Prepared for **Square Foot Ventures, LLC**

PRELIMINARY GEOTECHNICAL INVESTIGATION
FOR DUE DILLIGENCE EVALUATION
585 22ND STREET
OAKLAND, CALIFORNIA

July 30, 2015
Project No. 15-927

TABLE OF CONTENTS

ROCKRIDGE GEOTECHNICAL

PRELIMINARY GEOTECHNICAL INVESTIGATION
DUE DILIGENCE EVALUATION
585 22ND STREET
Oakland, California

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Rockridge Geotechnical to support the due diligence evaluation of the property located at 585 22nd Street in Oakland, California. The subject property is located approximately 500 feet west of the intersection of 22nd Street and Telegraph Avenue and spans between 22nd Street and 21st Street, as shown on the Site Location Map (Figure 1).

The property has rough plan dimensions of approximately 100 feet by 200 feet and is currently occupied by an asphalt-paved parking lot that is used for parking US Postal Service (USPS) trucks. We understand the proposed site development tentatively being considered for the site consists of an apartment building that will occupy most of the site. The building will likely consist of four stories of wood-framed construction over a single, at-grade concrete podium that will house parking. The parking level may contain car stacker pits that will extend below grade.

Structural design loads were not available at the time this report was prepared. However, based on our experience with similar structures, we estimate the proposed building will have maximum dead-plus-live interior column loads on the order of 400 kips.

2.0 SCOPE OF SERVICES

Our preliminary investigation was performed in accordance with our proposal dated June 17, 2015. Our scope of work consisted of evaluating subsurface conditions at the site by reviewing published geologic maps and previous geotechnical reports in the site vicinity, performing three cone penetration tests (CPTs), and performing engineering analyses to develop preliminary conclusions and recommendations regarding:

- the most appropriate foundation type(s) for the proposed structure
- preliminary design criteria for the recommended foundation type(s)
- estimates of foundation settlement
- 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
- construction considerations.

3.0 DATA REVIEW

We reviewed the results of a previous geotechnical investigations performed within the site vicinity. In 2003, Treadwell & Rollo (T&R) performed a geotechnical investigation for the Thomas L. Berkeley Square located at the southeast corner of 20th Street and San Pablo Avenue. T&R prepared a report dated March 8, 2004, titled *Geotechnical Investigation, Proposed Commercial Development, Thomas L. Berkeley Square, Oakland, California*. T&R's investigation included drilling three test borings each to a depth of 50 feet below the ground surface (bgs), advancing four CPTs to refusal at depths ranging from 18 to 19-1/2 feet bgs, performing two dynamic penetrometer tests (DPTs), and performing laboratory tests on select soil samples. The results of the borings, CPTs, DPTs, and laboratory tests were presented in T&R's report.

4.0 FIELD INVESTIGATION

Prior to performing our field investigation, we obtained a drilling permit from the Alameda County Public Works Agency (ACPWA) and contacted Underground Service Alert (USA) to notify them of our work, as required by law. We also retained Precision Locating, LLC, a private utility locator, to check that the CPT locations were clear of existing utilities.

Subsurface conditions at the site were investigated on July 9, 2015, by performing three CPTs, designated as CPT-1 through CPT-3, at the approximate locations shown on Figure 2. The CPTs were advanced to refusal at depths ranging from 14 to 15-1/2 feet bgs by Middle Earth Geo

Testing, Inc. of Fremont, California. 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,approximate strength characteristics, and liquefaction potential of the soil encountered. The CPT logs showing tip resistance and friction ratio, as well as interpreted soil behavior type, are presented in Appendix A on Figures A-1 through A-3.

5.0 SUBSURFACE CONDITIONS

A regional geologic map of the site and vicinity (Figure 3) indicates the site is underlain by beach and dune sand deposits of the Merritt Formation (Qs). The results of our CPTs and the previous borings performed within the site vicinity indicate the site is likely blanketed by 2 to 3 feet of undocumented fill generally consisting of soft to stiff clay with varying sand content and loose to dense sand with varying clay and gravel content. Due to the nature of CPT data, the material cannot be definitively classified as fill, however, this will need to be further evaluated using exploratory borings during the final geotechnical investigation. The fill is underlain by a 5- to 10-foot-thick layer of clay with varying sand content. The clay deposit generally grades stiffer with depth. The upper 2 to 3-1/2 feet of the clay deposit is medium stiff to stiff and the lower 2 to 6 feet is very stiff to hard. The clay is underlain by dense to very dense sand with varying fines content to the maximum depth explored of 15 feet bgs. The top of this dense sand layer, whichwe interpret to be Merritt Formation was encountered at depths of about 9-1/2 to 11 feet bgs.

5.1 Groundwater

Groundwater was measured at depths ranging from 11 to 13 feet bgs in our CPTs. Groundwater was encountered in the borings previously performed by T&R in the site vicinity at depths ranging from 15 to 17 feet bgs. The groundwater level at the site is expected to fluctuate several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall.

To estimate the highest potential groundwater level at the site, we reviewed information on the State of California Water Resources Control Board GeoTracker website (http://geotracker.swrcb.ca.gov). The two closest sites with historic groundwater data on the GeoTracker website are at 2225 Telegraph Avenue and 2103 San Pablo Avenue. The data from these two sites indicate the groundwater table slopes down gently to the west. Between July 1992 and March 2015, groundwater was measured in multiple monitoring wells at each of the two locations. The highest (i.e., shallowest) groundwater levels were measured at approximately Elevation 9.5 and 13.1 feet[1] at the 2103 San Pablo Avenue and the 2225 Telegraph Avenue sites, respectively. Using the high groundwater levels and distance from the two locations to the project site, we performed a linear interpolation to estimate the high groundwater level at the project site to be approximately Elevation 12 feet. Assuming the average existing ground surface elevation at the project site to be approximately 22 feet, this groundwater level corresponds to a depth of about 10 feet below existing grade. A topographic survey of the site should be performed to confirm the ground surface elevation and the groundwater elevation should be further evaluated during the final investigation.

