ROCKRIDGE
GEOTECHNICAL

Prepared for **DTSM Housing, LLC**

FINAL GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
401-445 S. B STREET
SAN MATEO, CALIFORNIA

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April 19, 2023
Project No. 20-1869

ROCKRIDGE
GEOTECHNICAL

April 19, 2023
Project No. 20-1869

DTSM Housing, LLC

c/o Mr. Scotty Nowak | Project Manager
Harvest Properties, Inc.
180 Grand Avenue, Suite 1400
Oakland, California 94612

Subject: Final Geotechnical Investigation Report
Proposed Mixed-Use Development
401-445 S. B Street
San Mateo, California

Dear Mr. Nowak:

We are pleased to present the results of our final geotechnical investigation for the proposed mixed-use development to be constructed at 401 through 445 S. B Street and the 4th and Railroad Lot (existing public parking lot) in San Mateo, California. Our services were provided in accordance with our proposal dated October 26, 2021.

The project site consists of six contiguous parcels encompassing a total of about 1.2 acres on the block bordered by S. B Street to the southwest, E. 4th Avenue to the northwest, S. Railroad Avenue to the northeast, and E. 5th Avenue to the southeast. The combined parcels are rectangular shaped and have maximum plan dimensions of about 220 by 230 feet and are currently occupied by one- to two-story commercial buildings, asphalt parking areas, and landscaped areas. We previously performed a preliminary geotechnical paper study for the site, the results of which were presented in our report dated July 29, 2021.

Current development plans[1] include demolishing the existing buildings and parking lot at the site and constructing a seven-story above grade affordable housing building at the northern corner of the site and a five-story office building in the remainder of the site. The affordable housing building will be rectangular-shaped with maximum plan dimensions of approximately 110 feet by 120 feet and will consist of five levels of wood-framed construction over a two-level concrete podium. The office building will be L-shaped with maximum plan dimensions of approximately 110 feet by 220 feet and will consist of mass timber construction. Both buildings will be constructed over a single

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[1] *Planning Application Submission for Bespoke, 401-445 S B Street, San Mateo, CA 94401 and Planning Application Submission for 4th & Railroad, 307 E 4th Avenue, San Mateo, CA 94401 by RMW Architecture Interiors, dated February 1, 2023*

below-grade parking level with a finished floor at a depth of about 15 feet below existing grades (bgs).

Based on the results of our field investigation, laboratory testing, and engineering analyses, we conclude there are no major geotechnical or geological issues that would preclude development of the site as planned. The primary geotechnical concerns affecting the proposed development include: 1) relatively shallow groundwater relative to the proposed building foundation levels and excavation depth, and 2) providing suitable lateral support and dewatering for the proposed excavation, while minimizing impacts to the surrounding improvements, including the nearby Caltrain tracks.

Provided the estimated total and differential settlements presented in our report are acceptable, we conclude the proposed building may be supported on a stiffened mat foundation that is underlain by waterproofing and designed to resist hydrostatic uplift pressures.

Our report contains specific recommendations regarding earthwork and grading, foundation design, excavation shoring, dewatering, and other geotechnical issues. The recommendations contained in our report are based on limited subsurface exploration. Consequently, variations between expected and actual soil conditions may be found in localized areas during construction. Therefore, we should be engaged to observe foundation and shoring installation, as well as grading and fill placement, during which time we may make changes in our recommendations, if deemed necessary.

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

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC.

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EXP. 12/31/24

2957
12/31/23

Timothy J. Forrest, P.E.
Project Engineer

Logan D. Medeiros, P.E., G.E.
Principal Engineer

Enclosure

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FINAL GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
401-445 S. B STREET
San Mateo, California

1.0 INTRODUCTION

This report presents the results of the final geotechnical investigation performed by Rockridge Geotechnical, Inc. for the proposed mixed-use development to be constructed at 401 through 445 S. B Street and the 4th and Railroad Lot (existing public parking lot) in San Mateo, California. The project site consists of six contiguous parcels encompassing a total of about 1.2 acres on the block bordered by S. B Street to the southwest, E. 4th Avenue to the northwest, S. Railroad Avenue to the northeast, and E. 5th Avenue to the southeast, as shown on the Site Location Map, Figure 1. The combined parcels are rectangular shaped and have maximum plan dimensions of about 220 by 230 feet as shown on the Site Plan, Figure 2. The parcels are currently occupied by one- to two-story commercial buildings, asphalt parking areas, and landscaped areas. We previously performed a preliminary geotechnical paper study for the site, the results of which were presented in our report dated July 29, 2021.

Current development plans[1] include demolishing the existing buildings and parking lot at the site and constructing a seven-story above grade affordable housing building at the northern corner of the site and a five-story office building in the remainder of the site. The affordable housing building will be rectangular-shaped with maximum plan dimensions of approximately 110 feet by 120 feet and will consist of five levels of wood-framed construction over a two-level concrete podium. The office building will be L-shaped with maximum plan dimensions of approximately 110 feet by 220 feet and will consist of mass timber construction. Both buildings will be constructed over a single below-grade parking level with a finished floor at a depth of about 15 feet below existing grades (bgs). Based on our discussion with DCI, the project structural engineer for the office building, we understand the proposed office building will have estimated

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[1] *Planning Application Submission for Bespoke, 401-445 S B Street, San Mateo, CA 94401 and Planning Application Submission for 4th & Railroad, 307 E 4th Avenue, San Mateo, CA 94401 by RMW Architecture Interiors, dated February 1, 2023*

column loads of about 1,000 kips (dead-plus-live condition). The structural loads for the affordable housing building were not available during preparation of this report.

2.0 SCOPE OF SERVICES

We previously performed a preliminary geotechnical “paper” study for the 407-445 S. B Street portion of the site during a due diligence evaluation period, which consisted of reviewing existing subsurface data for the site vicinity, geologic maps, earthquake hazard maps, and regional historic groundwater data and performing limited engineering analyses to develop preliminary conclusions and recommendations regarding the geotechnical aspects of the project. The results of our preliminary study were presented in our letter report dated July 29, 2021.

Our final geotechnical investigation for the site was performed in accordance with our proposal dated October 26, 2021. Our scope of work consisted of evaluating subsurface conditions at the site by drilling two exploratory borings, performing laboratory testing on select soil samples, and performing engineering analyses to develop final conclusions and recommendations regarding:

- the most appropriate foundation type(s) for the proposed building
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of static and seismically-induced foundation settlement
- design ground water level
- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- lateral earth pressures for design of permanent below-grade walls
- temporary cut slopes and excavation shoring
- excavation dewatering
- site grading and excavation, including criteria for fill quality and compaction
- subgrade preparation for floor slabs, pavements, and exterior concrete flatwork
- soil corrosivity
- 2022 California Building Code (CBC) site class and mapped design spectral response acceleration parameters
- construction considerations.

3.0 FIELD INVESTIGATIONS AND LABORATORY TESTING

We explored the subsurface conditions at the site by drilling two exploratory borings, designated B-1 and B-2, at the approximate locations shown on the attached Site Plan, Figure 2. Prior to drilling borings, we filed drilling notification forms with San Mateo County Environmental Health (SMCEH) and contacted Underground Service Alert (USA) to notify them of our work, as required by law. We also retained C. Cruz Sub-Surface Locators, a private utility locator, to check that the boring locations were clear of underground utilities. Details of our field exploration are described in the remainder of this section.

3.1 Geotechnical Borings

Two test borings, designated as B-1 and B-2, were drilled on January 17, 2022 by Exploration GeoServices of San Jose, California at the approximate locations shown on Figure 2. The borings were each drilled to a depth of about 50 feet bgs using a Mobile B-61 drill rig equipped with eight-inch-outside-diameter hollow-stem augers. During drilling, our field engineer logged the soil encountered and obtained representative samples for visual classification and laboratory testing. Boring logs were developed based on laboratory test data and the conditions recorded on the field logs and are presented on Figures A-1 through A-2 in Appendix A. The soil encountered in the borings was classified in accordance with the classification chart shown on Figure A-3.

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 brass or stainless steel tubes.
- 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 MC and SPT samplers were driven with a 140-pound, down-hole safety hammer falling about 30 inches per drop. The samplers were driven up to 18 inches and the hammer blows required to drive the samplers were recorded every six inches and are presented on the boring logs. A "blow count" is defined as the number of hammer blows per six inches of penetration or

50 blows for six inches or less of penetration. The blow counts required to drive the MC and SPT samplers were converted to approximate SPT N-values using factors of 0.63 and 1.08, respectively, to account for sampler type, approximate hammer energy (previously measured by drilling subcontractor), and the fact that the SPT sampler was designed to accommodate liners, but liners were not used. The blow counts used for this conversion were: (1) the last two blow counts if the sampler was driven more than 12 inches, (2) the last one blow count if the sampler was driven more than six inches but less than 12 inches, and (3) the only blow count if the sampler was driven six inches or less. The converted SPT N-values are presented on the boring logs.

The groundwater level prior to backfilling with cement grout was measured and recorded on the boring logs.

Upon completion of drilling, the boreholes were backfilled with cement grout in accordance with SMCEH requirements and the pavement was patched with quick-set concrete. Upon completion of drilling, the soil cuttings from the borings were placed in 55-gallon drums and temporarily stored on site. Laboratory analytical testing was performed on representative samples of the drum contents. The test results indicated the material was non-hazardous and the drums were removed from the site and disposed of at a landfill.

3.2 Laboratory Testing

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

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

4.0 SITE AND SUBSURFACE CONDITIONS

The conditions at the site are described in the following sections based on the results of our subsurface investigation, as well as, review of published geologic data and subsurface information collected for other projects in the vicinity. Site-specific descriptions of surface, subsurface, and groundwater conditions are provided in this section.

4.1 Site Conditions

The subject site is made up of six existing parcels, totaling about 1.16 acres and occupies the entire block. The site is bounded by S. B Street to the southwest, E. 4th Avenue to the northwest, S. Railroad Avenue and Peninsula Corridor Joint Powers Board (Caltrain) tracks to the northeast, and E. 5th Avenue to the southeast. The north to northeast portion of the project is currently occupied by asphalt parking and landscaped areas. The south to southwest portion of the site is currently occupied by one- to two-story commercial buildings. The structures cover about 64 percent of the parcels, with asphalt parking and landscaping areas accounting for the remaining area. According to the topographic survey developed by Sherwood Design Engineers and dated February 1, 2023, the site is relatively flat and slopes down to the northeast from about Elevation 100 to 101 feet[2] along South B Street to 98 feet at the intersection of East 4th Avenue and Railroad Avenue and 97 feet at the intersection of East 5th Avenue and Railroad Avenue.

4.2 Subsurface Conditions

As presented on the Regional Geologic Map (Figure 3), the site is mapped as being underlain by Holocene-age alluvial deposits (Qha) but is adjacent to a narrow band of artificial fill (af) at the Caltrain tracks running along the northeastern edge of the site.

The results of our borings indicate the alluvium primarily consists of stiff to hard clay with variable sand and gravel content with interbedded layers of medium dense to very dense sand and gravel with variable clay content to the maximum depth explored of about 50 feet bgs. The results of previous investigations in the site vicinity indicate the alluvium extends to maximum

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[2] Elevation datum not referenced on topographic survey drawing.

depth explored of about 80 feet bgs. A boring drilled by Cornerstone immediately southwest of the site, designated EB-1 on Figure 2, encountered 5 feet of clayey sand and sandy clay fill over alluvium. The granular layers encountered at this site varied in thickness from about 2 to 11 feet.

The results of Atterberg limits tests performed on near-surface soil samples obtained from the borings indicate the near-surface soil consists of clay that has low expansion potential[3].

4.3 Groundwater

Groundwater level measurements were taken while drilling borings. In borings, free groundwater was recorded when first encountered, as well as after withdrawing the augers upon completion. Stabilized groundwater was measured at about 19 feet bgs in our borings, which corresponds to approximately Elevation 78 and 80 feet[4].

To estimate the highest potential groundwater level that may occur at the site in the future, we reviewed information on the State of California Water Resources Control Board GeoTracker website (http://geotracker.swrcb.ca.gov). We reviewed records for a groundwater monitoring well, designated MW-2, located in the northeast corner of 400 East 4th Avenue, as well as for a well designated MW-1, located at 405 East 4th Avenue. Readings taken at these two monitoring wells between May 2010 and March 2012, as well as in MW-1 from January 2000 to October 2003 showed the groundwater levels fluctuated by about 5 feet over the monitoring period with the shallowest groundwater measured at MW-2 at a depth of 15 feet bgs in March 2011. The monitoring wells were located approximately 350 feet north of the subject site.

We also reviewed records for five monitoring wells (designated MW-1 through MW-5), located at 402 South Delaware, roughly 800 feet from the subject site, which documented groundwater readings from April 1994 to March 2005. The monitoring well data indicated groundwater fluctuations in excess of 12 feet with the shallowest reading in 1998 at approximately 6 feet bgs.

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[3] Highly expansive soil undergoes large volume changes with changes in moisture content.
[4] Elevation datum currently unknown.

Lastly, we reviewed records for eleven monitoring wells (designated MW-1 through MW-11), located at 2 East 3rd Avenue, roughly 1000 feet southwest from the subject site, which documented groundwater readings from May 1988 to August 2004. The monitoring well data indicated groundwater fluctuations up to 21 feet with the shallowest reading at MW-7 in 1998 at approximately 16-1/2 feet bgs.

