ROCKRIDGE
GEOTECHNICAL

Prepared for **Newlife Investments, LLC**

FINAL GEOTECHNICAL INVESTIGATION
PROPOSED RAILROAD RESIDENCE DEVELOPMENT
RAILROAD AND S. LINDEN AVENUES
SOUTH SAN FRANCISCO, CALIFORNIA

September 18, 2023
Project No. 21-2085

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Newlife Investments, LLC
3646 Maxon Street
Chino, California 91710

ATTN: Ken Cui

Subject: Final Geotechnical Investigation
Proposed Railroad Residence Development
Railroad and S. Linden Avenues
South San Francisco, California

Dear Mr. Cui:

We are pleased to present the results of our final geotechnical investigation report for the proposed residential development and linear park to be constructed on the southern side of Railroad Avenue, between its intersections with S. Spruce and S. Linden avenues in South San Francisco, California. Our geotechnical investigation was performed in accordance with our proposal dated June 27, 2023.

The site consists of two parcels (APNs 014072050 and 014061170) and is bordered by Railroad Avenue to the north, S. Linden Avenue to the east, and commercial properties to the south and west. The site for the proposed residential development is a strip of land along Railroad Avenue that has plan dimensions of approximately 50 by 1,467 feet. The ground surface elevations on the residential development site are close to the grade on Railroad Avenue at the eastern and western ends and up to approximately 20 feet below the Railroad Avenue grade near the center. Where the grades on the residential development site are lower than Railroad Avenue, the northern portion of the site slopes down towards the south from an existing retaining wall along Railroad Avenue at a gradient as steep as 1.7:1 (horizontal to vertical).

The proposed residential development consists of constructing residential buildings containing 73 townhouse units. The proposed residential buildings will be 3 to 4 stories and of wood-framed construction. The ground level of the residential buildings will have finished floor near Railroad Avenue grade. Some of the buildings will have an accessory dwelling unit (ADU) below the ground floor. Improvements to the residential development site will also include publicly accessible open spaces and a shared rear drive aisle. The existing retaining wall supporting Railroad Avenue will be left in-place and a new retaining wall will be constructed downslope (to the south) of the existing wall.

Other site improvements include constructing a new linear park between existing neighboring commercial properties. The proposed linear park will extend from Railroad Avenue to N. Canal Street.

Based on the results of our geotechnical investigation, we conclude there are no major geotechnical issues that would preclude development of the site as proposed. The primary geotechnical issues affecting the proposed development include providing adequate foundation support for the proposed buildings and lateral support for the proposed retaining walls.

The recommendations contained in our final 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 preparation, shoring installation, 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 yours,
ROCKRIDGE GEOTECHNICAL, INC.

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FINAL GEOTECHNICAL INVESTIGATION
PROPOSED RAILROAD RESIDENCE DEVELOPMENT
RAILROAD AND S. LINDEN AVENUES
South San Francisco, California

1.0 INTRODUCTION

This report presents the results of the final geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed residential development and linear park to be constructed on the southern side of Railroad Avenue, between its intersections with S. Spruce and S. Linden avenues in South San Francisco, California, as shown on the Site Location Map, Figure 1. We previously performed a preliminary geotechnical investigation for the project, the results of which were presented in our report dated October 22, 2021.

The site consists of two parcels (APNs 014072050 and 014061170) and is bordered by Railroad Avenue to the north, S. Linden Avenue to the east, and commercial properties to the south and west. The site for the proposed residential development is a strip of land along Railroad Avenue that has plan dimensions of approximately 50 by 1,467 feet, as shown on the Site Plan, Figure 2. The ground surface elevations on the residential development site are close to the grade on Railroad Avenue at the eastern and western ends and up to approximately 20 feet below the Railroad Avenue grade near the center. Where the grades on the residential development site are lower than Railroad Avenue, the northern portion of the site slopes down towards the south from an existing retaining wall along Railroad Avenue at a gradient as steep as 1.7:1 (horizontal to vertical).

The proposed residential development consists of constructing residential buildings containing 73 townhouse units. The proposed residential buildings will be 3 to 4 stories and of wood-framed construction. The ground level of the residential buildings will have finished floor near Railroad Avenue grade. Some of the buildings will have an accessory dwelling unit (ADU) below the ground floor. Improvements to the residential development site will also include publicly accessible open spaces and a shared rear drive aisle. The existing retaining wall supporting

Railroad Avenue will be left in-place and a new retaining wall will be constructed downslope (to the south) of the existing wall.

Other site improvements include constructing a new linear park between existing neighboring commercial properties. The proposed linear park will extend from Railroad Avenue to N. Canal Street.

2.0 SCOPE OF SERVICES

Our final geotechnical investigation was performed in accordance with our proposal dated June 27, 2023. Our scope of services consisted of exploring subsurface conditions at the site by drilling supplemental borings, performing laboratory testing on selected soil samples, and performing engineering analyses to develop conclusions and recommendations regarding:

- subsurface conditions
- site seismicity and seismic hazards, including the potential for liquefaction, cyclic densification, and seismically-induced landslides
- the most appropriate foundation type(s) for the proposed buildings
- design criteria for the recommended foundation type(s), including vertical and lateral capacities for each of the foundation type(s)
- estimates of foundation settlement
- design pressures for permanent walls
- temporary cut slopes and shoring
- site grading, subgrade preparation, and fill quality and compaction
- exterior concrete flatwork
- non-permeable and permeable concrete pavers
- 2022 California Building Code (CBC) site class and design spectral response acceleration parameters
- corrosivity of the near-surface soil and groundwater and the potential effects on buried concrete and metal structures and foundations
- construction considerations.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

We previously performed a preliminary geotechnical investigation for this project, the results of which were presented in our report dated October 22, 2021. Our preliminary investigation consisted of evaluating subsurface conditions at the site by drilling seven test borings, performing five dynamic penetrometer tests (DPTs), and performing laboratory tests on selected soil samples. For our final investigation, we supplemented the subsurface data on the Railroad Avenue retaining wall slope by drilling six borings and performing laboratory tests on selected soil samples. The approximate locations of the borings and DPTs, including those performed for our preliminary investigation, are shown on Figure 2.

Prior to drilling we obtained a drilling permit from the San Mateo County Environmental Health Services Division (SMCEHSD) and contacted Underground Service Alert (USA) to notify them of our work, as required by law. We also retained private utility locators, Precision Locating, LLC and C. Cruz Sub-Surface Locators, to check for buried utilities at the boring locations to reduce the potential of encountering utilities during drilling. Details of the preliminary and final field investigations and laboratory testing are described below.

3.1 Test Borings

Seven borings, designated as B-1 through B-7, were drilled at the approximate locations shown on Figure 2 as part of our preliminary investigation. Borings B-1 through B-4 were drilled on September 8, 2021 by Exploration Geoservices, Inc. of San Jose, California, using a Mobile B-61 truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers. Borings B-5 through B-7 were drilled on September 24, 2021 by Access Soil Drilling, Inc. of San Mateo, California, using a limited access drill rig equipped with 3-inch-diameter solid-stem augers. Borings B-1 through B-5 were drilled to depths between 7.9 and 21.4 feet below the ground surface (bgs) where the boreholes bottomed in bedrock. Borings B-6 and B-7 were drilled to depths of 11.5 and 13 feet bgs, respectively, where the boreholes bottomed in Bay Mud tidal deposits.

Six additional borings, designated as B-8 through B-13, were drilled at the approximate locations shown on Figure 2 on the sloped portion of the site as part of our final investigation. Borings B-8 through B-13 were drilled on July 17 and 18, 2023 by Access Soil Drilling, Inc. of San Mateo, California, using a limited access drill rig equipped with 3-inch-diameter solid-stem augers. Borings B-8 through B-13 were drilled to depths between 2.3 and 14.3 feet bgs and bottomed in bedrock.

During drilling, our field engineer logged the soil and bedrock encountered and obtained representative samples of the soil and bedrock for visual classification and laboratory testing. The logs for borings B-1 through B-13 are presented on Figures A-1 through A-13 in Appendix A. The soil and bedrock encountered in the borings were classified in accordance with the classification systems shown on Figures A-14 and A-15, respectively.

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.
- California (CA) split-barrel sampler with a 2.5-inch outside diameter and a 2.0-inch inside diameter, lined with 1.875-inch diameter liners.
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter; the sampler can accommodate liners, but liners were not used.

The SPT, CA, and MC samplers were driven with a 140-pound hammer falling 30 inches per drop. For the borings drilled by Exploration Geoservices, Inc., a downhole wireline hammer on a Mobile B61 rig was used. For the borings drilled by Access Soil Drilling, Inc., a rope-and-cathead safety hammer on the limited access rig was used. The samplers were driven up to 18 or 24 inches and the hammer blows required to drive the samplers were recorded every 6 inches and are presented on 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, for the Mobile B-61 rig to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was designed to accommodate liners, but liners were not used. Factors of 0.7, 0.9, and 1.2 were used to convert the blow counts for MC, CA, and SPT samplers to approximate SPT N-values, respectively, for the limited access rig to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was designed to accommodate liners, but liners were not used. The blow counts used for this conversion were: 1) the last two blow counts if the sampler was driven more than 12 inches, or 2) the last blow count if the sampler was driven less than 12 inches. The converted SPT N-values are presented on the boring logs.

Upon completion, the boreholes were backfilled with neat cement grout in accordance with SMCEHSD requirements. The soil cuttings generated during drilling were spread in landscape areas onsite.

3.2 Dynamic Penetrometer Tests

Subsurface conditions of the slope along Railroad Avenue were also investigated by performing five DPTs, designated as DPT-1 through DPT-5, at the approximate locations shown on Figure 2 during our preliminary investigation. 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 consist 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 SPT N-values for use in our engineering analyses. The DPTs were advanced to practical refusal, defined as more than 50 blows per 10-centimeter interval, at depths ranging from approximately 1.6 to 4.6 feet bgs. The DPT results are presented on Figure A-16.

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. Soil samples were tested by B. Hillebrandt Soils Testing, Inc. of Alamo, California to measure moisture content, fines content, and Atterberg limits (plasticity index). Near-surface soil samples were tested by Project X Corrosion

Engineering in Murrieta, California. The laboratory test results are presented on the boring logs and in Appendix B.

4.0 SUBSURFACE CONDITIONS

The Regional Geologic Map (Figure 3) for the site vicinity indicates the proposed residential development portion of the site (along Railroad Avenue) is underlain by slope debris and ravine fill (Qsr), Colma formation (Qc), and sandstone and shale bedrock (KJs). The geology map shows the proposed linear park area as being underlain by slope debris and ravine fill (Qsr), Colma formation (Qc), sandstone and shale bedrock (KJs), and artificial fill over tidal flats (Qaf/tf).

The results of our borings and DPTs indicate the proposed residential development site is underlain by slope debris overlying Colma formation overlying residual soil and bedrock. Where explored, the slope debris consists of dense to very dense clayey sand and hard sandy clay; the Colma formation consists of very stiff to hard sandy silt/clay and dense to very dense clayey sand; and the residual soil consists of medium dense to very dense sand and silty sand and very stiff to hard clay with sand and gravel. Top of bedrock was encountered at depths between of 1 and 23 feet bgs. The bedrock consists of sandstone and shale that has low hardness and is friable to weak and deeply to moderately weathered.

The results of borings indicate the proposed linear park area site is underlain by fill consisting of stiff to very stiff sandy clay and medium dense to dense clayey sand with gravel and gravel with varying amounts of silt and sand to depths of 3 to 8 feet bgs; the thickness of the fill increases towards the south. The fill on the northern portion of the proposed linear park site (i.e., Boring B-5) is underlain by about 2 feet of residual soil consisting of very dense silty sand with gravel overlying bedrock. The fill on the southern portion of the proposed linear park site (i.e., borings B-6 and B-7) extends to depths of about 7 to 8 feet bgs and is underlain by Bay Mud tidal deposits consisting of medium stiff to stiff clay and silty clay. The Bay Mud tidal deposits extend to the maximum depths explored in borings B-6 and B-7 of 11.5 and 13 feet bgs, respectively.

