ROCKRIDGE GEOTECHNICAL

Prepared for **United Playaz**

GEOTECHNICAL INVESTIGATION
PROPOSED IMPROVEMENTS
1044 HOWARD STREET
SAN FRANCISCO, CALIFORNIA

April 26, 2023
Project No. 23-2354

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GEOTECHNICAL

April 26, 2023
Project No. 23-2354

Ms. Carolyn Caldwell
United Playaz
1038 Howard Street
San Francisco, California 94103

Subject: Geotechnical Investigation
Proposed Improvements
1044 Howard Street
San Francisco, California

Dear Ms. Caldwell,

We are pleased to present our geotechnical investigation report for the improvements to be constructed at 1044 Howard Street in San Francisco, California. Our geotechnical investigation was performed in accordance with our proposal dated January 3, 2023.

The subject property is located on the northern corner of the intersection of Howard and Russ streets. The subject property is rectangular shaped with maximum plan dimensions of 32 by 80 feet. The site is bordered by three-story residential buildings to the northwest and northeast, Howard Street to the southeast, and Russ Street to the southwest. The site is currently occupied by a one- to two-story building. The second story occupies the northern third of the existing building. Available as-built plans indicate the existing building is currently supported on a waffle slab (floor slab with interconnected footings).

Current plans are to renovate the existing building. Proposed improvement plans include a new roof, a new replacement second story that will be within the same footprint as the existing second story, a basketball court on the new roof over the one-story portion of the building, and structural upgrades. Project plans may include new walls for the first story.

From a geotechnical standpoint, we conclude the proposed renovation and improvements 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 are:

- the presence of fill and marsh deposits underlying the site that are susceptible to cyclic densification and liquefaction
- the potential for lateral spread to occur at the site vicinity

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- potential for seismically induced ground settlement and lateral spreading to occur at the site, resulting in damages to building foundations and underground utilities
- providing adequate vertical and lateral support for the proposed improvements.

Considering that lateral spread is a neighborhood-wide phenomenon and the relatively small footprint of the subject property, we conclude lateral spread at the site vicinity cannot be practically eliminated through soil improvement or foundation upgrade. To reduce the potential amount of building damage from the seismically induced differential settlement and to support new improvements, we conclude the most practical method is to strengthen, if necessary, the existing waffle slab (mat) foundation and plan to relevel the building by mud jacking, if necessary, following a major earthquake.

The recommendations contained in our report are based on a limited subsurface exploration. Consequently, variations between expected and actual subsurface conditions may be found in localized areas during construction. Therefore, we should be engaged to observe site grading and foundation installations 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 yours,
ROCKRIDGE GEOTECHNICAL, INC.

Linda H. J. Liang, P.E., G.E.	Craig S. Shields, P.E., G.E.
Principal Engineer	Principal Engineer

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PROPOSED IMPROVEMENTS
1044 HOWARD STREET
San Francisco, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. (Rockridge) for the proposed improvements to the existing building at 1044 Howard Street in San Francisco, California. The site is located on the northern corner of the intersection of Howard and Russ streets, as shown on the Site Location Map (Figure 1).

The subject property is rectangular shaped with maximum plan dimensions of 32 by 80 feet, as shown on the Site Plan (Figure 2). The site vicinity has a ground surface gradient of about 1.5 percent down towards the south. The site is bordered by three-story residential buildings to the northwest and northeast, Howard Street to the southeast, and Russ Street to the southwest. The site is currently occupied by a one- to two-story building. The second story occupies the northern third of the existing building. Available as-built plans indicate the existing building is supported on a waffle slab (floor slab with interconnected footings).

Current plans are to renovate the existing building to be occupied by United Playaz, a violence prevention and youth development community organization. Proposed improvement plans include a new roof, a new replacement second story that will encompass the same footprint as the existing second story, a basketball court on the new roof of the one-story portion of the building, and structural upgrades. Project plans may include new walls for the first story.

2.0 SCOPE OF SERVICES

Our investigation was performed in accordance with our proposal dated January 3, 2023. Our scope of services consisted of reviewing available subsurface information and geologic maps of the site and vicinity, exploring subsurface conditions at the site by performing two dynamic penetrometer tests (DPTs), advancing one hand auger boring, and performing engineering analyses to develop conclusions and recommendations regarding:

- subsurface conditions
- site seismicity and seismic hazards, including the potential for liquefaction and lateral spreading, and total and differential settlement resulting from liquefaction and/or cyclic densification
- the most appropriate foundation type(s) for the proposed improvements
- design criteria for the recommended foundation type(s), including vertical and lateral capacities for each of the foundation type(s)
- estimates of foundation settlements
- 2022 San Francisco Building Code (SFBC) site class and design spectral response acceleration parameters
- construction considerations.

3.0 FIELD INVESTIGATION AND DATA REVIEW

To evaluate the subsurface conditions at the site, we performed two dynamic penetrometer tests (DPTs) and advanced one hand-auger boring at the site, and reviewed several geotechnical reports of the site vicinity in our files.

3.1 Field Investigation

On February 1, 2023, we explored the subsurface conditions at the site by performing two DPTs, designated as DPT-1 and DPT-2, and advancing one hand-auger boring, designated as HA-1, at the approximate locations shown on Figure 2. Boring HA-1 was advanced at the same location as DPT-2. The floor slab was pre-cored prior to advancing the DPTs and boring.

The DPTs were performed following the methodology presented in the technical paper titled *A Portable Dynamic Penetrometer for Geotechnical Investigations*, prepared by J.R. Triggs and P.D. Simpson. The DPTs consisted of manually driving a 1.4-inch-diameter, cone-tipped probe with a 35-pound hammer falling 15 inches. The blow counts required to drive the probe were recorded at 10-centimeter intervals and converted to Standard Penetration Test (SPT) N-values for use in our engineering analyses. Both DPTs were advanced to a depth of 16.4 feet below the bottom of the floor slab. The floor slab is 17 and 12 inches thick at DPT-1 and DPT-2, respectively. The results of DPT-1 and DPT-2 are presented on Figures A-1 and A-2, respectively, in Appendix A.

Boring HA-1 was advanced using a 3-inch-diameter hand auger to a depth of 3 feet below top of slab (corresponding to 2 feet below bottom of slab) at DPT-2 location. Our field engineers obtained soil samples for visual classification. Upon completion, the borehole was backfilled with soil cuttings. A log of the boring is presented on Figure A-3. The soil encountered in our boring was classified in general conformance with the classification chart shown on Figure A-4.

3.2 Data Review

We reviewed available subsurface information in our files of the site vicinity. Specifically, we reviewed the following reports:

- *Geotechnical Investigation, Proposed Mixed-Use Building, 1088 Howard Street, San Francisco, California*, prepared by Rockridge Geotechnical, Inc. and dated December 18, 2020 (Rockridge 2020).
- *Revised Geotechnical Report, Proposed Residential Building, 119 7th Street, San Francisco, California*, prepared by Rockridge Geotechnical, Inc. and dated October 16, 2014 (Rockridge 2014).
- *Revised Final Report, Geotechnical Investigation, Proposed Residential Building, 36 & 38 Harriet Street, San Francisco, California,* prepared by Rockridge Geotechnical, Inc. and dated April 9, 2010 (Rockridge 2010).

The site locations for the above-referenced geotechnical reports are shown on Figure 1. As part of the investigation at 1088 Howard Street, Rockridge (2020) performed two cone penetration tests (CPTs) to a depth of 100 feet below the ground surface (bgs). The report for 1088 Howard Street also included log of a test boring previously drilled at the site by another consultant (Ninyo and Moore). For the investigation at 119 7th Street, Rockridge (2014) drilled one rotary-wash boring to 91 feet bgs and performed three CPTs to 47-1/2 to 70-1/2 feet bgs. In addition, for the investigation at 36 & 38 Harriet Street, Rockridge (2010) performed two CPTs to 54 and 57 feet bgs. Site plans showing the boring and CPT locations and copies of boring logs, CPT results, and laboratory test results from these three previous geotechnical investigations (Rockridge 2010, 2014, and 2020) are attached in Appendix B.

4.0 SUBSURFACE CONDITIONS

Historical development of the San Francisco Bay Area resulted in placement of artificial fill material over substantial portions of modern estuaries, marshlands, tributaries, and creek beds in an effort to reclaim land. The site is in an old marsh area, known previously as Sullivan Marsh, which was progressively filled between 1850 and 1900. Records indicate the filling was essentially complete by 1900; however, additional fill may have been placed after 1906 to dispose of rubble from the 1906 earthquake and fire. Maps showing the limits of the former Sullivan Marsh[1] indicate the site lies inside the northern lobe of the former marsh (Figure 3).

As presented on the Regional Geologic Map (Figure 4), the site is mapped as being underlain by artificial fill (af). The results of our DPTs and other subsurface investigations in the site vicinity (Rockridge 2010, 2014, and 2020) indicate the site is blanketed by about 15 feet of undocumented fill consisting of loose to medium dense sand with variable amounts of silt. Based on our experience with undocumented fill in the site vicinity, the fill may also contain variable amounts of gravel, brick, and concrete rubble.

Based on subsurface data available for the site vicinity, the fill is underlain by marsh deposits that extend to a depth of about 35 to 40 feet bgs. The marsh deposits generally consist of sand with interbedded layers of silty sand, sandy silt, and clay. We anticipate the marsh deposits are medium dense (medium stiff to stiff for silt and clay) to a depth of about 30 feet bgs and dense (stiff for silt and clay) between depths of 30 and 40 feet bgs. Beneath the marsh deposits is likely medium stiff, highly compressible, high-plasticity clay, locally known as Bay Mud. We anticipate the Bay Mud extends to a depth of approximately 60 feet bgs. The Bay Mud is likely underlain by older bay and alluvial deposits. The older bay and alluvial deposits underlying the Bay Mud generally consist of stiff to very stiff clay with variable amounts of silt and sand to a depth of about 80 to 85 feet bgs. The older bay and alluvial deposits below depths of about 80 to 85 feet bgs generally consist of very dense sand with variable amounts of silt and clay that extend to the maximum depth explored of 100 feet bgs in the site vicinity. Available subsurface

1 Plate 2-11, “Ground Surface Elevation” from Final Report, Liquefaction Study, Marina District and Sullivan Marsh Area, San Francisco, California, prepared by Harding Lawson Associates, 1991.

information indicates the depth to bedrock in the site vicinity is about 190 feet bgs (Pease and O'Rourke, 1993).

Available historic data indicate the depth to groundwater in the site vicinity is about 10 feet bgs (Pease & O'Rourke, 1993). The depth to groundwater is expected to vary several feet annually, depending on rainfall amounts.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity

The site is located within the Coast Ranges Geomorphic Province of California, which 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 and extends from Point Arena in the north to the Gulf of California in the south. The Coast Ranges Geomorphic Province is bounded on the east by the Great Valley and on the west by the Pacific Ocean.

The major active faults in the area are the San Andreas, San Gregorio, Hayward and Calaveras faults. These and other faults in the region are shown on Figure 5. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[2] [Petersen et al. (2014) & Thompson et al. (2016)] are summarized in Table 1. These references are based on the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013).

2 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.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Characteristic Moment Magnitude
Total North San Andreas (SAO+SAN+SAP+SAS)	13	Southwest	8.04
North San Andreas (Peninsula, SAP)	13	Southwest	7.38
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	17	East	7.58
Hayward (North, HN)	17	East	6.90
San Gregorio (North)	18	West	7.44
Hayward (South, HS)	20	East	7.00
North San Andreas (North Coast, SAN)	26	West	7.52
Total Calaveras (CN+CC+CS+CE)	33	East	7.43
Calaveras (North, CN)	33	East	6.86
Mount Diablo Thrust North CFM	34	East	6.72
Mount Diablo Thrust	34	East	6.67
Monte Vista - Shannon	35	South	7.14
Concord	39	East	6.45
Green Valley	41	Northeast	6.30
Rodgers Creek - Healdsburg	43	North	7.19
Mount Diablo Thrust South	44	East	6.50
Clayton	45	East	6.57
West Napa	45	Northeast	6.97
Greenville (North)	48	East	6.86

Damaging earthquakes have occurred along many of these faults in recorded history, as depicted on Figure 5 (USGS, 2021). Notable historic earthquakes which have impacted the Bay Area in recorded history include:

- 1838 San Andreas Earthquake, M_w = 7.4 (estimated)
- 1865 San Andreas Earthquake, M_w = 6.5 (estimated)
- 1868 Hayward Earthquake, M_w = 7.0 (estimated)
- 1906 Great San Francisco Earthquake (San Andreas Fault), M_w = 7.9 (estimated)
- 1989 Loma Prieta Earthquake (San Andreas Fault), M_w = 6.9
- 2014 West Napa Earthquake, M_w = 6.0

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 San Andreas (Santa Cruz Mountains) faults. The respective probabilities are approximately 25, 21, and 17 percent.

5.2 Historic Earthquake Damages

The 1906 and 1989 earthquakes caused severe shaking, ground movements, and liquefaction which damaged buildings and underground utilities in the area. These effects have been studied by numerous investigators. More recently, Pease and O'Rourke (1993) expanded the available database based on detailed review of a large number of aerial photographs taken shortly after the 1906 earthquake that revealed significant features of the larger ground deformations that had not been previously noted. Specifically, Pease and O'Rourke (1993) documented the following damages and ground movements in the site vicinity caused by the 1906 earthquake:

Ground subsidence in the site vicinity:

- Ground settlement at the intersection of 7th and Natoma streets.
- Ground settlement of 0.6 meters at the intersection of Howard and 6th streets, including flexure of train tracks.
- Wave-like deformation on Howard Street, just west of its intersections with 6th and 7th streets
- Several water main breaks on Howard Street, between 6th and 7th streets.

Less ground damage was observed in the Sullivan Marsh area during the 1989 earthquake, as expected from an earthquake of lesser intensity. Documented damages (Pease and O'Rourke, 1993) in the site vicinity caused by the 1989 earthquake include:

- Differential settlement along 7th Street, between Howard and Folsom streets.
- Sand boils on Howard Street, between 6th and 7th streets.
- Damage to water supply system on 7th Street, between Natoma and Howard streets.

5.3 Geologic Hazards

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[3] lateral spreading,[4] and cyclic densification[5]. We used the results of our investigation at the site and vicinity to evaluate the potential of these phenomena occurring at the project site.

5.3.1 Ground Shaking

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

5.3.2 Ground Surface 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 there is no risk of fault offset at the site from a known active fault. 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.

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

4 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.

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

5.3.3 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 site is located within a zone of liquefaction potential as shown on the map titled *State of California Earthquake Zones of Required Investigation, San Francisco North Quadrangle,* prepared by the California Geological Survey (CGS), released November 17, 2000 (Figure 6). We used the results of CPTs performed at 1088 Howard Street to evaluate the potential for liquefaction to occur at the site.

Liquefaction susceptibility was assessed using the software CLiq v3.5.2 (GeoLogismiki, 2022). 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 & Idriss (2014). We also used the relationship proposed by Zhang, Robertson, and Brachman (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement; a relationship that is an extension of the work by Ishihara and Yoshimine (1992). Our analysis was performed using a high groundwater depth of 10 feet bgs. In accordance with the 2022 SFBC, we used a peak ground acceleration of 0.60 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) for a Site Class D. We recommended a Site Class D for this project based on shear wave velocity measurements from 1088 Howard Street (see Section 6.2). 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.

Our liquefaction analysis indicates there are loose to medium dense sand layers below the groundwater that are susceptible to liquefaction. The majority of the potentially liquefiable sand layers are located between depths of 10 and 30 feet bgs and are up to 15 feet thick. We estimate total liquefaction-induced ground settlement resulting from post-liquefaction reconsolidation of the liquefiable sand layers following a Maximum Considered Earthquake (MCE) event with a PGA_M of 0.60g could be on the order of 4 to 5 inches and differential settlement could be up to 2 to 2-1/2 inches across 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. Our analysis indicates the non-liquefiable soil overlaying the potentially liquefiable soil layers is relatively thin and the potentially liquefiable soils are relatively thick, such that the potential for surface manifestations from liquefaction, such as sand boils is high for ground surfaces not confined by concrete slabs.

Because the site and vicinity may be underlain by a layer of potentially continuous liquefiable soil between depths of 10 and 30 feet bgs and there is a ground-surface downward gradient of about 1.5 percent to the south in the area, we judge the risk of lateral spreading exists. The potential amount of lateral ground movement is difficult to predict, as it is dependent on: (1) the thickness, relative density, and fines content of the liquefiable soil; (2) ground-surface gradient in the area; (3) underground structures (i.e., foundations and basements) in the path of lateral spreading; and (4) the earthquake magnitude and intensity. The earthquake in 1906 caused lateral extensions (1.5 to 2.1 meters) on Minna Street at its intersection with 7th Street. Based on this information, we conclude up to about 6 feet of lateral spreading could occur at the site vicinity during a major earthquake on a nearby fault.

5.3.4 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. There are about 10 feet of loose to medium dense sandy fill above the groundwater table that is susceptible to cyclic densification. We evaluated the cyclic densification potential of the soil in the site vicinity using our CPT data from 1088 Howard Street with the methodology by Yee, Stewart, and Duku (2012) and the methodology by Pradel (1998) using our data from the DPTs performed at the site. In accordance with the 2022 SFBC, we used a PGA_M of 0.60g and moment magnitude 8.04 earthquake in our cyclic densification evaluation. We estimate total and differential ground settlement as a result of cyclic densification at the site will be up to about 1/4 to 1/2 inch and 1/4 inch across a horizontal distance of 30 feet, respectively.