In addition, according to the California Geologic Survey (CGS) report *Seismic Hazard Zone Report for the San Mateo 7.5-Minute Quadrangle, San Mateo County, California*, the historic high groundwater in the site vicinity is approximately 11 feet bgs.

Based on the available historic groundwater information for the site vicinity and our measurements on site, we conclude a high groundwater level of about 11 feet bgs (Elevation 88 feet, based on an average existing ground surface level of about Elevation 99 feet) should be used for design. Based on observed groundwater levels during our investigation, we anticipate that groundwater will likely be encountered during the excavation of the proposed development. The groundwater level at the site is expected to fluctuate several feet seasonally, depending on the amount of annual rainfall.

5.0 SEISMIC CONSIDERATIONS

5.1 Regional Seismicity and Faulting

The site is located in the Coast Ranges Geomorphic Province of California that is characterized by northwest-trending valleys and ridges. These topographic features are controlled by folds and faults that resulted from the collision of the Farallon North American plates and subsequent strike-slip faulting along the San Andreas Fault system. The San Andreas Fault is more than 600 miles long from Point Arena in the north to the Gulf of California in the south. The Coast Ranges 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, Hayward, and Calaveras faults. These and other faults in the region are shown on Figure 4. Numerous damaging earthquakes have occurred along these faults in recorded time. For these and other active faults within a 50-kilometer radius of the site, the distance from the site and estimated characteristic moment magnitude[5] [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).

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)	5.6	West	8.04
North San Andreas (Peninsula, SAP)	5.6	West	7.38
Monte Vista - Shannon	9.8	South	7.14
San Gregorio (North)	17	West	7.44
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	24	East	7.58
Hayward (South, HS)	24	East	7.00
Hayward (North, HN)	27	Northeast	6.90
Butano	29	South	6.93
Total Calaveras (CN+CC+CS+CE)	37	East	7.43
Calaveras (North, CN)	37	East	6.86
Mount Diablo Thrust North CFM	41	Northeast	6.72
Zayante-Vergeles (2011 CFM)	42	South	7.48
Mount Diablo Thrust	43	Northeast	6.67
Mount Diablo Thrust South	43	East	6.50
Las Positas	45	East	6.50
Calaveras (Central, CC)	46	East	6.85
Concord	47	Northeast	6.45
North San Andreas (North Coast, SAN)	48	Northwest	7.52
Hayward (Extension, HE)	49	East	6.18

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[5] 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.

Since 1800, four major earthquakes have been recorded on the North San Andreas Fault. In 1836, an earthquake with an estimated maximum intensity of VII on the Modified Mercalli (MM) scale occurred east of Monterey Bay on the San Andreas Fault (Toppozada and Borchardt 1998). The estimated moment magnitude (M_w) for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to an M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of loss of lives and property damage. This earthquake created a surface rupture along the San Andreas Fault from Shelter Cove to San Juan Bautista approximately 470 kilometers in length. It had a maximum intensity of XI (MM), an M_w of about 7.9, and was felt 560 kilometers away in Oregon, Nevada, and Los Angeles. The Loma Prieta Earthquake of October 17, 1989 had an M_w of 6.9 and occurred about 70 kilometers south of the site. On August 24, 2014, an earthquake with an estimated maximum intensity of VIII (severe) on the MM scale occurred on the West Napa fault. This earthquake was the largest earthquake event in the San Francisco Bay Area since the Loma Prieta Earthquake. The M_w of the 2014 South Napa Earthquake was 6.0.

In 1868, an earthquake with an estimated maximum intensity of X on the MM scale occurred on the southern segment (between San Leandro and Fremont) of the Hayward Fault. The estimated M_w for the earthquake is 7.0. In 1861, an earthquake of unknown magnitude (estimated M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake, which corresponds to an M_w of 6.2.

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

5.2 Seismic Hazards

Because the project site is in a seismically active region, we evaluated the potential for earthquake-induced geologic hazards, including ground shaking, ground surface rupture, liquefaction[6], lateral spreading[7] and cyclic densification.[8] We used the results of the borings and available subsurface information in the site vicinity to evaluate the potential of these phenomena occurring at the project site.

5.2.1 Ground Shaking

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

5.2.2 Fault Rupture

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

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

[7] 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.

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

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 shown on Figure 5, the site is not within a liquefaction hazard zone, defined by the map titled *State of California, Seismic Hazard Zones, Palo Alto Quadrangle, Official Map,* prepared by the California Geological Survey (CGS), dated October 18, 2006. Considering the soil encountered in our borings consist of stiff to hard clay with variable sand content and the medium dense to very dense sand and gravel layers had sufficient fines content, plasticity, and relative density, we judge the soil is not susceptible to liquefaction because of its cohesion and/or relative density. Therefore, we conclude the potential for liquefaction and associated hazards to occur at the site is very 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. The results of our borings indicate the soil above the groundwater at the site is not susceptible to cyclic densification due to its cohesion or relative density. Therefore, we conclude the potential for ground surface settlement resulting from cyclic densification at the site is very low.

6.0 DISCUSSION AND CONCLUSIONS

Based on the results of our field investigation, laboratory testing, and engineering analyses, we conclude there are no major geotechnical or geological issues that would preclude development of the site as planned. The primary geotechnical concerns affecting the proposed development include:

- relatively shallow groundwater relative to the proposed building foundation levels and excavation depth, and
- providing suitable lateral support and dewatering for the proposed excavation, while minimizing impacts to the surrounding improvements.

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

6.1 Groundwater

Based on the historical groundwater data discussed in Section 4.3, we recommend using a design high groundwater level of about 11 feet bgs (Elevation 88 feet) for the proposed project. As discussed in Section 1.0, we understand the proposed building will include one level of below-grade parking that will support both the affordable housing and office buildings. Current drawings indicate the basement finish floor depth will be about 15 feet below grade for both buildings. We estimate the construction of the proposed buildings will require an excavation bottomed roughly 19 feet bgs, assuming a mat foundation thickness of about 3 feet, a 12-inch-thick underslab rock drainage layer (for passive dewatering option), and a potential mudslab substrate for the waterproofing system, which are discussed in more detail later in this report. Therefore, the bottom-of-foundation may be as much as about 7 feet below the design high groundwater level. As a result, the proposed building’s foundation and below grade walls will need to be designed to resist hydrostatic pressures and include waterproofing.

Considering the proposed excavation will extend below the groundwater, the excavation will need to be temporarily dewatered, and the excavation shoring system will need to be designed for the effects of groundwater. A more detailed discussion regarding temporary excavation shoring and dewatering is presented in Section 6.3.

6.2 Foundations and Settlement

The soils encountered in our borings at the site are generally moderately to highly overconsolidated and capable of supporting new buildings loads without excessive static settlement.

Considering the proposed bottom-of-foundation will be as much as about 7 feet below the design high groundwater level, it will need to be designed to resist hydrostatic uplift pressures and be underlain by waterproofing. Although the native soils beneath the site are capable of supporting the building loads on conventional spread footings, a stiffened mat foundation system generally simplifies construction dewatering (discussed below) and the detailing of the waterproofing system. In addition, the weight of a stiffened mat foundation will provide greater resistance to the relatively high hydrostatic uplift pressures. Therefore, we conclude the proposed building may be supported on a stiffened mat foundation designed to resist hydrostatic uplift pressures. If the new foundation does not have sufficient uplift capacity, soil anchors (i.e., tiedowns) can be installed to resist uplift forces.

Our settlement analyses indicate total settlement of the mat foundation under static load conditions, assuming a maximum average contact pressure of about 1,500 psf, will be less than 1 inch. We anticipate most of the settlement will occur during construction. The amount of differential settlement between columns will be a function of the mat stiffness and hence its ability to spread the loads between columns, however, we expect the mat can be designed to limit differential settlements to 1/2 inch in 30 feet.

6.3 Construction Considerations

6.3.1 Excavation

The soil to be excavated consists of native soils, which can be excavated with conventional earth-moving equipment such as loaders and backhoes. Existing building foundation elements and abandoned utilities should be removed in their entirety within the proposed building footprint, which may require special handling and use of a hoe-ram may be required for removal during the excavation process.

6.3.2 Excavation Support

We estimate construction of the below-grade structure will require an excavation bottomed as deep as about 19 feet below grade. There is insufficient property line setback to slope cut the excavation. Therefore, excavation shoring will be required.

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

- protection of surrounding improvements, including neighboring structures, underground utilities, pavements, and sidewalks
- the presence of relatively shallow groundwater and the desire to minimize lowering of the water table outside the limits of the excavation in areas sensitive to ground settlement
- proper construction of the shoring system to reduce potential for vertical and lateral ground movement, and
- cost.

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

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

Because the proposed excavation depth is greater than about 15 feet, we conclude cantilevered shoring systems are not cost effective—therefore, both systems listed above would require tiebacks or internal bracing. Tieback anchors will extend beneath the neighboring properties, which will require encroachment agreements with the City of San Mateo and the Peninsula Corridor Joint Powers Board.

Soldier Pile-and-Lagging

A conventional tied back soldier pile-and-lagging system would be feasible, however, because the system is pervious, an active dewatering system consisting of a series of extraction wells installed outside the excavation, would likely be required to prevent caving of the soil and excessive water from seeping through the lagging boards into the excavation. Furthermore, as discussed in Section 6.3.3, an active dewatering system outside the excavation would likely lower the groundwater level beneath the Caltrain tracks and surrounding streets, which could result in ground settlement. We do not know the settlement tolerances of the Caltrain tracks, but we conclude that efforts should be made to reduce the potential for excessive groundwater drawdown and settlement beneath the tracks. The tolerances for dewatering-induced settlement beneath the city streets should also be considered in the design of the proposed shoring and dewatering system. Conventional tiedback soldier pile-and-lagging with active dewatering may be feasible along the edges of the site adjacent to city streets, if some ground settlement is acceptable during construction, however, at a minimum, we recommend that a permeable system with active dewatering not be used along edges of the site that are within approximately 200 feet of the Caltrain tracks, as indicated on the Site Plan (Figure 2). This setback distance may be re-evaluated following detailed groundwater drawdown analyses and feedback from Caltrain regarding their settlement tolerances.

Continuous Soil-Cement Mix (SMX) Soldier Pile Wall

Soil-cement mixing (SMX), also called deep soil mixing (DSM), is a viable option for creating a continuous soldier pile shoring wall that supports the excavation, as well as provides a hydraulic barrier when properly constructed. SMX columns are installed by injecting and blending cement and bentonite into the soil using a drill rig equipped with single or multiple augers/paddles, or a specialized proprietary cutterhead. The soil is mixed with the binder material(s) in situ, forming continuous, overlapping, soil-cement columns or a continuous wall of uniform thickness. Steel beams are placed in the soil-cement columns to provide rigidity. The SMX system, in combination with steel soldier beams and tiebacks, serves to shore the excavation as well as cut off lateral groundwater flow, thus reducing the amount of dewatering required from within the excavation. The portion of the SMX wall embedded below the bottom of excavation also greatly reduces the upward flow of water into the excavation bottom. This approach should be used, at a minimum, at portions of the excavation near the Caltrain tracks or any other areas that are sensitive to dewatering-induced ground settlement, as discussed above. Soil-cement walls are considered temporary and permanent building walls are built inside of the soil-cement walls following application of drainage panels (if used) and waterproofing.

SMX systems are generally installed under design-build contracts by specialty contractors. The required size, spacing, length, and strength of the SMX columns, beams, and tieback elements should be determined by the shoring designer, based on the design soil, water, and surcharge pressures presented in Section 7.4 of this report. However, there are numerous factors that influence the quality, consistency, strength, and permeability of the resulting soil-cement mix, which are controlled by the materials, methods, and equipment employed by the contractor performing the soil mixing. These factors include, but are not limited to:

- Types of binder material(s) used – i.e., cement, bentonite, etc.; wet-mixed vs. dry-mixed,
- quantities and proportions of binder material(s) used – i.e., water-to-binder ratio; volume ratio of SMX,
- equipment used to perform the mixing – i.e., single-auger, multi-auger, or cutter-based equipment,
- plumbness and amount of overlap between adjacent SMX columns,
- homogeneity of soil-cement mixture – controlled by rate of mixing, number of stages, and equipment used, and
- depth and diameter of predrilling, which may be required within hard clay or dense sand layers, depending on equipment selected.

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

6.3.3 Excavation Dewatering

Due to the low permeability of most of the soil underlying the site, an active dewatering system, such as a series of dewatering wells installed outside the perimeter of the excavation, may have limited effectiveness in drawing down the water level in the center of the excavation, where the subgrade soil consists of clay. Furthermore, as discussed in the previous section, a perimeter active dewatering system will temporarily lower the groundwater level outside the site, such as beneath city streets and sidewalks, as well as below the Caltrain railway tracks to the northeast. Where limiting potential dewatering-induced ground settlement is desired, we conclude the excavation dewatering employed during construction of the proposed building should consist of an internal system operating within the excavation footprint (shallow sumps and/or wells), combined with a continuous cut-off wall shoring, such as SMX. A combination of active and passive approaches will likely be required to adequately manage water in the excavation during construction, depending on the final shoring configuration selected. The design and proper implementation of the excavation dewatering system should be the responsibility of the contractor. Where/if an active dewatering approach is used, the system should be capable of drawing the water level down at least three feet below the bottom of excavation during construction. Where/if a passive approach is used, to facilitate the collection of groundwater at discrete extraction well and sump locations, we recommend over-excavating by at least 12 inches below the design bottom-of-mat and installing a minimum 12-inch-thick continuous layer of clean 3/4-inch drain rock. The drainage layer will help protect the soil subgrade, which will be sensitive to disturbance from construction equipment, as well as provide a means for water to flow to the extraction points, reducing the potential for hydrostatic pressure to prematurely build up beneath the mat.

