ROCKRIDGE GEOTECHNICAL

Prepared for **SummerHill Homes**

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
210 BAYPOINTE PARKWAY
SAN JOSE, CALIFORNIA

April 14, 2022
Project No. 22-2192

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Project No. 22-2192

Mr. Marshall Torre
Director of Development
SummerHill Homes
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Subject: Preliminary Geotechnical Investigation
Proposed Residential Development
210 Baypointe Parkway
San Jose, California

Dear Mr. Torre:

We are pleased to present our preliminary geotechnical investigation report to support your due diligence evaluation for the proposed residential development at 210 Baypointe Parkway in San Jose, California. Our preliminary geotechnical investigation was performed in accordance with our proposal dated February 7, 2022.

The site is located on the southeastern side of Baypointe Parkway, south of its intersection with Zanker Road. The subject property is relatively level and encompasses approximately 4.3 acres. The site is bordered by a vacant lot to the northeast, Baypointe Parkway to the northwest, and residential properties to the southeast and southwest. Currently, the site is occupied by a one-story commercial building and paved surface parking lots and driveways.

Plans are to demolish the existing building and construct six condominium buildings and one apartment building. The condominium buildings will be constructed on the northern half of the site and the apartment building will be constructed on the southern half of the site. A paseo will be constructed near the center of the site, between the condominium buildings and the apartment building. The condominium buildings will be three stories, some of which will have a roof deck, and will contain of 42 residential units. The apartment building will be seven stories consisting of five levels of Type III-A construction over two levels of Type I-A podium and will be constructed at-grade. The apartment building will contain 287 residential units and will include indoor and outdoor recreational amenities including a pool and spa on the podium courtyard, roof top decks, and a clubroom and fitness studio.

We understand a portion of the site may be impacted by the flood zone and the project may be designed to increase the finished floor elevation of proposed buildings to above the flood zone.

From a geotechnical standpoint, we preliminarily conclude the site can be developed as planned. The primary geotechnical concerns are:

- the presence of highly to very highly expansive near-surface clay;
- the presence of potentially liquefiable soil underlying the site; and
- the presence of medium stiff to stiff clay that is moderately compressible underlying the site.

We preliminarily conclude the proposed buildings may be supported on mat foundations.

This report presents our preliminary conclusions and recommendations regarding foundation design and other geotechnical aspects of the project. The recommendations contained in our report are based on limited subsurface exploration and are not intended for final design. A final geotechnical report should be prepared for the project once the development plans and building design have been further developed.

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

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
HUI JUN LIANG
GE2663
Exp. 6/30/2022
GEOTECHNICAL
STATE OF CALIFORNIA

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Principal Engineer

Enclosure

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PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
210 BAYPOINTE PARKWAY
SAN JOSE, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed residential development at 210 Baypointe Parkway in San Jose, California. The site is located on the southeastern side of Baypointe Parkway, south of its intersection with Zanker Road, as shown on the Site Location Map, Figure 1.

The subject property is relatively level and encompasses approximately 4.3 acres. The site is bordered by a vacant lot to the northeast, Baypointe Parkway to the northwest, and residential properties to the southeast and southwest. Currently, the site is occupied by a one-story commercial building and paved surface parking lots and driveways, as shown on the Site Plan, Figure 2.

Plans are to demolish the existing building and construct six condominium buildings and one apartment building. The condominium buildings will be constructed on the northern half of the site and the apartment building will be constructed on the southern half of the site. A paseo will be constructed near the center of the site, between the condominium buildings and the apartment building. The condominium buildings will be three stories, some of which will have a roof deck, and will contain of 42 residential units. The condominiums will have common open space with amenities including barbeque, seating, landscaping, and bicycle parking. The apartment building will be seven stories consisting of five levels of Type III-A construction over two levels of Type I-A podium and will be constructed at-grade. The apartment building will contain 287 residential units and will include indoor and outdoor recreational amenities including a pool and spa on the podium courtyard, roof top decks, and a clubroom and fitness studio.

We understand a portion of the site may be impacted by the flood zone and the project may be designed to increase the finished floor elevation of proposed buildings to above the flood zone.

2.0 SCOPE OF SERVICES

Our preliminary investigation was performed in accordance with our proposal dated February 7, 2022. Our scope of services consisted of performing eight cone penetration tests (CPTs), advancing four hand-auger borings, performing laboratory tests on selected soil samples, and performing engineering analyses to develop preliminary conclusions and recommendations regarding:

- subsurface conditions
- design high groundwater level
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- the most appropriate foundation type(s) for the proposed buildings
- preliminary design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlements
- 2019 California Building Code (CBC) site class and mapped design spectral response acceleration parameters
- flexible and rigid pavement design
- 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

Our preliminary field investigation consisted of performing eight CPTs, designated as CPT-1 through CPT-8, and advancing four hand-auger borings, designated as HA-1 through HA-2. The approximate locations of the CPTs and borings are shown on Figure 2. Prior to our field investigation, we contacted Underground Service Alert (USA) to notify them of our work. In addition, we retained a private utility locator, C. Cruz Sub-Surface Locators, to check for buried utilities at CPT and hand-auger boring locations to reduce the potential for encountering utilities during our field investigation. Details of the field investigation and laboratory testing are described in this section.

3.1 Cone Penetration Tests

The CPTs were performed by Middle Earth Geo Testing, Inc. of Orange, California on March 22, 2022. The CPTs were all advanced to a depth of about 50-1/2 feet below the existing ground surface (bgs), except for CPT-4 that was advanced to a depth of about 60-1/2 feet bgs.

The CPTs were performed by hydraulically pushing a 1.7-inch-diameter cone-tipped probe into the ground with a 25-ton truck. 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 at approximately two-inch recording interval for the entire depth advanced. Soil data, including tip resistance, frictional resistance, and pore water pressure, were recorded by a computer while the test was 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 are presented in Appendix A on Figures A-1 through A-8.

Upon completion, the CPT holes were backfilled with cement grout.

3.2 Hand-Auger Borings

The hand-auger borings were advanced using a three-inch-diameter, stainless steel hand-auger to a depth of five feet bgs. Soil samples were obtained from each boring for visual classification and laboratory testing. The subsurface conditions encountered in the borings are presented on Figures A-9 through A-12. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-13. Upon completion, the borings were backfilled with the soil cuttings and cement grout.

3.3 Laboratory Testing

We re-examined each soil sample obtained from our hand-auger borings to confirm the field classifications and selected representative samples for laboratory testing. Soil samples were tested to measure moisture content, Atterberg limits (plasticity index), percent passing the No.

200 sieve, and 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 Holocene-age alluvium (Qha). Alluvial deposits generally consist of a mixture of fine-grained and coarse-grained deposits and are deposited by rivers and streams. We encountered about 3 to 4 feet of fill in CPT-4 and CPT-7 that consists of sandy clay and clayey sand. The fill or the site, where fill is not present, is underlain by clay that extends to depths of 16 to 28 feet bgs. Where explored, the clay is stiff to very stiff to depths of about 10 to 16 feet bgs and becomes medium stiff to stiff below these depths. The clay is underlain by a layer of medium dense to dense sand with variable silt and clay content that extends to depths of about 25 to 38 feet bgs. This sand is underlain by a medium stiff to stiff clay with variable silt and sand content that extends to depths of 40 to 48 feet bgs; this clay layer was not present in CPT-5. Below depths of 40 to 48 feet bgs is dense to very dense sand with variable silt and clay content that extends to the maximum depths explored of 50-1/2 and 60-1/2 feet bgs.

Atterberg limits tests performed on samples of the near-surface clay obtained from hand-auger borings HA-1, HA-3, and HA-4 at depths of 4, 2, and 2.5 feet bgs indicate the near surface clay has plasticity indices of 45, 28, and 30, respectively, and therefore has high to very high expansion potential[1].

Groundwater was measured in the CPTs at depths of 8 to 13 feet bgs using a weighted tape prior to grouting. The groundwater levels in the CPTs may not have been fully stabilized at the time of these measurements. We reviewed the report Seismic Hazard Zone Report (2001) prepared by the California Geological Survey (CGS) for the Milpitas 7.5-Minute Quadrangle. The report indicates a historic high groundwater level at the site vicinity to be between 5 and 10 feet bgs. The groundwater level at the site is expected to fluctuate several feet seasonally, with potentially

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

more significant fluctuations annually, depending on the amount of rainfall. We preliminarily conclude that a high groundwater level of 6 feet bgs should be used for this project.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity and Faulting

The site is located in 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 plate and North American plate and subsequent strike-slip faulting along the San Andreas Fault system. The San Andreas Fault is more than 600 miles long from Point Arena in the north to the Gulf of California in the south. The Coast Ranges 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, Calaveras, Monte Vista and San Andreas faults. Numerous damaging earthquakes have occurred along these faults in recorded time. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[2] [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
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	7.2	Northeast	7.58
Hayward (South, HS)	7.2	Northeast	7.00
Total Calaveras (CN+CC+CS+CE)	12	East	7.43
Calaveras (Central, CC)	12	East	6.85
Calaveras (North, CN)	12	East	6.86
Hayward (Extension, HE)	12	East	6.18
Monte Vista - Shannon	16	West	7.14
Total North San Andreas (SAO+SAN+SAP+SAS)	22	Southwest	8.04
North San Andreas (Peninsula, SAP)	22	Southwest	7.38
Las Positas	24	Northeast	6.50
Butano	27	Southwest	6.93
North San Andreas (Santa Cruz Mts, SAS)	27	South	7.15
Sargent	30	South	6.71
Zayante-Vergeles (2011 CFM)	35	South	7.48
Greenville (North)	36	East	6.86
Zayante-Vergeles	36	South	7.00
Greenville (South)	37	East	6.64
Mount Diablo Thrust	37	Northeast	6.67
Mount Diablo Thrust South	37	Northeast	6.50
Mount Diablo Thrust North CFM	40	North	6.72
San Gregorio (North)	41	West	7.44
Hayward (North, HN)	46	Northwest	6.90

Since 1800, four major earthquakes have been recorded on the North San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt, 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista, approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9. It was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of October 17, 1989, had an M_w of 6.9 and occurred about 42 kilometers south of the site.

