ROCKRIDGE GEOTECHNICAL

Prepared for **Steel Wave**

GEOTECHNICAL INVESTIGATION
PROPOSED LIFE SCIENCE BUILDING
787 BANCROFT WAY
Berkeley, California

April 13, 2021
Project No. 21-1973

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Project No. 21-1973

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

Subject: Final Report
Proposed Life Science Building
787 Bancroft Way
Berkeley, California

Dear Ms. Metz:

We are pleased to present our geotechnical investigation report, dated April 13, 2021, for the proposed life science building to be constructed at 787 Bancroft Way in Berkeley, California. Our investigation was performed in accordance with our proposal dated January 21, 2021.

The project site is a rectangular-shaped lot with plan dimensions of approximately 250 by 389 feet. It is bordered by Fourth Street to the east, Bancroft Way to the south, a railroad right-of-way with several railroad tracks to the west, and industrial buildings to the north. The site is currently occupied by seven commercial/warehouse buildings and a small asphalt-paved parking and storage area and is accessed from a driveway on Bancroft Way. Other than the driveway and an approximately five-foot-high slope along the western portion of the southern property line, the site is relatively level with ground surface elevations ranging from approximately 24 to 25.4 feet (NAVD 1988 datum). The sidewalk grade along the southern side of the site slopes gently down from approximately Elevation 23 feet at the corner of Bancroft Way and Fourth Street to Elevation 20 feet at the southwestern corner of the site. There is a combination of irregular slopes and non-engineered retaining structures along both the southern and western edges of the property.

Plans for the proposed life science building are to demolish the existing buildings on site and to construct a three-story at-grade life science building with a finished floor elevation near existing site grades. The proposed building will be L-shaped with a width of approximately 320 feet and a length of approximately 130 to 220 feet. The life science building will include spaces for lab, support, and office for multiple tenants. The proposed development will also include a pavilion on the northwestern portion of the site, a surface parking lot west of the proposed building, and a courtyard north of the proposed structure.

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 highly to very highly expansive near-surface soil, 2) the presence of up to approximately six feet of undocumented fill on the site, and 3) providing adequate foundation support for the proposed structure. Foundation alternatives for sites underlain by highly to very highly expansive clay include deepened spread footings, stiffened shallow foundations such as conventionally reinforced concrete mats or post-tensioned (P-T) slabs-on-grade, and drilled, cast-in-place concrete piers. Based on our experience with similar structures and soil conditions, we conclude the most appropriate foundation type for the proposed structure consists of deepened spread footings.

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.

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Enclosure

ROCKRIDGE GEOTECHNICAL

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GEOTECHNICAL INVESTIGATION
PROPOSED LIFE SCIENCE BUILDING
787 BANCROFT WAY
Berkeley, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed life science building to be constructed at 787 Bancroft Way in Berkeley, California. The subject property is on the northwestern corner of the intersection of Bancroft Way and Fourth Street, as shown on the Site Location Map, Figure 1.

The project site is a rectangular-shaped lot with plan dimensions of approximately 250 by 389 feet. It is bordered by Fourth Street to the east, Bancroft Way to the south, a railroad right-of-way with several railroad tracks to the west, and industrial buildings to the north. The site is currently occupied by seven commercial/warehouse buildings and a small asphalt-paved parking and storage area and is accessed from a driveway on Bancroft Way. Other than the driveway and an approximately five-foot-high slope along the western portion of the southern property line, the site is relatively level with ground surface elevations ranging from approximately 24 to 25.4 feet (NAVD 1988 datum). The sidewalk grade along the southern side of the site slopes gently down from approximately Elevation 23 feet at the corner of Bancroft Way and Fourth Street to Elevation 20 feet at the southwestern corner of the site. There is a combination of irregular slopes and non-engineered retaining structures along both the southern and western edges of the property.

Plans for the proposed life science building are to demolish the existing buildings on site and to construct a three-story at-grade life science building with a finished floor elevation near existing site grades. The proposed building will be L-shaped with a width of approximately 320 feet and a length of approximately 130 to 220 feet. The life science building will include spaces for lab, support, and office for multiple tenants. The proposed development will also include a pavilion on the northwestern portion of the site, a surface parking lot west of the proposed building, and a courtyard north of the proposed 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, performing two cone penetration tests (CPTs), 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 life science building
- 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 retaining walls
- subgrade preparation for the concrete slab-on-grade floor for the building 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 and performing two CPTs at the approximate locations 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 the presence of existing underground utilities at each boring and CPT location. We also obtained a drilling permit and a construction parking permit from the City of Berkeley Toxics Management Division (CBTMD). Details of the field exploration are described in the sections below.

3.1 Cone Penetration Tests

Our subsurface investigation included performing two CPTs, designated as CPT-1 and CPT-2, on February 3, 2021. The CPTs were each advanced to a depth of approximately 50.7 feet below the existing ground surface (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-1 and A-2.

Pore pressure dissipation tests were performed in each CPT. The interpreted water levels associated with the dissipation tests are presented on the respective CPT logs in Appendix A.

3.2 Test Borings

The test borings, designated as B-1 and B-2, were drilled on February 3 and 5, 2021 by Exploration Geoservices Inc. of San Jose, California. The borings were drilled to depths of about 46 to 46-1/2 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-3a through A-4b in Appendix A. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-5.

Soil samples were obtained from the borings 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 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 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 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 borings were backfilled with neat cement grout in accordance with CBTMD requirements. The soil cuttings generated by the borings 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 and a previous test boring by Treadwell & Rollo, Inc. indicate the site is blanketed by about 3 to 4-1/2 feet of medium stiff to very stiff clay with variable sand content. Atterberg limits tests performed on the clay indicates it is highly to very highly expansive[1] with plasticity indices ranging from 34 to 43; however, at its current moisture content, which is well above the plastic limit, its expansion potential is relatively low. Based on the ground surface elevations on the site relative to the sidewalk elevation on Bancroft Way, we believe the fill thickness on the site increases to the west and most of the surficial clay layer is fill. Previous environmental borings drilled in the northwestern portion of the site encountered up to about 4 to 6 feet of fill consisting of medium stiff to stiff clay with variable sand and gravel content and loose to medium dense sand and gravel with variable silt and clay content.

Because of the past development of the site, we anticipate thicker fill may be present locally. A report found on the State of California's Water Resources Control Board's website: www.geotracker.com, titled *Groundwater Sampling Report*, *Former Peerless Lighting, Case File Nos. 01-1147 & 01S0490 (CCM) GeoTracker ID No. T0600101057, Report 2220 Fourth Street, Berkeley, California,* prepared by Fugro (December 21, 2020) indicates there was previously an excavation in the northwestern portion of the site to remove soil impacted with pesticides, as shown on the Site Plan, Figure 2.

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 50.7 feet bgs.

Groundwater was measured in Borings B-1 and B-2 at depths of 32.5 and 14.5 feet bgs, respectively, prior to grouting. The groundwater level was also estimated at approximately 11.5 and 10.7 feet bgs using pore pressure dissipation test data from CPT-1 and CPT-2, respectively. 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. Review of the GeoTracker report (Fugro 2020) indicates monitoring wells or piezometers were installed onsite between 1985 and 1994, the locations of which are shown on the Site Plan, Figure 2. Five groundwater readings in monitoring wells, installed by Brown and Caldwell, between June 1985 and March 1986 indicate the groundwater fluctuated between 7.16 and 12.38 feet bgs. Mark Group installed several wells onsite in 1988. Readings taken in the wells between April 25, 1988 and October 31, 1994 indicate the depth to groundwater varied from 9.43 to 19.50 feet bgs. Fugro measured groundwater in the same wells at depths of 9.52 to 12.46 feet bgs on October 14, 2020. Measurements in temporary monitoring wells/ piezometers, installed by Subsurface Consultants in 1994, indicate the groundwater fluctuated from 10.96 to 16.34 feet bgs in four readings taken between October 1994 and October 2020.

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. For design, recommend using a design groundwater elevation of 18 feet.

