

December 18, 2024

**GEOTECHNICAL INVESTIGATION
1815 CORDILLERAS ROAD
EMERALD HILLS, CALIFORNIA
SFB PROJECT NO. 659-5**

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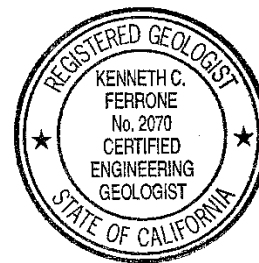
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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed new residential development to be located at 1815 Cordilleras Road in the unincorporated Emerald Hills area of San Mateo County, California. The project site location is shown on the **Vicinity Map, Figure 1**, and **Site Plan, Figure 2**. The purpose of our investigation was to evaluate the geological and geotechnical conditions at the site and provide recommendations regarding the geological and geotechnical engineering aspects of the project.

Based on to the project information provided by Edenbridge, and the project preliminary site plan prepared by MacLeod and Associates and dated November 5, 2024, we understand that the project will consist of developing two suburban parcels (APN 057-070-090 & 057-063-010) with a combined area of about 1.9-acres for a new residential subdivision. The two parcels are separated by a property owned by San Francisco Public Utilities Commission (SFPUC). The new development will include about six single-family home lots with paved roadways and underground utilities. Actual details of the new development are to be determined. Nominal grading is anticipated.

The conclusions and recommendations provided in this report are based upon the information presented above; Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) should be consulted if any changes to the project occur to assess if the changes affect the validity of this report.

2.0 SCOPE OF WORK

This investigation included the following scope of work:

- Reviewing published and unpublished geotechnical and geological literature relevant to the site;
- Reviewing historical aerial photographs and topographic maps of the site and surrounding area;
- Performing reconnaissance of the site and surrounding area;
- Performing four exploratory borings to a maximum depth of about 31 feet;
- Performing laboratory testing of soil samples retrieved from the borings;
- Performing engineering analysis of the field and laboratory data; and
- Preparing this report.

The data obtained and the analyses performed were for the purpose of providing geotechnical design and construction criteria for site earthwork, underground utility, drainage, building foundation, retaining wall, and pavement. Evaluating the potential for flooding and toxicity potential assessment of onsite materials or groundwater (including mold) were beyond our scope of work.

3.0 SITE INVESTIGATION

3.1 Field Exploration

Our geotechnical field exploration program for the project consisted of performing four exploratory borings (B-1 to B-4) a maximum depth of about 31 feet on December 5, 2024. The approximate locations of the borings are shown on the **Site Plan, Figure 2**. The borings were performed by West Coast Exploration of Escalon, California by using a truck-mounted Mobile B-24 drill rig equipped with 4-inch diameter, continuous flight, solid stem augers and a 140-pound safety hammer.

Our field engineer continuously logged the soils encountered in the borings. The soils are classified in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). Logs of the explorations as well as keys for the classification of the soil (**Figure A-1**) are included in **Appendix A**. Upon completion of our field explorations, the borings were backfilled with cement grout in accordance with San Mateo County Environmental Health permit requirements.

The approximate locations of our borings were determined by pacing, tape measurements, and/or alignment from landmark references. Latitude and longitude of exploration locations shown on the exploration logs are estimated from online map data from Microsoft; actual locations were not surveyed.

Representative samples were obtained from our exploratory borings at selected depths appropriate to the investigation. Relatively undisturbed samples were obtained using a 3-inch O.D. Modified California split barrel sampler with liners, and disturbed samples were obtained using a 2-inch O.D. Standard Penetration Test (SPT) split spoon sampler without liners. All samples were transported to our geotechnical laboratory for evaluation and appropriate testing. Both sampler types are indicated in the "Sampler" column of the exploration logs as designated in **Figure A-1**.

Resistance blow counts were obtained in our borings with the samplers by dropping a 140-pound safety hammer through a 30-inch fall with rope and cathead. The sampler was driven 18 inches and the number of blows were recorded as field blow counts for each 6 inches of penetration. The SPT N-value shown on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. A sampler barrel size correction factor of 0.6 was applied to the blow counts from the Modified California sampler. The recorded blow counts have not been corrected for other factors, such as hammer efficiency, borehole diameter, rod length, overburden pressure, and fines content.

It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report. In addition, our attached exploration logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

3.2 Laboratory Testing

Our laboratory testing program for the project was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. This program included the following testing:

- Eight moisture content and dry unit weight determinations per ASTM D2937;
- One Atterberg Limits determination (plastic and liquid limits) per ASTM D4318;
- Two sieve and hydrometer tests per ASTM D422 or D1140; and
- Six unconfined compressive strength tests per ASTM D2166.

All tests were performed by our geotechnical laboratory in Concord, California. The results of the testing are included on the exploration logs and plotted laboratory results are also included in **Appendix B**.

Two selected onsite soil samples were tested by CERCO Analytical, Inc. in Concord, California for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498). The test results and a brief evaluation summary report prepared by CERCO regarding the onsite soils' potential for corrosion of concrete and buried metal such as utilities and reinforcing steel are included in **Appendix B**. We recommend these corrosion test results be forwarded to the project's underground contractors, pipeline designers, concrete contractors, and foundation designers and contractors.

3.3 Surface Conditions and Site Development History

As shown on **Figure 2**, the site is bounded by Cordilleras Road on the north, existing residential developments on the east and west, and hillside open space slopes on the south. The generally north facing hillside slopes have inclinations varying from about 2:1 (horizontal to vertical) to 5:1, and have numerous large and small diameter trees. Existing stone retaining walls up to about 3 feet high are located at base of the slopes along the southern site boundary, and appear to be in good to excellent condition.

Cordilleras Creek is located near the north side of Cordilleras Road. The unlined creek bank is about 10 to 12 feet high and has steep side slopes. The top of the creek bank was located at about 70 feet from the site northern boundary at its closest location. Easterly flowing water was observed at the bottom of the creek at the time of our field exploration.

The site includes two parcels (western and eastern parcels) that are separated by a property owned by SFPUC. The large western parcel is approximately trapezoid in shape and has a plan area of about 1.8 acres with maximum dimensions of about 420 feet by 270 feet. The small eastern parcel is triangular in shape and has a plan area of about 5,000 square feet with maximum dimensions of about 120 feet by 85 feet. In general, the existing site grades slope slightly downward toward the northeast, except for the southwestern corner where a steeper hillside slope is located.

At the time of our investigation, the western large parcel was occupied by several residential buildings with associated facilities and landscaping, including an in-ground swimming pool. An asphalt concrete paved driveway provided access to the site from Cordilleras Road. The eastern small parcel and the adjacent SFPUC property were vacant. Large and small diameter trees were located throughout the site.

Based on our review of historical topographic maps and aerial photographs, the buildings within the site were probably built in the 1950s. An orchard appeared to occupy the northern half of the western large parcel before the 1960s.

3.4 Subsurface Conditions

The subsurface soil materials encountered in our borings generally consisted of firm to hard clays with interbeds of loose to medium dense sand and gravel lenses that extend to the maximum depth explored of about 31 feet. Drilling refusal in very dense gravels was encountered by the Mobile B-24 drill rig at the bottom of Boring B-1. In addition, Boring B-1 encountered about 2 feet of sandy fills at the ground surface. These fills are heterogenous, and potentially weak and compressible. In addition, we estimate the upper about 2 to 3 feet of surface soils within the northern half of the western large parcel have been weakened due to removal of the previous orchard trees.

According to the results of laboratory testing, the near-surface more clayey soils have a medium plasticity and moderate expansion and shrinkage potential. However, the sandy soils are non-plastic and have a low expansion and shrinkage potential. Detailed descriptions of soils encountered in our exploratory borings are presented on the exploration logs in **Appendix A**. Results of laboratory testing of retrieved onsite soils are included in **Appendix B**.

3.5 Surface Water and Groundwater

Onsite, we observed areas where surface water had ponded in the past. The slopes to the south of the site direct surface water flows into the site. A man-made access road (approximately shown on Figure 2) collects surface water and directs the water toward and into the site.

Groundwater levels were measured in Borings B-1 and B-4, respectively, at depths of about 27 and 18 feet at the end of drilling. No groundwater was encountered in the other borings to the maximum depths explored of these borings at about 21-1/2 feet. It should be noted that our borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions.

Due to the site's close proximity to Cordilleras Creek, the site groundwater level can be heavily influenced by the water flow within the creek. According to California Geological Survey (2019)¹, the historically high groundwater levels in areas near Cordilleras Creek are less than 10 feet deep.

Fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, subsurface seepage, water pumping in nearby wells, and other factors. It is also likely that during rainfall events, localized high groundwater level or seepage may develop on the adjacent hillside slopes and seep toward the site.

3.6 Hydrologic Soil Group

The surface soils of the site have been mapped by the USDA Natural Resource Conservation Services (NRCS) Web Soil Survey (WSS)² and categorized into the following two map units:

- a) Orthents, cut and fill-urban land complex, 5 to 75 percent slopes (Unit 124) within the northern half of the site; and
- b) Los Gatos loam, 30 to 75 percent slopes, MLRA 15 (Unit 115) within the southern half of the site.

The Orthents, cut and fill-urban land complex (Unit 124) has not been evaluated and assigned to any hydrologic soil groups by USDA. The Los Gatos loam (Unit 115) has been assigned to Hydrologic Soil Group C and was estimated to have moderately high to high transmission rates (approximately 0.6 to 2 inches per hour). Group C soils are defined as having a slow infiltration rate when thoroughly wet and may consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture.

¹California Geological Survey, 2019, Seismic Hazard Zone Report for the Woodside 7.5-Minute Quadrangle, San Mateo County, California, Seismic Hazard Zone Report 129, Plate 1.3.

²USDA NRCS, <https://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx>, accessed 12/16/2024.

Actual field infiltration rates will depend on the in-situ soil type, moisture, relative density, gradation, and fines content of soils, whether any water impeding clay layers exist at shallow depth, and proper and regular maintenance of the infiltration facilities. If needed, we recommend field Double Ring Infiltrometer Tests (ASTM D3385) be performed at the potential infiltration depths to evaluate the field infiltration rates.

Sand layers with higher infiltration rates may exist below the site at shallow depths in some areas. Compacted/engineered fills should be considered as having limited infiltration ability.

3.7 Geology and Seismicity

According to Brabb et al. (1998)³, the site (below surficial fills) is underlain by Holocene alluvial fan and fluvial deposits that are generally composed of brown or tan, medium dense to dense, gravely sand or sandy gravel that generally grades upward to sandy or silty clay. The adjacent hillside open space slopes are mapped as underlain by Cretaceous and Jurassic Sandstone (part of the Franciscan Complex) that consists of greenish-gray to buff, fine- to coarse-grained sandstone (graywacke) with interbedded siltstone and shale. The subsurface soil conditions encountered by our borings within the site are consistent with the regional geologic mapping.

According to Seismic Hazard Zones Map of the Woodside Quadrangle (2019)⁴, the hillside open space slopes to the south and southwest of the site are located in an earthquake-induced landslide seismic hazard zone as designated by the State of California. California Geological Survey (CGS, 2019)⁵ also mapped a possible landslide on the open space slope to the southwest of the site boundary. U.S. Geological Survey Open-File Report 97-745 (1997)⁶ mapped the site and the adjacent slopes as a typical hillside area that may have few landslides (if any). The open space slopes were also mapped as potential source areas of debris flow.

During our field reconnaissance by our Certified Engineering Geologist, we did not observe evidence of landsliding and adverse drainage conditions within the site. In addition, the existing stone retaining walls (up to about 3 feet high) locating along the base of the slope along the southern site boundary appeared to be in a good to excellent condition. The open space slopes to the south and southwest of the site had numerous small to large diameter trees. No obvious active

³McLaughlin, Clark, Brabb, Helley and Colon, 2001, Geologic Maps and Structure Sections of the Southwestern Santa Clara Valley and Southern Santa Cruz Mountains, Santa Clara and Santa Cruz Counties, California, U.S. Geological Survey, Miscellaneous Field Studies Map MF-2373, Sheet 5.

⁴State of California, 2019, Seismic Hazard Zones, Woodside Quadrangle, Official Map, released: April 4, 2019.

⁵California Geological Survey, 2019, Seismic Hazard Zone Report for the Woodside 7.5-Minute Quadrangle, San Mateo County, California, Seismic Hazard Zone Report 129, Plate 1.2.

⁶Ellen, Mark, Wiczorek, Wentworth, Ramsey, and May, 1997, San Francisco Bay Region Landslide Folio, Part C (Summary Distribution of Slides and Earth Flows) and Part E (Debris Flow Source Maps), U.S. Geological Survey Open-File Report 97-745.

landslide features were observed on the slopes south of the site, however the hillside was mantled with colluvial soils that show evidence of active soil creep. Old, eroded landslide features were observed directly below Fox Crossing Court, a roadway located upslope of the site and the features were covered with a thick growth of small and large diameter trees. Surface water flows from the hillsides to the south of the site are directed toward and into the site. Figure 2 shows the results of our engineering geology mapping.

It is our opinion that, based on the results of geologic literature review, field reconnaissance, and subsurface explorations, the potential for landsliding at the planned development is low provided the onsite slopes are regularly and properly maintained, and the recommendations contained in this report (which include surface and subsurface drainage) are implemented in the design and construction of the project.

The project site is in the San Francisco Bay Area, which is considered one of the most seismically active regions in the United States. Significant earthquakes have occurred in the region and are believed to be associated with crustal movements along a system of sub-parallel fault zones that generally trend in a northwesterly direction. According to the Alquist-Priolo Earthquake Fault Zones Map of the Woodside Quadrangle (1974)⁷, the site is not located in an earthquake fault zone as designated by the State of California. Therefore, it is our opinion that the potential for ground surface rupture due to a fault crossing the site is low.

Earthquake intensities will vary throughout the region, depending upon numerous factors including the magnitude of earthquake, the distance of the site from the causative fault, and the type of materials underlying the site. The U.S. Geological Survey (2016)⁸ indicated that there is a 72 percent chance of at least one magnitude 6.7 or greater earthquake striking the San Francisco Bay region between 2014 and 2043. Therefore, the site will be subjected to earthquakes that cause strong ground shaking.

According to 2022 CBC/ASCE 7-16, the site modified geometric mean peak ground acceleration (PGA_M) from a Maximum Considered Earthquake (MCE) event is estimated to be about 0.99g based on a stiff soil condition (Site Class D). The MCE peak ground acceleration generally has a 2% probability of being exceeded in 50 years (a mean return period of 2,475 years) except where deterministically capped along highly active faults.

According to the U.S. Geological Survey's Unified Hazard Tool and applying the Dynamic: Conterminous U.S. 2014 model (v4.2.0)⁹, the resulting deaggregation calculations indicate that

⁷State of California, Earthquake Fault Zones, Woodside Quadrangle, Official Map, Effective: January 1, 1974.

⁸Aagaard, Blair, Boatwright, Garcia, Harris, Michael, Schwartz, and DiLeo, 2016, Earthquake Outlook for the San Francisco Bay Region 2014–2043, USGS Fact Sheet 2016–3020 (ver. 1.1, August 2016).

⁹USGS Unified Hazard Tool, <https://earthquake.usgs.gov/hazards/interactive/>, accessed 12/16/2024.

the site has a 10% probability of exceeding a peak ground acceleration of about 0.57g in 50 years (a design earthquake ground motion based on a Site Class D with a mean return period of 475 years).

The actual ground surface acceleration might vary depending upon the local seismic characteristics of the underlying bedrock and the overlying soils.

3.8 Liquefaction

Soil liquefaction is a phenomenon primarily associated with saturated cohesionless soil layers. These soils can dramatically lose strength due to increased pore water pressure during cyclic loading, such as imposed by earthquakes. During the loss of strength, the soils acquire mobility sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose, uniformly graded, saturated sands that lie close to the ground surface; although, liquefaction can also occur in fine-grained soils, such as low-plasticity silts.

According to Seismic Hazard Zones Map of the Woodside Quadrangle (2019)¹⁰, the site is located in a liquefaction seismic hazard zone as designated by the State of California. The site and surrounding areas are mapped by Witter et al. (2006)¹¹ as being within an area having a high susceptibility to liquefaction hazard.

To evaluate the soil liquefaction potential of the site, we performed SPT-based seismic soil liquefaction triggering and post-liquefaction deformation analyses in general accordance with the guidelines in CGS Special Publication 117A (2008)¹² and the recommended procedures by Southern California Earthquake Center (SCEC, 1999)¹³. The analyses were performed by using methodologies developed by Idriss and Boulanger (2008 and 2014)^{14,15}. Subsurface soil data from our borings and an assumed groundwater level at 10 feet deep were used in our liquefaction analyses.

¹⁰State of California, 2019, Seismic Hazard Zones, Woodside Quadrangle, Official Map, released: April 4, 2019.

¹¹Witter, Knudsen, Sowers, Wentworth, Koehler, and Randolph, 2006, Maps of Quaternary Deposits and Liquefaction Susceptibility in the Central San Francisco Bay Region, California, U.S. Geological Survey Open File Report 2006-1037.

¹²California Geological Survey, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, CGS Special Publication 117A.

¹³Southern California Earthquake Center, 2002, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, June.

¹⁴Idriss and Boulanger, 2008, Soil Liquefaction During Earthquakes, EERI Monograph MNO-12.

¹⁵Boulanger and Idriss, 2014, CPT and SPT Based Liquefaction Triggering Procedures, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, Report No. UCD/CGM-14/01, April.

A site modified peak ground acceleration (PGA_M) of 0.99g from a Maximum Considered Earthquake (MCE) event (per 2022 California Building Code and based on Site Class D stiff soil) and a modal earthquake magnitude (M_w) of 7.9 (based on the USGS Unified Hazard Tool deaggregation model with a return period of 2,475 years) were used in our analyses. Factor of safety against liquefaction triggering is defined as Cyclic Resistance Ratio (CRR) divided by the Cyclic Stress Ratio (CSR). Saturated soils are considered to be potentially liquefiable if the calculated factor of safety against liquefaction triggering is less than about 1.3 (per CGS Special Publication 117A). Post-liquefaction residual volumetric strains and deformations were estimated for the potentially liquefiable soils based on the methodologies used.

The results of our analyses indicate that the loose to medium dense sand lenses encountered in our borings below the site have a moderate to high potential for liquefying where they are subjected to an MCE earthquake event. These sand lenses appear to be in the form of randomly distributed pockets of possibly 2 to 4 feet in thickness. We estimate the MCE earthquake induced liquefaction in these sand lenses could result in residual volumetric strains varying from about 1 to 3 percent and may cause total ground surface settlements up to about 2 inches at the site.

The actual ground surface damage or manifestation depends on characteristics of the triggering earthquakes and thicknesses of overlying non-liquefiable soil crust and underlying liquefiable soils as indicated by the empirical criteria developed by Ishihara (1985)¹⁶. According to Bowen and Jacka (2013)¹⁷, the observations after the 2010-2011 Canterbury earthquakes (M_w 7.1, 6.2, 5.9 and 6.0) suggest liquefaction induced ground damage likely did not occur in locations where the non-liquefiable soil crust thickness is greater than 4 meter thick (or about 13 feet thick). It is our opinion the potential for ground surface damage or manifestation due to liquefaction occurring at the site (with a non-liquefiable soil crust of about 10 feet over randomly distributed potential liquefiable sand pockets of possibly 2 to 4 feet in thickness) is low.

It is our understanding that the current building codes and the California statewide safety standard as outlined in California Geologic Survey (CGS) Special Publication 117A are only intended to protect life-safety and prevent collapse of non-essential buildings but not to prevent structural damage. To reduce the potential soil liquefaction impacts, the building foundations and superstructures should be designed to tolerate differential settlements caused by liquefaction without loss of the ability to support gravity loads. We recommend the building foundations and superstructures be designed to resist 2 inches of total settlement and 1 inch of differential settlement. The magnitude of differential settlement could occur directly below the center of a

¹⁶Ishihara, 1985, Stability of Natural Deposits During Earthquakes, Proceedings of the Eleventh International Conference on Soil Mechanics and Foundation Engineering, San Francisco, CA Volume 1, p. 321-376.

¹⁷Bowen and Jacka, 2013, Liquefaction Induced Ground Damage in the Canterbury Earthquakes: Predictions vs. Reality, Proceedings 19th NZGS Geotechnical Symposium, Queenstown.

building shallow foundation system (or over a distance of about 30 feet) as a result of a “cupping” shape of the underlying supporting subgrade after soil liquefaction.

Underground pipelines (such as gas lines, sanitary sewers, storm drains, and water services) should be properly designed to compensate for the settlement caused by the liquefaction of the underlying supporting soils. It should be noted that after a liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as structures, driveways, roadways, and utilities can occur and may require repair.

3.9 Lateral Spreading

As part of our soil liquefaction analyses, we evaluated the potential for lateral spreading impacting the site. Lateral spreading occurs when soils liquefy during an earthquake event, and the liquefied soils along with the overlying soils move laterally to unconfined spaces (such as the nearby Cordilleras Creek), which causes significant horizontal ground displacements.

The potentially liquefiable sand lenses below the site appear to be in the form of randomly distributed pockets of possibly 2 to 4 feet in thickness and have an equivalent SPT- $N_{1,60}$ value greater than or close to 15, which is considered by Youd et al. (2002)¹⁸ as unlikely to cause significant lateral spreading for an earthquake event of smaller than moment magnitude 8. In addition, the site boundary is located at least 70 feet from the bank of the relatively shallow creek (about 10 to 12 feet deep).

Based on our review of available literature, the results of the field explorations, and results of our analyses, it is our opinion that the potential for lateral spreading adjacent to the nearby Cordilleras Creek adversely impacting the site development is low.

¹⁸Youd, Hansen, and Bartlett, 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement, *Journal of Geotechnical and Geoenvironmental Engineering*, p. 1007-1017, December.

4.0 CONCLUSIONS AND RECOMMENDATIONS

It is our opinion that the site is suitable for the proposed project from a geotechnical engineering standpoint. The conclusions and recommendations presented in this report should be incorporated in the design and construction of the project to reduce soil or foundation related issues. The following are the primary geotechnical considerations for development of the site.

WEAK SOIL AND FILL MATERIALS: As described in **Section 3.4**, our Boring B-1 encountered about 2 feet of sandy fills at the ground surface. These fills are heterogenous, and potentially weak and compressible. The removal of the previous orchard trees might have weakened the upper about 2 to 3 feet of surface soils within the northern half of the western large parcel. We also anticipate that the demolition of the existing structures (and their associated foundations) and improvements will disturb and weaken the upper 2 to 3 feet of surface soils in the existing development area.

In order to reduce the potential for damaging differential settlement of overlying improvements (such as structural fill, building foundations, driveways, exterior flatwork, and pavements), we recommend these weak soils and fills be over-excavated and re-compacted. The process can consist of over-excavating to depths of about 2 feet below the existing ground surface, scarifying and compacting the bottom 12 inches before placing any new fill, and placing well-blended, moisture conditioned, and properly compacted fill over the properly prepared subgrade. Deeper removal will be needed in areas if thicker weak soils or fills are encountered during grading. The over-excavation should extend to depths where competent soils are encountered. The cuts for new roads will most likely remove most of these weak soils and fills.

Over-excavation and re-compaction should extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible. There would be no need to over-excavate and re-compact the soils or fills within areas that do not support improvements, such as within planned open space areas where ground instability or settlement is less a concern. Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent properties are not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below proposed building foundations. The extent of the removal and re-compaction may vary across the site and should be determined in the field by SFB at the time of the earthwork operation. The removed soil and fill materials can be reused as new engineered fills at the site.

SURFACE WATER FLOWS: Surface water from the hillsides south of the site is directed toward and into the planned development area. We recommend the project's Civil Engineer

account for potential water flow volumes from the hillsides and design concrete ditches to collect and transmit the water to appropriate facilities prior to the water impacting the planned building pads.

LIQUEFACTION SETTLEMENT: As described in **Section 3.8** of this report, our analyses indicate that the loose to medium dense sand lenses encountered in our borings below the site have a moderate to high potential for liquefying where they are subjected to an MCE earthquake event. These sand lenses appear to be in the form of randomly distributed pockets of possibly 2 to 4 feet in thickness. We estimate the MCE earthquake induced liquefaction in these sand lenses could result in residual volumetric strains varying from about 1 to 3 percent and may cause total ground surface settlements up to about 2 inches at the site.

To reduce the potential soil liquefaction impacts, the building foundations and superstructures should be designed to tolerate differential settlements caused by liquefaction without loss of the ability to support gravity loads. We recommend the building foundations and superstructures be designed to resist 2 inches of total settlement and 1 inch of differential settlement. The magnitude of differential settlement could occur directly below the center of a building shallow foundation system (or over a distance of about 30 feet) as a result of a “cupping” shape of the underlying supporting subgrade after soil liquefaction.

Underground pipelines (such as gas lines, sanitary sewers, storm drains, and water services) should be properly designed to compensate for the settlement caused by the liquefaction of the underlying supporting soils. It should be noted that after a liquefaction event, phenomena such as sand boils, ground cracking, and differential movement of overlying improvements such as structures, driveways, roadways, and utilities can occur and may require repair

DISSIMILAR SUBGRADE SOILS AND SHRINKAGE/EXPANSION POTENTIAL: As described in **Section 3.4**, the near-surface more clayey soils have a medium plasticity and moderate expansion and shrinkage potential. However, the sandy soils are non-plastic and have a low expansion and shrinkage potential.

To reduce the potential for damaging differential movement of the proposed structures, we recommend the proposed grading be performed so that the buildings and surrounding flatwork can be supported on fills with similar expansion potential. In no case should the buildings be underlain by subgrade (and within 3 feet of subgrade) consisting of both expansive clayey soils or fills and relatively non-expansive sandy soils or fills.

We recommend at least 3 feet thick of well-blended, moisture conditioned, engineered fill be provided below the buildings and surrounding flatwork located. The compacted, engineered fill

layers should extend at least 5 feet beyond building footprints and at least 3 feet beyond surrounding exterior flatwork, whichever is greater.

CORROSION POTENTIAL: Two selected onsite soil samples were tested for pH (ASTM D4972), chlorides (ASTM D4327), sulfates (ASTM D4327), sulfides (ASTM D4658M), resistivity at 100% saturation (ASTM G57), and Redox potential (ASTM D1498) for use in evaluating the potential for corrosion on concrete and buried metal, such as utilities and reinforcing steel. The results of these tests and brief evaluation summary of the results are included in **Appendix B**. We recommend these test results and brief evaluation summary be forwarded to your concrete contractors, underground contractors, pipeline designers, and foundation designers and contractors so they can design and install corrosion protection measures.

Please be aware that we are not corrosion protection experts; we recommend corrosion protection measures be designed and constructed so that all concrete and metal, including foundation reinforcement, are protected against corrosion. We also recommend additional testing be performed if the test results are deemed insufficient by the designers and installers of the corrosion protection. Landscaping soils typically contain fertilizers and other chemicals that can be highly corrosive to metals and concrete; landscaping soils commonly are in contact with foundations. Consideration should be given to testing the corrosion potential characteristics of proposed landscaping soils and other types of imported or modified soils in order to design and provide protection against corrosion for the foundation and pipelines.

SEEPAGE, SURFACE, AND SUBSURFACE WATER: Water seepage will occur during and after periods of rainfall and as a result of irrigation by “upstream” neighbors. To reduce the potential for seepage below and within planned improvements, we recommend installing subdrains where surface and seepage water is directed from open spaces toward planned improvements such as along the base of the hillside slopes along the southern site boundary. After construction is complete, seepage may occur below the ground surface resulting from irrigation and storm water flow develop over time. Surface water should not be allowed to flow over the top of slopes and retaining walls. The actual location and extent of subdrains should be assessed by SFB during the development of the grading and improvement plans, and determined in the field by SFB at the time of construction.

EROSION AND SLOPE MAINTENANCE: Drainage and erosion control measures should be maintained during and after construction. Short-term and long-term erosion control are critical for the stability of any exposed cut and fill slopes, and may be necessary for the natural slopes in order to reduce sediment accumulation in the drainage systems. We recommend all exposed cut and fill slopes be seeded or planted with appropriately designed erosion resistant vegetation and fertilizer. The vegetation should be appropriately irrigated in order to establish and maintain growth. Over-watering must be avoided in order to reduce surficial instability and erosion. Vegetation should be

deeply rooted to aid in the interlocking of the near-surface soils. Additional seeding and planting may be necessary in localized areas if the initial seeding or planting is unsuccessful. After seeding, fertilizing, and planting, staked erosion control blankets might be necessary to further stabilize the surficial soils.

Additional erosion control measures will need to be designed and implemented prior to the rainy season based upon the site's configuration. The measures could include straw wattles, silt fencing, hay bales, sediment collection basins, and filtration systems. Silt fencing should be designed for the site's soil type. Storm water discharge and release points from silt fencing should be designed to reduce erosion. In areas exposed to winter rains, we recommend an erosion control plan be prepared and implemented at least one month prior to the beginning of the rainy season. The erosion control measures will require inspection, modification, and re-mediation during the rainy season in order to comply with regulatory requirements.

ADDITIONAL RECOMMENDATIONS: Detailed earthwork, underground utility, drainage, building foundation, retaining wall, and pavement recommendations for use in design and construction of the project are presented below. We recommend SFB review the design and specifications to verify that the recommendations presented in this report have been properly interpreted and implemented in the design, plans, and specifications. We also recommend SFB be retained to provide consulting services and to perform construction observation and testing services during the construction phase of the project to observe and test the implementation of our recommendations, and to provide supplemental or revised recommendations in the event conditions different than those described in this report are encountered. We assume no responsibility for misinterpretation of our recommendations.

It is the responsibility of the contractors to provide safe working conditions at the site at all times. We recommend all OSHA regulations be followed, and excavation safety be ensured at all times. It is beyond our scope of work to provide excavation safety designs.

4.1 Earthwork

4.1.1 Clearing and Site Preparation

The site should be cleared of all obstructions, including existing structures and their entire foundation systems, existing driveway, designated trees and their associated entire root systems, and debris. Holes resulting from the removal of underground obstructions extending below the proposed finish grade should be cleared and backfilled with fill materials as specified in **Section 4.1.5, *Fill Material***, and compacted to the requirements in **Section 4.1.6, *Compaction***. Tree roots may extend to depths of about 3 to 4 feet. Wells and septic systems, if they exist, should be abandoned in accordance with San Mateo County standards.

