Mr. Jeremy Harris
1919 Crew LLC
Pier 54 Suite 202
San Francisco, CA 94158

Ms. Dilan Roe
Alameda County Health Care Services Agency
Department of Environmental Health
1131 Harbor Bay Parkway, Suite 250
Alameda, CA 94502-6577

Re: 1919 Market Street – Acknowledgement Statement
Oakland, California 94805
ACEH Case# RO0003205
APNs 5-410-13-1, 5-410-14, 5-410-25

Dear Ms. Roe:

1919 Crew LLC has retained the environmental consultant referenced on the attached report for the project referenced above. The attached report is being submitted on behalf of 1919 Crew LLC.

I have read and acknowledge the content, recommendations and/or conclusions contained in the attached document or report submitted on my behalf to ACDEH's FTP server and the State Water Resources Control Board's GeoTracker website.

Sincerely,

Jeremy Harris

ROCKRIDGE GEOTECHNICAL

Prepared for **1919 Crew**

GEOTECHNICAL INVESTIGATION
PROPOSED LIVE/WORK CONVERSION
1919 MARKET STREET
OAKLAND, CALIFORNIA

April 29, 2016
Project No. 16-1090

270 Grand Avenue
Oakland, CA 94610

www.rockridgegeo.com

510 420-5738 tel
510 652-3096 fax

April 29, 2016
Project No. 16-1090

Mr. Marvin Winegar
1919 Crew

Subject: Geotechnical Investigation
Proposed Live/Work Conversion
1919 Market Street
Oakland, California

Dear Mr. Winegar,

The attached report, dated April 29, 2016, presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed improvements to be constructed at 1919 Market Street in Oakland, California. Our services were provided in accordance with our Authorization to Provide Geotechnical Services dated March 24, 2016.

The site is relatively level with maximum plan dimensions of approximately 250 by 290 feet. The majority of the site is occupied by a two-story, warehouse-type building that has about 230 feet of frontage on Market Street and the front of the building and 200 feet of frontage on Myrtle Street at the rear. An asphalt-paved parking lot occupies the southern portion of the site. We understand plans are to seismically strengthen the building and to make other improvements to convert the building to live/work lofts.

On the basis of the results of our geotechnical investigation, we conclude the proposed improvements can be constructed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and properly implemented during construction. The primary geotechnical concerns are the presence of up to four feet of undocumented fill and the potential for 1/2 inch of liquefaction-induced differential settlement following a major earthquake. We conclude the proposed improvements may be supported on spread footings bottomed on well-compacted fill and/or stiff native clay. Alternatively, a mat foundation or drilled, cast-in-place concrete piers may be used to support improvements.

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

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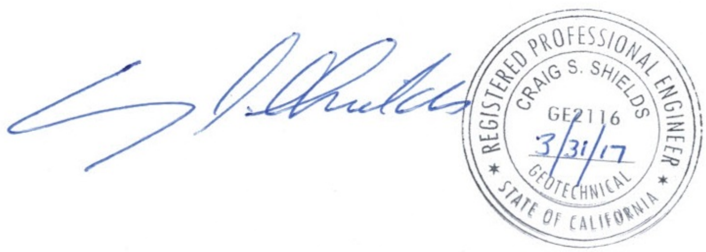
foundation installation during which time we may make changes in our recommendations, if deemed necessary.

We appreciate the opportunity to provide our services to you on this project. If you have any questions, please call.

Sincerely yours,
ROCKRIDGE GEOTECHNICAL, INC.



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

Enclosure

ROCKRIDGE GEOTECHNICAL

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APPENDIX A

ROCKRIDGE GEOTECHNICAL

GEOTECHNICAL INVESTIGATION
PROPOSED LIVE/WORK CONVERSION
1919 MARKET STREET
Oakland, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed improvements to the existing building at 1919 Market Street in Oakland, California. The subject property is on the eastern side of Market Street east of its intersection with 20th Street, as shown on the Site Location Map, Figure 1.

The site is relatively level with maximum plan dimensions of approximately 250 by 290 feet. As shown on Figure 2 (Site Plan), the majority of the site is occupied by a two-story, warehouse-type building that has about 230 feet of frontage on Market Street and the front of the building and 200 feet of frontage on Myrtle Street at the rear. An asphalt-paved parking lot occupies the southern portion of the site. We understand plans are to seismically strengthen the building and to make other improvements to convert the building to live/work lofts.

2.0 SCOPE OF SERVICES

Our scope of our services, which was was outlined in our Authorization to Provide Geotechnical Services, dated March 24, 2016, included reviewing available subsurface information for the site vicinity in our files, obtaining a drilling permit, and performing three cone penetration tests (CPTs) at the site. We used the data acquired from the CPT to perform engineering analyses to develop conclusions and recommendations regarding:

- allowable bearing capacity, friction factor, and passive pressure for existing footings
- the most appropriate foundation type(s) where new foundations are needed
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlement under static and seismic conditions

- subgrade preparation for new concrete slab-on-grade floor areas
- site grading and excavation, including criteria for fill quality and compaction
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- 2013 California Building Code (CBC) site class and design spectral response acceleration parameters.

3.0 FIELD INVESTIGATION

Subsurface conditions at the site were investigated by performing three CPTs. Prior to beginning our field investigation, we obtained a drilling permit from the Alameda County Public Works Agency (ACPWA). We also contacted Underground Service Alert (USA) to notify them of our work.

Middle Earth Geo Testing, Inc. of Orange, California performed the CPTs, designated as CPT-1 through CPT-3, on April 11 and 13, 2016 at the approximate locations shown on the attached Site Plan, Figure 2. 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 tests were 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 advanced to depths ranging from about 30 to 50 feet below the existing ground surface (bgs). CPT-3 met refusal in very dense sand at a depth of about 30 feet 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-3 in Appendix A. Upon completion, the CPT holes were backfilled with cement grout.

4.0 SUBSURFACE CONDITIONS

As shown on the Regional Geologic Map (Figure 3), the site is mapped as being underlain by Holocene- and Pleistocene-age Merritt sand. Based on our interpretation of the CPT data and our experience in the area, we conclude the site is blanketed by about 2 to 4 feet of undocumented fill that was likely placed during the original site development. The fill consists of medium stiff to stiff clay and medium dense to dense sand and silty sand. The fill is underlain by alluvium consisting of interbedded clay and sand layers that extend to the maximum depth explored of 50 feet bgs. Above a depth of about 13 to 14 feet bgs, the alluvium consists primarily of stiff to very stiff clay. This upper clay layer is underlain by an approximately 4- to 12-foot thick layer of dense to very dense Merritt sand. Below the Merritt sand are interbedded sand and clay layers extending to depths ranging from about 30 to 32 feet bgs. Below depths of about 30 to 32 feet bgs, the alluvium consists of very stiff to hard clay that extends to a depth of at least 50 feet bgs. The depth to bedrock is anticipated to be several hundred feet in this area.

