CORNERSTONE
EARTH GROUP

TYPE OF SERVICES Geologic Hazard Evaluation and Geotechnical Investigation

PROJECT NAME Embedded Way Industrial Building

LOCATION 865 Embedded Way
San Jose, California

CLIENT Oppidan

PROJECT NUMBER 1345-1-3

DATE May 2, 2023

GEOTECHNICAL

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Client Address	**1100 Lincoln Avenue San Jose, California 95125**
Project Number	**1345-1-3**
Date	**May 2, 2023**

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SECTION 1: INTRODUCTION

This geologic hazard evaluation and geotechnical investigation was prepared for the sole use of Oppidan for the site located at 865 Embedded Way in San Jose, California. The purpose of this study was to evaluate the existing subsurface conditions and develop an opinion regarding potential geotechnical and geologic concerns that could impact the proposed development. We have previously provided a preliminary study of the site titled "Geologic Hazard Evaluation and Preliminary Geotechnical Investigation, Embedded Way Industrial Building" dated August 2, 2022. For our use, we were provided with the following documents:

- A partial set of civil plans titled, "Coyote Creek Industrial R&D" prepared by AMS Associates, Inc, dated March 17, 2023.

1.1 PROJECT DESCRIPTION

The approximately 10.2-acre project site, currently designated as APN 679-01-020, is located at 865 Embedded Way in San Jose, California. The site is currently a vacant parcel that had previously been graded for future development in the 1990s. The parcel consists of a flat building pad that slopes down along the northern, western and southern edges.

Based on our understanding, a warehouse structure is currently planned for the site totaling approximately 130,000 square feet. The planned improvements will include appurtenant parking, truck loading docks, utilities, landscaping and other improvements necessary for site development. A below-grade storm water storage system is planned to the west of the new building within new parking areas. The water storage system will be approximately 106 feet by 126 feet by 10 feet deep and will include large diameter water storage pipes surrounded by permeable gravel. In addition, a storm water retention basin is planned at the south edge of the site that will require retaining walls from approximately 4 to 15 feet high.

Structural loads are not currently known for the proposed structure; however, structural loads are expected to be typical of similar type structures. We assume moderate cuts and fills on the order of a few to several feet will be required to create the building pad, retaining walls, parking lots and drive aisles. Perimeter site walls will be constructed on existing natural and man-made fill slopes, are planned to be approximately 5 to 12 feet high, and are likely to consist of mechanically stabilized earth (MSE) or criblock wall systems.

The site lies at northernmost end of a Santa Clara County and City of San Jose fault rupture hazard zone for the potentially active Piercy fault (aka Coyote Creek fault). As a result, our scope has also been focused on determining if prior studies for the adjacent commercial buildings have already located the fault and developed building exclusions zones adjacent to the fault.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated March 28, 2023, and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 PRIOR GEOTECHNICAL INVESTIGATIONS

The project site is located within the Edenvale Industrial Area of southeastern San Jose. The geology of Santa Clara County and the greater San Francisco Bay Area has been mapped extensively by both the USGS and the CGS. Published geologic and seismic literature and maps reviewed as part of our scope for this assessment are listed in the References section of this report. Several geotechnical and geological studies were reviewed for preparation of this report. Two reports in the immediate vicinity are as follows.

Engeotech, Inc., May of 1995, Report to Berg & Berg Developers, Cupertino, California. Soil and Foundation Investigation on the King Ranch (Areas F through J Portion). This report was conducted to support industrial development of a portion of the former King Ranch, with boundaries between East Branham Lane (now Embedded Way) on the north, Hellyer Avenue to the east and Loop Street (now Fontenoso Way) to the south. This investigation included drilling 11 borings in alluvial soils and recommendations for foundations and grading for 2 to 3 story industrial tilt-up structures that are now located between Embedded Way, Hellyer Avenue and Fontenoso Way and Coyote Creek. This report did not address developments north of Embedded Way (including the project site) nor did it address geologic hazards associated with the Piercy/Coyote Creek fault.

Engeotech, Inc., December of 1995, Report to Berg & Berg Developers, Cupertino, California. Soil and Foundation Investigation on the Kings Ranch (Areas A-E and K-M Parts), Hellyer Avenue and Branham Lane, San Jose, California. Consultants Report, Dec. This report was conducted to support industrial development of a portion of the former King Ranch, with boundaries between East Branham Lane (now Embedded Way) to the south, Hellyer Avenue to the east and the current hilltop containing the 875 and 865 Embedded Way parcels. This investigation included drilling 16 borings in alluvial soils and recommendations for foundations and grading for 2 to 3 story industrial tilt-up structures. As with the earlier work in 1995, this report did not address geologic hazards associated with the Piercy/Coyote Creek fault.

Other geotechnical and geologic reports in the general vicinity include:

Kleinfelder, 2016a, Geologic Literature Review and Desktop Assessment Pacific Gas and Electric Company (PG&E) Stone-Evergreen-Metcalf 115 kV Transmission Line Project Santa Clara County, California. File No. 20164728.001A, dated June 8.

Kleinfelder, 1998, Geologic and Preliminary Geotechnical Investigation, Proposed Canyon Creek Plaza Development, along Silver Creek Valley Road in San Jose, California, File No. 12-3039-30, dated February 24.

Kleinfelder, 2018, Geotechnical Investigation Report, PG&E Piercy Substation CB122 Relay Upgrade Project, 5444 Hellyer Avenue, San Jose, California. File No. 20191583.001A, dated October 4.

Engeotech, Inc., 1997, Report to Berg & Berg Developers, Cupertino, California. Soil and Foundation Investigation of Hellyer I and Hellyer II. This parcel is situated between Hellyer Avenue and Silver Creek Valley Road.

These last four reports do not address geologic hazards specifically but provide general observations of site conditions and subsurface data. References specific to faulting are discussed in Section 4.

1.4 RECENT GEOTECHNICAL EXPLORATION PROGRAM

Our previous field exploration consisted of five borings drilled on October 15, 2021, with truck-mounted, hollow-stem-auger drilling equipment. The borings were drilled to depths ranging from approximately 6½ to 15 feet below ground surface. Two additional borings were drilled for the infiltration tests to depths of approximately 5 to 8 feet.

Field exploration for the current project scope consisted of six borings drilled on April 13 and 14, 2023, with truck-mounted and track mounted, limited access, hollow-stem-auger drilling equipment. The borings were drilled to depths ranging from approximately 8¾ to 24¼ feet below ground surface. The borings were backfilled with cement grout in accordance with local requirements.

The approximate locations of our exploratory borings are shown on the Site Plan & Geologic Map, Figure 2. Details regarding our previous and current field program are included in Appendix A.

1.5 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for grading and foundation design. Testing included moisture contents, dry densities, washed sieve analyses, a Plasticity Index test, and an R-value test. Details regarding our laboratory program are included in Appendix B.

1.6 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including a Phase 1 site assessment; environmental findings and conclusions are provided under separate covers.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The project site is located in the Santa Clara Valley, on the east side of Coyote Creek and approximately 12 miles southeast of the southern shore of the San Francisco Bay (Figure 1). The Santa Clara Valley separates the Santa Cruz Mountains to the west and the Diablo Range to the east (California Geological Survey, 2002). The valley is underlain by Pleistocene and Holocene alluvial sediments deposited by several creeks and rivers, including the Guadalupe River and Coyote Creek, that generally flow northwest towards the San Francisco Bay. Geologic formations in the Santa Clara Valley region range in age from Jurassic (190 to 135 million years ago) to recent Holocene (younger than 11,800 years). The basement rocks of the Santa Cruz Mountains consist of accreted Franciscan Complex structurally overlain by Coast Range ophiolite and marine clastics of the Mesozoic Great Valley sequence (Wentworth et al., 1999). The Franciscan Complex contains various types of Cretaceous rocks including sandstone, shale, greywacke, greenstone, and serpentinite.

Based on mapping by Wentworth, et al. (1999), Helley and Wesling (1990) and Helley et al (1994), the site vicinity is underlain by Holocene or older alluvial fan deposits underlain by shallow bedrock (i.e., Franciscan Complex). The surficial sediment deposits include alluvial gravel, sand, silt, and clay. The hills around the site expose Franciscan Complex bedrock of Jurassic to Cretaceous age (about 65 to 205 million years old). The exposures include mélange and exotic serpentinized dunite as shown on the Regional Geologic Map, Figure 3.

The valley is located at the southern end of, and is part of, the larger San Francisco Bay basin. Santa Clara Valley is bordered on the west by the Santa Cruz Mountains and on the east by the rugged East Bay Hills. Both western and eastern margins of the valley are defined by a series of reverse faults that bound the range fronts.

On the eastern margin of Santa Clara Valley, reverse faults separate the valley margin from the uplifted East Bay structural domain to the east. This system of east-dipping reverse faults trending northwest-southeast along the base of the foothills includes the Piercy, Coyote Creek, Evergreen, Quimby, Berryessa, Crosley, and Warm Springs faults.

Santa Clara Valley has been shaped, in large part, by the San Andreas, Calaveras, and Hayward right-lateral strike-slip fault systems. These major fault systems accommodate much of the movement along the plate boundary between the Pacific and North American plates. Local, and possibly regional, compressional deformation associated with these faults hypothetically may have three main possible origins: (1) shortening attributed to the orthogonal component of relative motion between the Pacific and North American plates, (2) shortening occurring at restraining bends and stopovers in dextral fault systems, and (3) shortening caused by rotation of large crustal blocks (Hitchcock and Brankman, 2002).

2.2 REGIONAL SEISMICITY

While seismologists cannot predict earthquake events, geologists from the U.S. Geological Survey have recently updated earlier estimates from their 2014 Uniform California Earthquake Rupture Forecast (UCERF, Version 3) publication. The estimated probability of one or more magnitude 6.7 earthquakes (the size of the destructive 1994 Northridge earthquake) expected to occur somewhere in the San Francisco Bay Area has been revised (increased) to 72 percent for the period 2014 to 2043 (Aagaard et al., 2016). The faults in the region with the highest estimated probability of generating damaging earthquakes between 2014 and 2043 are the Hayward (33%), Rodgers Creek (33%), Calaveras (26%), and San Andreas Faults (22%). In this 30-year period, the probability of an earthquake of magnitude 6.7 or larger occurring is 22 percent along the San Andreas Fault and 33 percent for the Hayward or Rodgers Creek Faults. During such an earthquake the danger of fault surface rupture at the site is slight, but very strong to severe ground shaking would occur.

The San Francisco Bay area including the coastal region is recognized by geologists and seismologists as one of the most seismically active regions in the United States. Significant earthquakes occurring in the area are generally associated with crustal movement along well-defined, active fault zones of the San Andreas Fault system. The San Andreas Fault generated the great San Francisco earthquake of 1906 and the Loma Prieta earthquake of 1989 and passes about 12.6 miles west of the subject site. The Hayward fault is located about 3.2 miles west the site and the Calaveras Fault is located approximately 6.8 miles east of the site.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. Table 1 presents the State-considered active faults located within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	**Distance**	
	(miles)	**(kilometers)**
Monte Vista-Shannon Fault	4.6	7.5
San Andreas Fault	12.6	20.3
Hayward Fault	3.2	5.2
Calaveras Fault	6.8	10.9

A regional fault map is presented as Figure 4, illustrating the relative distances of the site to significant fault zones.

2.3 LOCAL FAULTING AND SEISMICITY

An active fault is a fault that has experienced seismic activity during historic time (since roughly 1800) or exhibits evidence of surface displacement during Holocene time (Bryant and Hart, 2007). The definition of “potentially active” varies. A generally accepted definition of “potentially active” is a fault showing evidence of displacement that is older than 11,700 years (Holocene age [USGS, 2010]) and younger than 2.6 million years (Pleistocene age [USGS, 2010]).

However, “potentially active” is no longer used as a criterion for zoning by the California Geological Survey (CGS), formerly known as the Division of Mines and Geology (DMG). The terms “sufficiently active” and “well-defined” are now used by the CGS as criteria for zoning faults under the Alquist-Priolo Earthquake Fault Zoning Act. A “sufficiently active fault” is a fault that shows evidence of Holocene surface displacement along one or more of its segments and branches, while a “well-defined fault” is a fault whose trace is clearly detectable by a trained geologist as a physical feature at or just below the ground surface. The definition “inactive” generally implies that a fault has not been active since the beginning of the Pleistocene Epoch (older than 2.6 million years).

Based on the data provided in Bryant and Hart (1997 and 2007) and CGS (2000), the project site is not within an Alquist-Priolo Earthquake Fault Zone (AP Zone) established by the CGS around active fault traces. However, portions of the project encroached onto a City of San Jose Fault Hazard Zones (1983) and Santa Clara County Geologic Hazard Zones (2012). Other less prominent faults in the area that are located along the base of the San Jose Foothills include the Piercy, Silver Creek, and Coyote Creek faults. These faults are not zoned by the CGS; however, they are zoned by the City of San Jose and Santa Clara County.

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

3.1.1 Review of Aerial Photographs

Historic aerial photographs taken over a period of time were analyzed to identify subtle features not readily seen on the ground. These features include scarps, concave surfaces, topographic expression and seepage indicative of potential landslides. Faults are commonly associated with topographic and geomorphic expression, tonal differences, lineaments and offset drainages. Findings from the aerial photo analysis are presented in the following sections of this report.

For this project, a total of 10 aerial photo sets (either single or stereo-paired) were evaluated that represented aircraft flights between 1960 and 1996. Flight dates included a range of months from April to November, representative of both wet and dry terrain conditions. These photos were viewed through a stereoscope to provide a timeline of site development. In addition, Google Earth imagery more recent than 1996 was also reviewed. A summary of the review photography is included in Table 1 of the References section.

3.1.2 Geomorphology

As previously discussed, the project site originally occupied a northwest trending ridgeline that was bisected by the north flowing Coyote Creek (USGS, 1961). Agricultural activities dominated this area for over a century with cultivated fields and orchards occupying the river plain and grazing land on the ridgelines and slopes. Access roads generally ran along the toe of the ridge line. This ridgeline and surrounding areas were graded in the 1990 to 2004 timeframe and fill was placed within these alluvial areas to the south to construct roads and building pads to support the industrial development of the area.

3.1.3 Site Development History

A review of these aerial photographs and some historic U.S. Geological Survey 7.5-minute quadrangles (USGS, 1961 1980, CDMG, 1982, USGS 2012) indicate a significant development of this portion of the Edenvale Industrial Development in the 1998 to 2001 time frame. Prior to the early 1980s, the Coyote Creek area between US Highway 101 and the foothills to the east was primarily in agricultural use. The project site occupied the nose of a northwest trending ridgeline of about Elevation 300 feet (NGVD 1929) with orchards situated on the alluvial floor to the north and south. The nose of the ridge appears to have been used for grazing. Urban encroachment is evident to the west side of Coyote Creek beginning in 1982. By 1986, Hellyer Avenue and Branham Lane (predecessor to Embedded Way) have been extended to the area and the site currently occupied to the north (5225 Hellyer Ave.) has been graded but no structures exist. In the 1990s, the project site and areas to the south have been graded with cuts and fills of unknown depth or height.

In 1999, a request was submitted to the City of San Jose by the developer of 800 Embedded Way to vacate the existing Branham Lane and shift the roadway to the north between Hellyer Avenue and Coyote Creek. The former roadway alignment then became a utility easement and allowed for a larger structure to be built on the parcel. It is unclear whether this action was in response to fault investigations performed in the area which may have required fault setbacks. Grading plans prepared by Kier & Wright (1998) for the roadway relocation indicate dual fault and setback lines that cross the new Embedded Way at an oblique angle based on reference to a Plate in a 1997 Kleinfelder report (as shown on Figure 8). Photos indicated that by April of 2000 the new Embedded Way roadway was being graded and the 800 Embedded Way structure was constructed.

Grading activity on the project site commenced in the 1990s and the adjacent 875 building was constructed. It is unclear as to when significant cutting of the ridge took place or when and why a sliver fill buttress was built on the south side of the site. Photos from 2002 show haul roads leading up to the project site with grading activities (dumping of fill, erosion control measures) evident. These activities were focused on the east end of the project site until 2011, at which time the dumping of fill was mostly on the west end as shown starting in 2014.

3.2 SITE RECONNAISSANCE AND SITE DEVELOPMENT DESCRIPTION

Our engineering geologist performed a reconnaissance of the site and adjacent areas on October 7, 2021. Refer to our Site Plan and Geologic Map and Conceptual Development Plan (Figures 2 and 2A). The site is bounded by existing commercial buildings and parking lots to the north, south and east, and by Coyote Creek to the west. At the time of the reconnaissance, the site was vacant and covered with low grasses and weeds. A paved pedestrian trail flanks the bottom of the west-facing slope adjacent to Coyote Creek. Several mature trees were observed at the northwest corner of the site.

The site has been graded to a near flat surface with the exception of mounds of undocumented fill. Site grades range from between Elevation 244 and 245 feet (datum unknown) within the pad areas to roughly Elevation 205 feet along the toe of the perimeter slope. Numerous 6-inch-diameter PVC pipes protrude from the ground at the top of the southern slope. Some but not all of these pipes may serve as cleanout access for servicing subdrains beneath the sliver fill.

Evidence of previous silt fencing exists along the entire edge of slope on the site. Two gravel driveways consisting of what appears to be ground asphalt extend into the site from two gates at the eastern fence that leads to the 875 Embedded Way facility.

The parcel is bounded by existing natural slopes to the north and west that range from approximately 30 to 35 feet high and are inclined at approximately 2:1 to 3:1 (horizontal:vertical). The southern and eastern slopes appear to be comprised of man-made fill, are approximately 30 and 10 feet high, respectively, and are inclined at approximately 2:1.