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

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)	4.2	East	7.58
Hayward (North, HN)	4.2	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)	26	West	8.04
North San Andreas (Peninsula, SAP)	26	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,[3] lateral spreading[4] and cyclic densification.[5] We used the results of our borings and CPTs to evaluate the potential of these

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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, 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 0.87 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 0.87g 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 highly to very highly expansive near-surface soil, 2) the presence of up to approximately six feet of undocumented fill on the site, and 3) providing adequate foundation support for the proposed structure. These and other issues are discussed in this section.

6.1 Foundation Support and Settlement

The 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 resist the effects of the expansive clay. These effects can be mitigated by moisture-conditioning the expansive soil, providing select, non-expansive fill below interior and exterior slabs, and either supporting foundations below the zone of severe moisture change or providing a stiff, shallow foundation that can limit deformation of the superstructure as the underlying soil shrinks and swells.

Foundation alternatives for sites underlain by highly to very highly expansive clay include deepened spread footings, stiffened shallow foundations such as conventionally reinforced concrete mats or post-tensioned (P-T) slabs-on-grade, and drilled, cast-in-place concrete piers. Based on our experience with similar structures and soil conditions, we conclude the most appropriate foundation type for the proposed structure consists of deepened spread footings. We can provide recommendations for other foundation types upon request.

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. Most of this settlement will occur during construction. As discussed above in Section 5.2.3, we estimate additional total building settlement of up to 1/2 inch and differential settlement of up to 1/4 inch over a horizontal distance of 30 feet may occur following a major earthquake as excess pore pressures dissipate in soil layers in which cyclic softening occurs.

6.2 Construction Considerations

The soil to be excavated consists of predominantly 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.

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

6.3 Soil Corrosivity

A corrosivity test was performed by Project X Corrosion Engineering of Murrieta, California on a selected soil sample obtained 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. The resistivity test results (670 ohm-cm) indicate the surficial clayey soil is "extremely corrosive[6]" 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.

The results indicated the sulfate ion concentration (29.3 mg/kg) is insufficient to damage reinforced concrete structures below ground and the chloride concentrations(17.4 mg/kg) of the soil does not present a problem with reinforcing steel in the buried concrete structures. The pH test results (8.5) indicate the soil is "negligibly corrosive" to buried steel or concrete; however, alkaline soil with a pH greater than 8.5 can cause accelerated corrosion of copper and aluminum alloys. Based on the results of the sulfate, chloride and pH tests, we conclude no special measures are required to protect below-grade reinforced concrete.

7.0 RECOMMENDATIONS

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

7.1 Site Preparation and Grading

Site demolition should include removal of all existing buildings and their foundations, pavements, and underground utilities. Demolished asphalt concrete should be taken to an asphalt recycling facility. 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 or will not interfere with the proposed construction, they may be abandoned in place provided the lines are filled with controlled low-strength material (CLSM) or cement grout to the property line. Voids resulting from demolition activities should be properly backfilled with compacted fill or CLSM following the recommendations provided later in this section. A field engineer from our office should be on

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

site during backfilling of voids resulting from demolition to confirm the backfill is placed and compacted in accordance with the recommendations below.

Following site demolition, the existing fill below the proposed building should be overexcavated and recompacted. The thickness of fill to be excavated should be further evaluated with test pits following demolition. The bottom of the overexcavation should extend at least five feet outside the proposed building footprint except where constrained by property lines or existing buildings. Excavations adjacent to neighboring buildings that extend below a 1.5:1 (horizontal to vertical) line projected downward from the bottom of adjacent foundations should be performed in slots not exceeding 10 feet in length. In general, the fill should be excavated so that no more than one foot of fill remains below the excavation subgrade. We estimate the excavation depth will range from about 2 to 5 feet below the existing ground surface with an average excavation depth of about three feet across the building footprint. The excavation subgrade should be scarified to a depth of 12 inches and moisture-conditioned and compacted based on the expansive soil recommendations in Table 2. If field density tests indicate the in-situ moisture content of the subgrade soil meets the requirements in Table 2, scarification and moisture-conditioning prior to compaction are not required. If soil that is too wet to properly compact is encountered at the excavation subgrade and aeration is not feasible due to time and/or weather constraints, lime treatment of the exposed soil may be needed to provide a stable subgrade on which to place the excavated 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 eight 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 takes the most time to complete. Other soil stabilization alternatives include overexcavating the wet soil and replacing or mixing it with drier soil, and lime treatment.

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

7.1.1 Subgrade Preparation

After site clearing is completed, in areas that will receive improvements (i.e. building pad and exterior concrete flatwork) or fill (including select fill), the soil subgrade exposed should be scarified to a depth of at least eight inches, moisture-conditioned and compacted in accordance with the requirements provided in Table 2 (Section 7.1.2). Subgrade preparation for the proposed building pad and exterior concrete flatwork are presented in this section.

Building Pad Subgrade

The near-surface clay at the site is highly to very highly expansive. To mitigate the detrimental effects of expansive near-surface soil, the building slab-on-grade floor should be underlain by at least 12 inches of non-expansive soil consisting of select fill or lime-treated on-site soil. The non-expansive soil should extend at least five feet beyond the perimeter of the proposed building, except where constrained by the property line or adjacent buildings. The soil exposed at the base of the excavation for placement of select fill should be scarified to a depth of at least eight inches, moisture-conditioned and compacted in accordance with the requirements presented in Table 2.

Exterior Concrete Flatwork

We recommend a minimum of eight inches of select fill be placed beneath proposed exterior concrete flatwork, including patio slabs and sidewalks; the select fill should extend at least six inches beyond the slab edges, except where constrained by property lines. Select fill beneath exterior slabs-on-grade, such as patios and sidewalks, should be moisture-conditioned and compacted in accordance with the requirements presented in Table 2. Lime treatment of the upper 12 inches of the native clay may be used in lieu of placement of select fill; however, we recommend at least four inches of Class 2 aggregate base be placed beneath flatwork where lime treatment is performed.

Even with eight inches of select fill or lime treatment, 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 the building, 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.

7.1.2 Fill Materials and Compaction Criteria

In areas to receive fill where native soil is exposed at the base of excavation, the soil subgrade should be scarified to a depth of at least eight inches, moisture-conditioned, and compacted to the specified percent relative compaction,[7] as presented below in Table 2. Note that "moisture-conditioning" may require wetting or drying of the soil, depending on the conditions encountered. All fill should be placed in horizontal lifts not exceeding eight inches in loose thickness, moisture-conditioned, and compacted in accordance with the requirements provided below in Table 2. Each type of material is described in the following text according to its uses and specifications.

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[7] 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

Notes:
1. Select fill and lime-treated onsite soil are considered low-plasticity soil.
2. Granular soil and clean sand or gravel are defined as soil with less than five percent fines.

Where the above recommended compaction requirements are in conflict with the City of Berkeley standard details for pavements and sidewalks within the public right-of-way, the City Engineer or inspector should determine which compaction requirements should take precedence.

On-site Soil

On-site soil may be used as general fill, provided it is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, and be approved by the Geotechnical Engineer. On-site soil can be compacted to the relative compaction presented in Table 2. Because of the shallow groundwater table, we anticipate it will be necessary to aerate the on-site soil to reduce its moisture content before the specified compaction can be achieved.

Select Fill

Select fill should consist of on-site or 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 15, and be approved by the Geotechnical Engineer. Samples of proposed select fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site.

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

Aggregate Base Material

Imported aggregate base (AB) may be used as select fill or trench backfill (above bedding materials). AB placed beneath sidewalks and vehicular pavements within public right-of-way should meet the requirements in the 2018 Caltrans Standard Specifications, Section 26, for Class 2 Aggregate Base (3/4 inch maximum).

Controlled Low-Strength Material

Controlled low-strength material (CLSM) may be considered as an alternative to fill beneath the building, concrete flatwork, or pavement. CLSM should meet the requirements in the 2018 Caltrans Standard Specifications. It is an ideal backfill material when adequate room is limited or not available for conventional compaction equipment, or when settlement of the backfill must be minimized. No compaction is required to place CLSM. CLSM should have a minimum 28-day unconfined strength of 50 pounds per square inch (psi).

