Appendix D: Geotechnical Report

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ROCKRIDGE GEOTECHNICAL

Prepared for **Habitat for Humanity East Bay/Silicon Valley**

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED RESIDENTIAL DEVELOPMENT
101 SOUTH JACKSON AVENUE
SAN JOSE, CALIFORNIA

July 23, 2019
Project No. 18-1627

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ROCKRIDGE GEOTECHNICAL

July 23, 2019
Project No. 18-1627

Mr. Robert Simonds
Habitat for Humanity East Bay/Silicon Valley
2619 Broadway
Oakland, California 94612

Subject: Geotechnical Investigation Report
Proposed Residential Development
101 South Jackson Avenue
San Jose, California

Dear Mr. Simonds,

We are pleased to present our geotechnical investigation report prepared for the proposed residential development to be constructed at 101 South Jackson Avenue in San Jose. Our geotechnical study was performed in accordance with our proposal dated March 12, 2018.

The project site is located on the southwestern side of South Jackson Avenue between Alum Rock Avenue and East San Antonio Street. The subject property is a relatively level, rectangular-shaped parcel with plan dimensions of approximately 100 by 370 feet. The front of the site is currently occupied by a single-family home. The remainder of the site is vacant and bordered by single-family homes to the southeast and southwest, and a church, street terminus, and single-family residence to the northwest.

Proposed improvement plans call for Woodset Drive to extend through the western end of the site, leaving a small portion of the western end of the site separated from the remainder of the site by the street. Plans also call for construction of four 2-story townhome buildings containing a total of 14 dwelling units. Three of the townhome buildings would be located along the northern side of the site and one would be located at the western end of the site. Other site improvements include surface parking areas and driveways.

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concerns at the site are: (1) the presence of moderately expansive near-surface clay, and (2) providing adequate foundation support. We conclude the proposed buildings may be supported on a well-reinforced mat foundation.

The recommendations contained in our report are based on a limited subsurface exploration. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe excavation, grading, and installation of foundations, 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 yours,
ROCKRIDGE GEOTECHNICAL, INC.

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Enclosure

ROCKRIDGE GEOTECHNICAL

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GEOTECHNICAL INVESTIGATION REPORT
PROPOSED RESIDENTIAL DEVELOPMENT
101 SOUTH JACKSON AVENUE
San Jose, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. (Rockridge) for the proposed residential development to be constructed at 101 South Jackson Avenue in San Jose, California. The project site is located on the southwestern side of South Jackson Avenue between Alum Rock Avenue and East San Antonio Street, as shown on the Site Location Map, Figure 1.

The subject property is a relatively level, rectangular-shaped parcel with plan dimensions of approximately 100 by 370 feet, as shown on the Site Plan, Figure 2. The front of the site is currently occupied by a single-family home. The remainder of the site is vacant and bordered by single-family homes to the southeast and southwest, and a church, street terminus, and single-family residence to the northwest.

Proposed improvement plans call for Woodset Drive to extend through the western end of the site, leaving a small portion of the western end of the site separated from the remainder of the site by the street (see Figure 2). Plans also call for construction of four 2-story townhome buildings containing a total of 14 dwelling units. Three of the townhome buildings would be located along the northern side of the site and one would be located at the western end of the site. Other site improvements include surface parking areas and driveways.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal dated March 2, 2018. Our scope of services consisted of evaluating the subsurface conditions at the site by drilling three test borings and performing one cone penetration test (CPT), performing laboratory testing on selected soil samples, and performing engineering analyses to develop conclusions and recommendations regarding:

- subsurface conditions
- 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 foundation settlement
- lateral earth pressure for design of site retaining walls
- surface drainage and bioswales
- subgrade preparation for slab-on-grade floors and concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- pavement sections for asphalt concrete and Portland cement concrete
- 2016 California Building Code (CBC) site class and design spectral response acceleration parameters
- corrosivity of the near-surface soil
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Subsurface conditions at the site were explored by advancing drilling three test borings, performing one CPT, and performing laboratory testing on selected soil samples. Prior to performing the field exploration, we obtained a drilling permit from Santa Clara Valley Water District (SCVWD) and contacted Underground Service Alert (USA) to notify them of our work, as required by law. Details of our field investigation and laboratory testing are presented in this section.

3.1 Test Borings

Three test borings, designated as B-1 through B-3, were drilled on April 8, 2019 at the approximate locations shown on Figure 2. The borings were drilled by Benevent Building of Concord, California using a portable drill rig equipped with four-inch-diameter solid-stem flight augers. All three borings were drilled to depths of approximately 26-1/2 feet below the ground surface (bgs). 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 A on Figures A-1 through A-3. The soil was classified in accordance with the classification system presented on Figure A-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 tubes.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.

The samplers were driven with a 140-pound safety hammer falling about 30 inches per drop using a rope-and-cathead pulley system. The samplers were driven 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 and approximate hammer energy. The blow counts used for this conversion were the last two blow counts. The converted SPT N-values are presented on the boring logs.

Upon completion of drilling, the boreholes were backfilled with cement grout to the ground surface per SCVWD requirements. The soil cuttings generated from the borings were left on site.

3.2 Cone Penetration Test

Middle Earth GeoTesting, Inc. of Orange, California performed one CPT, designated as CPT-1, on February 7, 2019 at the approximate location shown on Figure 2. The CPT was advanced to a depth of 51 feet bgs by hydraulically pushing a 1.7-inch-diameter cone-tipped probe with a projected area of 15 square centimeters into the ground. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the types and approximate strength characteristics of the soil encountered.

The CPT log showing tip resistance, friction ratio, and pore pressure, as well as interpreted soil behavior type, is presented in Appendix A on Figure A-5. Upon completion, the CPT was backfilled with cement grout in accordance with SCVWD requirements.

3.3 Laboratory Testing

We re-examined each soil sample in the office to confirm field classification and selected representative samples for laboratory testing. Soil samples were tested to measure moisture content, dry density, Atterberg limits[1] (plasticity index), and corrosion potential. The laboratory test results are presented on the boring logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS

A regional geologic map prepared by Graymer (2000), a portion of which is presented on Figure 3, indicates the site is underlain by Holocene-age alluvium (Qha). The results of our field investigation indicate the site is blanketed by 4 to 6 feet of stiff to very stiff clay with variable sand content. The results of Atterberg limits tests performed on three samples of the near-surface clay indicate the near-surface clay is moderately expansive[2] with plasticity indices (PIs) of 22 to 25.

Where explored, the near-surface clay is underlain by medium dense sand with variable amounts of silt and clay that extends to depths of 9 to 11 feet bgs. This sand is underlain by alluvium that generally consists of stiff to very stiff clay and silt with variable amounts of sand that extends to the maximum depth explored of 51 feet bgs. CPT-1 encountered occasional lenses of sand with

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[1] Atterberg limits are an indirect measure of the expansion potential of the soil.

variable silt content interbedded in the clay alluvium; the sand lenses are relatively thin (less than one foot thick).

Groundwater was measured in Borings B-1 and B-2 immediately after completion of drilling at depths of 12.2 and 13.8 feet bgs, respectively. Groundwater was not encountered in Boring B-3. In addition, a pore pressure dissipation (PPD) test performed in CPT-1 indicated the depth to groundwater was about 17 feet bgs. Considering the low permeability of the clayey soil, the groundwater levels encountered in the borings and CPT may not represent stabilized groundwater conditions. To further evaluate depth to high groundwater, we reviewed the California Geological Survey report titled *Seismic Hazard Zone Report for the San Jose East 7.5-Minute Quadrangle, Santa Clara County, California*, dated 2002. This report indicates the historic high groundwater in the site vicinity is approximately 15 feet bgs. The depth to groundwater may vary several feet seasonally, depending on the amount of rainfall.

