APPENDIX D

Geotechnical Investigation Report

Little Portugal Gateway Mixed-Use Project
Initial Study/MND

ROCKRIDGE GEOTECHNICAL

Prepared for **Silicon Sage Builders**

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
ALUM ROCK AVENUE AND NORTH KING ROAD
1661, 1663, 1665 ALUM ROCK AVENUE
SAN JOSE, CALIFORNIA

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September 15, 2018
Project No. 18-1488

ROCKRIDGE GEOTECHNICAL

September 15, 2018
Project No. 18-1488

Ms. Shaivali Desai
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Silicon Sage Builders
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Subject: Geotechnical Investigation Report
Proposed Mixed-Use Development
1661, 1663, & 1665 Alum Rock Avenue
San Jose, California

Dear Ms. Desai:

We are pleased to present the results of our geotechnical investigation for the proposed mixed-use development to be constructed at 1661, 1663, & 1665 Alum Rock Avenue in San Jose, California. Our services were provided in accordance with our proposal dated April 12, 2018.

The site is located on the north side of Alum Rock Avenue on the block bounded by North 34th Street and North King Road. The project site consists of three adjacent parcels with a combined area of about 0.9 acre. The site is approximately rectangular-shaped and measures about 150 by 260 feet in plan. The project site is currently occupied by multiple single-story commercial buildings and parking areas. The site is bounded by Alum Rock Avenue to the southeast and multiple single-story commercial and residential properties on the remaining three sides. The ground surface at the site is relatively level.

Based on our discussions with the project team, we understand conceptual development plans are to construct a mixed-use building with of five levels of wood-framed residential units over a one-level concrete podium structure that will house commercial space. The building will have a single basement level to house parking, with some areas deepened to accommodate parking stackers.

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 associated with the development currently proposed for the site include: 1) shallow groundwater relative to the proposed building foundation and excavation depth, 2) providing suitable lateral support and dewatering for the proposed excavation while minimizing impacts to the surrounding improvements, and 3) providing adequate foundation support for the proposed building. These and other geotechnical issues for the proposed development are addressed in detail in the attached report.

Provided the estimated total and differential settlements presented in our report are acceptable, the proposed building may be supported on a stiffened mat foundation that is underlain by waterproofing and designed to resist hydrostatic uplift pressures. If the building weight is not sufficient to resist the hydrostatic uplift pressures imposed by the groundwater, tiedown anchors may be required to provide the mat foundation with additional uplift resistance. Feasible methods of temporary shoring during excavation include permeable soldier-pile-and-lagging system (with or without tiebacks) or a tied-back soil-cement mix (SMX) cut-off wall reinforced with steel soldier beams. The most appropriate method will depend on the final excavation depth, setback from adjacent property lines, and the tolerance for ground deformation behind the shoring (i.e. the presence of neighboring structures near the excavation).

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

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GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
ALUM ROCK AVENUE AND NORTH KING ROAD
1661, 1663, 1665 ALUM ROCK 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 1661, 1663, & 1665 Alum Rock Avenue in San Jose, California. The site is located on the north side of Alum Rock Avenue on the block bounded by North 34th Street and North King Road, as shown on the attached Site Location Map (Figure 1).

The project site consists of three adjacent parcels with a combined area of about 0.9 acre. The site is approximately rectangular-shaped and measures about 150 by 260 feet in plan. The project site is currently occupied by multiple single-story commercial buildings and parking areas, as shown on the Site Plan (Figure 2). The site is bounded by Alum Rock Avenue to the southeast and multiple single-story commercial and residential properties on the remaining three sides. The ground surface at the site is relatively level.

Based on our discussions with the project team, we understand conceptual development plans are to construct a mixed-use building with of five levels of wood-framed residential units over a one-level concrete podium structure that will house commercial space. The building will have a single basement level to house parking, with some areas deepened to accommodate parking stackers.

Structural design loads were not available at the time this report was prepared. However, based on our experience with similar structures, we estimate the proposed building will have typical dead-plus-live interior column loads on the order of about 600 kips.

2.0 SCOPE OF SERVICES

Our geotechnical investigation included exploring subsurface conditions at the site by drilling two test borings, advancing three cone penetration tests (CPTs), and performing geotechnical laboratory testing on select samples from our borings. Our investigation was performed in accordance with our proposal dated April 12, 2018. We used the data collected during our field investigation to perform engineering analyses to develop conclusions and recommendations regarding:

- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of static and seismically-induced foundation settlement
- lateral earth pressures (static and seismic) for design of the basement walls
- recommended design groundwater elevation
- subgrade preparation for pavements and exterior concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- flexible and rigid pavement design
- temporary slopes, shoring, and dewatering
- 2016 California Building Code (CBC) site class and design spectral response acceleration parameters
- soil corrosivity
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

We investigated the subsurface conditions beneath the site by drilling two test borings and advancing three CPTs. The approximate locations of the test borings and CPTs are shown on the Site Plan (Figure 2). Prior to mobilizing to the site, we contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained a private utility locator to check for underground utilities at each investigation location. We also obtained a drilling permit from Santa Clara Valley Water District (SCVWD) for the CPTs. Details of the field exploration are described in the remainder of this section.

3.1 Test Borings

Borings B-1 and B-2 were drilled on April 24, 2018 by Exploration Geoservices, Inc. of San Jose, California, using Mobile B-40 drill rig with equipped with hollow-stem augers. The borings were advanced to depths of about 45 feet below the existing ground surface (bgs). During drilling, our field engineer logged the soil encountered in the borings and collected representative samples of the soil encountered for further classification and laboratory testing. The boring logs are presented in Appendix A as Figures A-1 and A-2. The soil was classified in accordance with the classification system presented on Figure A-3.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter brass tubes.
- Shelby Tube (ST) thin-walled stainless steel tubes with a 2.875-inch inside diameter.

The S&H sampler was driven with a 140-pound downhole safety hammer falling about 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every 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 Standard Penetration Test (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 pushed into the soil under the weight of the drill rods and hydraulic pressure from the drill rig.

Following completion, the borings were backfilled with neat cement grout in accordance with SCVWD requirements and the pavement surface was patched. Soil cuttings from the borings were collect and stored in drums onsite. Analytical testing of the cuttings indicated they were non-hazardous and were subsequently transported to an appropriate disposal facility.

3.2 Cone Penetration Tests

Middle Earth GeoTesting, Inc. of Orange, California advanced three CPTs, designated as CPT-1 through CPT-3, on May 4, 2018. The CPTs were performed by hydraulically pushing a 1.7-inch-diameter cone-tipped probe into the ground to a depth of about 80 feet bgs. 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 presented in Appendix A on Figures A-4 through A-6. Upon completion, the CPT holes were backfilled with neat cement grout in accordance with SCVWD requirements and the pavement surface was patched.

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. Soil samples were tested to measure moisture content, dry density, Atterberg limits[1] (plasticity index), particle size distribution, consolidation properties, shear strength, and corrosivity. The laboratory test results are presented on the boring logs and in Appendix B on Figures B-1 through B-6.

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

4.0 SUBSURFACE CONDITION

The Regional Geologic Map (Figure 3) indicates the site is underlain by Holocene-age (less than 11,000 years before present) alluvium (Qha).

The borings and CPTs advanced as part of our investigation indicate the site is underlain by alluvium that consists of predominantly medium stiff to very stiff clay with variable amounts of sand and gravel to the maximum depth explored, 80 feet bgs. The clay is generally medium stiff to stiff to a depth of about 15 feet bgs, stiff to very stiff between a depth of 15 and 40 feet bgs, and very stiff below 40 feet bgs. The clay is interbedded with occasional thin (less than 2 feet thick) layers of sand with varying fines and gravel content. In CPT-1 and CPT-2, dense to very dense sand layers, approximately 10 feet thick, were encountered at depths of about 66 feet and 56 feet bgs, respectively. Atterberg Limits test performed on a shallow samples of the native clay indicate the near-surface clay materials at the site have low expansion potential.

4.1 Groundwater

Groundwater was encountered in our in borings B-1 and B-2 at depths of about 7 feet and 9 feet bgs, respectively. Similarly, pore pressure dissipation tests performed in the CPTs indicates an approximate groundwater depth ranging from approximately 7 to 9 feet bgs. Because these measurements were taken over a short period of time, the measured groundwater levels may not represent fully stabilized conditions. Furthermore, the depth of groundwater is expected to fluctuate a few feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall.

To better estimate the highest potential groundwater level at the site, we also reviewed information on the State of California Water Resources Control Board GeoTracker website (http://geotracker.waterboards.ca.gov/) for well readings of various locations in the immediate site vicinity (1639, 1665, and 1694 Alum Rock Avenue). Groundwater depths typically ranged from about 7 to 11 feet bgs, but groundwater was measured as shallow as 5.6 feet in December 1997.

Based on the groundwater levels observed during our investigation and to account for approximate seasonal groundwater fluctuations, we conclude a design high groundwater level of approximately 5 feet below the existing ground surface, should be used for design.

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[2], lateral spreading[3] and cyclic densification.[4] 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 located 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 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[5] [2007 Working Group on California Earthquake Probabilities (USGS 2008) and Cao et al. (2003)] are summarized in Table 1.

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

3 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.

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

5 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
Total Calaveras	9.9	Northeast	7.0
Total Hayward	11	North	7.0
Total Hayward-Rodgers Creek	11	North	7.3
Monte Vista-Shannon	15	South	6.5
N. San Andreas - Peninsula	23	Southwest	7.2
N. San Andreas (1906 event)	23	Southwest	8.0
N. San Andreas - Santa Cruz	24	Southwest	7.1
Zayante-Vergeles	31	Southwest	7.0
Greenville Connected	32	East	7.0
Mount Diablo Thrust	42	North	6.7
San Gregorio Connected	46	West	7.5

Since 1800, four major earthquakes have been recorded on the San Andreas fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas fault (Toppozada and Borchardt 1998). The estimated Moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to 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 35 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 (Field, 2015). 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 our 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. 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 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.

We evaluated the liquefaction potential of soil encountered below groundwater at the site using data collected in our CPTs with consideration of laboratory testing 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 5 feet bgs. In accordance with the 2016 California Building Code (CBC), we used a peak ground acceleration of 0.50 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 very thin discontinuous layers of medium dense sand, 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 discontinuous nature 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 alsovery 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 site is primarily underlain by medium stiff to very stiff fine-grained deposits and thin granular layers that are sufficiently dense and/or cohesive to resist cyclic densification. Therefore, we conclude the potential for cyclic densification to occur at the site is very low.

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 DISCUSSION AND CONCLUSIONS

From a geotechnical engineering standpoint, we conclude the site can be developed as planned, provided the recommendations presented in the report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical issues associated with the development currently proposed for the site include:

1) shallow groundwater relative to the proposed building foundation and excavation depth

2) providing suitable lateral support and dewatering for the proposed excavation while minimizing impacts to the surrounding improvements

3) providing adequate foundation support for the proposed building

These and other issues are discussed in the following sections.

6.1 Design Groundwater Level

Based on the available groundwater data discussed in Section 4.1, we recommend using a design high groundwater level of about 5 feet below existing grades (bgs) for the proposed project. To date, we have not been provided with a topographic survey of the site, and therefore, we have not yet evaluated the elevation of the recommended design high water level. As discussed in Section 1.0, we understand the proposed development will include one level of below-grade parking with deeper areas for parking stackers and elevator pits. Assuming a basement depth of 10 feet, stacker depth of 8 feet, and a mat foundation thickness of about 2 feet, we estimate the construction of the proposed building will require an excavation between 12 and 20 feet below existing grades. Therefore, the bottom-of-foundation may be about 7 to 15 feet below the design high groundwater level. As a result, the proposed building foundation and below grade walls will need to be designed to resist hydrostatic pressures and include waterproofing.

Considering the proposed excavation will extend below the groundwater, the excavation will need to be temporarily dewatered and the excavation shoring system will need to be designed for the effects of groundwater. A more detailed discussion regarding temporary excavation shoring and dewatering is presented in Section 6.3.1.

6.2 Foundations and Settlement

The soils encountered in our borings and CPTs at the site are generally moderately overconsolidated and capable of supporting low to moderate buildings loads without excessive static or seismic settlement.

Considering the proposed bottom-of-foundation will be between about 7 and 15 feet below the design high groundwater level, it will need to be designed to resist hydrostatic uplift pressures and be underlain by a waterproofing. Although the native soils beneath the site could potentially support the building loads on conventional spread footings, a stiffened mat foundation system generally simplifies construction dewatering (discussed below) and the detailing of the waterproofing system. In addition, the weight of a stiffened mat foundation will provide greater resistance to the relatively high hydrostatic uplift pressures. Therefore, we conclude the proposed building may be supported on a stiffened mat foundation designed to resist hydrostatic uplift pressures. If the new foundation does not have sufficient uplift capacity, soil anchors (i.e., tiedowns) can be installed to resist uplift forces.

Our settlement analyses indicate total settlement of the mat foundation under static load conditions, assuming an average contact pressure of about 800 psf, will be about two inches. 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 1/2 inch in 30 feet.

6.3 Excavation Support

We estimate construction of the below-grade structure will require an excavation bottomed as deep as about 20 feet below grade. There is likely insufficient property line setback to slope cut the excavation. Therefore, excavation shoring will likely be required.

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

- protection of surrounding improvements, including neighboring structures, underground utilities, pavements, and sidewalks
- the presence of relatively shallow groundwater and the desire to minimize lowering of the water table outside the limits of the excavation
- 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:

- steel sheet piles
- soldier pile-and-lagging
- secant pile wall
- soil-cement mixed (SMX) columns

Where the excavation is deeper than about 12 feet or where the wall will need to resist hydrostatic pressures (i.e. cutoff wall), cantilevered shoring systems are generally not cost-effective—therefore, any of the systems listed above would require tiebacks or internal bracing. Tieback anchors may extend beneath the neighboring properties, which will require an encroachment agreement with neighboring property owners and City of San Jose, if needed.

Where the neighboring building foundations are supported above an imaginary line that lies at an inclination of 1.5:1 (horizontal to vertical) projected upward from the bottom edge of the proposed excavation, the shoring should be designed using at-rest pressure, as well as the surcharge load from the neighboring building foundation, to limit the amount of horizontal movement at the top of the shoring.

A structural/civil engineer knowledgeable in this type of construction should be retained to design the shoring. The shoring designer should determine deflection tolerances and establish a monitoring program to measure the deformations of the shoring system as construction proceeds. In our experience, deflections of about 1/2 inch and 1 inch are typical tolerances for shoring near neighboring buildings (as defined above) and away from buildings, respectively. We should review the final shoring plans and calculations to check that they are consistent with the recommendations presented in this report. During construction, we should observe the installation of the shoring system and check the condition of the soil encountered during excavation.

Steel Sheetpiles

We considered tied-back steel sheetpiles, but in our opinion this system is cost-prohibitive. We can provide recommendations for sheetpile walls upon request.

Soldier Pile-and-Lagging

A conventional tied back soldier pile-and-lagging system would be feasible, however, because the system is pervious, an active dewatering system consisting of a series of extraction wells installed outside the excavation, would likely be required to prevent caving of the soil and excessive water from seeping through the lagging boards into the excavation. The influence of dewatering wells on adjacent improvements, including the potential for inducing settlement, should be fully addressed during the design of the shoring system.

Continuous Soil-Cement Mix (SMX) Wall

Soil-cement mixing (SMX), also called deep soil mixing (DSM), is a viable option for creating a continuous shoring wall that supports the excavation, as well as provides a hydraulic barrier when properly constructed. SMX columns are installed by injecting and blending cement into the soil using a drill rig equipped with single or multiple augers/paddles, or a specialized proprietary cutterhead. The soil is mixed with the binder material(s) in situ, forming continuous, overlapping, soil-cement columns or a continuous wall of uniform thickness. Steel beams are placed in the soil-cement columns to provide rigidity. The SMX system, in combination with steel soldier beams and tiebacks, serves to shore the excavation as well as cut off lateral groundwater flow, thus reducing the amount of dewatering required from within the excavation. Soil-cement walls are considered temporary and permanent building walls are built inside of the soil-cement walls following application of drainage panels and waterproofing.

