Appendix GEO

Geotechnical Report

ROCKRIDGE GEOTECHNICAL

Prepared for **Aralon Properties**

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED OFFICE BUILDING AND
PARKING STRUCTURE
493 FORBES BOULEVARD
SOUTH SAN FRANCISCO, CALIFORNIA

May 10, 2019
Project No. 19-1674

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Mr. Colum Regan
Aralon Properties
482 Bryant Street
San Francisco, California 94107

Subject: Preliminary Geotechnical Investigation Report
Proposed Office Building and Parking Structure
493 Forbes Boulevard
South San Francisco, California

Dear Mr. Regan,

We are pleased to present our preliminary geotechnical investigation report for the proposed office building and parking structure to be constructed at 493 Forbes Boulevard in South San Francisco, California. Our preliminary geotechnical investigation was performed in accordance with our proposal dated March 18, 2019.

The subject property is located on the northern side of Forbes Boulevard west of its intersection with Allerton Avenue. The site is a relatively level, trapezoidal-shaped, 2.255-acre lot with maximum plan dimensions of about 215 by 425 feet. The northern perimeter of the site is bordered by a relatively steep slope down to the railroad tracks. The steepest portion of the slope is approximately 1.5:1 (horizontal:vertical) at the northeastern corner of the site with approximately 7 feet of elevation change. The project site is currently occupied by a one-story, 53,000-square-foot commercial building surrounded by asphalt and concrete pavements. Current plans are to demolish the existing building and construct a four-story, at-grade office building at the front of the site and a three-story parking structure at the rear of the site.

From a geotechnical standpoint, we preliminarily conclude the site can be developed as planned. The primary geotechnical concerns are: (1) foundation settlement due to compression of the underlying clay soils, (2) the potential for up to about one inch and 1-1/2 inches of seismically-induced settlement due to liquefaction beneath the proposed office building and parking garage, respectively, and 3) providing adequate vertical and lateral support for the proposed new structures.

We preliminarily conclude the proposed new parking structure may be supported on a shallow foundation system, such as spread footings, bearing on improved soil. The proposed new office building may be supported on a stiffened foundation system, such as a conventional reinforced concrete mat or interconnected continuous footings (i.e., a stiffened grid). If it is determined that the estimated total settlement (static plus seismic) can be tolerated by the structure, the office building may also be supported by a shallow foundation system on improved soil.

This report presents preliminary conclusions and recommendations regarding geotechnical aspects of the project. A final geotechnical investigation, potentially including additional CPTs and shear wave velocity measurements, should be performed to develop final geotechnical conclusions and recommendations for the project.

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

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC.

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**PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED OFFICE BUILDING AND PARKING STRUCTURE
493 FORBES BOULEVARD
South San Francisco, 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 493 Forbes Boulevard in South San Francisco, California. The subject property is located on the northern side of Forbes Boulevard west of its intersection with Allerton Avenue, as shown on the Site Location Map (Figure 1).

The site is a relatively level, trapezoidal-shaped, 2.255-acre lot with maximum plan dimensions of about 215 by 425 feet. The northern perimeter of the site is bordered by a relatively steep slope down to the railroad tracks. The steepest portion of the slope is approximately 1.5:1 (horizontal:vertical) at the northeastern corner of the site with approximately 7 feet of elevation change. The project site is currently occupied by a one-story, 53,000-square-foot commercial building surrounded by asphalt and concrete pavements.

Current plans are to demolish the existing building and construct a four-story, at-grade office building at the front of the site and a three-story parking structure at the rear of the site. Structural design loads were not available at the time this report was prepared. Based on our experience with similar buildings, we estimate the average building pressures will be on the order of approximately 400 and 525 psf for the proposed office building and parking garage, respectively.

2.0 SCOPE OF SERVICES

Our preliminary investigation was performed in accordance with our proposal dated March 18, 2019. 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 four cone penetration tests (CPTs), and performing engineering analyses to develop preliminary conclusions and recommendations regarding:

- the most appropriate foundation type(s) for the proposed buildings
- preliminary design criteria for the recommended foundation type(s)
- estimates of foundation settlement
- design groundwater level
- site seismicity and seismic hazards, including the potential for liquefaction and lateral spreading, and total and differential settlement resulting from liquefaction and/or cyclic densification
- 2016 California Building Code site class and design spectral response acceleration parameters.

3.0 FIELD INVESTIGATION

Our subsurface investigation consisted of performing four CPTs. Prior to advancing the CPTs, we obtained a drilling permit from San Mateo County Environmental Health Services Division (SMCEHS), contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained a private utility locator, Precision Locating, LLC, to confirm the CPT locations were clear of existing utility lines.

Gregg Drilling and Testing, Inc. (Gregg) of Martinez, California performed the four CPTs, designated as CPT-1 through CPT-4, on April 4, 2019. The CPTs each were advanced until practical refusal was met at depths ranging from 21 to 81 feet bgs by hydraulically pushing an approximately 1.7-inch-diameter cone-tipped probe with a projected area of 15 square centimeters into the ground using a 30-ton truck rig. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance, frictional resistance, and pore water pressure, were recorded by a computer while the test was conducted. Accumulated data were processed by a computer to provide engineering information such as the soil behavior types, approximate strength characteristics, and the liquefaction potential of the soil encountered. The CPT logs showing tip resistance, friction ratio, and pore water pressure by depth, as well as correlated soil behavior type (Robertson, 2010), are presented in Appendix A on Figures A-1 through A-4. Upon completion, the CPTs were backfilled with cement grout in accordance with SMCEHS grouting guidelines.

4.0 SUBSURFACE CONDITIONS

A regional geologic map of the site and vicinity (Figure 3) indicates most of the site is underlain by early Pleistocene-age alluvium (Qoa). The regional geologic map also indicates the northeastern corner of the site is underlain by hillslope deposits (Qsl) and localized areas near the center and southwestern corner of the site are underlain by artificial fill (af). The results of our CPTs indicate the site is generally underlain by heterogeneous alluvial sediments that consist predominantly of medium stiff to hard clays and silts interbedded with discontinuous dense to very dense granular (sand and/or gravel) layers to the maximum depth explored of about 81 feet bgs. The dense to very dense granular layers were encountered at varying depths in each CPT and range in thickness from less than one foot to up to about six feet.

Within the generally stiff soil profile, we encountered several 1- to 3-foot-thick layers of soft to medium stiff fine-grained soil at the location of CPT-3 in the northwestern corner of the site. The soft to medium stiff layers were encountered between depths of about 11 and 16 feet bgs. Based on the results of our CPTs, we conclude these layers are normally consolidated to lightly over-consolidated. Normally consolidated fine-grained deposits have an in-situ stress state close to that of their maximum past pressure and are highly compressible under new loads, compared to over-consolidated deposits.

