C2EARTH, INC.
GEOLOGIC & GEOTECHNICAL ENGINEERING SERVICES

15 August 2025
Document Id. 23103C-02L3
Serial No. 21371

[redacted]

SUBJECT: SUPPLEMENTAL FINDINGS AND RECOMMENDATIONS
PROPOSED SITE DEVELOPMENT
[redacted]
APN 544-39-035
HELEN WAY
SANTA CLARA COUNTY, CALIFORNIA

Dear [redacted]:

INTRODUCTION

As requested, we are providing the following supplemental findings and recommendations in response to comments from the County of Santa Clara Department of Planning and Development for the proposed residential development of your property at APN 544-39-035 on Helen Way in the Redwood Estates community of unincorporated Santa Clara County, California.

Our Geologic and Geotechnical Study report dated 31 October 2023 (Document Id. 23103C-01R1) presented our findings for the project. Following the preparation of our Report, we reviewed plans for the site development and the on-site wastewater treatment system and issued Plan Review letters dated 14 March 2024 and 30 May 2025 (Document Ids. 23103C-02L1 and -02L2).

Subsequently, the County of Santa Clara has issued comments in a review letter dated 16 August 2024 and by email. We have conferred with Mr. Peter Estes with Environmental Health, and we are providing the following information to address the setback requirements between the building foundation and the proposed drip drainfield.

SUPPLEMENTAL FINDINGS AND RECOMMENDATIONS

As currently planned, the site will be developed with a two-story residence over a daylighting basement in the southwest portion of the property, and a detached garage north of the residence. The property will be serviced by an on-site wastewater treatment system that will be located in the southeast portion of the property. In our opinion, the proposed site development may proceed as planned provide the recommendations presented in our Report and recommendations below are incorporated into the design and construction of the proposed improvements. The following recommendations must be considered a part of, and supplemental to, our Geologic and Geotechnical Study.

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On-site Wastewater Treatment System

We have reviewed the on-site wastewater treatment system plans, and further evaluated the site conditions and the proposed development concept. Based upon acceptable percolation test rates and our evaluation of the subsurface conditions, we recommend a minimum setback of 10 feet between the proposed drainfields and the building foundations, provided the basement is designed and constructed with undrained retaining walls in accordance with the recommendations presented below.

Undrained Retaining Walls

In addition to the pertinent recommendations presented in our Report, we recommend that site and building retaining walls that are downslope or within 25 feet of the proposed on-site wastewater treatment system drainfields be designed as undrained retaining walls in accordance with the following.

- Support undrained site and basement retaining walls on drilled pier foundations in accordance with the recommendations provided in our Report.
- Design unrestrained (active condition) undrained walls to resist an equivalent fluid pressure of 85 pcf. Design undrained walls that are restrained from movement at the top or sides (at-rest condition) to resist an equivalent fluid pressure of 100 pcf
- Design retaining walls for seismic-loading as the structural engineer deems appropriate. In our opinion, undrained retaining walls are not subject to additional seismic loading increments.
- Fully waterproof undrained retaining walls. In addition, water stops should be cast into cold joints at the retaining wall and foundation interface. We are not qualified to recommend specific waterproofing materials or their applications. Any waterproofing product must be applied in **strict** compliance with the manufacturer's and/or architect's specifications.
- Compact the backfill placed behind the walls to at least 90% relative compaction, using light compaction equipment, in accordance with the compaction procedures given in our Report. If heavy compaction equipment is used, the walls should be appropriately temporarily braced, as the situation requires. Backfill must consist of Caltrans Class II baserock or engineered fill. Drainrock is not an acceptable backfill material for undrained retaining walls.

We trust that these supplemental findings and recommendations adequately address the County's comments. Please contact us if there are any questions.

Sincerely,
C2Earth, Inc.

REGISTERED PROFESSIONAL ENGINEER CRAIG N. REID No. 3060 GEOTECHNICAL STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST CRAIG N. REID NO. 2471 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

THIS DOCUMENT HAS BEEN DIGITALLY SIGNED

Craig N. Reid, Principal
Certified Engineering Geologist 2471
Registered Geotechnical Engineer 3060

Distribution: Addressee (via e-mail to [redacted])

GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED SITE DEVELOPMENT

[redacted]

APN 544-39-035
HELEN WAY
SANTA CLARA COUNTY, CALIFORNIA

Prepared For:

[redacted]

31 October 2023
Document Id. 23103C-01R1

C2EARTH

31 October 2023
Document Id. 23103C-01R1
Serial No. 20541

[redacted]

SUBJECT: GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED SITE DEVELOPMENT
[redacted]
APN 544-39-035
HELEN WAY
SANTA CLARA COUNTY, CALIFORNIA

Dear [redacted]:

As you requested, we have performed a geologic and geotechnical study for the proposed site development of your property with APN 544-39-035 on Helen Way in the Redwood Estates community of unincorporated Santa Clara County, California. The accompanying report presents the results of our study and testing, and our conclusions and recommendations concerning the geologic and geotechnical engineering aspects of the project. The findings and recommendations presented in this report are contingent upon our review of the final grading, foundation, and drainage control plans; our observation of the grading; and the installation of the foundation and drainage control systems.

This report includes information that is vital to the success of your project. We strongly urge you to thoroughly read and understand its contents. Please refer to the text of the report for detailed findings and recommendations.

Sincerely,
C2Earth, Inc.

Sheldon Blair

Project Geologist

THIS DOCUMENT HAS BEEN DIGITALLY SIGNED

Craig N. Reid, Principal
Certified Engineering Geologist 2471
Registered Geotechnical Engineer 3060

REGISTERED PROFESSIONAL ENGINEER
CRAIG N. REID
No. 3060
GEOTECHNICAL
STATE OF CALIFORNIA

Distribution: Addressee (1 hard copy via mail and via e-mail to [redacted])
[redacted] (via e-mail to [redacted])

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APPLICATION TO USE

1. INTRODUCTION

This report presents the results of our geologic and geotechnical study for the proposed residential development of your property with APN 544-39-035 on Helen Way in the Redwood Estates community of unincorporated Santa Clara County, California (see Figure 1, Site Location Map). The purpose of our study was to explore the geologic and geotechnical conditions on the subject property in the area of the proposed improvements and to develop findings and recommendations for the earthwork and foundation engineering aspects of the proposed development. Our study was limited to the subject parcel and proposed building area.

We understand that you are planning to develop the site with a three-story residence that will be constructed into the hillside with a daylighting basement in the southwestern portion of the property. We also understand that you are planning to construct a detached garage in the northeastern portion of the property. The site is steep and we anticipate that building and site retaining walls will be required for the site development. The development will be serviced by an on-site wastewater treatment system with drip drainfields planned on the slope in the southeastern portion of the property.

We issue this report with the understanding that it is the responsibility of the owner or the owner's representative to ensure that the information and recommendations contained in this report are brought to the attention of the project architect and engineer and are incorporated into the plans and specifications of the development. You must also ensure that the contractor and sub-contractors follow the recommendations during construction.

2. SCOPE OF SERVICES

We conducted this study in accordance with the scope and conditions presented in our proposal dated 11 July 2023 (Document Id. 23103C-01P1). The methodology of our evaluation is discussed in the body of this report. We make no other warranty, either expressed or implied. Our scope of services for this study included:

- Reviewing selected geologic literature, aerial photographs, and a previous consultants' report of the area, to evaluate the prevailing geologic and geotechnical conditions;
- performing an engineering geologic reconnaissance and mapping of the site;
- preparing a site plan and slope profiles;
- conducting subsurface exploration;
- performing field and laboratory testing;
- analyzing geologic and geotechnical engineering properties from collected data;
- performing a global, quantitative slope stability analysis of the regional vicinity, based upon published rock and soil material parameters; and
- preparing this report.

We have prepared this report as a product of our service for your exclusive use in designing and developing the subject property for residential use. Other parties may not use this report, nor may the report be used for other purposes, without prior written authorization from C2Earth, Inc (C2).

Because of possible future changes in site conditions or the standards of practice for geotechnical engineering and engineering geology, the findings and recommendations of this report may not be considered valid beyond three years from the report date, without review by C2. In addition, in the event that any changes in the nature or location of the proposed improvements are planned, the conclusions and recommendations of this report may not be considered valid unless we review such changes, and modify or affirm in writing the conclusions and recommendations presented in this report remain valid with respect to the changes.

Our study excluded an evaluation of hazardous or toxic substances, corrosion potential, chemical properties, and other environmental assessments of the soil, subsurface water, surface water, and air on or around the subject property. The lack of comments in this report regarding the above does not indicate an absence of such substances and/or conditions.

3. GEOLOGY AND SEISMICITY

We reviewed selected geologic maps, literature, aerial photographs, topographic maps, and a other consultant's report to evaluate the prevailing geologic conditions of the site and in the vicinity. The Regional Geologic Map, Regional Landslide Inventory Map, Regional Seismic Hazard Zones Map, and County Hazard Zones Map are presented on Figures 2 through 5, respectively.

3.1. Geology

The subject property is on the eastern flank, near the crest of the central Santa Cruz Mountains, a northwest-trending range within the California Coast Ranges geomorphic province (see Figure 1). The ridge's eastern flank descends steeply towards the east in the vicinity of the subject property. According to the Geologic Map of the Los Gatos Quadrangle, California (McLaughlin et al., 2001), the subject site is underlain by a Holocene to Pleistocene age (approximately younger than 5.3 million years old) large dormant landslide (see Figure 2). The dormant landslide materials are shown to be derived from and underlain by Oligocene to late Eocene age (approximately 23 to 48 million year old) Rices Mudstone Member of the San Lorenzo Formation.

The landslide materials are overlain by slope debris (colluvium) on the subject property and across most of the hillside areas in the site vicinity. Where the colluvial soil is located on moderate to steep slopes, it is subject to downhill creep, a process by which the soil moves downslope at an imperceptibly slow rate as a result of gravity.

The originally flat lying sedimentary bedrock has been uplifted, tilted, and folded by the mountain-building processes that formed the Santa Cruz Mountains, then displaced by landsliding. A review of the geologic maps show that bedding attitudes in the site vicinity are sparse. Approximately 1,200 feet northwest of the site, the in-place bedrock bedding strikes (is oriented) approximately northwest and dips (sloped downward) 30 degrees to the southwest (see Figure 2). In addition, approximately 1,800 feet southeast of the site, the in-place bedrock has bedding that strikes to the northwest and dip of 75 degrees to the southwest.

3.2. Landsliding

As shown on the Geologic Map and according to the Landslide Inventory Map of the Los Gatos 7.5-Minute Quadrangle, the subject site is mapped within a large dormant, mature landslide (see Figure 3). The mapped landslide measures about 2,900 feet wide by 2,200 feet long and is denoted as having a 75% confidence that it exists, based upon one or two geomorphic features that have been subdued by erosion. Our reconnaissance and review of stereo-paired aerial photographs revealed hummocky disturbed terrain and geomorphology consistent with landsliding shown on geologic maps. Other consultants have reported movement within portions of the mapped landslide as a result of earthquake shaking during the Loma Prieta Earthquake of 1989.

Because of the steep slopes and evidence of landsliding, the site and site area are mapped within the State Seismic Hazard Zone for earthquake-induced landsliding and the Santa Clara County Landslide Hazard Zone (see Figures 4 and 5, respectively). These zones were established to minimize the loss of life and property by identifying and mitigating seismic hazards related to landslides. Consequently, we have characterized the landslide configuration and conducted a quantitative slope stability analysis to evaluate the risk of landsliding. Our interpretation of the landslide configuration is based upon the results of our quantitative slope stability analysis (discussed in Section 5) and is shown on Figure 6, Geologic Cross-Section A-A'.

3.3. Seismicity

Geologists and seismologists recognize the greater San Francisco Bay Area as one of the most active seismic regions in the United States. The seismicity in the region is related to activity within the San Andreas fault system, a major rift in the earth's crust that extends for at least 700 miles along the California Coast. Faults within this system are characterized predominantly by right-lateral, strike-slip movement. The four major faults that pass through the Bay Area in a northwest direction have produced approximately 12 earthquakes per century strong enough to cause structural damage. These major faults are the San Andreas, Hayward, Calaveras, and San Gregorio faults.

The site can be expected to experience periodic minor earthquakes or even a major earthquake (Moment magnitude 6.7 or greater) on one of the nearby active or potentially active faults during the design life of the proposed project. The Moment magnitude scale is directly related to the amount of energy released during an earthquake and provides a physically meaningful measure of the size of an earthquake event.

The U.S. Geological Survey (2016) estimates that by 2043, the probability of a Moment magnitude 6.0 earthquake occurring on one of the active faults in the San Francisco region is 98%. The probability of a Moment magnitude 6.7 or greater earthquake occurring on one of the active faults in the San Francisco region is 72%. The following table provides corresponding estimates for the probability of a major earthquake (Moment magnitude 6.7 or greater) for four major faults in the Bay Area.

Fault	Probability (%)
Hayward	33
Calaveras	26
San Andreas	22
San Gregorio	6

30-Year Probability of Magnitude 6.7 or Greater Earthquake

The property is within the San Andreas fault zone. The San Andreas fault has a regional trend of approximately N34W; however, the segment of the San Andreas fault located within the central Santa Cruz Mountains east of the site strikes approximately N44W, forming a restraining bend. This restraining bend has created a compressional zone along the east side of the Santa Cruz Mountains, resulting in the formation of the Frontal thrust fault system. The Frontal thrust fault system is comprised of reverse and right-reverse faults, including the Berrocal, Monte Vista, and Shannon faults within the eastern foothills and the alluvial plain adjacent to the foothills (Angell et al. 1997). This thrust fault system bounds the southwest margin of the Santa Clara Valley. The following table indicates the approximate distance and direction from the site to active and potentially active faults.

Fault	Approx. Distance To Fault	Direction From Site
San Andreas	2,800 feet	Northeast
Butano	3,700 feet	South
Monte Vista-Shannon	6 miles	Northeast
Hayward	17 miles	Northeast
Calaveras	20¾ miles	east
San Gregorio	17¾ miles	west

Regional Fault Distances and Directions

According to the California State Special Studies Zones Map by the California Division of Mines and Geology, the site is mapped outside of the current Alquist-Priolo Earthquake Fault Zone for areas prone to earthquake ground rupture. In addition, the site is not within a Santa Clara County geologic hazard zone corresponding to potential for fault rupture (see Figure 5).

The intensity of an earthquake differs from the Moment magnitude, in that intensity is a measure of the effects of an earthquake, rather than a measure of the energy released. These effects can vary considerably based on the earthquake magnitude, distance from the earthquake's epicenter, and site geology.

Because of the site's proximity to several active faults and the site’s geology, maximum anticipated ground shaking intensities for the area are characterized as violent and equal to a Modified Mercalli (MM) intensity of IX (Association of Bay Area Governments, 2023). An earthquake having a MM intensity of IX generally causes considerable damage in specially designed structures, well-designed framed structures thrown out of plumb, great damage in substantial buildings with partial collapse, and buildings shifted off foundations (Yanev, 1974) (see Table I, Modified Mercalli Scale of Earthquake Intensities).

Since 1800, four major earthquakes have been recorded on the San Andreas fault. In 1836, an earthquake with an estimated maximum intensity of VII on the MM scale occurred east of the Monterey Bay on the San Andreas fault (Toppozada and Borchardt, 1998). The estimated Moment magnitude (M_w) for this earthquake is about 6.25. In 1838, an earthquake occurred with an estimated intensity of about VIII-IX (MM), corresponding to a M_w of about 7.5. The San Francisco Earthquake of 1906 caused the most significant damage in the history of the Bay Area in terms of lives lost and cost of property damage. This earthquake created a surface rupture along the San Andreas fault from Shelter Cove to San Juan Bautista, about 290 miles in length. It had a maximum intensity of XI (MM), a M_w of about 7.9, and was felt as far away as Oregon, Nevada, and Los Angeles. The most recent earthquake to affect the Bay Area was the Loma Prieta earthquake of 17 October 1989, occurring in the Santa Cruz Mountains, which had a M_w of about 6.9. Ground shaking equal to an MM intensity of between VI and VII was felt at the site during the Loma Prieta Earthquake (Stover, et al., 1990).

In 1868, an earthquake with an estimated maximum MM intensity of X and M_w of about 7.0 occurred on the southern segment of the Hayward fault, between San Leandro and Fremont. In 1861, an earthquake of unknown magnitude (likely having an M_w of about 6.5) was reported on the Calaveras fault. The most recent significant earthquake on this fault was the 1984 Morgan Hill Earthquake, that had an M_w of about 6.2.

4. SITE CHARACTERIZATION

4.1. Regional Setting

We reviewed aerial photographs and topographic maps for the site and vicinity. The irregularly shaped, approximately 0.2-acre site is situated along an eastern facing flank of the northwest-southeast trending Santa Cruz Mountains. The flank is incised by unnamed drainage courses, northwest (also know as Muddy Gulch) and southeast of the site area, that flow northeast into the Los Gatos Creek. The subject property is bounded by undeveloped land to the southwest, Helen Way to the northwest, and developed private properties to the southeast and northeast.

4.2. Site Description

On 2 August 2023, our principal engineer/geologist performed an engineering site reconnaissance and site mapping as part of our evaluation of the residential site development and on-site wastewater treatment system. On 7 August 2023, our staff geologist and project geologist visited the site to perform a subsurface exploration, however, the site was inaccessible for exploration. On 24 August 2023, our principal engineer/geologist visited the site and met with you about the site's accessibility. On 28 August 2023, our project geologist visited the site to perform additional site reconnaissance, mapping, and subsurface exploration.

The site plan and slope profile we developed is based upon the On-site Wastewater Treatment System Plan by Biosphere Consulting, dated 19 March 2021, supplemented by tape and compass mapping techniques. The site plan is shown on Figure 7, Site Plan and Engineering Geologic Map and the slope profile is depicted on Figure8, Geologic Cross-Section B-B'. The site plan and slope profile are only as accurate as implied by the mapping technique used. The following is a summary of the surficial site characteristics.

The topography across the subject site and vicinity descends steeply towards the southeast with slope gradients varying between approximately 1¼:1 to 2½:1 (horizontal to vertical). Helen Way and the upper, northwest, portion of the site are underlain by fill. From a narrow flat area adjacent to Helen Way, an over-steepened fill slope descends steeply with a slope gradient of about 1¼ to 1. The fill slope has loose debris and rubbish evident along the ground surface. The native grade beneath the fill appears to have a natural slope gradient between 2½:1 and 2:1. The grade becomes less steep near the southeastern property boundary. Adjacent to the southeastern property boundary are developed, private, residential properties that are accessed from Mary Alice Way.

The flat areas on the subject property near Helen Way are sparsely vegetated with grasses and bushes, while the slope areas densely vegetated with trees, bushes, and grasses. Drainage across the site is generally characterized as uncontrolled sheet flow to the southeast. To our knowledge there has not been prior development of the site.

4.3. Subsurface

On 28 August 2023, our project geologist visited the site to observe the subsurface conditions at discrete locations in the vicinity of the proposed improvements. Our project geologist logged three test borings to depths of approximately 12, 12, and 11 feet below ground surface using a tripod, rope, cathead, and sampling equipment. Samples were collected with conventional split-spoon samplers advanced using a rope and cathead driven 140-pound hammer.

Boring 1 was drilled in the area of the proposed residence in the central portion of the site. Boring 2 was drilled in the area of the proposed drip drainfield in the lower, southeastern, portion of the site. Boring 3 was drilled in the area of the proposed garage in the upper, northeastern, portion of the site. The approximate locations of the borings are shown on Figure 7. We determined the approximate boring locations by measuring distance and bearing from points on the site plan; these locations are only as accurate as implied by the mapping technique used.