5.0 SEISMIC CONSIDERATIONS

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards, including ground shaking, ground surface rupture, liquefaction[3], lateral spreading[4] and cyclic densification.[5] The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity

The site is in the Coast Ranges geomorphic province that is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the

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2 Expansive soil undergoes volumetric changes with changes in moisture content (i.e. it shrinks when dried and swells when wetted).

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

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

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

collision of the Farallon and North American plates and subsequent shearing along the San Andreas Fault system. Movements along this plate boundary in the Northern California region occur along right-lateral strike-slip faults of the San Andreas Fault system.

The major active faults in the area are the Hayward and Calaveras faults. These and other faults in the region are shown on Figure 4. Active faults within a 50-kilometer radius of the site, the distance from the site and mean characteristic moment magnitude[6] [2007 Working Group on California Earthquake Probabilities (USGS 2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Total Calaveras	8.5	Northeast	7.03
Total Hayward	11	North	7.00
Total Hayward-Rodgers Creek	11	North	7.33
Monte Vista-Shannon	16	South	6.50
N. San Andreas - Peninsula	24	Southwest	7.23
N. San Andreas (1906 event)	24	Southwest	8.05
N. San Andreas - Santa Cruz	25	Southwest	7.12
Greenville Connected	31	East	7.00
Zayante-Vergeles	32	Southwest	7.00
Mount Diablo Thrust	42	North	6.70
San Gregorio Connected	48	West	7.50
Great Valley 7	50	Northeast	6.90

6 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.

Since 1800, 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 most recent major earthquake to affect the Bay Area was the Loma Prieta Earthquake of October 17, 1989 with an M_w of 6.9. This earthquake occurred in the Santa Cruz Mountains about 36 kilometers south of the site.

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

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

5.2 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong ground shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction, lateral spreading, and cyclic densification. We used the results of the borings and CPT 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. Due to the proximity of the site to active faults (Table 1), 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 Liquefaction-Induced Settlement

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 within a liquefaction hazard zone, as shown on Figure 5 from the map titled *State of California, Seismic Hazard Zones, San Jose East Quadrangle, Official Map,* prepared by the California Geological Survey (CGS), dated January 17, 2001. We evaluated the liquefaction potential of soil encountered below groundwater at the site using data collected in our CPT with consideration of visual classification of soil samples obtained during drilling. Our liquefaction analyses were performed using the methodology proposed by Boulanger & Idriss (2014).

Our analyses were performed using an assumed high groundwater depth of 15 feet bgs. In accordance with the 2016 CBC, we used a peak ground acceleration of 0.51 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Risk-Targeted Maximum Considered Earthquake (MCE_R) peak ground acceleration adjusted for site effects (PGA_M). We also used a moment magnitude 7.3 earthquake, which is consistent with the mean characteristic moment magnitude for the Hayward Fault, as presented in Table 1. A summary of our liquefaction analyses is presented in Appendix C.

The results of the liquefaction analysis indicate that, except for relatively thin lenses (i.e. about one foot thick) of medium dense "silty sand" between depths of about 35 and 40 feet bgs, the soils at the site are sufficiently cohesive and/or dense to resist liquefaction. We estimate total ground surface settlement associated with liquefaction (referred to as post-liquefaction reconsolidation) following a major earthquake on a nearby fault will be 1/4 inch or less. Based on these findings, we conclude the potential for liquefaction-induced building damage is very low.

Considering the relatively flat site grades, as well as the depth, relative thickness, and relative density of the potentially liquefiable layers, we conclude the risk of lateral spreading and other types of ground failure associated with liquefaction occurring at the site is also very low.

5.2.3 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The soil encountered above the groundwater table is predominantly clay and medium dense sand with variable amounts of clay and silt. We conclude these soils are not susceptible to cyclic densification because of their cohesion and/or relative density.

5.2.4 Ground Surface Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake

Fault Zoning Act, 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 DISCUSSIONS AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concerns at the site are: (1) the presence of moderately expansive near-surface clay, and (2) providing adequate foundation support. These and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

6.1 Expansive Soil

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

At expansive soil sites, it is critical to properly manage surface and subsurface drainage to prevent water from collecting beneath pavements, slabs and foundations. If permeable pavements, tree wells, irrigated landscaped zones, and storm water infiltration basins will be constructed close to the proposed buildings, they should incorporate design elements that prevent saturation of the soil adjacent to and below building foundations. While the objective of permeable pavement systems and infiltration basins is to allow for water storage and infiltration, we conclude that infiltration into the subgrade soil is not feasible at this site due to the low permeability of the moderately expansive clay. Furthermore, from a geotechnical standpoint, water should not be allowed to collect alongside or beneath the building foundations, pavements and flatwork. This can be achieved by providing subdrain systems and impermeable liners beneath permeable surfaces and installing vertical barriers between permeable surfaces underlain by subdrains and non-permeable surfaces underlain by conventional aggregate base.

6.2 Foundation Support and Settlement

The site is underlain by firm native alluvium that has moderate strength and relatively low compressibility that can provide adequate foundation support. As previously discussed in Section 6.1, the near-surface clay is moderately expansive. Based on our experience on previous Habitat for Humanity East Bay/Silicon Valley projects, we conclude the most appropriate foundation type for the proposed buildings consists of a well-reinforced concrete mat foundation. The mat can minimize distortion of the superstructure from shrink/swell of the moderately expansive near-surface clay. The edges of the mat should be deepened to reduce the potential for infiltration of water beneath the mat.

We estimate total settlement of the two-story residential buildings supported on a mat foundation would be less than 3/4 inch and differential settlement will be less than about 1/2 inch over a horizontal distance of 30 feet.

6.3 Soil Corrosivity

Laboratory testing was performed by Project X Corrosion Engineering of Murrieta, California on a sample of soil obtained from Boring B-2 at a depth of 3.5 feet bgs. The results of the test are presented on Figure B-2 in Appendix B.

The resistivity test results (938 ohm-cm for saturated condition) indicate the near-surface soil is "corrosive" to buried metallic structures. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron may need to be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.

The chloride ion concentration (24 mg/kg) and pH (9.4) indicate the near-surface soil is “negligibly corrosive” to buried metallic structures and reinforcing steel in concrete structures below ground. The results also indicate the sulfate ion concentration (30 mg/kg) is sufficiently low such that sulfates do not to pose a threat to buried concrete.

6.4 Construction Considerations

The soil to be excavated for the new foundations and underground utilities is expected to be mostly clay. If site grading is performed during the rainy season, the clay will likely be wet and will have to be dried before compaction can be achieved. Heavy rubber-tired equipment could cause excessive deflection (pumping) of the wet clay and, therefore, should be avoided. If the project schedule or weather conditions do not permit enough time for drying of the soil by aeration, the subgrade can be treated with lime prior to compaction. The appropriate amount of lime should be determined based on laboratory testing of the soil to be treated. It is also important that the moisture content of subgrade soil is sufficiently high to reduce the expansion potential. If the grading work is performed during the dry season, moisture-conditioning may be required.