6.0 SEISMIC CONSIDERATIONS

6.1 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, Calaveras, and San Andreas Faults. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and mean characteristic Moment magnitude[2] [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	Mean Characteristic Moment Magnitude
Total Hayward	5.3	East	7.00
Total Hayward-Rodgers Creek	5.3	East	7.33
Mount Diablo Thrust	22	East	6.70
Total Calaveras	23	East	7.03
N. San Andreas - Peninsula	24	West	7.23
N. San Andreas (1906 event)	24	West	8.05
Green Valley Connected	27	East	6.80
N. San Andreas - North Coast	27	West	7.51
San Gregorio Connected	30	West	7.50
Rodgers Creek	34	Northwest	7.07
Greenville Connected	39	East	7.00
West Napa	39	North	6.70
Monte Vista-Shannon	42	South	6.50
Great Valley 5, Pittsburg Kirby Hills	44	East	6.70

1 Elevations in this report reference the North American Vertical Datum of 1988 (NAVD88).
2 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 30 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).

6.2 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction, lateral spreading, and cyclic densification. We used the results of the CPTs to evaluate the potential of these phenomena occurring at the project site. The results of our analyses and evaluation are presented in the following sections.

6.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the Hayward and San Andreas faults, 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.

6.2.2 Liquefaction and Associated Hazards

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

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 P.K. Robertson (2009). 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 depth of 10 feet bgs. In accordance with the 2013 California Building Code (CBC), we used a peak ground acceleration of 0.675 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.

The results of our CPTs and our liquefaction analyses indicate the soils beneath the site generally have substantial cohesion and/or relative density, and therefore, they are not susceptible to liquefaction. Therefore, we conclude the potential for liquefaction and associated hazards, such as lateral spreading, are very low.

6.2.3 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 site is primarily underlain by stiff to hard fine-grained deposits and potential granular layers are sufficiently dense and very thin. Therefore, we conclude the potential for cyclic densification to occur at the site is very low.

6.2.4 Fault Rupture

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

7.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our engineering analyses using the data from the CPTs and previous geotechnical report for the site vicinity, we conclude there are no major geotechnical or geological issues that would preclude development of the site as proposed. The primary geotechnical concerns are (1) shallow groundwater relative to the proposed bottom of excavation for the stacked parking, and (2) providing adequate vertical and lateral support for the proposed structure. These and other issues are discussed in this section.

7.1 Foundations and Settlement

It is our preliminary conclusion that the proposed structure may be supported on a shallow foundation system, provided it gains support below the potential undocumented fill layer, which we preliminarily estimate to be about 2 to 3 feet below existing grades. If the bottom of foundation level for the proposed structure is above eight feet bgs, the foundation system may consist of continuous or isolated spread footings bearing on native undisturbed soil without waterproofing. If the depth to bottom of foundation for the proposed structure is below eight feet bgs, the shallow foundation system should consist of a reinforced concrete mat, which should be waterproofed. If the proposed bottom of foundation elevation is below a depth of 10 feet bgs, the mat foundation should be designed to hydrostatic uplift pressures.

For preliminary design of the footings and/or mat foundation, we recommend using allowable bearing pressures of 2,500 pounds per square foot (psf) for dead-plus-live loads and 3,300 psf for total design loads. We preliminarily estimate differential settlement of a properly constructed new shallow foundation system based on the recommendations presented in this report will be less than 3/4 inch across a 30-foot horizontal distance.

7.2 Groundwater

As discussed in Section 5.1, groundwater was measured at depths ranging from 11 to 13 feet bgs across the site during our preliminary subsurface investigation. However, based on the historic groundwater data we reviewed for two sites in the vicinity of the subject property, we believe the groundwater level will rise about 1 to 3 feet above the measured levels during periods of heavy rainfall. We preliminarily conclude a design groundwater level of Elevation 12 Feet[1] should be used for preliminary design and planning purposes, which corresponds to approximately 10 feet below existing grade. If the proposed construction is to extend close to or below this groundwater elevation, , we recommend a piezometer be installed at the site during the final geotechnical investigation to allow monitoring of the groundwater level so that seasonal fluctuations in the depth to groundwater can be better evaluated.

Assuming the proposed development will include below-grade stacked parking on a roughly 2-foot-thick mat foundation, the foundation subgrade may extend about 8 to 14 feet below existing grades. Therefore, depending on the time of year that excavation is performed, the foundation subgrade may be as much as 4 feet below the groundwater.

Depending on the final parking pit slab elevation, excavation dewatering may be necessary to construct the below-grade portion of the building. In addition, as discussed in Section 7.1, the mat foundation will need to be designed to resist hydrostatic uplift forces and be underlain by a waterproofing if the proposed bottom of foundation level is below a depth of 10 feet bgs. If the proposed depth to bottom of foundation is below a depth of 8 feet bgs, but above a depth of 10 feet bgs, the foundation will need waterproofing only. The construction dewatering system must be capable of maintaining the groundwater level below the foundation subgrade until sufficient building weight is available to resist the hydrostatic uplift pressure and/or micropiles have been installed, at which time the groundwater may be allowed to rise to its normal elevation.

7.3 Excavation and Shoring

We estimate the proposed below-grade stacked parking configuration will require an excavation on the order of 8 to 14 feet below existing grades (including excavation for the foundation). In some locations, the sides of the excavation may be cut at temporary slopes and the walls subsequently backfilled following construction of the podium. For planning purposes, a maximum cut slope inclination of 1:1 (horizontal:vertical) may be assumed, which corresponds to OSHA Type B soil. If seepage is observed in the cut-slopes during construction, the slopes should have a maximum inclination of 1.5:1, which corresponds to OSHA Type C soil.

