Appendix F1

Geotechnical Investigation

ROCKRIDGE GEOTECHNICAL

Prepared for **The Sobrato Organization**

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
123 INDEPENDENCE DRIVE
MENLO PARK, CALIFORNIA

April 27, 2021
Project No. 20-1950

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April 27, 2021
Project No. 20-1950

Ms. Sierra Sousa
The Sobrato Organization
599 Castro Street, Suite 400
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Subject: Geotechnical Investigation Report
Proposed Residential Development
123 Independence Drive
Menlo Park, California

Dear Ms. Sousa:

We are pleased to present our geotechnical investigation report for the proposed residential development to be constructed at 123 Independence Drive in Menlo Park, California. Our services were provided in accordance with our proposal dated April 10, 2020.

The subject property consists of five contiguous parcels (119, 123-125, and 127 Independence Drive, 1205 Chrysler Drive, and 130 Constitution Drive) encompassing a total of about 8.15 acres. The parcels form a "T" shape with maximum plan dimensions of about 555 by 865 feet. The site is bordered by Constitution Drive to the north, Independence Drive to the south, commercial properties to the west, and commercial properties and Chrysler Drive to the east. Each parcel is currently occupied by a one- to two-story commercial/industrial building surrounded by asphalt-paved driveways and parking lots and landscaped areas.

Current development plans (prepared by Studio T Square Architecture, dated July 08, 2020 and subsequent electronic mail correspondence with you on April 21, 2021) include demolishing the existing buildings and surrounding improvements and constructing a residential development that will include:

- An apartment building on the northern portion of the site, fronting Constitution Drive. The building will consist of four levels of residential units over two levels of podium parking; the lower parking level will be below-grade. Along the northern, southern, and western sides of the building, five levels of residential units will be constructed at-grade.

- Twenty-three low-rise townhome buildings that will be three stories high with ground-level parking garages/stalls.

- Other site improvements, including a new playground, community gathering spaces, surface parking lots and driveways, and hardscapes and landscapes.

From a geotechnical standpoint, we conclude the proposed development can be constructed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concerns at the site are the placement of 3.2 to 5.2 feet of new, engineered fill to raise site grades to Elevation 13 feet that will result in ground settlement, potentially liquefiable soil layers underlying the site that can result in liquefaction-induced, differential settlement and reduction in bearing capacity at localized areas, and providing adequate foundation support.

We conclude the proposed apartment building may be supported on a mat foundation, provided the static and seismic-induced settlements presented in the report are acceptable. Excavation for the below-grade parking for the apartment building will require temporary shoring and dewatering. We conclude the proposed townhome buildings may be supported on mat foundations or post-tensioned slabs-on-grade (P-T slab).

The recommendations contained in our report are based on a limited subsurface exploration and laboratory testing program. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe excavation, grading, and foundation installation, during which time we may make changes in our recommendations, if deemed necessary.

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

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC.

Linda H.J. Liang, G.E.
Associate Engineer

Quintin A. Flores, P.E.
Project Engineer

Enclosure

QUALITY CONTROL REVIEWER:

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

ROCKRIDGE GEOTECHNICAL

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GEOTECHNICAL INVESTIGATION REPORT
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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed residential development to be constructed at 123 Independence Drive in Menlo Park, California. The site is located on the northern side of Independence Drive, west of its intersection with Chrysler Drive, as shown on the Site Location Map, Figure 1.

The subject property consists of five contiguous parcels (119, 123-125, and 127 Independence Drive, 1205 Chrysler Drive, and 130 Constitution Drive) encompassing a total of about 8.15 acres. The parcels form a "T" shape with maximum plan dimensions of about 555 by 865 feet. The site is bordered by Constitution Drive to the north, Independence Drive to the south, commercial properties to the west, and commercial properties and Chrysler Drive to the east, as shown on the Site Plan, Figure 2. Each parcel is currently occupied by a one- to two-story commercial/industrial building surrounded by asphalt-paved driveways and parking lots and landscaped areas.

Current development plans[1] include demolishing the existing buildings and surrounding improvements and constructing a mixed-use development that will include:

- An apartment building on the northern portion of the site, fronting Constitution Drive. The building will consist of four levels of residential units over two levels of podium parking; the lower parking level will be below-grade. Along the northern, southern, and western sides of the building, five levels of residential units will be constructed at-grade.

[1] *123 Independence, Menlo Park 30% Schematic Design*, prepared by Studio T Square Architecture and dated July 08, 2020, and subsequent electronic mail correspondence, dated April 21, 2021

- Twenty-three low-rise townhome buildings that will be three stories high with ground-level parking garages/stalls.

Other site improvements include a new playground, community gathering spaces, surface parking lots and driveways, and hardscapes and landscapes.

Based on the topographic survey conducted by Kier & Wright, current site grade varies from approximately Elevation 7.8 feet to 9.8 feet[2]. We understand the finished grade for the proposed development will be at Elevation 13 feet, which corresponds to approximately 2.6 feet above the 5-foot FEMA floodplain. Therefore, between approximately 3.2 and 5.2 feet of engineered fill will be placed to reach proposed finished grades. We also understand the finished floor for ground-floor levels of proposed buildings will be at Elevation 13 feet, and the finished floor for the basement level of the apartment building will be at Elevation 2.4 feet.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal dated April 10, 2020. Our scope of services consisted of evaluating subsurface conditions at the site by performing twenty cone penetration tests (CPTs), drilling six test borings, performing laboratory testing on selected soil samples, and performing engineering analyses to develop conclusions and recommendations regarding:

- subsurface conditions
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- design groundwater table
- estimates of static settlement due to placement of new engineered fill to raise site grades
- the most appropriate foundation type(s) for the proposed buildings
- design criteria for the recommended foundation type(s), including vertical and lateral capacities

[2] *Existing Conditions* plan, prepared by Kier and Wright, dated July 8, 2020. NAVD 88 Datum.[3] Highly expansive soil undergoes large volume changes with changes in moisture content.

- estimates of static and seismically-induced foundation settlement
- slab-on-grade floors
- lateral earth pressures for permanent below-grade walls
- temporary cut slopes and shoring
- dewatering
- site grading, fill placement, and excavation, including criteria for fill quality and compaction
- corrosivity of the near-surface soil and the potential effects on buried concrete and metal structures and foundations, and recommendations for corrosion protection
- 2019 California Building Code (CBC) site class and mapped design spectral response acceleration parameters
- rigid and flexible pavement design
- permeable and non-permeable pavers
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

We explored the subsurface conditions at the site by performing twenty CPTs and drilling six test borings. Prior to performing the CPTs and drilling the borings, we filed drilling notification forms with San Mateo County Environmental Health (SMCEH) for each parcel 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 boring and CPT locations were clear of underground utilities. Details of our field exploration are described in this section.

3.1 Cone Penetration Tests

Twenty CPTs, designated as CPT-1 through CPT-20, were performed by Middle Earth Geo Testing, Inc. of Orange, California on December 17, 18, and 21, 2020 at the approximate locations shown on Figure 2. The CPTs were advanced to depths of 50 and 100 feet below ground surface (bgs) 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. A special cone was also used to measure the in-situ soil shear wave velocity in approximately five-foot intervals at CPT-6 and CPT-14. Soil data, including tip resistance, frictional resistance, and shear wave velocity, 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 and approximate strength characteristics of the soil encountered. The CPT logs showing tip resistance and friction ratio, as well as interpreted soil behavior type and shear wave velocity profiles, are presented on Figures A-1 through A-20 in Appendix A.

Upon completion, the CPT holes were backfilled with neat cement grout in accordance with SMCEH requirements, and the pavement surface was patched with quick-set concrete.

3.2 Test Borings

Six test borings, designated as Borings B-1 through B-6, were drilled on December 22 and 23, 2020 by Exploration Geoservices, Inc. (EGI) of San Jose, California at the approximate locations shown on Figure 2. The borings were drilled to depths between 30-1/2 and 45 feet bgs using a Mobile B-61 drill rig equipped with eight-inch-outside-diameter, hollow-stem augers. During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. Boring logs were prepared based on laboratory test data and the conditions recorded on the field logs and are presented on Figures A-21 through A-26b in Appendix A. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-27.

Soil samples were obtained using the following samplers:

- Modified California (MC) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter stainless steel tubes.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter; sampler can accommodate liners, but liners were not used.

- Shelby Tubes (ST) - thin-walled steel tubes with a 3.0-inch outside diameter and 2.875-inch inside diameter.

The MC and SPT samplers were driven with a 140-pound, downhole, wireline hammer falling about 30 inches per drop. The samplers were driven up to 18 inches, and the hammer blows required to drive the samplers were recorded every six inches and are presented on the boring logs. A “blow count” is defined as the number of hammer blows per six inches of penetration or 50 blows for six inches or less of penetration. The blow counts required to drive the MC and SPT samplers were converted to approximate SPT N-values using factors of 0.63 and 1.08, respectively, to account for sampler type, approximate hammer energy (previously measured by drilling subcontractor), and the fact that the SPT sampler was designed to accommodate liners, but liners were not used. The blow counts used for this conversion were the last two 6-inch blow counts, the last one blow count if the sampler was driven more than six inches but less than 12 inches, or the only blow count if the sampler was driven six inches or less. The converted SPT N-values are presented on the boring logs.

The Shelby tubes were used in an attempt to obtain relatively undisturbed samples of medium stiff, fine-grained soils. The Shelby tubes were slowly advanced using the weight of the drill rods and hydraulic pressure, as needed.

Upon completion of drilling, the boreholes were backfilled with cement grout in accordance with SMCEH requirements, and the pavement was patched with quick-set concrete. The soil cuttings from the borings were loaded onto a trailer, removed from the site, and disposed of at a landfill by EGI.

3.3 Laboratory Testing

Laboratory tests were performed on selected soil samples from our borings to assess their engineering properties and physical characteristics. Soil samples were tested by B. Hillebrandt Soils Testing, Inc. of Alamo, California to measure moisture content, dry density, plasticity (Atterberg limits), fines content, and undrained shear strength. The results of the geotechnical laboratory tests are presented on the boring logs and attached in Appendix B.

Soil corrosivity testing was also performed on near-surface soil samples by Project X Corrosion Engineering of Murrieta, California. The results of the soil corrosivity testing are presented in Appendix C.

4.0 SUBSURFACE CONDITIONS

As presented on the Regional Geologic Map (Figure 3), the site is mapped as being underlain by Holocene-age alluvial deposits (Qha). The results of our borings and CPTs indicate the alluvium primarily consists of stiff to very stiff clay with occasional medium stiff layers up to about two feet thick. The clay is interbedded with layers of medium dense to very dense sand and gravel to the maximum depth explored of about 100 feet bgs. The granular layers encountered at this site varied in thickness from 1 to 9 feet. Below depths of about 32 feet bgs (northwest corner of the site) and 52 feet bgs (southeast corner of the site), the clays become very stiff to hard, and the sand and gravels become dense to very dense.

The results of Atterberg limits tests performed on near-surface soil samples obtained from the borings indicate much of the near-surface soil consists of clay that is very highly expansive[3].

4.1 Groundwater

Groundwater was estimated in CPT soundings by performing pore pressure dissipation tests and by measuring the water level in the CPT holes with a weighted tape measure immediately following removal of the CPT rods. In borings, depth to groundwater was recorded when first encountered and right after drilling. Results of our groundwater measurements taken from the CPTs and borings indicate the depth to groundwater ranged from about 4-1/2 and 7 feet bgs at the time of our field investigation. The site is located approximately 600 feet south of the current bay margin; due to this proximity, the groundwater level at the site may experience tidal fluctuations. The groundwater level at the site is also expected to fluctuate several feet seasonally, depending on the amount of rainfall. We conclude a high groundwater level at

[3] Highly expansive soil undergoes large volume changes with changes in moisture content.

Elevation 6 feet (corresponding to about of 1.8 to 3.8 feet below existing grades) should be used for this project.

5.0 SEISMIC CONSIDERATIONS

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards, including ground shaking, ground surface rupture, liquefaction[4], lateral spreading[5] and cyclic densification[6]. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity and Faulting

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 and North American plates 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, Hayward, and Calaveras faults. These and other faults in the region are shown on Figure 4. Numerous damaging earthquakes have occurred along these faults in recorded time. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[7] [Petersen et al. (2014) & Thompson et al. (2016)] are summarized in Table 1. These

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

[5] 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.

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

[7] Moment magnitude (M_w) 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.

references are based on the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013).

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Characteristic Moment Magnitude
Monte Vista - Shannon	7.8	Southwest	7.14
Total North San Andreas (SAO+SAN+SAP+SAS)	10	Southwest	8.04
North San Andreas (Peninsula, SAP)	10	Southwest	7.38
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	20	East	7.58
Hayward (South, HS)	20	East	7.00
San Gregorio (North)	24	West	7.44
Butano	25	Southwest	6.93
Total Calaveras (CN+CC+CS+CE)	29	East	7.43
Calaveras (North, CN)	29	East	6.86
Calaveras (Central, CC)	32	East	6.85
Hayward (North, HN)	33	North	6.90
Hayward (Extension, HE)	35	East	6.18
Las Positas	35	East	6.50
Zayante-Vergeles (2011 CFM)	36	Southwest	7.48
North San Andreas (Santa Cruz Mts, SAS)	38	Southeast	7.15
Mount Diablo Thrust South	40	Northeast	6.50
Mount Diablo Thrust North CFM	40	Northeast	6.72
Mount Diablo Thrust	41	Northeast	6.67
Sargent	43	Southeast	6.71
Zayante-Vergeles	48	Southeast	7.00
Greenville (North)	49	Northeast	6.86
Concord	49	Northeast	6.45

Since 1800, four major earthquakes have been recorded on the North 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 Loma Prieta Earthquake of October 17, 1989 had an M_w of 6.9 and occurred about 56 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 (estimated 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 ($M_w = 6.2$).

As a part of the UCERF3 project, researchers estimated that the probability of at least one $M_w \geq 6.7$ earthquake occurring in the greater San Francisco Bay Area during a 30-year period (starting in 2014) is 72 percent. The highest probabilities are assigned to sections of the Hayward (South), Calaveras (Central), and the North San Andreas (Santa Cruz Mountains) faults. The respective probabilities are approximately 25, 21 , and 17 percent.

5.2 Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong ground shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction, lateral spreading, and cyclic densification.

5.2.1 Ground Shaking

The seismicity of the site is governed by the activity of the San Andreas and Hayward faults, although ground shaking from future earthquakes on other faults, including the Monte Vista-Shannon and Calaveras faults, will also be felt at the site. These and other faults in the region are shown in relation to the site on Figure 4. 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) site-specific soil conditions. We judge that strong to very strong ground shaking could occur at the site during a large earthquake on one of the nearby faults.

5.2.2 Liquefaction and Associated Hazards

When a saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by strong ground motion. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction.

The subject property is located in an area of Menlo Park designated as a potential liquefaction hazard zone on the map prepared by California Geological Survey (CGS) titled *State of California, Earthquake Zones of Required Investigation, Palo Alto Quadrangle*, dated October 18, 2006 (Figure 6). Special Publication 117 prepared by the CGS (2008) recommends subsurface investigations in mapped liquefaction potential areas be performed using rotary-wash borings and/or CPTs.

We evaluated the liquefaction potential of soil encountered below groundwater at the site using data collected in the CPTs with consideration of subsurface information from the hollow-stem borings and laboratory test results. We assessed the liquefaction susceptibility using the software CLiq 3.0.2.4 (GeoLogismiki, 2020). CLiq uses measured CPT data and assesses liquefaction susceptibility and 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 calculated “free-field” liquefaction-induced settlements of these layers and then modified the settlements using the methodology proposed by Çetin et al. (2009) to account for the depth of the liquefiable layers.

Our analyses were performed using an assumed high groundwater depth of three feet below existing grades for the “during earthquake” groundwater level. In accordance with the 2019

CBC, we used a peak ground acceleration of 0.69 times gravity (g) in our liquefaction evaluation; this peak ground acceleration is consistent with the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). We also used a moment magnitude 8.04 earthquake, which is consistent with the characteristic moment magnitude for the San Andreas fault, as presented in Table 1.

Most of the soils at the site are sufficiently cohesive and/or dense to resist liquefaction, however, several layers of potentially liquefiable material were encountered in the CPTs below a depth of nine feet bgs. The layers consist of loose to medium dense sand to silty sand/sandy silt that are discontinuous and vary from about 6 inches to 5 feet in thickness. We estimate total ground surface settlement associated with liquefaction (referred to as post-liquefaction reconsolidation) following a major earthquake on a nearby fault will be up to one inch, with differential settlement of up to 1/2 inch over a horizontal distance of 30 feet.

Ishihara (1985) presented an empirical relationship that provides criteria used to evaluate whether liquefaction-induced ground failure, such as sand boils, would be expected to occur under a given level of shaking for a liquefiable layer of given thickness overlain by a resistant, or protective, surficial layer. We conclude the non-liquefiable soil overlying the potentially liquefiable soil layers is sufficiently thick such that the potential for liquefaction-induced ground failure at the ground surface is low. There are lenses of potentially liquefiable soil slightly below proposed basement subgrade that may result in a reduction in bearing capacity during a major seismic event in localized areas.

Considering the potentially liquefiable layers are not continuous, we also conclude the risk of lateral spreading is nil.

5.2.3 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The soil above the groundwater at the site primarily consists of fine-grained deposits that are sufficiently cohesive or coarse-grained deposits that are sufficiently dense, such that they are not susceptible to cyclic densification. Therefore, we conclude the potential for cyclic densification to impact the proposed development is very low.

5.2.4 Ground Surface Fault Rupture

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

6.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the project is designed and constructed in accordance with the geotechnical recommendations presented herein. The primary geotechnical concerns affecting the proposed development include:

- placement of 3.2 to 5.2 feet of new engineered fill to raise site grades to Elevation 13 feet that will result in ground settlement,
- potentially liquefiable soil layers underlying the site that can result in liquefaction-induced differential settlement and reduction in bearing capacity at localized areas, and
- providing adequate foundation support.

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

6.1 Design Groundwater Table

As discussed in Section 4.1, we recommend a design groundwater table at Elevation 6 feet (corresponding to about of 1.8 to 3.8 feet below existing grades) be used for this project. Any below-grade walls, building foundations and floor slabs extending below the design groundwater table should be waterproofed and designed to resist hydrostatic pressures.

6.2 Expansive Soil

Atterberg limits tests performed on samples of the existing near-surface clay indicate the surficial soil has very high expansion potential. Expansive near-surface soil is subject to volume changes during fluctuations in moisture content. These volume changes can cause movement and cracking of foundations, pavements, slabs, and below-grade walls. Up to about 3.2 to 5.2 feet of engineered fill will be placed to raise the site grade to Elevation 13 feet. We judge the effects of expansive soil will be mitigated by using non-expansive soil as engineered fill within the zone of severe moisture change (i.e. within the upper two feet of finished soil subgrade).

6.3 New Fill and Settlement

We understand site grading for the proposed development will include placing between 3.2 and 5.2 feet of engineered fill to raise the site grades to Elevations 13 feet. The site is underlain by clayey soil that is slightly to moderately overconsolidated. When new fill is placed, a new cycle of consolidation will begin, and settlement will occur due to consolidation of the underlying clayey soil. We estimate about 1/2 and 1-1/2 inches of settlement will occur for 3.2 and 5.2 feet of engineered fill placed on the site, respectively. Because most of the settlement that will occur will consist of loading below the preconsolidation pressure (i.e., loading in recompression) rather than primary consolidation, at least half of the settlement is expected to occur within a few months of fill placement. We anticipate the remaining settlement will occur slowly over a period of several years.

Static settlement will affect various aspects of the planned development, including utilities and building entrances. Design of these elements should incorporate the effects of the predicted settlement. The potential adverse impacts of the settlement on the proposed improvements can be limited by placing the fill a minimum of three months prior to construction of the foundations for the proposed buildings.

6.4 Foundations and Settlement

The factors influencing the selection of a safe, economical foundation system are adequate foundation support, total and differential settlement of the structure resulting from engineered fill and building loads, and liquefaction-induced ground settlement and reduced bearing capacity of foundation soils. Our conclusions for appropriate foundation systems and estimated foundation settlements are presented in this section.

6.4.1 Apartment Building with Basement

Based on the results of our investigation, we anticipate the foundation of the proposed apartment building with one basement level will be underlain by alluvium that can provide adequate foundation support for the new building loads without excessive static settlement; however, the foundation level may be underlain by potentially liquefiable soil layers in localized areas that may result in reduction in bearing capacity for shallow foundations during a major seismic event. We judge spread footings bearing on localized liquefiable layers may experience bearing failures during a major seismic event. Therefore, we judge conventional spread footings are not appropriate for support of the proposed apartment building with one basement level. We conclude, however, the proposed apartment building with one basement level may be supported on a mat foundation.

The proposed finished floor for the apartment building basement will be at Elevation 2.4 feet, corresponding to about 3.6 feet below the design groundwater table. The mat foundation will need to be designed to resist hydrostatic uplift pressures and be waterproofed. A mat foundation system generally simplifies the detailing of the waterproofing system, and the weight of a mat foundation will provide greater resistance to the hydrostatic uplift pressures.

We estimate total and differential settlements of properly constructed mat foundation designed using the recommendations presented in Section 7.3 of this report will be less than 3/4 inch. The amount of differential settlement between columns will be a function of the mat stiffness and hence its ability to spread the loads between columns, however, we expect the mat can be designed to limit differential settlements to 1/2 inch in 30 feet. As discussed in Section 5.2.2, the mat should be designed for an additional one inch of total liquefaction-induced settlement and 1/2 inch of liquefaction-induced differential settlement over a horizontal distance of 30 feet.

6.4.2 Apartment Building (At-Grade)

Along the northern, southern, and western sides of the apartment building, there will be five levels of residential units constructed at-grade. The proposed at-grade portions of the apartment building will be underlain by engineered fill overlying native alluvium. We judge that the anticipated total and differential settlements due to engineered fill loads, static foundation loads, and post-liquefaction reconsolidation will exceed the typical tolerance of a conventional spread footing foundation system. Therefore, we judge conventional spread footings are not appropriate for support of the at-grade portions of the proposed apartment building. We conclude the at-grade portions of the proposed apartment building may be supported on a reinforced concrete mat, provided the static and liquefaction-induced settlements are acceptable from a structural standpoint.

Structural design loads were not available at the time this report was prepared. Based on our experience with similar buildings, we estimate the bearing pressure due to dead-plus-live loads imposed by the mat for the at-grade portions of the apartment building will be on the order of 550 pounds per square foot (psf). Therefore, we anticipate total and differential static settlement of the mat foundation will be less than 3/4 inch and 1/2 inch across a horizontal distance of 30 feet, respectively. As presented in Section 6.3, we estimate about 1/2 and 1-1/2 inches of additional static settlement will occur for 3.2 and 5.2 feet of engineered fill placed to raise the site grade to finish subgrade, respectively. We estimate total and differential settlements due to the engineered fill can be reduced to approximately 1/2 inch and 1/4 inch over a horizontal distance of 30 feet by placing the fill at least three months prior to building construction.

As discussed in Section 5.2.2, the mat should be designed for an additional one inch of total liquefaction-induced settlement and 1/2 inch of liquefaction-induced differential settlement over a horizontal distance of 30 feet.

6.4.3 Townhome Buildings

The proposed townhome buildings will be three stories high and constructed at-grade with finished floor at Elevation 13 feet. The proposed townhome buildings will be underlain by engineered fill overlying native alluvium. We judge the anticipated total and differential settlements due to engineered fill loads, static foundation loads and post-liquefaction reconsolidation will exceed the typical tolerance of a conventional spread footing foundation system. Therefore, we judge conventional spread footings are not appropriate for support of the proposed townhome buildings. We conclude the townhome buildings may be supported on a reinforced concrete mat or post-tensioned slab-on-grade (P-T slab), provided the static and liquefaction-induced settlements are acceptable from a structural standpoint.

Structural design loads were not available at the time this report was prepared. Based on our experience with similar buildings, we estimate the bearing pressure due to dead-plus-live loads imposed by the mat for the townhome buildings will be on the order of 300 psf. Therefore, we anticipate total and differential static settlement of the mat foundation will be less than 1/2 inch and 1/4 inch across a horizontal distance of 30 feet, respectively. As presented in Section 6.3, we estimate about 1/2 and 1-1/2 inches of additional static settlement will occur for 3.2 and 5.2 feet of engineered fill placed to raise the site grade to finish subgrade, respectively. We estimate total and differential settlements due to the engineered fill can be reduced to approximately 1/2 inch and 1/4 inch over a horizontal distance of 30 feet by placing the fill at least three months prior to building construction.

As discussed in Section 5.2.2, the mat should be designed for an additional one inch of total liquefaction-induced settlement and 1/2 inch of liquefaction-induced differential settlement over a horizontal distance of 30 feet.

6.5 Temporary Cut Slopes and Shoring

We anticipate excavations up to a depth of about 13 feet (below proposed finished grade) will be needed to construct the proposed basement level and mat foundation for the residential building. 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 shoring engineer should be responsible for shoring design. The contractor should be responsible for the construction and safety of temporary slopes and shoring.

There are several key considerations in selecting a suitable shoring system. Those we consider of primary concern are:

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

Several methods of shoring are available; we have qualitatively evaluated the following systems:

- soldier pile-and-lagging
- secant pile wall
- soil-cement mixed (SMX) columns.

6.5.1 Soldier Pile-and-Lagging

A soldier pile-and-lagging system usually consists of steel H-beams and concrete placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds from the top down. Where the required cut is less than about 13 feet, a soldier pile and lagging system can typically provide economical shoring without tiebacks. Where cuts exceed about 13 feet in height, soldier pile and lagging systems are typically more economical if they include tieback anchors.

Where caving soil layers are encountered below the groundwater, installing the soldier piles will require casing or use of drilling slurry to reduce caving of the holes. If drilling slurry is used, or groundwater is present, concrete should be placed using the tremie method. Where voids are developed behind wood laggings, the voids should be promptly filled by hand-packing dry material and/or filling the voids with flowable, sand-cement slurry mix.

6.5.2 Secant Pile Wall

A secant pile wall is a viable option for creating a continuous shoring wall that supports the excavation, as well as providing a hydraulic barrier when properly constructed. A secant pile wall is similar to a conventional soldier pile-and-lagging system, in which steel H-beams and lean concrete are placed in predrilled holes extending below the bottom of the excavation. However, instead of installing wood lagging to support the soil between the reinforced soldier piles, additional shafts are drilled and filled with lean concrete between the soldier piles in an overlapping fashion, such that a continuous wall of lean concrete is created.

6.5.3 Continuous Soil-Cement Mix (SMX) Wall

Similar to a soldier pile-and-lagging wall system, a continuous SMX wall is also a viable option for creating a continuous shoring wall that supports the excavation. An SMX wall can also provide a hydraulic barrier when properly constructed, which is an added benefit given the relatively shallow groundwater at the site. SMX columns are installed by injecting and blending cement into the soil using a drill rig equipped with single or multiple augers/paddles, or a specialized proprietary cutterhead. The soil is mixed with the binder material(s) in situ, forming continuous, overlapping, soil-cement columns or a continuous wall of uniform thickness. Steel beams are placed in the soil-cement columns to provide rigidity. The SMX system, which can also be installed in combination with steel soldier beams and tiebacks, serves to shore the excavation as well as cut off lateral groundwater flow, thus reducing the amount of dewatering required from within the excavation. Soil-cement walls are considered temporary, and permanent building walls are built inside of the soil-cement walls following application of drainage panels and waterproofing.

SMX systems are generally installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of the SMX columns, beams, and tieback elements (if necessary) should be determined by the shoring designer, based on the design soil, water, and surcharge pressures presented in Section 7.6.3 of this report. However, there are numerous factors that influence the quality, consistency, strength, and permeability of the resulting soil-cement mix, which are controlled by the materials, methods, and equipment employed by the contractor performing the soil mixing. These factors include, but are not limited to:

- Types of binder material(s) used – i.e. cement, bentonite, etc.; wet-mixed vs. dry-mixed.
- Quantities and proportions of binder material(s) used – i.e. water-to-binder ratio; volume ratio of SMX.
- Equipment used to perform the mixing – i.e. single-auger, multi-auger, or cutter-based equipment.
- Plumbness and amount of overlap between adjacent SMX columns.
- Homogeneity of soil-cement mixture – controlled by rate of mixing, number of stages, and equipment used.
- Depth and diameter of predrilling, depending on equipment selected.

A contractor experienced in installing SMX systems in similar soil conditions and below the groundwater table should be responsible for selecting appropriate materials, equipment, and methods based on the soil and groundwater conditions at this site, as well as their expertise, in order to meet the performance criteria established by the shoring designer. The design and construction of a SMX system should also consider the capacity of the dewatering system selected by the contractor.

6.6 Temporary Dewatering

The proposed basement excavation for the residential building will extend about six feet below the design groundwater table. During excavation of the basement level, groundwater will flow into the excavation unless collected and removed prior to reaching the work area. Therefore, a temporary dewatering system should be installed to provide a firm, relatively dry base from which to construct the foundation system. We anticipate an active dewatering system consisting of a series of extraction wells installed outside the excavation would be the most appropriate temporary dewatering system provided that dewatering-induced total and differential settlements of adjacent improvements are within acceptable limits. The potential effects of dewatering on nearby buildings should be evaluated during design of the shoring and dewatering systems. If the estimated settlements are not acceptable, mitigation measures, such as installing a secant pile or SMX wall, should be taken.

Where the temporary shoring system consists of a groundwater cut-off wall (i.e. secant pile wall or SMX wall), we anticipate a passive dewatering system, in which water is collected from a series of trench drains around the perimeter and across the base of the excavation, would be the most appropriate temporary dewatering system to be used in combination with a cut-off wall shoring system.

The method used to shore and dewater the excavation should be the responsibility of the contractor.

6.7 Excavation, Monitoring, and Construction Considerations

The soil to be excavated for the proposed residential basement and mat foundation is expected to consist primarily of clay with occasional thin sand layers which can be excavated with conventional earth-moving equipment, such as backhoes.

During excavation, the shoring system may deform laterally, which could cause the ground surface adjacent to the shoring to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Ground movements due to a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch where there are no improvements within a horizontal distance equal to 1.5 times the height of the shoring and 1/2 inch where there are improvements within that horizontal distance. A monitoring program should be established to evaluate the effects of the excavation on the adjacent buildings and surrounding ground.

The contractor should also survey and take photographs of existing buildings within a horizontal distance 1.5 times the excavation depth prior to the start of construction. The survey points should be used to monitor the vertical and horizontal movements of the shoring and surrounding structures and streets during construction.

6.8 Soil Corrosivity

Corrosivity tests were performed by Project X Corrosion Engineering of Murrieta, California on four selected soil samples obtained from Borings B-1, B-2, B-4, and B-6 at 5.75, 3.0, 5.0, and 3.5 feet bgs, respectively. The corrosivity test results and soil corrosivity evaluation report are presented in Appendix C of this report.

7.0 RECOMMENDATIONS

Our recommendations for site preparation and grading, design of foundations, temporary cut-slope and shoring, and other geotechnical aspects of the project are presented in this section.

7.1 Site Preparation and Grading

Site demolition should include removal of all existing pavements, former foundation elements, and underground utilities. Demolished asphalt concrete should be taken to an asphalt recycling facility. Aggregate base beneath existing pavements and floor slabs (if present) may be re-used as select fill if carefully segregated and meets the requirements for select fill presented in Section 7.1.1. 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 footprint of the proposed improvements or 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 engineered fill under our direction following the recommendations provided later in this section.

Any vegetation and organic topsoil (if present) should be stripped in areas to receive engineered fill or improvements (i.e., building, pavement, or flatwork). Tree roots with a diameter greater than 1/2 inch within three feet of subgrade should be removed.

After site clearing is completed, in areas that will receive fill or improvements (i.e. building pad, exterior concrete flatwork, and pavements), the soil subgrade exposed should be scarified to a depth of at least eight inches, moisture-conditioned, and compacted in accordance with the requirements provided in Table 2 (Section 7.1.1).

7.1.1 Fill Materials and Compaction Criteria

Between about 3.2 and 5.2 feet of engineered fill will be placed to raise site grades to Elevation 13 feet. Engineered fill may consist of on-site soil or imported soil (select fill) that is free of organic matter and debris and contains no rocks or lumps larger than three inches in greatest dimension. To mitigate the effects of very highly expansive near-surface clay, we recommend the engineered fill placed in the upper two feet of soil subgrade consists of non-expansive soil meeting the requirements of select fill presented in the following paragraph.