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

As 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 the sections of the Hayward (South), Calaveras (Central), and the North San Andreas (Santa Cruz Mountains) faults. The respective probabilities are approximately 25, 21, and 17 percent.

5.2 Geologic Hazards

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

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

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

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

5.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the Hayward and San Andreas faults, although ground shaking from future earthquakes on other faults will also be felt at the site. The intensity of earthquake ground motion at the site will 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 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 Fault Zoning Act, and no known active or potentially active faults exist on the site. We, therefore, conclude the risk of fault offset at the site from a known active fault is very low. 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 also very low.

5.2.3 Liquefaction and Associated Hazards

Strong shaking during an earthquake can result in ground failures such as those associated with soil liquefaction and lateral spreading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures, and sand boils are evidence of excess pore pressure generation and liquefaction.

The site has been mapped inside a zone of liquefaction potential on the map titled *Earthquake Zones of Required Investigation, Milpitas Quadrangle, Official Map*, prepared by the California Geological Survey (CGS), dated October 19, 2004 (Figure 5). 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 subsurface investigations in mapped liquefaction hazard zones be performed using rotary-wash borings and/or CPTs to a depth of at least 50 feet bgs.

We preliminarily evaluated the liquefaction potential at the site using data collected from our CPTs. Liquefaction susceptibility was assessed using the software CLiq v3.3.1.13 (GeoLogismiki, 2022). CLiq uses measured field CPT data and assesses liquefaction potential, including post-earthquake vertical settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). Our liquefaction analyses were performed using the methodology proposed by Boulanger and Idriss (2014). We also used the relationship proposed by Zhang, et al (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement; a relationship that is an extension of the work by Ishihara and Yoshimine (1992).

Our analyses were performed using the approximate in-situ groundwater depths measured in our CPTs and a “during earthquake” groundwater depth of six feet bgs. In accordance with the 2019 CBC, we used a peak ground acceleration of 0.74 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 (PGA_M). We also used a moment magnitude 7.58 earthquake, which is consistent with the mean characteristic moment magnitude for the San Andreas Fault, as presented in Table 1.

Our liquefaction analyses indicate there are relatively thin layers of potentially liquefiable soil below depths of 16 to 28 feet bgs. The potentially liquefiable layers generally have soil behavior types “sand”, “silty sand” and “sandy silt” and are generally less than four feet thick, except for an eight-foot thick layer of “sand” and “silty sand” between 23 and 31 feet bgs in CPT-6. We estimate total free-field ground settlement associated with liquefaction (referred to as post-liquefaction reconsolidation) at the site after the above-defined MCE event will on the order of 1/2 to 1-1/2 inches and differential settlement can be up to about 1/2 inch over a horizontal distance of 30 feet.

Ishihara (1985) presented an empirical relationship that provides criteria that can be used to evaluate whether liquefaction-induced ground failure, such as sand boils, would be expected to occur under a given level of shaking for a liquefiable later of given thickness overlain by a resistant, or protective, surficial layer. Our analysis indicated the non-liquefiable soil overlying the potentially liquefiable soil layers is sufficiently thick and the potentially liquefiable layers are sufficiently thin such that the potential for surface manifestations of liquefaction, such as sand boils, is low.

Considering the site topography is relatively flat and the potentially liquefiable layers are not continuous, we conclude the risk of lateral spreading is very low.

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. The soil encountered above the groundwater table is not susceptible to cyclic densification due to its cohesion. Therefore, we conclude the potential for cyclic densification to occur at the site is nil

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, we preliminarily conclude the site can be developed as planned. The primary geotechnical concerns are:

- the presence of highly to very highly expansive near-surface clay;
- the presence of potentially liquefiable soil underlying the site; and
- the presence of medium stiff to stiff clay that is moderately compressible underlying the site.

These and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

6.1 Expansive Soil

Atterberg limits tests performed on samples of the near-surface clay obtained from our hand-auger borings indicate the near-surface clay has high to very high expansion potential. Expansive near-surface soil is subject to volume changes during seasonal fluctuations in moisture content. These volume changes can cause movement and cracking of foundations, slabs and pavements. Therefore, foundations and slabs should be designed and constructed to resist the effects of the expansive clay. These effects can be mitigated by moisture-conditioning the expansive soil below slabs, providing non-expansive soil below slabs, and either supporting foundations below the zone of severe moisture change or providing a stiff, shallow foundation that can limit deformation of the superstructure as the underlying soil shrinks and swells.

We recommend the upper 18 inches of soil subgrade beneath slab-on-grade floors and exterior concrete flatwork be replaced with non-expansive fill. The non-expansive fill may consist on lime-treated onsite clay or select fill. Select fill should consist of imported or on-site soil that is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the Geotechnical Engineer.

For slab-on-grade floors, the 18 inches of non-expansive fill should be measured from the bottom of the capillary moisture break. The 18 inches of non-expansive fill may be omitted if the building is supported on a mat foundation that is at least 18 inches thick.

Even with 18 inches of non-expansive fill, exterior slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slab edges and adding additional reinforcement will control this cracking to some degree. In addition, where slabs provide access to buildings, 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.

6.2 New Fill and Settlement

We understand a portion of the site may be impacted by the flood zone and the project may be designed to increase the finished floor elevation of proposed buildings to above the flood zone. Grading plans showing existing and proposed finished floor elevations were not available when this report was prepared.

The underlying soil will compress/consolidate from the weight of the new fill. We estimate total static settlement will be about 1/4 inch and 1/2 inch for 2 and 4 feet of new fill, respectively. We anticipate the settlement will occur soon after fill placement (i.e., within about 1 to 2 months).

6.3 Foundations and Settlement

Based on the results of our preliminary field investigation, we anticipate the building pads are underlain by medium stiff to stiff clay that is moderately compressible below depths of 10 to 16 feet bgs. If the proposed buildings are supported on a shallow foundation system, settlement will occur due to compression of the underlying clay under static foundation loads. On the basis of our experience, we judge the anticipated total and differential settlements due to static foundation loads will exceed the typical tolerance of a conventional spread footing foundation system. We preliminarily conclude a stiffened mat foundation system would be the most appropriate foundation system for the proposed buildings, provided the estimated settlements are acceptable from a structural standpoint.

The mat foundation for the proposed structures should be at least 18 inches thick and the edges of the mat should be thickened such that the mat edge is bottomed at least 9 inches below the lowest adjacent exterior grade. For mat design, we recommend using a modulus of subgrade reaction of 20 and 15 pounds per cubic inch (pci) for dead-plus-live loads for the condominium buildings and the apartment building, respectively. These values have already been scaled to take into account the plan dimensions of the foundation and may be increased by one-third percent for total load conditions.

Considering the large area of the mat, we expect the average bearing stress under the mat to be low; however, concentrated stresses will occur at column locations and at the edges of the mat.

The mat should be designed to impose a maximum dead-plus-live bearing pressure of 3,500 pounds per square foot (psf) on the foundation subgrade soil. This pressure may be increased by one-third for total load conditions.

Conventionally reinforced mat foundations should be designed in accordance with the Wire Reinforcement Institute's (WRI's) publication title *Design of Slab-on-Grade Foundations, An Update* (1996). We recommend the following parameters should be used in conjunction with the WRI design method:

- Climatic rating (C_w) – 15
- Equivalent Plasticity Index (PI) – 45
- Slope Correction Coefficient (C_s) – 1.0
- Consolidation Correction Coefficient (C_o) – 1.0

We estimate the total settlement of a mat-supported buildings with an average bearing pressure of 500 psf (condominium buildings) and 1,200 psf (apartment building) for dead-plus-live load conditions will be up to about 1-1/4 and 2-1/4 inches, respectively. The amount of differential settlement between columns will be a function of the mat stiffness and hence its ability to spread the loads between columns, however, we expect the mats can be designed to limit differential settlements to about 1/2 inch in 30 feet. We estimate total and differential settlements associated with liquefaction at the site during an MCE event generating a PGA_M of 0.74g will be less than 1-1/2 inches and 1/2 inch across a horizontal distance of 30 feet, respectively

Assuming the mat is supported on a vapor retarder, a friction factor of 0.20 may be used to compute base friction. Where the mat foundation is supported directly on soil, a friction factor of 0.30 may be used. To compute lateral resistance, we recommend using a uniform pressure of 1,500 psf for transient load conditions; the upper foot of soil should be ignored unless confined by a slab or pavement. The values for friction coefficient and passive pressure include a factor of safety of 1.5 and may be used in combination without further reduction

6.4 Seismic Design

As discussed in Section 5.2.3, the site is underlain by relatively thin layers of potentially liquefiable soil. Although the 2019 CBC call for a Site Class F designation for sites underlain by potentially liquefiable soil, we conclude a Site Class D designation is more appropriate because the potentially liquefiable layers are thin and relatively dense such that the site will not incur significant non-linear behavior during strong ground shaking. Therefore, for seismic design, we recommend Site Class D be used.

The latitude and longitude of the site are 37.4131° and -121.9402°, respectively. For design in accordance with 2019 CBC (ASCE 7-16), we recommend the following:

- Site Class D – stiff soil
- S_S = 1.594g, S_1 = 0.601g

The 2019 CBC is based on the guidelines contained within ASCE 7-16. Per ASCE 7-16, where S_1 is greater than 0.2 times gravity (g) for Site Class D, a ground motion hazard analysis is needed unless the seismic response coefficient (C_s) value will be calculated as outlined in Section 11.4.8, Exception 2 of ASCE 7-16. Assuming the C_s value will be calculated as outlined in Section 11.4.8, Exception 2 of ASCE 7-16, we recommend the following seismic design parameters:

- F_a = 1.0, F_v =1.7
- S_{MS} = 1.594g, S_{M1} = 1.022g
- S_{DS} = 1.063g, S_{D1} = 0.681g
- Seismic Design Category D for Risk Factors I, II, and III

6.5 Soil Corrosivity

Corrosivity tests were performed by Project X Corrosion Engineering of Murrieta, California on selected soil samples obtained from borings HA-1, HA-2 and HA-4 at 1.5, 2.5, and 4.5 feet bgs, respectively. The corrosivity test results are presented in Appendix B of this report.

