ROCKRIDGE GEOTECHNICAL

Prepared for **Mazel Management**

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL BUILDING
4709, 4715, & 4719 TIDEWATER AVENUE
OAKLAND, CALIFORNIA

October 26, 2016
Project No. 16-1212

ROCKRIDGE GEOTECHNICAL

October 26, 2016
Project No. 16-1212

Mr. Alan Spiegel
Mazel Management
1275 Hall Avenue, Suite D
Richmond, California 94804

Subject: Preliminary Geotechnical Investigation
Proposed Industrial Building
4709, 4715, and 4719 Tidewater Avenue
Oakland, California

Dear Mr. Spiegel,

The results of our preliminary geotechnical investigation in support of the due diligence evaluation of the properties located at 4709, 4715, and 4719 Tidewater Avenue in Oakland, California. Our preliminary geotechnical investigation was performed in accordance with our proposal dated October 10, 2016.

The project site encompasses an area of about 100,000 square feet and is approximately L-shaped. The site is relatively level and is currently an unpaved lot used as parking for big rig truck trailers. Preliminary plans are to develop the subject properties for commercial or industrial use and will include construction of an industrial building with a footprint of about 50,000 square feet. Conceptual plans for the proposed building, such as type of construction and number of stories, are not available when this report was prepared.

From a geotechnical standpoint, we preliminarily conclude the subject properties can be developed as planned. The primary geotechnical concerns are:

- the presence of about 9 feet of undocumented fill across the site;
- the presence of 7 to 11 feet of weak, highly compressible Bay Mud that will initiate a new cycle of consolidation if subjected to new loads (i.e. new foundation loads); and
- providing adequate foundation support for the proposed improvements.

We preliminarily conclude proposed lighter buildings, such as buildings with bearing pressures less than 500 pounds per square feet (psf) for dead-plus-live loads, may be supported on stiffened shallow foundations, such as a mat or interconnected continuous spread footings. Heavier buildings may be supported on spread footings bearing on improved ground. Preliminary conclusions and recommendations regarding seismic hazards, foundation design, and seismic design are presented in the attached report.

Prior to final design, additional borings and/or CPTs should be performed within the proposed building footprints to supplement existing subsurface information and to develop final geotechnical conclusions and recommendations.

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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PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL BUILDING
4709, 4715, AND 4719 TIDEWATER AVENUE
Oakland, California

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Rockridge Geotechnical, Inc. in support of the due diligence evaluation of the properties located at 4709, 4715, and 4719 Tidewater Avenue in Oakland, California. The subject properties are located southwest of the intersection of Tidewater Avenue and Lesser Street at the approximate location shown on the Site Location Map, Figure 1.

The project site encompasses an area of about 100,000 square feet and is approximately L-shaped, as shown on the Site Plan, Figure 2. The site is relatively level and is currently an unpaved lot used as parking for big rig truck trailers. Preliminary plans are to develop the subject properties for commercial or industrial use and will include construction of an industrial building with a footprint of about 50,000 square feet. Conceptual plans for the proposed building, such as type of construction and number of stories, are not available when this report was prepared.

2.0 SCOPE OF SERVICES

Our preliminary geotechnical investigation was performed in accordance with our proposal dated October 10, 2016. Our scope of services consisted of reviewing available subsurface information and geologic maps of the site and vicinity, exploring subsurface conditions at the site by advancing two cone penetration tests (CPTs), and performing engineering analyses to develop preliminary conclusions and recommendations regarding:

- subsurface soil and groundwater conditions
- site seismicity and seismic hazards, including the potential for liquefaction and lateral spreading, and total and differential resulting from liquefaction and/or cyclic densification
- the most appropriate foundation type(s) for the proposed building

- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimated foundation settlement under static and seismic conditions
- 2013 and 2016 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations.

3.0 FIELD INVESTIGATION

Our subsurface investigation consisted of performing two CPTs to provide in-situ soil data at the approximate locations shown on Figure 2. The CPTs, designated as CPT-1 and CPT-2, were advanced to a depth of about 50 feet below the existing ground surface (bgs). Prior to performing the CPTs, we obtained a permit from Alameda County Public Works Agency (ACPWA), 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.

The CPTs were performed by Middle Earth Geo Testing, Inc. of Orange, California on October 19, 2016. The CPTs were performed by hydraulically pushing a 1.4-inch-diameter cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the soil behavior types and approximate strength characteristics of the soil encountered. The CPT logs showing tip resistance and friction ratio, as well as interpreted soil behavior type, are presented on Figures A-1 and A-2 in Appendix A. Upon completion, the CPTs were backfilled with cement grout.

4.0 SUBSURFACE CONDITIONS

As presented on the Regional Geologic Map (Figure 3), the site is underlain by artificial fill (af) (Graymer, 2000). Where explored, the results of the CPTs indicate the site blanketed by about 9 feet of fill consisting of medium dense silty sand. A weak and highly compressible marine clay deposit, known locally as Bay Mud, is present beneath the fill. Where explored, the Bay Mud is soft to medium stiff and normally consolidated to lightly overconsolidated, and extends to depths of about 16 to 20 feet bgs. The Bay Mud is underlain by alluvium to the maximum depth explored of about 50 feet bgs. The alluvium consists of interbedded layers of stiff clay and medium dense sand with variable amounts of clay and silt to depths of about 30 to 35 feet bgs. Below depths of 30 and 35 feet bgs, the alluvium consists of very stiff to hard clay and dense to very dense sand with variable amounts of clay and silt.

