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GEOTECHNICAL

Prepared for **San Pablo Investors Two, LLC**

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE BUILDING
2136 – 2154 SAN PABLO AVENUE
BERKELEY, CALIFORNIA

October 23, 2019
Project No. 19-1740

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ROCKRIDGE GEOTECHNICAL

October 23, 2019

San Pablo Investors Two, LLC
c/o Realtex Inc.
505 Sansome Street, Suite 400
San Francisco, California 94111

Attention: Mr. Tomas Janik, Senior Project Manager

Subject: Final Report
Geotechnical Investigation
Proposed Mixed-Use Building
2136 – 2154 San Pablo Avenue
Berkeley, California

Dear Mr. Janik,

We are pleased to present our final geotechnical report for the proposed mixed-use building to be constructed at 2136 – 2154 San Pablo Avenue in Berkeley, California. Our geotechnical investigation was performed in accordance with our proposal dated June 7, 2019 and our Agreement with San Pablo Investors Two, LLC, dated August 13, 2019.

The project site is a rectangular-shaped lot encompassing approximately 23,000 square feet. The site is bordered by San Pablo Avenue to the east, a small, one-story structure and paved parking lot to the north, residential units and a small park to the west, and a one-story commercial building, a parking lot, and bee-keeping equipment to the south. The eastern portion of the site is currently occupied by a two-story commercial building with car lifts and a gravel parking lot and driveway in the rear.

Plans are to construct a 126-unit residential building that will occupy most of the site. The building will consist of 3 to 5 stories of residential units over a one-story concrete podium. The ground floor will be at grade and will include parking lift pits, gym space, and residential lofts on the western half of the building and live/work units and the residential lobby on the eastern half.

Based on our investigation, we conclude the proposed building may be constructed as planned, provided the recommendations presented in the attached report are incorporated into the project plans and specifications. The primary geotechnical concerns at the site are the presence of near-surface highly expansive soil and providing adequate foundation support for the proposed building. The site is underlain primarily by stiff to very stiff clay which has moderate to high strength and relatively low compressibility. Therefore, we conclude the proposed building can be supported on individual spread and/or continuous footings bottomed on stiff to very stiff native soil.

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

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

Quintin A. Flores, P.E.
Senior Staff Engineer

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GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE BUILDING
2136 – 2154 SAN PABLO AVENUE
Berkeley, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed mixed-use building to be constructed at 2136 – 2154 San Pablo Avenue in Berkeley, California. The site is located on the western side of San Pablo Avenue, south of its intersection with Cowper Street, as shown on the Site Location Map, Figure 1.

The subject property is a rectangular-shaped lot on the western side of San Pablo Avenue between Allston Way to the south and Addison Street to the north. The site is relatively level and has plan dimensions of approximately 133 by 175 feet. The eastern portion of the site is currently occupied by a two-story commercial building with car lifts in the rear. Most of the remainder of the site is occupied by a gravel parking lot and driveway. The site is bordered by San Pablo Avenue to the east, a small, one-story structure and paved parking lot to the north, residential units and a small park to the west, and a one-story commercial building, a parking lot, and bee-keeping equipment to the south.

Based on our review of the Progress Set prepared by Trachtenberg Architects, dated September 4, 2019, current plans are to demolish the existing two-story structure and to construct a new, 4- to 6-story, 126-unit, mixed-use building that will occupy most of the site. The proposed mixed-use building will consist of 3 to 5 stories of wood-framed, residential units constructed over a one-story concrete podium with an open, mezzanine layout. The ground floor will be at-grade and will include parking lift pits, gym space, and residential lofts on the western half of the building and live/work units and the residential lobby that front San Pablo Avenue.

2.0 SCOPE OF WORK

Our investigation was performed in accordance with our proposal dated June 7, 2019 and our Agreement with San Pablo Investors Two, LLC, dated August 13, 2019. Our scope of services included performing three cone penetration tests (CPTs) and advancing two test borings, performing laboratory testing on selected soil samples, and performing engineering analyses to develop conclusions and recommendations regarding:

- site seismicity and seismic hazards, including the potential for liquefaction and lateral spreading, and total and differential settlement resulting from liquefaction and/or cyclic densification
- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities for each foundation type(s)
- estimates of foundation settlement
- design groundwater level
- lateral earth pressures for design of below-grade parking lift-pit and elevator pit walls
- site preparation and grading, including criteria for fill quality and compaction
- subgrade preparation for interior and exterior concrete slabs-on-grade
- 2016 California Building Code (CBC) site class and design spectral response acceleration parameters
- corrosivity of the near-surface soil and the potential effects on buried concrete and metal structures and foundations
- rigid pavement design
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Subsurface conditions at the site were investigated by drilling two borings, performing three CPTs, and performing laboratory tests on selected soil samples from our borings. Prior to performing our field investigation, we obtained a drilling permit from the City of Berkeley, Planning and Development Department, Toxic Management Division and contacted

Underground Service Alert (USA) to notify them of our work. Because the borings and CPTs are located on private property, we also retained a private utility locator, Precision Locating LLC, to confirm the boring and CPT locations were clear of existing utilities. Details of the field investigation and laboratory testing are described below.

3.1 Test Borings

Two test borings, designated as B-1 and B-2, were drilled by Benevent Building of Concord, California on August 30, 2019 at the approximate locations shown on Figure 2. The borings were each drilled to a depth of 36-1/2 feet below ground surface (bgs) using a limited access drill rig equipped with four-inch-diameter, solid-stem augers. During drilling, our field geologist logged the soil encountered and collected representative samples of the soil for visual classification and laboratory testing. The boring logs are presented in Appendix A on Figures A-1 and A-2. 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 1.5-inch inside diameter, not designed to accommodate liners.

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. The S&H sampler was generally 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, above-ground safety 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 0.7 and 1.2, respectively, to account for sampler type and approximate hammer energy. 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.

Upon completion of drilling, the boreholes were backfilled with neat cement grout in accordance with City of Berkeley TMD grout requirements and under the observation of the City of Berkeley grout inspector. The soil cuttings were transported and disposed of offsite.

3.2 Cone Penetration Tests

Middle Earth Geo Testing, Inc. of Orange, California performed the CPTs, designated as CPT-1 through CPT-3, on September 9, 2019 at the approximate locations shown on Figure 2. The CPTs were each advanced to a depth of approximately 50 feet bgs by hydraulically pushing a 1.7-inch-diameter, cone-tipped probe with a projected area of 15 square centimeters into the ground. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information, such as the soil behavior types and approximate strength characteristics of the soil encountered. The CPT logs showing tip resistance, friction ratio, and pore pressure, as well as correlated soil behavior type, are presented in Appendix A on Figures A-4 through A-6.

Upon completion, the CPTs were backfilled with cement grout in accordance with City of Berkeley TMD requirements.

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. In addition, one soil sample obtained from Boring B-2 at 3.5 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 regional geologic map prepared by Graymer (2000), a portion of which is presented on Figure 3, indicates the site is underlain by Holocene-age alluvial deposits (Qha). The results of our borings and CPTs indicate the site is underlain by alluvium primarily consisting of stiff to very stiff clay with varying amounts of sand and gravel and medium dense sand with varying amounts of clay and gravel to the maximum depth explored of 50 feet bgs. The results of Atterberg limits tests performed on selected samples of the near-surface clay indicate the clay is highly expansive[1]. Where explored, the near-surface highly expansive clay was encountered to depths up to about five feet bgs.

During drilling, groundwater was encountered at depths between 17-1/2 and 26-1/2 feet bgs in Borings B-1 and B-2, respectively. Pore pressure dissipation tests performed using the CPT probe indicated groundwater depths of approximately 4 and 12 feet bgs in CPT-1 and CPT-3, respectively. The depth to groundwater was also measured in the CPT holes between depths of 11 and 20 feet with a drop tape; however, the groundwater level may not have been stabilized at the time the measurements were taken. The groundwater level at the site is expected to fluctuate several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall. Available historic groundwater information presented in the California Geologic Survey

[1] Expansive soil undergoes volume changes with changes in moisture content.

(CGS) Seismic Hazard Zone Report for the Oakland West Quadrangle indicates the historic high groundwater at the site is approximately 5 feet bgs.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity and Faulting

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

The major active faults in the area are the Hayward, San Andreas and Calaveras faults. These and other faults in the region are shown on Figure 4. The fault systems in the Bay Area consist of several major right-lateral strike-slip faults that define the boundary zone between the Pacific and the North American tectonic plates. Numerous damaging earthquakes have occurred along these fault systems in recorded time. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated mean characteristic moment magnitude[2] [Working Group on California Earthquake Probabilities (USGS 2008) and Cao et al. (2003)] are summarized in Table 1.

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[2] 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	Maximum Magnitude
Total Hayward	3.3	East	7.00
Total Hayward-Rodgers Creek	3.3	East	7.33
Mount Diablo Thrust	22	East	6.70
Total Calaveras	25	East	7.03
Green Valley Connected	25	East	6.80
N. San Andreas - Peninsula	26	West	7.23
N. San Andreas (1906 event)	26	West	8.05
N. San Andreas - North Coast	26	West	7.51
Rodgers Creek	28	Northwest	7.07
San Gregorio Connected	31	West	7.50
West Napa	33	North	6.70
Greenville Connected	40	East	7.00
Great Valley 5, Pittsburg Kirby Hills	43	East	6.70
Monte Vista-Shannon	48	South	6.50

Since 1800, four major earthquakes (i.e., Magnitude > 6) have been recorded on the San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) Intensity Scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt, 1998). The estimated moment magnitude, M_w, for this earthquake is about 6.25. In 1838, an earthquake occurred on the Peninsula segment of the San Andreas Fault. Severe shaking occurred with an MM of about VIII-IX, corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of October 17, 1989 with an M_w of 6.9. This earthquake occurred in the Santa Cruz Mountains about 99 kilometers southwest of the site.