7.1.3 Lime Treatment

Lime treatment of fine-grained soils generally includes site preparation, application of lime, mixing, compaction, and curing of the lime treated soil. Field quality control measures should include checking the depth of lime treatment, degree of pulverization, lime spread rate measurement, lime content measurement, and moisture content and density measurements, and mixing efficiency. Quality control may also include laboratory tests for unconfined compressive strength tests on representative samples.

If the non-expansive soil to be placed beneath the building pad and exterior concrete flatwork will consist of lime-treated on-site soil, the soil subgrade should be treated in place with Dolomitic Quicklime. The lime treatment process should be designed by a contractor specializing in its use and who is experienced in the application of lime in similar soil conditions. Based on our experience with lime treatment, we judge that the specialty contractor should be able to treat the highly to very highly expansive on-site material to produce a non-expansive fill for the proposed building. For planning purposes, we recommend assuming the lime treatment will consist of five percent of Dolomitic Quicklime by dry weight of soil. The dry weight of soil should be assumed to be 105 pcf for calculating lime quantities. The specialty contractor should: 1) perform a lime demand test prior to treatment to determine the percentage of Quicklime required to achieve a pH of 12.4 or higher in the treated soil, 2) perform an Atterberg limits test to confirm the proposed percentage of Quicklime will reduce the plasticity index of the treated soil to 15 or less, and 3) prepare a lime treatment procedure for our review prior to construction.

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 as the high pH will prevent plant growth.

7.1.4 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of 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 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.

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. 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.

7.2 Surface Drainage and Landscaping

In the following sections, we present our recommendations for surface drainage, landscaping and bioswales.

7.2.1 Surface Drainage

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 into 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.

7.2.2 Landscaping

Prior experience and industry literature indicate that some species of high water-demand[8] 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 buildings: *Eucalyptus, Populus, Quercus, Crataegus, Salix, Sorbus* (simple-leafed), *Ulmus, Cupressus,*

[8] "Water-demand" refers to the ability of the tree to withdraw large amounts of water from the soil.

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.

7.2.3 Bioswales

Where bioswales will be part of the project, we recommend the bioswales be constructed at least five feet from the building and provided with underdrains and/or drain inlets. The subdrain pipes should be installed eight inches above the bottom of the infiltration area for treatment areas that are at least five feet away from the new building and pavements. The intent of this recommendation is to allow infiltration into the underlying soil, but to reduce the potential for bio-retention areas to flood during periods of heavy rainfall.

Where it is necessary for a bioswale to be constructed within five feet of the building and pavements because of site constraints, the bottom of the bioswale should be lined with an impermeable liner. Where a vertical curb or foundation is constructed near a bioswale, the curb and the edge of the foundation should be founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal: vertical) from the base of the bioswale.

7.2 Foundation Design

As discussed above, spread footings bottomed on very stiff/dense native soil and/or engineered fill may be used to support the proposed life science building. The recommendations for spread footings may also be used to design the mat foundation for the elevator pits. 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 36 inches below the lowest adjacent finished grade (for exterior footings), whichever is lower.

Spread footings for the building 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 40 pounds per cubic inch (pci) be used for design of footings and the elevator pit mat subgrade. This modulus has been scaled to take into account the footing/mat dimensions.

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 Capillary Moisture Break and Water Vapor Retarder

The subgrade for the building concrete slab-on-grade floor should be prepared in accordance with our recommendations in Section 8.1.1. The floor slab should be underlain by at least 12 inches of non-expansive soil consisting of select fill or lime-treated on-site soil (measured below the capillary moisture break discussed below). Where water vapor transmission through the slab-on-grade floor is undesirable, we recommend installing a capillary moisture break and a water vapor retarder beneath the floor. 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 3.

TABLE 3
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 A vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

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

7.4 Permanent Retaining Walls and Below-Grade Walls

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 and building stem 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, such as elevator pit walls, at the site be designed for the more critical of:

- at-rest pressure using an equivalent fluid weight of 60 pcf (triangular distribution); or
- active pressure using an equivalent fluid weight of 40 pcf (triangular distribution) plus a seismic increment of 37 pcf (triangular distribution)

We recommend unrestrained retaining walls at the site be designed for the more critical of:

- active pressure using an equivalent fluid weight of 40 pcf (triangular distribution); or

- active pressure using an equivalent fluid weight of 40 pcf (triangular distribution) plus a seismic increment of 15 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.

Where there will be vehicular traffic behind the top of a permanent wall within a horizontal distance equal to 1.5 times the height of the wall, the wall should be designed for vehicular surcharge of 50 psf (uniform) acting over the entire wall height. 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.

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 collector pipes.

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).

Site retaining walls should be supported spread footings bearing on engineered fill and/or stiff native soil. The walls should be bottomed at least 24 inches below the lowest adjacent grade where the grade adjacent to the bottom of the wall is covered with pavement or concrete and at least 36 inches below the lowest adjacent grade where the grade consists of exposed soil. Retaining wall footings may be designed using allowable bearing pressures of 3,000 psf for dead-plus-live loads and 4,000 psf for total loads. The passive pressure and friction design values provided above for building footings may be used design of retaining walls.

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 B soil according to the CAL-OSHA classification system. The maximum allowable slope for Type B 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 40 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 60 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

The latitude and longitude of the site are 37.8634° and -122.2991°, respectively. For design in accordance with 2019 CBC, we recommend the following:

- Site Class D – stiff soil
- S_S = 1.881, S_1 = 0.718g

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} = 1.881g, S_{M1} = 1.221g
- S_{DS} = 1.254g, S_{D1} = 0.814g
- 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 borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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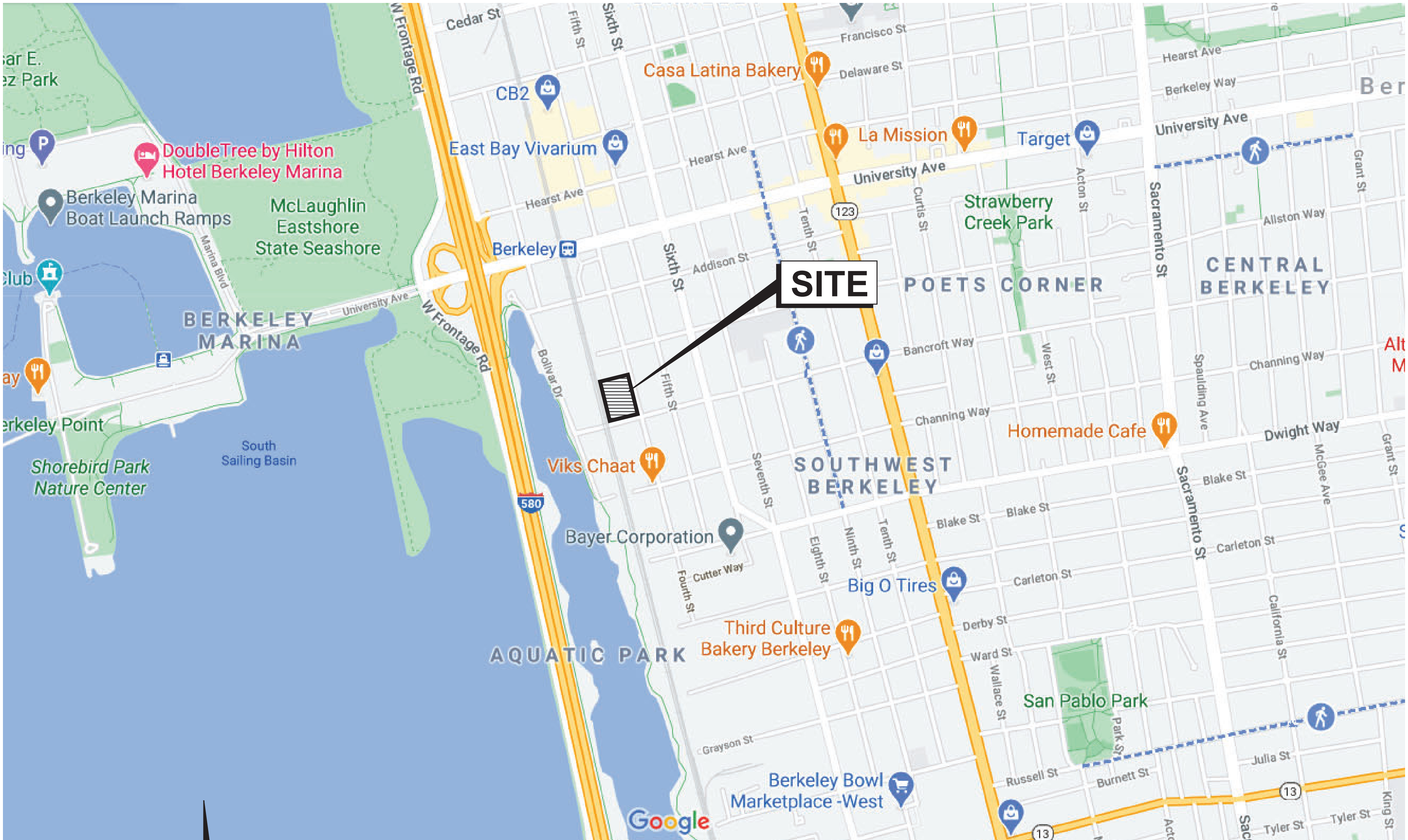
ROCKRIDGE GEOTECHNICAL

