Attachment G: Geotechnical Investigation

THIS PAGE INTENTIONALLY LEFT BLANK

ROCKRIDGE
GEOTECHNICAL

Prepared for **Riaz Capital**

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
820 WEST MACARTHUR BOULEVARD
OAKLAND, CALIFORNIA

UNAUTHORIZED USE OR COPYING OF THIS DOCUMENT IS STRICTLY PROHIBITED BY ANYONE OTHER THAN THE CLIENT FOR THE SPECIFIC PROJECT

July 23, 2020
Project No. 18-1557

July 23, 2020
Project No. 18-1557

Mr. Alex Walter
Riaz Capital
2744 East 11th Street
Oakland, California 94601

Subject: Geotechnical Investigation
Proposed Residential Building
820 West MacArthur Boulevard
Oakland, California

Dear Mr. Walter,

We are pleased to present our geotechnical investigation report for the proposed residential building to be constructed at 820 West MacArthur Boulevard in Oakland, California. Our geotechnical investigation was performed in accordance with our proposal dated May 21, 2020.

The subject property is located on the northwestern corner of the intersection of West MacArthur Boulevard and West Street. The site is relatively level and irregular shaped with maximum plan dimensions of about 140 by 195 feet. Currently, the site is occupied by a one-story building (formerly a gasoline station building), paved parking lot and a driveway. The site was a former gasoline station and there are underground storage tanks (USTs), fuel dispenser pipelines, and environmentally impacted soil beneath localized areas.

Plans are to demolish the existing improvements on the site and construct a residential building that will occupy most of the site. The proposed residential building will be constructed at-grade and will be five stories high. Prior to constructing the proposed building, environmental remediation, which may include excavation and removal of environmentally impacted soil to depths of 5 to 10 feet, will be performed.

From a geotechnical standpoint, we conclude the site can be developed as planned. The primary geotechnical concerns are: (1) the highly expansive near-surface soil, and (2) providing adequate foundation support for the proposed building. The firm native alluvium encountered beneath the site has moderate strength and relatively low compressibility that can provide adequate foundation support for the proposed building. However, we understand environmental remediation to be performed at the site will include excavation and removal of environmentally impacted soil to depths of 5 to 10 feet bgs and backfilling these excavations with compacted, engineered fill. Isolated spread footings bearing on engineered fill and native alluvium transitions will be susceptible to abrupt differential settlements. To reduce the potential for abrupt differential settlement at engineered fill and native alluvium transitions, we conclude the proposed building should be supported on a stiffened shallow foundation system, such as interconnected continuous spread footings or a mat.

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

REGISTERED PROFESSIONAL ENGINEER
HUI JUN LIANG
GE2663
Exp. 6/30/2022
GEOTECHNICAL
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
CRAIG S. SHIELDS
GE2116
3/31/21
GEOTECHNICAL
STATE OF CALIFORNIA

Linda H. J. Liang, P.E., G.E.
Associate Engineer

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

Enclosure

TABLE OF CONTENTS

LIST OF FIGURES

ROCKRIDGE GEOTECHNICAL

APPENDIX A

Figures A-1 to A-5	Cone Penetration Test Results

APPENDIX B

Figure B-1	Plasticity Chart

Corrosion Test Results

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL BUILDING
820 WEST MACARTHUR BOULEVARD
Oakland, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed residential building to be constructed at 820 West MacArthur Boulevard in Oakland, California. The subject property is located on the northwestern corner of the intersection of West MacArthur Boulevard and West Street, as shown on the Site Location Map, Figure 1.

The site is relatively level and irregular shaped with maximum plan dimensions of about 140 by 195 feet, as shown on the Site Plan, Figure 2. Currently, the site is occupied by a one-story building (formerly a gasoline station building), paved parking lot and a driveway. The site was a former gasoline station and there are underground storage tanks (USTs), fuel dispenser pipelines, and environmentally impacted soil beneath localized areas.

Plans are to demolish the existing improvements on the site and construct a residential building that will occupy most of the site. The proposed residential building will be constructed at-grade and will be five stories high. Prior to constructing the proposed building, environmental remediation, which may include excavation and removal of environmentally impacted soil to depths of 5 to 10 feet, will be performed.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal dated May 21, 2020. Our scope of services consisted of reviewing available subsurface information and geologic maps of the site and vicinity, exploring subsurface conditions at the site, and performing engineering analyses to develop conclusions and recommendations regarding:

- subsurface conditions at the site
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure

- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlement
- subgrade preparation for slab-on-grade floors and exterior concrete flatwork
- surface drainage and bio-swales
- site grading and fill placement, including fill quality and compaction requirements
- 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 AND LABORATORY TESTING

Subsurface conditions at the site were investigated by performing five cone penetration tests (CPTs). We also advanced three hand-auger borings to obtain near-surface soil samples for visual classification and laboratory tests. Prior to performing the field investigation, we obtained a permit from Alameda County Public Works Agency (ACPWA). We also contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained Precision Locating, LLC, a private utility locator, to check that the CPT locations were clear of underground utilities. Details of the field exploration are described below.

3.1 Cone Penetration Tests

Middle Earth Geo Testing, Inc. of Orange, California performed the CPTs, designated as CPT-1 through CPT-5, on June 19, 2020 at the approximate locations shown on Figure 2. 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 and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the types and approximate strength characteristics of the soil encountered. The CPTs were each advanced to a depth of 50 feet below the ground surface (bgs). The CPT logs, showing tip resistance and friction ratio by depth, as well as pore pressure and soil behavior type, are presented on Figures A-1 through A-5 in Appendix A.

Upon completion, the CPT holes were backfilled with cement grout in accordance with ACPWA grout standards.

3.2 Hand-Auger Borings

Three hand-auger borings were advanced on June 19, 2020 to obtain samples of the near surface soil for visual classification and laboratory testing. The borings, designated as HA-1 through HA-3, were each advanced using a three-inch-diameter hand auger to a depth of 3.5 feet bgs. The approximate locations of HA-1 through HA-3 are shown on Figure 2. Upon completion, the boreholes were backfilled with the soil cuttings. Descriptions of soil encountered in the hand-auger borings are presented in Table 1.

TABLE 1
Soil Descriptions from Hand-Auger Borings

Boring	Depth (feet)	Soil Description	Laboratory Tests
HA-1	0 to 3.5 feet	CLAY with SAND (CH) green-gray	Moisture content (19.2% at 3 ft); Plasticity test (LL=50, PI=33 at 3 ft)
HA-2	0 to 3.5 feet	SANDY CLAY (CH) dark brown	Moisture content (17.9% at 1 ft); Corrosivity test (at 2 ft)
HA-3	0 to 3.5 feet	SANDY CLAY (CH) dark brown	Moisture content (17.5% at 1 ft); Plasticity test (LL=50, PI=29 at 1 ft); Corrosivity test (at 3 ft)

3.3 Laboratory Testing

We re-examined the soil samples obtained from the hand-auger borings to confirm the field classifications and selected representative samples for laboratory testing. We performed laboratory tests to measure moisture content, plasticity, and corrosivity. The results of the laboratory tests are presented in Table 1 and attached in Appendix B.

4.0 SUBSURFACE CONDITIONS

A regional geologic map prepared by Graymer, et al. (2006), a portion of which is presented on Figure 3, indicates the site is underlain by Holocene-aged alluvial fan and fluvial deposits (Qhaf). Alluvial fan and fluvial deposits are generated when sediments are transported and deposited by rivers and streams. These types of deposits can be relatively uniform, but are often composed of different layers of different particle mixtures of gravelly, sandy, and clayey soils.

The results of our hand-auger borings and CPTs indicate the site is underlain by up to about two feet of fill in localized areas. The fill consists of medium dense sand and silty sand. The fill, or ground surface where fill is not present, is underlain by alluvium. Where explored, the alluvium consists of interbedded layers of stiff to hard clay with varying sand and gravel content and medium dense to very dense sand with varying clay and silt content that extends to the maximum depth explored of 50 feet bgs. The sand layers are typically less than two feet thick. Atterberg limits tests performed on samples of the near-surface clay with sand and sandy clay indicate the near-surface clay is highly expansive[1] with plasticity indices (PIs) of 29 and 33.

Groundwater was measured in our CPTs at depths of approximately 14 to 15 feet bgs during drilling. The groundwater levels may not have been fully stabilized at the time of these measurements. We reviewed the report Seismic Hazard Zone Report (2003) prepared by the California Geological Survey (CGS) for the Oakland West 7.5-Minute Quadrangle. The report indicates a historic high groundwater level in the site vicinity of about 7 feet bgs. The depth to groundwater is expected to vary several feet seasonally, depending on rainfall amounts.

[1] Expansive soil undergoes large volume changes with changes in moisture content (i.e. it shrinks when dried and swells when wetted).

