APPENDIX G

Geotechnical Investigation

ROCKRIDGE GEOTECHNICAL

Prepared for **Mid Pen Housing Corporation**

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL REDEVELOPMENT
MIDWAY VILLAGE
DALY CITY, CALIFORNIA

February 5, 2020
Project No. 18-1569

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February 5, 2020
Project No. 18-1569

Mr. Matthew Lewis
MidPen Housing Corporation
303 Vintage Park Drive, Suite 250
Foster City, California 94404

Subject: Final Report
Preliminary Geotechnical Investigation
Proposed Midway Village Redevelopment Project
47 Midway Drive
Daly City, California 94014

Dear Mr. Lewis:

We are pleased to present the results of our preliminary geotechnical investigation for the for the proposed Midway Village Redevelopment project in Daly City, California. Our preliminary geotechnical investigation was performed in accordance with our amended proposal dated November 14, 2019.

Midway Village is an existing residential development located on the eastern side of Schwerin Avenue and north of its intersection with Martin Street. The site consists of 33 parcels that encompass a total area of about 15.55 acres. It is bordered by a PG&E property to the north, vacant land and a single-family home subdivision under construction to the east, Schwerin Street to the west, and Martin Street to the south. The site is currently occupied by 35 multi-unit two-story residential buildings, asphalt-paved parking lots and interior streets and drive aisles, concrete flatwork and landscaping. The northeastern corner of the site is currently occupied by Bayshore Park. The ground surface across the site slopes gently down to the north and east with ground surface elevations (National Geodetic Vertical Datum) ranging from approximately 80 feet in the southwestern corner of the site (at the intersection of Schwerin and Martin streets) to 15 feet in the northeastern corner of Bayshore Park.

Current redevelopment plans for the Midway Village Redevelopment project call for demolishing the existing structures and other improvements on the site and constructing a combination of 2- to 3-story townhomes, 2- to 3-story walk-up flats, 3- and 4-story apartment buildings, a 1- to 2-story community center, and a 4-story parking structure. The redevelopment will be constructed in five phases. Except for Buildings A and A2, Parking Garage A, and Building D/Parking Garage D, the buildings will be entirely framed in wood. Building A, which will wrap around three sides of Parking Garage A, will consist of three stories of wood-framed construction above a one-story concrete podium. Parking Garage A will be four stories high. A portion of Building A2 will consist of a partial one-story podium with three stories of wood-framed residential units above the podium; the remainder of the building will consist of four stories of wood framing. Building D will consist of a two-story concrete podium with 2 to 3 stories of wood-framed residential units above the podium.

Redevelopment plans also include constructing interior roadways, new infrastructure, landscaping and courtyards, and a 3.3-acre park in the northwestern portion of the site.

Based on the results of our engineering analyses using the data from our cone penetration tests (CPTs), we conclude the primary geotechnical issues affecting the proposed redevelopment include: (1) the presence of up to about 10 feet of fill underlain by up to approximately 11 feet of a highly compressible marsh deposit in the northern portion of the site, and (2) providing uniform support for the proposed buildings.

Our investigation indicates the soils underlying the portion of the site south of Midway Drive have moderate to high strength and low to moderate compressibility. Therefore, we preliminarily conclude new buildings south of Midway Drive can be supported on conventional spread footings bottomed on well-compacted fill and/or native soil. The subsurface conditions north of Midway Drive vary significantly in both thickness of fill and the thickness of the marsh deposit. Based on our settlement analyses, we preliminarily conclude Building A, Garage A, and Building A2 should be supported on spread footings or a mat foundation bearing on improved soil to reduce differential settlements resulting from the consolidation of the marsh deposit, which varies from about 0 to 11 feet thick beneath Building A2 and 0 to 8 feet beneath Building A and Garage A. We preliminarily conclude Buildings B and B2 can be supported on mat foundations bottomed on two feet of recompacted fill.

This report presents our preliminary recommendations regarding foundation design, seismic design, and other geotechnical aspects of the project. The recommendations contained in our report are based on limited subsurface exploration and review of available data for the site, and are not intended for final design. Final geotechnical design values should be confirmed by a detailed geotechnical investigation. In addition, variations between expected and actual soil conditions may be found in localized areas during construction. Therefore, we should be engaged to observe ground improvement, foundation installation, and fill placement, 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,
ROCKRIDGE GEOTECHNICAL, INC.

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

Enclosure

ROCKRIDGE GEOTECHNICAL

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PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL REDEVELOPMENT
MIDWAY VILLAGE
Daly City, California

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed Midway Village Redevelopment project in Daly City, California. Midway Village is an existing residential development located on the eastern side of Schwerin Avenue and north of its intersection with Martin Street, as shown on the Site Location Map (Figure 1).

The site consists of 33 parcels that encompass a total area of about 15.55 acres. It is bordered by a PG&E property to the north, vacant land and a single-family home subdivision under construction to the east, Schwerin Street to the west, and Martin Street to the south. The site is currently occupied by 35 multi-unit two-story residential buildings, asphalt-paved parking lots and interior streets and drive aisles, concrete flatwork and landscaping. The northeastern corner of the site is currently occupied by Bayshore Park. The ground surface across the site slopes gently down to the north and east with ground surface elevations (National Geodetic Vertical Datum) ranging from approximately 80 feet in the southwestern corner of the site (at the intersection of Schwerin and Martin streets) to 15 feet in the northeastern corner of Bayshore Park.

Current redevelopment plans for the Midway Village Redevelopment project call for demolishing the existing structures and other improvements on the site and constructing a combination of 2- to 3-story townhomes, 2- to 3-story walk-up flats, 3- and 4-story apartment buildings, a 1- to 2-story community center, and a 4-story parking structure. The redevelopment will be constructed in five phases. Except for Buildings A and A2, Parking Garage A, and Building D/Parking Garage D, the buildings will be entirely framed in wood. Building A, which will wrap around three sides of Parking Garage A, will consist of three stories of wood-framed construction above a one-story concrete podium. Parking Garage A will be four stories high. A portion of Building A2 will consist of a partial one-story podium with three stories of wood-framed residential units above the podium; the remainder of the building will consist of four stories of wood framing. Building D will consist of a two-story concrete podium with 2 to 3 stories of wood-framed residential units above the podium.

Redevelopment plans also include constructing interior roadways, new infrastructure, landscaping and courtyards, and a 3.3-acre park in the northwestern portion of the site.

2.0 SCOPE OF SERVICES

Our preliminary geotechnical investigation was performed in accordance with our amended proposal dated November 14, 2019. Our scope of services consisted of reviewing available geotechnical and geologic data for the site and vicinity, exploring subsurface conditions for the proposed redevelopment by advancing 12 cone penetration tests (CPTs), and performing engineering analyses to develop preliminary conclusions and recommendations regarding:

- the most appropriate foundations type(s) for the proposed buildings
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlement
- lateral earth pressures for design of retaining walls
- subgrade preparation for slab-on-grade floors and concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- pavement sections for asphalt concrete and Portland cement concrete
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- 2019 California Building Code (CBC) site class and design spectral response acceleration parameters
- construction considerations.

3.0 FIELD INVESTIGATION

Our subsurface investigation consisted of performing 12 CPTs, designated as CPT-1 through CPT-12, at the approximate locations shown on the attached Site Plan (Figure 2). Prior to performing the CPTs, we obtained a drilling permit from the City of Daly City 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 buried utilities.

The CPTs were performed by Middle Earth Geo Testing, Inc. of Orange, California on January 15 and 16, 2020. The CPTs were advanced to depths of 50 to 51 feet below ground surface (bgs), except for CPT-10 which met refusal in very dense soil at a depth of 39 feet bgs. The CPTs were performed by hydraulically pushing a 1.7-inch-diameter cone-tipped probe with a projected area of 15 square centimeters into the ground. The cone-tipped probe measured tip resistance and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone continuously measured soil parameters for the entire depth advanced. Soil data, including tip resistance and frictional resistance, were recorded by a computer while the test was conducted. Accumulated data were processed by computer to provide engineering information such as the types and approximate strength characteristics of the soil encountered.

The CPT logs showing tip resistance, friction ratio, and pore pressure, as well as correlated soil behavior type, are presented in Appendix A on Figures A-1 through A-12. Upon completion, the CPTs were backfilled with cement grout.

4.0 SUBSURFACE CONDITIONS

A regional geologic map prepared by Graymer (2000), a portion of which is presented on Figure 3, indicates the northern portion of the site is underlain by artificial fill (af) and the southern portion of the site is underlain by Quaternary-age hillslope deposits (Qsl). Pleistocene-age alluvium is mapped along the northern edge of the site.

The area mapped as artificial fill in the northern portion of the site was formerly a marsh that extended inland from San Francisco Bay, as shown on Figure 3. The marsh was reportedly filled by the U.S. Federal Public Housing Authority after it took possession of the land in 1944. The report titled *Remedial Investigation Report for the Remedial Investigation/Feasibility Study and the Remedial Action Plan for Midway-Bayshore Site* prepared by Ecology and Environment, Inc., dated May 14, 1993, indicates the fill placed in the former marsh is up to about 10 feet thick, with the thickest fill occurring beneath the park in the northeastern portion of the site. Contours of the fill, which were shown on Figure 5-2 (Estimated Fill Thickness and Cross Section Locations) of the above-referenced report, are plotted on Figure 4. The data from CPT-1 through CPT-8, which were performed in the northern portion of the site, indicate the fill thickness is consistent with these contours. Where explored, the fill in the northern portion of the site consists predominantly of medium dense to dense sand, silty sand and clayey sand and stiff to very stiff clay. The Ecology and Environmental, Inc. report indicates the fill contains construction debris such as brick, metal, wood, glass and concrete.

The fill in the northern portion of the site is underlain by a marsh deposit consisting of soft to medium stiff clay with varying amounts of organics. We estimate the thickness of the marsh ranges from about 2 to 4 feet at the locations of CPT-1, CPT-4 and CPT-7. At the locations of CPT-2 and CPT-3, we estimate the marsh deposit is 7-1/2 and 11 feet thick, respectively. The marsh deposit was not encountered at the CPT-6 location. At the CPT-5 location, the four feet of fill is underlain by about five feet of stiff to very stiff clay. At a depth of nine feet bgs is about two feet of medium stiff clay, which may be interpreted to be the marsh deposit.

Beneath the marsh deposit at the locations of CPT-1 through CPT-5, CPT-7 and CPT-8, and below the ground surface at the CPT-6 location is heterogeneous alluvium consisting of interbedded medium dense to very dense sand with varying silt and clay content and stiff to hard clay that extends to the maximum depth explored of 50.5 feet bgs. The thickness of the sand, silty sand and clayey sand layers ranges from less than one foot up to about eight feet.

In the southern portion of the site (i.e., south of Midway Drive), there appears to be less than two feet of existing fill at the locations of the four CPTs (CPT-9 through CPT-12) performed in this area. Below depths of 0 to 2 feet bgs, the CPTs encountered alluvium consisting primarily of interbedded layers of very stiff to hard clay and medium dense to very dense clayey sand that extends to the maximum depth explored. As discussed above, CPT-10 met refusal in very dense soil (possibly bedrock) at a depth of about 39 feet bgs.

4.1 Groundwater

The depth to groundwater at the site is complex and varies both with location on the site and the depth of the water-bearing zone in which the measurements are taken. The geologic cross section (Figure 5-2) presented in the May 14, 1993 Ecology and Environmental, Inc. report referenced above shows the depth to groundwater on October 1, 1992 ranging from about one foot bgs at about the middle of the northern edge of the site to about 12-1/2 feet bgs along the western edge of the site just north of Midway Drive. The groundwater in the northeastern corner where the existing fill is thickest was about seven feet bgs. The reason for the shallow groundwater along the northern edge of the site is stated in the report as due to "groundwater mounding" due to frequent irrigation in the vicinity of the well.

We also reviewed groundwater levels measured in nearby off-site wells presented in the report prepared by Haley & Aldrich titled *Fourth Five-Year Review for Pacific Gas and Electric Company's Martin Service Center, 731 Schwerin Street, Daly City, California,* dated October 2015. The report includes groundwater-level measurements between April 24, 1987 and August 22, 2014 for wells both north and east of Midway Village. The report identifies three water-bearing zones referred to as fill, shallow and deep zones. The shallow zone refers to a 5- to 10-foot-thick layer of silty sand underlying about 10 to 15 feet of low-permeability alluvium and the deep zone refers to a sand layer below the marsh and alluvium at depths of 30 to 40 feet bgs. Based on measurements taken on August 22, 2014, the report states "the unconfined groundwater within the artificial fill is encountered at approximately 7 feet bgs, from a range of ground surface elevations". The report also states the groundwater flows from northwest to southeast in all three of the water-bearing zones. In one cluster of wells near the northeastern corner of the Midway Village site, the groundwater elevations in the shallow and deep zones were about 3 and 5 feet higher, respectively, than the groundwater elevation in the fill on August 22, 2014.