Each of the four CPTs was advanced to refusal in dense soil and/or bedrock at depths ranging from 21 to 25 feet bgs at the northern perimeter of the site and depths of 59 to 81 feet bgs at the southern perimeter of the site.

4.1 Groundwater Conditions

To estimate the 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 closest site with historic groundwater data on the GeoTracker website is 477 Forbes Boulevard located directly west of the site. Between October 1999 and August 2011, groundwater was measured in multiple monitoring wells. The highest (i.e., shallowest) groundwater levels were measured at depths ranging from approximately 8.2 to 10.1 feet bgs. Based on the groundwater-level data, we preliminarily conclude a design groundwater depth of about eight feet bgs should be used for this project.

5.0 SEISMIC CONSIDERATIONS

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,[1] lateral spreading,[2] and cyclic densification[3]. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity

The site is located in the Coast Ranges geomorphic province that is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent shearing along the San Andreas fault system. Movements along this plate boundary in the Northern California region occur along right-lateral strike-slip faults of the San Andreas Fault system.

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

2 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.

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

The major active faults in the area are the San Andreas, San Gregorio, Hayward, and Calaveras faults. These faults and other known Quaternary-aged faults that are believed to be sources of major earthquakes (i.e., Magnitude>6.0) in the region are shown on Figure 4 as accessed from the U.S. Geological Survey (USGS) database (USGS, 2010). Known faults within a 50-kilometer radius of the site, the distance from the site and estimated mean characteristic Moment magnitude[4] [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
N. San Andreas - Peninsula	5.7	West	7.23
N. San Andreas (1906 event)	5.7	West	8.05
San Gregorio Connected	15	West	7.50
N. San Andreas - North Coast	22	Northwest	7.51
Total Hayward	24	Northeast	7.00
Total Hayward-Rodgers Creek	24	Northeast	7.33
Monte Vista-Shannon	27	Southeast	6.50
Total Calaveras	38	East	7.03
Mount Diablo Thrust	39	Northeast	6.70
Green Valley Connected	44	Northeast	6.80
Rodgers Creek	48	North	7.07

In the past 200 years, four major earthquakes (i.e., Magnitude > 6) have been recorded on the

4 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.

San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) Intensity 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 on the Peninsula segment of the San Andreas Fault. Severe shaking occurred with an MM of about VIII-IX, 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 October 17, 1989 with an M_w of 6.9. This earthquake occurred in the Santa Cruz Mountains about 82 kilometers south of the site.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (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 ($M_w = 6.2$).

On August 24, 2014 an earthquake with an estimated maximum intensity of VIII (severe) on the MM scale occurred on the West Napa fault. This earthquake was the largest earthquake event in the San Francisco Bay Area since the Loma Prieta Earthquake. The M_w of the 2014 South Napa Earthquake was 6.0.

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 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 our 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.

5.2.1 Ground Shaking

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

5.2.2 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.

Liquefaction susceptibility was assessed using the software CLiq v2.2.1.14 (GeoLogismiki, 2016). CLiq uses measured field CPT data and assesses liquefaction potential given a user-defined earthquake magnitude and peak ground acceleration (PGA). Our liquefaction analyses were performed using the methodology proposed by Boulanger and Idriss (2014). We also used the relationship proposed by Zhang, Robertson, and Brachman (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement; this relationship is an extension of the work by Ishihara and Yoshimine (1992).

Our preliminary analyses were performed using a "during earthquake" groundwater depth of eight feet bgs. In accordance with the 2016 CBC, we used a peak ground acceleration of 0.73 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 8.05 earthquake, which is consistent with the mean characteristic moment magnitude for the San Andreas Fault, as presented in Table 1.

Our liquefaction analyses indicate there are thin layers of potentially liquefiable soil between depths of approximately 8 and 38 feet bgs. The localized potentially liquefiable layers the site are generally less than four feet thick and a majority of the material identified as potentially liquefiable in the liquefaction analyses generally consists of silty and sandy clay, as well as silty sand to sandy silt. We preliminarily estimate total and differential ground settlement resulting from post-earthquake reconsolidation following an MCE event with PGA_M of 0.73g will be on the order of about one inch and 3/4 inch across a horizontal distance of 30 feet, respectively, beneath the proposed new office building. We preliminarily estimate total and differential ground settlement results from post-earthquake reconsolidation beneath the proposed parking garage will be on the order of 1-1/2 inches and one inch across a horizontal distance of 30 feet, respectively.

Our preliminary analysis indicate the potentially liquefiable layers are sufficiently thin and/or have a sufficient amount of plastic fines such that the potential for surface manifestations from liquefaction, such as sand boils, and loss of bearing capacity for shallow foundations are low.

Lateral spreading occurs when a continuous layer of soil liquefies at depth and the soil layers above move toward an unsupported face, such as a shoreline slope, or in the direction of a regional slope or gradient. Because of the presence of potentially liquefiable soils and the topographic conditions, liquefaction-induced lateral spreading may be a concern for this site. Potential lateral spread displacements are difficult to estimate, as they depend on numerous factors, such as the geometry and continuity of the liquefiable layers beneath the site, the topography and the stratigraphy of the subsurface materials adjacent to the site, and the presence, geometry and integrity of structural systems. Based on our analyses, we preliminarily conclude the risk of lateral spreading to occur during a major seismic event is low, but should be further evaluated during the final investigation.

5.2.3 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 blanketed by material with a soil behavior type of loose to medium dense sand and silty sand, and medium stiff to very stiff silty clay. We preliminarily conclude the potential for cyclic densification to occur at the site is low. Additional borings and laboratory testing should be performed during the final investigation to further evaluate the potential for settlement due to cyclic densification at the site.

5.2.4 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.

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our preliminary engineering analyses using the data from our CPTs, 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) foundation settlement due to compression of the underlying clay soils, (2) the potential for up to about one inch and 1-1/2 inches of seismically-induced settlement due to liquefaction beneath the proposed office building and parking garage, respectively, and 3) providing adequate vertical and lateral support for the proposed new structures. These and other issues are discussed in this section.

6.1 Foundations and Settlement

We performed preliminary analyses to estimate settlement of the proposed new buildings under static and seismic loads using assumed average foundation pressures of 400 and 525 psf for the proposed office building and parking garage, respectively. Settlement estimates for the proposed new buildings supported on shallow foundation systems bearing on native soil are presented in Table 2 below.

TABLE 2
Preliminary Settlement Summary
New Buildings Supported on Shallow Foundations

Structure	Static (in)	Seismic (in)	Total (in)
Office Building	1.0	1.0	2.0
Parking Garage	1.0	1.5	2.5

Provided the estimated settlements presented in Table 2 can be tolerated from a structural and architectural standpoint, we preliminarily conclude the proposed office building may be supported on a stiffened foundation system, such as a conventional reinforced concrete mat or interconnected continuous footings (i.e., a stiffened grid). If the estimated total settlements are not acceptable to the project team and/or the stiffened foundation system cannot be economically designed to limit differential settlement to a value that can be tolerated by the structure, then the proposed new structure may be supported on spread footings bearing on improved soil provided the soil improvement extends to a depth that would reduce differential settlement of the structure under both static and seismic conditions to a tolerable amount. Because it is not practical to support the parking garage on a mat foundation designed for relatively low bearing pressures, we conclude the foundation system for the garage should consist of spread footings bearing on improved ground. Preliminary recommendations for design of a stiffened mat foundation bearing on firm, native soil and spread footings bearing on improved soil are presented in the sections below.

