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REVISED - GEOTECHNICAL DESIGN & DEBRIS FLOW HAZARD RISK ASSESSMENT REPORT

VETERANS VILLAGE HOUSING PROJECT

CE&G DOCUMENT: 220300.001

JUNE 24, 2022

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1.0 INTRODUCTION

1.1 GENERAL

Cal Engineering & Geology, Inc. (CE&G) has provided geotechnical and engineering geologic services for the Veterans Village Housing Project for APNs 078-273-15 & 078-272-06, located in Ben Lomond, California. The work has been completed to assess the site for potential debris flow hazards and to provide geotechnical design recommendations for the design and construction of multiple affordable housing unit foundations and new retaining walls.

1.2 INFORMATION PROVIDED

The following documents were provided and reviewed for this investigation and are included in Appendices A and B:

- Sherwood Design Engineers, 2022, Slope Analysis and Site Overview Figure for the Veterans Village Project in Ben Lomond, California: dated February 25, 2022.
- Santa Cruz County, 2022, Geologic Hazards Assessment for 8705 Highway 9, Ben Lomond, California: dated March 16, 2022.

1.3 PROJECT DESCRIPTION

The Veterans Village Housing Project is located at 8705 Highway 9, in Ben Lomond, California (Figure 1), and consists of converting the existing on-site Jaye's Timberlane Resort into affordable housing units for veterans (Veterans Village). The existing resort consists of 11 residential units, a garage, 2 dirt parking areas, a wood retaining wall, and a swimming pool (Figure 2). According to a Site Overview Figure (See Keynotes in Appendix-A) provided by Sherwood Design Engineers (Sherwood), as well as discussions with Sherwood and Swift Consulting Services, Inc., the conversion will involve:

- Remodeling the existing residential cabins (Keynotes 1 through 11)
- Replacing and extending an existing wooden retaining wall (Keynote 15)
- Demolishing the existing in-ground swimming pool (Keynote 16)
- Construction of 6 new single bedroom units (Keynotes 13, 14, and 17 through 20) and a new two-story four-unit structure (Keynote 12).

Foundation loads are anticipated to be relatively light and will likely be supported by concrete slabs-on-grade. Six new units (Keynotes 13, 14, and 17 through 20) will be located on the relatively flat areas of the site and will require new engineered foundations. We understand the existing units to be remodeled will not require geotechnical input as long as no modifications are made to the existing foundations (Appendix-A).

Site improvements currently include two new retaining walls that will likely be designed with soldier-piles and wood lagging along the base of the hillslope (See Figure 9). Based on the site topography and discussions with Swift Consulting Services, Inc., Nielsen Architects, and Sherwood Design Engineers, the retaining wall heights will likely range from about 4 to 6 feet depending on their locations. We understand the design engineer will decide on the final retaining wall setbacks.

1.4 BACKGROUND

Due to the site's location within steep hilly terrain, the County of Santa Cruz performed a Geologic Hazards Assessment (GHA) to determine whether geotechnical and/or geological constraints would need to be addressed before providing the project approval (Appendix-B). The assessment included a geologic data review and site reconnaissance by the County geologist and resulted in the following conclusions and project requirements:

- Portions of the planned development are located on an alluvial fan, near the mouth of an active channel, and are potentially at risk of debris flow impacts. A geologic evaluation is required to assess debris flow hazard risks for the planned development and to provide recommendations for mitigating any recognized hazards.

- A geotechnical design report is required for the proposed new structures and remodeled structures if foundation modifications are proposed.

1.5 PURPOSE AND SCOPE OF SERVICES

The investigation completed by CE&G was undertaken to evaluate the existing surface and subsurface conditions in the vicinity of the project area to assess debris flow hazard risks for the site and to develop geotechnical design recommendations for the design and construction of the planned improvements.

The scope of work completed for the geotechnical investigation and debris flow hazard risk assessment included:

1. Completion of a desktop study to identify and evaluate relevant geologic and geotechnical information available for the site and nearby sites, including published geologic maps, and unpublished geotechnical information in our files regarding the site and vicinity. The study also consisted of reviewing and analyzing existing Lidar

datasets to identify geomorphic features, including past landslide scars and source areas, and to delineate potential debris flow source areas.

2. Field geologic mapping to identify potential geologic and/or geotechnical hazards within the project areas, document existing site features, and structures, and confirm geomorphic features that were identified during desktop mapping of Lidar datasets.

3. Additional geologic reconnaissance to observe site conditions before subsurface explorations and to mark for Underground Service Alert (USA).

4. Subsurface exploration of six borings drilled in the areas of the planned improvements.

5. Excavation and logging of one test pit within the on-site alluvial fan deposits to evaluate potential past debris flow deposits.

6. Laboratory testing to determine key engineering index properties of selected earth materials.

7. Slope stability analysis of current slope conditions.

8. Development of geotechnical design recommendations.

9. Preparation of this geotechnical design and debris flow hazard risk assessment report.

2.0 SITE CONDITIONS AND DESCRIPTION

2.1 SITE DESCRIPTION

The project site is located in a moderately forested area of Ben Lomond, California, and is bounded by steep hilly terrain to the northwest, southwest, and southeast, and by Highway 9 to the northeast. The central and western portions of the site consist of steep hilly terrain with a northeast-trending stream channel, that actively flows to the northeast towards the gentle sloping portion of the site. The eastern portion of the project site gently slopes to the northeast towards Highway 9. The natural and fill slopes within the project area contain large redwood trees with a moderately brushy understory. On-site slopes that are not alongside the stream channel generally range from approximately 20° to 40°, whereas the slopes along the actively incising channel range from about 25° to 60°. These steeper slopes also contain small to large-sized trees, some of which have fallen due to slope instabilities along the channel. At the mouth of the channel is the apex of a gently sloping alluvial fan, which extends beyond highway 9 and makes up the gently sloping (0° to 10°) portions of the project site. The alluvial fan area has some large trees near the mouth of the channel and along the base of the slope but is mostly cleared of vegetation due to the existing development. The majority of the proposed development area sits within the gently sloping alluvial fan limits, except for four cabins (Keynotes 7 through 10), which are located on the adjacent hillside (Figure 2).

Elevations at the site range from approximately 375 to 580 feet above mean sea level. However, project improvements are planned in areas ranging in elevation from approximately 378 to 430 feet above sea level.

2.2 EXISTING STRUCTURES AND FEATURES

There are currently 6 existing wood cabins (Keynotes 1 through 6), a four-bedroom house (Keynote 11), and a 3-car garage (Keynote 12) located along the outer boundary of the alluvial fan portion of the project site. The central portion of the alluvial fan consists of an in-ground swimming pool (Keynote 16) and an open grass area, which overlays the development's septic leach field. These structures are accessed by an asphalt-paved driveway that loops around the pool and grass area and can be entered from Highway 9 at two locations. This asphalt-paved road also extends and switchbacks up the on-site hillslope to access four additional cabins (Keynotes 7 through 10) and to cross the on-site stream channel.

Key features of the project site are depicted on the attached Figure 2 and Appendix-A.

3.0 GEOLOGIC CONDITIONS

3.1 GEOLOGIC SETTING

The project site lies in the Santa Cruz Mountains, within the Coast Ranges geomorphic province of California (Figure 1). This province is characterized by northwest-southeast trending mountain ranges and intervening valleys such as that occupied by San Francisco Bay and San Lorenzo Valley. The Santa Cruz Mountains are one such range, marking a mountain-range scale regional uplift southwest of the San Andreas fault. This mountain range consists of steep terrain shaped by actively incising rivers and creeks, which commonly result in landsliding along the channel slopes. Landslide debris within the channels can result in debris flows, which are commonly deposited as they exit the channel, resulting in alluvial fan topography.

Some portions of the project site are located on steep slopes associated with the bounding hills of the San Lorenzo Valley, within the Santa Cruz Mountains. Other portions of the site are located on a gently sloping alluvial fan feature along the base of the slope and at the mouth of an active creek channel.

3.2 BEDROCK GEOLOGY

The geologic setting is shown on the Regional Geology Map, Figure 3.

The general vicinity of the site has been mapped several times, with geologic mapping having different emphases. Brabb and others (1997) mapped geologic materials and structures in detail for much of the Peninsula, including the site. According to Brabb and others (1997), the project site is underlain by a single bedrock unit, the middle Miocene Monterey formation, which generally consists of "medium- to thick-bedded and laminated olive-gray to light gray, semi-siliceous organic mudstone and sandy siltstone. Bedding orientations documented outside of the project boundary generally strike NE-SW and dip 15° to 30° to the southeast and downslope east into the local hillslopes (Brabb and others, 1997).

A more detailed discussion of the site-specific surface geology and subsurface conditions, local landslide scarps, and deposits are included in Section 4.0 and are shown in Figure 6, based on site-specific geologic mapping and subsurface exploration.

3.3 SEISMICITY

The project site is located within the greater San Francisco Bay Area which is recognized as one of the more seismically active regions of California. The seismic activity in this region results from the complex movements along the transform boundary between the Pacific

Plate and the North American Plate. Along this transform boundary, the Pacific Plate is slowly moving to the northwest relative to the more stable North American Plate at approximately 40 mm/yr in the Bay Area (Page, 1992). The differential movements between the two crustal plates caused the formation of a series of active fault systems within the transform boundary. The transform boundary between the two plates extends across a broad zone of the North American Plate within which right-lateral strike-slip faulting predominates. In this broad transform boundary, the San Andreas fault accommodates less than half of the average total relative plate motion. Much of the remainder of the motion in the South Bay Area is distributed across faults such as the San Gregorio, Monte Vista-Shannon, Sargent, Hayward, Calaveras, and Zayante-Vergeles fault zones.

Due to the site's location in the seismically active San Francisco Bay Area, they will likely experience strong ground shaking from a large (Moment Magnitude [Mw] 6.7) or greater earthquake along with one or more of the nearby active faults during the design lifetime of the project (WGCEP, 2003). It should be noted that the third Uniform California Earthquake Rupture Forecast (UCERF3) time-independent model supports a magnitude-dependent methodology that accounts for historic open intervals on faults without a date of last event constraint. The exact factors influencing differences between UCERF2 and UCERF3 vary throughout the region and depend on the evaluation of specific seismogenic sources. For example, with the 30 yr M≥6.7 probabilities, the most significant changes from UCERF2 are a threefold increase on the Calaveras fault and a threefold decrease on the San Jacinto fault. The model also suggests that the average time between 6.7 Mw or larger events has increased from every 4.8 years to every 6.3 years. The UCERF3 model indicates that M≥6.7 probabilities may not be representative of other hazard or loss measures and the applicability of UCERF3 should be evaluated on a case-by-case basis if required during site-specific ground motion analyses or at the behest of the regulatory agencies (WGCEP, 2014).

Some contributors to seismic risk for the project include the Monte Vista/Shannon, San Andreas, Hayward, Calaveras, Sargent, Zayante-Vergeles, and San Gregorio faults. A large magnitude earthquake on any of these fault systems has the potential to cause significant ground shaking in the vicinity of the site (Figure 4). The intensity of ground shaking that is likely to occur in the area is generally dependent upon the magnitude of the earthquake and the distance to the epicenter. Relevant seismic sources in the San Francisco Bay area and their distances from the site are summarized in Table 3-1.

Table-3-1. Distances to Selected Major Active Fault Surface Traces

Fault Name	Distance and Direction from Site to Mapped Surface Fault Traces
Zayante-Vergeles	3.8 km northeast
Butano	10.7 km northeast
San Andreas	12.8 km northeast
Sargent	13.9 km northeast
San Gregorio	15.8 km west
Monte Vista-Shannon	20.3 km northeast
Hayward (South)	37.4 km northeast
Calaveras	41.4 km northeast

3.4 GEOHAZARD MAPPING

3.4.1 Active Faulting and Fault Rupture

According to CGS (2018), a Holocene-active fault is defined as a fault that has had surface displacement within Holocene time (the last 11,700 years), and a pre-Holocene fault is defined as a fault whose recency of past movement is older than 11,700 years. The Alquist-Priolo Earthquake Fault Zoning Act only addresses the hazard of surface fault rupture for Holocene-active faults, although pre-Holocene-active faults may also have the potential for future surface fault rupture (CGS, 2018). The Alquist-Priolo Earthquake Fault Zoning Act's main purpose is to prevent the construction of buildings used for human occupancy on the surface trace of active faults. Before a new project is permitted, cities and counties require a geologic investigation to demonstrate that proposed buildings will not be constructed on active faults. According to the California Geological Survey (CGS), the project site is not located within an Alquist-Priolo Earthquake Fault Zone.

According to the United States Geological Survey's (USGS) Quaternary fault and fold database, there are no active faults mapped as crossing the project site (Figure 4).

The County of Santa Cruz Geologic Hazard Maps (accessed 2022) show no Fault Hazard Zones as crossing the project area and no fault hazard zones are established by the local jurisdictions.

The Ben Lomond fault has been mapped by Brabb and others (1997) as crossing the south-westernmost end of the site, where no improvements are currently planned (Figure 3). As far as tectonic and fault implications for the project, the fault has not been documented to show evidence suggestive of Holocene (11,700 years to present) ground rupture, and/or ground deformation (USGS, 2021-Archived report for Jennings ID#498). Santa Cruz County acknowledged the Ben Lomond fault in their GHA for the site and stated that "the Ben

Lomond fault is not considered to be active and therefore no fault hazard zone has been designated along this fault" (Appendix B).

3.4.2 Landsliding Hazards

The CGS has developed landslide inventory maps for parts of California, including areas within the project vicinity, which show recently mapped landslides by CGS and others from over the past 50 years. These mapped landslides for the project area heavily rely on mapping by Cooper-Clark and Associates (1975), who mapped landslide features for much of Santa Cruz County. The landslide data has been compiled in a way that presents landslide activity as either; Active/Historic, Dormant Young; Dormant Mature; Dormant Old/Relict; or Dormant Age Not Specified. Because some of the mapped landslides are based on aerial image and/or lidar mapping, the interpretation confidence of the slides is not certain for all slide and have been designated as either; definite; probable, or questionable. According to the CGS landslide inventory map, the project site is located within the boundaries of a very large "probable" landslide (see Figure 5 in Appendix-B).

The above-described "probable" landslide was assessed during Santa Cruz County's GHA for the site by reviewing Lidar imagery and aerial photos, as well as performing a limited site reconnaissance. Based on their findings, Santa Cruz County noted that the potential for large-scale landsliding at the site is judged to be low. The GHA also noted that some of the topography on the hill slope above the project site could be indicative of an older desiccated landslide.

According to mapping by Keefer (1989), earthquake-induced landslide features and ground failures due to the Loma Prieta earthquake were not recorded at the project site.

3.4.3 Debris Flow Hazards

Debris flows are a subset of landslides that generally occur during periods of intense rainfall or rapid snowmelt and usually start on hillsides or mountains and can then be channelized into streams or ravines. Debris flows can travel at speeds up to and exceeding 35 mph and can carry large items such as boulders, trees, and cars. If a debris flow enters a steep stream channel, it can travel for several miles, impacting areas distal from the initial landslide hazard. Areas recently burned by a forest fire are especially susceptible to debris flows, including the areas downslope and outside of the burned area (USGS, 1997).

Per the county's GHA assessment (Appendix B) there is a potential for small-scale landsliding along the flanks of the stream drainage to produce debris flows that flow down the axis of the channel into development areas. An alluvial fan has been formed where the stream emerges from the narrow, steep-sided channel on the western part of the property.

This alluvial fan may, and likely does, include older debris flow deposits. Structures sited on this fan could be at risk of debris flow impacts.

3.4.4 Liquefaction

Soil liquefaction is a phenomenon in which saturated, cohesionless soils (generally sands) lose their strength due to the build-up of excess pore water pressure during cyclic loading, such as that induced by earthquakes. Soils most susceptible to liquefaction are saturated clean, loose, fine-grained sands and silts. The primary factors affecting soil liquefaction include: 1) intensity and duration of seismic shaking; 2) soil type and relative density; 3) overburden pressure; and 4) depth to groundwater.

According to the County of Santa Cruz Liquefaction Hazard Area map, the easternmost corner of the project site is located within an area mapped as having high liquefaction potential (accessed 2022). However, the majority of the site is not located within a mapped liquefaction hazard area as designated by the County.

3.5 REGIONAL GROUNDWATER

The California Department of Water Resources identifies the area of the site as part of the Santa Margarita Groundwater Basin.

Groundwater within the hillslope areas encompassing the site is likely variable, with the water table commonly sloping downhill toward the closest drainage axis.

Site-specific groundwater data from our investigation is discussed in Section 4.3.4.

4.0 FIELD INVESTIGATIONS

4.1 SITE RECONNAISSANCE

CE&G performed field reconnaissance of the site on March 29, 2022, and again on April 06, 2022, in advance of performing subsurface explorations. Site reconnaissance consisted of photographic documentation of the project site, determining site access for drilling and backhoe equipment, and identifying and marking boring and test pit locations for clearance by Underground Service Alert (USA). A private utility locator was used to clear the exploration locations of existing utilities.

4.2 LIDAR GEOMORPHIC ANALYSIS AND GEOLOGIC MAPPING

Geologic site reconnaissance was completed on April 06, 2022, to document surface features and potential geologic hazards and/or geotechnical constraints within the project vicinity. Mapped features were documented on a LiDAR bare earth (hillshade) and topographic basemap, which was also used for our initial desktop geomorphic analysis. Our geologic and geomorphic interpretations are presented in Figure 2, which incorporates the LiDAR bare earth topographic dataset and regional geologic mapping (Figure 3). Some field observations made during our site reconnaissance consist of the following.

<u>Alluvial Fan Area:</u>

- The majority of the gently sloping portion of the site is developed with wooden cabins along the rim of the alluvial fan.
- There is a horseshoe-shaped, asphalt-paved road that loops around the alluvial fan, parallel to the housing layout, which intersects Highway 9 at two locations.
- The center of the alluvial fan area consists of a grass field with no trees and is the location of the existing subsurface leach field.
- Downslope of the grass area is an inground swimming pool with a concrete-lined area around the pool for lounging.
- Overhead utilities were observed along the asphalt paved roads within the project area.
- The upper limit of the alluvial fan area near the mouth of the channel (apex) consists of a flat gravel parking area along the asphalt-paved road. There is a storm drain inlet where the channel meets the gravel parking area that directs water from the stream channel beneath the road and to the northeast. No other storm drains were observed on the alluvial fan area.

Hillslopes and Stream Channel Area:

- The cut slope along the base of the hillside appears over-steepened (60° to 70°) and is partially supported by an existing wooden retaining wall. The cut exposes the road fill prism from the road above and the underlying colluvium.
- The hillslopes adjacent to the planned improvements (keynotes 7 through 10) appeared stable with minimal hummocky terrain suggested by older shallow landsliding.
- The hillslopes flanking the channel are heavily vegetated by mature redwood, oak, and madrone trees and shrubs.
- Existing cuts made into the hillslope along the access road range in height from about 3 to 9 feet and expose colluvium and some uppermost weathered bedrock in some areas. The cuts generally range from 40° to 70° and are near vertical in some locations. Evidence of landsliding or sloughing was not observed along the road cuts. Large diameter tree roots were observed throughout the cuts and appear to provide some stability within the cuts.
- Current shallow landsliding was observed on the slopes flanking the actively flowing stream channel. Observed landslide masses involved shallow colluvium, with possible involvement of the shallowest weathered bedrock. Some younger landslide scars expose Monterey Formation bedrock. Clasts within the landslide debris ranged from gravel to cobble-sized.
- Landslide masses along the channel flanks commonly block the stream channel, resulting in vertical incision into the mass to allow for creek flow. The landslide debris generally includes soil, gravel, and cobble-sized clasts from the underlying bedrock, and vegetation debris (e.g., branches, trunks, and brush.
- Areas without landsliding as well as older slide scarps along the channel slopes consist of small to large trees with some vines and ferns covering the soil surfaces.
- The channel bottom is generally lined with gravel and cobble-sized material.
- Downcutting in the channel bottom due to incision has resulted in rectangular cuts ranging from about 0.5 to 5 feet tall and 2 feet wide within the channel/landslide toe deposits. This downcutting erosion has left behind stream terraces consisting of landslides and possibly older stream deposits.
- The overall stream channel appears relatively U-shaped from its mouth at the apex of the alluvial fan up to the existing access road that crosses the channel. The channel becomes more V-shaped upslope of this area.
- The access road that crosses the stream is unpaved. There is a storm drain inlet just upstream of the road's intersection with the channel that allows flow beneath the road and out of an outfall downstream of the road. The outfall is approximately 18 inches in diameter and the water flows out to a 4-foot vertical drop in continues to flow downstream.

- Adjacent to the outfall is a wooden retaining wall that supports the road at its intersection with the channel. There is also a large redwood tree within the center of the channel, just downslope of the road and next to the outfall, which appears to add lateral support for the road. The tree did not appear to have physical damage to its trunk that could indicate past debris flow impacts.
- Seepage from the hillside just upslope of the road in this area was observed.
- Overall, the flat broad area of the road that intersects the channel likely slows down large flow events and may result in the deposition of larger debris.
- Much of the channel upslope of the intersecting road was inaccessible to steeper slopes as well as organic debris (e.g., trees, trunks, branches, etc.) within the stream channel.
- An existing foot trail along the upper portion of the channel allowed access to the uppermost limit of the channel. Which consisted of a relatively flat meadow-like area before transitioning to an asphalt paved road farther upslope. Stream incision in the meadow area appears to have been hand-dug to allow drainage from multiple storm drains along the asphalt-paved roads in the adjacent neighborhoods. Water was not observed in this part of the channel during the site visit.
- During the time of the visit, stream flow started between elevations of 460 and 490 along the channel, which likely indicates current groundwater table levels in the area.

Based on our site observations, the primary geotechnical considerations for the project consist of cut slope stability, erosion control, surface drainage, debris flow potential, and grading.

4.3 GEOTECHNICAL BORINGS

4.3.1 Scope of Explorations

Six geotechnical borings were drilled in the vicinity of the planned improvements as part of our investigation. Before drilling, CE&G marked planned boring locations and coordinated utility clearance through USA and a private utility locator. The approximate boring locations are shown on the attached Figure 2.

Four geotechnical borings were drilled by Cenozoic Exploration on April 11, 2022, using a SIMCO 2400 truck-mounted drill rig, equipped with 6-inch-diameter solid-flight augers. Two of the geotechnical borings were drilled using a hand-augur. The upper 3 to 5 feet of the deeper borings were hand-augered due to the presence of subsurface utility lines.

Upon completion, the borings were backfilled with cement grout in accordance with Santa Cruz County requirements. Drilling spoils were discretely spread on site.

4.3.2 Logging and Sampling

The materials encountered in the borings were logged in the field by a CE&G geologist. The soil was visually classified in the field, office, and laboratory according to the Unified Soil Classification System (USCS) in general accordance with ASTM D2487 and D2488.

During the drilling operations, soil and rock samples were obtained using the following sampling methods:

- California Modified (CM) Sampler; 3.0-inch outer diameter (O.D.), 2.5-inch inner diameter (I.D.) (ASTM D1586)
- Standard Penetration Test (SPT) Split Spoon Sampler; 2.0-inch O.D., 1.375-inch I.D. (ASTM D1586)

The CM and SPT samplers were driven 18 inches (unless otherwise noted on the boring logs) with a 140-pound hammer dropped from a height of 30 inches, using a cathead setup. The number of blows required to drive the samplers through each 6-inch interval was recorded for each sample and is included on the boring logs in Appendix C. The blow counts included on the boring logs are uncorrected and represent the field values.

Soil and rock samples obtained from the borings were packaged and sealed in the field to reduce the potential for moisture loss and disturbance. The samples were taken to CE&G's local laboratory, in Hayward, California, and Cooper Testing Labs, in Palo Alto, California, for further analysis and storage.

4.3.3 Soil and Bedrock Conditions Encountered

<u>Asphalt Pavement:</u> Approximately one inch of asphalt pavement was encountered at the surface of boring B-2.

<u>Artificial Fill:</u> Fill material was encountered in the upper portions of the six borings and generally consists of moist, medium dense, sandy lean clay to sandy lean clay with gravel and occasional silt. The encountered fill ranged from approximately 2 to 6.5 feet in thickness.

<u>Quaternary Colluvium:</u> Colluvium was encountered beneath the fill material in borings B-1, B-2, B-4, and possibly B-6. The encountered colluvial soils were generally logged as moist, medium dense, and hard sandy silt with gravel and sandy lean clay with gravel. The gravels within the colluvial soils consist of angular sandy siltstone fragments of Monterey Formation. The colluvial soils encountered in the borings were similar to colluvium that currently resides on the adjacent hillslopes, which also contain cobble-sized rock fragments. Thus, it is likely that the colluvium underlying the encountered fill material also

contains cobble-sized fragments. The encountered colluvium ranged from about 1.5 to 2 feet in thickness.

Quaternary Alluvial Fan Deposits: Alluvial soils were encountered beneath the fill materials in borings B-3 and B-4. The alluvium encountered in boring B-3, which was drilled near the center of the alluvial fan, about 200 feet from the mouth of the stream channel consists of moist, medium stiff to stiff, low to medium plasticity, sandy lean clay with varying amounts of angular to subangular pea-sized gravel. The alluvium encountered in boring B-4, which was drilled in the upslope limit of the alluvial fan near the stream channel mouth, consists of four differing layers of deposits to the maximum depth explored of 10.5 feet below the ground surface. The upper layer from approximately 3 to 4.5 feet below the ground surface consists of soft gravely lean to fat clay, which may be representative of stream channel deposits. Beneath this unit is sandy silt with angular to subrounded gravel which may be representative of debris flow deposits. The lower two layers that were encountered from about 6 to 10 feet below the ground surface consist of saturated clayey gravel with sand and well-graded gravel with clay and sand, which are indicative of stream channel deposits.

Monterey Formation Bedrock: Bedrock was encountered beneath colluvial soils in borings B-1 and B-2 at depths of about 8 feet and 3.5 feet below the ground surface, respectively. Much of the shallow encountered bedrock consists of silty sandstone, sandstone, and sandy siltstone. The encountered shallow sandstone and siltstone are fine-grained, highly to slightly weathered, extremely weak to very weak, and range from light yellowish brown to dark yellowish brown. Conglomerate bedrock was encountered beneath the sandstone in boring B-1 at approximately 22 feet below the ground surface. The conglomerate is slightly weathered, very weak, is matrix-supported with fine- to coarse-grained granitic sand, has little to no cementation, and has subrounded gravel up to 1.5 inches in diameter.

For a more detailed description of the subsurface materials encountered in the borings, the boring logs and laboratory test results are included in Appendices C and D. The materials encountered in borings B-1 and B-2 are also depicted on Cross-Section A-A’ in Figure 5.

4.3.4 Groundwater Conditions Encountered

Groundwater was encountered within alluvial fan deposits in boring B-4, at approximately 6 feet below the ground surface. Groundwater was not encountered in the other borings, which ranged in depth from 3 to 30 feet below the ground surface.

4.4 DEBRIS FLOW STUDY TEST PIT

4.4.1 Scope of Exploration

One exploratory test pit was excavated by Keith E. Dick Construction, Inc., on April 13, 2022, within the uppermost limits of the alluvial fan near the mouth of the on-site stream channel. The test pit was only excavated to about 5 feet below grade due to shallow groundwater conditions and was about 5 feet wide by 14 feet long. The test pit was excavated with a Case 580 Super M, 4WD rubber tire backhoe equipped with a 24-inch bucket. The intent of the test pit was to log the encountered soils to gather information regarding thicknesses of past debris flow events if present.

The materials encountered in the test pit were logged in the field by a CE&G certified engineering geologist. In addition, the Santa Cruz County geologist (Jeff Nolan) was on-site briefly, to observe the exposed test pit. Upon completion, the test pit was backfilled with the spoils, and bucket tamped.

4.4.2 Test Pit Findings

The orientation and composition of the encountered subsurface layers indicate that the test pit exposed a portion of the original stream channel and its southeastern bank before having been filled in with artificial fill to build up the existing road.

The majority of the encountered materials within the test pit consisted of two generations of artificial fill, which together extended to depths ranging from about 0.5 to 5 feet below grade. The fill gradually increases in thickness as the center of the older stream channel is approached. The encountered fill materials indicate that the original stream channel was initially filled in with un-engineered fill (Unit 1B on the test pit log) consisting of loose silty sand with gravel and debris (e.g., glass bottles, brick fragments, and metal piping). The hard silty and clayey fill (Unit 1a) overlaying Unit 1B fill appears to have been placed to fill the remaining portion of the stream channel as well as for the road base.

What appears to be the upper portion of the original stream channel bank, which is overlayed by Unit 1A fill, was documented as Unit 2 and consists of sandy silt with angular to subrounded gravel and cobbles. This unit ranges from about 1 to 1.5 feet in thickness and may be representative of a relatively gentle debris flow due to varying clast rounding and angularity. The older Unit 3 that underlies Unit 2 at the southeastern end of the trench consists of sandy silt with angular gravel and appears similar to colluvial deposits that cover the adjacent hill slopes and have been classified as such. The bottom-most unit encountered (Unit 4) is partially overlain by both Units 2 and 3 and consists of black, soft, gravelly lean to fat clay with a high percentage of organic matter (e.g., roots, woody debris, charcoal). Gravels within Unit 4 are subangular and were measured up to 0.75 inches in diameter. Unit 4 is likely representative of fluvial deposits that were later covered by Unit 3 and then Unit 2.

For a more detailed description of the subsurface materials encountered in the test pit, the test pit log is included in Appendix D.

4.5 GEOTECHNICAL LABORATORY TESTING

Testing was performed to obtain information concerning the qualitative and quantitative physical properties of the samples recovered during the subsurface exploration program. Tests were performed by Cooper Testing Laboratory in Palo Alto, California, and the CE&G Testing Laboratory in Hayward, California, in general conformance with applicable ASTM and Caltrans standards. The following tests were performed:

- Moisture Content and Dry Unit Weight (ASTM D2216)
- Atterberg Limits (ASTM D4318)
- Wash Over #200 Sieve (ASTM 1140)
- Unconsolidated-Undrained Triaxial Compression Test (ASTM D2850)
- Corrosion Caltrans Package includes:
    - Resistivity (Minimum) (Caltrans 643)
    - pH (Caltrans 643)
    - Chloride (Caltrans 422m)
    - Sulfate (Caltrans 417m)

The results of the laboratory testing program are presented in Appendix E.

6.0 GEOLOGIC & ENGINEERING ANALYSES

6.1 SLOPE STABILITY ANALYSIS

Slope stability analyses were conducted using the limit equilibrium software program SLIDE2 (Version 9.019, ROCSCIENCE). The Factors of Safety against slope failures were calculated using Spencer's Method ("entry and exit" search routine) with pore water pressures derived from piezometric data and typical water level elevation data. The Dry Season Model assumed deep water levels through Monterey formation at the major body of the slope. The Wet Season Model assumed that both colluvium and alluvium layers are saturated by water, in addition to the deep groundwater level assumed in the Dry Season Model. Spencer's Method is a two-dimensional, limit-equilibrium method that satisfies the force equilibrium of slices and overall moment equilibrium of the potential sliding mass. The inclination of side forces between vertical slices is assumed to be the same for all slices and is calculated along with the Factor of Safety (FS).