Pore pressure dissipation tests performed in CPT-1 and CPT-2 indicate groundwater to be at about 7 and 5 feet bgs, respectively. Available historic groundwater information presented in the Seismic Hazard Zone Report for the Oakland East Quadrangle indicates the historic high groundwater at the site is approximately 5 feet bgs at the site. The depth to groundwater is expected to vary several feet annually, depending on rainfall amounts.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity

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 San Gregorio 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 from the site and estimated mean characteristic Moment magnitude[1] [2007 Working Group on California Earthquake Probabilities (WGCEP) (USGS 2008) and Cao et al. (2003)] are summarized in Table 1.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
Total Hayward	4.6	Northeast	7.00
Total Hayward-Rodgers Creek	4.6	Northeast	7.33
Total Calaveras	19	East	7.03
Mount Diablo Thrust	20	Northeast	6.70
N. San Andreas - Peninsula	25	West	7.23
N. San Andreas (1906 event)	25	West	8.05
Green Valley Connected	25	Northeast	6.80
N. San Andreas - North Coast	31	West	7.51
San Gregorio Connected	33	West	7.50
Monte Vista-Shannon	36	South	6.50
Greenville Connected	37	East	7.00
Rodgers Creek	41	Northwest	7.07
Great Valley 5, Pittsburg Kirby Hills	43	Northeast	6.70
West Napa	45	North	6.70

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

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 most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with an Mw of 6.9. This earthquake occurred in the Santa Cruz Mountains about 86 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).

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

5.2 Seismic 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,[2] lateral spreading,[3] and cyclic densification[4]. We used the results of our preliminary field investigation to evaluate the potential of these phenomena occurring at the project site.

5.2.1 Ground Shaking

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

5.2.2 Ground Surface Rupture

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

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

[3] 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.

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

5.2.3 Liquefaction and Associated Hazards

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

The subject property is located in an area of Oakland designated as a potential liquefaction hazard zone on the map prepared by California Geological Survey (CGS) titled *State of California, Seismic Hazard Zone, Oakland East Quadrangle*, dated February 14, 2003 (Figure 5). We evaluated the liquefaction potential of soil encountered below groundwater at the site using data collected in our CPTs. Our liquefaction analyses were performed 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 an assumed high groundwater at 5 feet bgs. In accordance with the 2013 and 2016 CBC, we used a peak ground acceleration of 0.708 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). We also used a moment magnitude 7.33 earthquake, which is consistent with the mean characteristic moment magnitude for the Hayward Fault, as presented in Table 1.

Our liquefaction analyses indicate there are thin layers of potentially liquefiable soil underlying the site. The potentially liquefiable layers are generally less than three feet thick and are discontinuous. The majority of the material identified as potentially liquefiable in the liquefaction analyses has a soil behavior type of “sand”, “silty sand”, and “silty clay” based on interpretations of the CPT data.

Based on the results of our preliminary geotechnical investigation, we estimated total and differential settlements associated with liquefaction at the site during a MCE event generating a PGA_M of 0.708g will be up to 1-1/2 inches and 3/4 inch across a horizontal distance of 30 feet, respectively.

Our analysis indicates the non-liquefiable soil overlying the potentially liquefiable soil layers is sufficiently thick and the potentially liquefiable layers are sufficiently thin such that the potential for surface manifestations from liquefaction, such as sand boils, are low.

Considering the potentially liquefiable layers are not continuous and the potentially liquefiable soil layers have interpreted SPT N-values greater than 15 blows per foot, we conclude the risk of lateral spreading is low.

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 consists of medium dense silty sand and has a low susceptibility to cyclic densification because of its relative density and high fines content. Therefore, we conclude the potential for cyclic densification to occur at the site is low.

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, we conclude the subject properties at 4709, 4715, and 4719 Tidewater Avenue can be developed as planned. The primary geotechnical concerns are:

- the presence of about 9 feet of undocumented fill across the site;
- the presence of 7 to 11 feet of weak, highly compressible Bay Mud that will initiate a new cycle of consolidation if subjected to new loads (i.e. new foundation loads); and
- providing adequate foundation support for the proposed improvements.

These and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

6.1 Foundation and Settlement

Information regarding conceptual building height and construction type was not available when this report was prepared. The factors influencing the selection of a safe, economical foundation system are adequate foundation support, total and differential settlement of the structure resulting from new building loads, and liquefaction-induced ground settlement. The results of our preliminary field investigation indicate the proposed building will be underlain by undocumented fill and Bay Mud that is highly compressible. If a shallow foundation system placed directly on the existing fill is used to support the proposed building, the Bay Mud beneath the site would experience large total and differential consolidation settlement under the new loads. On the basis of our experience, we judge the anticipated differential settlements due to both static load conditions and liquefaction-induced ground settlement exceed the typical tolerance of a conventional spread footing foundation system.

