UPDATED GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT

GSCHWEND PROPERTY
APNS 708-21-004 AND -005
SANTA TERESA BOULEVARD
SAN JOSE, CALIFORNIA

Prepared For:

Mr. Marc Gschwend
1802 La Terrace Circle
San Jose, California

8 January 2016
Document Id. 15077C-01R1

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

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a division of C2EARTH, INC.

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Mr. Marc Gschwend
1802 La Terrace Circle
San Jose, CA 95123

SUBJECT: UPDATED GEOLOGIC AND GEOTECHNICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
GSCHWEND PROPERTY
APNS 708-21-004 AND -005
SANTA TERESA BOULEVARD
SAN JOSE, CALIFORNIA

Dear Mr. Gschwend:

As you requested, we have performed an updated geologic and geotechnical study for the proposed residential development of your property (composed of two parcels, APNs 708-21-004 and -005) located southwest of Santa Teresa Boulevard in San Jose, California. The accompanying report presents the results of our study, 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,
Upp Geotechnology
a division of C2Earth, Inc.

Jennifer Buckley

Jennifer Buckley
Project Geologist

THIS DOCUMENT HAS BEEN DIGITALLY SIGNED

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1. INTRODUCTION

This report presents the results of our updated geologic and geotechnical study for the proposed residential development of your property (composed of two parcels, APNs 708-21-004 and -005) located southwest of Santa Teresa Boulevard in San Jose, California (see Figure 1, Site Location Map). The purpose of our study was to further explore the geologic and geotechnical conditions on the subject property in the area of the proposed improvements and to update our previous findings and recommendations for the earthwork and foundation engineering aspects of the proposed development.

We previously performed a geotechnical investigation for a similar development concept by a prior owner, and submitted the results of that study in a Geotechnical Investigation report dated 8 December 1998 (Serial No. 9011). That study included excavating and logging eight exploration pits within the vicinity of the proposed home, leachfield, and driveway. Because of the time that has elapsed since our prior study, the changes in the building code and state and local ordinances, and the slightly modified development concept, we are providing this Updated Geologic and Geotechnical Study.

We understand that the current development concept consists of constructing a new single-family residence with a daylighting basement along the crest of a northwest-plunging spur ridge within the central portion of the property. Additionally, an accessory structure is planned upslope and southeast of the residence. We understand that a level pad for the structures will be created primarily by cutting into the hillside. A minor amount of fill may be placed to create a level yard area toward the north and east of the residence.

A driveway will be constructed, using cut and fill grading techniques, from Santa Teresa Boulevard to the proposed residence. In addition, we understand that an existing unpaved graded road that runs along a concrete-lined drainage course will be used as a secondary access to the homesite.

Landscape retaining walls, with a height of 3 feet or less, may be utilized to support cuts along the uphill side of the access road and/or to support fill for the yard near the residence. The development will be serviced by an on-site leachfield system planned for a gently to moderately sloping area north of the residence.

We anticipate that a moderate amount of grading will be required to construct the proposed improvements. We issue this report with the understanding that it is your responsibility as the owner 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. The owner 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 26 June 2015 (Document Id. 15077C-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 our prior report for the subject property, to evaluate the prevailing geologic and geotechnical conditions;
- performing an engineering geologic reconnaissance and mapping of the site in the area of the proposed improvements;
- preparing a partial site plan and engineering geologic map and updated geologic cross-sections;
- analyzing geologic and geotechnical engineering properties from collected data;
- performing qualitative and quantitative slope stability analyses; and
- preparing this report.

We have prepared this report as a product of our service for your exclusive use in designing and constructing the proposed improvements. Other parties may not use this report, nor may the report be used for other purposes, without prior written authorization from Upp Geotechnology, a division of 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 verify in writing the conclusions and recommendations presented in this report.

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, aerial photographs, and our prior reports to evaluate the prevailing geologic conditions of the site and in the vicinity. The Regional Geologic Map, County Hazard Zones Map, and Regional Seismic Hazard Zones Map are presented on Figures 2 through 4, respectively.