Spencer's Method utilizes the slope configuration, unit weight, and shear strength properties of the soil materials, and boundary and internal forces due to water pressures. After a potential failure surface has been assumed, the soil mass located above the failure surface is divided into a series of vertical slices. Forces acting on each slice include the slice weight, the pore pressure, the effective normal force on the base, the mobilized shear force (including both cohesion and friction), and the horizontal side forces due to earth pressures. The FS is calculated by determining the ratio of the resisting forces (cohesion and friction along the failure surface) to the driving forces about the center of the assumed failure surface.

The stability conditions that were considered for the model had a minimum slip surface depth of 2 feet to exclude erosion-related shallow failure surfaces. Both dry season model and wet season model conditions were considered in the model. The factors of safety for the slope stability analysis are outlined in Table 6-1, and output results are included in Appendix F. The outputs present the model geometry, material properties, and the estimated phreatic surface.

Pseudo-static slope stability analyses have been completed using a pseudo-static coefficient determined according to the methods described in the 2008 California Geologic Survey document SP117A titled, "Guidelines for Evaluating and Mitigating Seismic Hazards in California." The method is commonly used for evaluating the seismic slope stability of slopes. As part of the method, the mean moment magnitude and peak ground acceleration are used in the selection of the pseudo-static seismic coefficient. These parameters are determined from a probabilistic seismic hazard deaggregation to determine the peak ground acceleration and moment magnitude for the earthquake event having a 475-year return period. The USGS Unified Seismic Hazard Tool resulted in an estimated peak ground acceleration of 0.51 g. A seismic load coefficient of 0.20 was calculated using a simplified method developed by Bray and others (1998) using the following formula:

$$k_{eq} = f_{eq} * MHA_r$$

where MHA_r is the maximum horizontal acceleration at the site for a soft rock site condition and f_{eq} is a factor related to the seismicity of the site. The factor related to seismicity, f_{eq} , was determined by using Blake and others (2002) curves for ranges of magnitude and distance for a displacement of 15 cm.

The specific factors of safety for seismic slope stability analysis are outlined in Table 6-1 and output results are included in Appendix F. The outputs present the model geometry, material properties, and the estimated phreatic surface.

Table 6-1. Factor of Safety of Slope Stability Analysis

Analysis Condition	Cross Section A-A’
Dry Season - Static	1.79
Dry Season - Seismic	1.16
Wet Season - Static	**0.97**
Wet Season - Seismic	**0.68**

The results in Table 6-1 indicate that the slopes are stable under static and seismic conditions with the assumed dry season groundwater levels. However, during the wet season, the slopes are considered unstable under both static and considered seismic conditions. To analyze slope failures in the wet season, model results with filtered failure surfaces are plotted and included in Appendix F, where failure surfaces with FS<1.3 under the static case and failure surfaces with FS<1.1 under the seismic case are selected and shown. After a review of the locations of those surfaces, it is noted that the potential slope failures occur at the toe of the analyzed slope, where previous cuts are present.

6.2 DEBRIS FLOW HAZARD RISK ASSESSMENT

An assessment of the on-site stream channel was performed to evaluate its potential for producing debris flows that may adversely impact the planned development, which will be located on the alluvial fan, near the mouth of the active channel. Our assessment consisted of the following:

- Determined the approximate watershed boundary for the on-site channel and evaluated the stream channel geometry (Figures 6, 7, and 8).
- Geological mapping of accessible portions of the stream channel.
- Excavated one test pit (TP-1) to log shallow alluvial deposits within the alluvial fan near the mouth of the channel (see Section 4.4).
- Calculated potential debris flow volumes that may be entrained during a single debris flow event.
- Calculate the velocity and impact force of the potential debris flow at the location of prosed mitigations.

6.2.1 Stream Channel Watershed and Geometry

The approximate watershed boundary for the stream channel is shown in Figure 6 and has an approximate area of 61 acres. The approximate debris source area (~2.9 acres) for the channel is also outlined in Figure 6 and is further described in the next section.

We calculated two average channel gradients from a stream channel profile, which is shown in Figure 7. As previously discussed, the stream channel gradient is relatively gentle (~3° to 5°) with occasional vertical steps between the mouth of the channel and where the dirt access road intersects the channel. The average gradient upslope of the access road is approximately 19°.

6.2.2 Geologic Mapping Observations

The hillslopes along the on-site stream channel are lined with active landslides that occasionally add soil, rocks (e.g., gravel and cobbles), and organic debris (e.g., logs, tree stumps, branches, etc.) into the actively flowing channel (Figure 2). The toes of the landslides that have obstructed the stream channel have been incised by about 1 to 4 feet, leaving behind fill terraces along parts of the channel. Aside from oversteepening of the channel flanks, these landslides likely most often occur during higher flow events that erode the toes of the adjacent slopes, resulting in a loss of lateral support and eventually, slope failure. Because the stream is actively flowing, small debris within the channel is often transported and cleared from the stream bottom.

Organic debris within the channel greatly increases in volume upslope of where the dirt access road crosses the channel. This portion of the channel exhibits a decrease in flow and has larger trees that have fallen into the channel. The minimal water that was observed was flowing in small, incised channels beneath the organic debris. This debris may have the potential of damming up the channel during future high flow events.

Once the upper western limit of the channel was reached at an elevation of about 548 feet, there is a relatively flat meadow-like geomorphic break before transitioning to a gently sloping asphalt-paved area within a residential neighborhood (Figure 6). The asphalt-paved area consists of various storm drain systems that lead to the upper limit of the stream channel of interest. Although the storm drain system would add water to the channel during storm events, major volumes of debris would from this area are unlikely to enter the main stream channel. Due to these observations, as well as the site topography, a boundary of potential debris source areas for the channel has been outlined in Figure 6.

6.2.3 Potential Debris Flow Volumes

Based on our field observations and topographic morphology of the area, shallow landsliding along the channel flanks appears to be the primary source of debris within the stream channel and has the potential for supplying future debris flows. The observed landslide masses within the channel generally contain what appears to be about 50% of the overall original landslide volumes. The remaining portions of the landslide masses that have not been washed away generally still sit on the lower portions of the slope. Each of these mapped landslides represents a volume of material that has instantly been added to the channel in its past. It is also likely that each of these slides occurred at separate times from one another. Thus, the volume of one of the largest mapped landslides along the channel was calculated and used as the volume of a potential debris flow that may occur within the channel for a single event.

The landslide volume was calculated by multiplying the surface area of the mapped slide by the estimated thickness of the landslide, based on field observations, and using topography to estimate the pre-slide surface conditions. With an approximate landslide area of 1,993 square feet and an average thickness of 4 feet, the volume was calculated at about 295 cubic yards. Since approximately 50% of landslide masses generally remain on the lower portions of the slope, it is assumed that about 50% of 295 cubic yards (~148 cubic yards) of material is likely to be entrained during a single debris flow event.

6.2.4 Potential Debris Flow Velocity & Impact Pressure

The morphology, physical setting, and fan deposits of the project area suggest the potential for previous and future debris flow events. Estimating the potential velocity and impact of debris flows can require complex calculations that require detail observations of past events (Prochaska et.al., 2008). We have assessed that there is not enough information available to back-calculate events for application in estimating the velocity and impact. We have thus, adopted an empirical approach based on two published relationships that use input parameters of potential flow height in a channel (assumed depth of channel filled with debris) and the channel angel the flow will travel (Lo, 2000 & Prochaska et.al., 2008).

Velocity Equation Parameters:

- Velocity-V (meters/second), modified from and to feet/second
- Flow height-h (meters), modified from and to feet
- Slope-S (sine of channel angle)

We chose to use the maximum gradient or angle of the channel has been measured as 19° (Section 6.2.1 and Appendix G - Figure G1). We reviewed the cross-sectional morphology of the stream channel for breaks in slope or terraces, that may suggest the height of previous floods or flows and measured the height from channel base to the slope breaks to estimate flow height. The flow height was averaged from three points/cross sections resulting in an estimate potential flow height of 4.33 feet (Appendix G).

We estimate the potential debris flow velocity to be 11.3 feet/second and 17.9 feet per second; using the Lo, 2000 and Prochaska et.al., 2008, respective equations.

The impact pressure of a debris flow, when it collides with a barrier requires detailed calculation based upon the exact planned structure and the flow dynamics of the debris flow itself. Again, we have decided to provide an estimate impact pressure based on empirically derived relationships. Lo (2000) provides tabulated values of impact pressures correlated to the flow height/depth. For "small scale debris flows", the estimated impact pressure is stated to be 1,150 psf +/-100 (55kPa +/-5) (Lo, 2000-Table 9).

7.0 CONCLUSIONS AND DISCUSSION

7.1 GENERAL

The residential units are planned to be constructed in the approximate locations shown in Appendix-A. It is our professional opinion that the planned residential structures within the flat-lying area of the site (Keynotes 1 through 6, 11 through 14, and 17 through 20) may be designed to be supported on conventional isolated spread or continuous footings provided the recommendations presented in this report are followed. We understand the existing foundations for the four units (Keynotes 7 through 10) that are located on the hillslope will not be modified. However, if modifications occur, they may be supported on pier and grade beam foundations that are extended into the underlying bedrock.

The locations of the recommended retaining and staggered debris flow impact walls are shown in Figure 9. It is our professional opinion that the base of the on-site hillslope should be retained by a soldier-pile and wood lagging retaining wall at two locations. In addition, a staggered debris flow impact wall is recommended near the mouth of the on-site channel and may also consist of soldier piles and wood lagging.

Geotechnical considerations to note during project design and construction are:

- Drillability and Excavatability of encountered materials;
- Seismic design considerations across the project site;
- Settlement of the existing fill and alluvial soils;
- Landsliding along the on-site hillslopes;
- Debris Flows; and
- Corrosivity of on-site soils.

Detailed recommendations for these and other geotechnical aspects of the proposed improvements are presented in the following sections of this report. Our evaluations and recommendations are based upon the previously discussed development information provided to us and information obtained during this investigation. The following recommendations may need to be modified if there are any changes in the proposed improvements.

7.2 DRILLABILITY AND EXCAVATABILITY

Subsurface exploration was completed using solid-flight augers and did not encounter auger refusal to the depths explored of about 30 feet. Based on the subsurface exploration, we anticipate conventional earthwork and excavation equipment may be used for construction.

7.3 SEISMIC CONSIDERATIONS

Large magnitude earthquakes and strong ground shaking are likely to affect the project area within the design lifetime of the proposed improvements. Peak ground shaking parameters are presented below in Section 8.2.3 and should be considered in the design of the proposed improvements. Local ground-modifying effects of high-intensity ground shaking are considered secondary seismic effects. Our review of these processes is presented below.

- In our judgment, the potential for fault ground rupture or coseismic faulting to significantly affect the proposed improvements is low.
- In our judgment, the potential for ridgetop fissuring, ridgetop shattering, ridgetop spreading or other seismically induced ground deformation to significantly affect the proposed improvements is low.
- In our judgment, the potential for soil liquefaction to significantly affect the proposed project is low.

7.4 SETTLEMENT

Based on our boring and laboratory data, it appears that the upper 5 feet of fill and alluvial soils encountered in boring B-3 may be moderately compressible under the anticipated loads. To minimize potential settlement and structural distress due to this compressible layer, we recommend that the upper 18 inches of the site soils be recompacted in areas where new fill and other improvements are planned as described in our recommendations.

7.5 LANDSLIDING

As described above, no evidence of deep-seated landsliding was observed at the site. In our judgment, the potential for deep-seated landsliding (involving bedrock) to adversely affect the site improvements is low under static seismic conditions.

As described in Section 6.1, shallow landsliding of fill prisms, colluvium, and uppermost weathered bedrock, under static and seismic conditions, is likely to occur during the wet season if not properly supported by a retaining wall at the base of the slope. We judge the potential for shallow-seated landsliding (under static and seismic conditions) to adversely affect the site improvements to be low, provided site improvements (including the recommended retaining wall) are appropriately designed and constructed and surface runoff is appropriately managed.

7.6 DEBRIS FLOWS

Based on our site observations and debris flow hazard risk assessment, it is our professional opinion that the planned development, which will be located on the alluvial fan, near the mouth of the active channel is at risk of being impacted by future debris flow events. It is estimated that isolated debris flow events may entrain volumes of soil and rock of about 148 cubic yards along with other organic debris. At the stream outlet/ location of potential debris flow mitigation measures we estimate a velocity between 11.3 and 17.9 ft/sec, and a impact pressure of 1,150 psf +/-100 (55kPa +/-5).

The potential for debris flows should be addressed per our recommendations in Section 8.4 below.

7.7 CORROSION

Corrosion testing was performed on one soil sample collected from boring B-1 in general accordance with Caltrans methods. Testing results are presented below:

Table 7-1. Corrosion Testing Results

Boring (sample depth in feet)	Resistivity (Ohm-cm)	Chloride (mg/kg)	Sulfate (mg/kg)	pH
B-1 (6.5)	3,906	5	12	4.8

Caltrans Corrosion Guidelines, May 2021 (Caltrans, 2021), identifies a site to be corrosive for structural elements (metals and/or concrete) if one or more of the following conditions exist:

- Chloride concentration is 500 ppm or greater;
- Sulfate concentration is 1500 ppm or greater;
- pH is 5.5 or less.

A minimum resistivity value for soil and/or water less than 1000 ohm-cm indicates the presence of high quantities of soluble salts and a higher propensity for corrosion. Based on the results of the laboratory testing performed, the soil sample collected from boring B-1 has Chloride and Sulfate values that do not meet the Caltrans criteria for a corrosive site but do have a pH value that meets the Caltrans criteria for a corrosive site. The resistivity value of the tested soil sample is above the 1000 ohm-cm threshold.

According to ACI 318 Section 4.3, Table 4.3.1:

- Sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible (no restrictions on concrete type)
- Water-soluble chloride content of less than 500 ppm is generally considered non-corrosive to concrete.

Based on the results of the laboratory testing performed, the soil sample tested had values for Sulfate and Chloride that do not meet ACI criteria and are considered non-corrosive to concrete.

Corrosion test results should be considered preliminary and are an indicator of potential soil corrosivity for the sample tested. Other soils found onsite may be more, less, or of similar corrosive nature. Our scope of services does not include corrosion engineering; therefore, a detailed analysis of the corrosion tests is not included.

8.0 RECOMMENDATIONS

Detailed recommendations for the geotechnical aspects of the proposed improvements are presented in the subsequent sections of this report. Our evaluations and recommendations are based upon the previously discussed information that has been provided to us. The following recommendations may need to be modified if there are any changes in the proposed improvements, their layout or location, or the proposed grading.

8.1 DESIGN GROUNDWATER

Groundwater was encountered in boring B-4 at approximately 6 feet below the ground surface near the mouth of the channel and within the alluvial fan deposits. Groundwater was not encountered in the other borings. Groundwater levels are likely to fluctuate depending on rainfall. Groundwater is unlikely to be encountered during excavation of colluvial and alluvial soils for the installation of the residential unit foundations. However, groundwater may be encountered in deeper excavations for installation of underground utilities and pier drilling and installation. We recommend a design groundwater level of 6 feet below the ground surface within the flat-lying soils on the site. Groundwater encounters should be accounted for in the design of temporary shoring by the contractor. Groundwater should also be anticipated to be encountered during the construction of drilled piers.

8.2 FOUNDATIONS

We recommend foundations for the planned new residential units (Keynotes 12, 13, 14, and 17 through 20 [Appendix-A]) within the relatively flat portions of the site be supported on isolated spread or continuous strip footings designed and constructed in accordance with the following recommendations.

The footings should be embedded at least 18 inches below the finished pad grade or the lowest adjacent finish grade, whichever provides deeper embedment. Continuous and isolated footings may be designed using a net allowable soil bearing pressure of 2,500 pounds per square foot (psf) for dead plus live loads. This value may be increased by one-third when considering short-term loads such as wind and seismic forces.

Concrete should only be placed in excavations that are clean and free of loose soils and debris. Foundation excavations should be maintained in a moist condition before the placement of concrete. A member of our staff should observe foundation excavations to verify that adequate foundation bearing soils have been reached and to advise regarding moisture treatment of the underlying soils.

Lateral loads may be resisted by a combination of friction between the bottom of foundations and the supporting subgrade, and by passive resistance acting against the vertical sides of the foundations. An ultimate friction coefficient of 0.35 may be used for friction between the foundations and supporting subgrade. Ultimate passive resistance equal to an equivalent fluid weight of 350 pcf acting against the embedded sides of the foundations may be used for design purposes. These values may be used in combination without reduction. The passive pressure can be assumed to act starting at the top of the lowest adjacent grade in paved areas. In unpaved areas, the passive pressure can be assumed to act starting at a depth of 1 foot below grade. It should be noted that the passive resistance value discussed above is only applicable where the concrete is placed directly against undisturbed soil or engineered fills. Voids created by the use of forms should be backfilled with soil compacted to the requirements provided in this report or with concrete.

To maintain foundation support, utility trenches located near footings should be deepened so that the bearing surfaces are below an imaginary plane having an inclination of 1½:1 (horizontal to vertical). This imaginary plane shall be drawn extending upward from the bottom edge of the adjacent utility trench.

8.2.1 Concrete Slabs-On-Grade

We recommend that the slabs be a minimum of 5 inches thick. The slabs should include minimum reinforcement of #3 bars in both directions at 12-inch centers or #4 bars in both directions at 18-inch centers. The steel should be placed in the middle of the slab and should be held in place by dobie blocks or other suitable means. Actual dimensions and reinforcement should be determined by the project Structural Engineer.

Interior concrete slabs-on-grade that are covered with moisture-sensitive floor coverings, or where minimal vapor transmission through the slab is desirable, should be underlain by at least 4 inches of capillary break material such as a free-draining, clean drain rock or 3/8-inch pea gravel. A plastic membrane should be placed over the capillary break material. The membrane should be a high-quality polymer at least 15-mils thick that is resistant to puncture during slab construction. The membrane should also meet the specifications outlined in ASTM E 1745 latest revisions -*Standard Specification of Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*. To minimize damage to the barrier during concrete placement, a 2-inch sand layer may be placed above the plastic vapor barrier.

A lower water-cement ratio (0.45 to 0.50) for the concrete will help to reduce the permeability of the floor slab, and thus reduce the moisture transmission. It should be understood that the required plastic membrane is not intended to waterproof the concrete slab floor. If waterproofing is desired, the project designers should be contacted.

The use of concrete slabs-on-grade is also anticipated if exterior patios, walkways, etc., are to be added. Soil subgrade should be maintained in a moist condition before pouring the concrete slab.

8.2.2 Pier-and-Grade Beam Foundations

If existing foundations for Keynotes 7, 8, 9, and 10 (Appendix-A) are modified, they may be supported on pier and grade beam foundations. The foundation piers for the new units (if needed) should be designed as drilled cast-in-place concrete piers that derive their load-carrying capacity from frictional resistance between the pier shaft and the surrounding soil materials.

The recommended design parameters for a pier and grade beam foundation system are as follows.

Minimum pier diameter:	16 inches.
Allowable skin friction:	500 psf within competent soil below a depth of 3 feet below the finished grade.
Pier spacing:	Minimum three diameters on center.
Minimum reinforcing steel:	Four #4 bars w/ #3 closed ties.
Minimum pier depths:	8 feet deep. Piers should be embedded a minimum of 6 feet into the underlying bedrock.

The final design of pier depths and spacing should be determined by the project structural engineer. Perimeter piers and piers supporting shear walls should be structurally connected with grade beams and tie beams. The grade beams and tie beams should be designed by the project structural engineer. Grade beam and tie-beam dimensions and steel reinforcing requirements should be determined based on the design structural loads.

At a minimum, the grade beams and tie beams should be reinforced with no less than four #4 bars, two near the top and two near the bottom. Care should be taken to design the grade beams and tie beams such that they do not interfere with the air cross-flow ventilation beneath the residence.

The bottoms of the foundation pier holes should be dry and free of loose cuttings and debris before the installation of the reinforcing steel and concrete. This shall be done to the satisfaction of the engineer or geologist from Cal Engineering & Geology, Inc. observing the drilling operations. The concrete should be placed carefully in the pier holes so that over-pouring of the piers (mushrooming at the top) does not occur and the concrete does not have a free-fall drop over 6 feet.

Free groundwater is not anticipated to be encountered in pier excavations. However, the contractor must be prepared to drill and place the steel and concrete for the foundation piers on the same day, should adverse groundwater conditions be encountered during construction. Water should not be allowed to remain in a drilled pier hole overnight. Should this occur, it will be necessary for the contractor to enlarge the hole to a wider diameter and/or a greater depth to the satisfaction of the engineer or geologist from our office who is observing the drilling operation.

Our firm should be commissioned to review the foundation plans to determine if our recommendations are incorporated into the design. Our representative should observe the foundation excavations to determine if the excavations extend into suitable bearing materials and that they are cleaned of all soil and debris before pouring concrete.

8.2.3 Seismic Design Parameters

Due to the proximity of the site to the numerous active fault systems which traverse the greater San Francisco Bay Area, the project site will likely be subjected to the effects of a major earthquake during the design life of the proposed improvements. The effects are likely to consist of significant ground accelerations. These ground movements may cause damage to the proposed improvements. The following seismic design parameters in Table 7-1 are from Chapter 16 of the 2019 California Building Code and ASCE 7-16 for Site Class D soil (California Building Code, 2019).

Table 7-1. 2019 CBC Seismic Design Parameters

Item	Design Value	
Site Soil Class	D	
MCE_R Spectral Acceleration (g)	$S_s = 1.866$	$S_1 = 0.736$
Site Coefficients	Fa = 1.0	Fv = 1.7
MCER Mapped Spectral Acceleration Adjusted for Site Class Effects (g)	$S_{MS} = 1.866$	$S_{M1} = 1.25$
Design Spectral Acceleration (g)	$S_{DS} = 1.244$	$S_{D1} = 0.834$
Seismic Design Risk Category	III	
PGA	0.797	
PGA_M	0.876	

Note: The above parameters assume the structure is not seismically isolated and does not incorporate a damping system. If this is not the case, a ground motion hazard analysis may be required. Reference: https://asce7hazardtool.online/.

8.3 RETAINING WALLS

8.3.1 Lateral Earth Pressures

Two retaining wall are currently recommended along the base of the hillslope adjacent to proposed units 6, 5, 13, and 14 and is shown on Figure 9. This area currently consists of a cut slope with parts of the slope retained by tree logs. Based on the topography and subsurface conditions, we estimate wall heights ranging from 4 to 6 feet and recommend a steel soldier beam and lagging retaining wall to replace the existing wood retaining wall for support of the adjacent cut slope. For retaining walls less than 4 feet in height, timber soldier beams may be considered.

We recommend the design utilize the following parameters:

- Active equivalent fluid pressure of **60 pcf** acting over the full height of the retaining wall, **assuming a 2:1 slope above the wall**. The design height of the wall should be assumed to be the final exposed height plus a minimum of 1 foot of embedment;

- Active equivalent fluid pressure of **45 pcf** acting over the full height of the retaining wall, **assuming level backfill behind the wall**. The design height of the wall should be assumed to be the final exposed height plus a minimum of 1 foot of embedment;

- A **seismic equivalent fluid pressure of 22 pcf** acting over the full height of the retaining wall. Seismic loading should be applied in addition to the above active equivalent fluid pressure ignoring traffic live load.

- A passive equivalent fluid pressure of 350 pcf starting 2 feet below the exposed wall height acting over two pier diameters;

- Minimum pile diameter of 16 inches;

- Minimum pile spacing of three diameters on center;

- Minimum pile depth of 8 feet into competent materials.

Active and seismic equivalent fluid pressures assume the retaining wall will be backfilled using on-site materials excavated during soldier pile drilling operations or select import backfill with a minimum friction angle of 34 degrees and as outlined in Section 6.1.

8.3.2 Retaining Wall Drainage

Drainage for the retaining structure may be provided by a subdrain system behind the retaining wall. The system should consist of a 4-inch minimum diameter perforated pipe, placed with the perforations placed facing downward, and embedded in a 12-inch-wide layer of Caltrans Class 2 permeable material. As an alternative to the Class 2 Permeable drainage material, clean coarse gravel or drain rock may be used. If coarse gravel or drain rock is selected as a drainage material it should be separated from all adjacent soil by an engineering filter fabric such as Mirafi 140N, or a similar geotextile. The subdrain pipe should be connected to a free-draining outlet. Native clayey soil should be used for the upper 2 feet of wall backfill to cap the drainage material from infiltrating surface water.

8.3.3 Construction Considerations

The bottoms of soldier piles should be dry and free of loose cuttings and debris before the installation of the steel beams and concrete. This shall be done to the satisfaction of the engineer or geologist from Cal Engineering & Geology, Inc. who observes the drilling operations. The concrete should be placed carefully in the drilled holes so that over-pouring of the piles (mushrooming at the top) does not occur and the concrete does not have a free-fall drop over 6 feet.

Free groundwater was encountered in boring B-4 at approximately 6 feet below grade within the alluvial fan deposits. The drilling contractor should be prepared to drill and place steel and concrete for the piles on the same day. Under no circumstances shall water be allowed to remain in a drilled pile hole overnight. Should this occur, it will be necessary for the contractor to enlarge the hole to a wider diameter and/or a greater depth to the satisfaction of the engineer or geologist from our office who is observing the drilling operation.

8.4 DEBRIS FLOW IMPACT WALL

A staggered debris flow impact wall is recommended within the stream channel area, approximately 5 to 10 feet upstream of the channel mouth and is shown on Figure 9. The purpose of the impact wall would be to slow down an initial debris flow impact and block large debris from being transported out of the channel area. We are recommending a staggered wall over a single continuous wall to allow future tenants access to the channel area.

It is recommended that the staggered impact walls each be approximately 9 feet in length and overlap by approximately 6 feet on center. The walls should consist of soldier piles and wood lagging. The bottom lagging should extend approximately 1-foot over the top of the actively flowing stream so that it allows natural flow.

We recommend the design utilize the following parameters:

- Designed to withstand an impact pressure of 1,150 psf +/-100 (55kPa +/-5).
- Minimum wall height of 4 feet
- Maximum space of 10 feet between the two walls
- Minimum pile diameter of 12 inches;
- Minimum of 3 piles per wall;
- Minimum pile depth of 6 feet;
- Each wall should have piles on both sides of the stream channel.

Routine maintenance should be performed regularly to remove debris build-up that may occur behind the wall. The impact wall should also be inspected by a licensed engineer after debris flow events to evaluate and document the wall conditions and determine whether repairs and/or replacements are necessary for proper functionality. The property owner should be responsible for all debris flow impact wall maintenance.

8.5 CONCRETE SLABS-ON-GRADE - EXTERIOR

The use of concrete slabs-on-grade is also anticipated for exterior walkways. Soil subgrade shall be maintained in a moist condition before pouring the concrete slab. A lower water-cement ratio (0.45 to 0.50) for the concrete will help to reduce the permeability of the floor slab, and thus reduce the moisture transmission. It should be understood that the required plastic membrane is not intended to waterproof the concrete slab floor. If waterproofing is desired, the project designers should be contacted.

To reduce the potential for cracking of the concrete slabs, we recommend that the slabs be a minimum of 5 inches thick. The slabs should include minimum reinforcement of #3 bars in both directions at 12-inch centers or #4 bars in both directions at 18-inch centers. The steel should be placed in the middle of the slab and should be held in place by dobie blocks or other suitable means. Actual dimensions and reinforcement shall be determined by the project Structural Engineer.

Even with the steel reinforcement and base rock, it should be recognized that some cracking and differential movement of the slabs will likely occur and should be expected. Exterior concrete slabs-on-grade shall be cast free from adjacent footings or other non-heaving edge restraints. This may be accomplished by using a strip of 1/2-inch asphalt-impregnated felt divider material between the slab edges and the adjacent structure.

Construction and/or control joints should be provided in concrete slabs as recommended by the structural engineer.

8.6 SURFACE AND SUBSURFACE DRAINAGE

Engineering design of grading and drainage at the site is the responsibility of the project Civil Engineer. We recommend that the following points be considered by the project Civil Engineer and incorporated into the project plans where appropriate.

Generally, surface drainage should be directed away from building foundations, concrete slabs-on-grade, and pavements and directed towards suitable discharge locations. Ponding of surface water should be avoided by establishing positive drainage away from all improvements. Collected surface water and discharge from roof downspouts should be discharged into a pipe or towards drainage structures and the water carried to a suitable discharge point.

8.7 EARTHWORK

8.7.1 Clearing and Stripping

Site clearing should include removal of the existing wood log retaining wall, swimming pool, and structure foundations, deleterious materials, debris, obstructions, stumps, and primary roots of trees and brush that are designated for removal. Roots about 1 inch or larger in diameter or about 3 feet or longer should be removed. Depressions, voids, and holes that extend below the proposed finish grade should be cleaned and backfilled with engineered fill compacted to the recommendations in this report.

Residential units 17, 18, 19, and 20 will be located in the area of an existing in-ground swimming pool (Appendix-A). As part of the clearing and stripping phase of earthwork, the on-site swimming pool bottom will need to be cracked in multiple locations to allow drainage and then backfilled with engineered fill compacted to the recommendations in this report.

8.7.2 Excavations

Excavations for this project will include subexcavation of existing and fill and alluvial soils, general cuts to achieve design grades, trenching for underground utilities, and foundation excavations.

Excavations should be constructed in accordance with the current CAL-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, are the responsibility of the contractor.

8.7.3 Subgrade Preparation

Subgrade soil in areas to receive slabs-on-grade or pavements should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted to the recommendations given in the "Engineered Fill Placement and Compaction" section of this report. Prepared soil subgrades should be non-yielding when proof-rolled by a fully loaded water truck or equipment of similar weight. After moisture conditioning, subgrade soils should not be allowed to dry out.

Subgrade preparation should extend a minimum of 3 feet beyond the outermost limits of the proposed foundations and pavements. For exterior flatwork not connected to structures and for pavement areas, subgrade preparation should extend at least 3 feet beyond the limits of exterior flatwork or pavements. After the subgrades have been prepared, the areas may be raised to design grades by the placement of engineered fill.

Soil with moisture content above optimum value should be anticipated during and shortly after rainy seasons. Where unstable, wet, or soft soil is encountered, the soil will require processing before compaction can be achieved. When the construction schedule does not allow for air-drying, other means such as lime or cement treatment of the soil or excavation and replacement with suitable material may be considered. Geotextile fabrics may also be used to help stabilize the subgrade. The method to be used should be determined at the time of construction based on the actual site conditions.

8.7.4 Engineered Fill Placement and Compaction

Engineered fill should be placed on soil subgrades that are prepared as recommended in this report. Engineered fill should be placed in horizontal lifts each not exceeding 8 inches in thickness and mechanically compacted to the requirements below at the recommended moisture content. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory maximum dry density as determined by ASTM Test Method D1557, latest edition, expressed as a percentage. Moisture conditioning for soils outside the range of optimum moisture of soils should consist of adding water to the soils if they are too dry and allowing the soils to dry if they are too wet.

