ROCKRIDGE GEOTECHNICAL

Prepared for **Sares Regis Group of Northern California**

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
HAYWARD PARK STATION
SAN MATEO, CALIFORNIA

June 7, 2019
Project No. 15-972

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June 7, 2019
Project No. 15-972

Mr. Ken Busch
Sares Regis Group of Northern California
901 Mariners Island Blvd. #700
San Mateo, California 94404

Subject: Preliminary Geotechnical Investigation Report
Proposed Residential Development
Hayward Park Station
San Mateo, California

Dear Mr. Busch:

We are pleased to present our preliminary geotechnical investigation report for the proposed residential development to be constructed at the Hayward Park Station in San Mateo, California. Our preliminary geotechnical investigation was performed in accordance with our Authorization to Provide Geotechnical Services dated August 17, 2015 and our contract with Sares Regis Group of Northern California, dated October 12, 2015.

The project site consists of a triangular-shaped relatively level lot that is bordered by Concar Drive to the southeast, railroad tracks to the southwest, and asphalt-paved parking for the adjacent property to the northeast. The site is currently occupied by asphalt-paved parking for the CalTrain Station and the adjacent property to the east is currently under construction. We understand plans are to construct a residential development consisting of two five-story, at-grade buildings, which will occupy most of the site. The southeasternmost building, designated as Building A, will consist of three levels of wood-framed, residential units over two levels of reinforced-concrete parking. The northwesternmost building, designated as Building B, will consist of five levels of wood-framed residential units. Other proposed improvements include interior driveways, concrete flatwork, and landscaping.

From a geotechnical standpoint, we preliminarily conclude the site can be developed as planned. The primary geotechnical concerns are: (1) the potential for up to one inch and 1-3/4 inches of total settlement due to a combination of liquefaction and cyclic softening beneath the proposed Building A and Building B, respectively; (2) the presence of relatively weak and moderately to highly compressible clay deposits underlying the site; and (3) providing adequate vertical and lateral support for the proposed structures. We preliminarily conclude that the proposed new building B may be supported on footings bearing on improved soil or a deep foundation system. We preliminarily conclude that the proposed new Building A may be supported on a stiffened foundation system, such as a conventional reinforced concrete mat or interconnected continuous footings (i.e., a stiffened grid), provided the settlement (static plus seismic) is acceptable from a structural and architectural standpoint.

This report presents preliminary conclusions and recommendations regarding geotechnical aspects of the project. A final geotechnical investigation, potentially including additional CPTs and borings, 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.

REGISTERED PROFESSIONAL ENGINEER TESSA E. WILLIAMS C 84312 EXP. 9/30/19 CIVIL STATE OF CALIFORNIA

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Enclosure

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Boring Logs, Cone Penetration Test Results, and Laboratory Test Results by Others

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESDIENTIAL DEVELOPMENT
HAYWARD PARK STATION
San Mateo, California

1.0 INTRODUCTION

This report presents the results of the preliminary geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed residential development to be constructed at the Hayward Park Station in San Mateo, California. The site is located on the northern corner of the intersection of Concar Drive and Pacific Boulevard, as shown on the Site Location Map, Figure 1.

The project site is a triangular-shaped relatively level lot that is bordered by Concar Drive to the southeast, railroad tracks to the southwest, and asphalt-paved parking for the adjacent property to the northeast. The site is currently occupied by asphalt-paved parking for the CalTrain Station and the adjacent property to the east is currently under construction. We understand plans are to construct a residential development consisting of two five-story, at-grade buildings, which will occupy most of the site. The southeasternmost building, designated as Building A, will consist of three levels of wood-framed, residential units over two levels of reinforced-concrete parking. The northwesternmost building, designated as Building B, will consist of five levels of wood-framed residential units. Other proposed improvements include interior driveways, concrete flatwork, and landscaping.

2.0 SCOPE OF SERVICES

Our preliminary geotechnical investigation was performed in accordance with our Authorization to Provide Geotechnical Services dated August 17, 2015 and our Contract with Sares Regis Group of Northern California, dated October 12, 2015. Our scope of work consisted of reviewing previous geotechnical reports within the site vicinity, evaluating subsurface conditions at the site by performing six cone penetration tests (CPTs), and performing engineering analyses to develop preliminary conclusions and recommendations regarding:

- soil and groundwater conditions

- 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
- the most appropriate foundation type(s) for the proposed structures
- preliminary design criteria for the recommended foundation type(s), including vertical and lateral capacities for each of the foundation type(s)
- estimates of foundation settlement under static and seismic loads
- lateral earth pressures for design of below-grade walls
- 2016 and 2019 California Building Code (CBC) site class and design spectral response acceleration parameters.

3.0 DATA REVIEW

In 2007, Ove Arup & Partners California Ltd (Arup) performed a geotechnical investigation and prepared a report titled *EBL&S, Hayward Park Green, Preliminary Geotechnical Investigation, San Mateo, California* for an adjacent site at the western corner of Concar Drive and South Delaware Street. Arup's investigation included drilling seven test borings to depths ranging from 20.5 to 80.5 feet below the ground surface (bgs), advancing seven CPTs to depths ranging from 30 to 60 feet bgs, and performing laboratory tests on soil samples obtained from the borings. We reviewed the results of the borings, CPTs, and laboratory tests presented in Arup's report and used pertinent data in our engineering analyses.

4.0 FIELD INVESTIGATION

Subsurface conditions at the site were investigated on October 29, 2015 by performing six CPTs, designated as CPT-1 through CPT-6, at the approximate locations shown on the Site Plan, Figure 2. Prior to mobilizing to the site, we contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained a private utility locator to check for existing utilities at each CPT location. We also obtained a drilling permit from San Mateo County Environmental Health Services (SMCEHS) as well as a service agreement with San Mateo County Transit District (SamTrans) and a Right of Entry Permit Agreement with Peninsula Corridor Joint Powers Board (JPB).

Middle Earth Geo Testing, Inc. of Orange, California advanced each CPT to a depth of approximately 50 feet bgs, with the exception of CPT-3 and CPT-5. CPT-5 met refusal in very dense granular soil at a depth of about 37 feet bgs and CPT-3 met refusal due to an obstruction at approximately 2 feet bgs. The CPTs were advanced by hydraulically pushing a 1.4-inch-diameter cone-tipped probe with a projected area of 10 square centimeters into the ground. The cone 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 computer to provide engineering information such as the soil behavior types, approximate strength characteristics, and liquefaction potential of the soil encountered. Upon completion, the CPTs were backfilled with cement grout in accordance with SMCEHS requirements.

The CPT logs, showing tip resistance, friction ratio, and pore water pressure with depth, as well as interpreted soil behavior types, are presented in Appendix A on Figures A-1 through A-5.

5.0 SUBSURFACE CONDITIONS

The geologic map prepared by Brabb (1998), a portion of which is presented on Figure 3, indicates the site is underlain by artificial fill (af). Based on the results of our CPTs and our review of the logs from the borings drilled by Arup on the adjacent site, we conclude the site is blanketed by approximately 3 to 4 feet of undocumented fill generally consisting of medium dense to dense sand with varying clay and gravel content. The fill is underlain by medium stiff to very stiff clay with thin, interbedded layers of medium dense to dense sand and gravel with varying fines content to the maximum depth explored within the site vicinity of approximately 61 feet bgs. The clay deposits generally grade to very stiff to hard below a depth of approximately 30 to 35 feet bgs across the site.

Results of Atterberg limits testing performed by Arup on samples of clay between depths of 2 and 4 feet bgs indicate the material is highly to very highly expansive. Expansive clay is subject to large volume changes with changes in moisture content.

5.1 Groundwater

Pore pressure dissipation tests performed in two of our CPTs indicate the groundwater level is at depths of approximately 20 to 25 feet bgs. During their previous investigation on the adjacent site, Arup encountered groundwater measured in borings BH-6 and BH-7 at depths of 6 and 2-1/4 feet, respectively. Arup also determined the approximate depth to groundwater using pore pressure dissipation test data from two of their CPTs at depths of about 4 to 5-1/2 feet bgs. The groundwater level at the site is expected to fluctuate several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall.

To estimate the highest potential groundwater level at the site, we reviewed information on the State of California Water Resources Control Board GeoTracker website (http://geotracker.swrcb.ca.gov). The three closest sites with significant historic groundwater data on the GeoTracker website are at 149 South Boulevard, 1740 Leslie Street, and 1790 South Delaware Street. The data from these three sites indicates the groundwater table slopes down gently to the southwest. Between December 1998 and January 2008, groundwater was measured in multiple monitoring wells at each of the three sites. The highest groundwater levels measured during that time period ranged from 0.05 to 3 feet bgs at each of the locations. Using the high groundwater levels and distance from the three locations to the project site, we performed linear interpolations to estimate the high depth to groundwater at the project site to be approximately one foot bgs.

6.0 SEISMIC CONSIDERATIONS

6.1 Regional Seismicity and Faulting

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.

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

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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	5.1	West	7.23
N. San Andreas (1906 event)	5.1	West	8.05
Monte Vista-Shannon	14	Southeast	6.50
San Gregorio Connected	17	West	7.50
Total Hayward	24	Northeast	7.00
Total Hayward-Rodgers Creek	24	Northeast	7.33
N. San Andreas - North Coast	35	Northwest	7.51
Total Calaveras	37	East	7.03
Mount Diablo Thrust	43	Northeast	6.70
Green Valley Connected	48	Northeast	6.80
N. San Andreas - Santa Cruz	50	Southeast	7.12

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 69 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 ($M_w = 6.2$).

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.

6.2 Geologic Hazards

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards, including ground shaking, ground surface rupture, liquefaction[2], lateral spreading[3] and cyclic densification.[4] The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

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.

6.2.1 Ground Shaking

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and 4) subsurface conditions. The site is about 5 kilometers from the San Andreas Fault, although ground shaking from future earthquakes on other faults, including the Hayward, San Gregorio, and Calaveras faults will also be felt at the site. We judge that strong to very strong ground shaking could occur at the site during a large earthquake on one of the nearby faults.

6.2.2 Liquefaction and Associated Hazards

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

The site is located within a zone of liquefaction potential as shown on the map titled *State of California Seismic Hazard Zones, San Mateo Quadrangle, Official Map*, prepared by the California Geological Survey (CGS), dated January 11, 2018 (see Figure 5). CGS has provided recommendations for procedures and report content for site investigations performed within seismic hazard zones in Special Publication 117 (SP-117), titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California*, dated September 11, 2008. SP-117 recommends subsurface investigations in mapped liquefaction hazard zones be performed using rotary-wash borings and/or CPTs.

Liquefaction susceptibility was assessed using the software CLiq v1.7 (GeoLogismiki, 2014). 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). We performed a liquefaction triggering analysis using our CPT data in accordance with the methodologies proposed by Boulanger and Idriss (2014). Post-earthquake settlements were evaluated using the methodology proposed by Zhang, Robertson, and Brachman (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement; this methodology is an extension of the work by Ishihara and Yoshimine (1992).

Our analyses were performed using an assumed high groundwater at one foot bgs. In accordance with the 2016 CBC and the 2019 CBC (ASCE 7-16), we used peak ground acceleration values of 0.75 and 0.88 times gravity (g) in our liquefaction evaluation, respectively; these peak ground accelerations are 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 preliminary liquefaction analyses indicates there are isolated lenses of silty sand and sandy silt beneath a depth of about 4 feet bgs that are susceptible to liquefaction. These silty sand and sandy silt lenses are generally less than two feet thick. The clayey fill and the clay deposits underlying the fill are not susceptible to liquefaction because of their cohesion; however, our analyses indicate the clayey fill and clay deposits may experience pore pressure buildup and strength loss, referred to as cyclic softening, from cyclic loading during an earthquake. Dissipation of the excess pore pressures in the clayey/organic fill and clay deposits after the earthquake will result in ground surface settlement. We estimate total ground settlement during post-earthquake reconsolidation of the underlying fill, clay deposits, and silty sand/sandy silt layers following a Maximum Considered Earthquake (MCE) event with PGA_M ranging from 0.75g to 0.88g could be on the order of one inch in the southeastern portion of the site (under Building A) and about 1-3/4 inches in the northwestern portion of the project site (under Building B). We estimate differential settlement due to liquefaction beneath Building A and Building B could be on the order of 1/2 inch and one inch across a horizontal distance of 30 feet, respectively.

Because the uppermost potentially liquefiable layers underlying the proposed Building B in the northwestern portion of the site are relatively shallow, there is potential for reductions in the bearing capacity of the soil if the proposed building is supported on a shallow foundation system on unimproved soil. Consequently, the actual building deformations of Building B could be significantly greater than that estimated above for the free-field ground surface during an earthquake. As discussed in later sections of this report, the potential for liquefaction within these relatively shallow layers should be mitigated if the proposed parking structure is to be supported on a shallow foundation system.

Our preliminary analysis indicates the non-liquefiable soil overlying the potentially liquefiable soil layers underlying the proposed Building A footprint in the southeastern portion of the site is sufficiently thick and the potentially liquefiable layers are sufficiently thin such that the potential for surface manifestations from liquefaction, such as sand boils are very low, provided the building is constructed at grade.

