Appendix F

Geotechnical Investigation

ROCKRIDGE
GEOTECHNICAL

Prepared for **Aldersly, Inc.**

GEOTECHNICAL INVESTIGATION
PROPOSED PHASE 1A & 1B RESIDENTIAL BUILDINGS
ALDERSLY RETIREMENT COMMUNITY
326 MISSION AVENUE
SAN RAFAEL, CALIFORNIA

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August 31, 2020
Project No. 19-1779

ROCKRIDGE
GEOTECHNICAL

August 31, 2020
Project No. 19-1779

Mr. Peter Schakow
Aldersly, Inc.
326 Mission Ave.
San Rafael, CA 94901

Subject: Geotechnical Investigation Report
Proposed Phase 1A & 1B Residential Buildings
Aldersly Retirement Community
326 Mission Avenue
San Rafael, California

Dear Mr. Schakow:

We are pleased to present the results of our geotechnical investigation for the proposed Phase 1A and 1B residential buildings to be constructed at the Aldersly Retirement Community, located at 326 Mission Avenue in San Rafael, California. Our services were provided in accordance with our proposal dated November 11, 2019.

The subject site is an existing retirement community located on the north side of Mission Avenue, between Union Street and Grand Avenue, and is situated along a hillside, sloping mildly down from north to south. The site is currently occupied by several existing buildings, landscaped areas, and two small asphalt-paved parking lots. The proposed Phase 1A and 1B buildings will be located at the south edge of the property.

Our understanding of the proposed project is based on review of the drawings titled *Conceptual Design Review Submittal*, prepared by Perkins-Eastman, the project architect, dated May 29, 2020. We also reviewed the existing site topographic information presented on the drawings titled *Aldersly Retirement Community, Topographic Map, 326 Mission Ave.*, prepared by CSW|ST2, dated October 5, 2017.

We understand Phase 1A will consist of demolishing the existing Graasten, Marselisborg, and Liselund buildings and constructing the proposed Mission Avenue Independent Living building (Mission IL building), which will connect to the southeast end of the existing Fredensborg building. The Mission IL building will consist of three stories at the south half and four stories at the north half of the building, with a lower finished floor at Elevation 16 feet. The new Mission IL building will house 35 residential units and include parking and administration space on the north half of the lower level. We understand Phase 1B will consist of demolishing the west wing of the existing Frederiksborg building and replacing it with a new two-story addition with a lower finished floor at Elevation 15 feet. The new Frederiksborg addition will house four new residential units and parking spaces on the southeast end of the lower level. Structural design loads were not available at the time this report was prepared.

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 are (1) the presence of highly expansive near-surface soil in portions of the site; (2) the potential for differential settlement due to the variability in depth to bedrock across the proposed building footprints; and (3) the potential for future groundwater seepage from the underlying bedrock or along the soil-bedrock contact. These and other geotechnical issues are discussed in more detail in the attached report.

We conclude the proposed buildings may be supported on a continuous perimeter and isolated spread footings bearing on undisturbed bedrock. In locations where the design footing depth does not extend to bedrock, the structural concrete may either be deepened to bedrock, or the footing may be over-excavated down to bedrock and replaced with CDF or lean concrete up to the design bottom-of-footing elevation.

Our report contains specific recommendations regarding earthwork and grading, foundation design, excavation shoring, 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, 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.

LOGAN D. MEDEIROS
2957
12/31/21

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

Enclosure

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GEOTECHNICAL INVESTIGATION
PROPOSED PHASE 1A & 1B RESIDENTIAL BUILDINGS
ALDERSLY RETIREMENT COMMUNITY
326 Mission Avenue
San Rafael, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Rockridge Geotechnical for the proposed Phase 1A and 1B residential buildings to be constructed at the Aldersly Retirement Community, located at 326 Mission Avenue in San Rafael, California. The subject site is an existing retirement community located on the north side of Mission Avenue, between Union Street and Grand Avenue, as shown on the Site Location Map (Figure 1).

The subject property is located along a hillside, sloping mildly down from north to south, and is currently occupied by several existing buildings, landscaped areas, and two small asphalt-paved parking lots. The proposed Phase 1A and 1B buildings will be located at the south edge of the property, as shown on the Site Plan (Figure 2).

Our understanding of the proposed project is based on review of the drawings titled *Conceptual Design Review Submittal*, prepared by Perkins-Eastman, the project architect, dated May 29, 2020. We also reviewed the existing site topographic information presented on the drawings titled *Aldersly Retirement Community, Topographic Map, 326 Mission Ave.*, prepared by CSW|ST2, dated October 5, 2017.

We understand Phase 1A will consist of demolishing the existing Graasten, Marselisborg, and Liselund buildings and constructing the proposed Mission Avenue Independent Living building (Mission IL building), which will connect to the southeast end of the existing Fredensborg building, as shown on the Site Plan (Figure 2). The Mission IL building will consist of three stories at the south half and four stories at the north half of the building, with a lower finished floor at Elevation 16 feet[1]. Existing grades in the proximity of the proposed Mission IL building range from roughly Elevation 16 feet near the southwest corner to Elevation 20 feet near the northeast corner to Elevation 32 feet near the northwest corner. The new Mission IL building will house 35 residential units and include parking and administration space on the north half of the lower level.

We understand Phase 1B will consist of demolishing the west wing of the existing Frederiksborg building and replacing it with a new two-story addition with a lower finished floor at Elevation 15 feet, as shown on the Site Plan (Figure 2). Existing grades in the proximity of the proposed Frederiksborg addition range from roughly Elevation 15 feet near the south corner to Elevation 26 feet near the north corner. The new Frederiksborg addition will house four new residential units and parking spaces on the southeast end of the lower level.

Structural design loads were not available at the time this report was prepared.

2.0 SCOPE OF SERVICES

Our geotechnical investigation was performed in accordance with our proposal, dated November 11, 2019. Our scope of work consisted of evaluating subsurface conditions at the site by drilling six exploratory borings, performing laboratory testing on select soil samples, and performing engineering analyses to develop conclusions and recommendations regarding:

- site seismicity and seismic hazards, including the potential for liquefaction and liquefaction-induced ground failure
- the most appropriate foundation type(s) for the proposed new buildings
- design criteria for the recommended foundation type(s), including vertical and lateral capacities
- estimates of foundation settlement

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[1] Elevation Datum: NAVD 88 – Per *Aldersly Retirement Community, Topographic Map, 326 Mission Ave.*, prepared by CSW|ST2, dated October 5, 2017

- subgrade preparation for slab-on-grade floors and exterior concrete flatwork
- site grading and excavation, including criteria for fill quality and compaction
- lateral earth pressures for the design of permanent below-grade walls and temporary excavation shoring systems
- corrosivity of the near-surface soil
- 2019 California Building Code (CBC) site class and mapped design spectral response acceleration parameters
- construction considerations.

3.0 FIELD INVESTIGATION

Subsurface conditions at the site were investigated by drilling six test borings and performing laboratory testing on select soil samples. Prior to our field investigation, we contacted Underground Service Alert (USA) to notify them of our work, as required by law, and retained Precision Locating, LLC, a private utility locator, to check that the boring locations were clear of existing underground utilities. We also obtained a drilling permit from the Marin County Environmental Health Services Division. Upon completion, the test borings were backfilled with cement grout in accordance with County requirements. Details of the field investigation and laboratory testing are described below.

3.1 Test Borings

The subsurface conditions were explored during our field investigation by drilling six test borings. The borings were drilled on February 24 and 25, 2020 by Benevent Building of Concord, California. The borings, designated B-1 through B-6, were drilled to depths ranging from 6-1/4 to 17-1/2 feet bgs, where they met practical refusal in bedrock, using a portable hydraulic drill rig equipped with four-inch-diameter solid-stem augers. During drilling, our field engineer logged the soil and rock encountered and obtained representative samples for visual classification and laboratory testing. The logs of the borings are presented on Figures A-1 through A-6 in Appendix A. The soil and rock encountered in the borings was classified in accordance with the classification charts shown on Figures A-7 and A-8, respectively.

Soil samples were obtained using the following samplers:

- Sprague and Henwood (S&H) split-barrel sampler with a 3.0-inch outside diameter and 2.5-inch inside diameter, lined with 2.43-inch inside diameter tubes
- California (CA) split-barrel sampler with a 2.5-inch outside diameter and 2.0-inch inside diameter, lined with 1.875-inch inside diameter tubes
- Standard Penetration Test (SPT) split-barrel sampler with a 2.0-inch outside and 1.5-inch inside diameter, without liners.

The type of sampler used was selected based on soil type and the desired sample quality for laboratory testing. In general, the S&H sampler was used to obtain samples in medium stiff to very stiff cohesive soil and the Standard California and SPT sampler was used to evaluate the relative density of sandy soil and to sample hard clays and bedrock. The samplers were driven with a 140-pound, rope and cathead 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 S&H, CA, and SPT samplers were converted to approximate SPT N-values using factors of 0.7, 0.9, and 1.2, respectively, to account for sampler type, approximate hammer energy, and the fact that the SPT sampler was used without liners. 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.

Upon completion of drilling the boreholes were backfilled with cement grout. The soil cuttings were spread on the ground surface in landscape areas.

3.2 Laboratory Testing

We re-examined each soil sample obtained from our borings to confirm the field classifications and select representative samples for laboratory testing. Soil samples were tested to measure moisture content, dry density, Atterberg limits (plasticity index), particle size distribution, and corrosivity. The Atterberg limits test is an indirect measurement of the expansion potential of soil. The results of the geotechnical laboratory tests are presented on the boring logs and in Appendix B. The results of the corrosivity analyses are also presented in Appendix B.

4.0 SUBSURFACE CONDITIONS

The geology map of Marin County indicates the site is underlain by Holocene-aged alluvium (Qha) and Franciscan Complex Melange (fsr), as shown on the Regional Geologic Map (Figure 3). The results of our borings indicate the site is underlain by stiff to hard clay with varying amounts of sand and gravel and medium dense to very dense clayey sand with varying amounts of gravel, which is underlain by bedrock. The bedrock consists of interbedded sedimentary rock with varying degrees of weathering, fracturing, and hardness, including claystone, mudstone, and sandstone. The top-of-bedrock elevation varies substantially throughout the site. Within the footprint of the proposed Mission IL building, the top of weathered bedrock was encountered in our borings at depths ranging from about 2-1/2 feet to 6 feet below existing grades (bgs)—these depths correspond to approximately Elevation 10 to 23-1/2 feet. In the vicinity of the proposed Frederiksborg addition, the top of weathered bedrock was encountered in our borings at a depth of about 7 bgs, which corresponds to approximate Elevations 8 to 18-1/2 feet.

4.1 Groundwater

During our field investigation, groundwater was not encountered in our test borings, which were drilled on February 24 and 25, 2020. Due to the sloping topography of the site and surrounding area and the presence of shallow bedrock, we expect there is potential for groundwater to impact the proposed buildings as a result of complex surface drainage and subsurface seepage through fractures in bedrock and/or seepage along the soil-bedrock interface, especially during or following periods of heavy rainfall. We reviewed the geotechnical report prepared by Earth Science Consultants (ESC), dated March 5, 1999, for the existing Rosenborg building, located immediately north and upslope of the proposed Mission IL building. ESC did not encounter free water in their exploratory borings during drilling. ESC's borings were relatively shallow as a result of the shallow bedrock. However, in their report, ESC described observed evidence of seepage from cracks in former asphalt pavements prior to construction of the Rosenborg building. These observations are consistent with our experiences on other sloping sites with shallow bedrock throughout the Bay Area.

5.0 SEISMIC CONSIDERATIONS

The San Francisco Bay Area is considered to be one of the most seismically active regions in the world. We evaluated the potential for earthquake-induced geologic hazards including ground shaking, ground surface rupture, liquefaction,[2] lateral spreading,[3] and cyclic densification[4]. The results of our evaluation regarding seismic considerations for the project site are presented in the following sections.

