March 4, 2021
Project No. 21-1973

Ms. Bridget Metz
Steel Wave
101 California Street, Suite 800
San Francisco, California 94111

Subject: Final Report
Proposed Parking Structure
2221 Fourth Street
Berkeley, California

Dear Ms. Metz:

We are pleased to present our geotechnical investigation report, dated March 4, 2021, for the proposed parking structure, called TheLAB Garage, to be constructed at 2221 Fourth Street in Berkeley, California. Our investigation was performed in accordance with our proposal dated January 21, 2021.

The subject property is a relatively level, L-shaped site with a length of 249 feet and a width varying from about 120 to 150 feet. It is currently occupied by two commercial/warehouse buildings and a vacant two-story residence. Plans are to construct a 4- to 5-story at-grade parking structure that will have plan dimensions of 122 by 248.33 feet.

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 for the proposed project are: 1) the presence of about 2 to 4 feet of undocumented fill blanketing the site, and 2) providing adequate foundation support for the proposed structure. We conclude the proposed new structure can be supported on conventional spread footings bottomed on firm native soil and/or engineered fill.

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

Enclosure

ROCKRIDGE
GEOTECHNICAL

Prepared for **Steel Wave**

GEOTECHNICAL INVESTIGATION
PROPOSED PARKING STRUCTURE
2221 FOURTH STREET
BERKELEY, CALIFORNIA

March 4, 2021
Project No. 21-1973

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ROCKRIDGE GEOTECHNICAL

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed parking structure, called TheLAB Garage, to be constructed at 2221 Fourth Street in Berkeley, California. The subject property is located on the eastern side of Fourth Street, between its intersections with Allston and Bancroft ways, as shown on the Site Location Map, Figure 1.

The subject property is a relatively level, L-shaped site with a length of 249 feet and a width varying from about 120 to 150 feet. It is bordered by an asphalt-paved parking lot and commercial building to the north, a commercial building to the south, Fourth Street to the west, and Fifth Street to the east. The site is currently occupied by two commercial/warehouse buildings and a vacant two-story residence.

Plans are to demolish the existing buildings on the site and to construct a 4- to 5-story at-grade parking structure that will have plan dimensions of 122 by 248.33 feet. The proposed parking structure will include bike parking and an electrical room on the ground floor. There will be storage rooms on all levels and portions open to below on the second and fifth levels. Plans also include an elevator on the western side of the structure and two staircases - one near the elevator and one on the southeastern side of the structure.

2.0 SCOPE OF SERVICES

Our investigation was performed in accordance with our proposal dated January 21, 2021. Our scope of services consisted of investigating subsurface conditions at the site by drilling two test borings, advancing two cone penetration tests (CPTs), performing one dynamic penetrometer test (DPT), performing laboratory tests 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 liquefaction-induced ground failure
- the most appropriate foundation type(s) for the proposed parking structure
- design criteria for the recommended foundation type, including vertical and lateral capacities
- estimates of foundation settlement
- lateral earth pressures and design groundwater level for design of elevator pit walls
- subgrade preparation for the concrete slab-on-grade floor for the garage and concrete sidewalks
- site grading and excavation, including criteria for fill quality and compaction
- 2019 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
- construction considerations.

3.0 FIELD INVESTIGATION

We investigated the subsurface conditions beneath the site by drilling two test borings, advancing two CPTs, and performing one DPT. The approximate locations of the test borings, CPTs, and DPT are shown on the Site Plan (Figure 2). Prior to mobilizing to the site, we contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained a private utility locator to check for existing utilities at each boring, CPT, and DPT location. We also obtained a drilling permit from the City of Berkeley Toxics Management Division (CBTMD). Details of the field exploration are described in the sections below.

3.1 Dynamic Penetrometer Test

We originally planned to perform a CPT near the western side of the site; however, the pavement and underlying soil were too weak to support the CPT outrigger and, therefore, a DPT was performed to determine the depth to the bottom of the weak soil at that location. The DPT was performed following the methodology presented in the technical paper titled *A Portable Dynamic Penetrometer for Geotechnical Investigations*, prepared by J.F. Triggs and P.D. Simpson. The

DPT consists of manually driving a 1.4-inch-diameter cone-tipped probe with a 30-pound hammer falling 15 inches. The blow counts required to drive the probe are recorded in 10-centimeter intervals. The blow counts required to drive the probe are recorded at 10-centimeter intervals and converted to Standard Penetration Test (SPT) N-values for use in our engineering analyses. DPT-1 was advanced to practical refusal (defined as 50 blows per 4 inches) in hard clay at a depth of approximately 4.6 feet bgs. The results of the DPT are presented on Figure A-1 in Appendix A.

3.2 Cone Penetration Tests

Our subsurface investigation included performing two CPTs, designated as CPT-1 and CPT-2, on February 3, 2021. The approximate locations of the CPTs are presented on the Site Plan, Figure 2. The CPTs were advanced to depths of approximately 50.7 and 82.4 feet bgs by Middle Earth Geo Testing, Inc. of Orange, California.

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. 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, frictional resistance, and pore water pressure 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, approximate strength characteristics, and liquefaction potential of the soil encountered. The CPT logs showing tip resistance, friction ratio, pore water pressure, and soil behavior type are presented in Appendix A on Figures A-2 and A-3.

Groundwater was measured in CPT-1 using a weighted tape measure at the end of the CPT. A pore pressure dissipation test was also performed in CPT-2. The interpreted water level associated with the dissipation test and the water level measured from the tape drop are presented on the respective CPT logs in Appendix A.

3.3 Test Borings

The test borings were drilled on February 3 and 5, 2021 by Exploration Geoservices Inc. of San Jose, California. The borings, designated as B-1 and B-2, were drilled to depths of approximately 41.5 and 51.5 feet bgs using a truck-mounted, Mobile B-53 drill rig equipped with hollow-stem augers. During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. The logs of the borings are presented on Figures A-4a through A-5b in Appendix A. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-6.

Soil samples were obtained from the boring using the following samplers:

- Modified California (MC) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter tubes.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.375-inch inside diameter, without liners.
- 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. In general, the MC sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the SPT sampler was used to evaluate the relative density of sandy soils. The MC and SPT samplers were driven with a 140-pound, downhole, wireline hammer falling about 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every six inches and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The blow counts required to drive the MC and SPT samplers were converted to approximate SPT N-values using factors of 0.7 and 1.0, respectively, to account for sampler type, and approximate hammer energy (previously measured by the drilling subcontractor). The blow counts used for this conversion were the last two 6-inch blow counts. The converted SPT N-values are presented on the boring logs. The D&M tubes were slowly advanced using the weight of the drill rods and hydraulic pressure, as needed.

After completion, the boring was backfilled with neat cement grout in accordance with CBTMD requirements. The soil cuttings generated by the boring were placed in 55-gallon drums and temporarily stored on site. The drums will be disposed of offsite after completion of analytical testing on the drum contents.

3.4 Laboratory Testing

We re-examined each soil sample obtained from our borings to confirm the field classifications and select representative samples for laboratory testing. Soil samples were tested to measure moisture content, dry density, Atterberg limits, undrained shear strength, and corrosivity. The Atterberg limits test is an indirect measurement of the expansion potential of soil. The results of the laboratory tests are presented on the boring logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS

Regional geologic information (Figure 3) indicates the site is underlain by Holocene-age alluvial fan and fluvial deposits (Qhaf). The results of our field investigation indicate the site is blanketed by approximately 2 to 4 feet of fill generally consisting of stiff sandy clay and medium dense clayey sand with gravel. A previous boring drilled on the site by Treadwell & Rollo, Inc. (see Figure 2, Site Plan for boring location) indicated 10 feet of fill was present; however, due to the high SPT N-values in the reported fill material, which were confirmed with CPT-2, we judge the material below a depth of about 3 to 4 feet bgs is native soil. Because of the past development of the site, we anticipate thicker fill may be present locally. The fill is underlain by native alluvium consisting of interbedded layers of stiff to hard clay with varying sand content and medium dense to very dense clayey sand with varying gravel content that extend to the maximum depth explored of 82.4 feet bgs.

Groundwater was measured in CPT-1, Boring B-2, and Boring B-1 at depths of 9, 12-1/2 and 19-1/2 feet bgs, respectively, prior to grouting. The groundwater level was also estimated at approximately 14-1/4 feet bgs using pore pressure dissipation test data from CPT-2. Considering the low permeability of the soil encountered in the borings and CPTs, it is likely there was not enough time for groundwater to stabilize. Subsurface Consultants, Inc. installed a well across

Fourth Street from the proposed garage site in 1988. Four reading taken in the well between June 1, 1988 and October 31, 1994 indicate the depth to groundwater varied from 9.57 to 10.15 feet bgs. Treadwell & Rollo, Inc. measured groundwater in this same well at a depth of 10.2 feet bgs on May 15, 2008.

The depth to groundwater is expected to vary several feet annually, depending on rainfall amounts. Considering the groundwater level readings discussed above were not taken during years with above-average rainfall, we anticipate the historic high groundwater is several feet above the previous measurements. Based on the available information, we conclude the historic high groundwater is approximately seven feet bgs at the site.

5.0 SEISMIC CONSIDERATIONS

The San Francisco Bay Area is one of the more seismically active regions in the world. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity

The site is in the Coast Ranges geomorphic province 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 and North American plates and subsequent strike-slip faulting along the San Andreas fault system. The San Andreas Fault is more than 600 miles long 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, Calaveras, and San Andreas faults. These and other faults in the region are shown on Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance and direction from the site, and characteristic moment magnitude [Petersen et al. (2014) & Thompson et al. (2016)] are summarized in Table 1 below. These references are based on the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013).

