ROCKRIDGE
GEOTECHNICAL

Prepared for **Mr. Richard Truempler**

GEOTECHNICAL INVESTIGATION REPORT
PROPOSED BLOCK 8 OFFICE BUILDING
282 SOUTH MARKET STREET
SAN JOSE, CALIFORNIA

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June 24, 2019
Project No. 18-1602

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GEOTECHNICAL

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Mr. Richard Truempler
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Subject: Geotechnical Investigation Report
Proposed Block 8 Office Building
282 S. Market Street
San Jose, California

Dear Mr. Truempler:

We are pleased to present our geotechnical investigation report for the proposed Block 8 office building to be constructed at 282 S, Market Street in San Jose, California. Our services are being provided in accordance with our proposal dated October 22, 2018.

The subject property is a relatively level, approximately rectangular-shaped parcel with plan dimensions of about 297 feet in the east-west direction by 223 feet in the north-south direction. It is bordered by W. San Carlos Street to the south, S. Market Street to the west, First Street to the east, and the four-story Four Points by Sheraton hotel and the 12-story United Food and Commercial Workers Union office building to the north.

Plans are to construct a 17-story, at-grade office building on the site. The northern edge of the proposed building will be set back at least 22 feet from the Four Points by Sheraton hotel and at least 30 feet from the United Food & Commercial Union office building. The first floor of the building will be occupied by the building lobby, a loading dock, MEP and trash rooms, and commercial space. The 2nd through 6th floors will be used for parking. The remaining 9 to 11 floors will be occupied by offices. Other proposed features include a sky garden above a portion of the 15th story of the building.

On the basis of our investigation, we conclude the proposed buildings may be constructed as planned, provided the recommendations presented in the attached report are incorporated into the project plans and specification. The primary geotechnical concern affecting the proposed development is the presence of medium stiff to stiff clay that is moderately compressible underlying the site

We conclude the most appropriate foundation system for the proposed buildings would be deep foundations or a mat foundation bearing on ground improved. These and other issues are discussed in greater detail in the attached report.

The recommendations contained in our report are based on a limited subsurface exploration and laboratory testing program. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe excavation, grading and installation of foundations and/or ground improvement, during which time we may make changes in our recommendations, if deemed necessary.

We appreciate the opportunity to provide our services to you on this project. If you have any questions, please call.

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.

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Enclosure

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GEOTECHNICAL INVESTIGATION
PROPOSED BLOCK 8 OFFICE BUILDING
282 S. MARKET STREET
San Jose, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed Block 8 office building to be constructed at 282 S. Market Street in San Jose, California. The subject property is located at the northeastern corner of the intersection of S. Market and W. San Carlos streets, as shown on the Site Location Map, Figure 1.

The subject property is a relatively level, approximately rectangular-shaped parcel with plan dimensions of about 297 feet in the east-west direction by 223 feet in the north-south direction. It is bordered by W. San Carlos Street to the south, S. Market Street to the west, First Street to the east, and the four-story Four Points by Sheraton hotel and the 12-story United Food and Commercial Workers Union office building to the north.

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Structural loads for the proposed building were not available to us at the time we prepared this report. Based on our experience with similar building types, we estimate an average contact bearing pressure of 3,000 pounds per square foot (psf) for dead-plus-live-load conditions.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal dated October 22, 2018. Our scope of services consisted of reviewing available subsurface information and geologic maps of the site and vicinity, exploring subsurface conditions at the site by drilling two rotary-wash borings, performing five cone penetration tests (CPTs), and performing engineering analyses to develop conclusions and recommendations regarding:

- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- design groundwater table
- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s)
- estimates of foundation settlement of the proposed building and the adjacent streets and buildings
- slab-on-grade floor
- lateral earth pressures for elevator pit walls
- site grading and fill placement, including fill quality and compaction requirements
- 2016 California Building Code (CBC) site class and design spectral response acceleration parameters
- corrosion potential
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Subsurface conditions at the site were explored by drilling two rotary-wash borings and performing five CPTs. Prior to drilling, we obtained a drilling permit from the Santa Clara Valley Water District (SCVWD) for the borings and CPTs. We also contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained Precision Locating, LLC, a private utility locator, to check that the boring and CPT locations were clear of underground utilities. Details of our field exploration are described in the remainder of this section.

3.1 Rotary-Wash Borings

Two borings, designated as B-1 and B-2, were drilled at the approximate locations shown on Figure 2 by Pitcher Services, LLC of East Palo Alto, California on February 11 and 12, 2019. The borings were drilled using a truck-mounted drill rig equipped with rotary-wash drilling equipment, to depths of 100 feet bgs and 101-1/2 feet bgs for B-1 and B-2, respectively.

During drilling, our field geologist logged the soil encountered and collected representative samples of the soil for visual classification and laboratory testing. The logs of borings are presented in Appendix A on Figures A-1a through A-2d. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-3 in Appendix A.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter stainless steel tubes.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and a constant 1.375-inch inside diameter.
- Dames & Moore (D&M) thin-walled brass tubes with a 2.5-inch outside diameter

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. The S&H sampler was used to obtain samples in cohesive soil, and the SPT sampler was used to evaluate the relative density of granular soils. The S&H and SPT samplers were driven with a 140-pound, automatic hammer falling 30 inches per drop. The S&H and SPT 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 per six inches or less of penetration. The blow counts required to drive the S&H and SPT samplers were converted to approximate SPT N-values using factors of 0.84 and 1.2, respectively, to account for sampler type and approximate hammer energy. The SPT sampler used could not accommodate liners. The blow counts used for this conversion were: (1) the last two blow counts if the sampler was driven more than 12 inches, (2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and (3) the only blow count if the sampler was driven six inches or less. The converted SPT N-values are presented on the boring logs.

The D&M tubes were used to obtain relatively undisturbed samples of cohesive soil. The tubes were pushed into the soil under the weight of the drill rods and the hydraulic pressure from the drill rig. The hydraulic pressure required to advance the D&M tube is presented on the boring logs.

Upon completion of drilling, the boreholes were backfilled with neat cement grout in accordance with SCVWD requirements. The soil cuttings and drilling fluid from the borings were placed in 55-gallon drums and removed from the site by Pitcher Services, LLC.

3.2 Cone Penetration Tests

On February 11 and 12, 2018, ConeTec, of San Leandro, California performed five CPTs designated as CPT-1 through CPT-5, at the approximate locations shown on the Site Plan, Figure 2. CPT-1, CPT-2, CPT-3, and CPT-5 met practical refusal at depths ranging from 114 to 158 feet below the existing ground surface. CPT-4 encountered an obstructed at a depth of 9 feet bgs and was terminated at that depth.

The CPTs were performed by hydraulically pushing a 1.7-inch-diameter, cone-tipped probe with a projected area of 15 square centimeters into the ground using a 30-ton truck rig. The cone-tipped probe measured tip resistance, and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the soil behavior types and approximate strength characteristics of the soil encountered. The CPT logs showing tip resistance and friction ratio, as well as interpreted soil behavior type, are presented on Figures A-4 through A-7 in Appendix A. Upon completion, the CPTs were backfilled with cement grout in accordance with SCVWD requirements and the pavement was patched with quick-set concrete.

3.3 Laboratory Testing

We re-examined each soil sample in the office to confirm the field classification and selected representative samples for laboratory testing. Geotechnical laboratory tests were performed on selected soil samples to measure 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, fines content, plasticity (Atterberg limits), and undrained shear strength. Soil samples were also tested by Inspection Services, Inc. of Berkeley, California to measure compressibility (consolidation characteristics). In addition, one soil sample obtained from Boring B-2 at 3 feet bgs was tested by Project X Corrosion Engineering of Murrieta, California to evaluate corrosivity of the near-surface soil. The results of the laboratory tests are presented on the boring logs in Appendix A and in Appendix B.

4.0 SUBSURFACE CONDITIONS

The geologic units in the site vicinity, as shown on the regional geologic map prepared by Graymer, et al. (2006) for the U.S. Geological Survey, are presented on Figure 3 (Regional Geologic Map). This map indicates the site is blanketed by Holocene-age alluvium (Qha). The alluvium primarily consists of layers of clay with varying sand and gravel content that are interbedded with sand and gravel layers with varying fines content to the maximum depth explored of 158 feet bgs. The CPTs and borings advanced indicate the site is blanketed by about 3 to 5 feet of fill consisting of medium dense clayey sand and medium stiff to stiff sandy clay. Where explored, the fill is underlain by soft to stiff clay and sandy clay and loose to medium dense silty sand to a depth of approximately 15 feet bgs. Below a depth of 15 feet bgs to an approximate depth of 50 feet bgs, the clay becomes medium stiff to very stiff while the sand and gravel layers are generally medium dense. Below a depth of 50 feet bgs to the maximum explored depth, the soil consists of stiff to hard clay and sandy clay and medium dense to very dense sand and gravel layers with varying fines content.

4.1 Groundwater

The results of pore pressure dissipation tests performed in CPT-1, CPT-2, CPT-3, and CPT-5, performed on February 11, 2019, indicate the depth to groundwater ranged from 11.3 to 24.9 feet bgs. Groundwater was measured in Boring B-1 at a depth of 14.5 feet bgs prior to implementing drilling fluid. Groundwater measurements in Boring B-2 were obscured by the rotary-wash drilling method. Available historic groundwater information presented in the Seismic Hazard Zone Report for the San Jose West Quadrangle indicate the historic high groundwater at the site is approximately 12 feet bgs.

We also reviewed available data on the State of California, Regional Water Quality Control Board (RWQCB) Geotracker website (https://geotracker.waterboards.ca.gov). The groundwater data most pertinent for the 282 South Market Street property was obtained from documents from four properties within a 2000 foot radius of the site. Groundwater monitoring has been performed at the 95 South Almaden Avenue site quarterly or semi-annually between April 1991 and March 2019 to monitor groundwater contamination resulting from leaking underground storage tanks on the 95 South Almaden Avenue property. The depth to groundwater at 95 Almaden Avenue ranged from 12.71 feet to 25.79 feet. Groundwater measured at 520 South First Street quarterly from June 2002 to March 2004 indicated the depth to water ranged from 10 feet to 15 feet. Additionally, groundwater monitoring has been performed at 598 South First Street, a former Texaco gas station, quarterly or semi-annually between November 1994 and May 2017. The depth to groundwater at the former Texaco station site ranged from 8.59 feet to 17.82 feet. Finally, groundwater was monitored at the Spartan gas station located at 498 South Fourth Street monthly from March to December in 1998 and quarterly or semi-annually from February 2005 to January 2015. The groundwater measurements at 498 South Fourth Street indicated a depth to groundwater ranging from 8.08 feet to 21.43 feet.

The depth to groundwater is expected to vary several feet annually, depending on rainfall amounts. We recommend using a design groundwater depth of 10 feet bgs.

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[1], lateral spreading[2] and cyclic densification.[3] The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity

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 Calaveras, Hayward, and San Andreas faults. These faults and other known Quaternary-aged faults that are believed to be sources of major earthquakes (i.e., Magnitude>6.0) in the region are shown on Figure 4, as accessed from the U.S. Geological Survey (USGS) database (USGS, 2010). Active faults within a 50-kilometer radius of the site, the distance from the site, and mean characteristic moment magnitude[4] [2007 Working Group on California Earthquake Probabilities (USGS 2008) and Cao et al. (2003)] are summarized in Table 1.

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

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

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

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

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Monte Vista-Shannon	12	Southwest	6.50
Total Calaveras	13	Northeast	7.03
Total Hayward	14	Northeast	7.00
Total Hayward-Rodgers Creek	14	Northeast	7.33
N. San Andreas - Peninsula	19	Southwest	7.23
N. San Andreas (1906 event)	19	Southwest	8.05
N. San Andreas - Santa Cruz	20	Southwest	7.12
Zayante-Vergeles	28	Southwest	7.00
Greenville Connected	36	East	7.00
San Gregorio Connected	43	West	7.50
Mount Diablo Thrust	45	North	6.70

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 33 kilometers south of the site.

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

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

5.2 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong ground shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failures, such as that associated with soil liquefaction, lateral spreading, and cyclic densification. We used the results of the borings and CPTs to evaluate the potential of these phenomena occurring at the project site.

5.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the Calaveras and Hayward faults, although ground shaking from future earthquakes on other faults, including the Monte Vista-Shannon and San Andreas Faults, will also be felt at the site. These and other faults in the region are shown in relation to the site on Figure 4. The ground shaking intensity felt at the project site will depend on: 1) the size (magnitude) and duration of the earthquake, 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) site-specific soil conditions. We judge that strong to very strong ground shaking could occur at the site during a large earthquake on one of the nearby faults.

5.2.2 Liquefaction and Associated Hazards

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

The subject property is located in an area of San Jose designated as a potential liquefaction hazard zone on the map prepared by the California Geological Survey (CGS) titled *State of California, Seismic Hazard Zone, San Jose West Quadrangle*, dated February 7, 2002 (Figure 5). We evaluated the liquefaction potential of soil encountered below groundwater at the site using data collected in our CPTs.

Our liquefaction analyses using CPT data were performed following the methodology by Boulanger & Idriss (2014), and liquefaction susceptibility was assessed using the software CLiq v2.2 (GeoLogismiki, 2016). CLiq uses measured CPT data and assesses liquefaction susceptibility and post-earthquake vertical settlement given a user-defined earthquake magnitude and peak ground acceleration (PGA).

Our liquefaction analyses were performed using an assumed high groundwater of 10 feet bgs. In accordance with the 2016 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 Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). We also used a moment magnitude 7.33 earthquake, which is consistent with the mean characteristic moment magnitude for the Hayward Fault, as presented in Table 1.

Our liquefaction analyses indicate there are thin layers of potentially liquefiable soil underlying the site generally between depths of 25 and 35 feet. The potentially liquefiable layers are discontinuous and generally less than three feet thick. Most of the material identified as potentially liquefiable has a soil behavior type of “sand”, “silty sand”, and “silty clay” based on the interpretations of the CPT data. We estimate total and differential settlements associated with liquefaction at the site during an MCE event generating a PGA_M of 0.50g will be less than 1 inch and 1/2 inch across a horizontal distance of 30 feet, respectively.

Ishihara (1985) presented an empirical relationship that provides criteria 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. Our analysis indicates the non-liquefiable soil overlying the potentially liquefiable soil layers at the site is sufficiently thick and the potentially liquefiable layers are sufficiently thin, such that the potential for surface manifestations from liquefaction, such as sand boils and reduced bearing capacity, is low.

Considering the relatively flat site grades and the absence of a free face in the site topography, as well as the depth of the potentially liquefiable layer, we conclude the risk of lateral spreading is nil.

5.2.3 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The soil above the groundwater at the site primarily consists of fine-grained deposits that are sufficiently cohesive, such that they are not susceptible to cyclic densification. Therefore, we conclude the potential for cyclic densification to impact the proposed development is nil.

5.2.4 Ground Surface Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act, and no known active or potentially active faults exist on the site. We, therefore, conclude the risk of fault offset at the site from a known active fault is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure from previously unknown faults is also very low.

6.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, we conclude the subject property can be developed as planned. The primary geotechnical concern affecting the proposed development is the presence of medium stiff to stiff clay that is moderately compressible underlying the site. These and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

6.1 Foundation and Settlement

The results of our investigation indicate the site is underlain by alluvium that is moderately compressible. If the proposed building is supported on a shallow foundation system, settlement will occur due to consolidation of the underlying clay under static foundation loads. In addition, the building will be underlain by potentially liquefiable soil layers that will experience strength loss and post-liquefaction reconsolidation during and following a major earthquake, resulting in settlement of improvements constructed above these layers. We conclude the estimated total and differential settlements due to both static load conditions and post-liquefaction reconsolidation exceed the typical tolerance of shallow foundation systems (i.e. conventional spread footings or a mat foundation) bearing on existing (unimproved) ground conditions.