Select fill should consist of on-site or imported material that is free of organic matter and debris, contains no rocks or lumps larger than four 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. Imported soil should also have a resistivity of 2,000 ohm-cm or above, chloride and sulfate concentrations of 100 mg/kg or less, and a pH between 6.5 and 7.5, or be approved by the Corrosion Engineer. Select fill placed in the upper foot of asphalt pavement subgrade should also have a minimum resistance value (R-value) of 15. 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.

All fill should be placed in horizontal lifts not exceeding eight inches in loose thickness, moisture-conditioned, and compacted in accordance with the requirements provided below in Table 2. Each type of material is described in the following text according to its uses and specifications.

TABLE 2
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
General fill – expansive clay	88 – 93	4+% above optimum
General fill – select fill (less than 5 ft)	90+	Above optimum
General fill – select fill (more than 5 ft)	95+	Above optimum
Utility trench backfill – expansive clay	88 – 93	4+% above optimum
Utility trench backfill – select fill	90+	Above optimum
Utility trench - clean sand or gravel*	95+	Near optimum
Exterior slabs – expansive clay	88 – 93	4+% above optimum
Exterior slabs – select fill	90+	Above optimum
Pavement subgrade –expansive clay	92+	2+% above optimum
Pavement subgrade – select fill	95+	Above optimum
Pavement - aggregate base*	95+	Near optimum

*Note: 1. Clean sand or gravel are granular material with less the five percent fines.
2. Aggregate base is a type of select fill.

Where the above-recommended compaction requirements are in conflict with the City of Menlo Park standard details for pavements and sidewalks within the public right-of-way, the City Engineer or inspector should determine which compaction requirements should take precedence.

7.1.2 Exterior Concrete Flatwork

For all concrete flatwork and exterior slabs, the upper eight inches of soil subgrade should be scarified and recompacted, and the subgrade should be proof-rolled to provide a firm and non-yielding surface. On-site concrete flatwork and exterior slabs can be supported directly on the imported select fill. New public sidewalks, if any, should be underlain by at least four inches of Class 2 aggregate base compacted to at least 90 percent relative compaction.

7.1.3 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All temporary excavations used in construction should be designed, planned, constructed, and maintained by the contractor and should conform to all state and/or federal safety regulations and requirements, including those of CAL-OSHA.

To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of clean 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 clean sand or fine gravel, which should be mechanically tamped with lightweight equipment. The pipe bedding and cover should be eliminated where an impermeable plug is required as described in the following paragraph and detailed on Figure 6. Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted as according to the recommendations previously presented in Table 2. If imported clean sand or gravel (defined as poorly-graded soil with less than five percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. 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 improvements.

Impermeable plugs consisting of lean concrete or sand-cement slurry, at least three feet in length, should be installed in lieu of sand or fine gravel pipe bedding where utility trenches enter the at-grade building footprints. A typical detail for the recommended utility trench low-permeability plug at building perimeters is presented on Figure 6. Furthermore, where sand- or gravel-backfilled trenches cross planter areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the buildings or pavements. This trapped water can cause heaving of soils beneath slabs and softening of subgrade soil beneath pavements.

7.2 Surface Drainage and Landscaping

7.2.1 Surface Drainage

Positive surface drainage should be provided around the buildings to direct surface water away from the 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. The use of water-intensive landscaping around the perimeter of the buildings should be avoided to reduce the amount of water introduced to the expansive clay subgrade.

Care should be taken to minimize the potential for subsurface water to collect beneath flatwork and pavements. Where landscape beds and tree wells are immediately adjacent to pavements and flatwork that are not designed as permeable systems, we recommend vertical cutoff barriers be incorporated into the design to prevent irrigation water from saturating the subgrade and aggregate base. These barriers may consist of either flexible impermeable membranes or deepened concrete curbs.

7.2.2 Landscaping

Prior experience and industry literature indicate that some species of high water-demand[8] trees can induce ground-surface settlement by drawing water from the expansive clay, causing it to shrink. Where these types of trees are planted near buildings, the ground-surface settlement may result in damage to structure. This problem usually occurs 10 or more years after planting, as the trees reach mature height. To reduce the risk of tree-induced, building settlement, we recommend trees of the following genera not be planted within 25 feet of the proposed buildings unless adequate deep irrigation is provided at the tree locations: Eucalyptus, Populus, Quercus, Crataegus, Salix, Sorbus (simple-leafed), Ulmus, Cupressus, Chamaecyparis, and

[8] "Water-demand" refers to the ability of the tree to withdraw large amounts of water from the soil subgrade, rather than soil suction exerted by the root system.

Cupressocyparis. Because this is a limited list and does not include all genera that may induce ground-surface settlement, a tree specialist should be consulted prior to selection of trees to be planted at the site.

7.2.3 Bioswales

Where bioswales will be part of the project, we recommended that bioswales be constructed at least five feet from the buildings and provided with underdrains and/or drain inlets. The subdrain pipes should be installed eight inches above the bottom of the infiltration area for treatment areas that are at least five feet away from the new buildings and pavements. The intent of this recommendation is to allow infiltration into the underlying soil, but to reduce the potential for bio-retention areas to flood during periods of heavy rainfall.

Where it is necessary for a bioswale to be constructed within five feet of the buildings and pavements because of site constraints, the bottom of the bioswale should be lined with an impermeable liner. Where a vertical curb or foundation is constructed near a bioswale, the curb and the edge of the foundation should be founded below an imaginary line extending up at an inclination of 1.5:1 (horizontal: vertical) from the base of the bioswale.

7.3 Foundations

7.3.1 Mat Foundations

For mat design, we recommend using the following moduli of subgrade reaction for dead-plus-live load conditions:

- Apartment building with basement: 20 pounds per cubic inch (pci)
- Apartment building (at-grade): 30 pci
- Townhome buildings (at-grade): 30 pci

The moduli of subgrade reaction values presented above have been reduced to account for the size of the mat/equivalent footings (therefore, this is <u>not</u> k_{v1} for 1-foot-square plate) and may be increased by one-third for total loads. Once the structural engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of subgrade reaction, if appropriate.

Considering the large area of the mat foundations, we expect the average bearing stress under the mats to be relatively low; however, concentrated stresses will occur at column locations and at the edges of the mats. The mats should be designed to impose a maximum bearing pressure of 3,500 psf on the foundation subgrade soil for dead-plus-live load conditions; this pressure may be increased by one-third for total load conditions. The allowable bearing pressures recommended for dead-plus-live and total load conditions include factors of safety of at least 2.0 and 1.5, respectively.

Lateral loads may be resisted by a combination of friction along the base of the mat and passive resistance against the vertical faces of the mat foundation. To compute lateral resistance, we recommend using an equivalent fluid weight (triangular distribution) of 270 pounds per cubic foot (pcf) and 115 pcf above and below the design groundwater table, respectively; the upper foot of soil should be ignored unless confined by a slab or pavement. The allowable friction factor will depend on the type of waterproofing or vapor retarder used at the base of the mat. For bentonite-based water proofing membranes, such as Paraseal or Voltex, a friction factor of 0.12 should be used (assumes a bentonite friction angle of 10 degrees). If Preprufe is used, a base friction factor of 0.20 should be used. Friction factors for other types of waterproofing membranes can be provided upon request. If the mat is underlain by a vapor retarder (for at-grade buildings), a friction factor of 0.20 may be used to compute base friction. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

The mat subgrade for apartment buildings with basements will be sensitive to disturbance due to its proximity to the groundwater table. The final two feet of excavation and fine grading of the basement mat subgrade should be performed with tracked equipment to minimize heavy concentrated loads that may disturb the wet soil. A three-inch-thick rat slab should be placed on the basement mat subgrade to protect it from disturbance during placement of waterproofing and reinforcing steel. The subgrade should be free of standing water, debris, and disturbed materials and be approved by the geotechnical engineer prior to placing a rat slab.

The mat subgrade for at-grade buildings should be also be free of standing water, debris, and disturbed materials, be maintained in a moist condition, and be approved by the geotechnical engineer prior to placing the vapor retarder.

7.3.2 P-T Slab for Townhomes

As an alternate to mats, we conclude the proposed townhomes may be supported on P-T slabs. Geotechnical design recommendations for P-T slabs, such as subgrade preparation, allowable bearing pressures, subgrade modulus, static settlements, and lateral resistance, are the same as those presented above for mats.

The P-T slab subgrade will be underlain by non-expansive fill that is not expansive and not collapsible. For design of P-T slabs for non-expansive, non-collapsible subgrade soil conditions, P-T slabs should be designed as Type II (lightly reinforced against shrinkage and temperature cracking) and/or as a compressible soil using the Post-Tensioned Institute (PTI) methodology.

7.3.3 Vapor Retarder

The mat foundation for the apartment building with basement, which will bottom below the design groundwater table, should be waterproofed.

The mat foundation for the at-grade portions of the apartment building and for the at-grade townhomes should be underlain by a vapor retarder. 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.

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 foundations should have a low w/c ratio - less than 0.45. If necessary, workability should be increased by adding plasticizers. In addition, the mats 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.

7.4 Permanent Below-Grade Walls

Below-grade walls, such as the apartment building basement walls should be designed to resist static lateral earth pressures, lateral pressures caused by earthquakes, vehicular surcharge pressures, and surcharges from adjacent foundations, where appropriate. We recommend restrained below-grade walls at the site be designed for the more critical of the following criteria:

- At-rest equivalent fluid weight of 60 pcf above the design groundwater table and 91 pcf below.
- Active pressure of 40 pcf plus a seismic increment of 28 pcf (triangular distribution) above the design groundwater level, and 82 pcf below the groundwater level plus a seismic increment of 13 pcf (triangular distribution).

The design pressures recommended for "above the design groundwater level" are based on fully drained walls. One acceptable method for back-draining a basement wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to the design groundwater table. If a continuous cut-off wall shoring system, as presented in Section 7.6, is installed to provide excavation support of the subterranean parking levels, then installation of the drainage panel above the design groundwater table will not be required and the design pressures recommended for "above the design groundwater level" condition may still be used for design of the portion of the basement wall above the design groundwater level.

We recommend unrestrained retaining walls (i.e. site retaining walls) be designed for the more critical of the following criteria:

- Active equivalent fluid weight of 40 and 82 pcf for static conditions if walls are fully drained and not drained, respectively.

- Active equivalent fluid weight of 40 pcf plus a seismic increment of 11 pcf (triangular distribution) for seismic conditions if walls are fully drained; and active equivalent fluid weight of 82 pcf plus a seismic increment of 13 pcf (triangular distribution) for seismic conditions if walls are not drained.

Although the unrestrained site retaining walls will be above the design groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines, etc. If the "drained" earth pressures (i.e. pressures for above design groundwater table) presented above are used to design the walls, they will need to incorporate a drainage system. Alternatively, the walls may be designed for the recommended "undrained" earth pressures (i.e. pressures for below the groundwater table) presented above over their entire height, in which case the drainage system may be omitted.

Where traffic loads are expected within 10 feet of the basement walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall for static loading conditions. Where the mat foundation for the at-grade portion of the buildings will be adjacent to the basement wall for the portion of the building with one level below grade, the basement wall should be designed for the surcharge pressure imposed by the mat; the surcharge pressure can be calculated by multiplying the average mat pressure within a horizontal distance of 1.5 times height of wall by a factor of 0.5.

To protect against moisture migration, below-grade basement walls should be waterproofed and water stops should be placed at all construction joints. If backfill is required behind below-grade walls, the walls should be braced, or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the structural engineer).

7.5 Temporary Cut Slopes and Shoring

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 shoring engineer should be responsible for shoring design. The contractor should be responsible for the construction and safety of temporary slopes. We should review the geotechnical aspects of the proposed shoring system to ensure that it meets our requirements. During construction, we should observe the installation of the shoring system and check the condition of the soil encountered during excavation.

We judge that temporary slope cuts in clayey and sandy soils, corresponding to CAL-OSHA Types B and C soils, above the groundwater table inclined no steeper than 1:1 (horizontal:vertical) and 1.5:1, respectively, will be stable provided that they are not surcharged by equipment or building material. Temporary shoring will be required where temporary slopes are not possible because of space constraints.

As discussed in Section 6.5, we conclude the most appropriate shoring system for the proposed basement excavations consists of soldier pile-and-lagging shoring system with an active dewatering system, or a continuous cut-off wall system, which may consist of either a secant pile wall or a continuous SMX wall reinforced with steel soldier beams. The purpose of the continuous cut-off wall is to reduce the amount of groundwater seepage into the excavation and reduce dewatering costs. We anticipate the depth of excavation for the garages will be about 13 feet. We judge that a cantilevered shoring system is appropriate for support of excavations up to about 13 feet in depth, although it may be more economical to use tiebacks for shoring walls more than 10 feet high. Recommendations for a tied-back shoring system can be provided upon request.

7.5.1 Cantilevered Soldier Pile-and-Lagging Shoring System

We recommend a cantilevered soldier pile-and-lagging shoring system be designed to resist an active equivalent fluid weight of 40 pcf. In locations where minimizing lateral deflections is critical, such as near adjacent buildings or near sensitive underground utilities, the shoring system should be designed to resist an at-rest equivalent fluid weight of 60 pcf, plus any foundation surcharge loads. In calculating these design pressures, we assumed drained conditions with no hydrostatic pressure acting on the soldier pile-and-lagging shoring. Where traffic loads are expected within 10 feet of the shoring walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall. Where construction equipment will be working behind the walls within a horizontal distance equal to the wall height, the design should include a surcharge pressure of 250 psf. The above pressures should be assumed to act over the entire width of the lagging installed above the excavation.

Passive resistance at the toe of the soldier piles should be computed using an equivalent fluid weight of 240 pcf above the groundwater table (after dewatering) and 115 pcf below the groundwater table. Passive pressure can be assumed to act over an area of three pile widths assuming the toe of the soldier pile is filled with structural concrete. These passive pressure values include a factor of safety of at least 1.5.

Soldier piles should be placed in pre-drilled holes backfilled with concrete. Where caving soil is encountered during drilling, the shoring contractor should be prepared to drill the holes with casing or use of drilling slurry to reduce caving of the holes. Installing soldier piles by driving or using vibratory methods is also acceptable, but should not be permitted within 25 feet of existing structures.

7.5.2 Secant Pile or Continuous Soil-Cement Mix (SMX) Wall

Secant pile or continuous SMX walls are groundwater cut-off walls and should be designed for undrained conditions. We recommend the secant pile or SMX wall shoring systems be designed to resist the following pressures:

- <u>Above Design Groundwater Table</u>: An active equivalent fluid weight of 40 pcf. In locations where minimizing lateral deflections is critical, such as near adjacent buildings or near sensitive underground utilities, the shoring system should be designed to resist an at-rest equivalent fluid weight of 60 pcf, plus any foundation surcharge loads.
- <u>Below Design Groundwater Table</u>: An active equivalent fluid weight of 82 pcf. In locations where minimizing lateral deflections is critical, such as near adjacent buildings or near sensitive underground utilities, the shoring system should be designed to resist an at-rest equivalent fluid weight of 91 pcf, plus any foundation surcharge loads.
- Where traffic loads are expected within 10 feet of the shoring walls, an additional design load of 50 psf should be applied to the upper 10 feet of the wall.
- Where construction equipment will be working behind the walls within a horizontal distance equal to the wall height, the design should include a surcharge pressure of 250 psf.

The above pressures should be assumed to act over the entire width of the lagging installed above the excavation. Considering the cut-off wall will extend below the bottom of excavation, the active and at-rest pressures should be assumed to act over the entire width of the shoring below the bottom of the excavation.

Passive resistance at the toe of the soldier pile should be computed using an equivalent fluid weight of 240 pcf above the groundwater table (after dewatering) and 115 pcf below the groundwater table. Passive pressure can be assumed to act over an area of three pile widths, provided the concrete or soil-cement mix is sufficiently strong to accommodate the corresponding stresses (shoring designer should confirm). These passive pressure values include a factor of safety of at least 1.5.

7.5.3 Soil-Cement Mix (SMX) Shoring

The design strength and thickness of the SMX wall should be established by the shoring designer based on the recommended design pressures presented in the previous section and the design requirements of the structural system. A contractor experienced in installing SMX systems in similar soil and groundwater conditions should be responsible for selecting appropriate materials, equipment, and methods to provide a consistent SMX product that meets the design requirements set forth by the shoring designer.

Prior to the start of SMX production, the contractor should prepare a detailed work plan, including the following items:

- Detailed descriptions of sequence of construction and all construction procedures, equipment, and ancillary equipment to be used to penetrate the ground, proportion and mix binders, and inject and mix the site soils.
- Proposed mix design(s), including binder types, additives, fillers, reagents, and their relative proportions, and the required mixing time, water-to-binder ratio of the slurry (for wet mixing), binder factor (for dry mixing and wet mixing), and volume ratio (for wet mixing) for a deep mixed element.
- Proposed injection and mixing parameters, including mixing slurry rates, slurry pumping rates, air injection pressure, volume flow rates, mixing tool rotational speeds, and penetration and withdrawal rates.

- Methods for controlling and recording the verticality and the top and bottom elevation of each SMX element.
- Drawings indicating the identification number of every SMX element, as well as a schedule of all the SMX elements and their tip elevations, mix design (if variable), element type (primary or secondary), binder factors, volume ratios, etc.
- Details of all means and methods proposed for QC/QA activities, including surveying, process monitoring, sampling, testing, and documenting.

The work plan should be submitted to the shoring designer and the Geotechnical Engineer for review prior to the start of construction, and the approved document should be provided to the contractors' field personnel and our field engineer.

7.5.4 Construction Monitoring

During excavation, the shoring system is expected to yield and deform laterally, which could cause the ground surface adjacent to the shoring wall to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Ground movements due to a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch where there are no buildings with a horizontal distance equal to 1.5 times the excavation depth of 1/2 inch where there are buildings within that horizontal distance. A monitoring program should be established to evaluate the effects of the construction on the adjacent properties.

The conditions of existing buildings within a horizontal distance equal to 1.5 times the proposed excavation depth should be photographed and surveyed prior to the start of construction and monitored periodically during construction. In addition, prior to the start of excavation, the contractor should establish survey points on the shoring system, on the ground surface at critical locations behind the shoring, and on adjacent buildings. These survey points should be used to monitor the vertical and horizontal movements of the shoring and the ground behind the shoring throughout construction.

The survey points should be monitored regularly, and the results should be submitted to us in a timely manner for review. For estimating purposes, assume that the instrumentation will be read as follows:

- Prior to any excavation or shoring work at the site
- After installing temporary shoring
- After the excavation reaches its lowest elevation
- Every two weeks until the street-level floor slab is constructed

7.6 Pavement Design

Design recommendations for asphalt concrete and Portland-cement concrete (PCC) pavements and concrete pavers are presented in this section.

7.6.1 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. For pavement design, we assumed the upper foot of pavement subgrade will consist of non-expansive engineered fill (select fill) that has an R-value of at least 15. Recommended pavement sections for traffic indices (Tis) ranging from 4.5 to 7.5 are presented in Table 3. The project civil engineer should determine the appropriate design TI based on the anticipated vehicular traffic the pavement will experience. We can provide additional pavement sections for different TIs upon request.

TABLE 3
Asphalt Concrete Pavement Sections
(Subgrade R-Value of 15)

TI	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.5	2.5	8.0
5.0	3.0	8.0
5.5	3.0	10.0
6.0	3.5	10.5
6.5	4.0	11.5
7.0	4.0	13.0
7.5	4.5	14.0

The upper six inches of the subgrade should be moisture-conditioned and compacted in accordance with requirements presented in Table 2 in Section 7.1.1 and the subgrade should be non-yielding. The aggregate base should be moisture-conditioned to near optimum and compacted to at least 95 percent relative compaction and be non-yielding.

Where pavements are adjacent to irrigated landscaped areas, curbs adjacent to those areas should extend through the aggregate base and at least three inches into the underlying soil to reduce the potential for irrigation water to infiltrate into the pavement section. Where pavements are adjacent to storm water treatment facilities, such as bio-swales, flow-through planters, or bio-retention basins, or any other landscaped areas in which a significant thickness of loose, uncompacted soil will be present the curbs may need to extend deeper, as presented in Section 7.2.3.

7.6.2 Rigid (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 (i.e., several garbage trucks per week). The recommended rigid pavement section for these axle loads is 6.5 inches of PCC over six inches of Class 2 aggregate base. For areas that will receive fire truck traffic, the pavement section should consist of seven inches of PCC over six inches of Class 2 aggregate base. For areas that will experience only passenger vehicle traffic, the recommended pavement section is five inches of PCC over six inches of Class 2 aggregate base.

The modulus of rupture and unconfined compressive strength of the concrete should be at least 500 and 3,200 psi at 28 days, respectively. Contraction joints should be placed at a maximum 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. For areas that will garbage/recycling truck traffic, we recommend the concrete slab be reinforced with a minimum of No. 4 bars at 16 inches on center in both directions.

Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those we have described above for asphalt concrete pavement. Recommendations for pavements adjacent to irrigated landscaped areas, bio-swales, or other storm water treatment areas are also the same as those presented above for asphalt concrete pavement.

7.7 Pavers

Recommendations for non-permeable and permeable pavers are presented in this section. In preparing our recommendations for pavers, we assumed the upper two feet of paver subgrade will consist of non-expansive engineered fill (select fill) as presented in Section 7.1.

7.7.1 Non-Permeable Concrete Pavers

We recommend non-permeable pedestrian pavers and sand bedding be underlain by at least six inches of Class 2 aggregate base compacted to at least 90 percent relative compaction. Where non-permeable concrete pavers will be subject to vehicular traffic, we recommend they consist of fully dentated, interlocking shapes and be at least 80 millimeters (3.15 inches) thick. We recommend non-permeable pavers subject to vehicular traffic be underlain by Class 2 aggregate base compacted to at least 95 percent relative compaction. The aggregate base thickness beneath non-permeable pavers subject to vehicular traffic should be consistent with that recommended in Table 3 for asphalt pavement for the appropriate TI.

7.7.2 Permeable Concrete Pavers

We recommend permeable interlocking concrete pavements (ICP) be designed in accordance with the guidelines presented by the Interlocking Concrete Pavement Institute (ICPI 2005). These guidelines include specific recommendations for permeable aggregate subbase, base, and bedding courses to be placed beneath ICP pavements. We recommend permeable pavements for both vehicular and pedestrian traffic be designed for **partial exfiltration** of water into the subgrade soil. This requires installing a subdrain system at the base of the pervious aggregate materials, which are underlain by a filter fabric. ICPI's generalized paver section for partial exfiltration is presented on Figure 7.

The soil subgrade beneath ICP pavements should be prepared and compacted in accordance with the recommendations presented in Section 7.1. In addition, the subgrade should be a firm and non-yielding surface. The subgrade should be proof-rolled under the observation of our field engineer to confirm it is non-yielding prior to placing the impermeable membrane and aggregate base materials. The soil subgrade at the bottom of the permeable section should slope down toward the drainpipe trench at a gradient of at least two percent. The perforated pipe should slope down to a suitable outlet at a minimum gradient of one percent. The pipe should be placed with the perforations down on a minimum of two inches of permeable subbase.

ICPI's guidelines call for 1-1/2 to 2 inches of bedding material consisting of ASTM No. 8 crushed aggregate directly below the pavers. This material is also recommended for fill material between the pavers. As shown in Table 4 below, this material consists of fine gravel with 10 to 30 percent sand.

TABLE 4
Gradation Requirements for ASTM No. 8 Crushed Aggregate

Sieve Size	Percentage Passing Sieve
1/2 inch	100
3/8 inch	85 – 100
No. 4	10 – 30
No. 8	0 – 10
No. 16	0 – 5

The ASTM No. 8 bedding should be underlain by a permeable base course of ASTM No. 57 crushed aggregate. As shown in Table 5, ASTM No. 57 aggregate consists of crushed, open-graded gravel with a gradation between that of the 3/4-inch drain rock and the ASTM No. 8 aggregate.

TABLE 5
Gradation Requirements for ASTM No. 57 Crushed Aggregate

Sieve Size	Percentage Passing Sieve
1-1/2 inch	100
1 inch	95 – 100
1/2 inch	25 – 60
No. 4	0 – 10
No. 8	0 – 5

The ASTM No. 57 permeable base course should be underlain by a permeable subbase course of ASTM No. 2 crushed aggregate. The gradation requirements for ASTM No. 2 crushed aggregate subbase are presented in Table 6.

TABLE 6
Gradation Requirements for ASTM No. 2 Crushed Aggregate

Sieve Size	Percentage Passing Sieve
3 inch	100
2-1/2 inch	90-100
2 inch	35-70
1-1/2 inch	0-15
3/4 inch	0 -5

The No. 2 crushed aggregate subbase course should be placed in lifts not exceeding 6 inches in loose thickness and compacted using a smooth-drum roller that weighs a minimum of 10 tons, operated in static (non-vibratory) mode. The subsequent course of No. 57 crushed aggregate may be placed in one lift and should be compacted with a smooth-drum roller in vibratory mode with sufficient passes to create an unyielding surface. Placement and compaction of the permeable aggregate base and subbase should be performed under the observation of our field engineer. Following compaction of the No. 57 aggregate, the No. 8 bedding, not exceeding 2 inches in loose thickness, should be placed and screeded to a level, undisturbed surface immediately prior to paver installation.

The required thicknesses of the permeable aggregate base and subbase courses depends on the infiltration and water storage design requirements, as well as the traffic loading demand. Our recommendations for the minimum permeable ICP pavement sections subject to vehicular traffic (including fire and garbage trucks) are presented in Table 7. Also included in Table 7 is a recommended section for permeable ICPs subject to pedestrian traffic only.

TABLE 7
Recommended Pavement Sections for Permeable Interlocking Concrete Pavers

Pavement Type	ASTM No. 8 Crushed Bedding Aggregate (inches)	ASTM No. 57 Crushed Base (inches)	ASTM No. 2 Crushed Subbase (inches)
Pedestrian	1.5-2.0	4.0	6.0
Vehicular	1.5-2.0	4.0	10.0

The above recommended ICP pavement sections are based on the ICPI technical guidelines (ICPI 2005). From a geotechnical standpoint, it is acceptable to design the pedestrian ICP section to exclude the No. 2 subbase course, in which case the No. 57 base course should be increased to 10 inches. If this approach is used, the perforated pipe should include a filter fabric sleeve to prevent the finer aggregate from entering the perforations.

7.8 Seismic Design

The results of our CPT-6 and CPT-14 indicate the shear wave velocities for the upper 100 feet of soil (V_{s30}) at the site are about 780 and 750 feet per second, respectively. As discussed in Section 5.2.2, thin layers of potentially liquefiable soil were encountered beneath the site. The 2019 CBC calls for a Site Class F designation for sites underlain by potentially liquefiable soil; however, we judge that these layers are relatively thin and discontinuous and conclude that the soil at the site will not incur significant non-linear behavior. Therefore, we conclude a Site Class D designation is more appropriate.

The latitude and longitude of the site are 37.4837° and -122.1771°, respectively. Hence, in accordance with the 2019 CBC, we recommend the following:

- Site Class D (stiff soil)
- S_S = 1.5g, S_1 = 0.6g

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} = 1.5g, S_{M1} = 1.02g
- S_{DS} = 1.0g, S_{D1} = 0.68g
- Seismic Design Category D for Risk Factors I, II, and III

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

Prior to construction, Rockridge Geotechnical, Inc. should review the project plans and specifications to confirm that they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during site preparation, grading and excavation, shoring installation and testing, fill placement and compaction, and foundation installation. These observations will allow us to compare actual with anticipated soil conditions and to verify that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

9.0 LIMITATIONS

This 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 recommendations made in this report assume that the soil and groundwater conditions do not deviate appreciably from those disclosed in the exploratory borings and CPTs. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The 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: Google Maps, 2017

0 1,000 2,000 Feet

Approximate scale

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 01/06/21	Project No. 20-1950	Figure 1

EXPLANATION

CPT-1 Approximate location of cone penetration test by Rockridge Geotechnical Inc., December 17, 18 & 21, 2020

B-1 Approximate location of boring by Rockridge Geotechnical Inc., December 22 & 23, 2020

Project limits

Reference: Base map from a drawing titled "Townhomes Option 5", by Studio T Square Architecture, dated February 16, 2021.