5.0 SEISMIC CONSIDERATIONS

The San Francisco Bay Area is considered to be 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 and Faulting

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

The major active faults in the area are the Hayward, San Andreas, and Calaveras faults. These and other faults in the region are shown on Figure 4. Numerous damaging earthquakes have occurred along these faults in recorded time. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[2] [Petersen et al. (2014) & Thompson et al. (2016)] are summarized in Table 2. These references are based on the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013).

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

TABLE 2
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction	Characteristic Moment Magnitude
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	4.4	East	7.58
Hayward (North, HN)	4.4	East	6.90
Hayward (South, HS)	9.7	East	7.00
Total Calaveras (CN+CC+CS+CE)	20	East	7.43
Calaveras (North, CN)	20	East	6.86
Mount Diablo Thrust North CFM	21	East	6.72
Mount Diablo Thrust	21	East	6.67
Concord	26	East	6.45
Total North San Andreas (SAO+SAN+SAP+SAS)	26	Southwest	8.04
North San Andreas (Peninsula, SAP)	26	Southwest	7.38
Green Valley	29	Northeast	6.30
San Gregorio (North)	31	West	7.44
Clayton	32	East	6.57
Mount Diablo Thrust South	33	East	6.50
Greenville (North)	35	East	6.86
North San Andreas (North Coast, SAN)	35	West	7.52
West Napa	38	North	6.97
Monte Vista - Shannon	39	South	7.14
Great Valley 05 (Pittsburg - Kirby Hills alt1)	40	Northeast	6.60
Rodgers Creek - Healdsburg	40	Northwest	7.19
Great Valley 05 (Pittsburg - Kirby Hills alt2)	43	East	6.66
Las Positas	47	East	6.50

Since 1800, four major earthquakes have been recorded on the North San Andreas fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas fault (Toppozada and Borchardt 1998). The estimated moment magnitude (M_w) for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of October 17, 1989 had an M_w of 6.9 and occurred about 95 kilometers south of the site.

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

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

5.2 Geologic Hazards

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

---

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

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

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

5.2.1 Ground Shaking

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

5.2.2 Liquefaction and Associated Hazards

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

The site is located within a zone of liquefaction potential, as shown on the map titled *State of California Earthquake Zones of Required Investigation, Oakland West Quadrangle*, prepared by the California Geological Survey (CGS), released February 14, 2003 (Figure 5). 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.

We used the results of our CPTs to evaluate the potential for liquefaction to occur at the site. Liquefaction susceptibility was assessed using the software CLiq v3.0 (GeoLogismiki, 2019). CLiq uses measured field CPT data and assesses liquefaction potential, including post-earthquake vertical settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). Our liquefaction analyses were performed using the methodology proposed by Boulanger and Idriss (2014). We also used the relationship proposed by Zhang, et al (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 7 feet bgs. In accordance with the 2019 CBC, we used a peak ground acceleration of 0.86 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 Total Hayward + Rogers Creek fault, as presented in Table 2.

Our liquefaction analyses indicate there are several thin layers of medium dense sand below the groundwater table that are potentially liquefiable. The potentially liquefiable layers are generally less than two feet thick and have soil behavior types of "sand", "silty sand", and "sandy silt". We estimate total free-field ground settlement associated with liquefaction (referred to as post-liquefaction reconsolidation) at the site after the above-defined MCE event will be less than 3/4 inch and differential settlement will be less than 1/2 inch over a horizontal distance of 30 feet.

Ishihara (1985) presented an empirical relationship that provides criteria used to evaluate whether liquefaction-induced ground failure, such as sand boils, would be expected to occur under a given level of shaking for a liquefiable layer of given thickness overlain by a resistant, or protective, surficial layer. Our analysis indicates the non-liquefiable soil overlying the potentially liquefiable soil layers at the site is sufficiently thick and the potentially liquefiable layers are sufficiently thin such that the potential for surface manifestations from liquefaction, such as sand boils and loss of bearing capacity for shallow foundations, is low.

Considering the relatively flat site grades, the absence of a free face in the site topography, and the discontinuous nature of the potentially liquefiable layers, we conclude the risk of lateral spreading is very low.

5.2.3 Cyclic Densification

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

5.2.4 Ground Surface Rupture

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

6.0 DISCUSSIONS AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned. The primary geotechnical concerns are: (1) the highly expansive near-surface soil, and (2) providing adequate foundation support for the proposed building. These and other issues are discussed in this section.

6.1 Expansive Soil

The site is underlain by near-surface soil that has a high expansion potential. Expansive near-surface soil is subject to volume changes during fluctuations in moisture content. These volume changes can cause movement and cracking of foundations, pavements, and slabs. In general, the adverse effects associated with expansive soil can be mitigated by moisture-conditioning the expansive soil, providing non-expansive fill below slabs, and either supporting foundations below the zone of severe moisture change or by providing a stiff, shallow foundation that can limit deformation of the superstructure as the underlying soil shrinks and swells.

In addition, at expansive soil sites it is critical to properly manage surface and subsurface drainage to prevent water from collecting beneath pavements and slabs or behind below-grade walls, where it can lead to swelling and shrinking of the subgrade soil and can cause subgrade instability under vehicular loads. If permeable pavements, tree wells, irrigated landscaped zones, and storm water infiltration basins will be constructed in close proximity to the proposed buildings, they should incorporate design elements that prevent saturation of the soil adjacent to and below building foundations. While the objective of permeable pavement systems and infiltration basins is to allow for water storage and infiltration, we conclude that infiltration into the subgrade soil is not feasible at this site due to the low permeability of the highly expansive clay. Furthermore, from a geotechnical standpoint, water should not be allowed to collect alongside or beneath the building foundations, pavements and flatwork. This can be achieved by providing subdrain systems and impermeable liners beneath permeable surfaces and installing vertical barriers between permeable surfaces underlain by subdrains and non-permeable surfaces underlain by conventional aggregate base.

6.2 Foundation and Settlement

The firm native alluvium encountered beneath the site has moderate strength and relatively low compressibility that can provide adequate foundation support for the proposed building. However, we understand environmental remediation to be performed at the site will include excavation and removal of environmentally impacted soil to depths of 5 to 10 feet bgs and backfilling these excavations with compacted, engineered fill. Isolated spread footings bearing on engineered fill and native alluvium transitions will be susceptible to abrupt differential settlements. To reduce the potential for abrupt differential settlement at engineered fill and native alluvium transitions, we conclude the proposed building should be supported on a stiffened shallow foundation system, such as interconnected continuous spread footings or a mat.

We estimate total settlement of the proposed building supported on properly designed and constructed continuous footings or a mat will be less than 3/4 inch and differential settlement will be less than 1/2 inch in 30 feet. As discussed in Section 5.2.2, the continuous footings or mat should be designed for additional liquefaction-induced total and differential settlements on the order of 3/4 inch and 1/2 inch over a horizontal distance of 30 feet, respectively.

6.3 Interior and Exterior Slabs-on-Grade

To mitigate the effects of highly expansive near-surface soil, the building slab-on-grade floor and capillary break/vapor barrier should be underlain by at least 18 inches of non-expansive soil. Where the building will be supported on a mat foundation, the upper 6 inches of mat subgrade should consist of non-expansive soil. In addition, exterior concrete flatwork should perform satisfactorily if it is supported on a layer of non-expansive soil at least 12 inches thick (measured beneath the aggregate base layer). Non-expansive soil may consist of select fill or lime-treated onsite soil, as presented in Section 7.1.

6.4 Construction Considerations

The soil to be excavated for the proposed foundations and utilities is expected to consist primarily of clay which can be excavated with conventional earth-moving equipment, such as backhoes. Removal of existing foundations will require equipment capable of breaking up reinforced concrete.

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

We understand the proposed building will be constructed at-grade and, therefore, we do not anticipate significant deep excavations. However, construction of the proposed elevator(s) and any underground vaults, if planned, may require excavations in excess of five feet below the existing ground surface. Where there is sufficient clearance from the property line, the excavation sides may be slope cut at a maximum inclination of 1:1 (horizontal:vertical), which is consistent with OSHA Type B soil. Where there is insufficient space to slope-cut the excavations, shoring may be required. The selection, design, construction, and performance of the shoring system (if needed) should be the responsibility of the contractor.

6.5 Soil Corrosivity

Laboratory testing was performed by Project X Corrosion Engineering of Murrieta, California on two samples of soil obtained during our field investigation from borings HA-2 and HA-3 at a depths of 2 and 3 feet bgs, respectively. The results of the test are presented in Appendix B of this report.

The resistivity test results (536 and 3,350 ohm-cm) indicate the near-surface soil is "moderately to highly corrosive" to buried metallic structures. Accordingly, all buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric-coated steel or iron may need to be protected against corrosion, depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.

