ROCKRIDGE GEOTECHNICAL

Prepared for **Affirmed Housing**

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
3090 SOUTH BASCOM AVENUE
SAN JOSE, CALIFORNIA

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September 24, 2019
Project No. 19-1736

ROCKRIDGE GEOTECHNICAL

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Mr. Rob Wilkins
Regional Director, Northern California
Affirmed Housing
13520 Evening Creek Drive N, Suite 160
San Diego, California 92128

Subject: Geotechnical Investigation
Proposed Residential Building
3090 South Bascom Avenue
San Jose, California

Dear Mr. Wilkins:

We are pleased to present our geotechnical investigation report for the proposed development to be constructed at 3090 South Bascom Avenue in San Jose, California. Our services were provided in accordance with our proposal dated July 29, 2019.

The site is located on the southeast side of South Bascom Avenue between Camden and Foxworthy avenues. The parcel is irregularly shaped with maximum plan dimensions of about 150 by 210 feet, and a total area of 0.64 acres. The site is currently occupied by a two-story commercial building surrounded by asphalt-paved parking lots, drive aisles, and landscaped areas.

We understand Affirmed Housing is planning to purchase and develop the parcel. As currently envisioned, the development will consist of a 5- or 6-story apartment building. The building will consist of four levels of wood-framed construction (Type V) over one- or two-level concrete podium (Type 1) supported at grade. The planned ground-level finished floor elevation and structural loads are not known at this time.

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 concerns at the site are the presence of 3 to 7 feet of loose to medium dense material blanketing the site and providing adequate foundation support for the proposed building. Therefore, we conclude the proposed building should include foundations that gain support below the surficial loose layer. This can be achieved by either:

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- deepening the structural footings,

- over-excavating footings down to competent native material and backfilling with controlled density fill (CDF) or sand-cement slurry up to the design bottom-of-footing elevation, or

- over-excavating the surficial loose material across the entire site during mass grading of the building pad and re-compacting the material as a properly engineered fill.

The recommendations contained in our report are based on a limited subsurface exploration and laboratory testing program. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe excavation, grading, and foundation installation, 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.

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Enclosure

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GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
3090 SOUTH BASCOM AVENUE
San Jose, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed residential building to be constructed at 3090 South Bascom Avenue in San Jose, California. The parcel is irregularly shaped with maximum plan dimensions of about 150 by 210 feet, and a total area of 0.64 acres. It is bordered by South Bascom Avenue to the northwest, single story commercial buildings to the northeast and southwest, and single-family residential parcels to the east, as shown on Figure 1, Site Location Map and Figure 2, Site Plan. The site is currently occupied by a two-story commercial building surrounded by asphalt-paved parking lots, drive aisles, and landscaped areas. The ground surface elevation at the site is relatively flat.

We understand the proposed development consists of a 5- or 6-story apartment building which will be supported at grade. The building will consist of four levels of wood-framed construction (Type V) over one- or two-level concrete podium (Type 1).

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal dated July 29, 2019. The objective of our investigation was to evaluate subsurface conditions at the site and develop conclusions and recommendations regarding the geotechnical aspects of the proposed project. Our scope of services consisted of evaluating subsurface conditions at the site by drilling two exploratory borings, advancing six cone penetration tests (CPTs) and performing engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions beneath the site
- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of static and seismically induced foundation settlement
- subgrade preparation for floor slabs, pavements, and exterior concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- soil corrosivity
- 2016 and 2019 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations.

3.0 FIELD INVESTIGATION

Subsurface conditions at the site were investigated by drilling two borings, advancing six CPTs, and performing laboratory testing on select soil samples. Prior to our field investigation, we contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained Precision Locating, LLC, a professional utility locator, to check that the boring and CPT locations were clear of existing underground utilities. Details of the field investigation and laboratory testing are described below.

3.1 Cone Penetration Tests

CPT-1 through CPT-6 were advanced on August 5, 2019 by Middle Earth Geo Testing, Inc. (Middle Earth) of Orange, California. The approximate locations of the CPTs are shown on the Site Plan, Figure 2. The CPTs were advanced until practical refusal, which occurred between depths of 23 and 28 feet below ground surface (bgs).

The CPTs were performed using a truck-mounted rig hydraulically pushing a 1.7-inch-diameter cone-tipped probe into the ground. The probe measured tip resistance, pore water pressure, and frictional resistance on a sleeve behind the cone tip. Electrical sensors within the cone continuously measured these parameters for the entire depth advanced, and the readings were digitized and recorded by a computer. Accumulated data were processed by computer to provide engineering information such as soil behavior types, correlated strength characteristics, and estimated liquefaction resistance of the soil encountered. The CPT logs, showing normalized tip resistance, friction ratio, pore water pressure, and soil behavior type, are attached in Appendix A. Upon completion, the CPT holes were backfilled with neat cement grout and the pavement was patched with cold-mix asphalt.

3.2 Test Borings

Subsurface conditions at the site were further explored by drilling two geotechnical borings to depths of about 20 and 44 feet bgs. The borings, designated B-1 through B-2, were drilled on August 23, 2019 by Exploration GeoServices of San Jose, California at the approximate locations on the Site Plan, Figure 2. Exploration GeoServices drilled the borings using a Mobile B-53 truck-mounted drill rig equipped with hollow-stem augers. During drilling, our field geologist logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The boring logs are presented in Appendix B on Figures B-1 and B-2. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure B-3.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter stainless steel tubes
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. The S&H and SPT samplers were driven with a 140-pound, downhole, wireline hammer falling about 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every six inches and are presented on the boring logs. A “blow count” is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.6 and 1.0, respectively, to account for sampler type, and approximate hammer energy. The blow counts used for this conversion were the last two blow counts. The converted SPT N-values are presented on the boring logs.

Upon completion, the borings were backfilled with cement grout and the pavement surface was patched with quickset concrete.

3.3 Laboratory Testing

We re-examined the soil samples obtained from our borings to confirm the field classifications and selected representative samples for laboratory testing. Geotechnical laboratory tests were performed on soil samples to assess their engineering properties and physical characteristics. Soil samples were tested by B. Hillebrandt Soils Testing, Inc. of Alamo, California to measure moisture content, particle size distribution, and fines content. Corrosivity testing of two samples of near-surface soil was performed by Project X Corrosion of Murrieta, California. The results of the laboratory tests are presented on the boring logs and on Figure B-4 and B-5.

4.0 SUBSURFACE CONDITIONS AND SITE GEOLOGY

This section summarizes subsurface conditions at the site based on available geologic data from others and subsurface information from this investigation.

4.1 Subsurface Soil Conditions

As presented on Figure 3, the Regional Geologic Map, the site is mapped in a zone of alluvial deposits (Qpa) of the Pleistocene epoch (2.6 million to 11,000 years ago) (Graymer, 2006).

Based on the results of our geotechnical investigation, we conclude that the site is generally underlain by a layer of fill and/or geologically young material which varies between 3 and 7 feet thick. The where encountered in our investigation, the material consists of silty sand with gravel and has a consistency of loose to medium dense. This surficial layer is underlain by dense to very dense sands and gravels to the maximum depth explored of 44 feet bgs.

4.2 Groundwater Conditions

Groundwater was not encountered during our investigation. According to the document titled *Seismic Hazard Zone Report for the San Jose West 7.5-Minute Quadrangle, Santa Clara County, California,* prepared by the California Geological Survey (CGS) and dated 2002, the historic high groundwater level at the site is deeper than 50 feet bgs.

To help estimate the highest potential groundwater level at the site, we reviewed information on the State of California Water Resources Control Board GeoTracker website (http://geotracker.waterboards.ca.gov/). The closest monitoring well with groundwater data on the GeoTracker website is the Valero gas station at the intersection of South Bascom and Camden avenues, approximately 200 feet west of the site. The groundwater level at this well was measured semi-annually between 2017 and 2018. Measured groundwater levels ranged from 95 to 112 feet bgs.

5.0 SEISMIC CONSIDERATIONS

The San Francisco Bay Area is considered to be one of the more seismically active regions in the world. We evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[1] lateral spreading,[2] cyclic densification[3]. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity and Faulting

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

The major active faults in the area are the Monte Vista-Shannon, San Andreas, and Hayward faults. These and other faults of the region are shown on Figure 4. The fault systems in the Bay Area consist of several major right-lateral strike-slip faults that define the boundary zone between the Pacific and the North American tectonic plates. Numerous damaging earthquakes have occurred along these fault systems in recorded time. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated mean

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

2 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.