We logged the borings in general accordance with the Unified Soil Classification System and our Rock Classification System described on Figures 9 and 10, Key to Logs and Rock Classification System, respectively. A Summary of Field Sampling Procedures is presented on Figure 11. The boring logs are presented on Figures 12, 13, and 14, Log of Borings 1, 2, and 3. The boring logs show our interpretation of the subsurface conditions at the locations and on the date indicated and we do not warrant that they are representative of the subsurface conditions at other locations and times.

Borings 1 and 2 encountered colluvium (a soil material that is deposited on or at the base of a slope from sheet flow runoff) underlain by displaced bedrock. Boring 3 encountered fill underlain by displaced bedrock.

The colluvium observed in Boring 1 and Boring 2 consists of medium stiff to hard, light brown and brown, low plasticity sandy clayey silt. The colluvium is approximately 7½ and 6½ feet thick in Borings 1 and 2, respectively. The colluvium is underlain by displaced bedrock. The fill encountered in Boring 3 consists of very stiff to hard, brown clayey silt. The fill is approximately 9¼ feet thick in Boring 3. The fill is underlain by the displaced bedrock as described for Boring 1 and Boring 2. The displaced bedrock persisted to the bottom of both borings. The displaced bedrock are comprised of hard, light yellowish brown and brown siltstone.

Detailed descriptions of the subsurface materials observed in the borings are presented on the boring logs and our interpretations of the subsurface conditions are shown on Figure 8.

4.4. Groundwater

We did not encounter groundwater in our borings. Our review of topography maps revealed topography that is deeply eroded by several hundred feet along the drainage swales southeast and northwest of the site area, and along Los Gatos Creek at the base of the ridge. We anticipate groundwater to be at or near the bottom of the drainage swales and creek channel. By interpolating between the swales and drainage course, we suspect groundwater to be hundreds to several hundreds of feet below the subject site. Fluctuations in the level of subsurface water could occur due to variations in rainfall, temperature, and other factors not evident at the time our observations were made.

4.5. Laboratory Testing

We developed our laboratory testing program to supplement our evaluation of the geotechnical engineering properties of the soil and bedrock at the site. We retained soil samples from the borings for laboratory classification and testing. Additionally, we had Cooper Testing Laboratory conduct a staged, consolidated, undrained triaxial compression, shear strength test on a sample of the siltstone obtained from Boring 2 at a depth of about 8 feet. The results of moisture content, dry density, and shear strength tests are presented on the boring logs. The results of the shear strength testing is also presented on Figure 15, Shear Strength Test.

5. SLOPE STABILITY ANALYSES

5.1. Overview

The following paragraphs describe the methodology and results of a quantitative slope stability analyses that we performed to evaluate the relative risk of future landslide movement at the subject property. We performed the analyses using the computer program Slide2 Modeler Version 9.028 by Rocscience, Inc., utilizing the GLE/Morgenstern-Price methodology with non-circular Cuckoo slip surface search to calculate failure surfaces and the factor of safety against sliding. The analyses were performed in general accordance with the guidelines presented in the Special Publication 117A by the California Geological Survey (2008).

You should note that computer-aided slope stability analyses are mathematical models of the slopes and soil and they contain many assumptions. Slope stability analyses and the generated factors of safety only indicate general slope stability trends. In general, factors of safety below 1.00 indicate a potential failure. However, a slope with a factor of safety of less than 1.00 will not necessarily fail, but the probability of failure will be greater than that for a slope with a higher factor of safety. Conversely, a slope with a factor of safety greater than 1.00 may fail but the probability of stability is higher than that for a slope with a lower factor of safety.

5.2. Slope Geometry

We performed the slope stability analyses utilizing the surface profile depicted on Geologic Cross-Section A-A' (see Figure 6). We generated this profile and characterized the subsurface conditions based on data shown on the Regional Geologic Map (see Figure 2).

The subsurface geometry of the mapped landslide beneath the subject property, as shown on Figure 2, is based upon a calculated critical failure surface within the Rices mudstone bedrock using published shear strength values (as discussed below) under pseudo-static conditions. The specified entry (upslope) and exit (downslope) search limits were consistent with the upper and lower limits of the subject landslide depicted on Figure 2. The calculated critical failure surface is depicted on Figure 6.

5.3. Soil Strength Parameters

We assigned material shear strength parameters for the landslide deposits based upon laboratory testing performed by Cooper Testing Laboratories, as described above. We obtained soil strength parameters for the landslide gouge and in-situ bedrock units from the published values provided in the Seismic Hazard Zone Report for the Los Gatos 7½-Minute Quadrangle, Santa Clara County, California (California Geologic Survey, 2002). For the published bedrock units, we used the mean cohesion along with the recommended phi angle. For the landslide gouge layer, we modeled an approximate 1 foot thick gouge layer between the landslide deposits and the Rices mudstone, and utilized the recommended phi angle for the mapped “Qls” unit with a nominal cohesion. In addition, we assigned wet unit weights based upon laboratory testing and our experience in the area. A table of the soil and rock parameters is presented below.

Unit	Phi Angle (degrees)	Cohesion (psf)	Wet Unit Weight (pcf)
Landslide Deposits (Displaced Bedrock)	30	200	117
Gouge	12	1	110
Vaqueros Sandstone	34	656	125
Rices Mudstone	25	1009	125

Soil and Rock Parameters

5.4. Groundwater Conditions

We performed our analysis without an assumed permanent water table. As mentioned discussed above, we anticipate groundwater to be hundreds to several hundred feet below the site, and deeper than the modeled subject landslide.

5.5. Seismic Coefficient

A static (non-seismic) analysis was initially performed using no seismic coefficient. In accordance with California Geological Survey Note 48 (2019) and information included in Special Publication 117A, we derived a seismic coefficient of 0.29 for the site and utilized it in our pseudo-static analyses. The coefficient is based on peak ground acceleration (PGAm) of 0.95 computed using the ASCE 7 Hazard Tool based upon the ASCE/SEI 7-22 (2021) design standard and USGS Seismic Design Maps.

5.6. Slope Stability Analysis Results

Our analysis consisted of dozens of iterations to evaluate subsurface conditions and the potential for new or reactivated landsliding. Each analysis that we ran searched thousands of potential failure surfaces. The following is a summary of pertinent slope stability analysis results.

Slope Stability Analysis No. 1 and 2 evaluated the potential for reactivation of deep-seated landsliding under static and seismic conditions, respectively. Slope Stability Analysis No. 3 and 4 evaluated the potential for landsliding to initiate anywhere on the subject slope under static and seismic conditions, respectively.

The lowest factors of safety for each analysis is presented in the following table and graphical illustrations of potential failure surfaces are shown on Figures 16 through 19, Slope Stability Analysis No. 1 through 4).

Analysis No.	Slope	Seismic	Factor of Safety
1	Cross-Section A-A' (reactivation of landslide)	Static	2.22
2	Cross-Section A-A' (reactivation of landslide)	0.29	0.92
3	Cross-Section A-A' (initiating anywhere on the slope)	Static	1.67
4	Cross-Section A-A' (initiating anywhere on the slope)	0.29	0.88

Slope Stability Analyses and Results

5.7. Displacement Analysis

Because the pseudo-static (seismically induced) slope stability analyses yielded Factor of Safety values of less than 1.0, we also performed displacement analyses to evaluate the potential lateral displacement of both a reactivated landslide and landslide initiating anywhere on the subject slope during a large seismic event. Our analysis revealed that the most critical slide surface would have an average depth of approximately 150 feet for the reactivated landslide (see Figure 17) and about 80 feet for a landslide initiating anywhere on the subject slope (see Figure 19).

Using the Procedure for Estimating Shear-Induced Seismic Slope Displacement for Shallow Crustal Earthquakes by Bray and Macedo (2019), we obtained approximate slope displacement for the reactivated landslide with a 50% probability of exceedance. As indicated by Bray and Macedo, seismic displacement estimates will always be approximate in nature due to the complexities of the dynamic response of the earth materials involved and the variability of the earthquake ground motion.

We calculated displacements based upon a mean earthquake magnitude of 7.9 as denoted in the Seismic Hazard Zone Report for the Los Gatos 7.5-Minute Quadrangle (CGS, 2002). Shear wave velocities were estimated to be in the range of 2,500 feet per second. For the critical failures for the deep-seated landslide and a shallower landslide with an average depth of approximately 150 feet and 80 feet (see Figures 17 and 19), we derived an initial fundamental periods between 0.24 to 0.13 seconds, respectively with corresponding spectral accelerations of 1.74g and 1.81g based data from the ASCE/SEI 7-22 Hazard Report.

Based on a yield coefficients of 0.25g and 0.22g, fundamental periods, spectral acceleration, and depth to the critical landslide surface, both the deep-seated landslide and the shallower landslide revealed potential seismic slope-displacements on the order of approximately 15 inches with a 50% probability of exceedance during a strong seismic event.

6. FINDINGS

Based upon the results of our study, it is our opinion that, from a geotechnical engineering perspective, the subject property may be developed as planned, provided that the recommendations presented in this report are incorporated into the design and construction of the proposed improvements. In our opinion, the primary constraints to the proposed development include:

- The steep slope on the site;
- the presence of undocumented fill near the top of the steep slope and the potential for future fill creep or landsliding;
- the potential for earthquake-induced landsliding; and
- the site's seismic setting.

6.1. Proposed Building Site

Our subsurface study showed that the proposed building site is underlain at depth by displaced siltstone bedrock (displaced by landsliding; herein reference to bedrock includes bedrock displaced by landsliding). The supportive bedrock is blanketed by approximately 10 feet or less of non-supportive, undocumented fill or colluvium. Where located on moderate to steep slopes, this material can experience imperceptibly slow downhill creep under the force of gravity. The underlying, supportive material is comprised of competent siltstone. In our opinion, the siltstone should provide adequate support for the foundations of a proposed residence and associated improvements.

Standard penetration test results suggest that the siltstone at the site has variable consistency and can be very hard locally. We recommend that a high-powered, well-maintained drill rig equipped with rock teeth be used to drill the pier excavations. The contractor should plan for this condition in choosing the appropriate means and methods of drilling.

6.2. Proposed Drainfields

As currently proposed, the on-site wastewater treatment system's drip drainfields are planned in the southeast portion of the site and east of the proposed residence, with a minimum 10-foot setback to adjacent site improvements and property lines. Our subsurface study in this area revealed relatively dry soil conditions, and our slope stability evaluation revealed static slope stability with a factor of safety greater than 1.5. In our opinion the subsurface material underlying the subject site and area should be considered as a stable landmass for the purposes of siting an on-site wastewater treatment system.

Because of the moderate slopes in the area of the proposed drainfields, relatively shallow competent material, and setback from the property lines, it is our opinion that the proposed drainfields will not have a significant impact on the stability of the slope on the subject property. We understand that percolation testing in the area yielded acceptable percolation rates and that groundwater was not encountered. Consequently, the proposed drainfields should not degrade the quality of the local groundwater. In addition, in our opinion, it is unlikely that effluent from the drainfields will surface. Furthermore, it is unlikely that effluent introduced into the subsurface soil will present a threat to the public health and safety or create a public nuisance. From a geotechnical engineering perspective, the proposed drainfield may proceed as planned.

6.3. Slope Stability

As discussed above, the subject property is within the central portion of a large old landslide, and appears to be underlain by displaced competent siltstone bedrock. The results of our slope stability analysis suggest that the landslide could be about 150 feet thick. Following a strong seismic event, reactivation of portions of the slide could occur or a new slide could develop, upslope of the subject property. Our slope stability analysis shows potential for landslide displacements within an order of magnitude of one foot as a result of seismic shaking. In our opinion, the amount of modeled movement could result in damage to your property or residence. The risk of damage can be mitigated (but not eliminated) by improving the rigidity of the foundation system in accordance with the recommendations presented below.

Additionally, and because of the steep slopes, undocumented fill and colluvium, the occurrence of a new shallow landslide within or adjacent to the subject property cannot be excluded. A new shallow landslide (approximately less than 10 feet deep) in this area could be triggered by excessive precipitation or strong ground shaking associated with an earthquake. In our opinion, a landslide of this nature should not constitute an immediate threat to the integrity of the proposed residence and associated improvements, provided that they are designed and constructed in accordance with the recommendations of this report.

Generally speaking, in our opinion, the risk of landsliding to affect the subject property is greater than that of the average hillside residential property in Santa Clara County. Provided you are aware of and willing to accept these conditions, we believe the modeled movement and proposed mitigation measures constitute an acceptable level of risk.

The long-term stability of many hillside areas is difficult to predict. A hillside will remain stable only as long as the existing slope equilibrium is not disturbed by natural processes or human activity. Landslides can be activated by a number of natural processes, such as the loss of support at the bottom of a slope by stream erosion or the reduction of soil strength by an increase in groundwater level from excessive precipitation. Artificial processes caused by human activity may include improper grading activities; or the introduction of excess water through excessive irrigation and improperly designed or constructed leachfields; or poorly controlled surface runoff.

Although our knowledge of the causes and mechanisms of landslides has greatly increased in recent years, it is not yet possible to predict with certainty exactly when and where all landslides will occur. At some time over the span of thousands of years, most hillsides will experience landslide movement as mountains are reduced to plains. Therefore, a small but unknown level of risk is always present to structures located in hilly terrain. Owners of property located in these areas must be aware of, and willing to accept, this unknown level of risk.

6.4. Seismicity

Our reconnaissance and review of published geologic maps and aerial photographs revealed that no known active or potentially active faults pass through the subject property. However, it is reasonable to anticipate that the site will be subjected to strong to very violent ground shaking from a major earthquake on at least one of the nearby active faults during the design life of future improvements (Borcherdt, et. al., 1975). During such an earthquake, it is our opinion that the danger from fault offset through the site is negligible. It should be noted that following a large seismic event, cosmetic damage caused by ground shaking or sympathetic movement may occur and may have to be repaired.

7. RECOMMENDATIONS

Because the proposed project is still in a relatively early phase of development, it is conceivable that changes and additions will be made to the proposed development concept following submission of this report. We recommend that as various changes and additions are made, you contact us to evaluate the geotechnical aspects of these modifications.

As currently planned, a new single-family residence with a daylighting basement is planned in southwestern portion of the site. A new, short, driveway will access the property and lead to a detached garage in the northern portion of the property. Because of the steep slopes, we anticipate that site retaining walls or elevated decks and walkways will be constructed for landscaping purposes.

The following recommendations must be incorporated into all aspects of future development.

7.1. Location of Proposed Improvements

The proposed improvements must be confined to the approximate building area shown on Figure 7. Do not construct improvements outside of this generalized area without written approval from C2. If other structures are planned in the future, we must evaluate their location to provide appropriate geotechnical engineering design criteria.

7.2. Seismic Design Criteria

We recommend that the project structural design engineer provide appropriate seismic design criteria for proposed foundations and associated improvements. The following information is intended to aid the project structural design engineer to this end and is based on criteria set forth in the 2022 California Building Code (CBC). The mapped spectral accelerations and site coefficients have been computed using the ASCE/SEI 7-22 and 7-16 design standards, and the ASCE 7-22 and 7-16 Hazard Reports are presented in the Appendix of this report. The structural designer should select the applicable standard for use, and confirm the Seismic Risk Category, and make revisions as they deem appropriate.

Experience has shown that earthquake-related distress to structures can be substantially mitigated by quality construction. We recommend that a qualified and reputable contractor and skilled craftsmen build the associated improvements. We also recommend that the project structural design engineer and project architect monitor the construction to make sure that their designs and recommendations are properly interpreted and constructed.

7.3. Earthwork

At the time of this study, the full extent of any proposed earthwork had not been finalized. We anticipate that a significant amount of grading will be required to construct the proposed improvements. Where practical, we recommend that the existing undocumented fill be removed or replaced as engineered fill that is retained with retaining walls. Alternatively, the existing fill may remain in place provided the foundations are supported in the bedrock and designed to resist lateral loads from the fill and non-supportive material. Any proposed earthwork should be performed in accordance with the recommendations provided below.

7.3.1. Clearing and Site Preparation

- Clear all obstructions, including brush, trees not designated to remain, and debris on any areas to be graded.
- Clear and backfill any holes or depressions resulting from the removal of underground obstructions below proposed finished subgrade levels with suitable material compacted to the requirements for engineered fill given below.
- After clearing, strip the site to a sufficient depth to remove all surface vegetation and organic-laden topsoil. At the time of our field study, we estimated that a stripping depth of approximately 3 inches would be required on natural slope areas. This material must not be used as engineered fill; however, it may be used for landscaping purposes.

7.3.2. Fill Material

Based on our study, it is our opinion that the materials encountered in the borings should be suitable for use as fill. On-site materials must meet the requirements specified below to be used as engineered fill:

- Materials used for engineered fill must meet the following requirements:
  1) Have an organic content less than 3% by volume;
  2) no rocks or lumps greater than 6 inches in maximum dimension;
  3) no more than 15% of the fill may be greater than 2½ inches in maximum dimension; and
- If on-site materials do not meet the requirements given above, they may be off-hauled or used for landscaping purposes only.
- **Contact C2 with samples of proposed fill materials at least four days prior to fill placement for laboratory testing and evaluation.**

7.3.3. Benches

- Because of the steep slopes, fill must be retained with retaining walls and be benched into suitable material to provide a firm, stable surface for support of the fill.
- Benches generally must be a minimum of 4 feet wide.
- Temporary back slopes may be vertically excavated provided they are constructed in the dry season and meet Cal OSHA requirements.
- Benches must be excavated near level in the direction parallel to the natural slope and must be provided with an approximately 2% gradient sloping into the hillside to provide resistance to lateral movement and to facilitate proper subdrainage.
- **Contact C2 to evaluate the actual location, size, and depth of the required benches at the time of construction**.

7.3.4. Subdrains

- C2 must determine the need for subdrains at the time of construction.
- In general, fill exceeding 5 feet deep should be provided with subdrains.
- Subdrains must consist of a 4-inch diameter, rigid, heavy-duty, perforated pipe (Schedule 40, SDR 17, or equivalent), approved by C2, embedded in drainrock (crushed rock or gravel).
- Flexible corrugated pipe must not be used.
- The pipe must be placed with the perforations down on a 2- to 3-inch bed of drainrock. The drainrock must be separated from the fill and the native material by a geotextile filter fabric, approved by C2.
- Subdrain pipes must be provided with cleanout risers at their up-gradient ends and at all sharp changes in direction.
- Changes in pipe direction must be made with "sweep" elbows to facilitate future inspection and cleanout.
- Subdrain systems must be provided with a minimum 1% gradient and must discharge onto an energy dissipater at an appropriate downhill location approved by C2.

7.3.5. Compaction Procedures

- Prior to fill placement, scarify the surface to receive the fill to a depth of 6 inches.
- Moisture condition the fill to the materials' approximate optimum moisture content.
- Spread and compact the fill in lifts not exceeding 8 inches in loose thickness.
- Compact the fill to at least 90% relative compaction by the Modified Proctor Test method, in general accordance with the ASTM Test Designation D1557 (latest revision).
- **Contact C2 to observe the placement and test the compaction of engineered fill.** Provide at least two working days notice prior to placing fill.

7.3.6. Permanent Slopes

- Construct the gradients of permanent cut or fill slopes no steeper than 2:1.
- Re-vegetate all graded surfaces or areas of disturbed ground prior to the onset of the rainy season following construction to control soil erosion.
- Install other erosion control provisions if vegetation is not established by the rainy season.
- Maintain ground cover vegetation once it is established to provide long-term erosion control.