Considering the relatively flat site grades and the absence of a free face in the site topography, as well as the depth and relative thickness of the potentially liquefiable layers, we conclude the risk of lateral spreading is low.

6.2.3 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The CPTs indicate the fill above the groundwater at the site is not susceptible to cyclic densification because of its cohesion or relative density. Accordingly, we conclude the potential for ground surface settlement resulting from cyclic densification is low.

6.2.4 Fault Rupture

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

7.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, we conclude the site can be developed as planned. The primary geotechnical concerns are: (1) the potential for up to one inch and 1-3/4 inches of total settlement due to a combination of liquefaction and cyclic softening beneath the proposed Building A and Building B, respectively; (2) the presence of relatively weak and moderately to highly compressible clay deposits underlying the site; and (3) providing adequate vertical and lateral support for the proposed structures.

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

7.1 Foundation Support and Settlement

The factors influencing the selection of a safe, economical foundation system are providing an adequate factor of safety against bearing capacity failure, limiting differential settlement to an amount that can be tolerated by the structure, constructability, and cost. The results of our preliminary field investigation indicate the site is blanketed by heterogeneous fill overlying medium stiff to stiff clay deposits. The preliminary analyses we performed using the data from the CPTs indicate the proposed Building B, if supported on conventional shallow foundations (footings or mat), would experience excessive and erratic differential settlement on the order of 2 to 3 inches in 30 feet as a result of: (1) compression and consolidation of the fill layer, (2) consolidation of the medium stiff to stiff clay deposits, (3) post-earthquake reconsolidation from the medium dense sand layers, and (4) loss of bearing due to liquefaction of the supporting soil. This anticipated amount of differential settlement would exceed the typical tolerance of conventional shallow foundations. It is our preliminary conclusion that Building B should be supported on spread footings bearing on improved soil or a deep foundation system.

Our analyses performed using the CPT data indicate the liquefiable soil layers are generally thinner and deeper beneath the proposed Building A in the southeastern portion of the site. Our settlement analyses indicate total settlement of Building A supported on a shallow foundation system under static load conditions, designed using the allowable bearing pressures presented below, will be on the order of about 1-1/2 inches and differential settlement will be on the order of one inch over a 30-foot horizontal distance. Shallow foundations supporting Building A may experience an additional one inch of total settlement and 1/2 inch of differential settlement due to post-liquefaction reconsolidation following a major earthquake, as discussed in Section 6.2.2.

We preliminarily conclude Building A 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 (static plus seismic) 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 Building A may be supported on spread footings bearing on improved soil as well.

Preliminary mat foundation recommendations for Building A are presented below, as well as recommendations for spread footings bearing on improved soil for Building B. We can provide recommendations for a deep foundation system upon request.

7.1.1 Building A

If the estimated settlements presented above are acceptable, the proposed Building A may be supported on a well-reinforced mat foundation bearing on firm native soil or engineered fill. . Atterberg Limits tests performed on samples of the clay at approximately 2 and 4 feet bgs on the adjacent site indicate the clayey soil is highly expansive; therefore, an at-grade mat foundation should be underlain by 12 inches of non-expansive fill or lime-treated native soil. The on-site granular fill may be recompacted and used as select fill. The thickness and physical properties of the existing granular fill should be further evaluated during the final geotechnical investigation.

The perimeter of the mat should be thickened to achieve an embedment depth of at least nine inches below the adjacent outside finished grade. Where a mat is constructed near underground utilities, bio-swales or other storm water treatment areas, the edge of the mat should be founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal:vertical) from the base of the utility trench or bio-swale/treatment area.

For preliminary structural design of the mat foundation, we recommend using a coefficient of vertical subgrade reaction of 10 kips per cubic foot (kcf). This value has been reduced to account for the size of the mat/equivalent footings (therefore, this is not k_{v1} for 1-foot-square plate). Lateral loads can be resisted by a combination of passive pressure on the vertical faces of the mat and friction along the bottom of the mat. To limit total static settlement of the mat to 1-1/2 inches, localized bearing pressures should not exceed 2,000 psf for dead-plus-live loads. This pressure may be increased by one-third for total loads (including wind and seismic loads); we anticipate the average bearing pressure will be significantly lower.

Lateral resistance may be computed using equivalent fluid weights (triangular distribution) of 260 and 120 pcf for sustained loads above and below the design groundwater level, respectively. For transient load conditions, a uniform passive pressure of 1,300 psf may be used both above and below the design groundwater table. Passive resistance in the upper one foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance should be computed using a base friction coefficient of 0.30 where the mat is in contact with soil. Where a vapor retarder is placed beneath the mat, a base friction coefficient of 0.20 should be used. These values include a factor of safety of at least 1.5 and may be used in combination without reduction.

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

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 A 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. *If required by the structural engineer*, the vapor retarder may be covered with two inches of sand to aid in curing the concrete and to protect the vapor retarder during slab construction. The sand overlying the vapor retarder should be moist at the time concrete is placed. However, excess water trapped in the sand could eventually be transmitted as vapor through the mat. Therefore, if rain is forecast prior to concrete placement, the sand should be covered with plastic sheeting to avoid wetting. If the sand becomes wet, concrete should not be placed until the sand has been dried or replaced.

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.

7.1.2 Building B

Spread footings bearing on improved ground may be used to support the proposed Building B. We conclude drill displacement sand-cement (DDSC) columns or soil-cement (SMX) columns to be the most appropriate ground improvement methods for this project. DDSC columns are installed by advancing a continuous flight, hollow-stem auger that mostly displaces the soil and then pumping a sand-cement mixture into the hole under pressure as the auger is withdrawn. SMX columns are installed by injecting and blending cement into the soil using a drill rig equipped with single or multiple augers. These systems result in very low vibrations during installation and generate little to no drilling spoils for off-haul.

DDSC and SMX columns are installed under design-build contracts by specialty contractors. For planning purposes, we preliminarily recommend the ground improvement elements extend at least 5 feet into dense/stiff soil below a depth of approximately 35 to 40 feet bgs. The length and spacing of the DDSC or SMX columns should be sufficient to limit the combined static and seismic total settlement to less than one inch.

The DDSC and SMX columns, if properly designed, should be capable of increasing the allowable dead-plus-live-load bearing pressure to about 4,000 to 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. Perimeter footings should be bottomed at least 30 inches below the lowest adjacent outside finished grade and interior footings should be bottomed at least 24 inches below the bottom of the floor slab.

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 equivalent fluid weights (triangular distribution) of 260 and 120 pounds per cubic foot (pcf) above and below the design groundwater elevation, respectively. For transient load conditions, a uniform passive pressure of 1,300 psf may be used both above and below the design groundwater table. The upper foot of soil should be ignored for lateral resistance unless confined by a slab or pavement. The recommended passive pressure values 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.

Foundation excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. If the foundation soil dries during construction, the footing will eventually heave, which may result in cracking and distress. We should check footing excavations prior to placement of reinforcing steel.

The floor slab for the parking garage may be supported on grade provided the potential for up to 1-3/4 inches of seismically induced differential settlement to occur between the floor slab and the spread footings during a major earthquake is acceptable. If this potential is not acceptable, the floor slab should be designed to span between the DDSC columns. For both options, the upper 12 inches of soil beneath the floor slab should be treated with lime to mitigate the potential for shrink/swell movement of the highly expansive subgrade soil. For the slab-on-grade floor option, a minimum of six inches of Class 2 aggregate base should be placed beneath the floor slab.

7.2 Groundwater

As discussed in Section 5.1, groundwater was interpreted to be at depths ranging from 20 to 25 feet bgs based on pore pressure dissipation tests performed during our investigation and was estimated to range from 4 to 17 feet bgs during the Arup subsurface investigation. However, based on the historic groundwater data we reviewed for three sites in the vicinity of the subject property, we preliminarily conclude a design groundwater level of approximately one foot below existing grade should be used. Considering the potential adverse impacts of this very shallow groundwater depth on both design and construction, we recommend 1 to 2 piezometers be installed at the site during the final geotechnical investigation to allow monitoring of the groundwater level.

7.3 Permanent Below-Grade Walls

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

- At-rest equivalent fluid weight of 60 pcf above the design groundwater table and 90 pcf below.
- Active pressure of 40 pcf and seismic increment of 34 pcf above the design groundwater level, 82 pcf plus a seismic increment of 16 pcf below (triangular distribution).

The recommended lateral earth pressures above are based on a level backfill conditions with no additional surcharge loads. Where the below-grade walls are subject to traffic loading within 10 feet of the wall, an additional uniform lateral pressure of 50 psf applied to the upper 10 feet of the wall.

To protect against moisture mitigation into the below-grade parking level, we recommend that the below-grade walls be waterproofed and water stops be installed at all construction joints.

7.4 Seismic Design

We understand the proposed structures will be designed using the seismic provisions in either the 2016 CBC or the 2019 CBC. The latitude and longitude of the site are 37.5528° and -122.3092°, respectively. Recommendations in accordance with the 2016 and 2019 CBC are presented in the sections below.

<u>2016 CBC</u>

Section 1613A of the 2016 California Building Code (CBC) and Section 20.3.1 of ASCE 7-10 indicate if liquefiable soil is present at a site, it is classified as Site Class F and a site-specific response study is required; however, if the period of the structure is less than 0.5 second, the site class can be determined from Section 20.3 of ASCE 7-10. If the period of the proposed structures will be less than 0.5 second, we recommend Site Class D be used. Hence, in accordance with the 2016 CBC, we recommend the following:

- S_S = 1.909g, S_1 = 0.892g
- S_{MS} = 1.909g, S_{M1} = 1.338g
- S_{DS} = 1.273g, S_{D1} = 0.892g
- PGA_M = 0.75g
- Seismic Design Category E for Risk Categories I, II, and III.

<u>2019 CBC</u>

Section 20.3.1 of ASCE 7-16 indicate if liquefiable soil is present at a site, it is classified as Site Class F and a site-specific response study is required; however, if the period of the structure is

less than 0.5 second, the site class can be determined from Section 20.3 of ASCE 7-10. If the period of the proposed structures will be less than 0.5 second, we recommend Site Class D be used. Hence, in accordance with ASCE 7-16, we recommend the following:

- S_S = 1.85g, S_1 = 0.76g

Per ASCE 7-16, where S_1 is greater than 0.2 times gravity (g), a ground motion hazard analysis is needed unless C_S is conservatively increased per Exception 2 of Section 11.4.8 of ASCE 7-16.

> ASCE 7-16, Section 11.4.8, Exception 2*: Structures on Site Class D sites with S1 greater than or equal to 0.2, provided the value of the seismic response coefficient Cs is determined by Eq. (12.8-2) for values of T ≤ 1.5Ts and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for TL ≥ T > 1.5Ts or Eq. (12.8-4) for T > TL.*

Therefore, assuming Cs will be conservatively increased per Exception 2 of Section 11.4.8, we recommend the following seismic design parameters:

- F_a = 1, F_v =1.7
- S_{MS} = 1.85g, S_{M1} = 1.29g
- S_{DS} = 1.23g, S_{D1} = 0.86g
- PGA_M = 0.877g
- Seismic Design Category E for Risk Factors I, II, and III

If Cs is not increased per Exception 2 of Section 11.4.8, then a ground motion hazard analysis should be performed during the final investigation.

7.5 Construction Considerations

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

Excavations that will be deeper than five feet and will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes. We judge temporary slopes with a maximum inclination of 1.5:1 (horizontal to vertical) should be stable, provided the slope is not surcharged by adjacent structures, construction equipment, or stockpiled soil.