Excavations that will be deeper than five feet and will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes. We judge temporary slopes with a maximum inclination of 1:1 (horizontal to vertical) should be stable, provided the slope is not surcharged by adjacent structures, construction equipment, or stockpiled soil.

7.0 RECOMMENDATIONS

Our recommendations for site preparation and grading, foundation design, seismic design, and other geotechnical aspects of the project are presented in this section.

7.1 Site Preparation and Grading

Site demolition should include removal of vegetation and all existing pavements, foundations, and underground utilities. Any vegetation and the upper few inches of organic topsoil 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 the building subgrade should be removed. 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 footprint of the proposed improvements 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.

The soil (native or fill) exposed at the base of the excavation(s) should be scarified to a depth of at least eight inches, moisture-conditioned to at least three percent above moisture content, and compacted to at least 90 percent relative compaction.[7] The imported fill material should then be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned, and compacted in accordance with the requirements provided below in Table 2. The subgrade beneath concrete and asphalt concrete pavements should be firm and non-yielding under construction equipment wheel loads.

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

TABLE 2
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
Building pad subgrade	90+	3+% above optimum
General fill – native clay	90+	3+% above optimum
General fill – select fill	90+	Near optimum
Utility trench backfill – native clay	90+	3+% above optimum
Utility trench backfill – imported fill	90+	Near optimum
Utility trench - clean sand or gravel	95+	Near optimum
Vehicular pavement subgrade – select fill	95+	Near optimum
Vehicular pavement - aggregate base	95+	Near optimum
Exterior slabs – subgrade	90+	3+% above optimum
Exterior slabs – select fill	90+	Near optimum

7.1.1 Select Fill

Select fill should consist of onsite or imported soil that is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the geotechnical engineer. Sand or gravel with less than five percent fines (i.e., particles passing the No. 200 sieve), however, should be avoided because these soil types are easily disturbed and tend to cave in utility trenches. Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned near optimum moisture content, and compacted to at least 90 percent relative compaction. Samples of proposed select fill material should be submitted to the geotechnical engineer at least three business days prior to use at the site.

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

7.1.2 Exterior Concrete Flatwork

We recommend a minimum of six inches of Class 2 aggregate base (AB) or other suitable select fill be placed beneath the flatwork; the select fill should extend at least six inches beyond the slab edges. Select fill beneath concrete flatwork should be moisture-conditioned and compacted in accordance with the requirements provided above in Table 2.

7.1.3 Utility Trench Backfill

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

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

The proposed building foundation should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of any utility trench that is running parallel with the building foundation. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with controlled low-strength material (i.e., sand-cement slurry) with a 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).

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

7.1.4 Drainage and Landscaping

Positive surface drainage should be provided around the buildings to direct surface water away from the foundations. To reduce the potential for water ponding adjacent to the buildings, we recommend the ground surface within a horizontal distance of five feet from the buildings slope down away from the buildings 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 foundations. The use of water-intensive landscaping around the perimeter of the buildings should be avoided to reduce the amount of water introduced to the expansive clay subgrade. Bioswales constructed at the site should be provided with underdrains and/or drain inlets because of the low permeability of the near-surface soil. Bioswales should constructed be no closer than five feet from buildings.

Prior experience and industry literature indicate that some species of high water-demand[8] trees can induce ground-surface settlement by drawing water from the expansive clay, causing it to shrink. Where these types of trees are planted near buildings, the ground-surface settlement may result in damage to structure. This problem usually occurs 10 or more years after planting, as the trees reach mature height. To reduce the risk of tree-induced, building settlement, we recommend trees of the following genera are not planted within 25 feet of the proposed buildings: *Eucalyptus, Populus, Quercus, Crataegus, Salix, Sorbus* (simple-leafed), *Ulmus, Cupressus, Chamaecyparis,* and *Cupressocyparis.* Because this is a limited list and does not

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[8] "Water-demand" refers to the ability of the tree to withdraw large amounts of water from the soil subgrade, rather than soil suction exerted by the root system.

include all genera that may induce ground-surface settlement, a tree specialist should be consulted prior to selection of trees to be planted at the site.

7.2 Foundations

The proposed buildings should be supported on well-reinforced concrete mats. We recommend the mat foundations be at least 12 inches thick. The edges of the mat should be thickened such that the mat edge is bottomed at least nine inches below the adjacent exterior grade.

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

- Climatic rating (C_w) – 15
- Equivalent Plasticity Index (PI) – 25
- Slope Correction Coefficient (C_s) – 1.0
- Consolidation Correction Coefficient (C_o) – 0.85

The maximum bearing pressure beneath the mat should not exceed 3,000 pounds per square foot (psf) under dead-plus-live-load conditions and 4,000 psf under total load conditions.

Lateral loads can be resisted by a combination of passive pressure on the vertical faces of the foundation and friction along the bottom of the mat. Passive resistance may be calculated using a uniform pressure of 1,500 psf for transient loads and 270 pounds per cubic foot (pcf) for sustained loads. The upper one foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30 where the mat is in contact with native soil and 0.20 where the mat is in underlain by a vapor retarder. These values include a factor of safety of at least 1.5 and may be used in combination without further reduction.

The mat subgrade should be free of standing water, debris, and disturbed materials prior to placing concrete. The subgrade should be wetted following excavation and maintained in a moist condition until it is covered with the vapor retarder. We should check the foundation subgrade prior to placement of the vapor retarder.

7.3 Vapor Retarder

We recommend installing a capillary moisture break and water vapor retarder beneath the mat slab. The vapor retarder may be placed directly on the prepared soil subgrade and should meet the requirements for Class A vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and can result in excessive vapor transmission through the mat. Where the concrete is poured directly over the vapor retarder, we recommend the w/c ratio of the concrete not exceed 0.45. Water should not be added to the concrete mix in the field. If necessary, workability should be increased by adding plasticizers. In addition, the mat should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer’s requirements.

7.4 Site Retaining Walls

Retaining walls should be designed to resist static lateral earth pressures, vehicular surcharge pressures, and surcharges from adjacent foundations, where appropriate. For the design of unrestrained (retaining) walls, we recommend using an active pressure computed using an equivalent fluid weight of 40 pcf (triangular distribution). Restrained walls, including elevator pit walls, should be designed using an equivalent fluid weight of 60 pcf. Provided the walls retain less than six feet of soil, we conclude it is not necessary to design the walls for an extra increment of seismic earth pressure.

The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads from vehicles. Where the site retaining wall is subject to vehicular loading within 10 feet of the wall, an additional uniform lateral surcharge pressure of 50 psf should be applied to the upper 10 feet of the wall.

The lateral earth pressures recommended are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the walls. The pipe should be surrounded by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or 3/4-inch drain rock wrapped in filter fabric (Mirafi NC or equivalent). The collector pipe should be sloped to drain to a sump or another suitable outlet.

If backfill is required behind basement walls, the walls should be braced, or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the structural engineer).

7.5 Pavement Design

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

7.5.1 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. The final soil subgrade in pavement areas will likely consist of moderately expansive clay. Based on our experience with expansive clay sites, we selected a minimum resistance value (R-value) of 5 for asphalt concrete pavement design. Recommended pavement sections for traffic indices (TIs) ranging from 4.5 to 6.5 are presented in Table 3. The civil engineer for the project should check that the TI's presented in this report are appropriate for the intended use. We can provide additional pavement sections for different TIs upon request.