The depth to groundwater at the CPT-3 location was estimated by performing a pore pressure dissipation test at a depth of 15.8 feet bgs. The dissipation test indicates the depth to groundwater at the time of our field investigation was 13.6 feet bgs. The groundwater level is expected to fluctuate several feet seasonally, depending on the seasonal rainfall. Based on our review of published data, the historic high groundwater in the site vicinity is estimated to be approximately 10 feet bgs.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity and Faulting

The major active faults in the area are the Hayward, Calaveras, and San Andreas faults. These and other faults in the region are shown on Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated maximum Moment magnitude[1] [Working Group on California Earthquake Probabilities (WGCEP, 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	Maximum Magnitude
Total Hayward	6	East	7.00
Total Hayward-Rodgers Creek	6	East	7.33
Mount Diablo Thrust	22	East	6.70
N. San Andreas - Peninsula	23	West	7.23
N. San Andreas (1906 event)	23	West	8.05
Total Calaveras	24	East	7.03
N. San Andreas - North Coast	26	West	7.51
Green Valley Connected	27	East	6.80
San Gregorio Connected	30	West	7.50
Rodgers Creek	33	Northwest	7.07
West Napa	39	North	6.70
Greenville Connected	40	East	7.00
Monte Vista-Shannon	42	South	6.50
Great Valley 5, Pittsburg Kirby Hills	44	East	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, 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 most recent earthquake to affect the Bay Area was the Loma Prieta Earthquake of 17 October 1989 with an M_w of 6.9. This earthquake occurred in the Santa Cruz Mountains about 93 kilometers southwest of the site.

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

The U.S. Geological Survey's 2007 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 Bay Region during the next thirty years is 63 percent. The highest probabilities are assigned to the Hayward/Rodgers Creek Fault and the northern segment of the San Andreas Fault. These probabilities are 31 and 21 percent, respectively (USGS, 2008).

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,[2] lateral spreading,[3] and cyclic densification[4]. We used the results of the CPT 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 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.

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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.

As shown on Figure 5, the site has been mapped within a zone of liquefaction potential on the map titled *State of California Seismic Hazard Zones, Oakland West Quadrangle, Official Map*, prepared by the California Geological Survey (CGS), dated February 14, 2003. Special Publication 117 by the California Geological Survey (2008) recommends subsurface investigation in mapped liquefaction potential areas be performed using rotary-wash borings and/or cone penetration tests.

We evaluated the liquefaction potential of soil encountered below groundwater at the site using data collected in our CPT. Our analyses were performed using an in-situ groundwater depth of 13.6 feet and "during earthquake" groundwater depth of 10 feet. In accordance with the 2013 CBC, we used a peak ground acceleration of 0.65 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 were performed using the following methodologies:

- The publication titled *Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*, prepared by the National Center for Earthquake Engineering Research (NCEER), dated December 31, 1997.

- P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009.

Although both of the above liquefaction methodologies were used, the Robertson (2009) approach resulted in a greater liquefaction hazard than that of the NCEER (1997) approach. Therefore, the post-liquefaction settlement estimates are based on the Robertson (2009) methodology. In both cases, we 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).

The results of our liquefaction analysis indicate there are several thin layers of potentially liquefiable sand and silty sand underlying the site, which is typical for much of the Oakland flatlands. In addition, the analysis using the Robertson (2009) methodology indicates there are also zones of "transitional" soils that may experience pore pressure build-up during a major earthquake and, therefore, contribute to the post-liquefaction reconsolidation settlement. On the basis of our analysis, we estimate the containers may settle up to 1-1/4 inches following a major earthquake due to post-liquefaction reconsolidation. We estimate differential settlement following an earthquake would be on the order 1/2 inch over a horizontal distance of 30 feet. Considering the liquefiable layers are relatively thin and blanketed by at least 10 feet of primarily cohesive soil, we conclude the potential for formation of sand boils and bearing capacity failure of shallow foundations is low.

The area surrounding the site is essentially level and the potentially liquefiable layers are thin and discontinuous. Consequently, we conclude the potential for lateral spreading during seismic events 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 CPT data, we conclude the potential for cyclic densification of the soil above the groundwater table is low due to its cohesion or relative density.

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 CONCLUSIONS AND RECOMMENDATIONS

Our conclusions and recommendations regarding foundation support, slab-on-grade floors, and seismic design are presented in the following sections.

6.1 Foundations

We conclude the proposed improvements may be supported on spread footings bottomed on well-compacted fill and/or stiff native soil. Where overturning resistance is required for seismic frames, a mat foundation or drilled cast-in-place concrete piers may be used. Recommendations for spread footings, mat foundations, and drilled piers are presented in the following sections.

6.1.1 Footings

We conclude the proposed improvements may be supported on conventional spread footings bottomed on well-compacted fill and/or stiff native soil. Continuous and isolated spread footings should be at least 18 inches wide and should be bottomed at least 24 inches below the top of the existing floor slab. For design of footings, we recommend using an allowable bearing pressure of 3,000 pounds per square foot (psf) for dead-plus-live-load conditions. This value may be increased by one-third for total load conditions. We estimate total settlement of the footings under static loading will be less than 1/2 inch and differential settlement will be less than 1/4 inch over a horizontal distance of 30 feet. This settlement is expected to occur immediately as the loads are applied. As discussed above in Section 5.2.2, an additional 1-1/4 inches of total settlement and 1/2 inch of differential settlement over a horizontal distance of 30 feet may occur due to post-liquefaction reconsolidation after a major earthquake.

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 an equivalent fluid weight of 250 pounds per cubic foot (pcf). The upper one foot of soil should be ignored in computing passive resistance unless the soil is confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.3. 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.

The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. We should check footing excavations prior to placement of reinforcing steel. If poorly compacted fill is encountered at the bottom of the footing excavation, the fill should be compacted in place if it is less than 12 inches thick. If the poorly compacted fill is thicker than 12 inches, the fill should be removed and replaced with either properly compacted fill or controlled low-strength material (CLSM) with a 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).