SMX systems are generally installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of the SMX columns, beams, and tieback elements should be determined by the shoring designer, based on the design soil, water, and surcharge pressures presented in Section 7.4 of this report. However, there are numerous factors that influence the quality, consistency, strength, and permeability of the resulting soil-cement mix, which are controlled by the materials, methods, and equipment employed by the contractor performing the soil mixing. These factors include, but are not limited to:

- Types of binder material(s) used – i.e. cement, bentonite, etc.; wet-mixed vs. dry-mixed
- Quantities and proportions of binder material(s) used – i.e. water-to-binder ratio; volume ratio of SMX
- Equipment used to perform the mixing – i.e. single-auger, multi-auger, or cutter-based equipment
- Plumbness and amount of overlap between adjacent SMX columns
- Homogeneity of soil-cement mixture – controlled by rate of mixing, number of stages, and equipment used
- Depth and diameter of predrilling, which may be required within hard clay or dense sand layers, depending on equipment selected

A contractor experienced in installing SMX systems in similar soil conditions and below the groundwater table should be responsible for selecting appropriate materials, equipment, and methods based on the soil and groundwater conditions at this site, as well as their expertise, in order to meet the performance criteria established by the shoring designer. The design and construction of a SMX system should also consider the capacity of the dewatering system selected by the contractor.

Secant Pile Wall

A secant pile wall is similar to a conventional soldier-pile-and-lagging system, in which steel H-beams and lean concrete are placed in predrilled holes extending below the bottom of the excavation. However, instead of installing wood lagging to support the soil between the reinforced soldier piles, additional shafts are drilled and filled with lean concrete between the soldier piles in an overlapping fashion, such that a continuous wall of lean concrete is created. The final product is similar to the SMX wall described above, except the material is typically stronger than SMX and a greater amount of drill spoils requiring off-haul are generated.

6.3.1 Excavation Dewatering

Due to the low permeability of most of the soil underlying the site, an active dewatering system, such as a series of dewatering wells installed outside the perimeter of the excavation, may have limited effectiveness in drawing down the water level in the center of the excavation. Furthermore, a perimeter dewatering system may temporarily lower the groundwater level outside the site, such as beneath neighboring properties and City streets/sidewalks. Therefore, we conclude the excavation dewatering employed during construction of the proposed building should consist of an internal system operating within the excavation footprint. A combination of active and passive approaches will likely be required to adequately manage water in the excavation during construction. The design and proper implementation of the excavation dewatering system should be the responsibility of the contractor. The system should be capable of drawing the water level down about three feet below the bottom of excavation during construction. To facilitate the collection of groundwater at discrete extraction well and sump locations, we recommend over-excavating by at least 12 inches below the design bottom-of-mat and installing a minimum 12-inch-thick continuous layer of clean 3/4-inch drain rock. The drainage layer will help protect the soil subgrade, which will be sensitive to disturbance from construction equipment, as well as provide a means for water to flow to the extraction points, reducing the potential for hydrostatic pressure to prematurely build up beneath the mat.

The construction dewatering system must be capable of maintaining the groundwater level below the foundation subgrade until sufficient building weight is available to resist the hydrostatic uplift pressure, at which time the groundwater may be allowed to rise to its normal elevation. The project structural engineer should determine when the temporary dewatering system can be turned off.

6.4 Construction Considerations

The soil to be excavated for the new foundations and underground utilities is expected to be predominantly clay, which can be excavated with conventional earth-moving equipment such as conventional excavators, loaders, and backhoes. If site grading is performed during the rainy season, repeated loads by heavy equipment will reduce the strength of the surficial soil and decrease its ability to resist deformation; this phenomenon could result in severe rutting and pumping of the exposed subgrade. To reduce the potential for this behavior, heavy rubber-tired equipment as well as vibratory rollers, should be avoided within two feet of the foundation subgrade.

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 and shoring.

6.5 Soil Corrosivity

Corrosivity analyses were performed by Project X Corrosion on a composite sample of native clay from Boring B-1 at a depth of 3 feet. The corrosivity test results are presented in more detail in Appendix B of this report.

The results of the corrosivity analyses indicate the sample is “moderately corrosive” with respect to resistivity. 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 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 preparation, excavation, fill placement, excavation shoring and dewatering, foundations, basement wall design, rigid pavement design, and seismic design are presented in the following sections of the report.

7.1 Site Preparation, Excavation, and Fill Placement

Site clearing should include removal of all existing pavements, former foundation elements, and underground utilities. 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 subgrade should be removed. Excessively dry soil at tree removal locations, as determined in the field by the geotechnical engineer, should also be excavated and replaced. 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.

During excavation for the below-grade level, the excavation will extend below groundwater. The foundation excavation subgrade will consist of saturated clay and will be sensitive to disturbance, especially under construction equipment wheel loads. To provide a working surface on which to install the waterproofing system, and to facilitate dewatering, the native soil should be overexcavated to provide room for a minimum 12-inch-thick continuous layer of crushed rock. To minimize disturbance of the native clay subgrade, the last two feet of soil should be excavated with a track-mounted excavator with a smooth bucket or bar welded across the teeth. Even with tracked equipment, the exposed subgrade may be sensitive, especially if the excavation is not adequately dewatered. We do not recommend operating any trucks or rubber-tired equipment on the exposed mat subgrade. Any disturbed soil at or below subgrade level (i.e., bottom of overexcavation) should be removed by hand. Following approval by our engineer, the bottom of the overexcavation should be covered with a layer of woven geotextile tensile fabric (Mirafi 500X or equivalent). The geotextile should be covered with at least 12 inches of clean 3/4-inch crushed rock to provide a firm working surface. The crushed rock should meet the gradation requirements presented below in Table 2.

TABLE 2
Gradation Requirements for Gravel Blanket Beneath Mat

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

If any engineered fill will be placed above the crushed rock, it should then be covered with a nonwoven filter fabric (Mirafi 140NC or equivalent) prior to placement of engineered fill. A mud slab is generally required beneath most waterproofing products. If no engineered fill is to be placed above the crushed rock blanket, the mud slab may be placed directly over the rock (no filter fabric required).

For planning purposes, a maximum temporary cut slope inclination of 1:1 (horizontal to vertical) may be assumed for the native clay soil above the groundwater, which corresponds to OSHA Type B soil. If granular material or seepage is observed in the cut slope during construction, the material should be downgraded to OSHA Type C soil and a corresponding maximum inclination of 1.5:1 should be used. All soil below the design water table should be assumed to be Type C soil.

The results of our field investigation indicate the near-surface clay has a low expansion potential. However, if areas of expansive soil is encountered during preparation of soil subgrade beneath various surface improvements, such as pavements and concrete flatwork (both on-site and within the City right-of-way), these materials may require moisture-conditioning to limit its expansion potential. Where required, as determined by our field engineer, expansive clay subgrade soil should be scarified to a depth of at least 12 inches, moisture-conditioned, and compacted to the specified percent relative compaction,[6] as presented below in Table 3. All fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned, and compacted in accordance with the requirements provided below in Table 3.

TABLE 3
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
General fill – lime-treated clay	90+	Above optimum
General fill – native moderate plasticity clay	90+	2+% above optimum
Utility trench backfill – native moderate plasticity clay	90+	2+% above optimum
Utility trench backfill – low-plasticity	90+	Above optimum
Utility trench - clean sand or gravel	95+	Near optimum
Pavement subgrade – native moderate plasticity clay	92+	2+% above optimum
Pavement subgrade – low-plasticity	95+	Above optimum
Pavement - aggregate base	95+	Near optimum
Exterior slabs – native moderate plasticity clay	90+	2+% above optimum
Exterior slabs – low-plasticity	90+	Above optimum
Exterior slabs – select fill	90+	Above optimum

Where the above recommended compaction requirements are in conflict with the City of San Jose standard details for pavements, sidewalks, or trenches within the public right-of-way, the

6 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.

City Engineer or inspector should determine which compaction requirements should take precedence.

7.1.1 Soil Subgrade Stabilization

In some areas, soft, wet soil may be exposed during grading, causing the subgrade to deflect and rut under the weight of grading equipment. Although, the majority of the soil beneath the site consists of stiff clay, if heavy wheeled equipment is used close to the water table, or if grading is performed during the wet season, these materials may become disturbed and soften. In these areas, some form of subgrade stabilization may be required if disturbance occurs. Several options for stabilizing subgrade 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. Aeration will likely not be effective where the subgrade extends below or near the groundwater table; however, it depends on the time of year construction is performed and the effectiveness of the dewatering system.

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.

Where very soft subgrade conditions are encountered, a bi-directional geogrid, such as Tensar TriAx TX-140 or equivalent, may be required in lieu of tensile fabric. Where geogrids are used the depth of overexcavation will likely be on the order of 12 to 18 inches. The geogrids should be overlapped by at least two feet and tied with hog rings or nylon ties at a spacing not to exceed 10 feet. The geogrids should be covered with a well-graded granular fill such as Class 2 aggregate base; open-graded rock should not be used. All backfill placed over the geogrid should be compacted in accordance with our previous recommendations.

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 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 RC, and at least 95 percent RC in the upper six inches of pavement subgrade.

7.1.2 Select Fill

Select fill should consist of 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. Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction beneath concrete flatwork and sidewalks. Beneath vehicular pavements, or in areas where the fill thickness is greater than five feet, the select fill should be compacted to at least 95 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.

7.1.3 Exterior Flatwork Subgrade Preparation

We recommend a minimum of four inches of imported (select) material be placed beneath proposed exterior concrete flatwork, including patio slabs and sidewalks; the select fill should extend at least one foot beyond the slab edges. Select fill beneath exterior slabs-on-grade, such as patios and sidewalks, should be moisture-conditioned and compacted in accordance with the requirements provided above in Table 3.

7.1.4 Utility Trench Backfill

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

7.1.5 Drainage and Landscaping

Positive surface drainage should be provided around the buildings to direct surface water away from foundations and below-grade walls. 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 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. The use of water-intensive landscaping around the perimeter of the at-grade building should be avoided to reduce the amount of water introduced to the clay subgrade.

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

Storm water treatment systems (infiltration basins, rain gardens, bio-retention systems, vegetated swales, flow-through planters, etc.), if constructed at the site, should be provided with underdrains, as well as impermeable liners. Due to the low permeability of the near-surface soil, these systems should not be designed for exfiltration in to the subgrade soil. The drainage layer beneath the “treatment” soil should consist of a minimum 12-inch-thick layer of Caltrans Class 2 Permeable drainage material and include a minimum 6-inch-diameter perforated drain pipe with a filter sock. If the perforated pipe is placed with the perforations facing downward, the filter sock may be omitted. An impermeable liner consisting of a high density polyethylene membrane (or equivalent) that is at least 10 mils thick should line the entire bottom and sides of the system.

7.2 Foundations

Provided the estimated total and differential settlements presented in Section 6.2 are acceptable, the proposed building may be supported on a stiffened mat foundation that is underlain by waterproofing and designed to resist hydrostatic uplift pressures. If the building weight is not sufficient to resist the hydrostatic uplift pressures imposed by the groundwater, tiedown anchors may be required to provide the mat foundation with additional uplift resistance. The following sections present our recommendations for the design and construction of a mat foundation and tiedown anchors.

7.2.1 Mat Foundation

The mat foundation should be constructed on the required minimum 12-inch-thick layer of clean crushed rock. The purpose of the rock layer is to protect the soil subgrade and facilitate dewatering during construction. One or more mudslabs may be required between the crushed rock layer and the bottom of mat foundation depending on the waterproofing system requirements and construction methods selected for the project—this should be evaluated and specified by the waterproofing consultant and product manufacturer. The native soil subgrade beneath the rock layer should be firm and undisturbed, as described in Section 7.1. The top of the mat foundation may be used as the lowest basement floor or a thin layer of concrete (topping slab) may be placed above the mat to provide a smooth wearing surface.

For structural design of the mat foundation we recommend using an initial coefficient of vertical subgrade reaction of 10 pounds per cubic inch (pci) under DL+LL conditions. This value has been reduced to account for the size of the mat/equivalent footings (therefore, this is <u>not</u> k_{v1} for 1-foot-square plate). 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), which include factors of safety of at least 2.0 and 1.5, respectively.

Lateral forces can be resisted by friction along the base of the mat and 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 and an equivalent fluid weight (triangular distribution) of 150 pcf for sustained loads. The allowable friction factor will depend on the type of waterproofing used at the base of the mat. For bentonite-based water proofing membranes, such as Paraseal or Voltex, a friction factor of 0.12 should be used (assumes a bentonite friction angle of 10 degrees). If Preprufe is used, a base friction factor of 0.20 should be used. Friction factors for other types of waterproofing membranes can be provided upon request. 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 mat subgrade will be sensitive to disturbance due to its proximity to the groundwater table. The final two feet of excavation and fine grading of the mat subgrade should be performed with tracked equipment to minimize heavy concentrated loads that may disturb the wet soil. Rubber-tired equipment and dump trucks should not be operated on the final mat subgrade. The subgrade should be free of standing water, debris, and disturbed materials and be approved by the geotechnical engineer prior to placing the gravel drainage layer. The mat subgrade should be kept moist following excavation and maintained in a moist condition until drain rock is placed. If the foundation soil dries during construction, it will eventually heave, which may result in cracking and distress.

Considering the internal excavation dewatering system will need to be capable of continuously maintaining the water level below the bottom of the mat until the building has sufficient weight to resist hydrostatic uplift pressures associated with the design water level, the mat will need to be constructed with temporary block-outs to accommodate the extraction wells or sump pits used to extract the water from the drainage layer. Once it has been determined by the structural engineer that the dewatering system may be shutoff, the pumps will need to be removed and the block-outs promptly waterproofed and plugged. The detailing of the waterproofing and plugging system at these locations will be critical and should be evaluated by a waterproofing consultant and structural engineer experienced with such operations.

7.2.2 Tiedown Anchors

Tiedown anchors may be used in conjunction with the mat foundation at the site, if needed to resist the design hydrostatic uplift forces. Tiedowns are installed by advancing a small-diameter hole (typically between 5 to 8 inches in diameter) using either hollow-stem augers or air-track equipment that advances smooth steel drill casing (e.g., a Klemm rig). A large-diameter reinforcing bar or high-capacity steel strands are inserted into the hole, and then grout is injected into the hole under pressure as the auger or steel drill casing is withdrawn. Post-grouting can be performed to achieve higher capacities.

We recommend tiedowns be spaced at least four shaft diameters or three feet apart, center-to-center, whichever is greater. Tiedowns for this project will gain support through skin friction in primarily stiff to very stiff clay. Tiedown capacity depends significantly on installation procedures, and installation procedures vary. Assuming the tiedowns are installed with a Klemm rig and post-grouted, we recommend using allowable skin friction values of 1,000 psf. We estimate the allowable skin friction value includes a factor of safety of at least 2.0. If the contractor installing the tiedowns believes they can achieve a higher capacity than that assumed above, higher capacities may be used, provided the factor of safety is verified through a load testing program, as detailed below. We recommend using minimum free lengths of 5 and 10 feet for bars and strands, respectively, and a minimum bond length of 20 feet for both bars and strands. The skin friction values used in design should be verified by a testing program. Because the tiedowns will be permanent, they should have double corrosion protection.

The required tiedown bond length should be confirmed by a proof-test program conducted under our observation. We recommend proof-testing a minimum of two tiedowns in tension to 200 percent of the design load (DL) at the start of production installation. The two anchors tested to 200 percent DL may require larger bar diameter or additional strands, so that their structural capacity is not exceeded during testing. The remaining production anchors should be proof-tested to 150 percent DL. During testing, the deflection of each tiedown should be monitored with a free-standing, tripod-mounted dial gauge accurate to at least 0.001 inches. We recommend deflection of the tiedowns be measured at load increments equal to about 25 percent of the design load. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is more than 0.04 inches, the load should be held for an additional 50 minutes, with additional readings taken at 15, 20, 30, 45 and 60 minutes. If the deflection is more than 0.08 inches between the 10- and 60-minute readings or the tiedown design load should be re-evaluated. Replacement tiedowns should be provided, as directed by the structural engineer, for tiedowns that fail the tests. Tiedowns should be locked off at a load to limit movement during stabilizing of the groundwater level to less than 1/2 inch (structural engineer should confirm).

7.3 Permanent Below-Grade Walls

Below-grade walls should be designed to resist static lateral earth pressures, hydrostatic pressures, lateral pressures caused by earthquakes, vehicular surcharge pressures, and surcharges from adjacent foundations, where appropriate. We recommend below-grade walls at the site be designed for the more critical of the following criteria:

- At-rest equivalent fluid weight of 63 pcf above the design groundwater table and 95 pcf below.
- Active pressure of 42 pcf plus a seismic increment of 21 pcf (triangular distribution) above the design groundwater level, and 83 pcf plus a seismic increment of 10 pcf (triangular distribution) below the groundwater level.