TABLE 3
AC Pavement Sections

TI	Asphaltic Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4.5	2.5	9.5
5.0	3.0	10.0
5.5	3.0	12.0
6.0	3.5	13.0
6.5	4.0	13.5

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

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

Where pavement is constructed near bio-swales or other storm water treatment areas, curbs should be deepened so that the base is founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal:vertical) from the base of the bio-swale/treatment area. Further, deepened curbs near bioswales may require some type of lateral restraint. The need for lateral restraint of deepened curbs should be evaluated during design of the biotreatment features.

7.5.2 Rigid (Portland-Cement Concrete) Pavement Design

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

The modulus of rupture and unconfined compressive strength of the concrete should be at least 500 and 3,200 psi at 28 days, respectively. Contraction joints should be placed at a 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. For loading docks or bus stops, we recommend the concrete slab be reinforced with a minimum of No. 4 bars at 16 inches on center in both directions.

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

7.6 Porous Concrete Pavement or Pavers

If porous concrete pavement or porous pavers are used, the Class 2 aggregate base should be replaced with the same thickness of 3/4-inch, open-graded crushed rock (i.e., drain rock). Because the permeability of the near-surface native clay will be very low, it will be necessary to install a subdrain to collect surface water that infiltrates through the pavement and direct it to an appropriate discharge point. The soil subgrade beneath the drain rock should slope at a gradient of at least one percent toward perforated drain pipes spaced at no more than 20 feet apart. The drain pipes should consist of four-inch-diameter, perforated Schedule 40 PVC pipes installed in a 12-inch-wide by 12-inch-deep trench. The drain pipes should also slope at a gradient of at least one percent to the discharge point.

7.7 Seismic Design

For design in accordance with the 2016 CBC, we recommend Site Class D be used. The latitude and longitude of the site are 37.3568° and -121.8422°, respectively. Hence, in accordance with the 2016 CBC, we recommend the following:

- S_S = 1.5g, S_1 = 0.6g
- S_{MS} = 1.5g, S_{M1} = 0.9g
- S_{DS} = 1.0g, S_{D1} = 0.6g
- 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, grading, fill placement and compaction, and foundation 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 investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report assume that the soil and groundwater conditions do not deviate appreciably from those disclosed in the exploratory borings and CPT. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The 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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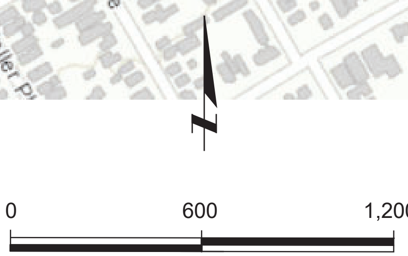
FIGURES

Base map: maps.conservation.ca.gov



101 SOUTH JACKSON AVENUE
San Jose, California

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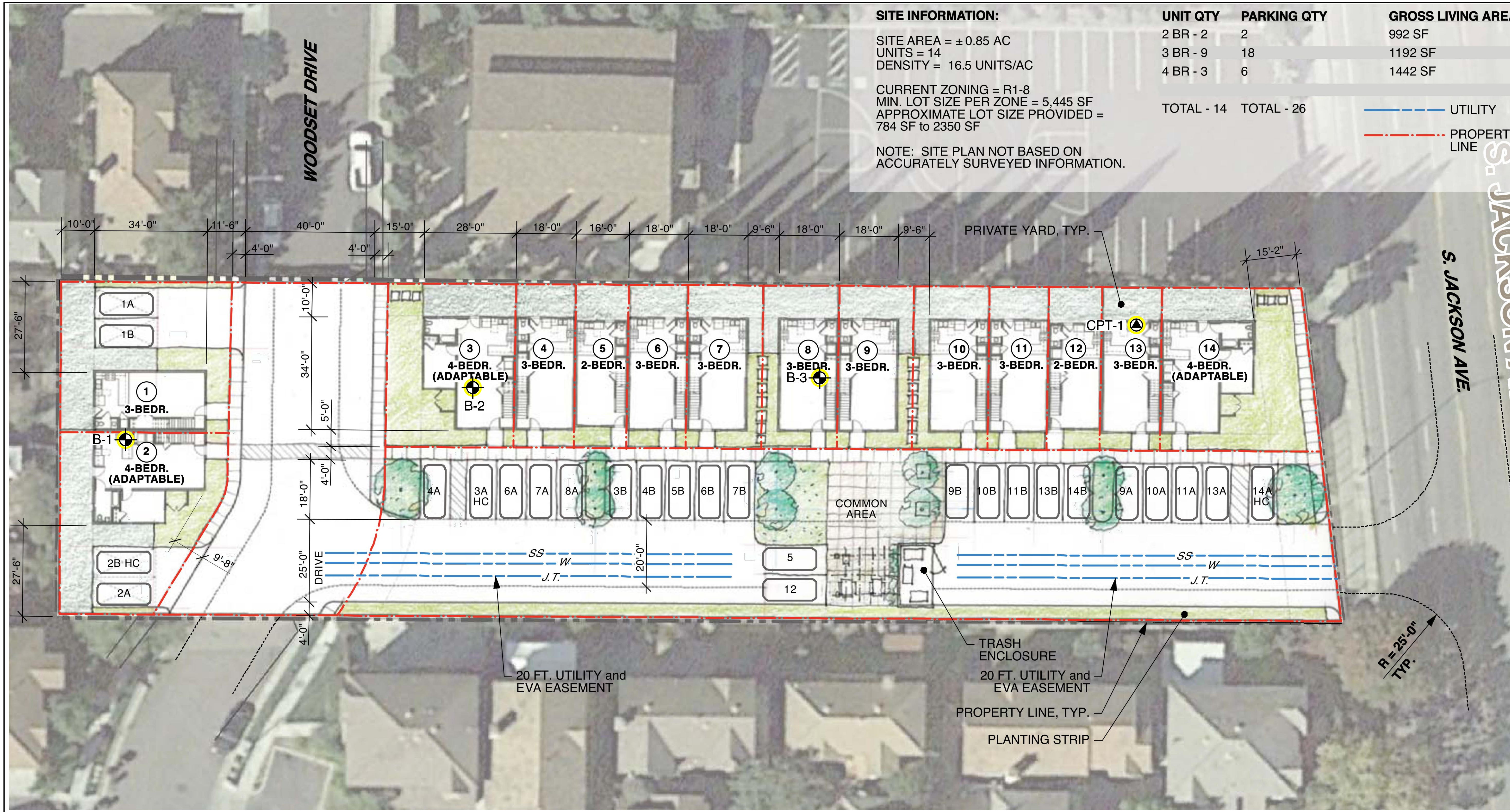
SITE LOCATION MAP

Date 04/22/19	Project No. 18-1627	Figure 1



EXPLANATION

CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical Inc., February 7, 2019

B-1 — Approximate location of boring by Rockridge Geotechnical Inc., April 8, 2019

Project limits

Reference: Base map from a drawing titled "Site Plan Sketch", by Geoffrey Holton & Associates, dated December 18, 2017.

