Appendix D:
Geotechnical Investigation and Paleontological Records Search

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D-1: Geotechnical Investigation

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ROCKRIDGE GEOTECHNICAL

Prepared for **Bay West Development**

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
1410 SOUTH BASCOM AVENUE
SAN JOSE, CALIFORNIA

March 23, 2018
Project No. 18-1437

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March 23, 2018
Project No. 18-1437

Mr. Pete Beritzhoff
Bay West Development
2 Henry Adams Street
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Subject: Geotechnical Investigation Report
Proposed Mixed Use Development
1410 South Bascom Avenue
San Jose, California

Dear Mr. Beritzhoff:

We are pleased to present the results of our geotechnical investigation for the proposed mixed-use development to be constructed at 1410 South Bascom Avenue in San Jose, California. Our services were provided in accordance with our proposal dated January 9, 2018.

The site is triangular shaped parcel encompassing an area of about 6.4 acres. It is bordered by South Bascom Avenue to the west, VTA rail tracks and station platform to the southeast, and commercial and residential properties to the north. The site is currently occupied by multiple single-story commercial buildings and an asphalt-paved parking lot. The ground surface elevation at the site varies by about five feet, sloping downward gently to the north.

Based on our review of the preliminary project drawings, titled *Gateway Station – Planned Development Zoning, PDZ Application Submittal*, prepared by WRNS Studio and KTGY Architects, dated October 12, 2017, we understand plans are to construct an eight-story residential building ("Building A") on the northern half of the site and a six-story office building ("Building B") on the southern portion of the site. The residential building will consist of five stories of wood-framed residential units over a four-story concrete podium with one below-grade level. The lower three levels of the podium structure will mostly house parking and the upper level of the podium structure will contain residential units. The office building will consist of six levels of office space over two below-grade parking levels. The below-grade parking associated with the office building will also extend beneath an at-grade plaza area in the central portion of the site, between the residential and office structures. The below-grade levels beneath the buildings will be constructed adjacent to each other. The development plan also includes plazas, landscaping areas, and exterior concrete flatwork.

ROCKRIDGE GEOTECHNICAL

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 issues affecting the proposed development include providing adequate foundation support for the proposed buildings and providing suitable lateral support for the proposed excavation while minimizing impacts to the surrounding improvements, including neighboring buildings, sidewalks, rail tracks, and roadways.

Provided the estimated settlements in this report are acceptable, we conclude the buildings may be supported on a shallow foundation system consisting of either conventional spread footings with a slab-on-grade or on a stiffened mat foundation. Feasible methods of temporary shoring during excavation include soil nails and soldier pile-and-lagging system. The most appropriate method will depend on the final excavation depth and setback from adjacent property lines.

Our report contains specific recommendations regarding earthwork and grading, foundation design, and other geotechnical issues. The recommendations contained in our report are based on limited subsurface exploration. Consequently, variations between expected and actual soil conditions may be found in localized areas during construction. Therefore, we should be engaged to observe foundation and shoring installation, grading, and fill placement, 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

ROCKRIDGE GEOTECHNICAL

TABLE OF CONTENTS

ROCKRIDGE GEOTECHNICAL

LIST OF FIGURES

APPENDIX A

APPENDIX B

APPENDIX C

ROCKRIDGE GEOTECHNICAL

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
1410 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 mixed-use development to be constructed at 1410 South Bascom Avenue in San Jose, California. The site is a triangular shaped parcel encompassing an area of about 6.4 acres. It is bordered by South Bascom Avenue to the west, VTA rail tracks and station platform to the southeast, and commercial and residential properties to the north, as shown on Figure 1, Site Location Map. The site is currently occupied by multiple single-story commercial buildings and an asphalt-paved parking lot. The ground surface elevation at the site varies by about five feet, sloping downward gently to the north.

Based on our review of the preliminary project drawings, titled *Gateway Station – Planned Development Zoning, PDZ Application Submittal*, prepared by WRNS Studio and KTGY Architects, dated October 12, 2017, we understand plans are to construct an eight-story residential building ("Building A") on the northern half of the site and a six-story office building ("Building B") on the southern portion of the site. The residential building will consist of five stories of wood-framed residential units over a four-story concrete podium with one below-grade level. The lower three levels of the podium structure will mostly house parking and the upper level of the podium structure will contain residential units. The office building will consist of six levels of office space over two below-grade parking levels. The below-grade parking associated with the office building will also extend beneath an at-grade plaza area in the central portion of the site, between the residential and office structures. The below-grade levels beneath the buildings will be constructed adjacent to each other. The development plan also includes plazas, landscaping areas, and exterior concrete flatwork.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal dated January 9, 2018. The objective of our investigation was to evaluate subsurface conditions at the site and develop conclusions and recommendations regarding the geotechnical aspects of the proposed project. Our scope of work consisted of evaluating subsurface conditions at the site by drilling three exploratory borings, advancing eight cone penetration tests (CPTs) and performing engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions beneath the site
- 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
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of static and seismically-induced foundation settlement
- subgrade preparation for pavements and exterior concrete flatwork
- recommended design groundwater elevation
- site grading and excavation, including criteria for fill quality and compaction
- excavation shoring design parameters
- soil corrosivity
- 2016 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 three borings, advancing eight 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 private 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-8 were advanced on January 25, 2018 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. CPT-1 was advanced to a depth of 44-1/2 feet bgs, however, the remaining seven CPTs encountered practical refusal at depths between 29 and 42 feet 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 explored by drilling three geotechnical borings, each to a depth of 44-1/2 feet. The borings, designated B-1 through B-3, were drilled on January 23, 2018 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-56 truck-mounted drill rig equipped with hollow-stem augers. During drilling, our field engineer 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 through B-3. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure B-4.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter stainless steel tubes
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.
- Shelby Tube (ST) thin-walled stainless steel tubes with 2.875-inch inside diameter.

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of granular soils. The Shelby tubes were used to obtain relatively undisturbed samples of medium stiff to stiff fine-grained soils. 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 0.7 and 1.2, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was designed to accommodate liners, but liners were not used. The blow counts used for this conversion were the last two blow counts. The converted SPT N-values are presented on the boring logs. The Shelby tubes were slowly advanced using the weight of the drill rods and hydraulic pressure, as needed.

Upon completion, the borings were backfilled with cement grout and the pavement surface was patched with quickset concrete. The drilling spoils generated during drilling were drummed and temporarily stored onsite. A representative sample of the drum contents was submitted to a laboratory for analytical testing, found to be non-hazardous, and scheduled for disposal at an appropriate landfill facility.

3.3 Laboratory Testing

We re-examined each soil sample in the office to confirm the field classification and select 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, dry density, plasticity (Atterberg limits), and fines content. Corrosivity testing of a sample of near-surface soil was performed by Project X Corrosion of Murrieta, California. The results of the geotechnical laboratory tests are presented on the boring logs in Appendix A and in Appendix C.

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 (Qha) of the Holocene epoch (11 thousand years ago to present) (Graymer, 2006). Alluvial fan deposits generally consist of a mixture of fine-grained and coarse-grained deposits.

Based on the results of our geotechnical investigation, we conclude that the site is generally underlain by clay with varying sand content to a depth ranging from about 24 to 29 feet bgs. The clay is typically stiff to very stiff with occasional soft to medium stiff zones. The clay is underlain by dense to very dese sands and gravels to the maximum depth explored of 44-1/2 feet.

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, the maximum depth included in the report.

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 near the intersection of Hamilton and Leigh avenues, approximately 3,000 feet southeast of the site (Well ID: 07S01W25L001M). The groundwater level at this well was measured at 1- to 3-month intervals from 2011 to 2016. Measured groundwater levels ranged from 74 to 137 feet bgs. The next-closest site listed on GeoTracker is located at 1030 Leigh Avenue, approximately 3,200 feet northeast of the 1410 South Bascom Avenue site. The shallowest observed groundwater at this location is approximately 73 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

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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.

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

[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.

TABLE 1

Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista-Shannon	6.6	Southwest	6.50
N. San Andreas - Peninsula	14	Southwest	7.23
N. San Andreas (1906 event)	14	Southwest	8.05
N. San Andreas - Santa Cruz	15	Southwest	7.12
Total Calaveras	19	East	7.03
Total Hayward	19	Northeast	7.00
Total Hayward-Rodgers Creek	19	Northeast	7.33
Zayante-Vergeles	24	South	7.00
San Gregorio Connected	38	West	7.50
Greenville Connected	41	East	7.00
Monterey Bay-Tularcitos	45	Southwest	7.30
Mount Diablo Thrust	49	North	6.70

In the past 200 years, four major earthquakes (i.e., Magnitude > 6) have been recorded on the San Andreas fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) Intensity Scale occurred east of Monterey Bay on the San Andreas fault (Toppozada and Borchardt, 1998). The estimated moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred on the Peninsula segment of the San Andreas fault. Severe shaking occurred with an MM of about VIII-IX, corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt

560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of October 17, 1989 had an M_w of 6.9 and occurred about 30 kilometers south of the site. On August 24, 2014 an earthquake with an estimated maximum intensity of VIII (severe) on the MM scale occurred on the West Napa fault. This earthquake was the largest earthquake event in the San Francisco Bay Area since the Loma Prieta Earthquake. The M_w of the 2014 South Napa Earthquake was 6.0.

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

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

5.2 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 less than seven kilometers from the Monte Vista-Shannon fault and less than 15 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, 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. In boring B-2, very loose to loose silty sand was encountered in the upper 5 feet, which is susceptible to cyclic densification. The remaining borings and CPTs encountered material which is either sufficiently dense or cohesive to resist cyclic densification. Based on these findings, we conclude there is potential for up to 1/2 inch of ground surface settlement in isolated areas of the site resulting from cyclic densification. This material will be removed where a basement is installed; therefore, we anticipate there will be negligible amounts of cyclic densification settlement beneath the proposed buildings.

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 issues affecting the proposed development include providing adequate foundation support for the proposed buildings and providing suitable lateral support for the proposed excavation while minimizing impacts to the surrounding improvements. These and other issues are discussed in more detail below.

6.1 Foundations and Settlement

Based on the current conceptual design drawings, we anticipate the foundations will be approximately 12 to 22 feet below grade, depending on the number of below-grade levels and foundation thickness. We anticipate medium stiff to very stiff clay with varying sand content will be exposed at foundation level. These soils have moderate strength and are moderately compressible. Based on the results of our investigation and engineering analyses, we conclude the proposed buildings can be supported on shallow foundations consisting of either spread footings or a mat foundation, provided that the estimated settlements are acceptable from a structural standpoint.

Our settlement analyses indicate total settlement of a mat foundation under static load conditions—assuming an average contact pressure of about 1,200 psf—will be about 1 inch. We anticipate most of the settlement will occur during construction. 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 mat can be designed to limit differential settlements to about 1/2 inch in 30 feet. For properly constructed spread footings designed in accordance with the recommendations in Section 7.2, we anticipate about 1 inch of total static settlement, most of which will occur during construction. Differential static settlement is estimated to be about 3/4 inch or less in 30 feet.

6.2 Excavation Support

Considering the proposed below-grade parking will extend as much as two levels below existing grades, construction will require an excavation extending as much as about 22 feet below the existing ground surface (including anticipated foundation thickness). Where the proposed buildings will include only one below-grade level, the excavation will likely be about 12 feet deep. The setbacks of the proposed buildings from the property lines varies from about 40 feet along the northern boundary, between 25 and 45 feet from the property line along the VTA tracks to the southeast, and minimal setback from the western property line along South Bascom Avenue.

Depending on the final basement layout and required excavation depth, portions of the excavation may be cut at temporary slopes and subsequently backfilled following construction of the below-grade walls. However, in locations where adjacent improvements (such as neighboring structures, sidewalks, utilities, roadways, and railway tracks) are within about two-times the proposed excavation height, the will need to be supported by a temporary shoring system.

There are several key considerations in selecting a suitable shoring system. Those we consider of primary concern are:

- protection of surrounding improvements, including structures, underground utilities, pavements, rail tracks, and sidewalks
- proper construction of the shoring system to reduce potential for ground movement,
- cost.

Several methods of shoring are available; we have qualitatively evaluated the following systems:

- soil nails,
- soldier pile-and-lagging with tiebacks, and
- cantilevered soldier pile-and-lagging.

Soil nail shoring systems consist of reinforcing bars, which are grouted in predrilled holes through the face of the excavation, and a reinforced shotcrete facing. Soil nail systems require a certain amount of ground movement to mobilize their lateral resistance, and therefore are only appropriate in locations where the excavation is not immediately adjacent to existing structures or critical underground utilities. In addition, where the excavation is close to the property line and there is insufficient setback, soil nails may need to extend beneath the neighboring property, which would require an encroachment agreement with neighboring property owners.

Soldier pile-and-lagging shoring systems usually consists of steel H-beams and concrete placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds from the top down, in maximum five-foot-thick lifts. Continuous soil-cement mixing reinforced with steel H-beams may be used in lieu of wood lagging. Where the required total cut is less than about 12 feet, a soldier pile-and-lagging system can typically provide economical shoring without tiebacks, and therefore will not encroach beyond the property line. Where cuts exceed about 12 feet in height, soldier pile-and-lagging systems are typically more economical if they include tieback anchors. Tiebacks consist of posttensioned steel strands or bars that are grouted into predrilled holes through the excavation face. Generally, tie-backs are installed in conjunction with a soldier pile-and-lagging (or soil-cement mix) system. However, tieback anchors will likely extend beneath the neighboring properties. Where there is insufficient property line set-back to accommodate soil nails or tiebacks, and an encroachment agreement is not possible, internal bracing will be required. Another alternative is to construct a cantilevered shoring system combined with partial slope-cuts, in order to reduce the vertical retained height.