6.1.2 Mat Foundations

A mat foundations may be used in lieu of spread footings. Mat foundations should be bottomed on either stiff native soil and/or well-compacted fill. The design parameters provided above for spread footings may be used for design of mat foundations. In addition, the total settlement estimates provided above for spread footings also apply to mat foundations, although the differential settlement for a mat foundation will be on the order of half the estimated differential settlement for spread footings.

Since up to four feet of existing undocumented fill may be present inside the building, we should check the mat foundation subgrade prior to placement of reinforcing steel to evaluate whether it is necessary to overexcavate and recompact any existing fill that may be present.

6.1.3 Drilled Piers

Where additional overturning resistance is needed for seismic frames, drilled, cast-in-place concrete piers may be used. Drilled piers should be at least 18 inches in diameter and spaced at least three pier diameters, center to center. To compute compression and tension capacities for drilled piers, we recommend using an allowable skin friction value of 600 psf. This value includes a factor of safety of 2.0 and may be increased by one-third for total load conditions. Support from end bearing should be ignored. We estimate total settlement of drilled piers will be less than 1/2 inch and differential settlement will be less than 1/2 inch over a horizontal distance of 30 feet.

Lateral loads on piers can be resisted by a combination of the bending resistance of the drilled piers and passive soil pressure acting against the vertical faces of the grade beams and the upper portion of the piers. Passive resistance for grade beams should be computed using an equivalent fluid weight of 250 pounds per cubic foot (pcf) on level ground. The upper foot of soil should be ignored unless it is confined by a slab or pavement.

To compute the lateral capacities of the drilled piers, we recommend using a passive pressure of 250 pcf acting over a width of three pier diameters. Passive pressure should not be used for lateral resistance below a depth of 10 feet (measured below the ground surface). Below this depth, excessive deflections of the pier head would be required to mobilize the passive pressure. Moment profiles for the piers can be developed upon request once the diameter and length of the piers are known.

The bottoms of the pier holes should be free of debris and water before placement of concrete. We do not anticipate groundwater will be encountered during pier drilling. If groundwater is encountered, however, the pier hole should be pumped dry prior to placement of concrete. If the hole cannot be pumped dry prior to placement of concrete, then the concrete should be placed by tremie methods. Concrete should be placed in the drilled pier holes within 48 hours of drilling if no groundwater is encountered. If groundwater is encountered in the pier holes, concrete should be placed on the same day the holes are drilled.

6.2 Fill Quality and Compaction

In areas that will receive fill or a new slab-on-grade floor, the soil subgrade exposed should be scarified to a depth of at least eight inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction[5]. The soil subgrade should be kept moist until it is covered by fill.

Material excavated at the site will primarily consist of clay and sand that may be reused as fill or backfill. If imported fill (select fill) is required, it should be free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the Geotechnical Engineer. Samples of proposed select fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed imported material.

Fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. Fill consisting of clean sand or gravel (defined as soil with less than 10 percent fines by weight)

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5 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-09 laboratory compaction procedure.

should be compacted to at least 95 percent relative compaction. Fill placed within six inches of soil subgrade for pavement (concrete or asphalt concrete) that will be subjected to vehicular traffic should be compacted to at least 95 percent relative compaction and be non-yielding.

6.3 Concrete Slab-on-Grade Floors

Concrete slab-on-grade floors may be supported on the on-site soil provided the subgrade is compacted in accordance with the recommendations presented above in Section 6.2. If water vapor moving through the floor slab is considered detrimental, we recommend installing a capillary moisture break and water vapor retarder beneath the slab. A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock. The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

If required by the Structural Engineer, the vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The sand overlying the vapor retarder should be moist at the time concrete is placed. However, excess water trapped in the sand could eventually be transmitted as vapor through the slab. Therefore, if rain is forecast prior to pouring the slab, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced. The particle size of the capillary break material and sand (if used) should meet the gradation requirements presented in Table 2.

TABLE 2
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
Gravel or Crushed Rock	
1 inch	90 – 100
¾ inch	30 – 100
½ inch	5 – 25
3/8 inch	0 – 6
Sand	
No. 4	100
No. 200	0 – 5

Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the floor slab should have a low w/c ratio - less than 0.50. If 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.

6.4 Seismic Design

As discussed in Section 5.2.2, the site is underlain by potentially liquefiable soil layers; however, the potentially liquefiable soil layers are thin and, therefore, we do not expect significant non-linear soil behavior to occur. Consequently, we conclude a Site Class D (stiff soil) can be used for design of the proposed improvements. The latitude and longitude of the site are 37.8125° and -122.2794°, respectively. Hence, in accordance with the 2013 CBC, we recommend the following:

- S_S = 1.685 g, S_1 = 0.665 g
- S_{MS} = 1.685 g, S_{M1} = 0.997 g

- S_{DS} = 1.123 g, S_{D1} = 0.665 g
- Seismic Design Category D for Risk Categories I, II, and III.

7.0 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 observe foundation excavations prior to placement of reinforcing steel. These observations will allow us to compare actual with anticipated soil conditions and to verify that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