Many factors can affect the corrosion potential of soil including, but not limited to, resistivity, pH, and chloride and sulfate concentrations. The resistivity test results (670 to 1,474 ohm-cm) indicate the soil is "highly to extremely corrosive[6]" to buried metal. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron should be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.

The results of the pH tests (6.2 to 8.2) indicate the near-surface is "moderately to negligibly corrosive" to buried metallic and concrete structures. The chloride ion concentrations (44.5 to 92.4 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 (173 to 467 mg/kg) are sufficiently low such that sulfates do not to pose a threat to buried concrete.

6.6 Pavement Design

6.6.1 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. Considering the soil subgrade in pavement areas will likely consist of highly to very highly expansive clay, we selected a minimum resistance value (R-value) of 5 for pavement design, which is appropriate for highly to very highly expansive clay.

Recommended pavement sections for traffic indices (TIs) ranging from 4.5 to 6.0 are presented in Table 2. The project civil engineer should determine the appropriate design TI based on the anticipated vehicular traffic the pavement will experience. We can provide additional pavement sections for different TIs upon request.

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6 Roberge, Pierre R. (2018). *Corrosion Basics, an Introduction, Third Edition*. NACE International, P. 189.

TABLE 2
Asphalt Pavement Sections

TI	Subgrade Lime Treated (?)	Asphaltic Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4.5	No	2.5	9.5
4.5	Yes	2.5	6.5
5.0	No	3.0	10.0
5.0	Yes	3.0	6.5
5.5	No	3.0	12.0
5.5	Yes	3.0	8.0
6.0	No	3.5	13.0
6.0	Yes	3.5	8.5

The pavement sections with lime treatment assume the upper 12 inches of the pavement subgrade is treated and is based on a conservative R-value of 25 for the lime-treated soil. The upper 12 inches of the subgrade (treated or untreated) should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction[7] (95 percent relative compaction if non-expansive soil subgrade) and be non-yielding. The aggregate base should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction and be non-yielding.

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

---

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

6.6.2 Rigid (Portland-Cement Concrete) Pavement

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle load of 32,000 pounds and moderate truck traffic (i.e., several trucks per week). The recommended rigid pavement section for these axle loads is 6.5 inches of Portland cement concrete (PCC) over six inches of Class 2 aggregate base. For areas that will receive fire truck traffic, the PCC thickness should be increased to seven inches. For areas that will experience only passenger vehicle traffic, the recommended pavement section is five inches of PCC over six inches of Class 2 aggregate base.

The modulus of rupture and unconfined compressive strength of the concrete should be at least 500 and 3,200 psi at 28 days, respectively. Contraction joints should be placed at maximum spacing of 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement, concrete pavement, or interlocking concrete pavers, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. Dowelling is not required where new pavement abuts existing pavement. Concrete slabs subject to vehicular traffic should be reinforced with a minimum of No. 4 bars at 16-inch spacing on center in both directions.

Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those we have described above for asphalt concrete pavement. Recommendations for pavements adjacent to irrigated landscaped areas, bioswales, or other storm water treatment areas are also the same as those presented above for asphalt concrete pavement.

6.6.3 Exterior Concrete Flatwork

Differential ground movement caused by the shrinking and swelling of the highly to very highly expansive near-surface clay should be expected. Differential ground movement can damage exterior concrete slabs. To reduce the potential for differential movement, the expansive clay within the upper 12 inches of the slab subgrade and a lateral extent of about one foot beyond the limits of the slab should be removed and replaced with non-expansive soil, such as Class 2 aggregate base material. The exterior concrete flatwork should be underlain by at least four inches of Class 2 aggregate base compacted to at least 90 percent relative compaction; this Class 2 aggregate base is in addition to the aforementioned 12 inches of non-expansive soil.

6.6.4 Non-Permeable Concrete Pavers

To reduce the potential for differential movement, the expansive clay within the upper 12 inches of the slab subgrade and a lateral extent of about one foot beyond the limits of the non-permeable pavers should be removed and replaced with non-expansive soil. We recommend non-permeable pedestrian pavers and sand bedding be underlain by at least four inches of Class 2 aggregate base compacted to at least 90 percent relative compaction; this Class 2 aggregate base is in addition to the aforementioned 12 inches of non-expansive soil. We recommend non-permeable pavers subject to vehicular traffic be underlain by Class 2 aggregate base compacted to at least 95 percent relative compaction. The aggregate base thickness beneath non-permeable pavers subject to vehicular traffic should be consistent with that recommended in Table 2 for asphalt pavement for the appropriate TI.

6.6.5 Permeable Interlocking Concrete Pavers

We recommend permeable interlocking concrete pavements (ICP) be designed in accordance with the guidelines presented by the Interlocking Concrete Pavement Institute (ICPI 2005). These guidelines include specific recommendations for permeable aggregate subbase, base, and bedding courses to be placed beneath ICP pavements. We recommend permeable pavements for vehicular and pedestrian traffic be designed for no exfiltration of water into the subgrade soil. This requires installing a subdrain system at the base of the pervious aggregate materials, which are underlain by an impermeable liner.

The soil subgrade beneath ICP pavements should be prepared and compacted in accordance with the recommendations presented in Section 6.6.1 for subgrade preparation of asphalt concrete pavements. In addition, the subgrade should be a firm and non-yielding surface. The subgrade should be proof-rolled under the observation of our field engineer to confirm it is non-yielding prior to placing the filter fabric and aggregate base materials. The soil subgrade at the bottom of the permeable section should slope down toward the drain pipe trench at a gradient of at least two percent. The perforated pipe should slope down to a suitable outlet at a minimum gradient of one percent. The pipe should be placed with the perforations down on a minimum of two inches of permeable subbase.

ICPI's guidelines call for 1-1/2 to 2 inches of bedding material consisting of ASTM No. 8 aggregate directly below the pavers. This material is also recommended for fill material between the pavers. As shown in Table 3 below, this material consists of fine gravel with 10 to 30 percent sand.

TABLE 3
Gradation Requirements for ASTM No. 8 Aggregate

Sieve Size	Percentage Passing Sieve
1/2 inch	100
3/8 inch	85 – 100
No. 4	10 – 30
No. 8	0 – 10
No. 16	0 – 5

The ASTM No. 8 bedding should be underlain by a permeable base course of ASTM No. 57 crushed aggregate. As shown in Table 4, ASTM No. 57 aggregate consists of open-graded gravel with a gradation between that of the 3/4-inch drain rock and the ASTM No. 8 aggregate.

TABLE 4
Gradation Requirements for ASTM No. 57 Aggregate

Sieve Size	Percentage Passing Sieve
1-1/2 inch	100
1 inch	95 – 100
1/2 inch	25 – 60
No. 4	0 – 10
No. 8	0 – 5

The ASTM No. 57 permeable base course should be underlain by a permeable subbase course of ASTM No. 2 crushed aggregate. The gradation requirements for ASTM No. 2 crushed aggregate subbase are presented in Table 5.

TABLE 5
Gradation Requirements for ASTM No. 2 Aggregate

Sieve Size	Percentage Passing Sieve
3 inch	100
2-1/2 inch	90-100
2 inch	35-70
1-1/2 inch	0-15
3/4 inch	0 -5

The No. 2 aggregate subbase course should be placed in lifts not exceeding 6 inches in loose thickness and compacted using a smooth-drum roller, operated in static (non-vibratory) mode. The subsequent course of No. 57 aggregate may be placed in one lift and should be compacted with a smooth-drum roller in vibratory mode with sufficient passes to create an unyielding surface. Placement and compaction of the permeable aggregate base and subbase should be performed under the observation of our field engineer. Following compaction of the No. 57 aggregate, the No. 8 bedding, not exceeding 2 inches in loose thickness, should be placed and screeded to a level, undisturbed surface immediately prior to paver installation.

The required thicknesses of the permeable aggregate base and subbase courses depends on the infiltration and water storage design requirements, as well as the traffic loading demand. Recommendation for the minimum permeable ICP pavement section (based on traffic demand) for TI of 6.0 is presented in Table 6; ICP pavement sections for other TI’s can be provided upon request. Where permeable pavement will be subject to fire trucks, we recommend a layer of triaxial geogrid (i.e., Tensar TriAx TX-140 Geogrid or equivalent) be placed atop the soil subgrade prior to placing the paver aggregates. Also included in Table 6 is a recommended section for permeable ICPs subject to pedestrian traffic only.

TABLE 6
Recommended Pavement Sections for
Permeable Interlocking Concrete Pavers

TI	ASTM No. 8 Bedding Aggregate (inches)	ASTM No. 57 Stone Base (inches)	ASTM No. 2 Stone Subbase (inches)
Pedestrian	1.5-2.0	4.0 (10)	6.0 (0)
6.0	1.5-2.0	4.0	8.0

The above recommended ICP pavement sections are based on the ICPI technical guidelines (ICPI 2005). From a geotechnical standpoint, it is acceptable to design the pedestrian ICP section to exclude the No. 2 subbase course, in which case the No. 57 base course should be increased to 10 inches. From a geotechnical standpoint, it is also acceptable to use compacted structural planting mix in lieu of the No. 57 and No. 2 base courses in locations where the pedestrian ICP section is adjacent to tree wells and is required for promoting root growth.

7.0 ADDITIONAL GEOTECHNICAL SERVICES

The preliminary conclusions and recommendations presented within the report are based on a preliminary field investigation and not intended for final design. Prior to final design, additional borings and/or CPTs should be performed to supplement existing subsurface information and to develop final geotechnical conclusions and recommendations.