The chloride ion concentrations (3.0 and 12.2 mg/kg) and pH (7.7 and 7.1) indicate the near-surface soil is "negligibly corrosive" to buried metallic structures and reinforcing steel in concrete structures below ground. The results also indicate the sulfate ion concentrations are sufficiently low such that sulfates do not pose a threat to buried 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 Fill Placement

Any vegetation and organic topsoil should be stripped in areas to receive improvements (i.e., building or flatwork). Site demolition should include the removal of all existing underground utilities and existing building foundations and slabs. 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 will not interfere with the proposed construction, they may be abandoned in-place provided the lines are filled with lean concrete or cement grout to the property line. Voids resulting from demolition activities and overexcavations from environmental remediation activities that extend below finished improvements should be properly backfilled with engineered fill under our observation and following the recommendations provided later in this section.

The near-surface clay at the site is highly expansive. To mitigate the detrimental effects of highly expansive near-surface soil, the building slab-on-grade floors or mat foundation, should be underlain by at least 18 and 6 inches of non-expansive soil, respectively, 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. In addition, exterior concrete flatwork should be underlain by a layer of non-expansive soil at least 12 inches thick (measured beneath the aggregate base layer).

In areas to receive fill, the soil subgrade should be scarified to a depth of at least eight inches, moisture-conditioned to at least four percent above optimum moisture content and compacted to between 87 and 92 percent relative compaction[6]. If material to be used as fill is imported to the site, it should meet the requirements for select fill provided below in Section 7.1.1. A summary of the compaction requirements for the various types of fill that may be used at the site is presented in Table 3.

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

TABLE 3
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

*Note: Select fill and lime-treated onsite soil are considered low-plasticity soil.

We recommend the upper eight inches of the soil subgrade beneath pavements be scarified, moisture-conditioned, and recompacted. The scarification and recompaction should extend at least two feet beyond the perimeters of the pavements, except where constrained by the property line. The scarified soil subgrade should be moisture-conditioned and compacted in accordance with the requirements provided in Table 3. The subgrade beneath pavements should be firm and non-yielding under construction equipment wheel loads.

7.1.1 Select Fill

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

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

7.1.2 Lime Treatment

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, mat, or exterior concrete flatwork will consist of lime-treated on-site soil, the upper 18, 6, and 12 inches the soil subgrade, respectively, 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 expansive on-site material to produce a non-expansive fill for the proposed buildings. 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 100 pounds per cubic foot (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.

7.1.3 Utility Trench Backfill

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

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 footprints (see Figure 6). Furthermore, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the building or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

7.2 Surface Drainage and Landscaping

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 to the expansive clay subgrade.

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

7.2.2 Landscaping

Prior experience and industry literature indicate that some species of high water-demand[7] trees can induce ground-surface settlement by drawing water from the expansive clay, causing it to shrink. Where these types of trees are planted near buildings, the ground-surface settlement may result in damage to structure. This problem usually occurs 10 or more years after planting, as the trees reach mature height. To reduce the risk of tree-induced, building settlement, we recommend trees of the following genera are not planted within 25 feet of the proposed

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

buildings: *Eucalyptus, Populus, Quercus, Crataegus, Salix, Sorbus* (simple-leafed), *Ulmus, Cupressus, Chamaecyparis,* and *Cupressocyparis.* Because this is a limited list and does not include all genera that may induce ground-surface settlement, a tree specialist should be consulted prior to selection of trees to be planted at the site.

7.2.3 Bioswales

Where bioswales will be part of the project, we recommended that 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.

7.3 Foundations

Recommendations for interconnected continuous footings and mat foundation are presented in this section.

7.3.1 Interconnected Continuous Footings

Interconnected continuous footings should be at least 18 inches wide and bottom on firm native alluvium or engineered fill. Perimeter footings should be bottomed least 36 inches below the lowest adjacent outside grade. The perimeter footing embedment depth may be decreased by six inches where pavement or concrete flatwork is adjacent to the new building. Interior footings should be bottomed on extend at least 24 inches below the bottom of the capillary moisture break. Footings may be designed using an allowable bearing pressure of 3,000 pounds per square foot (psf) for dead-plus-live loads; this value may be increased by one-third for total design loads, which include wind or seismic forces. The allowable bearing pressures for dead-plus-live and total loads include factors of safety of at least 2.0 and 1.5, respectively.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute lateral resistance, we recommend using a uniform pressure of 1,500 psf for transient load conditions and an equivalent fluid weight of 260 pounds per cubic foot (pcf) for sustained load conditions; the upper foot of soil should be ignored unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the footing will eventually heave, which may result in cracking and distress. We recommend rat slabs consisting of at least two inches of controlled low-strength material (CLSM) or structural concrete be placed in the bottoms of the footings to protect them from drying out, softening from ponding water and/or disturbance from foot traffic during construction. We should check footing excavations prior to placement of the rat slabs. The CLSM used to construct the rat slabs should have a 28-day unconfined strength of 100 psi and should be poured within two days of footing excavation. The rat slab thickness may be counted as part of the minimum footing embedment.

7.3.2 Mat Foundation

For mat design, we recommend using a modulus of subgrade reaction of 30 pounds per cubic inch (pci) for dead-plus-live loads. This value has already been scaled to take into account the plan dimensions of the foundation and may be increased by 50 percent for total load conditions. Considering the large area of the mat, we expect the average bearing stress under the mat to be low; however, concentrated stresses will occur at column locations and at the edges of the mat. The mat should be designed to impose a maximum dead-plus-live bearing pressure of 3,000 psf on the foundation subgrade soil. This pressure may be increased by one-third for total load conditions. The edge of the mat should be bottomed at least 12 inches below the lowest adjacent finished grade.

Assuming the mat is underlain by a vapor retarder, a friction factor of 0.20 may be used to compute base friction. Where the mat foundation is supported directly on soil, a friction factor of 0.30 may be used. To compute lateral resistance, we recommend using a uniform pressure of 1,500 psf for transient load conditions and an equivalent fluid weight of 260 pcf for sustained load conditions; the upper foot of soil should be ignored unless confined by a slab or pavement. The values for friction coefficient and passive pressure include a factor of safety of 1.5 and may be used in combination without reduction.

7.4 Concrete Slab-on-Grade Floor

The subgrade for slab-on-grade floor or mat should be prepared in accordance with our recommendations in Section 7.1. Where water vapor transmission through the floor slab/mat is not desirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor slab/mat. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The particle size of the capillary break material should meet the gradation requirements presented in Table 4.

TABLE 4
Gradation Requirements for Capillary Moisture Break

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

The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. For the mat foundation option, the four-inch capillary break can be eliminated provided the vapor retarder meets the requirements for Class A vapor retarders. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab or mat. Therefore, concrete for the floor slab and mat should have a low w/c ratio - less than 0.50. If the concrete is poured directly over the vapor retarder, we recommend the w/c ratio of the concrete not exceed 0.45. In either case, water should not be added to the concrete mix in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.5 Exterior Concrete Flatwork

We recommend a minimum of four inches of Class 2 aggregate base be placed over 12 inches of non-expansive soil (see Sections 7.1.1 and 7.1.2) beneath proposed exterior concrete flatwork; the non-expansive soil should extend at least six inches beyond the slab edges. Non-expansive soil beneath exterior slabs-on-grade, such as patios and sidewalks, should be moisture-conditioned and compacted in accordance with the requirements provided in Table 3. Class 2 aggregate base beneath concrete flatwork should be compacted to at least 90 percent relative compaction.

Even with 16 inches of non-expansive soil (including aggregate base layer), 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.

7.6 Permanent Below-Grade Walls

Below-grade walls (i.e., elevator pit walls) should be designed to resist lateral earth pressure imposed by the retained soil. Since the elevator pit walls will be restrained from movement at the sides, they should be designed for at-rest conditions. We recommend restrained walls be designed using at-rest equivalent fluid weights of 60 and 91 pcf if the walls are drained and undrained, respectively. To evaluate the below-grade walls for seismic loading, we recommend using an active equivalent fluid weight of 40 pcf plus a seismic increment of 34 pcf (triangular distribution) for drained conditions; and an active equivalent fluid weight of 82 pcf plus a seismic increment of 17 pcf (triangular distribution) for undrained conditions.

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

One acceptable method for backdraining an elevator pit wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the wall. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi NC or equivalent). A proprietary, prefabricated collector drain system, such as Tremdrain Total Drain or Hydroduct Coil (or equivalent), designed to work in conjunction with the drainage panel may be used in lieu of the perforated pipe surrounded by gravel described above. The pipe should be connected to a suitable discharge point; a sump and pump system may be required to drain the collector pipes.