6.1.1 Mat Foundation

The mat foundation should be designed to limit the amount of differential settlement to tolerable levels. To limit total static settlement of the mat to one inch, localized bearing pressures should not exceed 3,000 psf for dead-plus-live loads. To evaluate the pressure distribution beneath the mat foundation, we preliminarily recommend a modulus of vertical subgrade reaction (K_S) of 20 pounds per cubic inch (pci) be used. This value has been corrected to take into account the mat widths and may be increased by 1/3 for total load conditions. Once the structural engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of subgrade reaction, if appropriate.

Lateral loads can be resisted by a combination of passive pressure on the vertical faces of the mats and friction along the bottom of the foundation. Lateral resistance may be computed using an equivalent fluid weight of 300 pcf (triangular distribution); the upper foot of soil should be ignored unless confined by a slab or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30 where the mat is in contact with the soil. Where a vapor retarder is placed beneath the mat, a base friction coefficient of 0.20 should be used. The passive pressure and frictional resistance values include a factor of safety of at least 1.5.

Where water vapor transmission through the mat slab is undesirable, we recommend installing a water vapor retarder beneath the mat. The vapor retarder may be placed directly on the smooth, compacted soil subgrade. The retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745 and should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder. Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the mat foundation should have a low w/c ratio - less than 0.45. If necessary, workability should be increased by adding plasticizers. In addition, the mat should be properly cured. Before floor coverings, if any, are placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer’s requirements.

6.1.2 Spread Footings on Improved Ground

Spread footings bearing on improved ground may be used to support the proposed new buildings. We conclude drill displacement sand-cement (DDSC) columns or rammed aggregate piers (RAPs) to be the most appropriate ground improvement methods for this project. Descriptions of both ground improvement methods are presented below.

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. This system results in low vibration during installation and generate relatively few drilling spoils (approximately one-half cubic yard per DDSC) for off-haul. DDSC columns are installed under design-build contracts by specialty contractors.

For preliminary design of spread footings bearing on DDSCs, we recommend assuming ground improvement elements will extend to depths ranging from approximately 20 to 25 feet bgs, depending on column loads. The length and spacing of the DDSC columns should be sufficient to limit total settlement of the parking structure to less than one inch (static plus seismic). The DDSC columns, if properly designed, should be capable of increasing the allowable dead-plus-live-load bearing pressure to about 5,000 pounds per square foot (psf). The actual design allowable bearing pressure should be determined by the design-build ground improvement contractor, as it will be based on the size and spacing of the ground improvement elements.

A rammed aggregate pier is typically constructed by drilling a 30-inch-diameter shaft and replacing the excavated soil with compacted aggregate. The aggregate consists of clean, opengraded crushed rock below the water table and Class 2 aggregate base above the water table. The aggregate is compacted in approximately 12-inch-thck lifts using a modified hydraulic hammer mounted on an excavator. Rammed aggregate piers develop vertical support through a combination of frictional resistance along the shaft of the pier and improvement of the surrounding soil matrix, allowing use of significantly larger bearing capacities than feasible in unimproved soil. Rammed aggregate piers can also be designed to resist transient uplift loads by installing steel rods in the center of the pier; the rods are attached to a flat steel plate at the base of the of the footings. Installation of RAPs generate significantly more spoils than DDSCs so any cost comparison should consider the cost of off-site disposal of spoils.

For preliminary design of spread footings bearing on RAPs, we recommend assuming ground improvement elements will extend to depths of approximately 20 to 25 feet bgs, depending on column loads. The length and spacing of the RAPs columns should be sufficient to limit total settlement of the parking structure to less than one inch (static plus seismic). The RAPs, if properly designed, should be capable of increasing the allowable dead-plus-live-load bearing pressure to about 7,000 pounds per square foot (psf). The actual design allowable bearing pressure should be determined by the design-build ground improvement contractor, as it will be based on the size and spacing of the ground improvement elements.

For footings supported on improved ground, 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 passive resistance for sustained loading, we recommend using an equivalent fluid weight of 300 pcf (triangular distribution). The upper foot of soil should be ignored for lateral resistance unless confined by a slab or pavement. The recommended passive pressures include a factor of safety of at least 1.5 and may be used in combination with the frictional resistance without reduction. Allowable frictional resistance along the base of the footings should be calculated based on parameters provided by the design-build ground improvement contractor.

6.2 Floor Slabs

Slab-on-grade floors may be used for the proposed structures supported on spread footings bearing on improved soil, provided the potential for up to about 3/4 inch of differential settlement between the footings and the floor slab following a major earthquake is acceptable to the project team. If the potential for this differential settlement is not acceptable, the floor slab should be designed to span between ground improvement elements.

A capillary moisture break and vapor retarder are generally not required below parking garage slabs because there is sufficient air circulation to limit condensation of moisture on the slab surface. Where a capillary moisture break/vapor retarder is not used, we recommend six inches of Class 2 aggregate base compacted to at least 95 percent relative compaction be placed beneath the parking garage slab. To reduce the potential for excessive moisture in the electrical room, storage rooms, and other rooms with little ventilation, we recommend installing a capillary moisture break and water vapor retarder beneath the slab to reduce water vapor transmission through 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. The particle size of the capillary break material and sand (if used) should meet the gradation requirements presented in Table 3.

TABLE 3
Gradation Requirements for Capillary Moisture Break

Sieve Size	Percentage Passing Sieve
1 inch	90 – 100
3/4 inch	30 – 100
1/2 inch	5 – 25
3/8 inch	0 – 6

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 slabs should have a w/c ratio less than 0.45. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before floor coverings are placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirement.

6.3 Seismic Design

We understand the proposed buildings will be designed using the seismic provisions in the 2016 CBC. The latitude and longitude of the site are 37.6576° and -122.3894°, respectively. As discussed in Section 5.2.2, the site is underlain by thin potentially liquefiable soil layers. Although the CBC calls for a Site Class F designation for sites underlain by potentially liquefiable soil, we conclude a Site Class C or D designation is more appropriate because the potentially liquefiable layers are thin and therefore, the site will not incur significant nonlinear behavior during strong ground shaking.