8.0 ADDITIONAL GEOTECHNICAL SERVICES

The preliminary conclusions and recommendations presented within are based on a preliminary investigation and not intended for final design. Prior to final design, we should be retained to provide a final geotechnical report based on the final proposed development. 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, 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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ROCKRIDGE GEOTECHNICAL

FIGURES

Base map: The Thomas Guide
San Francisco County
2002

0 1/4 1/2 Mile

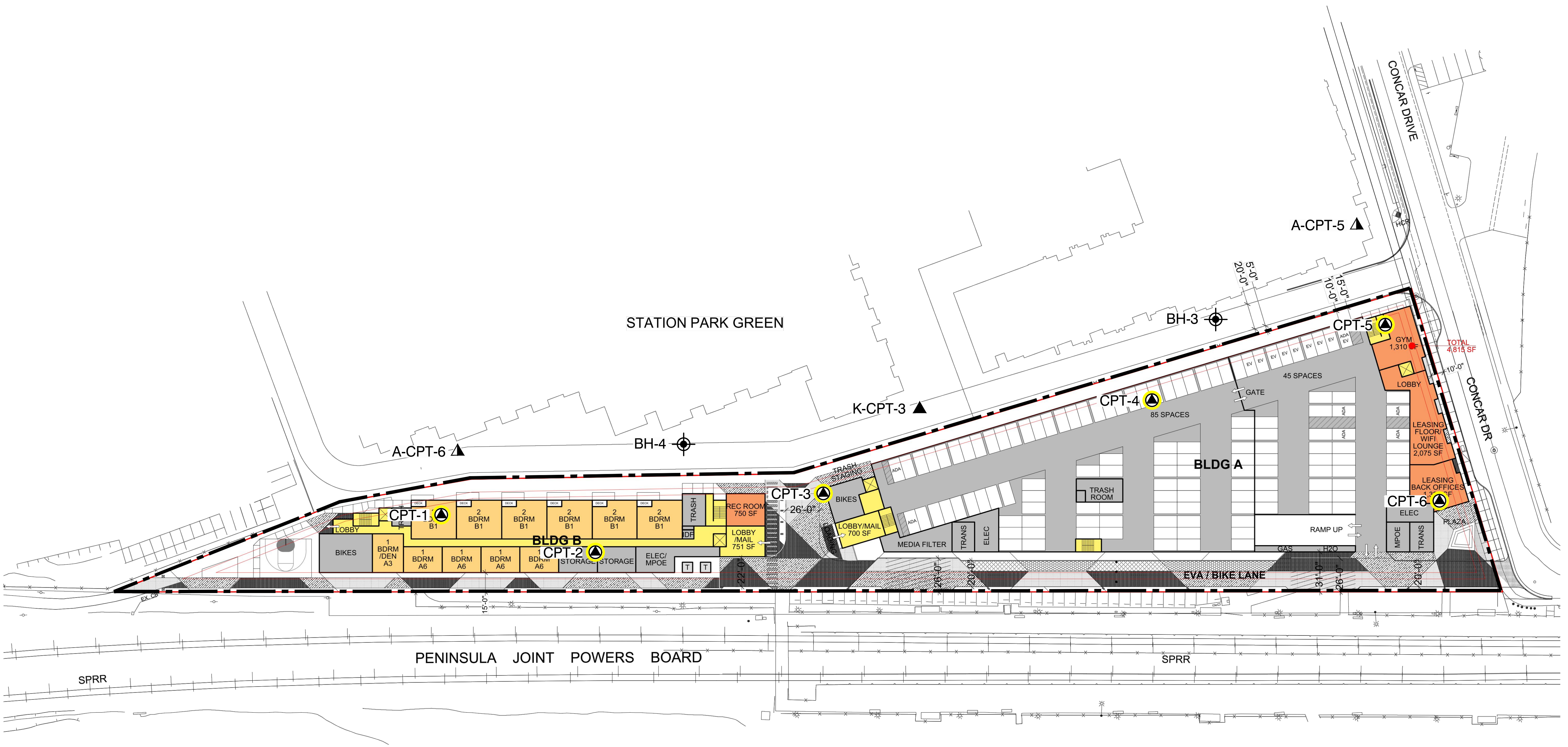
Approximate scale

HAYWARD PARK STATION
San Mateo, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 11/16/15	Project No. 15-972	Figure 1



EXPLANATION

CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical, Inc., October 29, 2015

A-CPT-6 — Approximate location of cone penetration test by Arup, September 2005 and November 2006

K-CPT-3 — Approximate location of cone penetration test by Kleinfelder, September 2005

BH-4 — Approximate location of test boring by Arup, September 2006

Property boundary

Reference: Base map from a drawing titled "Conceptual Floor Plan - Level 1", by BDE Architecture, dated February 26, 2019.

0 80 Feet

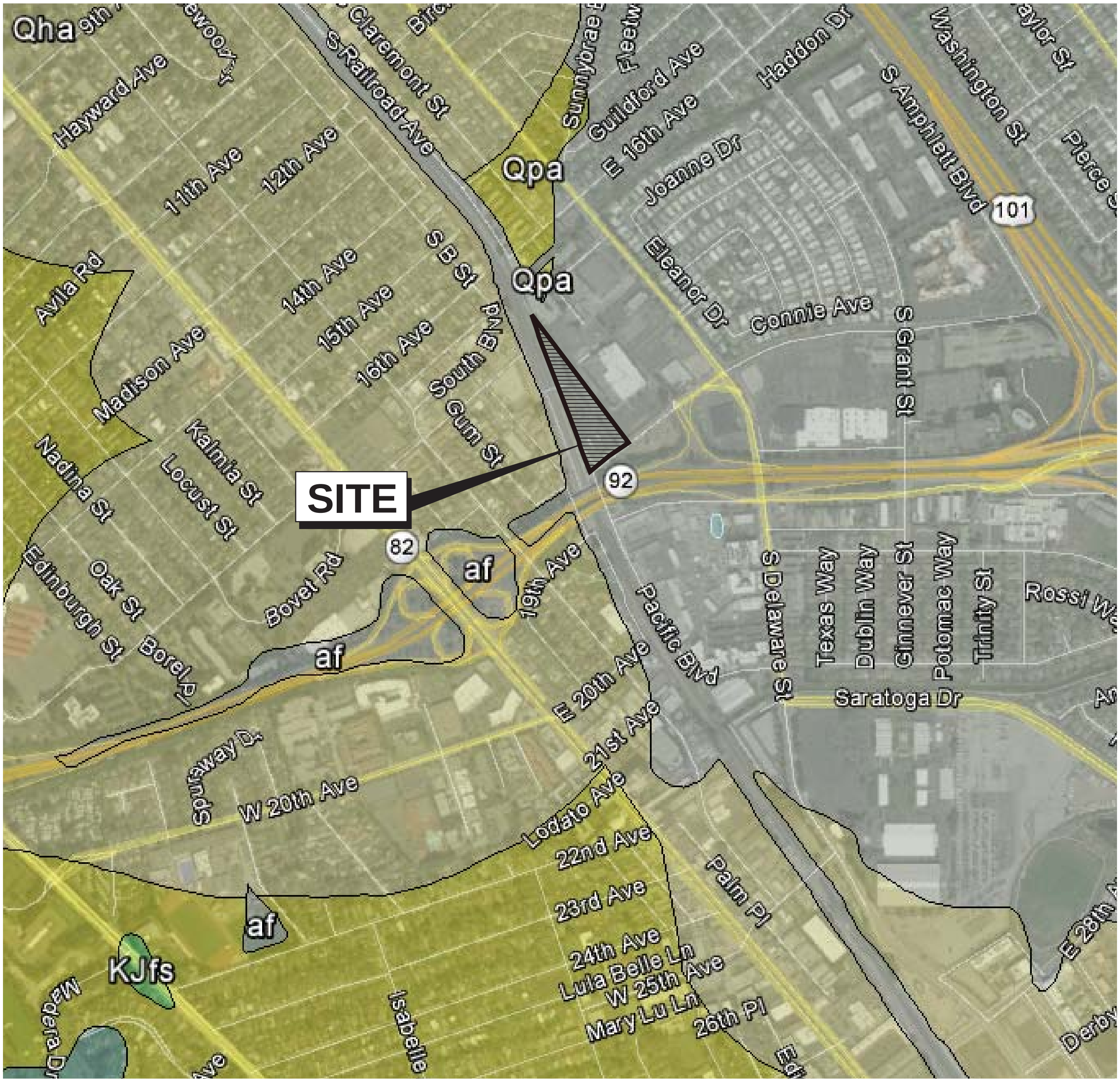
Approximate scale

HAYWARD PARK STATION
San Mateo, California

SITE PLAN

Date 06/06/19	Project No. 15-972	Figure 2

ROCKRIDGE GEOTECHNICAL



Base map: Goolge Earth with U.S. Geological Survey (USGS), San Mateo County, 2015.

EXPLANATION

af	**Artificial Fill**
Qha	**Alluvium (Holocene)**
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 uncertin

N

0 1000 2000 Feet

Approximate scale

HAYWARD PARK STATION
San Mateo, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 11/16/15 | Project No. 15-972 | Figure 3

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

EXPLANATION

Strike slip

Thrust (Reverse)

Normal

0 5 10 Miles

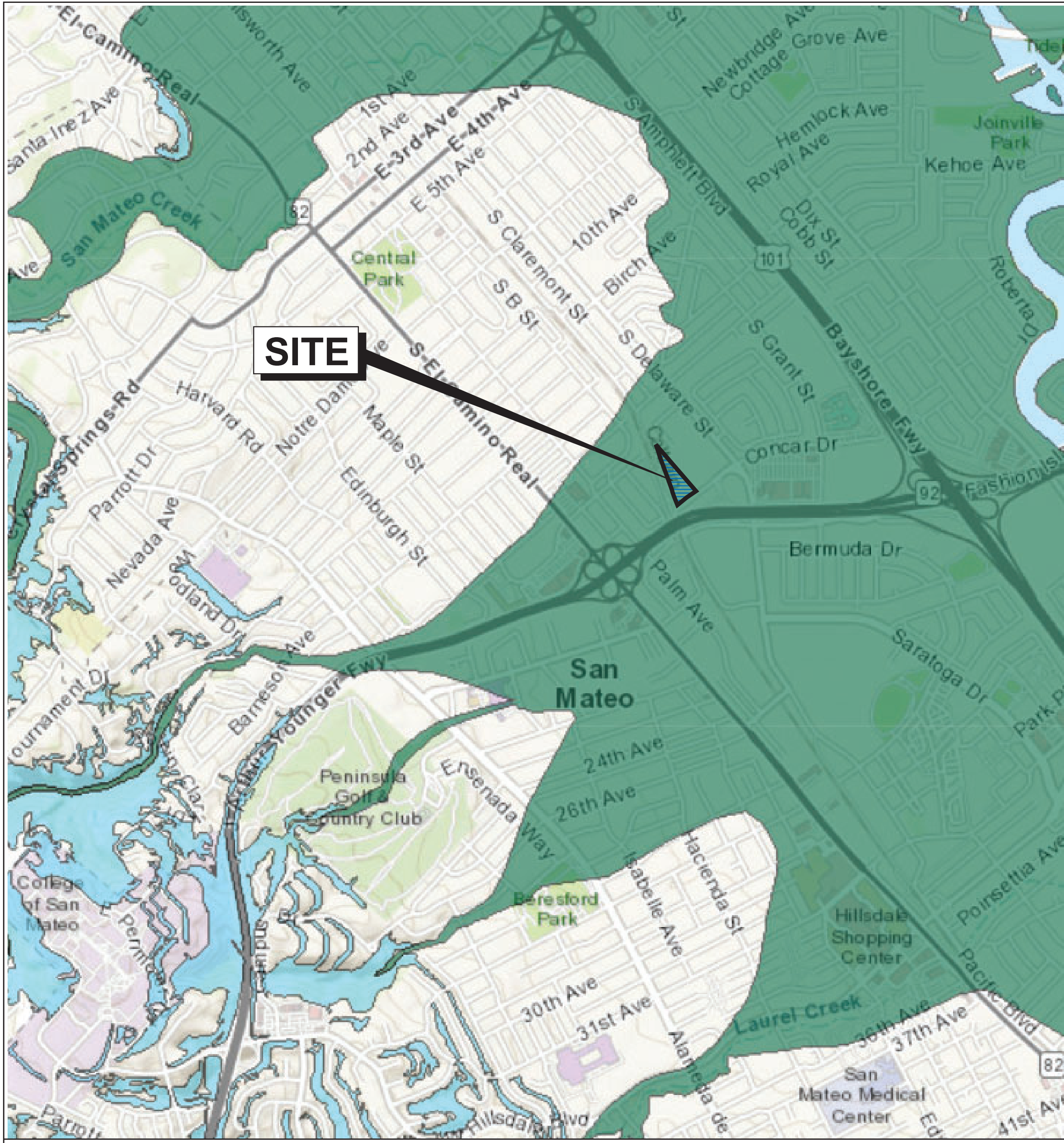
Approximate scale

HAYWARD PARK STATION
San Mateo, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 11/16/15	Project No. 15-972	Figure 4



Reference: https://maps.conservation.ca.gov/cgs/EQZApp/app/ (California Geological Survey, 2019)
San Mateo Quadrangle, Released January 11, 2018.