FIGURES



Base map: Google Maps, 2021.

0 1,300 2,600 Feet

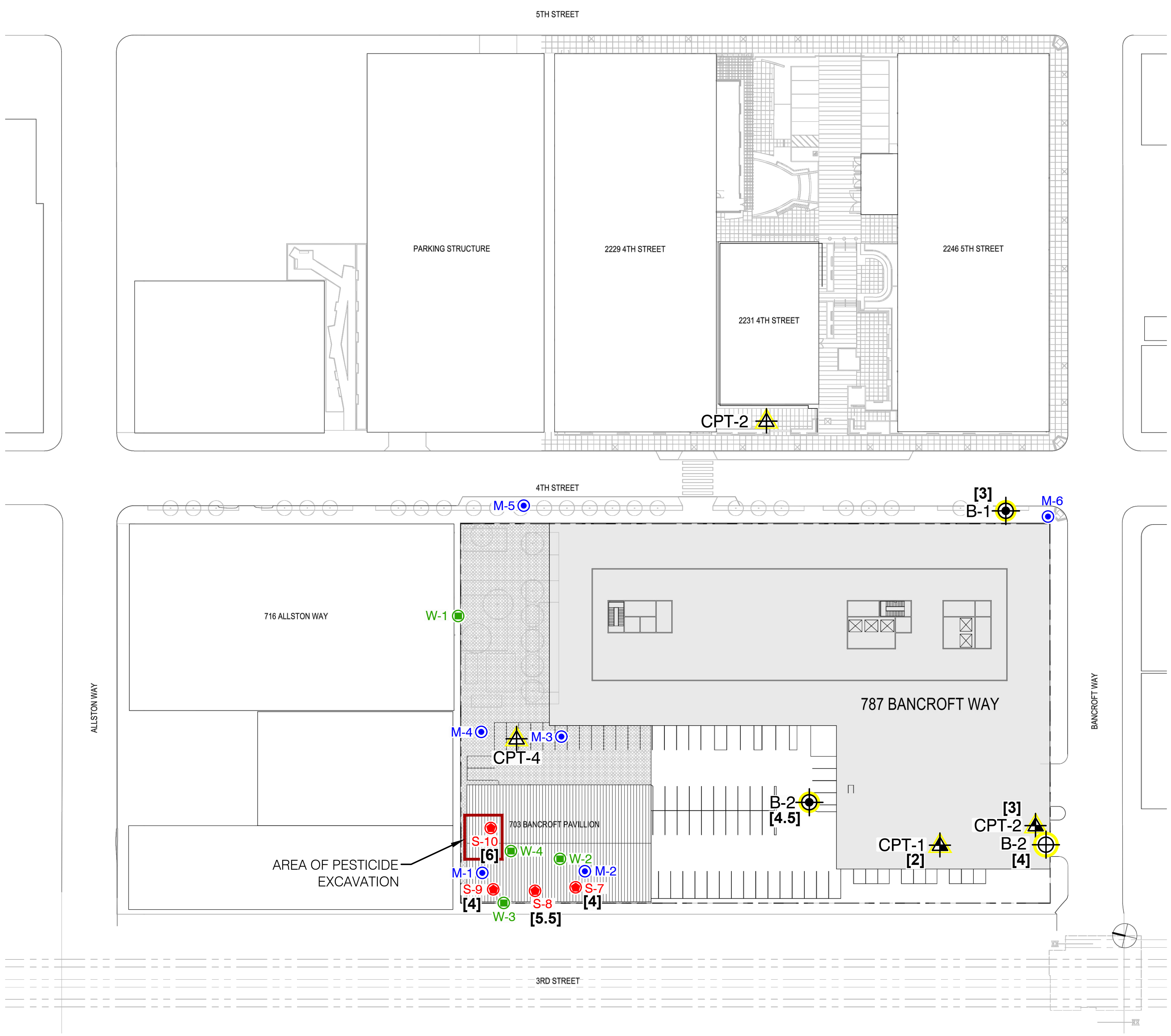
Approximate scale

LIFE SCIENCE BUILDING
787 BANCROFT WAY
Berkeley, California

ROCKRIDGE
GEOTECHNICAL

SITE LOCATION MAP

Date 03/02/21	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
CPT-2	Approximate location of cone penetration test by Treadwell & Rollo, Inc., March 2008
B-2	Approximate location of boring by Treadwell & Rollo, Inc., March 2008
[4]	Estimated thickness of existing fill in feet
W-1	Well/Peizometer groundwater sampling location by Brown and Caldwell, June 1985
M-1	Well/Peizometer groundwater sampling location by Mark Group, April 1988
S-7	Temporary well/peizometer groundwater sampling location by Subsurface Consultants, October 1994

0 80 Feet

Approximate scale

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SITE PLAN

Date 03/09/21	Project No. 21-1973	Figure 2

ROCKRIDGE GEOTECHNICAL

Reference: Base map from a drawing titled "Proposed Site Plan", by Skidmore, Owings & Merrill LLP, dated February 25, 2021.