Based on shear wave velocity correlations, we estimate the average shear wave velocity in the upper 30 meters ($V_{s,30}$) to be on the order of about 1,100 to 1,200 feet per second, which corresponds to Site Class D and Site Class C soil conditions, respectively. To determine which site class should be used for design of the proposed structures, we recommend shear wave velocity measurements be collected during the final investigation. If shear wave velocity measurements are not collected during the final investigation, we recommend Site Class D be used for design. In accordance with the 2016 CBC, we recommend the following for Site Class D:

- S_S = 1.85 g, S_1 = 0.86 g
- S_{MS} = 1.85 g, S_{M1} = 1.24 g
- S_{DS} = 1.23 g, S_{D1} = 0.86 g
- PGA_M = 0.73 g
- Seismic Design Category E for Risk Categories I, II, and III.

If shear wave velocity measurements confirm Site Class C may be used for design of one or both of the structures, we recommend the following in accordance with the 2016 CBC:

- S_S = 1.85 g, S_1 = 0.86 g
- S_{MS} = 1.85 g, S_{M1} = 1.12 g
- S_{DS} = 1.23 g, S_{D1} = 0.74 g
- PGA_M = 0.73 g
- Seismic Design Category D for Risk Categories I, II, and III.

7.0 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 and CPTs 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.

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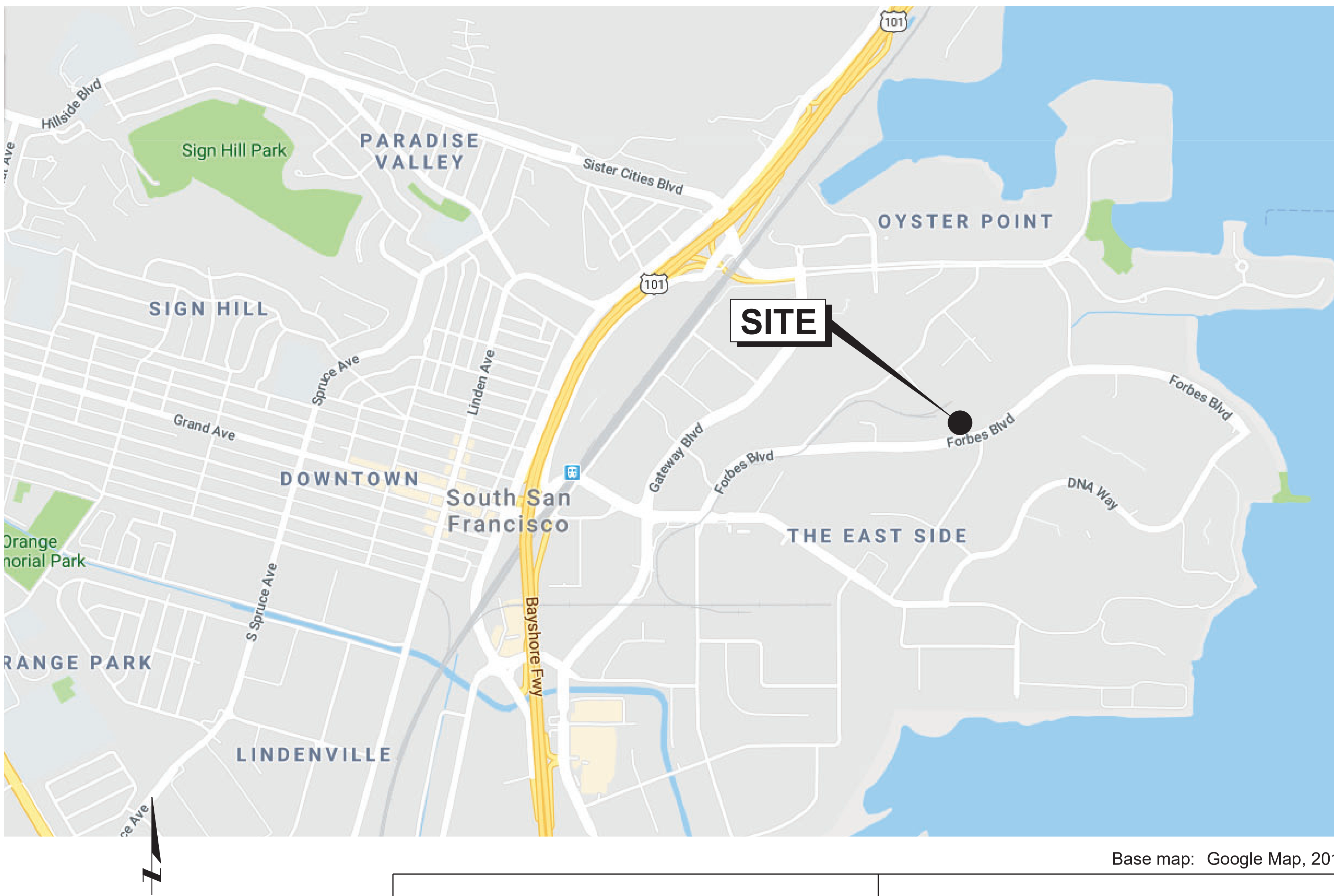
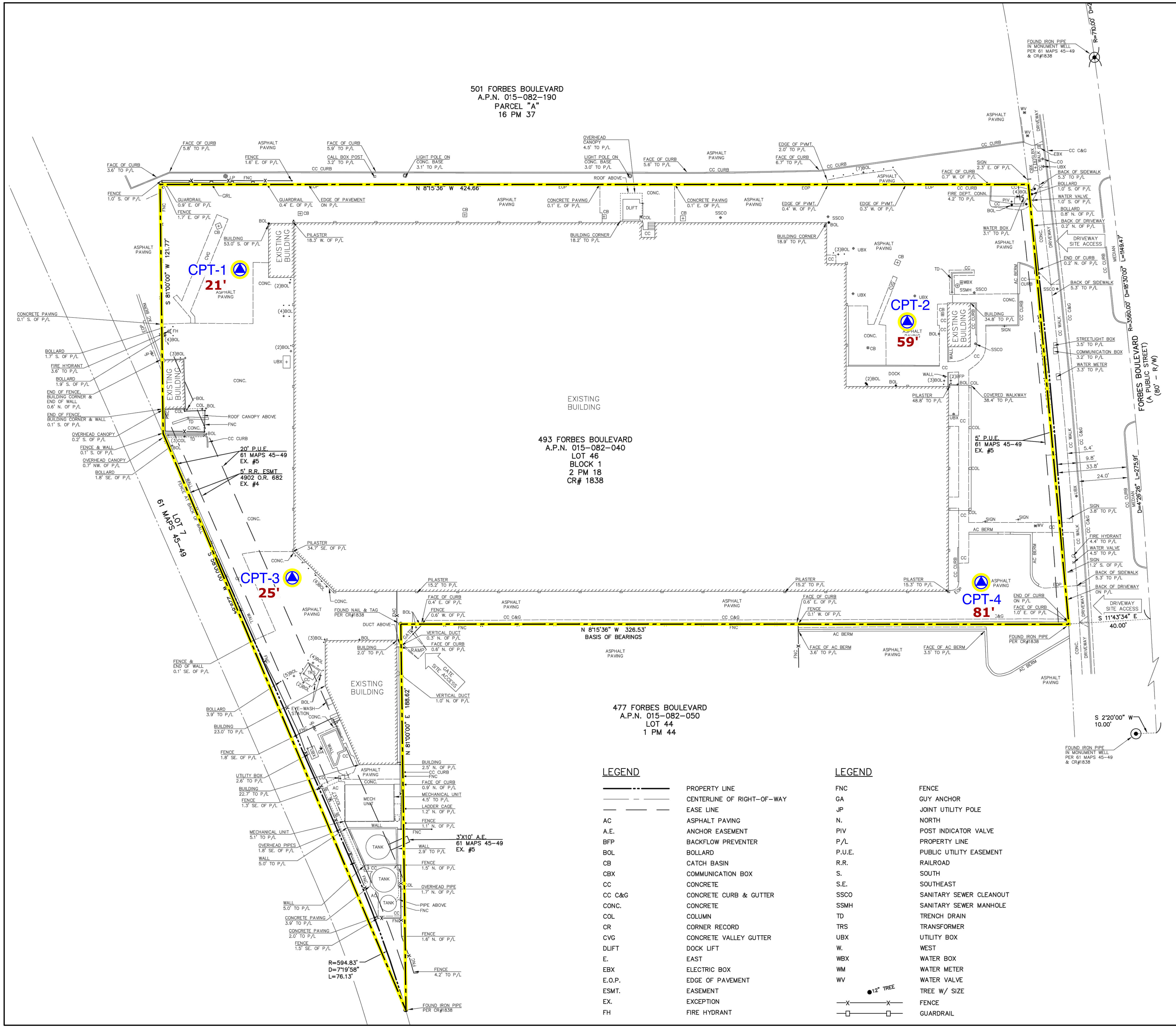
FIGURES