5.1 Regional Seismicity and Faulting

The site is located in the Coast Ranges geomorphic province of California that is characterized by northwest-trending valleys and ridges. These topographic features are controlled by folds and faults that resulted from the collision of the Farallon plate and North American plate and

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

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

subsequent strike-slip faulting along the San Andreas fault system. The San Andreas fault is more than 600 miles long from Point Arena in the north to the Gulf of California in the south. The Coast Ranges province is bounded on the east by the Great Valley and on the west by the Pacific Ocean.

The major active faults in the area are the Hayward and San Andreas faults. These and other faults in the region are shown on the regional Fault Map (Figure 3). 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[6] [Petersen et al. (2014) & Thompson et al. (2016)] are summarized in Table 1. These references are based on the Third Uniform California Earthquake Rupture Forecast (UCERF3), prepared by Field et al. (2013).

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

[6] Moment magnitude (M_w) is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

TABLE 1
Regional Faults and Seismicity

Fault Segment	Approximate Distance from Site (km)	Direction from Site	Characteristic Moment Magnitude
Total Hayward + Rodgers Creek (RC+HN+HS+HE)	13	Northeast	7.58
Hayward (North, HN)	13	Northeast	6.90
Total North San Andreas (SAO+SAN+SAP+SAS)	16	West	8.04
North San Andreas (North Coast, SAN)	16	West	7.52
North San Andreas (Peninsula, SAP)	16	West	7.38
San Gregorio (North)	17	West	7.44
Rodgers Creek - Healdsburg	23	Northeast	7.19
West Napa	32	Northeast	6.97
Hayward (South, HS)	37	Southeast	7.00
Green Valley	38	East	6.30
Concord	38	East	6.45
Mount Diablo Thrust North CFM	42	East	6.72
Mount Diablo Thrust	43	East	6.67
Total Calaveras (CN+CC+CS+CE)	44	East	7.43
Calaveras (North, CN)	44	East	6.86
Clayton	49	East	6.57

Since 1800, four major earthquakes have been recorded on the 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 118 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 (probably an M_w of about 6.5) was reported on the Calaveras Fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill earthquake ($M_w = 6.2$).

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

5.2 Geologic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the project site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction, lateral spreading, cyclic densification, and landsliding. We used the results of the borings to evaluate the potential of these phenomena occurring at the project site. The results of our analyses and evaluation are presented in the following sections.

5.2.1 Ground Shaking

The ground shaking intensity felt at the project site will depend on: 1) the size of the earthquake (magnitude), 2) the distance from the site to the fault source, 3) the directivity (focusing of earthquake energy along the fault in the direction of the rupture), and 4) site-specific soil conditions. The site is less than 20 kilometers from three major faults. Therefore, the potential exists for a large earthquake to induce strong to very strong ground shaking at the site during the life of the project.

5.2.2 Liquefaction and Associated Hazards

Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction and lateral spreading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, low-plasticity silt, and some low-plasticity clay deposits. Flow failure, lateral spreading, differential settlement, loss of bearing strength, ground fissures and sand boils are evidence of excess pore pressure generation and liquefaction. As presented on the Liquefaction Susceptibility Map (Figure 5), the subject site is located on the margin of two zones designated “moderate” and “very low” liquefaction susceptibility (USGS 2006).

The results of our exploratory borings indicate bedrock is present 2-1/2 to 7 feet below existing grades, where explored. Bedrock is not susceptible to liquefaction. Furthermore, the soil present above the bedrock, much of which will be removed during excavation for the proposed buildings, was found to have sufficient cohesion to resist liquefaction. Therefore, we conclude the potential for liquefaction and associated hazards, such as lateral spreading, are very low.

5.2.3 Cyclic Densification

Cyclic densification (also referred to as differential compaction) of non-saturated sand (sand above groundwater table) can occur during an earthquake, resulting in settlement of the ground surface and overlying improvements. The soil present above the groundwater (and bedrock), much of which will be removed during excavation for the proposed buildings, was found to have sufficient cohesion to resist cyclic densification. Therefore, we conclude the potential for cyclic densification to occur at the site is very low.

5.2.4 Fault Rupture

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

5.2.5 Landsliding

The subject site slopes up to the north-northeast at an average gradient of less than 20 percent from Mission Avenue to Belle Avenue. According to the Marin GeoHub digital database, the site is mapped in an area designated Landslide Category 3. The site is not located near an existing mapped landslide area on the map titled *Summary Distribution of Slides and Earth Flows in Marin County, California*, (Wentworth et al, 1997). In addition, the site is not located near a mapped potential debris-flow hazard area on the map titled *Map Showing Principal Debris-Flow Source Areas in Marin County, California*, (Ellen et al, 1997).

Based on review of available geologic maps and literature and our observations during a site reconnaissance, we conclude the risk of large scale landsliding at the site is low. Furthermore, provided the proposed project is designed and constructed in accordance with our recommendations for site drainage, lateral earth pressures for temporary shoring and permanent below-grade walls, and foundation design, we conclude the risk of localized slope instability at the site is also low.

6.0 DISCUSSIONS AND CONCLUSIONS

Based on the results of our engineering analyses using the data from our exploratory borings, we conclude there are no major geotechnical or geological issues that would preclude development of the site as proposed. The primary geotechnical concerns are (1) the presence of highly expansive near-surface soil in portions of the site; (2) the potential for differential settlement due to the variability in depth to bedrock across the proposed building footprints; and (3) the potential for future groundwater seepage from the underlying bedrock or along the soil-bedrock contact. These and other issues are discussed in the following sections.

6.1 Foundations and Settlement

The proposed Mission IL building will have a finished floor at Elevation 16 feet. In borings B-1 through B-4, which were drilled within the proposed building footprint, we encountered bedrock at elevations ranging from approximately 10 to 23-1/2 feet. We preliminarily estimate new foundations, if conventional spread footings are used, will be bottomed at roughly Elevation 13 to 14 feet. Therefore, many of the foundations will be bottomed in bedrock, which is suitable for support of spread footings with minimal settlement; however, in the southeast portion of the building (near boring B-1), the footings will be underlain by 3 to 4 feet of soil. To mitigate concerns for potential differential settlement across the soil-bedrock transition, we conclude all foundations for the Mission IL building should be bottomed in competent, undisturbed bedrock. This can be achieved by deepening the structural footings or, alternatively, footing excavations may be over-excavated down to competent bedrock and backfilled with controlled density fill (CDF) or sand-cement slurry up to the design bottom-of-footing elevation. The CDF would serve to transfer footing loads to the bedrock and prevent the need for extending reinforced structural concrete down to rock. As noted above, based on the results of our borings, much of the building pad excavation will likely expose bedrock, based on the currently planned finished floor at Elevation 16 feet. Structural loads for the building are not currently available, however, we anticipate total and differential settlements of footings bearing on bedrock, some of which is deeply weathered, will be less than 1/2 and 1/4 inch over a horizontal distance of 30 feet, respectively.

The proposed Frederiksborg building addition will have a finished floor at Elevation 15 feet. In borings B-5 and B-6, which were drilled in the vicinity of the proposed building addition footprint, we encountered bedrock at approximate Elevations 8.1 and 18-1/2 feet, respectively. We preliminarily estimate new foundations, if conventional spread footings are used, will be bottomed at roughly Elevation 12 to 13 feet. Therefore, the foundations for the north portion of the building will likely be bottomed in bedrock, which is suitable for support of spread footings with minimal settlement; however, the footings in the south portion of the building may be underlain by as much as several feet of soil. To mitigate concerns for potential differential settlement across the soil-bedrock transition, we conclude all foundations for the Frederiksborg building addition should be bottomed in competent, undisturbed bedrock, using the same options described above for the Mission IL building. Structural loads for the building are not currently available, however, we anticipate total and differential settlements of footings bearing on bedrock, some of which is deeply weathered, will be less than 1/2 and 1/4 inch over a horizontal distance of 30 feet, respectively.

Due to the presence of highly expansive soil and potentially expansive claystone bedrock beneath portions of the buildings (discussed in more detail below), we conclude the foundation system for each building should include a continuous perimeter footing, which will help control large seasonal fluctuations in moisture content beneath the slab-on-grade. Recommendations for the design of spread footings are presented in Section 7.2.

6.2 Expansive Soil

Atterberg limits tests performed on two samples of the near-surface clay indicate the clay has low to high expansion potential. It is anticipated that the weathered claystone bedrock, where encountered, may also have moderate expansion potential. The expansive near-surface clay is subject to volume changes during seasonal fluctuations in moisture content. These volume changes can cause cracking of foundations, slabs, and below-grade walls. Therefore, foundations, slabs, and below-grade walls should be designed and constructed to resist the effects of the expansive clay. In general, the effects of expansive soil can be mitigated by moisture-conditioning the expansive soil, providing select, non-expansive fill or lime-treated soil below interior and exterior slabs and behind retaining walls, and either supporting foundations below the zone of severe moisture change or by providing a stiff, shallow foundation that can limit deformation of the superstructure as the underlying soil shrinks and swells.

We anticipate the expansive near-surface clay will be removed during excavation for portions of the proposed building pads, however, due to the high variability of soil/rock composition and depth to bedrock relative to the proposed pad elevations, we conclude portions of the proposed building slabs will be underlain by expansive soil and/or weathered rock. Therefore, we recommend the proposed building foundations include a continuous, deepened perimeter to reduce the potential for extreme seasonal moisture change beneath the foundations and slab-on-grade floors.

To prevent the soil subgrade beneath the building slabs from drying during construction and to reduce the long-term effects of expansive subgrade soil, a minimum of 12 inches of select, non-expansive fill should be placed on the prepared subgrade. We conclude the proposed underslab drainage layer, discussed in more detail in Sections 6.3.2 and 7.3, may serve as the select fill layer, provided it is at least 12 inches thick.

In addition, on expansive soil sites it is critical to properly manage surface and subsurface drainage to prevent water from collecting beneath pavements and slabs or behind below-grade walls, where it can lead to cyclic swelling and shrinking of the subgrade soil and can cause subgrade instability under vehicular loads. At this time, we are not aware of the pavement types that are planned for the site. However, we anticipate that permeable pavements may be considered for the proposed project. Furthermore, storm water collection and/or treatment systems (bio-swales, infiltration basins, rain gardens, etc.) are increasingly common design features on projects throughout the Bay Area. While the objective of permeable pavement systems and infiltration basins is to allow for water storage and infiltration, we conclude that infiltration into the subgrade soil is not feasible at this site due to the low permeability of the moderately to expansive clay soil and bedrock. Furthermore, from a geotechnical standpoint, water should not be allowed to collect alongside or beneath the building foundations, pavements and concrete flatwork. This can be achieved by providing subdrain systems beneath permeable surfaces and installing vertical barriers between permeable surfaces underlain by subdrains and non-permeable surfaces underlain by conventional aggregate base. In addition, to prevent the subgrade soil from becoming saturated, we conclude that permeable aggregate base courses should be underlain by an impermeable liner in zones subject to regular vehicular traffic.

6.3 Groundwater, Drainage, and Dewatering

As discussed in Section 4.1, groundwater was not encountered in our borings during drilling. However, considering the site topography, the past documentation of observed seepage through the pavement at the site (ESC, 1999), and our experience with similar shallow-bedrock sites in the Bay Area, we conclude there is a high potential for periodic shallow perched water along the soil-bedrock interface and/or seepage through fractures in the bedrock beneath the proposed buildings in the future, especially following periods of heavy rainfall. Therefore, we conclude the proposed development should be designed to manage surface and subsurface drainage, in order to reduce the potential for shallow subsurface water adversely impacting the performance of the foundations, slabs, and below-grade walls. Temporary dewatering during construction, as well as permanent underslab and wall drainage systems, are discussed in more detail in the following sections.

6.3.1 Temporary Dewatering

If construction is performed during the dry season, it is unlikely substantial excavation dewatering will be required, based on the results of our exploratory borings. If the work is performed during the rainy season, there is potential for encountering water within excavations or seepage from vertical cuts or shoring walls as a result of the sloping terrain and shallow bedrock. In most cases, we anticipate groundwater seepage, if any, would have a relatively low flow rate. However, potential fractures in the bedrock may produce a significant amount of water in isolated areas. Flow of groundwater into the excavations during construction could result in sloughing, slumping, or caving of the sides of the excavation and/or wet, difficult working conditions. Therefore, we anticipate it may be necessary to temporarily dewater excavations and/or install water management features, such as temporary sumps or interceptor trenches, during wet weather conditions.