The construction dewatering system must be capable of maintaining the groundwater level below the foundation subgrade until sufficient building weight is available to resist the hydrostatic uplift pressure, at which time the groundwater may be allowed to rise to its normal elevation. The project structural engineer should determine when the temporary dewatering system can be turned off, based on the recommended design groundwater level presented in Sections 4.3 and 6.1.

In summary, we conclude the dewatering system for the project may consist of either a complete passive system with continuous cut-off wall shoring, or a combination of passive and active dewatering with a permeable soldier pile-and-lagging shoring, depending on the tolerances for ground settlement for the Caltrain tracks and within the streets around the proposed excavation. In either case, we recommend the dewatering contractor perform detailed groundwater drawdown analyses and develop estimated contours of water drawdown outside of the site, at which point we can perform settlement analyses to estimate the potential ground settlement that may occur for the proposed shoring and dewatering system.

6.4 Soil Corrosivity

Corrosivity testing was performed by Project X Corrosion of Murrieta, California on two soil samples obtained during our field investigation from B-1 (1.75 ft) and B-2 (4 ft). The results of the test are presented on Figure B-3 in Appendix B of this report. Based on the resistivity test results, the samples are classified as “highly corrosive” to buried metals. Accordingly, all buried iron, steel, cast iron, ductile 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 chloride, sulfide, and sulfate ion concentrations and pH of the soil do not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

7.0 RECOMMENDATIONS

Recommendations for site preparation, excavation, fill placement, excavation shoring and dewatering, foundations, basement wall design, pavement design, and seismic design are presented in the following sections of the report.

7.1 Site Preparation, Excavation, and Fill Placement

Site clearing should include removal of all existing pavements, former foundation elements, and underground utilities. Any vegetation and organic topsoil (if present) should be stripped in areas to receive improvements (i.e., building, pavement, or flatwork). Tree roots with a diameter greater than 1/2 inch within three feet of subgrade should be removed. Excessively dry soil at tree removal locations, as determined in the field by the geotechnical engineer, should also be excavated and replaced. Demolished asphalt concrete should be taken to an asphalt recycling facility. Aggregate base beneath existing pavements may be re-used as select fill if carefully segregated.

During excavation for the below-grade parking level, the excavation will extend below groundwater. The foundation excavation subgrade will consist of saturated soil and will be sensitive to disturbance, especially under construction equipment wheel loads. To provide a working surface on which to install the mudslab and waterproofing system, and to facilitate dewatering, the soil should be overexcavated to provide room for a minimum 12-inch-thick continuous layer of crushed rock, where passive dewatering will be used. Where/if an active dewatering system is used, the rock layer may be omitted, provided the dewatering system is capable of adequately dewatering the subgrade soil. To minimize disturbance of the soil subgrade, the last two feet of soil should be excavated with a track-mounted excavator with a smooth bucket or bar welded across the teeth. Even with tracked equipment, the exposed subgrade may be sensitive, especially if the excavation is not adequately dewatered. We do not recommend operating any trucks or rubber-tired equipment on the exposed mat subgrade. Any disturbed soil at or below subgrade level (i.e., bottom of overexcavation) should be removed by hand if it cannot be reached with a tracked excavator. Following approval by our engineer, the bottom of the excavation should be covered with at least 12 inches of clean 3/4-inch crushed rock (where required) and/or a mudslab, to provide a firm working surface. The crushed rock should meet the gradation requirements presented below in Table 2.

TABLE 2
Gradation Requirements for Gravel Blanket Beneath Mat (Passive Dewatering Approach)

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

If any engineered fill will be placed above the crushed rock, it should then be covered with a nonwoven filter fabric (Mirafi 140NC or equivalent) prior to placement of engineered fill. A mud slab is generally required beneath most waterproofing products. If no engineered fill is to be placed above the crushed rock blanket, the mud slab may be placed directly over the rock (no filter fabric required).

For planning purposes, a maximum temporary cut slope inclination of 1:1 (horizontal to vertical) may be assumed for the native clay soil above the groundwater, which corresponds to OSHA Type B soil. If granular material or seepage is observed in the cut slopes during construction, the material should be downgraded to OSHA Type C soil and a corresponding maximum inclination of 1.5:1 should be used. All soil below the design water table should be assumed to be Type C soil.

In areas to receive new fill, pavements, concrete flatwork, pavers, etc., the subgrade soil should be scarified to a depth of at least 8 inches, moisture-conditioned, and compacted to the specified percent relative compaction,[9] as presented below in Table 3. All fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned, and compacted in accordance with the requirements provided below in Table 3.

TABLE 3
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
General fill – select fill less than 5 feet thick	90+	Above optimum
General fill – low plasticity clay less than 5 feet thick	90+	Above optimum
General fill – select fill and low-plasticity clay greater than 5 feet thick	95+	Above optimum
Utility trench backfill – select fill	90+	Above optimum
Utility trench backfill – low plasticity clay	90+	Above optimum
Utility trench backfill – clean sand or gravel and low-plasticity fills greater than 5 feet thick	95+	Near optimum
Pavement subgrade – low-plasticity clay	95+	Above optimum
Pavement section - aggregate base	95+	Near optimum
Exterior slabs – select fill	90+	Above optimum
Exterior slabs – low-plasticity	90+	Above optimum

Where the above recommended compaction requirements conflict with the City of San Mateo standard details for pavements, sidewalks, or trenches within the public right-of-way, the City Engineer or inspector should determine which compaction requirements should take precedence.

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[9] 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.

7.1.1 Soil Subgrade Stabilization

In some areas, soft, wet soil may be exposed during grading, causing the subgrade to deflect and rut under the weight of grading equipment. Although the majority of the near-surface soil beneath the site consists of stiff to hard clay, if heavy, wheeled equipment is used close to the drawn-down water table, or if grading is performed during the wet season, these materials may become disturbed and soften. In these areas, some form of subgrade stabilization may be required if disturbance occurs. Several options for stabilizing subgrade are presented below.

Aeration

Aeration consists of mixing and turning the soil to naturally lower the moisture content to an acceptable level. Aeration typically requires several days to a week of warm, dry weather to effectively dry the material. Material to be dried by aeration should be scarified to a depth of at least 12 inches; the scarified material 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 previous recommendations. Aeration is typically the least costly subgrade stabilization alternative; however, it generally requires the most time to complete and may not be effective if the soft material extends to great depths. Aeration will likely not be effective where the podium subgrade extends below or near the groundwater table; however, it depends on the time of year construction is performed.

Overexcavation

Another method of achieving suitable subgrade in areas where soft, wet soil is exposed is to overexcavate the soft subgrade soil and replace it with drier, granular material. If the soft material extends to great depths, the upper 18 to 24 inches of soft material may be overexcavated and a geotextile tensile fabric (Mirafi 500X or equivalent) placed beneath the granular backfill to help span over the weaker material. The fabric should be pulled tight and placed at the base of the overexcavation, extending at least two feet laterally beyond the limits of the overexcavation in all directions. The fabric should be overlapped by at least two feet at all seams. Granular material, such as Class 2 aggregate base, should then be placed and compacted over the geotextile tensile fabric.

Where very soft subgrade conditions are encountered, a bi-directional geogrid, such as Tensar TriAx TX-140 or equivalent, may be required in lieu of tensile fabric. Where geogrids are used, the depth of overexcavation will likely be on the order of 12 to 18 inches. The geogrids should be overlapped by at least two feet and tied with hog rings or nylon ties at a spacing not to exceed 10 feet. The geogrids should be covered with a well-graded granular fill, such as Class 2 aggregate base; open-graded rock should not be used. All backfill placed over the geogrid should be compacted in accordance with our previous recommendations.

Chemical Treatment

Lime and/or cement have been used to dry and stabilize fine-grained soils with varying degrees of success. Lime- and/or cement-treatment will generally decrease soil density, change its plasticity properties, and increase its strength. The degree to which lime will react with soil depends on such variables as type of soil, mineralogy, quantity of lime, and length of time the lime-soil mixture is cured. Cement is generally used when a significant amount of granular material or low-plasticity silt is present in the soil. The quantity of lime and/or cement added generally ranges from 3 to 7 percent by weight and should be determined by laboratory testing. The specialty contractor performing the chemical treatment should select the most appropriate additive and quantity for the soil conditions encountered.

If chemical treatment is used to stabilize soft subgrade, a treatment depth of about 18 inches below the final soil subgrade will likely be required. The soil being treated should be scarified and thoroughly broken up to full depth and width. The treated soil should not contain rocks or soil clods larger than three inches in greatest dimension. Treated soil should be compacted to at least 90 percent RC.

7.1.2 Select Fill

Select fill should consist of imported soil that is free of organic matter, contain no rocks or lumps larger than three inches in greatest dimension, have a liquid limit less than 40 and plasticity index less than 12, and be approved by the geotechnical engineer. Select fill should be placed in lifts not exceeding eight inches in loose thickness, moisture-conditioned to above optimum moisture content, and compacted to at least 90 percent relative compaction beneath concrete flatwork and sidewalks. Beneath vehicular pavements, or in areas where the fill thickness is greater than five feet, the select fill should be compacted to at least 95 percent relative compaction. Samples of proposed select fill material should be submitted to the geotechnical engineer at least three business days prior to use at the site.

The grading contractor should provide analytical test results or other suitable environmental documentation indicating the imported fill is free of hazardous materials at least three days before use at the site. If this data is not provided, a minimum of two weeks will be required to perform any necessary analytical testing.

Aggregate Base Material

Imported aggregate base material may be used as general fill, trench backfill (above bedding materials), or as select fill beneath pavements or exterior concrete flatwork. Aggregate base beneath pavements should meet the requirements in the 2010 Caltrans Standard Specifications, Section 26, for Class 2 Aggregate Base (3/4 inch maximum).

Controlled Low Strength Material

Controlled low strength material (CLSM) may be considered as an alternative to fill beneath structures or pavement. CLSM should meet the requirements in the 2010 Caltrans Standard Specifications. It is an ideal backfill material when adequate room is limited or not available for conventional compaction equipment, or when settlement of the backfill must be minimized. No compaction is required to place CLSM. CLSM should have a minimum 28-day unconfined compressive strength of at least 100 pounds per square inch (psi).

7.1.3 Exterior Flatwork Subgrade Preparation

We recommend exterior concrete flatwork, including sidewalks, be underlain by a minimum of 6 inches of select material. Select fill should be moisture-conditioned and compacted in accordance with the requirements provided above in Table 3.

In areas to receive new concrete flatwork, the upper 8 inches of native clay should be scarified, moisture-conditioned, and re-compacted in accordance with the requirements presented in Table 3 prior to placement of select fill. This grading should be performed under the observation of our field engineer. If zones of weak or loose soil that extend deeper than the upper 8 inches are encountered during grading, the material should be over-excavated down to firm material, as determined by our field engineer, and replaced with engineered fill.

7.1.4 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All temporary excavations used in construction should be designed, planned, constructed, and maintained by the contractor and should conform to all state and/or federal safety regulations and requirements, including those of CAL-OSHA. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of clean sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with 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 in accordance with the recommendations previously presented. If imported clean sand or gravel (defined as poorly-graded soil with less than 5 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

The bottom of foundations for the proposed building or any surface structures, such as Caltrain, should be below an imaginary line extending up at a 1.5:1 (horizontal to vertical) inclination from the base of utility trenches. Alternatively, the portion of the utility trench (excluding bedding) that is below the 1.5:1 line can be backfilled with CLSM (see Section 7.1.2 for material requirements).

Where the above utility trench backfill recommendations are in conflict with the City of San Mateo standard details for underground utility trenches within the public right-of-way, the City Engineer or inspector should determine which material and compaction requirements should take precedence.

7.1.5 Drainage and Landscaping

Positive surface drainage should be provided around the buildings to direct surface water away from foundations and below-grade walls. To reduce the potential for water ponding adjacent to the buildings, we recommend the ground surface within a horizontal distance of five feet from the building slope down away from the building with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundation and below-grade walls. The use of water-intensive landscaping around the perimeter of the at-grade building should be avoided to reduce the amount of water introduced to the clay subgrade.

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

7.2 Foundation Design

Provided the estimated total and differential settlements presented in Section 6.2 are acceptable, the proposed building may be supported on a stiffened mat foundation that is underlain by waterproofing and designed to resist hydrostatic uplift pressures. If the building weight is not sufficient to resist the hydrostatic uplift pressures imposed by the groundwater, tiedown anchors may be required to provide the mat foundation with additional uplift resistance. The following sections present our recommendations for the design and construction of a mat foundation and tiedown anchors.

7.2.1 Mat Foundation

The mat foundation should be constructed on a minimum 12-inch-thick layer of clean crushed rock, where passive dewatering is used. The purpose of the rock layer is to protect the soil subgrade and facilitate dewatering during construction. If/where an active dewatering system is used and the system adequately dewaters the subgrade, the rock layer may be omitted. One or more mudslabs may be required beneath the bottom of mat foundation depending on the waterproofing system requirements and construction methods selected for the project—this should be evaluated and specified by the waterproofing consultant and product manufacturer. The native soil subgrade beneath the rock layer should be firm and undisturbed, as described in Section 7.1. The top of the mat foundation may be used as the lowest basement floor, or a thin layer of concrete (topping slab) may be placed above the mat to provide a smooth wearing surface.