3.1. Geology

The subject property is located on a ridge near the base of the Santa Teresa Hills, an isolated northwest-trending range in the southern portion of the Santa Clara Valley and within the California Coast Ranges geomorphic province (see Figure 1). The subject property is situated along a northern-facing slope dominated by two, north- to northwest-trending spur ridges. The spur ridges are flanked by broad swales. According to the Geologic Map and Structure Sections of the Southwestern Santa Clara Valley and Southern Santa Clara Mountains (McLaughlin et al., 2001), the subject site is underlain by Cretaceous age (approximately 66 to 145 million years old) Franciscan assemblage sandstone bedrock at depth (see Figure 2, Regional Geologic Map). The bedrock is described as coherent, bedded, locally conglomeratic, lithic graywacke sandstone with chert, shale, and volcanic detrius.

The bedrock outcrops along eroded creek channels and is at or near the ground surface along prominent ridgelines. The bedrock is overlain by slope debris (colluvium) on the subject property and across most of the hillside areas in the site vicinity. Where the colluvium 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. A review of the geologic maps show that bedding attitudes in the site vicinity are sparse but generally the bedrock bedding strikes (is oriented) approximately east-west, and dips (slopes downward) 57 degrees to the north (see Figure 2).

3.2. Landsliding

Our site reconnaissance, review of aerial photographs, and our prior subsurface exploration revealed evidence of an ancient landslide within a colluvial-filled swale located east of the proposed building area, within an area that will be crossed by the proposed driveway (see Figure 5). Exploration Pits 4 and 5, excavated within the landslide, revealed a 1- to 2-foot thick layer of ancient landslide debris below a thick sequence of younger and older colluvium, at a depth of about 15 feet below ground surface. During our site reconnaissance and review of aerial photographs, we observed slightly hummocky topography within the swale consistent the presence of an ancient landslide. The limits of the landslide appear to measure approximately 320 feet wide by 500 feet long. In addition, based on our review of aerial photographs, we identified a possible second smaller ancient landslide within a swale west of the proposed building site (see Figure 5).

Based upon limited surficial expression, the thick sequence of accumulated colluvium, and the thin layer of landslide debris, it is our opinion that the slope movement is not recent. The landslides are confined to the swales and are well removed from the proposed homesite atop the crest of the spur ridge. Because a portion of the driveway crosses the identified ancient landslide, we performed a quantitative slope stability analysis as described below.

We observed no evidence of recent landsliding on the subject property. According to the Landslide Inventory Map of the Santa Teresa Hills Quadrangle (CGS, 2006), no landslides are mapped within the subject property. The nearest mapped landslide is about 2,000 feet southwest of the site. You should also note that portions of the spur ridge flanks are mapped within the State Seismic Hazard Zone for earthquake-induced landsliding (see Figure 4, Regional Seismic Hazard Zones Map). These zones were established to minimize the loss of life and property by identifying and mitigating seismic hazards related to landslides. Consequently, we have conducted a qualitative slope stability analysis of the slopes around the homesite to evaluate the risk of landsliding to the home and accessory building.

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 (2015) estimates that by 2044, 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 three major faults in the Bay Area.

Fault	Probability (%)
Hayward	14.3
Calaveras	7.4
San Andreas	6.4

30-Year Probability of Magnitude 6.7 or Greater Earthquake

The following table indicates the approximate distance and direction from the site to active and potentially active faults.

Fault	Approx. Distance From Fault	Direction From Site
Coyote/Piercy	1¼ miles	Northeast
Shannon	1½ miles	Southwest
Hayward	4½ miles	Northeast
Calaveras	5½ miles	Northeast
San Andreas	11 miles	Southwest
San Gregorio	30 miles	Southwest

Regional Fault Distances and Directions

According to the City of San Jose Fault Hazard Maps (USGS, 1983), the site is mapped outside of the City of San Jose Potential Hazard Zone for areas prone to earthquake ground rupture (see Figure 4, Regional Fault Hazard Map).