6.0 CONCLUSIONS AND RECOMMENDATIONS

From a geotechnical standpoint, we conclude the existing building may be renovated and improved 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 project site are:

- the presence of fill and marsh deposits underlying the site that are susceptible to cyclic densification and liquefaction
- the potential for lateral spread to occur at the site vicinity
- potential for seismically induced ground settlement and lateral spreading to occur at the site during a major earthquake, resulting in damages to building foundations and underground utilities
- providing adequate vertical and lateral support for the proposed improvements.

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

6.1 Foundation Support and Settlement

Available as-built plans indicate the existing building is supported on a waffle slab (floor slab with interconnected footings). As-built plans showing the thickness of the floor slab and locations and dimensions of interconnected footings were not available when this report was prepared. At the DPT-1 and DPT-2 locations, the cores through the existing floor slab encountered 17 and 12 inches of concrete, respectively.

During a major earthquake, the existing building could experience total settlement of up to about 5 inches and differential settlement of up to about 2-1/2 inches over a horizontal distance of 30 feet due to cyclic densification and post-liquefaction reconsolidation. As presented in Section 5.3.3, we judge the risk of lateral spreading exists and based on historic information from the 1906 Earthquake, we conclude up to about 6 feet of lateral spreading could occur at the site vicinity during a major earthquake on a nearby fault.

To reduce the potential amount of building damage from the seismically induced differential settlement, the following three options were considered: 1) strengthen, if necessary, the existing waffle slab (mat) foundation and plan to relevel the building by mud jacking, if necessary, following a major earthquake, 2) mitigate the liquefaction potential by soil improvement, or 3) support the building on deep foundations. It should be noted that Options #2 and #3 would reduce, but not eliminate the potential for building damage during a major earthquake. Considering there are buildings directly adjacent to the site that may be damaged during ground improvement activities, we conclude Option #2 would be very risky and, therefore, is not recommended. Option #3 would necessitate installing deep foundations to depths of about 70 to 80 feet below grade inside the existing structure. It would be necessary to support the entire structure on the deep foundations to prevent large differential settlement of the building during an earthquake. Although technically feasible, the new deep foundation system would be extremely costly because of the limited access. Considering that lateral spread is a neighborhood-wide phenomenon and the relatively small footprint of the subject property, we conclude lateral spread at the site vicinity cannot be practically eliminated through soil improvement or foundation upgrade (Options #2 and 3). Accordingly, if it is desired to reduce the potential amount of damage from settlement during an earthquake, we conclude Option #1 is the most practical method.

The Structural Engineer should evaluate if the existing waffle mat foundation system meets the objective of Option #1 and mat foundation recommendations presented in Section 6.1.1 below.

In addition, the Owner should recognize that relevelling of the building will likely be required after a major earthquake with Option #1.

6.1.1 Mat Foundation

The edge of the mat should be bottomed at least 12 inches below the lowest adjacent outside grade. Where the mat is adjacent to the neighboring buildings, the bottom of the mat should match the bottom of the adjacent footings. The mat may be designed using a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) for dead-plus-live-load conditions. This value may be increased by one-third for total load conditions. To evaluate the pressure distribution for the mat foundation, we recommend using a modulus of vertical subgrade reaction of 10 pounds per cubic inch (pci); this value has already been scaled to take into account the plan dimensions of the foundation. The value may be increased by one-third for seismic conditions. The mat foundation should be designed to distribute the superimposed structural loads assuming an unsupported area of 20 feet in diameter at any location within the mat and a cantilever of 5 feet around the perimeter, limiting the maximum deflections to 1/360th of the span.

We estimate settlement of a mat foundation under the weight of the proposed seismic strengthening elements under static loads, which we anticipate will be relatively light, will be less than 1/2 inch. As previously discussed, the mat may experience additional cyclic densification and liquefaction-induced total and differential settlements up to about 5 inches and 2-1/2 inches across a horizontal distance of 30 feet, respectively, following a major earthquake.

Lateral loads can be resisted by a combination of passive pressure on the outside edges of the mat foundation and friction along the bottom of the mat. Passive resistance may be calculated using an equivalent fluid weight of 250 pounds per cubic foot (pcf). The upper one foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance should be computed using a base friction coefficient of 0.35 where the mat is in contact with soil and 0.20 where the mat is in underlain by a vapor retarder. It should be noted no vapor retarder was encountered in the two holes cored through the foundation during our investigation. The passive resistance and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without further reduction.

If water vapor moving through the mat foundation is considered detrimental, a vapor retarder meeting the requirements for Class A vapor retarders stated in ASTM E1745 should be placed directly on the subgrade soil below the mat foundation. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

The mat subgrade should be free of loose materials and debris prior to placing the vapor retarder. We should examine the subgrade prior to placement of the vapor retarder to confirm the condition of the subgrade is acceptable.

6.2 Seismic Design

The latitude and longitude of the site are 37.7791° and -122.4048°, respectively. Section 1613A of the 2022 California Building Code (CBC), on which the 2022 SFBC is based, and Section 20.3.1 of ASCE 7-16 indicate if liquefiable soil is present at a site, it is classified as Site Class F and a site-specific response study is required; however, if the period of the structure is less than 0.5 second, the liquefaction potential can be neglected when determining seismic design parameters. Since the period of the building is less than 0.5 seconds, and the results of our CPT-2 performed at 1088 Howard Street (Rockridge 2020) indicate the shear wave velocity for the upper 100 feet of soil (V_{s30}) at the site vicinity is about 700 feet per second, we recommend Site Class D be used. For design in accordance with 2022 SFBC, we recommend the following:

- Site Class D (stiff soil, non-default)
- S_S = 1.50g, S_1 = 0.60g

The 2022 SFBC is based on the guidelines contained within ASCE 7-16 (Supplement 3 revision) which stipulates that where S_1 is greater than 0.2 times gravity (g) for Site Class D, a ground motion hazard analysis is required unless the long-period spectral design parameters (S_{M1}, S_{D1}) are increased by 50%. Therefore, we recommend the following seismic design parameters, which include the 50% increase as designated by an asterisk:

- F_a = 1.0, F_v = 1.7
- S_{MS} = 1.50g, S_{M1}* = 1.53g
- S_{DS} = 1.00g, S_{D1}* = 1.02g
- Seismic Design Category D for Risk Factors I, II, and III

6.3 Fill Quality and Compaction

In areas that will receive fill or new foundations (mat), the soil subgrade exposed should be scarified to a depth of at least 8 inches, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction.[6] The soil subgrade should be compacted to at least 95 percent relative compaction if the soil consists of clean sand or gravel (defined as soil with less than 5 percent fines passing the No. 200 sieve). The soil subgrade should be kept moist until it is covered by fill or improvements.

Fill should consist of on-site soil or imported soil (select fill) that is free of organic matter, contains no rocks or lumps larger than 3 inches in greatest dimension, and is approved by the Geotechnical Engineer. Imported soil (select fill) should have a liquid limit less than 40 and plasticity index less than 12. Samples of proposed imported fill material should be submitted to the Geotechnical Engineer at least three business days prior to use at the site. The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not available, up to two weeks should be allowed to perform analytical testing on the proposed imported material.

Fill should be placed in lifts not exceeding 8 inches in loose thickness, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. The fill should be compacted to at least 95 percent relative compaction if it consists of clean sand or gravel (defined as soil with less than 5 percent fines passing the No. 200 sieve). Both subgrade and fill compaction should be performed with relatively small compaction equipment, such as a

6 Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557 laboratory compaction procedure.

"Turtle" vibratory plate. Within 5 feet of the adjacent structures, only a jumping-jack-type compactor should be used.

6.4 Construction Considerations

The soil to be excavated consists predominately of sandy fill, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. Removal of existing on-site improvements, including floor slabs and buried foundations, will require equipment capable of breaking concrete.

There are existing buildings adjacent to the site. Heavy equipment should not be used within 10 horizontal feet from adjacent shallow foundations and basement walls. Excavations for new or modified foundations should not undermine footings of adjacent structures.

7.0 ADDITIONAL GEOTECHNICAL SERVICES

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

8.0 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 are based on the assumption that the subsurface soil and groundwater conditions do not deviate appreciably from those disclosed in our field investigation. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The foundation recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

REFERENCES

Boulanger, R.W and Idriss, I.M. (2014). "CPT and SPT Based Liquefaction Triggering Procedures", Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, Report No. UCD/CGM-14/01, April.

California Geological Survey (2008). Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117.

California Geological Survey (2000). State of California Seismic Hazard Zones, City and County of San Francisco, Official Map.

Field, E.H., et al. (2013). *Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3)—The Time-Independent Model* (Open-File Report 2013-1165). United States Geological Survey. http://doi.org/10.3133/ofr20131165

Harding Lawson Associates, Dames & Moore, Kennedy/Jenks/Chilton, and EQE Engineering (1991). Liquefaction Study, Marina District and Sullivan Marsh Area, San Francisco, California.

Ishihara, K. and Yoshimine, M., (1992), Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes, Soils and Foundations, Volume 23, No. 1, pp 173-188.

Ninyo & Moore (2017, July 28). Preliminary Geotechnical Evaluation, 1088 Howard Street, San Francisco, California, dated July 28, 2017.

Pease, J.W. and O'Rourke, T.D. (1993). Liquefaction Hazards in the Mission District and South of Market Areas, San Francisco, California.

Petersen, M.D., et al. (2014, July 17). *Documentation for the 2014 Update of the United States National Seismic Hazard Maps* (Open-File Report 2014-1091). United States Geological Survey. https://pubs.usgs.gov/of/2014/1091/

Pradel, D. (1998). Procedure to Evaluate Earthquake-Induced Settlement in Dry Sand, Journal of Geotechnical and Geoenvironmental Engineering.

Robertson, P.K. (2009, June). "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo.

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

Rockridge Geotechnical, Inc. (2010, April 9). Revised Final Report, Geotechnical Investigation, Proposed Residential Building, 36 & 38 Harriet Street, San Francisco, California.

Rockridge Geotechnical, Inc. (2014, October 16). Revised Geotechnical Report, Proposed Residential Building, 119 7th Street, San Francisco, California.

Rockridge Geotechnical, Inc. (2020, December 18). Geotechnical Investigation, Proposed Mixed-Use Building, 1088 Howard Street, San Francisco, California.

Sitar, N. et al. (2012). Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls, ASCE GeoCongress 2012 Geotechnical Special Publiation No. 226.

Thompson, E.M., et al. (2016). *Shakemap earthquake scenario: Building Seismic Safety Council 2014 Event Set (BSSC2014)*. United States Geological Survey. https://doi.org/10.5066/F7V122XD

Triggs, J.R. and Simpson, P.D. A Portable Dynamic Penetrometer for Geotechnical Investigations, undated.

U.S. Geological Survey (2021). Earthquake Hazards Program, Advanced National Seismic System (ANSS) Comprehensive Catalog of Earthquake Events and Products, website accessed January 2022, https://earthquake.usgs.gov/earthquakes/search/

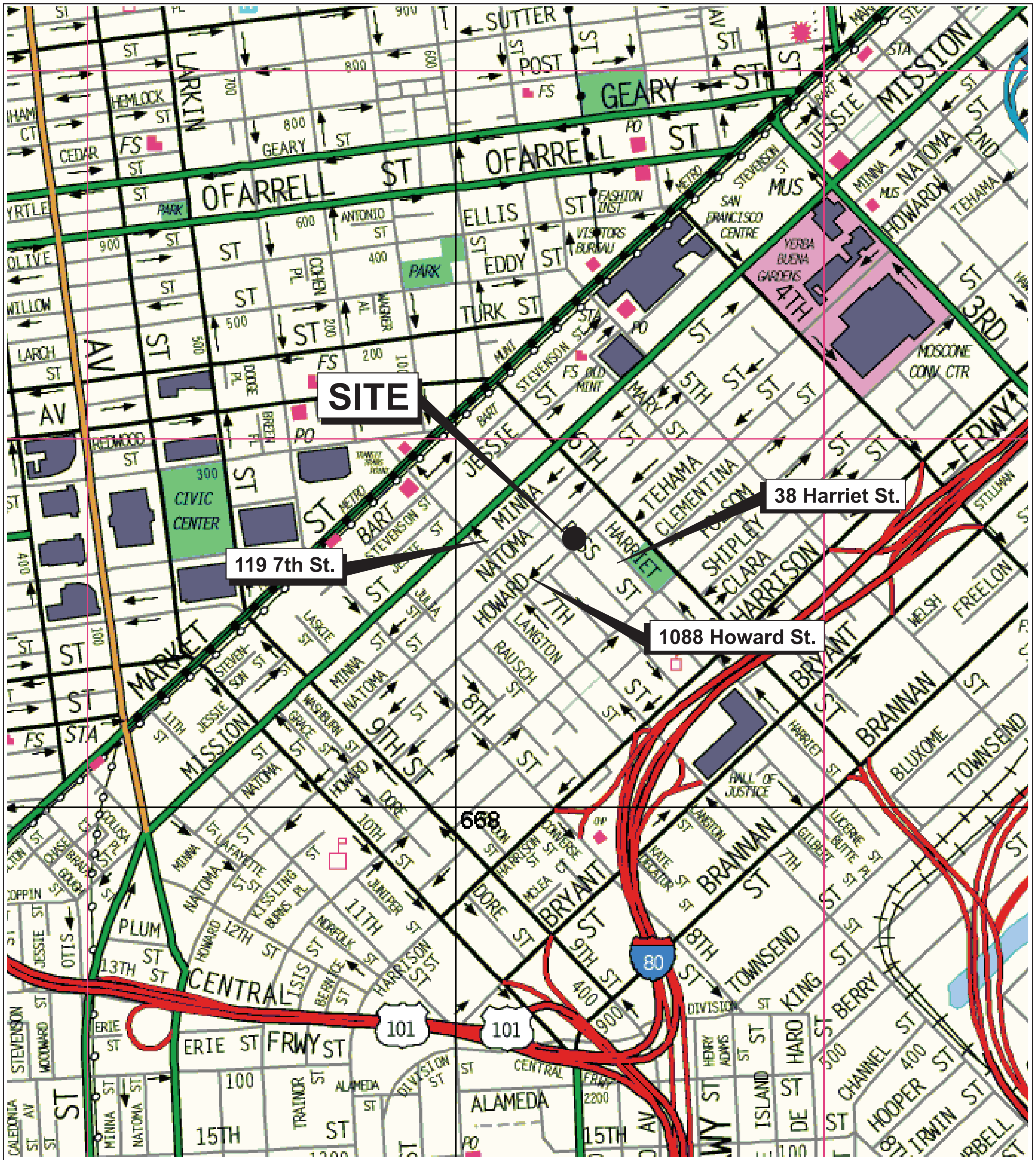
Youd, T. L. et al. (2001). Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering.

Yee, E., Stewart, J., and Duku, P. (2012). "Seismic Compression Behavior of Sands with Fines of Low Plasticity." GeoCongress 2012, pp 839-848.

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

ROCKRIDGE
GEOTECHNICAL

FIGURES



Base map: The Thomas Guide
San Francisco County
2002

0 1,050 2,100 Feet

Approximate scale

N

1044 HOWARD STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 02/20/23	Project No. 23-2354	Figure 1

EXPLANATION

DPT-1 Approximate location of dynamic penetrometer test by Rockridge Geotechnical, Inc., February 1, 2023

HA-1 Approximate location of hand-auger boring by Rockridge Geotechnical, Inc., February 1, 2023

0 16 Feet

Approximate scale

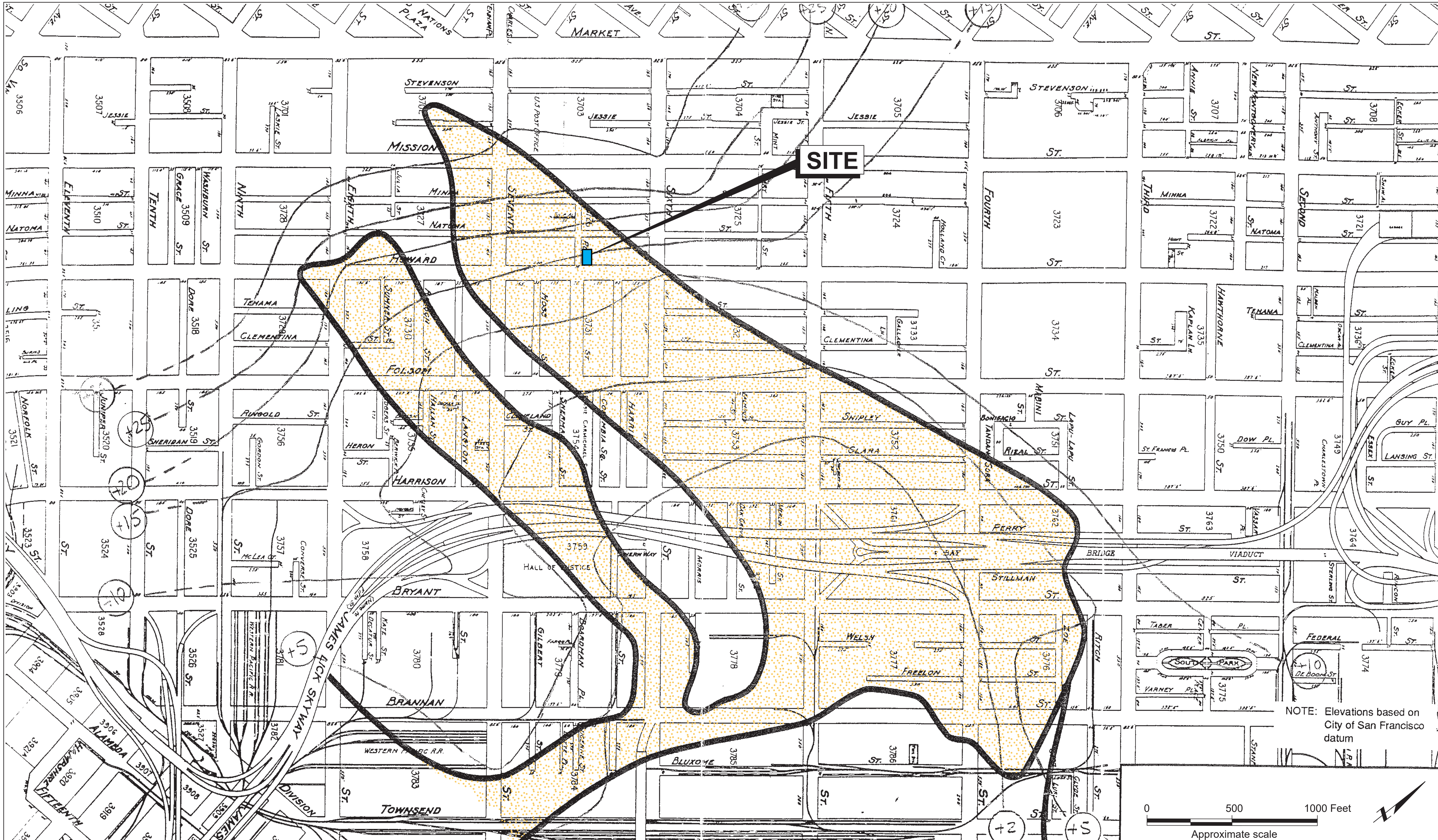
Reference: Base map from a drawing titled "Existing First & Second Floor Plans", by MEI Architecture, printed October 11, 2021.