4.1 Groundwater

Groundwater was measured at depths of approximately 16 and 11.5 feet bgs in borings B-4 and B-7, respectively, during drilling. Groundwater was not present in borings B-1, B-2, B-3, B-5, B-6, and B-8 through B-13 during drilling. It should be noted the groundwater level in the borings was likely not given adequate time to stabilize at the time of drilling and groundwater level measurements.

In the California Geologic Survey (CGS) report *Seismic Hazard Zone Report for the San Francisco South 7.5-Minute Quadrangle, San Mateo County, California*, Plate 1.3 shows the historic high groundwater at the site is approximately 10 feet bgs near the proposed residential development area and approximately 2 feet bgs near the southern end of the proposed linear park area.

To further evaluate depth to groundwater at the site, we reviewed groundwater data on the State of California Water Resources Control Board GeoTracker website (http://geotracker.waterboards.ca.gov). At a site approximately 400 feet east of the site, located at 7 S. Linden Avenue, groundwater monitoring wells were monitored from February 2004 to December 2018; the groundwater readings showed that the groundwater generally fluctuated from 3 to 11 feet bgs. Another site, approximately 650 feet north of the site, located at 123 Linden Avenue, presents groundwater level readings taken periodically between March 1999 and March 2013; groundwater readings showed that the groundwater fluctuated between 1.5 to 11 feet bgs. A third site, approximately 1000 feet south of the residential portion of the site, located at 114 S. Maple Avenue, had groundwater monitoring wells monitored from December 1998 to March 2015; the groundwater readings showed that the groundwater generally fluctuated from 1.7 to 6.4 feet bgs.

The groundwater level at the site is expected to fluctuate several feet seasonally with potentially larger fluctuations annually, depending on the amount of rainfall. Based on the available groundwater information for the site and vicinity, we conclude a groundwater depth of 10 feet bgs should be used for the residential portion of the site along Railroad Avenue, with the groundwater depth decreasing to approximately 2 feet bgs at the southern end of the linear park portion of the site.

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 Hayward faults. These and other faults in the region are shown on Figure 4. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[1] [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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[1] 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)	4.2	Southwest	8.04
North San Andreas (Peninsula, SAP)	4.2	Southwest	7.38
San Gregorio (North)	13	West	7.44
Monte Vista - Shannon	21	Southeast	7.14
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	25	East	7.58
Hayward (South, HS)	25	East	7.00
Hayward (North, HN)	25	Northeast	6.90
North San Andreas (North Coast, SAN)	35	Northwest	7.52
Butano	39	South	6.93
Total Calaveras (CN+CC+CS+CE)	39	East	7.43
Calaveras (North, CN)	39	East	6.86
Mount Diablo Thrust	41	Northeast	6.67
Mount Diablo Thrust North CFM	42	Northeast	6.72
Concord	46	Northeast	6.45
Mount Diablo Thrust South	47	East	6.50

Damaging earthquakes have occurred along many of these faults in recorded history, as depicted on 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

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with ground shaking, ground surface rupture, liquefaction,[2] lateral spreading,[3] and cyclic densification.[4] The results of our evaluation are presented in this section.

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 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 severe 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 is very low. In a seismically active area, the remote possibility exists for future faulting in areas where no faults

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

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

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

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

5.2.3 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.

As presented on Figure 5, the proposed residential development area and the northern portion of the proposed linear park area are not within a designated zone of liquefaction potential on the map titled *Earthquake Zones of Required Investigation, South San Francisco South Quadrangle,* prepared by the California Geological Survey (CGS), dated September 23, 2021. The southern portion of the proposed linear park is mapped within a designated zone of liquefaction potential.

The results of our subsurface investigation indicate the proposed residential development area and the northern portion of the proposed linear park area are underlain very stiff to hard clayey soil, dense to very dense sandy soil, and bedrock. We judge the clayey and sandy soils beneath these two areas are not susceptible to liquefaction due to their cohesion and/or relative density. Therefore, we conclude the potential for liquefaction to occur in these two areas is very low.

The results of our subsurface investigation indicate the southern portion of the proposed linear park is underlain by fill overlying Bay Mud tidal deposits. The fill may contain lenses of medium dense clayey sand that is susceptible to liquefaction. The Bay Mud tidal deposits may also contain lenses of silty sand and sandy silt that may be susceptible to liquefaction
Based on subsurface conditions encountered in borings B-6 and B-7, we estimate liquefaction-induced settlement will be less than 3/4 inch and less than 1/2 inch across a horizontal distance of 30 feet. The non-liquefiable soil overlying the potentially liquefiable soil layers is sufficiently thick and the potentially liquefiable layers are sufficiently thin such that the potential for surface

manifestations of liquefaction, such as sand boils, is low. Considering the potentially liquefiable layers are not continuous, we conclude the risk of lateral spreading is low.

5.2.4 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. Based on the subsurface data from our field investigation, we conclude the soil above the groundwater table is not susceptible to cyclic densification because of its cohesion and/or relative density.

6.0 DISCUSSIONS AND CONCLUSIONS

We conclude that from a geotechnical engineering standpoint, the site can be developed as planned, provided the recommendation presented in this report are incorporated into the project plans and specifications and implemented during construction. The primary geotechnical issues affecting the proposed development are providing adequate foundation support for the proposed buildings and lateral support for the existing and proposed retaining walls. These and other geotechnical issues as they pertain to the proposed development are presented in this section.

6.1 Slope Stability Considerations

The site for the proposed residential development is a strip of land along Railroad Avenue that has plan dimensions of approximately 50 by 1,467 feet. The ground surface elevations on the residential development site are close to the grade on Railroad Avenue at the eastern and western ends and up to approximately 20 feet below the Railroad Avenue grade near the center. Where the grades on the residential development site are lower than Railroad Avenue, the northern portion of the site slopes down towards the south from an existing retaining wall along Railroad Avenue at a gradient as steep as 1.7:1 (horizontal to vertical).

Based on the results of our investigation, we judge the existing slope is stable in its current condition. The project includes constructing retaining walls to provide lateral support for the existing retaining wall, new fill that will be placed on the slope, and new buildings and improvements on the slope. We conclude the existing slope along the proposed improvements will be stable under static and seismic conditions, provided the proposed improvements, including new retaining walls, and foundation elements are designed and constructed following the recommendations presented in this report, and surface water are prevented from being discharged onto slopes.

6.2 Foundation Support and Settlement

The site for the proposed residential development is underlain by firm native soil that can provide adequate foundation support for light to moderate building loads. Therefore, we judge the proposed buildings may be supported on spread footings where the ground is relatively level. Where the ground is steeper than 3:1, we recommend the proposed building be supported on drilled piers.

We estimate total and differential static settlements for properly designed and constructed spread footings will be less than 3/4 inch and 1/2 inch over a horizontal distance of 30 feet, respectively. We also estimate total and differential settlement of properly constructed drilled piers designed based on the recommendations presented in this report will be less than 1/2 inch and 1/4 inch over a horizontal distance of 30 feet, respectively.

6.3 Excavation Support

Excavations that will be entered by workers should be sloped or shored in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). Where space permits, the sides of the temporary excavation can be sloped. Where space does not permit sloping of the excavation perimeter, a shoring system will be required to support the sides of the proposed excavation. The contractor should be responsible for the construction and safety of temporary slopes and shoring. The shoring designer should be responsible for the shoring design.

We judge that a cantilevered soldier pile and lagging shoring system is appropriate for support of excavations that are less than 12 feet deep. Where cuts exceed about 12 feet in height, soldier pile-and-lagging systems are typically more economical if they include tieback anchors;

however, tieback anchors will likely extend beneath the streets, which will require an encroachment agreement with the City of South San Francisco. Where it is not feasible to install tiebacks, then internal bracing of the shoring will be required. If tiebacks or internal bracing are required, we can provide recommendations upon request.

6.4 Construction Considerations and Monitoring

The soil to be excavated consists predominantly of sand and clay, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. Excavations will also likely extend into bedrock and, therefore, contractors should be prepared to use equipment capable of excavating and drilling into rock. Removal of existing on-site improvements, including pavements and buried foundations will require equipment capable of breaking concrete.

Special care should be taken to not undermine the existing retaining wall along Railroad Avenue during construction. Where there are existing structures nearby, heavy equipment should not be used within 10 horizontal feet from existing structures, including the existing retaining wall. Jumping jack or hand-operated vibratory plate compactors should be used for compacting fill within this zone.

The contractor should establish survey points on the shoring, adjacent streets, and adjacent buildings to monitor the movement during and immediately after excavation. Further, because adjacent streets (i.e., Railroad Avenue) and buildings may experience settlement during construction of the proposed project, a crack survey should be performed on adjacent streets prior to the start of excavation.

6.5 Soil Corrosivity

Corrosivity tests were performed by Project X Corrosion Engineering of Murrieta, California on soil samples obtained from borings B-1, B-3, B-4, and B-8 at a depths between 1 and 6 feet bgs. The corrosivity test results are presented in Appendix B of this report.

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 ranging from 1,876 to 5,025 ohm-cm, we conclude the soil is “moderately 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 (7.8 to 8.4) indicate the near-surface soil is “negligibly corrosive” to buried metallic and concrete structures. The chloride ion concentration (11.7 to 100.3 mg/kg) indicates the chlorides in the near-surface soil are “mildly to negligibly corrosive” to buried metallic structures and reinforcing steel in concrete structures below ground. The results also indicate the sulfate ion concentration (32.1 to 96.6 mg/kg) is sufficiently low such that sulfates do not pose a threat to buried concrete and mortars.

7.0 RECOMMENDATIONS

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

7.1 Site Preparation and Grading

Any vegetation and organic topsoil should be stripped and disposed of off-site. Site demolition should include the removal of all existing foundation elements and underground utilities, if any. In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside of the proposed building footprints and/or will not interfere with the proposed construction, they may be abandoned in-place provided they 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 the observation of our field engineer and in accordance with our compaction recommendations provide in this section.

If grading is performed during the rainy season, the contractor may find the subgrade material too wet to compact to the recommended relative compaction and will have to be scarified and aerated to lower its moisture content so the recommended compaction can be achieved. Material to be dried by aeration should be scarified to a depth of at least 8 inches; the scarified soil should be turned at least twice a day to promote uniform drying. Once the moisture content of the aerated soil has been reduced to acceptable levels, the soil should be compacted in accordance with our recommendations. Aeration is typically the least costly method used to stabilize the subgrade soil; however, it generally takes the most time and favorable weather conditions to complete. Other soil stabilization alternatives include over-excavating the wet soil and replacing or mixing it with drier soil, and chemical treatment.

7.1.1 Subgrade Preparation

The soil subgrade in areas that will receive improvements (i.e., slab-on-grade floors or exterior concrete flatwork) or fill should be scarified to a depth of at least 8 inches, moisture-conditioned to near optimum moisture-content, and compacted to at least 90 percent relative compaction.[5] If bedrock is exposed at subgrade elevation, it is not necessary to scarify or compact the bedrock. Where the building pad will have a crawl space and will not have a slab-on-grade floor, the subgrade does not need to be scarified and recompacted. However, if there will be a crawl space, we recommend a 2- to 3-inch-thick concrete rat slab be placed on the subgrade.

7.1.2 Fill Quality and Compaction

Engineered fill may consist of on-site soil or imported fill that is free of organic matter and contains no rocks or lumps larger than 3 inches in greatest dimension. If imported fill (select fill) is required, it should also have a liquid limit less than 40 and plasticity index less than 12, 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

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[5] 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.

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. Fill consisting of clean sand or gravel (defined as poorly-graded soil with less than 5 percent fines by weight) or more than 5 feet thick should be compacted to at least 95 percent relative compaction. Fill placed within 8 inches of pavement soil subgrade that will be subjected to vehicular traffic should also be moisture-conditioned to near optimum moisture content, and compacted to at least 95 percent relative compaction and be non-yielding.

7.1.3 Utility Trenches

Excavations for utility trenches should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of 4 inches of sand or fine gravel. After the pipes and conduits are tested, inspected (if required), and approved, they should be covered to a depth of 6 inches with sand or fine gravel, which should be mechanically tamped. Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted as according to the recommendations previously presented. Jetting of trench backfill should not be permitted.

Spread footings should be bottomed below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of utility trenches running parallel to the footings. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with controlled low-strength material (CLSM) with a 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).