101 SOUTH JACKSON AVENUE
San Jose, California

SITE PLAN

Date 04/23/19	Project No. 18-1627	Figure 2

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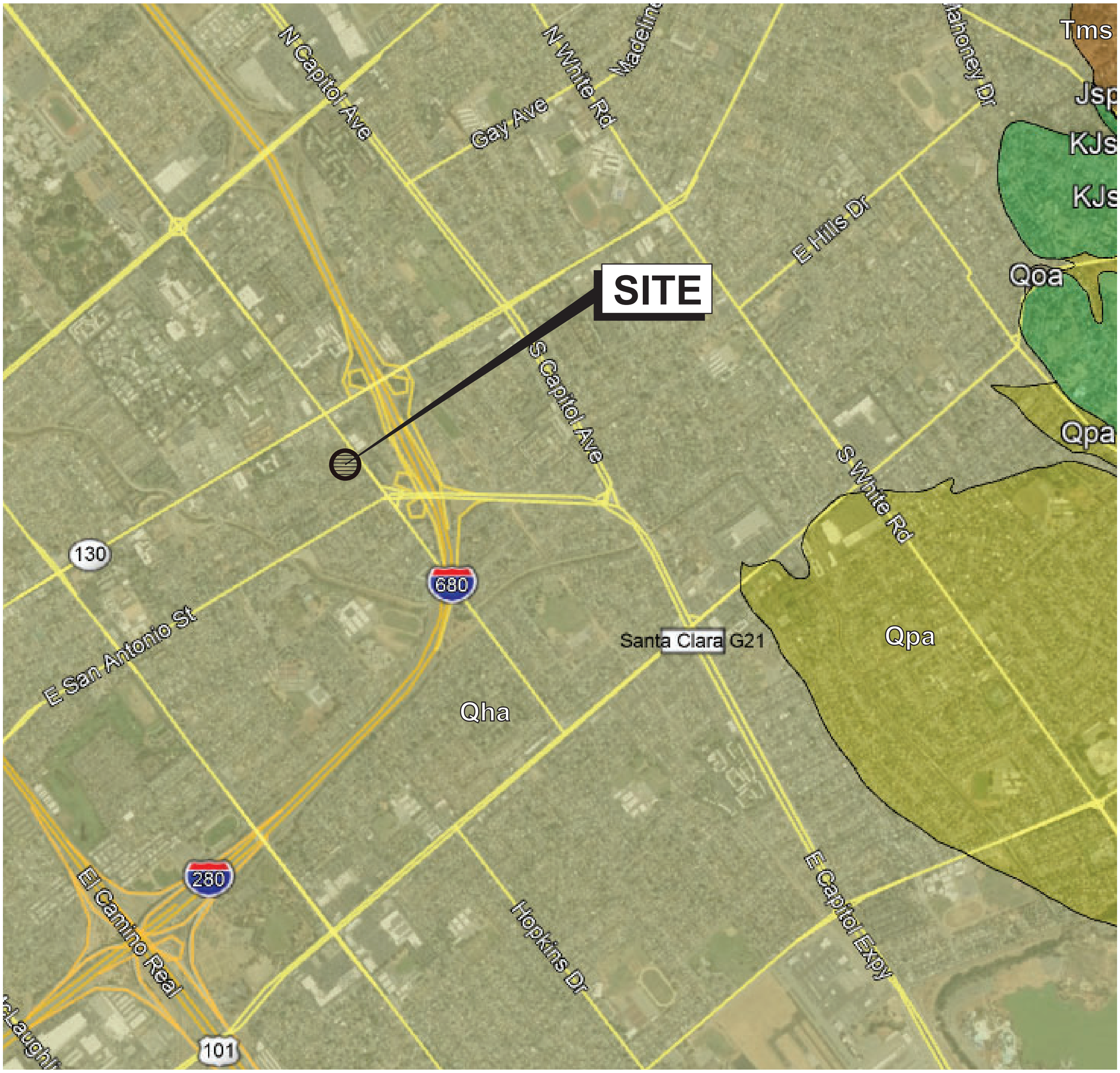

Base map: Google Earth with U.S. Geological Survey (USGS), Santa Clara County, 2018.

EXPLANATION

Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)
Qoa	Alluvium (early (Pleistocene)
Tms	Sedimentary rocks (Miocene)
KJs	Great Valley complex sedimentary rocks (Early Cretaceous and (or) Late Jurassic)
Jsp	Great Valley complex serpentinite (Jurassic)

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

0 2000 4000 Feet

Approximate scale

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REGIONAL GEOLOGIC MAP

Date 04/22/19 | Project No. 18-1627 | 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

101 SOUTH JACKSON AVENUE San Jose, California	**REGIONAL FAULT MAP**
ROCKRIDGE GEOTECHNICAL	Date 04/23/19 | Project No. 18-1627 | Figure 4

Reference: https://maps.conservation.ca.gov/cgs/EQZApp/app/ (California Geological Survey, 2019)
San Jose East Quadrangle, Released January 17, 2001.

EXPLANATION

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

Alquist-Priolo Special Studies Fault Zone

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

0 0.4 0.8 Miles

Approximate scale

101 SOUTH JACKSON AVENUE
San Jose, California

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SEISMIC HAZARDS ZONE MAP

Date 04/23/19	Project No. 18-1627	Figure 5

ROCKRIDGE GEOTECHNICAL

APPENDIX A

Boring Logs and Cone Penetration Test Results

PROJECT: **101 SOUTH JACKSON AVENUE**
San Jose, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2
Date started: 4/8/19 | Date finished: 4/8/19
Drilling method: 4" Solid-Stem Flight Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: D. Landkamer
Drilled by: Benevent Building
Rig: Portable Hydraulic Unit

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1.6				CL	CLAY (CL) brown to dark brown, very stiff, moist LL = 43, PI = 25; see Appendix B						
1.6–2.5	S&H	9 11 17	20	CL	CLAY with SAND (CL) olive-brown, very stiff, moist, fine sand					17.1	
3.6–4.5	S&H	6 9 14	16	CL	LL = 42, PI = 24; see Appendix B increase sand content at bottom of sample					18.9	108
5–6.5	SPT	9 11 10	25	SP-SM	SAND with SILT (SP-SM) olive-brown, medium dense, moist						
10–11.5	SPT	3 3 5	10	SP-SM / CL	CLAY (CL) olive-brown mottled gray, stiff, moist, trace fine sand ▽ (4/8/2019; 9:44 AM)						
15–16.5	SPT	3 4 6	12	CL	SANDY CLAY (CL) olive-brown mottled gray, stiff, moist, fine to medium sand						
20–21.5	SPT	5 7 7	17	CL	olive-brown, very stiff						
25–26.5	SPT	3 5 6	13	CL	olive-brown mottled gray, stiff						

Boring terminated at a depth of 26.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 12.2 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1627 | Figure: A-1

PROJECT:	101 SOUTH JACKSON AVENUE San Jose, California	**Log of Boring B-2** PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 4/8/19 | Date finished: 4/8/19
Drilling method: 4" Solid-Stem Flight Auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: D. Landkamer
Drilled by: Benevent Building
Rig: Portable Hydraulic Unit

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1.75				CL	CLAY (CL) dark brown, stiff, moist, trace fine sand LL = 39, PI = 22; see Appendix B						
2	S&H	8 9 10	13							15.9	105
1.75–4.5				CL	SANDY CLAY (CL) olive-brown, stiff, mist fine sand						
4	S&H	6 9 9	13		Corrosivity Test; see Appendix B						
4.5–9				SC	CLAYEY SAND (SC) olive-brown, medium dense, moist, fine to medium sand						
6	SPT	6 7 8	18								
9–26.3				CL	SANDY CLAY (CL) olive-brown mottled gray, very stiff, moist, fine sand						
11	SPT	5 8 13	25								
13.8					▽ (4/8/2019; 9:44 AM)						
16	S&H	4 5 5	7		medium stiff, wet, increased sand content, trace gravel						
21	S&H	6 10 12	15		stiff, no gravel						
26	S&H	7 9 11	14		increased sand content						

