ROCKRIDGE
GEOTECHNICAL

Prepared for **Mercy Housing California**

GEOTECHNICAL INVESTIGATION
PROPOSED AFFORDABLE HOUSING BUILDING
555 KELLY AVENUE
HALF MOON BAY, CALIFORNIA

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January 29, 2024
Project No. 23-2527

ROCKRIDGE
GEOTECHNICAL

January 29, 2024
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Ms. Emma Wynkoop
Mercy Housing California
1256 Market Street
San Francisco, California 94102

Subject: Geotechnical Investigation
Proposed Affordable Housing Building
555 Kelly Avenue
Half Moon Bay, California

Dear Ms. Wynkoop:

We are pleased to present the results of our geotechnical investigation for the proposed affordable housing building to be constructed at 555 Kelly Avenue in Half Moon Bay, California. Our investigation was performed in accordance with our Consulting Services Agreement, dated November 16, 2023.

The subject property consists of two adjacent parcels (the southern third of 535 Kelly Avenue and 555 Kelly Avenue) on the northern side of Kelly Avenue, between its intersections with the Cabrillo Highway and Church Street. The two parcels combined have maximum plan dimensions of about 120 feet by 150 feet. The parcels are currently occupied by a single-family residence with a basement, a detached garage, and a surface parking lot. The site is bordered by a shopping center surrounded by asphalt-paved drive aisles to the east, the remaining two-thirds of 535 Kelly Avenue consisting of a community center, skate park, and the Half Moon Bay Police Department to the north, a church surrounded by asphalt-paved drive aisles and landscaping to the east, and Kelly Avenue to the south.

Plans are to demolish the single-family residence, detached garage, and surface parking lot and construct an at-grade, five-story affordable housing building targeting local senior farmworkers. The building will consist of four stories of wood-framed residential units above a one-story, at-grade concrete podium. A parking garage and farmworker resource center will occupy the ground floor.

From a geotechnical standpoint, we conclude the proposed building 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 issues affecting the proposed development includeproviding adequate foundation support for the proposed building. We conclude the building may be supported on spread footings or a reinforced concrete mat foundation.

The recommendations contained in our report are based on 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 preparation and foundation installation, during which time we may make changes to our recommendations if deemed necessary.

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

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC.

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Enclosure

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GEOTECHNICAL INVESTIGATION
PROPOSED AFFORDABLE HOUSING BUILDING
555 KELLY AVENUE
HALF MOON BAY, CALIFORNIA

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical, Inc. (Rockridge) for the proposed affordable housing building to be constructed at 555 Kelly Avenue in Half Moon Bay, California. The subject property consists of two adjacent parcels (APN 056-120-200 and the southern third of APN 056-150-220) on the northern side of Kelly Avenue, between its intersections with the Cabrillo Highway and Church Street, as shown in Figure 1 (Site Location Map).

The two parcels combined have maximum plan dimensions of about 120 feet by 150 feet, as shown in Figure 2 (Site Plan). The parcels are currently occupied by a single-family residence with a basement, a detached garage, and a surface parking lot. The site is bordered by a shopping center surrounded by asphalt-paved drive aisles to the east, the remaining two-thirds of 535 Kelly Avenue consisting of a community center, skate park, and the Half Moon Bay Police Department to the north, a church surrounded by asphalt-paved drive aisles and landscaping to the east, and Kelly Avenue to the south.

Plans are to demolish the single-family residence, detached garage, and surface parking lot and construct an at-grade, five-story affordable housing building targeting local senior farmworkers. The building will consist of four stories of wood-framed residential units above a one-story, at-grade concrete podium. A parking garage and farmworker resource center will occupy the ground floor. Other improvements will include landscaping around the building and new utility connections and resurfacing the parking lot to the north of the proposed building.

2.0 SCOPE OF SERVICES

Our investigation was performed in accordance with our Consulting Services Agreement, dated November 16, 2023. Our scope of services consisted of exploring subsurface conditions at the site, performing laboratory tests on selected soil samples, and performing engineering analyses to develop conclusions and recommendations regarding:

- soil and groundwater conditions at the site
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- design groundwater table
- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlement
- lateral earth pressure for design of site retaining walls and below-grade walls (i.e., elevator pit walls)
- subgrade preparation for slab-on-grade floors and exterior concrete flatwork
- site grading and fill placement, including fill quality and compaction recommendations
- flexible and rigid pavement design
- 2022 California Building Code (CBC) site class and design spectral response acceleration parameters
- corrosivity of the near-surface soil and the potential effects on buried concrete and metal structures and foundations
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

Our field investigation consisted of performing four cone penetration tests (CPTs), drilling two test borings, and performing laboratory testing on selected soil samples. Before performing the field investigation, we filed drilling notification forms with the San Mateo County Environmental Health Services Division (SMCEHSD). We also contacted Underground Services Alert (USA) to notify them of our work, as required by law, and retained C. Cruz Sub-Surface Locators of Milpitas, California, a private utility locator, to check the CPT and boring locations to reduce the potential for encountering buried utilities during our field investigation. Details of our field investigation and laboratory testing are presented in this section.

3.1 Cone Penetration Tests

Four CPTs, designated as CPT-1 through CPT-4, were performed on December 14, 2023, by Middle Earth Geo Testing, Inc. of Hayward, California, at the approximate locations shown in Figure 2. CPT-1 was advanced to practical refusal at a depth of approximately 78 feet below the ground surface (bgs). CPT-2, CPT-3, and CPT-4 were each advanced to the target depth of 50 feet bgs.

Middle Earth Geo Testing, Inc. advanced the CPTs by hydraulically pushing a 1.7-inch-diameter cone-tipped probe with a projected area of 15 square centimeters into the ground using a 25-ton truck rig. The cone-tipped probe measured tip resistance, and the friction sleeve behind the cone tip measured frictional resistance. Electrical strain gauges within the cone measured soil parameters at a recording interval of approximately 2 inches for the advanced depth. Soil data, including tip resistance, frictional resistance, and pore water pressure, were recorded by a computer while the test was conducted. A computer processed accumulated data to provide engineering information, such as the soil behavior types (Robertson, 2010) and approximate strength characteristics of the soil encountered. The CPT logs showing tip resistance, friction ratio, pore pressure, and correlated soil behavior type with depth are presented in Appendix A in Figures A-1a through A-4. Groundwater was measured in all CPTs, and the depth of the groundwater and the measurement method are noted in the CPT logs. Shear wave velocities of the soil were measured at approximately 5-foot intervals while advancing CPT-1. A plot of the measured shear wave velocities at each interval is presented in Figure A-1b.

Upon completion, the CPT holes were backfilled with cement grout in accordance with SMCEHSD requirements and patched with concrete.

3.2 Test Borings

Two test borings, designated as Boring B-1 and Boring B-2, were drilled on December 21, 2023, by Exploration GeoServices, Inc of San Jose, California at the approximate locations shown in Figure 2. Borings B-1 and B-2 were drilled to depths of 40 and 50 feet bgs, respectively, using a truck-mounted drill rig equipped with 8-inch-outside-diameter hollow-stem flight augers. During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. Our field engineer noted the date and time when groundwater was encountered during drilling. The logs of the borings are presented in Figures A-5a through A-6b in Appendix A. The soil encountered in the borings was classified in accordance with the classification system presented in Figure A-7.

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; the sampler was designed to accommodate liners, but liners were not used.

The samplers were driven with a 140-pound downhole safety 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 6 inches and are presented in the boring logs. A “blow count” is defined as the number of hammer blows per 6 inches of penetration or 50 blows for 6 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, and the fact that the SPT sampler was not designed to accommodate liners. The blow counts used for this conversion were the last two blow counts. The converted SPT N-values are presented in the boring logs.

Upon completion of drilling, the boreholes were backfilled with neat cement grout in accordance with SMCEHSD requirements and patched with concrete. The soil cuttings generated by the borings were removed from the site for disposal as clean fill by Exploration Geoservices, Inc.

3.3 Laboratory Testing

We re-examined each soil sample in the office to confirm the field classification and selected representative samples for laboratory testing. Laboratory tests were performed by Inspection Services, Inc., of Berkeley, California, to measure moisture content, dry density, Atterberg limits (plasticity), and particle size distribution. Laboratory tests were performed by Project X Corrosion Engineering of Murrieta, California on two near-surface soil samples to provide data for evaluating the soil corrosivity. The results of the laboratory tests are presented in the boring logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS

The site is underlain by Holocene-age alluvium (Qha), as shown in Figure 3 (Regional Geologic Map). Alluvial deposits generally consist of a mixture of fine-grained and coarse-grained deposits deposited by rivers and streams. The results of our subsurface investigation indicate the site is underlain by about 10 to 20 feet of stiff to hard clay with variable sand content. Below about 10 to 20 feet bgs, the clay is underlain by medium dense to dense sand with variable clay and silt content interbedded with very stiff to hard clay with variable sand content to the maximum depth explored of about 73 feet bgs.

Atterberg limits tests performed on samples of the near-surface clay obtained from the borings at depths between 1.5 and 3.8 feet bgs indicate the sandy clay blanketing the site has plasticity indices (PI's) of 11 to 15 and, therefore, generally has a low to moderate expansion potential.[1] An Atterberg limits test performed on a soil sample obtained at a depth of 5.5 feet bgs in Boring B-1 resulted in a PI of 32, indicating the soil at that depth is highly expansive.

Groundwater was encountered between depths of 18 and 23 feet bgs in the CPTs and at 23 and 25 feet bgs in the borings. The groundwater encountered in the CPTs and borings was measured immediately prior to grouting. The groundwater measurements in both the CPTs and borings may not represent the stabilized groundwater level because the groundwater level may not have had enough time to stabilize before the measurements were taken.

Available historic groundwater information presented in the *Seismic Hazard Zone Report for the Half Moon Bay Quadrangle* indicates the historic high groundwater in the site vicinity is less than 10 feet bgs. To estimate the high groundwater level at the site, we reviewed available groundwater data on the State of California Water Resources Control Board GeoTracker website.[2] The nearest site with groundwater data is located at 501 Kelly Avenue, approximately

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[1] Expansive soil undergoes large volume changes with changes in moisture content (i.e., it shrinks when dried and swells when wetted).

[2] https://geotracker.waterboards.ca.gov

375 feet west of the site. Groundwater was measured between June 1992 and March 2009 between depths of 11 and 23 feet bgs.

The groundwater level at the site is expected to fluctuate several feet seasonally, depending on the amount of rainfall. Based on our review of available historic groundwater information within the site vicinity, we conclude a high groundwater level of about 10 feet should be used for the project.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity and Faulting

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, and Monte Vista-Shannon faults. These and other faults in the region are shown in Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[3] [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).