Because of the site's proximity to the Hayward fault and the site's geology, maximum anticipated ground shaking intensities for the area are characterized as strong and equal to a Modified Mercalli (MM) intensity of VI to VII (Borcherdt, et. al., 1975). An earthquake having a MM intensity of VII generally causes slight to moderate damage to well-built ordinary structures, and considerable damage to poorly built or designed structures (Yanev, 1974) (see Table I, Modified Mercalli Scale of Earthquake Intensities).

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.

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 VI 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 site is situated along a northern facing slope near the base of the Santa Teresa Hills. The site is comprised of two adjacent parcels. The larger parcel is approximately 12 acres and is within the City of San Jose. The smaller approximately 5-acre parcel is situated adjacent to the southeastern property boundary of the larger parcel and is within an unincorporated area of Santa Clara County.

The site is elongated with its long axis oriented in the northeast-southwest direction. The subject property is bound to the northwest and southeast by undeveloped land, to the south/southwest by Santa Teresa County Park, and to the northeast by Santa Teresa Boulevard.

4.2. Site Description

As part of our prior study, our founding principal engineer performed a site reconnaissance on 21 November 1997. In addition, our senior engineering geologist and staff geologist performed site mapping and explored the subsurface conditions on 29 and 30 October 1998. Our founding principal engineer returned to the site on 30 July 2015 to observe the site conditions and two soil profile pits excavated in the vicinity of the proposed leachfield. As part of this study, our principal geologist visited the site on 4 January 2016 to perform a site reconnaissance and update the geologic mapping.

We developed an updated partial site plan based upon a Google Earth aerial image and a site plan by RI Engineering, Inc., dated October 2015, supplemented by tape and compass mapping techniques (see Figure 5). In addition, we generated slope profiles from the site plan by RI Engineering, Inc. The slope profiles were used to develop updated geologic cross-sections, as depicted on Figures 6 through 9, Geologic Cross-Sections A-A' through D-D'. The site plan and profiles are only as accurate as implied by the mapping technique used. The following is a summary of the surficial site characteristics.

The site is situated on a north-facing slope along the flank of a northeast-trending ridge. Two north- to northwest-plunging spur ridges flanked by broad swales to the east and west dominate the site topography. The western spur ridge is more prominent than the eastern spur ridge. Slopes on the property vary from gentle along the crest of the spur ridges to moderately steep in the upper portion of the property and along the flanks of the spur ridges (see Figure 5).

In the area of the proposed residence along the crest of the western spur ridge, the ground surface slopes down gently to the north to northwest with a gradients of approximately 10:1 (horizontal to vertical). To the north of the proposed residence in the area of the proposed leachfield along the nose of the spur ridge, the slope increases to approximately 4:1.

The proposed driveway alignment enters the northeastern corner of the property from Santa Teresa Boulevard and leads toward the southwest. The proposed alignment crosses the eastern-most swale, the eastern spur ridge and the swale between the two spur ridges then curves to the west, up the eastern flank of the western spur ridge. The ground surface on the uphill and downhill sides of the proposed driveway alignment generally slopes down to the north with gradients varying from approximately 3:1 to 4:1 (horizontal to vertical).

An existing concrete-lined drainage course named Coyote Alamitos Canal exists along the northwestern property boundary. Cut slopes on the uphill side of the canal are steep and have experienced minor sloughing, which, in places has damaged the concrete canal. An unpaved maintenance road (which will serve as the secondary access to the homesite) is constructed on fill along the downslope side of the canal. At several locations, linear cracks were observed during our 1998 study that were parallel to the downhill side of this road, suggesting settlements and downhill creep of the fill. We did not observe evidence of additional cracks during our recent site visits.

Drainage across the property is characterized as uncontrolled sheet flow to the north and northwest, intercepted by the concrete canal at the northern property line. The site is vegetated with native grasses and scattered oak trees.