During our field investigation, Middle Earth Geo Testing attempted to obtain groundwater-level measurements at several of the CPT locations using pore pressure dissipation tests performed with the CPT probe; however, with the exception of tests performed at CPT-2 and CPT-9, the pore pressure did not equilibrate during the test due to the low permeability of the soil in which the test was performed. At the CPT-2 location, the pore pressure dissipation test was performed at a depth of 19.36 feet bgs and indicated an estimated groundwater depth of 13.9 feet bgs. At the CPT-9 location, the pore pressure dissipation test was performed at a depth of 32.81 feet bgs and resulted in an estimated groundwater depth of 2.1 feet bgs. The measurements at the CPT-2 and CPT-9 locations appear to be taken in the “shallow” and “deep” water-bearing zones, as described in the Haley & Aldrich report and, therefore, are probably not representative of the groundwater levels in the fill.

Based on our review of the groundwater data discussed above, the depth to groundwater in the existing fill blanketing the area north of Midway Drive fluctuated about 3-1/2 to 4-1/2 feet between 1987 and 2014 in the two monitoring wells closest to Midway Village. The shallowest depth to groundwater measured in the fill zone in the monitoring well closest to Midway Village was about four feet bgs in February 2000. For preliminary design, we recommend using a design groundwater depth of four feet bgs. It should be noted the monitoring well in which groundwater was measured at one foot bgs, as described above in the Ecology and Environment, Inc. report was screened in the “deep” water-bearing zone below the marsh deposit and, therefore, was not representative of the groundwater level in the fill.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity

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

The major active faults in the area are the San Andreas, San Gregorio, and Hayward faults. These and other faults in the region are shown on Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated mean characteristic Moment magnitude[1] [2007 Working Group on California Earthquake Probabilities (WGCEP) (USGS 2008) and Cao et al. (2003)] are summarized in Table 1.

[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.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Mean Characteristic Moment Magnitude
N. San Andreas - Peninsula	6.6	West	7.23
N. San Andreas (1906 event)	6.6	West	8.05
San Gregorio Connected	14	West	7.50
N. San Andreas - North Coast	17	West	7.51
Total Hayward	22	Northeast	7.00
Total Hayward-Rodgers Creek	22	Northeast	7.33
Monte Vista-Shannon	33	Southeast	6.50
Total Calaveras	38	East	7.03
Mount Diablo Thrust	38	East	6.70
Rodgers Creek	43	North	7.07
Green Valley Connected	43	East	6.80

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 88 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 2014 Working Group on California Earthquake Probabilities has compiled the earthquake fault research for the San Francisco Bay area in order to estimate the probability of fault segment rupture. They have determined that the overall probability of moment magnitude 6.7 or greater earthquake occurring in the San Francisco Region during the next 30 years (starting from 2014) is 72 percent. The highest probabilities are assigned to the Hayward Fault, Calaveras Fault, and the northern segment of the San Andreas Fault. These probabilities are 14.3, 7.4, and 6.4 percent, respectively.

5.2 Seismic Hazards

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[2] lateral spreading,[3] and cyclic densification[4]. We used the results of our field investigation to evaluate the potential of these phenomena occurring at the project site.

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

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

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

5.2.1 Ground Shaking

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

5.2.2 Ground Surface Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is located 7 km from the San Andreas Fault; however, 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.

5.2.3 Cyclic Densification

Seismically induced compaction or cyclic densification of non-saturated sand (sand above the groundwater table) caused by earthquake vibrations may result in differential settlement. The results of our preliminary field investigation indicate the granular soil above the groundwater table is generally not susceptible to cyclic densification due to its relative density and/or fines content. A zone of loose to medium dense sand to silty sand was encountered between depths of approximately 1-1/2 and 3-3/4 feet bgs at the CPT-6 location. We estimate ground surface settlement due to cyclic densification of this thin layer during a major earthquake would be less than 1/4 inch. The potential for settlement from cyclic densification at this location will be mitigated by overexcavating and recompacting two feet of soil below the foundation.

5.2.4 Liquefaction and Associated Hazards

Liquefaction is a phenomenon in which saturated soil temporarily loses strength from the build-up of excess pore water pressure, especially during earthquake-induced cyclic loading. 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.

A portion of the map titled *Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region,* prepared by the USGS in cooperation with the California Geological Survey (CGS), dated 2006, is shown on Figure 6. The map indicates the area mapped as artificial fill in the northern portion of the site is highly susceptible to liquefaction while the liquefaction potential of the native alluvium in the southern portion of the site is very low.

We evaluated the liquefaction potential at the site using data collected from our CPTs. Liquefaction susceptibility was assessed using the software CLiq v3.0 (GeoLogismiki, 2019). CLiq uses measured field CPT data and assesses liquefaction potential, including post-earthquake vertical settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). Our liquefaction analyses were performed using the methodology proposed by Boulanger and Idriss (2014). We also used the relationship proposed by Zhang, et al (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement; a relationship that is an extension of the work by Ishihara and Yoshimine (1992).

Our analyses were performed using an in-situ groundwater depth of four feet bgs and a “during earthquake” groundwater depth of four feet bgs. In accordance with the 2019 CBC, we used a peak ground acceleration of 0.79 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 isolated thin layers of potentially liquefiable silty sand and sandy silt at random depths in the native alluvium underlying the site. The layers are less than about one foot thick except for a two-foot-thick layer encountered between depths of 16 and 18 feet bgs at the CPT-10 location. Based on the results of our analyses, we estimate total and differential settlements associated with liquefaction after an MCE event generating a PGA_M of 0.79g will be up to about 1/2 inch and 1/4 inch over horizontal distance of 30 feet, respectively.

Based on the depth and thickness of the potentially liquefiable soil layers, we conclude the site is not susceptible to surface manifestations from liquefaction, such as sand boils. Considering the potentially liquefiable soil layers are not continuous, we conclude the risk of lateral spreading is very low.

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our preliminary field investigation, we conclude the site may be redeveloped as proposed. The primary geotechnical concerns for the proposed redevelopment are: (1) the presence of up to about 10 feet of fill underlain by up to approximately 11 feet of a highly compressible marsh deposit in the northern portion of the site, and (2) providing uniform support for the proposed buildings. These and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

6.1 Foundation and Settlement

Based on the data from our CPTs and review of existing subsurface data from the previous investigation, we have divided the site into “south of Midway Drive” and “north of Midway Drive” for the purposes of discussing foundation alternatives and providing preliminary foundations recommendations for the appropriate foundation types. Our preliminary conclusions and recommendations regarding foundations are presented in the following sections.

6.1.1 South of Midway Drive

Our investigation indicates the soil underlying the portion of the site south of Midway Drive has moderate to high strength and low to moderate compressibility. Therefore, we preliminarily conclude new buildings south of Midway Drive can be supported on conventional spread footings bottomed on well-compacted fill and/or native soil.

We preliminarily recommend that spread footings be designed using an allowable bearing pressure of 4,000 pounds per square foot (psf) for dead-plus-live loads; this pressure may be increased by one-third for total design loads, which include wind or seismic forces. Estimated total settlements will be on the order of 1/2 to 3/4 inch for wood-framed structures and 3/4 to 1 inch for Building D. We estimate differential settlements will be on the order of 1/2 to 3/4 inch over a 30-foot horizontal distance. Most of the settlement will occur during construction of the buildings. Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. Footings should extend at least 18 inches below the lowest adjacent soil subgrade.

Lateral loads may be resisted by a combination of friction along the base of the footing and passive resistance against the vertical faces of the footing. To compute lateral resistance, we recommend using an equivalent fluid weight of 300 pounds per cubic foot (pcf); 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 footing is in direct contact with soil. 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.

6.1.2 North of Midway Drive

The subsurface conditions north of Midway Drive vary significantly in both thickness of fill and the thickness of the marsh deposit. Based on our settlement analyses using our CPT data, we preliminarily conclude Building A, Garage A, and Building A2 should be supported on spread footings or a mat foundation bearing on improved soil to reduce differential settlements resulting from the consolidation of the marsh deposit, which varies from about 0 to 11 feet thick beneath Building A2 and 0 to 8 feet beneath Building A and Garage A. We preliminarily conclude Buildings B and B2 can be supported on mat foundations bottomed on two feet of recompacted fill. Recommendations for both spread footings on improved soil and mat foundations are presented below.

6.1.2.1 Spread Footings on Improved Soil

We preliminarily conclude proposed Buildings A and A2 and Garage A may be supported on shallow foundations, such as spread footings or mat, bearing on soil strengthened using ground improvement techniques. Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. Footings should extend at least 18 inches below the lowest adjacent soil subgrade. The edge of mat foundations should extend at least 12 inches below the lowest adjacent exterior finished grade.

Ground improvement can serve to stiffen the overall soil matrix by transferring foundation loads to more competent material below the existing fill and marsh deposit, thus reducing static and seismically induced settlements and providing increased bearing capacity for shallow foundations. Based on our experience, we believe the most appropriate ground improvement method for the site conditions consists of drilled displacement columns (DDCs). Drilled displacement 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 for off-haul. DDCs are installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of columns should be determined by the contractor, based on the desired level of improvement.

For preliminary design of spread footings or a mat foundation bearing on improved ground, we recommend assuming ground improvement elements will extend a minimum of 20 feet into the alluvium below the marsh deposit, resulting in DDSC columns ranging from about 20 to 40 feet across the building footprints. We anticipate the ground improvement should be capable of increasing the allowable bearing pressure for spread footings or a mat foundation to approximately 4,000 psf for dead-plus-live-loads and limiting combined static and seismic differential total settlement to less than one inch and differential settlement to less than 3/4 inch over a horizontal distance of 30 feet. The actual design allowable bearing pressures and estimated settlements should be determined by the design-build ground improvement contractor, as they will be based on the diameter, depth, and spacing of the ground improvement elements

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the shallow foundations and friction between the bottoms of the foundations and the supporting soil and ground improvement elements. To compute lateral resistance, we preliminarily recommend using an equivalent fluid weight (triangular distribution) of 300 pcf. Passive pressure in the upper one foot of soil should be neglected unless confined by a slab or pavement. The allowable base friction coefficient between the foundation and ground improvement elements should be determined by the ground improvement contractor, as it may be higher than that recommended for foundations on native (unimproved) soil, depending on the size and spacing of the ground improvement elements. Alternatively, the frictional resistance for footings may be computed using an allowable base friction coefficient of 0.35, which is conservative. For a mat foundation, an allowable base friction value of 0.20, which assumes a vapor retarder is placed between the bottom of the mat and tops of the ground improvement elements, should be used. 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.

6.1.2.2 Mat Foundation

We preliminarily conclude Buildings B and B2 may be supported on mat foundations underlain by at least two feet of engineered fill. For mat design, we preliminarily recommend using a modulus of subgrade reaction of 15 pounds per cubic inch (pci) for dead-plus-live loads: this value has already been scaled to take into account the plan dimensions of the foundation and may be increased by one-third percent for total load conditions.

Considering the large area of the mat, we expect the average bearing stress under the mat to be low; however, concentrated stresses will occur at column locations and at the edges of the mat. For preliminary design, an allowable dead-plus-live bearing pressure of 2,500 psf may be used; this pressure may be increased by one-third for total load conditions.

We estimate the total settlement of a mat-supported building with an average bearing pressure of 500 psf for dead-plus-live-load conditions will be approximately 1 to 1-1/2 inches and differential settlement would be approximately 3/4 inch over a horizontal distance of 30 feet.

To compute lateral resistance, we recommend using an equivalent fluid weight of 300 pcf; the upper foot of soil should be ignored unless confined by a slab or pavement. Assuming the mat is supported on a vapor retarder, a friction factor of 0.20 may be used to compute base friction. Where the mat foundation is supported directly on soil, a friction factor of 0.30 may be used. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without further reduction.