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

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

5.2 Geologic Hazards

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[3] lateral spreading,[4] and cyclic densification[5]. We used the results of the CPTs and borings to evaluate the potential of these phenomena occurring at the project site.

5.2.1 Ground Shaking

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

5.2.2 Ground Surface Rupture

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

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

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

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

5.2.3 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.

As shown on Figure 5, the site has been mapped along the boundary of a zone of liquefaction potential on the map titled *State of California Seismic Hazard Zones, Oakland West Quadrangle, Official Map*, prepared by the California Geological Survey (CGS), dated February 14, 2003. Special Publication 117 prepared by the California Geological Survey (2008) recommends subsurface investigation in mapped liquefaction potential areas be performed using rotary-wash borings and/or CPTs. We used the results of our CPTs, supplemented with subsurface information from the test borings, to evaluate the potential for liquefaction to occur at the site.

Liquefaction susceptibility was assessed using the software CLiq v3.0.2.4 (GeoLogismiki, 2019). 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). We performed the liquefaction triggering analyses using the methodology proposed by Idriss and Boulanger (2014). We also used the relationship proposed by Zhang, Robertson, and Brachman (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement; a relationship that is an extension of the work by Ishihara and Yoshimine (1992). Our analyses were performed using an in-situ groundwater depth of 12 feet bgs and "during earthquake" groundwater depth of 5 feet bgs. In accordance with the 2016 CBC, we used a peak ground acceleration of 0.78 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 the soil underlying the site is sufficiently cohesive and/or sufficiently dense such that the potential for liquefaction is low. Consequently, we conclude seismically induced ground settlement following a Maximum Considered Earthquake (MCE) event with PGA_M of 0.78g will be negligible.

Considering the relatively flat site grades, the absence of a free face in the site topography, and the site is sufficiently cohesive and/or the granular layers are sufficiently dense such that liquefaction will not occur, we conclude the risk of lateral spreading is nil.

5.2.4 Cyclic Densification

Seismically induced compaction (also referred to as cyclic densification) of non-saturated granular soil (granular soil above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. Based on the boring and CPT data, we conclude the potential for cyclic densification of the soil above the groundwater table is very low due to its cohesion.

6.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concerns at the site are the presence of near-surface highly expansive soil and providing adequate foundation support for the proposed building. Our conclusions and recommendations regarding these issues and other geotechnical aspects of the project are presented below.

6.1 Foundation Support and Settlement

The highly expansive near-surface clay is subject to volume changes during seasonal fluctuations in moisture content. These volume changes can cause cracking of foundations and slabs.

Therefore, foundations and slabs should be designed and constructed to mitigate the effects of the expansive clay. These effects can be mitigated by moisture-conditioning the expansive soil below slabs, providing non-expansive soil below slabs, and supporting foundations below the zone of severe moisture change.

The soil encountered at the foundation level has moderate strength and relatively low compressibility. Therefore, we conclude the proposed building may be supported on individual spread footings at interior column locations and continuous, deepened perimeter footings. The perimeter footings should be deepened to act as barriers to reduce the potential for moisture change beneath the slab-on-grade floors.

We estimate total settlement of the proposed building supported on properly designed and constructed footings will be less than one inch and differential settlement will be less than 1/2 inch in 30 feet.

6.2 Slabs-on-Grade

The building slab-on-grade and capillary break/vapor barrier should be underlain by at least 18 inches of non-expansive soil. The following two alternatives may be used to provide a 18-inch-thick layer of non-expansive soil beneath slabs-on-grade.

- Alternative No. 1: The building pad should be overexcavated to allow for placement of 18 inches of imported select (non-expansive) fill beneath the floor slab and capillary break/vapor retarder.
- Alternative No. 2: As an alternative to importing select fill, the upper 18 inches of soil beneath the building pad subgrade may be treated in place with five percent quicklime by dry weight of soil. The purpose of the lime treatment is to reduce the expansion potential of the surface soil and provide a firm surface for construction of the slabs.

We judge exterior concrete slabs-on-grade (i.e. concrete flatwork) should perform satisfactorily if they are supported on a layer of non-expansive soil at least eight inches thick.

6.3 Construction Considerations

The soil to be excavated consists primarily of clay, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. If site grading is performed during the rainy season, repeated loads by heavy equipment will reduce the strength of the surficial soil and decrease its ability to resist deformation; this phenomenon could result in severe rutting and pumping of the exposed subgrade. To reduce the potential for this behavior, heavy rubber-tired equipment as well as vibratory rollers, should be avoided. If the lime-treatment alternative is selected to provide 18 inches of non-expansive soil beneath building slabs, the lime treatment will also help stabilize/winterize the soil subgrade.

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

To reduce the potential for damage to the adjacent buildings, heavy equipment should not be used within 10 feet from adjacent foundations. Jumping jack or vibratory plate compactors should be used for compacting fill within this zone.

6.4 Soil Corrosivity

Corrosivity testing was performed by Project X of Murrieta, California on a sample of soil obtained during our field investigation from Boring B-2 at a depth of 3.5 feet bgs. The results of the test are presented in Appendix B of this report. Based on the resistivity test results (minimum resistivity of 1,943 ohm-cm), the sample is classified as moderately corrosive to buried steel. Accordingly, buried iron, steel, cast iron, galvanized steel, and dielectric-coated steel or iron should be properly protected against corrosion. The chloride and sulfate ion concentrations and pH of the soil do not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

7.0 RECOMMENDATIONS

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

7.1 Site Preparation, Grading, and Fill Placement

Site demolition should include the removal of existing pavements and all existing underground utilities and foundations. 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 proposed building footprint and will not interfere with the proposed construction, they may be abandoned in-place provided the lines are filled with lean concrete or cement grout to the property line. Voids resulting from demolition activities should be properly backfilled with compacted fill following the recommendations provided later in this section.

The soil subgrade beneath proposed improvements or areas for new fill should be scarified to a depth of at least eight inches, moisture-conditioned to at least four percent above optimum moisture content, and compacted to between 87 and 92 percent relative compaction[6]. On-site soil may be used as general fill, provided the material is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, and be approved by the Geotechnical Engineer. If material to be used as fill is imported to the site, it should meet the requirements for select fill provided below in Section 7.1.1. A summary of the compaction requirements for the various types of fill that may be used at the site is presented in Table 2.

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

TABLE 2
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
Building pad – expansive clay	87 – 92	4+% above optimum
Building pad – low-plasticity soil	90+	Above optimum
Exterior slabs – expansive clay	87 – 92	4+% above optimum
Exterior slabs – low-plasticity soil	90+	Above optimum
Pavements – expansive clay	90+	2+% above optimum
Pavements – low-plasticity soil	95+	Above optimum
Pavements - aggregate base	95+	Near optimum
General fill – expansive clay	87 – 92	4+% above optimum
General fill – low-plasticity soil	90+	Above optimum
General fill – granular soil	95+	Near optimum
Utility trench backfill – expansive clay	87 – 92	4+% above optimum
Utility trench backfill – low-plasticity	90+	Above optimum
Utility trench - clean sand or gravel	95+	Near optimum

*Low-plasticity soil includes select fill and lime-treated onsite soil.
*Granular soil consists of soil with less than 5 percent fines by weight of soil.

If grading work is performed during the rainy season, the contractor may find the subgrade material too wet to compact to the recommended relative compaction and will have to be scarified and aerated to lower its moisture content so the specified compaction can be achieved. Material to be dried by aeration should be scarified to a depth of at least 12 inches; the scarified soil 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 recommendations. Aeration typically is the least costly method used to stabilize the subgrade soil; however, it generally requires the most time to complete. Other soil stabilization alternatives include overexcavating and placing drier material, and lime treatment.

It is also important that the moisture content of subgrade soil is sufficiently high to reduce its expansion potential. If the grading work is performed during the dry season, moisture-conditioning may be required.

The near-surface clay at the site is highly expansive. The building slab-on-grade floor and capillary break/vapor barrier should be underlain by at least 18 inches of non-expansive soil consisting of imported select fill (Alternative No. 1) or lime-treated on-site soil (Alternative No. 2). The non-expansive soil should extend at least five feet beyond the perimeter of the proposed building, except where constrained by the property line.

7.1.1 Select Fill

If Alternative No. 1 is selected, we recommend the proposed building pad be overexcavated to allow placement of at least 18 inches of imported select fill beneath the building slab-on-grade floor.

Select fill should consist of imported soil that is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the Geotechnical Engineer. Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. In the parking garage, the upper six inches of the select fill should be compacted to at least 95 percent relative compaction. Samples of proposed select fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site.

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

7.1.2 Lime Treatment

If Alternative No. 2 is selected, the upper 18 inches of the building pad subgrade (measured below the capillary moisture break) should be treated in place with a minimum of five percent dolomitic quicklime by dry weight of soil. The dry weight of soil should be assumed to be 105 pounds per cubic foot (pcf) for calculating lime quantities. A specialty subcontractor typically performs lime treatment. Prior to lime treatment, we recommend the site be graded to a level pad elevation in accordance with our previous recommendations and all below-grade obstructions removed. The soil treated with lime should be mixed and compacted in one lift. The lime should be thoroughly blended with the soil and allowed to set for 24 hours prior to remixing and compaction. The lime-treated soil should be moisture-conditioned to above optimum moisture content and compacted to at least 90 percent relative compaction. It should be noted that disposal of lime-treated soil is typically expensive because of the high pH of the treated soil. In addition, lime-treated soil should be completely removed from landscaping areas.

7.1.3 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped. The pipe bedding and cover should be eliminated where an impermeable plug is required as described below. Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted as according to the recommendations previously presented. If imported clean sand or gravel (defined as poorly-graded soil with less than five percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

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

Where utility trenches enter the building pad, an impermeable plug consisting of CLSM, at least three feet in length, should be installed where the trenches enter the building footprint, as shown on Figure 6. Furthermore, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

7.1.4 Drainage and Landscaping

Positive surface drainage should be provided around the building to direct surface water away from the foundations. To reduce the potential for water ponding adjacent to the building, we recommend the ground surface within a horizontal distance of five feet from the building slope down away from the building with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. The use of water-intensive landscaping around the perimeter of the building should be avoided to reduce the amount of water introduced to the expansive clay subgrade.