8.0 LIMITATIONS

This preliminary 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 preliminary recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the CPTs performed at the site during our 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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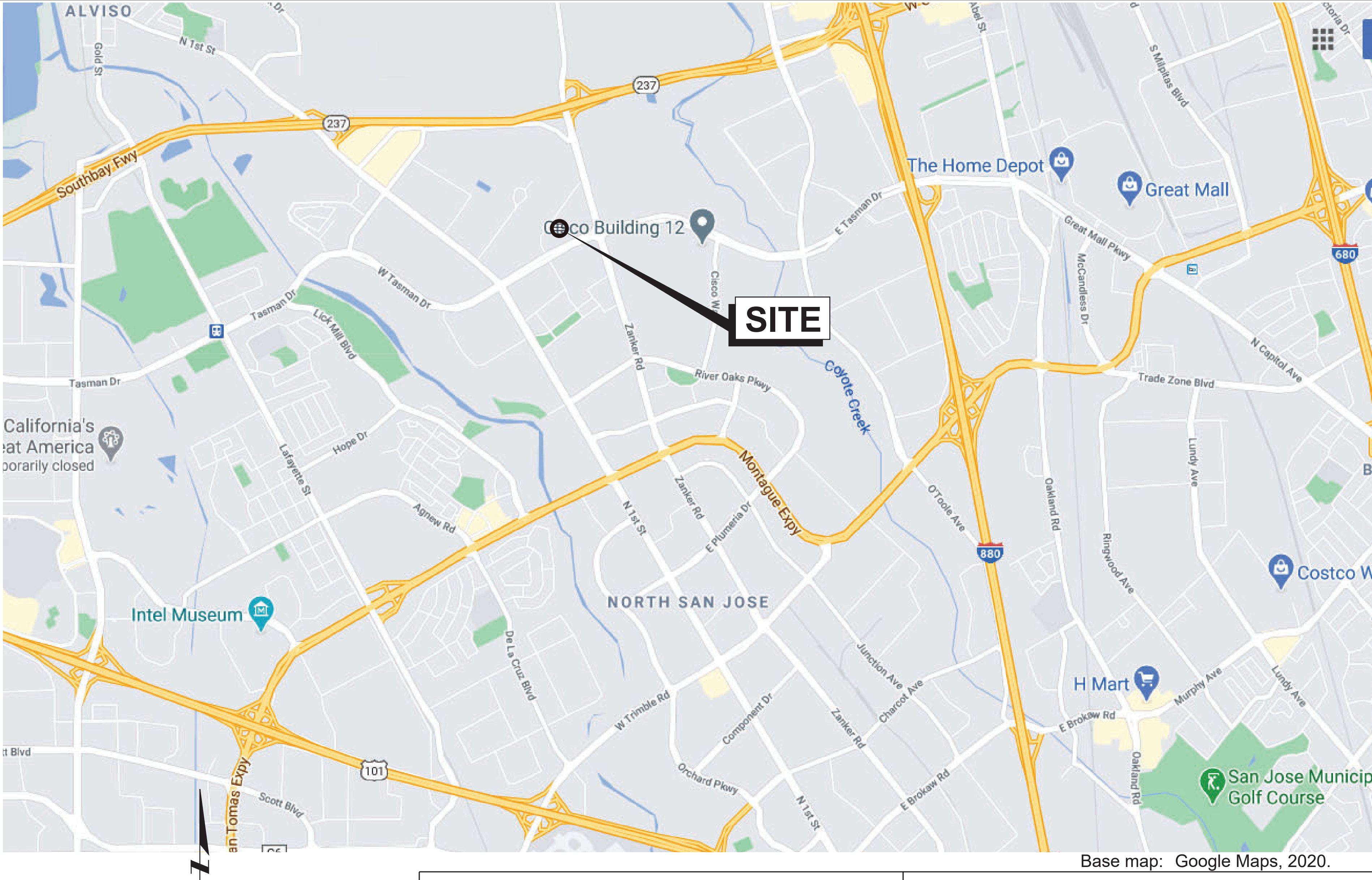
ROCKRIDGE GEOTECHNICAL

FIGURES



Base map: Google Maps, 2020.

0 3,000 6,000 Feet

Approximate scale

210 BAYPOINTE PARKWAY
San Jose, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 04/04/22	Project No. 22-2192	Figure 1

EXPLANATION

CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical, Inc., March 22, 2022

HA-1 — Approximate location of hand-auger boring by Rockridge Geotechnical, Inc., March 22, 2022

Project limits

0 80 Feet

Approximate scale

Base map: Google Earth, 2021

210 BAYPOINTE PARKWAY
San Jose, California

SITE PLAN

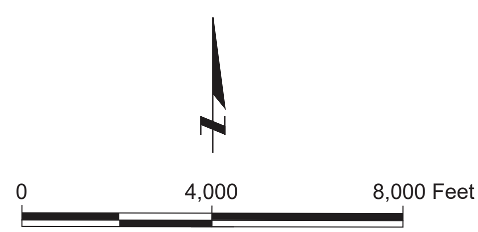
Date 04/13/22	Project No. 22-2192	Figure 2

ROCKRIDGE GEOTECHNICAL

Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2020.

EXPLANATION

af	Artificial Fill
Qhym	Mud deposits (late Holocene)
Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)
	Geologic contact: dashed where approximate and dotted where concealed, queried where uncertain



Approximate scale

210 BAYPOINTE PARKWAY
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 04/12/22 | Project No. 22-2192 | Figure 3

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

EXPLANATION

Strike slip

Thrust (Reverse)

N

0 4 8 Miles

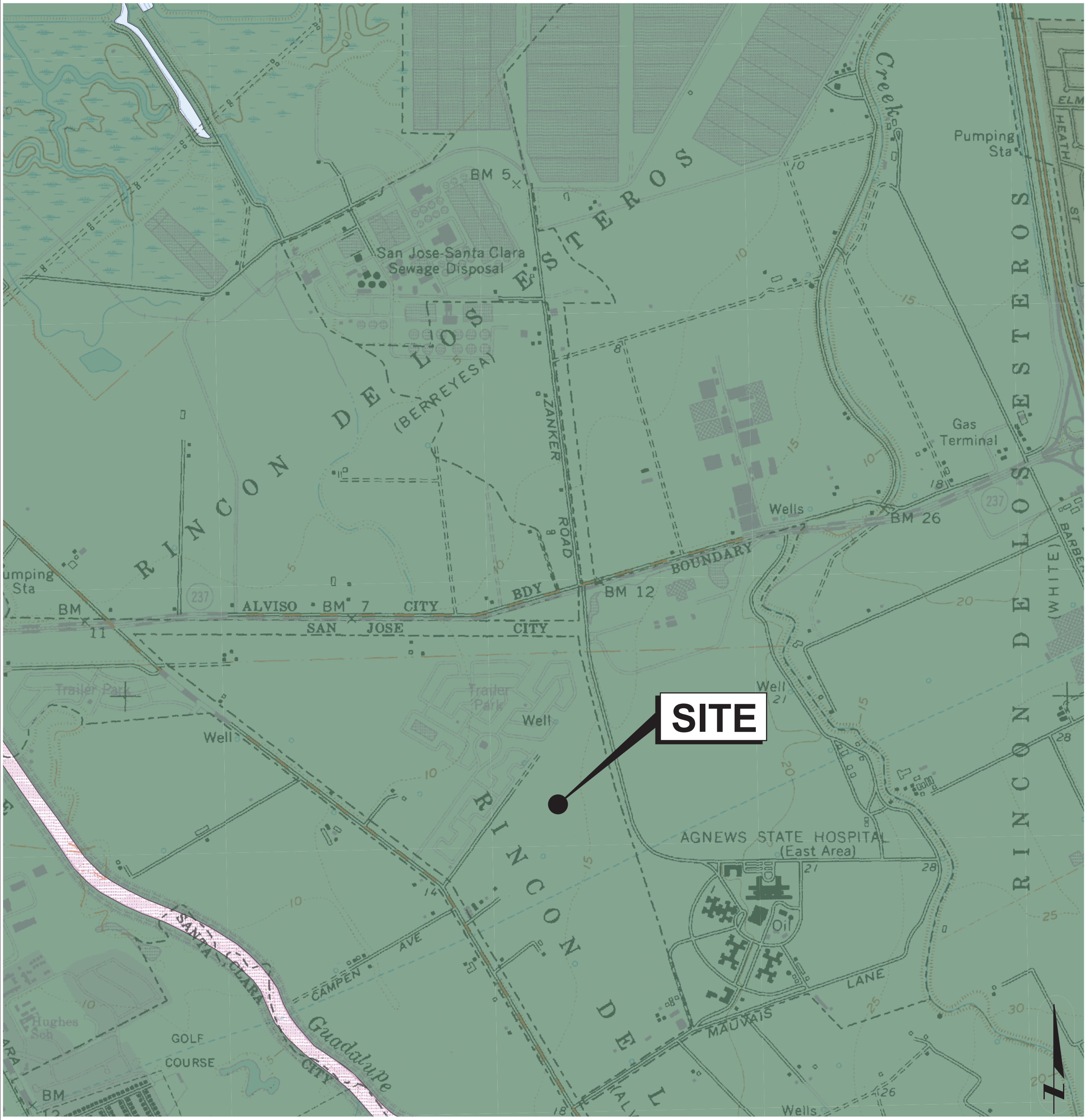
Approximate scale

210 BAYPOINTE PARKWAY
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 04/04/22	Project No. 22-2192	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
Milpitas Quadrangle
California Geological Survey
Released October 19, 2004