3.3 SITE GEOLOGY

The project site represents the last undeveloped parcel in this portion of the Edenvale Development area. As such the parcel provides excellent exposures of surficial soils and bedrock, as summarized below.

3.3.1 Artificial (Man-Made) Fill (af)

Undocumented fill was encountered in some of our explorations. On the building pad area, we encountered between 2 to 5½ feet of undocumented fill of either medium dense to dense clayey sands or hard sandy lean clays. Undocumented fill was also encountered in our boring drilled in the bioretention area (EB-11) that consisted of 14 feet of silty sand with gravel.

The presence of what appears to be subdrain cleanout pipes at the top and toe of the southern slope of the project site indicate the distinct possibility that a fill buttress of unknown width was built in the late 1990s to support development of the access road and parking for 855 Embedded Way. Rodent burrows in the slope have released what appears to be variable sands and gravels indicative of fill. The remnants of a haul road in the slope opposite the corner of 855 Embedded Way is apparent with that road being the dividing line between native bedrock to the west and northwest and an engineered slope to the east. Elsewhere on the project site, fill embankments were observed adjacent to the Coyote Creek trail system as shown on Figure 2. Generalized cross sections A-A′ and B-B′ depicting the surface and subsurface conditions across the site are presented on Figure 5.

3.3.2 Alluvial Soil (Qal)

The floodplain of Coyote Creek contains alluvium of unknown thickness. This alluvium occupies the lowest portions (northwest corner) of the project site that are beyond the development footprint. Alluvial soils were not encountered in our borings.

3.3.3 Serpentinite (sp)

Bedrock of the Cretaceous-Jurassic Franciscan Complex underlies the undisturbed slopes of the western to northern limits of the project site. Larger irregular shaped boulders on the slope are derived from ultramafic rocks consisting of serpentinite and serpentinized dunite and harzburgite. These rock assemblages are sometimes referred to as an ophiolite. A comparison of predevelopment and post development topographic contours suggests that as much as 15 feet of native “cap” material was removed from the rounded knob of the ridgeline thus exposing bedrock on a majority of what is now a flat site as shown on Figure 2.

Serpentinite was encountered in all borings at relatively shallow depths. The serpentinite was generally greenish brown to orange in color, low to moderately hard, weak to moderately strong, deeply weathered and intensely fractured.

Naturally occurring asbestos (NOA) is often associated with ultramafic bedrock such as serpentinite and dunite. Four samples of the bedrock were submitted for analysis and the results indicate trace to 1.25 percent chrysotile fibers detected. The results are presented in Appendix C. Further discussion of the potential impacts due to chrysotile asbestos is presented in the "Conclusions" section of this report and the Phase 1 Environmental Site Assessment prepared by Cornerstone Earth Group under separate cover.

3.2.4 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples of the bioretention boring. Test results were used to evaluate expansion potential of surficial soils. The results of the surficial PI tests indicated PIs ranging from 21 to 27, indicating moderate expansion potential to wetting and drying cycles.

3.2.5 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from optimum to 30 percent over the estimated laboratory optimum moisture.

3.4 GROUNDWATER

Groundwater was not encountered in any of our explorations; however, the borings were not left open instead were immediately backfilled when the boring was completed. Groundwater is likely at or near the Coyote Creek level in nearby alluvial soil areas. Fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

3.5 INFILTRATION TESTING

We previously performed two "quick" Infiltration Tests (P-1 and P-2) to evaluate the infiltration rate of the near surface soil. The tests were performed on October 19, 2021, within the previously drilled explorations at a depth ranging approximately from 5 to 8 feet below the existing grades. The material encountered within our test hole was predominately native bedrock consisting of serpentinite. The explorations were presoaked with water approximately 30 minutes prior to performing the infiltration test. The location of the infiltration test is shown on Figure 2.

Table 2: Summary Infiltration Test Results

Test Number	Depth of Infiltration (feet)	Average Rate of Infiltration (inches per hour)
P-1	7½	1.2
P-2	4⅔	0.6

Infiltration tests resulted in an infiltration rate of 0.6 and 1.2 inches per hour (in/hr). Based on the results of our field tests and experience in the vicinity of the site, we recommend that an infiltration rate of 0.5 in/hr be used for preliminary design of stormwater facilities at the site. Test results may not be truly indicative of the long-term, in-situ permeability. Other factors including stratifications, heterogenous deposits, overburden stress, and other factors can influence permeability results. In addition, for weathered bedrock materials such as those encountered at the site, the average horizontal permeability is typically greater than the average vertical permeability.

We recommend that if any underground infiltration systems are to be constructed, the locations and depth of the systems be further evaluated during the design-level geotechnical investigation or at the time of construction to confirm the above estimates are reasonable. We recommend the project civil engineer review the above information and provide additional recommendations as deemed necessary.

As discussed, the tests were performed at discrete locations and depths. In addition, some disturbance in preparing the tests can occur. Therefore, the above results can vary significantly and may not be representative across the entire site. Localized areas/depths containing higher or lower permeable materials can increase or decrease the actual infiltration rates, respectively. Therefore, we recommend the potential for variations be considered when evaluating the infiltration capacity or performance. In addition, we recommend the project civil engineer give consideration for handling/discharging of water when the infiltration rate is not sufficient or during a large storm event. We also recommend that subsurface water infiltration techniques and/or devices be designed in accordance with local agencies' guidelines and requirements.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above, several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist Priolo Earthquake Fault Zone. Most of the site is located within a Santa Clara County Fault Hazard Zone, or a City of San Jose Potential Hazard Zone. As shown in Figure 4, no known surface expression of active fault traces is thought to cross the site. Further discussion of potential faulting related to the nearby Piercy Fault is presented below.

4.1.1 Previous Consultant's Site-Specific Fault Investigations

A fault investigation was reportedly conducted in the immediate area of the site by Kleinfelder

Inc. (1997) to prior site development. A supplemental report by Kleinfelder (1997b) was submitted later that year. The results of this investigation were mapped onto record civil improvement drawings (Sheets C1.10A to C1.40A) prepared by Kier & Wright Civil Engineers dated 1998. The approximate location of the prior fault mapping and corresponding building exclusion zones are presented conceptually on Figure 8. A brief commentary on the previous findings is presented below.

The primary fault that comes in close proximity to the project site is the Piercy Fault. Information on the Piercy Fault is relatively obscure in the published literature. The fault was originally recognized by Dibblee (1972) based on an exposure of Serpentinite in a fault contact with the Santa Clara Formation over a mile southeast of the project site, although Dibblee shows the fault as concealed beneath alluvium (but not offsetting Holocene age alluvium) along most of its 3.9-mile-long mapped trace. Additional regional mapping has recognized the Piercy Fault trending through the area (Bailey and Everhart, 1964; Dibblee, 1972; City of San Jose, 1983; Helley and Herd, 1990; Dibblee and Minch, 2005). The California Division of Mines and Geology (now the CGS) had previously zoned a portion of the Piercy Fault along Piercy Road in their Alquist-Priolo Special Studies Zone (CDMG, 1974), but was later removed as part of Fault Evaluation Report FER-106 (Bryant, W.R., 1981b).

The most relevant fault investigation conducted in the immediate area is by Kleinfelder (1997a and b) and titled, “Fault Study and Geologic Hazards Assessment, King Ranch.” This report presumably does address the potential for fault rupture on developments on the former King Ranch. Currently this report is not available for review.

As summarized in an Initial Study/Negative Declaration Environmental document (City of San Jose, 2005) for the Rollin Ice development at 800 Embedded Way, local faults in the project area include the Piercy, Coyote, Metcalf and Silver Creek faults. The fault trace for the Piercy Fault has been mapped as running approximately through that project site’s parking lots, between the existing building and Embedded Way. Previous studies (Kleinfelder, 1997) have indicated that the potential for ground rupture or possible deformation associated with the fault cannot be precluded. The site is shown as a Fault Rupture Hazard Zone on the Santa Clara County Geologic Hazard Map. At the time of the previous development (Candescent Technologies, 1998), the shear zone for this fault was designated as a “building exclusion zone”, within which structures designed for human occupancy would not be allowed.

Cornerstone Earth Group conducted an extensive fault investigation in 2016 of the Piercy Fault, 4600 feet to the southwest, as documented in, Cornerstone Earth Group, 2016, “Fault Investigation, 455 Piercy Road, San Jose California,” Project No 920-1-1, dated November 23. This report concluded that “despite the differing interpretations concerning the location of the (Piercy) fault, there is compelling evidence that surface traces of the Piercy Fault project through the subject site”. Building setback lines were recommended for any future habitable structures at the 455 Piercy Road site.

4.1.2 Surficial Evidence Regarding Faulting

It is our opinion that the risk of fault rupture on the project site is low based exclusively on the previously mapped location of the Piercy Fault to the south of the project boundary and which is outside the limits of the development setback lines established by Kleinfelder (1997a and b).

Our site reconnaissance and aerial photo interpretation supports this opinion.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to F_{PGA}*PGA, as allowed in the 2022 edition of the California Building Code when an exception has been taken per ASCE 7-16, Section 11.4.8. For our analysis we used a PGA_M of 0.80g.

4.3 LIQUEFACTION POTENTIAL

The majority of the site is not located within a State-designated Liquefaction Hazard Zone (CGS, San Jose East Quadrangle, 2000) or a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2003). The northwestern and southwestern corners of the site, which are outside the planned development area, are located with a liquefaction hazard zone as shown on Figure 7. We screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 3 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

As discussed in the “Subsurface” section above, the site is underlain primarily by shallow bedrock and localized man-made fills exposed at or near the ground surface. Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically, lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

The site is underlain by shallow bedrock, therefore, the potential for lateral spreading to affect the site is low. The stability of existing natural and man-made slopes is discussed below.

4.5 LANDSLIDING

During our October 7, 2021 geologic reconnaissance by our Certified Engineering Geologist, evidence of slope instability was not observed on either natural or engineered slopes. No observable changes were reported during our recent site exploration in 2023. Soil cover over bedrock on the natural slopes is probably less than 1 foot in thickness and the slopes are flatter than 2:1 (horizontal: vertical), both favorable from a soil creep perspective. The presumption is that the slope on the southern side of the project site is a sliver fill, which may have been built as part of the overall development or possibly to buttress some pre-existing unstable slope. This seems unlikely however as our review of historic aerial photos did not show any slope failures or erosion as far back as 1960.

Santa Clara County and City of San Jose both show a small portion of the project site as being in a landslide hazard zone as shown in Figure 6. The State of California similarly shows the same “nose” at the western end of the project site as being in an earthquake-induced landslide zone (Figure 7). These designations probably result from a moderately steep slope having a free face towards the Coyote Creek drainage.

It is our opinion that this portion of the project site is not at risk for landslides, provided that the slope is not disturbed by future grading and that surface water runoff from new development be directed away from the face of the slope.

4.6 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, an area determined to be outside the 0.2% annual chance of floodplain. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Presence of undocumented fill
- Presence of moderately expansive soils
- Differential settlement due to material transitions
- Potential presence of naturally occurring asbestos (NOA)
- Presence of shallow bedrock

5.1.1 Undocumented Fill

As stated above, the future building pad is partially blanketed up to 5½ feet of fill and localized stockpiled soil, as shown on Figures 2 and 2A. Any fills encountered during site grading should be completely removed from within building areas and to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the "Material for Fill" requirements below, the fills may be reused when backfilling the excavations. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should be screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the "Compaction" section below.

5.1.2 Presence of Moderately Expansive Soils

Moderately expansive surficial soils generally blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. We anticipate the majority of the native soil and bedrock materials will be capped by imported soils needed to raise site grades. Where this is not feasible, to reduce the potential for damage to the planned structures, slabs-on-grade should have sufficient reinforcement and be supported on a layer of non-expansive fill; footings should extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Detailed grading and foundation recommendations addressing this concern are presented in the following sections.

5.1.3 Differential Settlement Due to Material Transitions

Material transitions occur when two or more materials with differing geotechnical characteristics interface in a small area, such as within a building foundation or pavement area. The materials that comprise these transitions can include bedrock, surficial soils, or engineered fill. Because the geotechnical characteristics of the materials are different, the long-term performance of the materials will also be different.

For instance, fills materials, even if well compacted, are typically more compressible than bedrock materials and as a result will usually experience a greater amount of settlement under various loading conditions. The differences in the amount of settlement or expansion between fill materials and bedrock materials can cause distress to residential foundations and other site improvements. Such distress will often either add to the long-term maintenance costs or reduce the design life associated with the structure.

The preliminary grading plan indicates the new building footprint will be entirely underlain by bedrock that will be covered with the planned engineered fills to raise the building pad to roughly Elevation 246 feet. Portions of the parking lot, truck dock apron and drive aisles may have cut/fill transitions. Cut/fill and material transitions should be over-excavated and rebuilt with engineered fill to reduce the potential for differential movement beneath future pavements and slabs. Recommendations addressing this concern is presented in the "Earthwork" section of this report.

5.1.4 Naturally Occurring Asbestos (NOA)

Chrysotile and amphibole asbestos occur naturally in certain geologic settings in the San Francisco Bay area most commonly in serpentinite and other ultramafic rocks. These are igneous and metamorphic rocks with a high content of magnesium and iron minerals. The most common type of asbestos is chrysotile, which is commonly found in serpentinite rock formations. When disturbed by construction, grading, quarrying, or surface mining operations, asbestos-containing dust can be generated. Long-term exposure to asbestos can result in lung cancer, mesothelioma, and asbestosis. As discussed in Section 3, published geologic maps and our current site evaluation indicate ultramafic rocks outcrop across most of the site.

The Asbestos Airborne Toxic Control Measure (ATCM) for Construction, Grading, Quarrying and Surface Mining Operations (California Code of Regulations, Title 17, Section 93105) was signed into State law on July 22, 2002, and became effective in the Bay Area Air Quality Management District (District) on November 19, 2002. The purpose of this regulation is to reduce public exposure to NOA from construction and mining activities that emit dust which may contain NOA.

The Bay Area Air Quality Management District (BAAQMD) locally enforces the ATCM regulation and requires sites that sites with soil detections greater than 0.25% asbestos to prepare an asbestos dust mitigation plan (ADMP). The ATCM requirements are based on the project area. Project areas that are greater than 1-acre are required to submit the ADMP to the BAAQMD for review and comment and are required to implement air monitoring and reporting protocols for the duration of earth disturbing activities.

As previously discussed, four samples of the bedrock were submitted for analysis and the results indicate trace to 1.25 percent chrysotile fibers detected. The results are presented in Appendix C. Further discussion of the potential impacts due to chrysotile asbestos is presented in the Phase 1 Environmental Site Assessment prepared by Cornerstone Earth Group under separate cover.

5.1.5 Shallow Bedrock Excavations

Prior grading activities at the site exposed serpentinite bedrock across most of the site. We assume that conventional earthwork equipment can be used for most of the planned site development. However, our site exploration indicates some localized hard, resistant rock may be encountered during the grading operations. Removal and excavation of resistant rock will depend upon the planned excavation depths, the types of equipment used, and effort put forth by the contractor. Consideration could be given to over-excavating resistant rock during the grading operations to a depth of about 1 foot below the lowest utility in planned utility corridors.

It has been our experience that bedrock over-excavation, although it increases mass grading costs, reduces potential cost overruns during utility construction. Our exploration indicates that the upper approximately 3 to 5 feet of the bedrock, where encountered, is moderately to severely weathered and behaves as a dense soil. Below these depths, the rock becomes harder, less weathered, and more resistant to excavation with lightweight equipment. Excavations for retaining wall foundation around the perimeter of the site will also encountered bedrock. Once retaining wall types are finalized, we should review the wall plans and details to confirm whether additional benching or bedrock over-excavation is required.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE CLEARING AND PREPARATION

6.1.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements to be removed within the proposed development area. Demolition of existing improvements is discussed in the prior paragraphs. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 2 to 3 inches below existing grade in vegetated areas.

6.1.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the "Compaction" section of this report.

6.1.3 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risk for owners associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout.

6.2 RE-COMPACTION OF UNDOCUMENTED FILLS

As discussed in the “Conclusions” section, up to 5½ feet of undocumented fill was encountered within the future building pad during our site investigation. Prior to placement of new site fills within the proposed building pad or future parking or truck dock areas, all undocumented fill should be over-excavated and re-compacted. For fills extending into the planned bioretention area, the upper 12 inches of fill below pavement or flatwork subgrade should be over-excavated and re-compacted. Once fill excavations are completed, the excavation bottom should be scarified at least 6 inches, then the exposed weathered bedrock should be moisture conditioned and re-compacted as engineered fill. The actual depth of fill over-excavation should be confirmed based on final grading plans and updated topographic data.

6.3 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 10 feet at the site may be classified as OSHA Soil Type B materials. A Cornerstone representative should be retained to confirm the preliminary site classification.

Excavations performed during site demolition and fill removal should be sloped at 2:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Actual excavation inclinations should be reviewed in the field during construction, as needed. Excavations below building subgrade and excavations in pavement and flatwork areas should be sloped in accordance with OSHA soil classification requirements.

6.4 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

6.5 WET SOIL STABILIZATION GUIDELINES

Native soil and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the site conditions.

6.5.1 Scarification and Drying

The subgrade may be scarified to a depth of 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.5.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthetic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.6 MATERIAL FOR FILL

6.6.1 Re-Use of On-site Soils

On-site soils and bedrock materials with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.6.2 Potential Import Sources

Imported fill for general use and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the habitable building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date.

Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant’s review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.7 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches and consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the “Wet Soil Stabilization Guidelines” section of this report.

Table 2: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site or Imported Soils	90	>2
General Fill (below a depth of 5 feet)	On-Site or Imported Soils	95	>2
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)

6.8 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

6.9 PERMANENT CUT AND FILL SLOPES

All permanent cut and fill slopes in soil or bedrock should have a maximum inclination of 2:1 (horizontal:vertical) for slopes up to 10 feet high; slopes greater than 10 feet should be inclined at no greater than 2.5:1. Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. Refer to the “Erosion Control” section of this report for a discussion regarding protection of slope surfaces.

6.9.1 Keyways and Benches

The preliminary grading plans referenced in Section 1 indicate site fills will be supported by new retaining walls that will be constructed on existing bedrock or previously placed man-made fill slopes. If retaining walls will be constructed at mechanically stabilized earth (MSE) walls, then existing slopes will need to be keyed and benched prior to wall construction to provide adequate stability for the new wall systems. In general, fill placed on existing ground inclined at 6:1 or greater should be benched into the existing slope and a keyway constructed at the toe of the fill. Benches should be angled slightly into the slope and be spaced vertically at no greater than 5 to 8 feet between benches and be at least 4 feet wide. Depending on the thickness of any residual soil layer that blankets the bedrock, the benches may need to be widened beyond the minimum width to extend into competent bedrock. The keyway should also be angled slightly into the slope (minimum 2 percent inclination), extend into competent soil orbedrock, and be at least 10 feet wide. The actual keyway width will depend on the type of wall system and type of compaction equipment used by the contractor. A typical retaining wall key and benching is depicted in Figure 9.

6.9.2 Fill Drainage

A permanent subsurface drainage system consisting of a series of perforated gravity pipes or drainage strips should be constructed between engineered fill placed against a bedrock slope and within all keyways. This system is intended to intercept perched water flowing through the bedrock and transmit it to suitable outlet structures and reduce the potential for hydrostatic pressures building up behind the fills and causing slope instability. The drain lines should be placed at the back of the keyways and benches.

The drainage system should be constructed in small trenches or v-ditches as shown in Figure 9, and will consist of a minimum 4-inch-diameter perforated SDR 35 (perforations placed downward), bedded and shaded in Caltrans Class 2 Permeable Material (latest version) or ¾-inch crushed rock; if crushed rock is used, the rock should be encapsulated in filter fabric (Mirafi 140N or equivalent). The bedding should be at least 2 inches, and the trench should be at least 8 inches in width and depth. Alternatively, geocomposite strip drains may be used. All drainage lines should slope towards suitable outlet structures at an inclination of at least 0.5 percent. Suitable outlet structures may consist of connecting the drainage lines to a storm drain system, with a sump if required; if the drain lines will outlet overland at the toe of the slope, an appropriate rock spill pad should be provided; the drain lines should not outlet onto the slope.

Vertical cleanouts should be provided at all upslope ends of the drainage lines and at all 90-degree bends.

6.9.3 Plan Review and Construction Monitoring

We should be retained to review the final grading and sub-drainage plans and we can provide more specific input regarding the location of keyways, benches and fill drainage for the final plans. A Cornerstone representative should be on site during keyway and fill slope construction. Field modifications to the planned keyway and benching may be required based on encountered field conditions.

We recommend that the project civil engineer or land surveyor be retained to survey in place all keyways, sub-drainage lines, solid pipes, and cleanouts, and create an as-built plan. This plan will be of use for any future maintenance or repair work.

6.10 SITE DRAINAGE

Surface runoff should not be allowed to flow over the top of or pond at the top or toe of engineered slopes or retaining walls. Ponding should also not be allowed on or adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. These facilities are not recommended where stormwater infiltration may affect slopes at lower elevations on or adjacent to the site. However, if slopes are not present at lower elevations that could potentially be affected, and if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

6.11 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project’s drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface conditions at the site consist of weathered bedrock that will covered with a few to several feet of imported fill material having PI of 15 or less. Site specific infiltration rates were estimated to be approximately 0.5 inches per hour.
- Locally, seasonal high groundwater isn't mapped but expected to be greater than 30 feet, and therefore is expected to be at least 10 feet below the base of the infiltration measure.

6.11.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

6.11.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding expansive clay.
- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.
- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

6.11.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.
- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12-inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

6.11.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth, or

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the “Retaining Walls” section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

6.12 PERMANENT EROSION CONTROL MEASURES

Hillside grading will require periodic maintenance after construction to reduce the potential for erosion and sloughing. At a minimum, all newly graded or disturbed slopes should be vegetated by hydroseeding or other landscape ground cover. The establishment of vegetation will help reduce runoff velocities, allow some infiltration and transpiration, trap sediment within runoff, and protect the soil from raindrop impact. Depending on the exposed material type and the slope inclination, more aggressive erosion control measures may be needed to protect slopes for one or more winter seasons while vegetation is establishing. For slopes with inclinations of 2:1 (horizontal:vertical) or greater, erosion control may consist of jute netting, straw matting, or erosion control blankets used in combination with hydroseeding.

Both construction and post-construction Storm Water Pollution Prevention Plans (SWPPPs) should be prepared for the project-specific requirements. We recommend that final grading plans be provided for our review.

6.13 LANDSCAPE CONSIDERATIONS

We recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. This can typically be achieved by:

- Using drip irrigation
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers
- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 7: 2022 CBC SEISMIC DESIGN CRITERIA

7.1 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2022 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system.

Our explorations generally encountered alluvial deposits to a depth of 1 foot below the existing surface overlying serpentinite. Based on our borings and review of local geology, the site is underlain by shallow rock with typical SPT "N" values above 50 blows per foot. Therefore, we have classified the site as Soil Classification C. The mapped spectral acceleration parameters S_s and S_1 were calculated using the web-based program ATC Hazards by Locations, located at https://hazards.atcouncil.org/, based on the site coordinates presented below and the site classification. Recommended values for design are presented in Table 4. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.267357°
Site Longitude	-121.793634°
0.2-second Period Mapped Spectral Acceleration[1], S_S	1.576g
1-second Period Mapped Spectral Acceleration[1], S_1	0.600g
Short-Period Site Coefficient – Fa	1.2
Long-Period Site Coefficient – Fv	1.4
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	1.891g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.840g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.260g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.560g
Site Amplification Factor at PGA – F_{PGA}	1.2
Site Modified Peak Ground Acceleration – PGA_M	0.795

SECTION 8: FOUNDATIONS

8.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structures may be supported on shallow foundations provided the recommendations in the “Earthwork” section and the sections below are followed.

8.2 SHALLOW FOUNDATIONS

8.2.1 Conventional Shallow Footings – Future Warehouse and Retaining Walls on Bedrock

Conventional shallow footings for the warehouse and retaining walls should bear on natural, undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 18 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Footings constructed to the above dimensions and in accordance with the “Earthwork” recommendations of this report are capable of supporting maximum allowable bearing pressures of 3,000 psf for dead loads, 4,500 psf for combined dead plus live loads, and 6,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.2.2 Conventional Shallow Footings – Retaining Walls on Artificial Fill

Conventional shallow footings for retaining walls constructed on existing undocumented fill (southern edge of development area) should bear on undisturbed soil or engineered fill, be at least 18 inches wide, and extend at least 24 inches below the lowest adjacent grade. If wall footings are constructed on existing fill slopes, the depth of the footing should be deepened such that at least 6 feet soil is maintained between the bottom edge of the footing and face of slope. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Footings constructed to the above dimensions and in accordance with the “Earthwork” recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

8.2.3 Footing Settlement

Structural loads were not provided to us at the time this report was prepared; therefore, we assumed the typical loading in the following table.

Table 4: Assumed Structural Loading

Foundation Area	Range of Assumed Loads
Interior Isolated Column Footing	100 kips
Perimeter Strip Footing	3 to 5 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above, we estimate that the total static footing settlement for the warehouse will be on the order of ½-inch, with less than ¼-inch of post-construction differential settlement between adjacent foundation elements. For retaining walls on bedrock cut, we anticipate settlement to be less than ½ inch to negligible. Retaining walls on previously placed undocumented fill (southern edge of site) we estimate settlement on the order of ½ to ¾ inch, with differential settlement up to ¼ to ½ inch across this fill area. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading and verify the settlement estimates above.

8.2.4 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footings and the supporting subgrade, and also by passive pressures generated against deepened footing edges. For warehouse footings supported on compacted fill overlying bedrock, an ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. For retaining walls on bedrock, an ultimate frictional resistance of 0.50 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 500 pcf may be used in design for footings or keys set back at least 6 feet from face of existing slopes. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. The upper 12 inches of soil should be neglected when determining passive pressure capacity unless the surrounding area is capped with pavement or flatwork.

8.2.5 Conventional Shallow Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence, or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

SECTION 9: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

9.1 WAREHOUSE SLABS-ON-GRADE

Warehouse slabs-on-grade should be at least 6 inches thick and should have a minimum compressive strength of 3,500 psi. At this time, rack loading information, etc. was not available. The slab should be designed for the specific warehouse loading (i.e., Forklifts, rack loads, etc., and should also be designed to accommodate potential slab settlement beneath heavily loaded areas (i.e., rack loading). We recommend we be retained to review the final layout and loading of the heavily loaded areas and provide estimated settlements. The warehouse slab should also be supported on at least 6 inches of non-expansive, crushed granular base having an R-value of at least 50 and no more than 10 percent passing the No. 200 sieve, such as Class 2 aggregate base or subbase. All base and sub-base materials should be placed and compacted in accordance with the “Compaction” section of this report. If there will be areas within the warehouse that are moisture sensitive, such as equipment and elevator rooms, a vapor barrier may be placed over the upper granular base prior to slab construction. Please refer to the recommendations in the “Interior Slabs Moisture Protection Considerations” section for vapor barrier construction. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

9.2 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 4 inches of non-expansive fill overlying subgrade prepared in accordance with the “Earthwork” recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the “Vehicular Pavements” section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 10: VEHICULAR PAVEMENTS

10.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 8. The design R-value was chosen based on the results of the laboratory testing performed on a surficial sample collected from the proposed pavement area and engineering judgment considering the variable surface conditions. Additionally, due to the anticipated low R-value of the existing and potential import soils, we have also included an option for lime-treated subgrade soils using an estimated design R-value of 40. If considered, additional laboratory testing should be performed during mass grading to confirm the design R-value of the treated pavement subgrade.

Table 5A: Asphalt Concrete Pavement Recommendations

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base[1] (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	7.5	10.0
4.5	2.5	9.0	11.5
5.0	3.0	9.5	12.5
5.5	3.0	11.5	14.5
6.0	3.5	12.0	15.5
7.0	4.0	15.0	19.0
8.0	5.0	17.0	22.0
9.0	6.0	19.0	25.0

[1] Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 8

Table 5B: Asphalt Concrete Pavement Recommendations, Lime Treated Subgrade

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base[1] (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	4.0	6.5
4.5	2.5	4.0	6.5
5.0	3.0	4.0	7.0
5.5	3.0	5.0	8.0
6.0	4.0	6.0	10.0
7.0	4.0	7.0	11.0
8.0	5.0	8.0	13.0
9.0	6.0	9.0	15.0

[1] *Caltrans Class 2 aggregate base; minimum R-value of 78; subgrade R-value of 40 assuming 12-inch-thick subgrade treatment with at least 3 percent high-calcium quicklime*

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will use the pavements.

10.2 PORTLAND CEMENT CONCRETE

The Portland Cement Concrete (PCC) pavement recommendations outlined below are based on methods presented in American Concrete Pavement Association (ACPA, 2006). In our opinion, the truck loading areas where trucks turn, brake, or stop should be constructed of reinforced PCC pavement. We have provided a few pavement alternatives below as the anticipated number of trucks and number of load repetitions per day on a given location of the pavement has not been provided at this time. An alternative should be chosen that is greater than what is expected for the development. At this time, we have assumed trucks will consist of tractor trailers. When more specific truck loading information is available, additional information can be provided.

Table 6: PCC Pavement Recommendations

Traffic Category	Minimum PCC Thickness[1] (inches)	Class 2 Aggregate Base (inches)
Maximum ADTT = 20	6.5	6.0
Maximum ADTT = 80	7.0	6.0
Maximum ADTT = 400	7.5	6.0

[1] Subgrade design R-Value = 8

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

10.2.1 Stress Pads for Trash Enclosures

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed on Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be increased to a minimum PCC thickness of 7 inches. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

SECTION 11: RETAINING WALLS

11.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls be designed for the following pressures:

Table 7: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	Lateral Earth Pressure*	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	40 pcf	40 pcf + 8H
3:1	55 pcf	55 pcf + 8H
2½:1	60 pcf	60 pcf + 8H
2:1	65 pcf	65 pcf + 8H
Additional Surcharge Loads	$^1/_3$ of vertical loads at top of wall	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure
** H is the distance in feet between the bottom of footing and top of retained soil

If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

11.2 SEISMIC LATERAL EARTH PRESSURES

11.2.1 Site Walls

The 2022 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. Because walls greater than about 6 feet are planned, and peak ground accelerations greater than 0.40g are expected, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2022 CBC.

The CBC prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With respect to the load from lateral earth pressure and groundwater pressure, the CBC prescribes the basic combinations shown in CBC equations 16-2 and 16-7 below.

$$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \qquad [\text{Eq. 16-2}]$$

In Eq. 16-2: H - should represent the total static lateral earth pressure, which for the site walls will be unrestrained (use 45 pcf)

$$0.9(D + F) + 1.0E + 1.6H \qquad [\text{Eq. 16-7}]$$

In Eq. 16-7: H - should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf)

E - should represent the seismic increment component in Eq. 16-7, a triangular load with a resultant force of $12H^2$, which should be applied one third of the height up from the base of the wall (and which can also be expressed as an equivalent fluid pressure equal to 24 pcf).

The interim recommendations in the SEAOC paper more appropriately split out "active" earth pressure from the seismic earth pressure increment so that different load factors can be applied in accordance with different risk levels.

11.3 WALL DRAINAGE

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer’s connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

11.4 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

11.5 FOUNDATIONS

Retaining walls may be supported on a continuous footing designed in accordance with the recommendations presented in the “Foundations” section of this report.

11.6 SEGMENTED WALLS

Any segmented walls that will be constructed on site should be designed in accordance with the soil parameters below. Where segmented walls will retain more than about 3 feet, requiring geogrid reinforcing elements, we recommend the use of wall systems with pinned connections between blocks, such as Keystone or Versalok wall systems, not just angled blocks that rely on gravity. These values may need to be field-revised based on review of import soil and the existing materials in the wall alignment area, as needed. All walls should be designed to include permeable granular fill behind the walls with an appropriate outlet.

Table 8: Recommended Soil Parameters – Segmented Walls

Material Type	Friction Angle (degrees)	Cohesion (psf)	Soil Moist Unit Weight (pcf)
Reinforced Soil Zone	32	500	120
Retained Soil Zone	32	500	120
Foundation Soil Zone (Bedrock Cut)	34	0	115
Foundation Soil Zone (Existing Fill Slope)	30	100	120

The above assumes that the reinforced and retained soil zone will generally consist of excavated bedrock cut materials or imported fill. These values also assume the foundation soil zone fill will consist of bedrock (northern or western edges of site) or existing undocumented fill materials (southern edge of site). Select fill in the reinforced soil zone should be relatively non-expansive, consisting predominantly of import fill soil or bedrock cut materials with less than 30 percent passing the No. 200 sieve and a Plasticity Index (PI) of 15 or less. The manufacturer's recommendations should be followed regarding design and construction. We should be retained to review the design calculations, plans and details for conformance with project requirements. Depending on the size and location of planned walls, segmented walls may need to be constructed with a base keyway to improve the global stability of the walls (refer to Figure 9). Further analysis should be performed once wall height and locations have been finalized.