7.7 Seismic Design

As discussed in Section 5.2.2, the site is underlain by relatively thin layers of potentially liquefiable soil. Although the 2019 CBC call for a Site Class F designation for sites underlain by potentially liquefiable soil, we conclude a Site Class D designation is more appropriate because the potentially liquefiable layers are thin and relatively dense such that the site will not incur significant non-linear behavior during strong ground shaking. Therefore, for seismic design, we recommend Site Class D be used.

The latitude and longitude of the site are 37.8279° and -122.2720°, respectively. For design in accordance with 2019 CBC (ASCE 7-16), we recommend the following:

- Site Class D (stiff soil)
- S_S = 1.866g, S_1 = 0.712g

The 2019 CBC is based on the guidelines contained within ASCE 7-16. Per ASCE 7-16, 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 of ASCE 7-16. Assuming the C_s value will be calculated as outlined in Section 11.4.8, Exception 2 of ASCE 7-16, we recommend the following seismic design parameters:

- F_a = 1.0, F_v =1.7
- S_{MS} = 1.866g, S_{M1} = 1.210g
- S_{DS} = 1.244g, S_{D1} = 0.807g
- Seismic Design Category D for Risk Factors I, II, and III

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

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

9.0 LIMITATIONS

This geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the 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.

REFERENCES

ASCE 7-16 (2019). Minimum design loads and associated criteria for buildings and other structures (with Supplement 1). American Society of Civil Engineers

Boulanger, R.W and Idriss, I.M. (2014). "CPT and SPT Based Liquefaction Triggering Procedures", Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, Report No. UCD/CGM-14/01, April.

California Building Code (2019).

California Division of Mines and Geology (1996). Probabilistic seismic hazard assessment for the State of California, DMG Open-File Report 96-08.

California Geological Survey (2008). Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A.

California Geological Survey (2003), State of California Seismic Hazard Zones, Oakland West Quadrangle, Official Map, February 14, 2003.

Cetin, K. O., Bilge, H. T., Wu, J., Kammerer, A. M., & Seed, R. B. (2009). Probabilistic model for the assessment of cyclically induced reconsolidation (volumetric) settlements. Journal of Geotechnical and Geoenvironmental Engineering, 135(3), 387-398.

Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, and Zeng, Y., (2013). Uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model: U.S. Geological Survey Open-File Report 2013–1165, 97 p.

Graymer, R.W. (2000), "Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa and San Francisco Bay Counties, California", U.S. Geological Survey Miscellaneous Field Studies MF-2342.

Graymer, R.W., Moring, B.C., Saucedo, G.J, Wentworth, C.M., Brabb, E.E., and Knudsen, K.L. (2006). Geologic Map of the San Francisco Bay Region, prepared in cooperation with U.S. Geological Survey and California Geological Survey, March 6.

Ishihara, K. (1985). "Stability of Natural Deposits During Earthquakes."

Ishihara, K. and Yoshimine, M., (1992), Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes, Soils and Foundations, Volume 23, No. 1, pp 173-188.

Petersen, M.D., Moschetti, M.P., Powers, P.M., Mueller, C.S., Haller, K.M., Frankel, A.D., Zeng, Y., Rezaeian, S., Harmsen, S.C., Boyd, O.S., Field, E.H., Chen, R., Rukstales, K.S., Luco, N., Wheeler, R.L., Williams, R.A., and Olsen, A.H., (2014). Documentation for the 2014 update of the United States national seismic hazard maps: U.S. Geological Survey Open-File Report 2014–1091, 243 p.

Robertson, P.K. (2009). "Interpretation of Cone Penetration Tests - A Unified Approach", Canadian Geotechnical Journal, Vol. 46, No. 11, pp 1337-1355.

Thompson, E.M., Wald, D.J, Worden, B., Field, E.H., Luco, N., Petersen, M.D., Powers, P.M., Badie, R. (2016). Shakemap earthquake scenario: Building Seismic Safety Council 2014 Event Set (BSSC2014). U.S. Geological Survey. DOI: 10.5066/F7V122XD

Toppozada, T.R. and Borchardt G. (1998). "Re-evaluation of the 1936 "Hayward Fault" and the 1838 San Andreas Fault Earthquakes." Bulletin of Seismological Society of America, 88(1), 140-159.

Zhang, G., Robertson. P.K., Brachman, R., (2002), "Estimating Liquefaction Induced Ground Settlements from the CPT", Canadian Geotechnical Journal, 39: pp 1168-1180.

ROCKRIDGE GEOTECHNICAL

FIGURES

Base Map: Google Maps, 2018.

0 200 400 Feet

Approximate scale

820 WEST MACARTHUR BOULEVARD
Oakland, California

SITE LOCATION MAP

Date 07/16/20	Project No. 18-1557	Figure 1

ROCKRIDGE GEOTECHNICAL

EXPLANATION

CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical Inc., June 19, 2020

HLA-1 — Approximate location of hand auger boring by Rockridge Geotechnical Inc., June 19, 2020

Project limits

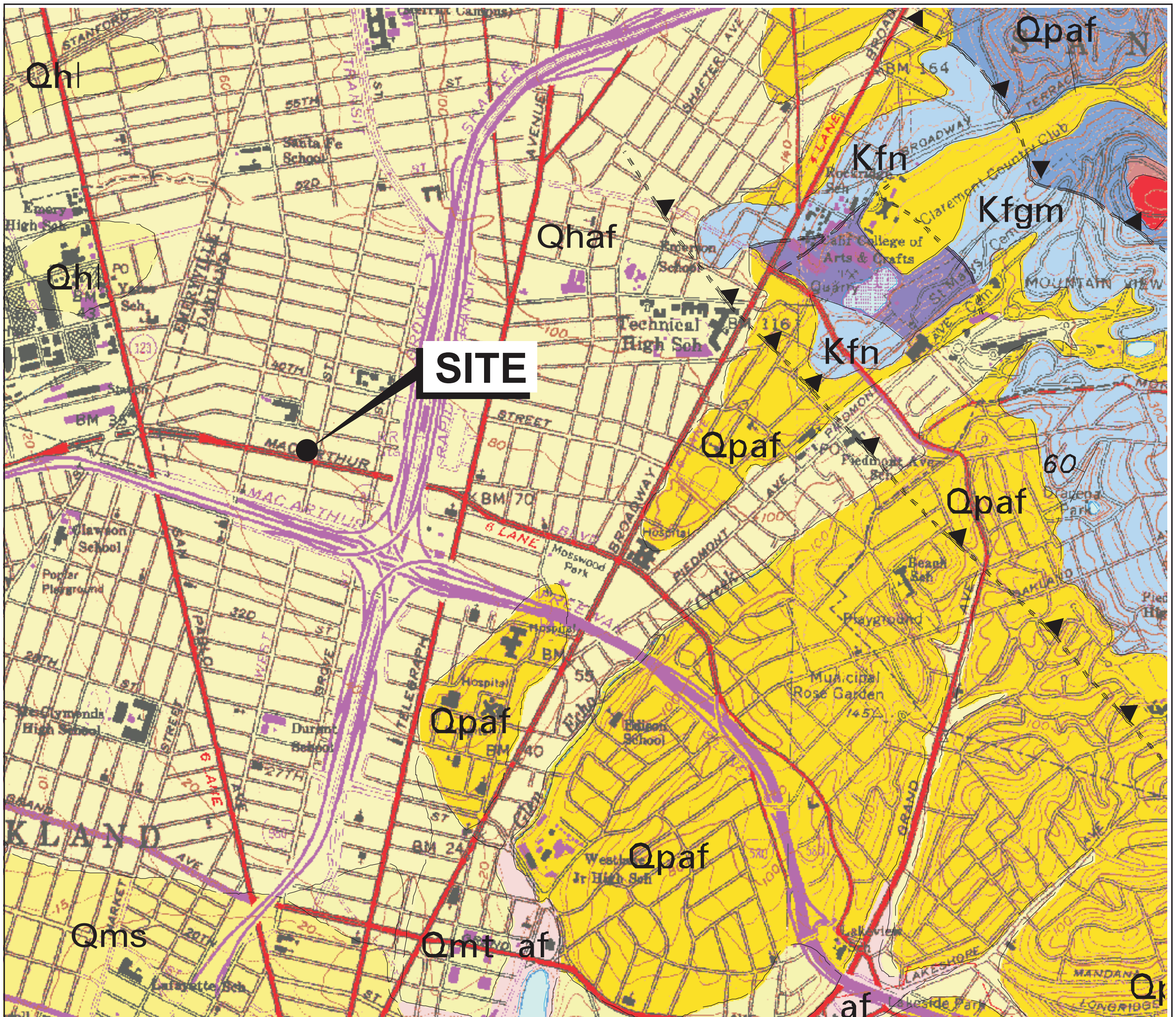
Reference: Base map from a drawing titled "820 W. Mac - ABD 3. 5 Story, 1st Floor", by Levy Design Partners Inc., dated April 27, 2020.