We conclude a stiffened shallow foundation system, such as interconnected continuous spread footings or a mat, would be the most appropriate foundation system for a light-weight building, provided the average bearing pressure will be less than 500 pounds per square foot (psf) for dead-plus-live loads. Interconnected continuous spread footings or mat foundations are capable of minimizing distortion of the superstructure from static and seismically induced differential settlement. Where the average bearing pressure will exceed 500 psf for dead-plus-live loads, we preliminary conclude the building should be supported on spread footings bearing on improved ground. Preliminary recommendations for stiffened shallow foundations and spread footings bearing on improved ground are presented in this section.

6.1.1 Stiffened Shallow Foundations

We preliminarily conclude a light-weight building may be supported on a stiffened shallow foundation system, such as interconnected continuous spread footings or a mat, provided the stiffened shallow foundation is designed to limit estimated differential settlement resulting from consolidation of the highly compressible Bay Mud under static loads. Considering the large area of the stiffened shallow foundation, we expect the average bearing stress under the foundation to be low; however, concentrated stresses will occur at column locations and at the edges of the foundation. The stiffened shallow foundation should be designed to impose an average bearing pressure of less than 500 psf and a maximum bearing pressure of 750 psf on the foundation subgrade soil for dead-plus-live loads. These pressures may be increased by one-third for total load conditions.

We estimate the total and differential settlements of a building supported on a stiffened shallow foundation with an average bearing pressure of 500 psf for dead plus live loads would on the order of 1-3/4 inches and one inch over a horizontal distance of 30 feet, respectively. As discussed in Section 5.2.3, the stiffened shallow foundation should be designed for an additional 1-1/2 inches of total liquefaction-induced settlement and 3/4 inch of differential liquefaction-induced settlement over a horizontal distance of 30 feet.

For stiffened shallow foundation design, we recommend using a modulus of subgrade reaction of 5 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.

Lateral loads can be resisted by a combination of passive pressure on the vertical faces of the foundation and friction along the bottom of the foundation. Passive resistance may be calculated using an equivalent fluid weight of 240 pounds per cubic foot (pcf). The upper one foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance should be computed using a base friction coefficient of 0.35 where the foundation is in contact with soil and 0.20 where the foundation is underlain by a vapor retarder. These values include a factor of safety of at least 1.5 and may be used in combination without reduction.

The stiffened shallow foundation subgrade should be free of standing water, debris, and disturbed materials prior to placing concrete. The subgrade should be wetted following excavation and maintained in a moist condition until it is covered with the vapor retarder or concrete.

6.1.2 Spread Footings on Improved Ground

We preliminarily conclude an appropriate foundation system for a heavier building would consist of spread footings bearing on improved ground. The ground improvement should extend to a depth that would reduce differential settlement of the structure under both static and seismic conditions to a tolerable amount. There are several types of ground improvement that are feasible for these soil conditions. We conclude viable ground improvement systems include soil-cement (SMX) columns or drill displacement sand-cement (DDSC) columns.

DDSC columns are installed by advancing a continuous flight, hollow-stem auger that mostly displaces the soil and then pumping a sand-cement mixture into the hole under pressure as the auger is withdrawn. SMX columns are installed by injecting and blending cement into the soil using a drill rig equipped with single or multiple augers. These systems result in low vibration during installation and generate little to no drilling spoils for off-haul. DDSC and SMX columns are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of columns should be determined by the contractor, based on the desired level of improvement. If soil improvement is to be considered, we recommend a preliminary design, including calculations of static and seismic settlement, be prepared by the ground improvement contractor and submitted for our review.

For preliminary design of spread footings bearing on improved ground, we recommend assuming ground improvement elements will extend about 35 feet bgs. We anticipate the ground improvement systems (DDSC and SMX columns), if properly designed, should be capable of increasing the average allowable bearing pressures to approximately 4,000 psf for dead-plus-live-loads and limiting static differential settlement to less than 1/2 inch and seismically induced differential settlement to less than 1/4 inch over a horizontal distance of 30 feet. The actual design allowable bearing pressure and estimated settlements should be evaluated by the design-build ground improvement contractor, as they will be based on the diameter, depth, and spacing of the ground improvement elements.

Lateral loads can be resisted by a combination of passive pressure on the vertical faces of the foundation and friction along the bottom of the foundation. Passive resistance may be calculated using an equivalent fluid weight of 240 pcf. The upper one foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance for spread footings bearing on ground improved with DDSC or SMX columns should be computed using a base friction coefficient of 0.40. These 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 of the footing excavations should be tamped with a whacker-type compactor to remove disturbance caused by the excavation. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed.

6.2 Seismic Design

As discussed in Section 5.2.3, the site is underlain by thin zones of potentially liquefiable soil. Although the 2013 and 2016 CBC calls for a Site Class F designation for sites underlain by potentially liquefiable soil, we conclude a Site Class D designation is more appropriate because the potentially liquefiable layers are relatively thin and the site will not incur significant nonlinear behavior during strong ground shaking. Therefore, for seismic design we recommend Site Class D be used. The latitude and longitude for the site are 37.7622° and -1212.2197°, respectively. Hence, in accordance with the 2013 or 2016 CBC, we recommend the following:

- S_S = 1.837g, S_1 = 0.736g
- S_{MS} = 1.837, S_{M1} = 1.104g
- S_{DS} = 1.225g, S_{D1} = 0.736g
- Seismic Design Category D for Risk Categories I, II, and III.