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Characteristic Moment Magnitude
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	4.1	East	7.58
Hayward (North, HN)	4.1	East	6.90
Hayward (South, HS)	14	Southeast	7.00
Mount Diablo Thrust North CFM	22	East	6.72
Total Calaveras (CN+CC+CS+CE)	23	East	7.43
Calaveras (North, CN)	23	East	6.86
Mount Diablo Thrust	23	East	6.67
Concord	26	East	6.45
Total North San Andreas (SAO+SAN+SAP+SAS)	27	West	8.04
North San Andreas (Peninsula, SAP)	27	West	7.38
Green Valley	27	Northeast	6.30
San Gregorio (North)	30	West	7.44
Clayton	32	East	6.57
North San Andreas (North Coast, SAN)	32	West	7.52
West Napa	34	North	6.97
Rodgers Creek - Healdsburg	36	North	7.19
Mount Diablo Thrust South	36	East	6.50
Greenville (North)	37	East	6.86
Great Valley 05 (Pittsburg - Kirby Hills alt1)	40	East	6.60
Monte Vista - Shannon	43	South	7.14
Great Valley 05 (Pittsburg - Kirby Hills alt2)	43	East	6.66

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, Mw, for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an Mw 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 Mw 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 Mw of 6.9 and occurred approximately 99 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 Mw for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (probably an Mw of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake (Mw = 6.2).

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

5.2 Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction,[1] lateral spreading[2] and cyclic densification.[3] We used the results of our borings and CPTs to evaluate the potential of these

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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, also referred to as differential compaction, is a phenomenon in which non-saturated, cohesionless soil is compacted by earthquake vibrations, causing ground-surface settlement.

phenomena occurring at the project site. The results of our analyses and evaluation are presented in the following sections.

5.2.1 Ground Shaking

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and 4) subsurface conditions. The site is about four kilometers from the Hayward Fault. Therefore, the potential exists for a large earthquake to induce strong to very strong ground shaking at the site during the life of the project.

5.2.2 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.

5.2.3 Liquefaction and Associated Hazards

Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction and lateral spreading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

The site has been mapped inside a zone of liquefaction potential on the map titled *State of California, Earthquake Zones of Required Investigation, Oakland West Quadrangle, Official Map,* prepared by the California Geological Survey (CGS), dated February 14, 2003 (Figure 5).

The California Geological Survey (CGS) has provided recommendations for procedures and report content for site investigations performed within seismic hazard zones in Special Publication 117 (SP-117), titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California*, dated September 11, 2008. SP-117 recommends subsurface investigations in mapped liquefaction hazard zones be performed using rotary-wash borings and/or CPTs.

Liquefaction susceptibility was assessed using the software CLiq v3.3.1.13 (GeoLogismiki, 2021). CLiq uses measured field CPT data and assesses liquefaction susceptibility and post-earthquake vertical settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). We performed the liquefaction triggering analyses using the methodology proposed by Boulanger and Idriss (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 the approximate in-situ groundwater depths measured in our CPTs and a "during earthquake" groundwater depth of seven feet bgs. In accordance with the 2019 CBC, we used a peak ground acceleration of 0.88 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 7.58 earthquake, which is consistent with the mean characteristic moment magnitude for the Hayward Fault, as presented in Table 1.

Our analysis indicates the underlying soils below the groundwater are not susceptible to liquefaction because of their cohesion; however, the analysis indicates that zones within the clay layers at depths between 7 and 44 feet bgs may experience pore pressure buildup and strength loss, referred to as cyclic softening, from cyclic loading during a major earthquake event. Dissipation of the excess pore pressures in the clay after the earthquake will result in ground surface settlement. We estimate total and differential ground settlement resulting from post-earthquake reconsolidation of the underlying clay following an MCE event with PGA_M of 0.88g will be on the order 1/2 inch and 1/4 inch across a horizontal distance of 30 feet, respectively.

Considering the site topography is relatively flat and the cohesive nature of the soil underlying the site, we conclude the risk of lateral spreading is nil.

5.2.4 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The soil encountered above the groundwater table is not susceptible to cyclic densification due to its cohesion. Therefore, we conclude the potential for cyclic densification to occur at the site is low.

6.0 DISCUSSIONS AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical issues affecting the proposed development include: 1) the presence of about 2 to 4 feet of undocumented fill blanketing the site, and 2) providing adequate foundation support for the proposed structure. These and other issues are discussed in this section.

6.1 Foundation Support and Settlement

Based on the results of our subsurface investigation, we anticipate the foundation level of the proposed garage will be underlain by very stiff sandy clay and medium dense to dense clayey sand. These materials have moderate strength and relatively low compressibility. Therefore, we conclude the proposed garage can be supported on conventional spread footings bottomed on firm native soil and/or engineered fill.

We estimate total static settlement of the proposed building supported on spread footings will be less than one inch and differential settlement will not exceed 3/4 inch in 30 feet. Additional total and differential settlements of up to 1/2 inch and 1/4 inch over a horizontal distance of 30 feet, respectively, may occur following a major earthquake as excess pore pressures dissipate in soil layers in which cyclic softening occurs.

6.2 Excavations, Temporary Cut Slopes and Shoring

The soil to be excavated mostly consists of sandy clay and clayey sand, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. If building pad grading and foundation construction will be performed during the rainy season, it would be prudent to place 2 to 3 inches of unreinforced concrete (a "rat slab") over the building pad subgrade and the footing excavation bottoms to protect against softening of the soil due to standing water.

Excavations that will be deeper than four feet and will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The sides of excavations may be sloped where space permits. Where sloping of the excavation is not feasible, shoring will be required. We judge that a cantilevered soldier pile and timber lagging shoring system is appropriate for support of excavations that are less than about 12 feet deep. The contractor should be responsible for the construction and safety of temporary slopes and shoring.

6.3 Soil Corrosivity

Corrosivity tests were performed by Project X Corrosion Engineering of Murrieta, California on selected soil samples obtained from Boring B-1 at 5 feet bgs and from Boring B-2 at 3 feet bgs. The corrosivity test results are presented in Appendix B of this report.

Many factors can affect the corrosion potential of soil including, but not limited to, resistivity, pH, and chloride and sulfate concentrations. Based on the resistivity test results (1,608 to 2,211 ohm-cm), we conclude the surficial clayey soil is "highly corrosive[4]" to buried metal. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron should be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.

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[4] Roberge, Pierre R. (2018). *Corrosion Basics, an Introduction, Third Edition*. NACE International, P. 189.

The results indicated that sulfate ion concentrations (25.6 to 105.5 mg/kg) are insufficient to damage reinforced concrete structures below ground and the chloride concentrations (7.2 to 13.2 mg/kg) of the soil do not present a problem with reinforcing steel in the buried concrete structures. The pH test results (6.4 to 7.1) indicate the soil is "mildly corrosive".

7.0 RECOMMENDATIONS

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

7.1 Site Preparation and Grading

Site clearing should include removal of all existing buildings and their foundations, pavements, and underground utilities. Any vegetation and organic topsoil (if present) should be stripped in areas to receive improvements (i.e., building, pavement, or flatwork). After site clearing is completed, the building footprint should be excavated to three feet below the existing ground surface and the soil should be stockpiled on site. We should be retained when the excavation is complete to confirm all undocumented fill has been excavated. The exposed subgrade should then be scarified to a depth of eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. The stockpiled soil should then 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[5] to create a smooth, non-yielding surface. If fill will be placed below the proposed bottom-of-footing elevation, it should be compacted to at least 95 percent relative compaction. The soil subgrade should be kept moist until it is covered by the capillary break (if used) or Class 2 aggregate base (AB).

Material excavated at the site will primarily consist of sandy clay and clayey sand that may be reused as backfill. If on-site soil will be used as fill within one foot of the building pad subgrade,

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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.

it should consist of soil with a plasticity index (PI) of less than 15. If imported fill (select fill) is required, it should be 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 15, and be approved by the Geotechnical Engineer. 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. Fill consisting of clean sand or gravel (defined as soil with less than 10 percent fines by weight), fill greater than five feet in thickness, and fill placed below footings should be compacted to at least 95 percent relative compaction.

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

7.1.1 Exterior Concrete Flatwork

We recommend a minimum of four inches of Class 2 aggregate base be placed below exterior concrete flatwork, including on-site and public sidewalks. The soil subgrade exposed should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. The soil subgrade should be kept moist until it is covered by AB. The AB should be moisture-conditioned to near optimum moisture content and compacted to at least 90 percent relative compaction.

7.1.2 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. Backfill for utility trenches and other excavations is also considered fill and should be placed and compacted according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Spread footings should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of existing and new utility trenches that run parallel to the footings. Alternatively, for new utility trenches, 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 50 pounds per square inch (psi) or Class 2 AB compacted to at least 95 percent relative compaction.

7.1.3 Drainage and Landscaping

Positive surface drainage should be provided around the garage to direct surface water away from the foundations. To reduce the potential for water ponding adjacent to the structure, we recommend the ground surface within a horizontal distance of five feet from the structure 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 clay subgrade. Bioswales constructed at the site should be provided with underdrains and/or drain inlets because of the low permeability of the near-surface soil.

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

7.2 Foundation Design

As discussed above, spread footings bottomed on firm native soil and/or engineered fill may be used to support the proposed garage. The recommendations for spread footings may also be used to design the mat foundation for the elevator pit.