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

characteristic moment magnitude[4] [Working Group on California Earthquake Probabilities (WGCEP, 2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1

Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista-Shannon	3.1	Southwest	6.50
N. San Andreas - Peninsula	11	Southwest	7.20
N. San Andreas (1906 event)	11	Southwest	8.05
N. San Andreas - Santa Cruz	12	Southwest	7.12
Zayante-Vergeles	20	South	7.00
Total Calaveras	21	East	7.03
Total Hayward	22	Northeast	7.00
Total Hayward-Rodgers Creek	22	Northeast	7.33
San Gregorio Connected	36	West	7.50
Monterey Bay-Tularcitos	42	Southwest	7.30
Greenville Connected	44	Northeast	7.00

In the past 200 years, four major earthquakes (i.e., Magnitude > 6) have been recorded on the San Andreas fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) Intensity Scale occurred east of Monterey Bay on the San Andreas fault (Toppozada and Borchardt, 1998). The estimated moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred on the Peninsula segment of the San Andreas fault. Severe shaking occurred with an MM of about VIII-IX, corresponding to an M_w of about 7.5.

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

The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of October 17, 1989 had an M_w of 6.9 and occurred about 26 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 ($M_w = 6.2$).

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

5.2 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong ground 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, lateral spreading, and cyclic densification. We used the results of the borings and CPTs to evaluate the potential of these phenomena occurring at the project site.

5.2.1 Ground Shaking

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and 4) subsurface conditions. The site is approximately 3 kilometers from the Monte Vista-Shannon fault and 11 kilometers from the San Andreas fault. Therefore, the potential exists for a large earthquake to induce strong to very strong ground shaking at the site during the life of the project.

5.2.2 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction. The site is not in a mapped liquefaction hazard zone, as shown on Figure 5 from the map titled *State of California, Seismic Hazard Zones, San Jose West Quadrangle, Official Map*, dated February 7, 2002 and prepared by the California Geological Survey (CGS).

Considering the historic high groundwater depth is greater than 50 feet bgs and the soil beneath the site is geologically old, we conclude the potential for liquefaction-induced damage to the proposed development is very low. We also conclude the risk of lateral spreading and other types of ground failure associated with liquefaction occurring at the site is very low.

5.2.3 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 loose to medium dense material in the upper 3 to 7 feet of the site is potentially susceptible to cyclic densification. We estimate up to 1/2 inch of ground surface settlement where this layer is not improved or removed, as discussed further in Section 6.1.

5.2.4 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.

6.0 DISCUSSION AND CONCLUSIONS

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 concerns at the site are the presence of 3 to 7 feet of loose to medium dense material blanketing the site and providing adequate foundation support for the proposed building. These and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

6.1 Foundations and Settlement

The site is blanketed by a surficial layer of loose to medium dense silty sand and sandy silt that is about 3 to 7 feet thick, which could potentially result in erratic foundation performance if the building is supported on shallow foundations. Therefore, we conclude the proposed building should include foundations that gain support below the surficial loose layer. This can be achieved by deepening the structural footings, or alternatively, footing excavations may be over-excavated down to competent native material and backfilled with controlled density fill (CDF) or sand-cement slurry up to the design bottom-of-footing elevation. The CDF would serve to transfer footing loads to the denser material and prevent the need for extending reinforced structural concrete down to native material.

An alternative to the deepened footing and CDF options presented above would be to over-excavate the surficial loose material across the entire site during mass grading of the building pad and re-compact the material as a properly engineered fill. This would eliminate the need for deepened foundation support, as discussed above. If this option is implemented, the upper 5 feet of existing undocumented fill should be removed and replaced as a properly engineered fill following scarification and recompaction of the base of the overexcavation.

We conclude the medium dense to very dense gravel and sand with varying clay content below the surficial loose material is capable of supporting the proposed structure on a shallow foundation system, such as conventional spread footings, without excessive settlement. Our settlement analyses indicate total and differential settlement of conventional footings bearing on undisturbed native soil or properly engineered fill, designed using the allowable bearing pressures presented in Section 7.2 of this report, will be less than about 3/4 inch and 1/2 inch over a 30-foot horizontal distance, respectively.

6.2 Construction Considerations

The soil to be excavated consists primarily of sand, silt, and gravel, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. If concrete debris or former foundation elements are encountered during grading, removal will require equipment capable of breaking concrete, such as a hoe-ram.

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. We judge temporary slopes with a maximum inclination of 1.5:1 (horizontal to vertical) should be stable, provided the slope is not surcharged by adjacent structures, construction equipment, or stockpiled soil.

6.3 Soil Corrosivity

Corrosivity testing was performed by Project X Corrosion of Murrieta, California on soil samples obtained from the near-surface soil during our field investigation from locations B-1 and B-2 at a depth of 3.5 feet bgs. The test results are presented on Figure B-5. Based on the resistivity test results, the soil samples are classified as mildly corrosive to buried steel. Based on the chloride, sulfide, and sulfate ion concentrations, and pH test results, the soil samples do not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures. Accordingly, based on the resistivity test results, any buried iron, steel, cast iron, galvanized steel, and dielectric-coated steel or iron should be properly protected against corrosion.

7.0 RECOMMENDATIONS

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

7.1 Site Preparation and Grading

Site demolition should include the removal of existing structures, pavements, underground utilities, and foundations. In addition, any on-site areas that will receive surface improvements, such as asphalt pavement or concrete flatwork, should be overexcavated one foot below existing grade and recompacted. The primary purpose of the overexcavation beneath surface improvements is to check for loose fill and buried debris or foundations from previous site development or demolition.

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

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

7.1.1 Subgrade Preparation

After site clearing and demolition is completed, in areas that will receive improvements (i.e. building pad, pavement, and exterior concrete flatwork) or new fill, the soil subgrade exposed should be scarified to a depth of at least eight inches, moisture-conditioned, and compacted in accordance with the requirements presented below in Table 2 (Section 7.1.2).

If the over-excavation and re-compaction option is selected to mitigate the loose upper soils blanketing the site, as discussed in Section 6.1, the material should be removed down to at least 5 feet below existing grades, or to undisturbed native material, under the direction of our field engineer, moisture-conditioned, placed in 8-inch-thick lifts, and compacted in accordance with the recommendations in Table 2.

7.1.2 Fill Materials and Compaction Criteria

All fill should be placed in horizontal lifts not exceeding eight inches in loose thickness, moisture-conditioned, and compacted in accordance with the requirements provided below in Table 2. Each type of material is described in the following text according to its uses and specifications.

TABLE 2
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
Building pad – low-plasticity soil	90+	Above optimum
Exterior slabs – low-plasticity soil	90+	Above optimum
Pavements – low-plasticity soil	95+	Above optimum
Pavements - aggregate base	95+	Near optimum
General fill – low-plasticity soil	90+	Above optimum
General fill – granular soil	95+	Near optimum
Utility trench backfill – low-plasticity	90+	Above optimum
Utility trench - clean sand or gravel	95+	Near optimum

Note: Select fill and lime-treated clay are considered low-plasticity soil.

On-site Soil

On-site soil may be used as general fill, provided the material is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, and be approved by the Geotechnical Engineer.

Select Fill

Select fill should consist of on-site or imported 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. Samples of proposed select fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site.

The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not provided, a minimum of two weeks will be required to perform any necessary analytical testing.

Aggregate Base Material

Imported aggregate base material may be used as general fill, trench backfill (above bedding materials), or as select fill beneath building pad or exterior concrete flatwork. Aggregate base should meet the requirements in the 2015 Caltrans Standard Specifications, Section 26, for Class 2 aggregate base (3/4-inch maximum).

Controlled Low-Strength Material

Controlled low-strength material (CLSM) may be considered as an alternative to fill beneath the building, concrete flatwork, or pavement. CLSM should meet the requirements in the 2015 Caltrans Standard Specifications. It is an ideal backfill material when adequate room is limited or not available for conventional compaction equipment, or when settlement of the backfill must be minimized. No compaction is required to place CLSM. CLSM should have a minimum 28-day unconfined strength of 100 pounds per square inch (psi).

7.1.3 Utility Trench Backfill

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

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

Foundations for the proposed building should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal:vertical) inclination from the base of the utility trenches running parallel to the foundation. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with CLSM (see Section 7.1.2 for material requirements). If utility trenches are to be excavated below this zone-of-influence line after construction of the building foundations, the trench walls need to be fully supported with shoring until CLSM is placed.

7.1.4 Drainage and Landscaping

Positive surface drainage should be provided around the building to direct surface water away from foundations and below-grade walls. To reduce the potential for water ponding adjacent to the building, we recommend the ground surface within a horizontal distance of five feet from the building slope down away from the building with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundation and below-grade walls.

7.2 Foundation Design

Provided the estimated total and differential settlements presented in Section 6.1 are acceptable, the proposed building may be supported on spread footings that derive support on firm native soil below the undocumented fill blanketing the site. This can be achieved by either bottoming the footings below the fill, which typically extends 3 to 7 feet bgs, or by over-excavating the undocumented fill and placing CDF up to the design bottom-of-footing elevation. Alternatively, the footings may be supported on over-excavated and properly compacted fill, as described in Sections 6.1 and 7.1.