7.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to final design, additional borings and/or CPTs should be performed within the proposed building footprint to supplement existing subsurface information and to develop final geotechnical conclusions and recommendations.

8.0 LIMITATIONS

This preliminary 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 preliminary recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the exploratory CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The preliminary foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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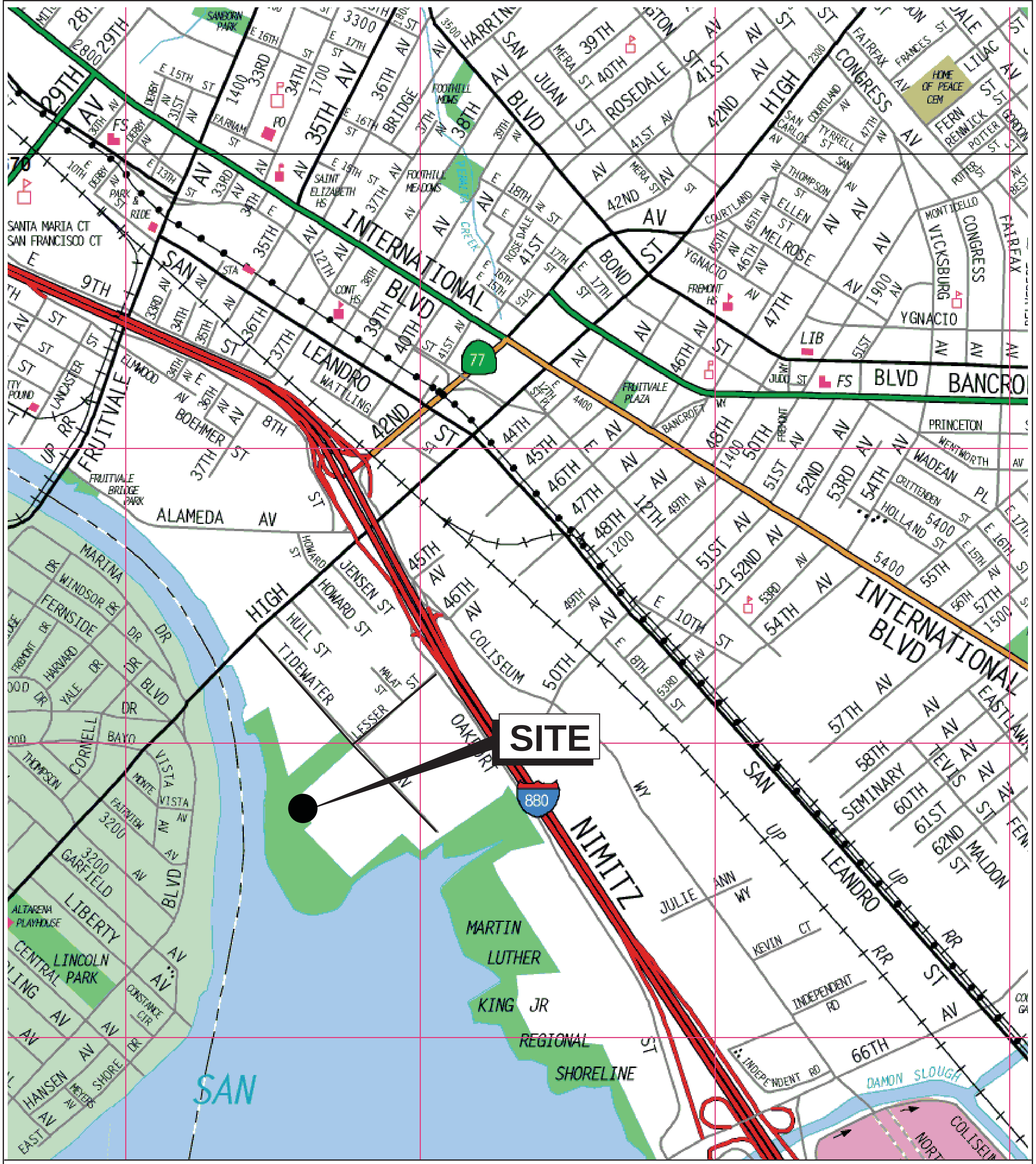
FIGURES



Base map: The Thomas Guide
Alameda County
2002

0 1/4 1/2 Mile

Approximate scale

4709, 4715, AND 4719 TIDEWATER AVENUE
Oakland, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 10/21/16	Project No. 16-1212	Figure 1

EXPLANATION

CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical Inc., October 19, 2016