8.0 LIMITATIONS

This geotechnical study 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 CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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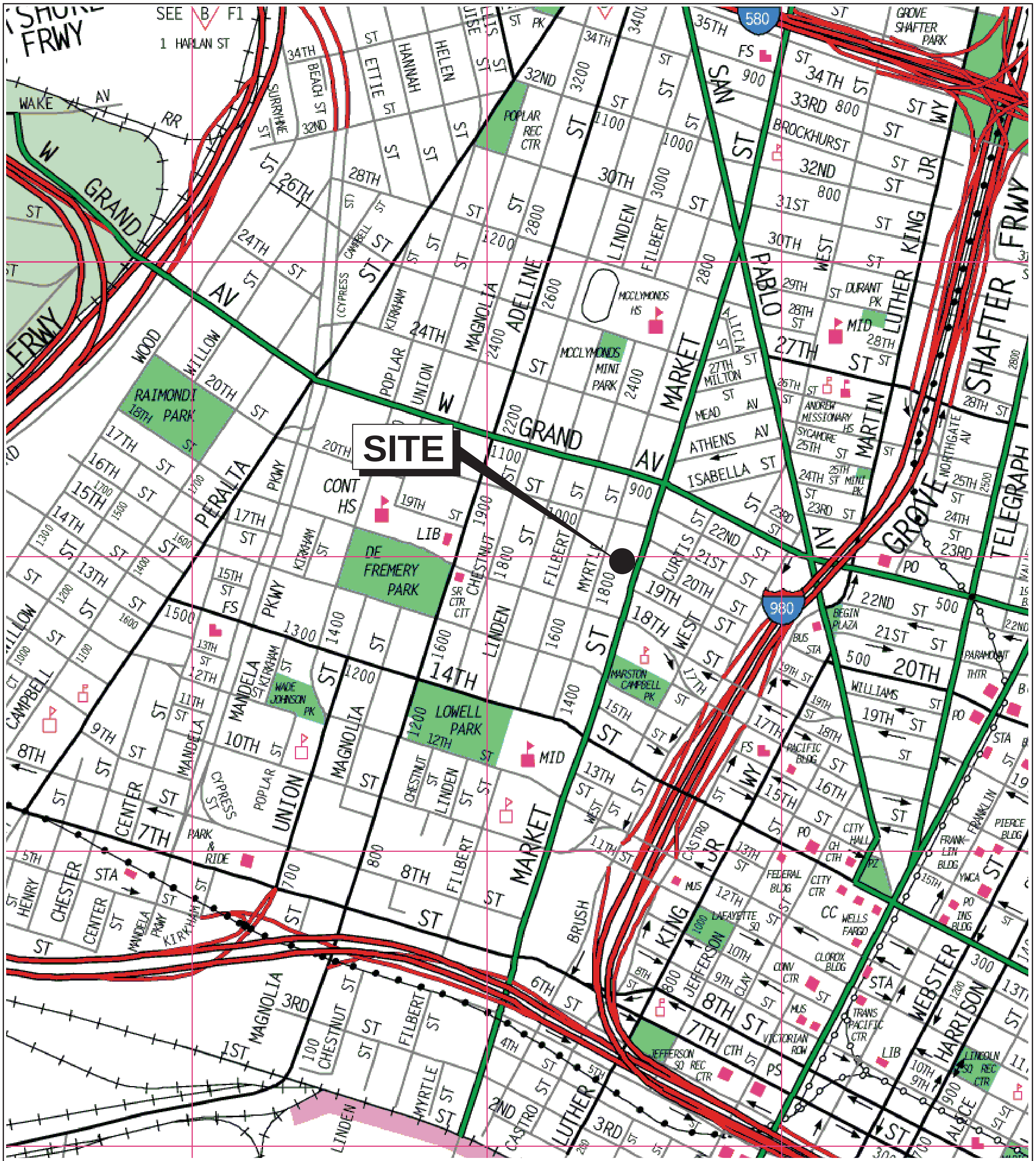
FIGURES



Base map: The Thomas Guide
Alameda County
2002

0 1/4 1/2 Mile

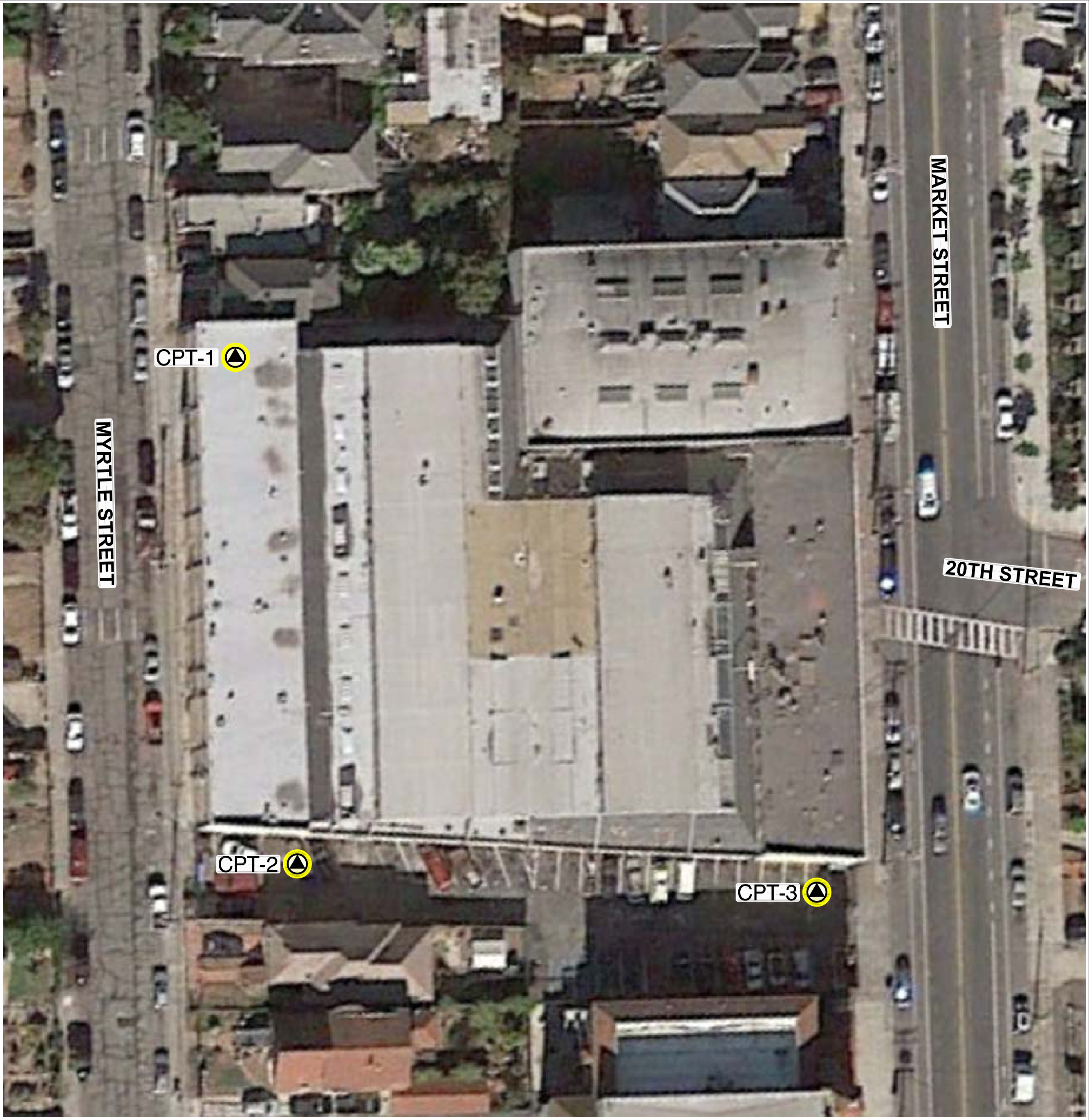
Approximate scale

1919 MARKET STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 04/27/16	Project No. 16-1090	Figure 1



Base map: Google Earth Pro, 2016.