Continuous footings and isolated spread footings should be at least 18 and 24 inches wide, respectively. Footings should bottom at least 24 inches below the building pad subgrade (i.e., bottom of capillary break or AB) or 36 inches below the lowest adjacent exterior subgrade (for exterior footings), whichever is lower. Spread footings for the garage should also bottom below the zone-of-influence line for the elevator pit, which is defined as an imaginary line extended upward at an inclination of 1.5:1 from the bottom of the elevator pit mat foundation. Similarly, footings and the elevator pit mat foundation should bottom below a 1.5:1 line extending up from the bottom of the existing footings supporting the neighboring buildings. If it is necessary to deepen the new footings significantly to achieve these criteria, the lower portion of the footing excavation (i.e., the portion below the minimum footing embedment depth) may be backfilled with CLSM with a minimum 28-day unconfined compressive strength of 100 psi.

Footings may be designed using allowable bearing pressures of 5,000 pounds per square foot (psf) for dead-plus-live loads and 6,600 psf for total design loads, which includes wind or seismic forces. These values include factors of safety of at least 2.0 and 1.5, respectively. We recommend a modulus of vertical subgrade reaction (k_{v1}) of 440 pounds per cubic inch (pci) be used for design of footings and the elevator pit mat subgrade. This modulus value should be scaled to account for footing width (B) using the following equation[6]:

$$k_s = \frac{k_{v1}}{B} [(m+0.5)/1.5m]$$

[6] For example, for a nine-foot-square footing, B would be 9 and m would be 1. The resulting scaled modulus of vertical subgrade reaction would be 50 pci.

Where: B = Width of loaded area

k_{v1} = Modulus of vertical subgrade reaction for one-foot-square plate

mB = Length of loaded area

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 underlying soil. To compute lateral resistance for footings, we recommend using a uniform pressure of 2,000 psf for transient load conditions and an equivalent fluid weight of 300 pounds per cubic foot (pcf) for sustained load conditions. The upper foot of soil should be ignored when computing passive resistance unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.3. If waterproofing is installed below the elevator pit mat foundation, frictional resistance provided by the elevator pit should be computed using design base friction values of 0.2 and 0.12 for Preprufe (or equivalent) and bentonite-based waterproofing membranes, respectively. 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.

Footing excavations should bottom in very stiff/dense native soil and/or engineered fill and should be free of standing water, debris, and weak or disturbed materials prior to placing concrete. The bottoms and sides of the footing excavations must be maintained in a moist condition until concrete is placed. If footings will be constructed during the rainy season, we strongly recommend a two-inch-thick unreinforced concrete “rat” slab be placed on the bottom of the footing excavations to protect the footing excavation subgrade from softening if exposed to rain. The CLSM used to construct the rat slabs should have a minimum 28-day compressive strength of 100 psi and be placed within two days of footing excavation. We should check footing excavations prior to placement of reinforcing steel. If a rat slab will be used, we should check the excavations prior to placement of the rat slab.

In general, we recommend all footings be founded below an imaginary plane extending up at an inclination of 1.5:1 (horizontal to vertical) from the base of any vault, utility trench, bioswale/storm water treatment area, and footings for neighboring buildings. If the design footing elevation is above this plane, the footing can either be deepened or over-excavated below the influence line and backfilled with CLSM to the design footing elevation.

7.3 Concrete Slab-on-Grade Floor

Where water vapor transmission through the floor slab is not desirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor slab. 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 2.

TABLE 2
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. 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. 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; however, we recommend a capillary break and vapor retarder be placed in areas where there is a floor covering, areas used for storage, and any enclosed rooms. 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.

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. 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 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 Retaining Walls

Permanent retaining walls should be designed to resist lateral earth pressure imposed by the retained soil, as well as a surcharge pressure from nearby foundations and vehicles, where appropriate. Below-grade retaining walls (i.e., elevator pit walls) 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. In addition, because the site is in a seismically active area, retaining walls that retain more than six feet of soil should be designed to resist pressures associated with seismic forces. We recommend restrained retaining walls at the site be designed for the more critical of:

- at-rest pressure using an equivalent fluid weight of 55 pcf (triangular distribution); or
- active pressure using an equivalent fluid weight of 35 pcf (triangular distribution) plus a seismic increment of 37 pcf (triangular distribution)

To avoid surcharging the elevator pit 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 (horizontal:vertical) from the bottom of the below-grade walls.

The lateral earth pressures recommended above are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. 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 140NC or equivalent). Where shoring is installed and there is insufficient room to install a perforated pipe between the shoring and the back of the wall, 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. The pipe should be connected to a suitable discharge point; a sump and pump system may be required to drain the collect pipes.

Where there will be vehicular traffic behind the top of a permanent wall within a horizontal distance equal to 1.5 times the height of the wall, the upper 10 feet of the wall should be designed for vehicular surcharge of 50 psf (uniform). Where existing foundations are supported above a "zone-of-influence" line extending up from a permanent wall at an inclination of 1.5:1 (horizontal: vertical), the wall should be designed for a surcharge pressure. We can provide the recommended surcharge pressure once the size and loading on the foundation are known.

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

7.5 Temporary Cut Slopes and Shoring

Excavations that will be deeper than four feet and will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). We conclude the soil underlying the site be classified as a Type A soil according to the CAL-OSHA classification system. The maximum allowable slope for Type A soil is 1:1 (horizontal to vertical).

Where sloping of the excavation is not feasible, shoring will be required. We judge that a cantilevered soldier-pile-and-lagging shoring system is appropriate for support of excavations that are less than about 12 feet deep.

7.5.1 Cantilevered Soldier-Pile-and-Lagging Shoring System

For design of a cantilevered shoring system, we recommend using an active earth pressure equivalent to a fluid weight of 35 pcf. Where existing buildings are within a horizontal distance equal to 1.5 times the height of the shoring from the edge of excavation, we recommend using at-rest earth pressure equivalent to a fluid weight of 55 pcf. Where traffic loads are expected within 10 feet of the shoring walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall. Shoring should be designed for surcharge loads where there will be construction equipment and/or stockpiled soil within a horizontal distance equal to 1.5 times the height of the shoring from the edge of excavation. We can provide recommendations for surcharge pressures once surcharge loads are known.

Lateral resistance can be gained by passive pressure acting on the face of the soldier piles. We recommend using an equivalent fluid weight of 300 pcf for passive resistance. This value includes a factor of safety of at least 1.5. Passive pressure can be assumed to act over an area of three soldier pile widths. The upper foot of soil should be ignored when computing passive resistance.

Soldier piles should be placed in pre-drilled holes backfilled with concrete. Installing soldier piles by driving or using vibratory methods may be attempted but should not be used within 25 feet of existing buildings.

A structural/civil engineer knowledgeable in this type of construction should be retained to design the shoring. We should review the final shoring plans to check that they are consistent with the recommendations presented in this report.

7.6 Seismic Design

Results of shear wave velocity measurements at the CPT-2 location indicate the average shear wave velocity is 1,020 foot per second for the upper 80 feet of soil. Therefore, we conclude the Site Class D designation, in accordance with the 2019 CBC, may be used for design. The latitude and longitude of the site are 37.8643° and -122.2983°, respectively. For design in accordance with 2019 CBC, we recommend the following:

- Site Class D – stiff soil
- $S_S = 1.895$, $S_1 = 0.724g$

The 2019 CBC is based on the guidelines contained within ASCE 7-16 which stipulates that where S_1 is greater than 0.2 times gravity (g) for Site Class D, a ground motion hazard analysis is needed unless the seismic response coefficient (C_s) value will be calculated as outlined in Section 11.4.8, Exception 2. Assuming the C_s value will be calculated as outlined in Section 11.4.8, Exception 2, we recommend the following seismic design parameters:

- $F_a = 1.0$, $F_v = 1.7$
- $S_{MS} = 1.895g$, $S_{M1} = 1.231g$
- $S_{DS} = 1.263g$, $S_{D1} = 0.821g$
- Seismic Design Category D for Risk Factors I, II, and III