4.3. Subsurface

On 29 and 30 October 1998, during our prior study, we observed the subsurface conditions at discrete locations in the vicinity of the proposed improvements by logging eight exploration pits. The pits were excavated to a depth of approximately 19 feet or less using a four-wheel drive backhoe equipped with a 2-foot wide bucket. The approximate locations of the exploration pits are shown on Figure 5. We determined the approximate pit locations by overlying the former site plan with the current site plan. The pit logs are presented in Appendix A of this report. The logs show our interpretation of the subsurface conditions at the locations and on the dates indicated and we do not warrant that they are representative of the subsurface conditions at other locations and times.

Pit 1 was excavated in the area of the proposed leachfiled, Pits 2 and 3 were excavated in the vicinity of the proposed residence, and Pits 4 through 8 were excavated along the proposed driveway alignment.

Exploration Pits 1 through 3 encountered a similar sequence of subsurface materials including approximately ½- to 1 foot of colluvium consisting of silt, underlain by sandstone bedrock that persisted to the bottoms of the excavations.

Exploration Pit 4 encountered approximately 5 to 6 feet of silty colluvium similar to that encountered in Pits 1 through 3. The colluvium is underlain by older colluvium consisting of layers of silty clay and silt. The older colluvium is approximately 8 feet thick. Beneath the older colluvium at a depth of approximately 13 to 14 feet, the pit encountered an approximately 1-foot thick layer of ancient landslide debris consisting of variegated sandy gravel. At the base of this unit, the pit exposed an approximately 1-inch thick layer of light gray gravelly clay, which is interpreted as an ancient landslide failure plane. Moderately fractured shale bedrock was encountered below the surficial soils at a depth of approximately 14½ feet. The shale persisted to the bottom of the excavation at a depth of approximately 16 feet.

Exploration Pit 5 encountered between approximately 2 to 4 feet of silty colluvium similar to that encountered in Pits 1 through 4. The colluvium is underlain by older colluvium consisting of shale and sandstone fragments in a sandy clay matrix. This older colluvium persisted to depths of between approximately 8 to 12 feet. Below the older colluvium, the excavation encountered a 1½- to 2-foot thick layer of the ancient landslide debris consisting of gravelly clay. A 1-inch thick discontinuous layer of gray clay was observed near the base of this unit at the uphill end of the pit. This layer is interpreted as ancient landslide failure plane. Sandstone bedrock was encountered below the older landslide debris at depths varying from 10½ to 14 feet. The sandstone bedrock persisted to the bottom of the excavation at a depth of approximately 14½-feet.

Exploration Pits 6 through 8 encountered similar sequences of subsurface materials, including colluvium and older colluvium underlain by bedrock. The colluvium consists of silt and varies in thickness from approximately 4 to 5 feet. The older colluvium generally consists of layers of silt, clayey silt and clayey sand ranging in depth from approximately 2 to 5 feet in Pits 7 and 8 and up to approximately 9 feet in Pit 6. Shale bedrock was encountered in Pit 6 at a depth of approximately 13 feet, persisting to the bottom of the excavation at a depth of approximately 14-feet. Sandstone bedrock was encountered in Pits 7 and 8 at depths of approximately 9 and 7 feet, respectively. The sandstone bedrock persisted to the bottom of these excavations at depths of approximately 11½ feet and 10½ feet, respectively. The septic pits we observed exposed about 1 to 2 feet of colluvium over fractured sandstone bedrock.

4.4. Groundwater

A minor amount of perched ground water was observed seeping into the base of Pit 5 at a depth of approximately 14½ feet. No free ground water was encountered in any of the exploration pits. According to the Seismic Hazard Report for the 7½-Minute Santa Teresa Hills Quadrangle (California Geological Survey, 2003), groundwater is deeper than 30 feet below the lowest portion of the site near Santa Teresa Boulevard, and is several tens of feet deeper than that below the areas of the proposed driveway and homesite. It should be noted that 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

Our prior study included a laboratory testing program to supplement the evaluation of the geotechnical engineering properties of the soil and bedrock at the site. Soil samples were retained from the pits for laboratory classification and testing. The results of moisture content, dry density, and shear strength tests are presented on the logs (see Appendix).