SECTION 12: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Oppidan specifically to support the design of the project located at Embedded Way Industrial Building in San Jose, California. The opinions, conclusions, and preliminary recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and ground water conditions encountered during our limited subsurface exploration. Preparation of a design-level investigation is anticipated to provide additional information and refine the preliminary recommendations presented herein. If variations or unsuitable conditions are encountered during the construction phase, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Oppidan may have provided Cornerstone with plans, reports and other documents prepared by others. Oppidan understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 13: REFERENCES

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Aerial Photographs

Table 1 - List of Reviewed Aerial Photos

Source	Form	Scale	Date	Flight	Line	Frames
Google Earth	Color	Various	9/26/2020	N/A	N/A	N/A
Google Earth	Color	Various	4/5/2016	N/A	N/A	N/A
Google Earth	Color	Various	10/31/2011	N/A	N/A	N/A
Google Earth	Color	Various	9/30/2002	N/A	N/A	N/A
Google Earth	Color	Various	3/28/2000	N/A	N/A	N/A
Google Earth	Black & White	Various	9/12/1998	N/A	N/A	N/A
PAS	Black & White	1:12000	7/31/1996	5200	32	84
PAS	Black & White	1:12000	6/2/1994	4625	32	81, 82, 83
PAS	Black & White	1:12000	7/24/1990	3845	30	82, 83
PAS	Black & White	1:33600	6/30/1986	2881	9	10, 11
PAS	Black & White	1:12000	4/30/1982	2135	18	20, 21, 22
PAS	Black & White	1:12000	11/4/1976	1277	18	17, 18
PAS	Black & White	1:12000	5/23/1974	1138	18	19, 20
PAS	Black & White	1:12000	10/12/1971	1006	18	21, 22
PAS	Black & White	1:12000	10/12/1971	1006	19	16, 17
PAS	Black & White	1:36000	7/23/1963	550	16	40, 41
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Notes: PAS – Pacific Aerial Surveys

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CORNERSTONE
EARTH GROUP

Vicinity Map

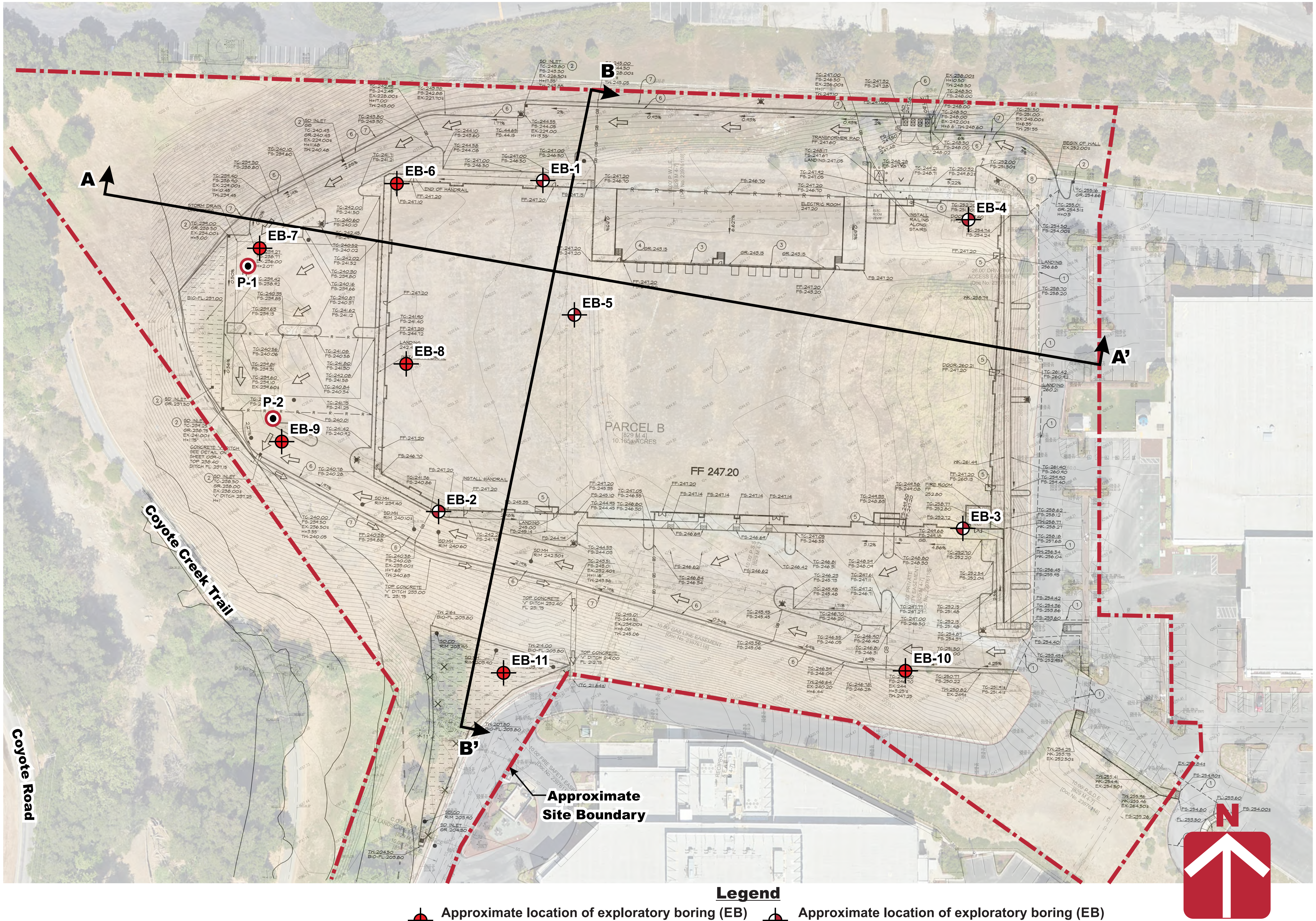
865 Embedded Way
San Jose, CA

Project Number
1345-1-3

Figure Number
Figure 1

Date
April 2023

Drawn By
RRN



Legend

Approximate location of exploratory boring (EB) (Cornerstone, April 2023)	Approximate location of exploratory boring (EB) (Cornerstone, October 2021)
A A' Approximate location of cross section	Approximate location of infiltration test (P) (Cornerstone, October 2021)

0 80 160
APPROXIMATE SCALE (FEET)

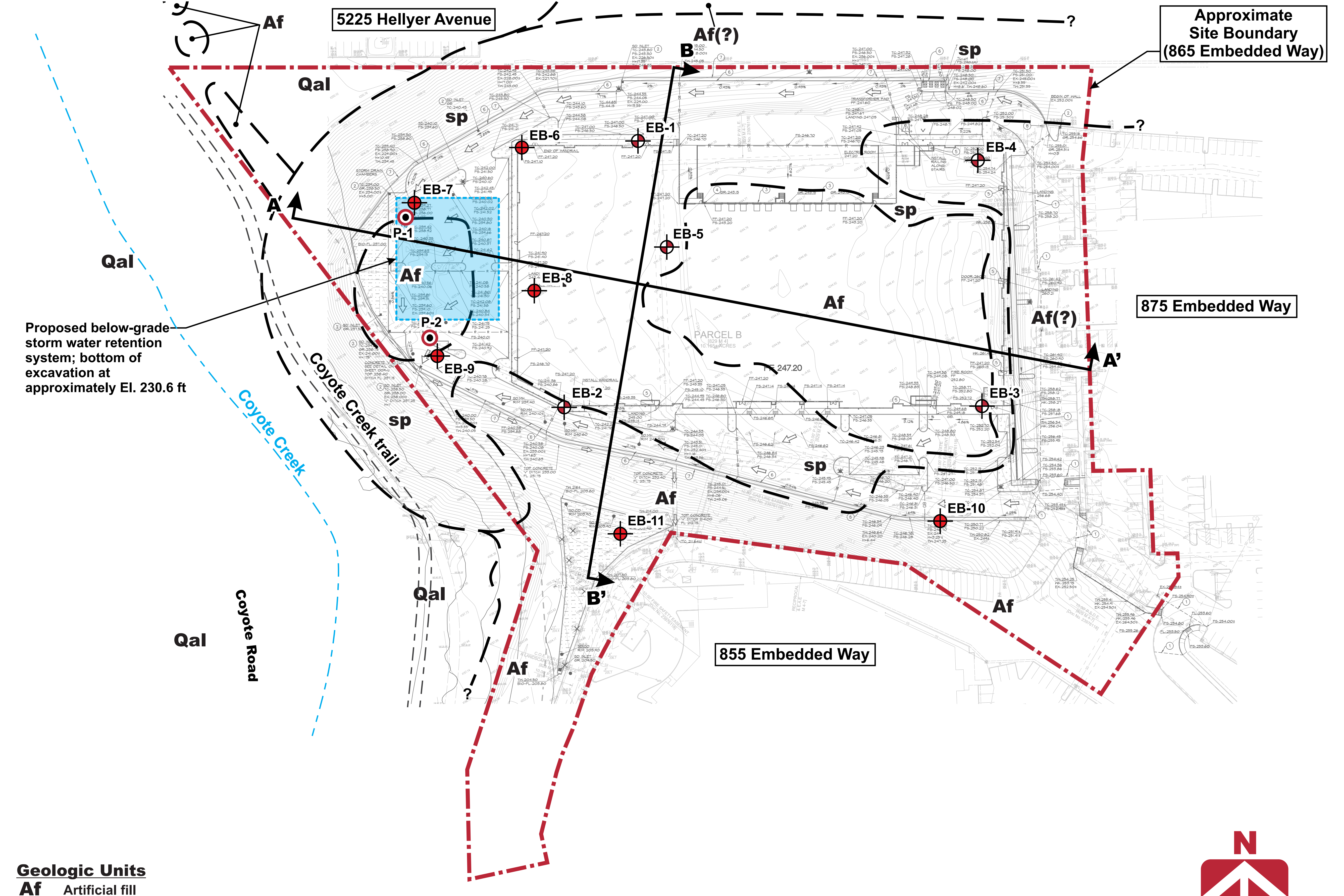
Base by Google Earth, dated 03/10/2022
Overlay by AMS Associates, Inc., Grading Plan - 006-C, dated 03/17/2023

Project Number 1345-1-3
Figure Number Figure 2
Drawn By RRN
Date April 2023

Site Plan

865 Embedded Way
San Jose, CA

CORNERSTONE
EARTH GROUP



Geologic Units

Af	Artificial fill
Qal	Alluvium
sp	Serpentinite Bedrock Ophiolite

Legend

?— —	Approximate geologic contract, queried where uncertain
⊕	Approximate location of exploratory boring (EB) (Cornerstone, April 2023)
⊕	Approximate location of exploratory boring (EB) (Cornerstone, October 2021)
A A'	Approximate location of cross section
◉	Approximate location of infiltration test (P) (Cornerstone, October 2021)

Base by AMS Associates, Inc., Grading Plan - 006-C, dated 03/17/2023

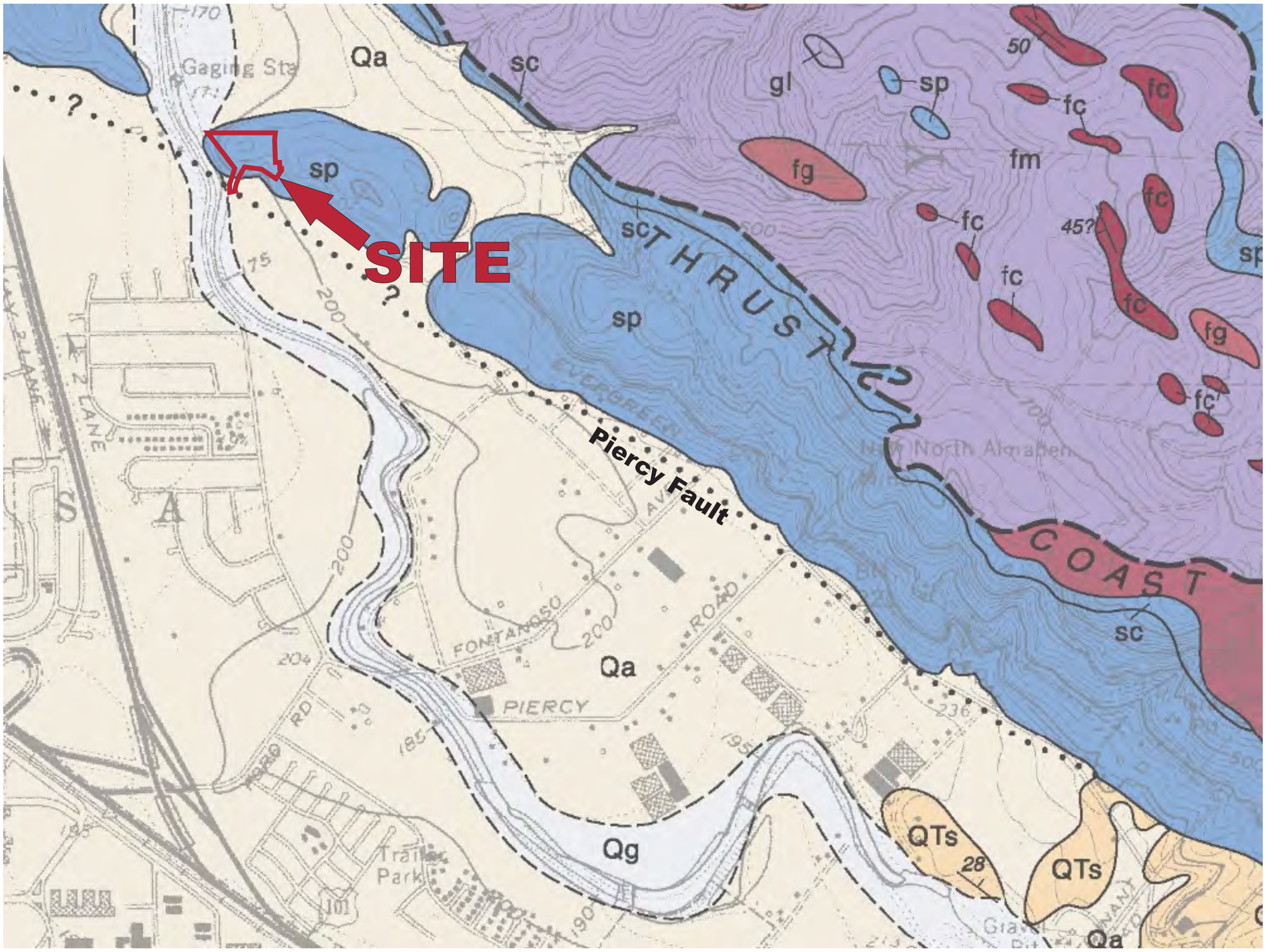
0 100 200
APPROXIMATE SCALE (FEET)

Project Number 1345-1-3
Figure Number Figure 2A
Drawn By RRN
Date April 2023

Conceptual Development Plan

865 Embedded Way
San Jose, CA

CORNERSTONE EARTH GROUP

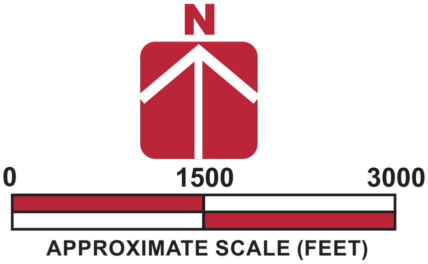


Geologic Units

Qg	**Surficial Sediments** (Holocene) Sand and gravel of Coyote Creek channel
Qa	Alluvial gravel, sand and clay soil of valley areas
QTs	**Santa Clara Formation** (Late Pliocene)
sp	**Coast Range Ophiolite Complex** (Late Jurassic and Cretaceous) Serpentinite
sc	Serpentinite altered to ferruginous silica-carbonate rock
fm	**Franciscan Assemblage** (Late Jurassic and Cretaceous) Melange
fc	Chert
fg	Greenstone

Explanation

Contact - dashed where approximate, dotted where concealed



Base: USGS, Geologic Map of the San Jose East Quadrangle, Santa Clara County, California, by Dibblee and Minch, 2005

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Regional Geologic Map

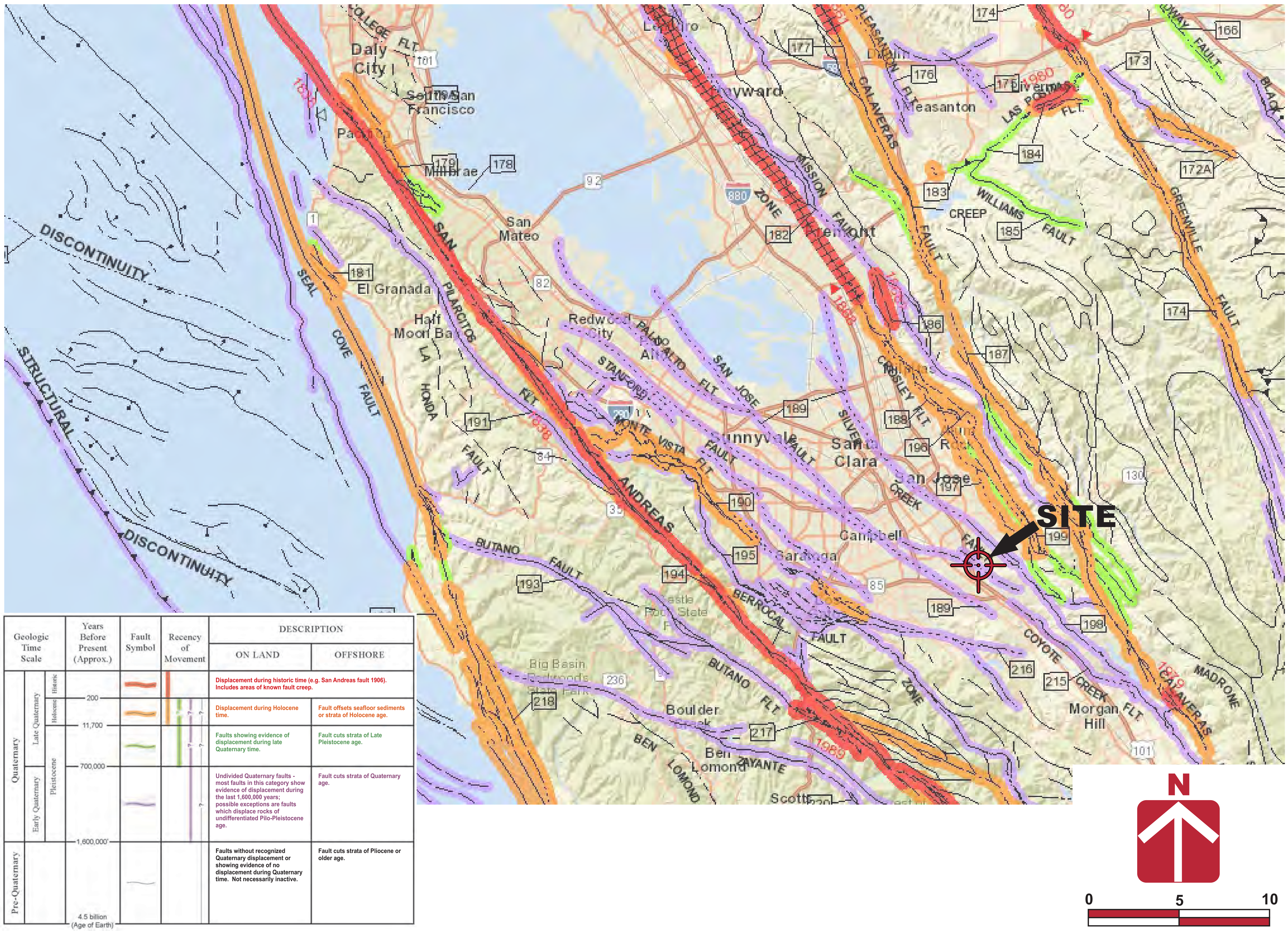
865 Embedded Way
San Jose, CA

Project Number
1345-1-3

Figure Number
Figure 3

Date
April 2023

Drawn By
RRN



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic		🟥		Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200	🟧		Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700	🟩		Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000	🟪		Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Pilo-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000 4.5 billion (Age of Earth)	—		Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Project Number
1345-1-3