Base map: Google Map, 2018.

493 FORBES BOULEVARD South San Francisco, California	**SITE LOCATION MAP**
ROCKRIDGE GEOTECHNICAL	Date 04/24/19 · Project No. 19-1674 · Figure 1



LEGEND	
	PROPERTY LINE
	CENTERLINE OF RIGHT-OF-WAY
	EASE LINE
AC	ASPHALT PAVING
A.E.	ANCHOR EASEMENT
BFP	BACKFLOW PREVENTER
BOL	BOLLARD
CB	CATCH BASIN
CBX	COMMUNICATION BOX
CC	CONCRETE
CC C&G	CONCRETE CURB & GUTTER
CONC.	CONCRETE
COL	COLUMN
CR	CORNER RECORD
CVG	CONCRETE VALLEY GUTTER
DLIFT	DOCK LIFT
E.	EAST
EBX	ELECTRIC BOX
E.O.P.	EDGE OF PAVEMENT
ESMT.	EASEMENT
EX.	EXCEPTION
FH	FIRE HYDRANT

LEGEND	
FNC	FENCE
GA	GUY ANCHOR
JP	JOINT UTILITY POLE
N.	NORTH
PIV	POST INDICATOR VALVE
P/L	PROPERTY LINE
P.U.E.	PUBLIC UTILITY EASEMENT
R.R.	RAILROAD
S.	SOUTH
S.E.	SOUTHEAST
SSCO	SANITARY SEWER CLEANOUT
SSMH	SANITARY SEWER MANHOLE
TD	TRENCH DRAIN
TRS	TRANSFORMER
UBX	UTILITY BOX
W.	WEST
WBX	WATER BOX
WM	WATER METER
WV	WATER VALVE
12" TREE	TREE W/ SIZE
	FENCE
	GUARDRAIL

EXPLANATION

CPT-1 Approximate location of cone penetration test by Rockridge Geotechnical Inc., April 4, 2019

21' Approximate depth of refusal in dense sand or rock

Project limits

0 50 Feet

Approximate scale

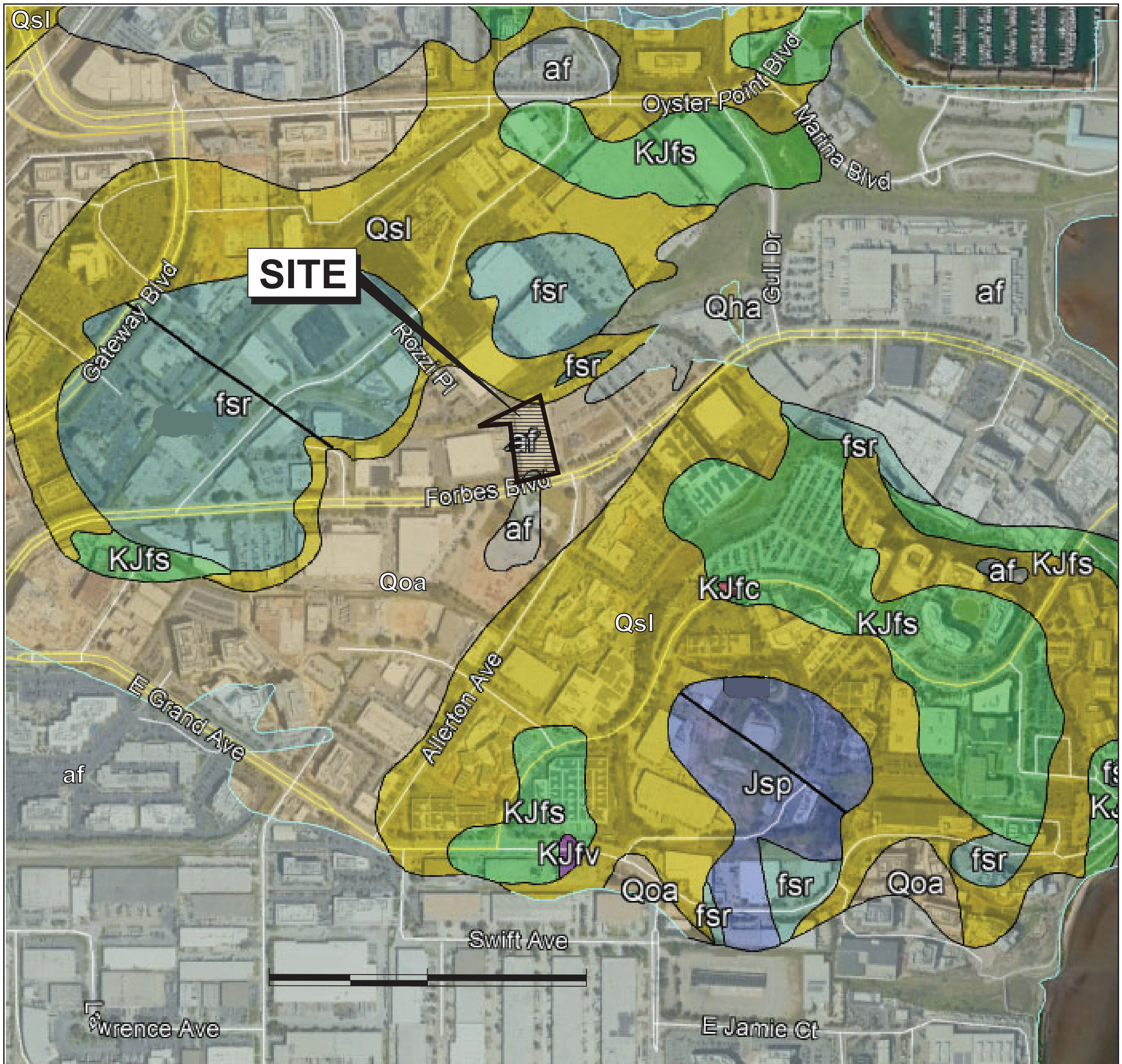
Reference: Base map from a drawing titled "ALTA/NSPS Land Title Survey", by Butler Armsden, dated February 4, 2018.