Engineered fills consisting of on-site soils and imported soils should be compacted to a minimum of 90 percent relative compaction with moisture content between about 1 and 3 percent above the laboratory optimum value. In pavement areas, the upper 12 inches of subgrade soil and the full section of the aggregate base should be compacted to a minimum of 95 percent relative compaction with moisture content slightly above the optimum value. Aggregate base in vehicle pavement areas should be compacted at slightly above the optimum moisture content to a minimum of 95 percent relative compaction.

8.7.5 Material for Engineered Fill

In general, on-site soils with an organic content of less than 3 percent by weight, free of any hazardous or deleterious materials, and meeting the gradation requirements below may be used as general engineered fill to achieve project grades, except when special material (such as or capillary break material) is required.

In general, engineered fill material should not contain rocks or lumps larger than 3 inches in greatest dimension, should not contain more than 15 percent of the material larger than 1½ inches, and should contain at least 20 percent passing the No. 200 sieve. In addition to these requirements, import fill should have a low expansion potential as indicated by a Plasticity Index of 15 or less, or an Expansion Index of less than 20.

All import fills must be approved by the project geotechnical engineer before delivery to the site. At least five (5) working days before importing to the site, a representative sample of the proposed import fill should be delivered to our laboratory for evaluation.

8.7.6 Utility Trench Excavation and Backfill

We estimate that excavations within the encountered soil should be able to be accomplished with conventional digging equipment, such as backhoes and excavators, and that jackhammers and/or blasting should not be necessary. Excavations should be constructed in accordance with the current CAL-OSHA safety standards and local jurisdiction. The stability and safety of excavations, braced or unbraced, are the responsibility of the contractor.

Pipe-zone backfill, extending from the bottom of the trench to about 1 foot above the top of the pipe, should consist of free-draining sand (at least 90% passing a No. 4 sieve and less than 5% passing a No. 200 sieve) compacted to a minimum of 90 percent relative compaction unless concrete or cement slurry is specified.

Above the pipe zone, underground utility trenches may be backfilled with free-draining sand, on-site soil, or imported soil. The trench backfill should be compacted to the requirements given in the section on "Engineered Fill Placement and Compaction." Trench backfill should be capped with at least 12 inches of compacted, on-site soil similar to that of the adjoining subgrade. The upper 12 inches of trench backfill in areas to be paved should be compacted to a minimum of 95 percent relative compaction. Compaction should be performed by mechanical means only. Water jetting or flooding to attain compaction of backfill is not permitted.

Trench excavations that extend below an imaginary plane inclined at 1½:1 (h:v) below the bottom edge of foundations should be properly shored to maintain the support of the existing facilities. Trenches that run parallel to the proposed foundations should not be excavated within the imaginary plane inclined at 1½:1 (h:v) below the bottom of the footing.

8.7.7 Considerations for Soil Moisture and Seepage Control

Subgrade soil and engineered fill should be compacted at moisture content meeting our recommendations. Once compacted, soils should be protected from drying and wetting. This may be accomplished by regular watering with a water truck to prevent excessive drying or covering with plastic sheeting to prevent excessive wetting from rainfall.

Consideration should be given to reducing the potential for water infiltration from the exterior to under the building through utility lines crossing the building perimeter. In utility lines crossing beneath perimeter foundations, permeable backfill should be terminated at least 1 foot outside of the perimeter foundation. Impermeable material, such as concrete or clay soil, should be used for the entire trench depth to act as a seepage cutoff.

Where concrete slabs or pavements abut against landscaped areas, the base rock layer and subgrade soil should be protected against saturation. If water is allowed to seep into the subgrade soil or pavement section, it could reduce the service life of the improvements. Methods that may be considered to reduce infiltration of water include: 1) subdrains installed behind curbs and slabs in landscape areas; 2) vertical cut-offs, such as a deepened curb section, or equivalent, extending at least 2 inches into the subgrade soil; and 3) use of drip irrigation system for landscape watering.

8.7.8 Wet Weather Construction

If site grading and construction are to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads and/or subgrades, or by flooding excavations.

Earthwork during rainy months may require extra effort and caution by the contractors. The grading contractor should be responsible to protect his work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

8.8 TECHNICAL REVIEW AND CONSTRUCTION OBSERVATION

Before construction, the geotechnical engineer should review the project plans and specifications for conformance with the intent of the recommendations presented in this report. The geotechnical engineer should be contacted a minimum of 48 hours in advance of excavation operations to observe the subsurface conditions.

9.0 LIMITATIONS

The conclusions and recommendations presented in this report are based on the information provided regarding the planned construction, and the results of the geologic mapping, subsurface exploration, and testing, combined with interpolation of the subsurface conditions between boring locations. Site conditions described in the text of this report are those existing at the time of our last field reconnaissance and are not necessarily representative of the site conditions at other times or locations. This information notwithstanding, the nature and extent of subsurface variations between borings may not become evident until construction. If variations are encountered during construction, Cal Engineering & Geology, Inc. should be notified promptly so that conditions can be reviewed and recommendations reconsidered, as appropriate.

It is the Owner's responsibility to ensure that recommendations contained in this report are carried out during the construction phases of the project. This report was prepared based on preliminary design information provided which is subject to change during the design process. At approximately the 90 percent design level, Cal Engineering & Geology, Inc. should review the design assumptions made in this report and prepare addenda or memoranda as appropriate. Any modifications included in these addenda or memoranda should be carefully reviewed by the project designers to make sure that any conclusions or recommendations that are modified are accounted for in the final design of the project.

The findings of this report should be considered valid for three years unless the conditions of the site change. After three years, CE&G should be contacted to review the site conditions and prepare a letter regarding the applicability of this report.

This report presents the results of a geotechnical and geologic investigation only and should not be construed as an environmental audit or study. The evaluation or identification of the potential presence of hazardous materials at the site was not requested and was beyond the scope of this investigation and report.

The conclusions and recommendations contained in this report are valid only for the project described in this report. We have employed accepted geotechnical engineering procedures, and our professional opinions and conclusions are made in accordance with generally accepted geotechnical engineering principles and practices. This standard is in lieu of all other warranties, either expressed or implied.

10.0 REFERENCES

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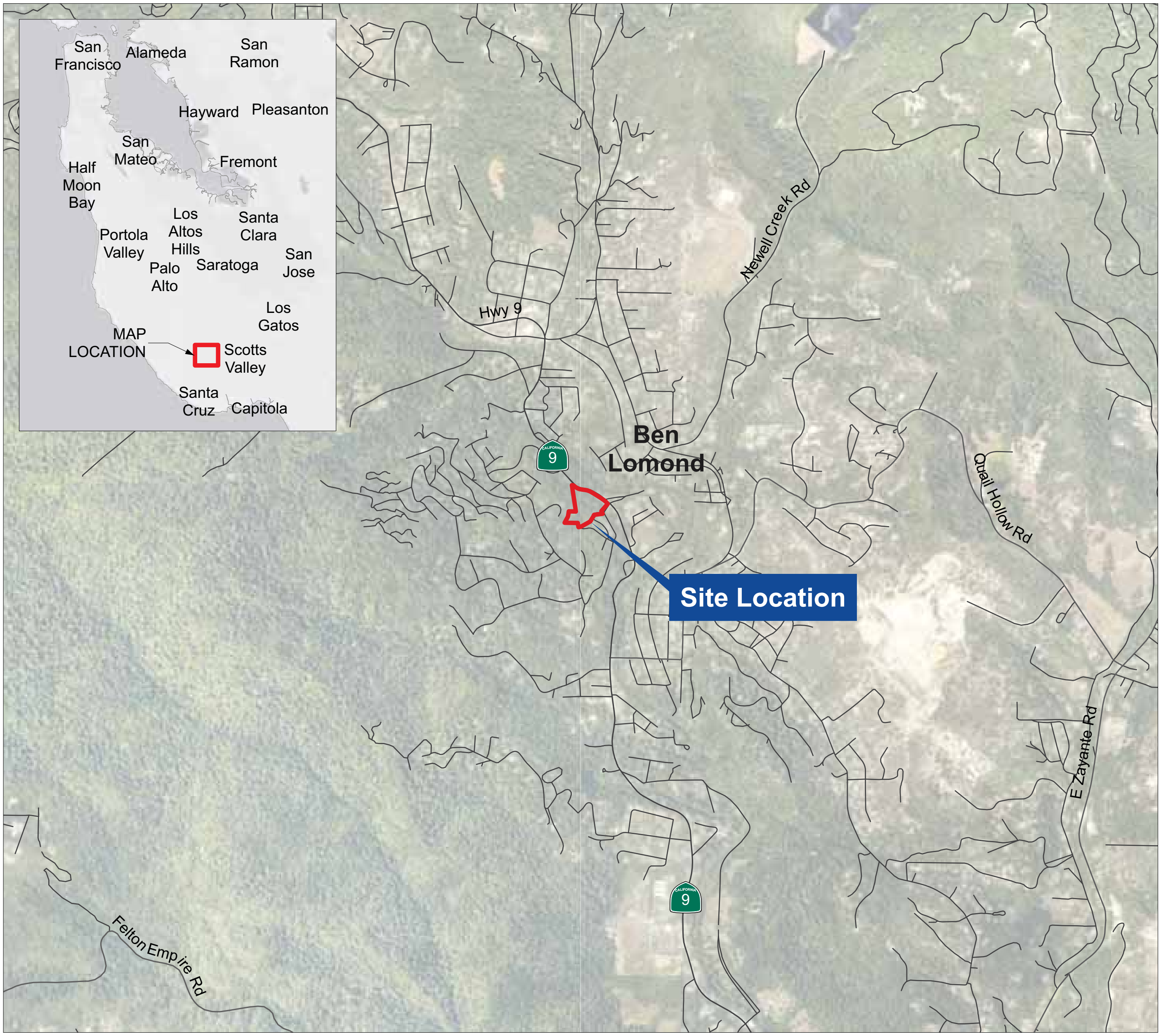

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1. STREET CENTERLINES FROM CALTRANS CALIFORNIA ROAD SYSTEM, DOWNLOADED ON 18 FEB 2020.
2. ORTHOIMAGERY FROM ESRI (MAXAR), 2019.

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VETERANS VILLAGE HOUSING PROJECT
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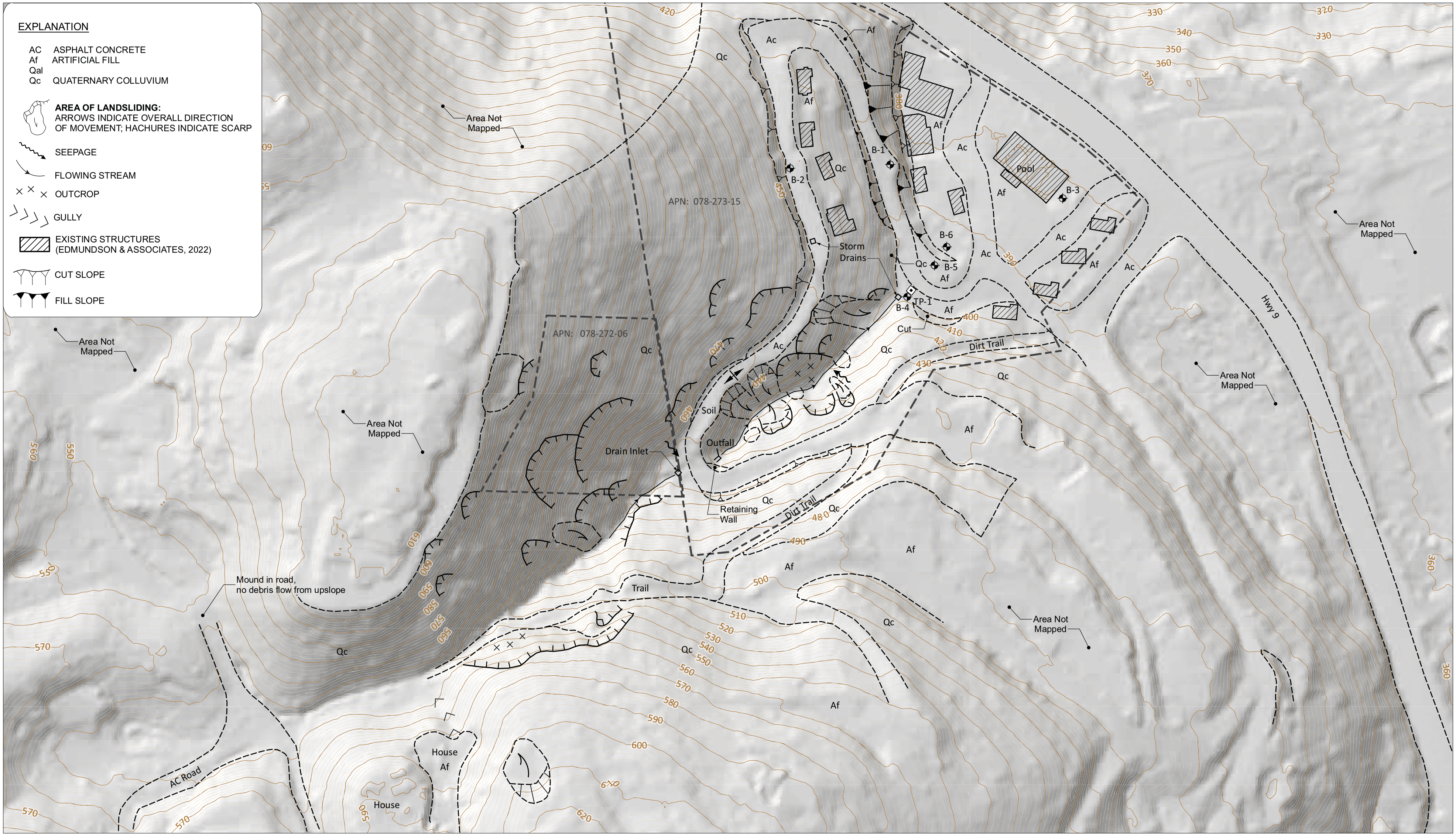
BEN LOMOND, CALIFORNIA

SITE LOCATION MAP

220300	June 2022	FIGURE 1



BASEMAP REFERENCE

1. GEOLOGIC MAPPING BY K. LOEB ON 4/6/2022.
2. 2-FT CONTOURS DERIVED FROM 2018-2020 LIDAR DATA (3-FT DEM) FOR SANTA CRUZ COUNTY.

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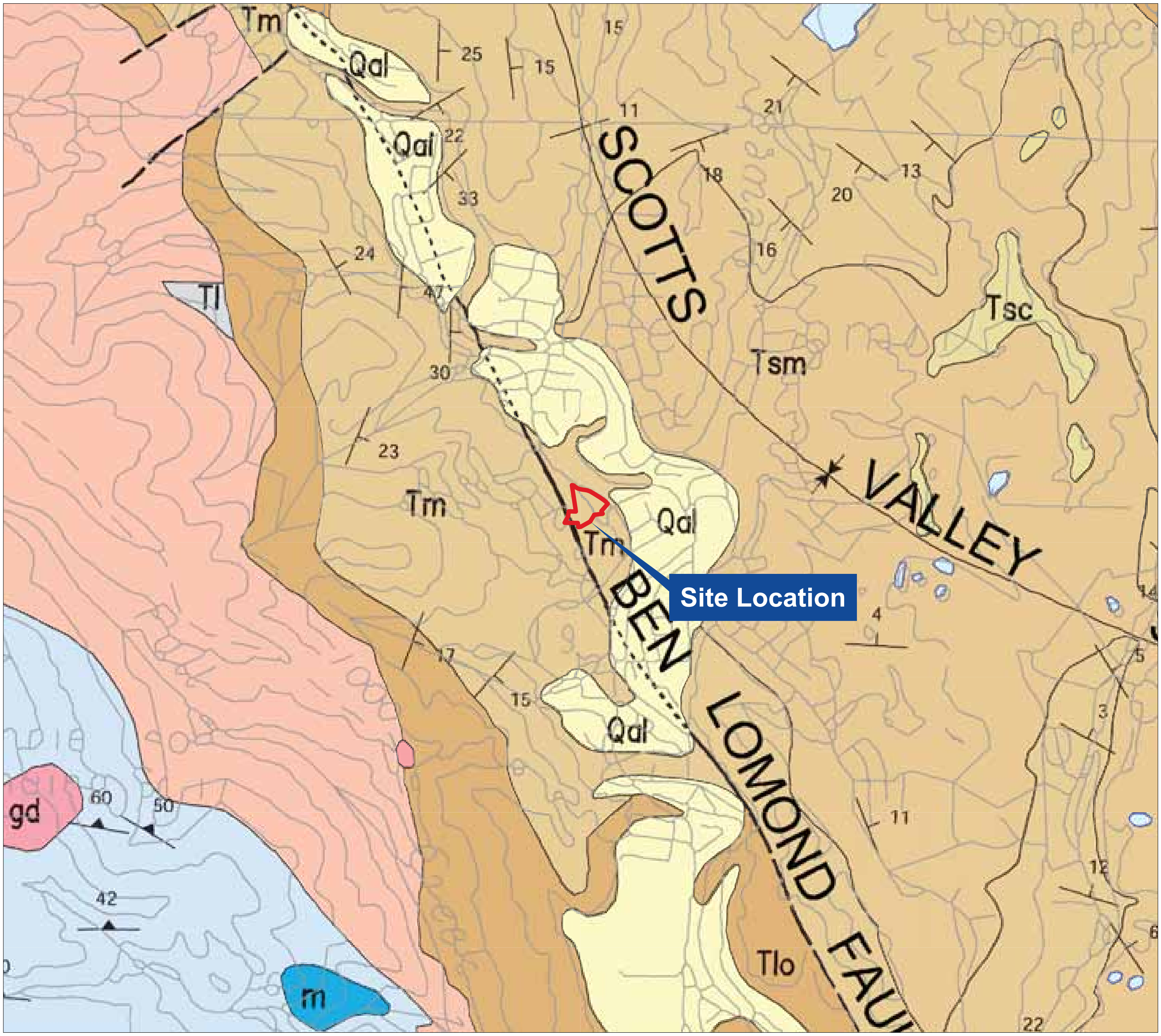
SITE PLAN AND GEOLOGIC MAPPING

220300 | June 2022 | FIGURE 2



BASEMAP REFERENCE

1. REGIONAL GEOLOGY FROM BRABB, 1997 (OPEN-FILE REPORT 97-489).

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MILES

MAP UNIT DESCRIPTION

Qal	Alluvial deposits, undifferentiated (Holocene)
Tsc	Santa Cruz Mudstone (upper Miocene)
Tsm	Santa Margarita Sandstone (upper Miocene)
Tm	Monterey Formation (middle Miocene)
Tlo	Lompico Sandstone (middle Miocene)
Tl	Locatelli Formation (Paleocene)
qd	Quartz diorite (Cretaceous)
gd	Gneissic granodiorite (Cretaceous)
hcg	Hornblende-cummingtonite gabbro (Cretaceous)
sch	Metasedimentary rocks (Mesozoic or Paleozoic)
m	Marble (Mesozoic or Paleozoic)

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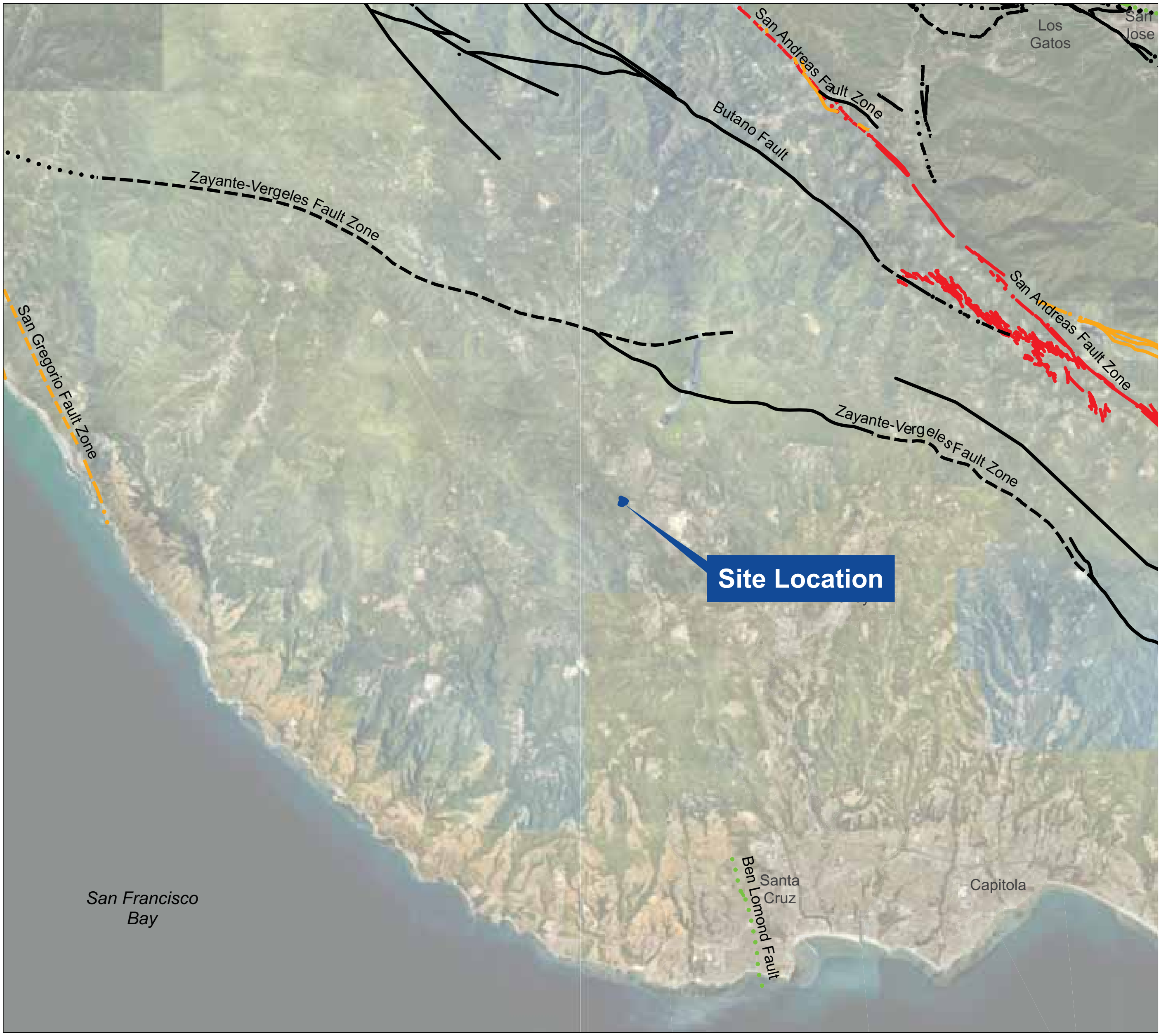
REGIONAL GEOLOGY MAP

220300	June 2022	FIGURE 3



BASEMAP REFERENCE

1. FAULT LOCATIONS FROM US GEOLOGICAL SURVEY QUATERNARY FAULTS AND FOLDS DATABASE, ACCESSED ONLINE ON 30 JULY 2021.

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MILES

MAP UNIT DESCRIPTION

- historical (<150 years), well constrained location
- historical (<150 years), moderately constrained location
- historical (<150 years), inferred location
- latest Quaternary (<15,000 years), well constrained location
- latest Quaternary (<15,000 years), moderately constrained location
- latest Quaternary (<15,000 years), inferred location
- late Quaternary (<130,000 years), well constrained location
- late Quaternary (<130,000 years), moderately constrained location
- late Quaternary (<130,000 years), inferred location
- middle and late Quaternary (<750,000 years), well constrained location
- middle and late quaternary (<750,000 years), moderately constrained location
- middle and late Quaternary (<750,000 years), inferred location
- undifferentiated Quaternary(<1.6 million years), well constrained location
- undifferentiated Quaternary(<1.6 million years), moderately constrained location
- undifferentiated Quaternary(<1.6 million years), inferred location
- Class B (various age), well constrained location
- Class B (various age), moderately constrained location
- Class B (various age), inferred location

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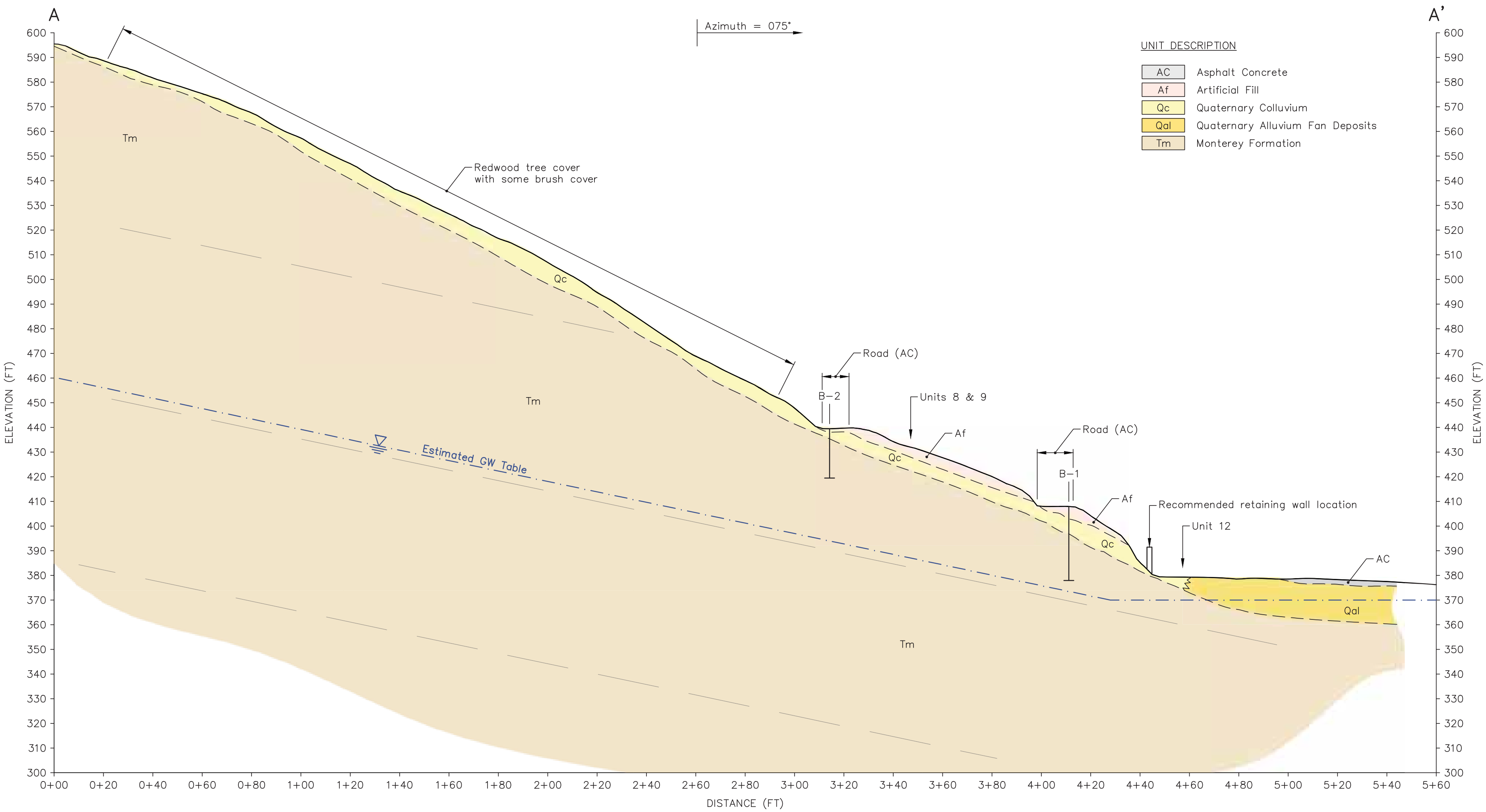
FAULT ACTIVITY MAP

220300	June 2022	FIGURE 4



NOTES

1. APPROXIMATE APPARENT DIP OF 021°, 21SE USED FOR Tm (BRABB, 1997).
2. BORINGS B-1 AND B-2 DRILLED BY CE&G ON 4/11/2022.
3. GW TABLE BASED ON OBSERVED SEEPAGE AND GROUNDWATER ENCOUNTERED IN BORING B-4.

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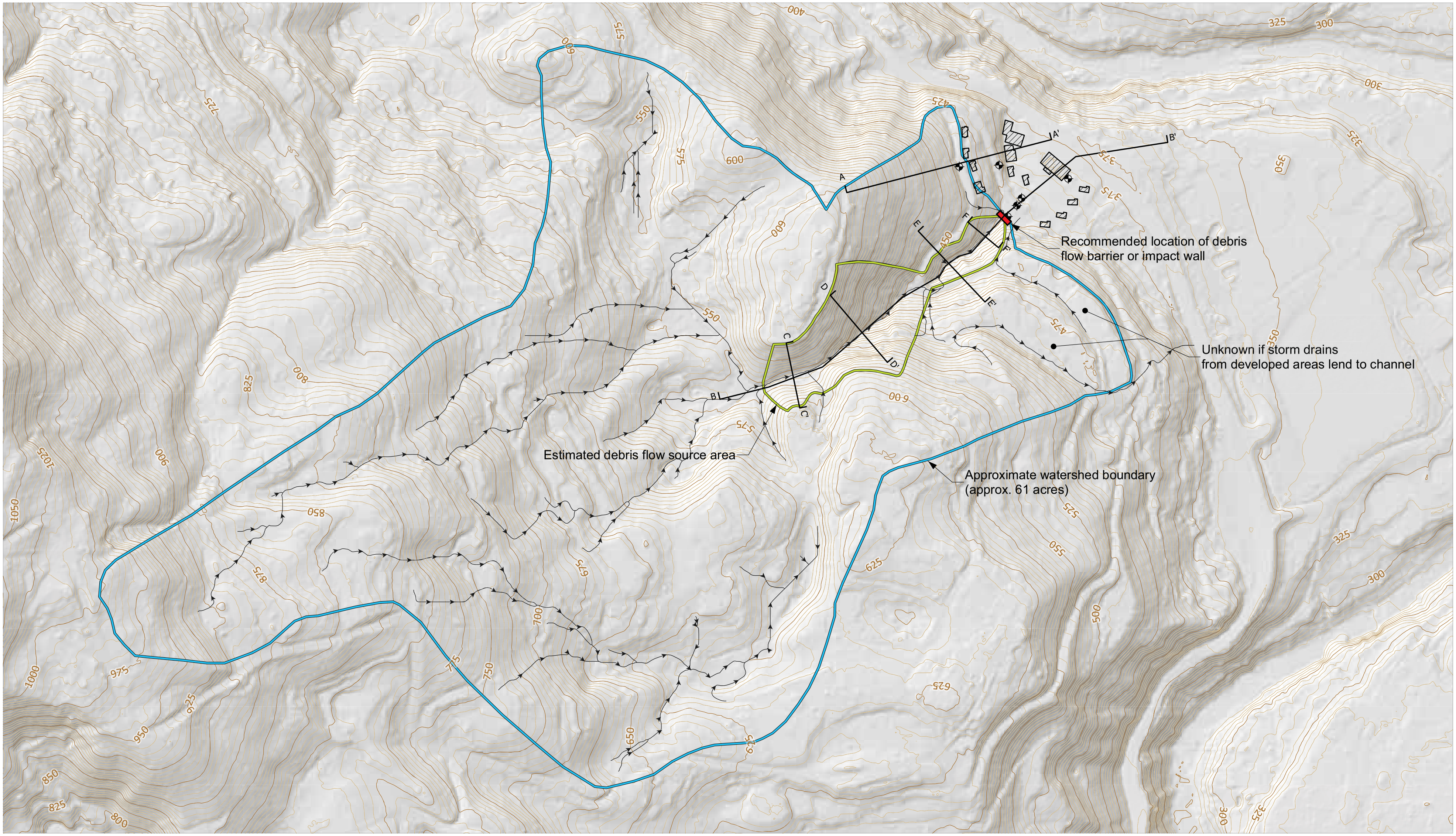
GEOLOGIC SECTION A-A'

220300	June 2022	FIGURE 5



BASEMAP REFERENCE

1. 5-FT CONTOURS DERIVED FROM 2018-2020 LIDAR DATA (3-FT DEM) FOR SANTA CRUZ COUNTY.