7.1.4 Exterior Concrete Flatwork

We recommend a minimum of 4 inches of Class 2 aggregate base be placed beneath exterior concrete flatwork (i.e., sidewalks and patios). The soil subgrade should be scarified to a depth of at least 8 inches, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction. If bedrock is exposed at subgrade elevation, it is not necessary to scarify or compact the bedrock. Class 2 aggregate base beneath concrete flatwork should also be compacted to at least 90 percent relative compaction.

7.1.5 Surface Drainage and Bioswales

Positive surface drainage should be provided around the buildings to direct surface water away from the foundations and slopes. Grades around the building should be determined 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 and slopes. The use of water-intensive landscaping around the perimeter of the building should be avoided to reduce the amount of water introduced to the soil subgrade.

Care should be taken to minimize the potential for subsurface water to collect beneath pavements and pedestrian walkways, and to be discharged onto slopes. 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.

Stormwater treatment systems (infiltration basins, rain gardens, bio-retention systems, vegetated swales, flow-through planters, etc.) should be provided with subdrains. Within 5 feet of the proposed buildings, excavations for stormwater treatment systems should have an impermeable liner in addition to the subdrain. Due to the relatively low estimated permeability of the near-surface soil, these systems should be designed for partial exfiltration. The drainage layer beneath the “treatment” soil should consist of a minimum 12-inch-thick layer of Caltrans Class 2 Permeable drainage material and include a minimum 4-inch-diameter perforated drain pipe (perforations facing down).

7.2 Foundation and Settlement

We conclude the proposed townhome buildings may be supported on conventional spread footings or drilled piers. Recommendations for spread footings and drilled piers are presented in this section.

7.2.1 Spread Footings

Spread footings should bottom at least 24 inches below the ground surface and bear on firm native soil or bedrock. Spread footings situated near a slope should be setback at least 5 feet horizontally from the top of slope. Alternatively, the footings may be deepened, such that there is at least 7 feet of horizontal distance between the bottom edge of the footing and the face of the slope. Footings adjacent to utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upwards from the bottom edge of the utility trench.

Spread footings may be designed using allowable bearing pressures of 5,000 pounds per square foot (psf) for dead-plus-live loads and 6,650 psf for total design loads, which includes wind or seismic forces; these values include factors of safety of at least 2.0 and 1.5, respectively.

Lateral loads may be resisted by a combination of passive pressures on the vertical faces of the footings and friction between the bottoms of the footings and the supporting soil. To compute lateral resistance, we recommend using an equivalent fluid weight of 300 pounds per cubic foot (pcf); the upper foot of soil should be ignored for lateral resistance unless confined by a slab. Where the footing is situated near a sloped surface, the depth where the soil can be relied upon for lateral resistance is beneath where there is at least 7 feet of horizontal distance between the edge of the footing and the face of the slope. Frictional resistance should be computed using a base friction coefficient of 0.35 or 0.4 for footings bearing on native soil or bedrock, respectively. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without further reduction.

The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. We should check footing excavations prior to placement of reinforcing steel to confirm the excavations are bottomed on suitable bearing material and have been properly prepared.

7.2.2 Drilled Piers

Drilled piers should be spaced at least three diameters on center. Drilled piers should be at least 8 feet long or extend at least 4 feet into bedrock, whichever is deeper.

Drilled piers should be designed to derive their axial capacity from skin friction in native soil and bedrock starting at a depth of 2 feet below bottom of the grade beam. To compute axial capacity for dead-plus-live loads acting in compression, we recommend using an allowable skin friction 500 psf in native soil and 1,000 psf in bedrock. Skin friction from the upper 2 feet of pier and end bearing should be ignored for vertical support. These skin friction values may be increased by one-third for total load conditions. To compute uplift resistance for the piers, the same skin friction values provided for dead-plus-live loads may be used.

Drilled piers situated on slopes steeper than 3:1 (horizontal to vertical) should be designed to resist downslope creep movement. The piers should be designed for a “creep load” that would act on the piers using an equivalent fluid weight of 51 pcf acting over one pier diameter and to the upper 8 feet of pier where the pier is situated near top of slope (i.e., near northern portion of site, adjacent to Railroad Avenue); and decreasing to 2 feet at base of slope (i.e., at southern limit of residential development site). Piers should be tied together with well-reinforced grade beams running perpendicular to the slope contours. Isolated piers should not be used.

To compute lateral resistance, we recommend using an equivalent fluid weight of 300 pcf; the upper foot of soil should be ignored for lateral resistance unless confined by a slab. Where the pier is near a sloped surface, the depth where the soil can be relied upon for lateral resistance is beneath where there is at least 7 feet of horizontal distance between the pier and the face of the slope. The passive pressure value includes a factor of safety of at least 1.5. The passive pressure may be assumed to act over a width of two pier diameters, or center-to-center spacing between piers, whichever distance is shorter. Passive pressure should not be used for lateral resistance below a depth of about 8 feet. Below this depth, excessive deflections of the pier head would be required to mobilize the passive pressure.

Drilled piers should be installed by a qualified contractor with demonstrated experience in this type of foundation and subsurface conditions, including drilling into bedrock. The bottoms of the pier holes should be free of debris and water before placement of concrete. If groundwater is encountered during pier drilling, the pier hole should be pumped dry prior to placement of concrete. If the hole cannot be pumped dry prior to placement of concrete, then the concrete should be placed by tremie methods.

Concrete used for pier construction should be discharged vertically using a hose to tremie fill the drilled holes to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during pouring. Concrete should be placed in the pier holes within 24 hours of completion of drilling if groundwater is encountered.

7.3 Concrete Slab-on-Grade Floor

Where the proposed buildings will have a crawl space underneath the floor, the recommendations below may be ignored. However, we recommend a 2- to 3-inch-thick concrete rat slab be placed on the soil subgrade of the crawl space.

The subgrade for the concrete slab-on-grade floor should be prepared in accordance with our recommendations in Section 7.1.1. We recommend installing a capillary moisture break and water vapor retarder beneath the floor slab. 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 2.

TABLE 2
Gradation Requirements for Capillary Moisture Break

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

The vapor retarder should meet the requirements for Class B vapor retarders stated in ASTM E1745. 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 moisture emission levels (if emission testing is required) meet the manufacturer’s requirements.

7.4 Permanent Retaining Walls

Permanent retaining walls, including site retaining walls and basement walls, should be designed to resist lateral earth pressures imposed by the retained soil, as well as surcharge pressures from nearby foundations and traffic, 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 to resist pressures associated with seismic forces.

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

- Restrained Wall - At-rest earth pressure using an equivalent fluid weight of 51 pcf for drained conditions
- Unrestrained Wall - Active earth pressure using an equivalent fluid eight of 33 pcf for drained 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 33 pcf plus a seismic increment of 47 pcf for drained conditions
- Unrestrained Wall - Active earth pressure using an equivalent fluid weight of 33 pcf plus a seismic increment of 21 pcf for drained conditions

The recommended lateral earth pressures above are based on a level backfill conditions with no additional surcharge loads. If the backfill behind the retaining walls will not be level, we can provide sloped pressures upon request. Where the retaining/below-grade wall is subject to traffic loading within a horizontal distance equal to 1.5 times the height of the wall, the wall should be designed for vehicular surcharge of 100 psf over the entire height of the wall. This surcharge pressure assumes the vehicular traffic on Railroad Avenue is at least 5 feet from the face of the existing wall.

The design pressures recommended above are based on fully drained walls. Although new 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 retaining walls is to place a prefabricated drainage panel against the shoring or the back of the walls. The drainage panel should extend down to a perforated PVC collector pipe at the base of the wall. The pipe should be surrounded on all sides by at least 4 inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi 150N or equivalent). Where shoring is installed and there is insufficient room to install a perforated pipe between the shoring and the back of the retaining/below-grade wall, the drainage panel should extend down to a proprietary, prefabricated collector drain system, such as Tremdrain Total Drain or Hydroduct Coil (or equivalent), designed to work in conjunction with the drainage panel and may be used in lieu of the perforated pipe surrounded by gravel described above. The pipe should be connected to a suitable discharge point.

Retaining walls may be supported by spread footings or drilled piers designed using the recommendations presented in Section 7.2 of this report. If backfill is required behind retaining walls, the walls should be braced or hand compaction equipment used to prevent unacceptable surcharges on walls (as determined by the structural engineer).

7.5 Temporary Cut Slopes and Shoring

Excavations that will be entered by workers should be sloped or shored in accordance with the Occupational Safety and Health Administration (OSHA) standards (29 CFR Part 1926). Where space permits, the sides of the temporary excavation can be sloped. We recommend temporary slopes not exceed an inclination of 1.5:1 (horizontal to vertical) in sand or silty/clayey sand (OSHA Type C soil) or 3/4:1 in bedrock.

7.5.1 Cantilevered Soldier Pile and Timber Lagging Shoring System

For design of a cantilevered soldier pile and timber lagging shoring system, we recommend using an at-rest earth pressure equivalent to a fluid weight of 51 pcf where there is a structure within a horizontal distance equal to 1.5 times the retained soil height and using an active earth pressure equivalent to a fluid weight of 33 pcf where there are no structures within that horizontal distance. The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads. If the backfill behind the shoring system will not be level, we can provide sloped pressures upon request.

Where there will be vehicular traffic behind the top of the shoring system within a horizontal distance equal to 1.5 times the height of the wall, the wall should be designed for vehicular surcharge of 100 psf acting over the upper 10 feet. Shoring should be designed for surcharge loads from construction equipment and/or stockpiled soil within a horizontal distance of 1.5

times the excavation height from the edge of excavation, and from adjacent foundations that are not underpinned and are located above an imaginary line that extends at an inclination of 1.5:1 (horizontal to vertical) projected upward from the bottom edge of the proposed excavation. We can provide recommendations for surcharge pressures once surcharge loads are known

Passive resistance at the toe of the soldier piles should be computed using an equivalent fluid weight of 300 pcf with a maximum pressure of 3,000 psf. The upper foot of soil should be ignored when computing passive resistance. Passive pressure can be assumed to act over an area of three soldier pile widths, assuming the toe of the soldier pile is filled with structural concrete. If lean concrete is placed in the soldier pile shaft, the passive pressure can be assumed to act over two pile diameters. These passive pressure values include a factor of safety of at least 1.5.

Soldier piles should be placed in pre-drilled holes backfilled with concrete. Soldier piles will likely extend into bedrock, so contractors should be prepared to use equipment capable of drilling in bedrock.

A structural/civil engineer knowledgeable in this type of construction should be retained to design the shoring. The shoring designer should design the shoring system for lateral deformation of less than 1/2 inch at any location on the shoring where there is a structure or improvements within a horizontal distance equal to 1.5 times the retained soil height and 1 inch where there are no structures or improvements within that horizontal distance. We should review the final shoring plans and calculations to check that they are consistent with the recommendations presented in this report.

7.6 Non-Vehicular Concrete Pavers

The section presents our recommendations for non-permeable and permeable concrete pavers for pedestrian traffic.

7.6.1 Non-Permeable Concrete Pavers for Pedestrian Traffic

Non-permeable concrete pavers for pedestrian traffic may be 60 millimeters (2.375 inches) thick and should be underlain by at least 4 inches of Class 2 aggregate base compacted to at least 90 percent relative compaction. The soil subgrade beneath the aggregate base should be scarified to a depth of at least 8 inches, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction.

7.6.2 Permeable Concrete Pavers for Pedestrian Traffic

We recommend permeable interlocking concrete pavements (ICP) be designed in accordance with the guidelines presented by the Interlocking Concrete Pavement Institute (ICPI 2017). These guidelines include specific recommendations for permeable aggregate subbase, base, and bedding courses to be placed beneath ICP pavements. We recommend permeable pavements for pedestrian traffic be designed for partial exfiltration of water into the subgrade soil. This requires installing a subdrain system at the base of the pervious aggregate materials, which are underlain by a filter fabric. ICPI's generalized paver section for pedestrian traffic is presented on Figure 6. Where partial exfiltration is installed, some movement should be anticipated if this results in drying and wetting of the subgrade soil.