Temporary dewatering is typically performed by installing a series of wells around the perimeter of the building, with interior wells also used for larger building footprints. However, based on the results of our investigation, we conclude the effectiveness of temporary dewatering wells will be limited due to the relatively low permeability of the soil and bedrock encountered at the site. Therefore, we believe a passive system, in which water is collected from the perimeter of the site using gravel-filled trenches, will be more appropriate, if needed. Once it is in place and functional, the permanent underslab drainage system (discussed in more detail in the following section) can be used to manage water in the excavation during construction.

The dewatering system, if needed, should be designed and installed by an experienced contractor. Water removed during dewatering should be disposed of in accordance with applicable state and local regulations.

6.3.2 Permanent Drainage

Long-term management of water should include installation of subdrain systems beneath the building slabs and behind the below-grade walls, as well as systems to collect and manage surface water around the proposed buildings and any site retaining walls. The underslab drainage systems for the two buildings should consist of a continuous 12-inch-thick layer of drain rock containing a network of collection pipes connected to suitable outlets outside the building footprints, which will mitigate potential the build-up of water beneath the floor slabs and reduce the potential for water vapor intrusion in conjunction with underslab vapor retarders.

In addition to the underslab drainage systems, wall backdrains should be installed along the perimeter below-grade walls to reduce the potential for hydrostatic pressures on the walls and to reduce the amount of water flowing to the underslab drainage system—the two systems should be piped and discharged separately.

6.4 Excavation Support

We estimate the proposed excavation for the Mission IL building may extend as much as roughly 18 feet below existing grades (including excavation for the foundation) near the northwest corner of the building Mission IL building.

In some locations, the sides of the excavation may be cut at temporary slopes and the walls subsequently backfilled following construction of the below-grade building walls; however, we anticipate there will be insufficient clearance to slope cut portions of the excavation due to the presence of nearby existing buildings and site retaining walls. Temporary excavation slopes should be no steeper than ¾:1 (horizontal:vertical) in weathered bedrock or residual soil (OSHA

Type A soil), 1:1 in stiff to very stiff cohesive soil (OSHA Type B soil), and 1.5:1 where seepage is observed in the cut during construction (OSHA Type C soil). Vertical cuts as high as four feet in stiff to very stiff cohesive soil and five feet in weathered bedrock may be used unless seepage is observed. Higher vertical cuts in hard bedrock may be feasible; however, the potential for adverse bedding should be checked by a registered geologist before the cuts are made. Slopes which exceed a vertical height of 15 feet should be checked individually for stability. The allowable cut criteria presented above assumes nearby building and retaining wall foundations are bearing below a 2:1 zone-of-influence extending up from the toe of excavation. Allowable temporary cut inclinations should be further evaluated based on the time of year excavation is performed and the soil/rock/groundwater conditions observed by our engineer in the field.

In locations where there is insufficient space to slope the excavation due to the presence of property lines, adjacent streets, sidewalks, critical underground utilities, or existing structures, a temporary shoring system will be required. There are several key considerations in selecting a suitable shoring system for this project. Those we consider of primary concern are:

- protection of surrounding improvements, including nearby buildings and retaining walls,
- proper construction of the shoring system to reduce potential for ground movement,
- shallow bedrock of varying hardness, some of which may require specialized drilling equipment
- cost.

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

- soil nails,
- soldier pile-and-lagging with tiebacks, and
- cantilevered soldier pile-and-lagging.

Soil nail shoring systems consist of reinforcing bars, which are grouted in predrilled holes through the face of the excavation, and a reinforced shotcrete facing. Soil nail systems require a certain amount of ground movement to mobilize their lateral resistance, and therefore are only appropriate in locations where the excavation is not immediately adjacent to existing structures or critical underground utilities. In addition, where the excavation is close to the property line and there is insufficient setback, soil nails may need to extend beneath the neighboring property, which would require an encroachment agreement with neighboring property owners.

Soldier pile-and-lagging shoring systems usually consist of steel H-beams and concrete placed in predrilled holes extending below the bottom of the excavation. Wood lagging is placed between the piles as the excavation proceeds from the top down, in maximum five-foot-thick lifts. Where the required total cut is less than about 12 to 14 feet, a soldier pile-and-lagging system can typically provide economical shoring without tiebacks, and therefore will not encroach beyond the property line. Where cuts exceed about 12 to 14 feet in height, soldier pile-and-lagging systems are typically more economical if they include tieback anchors. Tiebacks consist of post-tensioned steel strands or bars that are grouted into predrilled holes through the excavation face. Generally, tiebacks are installed in conjunction with a soldier pile-and-lagging system. However, tieback anchors will likely extend beneath the neighboring property, if installed along the north and west edges of the proposed Frederiksborg building addition. Where there is insufficient property line set-back to accommodate soil nails or tiebacks, and an encroachment agreement is not possible, internal bracing or a very stiff cantilever may be required. Another alternative is to construct a cantilevered shoring system combined with partial slope cuts, in order to reduce the vertical retained height.

Recommendations for the design and construction of both soil nails and soldier pile-and-lagging shoring are presented in Section 7.6.

6.5 Excavation in Rock

Excavations in rock will be required for construction of the below-grade portions of the buildings. Where encountered in our borings, the bedrock is generally friable to weak and highly weathered. Note that corrected SPT N-values within the rock, where explored, ranged from about 16 blows for 12 inches to about 60 blows for 1/4 inch of penetration and therefore, the rock varies substantially in hardness and degree of weathering. We anticipate the weathered rock in the upper portion of the excavation can be excavated with conventional grading equipment (excavators and bull dozers); and harder rock at depth may require the use of hydraulic breaking equipment (i.e. a hoe ram). Furthermore, because the bedrock was characterized by discrete borings during our investigation, it is possible that harder rock and difficult drilling or excavation may be encountered. Therefore, the contractors involved in shoring installation and excavation for the building pads and foundations should be prepared to excavate hard rock, including the possible use of hydraulic breaking equipment, and should bid the project accordingly. The material descriptions and SPT N-values presented on the boring logs should be evaluated by the contractor when bidding the project and selecting appropriate equipment.

6.6 Soil Corrosivity

Laboratory testing was performed by Project X Corrosion Engineering to evaluate the corrosivity of the near-surface clay and claystone from Boring B-3 and B-5 at depths 4 and 2-1/2 feet bgs, respectively. The corrosivity test results are presented in Appendix B of this report.

The minimum resistivity test results (4355 ohm-cm and 4288 ohm-cm, respectively) indicate that the near-surface soil is “mildly corrosive” to buried metallic structures. The chloride ion concentration (2.4 mg/kg and 0.5 mg/kg) and pH (8.0 and 7.9) indicate “negligible” corrosivity effects to buried metallic structures and reinforcing steel in concrete structures below ground. The results also indicate the sulfate ion concentrations (5.0 mg/kg and 2.5 mg/kg) are sufficiently low such that sulfates do not pose a threat to buried concrete and mortars.

7.0 RECOMMENDATIONS

7.1 Site Preparation and Grading

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., buildings, 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.

In general, abandoned underground utilities should be removed to the property line or service connections and properly capped or plugged with concrete. Where existing utility lines are outside of the proposed building footprints and will not interfere with the proposed construction, they may be abandoned in-place provided the lines are filled with lean concrete or cement grout to the property line. Voids resulting from demolition activities should be properly backfilled with engineered fill following the recommendations provided later in this section and under the observation of our field engineer.

We anticipate the excavation will be well above the static regional groundwater level. However, if excavation and grading is performed during the wet season, the excavation subgrade may become wet, especially along the northern edge, where perched water may seep from the cut slopes and/or shoring along the soil-rock interface. Where the excavation subgrade is bottomed in rock, the surface should be reasonably stable. However, where it is bottomed in soil, the soil may be sensitive to disturbance, especially under construction equipment wheel loads. If soft areas are encountered in the building pad, subgrade stabilization measures may be required.

The building pads should be excavated to accommodate a 12-inch-thick underslab drainage layer, as discussed in Sections 6.2 and 6.3.2 and detailed in Section 7.3. The subgrade for proposed concrete flatwork should also be overexcavated to accommodate at least 6 inches of non-expansive soil, such as Caltrans Class 2 aggregate base (AB). In areas that will receive new pavements and exterior concrete flatwork, the soil subgrade exposed following stripping and clearing should be scarified to a depth of at least eight inches, moisture-conditioned, and compacted in accordance with the recommendations provided below in Table 2. Where undisturbed bedrock is exposed at subgrade elevation, scarification and re-compaction will not be required.

All fill should consist of soil that is free of organic matter and contain no rocks or lumps larger than three inches in greatest dimension. Rock from the proposed excavation may be re-used as fill if it is broken down smaller than 3 inches in greatest dimension and sufficiently blended with soil such that the fill does not contain substantial void spaces. All fill should be placed in horizontal lifts not exceeding eight inches in uncompacted thickness, moisture-conditioned, and compacted in accordance with the relative compaction[7] requirements presented in Table 2. Fill consisting of clean sand or gravel (defined as soil with less than 5 percent fines by weight) should be compacted to at least 95 percent relative compaction. Non-expansive fill greater than five feet in thickness or placed within the upper foot of pavement subgrade should also be compacted to at least 95 percent relative compaction, and be non-yielding.

A summary of the compaction requirements for the various types of fill that may be used at the site is presented in Table 2. If material to be used as fill is imported to the site, it should meet the requirements for select fill provided below in Section 7.1.1.

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

TABLE 2
Summary of Compaction Requirements

Location	Required Relative Compaction (percent)	Moisture Requirement
Building pads – expansive clay	88 – 92	3+% above optimum
Building pads – low-plasticity soil	90+	Above optimum
Exterior slabs – expansive clay	88 – 92	3+% above optimum
Exterior slabs – low-plasticity soil	90+	Above optimum
Vehicular Pavements – expansive clay	92+	2+% above optimum
Vehicular Pavements – low-plasticity soil	95+	Above optimum
Vehicular Pavements - aggregate base	95+	Near optimum
General fill – expansive clay	88 – 92	3+% above optimum
General fill – low-plasticity soil	90+	Above optimum
General fill – granular soil	95+	Near optimum
Utility trench backfill – expansive clay	88 – 92	3+% above optimum
Utility trench backfill – low-plasticity soil	90+	Above optimum
Utility trench - clean sand or gravel	95+	Near optimum

7.1.1 Select Fill

Select fill should consist of 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 in accordance with the compaction requirements presented in Table 2. 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.

7.1.2 Exterior Flatwork Subgrade Preparation

We recommend a minimum of six inches of select, non-expansive soil beneath proposed exterior concrete flatwork. The select fill may consist of Class 2 aggregate base (AB). Select fill beneath exterior slabs-on-grade should be moisture-conditioned and compacted in accordance with the requirements provided above in Table 2.

Even with 6 inches of non-expansive soil, exterior slabs may experience some cracking due to shrinking and swelling of the underlying expansive soil present in portions of the site. Thickening the slab edges and adding additional reinforcement will control this cracking to some degree. Where slabs are adjacent to landscaped areas, thickening the concrete edge will help control water infiltration beneath the slabs. In addition, where slabs provide access to building, it would be prudent to dowel the entrance to the building to permit rotation of the slab as the exterior ground shrinks and swells and to prevent a vertical offset at the entries.

Where concrete flatwork, vertical curbs, or curb-and-gutters will be constructed adjacent to stormwater treatment facilities, such as bio-swales, flow-through planters, or bio-retention basins, or any other landscaped areas in which a significant thickness of loose, uncompacted soil will be present, the edge of the concrete flatwork should be thickened to prevent long-term settlement and subsequent cracking of the concrete. We should be consulted during final design to provide specific recommendations, on a case-by-case basis, where such conditions occur. In general, the bottom of the concrete should extend below an imaginary plane extending up from the bottom of the loose soil at an inclination of 2:1 (horizontal:vertical).