Where the perimeter footing is constructed near a bio-retention area or bio-swale, the footing should be founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal: vertical) from the base of the bio-retention area. Exterior perimeter footings should be bottomed at least 24 inches below the outside grade. Interior footings should be bottomed at least 24 inches below the bottom of the capillary moisture break.

Footings may be designed using an allowable bearing pressure of 4,000 pounds per square foot (psf) for dead-plus-live loads; this value may be increased by one-third for total design loads, which include wind or seismic forces. The recommended allowable bearing pressures for dead-plus-live loads and total loads include factors of safety of at least 2.0 and 1.5, respectively.

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 of footings, we recommend using an equivalent fluid weight (triangular distribution) allowable passive pressure of 250 pounds per cubic foot (pcf). Passive pressure in the upper one foot of soil should be neglected unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.3. The passive pressure and frictional resistance values have included a factor of safety of at least 1.5 and may be used in combination without reduction.

Footings should bottom on firm native alluvium or properly compacted fill. Where undocumented fill or other unsuitable bearing material is encountered at the bottom of footing excavations, the fill/unsuitable material should be removed and the overexcavation should be backfilled with lean concrete or CLSM.

7.3 Floor Slabs

The floor/garage slabs may consist of conventional slabs-on-grade. Where water vapor transmission through the floor slab is undesirable, we recommend installing a capillary moisture break and a water vapor retarder beneath the slab-on-grade. A vapor retarder and capillary moisture break are often not required beneath parking garage slabs because there is sufficient air circulation to allow evaporation of moisture that is transmitted through the slab; however, we recommend the vapor retarder and capillary break be installed below the slab-on-grade in utility rooms and any areas in or adjacent to the parking garage that will be used for storage and/or will receive a floor covering or coating. Where a capillary moisture break is not installed beneath the garage, the garage slab should be underlain by at least 4 inches of Class 2 AB compacted to at least 95 percent relative compaction.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. The particle size of the capillary break material should meet the gradation requirements presented in Table 3.

TABLE 3

Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6

The concrete slabs should be properly cured. Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the slabs should have a low w/c ratio - 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. Before floor coverings are placed on the mat or on slab-on-grade floors, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.4 Permanent Below-Grade Walls

Permanent below-grade walls, if required, should be designed to resist static lateral earth pressures, lateral pressures caused by earthquakes, and traffic loads (if vehicular traffic is expected within 10 feet of the wall). We recommend the permanent below-grade walls be designed for the more critical of the following criteria:

- At-rest equivalent fluid weight of 55 pounds per cubic foot (pcf).
- Active equivalent fluid weight of 35 pcf, plus a seismic increment of 25 pcf (triangular distribution)

The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads. Where the below-grade walls are subject to passenger vehicle traffic loading within 10 feet of the wall, an additional uniform lateral pressure of 50 psf, applied to the upper 10 feet of the wall, should be used.

The lateral earth pressures recommended are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. Although the basement walls will be well above the groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines, etc. One acceptable method for backdraining the wall is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the shoring or the back of the wall. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe at the base of the wall or just above the design groundwater level (whichever is higher). The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans 2015 Standard Specifications Section 68) or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140NC or equivalent). A proprietary, prefabricated collector drain system, such as Tremdrain Total Drain or Hydroduct Coil (or equivalent), designed to work in conjunction with the drainage panel may be used in lieu of the perforated pipe surrounded by gravel described above. The pipe should be connected to a suitable discharge point; a sump and pump system may be required to drain the collector pipes. We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use. To protect against moisture migration into the below-grade parking level, we recommend that the below-grade walls be water-proofed and water stops be installed at all construction joints.

As an alternative to installing a wall drainage system and sump, it may be more economical to design the below-grade walls for saturated earth pressures and omit the drainage system. Using this approach, we recommend the permanent below-grade walls be designed for the more critical of the following criteria:

- At-rest equivalent fluid weight of 90 pounds per cubic foot (pcf) plus a traffic increment where the wall will be within 10 feet of adjacent streets, or
- Active equivalent fluid weight of 81 pcf, plus a seismic increment of 11 pcf (triangular distribution)

If backfill is required behind below-grade walls prior to pouring the podium slabs, the walls should be temporarily braced and hand compaction equipment used in close proximity to the wall, to prevent unacceptable surcharges and potential deformation of the walls.

7.5 Seismic Design

We anticipate that the proposed building may be designed under either the 2016 or 2019 version of the California Building Code (CBC). For seismic design, we recommend Site Class D (non-default) be used. The latitude and longitude for the site are 37.2670° and -121.9398°, respectively. Recommended design parameters for each version of the code are presented in the following subsections.

7.5.1 2016 California Building Code

The 2016 CBC is based on the guidelines contained within *ASCE 7-10*. Hence, in accordance with the 2016 CBC, we recommend the following:

- S_S = 1.879g, S_1 = 0.643g
- F_a = 1.0, F_v = 1.5
- S_{MS} = 1.879, S_{M1} = 0.965g
- S_{DS} = 1.253g, S_{D1} = 0.643g
- Seismic Design Category D for Risk Categories I, II, and III.

7.5.2 2019 California Building Code

The 2019 CBC is based on the guidelines contained within *ASCE 7-16*. Assuming C_s will be calculated as outlined in *Section 11.4.8, Exception 2*, we recommend the following seismic design parameters:

- S_S = 1.963g, S_1 = 0.699g
- F_a = 1.0, F_v = 1.7
- S_{MS} = 1.963g, S_{M1} = 1.188g
- S_{DS} = 1.309g, S_{D1} = 0.792g
- Seismic Design Category D for Risk Factors I, II, and III

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

9.0 LIMITATIONS

This geotechnical study 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 test borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

REFERENCES

2016 California Building Code (CBC)

2019 California Building Code (CBC)

California Geological Survey (2008). Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A.

Cao, T., Bryant, W. A., Rowshandel, B., Branum D. and Wills, C. J. (2003). "The Revised 2002 California Probabilistic Seismic Hazard Maps"

Field, E.H., and 2014 Working Group on California Earthquake Probabilities, 2015, UCERF3: A new earthquake forecast for California's complex fault system: U.S. Geological Survey 2015-3009, 6 p.

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GeoLogismiki, (2019). CLiq, Version 3.0.

Jennings, C.W. (1994). Fault Activity Map of California and Adjacent Areas with Locations and Ages of Recent Volcanic Eruptions: California Division of Mines and Geology Geologic Data Map No. 6, scale 1: 750,000.

Pradel, D. (1998). Proceedure to Evaluate Earthquake-Induced Settlement in Dry Sand, Journal of Geotechncial and Geoenvironmental Engineering.

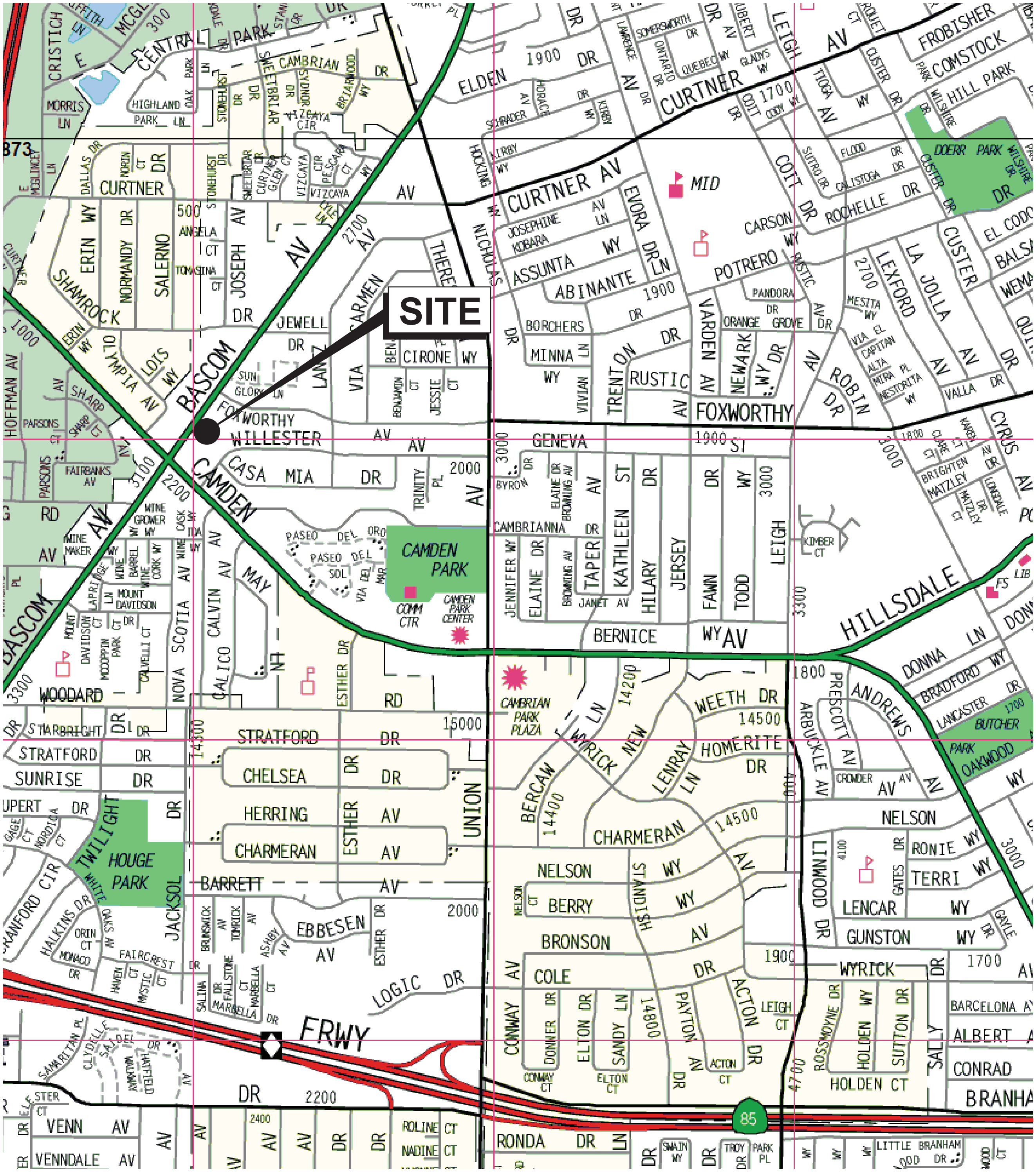
Robertson, P.K. (2009). "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009.