820 WEST MACARTHUR BOULEVARD
Oakland, California

SITE PLAN

Date 07/15/20	Project No. 18-1557	Figure 2

ROCKRIDGE GEOTECHNICAL



Base map: USGS MF 2342, Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California (Graymer, 2000).

EXPLANATION

Contact - Depositional or intrusive contact, dashed where approximately located, dotted where concealed

Fault - Dashed where approximately located, small dashed where inferred, dotted where concealed, queried where locations is uncertain

Reverse or thrust fault - Dotted where concealed

Anticline -Shows fold axis, dotted where concealed

Syncline

35 Strike and dip of bedding

Overturned bedding

Flat bedding

Vertical bedding

35 Strike and dip of foliation

Vertical foliation

35 Strike and dip of joints in plutonic rocks

Vertical joint

af	**Artificial fill (Historic)**
Qhaf	**Alluvial fan and fluvial deposits (Holocene)**
Qms	**Merrit sand (Holocene and Pleistocene)**
Qpaf	**Alluvial fan and fluvial deposits (Pleistocene)**
Qmt	**Marine terrace deposits (Pleistocene)**
Kfn	**Sandstone of the Novato Quarry terrane of Blake and others (1984) (Late Cretaceous)**

0 2,000 4,000 Feet

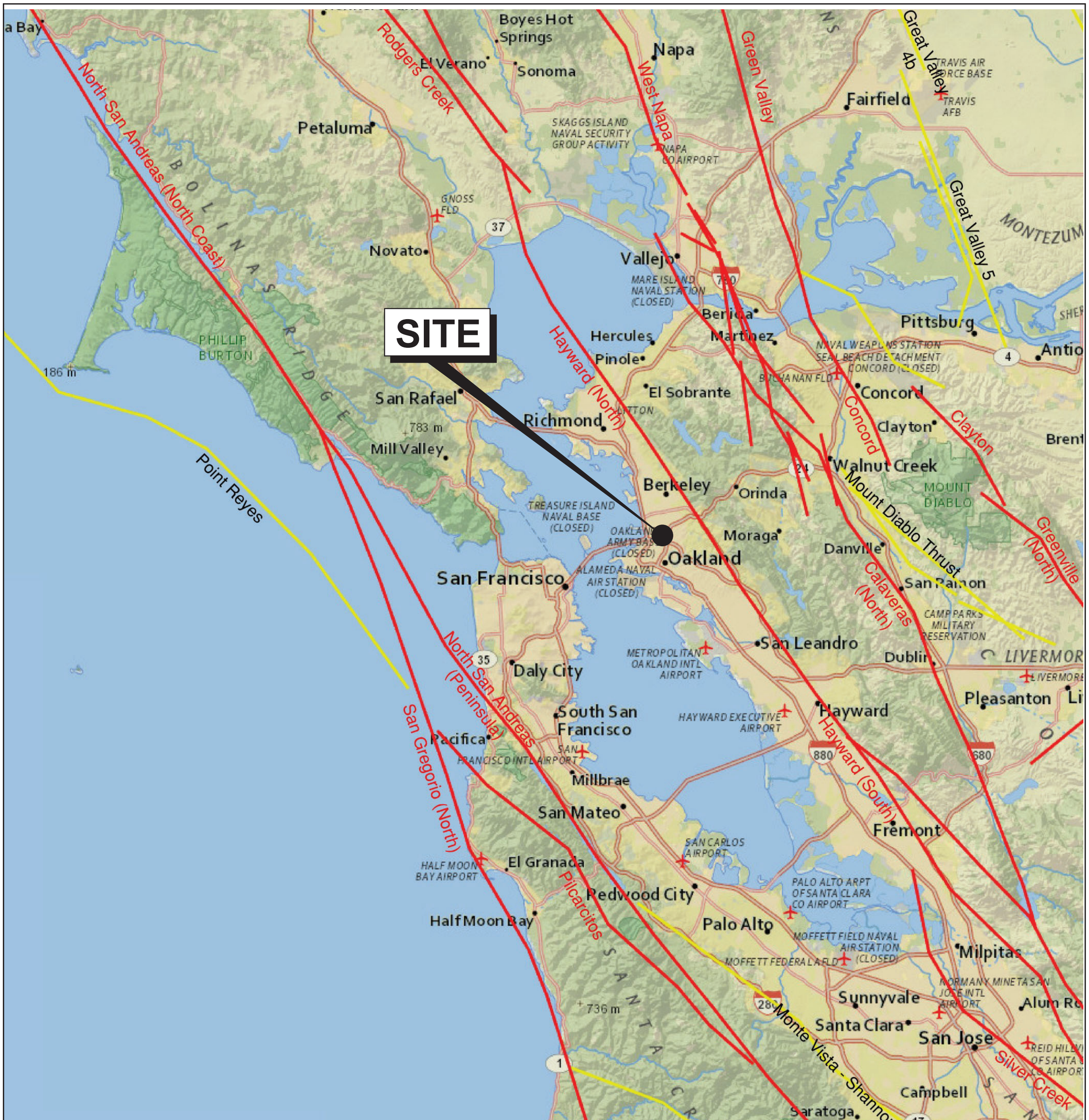
Approximate scale

820 WEST MACARTHUR BOULEVARD
Oakland, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 07/16/20 | Project No. 18-1557 | Figure 6



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

EXPLANATION

- Strike slip
- Thrust (Reverse)
- Normal

0 5 10 Miles

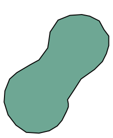
Approximate scale

820 WEST MACARTHUR BOULEVARD
Oakland, California

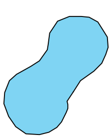
ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 07/16/20 | Project No. 18-1557 | 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

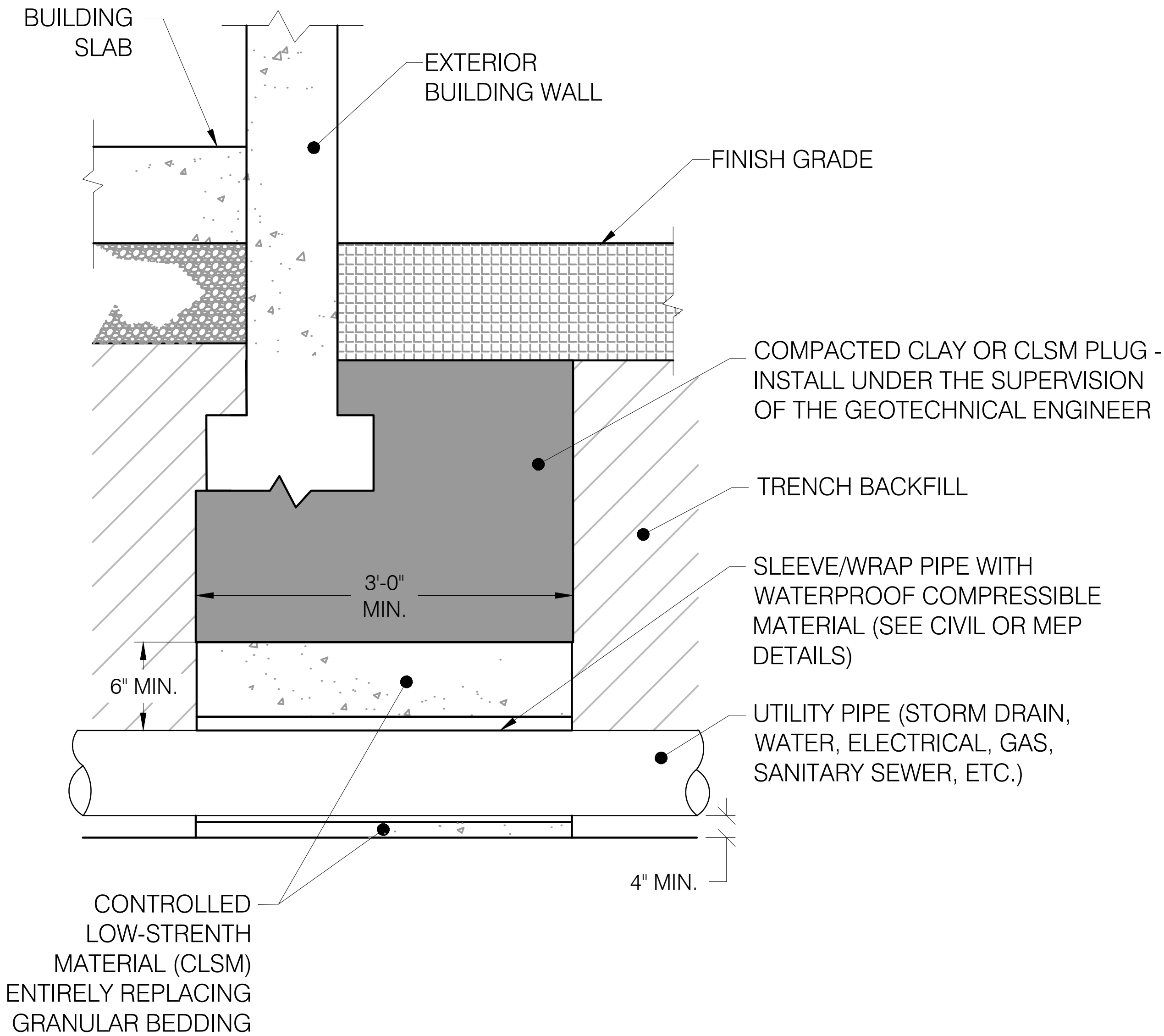
0 2,000 4,000 Feet
Approximate scale

820 WEST MACARTHUR BOULEVARD
Oakland, California

ROCKRIDGE GEOTECHNICAL

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 07/16/20 | Project No. 18-1557 | Figure 5