6.2 Slab-on-Grade Floors

Concrete slab-on-grade floors may be used for all the buildings south of Midway Drive and for Buildings A, Garage A, and Building A2. Where water vapor transmission through the floor slab is not desirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor 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 A vapor retarders stated in ASTM E1745. For the mat foundation option for Buildings B and B2, the four inches of capillary break material is not required. 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 should meet the gradation requirements presented in Table 2.

TABLE 2
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 mat and floor slab should have a w/c ratio of less than 0.45. Water should not be added to the concrete mix in the field. If necessary, workability should be increased by adding plasticizers. In addition, the mat/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.

6.3 Seismic Design

For design in accordance with the 2019 CBC, we preliminarily recommend Site Class D be used. It is possible the stiffer soils south of Midway Drive may be classified as Site Class C; however, a geophysical survey would be necessary to estimate the average shear-wave velocity of the upper 100 feet of soil (and possibly bedrock) to determine the appropriate site class. This survey could be performed during the final geotechnical investigation for the project.

The latitude and longitude of the site are 37.7020° and -122.4138°, respectively. For design in accordance with 2019 CBC, we preliminarily recommend the following:

- Site Class D
- S_S = 1.66g, S_1 = 0.67g

The 2019 CBC is based on the guidelines contained within ASCE 7-16 which stipulates that where S_1 is greater than 0.2 times gravity (g) for Site Class D, a ground motion hazard analysis is needed unless the seismic response coefficient (C_s) value will be calculated as outlined in Section 11.4.8, Exception 2. Assuming the C_s value will be calculated as outlined in Section 11.4.8, Exception 2, we recommend the following seismic design parameters:

- F_a = 1.0, F_v = 1.7
- S_{MS} = 1.66g, S_{M1} = 1.14g
- S_{DS} = 1.10g, S_{D1} = 0.76g
- Seismic Design Category D for Risk Factors I, II, and III

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

6.4 Site Preparation and Grading

Site demolition should include the removal of existing pavements, foundations, and underground utilities. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside of the proposed building footprint and will not interfere with the proposed construction, they may be abandoned in-place provided the lines are filled with lean concrete or cement grout to the property line. Voids resulting from demolition activities should be properly backfilled with compacted fill following the recommendations provided later in this section.

In areas that will receive improvements (i.e. building pads, exterior concrete flatwork, and new fill), the soil subgrade exposed following stripping and clearing 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 . If the subgrade is within eight inches of finished subgrade in areas to receive vehicular traffic, it should be moisture-conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction and be non-yielding. The soil subgrade should be kept moist until it is covered by fill or improvements.

Fill should consist of on-site soil or imported soil (select fill) that is free of organic matter, contains no rocks or lumps larger than three inches in greatest dimension, has a liquid limit of less than 40 and a plasticity index lower than 15, and is approved by the Geotechnical Engineer. Samples of proposed imported 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 horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction. Fill should be compacted to at least 95 percent relative compaction where the fill is greater than five feet in thickness or it consists of clean sand or gravel, defined as soil with less than five percent fines by weight. Fill placed within the upper foot of pavement subgrade should also be compacted to at least 95 percent relative compaction, and be non-yielding.

6.4.1 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with sand or fine gravel, which should be mechanically tamped.

Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted in accordance with the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than five percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Fill placed in the public right-of-way should be compacted in accordance with the City of Daly City Standard Specifications. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

6.4.2 Exterior Concrete Flatwork

Exterior concrete flatwork that will not receive vehicular traffic (i.e. sidewalk) should be underlain by at least four inches of Class 2 aggregate base compacted to at least 90 percent relative compaction. Prior to placement of the aggregate base, the upper eight inches of the subgrade soil should be scarified, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction.

6.4.3 Drainage and Landscaping

Positive surface drainage should be provided around the buildings to direct surface water away from foundations. To reduce the potential for water ponding adjacent to the buildings, we recommend the ground surface within a horizontal distance of five feet from the buildings slope down away from the buildings with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations.

6.5 Retaining Walls

Retaining walls should be designed to resist static lateral earth pressures, lateral pressures caused by earthquakes, and traffic loads (if vehicular traffic is expected within a horizontal distance equal to 1.5 times the wall height). All on-site walls, including low retaining walls in landscaped areas, should be designed in accordance with the recommendations presented in this section, although checking the walls for seismic loading is not required for walls less than six feet high. Retaining walls that are restrained from movement at the top or sides (e.g., a wall with a 90-degree turn) should be designed using the at-rest pressure presented in Table 3. Walls that are not restrained from rotation may be designed using the active pressure presented in Table 3.

TABLE 3
Lateral Earth Pressures for Retaining Wall Design

Soil Backfill Type	Active Static Condition (Unrestrained)	At-Rest Static Condition (Restrained)	Seismic Condition
On-site Soil - Drained	35 pcf[1]	55 pcf	35 pcf + 16 pcf
On-site Soil - Undrained	80 pcf	90 pcf	80 pcf + 8 pcf

1. Equivalent fluid weight (triangular distribution); pcf = pounds per cubic foot

The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads. If the retained soil will be sloped, we can provide additional recommendations after the degree to which the soil will be sloped has been determined. Where the below-grade walls are subject to traffic loading within a horizontal distance equal to 1.5 times the wall height, an additional uniform lateral pressure of 50 psf, applied to the entire height of the wall.

The design pressures recommended are based on fully drained walls. Although a majority of the retaining walls will be above the groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines, etc. One acceptable method for backdraining a retaining wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the retaining wall. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi NC or equivalent). The pipe should be connected to a suitable discharge point; a sump and pump system may be required to drain the collector pipes.

Wall backfill material and compaction should conform to the recommendations presented above in Section 6.4 of this report. Lightweight compaction equipment should be used to reduce stresses induced on the retaining walls during fill placement unless the walls are appropriately braced.

Site retaining walls may be supported on spread footings bottomed on one foot of engineered fill compacted to at least 90 percent relative compaction. The footings should be bottomed at least 18 inches below the lowest adjacent finished grade. The allowable bearing pressure, friction factor, and passive pressure presented for mat foundation design in Section 7.2 may be used for design of site retaining walls.

6.6 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt-concrete pavement sections. Based on our experience, we assumed an R-value of 40 for the near-surface soil, which consists mostly of silty sand. Several R-value tests should be performed on the near-surface soil during the final geotechnical investigation. Preliminary pavement design recommendations for asphalt-concrete pavements for the assumed R-value of 40 are presented below in Table 4.

TABLE 4
AC Pavement Sections

TI	Asphaltic Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4.5	2.5	6.0
5.0	3.0	6.0
5.5	3.0	6.0
6.0	3.5	6.0
6.5	4.0	6.0
7.0	4.0	7.0

The soil subgrade beneath AC pavements should be scarified to a depth of eight inches, moisture-conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction. In addition, the subgrade should be a firm and non-yielding surface. The subgrade should be proof-rolled to confirm it is non-yielding prior to placing the aggregate base. The Class 2 aggregate base should be moisture-conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction.

6.7 Portland Cement Concrete Pavement

The PCC pavement section design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle of 32,000 pounds. The recommended PCC pavement section for these axle loads is six inches of Portland cement concrete over six inches of Class 2 aggregate base. For PCC pavement areas that will not receive truck traffic, a PCC pavement section consisting of five inches of Portland cement concrete over six inches of Class 2 aggregate base may be used. The modulus of rupture and unconfined compressive strength of the concrete should be at least 500 and 3,000 pounds per square inch (psi) at 28 days, respectively. Contraction joints should be placed at a 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10.

The soil subgrade beneath PCC pavements should be scarified to a depth of eight inches, moisture-conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction to provide an unyielding surface. The Class 2 aggregate base should be moisture-conditioned to near optimum moisture content and compacted to at least 95 percent relative compaction.

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 and the final proposed development. Additional borings and CPTs will be required to further evaluate the subsurface conditions beneath the site. 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, ground improvement installation, 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.

8.0 LIMITATIONS

This preliminary geotechnical investigation has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The preliminary recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the exploratory CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The preliminary foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

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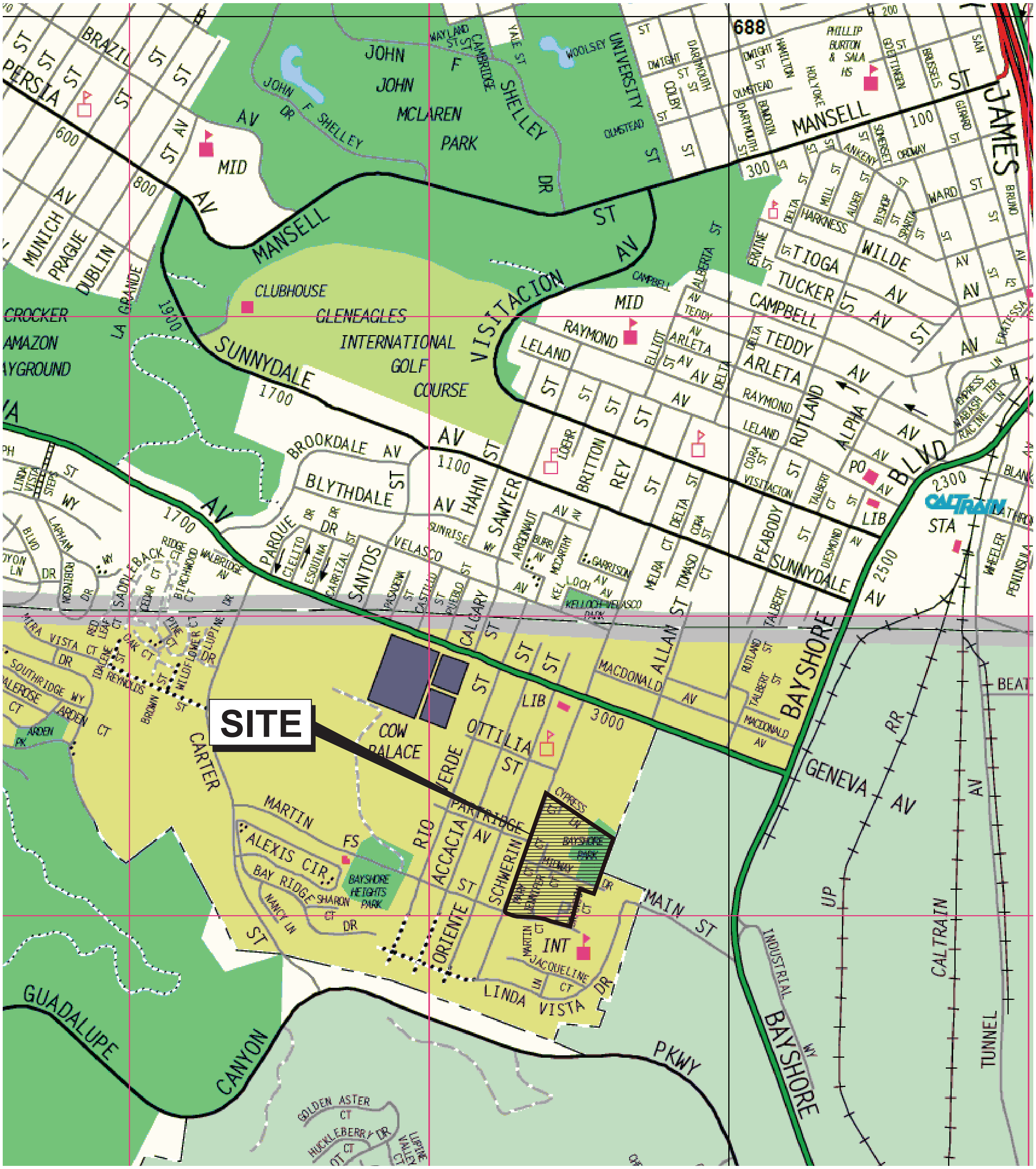
ROCKRIDGE GEOTECHNICAL

FIGURES



Base map: The Thomas Guide
San Francisco County
2002

0 1/4 1/2 Mile
Approximate scale

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 01/21/20	Project No. 18-1569	Figure 1

EXPLANATION

CPT-1 ▲ Approximate location of cone penetration test by Rockridge Geotechnical Inc., January 16, 2019