LABORATORY TEST DATA

Boring terminated at a depth of 26.3 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 13.8 feet during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1627 | Figure: A-2

ROCKRIDGE 18-1627.GPJ TR.GDT 7/21/19

PROJECT: **101 SOUTH JACKSON AVENUE**
San Jose, California

Log of Boring B-3

Boring location: See Site Plan, Figure 2
Date started: 4/8/19 Date finished: 4/8/19
Drilling method: 4" Solid-Stem Flight Auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT)

Logged by: D. Landkamer
Drilled by: Benevent Building
Rig: Portable Hydraulic Unit

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1.75				CL	CLAY with SAND (CL) dark brown, stiff, moist, fine sand						
2	S&H	6, 9, 11	14								
1.75–4.5				CL	SANDY CLAY (CL) olive-brown, stiff, moist, fine sand						
4	S&H	5, 10, 12	15							21.5	107
4.5–11				SC	CLAYEY SAND (SC) olive-brown, medium dense, moist						
6	SPT	3, 6, 10	19								
11	SPT	5, 4, 6	12								
11–14.25				CL	CLAY (CL) olive-brown mottled gray, stiff, moist						
14.25–25.5				CL	SANDY CLAY (CL) olive-brown mottled gray, medium stiff, wet, fine to medium sand						
16	S&H	4, 4, 4	6								
20					very stiff, fine sand						
21	S&H	7, 12, 16	20								
25.5–26.5				ML	SANDY SILT (ML) olive-brown, stiff, wet						
26	S&H	6, 9, 10	13								

Boring terminated at a depth of 26.5 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.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1627 Figure: A-3

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

- Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
- Classification sample taken with Standard Penetration Test sampler
- Undisturbed sample taken with thin-walled tube
- Disturbed sample
- Sampling attempted with no recovery
- Core sample
- Analytical laboratory sample
- Sample taken with Direct Push sampler
- Sonic

SAMPLER TYPE

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

101 SOUTH JACKSON AVENUE
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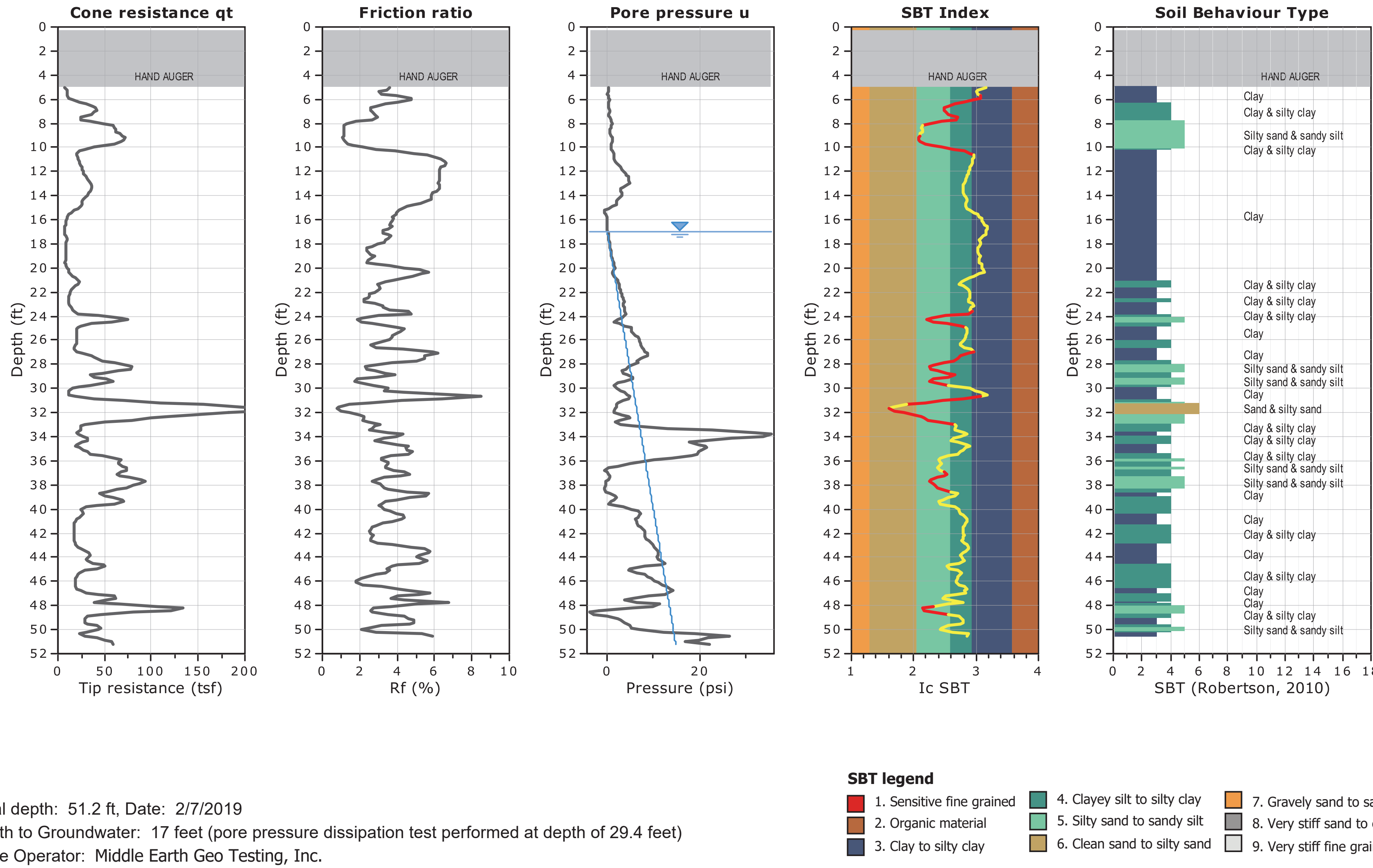
CLASSIFICATION CHART