7.1.3 Utility Trench Backfill

All trenches should conform to the current CAL-OSHA requirements. To provide uniform support, pipes or conduits should be bedded on a minimum of four inches of clean sand or fine gravel. After the pipes and conduits are tested, inspected (if required) and approved, they should be covered to a depth of six inches with clean sand or fine gravel, which should be mechanically tamped.

Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted as according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 5 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

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

7.1.4 Site Drainage and Landscaping

Positive surface drainage should be provided around the buildings to direct surface water away from the 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 buildings slope down away from the buildings with a surface gradient of at least two percent in unpaved areas and one percent in paved areas. In addition, roof downspouts should be discharged into controlled drainage facilities to keep the water away from the foundations. The use of water-intensive landscaping around the perimeter of the buildings should be avoided to reduce the amount of water introduced into the expansive clay. Unpaved areas should be planted with vegetation to prevent surficial erosion. Permanent slopes should have a maximum inclination of 2:1 (horizontal:vertical), although, highly expansive soils should have a maximum inclination of 3:1 to reduce the potential for long-term slope creep.

Bioswales or similar stormwater treatment features constructed at the site should include a minimum 12-inch-thick layer of Caltrans Class 2 Permeable rock below the treatment soil and include a subdrain due to the low permeability of the near-surface soil and bedrock. Bioswales should be constructed no closer than five feet from the buildings. If bioswales must be located within five feet of building foundations, they should be lined with impermeable liners below the collector drains and rock layer.

Below-grade building walls and site retaining walls should be well-drained in accordance with the recommendations presented in Section 7.5. Surface drainage from slopes above the walls should be collected and removed from the area in controlled systems, such as concrete-lined v-ditches, immediately behind the proposed below-grade building walls and site retaining walls. The below-grade wall drain systems should not be relied upon for collecting and removing surface run-off from the above slopes.

7.2 Foundation Design

We recommend the proposed buildings be supported on continuous perimeter footings and isolated interior footings bearing on firm, undisturbed bedrock. In locations where the design footing depth does not extend to bedrock, the structural concrete may either be deepened to bedrock, or the footing may be over-excavated down to bedrock and replaced with CDF or lean concrete up to the design bottom-of-footing elevation. Continuous footings should be at least 18 inches wide and isolated spread footings should be at least 24 inches wide. Interior footings should be founded at least 18 inches below the lowest adjacent soil subgrade and continuous perimeter footings should be founded at least 24 inches below the outside grade. Footings bearing on undisturbed weathered bedrock may be designed using allowable bearing pressures of 5,000 pounds per square foot (psf) for dead-plus-live loads and 6,600 psf for total design loads, which include wind or seismic forces; these values include factors of safety of at least 2.0 and 1.5, respectively.

Lateral loads may be resisted by a combination of passive pressure on the vertical faces of the footings and friction between the bottoms of the footings and the supporting CDF and/or bedrock. To compute passive resistance for sustained loading, we preliminarily recommend using an equivalent fluid weight (triangular distribution) of 350 pounds per cubic foot (pcf). For transient loads, including wind and seismic, we preliminarily recommend using an allowable passive pressure of 2,000 psf (uniform distribution). The upper foot of soil should be ignored unless confined by a slab or pavement. During final design, we can re-evaluate the recommended allowable passive pressures, on a case-by-case basis, once the building cross sections and footing locations have been determined, as some footings may be designed for higher values, where they are embedded in competent bedrock. Frictional resistance should be computed using an allowable base friction coefficient of 0.30. The passive pressure and frictional resistance values include a factor of safety of at least 1.5 and may be used in combination without reduction.

Footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. If footings are excavated during the rainy season, the footing concrete should be placed in a timely manner, following inspection by our field engineer, to prevent ponding water from disturbing and softening the highly weathered bedrock. Alternatively, the footings may be over-excavated by about 2 to 3 inches to allow for placement of a protective mudslab consisting of lean concrete or sand-cement slurry (following inspection by our engineer). A mud slab will help protect the footing subgrade from ponding water during placement of reinforcing steel. Water can then be pumped from the excavations prior to placement of structural concrete, if present. The bottoms and sides of the footing excavations should be moistened following excavation and maintained in a moist condition until concrete is placed. If expansive bedrock dries during construction, the footing will eventually heave, which may result in cracking and distress. We should check footing excavations prior to placement of the mud slab and/or reinforcing steel.

7.3 Underslab Drainage Systems

The proposed building slabs should be underlain by permanent underslab drainage systems to provide a controlled outlet for potential seepage from the underlying bedrock and water that may flow along the soil-bedrock interface during the wet season to prevent the build-up of hydrostatic pressures beneath the slabs. We recommend the permanent underslab drainage system consist of a 12-inch-thick layer of drain rock containing a network of perforated collection pipes spaced approximately 15 feet on center. The drain rock layer should meet the gradation requirements presented below in Table 3 and be underlain by filter fabric (Mirafi 140N, or equivalent). The collection pipes should consist of four-inch-diameter, Schedule 40, perforated PVC pipes (perforations oriented downward). The pipes should be installed such that they are surrounded on all sides by at least four inches of rock. The perforated collection pipes should be connected to solid pipes, where they cross the building edges, which should be designed to transport the water to a suitable outlet, such as the storm drain system or a stormwater treatment feature located at least 10 feet from the buildings. Outside the building footprints, the pipes should drain at a gradient of at least one percent. Cleanouts should be provided to ensure the underslab drainage system can be cleared if it becomes clogged.

TABLE 3

Gradation Requirements for Underslab Drainage rock

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

7.4 Concrete Slab-on-Grade Floor

The floor slabs for the proposed buildings may consist of conventional slabs-on-grade. Where water vapor transmission through the floor slabs is undesirable, we recommend installing a water vapor retarder and capillary moisture break beneath the floor slabs. A vapor retarder is generally not required beneath parking garage floor slabs because there is sufficient air circulation to allow evaporation of moisture that is transmitted through the slab; however, we recommend the vapor retarder be installed below the slabs-on-grade beneath living spaces, utility rooms, and any areas that will be used for storage and/or will receive floor coverings or coatings.

A capillary moisture break consists of at least four inches of clean, free-draining gravel or crushed rock—the recommended rock layer described above for the underslab drainage system meets the requirements for a capillary moisture break. The vapor retarder should meet the requirements for Class A vapor retarders stated in ASTM E1745. The vapor retarder should be placed in accordance with the requirements of ASTM E1643. These requirements include overlapping seams by six inches, taping seams, and sealing penetrations in the vapor retarder.

The concrete slabs should be properly cured. Concrete mixes with high water/cement (w/c) ratios result in excess water in the concrete, which increases the cure time and results in excessive vapor transmission through the slab. Therefore, concrete for the slabs should have a low w/c ratio - less than 0.45. Water should not be added to the concrete mix in the field. If necessary, workability should be increased by adding plasticizers. Before floor coverings are placed on the slab-on-grade floors, the contractor should check that the concrete surface and the moisture emission levels (if emission testing is required) meet the manufacturer's requirements.

7.5 Permanent Below-Grade Walls and Site Retaining Walls

Permanent below-grade walls should be designed to resist static lateral earth pressures, lateral pressures caused by earthquakes, traffic loads (if vehicular traffic is expected within 10 feet of the wall), and foundation loads from adjacent buildings, where applicable. For preliminary planning, we recommend the permanent below-grade walls be designed for the more critical of the criteria:

- At-rest equivalent fluid weight of 60 pounds per cubic foot (pcf), or
- Active equivalent fluid weight of 40 pcf, plus a seismic equivalent fluid weight of 18 pcf.

Walls retaining less than six feet of soil do not need to be designed for the seismic increment of earth pressure. We are providing the above preliminary recommended lateral earth pressures for soil only at this time because it is currently unclear how much of the proposed below-grade walls will retain rock. Although the proposed excavation will extend below the top of rock in some locations, we don't currently know how much of the excavation will be slope-cut versus supported with shoring. Where slope cuts are made, the wall will essentially be retaining backfilled soil. Furthermore, a detailed grading plan has not yet been developed, so we cannot evaluate the proposed outside grades relative to the bottom-of-wall, which will be required to better define the proportions of retained soil versus retained rock. Once the excavation, shoring, grading, and foundation plans have been further developed, we can provide more specific earth pressure recommendations, as needed, which may include reduced earth pressures in rock.

The preliminary recommended lateral earth pressures above are based on a level backfill condition with no additional surcharge loads from vehicles, adjacent building foundations, or sloping ground conditions. Where the below-grade walls are subject to passenger vehicle loading within 10 feet of the back-of-wall, an additional uniform lateral pressure of 50 psf should be applied to the upper 10 feet of the wall. If the below-grade wall is within the zone of influence of adjacent building foundations, defined as a 1.5:1 (horizontal:vertical) plane extending downward from the bottom of footing, the wall should be designed for additional foundation surcharge pressures. Evaluation of potential surcharge pressures from nearby buildings will require detailed cross sections for the subject new buildings and as-built foundation details and structural loading for the existing neighboring buildings. Once this information has been determined, we can provide specific recommendations for design surcharge pressures on the proposed below-grade walls.

The lateral earth pressures recommended above assume the proposed walls will be properly drained to prevent hydrostatic pressure build-up behind the walls. Although we anticipate the regional static groundwater level is deeper than the proposed the basement walls, water can accumulate behind the walls from other sources, such as rainfall, irrigation, broken water lines, and perched water/seepage along the soil-bedrock interface. One acceptable method for backdraining the wall is to place a prefabricated drainage panel (Miradrain 6000 or equivalent) against the shoring or the back of the wall. The drainage panel should extend down to a four-inch-diameter perforated PVC collector pipe at the base of the wall. The pipe should be surrounded on all sides by at least four inches of Caltrans Class 2 permeable material (see Caltrans 2015 Standard Specifications Section 68) or 3/4-inch drain rock wrapped in filter fabric (Mirafi 140NC 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. We do <u>not</u> recommend connecting the wall drainage system to the underslab drainage system described in Sections 6.3 and 7.3 of this report. We should check the manufacturer's specifications regarding the proposed prefabricated drainage panel material to verify it is appropriate for its intended use. To protect against moisture migration into the below-grade levels, we recommend the below-grade walls be waterproofed and water stops be installed at all construction joints.

The purpose of the wall backdrain is to capture and remove subsurface water. Surface water, such as run-off from the hillside, pavements, and roof downspouts should not be directed into the wall drainage system and should be managed by a separate system. Accordingly, the wall drainage panel should be terminated about 12 inches below the ground surface and should be capped by compacted clay fill and/or concrete. In addition, we recommend a concrete v-ditch or bio-swale lined with an impermeable liner be constructed between the structure and the above slope (where present) to collect and remove surface run-off.

If backfill is required behind walls prior to construction of the podium slab, the walls should be braced to prevent unacceptable surcharges on walls (as determined by the structural engineer).

7.6 Excavation Shoring

The safety of workers and equipment in or near the excavation is the responsibility of the contractor. The selection, design, construction, and performance of the shoring system should be 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 ensure that it meets our requirements. During construction, we should observe the installation of the shoring system and check the condition of the soil and rock encountered during excavation.

As discussed in Section 6.4, we conclude viable shoring systems for the proposed excavations include soil nails or soldier piles-and-lagging. Recommendations for the design and construction of both shoring types are presented below.

7.6.1 Cantilever Soldier Pile-and-Lagging Shoring System

We recommend a cantilevered soldier pile-and-lagging shoring system be designed to resist active equivalent fluid weights of 40 and 20 pcf in soil and bedrock, respectively. The transition from soil to rock earth pressures should be assumed at an average depth of 6 feet below existing grades. In locations where minimizing lateral deflections is critical, such as near adjacent buildings or near sensitive underground utilities, the shoring system should be designed to resist at-rest equivalent fluid weights of 60 and 30 pcf in soil and bedrock, respectively. Where passenger vehicle traffic loads are expected within 10 feet of the shoring walls, an additional design load of 50 psf should be applied to the upper ten feet of the wall. Where construction equipment will be working behind the walls within a horizontal distance equal to the wall height, the design should include a surcharge pressure of 250 psf. The above pressures should be assumed to act over the entire width of the lagging installed above the excavation.