In locations where there is insufficient space to slope the excavation due to the presence of adjacent streets, sidewalks, critical underground utilities, or existing structures, a temporary shoring system may be required. There are several key considerations in selecting a suitable shoring system. Those we consider of primary concern are:

- protection of surrounding improvements, including roadways, utilities, and adjacent structures
- proper construction of the shoring system to reduce potential for ground movement
- the presence of shallow groundwater
- cost.

Several methods of shoring are available; however, in our experience conventional soldier pile and lagging shoring is most suitable and economical in these soil conditions. A soldier pile and lagging system usually consists of steel H-beams and concrete placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds. If the excavation extends deeper than about 12 feet, a soldier pile and lagging system will likely require tiebacks, which could necessitate the need for an encroachment agreement with neighboring property owners.

7.4 Permanent Below-Grade Walls

Permanent below-grade walls should be designed to resist static lateral earth pressures, lateral pressures caused by earthquakes, and traffic loads (if vehicular traffic is expected within 10 feet of the wall). We preliminarily recommend the permanent below-grade walls be designed for the more critical of the following criteria:

- At-rest equivalent fluid weight of 60 pcf above the design groundwater table and 90 pcf below.
- Active pressure of 38 pcf and seismic increment of 27 pcf above the design groundwater level, 81 pcf plus a seismic increment of 13 pcf below (triangular distribution).

The recommended lateral earth pressures above are based on a level backfill conditions with no additional surcharge loads. Where the below-grade wall is subject to traffic loading within 10 feet of the wall, an additional uniform lateral pressure of 100 psf applied to the upper 10 feet of the wall.

The lateral earth pressures recommended are applicable to walls that are backdrained to prevent the buildup of hydrostatic pressure. One acceptable method for backdraining the wall is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the shoring or the back of the wall. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe at the base of the walls. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans Standard Specifications Section 68-1.025) or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140N or equivalent). A proprietary, prefabricated collector drain system, such as Tremdrain Total Drain or Hydroduct Coil, designed to work in conjunction with the drainage panel may be used in lieu of the perforated pipe surrounded by gravel described above. We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel materials to verify it is appropriate for its intended use. The pipe should be connected to a suitable discharge point; a sump and pump system may be required to drain the collector pipes for the below-grade parking area. To protect against moisture mitigation into the below-grade parking level, we recommend that the below-grade walls be moisture-proofed and water stops be installed at all construction joints.

7.5 Seismic Design

For design in accordance with the 2013 CBC, we recommend Site Class D be used. The latitude and longitude of the site are 37.8110 and -122.2711, respectively. Hence, in accordance with the 2013 SFBC, we recommend the following:

- S_S = 1.748g, S_1 = 0.694g
- S_{MS} = 1.748g, S_{M1} = 1.041g
- S_{DS} = 1.166g, S_{D1} = 0.694g
- *Seismic Design Category D for Risk Categories I, II, and III.*

7.6 Construction Considerations

The soil to be excavated generally consists of sand and clay, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. If site grading is performed during the rainy season, repeated loads by heavy equipment will reduce the strength of the surficial soil and decrease its ability to resist deformation; this phenomenon could result in severe rutting and pumping of the exposed subgrade. To reduce the potential for this behavior, heavy rubber-tired equipment as well as vibratory rollers, should be avoided.

Excavations that will be deeper than five feet and will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes. We judge temporary slopes with a maximum inclination of 1:1 (horizontal to vertical) should be stable, provided the slope is not surcharged by adjacent structures, construction equipment, or stockpiled soil.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

The preliminary conclusions and recommendations presented within are based on a preliminary field investigation and not intended for final design. Prior to final design, we should be retained to provide a final geotechnical report based on a supplemental field investigation. Additional borings will be required to further evaluate the subsurface conditions beneath the site and develop final foundation design recommendations. Once our final report has been completed, the design team has selected a foundation system, and prior to construction, we should review the project plans and specifications to check their conformance with the intent of our final recommendations. During construction, we should observe site preparation, foundation installation, and the placement and compaction of fill. These observations will allow us to compare the actual with the anticipated soil conditions and to check if the contractor's work conforms with the geotechnical aspects of the plans and specifications.

REFERENCES

California Building Code (2013).

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

Cao, T., Bryant, W. A., Rowshandel, B., Branum D. and Wills, C. J. (2003). “The Revised 2002 California Probabilistic Seismic Hazard Maps”

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

Lew, M., Sitar, N. (2010), “Seismic Earth Pressures on Deep Building Basements,” SEAOC 2010 Convention Proceedings.

Robertson, P.K. (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.