— — — Project limits

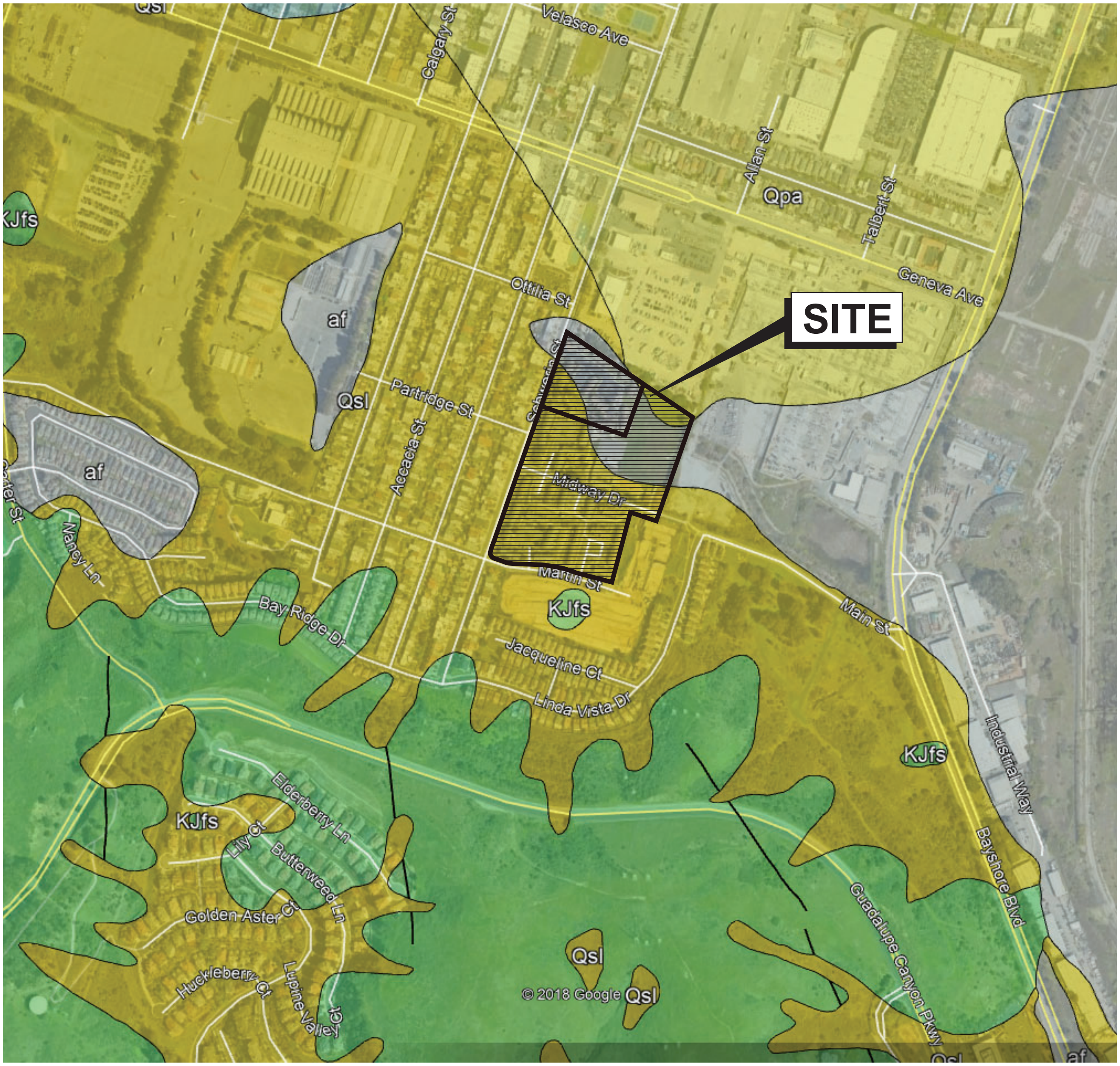
Reference: Base map from a drawing titled "Midway Village Site Plan", by Daivd Baker Architects, undated.

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

SITE PLAN

Date 02/05/20	Project No. 18-1569	Figure 2

ROCKRIDGE GEOTECHNICAL



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

EXPLANATION

af	Artificial Fill
Qsl	Hillslope Deposits (Quaternary)
Qpa	Alluvium (Pleistocene)
KJfs	Franciscan Complex sedimentary rocks (Early Cretaceous and (or) Late Jurassic)

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

N

0 800 1600 Feet

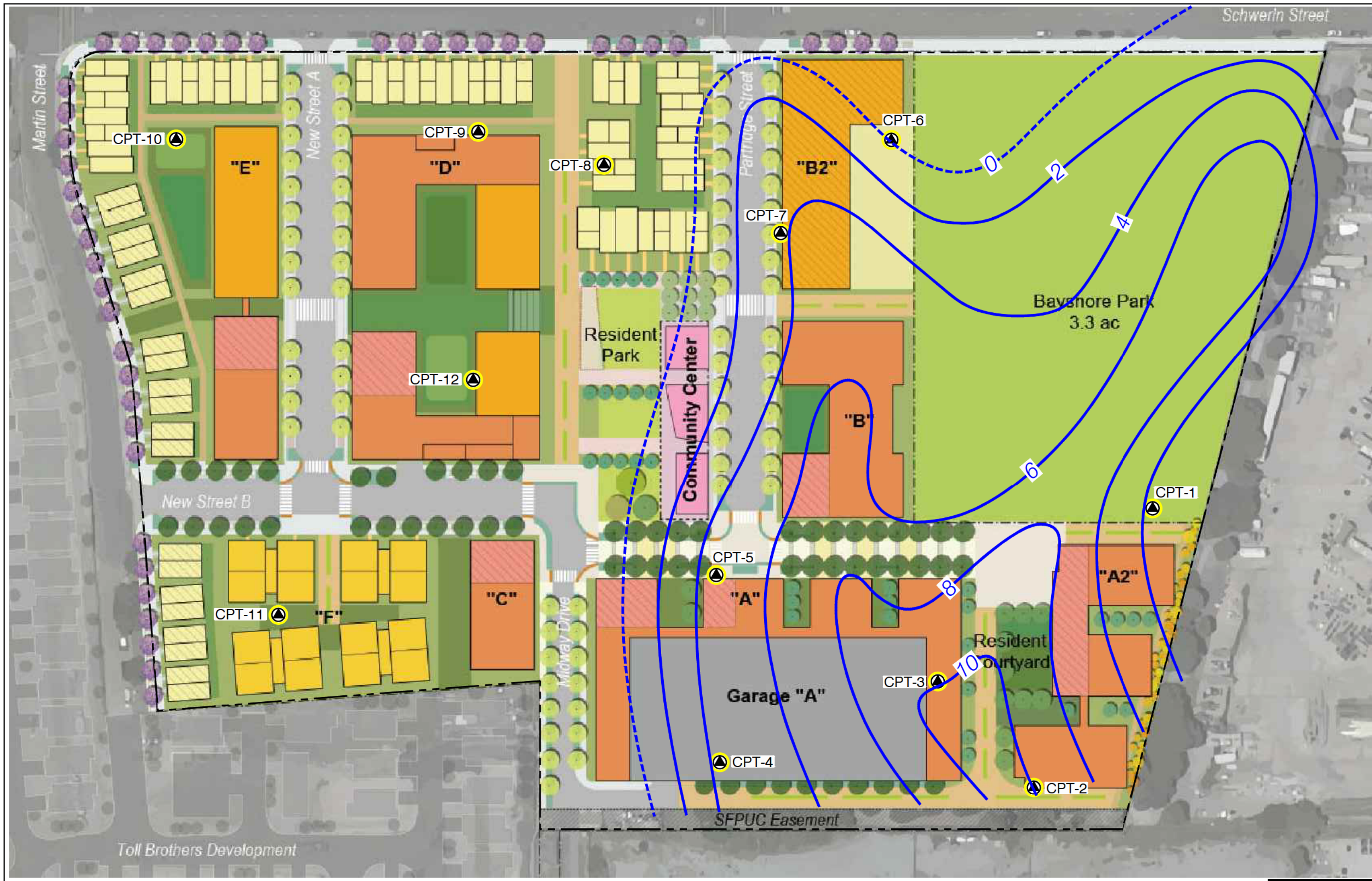
Approximate scale

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 01/29/20 | Project No. 18-1569 | Figure 3



EXPLANATION

CPT-1 Approximate location of cone penetration test by Rockridge Geotechnical Inc., January 16, 2019

2 Approximate existing fill thickness contours in feet from Figure 5-2 in report titled "Remedial Investigation Report for the Remedial Investigation/Feasibility Study and the Remedial Action Plan for Midway-Bayshore Site, prepared by Ecology and Environment, Inc., dated May 14, 1993

Project limits

Reference: Base map from a drawing titled "Midway Village Site Plan", by Daivd Baker Architects, undated.

0 100 Feet

Approximate scale

MIDWAY VILLAGE 47 MIDWAY DRIVE Daly City, California		
SITE PLAN SHOWING ESTIMATED THICKNESS OF EXISTING FILL		
Date 02/05/20	Project No. 18-1569	Figure 4

ROCKRIDGE GEOTECHNICAL

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

EXPLANATION

Strike slip

Thrust (Reverse)

Normal

0 5 10 Miles

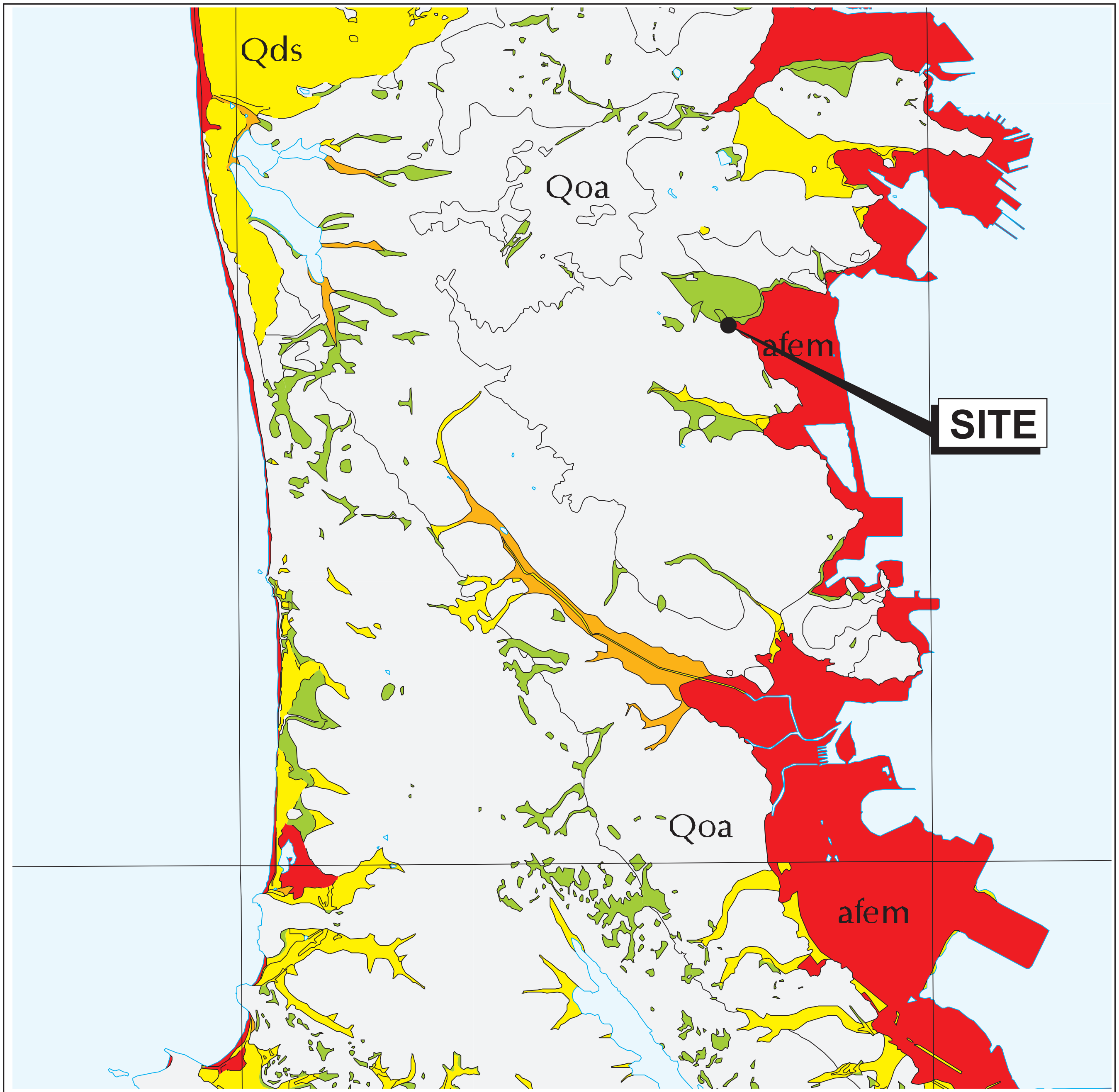
Approximate scale

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 01/23/20	Project No. 18-1569	Figure 5



LIQUEFACTION SUSCEPTIBILITY

VERY HIGH

HIGH

MODERATE

LOW

VERY LOW

Lines

Contact, dashed where location uncertainty is greater than about +- 100 m.