EXPLANATION

CPT-1 — Approximate location of cone penetration test (CPT) by Rockridge Geotechnical, Inc., April 11 and 13, 2016

0 — 60 Feet

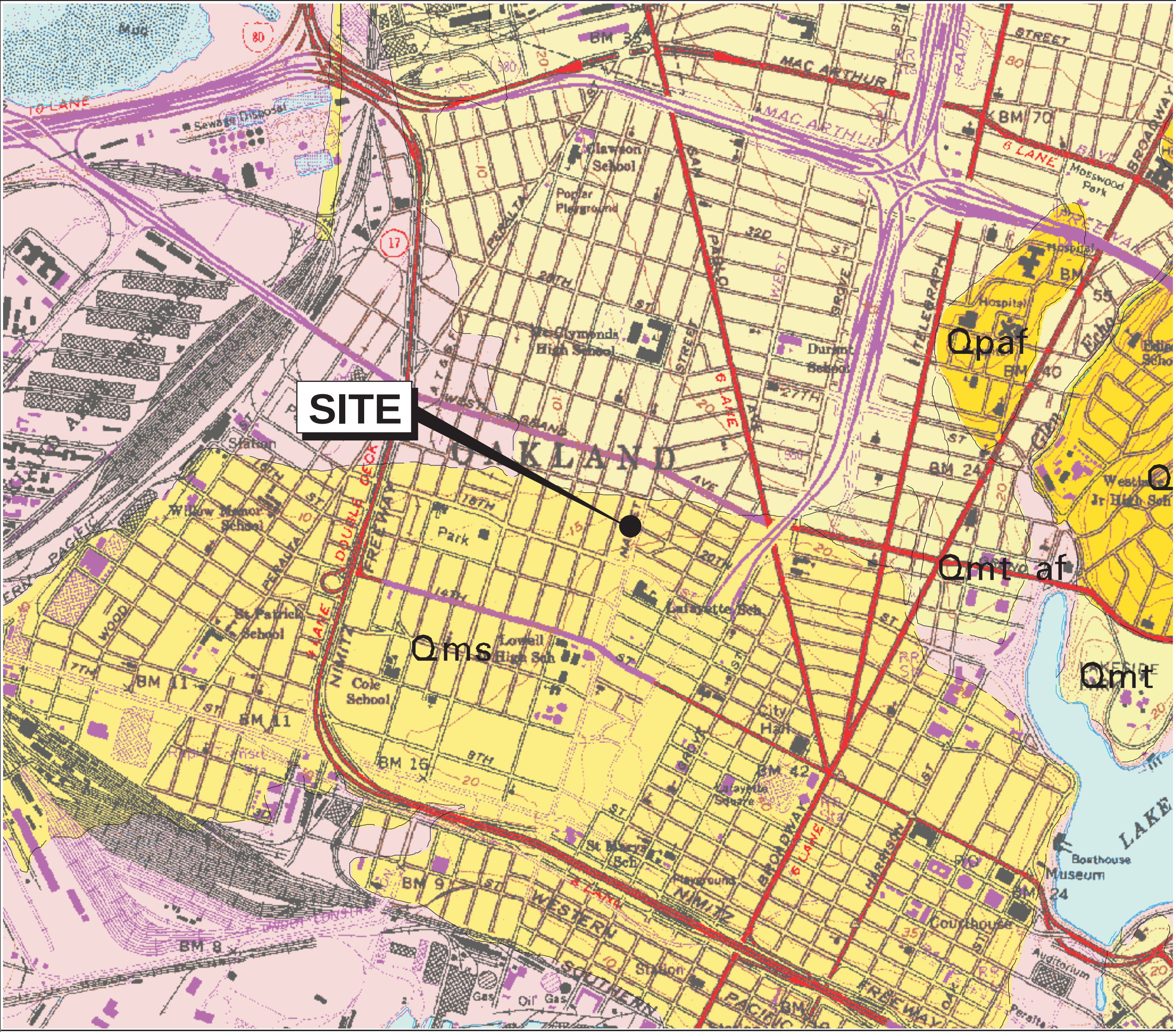
Approximate scale

1919 MARKET STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

SITE PLAN

Date 04/27/16	Project No. 16-1090	Figure 2



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**
- **Strike and dip of bedding** (35)
- **Overturned bedding**
- **Flat bedding**
- **Vertical bedding**
- **Strike and dip of foliation** (35)
- **Vertical foliation**
- **Strike and dip of joints in plutonic rocks** (35)
- **Vertical joint**

af	**Artificial fill (Historic)**
Qms	**Merrit sand (Holocene and Pleistocene)**
Qpaf	**Alluvial fan and fluvial deposits (Pleistocene)**
Qmt	**Marine terrace deposits (Pleistocene)**