From a geotechnical standpoint, any existing trench backfill materials, clay or concrete pipes, pavements, baserock, and concrete that are removed can be used as new fill onsite provided debris is removed and it is broken up to meet the size requirement for fill material in **Section 4.1.5, *Fill Material***. We recommend fill materials composed of broken up concrete or asphalt concrete not be located within 3 feet of the ground surface in yard areas. Consideration should be given to placing these materials below pavements, directly under building footprints, or in deeper excavations. We recommend backfilling operations for any excavations be performed under the observation and testing of SFB. Crushed concrete materials from concrete demolition can be re-used onsite as aggregate base or subbase if they meet current Caltrans specifications for aggregate base or subbase based on laboratory testing results.

We recommend that at least two weeks prior to grading, areas containing surface vegetation be mowed and the cut grasses and weeds removed from the site or stockpiled for use in landscaping. After mowing, the site should be disced or stripped. Portions of the site containing heavy surface vegetation that is not removed by discing should be stripped to an appropriate depth to remove these materials. The amount of actual stripping should be determined in the field by SFB at the time of construction. Stripped materials should be removed from the site or stockpiled for later use in landscaping, if desired.

4.1.2 Weak Soil and Fill Re-Compaction

As described previously, Boring B-1 encountered about 2 feet of sandy fills at the ground surface. These fills are heterogenous, and potentially weak and compressible. The removal of the previous orchard trees might have weakened the upper about 2 to 3 feet of surface soils within the northern half of the western large parcel. We also anticipate that the demolition of the existing structures (and their associated foundations) and improvements will disturb and weaken the upper 2 to 3 feet of surface soils in the existing development area.

In order to reduce the potential for damaging differential settlement of overlying improvements (such as structural fill, building foundations, driveways, exterior flatwork, and pavements), we recommend these weak soils and fills be over-excavated and re-compacted. The process can consist of over-excavating to depths of about 2 feet below the existing ground surface, scarifying and compacting the bottom 12 inches before placing any new fill, and placing well-blended, moisture conditioned, and properly compacted fill over the properly prepared subgrade. Deeper removal may be needed in areas if thicker weak soils or fills are encountered during grading. The over-excavation should extend to depths where competent soils are encountered. The cuts for new roads will most likely remove most of these weak soils.

Over-excavation and re-compaction should extend at least 5 feet beyond building footprints and at least 3 feet beyond exterior flatwork (including driveways) and pavement wherever possible. There would be no need to over-excavate and re-compact the soils or fills within areas that do not support improvements, such as within planned open space areas where ground instability or settlement is less a concern. Where the over-excavation limits abut adjacent property, SFB should be consulted to determine the actual vertical and lateral extent of over-excavation so that adjacent properties are not adversely impacted. Over-excavations should be performed so that no more than 5 feet of differential fill thickness exists below proposed building foundations. The extent of the removal and re-compaction may vary across the site and should be determined in the field by SFB at the time of the earthwork operation.

The removed soil and fill materials may be used as new fill onsite provided they satisfy the recommendations provided in **Section 4.1.5, *Fill Material***. Compaction should be performed in accordance with the recommendations in **Section 4.1.6, *Compaction***.

4.1.3 Building Pad

To reduce the potential for damaging differential movement of the proposed structures, we recommend the proposed grading be performed so that the buildings and surrounding flatwork can be supported on fills with similar expansion potential. In no case should the buildings be underlain by subgrade (and within 3 feet of subgrade) consisting of both expansive clayey soils or fills and relatively non-expansive sandy soils or fills.

We recommend at least 3 feet thick of well-blended, moisture conditioned, engineered fill be provided below the buildings and surrounding flatwork. The compacted, engineered fill layers should extend at least 5 feet beyond building footprints and at least 3 feet beyond surrounding exterior flatwork, whichever is greater.

4.1.4 Subgrade Preparation

After the completion of clearing, site preparation, and weak soil re-compaction, soil exposed in areas to receive improvements (such as structural fill, building foundations, driveways, exterior flatwork, and pavements) should be scarified to a depth of about 12 inches, moisture conditioned to approximately 2 to 3 percent over optimum water content, and compacted to the requirements for structural fill. Subgrade preparation would not be necessary in areas where over-excavation and re-compaction of the surface soils have occurred.

If completed building pads, driveway and pavement subgrades are allowed to remain exposed to sun, wind or rain for an extended period of time, are heavily disturbed by vehicle traffic or animal borrowing, or experience vegetation growth, the exposed pads and subgrades may need to be

reconditioned (moisture conditioned and/or scarified and recompacted) prior to foundation or pavement construction. SFB should be consulted on the need for pad and subgrade reconditioning.

4.1.5 Fill Material

From a geotechnical and mechanical standpoint, onsite soil and fill materials having an organic content of less than 3 percent by volume can be used as fill. Fill should not contain rocks or lumps larger than 6 inches in greatest dimension with not more than 15 percent larger than 2.5 inches. Larger sized rock may be used as fill onsite provided it is closely monitored, placed properly to achieve compaction, and are located at depths below anticipated, future excavations; SFB should be consulted regarding the use of larger rock pieces in fill materials. Imported fill should have a plasticity index of 15 or less and have a significant amount of cohesive fines.

In addition to the mechanical property specifications, all imported fill material should have a resistivity (100% saturated) no less than the resistivity for the onsite soils, a pH of between approximately 6.0 and 8.5, a total water-soluble chloride concentration less than 300 ppm, and a total water-soluble sulfate concentration less than 500 ppm. We recommend import samples be submitted for corrosion and geotechnical testing at least two weeks prior to being brought onsite.

4.1.6 Compaction

Within the upper 5 feet of the finished ground surface, we recommend structural fill be compacted at least 90 percent relative compaction, and structural fill below a depth of 5 feet be compacted to at least 95 percent relative compaction, as determined by ASTM D1557 (latest edition). We recommend the new fill be moisture conditioned approximately 2 to 3 percent over optimum water content. The upper 6 inches of subgrade soils beneath pavements should be compacted to at least 95 percent relative compaction. Fill material should be spread and compacted in lifts not exceeding approximately 8 to 12 inches in un-compacted thickness.

4.1.7 Utility Trench Backfill

Pipeline trenches should be backfilled with fill placed in lifts of approximately 8 inches in un-compacted thickness. Thicker lifts can be used provided the method of compaction is approved by SFB and the required minimum degree of compaction is achieved. Backfill should be placed by mechanical means only. Jetting is not permitted.

Onsite trench backfill should be compacted to at least 90 percent relative compaction. Imported sand trench backfill should be compacted to at least 95 percent relative compaction and sufficient water is added during backfilling operations to prevent the soil from "bulking" during compaction. The upper 3 feet of trench backfill in foundation, slab, and pavement areas should be entirely compacted to at least 95 percent relative compaction. To reduce piping and settlement of overlying

improvements, we recommend crushed rock bedding and backfill (if used) be completely surrounded by a filter fabric such as Mirafi 140N (or equivalent); alternatively, filter fabric would not be necessary if Caltrans Class 2 permeable material is used in lieu of crushed rock bedding and backfill.

Sand or gravel backfilled trench laterals that extend toward driveways, exterior slabs-on-grade, or under the building foundations, and are located below irrigated landscaped areas such as lawns or planting strips, should be plugged with onsite clays, low strength concrete, or sand/cement slurry. The plug for the trench laterals should be located below the edge of pavement or slabs, and under the perimeter of the foundation. The plug should be at least 24 inches thick, extend across the entire width of the trench, and extend from the bottom of the trench to the top of the sand or gravel backfill.

Where the bottoms of trenches are sloped steeper than 5 percent, we recommend a low permeability plug composed of low strength concrete, sand/cement slurry, or onsite clays be installed in the utility trenches every 50 feet on-center. The plug will reduce piping/consolidation from water seepage that may cause roadway and trench surface settlement. The plug should be at least 12 inches thick and extend to within 1 foot of the finished ground surface or to the base of the pavement section.

4.1.8 Exterior Flatwork

We recommend that exterior slabs (including driveways, patios, and walkways) be placed directly on the properly compacted fills. If imported granular materials are placed below these elements, subsurface water can seep through the granular materials and cause the underlying soils to saturate, pipe, and/or heave upward. Prior to placing concrete, subgrade soils should be moisture conditioned to increase their moisture content to approximately 2 to 3 percent above laboratory optimum moisture (ASTM D-1557).

The soils at the site could be subjected to volume changes during fluctuations in moisture content. As a result of these volume changes, some vertical movement of exterior slabs (such as driveways, sidewalks, patios, exterior flatwork, etc.) should be anticipated. This movement could result in damage to the exterior slabs and might require periodic maintenance or replacement. Adequate clearance should be provided between the exterior slabs and building elements that overhang these slabs, such as window sills or doors that open outward.

We recommend reinforcing exterior slabs with steel bars in lieu of wire mesh. To reduce potential crack formation, the installation of #4 bars spaced at approximately 18 inches on center in both directions should be installed. Score joints and expansion joints should be used to control cracking and allow for expansion and contraction of the concrete slab. We recommend appropriate flexible,

relatively impermeable fillers be used at all cold/expansion joints. The installation of dowels at all expansion and cold joints will reduce differential slab movements; the dowels should be at least 30 inches long and should be spaced at a maximum lateral spacing of 18 inches. Although exterior slabs that are adequately reinforced will still crack, trip hazards requiring replacement of the slabs will be reduced if the slab are properly reinforced.

We do not recommend the use of flatwork having permeable joints (such as pavers or tiles with sand or gravel infilled joints) unless the underlying soil subgrade is protected against water seepage or ponding. If not protected, the underlying subgrade will heave, settle, and/or pipe and cause damage to the overlying improvements.

4.1.9 Construction During Wet Weather Conditions

If construction proceeds during or shortly after wet weather conditions, the moisture content of onsite soils could be significantly above optimum. Consequently, subgrade preparation, placement and/or reworking of onsite soil or fills as structural fill might not be possible. Alternative wet weather construction recommendations can be provided by our representative in the field at the time of construction, if appropriate. All the drainage measures recommended in this report should be implemented and maintained during and after construction, especially during wet weather conditions.

4.1.10 Surface Drainage, Irrigation, and Landscaping

Ponding of surface water must not be allowed on pavements, adjacent to foundations, at the top or bottom of slopes, and at the top or adjacent to retaining walls. Ponding of water should also not be allowed on the ground surface adjacent to or near exterior slabs, including walkways and driveways. Surface water should not be allowed to flow over the top of slopes, down slope faces, or over retaining walls.

Surface drainage should be designed in accordance with the latest edition of the California Building Code to direct surface water away from the foundations and toward suitable discharge facilities. Roof downspouts and landscaping drainage inlets should be connected to solid pipes that discharge the collected water into appropriate water collection facilities.

In order to reduce differential foundation movements, landscaping (where used) should be placed uniformly adjacent to foundations and exterior slabs. We recommend trees be no closer to structures or exterior slabs than half the mature height of the tree; in no case should tree roots be allowed to extend near or below foundations or exterior slabs. Installation of root barriers may be necessary to restrict root development below structures.

Drainage inlets should be provided within enclosed planter areas and collected water should be discharged onto pavement, into drainage swales, or into storm water collection systems. In order to reduce the potential for water seepage, consideration should be given to lining planting areas and collecting the accumulated water in subdrain pipes that discharge to appropriate collection facilities. The drainage should be designed and constructed so that the moisture content of the soils surrounding the foundations do not become elevated and no ponding of water occurs. The inlets should be kept free of debris and be lower in elevation than the adjacent ground surface.

We recommend regular maintenance of the drainage systems be performed, including maintenance prior to rainstorms. The inspection should include checking drainage patterns to make sure they are performing properly, making sure drainage systems and inlets are functional and not clogged, and checking that erosion control measures are adequate for anticipated storm events. Immediate repairs should be performed if any of these measures appears to be inadequate.

Irrigation should be performed in a uniform, systematic manner as equally as possible on all sides of the foundations and exterior slabs to maintain moist soil conditions. Over-watering must be avoided. To reduce moisture changes in the natural soils and fills in landscaped areas, we recommend that drought resistant plants and low flow watering systems be used. All irrigation systems should be regularly inspected for leakage.

4.1.11 Subsurface Drainage

To reduce the potential for seepage below and within planned improvements, we recommend installing subdrains where surface and seepage water is directed toward planned improvements such as along the base of the hillside slopes along the southern site boundary. During the earthwork operations, additional subdrains may be necessary in areas of encountered or anticipated seepage. We recommend a subdrain be located below lined ditches or earthen swales that collect surface water from open space areas. The location and extent of subdrains should be assessed by SFB during the development of the grading and improvement plans, and determined in the field by SFB at the time of construction.

Where used, subdrains should consist of 4 inch diameter, rigid perforated pipe (perforations down) surrounded by free draining, uniformly graded, 1/2 to 3/4 inch crushed gravel wrapped in filter fabric such as Mirafi 140N or equivalent. The pipe should be underlain by about 1/2 to 1 inch of gravel, and on the sides by at least 4 inches of gravel. The filter fabric should overlap approximately 12 inches or more at joints. Subdrains should be connected to a solid, rigid, collector pipe with a minimum diameter of 4 inches. Subdrain pipes should consist of rigid PVC SDR-35 or PVC A-2000 (or equal) for fills less than 20 feet in height. Collector pipes should be connected to appropriate discharge facilities such as storm drains, drainage inlets, or storm drain manholes. Subdrain clean-outs should be provided. The clean-out locations should be based upon the reach

of the rotary cleaning systems and the restrictions of pipe bends. Caltrans Class 2 permeable material may be used in lieu of gravel and filter fabric.

Where used, subdrain trenches should be at least 12 inches wide and about 4 feet deep below adjacent ground surface. If a subdrain trench extends to the ground surface and is not covered with concrete lined ditch or concrete flatwork, we recommend the subdrain trench be covered with a 12-inch thick cap consisting of clayey soil compacted to at least 90 percent relative compaction.

4.1.12 Storm Water Treatment Facilities

To satisfy local and state permit requirements, most new development projects must control pollutant sources and reduce, detain, retain, and/or treat specified amounts of storm water runoff. The intent of these types of storm water treatment facilities is to conserve and incorporate on-site natural features, together with constructed hydrologic controls, to more closely mimic pre-development hydrology and watershed processes. These facilities include bio-retention swales and basins, porous paver and pavement, water detention basins, and any proprietary underground storage and treatment systems.

In general, we recommend the portion of the storm water treatment facilities that are within 10 feet of structure foundations and improvements (such as building foundations, exterior flatwork, and pavements) be lined with a relatively impermeable membrane to reduce water seepage and the potential for damage and distress to the adjacent structures and improvements. The lining can consist of a relatively impermeable membrane such as STEGO Wrap 15-mil or equivalent. The membrane should be lapped and sealed in accordance with the manufacturer's specifications, including taping joints where pipes penetrate the membrane.

Soil filter/bio-mix materials within basins and swales will consolidate over time causing long-term ground surface settlement. Additional filling within the basins and swales over time will be needed to maintain design surface elevations. The soil filter/bio-mix materials, infiltration testing and procedures, and associated compaction requirements should be specified by the Civil Engineer and shown in detail on the grading and improvement plans.

Soil filter/bio-mix materials provide little to no lateral restraint of excavation side walls. Sidewalls of bio-retention swale and basin excavations (excavations made prior to the installation of the soil filter/bio-mix) steeper than 2:1 (horizontal to vertical) will experience downward and lateral movements that can cause distresses to adjacent improvements such as foundations, utilities, pavements, driveways, walkways, and curbs and gutters. The magnitude and rate of movement depend upon the swale and basin backfill material type and compaction. To reduce the potential for damaging movements, we recommend 2:1 or flatter excavation sidewall slopes be used for bio-retention swales and basins, sidewalks be setback at least 3 feet from the top of slopes, and creep

sensitive improvements (such as roadway curbs) be setback at least 5 feet from the top of slopes. If the above sidewall slope and setback distance cannot be met, considerations should be given to using below-grade concrete sidewalls that are designed and constructed as retaining walls. Alternatively, deepened sidewalk slab edge or roadway curbs can be used and designed to resist lateral earth pressures and act as a retaining wall. SFB should be consulted to evaluate the need for sidewall restraint when swales or basins are planned. We also recommend SFB observe and document the installation of liners, subdrain pipes, and soil filter/bio-mix materials during construction for conformance to the recommendations in this report and the development's plans and specifications.

Where used, proprietary underground storage and treatment systems should be installed and maintained in accordance with the manufacturer's specifications. In addition, the manufacturer should be consulted for vertical and lateral bearing capacities and anticipated deformations of these systems if they will also support exterior slabs and pavements that are subjected to vehicular traffic.

4.1.13 Future Maintenance

In order to reduce water related issues, we recommend regular inspection and maintenance of the site be performed, including maintenance prior to rainstorms. Inspections should include checking drainage patterns, making sure drainage systems are functional and not clogged, and erosion control measures are adequate for anticipated storm events. Immediate repair should be performed if any of these measures appears to be inadequate. Temporary and permanent erosion and sediment control measures should be installed over any exposed soils immediately after repairs are made. Maintenance should include the re-compaction of loosened soils, collapsing and infilling holes with compacted soils or low strength sand/cement grout, removal and control of digging animals, modifying storm water drainage patterns to allow for sheet flow into drainage inlets or ditches rather than concentrated flow or ponding, removal of debris within drainage ditches and inlets, and immediately repairing any erosion or soil flow.

Differential movement of exterior slabs can occur over time as a result of numerous factors. We recommend development owners perform inspections and maintenance of the slabs, including infilling significant cracks, providing fillers at slab offsets, and replacing slabs if severely damaged.

4.1.14 Additional Recommendations

We recommend that the drainage, irrigation, landscaping, and maintenance recommendations provided in this report be forwarded to your designers and contractors, and we recommend they be also included in disclosure statements given to the development owners and their maintenance associations.

4.2 Foundation Support

4.2.1 Post-Tensioned Slab

The proposed residential buildings can be supported on a post-tensioned slab foundation that is designed for the expansion potential of onsite soils and fills. The building foundations and superstructures should also be designed to tolerate differential settlements caused by liquefaction without loss of the ability to support gravity loads. We recommend the building foundations and superstructures be designed to resist 2 inches of total settlement and 1 inch of differential settlement. The magnitude of differential settlement could occur directly below the center of the building's shallow foundation system (or over a distance of about 30 feet) as a result of a "cupping" shape of the underlying supporting subgrade after soil liquefaction.

The slab foundation should bear entirely on properly prepared and compacted, well-blended structural fill. In no case should a slab foundation bear upon fills with differential expansion characteristics. We recommend at least 3 feet thick of well-blended, moisture conditioned, engineered fill be provided below the buildings and surrounding flatwork. The compacted, engineered fill layers should extend at least 5 feet beyond building footprints and at least 3 feet beyond surrounding exterior flatwork, whichever is greater.

Recommendations for building pad preparation are described previously in **Sections 4.1.3, Building Pad**, and **4.1.4, Subgrade Preparation**, of this report. Prior to the concrete pour, we recommend the moisture content of subgrade materials be conditioned to approximately 2 to 3 percent above laboratory optimum moisture. If the building pads are left exposed for an extended period of time prior to constructing foundations, we recommend SFB be contacted for recommendations to re-condition the pads in order provide adequate building support. We also recommend SFB review the foundation drawings and specifications prior to submittal to verify that the recommendations provided in this report have been used and properly interpreted in the design of the slabs.

The thickness of post-tensioned slab should be determined by the Structural Engineer; however, we recommend the post-tensioned slabs be at least 10 inches thick. In addition, deflection of the slab foundations should be designed not to exceed the values recommended in the most recent PTI design manual. An allowable bearing pressure of 1,500 pounds per square foot can be used for localized point and line loads. We estimate the maximum total static settlement of PT-slab foundations under the recommended allowable bearing pressure to be on the order of 1 inch or less. Differential static settlement is estimated to be approximately 1/2 inch or less.

Lateral loads, such as derived from earthquakes and wind, can be resisted by friction between the post-tensioned slab foundation bottom and the supporting subgrade. A friction coefficient of 0.25 is considered applicable.

At least 10 feet of cover should be provided between the outer face of slabs and un-retained slope faces, as measured laterally between slope faces and the slabs. Where less than 10 feet of cover exists, deepening of the edge of slabs may be necessary in order to achieve 10 feet of cover for buildings located near tops of slopes. Where slabs are located adjacent to utility trenches, the slab bearing surface should bear below an imaginary 1:1 (horizontal to vertical) plane extending upward from the bottom edge of the adjacent utility trench. Alternatively, the slab reinforcing could be increased to span the area defined above assuming no soil support is provided.

A vapor retarder must be placed between subgrade soils and the bottom of the slabs-on-grade. We recommend the vapor retarder consist of a single layer of Stego Wrap Vapor Barrier 15 mil Class A or equivalent provided the equivalent satisfies the following criteria: a permeance as tested before and after mandatory conditioning of less than 0.01 Perms and strength of Class A as determined by ASTM E 1745 (latest edition), and a thickness of at least 15 mils. Installation of the vapor retarder should conform to the latest edition of ASTM E 1643 (latest edition) and the manufacturers requirements, including lapping and all joints at least 6 inches and sealing with Stego Tape or equal in accordance with the manufacturer's specifications. Protrusions where pipes or conduit penetrate the membranes should be sealed with either one or a combination of Stego Tape, Stego Mastic, Stego Pipe Boots, or a product of equal quality as determined by the manufacturer's instructions and ASTM E 1643. Care must be taken to protect the membrane from tears and punctures during construction. We do not recommend placing sand or gravel over the membrane.

Concrete slabs retain moisture and often take many months to dry. Any water added during the concrete pour further increases the curing time. If the slabs are not allowed to completely cure prior to constructing the super-structure, the concrete slabs will expel water vapor which will be trapped under impermeable flooring. The concrete mix design for slabs should have a maximum water/cement ratio of 0.45; the actual water/cement ratio may need to be reduced if the concentration of soluble sulfates or chlorides in the supporting subgrade is detrimental to the concrete. If a higher water/cement ratio is being considered, we recommend higher vapor transmission be taken into account in the design and construction of the buildings. We recommend the foundation designer determine if corrosion protection is needed for the foundation concrete and reinforcing steel. The results of sulfate and chloride testing of onsite soil samples are included in **Appendix B**; the foundation designer should determine if additional testing is needed. In addition, we recommend you consult with your concrete slab designers and concrete contractors regarding methods to reduce the potential for differential concrete curing.

During the curing process, concrete slabs will shrink in volume resulting in cracks developing in the slab. Curing of concrete can take many months (or possibly longer) to complete. These concrete cracks may be visible on the surface of the slab during and after the curing process. In order to reduce the potential for crack propagation through overlying brittle surfaces such as tile or stone flooring, we recommend appropriate crack isolation measures be used between the concrete slab and flooring to reduce the potential for slab cracks to propagate into these brittle flooring surfaces.

An experienced Structural Engineer should design the post-tensioned slabs to resist the differential soil movement. The preliminary soil design parameters presented below were generated using the procedures presented in the 3rd edition of the PTI design manual (2008)¹⁹, PTI standard requirements (2019)²⁰, and a PTI preferred computer program, VOLFLO (Version 1.5 Build 120704), was employed to simulate the wetting and drying scenarios of the soils beneath the post-tensioned slabs.

The values provided below are based upon the post-tensioned slab foundations being entirely surrounded by uniform, properly drained, moderately irrigated landscaping; if differing conditions will exist that will cause differential soil moisture adjacent or below the slabs, or if portions of the foundations will be located adjacent to relatively dry or wet soils, then we should be consulted. Modifications to the design values below would need to be made in writing. Please refer to **Section 4.1.10, Surface Drainage, Irrigation, and Landscaping**, for additional recommendations. We recommend the slab-subgrade friction values provided in the most recent PTI Manual be used in order to determine the friction that might be expected to exist during tendon stressing.

SWELLING MODE

	<u>Center Lift</u>	<u>Edge Lift</u>
Edge Moisture Variation Distance (e_m)	9.0 feet	5.0 feet
Differential Soil Movement (y_m)	0.5 inch	1.0 inch

4.2.2 Retaining Walls

If segmental block walls with geogrid will be used at the site, SFB should be contacted to provide block wall and geogrid designs and specifications. Any walls that retain soils should be designed to resist both lateral earth pressures and any additional lateral loads caused by roadway

¹⁹Post-Tensioning Institute, 2008, Design of Post-Tensioned Slabs-On-Ground (PTI DC10.1-08), Third Edition.

²⁰Post-Tensioning Institute, 2019, Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils (PTI DC10.5-19).

surcharging, earthquake loading, and hydrostatic pressure if wall back-drainage is not provided. The global stability of the walls should also be evaluated where the walls will be located on slopes or where multi-tiered walls will be used.

If walls are allowed to deflect or rotate (unrestrained walls), they can be designed to resist active pressures. If no movement is allowed at the top of walls (restrained walls), at-rest pressures should be used in wall design. The recommended active and at-rest lateral earth pressures under both drained and undrained conditions are provided in the table below.

LATERAL EARTH PRESSURES FOR RETAINING STRUCTURES				
Wall Condition	Backfill Condition	Drained Equivalent Fluid Pressure (pcf)	Undrained Equivalent Fluid Pressure (pcf)	Incremental Seismic Pressure (pcf)
Unrestrained (Active Pressure)	Level	40	85	45
Restrained (At-Rest Pressure)		60	95	90*

*Note: For restrained walls, use the static active pressure and restrained seismic increment in the seismic design.

For retaining walls that need to resist earthquake induced lateral loads from nearby foundations, walls that retain buildings, walls that are to be designed to resist earthquake loads, and any retaining walls that are higher than 6 feet (as required by the 2022 CBC), we recommend the walls be designed to also resist an incremental seismic lateral earth pressure listed in the above table, using a triangular fluid pressure distribution (not inverted). This seismic induced earth pressure is in addition to the active pressures listed above. The seismic lateral earth pressure increments for unrestrained and restrained walls were estimated, respectively, based on 50% and 100% of the peak ground acceleration (PGAM) from a Maximum Considered Earthquake (MCE) earthquake per ASCE 7-16/2022 CBC. Due to the transient nature of the seismic loading, a factor of safety of at least 1.1 can be used in the design of the walls when they resist seismic lateral loads. Some movement of the walls may occur during moderate to strong earthquake shaking and may result in distress as is typical for all structures subjected to earthquake shaking.

Walls with inclined backfill should be designed for an additional equivalent fluid pressure of 1 pound per cubic foot for every 2 degrees of slope inclination. Soil creep forces equal to an equivalent fluid pressure of 125 pcf should also be applied to the upper 3 feet of the wall height where native hill slopes steeper than 3:1 will exist above the walls.

Any surcharge loads located within an imaginary 1:1 (horizontal to vertical) plane projected upward from the base of the walls will increase the lateral earth pressures on the wall. Walls subjected to surcharge loads should be designed for an additional uniform lateral pressure (rectangular distribution) equal to one-third (0.33) and one-half (0.5) the anticipated surcharge load for unrestrained and restrained walls, respectively. Walls adjacent to areas subject to vehicular traffic should be designed for a 2-foot equivalent soil surcharge (250 psf). We should be consulted to provide load contributions from other particular surcharges located behind walls if needed.

It should be noted the lateral earth pressures depend upon the moisture content of the retained soils to be constant over time; if the moisture content of the retained soils will fluctuate or increase compared to the moisture content at time of construction, then SFB should be consulted and provide written modifications to this design criteria.

The above recommended drained lateral earth pressures assume walls are fully back drained to prevent the build-up of hydrostatic pressures. If drainage behind the wall is omitted, the wall should be designed for undrained condition. Wall back-drainage can be accomplished by using 1/2- to 3/4-inch crushed, uniformly graded gravel entirely wrapped in filter fabric, such as Mirafi 140N or equal (an overlap of at least 12 inches should be provided at all fabric joints). The gravel and fabric should be at least 12 inches wide and extend from the base of the wall to within about 1 foot of the finished grade at the top (Class 2 permeable material per Caltrans Specification Section 68 may be used in lieu of gravel and filter fabric). The upper 1 foot of cover backfill should consist of relatively impervious material.

Where wall back-drainage is used, a 4-inch diameter, perforated, PVC SDR-35 pipe should be installed at the base and centered within the gravel. The perforated pipe should be connected to a solid collector pipe that transmits the water directly to suitable discharge facilities. If weep holes are used in the wall, the perforated pipe within the gravel is not necessary provided the weep holes are kept free of animals and debris, are located no higher than approximately 6 inches from the lowest adjacent grade and are able to function properly. Weepholes can be spaced at about 10 to 15 feet apart. As an alternative to using gravel, pre-fabricated drainage panels (such as AWD SITEDRAIN Sheet 94 for walls or equal) may be used behind the walls in conjunction with perforated pipe (connected to solid collector pipe), weep holes, or strip drains (such as SITEDRAIN Strip 6000 or equal).

If heavy compaction equipment is used behind the walls, the walls should be appropriately designed to withstand loads exerted by the heavy equipment and/or temporarily braced. Fill placed behind walls should conform to the recommendations provided in **Section 4.1.5, *Fill Material***, and **Section 4.1.6, *Compaction***.

Retaining walls can be supported on drilled, cast-in-place, straight shaft friction piers that develop their load carrying capacity in the materials underlying the site. The piers should have a minimum diameter of 12 inches and a center-to-center spacing of at least three times the shaft diameter. We recommend that piers be at least 6 feet long. Pier reinforcing should be based on structural requirements, but in no case should less than two #4 bars for the entire length of the pier be used.

The actual design depth of the piers should be determined using an allowable skin friction of 500 pounds per square foot (psf) for dead plus live loads, with a one-third increase for all loads including wind or seismic. Eighty percent of the skin friction value can be used to resist uplift. Lateral load resistance can be developed in passive resistance for pier foundations. We recommend an allowable soil passive resistance (which includes a factor of safety of 1.5) equal to an equivalent fluid weighing 300 pounds per cubic foot be used for pier foundations. This value can be used up to a maximum value of 3,600 psf. The passive resistance can be applied against twice the projected diameter of pier shaft if the piers are spaced center-on-center at least 3 times of the pier shaft diameter.