Considering the depth and location of the excavation have not been finalized, both soil nails or soldier pile-and-lagging system—or a combination of both—may be the most economical shoring for the excavation. Recommendations for the design and construction of both soil nail walls and tiedback soldier pile-and-lagging shoring are presented in Section 7.5.

6.3 Construction Considerations

The soil to be excavated consists primarily of clay, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. If larger concrete debris is encountered, removal will require equipment capable of breaking concrete, such as a hoe-ram.

6.4 Soil Corrosivity

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

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

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 clearing should include removal of all existing pavements, former foundation elements, and underground utilities. Chunks of concrete and asphalt larger than 3 inches in greatest dimension that cannot be broken down by compaction equipment should be segregated and disposed of off-site. Any vegetation and organic topsoil (if present) should be stripped in areas to receive improvements (i.e., building, pavement, or flatwork). Tree roots with a diameter greater than 1/2 inch within three feet of building or flatwork subgrade should be removed. Demolished asphalt concrete should be taken to an asphalt recycling facility. Aggregate base beneath existing pavements may be re-used as select fill if carefully segregated and approved by the environmental consultant.

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 building footprints and will not interfere with the proposed construction, they may be abandoned in-place provided the lines are filled with lean concrete or cement grout to the property line. Voids resulting from demolition activities should be properly backfilled with engineered fill following the recommendations provided later in this section.

During excavation for the below-grade parking levels, portions of the excavation may encounter perched groundwater in isolated areas. If perched groundwater is encountered near the final subgrade, or if excavation is performed during the rainy season, the subgrade will be sensitive to disturbance, especially under construction equipment wheel loads. The potential for subgrade disturbance can be minimized by using tracked equipment when the excavation approaches two feet of the building subgrade. If soft areas are encountered in the slab subgrade or footing excavations, subgrade stabilization measures may be required. Recommendations for various subgrade stabilization options are presented below in Section 7.1.2.

7.1.1 Fill Materials and Compaction Criteria

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.

Fill should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction[5]. Fill consisting of clean sand or gravel (defined as soil with less than 10 percent fines by weight) should be compacted to at least 95 percent relative compaction. Fill greater than five feet in thickness or placed within the upper foot of pavement soil subgrade should also be compacted to at least 95 percent relative compaction, and be non-yielding.

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

<u>Aggregate Base Material</u>

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

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5 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.

Controlled Low Strength Material

Controlled low strength material (CLSM) may be considered as an alternative to soil fill beneath structures 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 compressive strength of at least 50 pounds per square inch (psi) and no more than 100 psi.

7.1.2 Soil Subgrade Stabilization

In some areas, soft, wet soil (resulting from perched groundwater) may be encountered in localized areas during grading, causing the subgrade to deflect and rut under the weight of grading equipment. Furthermore, if the excavation subgrade is exposed during periods of heavy rain, it will become soft and unstable. In these areas, some form of subgrade stabilization may be required. Several options for stabilizing subgrade, if needed, are presented below.

Aeration

Aeration consists of mixing and turning the soil to naturally lower the moisture content to an acceptable level. Aeration typically requires several days to a week of warm, dry weather to effectively dry the material. Material to be dried by aeration should be scarified to a depth of at least 12 inches; the scarified material should be turned at least twice a day to promote uniform drying. Once the moisture content of the aerated soil has been reduced to acceptable levels, the soil should be compacted in accordance with our previous recommendations. Aeration is typically the least costly subgrade stabilization alternative; however, it generally requires the most time to complete and may not be effective if the soft material extends to great depths.

Overexcavation

Another method of achieving suitable subgrade in areas where soft, wet soil is exposed is to overexcavate the soft subgrade soil and replace it with drier, granular material. If the soft material extends to great depths, the upper 18 to 24 inches of soft material may be overexcavated and a geotextile tensile fabric (Mirafi 500X or equivalent) placed beneath the granular backfill to help span over the weaker material. The fabric should be pulled tight and placed at the base of the overexcavation, extending at least two feet laterally beyond the limits of the overexcavation in all directions. The fabric should be overlapped by at least two feet at all seams. Granular material such as Class 2 aggregate base should then be placed and compacted over the geotextile tensile fabric.

Chemical Treatment

Lime and/or cement have been successfully used to dry and stabilize fine-grained soils with varying degrees of success. Lime- and/or cement-treatment will generally decrease soil density, change its plasticity properties, and increase its strength. The degree to which lime will react with soil depends on such variables as type of soil, mineralogy, quantity of lime, and length of time the lime-soil mixture is cured. Cement is generally used when a significant amount of granular material or low-plasticity silt is present in the soil. The quantity of lime and/or cement added generally ranges from 3 to 7 percent by weight and should be determined by laboratory testing. The specialty contractor performing the chemical treatment should select the most appropriate additive and quantity for the soil conditions encountered.

If chemical treatment is used to stabilize soft subgrade, a treatment depth of about 12 to 18 inches below the final soil subgrade will likely be required. The soil being treated should be scarified and thoroughly broken up to full depth and width. The treated soil should not contain rocks or soil clods larger than three inches in greatest dimension. Treated soil should be compacted to at least 90 percent relative compaction.

7.1.3 Utility Trench Excavations and Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. All temporary excavations used in construction should be designed, planned, constructed, and maintained by the contractor and should conform to all state and/or federal safety regulations and requirements, including those of CAL-OSHA.

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

The bottom of foundations for the proposed building should be below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of utility trenches. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with CLSM (see Section 7.1.1 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 buildings may be supported on spread footings with a slabs-on-grade or on mat foundations bearing on undisturbed, native soil. Specific recommendations for the design and construction of each foundation type are presented in the following sections.

7.2.1 Spread Footings

Continuous and isolated spread footings should be at least three feet wide and bottomed at least 18 inches below the adjacent soil subgrade. Footings to be constructed near underground utilities should be bottomed below an imaginary line extending up at an inclination of 1.5:1 (horizontal:vertical) from the bottom of the utility trench. The footings may be designed using allowable bearing pressures of 3,000 psf for dead plus live loads and 4,000 psf for total design loads, which include wind or seismic forces.

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 passive resistance for transient loading, we recommend using an allowable uniform pressure of 1,500 psf (rectangular distribution). To compute passive resistance for sustained lateral loads, we recommend using an equivalent fluid weight (triangular distribution) of 250 pounds per cubic foot (pcf). The upper foot of soil should be ignored unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. If footings are excavated during the rainy season or below the groundwater level they should incorporate a mud slab to protect the footing subgrade. This will involve over-excavating the footing by about 2 to 3 inches and placing lean concrete or CLSM in the bottom (following our engineer checking the subgrade). A mud slab will help protect the footing subgrade during placement of reinforcing steel. Water can then be pumped from the excavations prior to placement of structural concrete, if present.

7.2.2 Mat Foundations

For structural design of mat foundations, we recommend using a coefficient of vertical subgrade reaction of 20 pounds per cubic inch (pci). This value has been reduced to account for the size of the mat/equivalent footings (therefore, this is not k_{v1} for 1-foot-square plate). Once the structural engineer evaluates the initial distribution of bearing stress on the bottom of the mat, we can review the distribution and revise the coefficients of subgrade reaction, if appropriate. We recommend the mat be designed for allowable bearing pressures of 3,000 psf for dead-plus-live loads and 4,000 psf for total loads (including seismic and wind loads); we anticipate the average bearing pressure will be significantly lower.

Lateral forces can be resisted by friction along the base of the mat and passive pressure against the sides of the mat foundation. To compute lateral resistance, we recommend using an allowable uniform pressure of 1,500 psf (rectangular distribution) for transient loads. To compute passive resistance for sustained lateral loads, we recommend using an equivalent fluid weight (triangular distribution) of 250 pounds per cubic foot (pcf) and an allowable base friction coefficient of 0.30 may be used, where the mat is in contact with soil. Where/if a vapor retarder is placed beneath the mat, a base friction coefficient of 0.20 should be used. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

The subgrade should be free of standing water, debris, and disturbed materials and be approved by the geotechnical engineer prior to placing a mud slab, vapor retarder, or reinforcing steel.

7.3 Floor Slabs

If the buildings are supported on footings, 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.

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 2.

TABLE 2

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 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 63 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 42 pcf, plus a seismic increment of 22 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 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 94 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 83 pcf, plus a seismic increment of 11 pcf (triangular distribution)

If backfill is required behind basement 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 Temporary Cut Slopes and Shoring

The safety of workers and equipment in or near the excavation is the responsibility of the contractor. The selection, design, construction, and performance of the shoring system should be the responsibility of the contractor. A structural engineer knowledgeable in this type of construction should design the shoring. We should review the geotechnical aspects of the proposed shoring system to ensure that it meets our recommendations. During construction, we should observe the installation of the shoring system and check the condition of the soil encountered during excavation.

We recommend that temporary cuts between 5 and 20 feet in height, that are not subjected to surcharges and not close to neighboring buildings, should be inclined no steeper than 1:1 (horizontal:vertical), which corresponds to OSHA Type B soil. If the excavation is performed during the rainy season, or a substantial amount of granular soil is encountered in the cut, the soil should be downgraded to OSHA Type C soil, which requires a maximum inclination of 1.5:1

(horizontal:vertical). Temporary shoring will be required where temporary slopes are not possible because of space constraints.

As discussed in Section 6.2, we conclude soil nails or soldier pile-and-lagging with tiebacks are likely the most suitable shoring systems for the proposed excavation, where/if the buildings include two below-grade levels. A combination of slope cuts with cantilevered soldier pile-and lagging shoring may also be viable, however the design earth pressures will depend on the various cut configurations and retained heights being considered—we can provide specific recommendations for these pressures once the proposed excavation and shoring scheme has been established. Where/if the buildings include one below-grade level, cantilevered soldier pile-and-lagging (without tiebacks) will likely be the most economical system. Geotechnical recommendations for the design and construction of soil nails and soldier pile-and-lagging shoring systems are presented in the following sections.

7.5.1 Soil Nail Walls

All or portions of the proposed excavation may be supported by a soil nail shoring system. Soil nail walls should be designed to resist static lateral earth pressures, as well as traffic loads, construction equipment loads, and foundation surcharge loads, where applicable. In general, we recommend the walls be designed and constructed in accordance with the guidelines presented in the Federal Highway Administration report on soil nail walls (FHWA, 2015)[6]. Several computer programs, such as SNAIL (California Department of Transportation, 2014) and GoldNail (Golder Associates, 1996), are available for designing a soil-nail wall. SNAIL uses a force equilibrium method of analysis; the failure planes are assumed bi-linear if they pass through the toe of the wall and tri-linear if they pass below the toe of the wall. GoldNail uses a slope-stability model that satisfies overall limiting equilibrium of free bodies defined by circular slip surfaces.

[6] Federal Highway Administration (2003), *Geotechnical Engineering Circular No. 7 – Soil Nail Walls*, March 2003 (FHWA Report No. FHWA0-IF-03-017)

Soil-nail systems are typically installed under a design-build contract by specialty contractors; therefore, we are not providing a specific design. However, we are providing estimated input parameters for preliminary design. The actual soil nail capacities and lengths should be determined by a design-build contractor with experience designing, building, and testing soil-nail walls in similar soil conditions. We should review the geotechnical aspects of their design prior to installation. For preliminary design, we recommend the input parameters presented in Table 3.

TABLE 3
Recommended Input Parameters for Design of Soil-Nail Walls

Soil Type	Total Density (pcf)	Ultimate Bond Strength (psf) (Factor of Safety = 1.0)	Shear Strength Parameters	
			c^1 (psf)	ϕ^2 (deg)
Native Sandy Clays	125	800	400	20

Notes:
[1] Cohesion intercept or undrained shear strength, without a factor of safety
[2] Angle of internal friction, without a factor of safety

Where construction equipment will be working or driving behind the walls, the design should include a surcharge pressure of 250 psf. The soil-nail wall should be designed with a minimum factor of safety of 1.5 against slope stability failure for temporary walls and a factor of safety of 2.0 for permanent walls.

We should be allowed to review the design plans and design calculations prior to their issuance for construction to check for conformance with our recommendations.

Soil Nail Installation

The drilling method and equipment should be determined by the contractor and modified, as needed, based on the soil conditions encountered during excavation and drilling. If the drilling methods and equipment deviate from those used during installation of the load-tested verification nails, additional verification tests may be required. The holes should be cleaned of loose soil prior to placement of bars, centralizers, and grout. If caving soil is encountered, casing of the holes may be required. We recommend all soil nails be grouted the same day they are drilled and that grout be placed using the tremmie method from the bottom of the hole.

Maintaining a consistent grout mix is critical to achieving consistent nail performance and is the responsibility of the contractor. Mud balance measurements of the specific gravity of the grout mixture may be used in the field to provide immediate indications of the grout consistency (water-cement ratio). We recommend a minimum specific gravity of 1.80 be used for grout mixes containing cement and water. In our experience, grout mixes with specific gravities significantly lower than 1.80 can result in inadequate soil nail bond strengths, longer required cure times before proof testing, and increased load test failures.