0 2,000 4,000 Feet

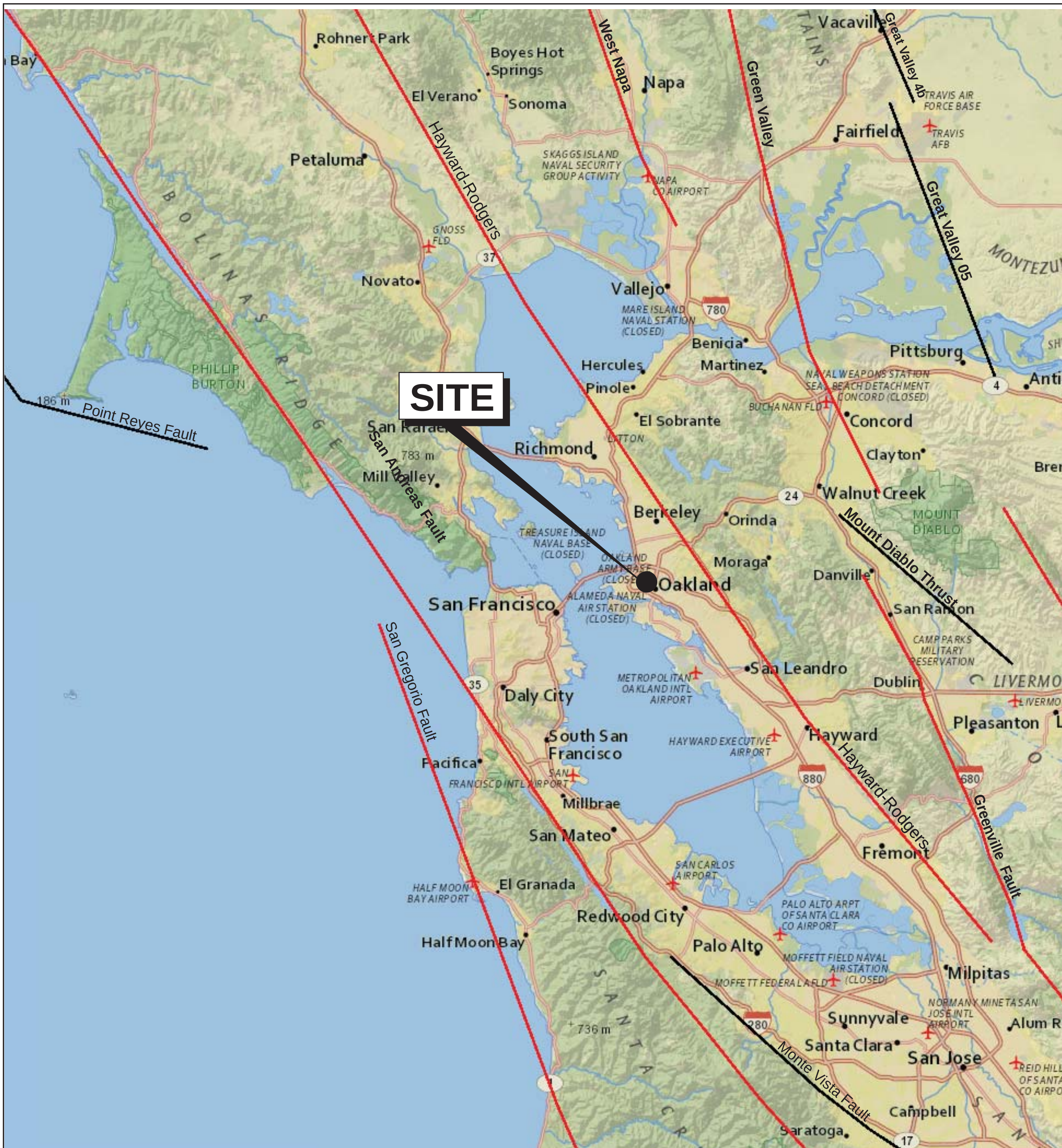
Approximate scale

1919 MARKET STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 04/27/16 | Project No. 16-1090 | Figure 3



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

EXPLANATION

Strike slip

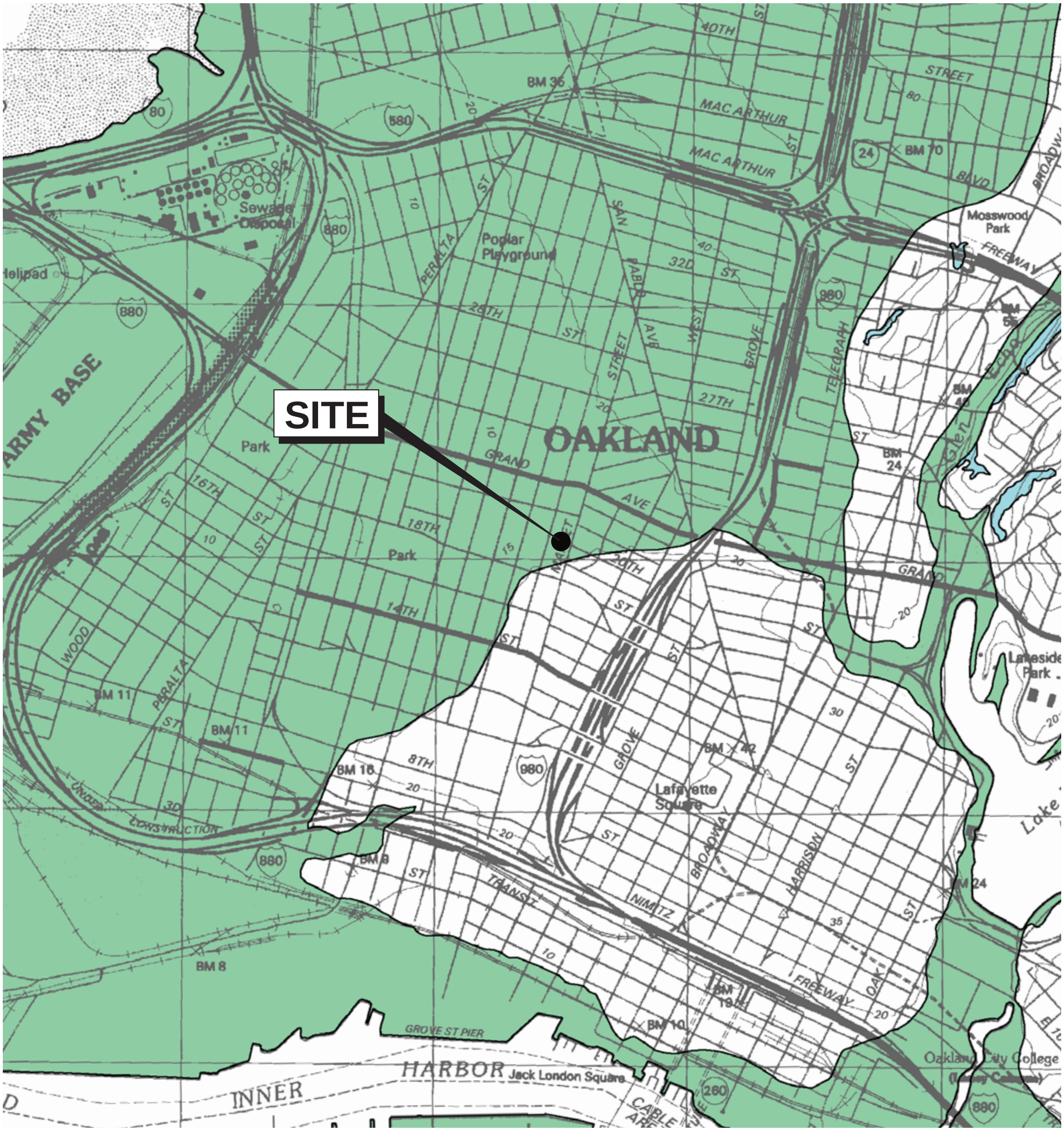
Thrust (Reverse)

Normal

0 5 10 Miles

Approximate scale

1919 MARKET STREET Oakland, California	**REGIONAL FAULT MAP**		
ROCKRIDGE GEOTECHNICAL	Date 04/27/16	Project No. 16-1090	Figure 4