493 FORBES BOULEVARD
South San Francisco, California

SITE PLAN

Date 04/26/19	Project No. 19-1674	Figure 2

ROCKRIDGE GEOTECHNICAL



Base map: Google Earth with U.S. Geological Survey (USGS), San Mateo, 2018.

EXPLANATION

af	Artificial Fill
Qhym	Mud deposits (late Holocene)
Qha	Alluvium (Holocene)
Qsl	Hillslope Deposits (Quaternary)
Qoa	Alluvium (early Pleistocene)
fsr	Franciscan Complex melange (Eocene, Paleocene, and (or) Late Cretaceous)
KJfs	Franciscan Complex sedimentary rocks (Early Cretaceous and (or) Late Jurassic)
KJfv	Franciscan Complex volcanic rocks (Early Cretaceous and (or) Late Jurassic)
Jsp	Great Valley complex serpentinite (Jurassic)

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

0 800 1600 Feet

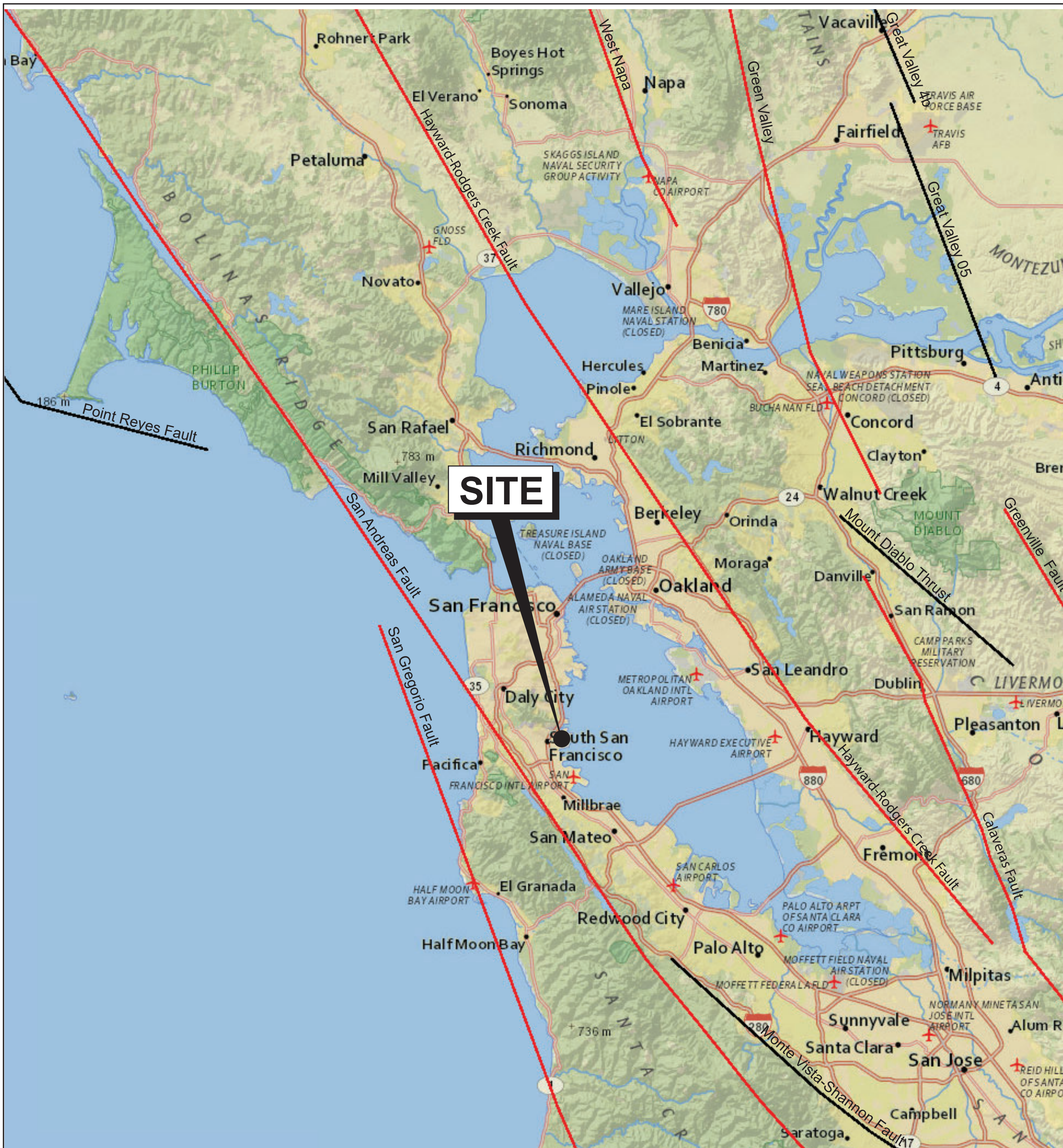
Approximate scale

493 FORBES BOULEVARD
South San Francisco, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 04/26/19 | Project No. 19-1674 | 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