0 1.5 3 Miles

Approximate scale

Reference:
Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, by USGS, 2006

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

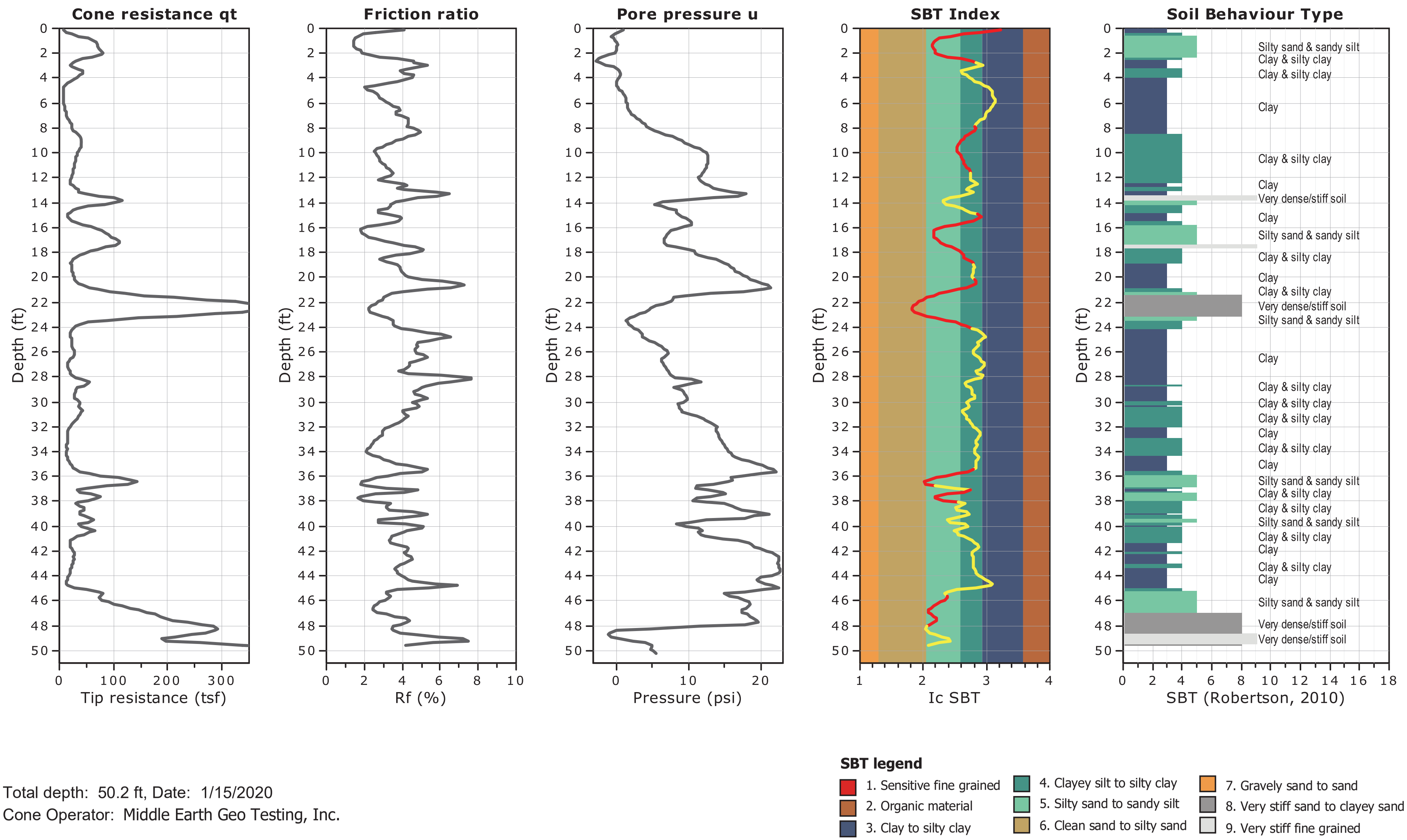
LIQUEFACTION SUSCEPTIBILITY MAP

Date 02/04/20 | Project No. 18-1569 | Figure 6

ROCKRIDGE GEOTECHNICAL

APPENDIX A

Cone Penetration Test Results



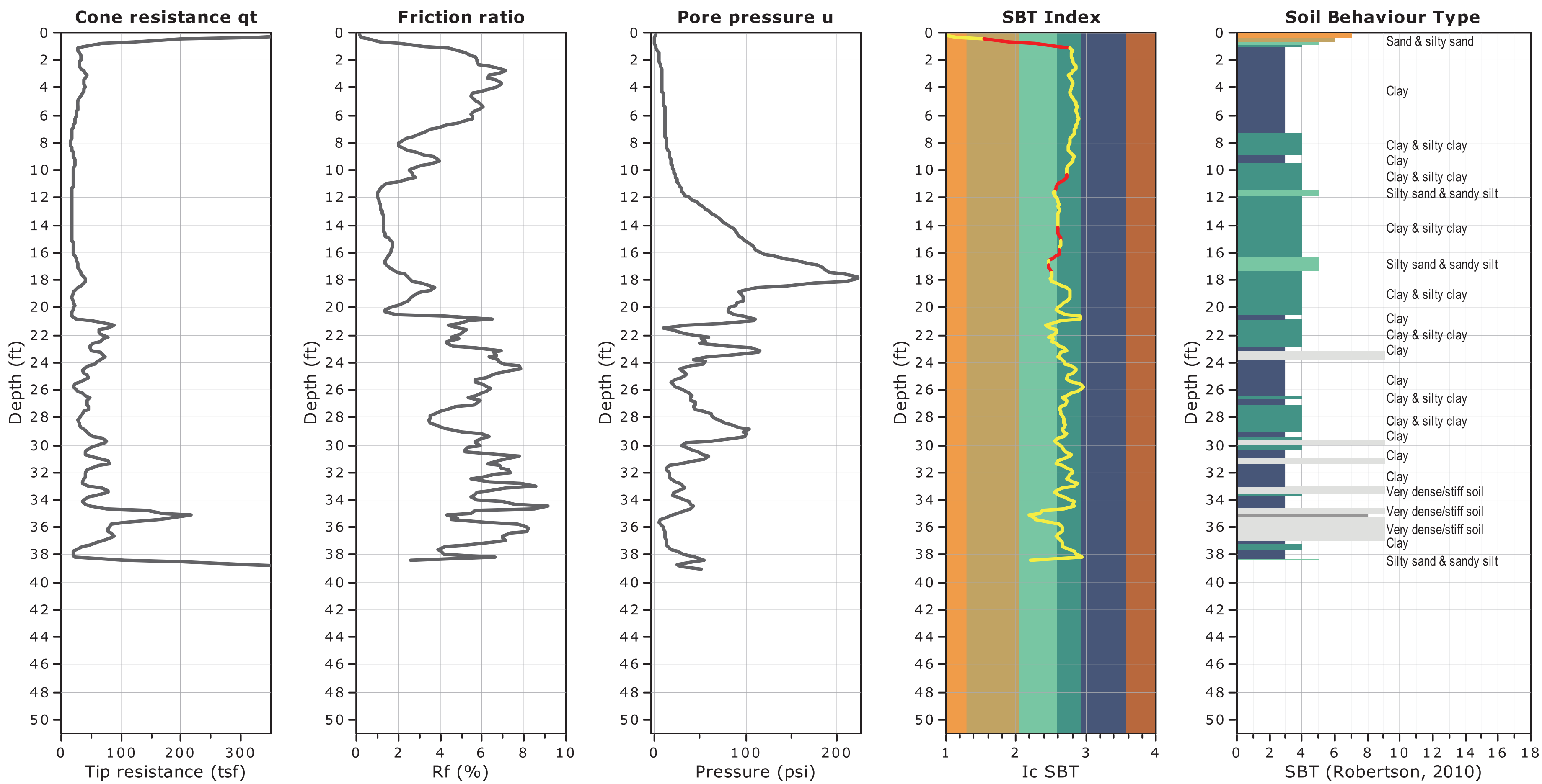
Total depth: 50.2 ft, Date: 1/15/2020
Cone Operator: Middle Earth Geo Testing, Inc.

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 01/29/20	Project No. 18-1569	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

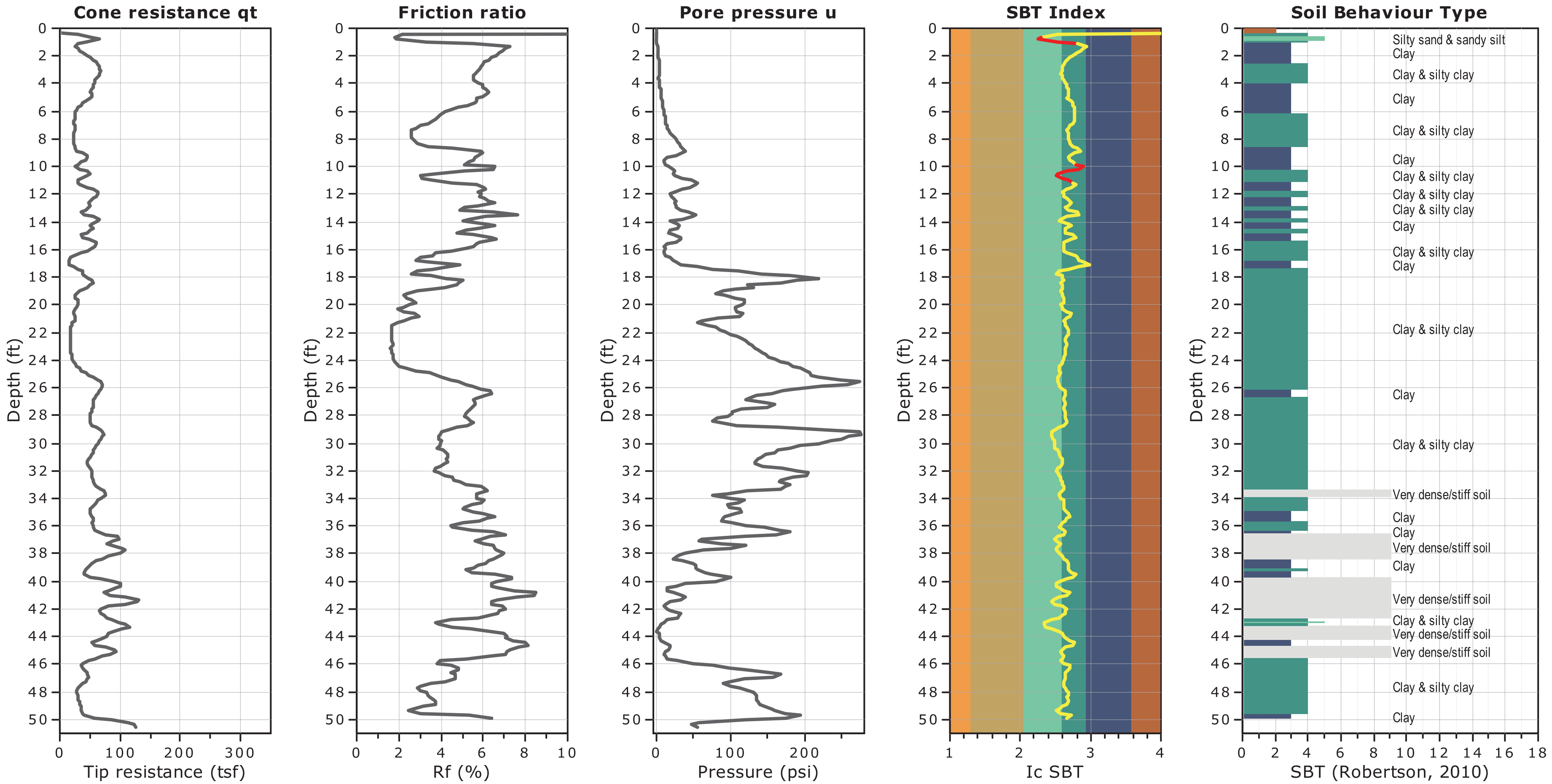
Total depth: 39.0 ft, Date: 1/16/2020
Cone Operator: Middle Earth Geo Testing, Inc.