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

9.0 LIMITATIONS

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

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ROCKRIDGE
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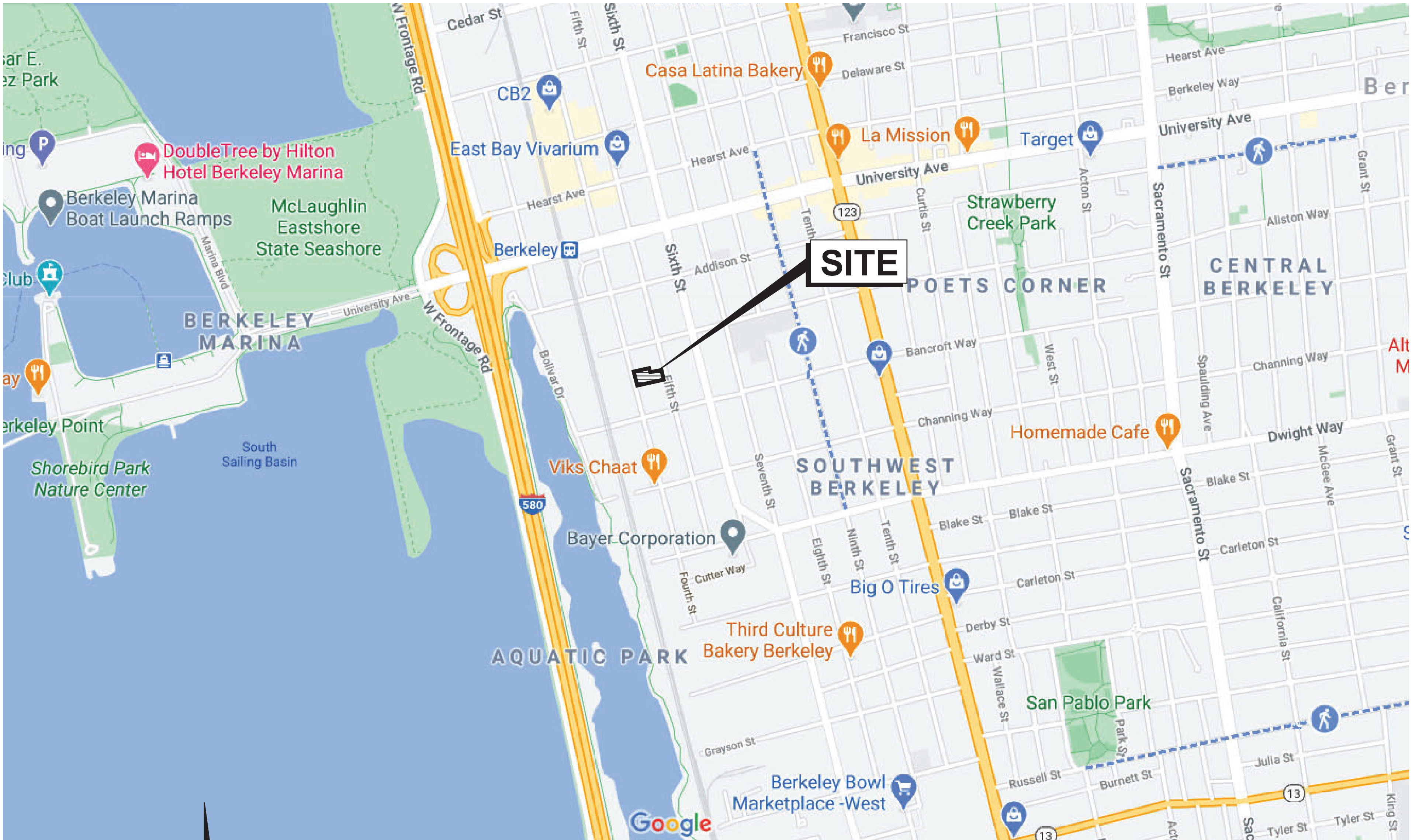
FIGURES



Base map: Google Maps, 2021.

0 | 1,300 | 2,600 Feet

Approximate scale

THE LAB GARAGE
2221 FOURTH STREET
Berkeley, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 02/22/21	Project No. 21-1973	Figure 1

EXPLANATION

CPT-1	Approximate location of cone penetration test by Rockridge Geotechnical, Inc., February 3, 2021
B-1	Approximate location of boring by Rockridge Geotechnical, Inc., February 3 & 5, 2021
DPT-1	Approximate location of dynamic penetrometer test by Rockridge Geotechnical, Inc., February 5, 2021
CPT-3	Approximate location of cone penetration test by Rockridge Geotechnical, Inc., May 7, 2016
B-2	Approximate location of boring by Rockridge Geotechnical, Inc., May 7, 2016
B-4	Approximate location of boring by Treadwell & Rollo, Inc., March 2008

Reference: Base map from a drawing titled "Option - 4A, Ground Level Plan", by Steelwave, dated December 30, 2020.

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Berkeley, California

SITE PLAN

Date 02/24/21	Project No. 21-1973	Figure 2

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Base map: Google Earth with U.S. Geological Survey (USGS), Alameda County, 2016.

EXPLANATION

- **af** — **Artificial Fill**
- **Qha** — **Alluvium (Holocene)**
- **Qpa** — **Alluvium (Pleistocene)**
- **Qs** — **Beach and dune sand (Quaternary)**
- **Geologic contact: dashed where approximate and dotted where concealed, queried where uncertain**

0 1000 2000 Feet

Approximate scale

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Berkeley, California

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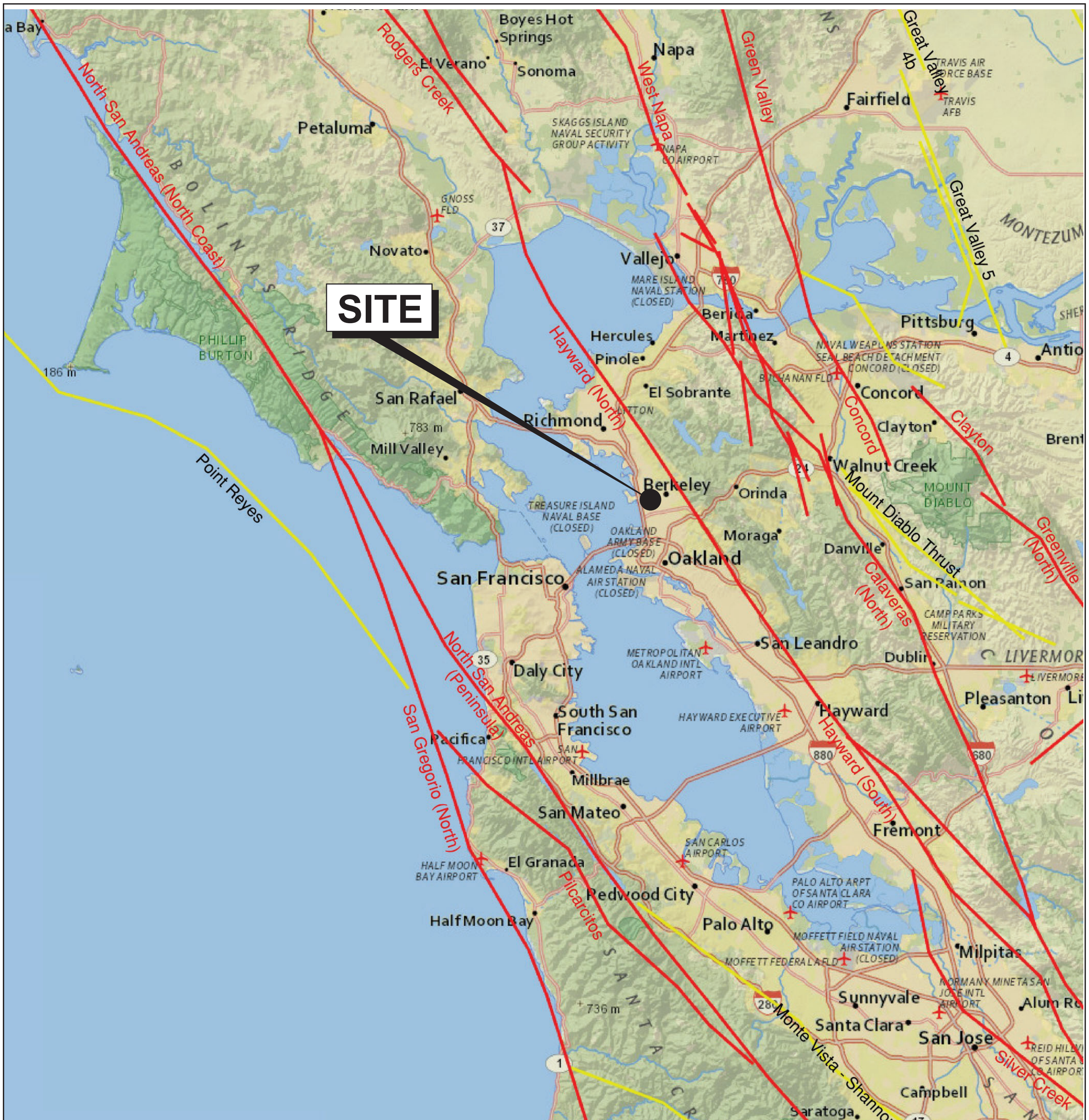
REGIONAL GEOLOGIC MAP

Date 02/22/21	Project No. 21-1973	Figure 3



Base Map: U.S. Geological Survey (USGS), National Seismic Hazards Maps - Fault Sources, 2014.

EXPLANATION

Strike slip

Thrust (Reverse)