Care should be taken to minimize the potential for subsurface water to collect beneath pavements and pedestrian walkways. Where landscape beds and tree wells are immediately adjacent to pavements and flatwork, we recommend vertical cutoff barriers be incorporated into the design to prevent irrigation water from saturating the subgrade and aggregate base. These barriers may consist of either flexible impermeable membranes or deepened concrete curbs.

Prior experience and industry literature indicate that some species of high water-demand[7] trees can induce ground-surface settlement by drawing water from the expansive clay, causing it to shrink. Where these types of trees are planted near buildings, the ground-surface settlement may result in damage to structure. This problem usually occurs 10 or more years after planting, as the trees reach mature height. To reduce the risk of tree-induced, building settlement, we recommend trees of the following genera are not planted within 25 feet of the proposed building: *Eucalyptus, Populus, Quercus, Crataegus, Salix, Sorbus* (simple-leafed), *Ulmus, Cupressus, Chamaecyparis,* and *Cupressocyparis.* Because this is a limited list and does not include all genera that may induce ground-surface settlement, a tree specialist should be consulted prior to selection of trees to be planted at the site.

7.2 Spread Footings

Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. Perimeter footings should be bottomed at least 36 inches below the lowest adjacent outside grade. The perimeter footing embedment depth may be decreased by six inches where pavement, concrete flatwork, or an existing building is adjacent to the new building. Interior footings should extend at least 24 inches below the bottom of the capillary moisture break. Spread footings may be designed using allowable bearing pressures of 4,000 pounds per square foot (psf) for dead-plus-live loads and 5,300 psf for total design loads, which include wind or seismic forces; these values include factors of safety of at least 2.0 and 1.5, respectively.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute lateral resistance, we recommend using a uniform pressure of 1,600 psf for transient load conditions and an equivalent fluid weight of 240 pcf for sustained load conditions; the upper foot of soil should be ignored unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30. The passive pressure and frictional resistance values include a factor of safety of at least 1.5.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the footing will eventually heave, which may result in cracking and distress. We recommend rat slabs consisting of at least two inches of CLSM be placed in the bottoms of the footings to protect them from drying out, softening from ponding water and/or disturbance from foot traffic during construction. We should check footing excavations prior to placement of the rat slabs. The CLSM used to construct the rat slabs should have a 28-day unconfined strength of 50 psi and should be poured within two days of footing excavation. The rat slab thickness may be counted as part of the minimum footing embedment.

7.3 Concrete Slab-on-Grade Floors

As discussed in Section 7.1, the slab-on-grade floor should be underlain by at least 18 inches of non-expansive soil consisting of either imported select fill or lime-treated on-site soil. The capillary break material discussed below should not be counted as part of the non-expansive soil.

To reduce water vapor transmission through the floor slab, we recommend installing a capillary moisture break and water vapor retarder beneath the slab where water vapor transmission through the slab may damage floor coverings and stored materials. A capillary moisture break and vapor retarder are generally not required below parking slabs-on-grade because there is sufficient air circulation to limit condensation of moisture on the slab surface. Where a capillary moisture break/vapor retarder is not used, we recommend four inches of Class 2 aggregate base compacted to at least 95 percent relative compaction be placed beneath the parking garage slab.

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

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

If required by the structural engineer, the vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The sand overlying the vapor retarder should be moist at the time concrete is placed. However, excess water trapped in the sand could eventually be transmitted as vapor through the slab. Therefore, if rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced. The particle size of the capillary break material and sand (if used) should meet the gradation requirements presented in Table 3.

TABLE 3
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If the concrete is poured directly over the vapor retarder, we recommend the w/c ratio of the concrete not exceed 0.45. In either case, 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 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 Exterior Concrete Flatwork

We recommend a minimum of eight inches of Class 2 aggregate base (AB) be placed beneath proposed exterior concrete flatwork; the Class 2 AB should extend at least six inches beyond the slab edges. Class 2 AB placed beneath exterior slabs-on-grade, such as patios and sidewalks, should be moisture-conditioned and compacted in accordance with the requirements provided in Table 2. Where the flatwork will be subject to vehicular traffic, such as at driveways, the Class 2 AB should be compacted to at least 95 percent relative compaction.

Even with eight inches of non-expansive soil, exterior slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil. Thickening the slab edges and adding additional reinforcement will control this cracking to some degree. Where slabs are adjacent to landscaped areas, thickening the concrete edge will help control water infiltration beneath the slabs. In addition, where slabs provide access to buildings, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries.

We do not recommend the use of pervious interlocking pavers at this site, due to the expansive subgrade soil. If pavers must be incorporated into this project, we should provide additional recommendations for proper subgrade preparation and subsurface drainage.

7.5 Below-Grade Walls

Below-grade walls for elevator and parking lift pits, if used, should be designed to resist lateral earth pressure imposed by the retained soil, as well as a surcharge pressure from nearby vehicles and foundations, where appropriate. Since the elevator and parking lift pit walls will be restrained from movement at the sides, they should be designed for an at-rest condition. We recommend below-grade walls be designed using at-rest equivalent fluid weights of 60 and 91 pcf above and below the design groundwater level (i.e., 5 feet bgs), 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 31 pcf (triangular distribution) above the design groundwater level and 82 pcf plus a seismic increment of 15 pcf below the design groundwater level.

Where applicable, the elevator and parking lift pit walls should be designed for a uniform vehicular surcharge pressure of 50 psf applied to the entire height of the walls. To avoid surcharging the below-grade walls with lateral pressures imposed by the proposed footings, the footings should be bottomed below a zone-of-influence line projected upward at an inclination of 1.5:1 from the bottom of the below-grade walls.

The lateral earth pressures recommended are applicable to walls that are backdrained above the water table to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining below-grade walls is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the shoring or the back of the walls. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe at the base of the walls. The pipe should be surrounded by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140NC or equivalent). The collector pipe should outlet into a sump in the elevator and parking lift pits. Where shoring is installed and there is insufficient room to install a perforated pipe between the shoring and the back of the elevator and parking lift pit walls, the drainage panel should extend down to a proprietary, prefabricated collector drain system, such as Tremdrain Total Drain or Hydroduct Coil, designed to work in conjunction with the drainage panel. We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use. To protect against moisture migration into the elevator and parking lift pits, we recommend the below-grade walls be waterproofed and water stops be installed at all construction joints.

If it is desired to avoid installing a backdrain behind the below-grade walls, the walls should be designed for the undrained at-rest equivalent fluid weight (i.e., 91 pcf) for their full height.

If backfill is required behind the elevator and parking lift pit walls, the walls should be braced or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the Structural Engineer).

7.6 Seismic Design

For design in accordance with the 2016 CBC, we recommend Site Class D be used. The latitude and longitude of the site are 37.8669° and -122.2918°, respectively. Hence, in accordance with the 2016 CBC, we recommend the following:

- S_S = 2.026g, S_1 = 0.826g
- S_{MS} = 2.026g, S_{M1} = 1.240g
- S_{DS} = 1.351g, S_{D1} = 0.826g
- Seismic Design Category E for Risk Categories I, II, and III.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

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

9.0 LIMITATIONS

This geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the exploratory borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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FIGURES

Base map: The Thomas Guide
San Francisco County
2002

0 1/4 1/2 Mile

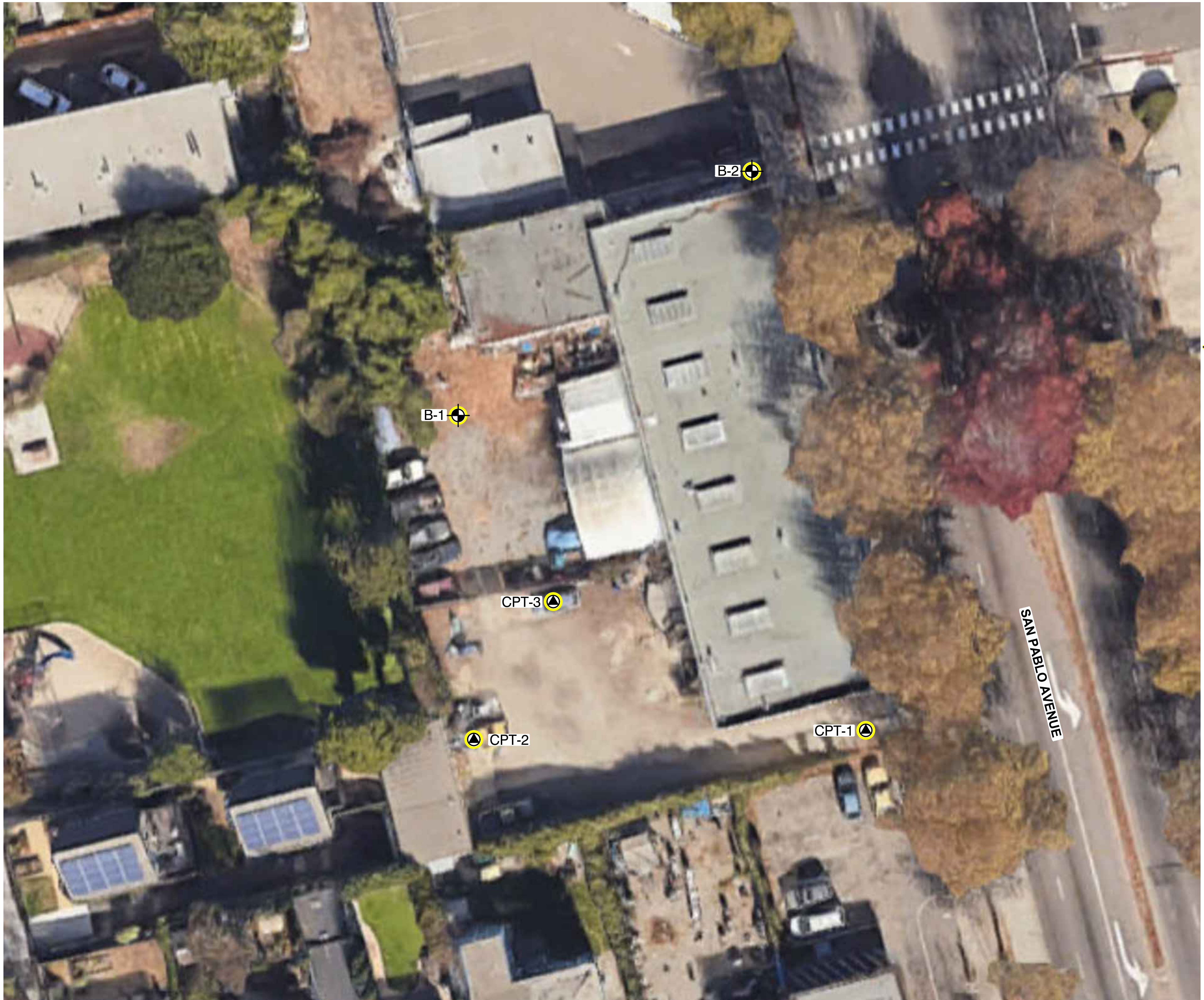
Approximate scale

2136 - 2154 SAN PABLO AVENUE
Berkeley, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 10/23/19 | Project No. 19-1740 | Figure 1