7.3.7. Trench Backfill

- Backfill all utility trenches with compacted engineered fill.
- Place suitable on-site soil into the trenches in lifts not exceeding 8 inches in uncompacted thickness, and compact it to at least 90% relative compaction by mechanical means only.
- If imported sand is used, compact it to at least 90% relative compaction. Do not use water jetting to obtain the minimum degree of compaction in imported sand backfill. If the trench is greater than 50 feet long, located on sloping ground greater than 5:1 (horizontal to vertical), and is backfilled with sand, check dams should be installed to reduce the potential of the sand washing out.
- Compact the upper 6 inches of trench backfill to at least 95% relative compaction in all pavement areas.
- **Contact C2 to observe and test compaction of the fill.**

7.3.8. Basement Excavations

- Excavate for the proposed retaining wall along the uphill side of the building footprint using an OSHA approved benching or sloping cut configuration selected by an OSHA "Competent Person". The Competent Person must be capable of identifying hazards during construction, such as slope instability, and take prompt corrective measures to mitigate any potential hazard.

- To aid the Competent Person in their selection of construction means and methods, consider the on-site engineered fill to be an OSHA Soil Type A. The Competent Person must evaluate the excavation during construction and confirm the suggested OSHA soil classification type.
- As an alternative to benching and sloping, shoring may be used to support temporary cuts. Consult with a shoring specialist for the design and installation of temporary shoring.
- The contractor is solely responsible for means and methods of construction and should designate appropriate personnel to act as the Competent Person.
- **Contact C2 to observe the subsurface conditions** exposed within the excavations to assess whether they are consistent with expected subsurface conditions.

7.4. Foundations

Because of the potential for landsliding, moderately steep slope, the site's seismic setting, and the presence of fill and/or colluvium in the area of the proposed residence and garage, we recommend that the structures be supported on a drilled, cast-in-place, straight-shaft concrete friction pier foundation interconnected with grade beams or a structural slab. In our opinion, a pier foundation, gaining support in the underlying bedrock, designed and constructed in accordance with the following recommendations will reduce the risk of differential foundation movement as a result of earthquake-induced landsliding. Site retaining walls must be supported on drilled pier foundations, in accordance with the recommendations provided below under the section headed "Retaining Walls".

We recommend that your engineer design and your contractor construct the proposed foundation elements in accordance with the following recommendations.

7.4.1. Drilled Piers

- Drill piers with a minimum diameter of 16 inches and embed them a minimum of 8 feet or the depth of overburden (which ever is greater) into the underlying bedrock below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the pier and the surface of the bedrock.
- Design and construct drilled piers no closer than 3 pier diameters apart (measured center of pier to center of pier).
- Neglect the upper 2 feet of bedrock for support and consider active and passive pressure loading to be negligible within the upper 2 feet of the bedrock (to achieve the recommended 5 feet of horizontal separation between the downhill face of the pier and the surface of the bedrock).
- Total pier depth will vary across the building site depending on the depth of the non-supportive soil and the extent of prior grading.

- Design the portion of the piers in the supportive bedrock using a skin friction value of 450 psf for dead plus live loads, with a ⅓ increase for transient loads, including wind and seismic.
- Neglect any portion of the piers in fill and non-supportive colluvium and any point-bearing resistance for support.
- Figure active loads on the upper portion of the piers in the fill and colluvium on the basis of an equivalent fluid weight of 45 pcf taken over **2 times** the pier diameter. The depth of the active loads will vary across the building site depending on the depth of prior grading. Where the fill and surficial soil is removed by grading, active loads will be negligible. Where proposed structures are built at existing grades, active loads may extend to depths of approximately 8 feet. To facilitate construction, it may be appropriate for the structural engineer to prepare a table that provides pier depths and design based on various wall heights and an active zone that could vary from 0 to 8 feet.
- Design for resistance to lateral loads using a passive pressure equal to an equivalent fluid weight of 400 pcf to a maximum of 4,000 psf taken over **1½ times** the pier diameter for the length of the piers in the bedrock below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the pier and the surface of the bedrock (see Figure 20, Conceptual Pier Pressure Diagram).
- Because the bedrock has variable consistency and can be locally very hard, it may be difficult to drill. The contractor should plan for this conditions and choose the appropriate means and methods of drilling.
- Clear the bottoms of the pier excavations of loose cuttings and soil fall-in prior to the installation of the reinforcing steel and the placement of concrete.
- Remove any accumulated water in the excavations prior to the placement of the steel and concrete.
- Reinforce the piers with a full-length cage containing a minimum of four No. 5 steel reinforcing bars.
- The structural engineer must determine the actual number, size, location, depth, spacing, and reinforcement of the piers, based on the anticipated building loads and the soil engineering design parameters provided above.
- **Contact C2 to observe the piers as they are being drilled** to assess whether the piers are founded in material of sufficient supporting capacity.

7.4.2. Grade Beams

- Interconnect the piers supporting the residence with a rigid grid of grade beams, or alternatively a structural concrete slab. Specify a maximum grid spacing of 10 feet by 10 feet, measured center of grade beam to center of grade beam.
- Design any grade beam retaining more than 2 feet of soil for an active pressure of 45 pcf.

- Reinforce grade beams with top and bottom reinforcement to provide structural continuity and to permit the spanning of local irregularities.
- Provide good structural continuity between the grade beam and the piers.
- The structural design engineer must determine the actual size and reinforcement of the grade beams.
- Remove any concrete overpour before the concrete has achieved its design strength.

7.4.3. Retaining Walls

We anticipate that retaining walls will be used for building and landscaping purposes. The following recommendations are for cantilever type walls. Contact us to provide appropriate recommendations if you consider other types of walls.

- Support residential and site retaining walls on drilled pier foundations designed in accordance with the recommendations given above.
- Design retaining walls to resist both lateral earth pressures and any additional lateral loads caused by surcharge loads on the adjoining ground surface.
- Deflection of cantilever retaining walls will occur in response to lateral loading. Anticipate horizontal deflections at the top of the wall to be 2% of the wall height or less.
- Design unrestrained (active condition) walls to resist an equivalent fluid pressure of 45 pcf. Design walls that are restrained from movement at the top or sides (at-rest condition) to resist an equivalent fluid pressure of 67 pcf (see Figure 21, Conceptual Pier and Retaining Wall Pressure Diagram).
- Add an additional equivalent fluid pressure increment to the active and at-rest condition for backfill steeper than 4:1 (horizontal to vertical), in accordance with the following:

  + 8 pcf for slopes between 3:1 and 4:1

  + 12 pcf for slopes between 2:1 and 3:1

  + Contact us for slopes steeper than 2:1

- Design for seismic-loading as the structural engineer deems appropriate. In our opinion, the requirements for seismic design of retaining walls are not clearly defined. If the structural engineer considers seismic loading, based upon the procedures presented by Sitar, et. al. (2012), design unrestrained (active condition) residential retaining walls to resist an additional earthquake equivalent fluid pressure (seismic increment) of 39 pcf.
- If seismic loading is considered, design building retaining walls to resist the appropriate loading condition: either the at-rest condition if the walls are restrained, or the active condition plus the seismic increment if the walls are unrestrained.

- Site walls less than 6 feet tall are not subject to additional earthquake loading requirements.
- Wherever the walls will be subjected to surcharge loads, they must be designed for an additional uniform lateral pressure equal to ½ or ⅓ the anticipated surcharge load for restrained or unrestrained walls, respectively.
- The preceding pressures require that sufficient drainage be provided behind the walls to prevent the buildup of hydrostatic pressures from surface or subsurface water infiltration.
- Provide a backdrain system consisting of an approximately 1 foot thick curtain of drainrock (crushed rock or gravel) behind the wall. Separate the drainrock from the backfill by a geotextile filter fabric, such as Mirafi 140 or an alternative, approved by C2.
- Place a 4-inch diameter heavy-duty rigid perforated subdrain pipe (SDR 35 or equivalent), approved by C2, with the perforations down on a 2- to 3-inch layer of drainrock at the base of the drain. **Do not use flexible corrugated pipe.**
- As an alternative, back drainage may consist of an approved drainage mat placed directly against the wall. The bottom of the drainage mat must be in contact with the rigid 4-inch perforated drainpipe embedded in gravel. The gravel must be fully encased in filter fabric.
- If lagging walls are planned, provide a backdrain system consisting of an approximately 1-foot wide curtain of ¾-inch crushed drainrock (crushed rock or gravel), wrapped in filter fabric, placed behind the wall. Provide ½-inch spacers between lagging boards to promote weeping and allow dissipation of hydrostatic pressure.
- The backdrains should extend up the height of the back of the retaining walls to within 1 foot of the height of the retained soil, and then be covered with a compacted clay soil cap or other impervious materiel.
- Install a drainage swale at the top of the wall to collect and direct surface water away from the retaining wall. Do not allow surface water to infiltrate into the retaining wall backdrainage system.
- Details of backdrain options are presented on Figure 22, Conceptual Retaining Wall Backdrain Diagram.
- Perforated retaining wall subdrain pipes must be dedicated pipes and must not connect to the surface drain system. Install the subdrain pipes with a positive gradient of at least 1% and provide them with cleanout risers at their up-gradient ends and at all sharp changes in direction. Changes in pipe direction must be made with "sweep" elbows to facilitate future inspection and cleanout. The perforated pipes must be connected to buried solid pipes to convey collected runoff to discharge onto an energy dissipater at an appropriate downhill location, approved by C2.

- Compact the backfill placed behind the walls to at least 90% relative compaction, using light compaction equipment, in accordance with the compaction procedures given above. If heavy compaction equipment is used, the walls should be appropriately temporarily braced, as the situation requires. If backfill consists entirely of drainrock, it should be placed in approximately 2-foot lifts and must be compacted with several passes of a vibratory plate compactor.
- Perform annual maintenance of retaining wall backdrain systems, which must include inspection and flushing to make sure that subdrain pipes are free of debris and are in good working order. This maintenance must also include inspection of subdrain outfall locations to verify that introduced water flows freely through the discharge pipes and that no excessive erosion has occurred.
- If erosion is detected, C2 must be contacted to evaluate its extent and to provide mitigation recommendations, if needed.
- Damp proof retaining walls that are adjacent to living spaces and/or site walls with decorative facing. We are not qualified to recommend specific damp proofing materials or their applications. Any damp proofing product must be applied in **strict** compliance with the manufacturer's and/or architect's specifications.

7.4.4. Flatwork

We anticipate that pavement or concrete slabs-on-grade may be used for the driveway and walkways. Where located on undocumented fill and/or colluvium, the overlaying flatwork will be subject to downslope migration and differential movement. We believe this condition will result in minor ongoing cosmetic damage to the flatwork. To mitigate the risk of differential movement of the flatwork, we recommend the following options:

- Option 1: Construct the flatwork using a flexible pavement system that can accommodate differential movement, such as pavers.
- Option 2: Remove and replace the colluvium and/or undocumented fill with engineered fill, retained with retaining walls and benched into the supportive material in accordance with the recommendation provided above.

For pavers we recommend the following minimum requirements:

- Support pavers on a minimum of 6 inches of non-expansive fill compacted to the requirements for compacted fill given above.
- Proof-roll the surface of the non-expansive fill to provide a smooth, firm surface, then place pavers on a leveling course per manufacturer's recommendations.
- Periodically repair cracked pavers or re-level pavers that experience differential movement.

For concrete slabs-on-grade we recommend the following minimum requirements:

- Support concrete slabs-on-grade on a minimum of 6 inches of non-expansive fill compacted to the requirements for compacted fill given above.
- Proof-roll the surface of the non-expansive fill to provide a smooth, firm surface for slab support prior to placement of reinforcing steel.
- Design slab reinforcement in accordance with anticipated use and loading, but at a minimum, reinforce slabs with No. 3 rebar on 18-inch centers each way, placed mid-height in the slab.
- Support the reinforcing steel from below on concrete blocks (or similar) during concrete pouring to make sure that it remains mid-height in the slab.
- Place grooves in the concrete slabs at 10-foot intervals or in accordance with the structural design engineer's recommendations to help control cracking.
- The structural designer must evaluate moisture conditions related to concrete slab curing and performance. The builder must provide appropriate drying time as determined by the designer.

Where floor wetness is undesirable:

- The building designer or qualified waterproofing consultant must provide moisture barrier requirements.
- The following recommendations are typical moisture barrier standards. We do not guarantee that these measures will prevent all future moisture intrusion. If necessary, you should consult a qualified waterproofing consultant to provide waterproofing design.
- We recommend as a minimum using a puncture resistant, heavy-duty membrane (such as a minimum of 15 mil Stego Wrap, or equivalent) in direct contact with the floor slab and underlain by 6 inches of free-draining gravel.
- Use the gravel and heavy-duty membrane in lieu of the upper 6 inches of recommended non-expansive fill.

7.5. Drainage

Control of surface drainage is critical to the successful performance of the proposed improvements. The results of improperly controlled runoff may include foundation heave and/or settlement, erosion, gullying, ponding, and potential slope instability. To mitigate the risks associated with improperly controlled runoff, we recommend that you implement the following:

- Prevent surface water from ponding in areas adjacent to the foundation of the proposed residence and associated improvements by grading adjacent areas to create proper drainage by sloping them away from the structures.
- As an alternative, install area drains to collect surface runoff.

- Provide roof gutters with downspouts on the structures. Provide downspouts with slip-joint connectors or cleanouts, where they are connected to buried pipes, to facilitate maintenance (see Figure 23, Conceptual Downspout Cleanout Diagram).
- Do not allow water collected in the gutters to discharge freely onto the ground surface adjacent to the foundation.
- Convey water from downspouts and/or area drains away from the residence and garage via buried, closed conduits or lined surfaces. Use buried conduits consisting of rigid, smooth-walled pipes (PVC). **Do not use flex-pipes.**
- Discharge collected water in an appropriate manner and at an appropriate location approved by C2.
- Convey all collected water away from the structures via buried, closed conduit or hard surfaced drainage way, and discharge onto an energy dissipater at an appropriate downslope location approved by C2. Energy dissipaters may consist of a short "T" fitting placed in a shallow bench and covered or surrounded with cobbles (see Figure 24, Conceptual Energy Dissipater Diagram). The discharge must not be located on, or adjacent to, steep, potentially unstable terrain or where runoff will adversely impact adjacent parcels.
- Perform annual maintenance of the surface drainage systems, including:
  1) Inspecting and testing roof gutters and downspouts to make sure that they are in good working order and do not leak;
  2) inspecting and flushing area drains to make sure that they are free of debris and are in good working order; and
  3) inspecting surface drainage outfall locations to verify that introduced water flows freely through the discharge pipes and that no excessive erosion has occurred.
- **Contact C2 if erosion is detected** so that we may evaluate its extent and provide mitigation recommendations, if needed.

8. PLAN REVIEW AND CONSTRUCTION OBSERVATION

We must be retained to review the final grading, foundation, and drainage control plans in order to assess whether our recommendations have been properly incorporated into the proposed project. **WE MUST BE GIVEN AT LEAST TWO WEEKS TO REVIEW THE PLANS AND PREPARE A PLAN REVIEW LETTER.**

We must also be retained to observe the grading and the installation of foundations and drainage systems in order to:

- assess whether the actual soil conditions are similar to those encountered in our study;
- provide us with the opportunity to modify the foundation design, if variations in conditions are encountered; and
- observe whether the recommendations of our report are followed during construction.

Sufficient notification prior to the start of construction is essential, in order to allow for the scheduling of personnel to ensure proper monitoring.

WE MUST BE NOTIFIED AT LEAST TWO WEEKS PRIOR TO THE ANTICIPATED START-UP DATE. IN ADDITION, WE MUST BE GIVEN AT LEAST TWO WORKING DAYS NOTICE PRIOR TO THE START OF ANY ASPECTS OF CONSTRUCTION THAT WE MUST OBSERVE.

The phases of construction that we must observe include, but are not necessarily limited to, the following.

1. **EARTHWORK:** During construction to observe excavations, evaluate the need for subdrainage, and to test compaction of engineered fill
2. **DRILLED PIER EXCAVATION:** During drilling to evaluate depth to supportive material and final pier depths
3. **RETAINING WALL BACKDRAIN:** During installation
4. **RETAINING WALL BACKFILL:** During backfill to observe and test compaction
5. **FLATWORK:** Prior to and during placement of non-expansive fill to observe the subgrade preparation and to test compaction of non-expansive fill
6. **SURFACE DRAINAGE SYSTEMS:** Near completion to evaluate installation and discharge locations

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A Bibliography, a List of Aerial Photographs, and the following Figures, Table, and Appendix are attached and complete this report.

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FIGURES AND TABLE

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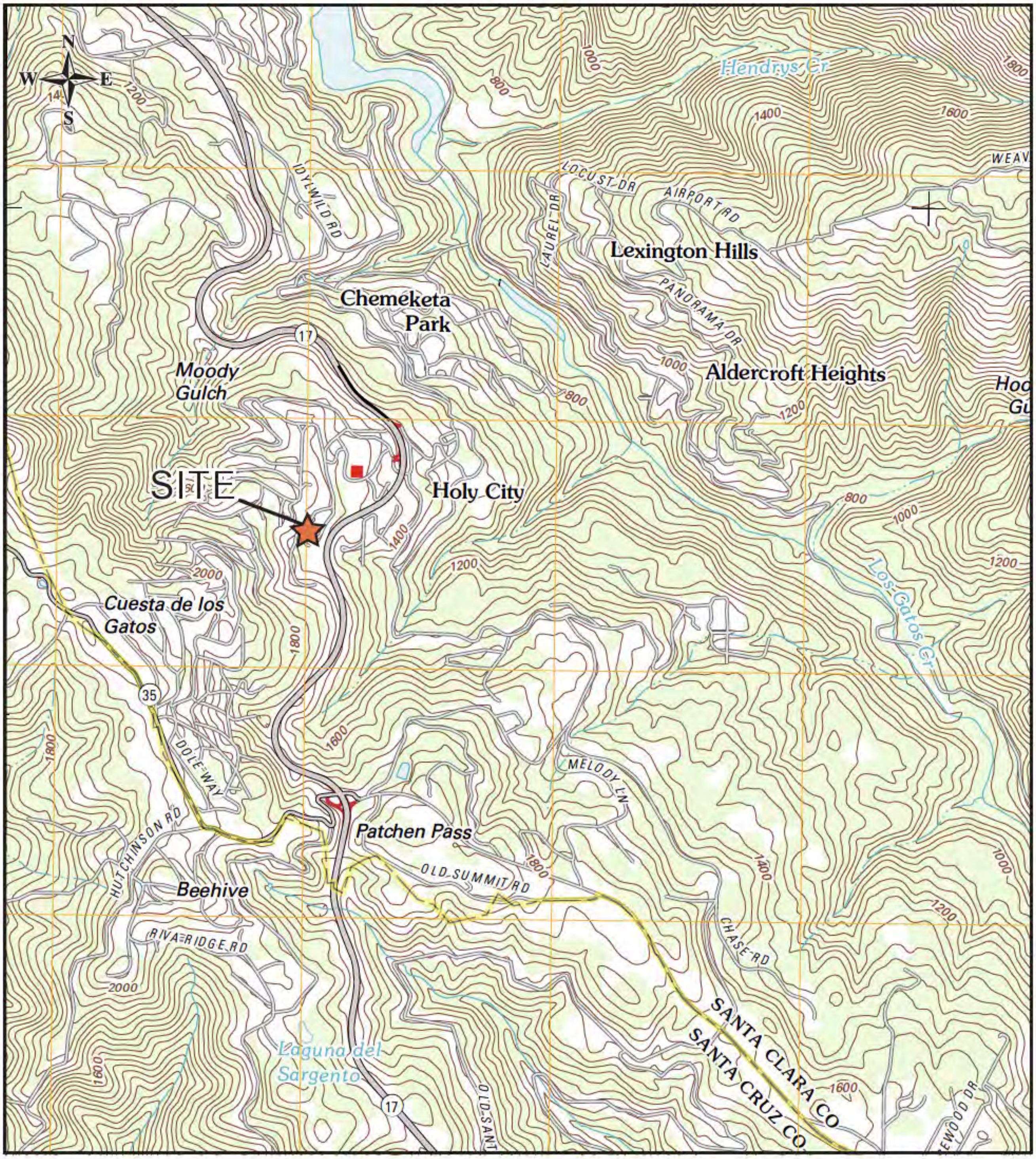
TABLE NO.