Passive resistance at the toe of the soldier piles should be computed using an equivalent fluid weight of 450 pcf with a maximum passive earth pressure of 4,000 psf, assuming the toes of the soldier piles are embedded entirely in weathered rock. The upper foot of bedrock should be neglected when computing passive resistance. Passive pressure can be assumed to act over an area of three soldier pile widths assuming the toe of the soldier pile is filled with structural concrete. The shoring designer should check that the specified minimum concrete strength is sufficient to spread the anticipated loads to three soldier pile widths. The passive pressure values include a factor of safety of at least 1.5.

Soldier piles should be placed in pre-drilled holes backfilled with concrete. Drilling the soldier piles will require equipment capable of penetrating bedrock. If water is encountered in the drilled holes, the contractor should be prepared to pump the water out immediately prior to placing concrete. Alternatively, the water may be displaced by placing concrete from the bottom-up using a tremie pipe. Installing soldier piles by driving or using vibratory methods is not feasible for this site.

Where retained heights exceed about 12 to 14 feet, a soldier pile-and-lagging system may be more cost-effective with tiebacks, although a partial slope cut may also be used to reduce the cantilevered height without tiebacks. The lateral earth pressures presented above are applicable to a cantilevered system supporting level ground. If a partial slope-cut configuration or a tiedback system is considered, we can provide additional recommendations once the proposed geometry has been determined.

7.6.2 Soil Nails

The proposed excavation may be supported by a soil nail shoring system. Soil nail walls should be designed to resist static lateral earth pressures, as well as traffic loads, construction equipment loads, and foundation surcharge loads, where applicable. In general, we recommend the walls be designed and constructed in accordance with the guidelines presented in the Federal Highway Administration report on soil nail walls (FHWA, 2015)[10]. Several computer programs, such as SNAIL (California Department of Transportation, 2014) and GoldNail (Golder Associates, 1996), are available for designing a soil-nail wall. SNAIL uses a force equilibrium method of analysis; the failure planes are assumed bi-linear if they pass through the toe of the wall and tri-linear if they pass below the toe of the wall. GoldNail uses a slope-stability model that satisfies overall limiting equilibrium of free bodies defined by circular slip surfaces.

Soil-nail systems are typically installed under a design-build contract by specialty contractors; therefore, we are not providing a specific design. However, we are providing estimated input

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[10] Federal Highway Administration (2003), *Geotechnical Engineering Circular No. 7 – Soil Nail Walls*, March 2003 (FHWA Report No. FHWA0-IF-03-017)

parameters for preliminary design. The actual soil nail capacities and lengths should be determined by a design-build contractor with experience designing, building, and testing soil-nail walls in similar soil and rock conditions. We should review the geotechnical aspects of their design prior to installation. For preliminary design, we recommend the input parameters presented in Table 4.

TABLE 4
Recommended Input Parameters for Design of Soil-Nail Walls

Soil Type	**Total Density (pcf)**	**Ultimate Bond Strength (psf) (Factor of Safety = 1.0)**	**Shear Strength Parameters**	
			c[1] (psf)	**ϕ[2] (deg)**
Soil	125	1,000	500	25
Weathered Bedrock	135	3,000	2,000	45

Notes:
[1] Cohesion intercept or undrained shear strength, without a factor of safety
[2] Angle of internal friction, without a factor of safety

Where construction equipment will be working or driving behind the walls, the design should include a surcharge pressure of 250 psf. The soil-nail wall should be designed with a minimum factor of safety of 1.5 against slope stability failure for temporary walls and a factor of safety of 2.0 for permanent walls.

The soil-nail wall should be properly backdrained. Typically, two-foot-wide, prefabricated drainage panels are placed behind the shotcrete facing at the same spacing as the nails. Wire mesh and shotcrete should be applied to the exposed soil face within eight hours of excavation.

We should be allowed to review the design plans and design calculations prior to their issuance for construction to check for conformance with our recommendations.

Soil Nail Installation

The drilling method and equipment should be determined by the contractor and modified, as needed, based on the soil conditions encountered during excavation and drilling. If the drilling methods and equipment deviate from those used during installation of the load-tested verification nails, additional verification tests may be required. The holes should be cleaned of loose soil prior to placement of bars, centralizers, and grout. If caving soil is encountered, casing of the holes may be required. We recommend all soil nails be grouted the same day they are drilled and that grout be placed using the tremie method from the bottom of the hole.

Maintaining a consistent grout mix is critical to achieving consistent nail performance and is the responsibility of the contractor. Mud balance measurements of the specific gravity of the grout mixture may be used in the field to provide immediate indications of the grout consistency (water-cement ratio). We recommend a minimum specific gravity of 1.80 be used for grout mixes containing cement and water. In our experience, grout mixes with specific gravities significantly lower than 1.80 can result in inadequate soil nail bond strengths, longer required cure times before proof testing, and increased load test failures.

Soil-Nail Testing

We recommend the soil-nails be load-tested prior to and during construction in accordance with the guidelines presented in the Federal Highway Administration document (FHWA, 2015). Test nails should be installed using the same equipment, method, and hole diameter as planned for the production nails. Verification and proof tests should be performed. Verification tests are performed prior to production nail installation to verify the pullout resistance (bond strength) value used in design and resulting from the contractor’s chosen installation methods. Two verification tests should be performed for each soil type assumed in design. Proof tests are performed during construction to verify that the contractor’s procedure remains consistent and that the nails are not installed in a soil type that was not adequately represented by the verification stage testing. At least five percent of the production nails should be proof tested.

Verification tests should be performed on non-production, sacrificial nails to a test load corresponding to the ultimate pullout resistance value used in the design. Test nails should have at least three feet of unbonded length and 10 feet of bond length. The nail bar grade and size should be designed such that the bar stress does not exceed 80 percent of its ultimate tensile strength for Grade 75 steel or 90 percent of the yield strength for Grade 60 steel during testing—a larger bar may be required for verification test nails.

The verification and proof tests should be performed in accordance with FHWA guidelines (FHWA, 2015), including the recommended load increments, maximum test load, and failure criteria. In the verification and proof tests, the load is applied to the nails in four increments (one complete load cycle). The maximum test load should be held for a minimum of 10 minutes; the movements of the nails should be recorded at 0, 1, 2, 3, 4, 5, 6, and 10 minutes. If the difference in movement between the 1- and 10-minute reading is less than 0.04 inch, the test is discontinued. If the difference is greater than 0.04 inch, the holding period is extended to 60 minutes, and the movements should be recorded at 15, 20, 25, 30, 45, and 60 minutes.

We should evaluate the test results and determine whether the test nail performance is acceptable. Generally, a test with a ten-minute hold is acceptable if the nail carries the maximum test load with less than 0.04 inch movement between one and 10 minutes. A test with a 60-minute hold is acceptable if the nail carries the maximum test load with less than 0.08 inch movement between six and 60 minutes.

7.7 Pavement Design

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

7.7.1 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 the expansive clays at the site. Recommended pavement sections for traffic indices ranging from 4.5 to 7.5 are presented in Table 5.

TABLE 5
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 six inches of the subgrade should be moisture-conditioned and compacted in accordance with requirements presented in Table 2 in Section 7.1. The aggregate base should be moisture-conditioned to near optimum and compacted to at least 95 percent relative compaction.

Where pavements are adjacent to irrigated landscaped areas, curbs adjacent to those areas should extend through the aggregate base and at least three inches into the underlying soil to reduce the potential for irrigation water to infiltrate into the pavement section. Where pavements are adjacent to storm water treatment facilities, such as bio-swales, flow-through planters, or bio-retention basins, or any other landscaped areas in which a significant thickness of loose, uncompacted soil will be present, the curbs may need to extend deeper, as outlined in Section 7.1.2

7.7.2 Rigid (Portland Cement Concrete) Pavement

Concrete pavement design is based on a maximum single-axle load of 20,000 pounds and a maximum tandem axle load of 32,000 pounds and light truck traffic (i.e., a few trucks per week). The recommended rigid pavement section for these axle loads is six inches of Portland cement concrete over 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 six inches of Class 2 aggregate base.

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 concrete pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. For areas that will receive weekly garbage truck traffic, we recommend the slab be reinforced with a minimum of No. 4 bars at 16-inch spacing 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.

7.8 Seismic Design

The latitude and longitude of the site are 37.9729° and -122.5161°, respectively. For design in accordance with 2019 California Building Code (CBC), we recommend the following:

- Site Class C
- S_S = 1.500g, S_1 = 0.600g
- F_a = 1.2, F_v = 1.4
- S_{MS} = 1.800g, S_{M1} = 0.840g
- S_{DS} = 1.200g, S_{D1} = 0.560g
- Seismic Design Category D for Risk Factors I, II, and III

8.0 GEOTECHNICAL SERVICES DURING CONSTRUCTION

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

9.0 LIMITATIONS

This geotechnical study has been conducted in accordance with the standard of care commonly used as state-of-practice in the profession. No other warranties are either expressed or implied. The recommendations made in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed in the 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.

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FIGURES

Base map: Google Maps, 2020.

0 200 400 Feet

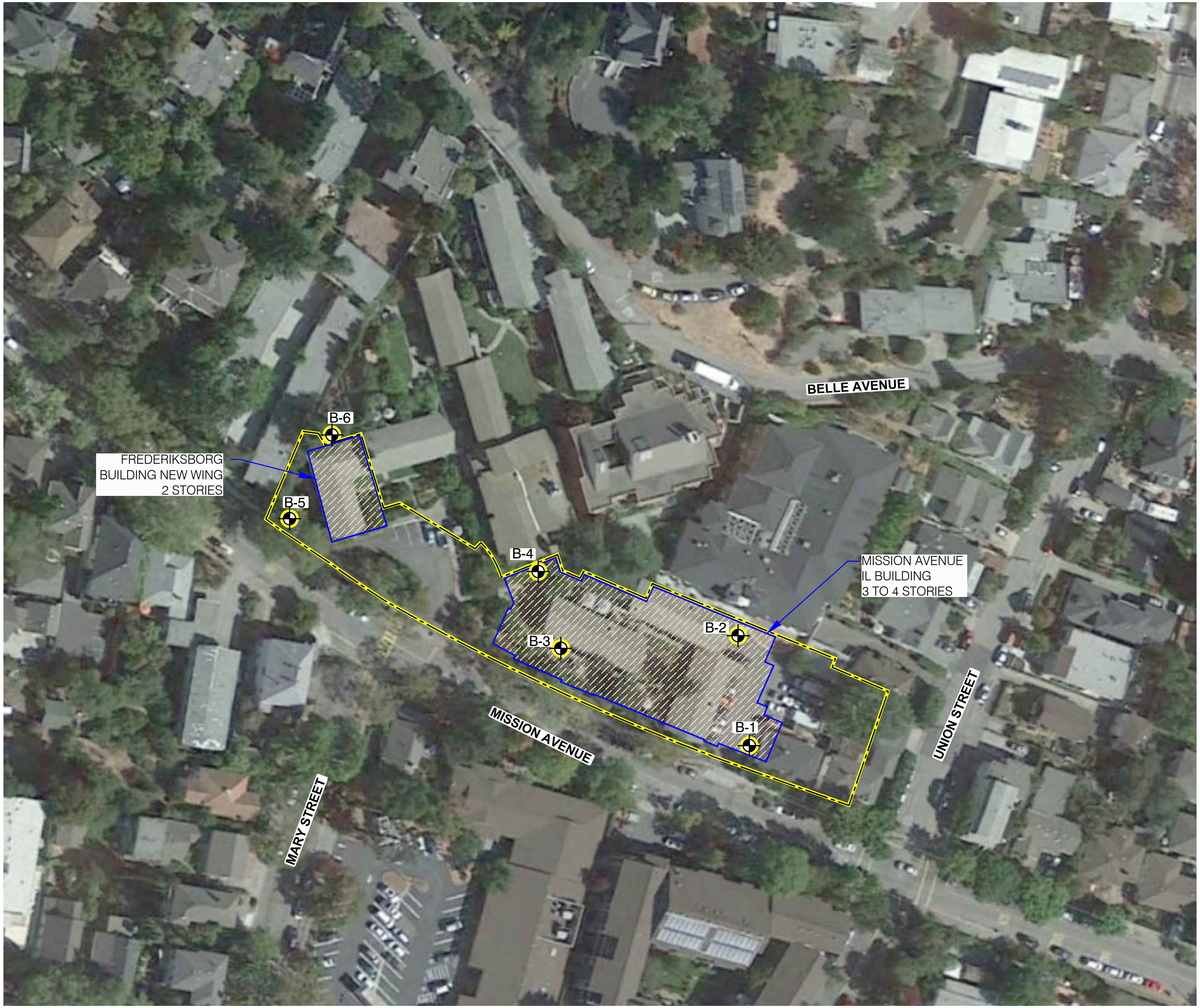
Approximate scale

ALDERSLY RETIREMENT COMMUNITY
San Rafael, California

ROCKRIDGE
GEOTECHNICAL

SITE LOCATION MAP

Date 02/26/20	Project No. 19-1779	Figure 1



EXPLANATION

B-1 ⊕	Approximate location of boring by Rockridge Geotechnical Inc., February 24-25, 2020
—·—	Project limits
▨	Approximate footprints of proposed buildings