The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads. Where the below-grade wall is subject to traffic loading within 10 feet of the wall, an additional uniform lateral pressure of 100 psf, applied to the upper 10 feet of the wall, should be used.

To protect against moisture migration, below-grade walls should be waterproofed and water stops should be placed at all construction joints. The design pressures recommended for above the design water level (approximately 5 feet below existing grades) are based on fully drained walls. Although part of the basement walls will be 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 a basement wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the design high groundwater level (or higher if allowed by the structural engineer). The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi NC 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.

If backfill is required behind basement walls prior to constructing the podium slabs, the walls should be braced (as determined by the structural engineer), to prevent excessive earth pressures and potential wall deformation.

7.4 Excavation Shoring

As discussed in Section 6.3, we conclude the most appropriate shoring system for the proposed excavation at the site consists of either a permeable soldier-pile-and-lagging system (with or without tiebacks) or a tied-back soil-cement mix (SMX) cut-off wall reinforced with steel soldier beams. The most appropriate shoring system will depend on many factors, including:

- the final excavation depth
- effectiveness of the temporary dewatering system and associated costs
- the ability to obtain encroachment permits for tiebacks beneath surrounding properties

The safety of workers and equipment in or near the excavation is 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 the intent of our geotechnical recommendations. During construction, we should observe the installation and load testing of the shoring system and check the condition of the soil encountered during excavation.

Recommendations regarding the design and construction of the shoring, as well as design, construction and load testing of tieback anchors, are presented in the following sections. Where it is not feasible to install tiebacks, then internal bracing of the excavation may be required. The shoring designer should determine if internal bracing should be preloaded to limit movement of the shoring.

7.4.1 Design Lateral Earth and Water Pressures

The recommended water and earth pressure distributions presented in Figure 6 through 8 have been developed to account for the variations in pressures resulting from varying groundwater conditions (cut-off wall versus active dewatering system), cantilevered versus tied-back walls, and tolerance for shoring movement (at-rest versus active conditions).

At-rest earth pressures should be used in critical areas where deformations need to be minimized. Where traffic loads are expected within 10 feet of the shoring walls, an additional design load of 100 psf should be applied to the upper 10 feet of the wall. The pressure diagrams do not account for loading from adjacent buildings, sloped-backfill, or heavy (construction) equipment. Additional loads caused by these or other loadings should be evaluated on a case-by-case basis and included in design.

7.4.2 Soldier-Pile-and-Lagging System

Soldier piles should be placed in pre-drilled holes backfilled with concrete. Because soldier piles will extend below the water table, the shoring contractor should be prepared to use casing or drilling slurry to reduce caving of holes, where necessary. If more than 6 inches of water is present at the bottom of the drilled holes, concrete should be placed from the bottom-up via tremie.

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 500 psf to compute the required soldier pile embedment. End bearing should be neglected.

7.4.3 Soil-Cement Mix (SMX) Shoring

The design strength and thickness of the SMX wall should be established by the shoring designer based on the recommended design pressures presented in the previous section and the design requirements of the structural system. A contractor experienced in installing SMX systems in similar soil and groundwater conditions should be responsible for selecting appropriate materials, equipment, and methods to provide a consistent SMX product that meets the design requirements set forth by the shoring designer.

Prior to the start of SMX production, the contractor should prepare a detailed work plan, including the following items:

- Detailed descriptions of sequence of construction and all construction procedures, equipment, and ancillary equipment to be used to penetrate the ground, proportion and mix binders, and inject and mix the site soils.
- Proposed mix design(s), including binder types, additives, fillers, reagents, and their relative proportions, and the required mixing time, water-to-binder ratio of the slurry (for wet mixing), binder factor (for dry mixing and wet mixing), and volume ratio (for wet mixing) for a deep mixed element.
- Proposed injection and mixing parameters, including mixing slurry rates, slurry pumping rates, air injection pressure, volume flow rates, mixing tool rotational speeds, and penetration and withdrawal rates.
- Methods for controlling and recording the verticality and the top and bottom elevation of each SMX element.
- Drawings indicating the identification number of every SMX element, as well as a schedule of all the SMX elements and their tip elevations, mix design (if variable), element type (primary or secondary), binder factors, volume ratios, etc.
- Details of all means and methods proposed for QC/QA activities, including surveying, process monitoring, sampling, testing, and documenting.

The work plan should be submitted to the shoring designer and the geotechnical engineer of record for review prior to the start of construction and the approved document should be provided to the contractors' field personnel and our field engineer.

Detailed specifications for minimum required SMX strength for the various stages of excavation should be established by the shoring designer and followed by the shoring contractor during construction. The construction schedule should allow time for adequate curing and strength gain of the SMX material prior to proceeding with successive lifts of excavation. A clear quality control program should be established and implemented to confirm the design strengths have been achieved.

7.4.4 Tiebacks

Temporary tiebacks may be used to restrain the shoring. Where tiebacks are not feasible, internal bracing would likely be required. The vertical load from the temporary tiebacks should be accounted for in the design. The recommended tieback design criteria are presented on Figures 7 and 8 and in the following paragraphs.

Tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation and sloping upwards at 60 degrees from the horizontal, where H is the wall height in feet. Tiebacks should have a minimum unbonded length of 15 feet. All tiebacks should have a minimum bonded length of 15 feet and spaced at least four feet on center. During construction, the bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Tieback allowable capacity will depend upon the drilling method, hole diameter, grout pressure, and workmanship. For estimating purposes, we recommend using the skin friction value presented on Figures 7 and 8, assuming the tiebacks are post-grouted at least once. Higher skin friction values may be used if confirmed with pre-production load testing.

The contractor should be responsible for determining the actual length of tiebacks required to resist the design lateral earth and water pressures imposed on the temporary retaining systems. Determination of the tieback length should be based on the contractor's familiarity with his installation method and experience in similar soil conditions. The computed bond length should be confirmed by a proof-testing program under the observation of an engineer experienced in this type of work. Replacement tiebacks should be installed for tiebacks that fail the load test. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer.

Tieback Testing

We should observe all tieback testing. A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. All production tiebacks should be confirmed by proof tests to at least 1.25 times the design load. The bar or strands selected for the system must be capable of safely holding the maximum test load such that their structural capacity is not exceeded.

The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during proof testing. During the test, the tieback load and axial deflection are measured at each loading increment. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings 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.

We should evaluate the tieback test results and determine whether the tiebacks are acceptable. A proof-tested tieback with a ten-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04 inch movement between one and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. A 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 six 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 fail to meet the first criterion will be assigned a reduced capacity. If the total movement of the tiebacks at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the unbonded length, the contractor should replace the tiebacks at no additional cost to the owner.

7.4.5 Construction Monitoring

Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. During excavation, the shoring system is expected to yield and 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. Deformation tolerances should be determined by the shoring designer, and a monitoring program should be established to evaluate the effects of the construction on the adjacent properties. In our experience, deformations of approximately one inch where not adjacent buildings are present and one-half inch adjacent to existing buildings, are typically acceptable.

The conditions of existing buildings within 40 feet of the proposed excavation should be photographed and surveyed prior to the start of construction and monitored periodically during construction. In addition, prior to the start of excavation, the contractor should establish survey points on the shoring system, on the ground surface at critical locations behind the shoring, and on adjacent buildings. These survey points should be used to monitor the vertical and horizontal movements of the shoring and the ground behind the shoring throughout construction.

The survey points should be monitored regularly, and the results should be submitted to us in a timely manner for review. For estimating purposes, assume that the instrumentation will be read as follows:

- Prior to any excavation or shoring work at the site
- After installing soldier piles / SMX columns
- After excavation of each lift
- After a row of tiebacks is locked-off
- After the excavation reaches its lowest elevation
- Every two weeks until the street-level floor slab is constructed

7.5 Pavement Design

Design recommendations for asphalt concrete 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. For pavement design, we assumed a resistance value (R-value) of 5, which is appropriate for moderate plasticity clays. Recommended pavement sections for traffic indices ranging from 4.5 to 7.0 are presented in Table 4.

TABLE 4
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
7.0	4.0	15.5

The upper six inches of the subgrade should be moisture-conditioned and compacted in accordance with requirements presented in Table 3 in Section 7.1. The aggregate base should be moisture-conditioned to near optimum and compacted to at least 95 percent relative compaction.

Where 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. Where pavements are adjacent to storm water treatment facilities, such as bio-swales, flow-through planters, or bio-retention basins, or any other landscaped areas in which a significant thickness of loose, uncompacted soil will be present, the curbs may need to extend deeper, as outlined in Section 7.1.5

7.5.2 Concrete Pavement Design

For the parking garage ramp and driveway, which will experience only passenger car and light truck traffic, we recommend the concrete slab be at least five inches thick over six inches of Class 2 aggregate base (AB). Concrete pavement around the buildings, if any, may be subject to traffic from heavier vehicles, such as garbage trucks. Assuming a maximum single-axle load of 20,000 pounds and a maximum tandem axle of 32,000 pounds, the recommended rigid pavement section for these axle loads is 6-1/2 inches of Portland cement concrete over six inches of Class 2 aggregate base compacted to at least 95 percent relative compaction. Prior to placement of the aggregate base, we should confirm by proof rolling that the native soil subgrade is firm and non-yielding. If the subgrade deflects excessively during proof rolling, it should be scarified, aerated, and recompacted as discussed in Section 7.1 of this report.

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 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. Concrete slabs subject to vehicular traffic should be reinforced with a minimum of No. 4 bars spaced at 16 inches in both directions.

7.6 Seismic Design

For design in accordance with the 2016 California Building Code (CBC), we recommend Site Class D be used. The latitude and longitude of the site are 37.3522° and -121.8568°, respectively. Hence, in accordance with the 2016 CBC, we recommend the following:

- S_S = 1.50g, S_1 = 0.60g
- S_{MS} = 1.50g, S_{M1} = 0.90g
- S_{DS} = 1.00g, S_{D1} = 0.60g
- Seismic Design Category D for Risk Categories I, II, and III.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, Rockridge Geotechnical 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, excavation, grading, fill placement and compaction, shoring installation and load testing, and foundation installation. These observations will allow us to compare actual with anticipated soil conditions and to check that the contractor's work conforms with the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

This geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report are based on the assumption that the soil and groundwater conditions do not deviate appreciably from those disclosed in the exploratory 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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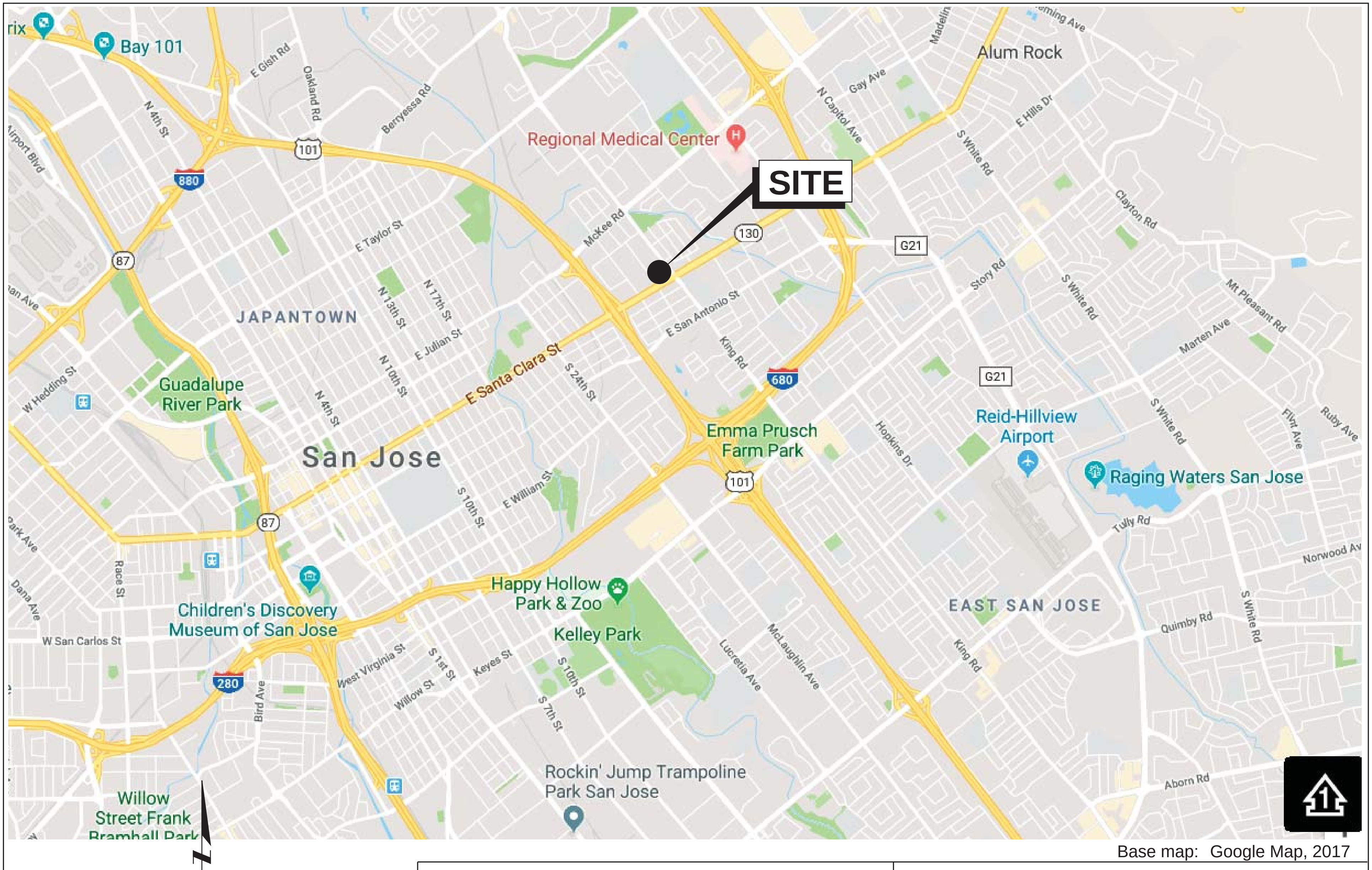
ROCKRIDGE GEOTECHNICAL

FIGURES



Base map: Google Map, 2017

0 4,000 8,000 Feet

Approximate scale

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 06/12/18	Project No. 18-1488	Figure 1


CPT-1
B-1
CPT-2
CPT-3
B-2
N 34TH STREET
ALUM ROCK AVENUE
EXPLANATION
CPT-1 Approximate location of cone penetration test by Rockridge Geotechnical Inc., May 4, 2018
B-1 Approximate location of boring by Rockridge Geotechnical Inc., April 24, 2018
Project limits
0
40 Feet
Approximate scale
Base map: Google Earth, 2018.
ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California
SITE PLAN
Date 06/19/18
Project No. 18-1488
Figure 2
ROCKRIDGE GEOTECHNICAL

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

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
Qsl	Hillslope Deposits (Quaternary)
Qpa	Alluvium (Pleistocene)
Qoa	Alluvium (early (Pleistocene)
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 1,500 3,000 Feet

Approximate scale

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 06/12/18 | Project No. 18-1488 | 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