Figure Number
Figure 4

Drawn By
RRN

Date
April 2023

Regional Fault Map

865 Embedded Way
San Jose, CA

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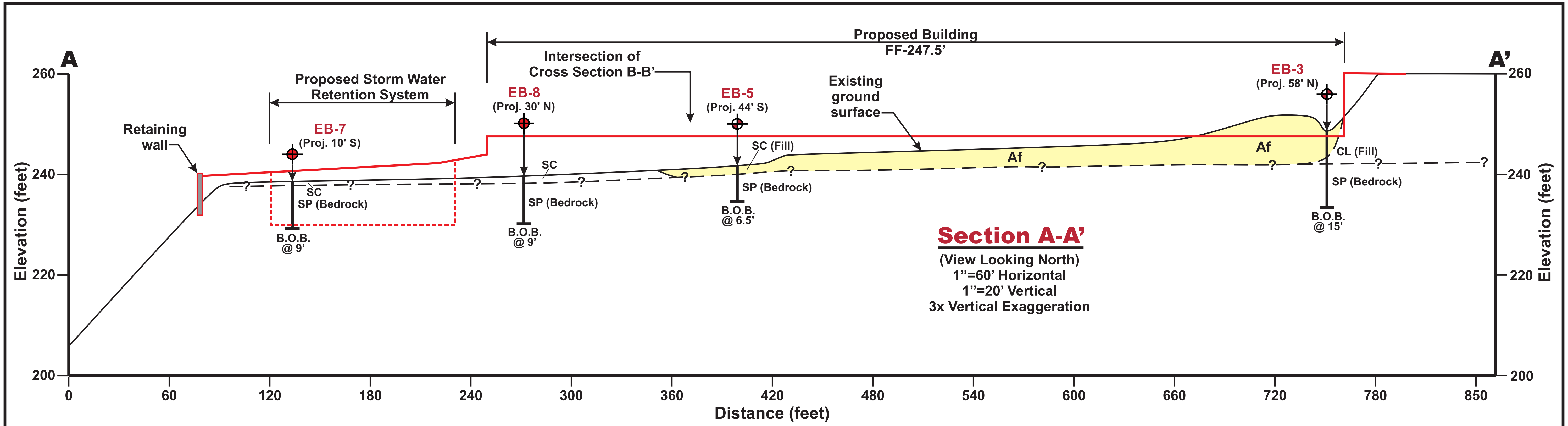
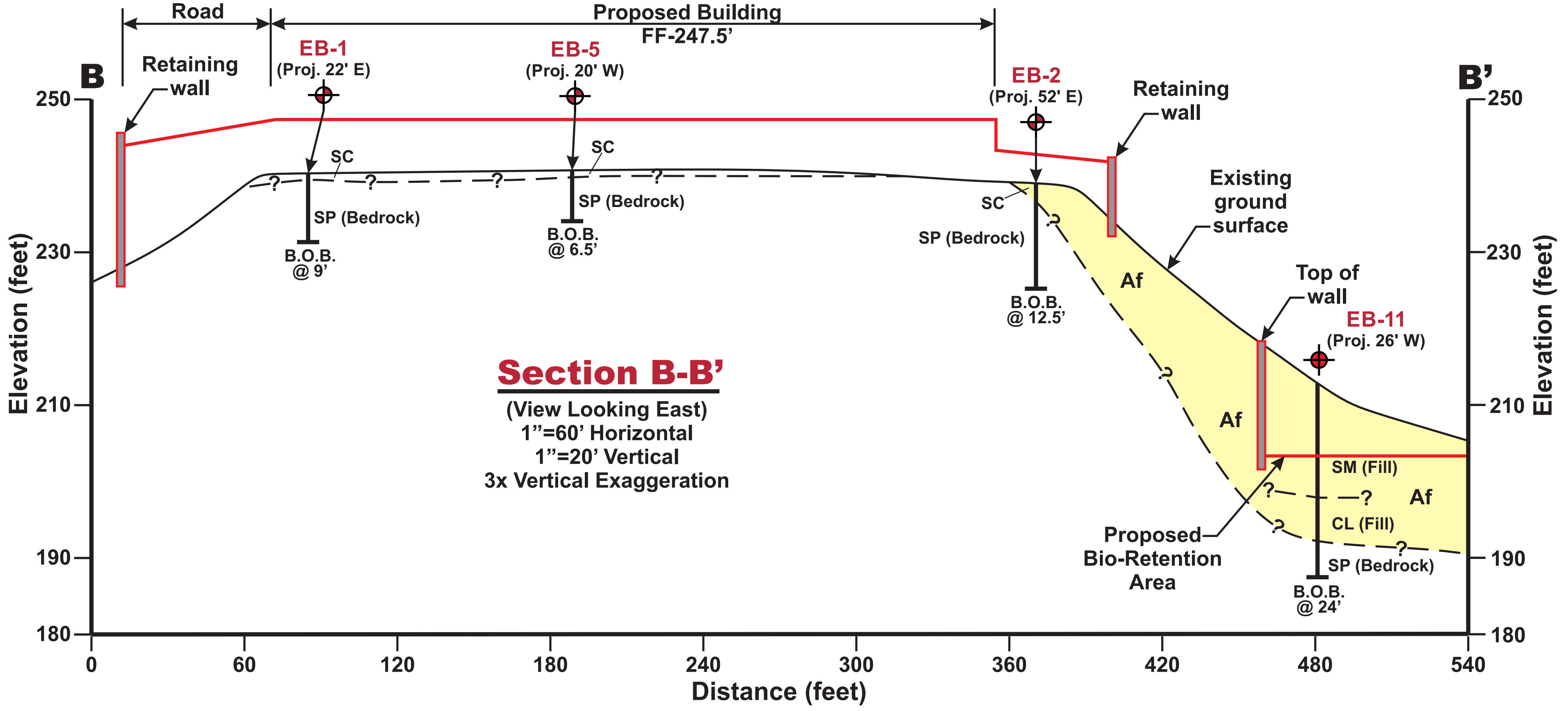


Section A-A'

(View Looking North)
1"=60' Horizontal
1"=20' Vertical
3x Vertical Exaggeration



Section B-B'

(View Looking East)
1"=60' Horizontal
1"=20' Vertical
3x Vertical Exaggeration

Symbols

Af	Artificial (undocumented) fill
CL	Lean Clay
SC	Clayey Sand (Weathered Bedrock)
SM	Silty Sand (Fill)
sp	Serpentinite (Bedrock)
⊕	Approximate location of exploratory boring (EB) (Cornerstone, April 2023)
⊕	Approximate location of exploratory boring (EB) (Cornerstone, October 2021)

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 2 for location of cross section.

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Geologic Cross Section A-A'

865 Embedded Way
San Jose, CA

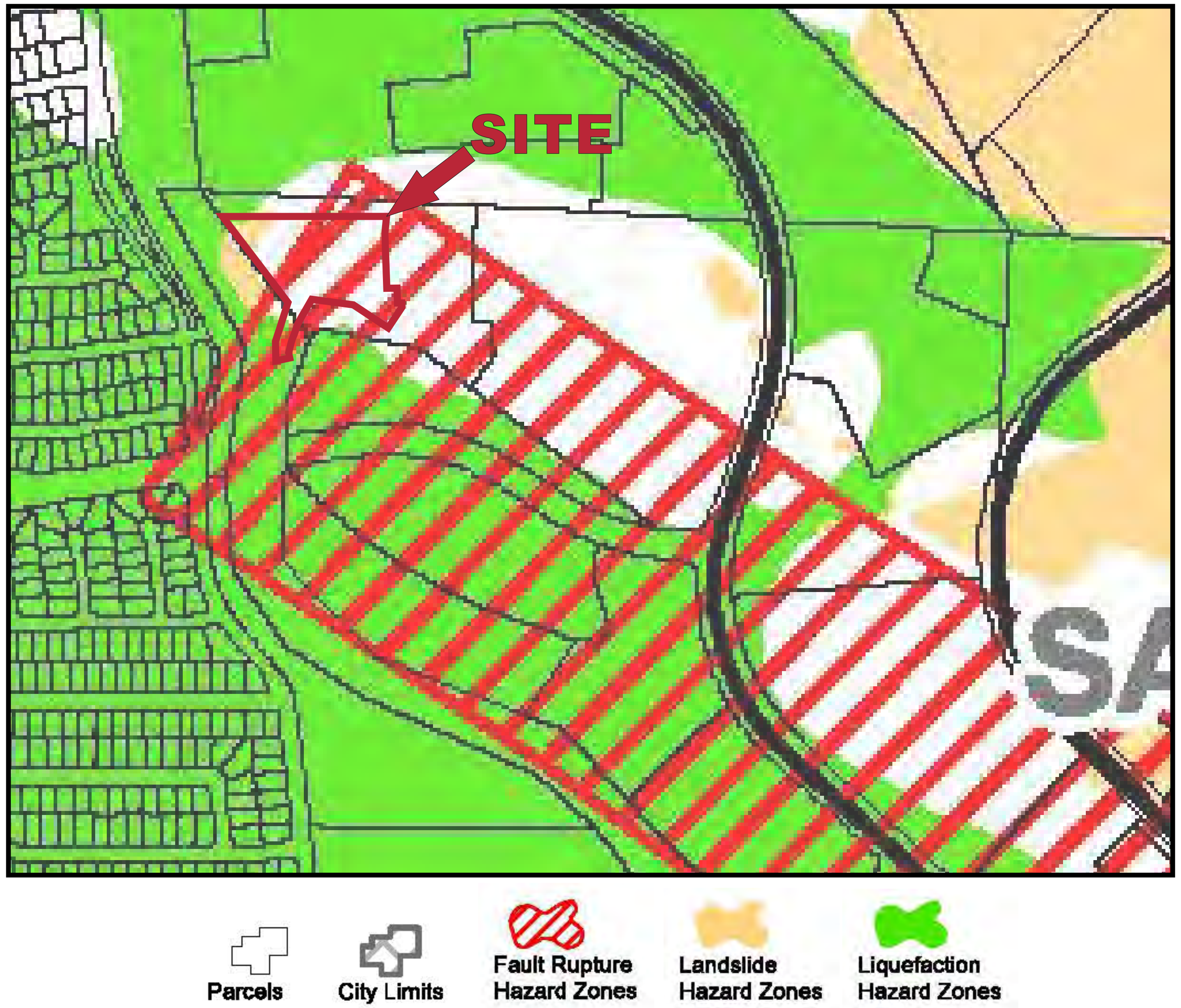
Project Number
1345-1-3

Figure Number
Figure 5

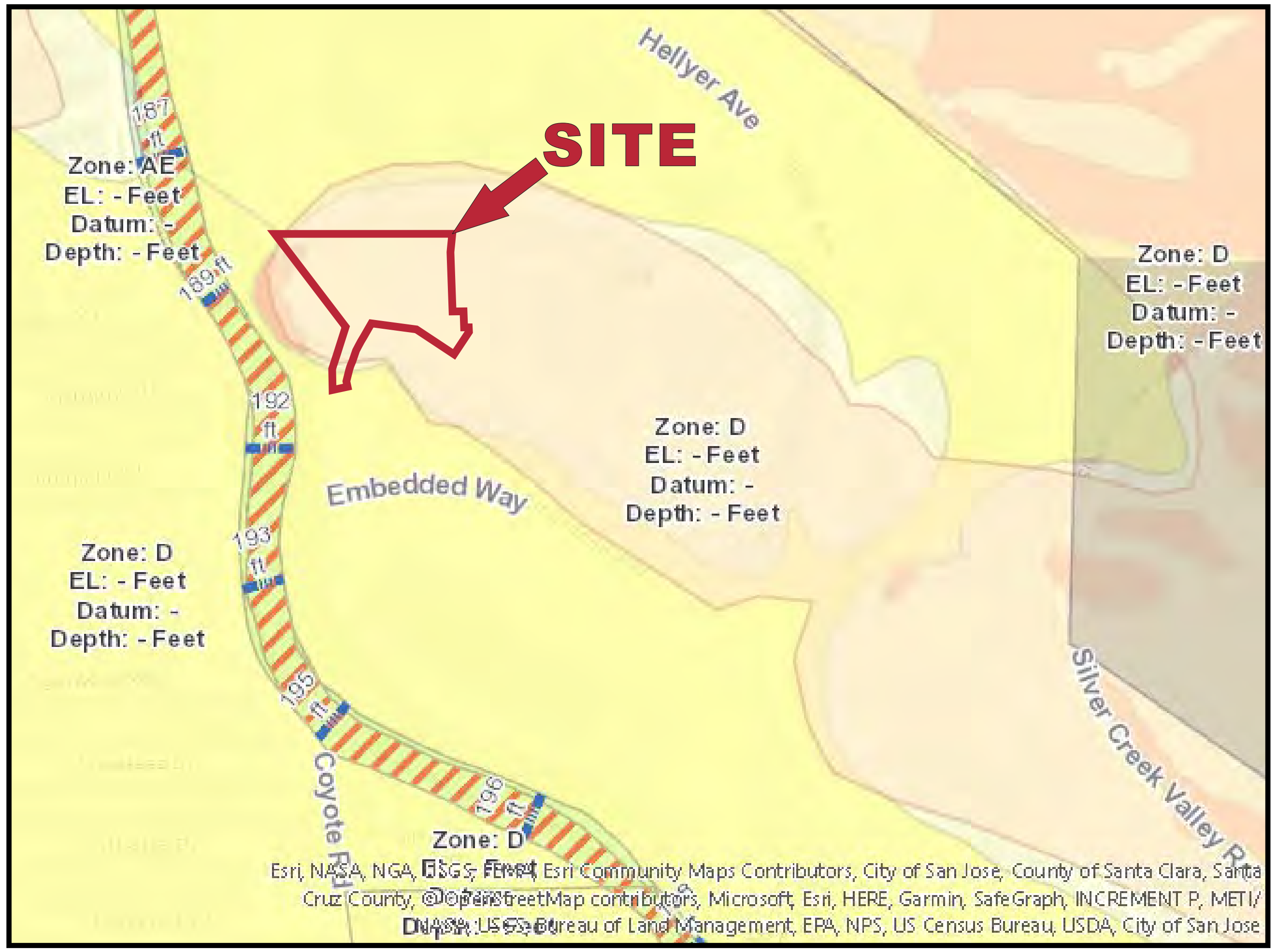
Date
April 2023

Drawn By
RRN

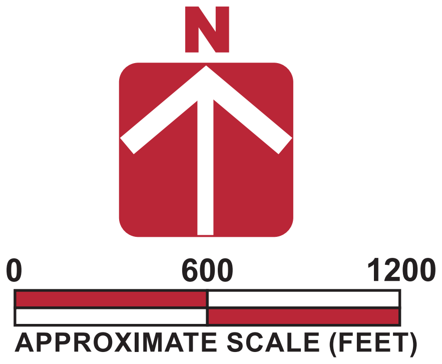
Santa Clara County Geologic Hazard Zones



City of San Jose Geologic Hazard Zones



Esri, NASA, NGA, USGS, FEMA, Esri Community Maps Contributors, City of San Jose, County of Santa Clara, Santa Cruz County, © OpenStreetMap contributors, Microsoft, Esri, HERE, Garmin, SafeGraph, INCREMENT P, METI/NASA, USGS, Bureau of Land Management, EPA, NPS, US Census Bureau, USDA, City of San Jose