The upper 2 feet of pier embedment should be neglected in the vertical and passive resistance design as measured from finished grade unless it is confined by a pavement or concrete slab. The portion of the pier shaft located within 10 feet (as measured laterally) of the nearest slope face or above an imaginary 1:1 (horizontal to vertical) plane extending upward from the bottom of any adjacent walls or utility trenches should also be ignored in both the vertical bearing and passive resistance designs.

The bottoms of pier excavations should be relatively dry and free of all loose cuttings or slough prior to placing reinforcing steel and concrete. Any accumulated water in pier excavation should be removed prior to placing concrete. We recommend that the excavation of all piers be performed under the direct observation of SFB to confirm that the pier foundations are founded in suitable materials and constructed in accordance with the recommendations presented herein. Preliminarily, we recommend concrete pour of pier excavations be performed within 24 hours of excavation and prior to any rainstorms. Where caving or high groundwater conditions exist, additional measures such as using dewatering, casing, slurry, tremie methods, and/or pouring concrete immediately after excavating may be necessary. SFB should be consulted for additional measures for pier construction as needed during construction.

As an alternative to using pier foundations to support the walls, footings may be used. Please contact SFB for footing foundation recommendations if footings will be used to support the walls.

4.3 Seismic Design Criteria

As described in **Section 3.8** of this report, some of the soils underlying the site are susceptible to soil liquefaction when subjected to an MCE event. These liquefiable soils are classified by the 2022 California Building Code (CBC) and ASCE 7-16 as Site Class “F” since they are vulnerable to potential failure or collapse under seismic loading. However, it is our understanding that the proposed project structures will have fundamental periods of vibration equal to or less than 0.5 second. Therefore, a site-specific response analysis is not required for liquefiable soils per Section 20.3.1 of ASCE 7-16, and the site class and seismic design parameters can be determined in accordance with ASCE 7-16 and its supplements.

Based on the site geology and subsurface soil conditions encountered at the site, we recommend the site be characterized as Site Class “D”, a “stiff soil” profile. For seismic designs using the 2022 CBC and ASCE 7-16, we recommend the following parameters be used. These parameters were calculated using the ASCE 7 Hazard Tool online program²¹, and are based on the site being located at approximate latitude 37.473151°N and longitude 122.263365°W.

We assumed the proposed project structures are categorized as Risk Category II, and the exceptions of ASCE 7-16 and Supplement 3 Section 11.4.8 (Site-Specific Ground Motion Procedures) will be taken by the Structural Engineer. We should be contacted if any of these assumptions are incorrect or a site-specific ground motion hazard analysis is required.

SEISMIC PARAMETER	DESIGN VALUE
Site Class	D
S_s	2.107
S_1	0.874
F_a	1.000
F_v	See Section 11.4.8 of ASCE 7-16*
S_{MS}	2.107
S_{M1}	See Section 11.4.8 of ASCE 7-16*
S_{DS}	1.405
S_{D1}	See Section 11.4.8 of ASCE 7-16*
SDC	See Section 11.4.8 of ASCE 7-16*
PGA	0.903
F_{PGA}	1.1

²¹ASCE, <https://asce7hazardtool.online/>, accessed 12/16/2024.

SEISMIC PARAMETER	DESIGN VALUE
PGA _M	0.993
T _L	12

*Note: The values of F_v, S_{M1}, S_{D1}, and Seismic Design Category (SDC) should be determined by the Structural Engineer based on the ASCE 7-16 Section 11.4.8 requirements and any applicable supplements.

4.4 Pavements

Based on the results of field explorations and laboratory testing; we recommend an R-value of 5 be used in preliminary asphalt concrete pavement design. We recommend additional R-value tests be performed once the pavement subgrade is established to confirm the R-value used in the design. Pavement subgrade completely composed of sandy and gravelly fills will result in higher R-values and thinner pavement sections.

We developed the following alternative preliminary pavement sections using Topic 608 of the State of California Department of Transportation Highway Design Manual, the recommended R-value, and typical traffic indices for residential developments. The project’s Civil Engineer or appropriate public agency should determine actual traffic indices. The pavement thicknesses shown below are SFB’s recommended minimum values; governing agencies may require pavement thicknesses greater than those shown.

PRELIMINARY PAVEMENT DESIGN ALTERNATIVES			
SUBGRADE R-VALUE = 5			
Location	Pavement Components		Total Thickness (inches)
	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)	
T.I. = 4.5 (auto & light truck parking)	2.5	9.0	11.5
T.I. = 5.0 (access ways/courts)	3.0	10.0	13.0

If the pavements are planned to be placed prior to or during construction, the traffic indices and pavement sections may not be adequate for support of what is typically more frequent and heavier construction traffic. If the pavement sections will be used for construction access by heavy trucks or construction equipment (especially fork lifts with outriggers), SFB should be consulted to provide recommendations for alternative pavement sections capable of supporting the heavier use and heavier loads. If requested, SFB can provide recommendations for a phased placement of the asphalt concrete to reduce the potential for mechanical scars caused by construction traffic in the

finished grade. Preliminary pavement sections should be revised, if necessary, when actual traffic indices are known and pavement subgrade elevations are determined.

We recommend the pavement materials and construction conform to Caltrans Standard Specifications. Pavement aggregate base and asphalt concrete should be compacted to at least 95 percent relative compaction as determined by ASTM D1557 or Caltrans Test Method 375. The asphalt concrete compacted unit weight should be determined using Caltrans Test Method 308-A or ASTM Test Method D1188. Asphalt concrete should also satisfy the S-value requirements by Caltrans.

We recommend regular maintenance of the asphalt concrete be performed at approximately five-year intervals. Maintenance may include sand slurry sealing, crack filling, and chip seals as necessary. If regular maintenance is not performed, the asphalt concrete layer could experience premature degradation requiring more extensive repairs.

5.0 CONDITIONS AND LIMITATIONS

SFB is not responsible for the validity or accuracy of information, analyses, test results, or designs provided to SFB by others or prepared by others. The analysis, designs, opinions, and recommendations submitted in this report are based in part upon the data obtained from our field work and upon information provided by others. Site exploration and testing characterize subsurface conditions only at the locations where the explorations or tests are performed; actual subsurface conditions between explorations or tests may be different than those described in this report. Variations of subsurface conditions from those analyzed or characterized in this report are not uncommon and may become evident during construction. In addition, changes in the condition of the site can occur over time as a result of either natural processes (such as earthquakes, flooding, or changes in ground water levels) or human activity (such as construction adjacent to the site, dumping of fill, or excavating). If changes to the site's surface or subsurface conditions occur since the performance of the field work described in this report, or if differing subsurface conditions are encountered, we should be contacted immediately to evaluate the differing conditions to assess if the opinions, conclusions, and recommendations provided in this report are still applicable or should be amended.

We recommend SFB be retained to provide geotechnical services during design, reviews, earthwork operations, paving operations, and foundation installation to confirm and observe compliance with the design concepts, specifications and recommendations presented in this report. Our presence will also allow us to modify design if unanticipated subsurface conditions are encountered or if changes to the scope of the project, as defined in this report, are made.

This report is a design document that has been prepared in accordance with generally accepted geological and geotechnical engineering practices for the exclusive use of Edenbridge, Inc. and their consultants for specific application to the proposed residential development at 1815 Cordilleras Road in Emerald Hills, California, and is intended to represent our design recommendations to Edenbridge, Inc. for specific application to the proposed project. The conclusions and recommendations contained in this report are solely professional opinions. It is the responsibility of Edenbridge, Inc. to transmit the information and recommendations of this report to those designing and constructing the project. We will not be responsible for the misinterpretation of the information provided in this report. We recommend SFB be retained to review geological and geotechnical aspects of construction calculations, specifications, and plans; we should also be retained to participate in pre-bid and pre-construction conferences to clarify the opinions, conclusions, and recommendations contained in this report.

It should be understood that advancements in the practice of geotechnical engineering and engineering geology, or discovery of differing surface or subsurface conditions, may affect the

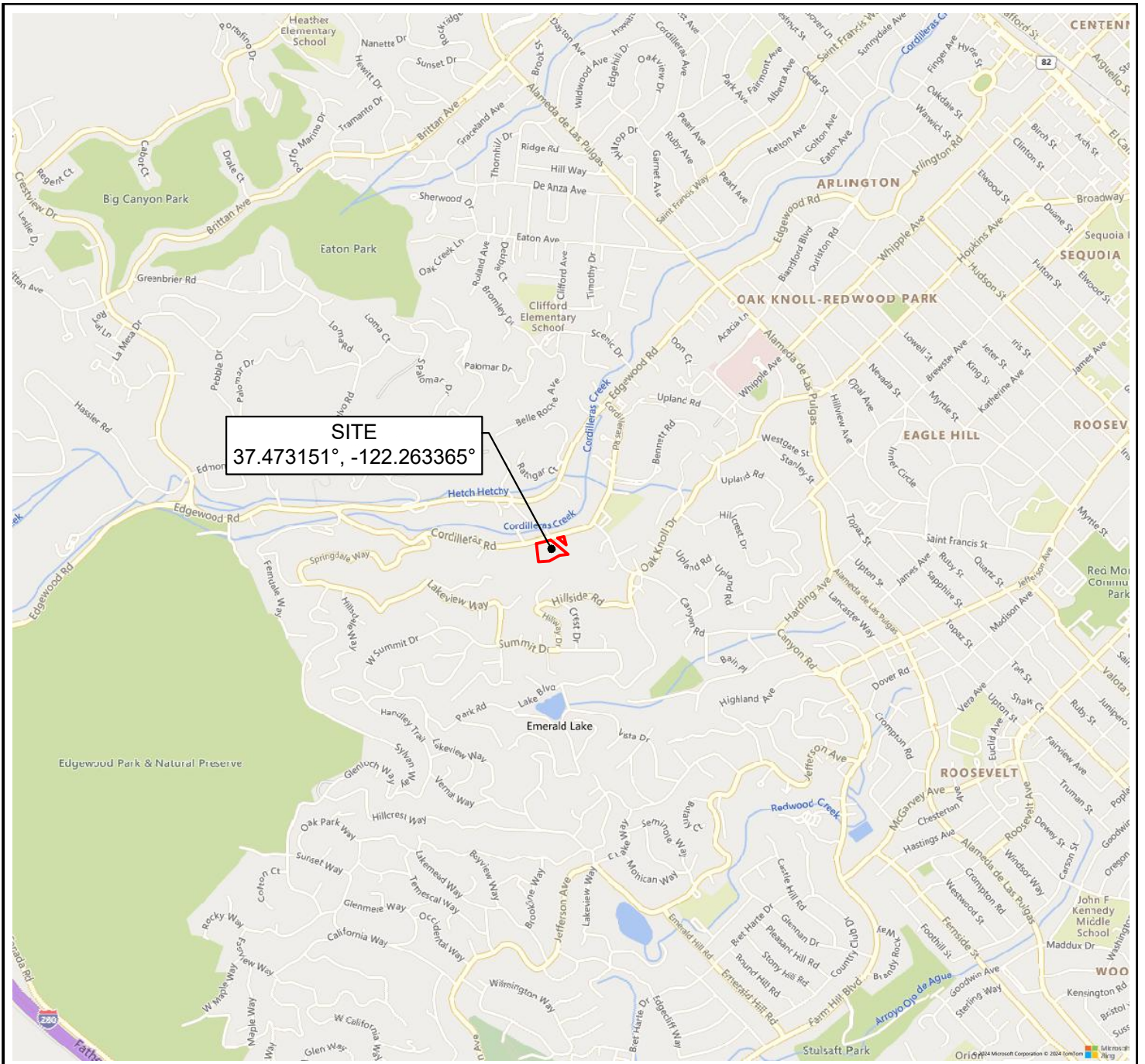
validity of this report and are not uncommon. SFB strives to perform its services in a proper and professional manner with reasonable care and competence but we are not infallible. Geological engineering and geotechnical engineering are disciplines that are far less exact than other engineering disciplines; therefore, we should be consulted if the limitations to using this are not completely understood.

In the event that there are any changes in the nature, design or location of the project, as described in this report, or if any future additions are planned, the conclusions and recommendations contained in this report shall not be considered valid unless we are contacted in writing, the project changes are reviewed by us, and the conclusions and recommendations presented in this report are modified or verified in writing. The opinions, conclusions, and recommendations contained in this report are based upon the description of the project as presented in the introduction section of this report.

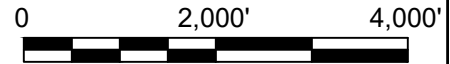
This report does not necessarily represent all of the information that has been communicated by us to Edenbridge, Inc. and their consultants during the course of this engagement and our rendering of professional services to Edenbridge, Inc. Reliance on this report by parties other than those described above must be at their own risk unless we are first consulted as to the parties' intended use of this report and only after we obtain the written consent of Edenbridge, Inc. to divulge information that may have been communicated to Edenbridge, Inc. We cannot accept consequences for use of segregated portions of this report.

Please refer to **Appendix C** for Geoprofessional Business Association (GBA) guidelines regarding use of this report.


FIGURES

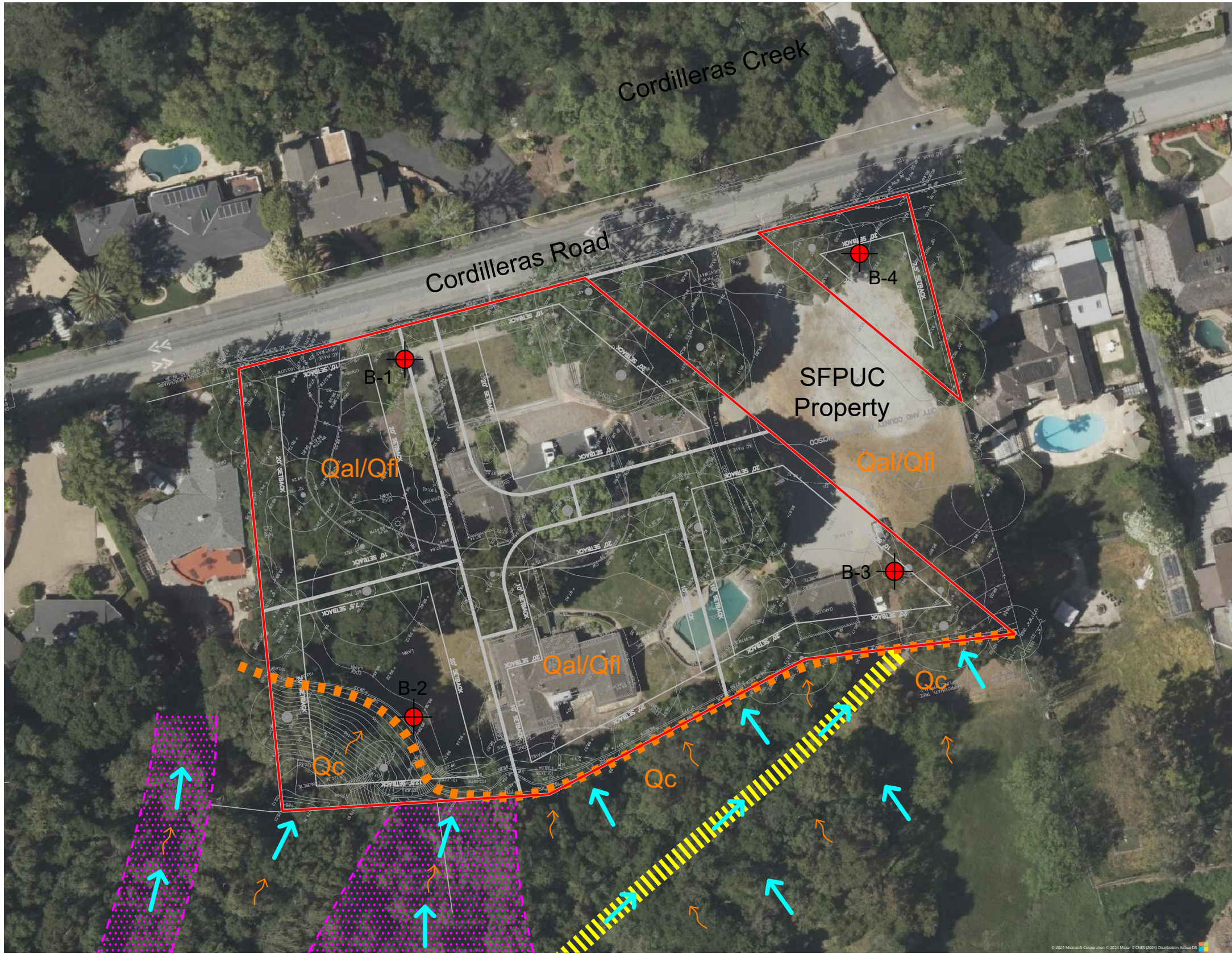


SCALE: 1" = 2,000'



NOTE: Basemap from Microsoft/TomTom 2024.

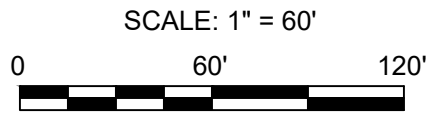
DATE		VICINITY MAP	FIGURE
December 2024		1815 CORDILLERAS ROAD	1
PROJECT NO.		Emerald Hills, Unincorporated San Mateo County, California	
659-5		1600 Willow Pass Court Concord, CA 94520 Tel 925.688.1001 Fax 925.688.1005 www.SFandB.com	



- KEY**
- Exploratory Boring by SFB (12/5/2024)
 - Approximate Project Limit
 - Approximate Geologic Contact
 - Qal/Qfl Alluvial and Fluvial Deposits (Fills May Exist in Local Areas)
 - Qc Colluvium
 - Soil Creep
 - Surface Water Flow Direction
 - Historical Man-Made Access Road
 - Hillside Drainage Swale
- NOTE: All locations shown are approximate.



NOTE: Basemap from the project preliminary site plan prepared by MacLeod and Associates and dated 11/5/2024. Aerial photo imagery from Microsoft 2024.



DATE	December 2024
PROJECT NO.	659-5

Stevens
Serrone &
Bailey
Engineering Company, Inc

1600 Willow Pass Court
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SITE PLAN AND ENGINEERING GEOLOGY MAP

1815 CORDILLERAS ROAD
Emerald Hills, Unincorporated San Mateo County, California

FIGURE

2

659-5, 1815 Cordilleras Road, Emerald Hills - December 2024.dwg
Printed: 12/18/2024

APPENDIX A
Field Exploration

KEY TO FIELD EXPLORATION LOGS

PROJECT:

1815 CORDILLERAS ROAD
Emerald Hills, California

PROJECT NO: **659-5**

FIGURE NO: **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487 & D2488)

MAJOR DIVISIONS		GRAPHIC LOG	GROUP SYMBOL	TYPICAL DESCRIPTION	MAJOR DIVISIONS	GRAPHIC LOG	GROUP SYMBOL	TYPICAL DESCRIPTION		
COARSE-GRAINED SOILS (More than 50% retained on #200 sieve)	CLEAN GRAVELS (Less than 5% fines)		GW	Well-graded gravels, gravel-sand mixtures, trace or no fines	FINE-GRAINED SOILS (More than 50% passes #200 sieve)		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands, clayey silts of low to medium plasticity		
			GP	Poorly-graded gravels, gravel-sand mixtures, trace or no fines			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
	GRAVELS WITH FINES (More than 12% fines)		GM	Silty gravels, gravel-sand-silt mixtures			OL	Organic silts and clays of low plasticity		
			GC	Clayey gravels, gravel-sand-clay mixtures			MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts of high plasticity		
	CLEAN SANDS (Less than 5% fines)		SW	Well-graded sands, gravelly sands, trace or no fines			CH	Inorganic clays of high plasticity, fat clays		
			SP	Poorly-graded sands, gravelly sands, trace or no fines			OH	Organic silts and clays of medium to high plasticity		
	SANDS WITH FINES (More than 12% fines)		SM	Silty sands, sand-silt mixtures			HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils
			SC	Clayey sands, sand-clay mixtures						

GRAIN SIZES

U.S. STANDARD SIEVE SIZE

BOULDERS		COBBLES		GRAVELS		SANDS			SILTS AND CLAYS	
12"		3"		3/4"		#4		#10	#40	#200
				Coarse	Fine	Coarse	Medium	Fine		
304.8 mm		76.2 mm		19.0 mm	4.75 mm	2.00 mm	0.425 mm	0.075 mm		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

SILTS AND CLAYS	BLOWS/FOOT*	UCS (KSF)**
Very Soft	0 - 2	0 - 1/2
Soft	2 - 4	1/2 - 1
Firm	4 - 8	1 - 2
Stiff	8 - 16	2 - 4
Very Stiff	16 - 32	4 - 8
Hard	Over 32	Over 8

*Number of blows for a 140-pound hammer falling 30 inches to drive a 2" O.D. (1-3/8" I.D.) split spoon sampler.

**UCS: Unconfined Compressive Strength.

SYMBOLS AND NOTES

Standard Penetration Test (SPT) Sampler (2" O.D. Split Barrel)

Shelby Tube

Groundwater Level During Drilling

Modified California Sampler (3" O.D. Split Barrel)

Bulk Sample

Groundwater Level at End of Drilling

California Sampler (2.5" O.D. Split Barrel)

Core Barrel (See Log Notes)

INCREASING VISUAL MOISTURE CONTENT

Wet
Moist
Dry

CONSTITUENT PERCENTAGE

trace < 5%
few 5 - 15%
with 16 - 30%
-y 31 - 49%

EXPLORATORY BORING B-1

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 25 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 27 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473364°; -122.263744°)	

PROJECT:
1815 CORDILLERAS ROAD
Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
FILL: SAND (SM), dark brown, fine- to medium-grained, silty, trace gravel (fine, subangular to subrounded), dry.	loose to medium dense		0		8 9	13	13.3	93.7	1.1	At 3.5 Feet: Liquid Limit = 35 Plasticity Index = 18 Fine Gravel = 5% Coarse Sand = 7% Medium Sand = 17% Fine Sand = 25% Fines = 46%
CLAY (CL)/SAND (SC), mottled gray brown, silty, sandy (fine- to coarse-grained), few to with gravel (fine, subangular to subrounded), trace roots, dry to moist.	hard		5		12 14 14	34	11.5			
	very stiff				30 28 20	29				
SAND (SM), brown, fine- to medium-grained, few to with silt, dry to moist.	medium dense		10		12 14 14	17				
Gravelly (fine, subangular to rounded) at 11.5 feet.										
CLAY (CL), mottled gray brown, silty, with to sandy (fine-grained), moist.	very stiff		15		10 13 20	20				
Sandy (fine- to medium-grained) at 16.5 feet.										
CLAY (CL), mottled light gray brown, silty, trace sand (fine-grained), moist.	firm		20		6 6 6	7				
	very stiff		25		7 7 21	28				
GRAVEL (GC), dark gray, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), with clay and silt, wet.	very dense		30							



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EXPLORATORY BORING B-1

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 25 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 27 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473364°; -122.263744°)	

PROJECT:
1815 CORDILLERAS ROAD
 Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
GRAVEL (GC), continued. Drilling refusal at 31 feet.	very dense		31		50/6"	50/6"				
Bottom of Boring = 31 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			35							
			40							
			45							
			50							
			55							
			60							



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EXPLORATORY BORING B-2

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: Not Encountered
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: Not Encountered
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.472824°; -122.263727°)	

PROJECT:
1815 CORDILLERAS ROAD
 Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
CLAY (CL), mottled dark gray brown, silty, with to sandy (fine- to coarse-grained), few gravel (fine, subangular to subrounded), dry to moist.	firm to very stiff		0		18 17 17	20				At 2 Feet: Corrosion Tests.
Moist.					13 14 15	29				
Sandy (fine- to coarse-grained), few to with gravel (fine to coarse, subangular to subrounded).	hard		5		25 23 40	38				
					9 11 15	16	16.4	109.6	2.9	
	very stiff		10		15 14 25	23				
			15		14 20 22	25	19.2	105.3	2.4	
Bottom of Boring = 21.5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			20							
			25							
			30							

EXPLORATORY BORING LOG 659-5 B-2.dwg STEVENS FERRONE & BAILEY 12/16/2024



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EXPLORATORY BORING B-3

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: Not Encountered
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: Not Encountered
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473044°; -122.262816°)	

PROJECT:
1815 CORDILLERAS ROAD
Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
SAND (SM)/SILT (ML), grayish brown, fine- to medium-grained, silty, few gravel (fine, subangular to subrounded), dry.	loose to medium dense		0		22 15 15	18	8.3	93.7	1.2	
SAND (SM), mottled gray brown, fine- to coarse-grained, silty, trace gravel (fine, subangular to subrounded), dry.	medium dense		5		7 9 12	21				
CLAY (CL), mottled gray brown, sandy (fine- to medium-grained), moist.	very stiff		10		19 20 25	27				
With sand (fine- to coarse-grained), trace gravel (fine, subangular to subrounded).			15		14 27 22	29				
SAND (SM), brown, fine- to medium-grained, silty, lightly cemented, trace roots, dry to moist.	medium dense		20		18 17 15	19	8.0	101.7	2.6	
CLAY (CL), brown, sandy (fine- to medium-grained), moist.	stiff		25		10 10 11	13				
Bottom of Boring = 21.5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.										
			30							

EXPLORATORY BORING LOG 659-5 B-3.dwg STEVENS FERRONE & BAILEY 12/16/2024



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 Concord, CA 94520
 Tel: (925) 688-1001

EXPLORATORY BORING B-4

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 20 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 18 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473524°; -122.262882°)	

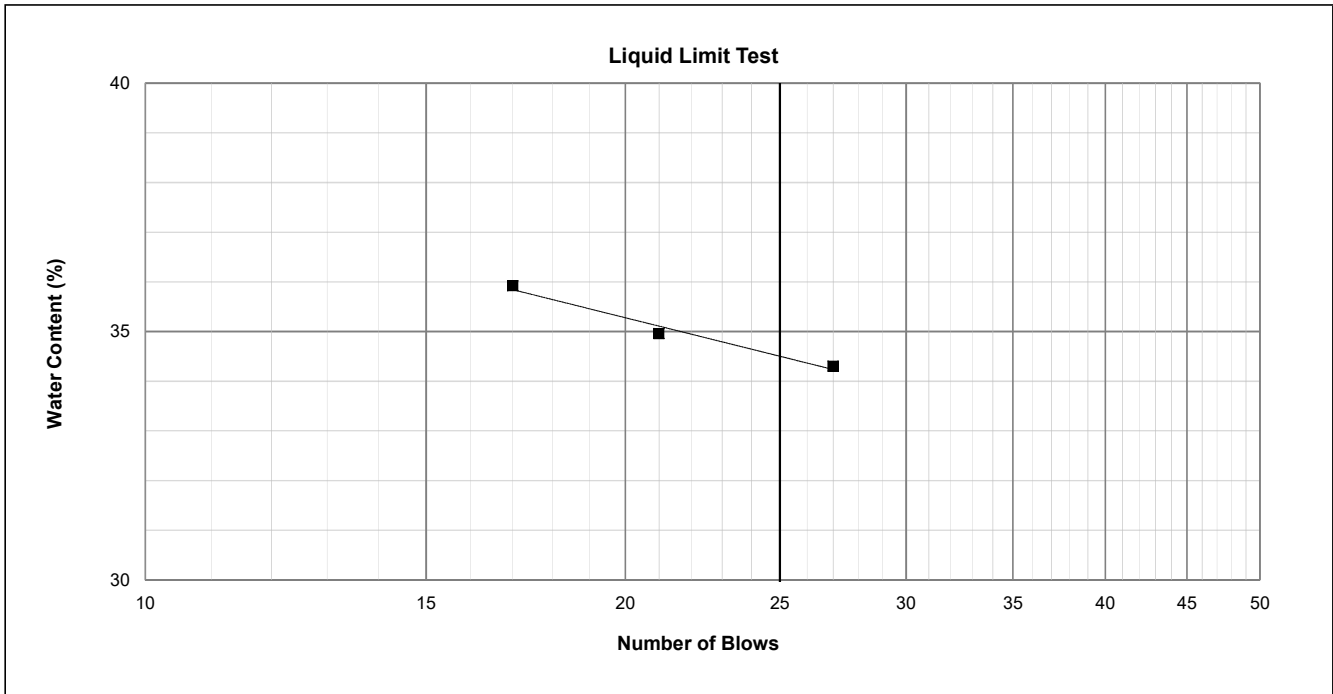
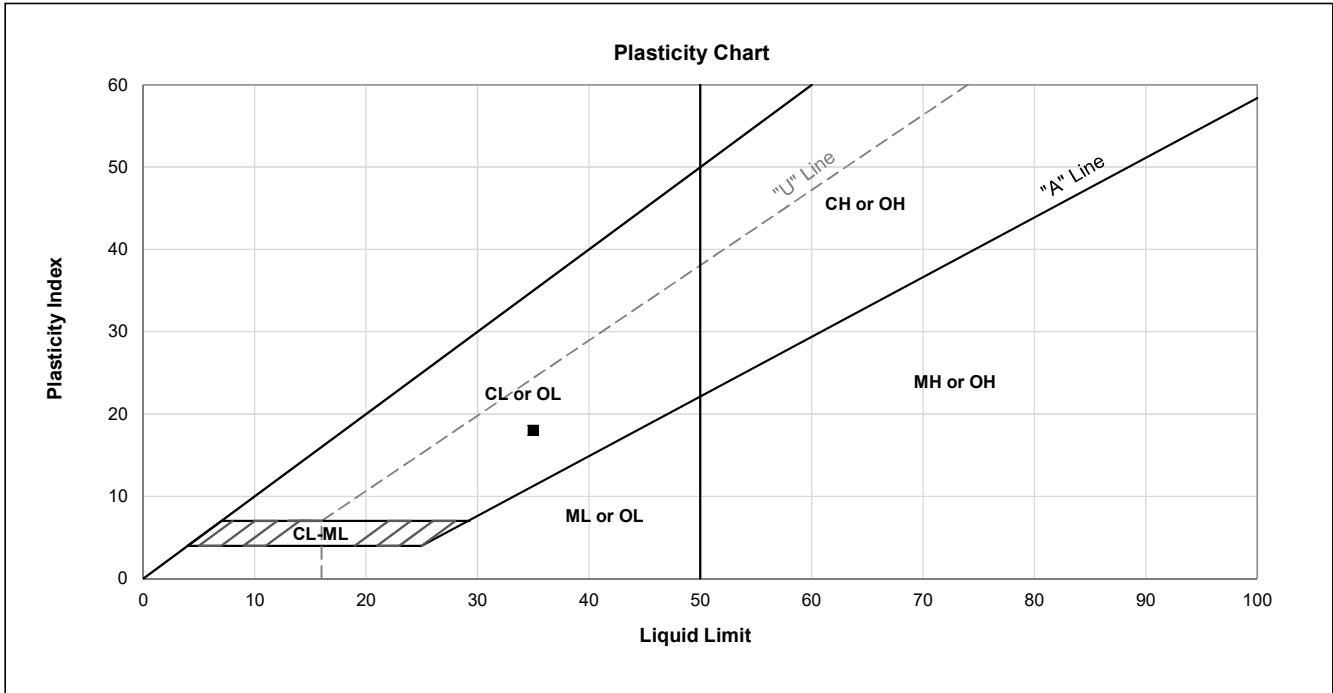
PROJECT:
1815 CORDILLERAS ROAD
 Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS	
DESCRIPTION	CONSIST	GRAPHIC LOG									
CLAY (CL), mottled dark gray brown, silty, with sand (fine- to medium-grained), few gravel (fine, subangular to subrounded), with large tree roots, dry.	firm to very stiff		0		11 18 20	23				At 2 Feet: Corrosion Tests.	
CLAY (CL), mottled gray yellowish brown, silty, with to sandy (fine- to coarse-grained), few gravel (fine, subangular to subrounded), dry.	very stiff				10 12 10	22					
Change color to brown, sandy (fine- to medium-grained), dry to moist.			5		18 17 17	20	12.2	111.5	5.9		
Change color to mottled light gray brown, trace gravel (fine, subangular to subrounded).	stiff		10		12 18 22	12					
SAND (SM), brown, fine- to medium-grained, few coarse-grained, with silt, few gravel (fine, subangular to subrounded), dry to moist.	loose		15		10 7 7	8	7.0				At 15.5 Feet: Fines = 15%
CLAY (CL), mottled gray brown, silty, trace sand (fine-grained), moist.	stiff										
SAND (SM), mottled gray brown, fine- to coarse-grained, silty, trace gravel (fine, subangular to subrounded), trace clay, moist to wet.	loose		20		7 7 7	8					
Bottom of Boring = 21.5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.											
			25								
			30								