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

N

0 1000 2000 Feet

Approximate scale

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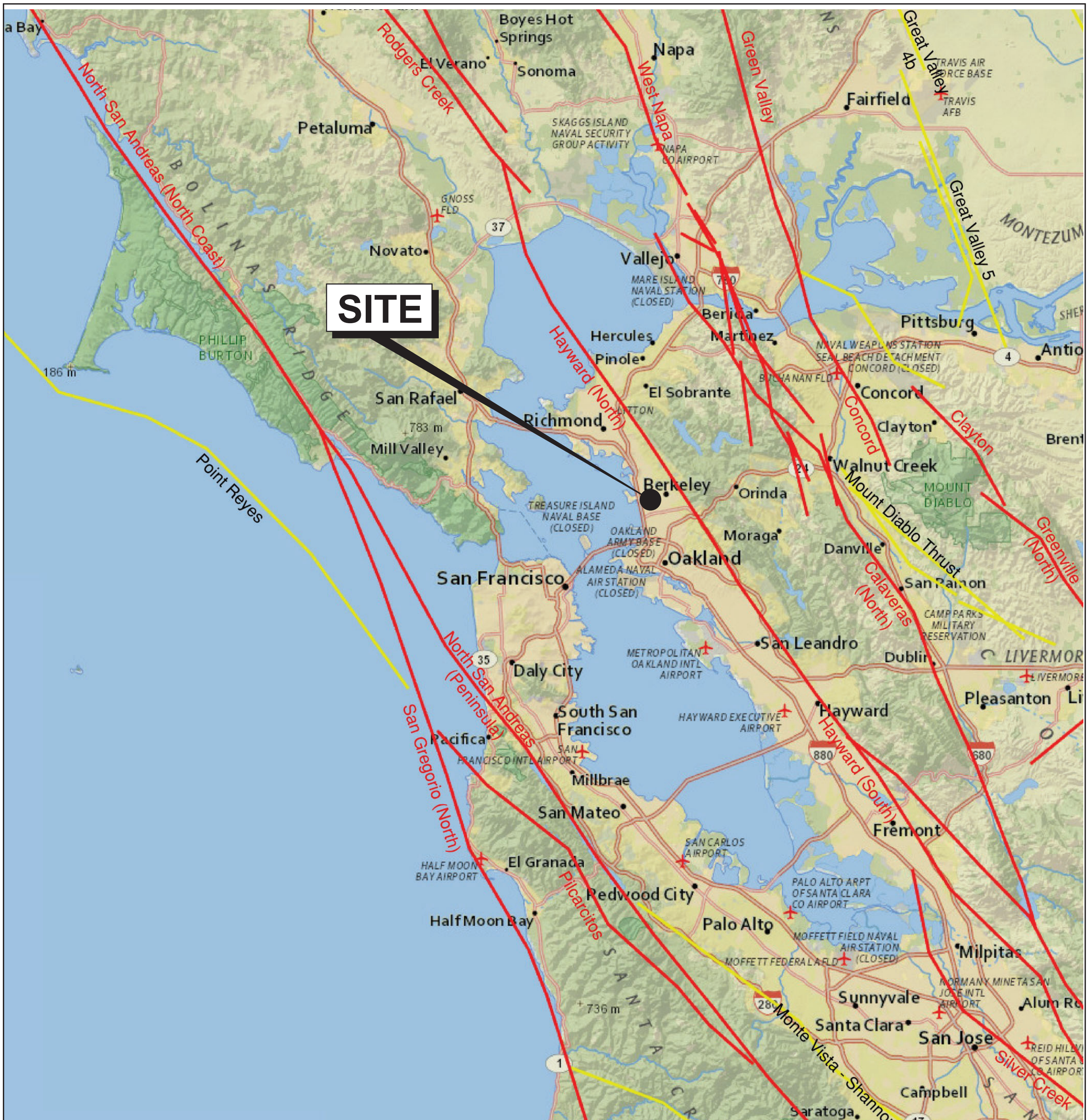
ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 03/02/21 | 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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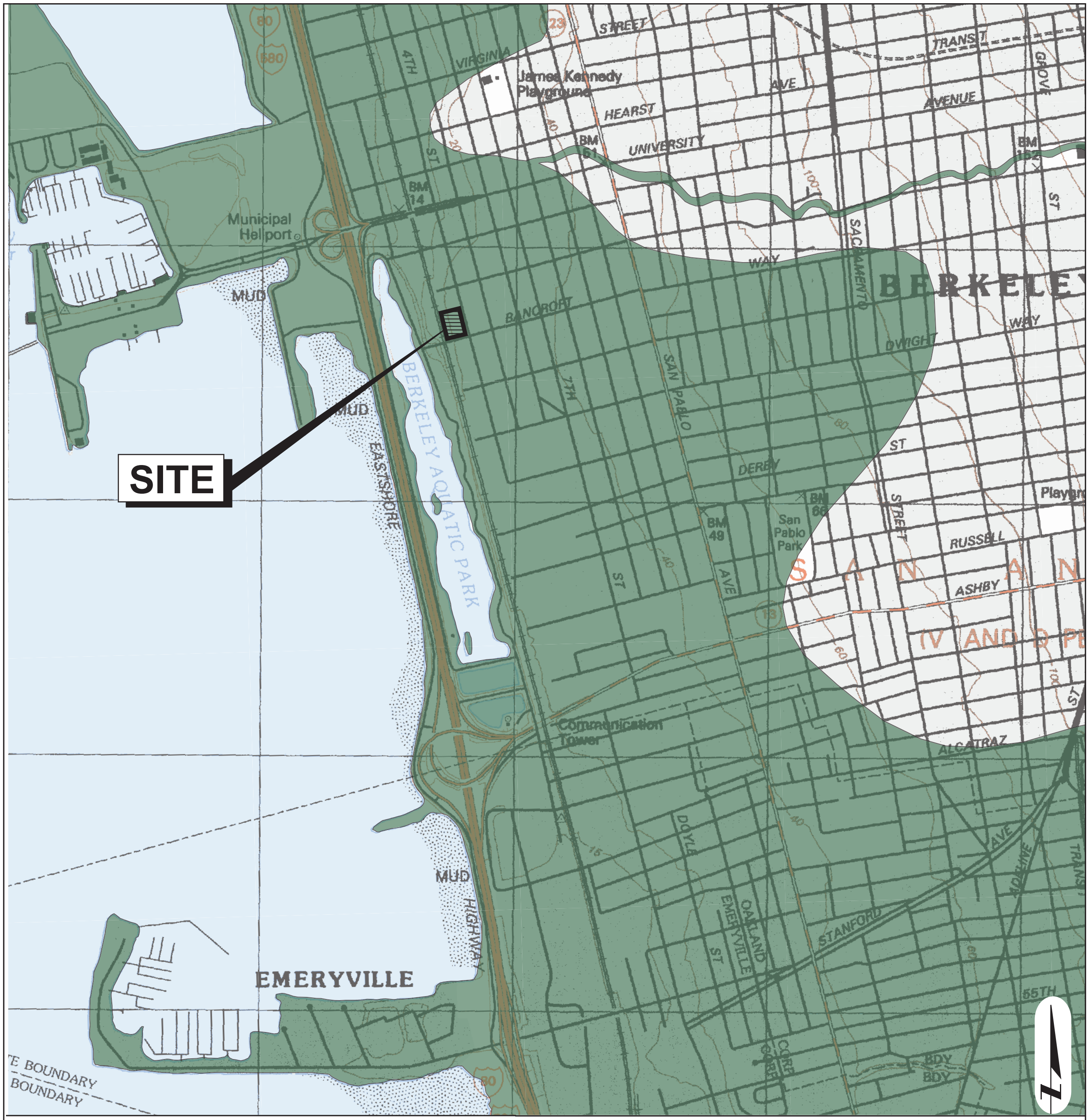
ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 03/01/21 | 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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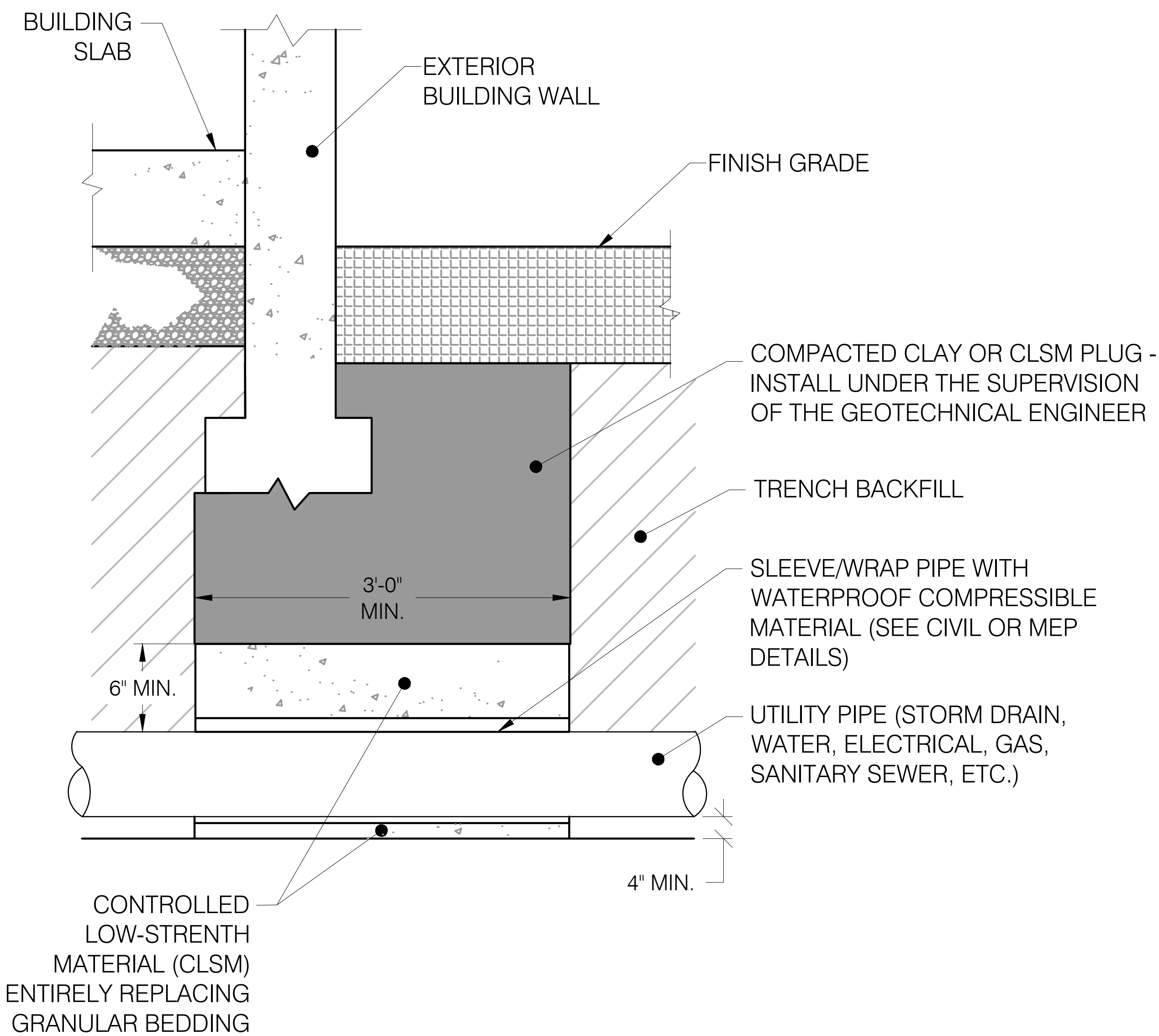
Berkeley, California