EXPLANATION

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

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

0 2,000 4,000 Feet

Approximate scale

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

1919 MARKET STREET
Oakland, California

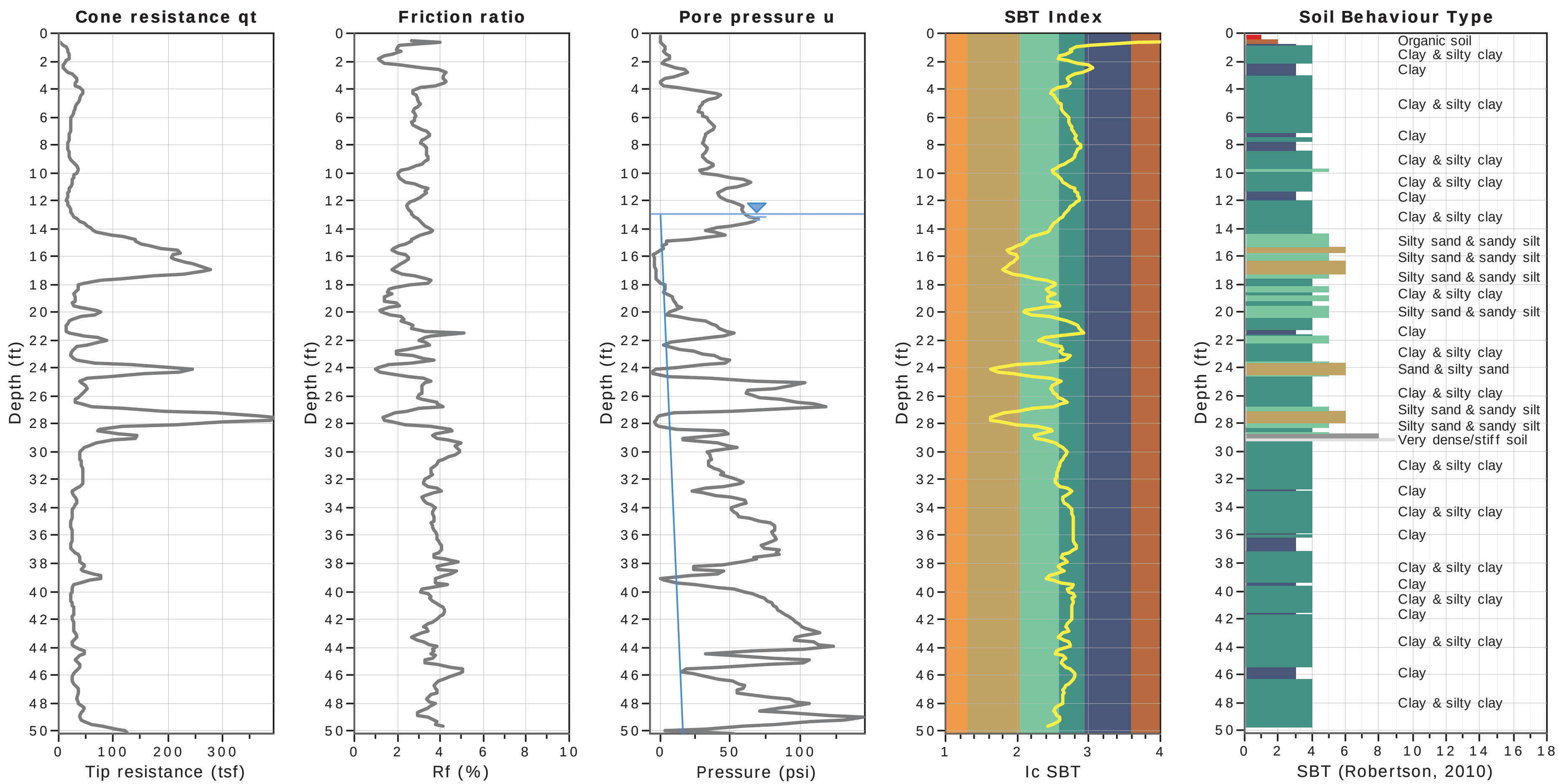
ROCKRIDGE GEOTECHNICAL

SEISMIC HAZARDS ZONE MAP

Date 04/27/16 | Project No. 16-1090 | 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

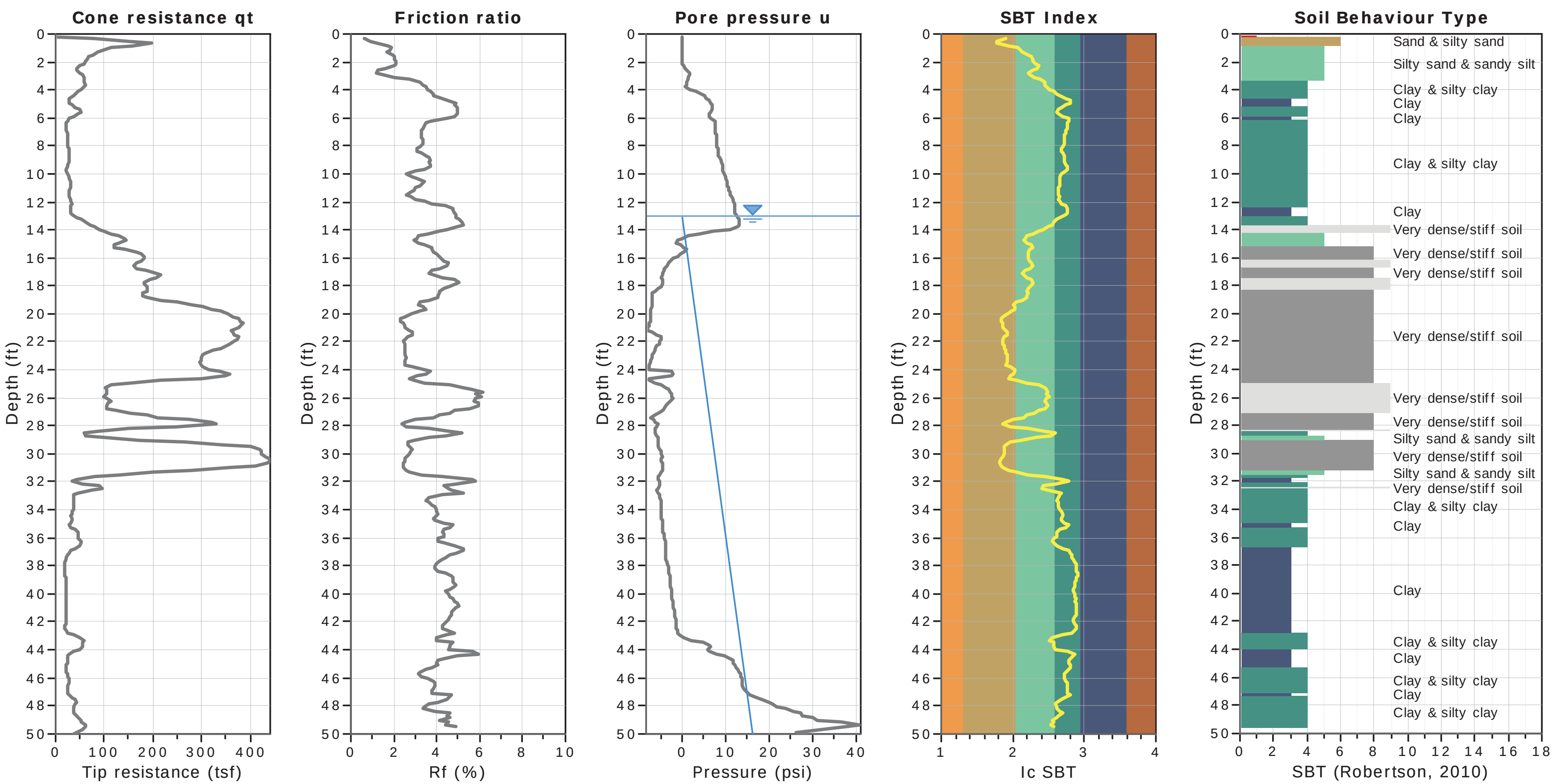
Total depth: 50.20 ft, Date: 4/13/2016
Assued Groundwater Depth: 13 feet
Cone Operator: Middle Earth Geo Testing, Inc.