EXPLORATORY BORING LOG 659-5 B-4.dwg STEVENS FERRONE & BAILEY 12/16/2024

APPENDIX B
Laboratory Testing

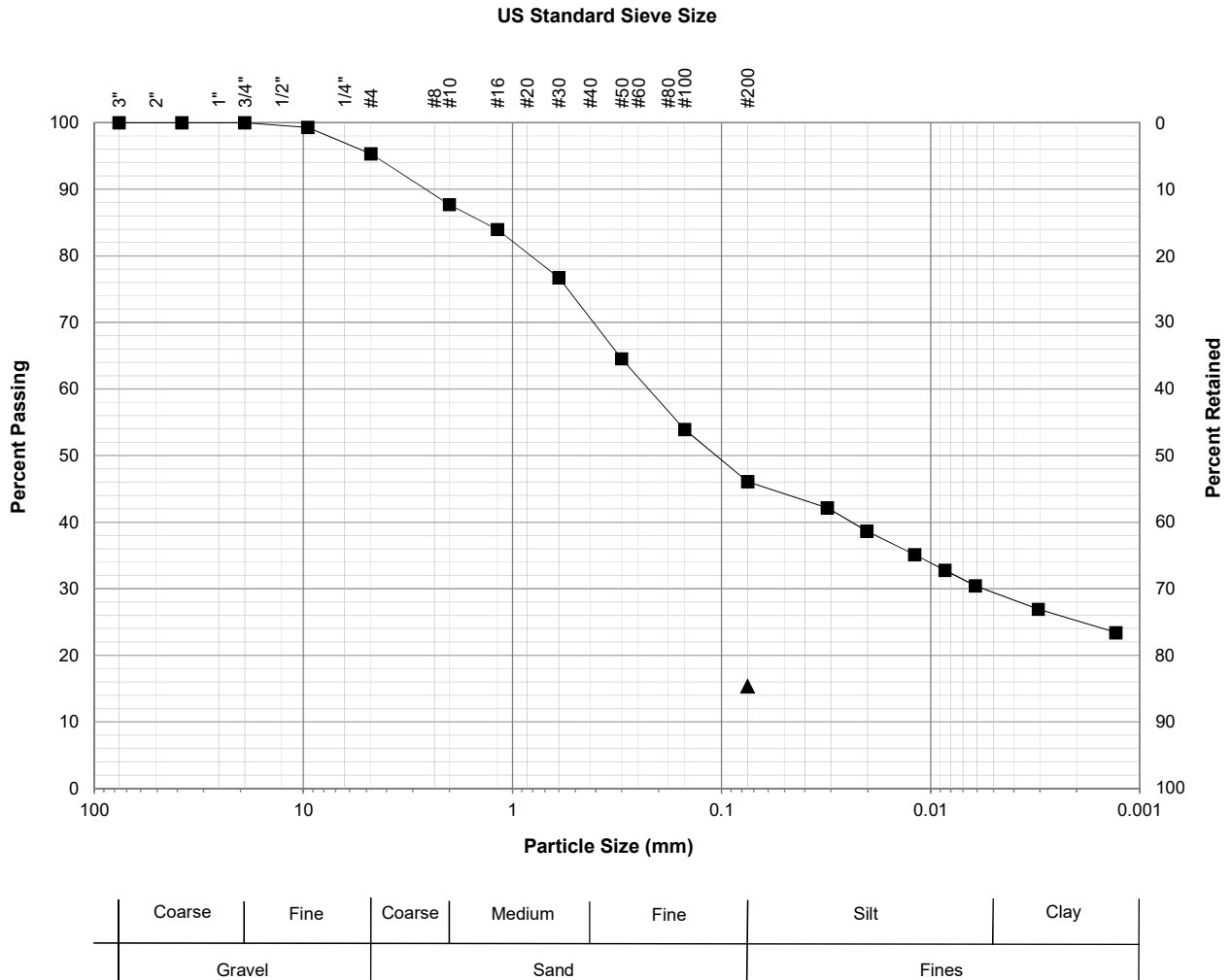
ATTERBERG LIMITS
 ASTM D4318



Test Results	Symbol	Sample Source	Sample Description	LL	PL	PI	% _≤ #200	USCS
	■	Boring B-1 at 3.5 ft.	Brown silty clayey SAND trace gravel	35	17	18	46.1	SC
Project Info	Project Number: 659-5			Test Report Date: 12/11/2024				
	Project Name: 1815 Cordilleras Rd.							
	Project Location: Emerald Hills, CA							
	Tested by: R. Tuazon							
	Checked by: T. Chen							

PARTICLE SIZE DISTRIBUTION

ASTM C136, D422 & D1140

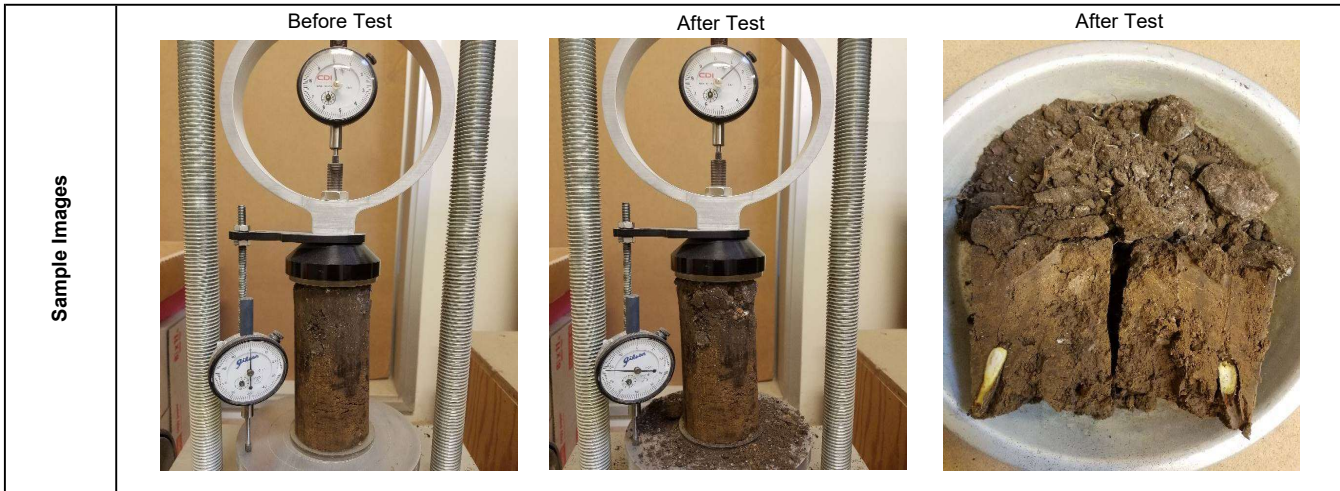


Test Results	Symbol	% >3"	% Gravel		% Sand			% Fines		D ₆₀	D ₃₀	D ₁₀	C _u	C _c
			Coarse	Fine	Coarse	Medium	Fine	Silt	Clay					
	■	0.0	0.0	4.7	7.6	17.0	24.6	46.1	0.223	0.006				
▲							15.4							

Sample Info	Symbol	Sample Source	Sample Description	USCS
	■	Boring B-1 at 3.5 ft.	Brown silty clayey SAND trace gravel	SC
▲	Boring B-4 at 15.5 ft.	Red brown SAND with silt trace gravel	SM	

Project Info	Project Number:	659-5	Test Report Date:	12/12/2024
		Project Name:	1815 Cordilleras Rd.	
	Project Location:	Emerald Hills, CA		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

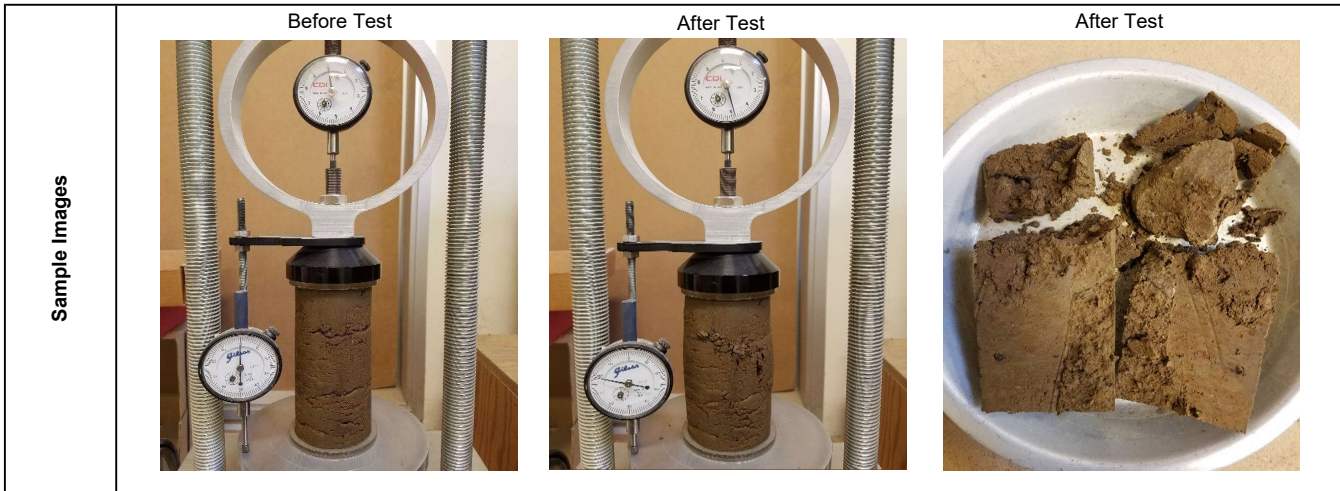
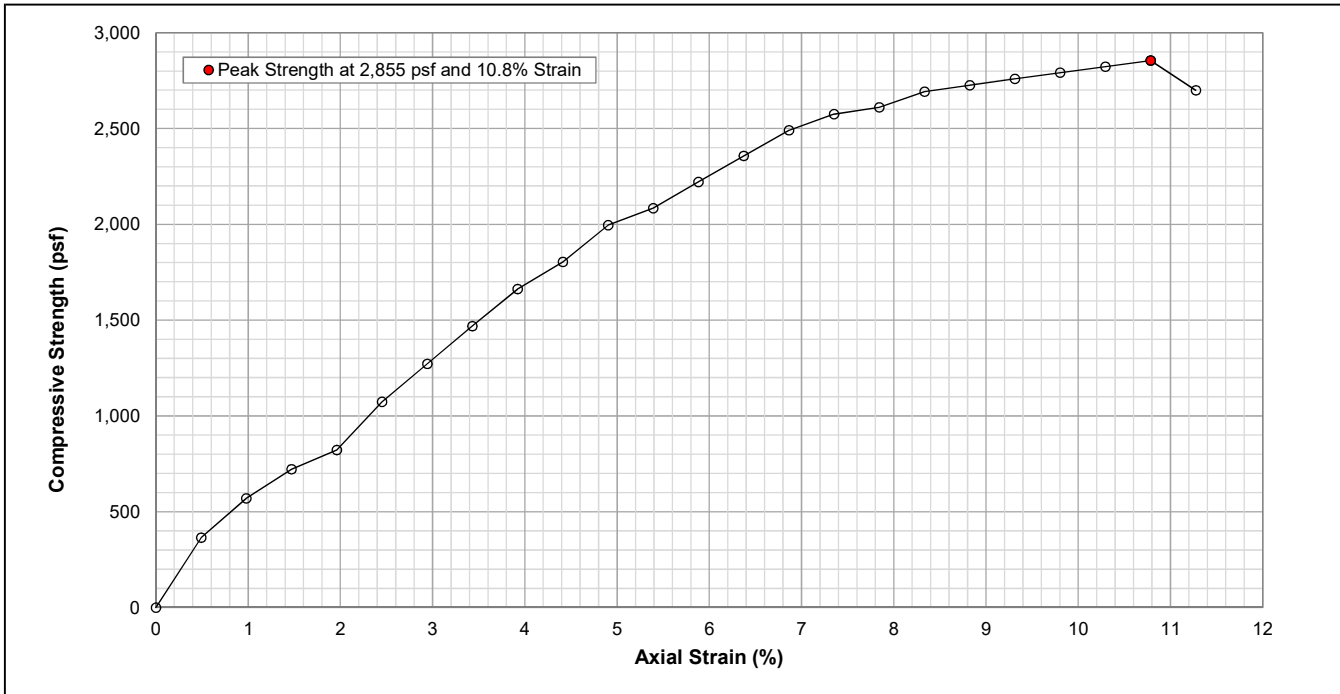
UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Dark brown sandy silty CLAY trace gravel and roots (CL)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	1,122 psf
	Height:	5.6 in		Axial Strain at Failure:	2.7 %
	Height/Diameter:	2.31		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	106.1 pcf		Test Time to Failure:	3 min
	Water Content:	13.3 %		Remarks:	
	Dry Unit Weight:	93.7 pcf			

Project Info	Project Number:	659-5	Test Report Date:	12/16/2024
	Project Name:	1815 Cordilleras Rd.		
	Project Location:	Emerald Hills, CA		
	Sample Source/No.:	Boring B-1		
	Sample Depth:	2 ft.		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

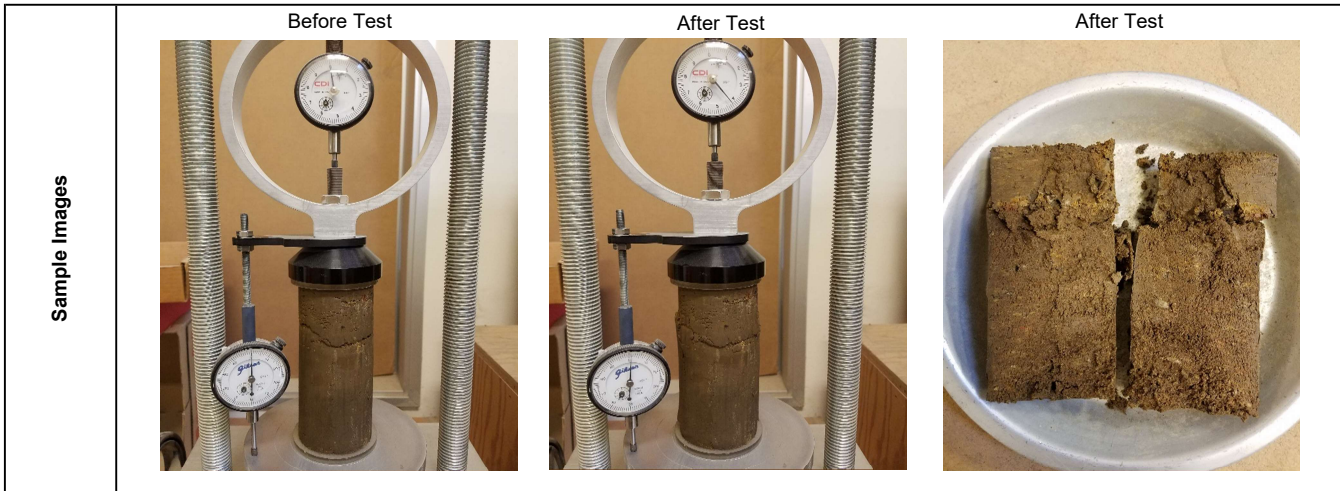
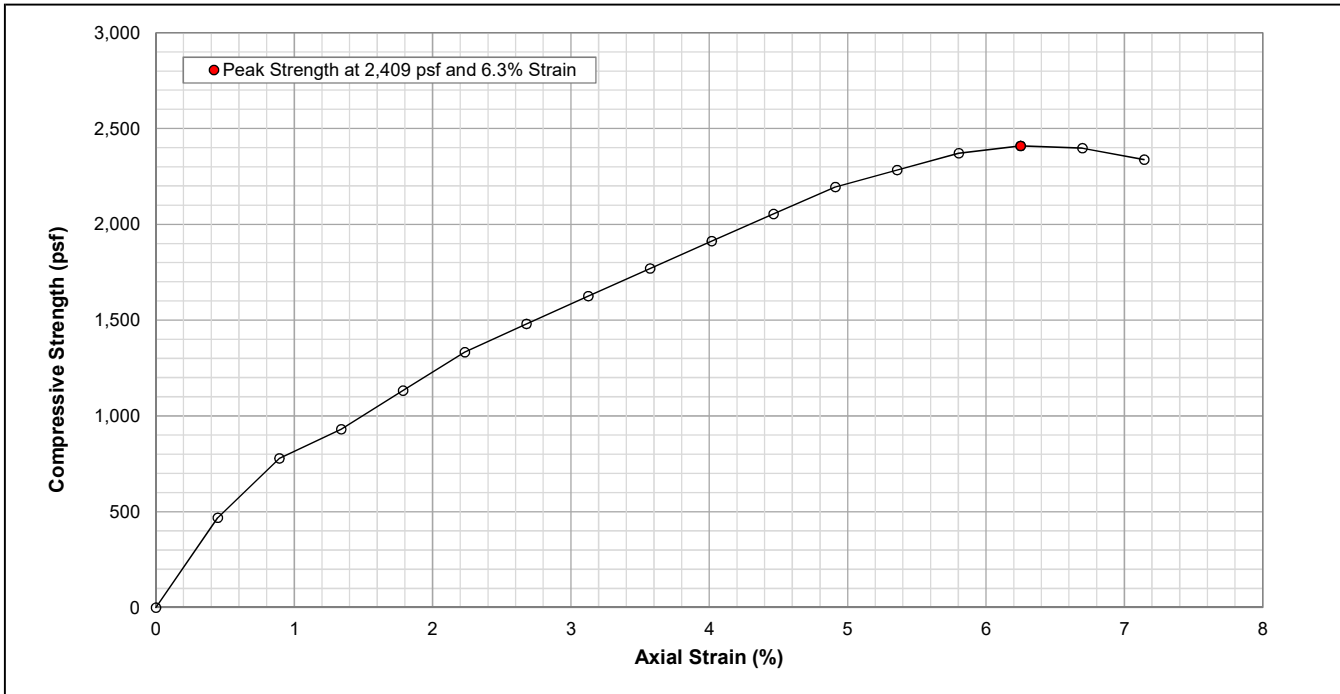
UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Brown silty CLAY with sand some gravel (CL)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	2,855 psf
	Height:	5.1 in		Axial Strain at Failure:	10.8 %
	Height/Diameter:	2.11		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	127.6 pcf		Test Time to Failure:	11 min
	Water Content:	16.4 %		Remarks:	
Dry Unit Weight:	109.6 pcf				

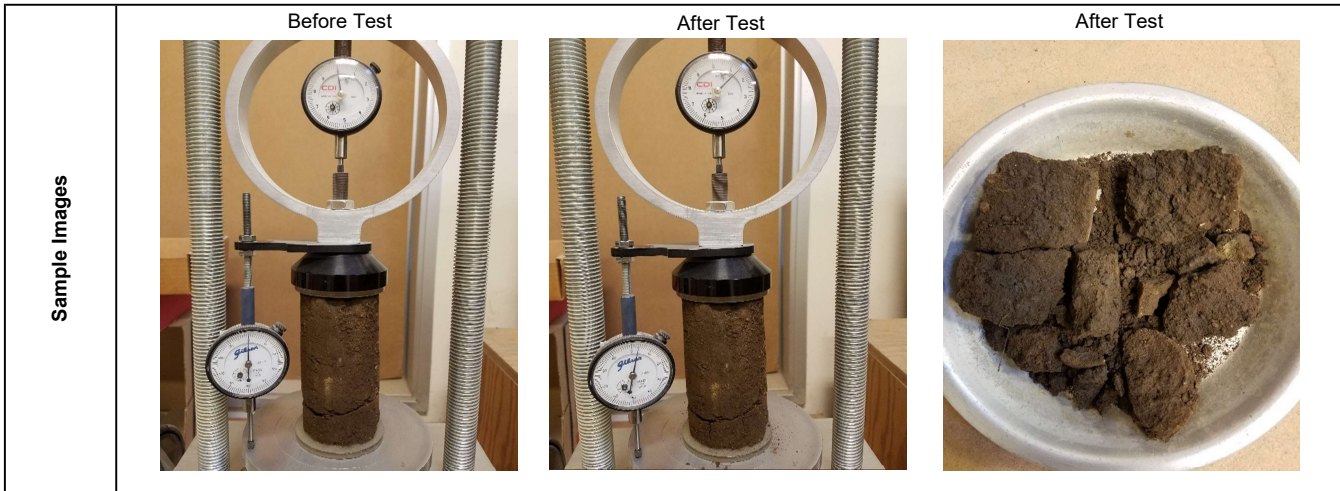
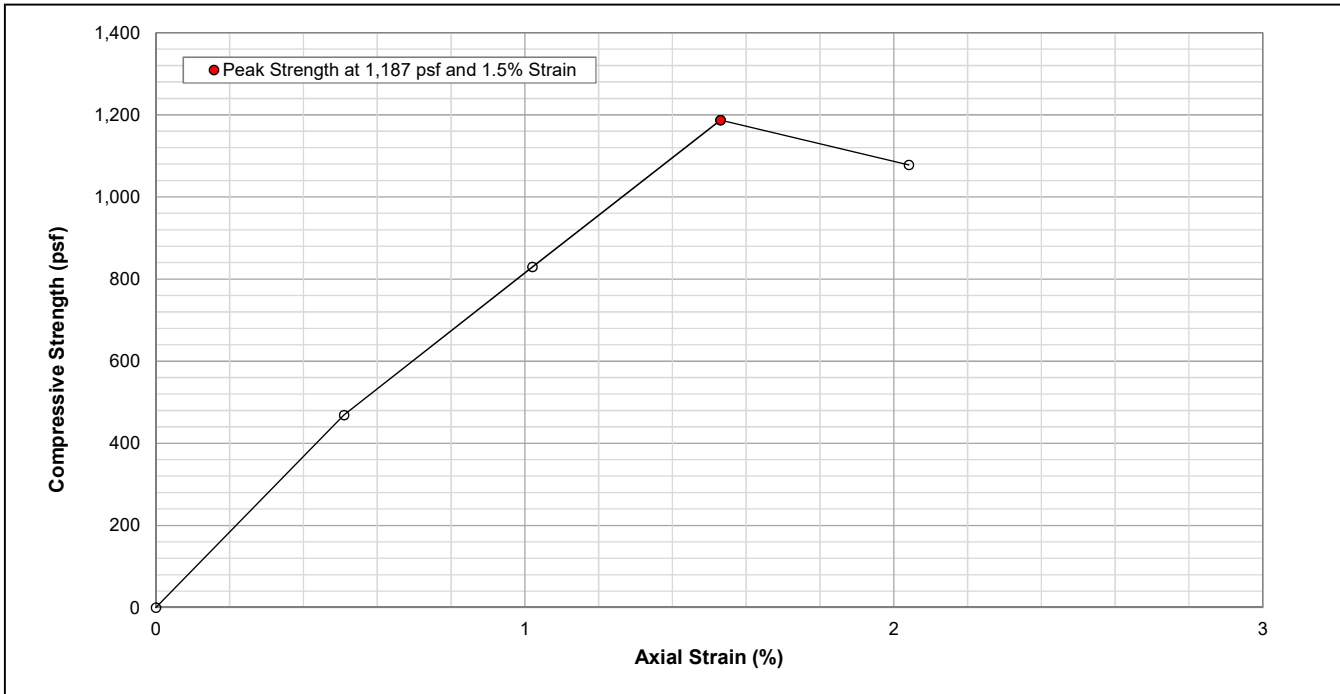
Project Info	Project Number:	659-5	Test Report Date:	12/16/2024
	Project Name:	1815 Cordilleras Rd.		
	Project Location:	Emerald Hills, CA		
	Sample Source/No.:	Boring B-2		
	Sample Depth:	11 ft.		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



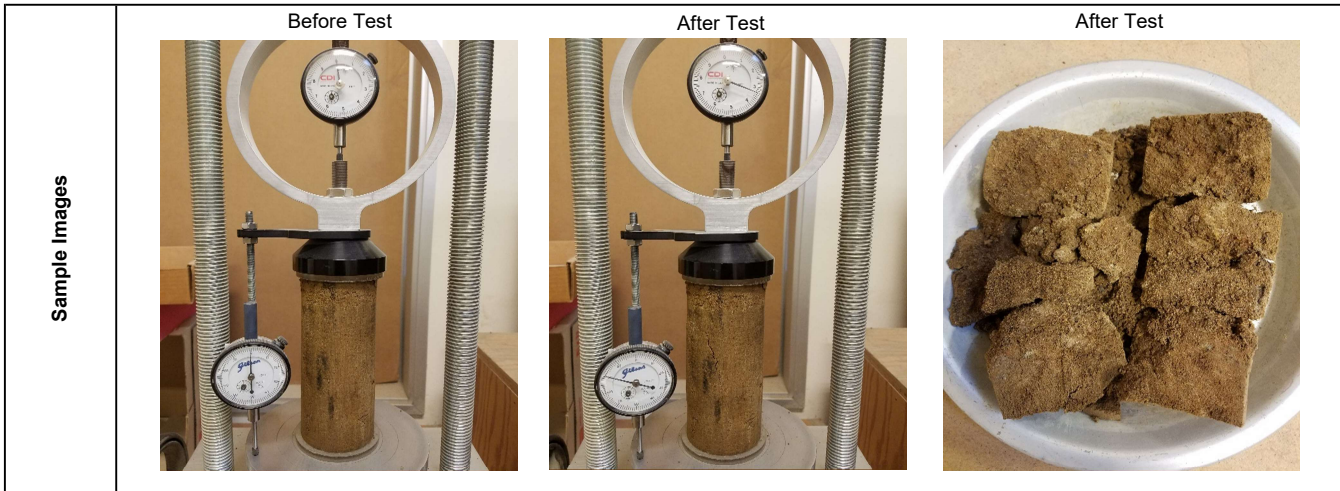
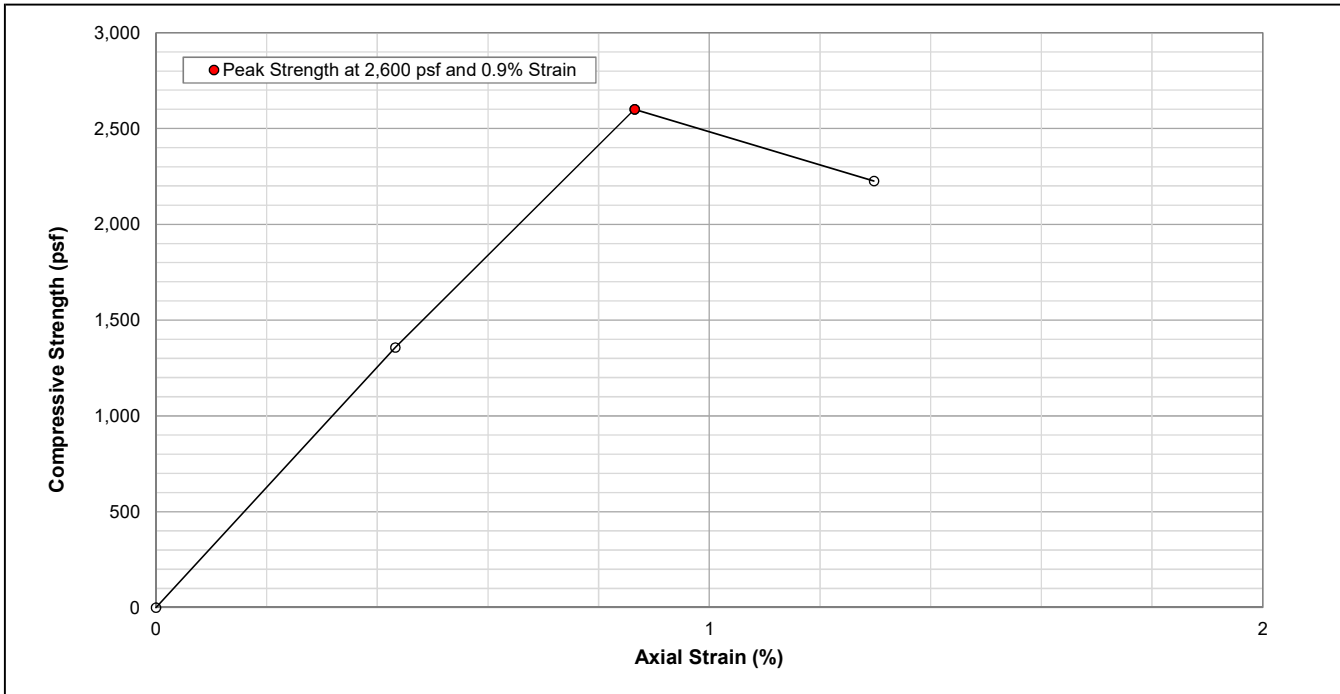
Sample Properties	Sample Description:	Brown sandy silty CLAY trace gravel (CL)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	2,409 psf
	Height:	5.6 in		Axial Strain at Failure:	6.3 %
	Height/Diameter:	2.31		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	125.5 pcf		Test Time to Failure:	7 min
	Water Content:	19.2 %		Remarks:	
	Dry Unit Weight:	105.3 pcf			
Project Info	Project Number:	659-5	Test Report Date: 12/16/2024		
	Project Name:	1815 Cordilleras Rd.			
	Project Location:	Emerald Hills, CA			
	Sample Source/No.:	Boring B-2			
	Sample Depth:	21 ft.			
	Tested by:	R. Tuazon			
Checked by:	T. Chen				

UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Dark brown sandy SILT (ML/SM)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	1,187 psf
	Height:	4.9 in		Axial Strain at Failure:	1.5 %
	Height/Diameter:	2.02		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	101.6 pcf		Test Time to Failure:	1.5 min
	Water Content:	8.3 %		Remarks:	
	Dry Unit Weight:	93.7 pcf			
Project Info	Project Number:	659-5	Test Report Date: 12/16/2024		
	Project Name:	1815 Cordilleras Rd.			
	Project Location:	Emerald Hills, CA			
	Sample Source/No.:	Boring B-3			
	Sample Depth:	2 ft.			
	Tested by:	R. Tuazon			
	Checked by:	T. Chen			

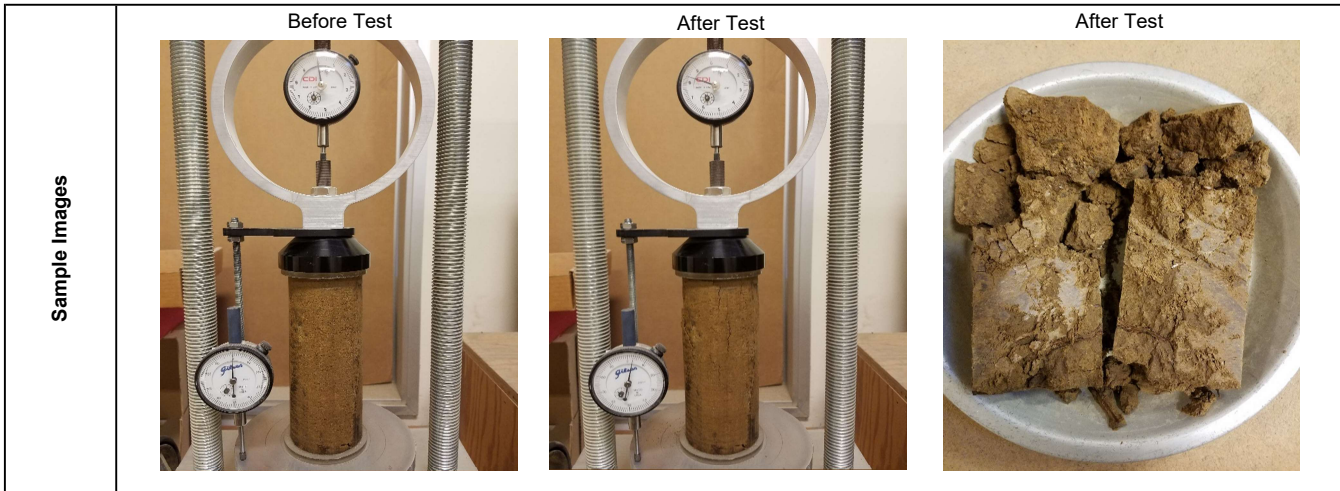
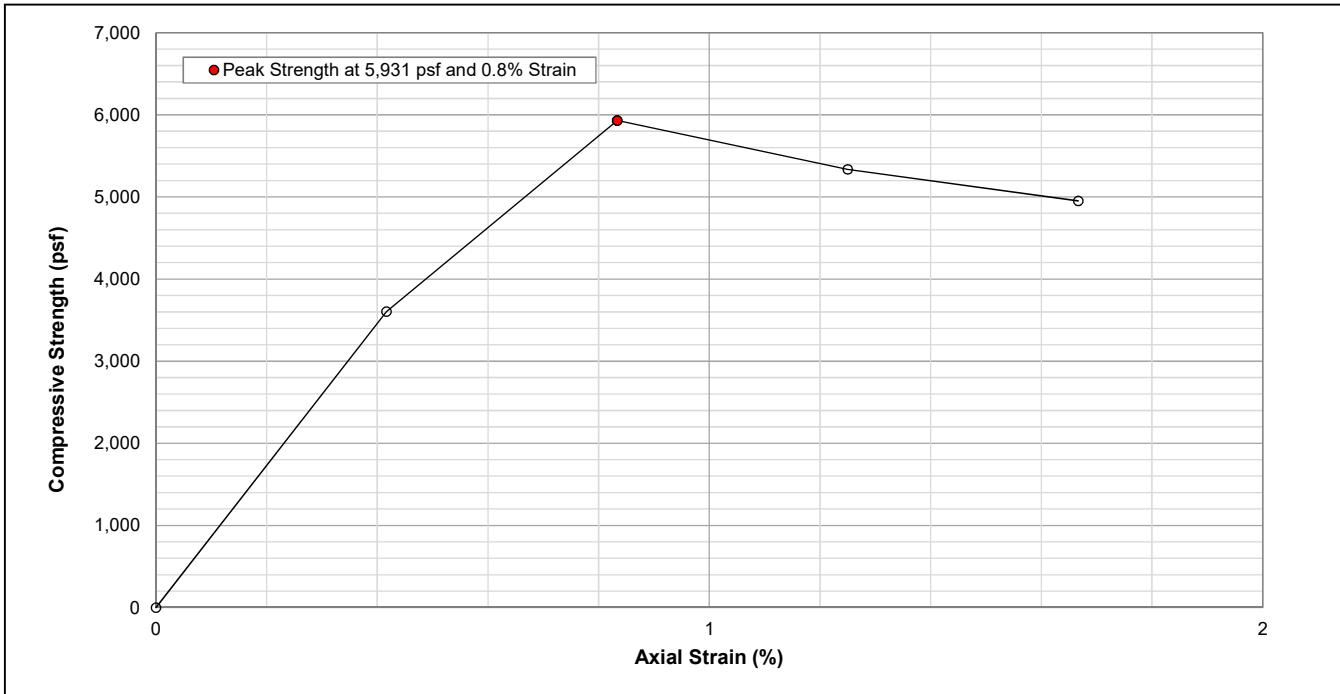
UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Light brown silty SAND (SM)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	2,600 psf
	Height:	5.78 in		Axial Strain at Failure:	0.9 %
	Height/Diameter:	2.39		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	109.8 pcf		Test Time to Failure:	1 min
	Water Content:	8.0 %		Remarks:	
	Dry Unit Weight:	101.7 pcf			

Project Info	Project Number:	659-5	Test Report Date:	12/16/2024
	Project Name:	1815 Cordilleras Rd.		
	Project Location:	Emerald Hills, CA		
	Sample Source/No.:	Boring B-3		
	Sample Depth:	16 ft.		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Red brown silty CLAY with sand (CL)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	5,931 psf
	Height:	6 in		Axial Strain at Failure:	0.8 %
	Height/Diameter:	2.48		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	125.2 pcf		Test Time to Failure:	1 min
	Water Content:	12.2 %		Remarks:	
	Dry Unit Weight:	111.5 pcf			

Project Info	Project Number:	659-5	Test Report Date:	12/16/2024
	Project Name:	1815 Cordilleras Rd.		
	Project Location:	Emerald Hills, CA		
	Sample Source/No.:	Boring B-4		
	Sample Depth:	6 ft.		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		



1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

12 December, 2024

Job No. 2412021
Cust. No. 11486

Mr. Taiming Chen
Stevens, Ferrone & Bailey
1600 Willow Pass Court
Concord, CA 94520

Subject: Project No.: 659-5
Project Name: 1815 Cordilleras Road, Emerald Hills, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Chen:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on December 09, 2024. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as “moderately corrosive”. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfide ion concentrations reflect none detected with a reporting limit of 50 mg/kg.

The pH of the soils are 7.26 and 8.07, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 270-mV and 350-mV, which are indicative of potentially “slightly corrosive” soils.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

A handwritten signature in blue ink that reads 'J. Darby Howard, Jr.'.

J. Darby Howard, Jr., P.E.
President


JDH/jdl
Enclosure

Client: Stevens, Ferrone & Bailey Engineering
 Client's Project No.: 659-5
 Client's Project Name: 1815 Cordilleras Road, Emerald Hills, CA
 Date Sampled: 5-Dec-24
 Date Received: 9-Dec-24
 Matrix: Soil
 Authorization: Signed Chain of Custody

Date of Report: 12-Dec-2024

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2412021-001	B-2 at 2'	350	7.26	-	5,400	N.D.	N.D.	N.D.
2412021-002	B-4 at 2'	270	8.07	-	7,400	N.D.	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	10-Dec-2024	10-Dec-2024	-	11-Dec-2024	11-Dec-2024	11-Dec-2024	11-Dec-2024


 Julia Clauson
 Chemist

* Results Reported on "As Received" Basis
 N.D. - None Detected

APPENDIX C
GBA Guidelines for Geotechnical Report

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer

will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it.* A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the “Findings” Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site’s subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report’s Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals’ misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals’ plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note*

conspicuously that you’ve included the material for information purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled “limitations,” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a “phase-one” or “phase-two” environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer’s services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer’s recommendations will not of itself be sufficient to prevent moisture infiltration.* **Confront the risk of moisture infiltration** by including building-envelope or mold specialists on the design team. **Geotechnical engineers are not building-envelope or mold specialists.**



Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

February 5, 2026

Mr. Eric Zweig
Edenbridge Homes
P.O. Box 1825
Los Altos, CA 94023

Re: Supplemental Geotechnical Investigation - PLN2025-00350/GP2025-023-02
1815 Cordilleras Road – Emerald Hills, California
SFB Project No.: 659-5

Mr. Zweig:

In accordance with your request to respond to the geotechnical review comments provided by San Mateo County Planning and Building in their letters dated November 21 and December 11, 2025, Stevens, Ferrone & Bailey Engineering Company, Inc. (SFB) performed a supplemental geotechnical investigation for the proposed new residential development to be located at 1815 Cordilleras Road in the unincorporated Emerald Hills area of San Mateo County, California.

Previously, SFB prepared a geotechnical investigation report¹ dated December 18, 2024 for the site. In addition, SFB also prepared a letter² dated December 4, 2025 in response to geotechnical review comments provided by Mr. Quinn Todzo, PE (the County Reviewer) of San Mateo County Planning and Building³.

The purposes of our supplemental investigation were to collect additional site subsurface soil and rock data, and perform supplemental geotechnical engineering analyses to further evaluate the soil liquefaction hazards potential of the site in accordance with the County review comments⁴. In addition, additional field reconnaissance and mapping were also performed on the open space

¹SFB, 2024, Geotechnical Investigation, 1815 Cordilleras Road, Emerald Hills, California, SFB Project No. 659-5, December 18.

²SFB, 2025, Responses to County Geotechnical Review Comments - PLN2025-00350/GP2025-023-02, SFB Project No. 659-5, December 4.

³San Mateo County Planning and Building, 2025, Geotechnical review of PLN Project, PLN2025-00350/GP2025-023-02, 1815 Cordilleras Road, Redwood City, APN 057070090, November 21.

⁴San Mateo County Planning and Building, 2025, Geotechnical supplementary review of Liquefaction Study, PLN2025-00350/GP2025-023-02, 1815 Cordilleras Road, Redwood City, APN 057070090, December 11.

hillside slopes located to the south and southwest of the planned development area (above the proposed Lot 3) to further evaluate slope stability and drainage conditions.

1.0 Supplemental Field Exploration

Our supplemental geotechnical field exploration program consisted of performing two additional exploratory borings (B-5 and B-6) within the site to a depth of about 40 feet (where drilling refusal was encountered in sandstone bedrock) on January 21, 2026. The approximate locations of the borings are shown on the **Site Plan and Engineering Geology Map, Figure 1**. The borings were performed by Terra-Ex LLC of Dixon, California by using a truck-mounted Diedrich D-70 drill rig equipped with rotary wash tooling and a 140-pound automatic trip hammer.

Our field engineer continuously logged the soils and rock encountered in the borings. The soils are classified in general accordance with the Unified Soil Classification System (ASTM D2487 and D2488). Logs of the explorations as well as keys for the classification of the soil (**Figure A-1**) and rock characteristics (**Figure A-2**) are included in **Appendix A**. Upon completion of our field explorations, the borings were backfilled with cement grout in accordance with San Mateo County Environmental Health requirements.

The approximate locations of our borings were determined by pacing, tape measurements, and/or alignment from landmark references. Latitude and longitude of exploration locations shown on the exploration logs are estimated from online map data from Microsoft; actual locations were not surveyed.

Representative samples were obtained from our borings at selected depths appropriate to the investigation. Relatively undisturbed samples were obtained using a 3-inch O.D. Modified California split barrel sampler with liners, and disturbed samples were obtained using a 2-inch O.D. Standard Penetration Test (SPT) split spoon sampler without liners. All samples were transported to our geotechnical laboratory for evaluation and appropriate testing. Sampler types are indicated in the "Sampler" column of the exploration logs as designated in **Figure A-1**.

Resistance blow counts were obtained in our borings with the samplers by dropping a 140-pound automatic trip hammer through a 30-inch fall. The sampler was driven 18 inches and the number of blows were recorded as field blow counts for each 6 inches of penetration. The SPT N-value shown on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. A sampler barrel size correction factor of 0.6 was applied to the blow counts from the Modified California sampler. In addition, the recorded blow counts (N_{60}) shown on our boring logs had been corrected to a standard 140-pound hammer that delivers 60% of energy. An energy ratio correction factor of 1.3 was used for the automatic trip hammer, assuming the hammer delivers about 78%

of energy. The recorded blow counts have not been corrected for other factors, such as borehole diameter, rod length, overburden pressure, and fines content.

Previously, four exploratory borings (B-1 to B-4) were performed at the site by SFB on December 5, 2024 to a maximum depth of about 31 feet. The locations of these previous explorations are also shown on **Figure 1**. Logs of SFB's previous explorations and laboratory testing results are provided in **Appendix C** for reference.

It should be noted that changes in the surface and subsurface conditions can occur over time as a result of either natural processes or human activity and may affect the validity of the conclusions and recommendations in this report. In addition, our attached exploration logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

2.0 Supplemental Laboratory Testing

Our supplemental laboratory testing program for the project was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site. This program included the following testing:

- Three Atterberg Limits determinations (plastic and liquid limits) per ASTM D4318; and
- Eleven sieve and hydrometer tests per ASTM D422 or D1140.

All tests were performed by our geotechnical laboratory in Concord, California. The results of the testing are included on the exploration logs and plotted laboratory results are also included in **Appendix B**.

3.0 Site Subsurface Conditions

Both of our supplemental borings (B-5 and B-6) encountered subsurface soil conditions consistent with those encountered in our previous borings (B-1 to B-4). The site (below pavement sections and/or surficial fills if any) is generally underlain by firm to hard clays with interbeds of loose to medium dense, thin sand and gravel lenses that extend to depths of about 26 to 28-1/2 feet. Below the predominately clay layers, very dense clayey gravels were encountered that extend to depths of about 33 to 37 feet. Underlying the gravel layer, friable to weak, moderately weathered, moderately hard sandstone bedrock was encountered that extended to the maximum depth explored of about 40 feet. Drilling refusal in bedrock was encountered by the Diedrich D-70 drill rig at the bottoms of Borings B-5 and B-6. Detailed descriptions of soils and rock encountered in our exploratory borings are presented on the exploration logs in **Appendix A**. Results of laboratory

testing of retrieved onsite soils are included in **Appendix B**. Logs of SFB's previous explorations and laboratory testing results are also provided in **Appendix C** for reference.

Due to the use of rotary wash drilling fluids for Borings B-5 and B-6, groundwater levels were not established in both borings. By inspection of the received soil samples, we estimated groundwater level was at about 15 feet deep within these two borings during drilling. Groundwater levels were previously measured in Borings B-1 and B-4, respectively, at depths of about 27 and 18 feet at the end of drilling. No groundwater was encountered in the other two borings (B-2 and B-3) to the maximum depths explored of these borings at about 21-1/2 feet. It should be noted that our borings might not have been left open for a sufficient period of time to establish equilibrium groundwater conditions.

Fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, subsurface seepage, water pumping in nearby wells, water recharging from the nearby creek, and other factors. It is also likely that during rainfall events, localized high groundwater level or seepage may develop on the adjacent hillside slopes and seep toward the site.

4.0 Supplemental Liquefaction Hazard Evaluation

We performed SPT-based seismic soil liquefaction triggering and post-liquefaction deformation analyses in general accordance with the guidelines in CGS Special Publication 117A (2008)⁵ and the recommended procedures by Southern California Earthquake Center (SCEC, 1999)⁶. The analyses were performed by using methodologies developed by Idriss and Boulanger (2008 and 2014)^{7,8}. Subsurface soil data from our borings with three probable scenarios of groundwater depths (the groundwater level encountered during our field exploration, an assumed groundwater level at 10 feet deep, and an assumed groundwater level at 0 feet deep) were used in our liquefaction analyses. Since the upper three feet of onsite surficial fills and soil will be over-excavated and re-compacted during site mass grading and building pad preparation, the existing sandy fills and soils within three feet deep (if any) were not included in our analyses.

A peak ground acceleration (PGA_M) of 0.92g from a Maximum Considered Earthquake (MCE) per the 2025 California Building Code/ASCE 7-22 (based on Site Class D - Default) and an earthquake magnitude (M_w) of 7.9 (based on the USGS NSHM Conterminous U.S. 2018 model

⁵California Geological Survey, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, CGS Special Publication 117A.

⁶Southern California Earthquake Center, 2002, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California, June.

⁷Idriss and Boulanger, 2008, Soil Liquefaction During Earthquakes, EERI Monograph MNO-12.

⁸Boulanger and Idriss, 2014, CPT and SPT Based Liquefaction Triggering Procedures, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA, Report No. UCD/CGM-14/01, April.

disaggregation with a return period of 2,475 years) were used in our analyses. Factor of safety against liquefaction triggering is defined as Cyclic Resistance Ratio (CRR) divided by the Cyclic Stress Ratio (CSR). Saturated soils are considered to be potentially liquefiable if the calculated factor of safety against liquefaction triggering is less than about 1.3 (per CGS Special Publication 117A). Post-liquefaction residual volumetric strains and deformations were estimated for the potentially liquefiable soils based on the methodologies used. No weighting factors were applied to the post-liquefaction volumetric strain.

The results of our analyses indicate that the loose to medium dense sand lenses encountered in our borings below the site have a moderate to high potential for liquefying where they are subjected to an MCE earthquake event. These sand lenses appear to be in the form of randomly distributed pockets of 2 to 4 feet in thickness. We estimate the MCE earthquake induced liquefaction in these sand lenses could result in residual volumetric strains varying from about 1/2 to 3-1/2 percent and may cause total ground surface settlements up to about 2 inches at the site.

The table below summarizes the calculated liquefaction-induced settlement for each boring location. Our detailed soil liquefaction analysis results are included in **Appendix D**.

SPT-BASED LIQUEFACTION ANALYSIS RESULTS					
Location	Depth of Boring (ft)	Combined Potentially Liquefiable Soil Thickness (ft)	Calculated Cumulative Ground Settlement (inch)		
			Groundwater at Time of Drilling	Groundwater at 10 Feet Deep	Groundwater at 0 Feet Deep
B-1	31.0	4.0	0	0.8	1.1
B-2	21.5	Not Encountered	0	0	0
B-3	21.5	5.5	0	0.9	1.0
B-4	21.5	5.0	1.1	2.0	2.0
B-5	40.1 (Refusal in Bedrock)	Not Encountered	0	0	0

SPT-BASED LIQUEFACTION ANALYSIS RESULTS					
Location	Depth of Boring (ft)	Combined Potentially Liquefiable Soil Thickness (ft)	Calculated Cumulative Ground Settlement (inch)		
			Groundwater at Time of Drilling	Groundwater at 10 Feet Deep	Groundwater at 0 Feet Deep
B-6	40.2 (Refusal in Bedrock)	3.5	0.7	1.0	1.0

The results of our supplemental field exploration, laboratory testing, and seismic soil liquefaction triggering and post-liquefaction deformation analyses fully support our opinions, conclusions, and recommendations included in our previous geotechnical investigation report (2024) and the subsequent response letter (2025) to the San Mateo County Planning and Building. Please refer to our previous report and letter for more details.

5.0 Supplemental Slope Stability and Drainage Evaluation

The field reconnaissance and mapping of the site and the neighboring hillside open space slopes were performed on December 17, 2024 and January 27, 2026 by SFB’s Certified Engineering Geologist and Geotechnical Engineers. We did not observe evidence of landsliding and adverse drainage conditions within the site, except for local areas along the southern site boundary where surface water had ponded in the past. The hillside slopes to the south of the site generally direct surface water toward the site. However, a historical man-made access road of about 10 feet wide within the neighboring properties (approximately shown on **Figure 1**) intercepts and collects most of the surface water from the hillside open space slopes, and directs the water toward the southeastern corner of the site. Based on the results of geologic literature review, field reconnaissance and mapping, and subsurface explorations within the site, it is our opinion that the proposed lateral setback of 10 feet from the base of the existing stone retaining wall is acceptable for the potential new house location at Lot 3 (at the southwestern portion of the site). We recommend the onsite slopes be regularly and properly maintained, and the recommendations contained in our geotechnical investigation report (2024) be implemented in the design and construction of the project including the recommendations contained in **Sections 4.1.10 and 4.1.11** of our 2024 report.

Opinions, conclusions and recommendations provided in this letter are meant to supplement our previous report and letter; all previous conditions and limitations apply. If you have any questions or need additional information, please contact us.

Sincerely,
Stevens, Ferrone & Bailey Engineering Company, Inc.



Taiming Chen, PE, GE
Civil/Geotechnical Engineer

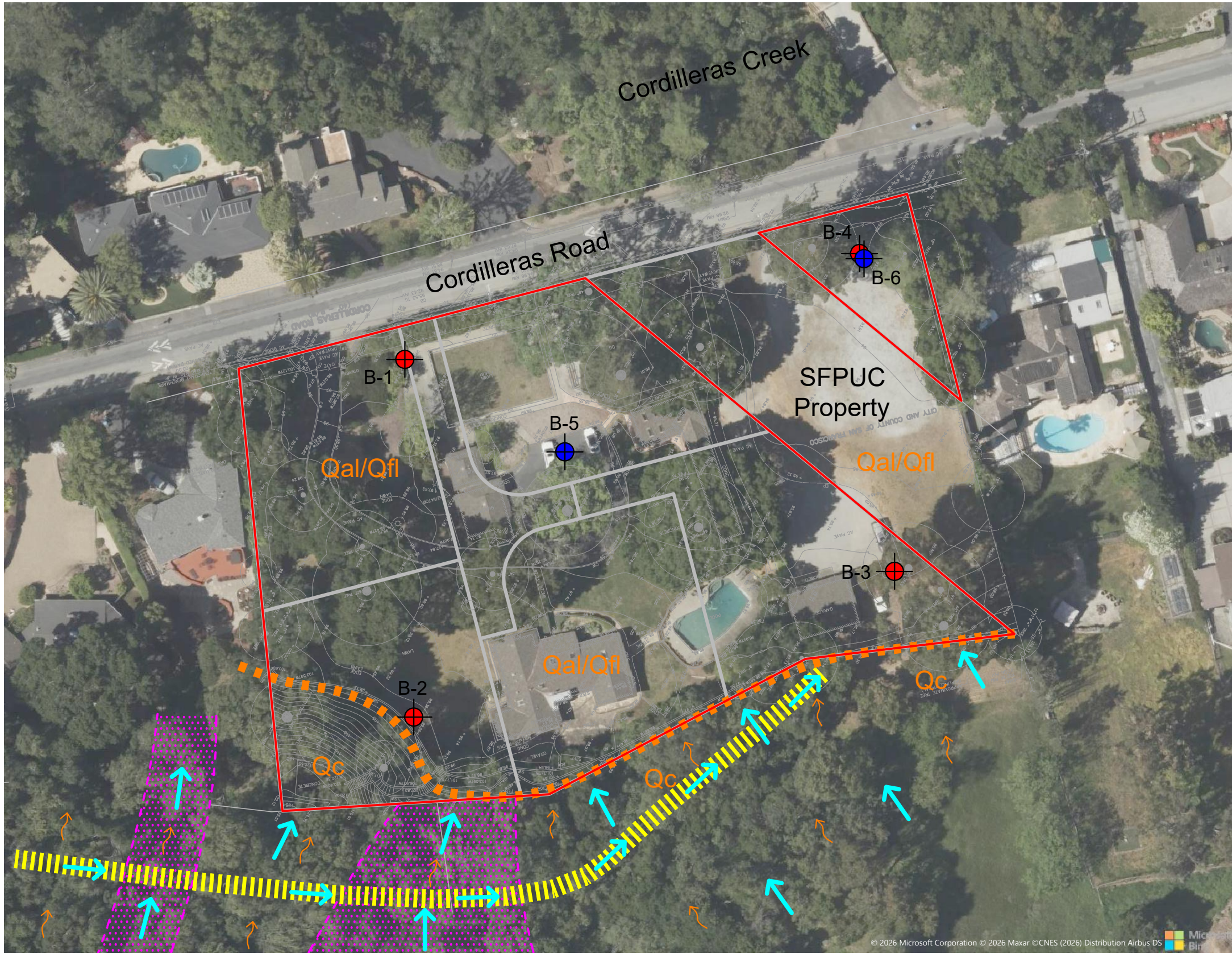


Kenneth C. Ferrone, P.E., G.E., C.E.G.
Civil/Geotechnical Engineer
Certified Engineering Geologist



- Copies: Addressee (via email)
- Attachments: Figure 1 – Site Plan and Engineering Geology Map
 - Appendix A – Supplemental Field Exploration
 - Appendix B – Supplemental Laboratory Testing
 - Appendix C – Previous Field Exploration and Laboratory Testing
 - Appendix D – Soil Liquefaction Analyses

FIGURE



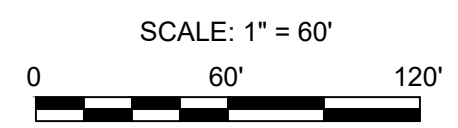
KEY

- Exploratory Boring by SFB (1/21/2026)
- Exploratory Boring by SFB (12/5/2024)
- Approximate Project Limit
- Approximate Geologic Contact
- Qal/Qfl Alluvial and Fluvial Deposits (Fills May Exist in Local Areas)
- Qc Colluvium
- Soil Creep
- Surface Water Flow Direction
- Historical Man-Made Access Road
- Hillside Drainage Swale

NOTE: All locations shown are approximate.



NOTE: Basemap from the project preliminary site plan prepared by MacLeod and Associates and dated 11/5/2024. Aerial photo imagery from Microsoft 2026.



DATE	February 2026
PROJECT NO.	659-5

Stevens
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SITE PLAN AND ENGINEERING GEOLOGY MAP	FIGURE
1815 CORDILLERAS ROAD Emerald Hills, Unincorporated San Mateo County, California	1

659-5, 1815 Cordilleras Road, Emerald Hills - February 2026.dwg
Printed: 2/5/2026

APPENDIX A
Supplemental Field Exploration

KEY TO FIELD EXPLORATION LOGS

PROJECT:

1815 CORDILLERAS ROAD
Emerald Hills, California

PROJECT NO: **659-5**

FIGURE NO: **A-1**

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487 & D2488)

MAJOR DIVISIONS		GRAPHIC LOG	GROUP SYMBOL	TYPICAL DESCRIPTION	MAJOR DIVISIONS	GRAPHIC LOG	GROUP SYMBOL	TYPICAL DESCRIPTION		
COARSE-GRAINED SOILS (More than 50% retained on #200 sieve)	CLEAN GRAVELS (Less than 5% fines)		GW	Well-graded gravels, gravel-sand mixtures, trace or no fines	FINE-GRAINED SOILS (More than 50% passes #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands, clayey silts of low to medium plasticity		
			GP	Poorly-graded gravels, gravel-sand mixtures, trace or no fines			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
	GRAVELS WITH FINES (More than 12% fines)	GM	Silty gravels, gravel-sand-silt mixtures	OL			Organic silts and clays of low plasticity			
		GC	Clayey gravels, gravel-sand-clay mixtures	SILTS AND CLAYS (Liquid Limit 50 or more)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts of high plasticity			
	CLEAN SANDS (Less than 5% fines)	SW	Well-graded sands, gravelly sands, trace or no fines			CH	Inorganic clays of high plasticity, fat clays			
		SP	Poorly-graded sands, gravelly sands, trace or no fines			OH	Organic silts and clays of medium to high plasticity			
	SANDS WITH FINES (More than 12% fines)		SM			Silty sands, sand-silt mixtures	HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils
			SC			Clayey sands, sand-clay mixtures				

GRAIN SIZES

U.S. STANDARD SIEVE SIZE

12"	3"	3/4"	#4	#10	#40	#200	
BOULDERS	COBBLES	GRAVELS		SANDS			SILTS AND CLAYS
		Coarse	Fine	Coarse	Medium	Fine	
304.8 mm	76.2 mm	19.0 mm	4.75 mm	2.00 mm	0.425 mm	0.075 mm	

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

CONSISTENCY

SILTS AND CLAYS	BLOWS/FOOT*	UCS (KSF)**
Very Soft	0 - 2	0 - 1/2
Soft	2 - 4	1/2 - 1
Firm	4 - 8	1 - 2
Stiff	8 - 16	2 - 4
Very Stiff	16 - 32	4 - 8
Hard	Over 32	Over 8

*Number of blows for a 140-pound hammer falling 30 inches to drive a 2" O.D. (1-3/8" I.D.) split spoon sampler.