0 5 10 Miles

Approximate scale

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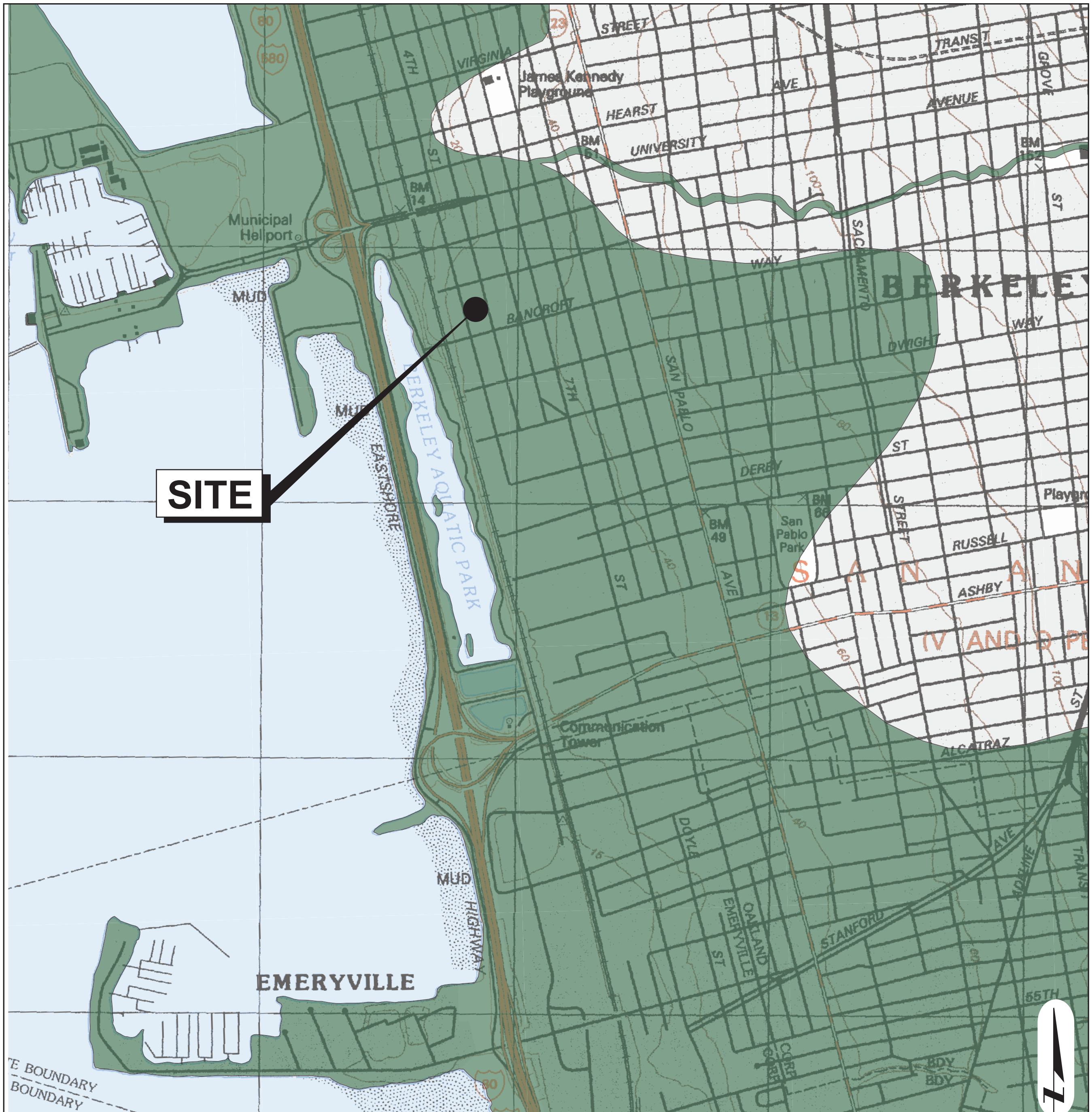
REGIONAL FAULT MAP

Date 02/22/21	Project No. 21-1973	Figure 4



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

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

Reference:
Earthquake Zones of Required Investigation
Oakland West Quadrangle
California Geological Survey
Released February 14, 2003

0 2,000 4,000 Feet
Approximate scale

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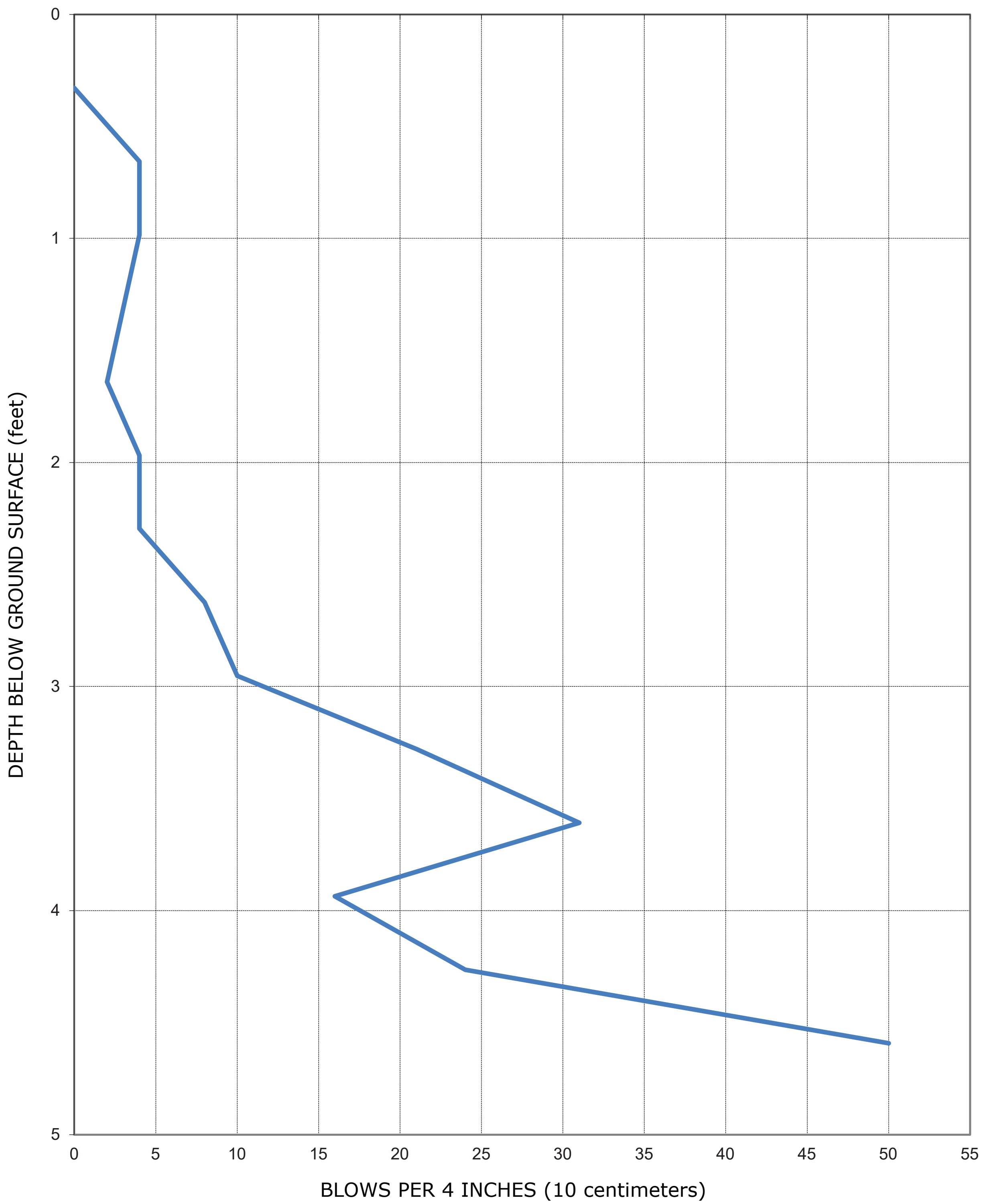
ROCKRIDGE GEOTECHNICAL

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

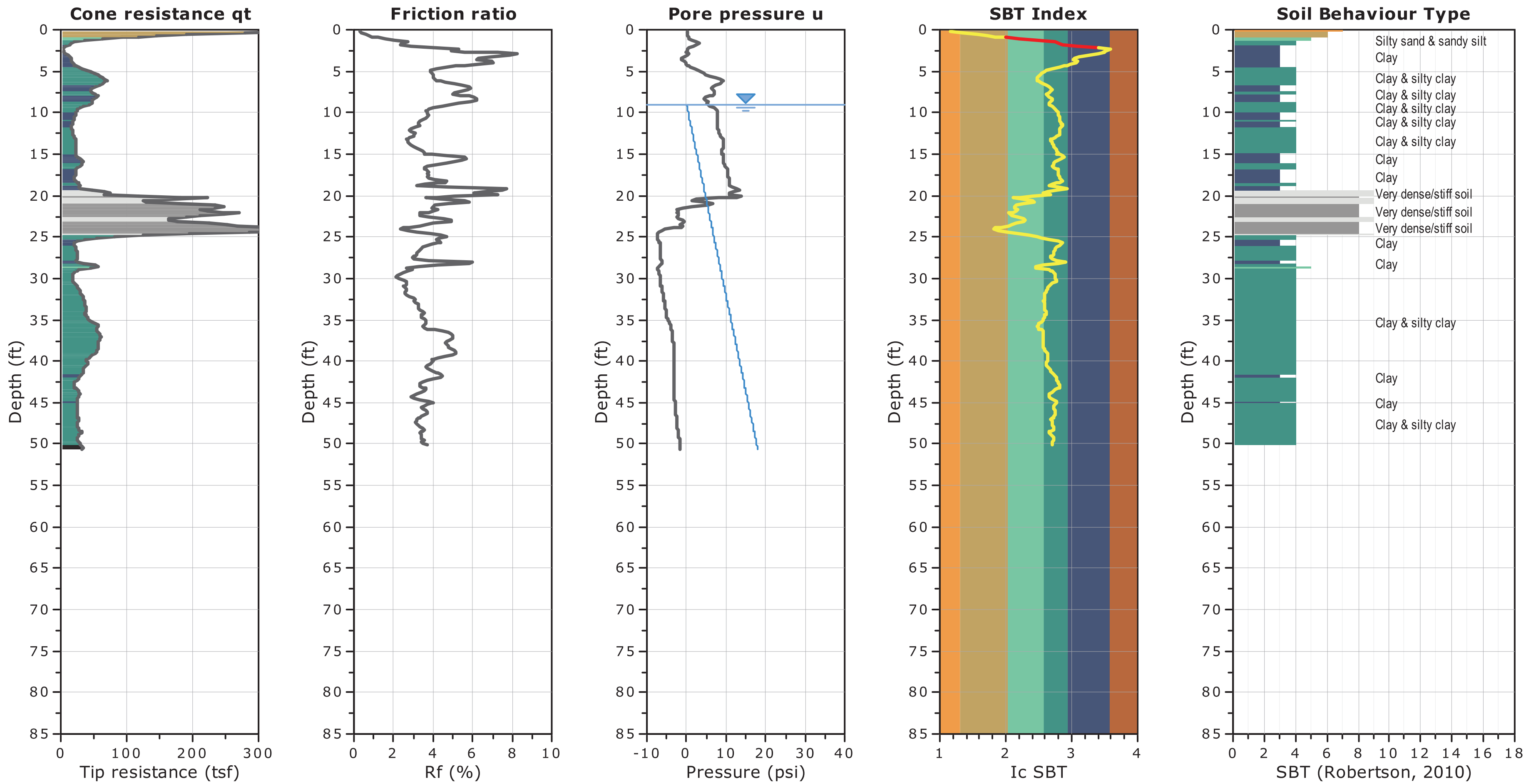
Date 02/22/21	Project No. 21-1973	Figure 5