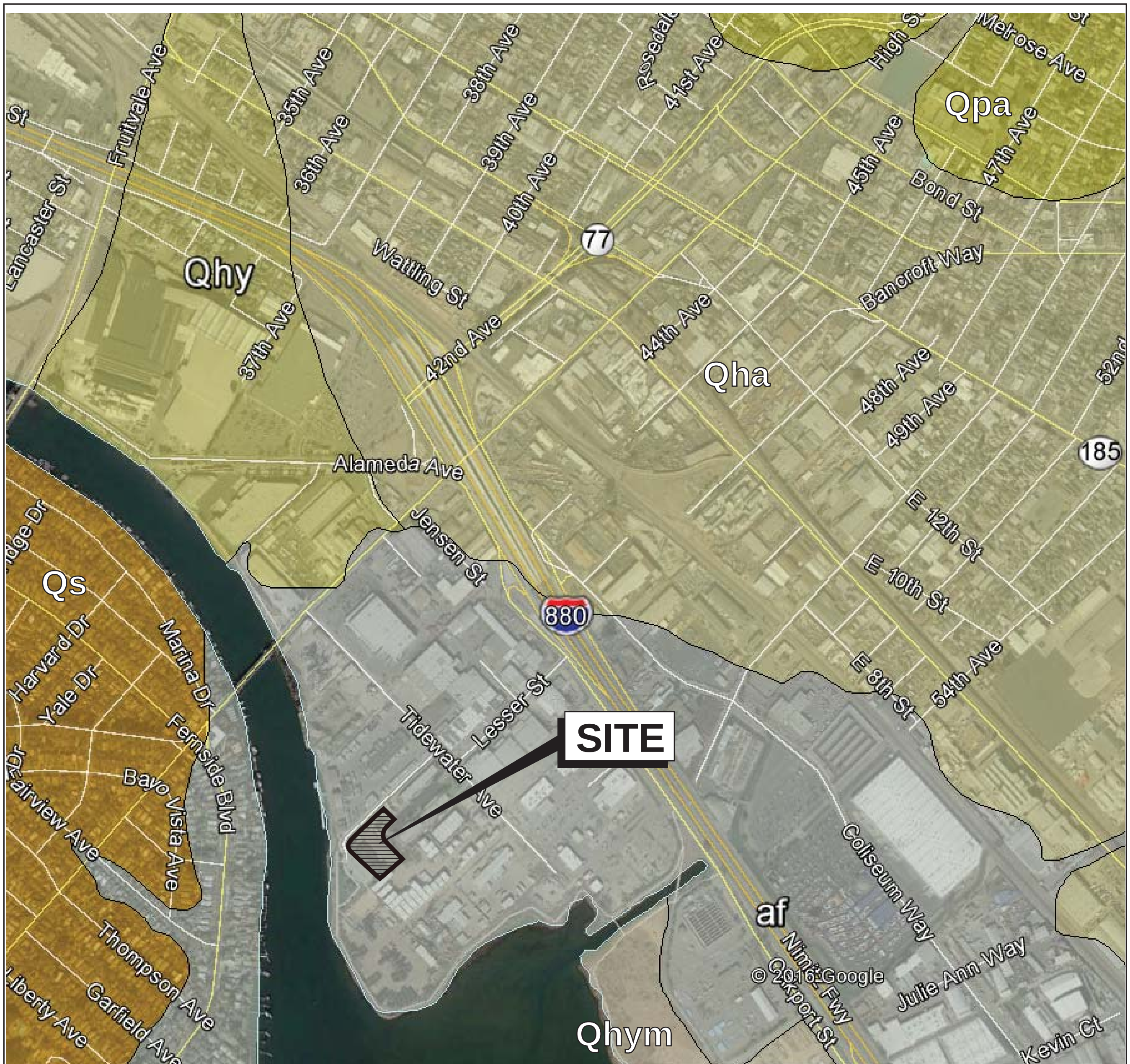
Approximate project limits

Base map: Google Earth Pro, 2016.

0 100 Feet

Approximate scale

4709, 4715, AND 4719 TIDEWATER AVENUE Oakland, California	**SITE PLAN**
ROCKRIDGE GEOTECHNICAL	Date 10/24/16 | Project No. 16-1212 | Figure 2



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

EXPLANATION

af	Artificial Fill
Qhy	Alluvium (late Holocene)
Qha	Alluvium (Holocene)
Qs	Beach and dune sand (Quaternary)
Qhym	Mud deposits (late Holocene)
Qpa	Alluvium (Pleistocene)

