁻ (mg/kg)							
B-1: SANDY CLAY (CL) brown to dark brown	2.5	0.0227	11.2	0.0011	1,876	1,742	7.4	162	ND	39.1	4.3	42.5	128.8	1.2	3.0								
B-4: SANDY CLAY (CL) dark brown with red	1.5	0.0668	24.3	0.0024	4,891	1,474	7.4	145	ND	38.9	7.7	49.9	115.2	8.3	0.5								

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops which is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

29990 Technology Dr., Suite 13, Murrieta, CA 92563 Tel: 213-928-7213 Fax: 951-226-1720
www.projectxcorrosion.com

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SOIL CORROSION TEST RESULTS

Date 04/21/23 Project No. 23-2398 Figure B-4