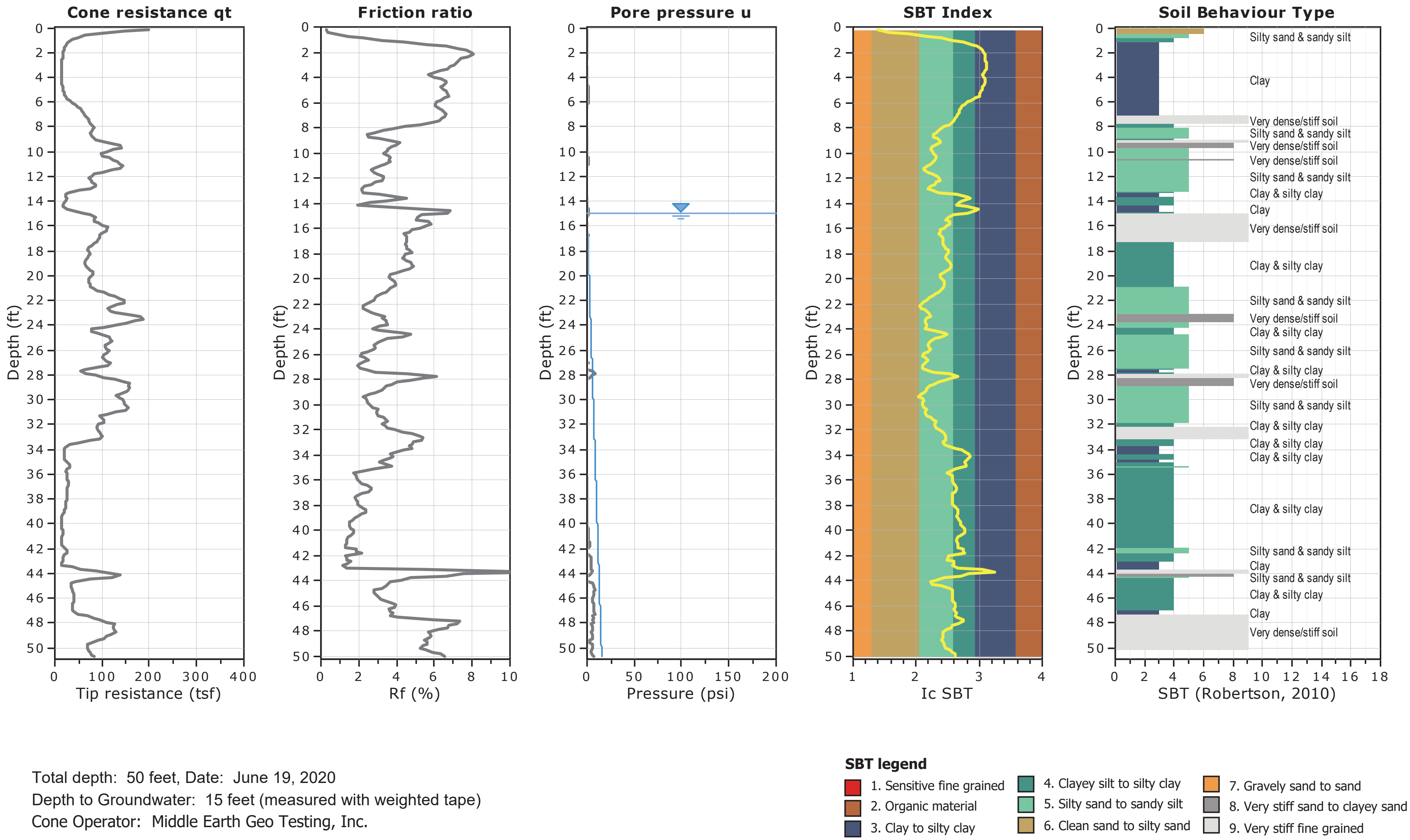


Not to Scale

820 WEST MACARTHUR BOULEVARD Oakland, California	**UTILITY TRENCH LOW-PERMEABILITY PLUG AT BUILDING PERIMETER**
ROCKRIDGE GEOTECHNICAL	Date 07/17/20 · Project No. 18-1557 · Figure 6

ROCKRIDGE GEOTECHNICAL

APPENDIX A
Cone Penetration Test Results



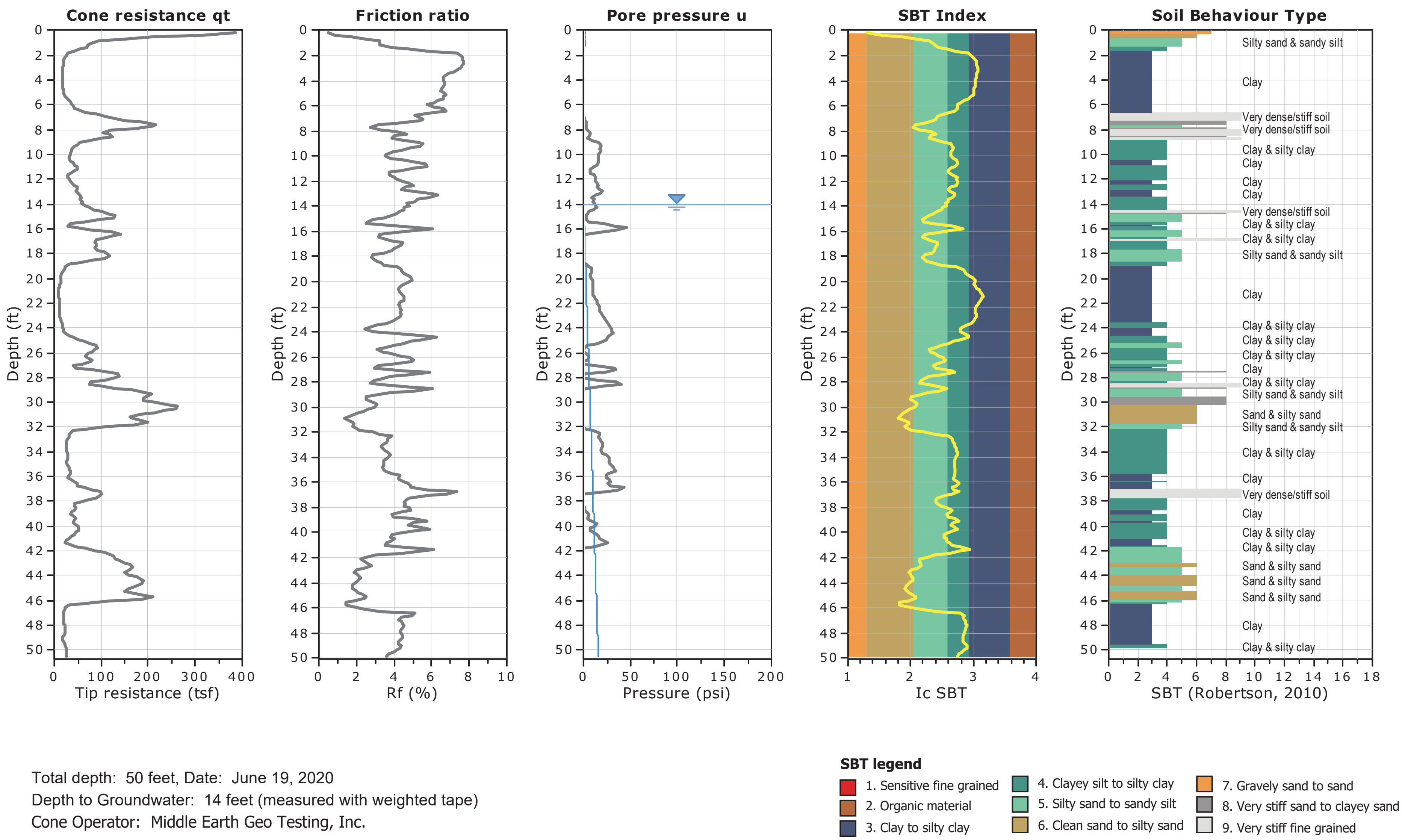
Total depth: 50 feet, Date: June 19, 2020
Depth to Groundwater: 15 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