Based on our experience, we judge the most appropriate foundation type for the proposed high-rise building consists of a mat foundation supported on a ground improvement system designed to reduce total and differential settlements to tolerable levels or deep foundations that derive support through skin friction and end bearing in stiff clays and dense sands of the alluvial deposits.

Static settlement will affect various aspects of the planned development, including utilities, building entrances, and sidewalks. Design of these elements should incorporate the effects of the predicted settlement, as appropriate. To mitigate the detrimental effects of seismically induced settlement, flexible connections should be used where utilities enter the building. If a structural slab is used, the below-slab utilities should be supported by hangers suspended from the floor slab. The hangars should be designed to resist corrosion. Additionally, exterior slabs and ramps attached to the building should be hinged to accommodate differential settlement between the building and outside ground. Maintenance of utilities, sidewalks and entry slabs should be expected throughout the life of the project. This may include periodically replacing some of the improvements at the building/outside area interface.

6.1.1 Mat Foundation on Improved Ground

Ground improvement serves to stiffen the overall soil matrix by densifying loose soil layers and transferring the foundation loads to more competent materials below the moderately compressible and liquefiable layers, thus reducing settlements and providing increased bearing capacity beneath the mat foundation.

There are several types of ground improvement that may be utilized to reduce total and differential settlements of the proposed building. We consider soil-cement mix (SMX) columns or drilled displacement sand-cement (DDSC) columns to be the most appropriate ground improvement methods for this project. SMX columns are installed by injecting and blending cement into the soil using a drill rig equipped with single or multiple augers. DDSC columns are installed by advancing a hollow-stem auger that mostly displaces the soil and then pumping a sand-cement mixture into the hole under pressure as the auger is withdrawn. Both DDSC and SMX columns result in low vibrations during installation and generate few drilling spoils for off-haul. DDSC columns and SMX columns are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of columns should be determined by the design-build contractor based on the desired level of improvement (i.e. the tolerable settlement and desired allowable bearing pressure), as determined by the project Structural Engineer. We recommend a preliminary soil improvement design, including calculations of static and seismic settlements, be prepared by the ground improvement contractor and submitted for review by us, as well as the project Structural Engineer.

We conclude the proposed building may be supported on a reinforced concrete mat, provided the static and liquefaction-induced settlements are acceptable from a structural standpoint. Structural design loads were not available at the time this report was prepared. Based on our experience with similar buildings, we estimate the average bearing pressure imposed by the mat will be on the order of 3,000 psf for dead-plus-sustained-live loads. The length and spacing of the DDSC or SMX columns should be sufficient to limit total and differential static settlement to 2-1/2 inches and 1-1/4 inch across a horizontal distance of 30 feet, respectively, and liquefaction-induced total and differential settlement to less than 3/4 inch and 1/2 inch across a horizontal distance of 30 feet, respectively. The acceptability of these settlement criteria should be confirmed by the project Structural Engineer.

6.1.2 Deep Foundations

We evaluated the feasibility of the following deep foundation systems:

- drilled piers
- driven concrete or steel piles
- torque-down piles
- auger cast-in-place piles

We conclude drilled piers are not desirable for the site because of relatively high groundwater table, the presence of sandy soil that is susceptible to caving, and the large amount of off-haul that would be required. Installation of drilled piers will require casing and/or drilling slurry.

We conclude driven concrete or steel piles are also not desirable for the site because of the relatively high vibrations and noise generated during pile driving.

We believe more appropriate deep foundation systems are proprietary pile types, such as torque-down piles (TDPs) and auger cast-in-place (ACIP) piles. A TDP is a steel pipe pile with a closed conical end with pitched flights that allow the pipe pile to be "screwed-in" to the soil, resulting in displacement and densification of the surrounding soil. The pipe typically used for the TDPs has an outside diameter of 12.75 inches and a typical wall thickness of 0.375 (3/8) inches. When the pipe pile is advanced to the design tip elevation, it is filled with structural concrete to provide additional bending resistance. TDPs are displacement piles installed with little spoils created to reduce off-haul. An advantage of the TDPs is they can be installed with minimal vibration and noise, as compared to driven piles. However, TDPs will likely meet refusal in the dense sand layer around 45 to 50 feet bgs and if the design requires the piles to go deeper, that may not be possible with TDPs. Therefore, we believe ACIP piles would be the most suitable deep foundation option.

ACIP piles are installed by drilling a continuous flight, hollow-stem auger into the ground to a specified depth. Sand-cement grout or concrete is pumped into the hole under pressure as the auger is removed, eliminating the need for temporary casing or slurry. After the auger is removed, reinforcement can be installed while the cement grout or concrete is still fluid. Unlike driven piles, very little noise and vibrations are generated during the installation of the ACIP piles. ACIP piles are available with variable diameters; however, 16-inch-diameter is typical. ACIP piles can be installed as displacement, partial displacement, or non-displacement, allowing for the design to reduce spoils and off-haul.

We estimate total settlement due to static loads of the new building supported on properly designed and constructed deep foundations will not exceed 2-1/2 inches and differential settlement will be less than 1-1/4 inch in 30 feet.

6.2 Construction Considerations

We anticipate excavation at the site will generally be limited to those required to construct foundations and elevator pit(s), for the proposed building and to install new underground utilities. Excavation at the site can be performed with typical earth-moving equipment. Removal of buried obstructions may require equipment capable of breaking up reinforced concrete. All disturbed soil resulting from demolition activities that will be beneath proposed improvements should be overexcavated and recompacted in accordance with the recommendations in Section 7.1 under the observation of our field engineer.

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.

If the site grading is performed during the rainy season, the near-surface clay will likely be wet and will have to be dried before compaction can be achieved. Heavy rubber-tired equipment, such as haul trucks, scrapers, and vibratory rollers, could cause excessive deflection (pumping) of the wet clay and, therefore, should be avoided if this condition occurs. If the project schedule or weather conditions do not permit sufficient time for drying of the soil by aeration, the subgrade can be treated with lime, Quicklime, or cement, as appropriate, prior to compaction. If the grading work is performed during the dry season, moisture-conditioning may be required to increase the moisture to above optimum moisture content, as recommended in Section 7.1.

6.3 Soil Corrosivity

Corrosivity tests were performed by Project X Corrosion Engineering of Murrieta, California on one soil sample obtained from Boring B-2 at 3.0 feet bgs. The corrosivity test results are presented in Appendix B of this report.

The resistivity test results (1,273 ohm-cm) indicate the near-surface soil is “corrosive” to buried metallic structures. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron may need to be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.

The chloride ion concentrations (27 mg/kg) indicate the chloride in the soil is “negligibly corrosive” to buried metallic structures and reinforcing steel in concrete structures below ground. The results of the pH tests indicate the near-surface soil has a pH of 8.23 which should not have an adverse effect on buried concrete but may be detrimental to buried metal. The results also indicate the sulfate ion concentrations (150 mg/kg) which indicate the sulfate in the soil is “negligibly to moderately corrosivity” to buried concrete.

7.0 RECOMMENDATIONS

Our recommendations for the 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 removal of all existing pavements, former foundation elements, and underground utilities. Demolished asphalt concrete should be taken to an asphalt recycling facility. Aggregate base beneath existing pavements and floor slabs (if present) may be re-used as select fill if carefully segregated. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside of the footprint of the proposed building 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 that extend below finished improvements should be properly backfilled with engineered fill under our observation and following the recommendations provided later in this section.

7.1.1 Fill Materials and Compaction Criteria

Prior to placement of new fill materials, the exposed subgrade soil should be scarified to a depth of at least eight inches, moisture-conditioned, and compacted to the specified percent relative compaction,[5] as presented below in Table 2. Note that "moisture-conditioning" may require wetting or drying of the soil, depending on the particular conditions encountered at the time of construction. All fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned, and compacted to the specified percent relative compaction, presented below in Table 2. Each type of material is described in the following text according to its uses and specifications.

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

TABLE 2
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
General fill – low-plasticity	90+	Above optimum
General fill – native moderate plasticity clay	90+	2+% above optimum
General fill – low-plasticity, greater than 5 feet in thickness	95+	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	90+	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 City Engineer or inspector should determine which compaction requirements should take precedence.

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. Samples of proposed select fill material should be submitted to the geotechnical engineer at least three business days prior to use at the site.

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

Aggregate Base Material

Imported aggregate base material may be used as general fill, trench backfill (above bedding materials), or as select fill beneath pavements, exterior concrete flatwork, or the at-grade building slab. Aggregate base beneath pavements should meet the requirements in the 2018 Caltrans Standard Specifications, Section 26, for Class 2 Aggregate Base (3/4 inch maximum).

Controlled Low-Strength Material

Controlled low-strength material (CLSM) may be considered as an alternative to fill beneath structures or pavement. CLSM should meet the requirements in the 2018 Caltrans Standard Specifications. It is an ideal backfill material when adequate room is limited or not available for conventional compaction equipment, or when settlement of the backfill must be minimized. No compaction is required to place CLSM. CLSM should have a minimum 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).

7.1.2 Soil Subgrade Stabilization

Soft, wet soil may be exposed during excavation of the foundations, causing the subgrade to deflect and rut under the weight of grading equipment. 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 excavation subgrade extends below or near the groundwater table; however, it depends on the time of year construction is performed.

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.

Tensile fabric or geogrid may interfere with the installation of deep foundation or ground improvement systems. Their use should be confirmed by the foundation or ground improvement contractor prior to use in areas in which these systems will be installed.

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 relative compaction, and at least 95 percent relative compaction in the upper six inches of pavement or flatwork subgrade.

7.1.3 Exterior Flatwork Subgrade Preparation

We recommend a minimum of six inches of select material be placed beneath proposed exterior concrete flatwork, including sidewalks. The six-inch-thick select fill layer is not required beneath the mat foundation. 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 2.

7.1.4 Utility Trench Backfill

Flexible connections that can tolerate at least one inch of vertical movement due to post-liquefaction reconsolidation should be used where utilities enter the building. Backfill for utility trenches is considered fill, and it should be compacted according to the recommendations presented in Section 7.1.1. Additionally, if a structural slab is used, the below-slab utilities should be supported by hangers suspended from the floor slab. Utilities supported by underslab hangers should be loosely backfilled with clean pea-gravel. Special care should be taken when backfilling utility trenches beneath pavements. Poor compaction may result in excessive settlement and damage pavements. Jetting of trench backfill as a mean of compaction should not be permitted.

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

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 foundations.

7.1.6 Bioswales

The primary concerns with bioswales are: 1) providing suitable support for foundations and curbs constructed near the bioswales, and 2) potential for subsurface water from the bioswales to migrate (and possibly build up) beneath pavements and the proposed building. Consequently, we recommend that: 1) bioswales constructed at the site be provided with underdrains and/or drain inlets, and 2) bioswales be constructed no closer than five feet from the building. The subdrain pipes should be installed eight inches above the bottom of the bottom of the bioswale for treatment areas that are at least five feet away from the new building and pavements. The intent of this recommendation is to allow infiltration into the underlying soil, but to reduce the potential for bio-retention areas to flood during periods of heavy rainfall. The sides of bioswales should be sloped at a maximum gradient of 1.5:1 (horizontal to vertical).

Where bioswales must be located within five feet of the building and pavements, the bottom of the bioswale should be lined with an impermeable liner. Where a vertical curb or foundation is constructed near a bioswale, the curb and the edge of the foundation should be founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal to vertical) from the base of the bioswale. Shallow foundation elements within five feet of bioswales should not be relied upon for lateral support.

7.2 Foundations

As discussed in Section 6.1, we conclude the proposed building should be supported on either: 1) a mat foundation bearing on soil strengthened using ground improvement, or 2) ACIP piles. Recommendations for these foundation systems are presented below.

7.2.1 Mat Foundation on Improved Ground

For preliminary design of a mat foundation bearing on improved ground, we recommend ground improvement elements extend into the very dense sand at a depth of about 55 feet bgs. We anticipate the ground improvement systems described in Section 7.2.2, if properly designed and constructed, should be capable of increasing the maximum allowable bearing pressure to 5,000 pounds per square foot (psf) for dead-plus-live loads and 6,600 psf for total loads. The allowable bearing pressures recommended for dead-plus-live and total load conditions include factors of safety of at least 2.0 and 1.5, respectively. For design of the mat bearing on improved ground, we recommend using a preliminary modulus of vertical subgrade reaction of 10 pounds per cubic inch (pci) for dead-plus-live loads and 15 pci for total loads; these values have been reduced to account for the size of the mat. 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 vertical subgrade reaction, if appropriate.

The ground improvement elements should also be spaced sufficiently close to mitigate the potential for liquefaction of the fill between the elements. The final design allowable bearing pressures, estimated settlements, modulus of vertical subgrade reaction, and spacing of the elements to mitigate liquefaction potential should be evaluated by the design-build ground improvement contractor, as these values will be based on the diameter, depth, and spacing of the ground improvement elements.

Lateral loads may be resisted by a combination of friction along the base of the mat and passive resistance against the vertical faces of the mat foundation. To compute lateral resistance, we recommend using a uniform pressure of 2,000 psf for transient loads and an equivalent fluid weight (triangular distribution) of 240 pcf for sustained loads; the upper foot of soil should be ignored unless confined by a slab or pavement. The allowable friction factor will depend on the type of material at the base of the mat. If the mat is underlain by a vapor retarder, a friction factor of 0.20 may be used to compute base friction. Where the mat foundation is supported directly on soil, a friction factor of 0.30 may be used. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

Recommendations for mat subgrade preparation are presented in Section 7.1.2. The mat subgrade should be free of standing water, debris, and disturbed materials prior to placing the vapor retarder and concrete. The subgrade should be wetted following excavation and maintained in a moist condition until it is covered. We should check the foundation subgrade prior to placement of the vapor retarder or reinforcement/concrete.

7.2.2 Ground Improvement

We conclude viable ground improvement systems include DDSC or SMX columns. Ground improvement systems are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of the ground improvement elements should be determined by the contractor based on the proposed structural loads and the desired level of improvement.

The capacities and lengths of the ground improvement elements should be determined by the design-build contractor that installs the system; however for planning purposes, it may be assumed that both DDSC and SMX columns will extend to a depth of about 55 feet bgs. The length and spacing of the DDSC or SMX columns should be sufficient to limit total and differential static settlement to 2-1/2 inches and 1-1/4 inch across a horizontal distance of 30 feet, respectively, and liquefaction-induced total and differential settlement to less than 3/4 inch and 1/2 inch across a horizontal distance of 30 feet, respectively.

Our geotechnical report should be provided to potential design-build ground improvement contractors, and we should be retained to provide technical input and review the geotechnical aspects of their final design prior to construction. The final allowable bearing pressures and estimated settlements should be provided by the design-build ground improvement subcontractor and confirmed with load tests prior to installation of production elements.