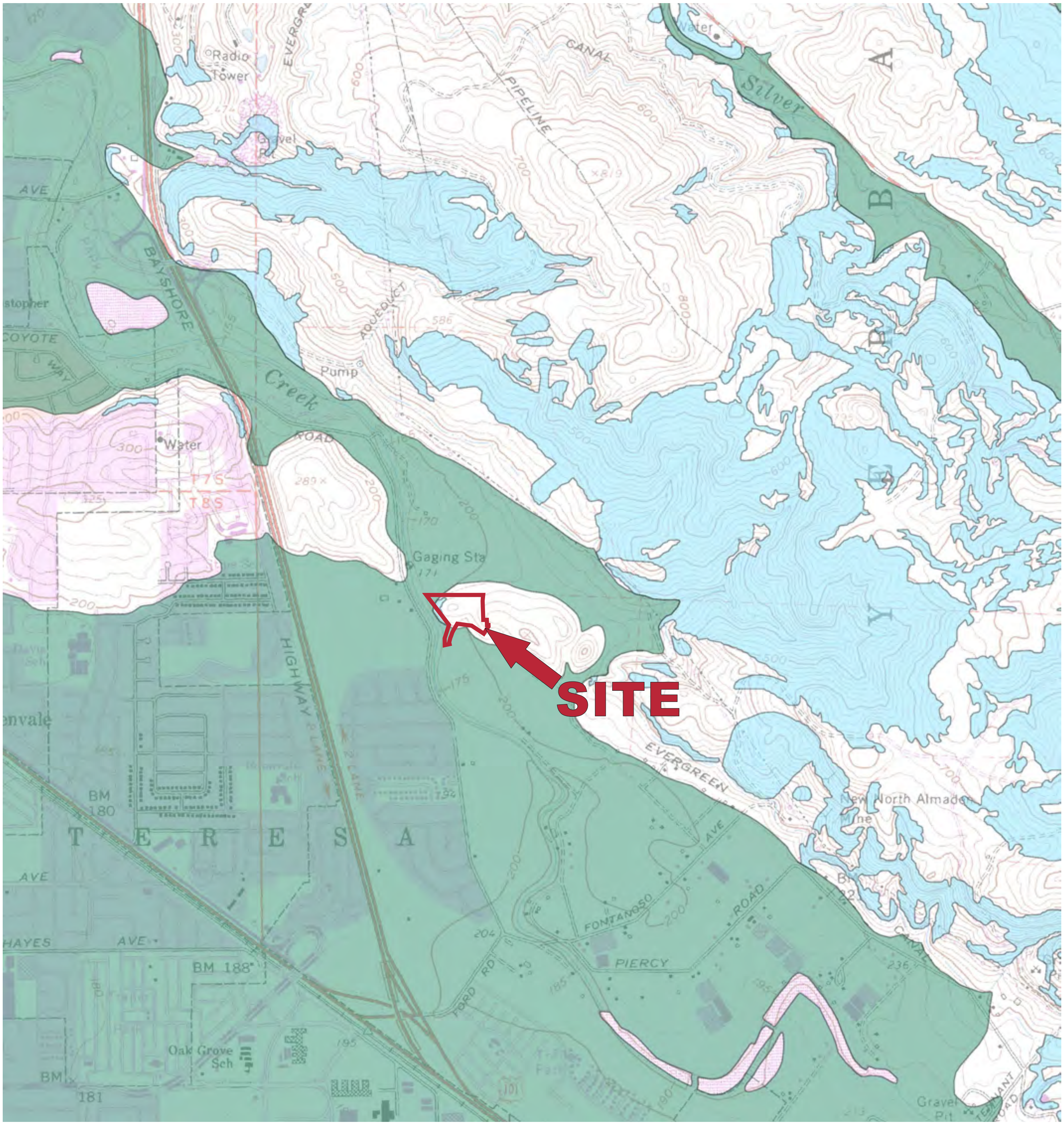
Base: Santa Clara County Geologic Hazard Zones and City of San Jose Geologic Hazard Zones

Project Number 1345-1-3
Figure Number Figure 6
Date April 2023
Drawn By RRN

City and County Geologic Hazard Zone Maps

**865 Embedded Way
San Jose, CA**

CORNERSTONE EARTH GROUP



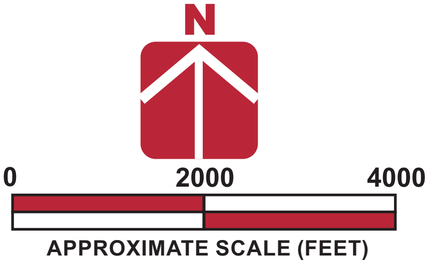
Explanation

Liquefaction

🟩 Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides

🟦 Areas where previous occurrence on landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.



Base: CGS, Earthquake Zones of Required
Investigation, San Jose East Quadrangle

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Liquefaction Hazard Zone Map

865 Embedded Way
San Jose, CA

Project Number
1345-1-3

Figure Number
Figure 7

Date
April 2023

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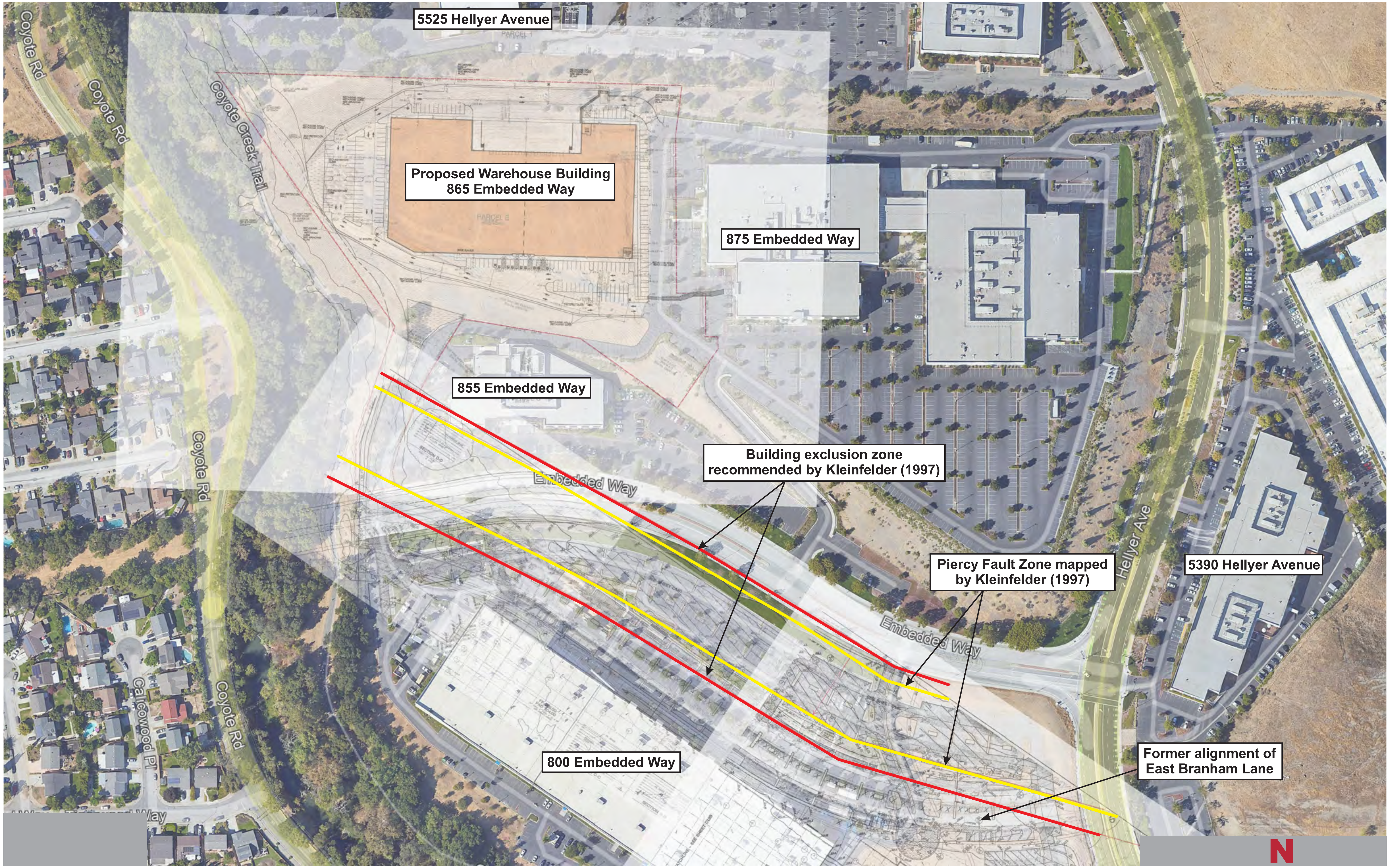

Note: The mapped Piercy Fault zone and building exclusion zone based on information presented on as-built civil plans titled "Grading Plan, Branham & Hellyer, San Jose, CA, Sheets C1.20, C1.30 and C1.40," prepared by Kier & Wright dated December 28, 2000. These plan sheets referenced the "approximate location of Piercy Fault per Kleinfelder plate dated 4/1/97" and the associated building setback.

Base by Google Earth with 1997 Kleinfelder fault zone overlay_bldg layout, dated 08/15/2020

Project Number 1345-1-3

Figure Number Figure 8

Date April 2023

Drawn By RRN

Previously Mapped Piercy Fault Building Exclusion Zone (1997)

865 Embedded Way
San Jose, CA

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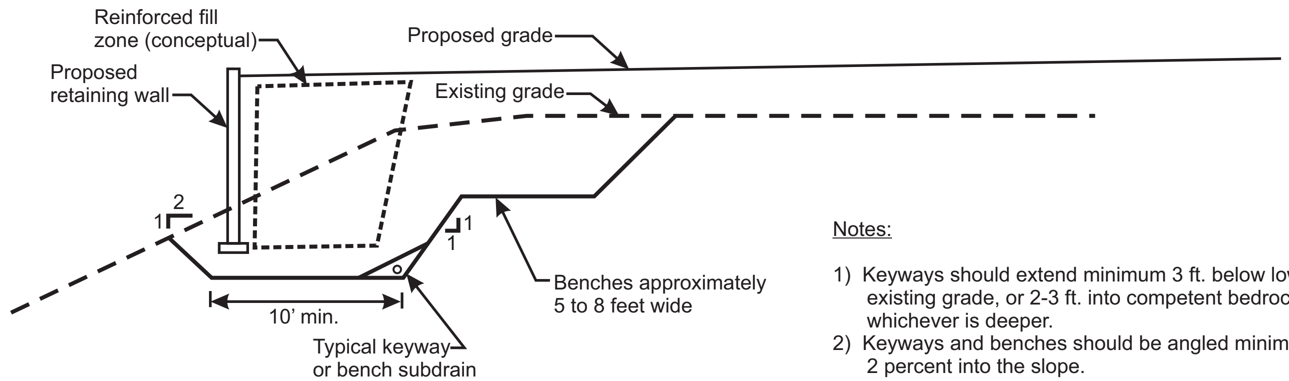
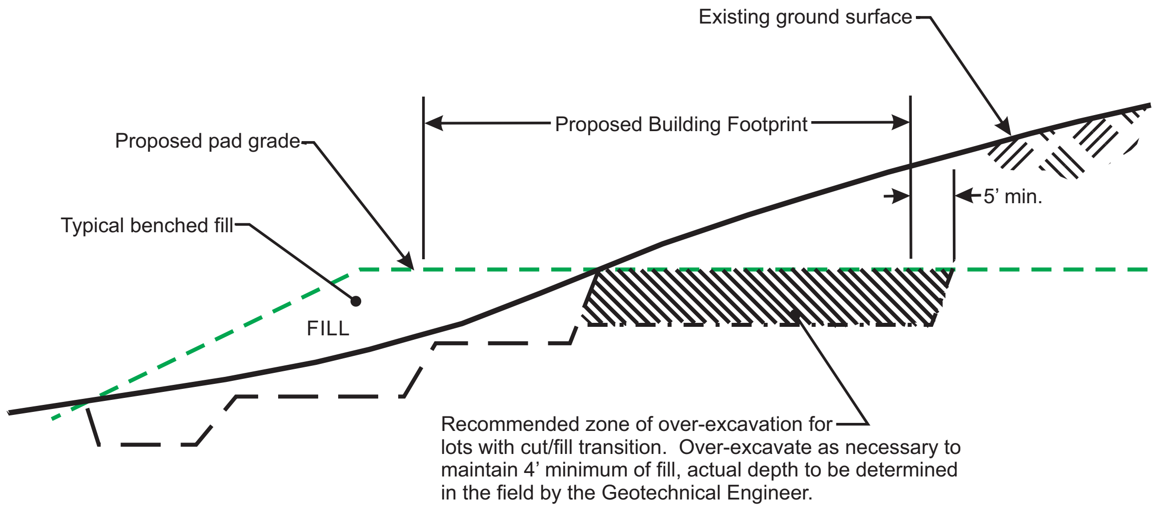


Notes:

1) Keyways should extend minimum 3 ft. below lowest existing grade, or 2-3 ft. into competent bedrock, whichever is deeper.
2) Keyways and benches should be angled minimum 2 percent into the slope.
3) This detail is preliminary; final dimensions to be determined based on actual conditions in the field
4) Geotechnical Engineer to observe keyway before placement of fill.

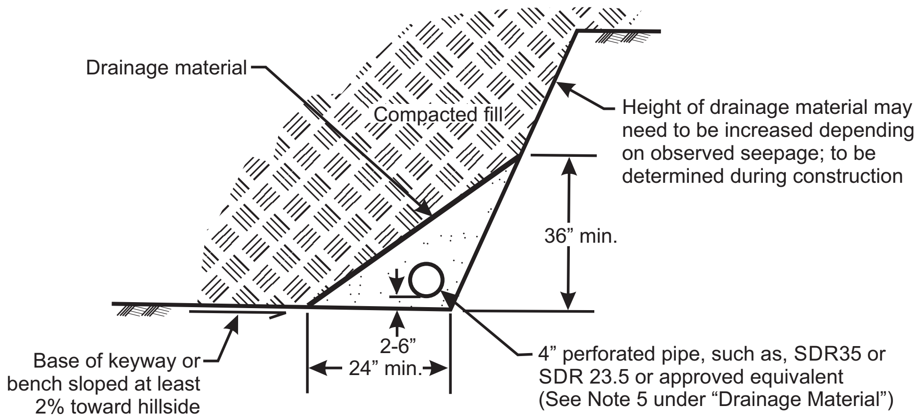
Detail 1 - Typical Bench and Keyway Detail for Perimeter Retaining Walls

Not to scale



Detail 2 - Conceptual Cut / Fill Transition Over-Excavation

Not to scale

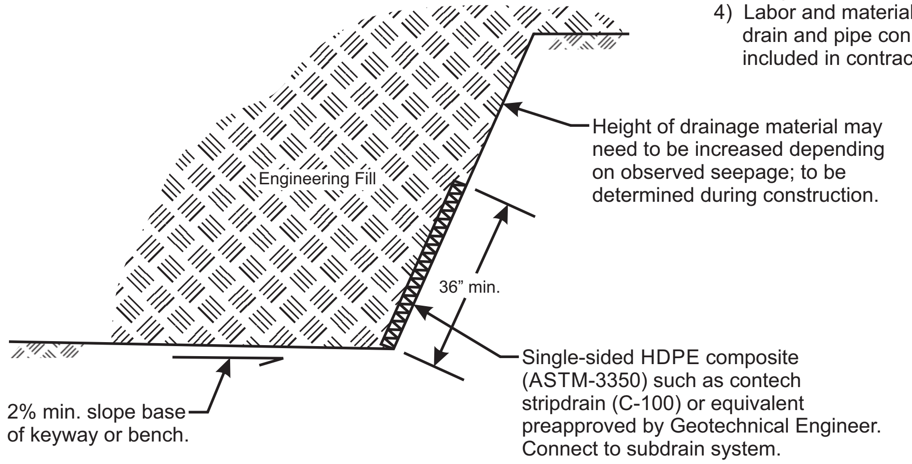


Detail 3 - Typical Bench and Keyway Subdrain

Not to scale

Notes:

1) See mitigation plan and cross sections for locations.
2) Geotechnical engineer to verify location and placement of these subdrains in the field at time of construction.
3) Contractor responsible to install connection of composite to solid discharge lines.
4) Labor and material cost to furnished install geocomposite drain and pipe connections to subdrain system to be included in contractors scope of work.



Detail 4 - Keyway and Bench Geocomposite Subdrain

Not to scale

DRAINAGE MATERIAL

Alternative 1

Class 2 Permeable Material
(Caltrans Standard Specs, latest edition)
Material shall consist of clean, coarse sand and gravel or crushed stone, conforming to the following gradation requirements:

Sieve Size	% Passing Sieve
1"	100
3/4"	90-100
3/8"	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

Alternative 2

1 /2- to 3/4- inch Clean Crushed Rock or
Gravel Wrapped in Filter Fabric
All non-woven filter fabric shall meet the following minimum average roll values unless otherwise specified by Cornerstone Earth Group

Grab Strength (ASTM D-4632):	180 lbs.
Mass Per Unit Area (ASTM D-4751):	5 oz/yd
Apparent Opening Size (ASTM D-4751):	70-100 U.S. std. sieve
Flow Rate (ASTM D-4491):	80 gal/min/ft
Puncture Strength (ASTM D-4833):	80 lbs.

Notes:

1. 1% fall (minimum) along all keyways, benches and subdrain lines.
2. All perforated pipe placed perforations down.
3. All pipe joints shall be glued.
4. All subdrains should be discharged to a free draining outlet approved by the Civil Engineer.
5. Subdrain pipe (perforated or solid connector) should consist of SDR-35 PVC pipe when placed in fills less than 30 feet deep. SDR-23.5 PVC pipe should be used when fill is greater than 30 feet deep.