EXPLANATION

CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical Inc., September 9, 2019

B-1 — Approximate location of boring by Rockridge Geotechnical Inc., August 30, 2019

Project limits

0 30 Feet

Approximate scale

Base map: Google Earth, 2019.

2136 - 2154 SAN PABLO AVENUE
Berkeley, California

SITE PLAN

Date 10/23/19	Project No. 19-1740	Figure 2

ROCKRIDGE GEOTECHNICAL

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

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
Qs	Beach and dune sand (Quaternary)
Qpa	Alluvium (Pleistocene)

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

0 600 1200 Feet

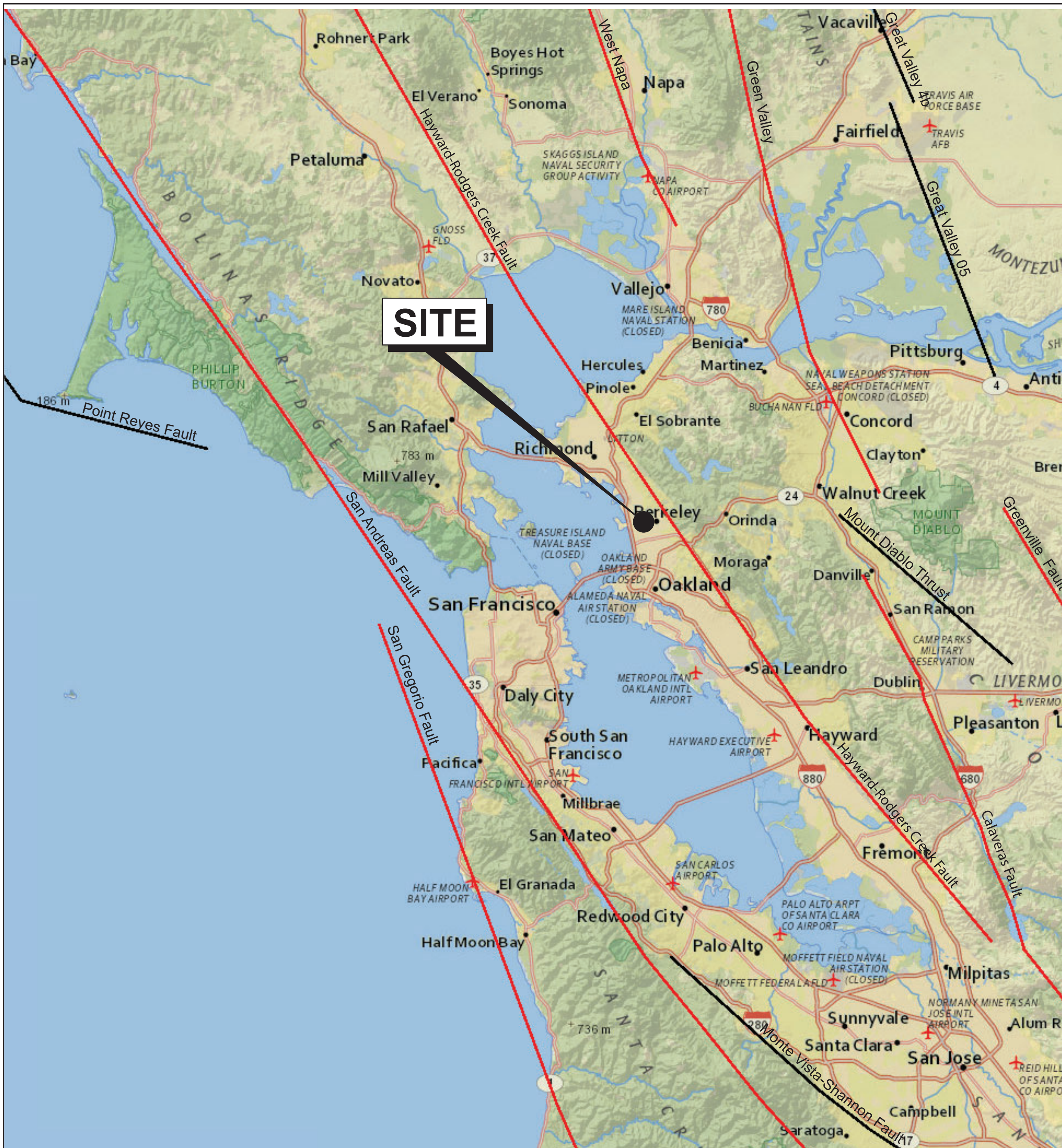
Approximate scale

2136 - 2154 SAN PABLO AVENUE
Berkeley, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 10/23/19 | Project No. 19-1740 | Figure 3



Base Map: U.S. Geological Survey (USGS), National Seismic Hazards Maps - Fault Sources, 2008.

EXPLANATION

Strike slip

Thrust (Reverse)

Normal

N

0 5 10 Miles

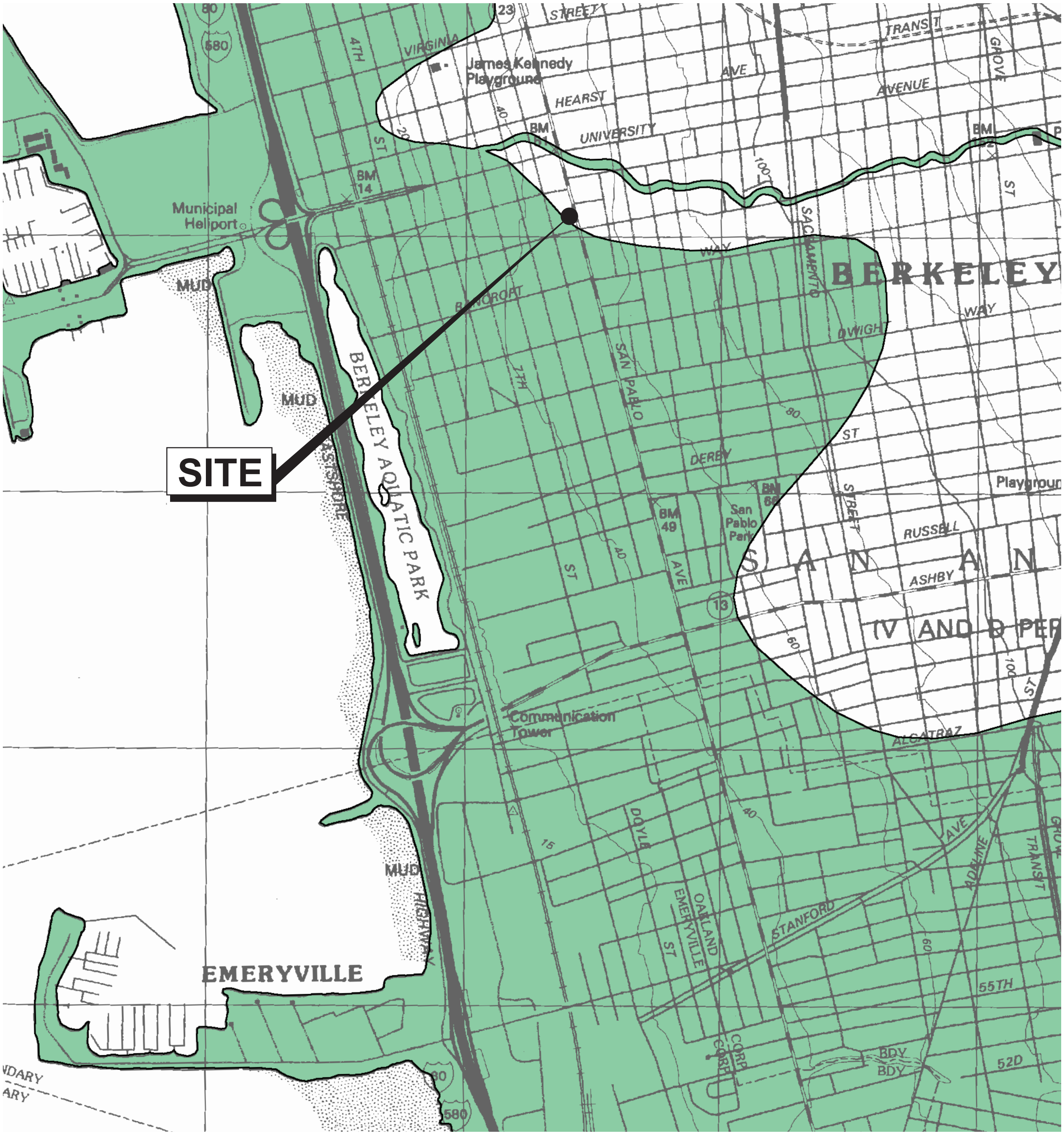
Approximate scale

2136 - 2154 SAN PABLO AVENUE
Berkeley, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 10/23/19	Project No. 19-1740	Figure 4