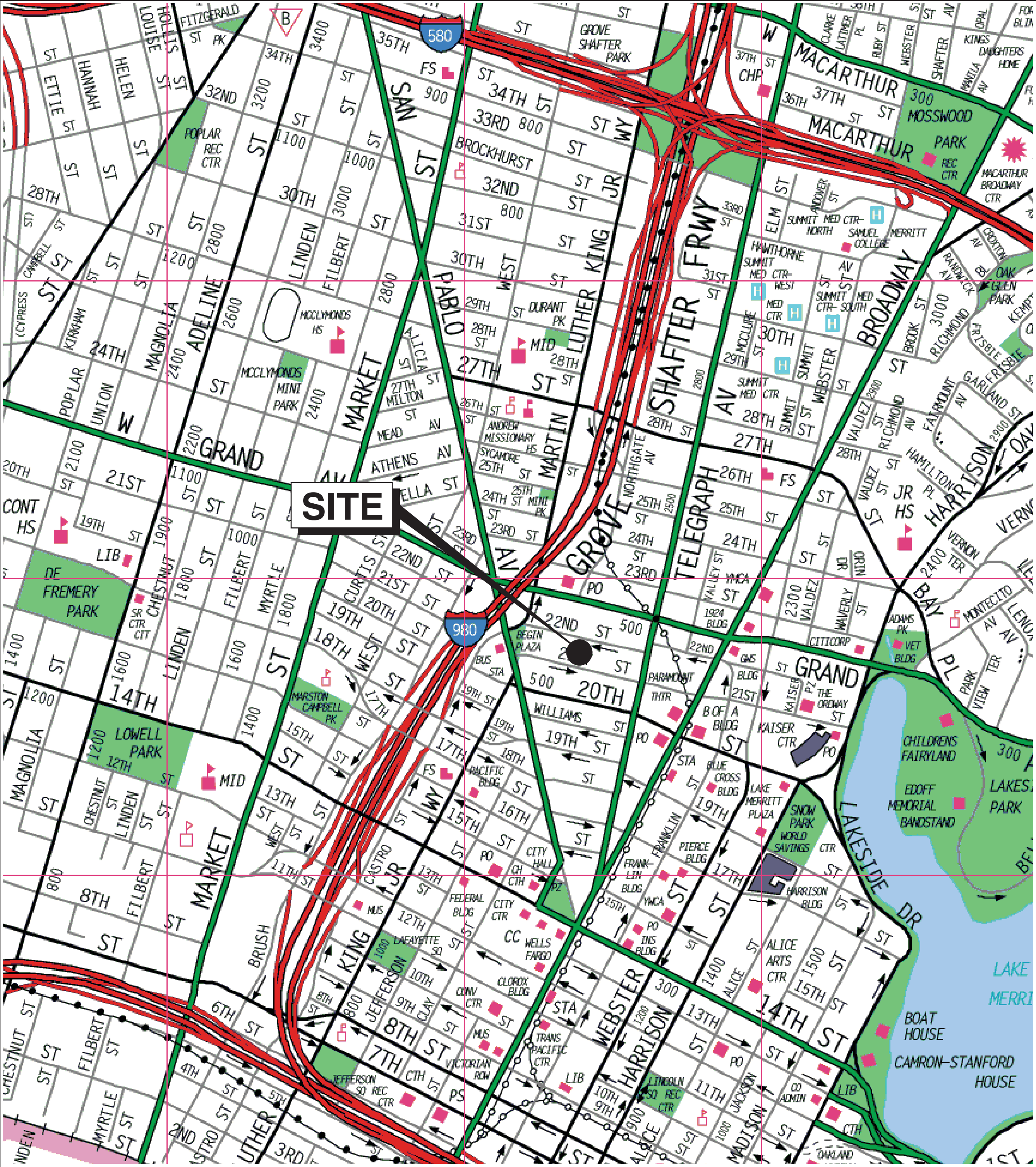
Robertson, P.K. (2009), “Interpretation of Cone Penetration Tests - A Unified Approach”, Canadian Geotechnical Journal, Vol. 46, No. 11, pp 1337-1355.

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.

U.S. Geological Survey, (2008), The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2): prepared by the 2007 Working Group on California Earthquake Probabilities, U.S. Geological Survey Open File Report 2007-1437.