0 2,000 4,000 Feet
Approximate scale

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San Jose, California

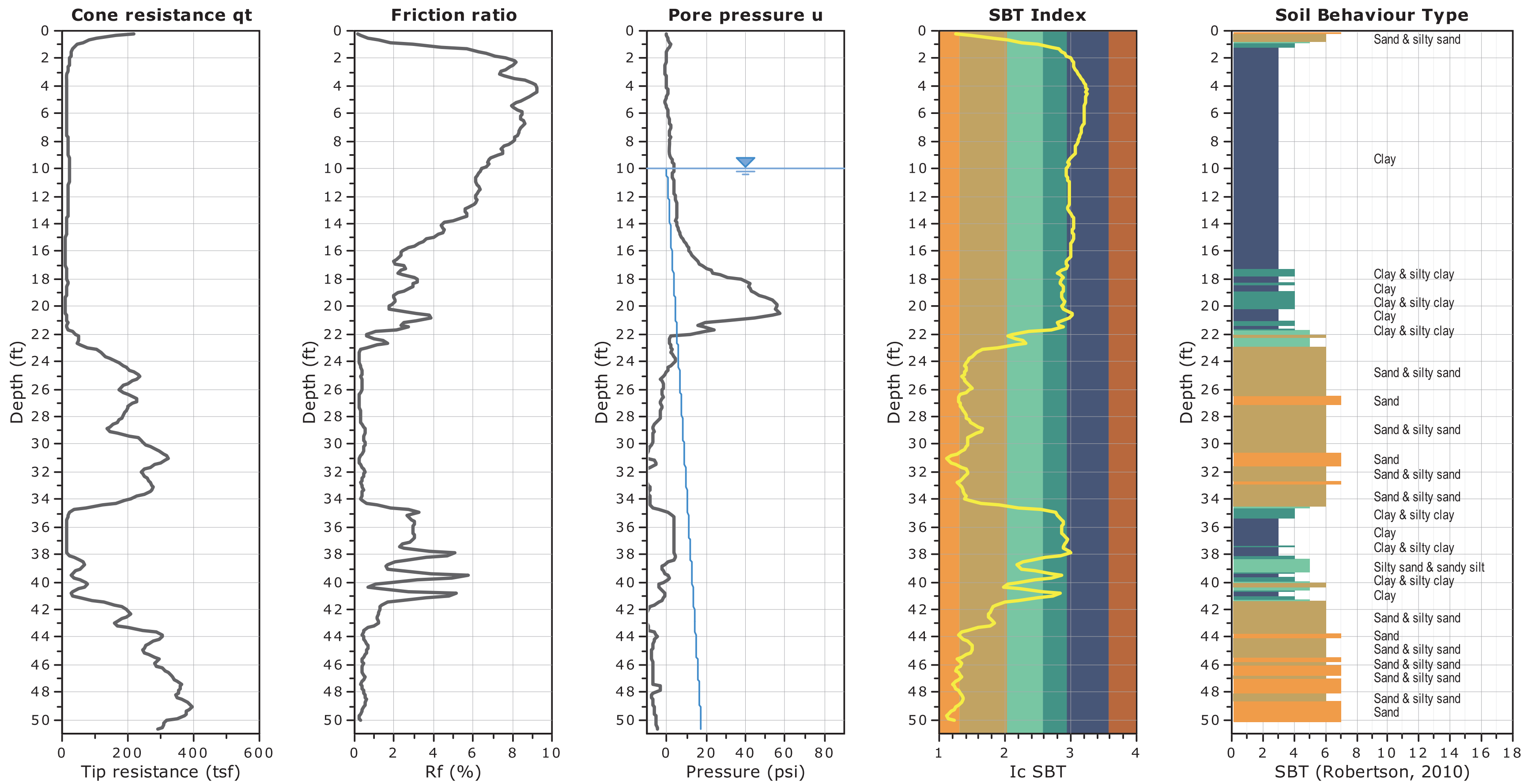
ROCKRIDGE
GEOTECHNICAL

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 04/04/22	Project No. 22-2192	Figure 5

APPENDIX A

Cone Penetration Test Results and Logs of Borings

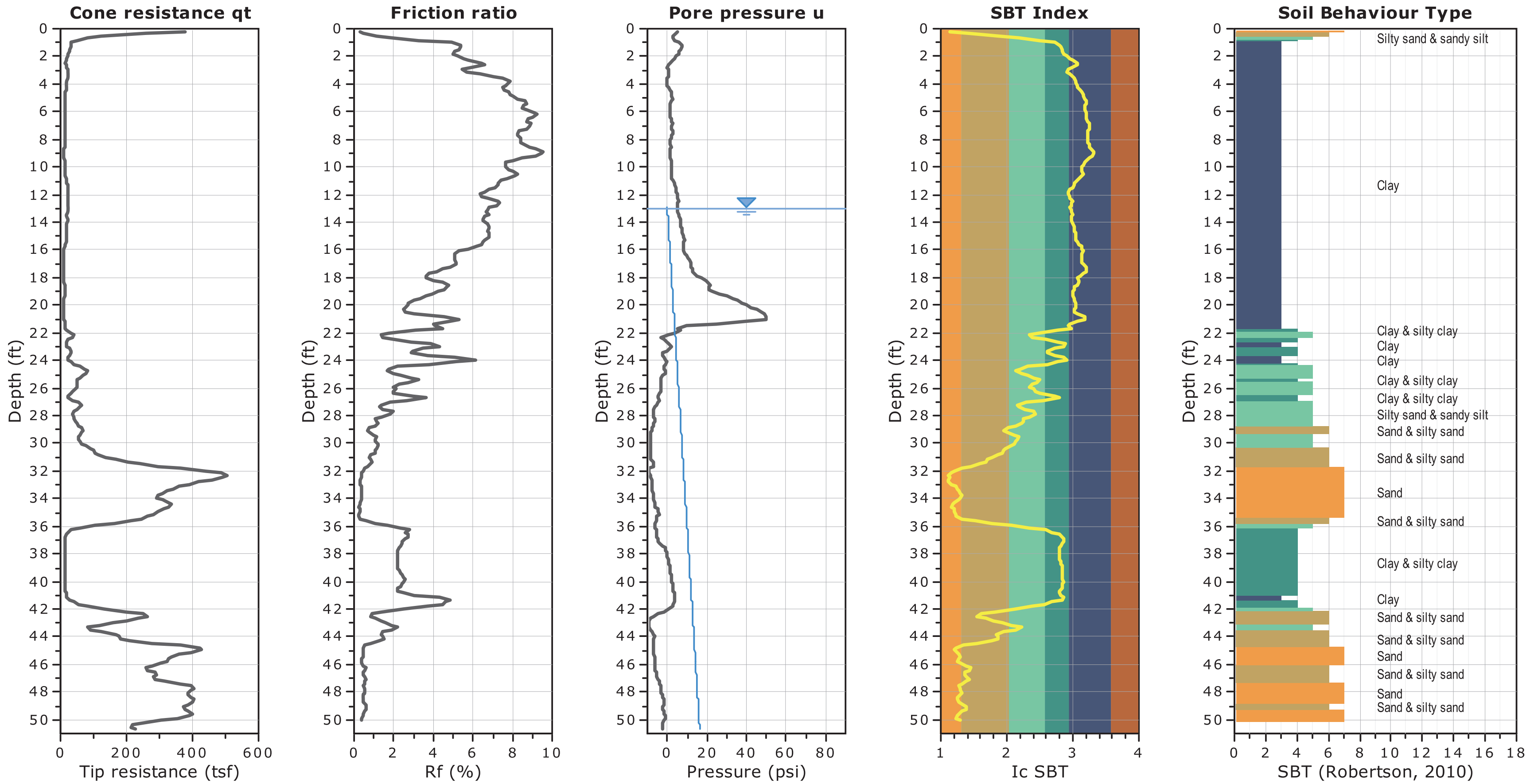


Total depth: 50.7 ft, Date: March 22, 2022
Depth to Groundwater: 10 feet (measured with drop tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

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

210 BAYPOINTE PARKWAY San Jose, California	**CONE PENETRATION TEST RESULTS** **CPT-1**
ROCKRIDGE GEOTECHNICAL	Date 04/12/22 | Project No. 22-2192 | Figure A-1



Total depth: 50.7 ft, Date: March 22, 2022
Depth to Groundwater: 13 feet (measured with drop tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

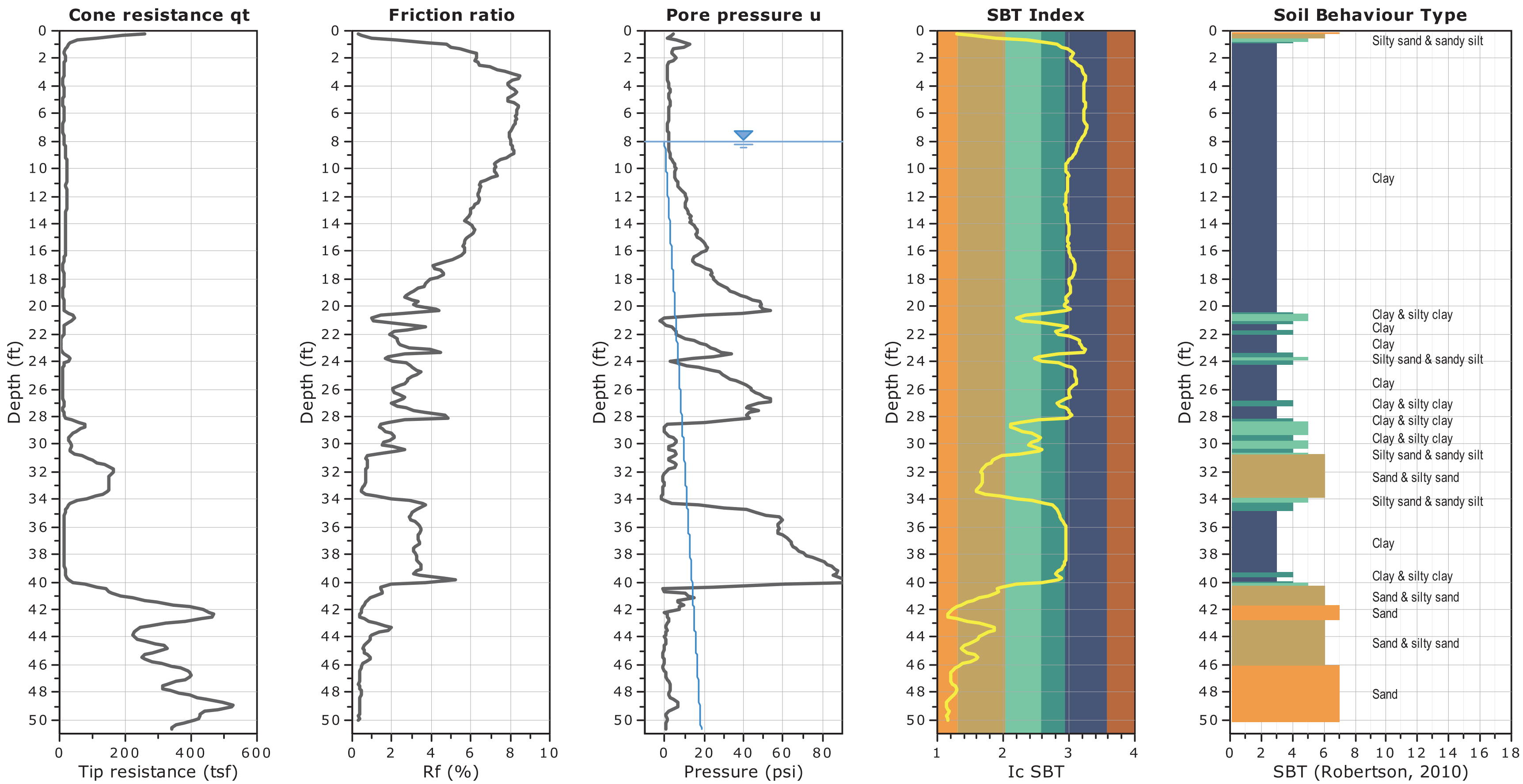
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