**UCS: Unconfined Compressive Strength.

SYMBOLS AND NOTES

Standard Penetration Test (SPT) Sampler (2" O.D. Split Barrel)

Shelby Tube

Groundwater Level During Drilling

Modified California Sampler (3" O.D. Split Barrel)

Bulk Sample

Groundwater Level at End of Drilling

California Sampler (2.5" O.D. Split Barrel)

Core Barrel (See Log Notes)

INCREASING VISUAL MOISTURE CONTENT

Wet
Moist
Dry

CONSTITUENT PERCENTAGE

trace < 5%
few 5 - 15%
with 16 - 30%
-y 31 - 49%

KEY TO ROCK CHARACTERISTICS

PROJECT:

1815 CORDILLERAS ROAD
Emerald Hills, California

PROJECT NO: **659-5**

FIGURE NO: **A-2**

STRENGTH/CONSISTENCY

PLASTIC - Very low strength; soil-like strength.

FRIABLE - Easily broken down with hand or finger pressure.

WEAK - Crumbles under light hammer blows.

MODERATELY STRONG - Withstands a few heavy hammer blows before breaking.

STRONG - Withstands a few heavy hammer blows while only yielding large fragments.

VERY STRONG - Resists breakage from heavy hammer blows while only yielding small chips and dust.

WEATHERING

COMPLETE - Rock reduced to soil-like structure; rock fabric not apparent or only in localized areas.

DEEP - Rock fabric apparent but extensive disintegration; moderate to complete mineral decomposition; fractures coated and/or filled with oxides, carbonates, and/or soil.

MODERATE - Rock fabric apparent; partial disintegration of minerals; all rock discolored or stained; moderately coated fractures.

SLIGHT - Rock generally fresh but fractures stained and may be slightly coated.

FRESH - No disintegration or discoloration; joints may show slight staining.

HARDNESS

SOFT - Easily scratched with fingernail; soil-like consistency.

LOW - Easily gouged or carved with knife blade.

MODERATE - Readily scratched with knife; scratch is readily visible and leaves heavy trace of dust.

HARD - Scratched with knife with difficulty; scatch is faintly visible and leaves little dust.

VERY HARD - Cannot be scratched with knife and leaves metallic streak.

FRACTURE SPACING

CRUSHED - Less than 1/4 inch

INTENSELY - 1/4 inch to 1 inch

CLOSELY - 1 inch to 6 inches

MODERATELY - 6 inches to 1 foot

OCCASIONALLY - 1 foot to 4 feet

LITTLE - Greater than 4 feet

BEDDING SPACING

THINLY LAMINATED - Less than 1/10 inch

LAMINATED - 1/10 inch to 1/2 inch

VERY THIN - 1/2 inch to 1 inch

THIN - 1 inch to 6 inches

THICK - 6 inches to 2 feet

VERY THICK - Greater than 2 feet

MASSIVE - No apparent bedding spacing

ROUGHNESS OF SURFACE

SMOOTH - Appears smooth and surface feels smooth to the touch; may be slickensided.

SLIGHTLY ROUGH - Asperities on surface are clearly visible.

MEDIUM ROUGH - Asperities are clearly visible and surface feels abrasive.

ROUGH - Large angular asperities are visible; some ridge and high side angle steps are evident.

VERY ROUGH - Near vertical steps and ridges are visible on surface.



1600 Willow Pass Court
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EXPLORATORY BORING B-5

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 1/21/2026
DRILLER: Terra-Ex	DATE FINISHED: 1/21/2026
DRILL RIG: Diedrich D70 Turbo	DEPTH TO INITIAL WATER: Not Established
DRILLING METHOD: 4-inch Rotary Wash	DEPTH TO FINAL WATER: Not Established
HAMMER TYPE / WEIGHT / DROP: Automatic Trip Hammer / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473224°; -122.263440°)	

PROJECT:
1815 CORDILLERAS ROAD
Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N ₆₀ -VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
Asphalt Concrete (AC) about 2" thick.	medium dense		0							At 3.5 Feet: Liquid Limit = 27 Plasticity Index = 11 Fine Gravel = 3% Coarse Sand = 4% Medium Sand = 9% Fine Sand = 19% Fines = 65%
Aggregate Base (AB) about 4" thick.										
FILL: SAND (SM), dark brown, fine- to coarse-grained, silty, trace gravel (fine to coarse, angular to subrounded), dry.	hard		0		14	24				At 6 Feet: Liquid Limit = 34 Plasticity Index = 18 Coarse Sand = 4% Medium Sand = 12% Fine Sand = 30% Fines = 54%
CLAY (CL), brown, silty, sandy (fine- to medium-grained, trace coarse-grained), trace gravel (fine, subrounded), dry.							14			
CLAY (CL) / SAND (SC), mottled gray brown, sandy (fine- to coarse-grained), silty, few to with gravel (fine, subangular to subrounded), dry.	hard		5		10	45				At 11 Feet: Fines = 71%
CLAY (CL), mottled gray brown, silty, with to sandy (fine- to medium-grained), moist.							14			
Trace sand (fine- to medium-grained), moist.	very stiff		10		11	27				At 16 Feet: Fines = 97%
Sandy (fine-grained), moist.							14			
With sand (fine-grained), moist to wet.	stiff		15		10	30				At 21 Feet: Fines = 63%
Sandy (fine- to coarse-grained), with gravel (fine, angular to subrounded), moist to wet.							15			
	stiff		20		9	19				At 26 Feet: Fines = 80%
							11			
	stiff		25		4	13				
							7			
	stiff		30		10					

EXPLORATORY BORING LOG 659-5 B-5.dwg STEVENS FERRONE & BAILEY 2/4/2026



1600 Willow Pass Court
 Concord, CA 94520
 Tel: (925) 688-1001

EXPLORATORY BORING B-5

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 1/21/2026
DRILLER: Terra-Ex	DATE FINISHED: 1/21/2026
DRILL RIG: Diedrich D70 Turbo	DEPTH TO INITIAL WATER: Not Established
DRILLING METHOD: 4-inch Rotary Wash	DEPTH TO FINAL WATER: Not Established
HAMMER TYPE / WEIGHT / DROP: Automatic Trip Hammer / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473224°; -122.263440°)	

PROJECT:
1815 CORDILLERAS ROAD
 Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N ₆₀ -VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
CLAY (CL), continued.	hard				16 18	78				At 31 Feet: Fines = 18%
GRAVEL (GC), fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), with silt and clay, moist to wet.	very dense				42					
SANDSTONE, gray, moderately weathered, moderately hard, fine- to medium-grained, dry. Very slow drilling below 35'.	friable to weak		35		50/2"	65/2"				
Drilling refusal at 40'. Bottom of Boring = 40.1 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			40		50/1"	65/1"				
			45							
			50							
			55							
			60							



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EXPLORATORY BORING B-6

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 1/21/2026
DRILLER: Terra-Ex	DATE FINISHED: 1/21/2026
DRILL RIG: Diedrich D70 Turbo	DEPTH TO INITIAL WATER: Not Established
DRILLING METHOD: 4-inch Rotary Wash	DEPTH TO FINAL WATER: Not Established
HAMMER TYPE / WEIGHT / DROP: Automatic Trip Hammer / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473516°; -122.262874°)	

PROJECT:
1815 CORDILLERAS ROAD
Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N ₆₀ -VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
See Boring B-4 (performed on 12/5/2024) for details from 0 to 10 feet.			0							
CLAY (CL), brown, silty, sandy (fine- to coarse-grained), few gravel (fine, subangular to subrounded), moist. Thin silty sand lens (fine- to coarse-grained) at 11'. Change color to mottled gray brown at 11.5'.	stiff		10		3 9 9	14				At 11 Feet: Fines = 14%
SAND (SM/SC), brown, fine- to medium-grained, few coarse-grained, with silt, few clay, moist.	medium dense		15		6 8 10	14				At 16 Feet: Liquid Limit = 25 Plasticity Index = 9 Medium Sand = 16% Fine Sand = 44% Fines = 40%
CLAY (CL), mottled gray brown, silty, few to with sand (fine-grained), moist.	stiff		20		5 7 10	13				At 21 Feet: Fines = 85%
Sandy (fine- to medium-grained).	very stiff		25		5 21 35	44				At 26 Feet: Fines = 20%
GRAVEL (GC), mottled brown gray, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), with silt and clay, moist.	very dense		30							

EXPLORATORY BORING LOG 659-5 B-6.dwg STEVENS FERRONE & BAILEY 2/4/2026



1600 Willow Pass Court
 Concord, CA 94520
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EXPLORATORY BORING B-6

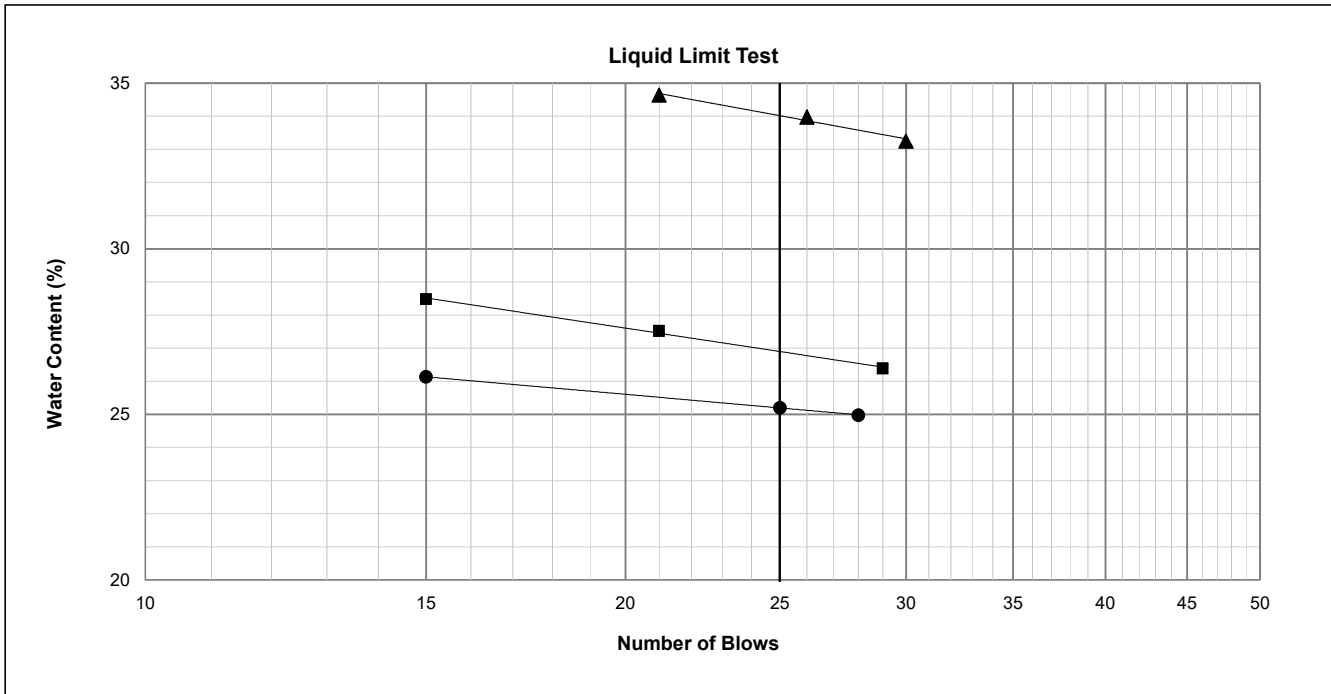
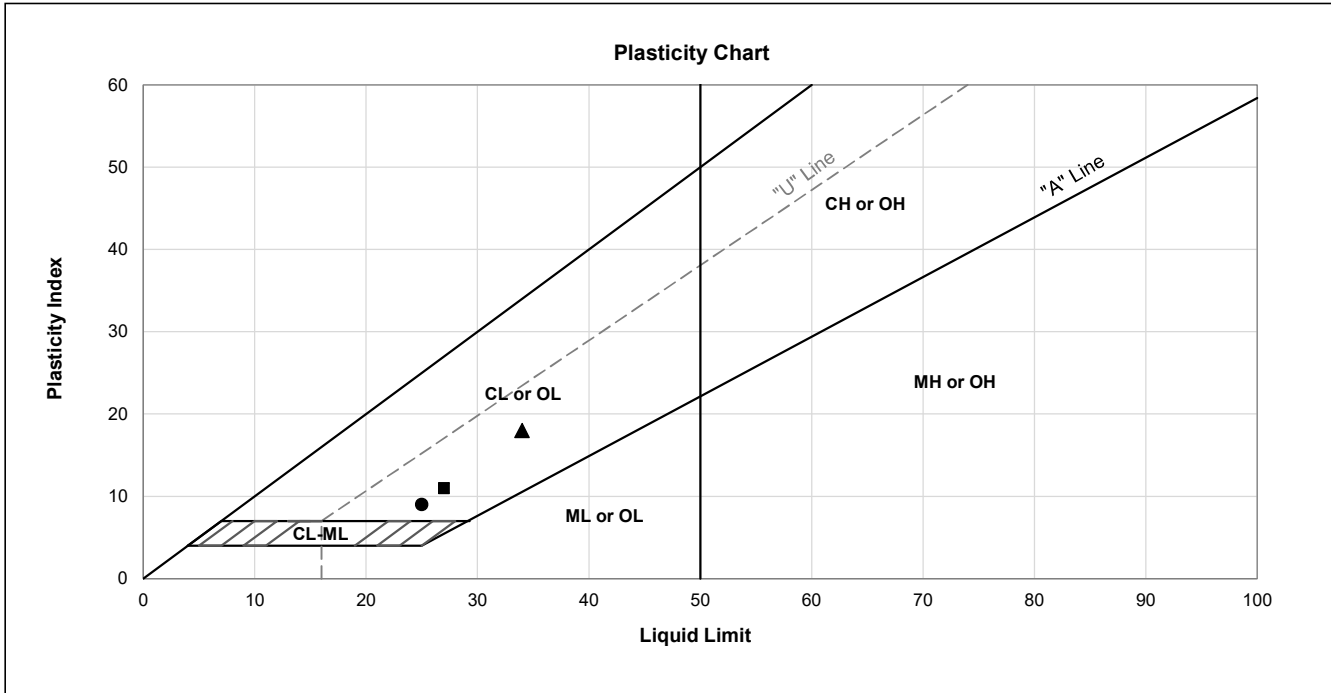
PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 1/21/2026
DRILLER: Terra-Ex	DATE FINISHED: 1/21/2026
DRILL RIG: Diedrich D70 Turbo	DEPTH TO INITIAL WATER: Not Established
DRILLING METHOD: 4-inch Rotary Wash	DEPTH TO FINAL WATER: Not Established
HAMMER TYPE / WEIGHT / DROP: Automatic Trip Hammer / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473516°; -122.262874°)	

PROJECT:
1815 CORDILLERAS ROAD
 Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N ₆₀ -VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
GRAVEL (GC), continued, moist to wet.	very dense		23 27 28		72					
			35		15 25 23	62				
SANDSTONE, gray, moderately weathered, moderately hard, fine- to medium-grained, dry.	friable to weak		40		53/2"	69/2"				
Very slow drilling.										
Bottom of Boring = 40.2 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			45							
			50							
			55							
			60							

APPENDIX B
Supplemental Laboratory Testing

ATTERBERG LIMITS
 ASTM D4318

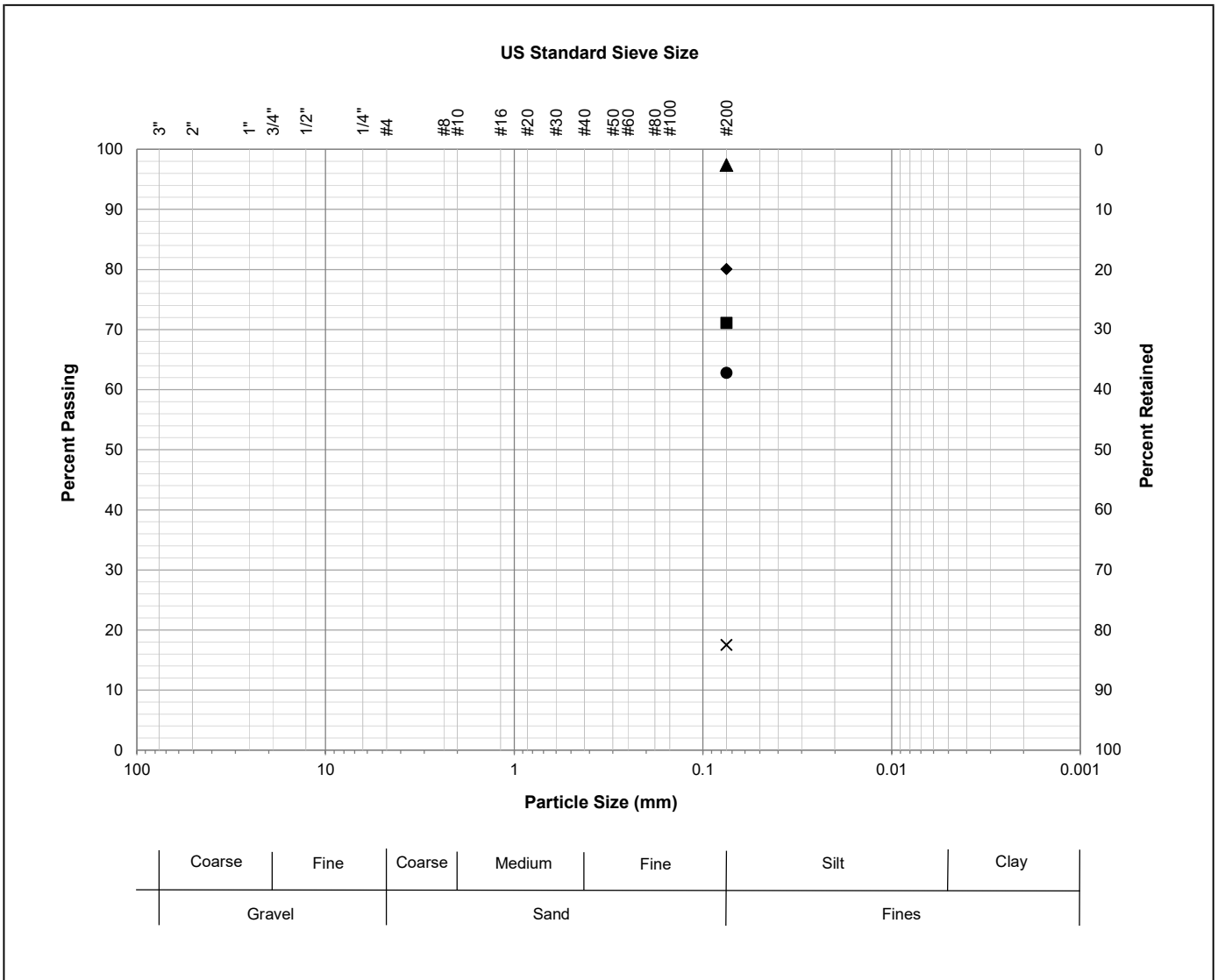


Test Results	Symbol	Sample Source	Sample Description	LL	PL	PI	% _≤ #200	USCS
	■	Boring B-5 at 3.5 ft.	Dark brown sandy silty CLAY	27	16	11	64.5	CL
	▲	Boring B-5 at 6 ft.	Red brown sandy silty CLAY	34	16	18	53.6	CL
	●	Boring B-6 at 16 ft.	Red brown silty clayey SAND	25	16	9	39.6	SC

Project Info	Project Number:	659-5	Test Report Date:	2/4/2026
	Project Name:	1815 Cordilleras Road		
	Project Location:	Emerald Hills, CA		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

PARTICLE SIZE DISTRIBUTION

ASTM C136, D422 & D1140



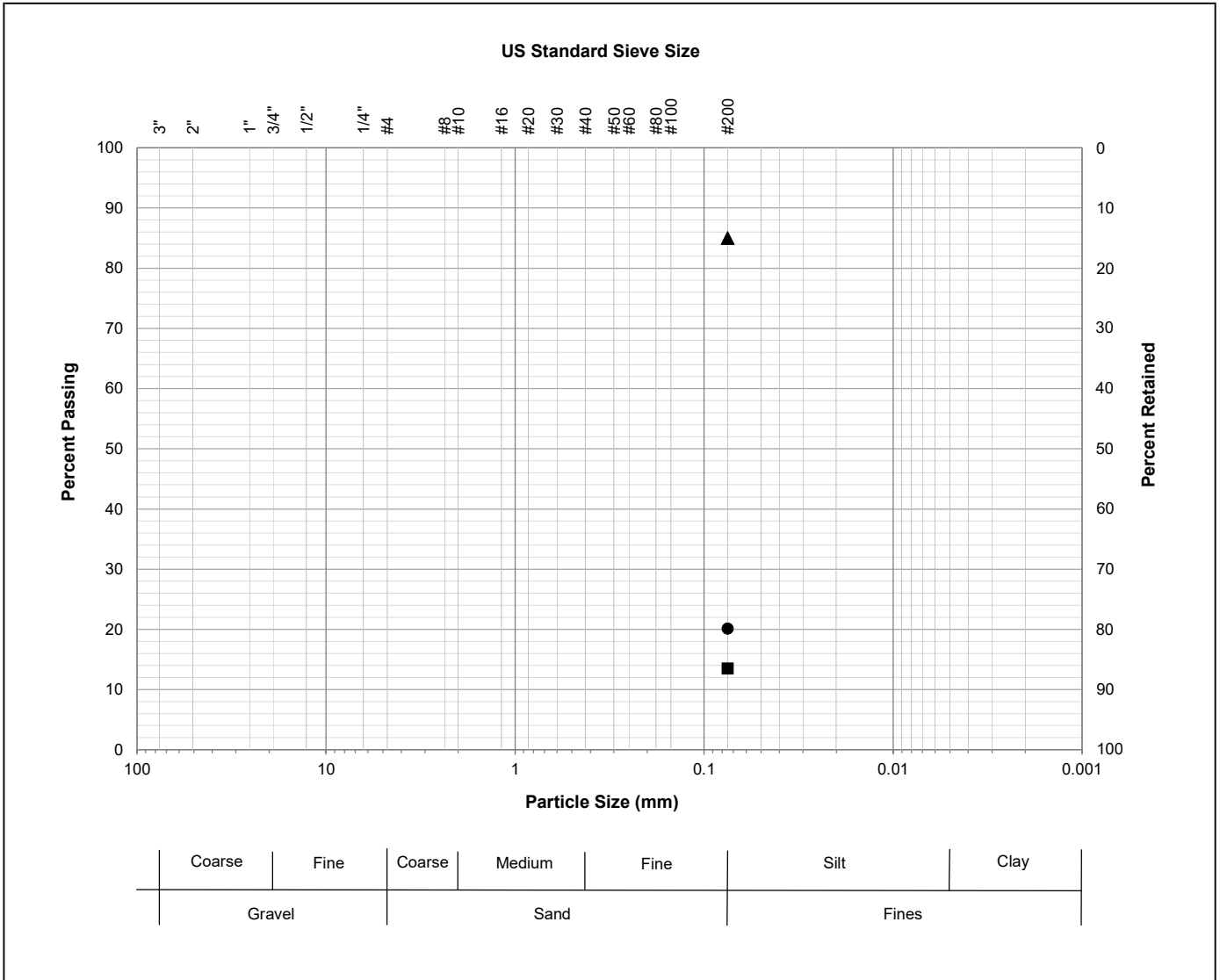
Test Results	Symbol	% >3"	% Gravel		% Sand			% Fines		D ₆₀	D ₃₀	D ₁₀	C _u	C _c
			Coarse	Fine	Coarse	Medium	Fine	Silt	Clay					
	■							71.1						
	▲							97.4						
	●							62.8						
	◆							80.1						
	×							17.5						

Sample Info	Symbol	Sample Source	Sample Description	USCS
		■	Boring B-5 at 11 ft.	Red brown silty CLAY with sand
	▲	Boring B-5 at 16 ft.	Red brown silty CLAY trace sand	CL
	●	Boring B-5 at 21 ft.	Red brown sandy silty CLAY	CL
	◆	Boring B-5 at 26 ft.	Red brown silty CLAY with sand	CL
	×	Boring B-5 at 31 ft.	Red brown sandy GRAVEL with clay	GC

Project Info	Project Number:	659-5	Test Report Date:	1/30/2026
		Project Name:	1815 Cordilleras Road	
	Project Location:	Emerald Hills, CA		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

PARTICLE SIZE DISTRIBUTION

ASTM C136, D422 & D1140



Test Results	Symbol	% >3"	% Gravel		% Sand			% Fines		D ₆₀	D ₃₀	D ₁₀	C _u	C _c
			Coarse	Fine	Coarse	Medium	Fine	Silt	Clay					
	■							13.5						
	▲							85.0						
	●							20.2						

Sample Info	Symbol	Sample Source	Sample Description	USCS
		■	Boring B-6 at 11 ft.	Red brown SAND some clay
	▲	Boring B-6 at 21 ft.	Red brown silty CLAY some sand	CL
	●	Boring B-6 at 26 ft.	Red brown sandy GRAVEL with clay	GC

Project Info	Project Number:	659-5	Test Report Date:	1/30/2026
		Project Name:	1815 Cordilleras Road	
	Project Location:	Emerald Hills, CA		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

APPENDIX C
Previous Field Exploration and Laboratory Testing

EXPLORATORY BORING B-1

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 25 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 27 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473364°; -122.263744°)	

PROJECT:
1815 CORDILLERAS ROAD
Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
FILL: SAND (SM), dark brown, fine- to medium-grained, silty, trace gravel (fine, subangular to subrounded), dry.	loose to medium dense		0		8 9	13	13.3	93.7	1.1	At 3.5 Feet: Liquid Limit = 35 Plasticity Index = 18 Fine Gravel = 5% Coarse Sand = 7% Medium Sand = 17% Fine Sand = 25% Fines = 46%
CLAY (CL)/SAND (SC), mottled gray brown, silty, sandy (fine- to coarse-grained), few to with gravel (fine, subangular to subrounded), trace roots, dry to moist.	hard		12		14	34	11.5			
	very stiff		14		20					
SAND (SM), brown, fine- to medium-grained, few to with silt, dry to moist. Gravelly (fine, subangular to rounded) at 11.5 feet.	medium dense		5		30 28	29				
			10		12 14 14	17				
CLAY (CL), mottled gray brown, silty, with to sandy (fine-grained), moist. Sandy (fine- to medium-grained) at 16.5 feet.	very stiff		15		10 13 20	20				
			20		6 6 6	7				
CLAY (CL), mottled light gray brown, silty, trace sand (fine-grained), moist. very stiff	firm		25		7 7 21	28				
			30							
GRAVEL (GC), dark gray, fine to coarse, angular to subrounded, sandy (fine- to coarse-grained), with clay and silt, wet.	very dense									



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EXPLORATORY BORING B-1

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 25 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 27 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473364°; -122.263744°)	

PROJECT:
1815 CORDILLERAS ROAD
 Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
GRAVEL (GC), continued. Drilling refusal at 31 feet.	very dense		31		50/6"	50/6"				
Bottom of Boring = 31 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.			35							
			40							
			45							
			50							
			55							
			60							



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EXPLORATORY BORING B-2

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: Not Encountered
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: Not Encountered
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.472824°; -122.263727°)	

PROJECT:
1815 CORDILLERAS ROAD
 Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS	
DESCRIPTION	CONSIST	GRAPHIC LOG									
CLAY (CL), mottled dark gray brown, silty, with to sandy (fine- to coarse-grained), few gravel (fine, subangular to subrounded), dry to moist.	firm to very stiff		0		18	20	16.4	109.6	2.9	At 2 Feet: Corrosion Tests.	
Moist.					17						
					17						
					13	29					
					14						
					15						
Sandy (fine- to coarse-grained), few to with gravel (fine to coarse, subangular to subrounded).	hard			5		25					38
						23					
						40					
				10		9					16
	very stiff					11					
						15					
				15		15					23
						14					
						25					
				20		14					25
						20					
						22					
Bottom of Boring = 21.5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.				25							
				30							

EXPLORATORY BORING LOG 659-5 B-2.dwg STEVENS FERRONE & BAILEY 12/16/2024



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Concord, CA 94520
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EXPLORATORY BORING B-3

PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: Not Encountered
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: Not Encountered
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473044°; -122.262816°)	

PROJECT:
1815 CORDILLERAS ROAD
Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS
DESCRIPTION	CONSIST	GRAPHIC LOG								
SAND (SM)/SILT (ML), grayish brown, fine- to medium-grained, silty, few gravel (fine, subangular to subrounded), dry.	loose to medium dense		0		22 15 15	18	8.3	93.7	1.2	
SAND (SM), mottled gray brown, fine- to coarse-grained, silty, trace gravel (fine, subangular to subrounded), dry.	medium dense		5		7 9 12	21				
CLAY (CL), mottled gray brown, sandy (fine- to medium-grained), moist.	very stiff		10		19 20 25	27				
With sand (fine- to coarse-grained), trace gravel (fine, subangular to subrounded).			15		14 27 22	29				
SAND (SM), brown, fine- to medium-grained, silty, lightly cemented, trace roots, dry to moist.	medium dense		20		18 17 15	19	8.0	101.7	2.6	
CLAY (CL), brown, sandy (fine- to medium-grained), moist.	stiff		25		10 10 11	13				
Bottom of Boring = 21.5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.										
			30							