APPENDIX A

Cone Penetration Test Results and Boring Logs



THE LAB GARAGE **2221 FOURTH STREET** Berkeley, California	**DYNAMIC PENETROMETER** **TEST RESULTS** **DPT-1**
ROCKRIDGE GEOTECHNICAL	Date 02/22/21 | Project No. 21-1973 | Figure A-1



Total depth: 50.69 ft, Date: Febuary 3, 2021
Depth to Groundwater: 9.0 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

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

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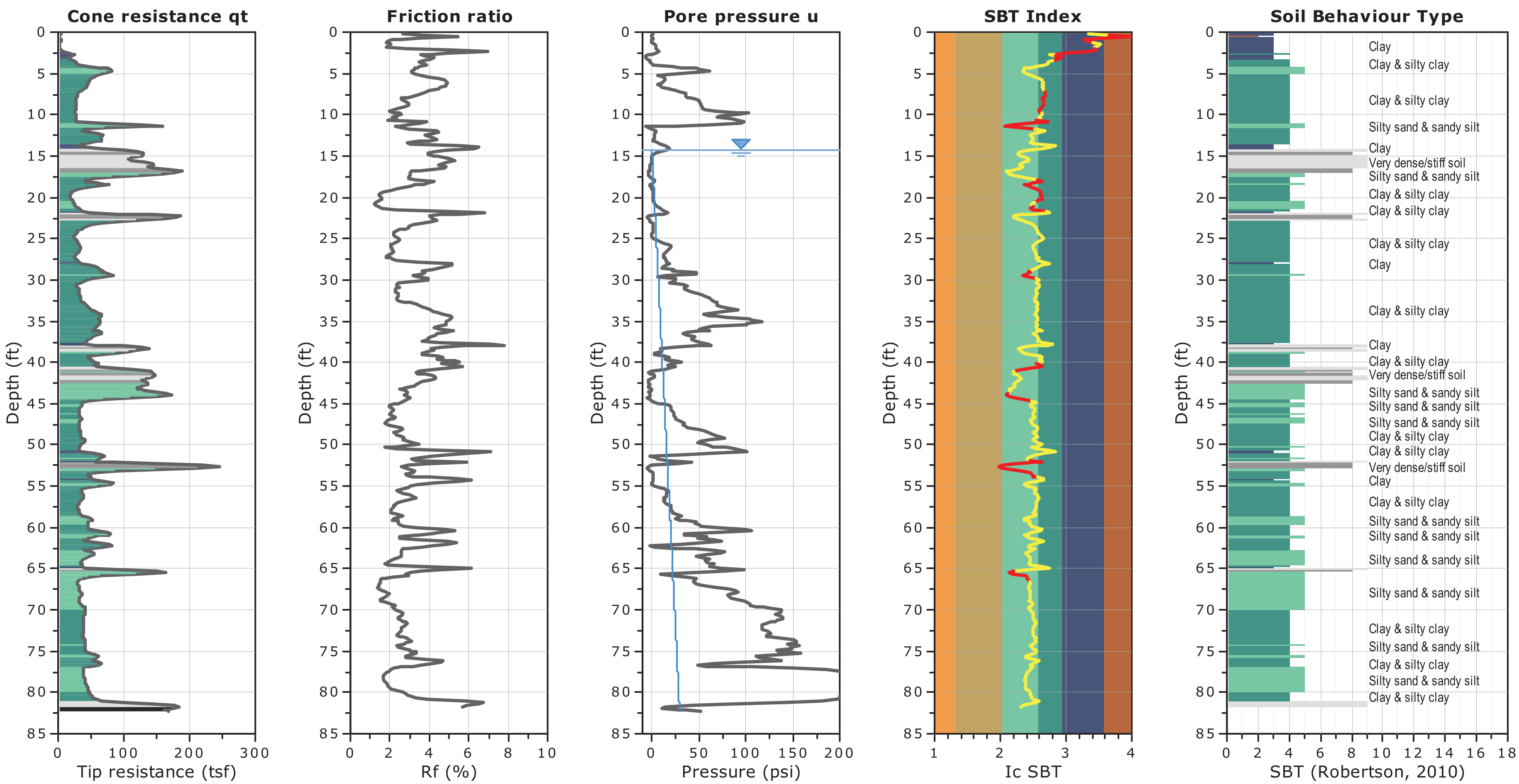
CONE PENETRATION TEST RESULTS
CPT-1

Date 02/22/21	Project No. 21-1973	Figure A-2



Total depth: 82.35 ft, Date: Febuary 3, 2021
Depth to Groundwater: 14.3 feet (pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

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

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ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 02/22/21	Project No. 21-1973	Figure A-3

PROJECT:	THE LAB GARAGE 2221 FOURTH STREET Berkeley, California	# Log of Boring B-1
		PAGE 1 OF 2

Boring location: See Site Plan, Figure 2

Date started: 02/05/2021 | Date finished: 02/05/2021

Drilling method: Hollow-Stem Auger

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

Sampler: Modified California (MC), Standard Penetration Test (SPT), Dames & Moore (D&M)

Logged by: J. Graham
Drilled by: Exploration Geoservices, Inc.
Rig: B-53 Blue

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1				CL	SANDY CLAY (CL) dark brown, stiff, moist, brick debris (FILL)						
2	MC	3 7 9	11	SC	CLAYEY SAND with GRAVEL (SC) red-yellow, medium dense, moist (FILL)						
4	MC	8 12 20	22	CL	CLAY with SAND (CL) olive-gray, very stiff, moist increasing sand content in bottom 6" LL = 30, PI = 13; see Appendix B					20.7	108
6	MC	14 28 41	48	CL	Soil Corrosivity Test; see Appendix B red-yellow, hard, decreasing sand content TxUU Test; see Appendix B	TxUU	550	4,450		19.4	108
8	MC	11 21 25	32	CL							
11	MC	7 10 11	15	CL	SANDY CLAY (CL) brown, stiff, moist, very fine to fine sand TxUU Test; see Appendix B	TxUU	1,100	1,700		27.0	99
14				CL	CLAY with SAND (CL) brown, very stiff, wet, trace fine subangular gravel						
16	MC	7 10 18	20	CL							
19–20				CL	▼ (02/05/2021; 9:30 AM) ▽ (02/05/2021; 9:10 AM)						
21	DM			SC	CLAYEY SAND with GRAVEL (SC) red-yellow, very dense, wet						
23	SPT	11 25 43	68	SC	CLAYEY SAND (SC) brown, very dense, wet, medium to coarse sand trace fine subangular to angular gravel						
25				CL	CLAY with SAND (CL) brown, very stiff, wet						
26	MC	8 14 18	22	SC	CLAYEY SAND with GRAVEL (SC) brown, medium dense, wet						
29–30				CL	CLAY with SAND (CL)						

LABORATORY TEST DATA

ROCKRIDGE GEOTECHNICAL

Project No.: 21-1973

Figure: A-4a

PROJECT: **THE LAB GARAGE**
2221 FOURTH STREET
Berkeley, California

Log of Boring B-1

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
30–35	D&M	200-400 psi		CL	CLAY with SAND (CL) (continued) brown, wet, trace angular gravel						
35–40	MC	14 28 44	50	CL	SANDY CLAY (CL) brown, hard, wet, trace fine gravel						
40–41.5	MC	12 24 40	45	CL	CLAY with SAND (CL) gray-brown, hard, wet						

Boring terminated at a depth of 41.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at depths of 19.5 & 20 feet during drilling.

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

ROCKRIDGE GEOTECHNICAL

Project No.: 21-1973

Figure: A-4b

PROJECT:	THE LAB GARAGE 2221 FOURTH STREET Berkeley, California	**Log of Boring B-2** PAGE 1 OF 2

Boring location: See Site Plan, Figure 2	Logged by: J. Graham
Date started: 02/03/2021 / Date finished: 02/03/2021	Drilled by: Exploration Geoservices, Inc.
Drilling method: Hollow-Stem Auger	Rig: B-53 Blue
Hammer weight/drop: 140 lbs./30 inches / Hammer type: Downhole Safety Hammer	
Sampler: Modified California (MC), Standard Penetration Test (SPT), Dames & Moore (D&M)	

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
0–1.6					CL	SANDY CLAY with GRAVEL (CL) black, very stiff, moist, wood debris (FILL)						
1–2.5	MC		13 17 14	22	CL	CLAY with SAND (CL) gray-brown mottled black, very stiff, moist Soil Corrosivity Test; see Appendix B						
3–4.5	MC		7 13 19	22		gray-brown with yellow-brown mottling						
5–6.5	MC		11 23 29	36		hard TxUU Test; see Appendix B	TxUU	550	4,350		18.8	111
7–8.5	MC		12 24 30	38	CL	SANDY CLAY (CL) yellow-brown with yellow mottling, hard, moist						
9						increasing sand content with depth						
10–11.5	MC		23 25 29	38	SC	CLAYEY SAND with GRAVEL (SC) brown, dense, moist					13.8	117
12.3						▼ (02/03/2021; 9:15 AM)						
14.8						▽ (02/03/2021; 9:05 AM) gray-brown, wet, fine to coarse sand						
15–16.5	MC		28 18 31	34								
20–21.5	SPT		14 32 12	44								
25–26.5	MC		7 13 15	20	CL	SANDY CLAY (CL) yellow- brown, very stiff, wet, gravel from 25.2 to 25.5 feet						
29–30					CL	CLAY with SAND (CL) brown, hard, wet, fine to medium sand						

ROCKRIDGE GEOTECHNICAL

Project No.: 21-1973

Figure: A-5a

PROJECT: **THE LAB GARAGE**
2221 FOURTH STREET
Berkeley, California

Log of Boring B-2

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
30–31.5	MC		18 20 31	36	CL	CLAY with SAND (CL) (continued)						
32.5–34	D&M		500 psi		CL	trace angular gravel						
35–36.5	MC		12 22 29	36	CL	SANDY CLAY (CL) yellow-brown, hard, wet						
40–41.5	SPT		14 29 44	73	CL	CLAYEY SAND with GRAVEL (SC) brown, very dense, wet, angular gravel						
43					CL	CLAY with SAND (CL) red-yellow, very stiff, wet						
45–46.5	MC		6 10 13	16	CL							
50–51.5	D&M		200 psi		CL	decreasing sand content						

Boring terminated at a depth of 51.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at depths of 12.5 & 15 feet during drilling.