EXPLANATION

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

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

0 0.4 0.8 Miles

Approximate scale

HAYWARD PARK STATION
San Mateo, California

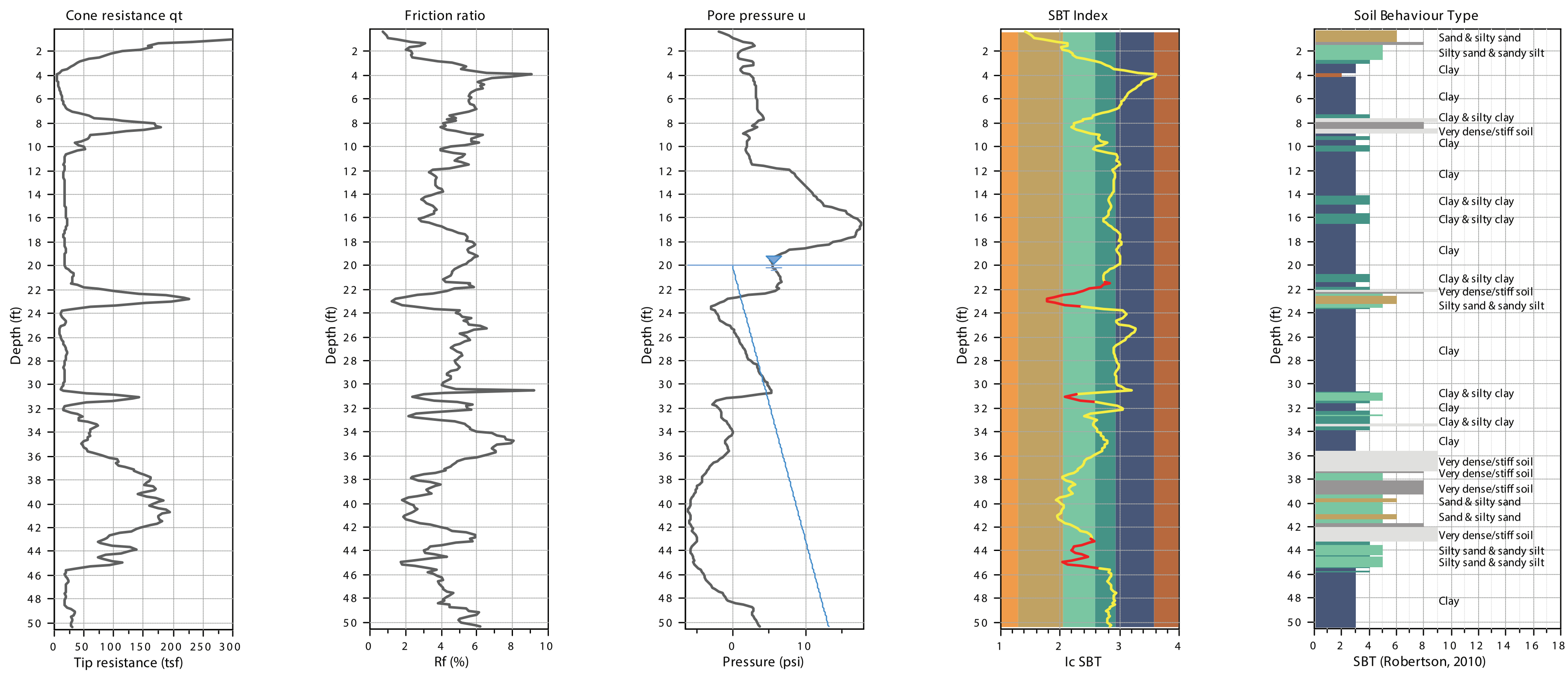
ROCKRIDGE GEOTECHNICAL

SEISMIC HAZARDS ZONE MAP

Date 06/07/19	Project No. 15-972	Figure 5

ROCKRIDGE GEOTECHNICAL

APPENDIX A
Cone Penetration Test Results



SBT legend

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

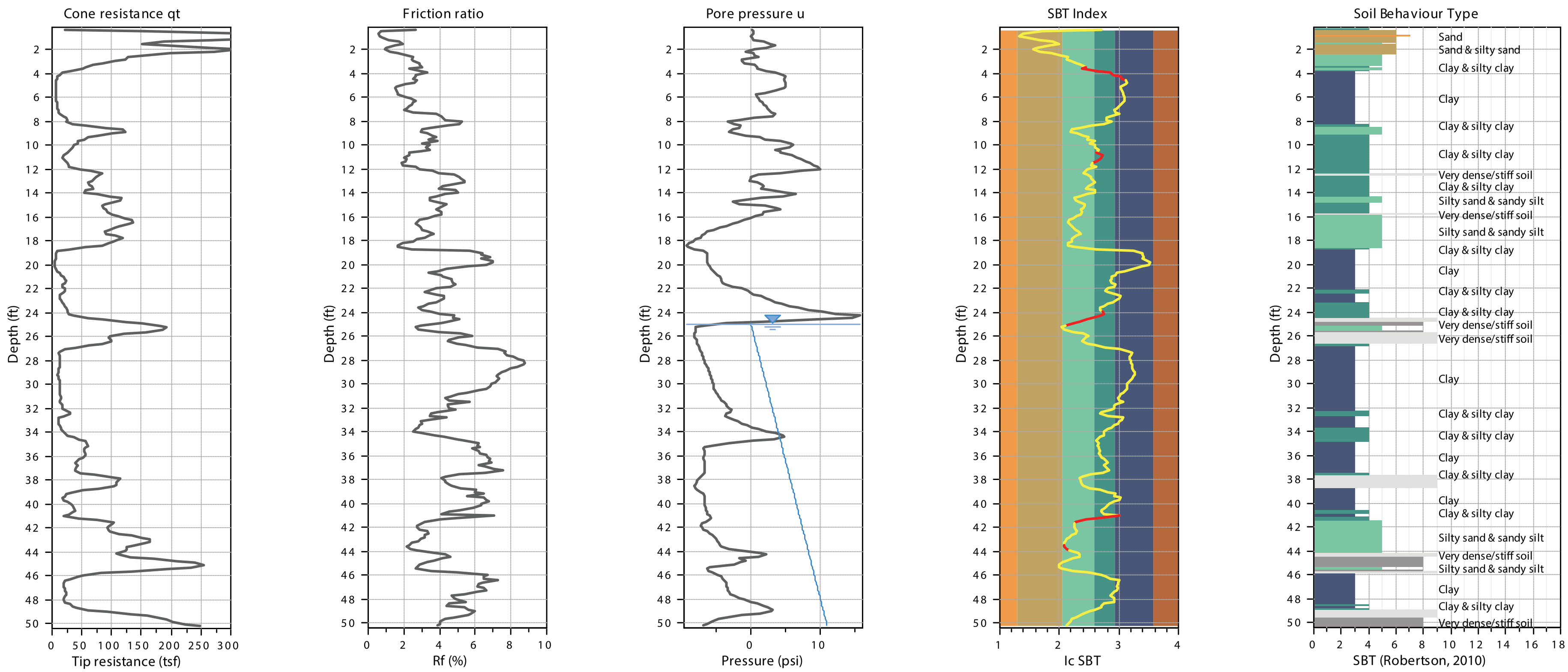
Total depth: 50.36 ft, Date: 10/29/2015
Measured Groundwater Depth: 20 feet (based on pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

HAYWARD PARK STATION
San Mateo, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 06/07/19	Project No. 15-972	Figure A-1



SBT legend

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

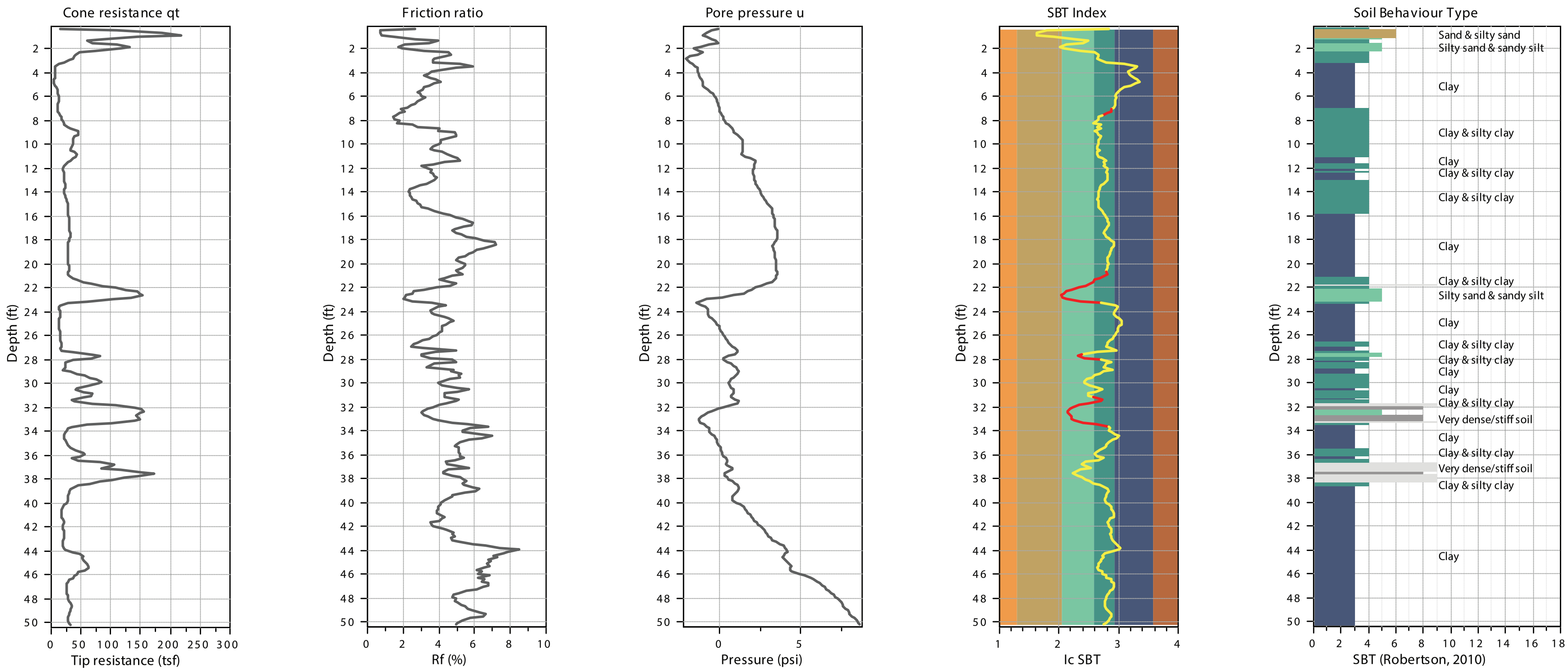
Total depth: 50.20 ft, Date: 10/29/2015
Measured Groundwater Depth: 25 feet (based on pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

HAYWARD PARK STATION
San Mateo, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 06/07/19 | Project No. 15-972 | 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

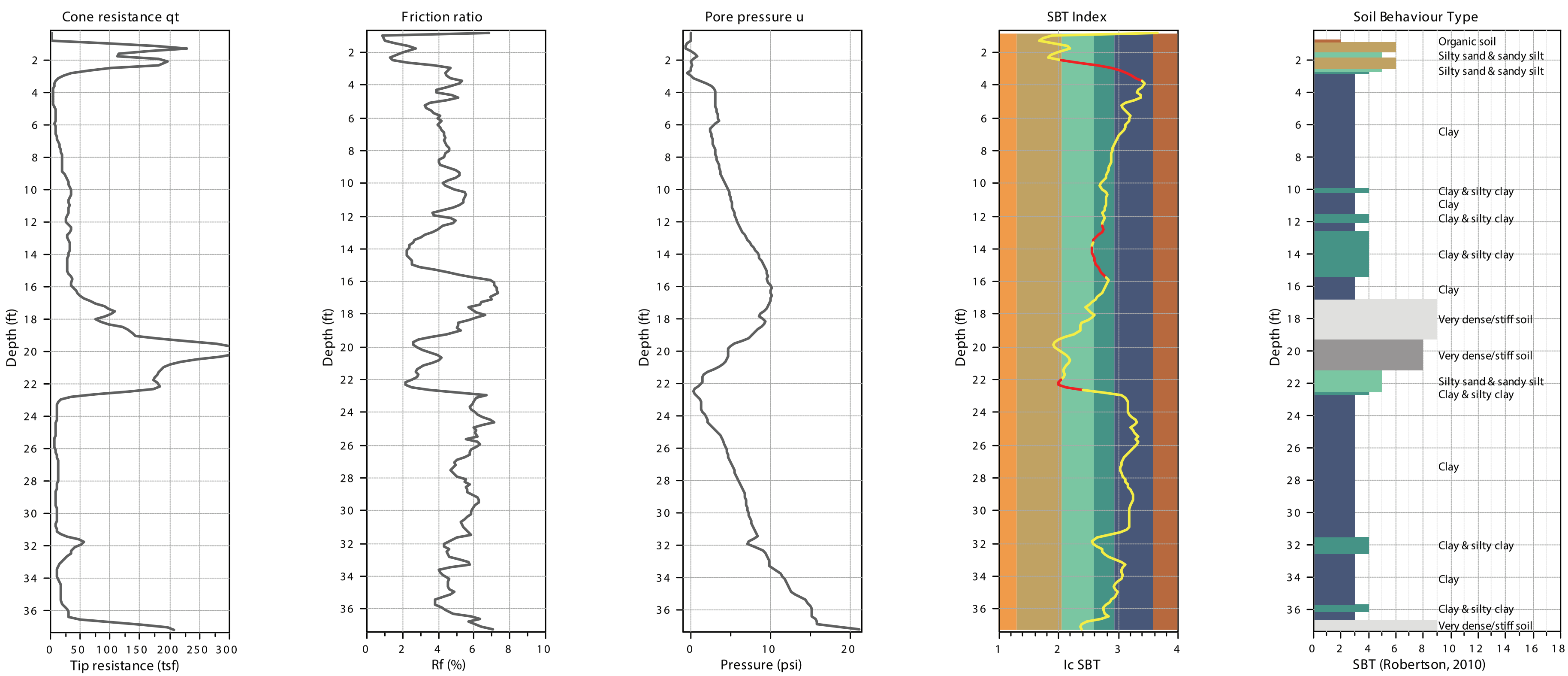
Total depth: 50.20 ft, Date: 10/29/2015
Groundwater not measured
Cone Operator: Middle Earth Geo Testing, Inc.