EXPLORATORY BORING LOG 659-5 B-3.dwg STEVENS FERRONE & BAILEY 12/16/2024



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EXPLORATORY BORING B-4

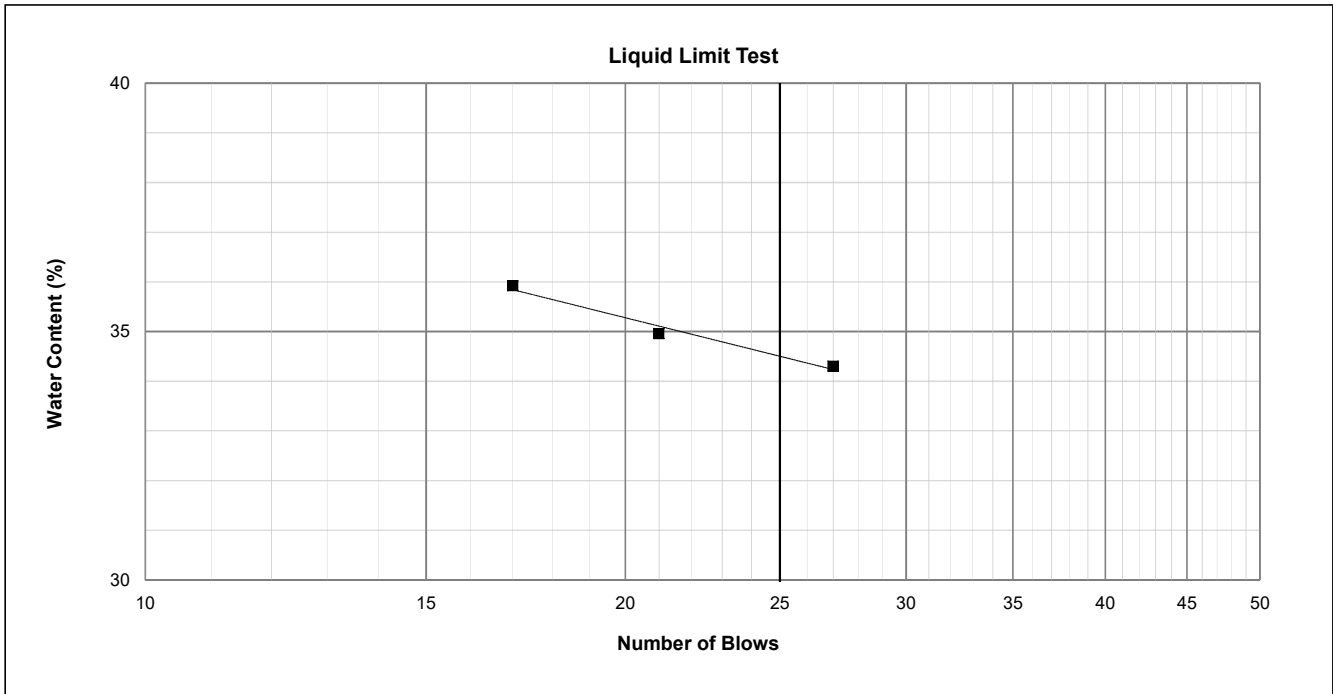
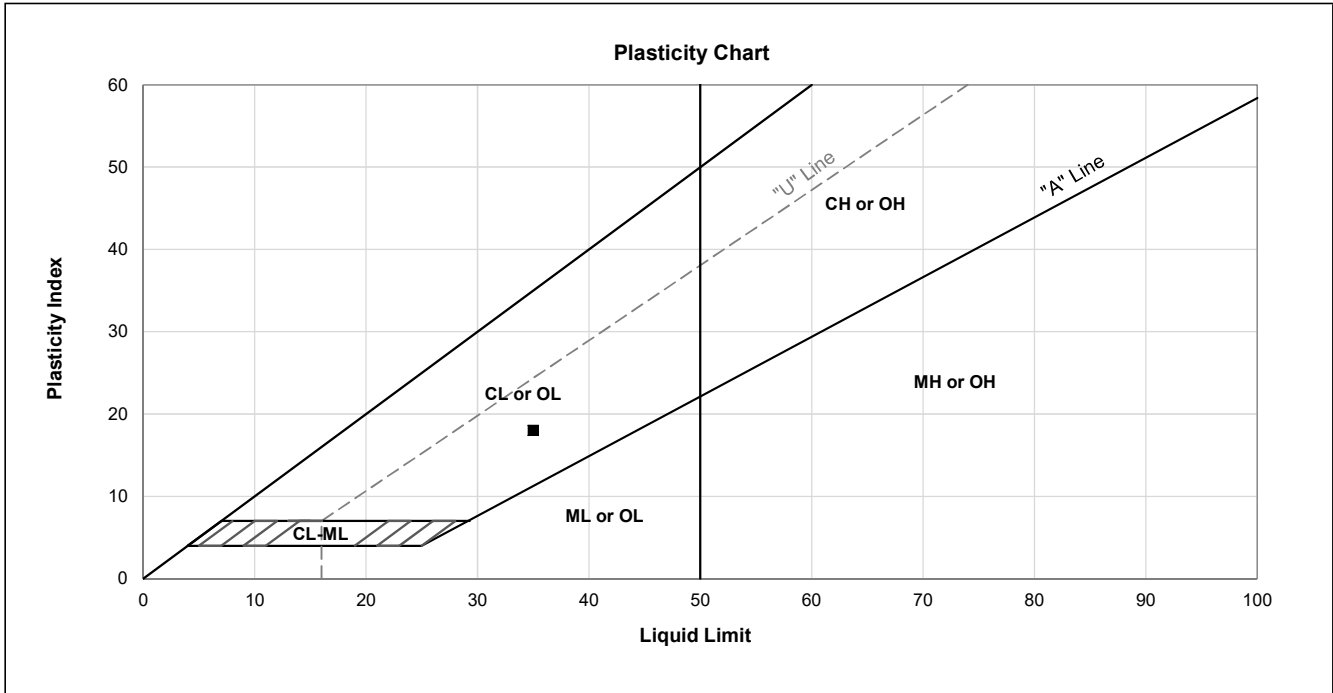
PROJECT NO: 659-5	SURFACE ELEVATION: --
LOGGED / CHECKED BY: R. Ceraolo / T. Chen	DATE STARTED: 12/5/2024
DRILLER: West Coast Exploration	DATE FINISHED: 12/5/2024
DRILL RIG: Mobile B-24	DEPTH TO INITIAL WATER: 20 Feet
DRILLING METHOD: 4-inch Solid Flight Auger	DEPTH TO FINAL WATER: 18 Feet
HAMMER TYPE / WEIGHT / DROP: Safety Hammer with Rope & Cathead / 140 pounds / 30 inches	
BORING LOCATION: See Site Plan, Figure 2 (37.473524°; -122.262882°)	

PROJECT:
1815 CORDILLERAS ROAD
Emerald Hills, CA

SUBSURFACE MATERIAL CLASSIFICATION			DEPTH (FEET)	SAMPLER	FIELD BLOW COUNT	SPT N-VALUE	WATER CONTENT (%)	DRY DENSITY (PCF)	UCS (KSF)	OTHER TESTS NOTES AND REMARKS	
DESCRIPTION	CONSIST	GRAPHIC LOG									
CLAY (CL), mottled dark gray brown, silty, with sand (fine- to medium-grained), few gravel (fine, subangular to subrounded), with large tree roots, dry.	firm to very stiff		0		11 18 20	23				At 2 Feet: Corrosion Tests.	
CLAY (CL), mottled gray yellowish brown, silty, with to sandy (fine- to coarse-grained), few gravel (fine, subangular to subrounded), dry.	very stiff				10 12 10	22					
Change color to brown, sandy (fine- to medium-grained), dry to moist.			5		18 17 17	20	12.2	111.5	5.9		
Change color to mottled light gray brown, trace gravel (fine, subangular to subrounded).	stiff		10		12 18 22	12					
SAND (SM), brown, fine- to medium-grained, few coarse-grained, with silt, few gravel (fine, subangular to subrounded), dry to moist.	loose		15		10 7 7	8	7.0				At 15.5 Feet: Fines = 15%
CLAY (CL), mottled gray brown, silty, trace sand (fine-grained), moist.	stiff										
SAND (SM), mottled gray brown, fine- to coarse-grained, silty, trace gravel (fine, subangular to subrounded), trace clay, moist to wet.	loose		20		7 7 7	8					
Bottom of Boring = 21.5 feet Notes: Stratification is approximate; variations must be expected. See report for additional details.											
			25								
			30								

EXPLORATORY BORING LOG 659-5 B-4.dwg STEVENS FERRONE & BAILEY 12/16/2024

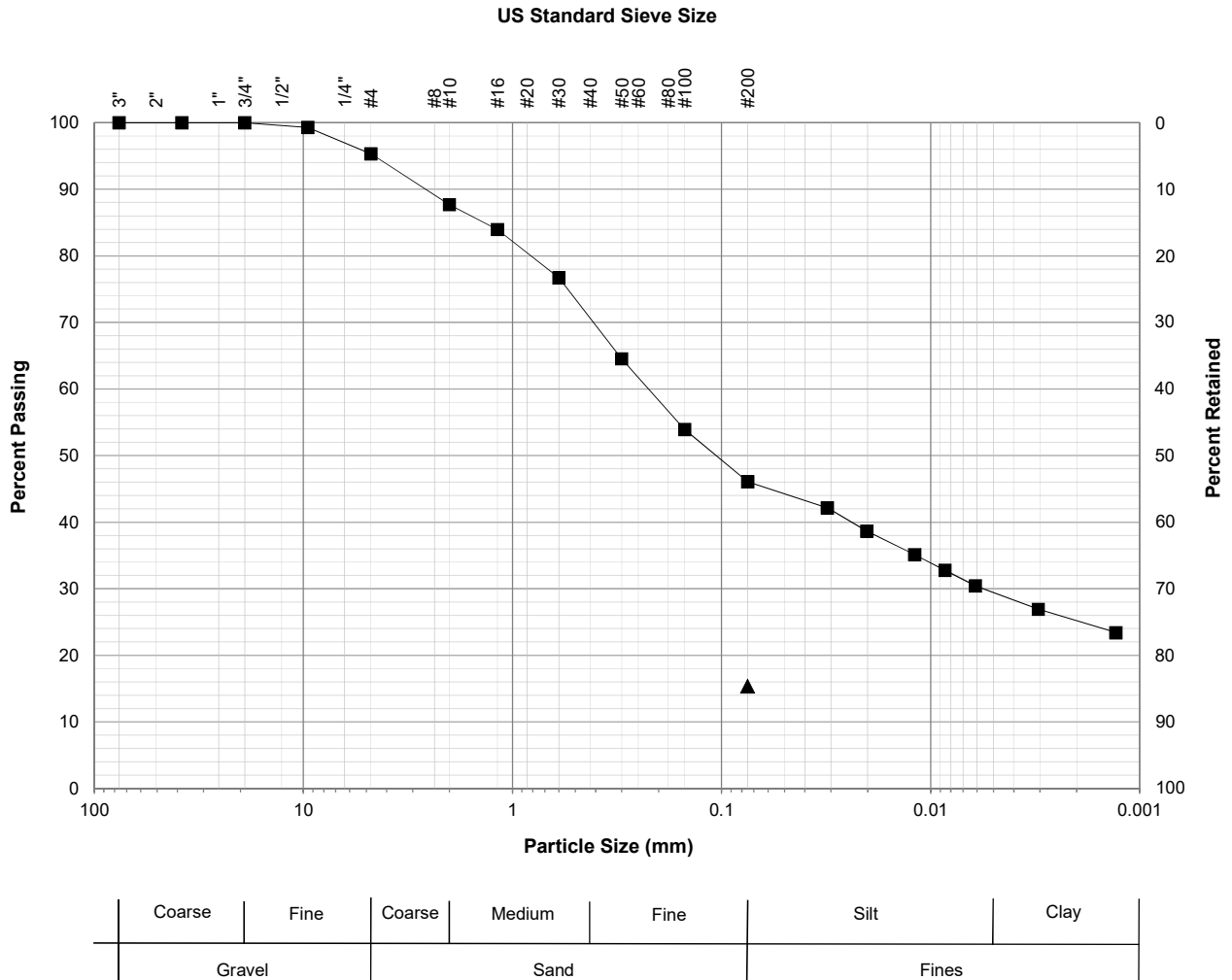
ATTERBERG LIMITS
 ASTM D4318



Test Results	Symbol	Sample Source	Sample Description	LL	PL	PI	% _≤ #200	USCS
	■	Boring B-1 at 3.5 ft.	Brown silty clayey SAND trace gravel	35	17	18	46.1	SC
Project Info	Project Number: 659-5			Test Report Date: 12/11/2024				
	Project Name: 1815 Cordilleras Rd.							
	Project Location: Emerald Hills, CA							
	Tested by: R. Tuazon							
	Checked by: T. Chen							

PARTICLE SIZE DISTRIBUTION

ASTM C136, D422 & D1140

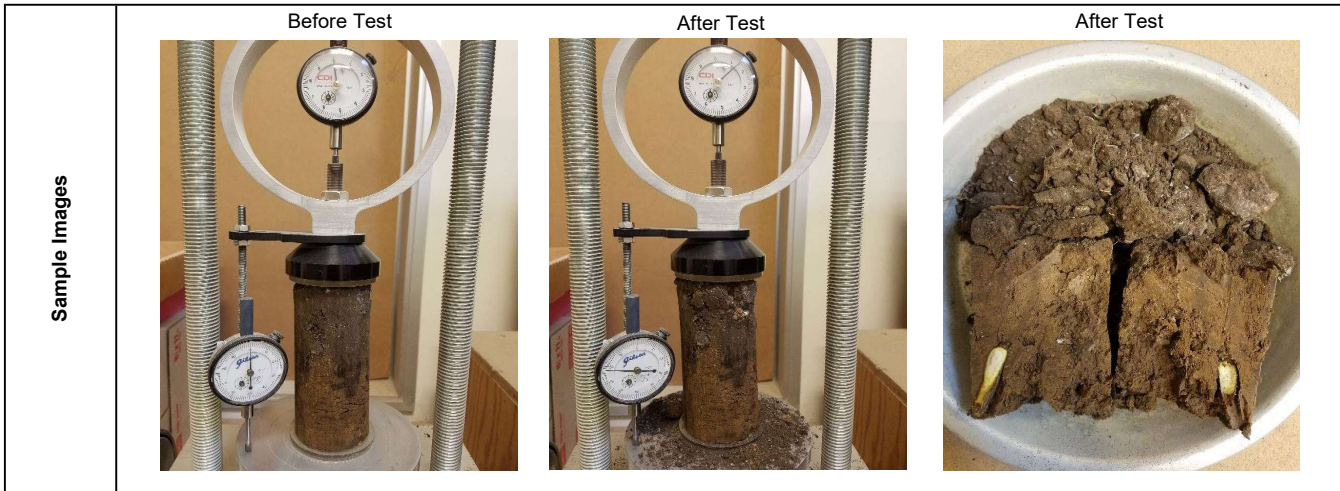
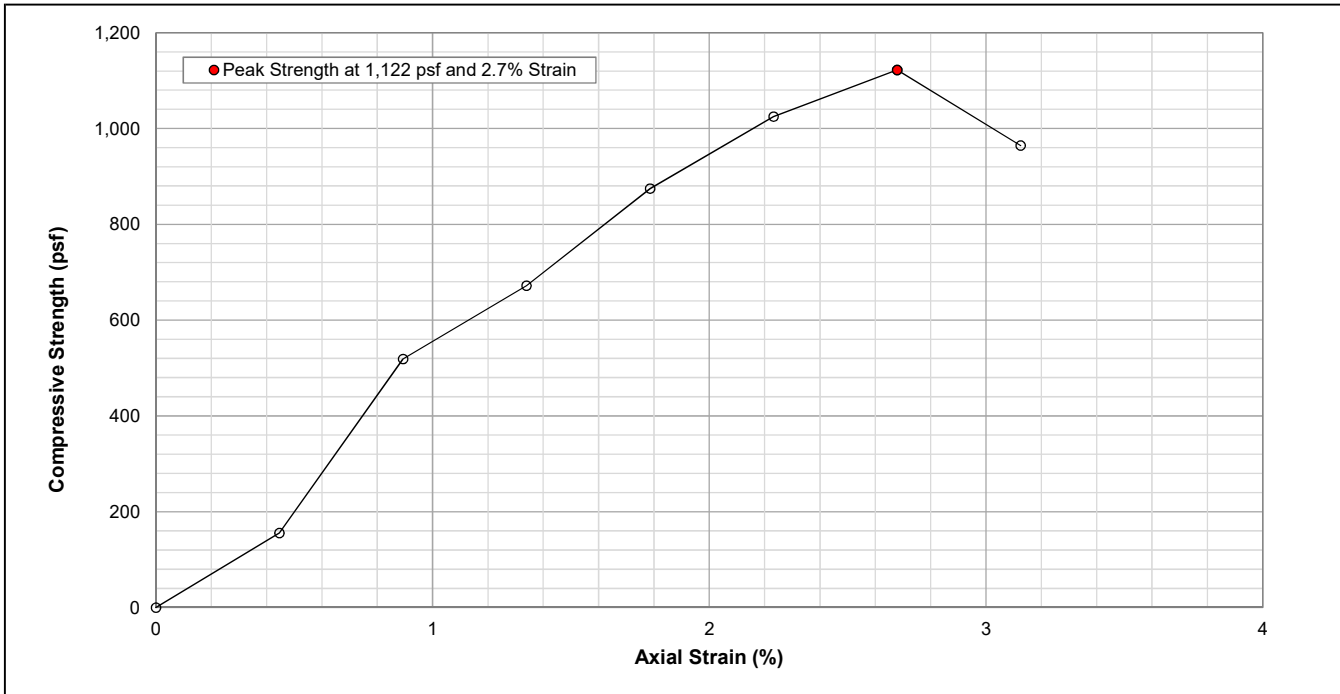


Test Results	Symbol	% >3"	% Gravel		% Sand			% Fines		D ₆₀	D ₃₀	D ₁₀	C _u	C _c
			Coarse	Fine	Coarse	Medium	Fine	Silt	Clay					
	■	0.0	0.0	4.7	7.6	17.0	24.6	46.1	0.223	0.006				
▲							15.4							

Sample Info	Symbol	Sample Source	Sample Description	USCS
	■	Boring B-1 at 3.5 ft.	Brown silty clayey SAND trace gravel	SC
▲	Boring B-4 at 15.5 ft.	Red brown SAND with silt trace gravel	SM	

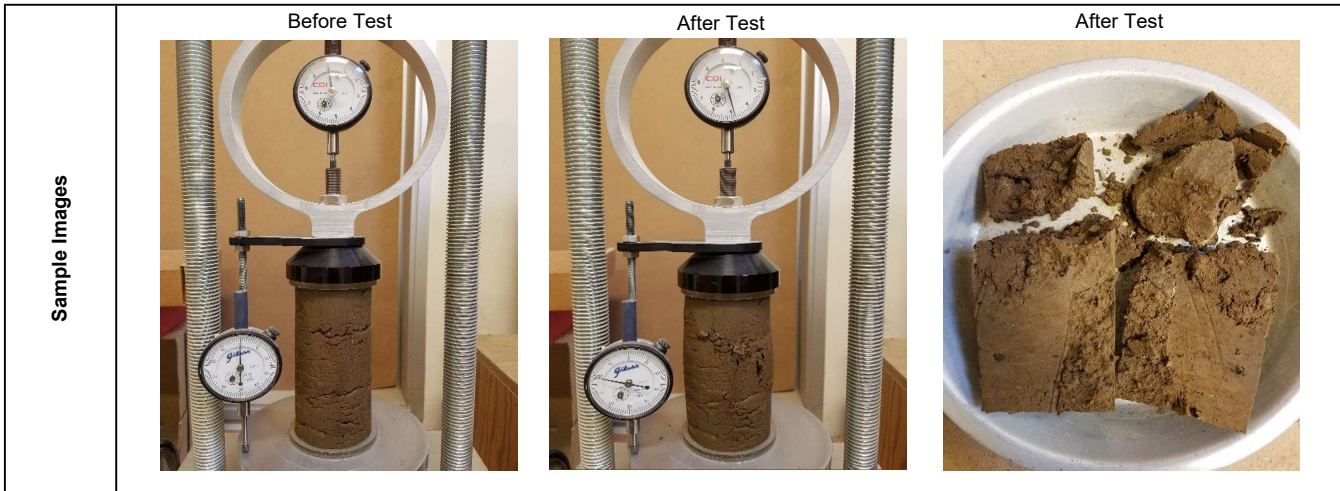
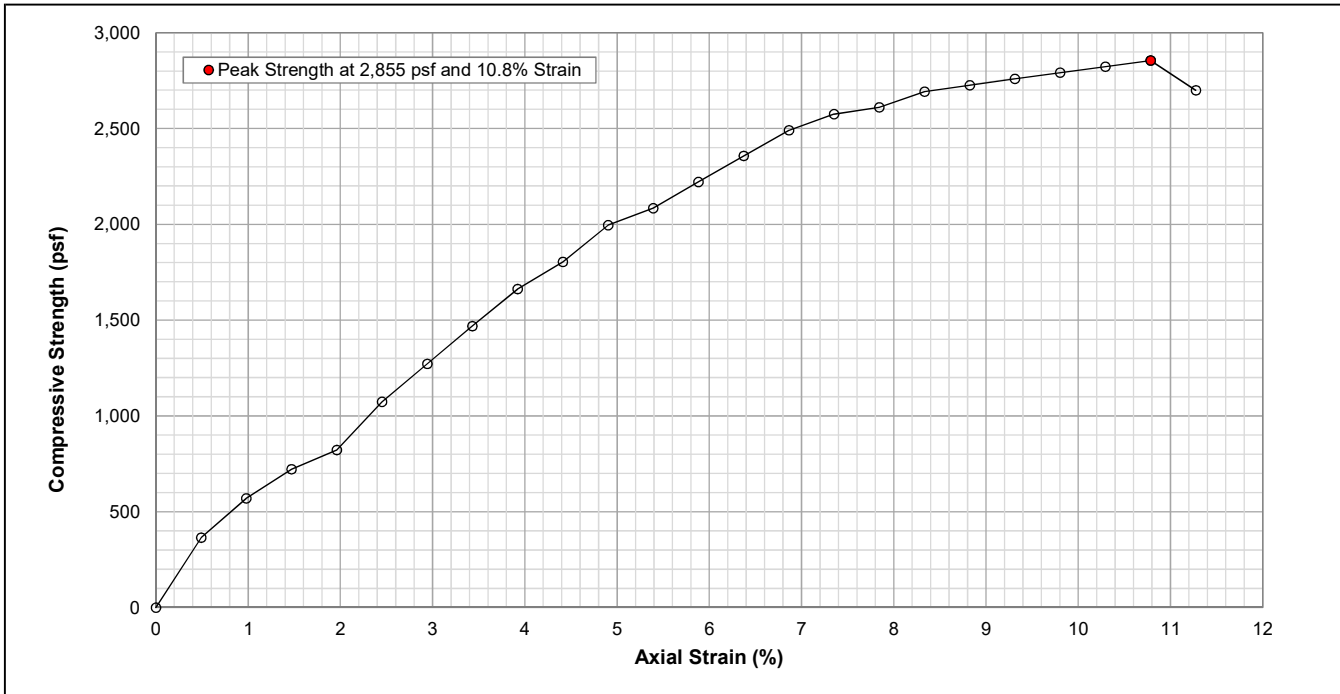
Project Info	Project Number:	659-5	Test Report Date:	12/12/2024
		Project Name:	1815 Cordilleras Rd.	
	Project Location:	Emerald Hills, CA		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



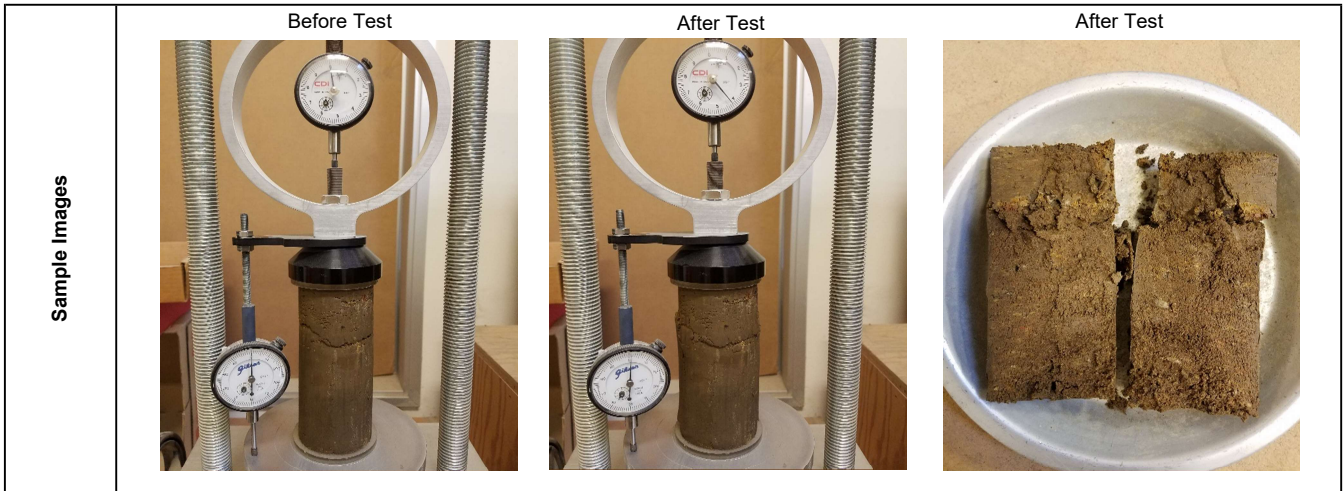
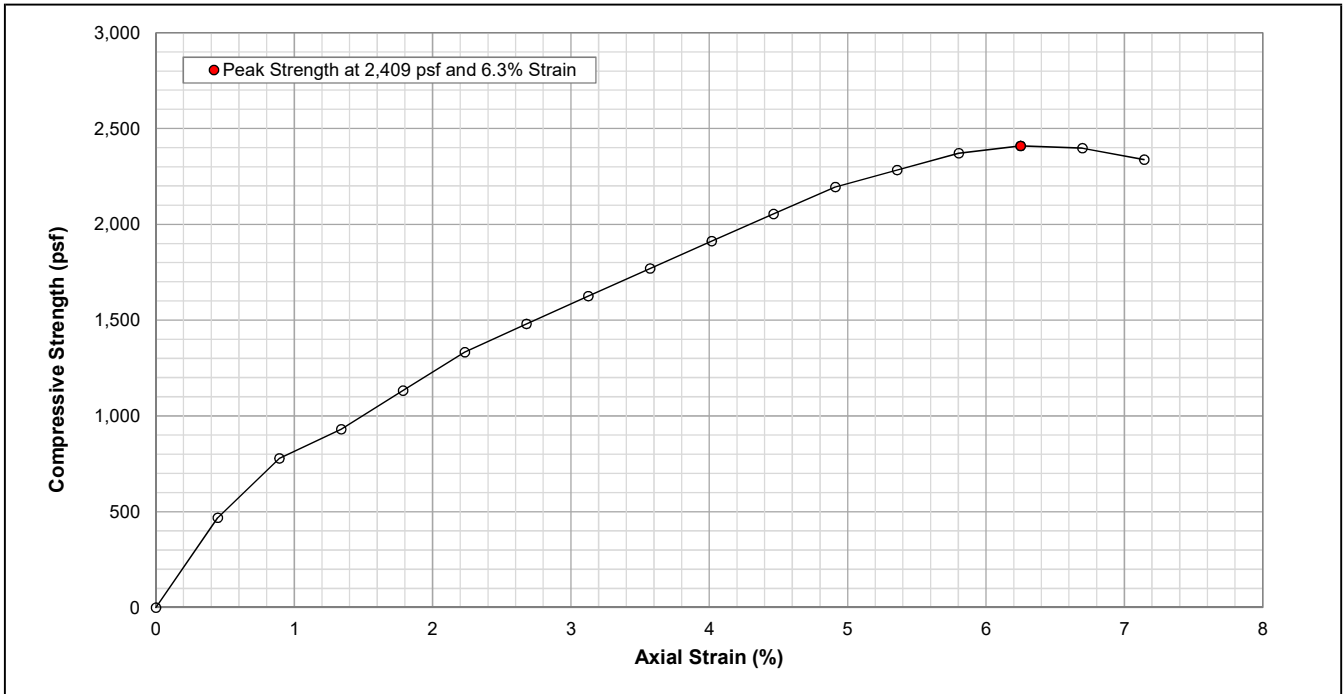
Sample Properties	Sample Description:	Dark brown sandy silty CLAY trace gravel and roots (CL)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	1,122 psf
	Height:	5.6 in		Axial Strain at Failure:	2.7 %
	Height/Diameter:	2.31		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	106.1 pcf		Test Time to Failure:	3 min
	Water Content:	13.3 %		Remarks:	
	Dry Unit Weight:	93.7 pcf			
Project Info	Project Number:	659-5	Test Report Date: 12/16/2024		
	Project Name:	1815 Cordilleras Rd.			
	Project Location:	Emerald Hills, CA			
	Sample Source/No.:	Boring B-1			
	Sample Depth:	2 ft.			
	Tested by:	R. Tuazon			
Checked by:	T. Chen				

UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Brown silty CLAY with sand some gravel (CL)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	2,855 psf
	Height:	5.1 in	Axial Strain at Failure:	10.8 %	
	Height/Diameter:	2.11	Test Strain Rate:	0.05 in/min	
	Wet Unit Weight:	127.6 pcf	Test Time to Failure:	11 min	
	Water Content:	16.4 %	Remarks:		
	Dry Unit Weight:	109.6 pcf			
Project Info	Project Number:	659-5	Test Report Date: 12/16/2024		
	Project Name:	1815 Cordilleras Rd.			
	Project Location:	Emerald Hills, CA			
	Sample Source/No.:	Boring B-2			
	Sample Depth:	11 ft.			
	Tested by:	R. Tuazon			
	Checked by:	T. Chen			