We recommend the ground improvement design be verified in the field by performing at least two full-scale load tests in compression and one load test in tension (if ground improvement elements will be used to resist uplift loads) for the proposed building. Details regarding the proposed load testing program should be included in the design-build submittal for our review prior to mobilization to the site. The load tests should be performed on pre-production elements, under our observation, constructed using the same equipment, means-and-methods, area replacement ratio, and grout factor proposed for the production elements. The results of the load testing program should be evaluated by the design-build contractor's engineer, as well as our engineer, to confirm the columns provide an adequate factor of safety with respect to axial failure and allowable axial deflection at the design load prior to commencing with production installation.

We recommend the interface between the ground improvement elements and bottoms of footings be separated by a 12-inch-thick compacted aggregate cushion, consisting of Class 2 aggregate base or crushed rock. The purpose of the aggregate cushion is to provide some degree of isolation between the two elements, which will help prevent excessive moments from being induced in the ground improvement columns during lateral loading, as the elements do not typically contain reinforcing steel to resist bending stresses. The aggregate cushion may either be placed over the entire footing subgrade, which would require 12 inches of extra excavation, or it can be placed only above the tops of the ground improvement elements. The latter option requires diligent effort by the ground improvement contractor to "post-drill" the tops of the elements to the correct elevation prior to placement of the aggregate down the holes

7.2.3 Deep Foundations

ACIP piles are proprietary foundations and are designed and installed by the piling contractor. Recommendations and estimated capacities for ACIP piles are presented in this section. We can provide recommendations for alternative pile types upon request.

Axial Capacity

Piles should be designed to gain support through a combination of skin friction and end bearing in dense sand and alluvial deposits beneath the fill. Dense to very dense sand and silty sand was encountered at depths ranging from about 45 to 53 feet bgs. Based on discussions with contractors with experience installing ACIP pile foundations in the San Jose area, an ACIP pile embedded about 5 to 10 feet into dense to very dense sand would likely have an allowable axial compressive capacity of about 250 kips for dead-plus-live loads; this compressive capacity includes a factor of safety of about 2 and may be increased by 1/3 for total loads, including wind and seismic loads. For temporary uplift loads, we estimate allowable uplift capacities would be approximately 100 kips per pile, and includes a factor of safety of about 1.5. The allowable compressive and uplift capacities presented above have been reduced to account for liquefaction. These axial capacities are preliminary; final design of the ACIP piles should be performed by the piling contractor. If it is desired to achieve higher vertical capacities with ACIP piles, an allowable dead-plus-live-load skin friction value of 1,100 psf may be used below a depth of 35 feet bgs. The computed pile capacities using this skin friction value should be reduced by 100 kips to account for downdrag due to liquefaction.

The piles should be designed accounting for the presence of moderately corrosive soil near the surface. We should review the pile design and the plans and specifications. To avoid axial compression capacity reduction caused by group effects, piles should be spaced at least three pile widths apart, measured center-to-center. We should observe installation of the test and production piles.

Downdrag Loads

Downdrag loads could develop on the piles because of liquefaction-induced settlement of the soil adjacent to the piles. The magnitude of the downdrag load due to liquefaction-induced settlement will depend on several factors, including the thickness of liquefiable soil beneath the building pad. We estimate the downdrag load will be on the order of 100 kips for 16-inch-diameter ACIP piles. The downdrag load will only be applied temporarily shortly following a large earthquake on a nearby fault.

Lateral Load Resistance

Lateral load resistance can be mobilized by the individual piles in combination with other foundation elements embedded below the ground surface. Lateral resistance of piles will depend on the stiffness of the pile, the strength of the surrounding soil, the allowable deflection of the pile top, and the bending moment capacity of the pile.

We have calculated the lateral capacity for 1/2-inch lateral deflection at the top of pile for fixed- and free-head conditions. The moment and deflection versus depth profiles for 16-inch-diameter ACIP piles are presented on Figures 6 and 7. The lateral load capacities shown on Figures 6 and 7 are for single piles only. To account for group effects, the lateral load capacity of a single pile should be multiplied by the appropriate reduction factors shown in Table 3. The reduction factors are based on a minimum pile spacing of three pile widths.

TABLE 3
Pile Group Reduction Factors

Number of Piles in Pile Group	Reduction Factor
2	0.9
3 to 5	0.8
6 to 9	0.7

Where piles have center-to-center spacing of at least six pile widths in the direction of loading, no group reduction factors need to be applied. Reduction for other pile group spacing can be provided once the number and arrangement of piles are known.

Additional lateral load resistance can be developed by passive resistance acting against the faces of the pile caps and grade beams. An equivalent fluid weight (triangular distribution) of 240 pounds per cubic foot (pcf) may be used to compute passive resistance. This value includes a factor of safety of 1.5.

Indicator Piles and Pile Load Tests

We recommend that before production ACIP pile lengths are selected, indicator piles be installed to: (1) aid in evaluating predrilling requirements, and (2) aid in estimating production pile lengths. We recommend eight indicator piles be installed within the building footprint. Indicator piles may be installed at production pile locations. We expect the indicator piles can be used for support of the proposed structure if installed in the proper location and are not damaged during installation. Indicator piles should be installed with the same equipment and methodology that will be used to install the production piles.

In addition, we recommend pile load tests of the ACIP piles be performed to confirm the axial compressive and tensile pile capacities. For ACIP piles, we recommend a minimum of two compressive and one uplift load tests be performed. The test piles should be selected by the Geotechnical Engineer and approved by the Structural Engineer. The load tests should be performed in accordance with ASTM D1143 (Standard Test Methods for Deep Foundations

Under Static Axial Compressive Load) and ASTM D3689 (Standard Test Methods for Deep Foundations Under Static Axial Tensile Load). Equipment used for the test (load frame, jacks, ad reaction piles) should be capable of applying at least 2.5 times the allowable dead plus live design loads. The Davisson Method or 90% Criterion (Brinch-Hanson) Method should be used to interpret the ultimate capacities of the piles.

7.3 Capillary Moisture Break and Water Vapor Retarder

A concrete slab-on-grade floor may be used for the pile option provided the potential for up to one inch of seismically induced differential settlement between the floor slab and the pile-supported elements (i.e., grade beams and pile caps) is acceptable. If the potential for this differential settlement is not acceptable, the floor slab should be designed to span between pile caps and grade beams.

The subgrade for floor slabs and mat foundations should be prepared in accordance with our recommendations in Section 7.1.2. Where water vapor transmission through the floor slab/mat is not desirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor slab/mat. A capillary moisture break and water vapor retarder are generally not required beneath parking garage floor slabs because there is sufficient air circulation to allow evaporation of moisture that is transmitted through the slab; however, we recommend the capillary moisture break and water vapor retarder be installed below the slab-on-grade in utility rooms and any areas in or adjacent to the parking garage that will be used for storage and/or will receive a floor covering or coating.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The particle size of the capillary break material should meet the gradation requirements presented in Table 4.

TABLE 4
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
1 inch	90 – 100
¾ inch	30 – 100
½ inch	5 – 25
3/8 inch	0 – 6

The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. Where the building will be supported on a mat, the capillary moisture break may be omitted provided the vapor retarder meets the requirements for Class A vapor retarders. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

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

7.4 Permanent Below-Grade Walls

Below-grade walls (i.e., elevator walls) should be designed to resist lateral earth pressure imposed by the retained soil. Since the elevator pit walls will be restrained from movement at the sides, they should be designed for at-rest conditions. We recommend restrained walls be designed using at-rest equivalent fluid weights of 60 and 90 pcf if the walls are drained and undrained, respectively. To evaluate the below-grade walls for seismic loading, we recommend using an active equivalent fluid weight of 40 pcf plus a seismic increment of 20 pcf (triangular distribution) for drained conditions; and an active equivalent fluid weight of 80 pcf plus a seismic increment of 10 pcf (triangular distribution) for undrained conditions.

To protect against moisture migration, below-grade walls should be waterproofed and water stops should be placed at all construction joints. Although the below-grade walls may be above the design groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines, etc. If the "drained" earth pressures presented above are used to design the walls, they will need to incorporate a drainage system. Alternatively, the walls may be designed for the recommended "undrained" earth pressures presented above over their entire height, in which case the drainage system may be omitted.

One acceptable method for backdraining an elevator pit 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 base of the wall. 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.

7.5 Seismic Design

As discussed in Section 5.2.2, the site is underlain by potentially liquefiable soil. Although the 2016 CBC calls 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. Therefore, for seismic design, we recommend Site Class D be used. The latitude and longitude for the site are 37.3315° and -121.8879°, respectively. Hence, in accordance with the 2016 CBC, we recommend the following:

- S_S = 1.50g, S_1 = 0.60g
- S_{MS} = 1.50, 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 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

9.0 LIMITATIONS

This geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report 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 recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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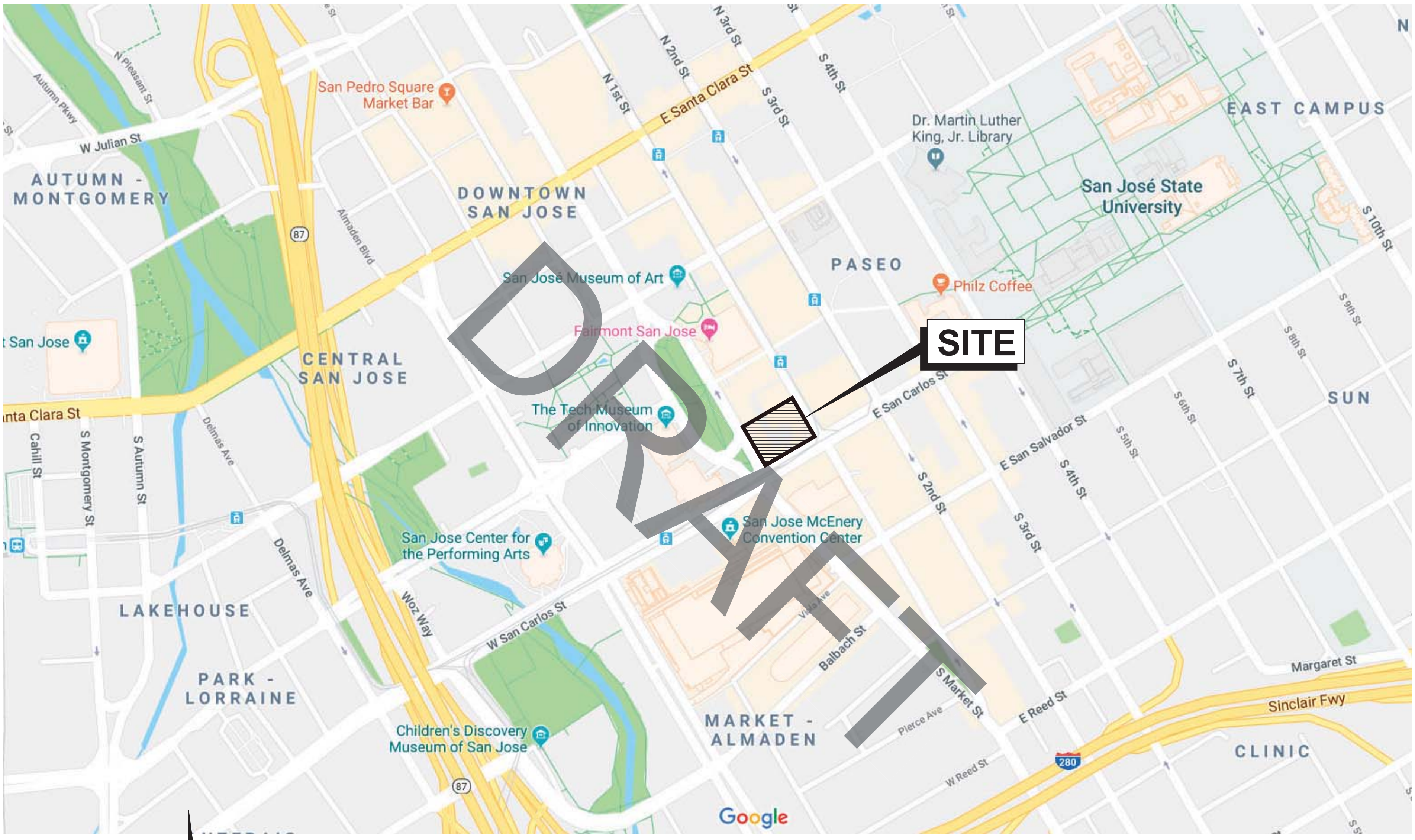
FIGURES



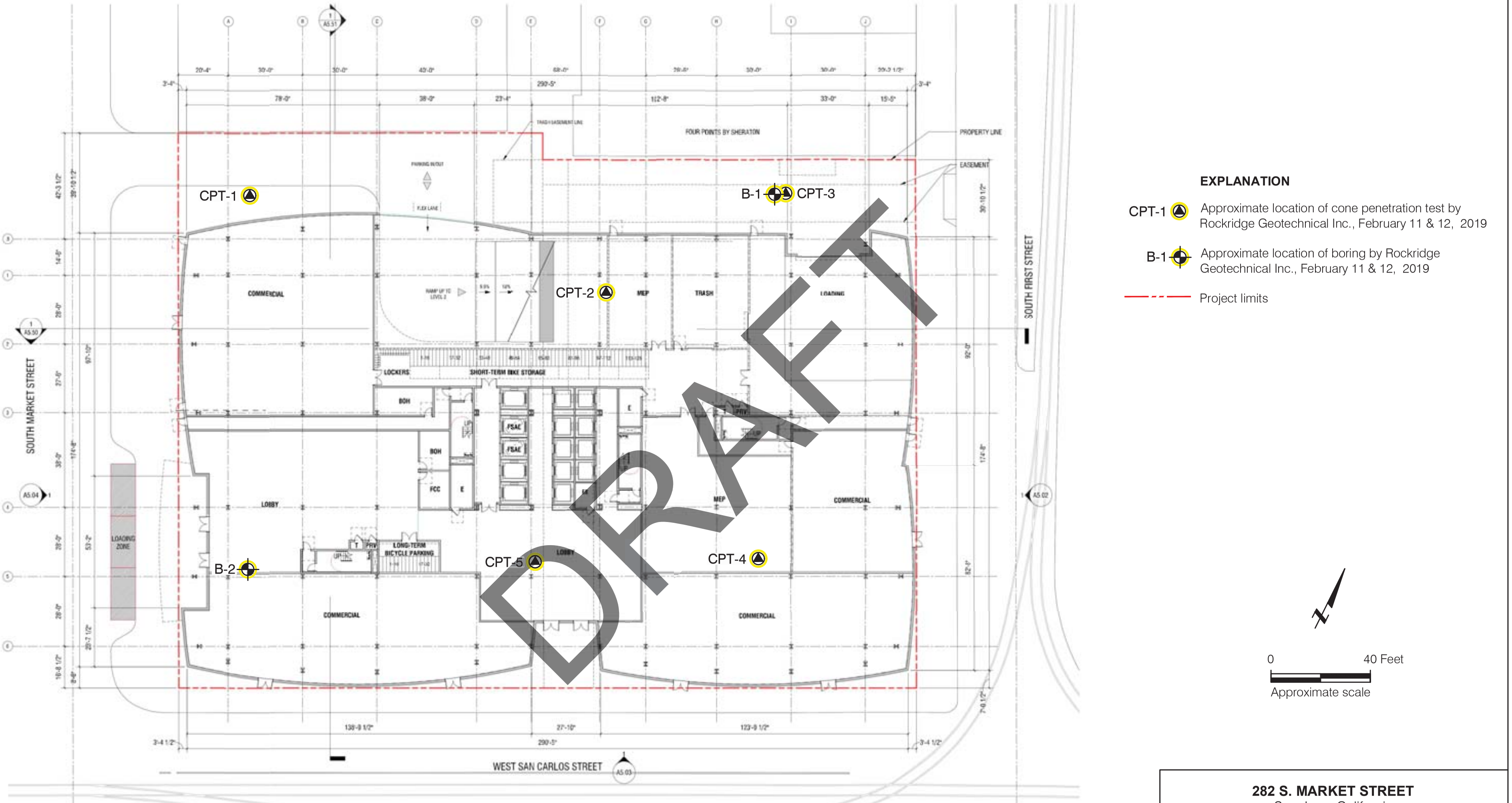
Base map: Google Map, 2018

282 S. MARKET STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 06/20/19	Project No. 18-1602	Figure 1



Reference: Base map from a drawing titled "Floor Plan - Level 1", by Arquitectonica Architect, dated September 18, 2018.