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

FIGURES



Base map: The Thomas Guide
Alameda County
1999

0 1/4 1/2 Mile

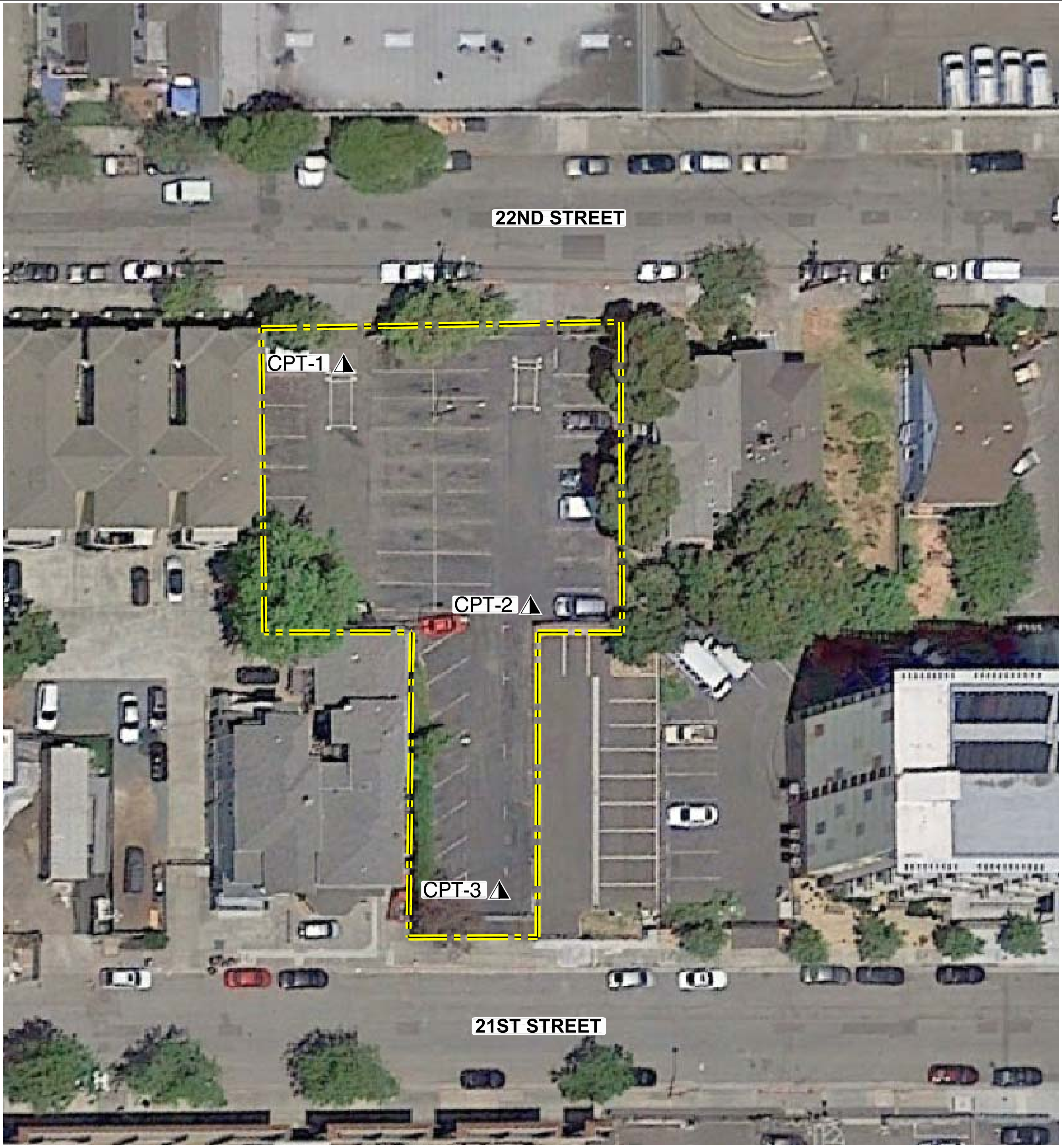
Approximate scale

585 22ND STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 07/15/15	Project No. 15-927	Figure 1



Base map: Google Earth, 2014.

EXPLANATION

CPT-1 ◭ Approximate location of cone penetration test (CPT) by Rockridge Geotechnical, Inc., July 9, 2015

Project limits

0 40 Feet

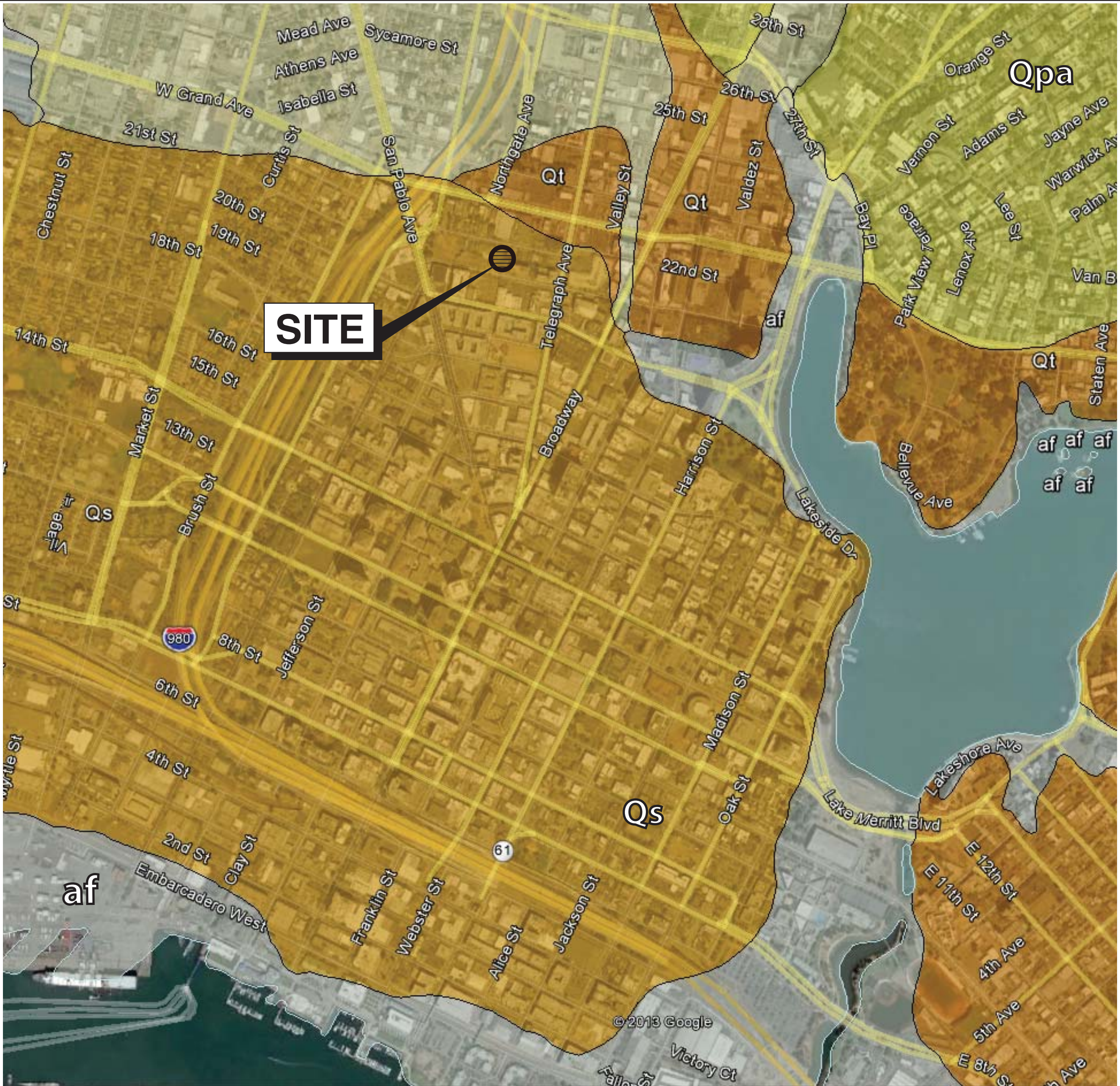
Approximate scale

585 22ND STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

SITE PLAN

Date 07/16/15	Project No. 15-927	Figure 2



Base map: Google Earth with U.S. Geological Survey (USGS), Alameda County, 2013.