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WATERSHED AND DEBRIS FLOW SOURCE AREA MAP

220300	June 2022	FIGURE 6

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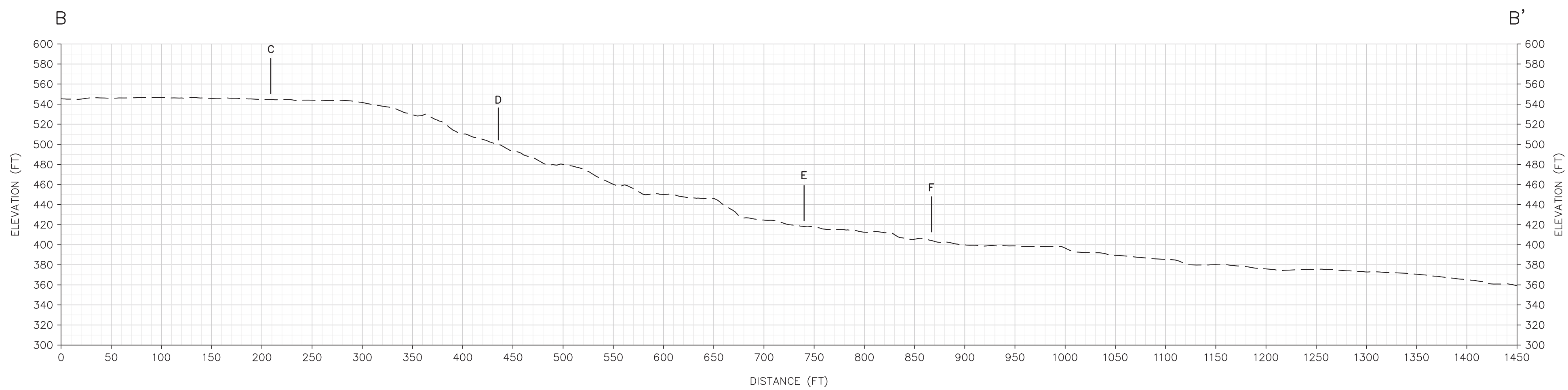

PROFILE B–B'

REFERENCES

1. EXISTING SURFACE PROFILE DERIVED FROM 2018–2020 LIDAR DATA (3–FT DEM) FOR SANTA CRUZ COUNTY.

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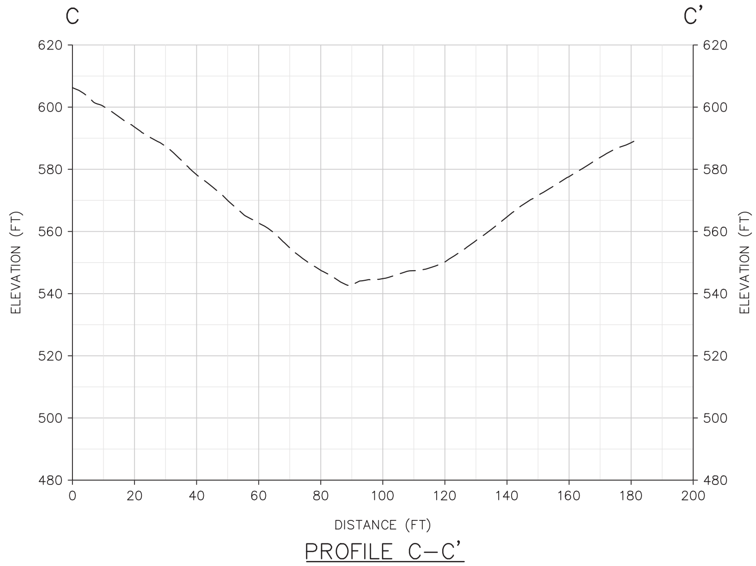
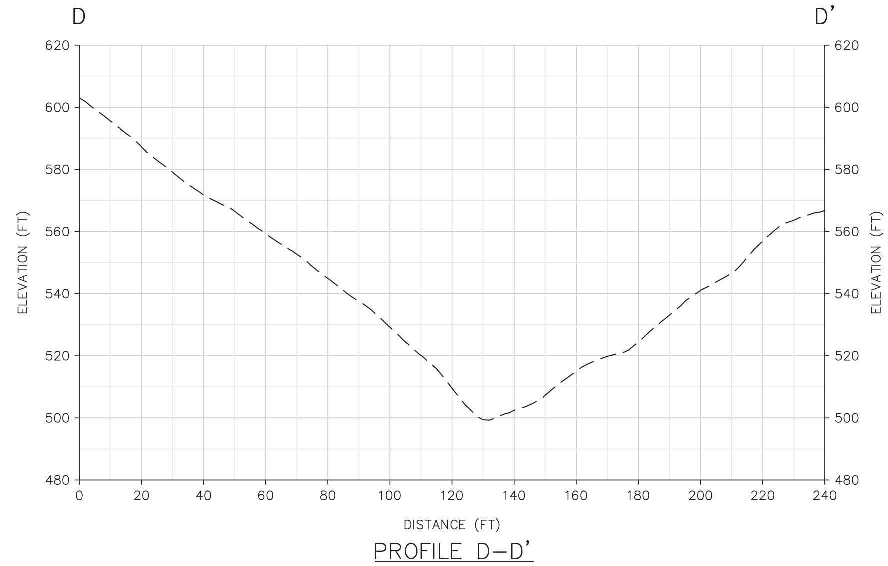
PROFILE B-B' (CHANNEL GRADIENT)

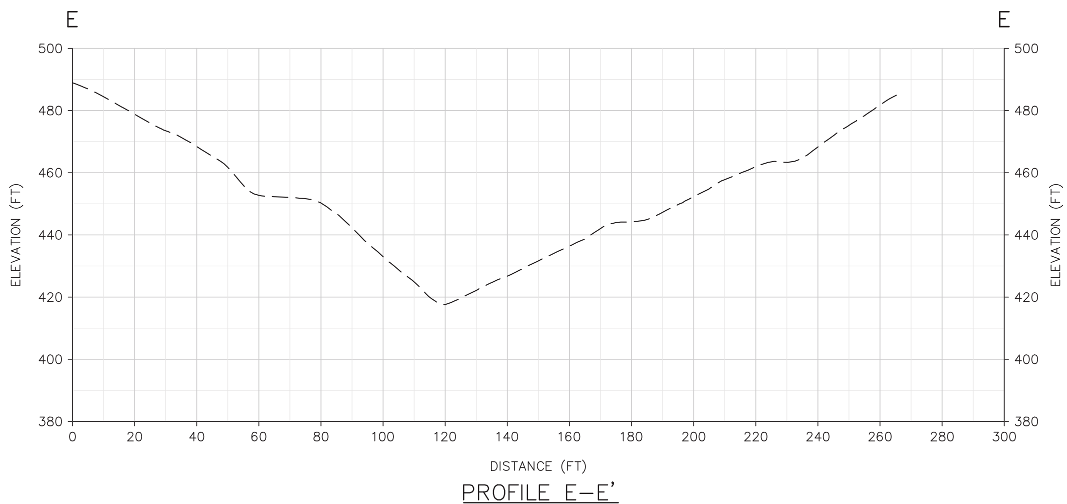
220300	June 2022	FIGURE 7



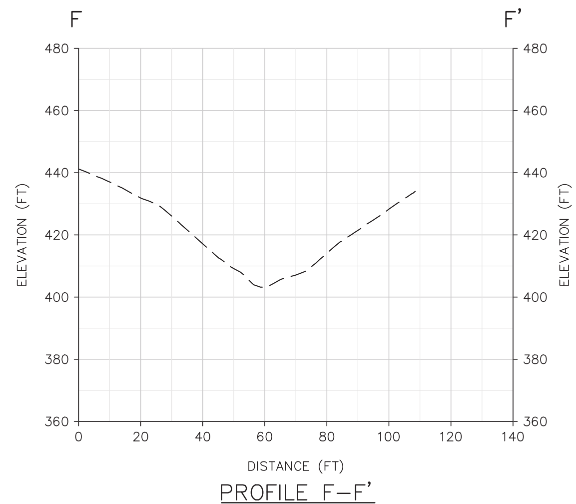
PROFILE C–C'



PROFILE D–D'



PROFILE E–E'



PROFILE F–F'

REFERENCES

1. EXISTING SURFACE PROFILE DERIVED FROM 2018–2020 LIDAR DATA (3–FT DEM) FOR SANTA CRUZ COUNTY.

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SCALE FEET

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PROFILES C-C' THROUGH F-F'
(CROSS CHANNEL PROFILES)

220300	June 2022	FIGURE 8

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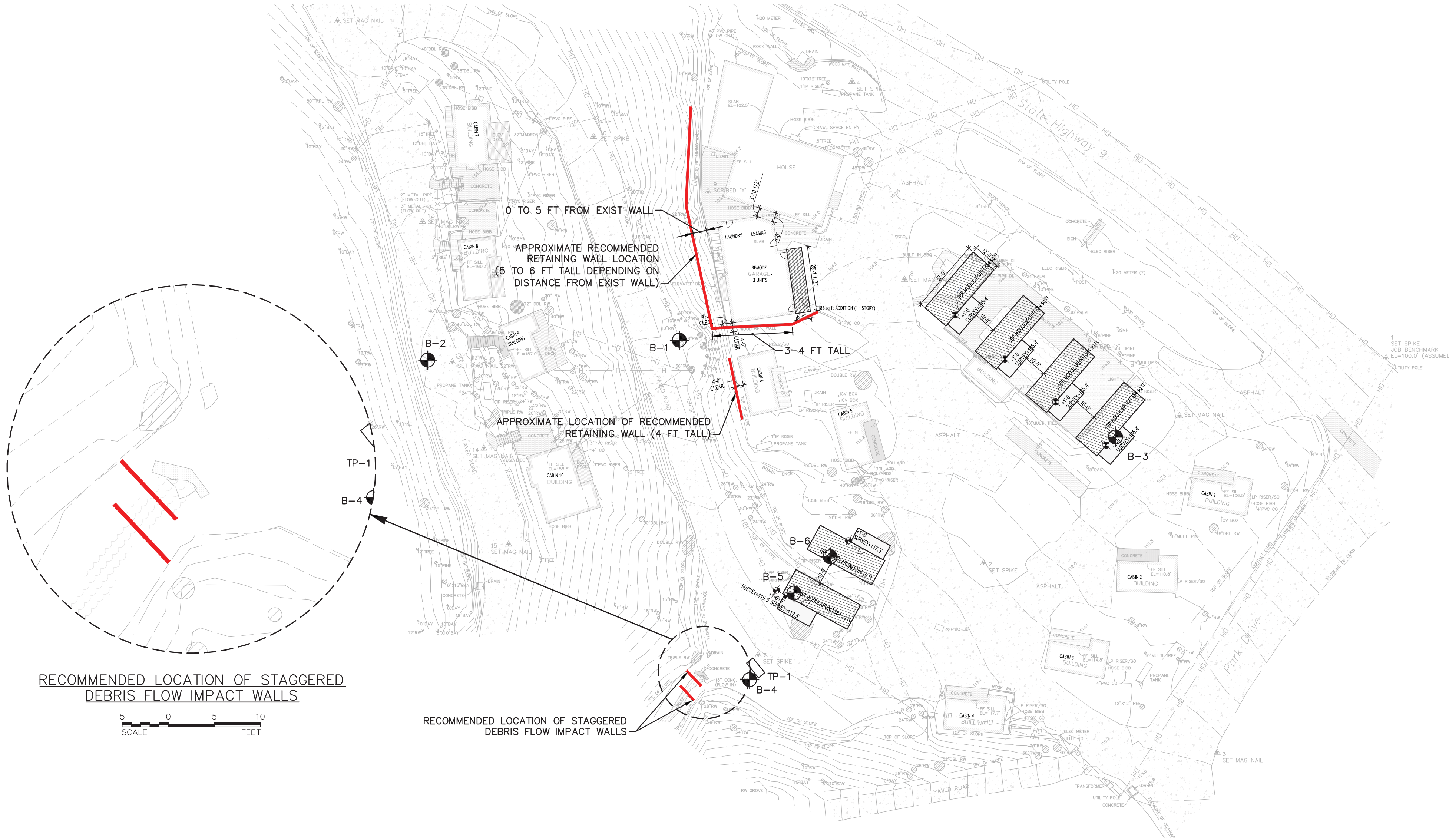

REFERENCES

1. TOPOGRAPHIC SURVEY AND IMPROVEMENTS SITE PLAN BASE MAP PREPARED BY NIELSEN ARCHITECTS: PRELIMINARY FILE DATED MAY 19, 2022; CAD FILE PROVIDED ON MAY 19, 2022.

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VETERANS VILLAGE HOUSING PROJECT
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RECOMMENDED RETAINING WALL AND DEBRIS FLOW IMPACT WALL LOCATIONS

220300	June 2022	FIGURE 9

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Appendix A. Site Overview Figure (Sherwood, 2022)

SITE LEGEND

- (E) CONTOUR (LIDAR)
- (E) EDGE OF PAVEMENT
- (E) EDGE OF DIRT ROAD
- PARCEL LINE (FROM SANTA CRUZ COUNTY GIS)

TABLE OF EXISTING AND PROPOSED CONDITIONS

KEYNOTE	DISCRIPTION	EXISTING CONDITIONS	PROPOSED CONDITIONS
1	CABIN 1	STUDIO	REMODELED ONE BEDROOM UNIT
2	CABIN 2	STUDIO	REMODELED ONE BEDROOM UNIT (ADA)
3	CABIN 3	STUDIO	REMODELED ONE BEDROOM UNIT (ADA)
4	CABIN 4	STUDIO	REMODELED ONE BEDROOM UNIT
5	CABIN 5	STUDIO	REMODELED ONE BEDROOM UNIT
6	CABIN 6	STUDIO	REMODELED ONE BEDROOM UNIT
7	CABIN 7	STUDIO	REMODELED ONE BEDROOM UNIT
8	CABIN 8	STUDIO	REMODELED ONE BEDROOM UNIT
9	CABIN 9	ONE BEDROOM	REMODELED ONE BEDROOM UNIT
10	CABIN 10	TWO BEDROOM	REMODELED TWO BEDROOM UNIT
11	MAIN HOUSE	FOUR BEDROOM	REMODELED FOUR ROOM BUILDING
12	GARAGE	THREE CAR GARAGE	NEW TWO STORY FOUR UNIT BUILDING
13	RV SPACE 1	DIRT PARKING	NEW ONE BEDROOM UNIT
14	RV SPACE 2	DIRT PARKING	NEW ONE BEDROOM UNIT
15	RETAINING WALL	WOOD RETAINING WALL	NEW RETAINING WALL
16	POOL	SWIMMING POOL	SEE KEYNOTE 17 - 20
17	MODULAR 1	SWIMMING POOL AREA	NEW MODULAR ONE BEDROOM UNIT
18	MODULAR 2	SWIMMING POOL AREA	NEW MODULAR ONE BEDROOM UNIT
19	MODULAR 3	SWIMMING POOL AREA	NEW MODULAR ONE BEDROOM UNIT
20	MODULAR 4	SWIMMING POOL AREA	NEW MODULAR ONE BEDROOM UNIT

SUMMARY OF SLOPE CONDITIONS

SLOPE RANGE (%)	AREA (ACRES)	COLOR
0-29.99	2.87	(green)
30-49.99	0.68	(yellow)
>50	2.38	(red)

0 20' 40' 80'

North

OVERVIEW FIGURE

SHERWOOD DESIGN ENGINEERS
2548 Mission Street
San Francisco, CA 94110
www.sherwoodengineers.com

NO	DATE	REVISION

PROJECT NO.
DATE 02/25/22
DRAWN CBH
DESIGNED XXX
CHECKED XXX

JAYE'S TIMBERLANE RESORT
VETERAN'S VILLAGE
BEN LOMAND, CALIFORNIA

SLOPE ANALYSIS

DRAWING NO.
FIG. 2

Appendix B. Santa Cruz Co. Geologic Hazards Assessment

COUNTY OF SANTA CRUZ

PLANNING DEPARTMENT

701 OCEAN STREET, 4TH FLOOR, SANTA CRUZ, CA 95060
(831) 454-2580 FAX: (831) 454-2131 TDD: (831) 454-2123

March 16, 2022

Shawn and Jason Moore
660 Memory Lane
Boulder Creek, CA 95006

Subject: GEOLOGIC HAZARDS ASSESSMENT
APNs 078-273-15 & 078-272-06
LOCATION: 8705 Highway 9
Boulder Creek, CA
APPLICATION NUMBER: REV221050
OWNERS: Vimal and Amita Patel

Dear Applicants:

I performed a site reconnaissance of the parcels referenced above on 3/15/22 where construction of seven new residential units and remodelling of 11 existing residential units are proposed. The project consists of conversion of an existing resort facility into affordable housing for veterans. The property is currently occupied by a main house and garage and ten guest cabins. All existing and proposed structures are on parcel 078-273-15.

This letter briefly discusses my site observations, outlines permit conditions, and provides requirements for further technical investigations, if any. The property location is shown on Figure 1, Topographic Index Map, attached.

Completion of this geologic hazards assessment included a site reconnaissance, a review of published maps and other pertinent documents on file with the Planning Department, and an evaluation of remote sensing imagery. The scope of this assessment is not intended to be as detailed as a full geologic or geotechnical report completed by a state registered geologic consultant.

GEOLOGIC SETTING

The subject properties occupy a gently to steeply sloping area on the southwest side of the San Lorenzo River valley about one-half mile south of the town of Ben Lomond (Figure 1). The property extends a short distance up the steep northeast flank of Ben Lomond Mountain from the flood plain of the river. The two subject parcels together form an irregularly shaped plot of about 5.9 acres. Slopes are relatively gentle on the eastern portion of the property that is proposed for new development, ranging from about 6% to 12% gradient. A small stream drainage flows easterly through the central portion of the subject properties (Figure 1). Slope gradients along the flanks of this drainage reach gradients of 70% or more.

The geologic map of Santa Cruz County shows the parcel underlain by sedimentary rock consisting of fissile siliceous shale of the Monterey Formation (Tm, Figure 2). Bedding (layering) in the shale unit is inclined generally eastward at dips of 15 to 30 degrees. This unit can be susceptible to landsliding along bedding planes.

SEISMIC SHAKING AND FAULTING

This property is located in a seismically active region of northern California, as the October 17, 1989 Magnitude 6.9 Loma Prieta earthquake amply demonstrated. The subject parcels lie approximately 7 miles southwest of the San Andreas fault zone, a very large, active strike-slip (horizontally moving) fault that extends for over 700 miles through California (Figure 3, Regional Seismicity Map). The active Zayante fault is located about 2.5 miles to the northeast of the property. Other active or potentially active faults in the area include the San Gregorio and Monterey Bay/Tularcitos faults to the southwest and the Sargent, Shannon, Calaveras, and Hayward faults to the northeast (Figure 3).

The earthquake history of the region around the subject property contains quite a few magnitude 4.0 or larger earthquakes have occurred during historical times (Figure 3). The largest historical earthquakes to have affected the area are the 1906 San Francisco Earthquake, an estimated magnitude 7.9, and the 1989 Loma Prieta earthquake, a magnitude 6.9. Other historical earthquakes of significance in the area include two magnitude 6.1 earthquakes in the Monterey Bay in 1926, magnitude 6.5 and 6.3 earthquakes on the portion of the San Andreas fault in south Santa Cruz County in 1836 and 1890, respectively, and a magnitude 6.8 earthquake on the San Andreas fault to the north of Santa Cruz County in 1938.

The subject property is not located in a State or County Fault Hazard Zone (Figure 4, Fault Zone Map). The property lies immediately to the east of the Ben Lomond fault. The Ben Lomond fault is not considered to be active and therefore no fault hazard zone has been designated along this fault.
Fault movement during earthquakes can offset the ground surface, which will severely damage or destroy structures built directly over the fault rupture zone. Projects sited in areas of active faulting must therefore be carefully evaluated for the potential for ground surface rupture. No evidence for active faulting was observed on the subject parcels during our site evaluation and therefore the potential for ground surface rupture is considered to be negligible.

Very strong ground shaking may occur on the parcel during the anticipated lifetime of the existing and proposed structures. Therefore, care must be taken in securing these structures against the possibility of strong seismic shaking. Intense ground shaking may be accompanied by shaking-related ground deformation that includes ridgetop shattering, liquefaction, lateral spreading, lurch cracking, ground subsidence, and seismically induced landsliding. The liquefaction hazard map for Santa Cruz County shows the entire parcel located outside any areas considered to have a liquefaction potential. Therefore, risks due to liquefaction and liquefaction related hazards of lurch cracking and lateral spreading are considered to be low on this parcel. Landslide potential is discussed in the following section.

LANDSLIDING

A "Preliminary Map of Landslide Deposits in Santa Cruz County" was prepared in 1975 as part of the County's General Plan, a portion of which is depicted on Figure 5. This interpretive map was prepared from aerial photographs and was designed only for "regional land use evaluations." The map indicates areas where questionable, probable, or definite past instability is suspected. While not a susceptibility map indicating potential site-specific stability problems, when utilized in conjunction with other published data and documents the map is a useful planning resource.

The entirety of the subject parcels are shown to be within the boundaries of a very large "probable" landslide on the Santa Cruz County Landslide Map (Figure 5). For this evaluation, we reviewed lidar imagery and 1975 and 1989 aerial photos in order to assess the potential extent of landsliding on or near the property. We also performed field observation of sloping portions of the parcel. The size and shape of the landslide depicted on the County Landslide Map is not consistent with our observations. In particular, there is no landsliding of the scale depicted on the County Landslide Map evident on the lidar imagery, which was not available at the time the County Landslide Map was constructed and which shows the ground surface morphology in much greater detail than the old aerial photographs.

We did note some topography on the slope above the subject property that could be indicative of a large, very old, highly dissected landslide. However, the lower portion of the slope steepens, clearly due to ancient downcutting by San Lorenzo River. The toe of this slope is not displaced in a way that would indicate significant, geologically recent movement. Consequently, the potential for large-scale landsliding to be hazard at this site is judged to be low.

Smaller scale landsliding was noted on the steepest slopes bordering the stream drainage on the western portion of the properties. These areas are located away from any proposed or existing development. However, there is a potential for small scale landsliding along the flanks of the stream drainage to produce debris flows that flow down the axis of the channel into development areas. An alluvial fan has been formed where stream emerges from the narrow, steep-sided channel on the western part of the property. This alluvial fan may, and likely does, include older debris flow deposits. Structures sited on this fan could be at risk of debris flow impact.

GEOLOGIC HAZARDS

Two potential geologic hazards are posed to the proposed development: strong seismic shaking due to an earthquake on one of the local active faults and debris flow impact on the portions of the property underlain by alluvial fan deposits. These hazards are discussed below.

The subject property is likely to be subjected to strong seismic shaking in the next 50 to 100 years. In addition to the San Andreas fault, there are a number of active or potentially active faults in the region that could cause strong shaking at the study site. Any "development", as defined by Santa Cruz County Code of Regulations (SCCCR) 16.10.040 (19) associated with the project shall be designed to the most current seismic standards of the California Building Code. In this case, the proposed new residential building will qualify as development. The remodels of existing structures may or may not be considered development depending on the amount and type of remodeling done.

The portions of the project located on the alluvial fan on the property are potentially at risk of debris flow impact. This area includes, at a minimum, the four new modular housing units proposed for the area of the existing pool (per the 2/26/22 Site Plan by Sherwood Design Engineers), but may well include other portions of the site.

CONCLUSIONS AND REPORT REQUIREMENTS

Based on my site visit and review of pertinent maps and other documents, further geologic evaluation in the form of a focused geologic report is indicated for the proposed development on these parcels. The focused geologic report shall provide an assessment of risks to the proposed development due to debris flow hazards and shall provide recommendations for mitigating any recognized risks. The project will also require a geotechnical report for design of the proposed new structures and for remodeling of existing structures if foundation modifications are proposed.

Please note that the report requirements outlined above are based on a review of the conceptual plans by Sherwood Design Engineers (dated 2/26/22) that were provided to us. It is possible that the detailed project plans submitted at the building permit stage may contain changes to the placement or design of structures that alter the report requirements.

If you have any questions concerning these conditions, the hazards assessment, or geologic issues in general, please contact me at 454-3175.

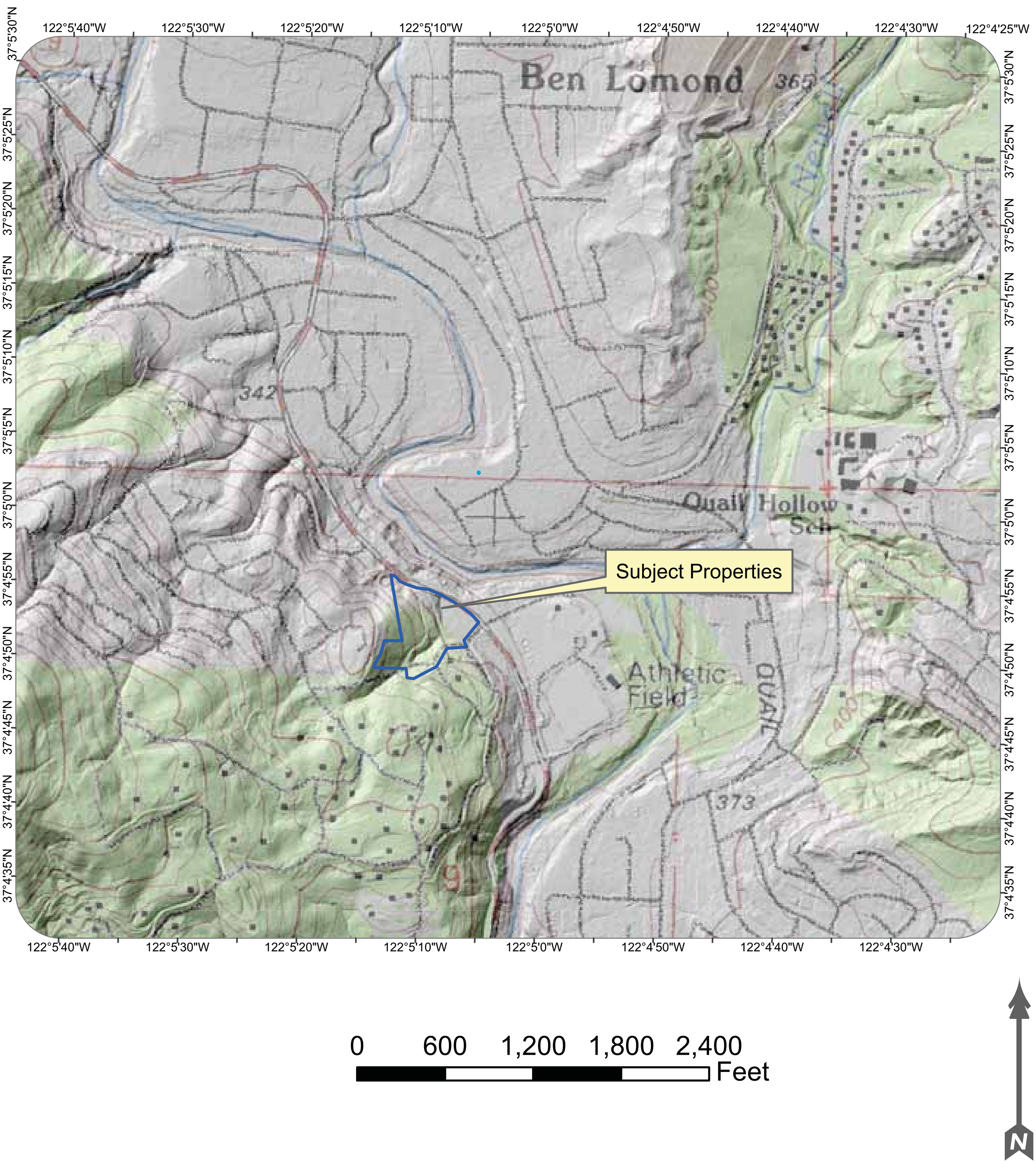
Sincerely,

____
Jeff Nolan
County Geologist
CEG #2247

______3/18/22____________
Date

Enclosure(s): Figures 1-5

Cc:	Jessica deGrassi



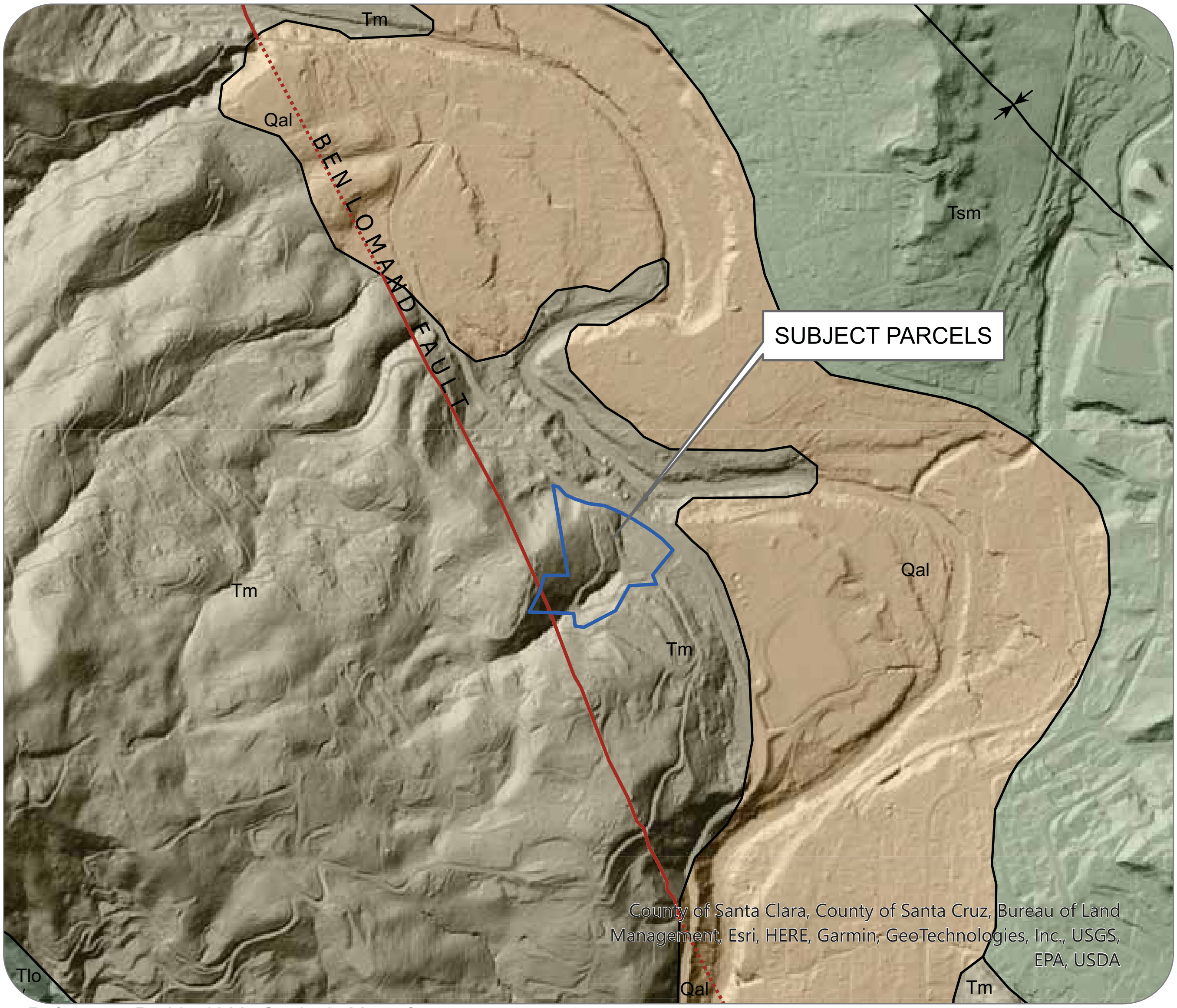
Planning Department
http://www.sccoplanning.com/