EXPLANATION

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

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

0 2,000 4,000 Feet

Approximate scale

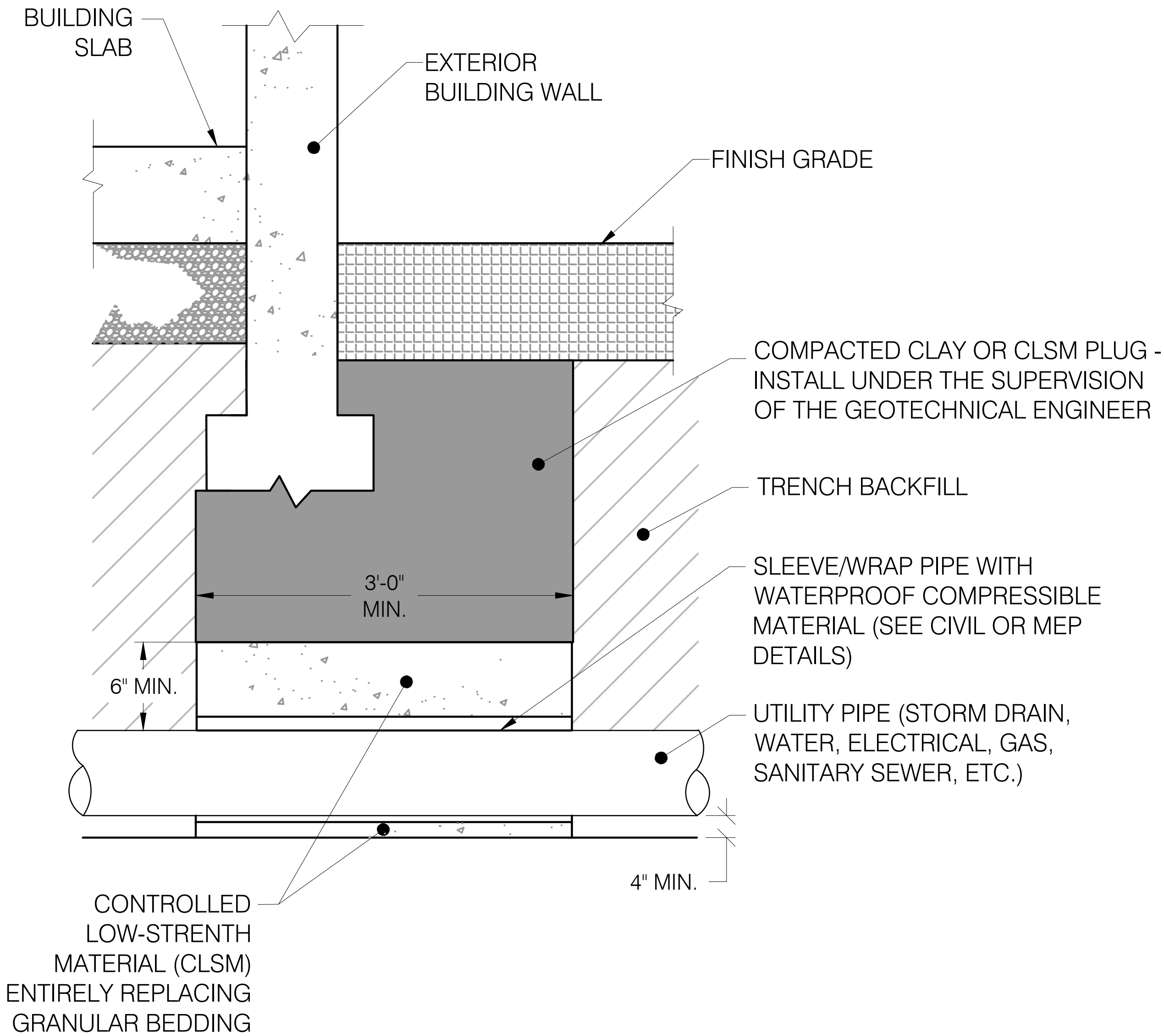
Reference:
State of California "Seismic Hazard Zones"
Oakland West Quadrangle.
Released on February 14, 2003

2136 - 2154 SAN PABLO AVENUE
Berkeley, California

ROCKRIDGE GEOTECHNICAL

SEISMIC HAZARDS ZONE MAP

Date 10/23/19 | Project No. 19-1740 | Figure 5



Not to Scale

2136 - 2154 SAN PABLO AVENUE
Berkeley, California

ROCKRIDGE GEOTECHNICAL

UTILITY TRENCH LOW-PERMEABILITY PLUG AT BUILDING PERIMETER

Date 10/23/19	Project No. 19-1740	Figure 6

APPENDIX A

Boring Logs and Cone Penetration Test Results

Log of Boring B-1

PROJECT: **2136 - 2154 SAN PABLO AVENUE**
Berkeley, California

Boring location:	See Site Plan, Figure 2
Date started:	8/30/19
Date finished:	8/30/19
Drilling method:	4-inch-diameter Solid Stem Auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Rope & Cathead
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	M. Hachey
Drilled by:	Benevent Building
Rig:	Limited Access

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3 inches of gravel parking lot						
1–2	S&H	11, 10, 10	14	CL	SANDY CLAY (CL) dark brown, stiff, moist, trace organics, trace brick debris; LL = 46, PI = 24; see Appendix B (FILL)					14.8	96
3–4	S&H	5, 9, 25	16	CL	dark brown to olive-gray, very stiff, trace fine gravel, fine- to coarse-grained sand					20.4	99
5–6	S&H	5, 8, 10	13	CL	CLAY with SAND (CL) brown, very stiff, moist, fine-grained sand, trace rootlets; TxUU Test; see Appendix B	TxUU	750	2970		20.4	108
7–8	S&H	5, 8, 11	13	CL	increasing sand content, trace coarse-grained sand						
10–11	S&H	15, 16, 16	22	SW-SC	SAND with CLAY and GRAVEL (SW-SC) red-brown to gray-brown, medium dense, moist, fine gravel						
14.5–16	S&H	9, 16, 26	29	SC	CLAYEY SAND (SC) red-brown, medium dense, moist, some oxidation staining, subrounded to subangular, fine gravel; Particle Size Distribution; see Appendix B				43		
17				SC	∇ wet (8/30/2019; 2:00 PM)						
19.5–21	SPT	8, 10, 15	30	CL	CLAY (CL) red-brown to gray-brown, very stiff to hard, wet, some fine-grained sand, trace coarse-grained sand						
25–26	SPT	5, 7, 6	16	CL	stiff, trace fine-grained sand						
30											

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1740

Figure: A-1a

PROJECT:	2136 - 2154 SAN PABLO AVENUE Berkeley, California	Log of Boring B-1 PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		5 7 9	19	CL	CLAY (CL) (continued) very stiff						
32												
33												
34												
35					SC	CLAYEY SAND (SC) gray-brown to red-brown, medium dense, wet						
36	SPT		5 6 10	19								
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
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Boring terminated at a depth of 36.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 17.5 feet after drilling but was not allowed time to stabilize.

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

ROCKRIDGE GEOTECHNICAL

Project No.:	Figure:
19-1740	A-1b

PROJECT:	2136 - 2154 SAN PABLO AVENUE Berkeley, California	**Log of Boring B-2** PAGE 1 OF 2

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

Logged by: M. Hachey
Drilled by: Benevent Building
Rig: Limited Access

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					4.5 inches of concrete slab						
1–2	S&H	12 10 10	14	CH	SANDY CLAY (CH) dark brown, stiff, dry to moist, trace, fine gravel					14.4	93
3					gray-brown						
4	S&H	9 13 18	22		dark brown, very stiff, moist LL = 50, PI = 29; see Appendix B					20.7	101
5–6	S&H	11 12 15	19	SC	CLAYEY SAND (SC), brown to gray-brown, medium dense, moist, trace gravel						
7–8	S&H	8 11 18	20	CL	CLAY with SAND (CL) brown, very stiff to hard, moist, fine- to coarse-grained sand, trace gravel TxUU Test; see Appendix B	TxUU	1,000	4,640		17.6	113
10–11	S&H	9 9 7	11		gray-brown						
11–12				CL	SANDY CLAY (CL) gray-brown to olive-brown, stiff, moist, trace coarse gravel						
12–14				CL	CLAY (CL) gray-brown to olive-brown, stiff, moist, trace coarse-grained sand, trace rootlets						
16	S&H	6 10 13	16		increasing fine-grained sand content						
19–21	SPT	11 15 19	41	CL	SANDY CLAY (CL) brown mottled with white, black, and red-brown, hard, moist, some gravel, fine- to coarse-grained sand						
25–26	SPT	6 8 13	25		gray-brown mottled with red and olive-brown, very stiff Particle Size Distribution; see Appendix B ▽ wet (8/30/2019; 11:15 AM)				64		
29					▽ (8/30/2019; 10:30 AM)						
30											

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1740 | Figure: A-2a

PROJECT: **2136 - 2154 SAN PABLO AVENUE**
Berkeley, California

Log of Boring B-2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	SPT		5 8 13	25		CLAYEY SAND (SC) (continued) increasing clay content, decrease in gravel						
32					SC							
33												
34												
35						brown to gray-brown						
36	SPT		9 10 10	24	CL	SANDY CLAY (CL) gray-brown, very stiff, wet, fine-grained sand						
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												
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60												

Boring terminated at a depth of 36.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 26.5 feet after during drilling.