Soil-Nail Testing

We recommend the soil-nails be load-tested prior to and during construction in accordance with the guidelines presented in the Federal Highway Administration document (FHWA, 2015). Test nails should be installed using the same equipment, method, and hole diameter as planned for the production nails. Verification and proof tests should be performed. Verification tests are performed prior to production nail installation to verify the pullout resistance (bond strength) value used in design and resulting from the contractor's chosen installation methods. Two verification tests should be performed for each soil type assumed in design. Proof tests are performed during construction to verify that the contractor's procedure remains consistent and that the nails are not installed in a soil type that was not adequately represented by the verification stage testing. At least five percent of the production nails should be proof tested.

Verification tests should be performed on non-production, sacrificial nails to a test load corresponding to the ultimate pullout resistance value used in the design. Test nails should have at least three feet of unbonded length and 10 feet of bond length. The nail bar grade and size should be designed such that the bar stress does not exceed 80 percent of its ultimate tensile strength for Grade 75 steel or 90 percent of the yield strength for Grade 60 steel during testing—a larger bar may be required for verification test nails.

The verification and proof tests should be performed in accordance with FHWA guidelines (FHWA, 2015), including the recommended load increments, maximum test load, and failure criteria. In the verification and proof tests, the load is applied to the nails in four increments (one complete load cycle). The maximum test load should be held for a minimum of 10 minutes; the movements of the nails should be recorded at 0, 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference in movement between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is greater than 0.04 inch, the holding period is extended to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the test results and determine whether the test nail performance is acceptable. Generally, a test with a ten-minute hold is acceptable if the nail carries the maximum test load with less than 0.04 inch movement between one and 10 minutes. A test with a 60-minute hold is acceptable if the nail carries the maximum test load with less than 0.08 inch movement between six and 60 minutes.

7.5.2 Soldier Pile-and-Lagging Shoring System

Soldier pile-and-lagging is an acceptable method to retain the excavation. Recommended lateral pressures for the design of cantilevered and tied-back soldier pile-and-lagging shoring are presented on Figures 6 and 7, respectively. The shoring should be designed by a shoring engineer.

Where traffic loads are expected within 10 feet of the shoring walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall. Where construction equipment will be working behind the walls within a horizontal distance of 10 feet, the design should include a surcharge pressure of 250 psf acting over the upper 10 feet of the wall. The above pressures should be assumed to act over the entire width of the lagging installed above the excavation. Passive resistance at the toe of the soldier pile should be computed using equivalent fluid weights of 250 pcf up to a maximum of 1,750 psf (trapezoidal distribution). These passive pressure values include a factor of safety of at least 1.5. The upper foot of soil should be ignored when computing passive resistance. Passive pressure can be assumed to act over an area of three soldier pile widths, or pile-to-pile spacing, whichever is less, assuming the toe of the soldier pile is filled with concrete or lean concrete that is sufficiently strong to accommodate the corresponding stresses.

Soldier piles should be placed in pre-drilled holes backfilled with concrete. Based on our investigation, we expect that the soil to be retained by the shoring has sufficient cohesion to stand vertically for four-foot cuts. If voids are created behind lagging boards due to localized caving or overcutting, they should be filled with cement slurry or hand-packed soil prior to proceeding with excavation.

The penetration of the soldier piles must be sufficient to ensure stability and resist the downward loading of tiebacks. Vertical loads can be resisted by skin friction along the portion of the soldier piles below the excavation. We recommend using an allowable skin friction value of 400 psf above a depth of 30 feet (from existing grades) and 1,500 psf below a depth of 30 feet to compute the required soldier pile embedment. End bearing should be neglected.

Design criteria for tiebacks are also presented on Figure 7. As shown, tiebacks should derive their load-bearing capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation at an angle of 60 degrees from horizontal, where H is the wall height in feet. The minimum stressing lengths for strand and bar tendons should be 15 and 10 feet, respectively. The minimum bond length for strand and bar tendons should both be 15 feet.

Allowable capacities of the tiebacks will depend upon the drilling method, hole diameter, grout consistency, grout pressure, and workmanship. The shoring contractor should use a smooth-cased method (such as a Klemm rig) to install the tiebacks to prevent caving beneath adjacent buildings and improvements. The bottom of excavation should not extend more than two feet below a row of unsecured tiebacks. The shoring designer should be responsible for determining the actual length of tiebacks required to resist the design loads. The determination should be based on the designer's familiarity with the installation method to be used.

Tieback Testing

The computed bond length of tiebacks should be confirmed by a performance- and proof-testing program under the observation of our field engineer. The first two production tiebacks and two percent of the remaining tiebacks should be performance tested to 1.5 times the design load. The remaining tiebacks should be confirmed by a proof-test to 1.25 times the design load. The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during performance and proof testing.

The performance test is used to verify the capacity and the load-deformation behavior of the tiebacks. It is also used to separate and identify the causes of tieback movement, and to check that the designed unbonded length has been established. In the performance test, the load is applied to the tieback in several cycles of incremental loading and unloading. During the test, the tieback load and movement are measured. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 2, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

Rockridge Geotechnical and the shoring engineer should evaluate the tieback test results and determine whether the tiebacks are acceptable. A performance- or proof-tested tieback with a

10-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inch movement between 1 and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. A performance- or proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with less than 0.08 inch movement between 6 and 60 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. Tiebacks that failed to meet the10- or 60-minute hold criterion will be assigned a reduced capacity. Tiebacks that do not exceed 80 percent of theoretical elastic elongation should be replaced by the contractor at no additional cost to the owner.

7.5.3 Construction Monitoring

During excavation, the shoring system may deform laterally, which could cause the ground surface adjacent to the shoring wall to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Ground movements due to a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch. A monitoring program should be established to evaluate the effects of the construction on the adjacent properties.

The contractor should establish survey points on the shoring and on the ground surface at critical locations behind the shoring prior to the start of excavation. These survey points should be used to monitor the vertical and horizontal movements of the shoring and the ground behind the shoring during construction.

7.6 Seismic Design

We understand the proposed building will be designed using the seismic provisions in the 2016 California Building Code (CBC). Using the USGS Seismic Design Maps website and a site latitude of 37.2990º and longitude of -121.9303º, we conclude the following seismic design parameters should be used:

- Site Class D
- S_S = 1.500 g, S_1 = 0.600 g
- S_{MS} = 1.500 g, S_{M1} = 0.900 g
- S_{DS} = 1.000 g, S_{D1} = 0.600 g
- Seismic Design Category D for Risk Categories 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, installation of foundations, and shoring installation. 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.

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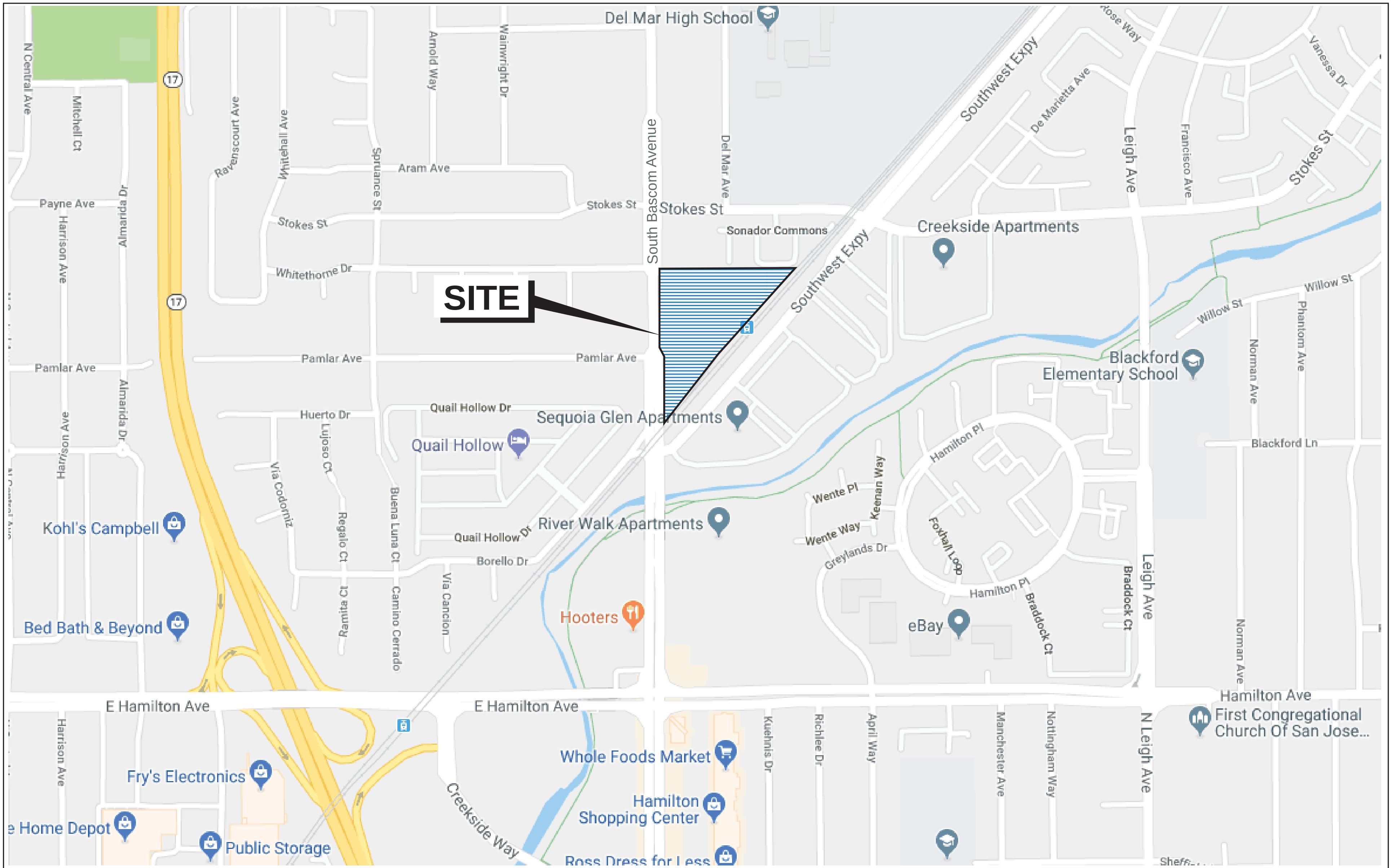
FIGURES



Base map: Google Map, 2017

0 800 Feet

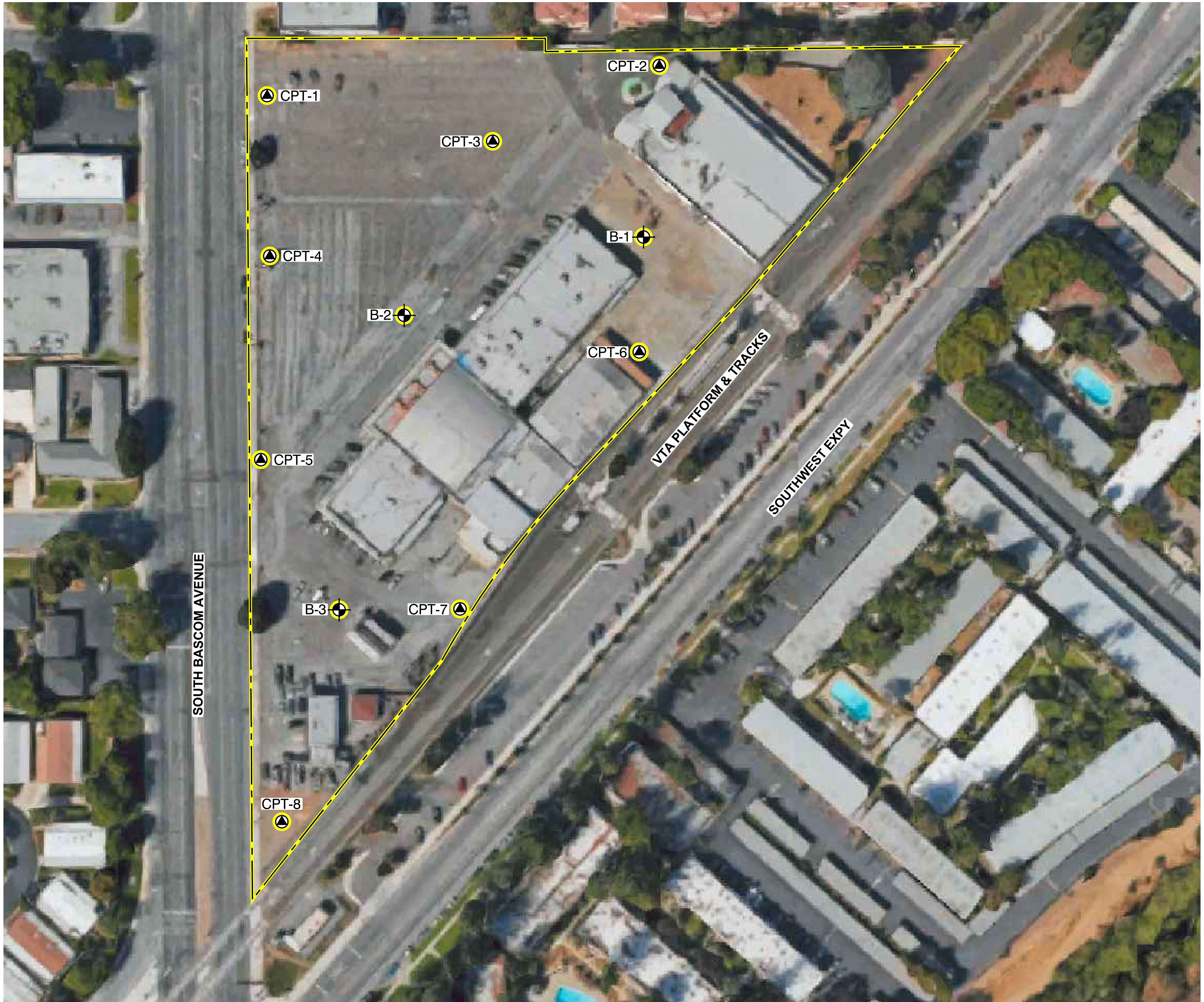
Approximate scale

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 03/21/18	Project No. 18-1437	Figure 1



EXPLANATION

CPT-1 Approximate location of cone penetration test by Rockridge Geotechnical Inc., January 25, 2018

B-1 Approximate location of boring by Rockridge Geotechnical Inc., January 23, 2018

Project limits

0 100 Feet

Approximate scale

Base map: Google Earth, 2017.