Robertson, P.K. (2009). "Interpretation of Cone Penetration Tests - A Unified Approach", Canadian Geotechnical Journal, Vol. 46, No. 11, pp 1337-1355.Toppozada, T.R. and Borchardt G. (1998). "Re-evaluation of the 1936 "Hayward Fault" and the 1838 San Andreas Fault Earthquakes." Bulletin of Seismological Society of America, 88(1), 140-159.

U.S. Geological Survey (USGS), (2008). The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): prepared by the 2007 Working Group on California Earthquake Probabilities, U.S. Geological Survey Open File Report 2007-1437.

ROCKRIDGE GEOTECHNICAL

FIGURES



Base map: The Thomas Guide
Santa Clara County
2002

0 1/4 1/2 Mile

Approximate scale

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE
GEOTECHNICAL

SITE LOCATION MAP

Date 08/13/19	Project No. 19-1736	Figure 1

EXPLANATION

B-1 — Approximate location of boring by Rockridge Geotechnical Inc., August 23, 2019

CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical Inc., August 5, 2019

Project limits

0 30 Feet

Approximate scale

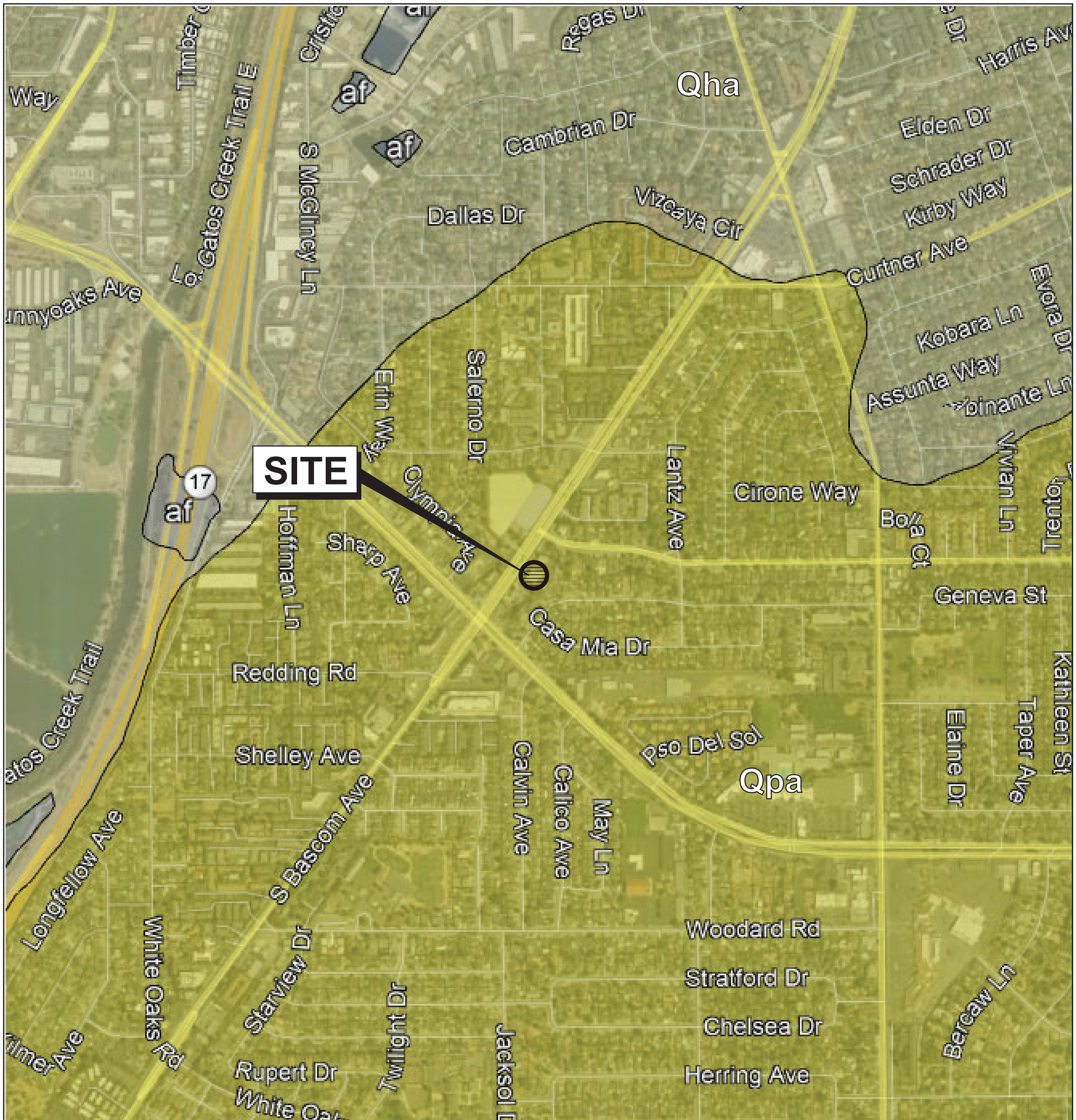
Base map: Google Earth, 2019.

3090 SOUTH BASCOM AVENUE
San Jose, California

SITE PLAN

Date 09/23/19	Project No. 19-1736	Figure 2

ROCKRIDGE GEOTECHNICAL



EXPLANATION

Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2018.

af	Artificial Fill
Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)

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

0 1000 2000 Feet

Approximate scale

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 08/13/19 | Project No. 19-1736 | Figure 3

Base Map: U.S. Geological Survey (USGS), National Seismic Hazards Maps - Fault Sources, 2008.

EXPLANATION

Strike slip

Thrust (Reverse)