ROCKRIDGE GEOTECHNICAL

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 03/02/21	Project No. 21-1973	Figure 5



Not to Scale

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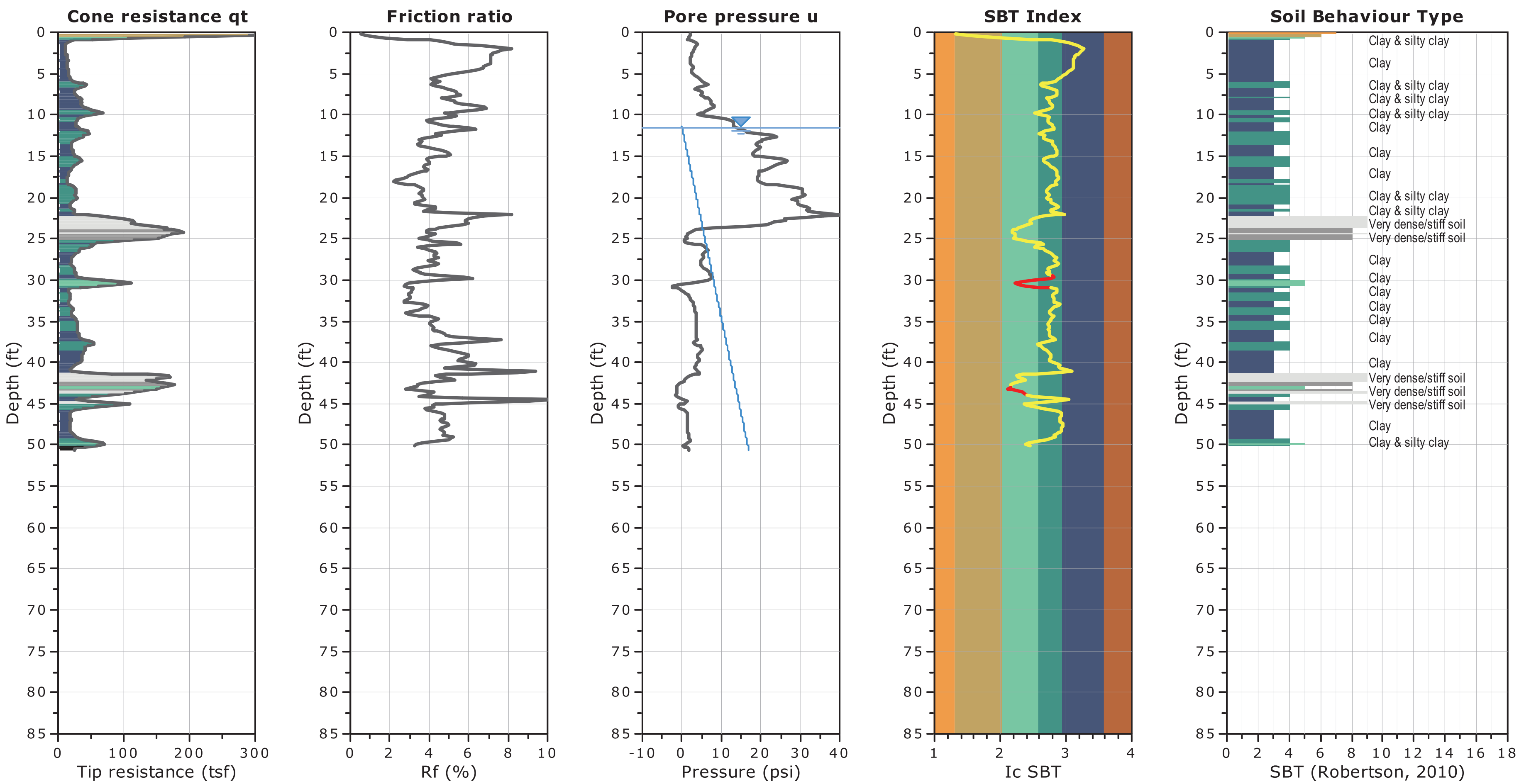
ROCKRIDGE GEOTECHNICAL