1919 MARKET STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 04/27/16	Project No. 16-1090	Figure A-1

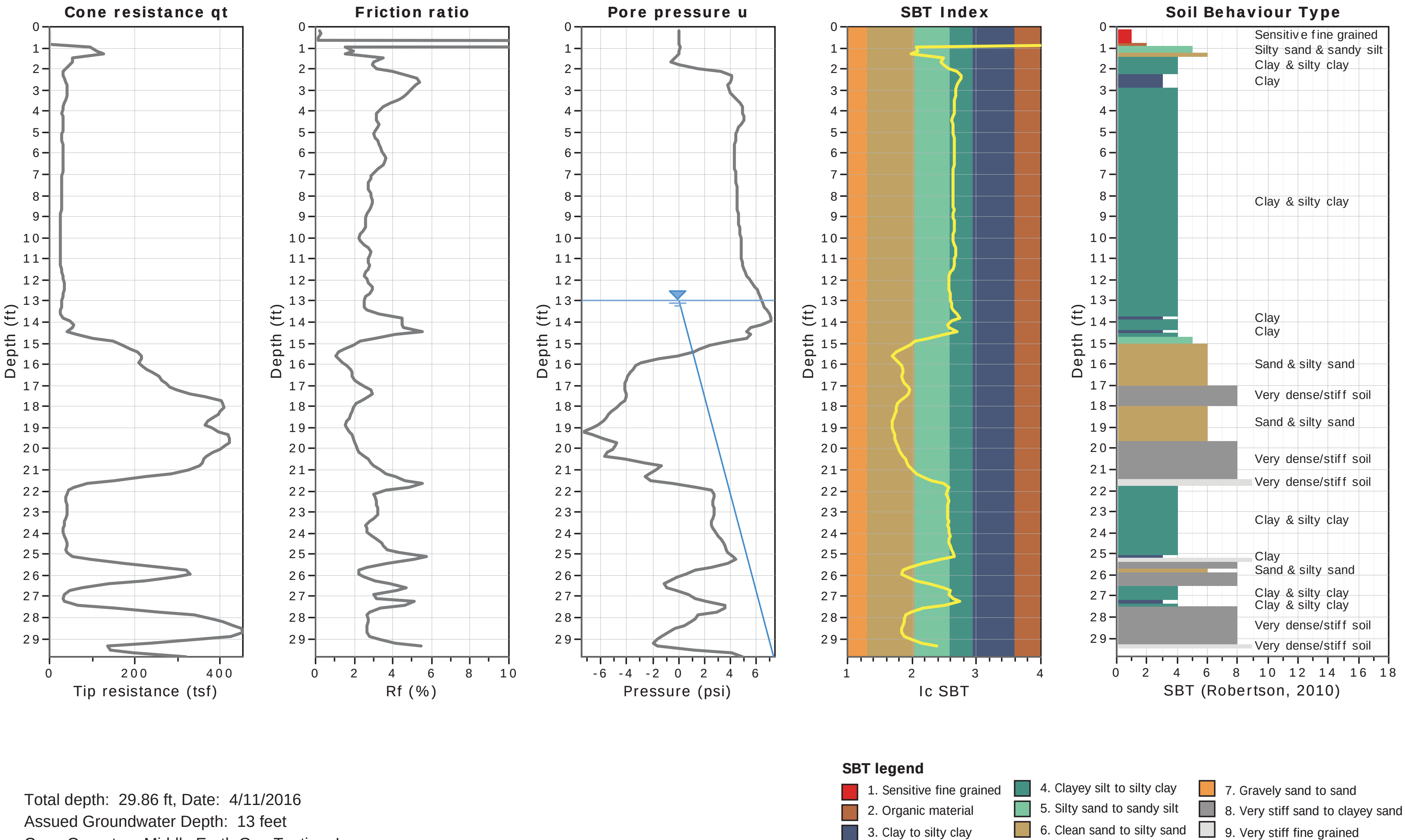


SBT legend

- 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: 50.03 ft, Date: 4/11/2016
Assued Groundwater Depth: 13 feet
Cone Operator: Middle Earth Geo Testing, Inc.

1919 MARKET STREET Oakland, California	CONE PENETRATION TEST RESULTS CPT-2
ROCKRIDGE GEOTECHNICAL	Date 04/27/16 | Project No. 16-1090 | Figure A-2



Total depth: 29.86 ft, Date: 4/11/2016
Assued Groundwater Depth: 13 feet
Cone Operator: Middle Earth Geo Testing, Inc.

1919 MARKET STREET Oakland, California	**CONE PENETRATION TEST RESULTS** **CPT-3**
ROCKRIDGE GEOTECHNICAL	Date 04/27/16 | Project No. 16-1090 | Figure A-3