EXPLANATION

af	Artificial Fill
Qs	Beach and Dune Sand (Quaternary)
Qpa	Alluvium (Pleistocene)
Qt	Marine terrace deposits (Pleistocene)
	Geologic contact: dashed where approximate and dotted where concealed, queried where uncertin

0 1,250 2,500 Feet

Approximate Scale

585 22ND STREET Oakland, California	**REGIONAL GEOLOGIC MAP**
ROCKRIDGE GEOTECHNICAL	Date 07/16/15 | Project No. 15-927 | 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

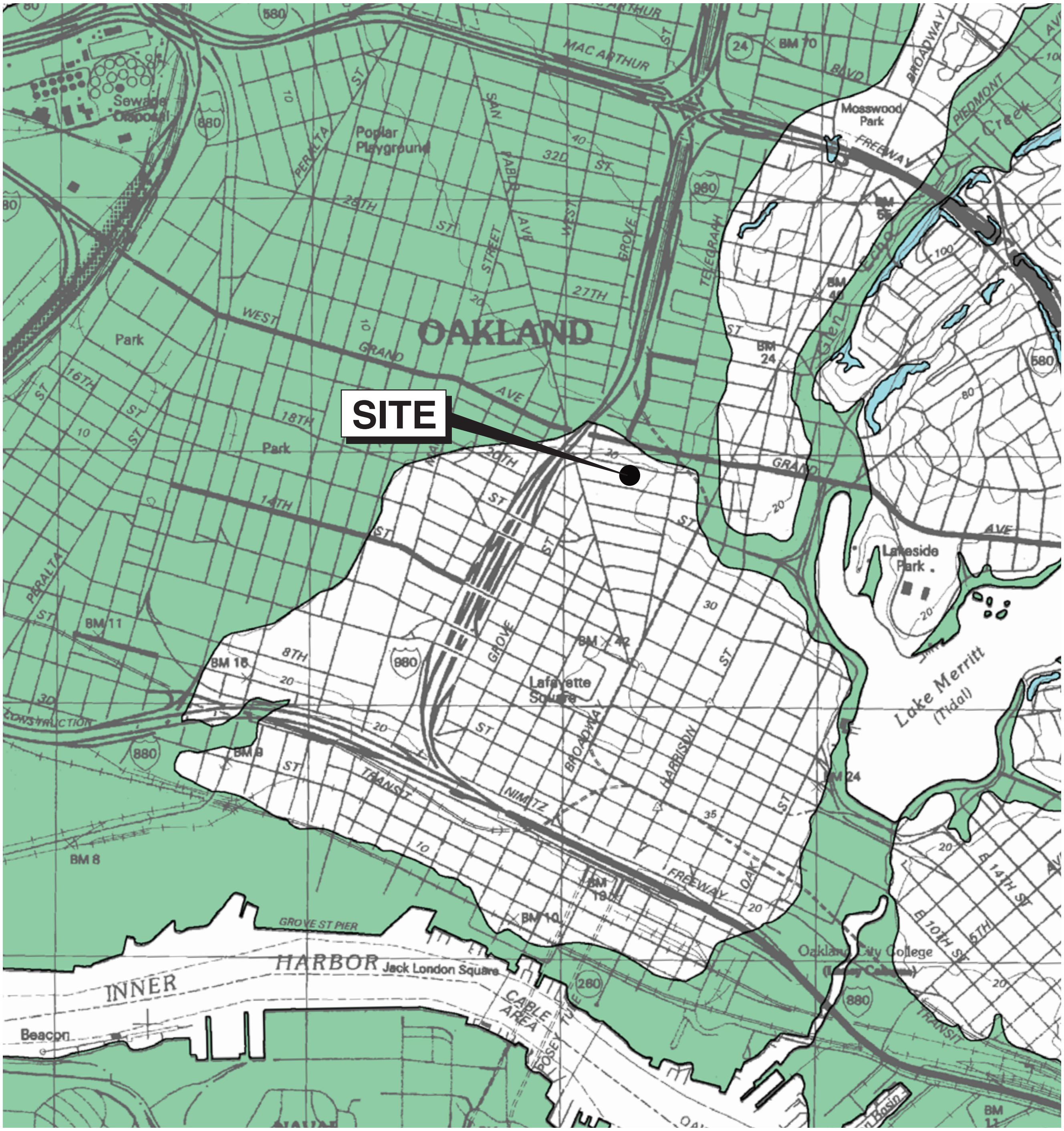
Approximate scale

585 22ND STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 07/15/15	Project No. 15-927	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