HAYWARD PARK STATION
San Mateo, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 06/07/19	Project No. 15-972	Figure A-3



Total depth: 37.24 ft, Date: 10/29/2015
Groundwater not measured
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

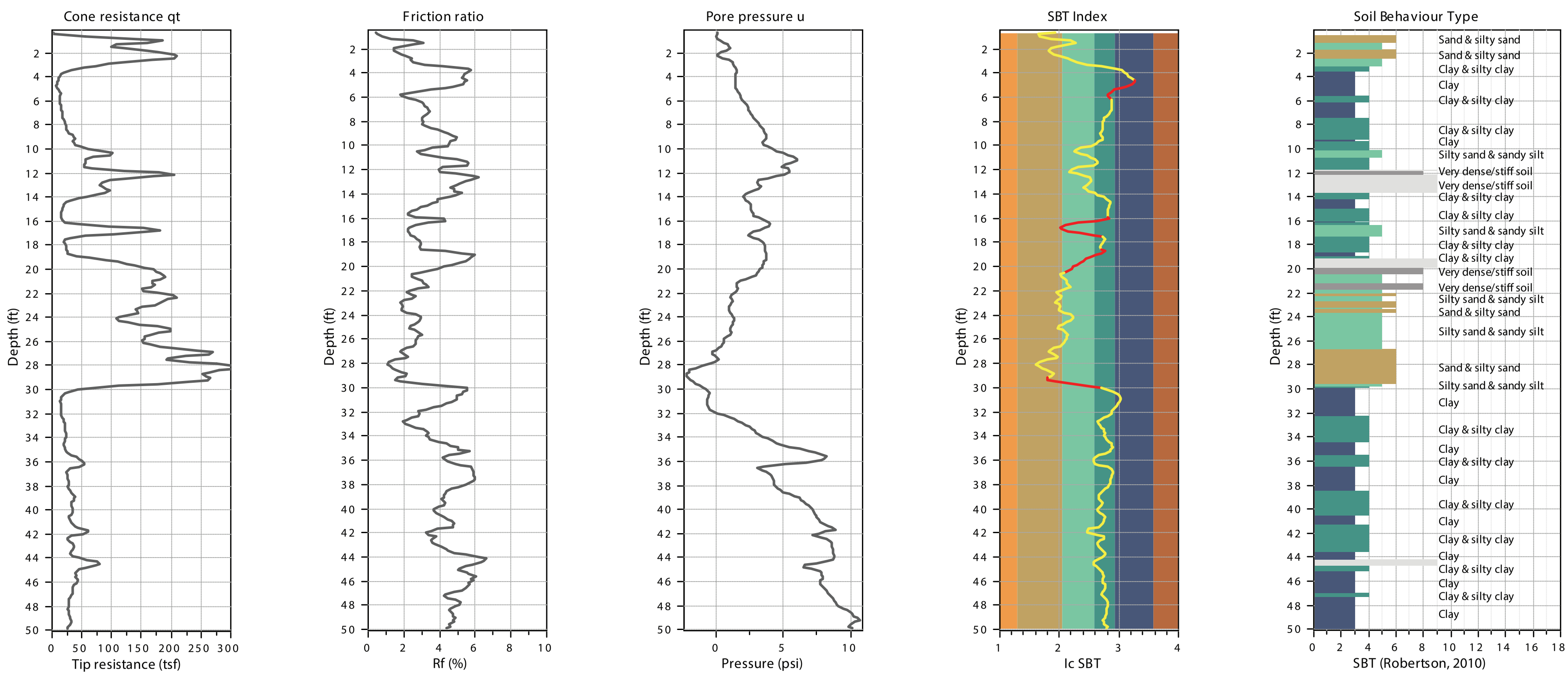
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

HAYWARD PARK STATION
San Mateo, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-5

Date 06/07/19	Project No. 15-972	Figure A-4



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: 49.87 ft, Date: 10/29/2015
Groundwater not measured
Cone Operator: Middle Earth Geo Testing, Inc.

HAYWARD PARK STATION
San Mateo, California

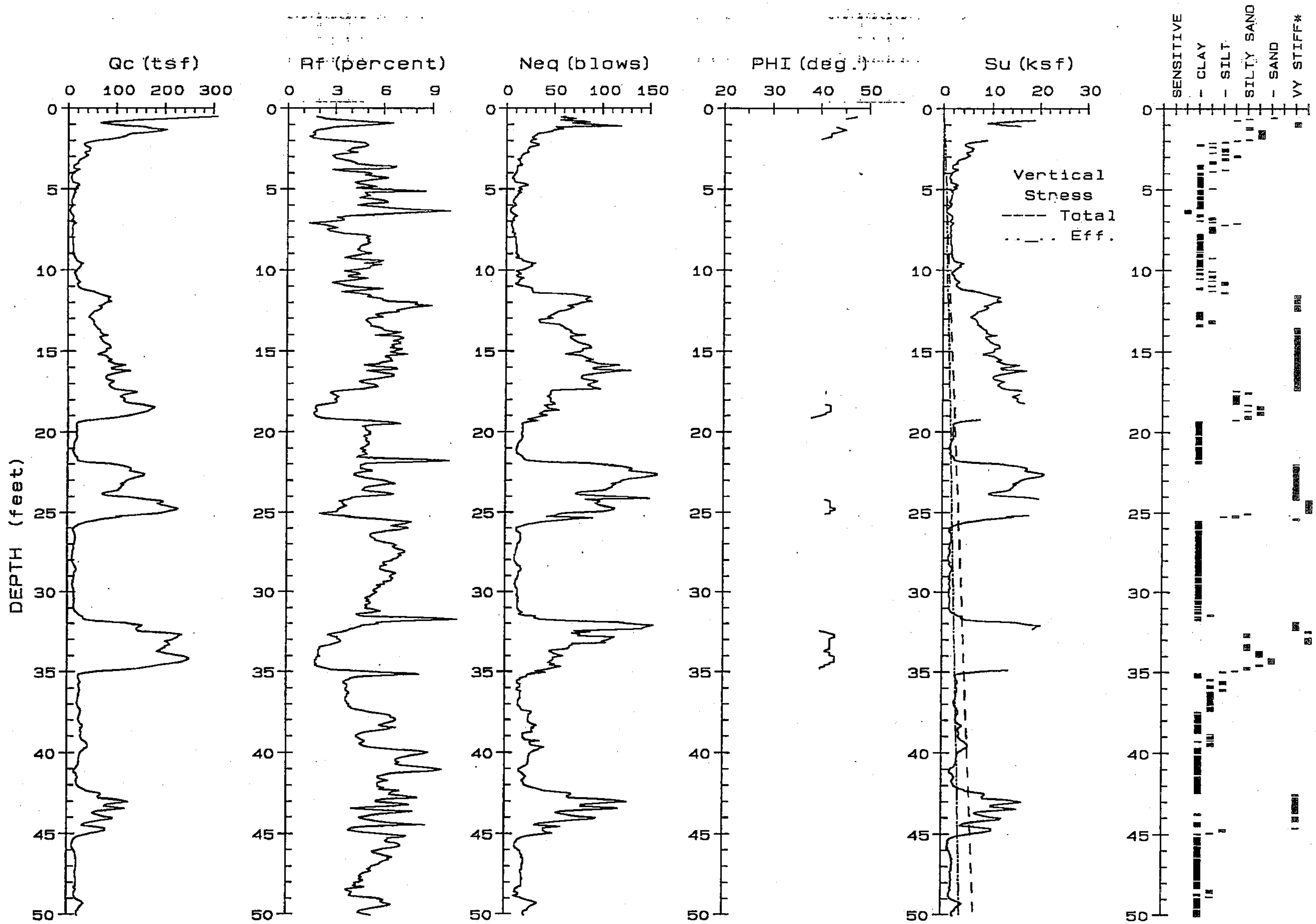
ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-6

Date 06/07/19	Project No. 15-972	Figure A-5

ROCKRIDGE
GEOTECHNICAL

APPENDIX B
Boring Logs, Cone Penetration Test Results, and Laboratory Test Results by Others



Terminated at 50.0 feet

Groundwater measured at 4.7 feet

PROJECT: 1700 SOUTH DELAWARE
LOCATION: San Mateo CA
PROJ. NO.: 61433 (KLF-111)

CPT NO.: CPT-3
DATE : 09-21-2005

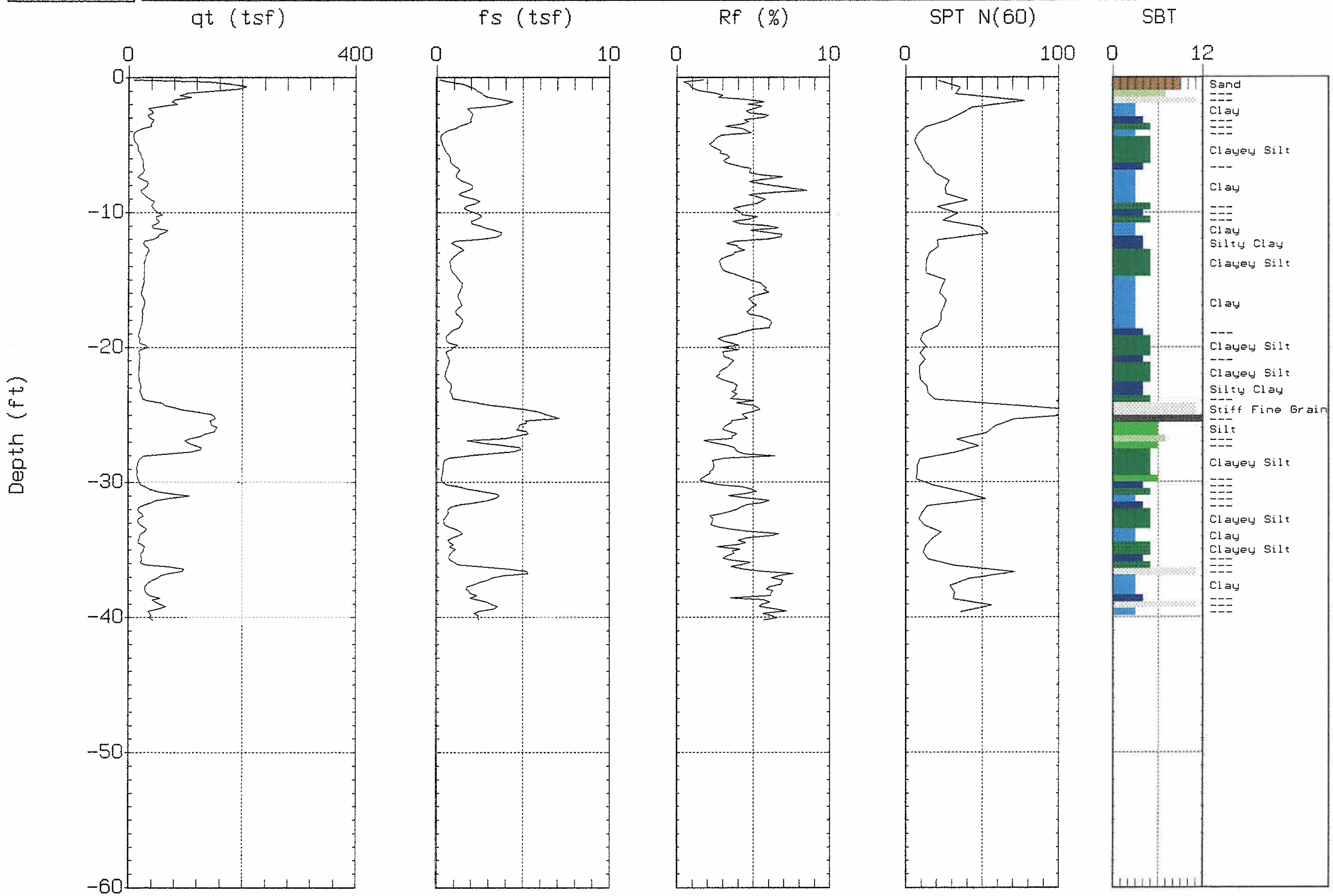
John Sarmiento & Associates
Cone Penetration Testing Service

GREGG

ARUP

Site: HAYWARD PARK GREEN
Location: CPT-05

Geologist: F.GREGURAS
Date: 11:07:06 04:45



Max. Depth: 40.19 (ft)
Depth Inc.: 0.164 (ft)