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

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1740

Figure: A-2b

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

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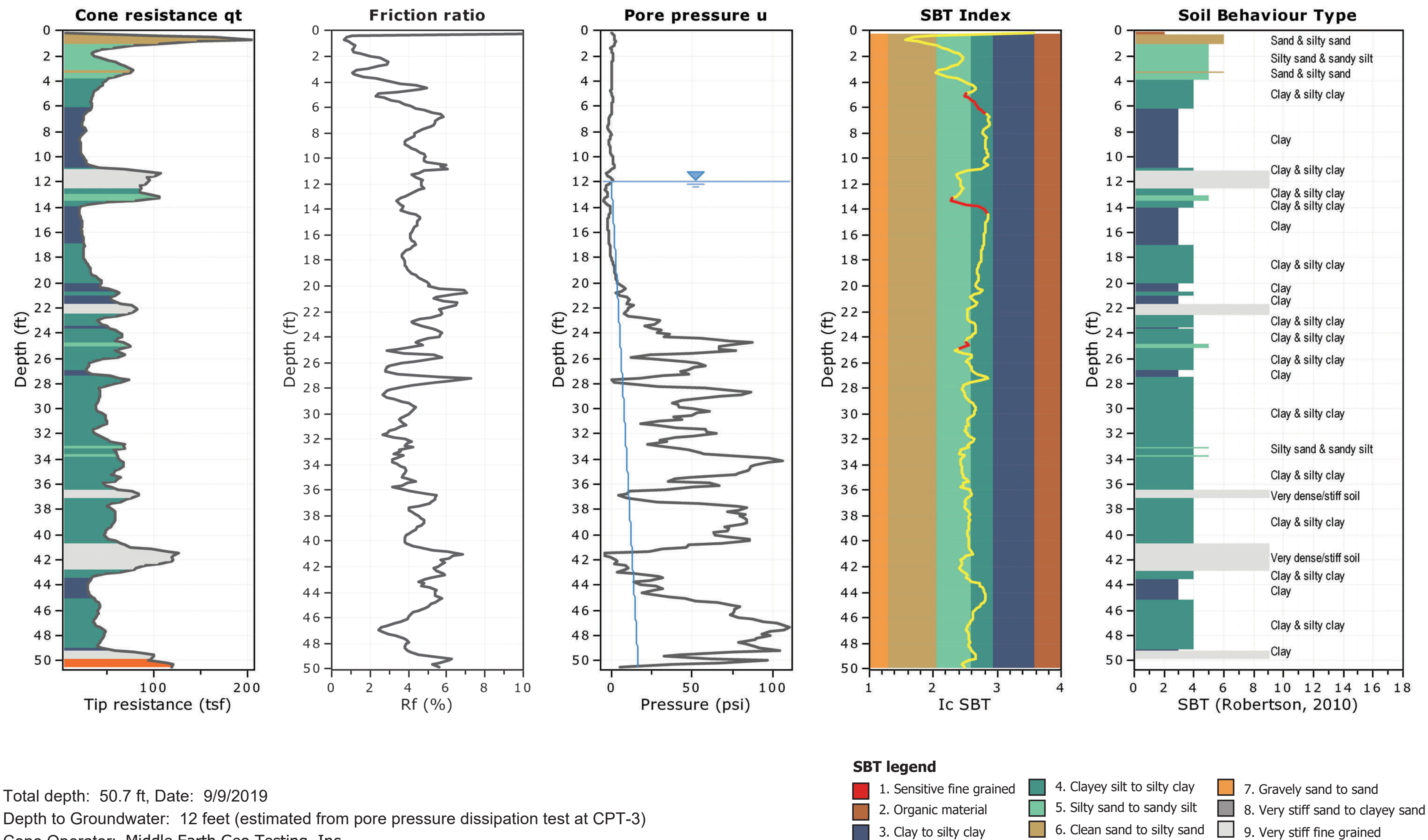
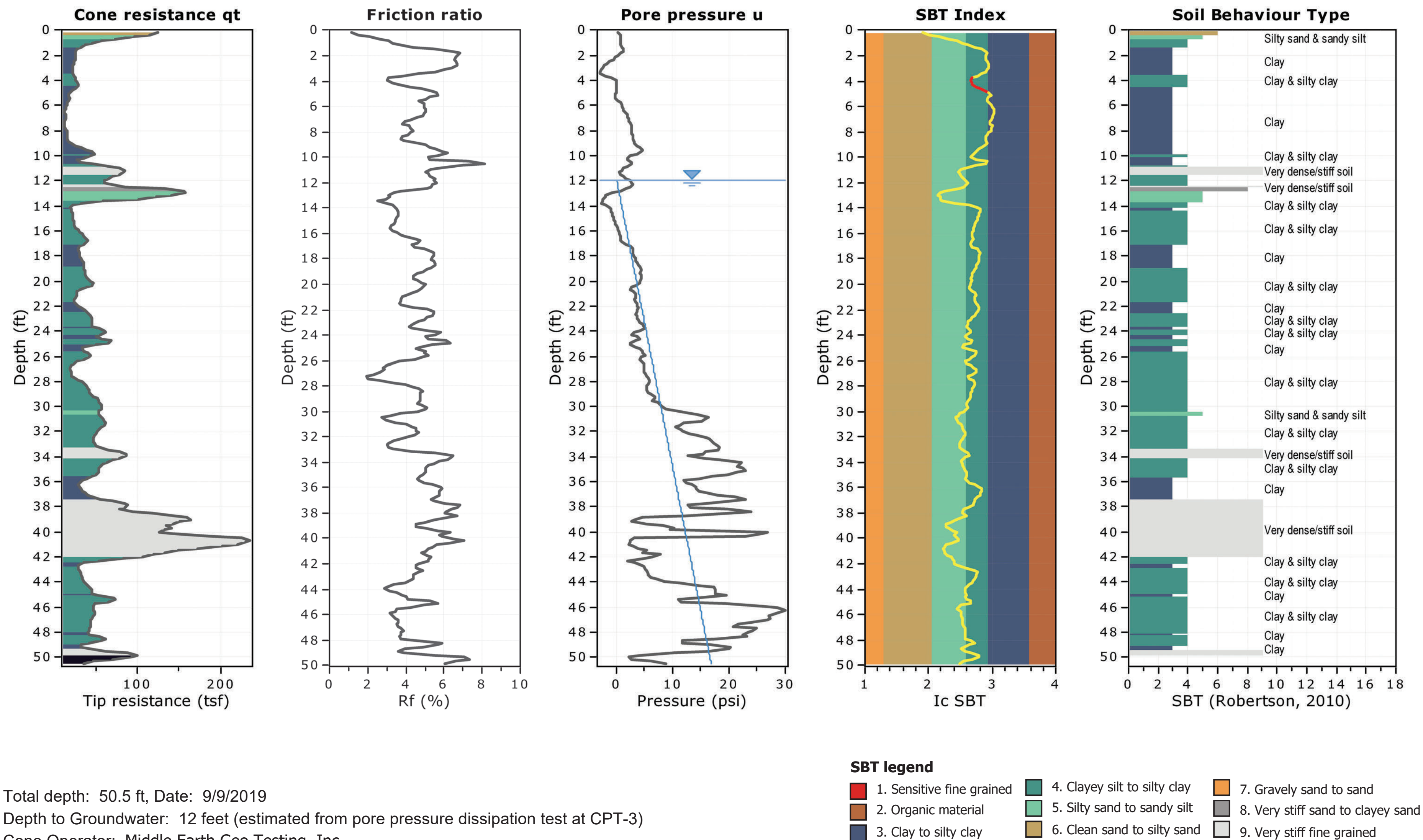
CLASSIFICATION CHART

Date 10/23/19 | Project No. 19-1740 | Figure A-3



Total depth: 50.7 ft, Date: 9/9/2019
Depth to Groundwater: 12 feet (estimated from pore pressure dissipation test at CPT-3)
Cone Operator: Middle Earth Geo Testing, Inc.

2136 - 2154 SAN PABLO AVENUE Berkeley, California	**CONE PENETRATION TEST RESULTS** **CPT-1**
ROCKRIDGE GEOTECHNICAL	Date 10/23/19 · Project No. 19-1740 · Figure A-4



Total depth: 50.5 ft, Date: 9/9/2019
Depth to Groundwater: 12 feet (estimated from pore pressure dissipation test at CPT-3)
Cone Operator: Middle Earth Geo Testing, Inc.