585 22ND STREET
Oakland, California

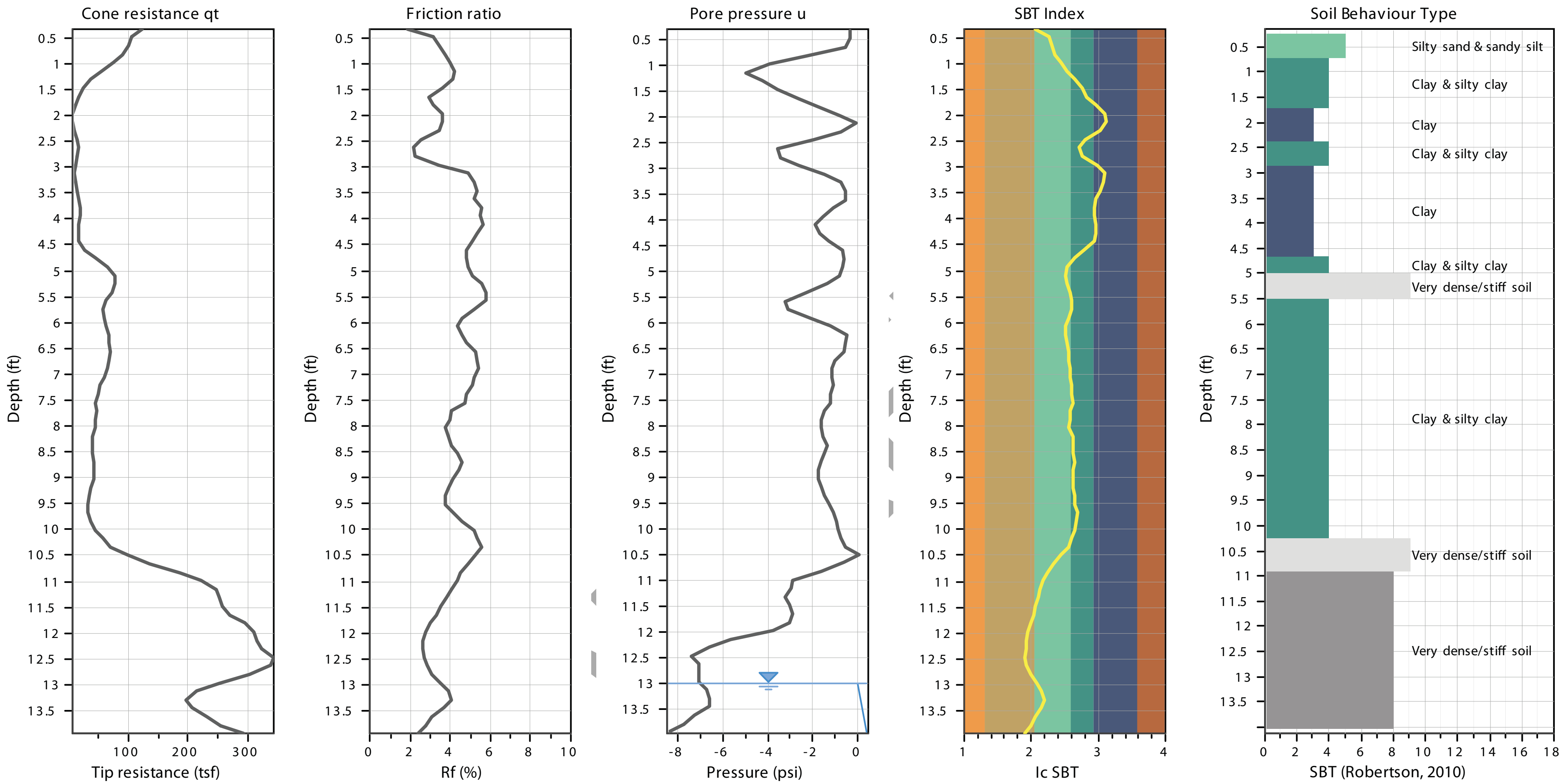
SEISMIC HAZARDS ZONE MAP

ROCKRIDGE GEOTECHNICAL

Date 07/15/15	Project No. 15-927	Figure 5

ROCKRIDGE GEOTECHNICAL

APPENDIX A
Cone Penetration Test Results



Total depth: 13.94 ft, Date: 7/9/2015
Measured Groundwater Depth: 13 feet
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

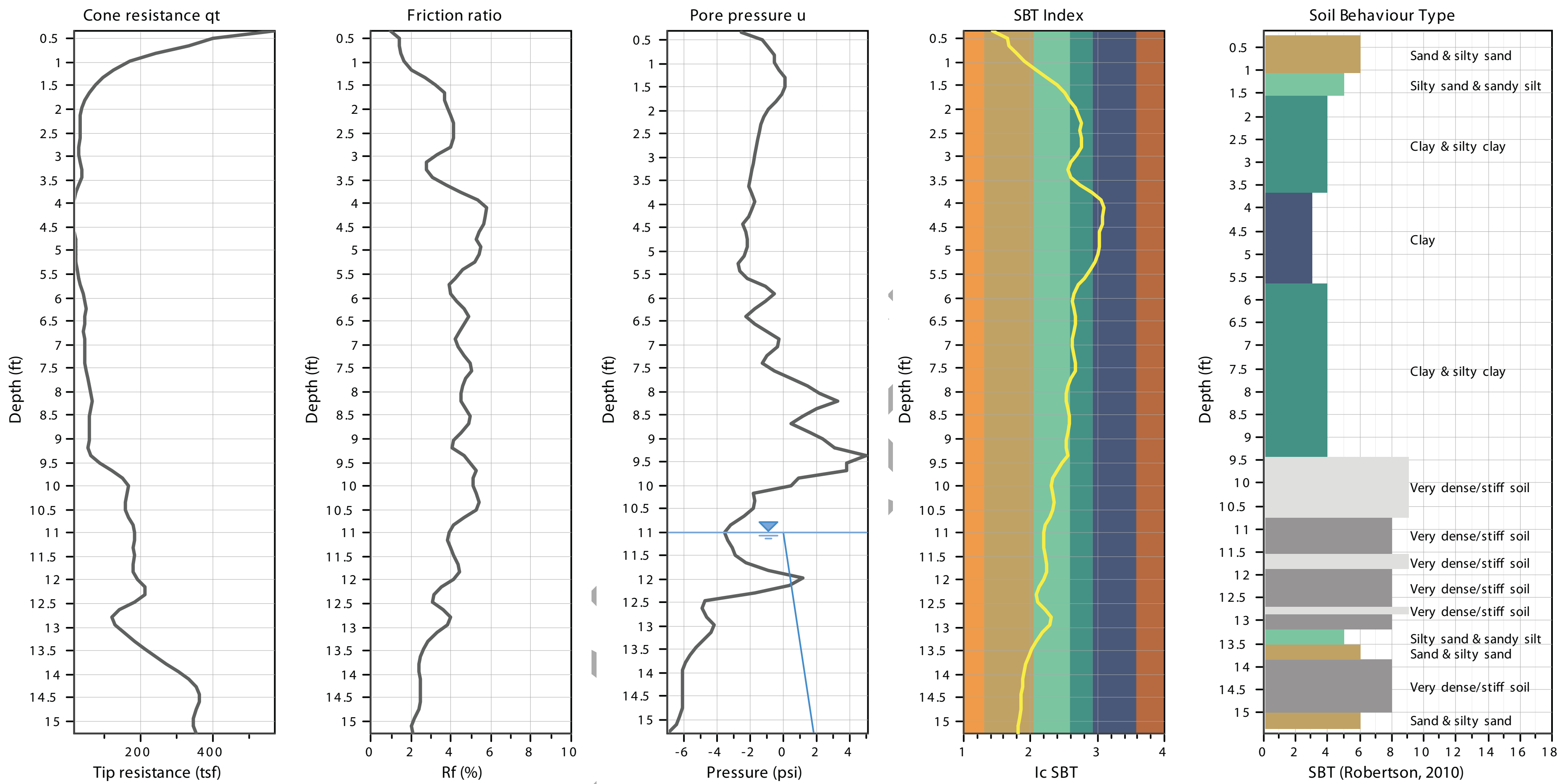
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