210 BAYPOINTE PARKWAY
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS CPT-2

Date 04/12/22	Project No. 22-2192	Figure A-2



Total depth: 50.7 ft, Date: March 22, 2022
Depth to Groundwater: 8 feet (measured with drop tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

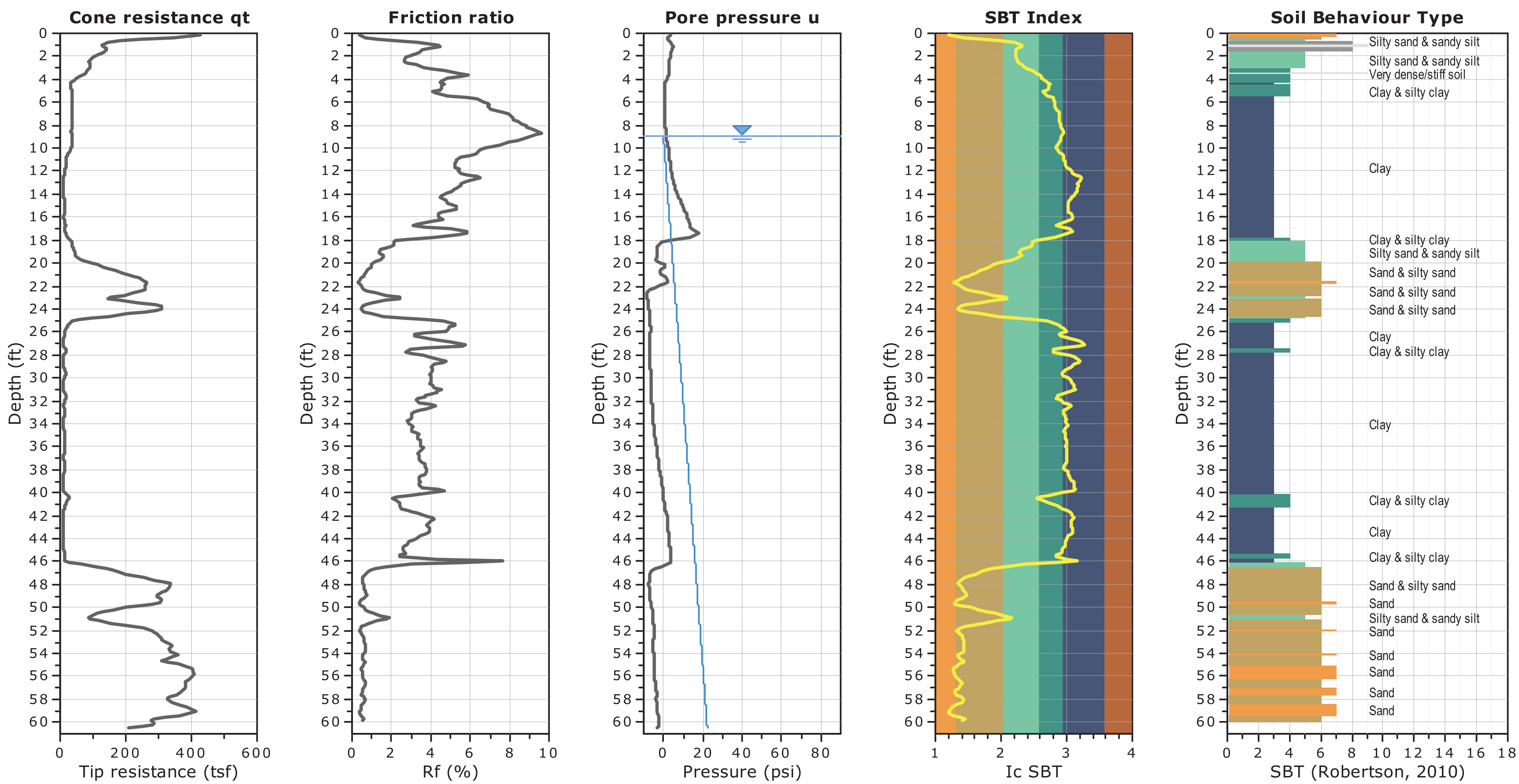
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

210 BAYPOINTE PARKWAY
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 04/12/22	Project No. 22-2192	Figure A-3



Total depth: 50.7 ft, Date: March 22, 2022
Depth to Groundwater: 9 feet (measured with drop tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

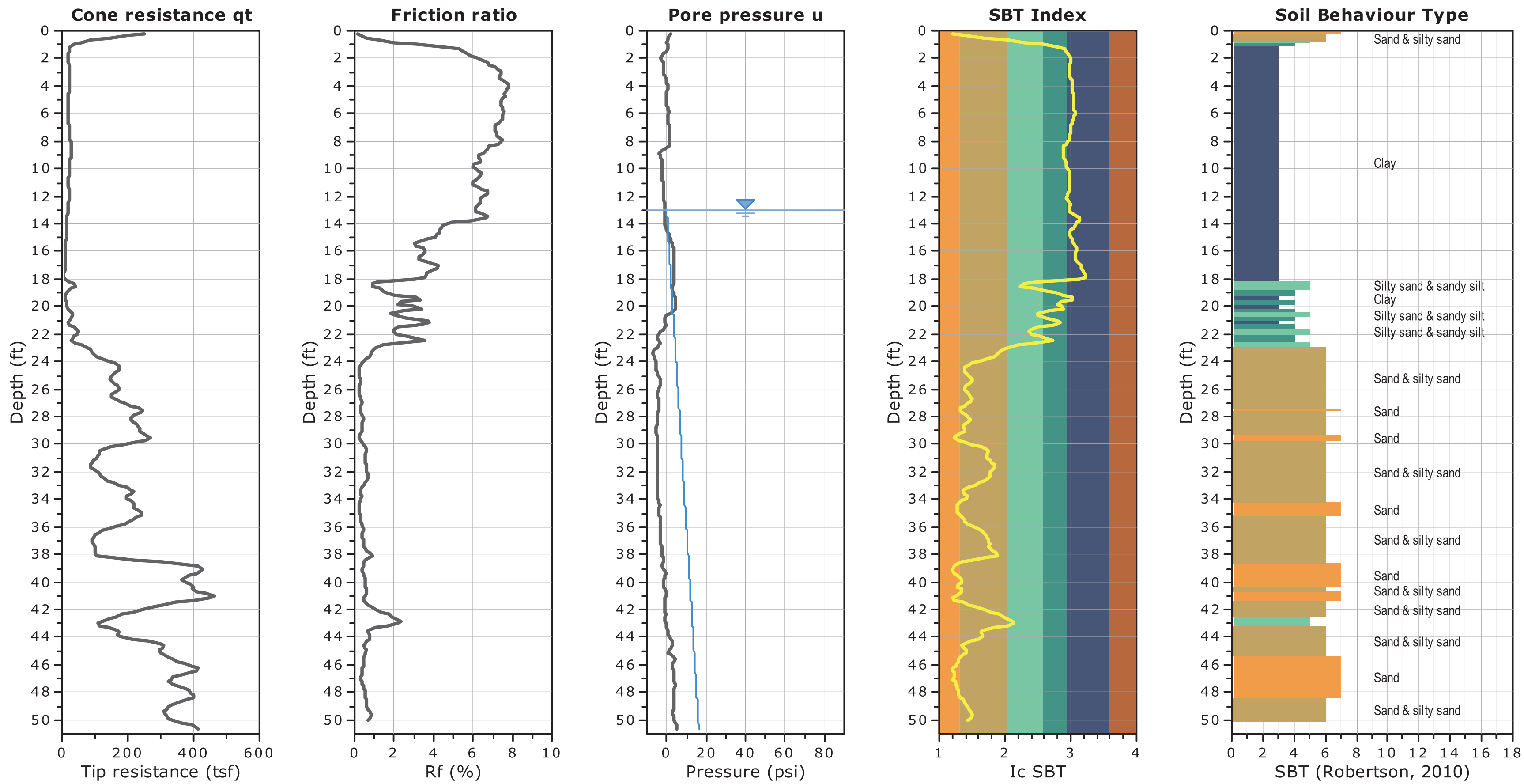
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

210 BAYPOINTE PARKWAY
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 04/12/22	Project No. 22-2192	Figure A-4



Total depth: 50.7 ft, Date: March 22, 2022
Depth to Groundwater: 13 feet (measured with drop tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