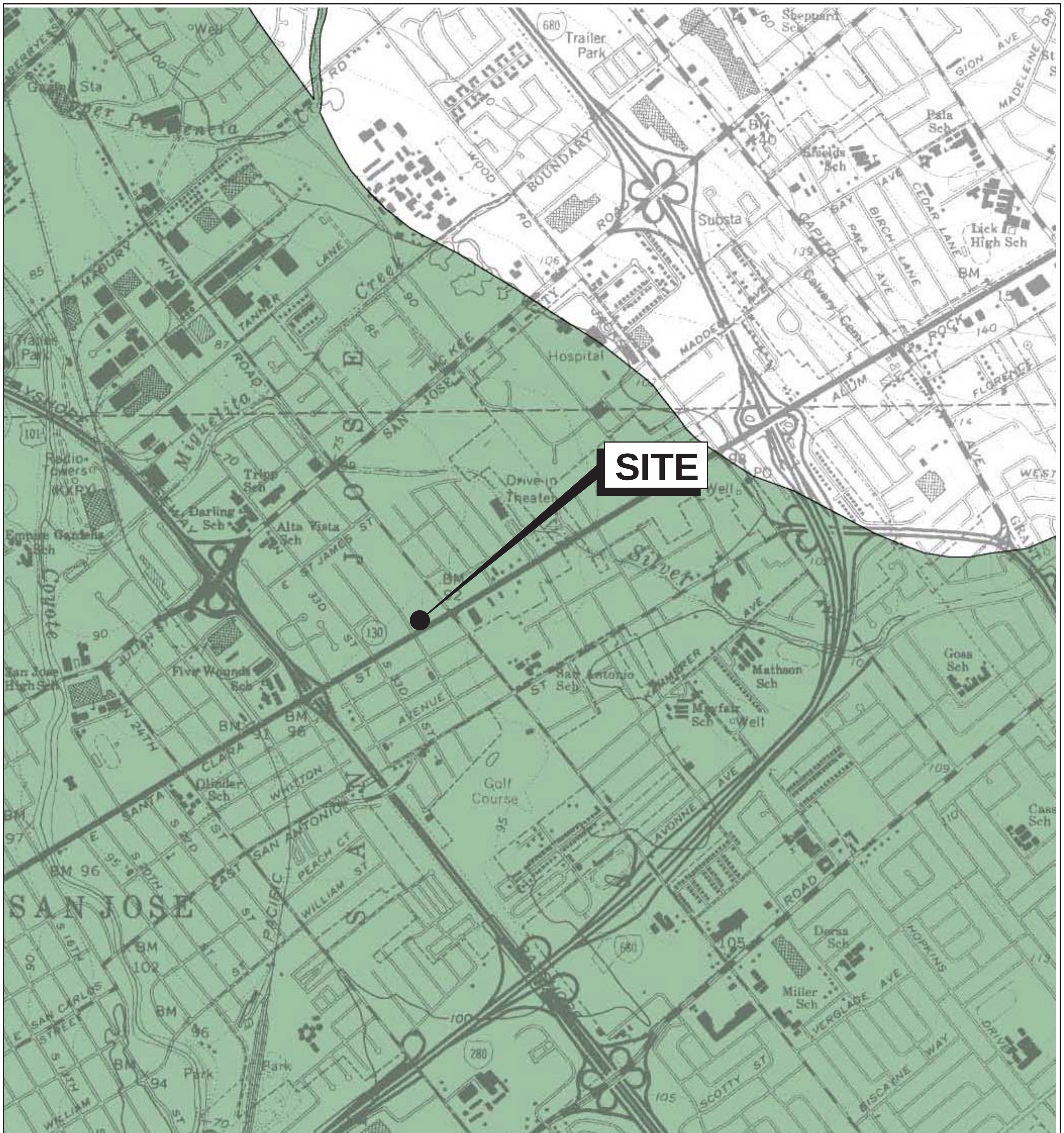
Approximate scale

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 06/12/18	Project No. 18-1488	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.

Reference:
State of California "Seismic Hazard Zones"
San Jose East Quadrangle.
Released on January 17, 2001

0 2000 4000 Feet

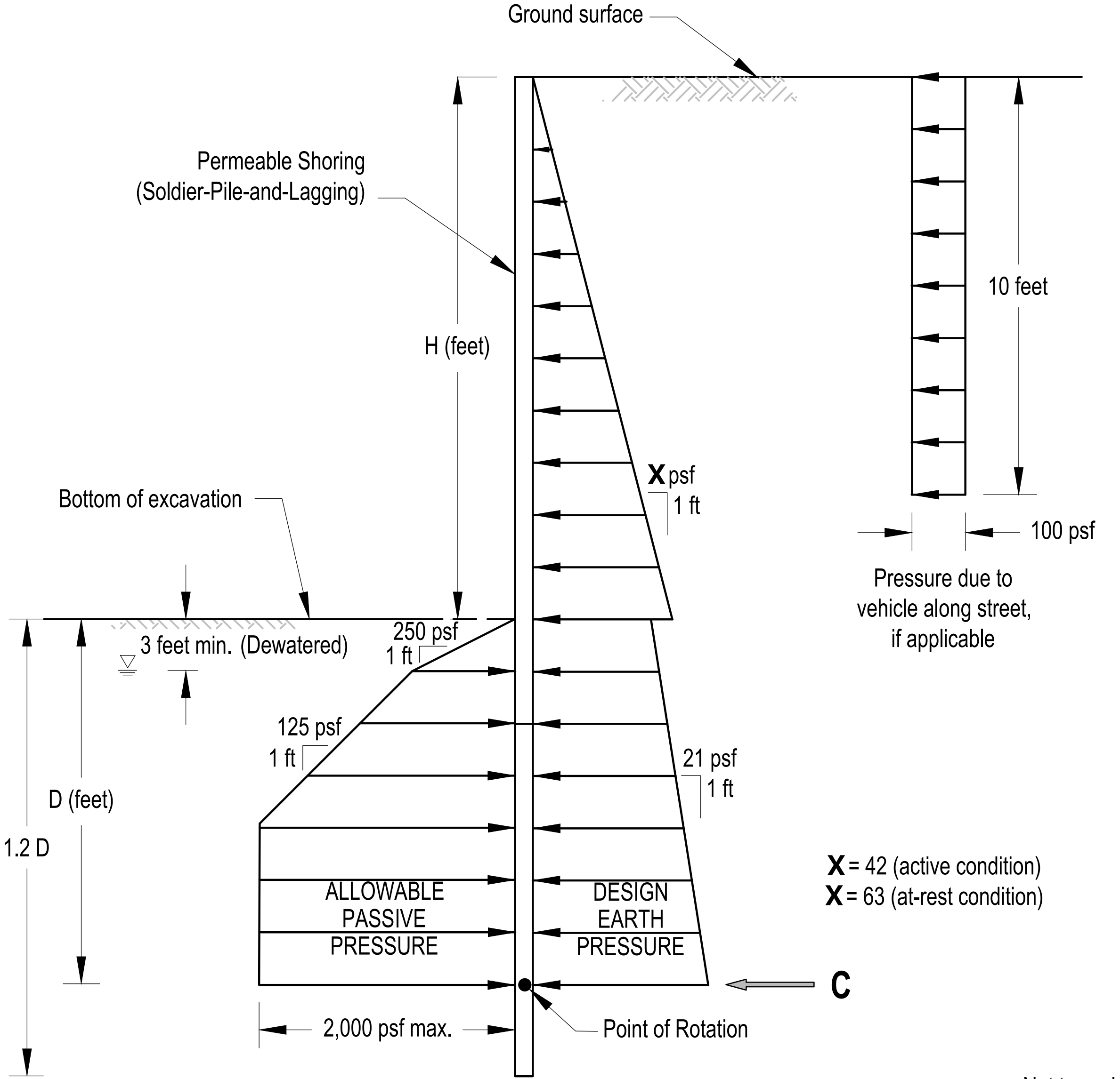
Approximate scale

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

SEISMIC HAZARDS ZONE MAP

ROCKRIDGE GEOTECHNICAL

Date 06/12/18	Project No. 18-1488	Figure 5

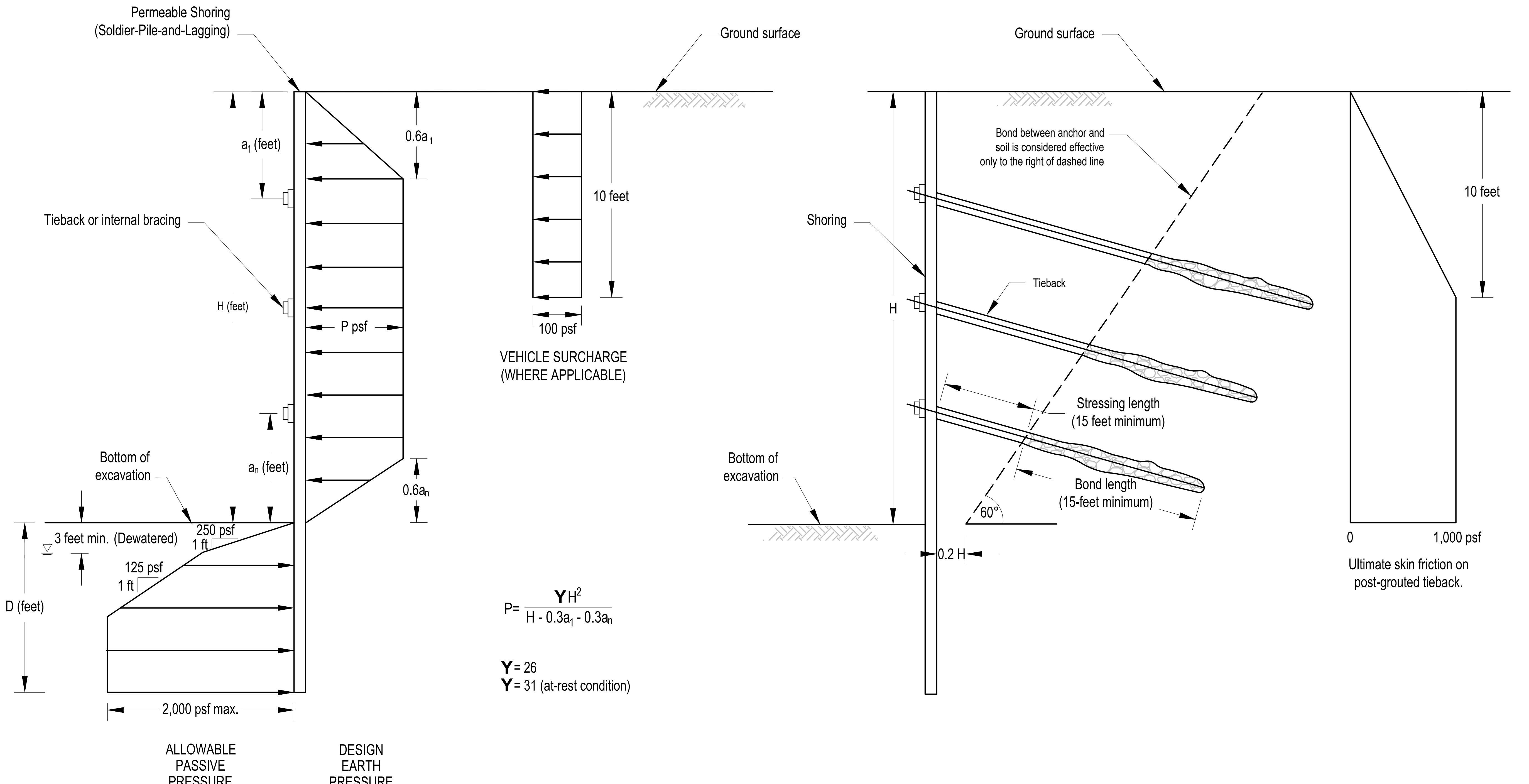


Not to scale

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. Active pressure below the excavation should be assumed to act over one pile diameter.
5. Calculated embedment depth, D, should be increased by at least 20 percent to obtain the design depth of penetration.
6. Does not include loading from adjacent buildings, sloped backfill, or heavy (construction) equipment. These loads should be evaluated on a case-by-case basis where they occur and included in design.

ALUM ROCK AVENUE AND NORTH KING ROAD San Jose, California	**DESIGN PARAMETERS FOR CANTILEVERED SOLDIER-PILE-AND-LAGGING SHORING SYSTEM (DEWATERED)**
ROCKRIDGE GEOTECHNICAL	Date 09/16/18 | Project No. 18-1488 | Figure 6



NOT TO SCALE

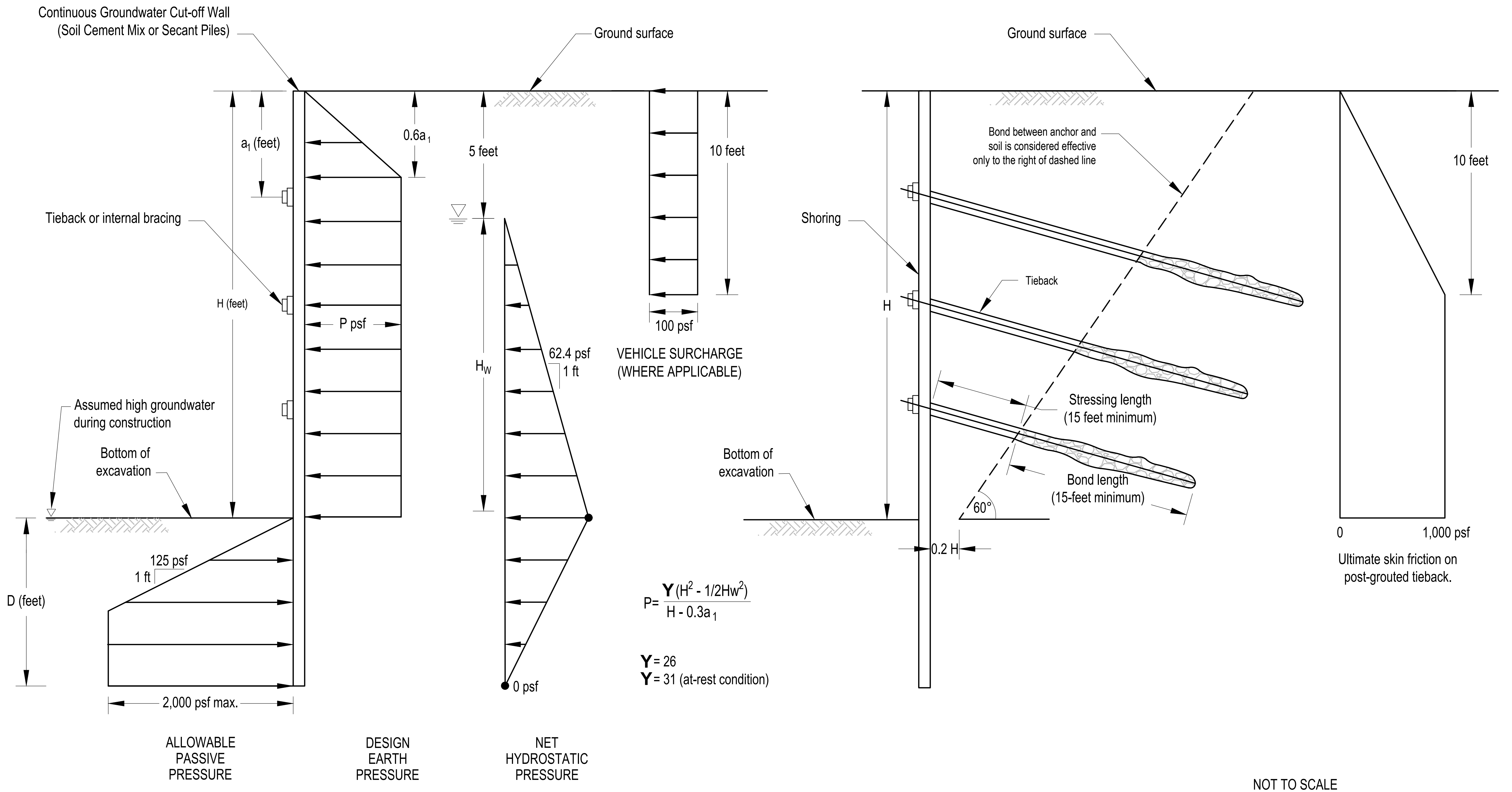
Notes:
1. Passive pressures include a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters, provided the concrete or soil-cement mix is sufficiently strong to accommodate the corresponding stresses (shoring designer should confirm).
3. Does not include loading from adjacent buildings, sloped backfill, or heavy (construction) equipment. These loads should be evaluated on a case-by-case basis where they occur and included in design.

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

DESIGN PARAMETERS FOR ANCHORED SOLIDIER-PILE-AND-LAGGING SHORING SYSTEM (DEWATERED)

Date 06/22/18	Project No. 18-1488	Figure 7

ROCKRIDGE GEOTECHNICAL



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, provided the concrete or soil-cement mix is sufficiently strong to accommodate the corresponding stresses (shoring designer should confirm).
3. Does not include loading from adjacent buildings, sloped backfill, or heavy (construction) equipment. These loads should be evaluated on a case-by-case basis where they occur and included in design.