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[3] 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
San Gregorio (North)	3.0	West	7.44
Total North San Andreas (SAO+SAN+SAP+SAS)	8.8	Northeast	8.04
North San Andreas (Peninsula, SAP)	8.8	Northeast	7.38
Monte Vista - Shannon	11	East	7.14
Butano	18	South	6.93
Zayante-Vergeles (2011 CFM)	32	Southeast	7.48
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	38	Northeast	7.58
Hayward (South, HS)	38	Northeast	7.00
Hayward (North, HN)	42	Northeast	6.90
North San Andreas (Santa Cruz Mts, SAS)	50	Southeast	7.15

Damaging earthquakes have occurred along many of these faults in recorded history, as depicted in Figure 4 (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 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 Geologic Hazards

Because the 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] We used the results of our geotechnical investigation to evaluate the potential of these phenomena occurring at the project site.

5.2.1 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, San Gregorio, and Monte Vista – Shannon 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 strong to very strong ground shaking could occur at the site during a large earthquake on one of the nearby faults.

5.2.2 Ground Surface Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is 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. Therefore, we conclude the probability of fault offset at the site from a known active fault to be very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults

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[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.

previously existed; however, we conclude the probability of surface faulting, and consequently secondary ground failure from previously unknown faults, is very low.

5.2.3 Liquefaction and Liquefaction-Induced Settlement

When saturated, cohesionless soil liquefies, it experiences a temporary loss of shear strength created by a transient rise in excess pore pressure generated by a 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 has been mapped within a zone of liquefaction potential as shown in the map prepared by the California Geologic Survey (CGS) *titled Earthquake Zones of Required Investigation, Half Moon Bay Quadrangle, Official Map*, dated September 23, 2021, a portion of which is shown on Figure 5. CGS has provided recommendations for procedures and report content for site investigations performed within seismic hazard zones in Special Publication 117 (SP-117), titled *Guidelines for Evaluating and Mitigating Seismic Hazard Zones in California*, dated September 11, 2008. SP-117 recommends subsurface investigations in mapped liquefaction hazard zones be performed using rotary-wash borings and/or CPTs. We used the results of our CPTs to evaluate the potential for liquefaction to occur at the site.

Liquefaction susceptibility was assessed using the software CLiq v3.5.2.22 (GeoLogismiki, 2023). CLiq uses measured field CPT data and assesses liquefaction potential, including post-earthquake vertical settlement, given a user-defined earthquake magnitude and peak ground acceleration (PGA). We performed a liquefaction triggering analysis using the CPTs in accordance with the methodology by Boulanger and Idriss (2014). The method was modified to consider an I_B (Robertson, 2016) cutoff of 28, which is similar to an I_c of 2.65 for "young" and "normally consolidated" soils (i.e., those most susceptible to liquefaction) and consistent with local experience (Proto, 2024). We also used the relationship proposed by Zhang, Robertson, and Brachman (2002) to estimate post-liquefaction volumetric strains and corresponding ground surface settlement. Volumetric strains were modified using the methodology proposed by Çetin et al. (2009) to account for the depth of the liquefiable layers.

Our analyses were performed using the estimated historic high groundwater depth of 10 feet bgs. In accordance with the 2022 CBC, we used a peak ground acceleration of 0.94 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 Total North San Andreas Fault, as presented in Table 1.

Our liquefaction analyses indicate that there are thin (i.e., less than 2 to 3 feet thick) medium dense sand and clayey sand layers between depths of approximately 23 and 55 feet bgs that are susceptible to liquefaction during a major earthquake. Based on the results of our analyses, we estimate total “free-field” ground settlement associated with liquefaction after an MCE event generating a PGA_M of 0.94g will be up to about 1/2 inch. We estimate liquefaction-induced differential settlement will be less than 1/4 inch over a horizontal distance of 30 feet.

Ishihara (1985) presented an empirical relationship that provides criteria that can be 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 restraint, or protective surficial layer. Our analysis indicates the non-liquefiable layers are sufficiently thin such that the potential for surface manifestations of liquefaction, such as sand boils, is low.

Considering the site topography is relatively flat and the potentially liquefiable layers are not continuous, we conclude the potential of lateral spreading is very low.

5.2.4 Cyclic Densification

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

6.0 DISCUSSION AND CONCLUSIONS

From a geotechnical standpoint, we conclude the site can be developed as planned, provided the recommendations presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical concern is providing adequate foundation support for the proposed building. These and other geotechnical issues as they pertain to the proposed development are discussed in the remainder of this section.

6.1 Expansive Soil

Based on Atterberg limits testing, we conclude the upper approximately 4 feet of soil at the site has low to moderate expansion potential while the soil below a depth of 4 feet bgs is highly expansive. Expansive near-surface soil is subject to volume changes during seasonal fluctuations in moisture content. These volume changes can cause movement and cracking of foundations and slabs. Therefore, foundations and slabs should be designed and constructed to mitigate the effects of the expansive soil. In general, the effects of expansive soil can be mitigated by moisture-conditioning the expansive soil, providing non-expansive soil below slabs, and either supporting foundations below the zone of severe moisture change or by providing a stiff, shallow foundation that can limit deformation of the superstructure as the underlying soil shrinks and swells.

At expansive soil sites it is critical to manage surface and subsurface drainage to prevent water from collecting beneath pavements and slabs, where it can lead to swelling and shrinking of the subgrade soil and can cause subgrade instability under vehicular loads. If permeable pavements, tree wells, irrigated landscaped zones, and stormwater infiltration basins will be constructed near the proposed building, they should incorporate design elements that prevent saturation of the soil adjacent to and below building foundations.

6.2 Foundations and Settlement

The soil encountered at the foundation level has moderate strength and compressibility. Therefore, we conclude that the proposed building may be supported on spread footings. Alternatively, we conclude the proposed building may be supported on a reinforced concrete mat foundation.

We estimate total and differential static settlements of the proposed building, supported on properly constructed shallow foundations designed using the allowable bearing pressures presented in Section 7.2, will be less than about 1-1/4 inches and less than 3/4 inch over a horizontal distance of 30 feet, respectively. As discussed in Section 5.2.3, shallow foundations may experience additional total and differential liquefaction-induced settlements of up to 1/2 inch and 1/4 inch across a horizontal distance of 30 feet, respectively, following a major seismic event.

6.3 Soil Corrosivity

Corrosivity tests were performed by Project X Corrosion Engineering, of Murrieta, California, on soil samples obtained from B-1 and B-2 at depths of 1.25 and 3.25 feet bgs, respectively. The corrosivity test results are presented in Appendix B.

Many factors can affect the corrosion potential of soil including, but not limited to, resistivity, pH, and chloride and sulfate concentrations. Based on the minimum soil resistivity measurements of 3,216 and 2,881 ohm-cm, we conclude the soil is “corrosive to highly corrosive” to buried metal (Roberge, 2018). Accordingly, all buried iron, steel, cast iron, galvanized steel, and dielectric-coated steel or iron should be protected against corrosion depending upon the critical nature of the structure. If it is necessary to have metal in contact with soil, a corrosion engineer should be consulted to provide recommendations for corrosion protection.

The results of the pH tests (7.5 and 7.3) indicate the near-surface soil is “negligibly corrosive” to buried metallic and concrete structures. The chloride ion concentrations (6.1 and 18.7 mg/kg) indicate the chlorides in the near-surface soil are “negligibly corrosive” to buried metallic structures and reinforcing steel in concrete structures below ground. The results also indicate the sulfate ion concentrations (8.5 and 10.9 mg/kg) are sufficiently low such that sulfates do not pose a threat to buried concrete and mortars.

6.4 Construction Considerations

The soil to be excavated generally consists of clay and sand, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. If site grading is performed during the rainy season, the near-surface clay will likely be wet and will have to be dried before compaction can be achieved. Heavy rubber-tired equipment, such as scrapers and vibratory rollers, could cause excessive deflection (pumping) of the wet clay and therefore should be avoided. If the project schedule or weather conditions do not permit sufficient time for drying of the soil by aeration, the subgrade can be treated with lime or cement prior to compaction. Alternatively, imported granular fill can be used.

Excavations that will be entered by workers should be sloped or shored in accordance with CAL-OSHA standards (29 CFR Part 1926). The contractor should be responsible for the construction and safety of temporary slopes.

7.0 RECOMMENDATIONS

Our recommendations for site preparation and grading, foundation design, and other geotechnical aspects of the project are presented in this section.

7.1 Site Preparation and Grading

In areas of proposed improvements, demolition should include removal of existing pavements and underground utilities. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside of the footprint of the proposed improvements and will not interfere with the proposed construction, they may be abandoned in place provided the lines are filled with lean concrete or cement grout to the property line. Voids resulting from demolition activities should be properly backfilled with 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 improvements (i.e., building pads, pavement, or flatwork). Tree roots with a diameter greater than 1/2 inch within 3 feet of finished subgrade should be removed.

7.1.1 Fill Materials and Compaction Criteria

The moderately to highly expansive soil subgrade beneath various surface improvements, such as building pads, pavements, and concrete flatwork, may require moisture-conditioning to limit its expansion potential. Where required, as determined by our field engineer, expansive clay subgrade soil should be scarified to a depth of at least 12 inches, moisture-conditioned, and compacted to the specified percent relative compaction,[7] as presented below in Table 2. Note that "moisture-conditioning" may require wetting or drying of the soil, depending on the conditions encountered. All fill should be placed in horizontal lifts not exceeding 8 inches in loose thickness, moisture-conditioned, and compacted in accordance with the recommendations 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 Recommendations

Location	Relative Compaction (percent)	Moisture Content
General fill – low-plasticity and select fill	90+	Above optimum
General fill – select fill (more than 5 ft)	95+	Above optimum
Utility trench backfill – low-plasticity and select fill	90+	Above optimum
Utility trench - clean sand or gravel	95+	Near optimum
Exterior slabs – low-plasticity and select fill	90+	Above optimum
Pavement subgrade – low-plasticity and aggregate base	95+	Above optimum

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[7] 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.

Select Fill

Select fill should consist of imported soil that is free of organic matter and contains no rocks or lumps larger than 3 inches in greatest dimension. It should also have a liquid limit less than 40, plasticity index less than 15, and be approved by the Geotechnical Engineer. Samples of proposed select 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 provided, a minimum of two weeks will be required to perform any necessary analytical testing.

Aggregate Base Material

Imported Class 2 aggregate base may be used as select fill or trench backfill (above bedding materials). Aggregate base placed beneath sidewalks and vehicular pavements within public right-of-way should meet the requirements in the 2022 Caltrans Standard Specifications, Section 26, for Class 2 Aggregate Base (3/4 inch maximum).

Controlled Low-Strength Material

Controlled low-strength material (CLSM) may be considered as an alternative to fill beneath the improvements, concrete flatwork, or pavement. CLSM should meet the requirements in the 2022 Caltrans Standard Specifications. It is an ideal backfill material when adequate room is limited or not available for conventional compaction equipment, or when settlement of the backfill must be minimized. No compaction is required to place CLSM. CLSM should have a minimum 28-day unconfined strength of 100 pounds per square inch (psi).