Normal

0 5 10 Miles

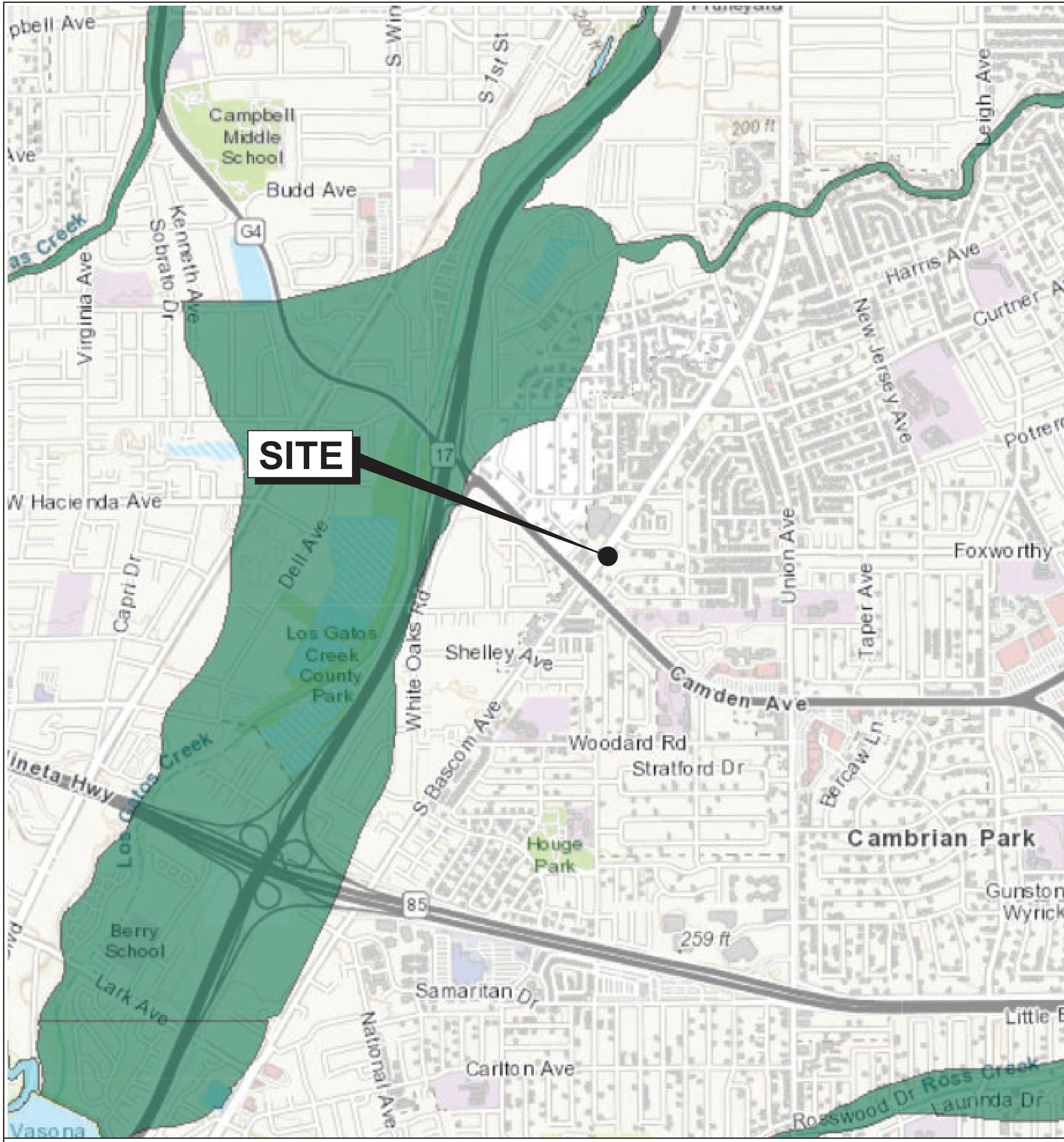
Approximate scale

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 08/13/19	Project No. 19-1736	Figure 4



Reference: https://maps.conservation.ca.gov/cgs/EQZApp/app/ (California Geological Survey, 2019)
San Jose West Quadrangle, Released February 7, 2002.

EXPLANATION

Liquefaction; Areas where historic occurence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

Earthquake-Induced Landslides; Areas where previous occurence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