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

DESIGN PARAMETERS FOR TEMPORARY ANCHORED SHORING SYSTEM/ CUT-OFF WALL

Date 06/22/18	Project No. 18-1488	Figure 8

ROCKRIDGE GEOTECHNICAL

ROCKRIDGE GEOTECHNICAL

APPENDIX A

Boring Logs and Cone Penetration Test Results

PROJECT: **ALUM ROCK AVENUE AND NORTH KING ROAD**
San Jose, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2

Date started: 4/24/18 | Date finished: 4/24/18

Drilling method: 8" Hollow Stem Auger

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

Sampler: Sprague & Henwood (S&H), Shelby Tube (ST)

Logged by: Q. Flores
Drilled by: Exploration Geoservices
Rig: Mobile B-40 Blue

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2 inches of asphalt concrete						
1–3	S&H	8, 8, 11	13	CL	SANDY CLAY (CL) dark brown, stiff, moist, fine sand, trace asphalt debris (FILL)						
3–5	S&H	5, 7, 7	10	CL	SANDY CLAY (CL) brown, stiff, moist, fine sand LL = 27, PI = 10; see Appendix B Corrosivity Test; see Appendix B				82	23.6	98
5–6.5	S&H	3, 5, 7	8	CL	CLAY (CL) brown with trace orange and black mottling, medium stiff to stiff, moist to wet, trace sand					26.9	100
6.5–9				SP-SC	▽ (4/24/2018; 2:05 PM) SAND with CLAY (SP-SC) brown, wet						
9	S&H	4, 5, 7	8	CL	CLAY with SAND (CL) gray-brown, medium stiff to stiff, wet					29.7	96
14	S&H	6, 13, 21	24		gray-brown with gray and orange-brown mottling, very stiff Triaxial Test; see Appendix B	TxUU	1,500	2,300		29.5	96
19	S&H	18, 20, 23	30		seams of light gray and light brown, very stiff to hard	PP		2,750		24.8	103
24	S&H	5, 8, 11	13		stiff	PP		2,000			
29	S&H	9, 11, 14	18		very stiff						

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1488 | Figure: A-1a

PROJECT: **ALUM ROCK AVENUE AND NORTH KING ROAD**
San Jose, California

Log of Boring B-1

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	ST				CL	CLAY with SAND (CL) (continued) olive colored sand nodules Triaxial Test; see Appendix B	TxUU	3,000	1,800		33.2	89
32						Consolidation Test; see Appendix B					37.8	75
33												
34												
35												
36												
37	S&H		8 9 15	17		very stiff, trace coarse-grained sand						
38	ST											
39					CL	SANDY CLAY with GRAVEL (CL) gray-brown, very stiff, wet, subrounded to subangular gravel						
40												
41												
42												
43					SC	CLAYEY SAND with GRAVEL (SC) gray to gray-brown, dense, wet, fine- to coarse-grained sand, subround to subangular gravel						
44	S&H		27 25 19	31	CL	SANDY CLAY (CL) orange-brown and gray, very stiff to hard, wet, trace coarse-grained sand						
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 45 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 7.2 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. SPT sampler used without liners.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1488

Figure: A-1b

Log of Boring B-2

PROJECT: **ALUM ROCK AVENUE AND NORTH KING ROAD**
San Jose, California

Boring location:	See Site Plan, Figure 2
Date started:	4/24/18
Date finished:	4/24/18
Drilling method:	8" Hollow Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Wireline
Sampler:	Sprague & Henwood (S&H), Shelby Tube (ST)
Logged by:	Q. Flores
Drilled by:	Exploration Geoservices
Rig:	Mobile B-40 Blue

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 concrete						
					2.5 inches of aggregate base						
1–2				CL	SANDY CLAY (CL) dark brown, stiff, moist trace coarse-grained sand						
3	S&H	8, 9, 11	14	SM	SILTY SAND (SM) brown, medium dense, moist, fine sand						
4	S&H	6, 5, 6	8	CL	CLAY (CL) light brown, medium stiff to stiff, moist, fine sand, trace sand						
6	S&H	4, 5, 5	7		LL = 31, PI = 10; see Appendix B				95	25.3	94
9	S&H	4, 5, 7	8		∇ wet					31.5	93
10				SC	CLAYEY SAND (SC) brown, loose, wet, fine sand						
11				CL-ML	SANDY SILTY CLAY (CL-ML) brown, medium stiff, wet, fine sand						
13.5					LL = 26, PI = 5; see Appendix B						
14	S&H	4, 4, 5	6						69	28.6	97
17				CL	CLAY with SAND (CL) gray-brown trace red-brown mottling, hard, wet						
19	S&H	18, 28, 36	45								
24	S&H	10, 10, 18	20		very stiff, increase in sand content						
26	ST										
29	S&H	4, 8, 13	15		gray with orange-brown mottling, stiff to very stiff	PP		2,000			
30											

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1488 | Figure: A-2a

PROJECT: **ALUM ROCK AVENUE AND NORTH KING ROAD**
San Jose, California

Log of Boring B-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
						CLAY with SAND (CL) (continued)						
31												
32												
33												
34	S&H		10 14 27	29		gray, very stiff to hard	PP		2,000			
35												
36	ST					Triaxial Test; see Appendix B Consolidation Test; see Appendix B	TxUU	3,500	2,000		27.2	96
37					CL						25.2	99
38												
39	S&H		8 11 14	18		very stiff						
40												
41												
42												
43												
44	S&H		8 8 14	15		stiff to very stiff						
45					SC	CLAYEY SAND (SC) light olive-brown, medium dense, wet						
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 45 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 9 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. SPT sampler used without liners.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1488

Figure: A-2b

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

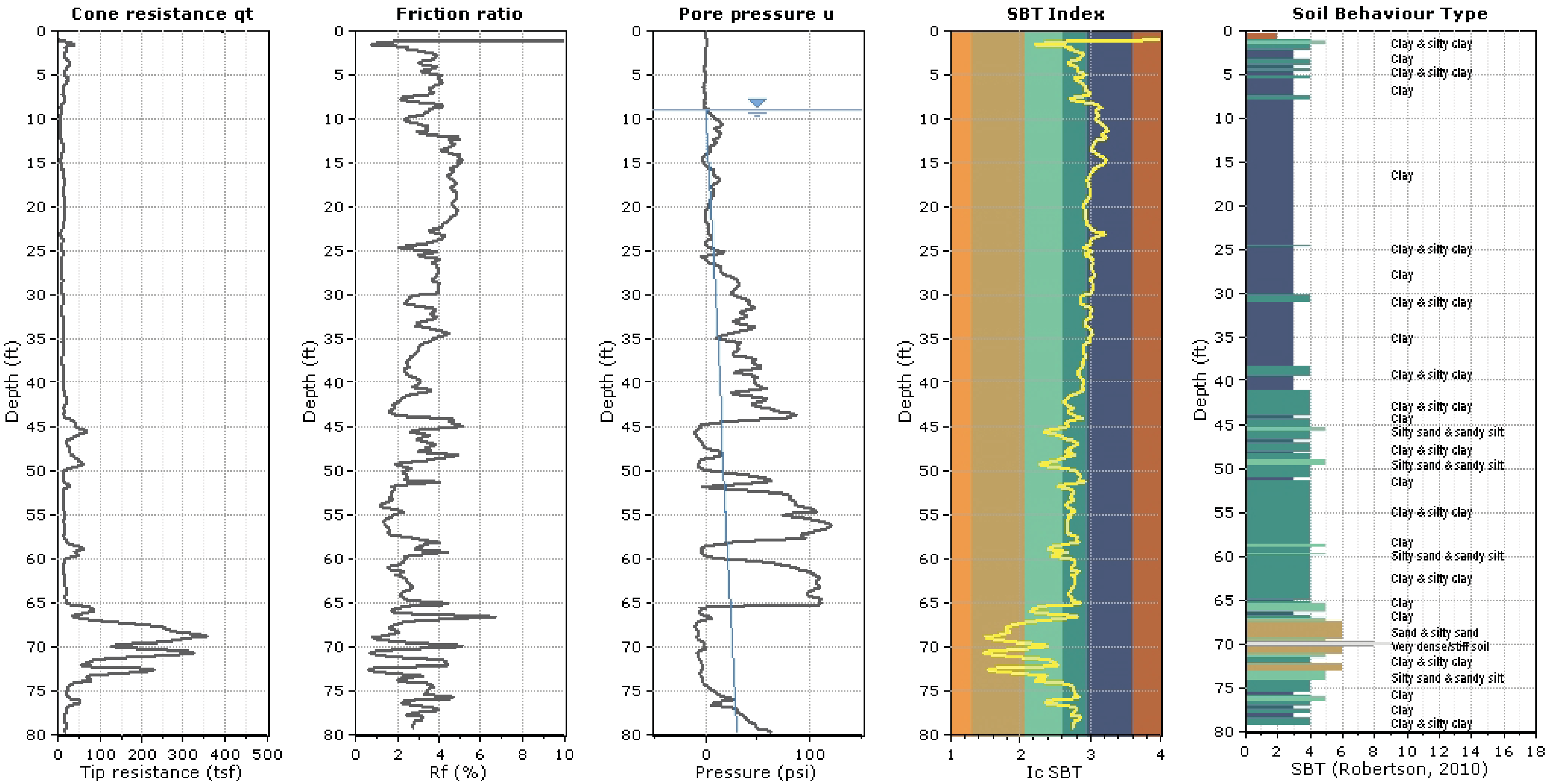
C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler 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

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

CLASSIFICATION CHART

Date 06/12/18 | Project No. 18-1488 | Figure A-3



SBT legend

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

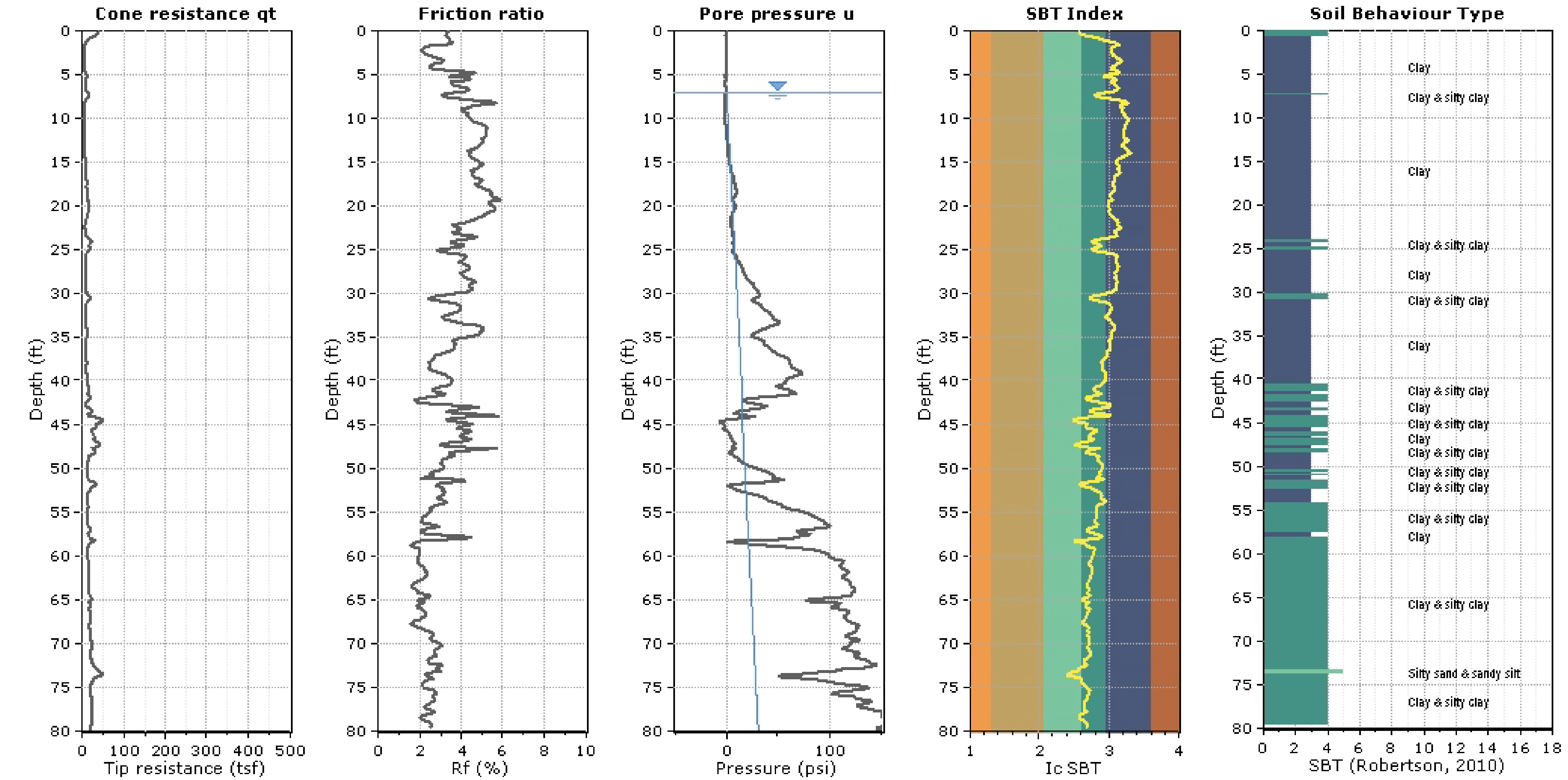
Total depth: 79.72 ft, Date: 5/4/2018
Depth to Groundwater: 9 feet
Cone Operator: Middle Earth Geo Testing, Inc.

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 06/20/18	Project No. 18-1488	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

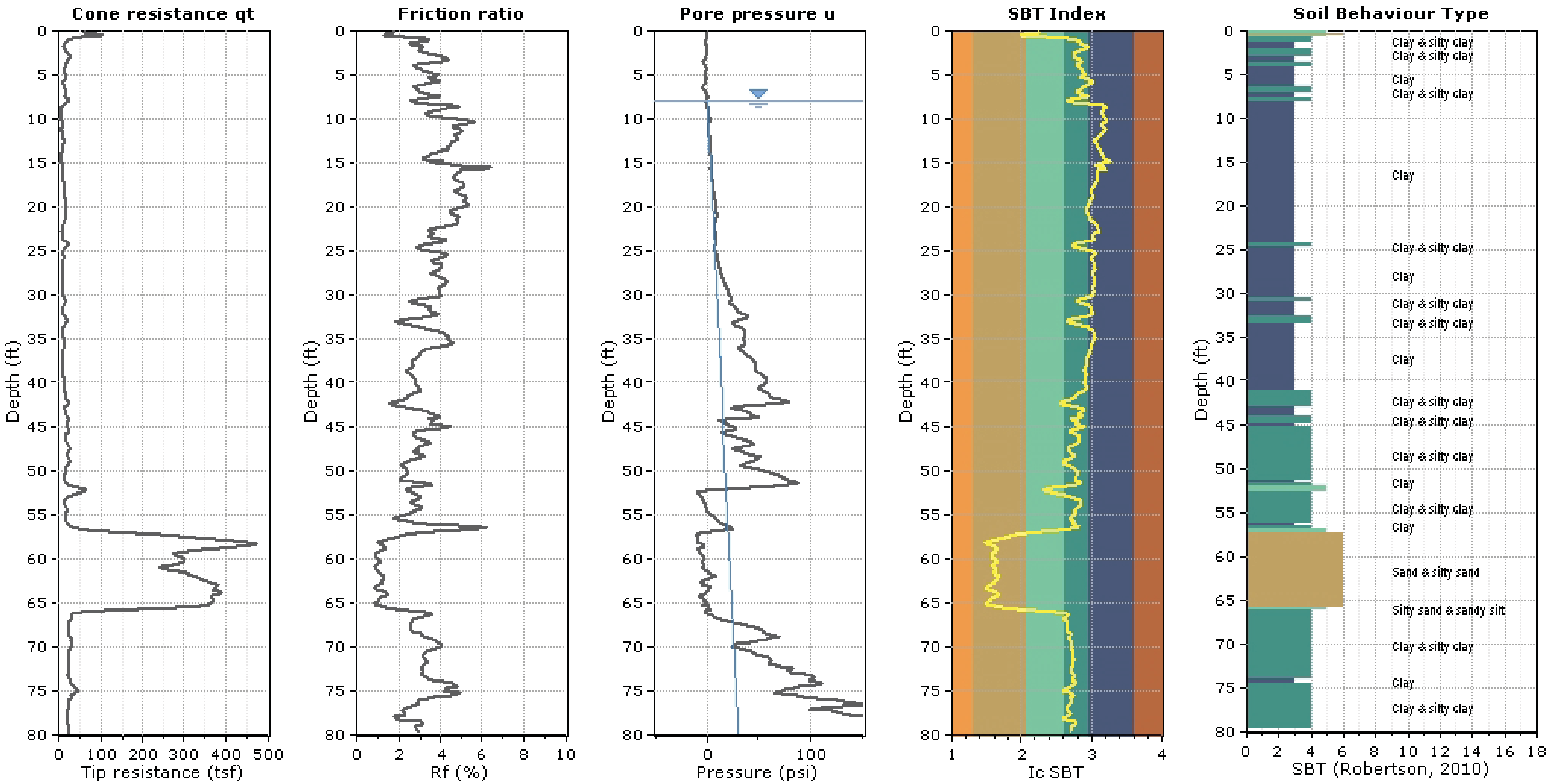
Total depth: 80.22 ft, Date: 5/4/2018
Depth to Groundwater: 7 feet
Cone Operator: Middle Earth Geo Testing, Inc.