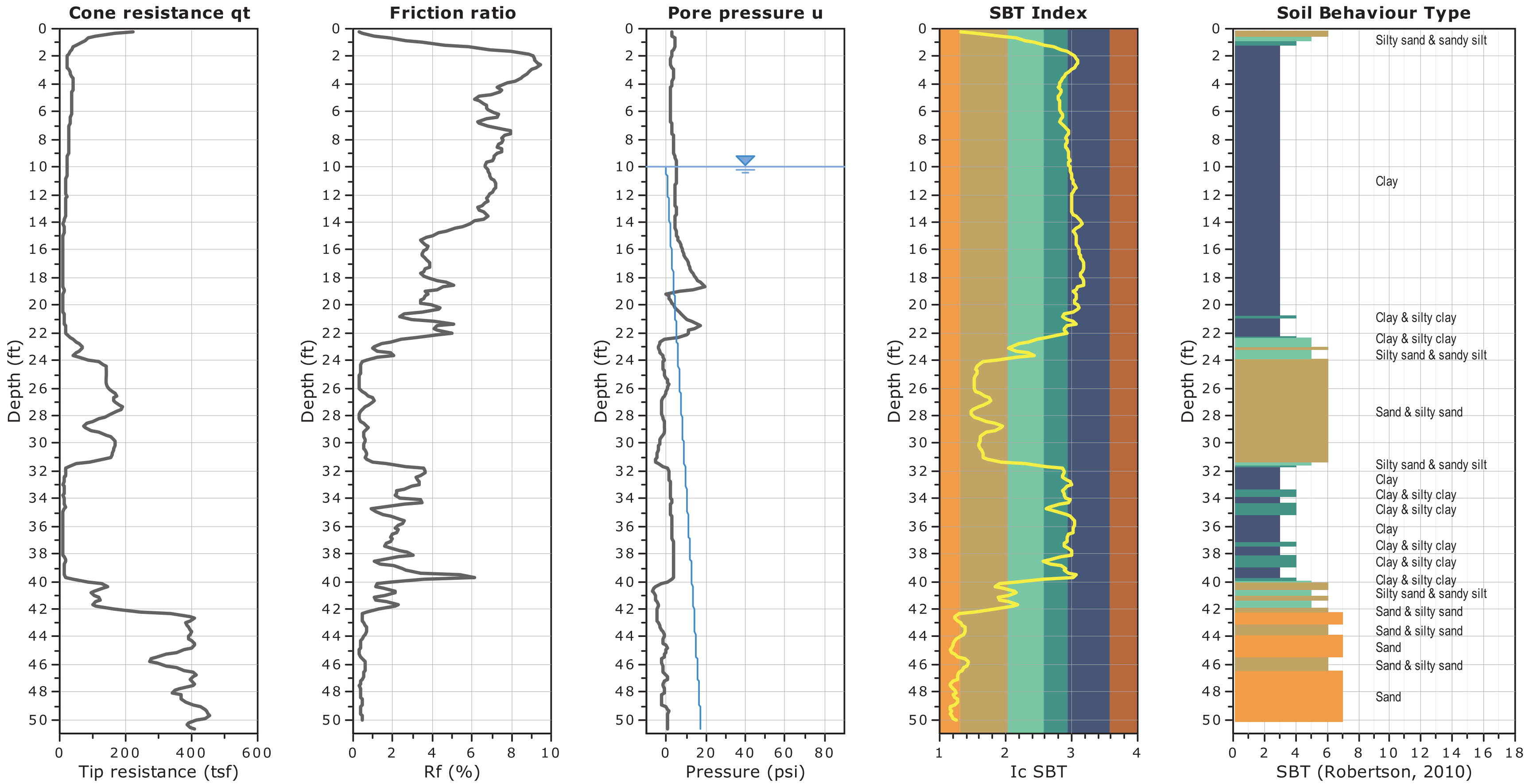
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

210 BAYPOINTE PARKWAY
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-5

Date 04/12/22	Project No. 22-2192	Figure A-5

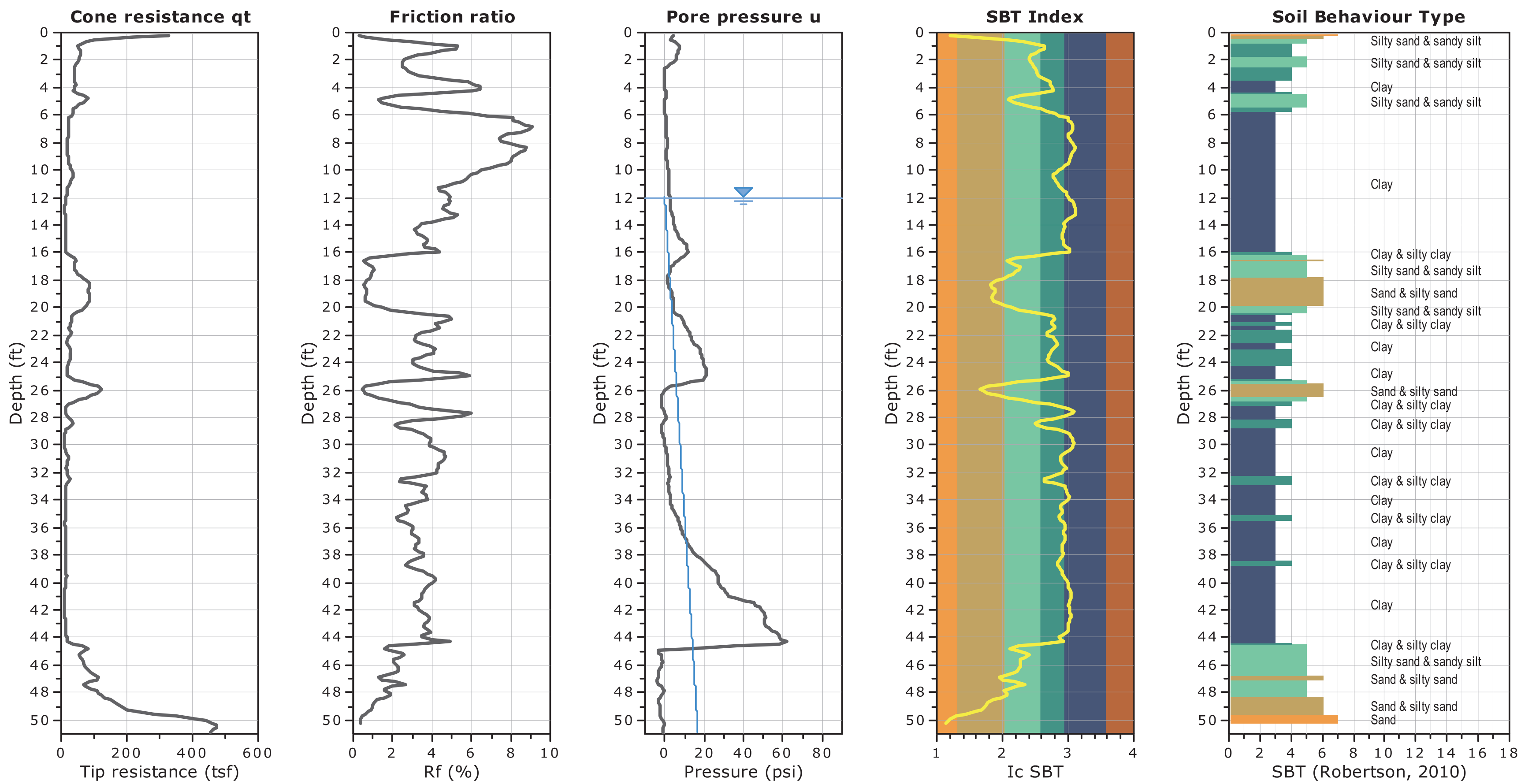


Total depth: 50.7 ft, Date: March 22, 2022
Depth to Groundwater: 10 feet (measured with drop tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

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

210 BAYPOINTE PARKWAY San Jose, California	**CONE PENETRATION TEST RESULTS** **CPT-6**
ROCKRIDGE GEOTECHNICAL	Date 04/12/22 · Project No. 22-2192 · Figure A-6



Total depth: 50.9 ft, Date: March 22, 2022
Depth to Groundwater: 12 feet (measured with drop tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

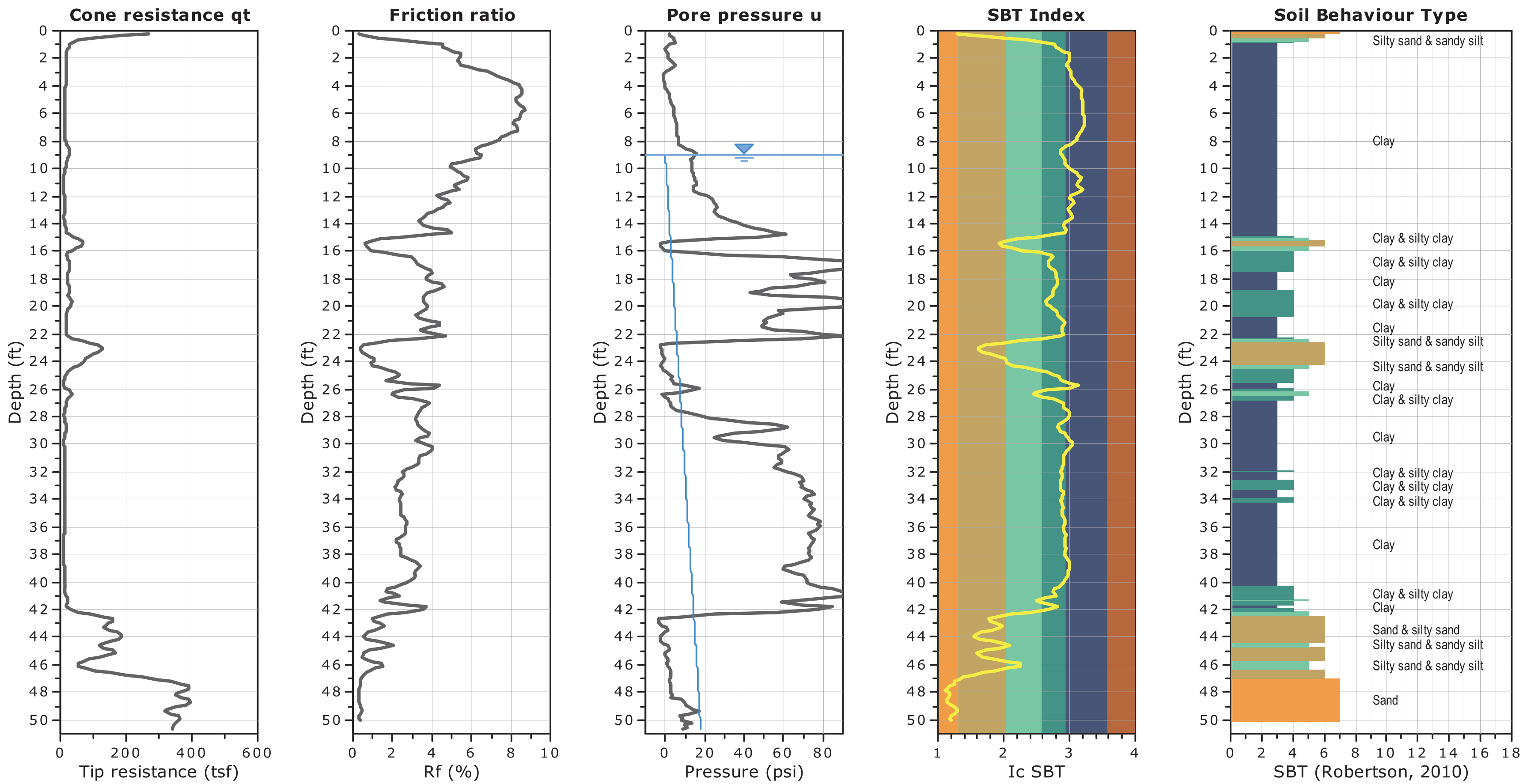
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravelly sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