282 S. MARKET STREET
San Jose, California

SITE PLAN

Date 06/20/19 | Project No. 18-1602 | 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)
Qpa	Alluvium (Pleistocene)
fsr	Franciscan Complex melange (Eocene, Paleocene, and (or) Late Cretaceous)
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 Mile

Approximate scale

282 S. MARKET STREET
San Jose, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 06/20/19 | Project No. 18-1602 | 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

282 S. MARKET STREET
San Jose, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL FAULT MAP

Date 06/20/19 | Project No. 18-1602 | 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 West Quadrangle.
Released on February 7, 2002

0 2000 4000 Feet

Approximate scale

282 S. MARKET STREET
San Jose, California

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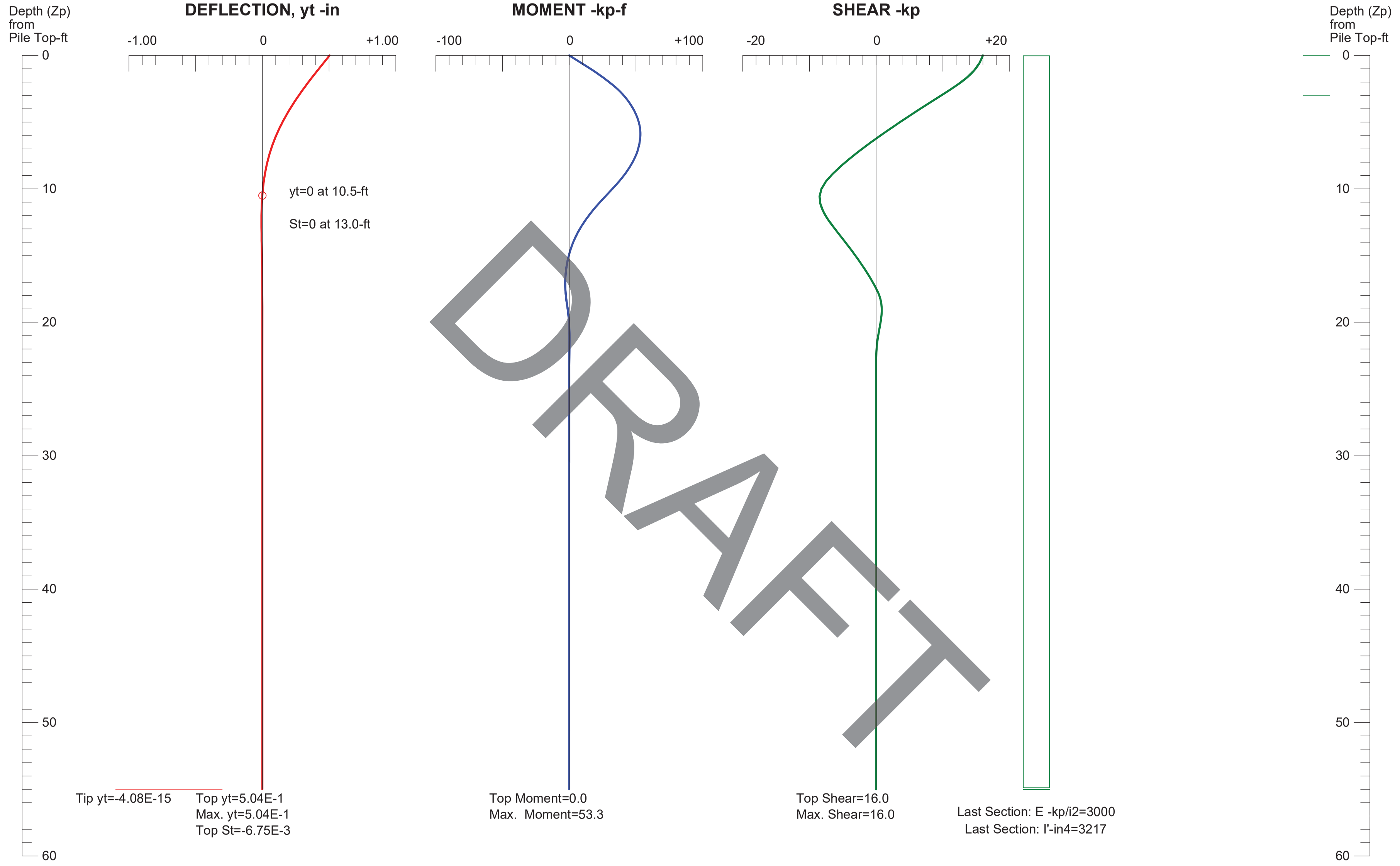
SEISMIC HAZARDS ZONE MAP

Date 06/20/19 | Project No. 18-1602 | Figure 5

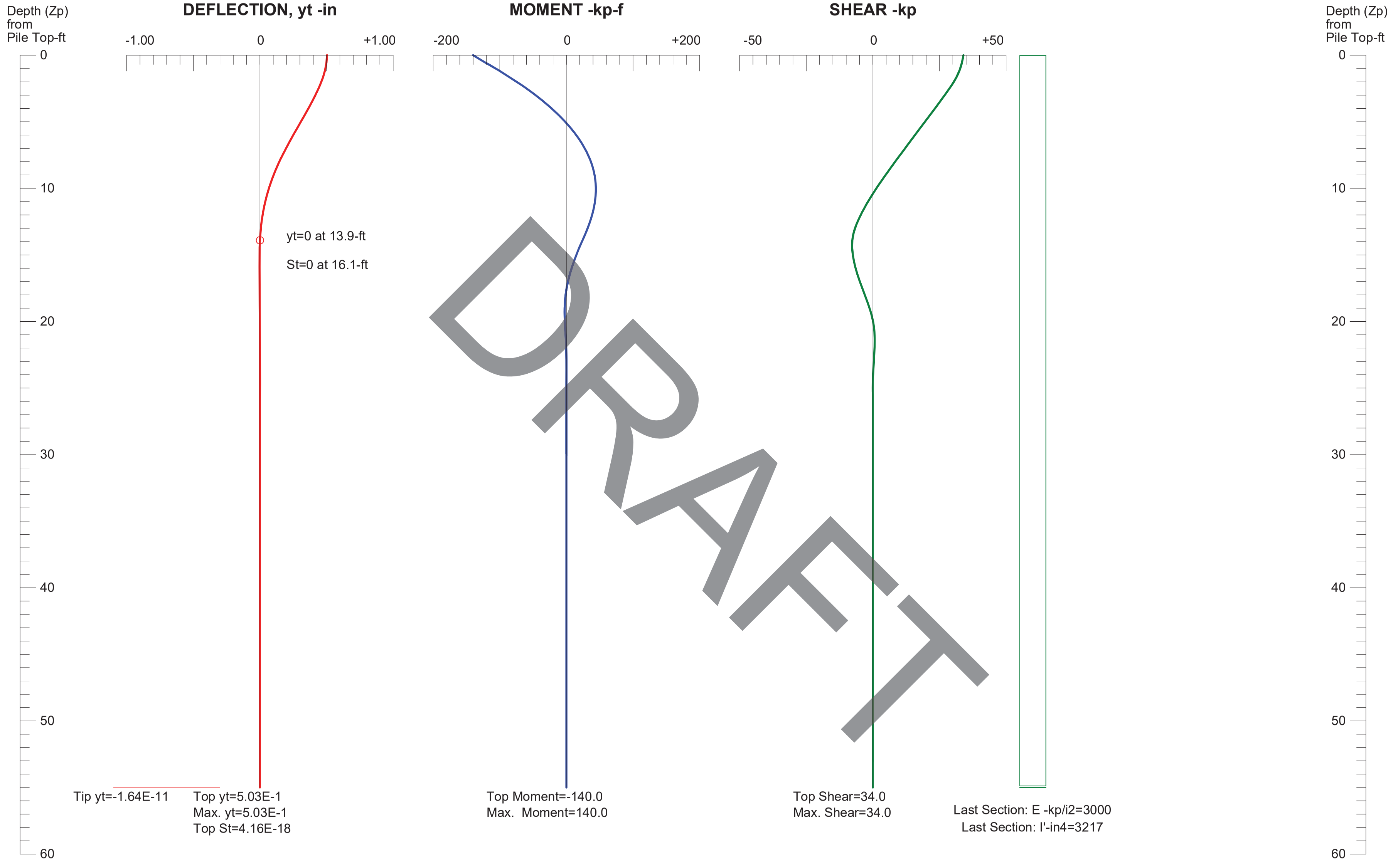


282 S. MARKET STREET
San Jose, California

ROCKRIDGE
GEOTECHNICAL

MOMENT AND DEFLECTION PROFILES
16-INCH DIAMETER AUGER CAST-IN-PLACE PILES
FREE-HEAD CONDITION

Date 06/20/19 | Project No. 18-1602 | Figure 6



282 S. MARKET STREET
San Jose, California

ROCKRIDGE
GEOTECHNICAL

MOMENT AND DEFLECTION PROFILES
16-INCH DIAMETER AUGER CAST-IN-PLACE PILES
FIXED-HEAD CONDITION

Date 06/20/19	Project No. 18-1602	Figure 7

APPENDIX A

Logs of Borings and Cone Penetration Test Results

PROJECT: **282 S. MARKET STREET**
San Jose, California

Log of Boring B-1

PAGE 1 OF 4

Boring location: See Site Plan, Figure 2
Date started: 2/11/19
Date finished: 2/11/19
Drilling method: Mud Rotary
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)

Logged by: R. Ford
Drilled by: Pitcher Services, LLC
Rig: Fraste MDXL

	SAMPLES						LABORATORY TEST DATA					
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
						4 inches of asphalt						
						4 inches of aggregate base						
1					SC	CLAYEY SAND with GRAVEL (SC) yellow, medium dense, moist, fine gravel FILL						
2	S&H		8 16 15	26								
4	S&H		4 4 6	8	CL	SANDY CLAY (CL) dark brown, medium stiff to stiff, moist, fine gravel, bricks present					18.2	103
6	SPT		3 3 3	7	CL	CLAY (CL) light brown, medium stiff, moist Particle Size Distribution; see Appendix B				87	27.3	
8					ML	SANDY SILT (ML) olive-brown, soft, moist						
9	S&H		3 2 2	3								
10					CL	CLAY with SAND (CL) olive-brown, medium stiff, moist LL = 40, PI = 20; see Appendix B						
11	S&H		6 3 3	5							34.0	90
13					SM	SILTY SAND (SM) olive brown, medium dense, moist						
14						▽ (2/11/2019; 9:00 AM)						
15					CH	CLAY (CH) gray-brown, very stiff, wet, trace sand						
16	D&M		300 psi				PP		3,500			
20					CL-ML	SANDY SILTY CLAY (CL-ML) olive-brown, medium stiff to stiff, wet, fine-grained sand Particle Size Distribution; see Appendix B LL = 24, PI = 5; see Appendix B						
21	S&H		4 4 5	8						53	22.0	105
24					CL	CLAY (CL) brown, medium stiff to stiff, wet						
25	D&M		7500 psi		SP-SM	SAND with SILT and GRAVEL (SP-SM) gray-brown, medium dense, wet, subrounded to rounded fine gravel brown Particle Size Distribution; see Appendix B						
26	SPT		10 14 10	29						10	11.1	
27												
28												
29												
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1602
Figure: A-1a

PROJECT: **282 S. MARKET STREET**
San Jose, California

Log of Boring B-1

DEPTH (feet)	SAMPLES Sampler Type	SAMPLES Sample	SAMPLES Blows/ 6"	SAMPLES SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	LABORATORY TEST DATA Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA Fines %	LABORATORY TEST DATA Natural Moisture Content, %	LABORATORY TEST DATA Dry Density Lbs/Cu Ft
31	SPT		10 7 4	13		SILTY SAND (SM) gray-brown, medium dense, wet, trace fine to coarse gravel				18	23.4	
32	SPT		8 7 8	18	SM	coarse-grained sand to pea-size gravel Particle Size Distribution; see Appendix B				25	19.4	
33												
34						SANDY CLAY (CL) gray, very stiff, wet						
35												
36	D&M		150 psi			TxUU Test; see Appendix B	TxUU	4,200	2,100		31.8	91
37												
38												
39					CL							
40												
41	S&H		4 8 11	16								
42												
43						yellow-brown						
44												
45						GRAVEL with SAND (GP)						
46	SPT		18 16 12	34		brown, dense, wet, fine to coarse-grained sand, fine subangular to subrounded gravel Particle Size Distribution; see Appendix B				5	9.1	
47					GP							
48												
49												
50						GRAVEL with SAND (GW)						
51	SPT		20 11 7	22		brown, medium dense, wet, 1.5-inch diameter subrounded gravel Particle Size Distribution; see Appendix B				4	9.1	
52												
53												
54												
55					GW							
56												
57												
58												
59												
60												

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1602

Figure: A-1b

PROJECT: **282 S. MARKET STREET**
San Jose, California

Log of Boring B-1

PAGE 3 OF 4

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
61	SPT	◪	18 18 18	43	GW	GRAVEL with SAND (GW) (continued) yellow-brown, dense						
62												
63												
64												
65												
66												
67					CL	SANDY CLAY with GRAVEL (CL) yellow-brown, very stiff, wet Consolidation Test; see Appendix B						
68	D&M	▮	500 psi								20.1	124
69												
70												
71												
72												
73												
74												
75												
76												
77												
78												
79												
80					CH	CLAY with GRAVEL (CH) olive-gray, very stiff, wet, fine subangular to subrounded gravel						
81	SPT	◪	6 9 10	23								
82												
83												
84					GP	GRAVEL with SAND (GP) olive-brown, very dense, wet, fine to coarse gravel						
85												
86												
87												
88												
89												
90												

PROJECT: **282 S. MARKET STREET**
San Jose, California

Log of Boring B-1

PAGE 4 OF 4

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	SPT	◩	36 25 27	62		GRAVEL with SAND (GP) (continued)						
92												
93												
94					GP							
95												
96												
97												
98	D&M	□	500 psi		CL	SANDY CLAY (CL) olive, hard, wet						
99	S&H	■	18 28 35	53		TxUU Test; see Appendix B	TxUU	8,500	6,100		19.7	111
100												
101												
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 100 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 14.5 feet during drilling.