For structural design of the mat foundation we recommend using an initial coefficient of vertical subgrade reaction of 75 kips per cubic foot (kcf) under DL+LL conditions. This value has been reduced to account for the size of the mat/equivalent footings (therefore, this is not k_{v1} for 1-foot-square plate). Once the structural engineer estimates the distribution of bearing stress on the bottom of the mat and the corresponding deflections, we should review the distribution and revise the modulus of subgrade reaction, if appropriate. We recommend the mat be designed for allowable bearing pressure of 4,500 psf for dead-plus-live loads, which can be increased by one-third for total loads (including seismic and wind loads).

Lateral forces can be resisted by friction along the base of the mat and passive pressure against the sides of the mat foundation. To compute lateral resistance, we recommend using an allowable uniform pressure of 2,000 psf (rectangular distribution) for transient loads and an equivalent fluid weight (triangular distribution) of 260 pcf for sustained loads above the groundwater and 125 pcf for sustained loads below the groundwater. The allowable friction factor will depend on the type of waterproofing used at the base of the mat. For bentonite-based water proofing membranes, such as Paraseal or Voltex, a friction factor of 0.12 should be used (assumes a bentonite friction angle of 10 degrees). If Preprufe is used, a base friction factor of 0.20 should be used. Friction factors for other types of waterproofing membranes can be provided upon request. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

The mat subgrade will be sensitive to disturbance due to its proximity to the groundwater table. The final two feet of excavation and fine grading of the mat subgrade should be performed with tracked equipment to minimize heavy concentrated loads that may disturb the wet soil. Rubber-tired equipment and dump trucks should not be operated on the final mat subgrade. The subgrade should be free of standing water, debris, and disturbed materials and be approved by the geotechnical engineer prior to placing the gravel drainage layer and/or mudslab. The mat subgrade should be kept moist following excavation and maintained in a moist condition until drain rock and/or mudslab is placed. If the foundation soil dries during construction, it will eventually heave, which may result in cracking and distress.

Considering the internal excavation dewatering system will need to be capable of continuously maintaining the water level below the bottom of the mat until the building has sufficient weight to resist hydrostatic uplift pressures associated with the design water level, the mat will need to be constructed with temporary block-outs to accommodate the extraction wells or sump pits used to extract the water from the drainage layer. Once it has been determined by the structural engineer that the dewatering system may be shutoff, the pumps will need to be removed, and the block-outs promptly waterproofed and plugged. The detailing of the waterproofing and plugging system at these locations will be critical and should be evaluated by a waterproofing consultant and structural engineer experienced with such operations.

7.2.2 Tiedown Anchors

Tiedown anchors may be used in conjunction with the mat foundation at the site, if needed, to resist the design hydrostatic uplift forces. Tiedowns are installed by advancing a small-diameter hole (typically between 5 to 8 inches in diameter) using either hollow-stem augers or air-track equipment that advances smooth steel drill casing (e.g., a Klemm rig). A large-diameter reinforcing bar or high-capacity steel strands are inserted into the hole, and then grout is injected into the hole under pressure as the auger or steel drill casing is withdrawn. Post-grouting can be performed to achieve higher capacities.

We recommend tiedowns be spaced at least four shaft diameters or three feet apart, center-to-center, whichever is greater. Tiedowns for this project will gain support through skin friction in primarily stiff to hard clay. Tiedown capacity depends significantly on installation procedures, and installation procedures vary. Assuming the tiedowns are installed with a Klemm rig and post-grouted, we recommend using allowable skin friction values of 1,500 psf. We estimate the allowable skin friction value includes a factor of safety of at least 2.0. If the contractor installing the tiedowns believes they can achieve a higher capacity than that assumed above, higher capacities may be used, provided the factor of safety is verified through a load testing program, as detailed below. We recommend using a minimum bond length of 15 feet. The skin friction values used in design should be verified by a testing program. Because the tiedowns will be permanent, they should have double corrosion protection.

The required tiedown bond length should be confirmed by a proof-test program conducted under our observation. We recommend proof-testing a minimum of two tiedowns in tension to 200 percent of the design load (DL) at the start of production installation. The two anchors tested to 200 percent DL may require larger bar diameter or additional strands, so that their structural capacity is not exceeded during testing. The remaining production anchors should be proof tested to 150 percent DL. During testing, the deflection of each tiedown should be monitored with a free-standing, tripod-mounted dial gauge accurate to at least 0.001 inches. We recommend deflection of the tiedowns be measured at load increments equal to about 25 percent of the design load. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute readings is more than 0.04 inches, the load should be held for an additional 50 minutes, with additional readings taken at 15, 20, 30, 45 and 60 minutes. If the deflection is more than 0.08 inches between the 10- and 60-minute readings, the tiedown design load should be re-evaluated. Replacement tiedowns should be provided, as directed by the structural engineer, for tiedowns that fail the tests. Tiedowns should be locked off at a load to limit movement during stabilizing of the groundwater level to less than 1/2 inch (structural engineer should confirm).

7.3 Permanent Below-Grade Walls

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

- At-rest equivalent fluid weight of 60 pcf above the design groundwater table and 90 pcf below.
- Active pressure of 40 pcf plus a seismic increment of 24 pcf (triangular distribution) above the design groundwater level, and 80 pcf plus a seismic increment of 11 pcf (triangular distribution) below the groundwater level.

The recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads. Where the below-grade wall is subject to traffic loading within 10 feet of the wall, an additional uniform lateral pressure of 100 psf, applied to the upper 10 feet of the wall, should be used. If the traffic loading will be limited to passenger vehicles only (e.g., a garage ramp), the vehicular surcharge may be reduced to 50 psf.

To protect against moisture migration, below-grade walls should be waterproofed, and water stops should be placed at all construction joints. The design pressures recommended for above the design water level are based on a **fully drained** condition. Although part of the basement walls will be above the groundwater level, water can accumulate behind the walls from other sources, such as rainfall, irrigation, and broken water lines, etc. One acceptable method for backdraining a basement wall is to place a prefabricated drainage panel against the back of the wall. The drainage panel should extend down to a perforated PVC collector pipe at the design high groundwater level (or higher if confirmed by the structural engineer). The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material or 3/4-inch drain rock wrapped in filter fabric (Mirafi NC or equivalent). 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 may be used in lieu of the perforated pipe surrounded by gravel described above. The pipe should be connected to a suitable discharge point; a sump and pump system may be required to drain the collector pipes. Wall drainage above the design high water table may be omitted if the wall is designed for saturated earth pressures over its entire height.

If backfill is required behind basement walls prior to pouring the podium slabs, the walls should be braced, or hand compaction equipment used, to prevent unacceptable surcharges on walls (as determined by the structural engineer).

7.4 Excavation Shoring

As discussed in Section 6.3, we conclude the shoring system for the proposed excavation at the site may consist of either a complete tiedback cut-off wall system, such as SMX, or a combination of a tiedback soldier pile-and-lagging and continuous cut-off wall system designed based on the dewatering configuration selected for each portion of the excavation. The purpose of the continuous cut-off wall is to reduce the potential for groundwater seepage into the excavation and to reduce the potential for groundwater drawdown beneath areas around the site that may be sensitive to ground settlement, such as the nearby Caltrain tracks.

The safety of workers and equipment in or near the excavation is the responsibility of the contractor. A structural engineer knowledgeable in this type of construction should design the shoring. We should review the geotechnical aspects of the proposed shoring system to check that it meets the intent of our geotechnical recommendations. During construction, we should observe the installation and load testing of the shoring system and check the condition of the soil encountered during excavation.

Recommendations regarding the design and construction of the shoring, as well as design, construction and load testing of tieback anchors, are presented in the following sections.

7.4.1 Design Lateral Earth and Water Pressures

Our recommendations for design lateral earth pressures and tiebacks for a soldier pile-and-lagging shoring system combined with active dewatering are presented on Figure 6. Our recommendations for design lateral earth and water pressures and tiebacks for a continuous cut-off wall shoring system combined with passive dewatering are presented on Figure 7. The recommended water and earth pressure distributions presented in Figure 7 have been adjusted to account for the non-hydrostatic water pressures and corresponding effective stresses behind the wall and in front of the toe that result from the excavation dewatering from within. In our analyses, we assumed the continuous cut-off wall (SMX) will extend at least 10 feet below the bottom of excavation. If a different cut-off wall embedment depth is proposed, we may need to re-evaluate the recommended design pressures.

7.4.2 Soil-Cement Mix (SMX) Shoring

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

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

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

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

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

Detailed specifications for minimum required SMX strength for the various stages of excavation should be established by the shoring designer and followed by the shoring contractor during construction. The construction schedule should allow time for adequate curing and strength gain of the SMX material prior to proceeding with successive lifts of excavation. A clear quality control program should be established and implemented to confirm the design strengths have been achieved prior to proceeding with excavation.

7.4.3 Tiebacks

Temporary tiebacks may be used to restrain the shoring. Alternatively, internal bracing would be required. The vertical load from the temporary tiebacks should be accounted for in the design. The recommended tieback design criteria are presented on Figures 6 and 7 and the following paragraphs.

Tiebacks should derive their load-carrying capacity from the soil behind an imaginary line sloping upward from a point H/5 feet away from the bottom of the excavation and sloping upwards at 60 degrees from the horizontal, where H is the wall height in feet. Tiebacks should have a minimum unbonded length of 15 feet. All tiebacks should have a minimum bonded length of 15 feet and spaced at least four feet on center. During construction, the bottom of the excavation should not extend more than two feet below a row of unsecured tiebacks.

Tieback allowable capacity will depend upon the drilling method, hole diameter, grout pressure, and workmanship. For estimating purposes, we recommend using the skin friction value presented on Figures 6 and 7, assuming the tiebacks are post-grouted at least once. Higher skin friction values may be used if confirmed with pre-production load testing.

The contractor should be responsible for determining the actual length of tiebacks required to resist the lateral earth and water pressures imposed on the temporary retaining systems. Determination of the tieback length should be based on the contractor's familiarity with the installation method and experience in similar soil conditions. The computed bond length should be confirmed by a proof-testing program under the observation of an engineer experienced in this type of work. Replacement tiebacks should be installed for tiebacks that fail the load test. If any tiebacks fail to meet the proof-testing requirements, additional tiebacks should be added to compensate for the deficiency, as determined by the shoring designer.

Tieback Testing

We should observe all tieback testing. A proof test is a simple test used to measure the total movement of the tieback during one cycle of incremental loading. All production tiebacks should be confirmed by proof tests to at least 1.25 times the design load. The bar or strands selected for the system must be capable of safely holding the maximum test load such that their structural capacity is not exceeded.

The movement of each tieback should be monitored with a free-standing, tripod-mounted dial gauge during proof testing. During the test, the tieback load and axial deflection are measured at each loading increment. The maximum test load should be held for a minimum of 10 minutes, with readings taken at 0, 1, 3, 6, and 10 minutes. If the difference between the 1- and 10-minute reading is less than 0.04 inch during the loading, the test is discontinued. If the difference is more than 0.04 inch, the holding period is extended by 50 minutes to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the tieback test results and determine whether the tiebacks are acceptable. A proof-tested tieback with a ten-minute hold is acceptable if the tieback carries the maximum test load with less than 0.04-inch movement between one and 10 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. A proof-tested tieback with a 60-minute hold is acceptable if the tieback carries the maximum test load with less than 0.08-inch movement between six and 60 minutes, and total movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the unbonded length. Tiebacks that fail to meet the first criterion will be assigned a reduced capacity. If the total movement of the tiebacks at the maximum test load does not exceed 80 percent of the theoretical elastic elongation of the unbonded length, the contractor should replace the tiebacks.

7.4.4 Construction Monitoring

Control of ground movement will depend as much on the timeliness of installation of lateral restraint as on the design. During excavation, the shoring system is expected to yield and deform laterally, which could cause the ground surface adjacent to the shoring wall to settle. The magnitudes of shoring movements and the resulting settlements are difficult to estimate because they depend on many factors, including the method of installation and the contractor's skill in the shoring installation. Ground movements due to a properly designed and constructed shoring system should be within ordinary accepted limits of about one inch. A monitoring program should be established to evaluate the effects of the construction on the adjacent properties.

The conditions of existing structures within 40 feet of the site should be photographed and surveyed prior to the start of construction and monitored periodically during construction. In addition, prior to the start of excavation, the contractor should establish survey points on the shoring system, on the ground surface at critical locations behind the shoring, and on adjacent buildings. These survey points should be used to monitor the vertical and horizontal movements of the shoring and the ground behind the shoring throughout construction.

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

- Prior to any excavation or shoring work at the site,
- after installing soldier piles / SMX columns,
- after excavation of each lift,
- after the excavation reaches its lowest elevation, and
- every two weeks until the street-level floor slab is constructed.

7.5 Pavement Design

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

7.5.1 Flexible (Asphalt Concrete) Pavement Design

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. For pavement design, we assumed a resistance value (R-value) of 5, which is appropriate for clays. Recommended pavement sections for traffic indices ranging from 4.5 to 7.5 are presented in Table 4.

TABLE 4
AC Pavement Sections

TI	Asphaltic Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4.5	2.5	9.5
5.0	3.0	10.0
5.5	3.0	12.0
6.0	3.5	13.0
6.5	4.0	13.5
7.0	4.0	15.5
7.5	4.5	16.5

The upper 12 inches of the subgrade soil should be scarified, moisture conditioned, and compacted in accordance with requirements presented in Table 3 in Section 7.1. The aggregate base should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction. Both the subgrade and the aggregate base should be firm and non-yielding during proof-rolling under the observation of our field engineer.