UTILITY TRENCH LOW-PERMEABILITY PLUG AT BUILDING PERIMETER

Date 03/01/21	Project No. 21-1973	Figure 6

APPENDIX A

Cone Penetration Test Results and Boring Logs



Total depth: 50.69 ft, Date: February 3, 2021
Estimated Groundwater Depth: 11.5 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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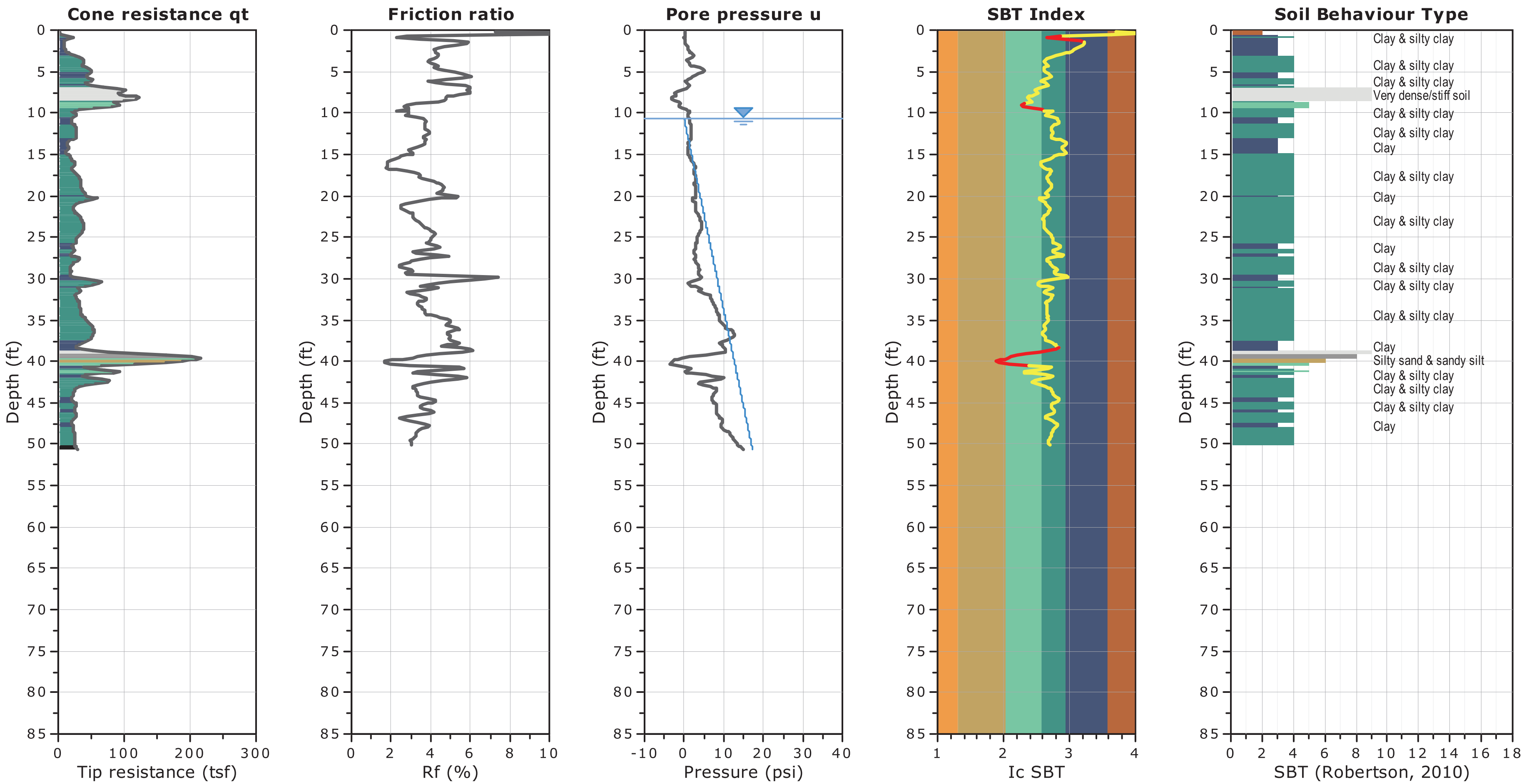
ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 03/01/21	Project No. 21-1973	Figure A-1



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: 50.69 ft, Date: February 3, 2021
Estimated Groundwater Depth: 10.7 feet (pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

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ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 03/01/21 | Project No. 21-1973 | Figure A-2

PROJECT: **LIFE SCIENCE BUILDING**
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Log of Boring B-1

Boring location: See Site Plan, Figure 2

Date started: 02/03/2021 | Date finished: 02/03/2021

Drilling method: Hollow-Stem Auger

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

Sampler: Modified California (MC), Dames & Moore (D&M)

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

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–3	MC (at 2)	3, 4, 8	8	CH	CLAY with SAND (CH) dark brown, medium stiff to stiff, moist LL = 54, PI = 34; see Appendix B (FILL?)					28.4	96
3–5	MC (at 4)	6, 9, 15	17	CL	CLAY with SAND (CL) gray, very stiff, moist, decreasing sand content						
5–7	MC (at 6)	7, 13, 25	27	CL	SANDY CLAY (CL) olive-gray to gray-brown, very stiff, moist						
7–10	MC (at 8)	12, 14, 18	22	SC	CLAYEY SAND (SC) gray, medium dense, moist, trace fine subrounded gravel						
10–15	MC (at 11)	8, 19, 26	32	CL	SANDY CLAY (CL) olive-gray, hard, moist, trace gravel						
15–30	MC (at 16)	6, 11, 14	18	CL	CLAY with SAND (CL) brown, very sitff, wet, trace coarse sand						
	MC (at 21)	11, 17, 25	29								
	MC (at 26)	12, 10, 21	22								

ROCKRIDGE GEOTECHNICAL

Project No.: 21-1973 | Figure: A-3a

PROJECT: **LIFE SCIENCE BUILDING**
787 BANCROFT WAY
Berkeley, California

Log of Boring B-1

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31	D&M		30-50 psi		CL	CLAY with SAND (CL) (continued) decreasing sand content						
32						∇ (02/03/2021; 2:50 PM)						
33												
34												
35						▼ (02/03/2021; 2:40 PM) brown mottled with gray, wet						
36	MC		14 18 23	29								
37												
38												
39												
40												
41	D&M		500 psi									
42												
43												
44												
45												
46	MC		10 12 17	20								
47–60												

Boring terminated at a depth of 46.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at depths of 35 and 32.5 feet during drilling.

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

ROCKRIDGE GEOTECHNICAL

Project No.: 21-1973

Figure: A-3b

PROJECT: **LIFE SCIENCE BUILDING**
787 BANCROFT WAY
Berkeley, California

Log of Boring B-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: Standard Penetration Test (SPT), Modified California (MC), Dames & Moore (D&M)

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

LABORATORY TEST DATA

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2 inches of asphalt; 2 inches of aggregate base						
1–2	MC	4, 6, 9	11	CH	CLAY (CH) dark gray to black, stiff, moist, trace medium sand (FILL?)						
3–4	MC	4, 9, 11	14	CH	Soil Corrosivity Test; see Appendix B. LL = 63, PI = 43; see Appendix B					28.2	97
5–6	MC	5, 9, 14	16	CL	CLAY with SAND (CL) gray, very stiff, moist						
7–8	MC	10, 17, 21	27	CL	brown, trace gravel. TxUU Test; see Appendix B	TxUU	725	4,550		19.7	109
10–11	MC	13, 15, 17	22	SC	CLAYEY SAND with GRAVEL (SC) gray mottled with yellow-brown, medium dense, moist						
14–15				SC	▼ (02/05/2021; 12:20 PM) ▽ (02/05/2021; 11:55 AM)						
15–16	SPT	6, 5, 8	13	SC							
17–18	MC	6, 8, 14	15	CL	CLAY with SAND (CL) yellow-brown, stiff to very stiff, wet						
20–21	MC	10, 13, 20	23	CL	SANDY CLAY (CL) brown, very stiff, wet, trace fine gravel						
25–26	MC	11, 16, 21	26	SC	CLAYEY SAND with GRAVEL (SC) gray-brown, medium dense, wet						
26				CL	SANDY CLAY (CL) yellow-brown, very stiff, wet						
29–30					CLAYEY SAND (SC)						

ROCKRIDGE GEOTECHNICAL
Project No.: 21-1973
Figure: A-4a

PROJECT: **LIFE SCIENCE BUILDING**
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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	MC		12 16 31	33	SC	CLAYEY SAND (SC) (continued) yellow-brown, dense, wet						
35–36	MC		12 18 23	29	CL	CLAY with SAND (CL) brown, very stiff, wet, trace gravel						
40–41	MC		18 26 48	52		hard, increasing sand content						
45–46	MC		22 50/6"	35/6"	SC	CLAYEY SAND (SC) brown, very dense, wet						

Boring terminated at a depth of 46 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at depths of 15 and 14.5 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