820 WEST MACARTHUR BOULEVARD
Oakland, California

ROCKRIDGE GEOTECHNICAL

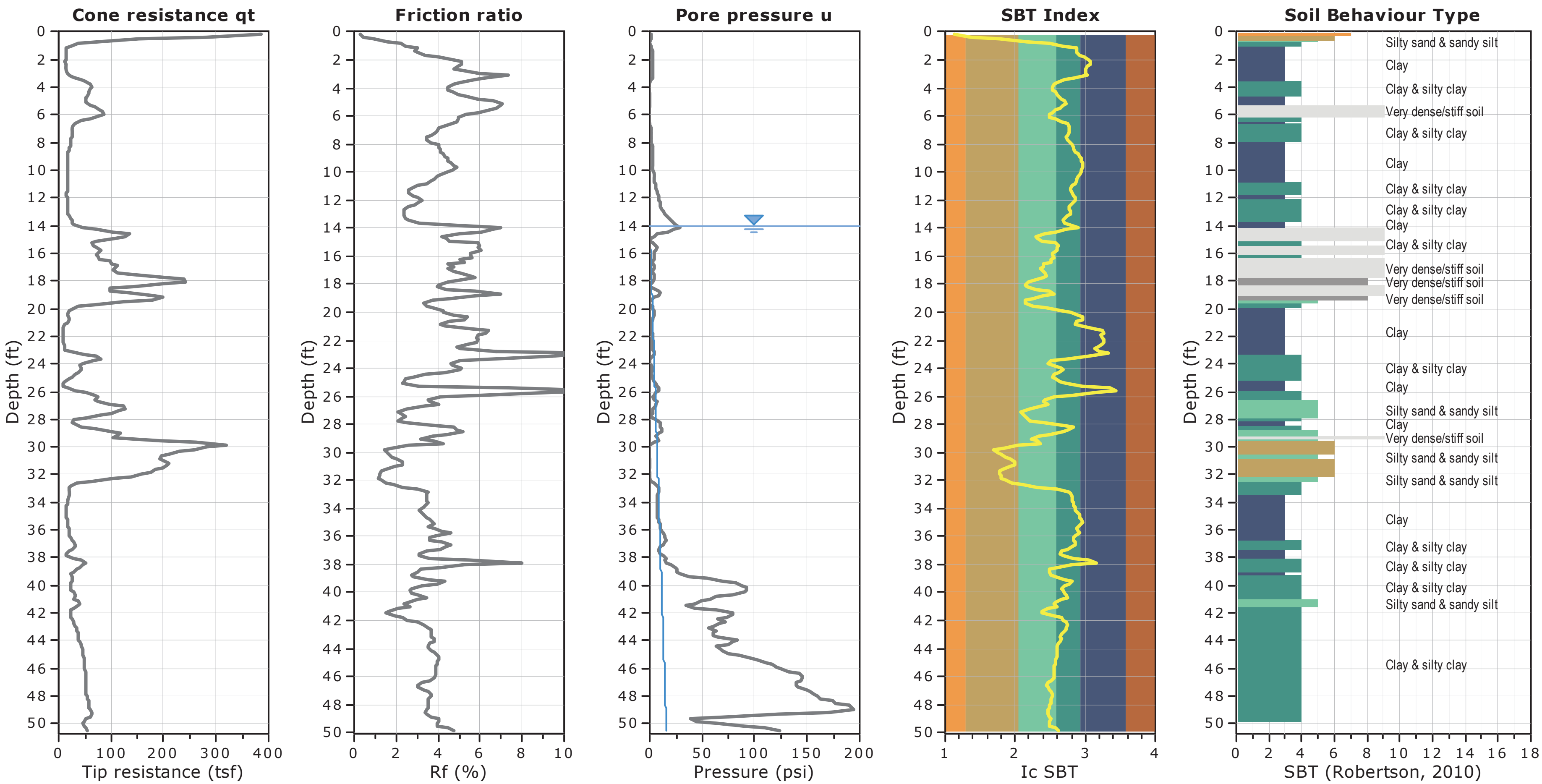
CONE PENETRATION TEST RESULTS
CPT-1

Date 07/16/20	Project No. 18-1557	Figure A-1



Total depth: 50 feet, Date: June 19, 2020
Depth to Groundwater: 14 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

820 WEST MACARTHUR BOULEVARD Oakland, California	**CONE PENETRATION TEST RESULTS** **CPT-2**
ROCKRIDGE GEOTECHNICAL	Date 07/16/20 · Project No. 18-1557 · Figure A-2



Total depth: 50 feet, Date: June 19, 2020
Depth to Groundwater: 14 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

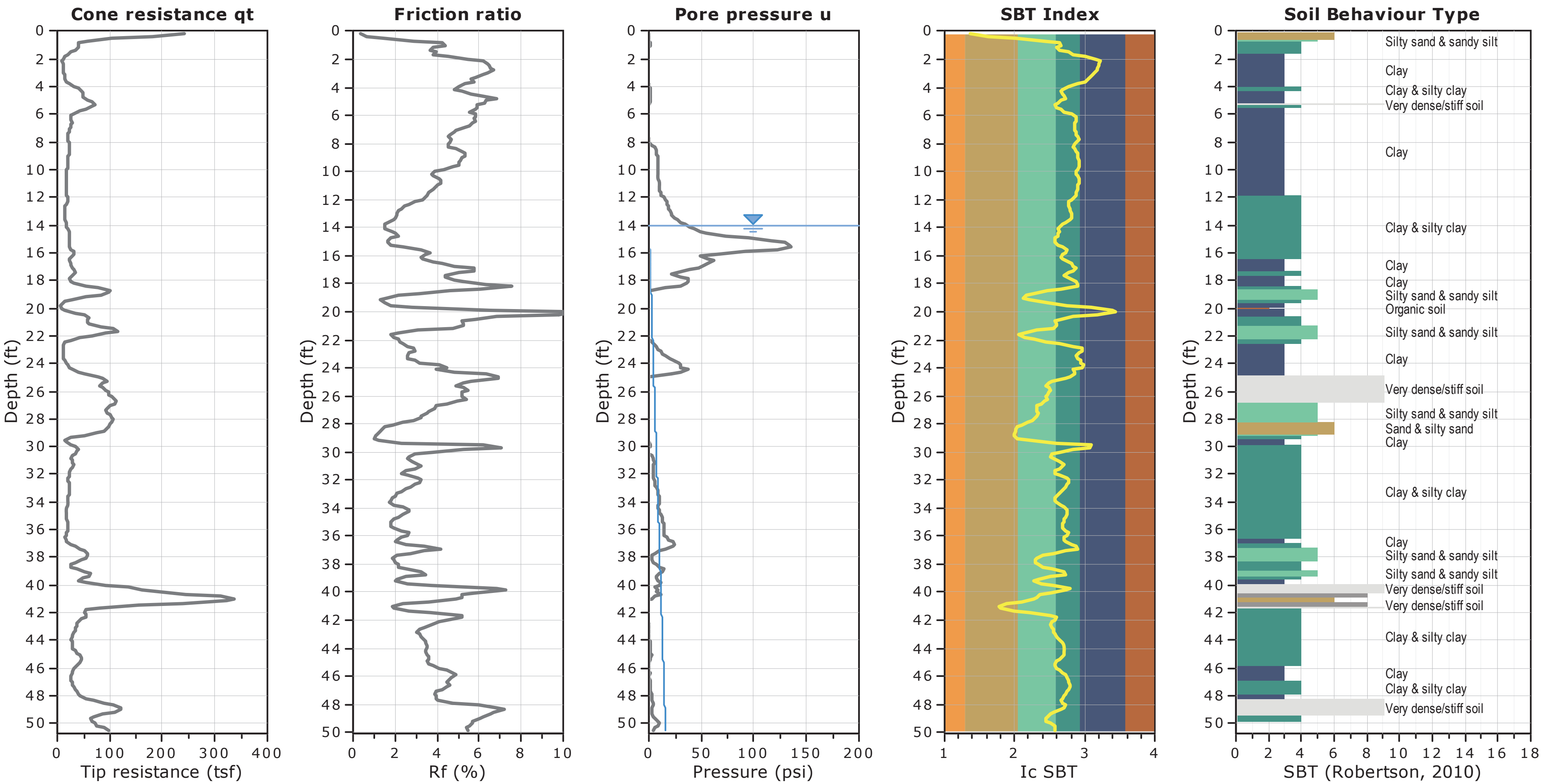
- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