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

0 600 1200 Feet

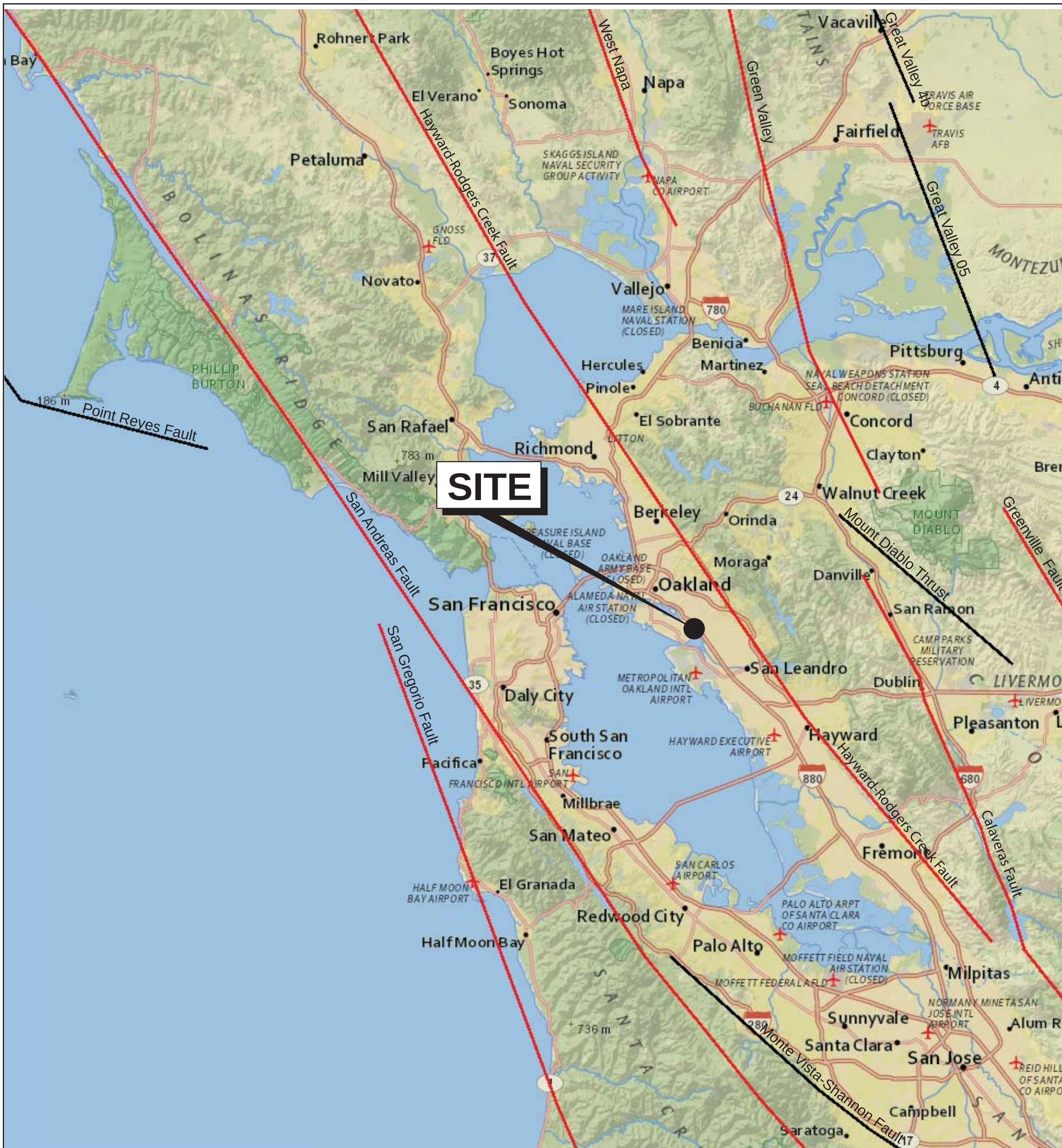
Approximate scale

4709, 4715, AND 4719 TIDEWATER AVENUE
Oakland, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 10/21/16 | Project No. 16-1212 | Figure 3



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

EXPLANATION

Strike slip

Thrust (Reverse)