MODIFIED MERCALLI SCALE OF EARTHQUAKE INTENSITIES.............................I



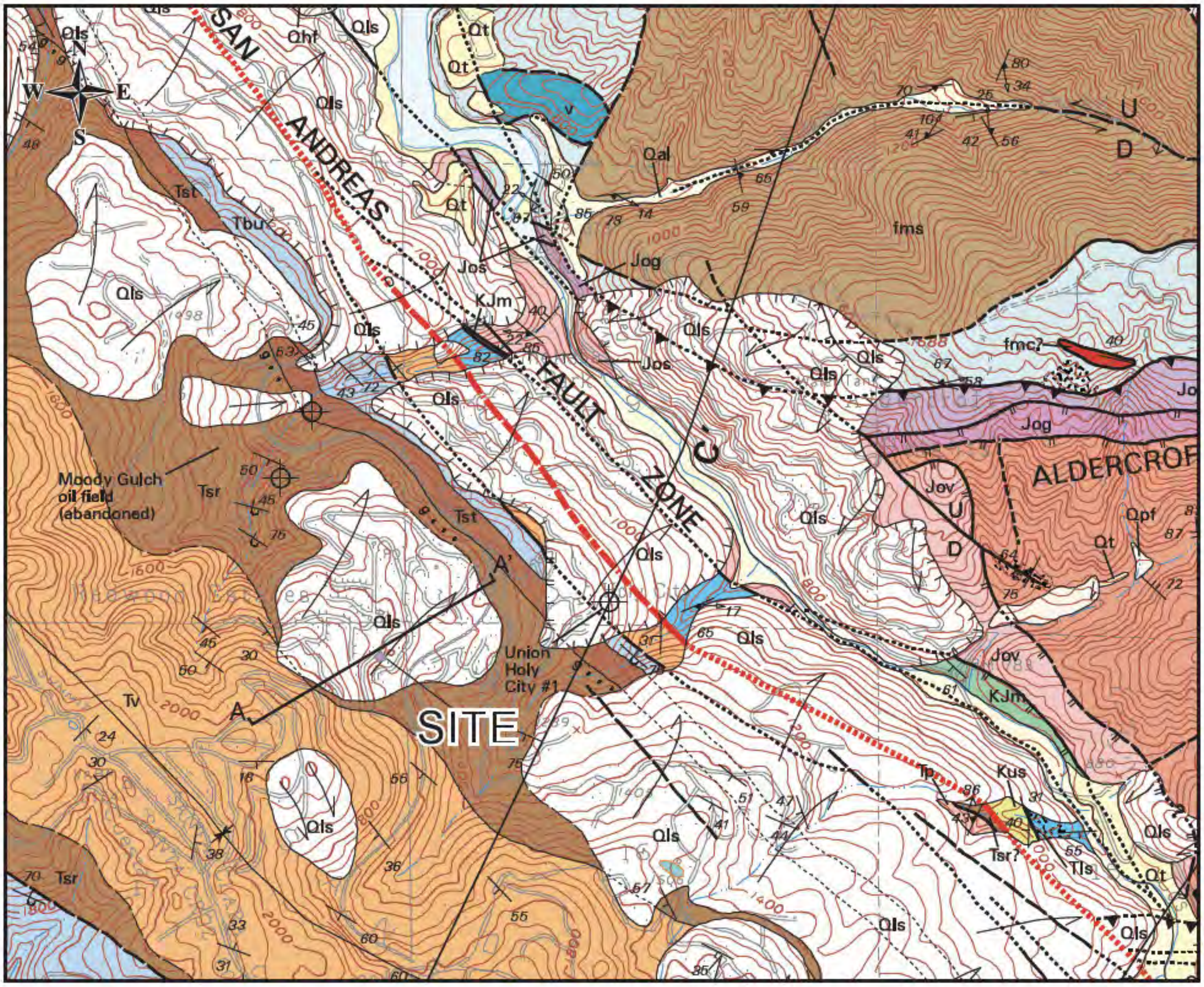
BASE: The National Map US Topo; UNITED STATES GEOLOGICAL SURVEY; 2012

SITE LOCATION MAP

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	1" = 2,000'	23103C-01R1	October 2023	Figure 1



Qpf	-	Alluvial fan deposits
Qls	-	Landslide deposits
Tsr	-	Rices mudstone member
Tv	-	Vaqueros sandstone
Tst	-	Twobar shale member
Tbu	-	Butano sandstone
v	-	Basaltic volcanic rock blocks
fms	-	Sandstone

EXPLANATION

A A' Cross-Section Location

15 Strike and Dip

15 Overturned Bedding

15 Inclined Shear Foliation

Thrust Fault
barbs on upper plate

Syncline
fold axis

Geologic Contact
dashed where approximate and dotted where concealed

D U Fault
U and D denote upthrown and downthrown blocks, dashed where approximate and dotted where concealed

Landslide
head scarp
general direction of movement

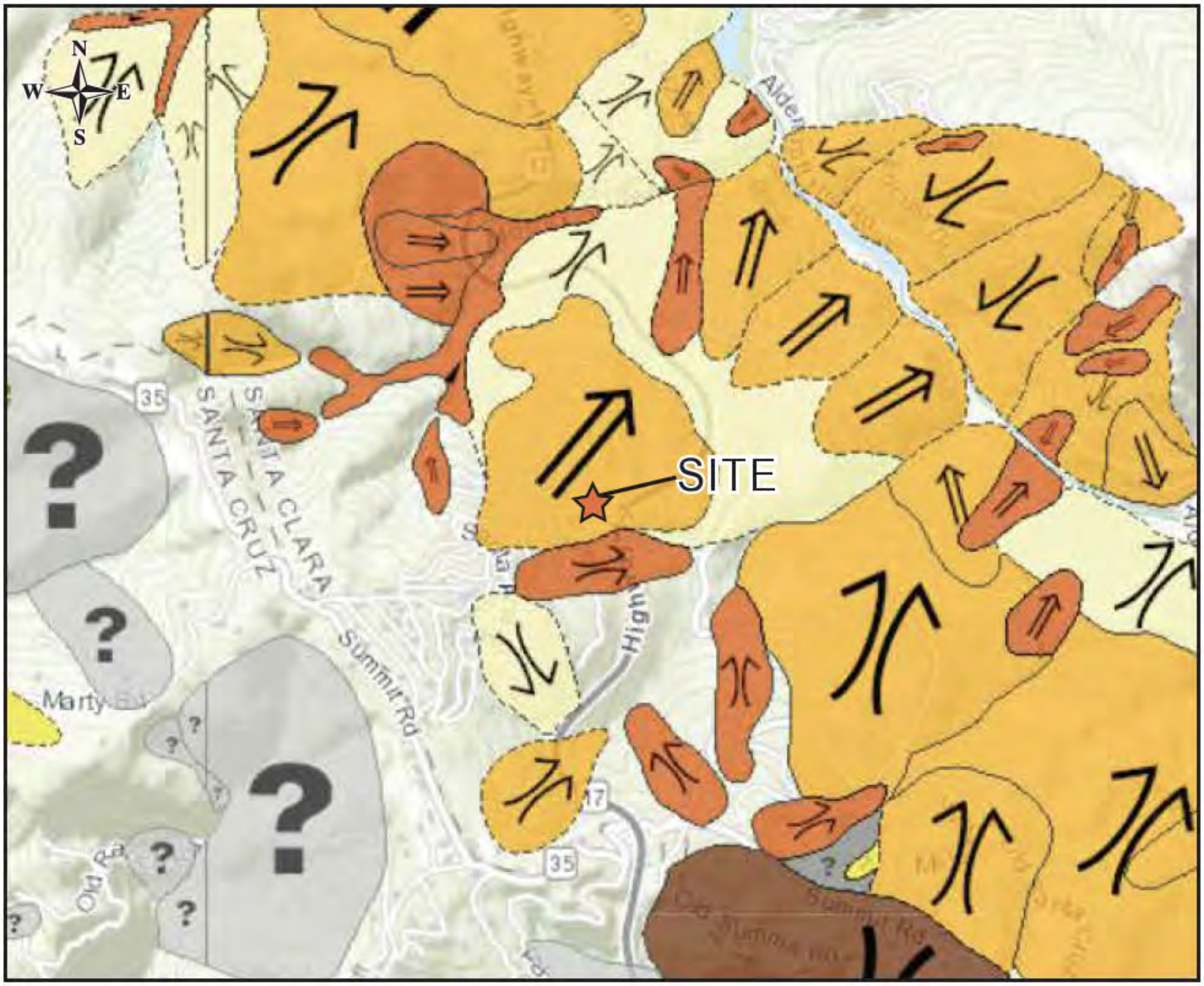
BASE: Sheet 1: Los Gatos Quadrangle, Geologic Maps and Structure Section of the Southwestern Santa Clara Valley and Southern Santa Cruz Mountains, Santa Clara and Santa Cruz Counties, California; MCLAUGHLIN, ET AL.; 2001

REGIONAL GEOLOGIC MAP

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	1" = 2,000'	23103C-01R1	October 2023	Figure 2



EXPLANATION

Landslide Activity

- Active or Historic
- Dormant, Young
- Dormant, Mature
- Dormant, Old

Landslide Types

- Rock Slide
- Soil Slide

Interpretation

- Definite
- Probable
- Questionable

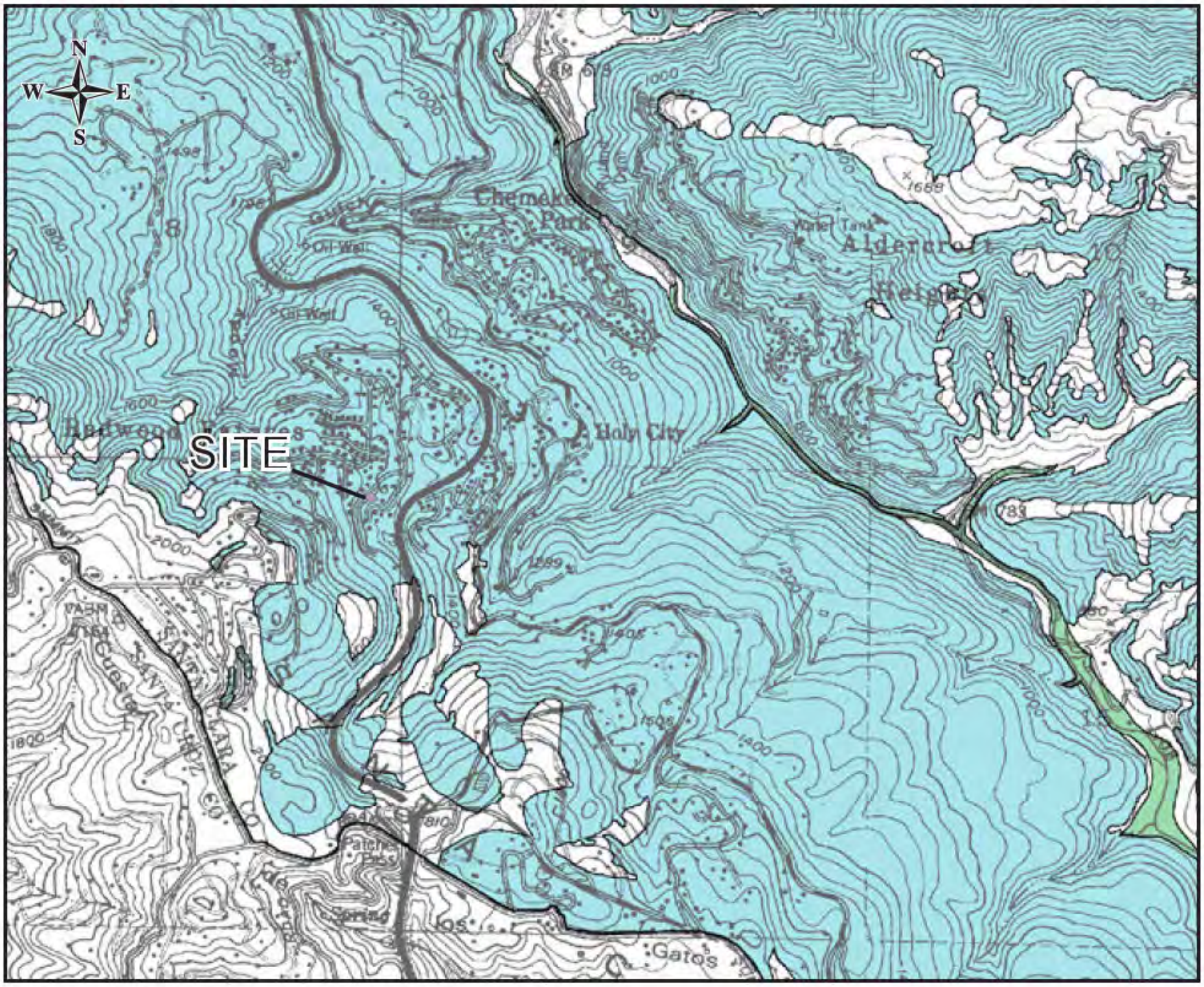
BASE: Landslide Inventory Map of the Los Gatos 7.5-minute Quadrangle Plste 2.1, Santa Clara County, California, Seismic Hazard Zone Report 069, California Geological Survey; 2002

REGIONAL LANDSLIDE INVENTORY MAP

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	1" = 2,000'	23103C-01R1	October 2023	Figure 3



EXPLANATION

- **Earthquake-Induced Landslide Hazard Zones;** Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

- **Earthquake-Induced Liquefaction Hazard Zones;** Areas where historic occurrence of liquefaction, or local topographic, geological, geotechnical, and subsurface water conditions indicate a potential for permanent ground displacements.

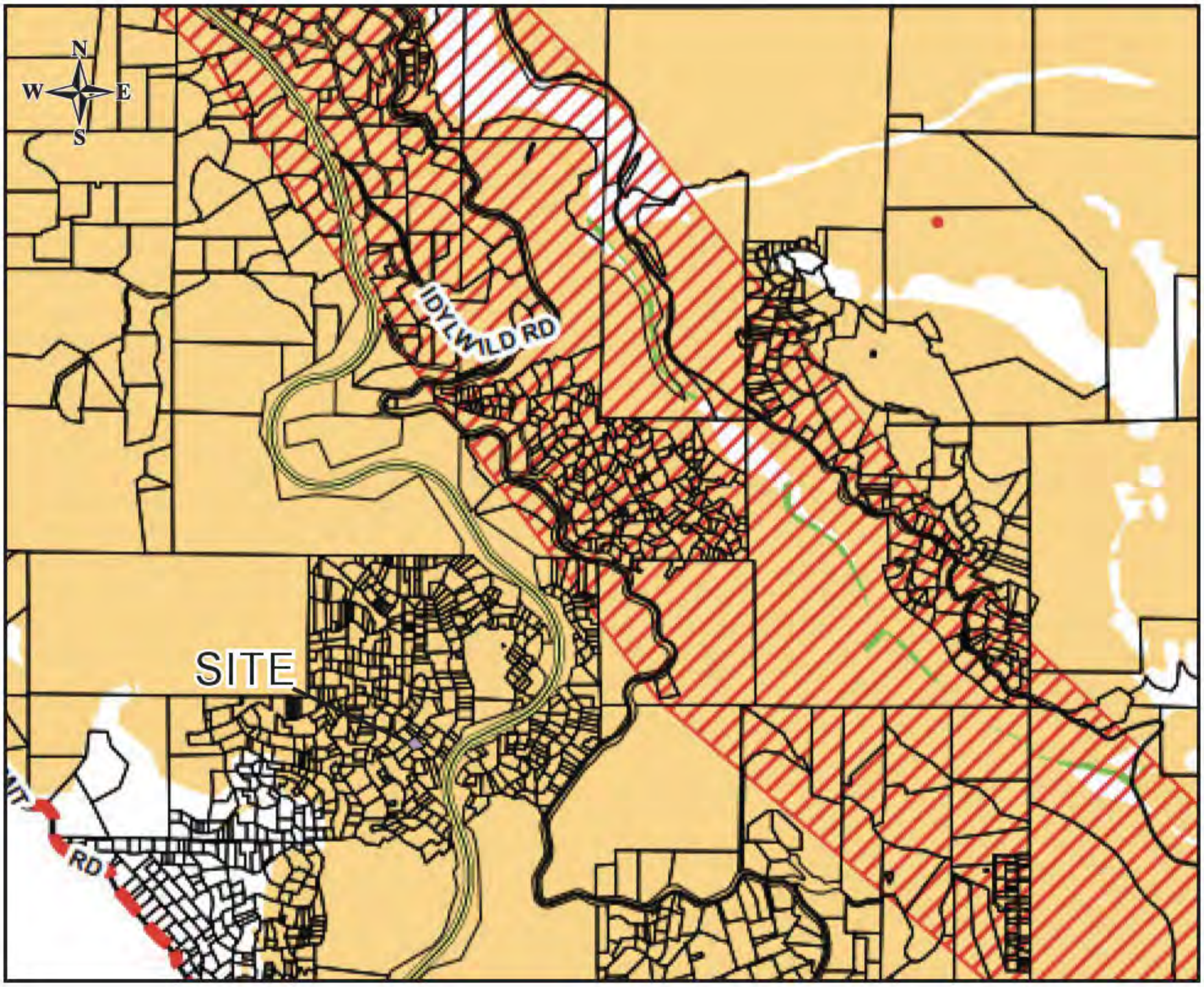
BASE: Earthquake Zones of Required Investigation, Los Gatos Quadrangle; California Geological Survey; 23 September 2002

REGIONAL SEISMIC HAZARD ZONES MAP

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	1" = 2,000'	23103C-01R1	October 2023	Figure 4



EXPLANATION

- Fault Rupture Hazard Zones
- Landslide Hazard Zones
- Liquefaction Hazard Zones