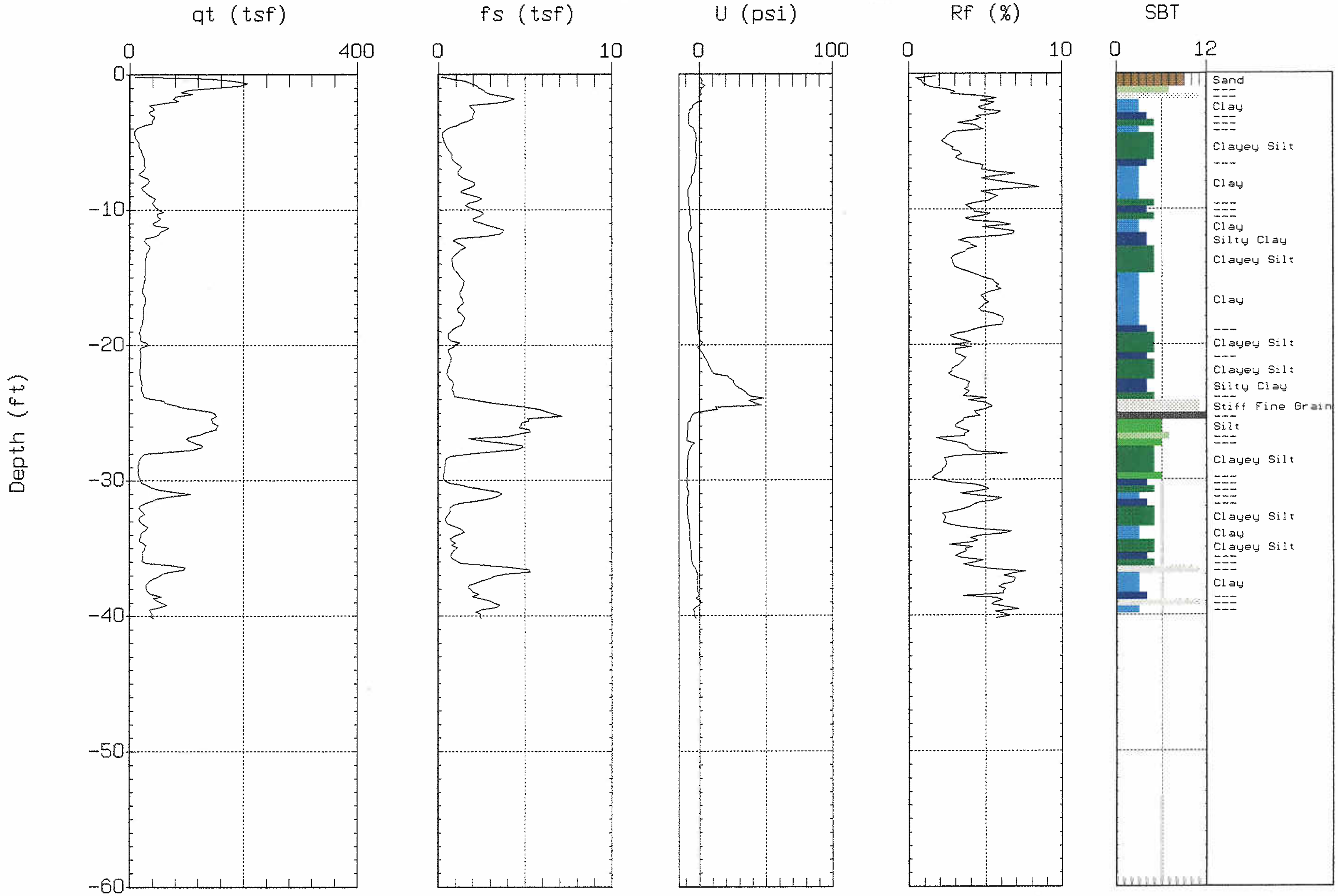
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ARUP

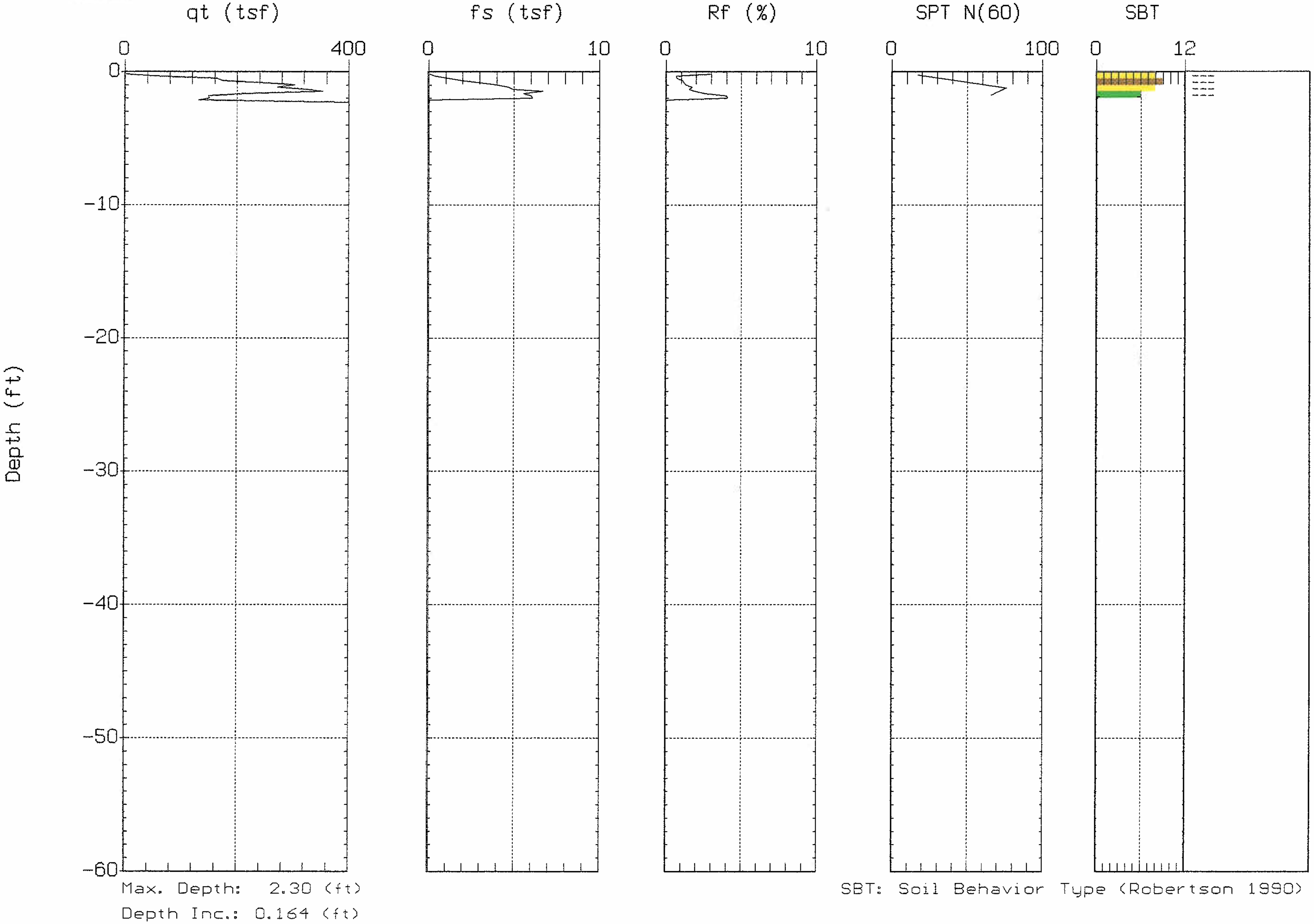
Site: HAYWARD PARK GREEN
Location: CPT-05

Geologist: F.GREGURAS
Date: 11:07:06 04:45



Max. Depth: 40.19 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

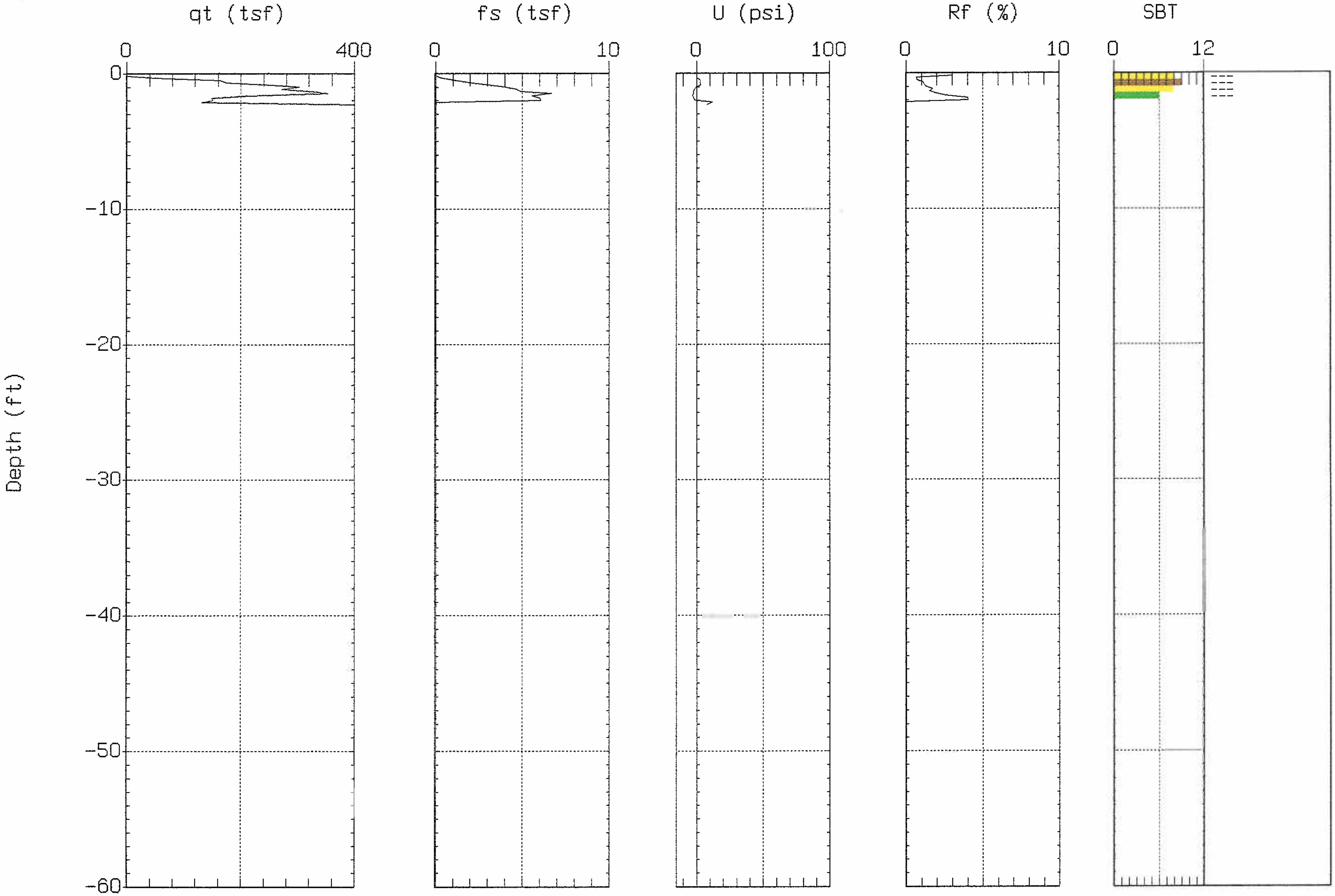

qt (tsf)
fs (tsf)
Rf (%)
SPT N(60)
SBT
0
400
10
100
12
Depth (ft)
-10
-20
-30
-40
-50
-60
Max. Depth: 2.30 (ft)
Depth Inc.: 0.164 (ft)
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ARUP