0 0.4 0.8 Miles

Approximate scale

3090 SOUTH BASCOM AVENUE
San Jose, California

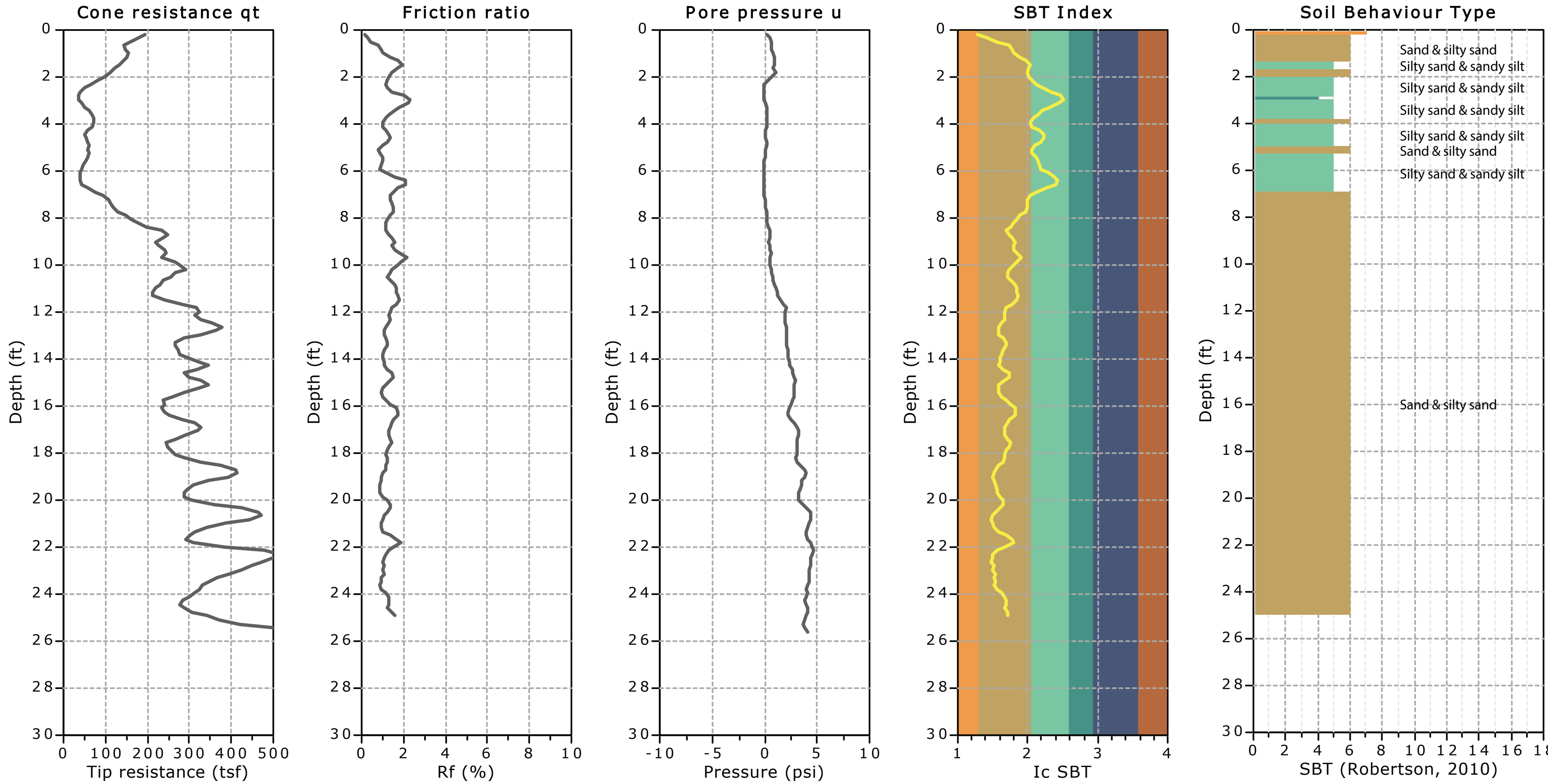
ROCKRIDGE GEOTECHNICAL

SEISMIC HAZARDS ZONE MAP

Date 08/13/19	Project No. 19-1736	Figure 5

APPENDIX A

Cone Penetration Test Results

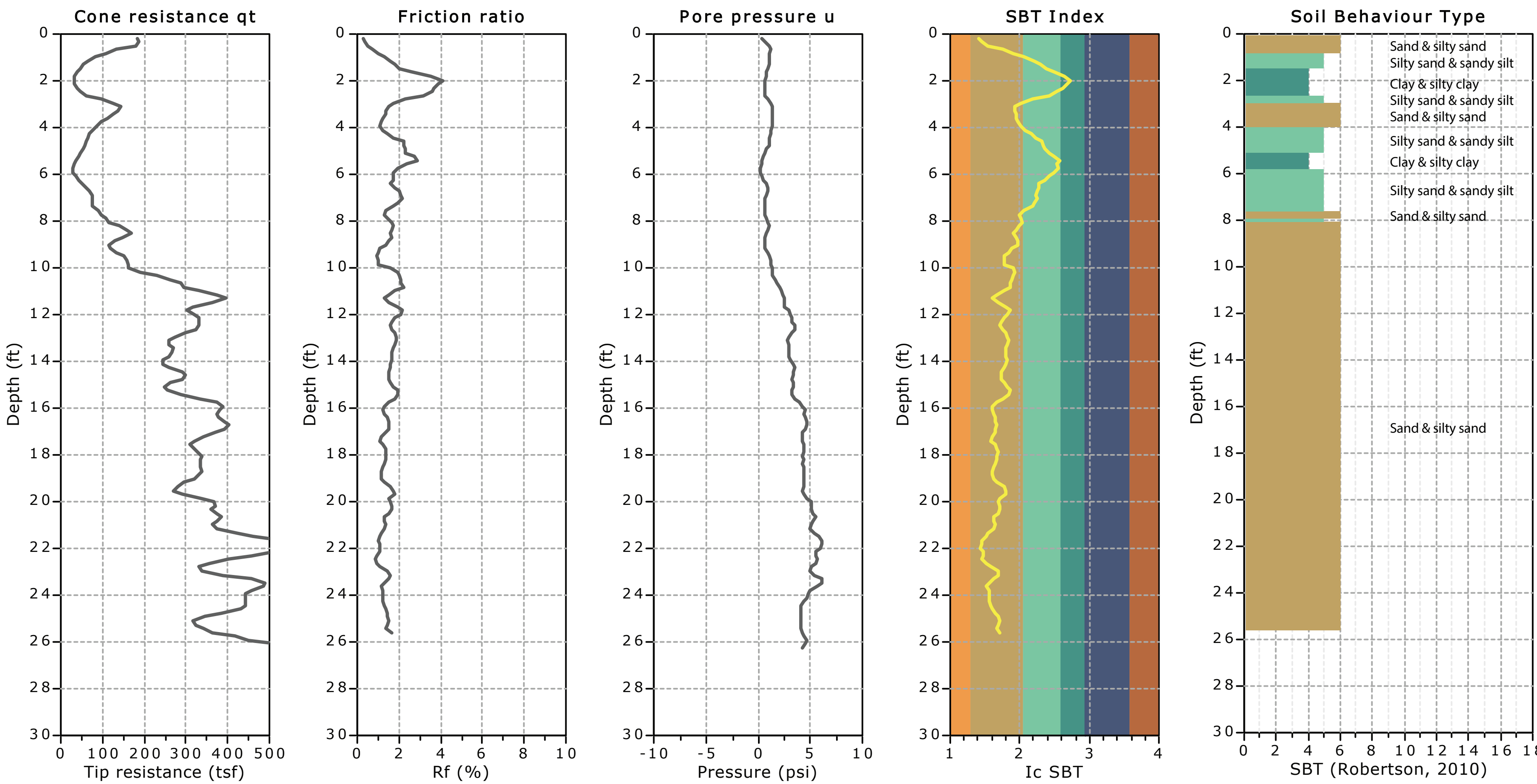


SBT legend

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

Total depth: 25.6 ft, Date: 8/5/2019
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

3090 SOUTH BASCOM AVENUE San Jose, California	**CONE PENETRATION TEST RESULTS** **CPT-1**
ROCKRIDGE GEOTECHNICAL	Date 08/13/19 · Project No. 19-1736 · Figure A-1



Total depth: 26.3 ft, Date: 8/5/2019
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

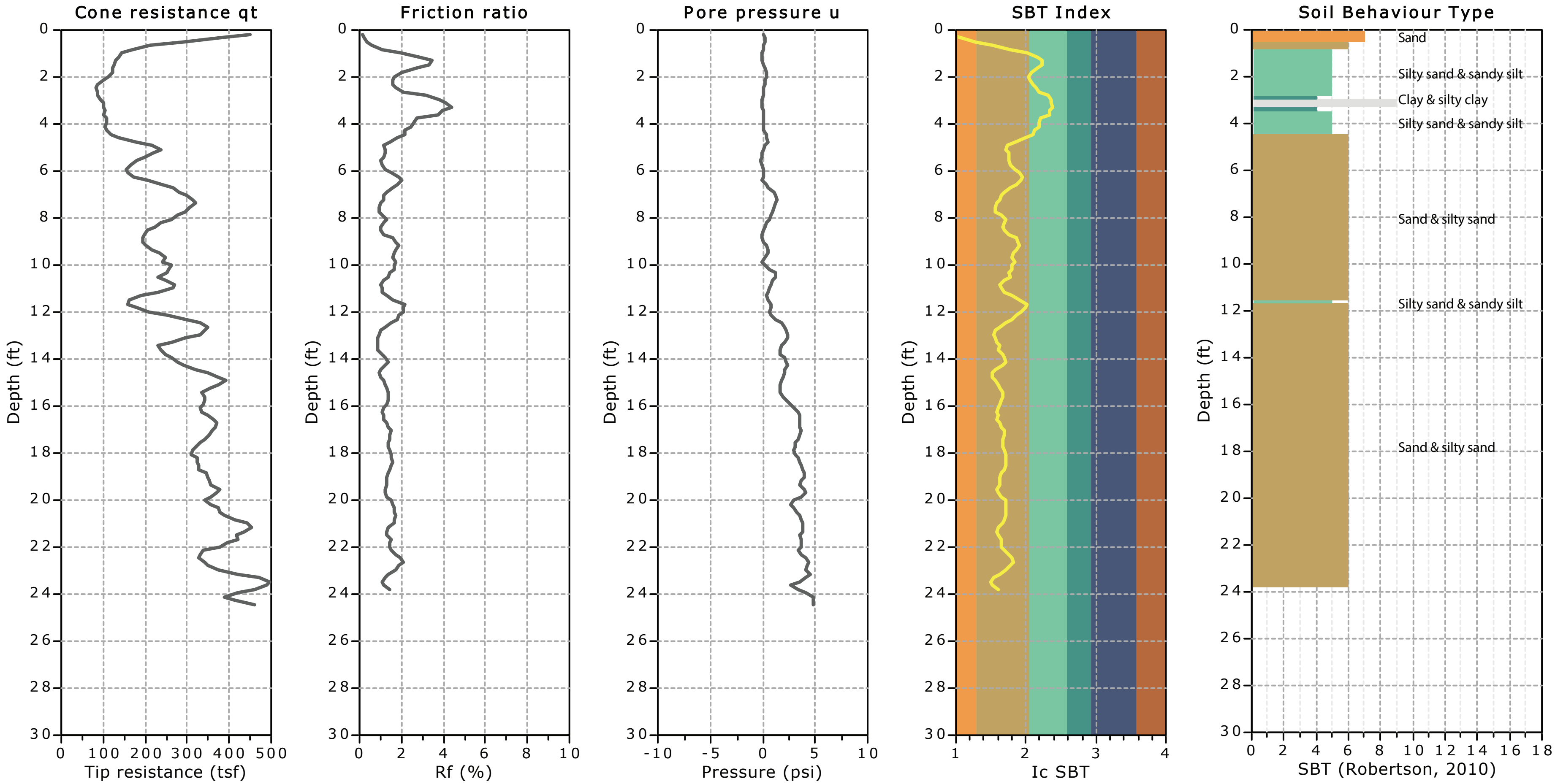
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 08/13/19	Project No. 19-1736	Figure A-2



SBT legend

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

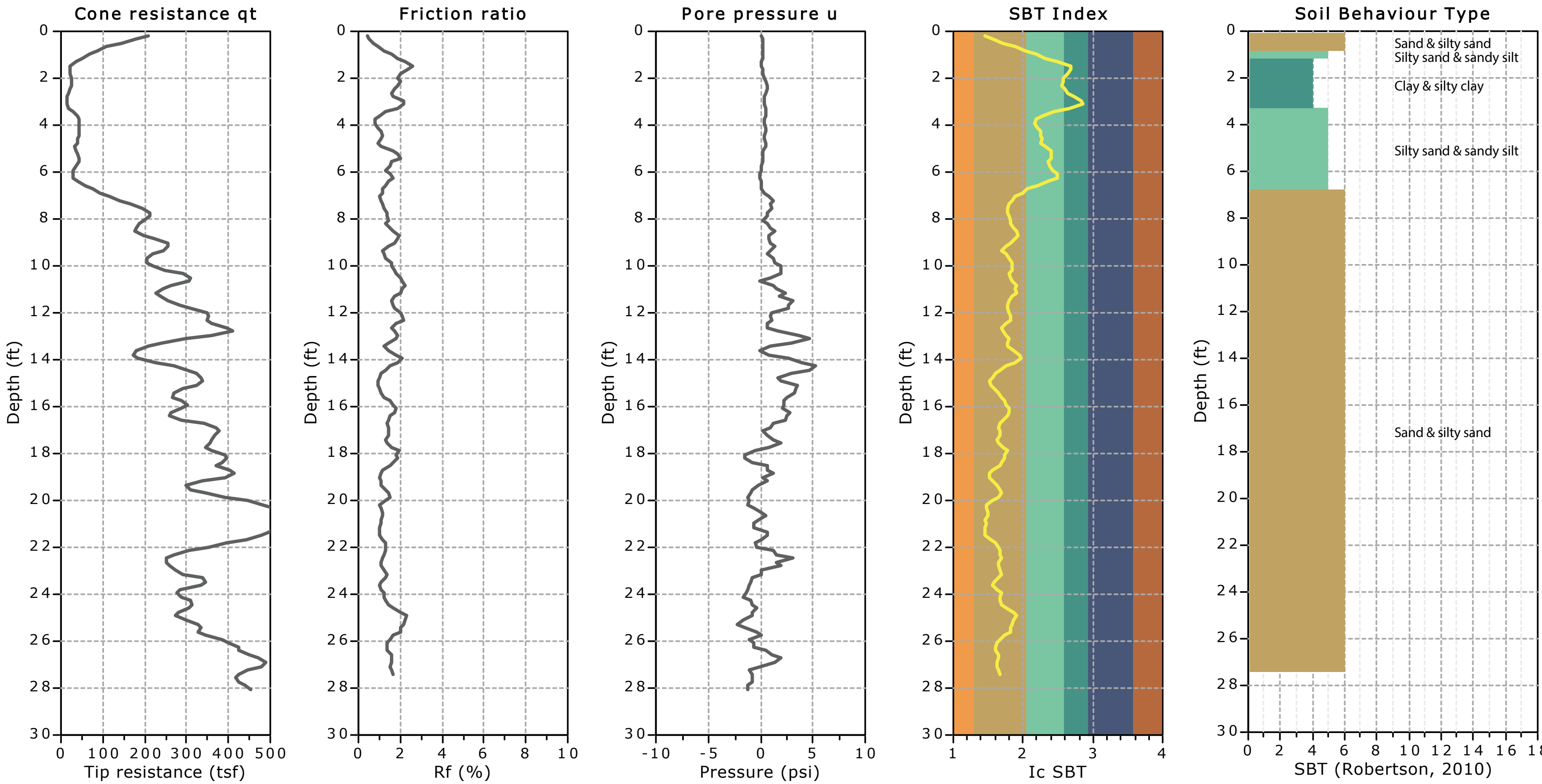
Total depth: 24.4 ft, Date: 8/5/2019
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 08/13/19 | Project No. 19-1736 | Figure A-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. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