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-10

Date 01/23/20	Project No. 18-1569	Figure A-10



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

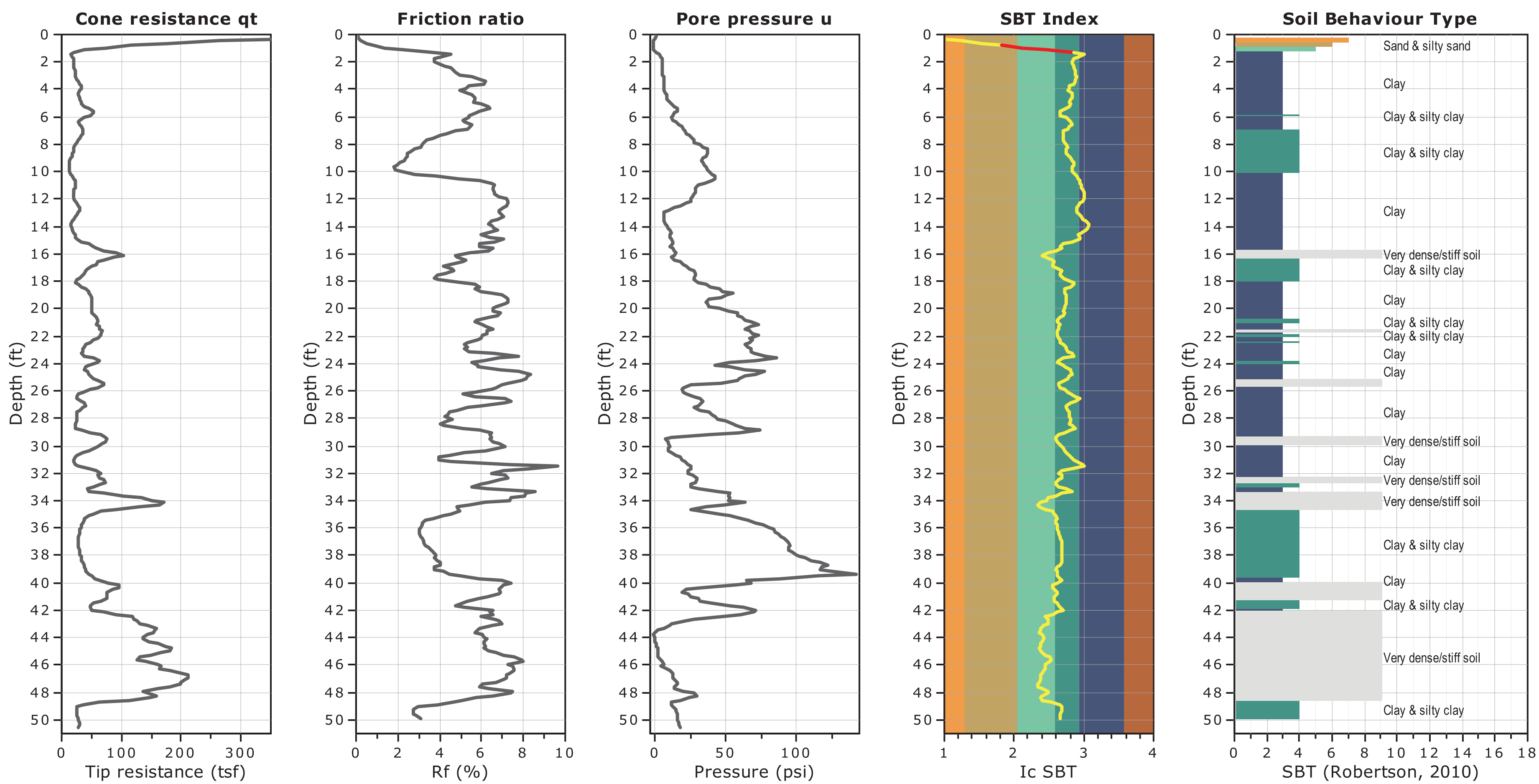
Total depth: 50.5 ft, Date: 1/16/2020
Cone Operator: Middle Earth Geo Testing, Inc.

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-11

Date 01/23/20	Project No. 18-1569	Figure A-11



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

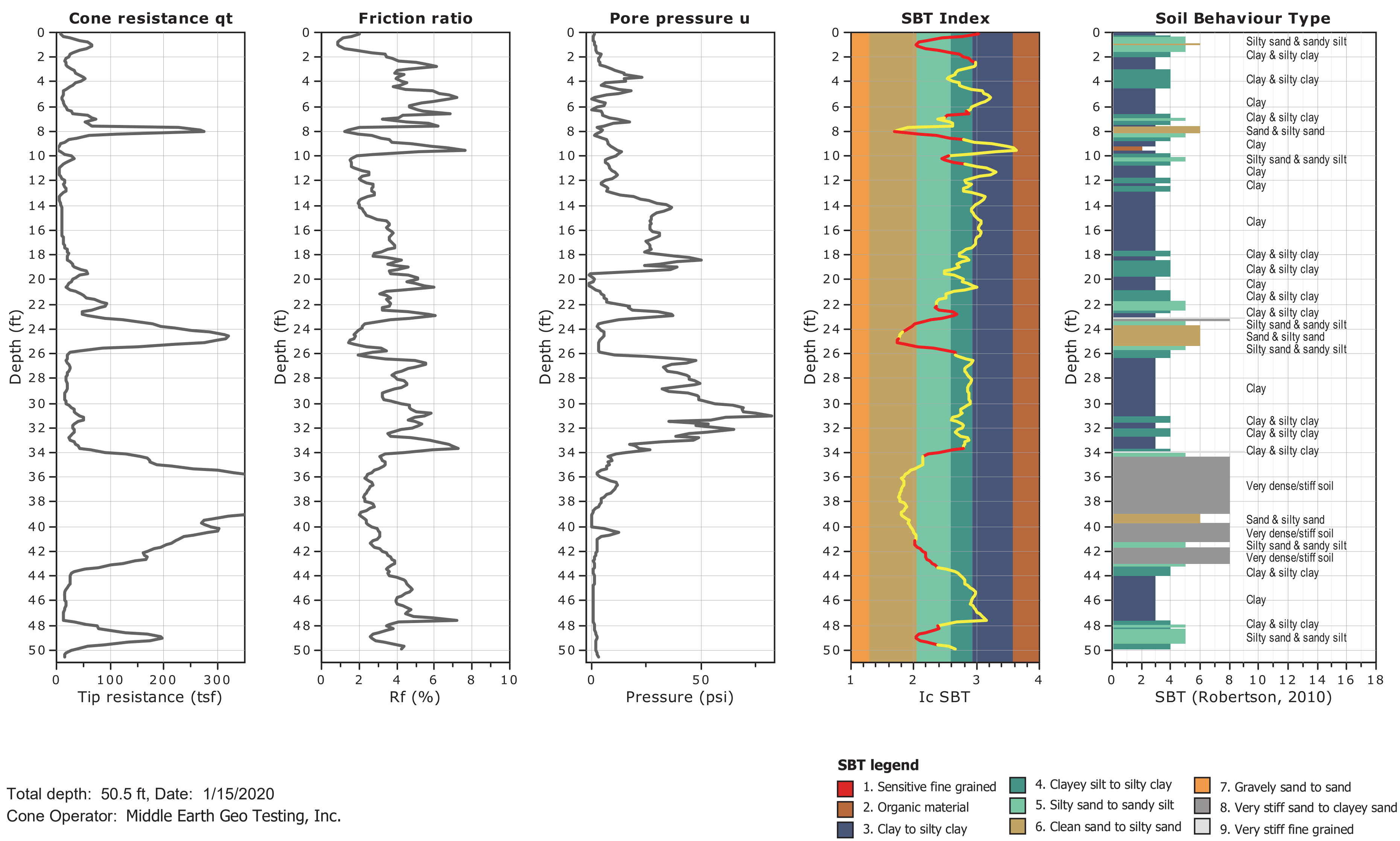
Total depth: 50.5 ft, Date: 1/16/2020
Cone Operator: Middle Earth Geo Testing, Inc.

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-12

Date 01/23/20	Project No. 18-1569	Figure A-12



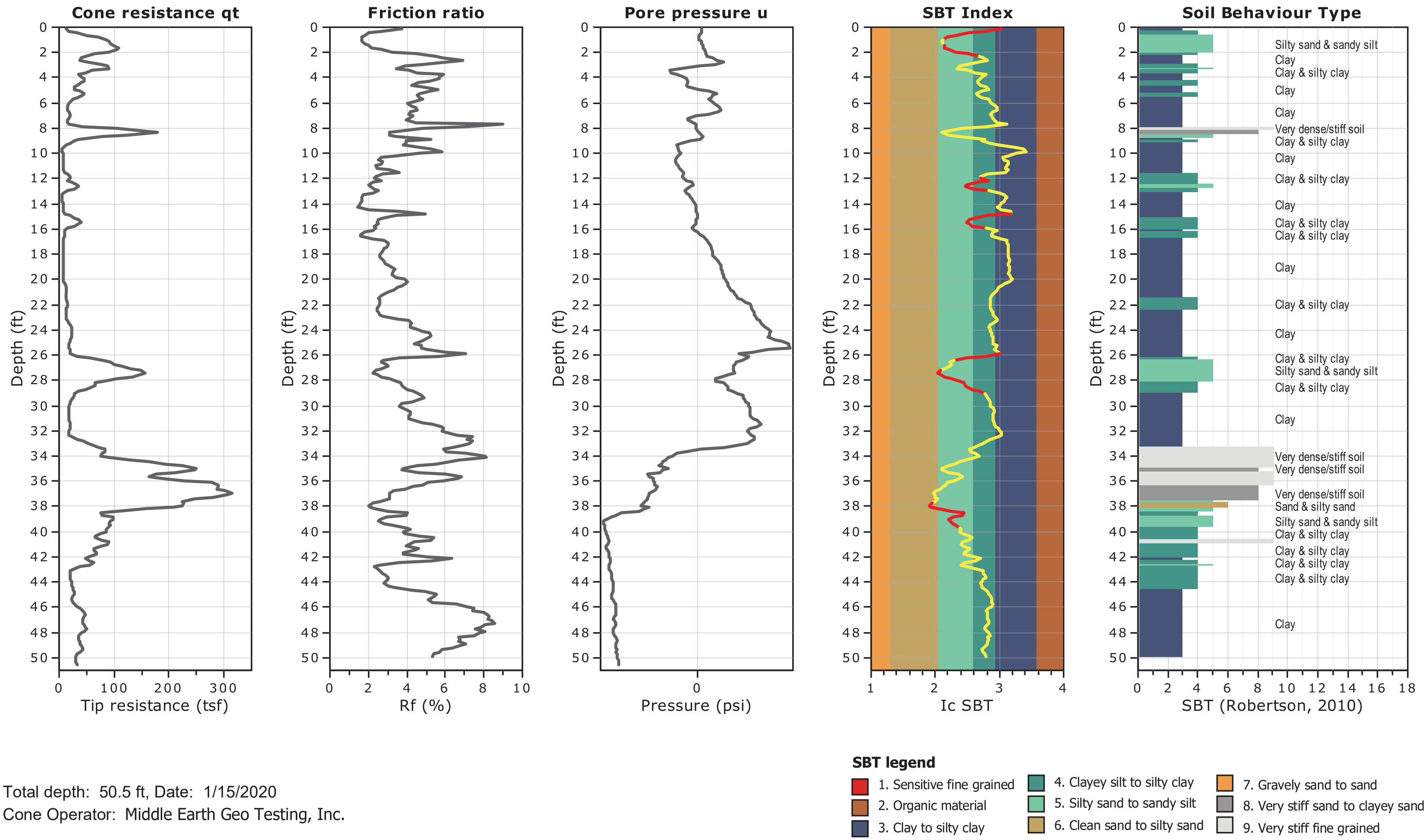
Total depth: 50.5 ft, Date: 1/15/2020
Cone Operator: Middle Earth Geo Testing, Inc.