493 FORBES BOULEVARD
South San Francisco, California

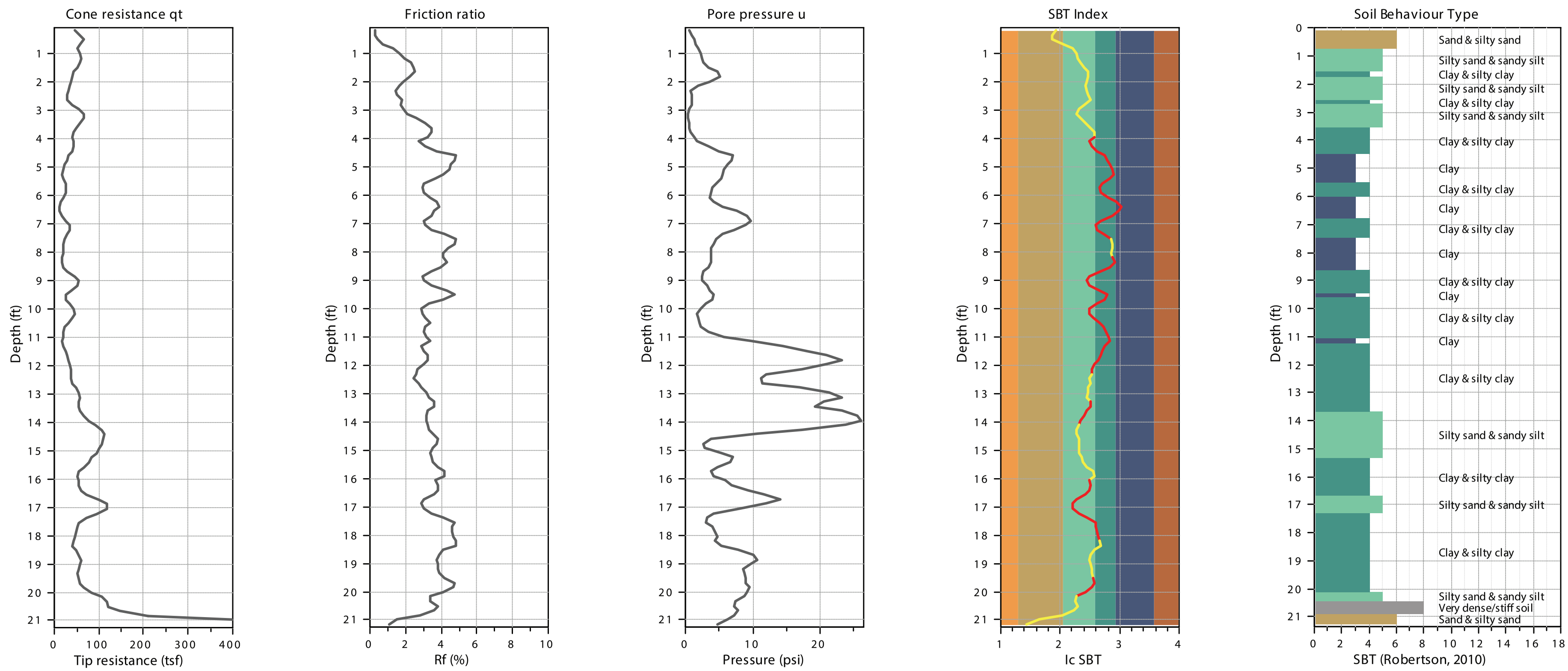
ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 04/24/19 | Project No. 19-1674 | Figure 4

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: 21.16 ft, Date: 4/4/2019
Groundwater not measured
Cone Operator: Gregg Drilling Inc.
Approximate Ground Surface Elevation: 42.5 feet

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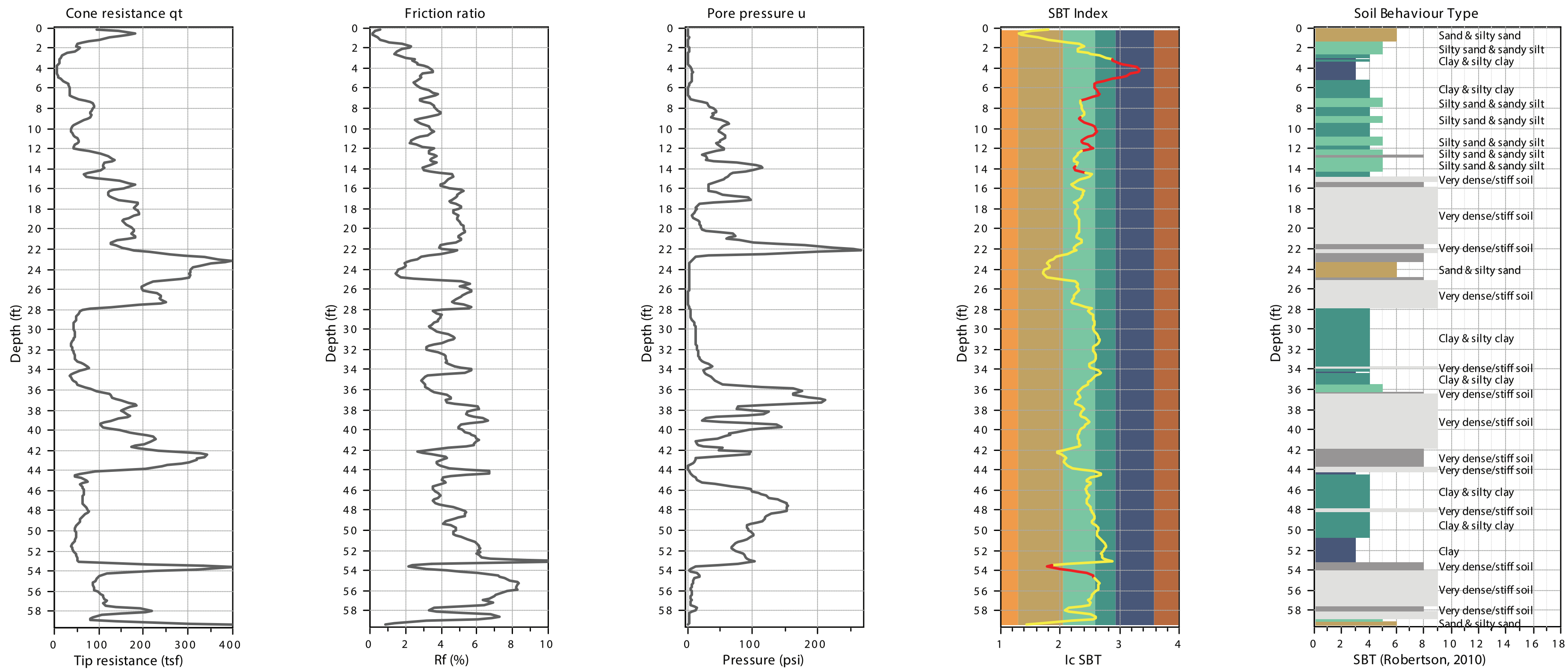
South San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 04/26/19 | Project No. 19-1674 | 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: 59.38 ft, Date: 4/4/2019
Groundwater not measured
Cone Operator: Gregg Drilling Inc.
Approximate Ground Surface Elevation: 41.0 feet