Project Number
1345-1-3

Figure Number
Figure 9

Drawn By
RRN

Date
May 2023

Grading and Mitigation Details

865 Embedded Way
San Jose, CA

CORNERSTONE
EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow stem auger drilling equipment. Ten 8-inch-diameter exploratory borings were drilled on October 15, 2021 and April 13, 2023 to depths of approximately 5 to 25 feet. One 6½-inch-diameter exploratory boring was drilled on April 14, 2023. The approximate exploration locations are shown on the Site Plan and Geologic Map, Figure 2. Exploration logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Exploration locations were approximated using existing site boundaries and other site features as references. Exploration elevations were not determined. The exploration locations should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

Attached exploration logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-10)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL	▨
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL	▨
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL	▨
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	▨
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND	▨
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND	▨
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND	▨
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	▨
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY	▨
			PI>4 AND PLOTS<"A" LINE	ML	SILT	▨
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT	▨
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY	▨
			PI PLOTS <"A" LINE	MH	ELASTIC SILT	▨
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT	▨
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT	▨

OTHER MATERIAL SYMBOLS

▨ Poorly-Graded Sand with Clay
▨ Clayey Sand
▨ Sandy Silt
▨ Artificial/Undocumented Fill
▨ Poorly-Graded Gravelly Sand
▨ Topsoil
▨ Well-Graded Gravel with Clay
▨ Well-Graded Gravel with Silt
▨ Sand
▨ Silt
▨ Well Graded Gravelly Sand
▨ Gravelly Silt
■ Asphalt
▨ Boulders and Cobble

SAMPLER TYPES

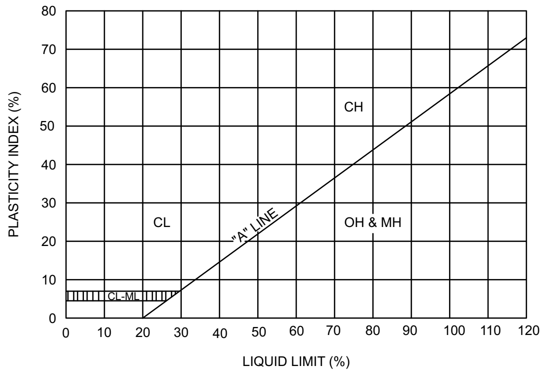
☒ SPT
▨ Modified California (2.5" I.D.)
▨ Rock Core
■ Shelby Tube
○ No Recovery
▨ Grab Sample

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)
CD - CONSOLIDATED DRAINED TRIAXIAL
CN - CONSOLIDATION
CU - CONSOLIDATED UNDRAINED TRIAXIAL
DS - DIRECT SHEAR
PP - POCKET PENETROMETER (TSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)
RV - R-VALUE
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE
▼ - WATER LEVEL

PI - PLASTICITY INDEX
SW SWELL TEST
TC - CYCLIC TRIAXIAL
TV - TORVANE SHEAR
UC - UNCONFINED COMPRESSION
(1.5) - (WITH SHEAR STRENGTH IN KSF)
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL

PLASTICITY CHART



PENETRATION RESISTANCE (RECORDED AS BLOWS / FOOT)				
SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

HARDNESS

Soft – Reserved for plastic material alone.
Low hardness – Can be gouged deeply or carved easily with a knife blade.
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.
Hard – Can be scratched with difficulty: scratch produces little powder and is often faintly visible.
Very hard – Cannot be scratched with knife blade: leaves a metallic streak.

STRENGTH

Plastic or very low strength.
Friable – Crumbles easily by rubbing with fingers.
Weak – An unfractured specimen of such material will crumble under light hammer blows.
Moderately strong – Specimen will withstand a few heavy hammer blows before breaking.
Strong – Specimen will withstand a few heavy ringing blows and will yield with difficulty only dust and small flying fragments.
Very strong – Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

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.

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.
Moderate – Slight change or partial decomposition of minerals: little disintegration: cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
Little – No megascopic decomposition of minerals: little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains or fracture surfaces.
Fresh – Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

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

BEDDING OF SEDIMENTARY ROCKS

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

CORNERSTONE
EARTH GROUP

Physical Properties of Rock Descriptions

Figure Number
A-2

BORING NUMBER EB-1

CORNERSTONE EARTH GROUP

PROJECT NAME 865 Embedded Way

PROJECT NUMBER 496-9-1

PROJECT LOCATION San Jose, CA

DATE STARTED 10/15/21 **DATE COMPLETED** 10/15/21

GROUND ELEVATION 239 FT +/- **BORING DEPTH** 8.9 ft.

DRILLING CONTRACTOR Exploration Geoservices Inc.

LATITUDE 37.269442° **LONGITUDE** -121.794834°

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY EA

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
239.0	0	**Clayey Sand with Gravel (SC)** medium dense, moist, yellowish brown and gray, fine to medium sand, fine to coarse subangular to angular gravel							
238.5		**Serpentinite [sp]** moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles							
			47	MC-1C	105	14			
			50 4"	MC-2B		23			
	5		50 5"	SPT					
			50 5"	SPT-4		1			
230.1		Bottom of Boring at 8.9 feet.							
	10								
	15								
	20								

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 10/27/21 12:31 - P:\DRAFTING\GINT FILES\496-9-1 EMBEDDED WAY.GPJ

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-2

PROJECT NAME 865 Embedded Way
PROJECT NUMBER 496-9-1
PROJECT LOCATION San Jose, CA
DATE STARTED 10/15/21 **DATE COMPLETED** 10/15/21
GROUND ELEVATION 240 FT +/- **BORING DEPTH** 12.5 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.268686° **LONGITUDE** -121.795140°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
240.0	0	▨	**Clayey Sand with Gravel (SC)** medium dense, moist, yellowish brown and gray, fine to medium sand, fine to coarse subangular to angular gravel							
239.5		▨	**Serpentinite [sp]** moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles							
				50 6"	MC-1C	92	22			
				32	SPT					
	5			50 6"	MC-3	94	6			
				62	SPT-4		26			
	10									
				50 5"	SPT					
227.5			Practical rufusal of auger at 12.5 feet. Bottom of Boring at 12.5 feet.							
	15									
	20									

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-3

PAGE 1 OF 1

PROJECT NAME 865 Embedded Way
PROJECT NUMBER 496-9-1
PROJECT LOCATION San Jose, CA
DATE STARTED 10/15/21 **DATE COMPLETED** 10/15/21
GROUND ELEVATION 246 FT +/- **BORING DEPTH** 15 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.268647° **LONGITUDE** -121.793647°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
∇ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
246.0	0	**Sandy Lean Clay (CL) [Fill]** hard, moist, dark brown to brown, fine to medium sand, some fine to coarse subangular gravel, moderate plasticity Liquid Limit = 41, Plastic Limit = 20							
			20	MC-1B	98	12	21		>4.5 ○
			32	MC-2B	104	14			>4.5 ○
241.5	5	**Sandy Lean Clay (CL)** hard, moist, brown, fine to medium sand, some fine to coarse subangular gravel, moderate plasticity							
			27	MC-3B	113	13			
240.0		**Serpentinite [sp]** moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles							
			50 6"	MC					
	10		50 5"	SPT-5		9			
			50 6"	SPT-6		22			
231.0	15	Bottom of Boring at 15.0 feet.							
	20								

BORING NUMBER EB-4

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 865 Embedded Way
PROJECT NUMBER 496-9-1
PROJECT LOCATION San Jose, CA
DATE STARTED 10/15/21 **DATE COMPLETED** 10/15/21
GROUND ELEVATION 248 FT +/- **BORING DEPTH** 9.3 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.269285° **LONGITUDE** -121.793606°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
248.0	0	▩	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, yellowish brown and gray, fine to medium sand, fine to coarse subangular to angular gravel	50 5"	MC-1B		26		
246.5		▨	**Serpentinite [sp]** moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles	50 4"	MC				
	5			50 6"	SPT-3		11		
				50 3"	SPT-4		24		
238.7			Bottom of Boring at 9.3 feet.						
	10								
	15								
	20								

CORNERSTONE EARTH GROUP

BORING NUMBER EB-5

PAGE 1 OF 1

PROJECT NAME 865 Embedded Way
PROJECT NUMBER 496-9-1
PROJECT LOCATION San Jose, CA
DATE STARTED 10/15/21 **DATE COMPLETED** 10/15/21
GROUND ELEVATION 242 FT +/- **BORING DEPTH** 6.5 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.269132° **LONGITUDE** -121.794753°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY EA
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
242.0	0		**Clayey Sand with Gravel (SC)** medium dense, moist, yellowish brown and gray, fine to medium sand, fine to coarse subangular to angular gravel							
241.5			**Serpentinite [sp]** moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles							
				70	MC-1B		20			
				50 6"	MC-2B		19			
	5									
				55	SPT-3		16			
235.5			Bottom of Boring at 6.5 feet.							
	10									
	15									
	20									

BORING NUMBER EB-6

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 865 Embedded Way

PROJECT NUMBER 1345-1-3

PROJECT LOCATION San Jose, CA

DATE STARTED 4/13/23 **DATE COMPLETED** 4/13/23

GROUND ELEVATION **BORING DEPTH** 9 ft.

DRILLING CONTRACTOR Exploration Geoservices Inc.

LATITUDE 37.2694318° **LONGITUDE** -121.7952580°

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

▽ **AT TIME OF DRILLING** Not Encountered

▼ **AT END OF DRILLING** Not Encountered

LOGGED BY JDS

NOTES

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UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0		**Clayey Sand with Gravel (SC)** medium dense, moist, brown and gray mottled, fine to medium sand, fine to coarse subangular gravel							
				50 5"	☒ SPT-1	83	22			
			Serpentinite [sp] moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles							
				50 3"	NR					
	5			50 1"	NR					
				50 6"	☒ SPT-2		41			
				50 6"	☒ SPT					
			Bottom of Boring at 9.0 feet.							
	10									
	15									
	20									
	25									

BORING NUMBER EB-7

PAGE 1 OF 1

PROJECT NAME 865 Embedded Way
PROJECT NUMBER 1345-1-3
PROJECT LOCATION San Jose, CA

DATE STARTED 4/13/23 **DATE COMPLETED** 4/13/23
DRILLING CONTRACTOR Exploration Geoservices Inc.
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
LOGGED BY JDS
NOTES

GROUND ELEVATION **BORING DEPTH** 8.8 ft.
LATITUDE 37.2692869° **LONGITUDE** -121.7956480°
GROUND WATER LEVELS:
▽ **AT TIME OF DRILLING** Not Encountered
▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0		**Clayey Sand with Gravel (SC)** medium dense, moist, brown and gray mottled, fine to medium sand, fine to coarse subangular gravel							
				50 1"	MC-1	73	27			
			Serpentinite [sp] moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles							
				50 1"	SPT-2		12			
	5			50 1"	SPT-3		15			
				50 3"	SPT-4		10			
			Bottom of Boring at 8.8 feet.							
	10									
	15									
	20									
	25									

BORING NUMBER EB-8

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 865 Embedded Way
PROJECT NUMBER 1345-1-3
PROJECT LOCATION San Jose, CA
DATE STARTED 4/13/23 **DATE COMPLETED** 4/13/23
GROUND ELEVATION **BORING DEPTH** 8.7 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.2690252° **LONGITUDE** -121.7952302°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY JDS
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	**Clayey Sand with Gravel (SC)** medium dense, moist, brown and gray mottled, fine to medium sand, fine to coarse subangular gravel							
			50 4"	MC-1	77	28			
		Serpentinite [sp] moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles							
			50 6"	SPT-2		16			
	5		50 5"	SPT-3		25			
			50 2"	SPT					
		Bottom of Boring at 8.7 feet.							
	10								
	15								
	20								
	25								

BORING NUMBER EB-9

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 865 Embedded Way
PROJECT NUMBER 1345-1-3
PROJECT LOCATION San Jose, CA
DATE STARTED 4/13/23 **DATE COMPLETED** 4/13/23
GROUND ELEVATION **BORING DEPTH** 18.6 ft.
DRILLING CONTRACTOR Exploration Geoservices Inc.
LATITUDE 37.2688483° **LONGITUDE** -121.7955866°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY JDS
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0		**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, brown and gray mottled, fine to medium sand, fine to coarse subangular gravel	31	MC-1B	94	22			
			Serpentinite [sp] moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles	50 5"	MC-2B		56			
	5			61	MC-3B	82	26			
				50 6"	SPT					
	10									
				50 6"	SPT-5		35			
	15									
			Bottom of Boring at 18.6 feet.	50 1"	SPT					
	20									
	25									

BORING NUMBER EB-10

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME 865 Embedded Way

PROJECT NUMBER 1345-1-3

PROJECT LOCATION San Jose, CA

DATE STARTED 4/13/23 **DATE COMPLETED** 4/13/23

GROUND ELEVATION **BORING DEPTH** 24.2 ft.

DRILLING CONTRACTOR Exploration Geoservices Inc.

LATITUDE 37.2683298° **LONGITUDE** -121.7938113°

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY JDS

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	**Clayey Sand with Gravel (SC) [Fill]** dense to medium dense, moist, brown and gray mottled, fine to medium sand, fine to coarse subangular gravel							
			71	MC-1B	106	16			
			59	MC-2B	101	21		30	
	5	**Serpentinite [sp]** moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles	50 5"	MC-3B	92	18			
			50 3"	SPT-4		13			
	10								
			50 5"	SPT-5		7			
	15								
			63	SPT-6		21			
	20								
			50 2"	SPT-7		17			
	25	Bottom of Boring at 24.2 feet.							

CORNERSTONE EARTH GROUP

BORING NUMBER EB-11

PAGE 1 OF 1

PROJECT NAME 865 Embedded Way
PROJECT NUMBER 1345-1-3
PROJECT LOCATION San Jose, CA
DATE STARTED 4/14/23 **DATE COMPLETED** 4/14/23
GROUND ELEVATION ____ **BORING DEPTH** 23.7 ft.
DRILLING CONTRACTOR Cuesta Geo
LATITUDE 37.2683242° **LONGITUDE** -121.7949560°
DRILLING METHOD MPP LAD Track Rig, 6½ inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY JDS
▽ **AT TIME OF DRILLING** Not Encountered
NOTES ____
▼ **AT END OF DRILLING** Not Encountered

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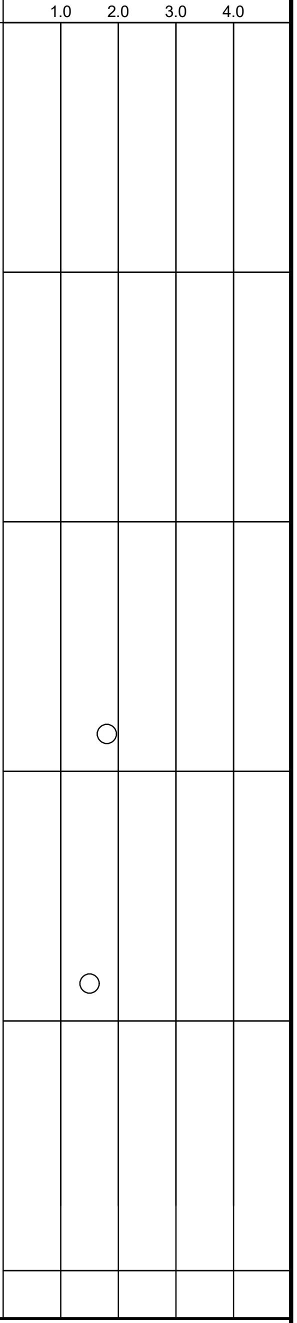
DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
Silty Sand with Gravel (SM) [Fill] medium dense, moist, yellowish brown and gray, fine to medium sand, fine to coarse subangular to angular gravel Liquid Limit = 50, Plastic Limit = 29	20	MC-1B	96	16	21	
	45	MC-2B	86	24		18
	39	MC				
becomes loose	17	MC-4B	79	26		
becomes medium dense	16	SPT				
	17	SPT-6		15		
Lean Clay with Sand (CL) stiff, moist, dark brown to brown, fine to medium sand, moderate plasticity Liquid Limit = 46, Plastic Limit = 20	11	MC-7B	103	20	27	
	26	MC				
Serpentinite [sp] moderately hard, moderately strong, moderate weathering, greenish gray with yellowish brown mottles	48	SPT-9		37		
Bottom of Boring at 23.7 feet.	50 2"	SPT-10		15		

ELEVATION (ft) | DEPTH (ft): 0, 5, 10, 15, 20, 25 | SYMBOL

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



APPENDIX B: LABORATORY TEST PROGRAM

The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 46 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

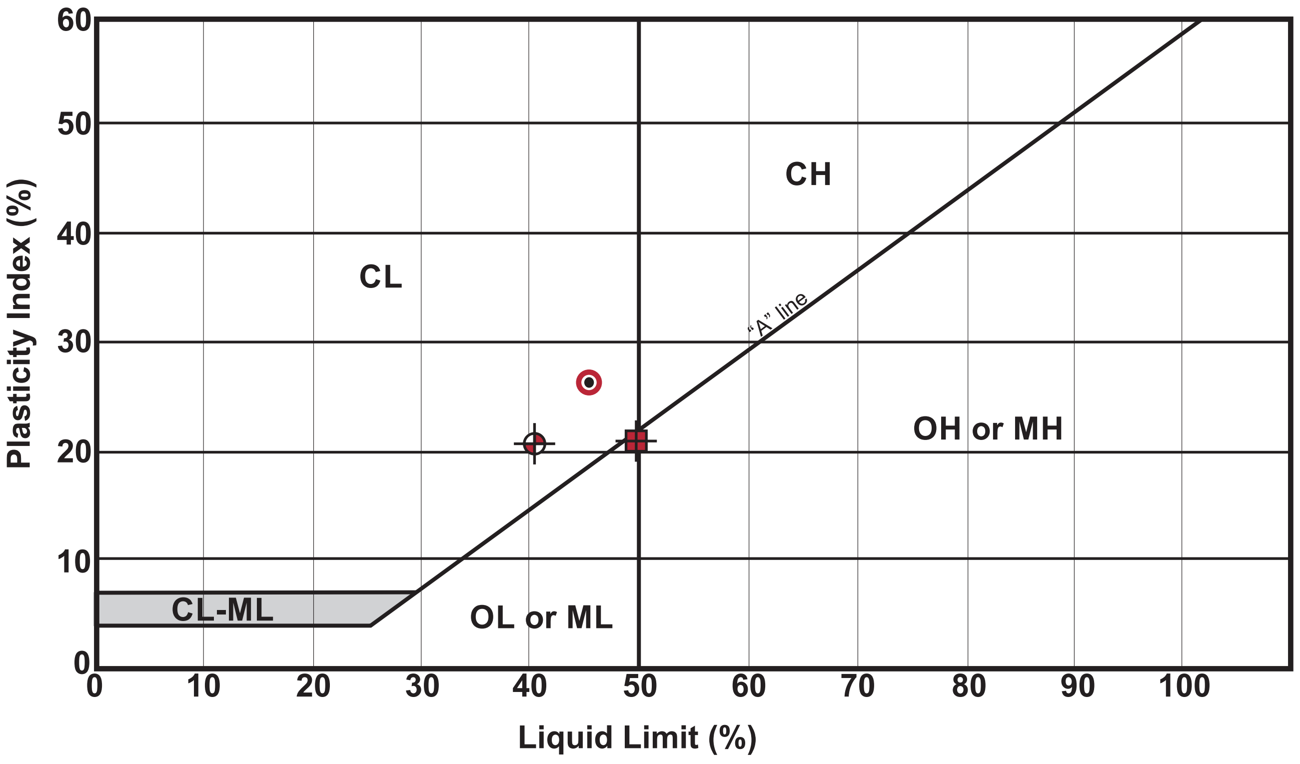
Dry Densities: In place dry density determinations (ASTM D2937) were performed on 36 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on three samples of the subsurface soil to aid in the classification of these soils. Results of these tests are shown on the boring log at the appropriate sample depth.