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 01/29/20	Project No. 18-1569	Figure A-2



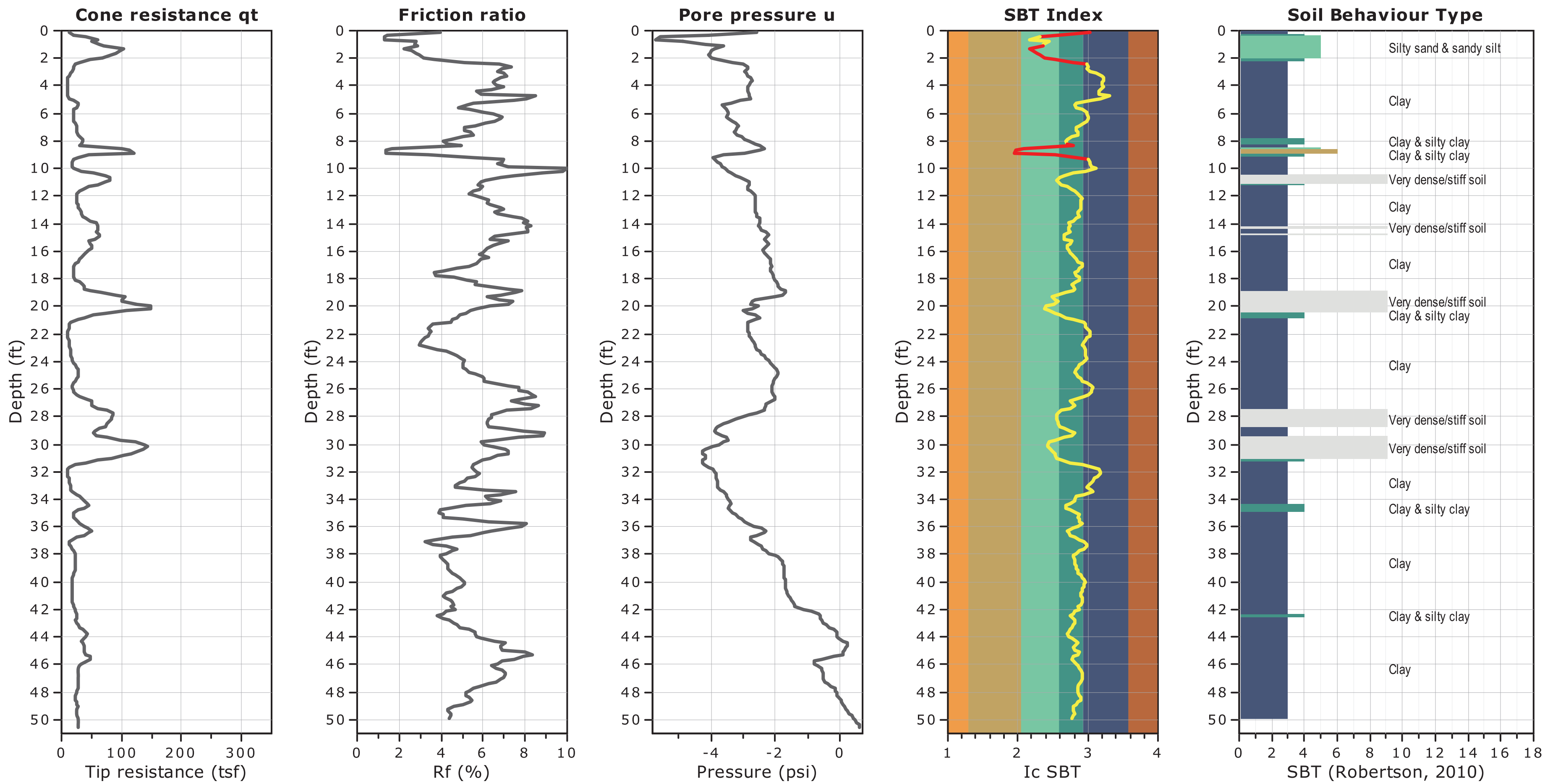
Total depth: 50.5 ft, Date: 1/15/2020
Cone Operator: Middle Earth Geo Testing, Inc.

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 01/29/20	Project No. 18-1569	Figure A-3



Total depth: 50.5 ft, Date: 1/15/2020
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

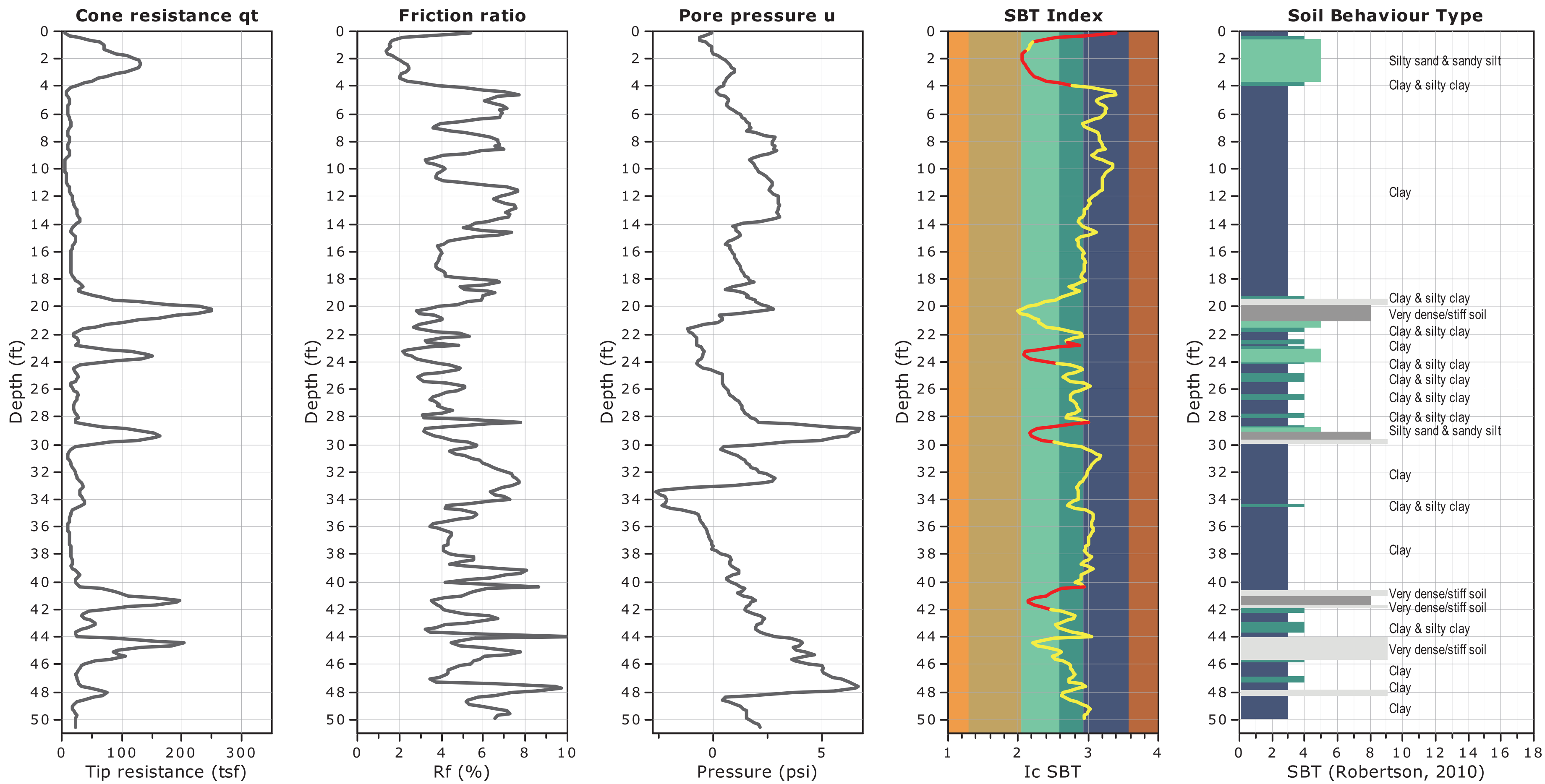
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 01/29/20	Project No. 18-1569	Figure A-4



Total depth: 50.2 ft, Date: 1/15/2020
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

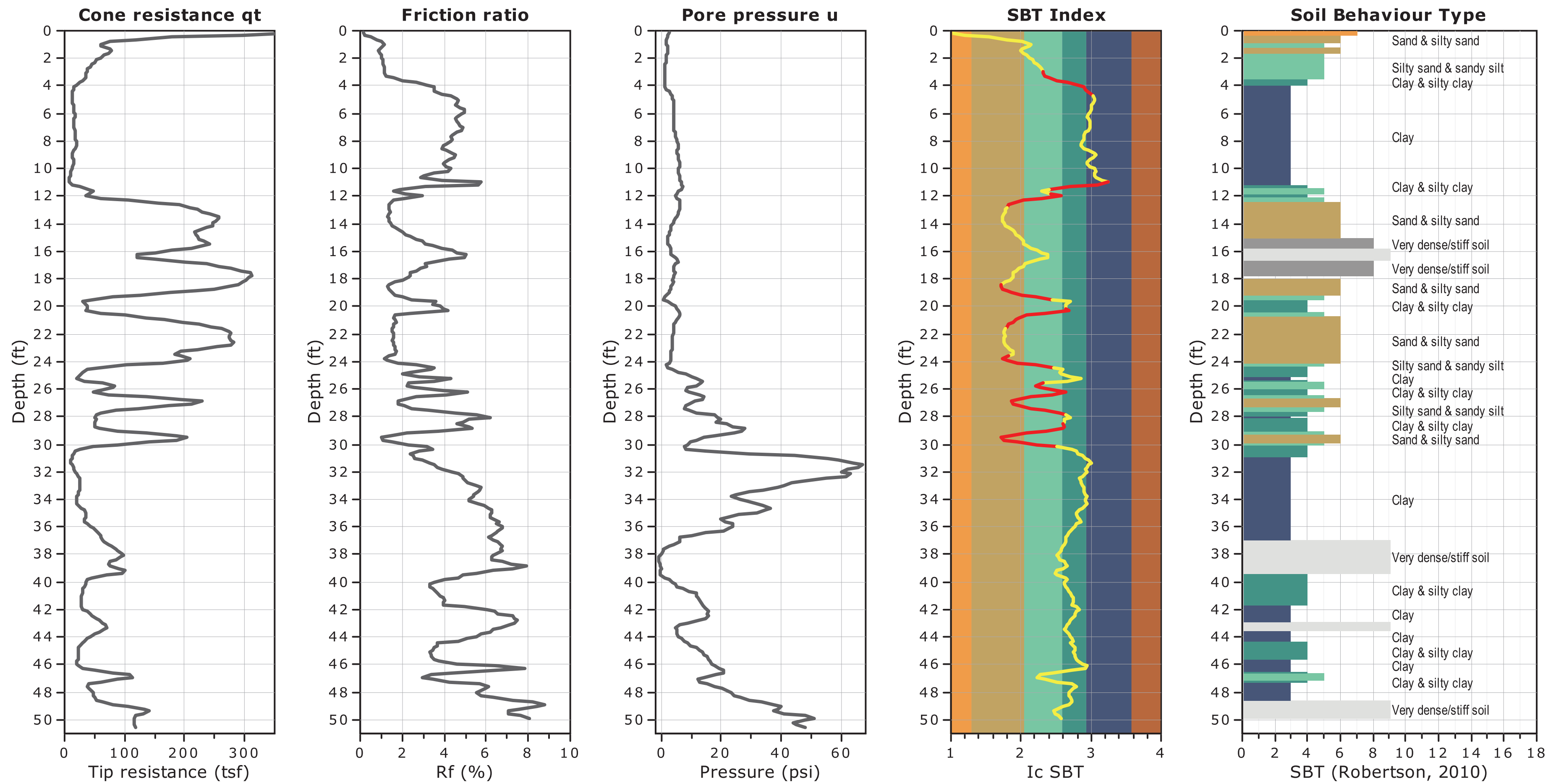
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-5