Normal

0 5 10 Miles

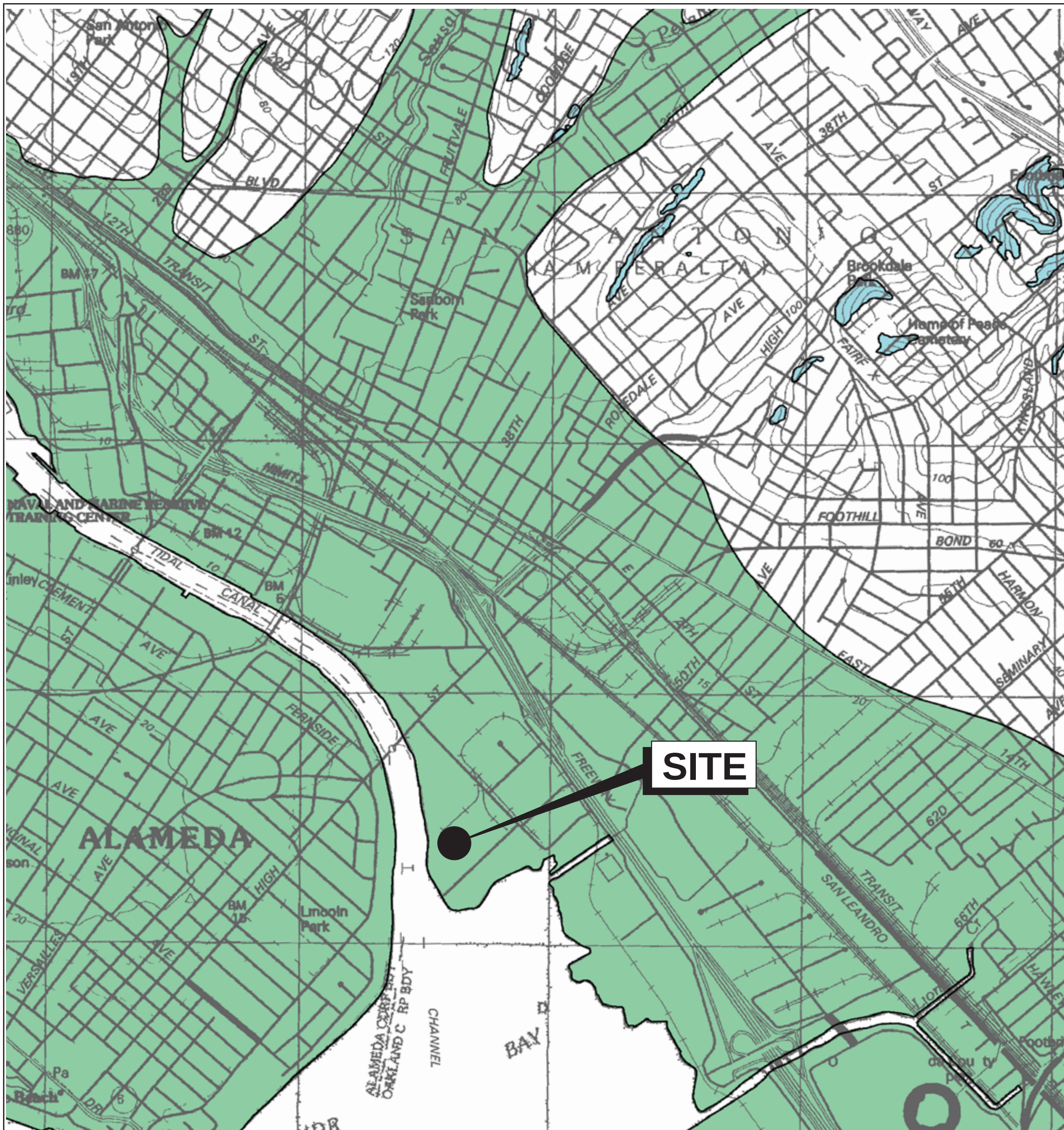
Approximate scale

4709, 4715, AND 4719 TIDEWATER AVENUE
Oakland, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 10/21/16	Project No. 16-1212	Figure 4