123 INDEPENDENCE DRIVE
Menlo Park, California

SITE PLAN

Date 04/27/21	Project No. 20-1950	Figure 2

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

EXPLANATION

af	Artificial Fill
Qhym	Mud deposits (late Holocene)
Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)

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

N

0 2000 4000 Feet

Approximate scale

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 12/31/20	Project No. 20-1950	Figure 3

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

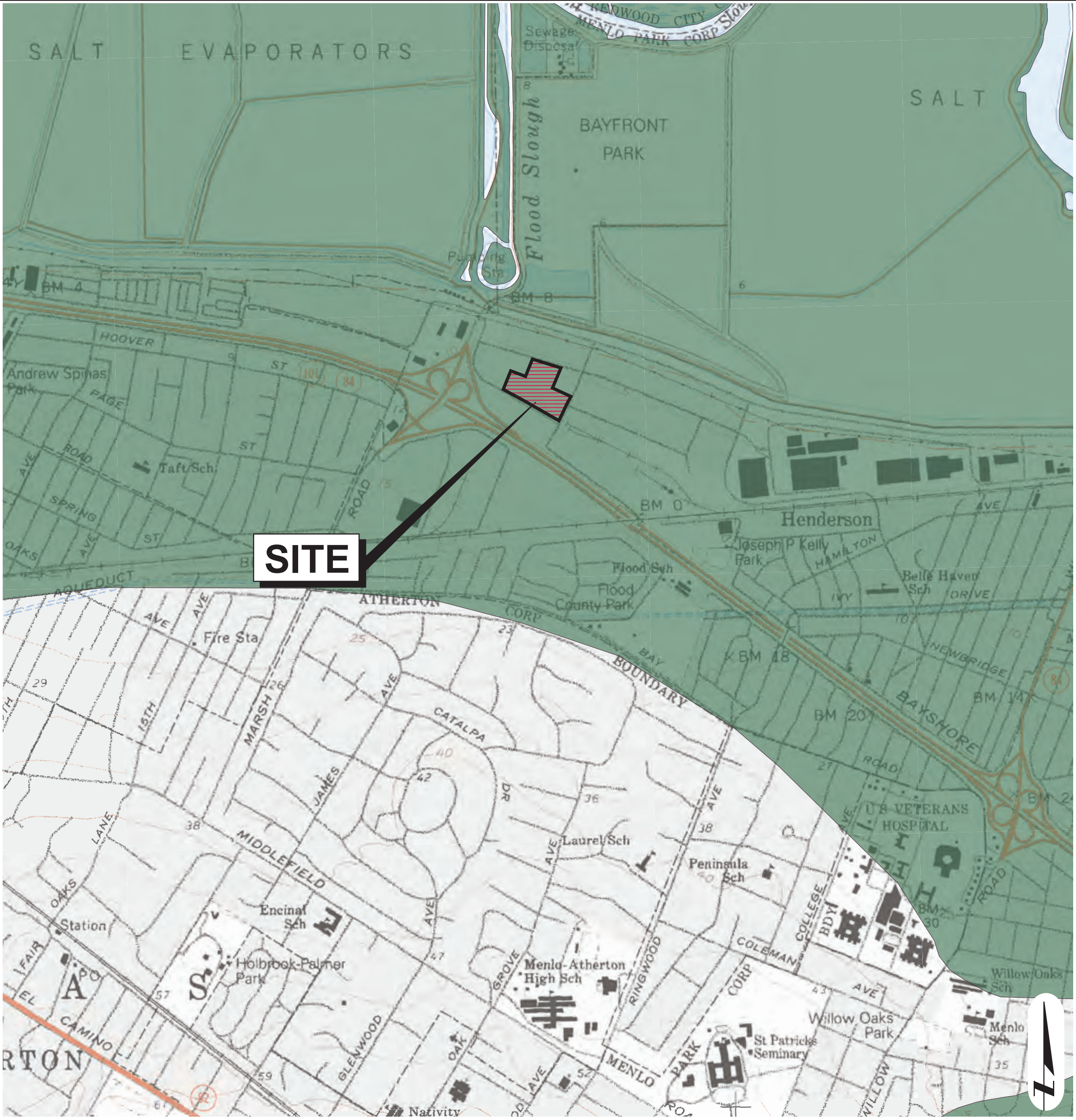
EXPLANATION

- Strike slip
- Thrust (Reverse)
- Normal

0 5 10 Miles

Approximate scale

123 INDEPENDENCE DRIVE Menlo Park, California	**REGIONAL FAULT MAP**
ROCKRIDGE GEOTECHNICAL	Date 12/31/20 · Project No. 20-1950 · Figure 4



Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Reference:
Earthquake Zones of Required Investigation
Palo Alto Quadrangle
California Geological Survey
Released October 18, 2006

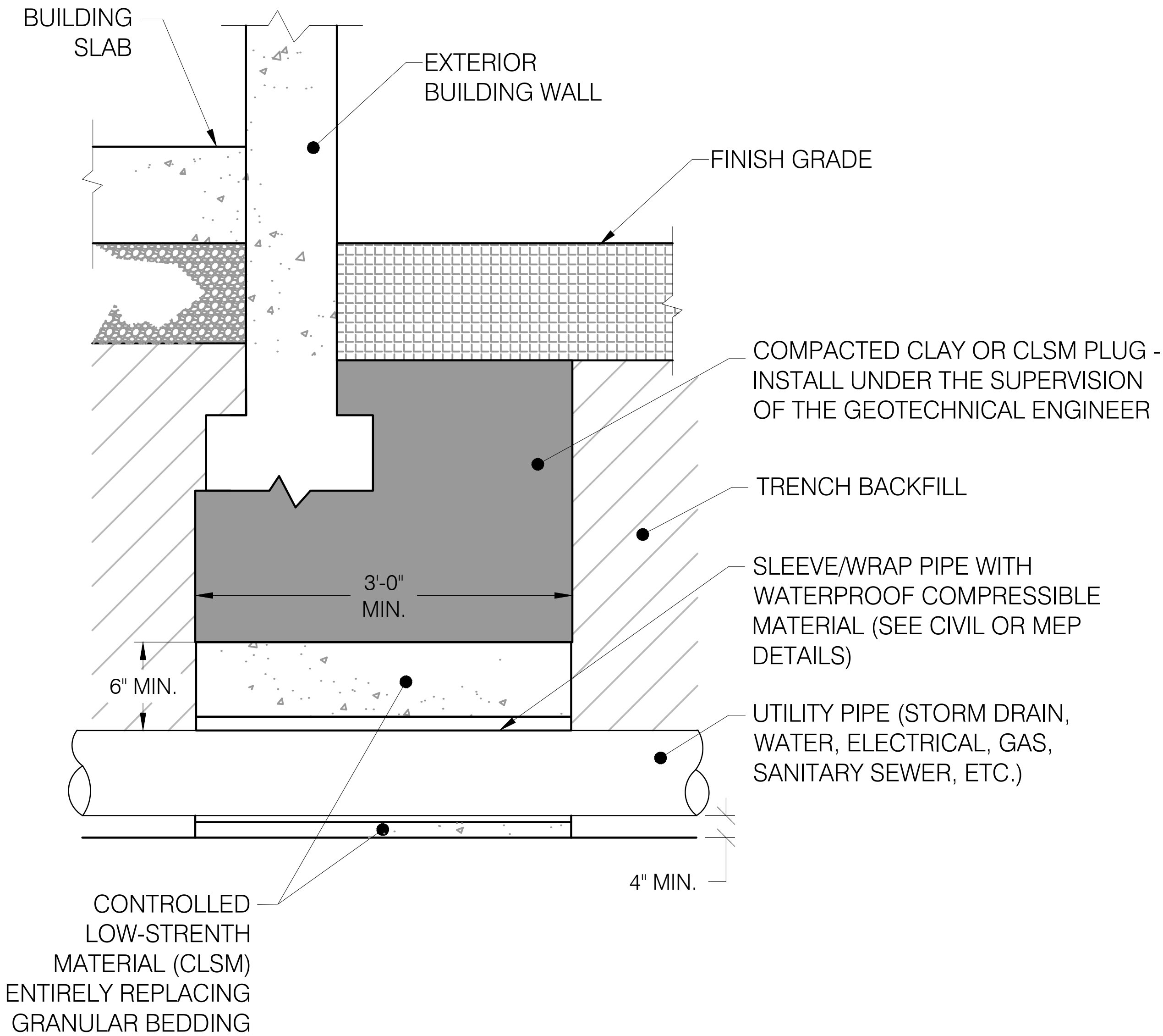
0 2,000 4,000 Feet
Approximate scale

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

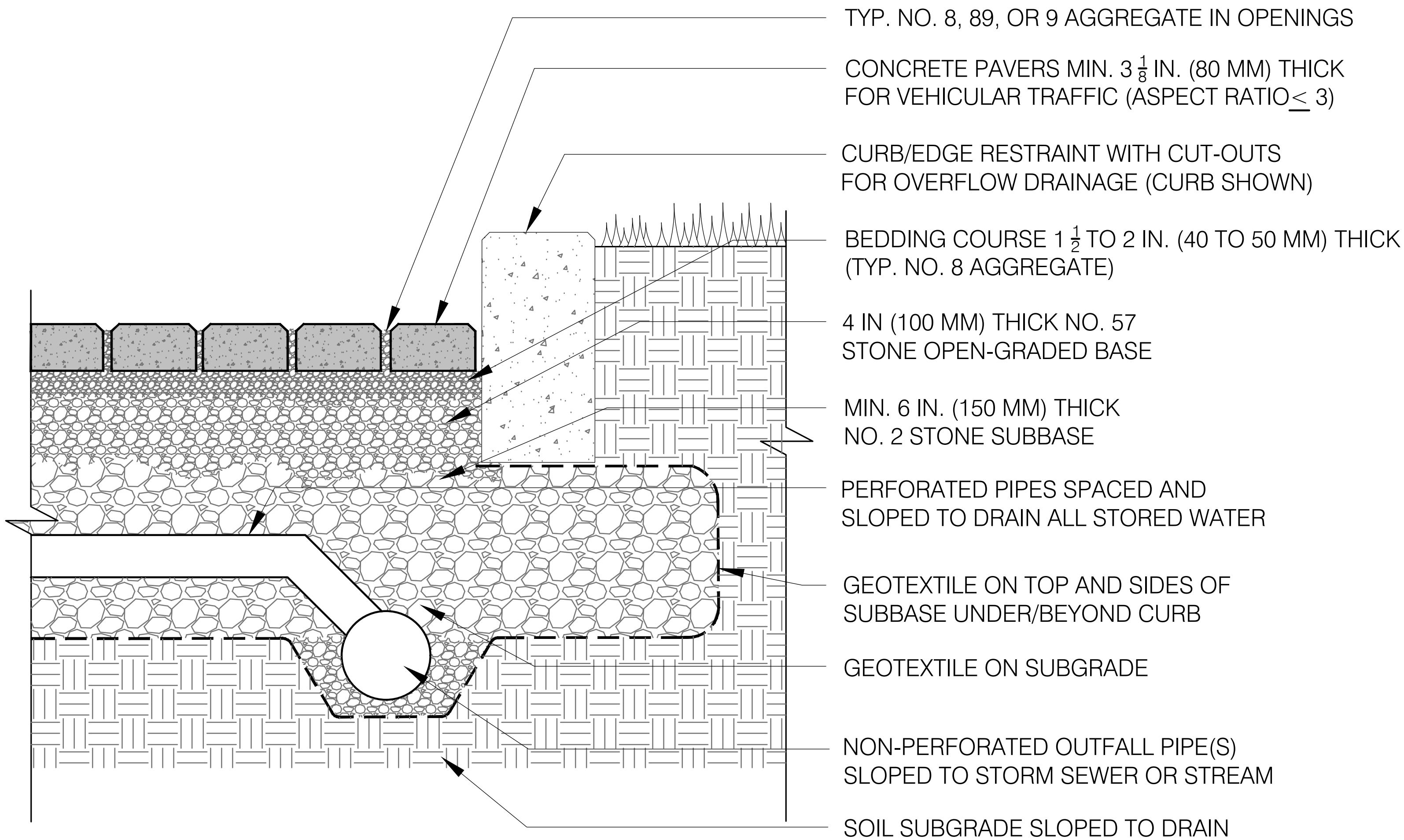
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 12/31/20	Project No. 20-1950	Figure 5



Not to Scale

123 INDEPENDENCE DRIVE Menlo Park, California	**UTILITY TRENCH LOW-PERMEABILITY PLUG AT BUILDING PERIMETER**
ROCKRIDGE GEOTECHNICAL	Date 01/26/21 | Project No. 20-1950 | Figure 6



NOTES:

1. $2\frac{3}{8}$ IN. (60 MM) THICK PAVERS MAY BE USED IN RESIDENTIAL APPLICATIONS.
2. NO. 2 STONE SUBBASE THICKNESS VARIES WITH DESIGN. CONSULT ICPI PERMEABLE INTERLOCKING CONCRETE PAVEMENT MANUAL..
3. PERFORATED PIPES MAY BE RAISED FOR WATER STORAGE FROM LARGE RAIN EVENTS WITH OUTLET(S) AT LINER BOTTOM TO DRAIN SMALL RAIN EVENTS.

Reference: "Permeable Interlocking Concrete Pavements", Third Edition, prepared by Interlocking Concrete Pavement Institute, dated 2005.

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

GENERALIZED ICPI PERMEABLE PAVER DETAIL FOR PARTIAL EXFILTRATION

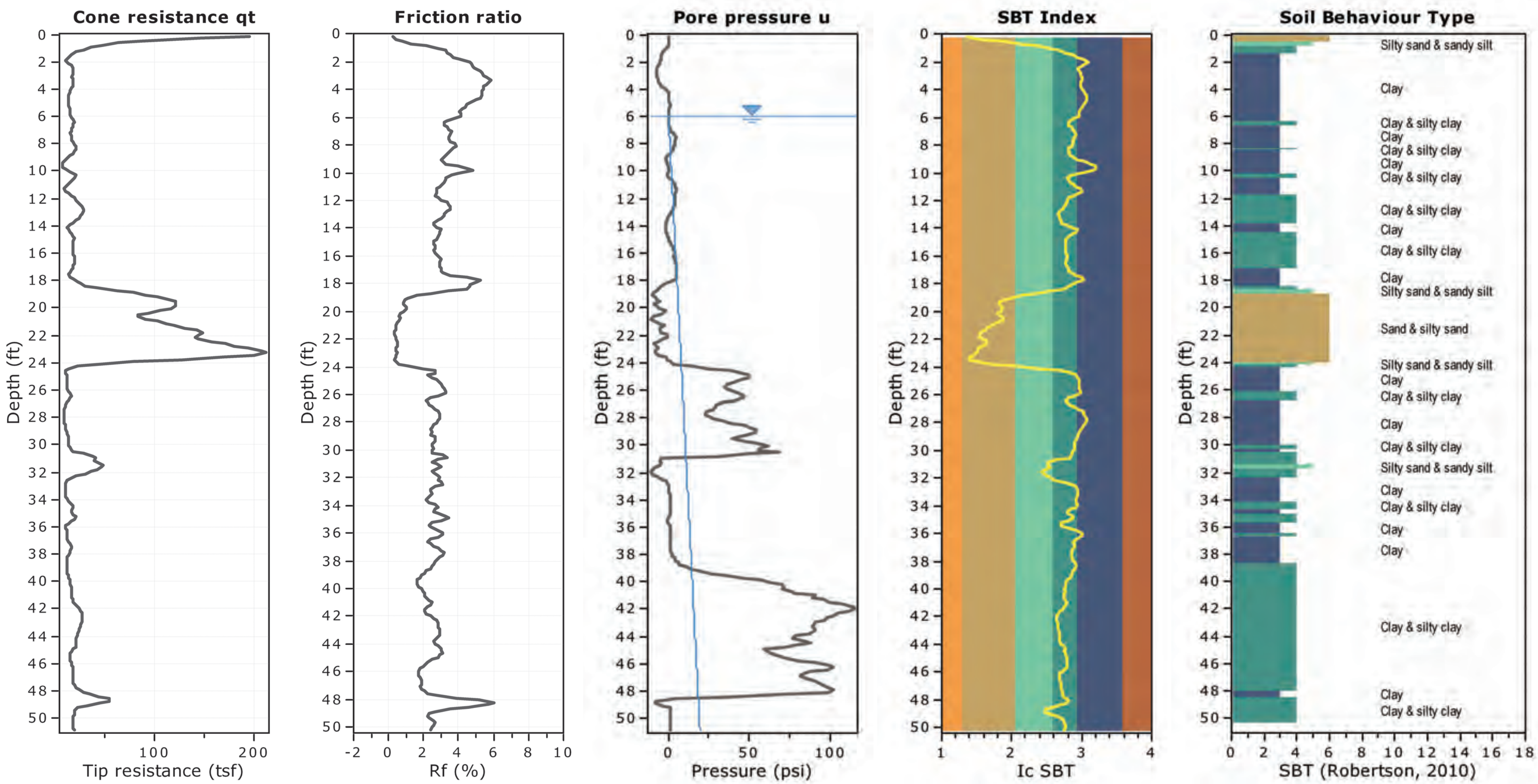
Date	Project No. 20-1950	Figure 7

ROCKRIDGE GEOTECHNICAL

APPENDIX A

Cone Penetration Test Results

Logs of Borings



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

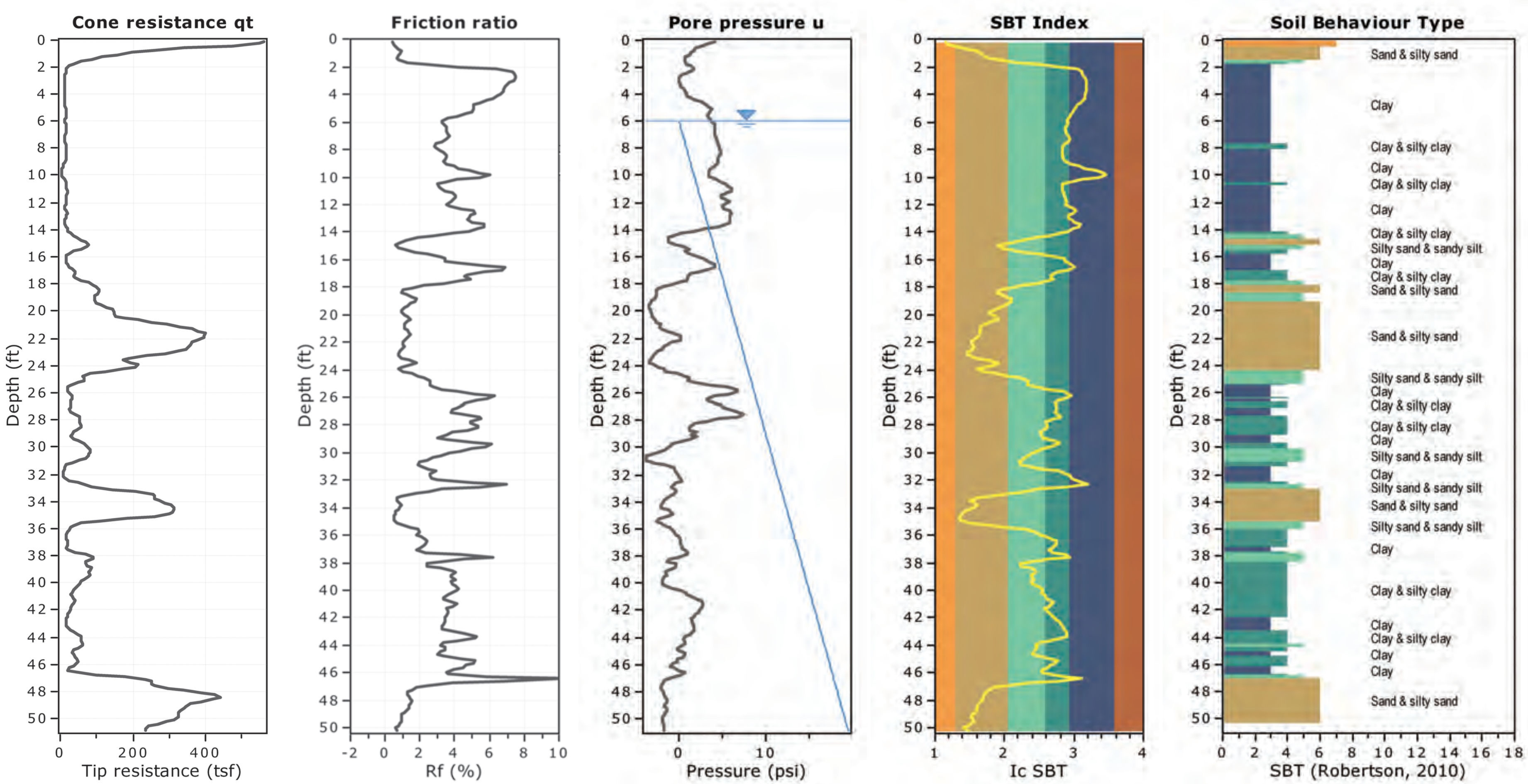
Approximate Ground Surface Elevation: 8.9 feet (NAVD 88)
Total depth: 50.9 ft, Date: December 21, 2020
Depth to Groundwater: 6 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 02/02/21 | Project No. 20-1950 | Figure A-1

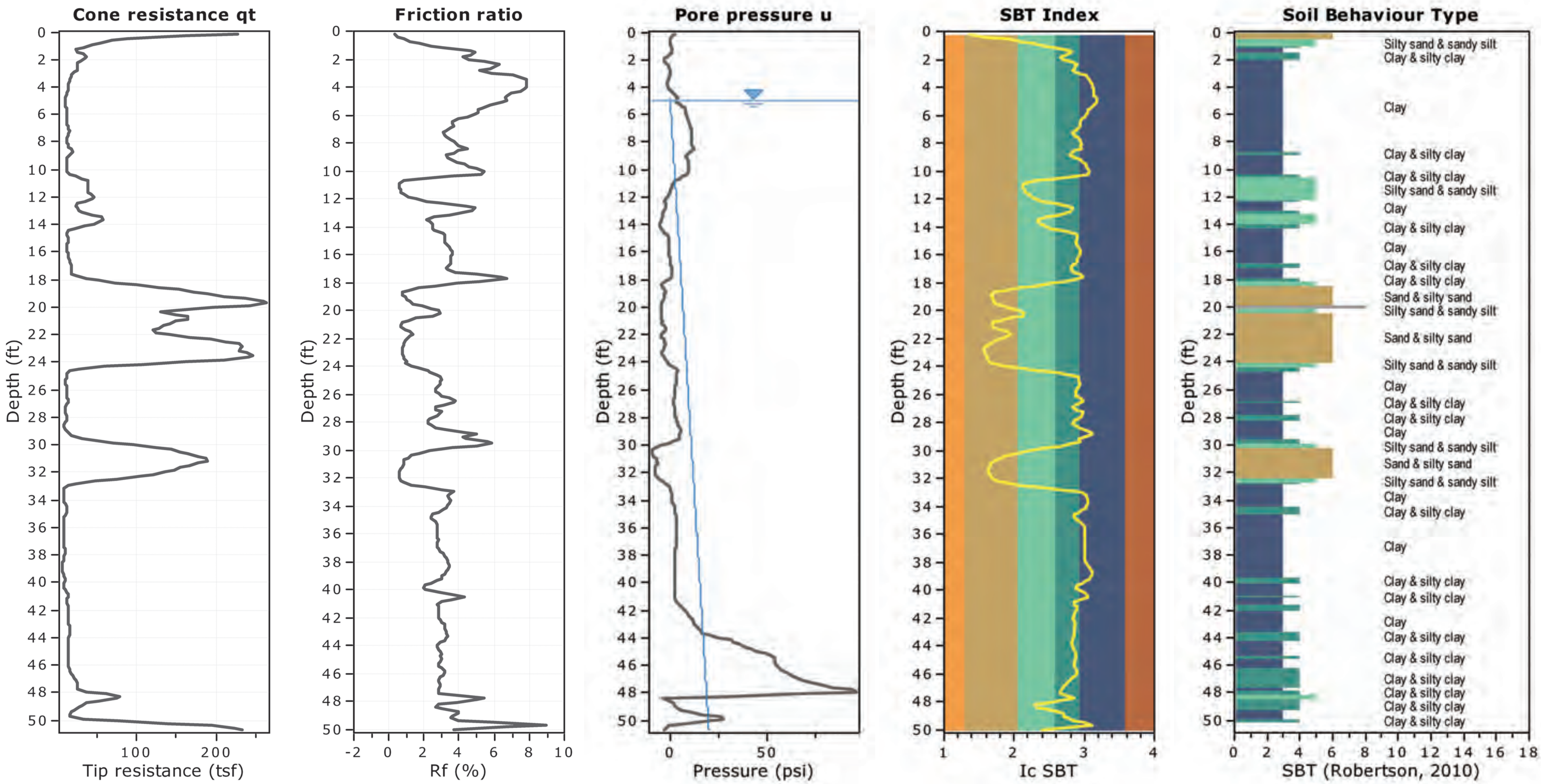


Approximate Ground Surface Elevation: 8.3 feet (NAVD 88)
Total depth: 50.9 ft, Date: December 21, 2020
Depth to Groundwater: 6 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

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

123 INDEPENDENCE DRIVE Menlo Park, California	CONE PENETRATION TEST RESULTS CPT-2
ROCKRIDGE GEOTECHNICAL	Date 02/02/21 · Project No. 20-1950 · Figure A-2



Approximate Ground Surface Elevation: 9.2 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 21, 2020
Depth to Groundwater: 5 feet (measured with weighted tape following rod removal)
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

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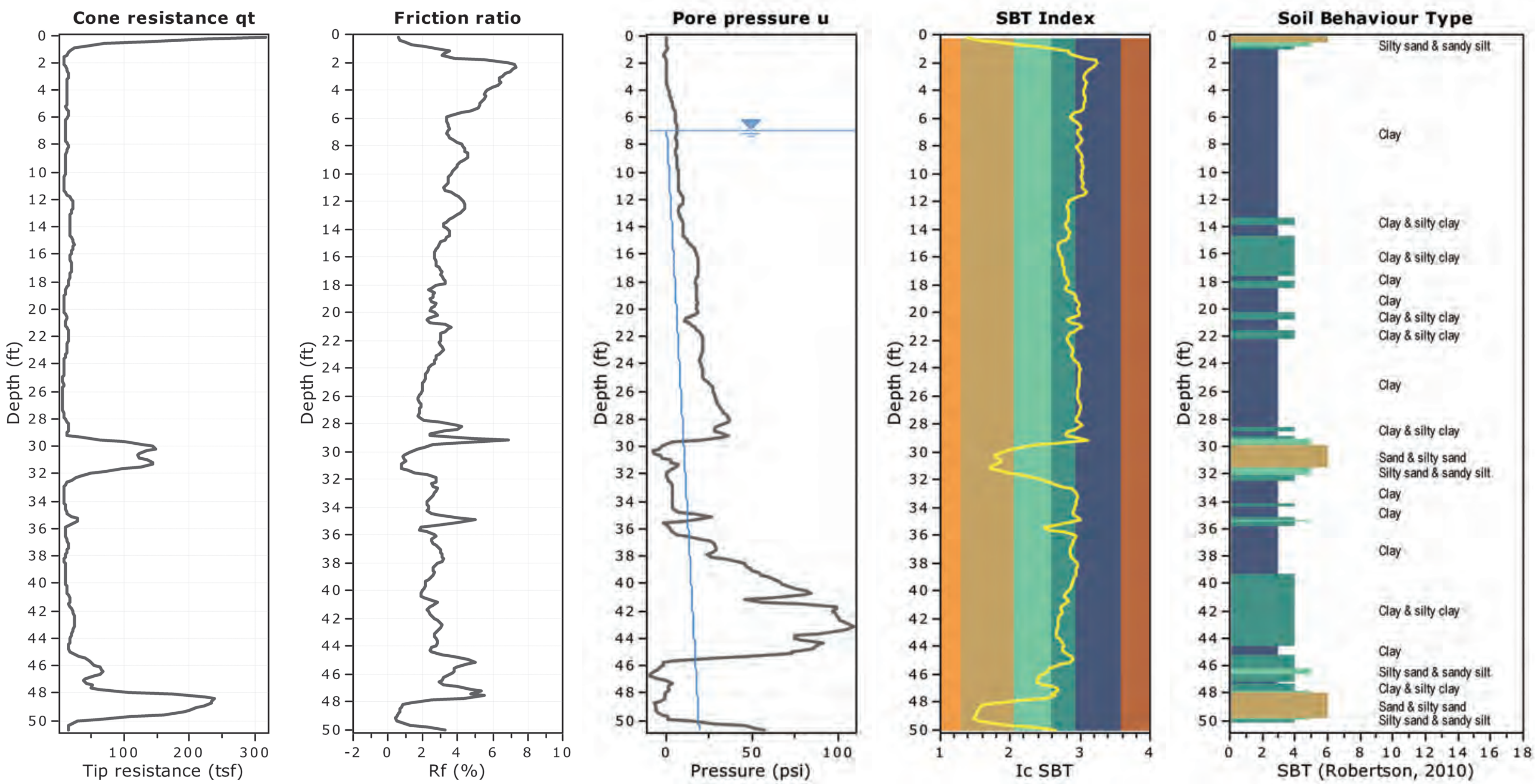
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 02/02/21	Project No. 20-1950	Figure A-3



Approximate Ground Surface Elevation: 8.8 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 21, 2020
Depth to Groundwater: 7 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

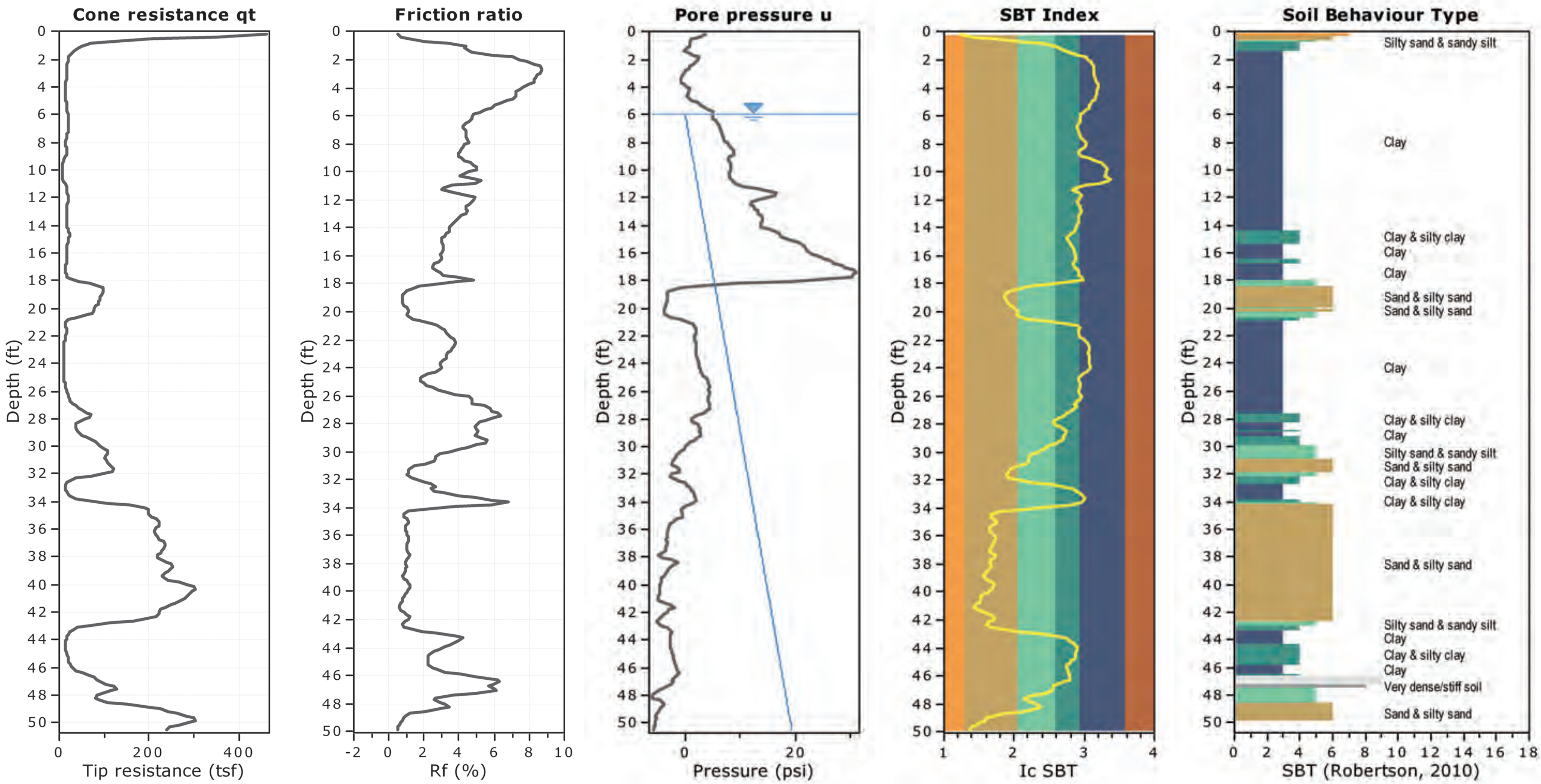
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-4

Date 02/02/21	Project No. 20-1950	Figure A-4

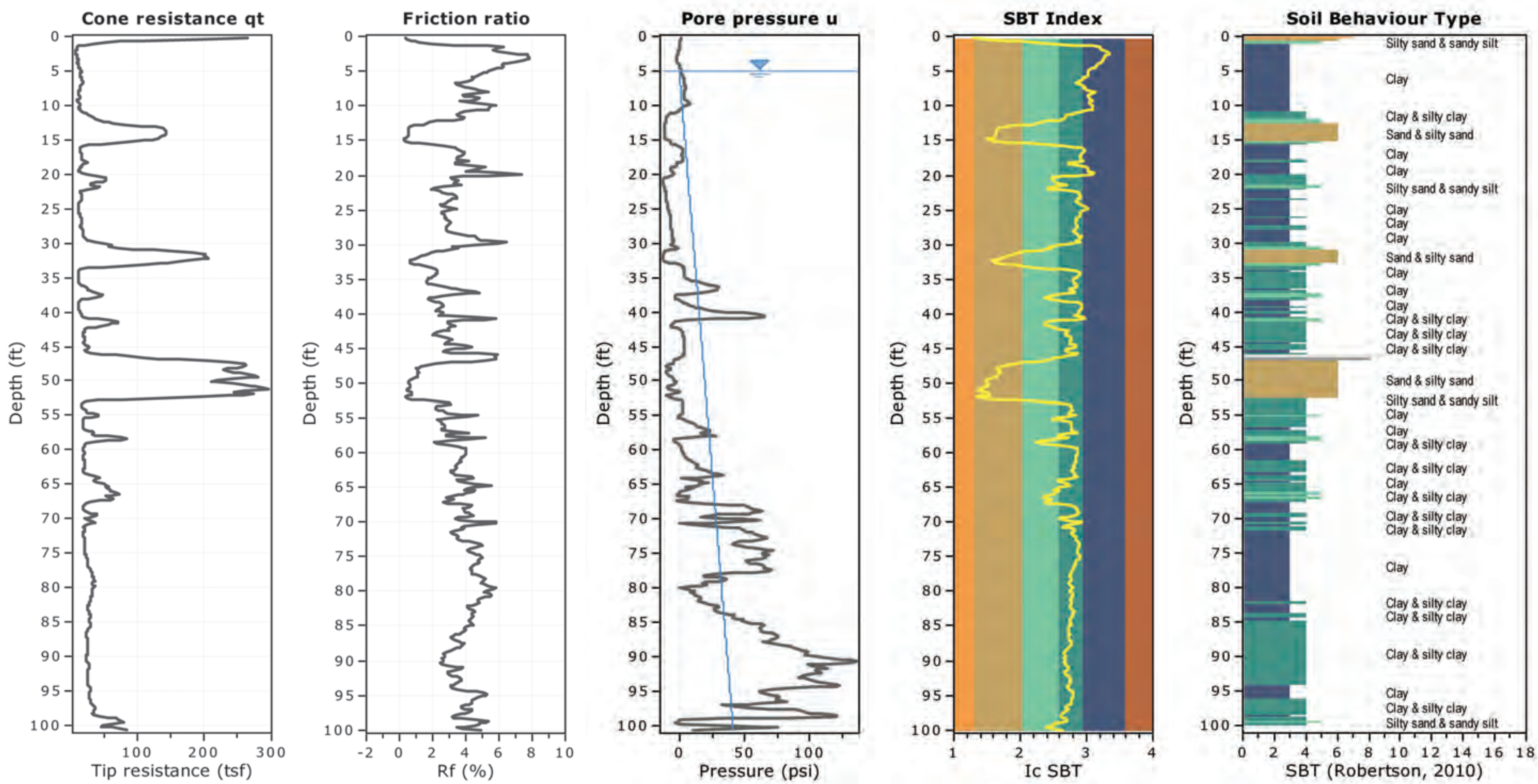


Approximate Ground Surface Elevation: 8.9 feet (NAVD 88)
Total depth: 50.5 ft, Date: December 21, 2020
Depth to Groundwater: 6 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

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

123 INDEPENDENCE DRIVE Menlo Park, California	CONE PENETRATION TEST RESULTS CPT-5
ROCKRIDGE GEOTECHNICAL	Date 02/02/21 | Project No. 20-1950 | Figure A-5



Approximate Ground Surface Elevation: 8.5 feet (NAVD 88)
Total depth: 100.7 ft, Date: December 21, 2020
Depth to Groundwater: 5 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

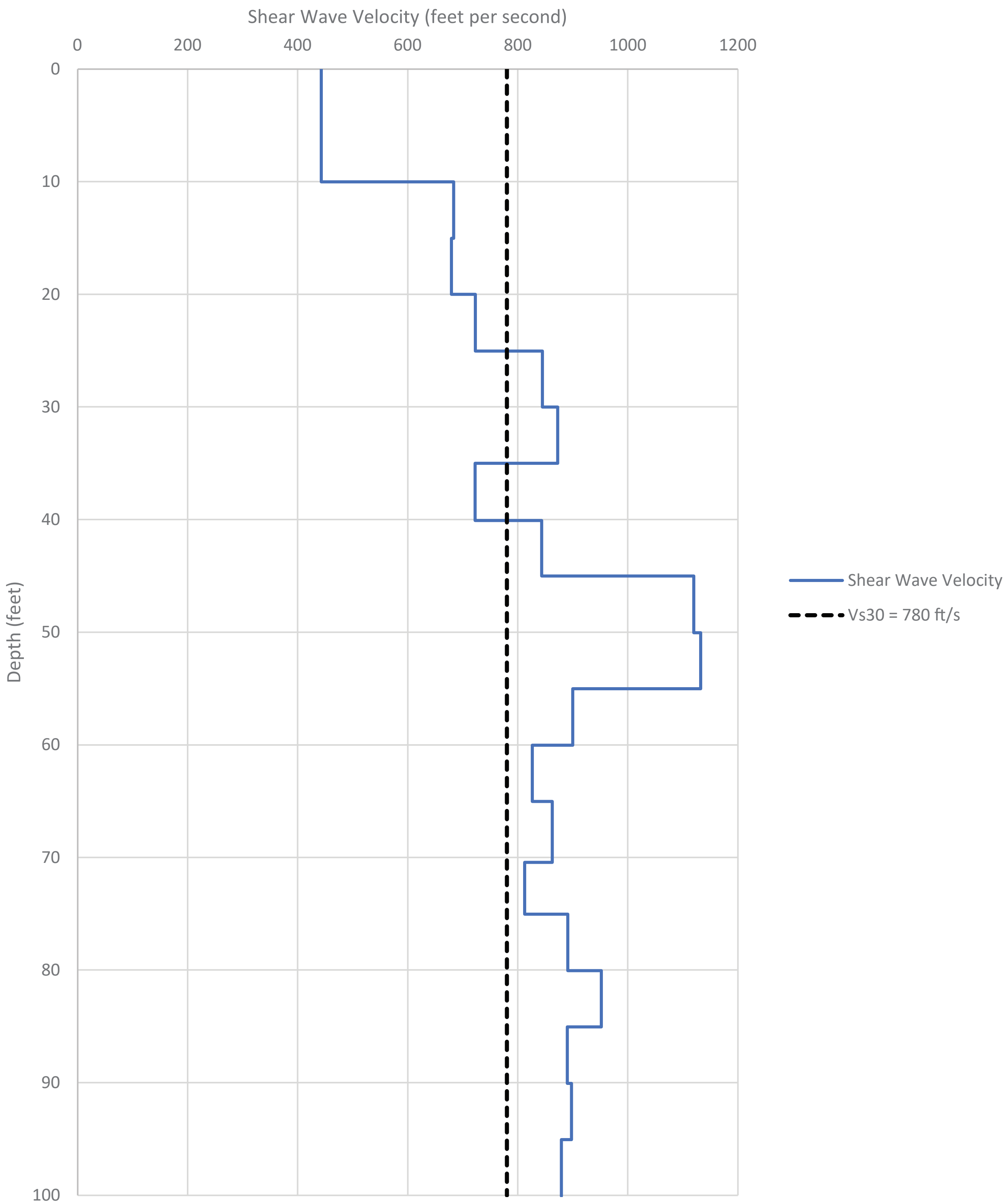
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

123 INDEPENDENCE DRIVE
Menlo Park, California

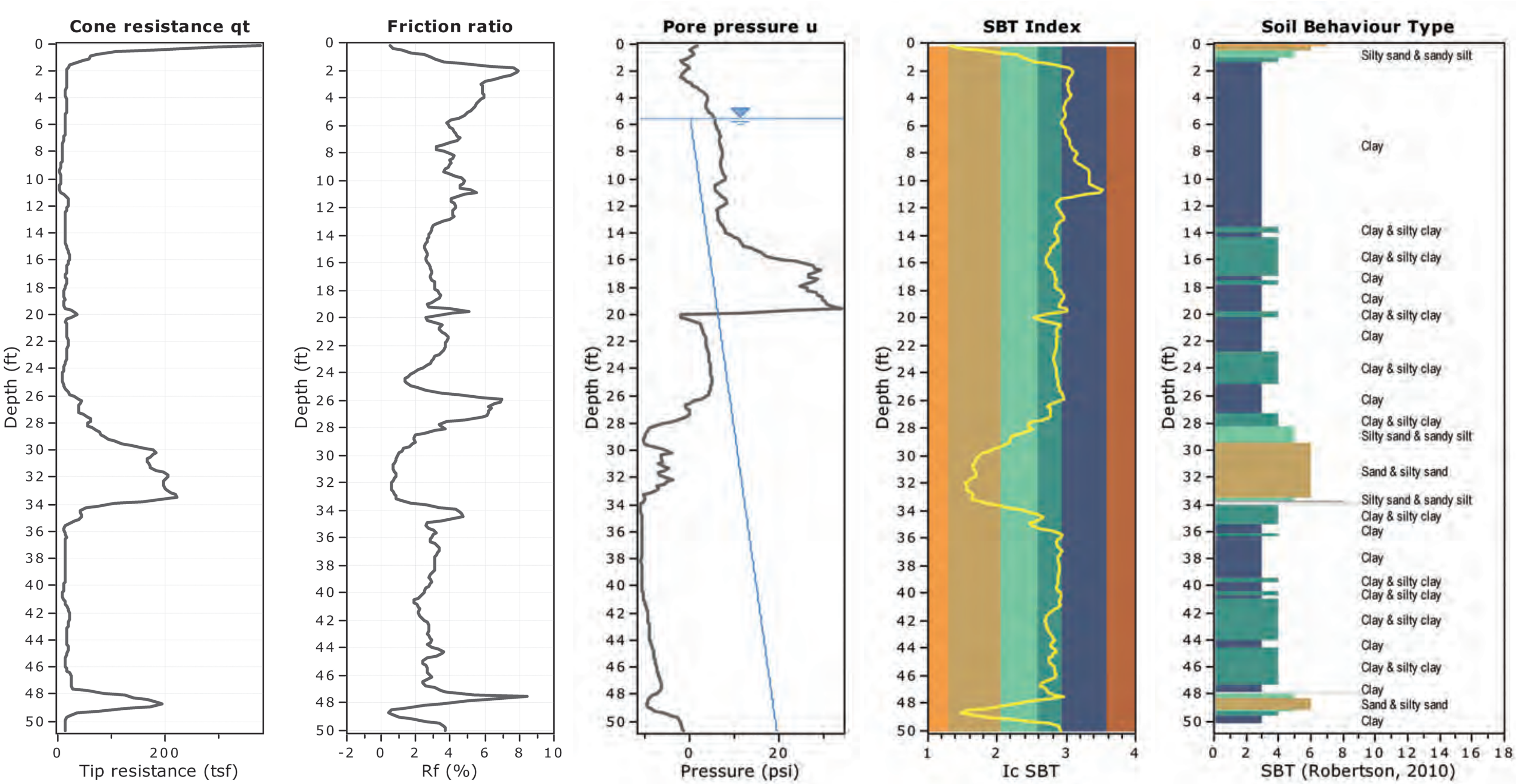
ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-6

Date 02/02/21	Project No. 20-1950	Figure A-6



123 INDEPENDENCE DRIVE Menlo Park, California	SHEAR WAVE VELOCITY PROFILE CPT-6		
ROCKRIDGE GEOTECHNICAL	Date 01/12/21	Project No. 20-1950	Figure A-6b

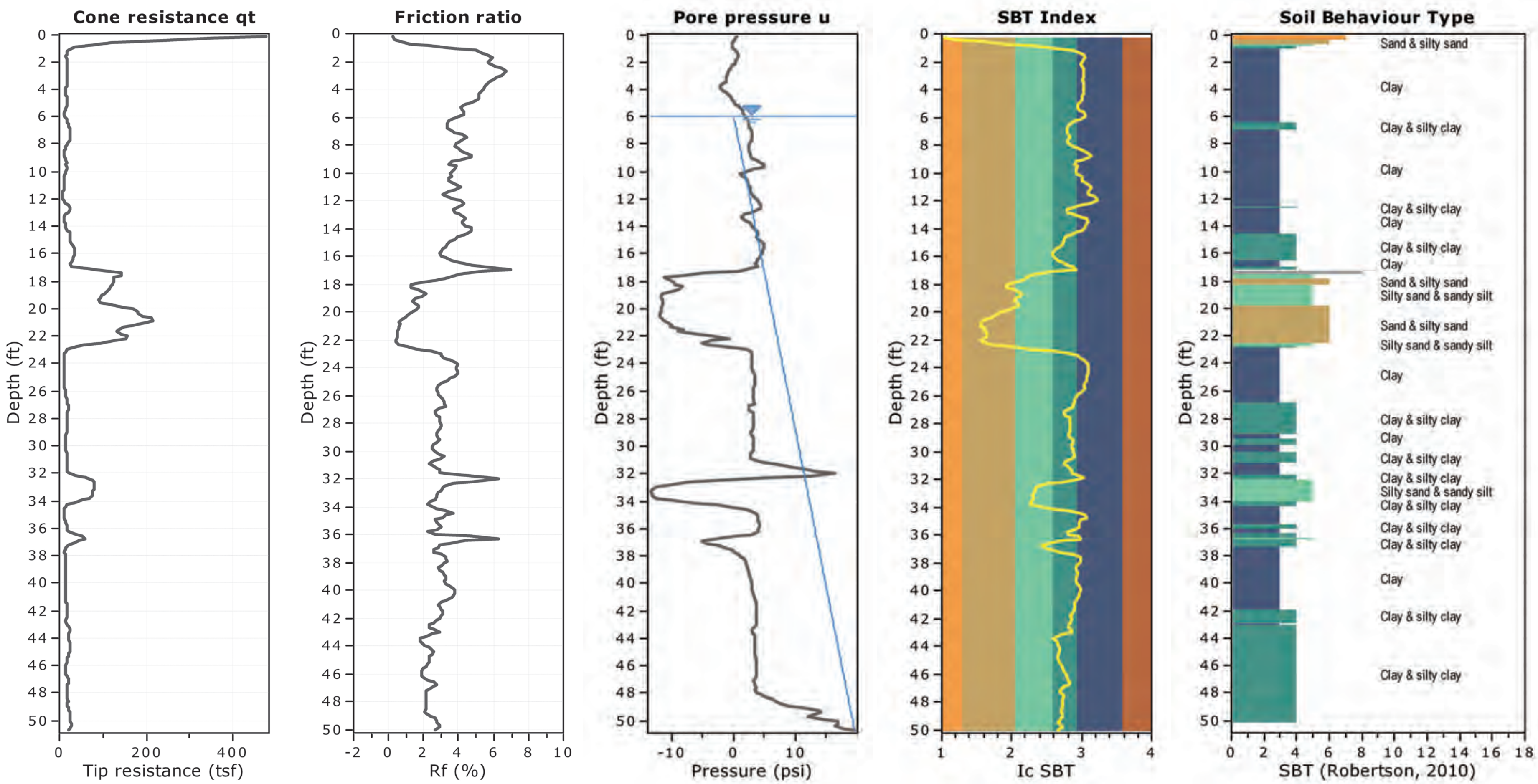


Approximate Ground Surface Elevation: 8.4 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 21, 2020
Depth to Groundwater: 5.5 feet (estimated from pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

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

123 INDEPENDENCE DRIVE Menlo Park, California	**CONE PENETRATION TEST RESULTS** **CPT-7**
ROCKRIDGE GEOTECHNICAL	Date 02/02/21 | Project No. 20-1950 | Figure A-7



Approximate Ground Surface Elevation: 9.0 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 18, 2020
Depth to Groundwater: 6 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

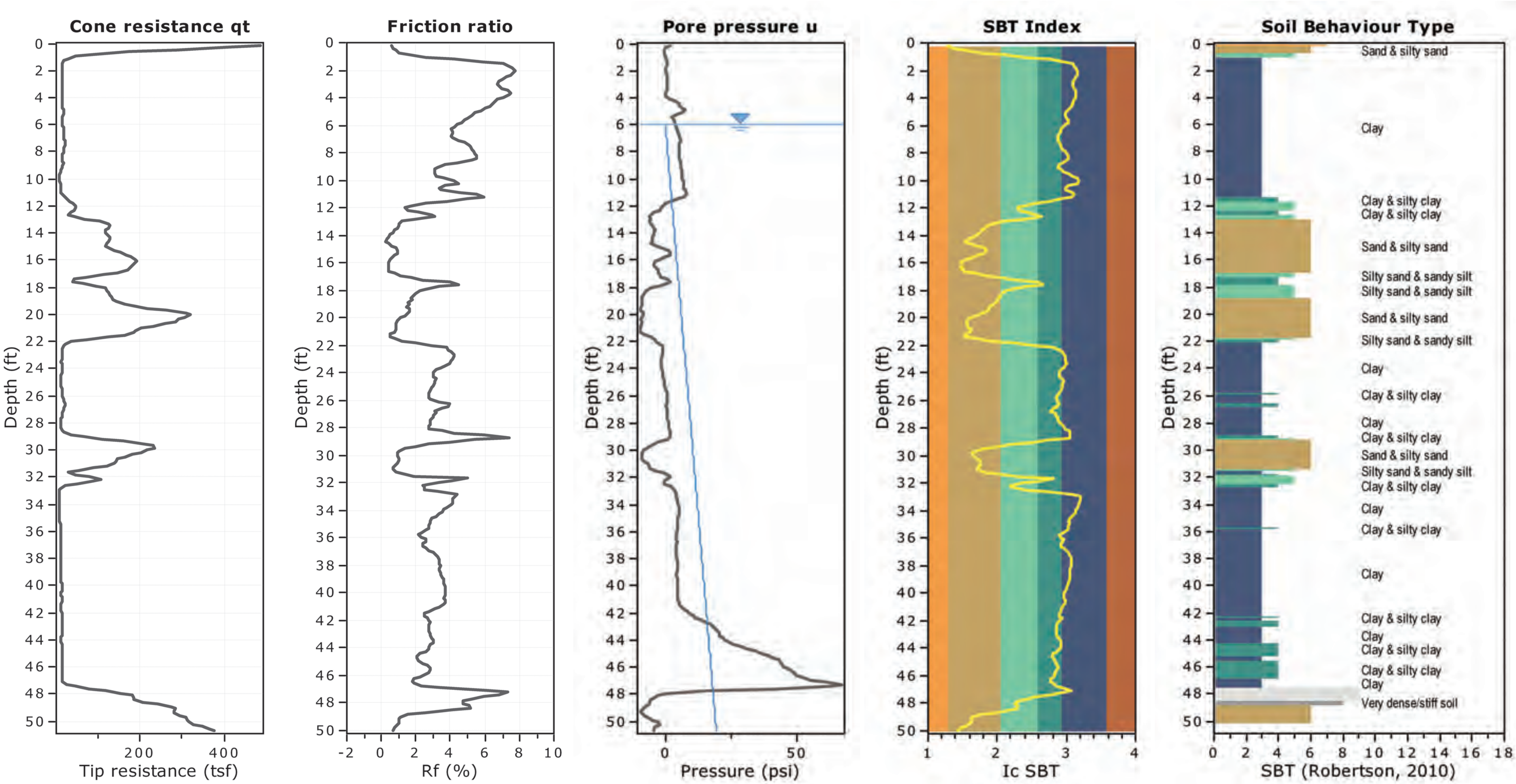
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-8

Date 02/02/21	Project No. 20-1950	Figure A-8

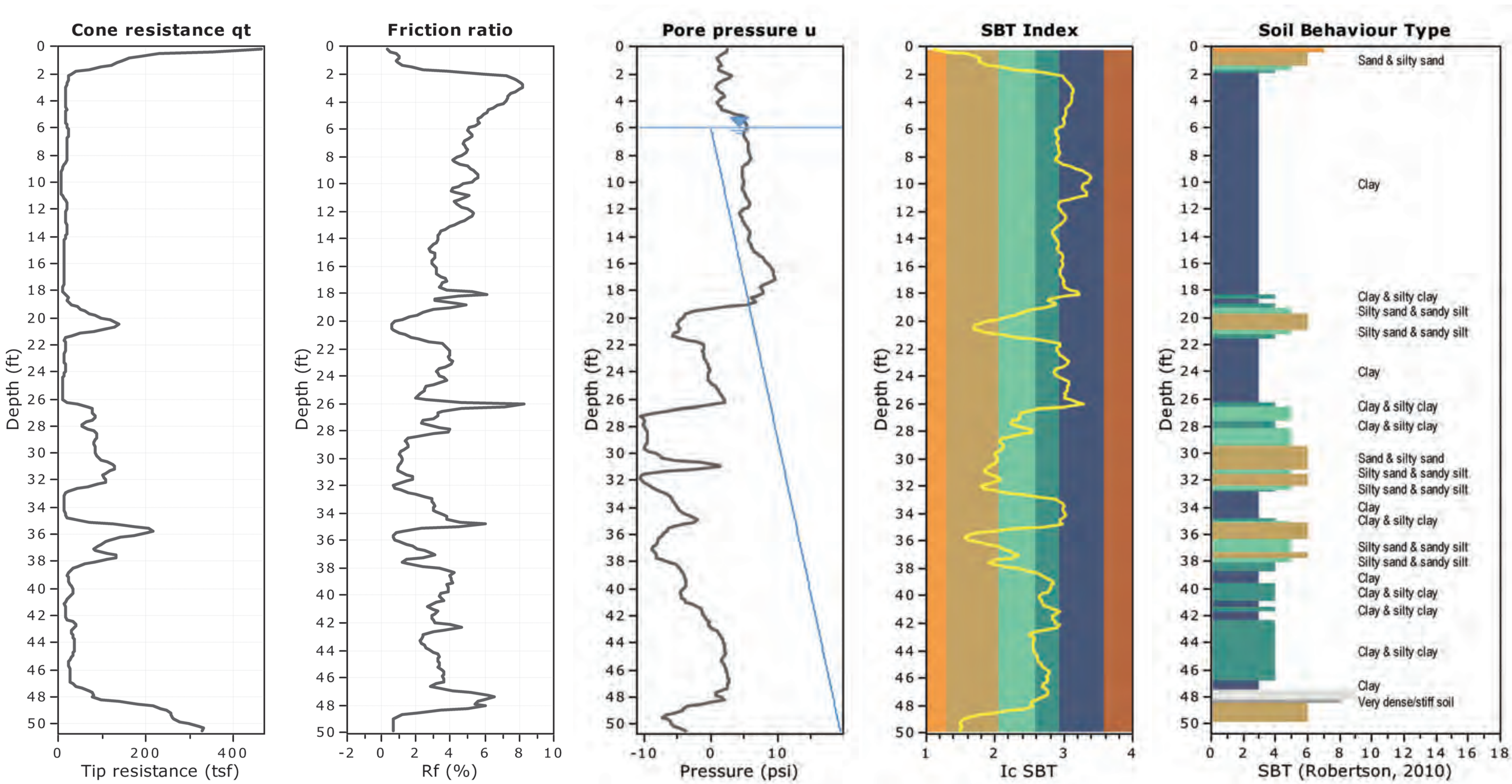


Approximate Ground Surface Elevation: 9.4 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 18, 2020
Depth to Groundwater: 6 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

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

123 INDEPENDENCE DRIVE Menlo Park, California	**CONE PENETRATION TEST RESULTS** **CPT-9**
ROCKRIDGE GEOTECHNICAL	Date 02/02/21	Project No. 20-1950	Figure A-9



Approximate Ground Surface Elevation: 8.8 feet (NAVD 88)
Total depth: 50.5 ft, Date: December 17, 2020
Depth to Groundwater: 6 feet (measured with weighted tape following rod removal)
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

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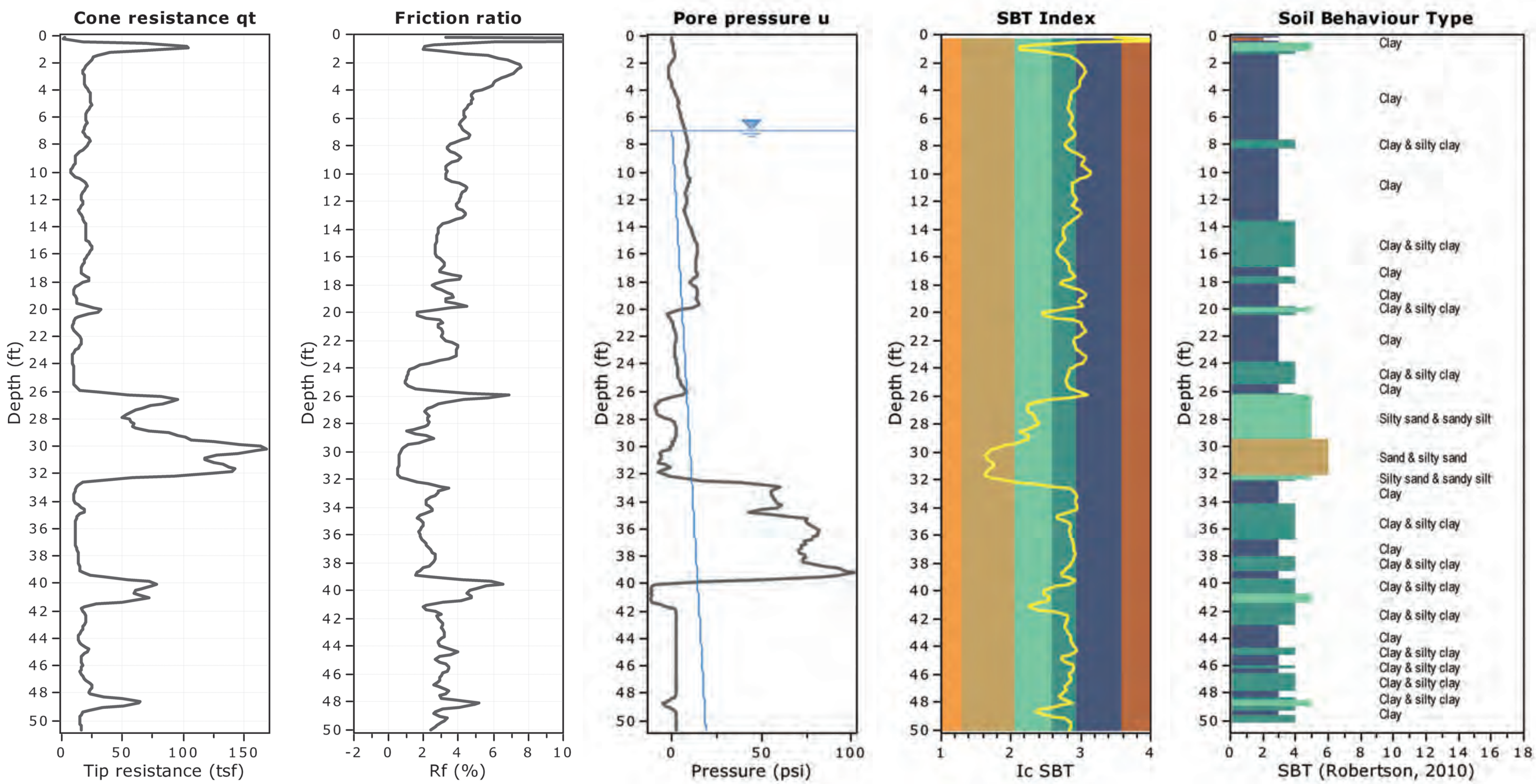
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-10

Date 02/02/21	Project No. 20-1950	Figure A-10



Approximate Ground Surface Elevation: 8.7 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 17, 2020
Depth to Groundwater: 7 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

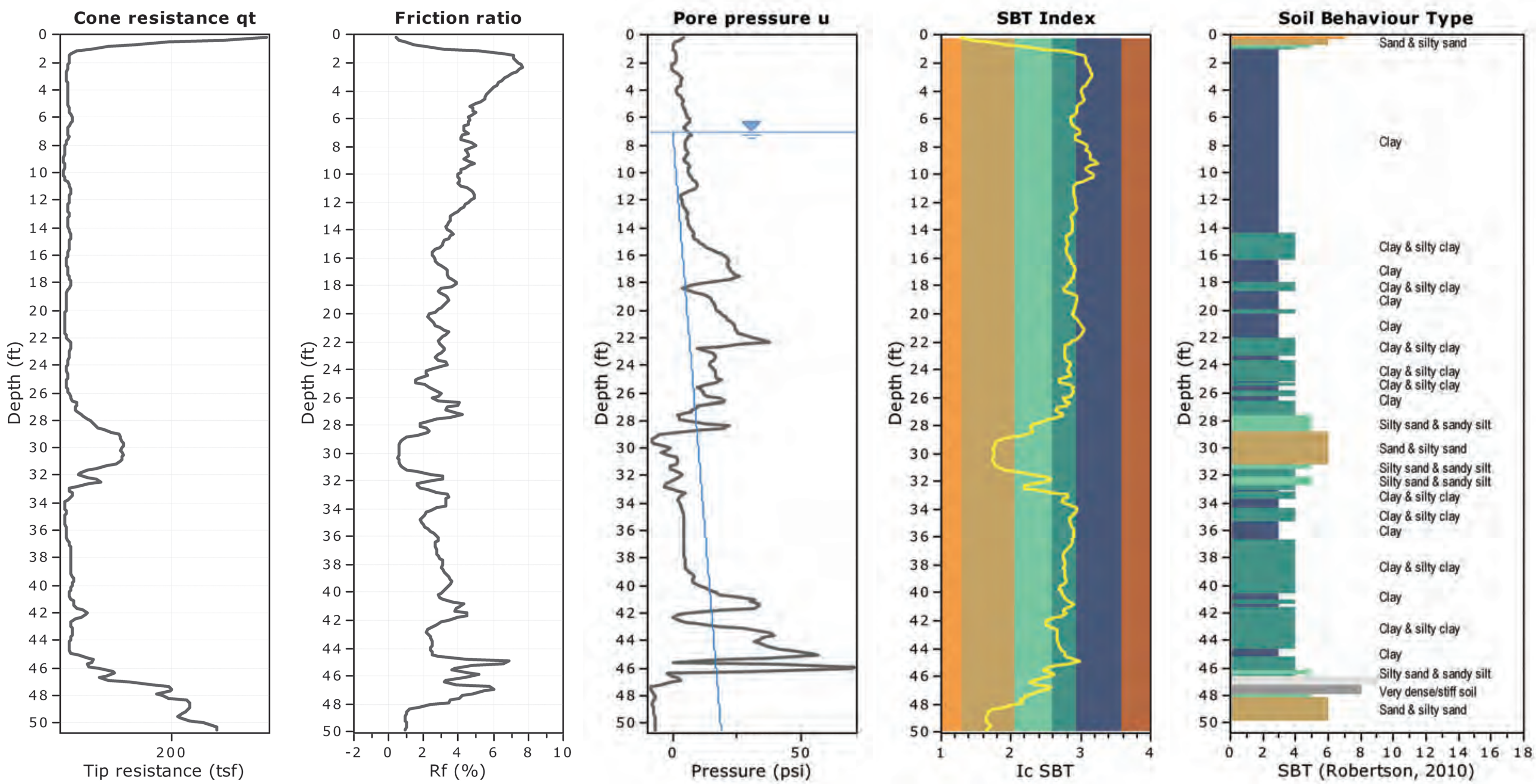
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-11

Date 02/02/21	Project No. 20-1950	Figure A-11



Approximate Ground Surface Elevation: 9.1 feet (NAVD 88)
Total depth: 50.5 ft, Date: December 17, 2020
Depth to Groundwater: 7 feet (measured with weighted tape following rod removal)
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

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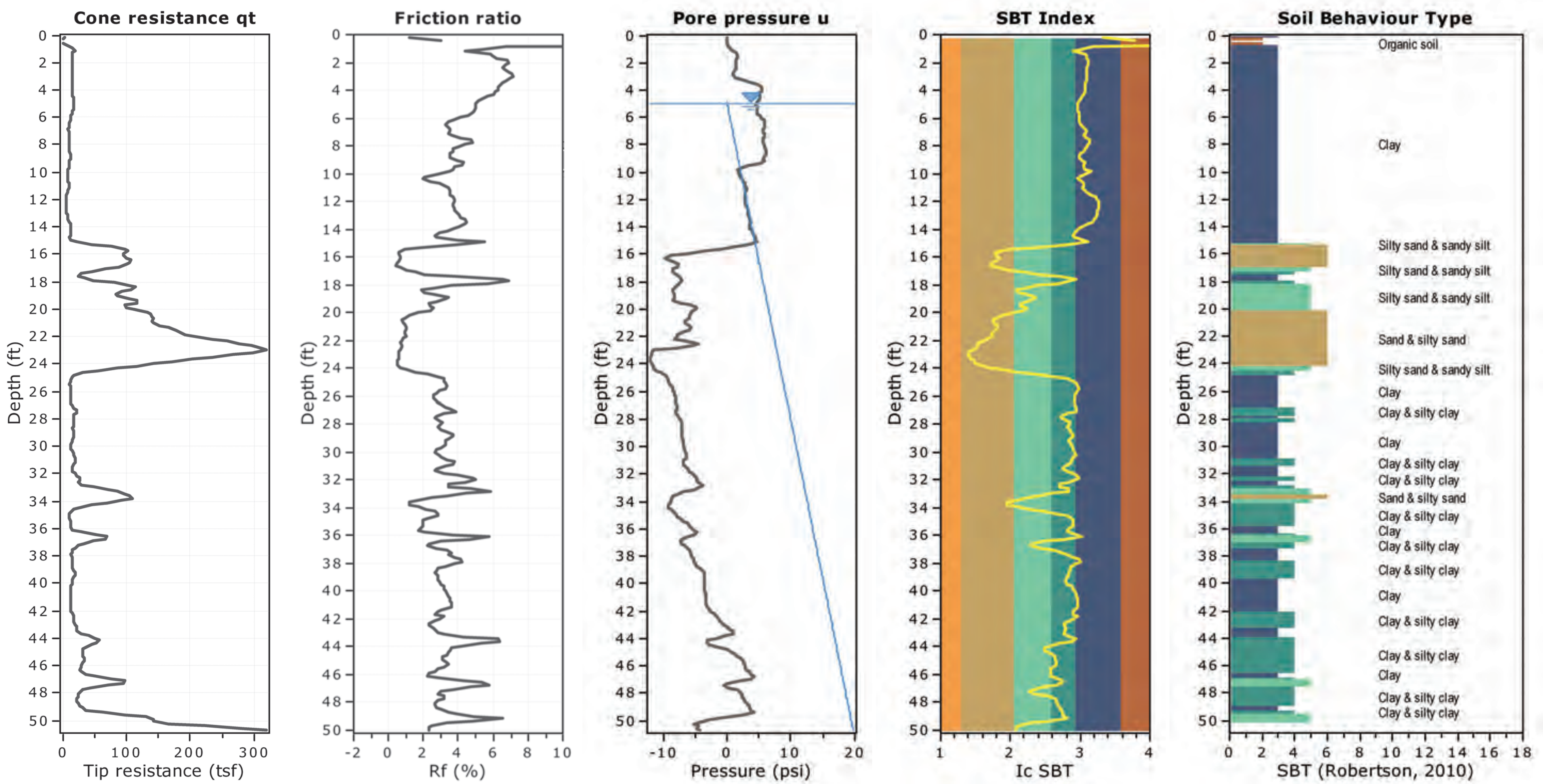
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-12

Date 02/02/21	Project No. 20-1950	Figure A-12



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

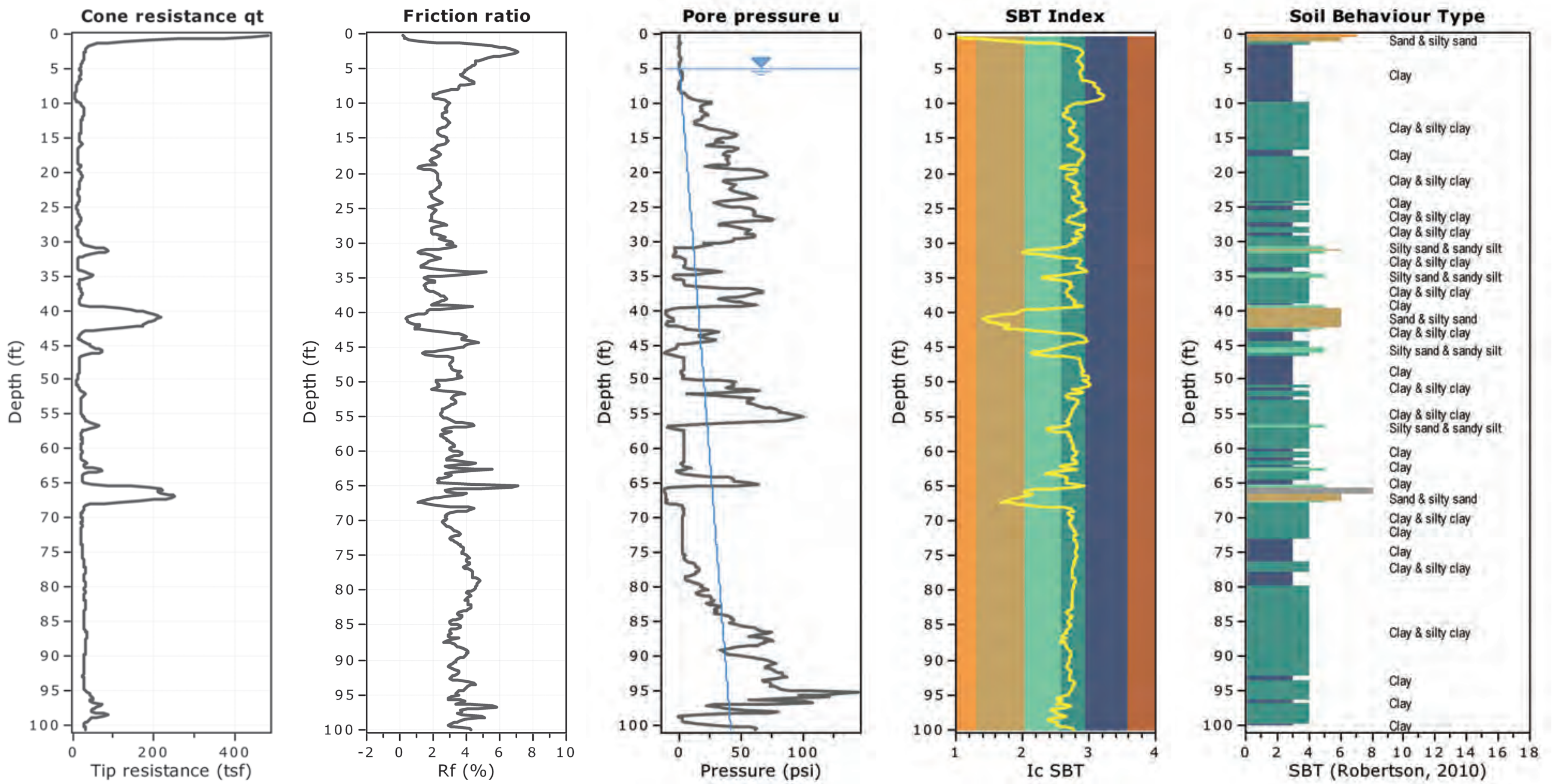
Approximate Ground Surface Elevation: 9.6 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 18, 2020
Depth to Groundwater: 5 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS CPT-13

Date 02/02/21	Project No. 20-1950	Figure A-13



Approximate Ground Surface Elevation: 8.2 feet (NAVD 88)
Total depth: 100.7 ft, Date: December 17, 2020
Depth to Groundwater: 5 feet (measured with weighted tape following rod removal)
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

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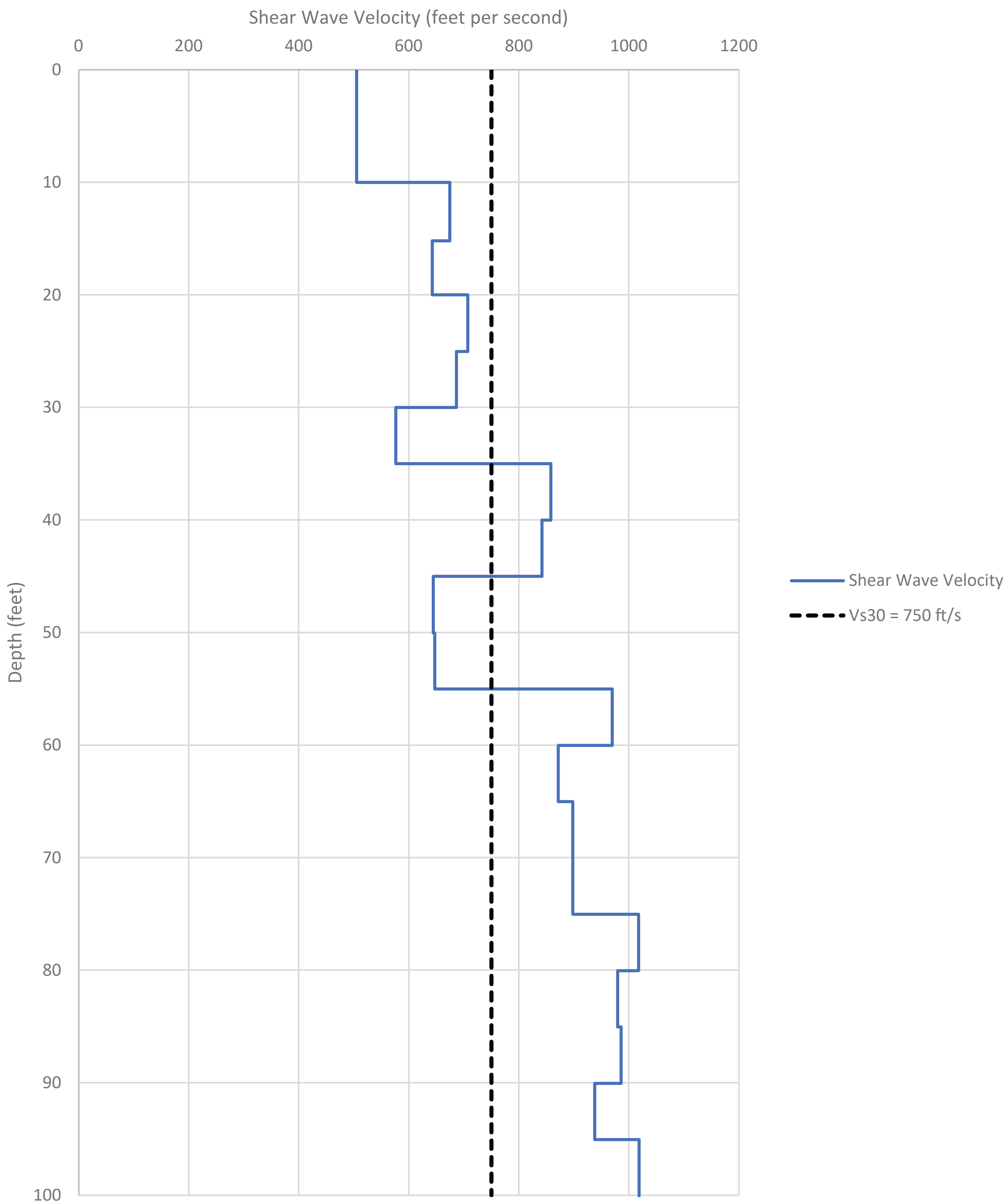
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-14

Date 02/02/21	Project No. 20-1950	Figure A-14

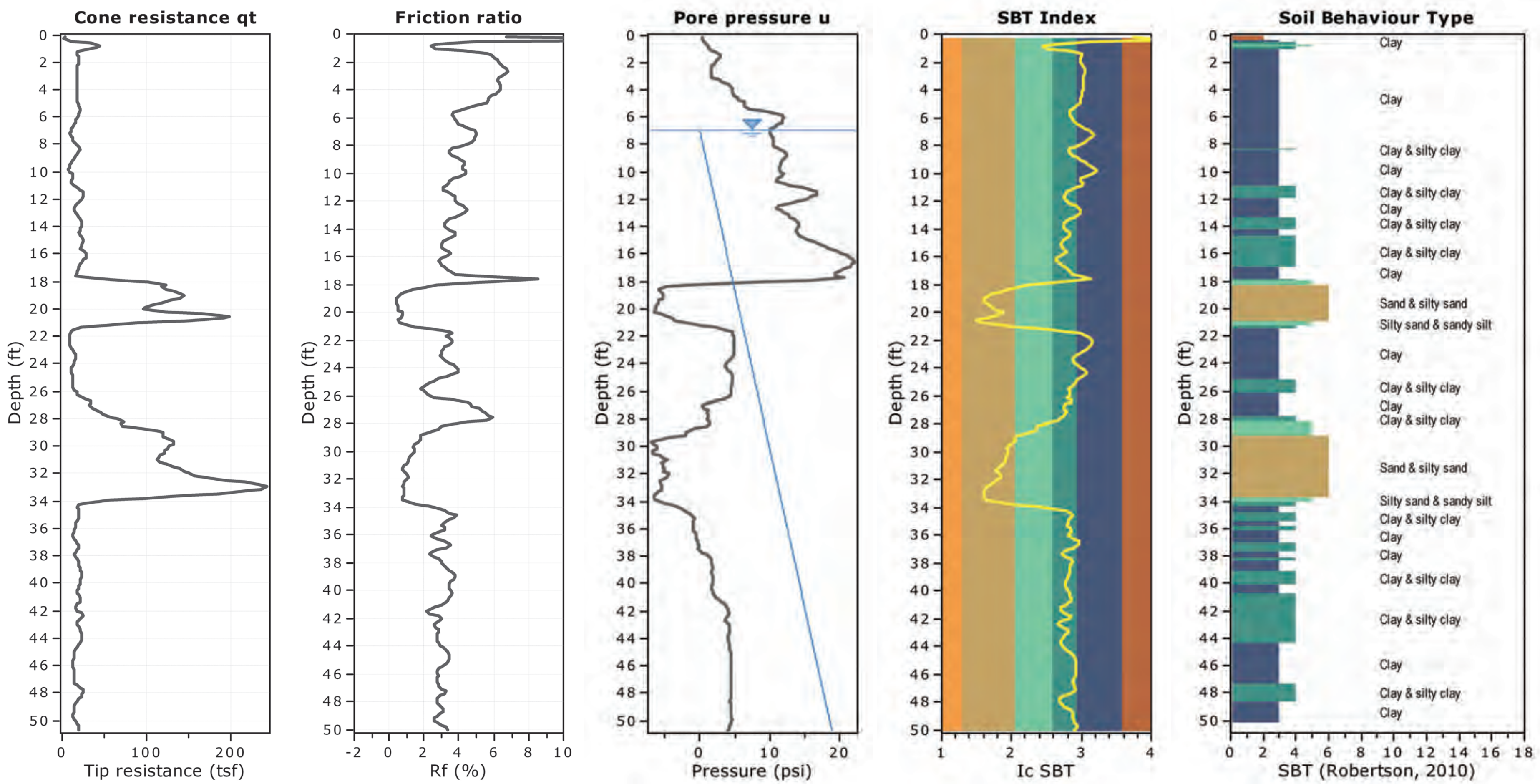


123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

SHEAR WAVE VELOCITY PROFILE
CPT-14

Date 01/12/21	Project No. 20-1950	Figure A-14b

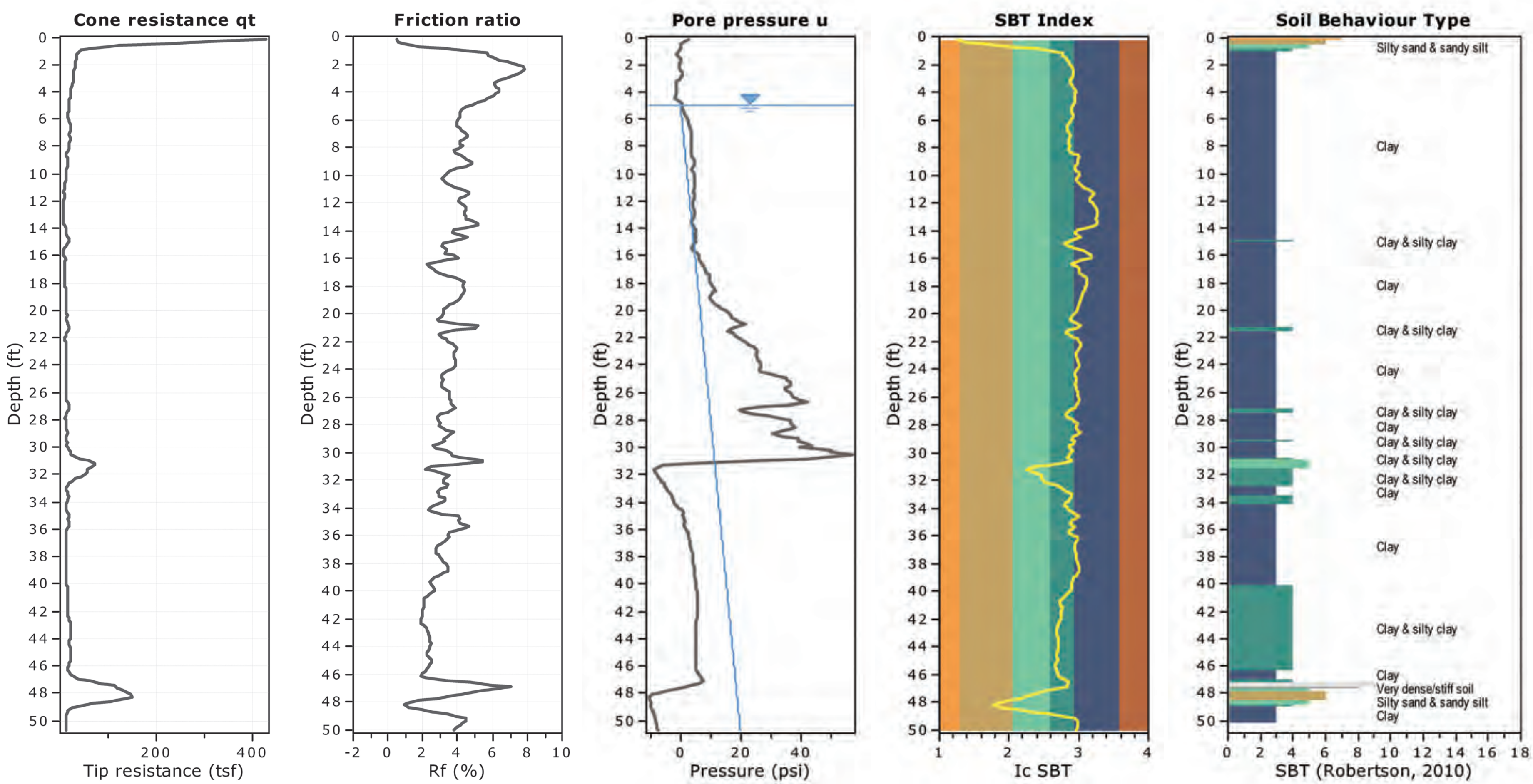


Approximate Ground Surface Elevation: 9.8 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 17, 2020
Depth to Groundwater: 7 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

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

123 INDEPENDENCE DRIVE Menlo Park, California	**CONE PENETRATION TEST RESULTS** **CPT-15**
ROCKRIDGE GEOTECHNICAL	Date 02/02/21 · Project No. 20-1950 · Figure A-15



Approximate Ground Surface Elevation: 8.8 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 18, 2020
Depth to Groundwater: 5 feet (estimated from pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

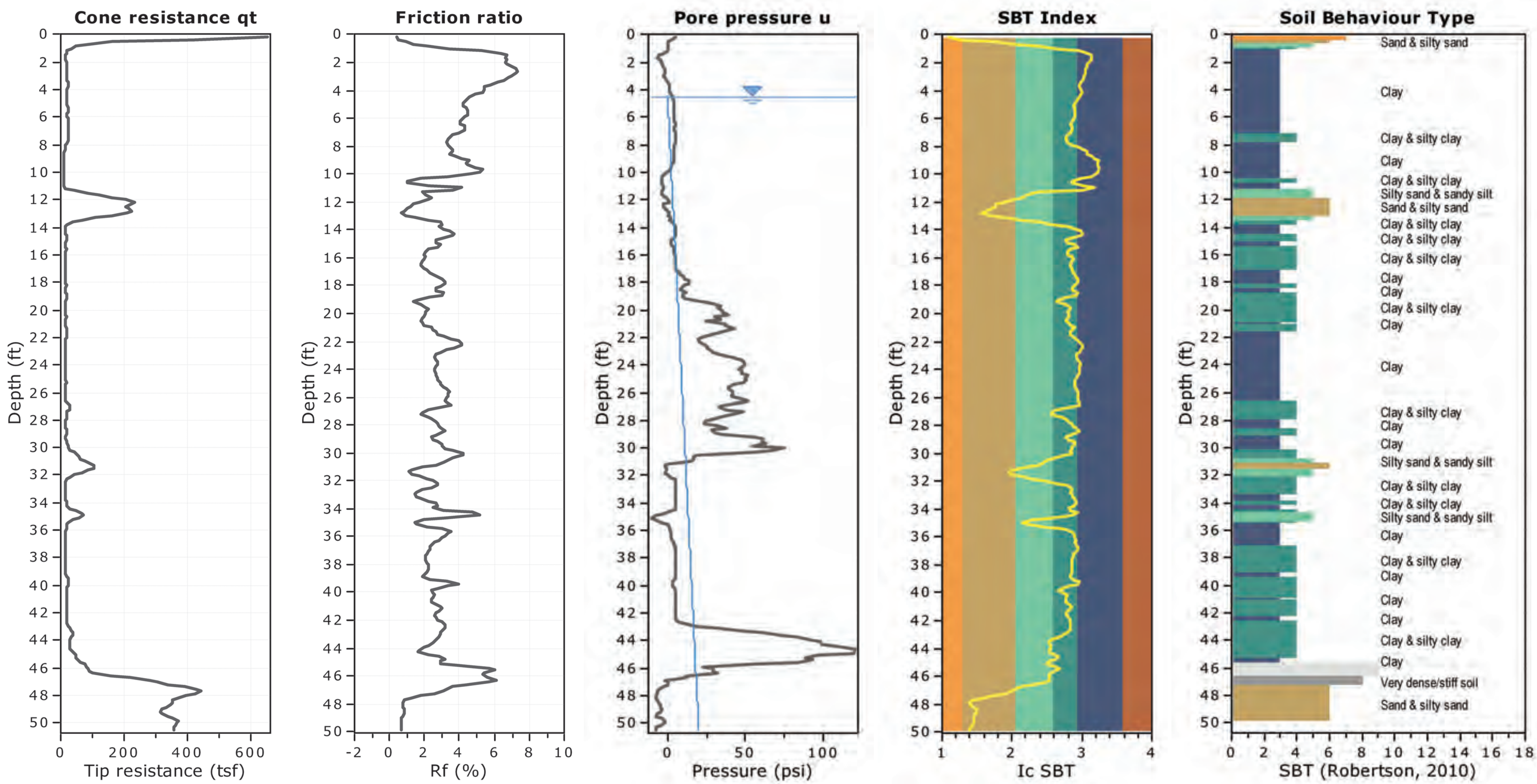
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-16

Date 02/02/21	Project No. 20-1950	Figure A-16

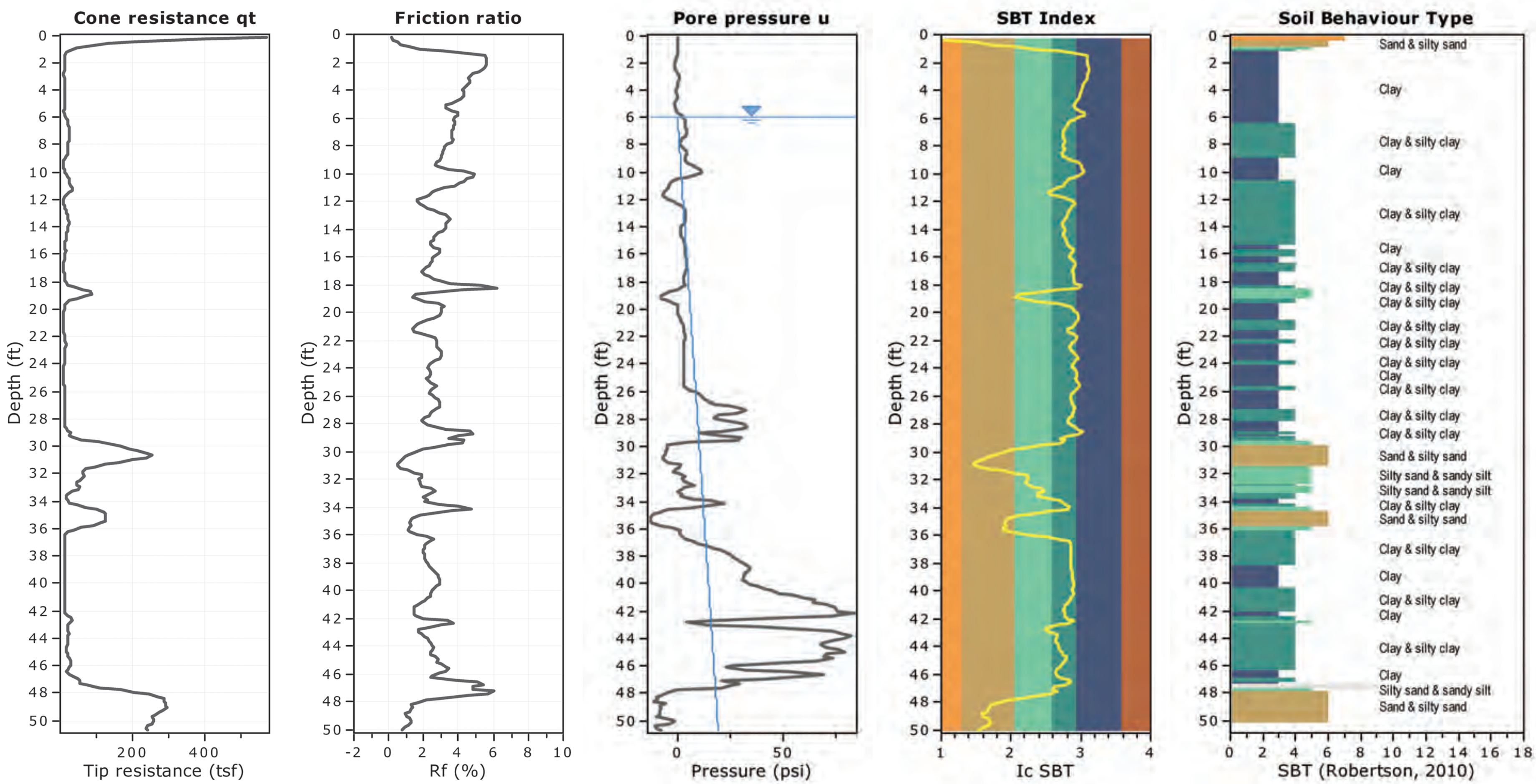


Approximate Ground Surface Elevation: 8.6 feet (NAVD 88)
Total depth: 50.5 ft, Date: December 18, 2020
Depth to Groundwater: 4.5 feet (estimated from pore pressure dissipation test)
Cone Operator: Middle Earth Geo Testing, Inc.

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

123 INDEPENDENCE DRIVE Menlo Park, California	CONE PENETRATION TEST RESULTS CPT-17		
ROCKRIDGE GEOTECHNICAL	Date 02/02/21	Project No. 20-1950	Figure A-17



Approximate Ground Surface Elevation: 9.3 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 18, 2020
Depth to Groundwater: 6 feet (measured with weighted tape following rod removal)
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

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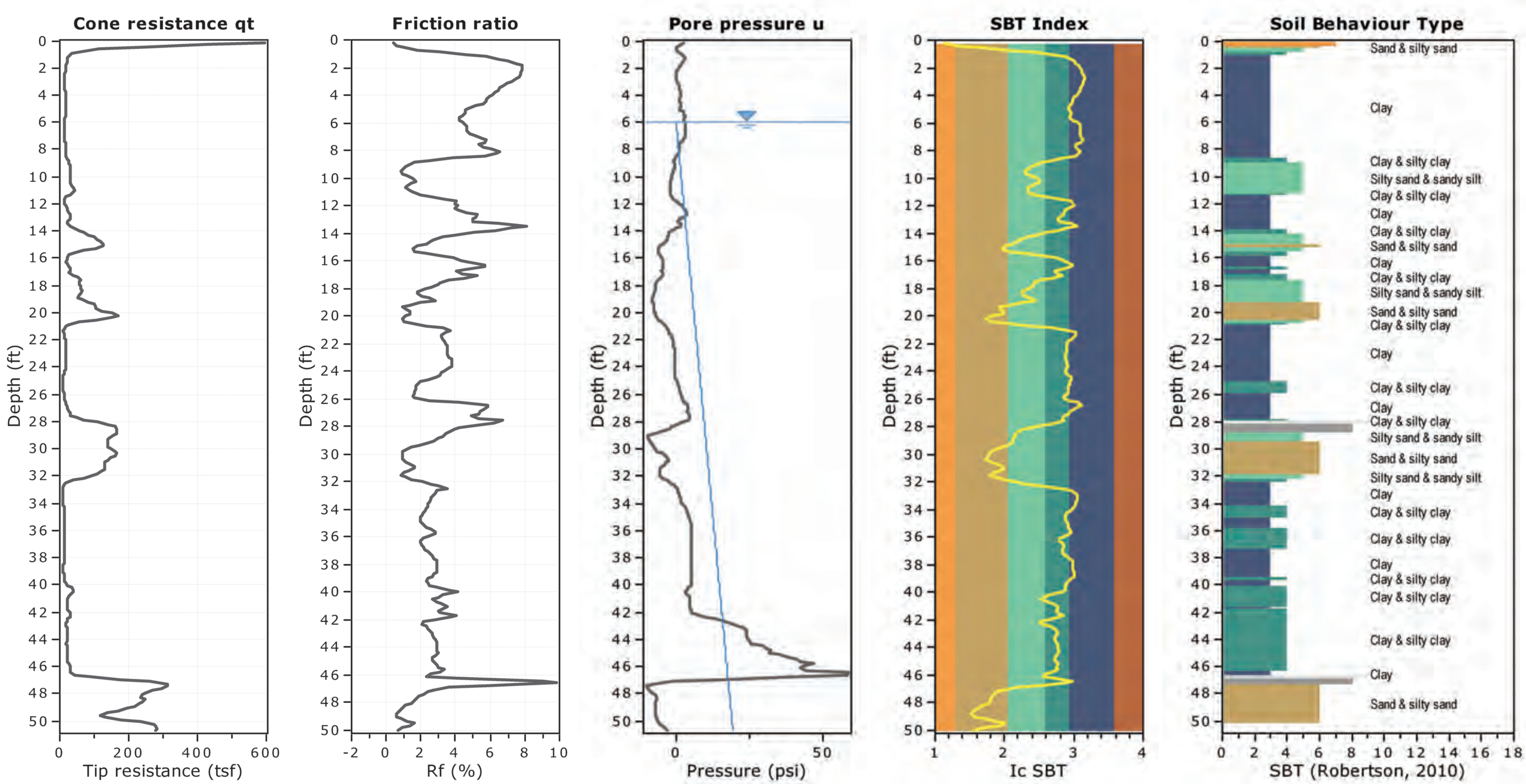
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-18

Date 02/02/21	Project No. 20-1950	Figure A-18



Approximate Ground Surface Elevation: 9.2 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 17, 2020
Depth to Groundwater: 6 feet (measured with weighted tape following rod removal)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

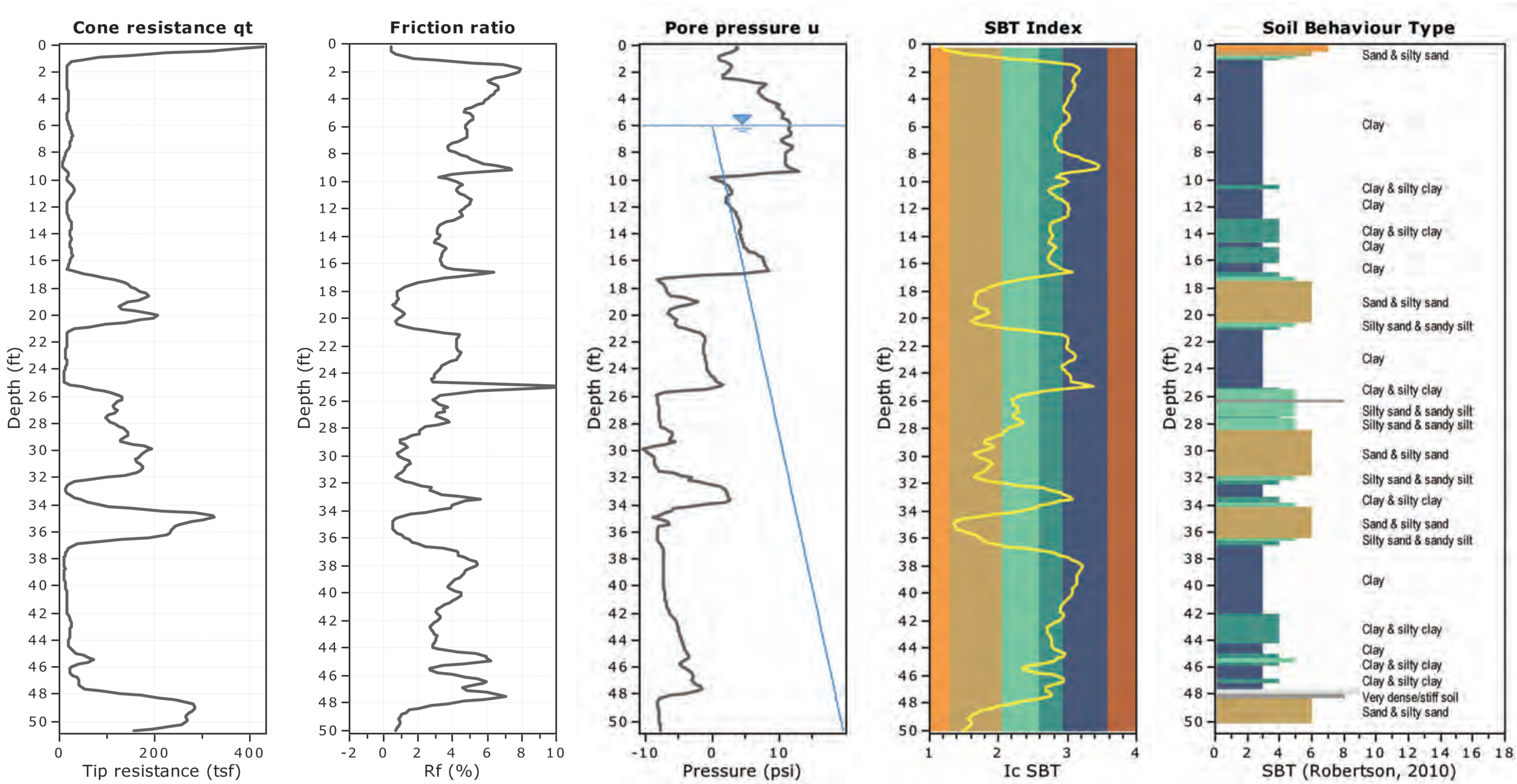
1. Sensitive fine grained
2. Organic material
3. Clay to silty clay
4. Clayey silt to silty clay
5. Silty sand to sandy silt
6. Clean sand to silty sand
7. Gravely sand to sand
8. Very stiff sand to clayey sand
9. Very stiff fine grained

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-19

Date 02/02/21	Project No. 20-1950	Figure A-19



Approximate Ground Surface Elevation: 9.2 feet (NAVD 88)
Total depth: 50.7 ft, Date: December 17, 2020
Depth to Groundwater: 6 feet (measured with weighted tape following rod removal)
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

123 INDEPENDENCE DRIVE Menlo Park, California	**CONE PENETRATION TEST RESULTS** **CPT-20**
ROCKRIDGE GEOTECHNICAL	Date 02/02/21 | Project No. 20-1950 | Figure A-20

PROJECT:	123 INDEPENDENCE DRIVE Menlo Park, California

Log of Boring B-1

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 12/23/2020 | Date finished: 12/23/2020

Drilling method: 8-inch-diameter hollow-stem auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Safety Hammer

Sampler: Modified California (MC), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: W. Gozali
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

LABORATORY TEST DATA

Approximate Ground Surface Elevation: 8.9 feet [2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3 inches of asphalt concrete						
0–1					12 inches of aggregate base						
1–2				CL	CLAY with SAND (CL) dark brown, hard, moist, fine to medium sand						
3	MC	11 22 30	33	CH	CLAY (CH) dark brown, hard, moist, trace sand					24.9	94
4–6	MC	11 20 25	28	CL	CLAY with SAND (CL) olive to olive-gray, very stiff, moist to wet ▽ (12/23/2020; 12:00 PM) ▽ (12/23/2020; 10:43 AM) Soil Corrosivity Test; see Appendix C					25.0	95
7				CL	SANDY CLAY (CL) yellow-brown mottled with red-brown and dark brown, very stiff, wet						
8	MC	9 9 6	9	SC	CLAYEY SAND (SC), light brown, loose, wet						
9–11	MC	7 9 27	23	CL	SANDY CLAY (CL) light brown to brown, stiff, wet, fine sand decrease in sand content very stiff, increase in fine to medium sand content decrease in medium sand						
15–16	MC	12 13 17	19	CL						22.4	106
19–21	SPT	15 16 17	36	SC	CLAYEY SAND with GRAVEL (SC) brown, dense, wet, fine to coarse sand, trace coarse gravel Particle Size Distribution; see Appendix B				16		
24				CL	CLAY with SAND (CL) light brown, very stiff, wet, trace coarse sand						
26	SPT	6 7 20	29	CL							
27–28	ST	200 psi		CL							
29				CL	trace gravel						
30–31	MC	30/6"	32/6"	SC	CLAYEY SAND with GRAVEL (SC) light brown, very dense, wet						

Boring terminated at a depth of 30.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 6 feet and 5.5 feet during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63 and 1.08, respectively, to account for sampler type and hammer energy.
[2] Elevation references the NAVD 88 datum.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1950 | Figure: A-21

PROJECT:	**123 INDEPENDENCE DRIVE** Menlo Park, California

Log of Boring B-2

Boring location: See Site Plan, Figure 2

Date started: 12/23/2020 | Date finished: 12/23/2020

Drilling method: 8-inch-diameter hollow-stem auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Safety Hammer

Sampler: Modified California (MC), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: W. Gozali
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

Approximate Ground Surface Elevation: 8.2 feet [2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2 inches of asphalt concrete						
					12 inches of aggregate base						
2	GRAB			CH	CLAY (CH) dark brown, very stiff, moist						
3	MC	8 14 36	25		Soil Corrosivity Test; see Appendix C; LL = 68, PI = 48; see Appendix B					29.5	96
5					(12/23/2020; 10:15 AM)						
5				CL	SANDY CLAY (CL) yellow-brown, very stiff to hard, wet; (12/23/2020; 7:45 AM)						
6	MC	12 18 30	30							20.2	110
7					gray-brown mottled with red-brown; red-brown						
8	MC	5 5 3	5		light brown, medium stiff; Particle Size Distribution; see Appendix B				52	28.3	97
10					yellow-brown, very stiff, trace gravel						
11	SPT	8 8 8	17								
12				SP	SAND (SP) yellow-brown to brown, medium dense, wet, trace fine gravel						
13	SPT	4 7 7	15								
13.5				GP	GRAVEL with SAND (GP) yellow-brown, medium dense, wet						
15				SP-SC	SAND with CLAY and GRAVEL (SP-SC) yellow-brown, medium dense, wet						
16	SPT	10 10 13	25								
16				CL	SANDY CLAY (CL) yellow-brown, very stiff, wet						
19				SP	SAND (SP) brown, very dense, wet						
20.5				SP	SAND with GRAVEL (SP) brown, very dense, wet, trace clay						
21	MC	12 28 50/ 54"	49/ 10"								
25				SC	CLAYEY SAND with GRAVEL (SC) tan, medium dense, wet						
26	SPT	5 8 9	18								
27	ST	300 psi									
28					brown						
29.5				SP	SAND (SP) brown, medium dense, wet						
31	SPT	4 5 11	17								

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1950 | Figure: A-22a

PROJECT: **123 INDEPENDENCE DRIVE**
Menlo Park, California

Log of Boring B-2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
33	SPT		20 30 38	43	SP	SAND (SP) (continued) trace clay						
34												
35	SPT		8 8 8	17	CL	SANDY CLAY (CL) olive-gray, very stiff, wet						
36												
37												
38	MC		20 30 38	43	SP	SAND (SP) olive-gray, dense, wet						
39												
40	ST		150 psi									
41												
42												
43												
44	SPT		12 18 16	37		dark brown						
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												
61												
62												
63												
64												

Boring terminated at a depth of 45 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 5.5 feet and 5 feet during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63 and 1.08, respectively, to account for sampler type and hammer energy.
[2] Elevation references the NAVD 88 datum.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1950

Figure: A-22b

PROJECT: **123 INDEPENDENCE DRIVE**
Menlo Park, California

Log of Boring B-3

Boring location: See Site Plan, Figure 2
Date started: 12/22/2020 Date finished: 12/22/2020
Drilling method: 8-inch-diameter hollow-stem auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Downhole Safety Hammer
Sampler: Modified California (MC), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: W. Gozali
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

MATERIAL DESCRIPTION

Approximate Ground Surface Elevation: 8.0 feet [2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3 inches of asphalt concrete						
0.25					6 inches of aggregate base						
0.75				CH	CLAY (CH) dark brown, hard, moist, trace coarse sand						
3	MC	20, 20, 30	32								
3.5				CL	SANDY CLAY (CL) brown mottled with olive, hard, moist, fine sand						
5–6	MC	14, 18, 20	24		very stiff, wet; ▽ (12/22/2020; 3:00 PM); ▽ (12/22/2020; 1:25 PM)					21.0	107
7.5				CL	CLAY with SAND (CL) light brown, medium stiff, wet						
8	MC	7, 8, 9	11		TxUU Test; see Appendix B	TxUU	1,000	690		29.3	95
10					yellow-brown, very stiff						
10.5				CL	SANDY CLAY (CL) yellow-brown, very stiff, wet						
11	MC	18, 20, 20	25								
16	MC	10, 14, 17	20		tan to light brown, trace gravel						
18–20	ST	300 psi									
19				SC	CLAYEY SAND with GRAVEL (SC) gray-brown, wet						
23				CL	CLAY with SAND (CL) yellow-brown, very stiff, wet, trace gravel						
25.5				CL	SANDY CLAY (CL) olive-gray, very stiff, wet						
26	MC	7, 12, 17	18								
28.5				CL	SANDY CLAY (CL) yellow brown, very stiff, wet						
30.5				CL	CLAY with SAND (CL) olive-brown, very stiff, wet, trace gravel						
31	MC	10, 14, 12	16								
32											

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1950
Figure: A-23a

PROJECT:	123 INDEPENDENCE DRIVE Menlo Park, California	Log of Boring B-3 PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
33					CL	CLAY with SAND (CL) (continued)						
34					CL	SANDY CLAY (CL) olive-gray, very stiff, wet						
35–36	MC		13 14 16	19								
37												
38					SC	CLAYEY SAND (SC) olive-gray, dense, wet						
39												
40–41	SPT		9 18 36	34								
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
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62												
63												
64												

Boring terminated at a depth of 41.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 6.5 feet and 6 feet during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63 and 1.08, respectively, to account for sampler type and hammer energy.
[2] Elevation references the NAVD 88 datum.

ROCKRIDGE GEOTECHNICAL

Project No.:	Figure:
20-1950	A-23b

PROJECT: **123 INDEPENDENCE DRIVE**
Menlo Park, California

Log of Boring B-4

Boring location:	See Site Plan, Figure 2
Date started:	12/23/2020
Date finished:	12/23/2020
Drilling method:	8-inch-diameter hollow-stem auger
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Downhole Safety Hammer
Sampler:	Modified California (MC), Standard Penetration Test (SPT), Shelby Tube (ST)
Logged by:	W. Gozali
Drilled by:	Exploration Geoservices, Inc.
Rig:	Mobile B-61

Approximate Ground Surface Elevation: 9.3 feet [2]

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					2 inches of asphalt concrete 4 inches of aggregate base						
1–2	MC	20 23 36	40	CH	CLAY (CH) dark brown, hard, moist					32.2	90
3–4	MC	8 22 24	29	CH	CLAY with SAND (CH) dark brown, very stiff, moist, fine to coarse sand LL = 58, PI = 40; see Appendix B					24.9	103
5–6	MC	13 14 16	19		gray-brown Soil Corrosivity Test; see Appendix C						
7–8	MC	3 5 7	8	SC	∇ (12/23/2020; 2:45 PM) CLAYEY SAND (SC) yellow-brown, loose, wet, trace gravel Particle Size Distribution; see Appendix B				41	21.7	106
10–11	MC	7 8 9	11		medium dense						
13–14	MC	7 9 13	14	CL	SANDY CLAY (CL) light brown, stiff, wet	TxUU	1,600	1,700		20.9	108
15–16	MC	8 14 28	26		very stiff, trace coarse sand						
18–19	SPT	6 9 9	19	SC	CLAYEY SAND (SC) yellow-brown, medium dense, wet						
20–21	SPT	50/6"	54/6"	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, wet						
23–24				CL	CLAY with SAND (CL) light brown, stiff , wet						
25–26	SPT	4 6 8	15								
30–31	SPT	6 8 8	16		light gray, very stiff						
31.5–32	ST										

PROJECT: **123 INDEPENDENCE DRIVE**
Menlo Park, California

Log of Boring B-4

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					CL	CLAY with SAND (CL) (continued)						
33	ST		200 psi		CL	CLAY with SAND (CL) light brown, very stiff, wet						
34												
35												
36												
37												
38												
39												
40												
41												
42												
43												
44												
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57												
58												
59												
60												
61												
62												
63												
64												

Boring terminated at a depth of 34 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 7 feet during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63 and 1.08, respectively, to account for sampler type and hammer energy.
[2] Elevation references the NAVD 88 datum.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1950

Figure: A-24b

PROJECT: **123 INDEPENDENCE DRIVE**
Menlo Park, California

Log of Boring B-5

Boring location: See Site Plan, Figure 2
Date started: 12/22/2020 | Date finished: 12/22/2020
Drilling method: 8-inch-diameter hollow-stem auger
Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Safety Hammer
Sampler: Modified California (MC), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: W. Gozali
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

Approximate Ground Surface Elevation: 9.3 feet [2]

DEPTH (feet)	Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0					3 inches of asphalt concrete						
					6 inches of aggregate base						
1–2	MC	9, 20, 40	38	CH	CLAY with SAND (CH) dark brown, hard, moist, trace coarse sand, trace fine gravel						
3–4	MC	14, 14, 44	58		gray-brown LL = 60, PI = 41; see Appendix B					23.4	105
5–6	MC	6, 8, 10	18	SC	CLAYEY SAND (SC) olive-gray, medium dense, moist to wet (12/22/2020; 7:57 AM) ▽						
6				CL	CLAY with SAND (CL), dark brown, very stiff, wet						
7				SC	CLAYEY SAND with GRAVEL (SC) olive-gray, medium dense, wet						
7–8	SPT	14, 9, 10	21	GP	GRAVEL with SAND (GP) brown, medium dense, wet, trace clay						
10–11	SPT	5, 5, 10	16		Particle Size Distribution; see Appendix B				4		
14				SC	CLAYEY SAND with GRAVEL (SC) yellow-brown, dense, wet						
15–17	ST	200 psi									
17–19	SPT	24, 20, 28	43								
25–27	ST	200 psi									
27				CL	SANDY CLAY (CL) olive-brown, medium stiff, wet, trace to some gravel	PP		750			
29.5				SM	SILTY SAND (SM) olive to light brown, very dense, wet, trace gravel						
30–31	MC	16, 26, 50/6"	48								
31					SAND (SP), brown, dense, wet, trace gravel						
32											

Boring terminated at a depth of 31.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 6 feet during drilling.
PP: Pocket Penetrometer Test

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7 and 1.2, respectively, to account for sampler type and hammer energy.
[2] Elevation references the NAVD 88 datum.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1950 | Figure: A-25

PROJECT: **123 INDEPENDENCE DRIVE**
Menlo Park, California

Log of Boring B-6

Boring location: See Site Plan, Figure 2

Date started: 12/23/2020 | Date finished: 12/23/2020

Drilling method: 8-inch-diameter hollow-stem auger

Hammer weight/drop: 140 lbs./30 inches | Hammer type: Downhole Safety Hammer

Sampler: Modified California (MC), Standard Penetration Test (SPT), Shelby Tube (ST)

Logged by: W. Gozali
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

Approximate Ground Surface Elevation: 9.3 feet[2]

DEPTH (feet)	SAMPLES: Sampler Type	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
					3 inches of asphalt concrete						
					6 inches of aggregate base						
1–2	GRAB			CH	CLAY (CH) dark brown mottled with brown, very stiff, moist						
3	MC	8 18 26	28	CH	Soil Corrosivity Test; see Appendix C					26.3	98
5				CL	SANDY CLAY (CL) brown mottled with red-brown, hard, moist						
6	MC	12 24 32	35	CL						21.7	107
7				CL	∇ wet (12/22/2020; 10:20 AM & 11:45 AM)						
10				CL	light brown to gray-brown, stiff, wet, fine sand						
11	MC	6 8 9	11	CL							
13	ST	300 psi		CL							
13				CL	SANDY CLAY with GRAVEL (CL) light brown, medium stiff to stiff, wet						
17				CL	SANDY CLAY (CL) yellow-brown, hard, wet, some sand						
21	MC	18 36 48	53	CL							
26	SPT	5 10 15	27	CL	light brown, very stiff						
28	SPT	20 28 17	49	SP-SC	SAND with CLAY and GRAVEL (SP-SC) light brown, dense, wet						
31	SPT	12 30 26	60	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, wet; 4” thick lense of sandy clay						
32											

LABORATORY TEST DATA

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1950

Figure: A-26a

PROJECT:	**123 INDEPENDENCE DRIVE** Menlo Park, California	**Log of Boring B-6** PAGE 2 OF 2

DEPTH (feet)	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						CLAYEY SAND with GRAVEL (SC) (continued)						
33												
34					SC							
35	SPT		7 7 11	19								
36						CLAY with SAND (CL) olive-gray, very stiff, wet, fine sand, trace silt						
37												
38												
39					CL							
40	SPT		10 10 10	22								
41												
42												
43												
44												
45												
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Boring terminated at a depth of 41.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 7 feet during drilling.

[1] MC and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.63 and 1.08, respectively, to account for sampler type and hammer energy.
[2] Elevation references the NAVD 88 datum.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1950	Figure: A-26b

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions		Symbols	Typical Names
Coarse-Grained Soils (more than half of soil > no. 200 sieve size)	Gravels (More than half of coarse fraction > no. 4 sieve size)	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	Sands (More than half of coarse fraction < no. 4 sieve size)	SW	Well-graded sands or gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
Fine -Grained Soils (more than half of soil < no. 200 sieve size)	Silts and Clays LL = < 50	ML	Inorganic silts and clayey silts of low plasticity, sandy silts, gravelly silts
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	Silts and Clays LL = > 50	MH	Inorganic silts of high plasticity
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic silts and clays of high plasticity
Highly Organic Soils		PT	Peat and other highly organic soils

GRAIN SIZE CHART

Classification	Range of Grain Sizes	
	U.S. Standard Sieve Size	Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel coarse fine	3" to No. 4 3" to 3/4" 3/4" to No. 4	76.2 to 4.76 76.2 to 19.1 19.1 to 4.76
Sand coarse medium fine	No. 4 to No. 200 No. 4 to No. 10 No. 10 to No. 40 No. 40 to No. 200	4.76 to 0.075 4.76 to 2.00 2.00 to 0.420 0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

Unstabilized groundwater level

Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

Sample taken with California or Modified California split-barrel sampler. Darkened area indicates soil recovered

Classification sample taken with Standard Penetration Test sampler

Undisturbed sample taken with thin-walled tube

Disturbed sample

Sampling attempted with no recovery

Core sample

Analytical laboratory sample

Sample taken with Direct Push sampler

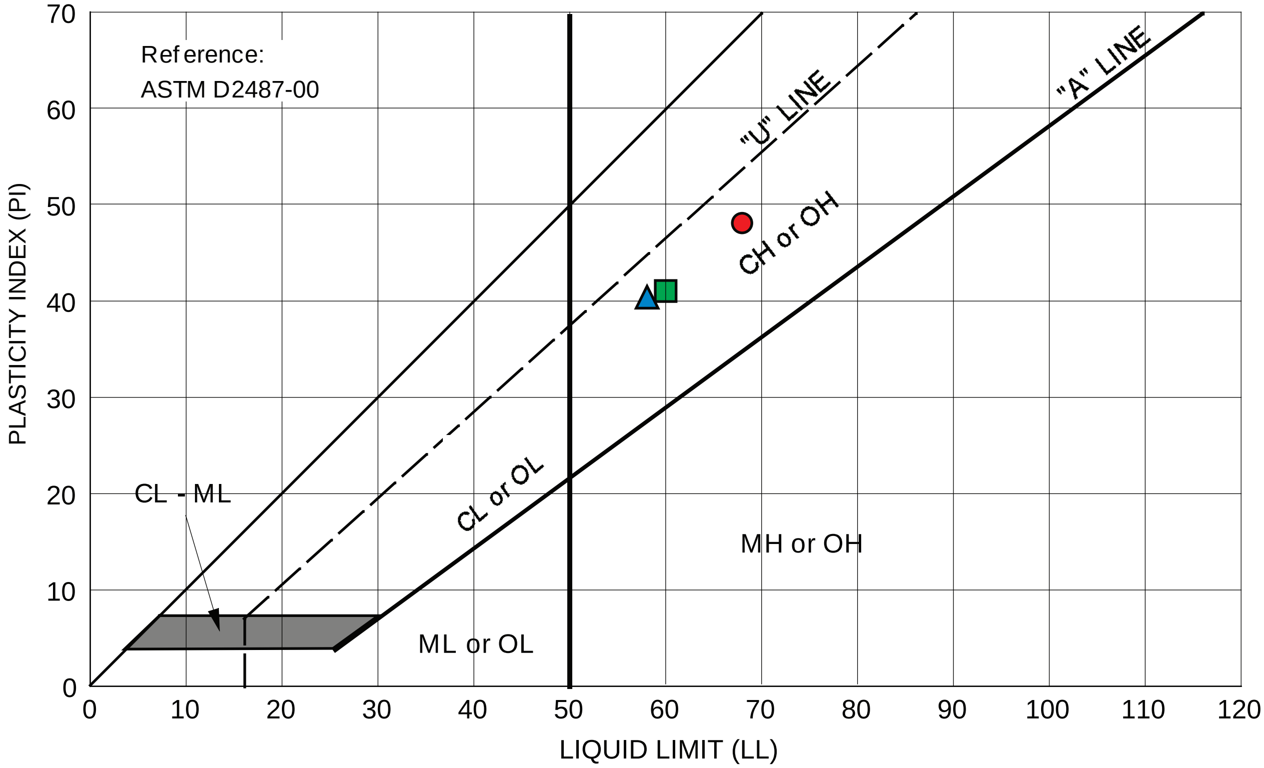
Sonic

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
MC	Modified California sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.5-inch inside diameter
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

ROCKRIDGE GEOTECHNICAL

APPENDIX B
Laboratory Test Results



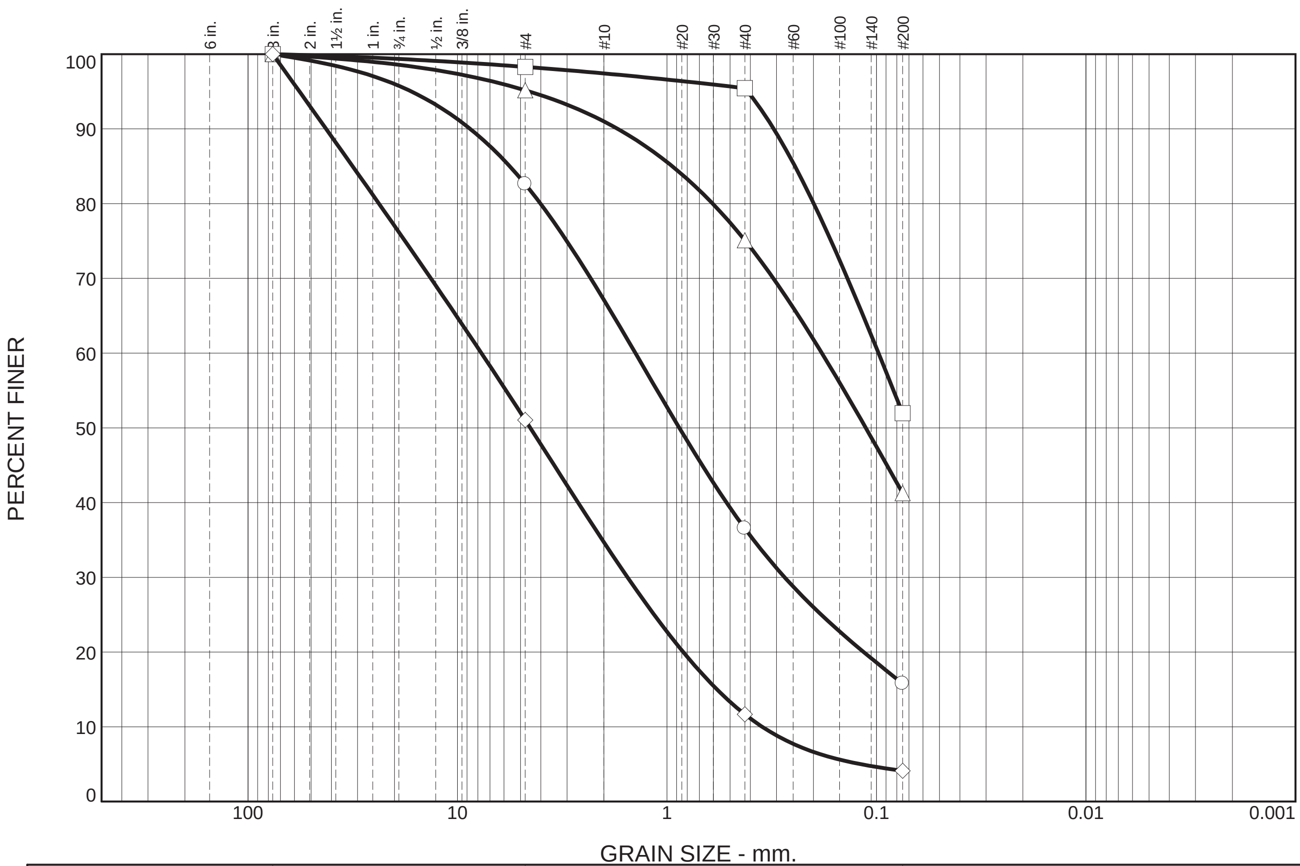
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-2 at 3.5 feet	CLAY (CH), dark brown	29.5	68	48	--
▲	B-4 at 4.0 feet	CLAY with SAND (CH), dark brown	24.9	58	40	--
■	B-5 at 3.25 feet	CLAY with SAND (CH), dark brown	23.4	60	41	--

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

PLASTICITY CHART

Date 02/01/21 | Project No. 20-1950 | Figure B-1



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	4.2	13.2	15.5	30.6	20.7	15.8	
□	0.0	0.6	1.1	0.9	2.0	43.4	52.0	
△	0.0	1.4	3.4	4.2	15.9	33.8	41.3	
◇	0.0	23.8	25.1	16.4	23.0	7.6	4.1	

SOIL DATA

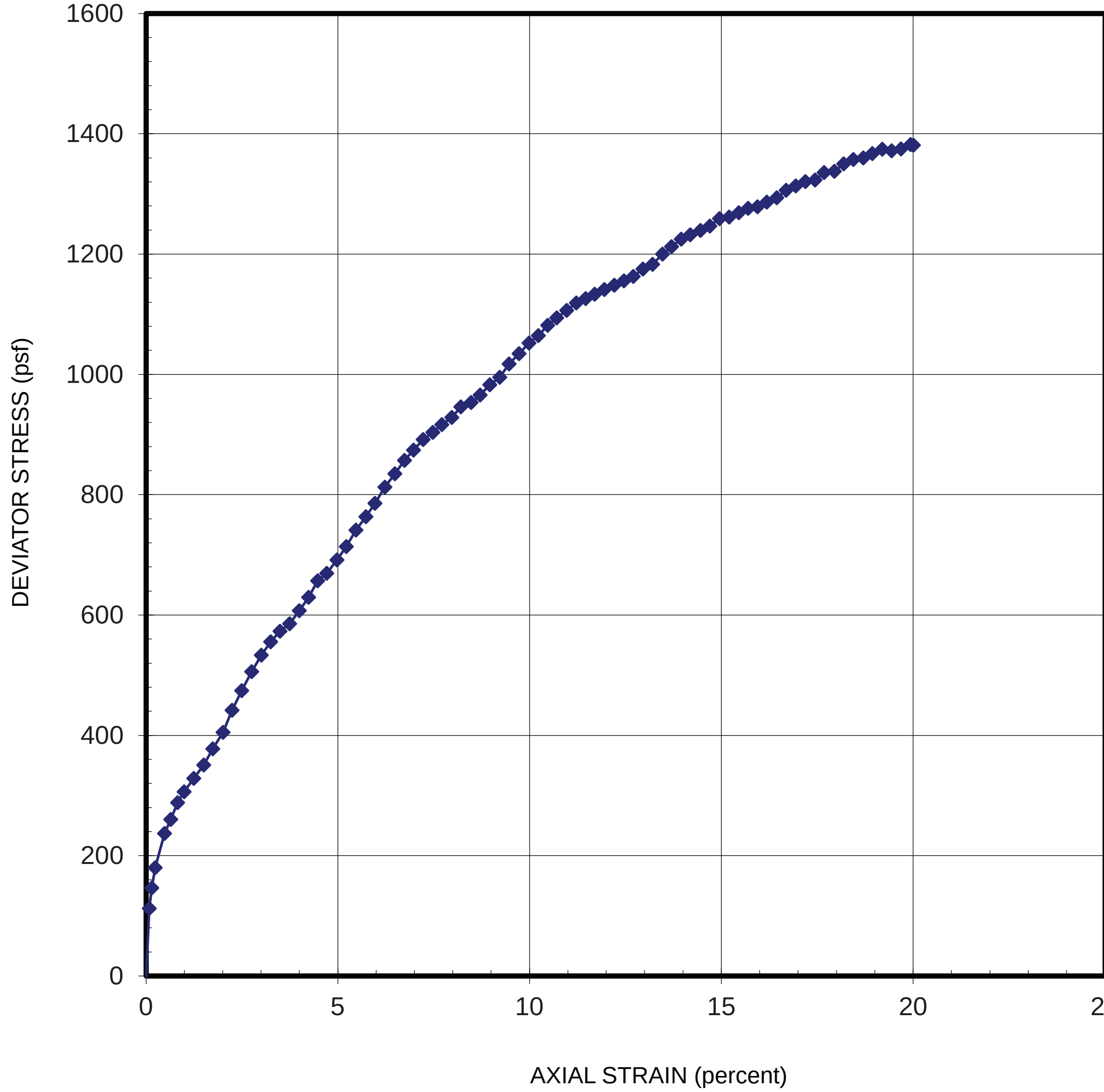
SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	20.0'	CLAYEY SAND with GRAVEL, brown	SC
□	B-2	8.5'	SANDY CLAY, light brown	CL
△	B-4	8.0	CLAYEY SAND, yellow-brown	SC
◇	B-5	10.0'	GRAVEL with SAND, brown	GP

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 01/26/21 | Project No. 20-1950 | Figure B-2



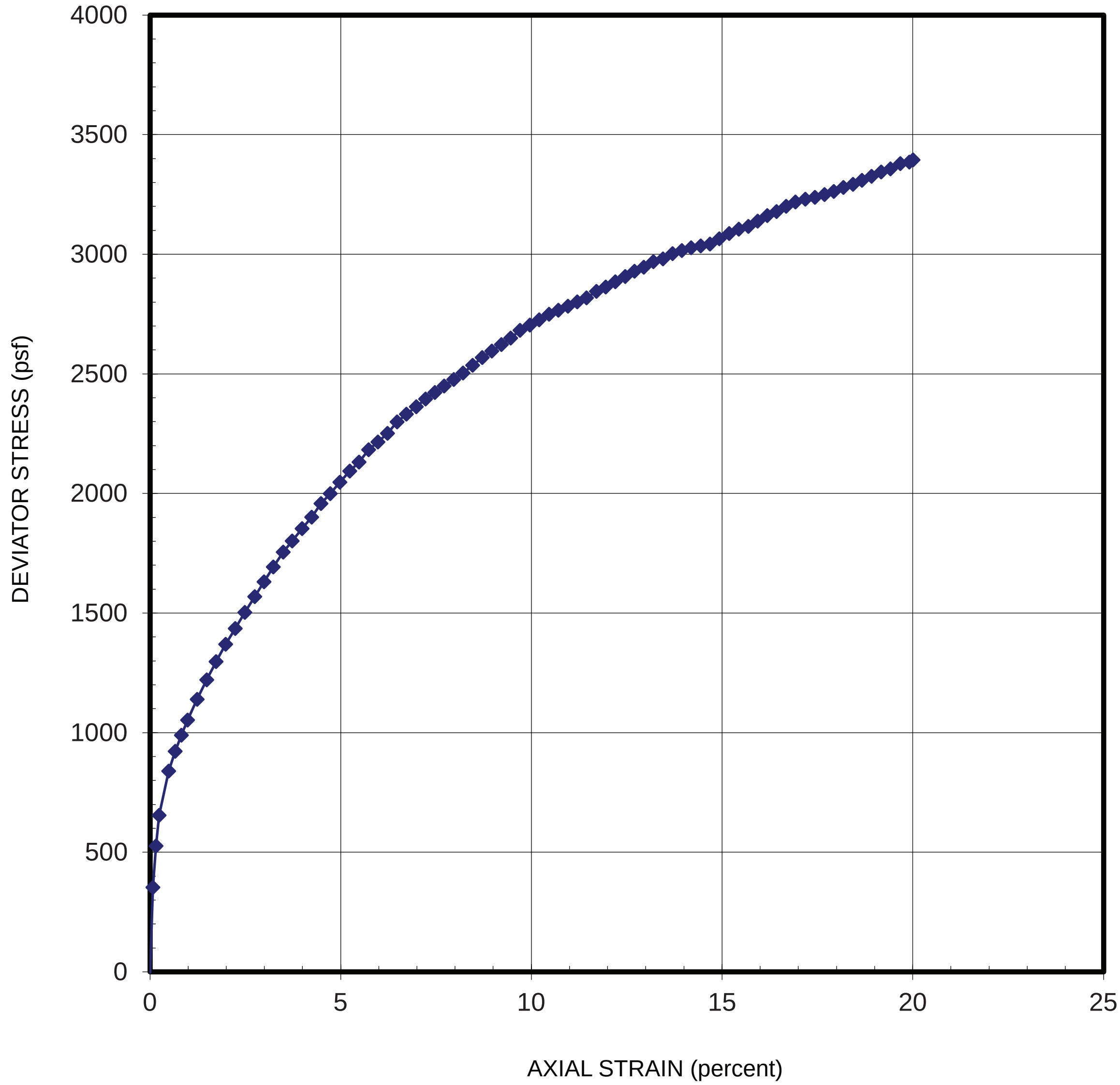
SAMPLER TYPE Modified California		SHEAR STRENGTH	690 psf
DIAMETER (in.) 2.41	HEIGHT (in.) 5.41	STRAIN AT FAILURE	20.0 %
MOISTURE CONTENT	29.3 %	CONFINING PRESSURE	1,000 psf
DRY DENSITY	95 pcf	STRAIN RATE	1 % / min.
DESCRIPTION CLAY with SAND (CL), light brown		SOURCE	B-3 at 8.5 feet

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 01/26/21 | Project No. 20-1950 | Figure B-3



SAMPLER TYPE	Modified California	SHEAR STRENGTH	1,700 psf
DIAMETER (in.) 2.40	HEIGHT (in.) 6.0	STRAIN AT FAILURE	20.0 %
MOISTURE CONTENT	20.9 %	CONFINING PRESSURE	1,600 psf
DRY DENSITY	108 pcf	STRAIN RATE	1 % / min.
DESCRIPTION	SANDY CLAY (CL), light brown	SOURCE	B-4 at 13.5 feet

123 INDEPENDENCE DRIVE
Menlo Park, California

ROCKRIDGE GEOTECHNICAL

UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

Date 01/26/21	Project No. 20-1950	Figure B-4

ROCKRIDGE GEOTECHNICAL

APPENDIX C
Soil Corrosivity Evaluation Report

Soil Corrosivity Evaluation Report for 123-Independence Dr

January 13, 2021

Prepared for:
Quintin Flores
Rockridge Geotechnical, Inc.
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Oakland, CA 94611
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Project X Job #: S210111F
Client Job or PO #: 20-1950

Contents

1 Executive Summary

A corrosion evaluation of the soils at 123 Independence Dr was performed to provide corrosion control recommendations for general construction materials. The site is located at 123 Independence Dr, Menlo Park, CA 94025 (37°29'01.7"N 122°10'35.4"W). Four (4) samples were tested to a depth of 5.8 ft. Site ground water and topography information was provided by Rockridge Geotechnical. Groundwater depth was determined to be 4 feet below finished grade.

Every material has its weakness. Aluminum alloys, galvanized/zinc coatings, and copper alloys do not survive well in very alkaline or very acidic pH environments. Copper and brasses do not survive well in high nitrate or ammonia environments. Steels and irons do not survive well in low soil resistivity and high chloride environments. High chloride environments can even overcome and attack steel encased in normally protective concrete. Concrete does not survive well in high sulfate environments. And nothing survives well in high sulfide and low redox potential environments with corrosive bacteria. This is why Project X tests for these 8 factors to determine a soil's corrosivity towards various construction materials. **Depending solely on soil resistivity or Caltrans corrosion guidelines (which concentrate on concrete/steel highways), will over-simplify descriptions as corrosive or non-corrosive. This approach will not detect these other factors attacking other metals because it is possible to have bad levels of corrosive ions and still have greater than 1,100 ohm-cm soil resistivity. We have observed this fact on thousands of soil samples tested in our laboratory.**

It should not be forgotten that import soil should also be tested for all factors to avoid making your site more corrosive than it was to begin with.

The recommendations outlined herein are not a substitute for any design documents previously prepared for the purpose of construction and apply only to the depth of samples collected.

Soil samples were tested for minimum resistivity, pH, chlorides, sulfates, ammonia, nitrates, sulfides and redox. 6Full

As-Received soil resistivities ranged between 556 ohm-cm and 737 ohm-cm. This data would be similar to a Wenner 4 pin test in the field and used in the design of a cathodic protection or grounding bed system. This resistivity can change seasonally depending on the weather and moisture in the ground. This reading alone can be misleading because condensation or minor water leaks will occur underground along pipe surfaces creating a saturated soil environment in the trench on infrastructure surfaces. This is why minimum or saturated soil resistivity measurements are more important than as-received resistivities.

Saturated soil resistivities ranged between 509 ohm-cm to 670 ohm-cm. The worst of these values is considered to be severely corrosive to general metals.

PH levels ranged between 8.6 to 8.9 pH. The average pH of these samples is alkaline and can cause accelerated corrosion of copper and aluminum alloys.

Chlorides ranged between 29 mg/kg to 219 mg/kg. Chloride levels in these samples are low and may cause insignificant corrosion of metals.

Sulfates ranged between 68 mg/kg to 185 mg/kg. Sulfate levels in these samples are negligible for corrosion of cement. Any type of cement can be used that does not contain encased metal.

Ammonia ranged between 48.2 mg/kg to 87.2 mg/kg. Nitrates ranged between 0.7 mg/kg to 2.5 mg/kg. Concentrations of these elements were high enough to cause accelerated corrosion of copper and copper alloys such as brass.

Sulfides presence was determined to be negative. REDOX ranged between + 134 mV to + 156 mV. The probability of corrosive bacteria was determined to be low due to the sulfide and positive REDOX levels determined in these samples.

2 Corrosion Control Recommendations

The following recommendations are based upon the results of soil testing.

2.1 Cement

The highest reading for sulfates was 185 mg/kg or 0.0 percent by weight.

Per ACI 318-14, Table 19.3.1.1, sulfate levels in these samples categorized as S0 and are negligible for corrosion of metals and cement. Per ACI 318-14 Table 19.3.2.1 any type of cement not containing steel or other metal can be used.

2.2 Steel Reinforced Cement/ Cement Mortar Lined & Coated (CML&C)

Chlorides in soil can overcome the corrosion inhibiting property of cement for steel, as it can also break through passivated surfaces of aluminum and stainless steels.[1,2] The highest concentration of chlorides was 219 mg/kg.

Chloride levels in these samples are not significantly corrosive to metals not in tension. Standard cement cover may be used in these soils.

Though soils at some locations are significantly corrosive to various metals, per ACI 318-14 Chapter 19 Table 19.3.1.1, all slabs on this site exposure categories and class for **Corrosion Protection of Reinforcement (C) would be considered C1** as Concrete exposed to moisture [mud/rain] (slab sides and bottom) but not to an external source of chlorides. Though there are chlorides in the soil, ACI 318's definition of "external source of chlorides" consists of deicing chemicals, salt, brackish water, seawater, or spray from these sources. The chloride levels in seawater are typically over 19,000 mg/L or 19,000 ppm.

When concrete is tested for water-soluble chloride ion content, the tests should be made at an age of 28 to 42 days. The limits in Table 5.3.2.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.[3]

[1] Design Manual 303: Cement Cylinder Pipe. Ameron. p.65
[2] *Chapter 19, Table 1904.2.2(1), 2012 International Building Code*

[3] ACI 381-14., BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI 318-14) AND COMMENTARY (ACI 318R-14)

2.3 Stainless Steel Pipe/Conduit/Fittings

Stainless steels derive their corrosion resistance from their chromium content and oxide layer which needs oxygen to regenerate if damaged. Thus stainless steel is not good for deep soil applications where oxygen levels are extremely low. Stainless steels should not be installed deeper than a plant root zone. Stainless steels typically have the same nobility as copper on the galvanic series and can be connected to copper. If stainless steel must be used, it must be backfilled with soil having greater than 10,000 ohm-cm resistivity and excellent drainage. 304 Stainless steel will also corrode if in contact with carbon materials such as activated carbon. Stainless steel welds should be pickled.

The soil at this site has low probability for anaerobic corrosive bacteria and moderate chloride levels. Per Nickel Institute guidelines, 316 Stainless steels should only be used in these soils.

2.4 Steel Post Tensioning Systems

The proper sealing of stressing holes is of utmost importance in PT Systems. Cut off excess strand 1/2" to 3/4" back in the hole. Coat or paint exposed anchorage, grippers, and stub of strands with "Rust-o-leum" or equal. After tendons have been coated, the cement contractor shall dry pack blockouts within ten (10) days. A non-shrink, non-metallic, non-porous moisture-insensitive grout (Master EMACO S 488 or equivalent), or epoxy grout shall be used for this purpose. If an encapsulated post-tension system is used, regular non-shrink grout can be used.

Due to the low chloride concentrations measured on samples obtained from this site, post-tensioned slabs should be protected in accordance with soil considered normal (non-corrosive).[4,5] Addition of grease caps to the cut strand at live end anchors can deter construction defect accusations but are not needed.

2.5 Steel Piles

Steel piles are most susceptible to corrosion in disturbed soil where oxygen is available. Further, a dissimilar environment corrosion cell would exist between the steel embedded in cement, such as pile caps and the steel in the soil. In the cell, the steel in the soil is the anode (corroding metal), and the steel in cement is the cathode (protected metal). This cell can be minimized by coating the part of the steel piles that will be embedded in cement to prevent contact with cement and reinforcing steel.

Piles driven into soils without disturbing soils will avoid oxygen introduction and low corrosion rates unless there is a probability for corrosive anaerobic bacteria. Galvanized steel's zinc coating can provide significant protection for driven piles. In corrosive soils in which normal zinc coatings are not enough, the life of piles can be extended by increasing zinc coating thickness, using sacrificial metal, or providing a combination of epoxy coatings and cathodic protection. Corrosion has been observed to be extremely localized even at and below underground water tables. Pit depths of this magnitude do not have an appreciable effect on the strength or useful life of piling structures because the reduction in pile cross section is not

[4] *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, PTI DC10.5-12,Table 4.1, pg 16*
[5] *Specification for Unbonded Single Strand Tendons. Post-tensioning Institute (PTI), Phoenix, AZ, 2000.*

significant.[6] Pitting is of more importance to pipes transporting liquids or gases which should not be leaked into the ground.

The following recommendations are recommended to achieve desired life. We defer to structural engineers to use our estimated corrosion rates and to choose from the corrosion control options listed below.

1) Sacrificial metal by use of thicker piles per non-disturbed soil corrosion rates, or
2) Galvanized steel piles per non-disturbed soil corrosion rates, or
3) Combination of galvanized and sacrificial metal per non-disturbed soil corrosion rates, or
4) For no loss of metal, coat entire pile with abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent, or
5) Use high yield steel which will corrode at the same rate as mild steel but have greater yield strength and thus be able to suffer more material loss than mild steel.

2.5.1 Expected Corrosion Rate of Steel and Zinc in disturbed soil

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils will also be considered disturbed simply because of their nature from dry to wet seasons.

In Melvin Romanoff's NBS Circular 579, the corrosion rates of carbon steels and various metals was studied over long term periods. Various metals were placed in various soil types to gather corrosion rate data of all metals in all soil types. Samples were collected and material loss measured over the course of 20 years in some sites. The following corrosion rates were estimated by comparing the worst results of soils tested with similar soils in Romanoff's studies and Highway Research Board's publications.[7] The corrosion rate of zinc in disturbed soils is determined per Romanoff studies and King Nomograph.[8]

Expected Corrosion Rate for Steel = 2.54 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 1.68 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

In undisturbed soils, a corrosion rate of 1 mil/year for steel is expected with little change in the corrosion rate of zinc due to it's low nobility in the galvanic series.

Per CTM 643: Years to perforation of corrugated galvanized steel culverts

- 20.5 Years to Perforation for a 18 gage metal culvert
- 26.6 Years to Perforation for a 16 gage metal culvert
- 32.8 Years to Perforation for a 14 gage metal culvert

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[6] Melvin Romanoff, Corrosion of Steel Pilings in Soils, National Bureau of Standards Monograph 58, pg 20.

[7] Field test for Estimating Service Life of Corrugated Metal Culverts, J.L. Beaton, Proc. Highway Research Board, Vol 41, P. 255, 1962

[8] King, R.A. 1977, Corrosion Nomograph, TRRC Supplementary Report, British Corrosion Journal

- 45.0 Years to Perforation for a 12 gage metal culvert
- 57.3 Years to Perforation for a 10 gage metal culvert
- 69.6 Years to Perforation for a 8 gage metal culvert

2.5.2 ***Expected Corrosion Rate of Steel and Zinc in Undisturbed soil***

Expected Corrosion Rate for Steel = 1 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 1.68 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

2.6 Steel Storage tanks

Underground fuel tanks must be constructed and protected in accordance with California Underground Storage Tank Regulations, CCR, Title 23, Division 3, Chapter 16. Metals should be protected with cathodic protection or isolated from backfill material with an epoxy coating.

2.7 Steel Pipelines

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286. **These are especially important for fire risers.**

The corrosivity at this site is corrosive to steel. Any piping that must be jack bored should use abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent. The corrosion control options for this site are as follows:

1) Wax tape per AWWA C217, or
2) Coal tar enamel per AWWA C203, or
3) Fusion bonded epoxy per AWWA C213
4) And install cathodic protection system per NACE SP0169.

   Or instead of CP and Dielectric coating

5) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the pipe that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.8 Steel Fittings

The corrosivity at this site is very corrosive to steel. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) And install cathodic protection system per NACE SP0169.

   Or instead of CP and Dielectric coating

7) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.9 Ductile Iron (DI) Fittings

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 14 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials.

The corrosivity at this site is very corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Wax tape per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213
5) And install cathodic protection system per NACE SP0169.

   Or instead of CP and Dielectric coating

6) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.10 Ductile Iron Pipe

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 14 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials.

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection. **If using thermite, perform one test bond using a half-charge then pressure test to confirm excess heat and pinholes were not created.**

Pea gravel is used by plumbers to lay pipes and establish slopes. If the gravel has more than 200 ppm chlorides or is not tested, a 25 mil plastic should be placed between the gravel and pipe to avoid corrosion.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286**. These are especially important for fire risers.**

The corrosivity at this site is corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Wax tape per AWWA C217, or
3) Coal tar enamel per AWWA C203, or
4) Fusion bonded epoxy per AWWA C213
5) And install cathodic protection system per NACE SP0169.

Or instead of CP and Dielectric coating

6) Apply 3 inch coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11 Copper Materials

Copper is an amphoteric material which is susceptible to corrosion at very high and very low pH. It is one of the most noble metals used in construction thus typically making it a cathode when connected to dissimilar metals. Copper's nobility can change with temperature, similar to the phenomenon in zinc. When zinc is at room temperature, it is less noble than steel and can provide cathodic protection to steel. But when zinc is at a temperature above 140F such as in a water heater, it becomes more noble than the steel and the steel becomes the sacrificial anode. This is why zinc is not used in steel water heaters or boilers. Cold copper has one native potential, but when heated it develops a more electronegative electro-potential aka open circuit potential. Thus hot and cold copper pipes should be electrically isolated from each other to avoid creation of a thermo-galvanic corrosion cell.

2.11.1 Copper Pipes

The lowest pH for this area was measured to be 8.6. Copper is greatly affected by pH, ammonia and nitrate concentrations[9]. The highest nitrate concentration was 2.5 mg/kg and the highest ammonia concentration was 87.2 mg/kg at this site.

These soils were determined to be corrosive to copper and copper alloys such as brass.

Aboveground, underground, cold water and hot water pipes should be electrically isolated from each other by use of dielectric unions and plastic in-wall pipe supports per NACE SP0286. The following are corrosion control options for underground copper water pipes.

1) Run copper pipes within PVC pipes to prevent soil contact, or
2) Cover piping with a 20 mil epoxy coating, or 8-mil polyethylene sleeve, or encase in double 4-mil thick polyethylene sleeves free of scratches and defects then backfill with clean sand with 2 inch minimum cover above and below tubing. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm

[9] Corrosion Data Handbook, Table 6, Corrosion Resistance of copper alloys to various environments, 1995

3) Cover copper pipes with minimum 8 mil polyethylene sleeve or incase in double 4-mil thick polyethylene sleeves over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.2 Brass Fittings

Brass fittings should be electrically isolated from dissimilar metals by use of dielectric unions or isolation joint kits per NACE SP0286.

These soils were determined to be corrosive to copper and copper alloys such as brass.

The following are corrosion control options for underground brass.

1) Prevent soil contact by use of impermeable coating system such as wax tape, or
2) Prevent soil contact by use of a 20 mil epoxy coating free of scratches and defects and backfill with clean sand with 4 inch minimum cover above and below brass. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm, or
3) Cover brass with minimum 10 mil polyethylene sleeve over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.3 Bare Copper Grounding Wire

It is assumed that corrosion will occur at all sides of the bare wire, thus the corrosion rate is calculated as a two sided attack determining the time it takes for the corrosion from two sides to meet at the center of the wire. The estimated life of bare copper wire for this site is the following:[10]

Size (AWG)	**Diameter (mils)**	**Est. Time to penetration (Yrs)**
14	64.1	5.5
13	72	6.2
12	80.8	7.0

[10] Soil-Corrosion studies 1946 and 1948: Copper Alloys, Lead, and Zinc, Melvin Romanoff, National Bureau of Standards, Research Paper RP2077, 1950

Size (AWG)	**Diameter (mils)**	**Est. Time to penetration (Yrs)**
11	90.7	7.8
10	101.9	8.8
9	114.4	9.9
8	128.5	11.1
7	144.3	12.4
6	162	14.0
5	181.9	15.7
4	204.3	17.6
3	229.4	19.8
2	257.6	22.2
1	289.3	24.9

If the bare copper wire is being used as a grounding wire connected to less noble metals such as galvanized steel or carbon steel, the less noble metals will provide additional cathodic protection to the copper reducing the corrosion rate of the copper.

It is recommended that a corrosion inhibiting and water-repelling coating be applied to aboveground and belowground copper-to-dissimilar metal connections to reduce risk of dissimilar corrosion. This can be wax tape, or other epoxy coating.

Tinned copper wiring or laying copper wire in conductive concrete can protect against chemical attack in soils with high nitrates, ammonia, sulfide and severely low soil electrical resistivity.

2.12 Aluminum Pipe/Conduit/Fittings

Aluminum is an amphoteric material prone to pitting corrosion in environments that are very acidic or very alkaline or high in chlorides.

Conditions at this site are unsafe for aluminum. Soils at this site were determined to be too alkaline for aluminum. Soil contact with aluminum alloys should be avoided at this site. This can be achieved with:

1) Impermeable minimum 20 mil polyethylene coatings, or
2) Epoxy coatings with minimum 20 mil thickness free of scratches and defects, or
3) Wax tape

Aluminum derives its corrosion resistance from its oxide layer which needs oxygen to regenerate if damaged, similar to stainless steels. Thus aluminum is not good for deep soil applications. Since aluminum corrodes at very alkaline environments, it cannot be encased or placed against cement or mortar such as brick wall mortar up against an aluminum window frame.

Aluminum is also very low on the galvanic series scale making it most likely to become a sacrificial anode when in contact with dissimilar metals in moist environments. Avoid electrical continuity with dissimilar metals by use of insulators, dielectric unions, or isolation joints per NACE SP0286. Pooling of water at post bottoms or surfaces should be avoided by integrating good drainage.

2.13 Carbon Fiber or Graphite Materials

Carbon fiber or other graphite materials are extremely noble on the galvanic series and should always be electrically isolated from dissimilar metals. They can conduct electricity and will create corrosion cells if placed in contact within a moist environment with any metal.

2.14 Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping from a corrosion viewpoint.

Protect all metallic fittings and pipe restraining joints with wax tape per AWWA C217, cement if previously recommended, or epoxy.

3 CLOSURE

In addition to soils chemistry and resistivity, another contributing influence to the corrosion of buried metallic structures is stray electrical currents. These electrical currents flowing through the earth originate from buried electrical systems, grounding of electrical systems in residences, commercial buildings, and from high voltage overhead power grids. Therefore, it is imperative that the application of protective wraps and/or coatings and electrical isolation joints be properly applied and inspected.

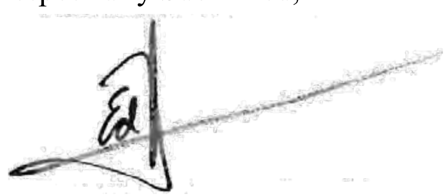
It is the responsibility of the builder and/or contractor to closely monitor the installation of such materials requiring protection in order to assure that the protective wraps or coatings are not damaged.

The recommendations outlined herein are in conformance with current accepted standards of practice that meet or exceed the provisions of the Uniform Building Code (UBC), the International Building Code (IBC), California Building Code (CBC), the American Cement Institute (ACI), Nickel Institute, National Association of Corrosion Engineers (NACE International), Post-Tensioning Institute Guide Specifications and State of California Department of Transportation, Standard Specifications, American Water Works Association (AWWA) and the Ductile Iron Pipe Research Association (DIPRA).

Our services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,



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NACE Corrosion Technologist #16592
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4 SOIL ANALYSIS LAB RESULTS

Client: Rockridge Geotechnical, Inc.
Job Name: 123-Independence Dr
Client Job Number: 20-1950
Project X Job Number: S210111F
January 13, 2021

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-1-2B Slay (CL) olive to olive-gray/sandy clay(cl) Yellow Brown	5.75	68.0	0.0068	62.5	0.0062	737	670	8.9	144	<0.01	2.0	55.1	ND	222.8	1.1	48.8	58.5	6.6	3.2
B-2-1A Clay (CH) Dark brown	3	78.8	0.0079	29.4	0.0029	643	630	8.6	134	<0.01	0.7	62.5	0.0	184.3	1.5	31.3	28.6	13.6	10.8
B-4 Clay with sand (CH): Gray Brown	5	70.8	0.0071	112.7	0.0113	670	657	8.7	156	<0.01	1.2	48.2	0.0	158.6	0.3	23.2	29.0	7.6	1.0
B-6-1B Clay(CH): Dark Brown	3.5	184.6	0.0185	218.9	0.0219	556	509	8.8	141	<0.01	2.5	87.2	0.0	391.2	1.7	29.4	26.6	10.9	0.3

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract
Anions and Cations tested via Ion Chromatograph except Sulfide.

Figure 1- Soil Sample Locations, 123 Independence Dr, Menlo Park, CA 94025

(37°29'01.7"N 122°10'35.4"W)

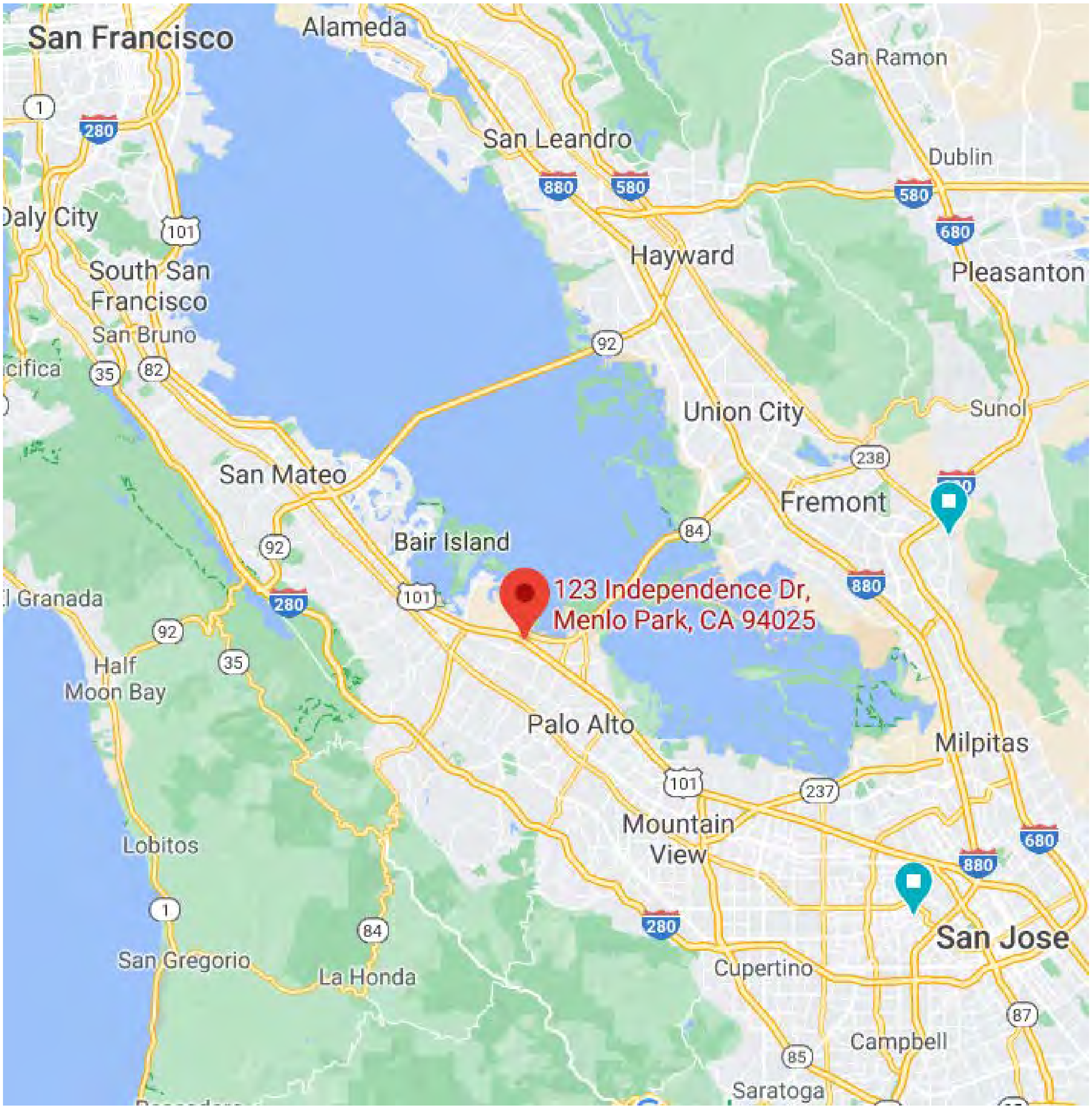


Figure 2 -Vicinity Map, 123 Independence Dr, Menlo Park, CA 94025
(37°29'01.7"N 122°10'35.4"W)

5 Corrosion Basics

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Oxygen content in soil can be increased during construction. These soils are considered disturbed soils. When construction equipment at a site is simply driving piles into soil without digging into the soil, the activity can still disturb soil down to 3 feet. Expansive soils will also be considered disturbed simply because of their nature from dry to wet seasons.

5.1 Pourbaix Diagram – In regards to a material's environment

All metals are unique and have a weakness. Some metals do not like acidic (low pH) environments. Some metals do not like alkaline (high pH) environments. Some metals don't like either high or low pH environments such as aluminum. These are called amphoteric materials. Some metals become passivated and do not corrode at high pH environments such as steel. These characteristics are documented in Marcel Pourbaix's book "Atlas of electrochemical equilibria in aqueous solutions"

In the mid 1900's, Marcel Pourbaix developed the Pourbaix diagram which describes a metal's reaction to an environment dependent on pH and voltage conditions. It describes when a metal remains passive (non-corroding) and in which conditions metals become soluble (corrode). Steels are passive in pH over 12 such as the condition when it is encased in cement. If the cement were to carbonate and its pH reduce to below 12, the cement would no longer be able to act as a corrosion inhibitor and the steel will begin to corrode when moist.

Some metals such as aluminum are amphoteric, meaning that they react with acids and bases. They can corrode in low pH and in high pH conditions. Aluminum alloys are generally passive within a pH of 4 and 8.5 but will corrode outside of those ranges. This is why aluminum cannot be embedded in cement and why brick mortar should not be laid against an aluminum window frame without a protective barrier between them.

5.2 Galvanic Series – In regards to dissimilar metal connections

All metals have a natural electrical potential. This electrical potential is measured using a high impedance voltmeter connected to the metal being tested and with the common lead connected to a copper copper-sulfate reference electrode (CSE) in water or soil. There are many types of reference electrodes. In laboratory measurements, a Standard Hydrogen Electrode (SHE) is commonly used. When different metal alloys are tested they can be ranked into an order from most noble (less corrosion), to least noble (more active corrosion). When a more noble metal is connected to a less noble metal, the less noble metal will become an anode and sacrifice itself through corrosion providing corrosion protection to the more noble metal. This hierarchy is known as the galvanic series named after Luigi Galvani whose experiments with electricity and muscles led Alessandro Volta to discover the reactions between dissimilar metals leading to the early battery. The greater the voltage difference between two metals, the faster the corrosion rate will be.

Table 1- Dissimilar Metal Corrosion Risk

	Zinc	Galvanized Steel	Aluminum	Cast Iron	Lead	Mild Steel	Tin	Copper	Stainless Steel
Zinc	None	Low	Medium	High	High	High	High	High	High
Galvanized Steel	Low	None	Medium	Medium	Medium	High	High	High	High
Aluminum	Medium	Medium	None	Medium	Medium	Medium	Medium	High	High
Cast Iron	High	Medium	Medium	None	Low	Low	Low	Medium	Medium
Lead	High	Medium	Medium	Low	None	Low	Low	Medium	Medium
Mild Steel	High	High	Medium	Low	Low	None	Low	Medium	Medium
Tin	High	High	Medium	Low	Low	Low	None	Medium	Medium
Copper	High	High	High	Medium	Medium	Medium	Medium	None	Low
Stainless Steel	High	High	High	Medium	Medium	Medium	Medium	Low	None

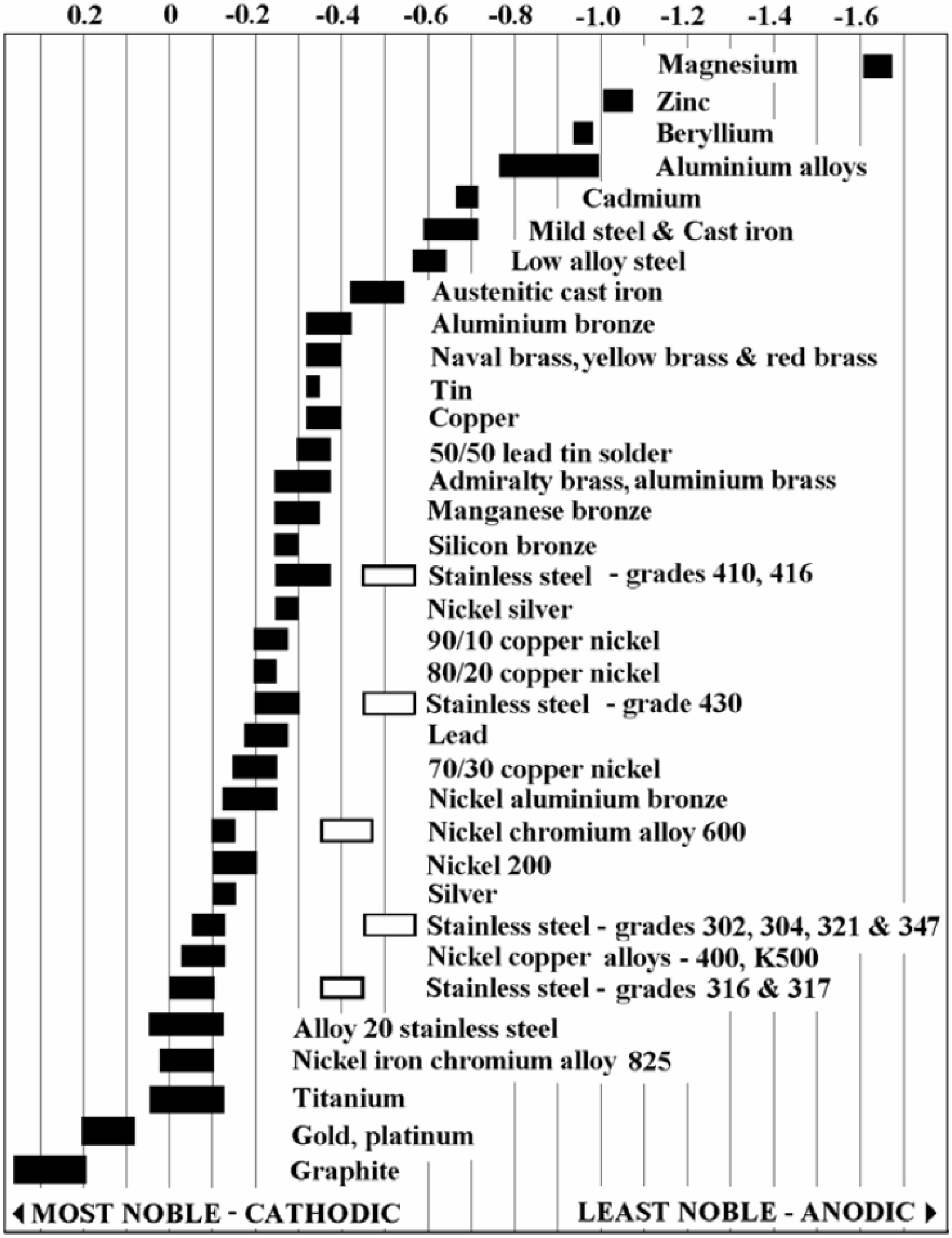
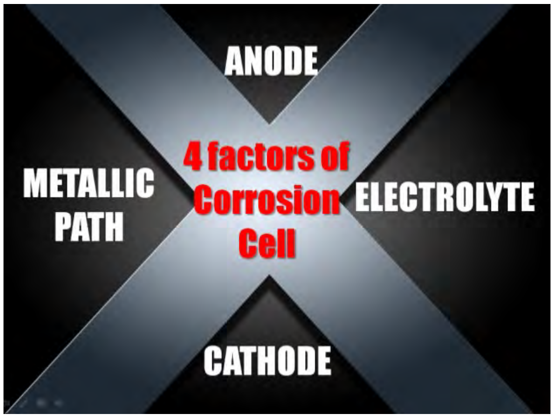
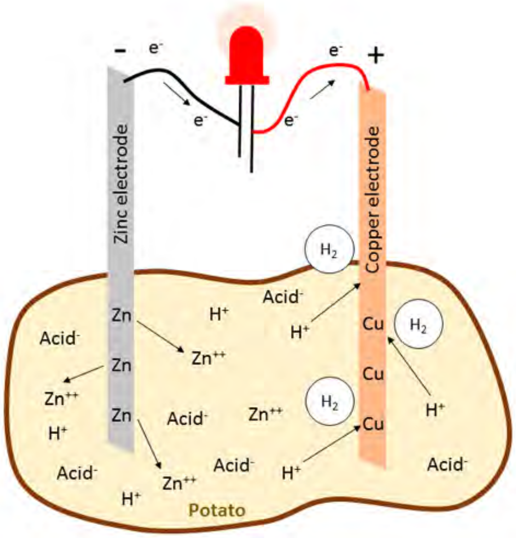


Figure 3 - Galvanic series of metals relative to CSE half cell.

5.3 Corrosion Cell



In order for corrosion to occur, four factors must be present. (1) The anode (2) the cathode (3) the electrolyte and (4) the metallic or conductive path joining the anode and the cathode. If any one of these is removed, corrosion activity will stop. This is how a simple battery produces electricity. An example of a non-metallic yet conductive material is graphite. Graphite is similar in nobility to gold. Do not connect graphite to anything in moist environments.



The anode is where the corrosion occurs, and the cathode is the corrosion free material. Sometimes the anode and cathode are different materials connected by a wire or union. Sometimes the anode and cathode are on the same pipe with one area of the pipe in a low oxygen zone while the other part of the pipe is in a high oxygen zone. A good example of this is a post in the ocean that is repeatedly splashed. Deep underwater, corrosion is minimal, but at the splash zone, the corrosion rate is greatest.

Low oxygen zones and crevices can also harbor corrosive bacteria which in moist environments will lead to corrosion. This is why pipes are laid on backfill instead of directly on native cut soil in a trench. Filling a trench slightly with backfill before installing pipe then finishing the backfill creates a uniform environment around the entire surface of the pipe.

The electrolyte is generally water, seawater, or moist soil which allows for the transfer of ions and electrical current. Pure water itself is not very conductive. It is when salts and minerals dissolve into pure water that it becomes a good conductor of electricity and chemical reactions. Metal ores are turned into metal alloys which we use in construction. They naturally want to return to their natural metal ore state but it requires energy to return to it. The corrosion cell, creates the energy needed to return a metal to its natural ore state.

The metallic or conductive path can be a wire or coupling. Examples are steel threaded into a copper joint, or an electrician grounding equipment to steel pipes inadvertently connecting electrical grid copper grounding systems to steel or iron underground pipes.

The ratio of surface area between the anode and the cathode is very important. If the anode is very large, and the cathode is very small, then the corrosion rate will be very small and the anode may live a long life. An example of this is when short copper laterals were connected to a large and long steel pipeline. The steel had plenty of surface area to spread the copper's attack, thus corrosion was not noticeable. But if the copper was the large pipe and the steel the short laterals, the steel would corrode at an amazing rate.

5.4 Design Considerations to Avoid Corrosion

The following recommendations are based upon typical observations and conclusions made by forensic engineers in construction defect lawsuits and NACE International (Corrosion Society) recommendations.

5.4.1 Testing Soil Factors (Resistivity, pH, REDOX, SO, CL, NO3, NH3)

As previously mentioned, different factors can cause corrosion. The most useful and common test for categorizing a soil's corrosivity has been the measure of soil resistivity which is typically measured in units of (ohm-cm) by corrosion engineers and geologists. Soil resistivity is the ability of soil to conduct or resist electrical currents and ion transfer. The lower the soil resistivity, the more conductive and corrosive it is. The following are "generally" accepted categories but keep in mind, the question is not "Is my soil corrosive?", the question should be, "What is my soil corrosive to?" and to answer that question, soil resistivity and chemistry must be tested. Though **soil resistivity is a good corrosivity indicator for steel materials, high chlorides or other corrosive elements do not always lower soil resistivity, thus if you don't test for chlorides and other water soluble salts, you can get an unpleasant surprise.** The largest contributing factor to a soil's electrical resistivity is its clay, mineral, metal, or sand make-up.

Table 2 - Corrosion Basics- An Introduction, NACE, 1984, pg 191

(Ohm-cm)	Corrosivity Description
0-500	Very Corrosive
500-1,000	Corrosive
1,000-2,000	Moderately Corrosive
2,000-10,000	Mildly Corrosive
Above 10,000	Progressively less corrosive

Testing a soil's pH provides information to reference the Pourbaix diagram of specific metals. Some elements such as ammonia and nitrates can create localized alkaline conditions which will greatly affect amphoteric materials such as aluminum and copper alloys.

Excess sulfates can break-down the structural integrity of cement and high concentrations of chlorides can overcome cement's corrosion inhibiting effect on encased ferrous metals and break down protective passivated surface layers on stainless steels and aluminum.

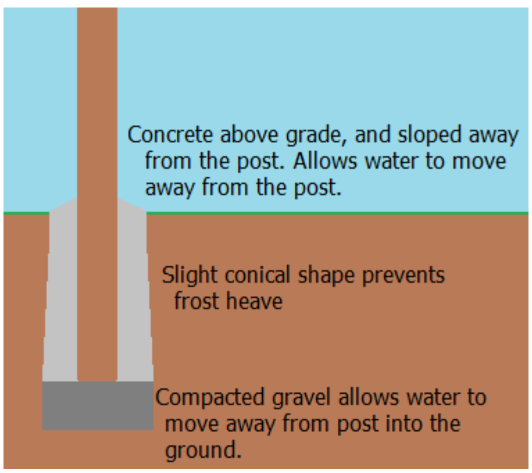
Corrosive bacteria are everywhere but can multiply significantly in anaerobic conditions with plentiful sulfates. The bacteria themselves do not eat the metal but their by-products can form corrosive sulfuric acids. The probability of corrosive bacteria is tested by measuring a soil's oxidation-reduction (REDOX) electro-potential and by testing for the presence of sulfides.

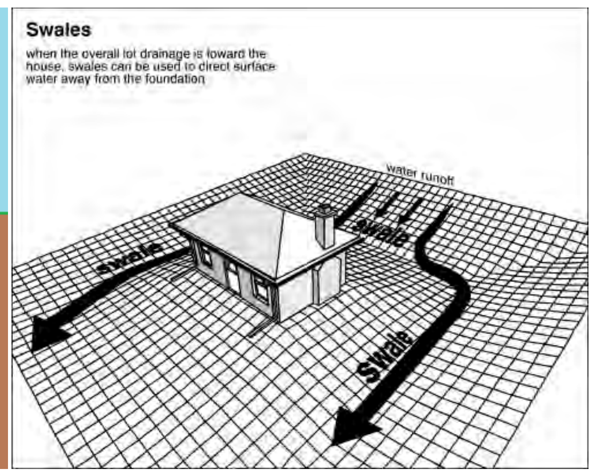
Only by testing a soil's chemistry for minimum resistivity, pH, chlorides, sulfates, sulfides, ammonia, nitrate, and redox potential can one have the information to evaluate the corrosion risk to construction materials such as steel, stainless steel, galvanized steel, iron, copper, brass, aluminum, and concrete.

5.4.2 *Proper Drainage*

It cannot be emphasized enough that pooled stagnant water on metals will eventually lead to corrosion. This stands for internal corrosion and external corrosion situations. In soils, providing good drainage will lower soil moisture content reducing corrosion rates. Attention to properly sealing polyethylene wraps around valves and piping will avoid water intrusion which would allow water to pool against metals. Above ground structures should not have cupped or flat surfaces that will pond water after rain or irrigation events.

Buildings typically are built on pads and have swales when constructed to drain water away from buildings directing it towards an acceptable exit point such as a driveway where it continues draining to a local storm drain. Many homeowners, landscapers and flatwork contractors appear to not be aware of this and destroy swales during remodeling. The majority of garage floor and finished grade elevations are governed by drainage during design. [11,12]



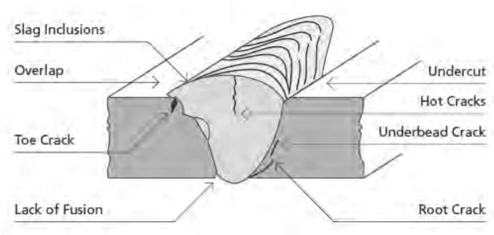


5.4.3 *Avoiding Crevices*

Crevices are excellent locations for oxygen differential induced corrosion cells to begin. Crevices can also harbor corrosive bacteria even in the most chemically treated waters. Crevices will also gather salts. If water's total alkalinity is low, its ability to maintain a stable pH can also become more difficult within a crevice allowing the pH to drop to acidic levels continuing a pitting process. Welds in extremely corrosive environments should be complete and well filleted without sharp edges to avoid crevices. Sharp edges should be avoided to allow uniform coating of protective epoxy. Detection of crevices in welds should be treated immediately. If pressures and loads are low, sanding and rewelding or epoxy patching can be suitable repairs. Damaged coatings can usually be repaired with Direct to Metal paints. **Scratches and crevice corrosion are like infections, they should not be left to fester or the infection will spread making things worse.**

[11] https://www.fencedaddy.com/blogs/tips-and-tricks/132606467-how-to-repair-a-broken-fence-post

[12] http://southdownstudio.co.uk/problme-drainage-maison.html



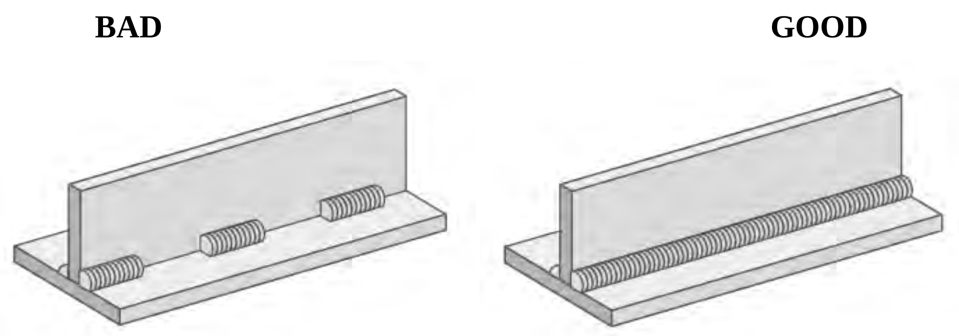


Figure 4 Defects which form weld crevices[13]

5.4.4 Coatings and Cathodic Protection

When faced with a corrosive environment, the best defense against corrosion is removing the electrolyte from the corrosion cell by applying coatings to separate the metal from the soil. During construction and installation, there is always some scratch or damage made to a coating. NACE training recommends that coatings be used as a first line of defense and that sacrificial or impressed current cathodic protection is used as a 2nd line of defense to protect the scratched areas. Use of a good coating dramatically reduces the amount of anodes a CP system would need. If CP is not installed as a 2nd line of defense in an extremely corrosive environment, the small scratched zones will suffer accelerated corrosion. CP details such as anode installation instructions must be designed by corrosion engineers or vessel manufacturers on a per project basis because it depends on electrolyte resistivity, surface area of infrastructure to be protected, and system geometry.

There are two types of cathodic protection systems, a Galvanic Anode Cathodic Protection (GACP) system and an Impressed Current Cathodic Protection (ICCP) system. A Galvanic Anode Cathodic Protection (GACP) system is simpler to install and maintain than an Impressed Current Cathodic Protection (ICCP) system. To protect the metals, they must all be electrically continuous to each other. In a GACP system, sacrificial zinc or magnesium anodes are then buried at locations per the CP design and connected by wire to a structure at various points in system. At the connection points, a wire connecting to the structure and the wire from the anode are joined in a Cathodic Protection Test Station hand hole which looks similar in size and shape to an irrigation valve pull box. By coating the underground structures, one can reduce the number of anodes needed to provide cathodic protection by 80% in many instances.

An ICCP system requires a power source, a rectifier, significantly more trenching, and more expensive type anodes. These systems are typically specified when bare metal is requiring protection

[13] http://www.daroproducts.co.uk/makes-good-weld/

in severely corrosive environments in which galvanic anodes do not provide enough power to polarize infrastructure to -850 mV structure-to-soil potential or be able to create a 100 mV potential shift as required by NACE SP169 to control corrosion. In severely corrosive environments, a GACP system simply may not last a required lifetime due to the high rate of consumption of the sacrificial anodes. ICCP system rectifiers must be inspected and adjusted quarterly or at a minimum bi-annually per NACE recommendations. Different anode installations may be possible but for large sites, anodes are placed evenly throughout the site and all anode wires must be trenched to the rectifier. For a large site, it may be beneficial to use two or more rectifiers to reduce wire lengths or trenching.

To simplify, a GACP system can be installed and practically forgotten with minor trenching because the anodes can be installed very close to the structures. An ICCP system must be inspected annually and anode wires run back to the rectifier which itself connects to the pile system. If any type of trenching or development is expected to occur at the site during the life of the site, it is a good idea to inspect the anode connections once a year to make sure wires are not cut and that the infrastructure is still being provided adequate protection. A common situation that occurs with ICCP systems is that a contractor accidently cuts the wires during construction then reconnects them incorrectly, turning the once cathode, into a sacrificing anode.

Design of a cathodic protection system protecting against soil side corrosion requires that Wenner Four Pin ground resistance measurements per ASTM G57 be performed by corrosion engineers at various locations of the site to determine the best depths and locations for anode installations. Ideally, a sample pile is installed and experiments determining current requirement are conducted. Using this data, the decision is made whether a GACP system is feasible or if an ICCP must be used.

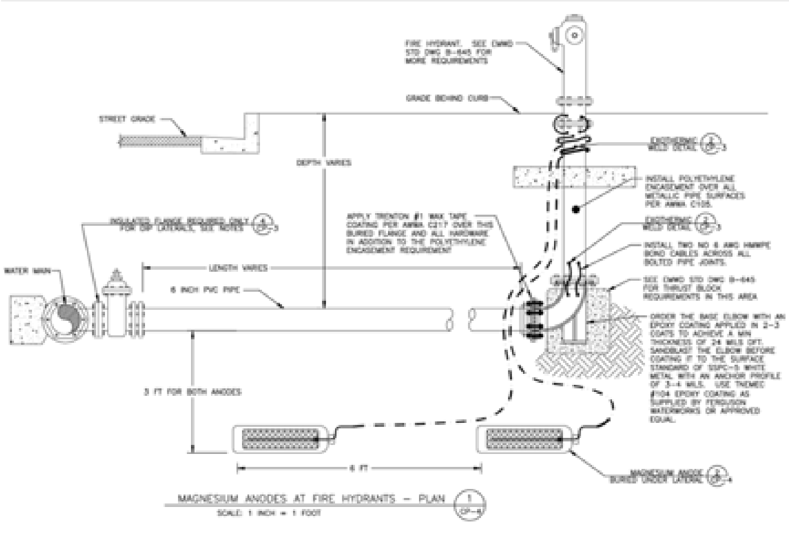


Figure 5 Sample anode design for fire hydrant underground piping

Vessels such as water tanks will have protective interior coatings and anodes to protect the interior surfaces. Anodes can also be buried on site and connected to system skid supports to protect the metal in contact with soil. A good example of a vessel cathodic protection system exists in all home water heaters which contain sacrificial aluminum or magnesium anodes. In environments that exceed 140F, zinc anodes cannot be used with carbon steel because they become the aggressor (Cathodic) to the steel instead of sacrificial (anodic). Anodes in vessels containing extremely brackish water with chloride levels over 2,000 ppm should inspect or change out their anodes every 6 months.

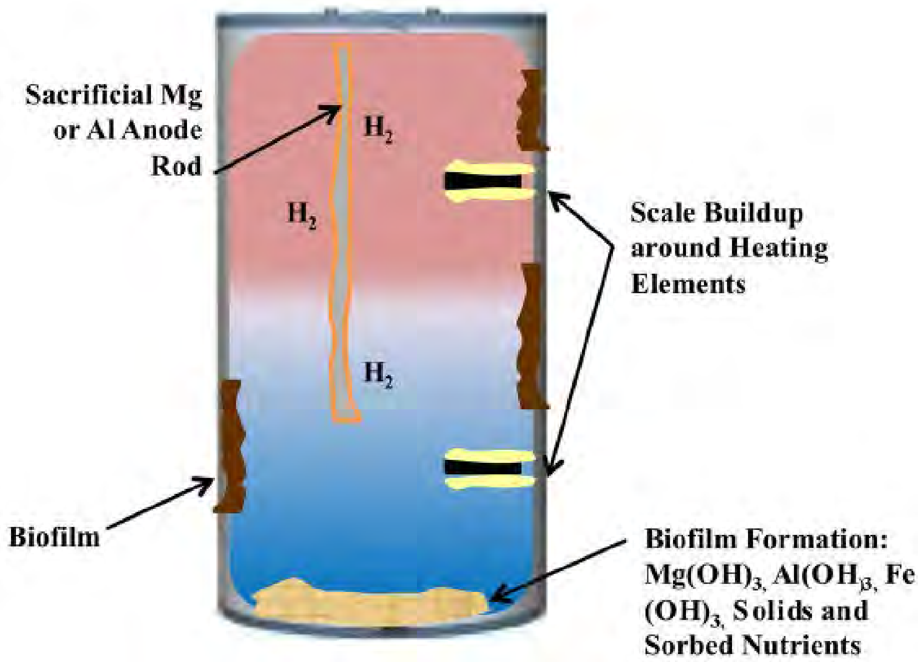


Figure 6 Cross section of boiler with anode

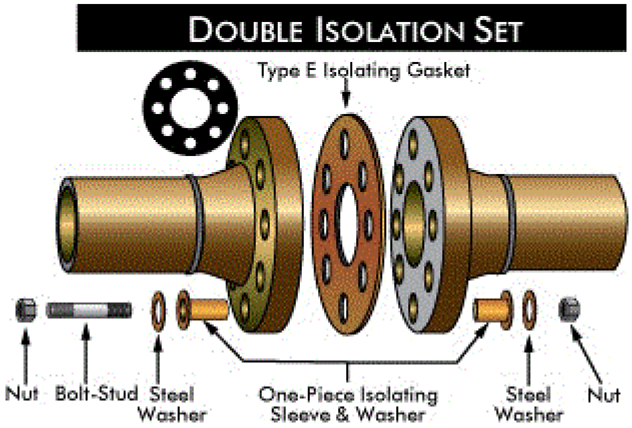
Cathodic protection can only protect a few diameters within a pipeline thus it is not recommended for small diameter pipelines and tubing internal corrosion protection. Anodes are like a lamp shining light in a room. They can only protect along their line of sight.

5.4.5 Good Electrical Continuity

In order for cathodic protection to protect a long pipeline or system of pipes from external soil side corrosion, they must all be electrically continuous to each other so that the electric current from the anode can travel along the pipes, then return through the earth to the anode. Electrical continuity is achieved by welding or pin brazing #8 AWG copper strand bond cable to the end of pipe sticks which have rubber gaskets at bell and spigots. If steel pipes are joined by full weld, bonding wires are not needed.

Electrical continuity between dissimilar metals is not desirable. Isolation joints or di-electric unions should be installed between dissimilar metals, such as steel pipes connecting to a brass valve per NACE SP0286. Bonding wires should then be welded onto the steel pipes by-passing the brass valve so that the cathodic protection system's current can continue to travel along the steel piping but isolate the brass valve from the steel pipeline. Another option would be to provide a separate cathodic protection system for steel pipes on both sides of the brass valve.

Typically, water heater inlets and outlets, gas meters and water meters have dielectric unions installed in them to separate utility property from homeowner property. This also protects them in the case that a home owner somehow electrically connects water pipes or gas pipes to a neighborhood electrical grounding system which can potentially have less noble steel in soil now connected to much more noble copper in soil which will then create a corrosion cell. This is exactly how a lemon powered clock works when a galvanized zinc nail and a steel nail are inserted into a lemon then connected to a clock. The clock is powered by the corrosion cell created.



5.4.6 Bad Electrical Continuity

Bad electrical continuity is when two different materials or systems are made electrically continuous (aka shorted) when they were not designed to be electrically continuous. Examples of this would be when gas lines are shorted to water lines or to electrical grounding beds. Very often, fire risers are shorted to electrical grounding systems, and water pipes at business parks. Since fire risers usually have a very short ductile iron pipe in the ground which connects to PVC pipe systems, they tend to experience leaks after 7 to 10 years of being attacked by underground copper systems.

It is absolutely imperative that any copper water piping or other metal conduits penetrating cement slab or footings, not come in contact with the reinforcing steel or post-tensioning tendons to avoid creation of galvanic corrosion cells.

5.4.7 Corrosion Test Stations

Corrosion test stations should be installed every 1,000 feet along pipelines in order to measure corrosion activity in the future. For a simple pipeline, two #8 AWG copper strand bond cable welded or pin brazed onto the pipeline are run up to finished grade and left in a hand hole. Corrosion test stations are used to measure pipe-to-soil electro potential relative to a copper copper-sulfate reference electrode to determine if the pipe is experiencing significant corrosion activity. By measuring test stations along a pipeline, hot spots can be determined, if any. The wires also allow for electrical continuity testing, condition assessment, and a multitude of other types of tests.

At isolation joints and pipe casings, two wires should be welded to either side of the isolation joint for a total of 4 wires to be brought up to the hand hole. This allows for future tests of the isolation joint, casing separation confirmation, and pipe-to-soil potential readings during corrosion surveys.

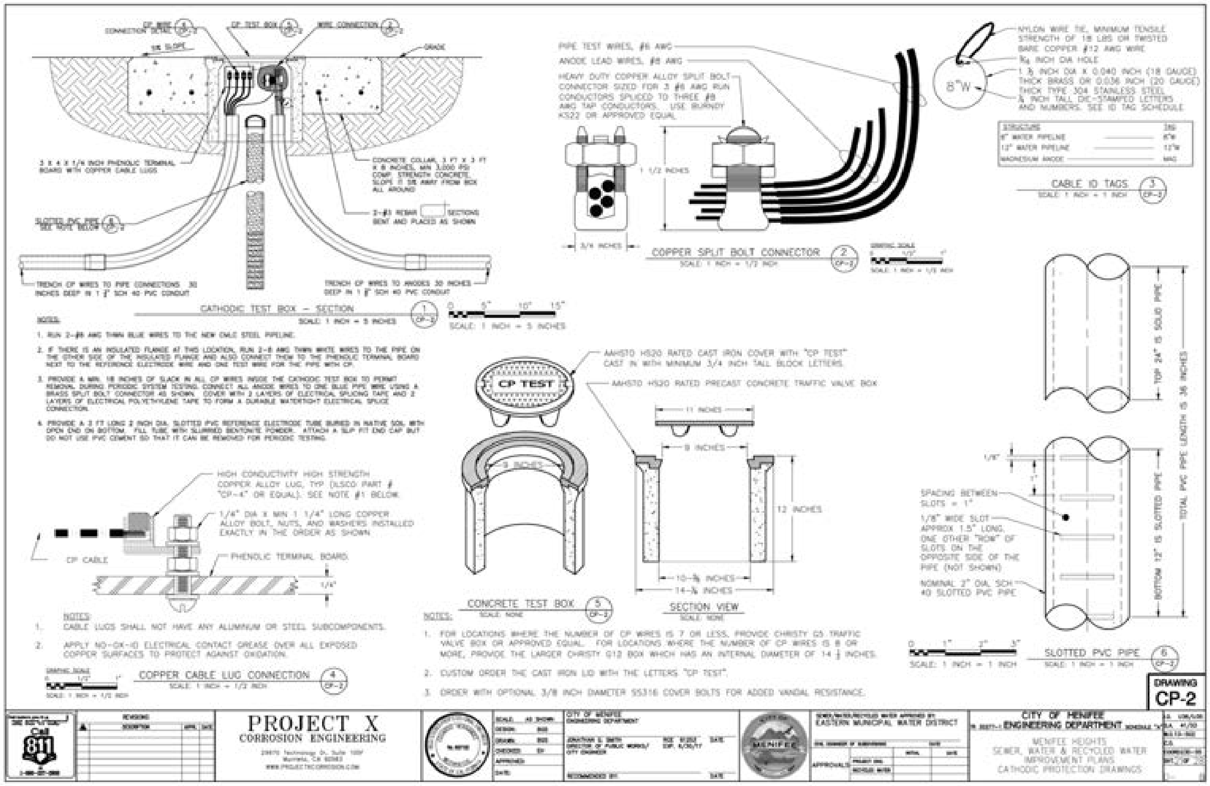


Figure 7 Sample of corrosion test station specification drawing

5.4.8 Excess Flux in Plumbing

Investigations of internal corrosion of domestic water plumbing systems almost always finds excess flux to be the cause of internal pitting of copper pipes. Some people believe that there is no such thing as too much flux. Flux runs have been observed to travel up to 20 feet with pitting occurring along the flux run. Flushing a soldered plumbing system with hot water for 15 minutes can remove significant amounts of excess flux left in the pipes. If a plumbing system is expected to be stagnant for some time, it should be drained to avoid stagnant water conditions that can lead to pitting and dezincification of yellow brasses.

5.4.9 Landscapers and Irrigation Sprinkler Systems

A significant amount of corrosion of fences is due to landscaper tools scratching fence coatings and irrigation sprinklers spraying these damaged fences. Recycled water typically has a higher salt content than potable drinking water, meaning that it is more corrosive than regular tap water. The same risk from damage and water spray exists for above ground pipe valves and backflow preventers. Fiber glass covers, cages, and cement footings have worked well to keep tools at an arm’s length.

5.4.10 Roof Drainage splash zones

Unbelievably, even the location where your roof drain splashes down can matter. We have seen drainage from a home’s roof valley fall directly down onto a gas meter causing it’s piping to corrode at an accelerated rate reaching 50% wall thickness within 4 years. It is the same effect as a splash zone in the ocean or in a pool which has a lot of oxygen and agitation that can remove material as it corrodes.

5.4.11 Stray Current Sources

Stray currents which cause material loss when jumping off of metals may originate from direct-current distribution lines, substations, or street railway systems, etc., and flow into a pipe system or other steel structure. Alternating currents may occasionally cause corrosion. The corrosion resulting from stray currents (external sources) is similar to that from galvanic cells (which generate their own current) but different remedial measures may be indicated. In the electrolyte and at the metal-electrolyte interfaces, chemical and electrical reactions occur and are the same as those in the galvanic cell; specifically, the corroding metal is again considered to be the anode from which current leaves to flow to the cathode. Soil and water characteristics affect the corrosion rate in the same manner as with galvanic-type corrosion.

However, stray current strengths may be much higher than those produced by galvanic cells and, as a consequence, corrosion may be much more rapid. Another difference between galvanic-type currents and stray currents is that the latter are more likely to operate over long distances since the anode and cathode are more likely to be remotely separated from one another. Seeking the path of least resistance, the stray current from a foreign installation may travel along a pipeline causing severe corrosion where it leaves the line. Knowing when stray currents are present becomes highly important when remedial measures are undertaken since a simple sacrificial anode system is likely to be ineffectual in preventing corrosion under such circumstances.[14] Stray currents can be avoided by installing proper electrical shielding, installation of isolation joints, or installation of sacrificial jump off anodes at crossings near protected structures such as metal gas pipelines or electrical feeders.

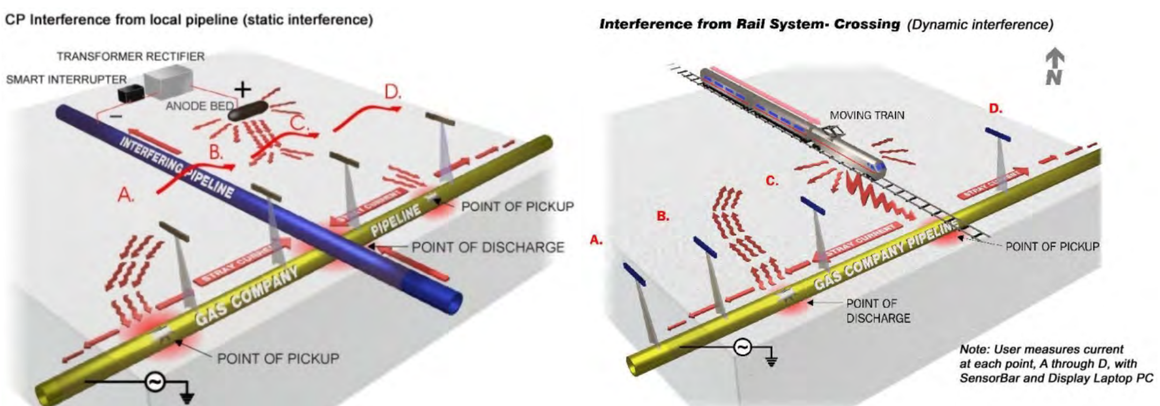


Figure 8 Examples of Stray Current[15]

[14] http://corrosion-doctors.org/StrayCurrent/Introduction.htm

[15] http://www.eastcomassoc.com/

Project X Corrosion Engineering

Corrosion Control – Soil, Water, and Metallurgy Lab

Lab Request Sheet Chain of Custody

Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com

Ship Samples To: 29990 Technology Dr, Suite 13, Murrieta, CA 92563

Project X Job Number S210111F Rockridge G. 20-1950 123 Ind. Dr. 4 Full +RPT

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name:	Rockridge Geotechnical	**Contact Name:**	Quintin Flores
		Phone No:	5305862513
Mailing Address:	270 Grand Ave, Oakland CA	**Contact Email:**	qaflores@rockridgegeo.com
Accounting Contact:	Jarka Stepanek	**Invoice Email:**	invoices@rockridgegeo.com
Client Project No:	20-1950	**Project Name:**	123 Independence Drive

P.O. #:		3-5 Day Standard	3 Day Guarantee 50% mark-up	24 Hr RUSH 100% mark-up
	(Business Days) Turn Around Time:	✓		

Results By: ☐ Phone ☐ Fax ☐ Email

Date & Received by : 1-11-2021

Special Instructions:

ANALYSIS REQUESTED (Please circle)

Analysis	Group	Methods (Default Method last)
Soil Resistivity	Full Corrosion Series / Geo Quad	Caltrans CTM643; AASHTO T2888; ASTM G187
pH	Full Corrosion Series / Geo Quad	Caltrans CTM643; AASHTO T289; ASTM G51
Sulfate	Full Corrosion Series / Geo Quad	Caltrans CTM417; AASHTO T290; ASTM D4327
Chloride	Full Corrosion Series / Geo Quad	Caltrans CTM422; AASHTO T291; ASTM D4327
Redox Potential	Full Corrosion Series	SM 2580B; ASTM G200
Sulfide	Full Corrosion Series	SM 4500-S2
Ammonia	Full Corrosion Series	SM 4500-NH3; ASTM D4327
Nitrate	Full Corrosion Series	SM 4500-NO3; ASTM D4327
Flouride	Full Corrosion Series	SM 2320B
Phosphate	Full Corrosion Series	ASTM D4327
Lithium	Full Corrosion Series	ASTM D4327
Sodium	Full Corrosion Series	ASTM D4327
Potassium	Full Corrosion Series	ASTM D4327
Magnesium	Full Corrosion Series	ASTM D4327
Calcium	Full Corrosion Series	ASTM D4327
BiCarbonate	Full Corrosion Series	SM 2320B
Full Corrosion Series		
Soil Corrosivity Evaluation Report	Reports	*Req: Min. 3 Samples, site map, and groundwater info
Water Corrosivity Mini Report	Reports	*Req: Min. 3 Samples, site map, and groundwater info
Moisture Content		ASTM D2216
Total Alkalinity		SM 2520B
Thermal Resistivity		
Metallurgical Analysis		
Langelier Index		
Puckorius Index		
XRF Elemental Analysis		
Water Hardness		

Lab #	#	SAMPLE ID - BORE #	DESCRIPTION	DEPTH (ft)	DATE COLLECTED	Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	Sulfide	Ammonia	Nitrate	Flouride	Phosphate	Lithium	Sodium	Potassium	Magnesium	Calcium	BiCarbonate	Full Corrosion Series	Soil Corrosivity Evaluation Report	Water Corrosivity Mini Report	Moisture Content	Total Alkalinity	Thermal Resistivity	Metallurgical Analysis	Langelier Index	Puckorius Index	XRF Elemental Analysis	Water Hardness
54	1	B-1-2B	CLAY (CL): olive to olive-gray / SANDY CLAY (CL): yellow-brown	5.75	12/23/2020																		X									
55	2	B-2-1A	CLAY (CH): dark brown	3.0	12/23/2020																		X									
56	3	B-4	CLAY WITH SAND (CH): gray-brown	5.0	12/22/2020																		X									
57	4	B-6-1B	CLAY (CH): dark brown	3.5	12/22/2020																		X									
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