If pavements will be adjacent to irrigated landscaped areas, curbs adjacent to those areas should extend through the aggregate base and at least three inches into the underlying soil to reduce the potential for irrigation water to infiltrate into the pavement section.

7.5.2 Rigid (Portland Cement Concrete) Pavement

For the parking garage ramp and driveway, which will experience only passenger car traffic, we recommend the concrete slab be at least five inches thick and placed over at least six inches of Class 2 aggregate base (AB). For concrete pavement that may be subject to traffic from heavier vehicles, such as garbage trucks or moving trucks, assuming a maximum single-axle load of 20,000 pounds and a maximum tandem axle of 32,000 pounds, the recommended rigid pavement section for these axle loads is 6 inches of Portland cement concrete over at least six inches of

Class 2 aggregate base. Where fire truck traffic is expected, the pavement section should consist of seven inches of Portland cement concrete over at least six inches of Class 2 aggregate base. Prior to placement of the aggregate base, we should confirm by proof rolling that the native soil subgrade is firm and non-yielding, and compacted in accordance with the specifications in Table 3. If the subgrade deflects excessively during proof rolling, it should be scarified, aerated, and recompacted as discussed in Section 7.1 of this report.

The modulus of rupture of the concrete should be at least 500 psi at 28 days. Contraction joints should be constructed at 15-foot spacing. Where the outer edge of a concrete pavement meets asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. Concrete slabs subject to vehicular traffic should be reinforced with a minimum of No. 4 bars spaced at 16 inches in both directions.

Recommendations for subgrade preparation and aggregate base compaction for concrete pavement are the same as those described above for asphalt concrete pavement.

7.6 Seismic Design

The latitude and longitude of the site are 37.5645° and -122.3210°, respectively. For design in accordance with the 2022 CBC, we recommend the following:

- Site Class D (stiff soil, non-default)
- $S_S = 1.873g$, $S_1 = 0.768g$

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

- F_a = 1.0, F_v = 1.7
- S_{MS} = 1.873g, S_{M1}* = 1.958g
- S_{DS} = 1.249g, S_{D1}* = 1.306g
- Seismic Design Category E for Risk Factors I, II, and III

Depending on the structural design methodology and fundamental period of the proposed building, it may be advantageous to perform a ground motion hazard analysis (the project structural engineer should confirm). We can perform a ground motion hazard analysis upon request.

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, excavation, grading, fill placement and compaction, shoring installation and load testing, and foundation installation. These observations will allow us to compare actual with anticipated soil conditions and to check that the contractor’s work conforms with 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 soil and groundwater conditions do not deviate appreciably from those disclosed in the exploratory borings. If any variations or undesirable conditions are encountered during construction, we should be notified so that additional recommendations can be made. The recommendations presented in this report are developed exclusively for the proposed development described in this report and are not valid for other locations and construction in the project vicinity.

REFERENCES

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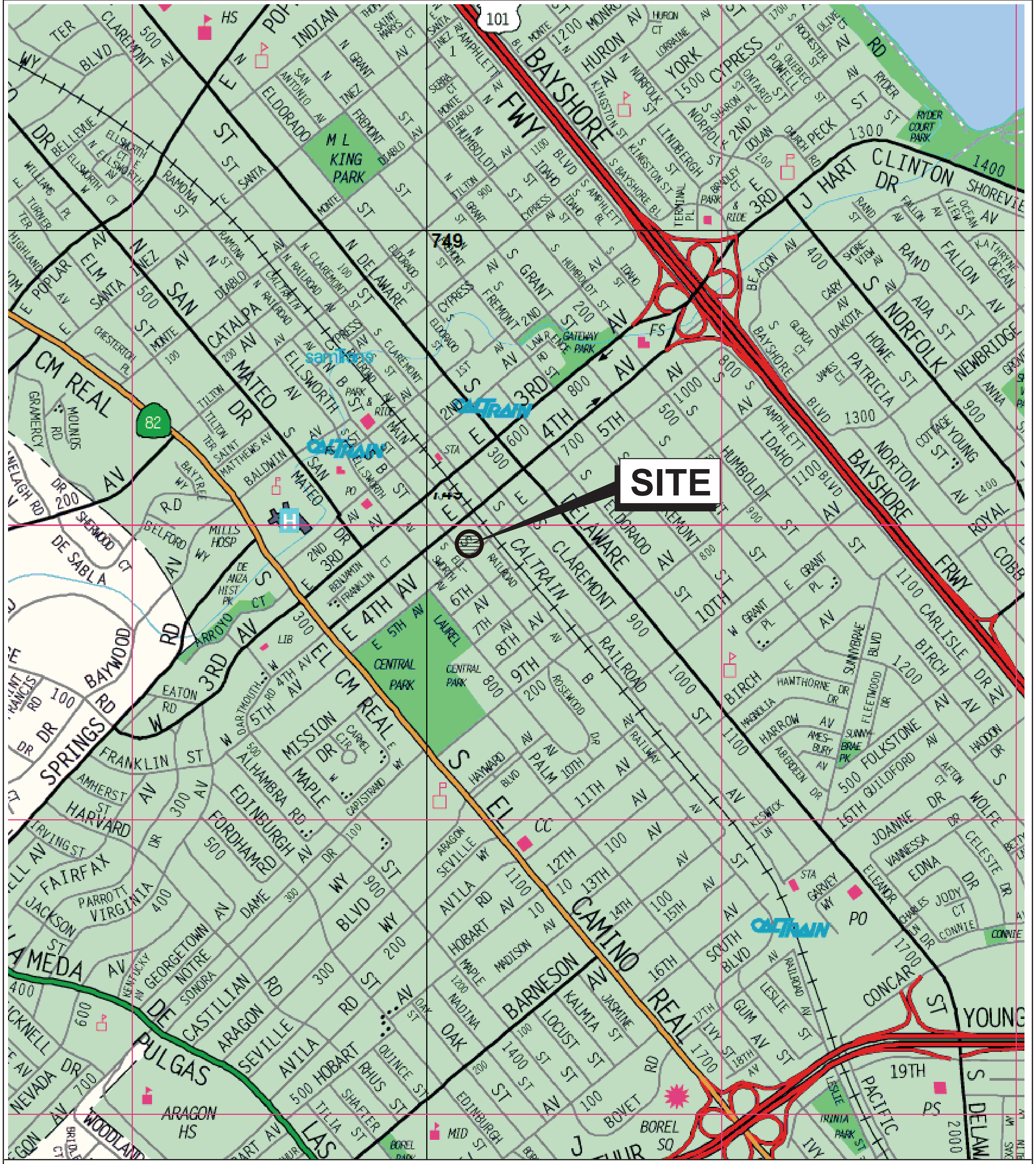
FIGURES



Base map: The Thomas Guide
San Mateo County
2002

0 1/4 1/2 Mile

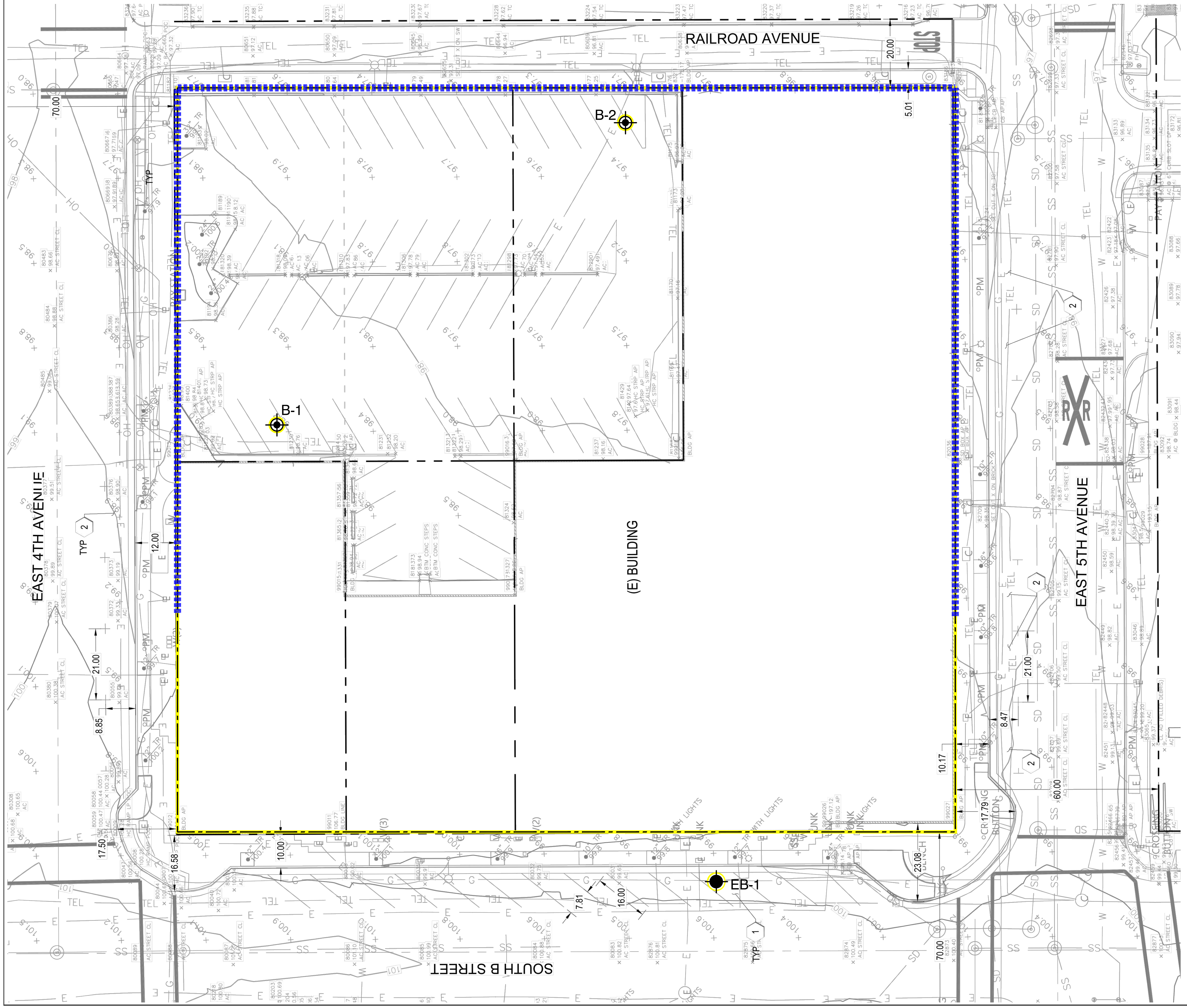
Approximate scale

401-445 S. B Street
San Mateo, California

ROCKRIDGE
GEOTECHNICAL

SITE LOCATION MAP

Date 04/05/22	Project No. 20-1869	Figure 1



EXPLANATION

B-1 ⊕	Approximate location of boring by Rockridge Geotechnical Inc., January 17, 2022
EB-1 ●	Approximate location of boring by Cornerstone Earth Group, November 6 & 21, 2019
—·—	Project limits
▪▪▪▪▪▪▪▪	Approximate locations (at a minimum) in which temporary shoring should consist of continuous cut-off wall, and passive dewatering system should be used, pending results of detailed groundwater drawdown analysis

0 30 Feet

Approximate scale

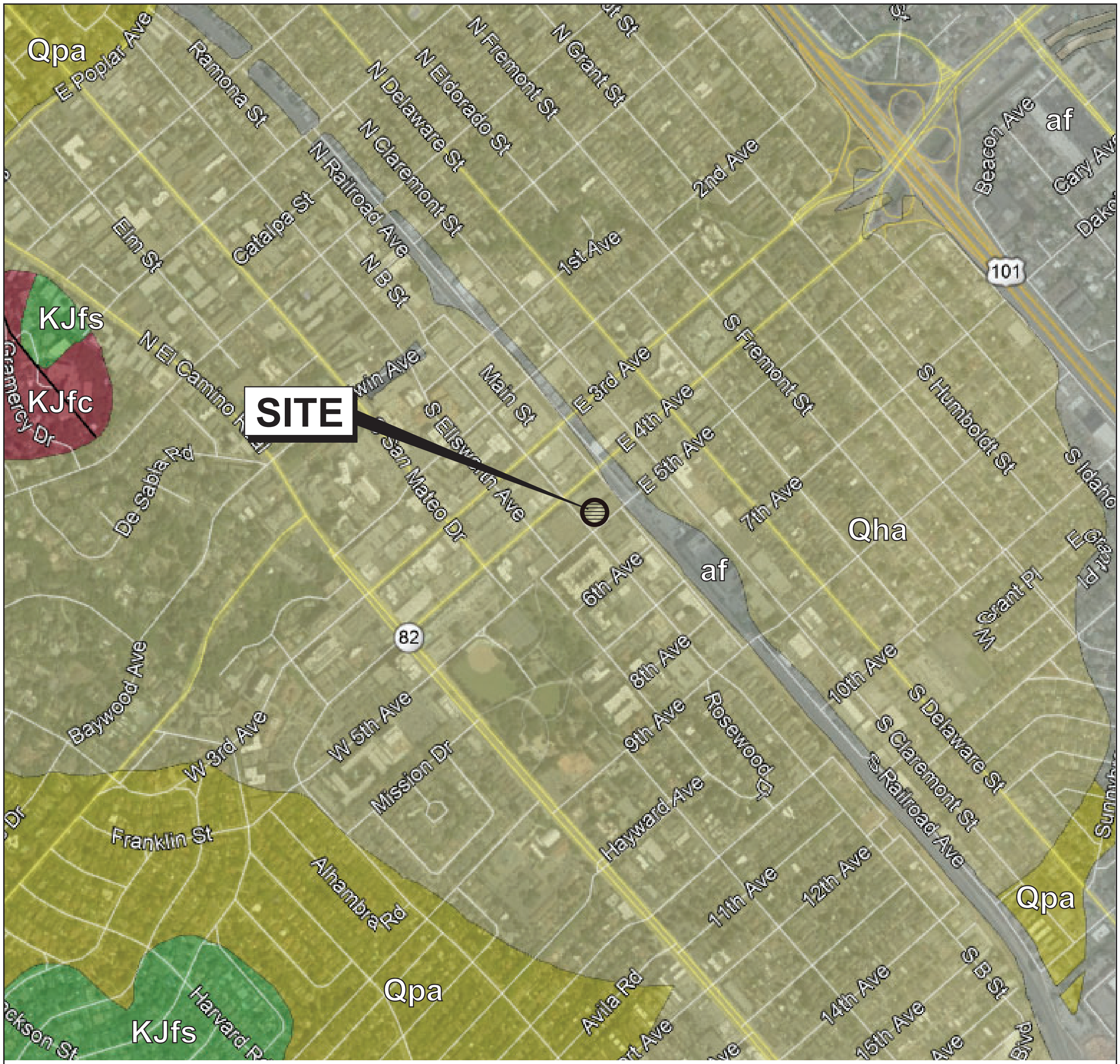
Reference: Base map from a drawing titled "Existing Conditions", by RMW Architecture, dated February 1, 2023.