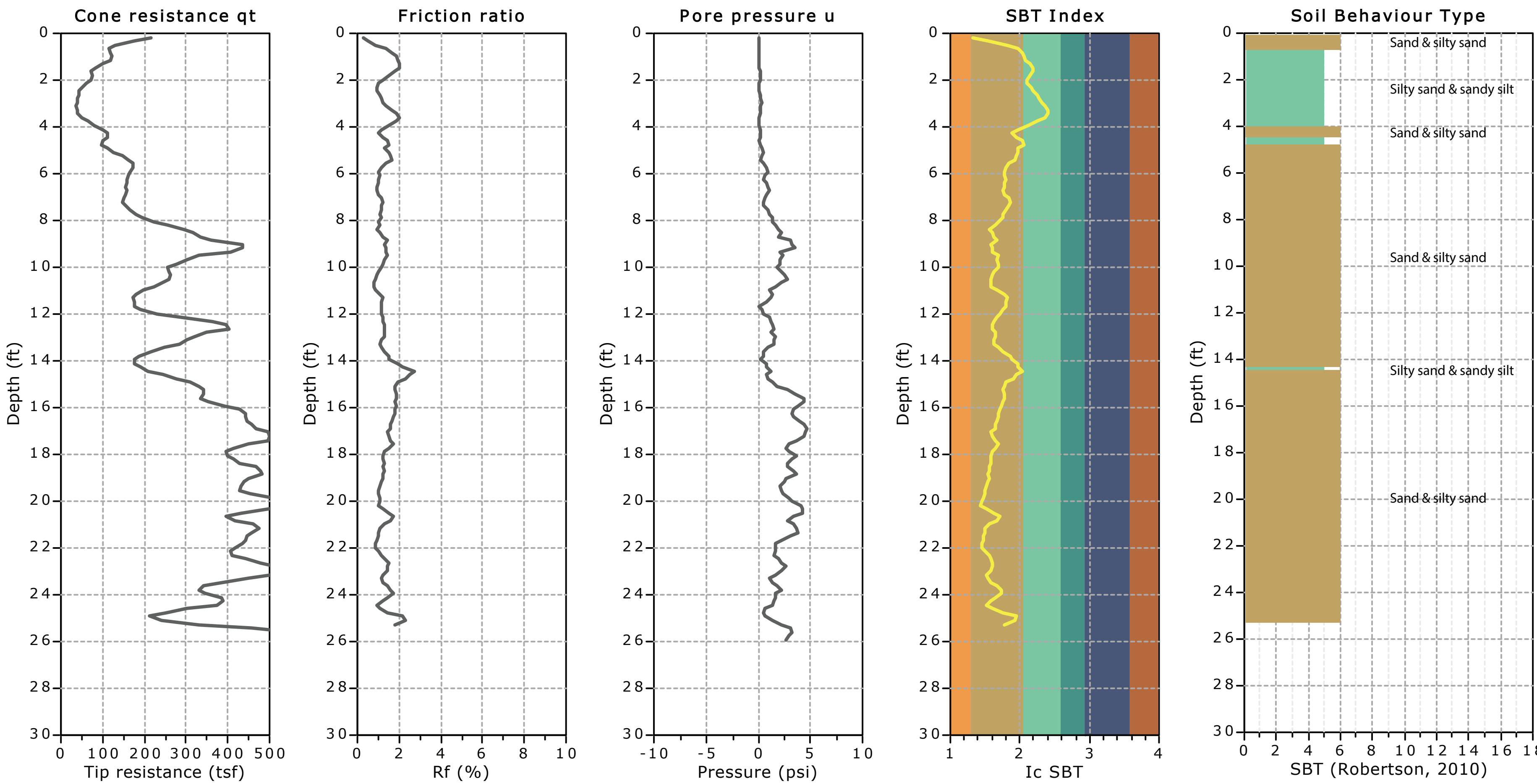
Total depth: 28.1 ft, Date: 8/5/2019
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 08/13/19 | Project No. 19-1736 | Figure A-4



SBT legend

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

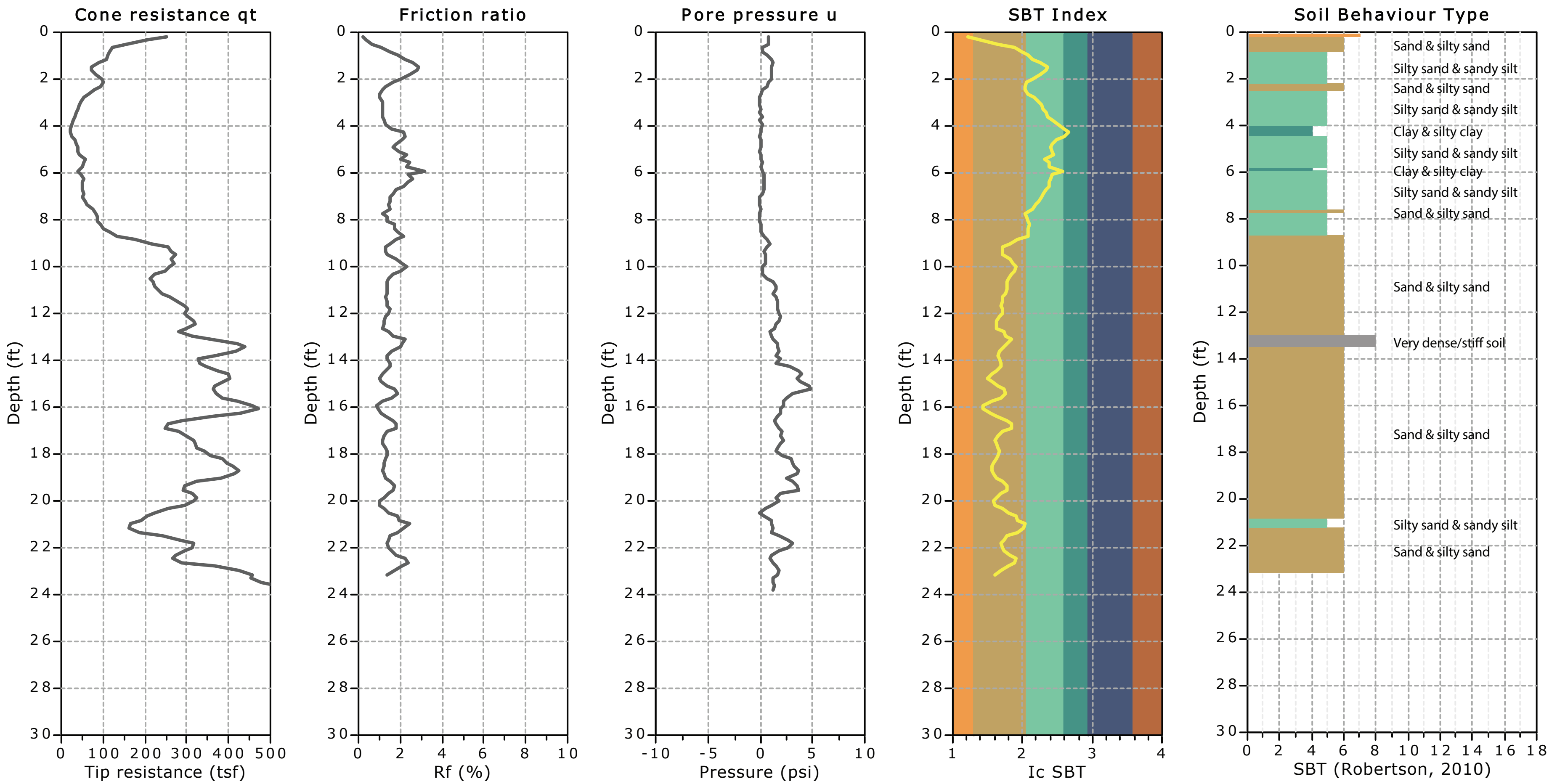
Total depth: 25.9 ft, Date: 8/5/2019
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-5