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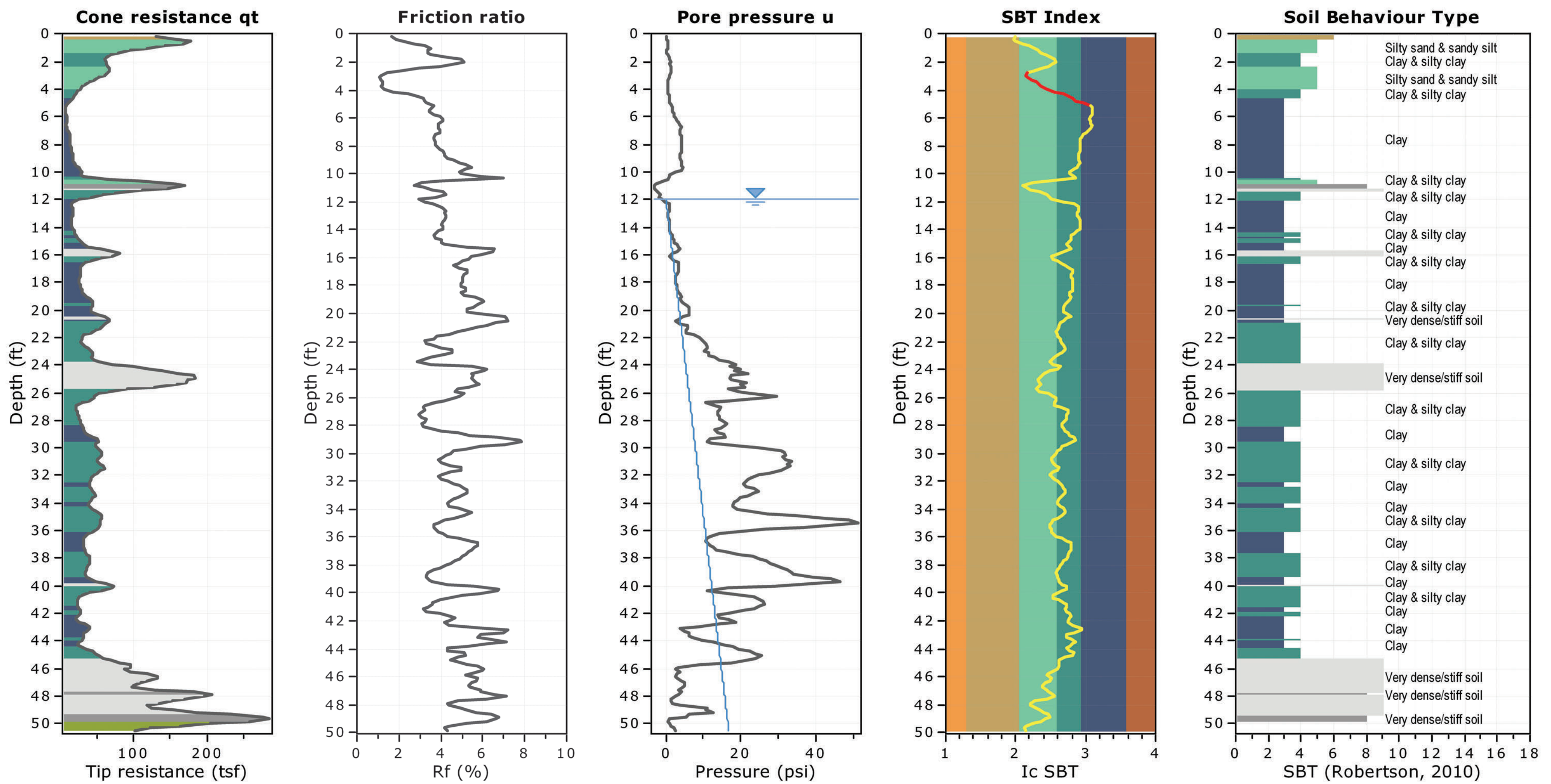
ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 10/23/19	Project No. 19-1740	Figure A-5



SBT legend

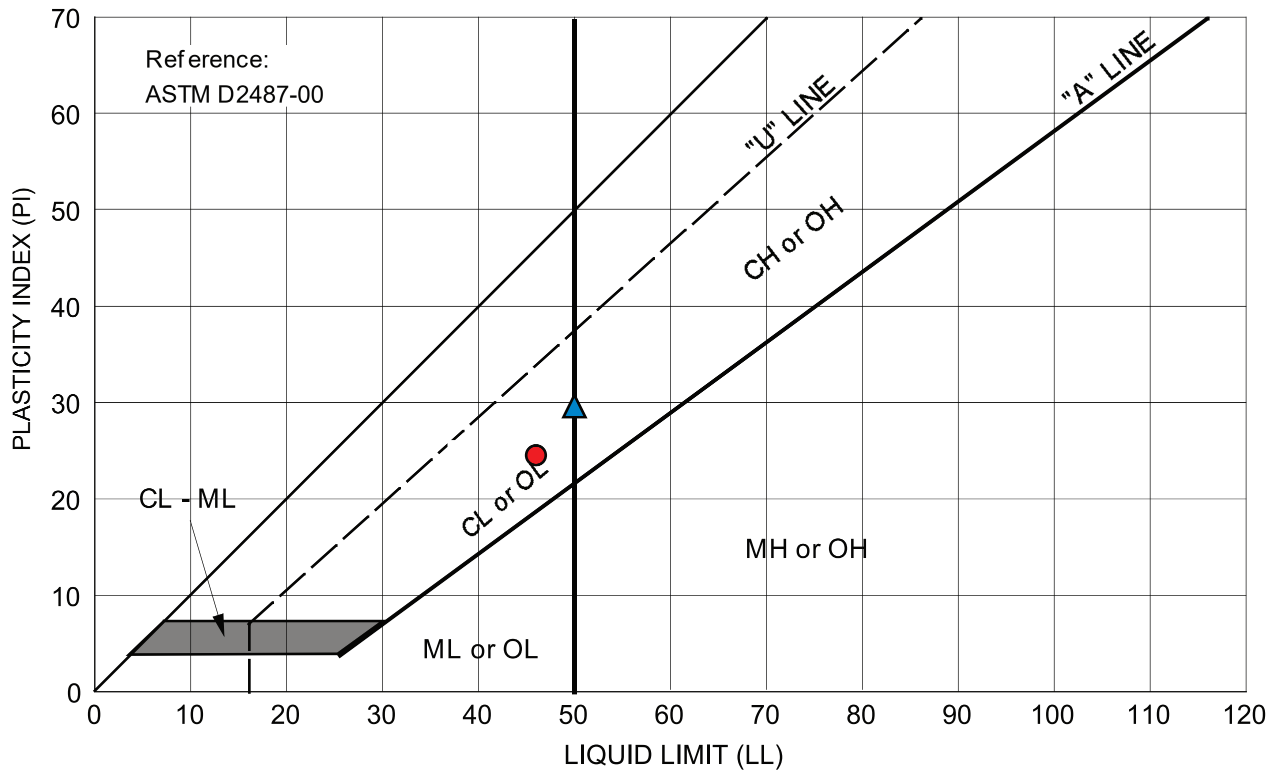
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

Total depth: 50.5 ft, Date: 9/9/2019
Depth to Groundwater: 12 feet (estimated from pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

2136 - 2154 SAN PABLO AVENUE Berkeley, California	**CONE PENETRATION TEST RESULTS** **CPT-3**
ROCKRIDGE GEOTECHNICAL	Date 10/23/19 | Project No. 19-1740 | Figure A-6

APPENDIX B

Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 1.25 feet	SANDY CLAY (CL), dark brown	14.8	46	24	--
▲	B-2 at 4.0 feet	SANDY CLAY (CH), dark brown	20.7	50	29	--

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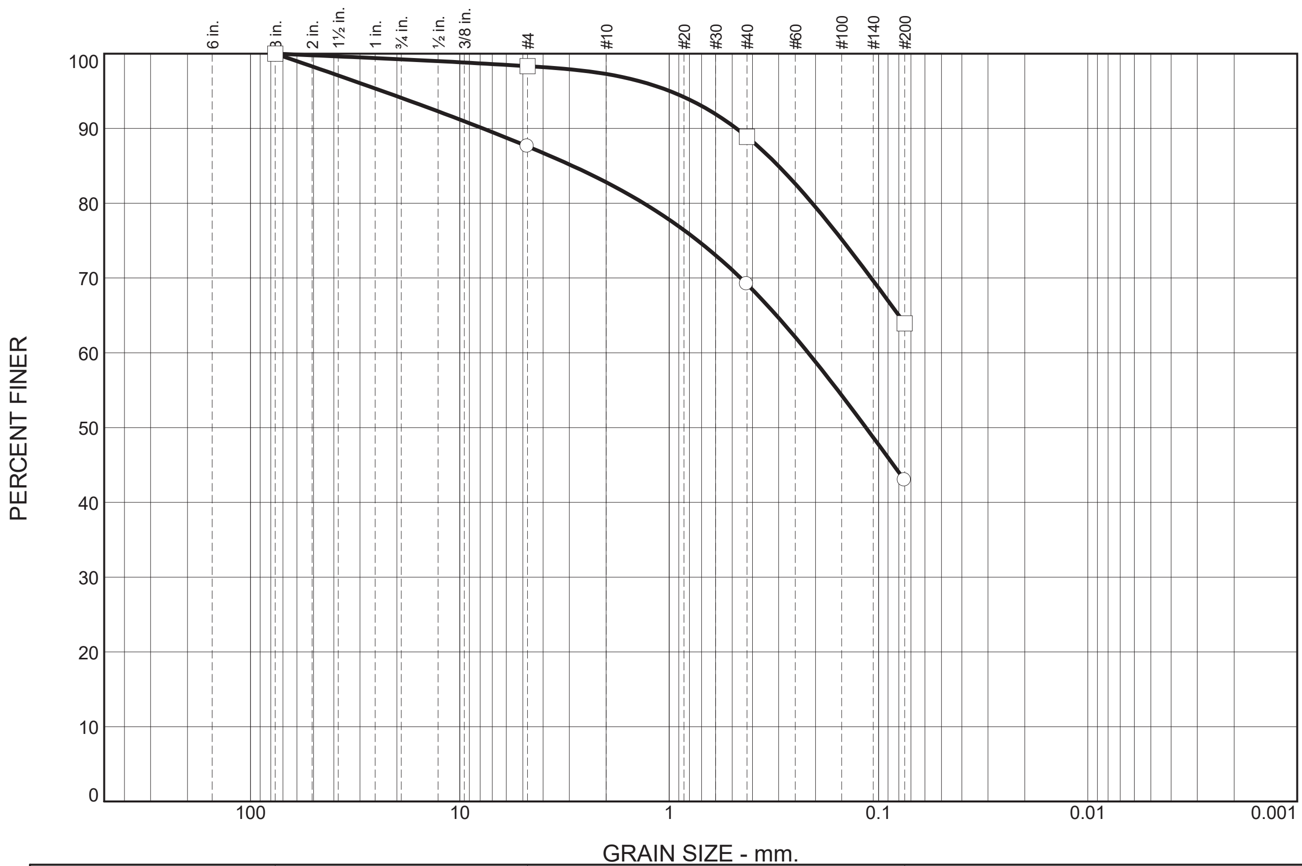
PLASTICITY CHART

Date 10/23/19 | Project No. 19-1740 | Figure B-1



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	5.9	6.5	4.8	13.6	26.2	43.0	
□	0.0	0.8	0.9	1.0	8.4	25.0	63.9	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	16.0'	CLAYEY SAND, red-brown	SC
□	B-2	25.0'	SANDY CLAY, gray-brown mottled with red and olive-brown	CL