1410 SOUTH BASCOM AVENUE
San Jose, California

SITE PLAN

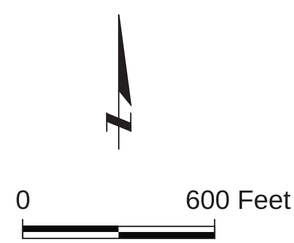
Date 03/21/18	Project No. 18-1437	Figure 2

ROCKRIDGE GEOTECHNICAL

Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2017.

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)
	Geologic contact: dashed where approximate and dotted where concealed, queried where uncertin



Approximate scale

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 02/07/18 | Project No. 18-1437 | 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

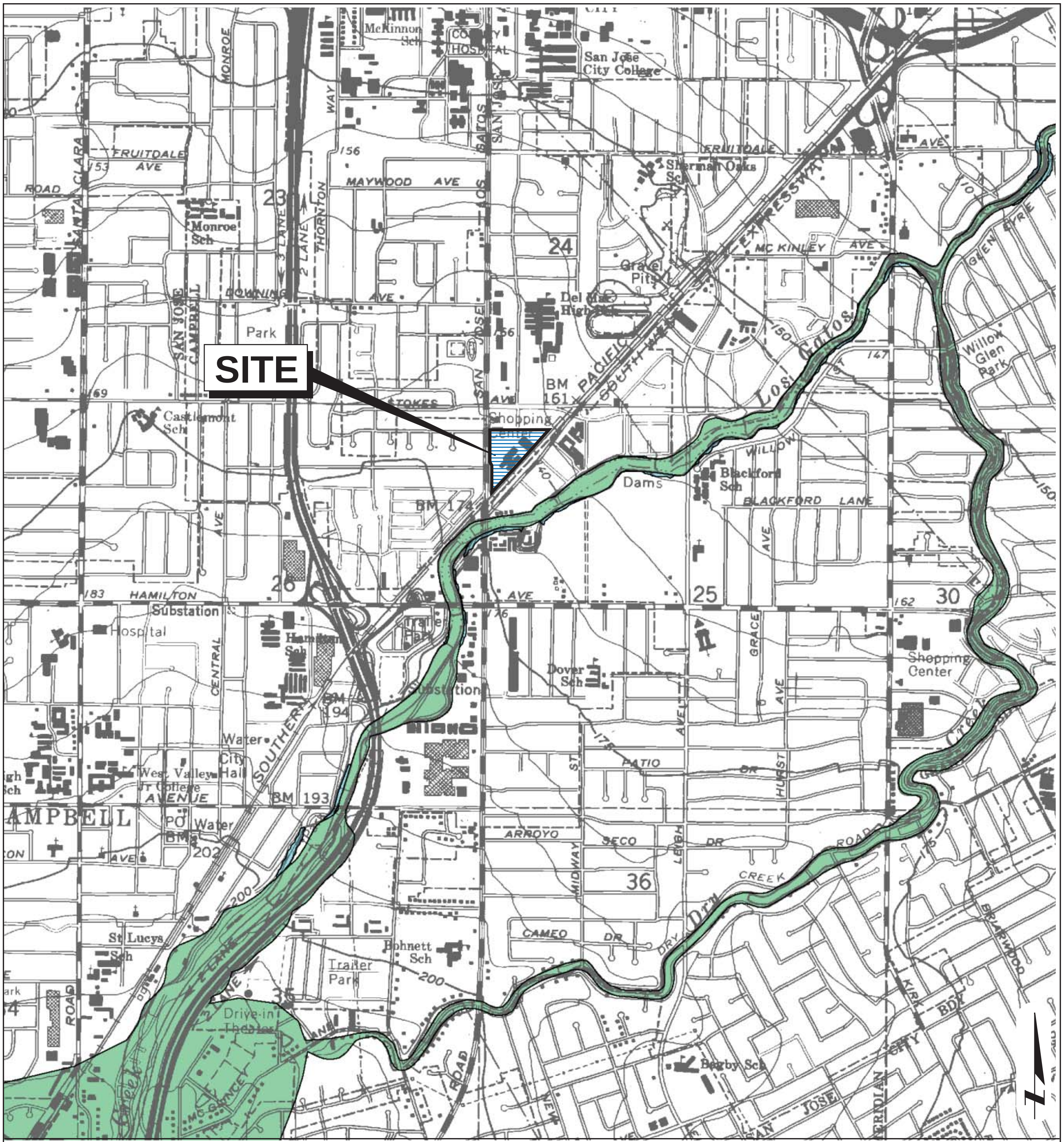
Approximate scale

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 02/07/18 | Project No. 18-1437 | Figure 4



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 2000 4000 Feet

Approximate scale

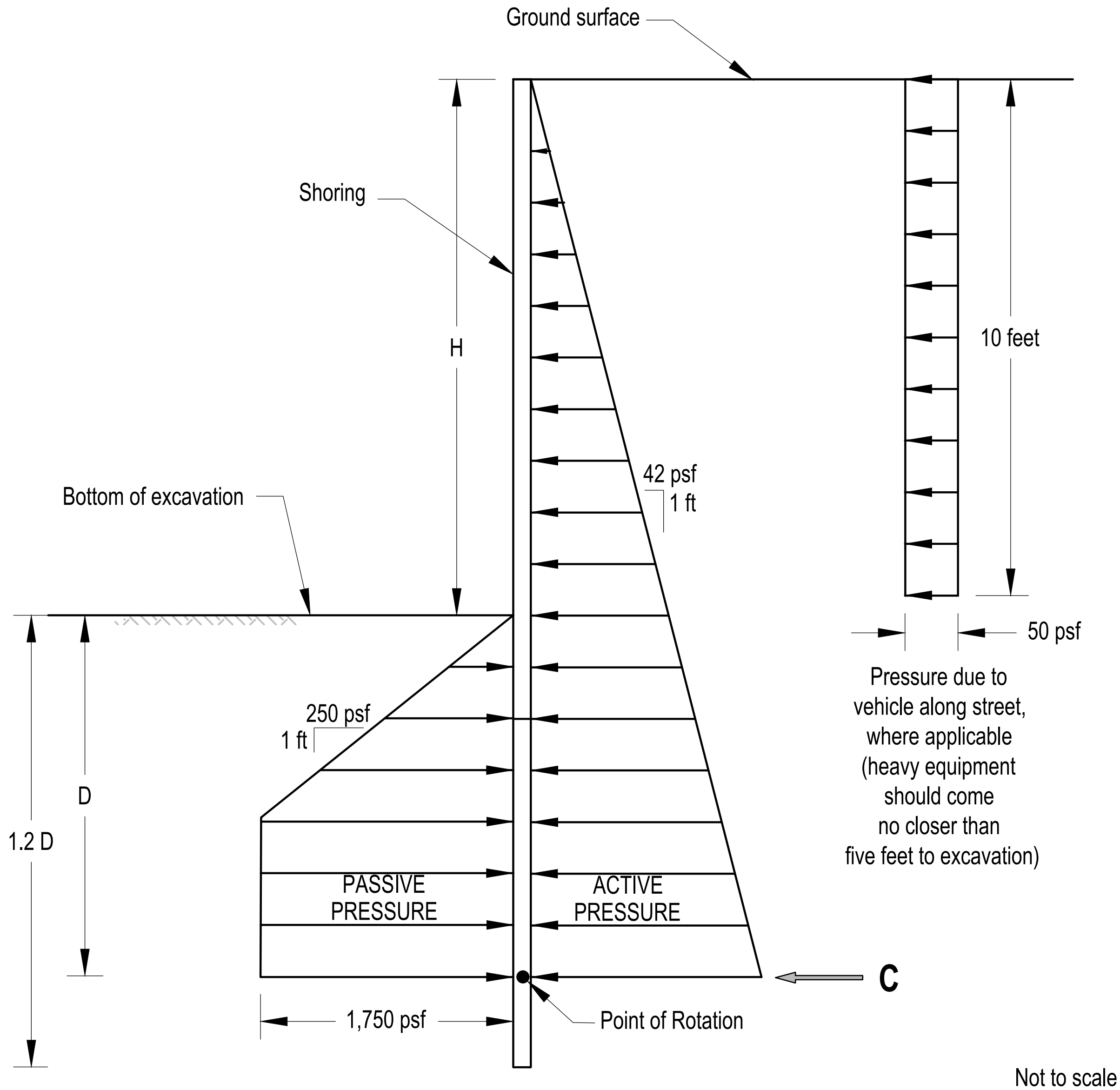
Reference:
State of California "Seismic Hazard Zones"
San Jose West Quadrangle.
Released on February 7, 2002

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

SEISMIC HAZARDS ZONE MAP

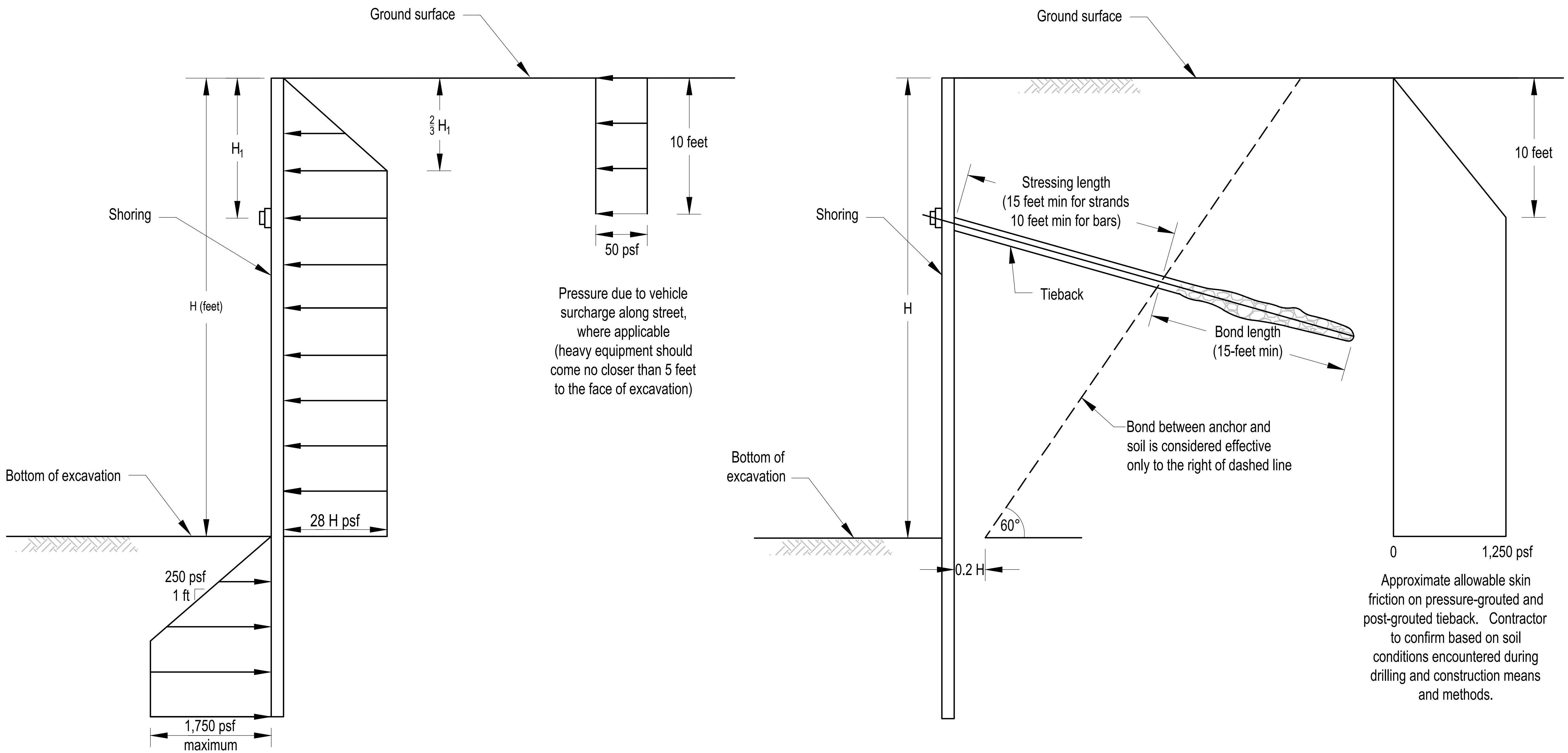
Date 02/09/18	Project No. 18-1437	Figure 5



Notes:

1. Simplified pressure diagram is presented above. The net passive pressure on the right side of the shoring below the point of rotation is replace by a concentrated force C.
2. Passive pressures include a factor of safety of about 1.5.
3. Passive pressures may be assumed to act over the pile spacing or three times the pile diameter, whichever is smaller (for piles with structural concrete).
4. Surcharge pressure, due to construction equipment, if any, should be added to the above shoring pressure.
5. Active pressure below the excavation should be assumed to act over one pile diameter.
6. Calculated embedment depth, D, should be increased by at least 20 percent to obtain the design depth of penetration.
7. The recommended pressures do not include surcharges from adjacent buildings. Where shoring system is adjacent to an at-grade building, at-rest lateral pressures should be used and surcharge pressure from footings should be added to the above shoring pressures.
8. pcf denotes pounds per cubic foot; psf denotes pounds per square foot.