Date 08/13/19 | Project No. 19-1736 | Figure A-5



SBT legend

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

Total depth: 23.8 ft, Date: 8/5/2019
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS CPT-6

Date 08/13/19 | Project No. 19-1736 | Figure A-6

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APPENDIX B

Logs of Borings and Laboratory Test Results

PROJECT: **3090 SOUTH BASCOM AVENUE**
San Jose, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2
Date started: 8/23/19 Date finished: 8/23/19
Drilling method: Hollow-Stem Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Downhole Wireline
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: M. Hachey
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-53R

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2 inches of asphalt concrete						
					4 inches of aggregate base						
1–3					SILTY SAND with GRAVEL (SM) red-brown, very dense, dry, fine- to coarse-grained sand, fine to coarse gravel						
3	S&H	27 50/6"	30/6"		Particle Size Distribution; see Figure B-4 Corrosivity Tests; see Figure B-5				20	5.0	
5	S&H	23 50/3"	30/3"	SM							
8	SPT	17 20 20	40		yellow-brown, dense, increased gravel content						
11	SPT	22 31 44	75		very dense						
13				GM	SILTY GRAVEL with SAND (GM) yellow-brown, very dense, dry, fine to coarse gravel, fine- to coarse-grained sand						
14	SPT	34 32 22	54								
19	SPT	30 50/6"	50/6"								
23	SPT	50/6"	50/6"								
29	SPT	22 30 32	62		trace clay						
30											

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1736 Figure: B-1a

PROJECT:	3090 SOUTH BASCOM AVENUE San Jose, California	Log of Boring B-1 PAGE 2 OF 2

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
31					GM	SILTY GRAVEL with SAND (GM) (continued) yellow-brown, very dense, dry, fine to coarse gravel, fine- to coarse-grained sand						
33	SPT		50/5"	50/5"								
39	SPT		24 29 37	66		moist						
44	SPT		46 50/6"	50/6"								

Boring terminated at a depth of 44.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1736

Figure: B-1b

PROJECT: **3090 SOUTH BASCOM AVENUE**
San Jose, California

Log of Boring B-2

Boring location: See Site Plan, Figure 2

Date started: 8/23/19 Date finished: 8/23/19

Drilling method: Hollow Stem Auger

Hammer weight/drop: 140 lbs./30 inches Hammer type: Downhole Wireline

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: M. Hachey
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-53R

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2.5 inches of asphalt concrete						
					5 inches of aggregate base						
1–3	S&H	12, 9, 13	13	SM	SILTY SAND with GRAVEL (SM) red-brown, medium dense, moist, fine- to coarse-grained sand, fine gravel, trace clay Particle Size Distribution; see Figure B-4 Corrosivity Tests; see Figure B-5				24	8.3	
6	S&H	12, 17, 30	28		fine and coarse gravel						
8	S&H	17, 21, 61	49		yellow-brown, dense						
10	S&H	50/6"	30/6"		very dense						
13					dry to moist, increased gravel content						
14	SPT	32, 50/6"	50/6"								
19	SPT	42, 50/6"	50/6"								

Boring terminated at a depth of 19.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.6 and 1.0, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1736 Figure: B-2

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 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

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

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

Sonic

SAMPLER TYPE

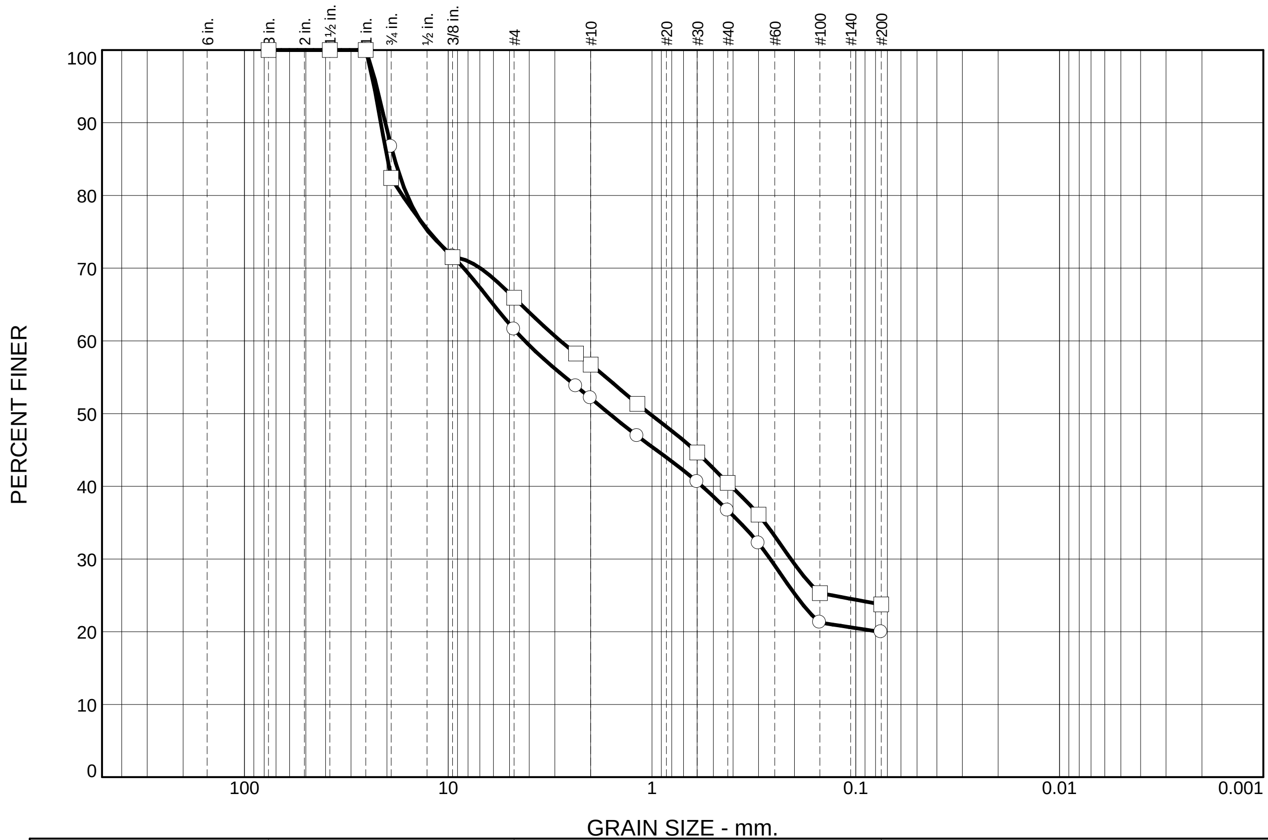
C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler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

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CLASSIFICATION CHART

Date 09/15/19 | Project No. 19-1736 | Figure B-3



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	13.3	25.1	9.5	15.4	16.7	20.0	
□	0.0	17.6	16.5	9.2	16.2	16.7	23.8	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1	1-1	3.0 - 3.5'	SILTY SAND with GRAVEL, red-brown	SM
□	B-2	2-1	3.0 - 3.5'	SILTY SAND with GRAVEL, red-brown	SM

3090 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 9/23/19 | Project No. 19-1736 | Figure B-4

Soil Analysis Lab Results

Client: Rockridge Geotechnical, Inc.
Job Name: 3090 South Bascom Ave.
Client Job Number: 19-1736
Project X Job Number: S190913H
September 18, 2019

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	SM-2320B
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+}	**Flouride** F_2^{--}	**Phosphate** PO_4^{3-}	**Bicarbonate** HCO_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)	(mg/kg)
B-1 #1a	3.5-4.0	62.7	0.0063	8.3	0.0008	40,870	4,489	8.0	145.0	0.5	3.8	2.7	ND	15.1	6.3	9.9	40.2	ND	1.0	120.9
B-2 #1a	3.5-4.0	38.2	0.0038	1.7	0.0002	14,740	5,628	7.8	158.0	0.8	10.1	0.5	ND	11.4	13.1	16.5	56.6	0.6	4.4	237.3

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