0 80 Feet

Approximate scale

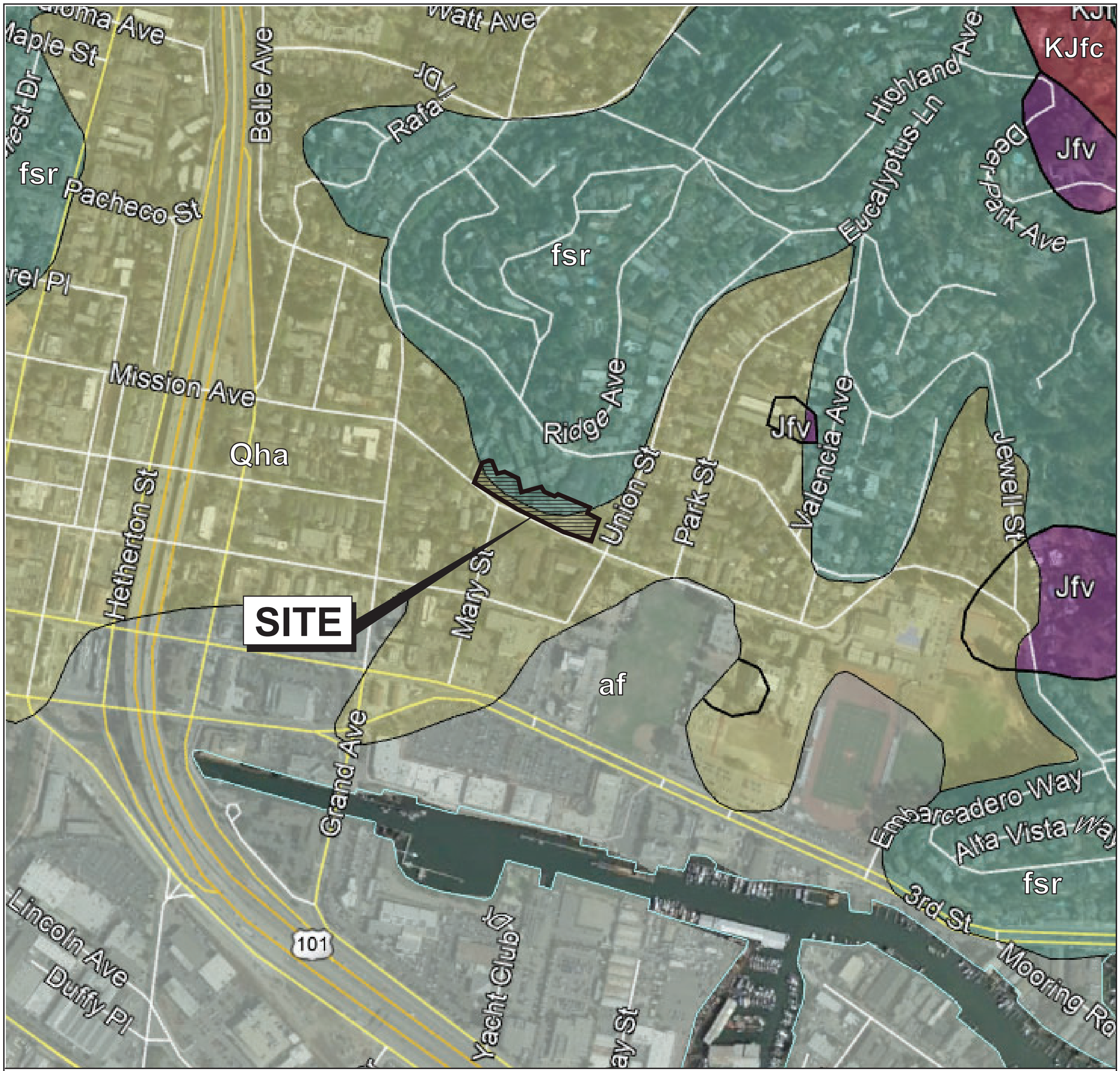
Base map: Google Earth, 2020.

ALDERSLY RETIREMENT COMMUNITY
San Rafael, California

SITE PLAN

Date 08/21/20 | Project No. 19-1779 | Figure 2

ROCKRIDGE
GEOTECHNICAL



Base map: Google Earth with U.S. Geological Survey (USGS), Marin County, 2020.

EXPLANATION

af	Artificial Fill
Qha	Alluvium (Holocene)
fsr	Franciscan Complex melange (Eocene, Paleocene, and (or) Late Cretaceous)
KJfc	Franciscan Complex chert (Early Cretaceous and (or) Late Jurassic)
Jfv	Franciscan Complex volcanic rocks (Jurassic)

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

N

0 600 1200 Feet

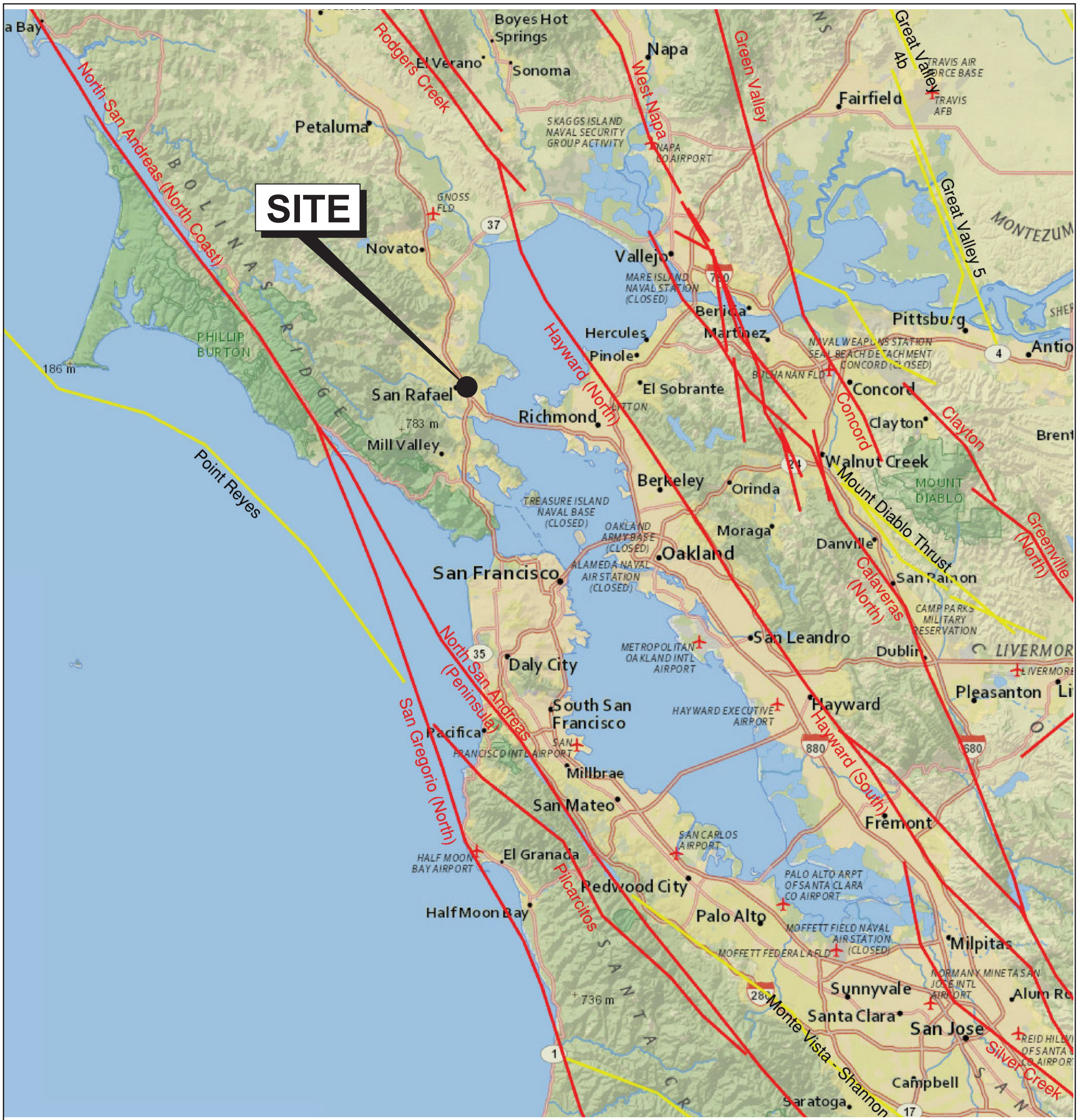
Approximate scale

ALDERSLY RETIREMENT COMMUNITY
San Rafael, California

ROCKRIDGE
GEOTECHNICAL

REGIONAL GEOLOGIC MAP

Date 04/16/20 | Project No. 19-1779 | 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