Site: HAYWARD PARK GREEN
Location: CPT-06

Geologist: F.GREGURAS
Date: 11:06:06 23:44

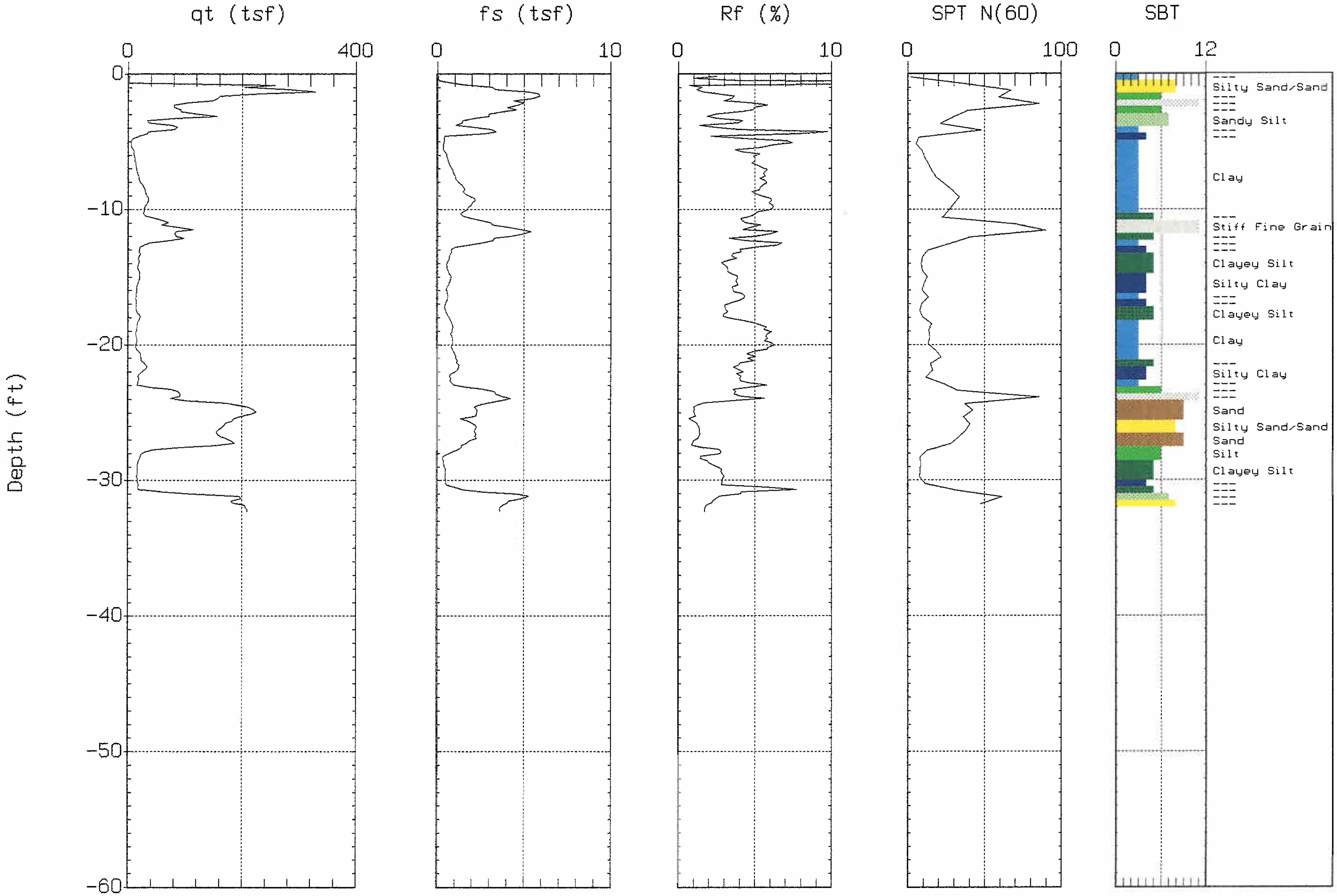


Max. Depth: 2.30 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

ARUP

Site: HAYWARD PARK GREEN
Location: CPT-06a
Geologist: F.GREGURAS
Date: 11:06:06 23:58



Max. Depth: 32.32 (ft)
Depth Inc.: 0.164 (ft)

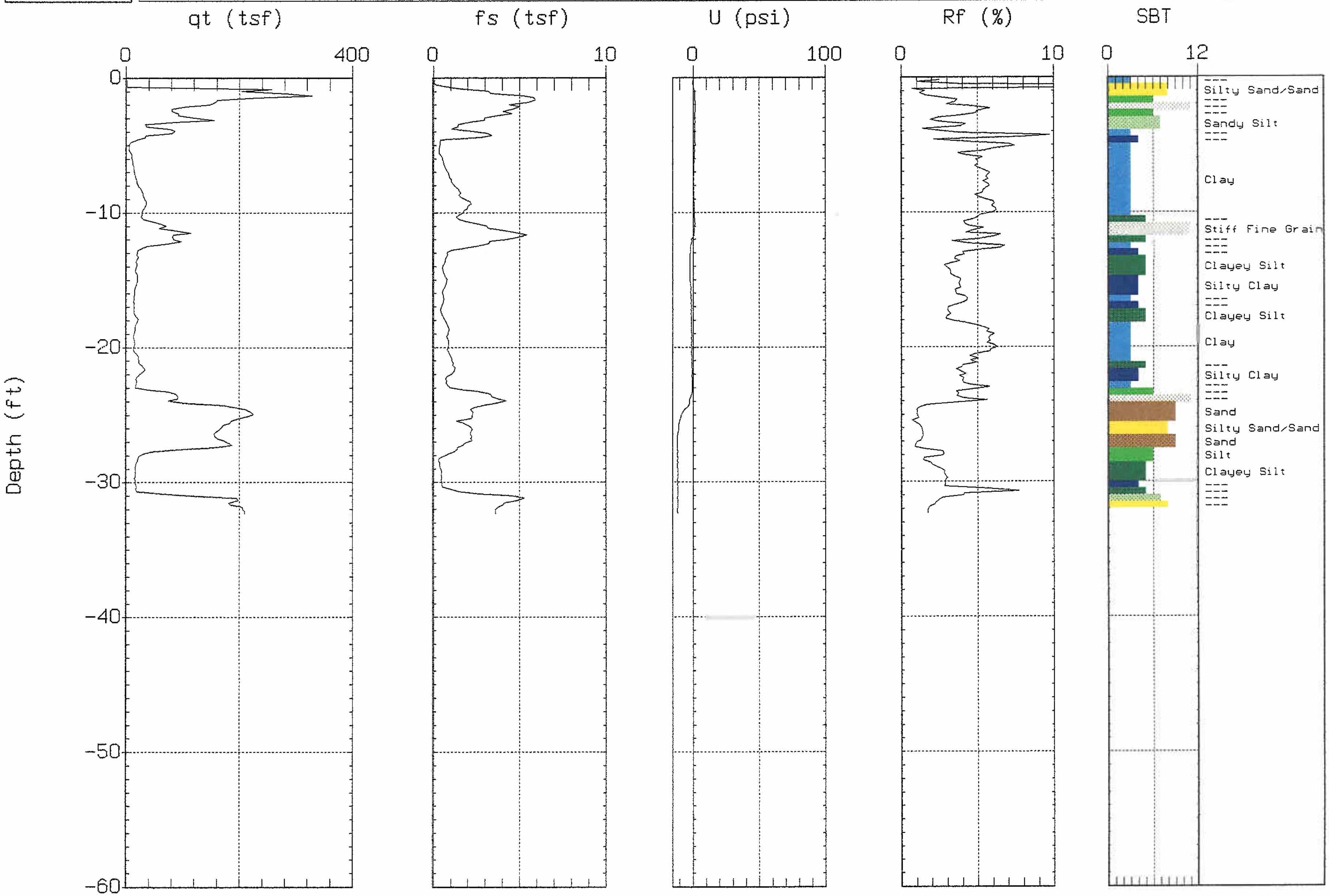
SBT: Soil Behavior Type (Robertson 1990)

GREGG

ARUP

Site: HAYWARD PARK GREEN
Location: CPT-06a

Geologist: F.GREGURAS
Date: 11:06:06 23:58



Max. Depth: 32.32 (ft)
Depth Inc.: 0.164 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TABLE E-2

SUMMARY OF INDEX PROPERTIES

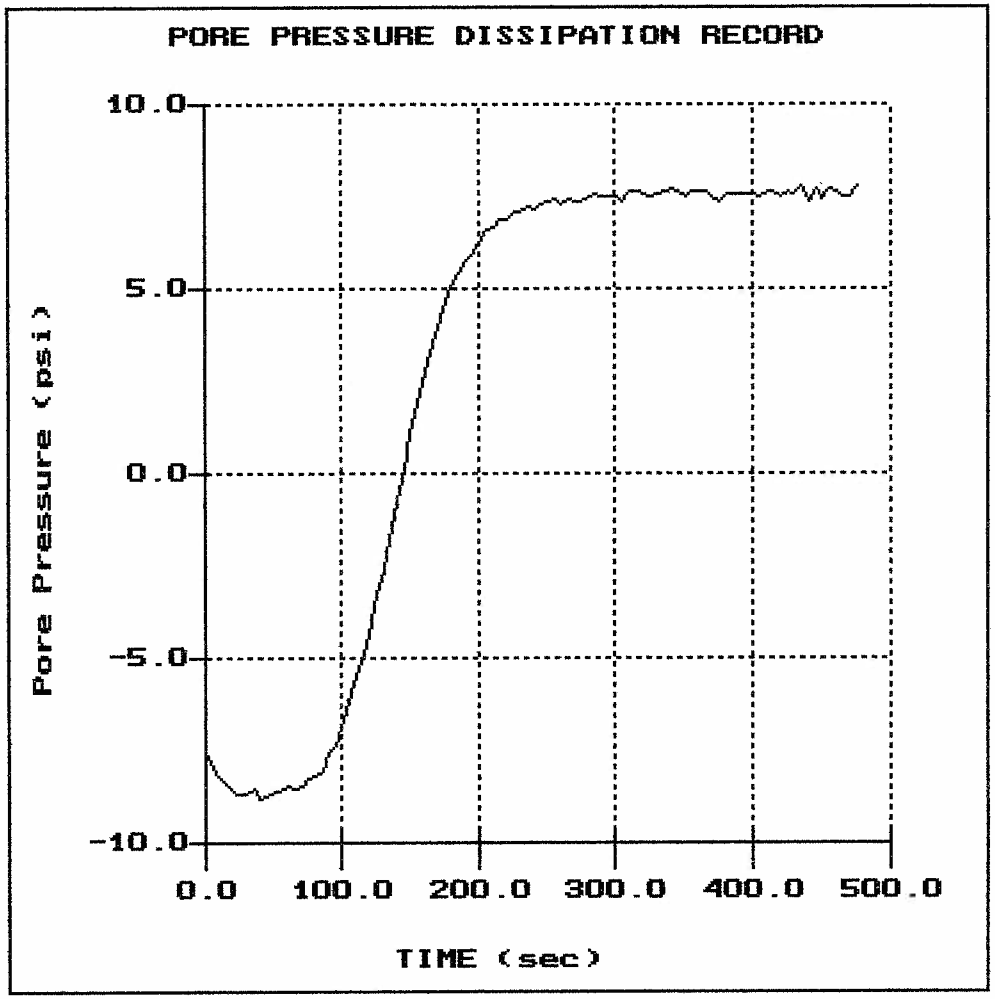
Borehole ID	Sample No.	Depth (ft)	Elevation (ft)	Soil Type	Moisture Content, wn (%)	Total Unit Weight, gt (pcf)	Liquid Limit, wl (%)	Plastic Limit, wp (%)	Plasticity Index, Ip (%)	Percent Fines (%)
BH-1	1B	1.5	100.27	Fill (ML)	27.2	115.5				
BH-1	3	10.0	91.77	Med. Stiff Clay (CL)	12.5	139.5				
BH-1	4B	14.5	87.27	Alluvium (CL)	21.9	133.6				
BH-1	6B	23.7	78.07	Alluvium (SM)	18.7	134.1				
BH-1	7B	30.8	70.97	Alluvium (SP-SM)	16.6	130.4				6.9
BH-1	8	34.5	67.27	Alluvium (SW-SM)						9.9
BH-1	10B	44.7	57.07	Alluvium (CL)	18.1	132.3				
BH-1	12B	54.6	47.17	Alluvium (GW-GM)						7.4
BH-1A	1	4.0	97.77	Bay Mud (CH)	75.3	95.7	84	29	55	
BH-1A	4	8.5	93.27	Med. Stiff Clay (CL)	22.3	132.2	32	13	19	
BH-1A	6B	69.5	32.27	Alluvium (CL)	17.3	131.9				
BH-1A	7B	79.8	21.97	Alluvium (SM)	16.6	130.4				20.9
BH-2	1B	2.0	100.26	Fill (SC)	14.6	128.3				
BH-2	4B	9.5	92.76	Alluvium (SC)	14.7	138.3				
BH-2	6	19.0	83.26	Alluvium (CL)	17.4	135.1	29	13	16	
BH-3	1C	2.0	102.56	Fill (SP)	11.9	103.3				
BH-3	3	9.0	95.56	Alluvium (ML)	17.0	133.3				
BH-3	5B	19.2	85.36	Alluvium (CL)	18.1	133.1				
BH-3	8A	34.3	70.26	Alluvium (SM)	16.0	128.5				13.6
BH-3	9	39.0	65.56	Alluvium (CL)	15.7	136.6				
BH-3	12B	59.2	45.36	Alluvium (CH)	19.1	130.2				
BH-4	1C	2.0	104.44	Fill (SM)	7.6	134.1				
BH-4	3B	9.5	96.94	Med. Stiff Clay (CL)	13.6	137.3				
BH-5	2	2.5	101.48	Bay Mud (CL)			49	17	32	
BH-5	2	2.5	101.48	Bay Mud (CH)	28.8	113.7				
BH-6	1B	1.5	104.28	Fill (GC)	11.7	128.9				
BH-6	4	6.0	99.78	Bay Mud (CH)	27.3	123.9				
BH-6	5	9.0	96.78	Alluvium (CL)	14.7	133.0	38	14	24	
BH-7	2	4.0	97.57	Bay Mud (CH)	23.4	130.0				
BH-7	3B	9.5	92.07	Alluvium (SC)	15.6	135.0				

ARUP

Site:HAYWARD PARK GREEN
Location:CPT-04

Oversite:F.GREGURAS
Date:11:07:06 02:44

File: 376C04.PPC
Depth (m): 6.95
(ft): 22.80
Duration : 475.0s
U-min: -8.81 40.0s
U-max: 7.77 475.0s