820 WEST MACARTHUR BOULEVARD
Oakland, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 07/16/20	Project No. 18-1557	Figure A-3



Total depth: 50 feet, Date: June 19, 2020
Depth to Groundwater: 14 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

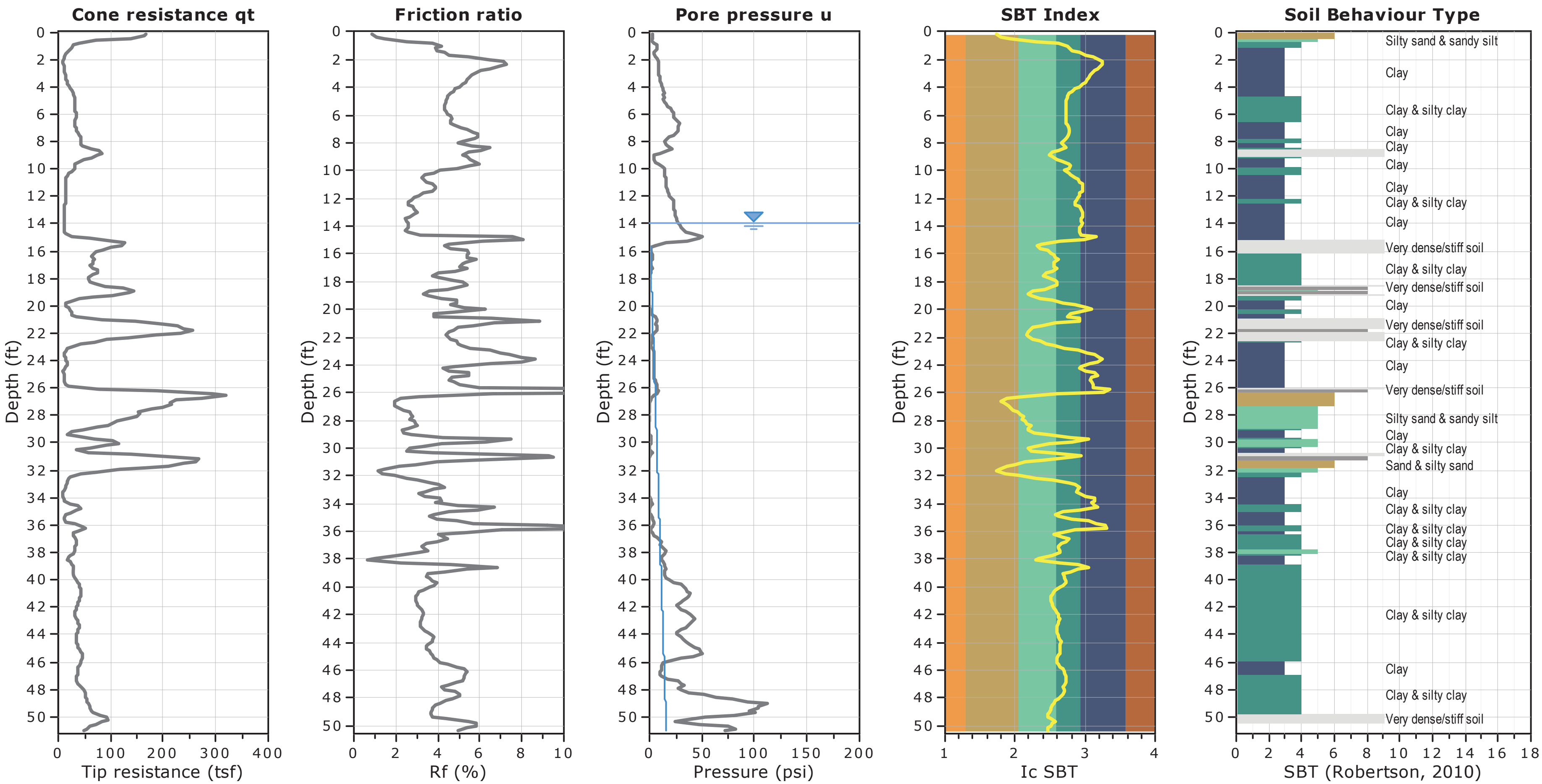
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

820 WEST MACARTHUR BOULEVARD
Oakland, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS CPT-4

Date 07/16/20	Project No. 18-1557	Figure A-4



Total depth: 50 feet, Date: June 19, 2020
Depth to Groundwater: 14 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

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

820 WEST MACARTHUR BOULEVARD
Oakland, California

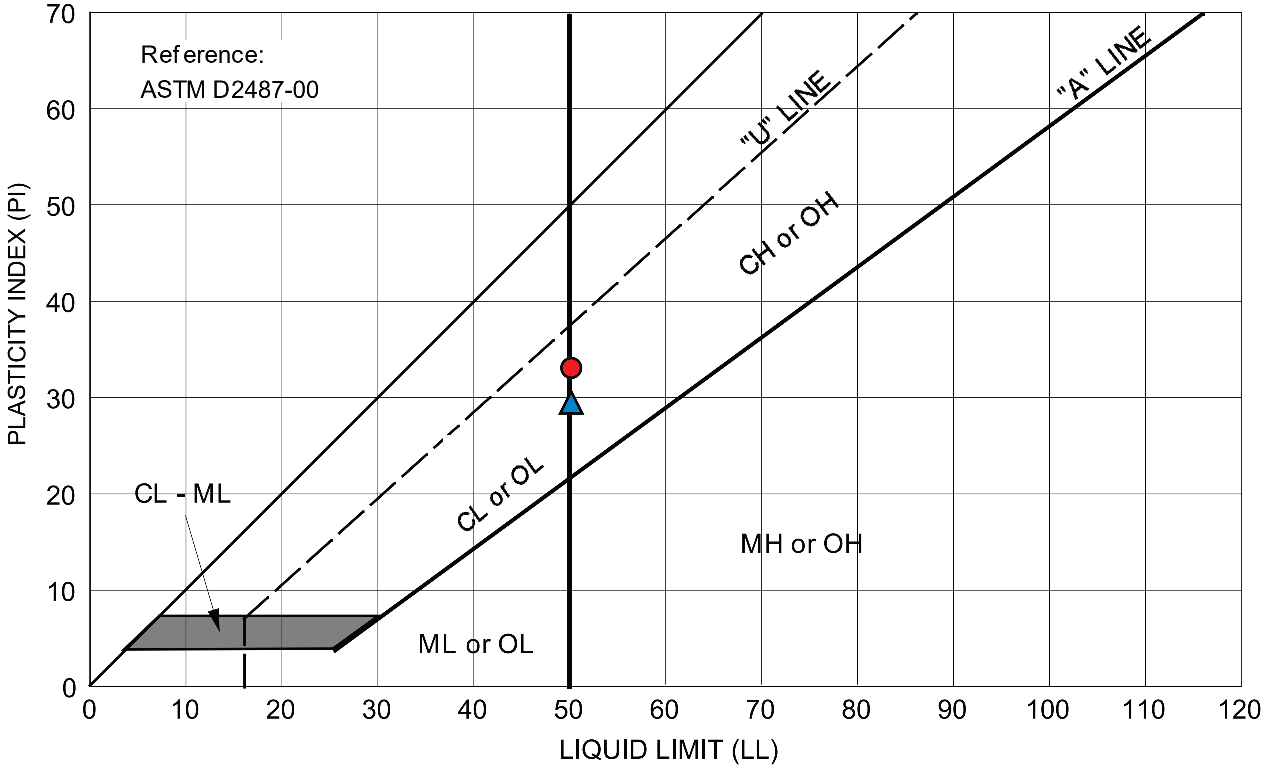
ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS CPT-5

Date 07/16/20	Project No. 18-1557	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
●	HA-1 at 3.0-3.5 feet	CLAY with SAND (CH), green-gray	19.5	50	33	--
▲	HA-3 at 1.0-1.5 feet	SANDY CLAY (CH), dark brown	17.5	50	29	--

820 WEST MACARTHUR BOULEVARD
Oakland, California

ROCKRIDGE GEOTECHNICAL

PLASTICITY CHART

Date 07/16/20 | Project No. 18-1557 | Figure B-1

Soil Analysis Lab Results

Client: Rockridge Geotechnical, Inc.
Job Name: 820 W MacArthur Blvd
Client Job Number: 18-1557
Project X Job Number: S200703E
July 8, 2020

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	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)
HA-2	2.0-2.5	48.3	0.0048	3.0	0.0003	40,200	3,350	7.73	190	ND	0.3	1.0	ND	29.3	ND	7.5	3.3	7.1	0.2
HA-3	3.0-3.5	47.1	0.0047	12.2	0.0012	536	536	7.09	195	ND	1.4	0.1	ND	40.2	0.4	21.1	17.4	4.1	1.1

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

THIS PAGE INTENTIONALLY LEFT BLANK