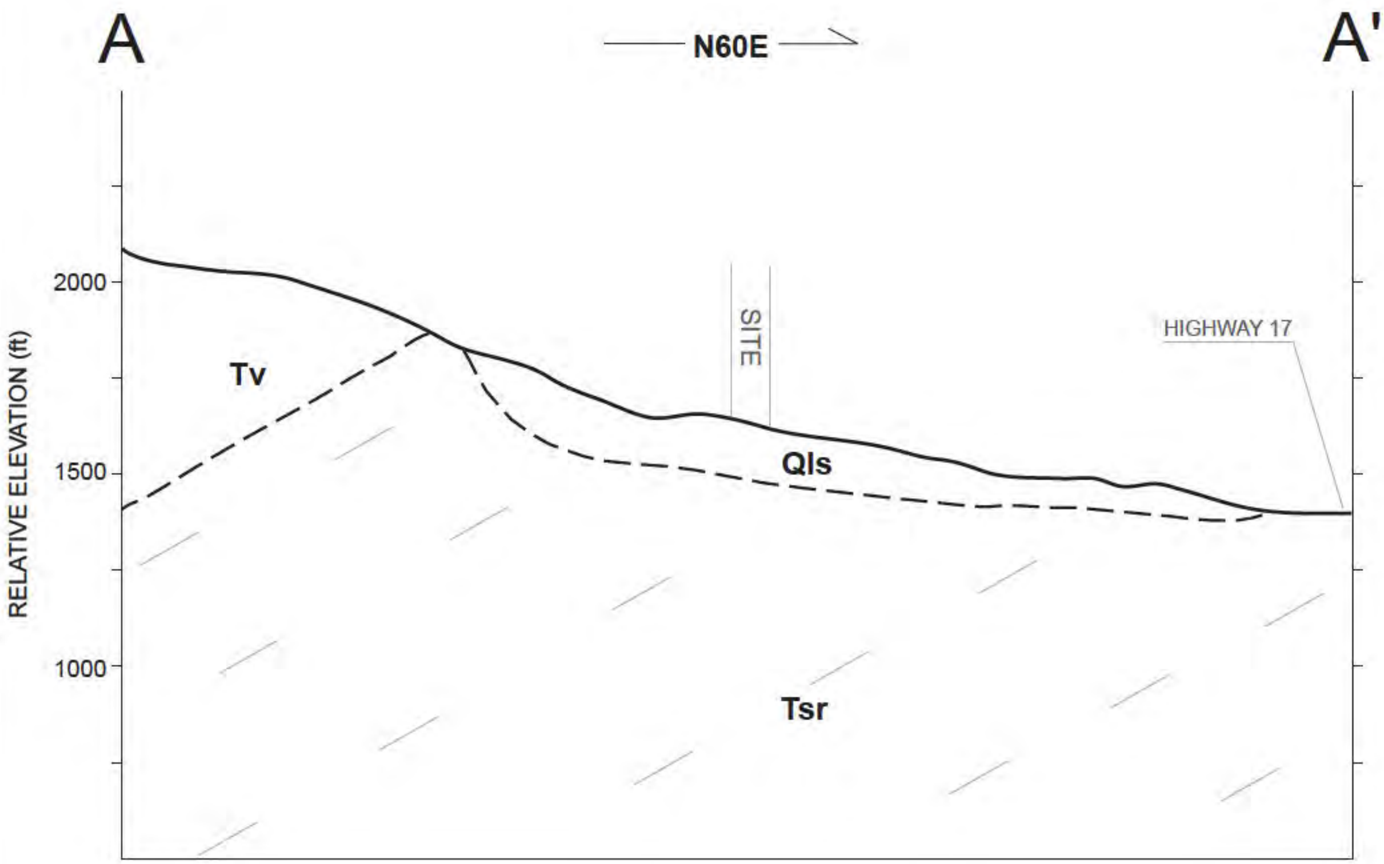
BASE: Santa Clara County Geologic Hazard Zones; Fault Rupture, Landslide, and Liquefaction Hazard Zones; Sheet 42; 26 October 2012

COUNTY HAZARD ZONES MAP

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	1" = 2,000'	23103C-01R1	October 2023	Figure 5



NOTE: This cross-section is a conceptual illustration of general geologic relationships and should not be used for any other purpose. Apparent dip of bedding was derived from the base map listed below, at the closest measurement to the site.

BASE: Sheet 1: Los Gatos Quadrangle, Geologic Maps and Structure Section of the Southwestern Santa Clara Valley and Southern Santa Cruz Mountains, Santa Clara and Santa Cruz Counties, California; MCLAUGHLIN, ET AL.; 2001

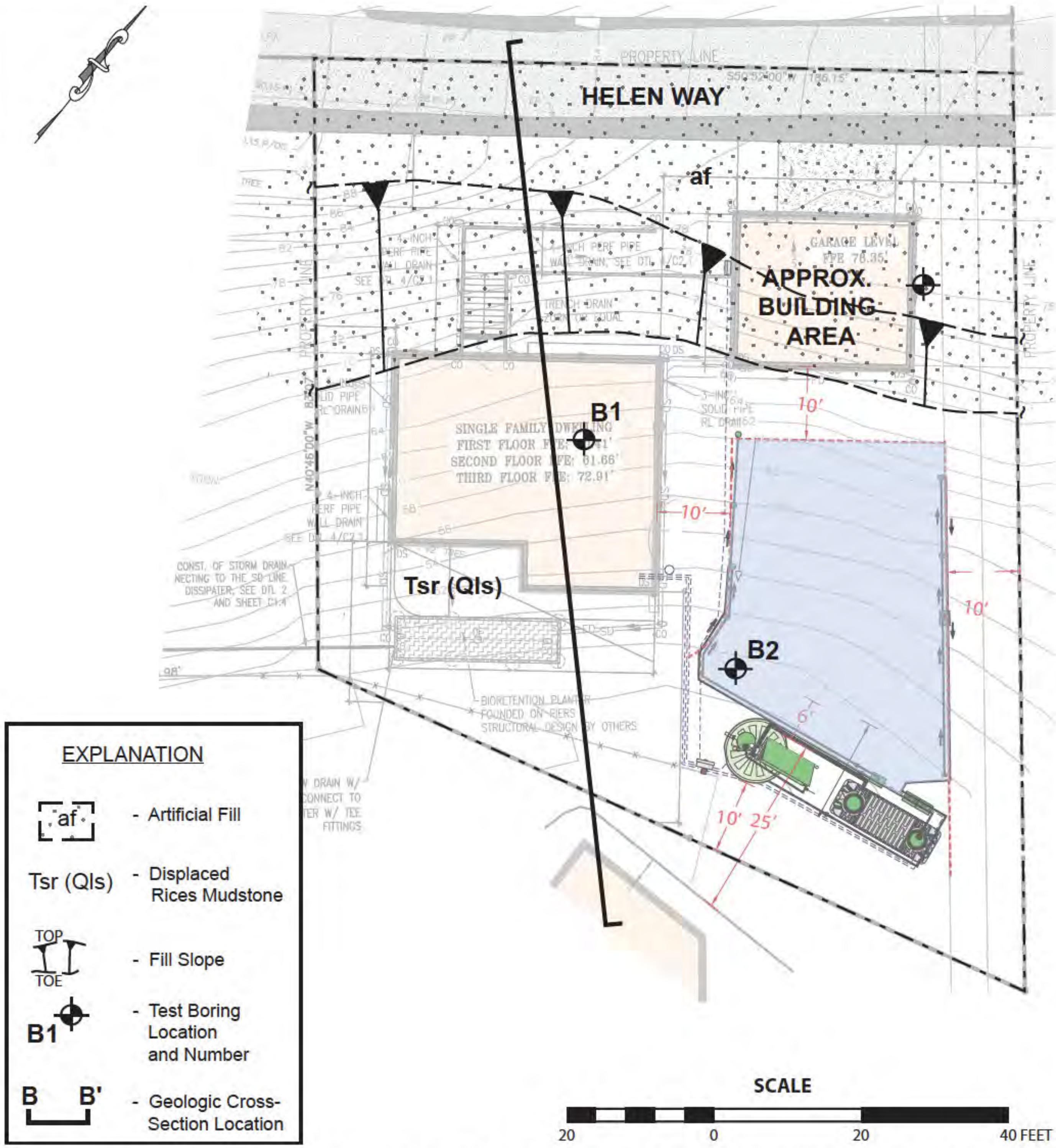
BASE: Topographic Map; COUNTY OF SANTA CLARA SCCMAP; 2023

GEOLOGIC CROSS-SECTION A-A'

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
JS/CR	1" = 500'	23103C-01R1	October 2023	Figure 6



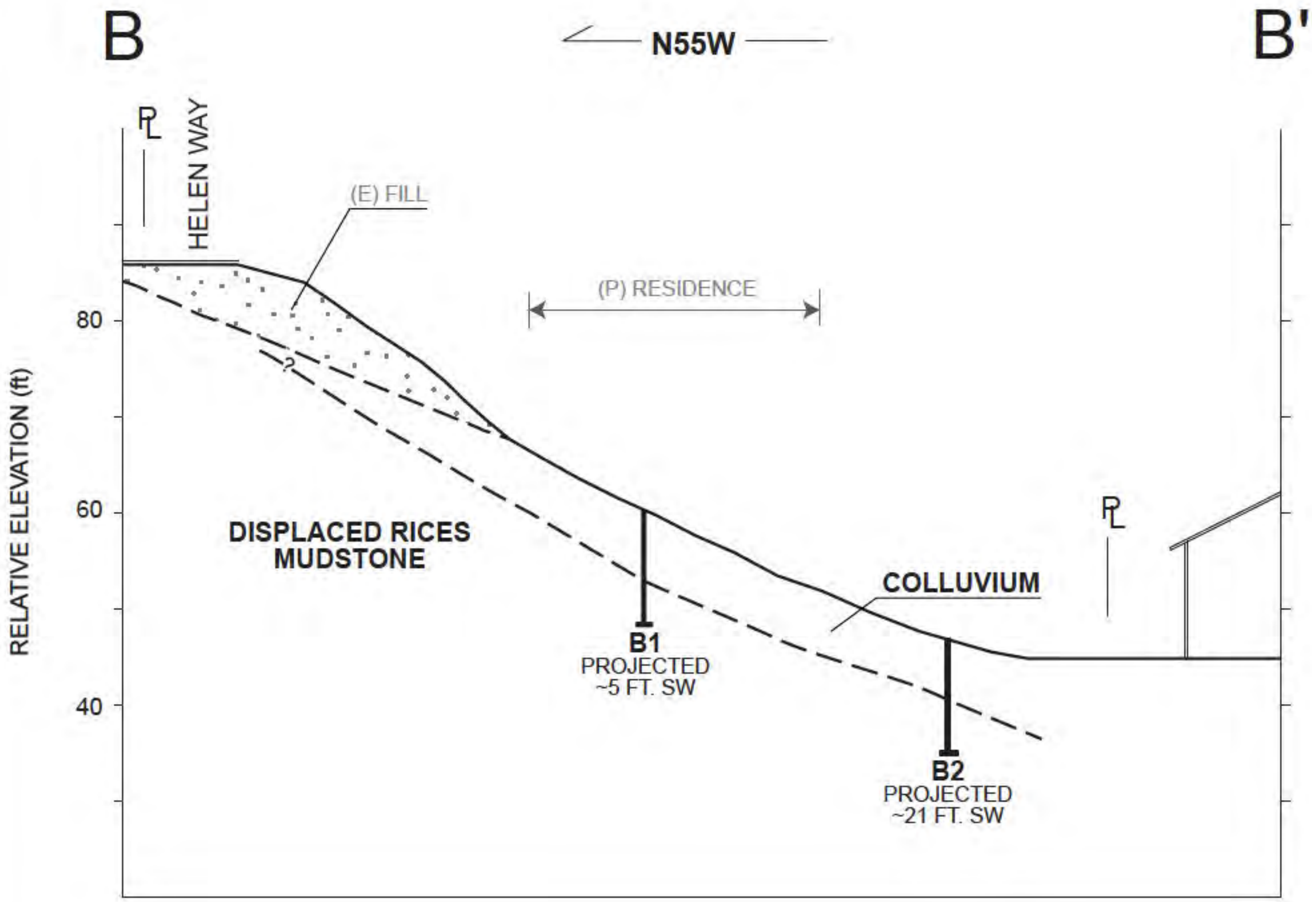
BASE: Onsite Wastewater Treatment System Design Plan, Sheet 1; BIOSPHERE CONSULTING; 19 March 2021

SITE PLAN AND ENGINEERING GEOLOGIC MAP

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	AS SHOWN	23103C-01R1	October 2023	Figure 7



NOTE: This cross-section is a conceptual illustration of general geologic relationships and should not be used for any other purpose.

BASE: Onsite Wastewater Treatment System Design Plan, Sheet 1; BIOSPHERE CONSULTING; 19 March 2021

GEOLOGIC CROSS-SECTION B-B'				
C2EARTH		[redacted] APN 544-39-035, Helen Way Santa Clara County, California		
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
JS/CR	1" = 20'	23103C-01R1	October 2023	Figure 8

UNIFIED SOIL CLASSIFICATION SYSTEM

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels; gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE	200	40	10	4	¾"	3"	12"	SIEVE OPENINGS

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

CONSISTENCY AND RELATIVE DENSITY

CONSISTENCY

SILTS AND CLAYS	STRENGTH[2]	BLOWS/FOOT[1]
VERY SOFT	0 - ¼	0 - 2
SOFT	¼ - ½	2 - 4
MEDIUM STIFF	½ - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT[1]
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

[1] Number of blows of 140-pound hammer falling 30 inches to drive a 2-inch O.D (1 3/8-inch I.D) split spoon

[2] Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated in general conformance with the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation

KEY TO LOGS

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DOCUMENT ID.	DATE	
23103C-01R1	October 2023	Figure 9

FRACTURING

INTENSITY	SIZE OF PIECES (FEET)
VERY LITTLE FRACTURED	Greater than 4.0
OCCASIONALLY FRACTURED	1 - 4
MODERATELY FRACTURED	0.5 - 1
CLOSELY FRACTURED	0.1 - 0.5
INTENSELY FRACTURED	0.05 - 0.1
CRUSHED	Less than 0.05

HARDNESS

SOFT	Reserved for plastic material alone
LOW	Can be gouged deeply or carved easily with a knife blade
MODERATELY	Can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
HARD	Can be scratched with difficulty; scratch produced a little powder and is often faintly visible.
VERY HARD	Cannot be scratched with knife blade; leaves a metallic streak.

STRENGTH

LOW	Plastic or very low strength.
FRIABLE	Crumbles easily by rubbing with fingers.
WEAK	An unfractured specimen of such material will crumble under light hammer blows.
MODERATELY	Specimen will withstand a few heavy hammer blows before breaking.
STRONG	Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
VERY STRONG	Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

WEATHERING[1]

DEEP	Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
MODERATE	Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
SLIGHT	No megascopic decomposition of minerals; little of no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture Surface.
FRESH	Unaffected by weathering agents. No disintegration of discoloration. Fractures usually less numerous than joints.

[1] The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

BEDDING OF SEDIMENTARY ROCKS

SPLITTING PROPERTY	THICKNESS (FEET)
MASSIVE	Greater than 4.0
BLOCKY	2.0 - 4.0
SLABBY	0.2 - 2.0
FLAGGY	0.05 - 0.2
SHALY OR PLATY	0.01 - 0.05
PAPERY	Less than 0.01

STRATIFICATION	THICKNESS (FEET)
VERY THICK-BEDDED	Greater than 4.0
THICK-BEDDED	2.0 - 4.0
THIN-BEDDED	0.2 - 2.0
VERY THIN-BEDDED	0.05 - 0.2
LAMINATED	0.01 - 0.05
THINLY LAMINATED	Less than 0.01

ROCK CLASSIFICATION SYSTEM

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DOCUMENT ID.	DATE	
23103C-01R1	October 2023	Figure 10

The standard penetration resistance (SPT) blow counts are obtained in general accordance with ASTM Test Designation D1586. The drive weight assembly consists of a 70 or 140-pound hammer dropped through a 30-inch free fall. A blow count is defined as the number of hammer blows per six inches of penetration, or 50 blows for 6 inches or less of penetration. The driving of samplers was discontinued if the observed blow count was 50 for 6 inches or less of penetration.

SPT samples are collected in a standard, 2-inch outer diameter, split-barrel sampler without liners (see Figure A below). Samplers holding 2-inch diameter liners (see Figure B below) and 2½-inch diameter liners (see Figure C below) are used to obtain “undisturbed” samples. Occasionally a portable power driven sampler holding 1-inch diameter liners is used for field sampling (see Figure D below). Resistance is measured in seconds per foot and does not correlate with the ASTM SPT. Undisturbed samples may also be collected using a Pitcher Barrel sampler (see Figure E below). Material recovered over the length of the sampler is shaded. A measure of resistance is not collected with this technique.

Blow counts are converted to SPT counts which are shown on the boring logs by the following relation:

$$B = \frac{N\,W\,H}{(140)(30)} \left(\frac{D_{o\,SPT}^2 - D_{i\,SPT}^2}{D_o^2 - D_i^2} \right)$$

B = Equivalent number of blows per foot with a SPT
N = Number of blows per foot actually recorded
W = Weight of hammer (lb)
H = Height of hammer drop (in)
D_o = Outside Diameter (in)
D_i = Inside Diameter (in)

The blow counts used for these conversions were taken over the last two sample intervals if the sampler was driven 12 inches or more. If the sampler is driven less than 12 inches, the blow counts of the last sample were converted to SPT counts of 50 blows over an equivalent SPT run length.

SPT Figure A | 2” Liner Figure B | 2.5” Liner Figure C | 1” Liner Figure D | Pitcher Barrel Figure E

= Undisturbed Sample = Disturbed Sample NR = Not Reported

Where obtained, the shear strength of the soil samples is shown on the boring logs in the far right-hand column.