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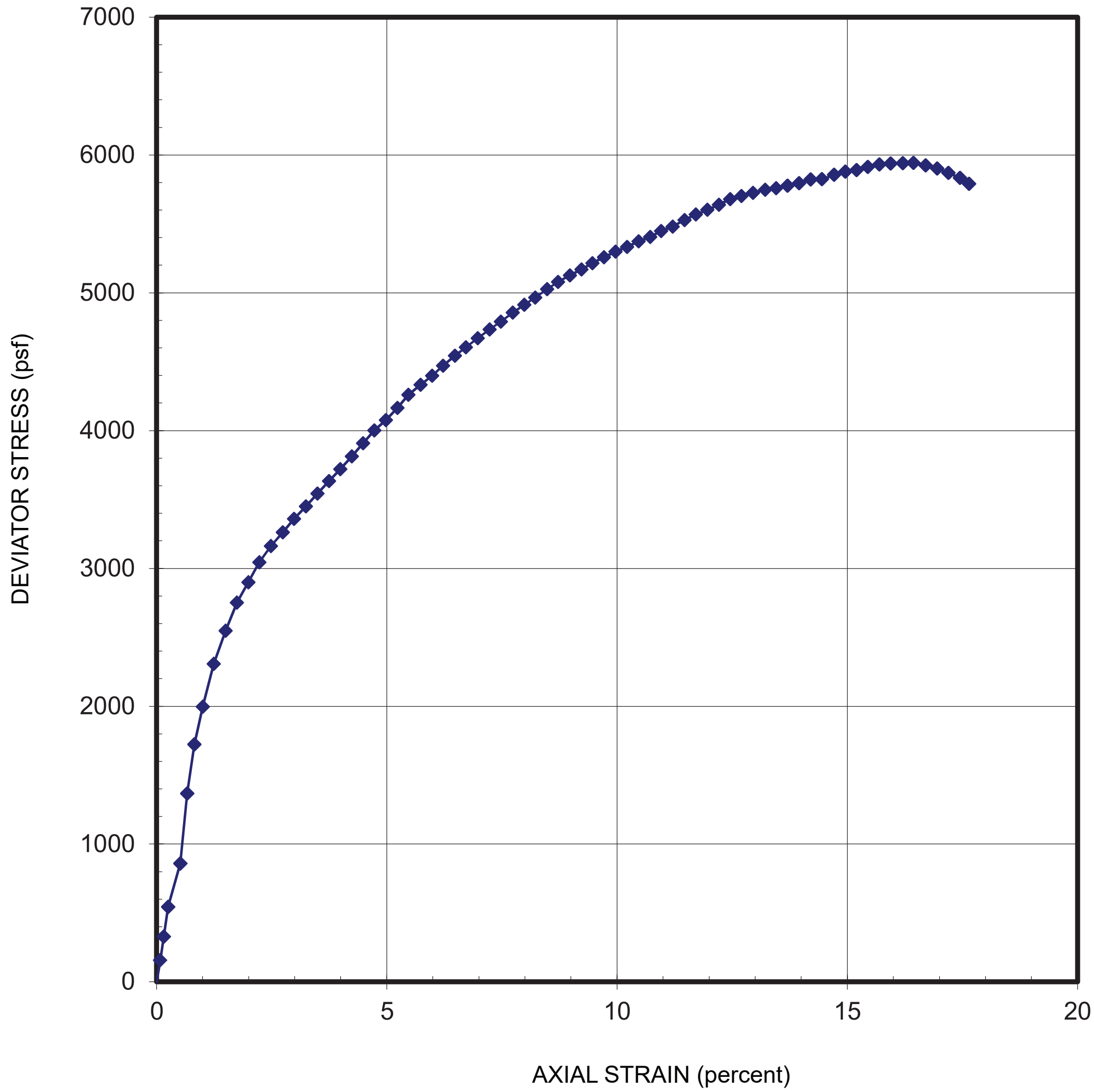
PARTICLE SIZE DISTRIBUTION REPORT

Date 10/23/19 | Project No. 19-1740 | Figure B-2



SAMPLER TYPE	Sprague and Henwood	SHEAR STRENGTH	2,970 psf
DIAMETER (in.) 2.39	HEIGHT (in.) 5.63	STRAIN AT FAILURE	16.4 %
MOISTURE CONTENT	20.4 %	CONFINING PRESSURE	750 psf
DRY DENSITY	108 pcf	STRAIN RATE	1 % / min.
DESCRIPTION	CLAY with SAND (CL), brown	SOURCE	B-1 at 6.0 feet

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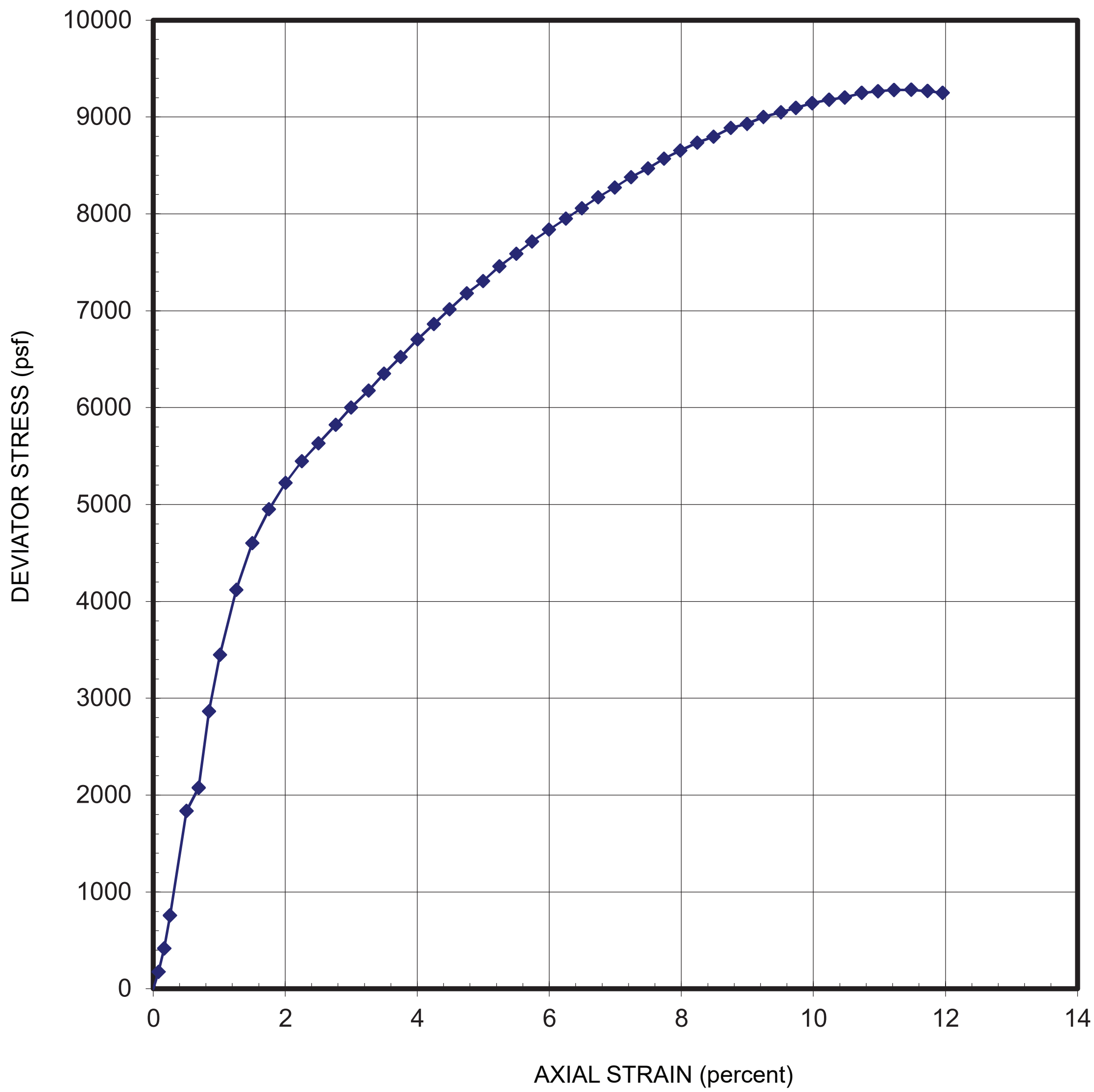
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 10/23/19 | Project No. 19-1740 | Figure B-3



SAMPLER TYPE	Sprague and Henwood	SHEAR STRENGTH	4,640 psf
DIAMETER (in.) 2.39	HEIGHT (in.) 5.61	STRAIN AT FAILURE	11.5 %
MOISTURE CONTENT	17.6 %	CONFINING PRESSURE	1,000 psf
DRY DENSITY	113 pcf	STRAIN RATE	1 % / min.
DESCRIPTION	CLAY with SAND (CL), brown	SOURCE	B-2 at 7.5 feet

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ROCKRIDGE GEOTECHNICAL

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 10/23/19 | Project No. 19-1740 | Figure B-4

Results Only Soil Testing
for
2136 San Pablo Avenue

September 20, 2019

Prepared for:
Quintin Flores
Rockridge Geotechnical, Inc.
271 Grand Ave,
Oakland, CA 94611
qaflores@rockridgegeo.com

Project X Job#: S190916D
Client Job or PO#: 19-1740

Respectfully Submitted,

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

Soil Analysis Lab Results

Client: Rockridge Geotechnical, Inc.
Job Name: 2136 San Pablo Avenue
Client Job Number: 19-1740
Project X Job Number: S190916D
September 20, 2019

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity**		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Flouride** F_2^-	**Phosphate** PO_4^{3-}
						As Rec'd	Minimum												
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-2-2	3.5-4.0	18.7	0.0019	3.1	0.0003	5,829	1,943	8.2	135.0	4.9	1.3	0.7	ND	18.1	4.0	30.7	11.9	1.8	2.5

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