---

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.84 and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1602

Figure: A-1d

PROJECT: **282 S. MARKET STREET**
San Jose, California

Log of Boring B-2

PAGE 1 OF 4

Boring location: See Site Plan, Figure 2
Date started: 2/12/19
Date finished: 2/12/19
Drilling method: Mud Rotary
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Automatic
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), Dames & Moore (D&M)

Logged by: R. Ford
Drilled by: Pitcher Services, LLC
Rig: Fraste MDXL

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value¹	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0						2.5 inches of asphalt						
1					SC	CLAYEY SAND with GRAVEL (SC) dark brown, medium dense, moist, debris present FILL						
2	S&H		16 19 16	29								
3					SM	SILTY SAND (SM) yellow-brown, loose, moist						
4	SPT		3 3 3	7								
6	S&H		5 4 5	8		LL = 21, PI = 3; see Appendix B					12.5	99
9	SPT		2 1 2	4								
10					CL	SANDY CLAY (CL) yellow-brown, stiff, moist						
11	S&H		3 6 6	10							17.5	93
15					ML	SILT (ML) olive-gray, soft to medium stiff, moist						
16	SPT		0 0 3	4	CH	CLAY (CH) gray, soft, moist, organic rich						
17	S&H		5 10 15	21	CL	CLAY (CL) gray, very stiff, wet, organic rich, trace angular gravel						
20						increase in sand content						
20					SC	CLAYEY SAND with GRAVEL (SC) gray-brown, medium dense, wet, fine gravel, trace organics						
21	SPT		5 4 5	11								
22					GP-GC	GRAVEL with SAND and CLAY (GP-GC) gray, medium dense, wet						
25					CL	CLAY (CL) brown, soft to medium stiff, wet						
26	SPT		3 1 2	4								
27	D&M		275 psi									
28					GP-GC	GRAVEL with SAND and CLAY (GP-GC) brown, medium dense, wet, subangular to rounded gravel up to 2-inch diameter						
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1602
Figure: A-2a

PROJECT: **282 S. MARKET STREET**
San Jose, California

Log of Boring B-2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31	SPT		5 5 6	13	GM	SILTY GRAVEL with SAND (GM) olive-gray, medium dense, wet, fine-grained sand Particle Size Distribution; see Appendix B				16	11.5	
33	S&H		10 9 10	16	SP-SM	SAND with SILT (SP-SM) olive-gray, medium dense, wet, coarse-grained sand, trace fine gravel Particle Size Distribution; see Appendix B				10	22.5	106
37	D&M		150 psi		CL	CLAY with SAND (CL) gray, stiff, wet Particle Size Distribution; see Appendix B				85	30.9	93
41	D&M		100 psi		CH	CLAY (CH) gray, very stiff, wet TxUU Test; see Appendix B	TxUU	4,800	2,200		34.9	87
46	S&H		4 7 10	14		trace gravel						
50					SC	CLAYEY SAND (SC) gray, medium dense, wet						
51	D&M		300 psi		GP	GRAVEL with SAND (GP) yellow-brown, very dense, wet						
52–60												

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1602

Figure: A-2b

PROJECT: **282 S. MARKET STREET**
San Jose, California

Log of Boring B-2

PAGE 3 OF 4

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT		17 24 26	61	GP SP	GRAVEL with SAND (GP) (continued) SAND (SP) yellow-brown, very dense, wet						
61												
62						GRAVEL with SAND (GP) yellow-brown, very dense, wet, subangular to subrounded fine gravel						
63												
64												
65												
66												
67												
68												
69												
70	SPT		24 21 17	46	GP	dense, coarse-grained sand						
71												
72												
73												
74												
75												
76												
77												
78												
79												
80	D&M		450 psi			CLAY (CL) gray, stiff, wet Consolidation Test; see Appendix B TxUU Test; see Appendix B	TxUU	10,000	1,945		25.5 25.7	97 116
81												
82												
83												
84					CL							
85												
86												
87												
88					GP	GRAVEL with SAND (GP) yellow-brown, very dense, wet, subrounded to subangular fine gravel, coarse-grained sand						
89												
90												

PROJECT: **282 S. MARKET STREET**
San Jose, California

Log of Boring B-2

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
91	SPT	◪	31 30 26	67		GRAVEL with SAND (GP) (continued)						
92												
93												
94					GP							
95												
96												
97												
98												
99						SAND with SILT and GRAVEL (SP-SM) yellow-brown, medium dense, wet						
100					SP-SM	Particle Size Distribution; see Appendix B						
101	SPT	◪	28							7	10.4	
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 101.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] S&H and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.84 and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 18-1602

Figure: A-2d

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

282 S. MARKET STREET
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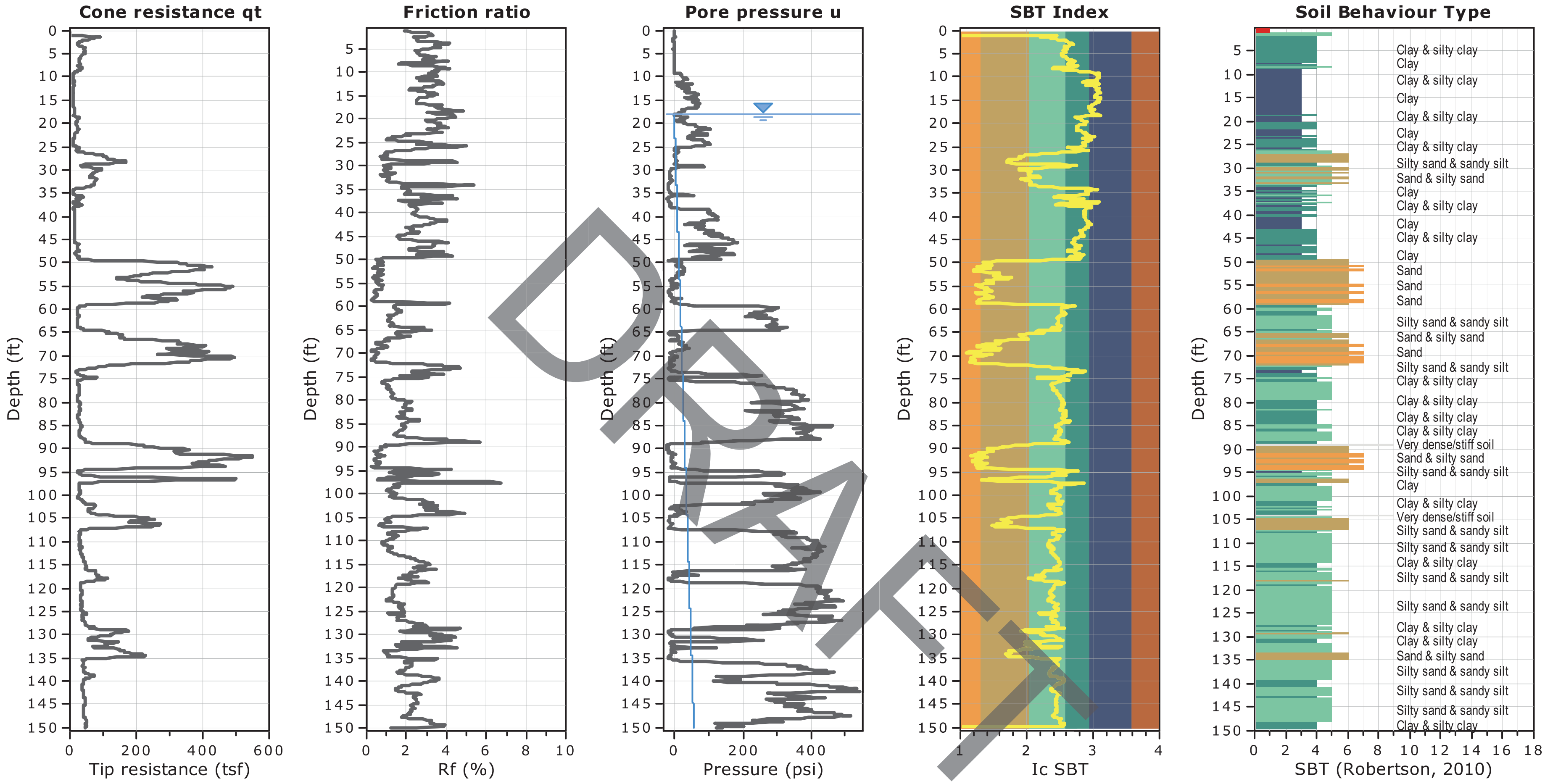
CLASSIFICATION CHART

Date 06/20/19 | Project No. 18-1602 | 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

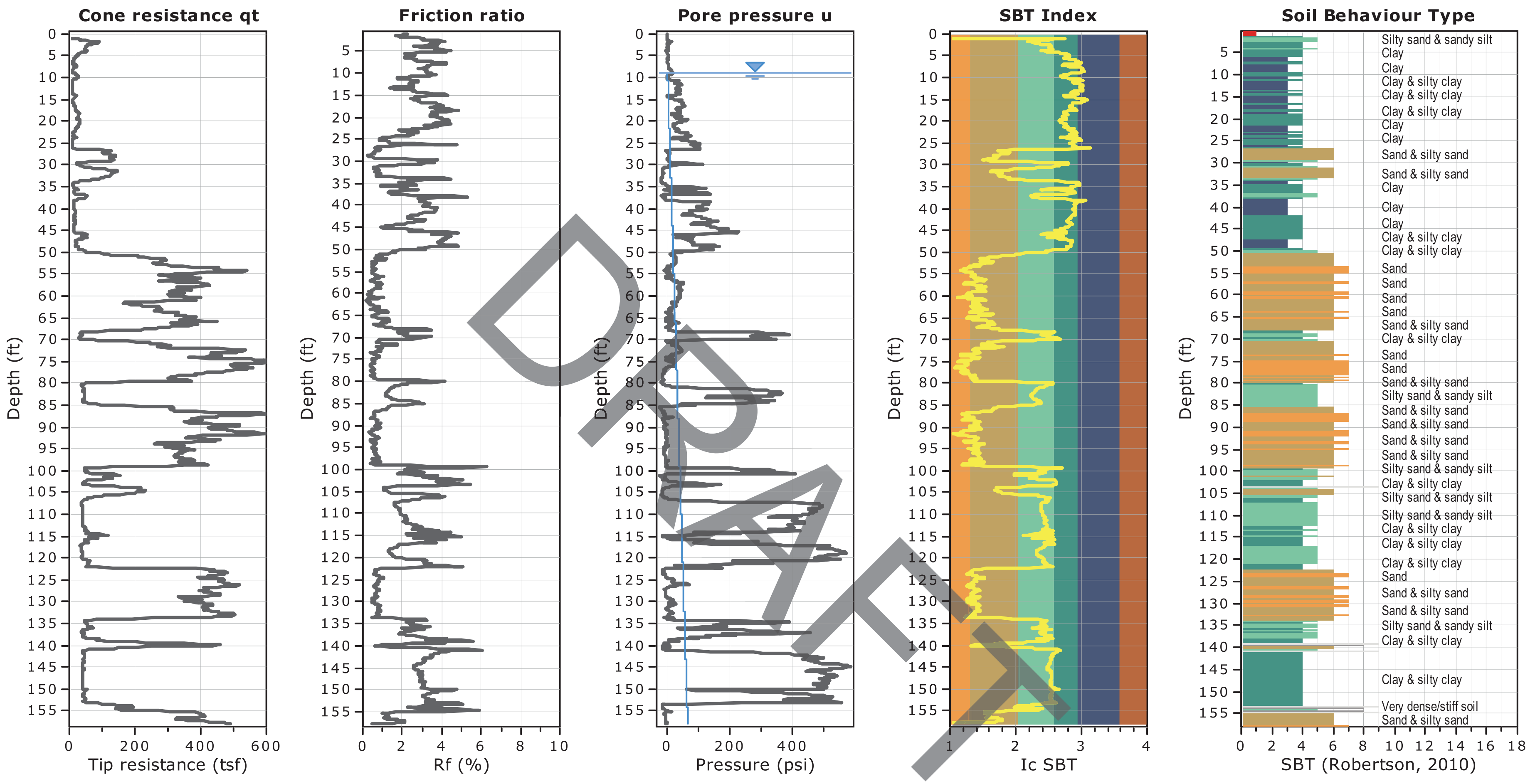
Total depth: 150.10 ft, Date: 2/11/2019
Depth to Groundwater: 18 feet (pore pressure dissipation test performed 89.7 ft bgs)
Cone Operator: Cone Tec

282 S. MARKET STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 06/20/19 | Project No. 18-1602 | Figure A-4



Total depth: 157.97 ft, Date: 2/11/2019
Depth to Groundwater: 9 feet (pore pressure dissipation test performed 140.3 ft bgs)
Cone Operator: Cone Tec

SBT legend

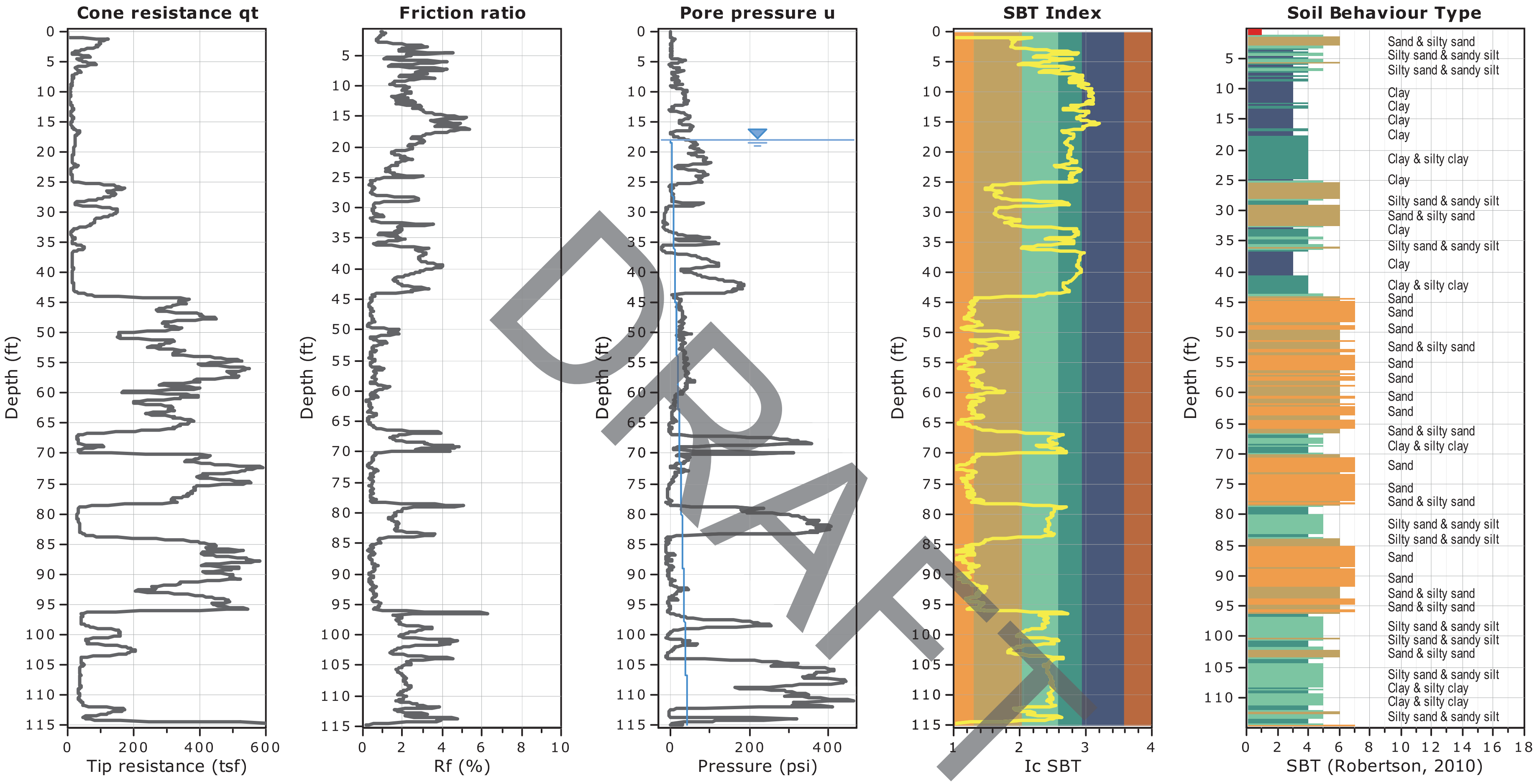
🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