1044 HOWARD STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

SITE PLAN

Date 02/17/23	Project No. 23-2354	Figure 2



EXPLANATION

(+5) Ground Surface Elevations (San Francisco City Datum)

Sullivan Marsh

Reference: Base map from a drawing titled "Ground Surface Elevation, Liquefaction Study, by Harding Lawson Associates, Dames & Moore, Kennedy/Jenks/Chilton, EQE Engineering, dated 5/17/91.

1044 HOWARD STREET San Francisco, California	**SULLIVAN MARSH AREA**
ROCKRIDGE GEOTECHNICAL	Date 02/01/23 | Project No. 23-2354 | Figure 3

Base map: Google Earth with U.S. Geological Survey (USGS), San Francisco County, 2017.

EXPLANATION

af	**Artificial Fill**
Qs	**Beach and Dune sand (Quaternary)**
Qpa	**Alluvium (Pleistocene)**
Kfs	**Franciscan Complex sedimentary rocks (Cretaceous)**

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

0 1/4 1/2 Mile

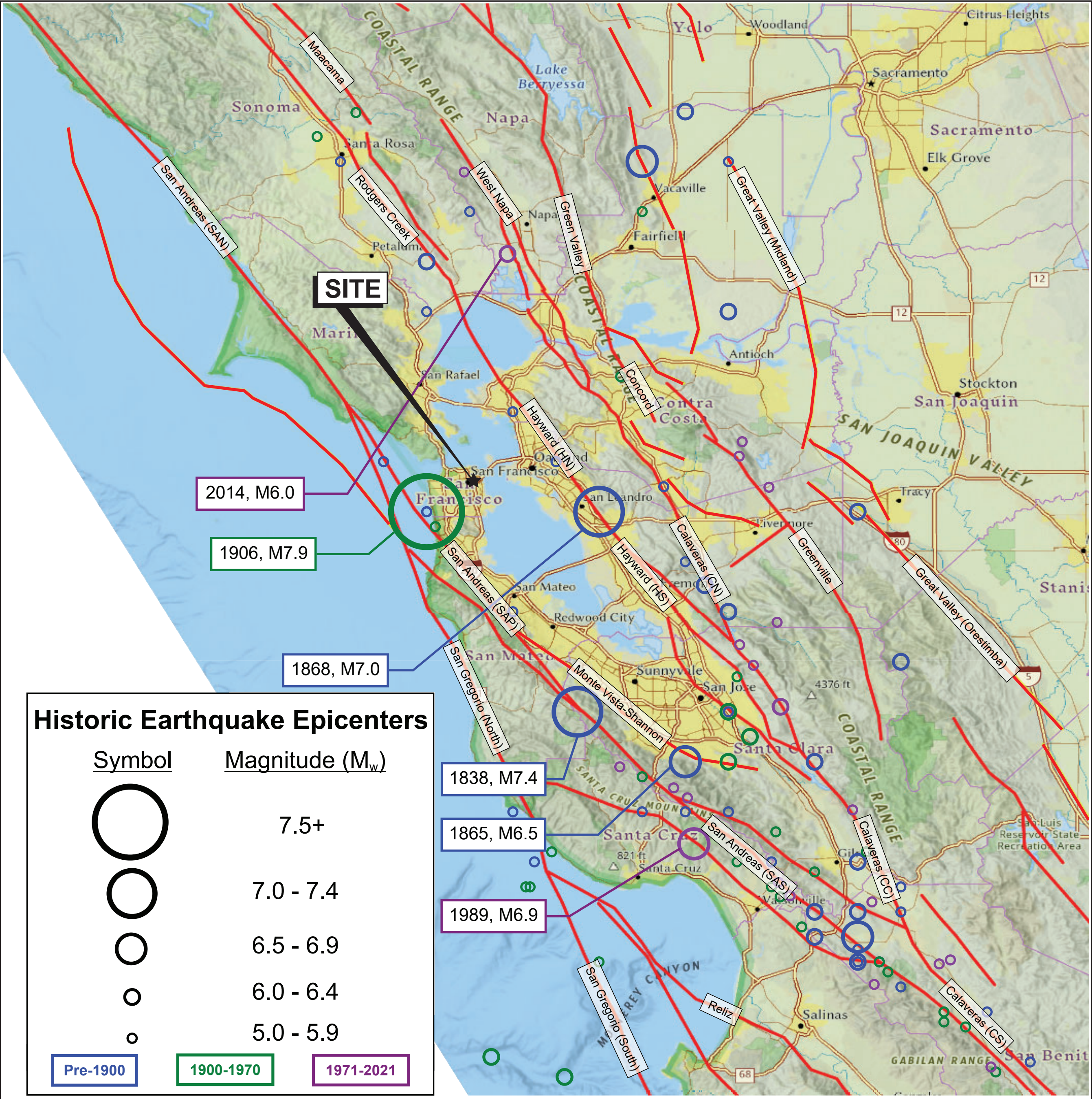
Approximate scale

1044 HOWARD STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 02/06/23 | Project No. 23-2354 | Figure 4



EXPLANATION

Faults (National Seismic Hazard Model)

0 10 20 Miles

0 16 32 Kilometers

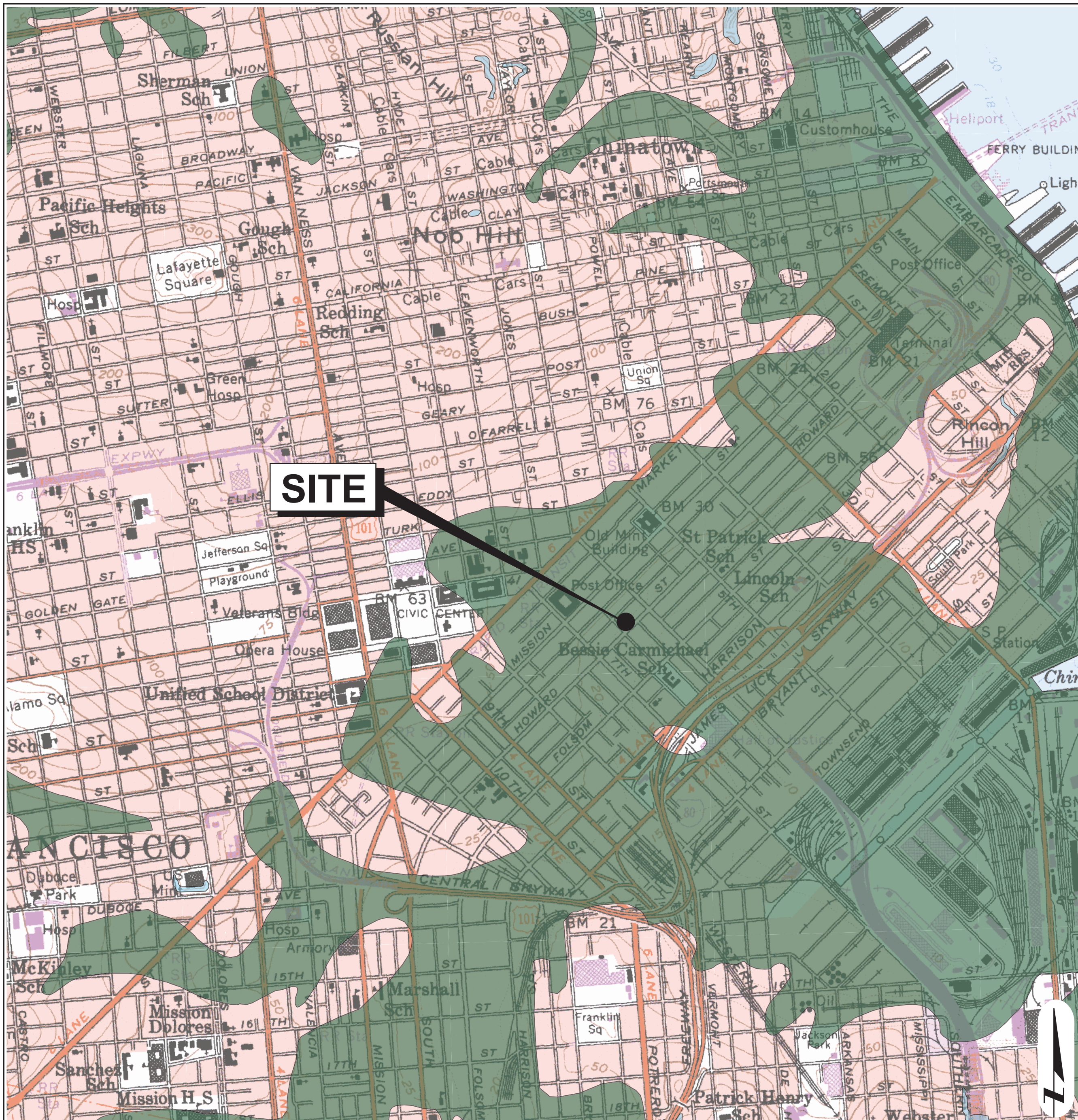
Approximate scale

1044 HOWARD STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Date 02/01/23	Project No. 23-2354	Figure 5



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
San Francisco North Quadrangle
California Geological Survey
Released November 17, 2000

0 2,000 4,000 Feet
Approximate scale

1044 HOWARD STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

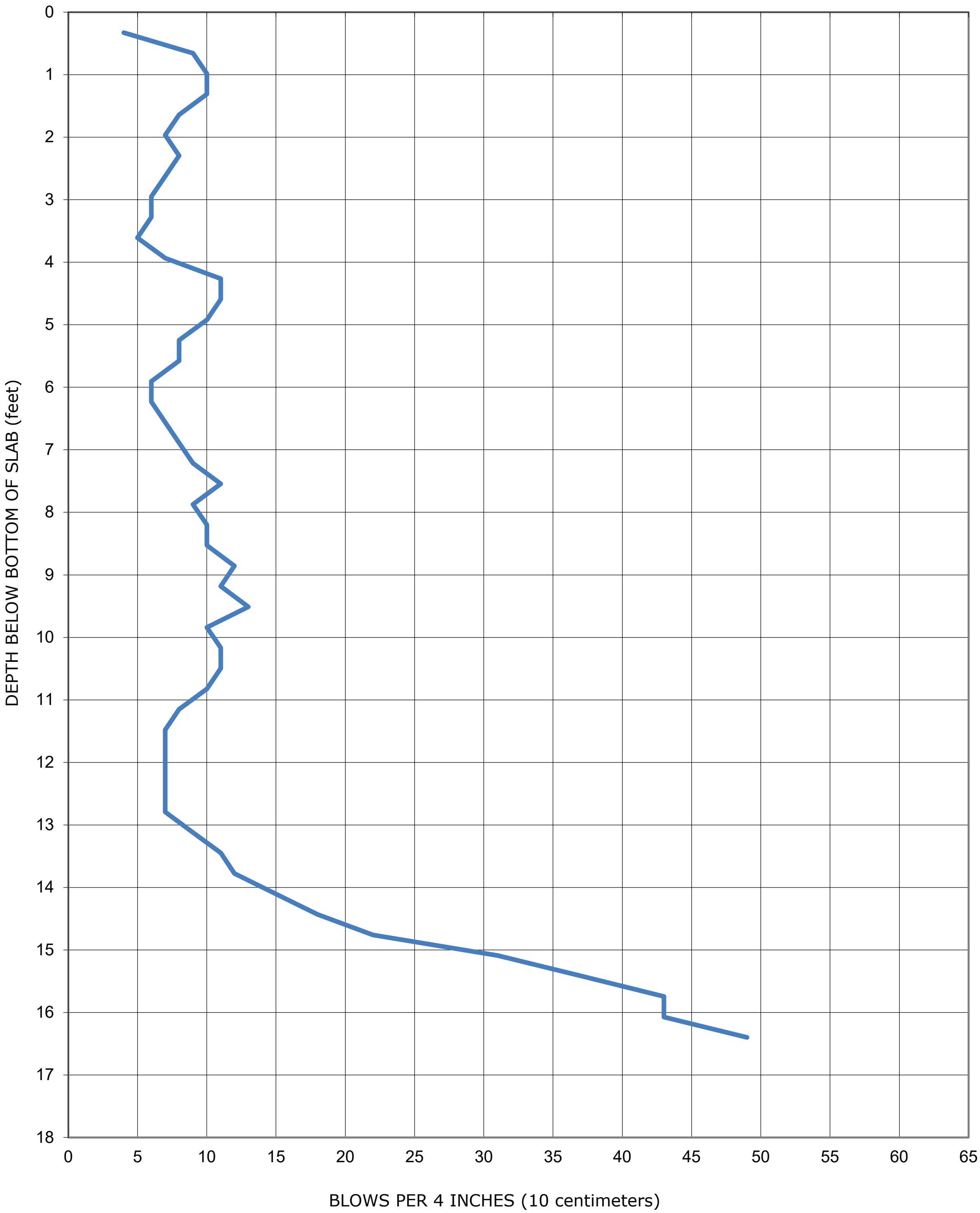
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 02/01/23 | Project No. 23-2354 | Figure 6

ROCKRIDGE GEOTECHNICAL

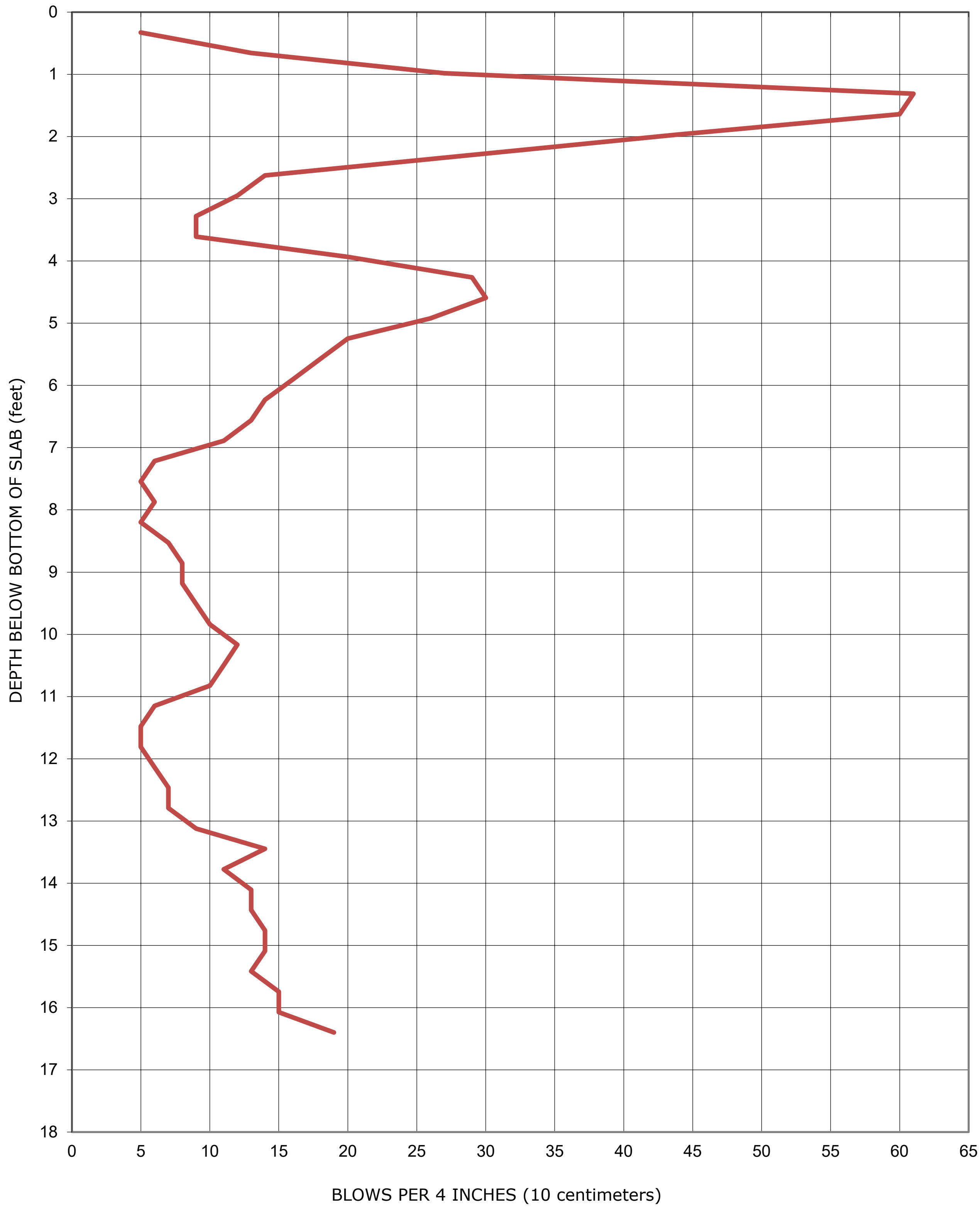
APPENDIX A

Dynamic Penetrometer Test Results and Log of Hand Auger Boring



Note: 17-inch-thick concrete slab

1044 HOWARD STREET San Francisco, California	DYNAMIC PENETROMETER TEST RESULTS DPT-1		
ROCKRIDGE GEOTECHNICAL	Date 02/20/23	Project No. 23-2354	Figure A-1



Note: 12-inch-thick concrete slab

1044 HOWARD STREET San Francisco, California	**DYNAMIC PENETROMETER TEST RESULTS** **DPT-2**
ROCKRIDGE GEOTECHNICAL	Date 02/20/23 | Project No. 23-2354 | Figure A-2

PROJECT:	**1044 HOWARD STREET** San Francisco, California	**Log of Boring HA-1** PAGE 1 OF 1

Boring location: See Site Plan, Figure 2	Logged by: A. Nasser/J. Pisenti
Date started: 02/01/2023 · Date finished: 02/01/2023	
Drilling method: 3-inch-diameter hand auger	
Hammer weight/drop: N/A · Hammer type: N/A	LABORATORY TEST DATA
Sampler: Grab	

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	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
						Hard wood flooring underlain by 12-inch-thck concrete slab						
1					GP	GRAVEL (GP) gray, medium dense, moist, fine to coarse angular gravel						
	GRAB	X			SP-SM	SAND with SILT (SP-SM) yellow-brown, very dense, moist, fine to medium sand						
2	GRAB	X			SP	SAND with GRAVEL (SP) olive-brown to gray-brown, very dense, moist, fine to coarse sub-angular gravel, fine sand, brick and concrete debris						
	GRAB	X										
3												
4												
5												
6												
7												
8												
9												
10												

FILL

Boring terminated at a depth of 3 feet below ground surface.
Boring backfilled with soil cuttings.
Groundwater not encountered during hand-augering.

ROCKRIDGE GEOTECHNICAL

Project No.:	Figure:
23-2354	A-3

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	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	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 1.38- or 1.5-inch inside diameter (refer to text)
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

1044 HOWARD STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

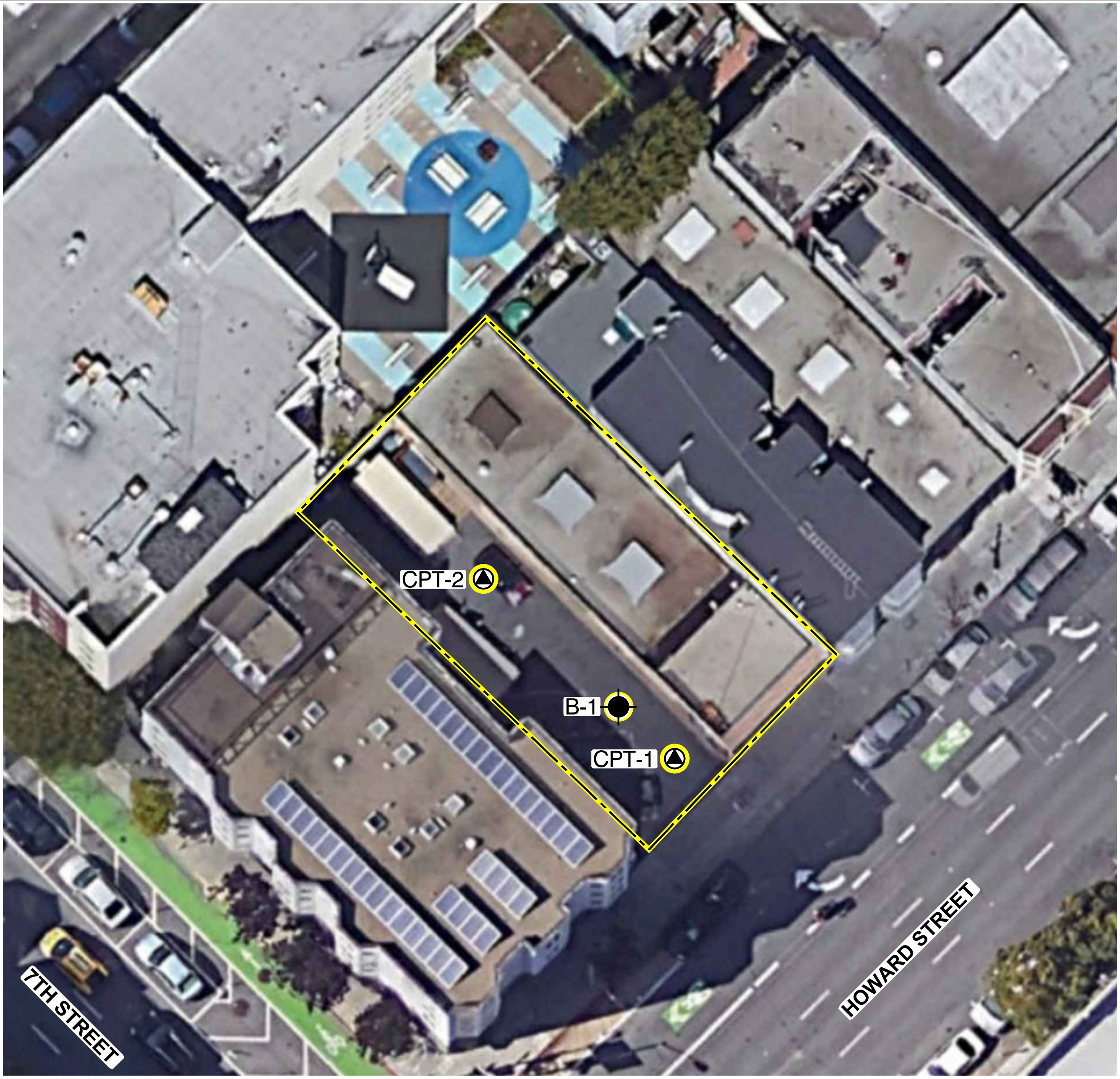
CLASSIFICATION CHART

Date 02/07/23 | Project No. 23-2354 | Figure A-4

ROCKRIDGE GEOTECHNICAL

APPENDIX B

Boring Logs, Cone Penetration Test Results, and Laboratory Test Results from Site Vicinity



Base map: Google Earth, 2020.

EXPLANATION

CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical Inc., May 10, 2019

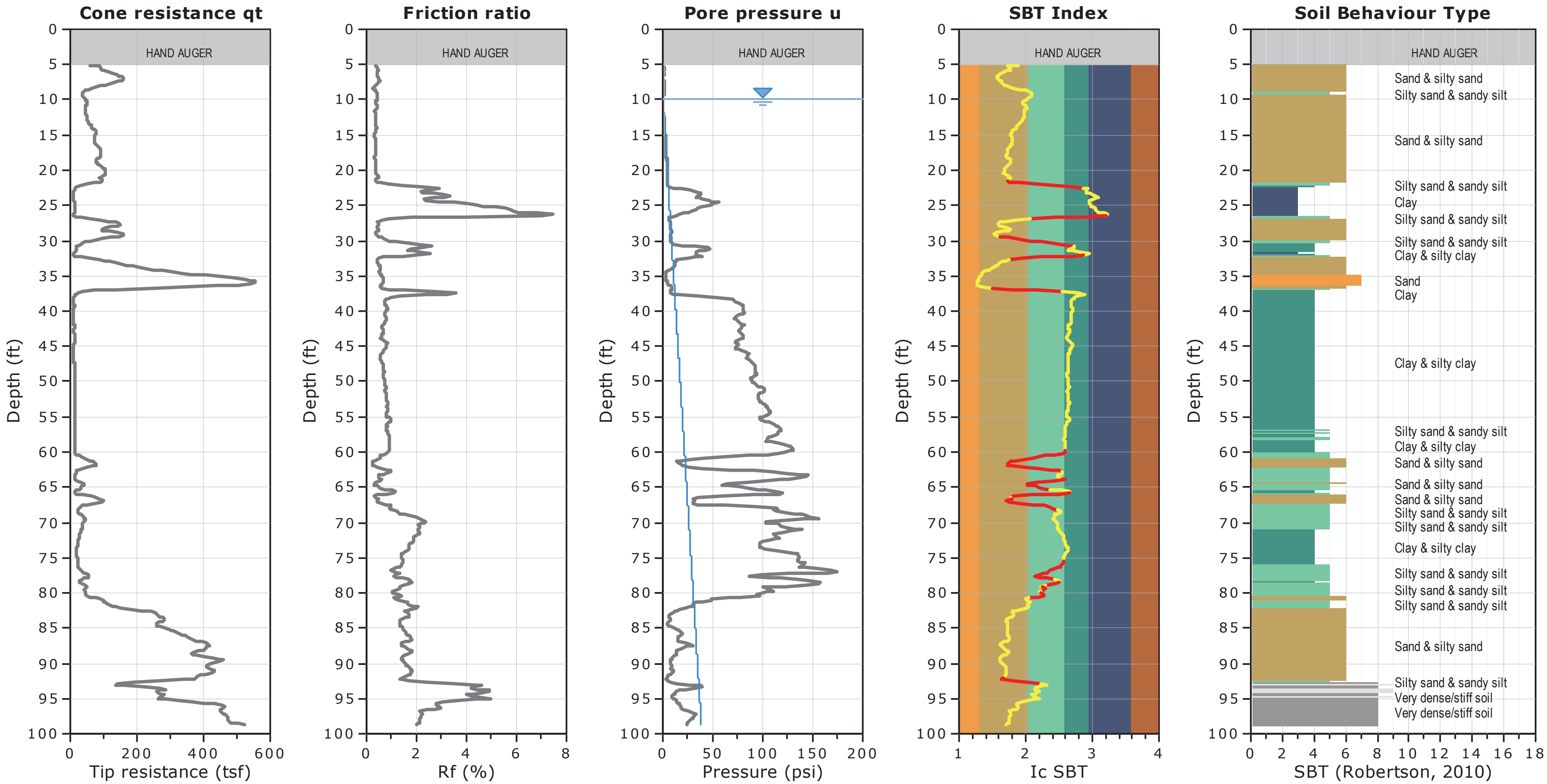
B-1 — Approximate location of boring by Ninyo & Moore, June 29, 2017

Project limits

0 30 Feet

Approximate scale

1088 HOWARD STREET San Francisco, California	**SITE PLAN**
ROCKRIDGE GEOTECHNICAL	Date 12/14/20 | Project No. 19-1689 | Figure 2



Total depth: 100 ft, Date: May 10, 2019
Estimated Groundwater Depth: 10 feet
Cone Operator: Gregg Drilling & Testing

SBT legend

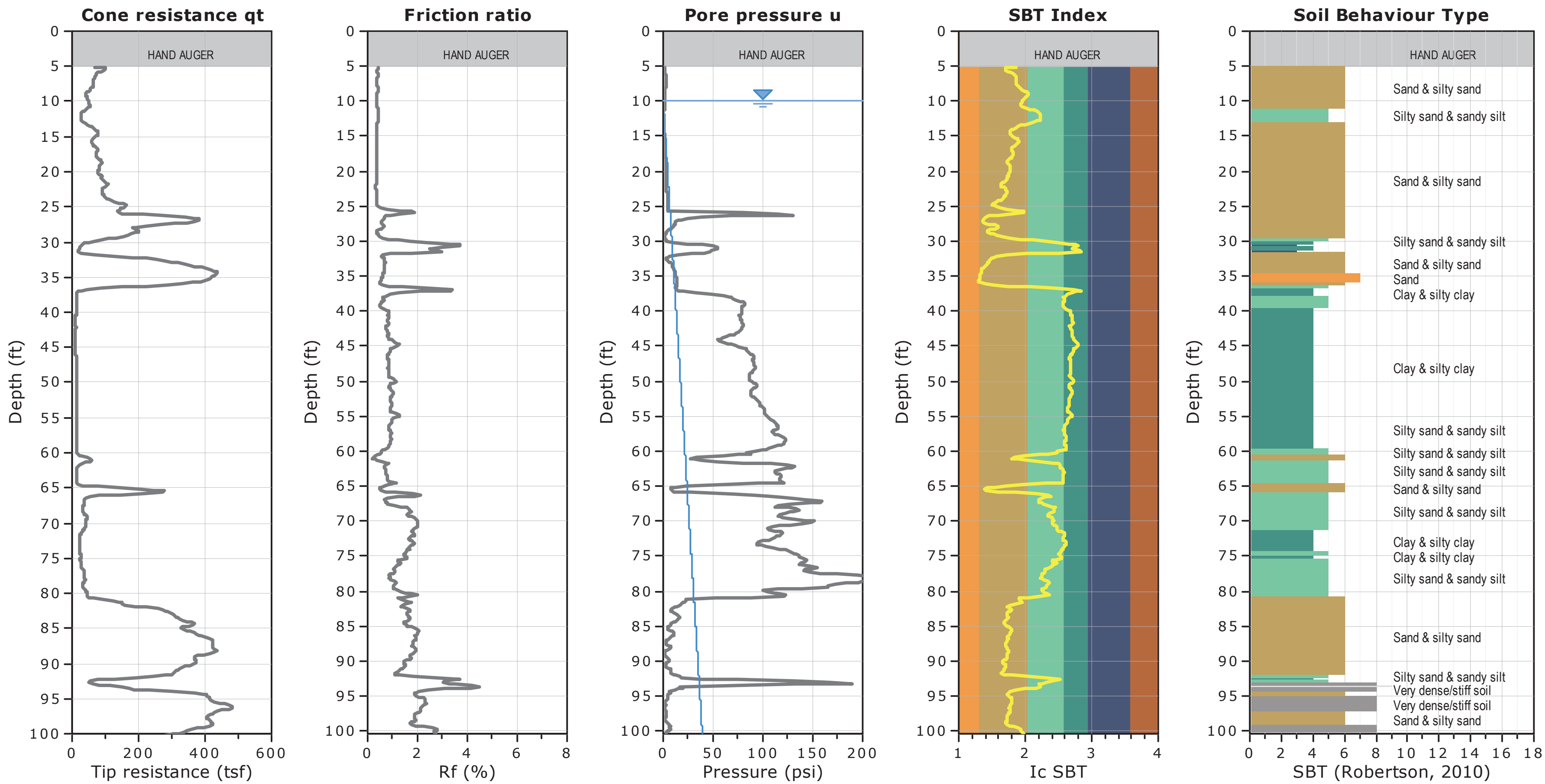
- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty clay
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to clayey sand
- 9. Very stiff fine grained

1088 HOWARD STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 12/15/20 | Project No. 19-1689 | Figure A-1



Total depth: 100 ft, Date: May 10, 2019
Estimated Groundwater Depth: 10 feet
Cone Operator: Gregg Drilling & Testing

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

1088 HOWARD STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 12/15/20 | Project No. 19-1689 | Figure A-2

Test Depth (Feet)	Geophone Depth (Feet)	Waveform Ray Path (Feet)	Incremental Distance (Feet)	Characteristic Arrival Time (ms)	Incremental Time Interval (ms)	Interval Velocity (Ft/Sec)	Interval Depth (Feet)
10.01	9.35	9.49	9.49	15.7500			
14.93	14.27	14.37	4.87	23.8000	8.0500	605.0	11.81
20.01	19.35	19.42	5.06	31.4000	7.6000	665.8	16.81
24.93	24.27	24.33	4.91	37.9500	6.5500	749.1	21.81
30.18	29.52	29.57	5.24	44.1000	6.1500	851.9	26.90
34.94	34.28	34.32	4.75	51.6500	7.5500	629.2	31.90
40.03	39.37	39.40	5.08	60.7000	9.0500	561.3	36.82
44.95	44.29	44.32	4.92	71.7500	11.0500	445.0	41.83
50.03	49.37	49.40	5.08	82.5500	10.8000	470.6	46.83
54.95	54.29	54.32	4.92	92.4000	9.8500	499.4	51.83
60.04	59.38	59.40	5.08	102.4500	10.0500	505.8	56.84
64.96	64.30	64.32	4.92	110.5000	8.0500	611.1	61.84
70.05	69.39	69.41	5.08	116.5500	6.0500	840.3	66.84
74.97	74.31	74.33	4.92	123.9000	7.3500	669.4	71.85
80.05	79.39	79.41	5.08	130.0000	6.1000	833.5	76.85
84.97	84.31	84.33	4.92	133.9500	3.9500	1245.6	81.85
90.06	89.40	89.41	5.08	137.9000	3.9500	1287.2	86.86
94.98	94.32	94.33	4.92	141.2500	3.3500	1468.8	91.86
100.07	99.41	99.42	5.08	144.7000	3.4500	1473.8	96.86

1088 HOWARD STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

SHEAR WAVES VELOCITY
CPT-2

Date 12/14/20 | Project No. 19-1689 | Figure A-2b

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following method.

Bulk Samples

Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	BORING LOG EXPLANATION SHEET
0								Bulk sample.
								Modified split-barrel drive sampler.
								2-inch inner diameter split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.
								Sample retained by others.
5								Standard Penetration Test (SPT).
								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
10								Seepage.
								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	<u>MAJOR MATERIAL TYPE (SOIL)</u>: Solid line denotes unit change.
							CL	Dashed line denotes material change.
								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

Ninyo & Moore

BORING LOG

Explanation of Boring Log Symbols

PROJECT NO.	DATE	FIGURE

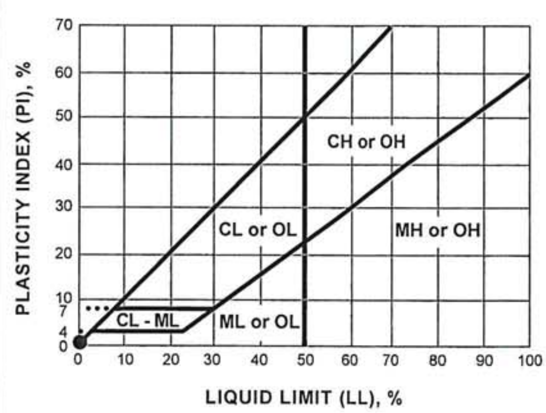
SOIL CLASSIFICATION CHART PER ASTM D 2488

PRIMARY DIVISIONS			SECONDARY DIVISIONS: GROUP SYMBOL	SECONDARY DIVISIONS: GROUP NAME
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

PLASTICITY CHART



APPARENT DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPOOLING CABLE OR CATHEAD: SPT (blows/foot)	SPOOLING CABLE OR CATHEAD: MODIFIED SPLIT BARREL (blows/foot)	AUTOMATIC TRIP HAMMER: SPT (blows/foot)	AUTOMATIC TRIP HAMMER: MODIFIED SPLIT BARREL (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

CONSISTENCY - FINE-GRAINED SOIL

CONSIS-TENCY	SPOOLING CABLE OR CATHEAD: SPT (blows/foot)	SPOOLING CABLE OR CATHEAD: MODIFIED SPLIT BARREL (blows/foot)	AUTOMATIC TRIP HAMMER: SPT (blows/foot)	AUTOMATIC TRIP HAMMER: MODIFIED SPLIT BARREL (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

Explanation of USCS Method of Soil Classification

PROJECT NO.	DATE	FIGURE

DATE DRILLED 6/29/17 BORING NO. B-1

GROUND ELEVATION 20'±(MSL) SHEET 1 OF 3

METHOD OF DRILLING 3" Hand Auger; 8" Hollow Stem Auger - Mobile B-53 (Expl. Geo.)

DRIVE WEIGHT 140 LBS (wireline) DROP 30 inches

SAMPLED BY GL LOGGED BY GL REVIEWED BY PCC

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 3 inches thick.
						SP-SM	AGGREGATE BASE: Approximately 4 inches thick. Brown, dry, medium dense, poorly graded SAND with silt and gravel.
	Bulk					SP	FILL: Dark brown, moist, medium dense, poorly graded SAND.
5							Start hollow stem auger.
	Driven	17					
	Driven	3	▽ 19.7				Wet; very loose.
10							
	Driven	6					Loose.
	Driven	8	20.2			SP	BAY DEPOSITS: Gray, wet, loose, poorly graded SAND.
15							
	Driven	15					Medium dense.
	Driven	39					Dense.
20							

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
20							SP	BAY DEPOSITS: (Continued) Gray, wet, dense, poorly graded SAND.
		◣	34					
25								
		◣	68					Very dense.
30								
		◣	14					Medium dense.
35								
		◣	5	22.5				Loose.
40								

DATE DRILLED 6/29/17 BORING NO. B-1

GROUND ELEVATION 20'±(MSL) SHEET 2 OF 3

METHOD OF DRILLING 3" Hand Auger; 8" Hollow Stem Auger - Mobile B-53 (Expl. Geo.)

DRIVE WEIGHT 140 LBS (wireline) DROP 30 inches

SAMPLED BY GL LOGGED BY GL REVIEWED BY PCC

DATE DRILLED	6/29/17	BORING NO.	B-1
GROUND ELEVATION	20'±(MSL)	SHEET	3 OF 3
METHOD OF DRILLING	3" Hand Auger; 8" Hollow Stem Auger - Mobile B-53 (Expl. Geo.)		
DRIVE WEIGHT	140 LBS (wireline)	DROP	30 inches
SAMPLED BY	GL	LOGGED BY	GL
REVIEWED BY	PCC		

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40							SP	BAY DEPOSITS: (Continued) Gray, wet, loose, poorly graded SAND.
		■	3					Very loose.
45								
		■	27				CL	Gray, wet, very stiff, lean CLAY.
50								Total Depth = 50 ft.
								Backfilled with cement grouton 6/29/17.
								Groundwater was encountered at a depth of approximately 9 feet in the borehole during drilling.
								Notes:The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
55								Groundwater may rise to a level higher than that measured in the borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
60								

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403062001 | 7/17

APPENDIX B

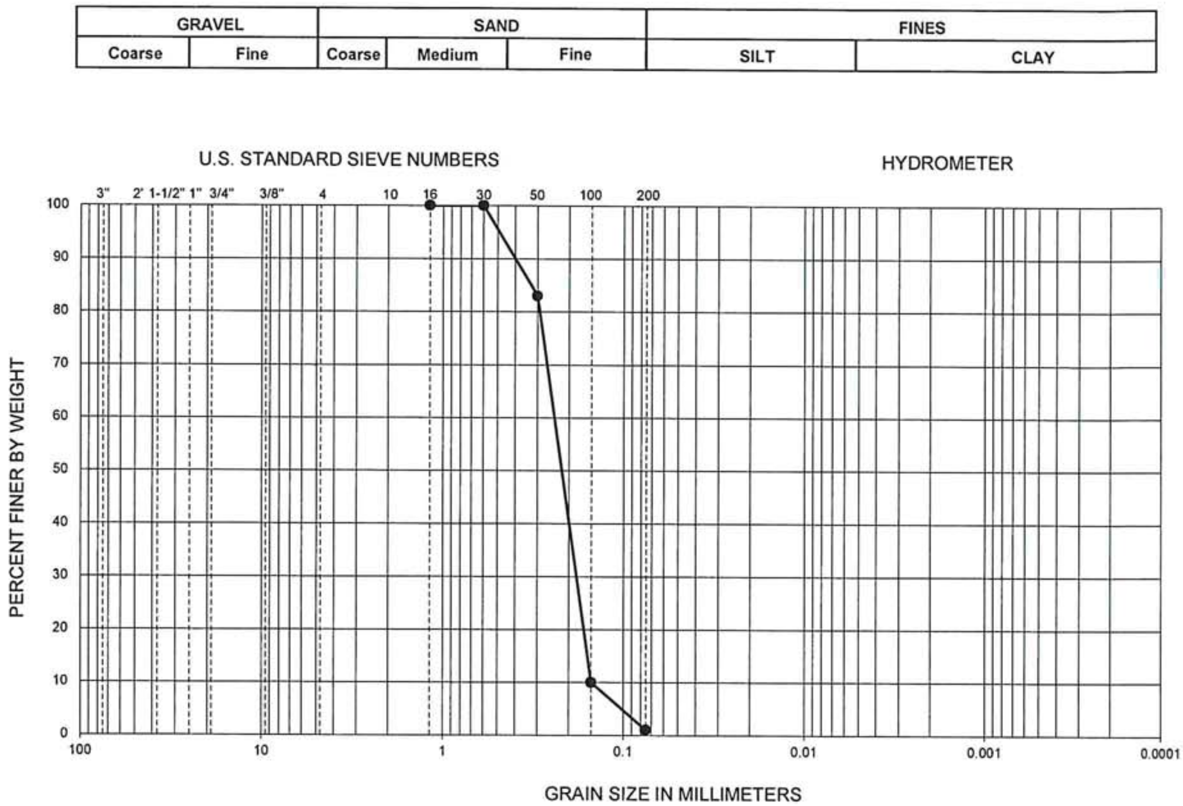
LABORATORY TESTING

Classification
Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488-00. Soil classifications are indicated on the logs of the exploratory borings in Appendix B.

Moisture Content
The moisture content of samples obtained from the exploratory boring was evaluated in accordance with ASTM D 2216. The test results are presented on the boring log in Appendix A.

Gradation Analysis
A gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain size distribution curves are shown on Figures B-1 through B-3. The test results were utilized in evaluating the soil classification in accordance with the Unified Soil Classification System (USCS).

Soil Corrosivity Tests
Soil pH, and resistivity tests were performed on a representative sample in general accordance with California Test (CT) 643. The soluble sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-4.



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	8.5-10.0	--	--	--	0.15	0.18	0.24	1.6	0.9	1	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-1

GRADATION TEST RESULTS

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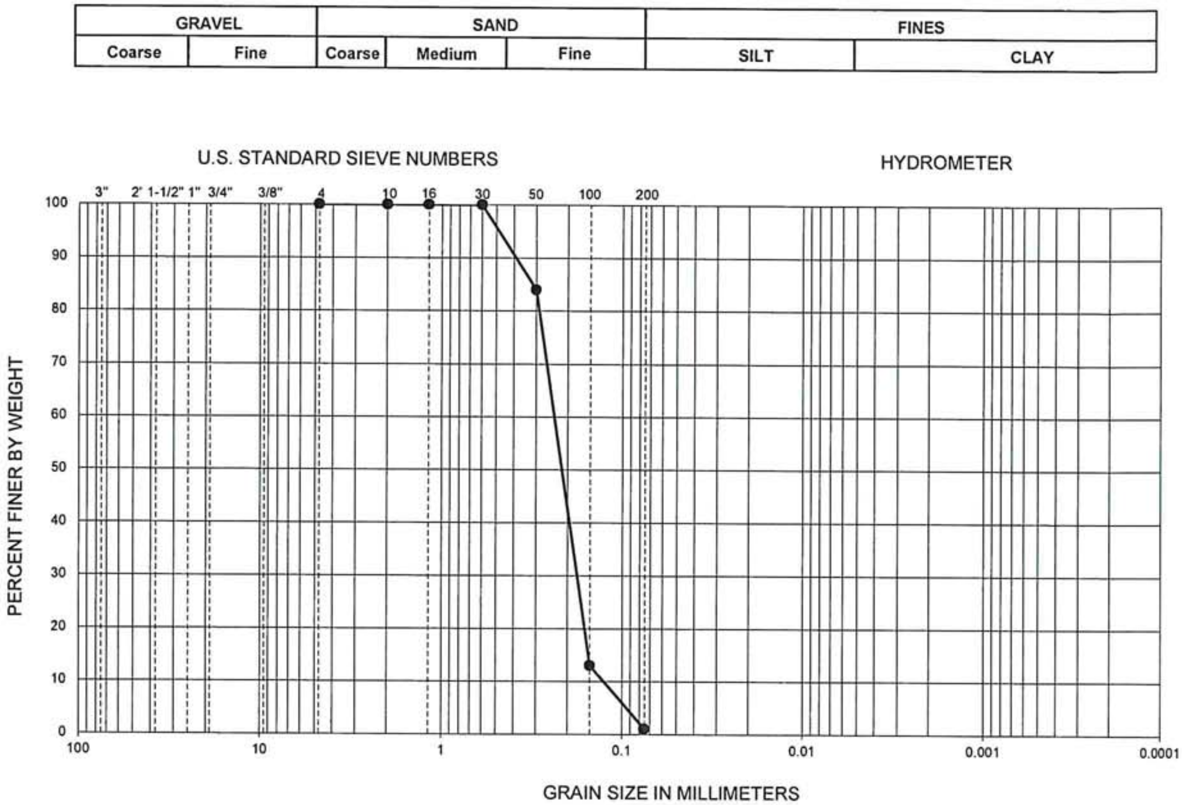

Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	13.5-15.0	--	--	--	0.13	0.18	0.24	1.8	1.0	1	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-2

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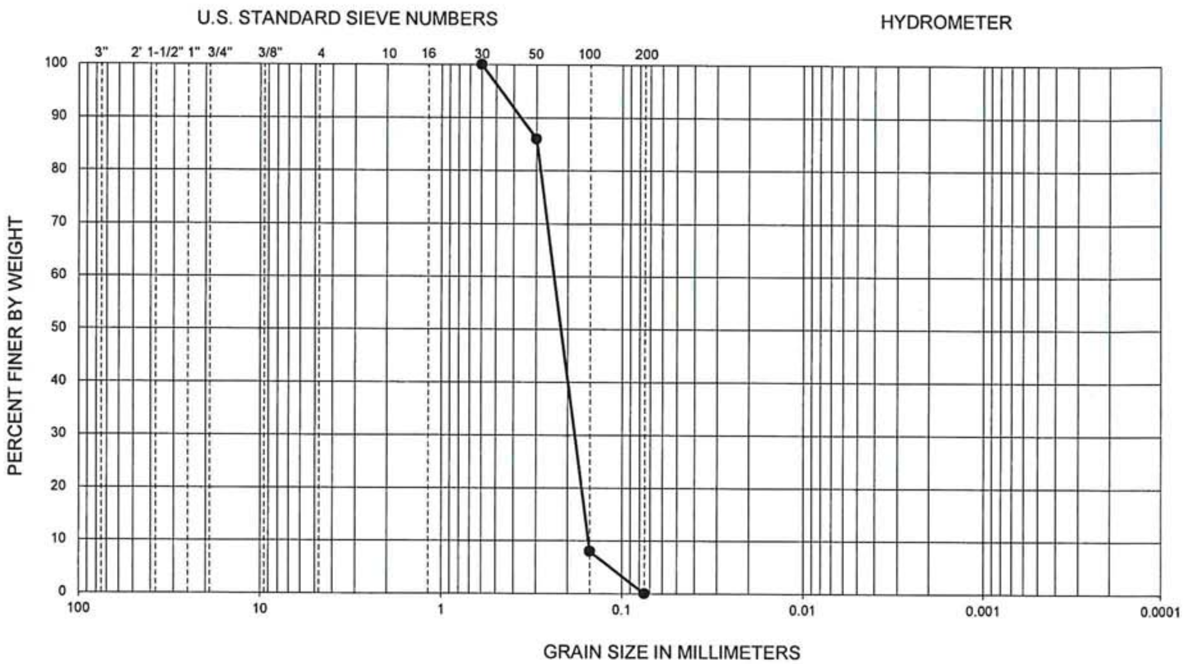
GRADATION TEST RESULTS

1088 HOWARD STREET
SAN FRANCISCO, CALIFORNIA

403062001 7/17

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	38.5-40.0	--	--	--	0.16	0.18	0.25	1.6	0.8	0	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-3

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GRADATION TEST RESULTS

1088 HOWARD STREET
SAN FRANCISCO, CALIFORNIA

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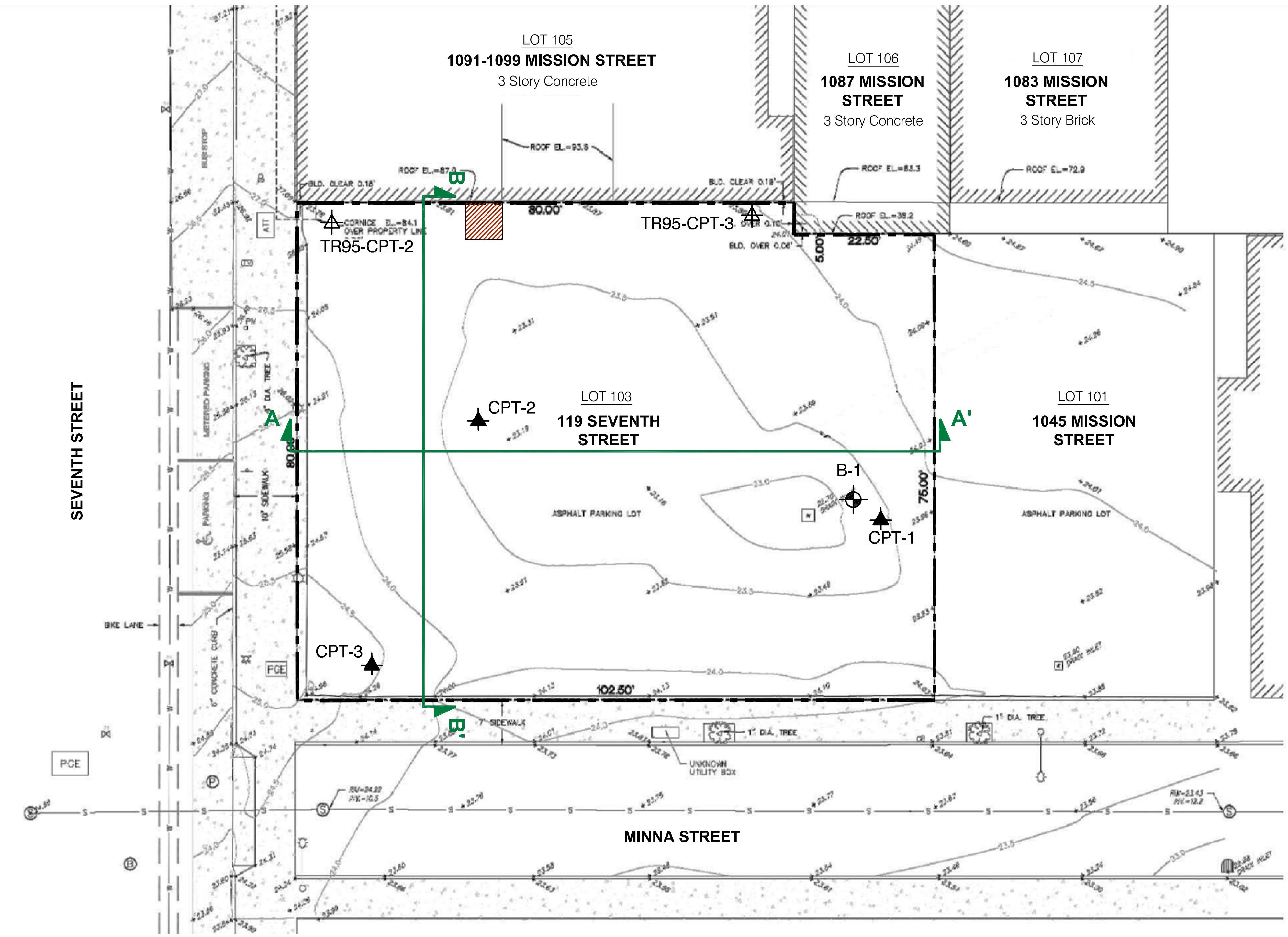


EXPLANATION

- B-1 — Approximate location of test boring by Rockridge Geotechnical, Inc., October 2012
- CPT-1 — Approximate location of cone penetration test by Rockridge Geotechnical, Inc.
- TR95-CPT-2 — Approximate location of cone penetration test by Treadwell & Rollo, Inc., 1995
- Approximate location of test pit excavated by Seton Pacific Construction in August 2014
- A — A' Idealized subsurface profiles

0 — 20 Feet
Approximate scale

Reference: Base map from a drawing titled "Plot Plan, 119 Seventh Street", by Heller Manus Architects, dated May 17, 2012.

119 7TH STREET
San Francisco, California

SITE PLAN

Date 10/15/14	Project No. 12-453	Figure 2

ROCKRIDGE GEOTECHNICAL

PROJECT: **119 7TH STREET**
San Francisco, California

Log of Boring B-1

Boring location:	See Site Plan, Figure 2
Date started:	10/2/12
Date finished:	10/2/12
Drilling method:	Rotary Wash
Hammer weight/drop:	140 lbs./30 inches
Hammer type:	Automatic
Sampler:	Sprague & Henwood (S&H), Standard Penetration Test (SPT)
Logged by:	T. Williams

Approximate Ground Surface Elevation: 24 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					1.5-inch asphalt concrete						
0.1					3-inch aggregate base						
0.4				CL	CLAY with SAND (CL) yellow-brown, stiff, moist, occasional fine gravel (FILL)						
3–4	BULK										
5–6.5	S&H	2, 3, 15	13		olive-brown					26.6	89
6					Concrete rubble						
6.5–7	SPT	50/3"	60/3"								
7.5					begin rotary-wash drilling						
8.5				SP	SAND (SP) olive-brown, loose, wet, fine-grained sand, trace of fines Particle Size Distribution, see Appendix B						
10–11.5	SPT	1, 2, 3	6						2	18.9	
15–16.5	SPT	3, 6, 5	13		medium dense						
20–21.5	SPT	3, 4, 6	12								
23				SP-SM	SAND with SILT (SP-SM) gray-brown, medium dense, wet, fine-grained sand						
25–26.5	SPT	3, 6, 8	17						6	21.7	
28				SP-SM	SAND with SILT (SP-SM) gray, medium dense to dense, wet, fine-grained sand						

LABORATORY TEST DATA

ROCKRIDGE GEOTECHNICAL

Project No.: 12-453

Figure: A-1a

PROJECT:	119 7TH STREET San Francisco, California	Log of Boring B-1 PAGE 2 OF 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
31	SPT		7 12 13	30	SP-SM	SAND with SILT (SP-SM) (continued) Particle Size Distribution, see Appendix B				6	21.1	
32												
33						SAND with SILT (SP-SM) gray, dense, wet, trace organics						
34												
35												
36	SPT		8 12 14	31	SP-SM							
37												
38												
39						SAND (SP) gray, medium dense, wet, fine-grained sand, trace of fines						
40												
41	S&H		10 24 15	27		Particle Size Distribution, see Appendix B				3	20.0	
42					SP							
43												
44												
45						CLAYEY SAND (SC) yellow-brown, medium dense, moist, fine-grained sand						
46	SPT		5 5 6	13						22	20.7	
47												
48												
49												
50						dense						
51	SPT		11 18 17	42								
52					SC							
53												
54												
55												
56												
57												
58												
59												
60												

PROJECT: **119 7TH STREET**
San Francisco, California

Log of Boring B-1

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
61	SPT		9 18 29	56	SC	CLAYEY SAND (SC) (continued) gray-brown, very dense, with less fines						
67						trace gravel						
68					SP	SAND (SP) gray-brown, very dense, wet, fine-grained sand						
70	SPT		24 50/6"	60/6"								
78					CL	CLAY with SAND (CL) olive-gray, very stiff, wet						
81	S&H		9 13 14	19								
84					SC	CLAYEY SAND (SC) olive-gray, very dense, wet, fine-grained sand						
90												

ROCKRIDGE GEOTECHNICAL

Project No.: 12-453

Figure: A-1c

PROJECT:	119 7TH STREET San Francisco, California	Log of Boring B-1 PAGE 4 OF 4

DEPTH (feet)	SAMPLES: Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
	SPT		28 50/6"	60/6"	SC	CLAYEY SAND (SC) (continued)						
91												
92												
93												
94												
95												
96												
97												
98												
99												
100												
101												
102												
103												
104												
105												
106												
107												
108												
109												
110												
111												
112												
113												
114												
115												
116												
117												
118												
119												
120												

Boring terminated at a depth of 91 feet below ground surface.
Boring backfilled with cement grout.
Groundwater level obscured by rotary wash drilling method.

[1] S&H 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] Elevations based on San Francisco City datum.

ROCKRIDGE GEOTECHNICAL

Project No.:	Figure:
12-453	A-1d

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	Range of Grain Sizes: Grain Size in Millimeters
Boulders	Above 12"	Above 305
Cobbles	12" to 3"	305 to 76.2
Gravel	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	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 Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. 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

S&H — Sprague & Henwood split-barrel 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

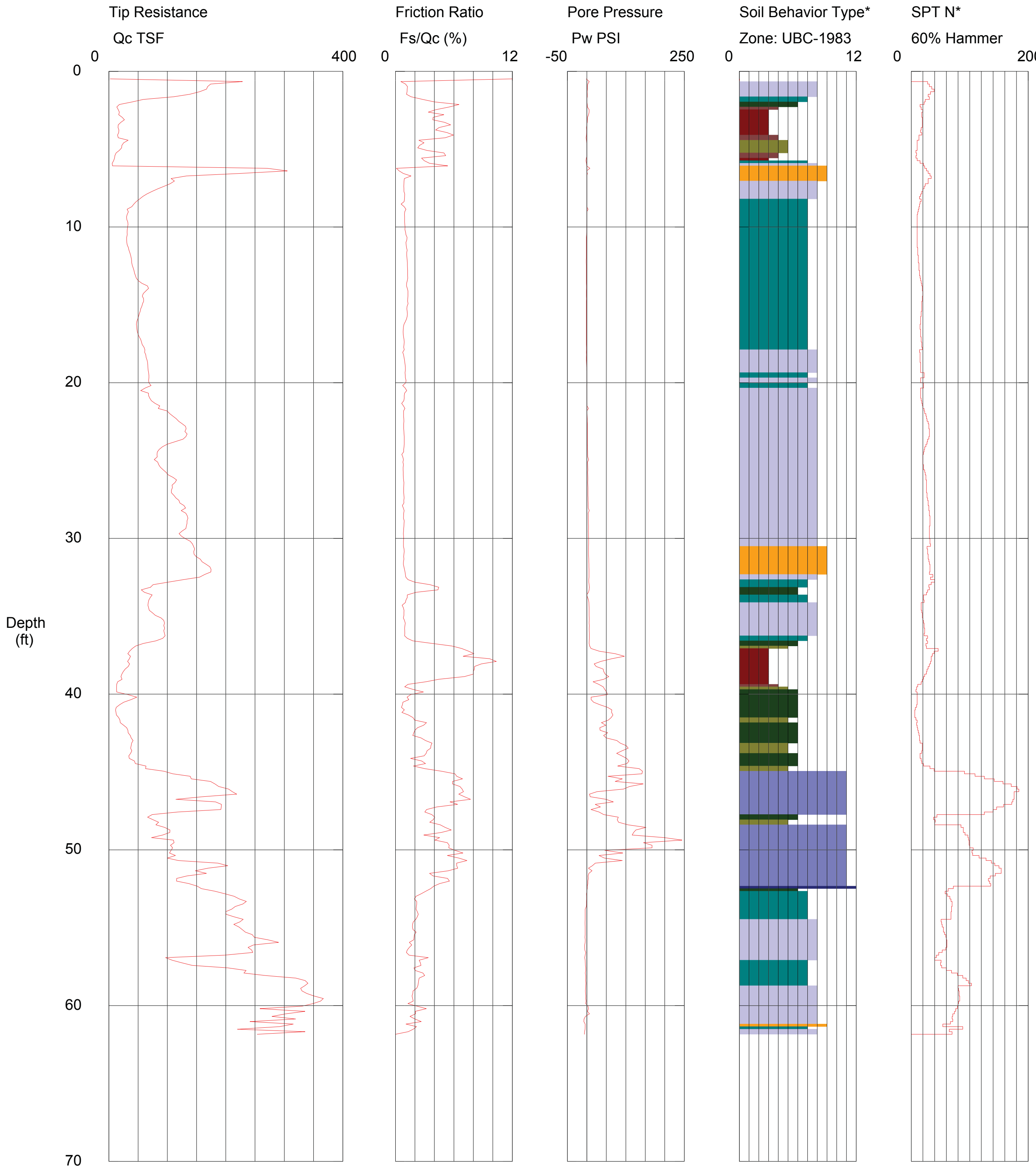
ST — Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

119 7TH STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CLASSIFICATION CHART

Date 10/09/12 | Project No. 12-453 | Figure A-2



Maximum Depth = 61.84 feet

Depth Increment = 0.164 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

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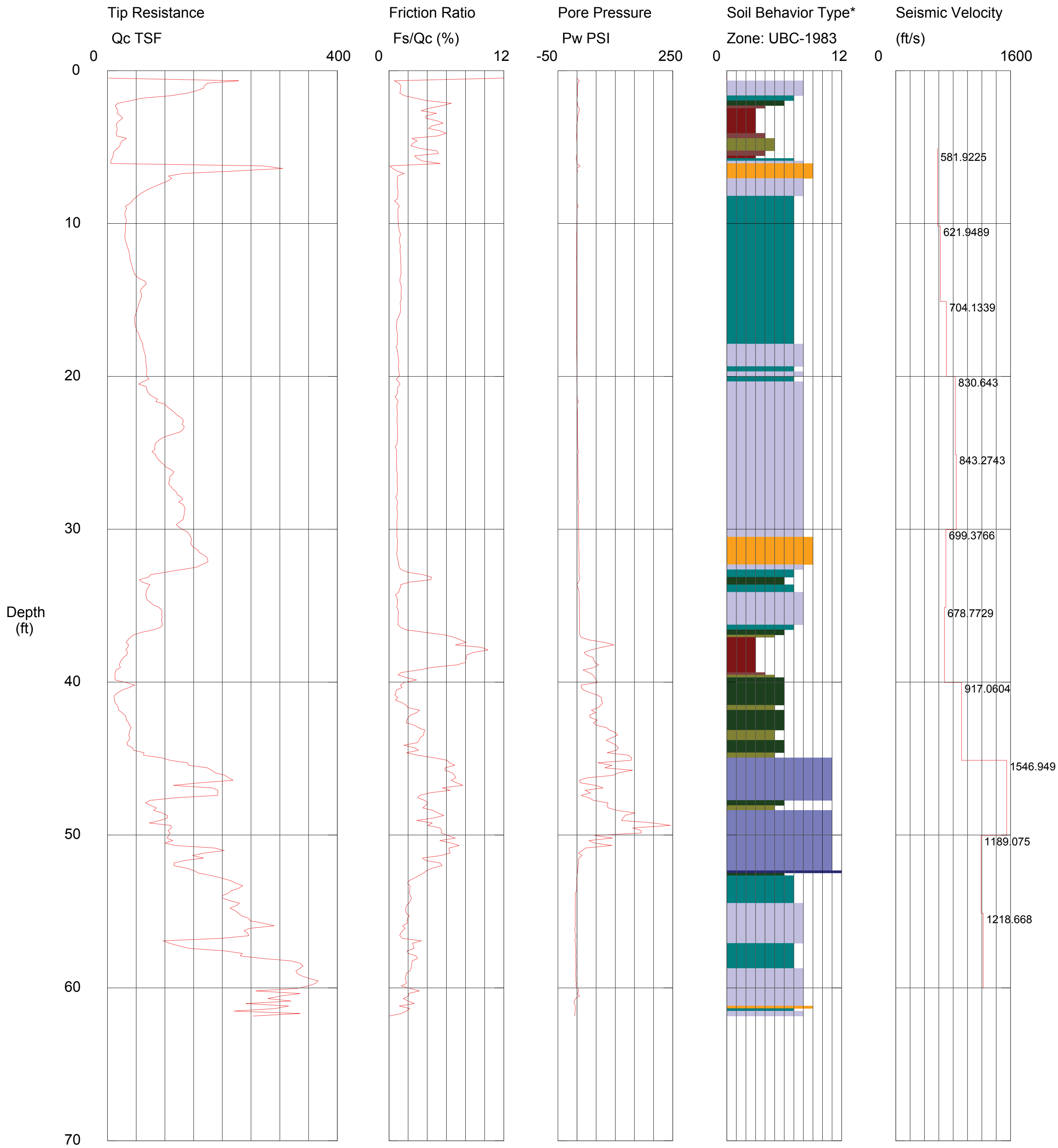
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 11/23/12	Project No. 12-453	Figure A-3



Maximum Depth = 61.84 feet

Depth Increment = 0.164 feet

1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

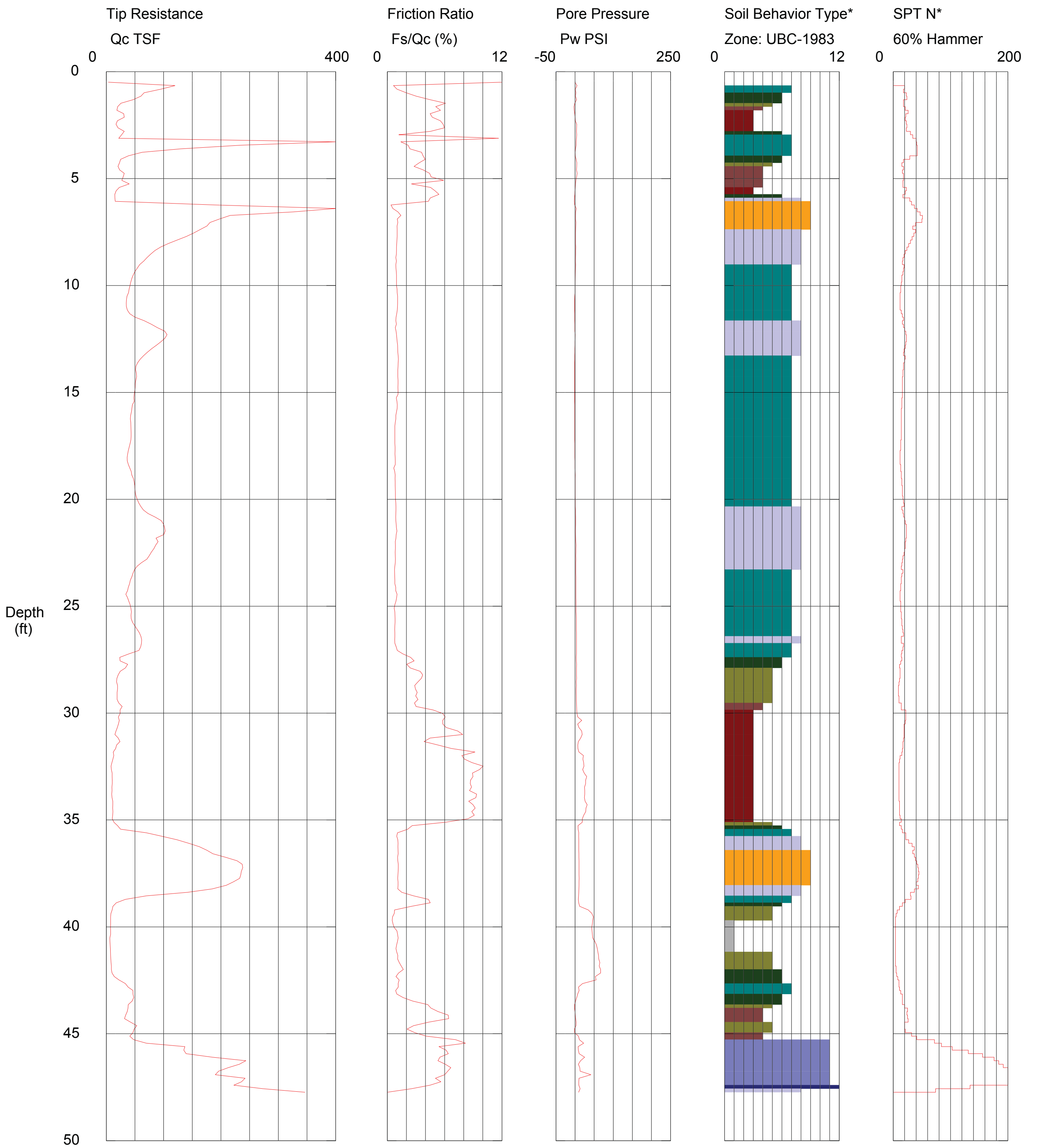
*Soil behavior type and SPT based on data from UBC-1983

119 7TH STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1, SEISMIC CONE

Date 11/23/12 | Project No. 12-453 | Figure A-4



Maximum Depth = 47.74 feet

Depth Increment = 0.164 feet

1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

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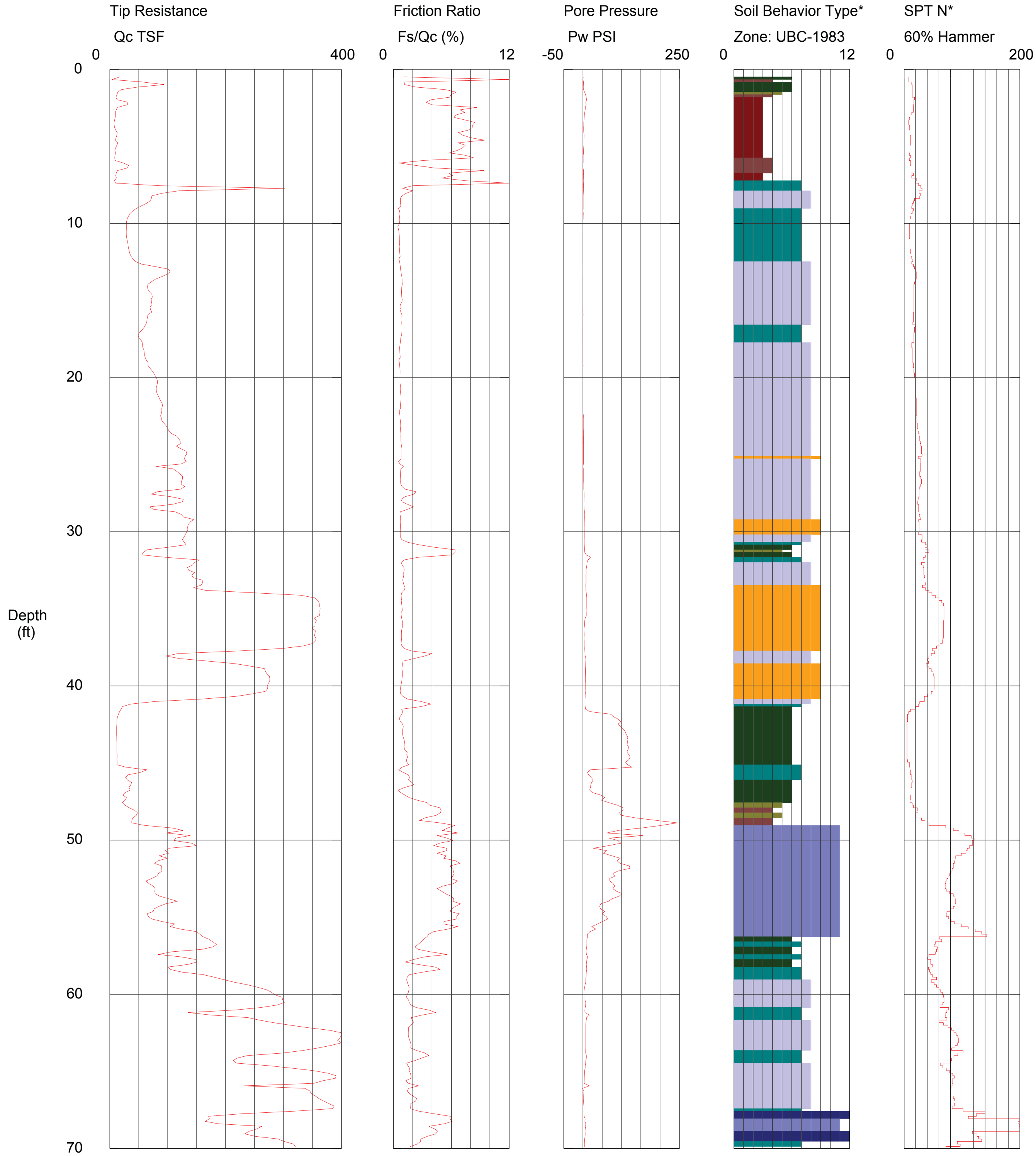
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-2

Date 11/23/12 | Project No. 12-453 | Figure A-5



Maximum Depth = 70.54 feet

Depth Increment = 0.164 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

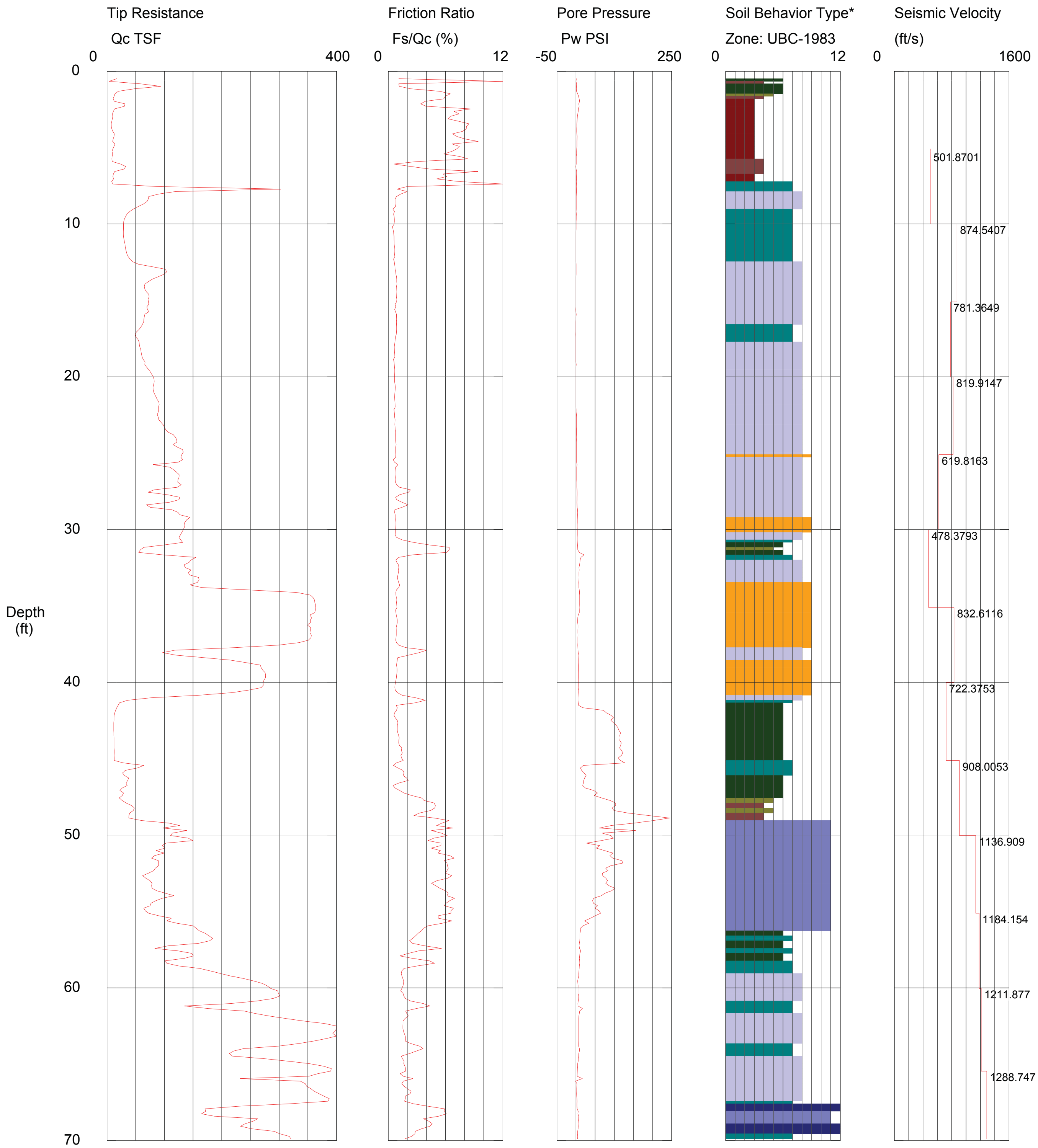
*Soil behavior type and SPT based on data from UBC-1983

119 7TH STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 11/23/12	Project No. 12-453	Figure A-6



Maximum Depth = 70.54 feet

Depth Increment = 0.164 feet

1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

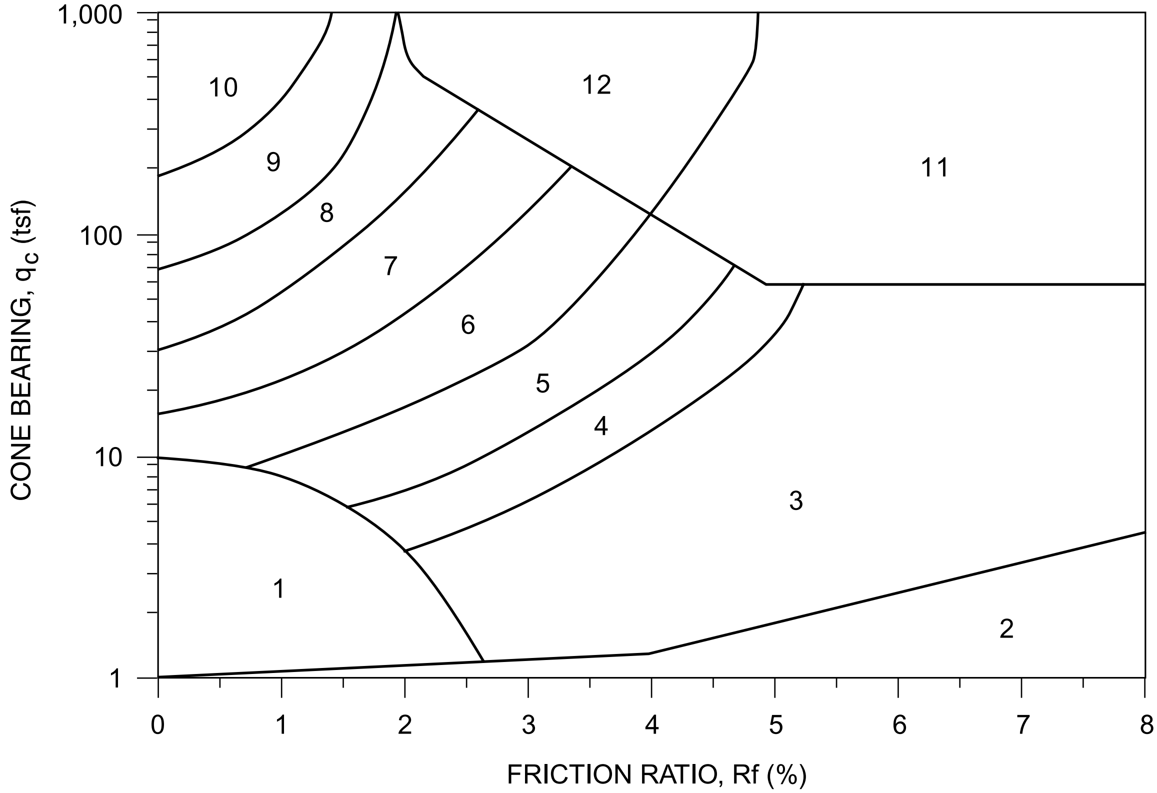
*Soil behavior type and SPT based on data from UBC-1983

119 7TH STREET
San Francisco, California

ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3, SEISMIC CONE

Date 11/23/12 | Project No. 12-453 | Figure A-7



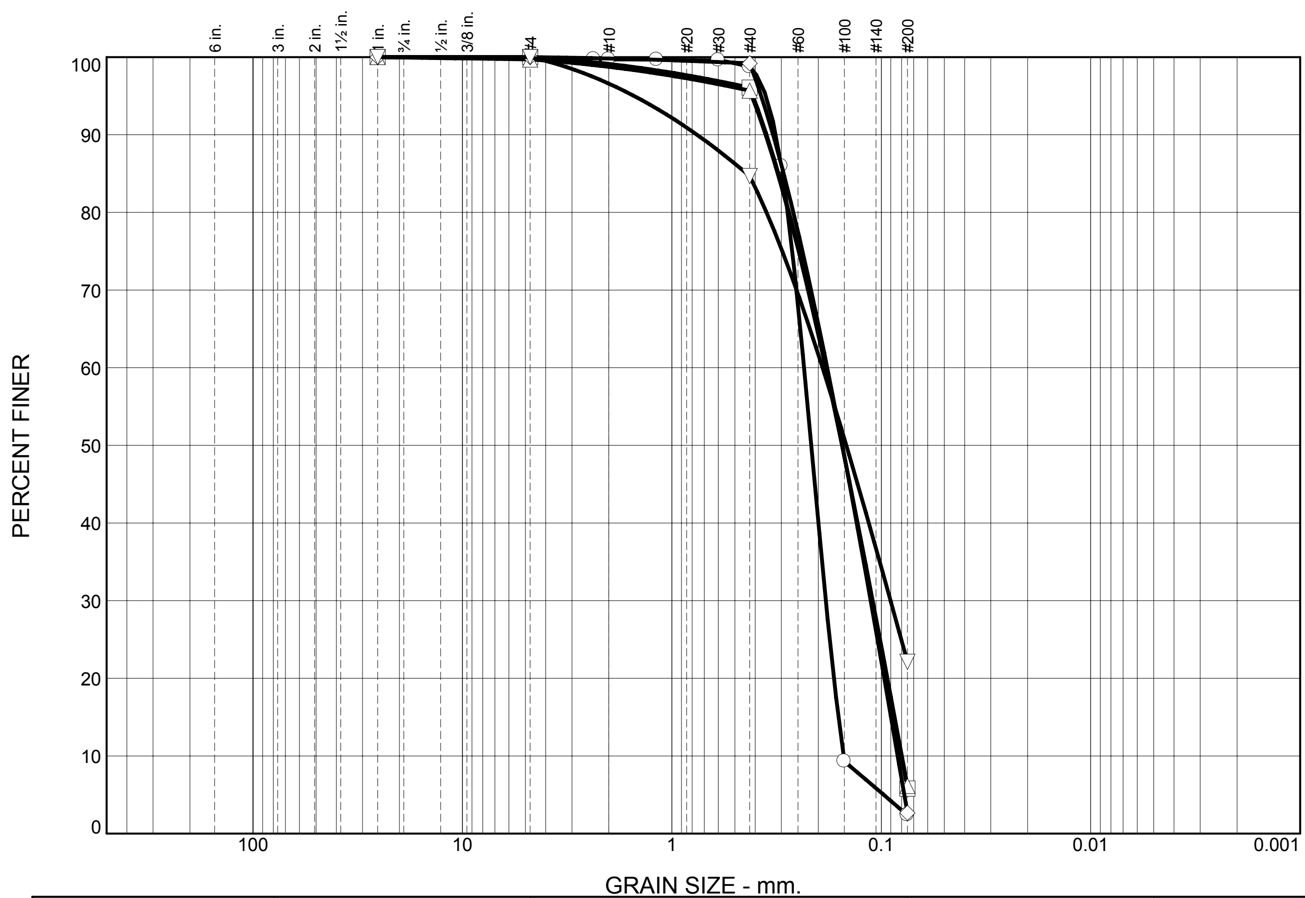
ZONE	q_c/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for $q_c \leq 9$ tsf)	Sensitive Fine-Grained
2	1	15 (10 for $q_c \leq 9$ tsf)	Organic Material
3	1	15 (10 for $q_c \leq 9$ tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
8	4	---	SAND to SILTY SAND
9	5	---	SAND
10	6	---	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	---	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
q_c = Tip Bearing
f_s = Sleeve Friction
$Rf = f_s/q_c \times 100$ = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

References: 1. UBC, 1983, Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud $q_c \leq 9$).
Estimated from local experience (fine-grained soils $q_c > 9$).

119 7TH STREET San Francisco, California	CLASSIFICATION CHART FOR CONE PENETRATION TESTS		
ROCKRIDGE GEOTECHNICAL	Date 11/28/12	Project No. 12-453	Figure A-8



	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0.0	0.1	97.5	2.4	
□	0.0	0.0	94.2	5.8	
△	0.0	0.3	93.6	6.1	
◇	0.0	0.0	97.3	2.7	
▽	0.0	0.0	77.9	22.1	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1		10.0 & 15.0'	SAND (SP), olive-brown	SP
□	B-1		25.0'	SAND with SILT (SP-SM), gray-brown	SP-SM
△	B-1		30.0'	SAND with SILT (SP-SM), gray	SP-SM
◇	B-1		40.0'	SAND (SP), gray	SP
▽	B-1		45.0'	CLAYEY SAND (SC), yellow-brown	SC

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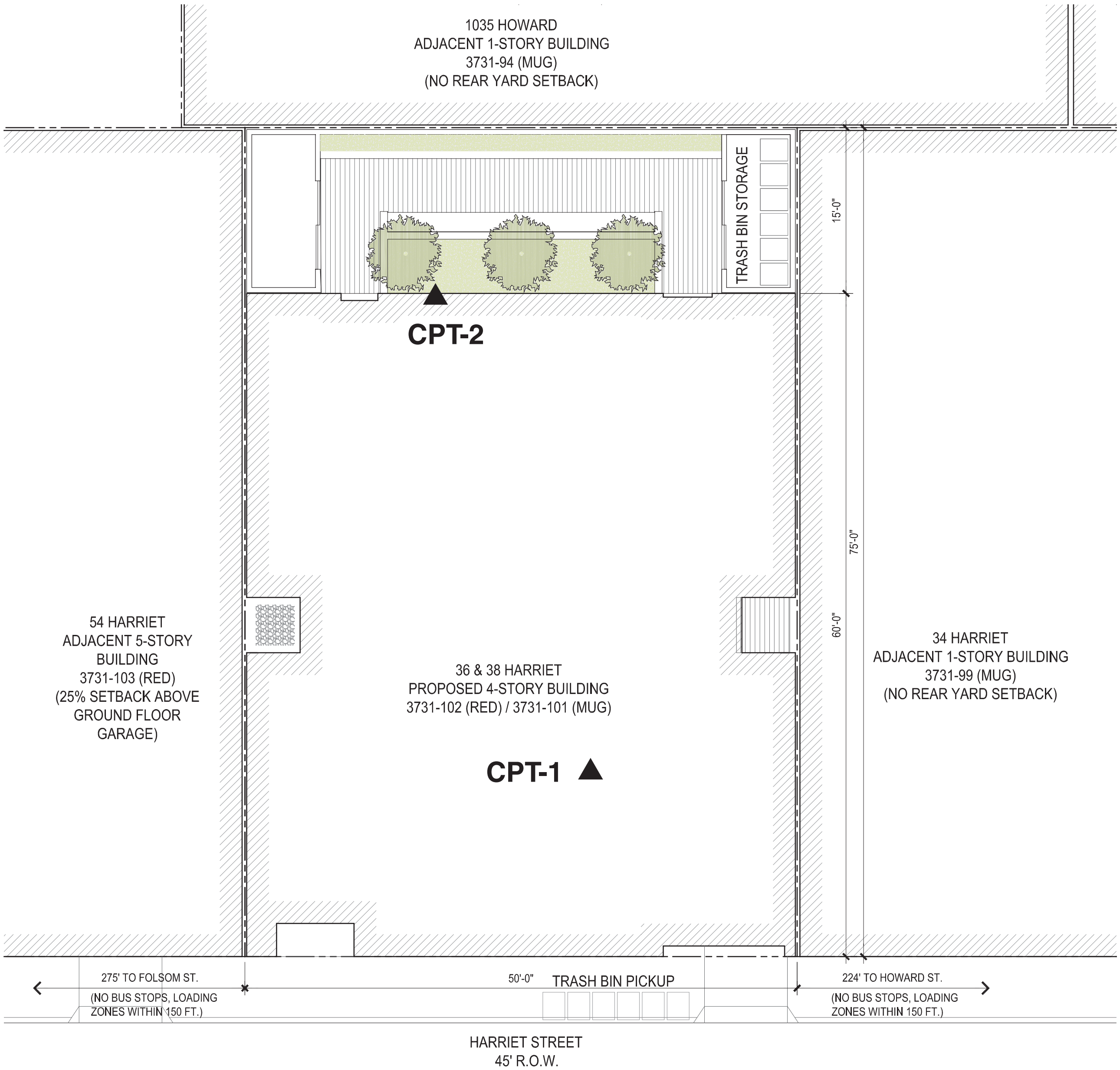
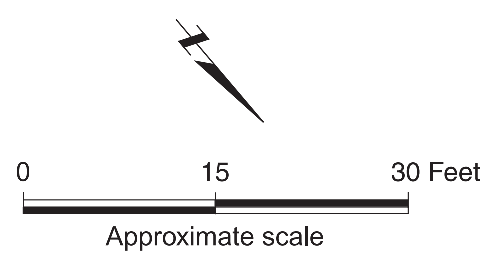
PARTICLE SIZE DISTRIBUTION REPORT

Date 11/23/12 | Project No. 12-453 | Figure B-1



EXPLANATION

CPT-1 ▲ Approximate location of cone penetration test by Rockridge Geotechnical, February 2010



Base map: "Zoning Information, Sheet A0.0" by Trachtenberg Architects, dated 11/18/2009.

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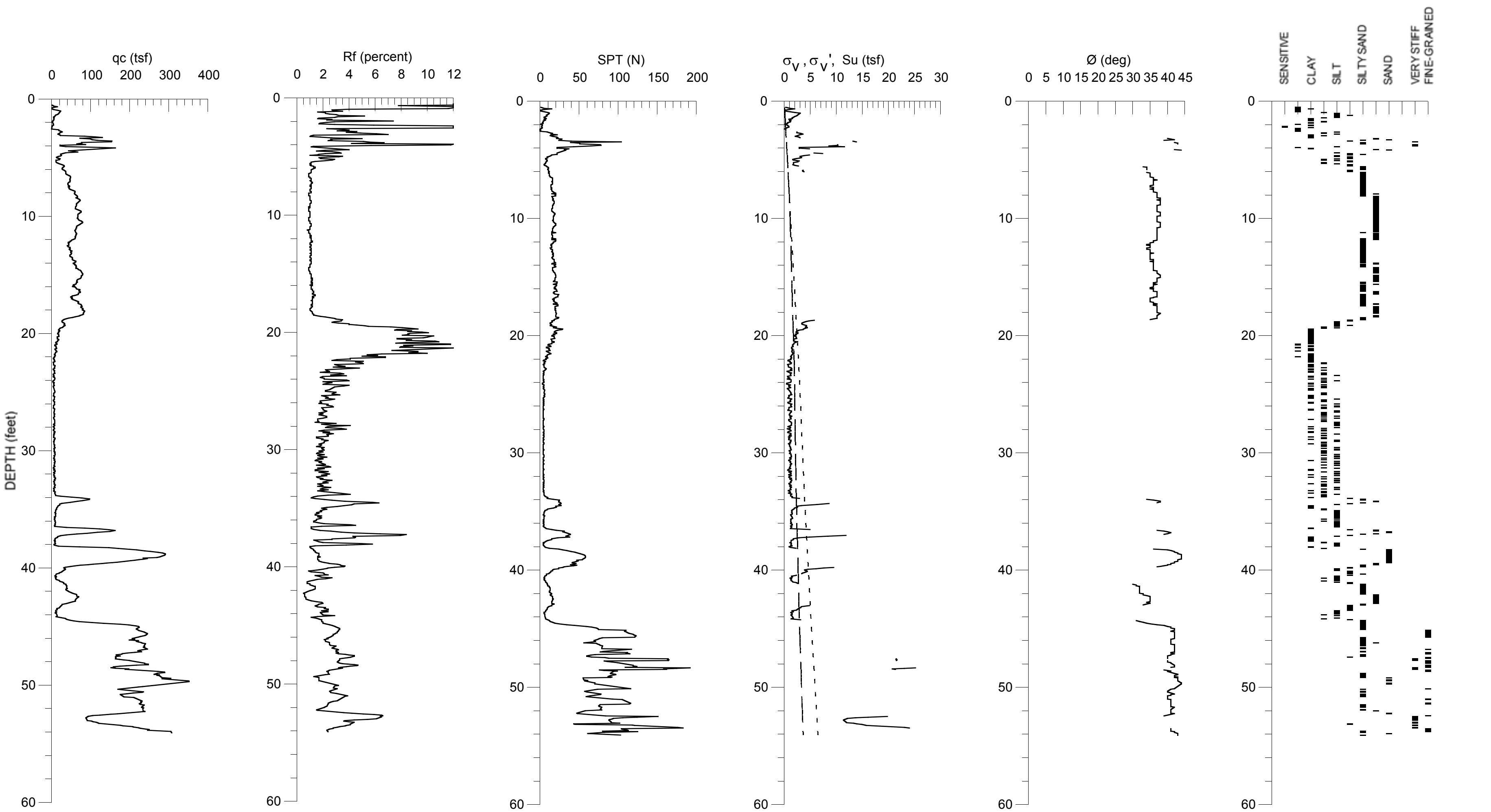
ROCKRIDGE GEOTECHNICAL

SITE PLAN

Date 02/11/10	Project No. 10-204	Figure 2



Effective vertical stress, σ_v'

Total vertical stress, σ_v

Undrained Shear Strength, s_u

Terminated at 54 feet
Groundwater encountered at 8.4 feet.
Date performed: 2/3/2010.

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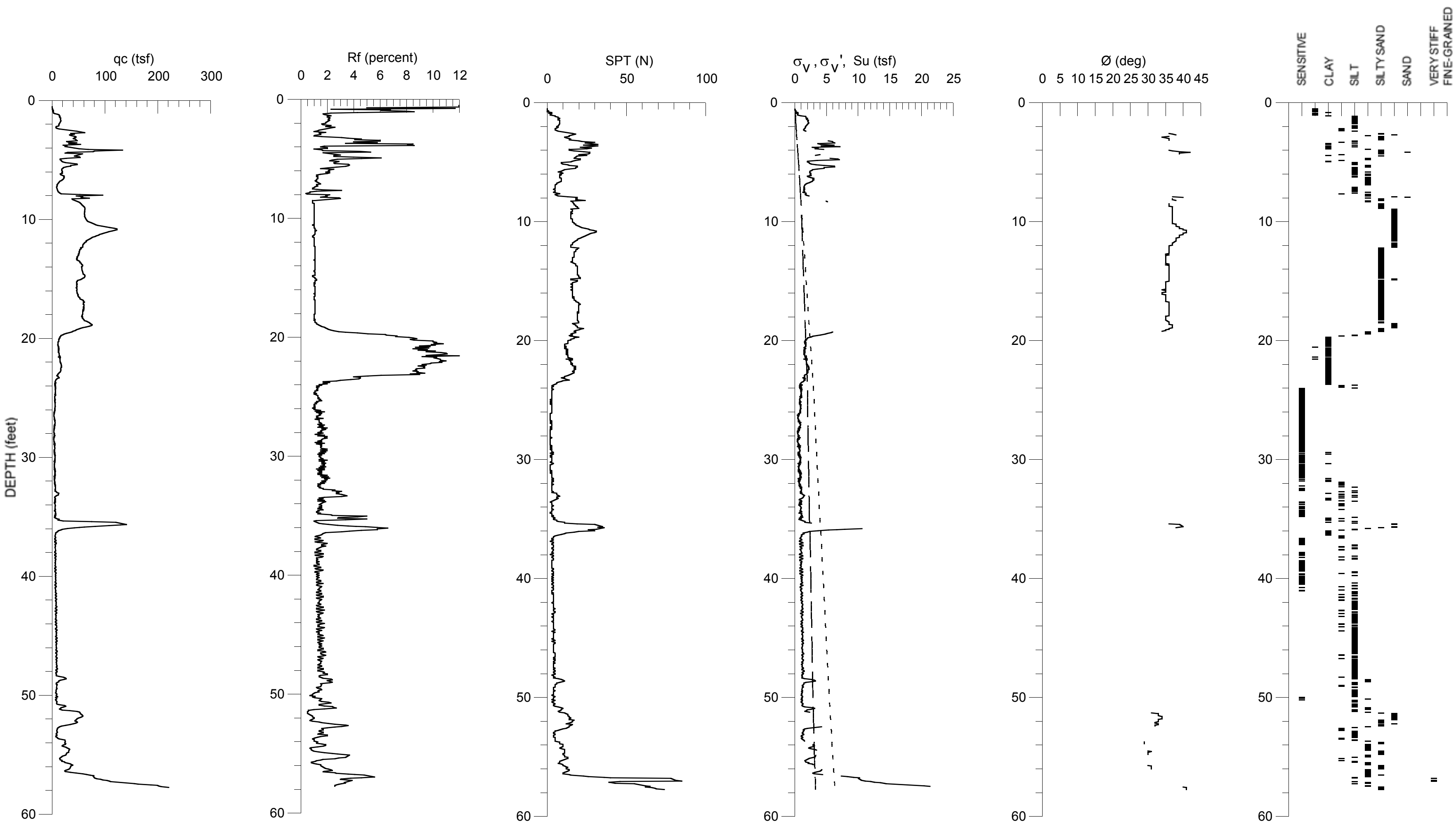
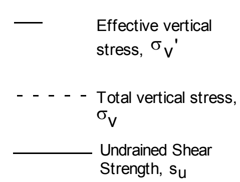
CONE PENETRATION TEST RESULTS
CPT-1

Date 02/11/10	Project No. 10-204	Figure A-1





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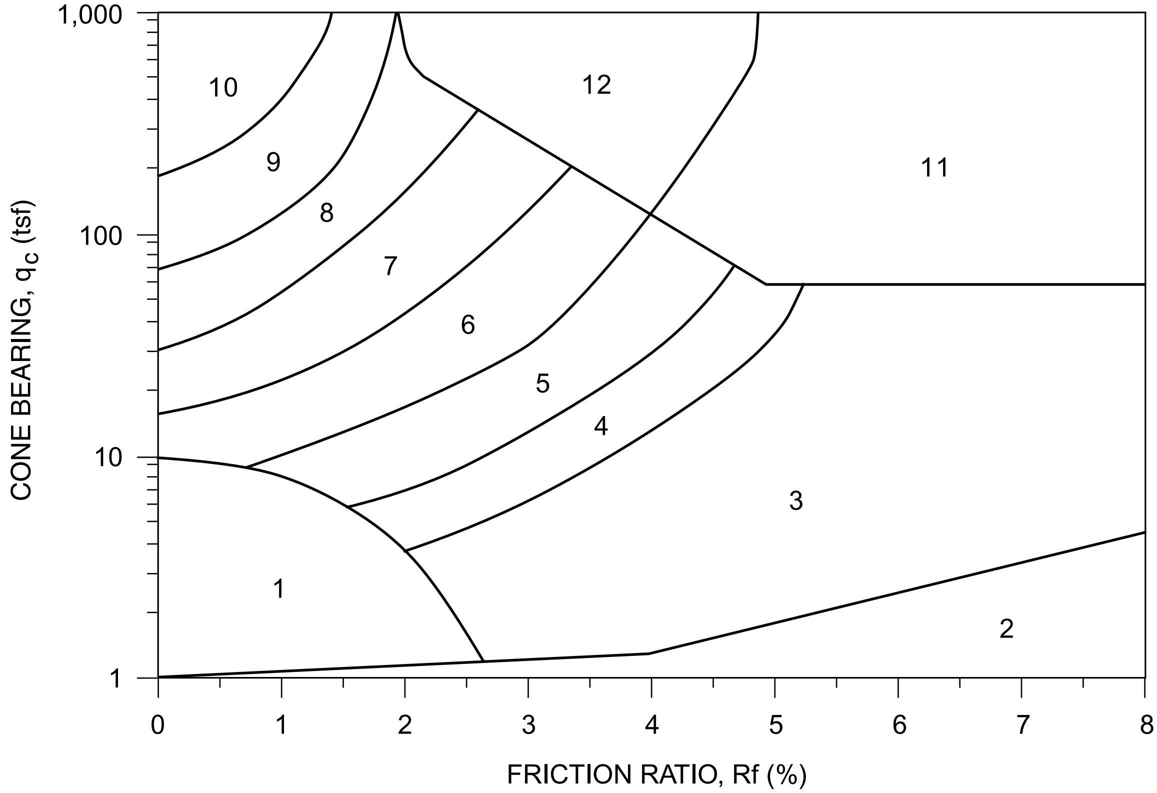
CONE PENETRATION TEST RESULTS
CPT-2

Date 02/11/10	Project No. 10-204	Figure A-2

Terminated at 57.7 feet
Groundwater encountered at 8.7 feet.
Date performed: 2/4/2010.



ZONE	q_c/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2	15 (10 for $q_c \le 9$ tsf)	Sensitive Fine-Grained
2	1	15 (10 for $q_c \le 9$ tsf)	Organic Material
3	1	15 (10 for $q_c \le 9$ tsf)	CLAY
4	1.5	15	SILTY CLAY to CLAY
5	2	15	CLAYEY SILT to SILTY CLAY
6	2.5	15	SANDY SILT to CLAYEY SILT
7	3	---	SILTY SAND to SANDY SILT
8	4	---	SAND to SILTY SAND
9	5	---	SAND
10	6	---	GRAVELLY SAND to SAND
11	1	15	Very Stiff Fine-Grained (*)
12	2	---	SAND to CLAYEY SAND (*)

(*) Overconsolidated or Cemented
q_c = Tip Bearing
f_s = Sleeve Friction
$Rf = f_s/q_c \times 100$ = Friction Ratio

Note: Testing performed in accordance with ASTM D3441.

References: 1. Robertson, 1986, Olsen, 1988.
2. Bonaparte & Mitchell, 1979 (young Bay Mud $q_c \le 9$).
Estimated from local experience (fine-grained soils $q_c > 9$).

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ROCKRIDGE GEOTECHNICAL

CLASSIFICATION CHART FOR CONE PENETRATION TESTS

Date 02/11/10	Project No. 10-204	Figure A-3