The soil subgrade beneath ICP pavements should be scarified to a depth of at least 8 inches, moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent relative compaction, prior to placing the filter fabric and aggregate materials. The soil subgrade at the bottom of the permeable section should slope down toward the drain pipe trench at a gradient of at least 2 percent. The perforated pipe should slope down to a suitable outlet at a minimum gradient of 1 percent. The pipe should be placed with the perforations down on a minimum of 2 inches of permeable subbase.

ICPI's guidelines call for 2 inches of bedding material consisting of ASTM No. 8 crushed aggregate directly below the pavers. This material is also recommended for fill material between the pavers. As shown in Table 3 below, this material consists of fine gravel with 10 to 30 percent sand.

TABLE 3
Gradation Requirements for ASTM No. 8 Crushed Aggregate

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

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

TABLE 4
Gradation Requirements for ASTM No. 57 Crushed Aggregate

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

The No. 57 aggregate should be placed in lifts not exceeding 8 inches in loose thickness and compacted with a smooth-drum roller in vibratory mode with sufficient passes to create an unyielding surface. Placement and compaction of the permeable aggregate base and subbase should be performed under the observation of our field engineer. Following compaction of the No. 57 aggregate, the No. 8 bedding, not exceeding 2 inches in loose thickness, should be placed and screeded to a level, undisturbed surface immediately prior to paver installation.

The required thicknesses of the permeable aggregate base and subbase courses depends on the infiltration and water storage design requirements, as well as the traffic loading demand. Our recommendations for the minimum permeable ICP pavement sections subject to pedestrian traffic is 2 inches of No. 8 bedding underlain by 6 inches of No. 57 aggregate over soil subgrade. A thicker base of No. 57 aggregate may be used for additional water storage.

The above recommended ICP pavement sections are based on the ICPI technical guidelines (ICPI 2017). From a geotechnical standpoint, it is also acceptable to use compacted structural planting mix in lieu of the No. 57 base courses in locations where the pedestrian ICP section is adjacent to tree wells and is required for promoting root growth.

7.7 Seismic Design

The latitude and longitude of the site are 37.6527° and -122.4134°, respectively. For design of the proposed buildings in accordance with 2022 CBC (ASCE 7-16), we recommend the following:

- Site Class C (very dense soil and soft rock)
- $S_S = 2.029$, $S_1 = 0.839g$
- $F_a = 1.2$, $F_v = 1.4$
- $S_{MS} = 2.435g$, $S_{M1} = 1.175g$
- $S_{DS} = 1.623g$, $S_{D1} = 0.783g$
- Seismic Design Category E for Risk Factors I, II, and III

8.0 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 shoring and 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.

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 are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the borings and DPTs. 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.

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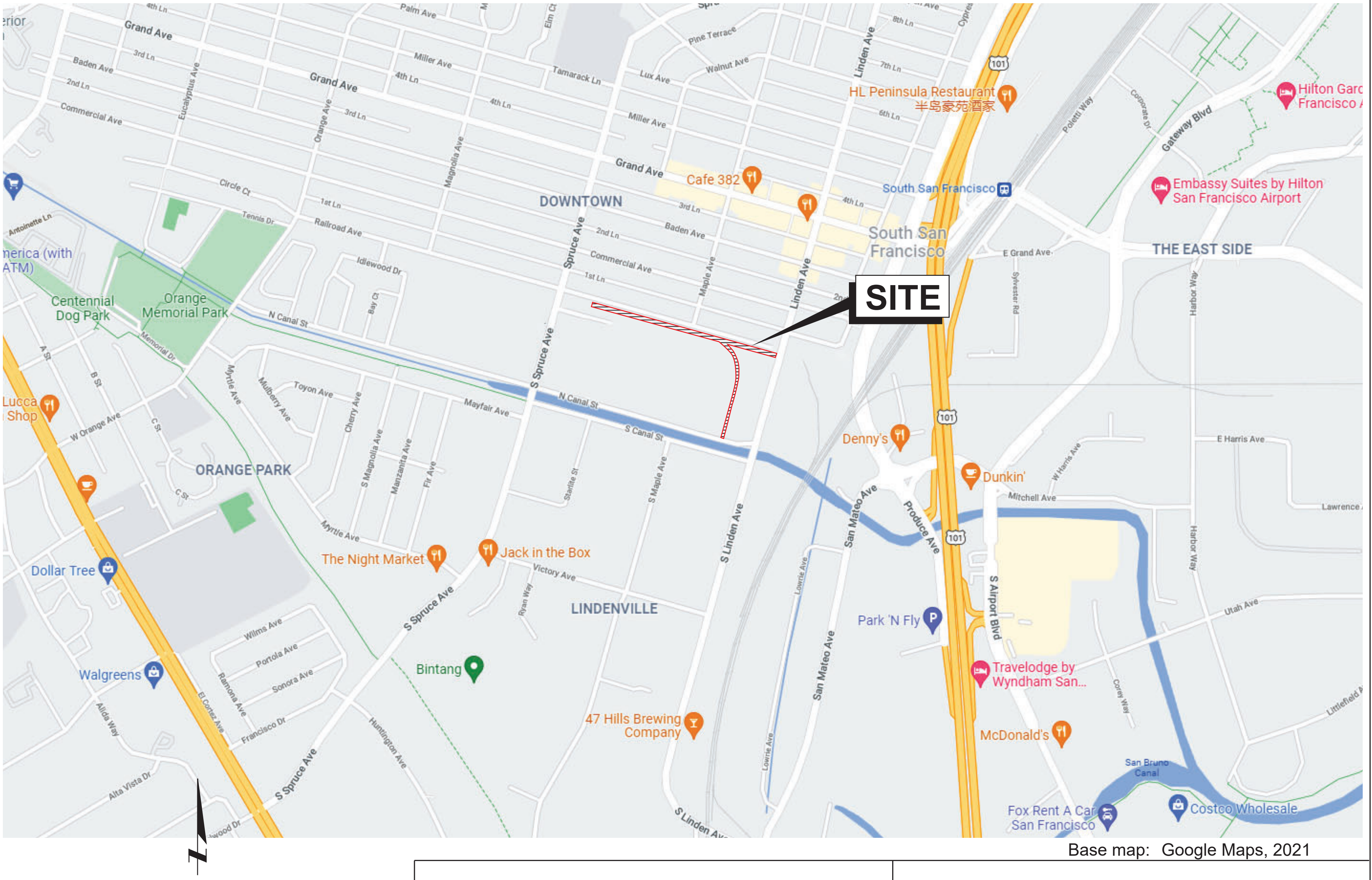
FIGURES



Base map: Google Maps, 2021

0 1,000 2,000 Feet

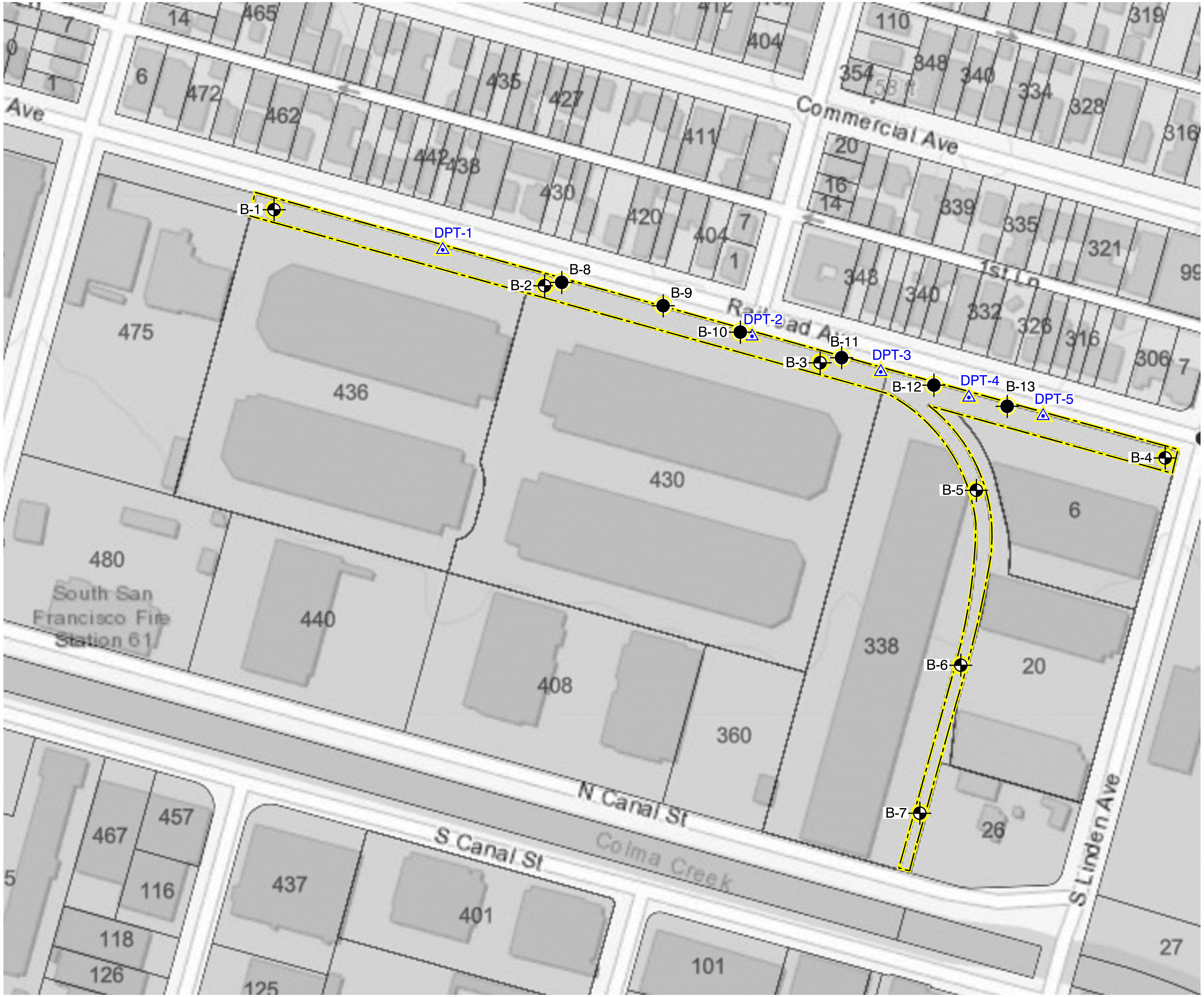
Approximate scale

RAILROAD RESIDENCE DEVELOPMENT
South San Francisco, California

ROCKRIDGE
GEOTECHNICAL

SITE LOCATION MAP

Date 08/30/23 | Project No. 21-2085 | Figure 1



EXPLANATION

B-8 ●	Approximate location of boring by Rockridge Geotechnical, Inc., July 17 and 18, 2023
B-1 ⊕	Approximate location of boring by Rockridge Geotechnical, Inc., September 8 and 24, 2021
DPT-1 △	Approximate location of dynamic penetrometer test by Rockridge Geotechnical, Inc., October 5, 2021
— · —	Project limits