Date 04/22/19 | Project No. 18-1627 | Figure A-4



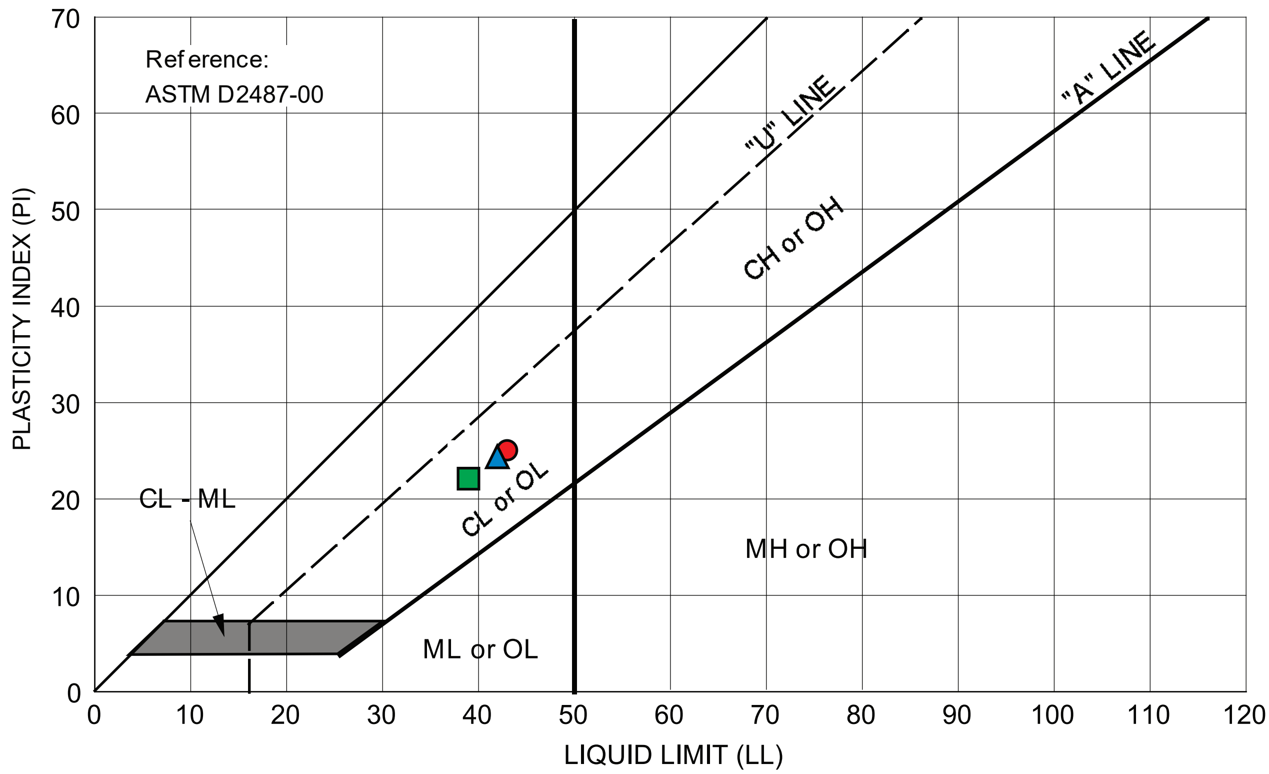
Total depth: 51.2 ft, Date: 2/7/2019
Depth to Groundwater: 17 feet (pore pressure dissipation test performed at depth of 29.4 feet)
Cone Operator: Middle Earth Geo Testing, Inc.

101 SOUTH JACKSON AVENUE San Jose, California	**CONE PENETRATION TEST RESULTS** **CPT-1**
ROCKRIDGE GEOTECHNICAL	Date 07/21/19 | Project No. 18-1627 | Figure A-5

ROCKRIDGE GEOTECHNICAL

APPENDIX B

Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 1.0 feet	CLAY (CL), brown to dark brown	17.1	43	25	--
▲	B-1 at 4.0 feet	CLAY with SAND (CL), olive-brown	18.9	42	24	--
■	B-2 at 1.5 feet	CLAY (CL), dark brown	15.9	39	22	--

101 SOUTH JACKSON AVENUE
San Jose, California

ROCKRIDGE GEOTECHNICAL

PLASTICITY CHART

Date 04/22/19 | Project No. 18-1627 | Figure B-1

	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**		**Sulfates**		**Chlorides**		**Nitrate**	**Ammonia**	**Sulfide**	**Redox**	**pH**
		As Rec'd	**Minimum**									
	(ft)	**(Ohm-cm)**	**(Ohm-cm)**	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mV)**	
B-2-3	3.5	1,005	938	30	0.0030	24	0.0024	18	8.5	0.84	79	9.38

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract

Prepared by,

Nathan Jacob
Lab Technician

Respectfully Submitted,

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ROCKRIDGE GEOTECHNICAL

CORROSION RESULTS

Date 04/22/19 | Project No. 18-1627 | Figure B-2

ROCKRIDGE GEOTECHNICAL

APPENDIX C

Summary of Liquefaction Analysis

Rockridge Geotechnical
270 Grand Ave
Oakland CA 94610
https://www.rockridgegeo.com/