1410 SOUTH BASCOM AVENUE San Jose, California	**LATERAL EARTH PRESSURES FOR CANTILEVERED SOLDIER-PILE-AND-LAGGING SHORING SYSTEM**
ROCKRIDGE GEOTECHNICAL	Date 03/21/18 | Project No. 18-1437 | Figure 6



NOT TO SCALE

Notes:

1. Passive pressures include a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
3. Surcharge pressure due to construction equipment, if any, should be added to the above shoring pressure.

1410 SOUTH BASCOM AVENUE
San Jose, California

DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING TEMPORARY SHORING SYSTEM WITH ONE ROW OF TIEBACKS

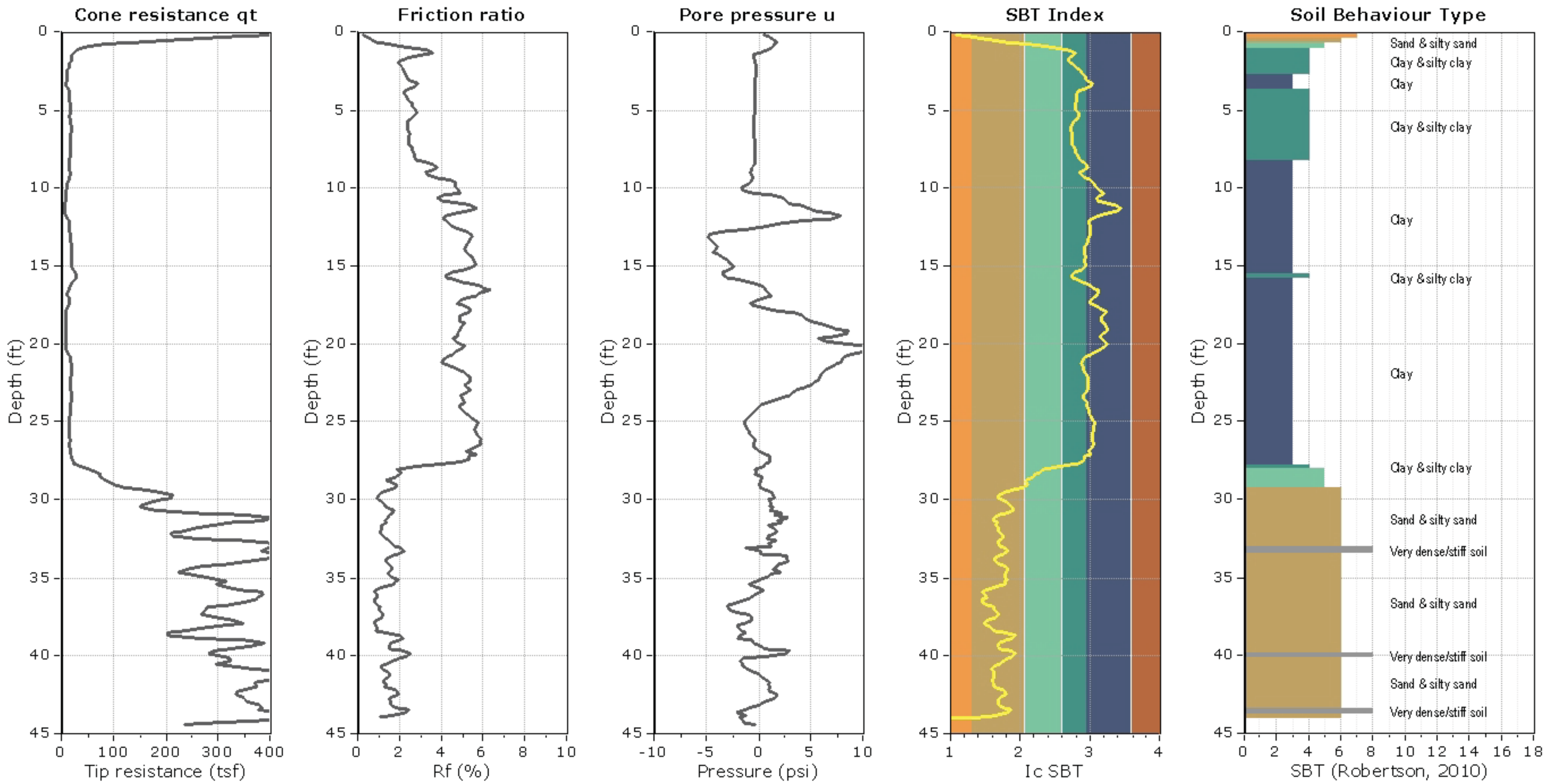
Date 03/21/18	Project No. 18-1437	Figure 7

ROCKRIDGE GEOTECHNICAL

ROCKRIDGE GEOTECHNICAL

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

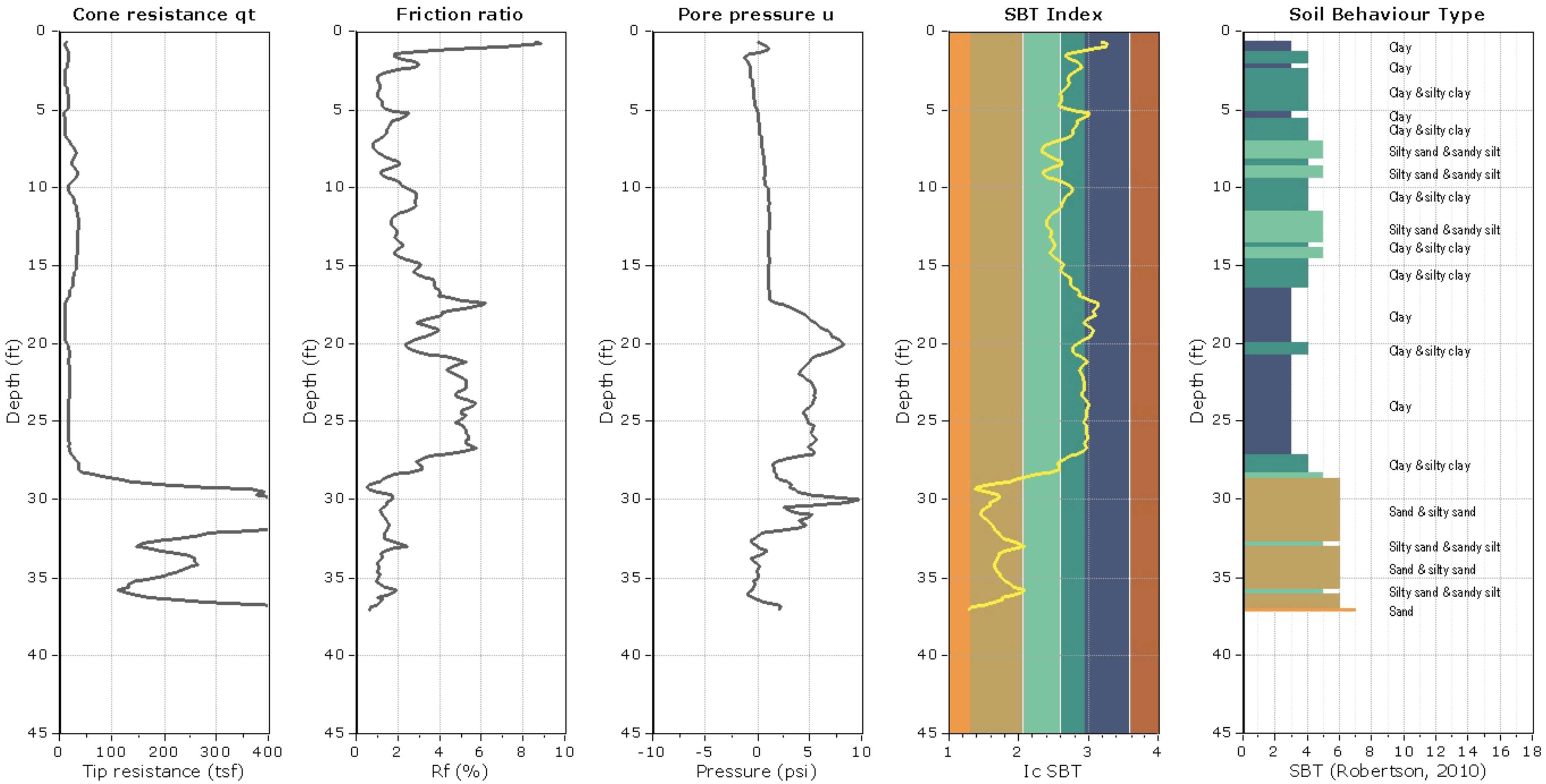
Total depth: 44.46 ft, Date: 1/25/2018
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 02/07/18 | Project No. 18-1437 | Figure A-1

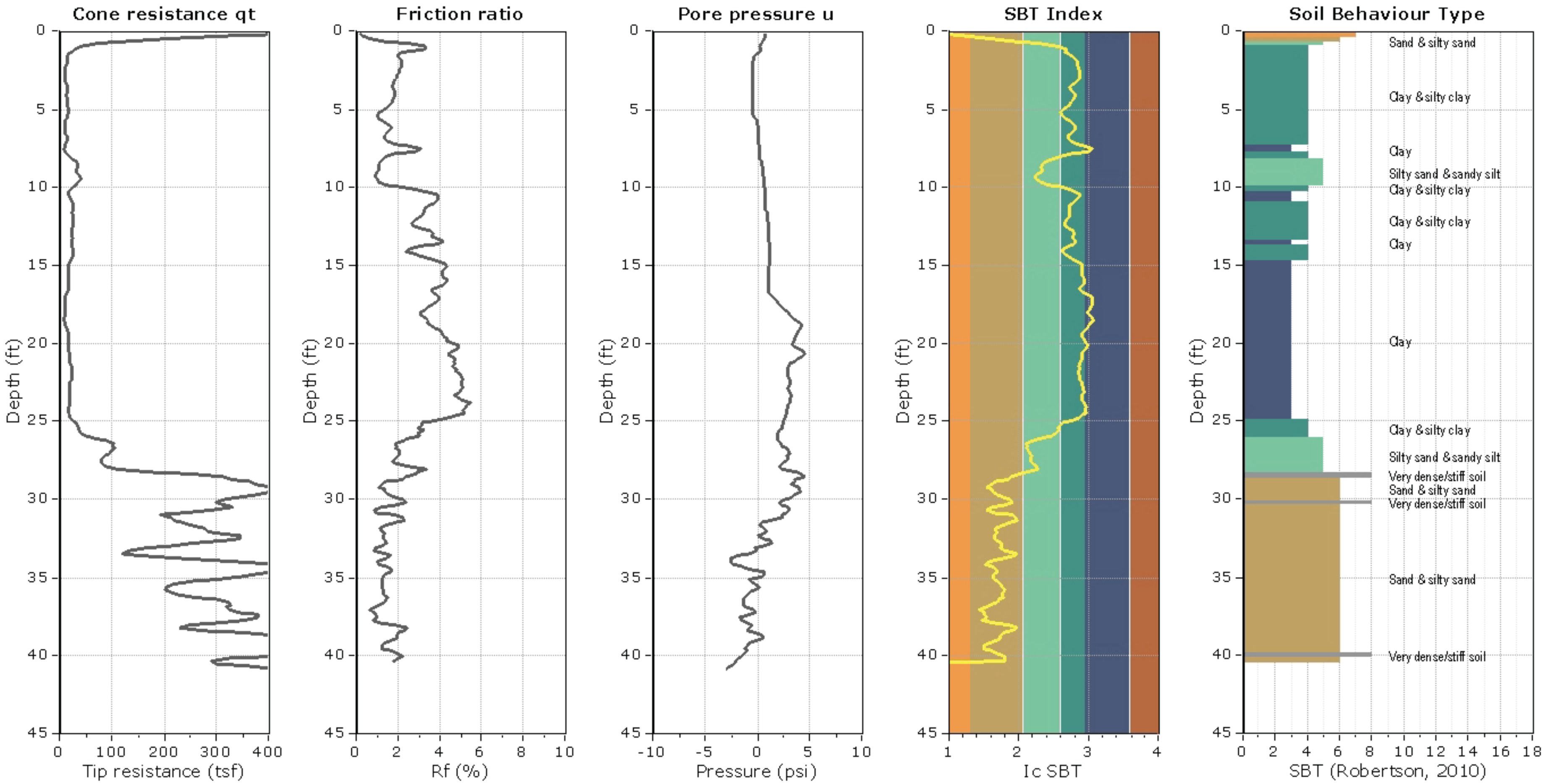


SBT legend

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

Total depth: 37.07 ft, Date: 1/25/2018
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