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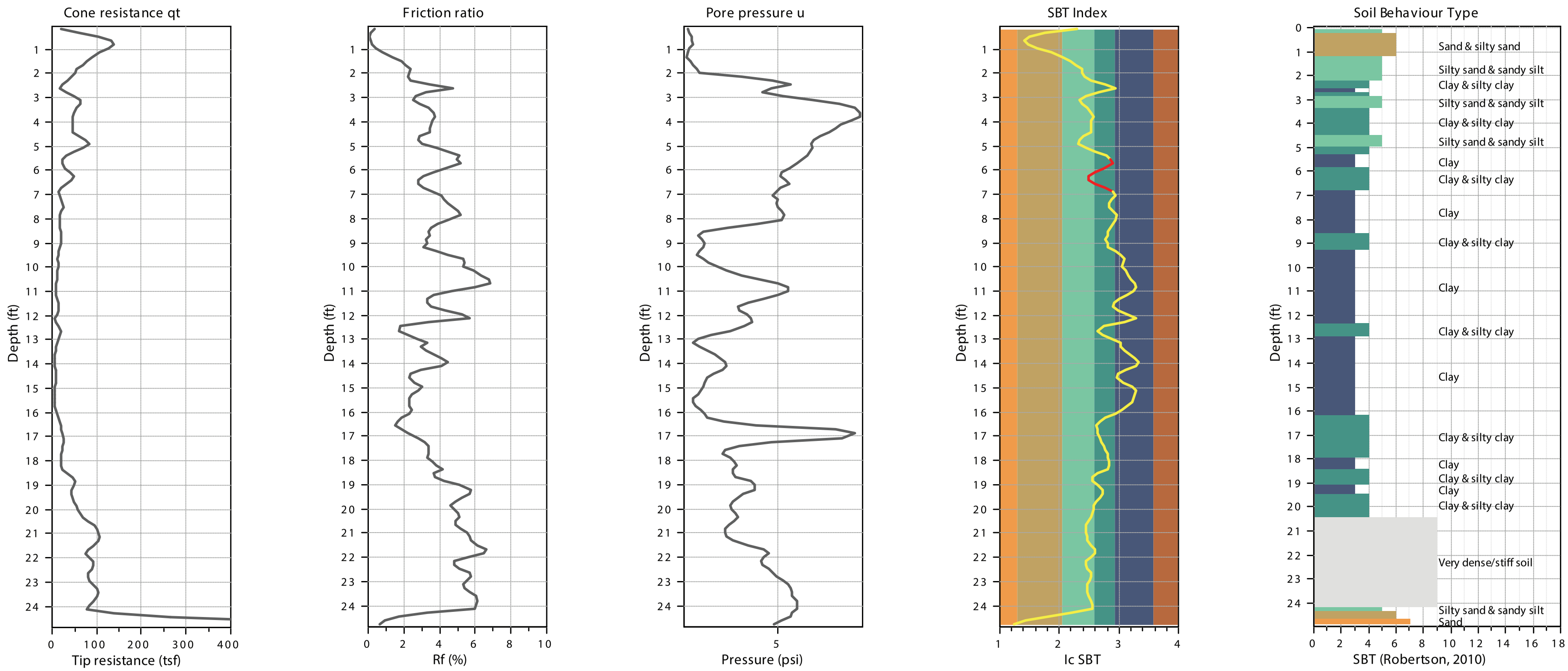
South San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 04/26/19 | Project No. 19-1674 | Figure A-2



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: 24.77 ft, Date: 4/4/2019
Groundwater not measured
Cone Operator: Gregg Drilling Inc.
Approximate Ground Surface Elevation: 43.7 feet

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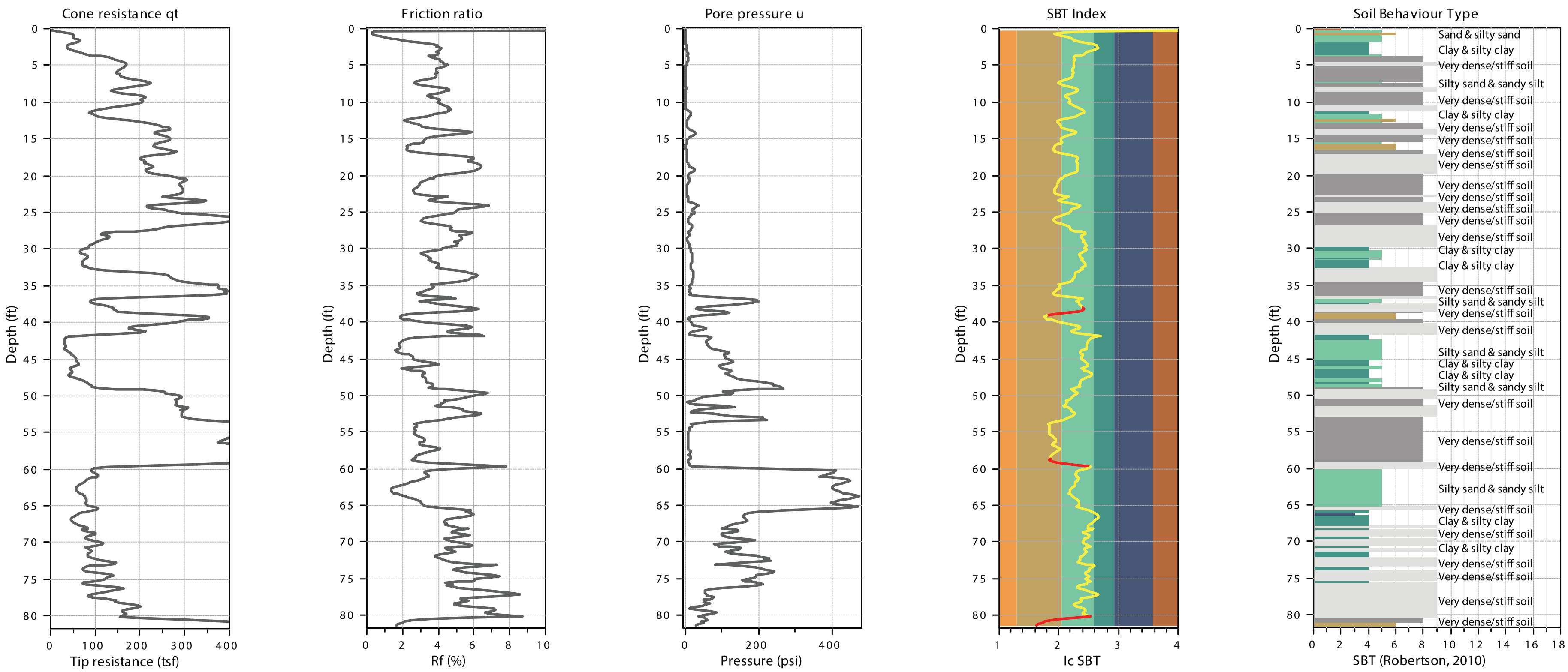
South San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 04/26/19	Project No. 19-1674	Figure A-3



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: 81.36 ft, Date: 4/4/2019
Groundwater not measured
Cone Operator: Gregg Drilling Inc.
Approximate Ground Surface Elevation: 42.8 feet

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South San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 04/26/19 | Project No. 19-1674 | Figure A-4