585 22ND STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 07/15/15 | Project No. 15-927 | Figure A-1



Total depth: 15.26 ft, Date: 7/9/2015
Measured Groundwater Depth: 11 feet
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

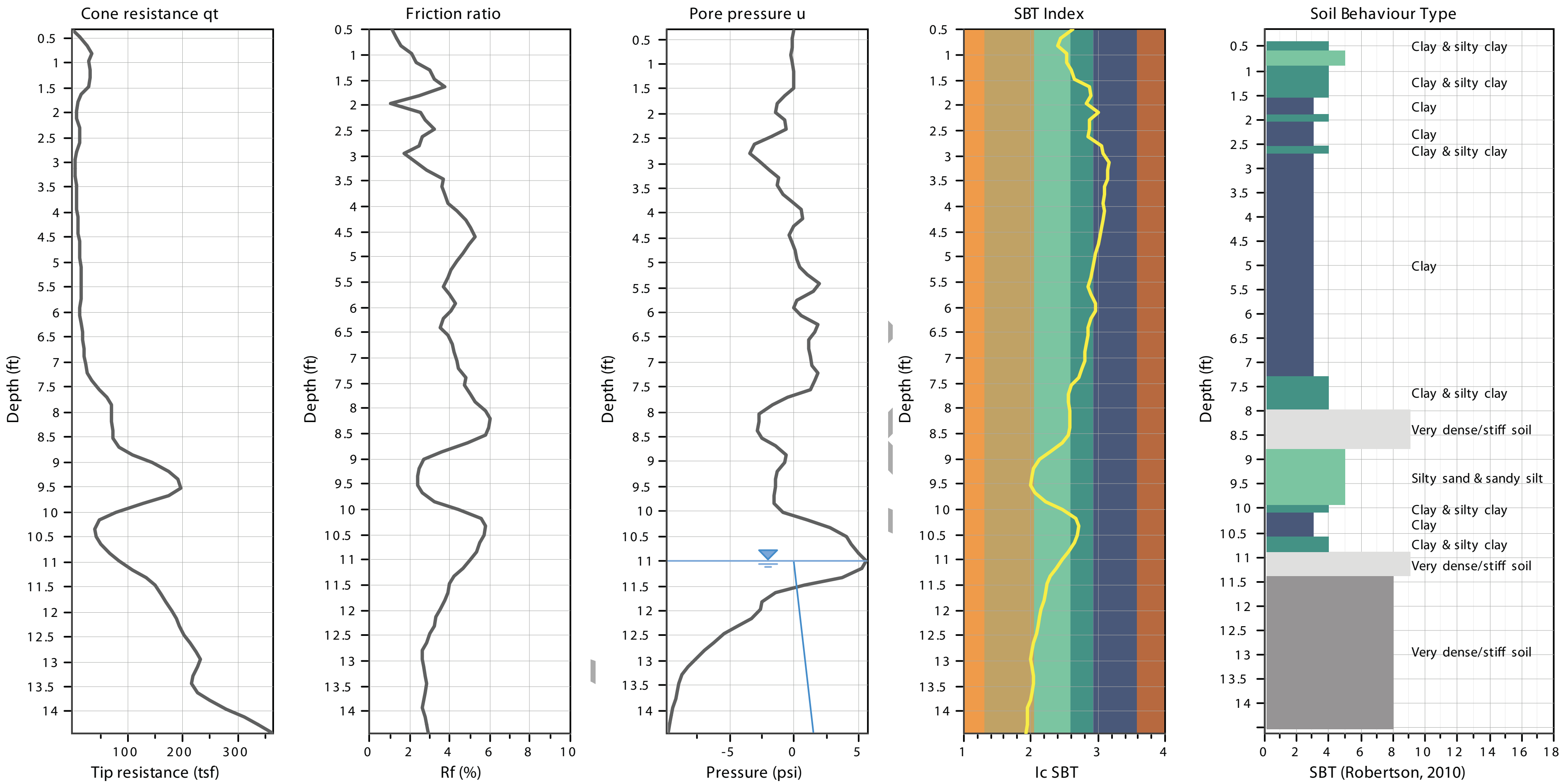
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

585 22ND STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 07/15/15	Project No. 15-927	Figure A-2



Total depth: 14.44 ft, Date: 7/9/2015
Measured Groundwater Depth: 11 feet
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

585 22ND STREET
Oakland, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 07/15/15	Project No. 15-927	Figure A-3