0 160 Feet

Approximate scale

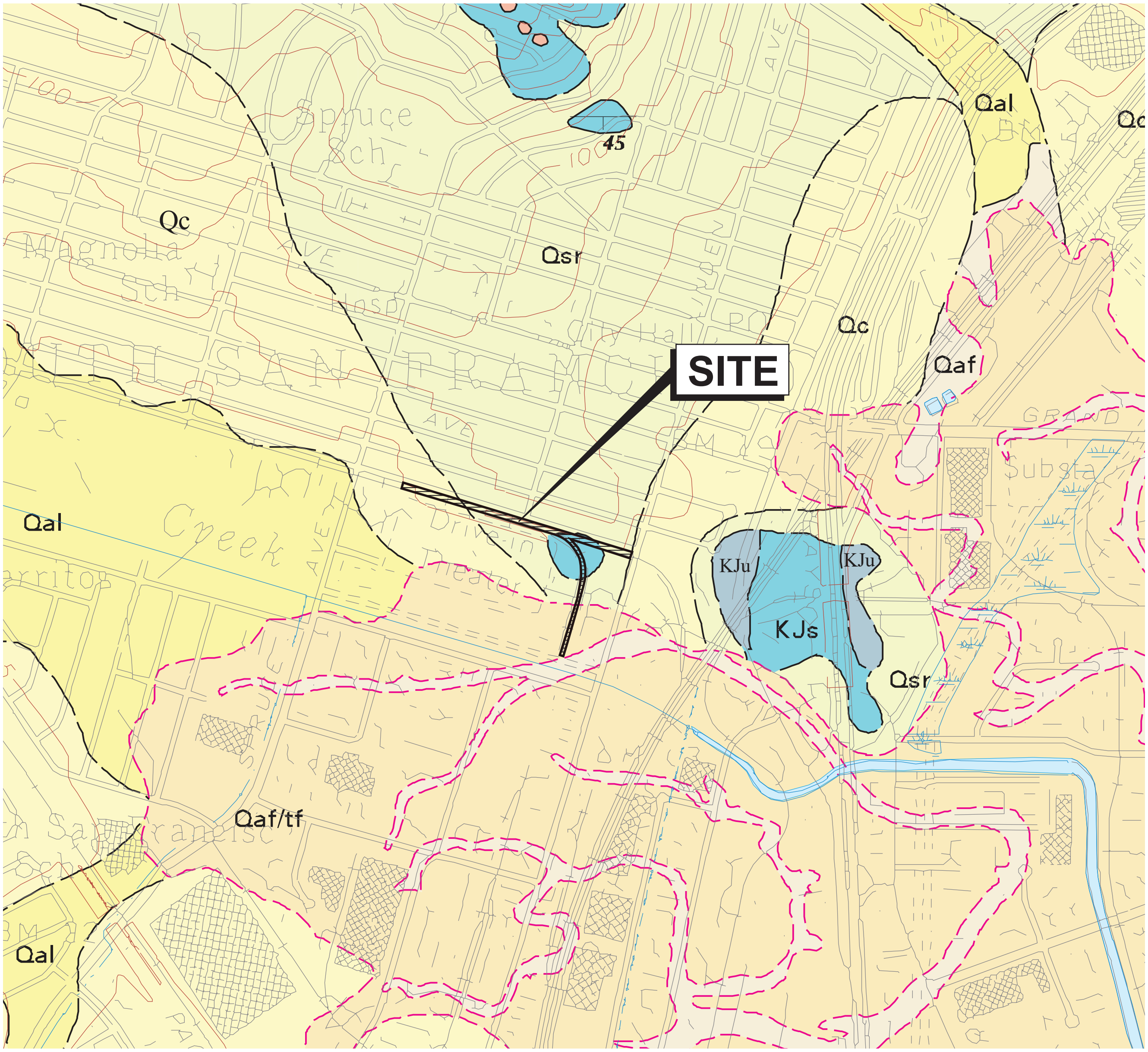
Base map: maps.conservation.ca.gov

RAILROAD RESIDENCE DEVELOPMENT
South San Francisco, California

SITE PLAN

Date 08/30/23	Project No. 21-2085	Figure 2

ROCKRIDGE GEOTECHNICAL



Base map: Preliminary Geologic Map of the SF South 7.5′ Quadrangle and part of the Hunters Points 7.5′ Quadrangle, San Francisco Bay Area, California, 1998

🟧	Qaf/tf	Artificial fill over tidal flat
🟨	Qal	Alluvium
■	Qsr	Slope debris and ravine fill
■	Qc	Colma Formation
🟦	KJs	Sandstone and shale
■	KJu	Sheared rocks
– – –		1800s shoreline and stream channels

N

0 1,000 2,000 Feet

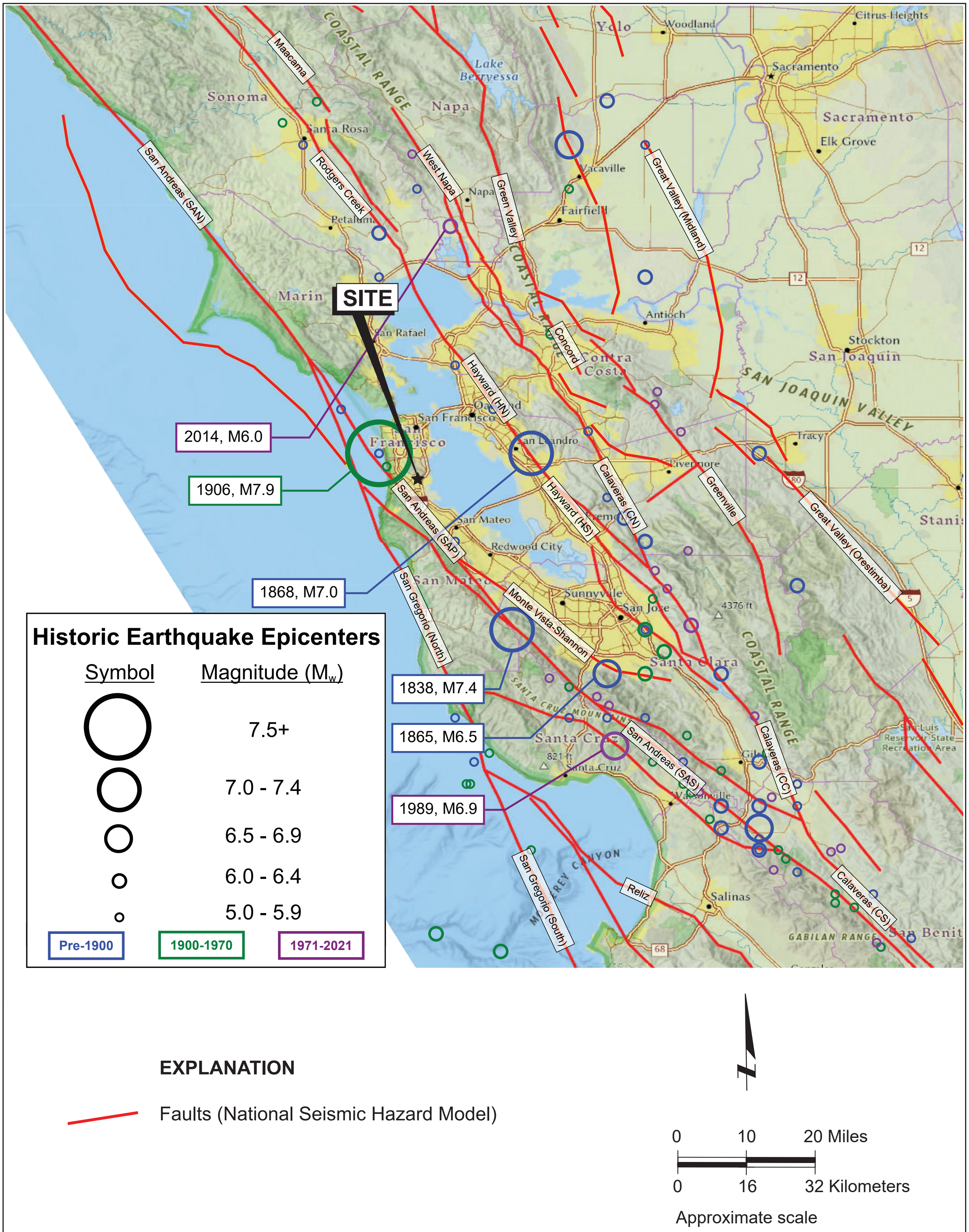
Approximate scale

RAILROAD RESIDENCE DEVELOPMENT
South San Francisco, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 08/30/23	Project No. 21-2085	Figure 3



RAILROAD RESIDENCE DEVELOPMENT
South San Francisco, California

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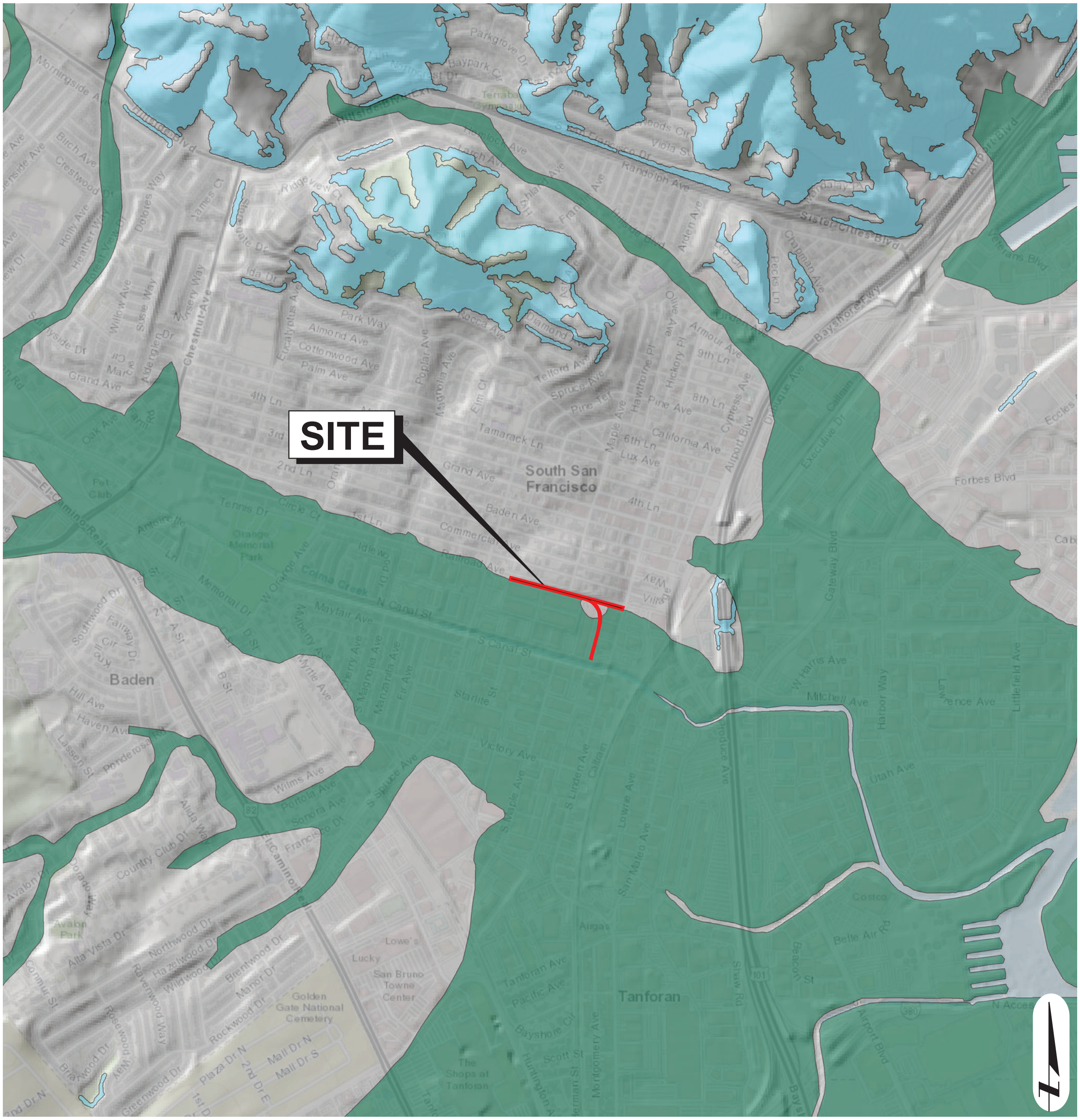
REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Date 08/30/23	Project No. 21-2085	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
San Francisco South Quadrangle
Seismic Hazard Zones
Released September 23, 2021