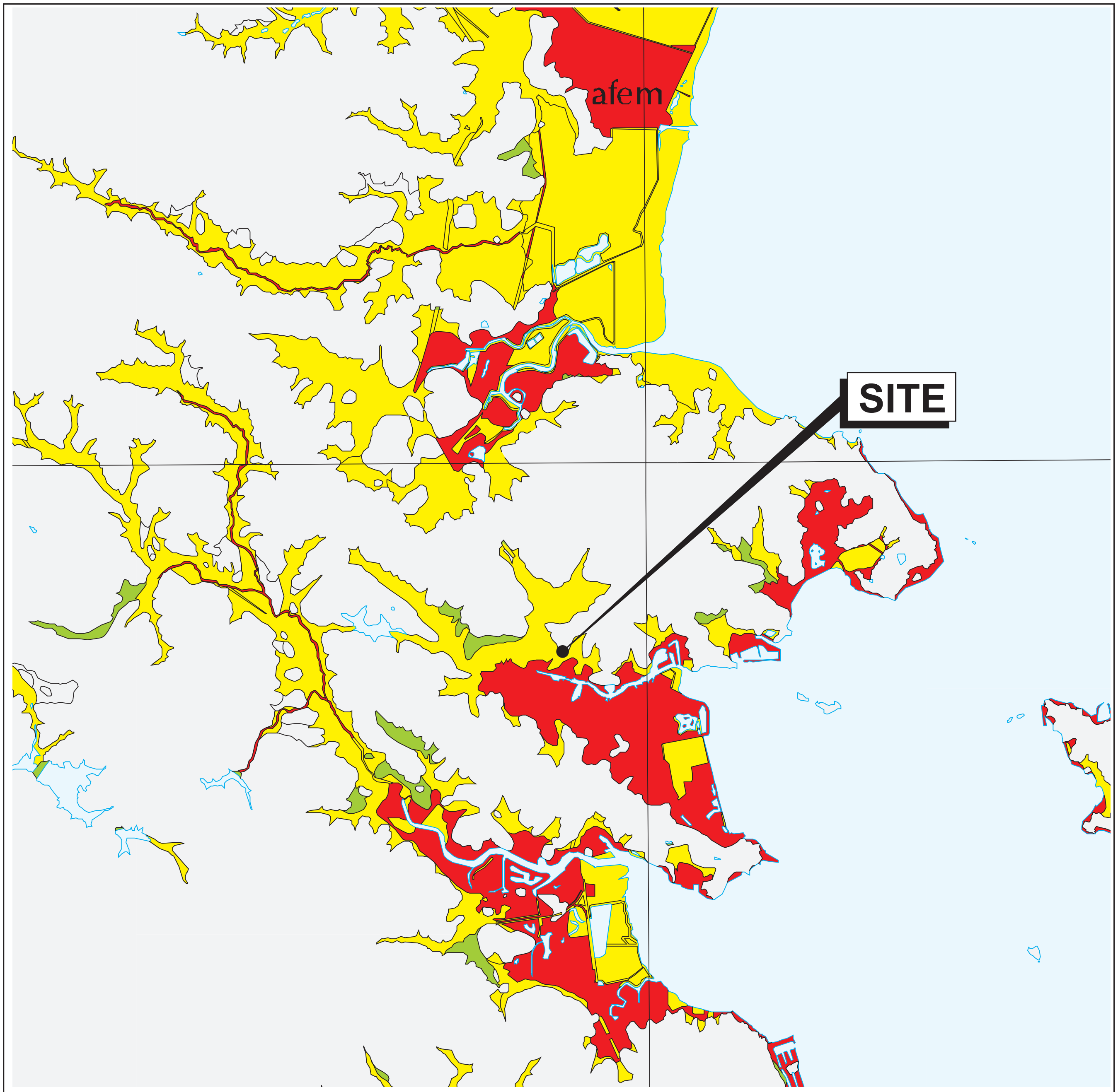
Approximate scale

ALDERSLY RETIREMENT COMMUNITY
San Rafael, California

ROCKRIDGE GEOTECHNICAL

REGIONAL FAULT MAP

Date 02/26/20 | Project No. 19-1779 | Figure 4



LIQUEFACTION SUSCEPTIBILITY

🟥 VERY HIGH

🟧 HIGH

🟨 MODERATE

🟩 LOW

⬜ VERY LOW

Lines

Contact, dashed where location uncertainty
is greater than about +- 100 m.

0 1.5 3 Miles

Approximate scale

Reference:
Maps of Quaternary Deposits and Liquefaction Susceptibility
in the Central San Francisco Bay Region, California, by USGS, 2006

ALDERSLY RETIREMENT COMMUNITY
San Rafael, California

ROCKRIDGE
GEOTECHNICAL

LIQUEFACTION SUSCEPTIBILITY MAP

Date 08/28/20 | Project No. 19-1779 | Figure 5

APPENDIX A
Logs of Borings

PROJECT: **ALDERSLY RETIREMENT COMMUNITY**
San Rafael, California

Log of Boring B-1

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 02/24/2020
Date finished: 02/24/2020
Drilling method: 4-inch Solid Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), California (CA)

Logged by: A. Limpert
Drilled by: Benevent Building, LLC
Rig: Portable Hydraulic Rig

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 14.0 feet[2]	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
						3 inches of ashalt concrete 3 inches of aggregate base						
1	S&H	▨	28 8 9	12		SANDY CLAY (CL) olive-gray, stiff, moist						
2					CL							
3	CA	▨	4 6 10	14		decreasing sand content Particle Size Distribution; see Appendix B				53	16.7	
4												
5	SPT	◪	5 10 20	36	BR	MUDSTONE gray, intensely fractured, low hardness						
6	CA	▨	20 34 50/ 4.5"	76/ 10.5"		SANDSTONE gray, intensely fractured, low hardness						
7	SPT	◪	24 19 33	62	BR							
8												
9	SPT	◪	42 50/4"	60/4"		soft						
10	SPT	◪	50/ 0.25"	60/ 0.25"								
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												

Boring terminated at a depth of 10.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

---

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

[2] Elevations NAVD 88 datum based on "Aldersly Retirement Community, Topographic Map, 326 Mission Avenue", dated October 5, 2017 by CSW/Stuber-Stroeh Engineering Group, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1779

Figure: A-1a

PROJECT: **ALDERSLY RETIREMENT COMMUNITY**
San Rafael, California

Log of Boring B-2

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 02/24/2020
Date finished: 02/24/2020
Drilling method: 4-inch Solid Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), California (CA)

Logged by: A. Limpert
Drilled by: Benevent Building, LLC
Rig: Portable Hydraulic Rig

DEPTH (feet)	SAMPLES: Sampler Type	SAMPLES: Sample	SAMPLES: Blows/ 6"	SAMPLES: SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 20.9 feet[2]	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
						landscaped area						
1	S&H	■	5 5 5	7	CL	CLAY with SAND and GRAVEL (CL) gray with black, medium stiff, moist, subangular gravel						
2						CLAYEY SAND with GRAVEL (SC) gray-brown with white, medium dense to dense, moist, large angular to subangular gravel, black pieces						
3	CA	■	8 12 20	29	SC	Particle Size Distribution; see Appendix B RESIDUAL SOIL				33	13.7	113
4												
5	SPT	◩	11 11 13	29		gray-brown with white						
6	CA	■	12 17 41	52		CLAYSTONE gray with white, highly weathered, rust stains, white staining						
7												
8	SPT	◩	14 43 31	89								
9	SPT	◩	15 15 29	53								
10						large piece of dark gray, hard CLAYSTONE in shoe						
11	SPT	◩	19 15 26	49								
12	SPT	◩	50/3"	60/3"								
13												
14												
15												
16												
17												
18												
19												
20												

Boring terminated at a depth of 12.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

---

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

[2] Elevations NAVD 88 datum based on "Aldersly Retirement Community, Topographic Map, 326 Mission Avenue", dated October 5, 2017 by CSW/Stuber-Stroeh Engineering Group, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1779

Figure: A-2

PROJECT: **ALDERSLY RETIREMENT COMMUNITY**
San Rafael, California

Log of Boring B-3

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Date started: 02/24/2020 Date finished: 02/24/2020

Drilling method: 4-inch Solid Stem Auger

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

Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), California (CA)

Logged by: A. Limpert
Drilled by:
Rig:

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 19.1 feet[2]	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
						4 inches of concrete paver						
1						CLAY (CL) gray with trace black gravel, stiff, moist, trace wood/ organics						
2	S&H	■	5 8 10	13	CL	LL = 48, PI = 31; see Appendix B					16.5	117
3	CA	■	15 25 36	55		CLAYSTONE gray with black mottled, fractured, highly weathered, weak Corrosivity Test; see Appendix B						
4												
5	SPT	◩	21 28 50/5"	60/ 11"								
6	SPT	◩	17 50/4"	60/4"								
7												
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												

Boring terminated at a depth of 6.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

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

[2] Elevations NAVD 88 datum based on "Aldersly Retirement Community, Topographic Map, 326 Mission Avenue", dated October 5, 2017 by CSW/Stuber-Stroeh Engineering Group, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1779

Figure: A-3

PROJECT: **ALDERSLY RETIREMENT COMMUNITY**
San Rafael, California

Log of Boring B-4

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 02/24/2020
Date finished: 02/24/2020
Drilling method: 4-inch Solid Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), California (CA)

Logged by: A. Limpert
Drilled by: Benevent Building, LLC
Rig: Portable Hydraulic Rig

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 26.1 feet[2]	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
						4 inches of concrete						
1	S&H	▨	5 5 12	12	CL	CLAY (CL) olive-brown, stiff, moist, rust stains, light brown mottled, large gravel/rock in shoe						
2												
3	CA	▨	12 10 15	23		CLAYSTONE brown with gray and white veins, fractured, highly weathered, weak						
4						light gray, less weathering with depth						
5	SPT	◩	12 13 27	48								
6	SPT	◩	10 14 50/4"	77/ 10"		light brown						
7	SPT	◩	50/ 1.5"	60/ 1.5"								
	SPT	◩	50/1"	60/1"								
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												

Boring terminated at a depth of 7.25 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

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

[2] Elevations NAVD 88 datum based on “Aldersly Retirement Community, Topographic Map, 326 Mission Avenue”, dated October 5, 2017 by CSW/Stuber-Stroeh Engineering Group, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1779
Figure: A-4

PROJECT: **ALDERSLY RETIREMENT COMMUNITY**
San Rafael, California

Log of Boring B-5

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 02/25/2020
Date finished: 02/25/2020
Drilling method: 4-inch Solid Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), California (CA)

Logged by: A. Limpert
Drilled by: Benevent Building, LLC
Rig: Portable Hydraulic Rig

DEPTH (feet)	SAMPLES Sampler Type	Sample	Blows/ 6"	SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 15.1 feet[2]	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
						landscaped area fill soil, organics						
1	S&H		8 8 7	11	CL	CLAY with SAND and GRAVEL (CL) brown with light brown, stiff, moist, some organics present						
2												
3	CA		3 4 7	10	CL	CLAY with SAND (CL) dark brown with some rust stains, stiff, moist Corrosivity Test; see Appendix B						
4	SPT		2 4 6	12	SC	CLAYEY SAND with GRAVEL (SC) brown with light brown mottled, medium dense, moist, rust stains, green gravel in shoe Particle Size Distribution; see Appendix B				30	16.0	
5												
6	S&H		5 8 15	16								
7	CA		6 7 11	16		SANDSTONE light brown with brown and red mottling, soft, moist, friable, deeply weathered WEATHERED BEDROCK						
8												
9	SPT		8 10 8	22								
10	SPT		5 8 15	28								
11												
12												
13	SPT		11 11 15	31		CLAYSTONE olive-gray, moderately hard, weak, moderately weathered						
14												
15	SPT		12 19 22	49								
16												
17	SPT		50/ 5.5"	60/ 5.5"								
18												
19												
20												

Boring terminated at a depth of 17.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

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

[2] Elevations NAVD 88 datum based on "Aldersly Retirement Community, Topographic Map, 326 Mission Avenue", dated October 5, 2017 by CSW/Stuber-Stroeh Engineering Group, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1779
Figure: A-5

PROJECT: **ALDERSLY RETIREMENT COMMUNITY**
San Rafael, California

Log of Boring B-6

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Date started: 02/25/2020
Date finished: 02/25/2020
Drilling method: 4-inch Solid Stem Auger
Hammer weight/drop: 140 lbs./30 inches
Hammer type: Rope & Cathead
Sampler: Sprague & Henwood (S&H), Standard Penetration Test (SPT), California (CA)

Logged by: A. Limpert
Drilled by: Benevent Building, LLC
Rig: Portable Hydraulic Rig

DEPTH (feet)	SAMPLES Sampler Type	 Sample	 Blows/ 6"	 SPT N-Value[1]	LITHOLOGY	MATERIAL DESCRIPTION Approximate Ground Surface Elevation: 25.7 feet[2]	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
						fill material for path						
1												
	S&H		6 7 8	11	SC	CLAYEY SAND (SC) brown with rust stains, medium dense, moist, trace gravel, debris Particle Size Distribution; see Appendix B				47	13.9	118
2												
						medium stiff						
3	CA		2 3 5	7	CL	CLAY (CL) gray, medium stiff grading to very stiff, moist, trace fine sand					15.3	109
4						LL = 30, PI = 12; see Appendix B						
5	SPT		6 7 15	26		very stiff						
6	S&H		6 12 36	34		hard						
7	SPT		36 50/5"	60/5"		CLAYSTONE gray with white, soft, crushed, highly fractured, low hardness						
8	SPT		26 23 24	56		moderately fractured						
9	SPT		50/ 0.5"	60/ 0.5"								
10												
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												

Boring terminated at a depth of 9.5 feet below ground surface.
Boring backfilled with cement grout.
Groundwater not encountered during drilling.

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

[2] Elevations NAVD 88 datum based on "Aldersly Retirement Community, Topographic Map, 326 Mission Avenue", dated October 5, 2017 by CSW/Stuber-Stroeh Engineering Group, Inc.