1410 SOUTH BASCOM AVENUE San Jose, California	**CONE PENETRATION TEST RESULTS** **CPT-2**
ROCKRIDGE GEOTECHNICAL	Date 02/07/18 | Project No. 18-1437 | 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

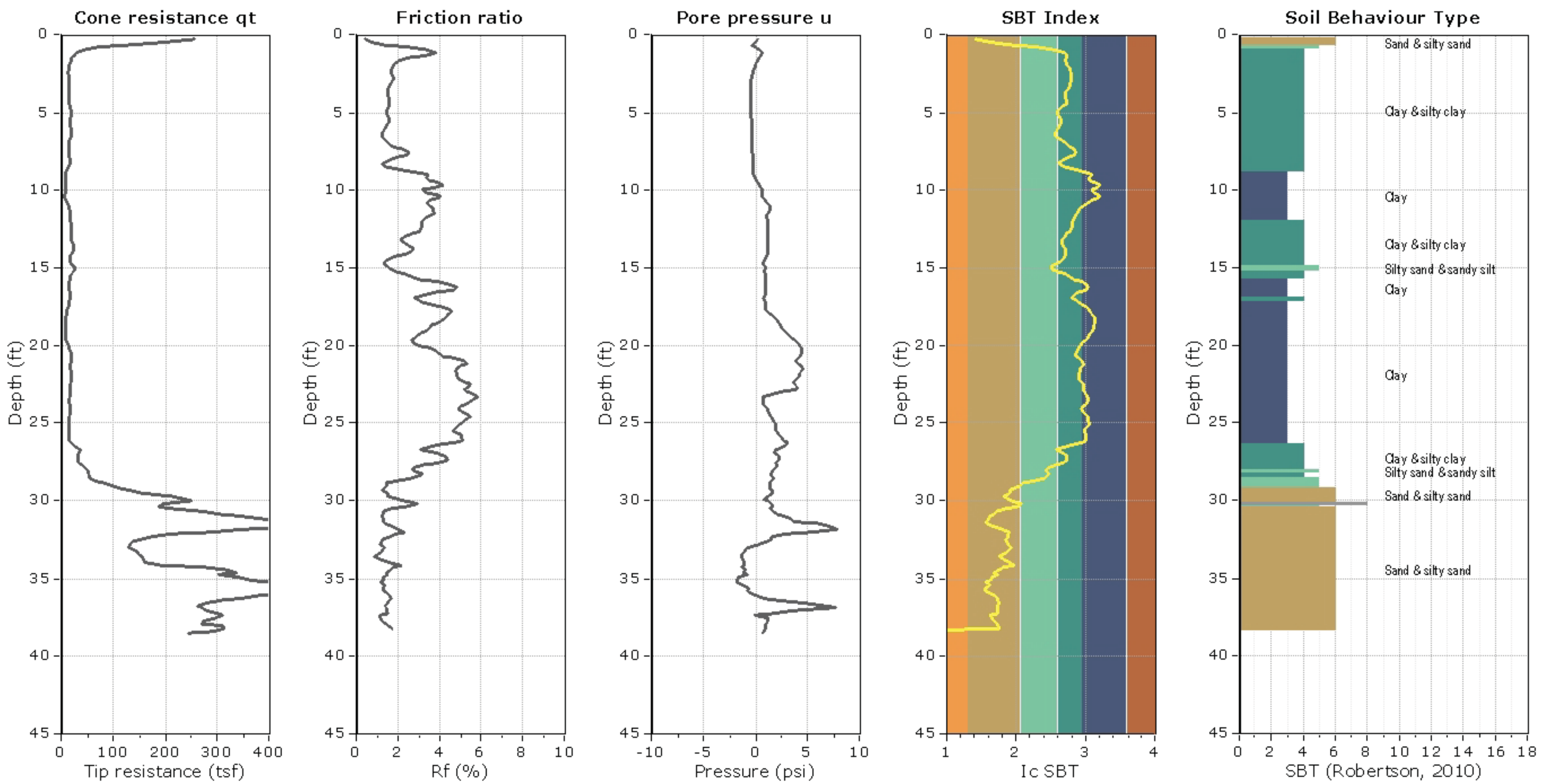
Total depth: 40.85 ft, Date: 1/25/2018
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 02/07/18	Project No. 18-1437	Figure A-3



Total depth: 38.55 ft, Date: 1/25/2018
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

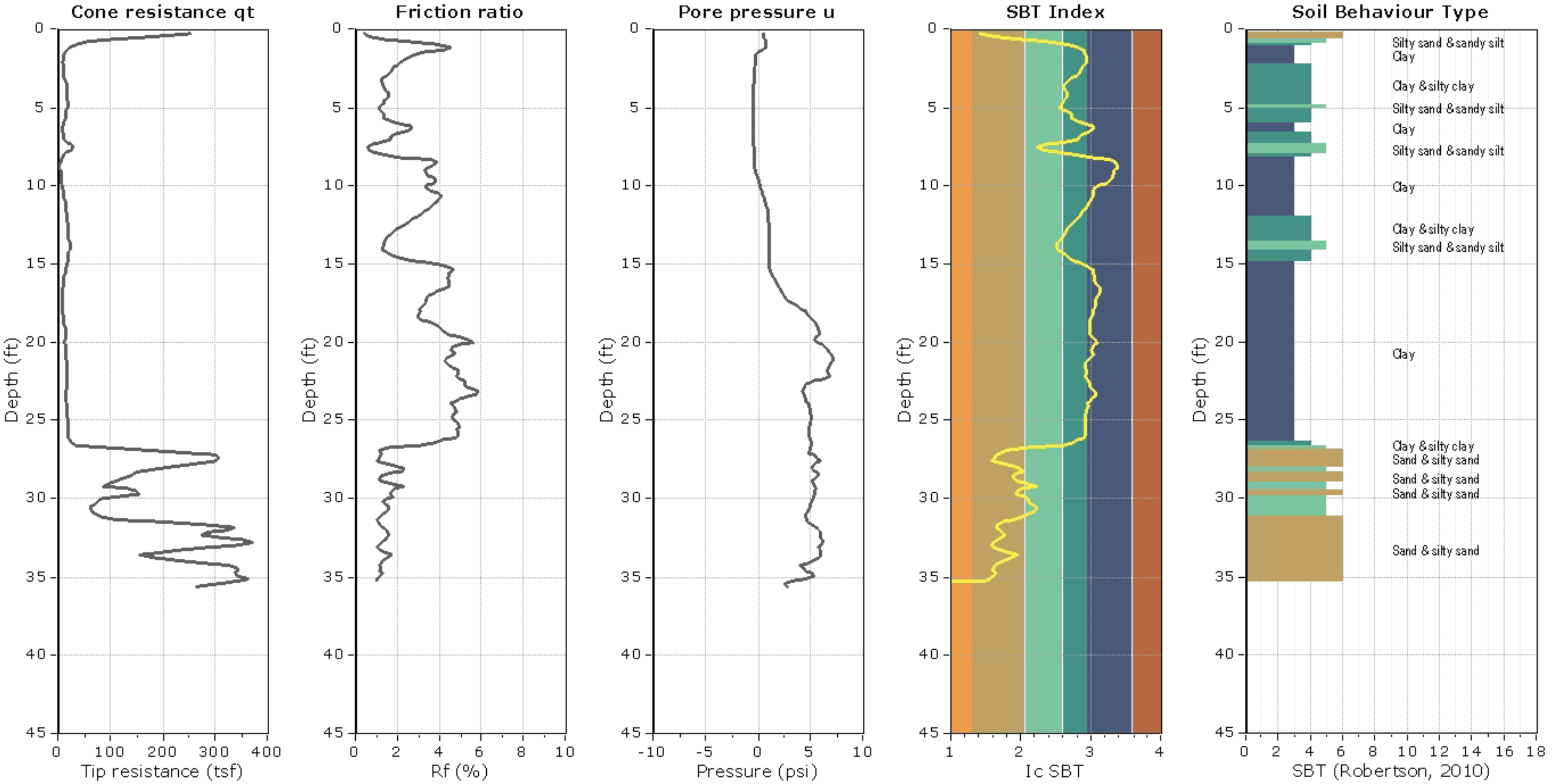
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 02/07/18 | Project No. 18-1437 | 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

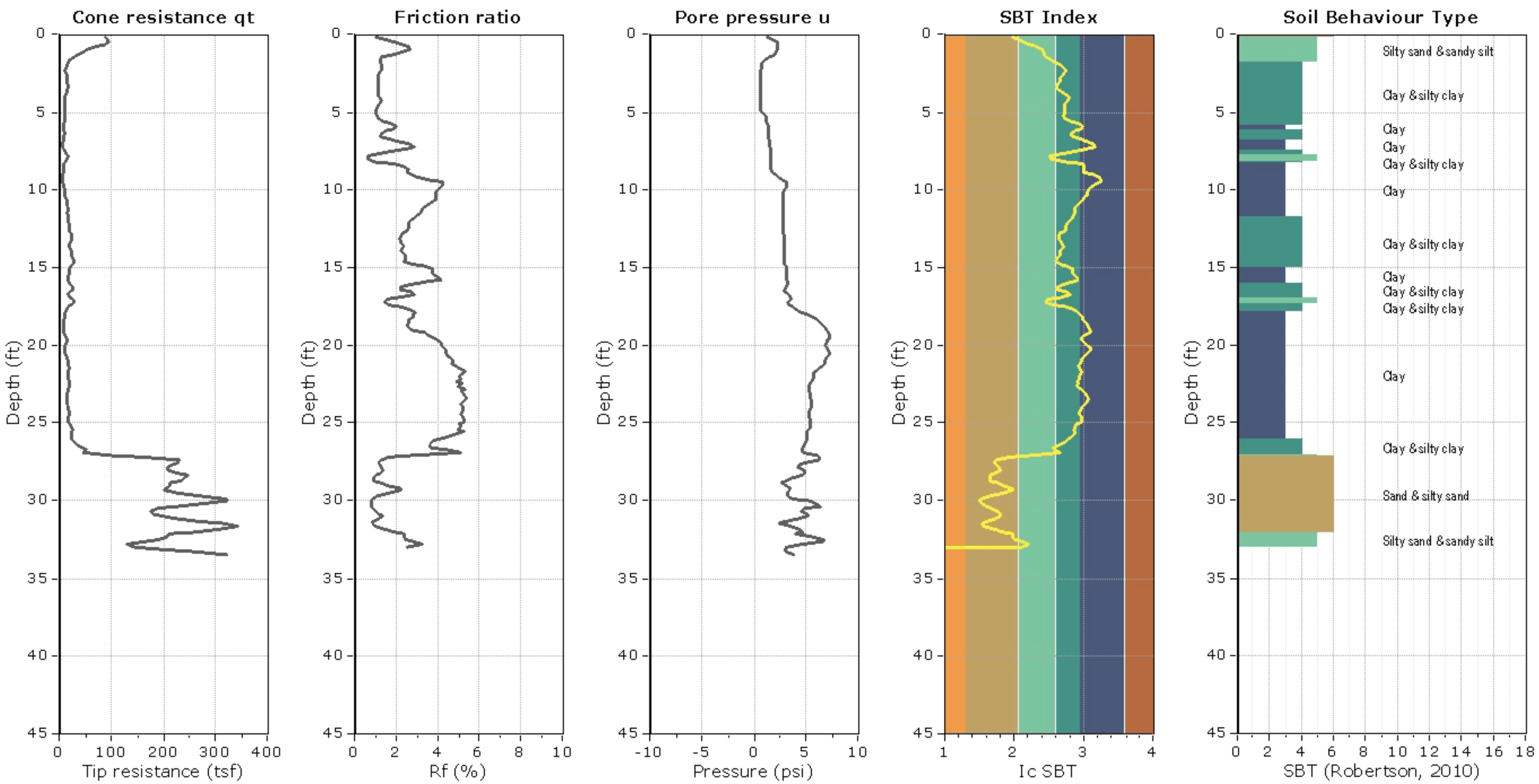
Total depth: 35.60 ft, Date: 1/25/2018
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-5