401-445 S. B STREET
San Mateo, California

SITE PLAN

Date 04/14/22	Project No. 20-1869	Figure 2

ROCKRIDGE GEOTECHNICAL



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

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
Qpa	Alluvium (Pleistocene)
KJfs	Franciscan Complex sedimentary rocks (Early Cretaceous and (or) Late Jurassic)
KJfc	Franciscan Complex chert (Early Cretaceous and (or) Late Jurassic)

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

N

0 1000 2000 Feet

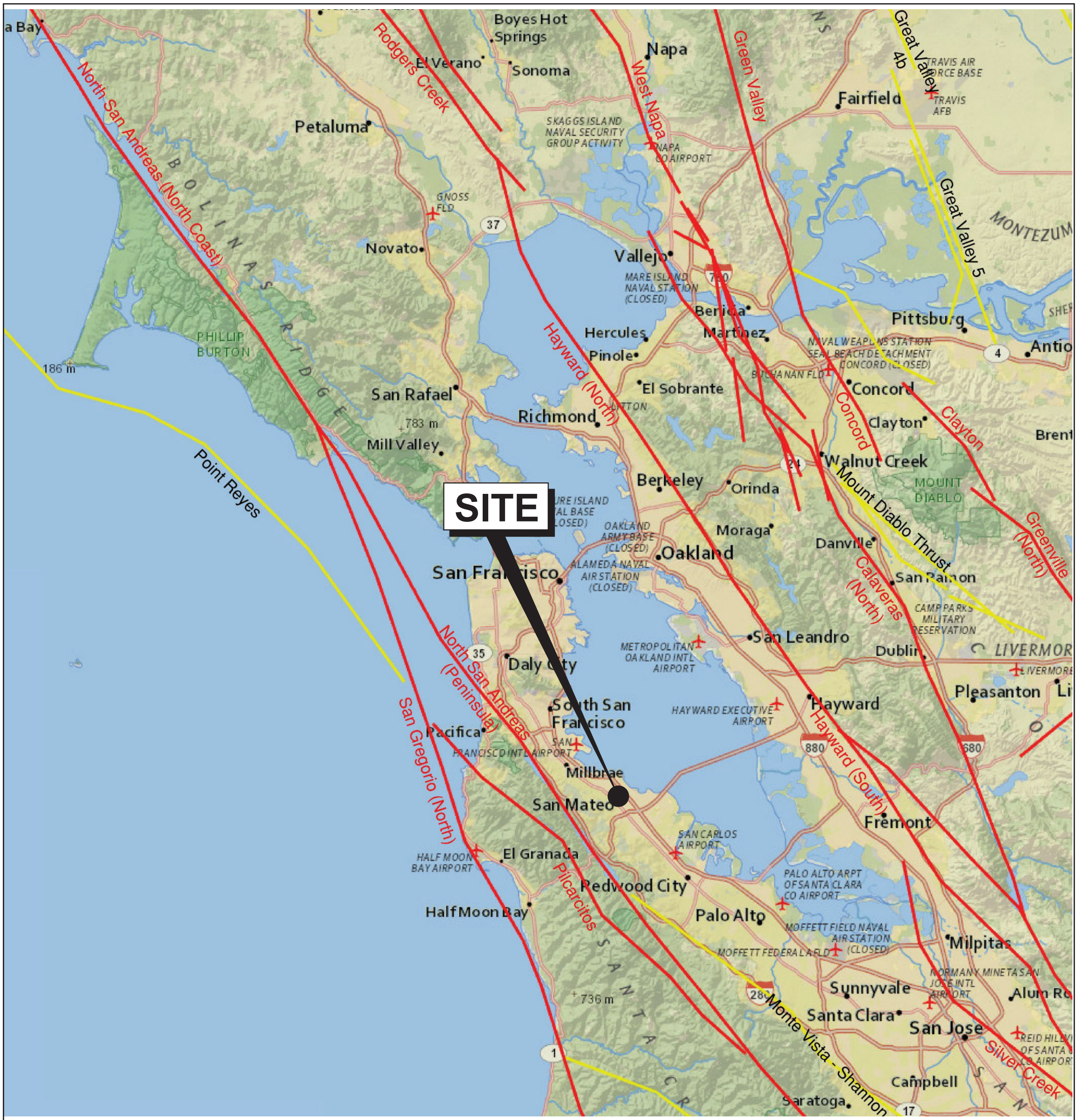
Approximate scale

401-445 S. B Street
San Mateo, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 04/05/22 | Project No. 20-1869 | Figure 3



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

EXPLANATION

— Strike slip

— Thrust (Reverse)

— Normal

0 5 10 Miles

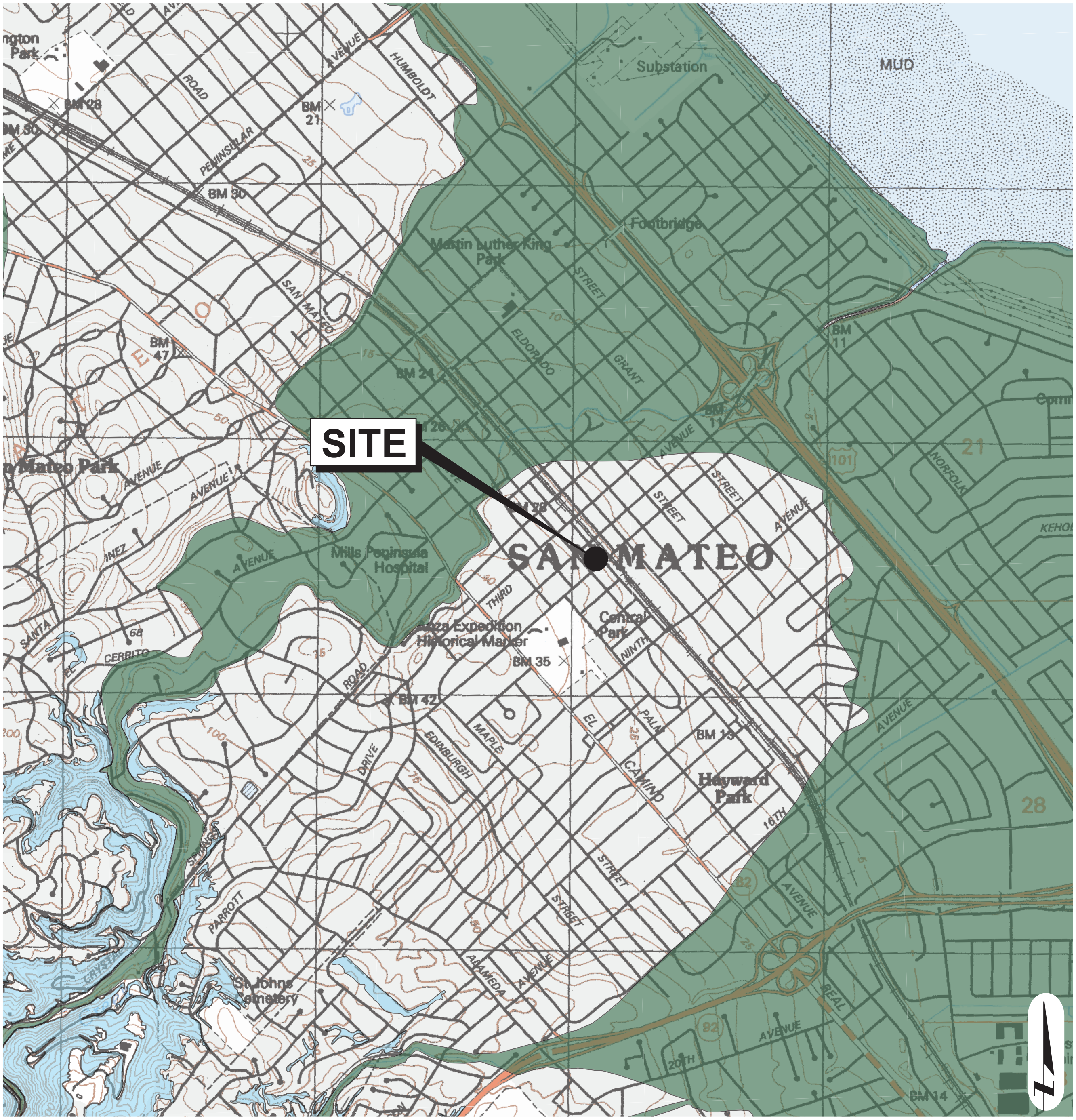
Approximate scale

401-445 S. B Street
San Mateo, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL FAULT MAP

Date 04/05/22	Project No. 20-1869	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 Mateo Quadrangle
California Geological Survey
Released January 11, 2018

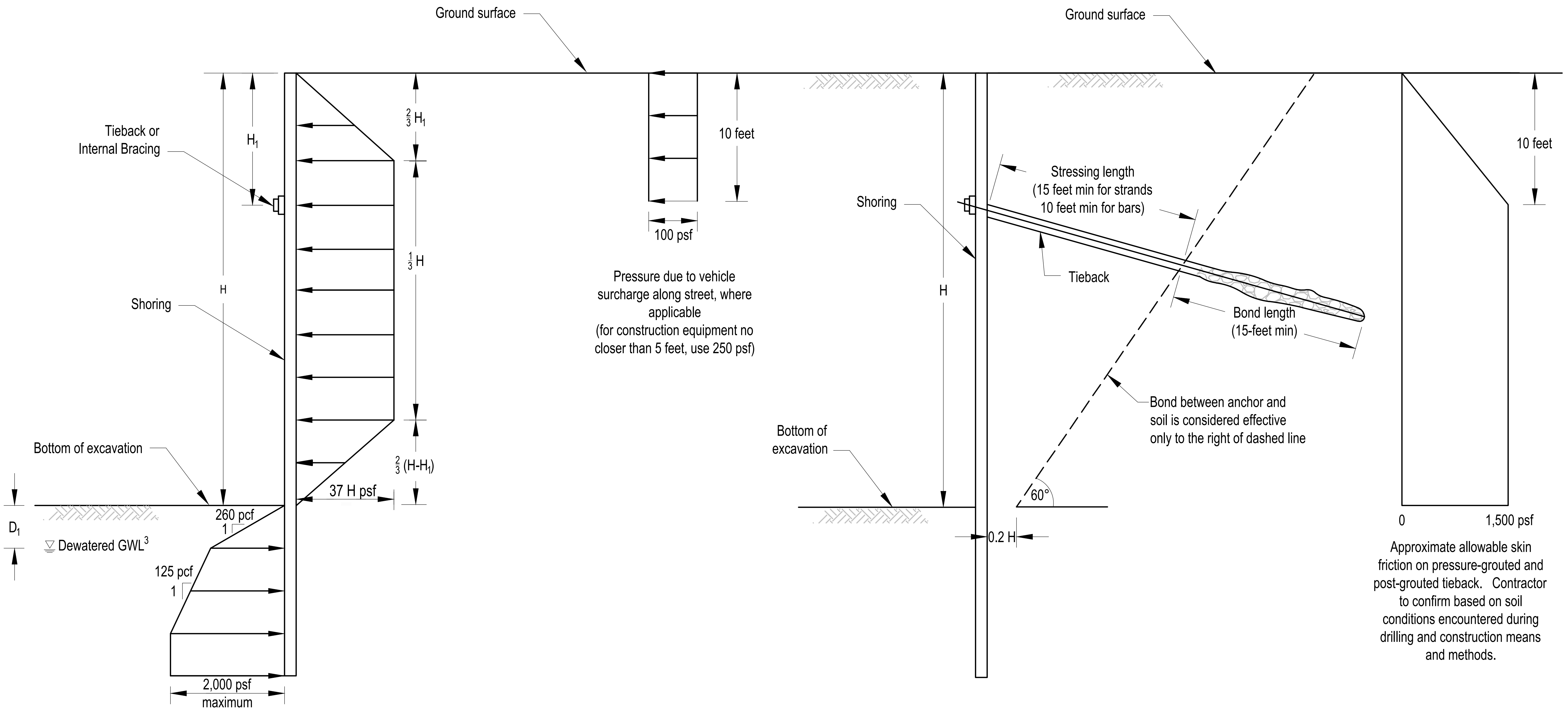
0 2,000 4,000 Feet
Approximate scale

401-445 S. B Street
San Mateo, California

ROCKRIDGE GEOTECHNICAL

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Date 04/05/22	Project No. 20-1869	Figure 5



Pressure due to vehicle surcharge along street, where applicable (for construction equipment no closer than 5 feet, use 250 psf)

Bond between anchor and soil is considered effective only to the right of dashed line

Approximate allowable skin friction on pressure-grouted and post-grouted tieback. Contractor to confirm based on soil conditions encountered during drilling and construction means and methods.