Date 01/29/20 | Project No. 18-1569 | Figure A-5

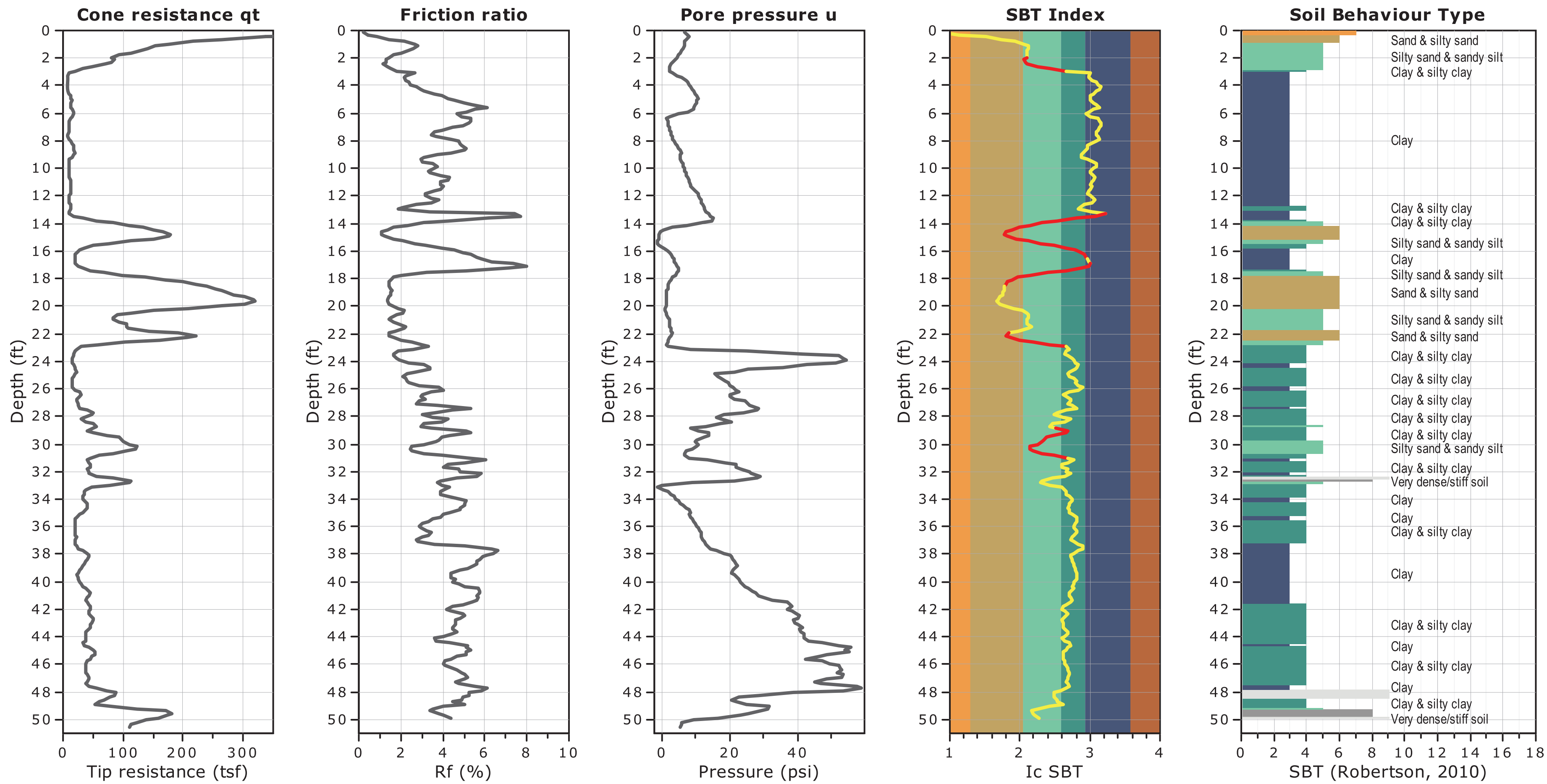


SBT legend

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: 50.5 ft, Date: 1/16/2020
Cone Operator: Middle Earth Geo Testing, Inc.

MIDWAY VILLAGE 47 MIDWAY DRIVE Daly City, California	CONE PENETRATION TEST RESULTS CPT-6
ROCKRIDGE GEOTECHNICAL	Date 01/23/20 · Project No. 18-1569 · Figure A-6



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

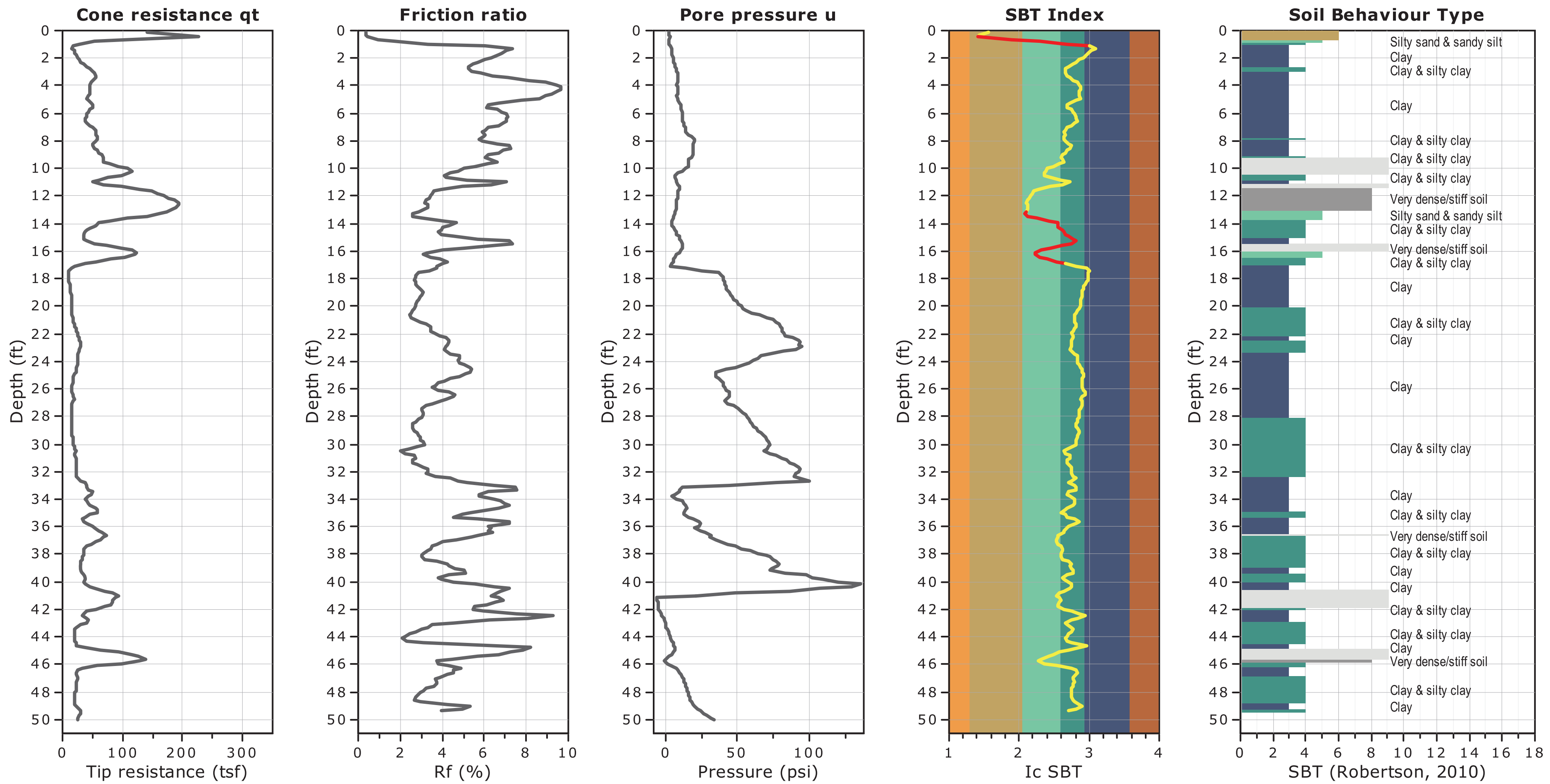
Total depth: 50.5 ft, Date: 1/16/2020
Cone Operator: Middle Earth Geo Testing, Inc.

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-7

Date 01/23/20	Project No. 18-1569	Figure A-7



Total depth: 50.0 ft, Date: 1/16/2020
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

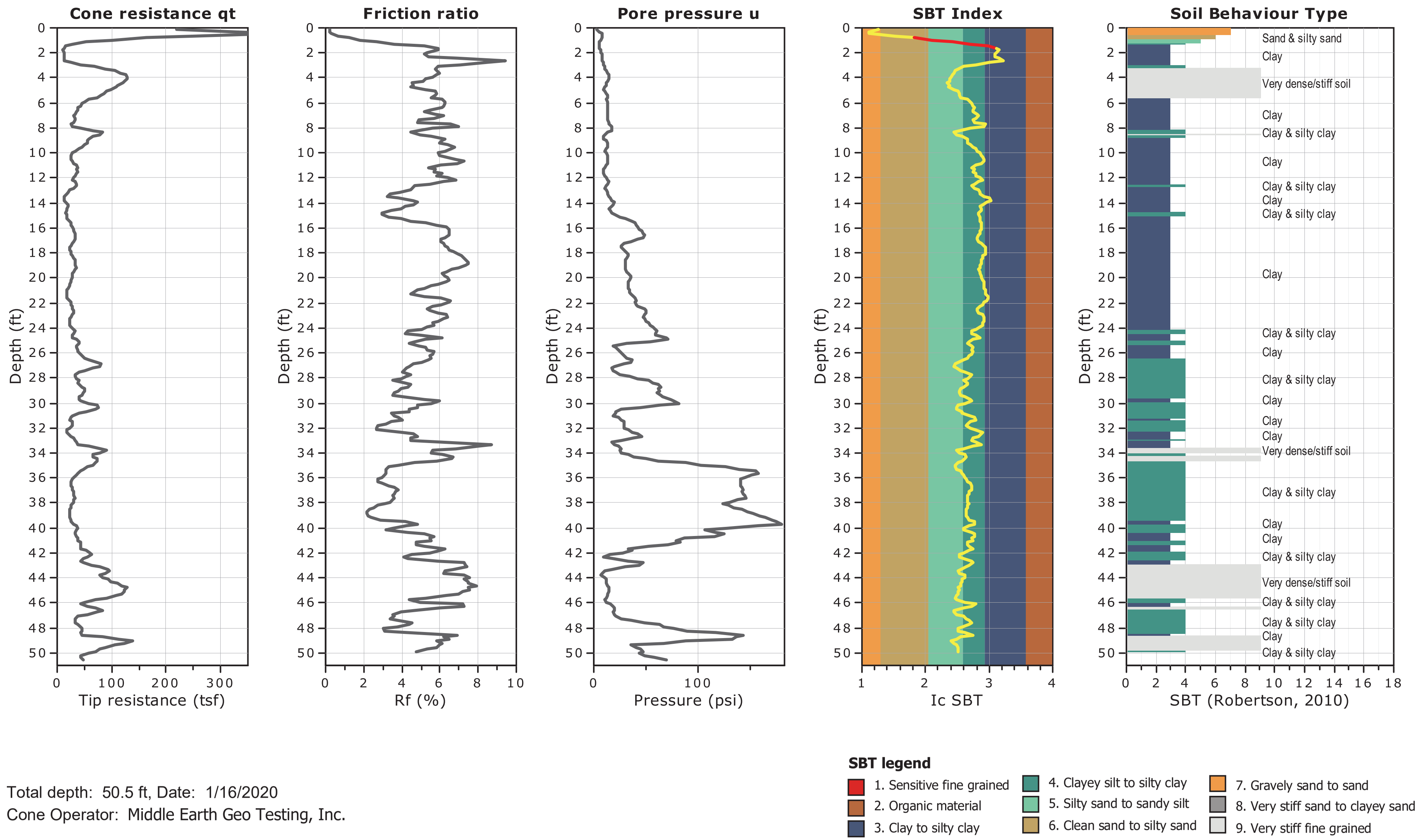
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

MIDWAY VILLAGE
47 MIDWAY DRIVE
Daly City, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-8

Date 01/23/20 | Project No. 18-1569 | Figure A-8



Total depth: 50.5 ft, Date: 1/16/2020
Cone Operator: Middle Earth Geo Testing, Inc.

MIDWAY VILLAGE **47 MIDWAY DRIVE** Daly City, California	**CONE PENETRATION TEST RESULTS** **CPT-9**
ROCKRIDGE GEOTECHNICAL	Date 01/23/20 | Project No. 18-1569 | Figure A-9