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

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Figure: A-5b

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 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 California or Modified California split-barrel sampler. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

Sonic

SAMPLER TYPE

C Core barrel

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

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

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

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

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

SPT Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter

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

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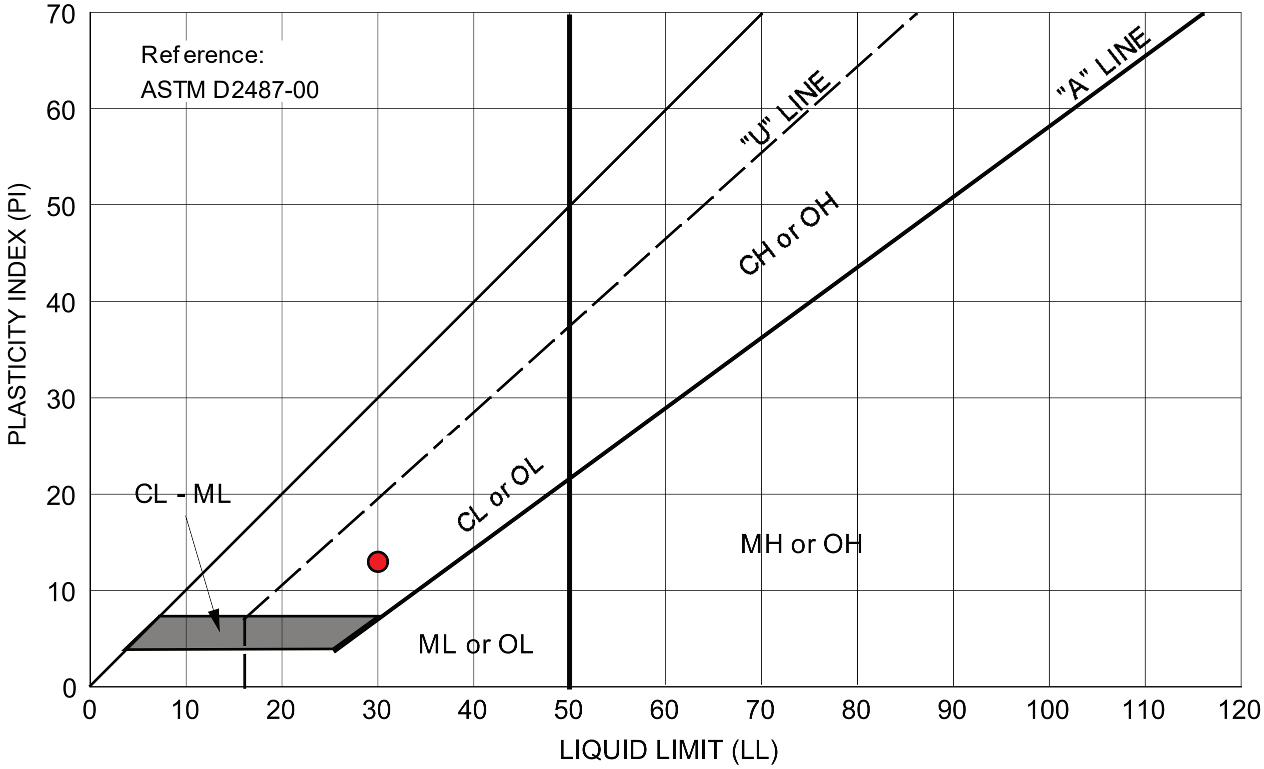
CLASSIFICATION CHART

Date 02/22/21 | Project No. 21-1973 | 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 4.0 feet	CLAY with SAND (CL), olive-gray	20.7	30	13	--

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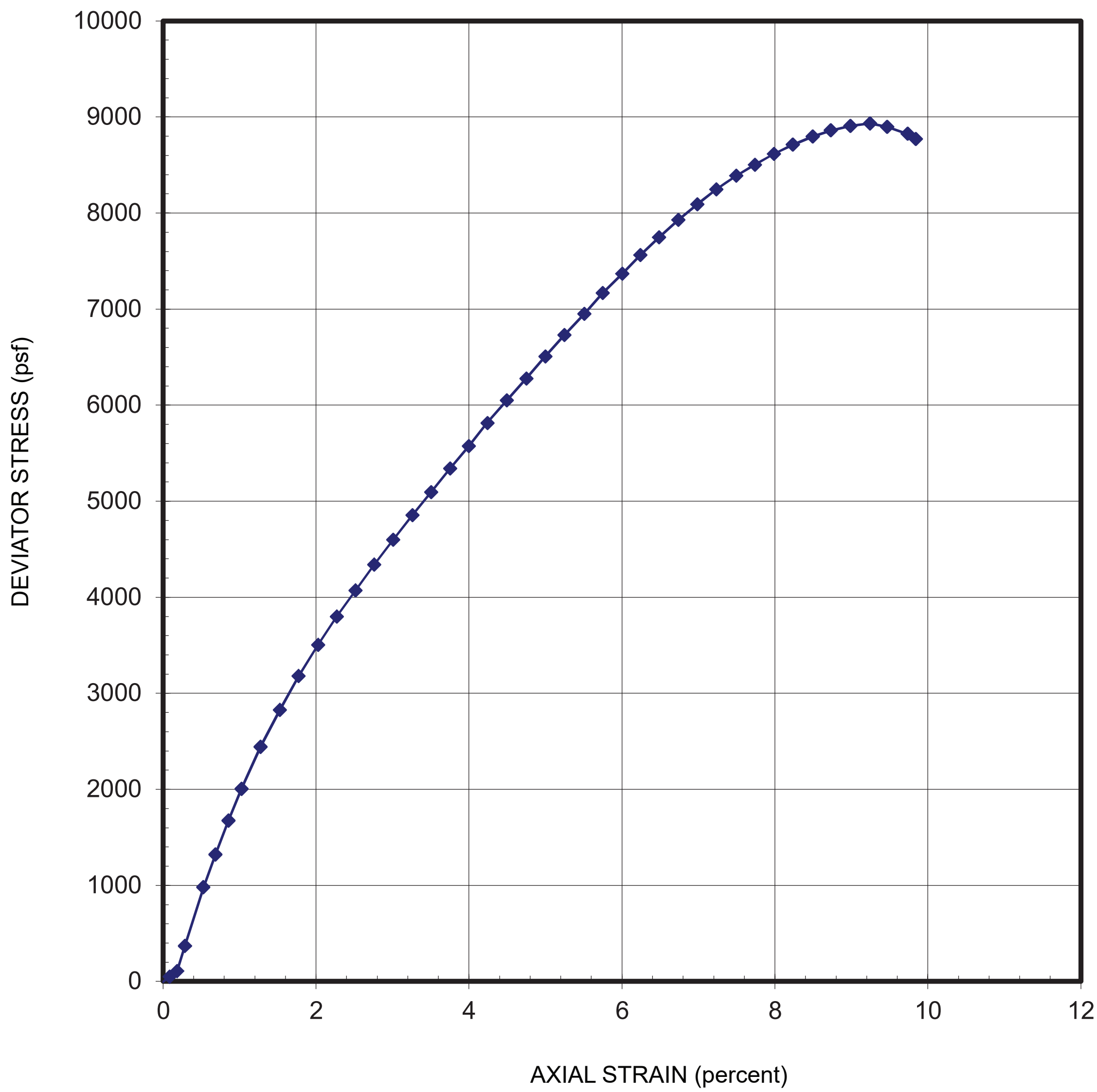
PLASTICITY CHART

Date 02/22/21 | Project No. 21-1973 | Figure B-1



SAMPLER TYPE Modified California		SHEAR STRENGTH	4,450 psf
DIAMETER (in.) 2.39	HEIGHT (in.) 5.77	STRAIN AT FAILURE	9.2 %
MOISTURE CONTENT	19.4 %	CONFINING PRESSURE	550 psf
DRY DENSITY	108 pcf	STRAIN RATE	1 % / min.
DESCRIPTION CLAY with SAND (CL), red-yellow			SOURCE B-1 at 5.5 feet