Topographic Index Map

8705 Highway 9
Santa Cruz County, California
APNs: 078-273-15; 078-272-06

Figure # 1

Date 3/16/22



Reference: Brabb, 1989, Geologic Map of Santa Cruz County, California

Strike and dip of Bedding
syncline, certain
contact, certain
fault, certain
fault, concealed
Qal Alluvium
Santa Margarita Sandstone
Tm Monterey Formation
Tlo Lompico Sandstone

N

0 310 620 1,240 1,860 2,480 Feet

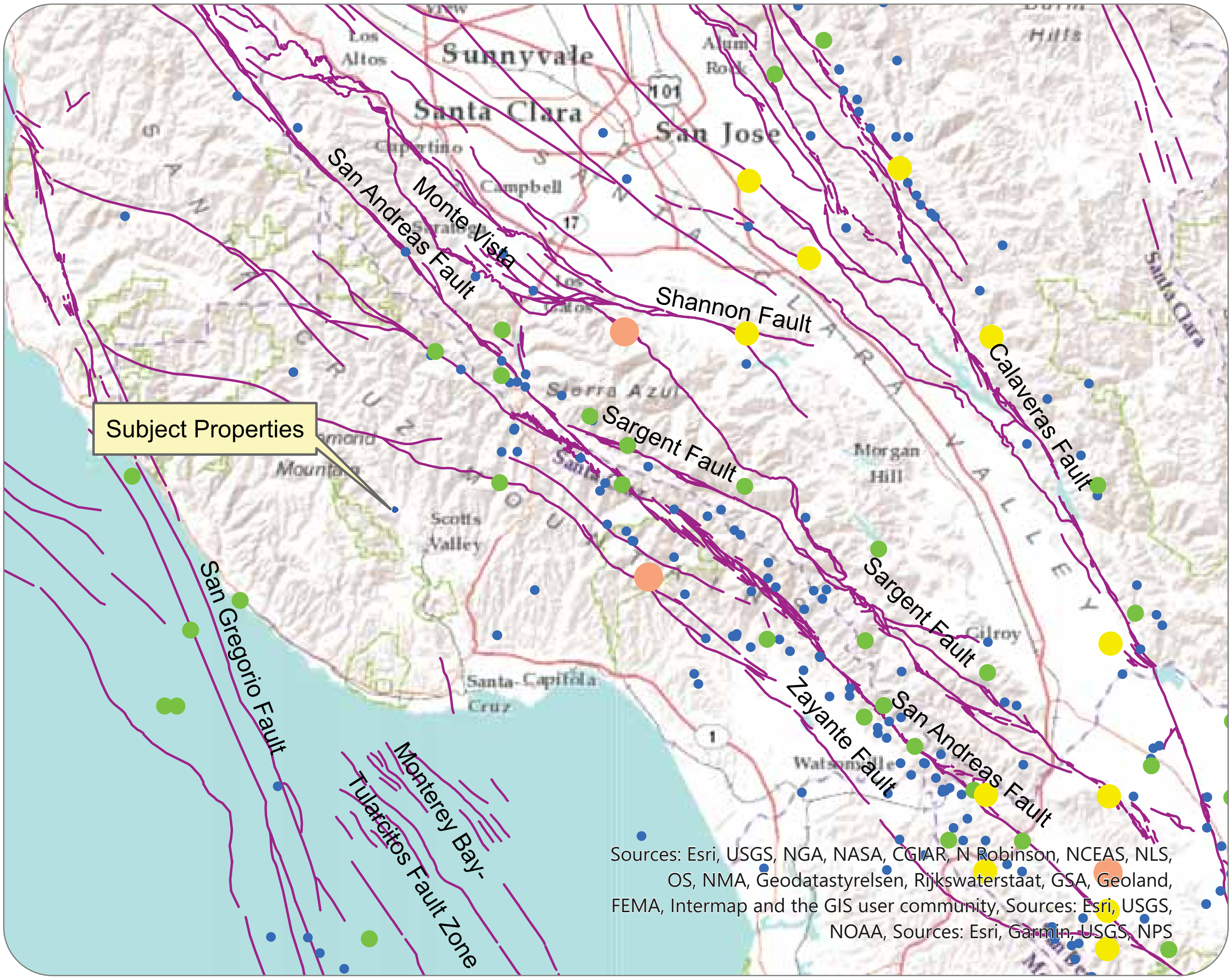
Planning Department
http://www.sccoplanning.com/

Regional Geologic Map

8705 Highway 9
Santa Cruz County, California
APNs: 078-273-15; 078-272-06

Figure # 2

Date: 3/16/22



Seismic events from U.S. Geological Survey earthquake catalog at:
https://earthquake.usgs.gov/earthquakes/search/ accessed 8/19/19

Faults from Jennings, C.W., 1994, Fault activity map of California and adjacent areas. California Division of Mines and Geology, California Geologic Data Map Series, Map No. 6.

Legend

Active or Potentially Active Faults

1800 to 2019 magnitude 4.0 or larger earthquakes

- Magnitude 4.0 to 4.99
- Magnitude 5.0 to 5.99
- Magnitude 6.0 to 6.49
- Magnitude 6.5 to 6.99
- Magnitude 7+

0 4.5 9 13.5 18 Mile

N

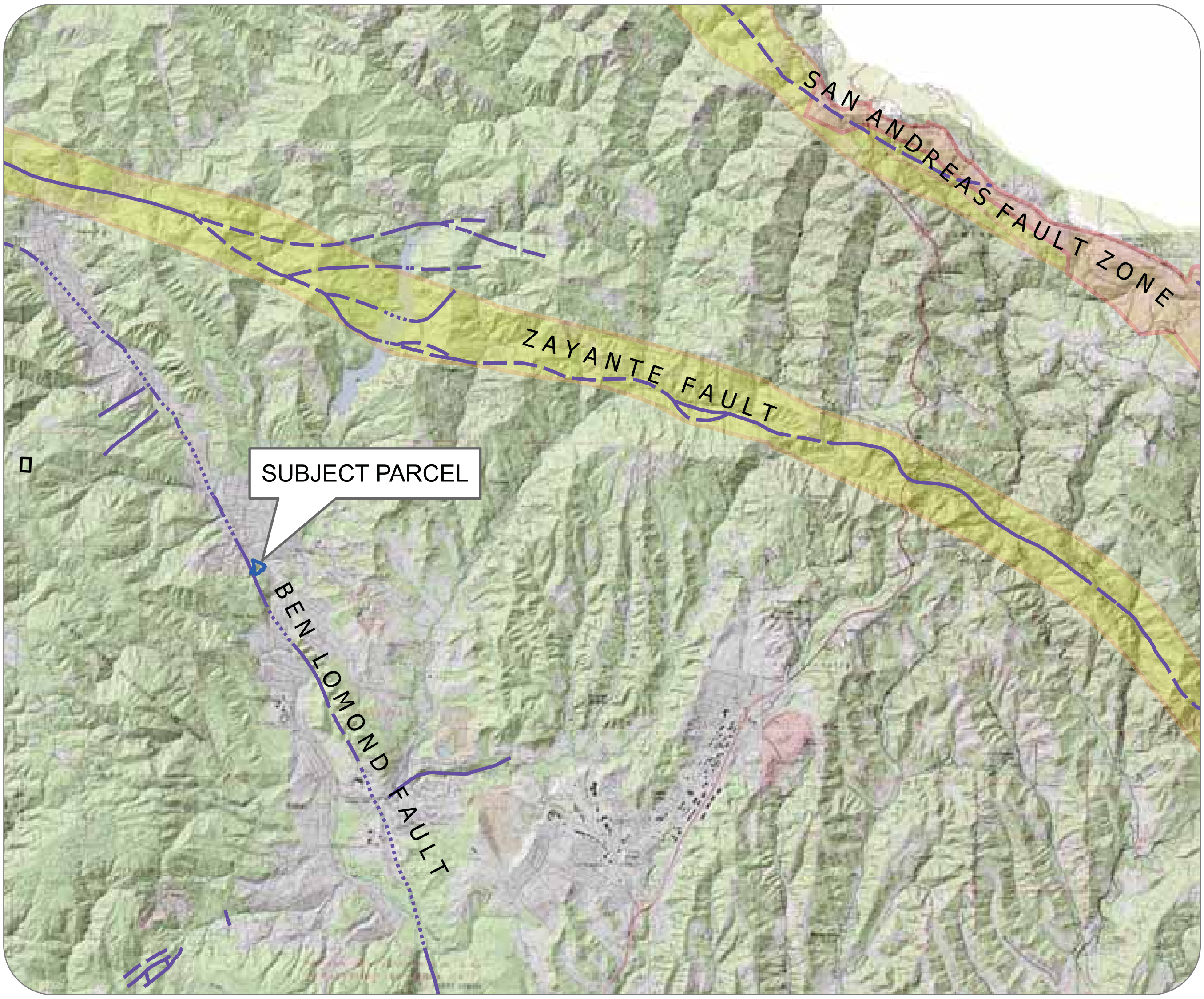
Planning Department
http://www.sccoplanning.com/

Regional Faulting and Seismicity Map

Jupiter Terrace
Santa Cruz County, California
APNs: 078-273-15; 078-272-06

Figure #5

Date: 3/16/22



Reference: U.S.Geological Survey MIscellaneous Investigation Series Map I-1905

Legend

Fault Zones
- County Fault Zone
- State Fault Zone

Faults from Brabb (1989)
- fault, certain
- fault, approx. located
- fault, concealed

0 1 2 4 Miles

N

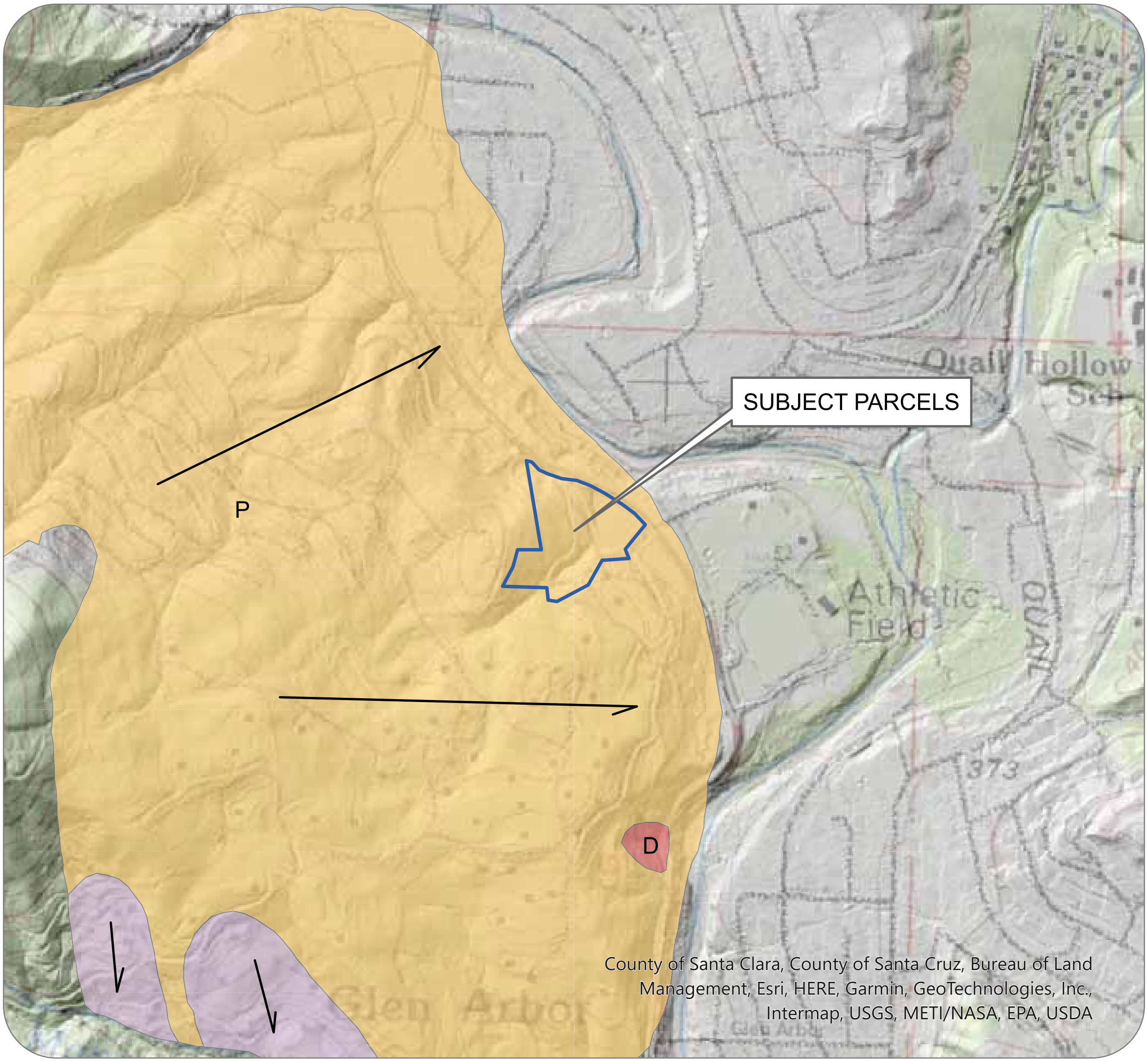
Planning Department
http://www.sccoplanning.com/

Fault Map

8705 HIghway 9
Santa Cruz County, California
APNs: 078-273-15; 078-272-06

Figure # 4

Date: 3/16/22



Reference: Landslide mapping by Cooper-Clark Associates. 1975, Preliminary map of landslide deposits in Santa Cruz County.

Small Landslides (too small to depict at map scale)

- Definite Landslide
- Probable Landslide
- Questionable Landslide

Large Landslides

- D- Definite Landslide Deposit
- P - Probable Landslide Deposit
- ? - Questionable Landslide Deposit
- Unattributed Landslide Deposit

Direction of Landslide Movement

N

0 490 980 1,470 Feet

Planning Department
http://www.sccoplanning.com/

Santa Cruz County Landslide Map

8705 Highway 9
Santa Cruz County, California
APNs: 078-273-15; 078-272-06

Figure #5

Date: 3/16/22

Appendix C. Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Field Identification			Group Symbols	Typical Names	Laboratory Classification Criteria	
Coarse-Grained Soils More than 50% of material is retained on the No. 200 sieve.	**Gravels** More than 50% coarse fraction retained on the No. 4 sieve	**Clean Gravels** < 5% Fines	**GW**	Well-graded gravels, gravel-sand mixtures, little or no fines	$C_U = D_{60} \div D_{10} \geq 4$ and $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1 \ \& \leq 3$	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	$C_U = D_{60} \div D_{10} < 4$ and/or $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) < 1 \ \& > 3$	
		Gravels with Fines >12% Fines	**GM**	Silty gravels, poorly graded gravel-sand-silt mixtures	Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol GC/GM
			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures	Fines classify as CL or CH	
	Sands More than 50% coarse fraction passes the No. 4 sieve	**Clean Sands** < 5% Fines	**SW**	Well-graded sands, gravelly sands, little or no fines	$C_U = D_{60} \div D_{10} \geq 6$ and $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) \geq 1 \ \& \leq 3$	
			SP	Poorly graded sands, gravelly sands, little or no fines	$C_U = D_{60} \div D_{10} < 6$ and/or $C_C = (D_{30})^2 \div (D_{10} \times D_{60}) < 1 \ \& > 3$	
		Sands with Fines >12% Fines	**SM**	Silty sands, poorly graded sand-silt mixtures	Fines classify as ML or MH	If fines classify as CL-ML, use dual symbol SC/SM
			SC	Clayey sands, poorly graded sand-clay mixtures	Fines classify as CL or CH	
Fine-Grained Soils More than 50% of material passes the No. 200 sieve.	**Identification Procedures on Percentage Passing the No. 40 Sieve**					
	Silts & Clays Liquid Limit less than 50%		**ML**	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands with slight plasticity		
			CL	Inorganic clays of low to medium plasticity, gravelly, sandy, and/or silty clays, lean clays		
			OL	Organic silts, organic silty clays of low plasticity		
	Silts & Clays Liquid Limit greater than 50%		**MH**	Inorganic silts, micaceous or diatomaceous fine sandy/-silty soil, elastic silts		
			CH	Inorganic clays of high plasticity, fat clays		
			OH	Organic clays of medium to high plasticity		
HIGHLY ORGANIC SOILS			**PT**	Peat and other highly organic soils		

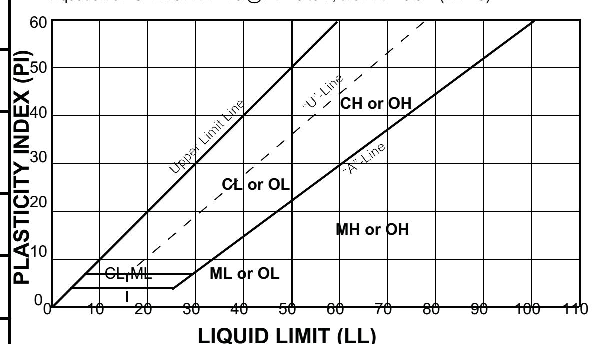
CLASSIFICATION OF GRAVELS & SANDS WITH 5% TO 12% FINES REQUIRES DUAL SYMBOLS

- GW/GM or GP/GM: Gravel/Silty Gravel
- GW/GC or GP/GC: Gravel/Clayey Gravel
- SW/SM or SP/SM: Sand/Silty Sand
- SW/SC or SP/SC: Sand/Clayey Sand

PLASTICITY CHART

For Classification of Fine-Grained Soils and Fine-Grained Fraction of Coarse-Grained Soils

Equation of "A"-Line: PI = 4 @ LL = 4 to 25.5, then PI = 0.73 × (LL − 20)
Equation of "U"-Line: LL = 16 @ PI = 0 to 7, then PI = 0.9 × (LL − 8)



KEY TO SAMPLER TYPES AND OTHER LOG SYMBOLS

CS	California Standard Sampler
CM	California Modified Sampler
SPT	Standard Penetration Test Sampler
SHL	Shelby Tube Sampler
BU	Bulk Sample
LL	Liquid Limit of Sample (ASTM D-4318)
PI	Plasticity Index of Sample (ASTM D-4318)
Q_U	Unconfined Compression Test (ASTM D-2166)
▽	Depth at which Groundwater was Encountered During Drilling
▼	Depth at which Groundwater was Measured After Drilling
PP	Pocket Penetrometer Test
PTV	Pocket Torvane Test
−#200	% of Material Passing the No. 200 Sieve Test (ASTM D-1140)
PSA	Particle-Size Analysis (ASTM D-422 & D-1140)
C	Consolidation Test (ASTM D-2435)
TXUU	Unconsolidated Undrained Compression Test (ASTM D-2850)

KEY TO SAMPLE INTERVALS

CS	Length of Sampler Interval with a CS Sampler
☒	Length of Sampler Interval with a CM Sampler
	Length of Sampler Interval with a SPT Sampler
SHL	Length of Sampler Interval with a SHL Sampler
BU	Bulk Sample Recovered for Interval Shown (i.e., cuttings)
	Length of Coring Run with Core Barrel Type Sampler
NR	No Sample Recovered for Interval Shown

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UNIFIED SOIL CLASSIFICATION SYSTEM AND KEY TO BORING LOG

Rock Hardness Descriptions

Very Hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimen requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately Hard	Can be scratched with knife or pick. Gouges or grooves to 1/4-inch deep can be excavated by hard blow of geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16-inch deep by firm pressure of knife or pick point. Can be excavated in small chips to pieces about 1-inch maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small tin pieces can be broken by finger pressure.
Very Soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-inch or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Bedding Thickness & Joint/Fracture Spacing Descriptions

Centimeters	Inches	Bedding	Joints/Fractures
< 2	< ¾	Laminated	Extremely Close
2-5	¾-2	Very Thin	Very Close
5-30	2-12	Thin	Close
30-90	12-36	Medium	Moderate
90-300	36-120	Thick	Wide
> 300	> 120	Very Thick	Very Wide

Rock Weathering Descriptions

Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very Slight	Rock generally fresh, joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dulled and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately Severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very Severe	All rock except quartz discolored or stained. Rock "fabric" discernible. But mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil." Rock "fabric" not discernible or discernible only in small scattered locations. Quartz may be present as dikes or stringers.

The above Bedrock Characteristics are based on the ASCE Manual No. 56, "Subsrface Investigation For Design And Construction Of Foundations Of Buildings," 1976.

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Bedrock Characteristics Chart

KEY TO SYMBOLS

CAL ENGINEERING & GEOLOGY

CLIENT SCC Veterans Memorial Building Board
PROJECT NAME Veterans Village Improvements Project
PROJECT NUMBER 220300
PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

ASPHALT: Asphalt

BEDROCK: Bedrock

CL: USCS Low Plasticity Clay

GC: USCS Clayey Gravel

GW-GC: USCS Well-graded Gravel with Clay

ML: USCS Silt

SANDSTONE: Sandstone

TOPSOIL: Topsoil

SAMPLER SYMBOLS

California Modified Sampler

Grab Sample

Standard Penetration Test

WELL CONSTRUCTION SYMBOLS

ABBREVIATIONS

LL	- LIQUID LIMIT (%)
PI	- PLASTIC INDEX (%)
W	- MOISTURE CONTENT (%)
DD	- DRY DENSITY (PCF)
NP	- NON PLASTIC
-200	- PERCENT PASSING NO. 200 SIEVE
PP	- POCKET PENETROMETER (TSF)
TV	- TORVANE
PID	- PHOTOIONIZATION DETECTOR
UC	- UNCONFINED COMPRESSION
ppm	- PARTS PER MILLION

▽ Water Level at Time Drilling, or as Shown

▼ Water Level at End of Drilling, or as Shown

▼ Water Level After 24 Hours, or as Shown

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BORING NUMBER B-1

CLIENT SCC Veterans Memorial Building Board
PROJECT NAME Veterans Village Improvements Project
PROJECT NUMBER 220300
PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA
DATE STARTED 4/11/2022 **COMPLETED** 4/11/2022
GROUND ELEVATION 405 ft **DATUM** NAVD88 **HOLE SIZE** 6" in.
DRILLING CONTRACTOR Cenozoic Exploration, LLC.
COORDINATES: LATITUDE 37.08143 **LONGITUDE** -122.08572
DRILLING RIG/METHOD Simco 2400/ 6-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING --- Not Encountered
LOGGED BY K. Loeb **CHECKED BY** D. Peluso
GROUNDWATER AT END OF DRILLING --- n/a
HAMMER TYPE 140 lb hammer with 30 in. cathead
GROUNDWATER AFTER DRILLING --- n/a

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0		Sandy Lean CLAY (CL): very dark grayish brown, moist, stiff, fine sand [Artificial Fill]									
			GB								
2.5											
		becomes dark yellowish brown, medium dense, trace angular gravel (up to 0.25")	GB				22	34	20	14	52
5.0											
		Sandy Lean CLAY w/ Gravel (CL): dark grayish brown, moist, medium dense/hard, fine sand, some oxidation [Quaternary Colluvium] Corrosion Test at 6.5 feet TXUU at 7 feet	CM	7-10-15		89	16 29				
7.5		Lean CLAY w/ Sand (CL): dark olive brown, moist, hard, low plasticity, subangular gravel (up to 2"), some oxidized pockets	SPT	6-12-19	>4.5 >4.5						
		Sandy SILTSTONE: dark grayish brown, moist, highly weathered, extremely weak, fine-grained sand, some oxidized pockets, some roots [Monterey Formation Bedrock]									
10.0											
		Silty SANDSTONE: light yellowish brown, damp, moderately to highly weathered, extremely weak, fine-grained sand, trace pea sized gravel, some roots and oxidized pockets									
12.5			CM	24-35-50/4"		98	19				
			SPT	17-23-29							
15.0											

(Continued Next Page)

BORING NUMBER B-1

CAL ENGINEERING & GEOLOGY

CLIENT SCC Veterans Memorial Building Board
PROJECT NAME Veterans Village Improvements Project
PROJECT NUMBER 220300
PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
15.0		Silty SANDSTONE: light yellowish brown, damp, moderately to highly weathered, extremely weak, fine-grained sand, trace pea sized gravel, some roots and oxidized pockets *(continued)*									
17.5											
		becomes fully oxidized along contact	SPT	12-15-21							
20.0		SANDSTONE: dark olive gray, moist, slightly weathered, extremely weak, no cementation, some clayey and silty pockets, more characteristic of a silty sand soil rather than rock									
		drilling difficulty increased at 22 feet									
22.5		CONGLOMERATE: various colors, dry, slightly weathered, very weak, matrix supported with fine- to coarse-grained granitic sand, subrounded gravel (up to 1.5"), oxidized, no cementation									
25.0			SPT	16-21-21							
27.5											
			SPT	14-19-25							
30.0		SANDSTONE: dark yellowish brown, extremely weak, fine-grained, oxidized									

Bottom of borehole at 30.0 ft. Borehole backfilled with neat cement grout.

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CAL ENGINEERING & GEOLOGY

CLIENT SCC Veterans Memorial Building Board
PROJECT NAME Veterans Village Improvements Project
PROJECT NUMBER 220300
PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA
DATE STARTED 4/11/2022 **COMPLETED** 4/11/2022
GROUND ELEVATION 439 ft **DATUM** NAVD88 **HOLE SIZE** 6" in.
DRILLING CONTRACTOR Cenozoic Exploration, LLC.
COORDINATES: LATITUDE 37.08128 **LONGITUDE** -122.08605
DRILLING RIG/METHOD Simco 2400/ 6-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING --- Not Encountered
LOGGED BY K. Loeb **CHECKED BY** D. Peluso
GROUNDWATER AT END OF DRILLING --- n/a
HAMMER TYPE 140 lb hammer with 30 in. cathead
GROUNDWATER AFTER DRILLING --- n/a

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0		Asphalt Pavement (approximately 1 to 2 inches)									
		Sandy Lean CLAY (CL): dark yellowish brown, dry, stiff, fine sand [Artificial Fill]	GB								
2.5		Sandy SILT w/ Gravel (ML): dark yellowish brown, dry to moist, medium dense, fine sand, angular gravel [Quaternary Colluvium]	GB				20				48
		Silty SANDSTONE: yellowish brown to dark yellowish brown, dry to moist, moderately weathered, very weak, fine-grained sand, some oxidized layers [Monterey Formation Bedrock]									
5.0			CM	13-16-35		88	25				
			SPT	7-15-20							
7.5											
10.0											
12.5		becomes highly to moderately weathered, extremely weak, oxidized	CM	11-16-36		91	26				
			SPT	10-11-15							
15.0		becomes light brownish gray, some clayey pockets									

(Continued Next Page)

BORING NUMBER B-2

CAL ENGINEERING & GEOLOGY

CLIENT SCC Veterans Memorial Building Board

PROJECT NAME Veterans Village Improvements Project

PROJECT NUMBER 220300

PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
15.0		Silty SANDSTONE: yellowish brown to dark yellowish brown, dry to moist, moderately weathered, very weak, fine-grained sand, some oxidized layers [Monterey Formation Bedrock] *(continued)*									
17.5											
		becomes slightly weathered, very weak	SPT	12-16-28							
20.0											

Bottom of borehole at 20.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-3

CE&G
CAL ENGINEERING & GEOLOGY

CLIENT SCC Veterans Memorial Building Board
PROJECT NAME Veterans Village Improvements Project
PROJECT NUMBER 220300
PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA
DATE STARTED 4/11/2022 **COMPLETED** 4/11/2022
GROUND ELEVATION 382 ft **DATUM** NAVD88 **HOLE SIZE** 6" in.
DRILLING CONTRACTOR Cenozoic Exploration, LLC.
COORDINATES: LATITUDE 37.081252 **LONGITUDE** -122.085067
DRILLING RIG/METHOD Simco 2400/ 6-in. Solid Flight Auger
GROUNDWATER AT TIME OF DRILLING --- Not Encountered
LOGGED BY K. Loeb **CHECKED BY** D. Peluso
GROUNDWATER AT END OF DRILLING --- n/a
HAMMER TYPE 140 lb hammer with 30 in. cathead
GROUNDWATER AFTER DRILLING --- n/a

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0	Topsoil									
	Lean CLAY (CL): very dark brown, moist, medium plasticity, medium stiff, silty, trace roots [Artificial Fill/Altered Ground]	GB		0.75		29	39	22	17	
2.5	Sandy SILT w/ Gravel (ML): very dark grayish brown, moist, medium plasticity, stiff, fine sand, subangular monterey formation gravel (up to 1.5"), some oxidized gravels [Quaternary Alluvium]	GB				35	49	29	20	41
5.0	increase in sand									
	increase in gravel , low plasticity, some oxidized pockets 2.5" rock fragment at 6.5 feet	CM	10-15-29	1.5 3.0	87	33				
7.5	Sandy Lean CLAY (CL): very dark gray, moist, low plasticity, stiff, oxidized pockets, trace angular pea-sized gravel, some manganese staining on coarser materials	SPT	5-8-10	2.5						
10.0	decrease in manganese staining, little oxidations, few subangular monterey formation gravel (up to 1")	CM	8-13-13	1.5	84	36				
12.5	Sandy Lean CLAY w/ Gravel (CL): dark olive brown, moist, medium stiff, fine to coarse sand, subangular gravel (up to 1.25"), some black staining, some oxidized fragments	SPT	6-6-9	1.25						
	Lean CLAY (CL): olive brown, moist, medium plasticity, medium stiff, silty									
15.0										

Bottom of borehole at 15.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-4

CE&G — CAL ENGINEERING & GEOLOGY

CLIENT SCC Veterans Memorial Building Board
PROJECT NAME Veterans Village Improvements Project
PROJECT NUMBER 220300
PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA
DATE STARTED 4/11/2022 **COMPLETED** 4/11/2022
GROUND ELEVATION 398 ft **DATUM** NAVD88 **HOLE SIZE** 6" in.
DRILLING CONTRACTOR Cenozoic Exploration, LLC.
COORDINATES: LATITUDE 37.081238 **LONGITUDE** -122.085527
DRILLING RIG/METHOD Simco 2400/ 6-in. Solid Flight Auger
▽ **GROUNDWATER AT TIME OF DRILLING** 6.3 ft / Elev 391.8 ft
LOGGED BY K. Loeb **CHECKED BY** D. Peluso
▼ **GROUNDWATER AT END OF DRILLING** 6.0 ft / Elev 392.0 ft
HAMMER TYPE 140 lb hammer with 30 in. cathead
GROUNDWATER AFTER DRILLING --- Not Measured

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0		Sandy SILT (ML): Dark yellowish brown, moist, medium dense [Artificial Fill]									
2.5		Monterey Formation cobble fragments									
		Gravelly Lean CLAY (CL): very dark brown clay with Monterey Formation sandstone gravel, moist, some roots, black woody debris [Quaternary Alluvium]	CM	8-8-7							
5.0		Sandy SILT w/ Gravel (ML): very dark brown to black, moist, soft/loose, fine sand, angular to subangular gravel (up to 2"), some roots and charcoal	CM	4-3-3							
	▼ ▽	Clayey GRAVEL w/ Sand (GC): very dark brown, wet, loose, angular gravel and cobble fragments, fine to coarse sand, trace charcoal fragments	CM	3-2-3							
7.5		Well Graded GRAVEL w/ Clay and Sand (GW-GC): very dark brown, wet/muddy fines, medium dense, angular gravel (up to 3"), cobble fragments	CM	4-4-10							
			CM	8-9-6							
10.0		Highly weathered sandstone cobble									

Bottom of borehole at 10.5 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-5

CAL ENGINEERING & GEOLOGY

CLIENT SCC Veterans Memorial Building Board
PROJECT NAME Veterans Village Improvements Project
PROJECT NUMBER 220300
PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA
DATE STARTED 4/11/2022 **COMPLETED** 4/11/2022
GROUND ELEVATION 398 ft **DATUM** NAVD88 **HOLE SIZE** 3" in.
DRILLING CONTRACTOR N/a
COORDINATES: LATITUDE 37.08129 **LONGITUDE** -122.08553
DRILLING RIG/METHOD Hand Augered by CE&G Staff
GROUNDWATER AT TIME OF DRILLING --- Not Encountered
LOGGED BY K. Loeb **CHECKED BY** D. Peluso
GROUNDWATER AT END OF DRILLING --- n/a
HAMMER TYPE n/a
GROUNDWATER AFTER DRILLING --- n/a

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0		Sandy Lean CLAY (CL): very dark grayish brown, moist, medium dense, fine sand [Artificial Fill]	GB								
			GB								
2.5			GB								

Bottom of borehole at 3.0 ft. Borehole backfilled with neat cement grout.