210 BAYPOINTE PARKWAY
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-7

Date 04/12/22	Project No. 22-2192	Figure A-7



Total depth: 50.7 ft, Date: March 22, 2022
Depth to Groundwater: 9 feet (measured with drop tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

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

210 BAYPOINTE PARKWAY
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-8

Date 04/12/22	Project No. 22-2192	Figure A-8

PROJECT:	210 BAYPOINTE PARKWAY San Jose, California	Log of Boring HA-1 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2	Logged by: J. Lawton
Date started: 03/22/2022 — Date finished: 03/22/2022	
Drilling method: Hand-Auger	
Hammer weight/drop: N/A — Hammer type: N/A	
Sampler: Grab	

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value [1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						4 inches asphalt concrete						
						4 inches gravel						
1	GRAB	X			CH	CLAY with SAND (CH) dark brown, very stiff, dry, fine sand, roots						
						CLAY (CH) dark brown, very stiff, moist, rootlets						
2	GRAB	X				Soil Corrosivity Test; see Appendix B						
3					CH							
4												
5	GRAB	X				gray-brown, stiff LL = 64, PI = 45; see Appendix B				98	22.3	
6												
7												
8												
9												
10												

Boring terminated at a depth of 5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand-augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 22-2192

Figure: A-9

PROJECT:	210 BAYPOINTE PARKWAY San Jose, California	Log of Boring HA-2 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 03/22/2022 | Date finished: 03/22/2022

Drilling method: Hand-Auger

Hammer weight/drop: N/A | Hammer type: N/A

Sampler: Grab

Logged by: J. Lawton

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						2 inches asphalt						
						4 inches gravel						
1	GRAB	X			CH	CLAY with SAND (CH) dark brown, stiff to very stiff, moist, fine sand						
2	GRAB	X			CH	CLAY with SAND (CH) dark brown, stiff to very stiff, moist, fine sand						
3	GRAB	X			CH	CLAY with SAND (CH) dark brown, stiff to very stiff, moist, fine sand Soil Corrosivity Test; see Appendix B						
4	GRAB	X			CH	CLAY (CH) gray-brown, stiff to very stiff, moist						
5												
6												
7												
8												
9												
10												

Boring terminated at a depth of 5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand-augering.

ROCKRIDGE GEOTECHNICAL

Project No.: 22-2192 | Figure: A-10

PROJECT: **210 BAYPOINTE PARKWAY**
San Jose, California

Log of Boring HA-3

Boring location:	See Site Plan, Figure 2
Date started: 03/22/2022	Date finished: 03/22/2022
Drilling method:	Hand-Auger
Hammer weight/drop: N/A	Hammer type: N/A
Sampler:	Grab
Logged by:	J. Lawton

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						4 inches asphalt						
						4 inches gravel						
1	GRAB	☒			CL	SANDY CLAY with GRAVEL (CL) gray-brown, very stiff, dry, fine to medium sand, fine gravel						
2	GRAB	☒			CL	SANDY CLAY (CL) gray, stiff, moist LL = 46, PI = 28; see Appendix B					16.7	
3												
4												
5	GRAB	☒			CH	CLAY with SAND (CH) gray-brown, stiff, moist, fine to medium sand						
6												
7												
8												
9												
10												

Boring terminated at a depth of 5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand-augering.

ROCKRIDGE GEOTECHNICAL

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Figure: A-11

PROJECT:	210 BAYPOINTE PARKWAY San Jose, California	Log of Boring HA-4 PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: J. Lawton

Date started: 03/22/2022 — Date finished: 03/22/2022

Drilling method: Hand-Auger

Hammer weight/drop: N/A — Hammer type: N/A

Sampler: Grab

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						4 inches asphalt						
						6 inches gravel						
1	GRAB	X			CL	SANDY CLAY (CL) yellow-brown, very stiff, dry, fine sand, rootlets, trace coarse angular gravel						
2	GRAB	X				CLAY (CL-CH) olive-brown and yellow-brown, very stiff, moist, trace brick debris, trace gravel						
3	GRAB	X			CL-CH	gray LL = 50, PI = 30; see Appendix B				99	24.2	
4												
5	GRAB	X			CH	CLAY (CH) gray-brown, very stiff, moist, trace fine sand Soil Corrosivity Test; see Appendix B						
6												
7												
8												
9												
10												

Boring terminated at a depth of 5 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand-augering.

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Figure: A-12

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	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

- Sample taken with 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
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
MC	Modified California sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.38- or 1.5-inch inside diameter (refer to text)
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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CLASSIFICATION CHART

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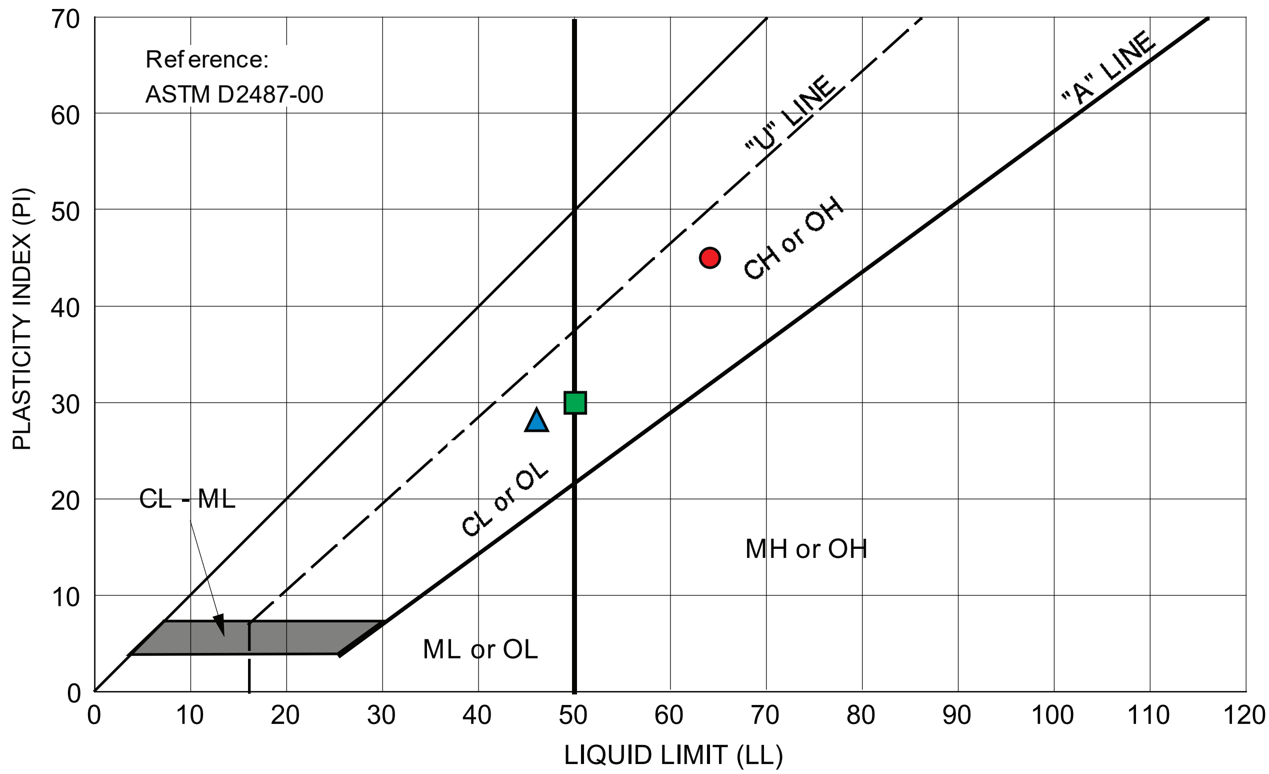
APPENDIX B

Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	HA-1 at 4.5 feet	CLAY (CH), gray-brown	22.3	64	45	98
▲	HA-3 at 2.0 feet	SANDY CLAY (CL), gray	16.7	46	28	--
■	HA-4 at 2.5 feet	CLAY (CL-CH), gray	24.2	50	30	99

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PLASTICITY CHART

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Project X
Corrosion Engineering
Corrosion Control – Soil, Water, Metallurgy Testing Lab

REPORT S220325D

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{-}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
HA-1: CLAY (CH), dark brown	1.5	173.0	0.0173	44.5	0.0045	2,814	1,474	6.2	141	0.39	0.5	2.0	0.04	156.0	14.0	1.7	0.5	5.2	3.3
HA-2: CLAY with SAND (CH), dark brown	2.5	237.5	0.0237	57.6	0.0058	1,541	1,072	8.1	157	0.06	0.7	0.5	0.03	140.9	28.2	3.6	3.0	4.6	0.2
HA-4: CLAY (CH), gray-brown	4.5	467.0	0.0467	92.4	0.0092	1,675	670	8.2	146	0.21	52.7	9.4	0.01	306.0	25.3	4.1	6.5	3.3	1.8

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)

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

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SOIL CORROSIVITY TEST RESULTS

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