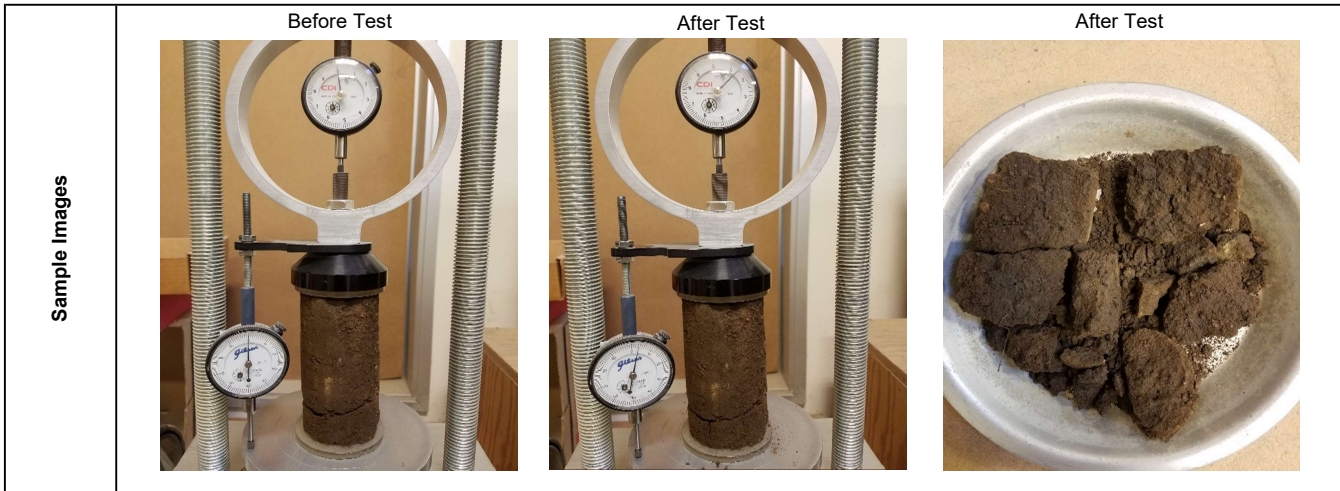
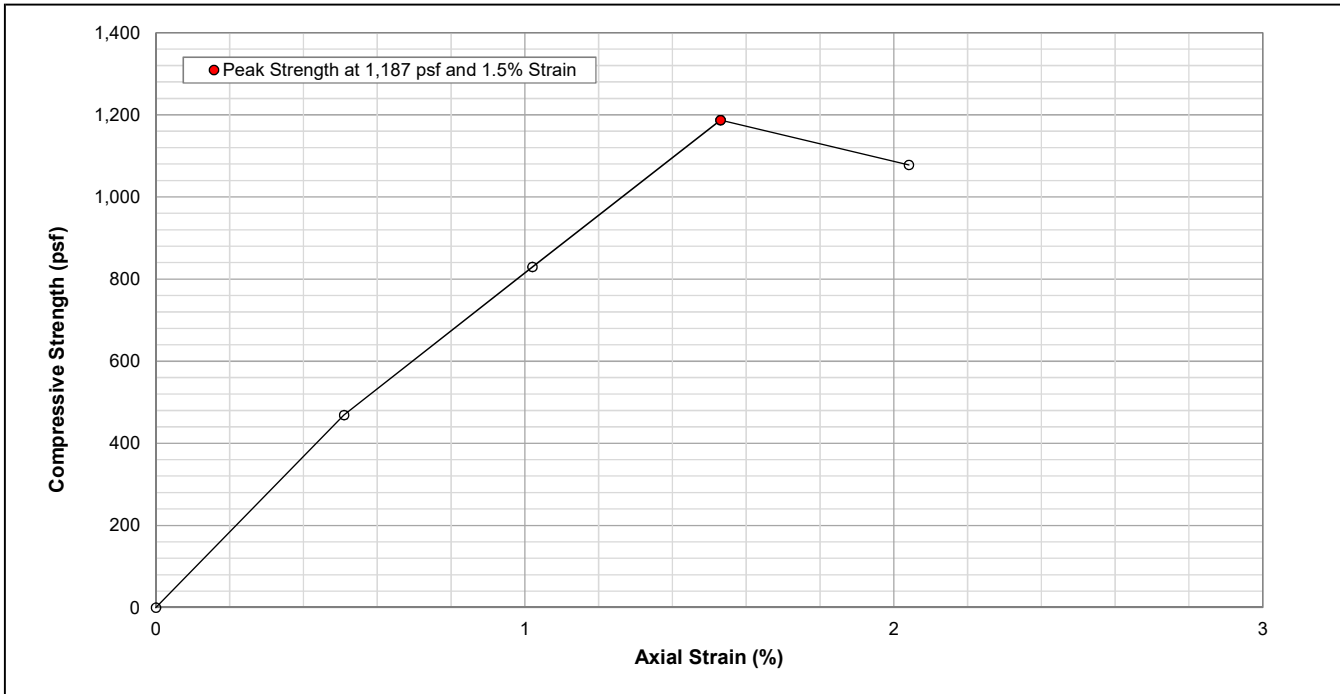
UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Brown sandy silty CLAY trace gravel (CL)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	2,409 psf
	Height:	5.6 in		Axial Strain at Failure:	6.3 %
	Height/Diameter:	2.31		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	125.5 pcf		Test Time to Failure:	7 min
	Water Content:	19.2 %		Remarks:	
	Dry Unit Weight:	105.3 pcf			

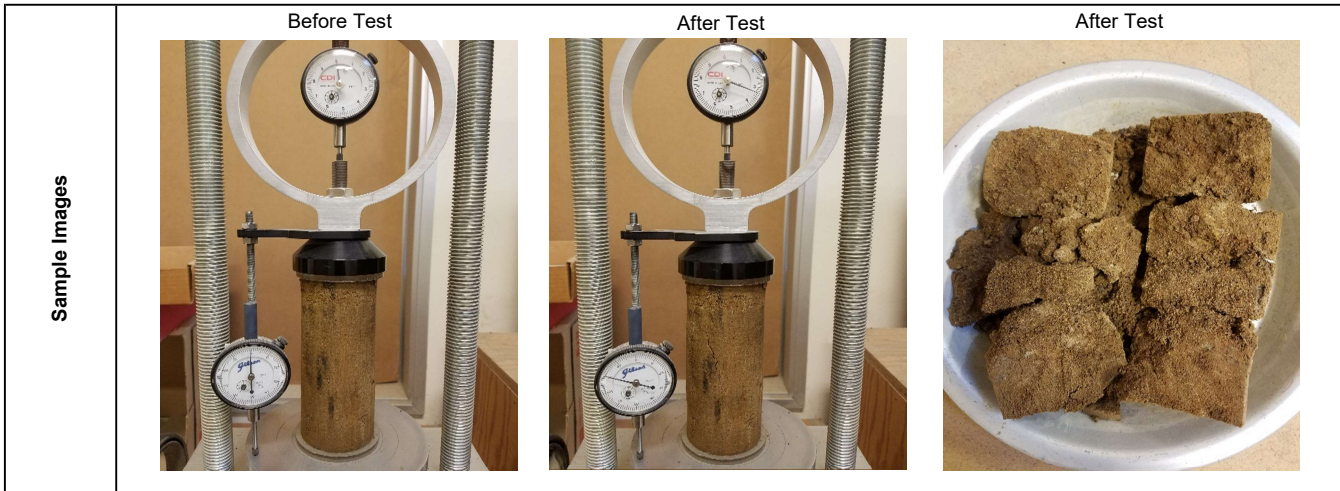
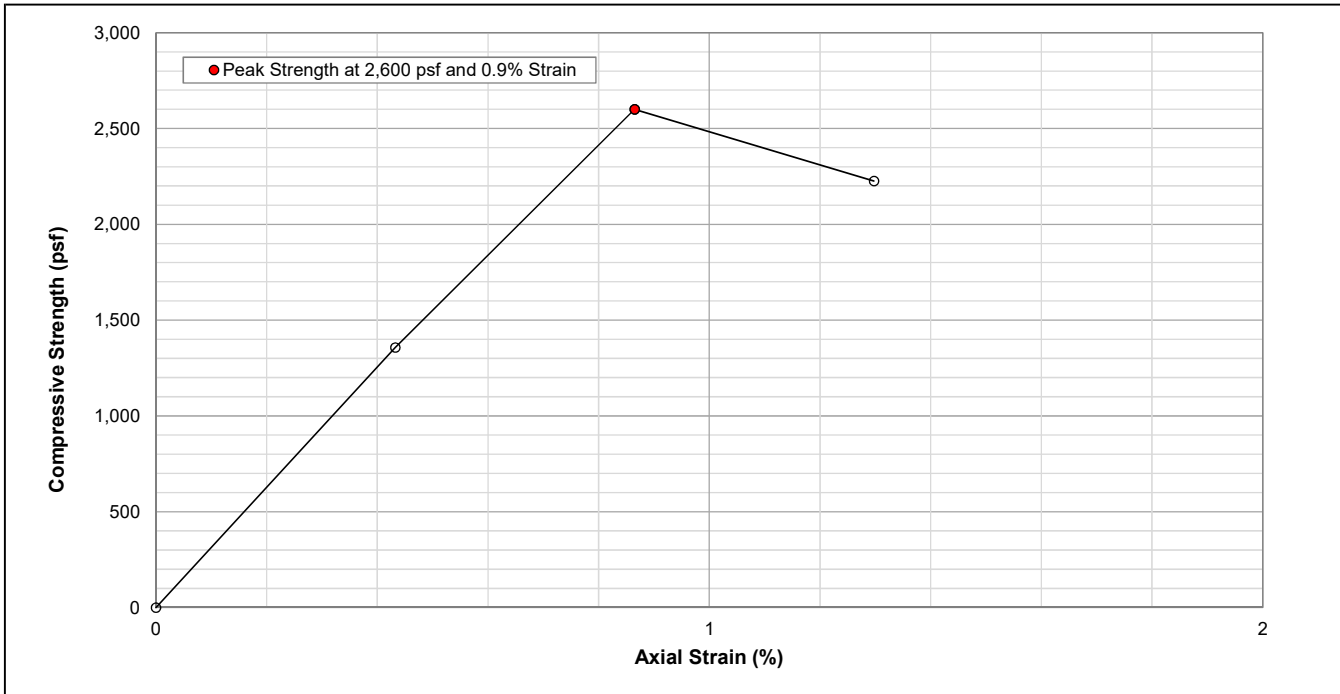
Project Info	Project Number:	659-5	Test Report Date:	12/16/2024
	Project Name:	1815 Cordilleras Rd.		
	Project Location:	Emerald Hills, CA		
	Sample Source/No.:	Boring B-2		
	Sample Depth:	21 ft.		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Dark brown sandy SILT (ML/SM)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	1,187 psf
	Height:	4.9 in		Axial Strain at Failure:	1.5 %
	Height/Diameter:	2.02		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	101.6 pcf		Test Time to Failure:	1.5 min
	Water Content:	8.3 %		Remarks:	
	Dry Unit Weight:	93.7 pcf			
Project Info	Project Number:	659-5	Test Report Date: 12/16/2024		
	Project Name:	1815 Cordilleras Rd.			
	Project Location:	Emerald Hills, CA			
	Sample Source/No.:	Boring B-3			
	Sample Depth:	2 ft.			
	Tested by:	R. Tuazon			
	Checked by:	T. Chen			

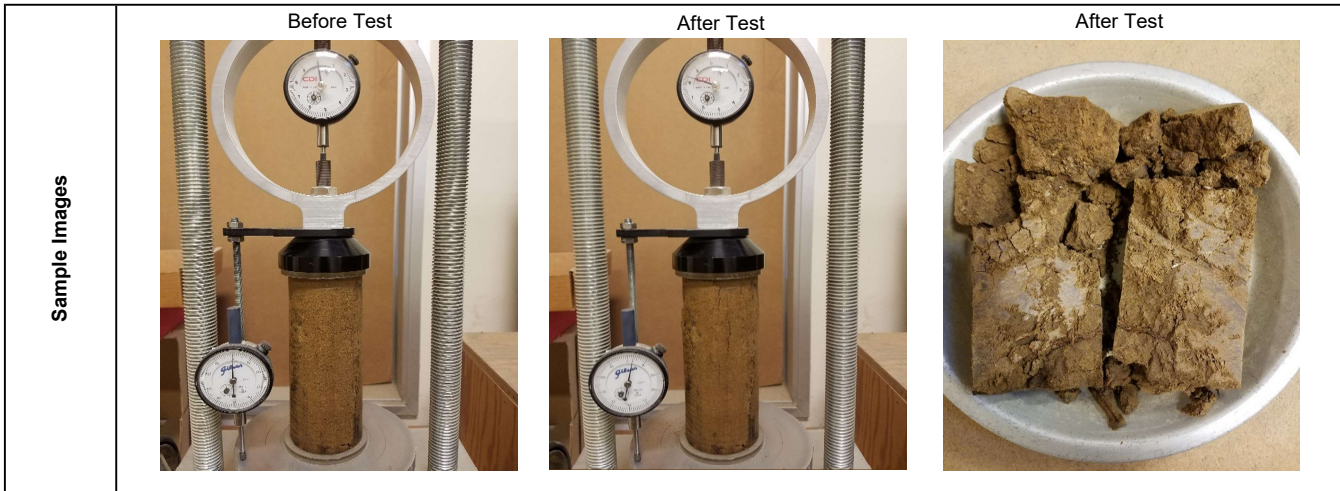
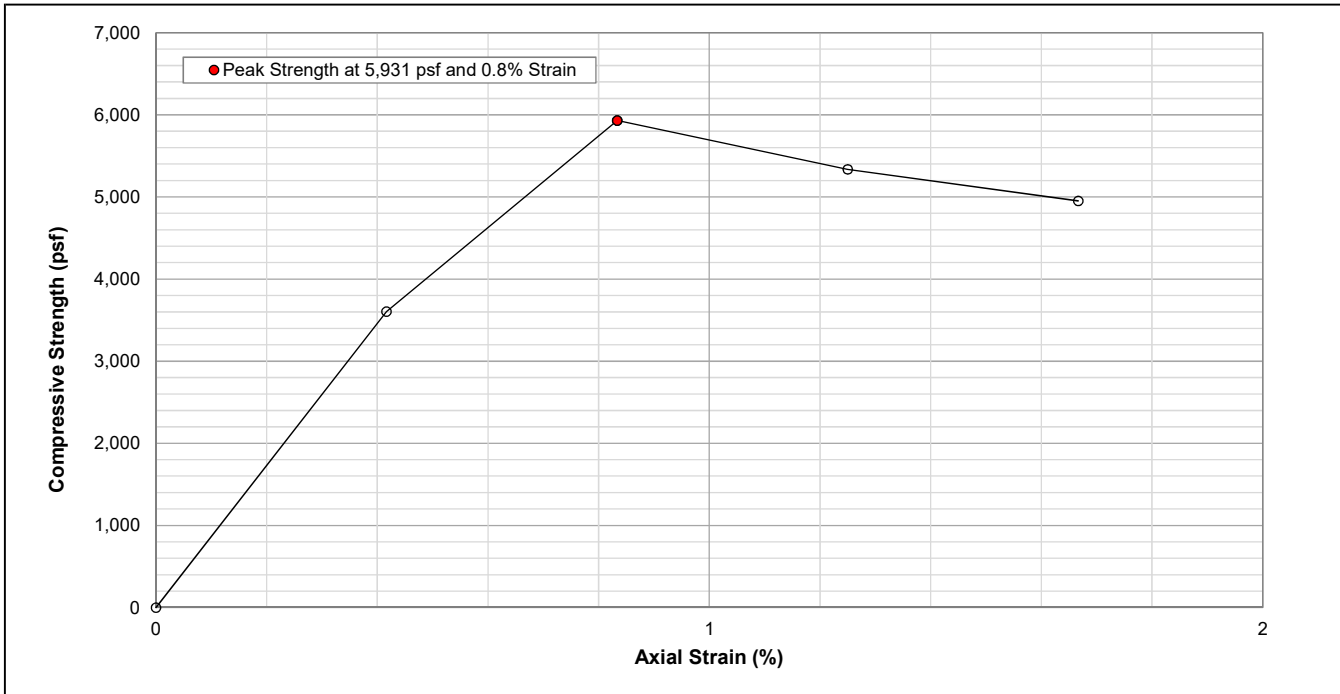
UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Light brown silty SAND (SM)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	2,600 psf
	Height:	5.78 in		Axial Strain at Failure:	0.9 %
	Height/Diameter:	2.39		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	109.8 pcf		Test Time to Failure:	1 min
	Water Content:	8.0 %		Remarks:	
	Dry Unit Weight:	101.7 pcf			

Project Info	Project Number:	659-5	Test Report Date:	12/16/2024
	Project Name:	1815 Cordilleras Rd.		
	Project Location:	Emerald Hills, CA		
	Sample Source/No.:	Boring B-3		
	Sample Depth:	16 ft.		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		

UNCONFINED COMPRESSIVE STRENGTH
 ASTM D2166



Sample Properties	Sample Description:	Red brown silty CLAY with sand (CL)	Test Results	Test Date:	12/13/2024
	Diameter:	2.42 in		Compressive Strength:	5,931 psf
	Height:	6 in		Axial Strain at Failure:	0.8 %
	Height/Diameter:	2.48		Test Strain Rate:	0.05 in/min
	Wet Unit Weight:	125.2 pcf		Test Time to Failure:	1 min
	Water Content:	12.2 %		Remarks:	
	Dry Unit Weight:	111.5 pcf			

Project Info	Project Number:	659-5	Test Report Date:	12/16/2024
	Project Name:	1815 Cordilleras Rd.		
	Project Location:	Emerald Hills, CA		
	Sample Source/No.:	Boring B-4		
	Sample Depth:	6 ft.		
	Tested by:	R. Tuazon		
	Checked by:	T. Chen		



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12 December, 2024

Job No. 2412021
Cust. No. 11486

Mr. Taiming Chen
Stevens, Ferrone & Bailey
1600 Willow Pass Court
Concord, CA 94520

Subject: Project No.: 659-5
Project Name: 1815 Cordilleras Road, Emerald Hills, CA
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Chen:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on December 09, 2024. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, both samples are classified as “moderately corrosive”. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfate ion concentrations reflect none detected with a reporting limit of 15 mg/kg.

The sulfide ion concentrations reflect none detected with a reporting limit of 50 mg/kg.

The pH of the soils are 7.26 and 8.07, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials are 270-mV and 350-mV, which are indicative of potentially “slightly corrosive” soils.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc. at (925) 927-6630.*

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

A handwritten signature in blue ink that reads 'J. Darby Howard, Jr.'.

J. Darby Howard, Jr., P.E.
President


JDH/jdl
Enclosure

Client: Stevens, Ferrone & Bailey Engineering
 Client's Project No.: 659-5
 Client's Project Name: 1815 Cordilleras Road, Emerald Hills, CA
 Date Sampled: 5-Dec-24
 Date Received: 9-Dec-24
 Matrix: Soil
 Authorization: Signed Chain of Custody

Date of Report: 12-Dec-2024

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2412021-001	B-2 at 2'	350	7.26	-	5,400	N.D.	N.D.	N.D.
2412021-002	B-4 at 2'	270	8.07	-	7,400	N.D.	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	10-Dec-2024	10-Dec-2024	-	11-Dec-2024	11-Dec-2024	11-Dec-2024	11-Dec-2024


 Julia Clauson
 Chemist

* Results Reported on "As Received" Basis
 N.D. - None Detected

APPENDIX D
Soil Liquefaction Analyses

**LIQUEFACTION ANALYSES BASED ON SPT DATA
1815 CORDILLERAS ROAD, EMERALD HILLS, CALIFORNIA**



SFB 659-5
2/5/2026 TC

a_{max} = 0.92 g ASCE 7-22 PGAm
Mw = 7.9

B-1
Ground Elevation = ft
Depth to Ground Water Table During Exploration = 27 ft = EL ft
Depth to Ground Water Table During Earthquake = 10 ft =EL ft
γ = 120 pcf
γ_{sat} = 125 pcf
Boring Diameter = 5 inch = 127 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation	Depth	Depth	N	σ _v Exploration	σ _v Earthquake	σ _v ' Exploration	σ _v ' Earthquake	C _R	Liner Correction	C _S	C _B	C _E	C _N	N _{1,60}	FC	ΔN	N _{1,60,cs}	r _d	CSR	BI2014		CRR _{M=7.5,1 atm}	CRR	FS	Yoshimine 2006						
																				MSF _{max}	MSF				K _σ	γ _{lim}	F _α	γ _{max}	ε _v	ΔH (ft)	Δs (in)
ft	ft	m	blow/ft	psf	psf	psf	kPa		Y/N						%	blow/ft									%	ft	in				
	11.0	3.4	17	1,320.0	1,325.0	1,320.0	1,262.6	0.85	N	1.00	1.05	1.0	1.25	19	10	1	20	0.99	0.62	1.50	0.94	1.07	0.21	0.21	0.3	0.158	0.514	0.158	2.3	3.0	0.8
																										Total	2.3	3.0	0.8		

**LIQUEFACTION ANALYSES BASED ON SPT DATA
1815 CORDILLERAS ROAD, EMERALD HILLS, CALIFORNIA**



SFB 659-5
2/5/2026 TC

a_{max} = 0.92 g ASCE 7-22 PGAm
Mw = 7.9

B-1
Ground Elevation = ft
Depth to Ground Water Table During Exploration = 27 ft = EL ft
Depth to Ground Water Table During Earthquake = 0 ft =EL ft
γ = 120 pcf
γ_{sat} = 125 pcf
Boring Diameter = 5 inch = 127 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation		Depth		N	σ _v Exploration	σ _v Earthquake	σ _v ' Exploration	σ _v ' Earthquake	C _R	Liner Correction	C _S	C _B	C _E	C _N	N _{1,60}	FC	ΔN	N _{1,60,cs}	r _d	CSR	BI2014		CRR _{M=7.5,1 atm}	CRR	FS	Yoshimine 2006						
ft	ft	m	blow/ft																		psf	psf				psf	kPa	Y/N	MSF _{max}	MSF	K _σ	γ _{lim}
				17	1,320.0	1,375.0	1,320.0	688.6	0.85	N	1.00	1.05	1.0	1.25	19	10	1	20	0.99	1.18	1.50	0.94	1.10	0.21	0.21	0.2	0.158	0.514	0.158	2.3	4.0	1.1
															Total					2.3	4.0	1.1										

**LIQUEFACTION ANALYSES BASED ON SPT DATA
1815 CORDILLERAS ROAD, EMERALD HILLS, CALIFORNIA**



SFB 659-5
2/5/2026 TC

a_{max} = 0.92 g ASCE 7-22 PGAm
Mw = 7.9

B-3
Ground Elevation = ft
Depth to Ground Water Table During Exploration = 21.5 ft = EL ft
Depth to Ground Water Table During Earthquake = 10 ft =EL ft
γ = 120 pcf
γ_{sat} = 125 pcf
Boring Diameter = 5 inch = 127 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation	Depth	Depth	N	σ _v Exploration	σ _v Earthquake	σ _v ' Exploration	σ _v ' Earthquake	Liner		C _S	C _B	C _E	C _N	N _{1,60}	FC	ΔN	N _{1,60,cs}	r _d	CSR	BI2014		CRR _{M=7.5,1 atm}	CRR	FS	Yoshimine 2006								
								C _R	Correction											MSF _{max}	MSF				K _σ	γ _{lim}	F _α	γ _{max}	ε _v	ΔH (ft)	Δs (in)		
ft	ft	m	blow/ft	psf	psf	psf	kPa		Y/N						%	blow/ft										%	ft	in					
	3.5	1.1	21	420.0	420.0	420.0	420.0	0.75	N	1.00	1.05	1.0	1.70	28	30	5	33	1.00	0.60	2.20	0.85	1.10	0.83	0.77						Soil Not Saturated	0.0		
	6.0	1.8	27	720.0	720.0	720.0	720.0	0.80	Y	1.27	1.05	1.0	1.51	44	30	5	49	1.00	0.60	2.20	0.85	1.10	2.00	1.87						Soil Not Saturated	0.0		
	16.0	4.9	19	1,920.0	1,950.0	1,920.0	1,575.6	0.95	N	1.00	1.05	1.0	1.04	20	30	5	25	0.97	0.72	1.73	0.91	1.05	0.29	0.28	0.4	0.087	0.222	0.087	1.9	4.0	0.9		
																														Total	1.9	4.0	0.9

**LIQUEFACTION ANALYSES BASED ON SPT DATA
1815 CORDILLERAS ROAD, EMERALD HILLS, CALIFORNIA**



SFB 659-5
2/5/2026 TC

a_{max} = 0.92 g ASCE 7-22 PGAm
Mw = 7.9

B-4
Ground Elevation = ft
Depth to Ground Water Table During Exploration = 18 ft = EL ft
Depth to Ground Water Table During Earthquake = 10 ft =EL ft
γ = 120 pcf
γ_{sat} = 125 pcf
Boring Diameter = 5 inch = 127 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation	Depth	Depth	N	σ _v Exploration	σ _v Earthquake	σ _v ' Exploration	σ _v ' Earthquake	Liner		C _S	C _B	C _E	C _N	N _{1,60}	FC	ΔN	N _{1,60,cs}	r _d	CSR	BI2014		K _σ	CRR _{M=7.5,1 atm}	CRR	FS	Yoshimine 2006			ε _v	ΔH (ft)	Δs (in)
								C _R	Correction											MSF _{max}	MSF					γ _{lim}	F _α	γ _{max}			
ft	ft	m	blow/ft	psf	psf	psf	kPa		Y/N						%	blow/ft										%	ft	in			
16.0	16.0	4.9	8	1,920.0	1,950.0	1,920.0	1,575.6	0.95	N	1.00	1.05	1.0	1.06	8	15	3	12	0.97	0.72	1.23	0.97	1.03	0.13	0.13	0.2	0.393	0.871	0.393	3.4	2.0	0.8
21.0	21.0	6.4	8	2,535.0	2,575.0	2,347.8	1,888.6	0.95	N	1.00	1.05	1.0	0.94	8	30	5	13	0.96	0.78	1.26	0.97	1.01	0.14	0.14	0.2	0.346	0.834	0.346	3.2	3.0	1.1
																										Total	3.3	5.0	2.0		

**LIQUEFACTION ANALYSES BASED ON SPT DATA
1815 CORDILLERAS ROAD, EMERALD HILLS, CALIFORNIA**



SFB 659-5
2/5/2026 TC

a_{max} = 0.92 g ASCE 7-22 PGAm
Mw = 7.9

B-6
Ground Elevation = ft
Depth to Ground Water Table During Exploration = 15 ft = EL ft
Depth to Ground Water Table During Earthquake = 15 ft =EL ft
γ = 120 pcf
γ_{sat} = 125 pcf
Boring Diameter = 5 inch = 127 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation	Depth	Depth	N	σ _v Exploration	σ _v Earthquake	σ _v ' Exploration	σ _v ' Earthquake	C _R	Liner Correction	C _S	C _B	C _E	C _N	N _{1,60}	FC	ΔN	N _{1,60,cs}	r _d	CSR	BI2014		K _σ	CRR _{M=7.5,1 atm}	CRR	FS	Yoshimine 2006					
																				MSF _{max}	MSF					γ _{lim}	F _α	γ _{max}	ε _v	ΔH (ft)	Δs (in)
ft	ft	m	blow/ft	psf	psf	psf	kPa		Y/N						%		blow/ft										%	ft	in		
	11.0	3.4	14	1,320.0	1,320.0	1,320.0	1,320.0	0.85	N	1.00	1.05	1.0	1.26	16	14	3	19	0.99	0.59	1.44	0.94	1.06	0.19	0.19			Soil Not Saturated		0.0		
	16.0	4.9	14	1,925.0	1,925.0	1,862.6	1,862.6	0.95	N	1.00	1.05	1.0	1.07	15	40	6	20	0.97	0.60	1.51	0.94	1.02	0.21	0.20	0.3	0.151	0.494	0.151	2.3	2.5	0.7
																										Total	2.3	2.5	0.7		

**LIQUEFACTION ANALYSES BASED ON SPT DATA
1815 CORDILLERAS ROAD, EMERALD HILLS, CALIFORNIA**



SFB 659-5
2/5/2026 TC

a_{max} = 0.92 g ASCE 7-22 PGAm
Mw = 7.9

B-6
Ground Elevation = ft
Depth to Ground Water Table During Exploration = 15 ft = EL ft
Depth to Ground Water Table During Earthquake = 0 ft =EL ft
γ = 120 pcf
γ_{sat} = 125 pcf
Boring Diameter = 5 inch = 127 mm
Rod Length Above Ground = 3 ft = 0.9 m

Elevation	Depth	Depth	N	σ _v Exploration	σ _v Earthquake	σ _v ' Exploration	σ _v ' Earthquake	Liner		C _S	C _B	C _E	C _N	N _{1,60}	FC	ΔN	N _{1,60,cs}	r _d	CSR	BI2014		K _σ	CRR _{M=7.5,1 atm}	CRR	FS	Yoshimine 2006			ε _v	ΔH (ft)	Δs (in)
								C _R	Correction											MSF _{max}	MSF					γ _{lim}	F _α	γ _{max}			
ft	ft	m	blow/ft	psf	psf	psf	kPa		Y/N						%	blow/ft										%	ft	in			
11.0	3.4	14	1,320.0	1,375.0	1,320.0	688.6	0.85	N	1.00	1.05	1.0	1.26	16	14	3	19	0.99	1.18	1.44	0.94	1.10	0.19	0.20	0.2	0.184	0.585	0.184	2.4	1.0	0.3	
16.0	4.9	14	1,925.0	2,000.0	1,862.6	1,001.6	0.95	N	1.00	1.05	1.0	1.07	15	40	6	20	0.97	1.16	1.51	0.94	1.10	0.21	0.22	0.2	0.151	0.494	0.151	2.3	2.5	0.7	
Total																															
2.3 3.5 1.0																															