BORING NUMBER B-6

CE&G
CAL ENGINEERING & GEOLOGY

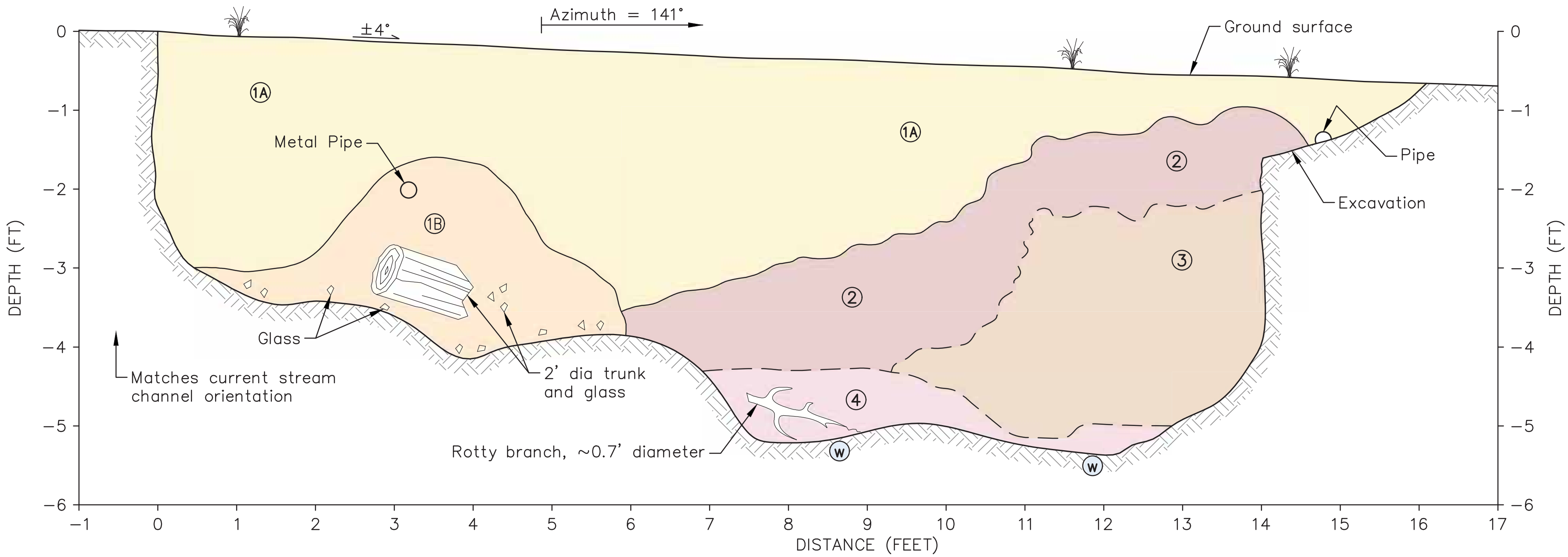
CLIENT SCC Veterans Memorial Building Board
PROJECT NAME Veterans Village Improvements Project
PROJECT NUMBER 220300
PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA
DATE STARTED 4/11/2022 **COMPLETED** 4/11/2022
GROUND ELEVATION 393 ft **DATUM** NAVD88 **HOLE SIZE** 3" in.
DRILLING CONTRACTOR N/a
COORDINATES: LATITUDE 37.0813 **LONGITUDE** -122.08544
DRILLING RIG/METHOD Hand Augered by CE&G Staff
GROUNDWATER AT TIME OF DRILLING --- Not Encountered
LOGGED BY K. Loeb **CHECKED BY** D. Peluso
GROUNDWATER AT END OF DRILLING --- n/a
HAMMER TYPE n/a
GROUNDWATER AFTER DRILLING --- n/a

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	BLOW COUNTS (FIELD VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTIC LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	FINES CONTENT (%)
0.0		Sandy Lean CLAY w/ Gravel (CL): dark yellowish brown, dry to moist, medium dense, fine sand, angular gravel [Artificial Fill/Quaternary Colluvium]									
			GB								
			GB								
2.5											
			GB								

Bottom of borehole at 3.0 ft. Borehole backfilled with neat cement grout.

Appendix D. Test Pit Log

M:\2022\220300-SantaCruzCountyVeterans-VillageHousing\AutoCAD\Figures\AppendixD-TesPitLog.dwg 5-02-22 12:08:47 PM kdrozynska



UNIT DESCRIPTION

(1A) Sandy Silt (ML): mottled-olive (5y, 5/3) to olive gray (5y, 5/2), moist, low plasticity, fine sand (approximately 35%), trace angular pea-sized gravel, no cementation, no HCl reaction, trace rootlets, no bedding observed, clasts with relict bedrock, moderately sorted [Artificial Fill/Road Base]

(1B) Silty Sand w/ Gravel (ML): very dark grayish brown (10YR, 3/2), dry to moist, loose, fine- to coarse-grained sand, approximately 15% fin-to coarse-grained gravel (subangular to subrounded), low plasticity, no cementation, no visible bedding, poorly sorted, large (2-foot-diameter) root ball/trunk, some debris (e.g., bricks, glass bottles and fragments, and metal pipe) [Artificial Channel Fill]

(2) Sandy Silt w/ Gravel (ML): very dark brown (10YR, 2/2), moist, low plasticity, fine-grained sand (approximately 25%), fine- to coarse-grained angular to subrounded gravel, subrounded and angular cobbles up to 5", some rootlets and multiple 4" diameter roots, no bedding, poorly sorted, no cementation, increase in gravel closer to stream invert, increase in angular clasts at base of unit [Channel + Debris Flow Deposits?]

(3) Sandy Silt w/ Gravel (ML): very dark brown (10YR, 2/2), moist, low plasticity, fine- to medium-grained sand (approximately 20%), fine- to coarse-grained angular to subangular gravel from local bedrock (approximately 10%), some rootlets and other organics, no cementation, no HCl reaction, no bedding, poorly sorted [Colluvium]

(4) Lean Clay w/ Gravel (CL): black (5Y, 7.5/1), moist to wet, low to medium plasticity, pea-sized gravel up to 0.75" (approximately 15%), some subangular clasts, little fine-grained sand and silt, no cementation, no HCl reaction, no bedding observed, poorly sorted, occasional 3" to 4" cobbles, high rootlet content, some woody debris, upper surface of unit lined with thin charcoal layer [Stream Channel Deposits]

LEGEND

- Ground surface
- (W) Standing Water
- Base excavation
- Clear boundary
- Gradual boundary
- Obscured boundary

NOTE

1. Test Pit logged by K. Loeb and K. Krug on 4/13/2022.

SCALE 1 0 1 2 FEET

CE&G
Cal Engineering & Geology
6455 Almaden Expwy.
Suite 100
San Jose, CA 95120
Phone: (408) 440-4542

VETERANS VILLAGE HOUSING PROJECT
8705 HIGHWAY 9
BEN LOMOND, CALIFORNIA

LOG OF TEST PIT TP-1

220300	MAY 2022	APPENDIX D

Appendix E. Laboratory Testing

SUMMARY OF LABORATORY RESULTS

CE&G
CAL ENGINEERING & GEOLOGY

CLIENT SCC Veterans Memorial Building Board

PROJECT NAME Veterans Village Improvements Project

PROJECT NUMBER 220300

PROJECT LOCATION 8705 HWY 9, Ben Lomond, CA

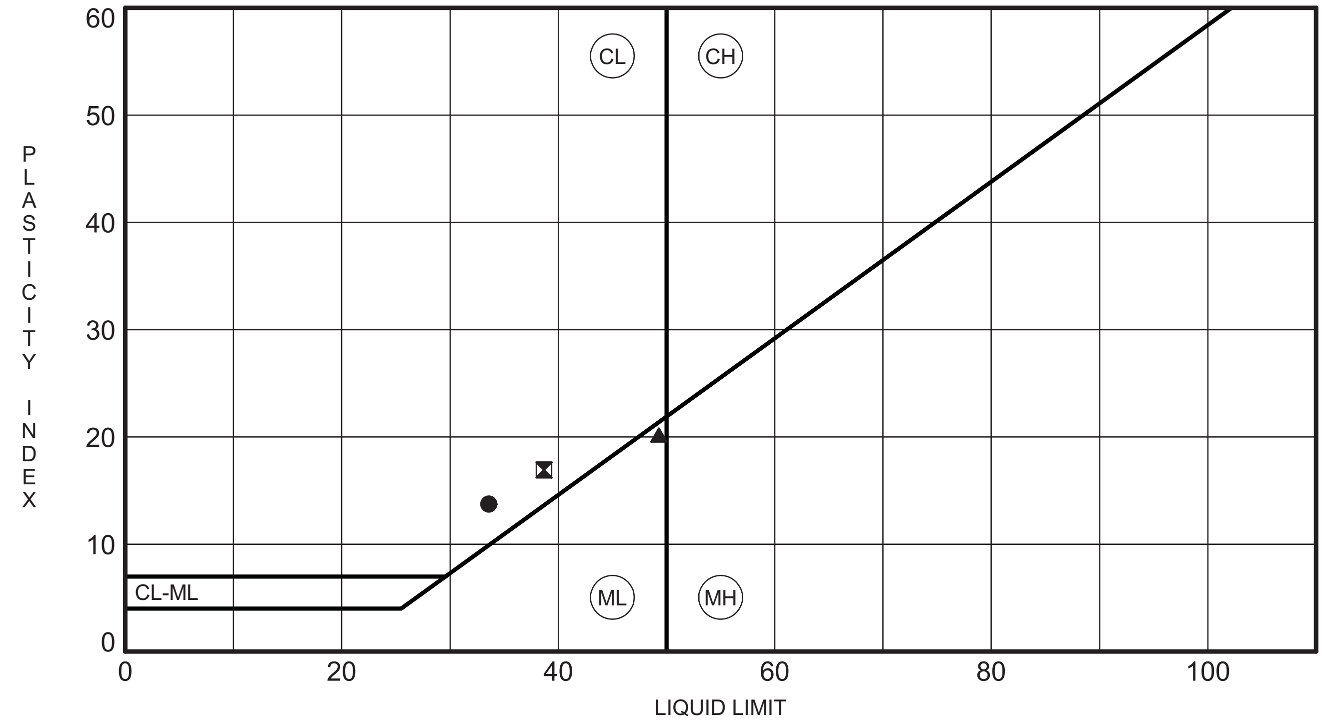
Borehole	Depth	Date Tested	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Screen Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-1	4.0	4/15/2022	34	20	14	0.106	52	CL	22.2			
B-1	13.0	4/15/2022							19.1	97.5		
B-2	2.5	4/15/2022				0.106	48		20.4			
B-2	5.5	4/15/2022							25.4	88.1		
B-2	13.0	4/15/2022							26.5	91.2		
B-3	1.0	4/15/2022	39	22	17				28.7			
B-3	3.5	4/15/2022	49	29	20	0.106	41	SM	34.8			
B-3	7.0	4/15/2022							33.1	87.3		
B-3	11.0	4/15/2022							36.2	83.7		

ATTERBERG LIMITS RESULTS

CE&G
Cal Engineering & Geology

CLIENT SCC Veterans Memorial Building Board **PROJECT NAME** Veterans Village Improvements Project

PROJECT NUMBER 220300 **PROJECT LOCATION** 8705 HWY 9, Ben Lomond, CA



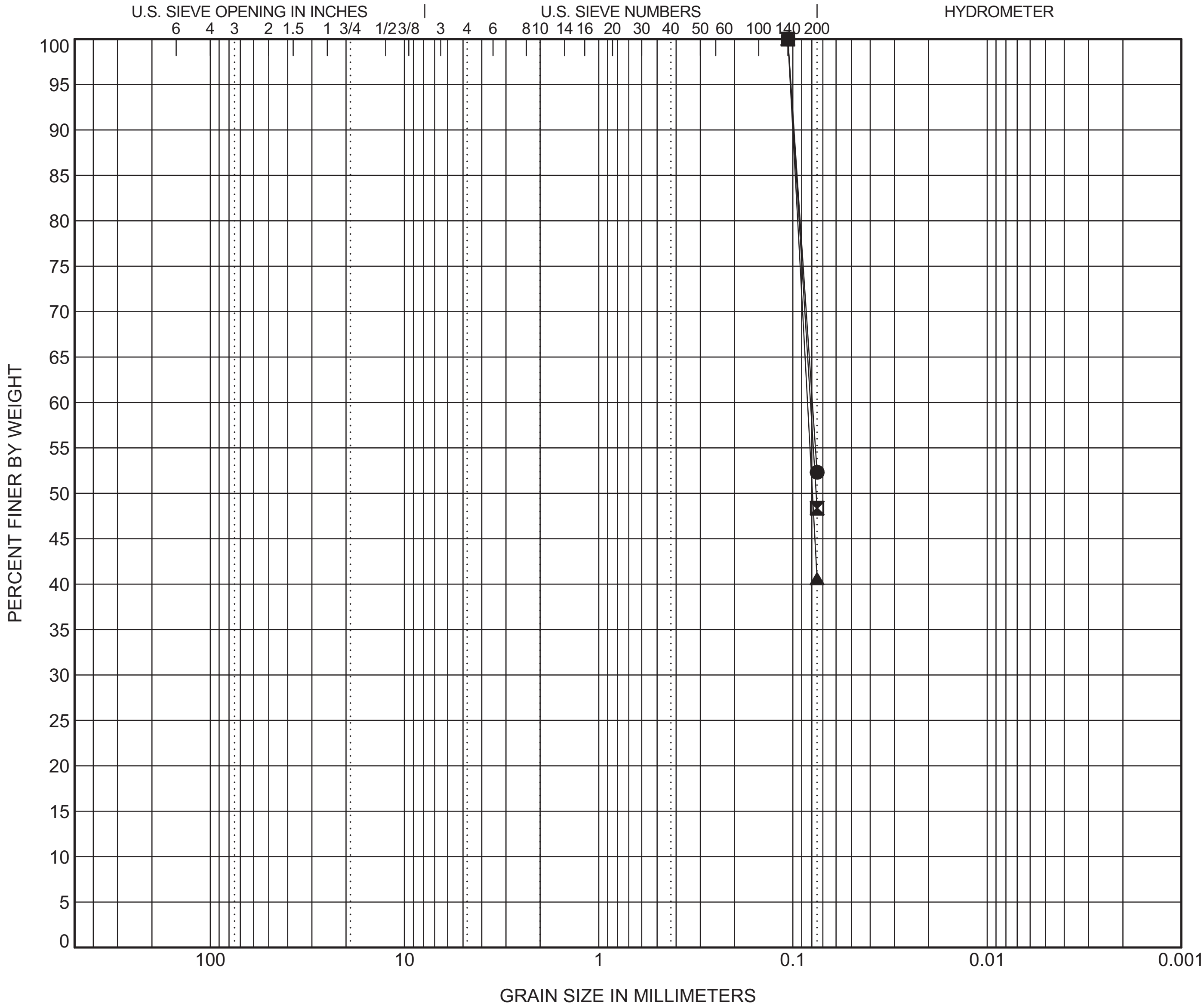
	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification	Date Tested
●	**B-1**	**4.0**	**34**	**20**	**14**	**52**	**SANDY LEAN CLAY(CL)**	**4/15/2022**
⊠	**B-3**	**1.0**	**39**	**22**	**17**			**4/15/2022**
▲	**B-3**	**3.5**	**49**	**29**	**20**	**41**	**SILTY SAND(SM)**	**4/15/2022**

GRAIN SIZE DISTRIBUTION

CAL ENGINEERING & GEOLOGY

CLIENT SCC Veterans Memorial Building Board **PROJECT NAME** Veterans Village Improvements Project

PROJECT NUMBER 220300 **PROJECT LOCATION** 8705 HWY 9, Ben Lomond, CA



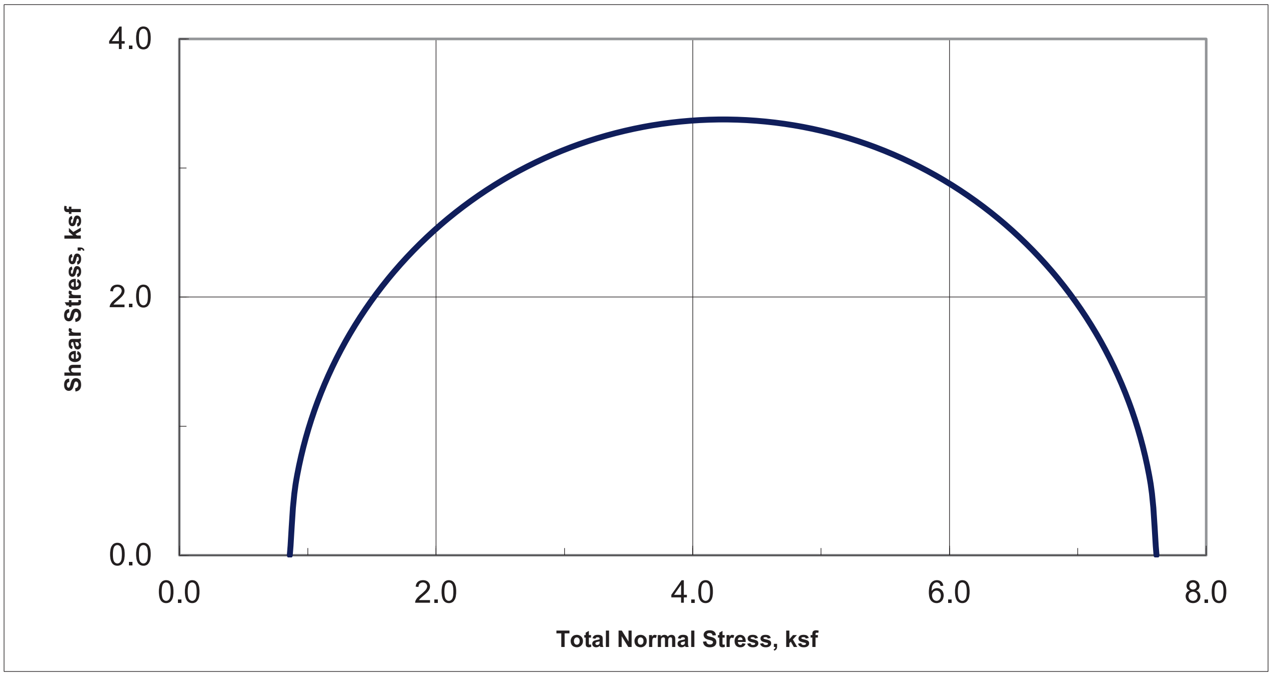
COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

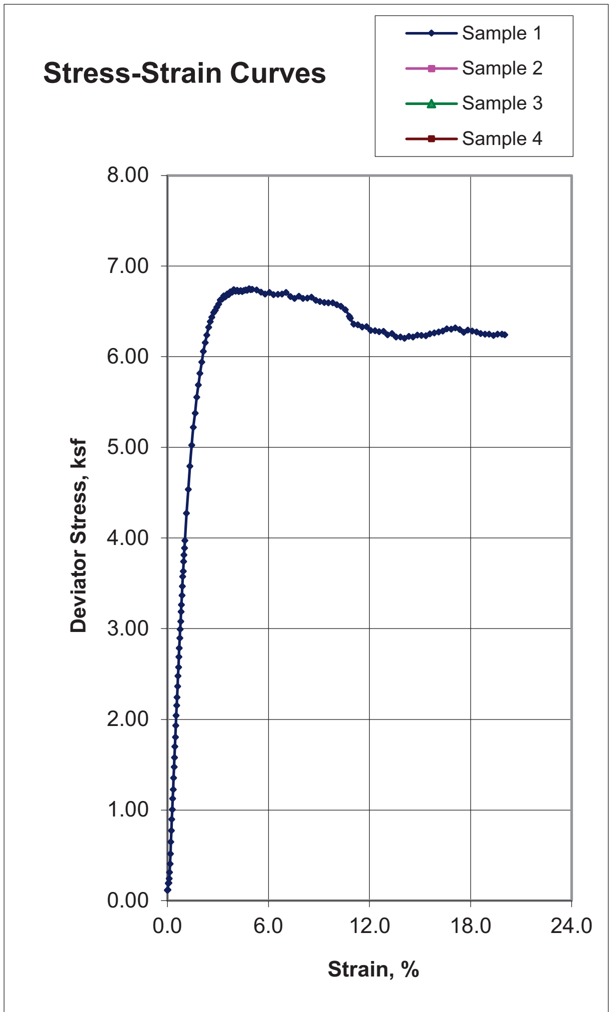
	BOREHOLE	DEPTH	DATE TESTED	Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**4.0**	**4/15/2022**	**SANDY LEAN CLAY(CL)**	**34**	**20**	**14**		
⊠	**B-2**	**2.5**	**4/15/2022**						
▲	**B-3**	**3.5**	**4/15/2022**	**SILTY SAND(SM)**	**49**	**29**	**20**		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**4.0**	**0.106**	**0.079**			**0.0**	**47.7**	**52.3**	
⊠	**B-2**	**2.5**	**0.106**	**0.081**			**0.0**	**51.6**	**48.4**	
▲	**B-3**	**3.5**	**0.106**	**0.084**			**0.0**	**59.4**	**40.6**	

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test
ASTM D2850





Sample Data	1	2	3	4
Moisture %	28.4			
Dry Den,pcf	89.2			
Void Ratio	0.890			
Saturation %	86.1			
Height in	5.01			
Diameter in	2.40			
Cell psi	6.0			
Strain %	4.84			
Deviator, ksf	6.752			
Rate %/min	1.00			
in/min	0.050			
Job No.:	471-377			
Client:	Cal Engineering & Geology			
Project:	220300			
Boring:	B-1			
Sample:	1-4			
Depth ft:	7			

Visual Soil Description

Sample #	
1	Yellowish Brown Sandy CLAY
2	
3	
4	

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL # 471-377 **Date:** 4/18/2022 **Tested By:** PJ **Checked:** PJ

Client: Cal Engineering & Geology **Project:** Veterans Village **Proj. No:** 220300

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
B-1	1-3	6.5	-	3906	-	5	12	0.0012	4.8	-	16.1	Dark Yellowish Brown Clayey SAND w/ Gravel, trace organics

Appendix F. Slope Stability Analysis

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Latitude

Decimal degrees

37.081252

Longitude

Decimal degrees, negative values for western longitudes

-122.085067

Site Class

259 m/s (Site class D)

Spectral Period

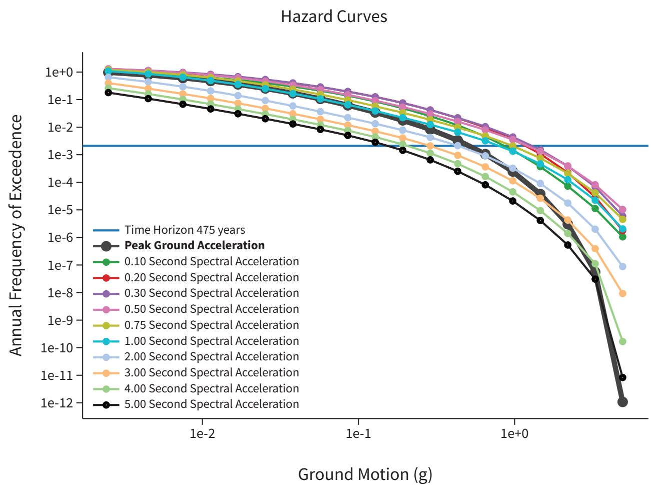
Peak Ground Acceleration

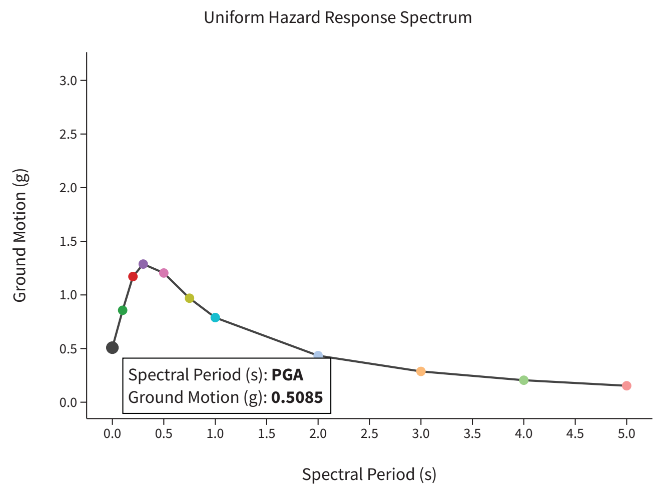
Time Horizon

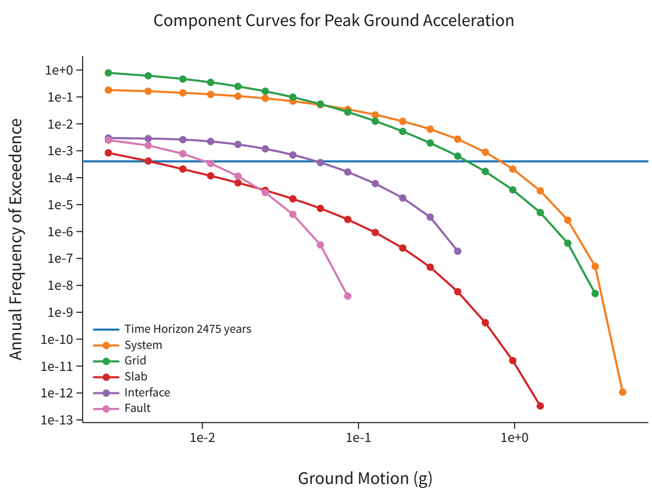
Return period in years

475

Hazard Curve





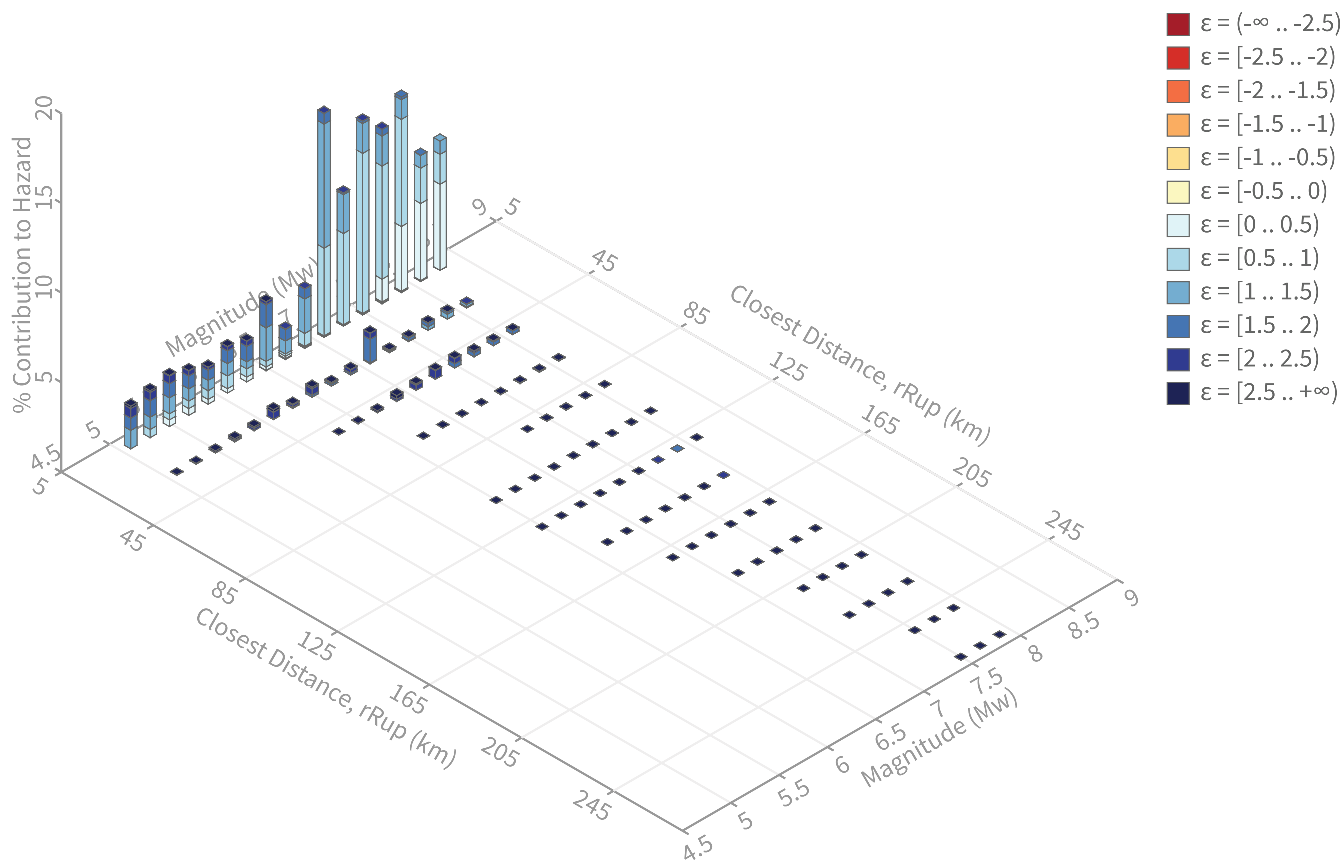


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.50845131 g

Recovered targets

Return period: 520.0006 yrs
Exceedance rate: 0.0019230747 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.12 %

Mean (over all sources)

m: 7.17
r: 14.51 km
ε_0: 1.04 σ

Mode (largest m-r bin)

m: 7.09
r: 13.97 km
ε_0: 1.08 σ
Contribution: 12.48 %

Mode (largest m-r-ε_0 bin)

m: 7.52
r: 12.68 km
ε_0: 0.72 σ
Contribution: 8.92 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							41.29
San Andreas (Santa Cruz Mts) [0]		12.51	7.70	0.70	121.993°W	37.169°N	39.64	22.68
San Gregorio (North) [19]		15.84	7.48	1.11	122.259°W	37.049°N	257.12	4.40
San Andreas (Santa Cruz Mts) [1]		13.72	7.07	1.07	121.943°W	37.134°N	64.93	2.45
San Andreas (Santa Cruz Mts) [2]		17.72	7.07	1.36	121.884°W	37.093°N	85.80	1.51
Butano [1]		11.03	7.49	0.77	122.012°W	37.161°N	36.27	1.25
UC33brAvg_FM32	System							41.16
San Andreas (Santa Cruz Mts) [0]		12.51	7.70	0.70	121.993°W	37.169°N	39.64	23.54
San Gregorio (North) [19]		15.84	7.50	1.10	122.259°W	37.049°N	257.12	4.27
San Andreas (Santa Cruz Mts) [1]		13.72	7.08	1.07	121.943°W	37.134°N	64.93	2.55
San Andreas (Santa Cruz Mts) [2]		17.72	7.09	1.35	121.884°W	37.093°N	85.80	1.55
Butano [1]		11.03	7.55	0.74	122.012°W	37.161°N	36.27	1.12
UC33brAvg_FM31 (opt)	Grid							8.91
PointSourceFinite: -122.085, 37.104		5.75	5.59	0.82	122.085°W	37.104°N	0.00	1.23
PointSourceFinite: -122.085, 37.104		5.75	5.59	0.82	122.085°W	37.104°N	0.00	1.23
UC33brAvg_FM32 (opt)	Grid							8.64
PointSourceFinite: -122.085, 37.104		5.62	5.65	0.78	122.085°W	37.104°N	0.00	1.64
PointSourceFinite: -122.085, 37.104		5.62	5.65	0.78	122.085°W	37.104°N	0.00	1.64

where NRF is a factor that accounts for the nonlinear response of the materials above the slide plane; u is displacement; and D_{5-95} is the duration of strong shaking, a function of earthquake magnitude and distance.