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 06/20/18 | Project No. 18-1488 | 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

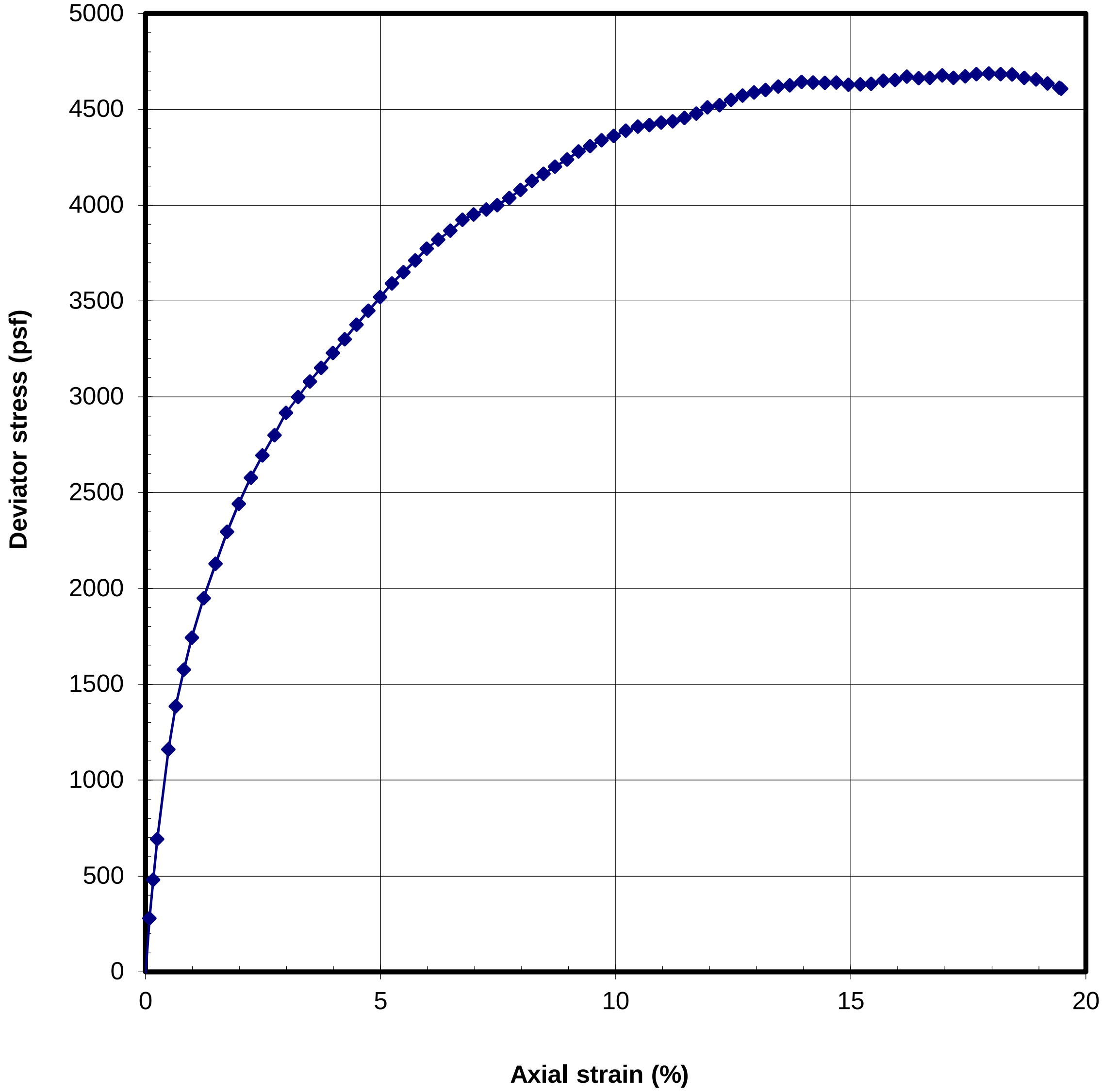
Total depth: 80.22 ft, Date: 5/4/2018
Depth to Groundwater: 8 feet
Cone Operator: Middle Earth Geo Testing, Inc.

ALUM ROCK AVENUE AND NORTH KING ROAD San Jose, California	CONE PENETRATION TEST RESULTS CPT-3		
ROCKRIDGE GEOTECHNICAL	Date 06/20/18	Project No. 18-1488	Figure A-6

ROCKRIDGE
GEOTECHNICAL

APPENDIX B

Laboratory Test Results



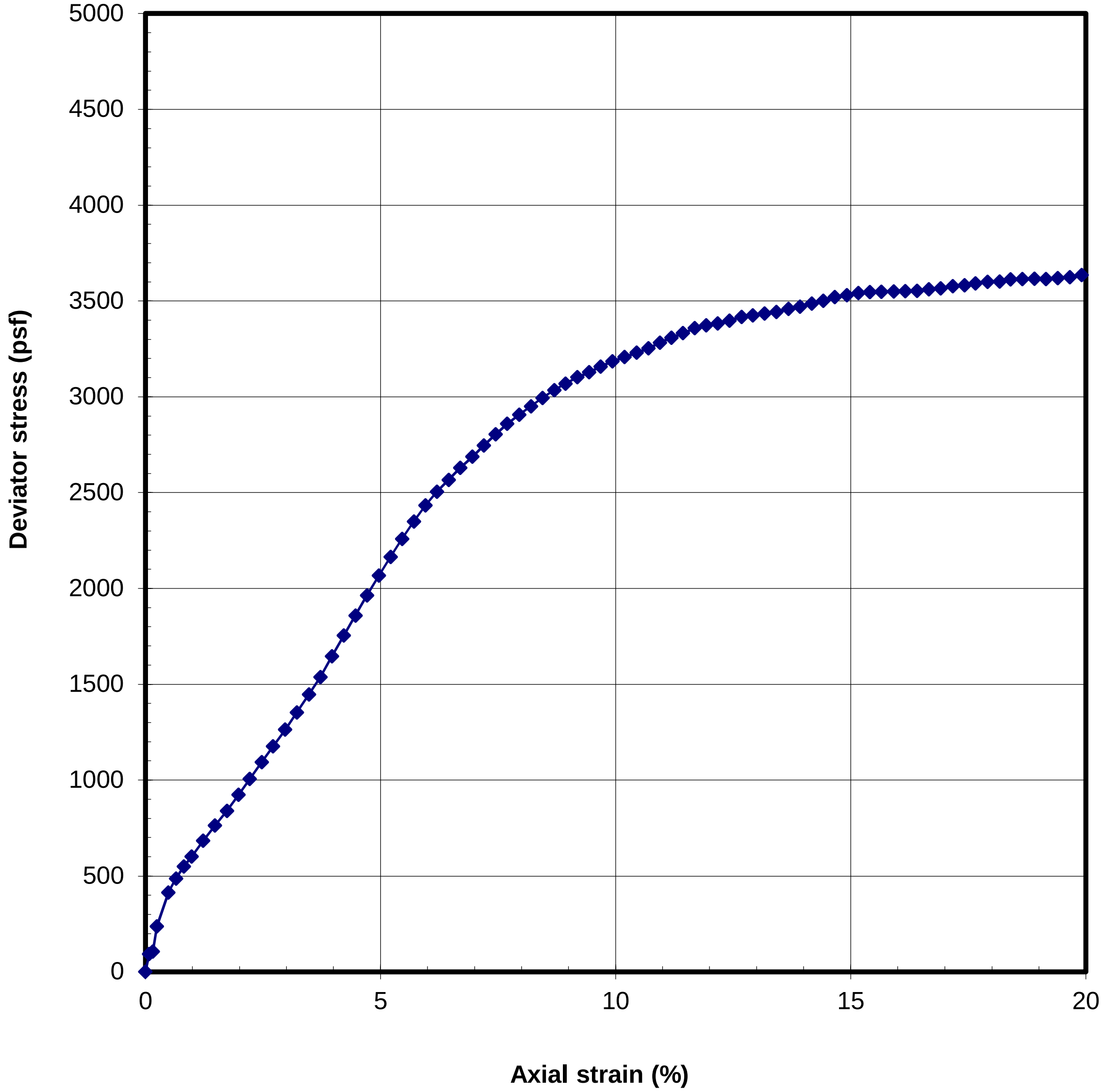
Sampler Type: Sprague & Henwood (S&H)	Shear Strength: 2,300 psf
Diameter (in): 2.39 — Height (in): 5.53	Strain at Failure: 17.90%
Moisture Content: 29.5 %	Confining Pressure: 1,500 psf
Dry Density: 96 pcf	Strain Rate: 1%/min
Source: B-1 at 14.5 feet	
Description: CLAY (CL), gray-brown with gray and orange mottling	

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date: 09/15/18 | Project No. 18-1488 | Figure B-1



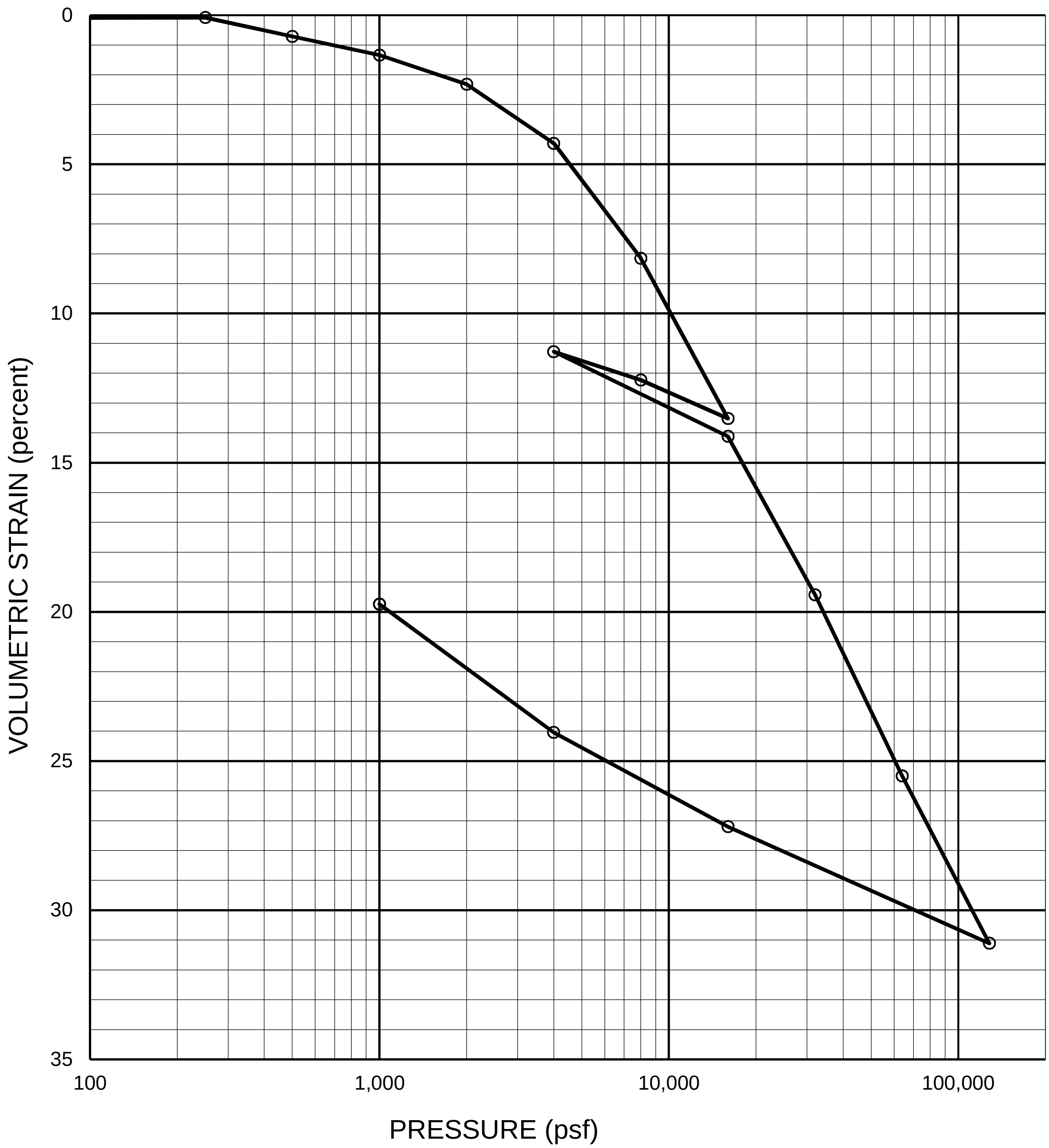
Sampler Type: Shelby Tube		Shear Strength:	1,800 psf
Diameter (in): 2.82	Height (in): 6.4	Strain at Failure:	20.00%
Moisture Content: 33.2 %		Confining Pressure:	3,000 psf
Dry Density: 89 pcf		Strain Rate:	1%/min
Source: B-1 at 31.0 feet			
Description: CLAY with SAND (CL), light gray-brown with olive			

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date: 09/15/18 | Project No. 18-1488 | Figure B-2



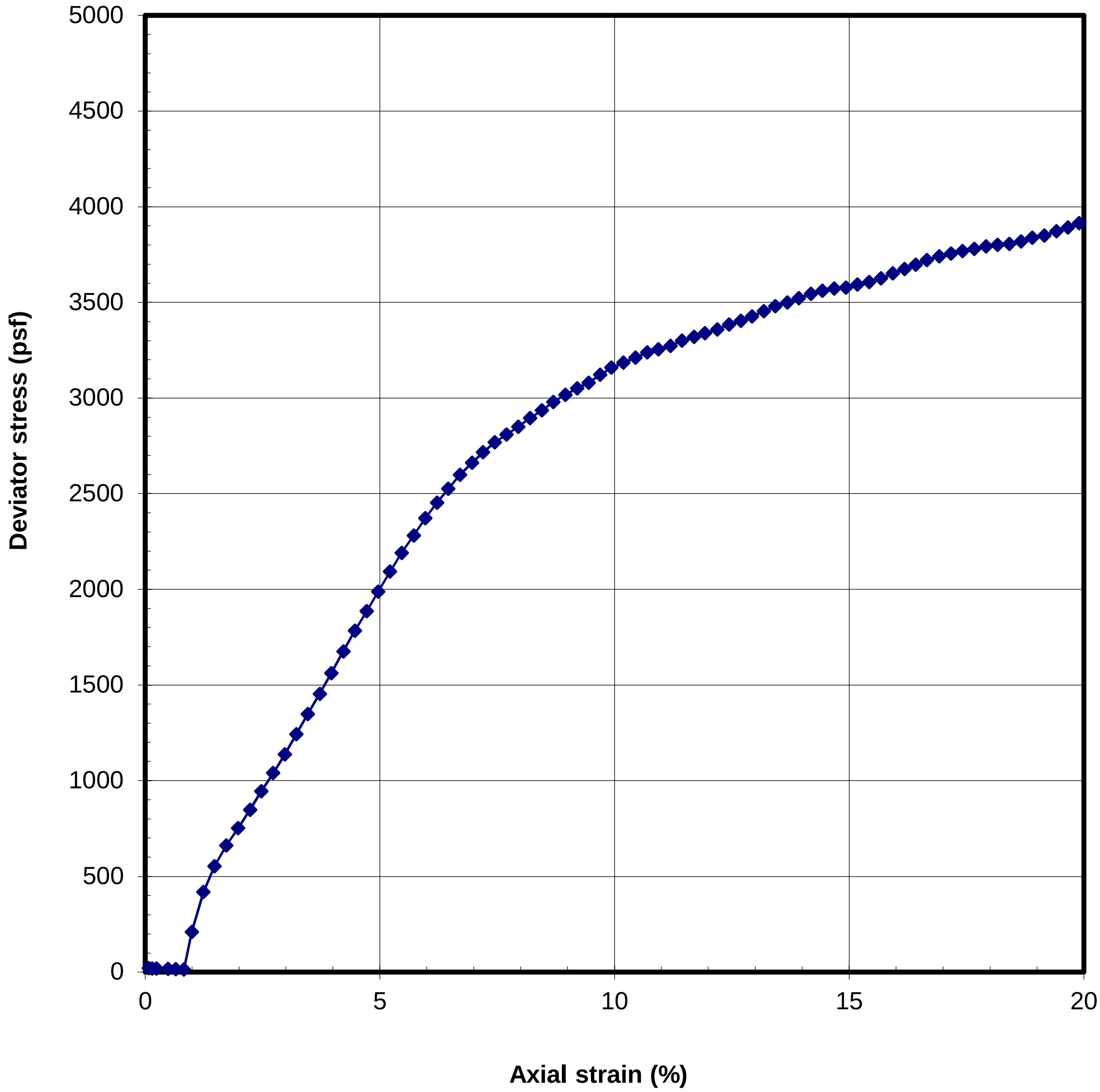
Sampler Type:	Shelby Tube			Condition		Before Test			After Test		
Diameter (in)	2.00	Height (in)	0.75	Water Content		w_o	37.8	%	w_f	30.3	%
Overburden Pressure, p_o'	2,000	psf		Void Ratio		e_o	1.28		e_f	0.83	
Preconsol. Pressure, p_c'	6,000	psf		Saturation		S_o	99.5	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.19			Dry Density		γ_d	75	pcf	γ_d	94	pcf
Recompression Ratio, $C_{\varepsilon r}$	0.04	LL	--	PL	--	PI	--	G_s	2.75	(assumed)	
Source:	B-1 at 31.5 feet										
Description:	CLAY with SAND (CL), light gray-brown with olive										

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONSOLIDATION TEST REPORT

Date	09/15/18	Project No.	18-1488	Figure B-3



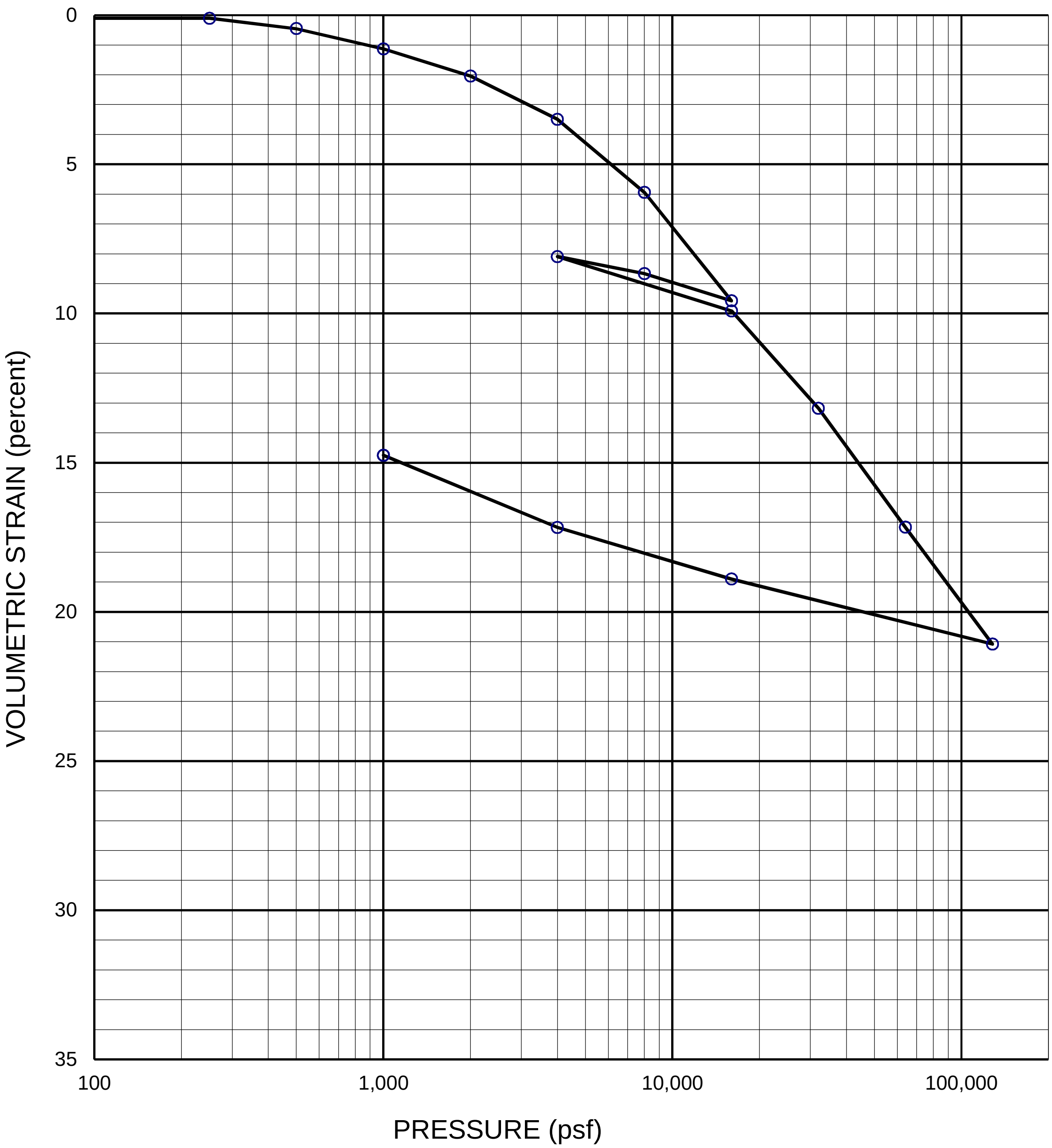
Sampler Type: Shelby Tube		Shear Strength:	2,000 psf
Diameter (in): 2.81	Height (in): 6.1	Strain at Failure:	20.00%
Moisture Content: 27.2 %		Confining Pressure:	3,500 psf
Dry Density: 96 pcf		Strain Rate:	1%/min
Source: B-2 at 36.0 feet			
Description: CLAY with SAND (CL), gray			

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date: 09/15/18 | Project No. 18-1488 | Figure B-4



Sampler Type:	Shelby Tube			Condition		Before Test			After Test	
Diameter (in)	2.00	Height (in)	0.75	Water Content	w_o	25.2	%	w_f	17.3	%
Overburden Pressure, p_o	2,200	psf		Void Ratio	e_o	0.73		e_f	0.47	
Preconsol. Pressure, p_c	6,500	psf		Saturation	S_o	95.1	%	S_f	100	%
Compression Ratio, $C_{\varepsilon c}$	0.13			Dry Density	γ_d	99	pcf	γ_d	117	pcf
Recompression Ratio, $C_{\varepsilon r}$	0.03	LL --	PL --	PI --		G_s	2.75	(assumed)		

Source: B-2 at 36.5 feet

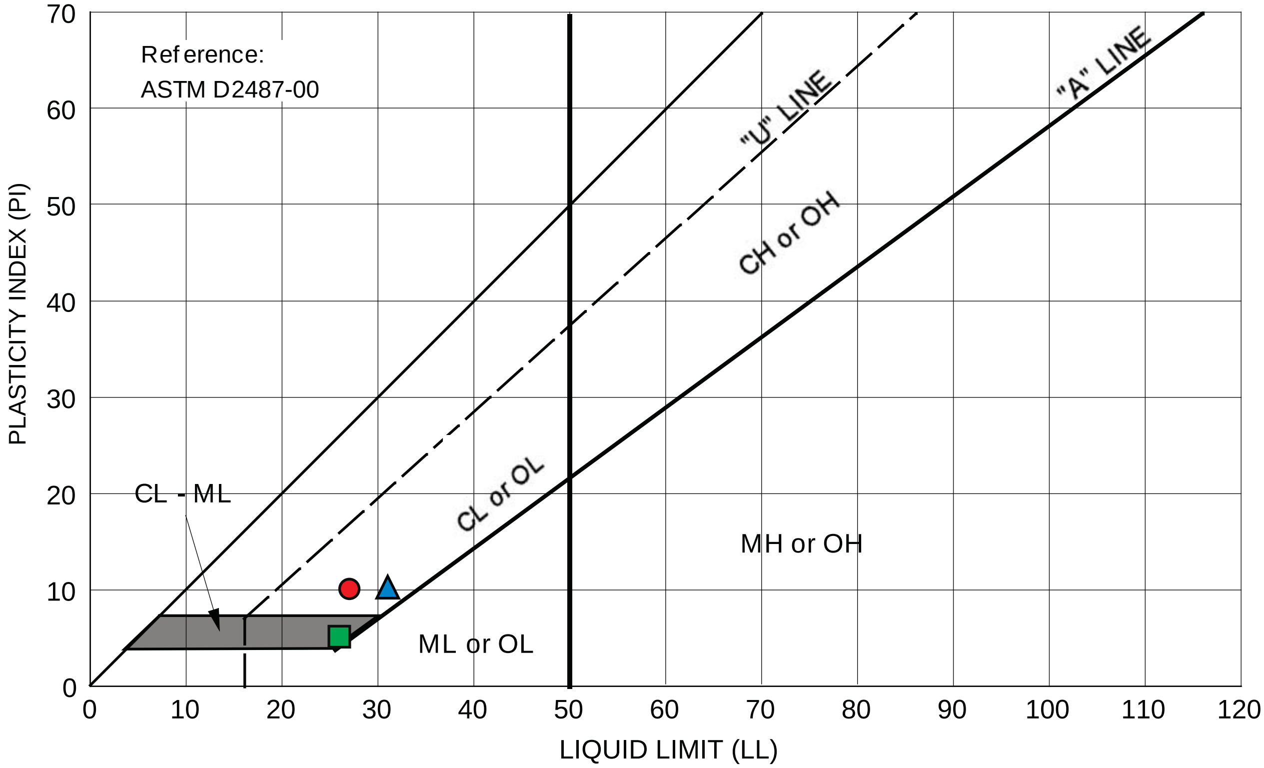
Description: CLAY with SAND (CL), gray

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONSOLIDATION TEST REPORT

Date 09/11/18 | Project No. 18-1488 | Figure B-5



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 4.0 feet	SANDY CLAY (CL), brown	23.6	27	10	82
▲	B-2 at 6.0 feet	CLAY (CL), light brown	25.3	31	10	95
■	B-2 at 14.5 feet	SANDY SILTY CLAY (CL-ML), brown	28.6	26	5	69

ALUM ROCK AVENUE AND NORTH KING ROAD
San Jose, California

ROCKRIDGE GEOTECHNICAL

PLASTICITY CHART

Date 06/19/18 | Project No. 18-1488 | Figure B-6

Soil Analysis Lab Results

Client: Rockridge Geotechnical
Job Name: Alum Rock Ave + North King
Client Job Number: 18-1488
Project X Job Number: S180614B
June 20, 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 #1 / Sandy Silty Clay	3.0	2,010	1,809	30	0.0030	15	0.0015	24	1.3	0.09	111	8.70

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

Please call if you have any questions.

Respectfully Submitted,

Eddie Hernandez, M.Sc., P.E.
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NACE Corrosion Technologist #16592
Professional Engineer
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REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

ROCKRIDGE GEOTECHNICAL

APPENDIX C

Summary of Liquefaction Analyses

Rockridge Geotechnical, Inc.
Oakland, California

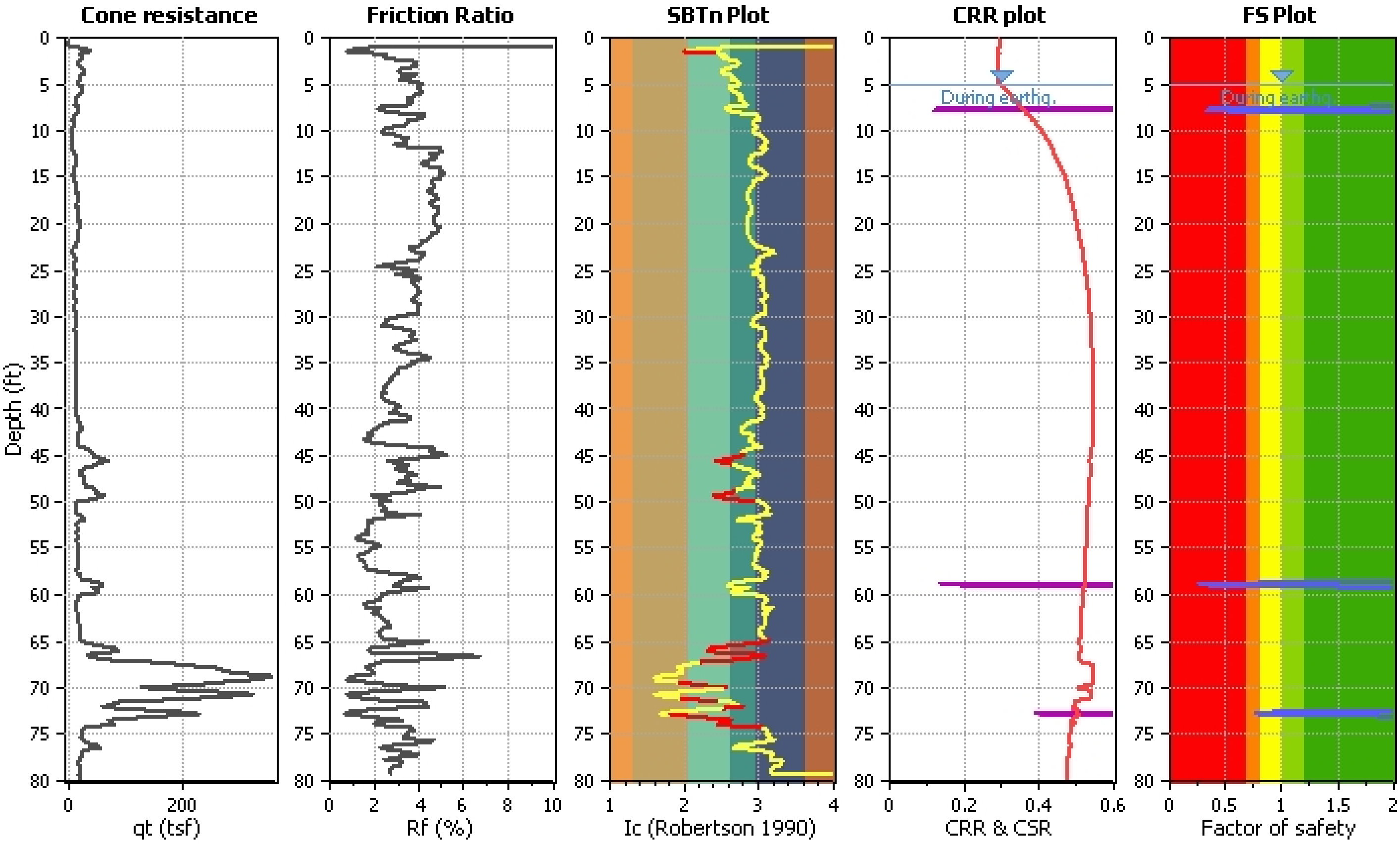
LIQUEFACTION ANALYSIS REPORT

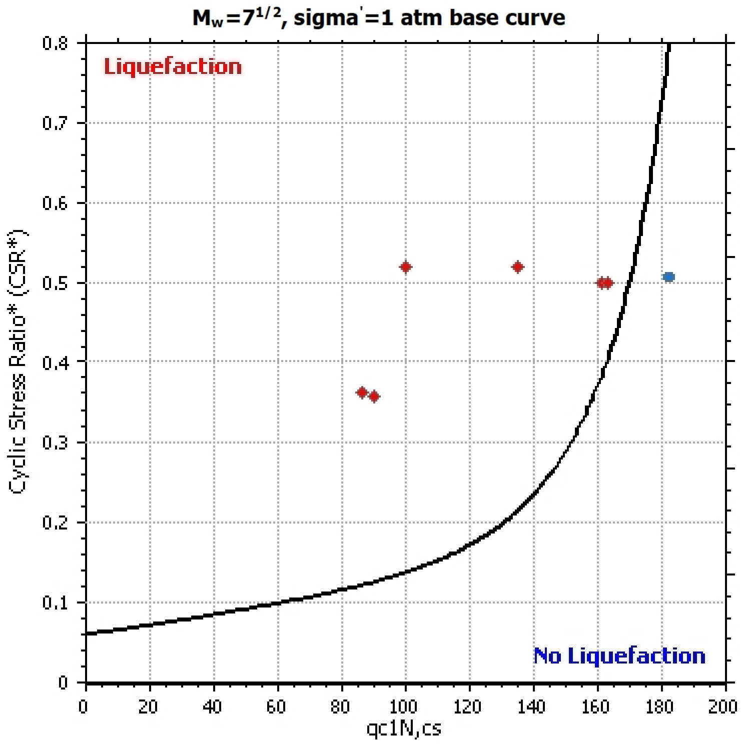
Project title : Alum Rock Avenue and North King Drive **Location : San Jose, California**