NOT TO SCALE

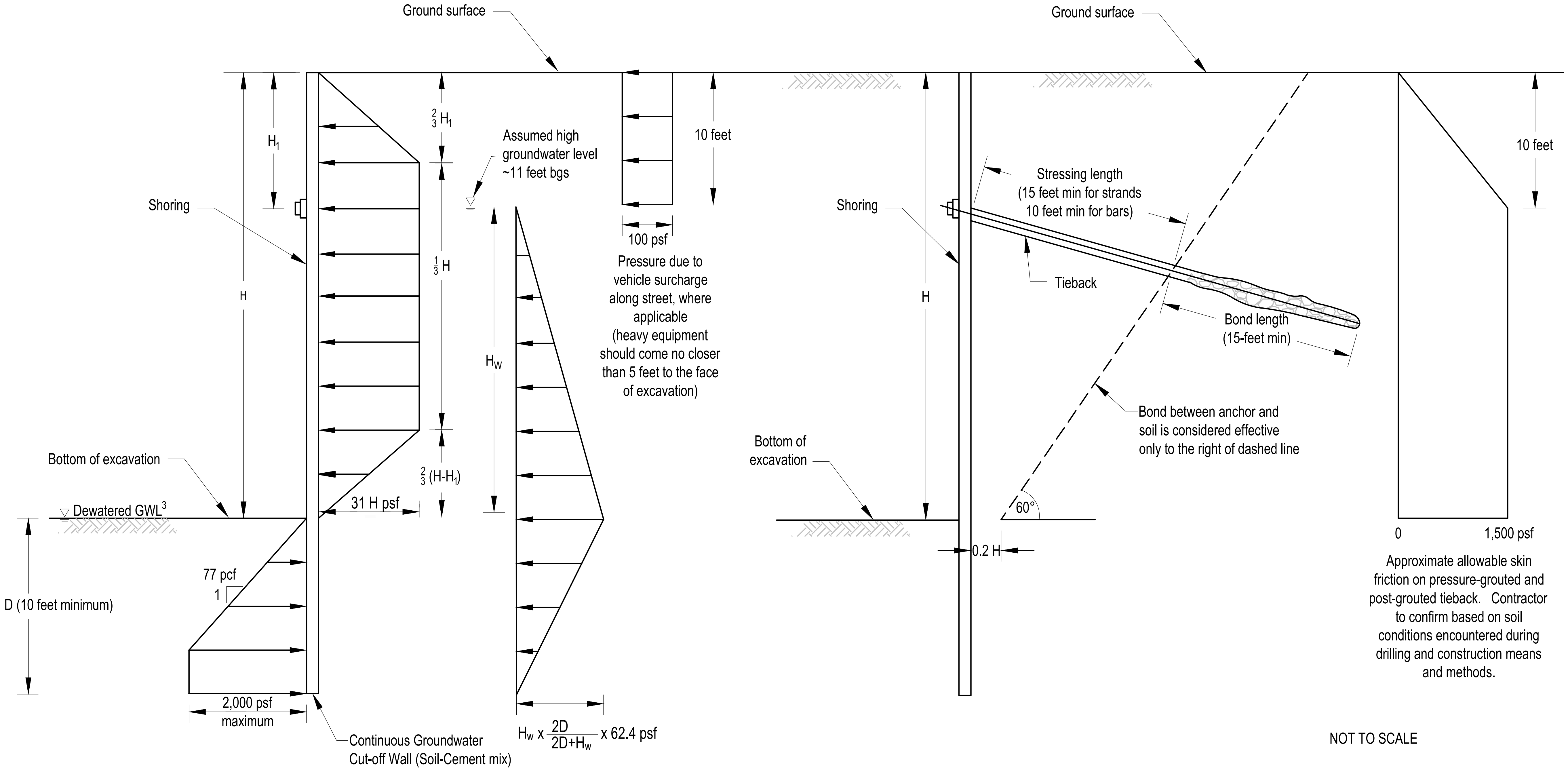
Notes:
1. Passive pressures include a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters.
3. Assumed temporary groundwater level (D_1) should be coordinated with the excavation dewatering design.
4. Recommended pressures do not include potential surcharge pressures that may result from cranes, boom pumps, or neighboring building foundations.

401-445 S. B STREET
San Mateo, California

DESIGN PARAMETERS FOR SOLDIER-PILE-AND-LAGGING TEMPORARY SHORING SYSTEM WITH ONE ROW OF TIEBACKS AND ACTIVE DEWATERING

Date 04/07/22	Project No. 20-1869	Figure 6

ROCKRIDGE
GEOTECHNICAL



Notes:

1. Passive pressure includes a factor of safety of about 1.5.
2. For soldier piles spaced at more than three times the soldier pile diameter, the passive pressure should be assumed to act over three diameters, provided the concrete or soil-cement mix is sufficiently strong to accommodate the corresponding stresses (shoring designer should confirm).
3. Stressing lenth; minimum 15 feet for strands, minimum 10 feet for bars.
4. The recommended design pressures are for preliminary design. These pressures are dependent on the depths of excavation, we may need to revise these recommended design pressures once the depths are finalized.
5. Recommended pressures do not include potential surcharge pressures that may result from cranes, boom pumps, or neighboring building foundations.

401-445 S. B STREET
San Mateo, California

DESIGN PARAMETERS FOR TEMPORARY ANCHORED CUT-OFF WALL SHORING SYSTEM WITH ONE ROW OF TIEBACKS AND PASSIVE DEWATERING

Date 04/07/22 | Project No. 20-1869 | Figure 7

ROCKRIDGE
GEOTECHNICAL

APPENDIX A

Logs of Borings by Rockridge Geotechnical

PROJECT: **401-445 S. B Street**
San Mateo, California

Log of Boring B-1

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 01/17/2022
Date finished: 01/17/2022
Drilling method: Hollow-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Safety Hammer
Sampler: Modified California (MC), Standard Penetration Test (SPT)

Logged by: J. Lei
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

DEPTH (feet)	SAMPLES				LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA					
	Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]			Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
						3 inches of asphalt concrete						
						6 inches of aggregate base						
1												
2	MC		7 9 13	14		CLAY (CL) brown, stiff, moist, trace sand Soil Corrosivity Test; see Appendix B						
3												
4	MC		13 16 19	22	CL	very stiff LL = 23, PI = 10; see Appendix B					10.9	108
5												
6	MC		14 21 35	35								
7						light brown with black speckling						
8	MC		30 50/6"	32/6"								
9						SILTY SAND (SM) brown, dense, moist, fine to coarse sand, trace fine to coarse subangular gravel						
10												
11	SPT		17 17 17	37		Particle Size Distribution; see Appendix B				17		
12												
13					SM							
14												
15	SPT		50/5"	54/5"		very dense						
16												
17												
18												
19						▼ (01/17/2022; 11:43 AM)						
20					CL	CLAY (CL) brown, hard, wet						
21	MC		32 50/5"	32/5"		GRAVELLY CLAY (CL) brown, hard, wet, fine subangular gravel						
22					CL							
23												
24												
25						CLAY with SAND (CL) brown, hard, wet, trace fine sand						
26	MC		31 41 42	52								
27					CL							
28												
29												
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1869
Figure: A-1a

PROJECT: **401-445 S. B Street**
San Mateo, California

Log of Boring B-1

PAGE 2 OF 2

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION	LABORATORY TEST DATA: Type of Strength Test	LABORATORY TEST DATA: Confining Pressure Lbs/Sq Ft	LABORATORY TEST DATA: Shear Strength Lbs/Sq Ft	LABORATORY TEST DATA: Fines %	LABORATORY TEST DATA: Natural Moisture Content, %	LABORATORY TEST DATA: Dry Density Lbs/Cu Ft
	MC	■	16 32 42	47	CL	CLAY with SAND (CL) (continued)						
31 32 33 34					CL	CLAY (CL) light brown, hard, wet						
35 36	MC	■	6 8 14	14	SC	CLAYEY SAND with GRAVEL (SC) gray-brown, medium dense, wet, fine to coarse sand, fine to coarse subangular gravel LL = 30, PI = 13; see Appendix B Particle Size Distribution; see Appendix B				17		
37 38 39					CL	CLAY (CL) brown, stiff, wet, trace sand						
40 41	MC	☐	28 50/6"	32/6"	CL	CLAY with SAND (CL) gray-brown with orange mottling, hard, wet, fine to coarse sand						
45 46	SPT	◪	13 16 26	45								
50	MC	■	50/6"	32/6"								
51 52 53 54 55 56 57 58 59 60												

Boring terminated at a depth of 50.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 19 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. SPT sampler used without liners.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1869

Figure: A-1b

PROJECT: **401-445 S. B Street**
San Mateo, California

Log of Boring B-2

PAGE 1 OF 2

Boring location: See Site Plan, Figure 2
Date started: 01/17/2022
Date finished: 01/17/2022
Drilling method: Hollow-stem auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Downhole Safety Hammer
Sampler: Modified California (MC), Standard Penetration Test (SPT)

Logged by: J. Lei
Drilled by: Exploration Geoservices, Inc.
Rig: Mobile B-61

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/Cu Ft
0						3 inches of asphalt concrete 9 inches of aggregate base						
1	MC		11 10 7	11	CL	CLAY with SAND (CL) brown, stiff, moist, fine and medium sand						
2					CL	CLAY (CL) brown, stiff, moist, trace fine sand						
3	MC		7 16 31	30		brown with black speckling, very stiff to hard Soil Corrosivity Test; see Appendix B						
5	MC		21 31 33	40	CL	CLAY with SAND (CL) brown, hard, miost, fine sand, trace coarse sand						
7	MC		21 23 20	27		very stiff						
8					SC	CLAYEY SAND (SC) light brown, dense, moist, fine sand						
10	SPT		21 31 38	75		very dense						
11					GC	CLAYEY GRAVEL with SAND (GC) light brown, very dense, moist, fine subangular gravel, fine sand						
14					GP-GC	GRAVEL with CLAY and SAND (GP-GC) brown, medium dense, moist, fine to coarse subangular gravel, fine to coarse sand LL = 44, PI = 26; see Appendix B Particle Size Distribution; see Appendix B						
15	MC		11 14 16	19						11	7.9	100
19						▼ (01/17/2022; 10:00 AM)						
20	MC		12 14 16	19	CL	CLAY (CL) brown, very stiff, wet, trace fine sand and subangular gravel						
25	MC		9 11 11	14		stiff, increased plasticity, no sand or gravel content						
27					CL	CLAY (CL) light brown with black speckling, very stiff, wet, trace fine sand						
30												

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1869
Figure: A-2a

PROJECT: **401-445 S. B Street**
San Mateo, California

Log of Boring B-2

PAGE 2 OF 2

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
31	MC	■	10 16 26	26	CL	CLAY (CL) (continued)						
32						CLAYEY SAND (SC) gray-brown, dense, wet, fine sand						
33												
34					SC							
35												
36	SPT	◪	20 20 20	43		Particle Size Distribution; see Appendix B				19		
37						CLAY (CL) light brown with orange mottling, hard, wet						
38												
39												
40					CL							
41	MC	■	20 32 50/6"	52		gray-brown with orange mottling, trace coarse sand						
42												
43												
44						CLAY with SAND (CL) gray-brown with orange mottling, hard, wet, fine to coarse sand						
45												
46	MC	■	30 50/4"	32/4"	CL							
47												
48												
49												
50					CL	CLAY (CL) brown, hard, wet, trace fine sand and subangular gravel						
51	MC	■	30 50/6"	32/6"								
52												
53												
54												
55												
56												
57												
58												
59												
60												

Boring terminated at a depth of 51 feet below ground surface.
Boring backfilled with cement grout.
Groundwater encountered at a depth of 19 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. SPT sampler used without liners.