ROCKRIDGE GEOTECHNICAL

Project No.: 19-1779
Figure: A-6

UNIFIED SOIL CLASSIFICATION SYSTEM

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

GRAIN SIZE CHART

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

▽	Unstabilized groundwater level
▼	Stabilized groundwater level

SAMPLE DESIGNATIONS/SYMBOLS

⬒	Sample taken with Sprague & Henwood split-barrel sampler with a 3.0-inch outside diameter and a 2.43-inch inside diameter. Darkened area indicates soil recovered
⧄	Classification sample taken with Standard Penetration Test sampler
◨	Undisturbed sample taken with thin-walled tube
☒	Disturbed sample
⊙	Sampling attempted with no recovery
◫	Core sample
◉	Analytical laboratory sample
▥	Sample taken with Direct Push sampler
▥	Sonic

SAMPLER TYPE

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

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CLASSIFICATION CHART

Date 03/01/20 | Project No. 19-1779 | Figure A-7

I FRACTURING

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

II HARDNESS

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

III STRENGTH

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

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

D. Deep - moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.

M. Moderate - slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.

L. Little - no megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.

F. Fresh - unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

ADDITIONAL COMMENTS:

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

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

VI BEDDING OF SEDIMENTARY ROCKS

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

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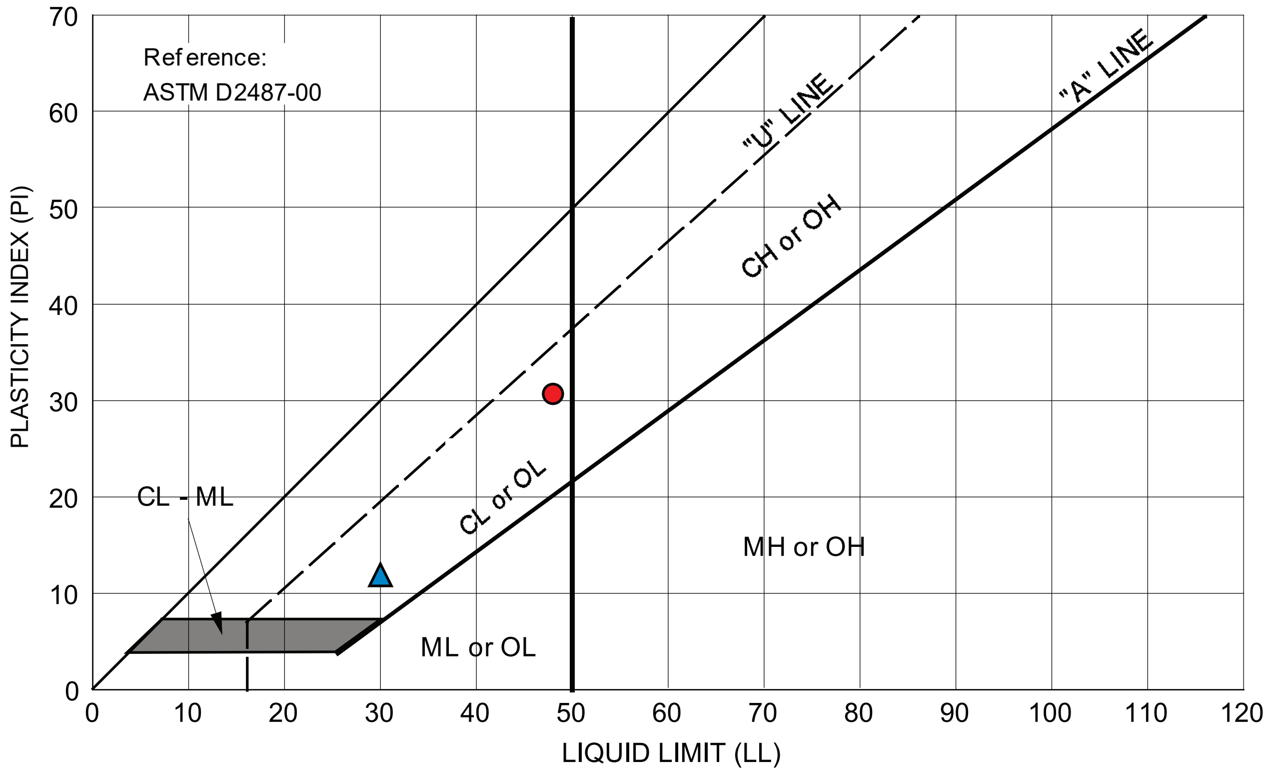
PHYSICAL PROPERTIES CRITERIA
FOR ROCK DESCRIPTIONS

Date 04/16/20 | Project No. 19-1779 | Figure A-8

APPENDIX B
Laboratory Test Results



Symbol	Source	Description and Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
●	B-3 at 2.0 feet	CLAY (CL), gray with trace black gravel	16.5	48	31	--
▲	B-6 at 3.5 feet	CLAY (CL), gray	15.3	30	12	--

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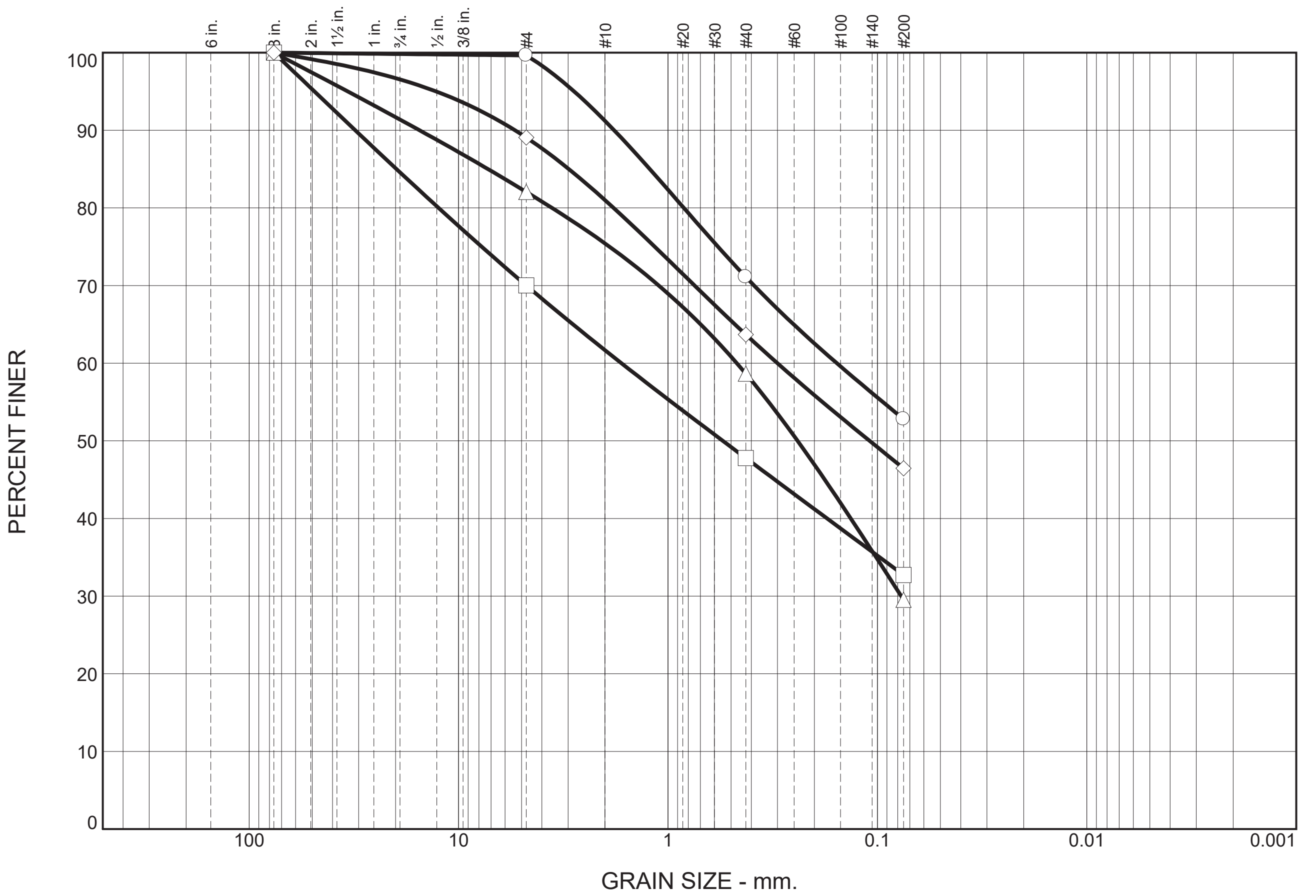
PLASTICITY CHART

Date 04/16/20 | Project No. 19-1779 | Figure B-1



	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0.0	0.4	46.8	52.8	
□	0.0	30.0	37.3	32.7	
△	0.0	17.9	52.6	29.5	
◇	0.0	10.9	42.6	46.5	

SOIL DATA

SYMBOL	SOURCE	DEPTH (ft.)	Material Description	USCS
○	B-1	3.75’	SANDY CLAY, olive-gray	CL
□	B-2	2.0’	CLAYEY SAND with GRAVEL, gray-brown with white	SC
△	B-5	4.0’	CLAYEY SAND with GRAVEL, brown with light brown mottled	SC
◇	B-6	1.0’	CLAYEY SAND, brown with rust stains	SC

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PARTICLE SIZE DISTRIBUTION REPORT

Date 04/16/20 | Project No. 19-1779 | Figure B-2

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity**		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Flouride** F_2^{--}	**Phosphate** PO_4^{3-}
						As Rec'd	**Minimum**												
	(ft)	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(Ohm-cm)**	**(Ohm-cm)**		**(mV)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**
B-3 Gray Claystone	4.0	5.0	0.0005	2.4	0.0002	10,050	4,355	8.0	124.0	0.1	1.0	ND	ND	17.9	0.2	16.9	10.1	3.7	0.1
B-5 Dark Brown Clay with Sand	2.5	2.5	0.0003	0.5	0.0001	4,690	4,288	7.9	95.0	0.8	1.7	ND	ND	5.6	1.0	7.8	50.3	0.3	2.6

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

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CORROSION RESULTS

Date 03/17/20 | Project No. 19-1779 | Figure B-3

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January 30, 2021
Project No. 19-1779

Mr. Peter Schakow
Aldersly, Inc.
326 Mission Ave.
San Rafael, CA 94901

Subject: Geotechnical Response to City Comments
Proposed Master Plan Amendments
Aldersly Retirement Community
326 Mission Avenue
San Rafael, California

Dear Mr. Schakow:

This letter presents our response to the following comment provided by the City of San Rafael, dated December 15, 2020, in regard to the proposed master plan amendments:

> *Geotechnical Investigation (Rockridge Geotechnical, August 31, 2020). This report addresses Phase 1A and 1B only, and does not address Phases 2, 3, and 4. The report should at a minimum acknowledge the proposed subsequent phases of development and indicate the extent to which the soil and geologic conditions, conclusions and recommendations contained in the report are applicable to other the entire property and to subsequent phases of the Master Plan. This is important because the CEQA document must address the entirety of the project (all phases). This information can be addressed in a letter addendum from geotechnical consultant with a focus on the items covered on CEQA checklist under Geology and Soils.*

To date we have been engaged to perform a geotechnical investigation and develop recommendations for the design and construction of the proposed Phase 1A and 1B residential buildings, only. We anticipate additional field investigation and engineering analyses will be performed to develop final geotechnical recommendations specific to proposed Phases 1C, 2, 3, and 4 prior to final design of those improvements. The depth to bedrock and quality of soil above bedrock is expected to vary throughout the site and, therefore, detailed foundation recommendations for the subsequent phases will need to be based on additional exploratory borings within those areas. However, we conclude general conclusions and recommendations presented in our report that pertain to the items listed in the *Appendix G – Environmental Checklist Form, Section VII. Geology and Soils* (i.e. fault rupture, seismic ground shaking, seismic-related ground failure, landslides, soil erosion, and expansive soil) are applicable to the entire site, including proposed future Phases 1C, 2, 3, and 4.

We trust this letter provides the information you require at this time. If you have any questions, please call.

Sincerely,
ROCKRIDGE GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
LOGAN D. MEDEIROS
2957
12/31/21
GEOTECHNICAL
STATE OF CALIFORNIA

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