SUMMARY OF FIELD SAMPLING PROCEDURES

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DOCUMENT ID.	DATE	
23103C-01R1	October 2023	Figure 11

EQUIPMENT	Tripod, Cathead, 140lb Hammer	RELATIVE ELEVATION	1605 feet	LOGGED BY	S. Blair
DEPTH TO GROUNDWATER	Not Encountered	DEPTH TO BEDROCK	7½ feet	DATE DRILLED	28 Aug 2023

DESCRIPTION AND CLASSIFICATION								
DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
SANDY CLAYEY SILT; light brown (7.5YR, 6/3); homogeneous; scattered fine-grained sand; trace siltstone gravel; low plasticity; scattered rootlets; scattered root holes; dry (Colluvium)	Medium Stiff to Hard	ML	2		8	7	81	
			4		14	9	92	
			6		39			
SILTSTONE; light yellowish brown (10 YR 6/4); low strength; low hardness; highly weathered; trace fine-grained sand; poorly indurated; trace rootlets; dry (Displaced Rices Mudstone)		(Rock)	8		64	10		
			10		44			
			12		63	8		
Bottom of Boring = 12 feet								

LOG OF BORING 1

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DOCUMENT ID.	DATE	
23103C-01R1	October 2023	Figure 12

EQUIPMENT	Tripod, Cathead, 140lb Hammer	RELATIVE ELEVATION	1589 feet	LOGGED BY	S. Blair
DEPTH TO GROUNDWATER	Not Encountered	DEPTH TO BEDROCK	6½ feet	DATE DRILLED	28 Aug 2023

DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
SANDY CLAYEY SILT; brown (10 YR, 5/3); homogeneous; scattered fine-grained sand; trace siltstone and sandstone gravel; low plasticity; trace rootlets; trace root holes; dry (Colluvium)	Medium Stiff to Very Stiff	ML	1					
			2		6	8	87	
			3					
			4		9	8	80	
			5					
			6		20	7		
SILTSTONE; brown (10 YR, 5/3) to light yellowish brown (10 YR, 6/4); low strength; low hardness; highly weathered; fine sandstone gravel; fine to coarse gravel; poorly indurated; dry (Displaced Rices Mudstone)		(Rock)	7					C = 0.20 Phi = 30
			8		76			
			9					
			10		86	9		
			11					
			12		103			
Bottom of Boring =12 feet			13 – 20					

LOG OF BORING 2

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DOCUMENT ID.	DATE	
23103C-01R1	October 2023	Figure 13

EQUIPMENT	Tripod, Cathead, 140lb Hammer	RELATIVE ELEVATION	1614 feet	LOGGED BY	S. Blair
DEPTH TO GROUNDWATER	Not Encountered	DEPTH TO BEDROCK	9¼ feet	DATE DRILLED	28 Aug 2023

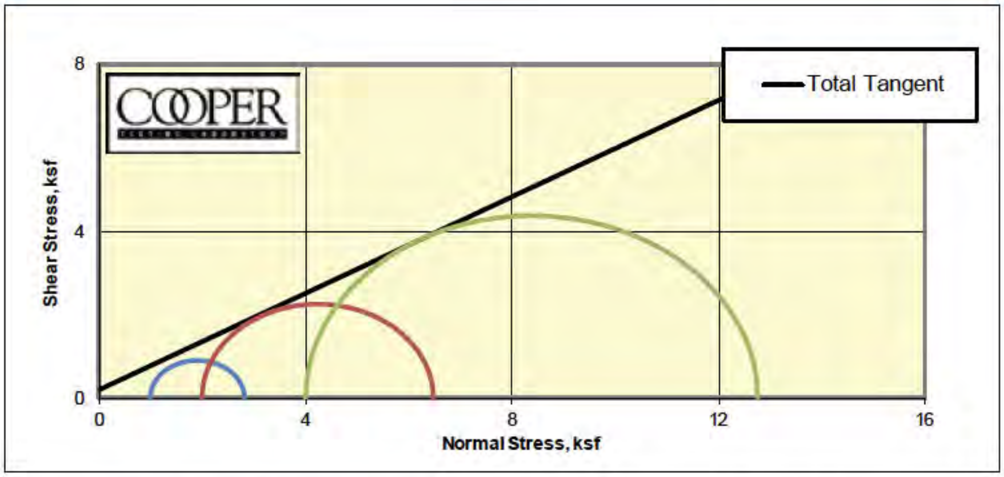
DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS	CONSIST.	SOIL TYPE	DEPTH (FEET)	SAMPLE	PENETRATION RESISTANCE (BLOWS / FT.)	WATER CONTENT (%)	DRY DENSITY (PCF)	SHEAR STRENGTH (KSF)
CLAYEY SILT; brown (10 YR, 5/3); heterogeneous; scattered fine-grained sand; abundant fine to coarse gravel; abundant rootlets; trace root holes; dry (Fill)	Very Stiff to Hard	ML	1					
			2		22	7	100	
			3		50			
no retrieval; encountered large rock			4–5		50			
			6–7		20	6		
SILTSTONE; brown (10 YR, 5/3) to light yellowish brown (10 YR, 6/4); low strength; low hardness; highly weathered; scattered fine to coarse siltstone and sandstone gravel; poorly indurated; dry (Displaced Rices Mudstone)		(Rock)	8–9		40			
			10–11		83			
Bottom of Boring =11 feet			12–20					

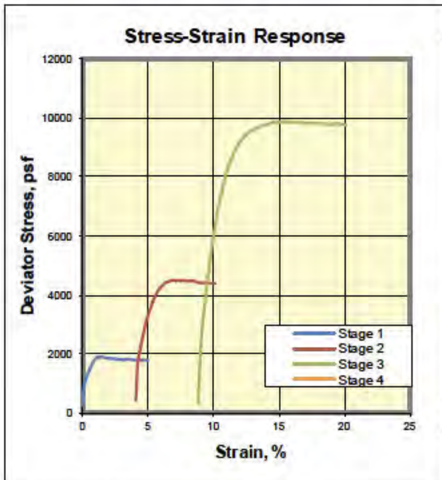
LOG OF BORING 3

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DOCUMENT ID.	DATE	
23103C-01R1	October 2023	Figure 14

Staged Consolidated Undrained Triaxial Compression
ASTM D4767m





CTL Number:	128-425	
Client Name:	C2 Earth	
Project Name:	Piglitsin	
Project Number:	23103C-01	
Date:	9/22/2023	By: MD/DC
Total C	0.200	ksf
Total phi	30.0	degrees
Eff. C	N/A	ksf
Eff. Phi	N/A	degrees

©

Remarks: Staged tests may show an abnormal strength envelope with the sample displaying a higher friction angle as the confining pressure increases which can show negative cohesion. This is more pronounced on granular materials. This is caused by the decreased initial void ratio of the second and/ or third stages created by the staged procedure. The sample failed during the first stage. This should be taken into account when evaluating all subsequent data.

Stage	1	2	3	4
Boring	B-2			
Sample	4			
Depth	8			
Visual Description	Brown Sandy CLAY near Clayey SAND			
MC (%)	9.0			
DryDensity(pcf)	107.4			
Saturation (%)	45.9			
Void Ratio	0.511			
Diameter (in)	1.92			
Height (in)	4.00			
	Final			
MC (%)	18.7	17.6	16.4	
DryDensity(pcf)	109.2	111.3	113.8	
Saturation (%)	100.0	100.0	100.0	
Void Ratio	0.487	0.458	0.426	
Diameter (in)	1.91	1.93	1.96	
Height (in)	3.97	3.81	3.62	
Cell Pressure (psi)	65.9	72.9	86.8	
Back Pressure (psi)	59.0	59.0	59.0	
	Total Stresses At:			
Strain (%)	3.0	3.0	3.0	
Deviator (ksf)	1.820	4.481	8.754	
Excess PP (psi)				
Sigma 1 (ksf)	2.814	6.482	12.758	
Sigma 3 (ksf)	0.994	2.002	4.003	
P (ksf)	1.904	4.242	8.380	
Q (ksf)	0.910	2.240	4.377	
Stress Ratio	2.832	3.239	3.187	
Rate (in/min)	0.0194	0.0192	0.0196	

SHEAR STRENGTH TEST

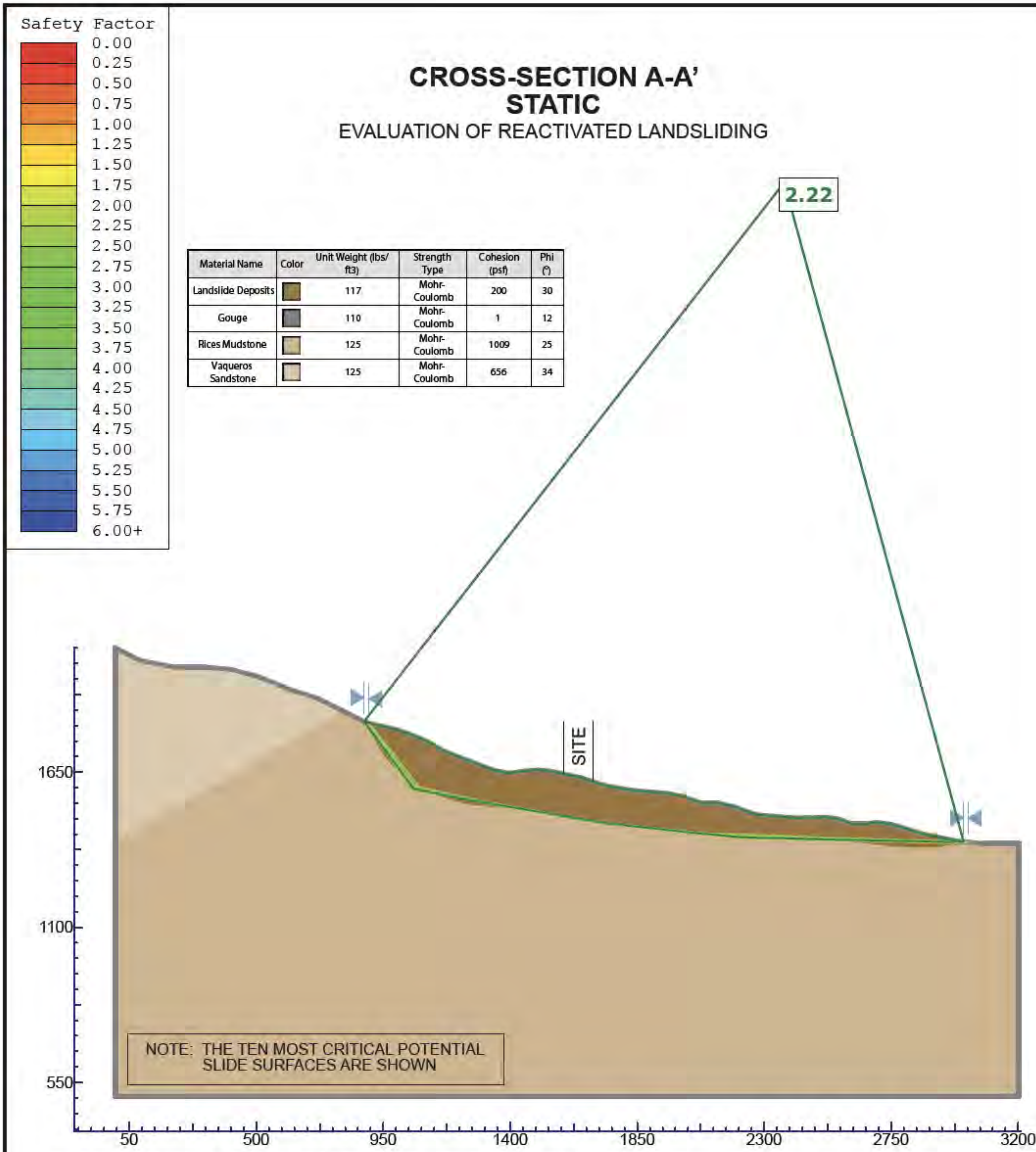
C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	Not Applicable	23103C-01R1	October 2023	Figure 15

CROSS-SECTION A-A'
STATIC
EVALUATION OF REACTIVATED LANDSLIDING

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Landslide Deposits		117	Mohr-Coulomb	200	30
Gouge		110	Mohr-Coulomb	1	12
Rices Mudstone		125	Mohr-Coulomb	1009	25
Vaqueros Sandstone		125	Mohr-Coulomb	656	34



BASE: Slide2 Modeler; ROCSCIENCE, INC.; Version 9.028

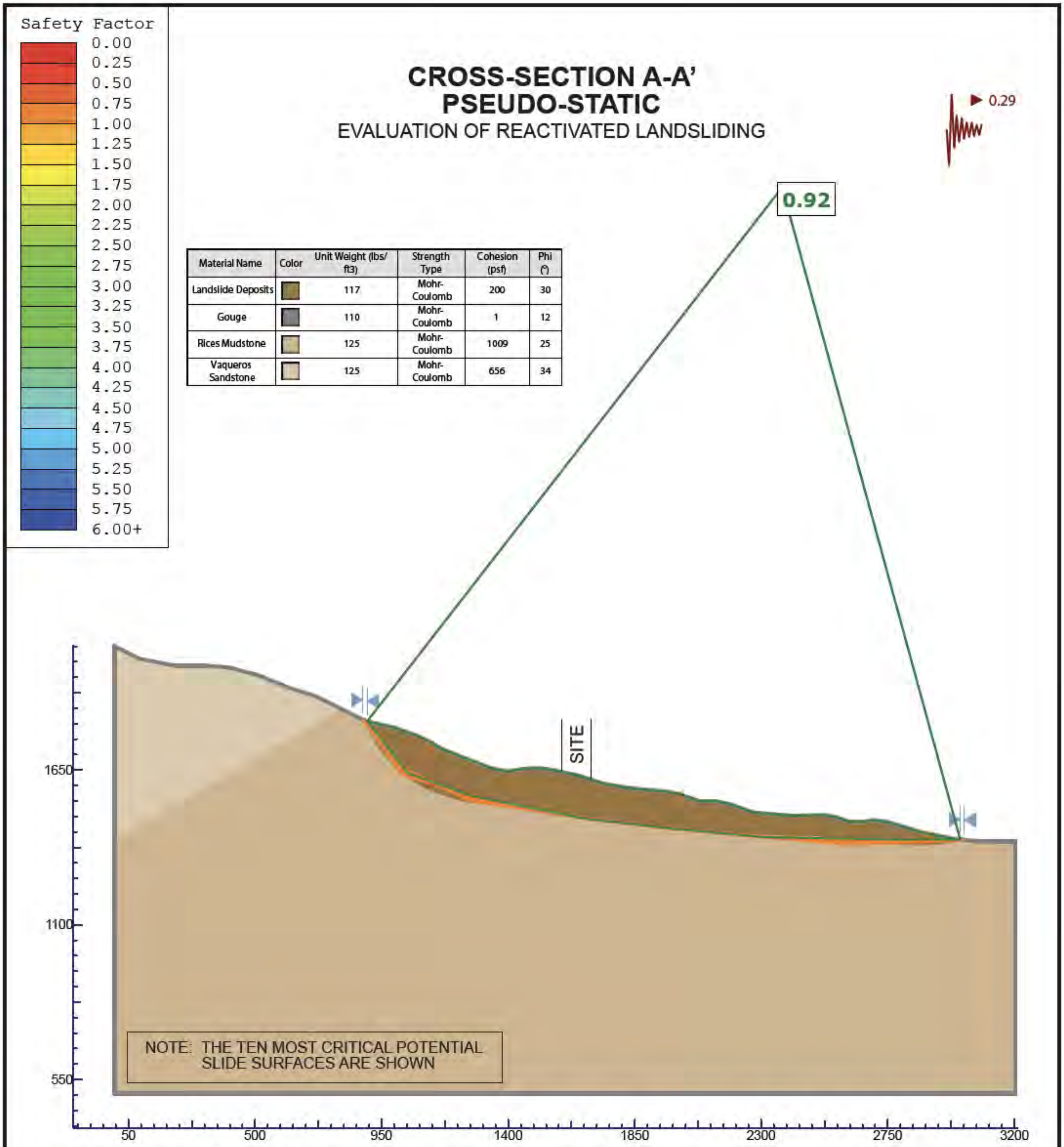
SLOPE STABILITY ANALYSIS NO. 1

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
KK/CR	AS SHOWN	23103C-01R1	October 2023	Figure 16

CROSS-SECTION A-A'
PSEUDO-STATIC
EVALUATION OF REACTIVATED LANDSLIDING



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Landslide Deposits		117	Mohr-Coulomb	200	30
Gouge		110	Mohr-Coulomb	1	12
Rices Mudstone		125	Mohr-Coulomb	1009	25
Vaqueros Sandstone		125	Mohr-Coulomb	656	34

BASE: Slide2 Modeler; ROCSCIENCE, INC.; Version 9.028

SLOPE STABILITY ANALYSIS NO. 2

C2EARTH

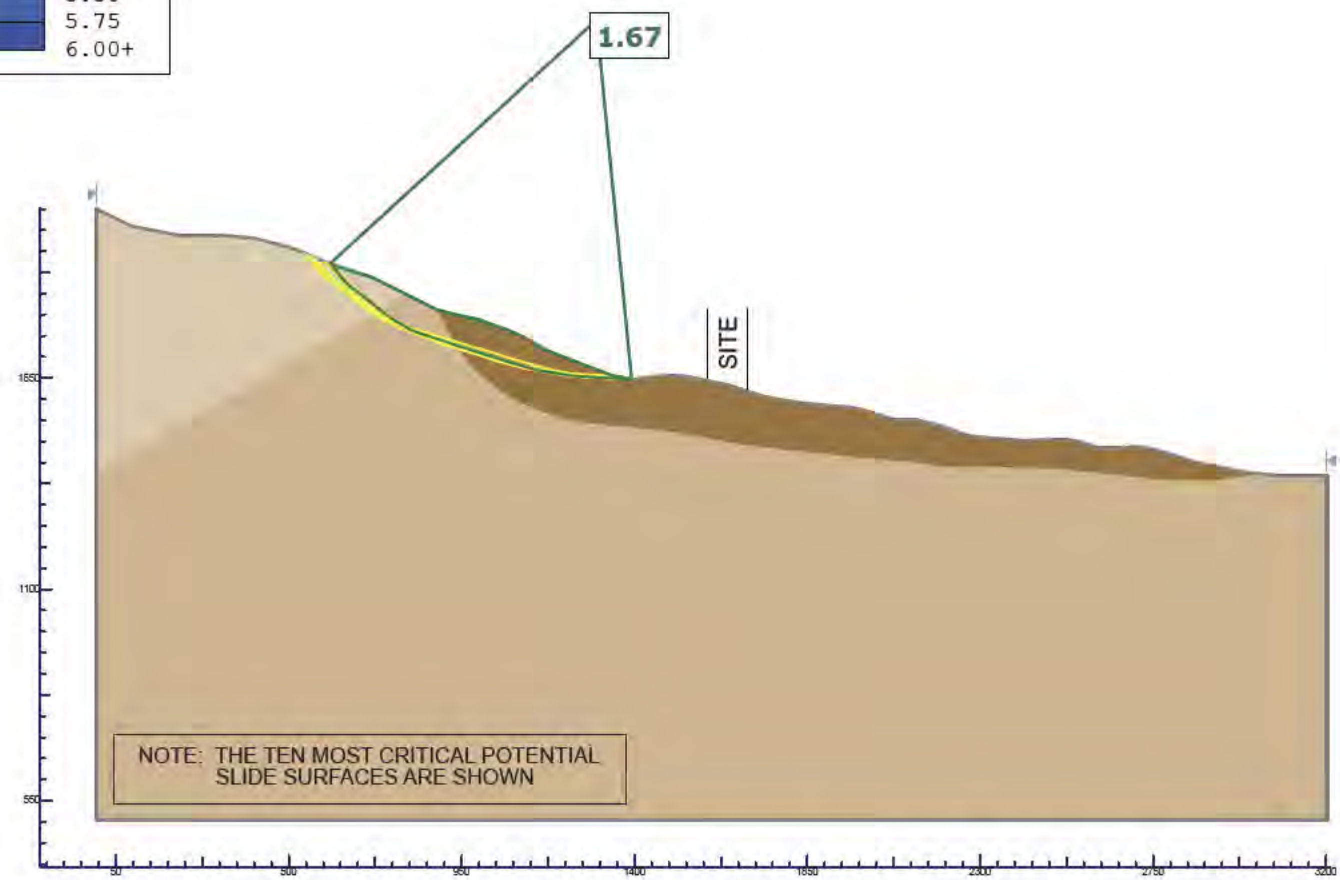
[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
KK/CR	AS SHOWN	23103C-01R1	October 2023	Figure 17

CROSS-SECTION A-A'
STATIC

EVALUATION OF LANDSLIDING INITIATING
ANYWHERE ON THE SUBJECT SLOPE

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Landslide Deposits		117	Mohr-Coulomb	200	30
Gouge		110	Mohr-Coulomb	1	12
Rices Mudstone		125	Mohr-Coulomb	1009	25
Vaqueros Sandstone		125	Mohr-Coulomb	656	34



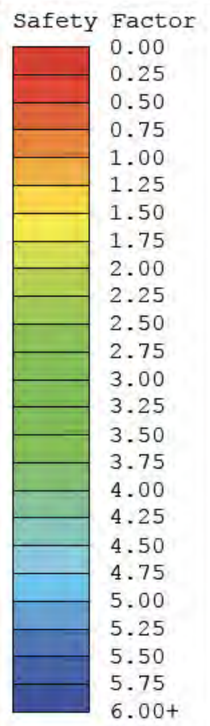
BASE: Slide2 Modeler; ROCSCIENCE, INC.; Version 9.028

SLOPE STABILITY ANALYSIS NO. 3

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
KK/CR	AS SHOWN	23103C-01R1	October 2023	Figure 18