0 2,000 4,000 Feet
Approximate scale

RAILROAD RESIDENCE DEVELOPMENT
South San Francisco, California

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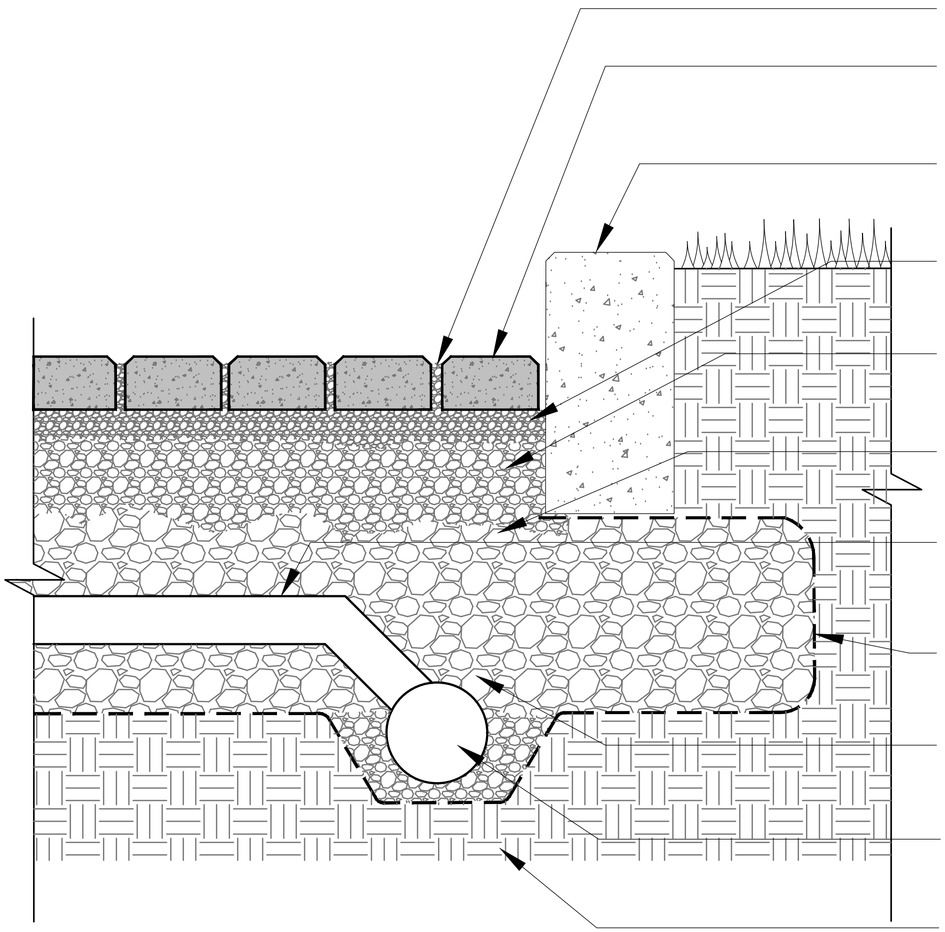
EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 08/30/23 | Project No. 21-2085 | Figure 5



TYP. NO. 8, 89, OR 9 AGGREGATE IN OPENINGS

CONCRETE PAVERS MIN. 3 $\frac{1}{8}$ IN. (80 MM) THICK
FOR VEHICULAR TRAFFIC (ASPECT RATIO $\leq$ 3)

CURB/EDGE RESTRAINT WITH CUT-OUTS
FOR OVERFLOW DRAINAGE (CURB SHOWN)

BEDDING COURSE 1 $\frac{1}{2}$ TO 2 IN. (40 TO 50 MM) THICK
(TYP. NO. 8 AGGREGATE)

4 IN (100 MM) THICK NO. 57
STONE OPEN-GRADED BASE

MIN. 6 IN. (150 MM) THICK
NO. 2 STONE SUBBASE

PERFORATED PIPES SPACED AND
SLOPED TO DRAIN ALL STORED WATER

GEOTEXTILE ON TOP AND SIDES OF
SUBBASE UNDER/BEYOND CURB

GEOTEXTILE ON SUBGRADE

NON-PERFORATED OUTFALL PIPE(S)
SLOPED TO STORM SEWER OR STREAM

SOIL SUBGRADE SLOPED TO DRAIN

NOTES:

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

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

RAILROAD RESIDENCE DEVELOPMENT
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GENERALIZED ICPI PERMEABLE
PAVER DETAIL
FOR PARTIAL EXFILTRATION

Date 09/05/23 | Project No. 21-2085 | Figure 6

APPENDIX A

Boring Logs and Dynamic Penetrometer Test Results

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-1

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 09/08/2021 Date finished: 09/08/2021
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: R. Ford
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

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	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						1 inch of old road asphalt							
1					SM	SILTY SAND with GRAVEL (SM) brown, dense, dry	FILL						
2	SPT	◩	15 16 19	38	SC	CLAYEY SAND (SC) yellow-brown with yellow and dark brown mottling, dense, moist, fine gravel	SLOPE DEBRIS AND RAVINE FILL					10.3	
3													
4	MC	■	13 15 38	33		SANDY SILT (ML) red-yellow with brown mottling, hard, moist, fine sand Soil Corrosivity Test; see Appendix B							
5					ML						57	10.4	
6	SPT	◩	18 18 26	48									
7						CLAYEY SAND (SC) light brown with yellow mottling, dense, moist, fine sand							
8	SPT	◩	14 14 21	38							36		
9							COLMA FORMATION						
10					SC	light brown with gray mottling, very dense							
11	SPT	◩	14 21 30	55									
12													
13													
14						hard drilling							
15	SPT	◩	30 50/6"	54/6"		SILTY SAND (SM) red-yellow, very dense, slightly moist, fine sand with occasional coarse sand size fragments							
16													
17					SM		RESIDUAL SOIL						
18													
19						SANDSTONE yellow-brown to yellow-orange, low hardness, friable, moderately weathered							
20							BEDROCK						
21	SPT	◩	40 44 50/5"	102/11"									
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 21.4 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered 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.

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Project No.: 21-2085
Figure: A-1

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-2

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 09/08/2021
Date finished: 09/08/2021
Drilling method: 8-inch-diameter hollow-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Safety Hammer
Sampler: Standard Penetration Test (SPT)

Logged by: R. Ford
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

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
					SC	CLAYEY SAND with GRAVEL (SC) brown, medium dense, dry	SLOPE DEBRIS AND RAVINE FILL						
1													
2	SPT	◪	12 18 22	43		CLAYEY SAND (SC) yellow-brown with red-brown mottling oxidation, dense, moist, roots LL = 28, PI = 15; see Appendix B olive with gray and yellow mottling, very dense							
3					SC						41	11.3	
4	SPT	◪	15 23 32	59									
5							COLMA FORMATION						
6	SPT	◪	25 30 32	67	SC	CLAYEY SAND (SC) yellow-brown with brown mottling, very dense, moist							
7						SANDY CLAY (CL) brown with red-yellow mottling, hard, moist							
8	SPT	◪	14 16 26	45							62	14.1	
9													
10													
11	SPT	◪	15 25 40	70									
12													
13													
14	SPT	◪	25 30 35	70	CL								
15													
16													
17													
18							RESIDUAL SOIL						
19	SPT	◪	14 23 28	55		SAND (SP) light gray, trace yellow-orange oxidation, very dense, moist, fine sand, cemented weakly							
20					SP								
21													
22													
23						SILTSTONE/SHALE gray-white with red-brown oxidation, very thin bedded, low hardness, friable to weak, deeply to moderately weathered	BEDROCK						
24	SPT	◪	18 45 50/4"	103/ 10"									
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 24.3 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.08 to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085
Figure: A-2

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-3

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 09/08/2021 Date finished: 09/08/2021

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

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

Sampler: Standard Penetration Test (SPT)

Logged by: R. Ford
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

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
					SC	CLAYEY SAND (SC) brown, medium dense, dry	**FILL**						
1													
2	SPT		9 11 11	24		SILTY SAND (SM) yellow, medium dense, dry, fine gravel Soil Corrosivity Test; see Appendix B	**RESIDUAL SOIL**						
3					SM								
4	SPT		11 12 14	28							46	9.5	
5													
6	SPT		18 28 50/3"	84/9"		SANDSTONE yellow with red-yellow mottling, fine grained, low hardness, friable to weak, deeply to moderately weathered	**BEDROCK**						
7													
8													
9	SPT		50/4"	54/4"									
10	SPT		50/2"	54/2"									
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 10.2 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.08 to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085

Figure: A-3

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-4

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 09/08/2021
Date finished: 09/08/2021
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: R. Ford
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

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
1					GW	GRAVEL with SAND and CLAY (GW) brown, medium dense, moist Soil Corrosivity Test; see Appendix B	FILL						
2	MC	■	25 15 16	20	SC	CLAYEY SAND (SC) olive-brown with dark brown and red-yellow mottling, dense, moist, rounded corase sand-size to pea size gravel LL = 28, PI = 15; see Appendix B	SLOPE DEBRIS AND RAVINE FILL						
4	MC	■	14 25 50	47									
6	SPT	◩	11 16 17	36							46	15.5	
7						medium dense							
8	MC	■	12 18 24	26									
9					CL	CLAY with SAND and GRAVEL (CL) yellow-brown with yellow mottling, very stiff to hard, moist	RESIDUAL SOIL						
11	SPT	◩	18 13 24	40									
12						SILTSTONE/SHALE brown to dark gray, very thin bedded, low hardness to plastic friable, pervasively sheared, deeply to completely weathered	BEDROCK						
16	SPT	◩	12 22 35	62		▽ (09/08/2021; 8:20 AM)							
21	SPT	◩	22 32 50/5"	89/ 11"		low hardness to weak with completly weathered zones							

Boring terminated at a depth of 21.4 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 16 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.: 21-2085
Figure: A-4

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-5

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 09/24/2021
Date finished: 09/24/2021
Drilling method: 3-inch-diameter solid stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & cathead safety hammer
Sampler: Standard Penetration Test (SPT)

Logged by: J. Pisenti
Drilled by: Access Soil Drilling
Rig: Limited Access

LABORATORY TEST DATA

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
1		◪	16		CL	SANDY CLAY with GRAVEL (CL) brown, yellow-brown, and olive-gray, hard, moist, fine to coarse sand, fine angular to subangular gravel	**SLOPE DEBRIS AND RAVINE FILL**						
2	SPT		16 15	37									
3		◪	32			SILTY SAND with GRAVEL (SM) pale yellow and gray, very dense, moist, fine to coarse sand and gravel, angular to subangular gravel	**RESIDUAL SOIL**						
4	SPT		35 38	88	SM								
5		◪	19			SANDSTONE pale yellow to light yellow-brown and gray, low hardness, moderately weak to weak, moderately to highly weathered	**BEDROCK**						
6	SPT		33 37	84									
7	SPT	◪	55 50/5"	60/5"									
8													
9													
10													
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 7.9 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.2 to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085
Figure: A-5

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-6

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 09/24/2021 Date finished: 09/24/2021
Drilling method: 3-inch-diameter solid stem auger
Hammer weight/drop: 140 lbs./30 inches Hammer type: Rope & cathead safety hammer
Sampler: Standard Penetration Test (SPT)

Logged by: J. Pisenti
Drilled by: Access Soil Drilling
Rig: Limited Access

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	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1 2	SPT	◩	9 10 14	29	CL	SANDY CLAY with GRAVEL (CL) brown mottled with yellow-brown, very stiff, dry to moist, fine to coarse sand, fine angular to subangular gravel	FILL						
3 4 5	SPT	◩	9 11 13	29	CL	SANDY CLAY (CL) yellow-brown mottled with olive-gray and gray, very stiff, moist, fine to medium sand							
6	SPT	◩	5 10 13	28	SC	CLAYEY SAND with GRAVEL (SC) gray-brown and red-yellow, medium dense, moist, fine to medium sand, fine subangular gravel							
7 8 9 10	SPT	●	3 4 3	8	CL-ML	SILTY CLAY (CL-ML) black, medium stiff to stiff, moist to wet, trace fine sand	BAY MUD						
11	SPT	◩	5 3 4	8	CL	CLAY (CL) olive-gray and black, medium stiff to stiff, moist to wet							
12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30													

Boring terminated at a depth of 11.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] SPT blow counts for the last two increments were converted to SPT N-Values using a factor of 1.2 to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085
Figure: A-6

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-7

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 09/24/2021
Date finished: 09/24/2021
Drilling method: 3-inch-diameter solid stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & cathead safety hammer
Sampler: Modified California (MC), Standard Penetration Test (SPT)

Logged by: J. Pisenti
Drilled by: Access Soil Drilling
Rig: Limited Access

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	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
1–2	MC	▨	38 31 36	47	SC	CLAYEY SAND with GRAVEL (SC) brown to ligh brown with light gray, dense, dry to moist, fine to coarse sand and gravel, angular to subangular gravel	FILL						
3–4	SPT	▨	17 13 12	30	GW-GM	GRAVEL with SILT and SAND (GW-GM) gray and yellow brown, medium dense to dense, dry to moist, fine to coarse sand and gravel, angular to subangular gravel							
5–6	SPT	▨	6 5 7	14	CL	SANDY CLAY with GRAVEL (CL) yellow brown and olive-gray mottled with gray, stiff, moist, fine to coarse sand, fine angular gravel							
7–8	SPT	▨	3 3 3	7	CL-ML	SILTY CLAY (CL-ML) dark gray to black, medium stiff, moist, trace fine sand	BAY MUD						
10–11	SPT	▨	2 2 2	5		medium stiff, moist to wet ▽ (09/24/2021; 9:50 AM)							
12–13	SPT	▨	2 2 3	6	CL	CLAY (CL) dark gray, medium stiff, wet							

Boring terminated at a depth of 13 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 11.5 feet during drilling.