7.1.2 Utility Trench Excavations and Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements. 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 4 inches of clean sand or fine gravel (defined as poorly graded soil with less than 5 percent fines by weight). After the pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of 6 inches with clean sand or fine gravel (defined as poorly graded soil with less than 5 percent fines by weight), which should be mechanically tamped.

Backfill for utility trenches and other excavations is also considered fill and it should be placed and compacted in accordance with the recommendations previously presented. If imported clean sand or gravel (defined as poorly graded soil with less than 5 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 the pavement section.

Foundations for the proposed improvements should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of utility trenches. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with CLSM in accordance with the material requirements presented in Section 7.1.1. If utility trenches are to be excavated below this zone-of-influence line for existing or new foundations, the trench walls need to be fully supported with shoring until CLSM is placed.

7.1.3 Exterior Concrete Flatwork

We recommend that a minimum of 4 inches of Class 2 aggregate base be placed beneath proposed exterior concrete flatwork, including patio slabs and sidewalks; the select fill should extend at least 12 inches beyond the slab edges, except where constrained by property lines. The aggregate base beneath exterior slabs-on-grade, such as patios and sidewalks, should be moisture-conditioned and compacted in accordance with the requirements presented in Table 2.

7.2 Surface Drainage and Landscaping

Positive surface drainage should be provided around the building to direct surface water away from the foundations. Grades around the building should be designed by the Civil Engineer and conform to the requirements of the 2022 CBC, which will help minimize stormwater accumulation adjacent to foundations. In addition, roof downspouts should be discharged into controlled drainage facilities to keep water away from the foundations. The use of water-intensive landscaping around the perimeter of the building 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 pavements and pedestrian walkways. Where landscape beds and tree wells are immediately adjacent to pavements and flatwork, 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.

7.3 Foundations

Recommendations for spread footings and a mat foundation are presented in the following sections.

7.3.1 Spread Footings

The proposed building may be supported on continuous and/or individual spread footings bearing on firm native soil. Continuous footings should be at least 18 inches wide, and isolated spread footings should be at least 24 inches wide. Perimeter footings should be bottomed at least 24 inches below the lowest adjacent soil subgrade and interior footings should be bottomed at least 18 inches below the building pad subgrade (i.e., bottom of capillary break). The perimeter footing embedment depth may be decreased by 6 inches where pavement or concrete flatwork is adjacent to the building.

Spread footings may be designed using an allowable bearing pressure of 3,000 pounds per square foot (psf) for dead-plus-live loads; this allowable bearing pressure may be increased by one-third for total design loads, which include wind or seismic forces. The allowable bearing pressures for dead-plus-live and total loads include factors of safety of at least 2.0 and 1.5, respectively.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute passive resistance for transient loading, we recommend using a uniform pressure of 1,500 psf (rectangular distribution). To compute passive resistance for sustained lateral loads, we recommend using an equivalent fluid weight (triangular distribution) of 250 pounds per cubic foot (pcf). The upper foot of soil should be ignored unless confined by a slab or pavement. Where the mat foundation is in contact with soil, frictional resistance should be computed using a base friction coefficient of 0.30. 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.

Footings to be constructed near underground utilities should be bottomed below an imaginary line extending up at an inclination of 1.5:1 (horizontal to vertical) from the bottom of the utility trench. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with CLSM with a 28-day unconfined compressive strength of at least 100 pounds per square inch (psi). In addition, footings should be bottomed below an imaginary line extending up at an inclination of 1.5:1 (horizontal to vertical) from the bottom of any bioswale/stormwater treatment area near the building.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. Where loose/weak soil is encountered at the bottom of footing excavations, the loose/weak soil should be removed to expose firm native soil that can provide adequate foundation support, and the over-excavation should be backfilled with CLSM or lean concrete. The bottoms and sides of the footing excavations should be maintained moist until the concrete is placed. We should be scheduled to check footing excavations for proper bearing and preparation prior to placement of reinforcing steel. The clay at the site is susceptible to significant softening from standing water. Therefore, the bottom of the footing excavations should be protected during the rainy season. One acceptable method for protecting the footing subgrade would be to place 2 to 3 inches of unreinforced concrete (“mud” slab) as soon as footing excavations are checked by our field engineer.

7.3.2 Mat Foundation

We recommend the proposed building be supported on a well-reinforced concrete mat foundation bottomed on stiff native soil. The perimeter of the mat should be bottomed at least 9 inches below the adjacent soil subgrade. For preliminary structural design of the mat foundation, we recommend using a coefficient of vertical subgrade reaction of 15 pounds per cubic inch (pci) for dead-plus-live loads; this value has already been scaled to take into account the plan dimensions of the mat foundation (therefore, this is not k_{v1} for 1-foot-square plate) and may be increased by one-third for total load conditions. Once the Structural Engineer estimates the distribution of bearing stress on the bottom of the mat, we should review the distribution and revise the modulus of subgrade reaction, if appropriate.

Considering the relatively large area of the mat, we expect the average bearing stress under the mat to be relatively low; however, concentrated stresses will occur at column locations and at the edges of the mat. We recommend the mat be designed using an allowable bearing pressure of 3,000 psf for dead-plus-live loads; this pressure may be increased by one-third for total load conditions. The allowable bearing pressures for dead-plus-live and total loads include factors of safety of at least 2.0 and 1.5, respectively.

Lateral forces can be resisted by friction along the base of the mat and by passive pressure against the sides of the mat foundation. To compute passive resistance for transient loading, we recommend using a uniform pressure of 1,500 psf (rectangular distribution). To compute passive resistance for sustained lateral loads, we recommend using an equivalent fluid weight (triangular distribution) of 250 pcf. The upper foot of soil should be ignored unless confined by a slab or pavement. Where the mat foundation is in contact with soil, frictional resistance should be computed using a base friction coefficient of 0.30. Where a vapor retarder is placed beneath the mat, a base friction coefficient of 0.20 should be used. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

If the subgrade for the mat is disturbed during excavation, it should be re-compacted in accordance with the recommendations presented above in Section 7.1. The mat subgrade should be kept moist following excavation and maintained in a moist condition until the vapor retarder is placed. We should check the mat subgrade prior to placing the vapor retarder or rebar to confirm it is free of standing water, debris, and disturbed materials. If the mat subgrade will be potentially exposed to rain, we recommend 2 to 3 inches of unreinforced concrete (“mud” slab) be placed to protect the subgrade from rain-induced softening prior to concrete/vapor retarder placement.

7.4 Capillary Moisture Break and Vapor Retarder

The subgrade for slab-on-grade floors and mats should be prepared in accordance with our recommendations in Section 7.1. Where water vapor transmission through the floor slab/mat is not desirable, we recommend installing a capillary moisture break and water vapor retarder beneath the floor slab/mat. A capillary moisture break consists of at least 4 inches of clean, free-draining gravel or crushed rock. The particle size of the capillary break material should meet the gradation requirements presented in Table 3.

TABLE 3
Gradation Requirements for Capillary Moisture Break

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

The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. Where the building will be supported on a mat, the capillary moisture break may be omitted provided the vapor retarder meets the requirements for Class A vapor retarders. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by 6 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 can result in excessive vapor transmission through the slab. Where the concrete is poured directly over the vapor retarder, we recommend the w/c ratio of the concrete not exceed 0.45. Water should not be added to the concrete mix in the field. If necessary, workability should be increased by adding plasticizers. In addition, the slab should be properly cured. Before the floor covering is placed, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.5 Permanent Below-Grade Walls

Permanent retaining walls should be designed to resist lateral earth pressures imposed by the retained soil, as well as a surcharge pressure from nearby foundations and vehicles where appropriate. In addition, because the site is in a seismically active area, retaining walls that retain more than 6 feet of soil should be designed for the more critical of static or seismic conditions.

For static conditions, we recommend restrained and unrestrained walls be designed for the following lateral earth pressures:

- Restrained Wall - At-rest earth pressure using an equivalent fluid weight of 60 pcf for drained conditions and 91 pcf for undrained conditions.
- Unrestrained Wall - Active earth pressure using an equivalent fluid weight of 40 pcf for drained conditions and 82 pcf for undrained conditions.

Walls that will retain more than 6 feet of soil will need to be designed for the more critical of static (presented above) or the following seismic conditions.

- Restrained Wall - Active earth pressure using an equivalent fluid weight of 40 pcf plus a seismic increment of 36 pcf for drained conditions; and 82 pcf plus a seismic increment of 17 pcf for undrained conditions.
- Unrestrained Wall - Active earth pressure using an equivalent fluid weight of 40 pcf plus a seismic increment of 16 pcf for drained conditions; and 82 pcf plus a seismic increment of 8 pcf for undrained conditions.

Where there will be vehicular traffic behind the top of a permanent wall within a horizontal distance equal to 1.5 times the height of the wall, the wall should be designed for vehicular surcharge of 50 psf over the upper 10 feet of the wall. Where existing foundations are supported above a “zone-of-influence” line extending up from a permanent wall at an inclination of 1.5:1 (horizontal to vertical), the wall should be designed for a surcharge pressure. The magnitude of the surcharge pressure will need to be evaluated on a case-by-case basis.

The recommended lateral earth pressures for “drained” conditions assume the walls are backdrained above the water table to prevent the buildup of hydrostatic pressure. Although site retaining walls will be above the groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines. One acceptable method for backdraining a site retaining wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the base of the wall. The pipe should be surrounded by at least 4 inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140NC or equivalent). The pipe should be sloped to drain by gravity to a suitable outlet. For below-grade retaining walls, such as elevator pit walls, the walls may be designed for the recommended “undrained” earth pressures presented above over their entire height, in which case the drainage system may be omitted. To protect against moisture migration, below-grade retaining walls should be waterproofed and water stops should be placed at all construction joints.

7.6 Pavement Design

Design recommendations for asphalt and Portland cement concrete pavements are presented in the following sections.

7.6.1 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete (AC) pavement sections. Based on our experience, we selected a resistance value (R-Value) of 5, which is appropriate for clay soils. Recommended pavement sections for traffic indices (TIs) ranging from 4.5 to 7.0 are presented in Table 4. The Civil Engineer for the project should check the TIs presented in this report are appropriate for the intended use. We can provide additional pavement sections for different TIs upon request.

TABLE 4
Asphalt Concrete Pavement Sections

TI	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
4.5	2.5	9.5
5.0	3.0	10.0
5.5	3.0	12.0
6.0	3.5	13.0
6.5	4.0	13.5
7.0	4.0	15.5

The upper 12 inches of the soil subgrade should be moisture-conditioned and compacted in accordance with our recommendations presented in Section 7.1 and should be non-yielding. The Class 2 aggregate base should be moisture-conditioned to near optimum, compacted to at least 95 percent relative compaction, and be non-yielding.

If pavements are adjacent to irrigated landscaped areas (including infiltration basins), curbs adjacent to those areas should extend through the aggregate base and at least 3 inches into the underlying soil to reduce the potential for irrigation water to infiltrate into the pavement section. If drip irrigation is used in the landscaping adjacent to the pavement, deepening the curbs is not required.

7.6.2 Rigid (Portland-Cement Concrete) Pavement Design

The minimum thickness for concrete pavements should be based on the anticipated traffic loading, the modulus of rupture of the concrete used, and the supporting characteristics of the subgrade below the pavement section. Pavements should be designed and constructed in accordance with the American Concrete Institute (ACI) Commercial Concrete Parking Lots and Site Paving Design and Construction Guide (ACI PRC-330-21). The compressive strength of the concrete should be at least 3,750 psi with a modulus of rupture of the concrete of 550 psi at 28 days. Reinforcing steel may be used for shrinkage crack control. The recommended minimum rigid pavement section and maximum spacing between joints are presented in Table 5 below.

TABLE 5
Rigid Concrete Pavement Design

Traffic Categories	Maximum ADTT[8]	Concrete Thickness (inches)	Class 2 Aggregate Base Thickness (inches)	Maximum Spacing Between Joints (feet)
Car parking areas and Access Lanes (Category A)	10	5.0	6	12.5
Entrance and Truck Service Lanes (Category B)	25	6.0	6	15
Garbage or Fire Truck lane (Category E)	1	7.0	6	15

Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those 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 Seismic Design

The latitude and longitude of the site are 37.4643° and -122.4323°, respectively. The results of the seismic CPT indicate the site has an estimated shear wave velocity (V_s) in the upper 100 feet (30 meters, V_{s30}) of 820 feet/second for CPT-1. For design per the 2022 CBC, we recommend the following:

- Site Class D (stiff soil, non-default)
- $S_S = 2.046g$, $S_1 = 0.778g$

The 2022 CBC 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})

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[8] ADDT is the Average Daily Truck Traffic in both directions (excludes panel trucks, pickup trucks, and other four-wheel vehicles).

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} = 2.046g, S_{M1}* = 1.984g
- S_{DS} = 1.364g, S_{D1}* = 1.323g
- Seismic Design Category E for Risk Factors I, II, and III

8.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, Rockridge Geotechnical, Inc. should review the project plans and specifications to check they conform to the intent of our recommendations. During construction, our field engineer should provide on-site observation and testing during grading and fill placement, as well as the installation of new foundations. These observations will allow us to compare actual and anticipated subsurface conditions to check 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 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 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

ACI Committee 330. (2021). *ACI PRC-330-21: Commercial Concrete Parking Lots and Site Paving Design and Construction*. American Concrete Institute (ACI).

American Society of Civil Engineers (ASCE). (2017). *ASCE/SEI 7-16: Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. ASCE Publications. http://doi.org/10.1061/9780784414248

ATC Associates Inc. (2009, May 1). *Semi-Annual Groundwater Monitoring Report – First Quarter 2009, Former Cheaper Store #44, 501 Kelly Street, Half Moon Bay, California.* ATC Associates Inc. https://documents.geotracker.waterboards.ca.gov/esi/uploads/geo_report/8401134669/T0608100224.PDF

Boulanger, R. W., and Idriss, I. M. (2014). *CPT and SPT Based Liquefaction Triggering Procedures*. Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, 134 pp.

California Department of General Services. (2022). *California Building Code*. State Government of California. https://www.dgs.ca.gov/BSC/Codes

California Department of Transportation (Caltrans). (2022). *Caltrans Standard Specifications*. State Government of California. https://dot.ca.gov/-/media/dot-media/programs/design/documents/2022_stdspecs-a11y.pdf

California Geological Survey. (2021, September 23). *Earthquake Zones of Required Investigation, Half Moon Bay Quadrangle, Official Map*. California Department of Conservation.

California Geological Survey. (2021). *Seismic Hazard Zone Report for the Half Moon Bay 7.5-Minute Quadrangle, San Mateo County, California*. California Department of Conservation.

Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., & Zeng, Y. (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

GeoLogismiki. (2022, March 15*). CLiq, Version 3.5.2.22*. Retrieved June 17, 2023, from https://geologismiki.gr/

GeoLogismiki. (2022, December 2). *CPeT-IT, Version 3.9.3.7*. Retrieved July 17, 2023, from https://geologismiki.gr/

Graymer, R.W., Moring, B.C., Saucedo, G.J., Wentworth, C.M., Brabb, E.E., & Knudsen, K.L. (2006, March 6). *Geologic Map of the San Francisco Bay Region* (Scientific Investigations Map 2918). United States Geological Survey. https://pubs.usgs.gov/sim/2006/2918/

Petersen, M.D., Moschetti, M.P., Powers, P.M., Mueller, C.S., Haller, K.M., Frankel, A.D., Zeng, Y., Rezaeian, S., Harmsen, S.C., Boyd, O.S., Field, N., Chen, R., Rukstales, K.S., Luco, N., Wheeler, R.L., Williams, R.A., & Olsen, A.H. (2014). *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/

Proto, C. (2024). *Improved CPT-based Liquefaction Assessments with the I_B Parameter: A Case Study in the San Francisco Bay Area*, Proceedings of the 8th International Conference on Earthquake Geotechnical Engineering.

Roberge, P. R. (2018). *Corrosion Basics: An Introduction: Third Edition*. NACE International.

Robertson, P.K. (2016). Cone penetration test (CPT)-based soil behaviour type (SBT) classification system — an update. *Canadian Geotechnical Journal, 53*(16). https://doi.org/10.1139/cgj-2016-0044

Sitar, N., Mikola, R.G., & Candia, G. (2012). Seismically Induced Lateral Earth Pressures on Retaining Structures and Basement Walls. *Proceedings of the ASCE GeoCongress 2012* (pp. 1-24). Geotechnical Special Publication. https://doi.org/10.1061/9780784412138.0013

Thompson, E.M., Wald, D.J., Worden, B., Field, E.H., Luco, N., Petersen, M.D., Powers, P.M., Badie, R. (2016). *Shakemap earthquake scenario: Building Seismic Safety Council 2014 Event Set (BSSC2014)*. United States Geological Survey. https://doi.org/10.5066/F7V122XD

United States Geological Survey. (2021). *Earthquake Hazards Program: Advanced National Seismic System (ANSS) Comprehensive Catalog of Earthquake Events and Products*. United States Department of the Interior. https://earthquake.usgs.gov/earthquakes/search/

Zhang G., Robertson. P.K., & Brachman R. (2002). Estimating Liquefaction Induced Ground Settlements from the CPT. *Canadian Geotechnical Journal, 39*(5), pp 1169-1180. https://doi.org/10.1139/t02-047

FIGURES

Base map: Google Map, 2023

0 1,000 2,000 Feet

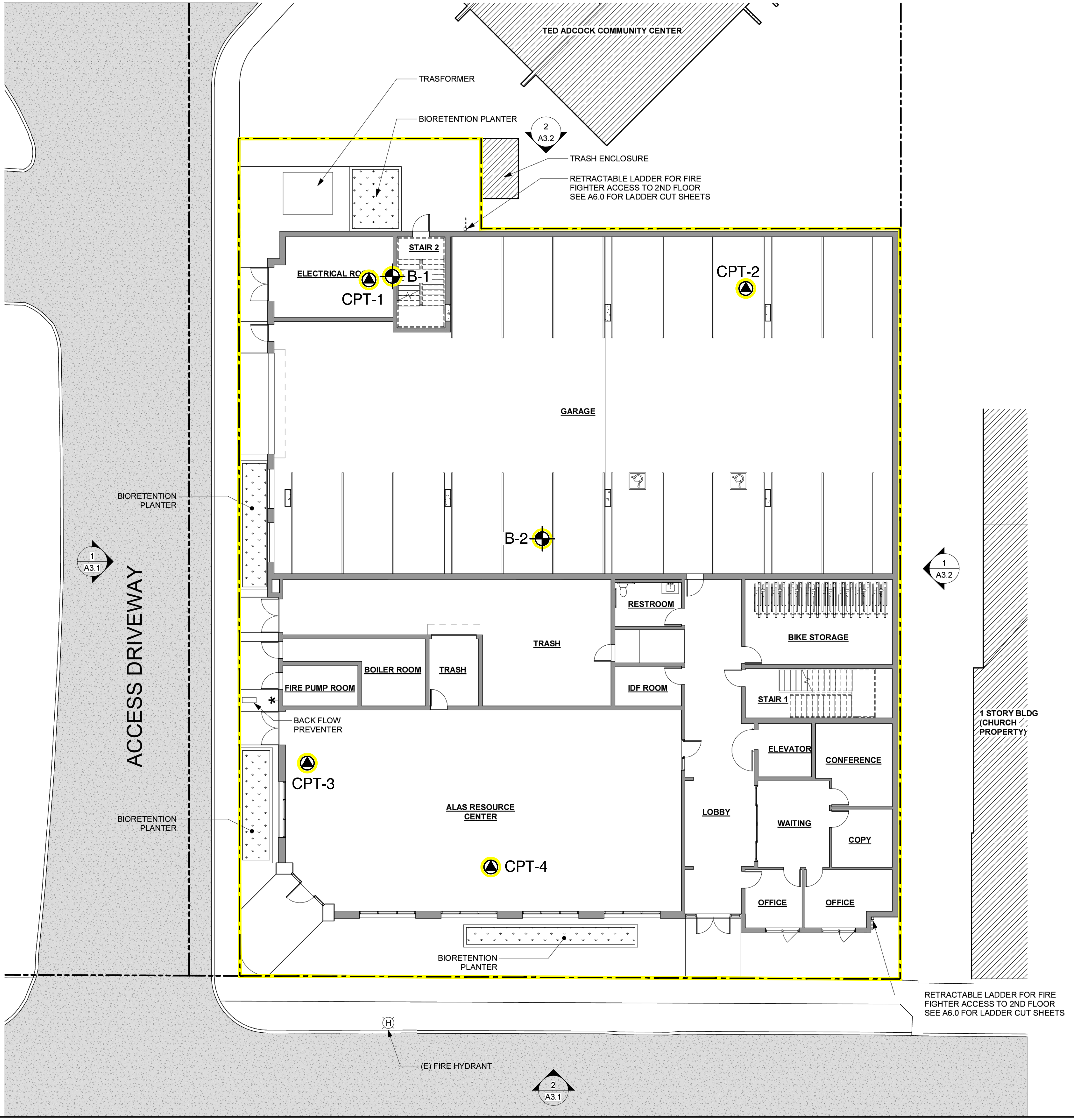
Approximate scale

555 KELLY AVENUE
Half Moon Bay, California

ROCKRIDGE GEOTECHNICAL

SITE LOCATION MAP

Date 12/27/23 | Project No. 23-2527 | Figure 1



EXPLANATION

CPT-1 ⊚	Approximate location of cone penetration test by Rockridge Geotechnical, Inc., December 14, 2023
B-1 ⊕	Approximate location of boring by Rockridge Geotechnical, Inc., December 21, 2023
— · —	Proposed project limits

0 20 Feet

Approximate scale

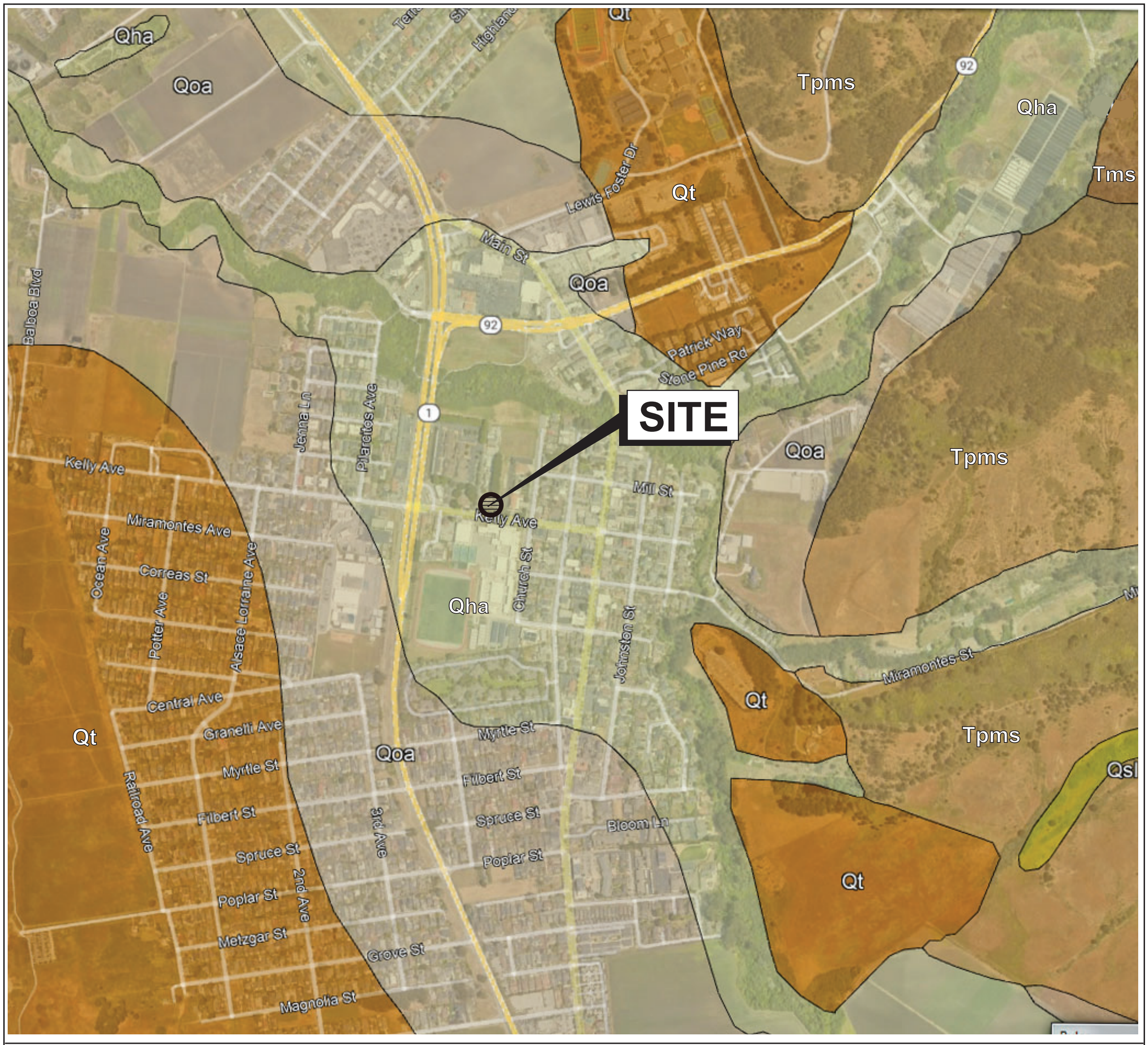
Reference: Base map from a drawing titled "Site Plan", by Van Meter Williams Pollack, LLP, dated July 3, 2023.

555 KELLY AVENUE
Half Moon Bay, California

SITE PLAN

Date 12/27/23 | Project No. 23-2527 | Figure 2

ROCKRIDGE GEOTECHNICAL



Base map: Google Earth with U.S. Geological Survey (USGS), San Mateo County, 2023.

EXPLANATION

Qsl	Hillslope Deposits (Quaternary)
Qha	Alluvium (Holocene)
Qt	Marine terrace deposits (Pleistocene)
Qoa	Alluvium (early (Pleistocene)
Tpms	Sedimentary rocks (Pliocene and early Miocene)
Tms	Sedimentary rocks (Miocene)

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

0 1,200 2,400 Feet

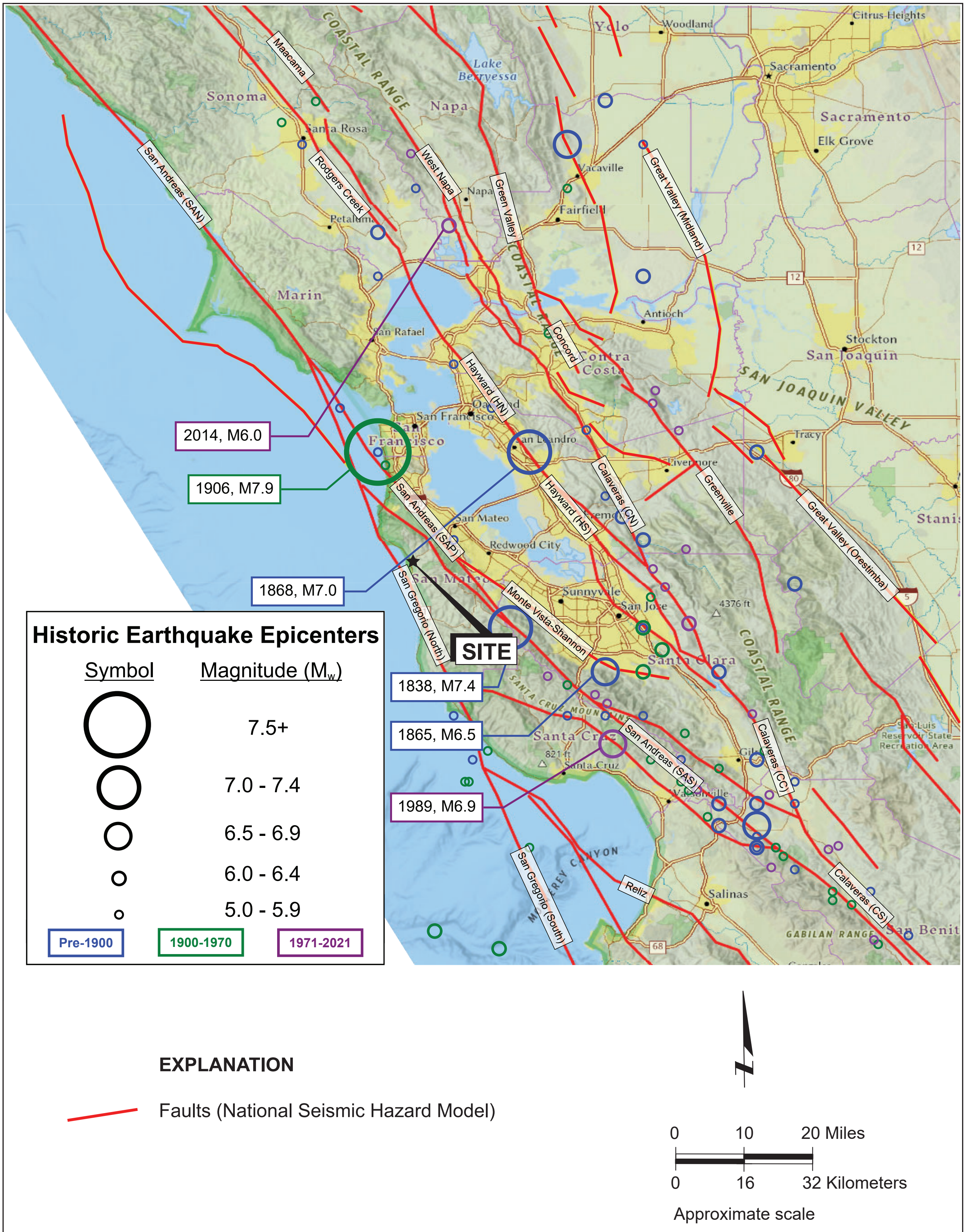
Approximate scale

555 KELLY AVENUE
Half Moon Bay, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 12/27/23 | Project No. 23-2527 | Figure 3

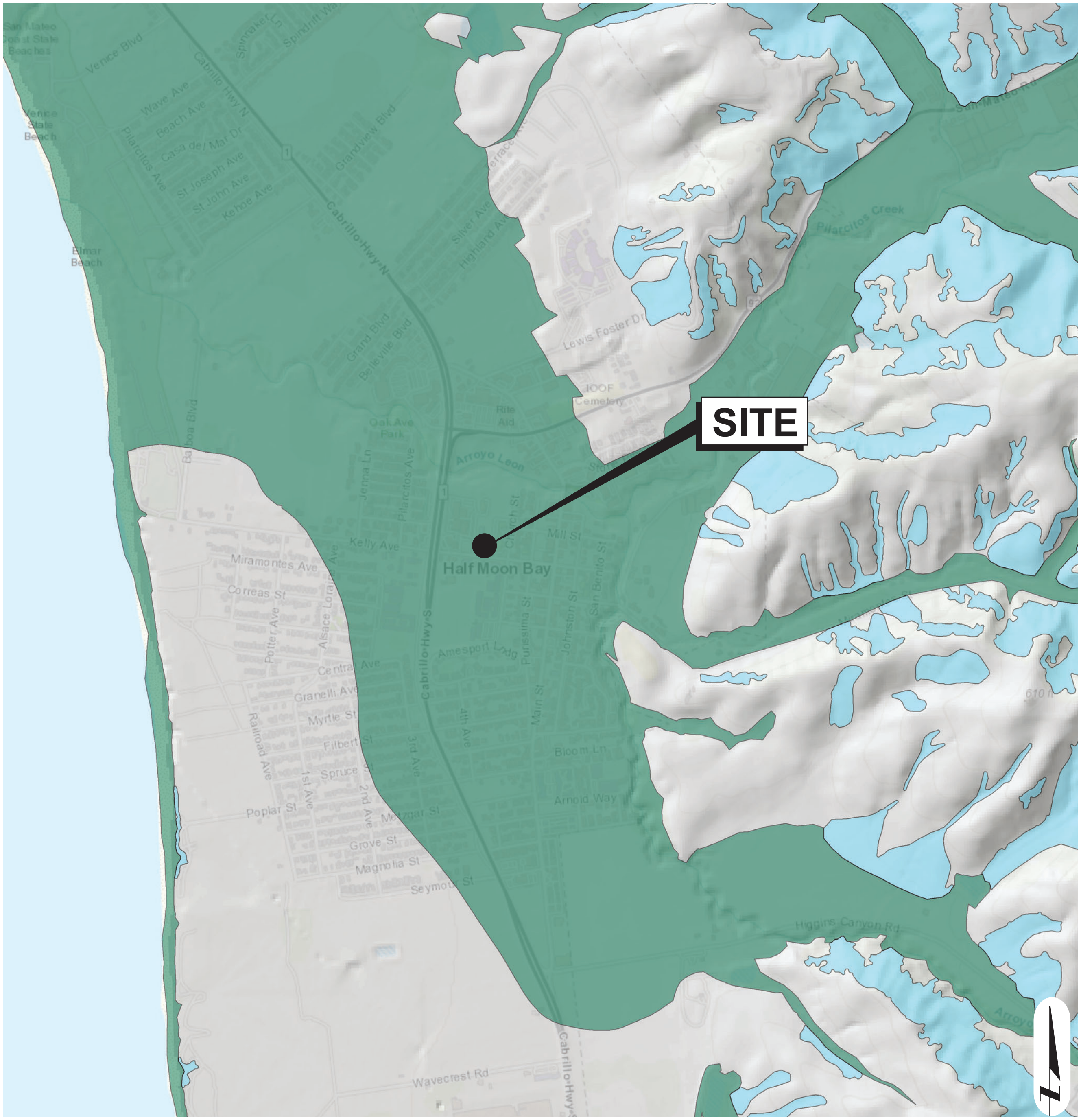


555 KELLY AVENUE
Half Moon Bay, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Date 11/21/23 | Project No. 23-2527 | 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
Half Moon Bay Quadrangle
California Geological Survey
Released September 23, 2021

0 2,000 4,000 Feet
Approximate scale

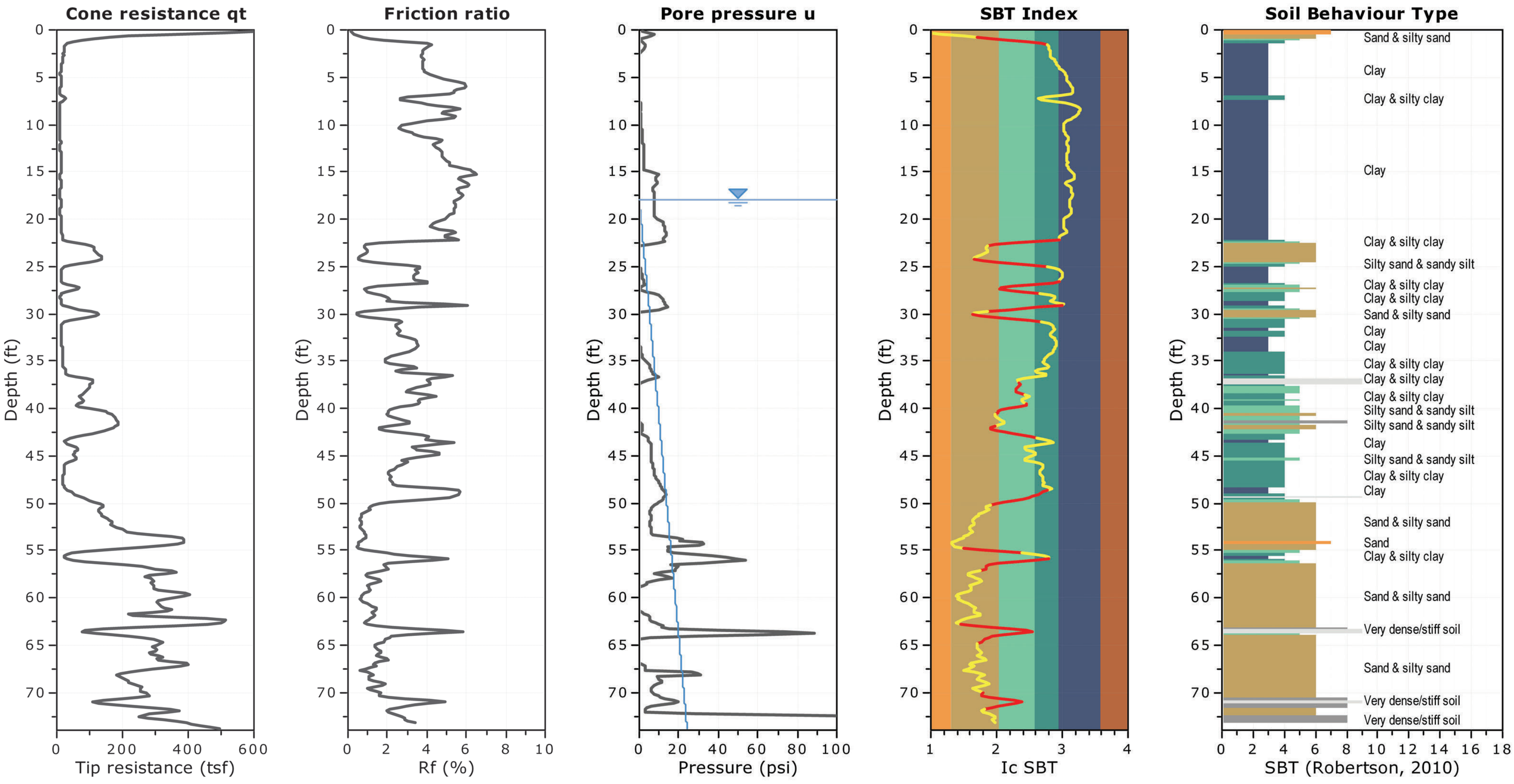
555 KELLY AVENUE
Half Moon Bay, California

ROCKRIDGE
GEOTECHNICAL

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 12/27/23
Project No. 23-2527
Figure 5

APPENDIX A
Cone Penetration Test Results and Logs of Borings



Total Depth: 73.8 ft, Date: December 14, 2023
Depth to Groundwater: 18 feet (measured with weighted tape)
Cone Operator: Middle Earth Geo Testing, Inc.

SBT legend

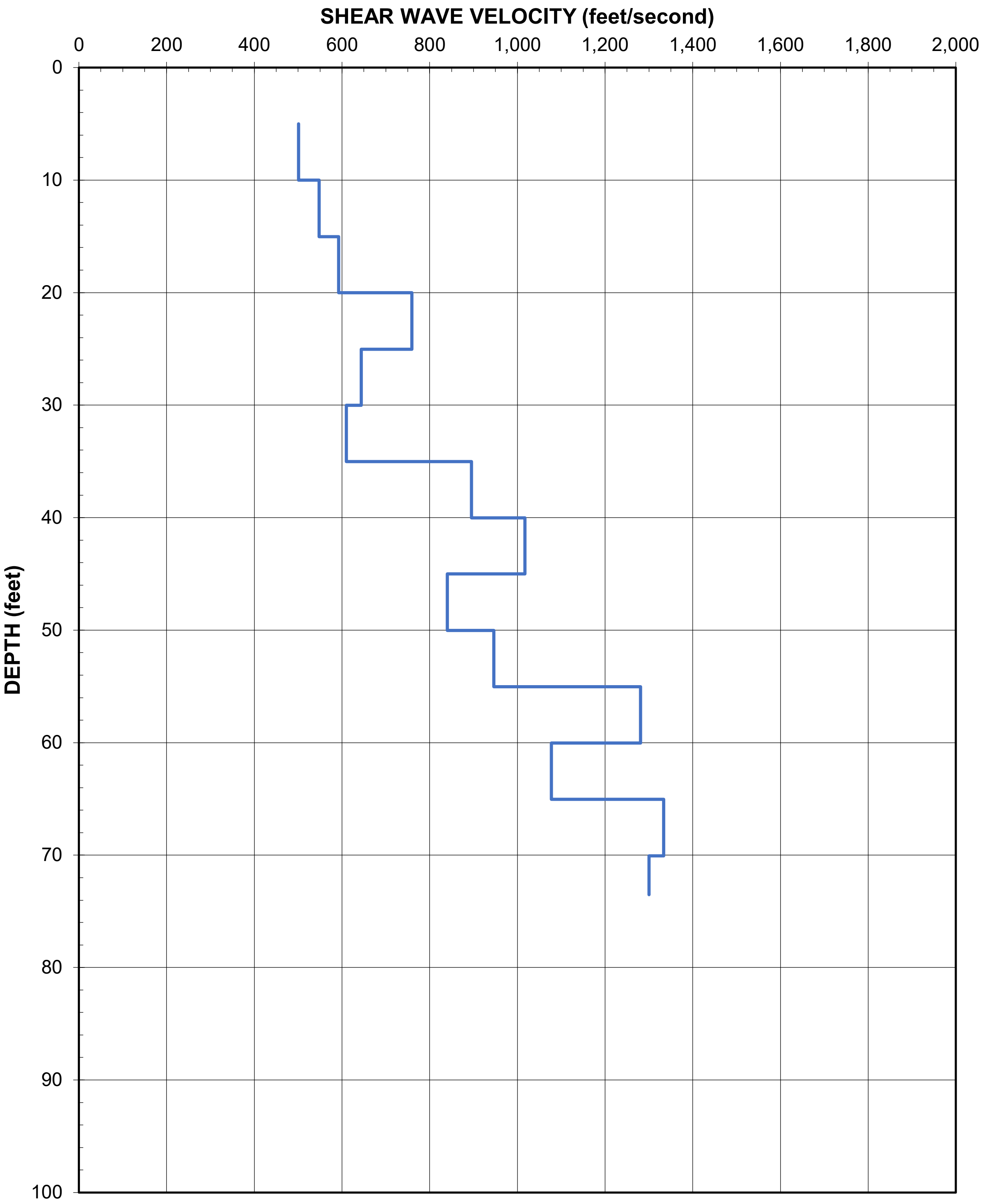
- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

555 KELLY AVENUE
Half Moon Bay, California

ROCKRIDGE
GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-1

Date 01/10/24 | Project No. 23-2527 | Figure A-1a

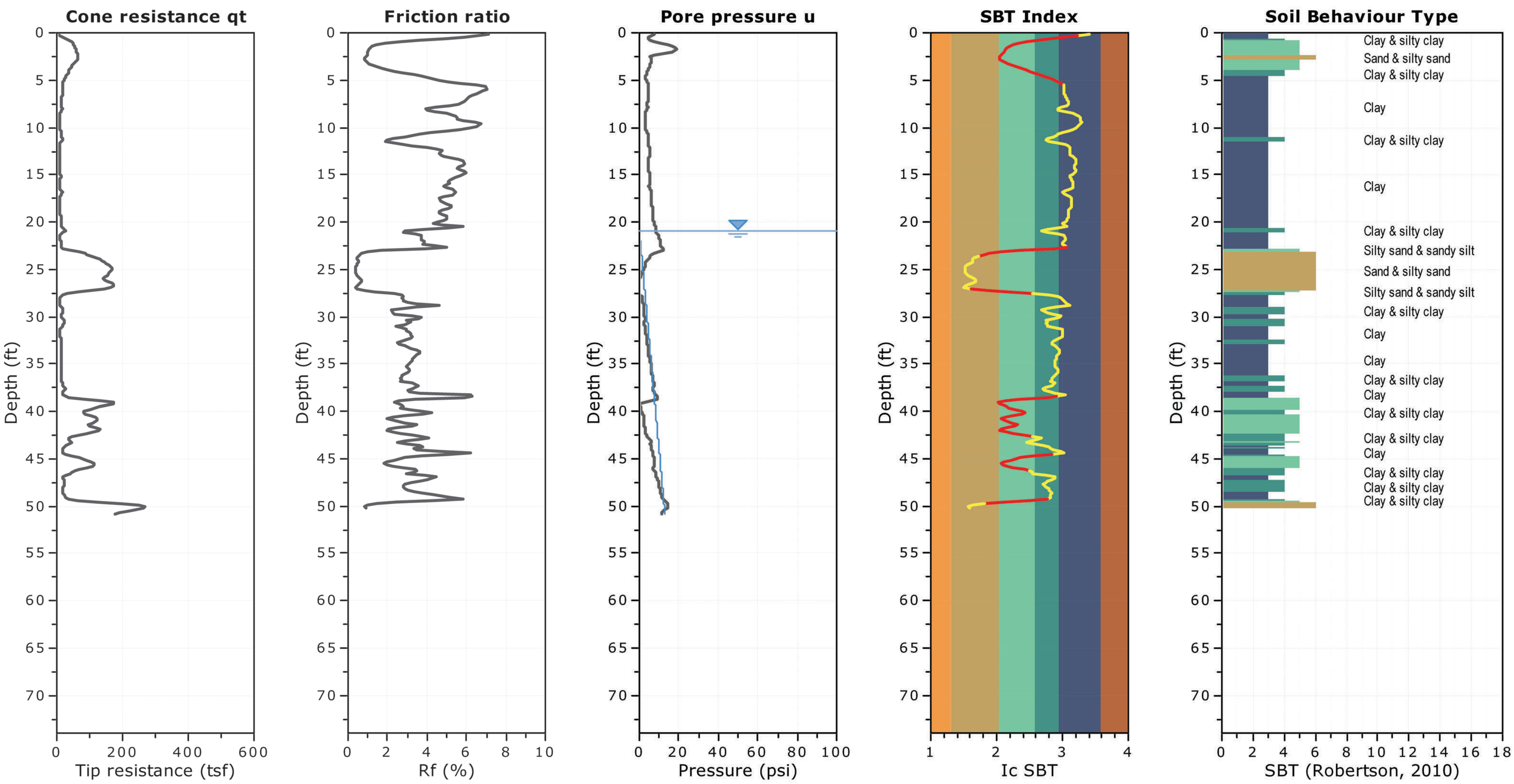


SHEAR WAVE VELOCITY PROFILE
CPT-1

555 KELLY AVENUE
Half Moon Bay, California

Date: 12/26/2023	Project No. 23-2527	Figure A-1b

ROCKRIDGE GEOTECHNICAL



Total Depth: 50.9 ft, Date: December 14, 2023
Depth to Groundwater: 21 feet (measured with weighted tape)
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. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

555 KELLY AVENUE
Half Moon Bay, California

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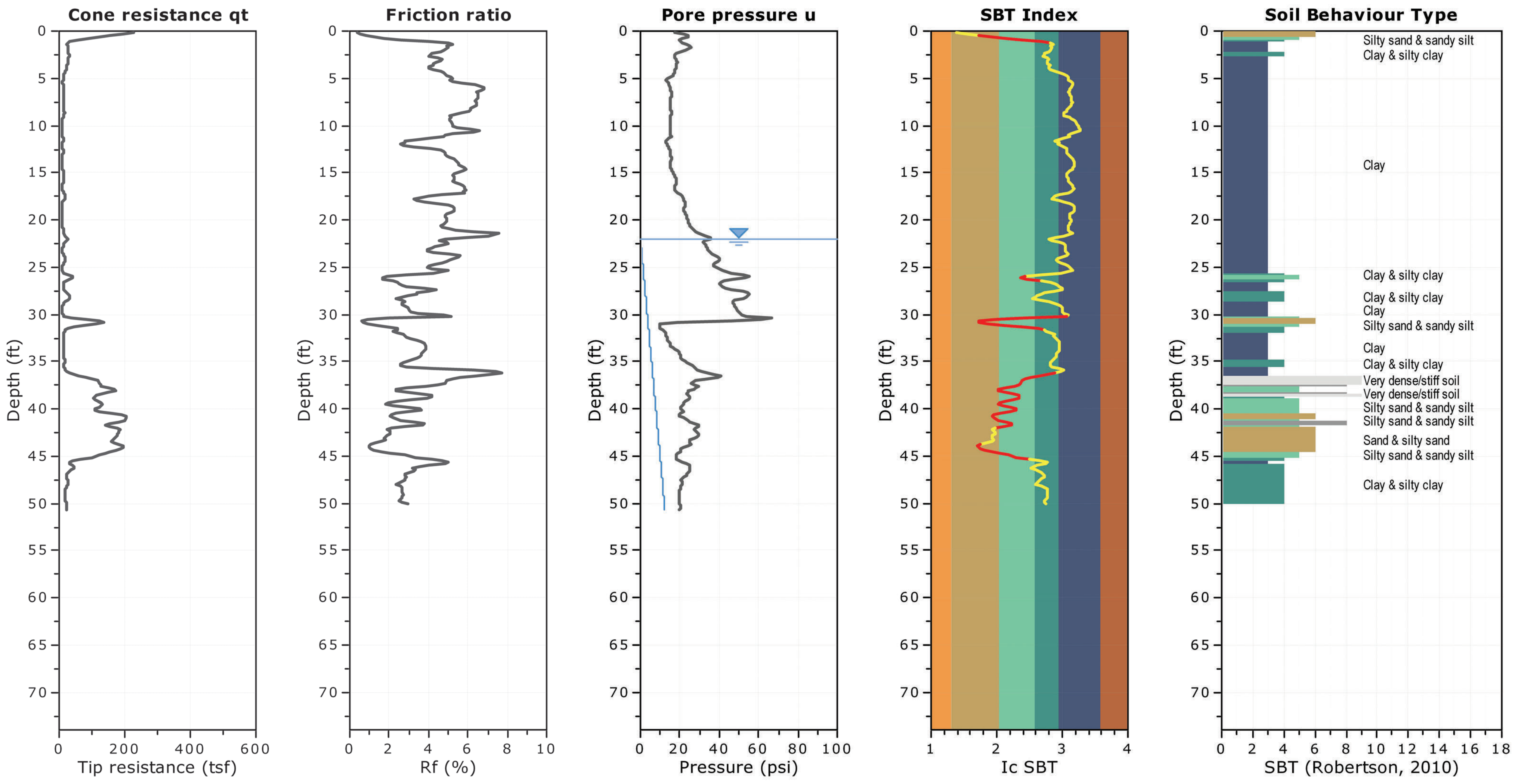
CONE PENETRATION TEST RESULTS
CPT-2

Date 01/10/24	Project No. 23-2527	Figure A-2



Total Depth: 50.7 ft, Date: December 14, 2023
Depth to Groundwater: 22 feet (measured with weighted tape)
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. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

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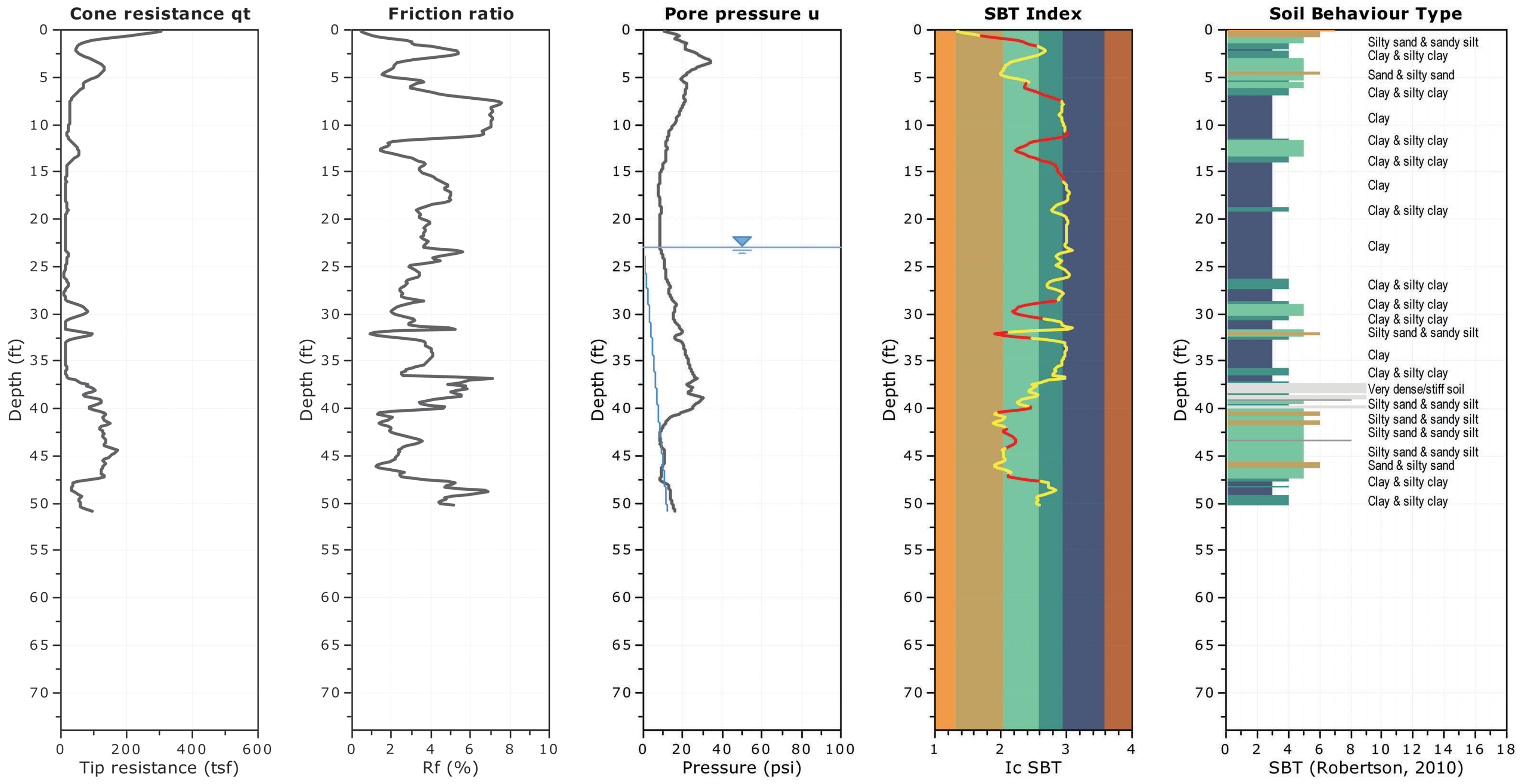
ROCKRIDGE GEOTECHNICAL

CONE PENETRATION TEST RESULTS
CPT-3

Date 01/10/24 | Project No. 23-2527 | Figure A-3



Total Depth: 50.9 ft, Date: December 14, 2023
Depth to Groundwater: 23 feet (measured with weighted tape)
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. Gravelly sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

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CONE PENETRATION TEST RESULTS
CPT-4

Date 01/10/24 | Project No. 23-2527 | Figure A-4

PROJECT: **555 KELLY AVENUE**
Half Moon Bay, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 12/21/2023
Date finished: 12/21/2023
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)

Logged by: J. Lei
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B53-R

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content,%	Dry Density Lbs/Cut Ft
						3 inches of asphalt concrete						
						6 inches of aggregate base						
1												
2	MC	▨	9 13 18	20	CL	SANDY CLAY (CL) dark brown, very stiff, moist, medium sand, trace roots Soil Corrosivity Test; see Appendix B LL = 30, PI = 15; see Appendix B					18.1	113
3						brown, fine to medium sand						
4	MC	▨	10 13 14	17								
5												
6	MC	▨	13 17 19	23	CL	CLAY with SAND (CL) brown and red-brown, very stiff, moist LL = 47, PI = 32; see Appendix B					15.9	109
7												
8												
9												
10					CL	SANDY CLAY (CL) olive-gray, stiff, moist, fine sand						
11	MC	▨	7 10 11	13	CL	CLAY (CL) light brown, stiff, moist, trace sand						
12												
13						gray-brown and red, very stiff						
14	MC	▨	9 13 17	19								
15												
16												
17												
18												
19	MC	▨	11 14 17	20								
20					CL	CLAY with SAND (CL) yellow-brown, very stiff, moist, fine sand						
21												
22												
23						▽ (12/21/2023; 12:28 PM)						
24	MC	▨	12 16 26	27	SP	SAND (SP) yellow, medium dense, wet, coarse sand, trace fines						
25												
26	SPT	◩	9 17 16	36		dense						
27					SP-SC	SAND with CLAY (SP-SC) gray, dense, wet, fine to medium sand						
28												
29					ML	SILT (ML) gray, hard, wet						
30	SPT	◩	9 18 24	45	SP-SM	SAND with SILT (SP-SM) gray, dense, wet, medium to trace coarse sand				8	18.0	105

ROCKRIDGE GEOTECHNICAL

Project No.: 23-2527

Figure: A-5a

PROJECT: **555 KELLY AVENUE**
Half Moon Bay, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
31						CLAY (CL) gray, hard, wet, trace sand						
32												
33					CL	stiff to very stiff						
34	MC	▨	6 10 13	15								
35												
36												
37												
38					GC	CLAYEY GRAVEL with SAND (GC) gray, dense, wet, medium to coarse sand, fine sub-angular gravel						
39	MC	▨	18 36 38	47								
40												
41												
42												
43												
44												
45												
46												
47												
48												
49												
50												
51												
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 40 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 23 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.

ROCKRIDGE GEOTECHNICAL

Project No.: 23-2527

Figure: A-5b

PROJECT: **555 KELLY AVENUE**
Half Moon Bay, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 12/21/2023
Date finished: 12/21/2023
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)
Logged by: J. Lei
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B53-R

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content,%	Dry Density Lbs/Cut Ft
						2 inches of asphalt concrete 6 inches of aggregate base						
1												
2	MC	■	8 14 15	18	CL	SANDY CLAY (CL) dark brown to gray-brown, very stiff, moist, fine to trace coarse sand LL = 30, PI = 14; see Appendix B					17.4	109
3												
4	MC	■	8 11 15	16	CL	CLAY with SAND (CL) brown, very stiff, moist, fine sand Soil Corrosivity Test; see Appendix B LL = 9, PI = 11; see Appendix B						
5												
6	MC	■	16 24 28	33	CL	CLAY (CL) gray-brown and red-brown, hard, moist, trace fine sand						
7												
8												
9												
10												
11	MC	■	7 9 11	13	CL	CLAY with SAND (CL) light brown and yellow, stiff, moist LL = 34, PI = 20; see Appendix B				77	27.6	97
12					CL	CLAY (CL) red-gray, stiff, moist, trace fine sand						
13												
14	MC	■	10 17 17	21		red-gray and red, very stiff						
15												
16												
17												
18												
19	MC	■	10 14 18	20		brown						
20					CL	SANDY CLAY (CL) gray-brown, very stiff, moist, coarse sand						
21												
22												
23												
24	MC	■	16 23 24	30	SP-SC	SAND with CLAY (SP-SC) yellow, medium dense to dense, wet, coarse sand						
25						∇ (12/21/2023; 10:00 AM) brown, fine sand						
26												
27												
28												
29	MC	■	13 26 32	37	SM	SILTY SAND (SM) gray, dense, wet, fine sand						
30					SP-SM	SAND with SILT (SP-SM)						

ROCKRIDGE GEOTECHNICAL

Project No.: 23-2527
Figure: A-6a

PROJECT: **555 KELLY AVENUE**
Half Moon Bay, California

Log of Boring B-2

PAGE 2 OF 2

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
31					SP-SM	SAND with SILT (SP-SM) (continued) red-yellow, dense, wet, fine to trace coarse sand						
34	SPT	◩	8 11 13	26	CL	CLAY (CL) gray, very stiff, wet, trace sand						
39	MC	■	24 30 36	42	SC	CLAYEY SAND with GRAVEL (SC) gray, dense, wet, fine to coarse sand, fine subrounded to subangular gravel						
40						very dense, increasing gravel content						
41	SPT	◩	13 18 37	59								
46	SPT	◩	18 26 30	61	SC	CLAYEY SAND (SC) gray, very dense, wet, fine sand						
47					CL	CLAY with SAND (CL) gray, hard, wet						
49	SPT	◩	13 19 27	50								
50												
60												

Boring terminated at a depth of 50 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 25 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.

ROCKRIDGE GEOTECHNICAL

Project No.: 23-2527

Figure: A-6b

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 1.38- or 1.5-inch inside diameter (refer to text)
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

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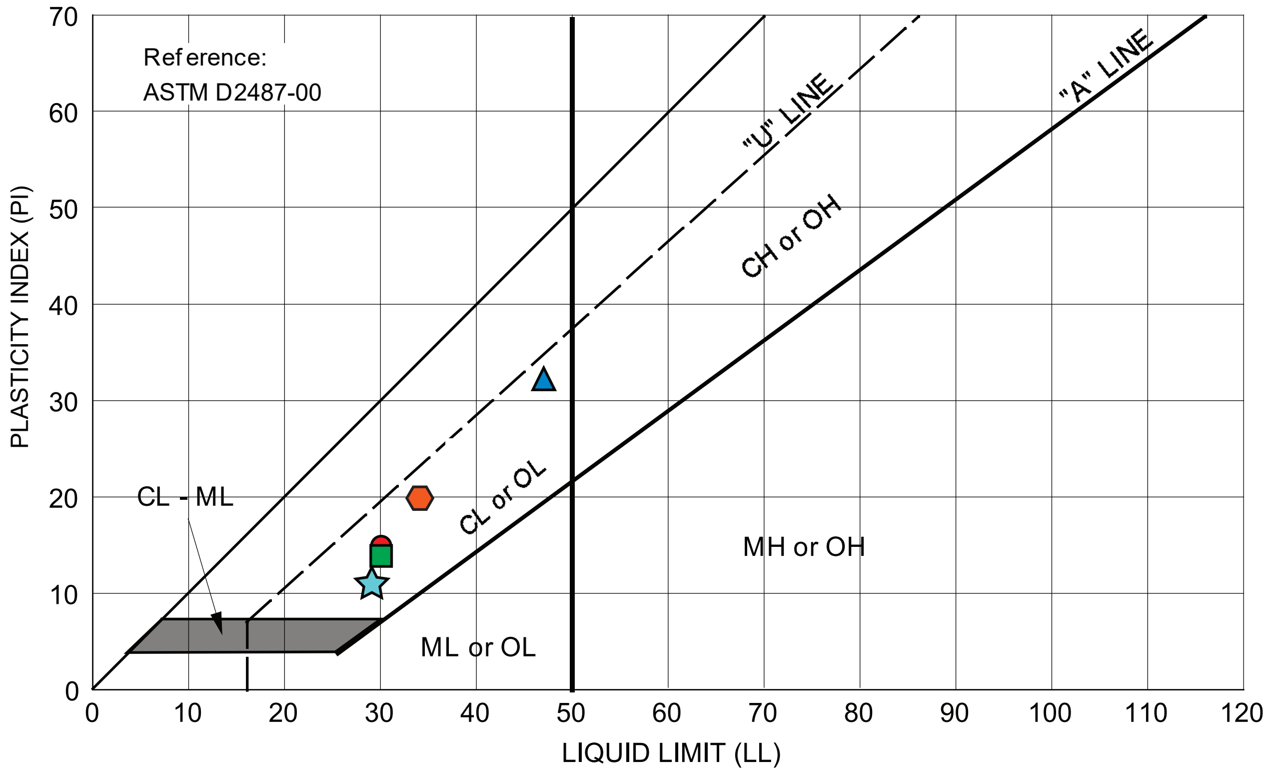
CLASSIFICATION CHART

Date 12/27/23 | Project No. 23-2527 | Figure A-7

APPENDIX B
Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 1.8 feet	SANDY CLAY (CL), dark brown	18.1	30	15	--
▲	B-1 at 5.5 feet	CLAY with SAND (CL), brown and red-brown	15.9	47	32	--
■	B-2 at 1.5 feet	SANDY CLAY (CL), dark brown to gray-brown	17.4	30	14	--
★	B-2 at 3.8 feet	CLAY with SAND (CL), brown	--	29	11	--
⬢	B-2 at 10.5 feet	CLAY with SAND (CL), light brown and yellow	27.6	34	20	77

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PLASTICITY CHART

Date 01/29/24 | Project No. 23-2527 | Figure B-1

Project X
Corrosion Engineering
Corrosion Control – Soil, Water, Metallurgy Testing Lab

REPORT S231226I

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-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: SANDY CLAY (CL), dark brown	1.3	8.5	0.0008	6.1	0.0006	3,484	3,216	7.5	156	2.5	8.1	1.6	ND	23.2	0.9	23.6	110.7	3.1	5.3
B-2: CLAY with SAND (CL), brown	3.3	10.9	0.0011	18.7	0.0019	2,948	2,881	7.3	158	2.8	11.2	1.2	ND	34.2	7.1	27.1	114.7	3.3	9.3

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops which is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

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SOIL CORROSIVITY
TEST RESULTS

Date 01/10/24 | Project No. 23-2527 | Figure B-2