Plasticity Index: Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of this test are shown on Figure B-1.

R-value: An R-value resistance test (California Test Method No. 301) was performed on a representative sample of the surface soils at the site to provide data for the pavement design. The test indicated an R-value of 8 at an exudation pressure of 300 pounds per square inch.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-3	2.0	12	41	20	21		Sandy Lean Clay (CL) [Fill]
■	EB-11	2.0	16	50	29	21		Silty Sand (SM) (MH fines) [Fill]
◉	EB-11	14.5	20	46	20	26		Lean Clay with Sand (CL)

Samples prepared in accordance with ASTM D421

CORNERSTONE EARTH GROUP

Plasticity Index Testing Summary

865 Embedded Way
San Jose, CA

Project Number
1345-1-3

Figure Number
Figure B1

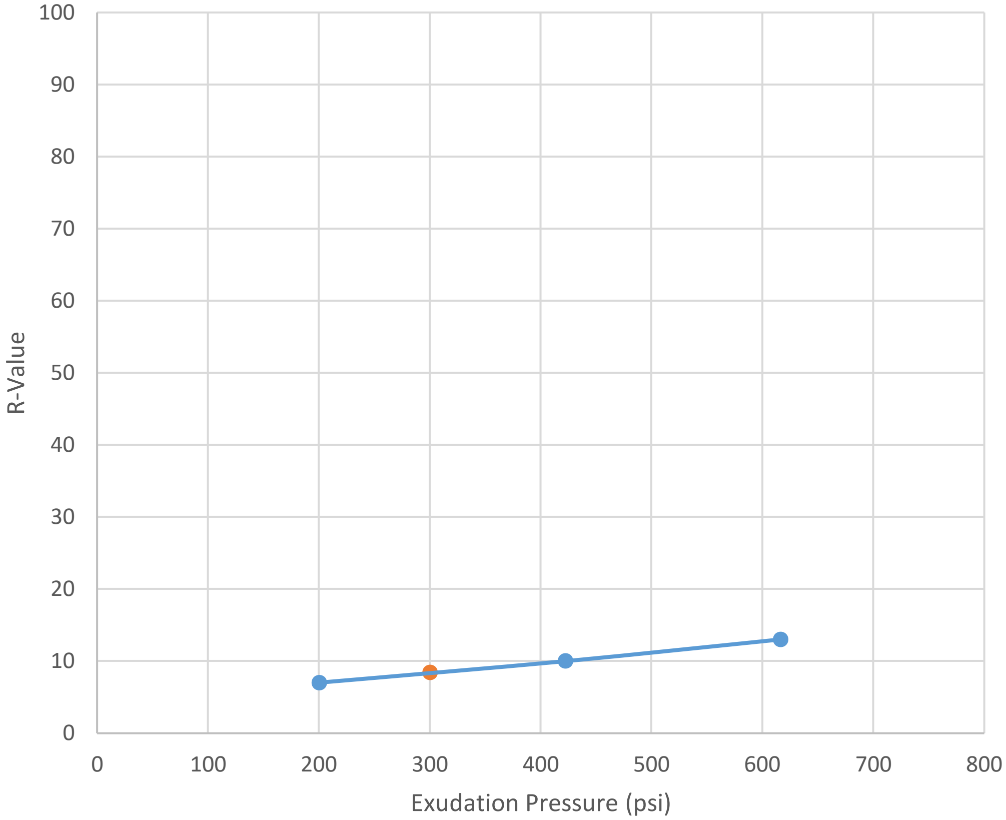
Date
April 2023

Drawn By
FLL

R-Value of Treated and Untreated Bases

CTM 301

Project Name: 865 Embedded Way (1345-1-3)
Sample Date: 4/13/2023
Project Number: 23-1548
Test Date: 4/19/2023
Project Address : --
Report Date: 4/21/2023
City/State: --
Sampled By: Others
Sample Location: EB-6 - 10, Upper 5'
Lab Number: M-2046
Material: Clayey Sand with gravel



Specimen Number	**1**	**2**	**3**
Dry Density of Briq. (lbs/ft^3)	91.3	92.5	94.3
Moisture @ Compaction	27.6%	26.6%	25.6%
Expansion Dial Reading	0	0	0
R-Value	7	10	13
R-Value @ 300 psi			**8**
Material Description	Clayey Sand with gravel		
Specification			

Reported By:

Suzanne Morgan
Laboratory Supervisor

APPENDIX C: RESULTS OF NATURALLY OCCURRING ASBESTOS (NOA) TESTING

ASBESTOS TEM LABORATORIES, INC.

CARB Method 435
Polarized Light Microscopy
Analytical Report

Laboratory Job # 1206-00723

3431 Ettie St.
Oakland, CA 94608
(510) 704-8930
FAX (510) 704-8429

ASBESTOS TEM LABORATORIES, INC

CA ELAP
Lab No. 1866

TESTING
NVLAP Lab Code: 101891-0
Oakland, CA

Oct/22/2021

Diana Lin
Cornerstone Earth Group, Inc.
1259 Oakmead Parkway
Sunnyvale, CA 94085

RE: LABORATORY JOB # 1206-00723
Polarized light microscopy analytical results for 4 bulk sample(s).
Job Site: 496-9-1
Job No.: 865 Embedded Way, San Jose

Enclosed please find the bulk material analytical results for one or more samples submitted for asbestos analysis. The analyses were performed in accordance with the California Air Resources Board (ARB) Method 435 for the determination of asbestos in serpentine aggregate samples.

Prior to analysis, samples are logged-in and all data pertinent to the sample recorded. The samples are checked for damage or disruption of any chain-of-custody seals. A unique laboratory ID number is assigned to each sample. A hard copy log-in sheet containing all pertinent information concerning the sample is generated. This and all other relevant paper work are kept with the sample throughout the analytical procedures to assure proper analysis.

Sample preparation follows a standard CARB 435 prep method. The entire sample is dried at 135-150 C and then crushed to ~3/8" gravel size using a Bico Chipmunk crusher. If the submitted sample is >1 pint, the sample was split using a 1/2" riffle splitter following ASTM Method C-702-98 to obtain a 1 pint aliquot. The entire 1 pint aliquot, or entire original sample, is then pulverized in a Bico Braun disc pulverizer calibrated to produce a nominal 200 mesh final product. If necessary, additional homogenization steps are undertaken using a 3/8" riffle splitter. Small aliquots are collected from throughout the pulverized material to create three separate microsope slide mounts containing the appropriate refractive index oil. The prepared slides are placed under a polarizing light microscope where standard mineralogical techniques are used to analyze the various materials present, including asbestos. If asbestos is identified and of less than 10% concentration by visual area estimate then an additional five sample mounts are prepared. Quantification of asbestos concentration is obtained using the standard CAL ARB Method 435 point count protocol. For samples observed to contain visible asbestos of less than 10% concentration, a point counting techinique is used with 50 points counted on each of eight sample mounts for a total of 400 points. The data is then compiled into standard report format and subjected to a thorough quality assurance check before the information is released to the client.

While the CARB 435 method has much to commend it, there are a number of situations where it fails to provide sufficient accuracy to make a definitive determination of the presence/absence of asbestos and/or an accurate count of the asbestos concentration present in a given sample. These problems include, but are not limited to, 1) statistical uncertainty with samples containing <1% asbestos when too few particles are counted, 2) definitive identification and discrimination between various fibrous amphibole minerals such as tremolite/actinolite/hornblende and the "Libby amphiboles" such as tremolite/winchite/richterite/arfvedsonite, and C) small asbestiform fibers which are near or below the resolution limit of the PLM microscope such as those found in various California coast range serpentine bodies. In these cases, further analysis by transmission electron microscopy is recommended to obtain a more accurate result.

Sincerely Yours,

Lab Manager
ASBESTOS TEM LABORATORIES, INC.

--- These results relate only to the samples tested and must not be reproduced, except in full, without the approval of the laboratory. ---

POLARIZED LIGHT MICROSCOPY
CARB 435 ANALYTICAL REPORT

Page: 1 **of**

Contact: Diana Lin

Address: Cornerstone Earth Group, Inc.
1259 Oakmead Parkway
Sunnyvale, CA 94085

Samples Submittec 4

Samples Analyzed: 4

Job Site / No. 865 Embedded Way, San Jose
496-9-1

Report No. **376055**

Date Submitted: Oct-19-21

Date Reported: Oct-22-21

SAMPLE ID	POINTS COUNTED	ASBESTOS %	ASBESTOS TYPE	LOCATION / DESCRIPTION
EB-1 Lab ID # 1206-00723-001	**400** - Total Points	**<0.25%**	**Chrysotile**	Trace Chrysotile fibers observed.
EB-3 Lab ID # 1206-00723-002	**1** **400** - Total Points	**0.25%**	**Chrysotile**	Chrysotile fibers observed.
EB-4 Lab ID # 1206-00723-003	**5** **400** - Total Points	**1.25%**	**Chrysotile**	Chrysotile fibers observed.
EB-5 Lab ID # 1206-00723-004	**3** **400** - Total Points	**0.75%**	**Chrysotile**	Chrysotile fibers observed.
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			
Lab ID #	- Total Points			

QC Reviewer

Analys Jo Ann Huerta

Asbestos TEM Laboratories, Inc. 3431 Ettie St., Oakland, CA 94608 PH. (510) 704-8930

ASBESTOS TEM LABORATORIES CHAIN OF CUSTODY – *www.asbestostemlabs.com*

CALIFORNIA: 600 Bancroft Way, Ste. A, Berkeley, CA 94710 Phone (510) 704-8930 Fax (510) 704-8429
NEVADA: 1350 Freeport Blvd. #104, Sparks, NV 89431 Phone (775) 359-3377 Fax (775) 359-2798

Please print and send completed CoC with your samples. If you wish to email CoC, send the form as an attachment to Berkeley <coc@asbestostemlabs.com > or Reno <sehrlich@asbestostemlabs.com>.

Company: Cornerstone Earth Group | *Contact:* Diana Lin | *Phone/Fax:* 408-470-9181 | *Email:* dlin@cornerstoneearth.com
Address: 1259 Oakmead Parkway | *City:* Sunnyvale | *State:* CA *Zip:* 94085 | *Country:* United States
Job Site: 865 Embedded Way, San Jose | *Job No:* 496-9-1 | *P.O. No:*

Reporting ☐ Fax ☐ Phone ☒ Email ☐ Mail ☐ FTP ☐ EDD/State Form ☐ Verbal ☐ Pickup **Billing** ☐ Fax ☒ Email ☐ Mail ☐ Pre-Paid ☐ 3rd Party

Results Due:* ☐ 2 hr ☐ 4 hr ☐ 6 hr ☐ 8 hr ☐ 24 hr ☐ 48 hr ☐ 3 day ☐ 4 day ☒ 5 day ☐ 10 day ☐ Time due:____ * Contact lab to confirm TAT

Asbestos Air ☐ PCM (NIOSH 7400A ☐ TEM AHERA ☐ TEM CARB Mod. AHERA ☐ TEM EPA Yamate Level ☐ TEM NIOSH 7402, Issue 2 ☐ ISO 10312 ☐ ISO 13794

Asbestos Bulk ☐ PLM Standard (EPA 600/R-93-1 ☐ PLM 400 PC ☐ PLM 1000 PC ☐ PLM 400 PC Grav. Red. ☐ PLM 1000 PC Grav. Red. ☐ TEM EPA Qualitative ☐ TEM EPA Quantitatve
☐ TEM Chatfield (Semi-Quant) ☐ PLM Vermiculite Attic Insulation ☐ **Custom Analysis:** Type:

Asbestos Soils ☐ CARB 435 Prep Only ☒ CARB 435 PLM 400 PC ☐ CARB 435 PLM 1000 PC ☐ EPA Soil Screening Qualitative ☐ TEM EPA/CARB Quantitative

Asbestos Dust ☐ ASTM D-5755 Fiber Count ☐ ASTM D-5756 Wt. % ☐ ASTM D-5756 Mass ☐ ASTM D-6840-99 Dust Wipe ☐ Total Particulates (Grav.)

Asbestos Water ☐ 100.2 Potable Drinking Water ☐ 100.1 Non Potable Water

Lead ☐ Paint Chips ☐ Dust Wipe ☐ Air Cassette ☐ Soil **Lead Waste Characterization:** ☐ TTLC ☐ STLC ☐ TCLP

Sample Storage *☐ No Test, Hold Sample Until: ________ *☐ Post Test, Hold Sample Until:________

Custom Order ☐ Reanalysis by:__________ ☐ Sensitivity: ________ ☐ Composite ☐ Other:

Sample #	Sample Type	Date Collected	Time On	Time Off	Total Time (min)	Flow Rate (lpm)			Volume or Area Sampled	8 Hour TWA Requested	Description
						On	Off	Average			
EB-1	Baggie	10.15.2021								☐	
EB-3	BAggie	10.15.2021								☐	
EB-4	BAggie	10.15.2021								☐	
EB-5	BA ggie	10.15.2021								☐	
										☐	
										☐	
										☐	
										☐	
										☐	
										☐	
										☐	
										☐	OCT 19 21 10:13AM
										☐	

Submitted By	**Received By** MTZ
Date/Time Submitted	**Date/Time Received**
Submitted By	**Received By**
Date/Time Submitted	**Date/Time Received**

*All samples will be held for 3 months from the date of receipt at ATEM. Additional sample storage time may be obtained through ATEM Customer Service

APPENDIX D: SITE CORROSIVITY EVALUATION

JDH CORROSION CONSULTANTS REPORT DATED APRIL 27, 2023

jdh corrosion consultants, inc.

Protecting the infrastructure
through innovative
Corrosion Engineering Solutions

April 27, 2023

Cornerstone Earth Group, Inc.
1220 Oakland Blvd Suite 200
Walnut Creek, CA 94596

Attention: **Diana Lin, P.E.**
Project Engineer

Subject: **Site Corrosivity Evaluation**
Embedded Way Industrial Building
San Jose, CA
Project: 1345-1-3

Dear Diana,

In accordance with your request, we have reviewed the laboratory soils data for the above referenced project site. Our evaluation of these results and our corresponding recommendations for corrosion control for the above referenced project foundations and buried site utilities are presented herein for your consideration.

Soil Testing & Analysis

Soil Chemical Analysis

Two (2) soil samples from the project site were chemically analyzed for corrosivity by **Cornerstone Earth Group**. Each sample was analyzed for chloride and sulfate concentration, pH and resistivity at 100% saturation. The test results are presented in Cornerstone Earth Group Geotechnical Report dated 1/11/2023. The results of the chemical analysis were as follows:

Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	3 – 7 mg/kg	Non-corrosive*
Sulfates	13 mg/kg	Non-corrosive**
pH	6.7 – 7.1	Non-corrosive*
Resistivity at 100% Saturation	1,813 – 2,767 ohm-cm	**Corrosive to Moderately Corrosive***

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Discussion

Shallow Reinforced Concrete Foundations

Due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The type of cement used should be in accordance with California Building Code (CBC) for soils which have less than 0.10 percent by weight of water-soluble sulfate (SO_4) in soil and the minimum depth of cover for the reinforcing steel should be as specified in CBC as well.

Underground Metallic Pipelines

The soils at the project site are generally considered to be “corrosive to moderately corrosive” to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at this site depending upon the critical nature of the piping. Pressure piping systems such as domestic and fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

LIMITATIONS

The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees, expressed or implied, is provided.

We thank you for the opportunity to be of service to **Cornerstone Earth Group** on this project and trust that you find the enclosed information satisfactory. If you have any questions, or if we can be of any additional assistance, please feel free to contact us at (925) 927-6630.

Respectfully submitted,

Brendon Hurley
JDH CORROSION CONSULTANTS, INC.
Field Technician

Mohammed Ali

REGISTERED PROFESSIONAL ENGINEER
MOHAMMED ALI
No. CR 1035
Exp. June 30, 2023
CORROSION
STATE OF CALIFORNIA

Mohammed Ali., P.E.
JDH Corrosion Consultants, Inc.
Senior Corrosion Engineer

CC: File 2023147