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

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085
Figure: A-7

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-8

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 07/17/2023
Date finished: 07/17/2023
Drilling method: 4-inch-diameter solid-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead Safety Hammer
Sampler: Modified California (MC), California (CA), Standard Penetration Test (SPT)

Logged by: J. Pisenti
Drilled by: Access Soil Drilling
Rig: Minuteman

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
1 2	MC		7 13 17 29	32	SC SC	CLAYEY SAND with GRAVEL (SC) brown, dense, moist, fine to coarse sand, fine subrounded gravel, roots CLAYEY SAND (SC) yellow-brown mottled with red-brown, dense, moist, fine to medium sand, roots	SLOPE DEBRIS AND RAVINE FILL						
3 4	MC		21 27 24 21	32	SP-SC	SAND with CLAY (SP-SC) red-yellow, dense, moist, fine sand							
5 6	MC		26 36 50/6" 58/6"	76	CL SP-SC	SANDY CLAY (CL) olive-brown, hard, moist SAND with CLAY (SP-SC) red-yellow, very dense, moist, fine sand Soil Corrosivity Test; see Appendix B	COLMA FORMATION					9.7	
7 8	CA		30 35 52 51	93		yellow-brown yellow, trace fine subangular gravel							
9 10	SPT		23 30 40 69	131	SP-SC	yellow-brown							
11 12													
13 14	SPT		30 72 50/3"	86/9"									
15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30													

Boring terminated at a depth of 14.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] MC, CA, and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7, 0.9, and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085
Figure: A-8

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-9

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 07/17/2023
Date finished: 07/17/2023
Drilling method: 4-inch-diameter solid-stem auger
Logged by: J. Pisenti
Drilled by: Access Soil Drilling
Rig: Minuteman
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead Safety Hammer
Sampler: Modified California (MC), California (CA), Standard Penetration Test (SPT)

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
1	MC	■	7 9 9	24	SC	CLAYEY SAND (SC) yellow-brown, medium dense, moist, fine to medium sand, trace fine subrounded gravel, roots	SLOPE DEBRIS AND RAVINE FILL						
2			25 30		CL	SANDY CLAY (CL) red-yellow, very stiff to hard, moist, fine sand, trace fine subangular gravel Particle Size Distribution; see Appendix B					71		
3	MC	■	28 43 52	67	SP-SC	SAND with CLAY (SP-SC) yellow, very dense, moist, fine sand red-yellow							
4			22 32										
5	CA	■	58 50/3"	97/9"	CL	SANDY CLAY (CL) yellow-brown, hard, moist, fine sand	RESIDUAL SOIL						
6	SPT	◩	50 50/3"	60/3"	SM	SILTY SAND (SM) yellow-brown, very dense, moist fine to medium sand							
7													
8					SP-SC	SAND with CLAY (SP-SC) red-yellow, very dense, moist, fine sand							
9			18										
10	SPT	◩	30 44 62	127									
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 11 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] MC, CA, and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7, 0.9, and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085
Figure: A-9

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-10

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 07/17/2023 Date finished: 07/17/2023

Drilling method: 4-inch-diameter solid-stem auger

Logged by: J. Pisenti
Drilled by: Access Soil Drilling
Rig: Minuteman

Hammer weight/drop: 140 lbs./30 inches Hammer type: Rope & Cathead Safety Hammer

Sampler: Modified California (MC), California (CA), Standard Penetration Test (SPT)

	SAMPLES							LABORATORY TEST DATA					
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/Cut Ft
1	MC		4 5 5 10	11	CL	SANDY CLAY (CL) olive-brown, stiff, moist, fine to coarse sand, trace fine subangular gravel Particle Size Distribution; see Appendix B	SLOPE DEBRIS AND RAVINE FILL				79	10.5	
2					SC	CLAYEY SAND (SC) red-yellow, medium dense, fine sand	RESIDUAL SOIL						
3	MC		14 34 50/5"	59/ 11"	SP-SC	SAND with CLAY (SP-SC) yellow-brown, very dense, moist, fine sand, trace fine subangular gravel, rootlets							
4	CA		47 50/4"	45/4"									
5	SPT		50 50/3"	60/3"		red-yellow							
6													
7													
8													
9													
10													
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
24													
25													
26													
27													
28													
29													
30													

Boring terminated at a depth of 5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

[1] MC, CA, and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7, 0.9, and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085

Figure: A-10

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-11

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 07/18/2023
Date finished: 07/18/2023
Drilling method: 4-inch-diameter solid-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead Safety Hammer
Sampler: Modified California (MC), Standard Penetration Test (SPT)
Logged by: J. Pisenti
Drilled by: Access Soil Drilling
Rig: Minuteman

	SAMPLES							LABORATORY TEST DATA					
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/Cut Ft
1	MC	■	2 5 30 50/3"	56/9"	SC	CLAYEY SAND with GRAVEL (SC) brown, very dense, moist, fine to coarse sand, fine subangular gravel, rootlets	**RESIDUAL SOIL**						
2	SPT	◪	104/ 6"	125/ 6"		SANDSTONE yellow-brown, closely to intensely fractured, low hardness to moderately hard, weak to moderately strong, moderately weathered	**BEDROCK**						
3													
4													
5													
6													
7													
8													
9													
10													
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
22													
23													
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26													
27													
28													
29													
30													

Boring terminated at a depth of 2.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

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

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085

Figure: A-11

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-12

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 07/18/2023
Date finished: 07/18/2023
Drilling method: 4-inch-diameter solid-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead Safety Hammer
Sampler: Modified California (MC), California (CA), Standard Penetration Test (SPT)
Logged by: J. Pisenti
Drilled by: Access Soil Drilling
Rig: Minuteman

DEPTH (feet)	SAMPLES: Sampler Type	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/Cut Ft
1	MC	12 20 28 22	35	SC SM	CLAYEY SAND with GRAVEL (SC) olive-gray, dense, moist, fine to coarse sand, fine to coarse subangular gravel SILTY SAND with GRAVEL (SM) gray, dense, moist, fine to coarse sand, fine to coarse angular gravel	**SLOPE DEBRIS AND RAVINE FILL** **RESIDUAL SOIL**						
3	MC	16 27 35 43	55									
4					trace clay Particle Size Distribution; see Appendix B					24	5.6	
5	CA	17 17 21 20	37									
7	CA	38 55 72 138	189		SILTSTONE/SHALE gray, intensely fractured, low hardness, friable, deeply to moderately weathered	**BEDROCK**						
9	SPT	47 61 49 49	118									
10					gray and light gray							
11	SPT	30 36 35 46	77									
12												

Boring terminated at a depth of 12 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

---

[1] MC, CA, and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7, 0.9, and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085
Figure: A-12

PROJECT: **RAILROAD RESIDENCE DEVELOPMENT**
South San Francisco, California

Log of Boring B-13

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 07/18/2023
Date finished: 07/18/2023
Drilling method: 4-inch-diameter solid-stem auger
Logged by: J. Pisenti
Drilled by: Access Soil Drilling
Rig: Minuteman
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead Safety Hammer
Sampler: Modified California (MC), California (CA), Standard Penetration Test (SPT)

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
1	MC	■	1 7 9 11	14	SC	CLAYEY SAND with GRAVEL (SC) brown, medium dense, moist, fine to coarse sand, fine to coarse angular gravel	**SLOPE DEBRIS AND RAVINE FILL**				7		
2						GRAVEL with SAND and CLAY (GW-GC) brown, medium dense, moist, fine to coarse sand, fine to coarse angular gravel Particle Size Distribution; see Appendix B olive-brown, medium dense to dense, trace silt							
3	MC	■	20 21 20 23	30	GW-GC								
4													
5	MC	■	29 23 43 36	55		CLAYEY GRAVEL with SAND (GC) olive-brown mottled with yellow, very dense, moist, fine to coarse sand, fine to coarse angular gravel, rootlets	**RESIDUAL SOIL**						
6					GC								
7	CA	■	22 34 48 50/3"	88/9"									
8	SPT	●	100/ 6"	120/ 6"		SILTSTONE/SHALE gray, closely to intensely fractured, low hardness to moderately hard, weak to moderately strong, moderately weathered	**BEDROCK**						
9													
10													
11													
12													
13													
14													
15													
16													
17													
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29													
30													

Boring terminated at a depth of 8.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

---

[1] MC, CA, and SPT blow counts for the last two increments were converted to SPT N-Values using factors of 0.7, 0.9, and 1.2, respectively, to account for sampler type and hammer energy.

ROCKRIDGE GEOTECHNICAL

Project No.: 21-2085
Figure: A-13

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

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

◩ Classification sample taken with Standard Penetration Test sampler

◨ Undisturbed sample taken with thin-walled tube

☒ Disturbed sample

⊙ Sampling attempted with no recovery

◫ Core sample

◙ Analytical laboratory sample

▥ Sample taken with Direct Push sampler

▥ Sonic

SAMPLER TYPE

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

RAILROAD RESIDENCE DEVELOPMENT
South San Francisco, California

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CLASSIFICATION CHART

Date 08/30/23 | Project No. 21-2085 | Figure A-14

I FRACTURING

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

II HARDNESS

1. **Soft** - reserved for plastic material alone.
2. **Low hardness** - can be gouged deeply or carved easily with a knife blade.
3. **Moderately hard** - can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** - can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
5. **Very hard** - cannot be scratched with knife blade; leaves a metallic streak.

III STRENGTH

1. **Plastic** or very low strength.
2. **Friable** - crumbles easily by rubbing with fingers.
3. **Weak** - an unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** - specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** - specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** - specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

IV **WEATHERING -** The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

- **D. Deep** - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
- **M. Moderate** - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
- **L. Little** - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
- **F. Fresh** - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

V **CONSOLIDATION OF SEDIMENTARY ROCKS:** usually determined from unweathered samples. Largely dependent on cementation.