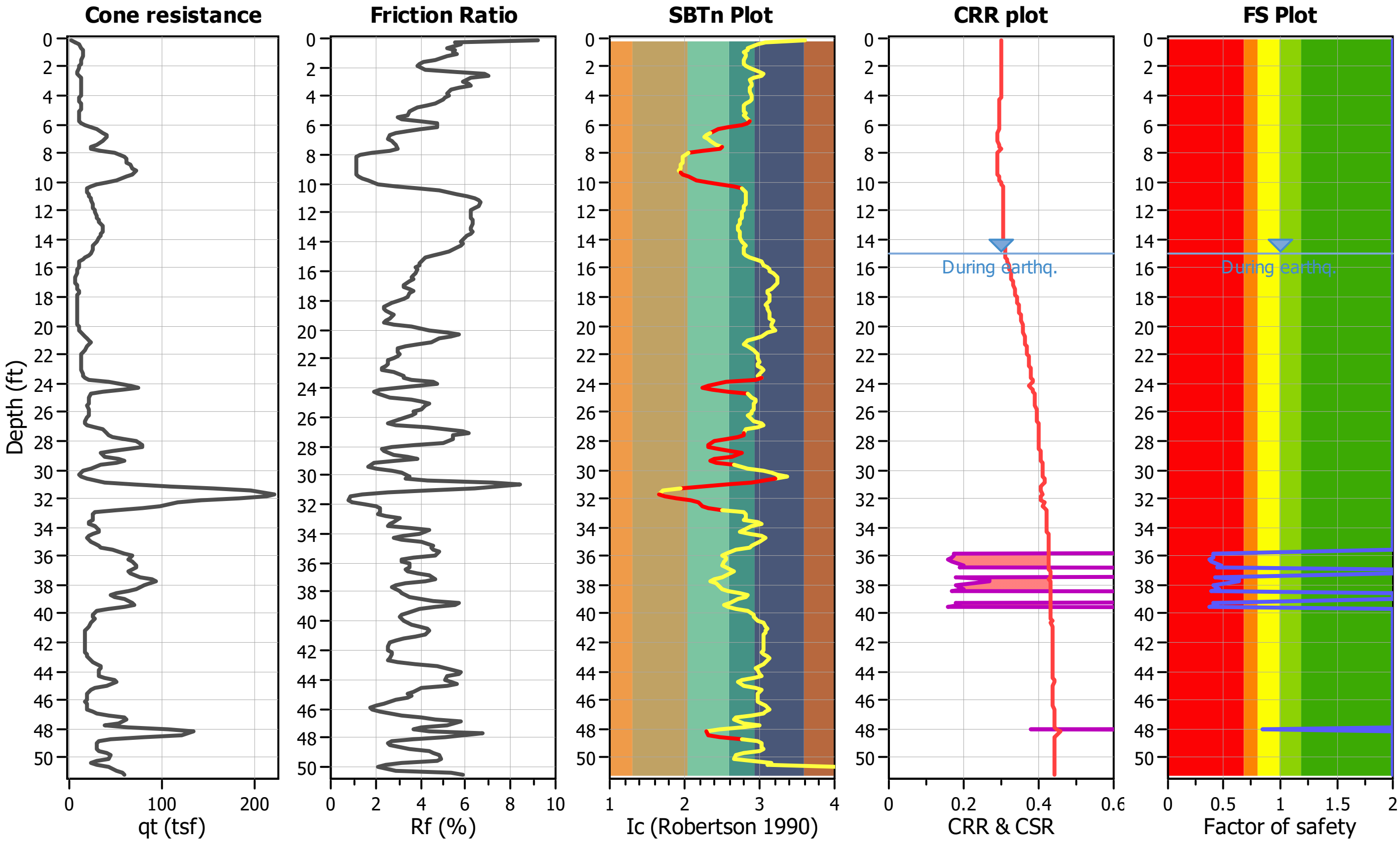
LIQUEFACTION ANALYSIS REPORT

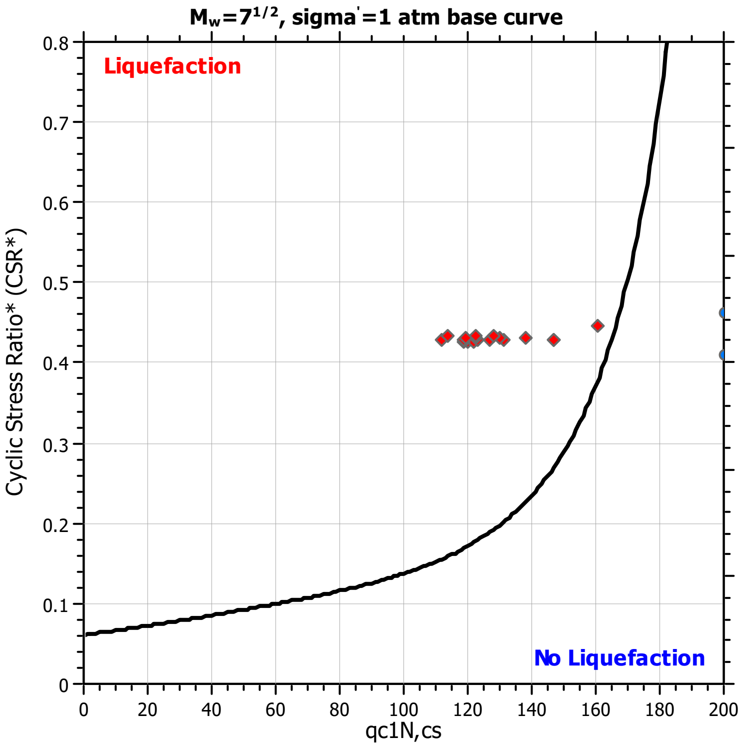
Project title : 101 S. Jackson Avenue, **Location : San Jose, California 95116**

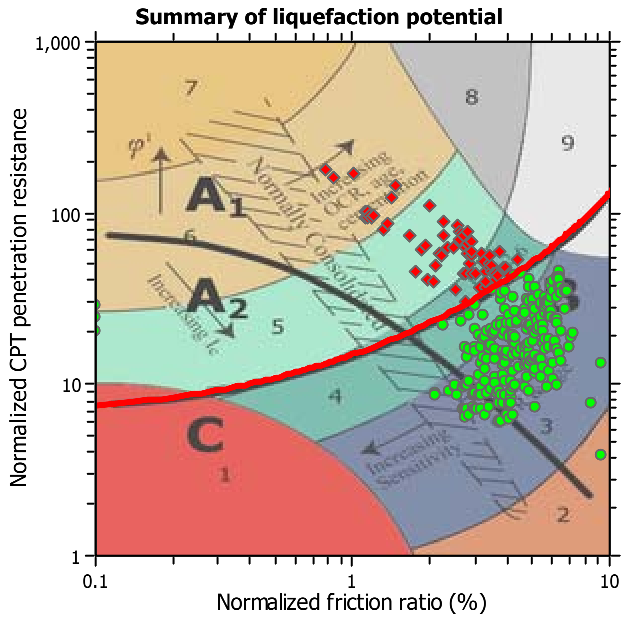
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	17.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	15.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.33	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

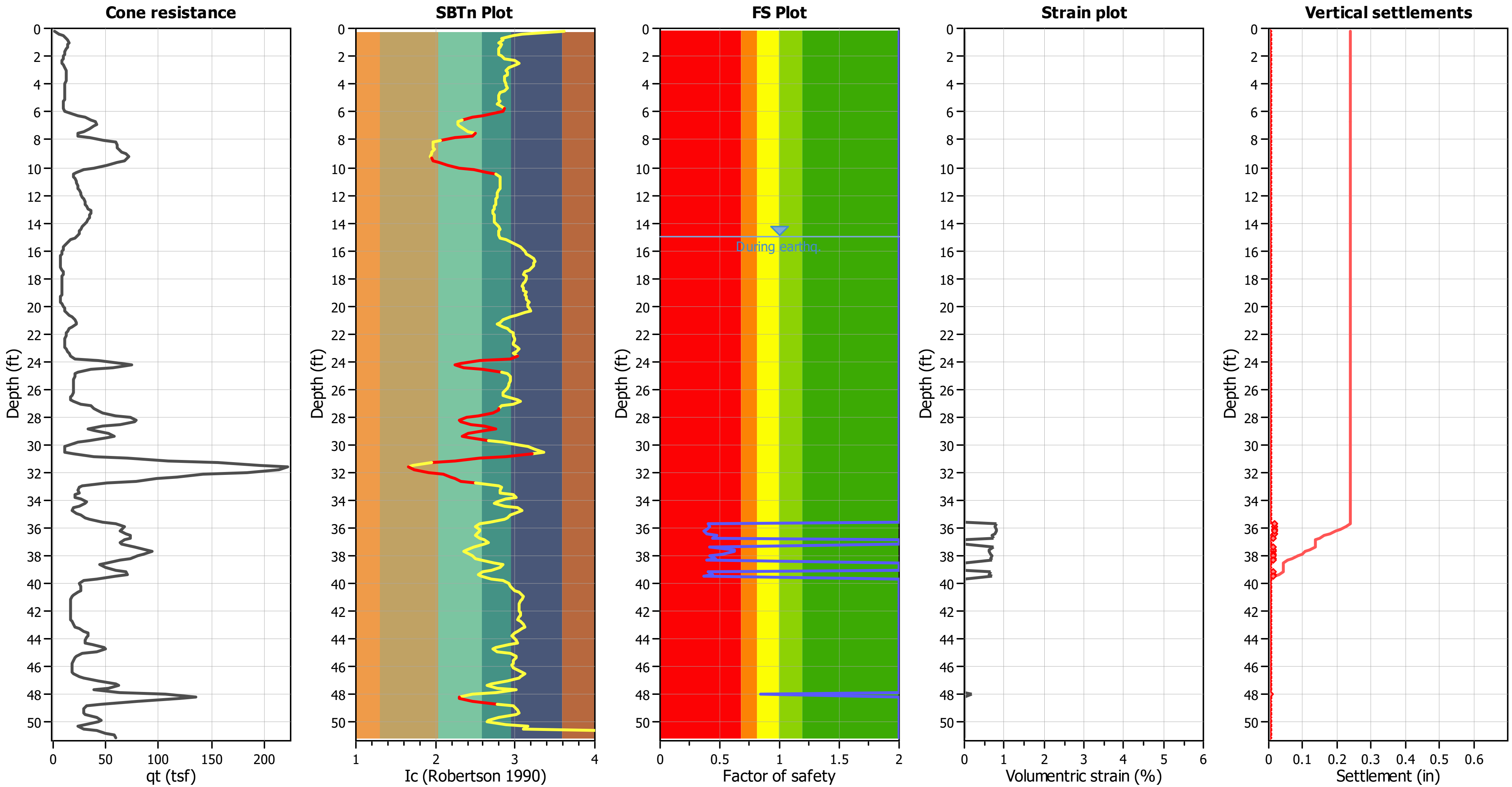






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain