Prepared for **MSASA Properties, LLC**

GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT BUILDING
380 NORTH FIRST STREET
SAN JOSE, CALIFORNIA

April 14, 2023
Project No. 23-2365

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ROCKRIDGE GEOTECHNICAL

April 14, 2023
Project No. 23-2365

Mr. Mostafa Aghamiri
MSASA Properties, LLC
130 Sisters Court
Los Gatos, California 95030

Subject: Geotechnical Investigation Report
Proposed Apartment Building
380 North First Street
San Jose, California

Dear Mr. Aghamiri:

We are pleased to present the results of our geotechnical investigation for the design and construction of the proposed apartment building to be constructed at 380 North First Street in San Jose, California. Our geotechnical investigation was performed in accordance with our proposal dated January 19, 2023.

The site is a relatively level, rectangular-shaped parcel on the northeastern corner of North First and Bassett streets. It has plan dimensions of approximately 75 by 270 feet. It is currently occupied by a two-story office building at the southwestern end of the site, with an adjacent asphalt-paved parking lot occupying the rest.

Plans are to construct a new eight-story, at-grade residential building that will occupy most of the site. The lower two stories will be of Type I-A (concrete) construction, and the upper six stories will be of Type III-A construction. The ground floor and first level will be occupied by parking, utility rooms, an amenities room, and a leasing office. Parking lift pits will be constructed along two sides of the building. The pits will extend about 8-1/2 feet below grade.

Based on the results of our geotechnical investigation, we conclude there are no major geotechnical issues that would preclude the development of the site as proposed. The primary geotechnical issues affecting the proposed development are providing adequate foundation support for the proposed building and the relatively shallow groundwater relative to the proposed parking pit excavations. We conclude that the proposed building may be supported on a mat foundation, provided the static and seismically induced settlements are acceptable from a structural standpoint.

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The recommendations contained in our report are based on a limited subsurface exploration. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe site preparation, shoring installation, and foundation installation, during which time we may make changes to our recommendations if deemed necessary. We appreciate the opportunity to provide our services to you on this project. Should you have any questions, please call.

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

Darcie Maffioli, P.E., G.E.
Senior Engineer

Craig S. Shields P.E., G.E.
Principal Engineer

Enclosure

TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX A

APPENDIX B

GEOTECHNICAL INVESTIGATION
PROPOSED APARTMENT BUILDING
380 NORTH FIRST STREET
San Jose, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed apartment building to be constructed at 380 North First Street in San Jose, California. The subject property is located between North First and North 2nd streets on the northern side of Bassett Street, as shown on the Site Location Map, Figure 1.

The project site is a relatively level, rectangular-shaped parcel with maximum plan dimensions of about 75 feet by 270 feet. It is bordered by a vacant lot and a commercial building to the northwest, North 2nd Street to the northeast, Bassett Street to the southeast, and North First Street to the southwest. It is currently occupied by a two-story office building at the southwestern end of the site with an adjacent asphalt-paved parking lot occupying the rest of the site.

Plans are to demolish the existing building and construct an at-grade, eight-story residential building that will occupy most of the site. The lower two stories will be of Type 1-A (concrete) construction, and the upper six stories will be of Type III-A construction. The ground floor and second level will be occupied by parking, utility rooms, an amenities room, and a leasing office. Parking lift pits will be constructed along two sides of the building. Based on the project plans developed by Studio T Square, we understand that the parking pits will extend about 8-1/2 feet below grade.

2.0 SCOPE OF SERVICES

Our investigation was performed in accordance with our proposal dated January 19, 2023. Our scope of services consisted of exploring subsurface conditions at the site by performing two cone penetration tests (CPTs), drilling two test borings, and performing laboratory testing and engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions at the site

- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of total and differential building settlement
- design groundwater level
- site grading and excavation, including criteria for fill quality and compaction
- subgrade preparation for slab-on-grade floors
- 2022 California Building Code (CBC) site class and design spectral response acceleration parameters
- shoring design parameters for lift-pit excavations
- soil corrosivity
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

We investigated the subsurface conditions beneath the site by performing two CPTs, drilling two test borings, and performing laboratory testing on selected soil samples from the borings. Before performing the field investigation, we obtained a boring permit from the Santa Clara Valley Water District (SCVWD). 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, C. Cruz Sub-Surface Locators, to minimize the potential that an underground utility was encountered during our work. Details of the field investigation are described below.

3.1 Cone Penetration Tests

Two CPTs were performed at the project site to provide in-situ soil data at the approximate locations shown on Figure 2. The CPTs, designated as CPT-1 and CPT-2, were advanced to depths of approximately 50 and 100 feet below the existing ground surface (bgs), respectively.

The CPTs were performed by Middle Earth Geo Testing, Inc. (Middle Earth) of Hayward, California, on February 17, 2023 by hydraulically pushing a 1.7-inch-diameter cone-tipped probe with a projected area of 15 square centimeters into the ground. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone measured soil parameters at a recording interval of approximately 2 inches for the depth advanced. Soil data, including tip resistance, frictional resistance, and pore water pressure, were recorded by a computer while the test was conducted. A computer processed accumulated data to provide engineering information, such as the soil behavior types (Robertson, 2010) and approximate strength characteristics of the soil encountered. The CPT logs showing tip resistance, friction ratio, pore pressure, and correlated soil behavior type with depth are presented in Appendix A in Figures A-1a, and A-2. Groundwater was measured with a tape drop or estimated with a pore pressure dissipation test in each CPT. The depth of the groundwater and the measurement method are noted on the CPT logs. Shear wave velocities of the soil were measured at approximately 5-foot intervals while advancing CPT-1. A plot of the measured shear wave velocities at each interval is presented in Figure A-1b.

Upon completion, the CPTs were backfilled with cement grout in accordance with SCVWD standards.

3.2 Test Borings

Two drilling subcontractors drilled borings to complete the subsurface investigation. Boring B-2 was drilled by Exploration Geoservices, Inc. of San Jose, California on February 13, 2023, using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem flight augers. Boring B-2 was drilled to a depth of 45 feet bgs. Boring B-1 was drilled by Stapleton Engineering and Exploration of Santa Rosa, California on February 22, 2023 using portable drilling equipment mounted on a skid steer (Bobcat) equipped with 4-inch-diameter solid-stem flight augers. Boring B-1 was drilled to a depth of 41-1/2 feet bgs.

During drilling, our field engineers logged the borings and obtained representative samples for visual classification and laboratory testing. Our field engineers noted the date and time when groundwater was encountered during drilling. The final boring logs were developed based on laboratory test data, a review of soil samples in the office, and the conditions recorded on the field logs. The boring logs are presented in Appendix A on Figures A-3a through A-4b. The soil was classified in accordance with the classification system presented on Figure A-5.

Soil samples were obtained using the following samplers:

- Modified California (MCH split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter tubes.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.

For Boring B-2, the MC and SPT samplers were driven with a 140-pound, automatic hammer falling about 30 inches per drop. For Boring B-1, the MC and SPT samplers were driven with a 140-pound safety hammer falling approximately 30 inches per drop using a rope-and-cathead pulley system. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every 6 inches and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per 6 inches of penetration or 50 blows for 6 inches or less of penetration. The blow counts required to drive the MC and SPT samplers were converted to approximate SPT N-values using factors of 0.63 and 1.08 for Boring B-2 and 0.7 and 1.2 for Boring B-1, respectively, to account for sampler type and approximate hammer energy. The blow counts used for this conversion were the last two blow counts. The converted SPT N-values are presented on the boring logs.

Upon completion of drilling, the boreholes were backfilled with cement grout to the ground surface. The soil cuttings generated from the borings were placed in 55-gallon soil drums. The drums were subsequently removed from the site and disposed of at an appropriate landfill facility after environmental testing was completed.

3.3 Laboratory Testing

Geotechnical laboratory tests were performed on selected soil samples obtained from the test borings to assess their engineering properties and physical characteristics. Soil samples were tested by B. Hillebrandt Soils Testing, Inc. of Alamo, California to measure moisture content, dry density, plasticity (Atterberg limits), and gradation. In addition, two corrosivity tests were performed on near-surface soil samples by Project X Corrosion Testing of Murrieta, California. The results of the geotechnical laboratory tests are presented on the boring logs in Appendix A and on figures in Appendix B.

4.0 SUBSURFACE CONDITIONS

The site is underlain by Holocene-age alluvial deposits (Qha), as shown on the attached Regional Geology Map, Figure 3 (Graymer et al., 2006). Alluvial fan deposits generally consist of a mixture of fine-grained and coarse-grained deposits and are deposited by rivers and streams.

The results of our borings and CPTs indicate the alluvium underlying the sie generally consists of about 12 to 14 feet of loose to dense silty sand interbedded with medium stiff to hard silty to sandy clay. This layer is underlain by medium stiff to very stiff clay with variable amounts of silt and sand content to as depth of approximately 45 feet bgs. At the location of CPT-1, which was advanced in the center of the site near the northern edge, the site is underlain by very stiff clay and silty clay interbedded with dense sand and silty sand below a depth of 45 feet to the maximum depth explored of 101 feet bgs.

The depth to groundwater was estimated at 20.3 feet bgs using a pore pressure dissipation test taken at 39 feet in CPT-2. Groundwater was measured with a tape measure at the completion of CPT-1 at a depth of 14 feet bgs and was encountered at depths of about 11 and 19 feet bgs in Borings B-1 and B-2, respectively prior to grouting the boreholes. The groundwater level measured in borings may not reflect stabilized groundwater levels due to the relatively short period in which the boreholes were open.

To estimate the highest potential groundwater level at the site, we reviewed information on the State of California Water Resources Control Board GeoTracker website (https://geotracker.waterboards.ca.gov/). From the GeoTracker website, we obtained information from monitoring wells installed for a former Shell service station located at 375 North First Street, about 250 feet southwest of the site. Summary of groundwater level measurements presented in the document titled *Closure Request, Former Shell Service Station, 375 North First Street at Bassett Street, San Jose, California* prepared by Pacific Environmental Group, Inc. indicate the groundwater level was measured between May 1993 to March 1996. Measured groundwater levels ranged from depths of 10.4 to 19.8 feet bgs.

The groundwater level at the site is expected to fluctuate several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall. Based on our review of available historic groundwater information within the site vicinity, we estimate the historic high groundwater level at the site is about 10 feet bgs. Accordingly, we recommend a groundwater depth of 10 feet be used for design.

5.0 SEISMIC CONSIDERATIONS

The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity and Faulting

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

The major active faults in the area are the Hayward, Calaveras, and San Andreas faults. These and other faults in the region are shown on Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[1] [Petersen et al. (2014) & Thompson et al. (2016)] are summarized in Table 1. These references are based on the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013).

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

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Characteristic Moment Magnitude
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	9.9	East	7.58
Hayward (Extension, HE)	9.9	East	6.18
Hayward (South, HS)	10	Northeast	7.00
Monte Vista - Shannon	12	Southwest	7.14
Total Calaveras (CN+CC+CS+CE)	12	East	7.43
Calaveras (Central, CC)	12	East	6.85
Calaveras (North, CN)	14	Northeast	6.86
Total North San Andreas (SAO+SAN+SAP+SAS)	19	Southwest	8.04
North San Andreas (Peninsula, SAP)	19	Southwest	7.38
North San Andreas (Santa Cruz Mts, SAS)	21	Southwest	7.15
Butano	23	Southwest	6.93
Sargent	23	South	6.71
Zayante-Vergeles (2011 CFM)	28	Southwest	7.48
Zayante-Vergeles	29	South	7.00
Las Positas	30	North	6.50
Greenville (South)	35	East	6.64
Greenville (North)	36	East	6.86
San Gregorio (North)	42	West	7.44
Mount Diablo Thrust South	44	North	6.50
Mount Diablo Thrust	44	North	6.67
Calaveras (South, CS)	46	Southeast	6.38
Mount Diablo Thrust North CFM	48	North	6.72
Reliz	50	Southwest	7.44

Damaging earthquakes have occurred along many of these faults in recorded history, as depicted on Figure 4 (USGS, 2021). Notable historic earthquakes which have impacted the Bay Area in recorded history include:

- 1838 San Andreas Earthquake, $M_w = 7.4$ (estimated)
- 1865 San Andreas Earthquake, $M_w = 6.5$ (estimated)
- 1868 Hayward Earthquake, $M_w = 7.0$ (estimated)

- 1906 Great San Francisco Earthquake (San Andreas Fault), M_w = 7.9 (estimated)
- 1989 Loma Prieta Earthquake (San Andreas Fault), M_w = 6.9
- 2014 West Napa Earthquake, M_w = 6.0

As a part of the UCERF3 project, researchers estimated that the probability of at least one $M_w \geq 6.7$ earthquake occurring in the greater San Francisco Bay Area during a 30-year period (starting in 2014) is 72 percent. The highest probabilities are assigned to sections of the Hayward (South), Calaveras (Central), and San Andreas (Santa Cruz Mountains) faults. The respective probabilities are approximately 25, 21, and 17 percent.

5.2 Geologic Hazards

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] We used the results of our CPTs and borings to evaluate the potential of these phenomena occurring at the project site.

5.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the Hayward, Monte-Vista Shannon, and Calaveras faults, although ground shaking from future earthquakes on other faults will also be felt at the site. The intensity of earthquake ground motion at the site will depend upon the characteristics of the generating fault, distance to the earthquake epicenter, and magnitude and duration of the earthquake. We judge that strong to very strong ground shaking could occur at the site during a large earthquake on one of the nearby faults.

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.

5.2.2 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.

5.2.3 Liquefaction and Liquefaction-Induced Settlement

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

As shown on Figure 5, the site is within a liquefaction hazard zone, defined by the map titled *State of California, Seismic Hazard Zones, San Jose West Quadrangle, Official Map*, prepared by the California Geological Survey (CGS), dated February 7, 2002. CGS has provided recommendations for procedures and report content for site investigations performed within seismic hazard zones in Special Publication 117 (SP-117), titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California*, dated September 11, 2008. SP-117 recommends that subsurface investigations in mapped liquefaction hazard zones be performed using rotary-wash borings and/or cone penetration tests (CPTs). We evaluated liquefaction potential at the site using the data collected in our CPTs.

Our liquefaction-triggering analyses using CPT data were performed using the methodology proposed by Boulanger and Idriss (2014). Calculated settlements were then modified using the methodology proposed by Çetin et al. (2009) to account for the depth of the liquefiable layers. These methods are used to estimate a factor of safety against liquefaction triggering by taking the ratio of soil strength (resistance of the soil to cyclic shaking) to the seismic demand that can be expected from a design-level seismic event. Specifically, the following two terms are used in the liquefaction-triggering analyses:

- Cyclic Resistance Ratio (CRR), which quantifies the soil's resistance to cyclic shaking; a function of soil depth, depth of groundwater, earthquake magnitude, and overall soil behavior
- Cyclic Stress Ratio (CSR), which quantifies the stresses that may develop during cyclic shaking.

The factor of safety (FS) against liquefaction triggering can be expressed as the ratio of CRR over CSR. For our analyses, if the FS for a soil layer is less than 1.3, it is considered possible that the soil layer may liquefy during a large seismic event. Therefore, for our calculations of estimated liquefaction-induced settlement, we assumed layers with an FS equal to or greater than 1.3 would not experience liquefaction-induced settlement.

CRR calculations are based on CPT tip resistance. The CPT tip pressures were normalized/corrected for overburden pressure, fines content, and thin layers, where appropriate. The CPT method also utilizes the soil behavior type index (I_c) and the exponential factor "n" applied to the Normalized Cone Resistance "q" to evaluate the cohesive nature of the soil. All of these are included in our analyses.

The CSR is obtained using the equations presented Boulanger and Idriss (2014) publication and is based on the density of the soil, the depth to design groundwater, the estimated peak horizontal acceleration at the ground surface (a_{max}), and a stress reduction coefficient (r_d).

The primary design parameters used in our liquefaction triggering calculations are partially based on the recommended values presented in the CGS Seismic Hazard Zone report for the San Jose West Quadrangle referenced above, and the seismic design parameters for Site Class D from the provisions in the 2022 CBC and are summarized below in Table 2.

TABLE 2
Values Used in Liquefaction Evaluation

Parameter	Value
Depth to groundwater (Historic depth to high groundwater)	10 feet below ground surface
Peak Ground Acceleration (PGA)	0.62g*
Predominant Earthquake Moment Magnitude (M_w)	7.58
Factor of Safety for Liquefaction Triggering	1.3
CPT conversion factor for tip resistance to SPT N-value	4 to 5 (depending on silt content)

* Value obtained from 2022 CBC and corresponds to the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M)

Liquefaction susceptibility was assessed using the software CLiq v3.5.2.10 (GeoLogismiki, 2022). CLiq uses measured field CPT data and assesses liquefaction susceptibility and post-earthquake vertical settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). We also used the relationship proposed by Zhang, Robertson, and Brachman (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement, a relationship that is an extension of the work by Ishihara and Yoshimine (1992). In our analyses, soil that has a significant amount of plastic fines, which is defined as an I_c greater than 2.6, was considered too cohesive to liquefy. Soil with a corrected cone tip resistance q_{c1N} greater than 160 tons per square foot (tsf) was considered too dense to liquefy. In addition, we assumed that soil below a depth of 50 feet was too deep to contribute to liquefaction-induced settlement. Because the predominant earthquake is a moment magnitude 7.58, the CRR has been scaled to a moment magnitude of 7.5 using magnitude scaling factors developed by Boulanger and Idriss (2014).

The results of our liquefaction analyses, which are presented in Appendix C, indicate there is the potential for liquefaction in about 1-foot thick layers of granular soil (primarily sand, silty sand, and sandy silt) between depths of about 10 to 13 feet and below a depth of about 46 feet bgs that are susceptible to liquefaction during a major earthquake. The layer thicknesses are presented below in Table 3. We estimate that total and differential settlements associated with liquefaction at the site during an MCE event generating a PGA_M of 0.62g will be less than 3/4 inch and 1/2 inch across a horizontal distance of 30 feet, respectively. A summary of our CPT data, as well as other pertinent parameters regarding liquefaction triggering and associated settlement, are presented below in Table 3. Note that the estimated liquefaction-induced ground settlements are applicable to the free-field and do not necessarily reflect actual building settlements that may result from liquefaction.

TABLE 3
Summary of Liquefaction Potential Analyses Results from CPT Data

CPT Number	Approx. Depth (feet)	Layer Thickness (feet)	Avg. $(q_{c1N})_{cs}$ (tsf)	$(N_1)_{60\text{-}cs}$	Avg. I_C	Avg. CSR_{EQ}	Avg. $CRR_{7.5}$	Avg. Factor of Safety	Avg. Volumetric Strain, ε_v (%)	Estimated Vertical Settlement (in)
CPT-1	10.01	1.31	94	23	2.38	0.41	0.13	0.32	2.03	0.32
	12.80	1.15	89	22	2.50	0.45	0.13	0.27	1.98	0.28
	46.75	0.33	115	29	2.25	0.57	0.21	0.38	0.47	0.02
	48.56	0.33	120	30	2.47	0.56	0.18	0.31	0.35	0.02
Total Estimated Settlement in CPT-1										**0.64**
CPT-2	10.66	1.15	103	26	2.49	0.42	0.14	0.34	1.85	0.25
Total Estimated Settlement in CPT-2										**0.25**

5.2.4 Liquefaction-Induced Ground Failure

The potential for liquefaction-induced ground failure depends on the thickness of the liquefiable soil layer relative to the thickness of the overlying non-liquefiable material. Ishihara (1985) presented empirical relationship that provides criteria that can be used to evaluate whether liquefaction-induced ground failure, such as sand boils, would be expected to occur under a given level of shaking for a liquefiable layer of given thickness overlain by a resistant, or protective, surficial layer.

We evaluated the potential for surface manifestation of liquefaction using an empirical relationship developed by Ishihara (1985). On the basis of our evaluation, we conclude that the potentially liquefiable layers are sufficiently thin, such that the potential for surface manifestations from liquefaction, such as sand boils and loss of bearing capacity, is low for the at-grade improvements. However, the parking pits are planned to extend to a depth of about 8-

1/2 feet bgs. We anticipate excavation for the parking lift pits will be 9 to 10 feet in depth, depending on the mat foundation thickness. There is potential for the parking pit lift foundations to experience larger settlements than the surrounding grades due to soil-structure interaction effects. These effects include settlements due to “ratcheting-type” deformations, where the foundations settle in increments during each earthquake cycle as the underlying liquefied soil deforms when the imposed bearing pressure approaches the ultimate bearing capacity of the liquefied soil. We judge that if the subgrade preparation extends to a depth of 11 feet bgs, the resulting potential for loss of bearing pressure will be reduced.

Lateral spreading is a phenomenon in which a surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. The surficial blocks are transported downslope or in the direction of a free face, such as a channel, by earthquake and gravitational forces. Lateral spreading is generally the most pervasive and damaging type of liquefaction-induced ground failure generated by earthquakes. Although thin, potentially liquefiable layers were encountered, the layers do not appear to be continuous, and the topography of the site and surrounding area is relatively flat. Therefore, we conclude that the risk of lateral spreading is low.

5.2.5 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. We evaluated the cyclic densification potential of soil encountered at the site using data collected in CPTs using the methodology by Yee, Stewart, and Duku (2012).

Using the earthquake parameters discussed in Section 5.2.3 and the data from the CPTs, we estimate that ground surface settlement will be less than 1/4 inch in the upper 6-1/2 to 9 feet of the site as a result of strong shaking during an MCE event on a nearby fault. Excavation of the parking pit lifts will remove this soil resulting in no cyclic densification settlement in the parking pit lift areas. Because of the variability in the density of the medium dense sand across the site, we conclude that differential settlements equivalent to the total settlements may occur over short distances at the site.

6.0 DISCUSSION AND CONCLUSIONS

Based on the results of our engineering analyses using the data from our borings and CPTs, we conclude there are no geotechnical or geological issues that would preclude the development of the site as proposed. The primary geotechnical issues affecting the proposed development are providing adequate foundation support for the proposed building and the relatively shallow groundwater relative to the proposed parking pit excavations. This and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

6.1 Design Groundwater Level

Based on the available groundwater data discussed in Section 4.0, we recommend using a design high groundwater level of 10 feet below the existing ground surface for the proposed project.

6.2 Foundation Support and Settlement

The soil encountered at the foundation levels has moderate strength and moderate compressibility. If the proposed building is supported on a conventional footing foundation system, settlement will occur due to compression of the underlying clay under static foundation loads. Considering that the proposed bottom of parking pit foundations may be at or about a foot above the design high groundwater level, the parking pit foundations will need to be underlain by waterproofing. A mat foundation system generally simplifies construction and the detailing of the waterproofing system. Based on our experience, we judge the anticipated differential settlements due to the static loading exceeding the typical tolerance of a conventional spread footing foundation system. Therefore, we conclude that the proposed building may be supported on a mat foundation, provided the static and seismically induced settlements are acceptable from a structural standpoint.

Based on preliminary dead-plus-sustained live loads of about 800 to 900 pounds per square foot (psf), we estimate the total static settlement of the proposed building will be up to about 1 to 1-1/2 inches. The 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 that the mats can be designed to limit differential settlements to about 3/4 inch in 30 feet. As discussed in Section 5.2.3, we estimate liquefaction-induced total and differential settlements of the building would be less than 3/4 inch and less than 1/2 inch over a horizontal distance of 30 feet, respectively. In addition, cyclic densification settlement would be less than 1/4 inch for at-grade improvements.

6.3 Soil Corrosivity

Laboratory testing was performed by Project X Corrosion Engineering to evaluate the corrosivity of near-surface soil samples obtained from Boring B-1 at depths of 2 and 2-1/2 feet bgs and Boring B-2 at depths of 10-1/4 and 10-3/4 feet bgs. The results of this corrosivity testing are presented in Appendix B.

Many factors can affect the corrosion potential of soil, including, but not limited to, resistivity, pH, and chloride and sulfate concentrations. The minimum soil resistivity measurements from Boring B-1 are 1,675 ohm-cm, and the measurements from Boring B-2 range from 938 to 3,685 ohm-cm. Based on these minimum soil resistivity measurements, we conclude that the near-surface soil is “highly” corrosive to buried metal and that the soil at a depth of about 10-1/2 feet bgs is “corrosive” to “extremely” corrosive to buried metal (Roberge, 2018). Accordingly, all buried iron, steel, cast iron, galvanized steel, and dielectric-coated steel or iron should be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.

The results of the pH tests (7.2 to 9.8) indicate that the soil tested is “negligibly corrosive” to buried metallic and concrete structures. However, alkaline soil with pH>8.5 at a depth of about 10-1/2 feet bgs can cause accelerated corrosion of copper and aluminum alloys, as well as zinc coatings if pH>10. The chloride ion concentrations (109.4 to 133.5 mg/kg) indicate that the chlorides in the soil tested are “mildly corrosive” to buried metallic structures and reinforcing steel in concrete structures below ground. The results also indicate that the sulfate ion concentrations (157.7 to 456.0 mg/kg) are sufficiently low such that sulfates do not pose a threat to buried concrete and mortars.

6.4 Excavation and Temporary Shoring

The proposed building will include parking lift pits on the ground floor. Installation of the parking lifts will involve excavating lift pits. We anticipate excavation for the parking lift pits, and foundations will be on the order of about 10 feet in depth, depending on the mat foundation thickness.

Excavations that will be entered by workers should be sloped or shored in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes. The shoring designer should be responsible for the shoring design.

Where space permits, the sides of the temporary excavation can be sloped. Where space does not permit sloping of the excavation perimeter, a shoring system will be required to support the sides of the proposed excavation. We judge that a cantilevered soldier pile and lagging shoring system is appropriate for foundation support of excavations that are less than 12 feet in depth.

6.5 Construction Monitoring

Because the project will involve an excavation that may impact the property to the northwest, the contractor should establish survey points on the shoring and on the ground surface at critical locations behind the shoring prior to the start of excavation. Monitoring points should be established on all structures within a horizontal distance equal to twice the proposed excavation depth. These survey points should be used to monitor the vertical and horizontal movements of the shoring and any improvements behind the shoring during construction.

During excavation, the shoring system may deform laterally, which could cause the ground surface adjacent to the shoring wall to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. A properly designed and constructed tied-back shoring system should be capable of limiting horizontal and vertical ground surface deflections to less than 3/4 inches during construction.

6.6 Construction Considerations

The soil to be excavated to construct building foundations, install utilities, and parking and elevator pit(s) is expected to consist of clay and silt with varying sand content and silty sand that can be reused as engineered fill. Removal of existing foundations will require equipment capable of breaking up reinforced concrete. Existing building elements should be removed in their entirety within the proposed building footprint.

If site grading is performed during the rainy season, the near-surface clay, silt, and silty sand will likely be wet and will have to be dried before compaction can be achieved. Heavy rubber-tired equipment could cause excessive deflection (pumping) of the wet clay and, therefore, should be avoided. If construction occurs during the winter, it may be necessary to winterize the site by cement treating the upper 12 inches of the building pad.

Where there are buildings adjacent to the site, heavy equipment should not be used within 10 horizontal feet from adjacent shallow foundations and basement walls (if any). A jumping jack or hand-operated vibratory plate compactor should be used for compacting fill within this zone.

We anticipate that the excavation for the parking pit foundations may extend to the design groundwater level. The actual groundwater level at the time of construction is uncertain, and the groundwater level may fluctuate depending on seasonal rainfall. If construction is performed during the wet season, the water level may be close to the design high groundwater level. If groundwater is encountered in the parking pit foundation excavation, we anticipate that a passive dewatering system can be performed where necessary. A passive system may consist of trench drains and sump pumps. The contractor is responsible for the need for, selection, and design of a temporary dewatering system.

7.0 RECOMMENDATIONS

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

7.1 Site Preparation and Grading

Site demolition should include the removal of existing pavements, foundations, and underground utilities, if any, where the new structure is proposed. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside the proposed building footprint and will not interfere with the proposed construction, they may be abandoned in-place provided the lines are filled with lean concrete or cement grout to the property line. Voids resulting from demolition activities should be properly backfilled with compacted fill following the recommendations provided later in this section. Removed asphalt concrete should be taken to an asphalt recycling facility.

Any vegetation and organic topsoil (if present) should be stripped in areas to receive improvements (i.e., building or flatwork). Tree roots with a diameter greater than 1/2 inch within 3 feet of subgrade should be removed. If zones of existing undocumented fill or weak/unstable soil are encountered during site grading, the fill should be over-excavated under the observation of our field engineer and replaced as a properly compacted fill.

f construction will occur during the rainy season, treatment of the upper 12 inches of the building pad with Portland cement may be needed to prevent softening of the pad due to exposure to rain. We should determine, in consultation with the project team, whether cement treatment of the building pad is warranted once the construction schedule is known.

7.1.1 Fill Materials and Compaction Criteria

Fill may consist of on-site soil that is free of organic matter and rocks or lumps larger than 3 inches in greatest dimension. If it is necessary to import soil (select fill), the material should be free of organic matter, contains no rocks or lumps larger than 3 inches in greatest dimension, have a liquid limit of less than 40 and a plasticity index lower than 12, and be approved by the

Geotechnical Engineer. Samples of proposed imported fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed imported material.

Where placement of fill or backfill is required, the fill should be placed in lifts not exceeding 8 inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 relative compaction.[5] Fill should be compacted to at least 95 percent relative compaction where the fill is: (1) greater than 5 feet in thickness, or (2) consists of clean sand or gravel, defined as soil with less than 5 percent fines by weight.

7.1.2 Subgrade Preparation

In the proposed at-grade building pad and areas that will receive exterior concrete flatwork, the soil subgrade exposed following stripping and clearing should be scarified to a depth of at least 8 inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction.

There is the potential for liquefaction-induced loss of bearing capacity in the parking pit lift foundations. The parking lift pit mat subgrade should be over-excavated to a depth of about 10 feet bgs, and the upper one foot of the exposed subgrade should be scarified, moisture-conditioned, and recompacted. The over-excavation may be backfilled with engineered fill. All fill should be placed in horizontal lifts not exceeding 8 inches in uncompacted thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction.

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

7.1.3 Exterior Flatwork Subgrade Preparation

We recommend that a minimum of 4 inches of Class 2 aggregate base (AB) be placed below exterior concrete flatwork, such as patios and sidewalks. The subgrade and Class 2 AB should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction. The prepared subgrade should be kept moist until it is covered with the Class 2 AB.

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 4 inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of 6 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 according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 5 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. Poor compaction may cause excessive settlements, resulting in damage to the overlying improvements.

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

7.1.5 Drainage and Landscaping

Positive surface drainage should be provided around the new building to direct surface water away from the foundations. Grades around the building should be determined by the project Civil Engineer and conform to the requirements of the 2022 CBC to mitigate the potential for stormwater to accumulate around and below foundations. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. The use of water-intensive landscaping around the perimeter of the building should be avoided.

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

7.2 Mat Foundation

The proposed building may be supported on a well-reinforced concrete mat bearing on firm native soil and/or engineered fill. For preliminary mat design, we recommend using a modulus of subgrade reaction of 20 pounds per cubic inch (pci) for dead-plus-live load. This value has been reduced to account for the size of the mat/equivalent footings (therefore, this is not k_{v1} for 1-foot-square plate) and may be increased by one-third for total load conditions. Once the Structural Engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of subgrade reaction, if appropriate.

Considering the large area of the mat, we expect the average bearing stress under the mat to be relatively low; however, concentrated stresses will occur at column locations and the edges of the mat. The maximum bearing pressure beneath the mat should not exceed 3,500 psf under dead-plus-live-load conditions and 4,500 psf under total load conditions. The allowable bearing pressures for dead-plus-live and total loads include factors of safety of at least 2.0 and 1.5, respectively.

Lateral forces can be resisted by a combination of friction along the base of the mat and passive pressure against the vertical faces of the mat foundation. To compute lateral resistance, we recommend using an equivalent fluid weight (triangular distribution) of 290 pounds per cubic foot (pcf) for sustained loads. The upper foot of soil should be ignored for passive resistance unless it is confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30 where the mat is in contact with soil. Where a vapor retarder is placed beneath the mat, a base friction coefficient of 0.20 should be used. In parking pits, the allowable friction factor will depend on the type of waterproofing used at the base of the mat. For bentonite-based waterproofing 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 foundation subgrade should be prepared following the recommendations presented in Section 7.1.2. We recommend that the mat be founded below an imaginary plane extending up at an inclination of 1.5:1 (horizontal to vertical) from the base of any vault, utility trench, bioswale/stormwater treatment area, etc. If the design bottom-of-mat elevation is above this plane, the edge of the mat can either be deepened, or it can be over-excavated below the zone-of-influence line and replaced with controlled low-strength material (CLSM) with a 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).

The mat subgrade in the parking pits may 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 in the parking pits should be performed with tracked equipment to minimize heavy concentrated loads that may disturb the wet soil. The mat subgrade should be free of standing water, debris, and disturbed materials before placing concrete. If the mat will be constructed during the rainy season, we recommend a 3-inch-thick mud slab be placed on the subgrade to protect it from saturation and softening from standing water following a rain event. We should check the mat subgrade prior to placement of the vapor retarder, concrete, or the mud slab, if used.

7.3 Water Vapor Retarder

If water vapor moving through the mat foundation for the at-grade portion of the structure is considered detrimental, we recommend installing a water vapor retarder beneath the mat. As a minimum, we recommend that a vapor retarder be placed beneath the mat in all living spaces, storage areas, and any areas that will receive a floor covering. The vapor retarder should meet the requirements for Class A vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by 6 inches, taping seams, and sealing penetrations in the vapor retarder.

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

7.4 Permanent Retaining Walls

Permanent retaining walls should be designed to resist lateral earth pressure imposed by the retained soil, as well as a surcharge pressure from nearby foundations and vehicles, where appropriate. Where permanent retaining walls will be restrained from movement at the top or sides (e.g., retaining walls with 90-degree angle turn, such as elevator pit walls), they should be designed for at-rest conditions. We recommend that restrained walls be designed using an at-rest equivalent fluid weight of 55 pcf. Retaining walls that are unrestrained at the top should be designed for active conditions using an active equivalent fluid weight of 35 pcf. To evaluate the restrained and unrestrained walls for seismic loading, we recommend using active equivalent fluid weight plus a seismic increment, as shown in Table 5.

TABLE 5
Lateral Earth Pressures for Retaining Wall Design

Wall Restraint	Static Condition	Seismic Condition
Unrestrained	35 pcf	35 pcf + 13 pcf
Restrained	55 pcf	35 pcf + 29 pcf

Where there will be vehicular traffic behind the top of a permanent wall within a horizontal distance equal to 1.5 times the height of the wall, the wall should be designed for a vehicular surcharge of 50 psf at the upper 10 feet. Where existing foundations are supported above a “zone-of-influence” line extending up from a permanent wall at an inclination of 1.5:1 (horizontal: vertical), the wall should be designed for surcharge pressure. The magnitude of the surcharge pressure will need to be evaluated on a case-by-case basis.

The design pressures recommended above are based on fully drained walls. Although below-grade retaining walls will be above the groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, broken water lines, etc. If the earth pressures presented above are used to design the walls, they will need to incorporate a drainage system (i.e., a back drain) behind the walls. One acceptable method for back-draining a retaining wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend to a perforated PVC collector pipe surrounded by at least four inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140NC or equivalent). A proprietary, prefabricated collector drain system, such as Tremdrain Total Drain or Hydroduct Coil (or equivalent), designed to work in conjunction with the drainage panel may be used in lieu of the perforated pipe surrounded by gravel described above. The pipe should be connected to a suitable discharge point; a sump and pump system may be required to drain the collector pipes for below-grade retaining walls, such as elevator pit or parking pit lift walls.

To protect against moisture migration, below-grade retaining walls, including elevator and parking pit lift walls, should be waterproofed, and water stops should be placed at all construction joints. If backfill is required behind retaining walls, the walls should be braced or hand compaction equipment used to prevent unacceptable surcharges on walls (as determined by the Structural Engineer).

7.5 Temporary Cut Slopes and Shoring

Excavations that will be entered by workers should be shored or sloped in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes. The shoring designer should be responsible for the shoring design.

Where space permits, the sides of the temporary excavation can be sloped. We recommend that temporary slopes not exceed an inclination of 1.5:1 (horizontal to vertical) in sandy soil (OSHA Type C Soil). Where space does not permit sloping of the excavation perimeter, a shoring system will be required to support the sides of the proposed excavation. We judge that a cantilevered soldier pile and lagging shoring system is appropriate for the support of less than 12 feet deep excavations.

A Structural or Civil Engineer knowledgeable in this type of construction should be retained to design the shoring. The shoring designer should design the shoring system for lateral deformation of less than 1 inch (1/2 inch if neighboring structures are within a horizontal distance equal to 1.5 times the height of the shoring) at any location on the shoring. We should review the final shoring plans and calculations to check that they are consistent with the recommendations presented in this report.

7.5.1 Cantilevered Soldier Pile and Timber Lagging Shoring System

Recommended lateral pressures for the design of cantilever soldier piles and lagging shoring are presented in Figure 6. A cantilevered soldier pile and lagging system should be designed using an active equivalent fluid weight of 35 pcf, provided no building foundations within a horizontal distance equal to two times the retained soil height. If foundations are within that horizontal distance, the shoring should be designed using an at-rest pressure of 55 pcf plus the surcharge load imposed by the building foundation.

Where traffic loads are expected within 10 feet of the shoring walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall. Shoring should be designed for surcharge loads where there will be construction equipment and/or stockpiled soil within a horizontal distance of 1.5 times the height of the shoring from the edge of the excavation, and/or there are nearby shallow foundations that will not be underpinned and are located above an imaginary line that extends at an inclination of 1.5:1 (horizontal: vertical), projected upward from the bottom edge of the proposed excavation. We can provide recommendations for surcharge pressures once surcharge loads are known.

Passive resistance at the toe of the soldier piles should be computed using an equivalent fluid weight of 290 and 150 pcf above and below the design groundwater table, respectively. The upper foot of soil should be ignored when computing passive resistance. Passive pressure can be assumed to act over an area of 2.4 times the soldier pile width assuming the toe of the soldier pile is filled with structural concrete. If lean concrete is placed in the soldier pile shaft, the passive pressure can be assumed to act over two times the pile width. These passive pressure values include a factor of safety of at least 1.5.

Soldier piles should be placed in pre-drilled holes backfilled with concrete. The shoring contractor should be prepared to use casing or drilling slurry to reduce the caving of holes, where necessary. Installing soldier piles using a vibratory method is not recommended within 25 feet of existing structures.

We recommend that the bottom of the excavation not extend more than 4 feet below the last row of lagging and not extend more than 1 foot when excavating in soil susceptible to caving. If voids are created behind lagging boards due to localized caving or overcutting, they should be filled with cement slurry or hand-packed soil prior to proceeding with excavation.

7.6 Seismic Design

As discussed in Section 5.2.3, the site is underlain by thin zones of potentially liquefiable soil. Although the 2022 CBC call for a Site Class F designation for sites underlain by potentially liquefiable soil, we conclude a Site Class D designation is more appropriate because the potentially liquefiable layers are relatively thin, and the site will not incur significant nonlinear behavior during strong ground shaking. Based on our field investigation and engineering analysis results, we conclude that a designation of Site Class D (V_{S30} = 750 feet/sec) is appropriate and consistent with the 2022 CBC.

The latitude and longitude of the site are 37.3422° and -121.8942°, respectively. For design per the 2022 CBC, we recommend the following:

- Site Class D (stiff soil, non-default)
- S_S = 1.50g, S_1 = 0.60g

The 2022 CBC is based on the guidelines contained within ASCE 7-16 (Supplement 3 revision), which stipulates that where S_1 is greater than 0.2 times gravity (g) for Site Class D, a ground motion hazard analysis is required unless the long-period spectral design parameters (S_{M1}, S_{D1}) are increased by 50%. Therefore, we recommend the following seismic design parameters, which include the 50% increase as designated by an asterisk:

- F_a = 1.0, F_v = 1.7
- S_{MS} = 1.50g, S_{M1}* = 1.53g
- S_{DS} = 1.00g, S_{D1}* = 1.02g
- Seismic Design Category D for Risk Factors I, II, and III

Depending on the structural design methodology and fundamental period of the proposed building, it may be advantageous to perform a ground motion hazard analysis (the project structural engineer should confirm). We can perform a ground motion hazard analysis upon request.

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

9.0 LIMITATIONS

This geotechnical 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 subsurface conditions do not deviate appreciably from those disclosed in our field investigation.

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

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FIGURES

0 2,400 4,800 Feet

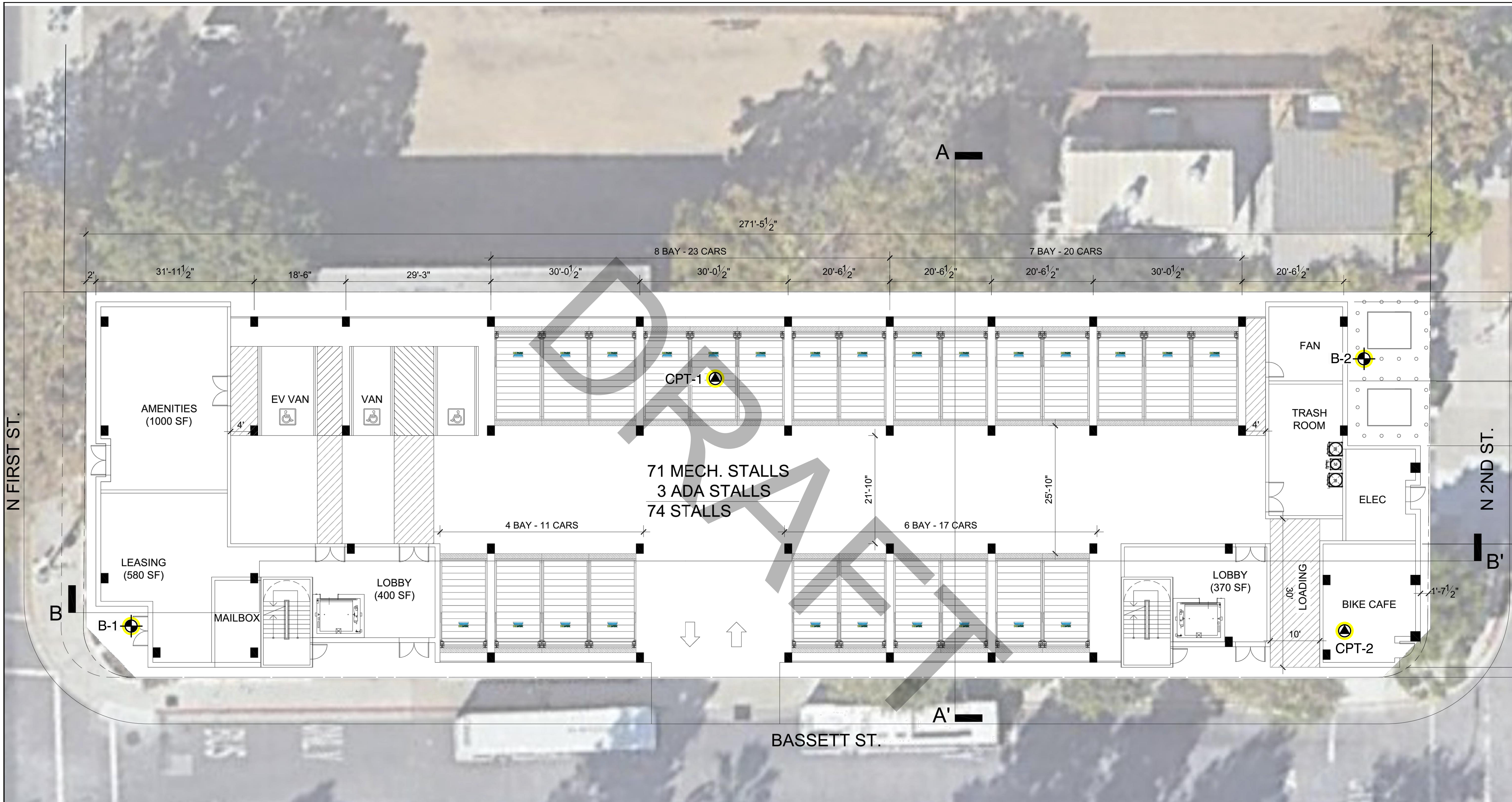
Approximate scale

380 NORTH FIRST STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 02/16/23	Project No. 23-2365	Figure 1



EXPLANATION

CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical, Inc., February 17, 2023

B-1 — Approximate location of boring by Rockridge Geotechnical, Inc., February 13 & 22, 2023

0 — 20 Feet

Approximate scale

Reference: Base map from a drawing titled "Level 1", by Studio T Square, dated September 6, 2022.

380 NORTH FIRST STREET
San Jose, California

SITE PLAN

Date 03/01/23	Project No. 23-2365	Figure 2

ROCKRIDGE GEOTECHNICAL

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

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
Qsl	Hillslope Deposits (Quaternary)
Qpa	Alluvium (Pleistocene)
Tms	Sedimentary rocks (Miocene)
KJs	Great Valley complex sedimentary rocks (Early Cretaceous and (or) Late Jurassic)
Jv	Great Valley complex volcanic rocks (Jurassic)
Jsp	Great Valley complex serpentinite (Jurassic)

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

0 1 2 Miles

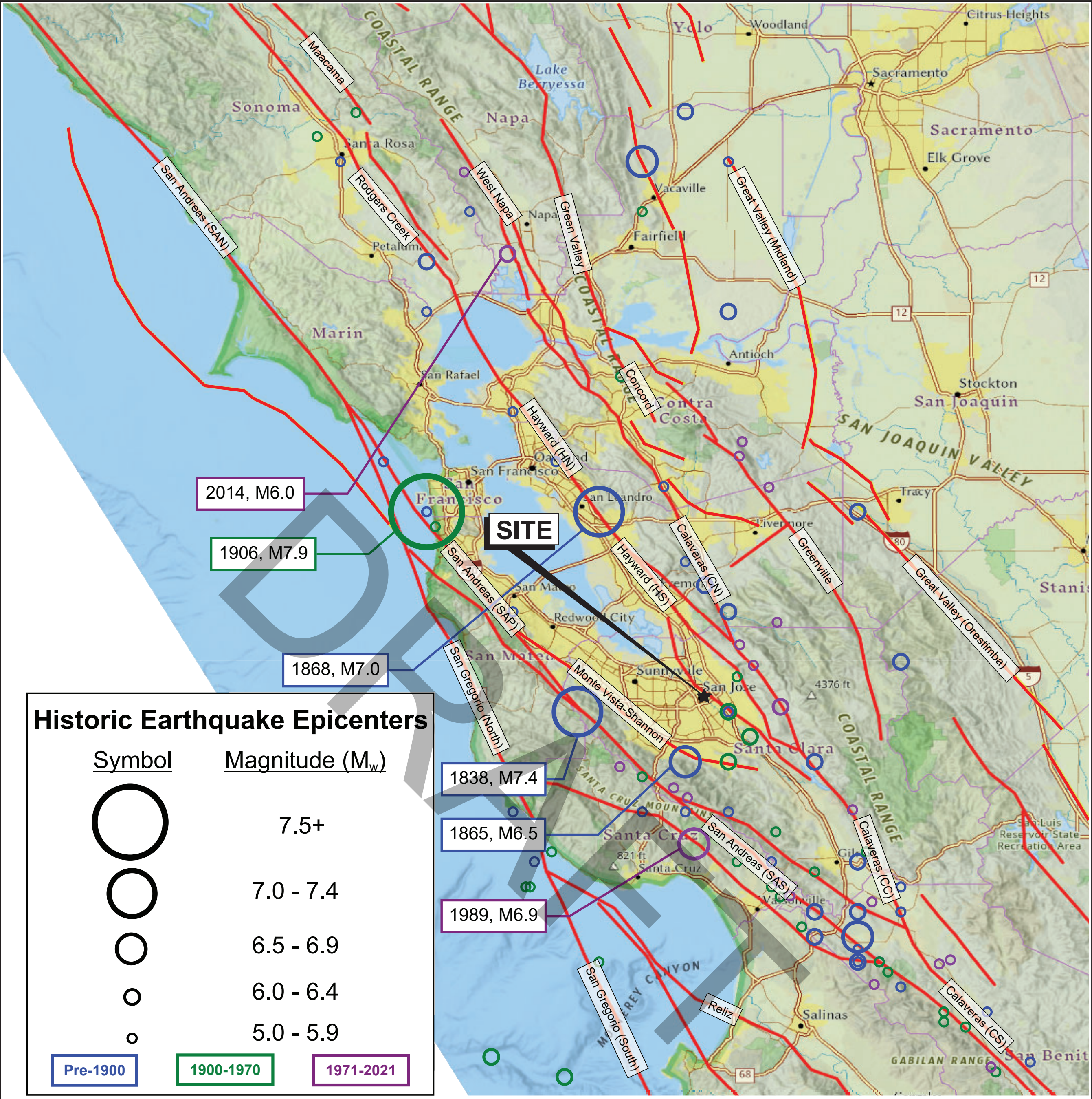
Approximate scale

380 NORTH FIRST STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 03/28/23 | Project No. 23-2365 | Figure 3



Historic Earthquake Epicenters

Symbol	Magnitude (M_w)
○	7.5+
○	7.0 - 7.4
○	6.5 - 6.9
○	6.0 - 6.4
○	5.0 - 5.9

Pre-1900 | 1900-1970 | 1971-2021

EXPLANATION

Faults (National Seismic Hazard Model)

0 10 20 Miles

0 16 32 Kilometers

Approximate scale

380 NORTH FIRST STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Date 02/15/23 | Project No. 23-2365 | Figure 4

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Reference:
Earthquake Zones of Required Investigation
San Jose West Quadrangle
California Geological Survey
Released February 7, 2002

0 2,000 4,000 Feet
Approximate scale

380 NORTH FIRST STREET
San Jose, California

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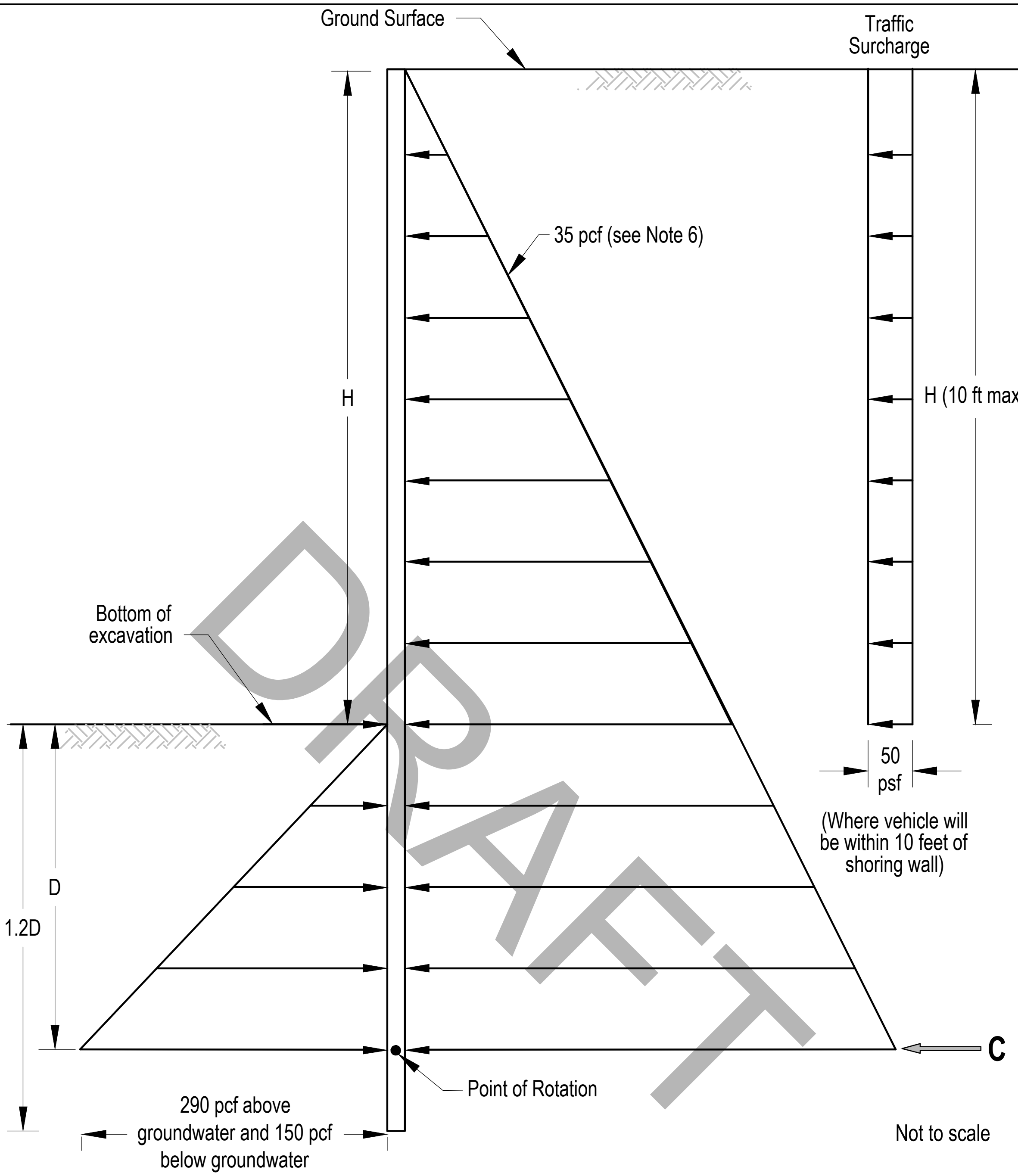
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 02/16/23	Project No. 23-2365	Figure 5



Notes:

1. Simplified pressure diagram is presented above. The net passive pressure on the right side of shoring below the point of rotation is replaced by a concentrated force C.
2. Passive pressures include a factor of safety of 1.5.
3. Passive pressures may be assumed to act over the pile spacing or 2.4 times the pile diameter, whichever is smaller, where the toe of drilled piles are backfilled with structural concrete. If lean concrete is placed in the toe, the passive pressure can be assumed to act over two pile diameters.
4. Active pressure below the excavation should be assumed to act over one pile diameter.
5. Surcharge pressure due to construction equipment within 1.5H of shoring wall, if any, should be added to the above shoring pressure.
6. Where adjacent building is located within 1.5H of the shoring wall, at-rest equivalent fluid weight of 55 pcf should be used and surcharge pressure from building foundations should be added to the above shoring pressures.
7. Calculated embedment depth, D, should be increased by at least 20 percent to obtain the design depth of penetration.
8. pcf = pounds per cubic foot; and psf = pounds per square foot

380 NORTH FIRST STREET
San Jose, California

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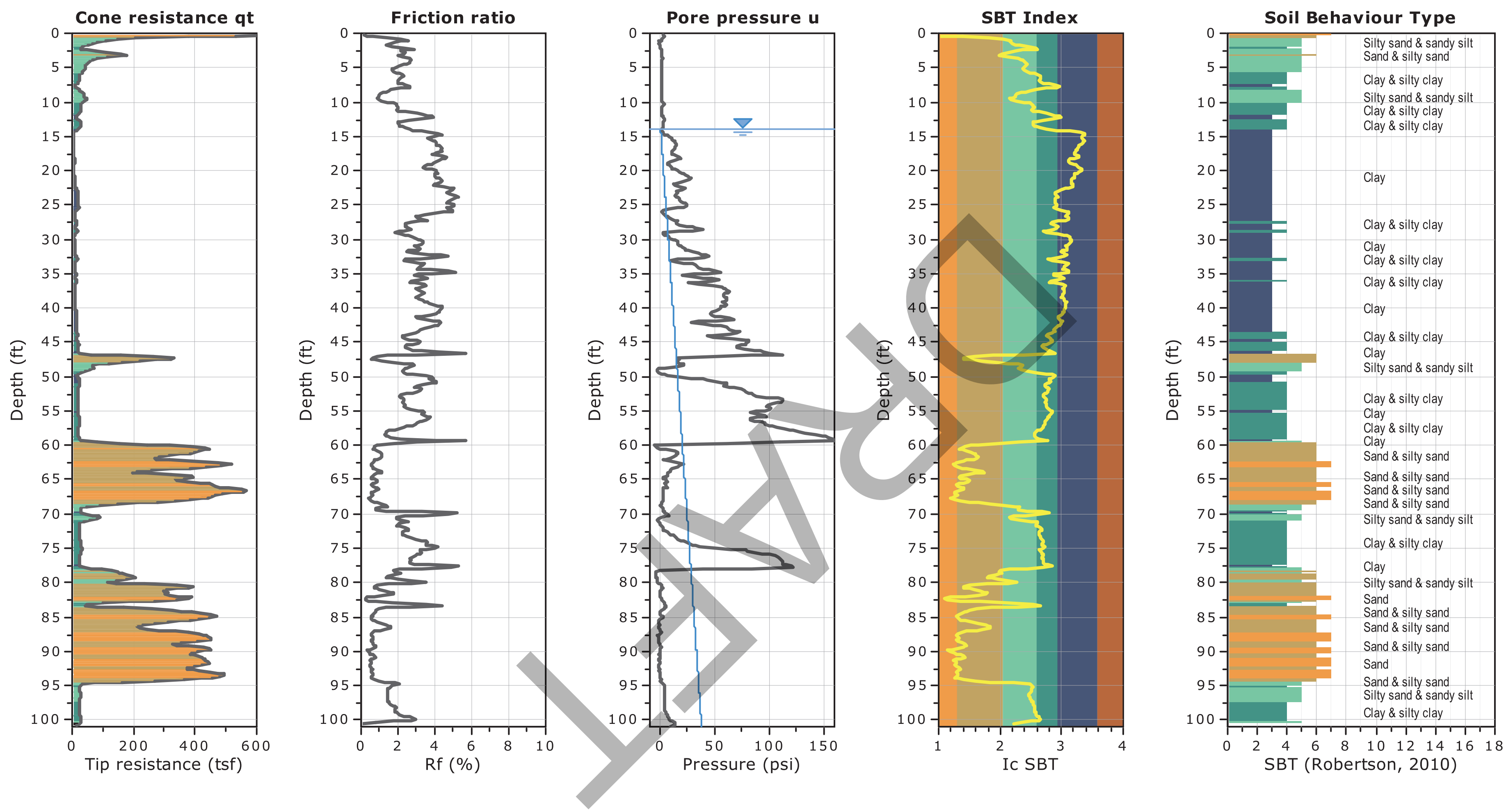
DESIGN PARAMETERS FOR CANTILEVERED SOLDIER-PILE-AND-LAGGING TEMPORARY SHORING SYSTEM

Date 04/12/23	Project No. 23-2365	Figure 6

ROCKRIDGE GEOTECHNICAL

APPENDIX B
Laboratory Test Results

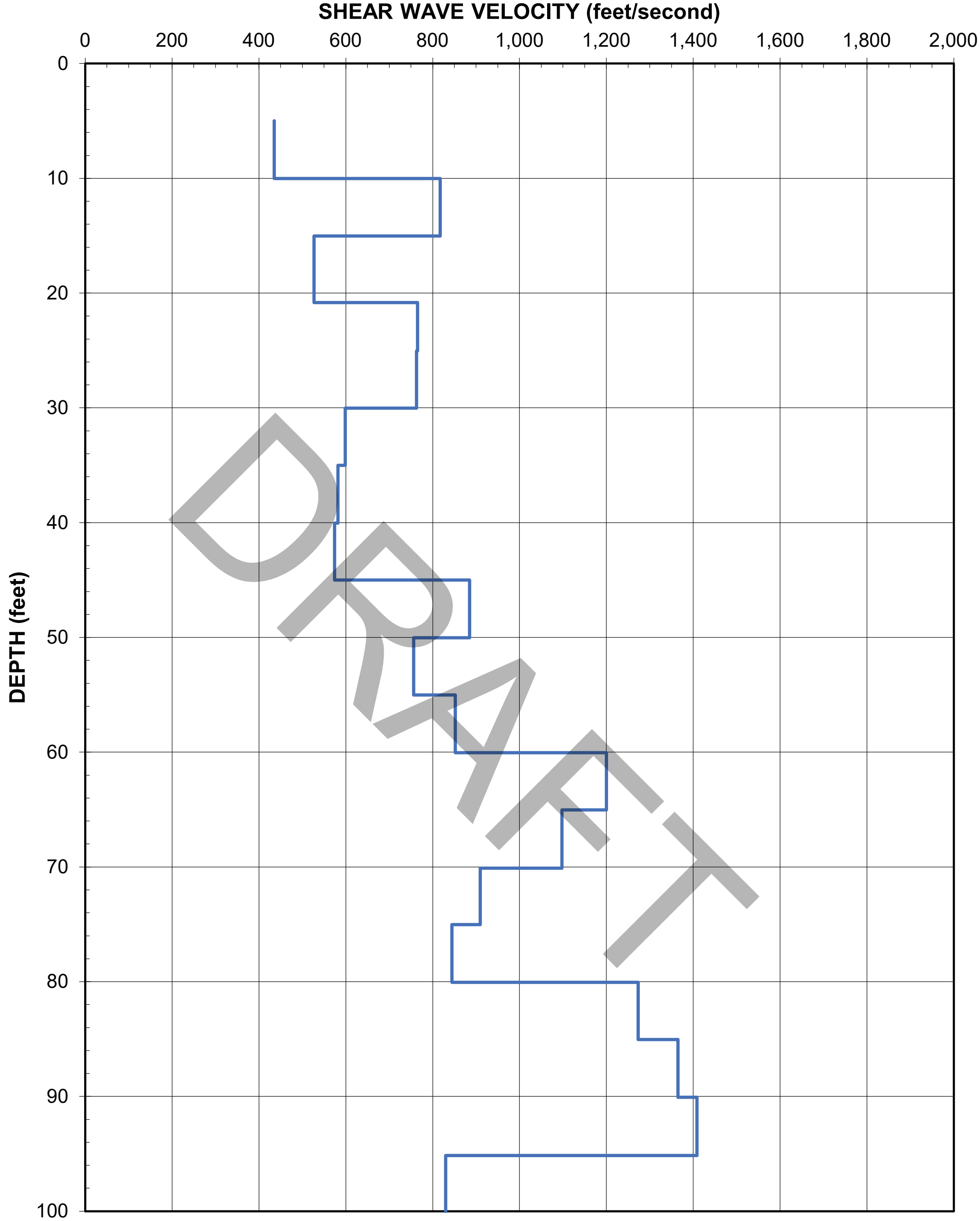


Total Depth: 101.21 ft, Date: Feburary 17, 2023
Depth to Groundwater: 14 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

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

380 NORTH FIRST STREET San Jose, California	**CONE PENETRATION TEST RESULTS** **CPT-1**
ROCKRIDGE GEOTECHNICAL	Date 03/01/23 | Project No. 23-2365 | Figure A-1a

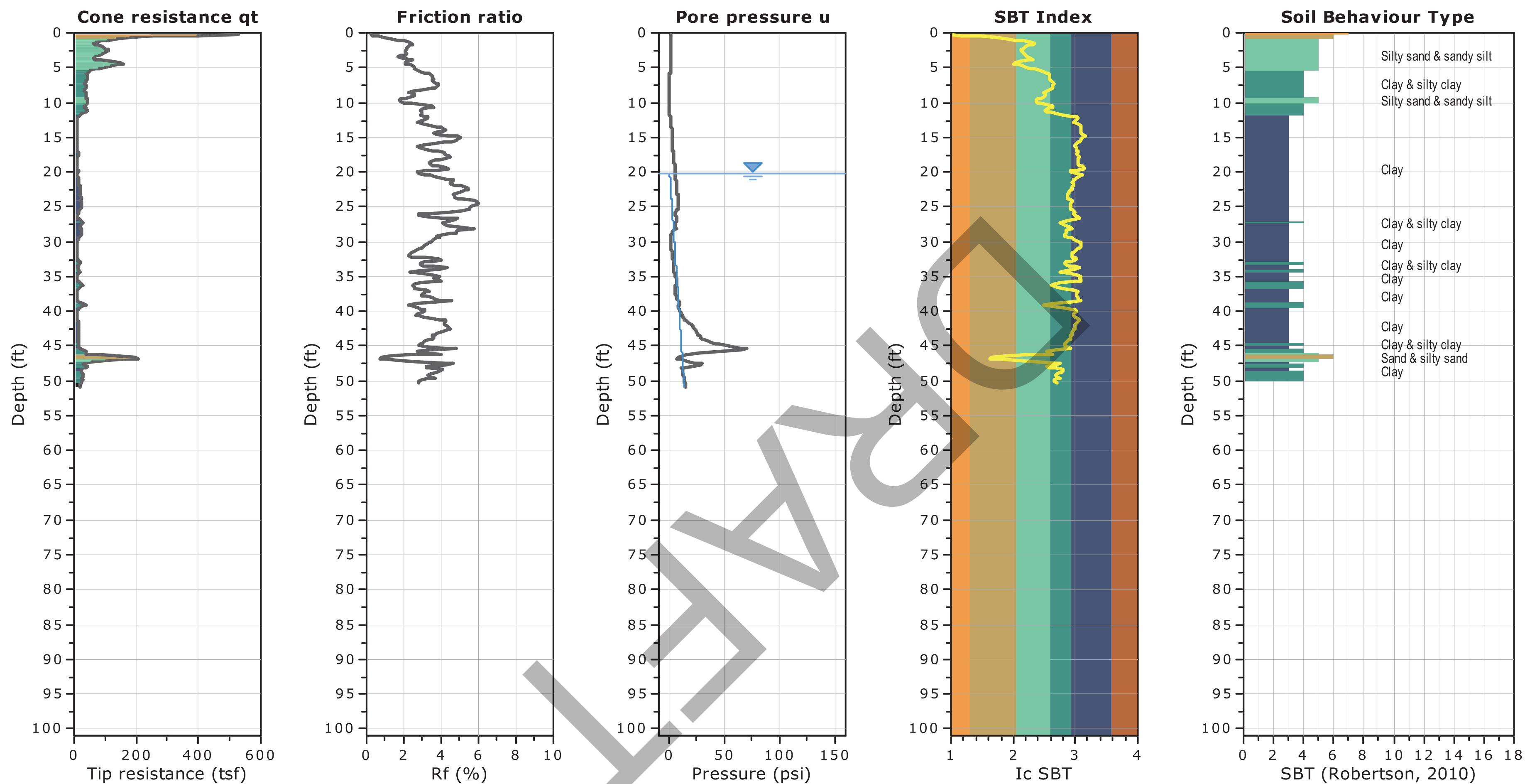


380 NORTH FIRST STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

SHEAR WAVE VELOCITY PROFILE
CPT-1

Date 03/01/23 | Project No. 23-2365 | Figure A-1b



Total Depth: 101.21 ft, Date: Feburary 17, 2023
Estimated Depth to Groundwater: 20.3 feet (based on Pore Pressure Dissipation Test at 39 feet)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

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

380 NORTH FIRST STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 03/10/23	Project No. 23-2365	Figure A-2

PROJECT: **380 NORTH FIRST STREET**
San Jose, California

Log of Boring B-1

Boring location: See Site Plan, Figure 2
Date started: 02/22/2023 | Date finished: 02/22/2023
Drilling method: 4-inch-diameter solid-stem auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Rope & cathead safety hammer
Sampler: Modified California (MC), Standard Penetration Test (SPT)

Logged by: D. Fernandez
Drilled by: Stapleton Engineering & Exploration
Rig: Limited Access Bobcat

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content,%	Dry Density Lbs/Cut Ft
0–1					SANDY CLAY (CL) red, medium stiff, dry to moist, fine sand, trace rootlets						
2–3	MC	5 5 6	8	CL	Soil Corrosivity Test; see Appendix B olive-brown						
4–5	MC	4 5 7	8		gray-brown, moist, increasing fine sand						
6–7	MC	3 4 4	6	CL	CLAY with SAND (CL) brown, medium stiff, moist, fine sand Particle Size Distribution; see Appendix B				85	11.7	96
8–9	SPT	3 3 4	7		SILT with SAND (ML) yellow-brown, medium stiff, moist, fine to medium sand LL = 21, PI = 3; see Appendix B Particle Size Distribution; see Appendix B				74		
9–10	SPT	3 3 4	7	ML							
11					▼ (02/22/2023; 12:55 PM)						
12–13					CLAY (CL) yellow-brown, medium stiff to stiff, wet, trace fine sand						
15–16	MC	4 5 7	8		Particle Size Distribution; see Appendix B				98	31.9	87
19					▽ (02/22/2023; 9:49 AM)						
20				CL							
21–22	MC	4 5 10	11		olive-brown, stiff						
25–26	MC	5 6 6	8		medium stiff to stiff LL = 35, PI = 18; see Appendix B Particle Size Distribution; see Appendix B				94	30.6	91
28–29				CL	SANDY CLAY (CL) gray, stiff, wet, fine sand						
30											

ROCKRIDGE GEOTECHNICAL

Project No.: 23-2365 | Figure: A-3a

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	SPT		4 5 5	10	CL	SANDY CLAY (CL) (continued)						
36	MC		5 6 9	11		olive-gray, fine to medium sand						
38						CLAY (CL) gray, stiff, wet, trace fine sand						
41	SPT		4 5 6	11	CL							

Boring terminated at a depth of 41.5 feet below the ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 11 feet and 19 feet during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.0, respectively, to account for sampler type and hammer energy.

PROJECT:	380 NORTH FIRST STREET San Jose, California	**Log of Boring B-2**

Boring location: See Site Plan, Figure 2

Date started: 02/13/2023 | Date finished: 02/13/2023

Drilling method: 8-inch-diameter hollow-stem auger

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

Sampler: Modified California (MC), Standard Penetration Test (SPT)

Logged by: J. Pisenti

Drilled by: Exploration Geoservices, Inc.

Rig: Mobile B40

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LAB: Type of Strength Test	LAB: Confining Pressure Lbs/Sq Ft	LAB: Shear Strength Lbs/Sq Ft	LAB: Fines %	LAB: Natural Moisture Content, %	LAB: Dry Density Lbs/Cu Ft
					5 inches of asphalt concrete						
					6 inches of aggregate base						
1–2	MC	44 72 66	87	ML	SANDY SILT (ML) gray-brown to dark brown, hard, dry to moist, fine to coarse sand, fine angular gravel, brick concrete debris (FILL)						
3–4	MC	23 15 14	18	SM	SILTY SAND (SM) brown, medium dense, dry to moist, fine sand, rootlets						
5–6	MC	7 7 10	11		light brown, no rootlets, increasing fines content						
7–8	MC	7 6 9	9	CL	SANDY CLAY (CL) brown, stiff, dry to moist, fine to coarse sand, trace fine rounded gravel Particle Size Distribution; see Appendix B				66	10.1	107
10–11	SPT	3 3 4	8		olive to olive-brown, medium stiff to stiff, fine sand, decreased gravel content, increase silt content Soil Corrosivity Test; see Appendix B						
13–14	MC	6 7 5	8	CH	CLAY (CH) olive-brown, medium stiff to stiff, moist, trace fine sand					27.9	94
19	MC	3 7 6	8		decreased sand content, moist to wet						
19–20				SM	(02/13/2023; 10:20 AM) SILTY SAND (SM) olive-brown, loose, moist to wet, fine sand						
20–22				CH	(02/13/2023; 10:08 AM) CLAY (CH) dark brown, medium stiff to stiff, wet, trace fine sand						
22–24	MC	6 11 10	13	CH	CLAY with GRAVEL (CH) olive-gray and gray, stiff, wet, trace fine subangular gravel						
28–30	MC	6 4 6	6	SM	SILTY SAND (SM) olive to olive-brown, loose, wet, fine sand Particle Size Distribution; see Appendix B				46	23.1	110

PROJECT: **380 NORTH FIRST STREET**
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
						SILTY SAND (SM) (continued)						
31					SM							
32												
33					CH	CLAY with SAND (CH) gray and yellow-brown, medium stiff, wet, fine sand and trace fine subangular gravel						
34	MC		3 5 5	6								
35					ML	SANDY SILT (ML) gray and yellow-brown, medium stiff, wet, fine sand						
36												
37					CL	CLAY with SAND (CL) gray with trace light brown, stiff, wet, fine sand						
38												
39	SPT		4 5 5	11								
40												
41					CL							
42												
43												
44	SPT		6 7 9	17		gray, very stiff, trace organics						
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 the ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 19.75 feet and 20.75 feet during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63 and 1.08, respectively, to account for sampler type and hammer energy.

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Project No.: 23-2365 | Figure: A-4b

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

- Sample taken with California or Modified California split-barrel sampler. Darkened area indicates soil recovered
- Classification sample taken with Standard Penetration Test sampler
- Undisturbed sample taken with thin-walled tube
- Disturbed sample
- Sampling attempted with no recovery
- Core sample
- Analytical laboratory sample
- Sample taken with Direct Push sampler
- Sonic

SAMPLER TYPE

C Core barrel

CA California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter

D&M Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube

O Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube

PT Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube

MC Modified California sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.38- or 1.5-inch inside diameter (refer to text)

ST Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

380 NORTH FIRST STREET
San Jose, California

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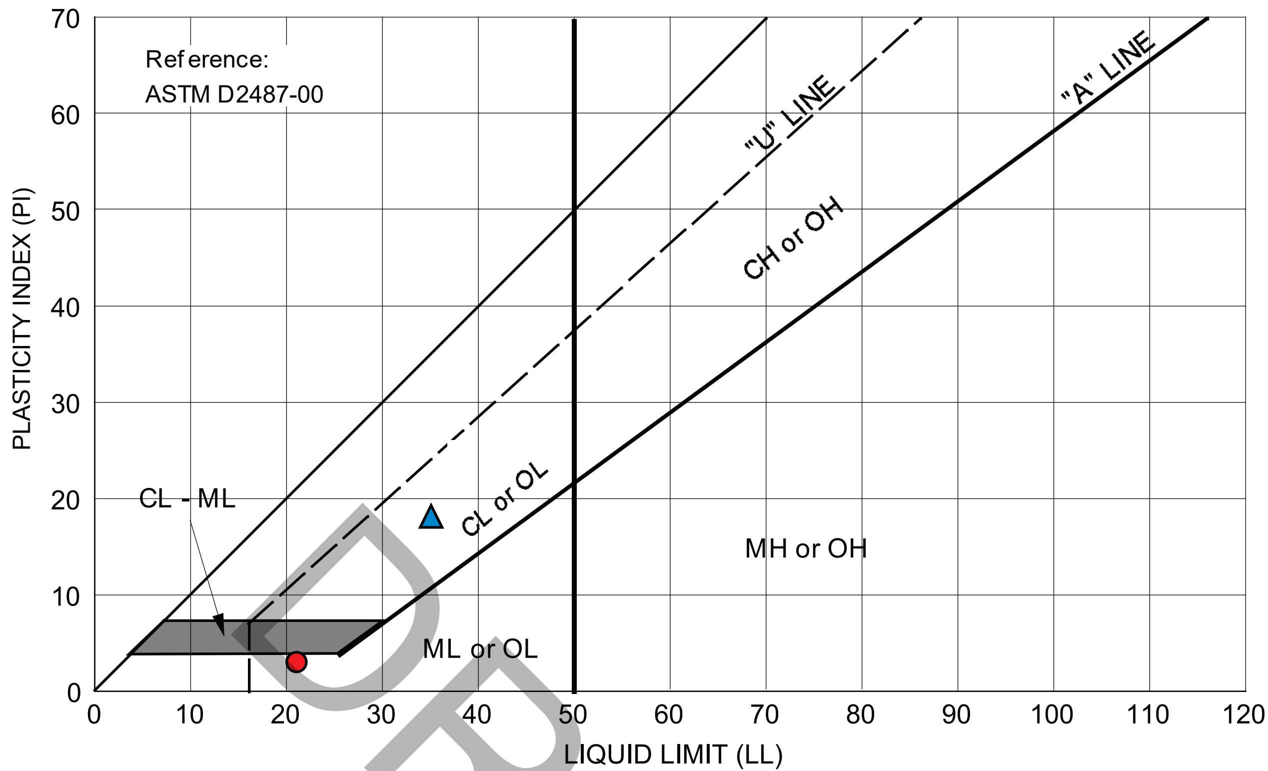
CLASSIFICATION CHART

Date 02/16/23 | Project No. 23-2365 | Figure A-5

ROCKRIDGE GEOTECHNICAL

APPENDIX A
Cone Penetration Test Results and Logs of Test Borings



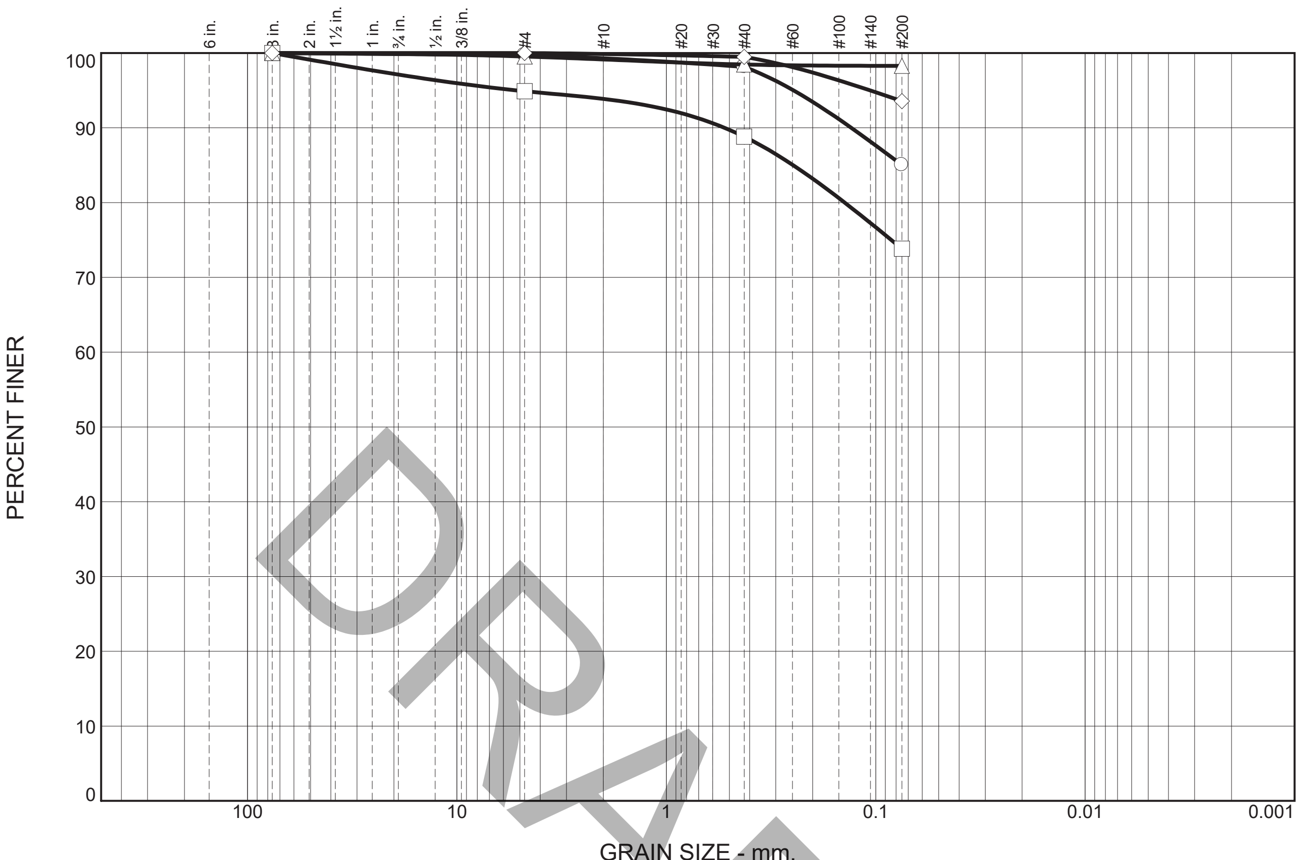
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 9.0 feet	SILTY with SAND (ML), yellow-brown	--	21	3	74
▲	B-1 at 26.0 feet	CLAY (CL), olive-brown	94	35	18	94

380 NORTH FIRST STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

PLASTICITY CHART

Date 04/12/23 | Project No. 23-2365 | Figure B-1



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.1	0.2	0.4	1.2	13.1	85.0	
□	0.0	2.9	2.2	1.0	5.1	15.0	73.8	
△	0.0	0.1	0.4	0.4	0.6	0.2	98.3	
◇	0.0	0.0	0.0	0.1	0.4	5.9	93.6	

SOIL DATA

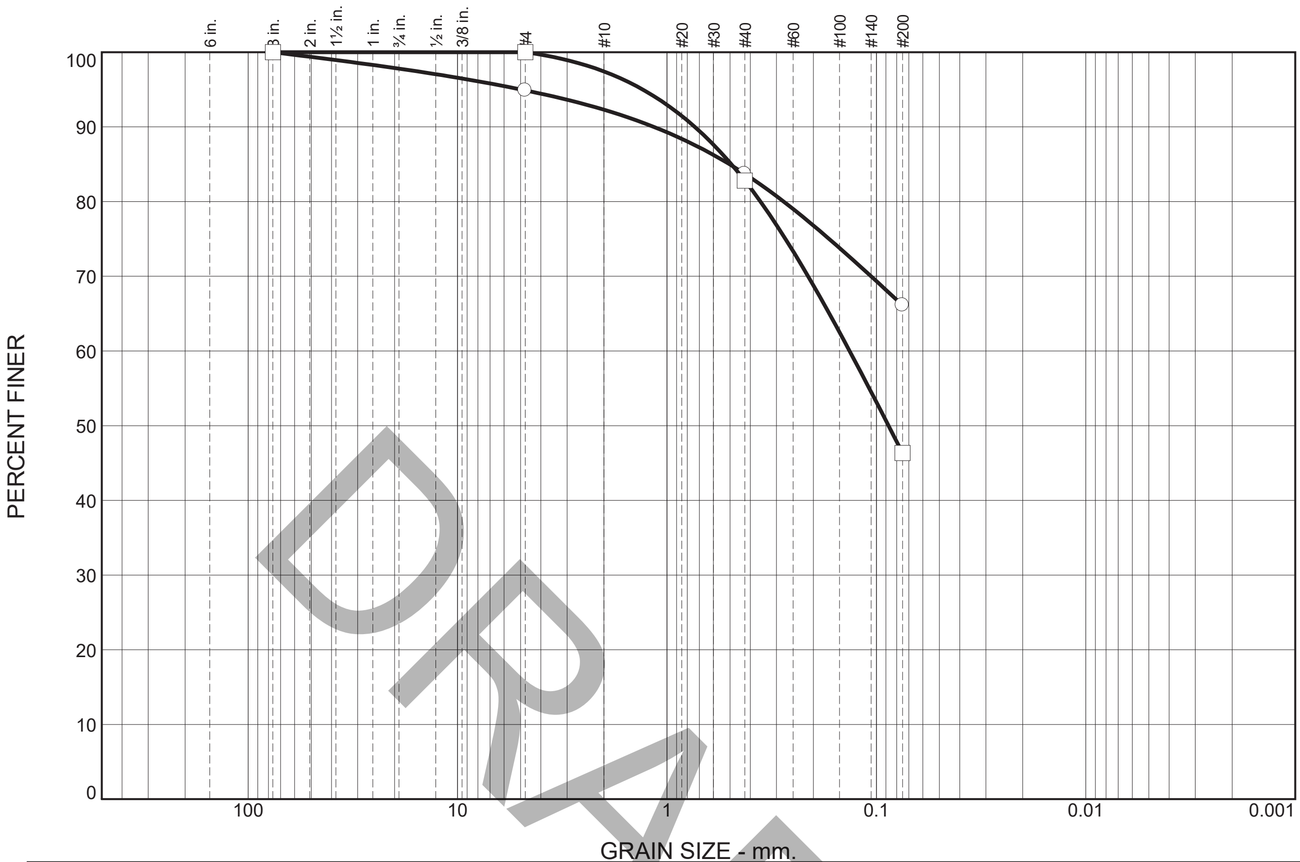
SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	6.0'	CLAY with SAND, brown	CL
□	B-1	9.0'	SILT with SAND, yellow-brown	ML
△	B-1	15.5'	CLAY, yellow-brown	CL
◇	B-1	26.0'	CLAY, olive-brown	CL

380 NORTH FIRST STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 03/10/23 | Project No. 23-2365 | Figure B-2



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	2.2	2.9	2.6	8.6	17.6	66.1	
□	0.0	0.0	0.0	2.6	14.6	36.5	46.3	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-2	7.75'	SANDY CLAY, brown	CL
□	B-2	29.5'	SILTY SAND, olive to olive-brown	SM

380 NORTH FIRST STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 03/10/23 | Project No. 23-2365 | Figure B-3

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

REPORT S230303B

Method		ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	Depth	Sulfates SO_4^{2-}		Chlorides Cl^-		Resistivity As Rec'd	Minimum	pH	Redox	Sulfide S^{2-}	Nitrate NO_3^-	Ammonium NH_4^+	Lithium Li^+	Sodium Na^+	Potassium K^+	Magnesium Mg^{2+}	Calcium Ca^{2+}	Fluoride F_2^-	Phosphate PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-1: SANDY CLAY (CL) olive-brown	2.5	210.8	0.0211	133.5	0.0134	3,216	1,675	7.6	170	0.3	90.6	1.5	0.0	108.2	11.1	52.8	204.8	8.9	3.2
B-2: SANDY CLAY (CL) brown	10.8	217.3	0.0217	126.9	0.0127	11,390	3,685	9.2	101	4.0	0.1	2.8	0.0	309.6	7.4	37.4	138.9	4.8	6.7
B-1: SANDY CLAY (CL) red	2.0	456.0	0.0456	109.4	0.0109	2,479	1,675	7.2	175	0.1	113.0	29.5	0.0	66.8	9.5	89.5	231.9	0.5	1.3
B-2: SANDY CLAY (CL) olive to olive-brown	10.75	157.7	0.0158	129.3	0.0129	1,206	938	9.8	61	7.9	36.6	49.0	0.0	705.1	14.7	55.4	172.0	5.3	71.9

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops which is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

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

380 NORTH FIRST STREET
San Jose, California

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SOIL CORROSIVITY TEST RESULTS

Date 04/12/23 | Project No. 23-2365 | Figure B-4

ROCKRIDGE GEOTECHNICAL

APPENDIX C
Liquefaction Analysis Results

Rockridge Geotechnical, Inc.
270 Grand Avenue
Oakland, California
http://www.rockridgegeo.com

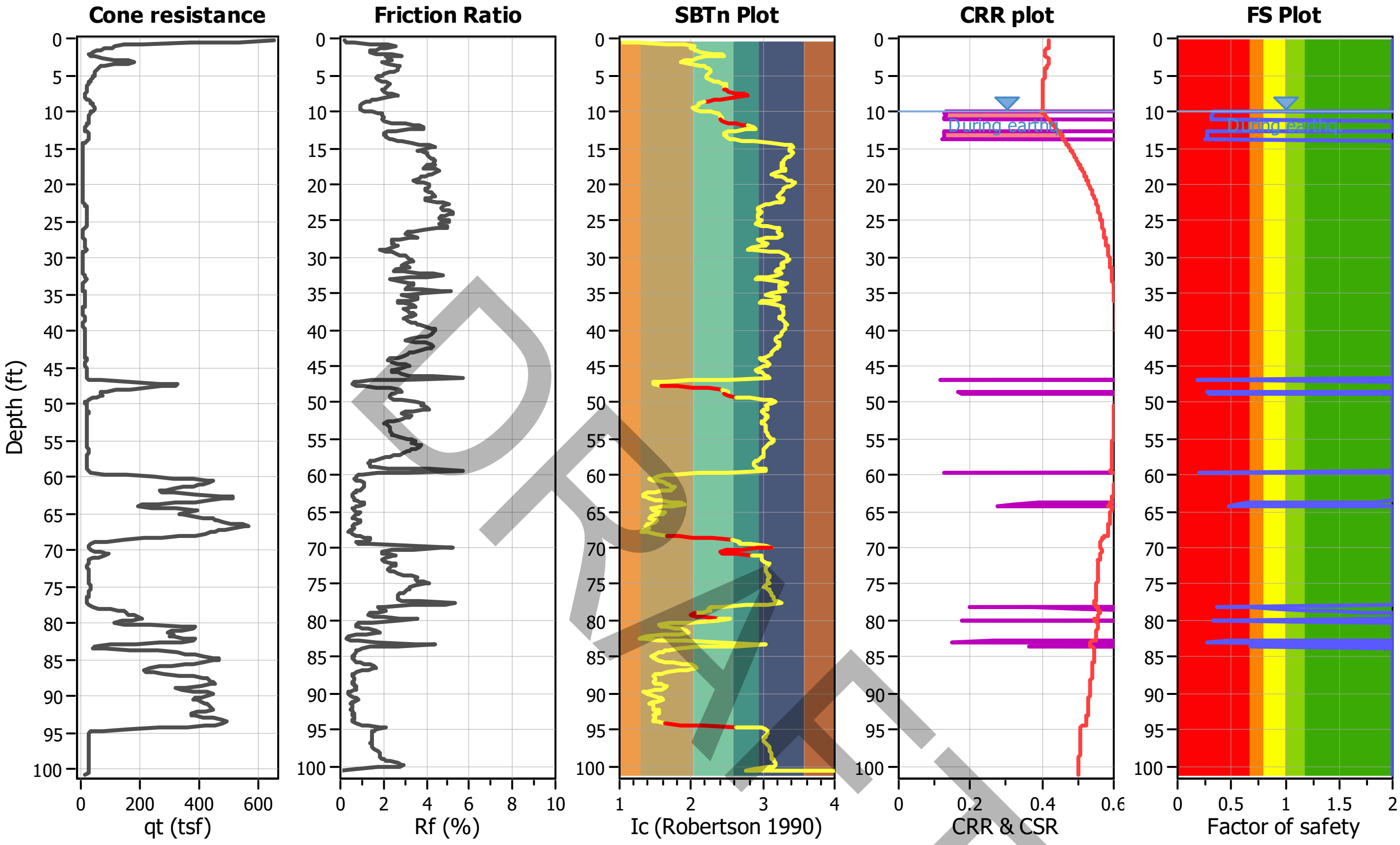
LIQUEFACTION ANALYSIS REPORT

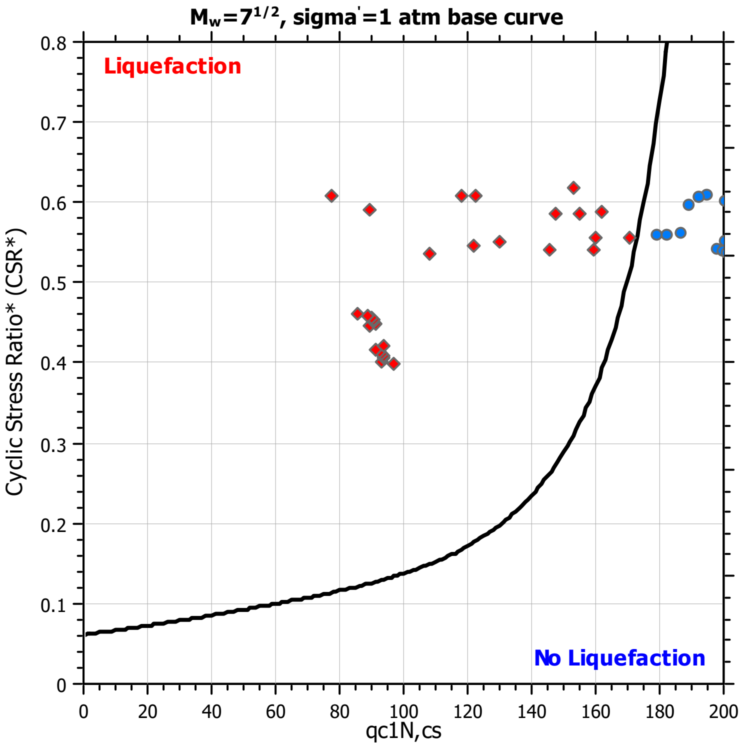
Project title : 380 N. First Street **Location : San Jose, California**

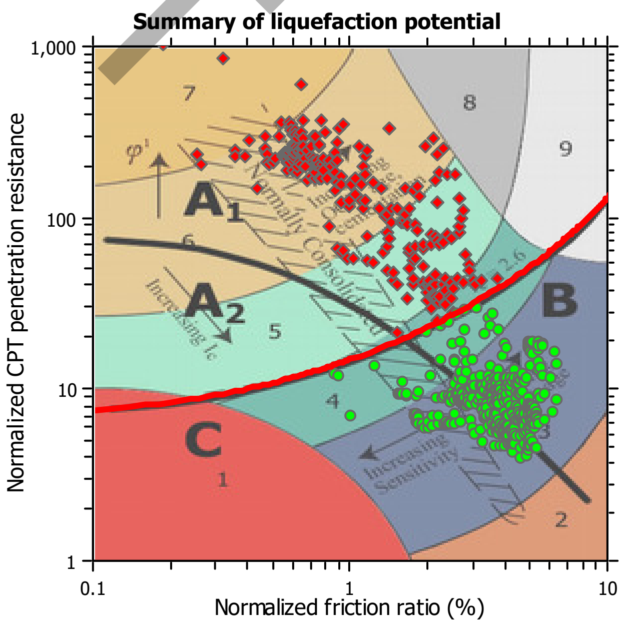
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	14.00 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	No
Earthquake magnitude M_w:	7.58	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	N/A
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	No	MSF method:	Method

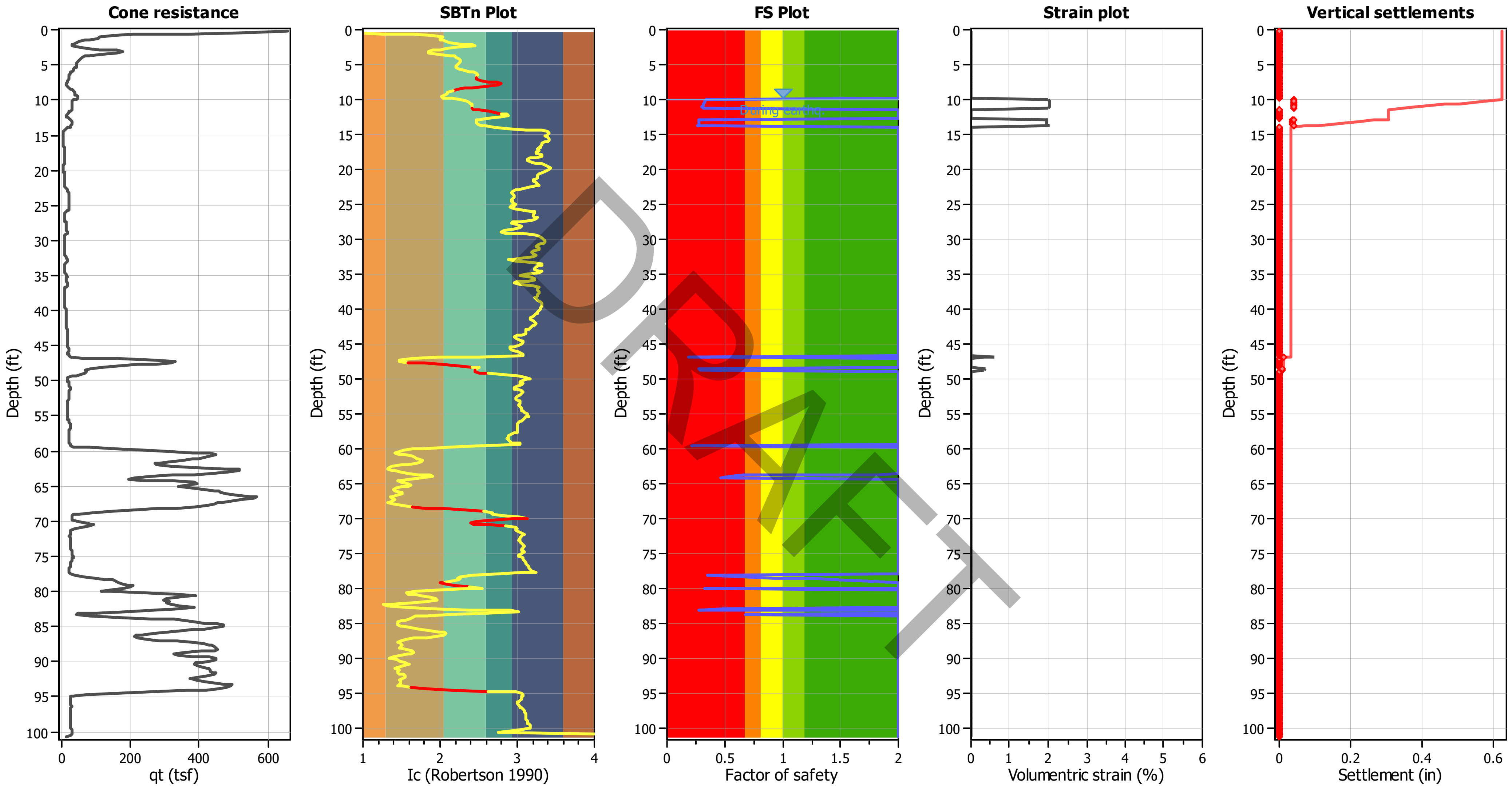






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

Estimation of post-earthquake settlements



Abbreviations

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

Rockridge Geotechnical, Inc.
270 Grand Avenue
Oakland, California
http://www.rockridgegeo.com

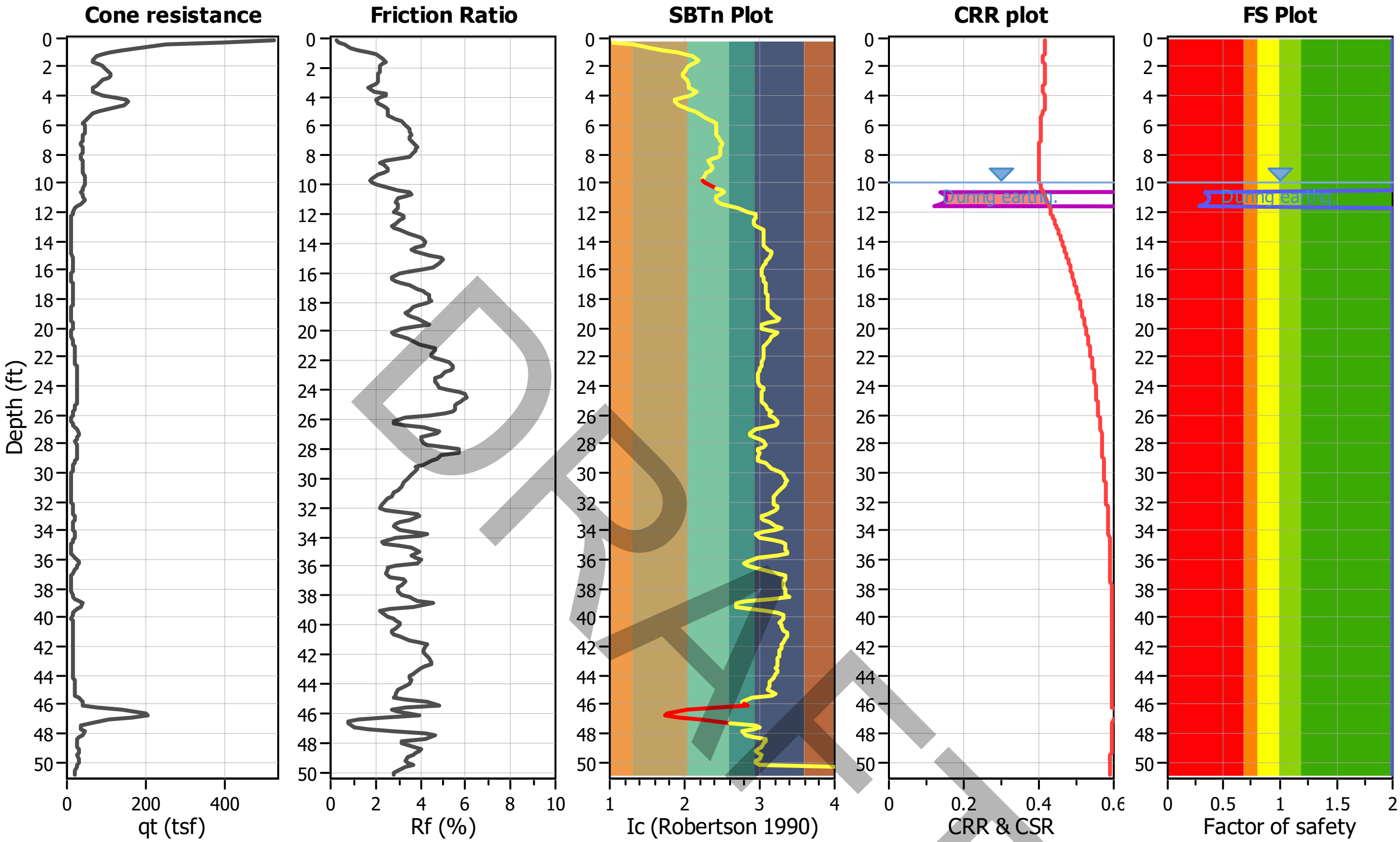
LIQUEFACTION ANALYSIS REPORT

Project title : 380 N. First Street **Location : San Jose, California**

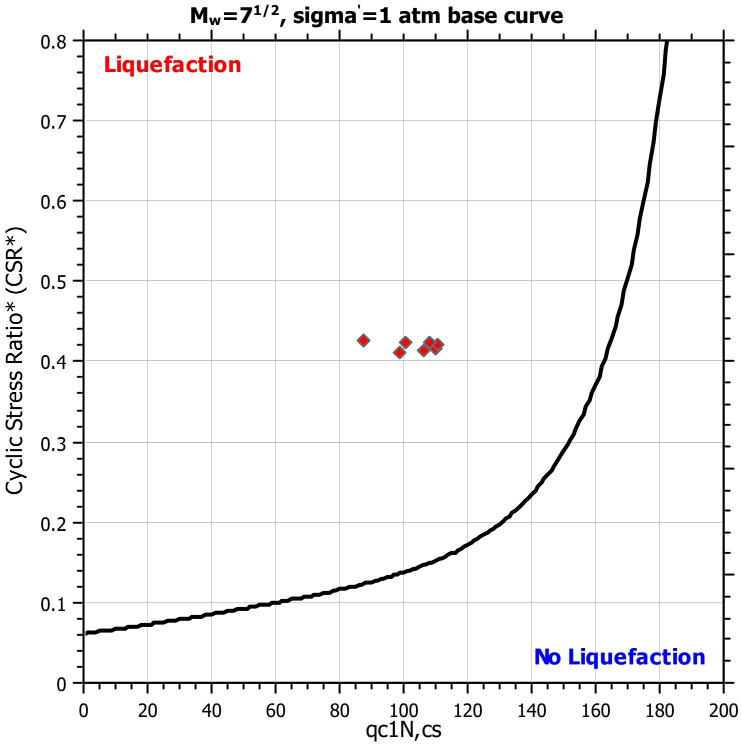
CPT file : CPT-02

Input parameters and analysis data

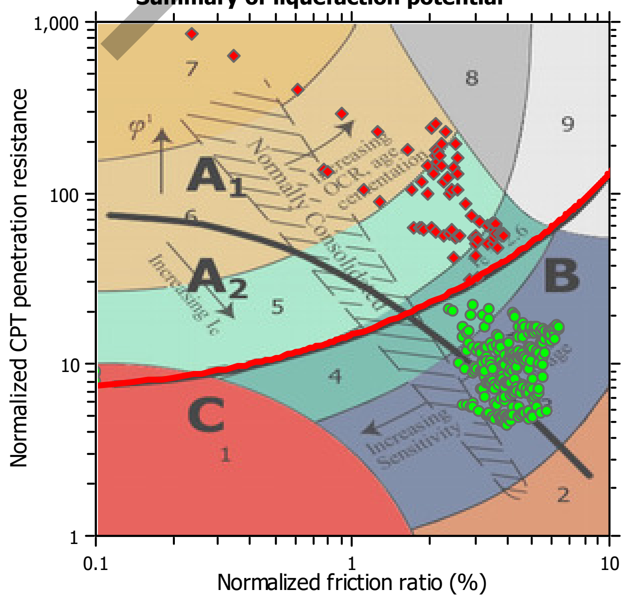
Analysis method:	B&I (2014)	G.W.T. (in-situ):	20.30 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	10.00 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.58	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.62	Unit weight calculation:	Based on SBT	K_σ applied:	No		



$M_w=7^{1/2}$, sigma'=1 atm base curve

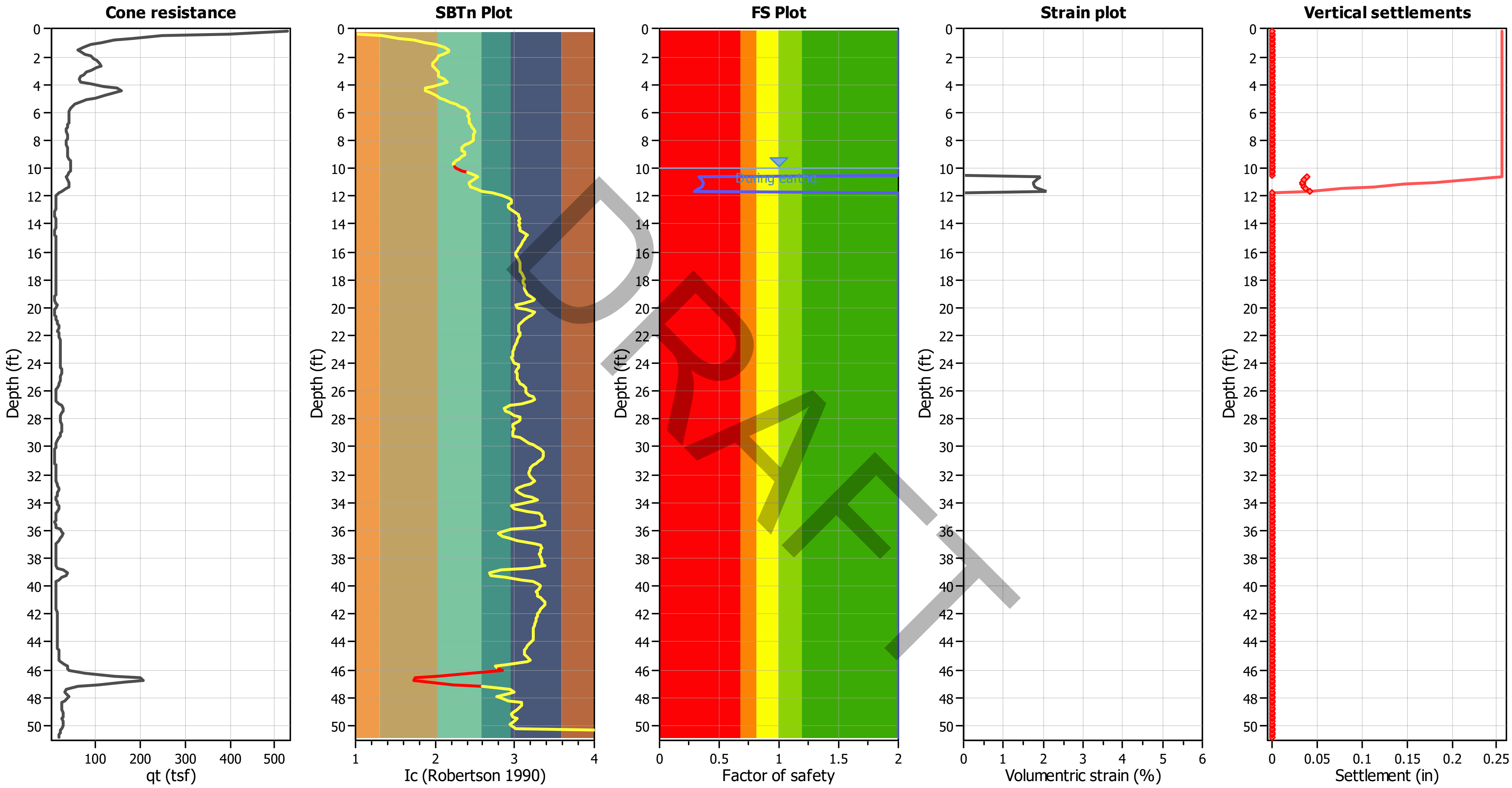


Summary of liquefaction potential



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

Estimation of post-earthquake settlements



Abbreviations

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