Blake and others (2002) have simplified the process of estimating f_{eq} for ranges of magnitude and distance by preparing sets of curves for two displacement (u) values, 5 cm and 15 cm. These curves are reproduced in Figure 1.

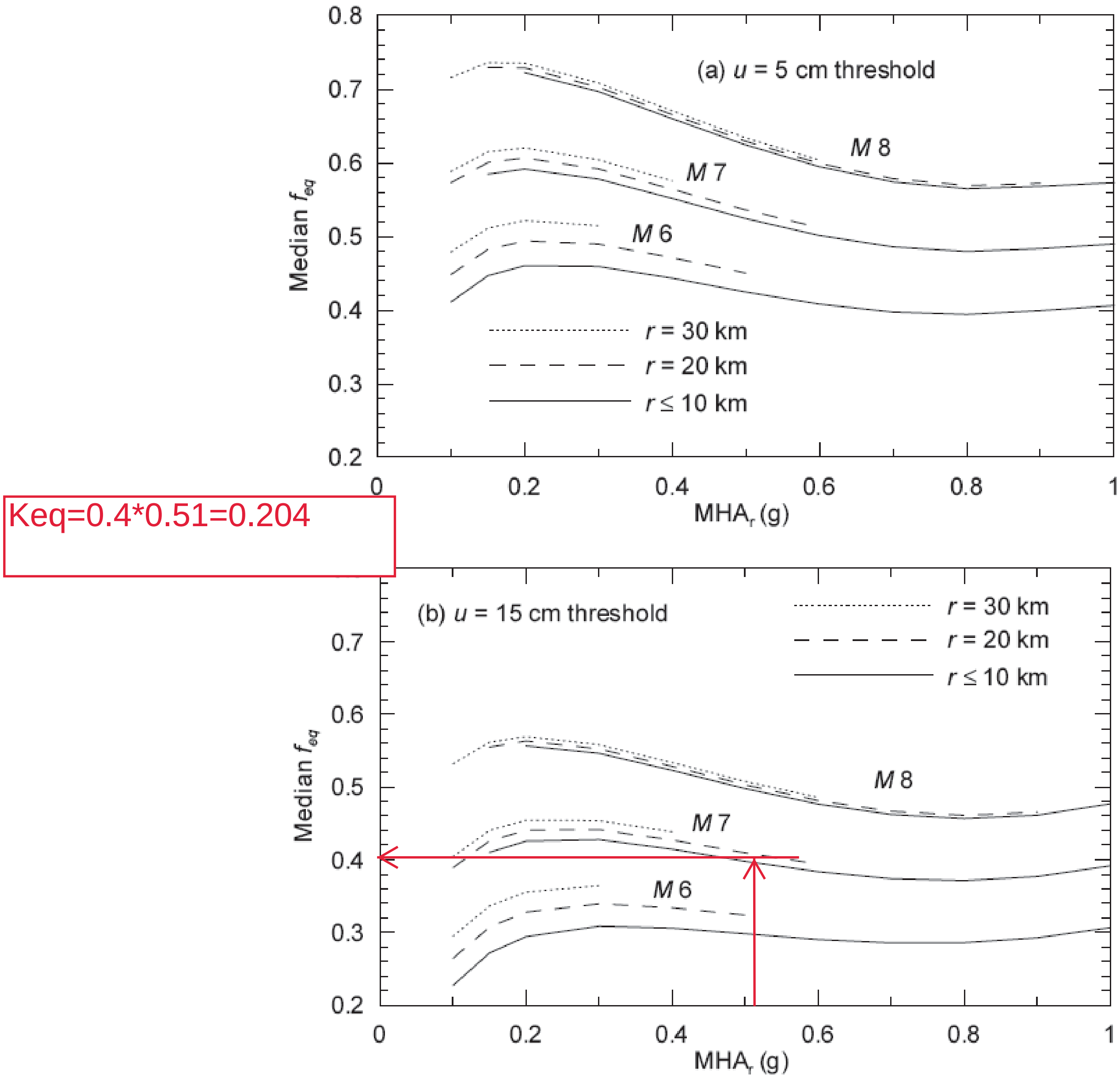
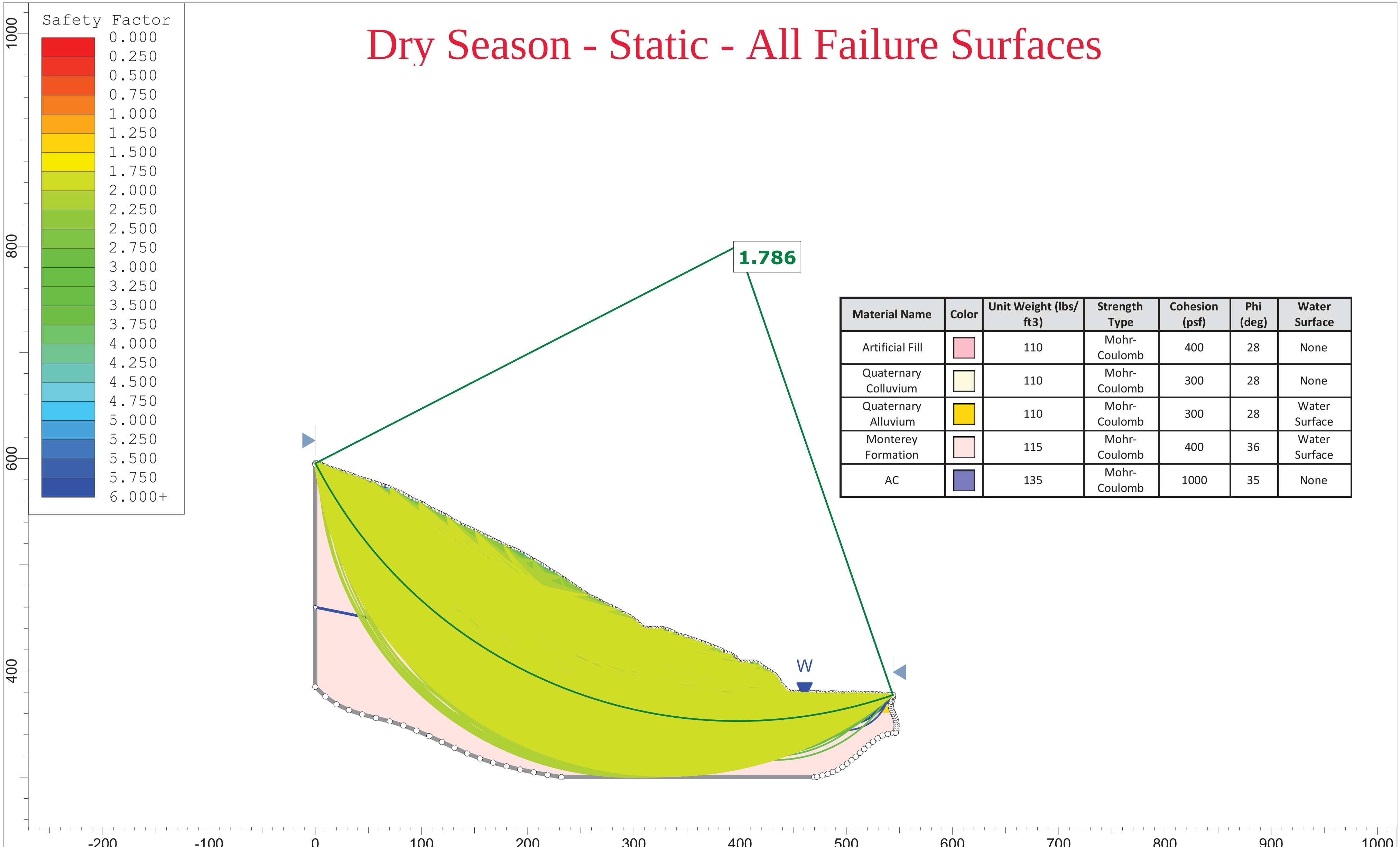


Figure 1. Values of f_{eq} as a Function of MHA_r, Magnitude and Distance for Threshold Displacements of (a) 5 cm and (b) 15 cm (Modified from Blake and others, 2002).

Dry Season - Static - All Failure Surfaces



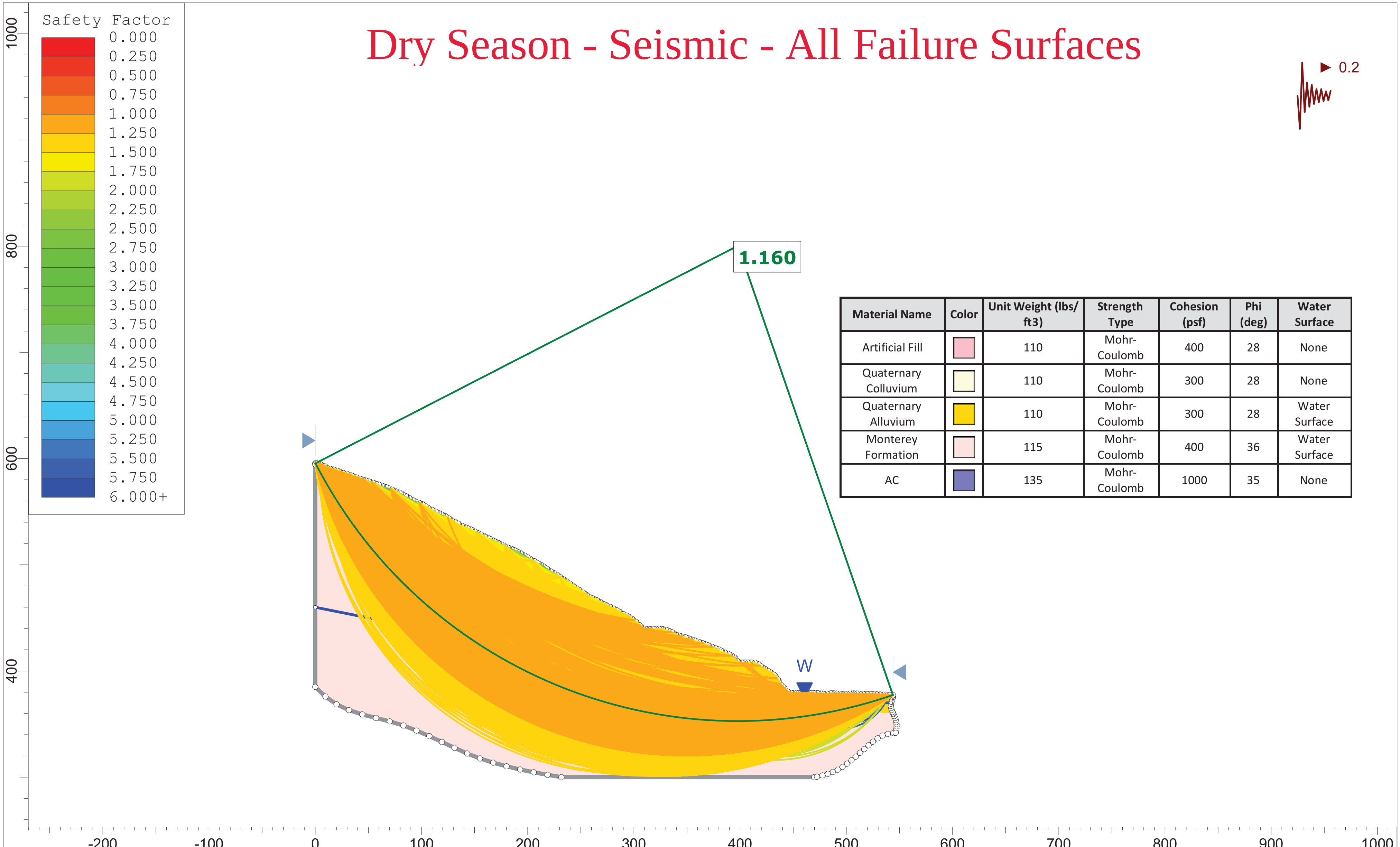
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Artificial Fill		110	Mohr-Coulomb	400	28	None
Quaternary Colluvium		110	Mohr-Coulomb	300	28	None
Quaternary Alluvium		110	Mohr-Coulomb	300	28	Water Surface
Monterey Formation		115	Mohr-Coulomb	400	36	Water Surface
AC		135	Mohr-Coulomb	1000	35	None

CE&G Pragmatic Expertise™

SLIDEINTERPRET 9.019

Project	Veterans Village at Santa Cruz County		
Cross Section	AA'	*Scenario*	Static
Drawn by	KF	*Company*	CE&G
Date	4/26/2022 1:55:03 PM	*File Name*	SectionAA'.slmd

Dry Season - Seismic - All Failure Surfaces



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Artificial Fill		110	Mohr-Coulomb	400	28	None
Quaternary Colluvium		110	Mohr-Coulomb	300	28	None
Quaternary Alluvium		110	Mohr-Coulomb	300	28	Water Surface
Monterey Formation		115	Mohr-Coulomb	400	36	Water Surface
AC		135	Mohr-Coulomb	1000	35	None

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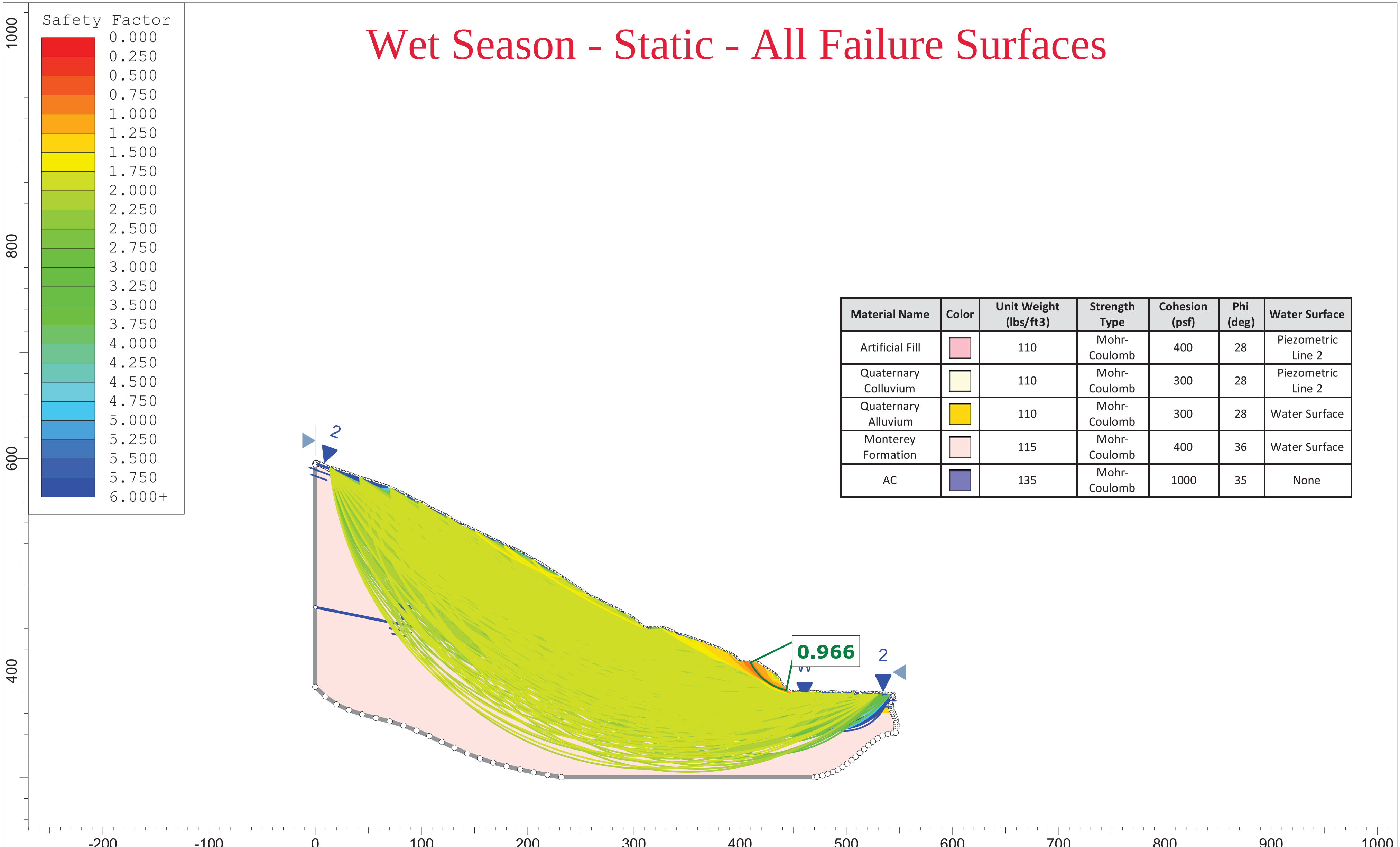
SLIDEINTERPRET 9.019

Project	Veterans Village at Santa Cruz County		
Cross Section	AA'	*Scenario*	Seismic
Drawn by	KF	*Company*	CE&G
Date	4/26/2022 1:55:04 PM	*File Name*	SectionAA'.slmd

Wet Season - Static - All Failure Surfaces



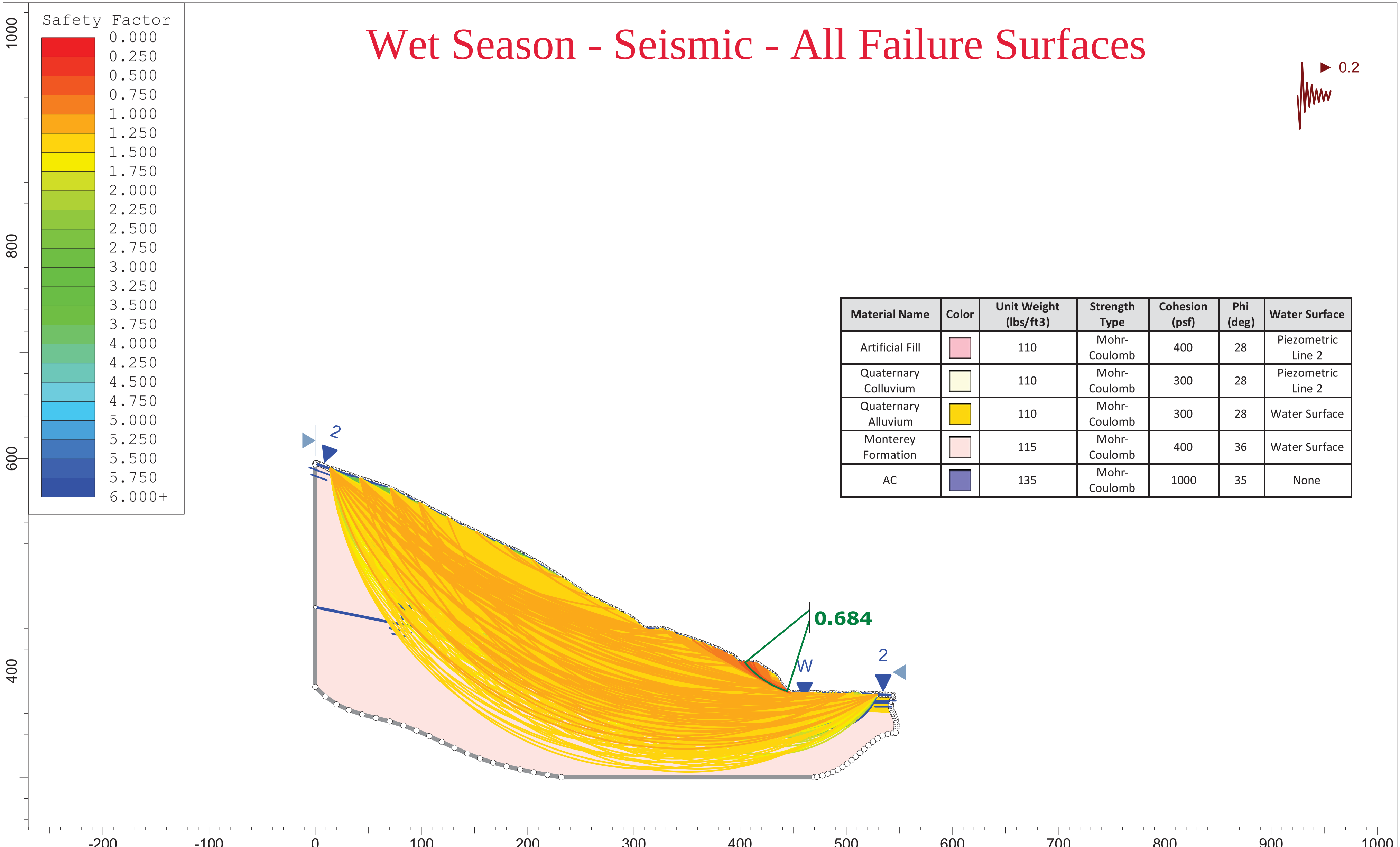
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Artificial Fill		110	Mohr-Coulomb	400	28	Piezometric Line 2
Quaternary Colluvium		110	Mohr-Coulomb	300	28	Piezometric Line 2
Quaternary Alluvium		110	Mohr-Coulomb	300	28	Water Surface
Monterey Formation		115	Mohr-Coulomb	400	36	Water Surface
AC		135	Mohr-Coulomb	1000	35	None

CE&G Pragmatic Expertise™

SLIDEINTERPRET 9.019

Project	Veterans Village at Santa Cruz County		
Cross Section	AA'	*Scenario*	Static
Drawn by	KF	*Company*	CE&G
Date	4/26/2022 1:55:04 PM	*File Name*	SectionAA'.slmd

Wet Season - Seismic - All Failure Surfaces



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Artificial Fill		110	Mohr-Coulomb	400	28	Piezometric Line 2
Quaternary Colluvium		110	Mohr-Coulomb	300	28	Piezometric Line 2
Quaternary Alluvium		110	Mohr-Coulomb	300	28	Water Surface
Monterey Formation		115	Mohr-Coulomb	400	36	Water Surface
AC		135	Mohr-Coulomb	1000	35	None

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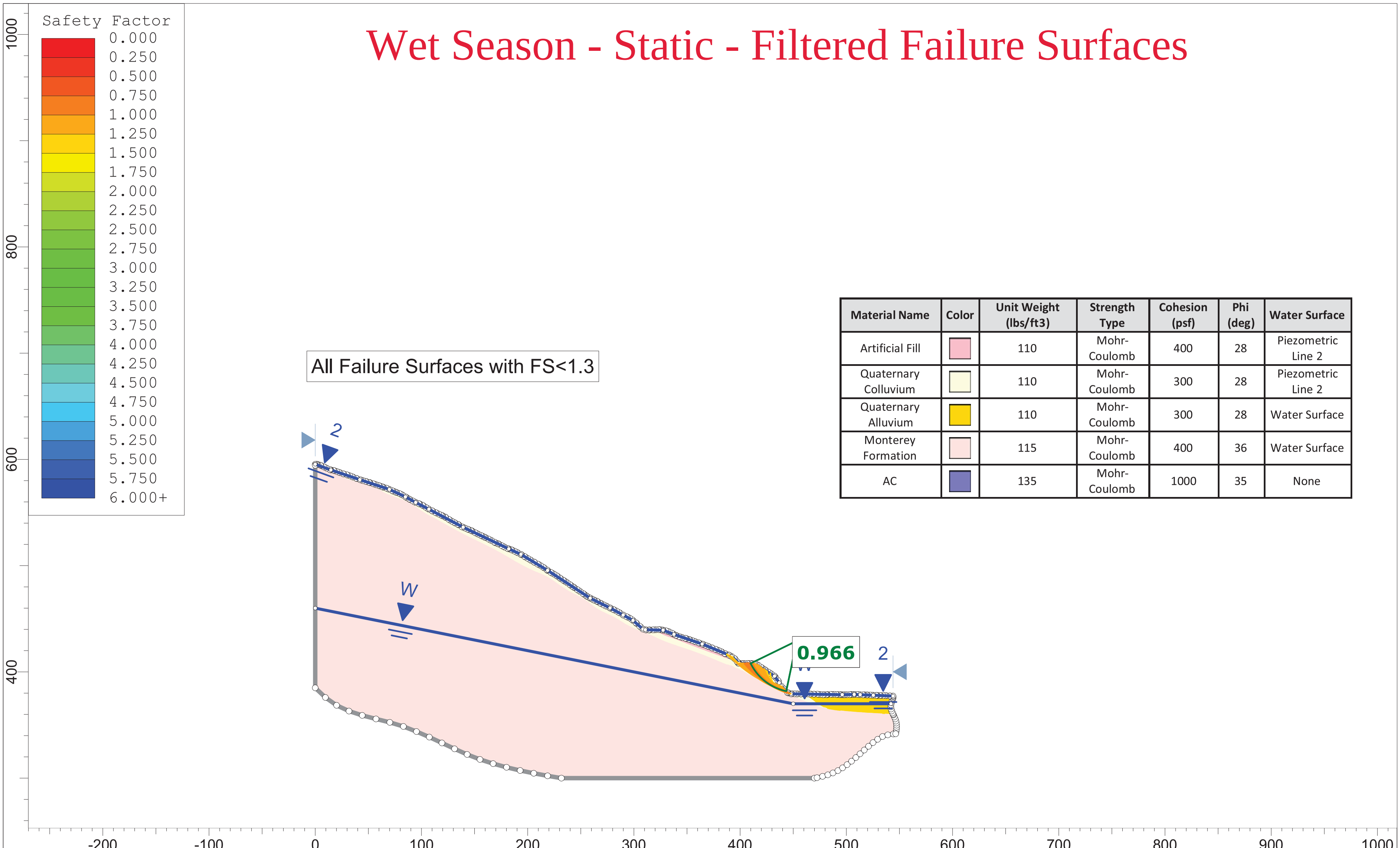
SLIDEINTERPRET 9.019

Project	Veterans Village at Santa Cruz County		
Cross Section	AA'	*Scenario*	Seismic
Drawn by	KF	*Company*	CE&G
Date	4/26/2022 1:55:05 PM	*File Name*	SectionAA'.slmd

Wet Season - Static - Filtered Failure Surfaces



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Artificial Fill		110	Mohr-Coulomb	400	28	Piezometric Line 2
Quaternary Colluvium		110	Mohr-Coulomb	300	28	Piezometric Line 2
Quaternary Alluvium		110	Mohr-Coulomb	300	28	Water Surface
Monterey Formation		115	Mohr-Coulomb	400	36	Water Surface
AC		135	Mohr-Coulomb	1000	35	None