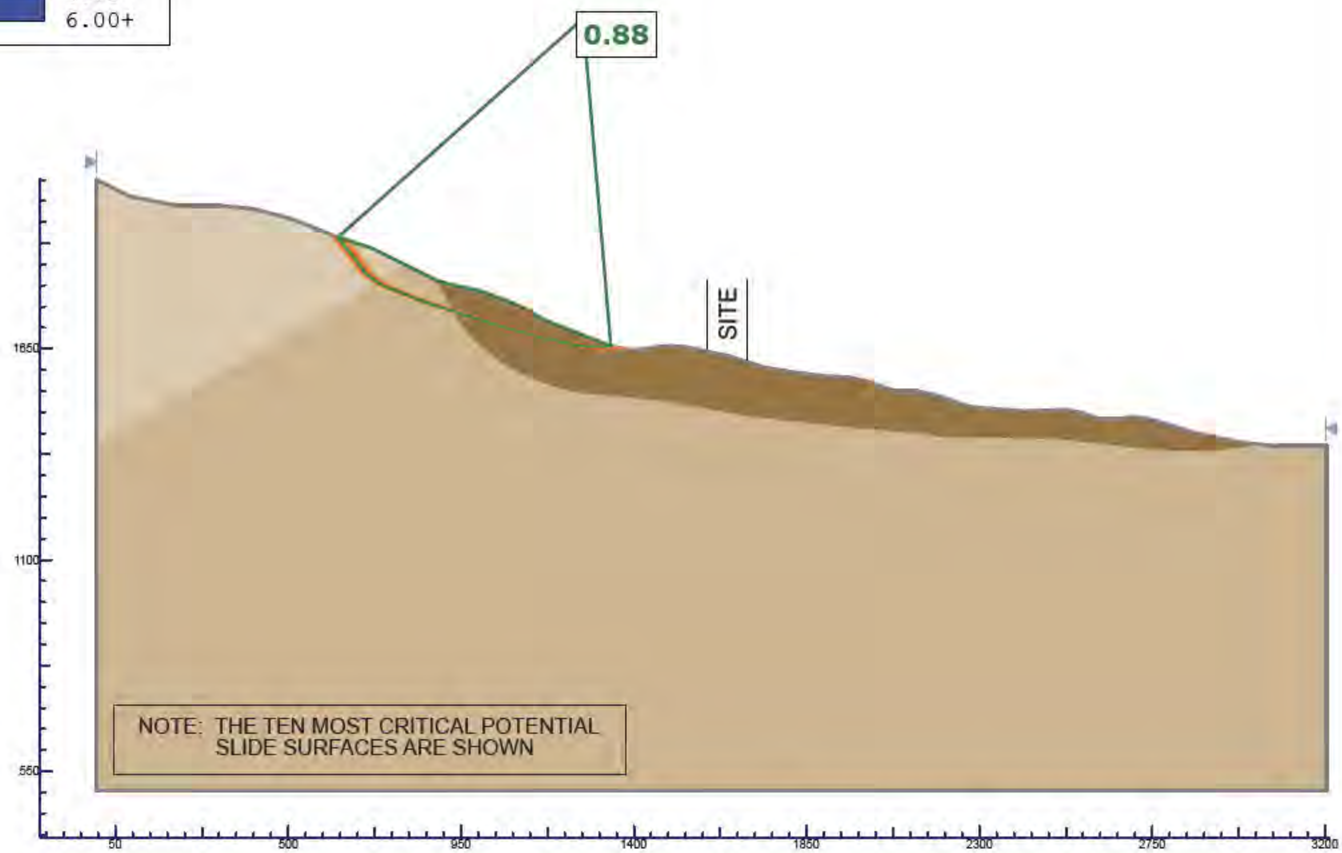


CROSS-SECTION A-A'
PSEUDO-STATIC

EVALUATION OF LANDSLIDING INITIATING
ANYWHERE ON THE SUBJECT SLOPE

▶ 0.29

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Landslide Deposits		117	Mohr-Coulomb	200	30
Gouge		110	Mohr-Coulomb	1	12
Rices Mudstone		125	Mohr-Coulomb	1009	25
Vaqueros Sandstone		125	Mohr-Coulomb	656	34



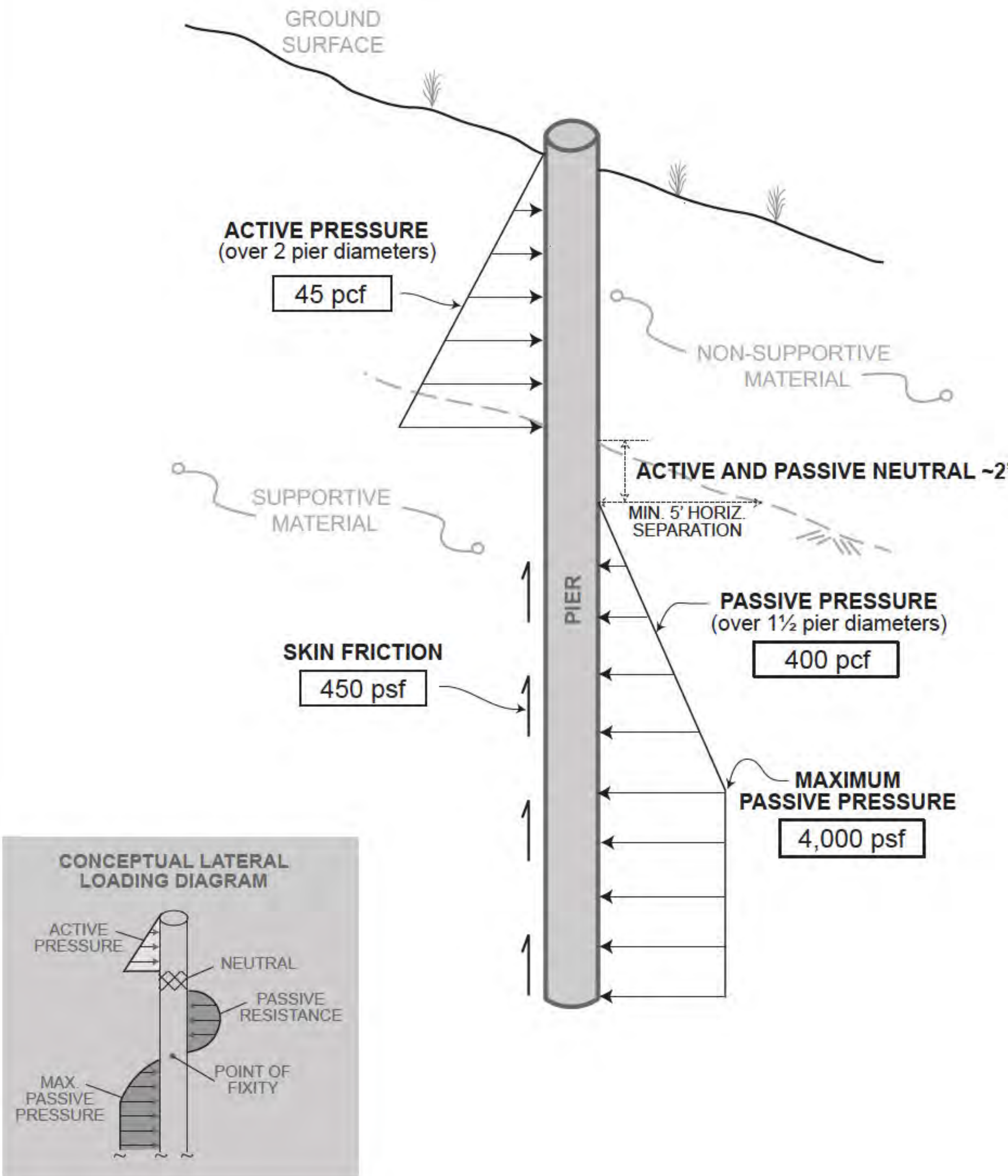
BASE: Slide2 Modeler; ROCSCIENCE, INC.; Version 9.028

SLOPE STABILITY ANALYSIS NO. 4

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
KK/CR	AS SHOWN	23103C-01R1	October 2023	Figure 19

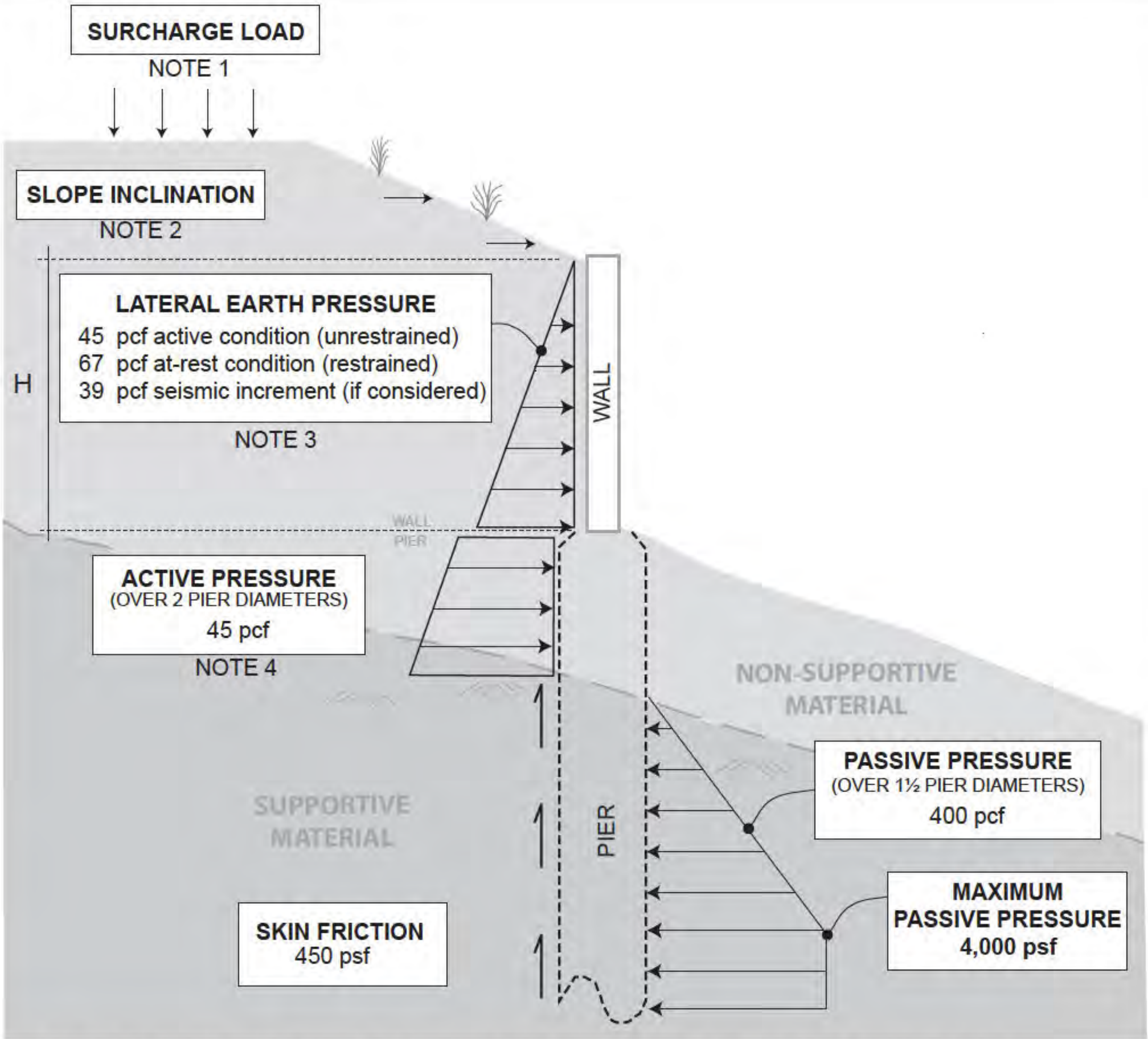


CONCEPTUAL PIER PRESSURE DIAGRAM

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	Not Applicable	23103C-01R1	October 2023	Figure 20



Note 1: Additional lateral load equal to 1/3 (unrestrained) or 1/2 (restrained) the anticipated surcharge load.

Note 2: Add an additional equivalent fluid pressure increment to the active and at-rest condition for sloping backfill above the wall:

+5 pcf for slope inclinations up to 4:1 (horizontal to vertical)
+8 pcf for slope inclinations between 3:1 and 4:1
+12 pcf for slope inclinations between 2:1 and 3:1

Note 3: Lateral earth pressures are shown for drained retaining walls. Contact us to provide additional recommendations if undrained walls are planned.

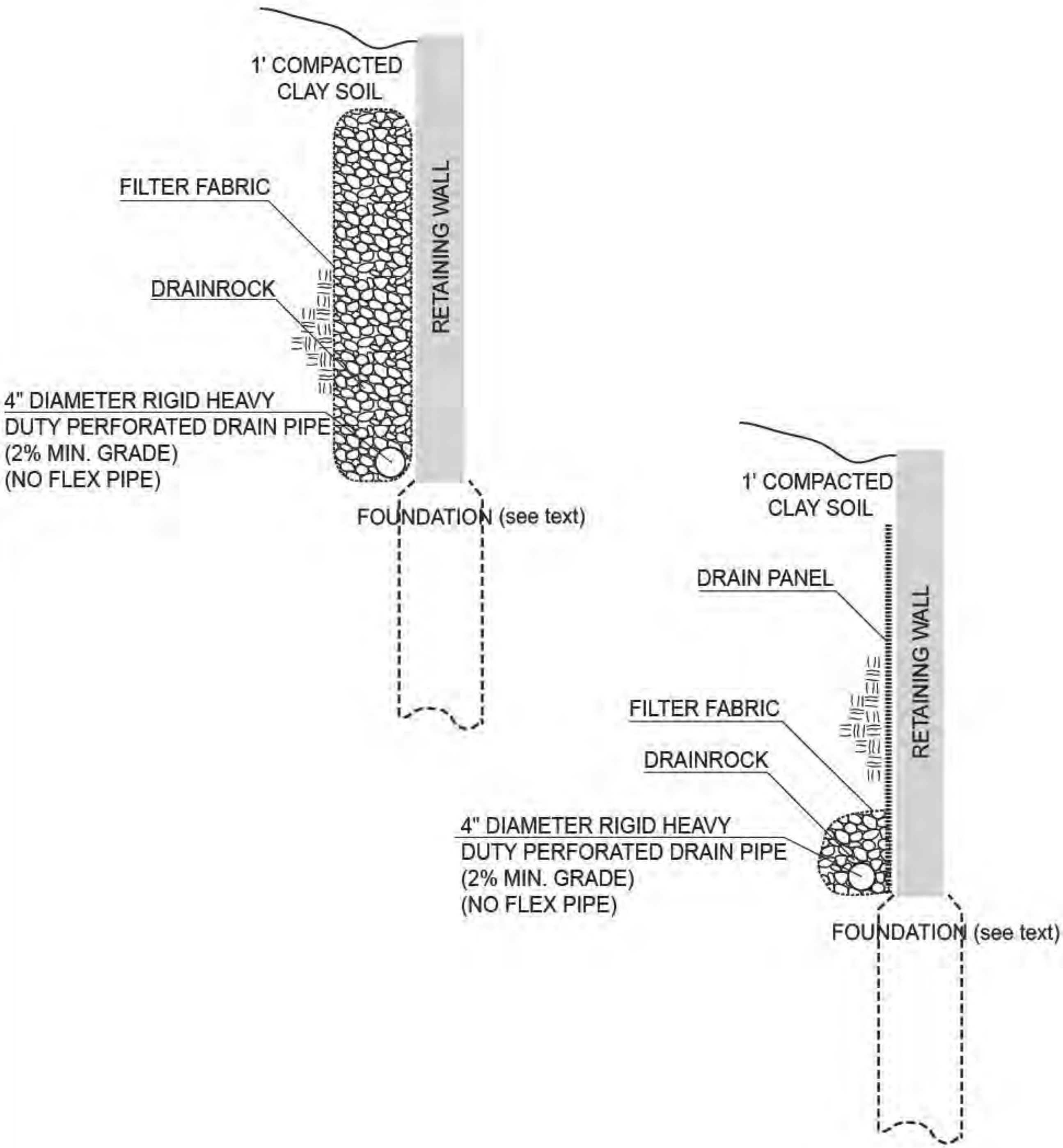
Note 4: Active pressure on piers continues from loading at the base of wall. Active pressure at the top of the pier is NOT equal to zero.

CONCEPTUAL PIER AND RETAINING WALL PRESSURE DIAGRAM

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	Not Applicable	23103C-01R1	October 2023	Figure 21

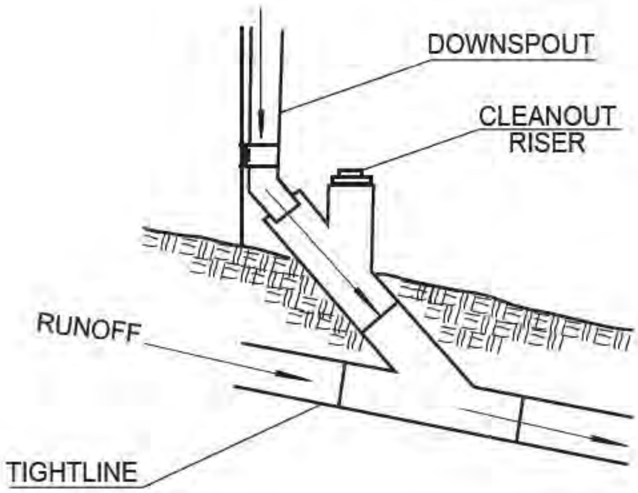


CONCEPTUAL RETAINING WALL BACKDRAIN DIAGRAM

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	Not Applicable	23103C-01R1	October 2023	Figure 22