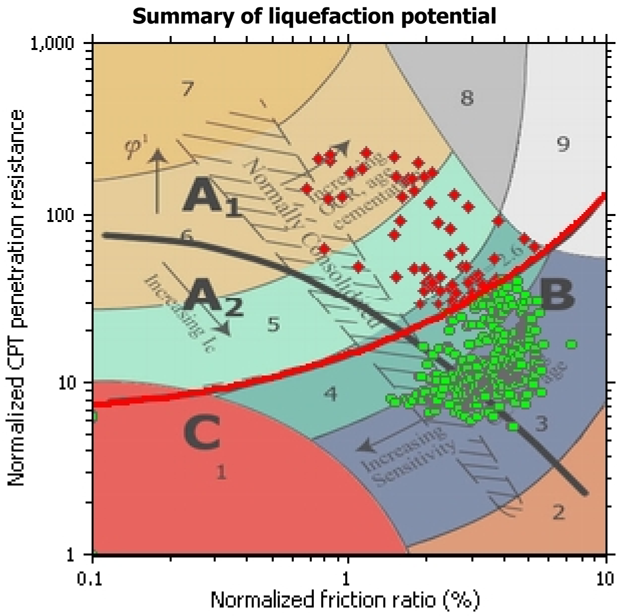
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 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:	0.50	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

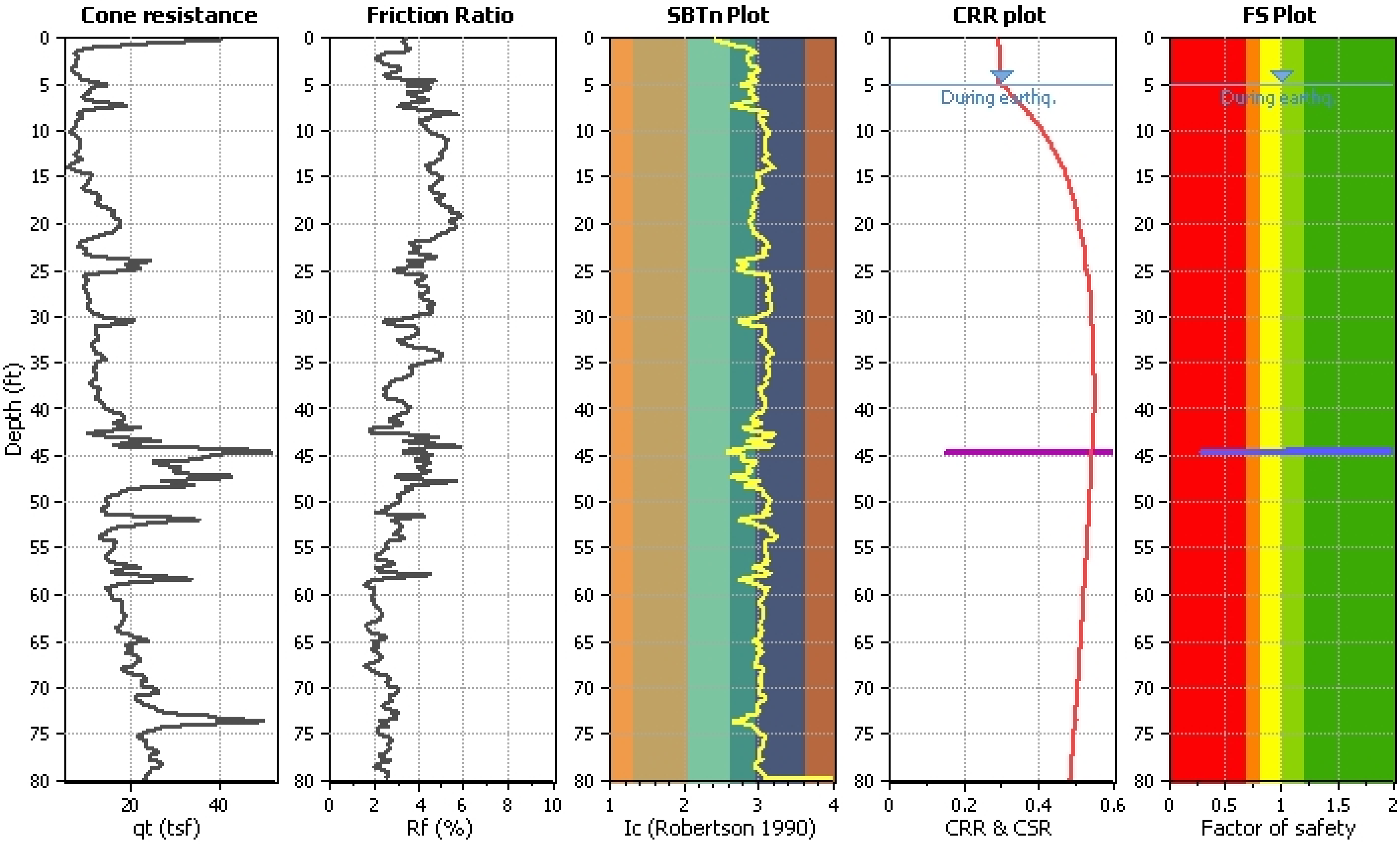
LIQUEFACTION ANALYSIS REPORT

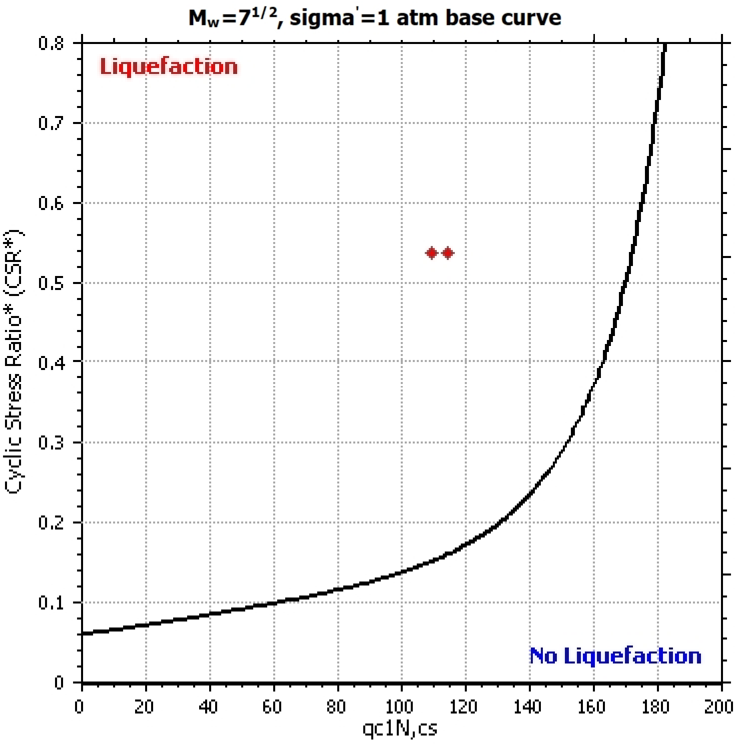
Project title : Alum Rock Avenue and North King Drive **Location : San Jose, California**

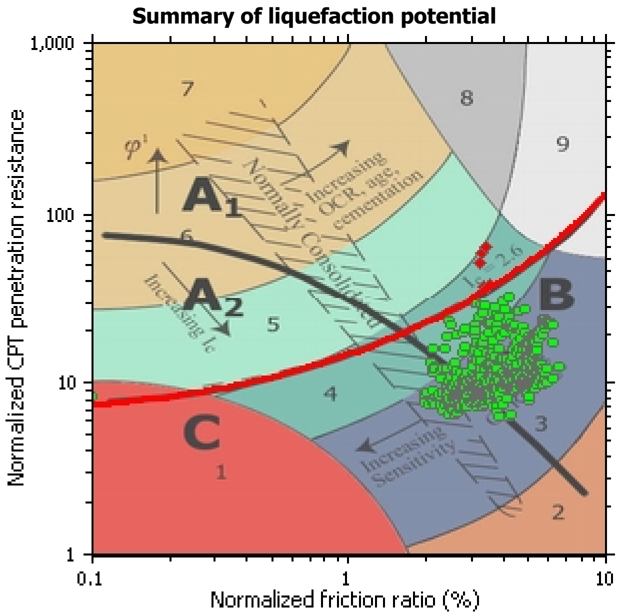
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	7.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 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:	0.50	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

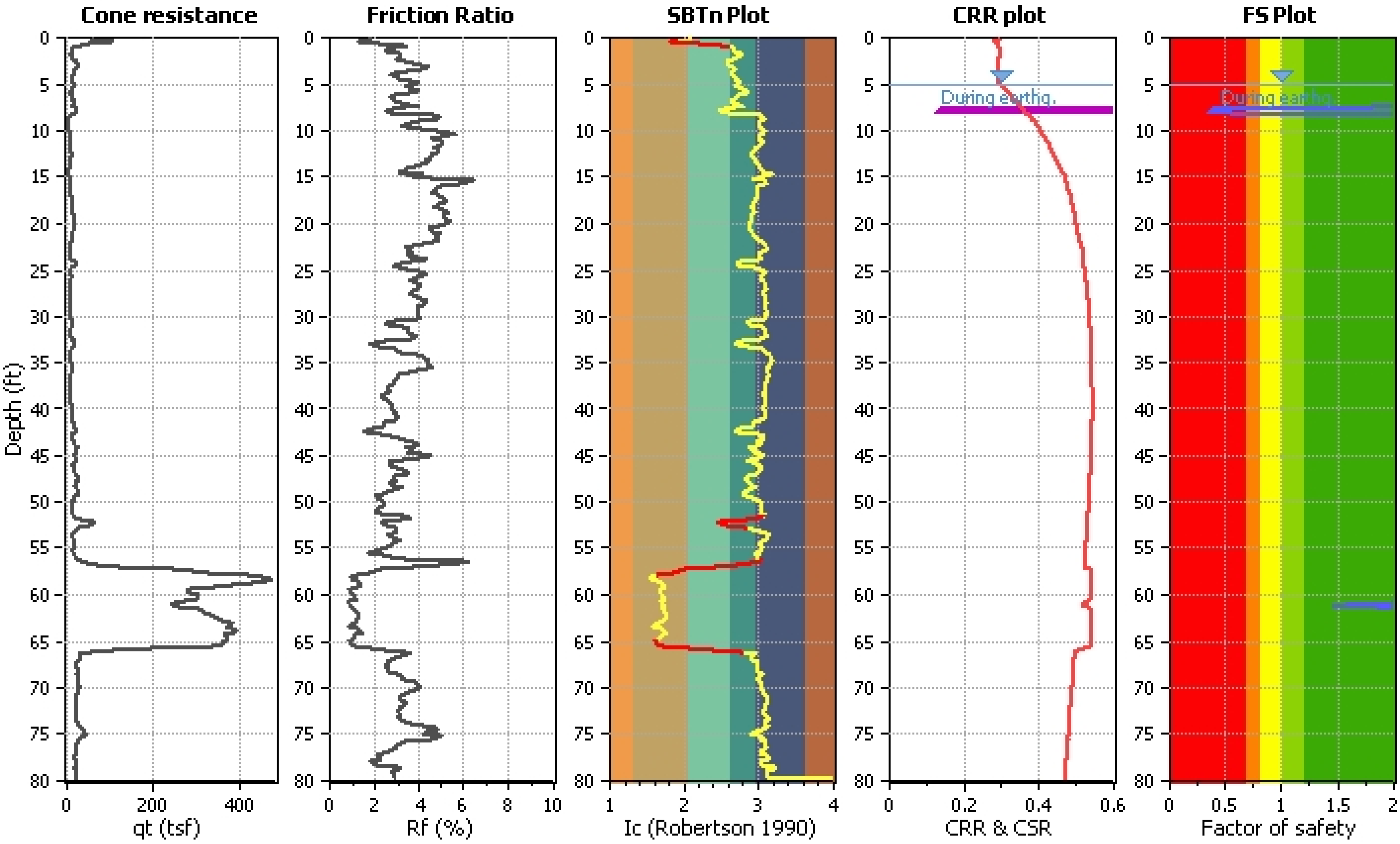
LIQUEFACTION ANALYSIS REPORT

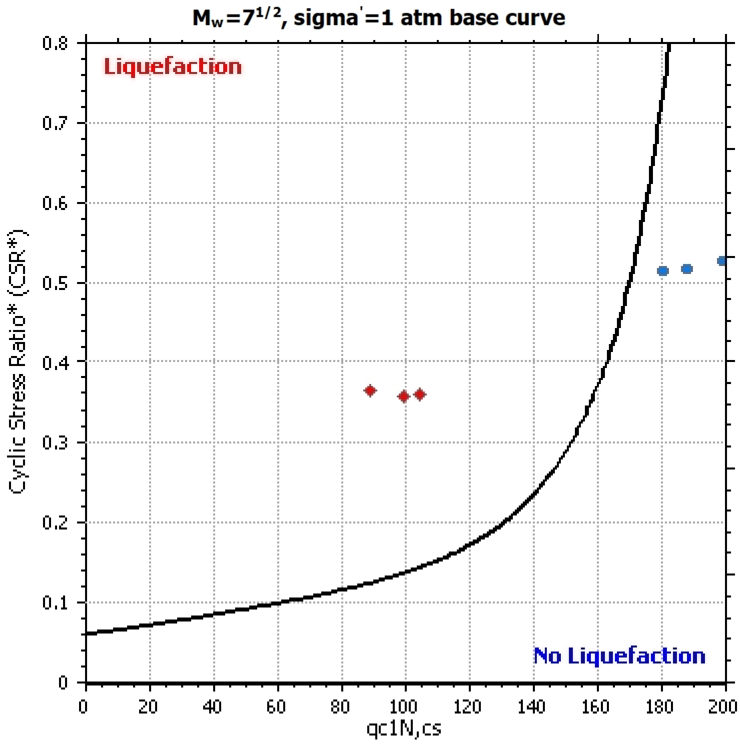
Project title : Alum Rock Avenue and North King Drive **Location : San Jose, California**

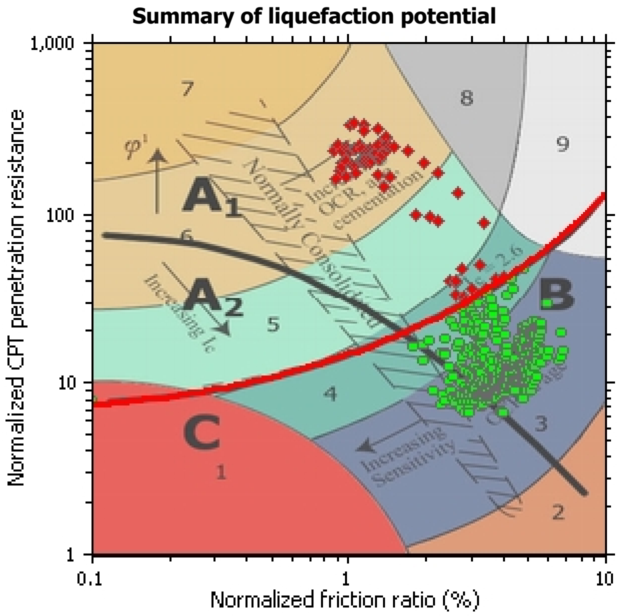
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	9.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	5.00 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:	0.50	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