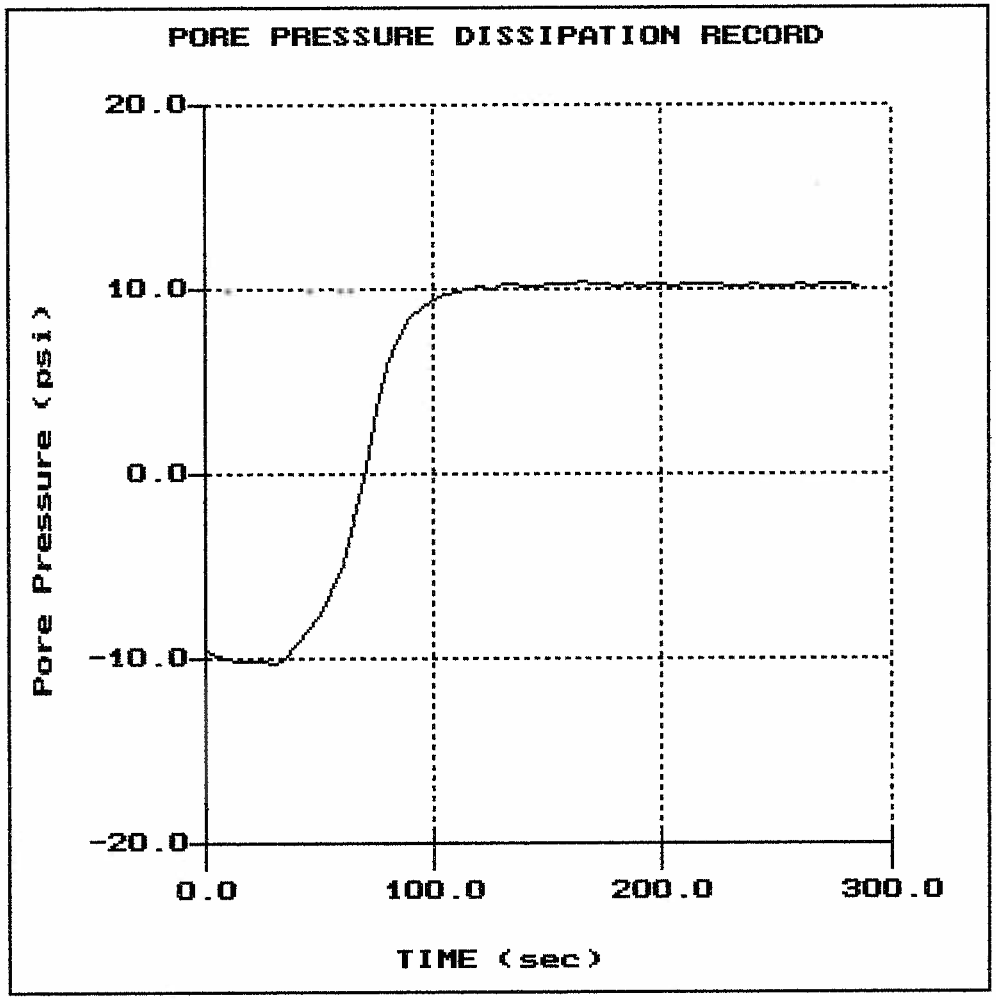


ARUP

Site:HAYWARD PARK GREEN
Location:CPT-05

Oversite:F.GREGURAS
Date:11:07:06 04:45

File: 376C05.PPC
Depth (m): 8.25
(ft): 27.07
Duration : 285.0s
U-min: -10.31 30.0s
U-max: 10.38 165.0s



BORING BH-3

DEPTH IN FEET	TESTS REPORTED ELSEWHERE	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	TYPE OF TEST	NORMAL OR CONFINING PRESSURE (PSF)	SHEAR STRENGTH (PSF)	MOISTURE CONTENT (%)	TOTAL DENSITY (PCF)	TYPE OF SAMPLER	SAMPLING RESISTANCE
							11.9	103.3	MC	22
									MC	9
				TXUU	1800	1799	17.0	133.3	ST	200 psi
									ST	125 psi
							18.1	133.1	MC	21
									MC	33
									PT	
	SA FC(13.6)						16.0	128.5	MC	48

Column groups: LABORATORY TEST DATA (ATTERBERG LIMITS; STRENGTH DATA); SAMPLING. Depth scale: 0, 5, 10, 15, 20, 25, 30, 35.

SAMPLES | SYMBOL | DESCRIPTION

SM
2-inches asphalt.
2-inches asphalt.
SANDY SILT: dark brown, with some gravel, moist. [FILL]

SP
SILTY FINE SAND: dark brown, with some gravel, moist.
Thin layer of fly ash at 2.5 feet.

CH
FAT CLAY: dark gray to black, with trace gravel, moist, medium stiff. [BAY MUD]
PTV: S_u = 880 psf at 4.6 feet

CL
LEAN CLAY: dark brown, with trace sand and occasional gravel, moist, stiff, some iron oxide. [ALLUVIUM]
PP: S_u = 1,120 psf at 5.1 feet
Grades less plasticity at 7.5 feet.

ML
SILT: olive brown, with sand and occasional gravel, moist, with reddish brown mottling, hard.
PP: S_u = 4,500 psf at 9.0 feet

SM
SILTY SAND: dark yellowish brown, fine to medium, with occasional gravel, moist.

ML
CLAYEY SILT: olive brown, with trace of fine sand, very moist, very stiff, with gray and black mottling.
Grades with occasional gravel and to lean clay at 15.0 feet.

CL
LEAN CLAY: reddish brown, moist, stiff.
PP: S_u = 2,133 psf at 16.2 feet
Grades less plasticity and sandier at 19.0 feet. PP: S_u = 2,250 psf at 19.2 feet
Grades with reddish brown mottling and very stiff at 24.5 feet.
PP: S_u = 3,500 psf at 24.8 feet
Grades to very dark gray brown with dark red clay nodules at 29.0 feet.

SM
SILTY FINE SAND: very dark gray, very moist, medium dense to dense.
Grades silty gravelly sand, dark grayish brown, sand

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Also See Notes on Page 3 of 3

Job No: 131178 Pt. ID: HAYWARDPARK_SOIL.GPJ / BH-3	ARUP	**Log of Boring**
Date Completed: 11/2/06 Boring Depth: 60.5 ft.	Surface Elev: 104.56 Adjusted City of San Mateo Datum Coordinates: N: 6097.9825 ; E: 5986.7123	Location: San Mateo, CA

BORING BH-3

DEPTH IN FEET	TESTS REPORTED ELSEWHERE	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	TYPE OF TEST	NORMAL OR CONFINING PRESSURE (PSF)	SHEAR STRENGTH (PSF)	MOISTURE CONTENT (%)	TOTAL DENSITY (PCF)	TYPE OF SAMPLER	SAMPLING RESISTANCE
35										
				TXUU	5000	1354	15.7	136.6	PT	
40										
45										
									MC	69
50										
									SPT	43
55										
							19.1	130.2	MC	42
60										
65										
70										

(Column groups: LABORATORY TEST DATA — ATTERBERG LIMITS, STRENGTH DATA; SAMPLING)

SYMBOL	DESCRIPTION
	is fine to medium, gravel is fine to medium, wet. Grades to clean sand with less gravel at 35.0 feet.
CL	SANDY CLAY: olive brown, moist, stiff to very stiff. Grades more gravelly at 39.0 feet.
SM	SILTY FINE SAND: olive brown with rust stained mottling, moist, dense. Lens of coarse sand and gravel at 45.0 feet. Grades coarser and more gravel.
SP	MEDIUM SAND: dark yellowish brown, with trace fine gravel, wet, dense.
GP	SANDY GRAVEL: dark olive brown, wet, dense, subrounded to subangular.
CL	CLAY: dark olive brown, with sand and gravel, moist, hard. Grades to fat clay at 58.8 feet.
CH	FAT CLAY: grayish brown with mottling. PP: S_u = 3,500 psf at 59.2 feet. Grades with black specks at 59.5 feet.

Borehole terminated at 60.5 feet.

NOTES:

1. Borehole was drilled with a truck mounted Failing 1500 rotary drill rig. Drilling and sampling started on 11/02/2006 at 3:18 AM and completed on 11/02/2006 at 7:40 AM.

2. Borehole advanced dry using a 6-inch-diameter garbage barrel to 3.0 feet and a 6-inch-diameter flight auger to 12.0 feet. After 12.0 feet, borehole advanced using a 4 7/8-inch drag bit and rotary wash drilling method. For rotary drilling, a 5-inch-diameter steel casing set to 13.0 feet.

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Also See Notes on Page 3 of 3

BORING BH-3

DEPTH IN FEET	TESTS REPORTED ELSEWHERE	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	TYPE OF TEST	NORMAL OR CONFINING PRESSURE (PSF)	SHEAR STRENGTH (PSF)	MOISTURE CONTENT (%)	TOTAL DENSITY (PCF)	TYPE OF SAMPLER	SAMPLING RESISTANCE	SAMPLES	SYMBOL	DESCRIPTION
	LABORATORY TEST DATA	ATTERBERG LIMITS		STRENGTH DATA					SAMPLING				
70													3. Samplers used: Modified California (3.0-inch O.D.), Shelby Tube (3.0-inch O.D.), Pitcher Barrel (3.0-inch O.D.), and SPT (2.0-inch O.D.). Automated hammer (140 lbs) was used to drive Modified California and SPT samplers. 4. No groundwater encountered within 12.0 feet during drilling using dry method. Groundwater level was not measured due to rotary drilling method. 5. Borehole backfilled with neat cement grout from 60.5 feet to surface using tremie method on 11/02/2006. 6. See PlateA-10 for Soil Classification Chart and Key to Test Data and Sampler Type.
75													
80													
85													
90													
95													
100													
105													

BORING BH-4

Depth in Feet	Tests Reported Elsewhere	Atterberg Limits: Liquid Limit (%)	Atterberg Limits: Plasticity Index (%)	Strength Data: Type of Test	Strength Data: Normal or Confining Pressure (PSF)	Strength Data: Shear Strength (PSF)	Moisture Content (%)	Total Density (PCF)	Sampling: Type of Sampler	Sampling: Sampling Resistance	Symbol	Description
0												3 1/4-inch asphalt.
							7.6	134.1	MC	79	SM	SILTY SAND WITH GRAVEL: very dark gray, well compacted, dry in upper 8-inches then moist. [FILL] Grades to dark gray at 2.5 feet.
									ST	75 psi	CH	CLAY: very dark gray, with black specks and decayed vegetation, moist, stiff. [BAY MUD/MARSH DEPOSIT]
5											CL-CH	CLAY: grades less plastic and to black. PTV: S_u = 1,000 psf PP: S_u = 600 psf at 5.5 feet
											CL	SANDY CLAY: dark gray, very moist, stiff. [ALLUVIUM] Grades to olive brown sandy clay with gravel and reddish brown mottling, moist, very stiff.
							13.6	137.3	MC	35		
10											ML	PP: S_u = 625 psf at 9.5 feet Grades to sandy silt.
											GP-SP	Layer of coarse sand and gravel.
											GC	GRAVEL: dark olive brown, with sandy clay matrix, fine to coarse gravel, subrounded to angular very moist to wet, medium dense.
15									SPT	18		
											CH	FAT CLAY: olive brown, with occasional gravel, stiff, moist.
20									SPT	8		
25												
30												
35												

Borehole terminated at a depth of 20.5 feet.

NOTES:

1. Borehole was drilled with a truck mounted Failing 1500 rotary drill rig. Drilling and sampling started on 11/04/2006 at 5:22 AM and completed on 11/04/2006 at 7:05 AM.

2. Borehole advanced dry using a 6-inch-diameter garbage barrel to 3.0 feet, and then advanced using a rotary wash method with a 4 7/8-inch drag bit.

3. Samplers used: Modified California (3.0-inch O.D.), SPT (2.0-inch O.D.), and Shelby Tube (3.0-inch O.D.). Automated hammer (140 lbs) was used to drive Modified California and SPT samplers.

4. Groundwater level was not measured due to rotary drilling method.

5. Borehole backfilled with neat cement grout from 20.5 feet to surface using tremie method on 11/04/2006.

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Also See Notes on Page 2 of 2

Job No: 131178 Pt. ID: HAYWARDPARK_SOIL.GPJ / BH-4	ARUP	**Log of Boring**
Date Completed: 11/4/06 Boring Depth: 20.5 ft.	Surface Elev: 106.44 Adjusted City of San Mateo Datum Coordinates: N: 6381.5146 ; E: 5798.7651	Location: San Mateo, CA

BORING BH-4

DEPTH IN FEET	LABORATORY TEST DATA								SAMPLING		SAMPLES	SYMBOL	DESCRIPTION
		ATTERBERG LIMITS		STRENGTH DATA									
	TESTS REPORTED ELSEWHERE	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	TYPE OF TEST	NORMAL OR CONFINING PRESSURE (PSF)	SHEAR STRENGTH (PSF)	MOISTURE CONTENT (%)	TOTAL DENSITY (PCF)	TYPE OF SAMPLER	SAMPLING RESISTANCE			
35													6. See PlateA-10 for Soil Classification Chart and Key to Test Data and Sampler Type.
40													
45													
50													
55													
60													
65													
70													