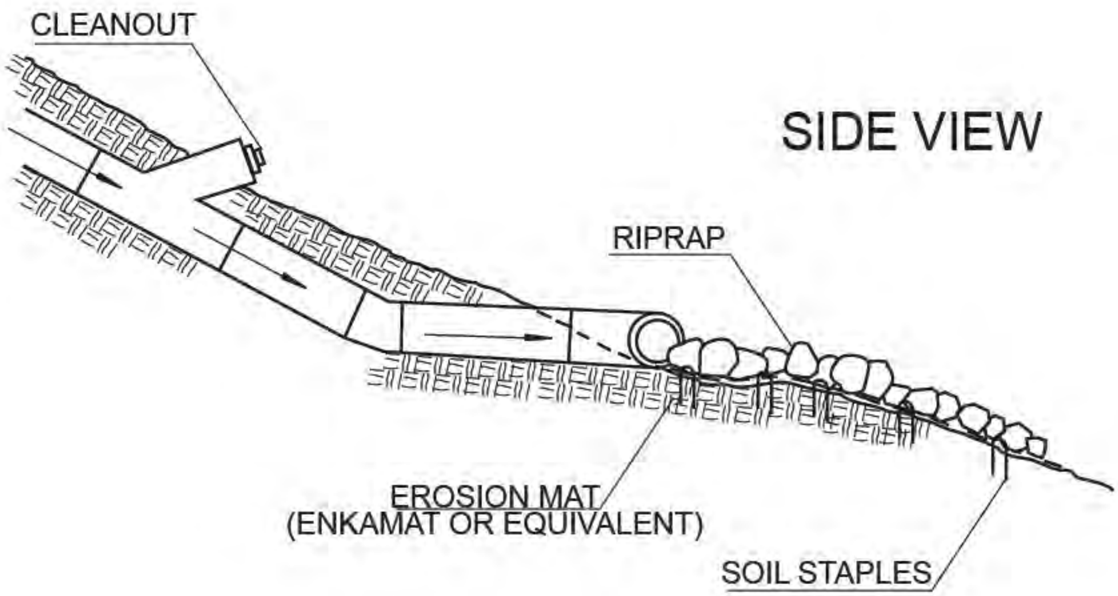


CONCEPTUAL DOWNSPOUT CLEANOUT DIAGRAM

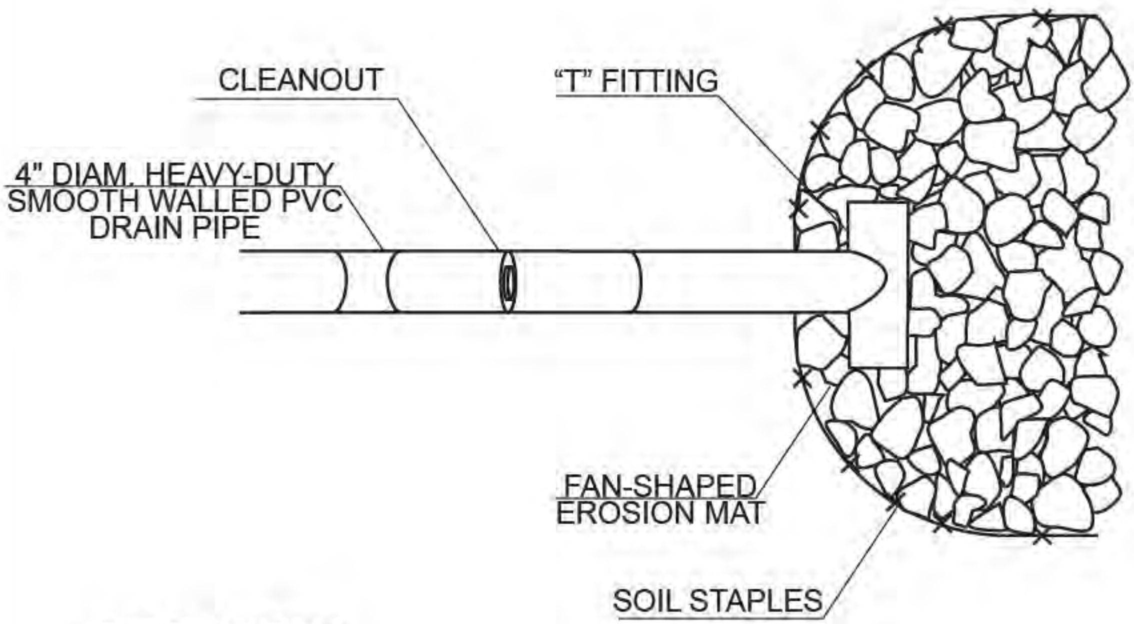
C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	Not Applicable	23103C-01R1	October 2023	Figure 23



SIDE VIEW



TOP VIEW

CONCEPTUAL ENERGY DISSIPATER DIAGRAM

C2EARTH

[redacted]
APN 544-39-035, Helen Way
Santa Clara County, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
SB/CR	Not Applicable	23103C-01R1	October 2023	Figure 24

TABLE I

MODIFIED MERCALLI SCALE OF EARTHQUAKE INTENSITIES

I. Not felt by people, except under especially favorable circumstances.

II. Felt only by persons at rest on the upper floors of buildings. Some suspended objects may swing.

III. Felt by some people who are indoors, but it may not be recognized as an earthquake. The vibration is similar to that caused by the passing of light trucks. Hanging objects swing.

IV. Felt by many people who are indoors, by a few outdoors. At night some people are awakenad. Dishes, windows and doors are disturbad: walls make creaking sounds; stationary cars rock noticeably. The sensation is like a heavy object striking a building; the vibration is similar to that caused by the passing of heavy trucks.

V. Felt indoors by practically everyone, outdoors by most people. The direction and duration of the shock can be estimated by people outdoors. At night, sleepers are awakened and some run out of buildings. Liquids are disturbed and sometimes spilled. Small, unstable objects and some furnishings are shifted or upset. Doors close or open.

VI. Felt by everyone, and many people are frightened and run outdoors. Walking is difficult. Small church and school bells ring. Windows, dishes, and glassware are broken; liquids spill; books and other standing objects fall; pictures are knocked from walls; furniture is moved or overturned. Poorly built buildings may be damaged, and weak plaster will crack.

VII. Causes general alarm. Standing upright is very difficult. Persons driving cars also notice the shaking. Damage is negligible in buildings of very good design and construction, slight to moderate in well-built ordinary structures, considerable in poorly built or designed structures. Some chimneys are broken; interiors and furnishings experience considerable damage; architectural ornaments fall. Small slides occur along sand or gravel banks of water channels; concrete irrigation ditches are damaged. Waves form in the water and it becomes muddied.

VIII. General fright and near panic. The steering of cars is difficult. Damage is slight in specially designed earthquake-resistant structures, considerable in well-built ordinary buildings. Poorly built or designed buildings experience partial collapses. Numerous chimneys fall; the walls of frame buildings are damaged; interiors experience heavy damage. Frame houses that are not properly bolted down may move on their foundations. Decayed pilings are broken off. Tress are damaged. Cracks appear in wet ground and on steep slopes. Changes in the flow or temperature of springs and wells are noted.

IX. Panic is general. Interior damage is considerable in specially designed earthquake-resistant structures. Well-built ordinary buildings suffer severe damage, with partial collapses; frame structures thrown out of plumb or shifted off of their foundations. Unreinforced masonry buildings collapse. The ground cracks conspicuously and some underground pipes are broken. Reservoirs are damaged seriously.

X. Most masonry and many frame structures are destroyed. Specially designed earthquake-resistant structures may suffer serious damage. Some well-built bridges are destroyed, and dams, dikes and embankments are seriously damaged. Large landslides are triggered by the shock. Water is thrown onto the banks of canals, rivers and lakes. Sand and mud are shifted horizontally on beaches and flat land. Rails are bent slightly. Many buried pipes and conduits are broken.

XI. Few, if any, masonry structures remain standing. Other structures are severely damaged. Broad fissures, slumps and slides develop in soft or wet soils. Underground pipe lines and conduits are put completely out of service. Rails are severely bent.

XII. Damage is total, with practically all works of construction severely damaged or destroyed. Waves are observed on ground surfaces, and all soft or wet soils are greatly disturbed. Heavy objects are thrown into the air, and large rock masses are displaced.

YANEV,P.,1974, Peace of Mind in Earthquake Country, Chronicle Books, San Francisco, California.

APPENDIX: ASCE 7 HAZARD REPORTS

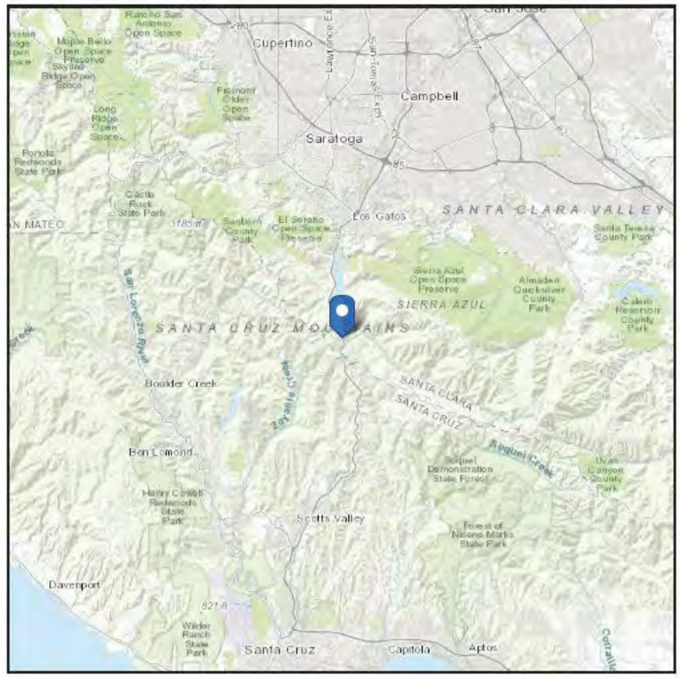
AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE 7 Hazards Report

Address:
No Address at This Location

Standard: ASCE/SEI 7-22
Risk Category: II
Soil Class: BC

Latitude: 37.155107
Longitude: -121.986846
Elevation: 1620.3151430410278 ft (NAVD 88)



Seismic

BC

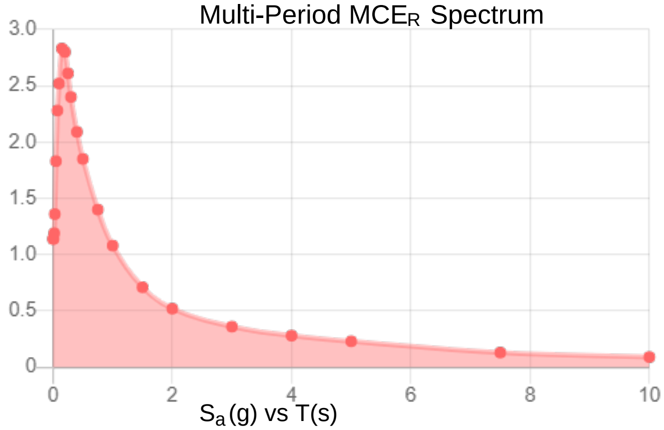
Site Soil Class:

Results:

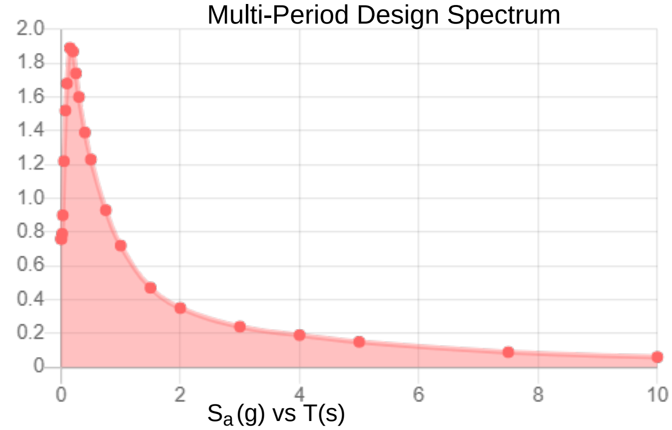
PGA $_M$:	0.95	T_L :	12
S_{MS} :	2.52	S_S :	2.8
S_{M1} :	1.08	S_1 :	1.08
S_{DS} :	1.68	V_{S30} :	760
S_{D1} :	0.72		

Seismic Design Category: E

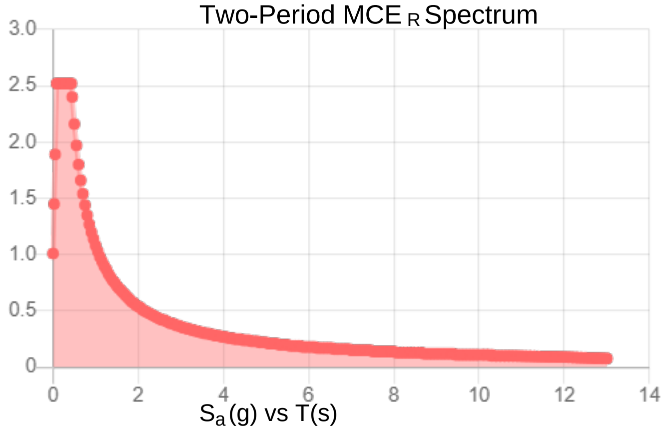
Multi-Period MCE$_R$ Spectrum



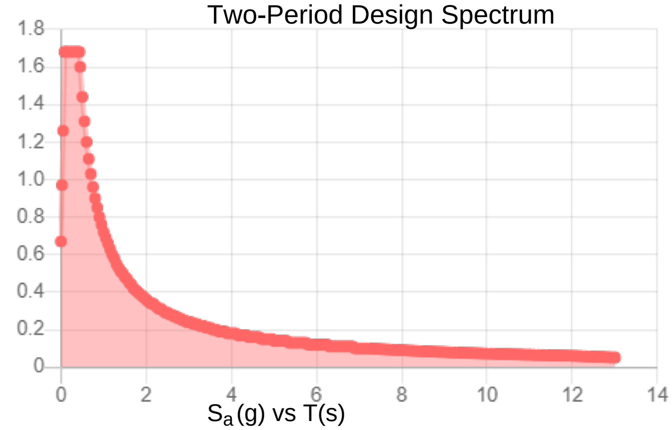
Multi-Period Design Spectrum



Two-Period MCE $_R$ Spectrum



Two-Period Design Spectrum



MCE$_R$ Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Design Vertical Response Spectrum

Vertical ground motion data has not yet been made available by USGS.

Data Accessed: **Wed Oct 04 2023**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-22 and ASCE/SEI 7-22 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-22 Ch. 21 are available from USGS.

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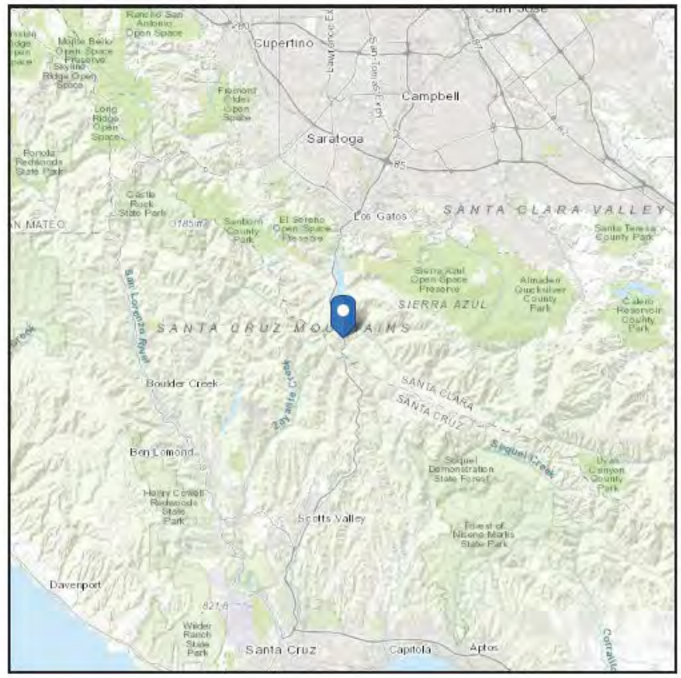
ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

ASCE 7 Hazards Report

Address:
No Address at This Location

Standard:	ASCE/SEI 7-16	**Latitude:**	37.155107
Risk Category:	II	**Longitude:**	-121.986846
Soil Class:	C - Very Dense Soil and Soft Rock	**Elevation:**	1620.3151430410278 ft (NAVD 88)



ASCE
AMERICAN SOCIETY OF CIVIL ENGINEERS

Seismic

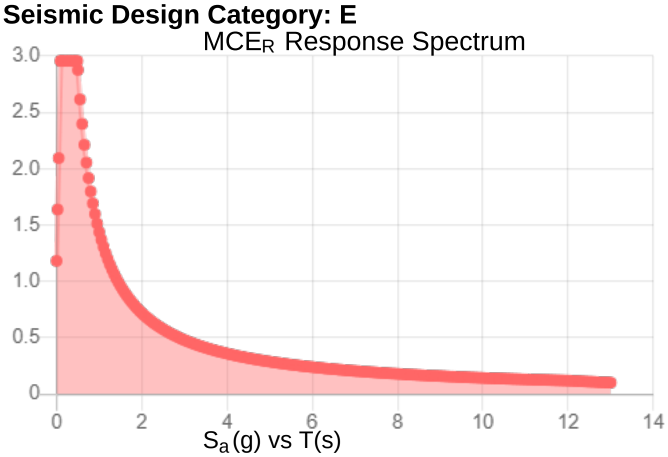
C - Very Dense Soil and Soft Rock

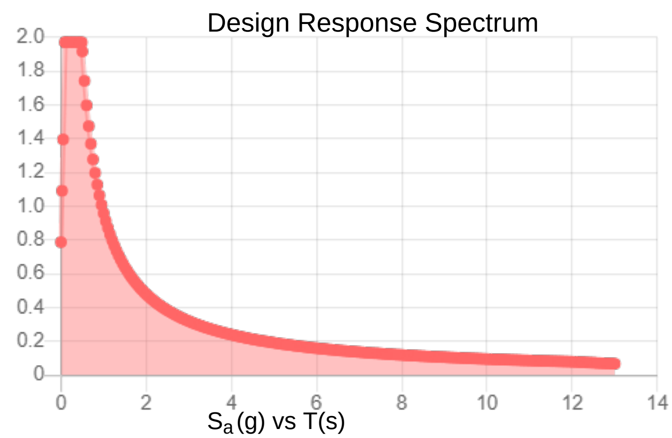
Site Soil Class:

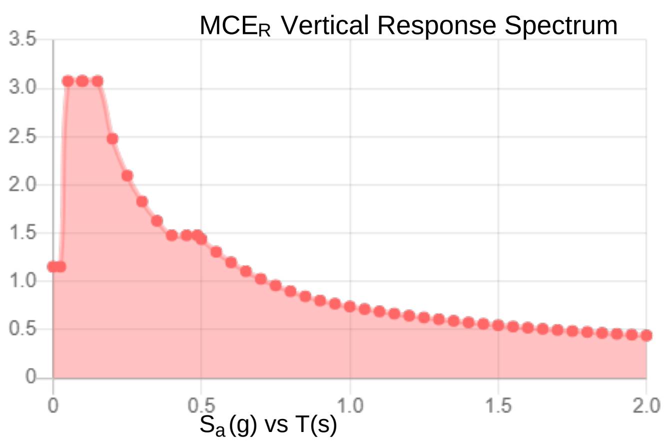
Results:

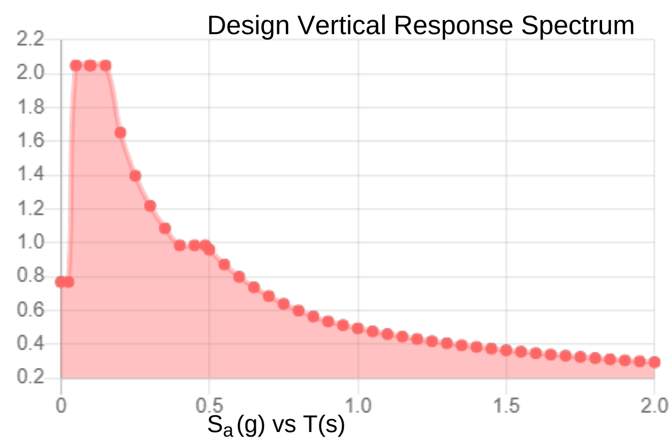
S_S :	2.463	S_{D1} :	0.959
S_1 :	1.027	T_L :	12
F_a :	1.2	PGA :	1.047
F_v :	1.4	PGA_M :	1.256
S_{MS} :	2.956	F_{PGA} :	1.2
S_{M1} :	1.438	I_e :	1
S_{DS} :	1.971	C_v :	1.3

Seismic Design Category: E









Data Accessed: **Wed Oct 04 2023**

Date Source:

USGS Seismic Design Maps based on ASCE/SEI 7-16 and ASCE/SEI 7-16 Table 1.5-2. Additional data for site-specific ground motion procedures in accordance with ASCE/SEI 7-16 Ch. 21 are available from USGS.

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GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED SITE DEVELOPMENT
[redacted]
APN 544-39-035
HELEN WAY
SANTA CLARA COUNTY, CALIFORNIA

DOCUMENT ID. 23103C-01R1
DATED 31 October 2023

TO: C2Earth, Inc.
750 Camden Avenue, Suite A
Campbell, CA 95008

FROM: ________________________________
____
____
____

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