U = unconsolidated
P = poorly consolidated
M = moderately consolidated
W = well consolidated

VI BEDDING OF SEDIMENTARY ROCKS

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick-bedded
Blocky	2.0 to 4.0 ft.	thick bedded
Slabby	0.2 to 2.0 ft.	thin bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01	thinly laminated

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South San Francisco, California

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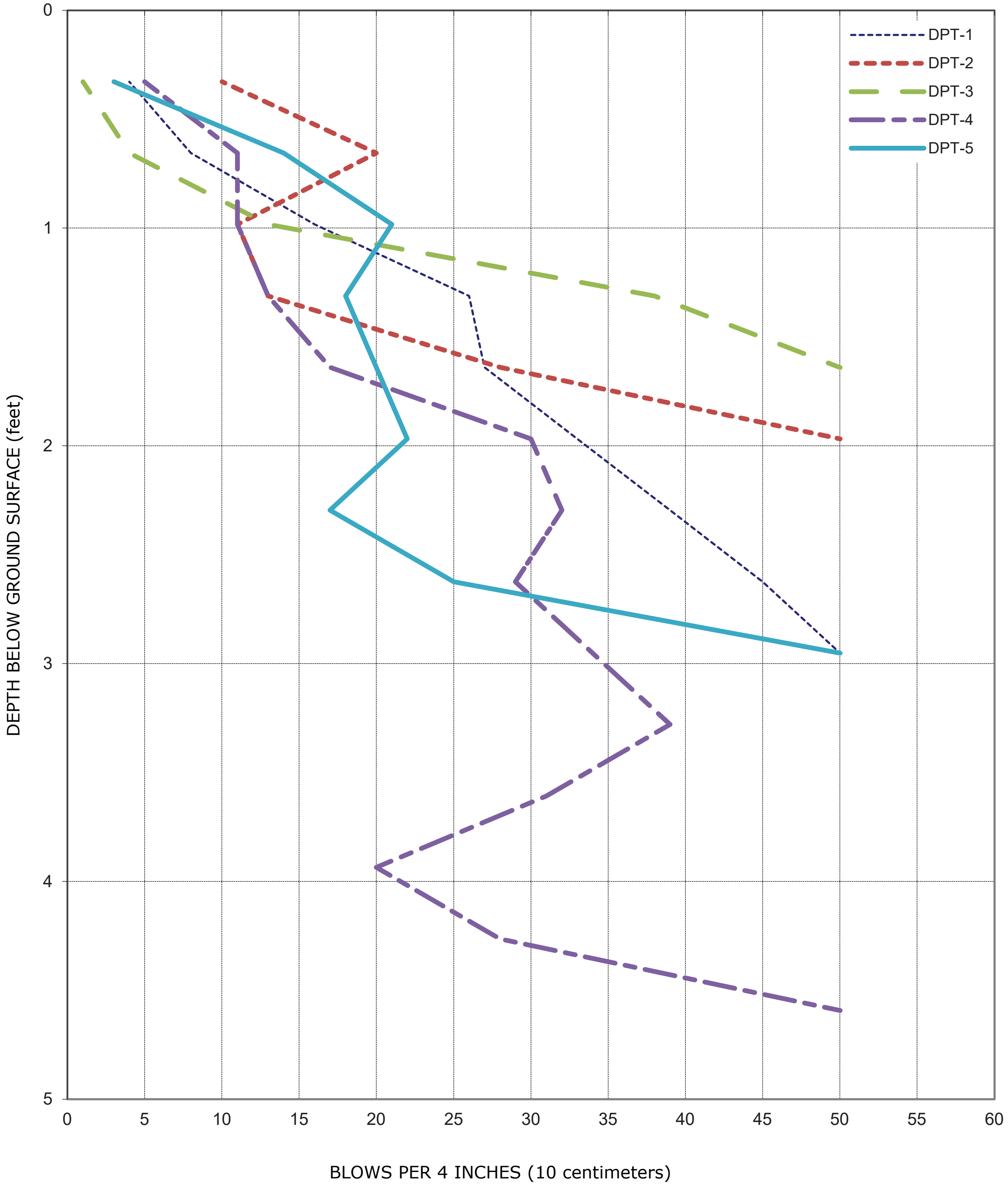
PHYSICAL PROPERTIES CRITERIA
FOR ROCK DESCRIPTIONS

Date 08/24/23 | Project No. 21-2085 | Figure A-15



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South San Francisco, California

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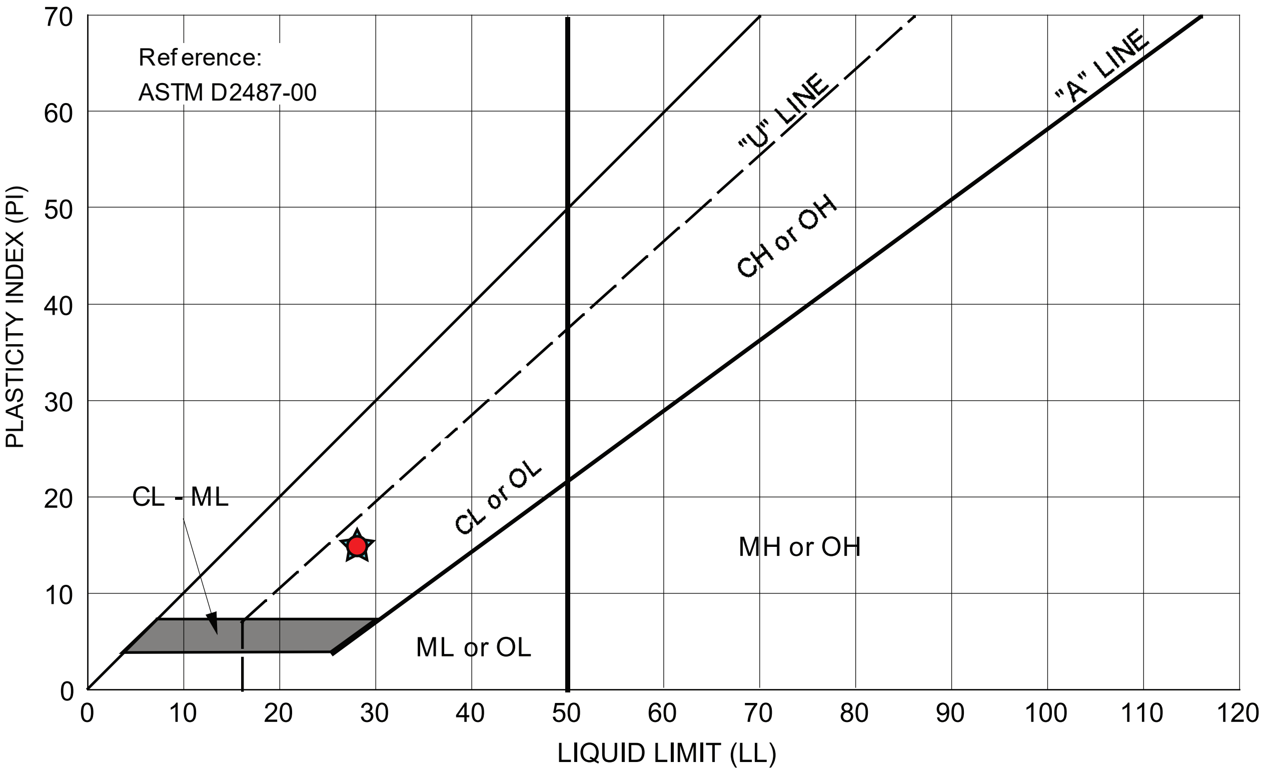
DYNAMIC PENETROMETER TEST RESULTS

Date 08/24/23	Project No. 21-2085	Figure A-16

APPENDIX B

Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-2 at 1-2.5 feet	CLAYEY SAND (SC) yellow-brown with red-brown mottling	11.3	28	15	--
★	B-4 at 2.0 feet	CLAYEY SAND (SC) olive-brown with dark brown and red-yellow mottling	--	28	15	--

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South San Francisco, California

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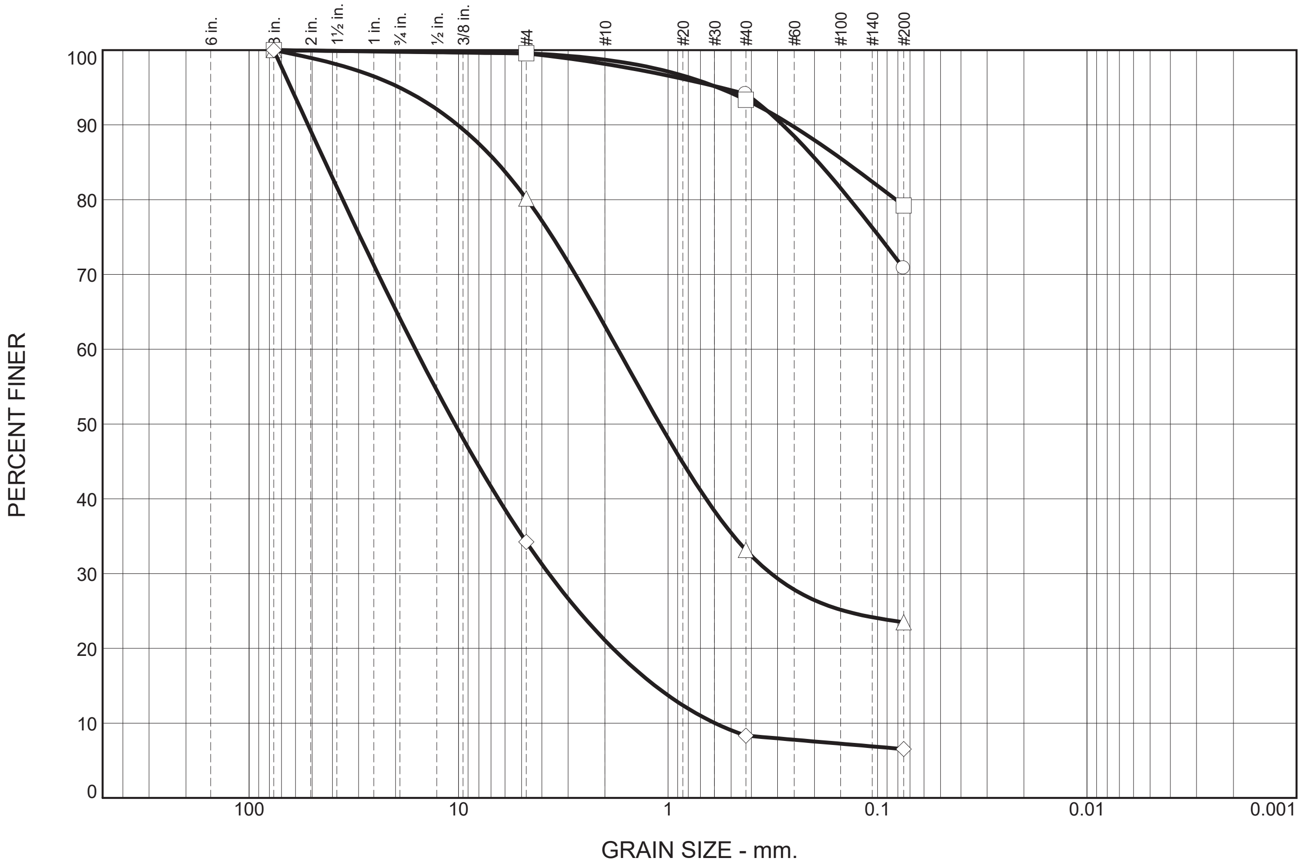
PLASTICITY CHART

Date 08/30/23 | Project No. 21-2085 | Figure B-1



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.2	0.2	1.5	4.0	23.3	70.8	
□	0.0	0.2	0.2	0.9	5.3	14.2	79.2	
△	0.0	5.0	14.8	17.1	29.9	9.7	23.5	
◇	0.0	35.9	29.9	13.1	12.8	1.8	6.5	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-9	2.3'	SANDY CLAY, red-yellow	CL
□	B-10	0.8'	SANDY CLAY, olive-brown	CL
△	B-12	4.0'	SILTY SAND with GRAVEL, gray	SM
◇	B-13	1.0'	GRAVEL with SAND and CLAY, brown	GW-GC

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PARTICLE SIZE DISTRIBUTION REPORT

Date 08/28/23 | Project No. 21-2085 | Figure B-2

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

REPORT S210924H

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	ASTM D4658	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 SILT (ML) red-yellow with brown mottling	4	96.6	0.0097	23.0	0.0023	2,881	1,943	8.4	106	<0.01	0.2	2.7	0.05	66.7	0.9	76.1	181.3	3.5	3.4
B-3: SANDY SILT (SM) yellow	1-2.5	51.5	0.0051	24.2	0.0024	18,090	5,025	7.8	95	<0.01	0.3	4.8	0.04	46.8	1.4	59.6	153.6	3.2	3.0
B-4: GRAVEL with SAND and CLAY (GW), brown	1.5	32.1	0.0032	11.7	0.0012	2,010	2,010	8.2	110	<0.01	0.9	2.2	0.03	54.1	2.0	96.5	208.9	4.3	2.2

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)

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www.projectxcorrosion.com

RAILROAD RESIDENCE DEVELOPMENT
South San Francisco, California

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SOIL CORROSIVITY
TEST RESULTS

Date 08/28/23
Project No. 21-2085
Figure B-3a

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

REPORT S230728A

	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-8 Sample #4	6.0	61.9	0.0062	100.3	0.0100	20,100	1,876	7.8	141	10.8	0.1	11.5	ND	145.5	7.4	94.6	187.7	6.4	98.1

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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South San Francisco, California

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SOIL CORROSIVITY
TEST RESULTS

Date 08/28/23 | Project No. 21-2085 | Figure B-3b