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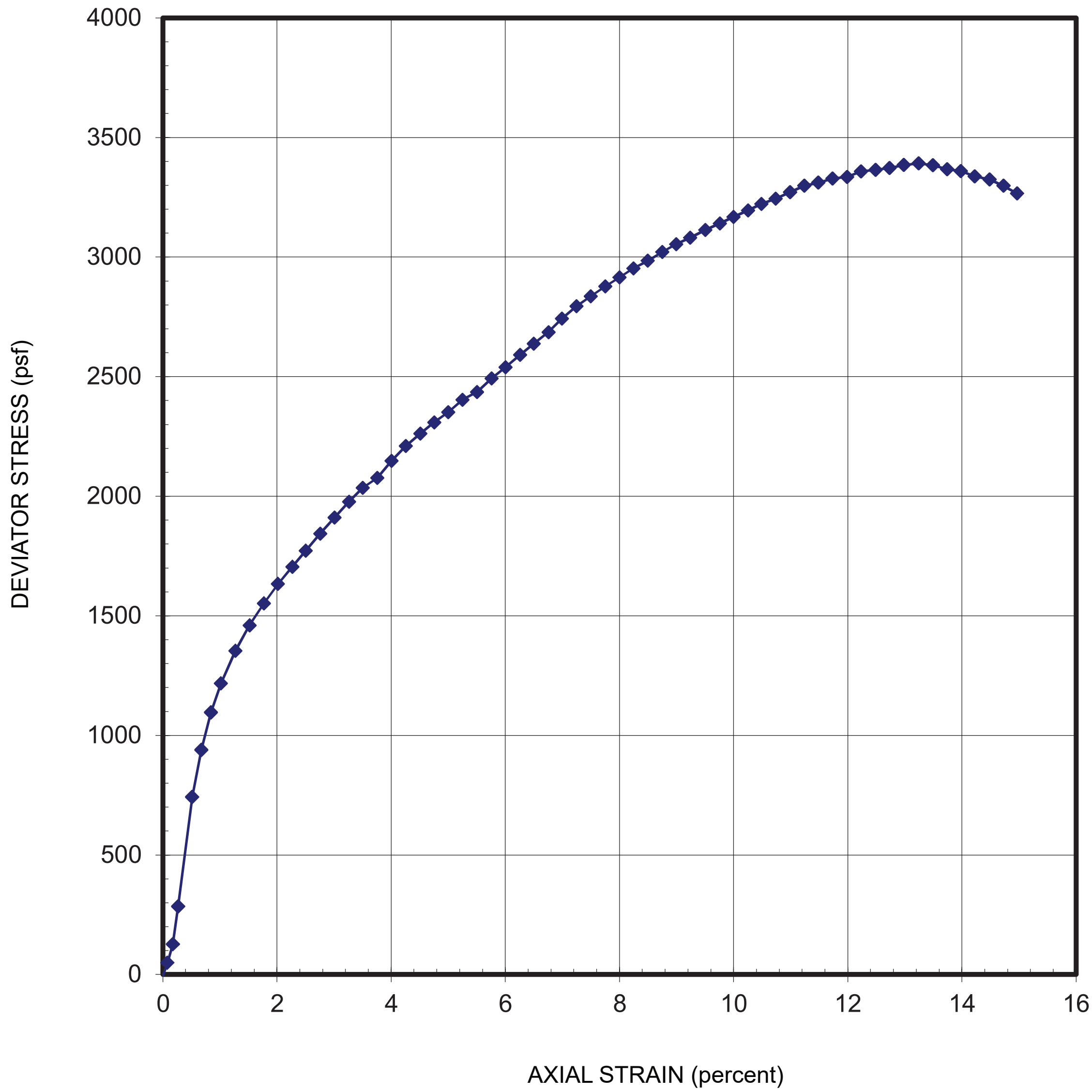
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 02/22/21 | Project No. 21-1973 | Figure B-2



SAMPLER TYPE Modified California		SHEAR STRENGTH	1,700 psf
DIAMETER (in.) 2.41	HEIGHT (in.) 5.69	STRAIN AT FAILURE	13.2 %
MOISTURE CONTENT 27.0 %		CONFINING PRESSURE	1,100 psf
DRY DENSITY 99 pcf		STRAIN RATE	1 % / min.
DESCRIPTION SANDY CLAY (CL), brown		SOURCE B-1 at 11.0 feet	

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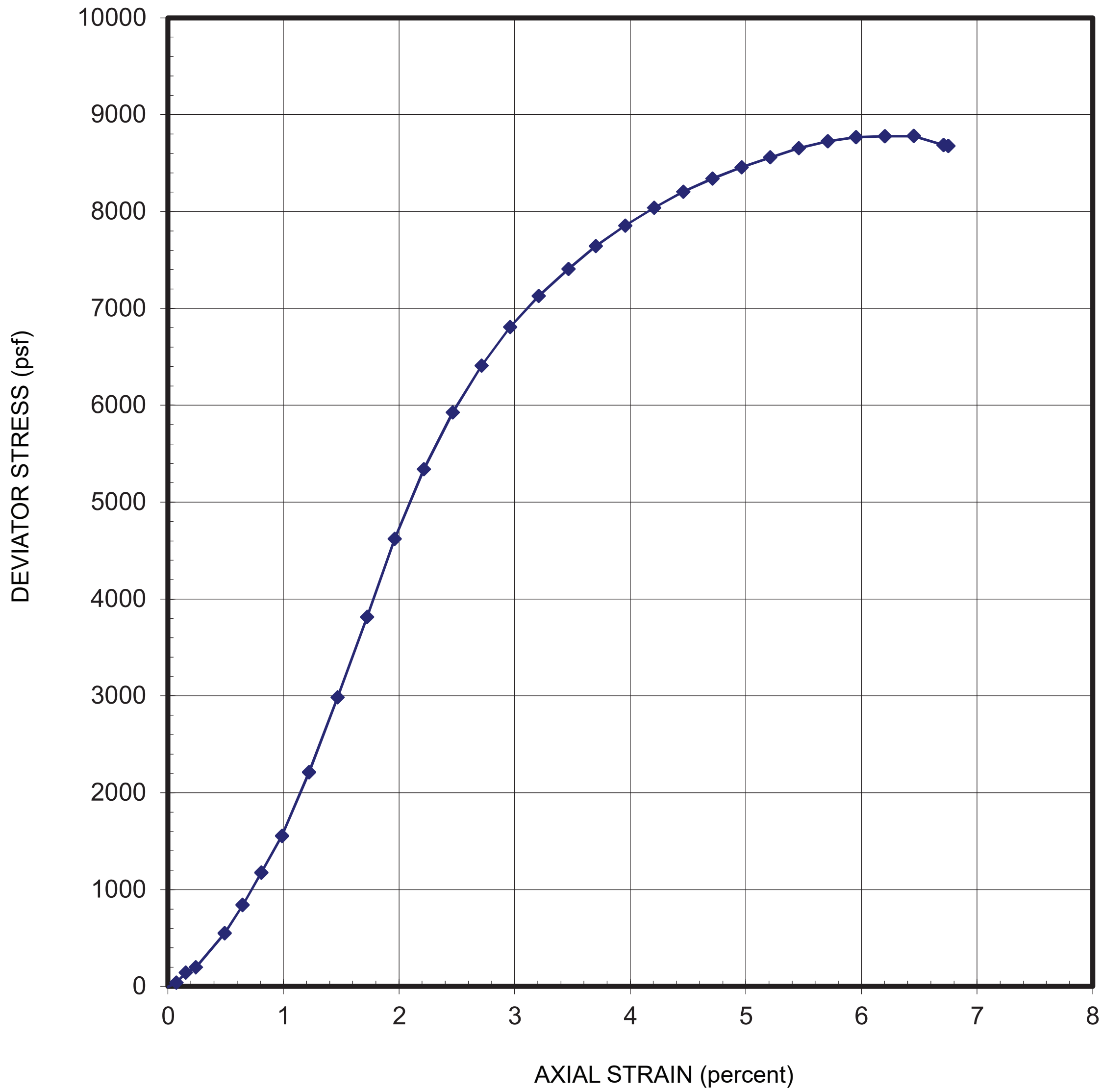
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 02/22/21	Project No. 21-1973	Figure B-3



SAMPLER TYPE Modified California	SHEAR STRENGTH 4,350 psf
DIAMETER (in.) 2.41 / HEIGHT (in.) 5.55	STRAIN AT FAILURE 6.5 %
MOISTURE CONTENT 18.8 %	CONFINING PRESSURE 550 psf
DRY DENSITY 111 pcf	STRAIN RATE 1 % / min.
DESCRIPTION CLAY with SAND (CL), gray-yellow with yellow-brown mottling	SOURCE B-2 at 5.5 feet

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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 02/22/21 | Project No. 21-1973 | Figure B-4

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

REPORT 210215D

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	Depth	Sulfates SO_4^{2-}		Chlorides Cl^-		Resistivity		pH	Redox	Sulfide S^{2-}	Nitrate NO_3^-	Ammonium NH_4^+	Lithium Li^+	Sodium Na^+	Potassium K^+	Magnesium Mg^{2+}	Calcium Ca^{2+}	Fluoride F_2^-	Phosphate PO_4^{3-}
						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: CLAY with SAND (CL), gray-brown	3	105.5	0.0105	7.2	0.0007	2,211	1,943	6.4	149	<0.01	19.0	17.7	0.02	48.6	0.8	59.3	96.0	2.8	2.4
B-1: CLAY with SAND (CL), red-yellow	5	25.6	0.0026	13.2	0.0013	1,608	1,608	7.1	140	<0.01	2.4	13.8	0.01	46.5	2.5	116.4	185.3	4.0	8.9

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

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

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SOIL CORROSIVITY TEST RESULTS

Date 02/22/21 | Project No. 21-1973 | Figure B-5