EXPLANATION

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

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

0 2,000 4,000 Feet

Approximate scale

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

4709, 4715, AND 4719 TIDEWATER AVENUE
Oakland, California

ROCKRIDGE GEOTECHNICAL

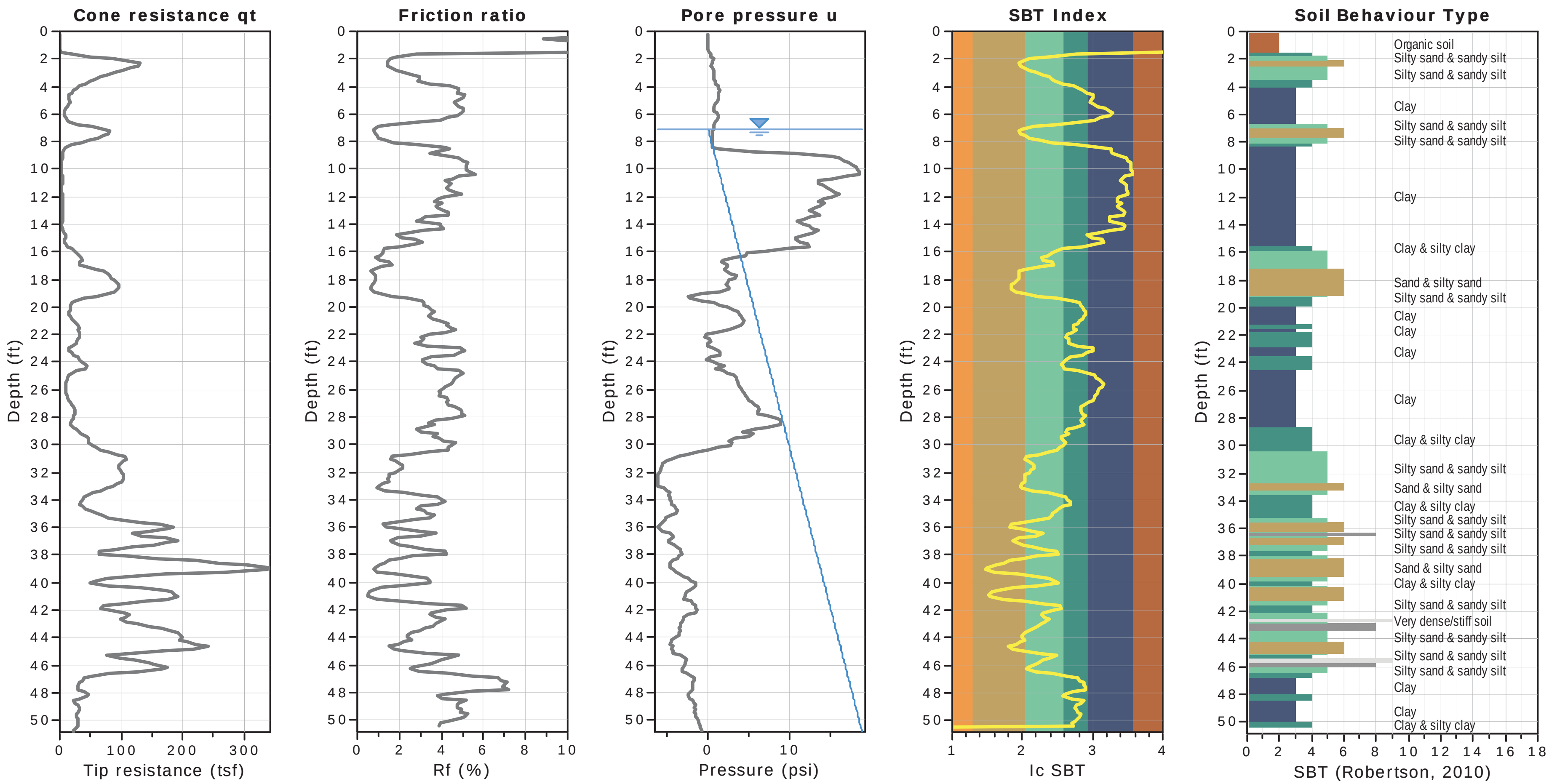
SEISMIC HAZARDS ZONE MAP

Date 10/21/16 | Project No. 16-1212 | Figure 5

ROCKRIDGE GEOTECHNICAL

APPENDIX A

Cone Penetration Test Results



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

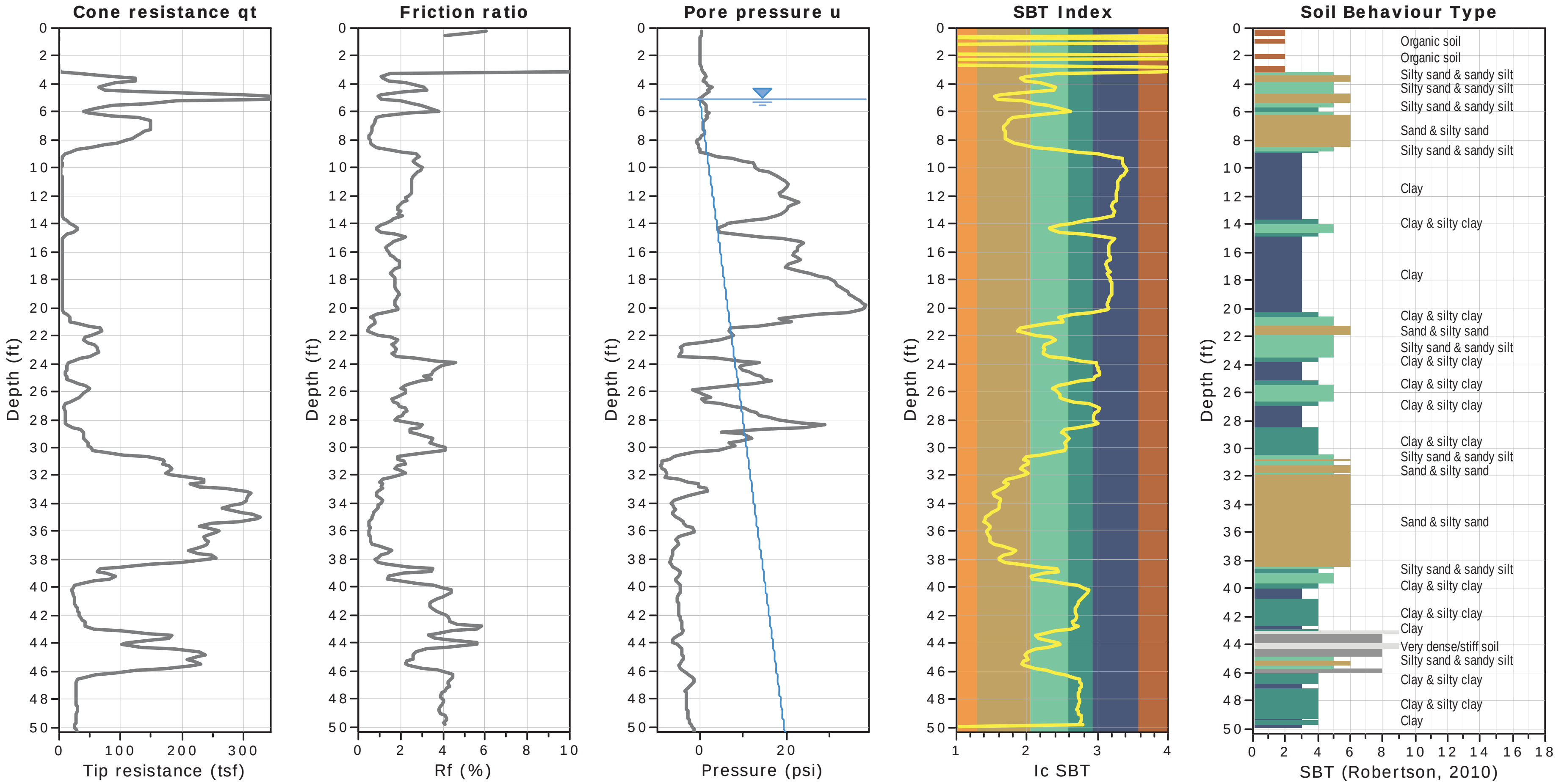
Total depth: 50.85 ft, Date: October 19, 2016
Measured Groundwater Depth: 7.1 feet (pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

4709, 4715, AND 4719 TIDEWATER AVENUE
Oakland, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 10/24/16	Project No. 16-1212	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: 50.36 ft, Date: October 19, 2016
Measured Groundwater Depth: 5.1 feet (pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

4709, 4715, AND 4719 TIDEWATER AVENUE
Oakland, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 10/24/16 | Project No. 16-1212 | Figure A-2