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

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

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
MC	Modified California sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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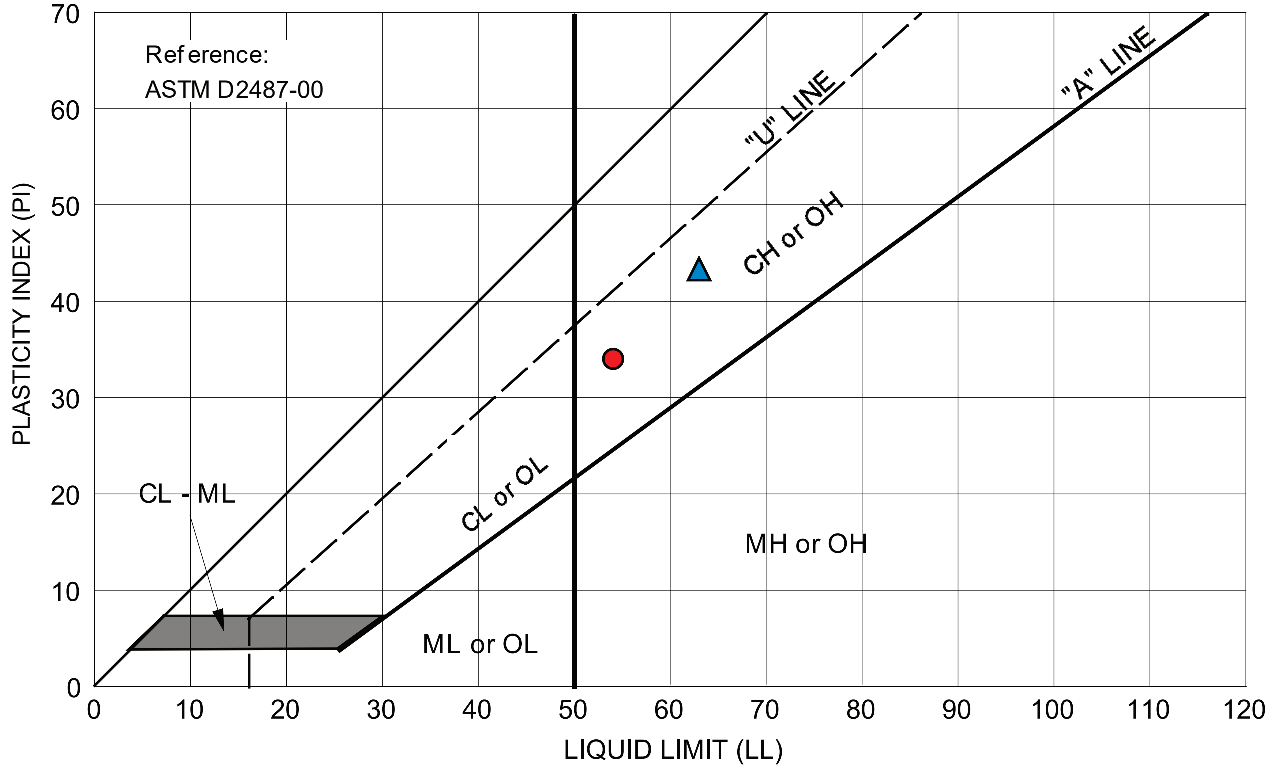
CLASSIFICATION CHART

Date 03/01/21 | Project No. 21-1973 | Figure A-5

ROCKRIDGE GEOTECHNICAL

APPENDIX B

Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 1.5 feet	CLAY with SAND (CH), dark brown	28.4	54	34	--
▲	B-2 at 3.5 feet	CLAY (CH), dark gray to black	28.2	63	43	--

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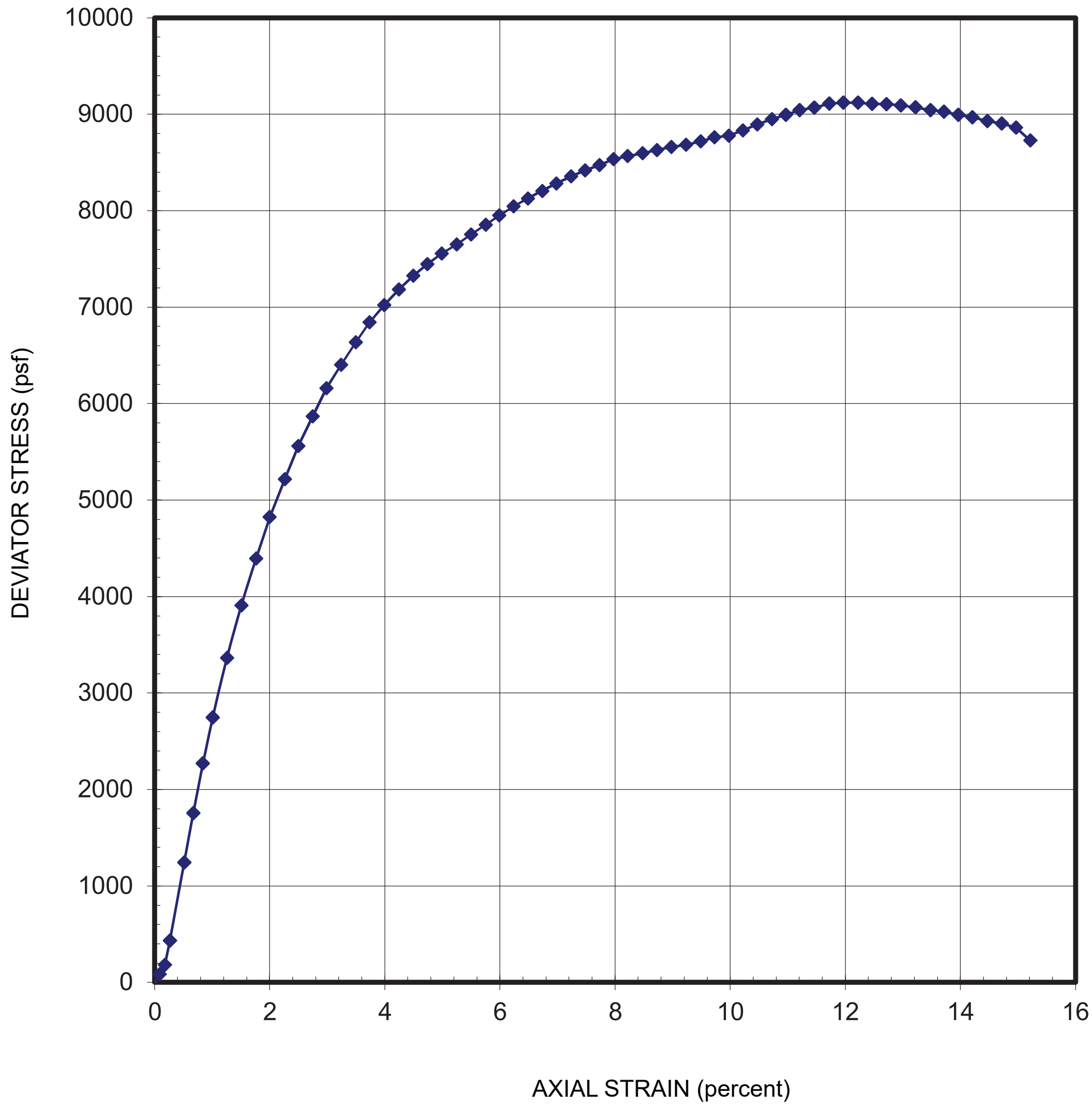
PLASTICITY CHART

Date 03/04/21	Project No. 21-1973	Figure B-1



SAMPLER TYPE	Modified California	SHEAR STRENGTH	4,550 psf
DIAMETER (in.) 2.37	HEIGHT (in.) 5.43	STRAIN AT FAILURE	12.0 %
MOISTURE CONTENT	19.7 %	CONFINING PRESSURE	725 psf
DRY DENSITY	109 pcf	STRAIN RATE	1 % / min.
DESCRIPTION	CLAY with SAND (CL), brown	SOURCE	B-2 at 7.3 feet

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UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 03/04/21 | Project No. 21-1973 | Figure B-2

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

REPORT S210226B

	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** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-2: CLAY (CH) dark gray to black	3	29.3	0.0029	17.4	0.0017	670	670	8.5	202	<0.01	1.3	25.6	0.02	67.7	0.2	52.0	47.2	3.1	2.7

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 03/03/21 | Project No. 21-1973 | Figure B-3