282 S. MARKET STREET
San Jose, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 06/20/19	Project No. 18-1602	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

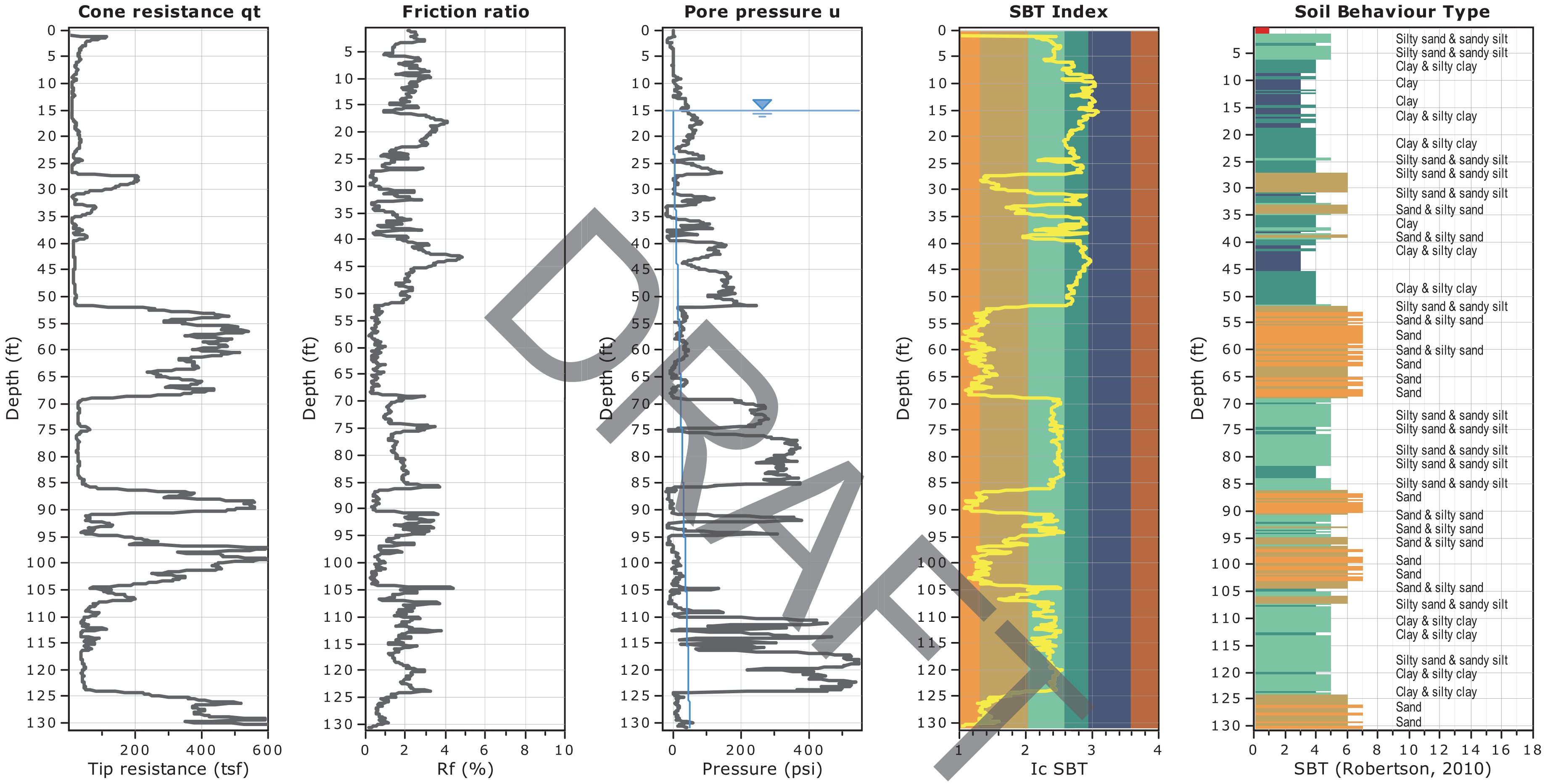
Total depth: 114.99 ft, Date: 2/11/2019
Depth to Groundwater: 18 feet (pore pressure dissipation test performed 86.7 ft bgs)
Cone Operator: Cone Tec

282 S. MARKET STREET
San Jose, California

ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 06/20/19	Project No. 18-1602	Figure A-6



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: 130.82 ft, Date: 2/11/2019
Depth to Groundwater: 15 feet (pore pressure dissipation test performed 86.3 ft bgs)
Cone Operator: Cone Tec

282 S. MARKET STREET
San Jose, California

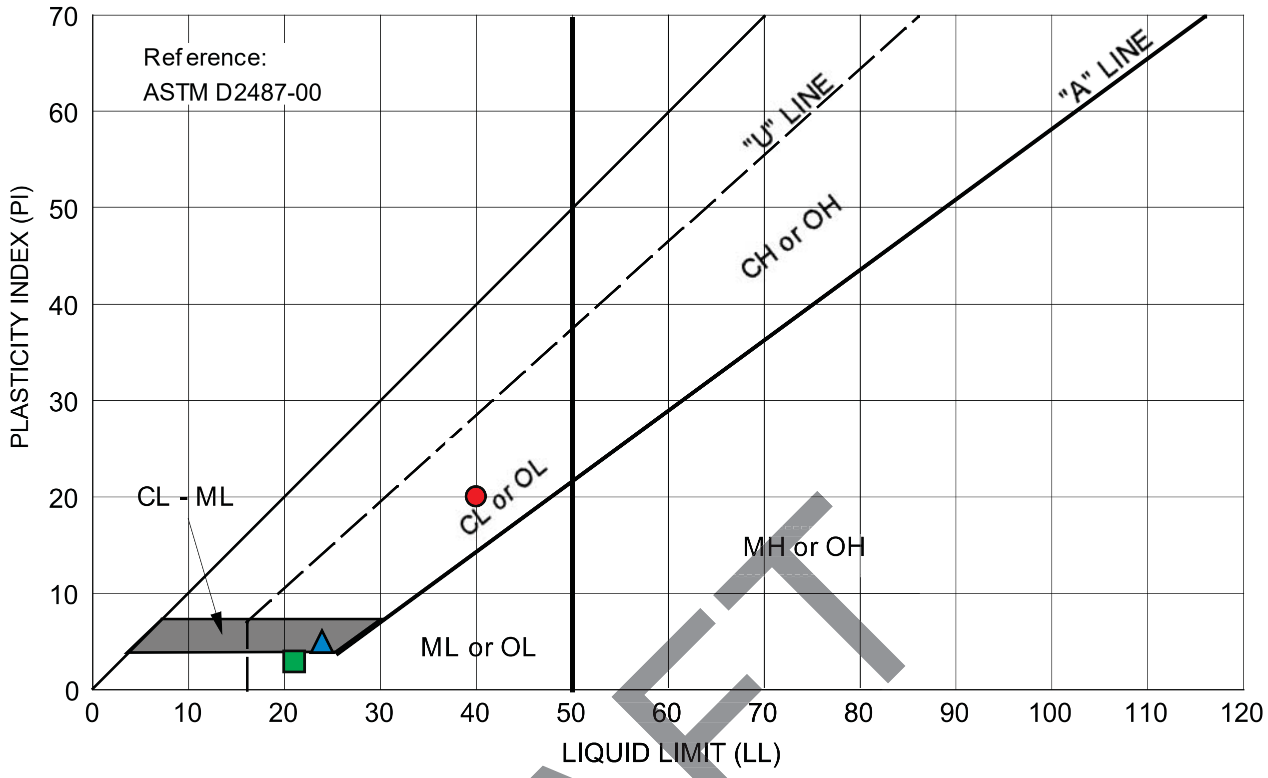
ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-5

Date 06/20/19 | Project No. 18-1602 | Figure A-7

APPENDIX B

Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 10.5 feet	CLAY with SAND (CL), olive-brown	34.0	40	20	--
▲	B-1 at 20.5 feet	SANDY SILTY CLAY (CL-ML), olive-brown	22.0	24	5	53
🟩	B-2 at 6.0 feet	SILTY SAND (SM), yellow-brown	12.5	21	3	--

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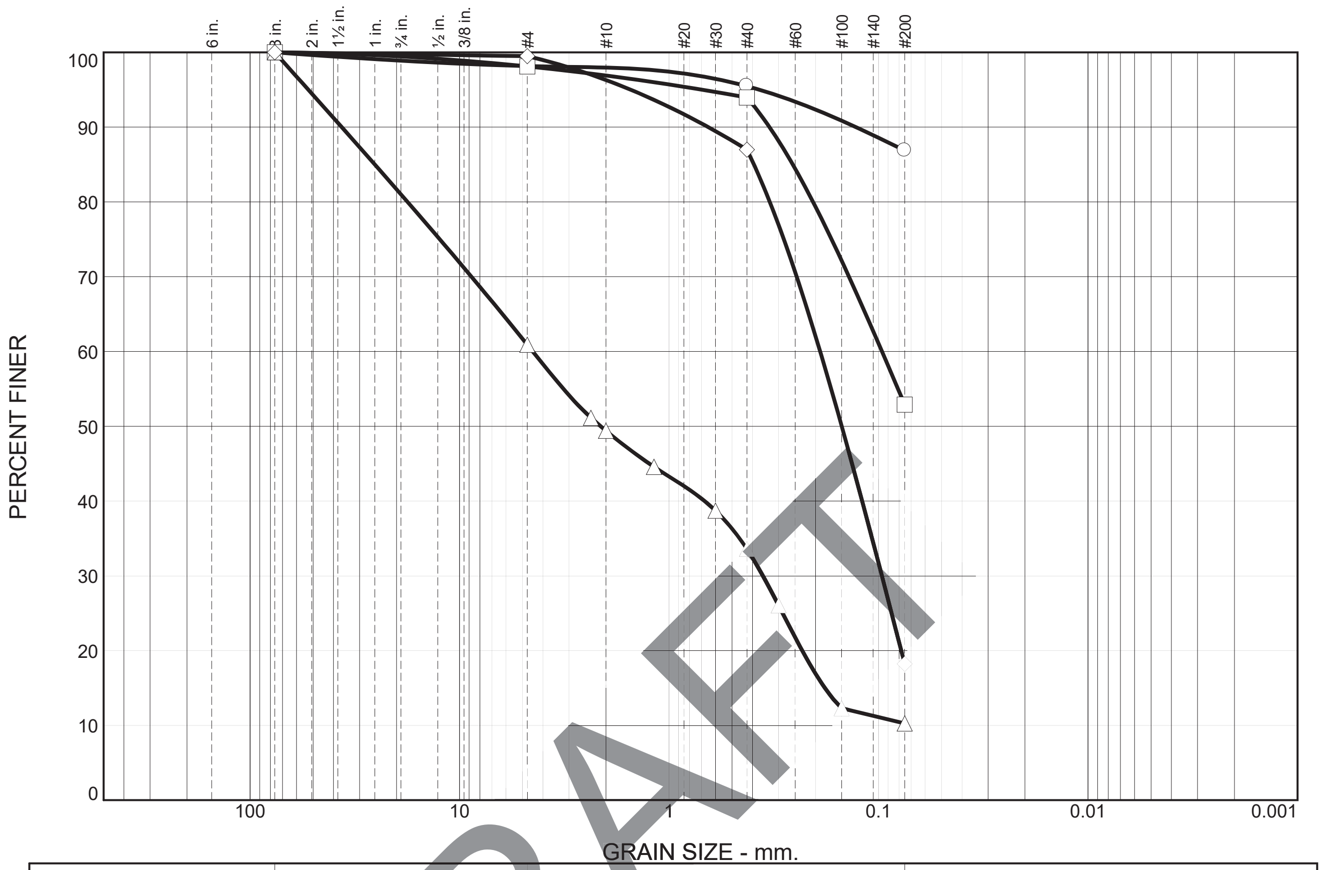
PLASTICITY CHART

Date 06/20/19 | Project No. 18-1602 | Figure B-1



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	1.1	0.7	0.3	2.4	8.6	86.9	
□	0.0	0.6	1.3	1.2	2.9	41.1	52.9	
△	0.0	19.0	20.1	11.5	15.8	23.3	10.3	
◇	0.0	0.3	0.2	3.2	9.3	68.8	18.2	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	5.0 - 6.5'	CLAY, light brown	CL
□	B-1	20.5'	SANDY SILTY CLAY, olive-brown	CL-ML
△	B-1	25.5 - 27.0'	SAND with SILT and GRAVEL, gray-brown	SP-SM
◇	B-1	30.0 - 31.5'	SILTY SAND, gray-brown	SM

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PARTICLE SIZE DISTRIBUTION REPORT

Date 06/20/19 | Project No. 18-1602 | Figure B-2

	% +3"	% Gravel		% Sand			% Fines	
		Coarse	**Fine**	**Coarse**	**Medium**	**Fine**	**Silt**	**Clay**
○	0.0	0.7	0.8	4.0	11.4	57.9	25.2	
□	0.0	24.1	25.1	14.3	18.8	12.8	4.9	
△	0.0	40.2	32.4	12.3	6.8	4.2	4.1	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	31.5 - 33.0'	SILTY SAND, gray-brown	SM
□	B-1	45.0 - 46.5'	GRAVEL with SAND, brown	GP
△	B-1	50.0 - 51.5'	GRAVEL with SAND, brown	GW