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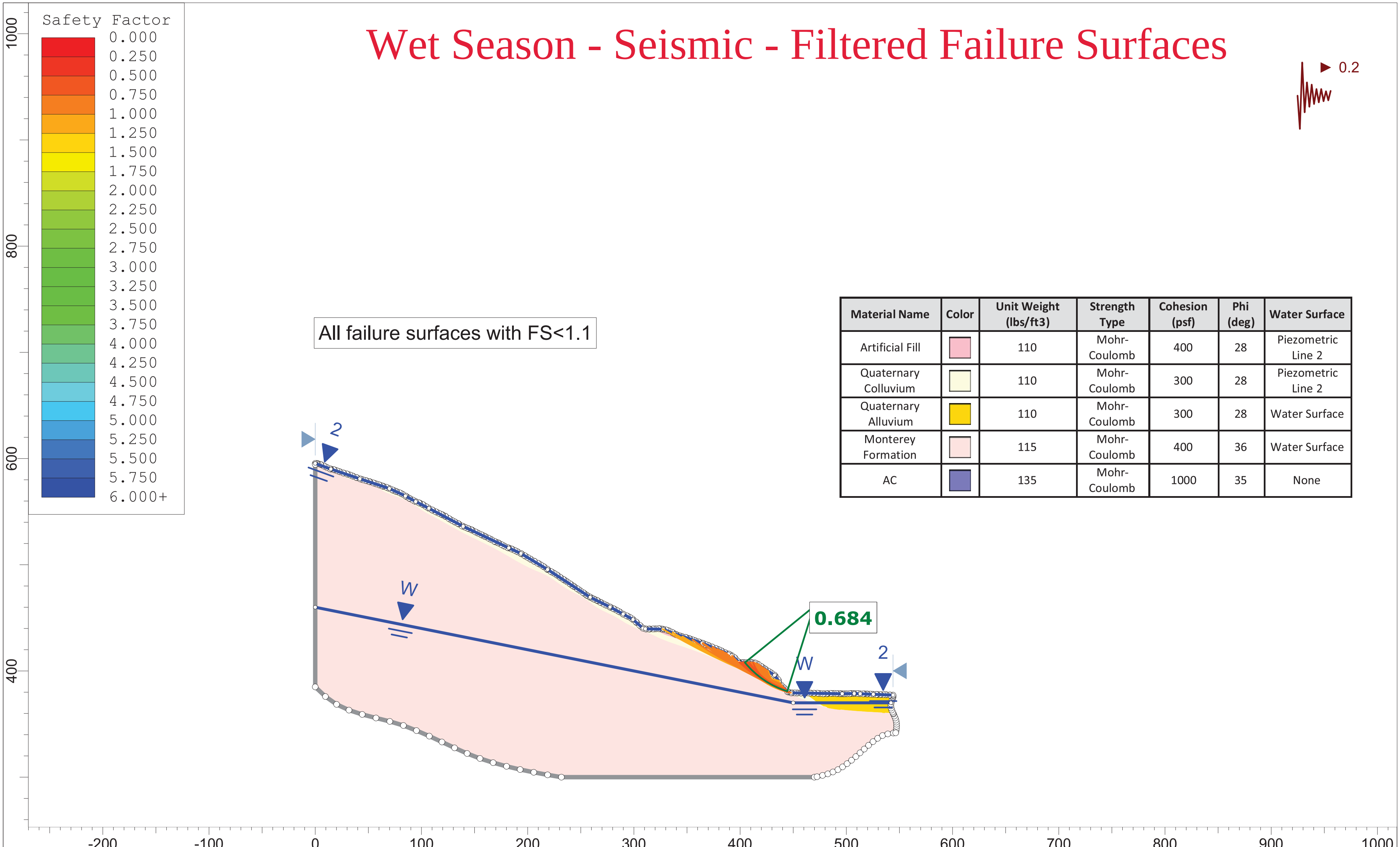
Project	Veterans Village at Santa Cruz County		
Cross Section	AA'	*Scenario*	Static
Drawn by	KF	*Company*	CE&G
Date	4/26/2022 2:01:10 PM	*File Name*	SectionAA'.slmd

Wet Season - Seismic - Filtered Failure Surfaces



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface
Artificial Fill		110	Mohr-Coulomb	400	28	Piezometric Line 2
Quaternary Colluvium		110	Mohr-Coulomb	300	28	Piezometric Line 2
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Monterey Formation		115	Mohr-Coulomb	400	36	Water Surface
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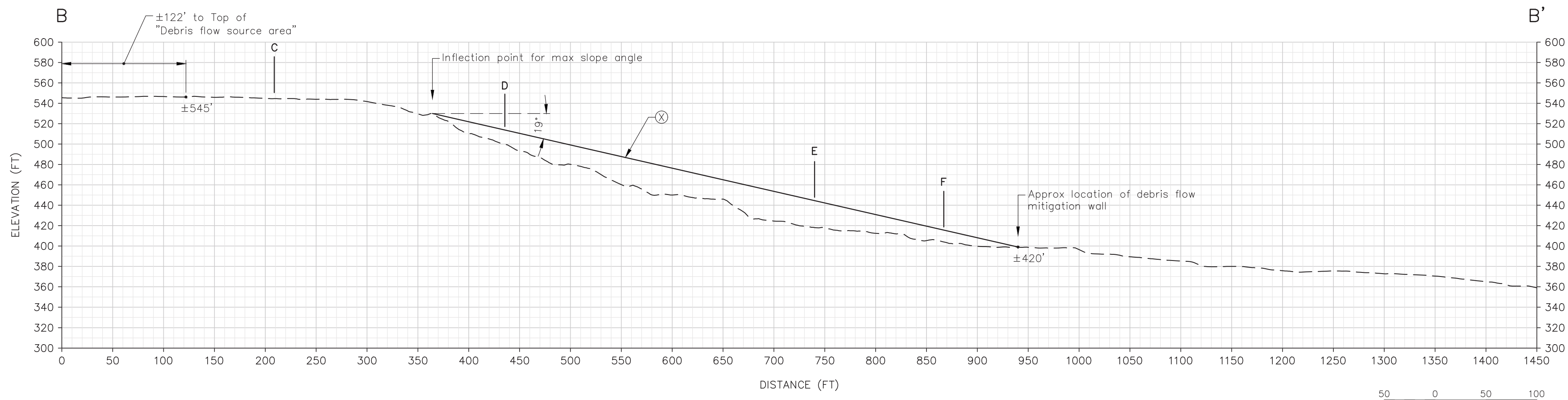
SLIDEINTERPRET 9.019

Project	Veterans Village at Santa Cruz County		
Cross Section	AA'	*Scenario*	Seismic
Drawn by	KF	*Company*	CE&G
Date	4/26/2022 2:01:11 PM	*File Name*	SectionAA'.slmd

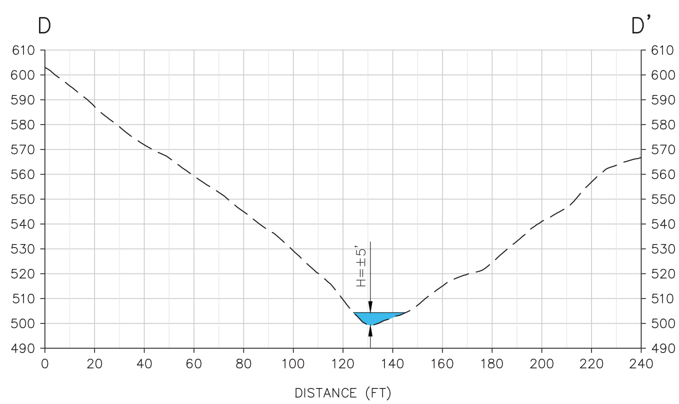
Appendix G. Debris Flow Velocity & Impact Analysis

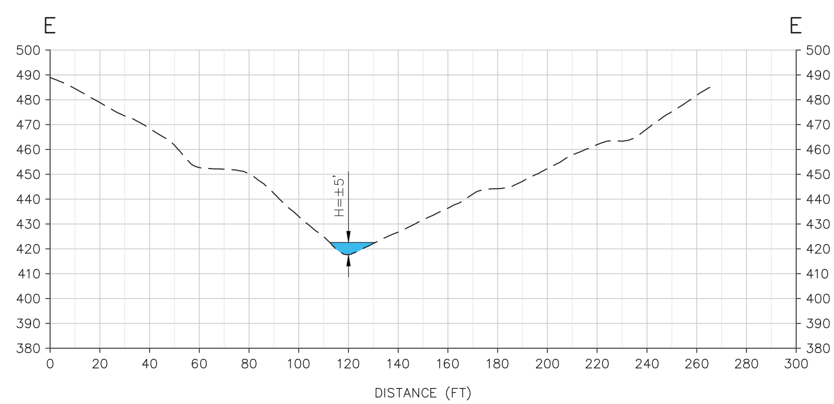


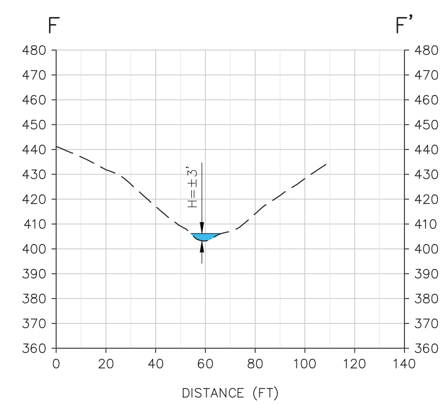
ELEVATION CHANGE: 545'-420'=125'

Ⓧ - MAX SLOPE ANGLE ALONG CHANNEL =±19°

PROFILE B-B'







H APPROXIMATED FROM CHANNEL MORPHOLOGY.

REFERENCES

1. EXISTING SURFACE PROFILE DERIVED FROM 2018-2020 LIDAR DATA (3-FT DEM) FOR SANTA CRUZ COUNTY.

CE&G
CAL ENGINEERING & GEOLOGY
6455 Almaden Expwy.
Suite 100
San Jose, CA 95120
Phone: (408) 440-4542

VETERANS VILLAGE HOUSING PROJECT
8705 HIGHWAY 9
BEN LOMOND, CALIFORNIA

VELOCITY CALCULATIONS

220300	JUNE 2022	FIGURE G1

Map Scale: 1:2,230 if printed on A portrait (8.5" x 11") sheet.

N

Meters: 0, 30, 60, 120, 180

Feet: 0, 100, 200, 400, 600

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils

Soil Rating Polygons
- 0 - 5
- 5 - 15
- 15 - 45
- 45 - 60
- 60 - 100
- Not rated or not available

Soil Rating Lines
- 0 - 5
- 5 - 15
- 15 - 45
- 45 - 60
- 60 - 100
- Not rated or not available

Soil Rating Points
- 0 - 5
- 5 - 15
- 15 - 45
- 45 - 60
- 60 - 100
- Not rated or not available

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Santa Cruz County, California
Survey Area Data: Version 15, Sep 9, 2021

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Apr 13, 2020—Apr 24, 2020

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Representative Slope

Map unit symbol	Map unit name	Rating (percent)	Acres in AOI	Percent of AOI
125	Danville loam, 2 to 9 percent slopes	6.0	1.8	16.6%
143	Lompico-Felton complex, 30 to 50 percent slopes, MLRA 4B	40.0	2.8	25.6%
158	Nisene-Aptos complex, 50 to 75 percent slopes	63.0	6.3	57.8%
Totals for Area of Interest			**10.9**	**100.0%**

Description

Slope gradient is the difference in elevation between two points, expressed as a percentage of the distance between those points.

The slope gradient is actually recorded as three separate values in the database. A low value and a high value indicate the range of this attribute for the soil component. A "representative" value indicates the expected value of this attribute for the component. For this soil property, only the representative value is used.

Rating Options

Units of Measure: percent

Aggregation Method: Dominant Component

Component Percent Cutoff: None Specified

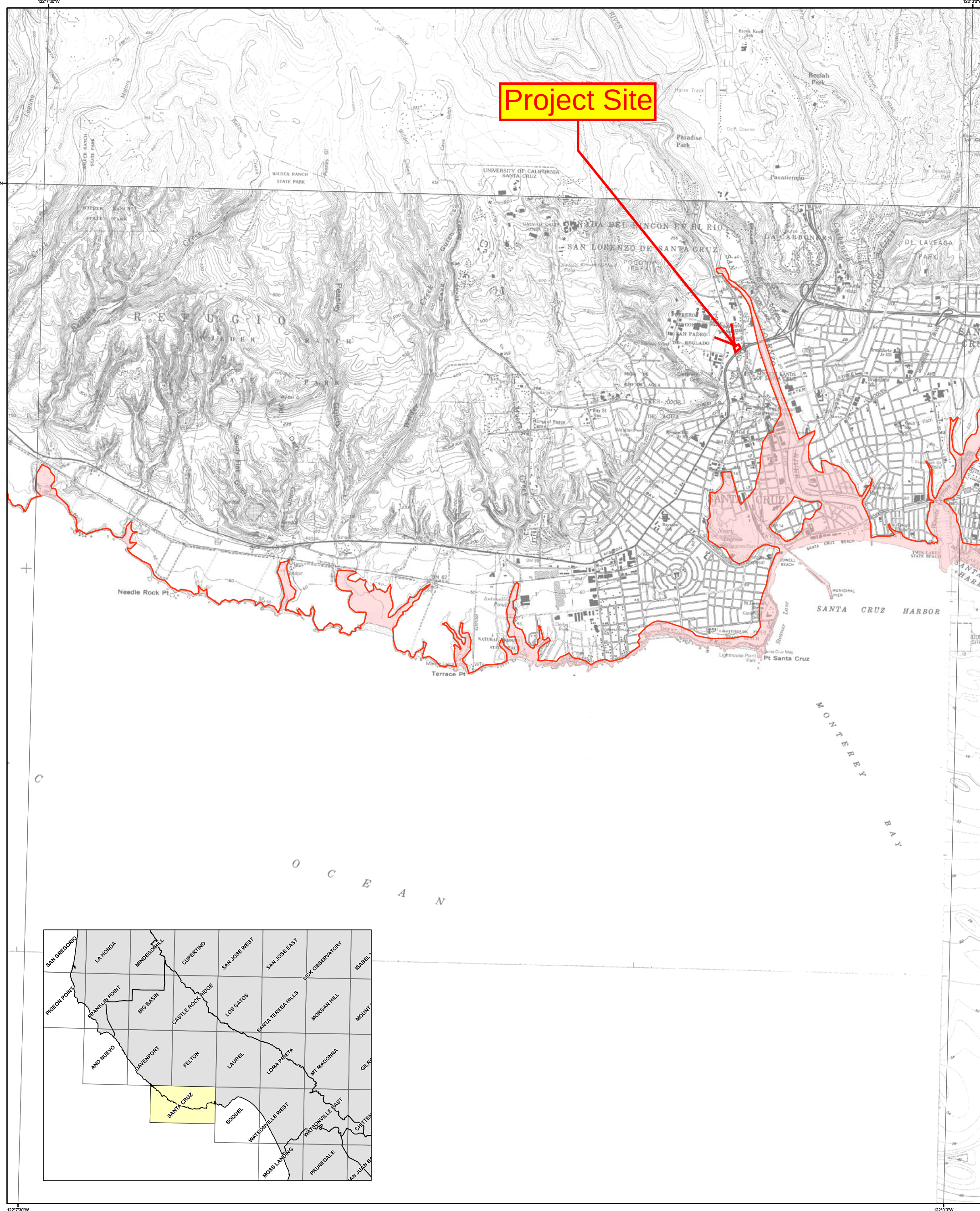
Tie-break Rule: Higher

Interpret Nulls as Zero: No

California Emergency Management Agency
California Geological Survey
University of Southern California

Tsunami Inundation Map for Emergency Planning
Santa Cruz Quadrangle

State of California
County of Santa Cruz



METHOD OF PREPARATION

Initial tsunami modeling was performed by the University of Southern California (USC) Tsunami Research Center funded through the California Emergency Management Agency (CalEMA) by the National Tsunami Hazard Mitigation Program. The tsunami modeling process utilized the MOST (Method of Splitting Tsunamis) computational program (Version 0), which allows for wave evolution over a variable bathymetry and topography used for the inundation mapping (Titov and Gonzalez, 1997; Titov and Synolakis, 1998).

The bathymetric/topographic data that were used in the tsunami models consist of a series of nested grids. Near-shore grids with a 3 arc-second (75- to 90-meters) resolution or higher, were adjusted to 'Mean High Water" sea-level conditions, representing a conservative sea level for the intended use of the tsunami modeling and mapping.

A suite of tsunami source events was selected for modeling, representing realistic local and distant earthquakes and hypothetical extreme undersea, near-shore landslides (Table 1). Local tsunami sources that were considered include offshore reverse-thrust faults, restraining bends on strike-slip fault zones and large submarine landslides capable of significant seafloor displacement and tsunami generation. Distant tsunami sources that were considered include great subduction zone events that are known to have occurred historically (1960 Chile and 1964 Alaska earthquakes) and others which can occur around the Pacific Ocean 'Ring of Fire."

In order to enhance the result from the 75- to 90-meter inundation grid data, a method was developed utilizing higher-resolution digital topographic data (3- to 10-meters resolution) that better defines the location of the maximum inundation line (U.S. Geological Survey, 1993; Intermap, 2003; NOAA, 2004). The location of the enhanced inundation line was determined by using digital imagery and terrain data on a GIS platform with consideration given to historic inundation information (Lander, et al., 1993). This information was verified, where possible, by field work coordinated with local county personnel.

The accuracy of the inundation line shown on these maps is subject to limitations in the accuracy and completeness of available terrain and tsunami source information, and the current understanding of tsunami generation and propagation phenomena as expressed in the models. Thus, although an attempt has been made to identify a credible upper bound to inundation at any location along the coastline, it remains possible that actual inundation could be greater in a major tsunami event.

This map does not represent inundation from a single scenario event. It was created by combining inundation results for an ensemble of source events affecting a given region (Table 1). For this reason, all of the inundation region in a particular area will not likely be inundated during a single tsunami event.

References:

Intermap Technologies, Inc., 2003, Intermap product handbook and quick start guide: Intermap NEXTmap document on 5-meter resolution data, 112 p.

Lander, J.F., Lockridge, P.A., and Kozuch, M.J., 1993, Tsunamis Affecting the West Coast of the United States 1806-1992: National Geophysical Data Center Key to Geophysical Record Documentation No. 29, NOAA, NESDIS, NGDC, 242 p.

National Atmospheric and Oceanic Administration (NOAA), 2004, Interferometric Synthetic Aperture Radar (IfSAR) Digital Elevation Models from GeoSAR platform (EarthData): 3-meter resolution data.

Titov, V.V., and Gonzalez, F.I., 1997, Implementation and Testing of the Method of Tsunami Splitting (MOST): NOAA Technical Memorandum ERL PMEL – 112, 11 p.

Titov, V.V., and Synolakis, C.E., 1998, Numerical modeling of tidal wave runup: Journal of Waterways, Port, Coastal and Ocean Engineering, ASCE, 124 (4), pp 157-171.

U.S. Geological Survey, 1993, Digital Elevation Models: National Mapping Program, Technical Instructions, Data Users Guide 5, 48 p.

TSUNAMI INUNDATION MAP FOR EMERGENCY PLANNING

State of California ~ County of Santa Cruz
SANTA CRUZ QUADRANGLE

July 1, 2009

SCALE 1:24,000

0.5 0.25 0 0.5 1 Miles
1,000 500 0 1,000 2,000 3,000 4,000 5,000 Feet
0.5 0.25 0 0.5 1 Kilometers

Table 1: Tsunami sources modeled for the Santa Cruz County coastline.

Sources (M = moment magnitude used in modeled event)		Areas of Inundation Map Coverage and Sources Used		
		Pescadero	Santa Cruz	Monterey Bay Big
Local Source	Monterey Canyon Landslide			X
Distant Sources	Cascadia Subduction Zone-full rupture (M9.0)		X	
	Central Aleutians Subduction Zone #1 (M8.9)	X	X	X
	Central Aleutians Subduction Zone #2 (M8.9)		X	
	Central Aleutians Subduction Zone #3 (M9.2)	X		X
	Chile North Subduction Zone (M9.4)		X	
	1960 Chile Earthquake (M9.3)		X	
	1964 Alaska Earthquake (M9.2)	X	X	X
	Japan Subduction Zone #2 (M8.8)		X	
	Kuril Islands Subduction Zone #2 (M8.8)		X	
	Kuril Islands Subduction Zone #3 (M8.8)		X	
	Kuril Islands Subduction Zone #4 (M8.8)		X	
	Marianas Subduction Zone (M8.6)	X		X

Cal EMA California Emergency Management Agency
CGS California Geological Survey
USC University of Southern California

MAP EXPLANATION

Tsunami Inundation Line

Tsunami Inundation Area

PURPOSE OF THIS MAP

This tsunami inundation map was prepared to assist cities and counties in identifying their tsunami hazard. It is intended for local jurisdictional, coastal evacuation planning uses only. This map, and the information presented herein, is not a legal document and does not meet disclosure requirements for real estate transactions nor for any other regulatory purpose.

The inundation map has been compiled with best currently available scientific information. The inundation line represents the maximum considered tsunami runup from a number of extreme, yet realistic, tsunami sources. Tsunamis are rare events; due to a lack of known occurrences in the historical record, this map includes no information about the probability of any tsunami affecting any area within a specific period of time.

Please refer to the following websites for additional information on the construction and/or intended use of the tsunami inundation map:

State of California Emergency Management Agency, Earthquake and Tsunami Program:
http://www.oes.ca.gov/WebPage/oeswebsite.nsf/Content/B1EC
51BA215931768825741F005E8D80?OpenDocument

University of Southern California – Tsunami Research Center:
http://www.usc.edu/dept/tsunamis/2005/index.php

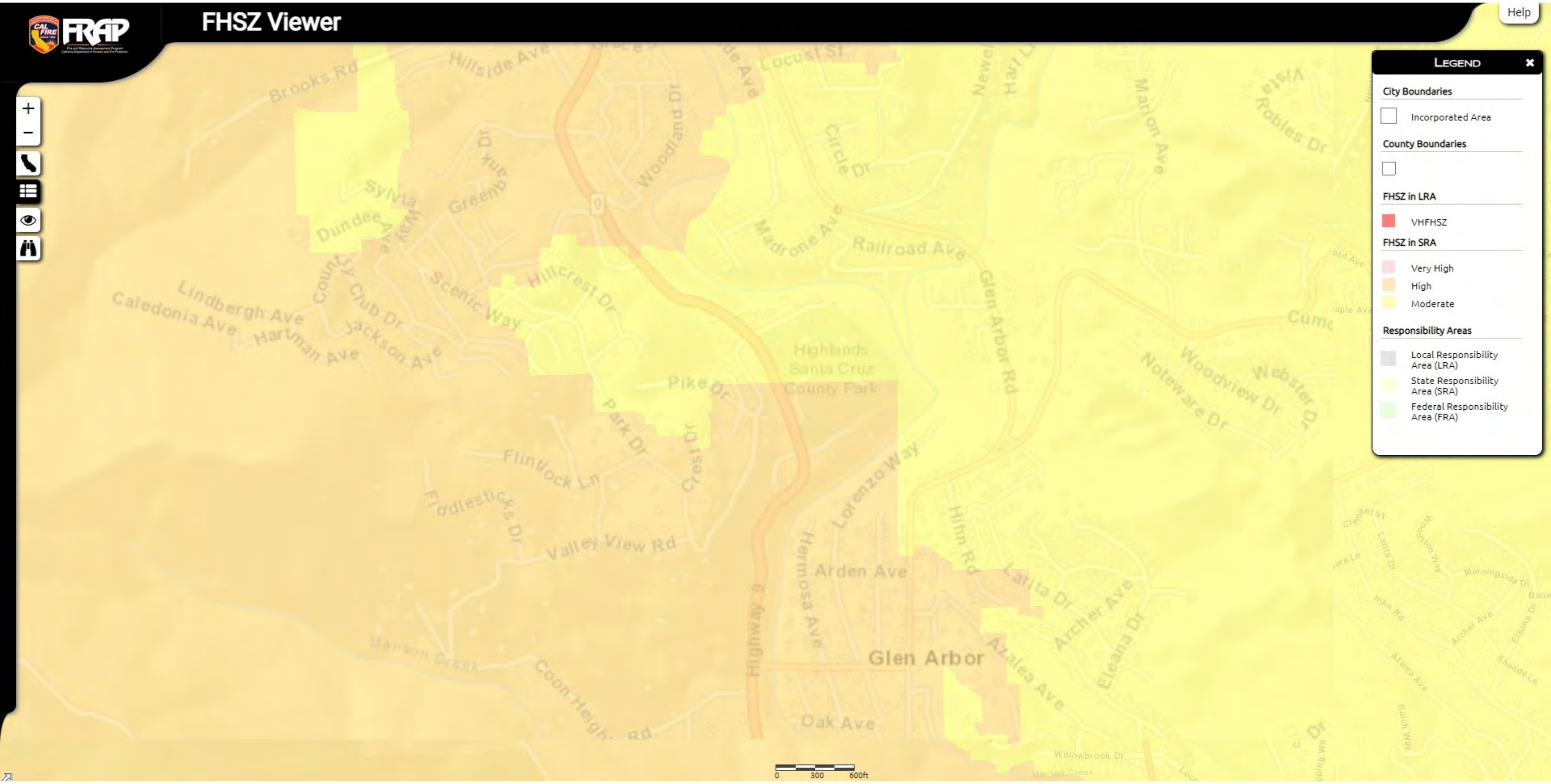
State of California Geological Survey Tsunami Information:
http://www.conservation.ca.gov/cgs/geologic_hazards/Tsunami/index.htm

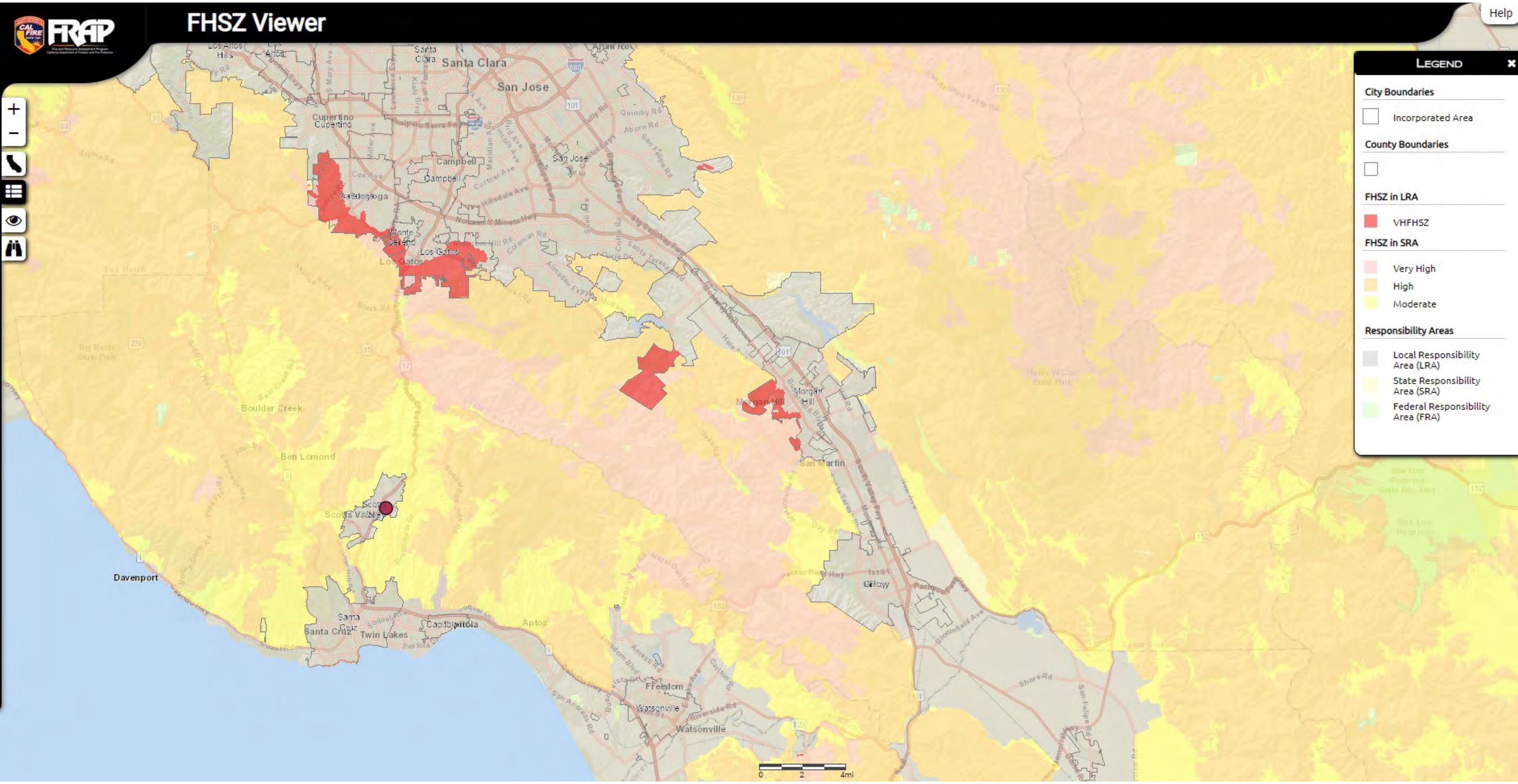
National Oceanic and Atmospheric Agency Center for Tsunami Research (MOST model):
http://nctr.pmel.noaa.gov/time/background/models.html

MAP BASE

Topographic base maps prepared by U.S. Geological Survey as part of the 7.5-minute Quadrangle Map Series (originally 1:24,000 scale). Tsunami inundation line boundaries may reflect updated digital orthophotographic and topographic data that can differ significantly from contours shown on the base map.

DISCLAIMER


FHSZ Viewer
Help
Legend
City Boundaries
Incorporated Area
County Boundaries
FHSZ in LRA
VHFHSZ
FHSZ in SRA
Very High
High
Moderate
Responsibility Areas
Local Responsibility Area (LRA)
State Responsibility Area (SRA)
Federal Responsibility Area (FRA)
Brooks Rd
Hillside Ave
Locust St
Woodland Dr
Circle Dr
Marion Ave
Vista Robles Dr
Sylvia Way
Dundee Ave
Madrone Ave
Railroad Ave
Caledonia Ave
Lindbergh Ave
Hartman Ave
Jackson Ave
Scenic Way
Hillcrest Dr
Glen Arbor Rd
Highlands Santa Cruz County Park
Pike Dr
Park Dr
Crest Dr
Flintlock Ln
Fiddlesticks Dr
Valley View Rd
Lorenzo Way
Hihn Rd
Woodview Dr
Noteware Dr
Webster Dr
Arden Ave
Hermosa Ave
Highway 9
Larita Dr
Archer Ave
Glen Arbor
Azalea Ave
Eleana Dr
Oak Ave
Coon Heights Rd
Manson Creek
Willowbrook Dr
0
300
600ft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FHSZ in LRA
VHFHSZ
FHSZ in SRA
Very High
High
Moderate
Responsibility Areas
Local Responsibility Area (LRA)
State Responsibility Area (SRA)
Federal Responsibility Area (FRA)
Santa Clara
San Jose
Cupertino
Campbell
Saratoga
Monte Sereno
Los Gatos
Morgan Hill
San Martin
Gilroy
Boulder Creek
Ben Lomond
Scotts Valley
Davenport
Santa Cruz
Twin Lakes
Capitola
Aptos
Freedom
Watsonville
0
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