Date 02/07/18	Project No. 18-1437	Figure A-5

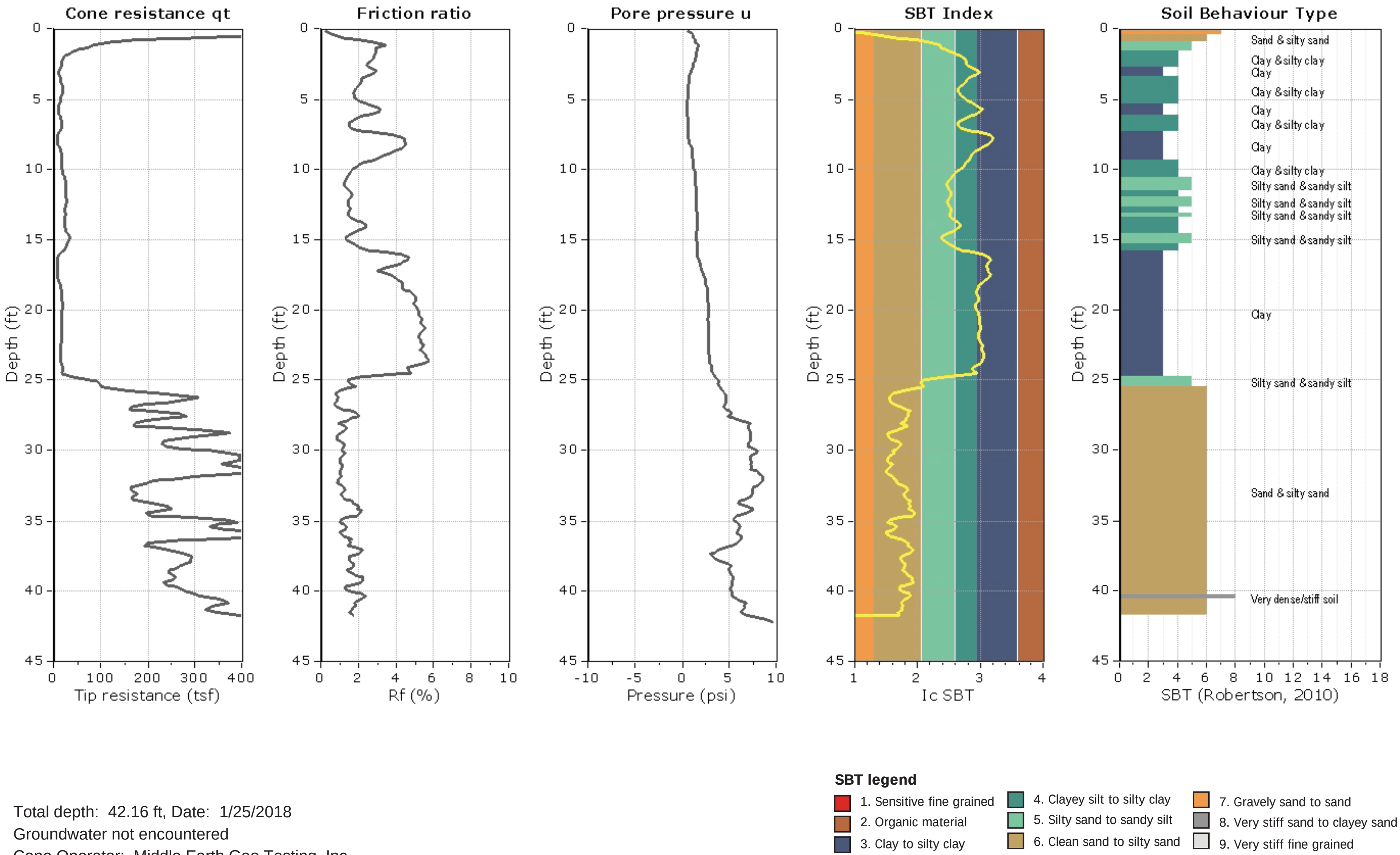


SBT legend

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

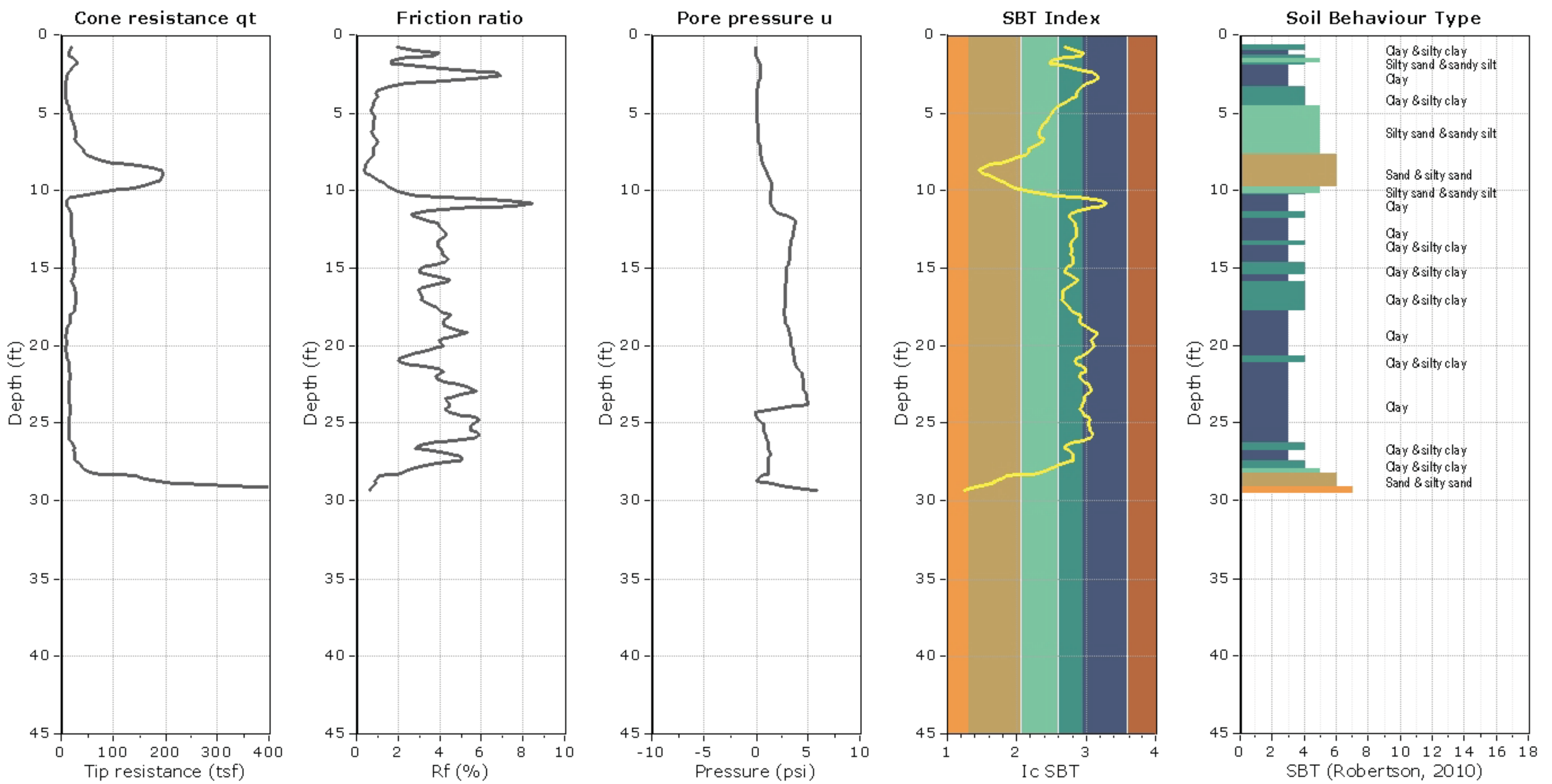
Total depth: 33.46 ft, Date: 1/25/2018
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

1410 SOUTH BASCOM AVENUE San Jose, California	**CONE PENETRATION TEST RESULTS** **CPT-6**
ROCKRIDGE GEOTECHNICAL	Date 02/07/18 · Project No. 18-1437 · Figure A-6



Total depth: 42.16 ft, Date: 1/25/2018
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

1410 SOUTH BASCOM AVENUE San Jose, California	**CONE PENETRATION TEST RESULTS** **CPT-7**
ROCKRIDGE GEOTECHNICAL	Date 02/07/18 | Project No. 18-1437 | Figure A-7



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: 29.36 ft, Date: 1/25/2018
Groundwater not encountered
Cone Operator: Middle Earth Geo Testing, Inc.

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-8

Date 02/07/18 | Project No. 18-1437 | Figure A-8

ROCKRIDGE GEOTECHNICAL

APPENDIX B

Logs of Borings

PROJECT:	1410 SOUTH BASCOM AVENUE San Jose, California	**Log of Boring B-1** PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 1/23/18 | Date finished: 1/23/18

Drilling method: 8" diameter 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: C. Proto
Drilled by: Exploration Geoservices
Rig: Mobile B56

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–0.5						6 inches of aggregate base						
0.5–3	BULK				CL	SANDY CLAY with GRAVEL (CL) brown, dry to moist						
2–3	S&H		12 12 8	14								
3–7					CL	CLAY with SAND (CL) yellow-brown, stiff, dry to moist, fine-grained sand, with silt						
3.5–5	S&H		6 4 3	5		medium stiff LL = 28, PI = 11; see Figure C-1					14.1	86
7–17					CL	SANDY CLAY (CL) yellow-brown, soft to medium stiff, moist, fine-grained sand, with silt						
8.5–10	S&H		2 3 3	4								
13.5–15	S&H		5 5 6	8		medium stiff to stiff LL = 27, PI = 10; see Figure C-1				61	14.2	107
17–29					CL	CLAY with SAND (CL) yellow-brown, medium stiff to stiff, moist, fine-grained sand, with silt						
18.5–20	S&H		5 5 7	8							22.0	99
23.5–25	S&H		5 6 10	11		stiff, decrease in silt content, trace gravel						
29–30					GP-GM	GRAVEL with SILT and SAND (GP-GM)						
28.5–30	S&H		9 9 21	21								

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1437 | Figure: B-1a

PROJECT:	**1410 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
31–33					GP-GM	GRAVEL with SILT and SAND (GP-GM) (continued) yellow, medium dense, moist, fine to coarse subrounded to subangular gravel, medium- to coarse-grained sand						
34	SPT		19 32 21	64		yellow-brown, very dense						
39	SPT		20 30 28	70								
44	SPT		23 18 14	39								
45–60												

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 0.7 and 1.2, respectively, to account for sampler type and hammer energy. SPT sampler used without liners.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1437 | Figure: B-1b

PROJECT: **1410 SOUTH BASCOM AVENUE**
San Jose, California

Log of Boring B-2

Boring location:	See Site Plan, Figure 2
Date started:	1/23/18
Date finished:	1/23/18
Drilling method:	8" diameter hollow-stem auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Wireline
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)
Logged by:	C. Proto
Drilled by:	Exploration Geoservices
Rig:	Mobile B56

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					3 inches of asphalt concrete						
0.25					6 inches of aggregate base						
0.75				SM	SILTY SAND (SM) brown, very loose to loose, moist, fine-grained sand						
3	S&H	3, 3, 3	4								
4	S&H	3, 5, 5	7		loose, trace clay						
5				CL	CLAY with SAND (CL) brown, medium stiff to stiff, moist to wet, fine-grained sand, with silt						
9	SPT	2, 2, 4	7								
12				CL	SANDY CLAY (CL) yellow-brown, stiff, moist, fine-grained sand, with silt						
14	S&H	4, 7, 8	11							17.3	106
17				CL	CLAY with SAND (CL) yellow-brown, stiff, moist, fine-grained sand, with silt						
19	S&H	4, 7, 11	13							23.1	100
24	S&H	7, 9, 11	14		stiff to very stiff						
26	ST	160-200 psi									
26				GM	SILTY GRAVEL with SAND (GM) yellow-brown, very dense, moist, fine to coarse subrounded gravel						
28	SPT	16, 22, 27	59								

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1437

Figure: B-2a

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
						SILTY GRAVEL with SAND (GM) (continued)						
31												
32												
33					GM							
34	SPT		18 18 20	46		dense, subrounded to subangular gravel						
35												
36												
37						GRAVEL with SILT and SAND (GP-GM) yellow-brown, dense, moist, fine to coarse subrounded to subangular gravel, medium- to coarse-grained sand						
38												
39	S&H		30 35 30	46								
40					GP-GM							
41												
42												
43												
44	SPT		18 27 37	77		very dense						
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

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 0.7 and 1.2, respectively, to account for sampler type and hammer energy. SPT sampler used without liners.