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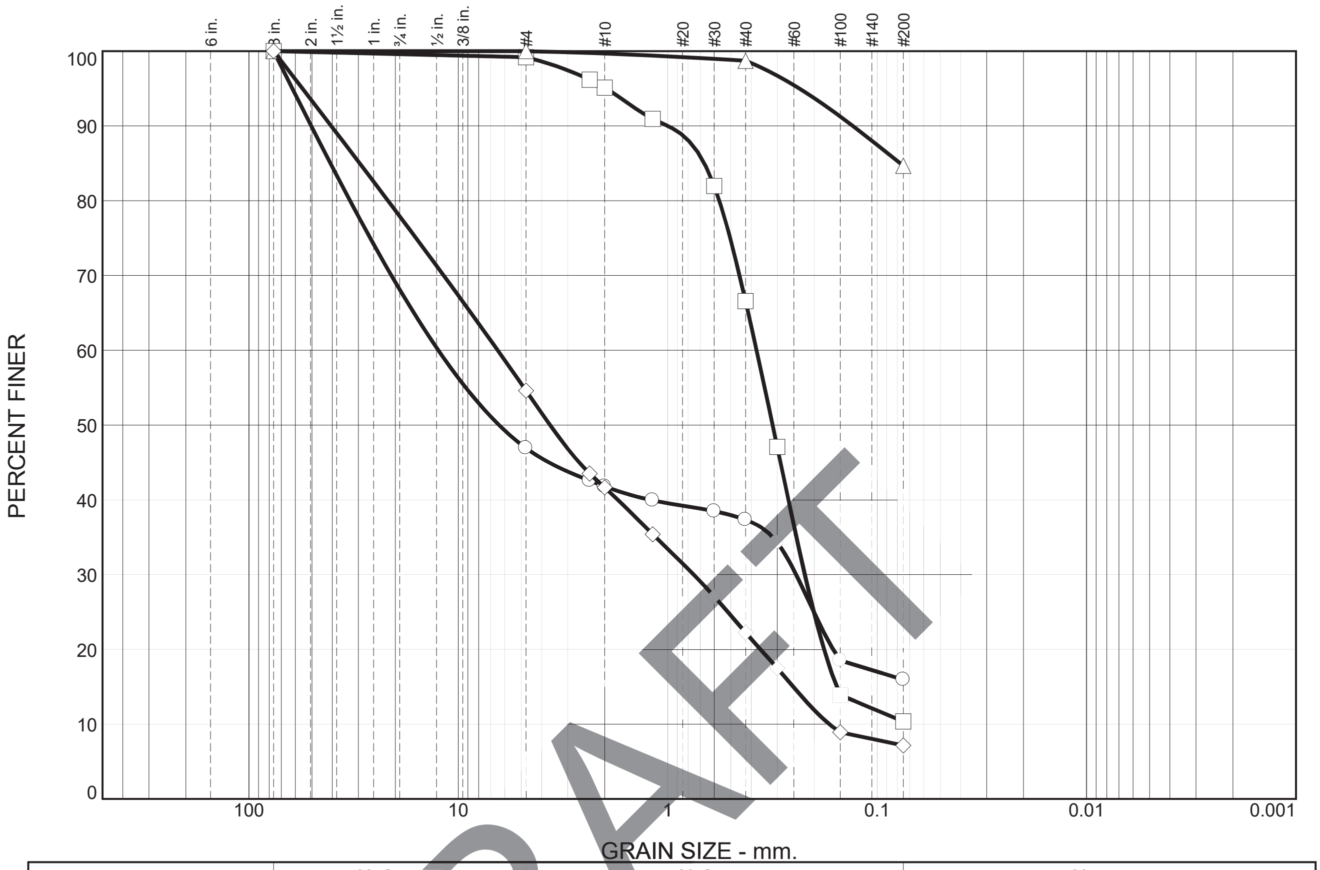
PARTICLE SIZE DISTRIBUTION REPORT

Date 06/20/19 | Project No. 18-1602 | Figure B-3



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	31.9	21.2	5.1	4.5	21.4	15.9	
□	0.0	0.4	0.4	4.1	28.5	56.2	10.4	
△	0.0	0.0	0.0	0.3	1.0	14.0	84.7	
◇	0.0	22.1	23.3	12.9	19.4	15.1	7.2	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-2	30.0 - 31.5'	SILTY GRAVEL with SAND, olive-gray	GM
□	B-2	32.5'	SAND with SILT, olive-gray	SP-SM
△	B-2	36.0'	CLAY with SAND, gray	CL
◇	B-2	100.0 - 101.5'	SAND with SILT and GRAVEL, yellow-brown	SP-SM

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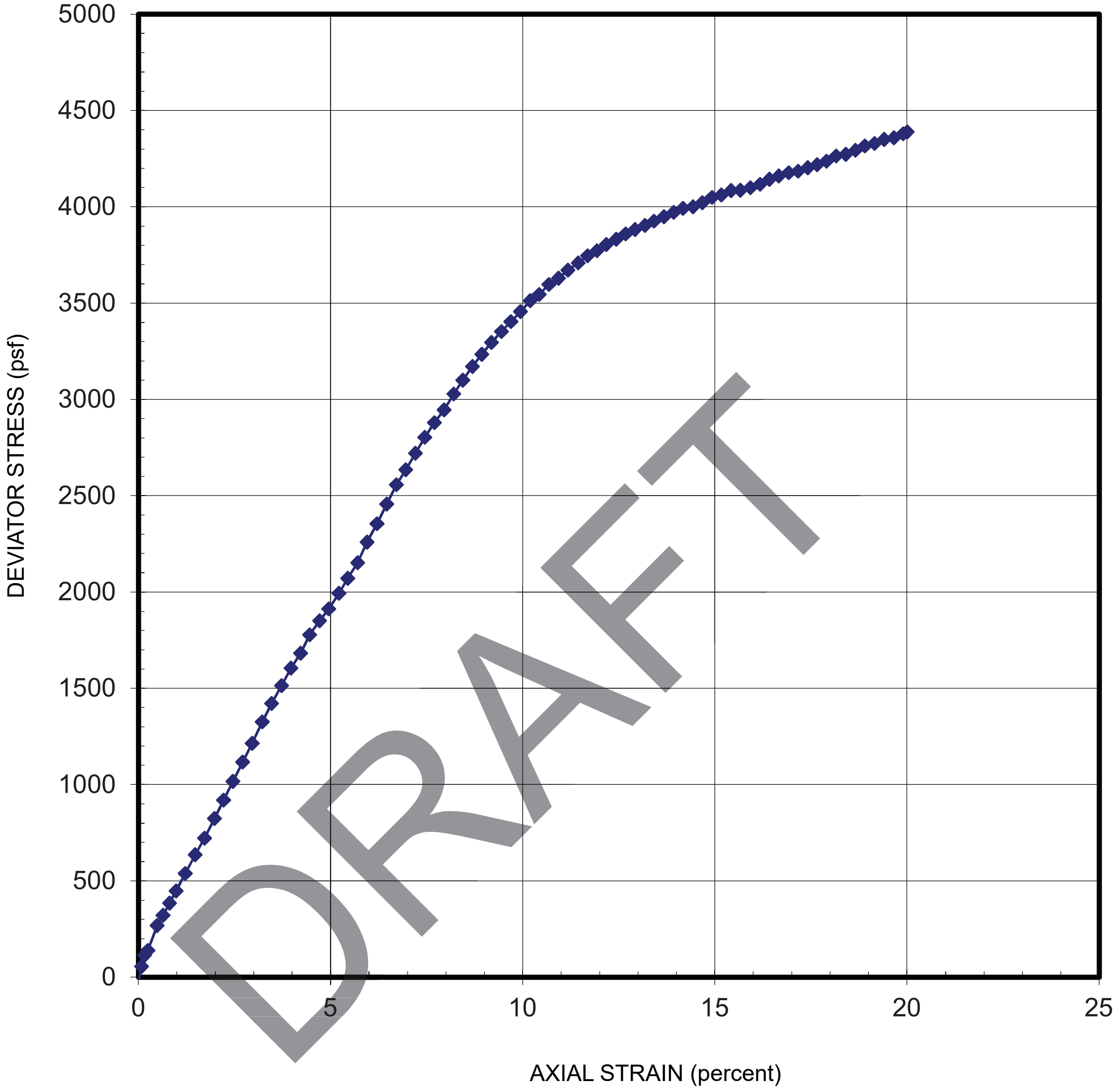
PARTICLE SIZE DISTRIBUTION REPORT

Date 06/20/19 | Project No. 18-1602 | Figure B-4



SAMPLER TYPE Dames & Moore

DIAMETER (in.) 2.40

HEIGHT (in.) 5.22

MOISTURE CONTENT 31.8 %

DRY DENSITY 91 pcf

SHEAR STRENGTH 2,100 psf

STRAIN AT FAILURE 20.0 %

CONFINING PRESSURE 4,200 psf

STRAIN RATE 1 % / min.

DESCRIPTION SANDY CLAY (CH), gray

SOURCE B-1 at 35 feet

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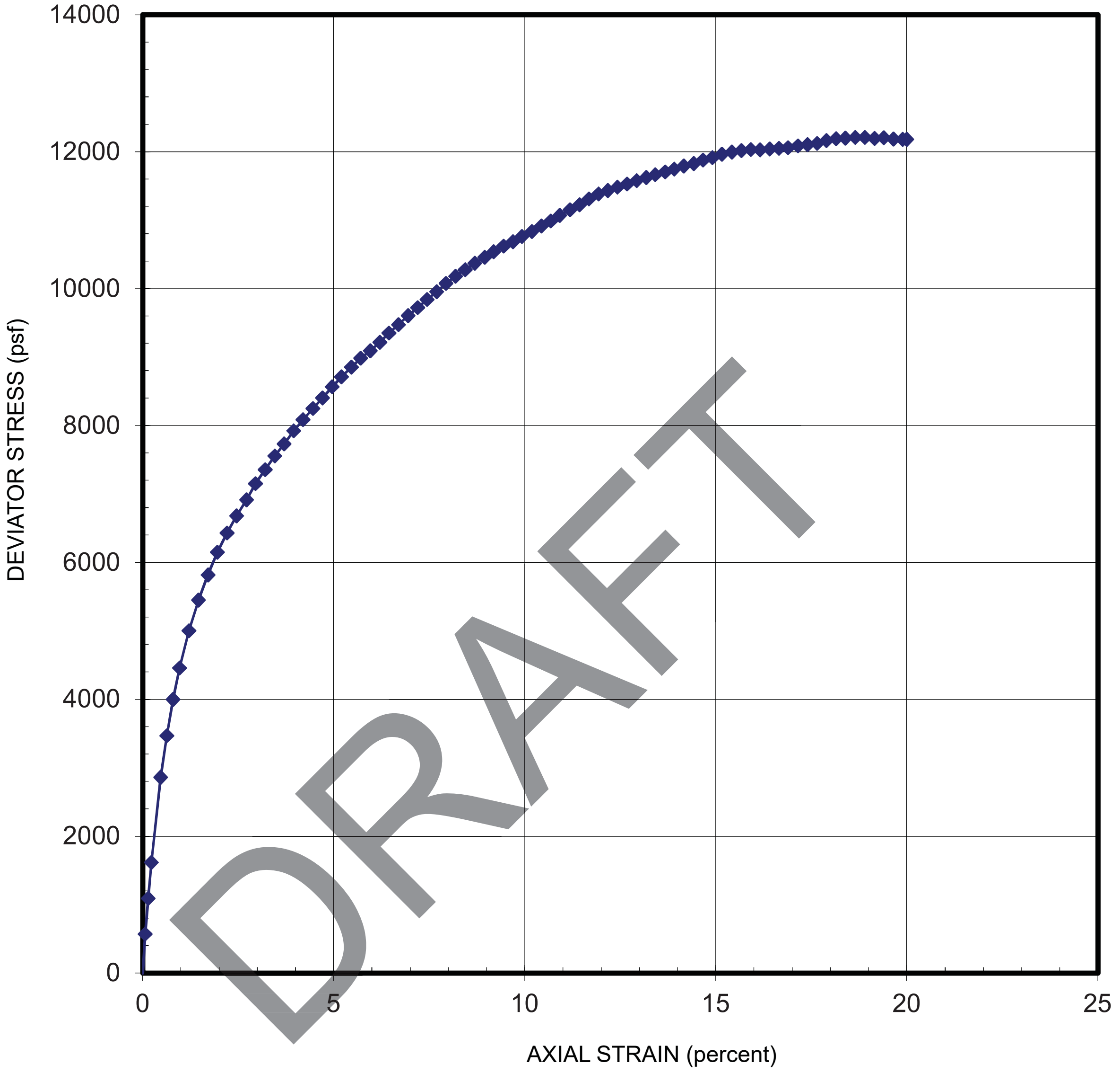
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 06/20/19

Project No. 18-1602

Figure B-5



SAMPLER TYPE	Sprague and Henwood	SHEAR STRENGTH	6,100 psf
DIAMETER (in.) 2.40	HEIGHT (in.) 6.02	STRAIN AT FAILURE	18.9 %
MOISTURE CONTENT	19.7 %	CONFINING PRESSURE	8,500 psf
DRY DENSITY	111 pcf	STRAIN RATE	1 % / min.
DESCRIPTION	SANDY CLAY (CL), olive	SOURCE	B-1 at 99.5 feet

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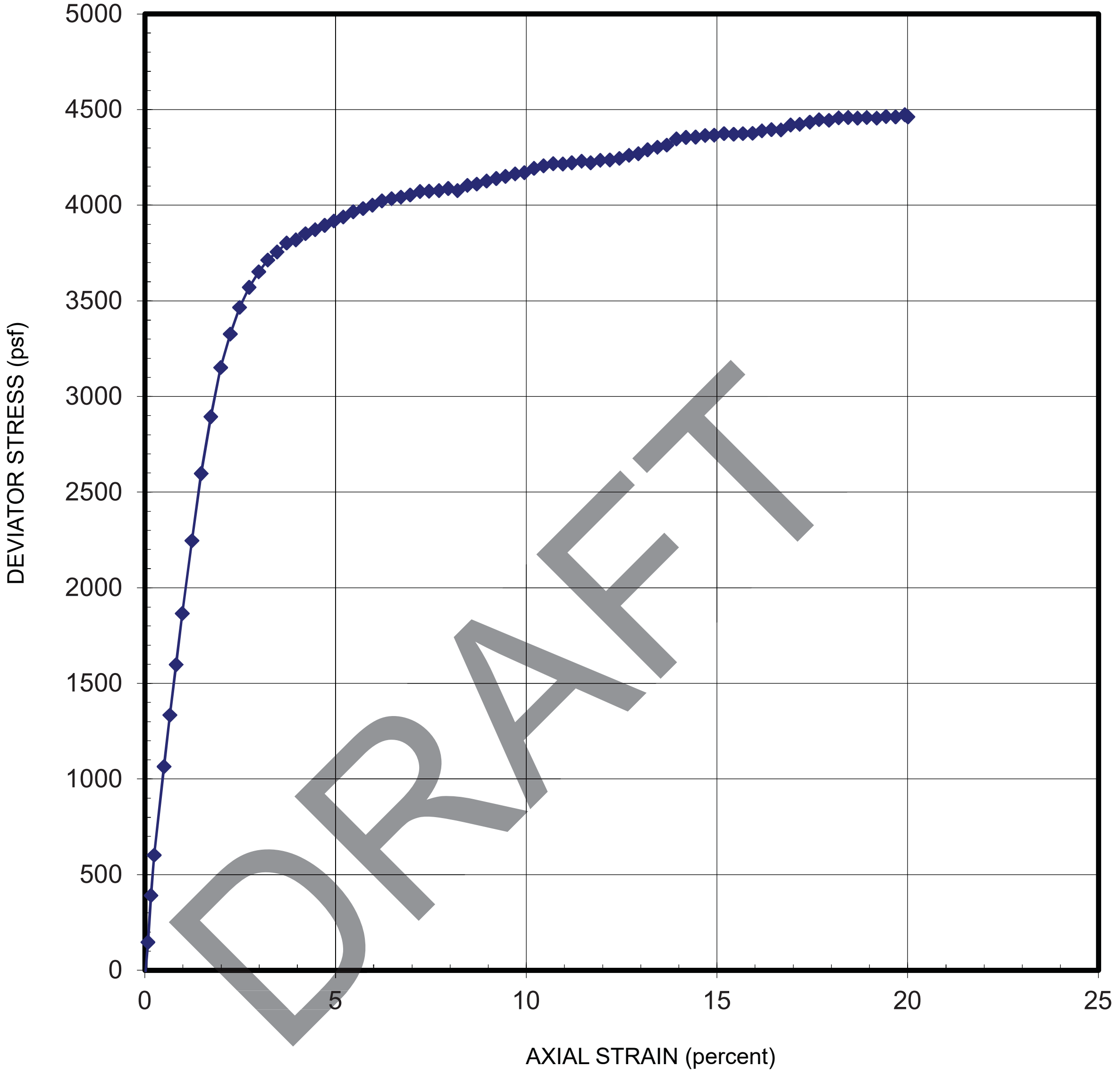
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 06/20/19 | Project No. 18-1602 | Figure B-6