ROCKRIDGE GEOTECHNICAL

Project No.: 20-1869

Figure: A-2b

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	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
Sand	No. 4 to No. 200	4.76 to 0.075
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.075
Silt and Clay	Below No. 200	Below 0.075

▽ Unstabilized groundwater level

▼ Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with California or Modified California split-barrel sampler. Darkened area indicates soil recovered
◩	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
⊠	Disturbed sample
⊡	Sampling attempted with no recovery
◫	Core sample
▣	Analytical laboratory sample
▥	Sample taken with Direct Push sampler
▥	Sonic

SAMPLER TYPE

C	Core barrel
CA	California split-barrel sampler with 2.5-inch outside diameter and a 1.93-inch inside diameter
D&M	Dames & Moore piston sampler using 2.5-inch outside diameter, thin-walled tube
O	Osterberg piston sampler using 3.0-inch outside diameter, thin-walled Shelby tube
PT	Pitcher tube sampler using 3.0-inch outside diameter, thin-walled Shelby tube
MC	Modified California sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter
SPT	Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside diameter and a 1.38- or 1.5-inch inside diameter (refer to text)
ST	Shelby Tube (3.0-inch outside diameter, thin-walled tube) advanced with hydraulic pressure

401-445 S. B Street
San Mateo, California

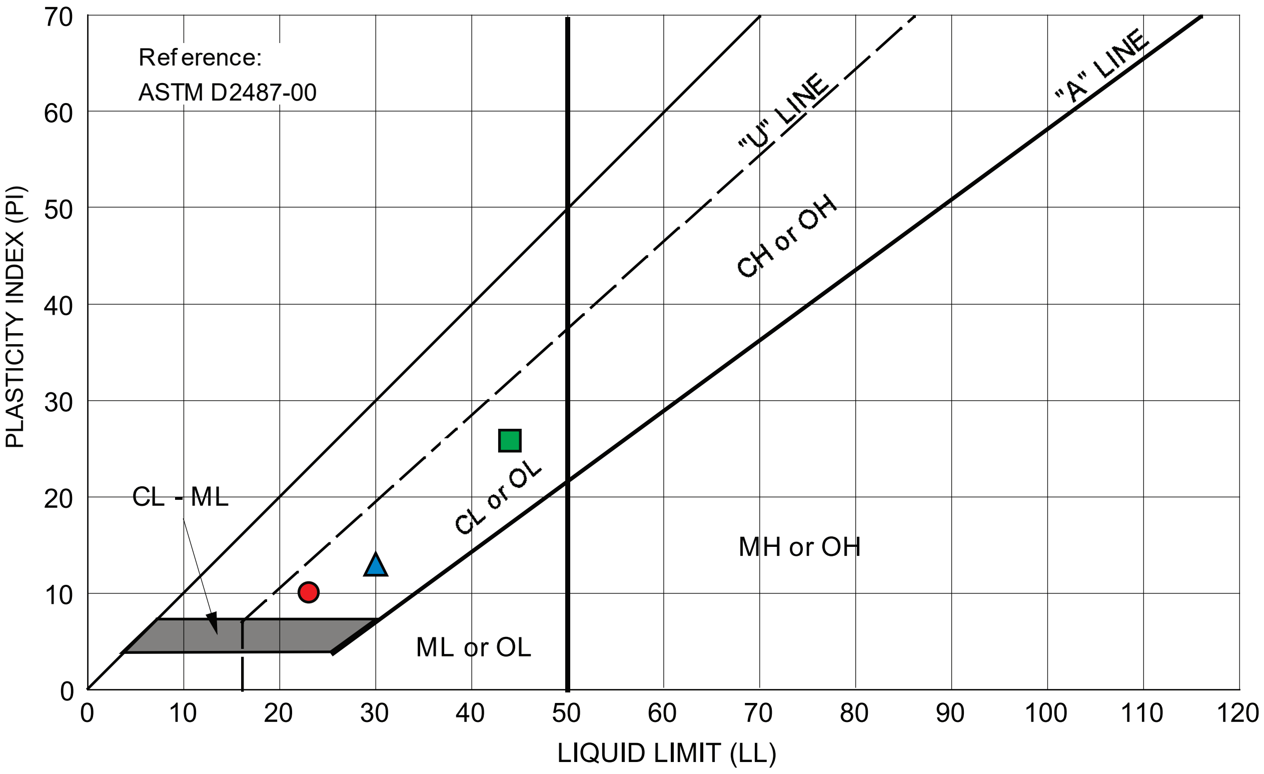
ROCKRIDGE
GEOTECHNICAL

CLASSIFICATION CHART

Date 04/05/22 | Project No. 20-1869 | Figure A-3

APPENDIX B

Laboratory Test Results



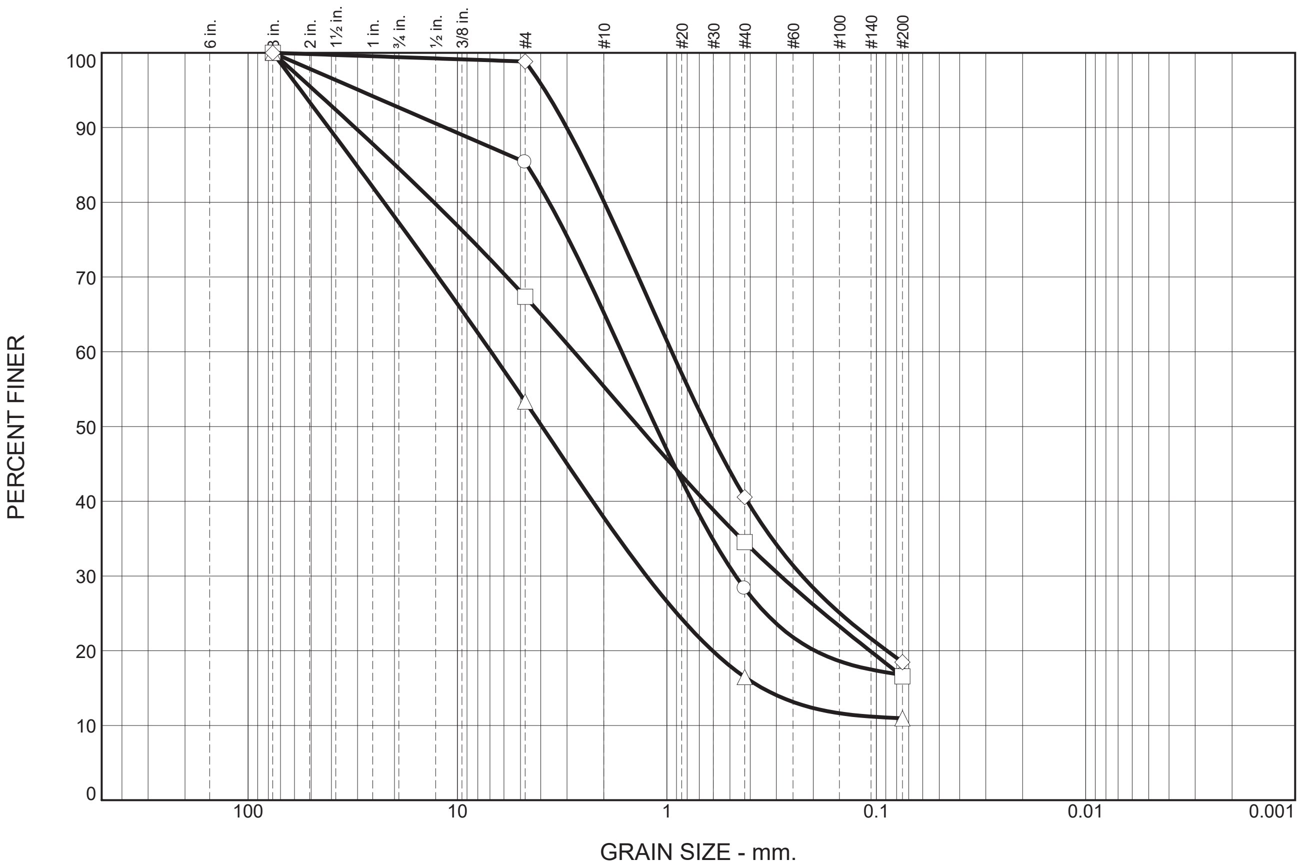
Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-1 at 3.5 feet	CLAY (CL), brown	10.9	23	10	--
▲	B-1 at 35.5 feet	CLAYEY SAND with GRAVEL (SC), gray-brown	--	30	13	16.6
■	B-2 at 15.25 feet	GRAVEL with CLAY and SAND (GP-GC), brown	7.9	44	26	11.0

401-445 S. B Street
San Mateo, California

ROCKRIDGE
GEOTECHNICAL

PLASTICITY CHART

Date 04/05/22 | Project No. 20-1869 | Figure B-1



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	7.3	7.3	20.2	36.9	11.5	16.8	
□	0.0	15.5	17.1	12.0	20.9	17.9	16.6	
△	0.0	22.7	24.0	15.5	21.3	5.5	11.0	
◇	0.0	0.6	0.6	18.7	39.6	22.0	18.5	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	10.0'	SILTY SAND, brown	SM
□	B-1	35.5'	CLAYEY SAND with GRAVEL, gray-brown	SC
△	B-2	15.3'	GRAVEL with CLAY and SAND, brown	GP-GC
◇	B-2	35.0'	CLAYEY SAND, gray-brown	SC

401-445 S. B Street
San Mateo, California

ROCKRIDGE
GEOTECHNICAL

PARTICLE SIZE DISTRIBUTION REPORT

Date 04/05/22 | Project No. 20-1869 | Figure B-2

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

Soil Analysis Lab Results

Client: Rockridge Geotechnical, Inc.
Job Name: 407-445 S B. Street
Client Job Number: 20-1869
Project X Job Number: S220208E
February 9, 2022

	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**		**Chlorides**		**Resistivity**		**pH**	**Redox**	**Sulfide**	**Nitrate**	**Ammonium**	**Lithium**	**Sodium**	**Potassium**	**Magnesium**	**Calcium**	**Fluoride**	**Phosphate**
		SO_4^{2-}		Cl^-		**As Rec'd**	**Minimum**			S^{2-}	NO_3^-	NH_4^+	Li^+	Na^+	K^+	Mg^{2+}	Ca^{2+}	F_2^{--}	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)**
Sample 1b, B-1 CLAY (CL), brown	1.75	321.1	0.0321	17.0	0.0017	10,050	1,206	7.8	235	ND	642.3	2.3	0.03	28.8	15.7	18.6	36.2	7.6	11.3
Sample 2b, B-2 CLAY (CL), brown	4	89.8	0.0090	3.7	0.0004	6,700	2,144	7.8	217	ND	190.0	0.9	0.01	23.2	3.4	14.3	19.3	7.6	0.1

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)

401-445 S. B STREET
San Mateo, California

ROCKRIDGE
GEOTECHNICAL

SOIL CORROSIVITY
TEST RESULTS

Date 04/05/22 | Project No. 20-1869 | Figure B-3

APPENDIX C

Log of Boring by Others

BORING NUMBER EB-1

PAGE 1 OF 2

CORNERSTONE
EARTH GROUP

PROJECT NAME 222 East 4th Avenue
PROJECT NUMBER 129-6-1
PROJECT LOCATION San Mateo, CA
DATE STARTED 11/21/19 **DATE COMPLETED** 11/21/19
GROUND ELEVATION 25.5 FT +/- **BORING DEPTH** 60 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.564387° **LONGITUDE** -122.321193°
DRILLING METHOD Mobile B-61, 8 inch Hollow-Stem Auger
GROUNDWATER LEVELS:
LOGGED BY BCG
▽ **AT TIME OF DRILLING** 30 ft.
NOTES B Street
▼ **AT END OF DRILLING** 32 ft.

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
25.5	0	8 inches asphalt concrete over 3 inches aggregate base							
24.8									
24.6		**Sandy Lean Clay (CL) [Fill]** dry, reddish brown, fine to medium sand, low plasticity		GB					
				GB-2		7			
				GB					
20.5	5	**Clayey Sand with Gravel (SC)** very dense, moist, brown to reddish brown, fine to coarse sand, fine to coarse subrounded to subangular gravel	50/5"	MC-4B	115	11			
17.5		**Poorly Graded Sand with Clay and Gravel (SP-SC)** dense, moist, brown, fine to coarse sand, fine to coarse subrounded to angular gravel	66	MC-5B	128	5			
	10								
		becomes very dense	50/6"	MC-6B	111	8			
	15								
		becomes dense	73	MC-7B	125	8			
	20								
3.5		**Sandy Lean Clay with Gravel (CL)** very stiff to hard, moist, brown, fine to coarse sand, fine to coarse subrounded to subangular gravel, low plasticity							
			51	SPT					○
	25								
			50/6"	MC-9B	118	16			>4.5 ○
-1.5									
-2.5									

Continued Next Page

BORING NUMBER EB-1

PAGE 2 OF 2

PROJECT NAME 222 East 4th Avenue
PROJECT NUMBER 129-6-1
PROJECT LOCATION San Mateo, CA

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT, %	PLASTICITY INDEX	PERCENT PASSING No. 200 SIEVE
-2.5			**Lean Clay with Sand (CL)** very stiff, moist, gray with brown mottles, fine to medium sand, trace gravel, low to moderate plasticity						
				75	MC-10B	108	21		
	▽ 30								
-5.5			**Sandy Lean Clay with Gravel (CL)** hard, moist, brown, fine to coarse sand, fine to coarse subangular to angular gravel, low to moderate plasticity						
	▼								
				50 6"	MC-11B	129	16		
	35								
-11.5			**Sandy Lean Clay (CL)** very stiff, moist, brown and gray mottled, fine to medium sand, some gravel, low plasticity						
				52	MC-12B	112	19		
	40								
				50 6"	MC-13B	105	23		
	45								
-21.5			**Sandy Lean Clay with Gravel (CL)** very stiff, moist, brown, fine to coarse sand, fine to coarse subangular to angular gravel, low plasticity						
				54	SPT-14		19		
	50								
-26.5			**Clayey Sand with Gravel (SC)** very dense, moist, brown, fine to coarse sand, fine to coarse subangular to angular gravel						
				74	SPT-15		15		
	55								
			becomes dense	46	SPT-16		14		
-34.5	60		Bottom of Boring at 60.0 feet.