PROJECT: **1410 SOUTH BASCOM AVENUE**
San Jose, California

Log of Boring B-3

Boring location:	See Site Plan, Figure 2		
Date started:	1/23/18	Date finished:	1/23/18
Drilling method:	8" diameter hollow-stem auger		
Hammer weight/drop:	140 lbs./30 inches	Hammer type:	Downhole Wireline
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT), Shelby Tube (ST)		

Logged by: C. Proto
Drilled by: Exploration Geoservices
Rig: Mobile B56

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					4 inches of asphalt						
0–5				GP-GM	GRAVEL with SAND and SILT (GP-GM) yellow-brown, dense, moist, fine to coarse subangular to subrounded gravel						
2–3.5	S&H	23 23 22	32								
3.5–5	S&H	12 9 12	15		medium dense						
5–17				CL	SANDY CLAY (CL) yellow-brown, stiff, moist, fine-grained sand, with silt						
8.5–10	S&H	18 8 8	11								
13.5–15	S&H	6 6 9	11							16.2	109
17–22				CL	CLAY (CL) yellow-brown, very stiff, moist, with silt, trace fine-grained sand LL = 31, PI = 12; see Figure C-1						
18.5–20	S&H	6 10 14	17							21.4	103
22–28				CL	CLAY with SAND (CL) yellow-brown, stiff, moist, fine-grained sand, with silt						
23.5–26	ST	200-300 psi									
28–30				SP-SM	SAND with SILT and GRAVEL (SP-SM) yellow-brown, dense, moist, medium- to coarse-grained sand, fine to coarse subrounded to subangular gravel						
28.5–30	S&H	18 28 32	42								

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1437

Figure: B-3a

PROJECT: **1410 SOUTH BASCOM AVENUE**
San Jose, California

Log of Boring B-3

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					SP-SM	SAND with SILT and GRAVEL (SP-SM) (continued)						
32					GP	GRAVEL with SAND (GP) yellow-brown, very dense, moist, fine to coarse gravel						
33												
34	SPT		18 28 28	67								
35												
36												
37												
38												
39	SPT		36 26 30	67		dry to moist						
40												
41												
42					SP-SC	SAND with CLAY and GRAVEL (SP-SC) yellow-brown, dense, moist						
43												
44	SPT		30 15 22	44								
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

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 0.7 and 1.2, respectively, to account for sampler type and hammer energy. SPT sampler used without liners.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1437

Figure: B-3b

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 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

1410 SOUTH BASCOM AVENUE
San Jose, California

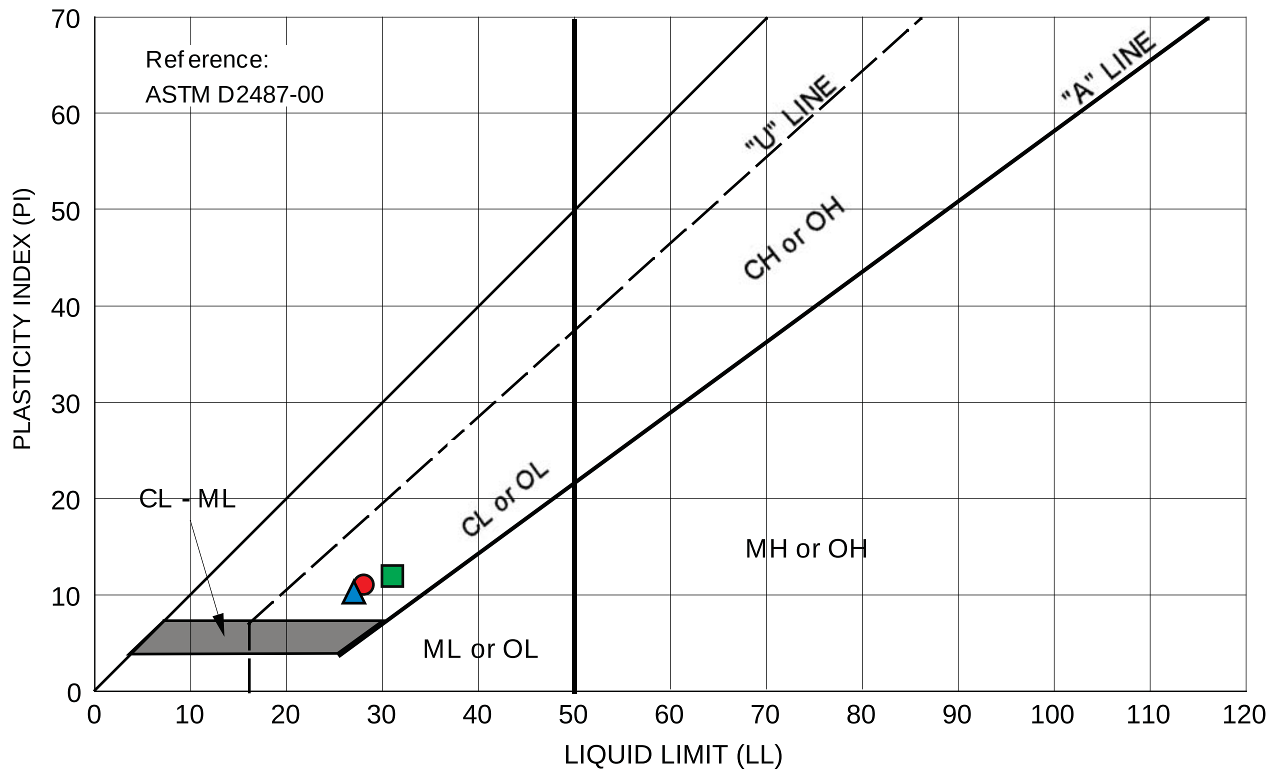
ROCKRIDGE GEOTECHNICAL

CLASSIFICATION CHART

Date 02/07/18 | Project No. 18-1437 | Figure B-4

APPENDIX C

Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 4.5 feet	CLAY with SAND (CL), yellow-brown	14.1	28	11	--
▲	B-1 at 14.5 feet	SANDY CLAY (CL), yellow-brown	14.2	27	10	61
■	B-3 at 19.0 feet	CLAY (CL), yellow-brown	21.4	31	12	--

1410 SOUTH BASCOM AVENUE
San Jose, California

ROCKRIDGE
GEOTECHNICAL

PLASTICITY CHART

Date 03/03/18 | Project No. 18-1437 | Figure C-1

Soil Analysis Lab Results

Client: Rockridge Geotechnical
Job Name: 1410 South Bascom Avenue
Client Job Number: 18-1437
Project X Job Number: S180215E
February 19, 2018

	Method	ASTM G187		ASTM D516		ASTM D512B		SM 4500-NO3-E	SM 4500-NH3-C	SM 4500-S2-D	ASTM G200	ASTM G51
Bore# / Description	**Depth**	**Resistivity** As Rec'd	Minimum	**Sulfates**		**Chlorides**		**Nitrate**	**Ammonia**	**Sulfide**	**Redox**	**pH**
	(ft)	(Ohm-cm)	(Ohm-cm)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(mg/kg)	(mg/kg)	(mg/kg)	(mV)	
B-1 #2	3.0	2,010	1,407	18	0.0018	12	0.0012	60	0.3	0.12	217	7.38

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

Please call if you have any questions.

Prepared by,

Ernesto Padilla, BSME
Field Engineer

Respectfully Submitted,

Eddie Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com

REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

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D-2: Paleontological Records Search

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Kenneth L. Finger, Ph.D.
Consulting Paleontologist

18208 Judy St., Castro Valley, CA 94546-2306   510.305.1080   klfpaleo@comcast.net

May 30, 2018

Dana DePietro
FirstCarbon Solutions
1350 Treat Boulevard, Suite 380
Walnut Creek, CA 94597

Re: Paleontological Records Search: Bascom Project (5026.0001), Campbell, Santa Clara County, California

Dear Dr. DePietro:

As per your request, I have investigated the paleontological potential and sensitivity of the geologic units in the vicinity of the proposed Bascom Project in Campbell. The project site is at 1410 S. Bascom, on the northwest side of the Southern Pacific Railroad within the southeast sector of the intersection of San Jose Road and Stokes Avenue. Its PRS location is Sec. 25, T7S, R1W, San Jose West quadrangle (1980 USGS 7.5-series topographic map). Google Earth imagery shows that the site is completely covered by commercial development (structures and parking lot).

<u>Geologic Units</u>

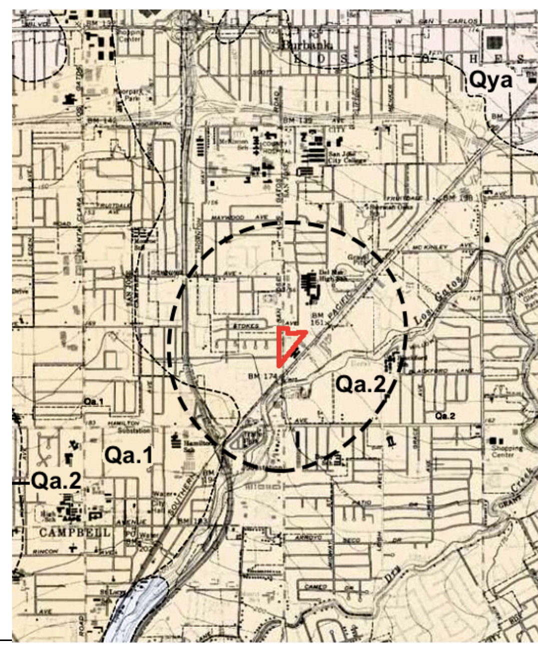
According to the part of the geologic map of Dibblee and Minch (2007) shown here, the entire project site (red outline in center) is on Holocene stream alluvium in fan deposits (Qa.2). The half-mile search area (dashed black line) also includes Holocene fan deposits (Qa.1). Farther to the north are distal alluvial fan deposits (Qya).

<u>Key to mapped units</u>

Qa.1 Alluvial fan deposits at base of slopes & upper fan areas

Qa.2 Alluvial gravel, sand, silt, and clay; represents younger stream alluvium in fan deposits

Qya Alluvial sand, fine-grained, silt, and clay; represents distal alluvial fan deposits at outer edge of fan deposits



<u>Records Search</u>

A records search on the University of California Museum of Paleontology database was not performed because all of the geologic units in the vicinity of the Bascom project are of Holocene age, which are too young to have any fossil potential. Older units are not in the vicinity and are likely to be too deeply buried at the site to be impacted by project-related earth-disturbing activities.

Remarks and Recommendations

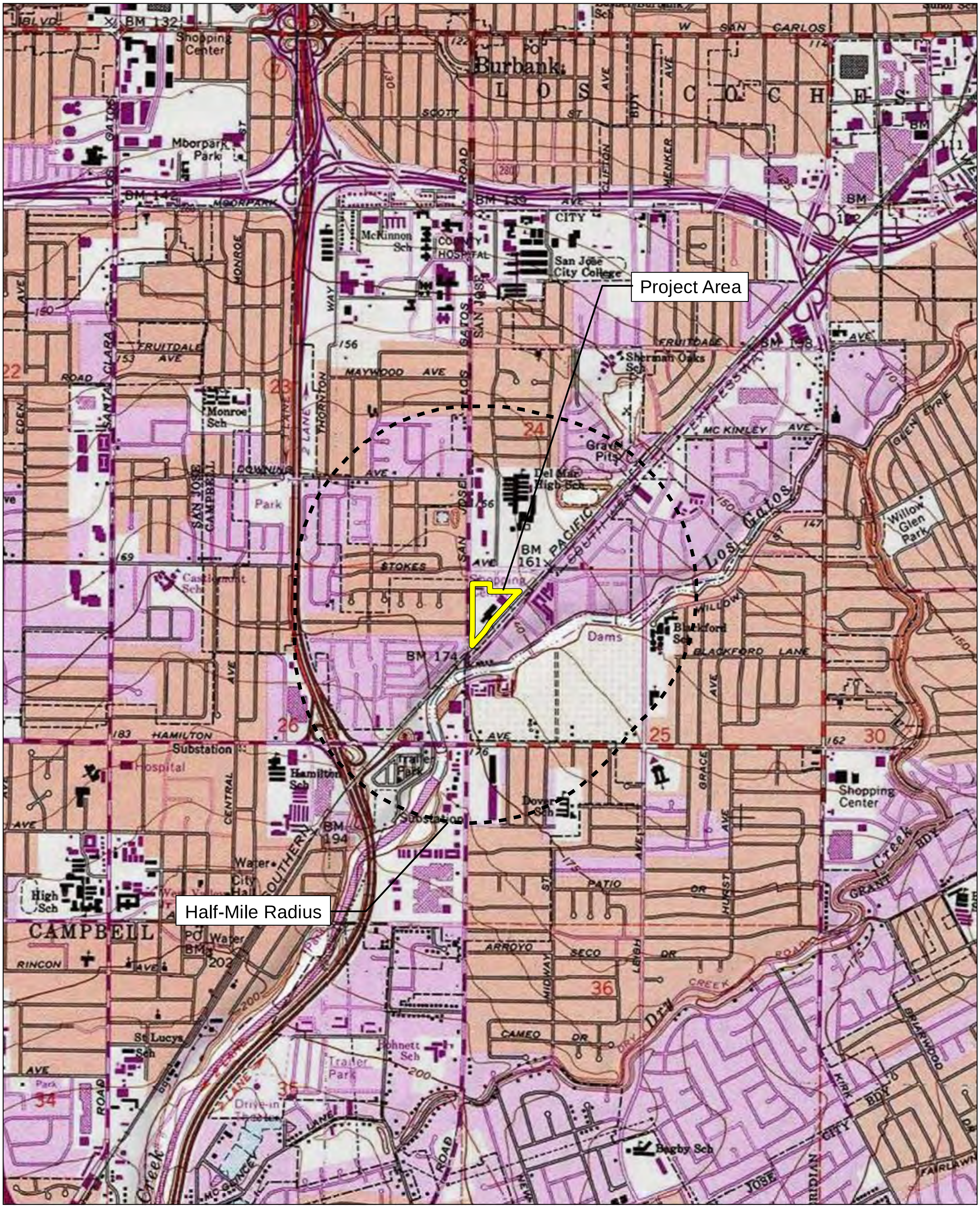
Because it is highly unlikely that potentially fossiliferous deposits will be encountered at the Bascom site, t here is no need for a pre-construction paleontological walkover survey or paleontological monitoring of project-related excavations. This report therefore satisfies CEQA guidelines and concludes the paleontological mitigation for this project.

Sincerely,

Ken Finger

Reference Cited

Dibblee, T.W., Jr., and Minch, J.A., 2007, Geologic map of the Cupertino and San Jose West quadrangles, Santa Clara and Santa Cruz counties, California: Dibblee Geology Center Geologic Map #DF-351. Scale 1:24,000.



Source: USGS San Jose West 7.5' Quadrangle / T7S,R1W,sec25

FIRSTCARBON SOLUTIONS™

2,000 1,000 0 2,000 Feet

Record Search Map

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