SAMPLER TYPE	Dames & Moore			SHEAR STRENGTH	2,200 psf
DIAMETER (in.)	2.39	HEIGHT (in.)	5.30	STRAIN AT FAILURE	20.0 %
MOISTURE CONTENT	34.9 %			CONFINING PRESSURE	4,800 psf
DRY DENSITY	87 pcf			STRAIN RATE	1 % / min.
DESCRIPTION	CLAY (CH), gray			SOURCE	B-2 at 40 feet

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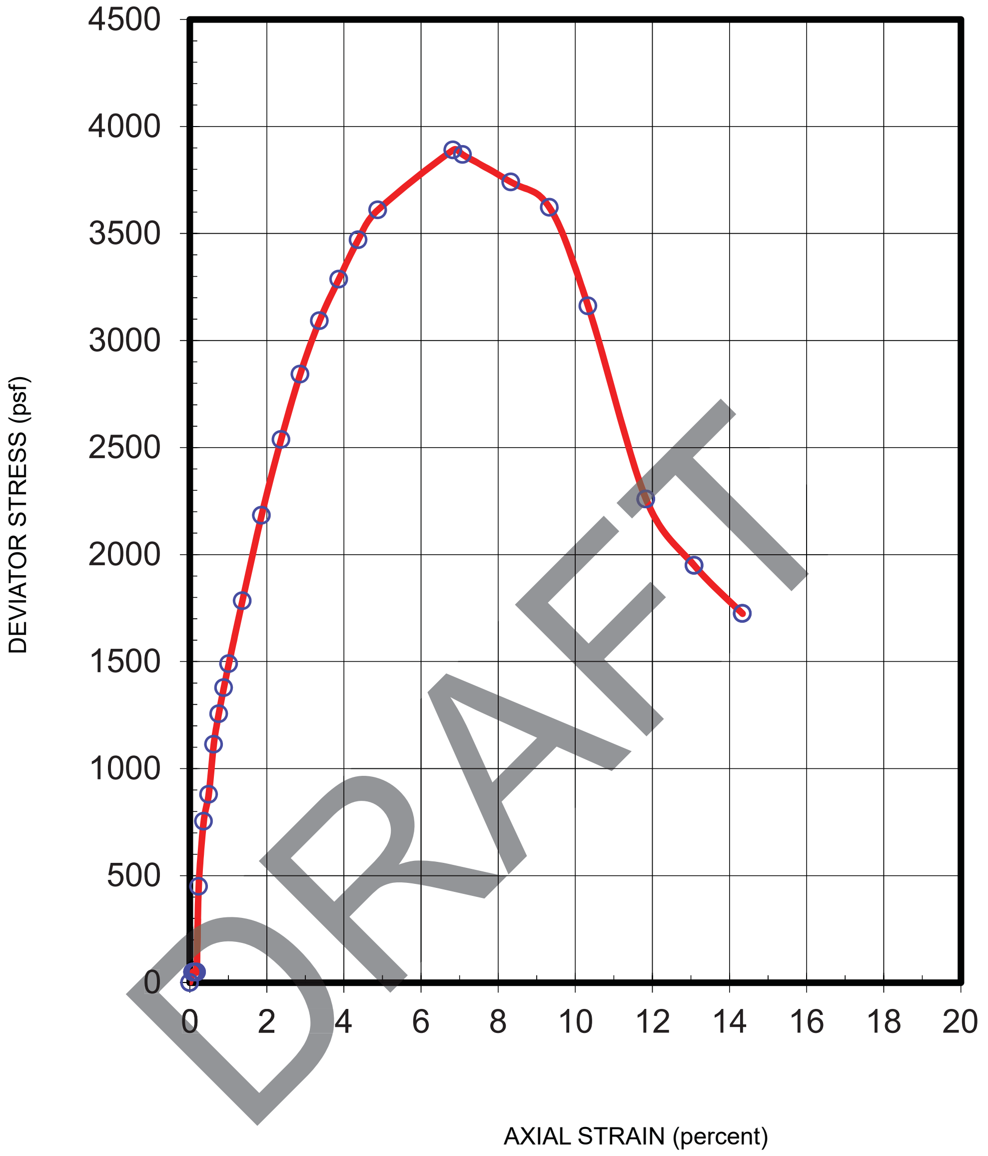
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 06/20/19 | Project No. 18-1602 | Figure B-7



SAMPLER TYPE	Dames & Moore	SHEAR STRENGTH	1,945 psf
DIAMETER (in.) 2.43	HEIGHT (in.) 5.75	STRAIN AT FAILURE	6.83 %
MOISTURE CONTENT	25.5 %	CONFINING PRESSURE	1,000 psf
DRY DENSITY	96.8 pcf	STRAIN RATE	0.50 % / min.
DESCRIPTION	CLAY (CL), gray	SOURCE	B-2 at 80 feet

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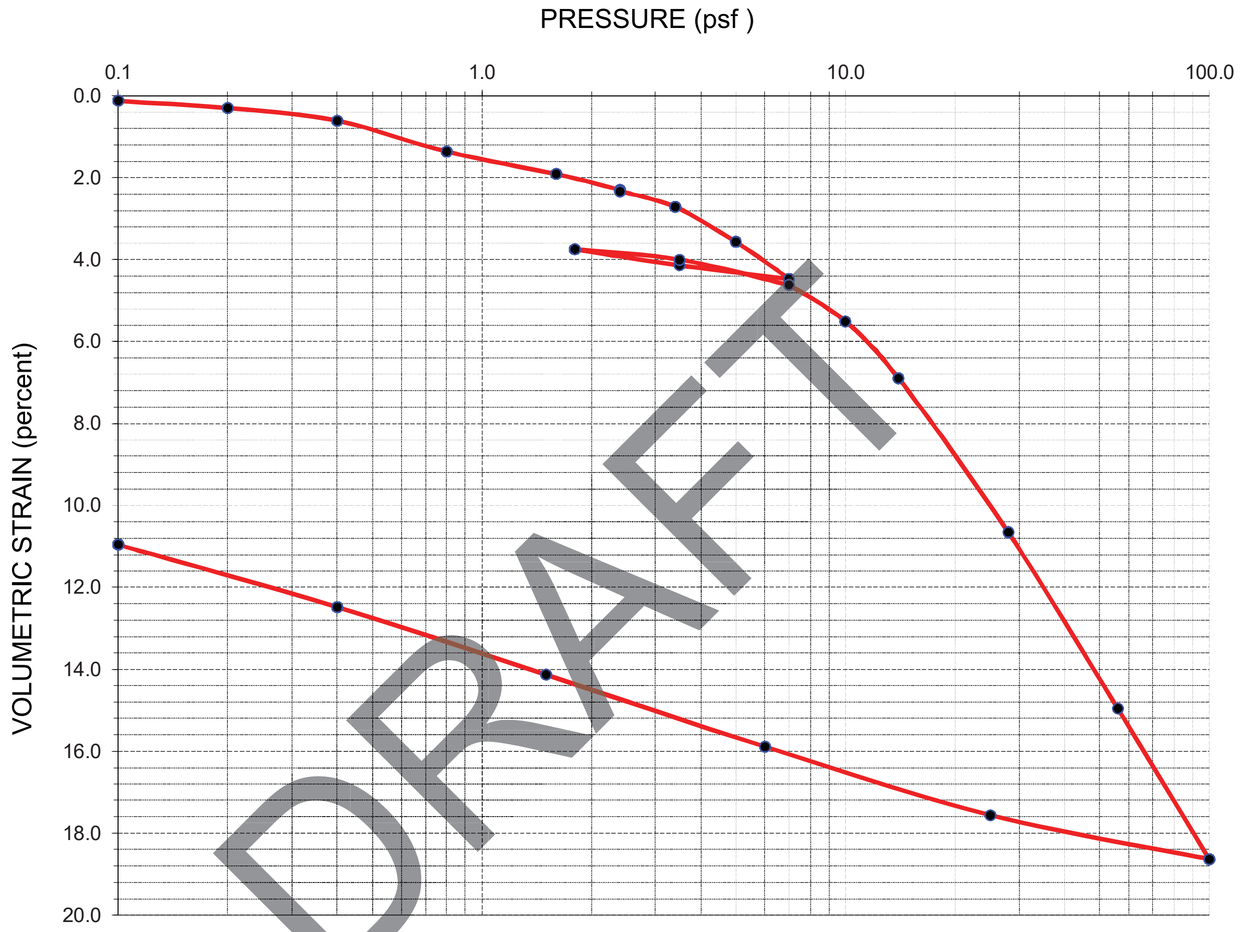
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 06/20/19 | Project No. 18-1602 | Figure B-8



Sampler Type	Dames & Moore (D&M)	Condition		Before test		After test
Diameter (in) 2.42	Height (in) 1.00	Water Content	w_o	20.1 %	w_f	16.8 %
Overburden Pressure, P_o	3,474 psf	Void Ratio	e_o	0.630	e_f	0.452
Preconsol. Pressure, P_c	12,000 psf	Saturation	S_o	86.2 %	S_f	100.2 %
Compression Ratio, $C_{\varepsilon c}$	0.16	Dry Density	γ_d	124.3 pcf	γ_d	135.6 pcf
Recompression Ratio, $C_{\varepsilon r}$	0.013	LL --	PL --	PI --	G_s	2.70 (assumed)
Description:	SANDY CLAY with GRAVEL (CL), yellow-brown	Source	B-1 at 67.5 feet			

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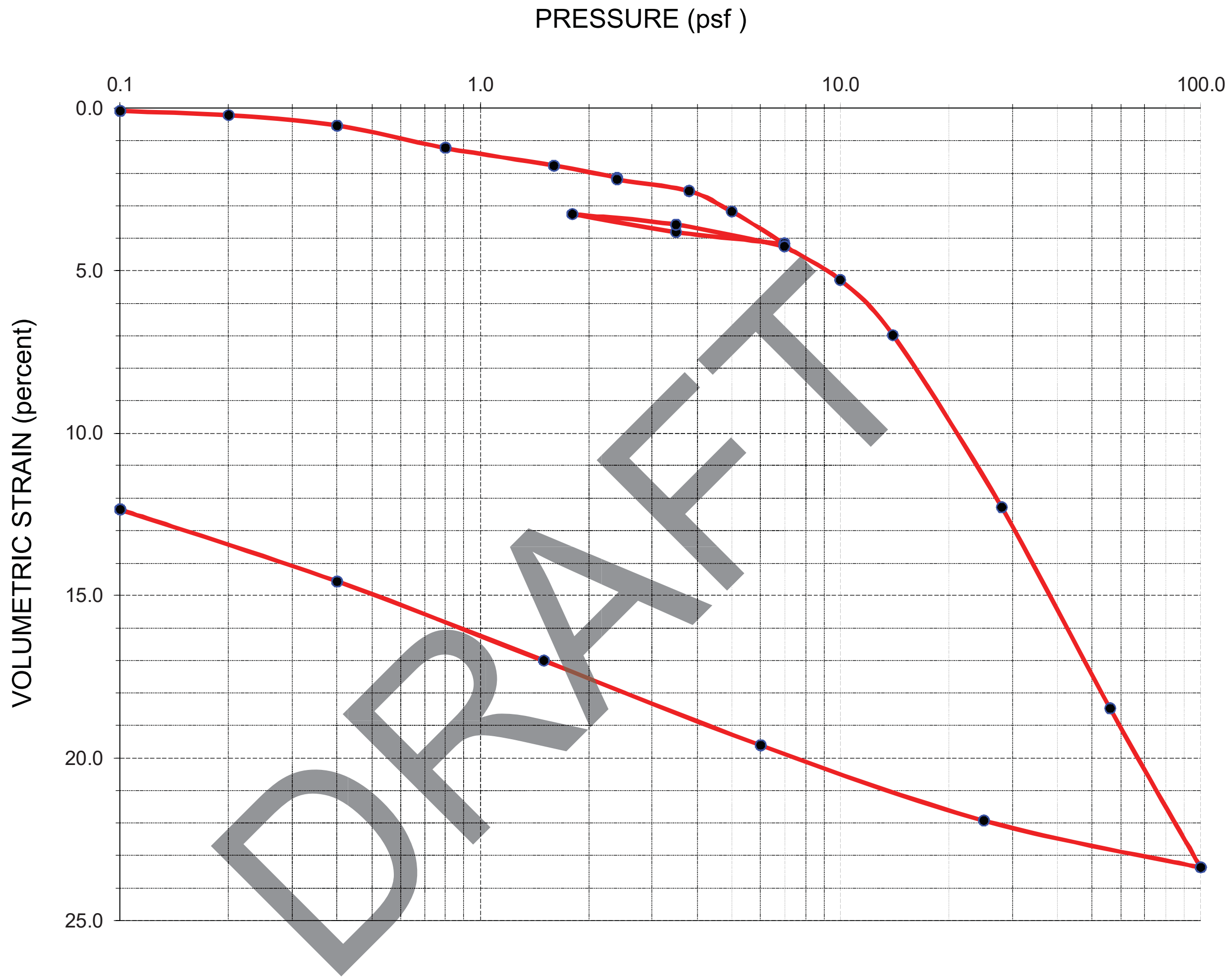
CONSOLIDATION TEST REPORT

Date 06/20/19 | Project No. 18-1602 | Figure B-9



Sampler Type	Dames & Moore (D&M)		Condition		Before test		After test
Diameter (in) 2.42	Height (in) 1.00		Water Content	w_o	25.7 %	w_f	22.1 %
Overburden Pressure, P_o	3,944	psf	Void Ratio	e_o	0.819	e_f	0.596
Preconsol. Pressure, P_c	11,000	psf	Saturation	S_o	84.5 %	S_f	100.4 %
Compression Ratio, $C_{\varepsilon c}$	0.23		Dry Density	γ_d	116.5 pcf	γ_d	129.1 pcf
Recompression Ratio, $C_{\varepsilon r}$	0.019	LL --	PL --	PI --	G_s 2.70 (assumed)		
Description: CLAY (CL), gray				Source B-2 at 80 feet			

282 S. MARKET STREET
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CONSOLIDATION TEST REPORT

Date 06/20/19 | Project No. 18-1602 | Figure B-10

Results Only Soil Testing
for
282 S. Market Street

February 20, 2019

Prepared for:
Katie Dickinson
Rockridge Geotechnical
270 Grand Ave,
Oakland, CA 94610
ksdickinson@rockridgegeo.com

Project X Job#: S190215A
Client Job or PO#: 18-1602

Soil Analysis Lab Results

Client: Rockridge Geotechnical
Job Name: 282 S. Market Street
Client Job Number: 18-1602
Project X Job Number: S190215A
February 20, 2019

	Method	ASTM G187		ASTM D516		ASTM D512B		SM 4500-NO3-E	SM 4500-NH3-C	SM 4500-S2-D	ASTM G200	ASTM G51
Bore# / Description	**Depth**	**Resistivity**		**Sulfates**		**Chlorides**		**Nitrate**	**Ammonia**	**Sulfide**	**Redox**	**pH**
		As Rec'd	Minimum									
	(ft)	(Ohm-cm)	(Ohm-cm)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(mg/kg)	(mg/kg)	(mg/kg)	(mV)	
B-2-2	3.0-4.5	2,278	1,273	150	0.0150	27	0.0027	30	1.3	0.09	153	8.23

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.

Prepared by,

Nathan Jacob
Lab Technician

Respectfully Submitted,

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