5. LANDSLIDE SCREENING ANALYSES

Portions of the subject site are mapped within State Seismic Hazard zones for earthquake-induced landsliding (see Figure 3). The purpose of this qualitative screening analysis is to evaluate the severity of the earthquake-induced landsliding hazard on the subject site in the vicinity of the proposed residence and accessory building, and to determine if further analysis is warranted (CDMG, 1996). In accordance with Special Publication 117A by the California Geological Survey (2008) our screening analysis includes an evaluation of the following questions:

- ***Are existing landslides, active or inactive, present on, or adjacent (either uphill or downhill) to the project site?*** As described above, one ancient landslide was identified within a swale east of the proposed ridge-top building site, and a suspected second smaller ancient landslide was mapped within a swale west of the home-site. Both landslides trend down the valley and are well removed from the proposed building site. The proposed homesite is located atop the crest of a bedrock ridge, in an area that would not be affected by a reactivation of the ancient landslides. As described above, a portion of the proposed driveway is planned across the eastern ancient landslide. Consequently, we performed a quantitative slope stability analysis of this landslide, as described in the following section of this report.

- ***Are there geologic formations or other earth materials located on or adjacent to the site that are known to be susceptible to landslides?*** According to the geologic map and our exploration pits, competent sandstone and shale bedrock underlies the proposed building site. These materials are not known to be susceptible to landsliding in the general area.

- ***Do slope areas show surface manifestations of the presence of subsurface water (springs and seeps), or can potential pathways or sources of concentrated water infiltration be identified on or upslope of the site?*** Slope areas on the property in the area of the proposed development are uniform and drainage courses are not disturbed. We did not observe any evidence of springs or seeps in areas that could affect the proposed ridge top building site.

- ***Are susceptible landforms and vulnerable locations present? These include steep slopes, colluvium-filled swales, cliffs or banks being undercut by stream or wave action, areas that have recently slid.*** The eastern swale has been infilled with up to 15 feet of colluvium. The upper, organic rich layers of the colluvium may be susceptible to minor slope movement as a result of creep. Such slope movement would not have an impact on the proposed building site; however periodic cosmetic repairs may be required for the access road.

- ***Given the proposed development, could anticipated changes in the surface and subsurface hydrology (due to watering of lawns, on-site sewage disposal, concentrated runoff from impervious surfaces, etc.) increase the potential for future landsliding in some areas?*** The current development concept will not increase the potential for landsliding on the subject site.

6. SLOPE STABILITY ANALYSES

6.1. Overview

We have conducted a quantitative slope stability analysis to asses the risk of landsliding to the driveway improvements that will cross the swale and ancient landslide deposits east of the homesite.

The following paragraphs describe the methodology and results of 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 Slide 5.0 by Rocscience, Inc., utilizing Janbu's simplified method to calculate 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 one may fail but the probability of stability is higher than that for a slope with a lower factor of safety.

6.2. Slope Geometry

We performed the slope stability analyses utilizing the surface profile depicted on Figure 7. We generated this profile from the supplied site plan and proposed driveway configuration. The subsurface conditions are based upon our 1998 test pits.

6.3. Soil Strength Parameters

We obtained soil and rock strength parameters for the proposed driveway fill, colluvium and ancient landslide debris, and the sandstone/shale bedrock from the published values provided in the Seismic Hazard Zone Report for the Santa Teresa Hills 7½-Minute Quadrangle, Santa Clara County, California (California Geologic Survey, 2003). For the published subsurface units, we used the mean cohesion along with the recommended phi angle. In addition, we assumed wet unit weights based upon our former 1998 laboratory testing. A table of the soil parameters is presented below.

Unit	Phi Angle (degrees)	Cohesion (psf)	Wet Unit Weight (pcf)
Proposed Engineered Fill	27	680	130
Colluvium/Ancient Landslide Debris	11	982	130
Sandstone and Shale Bedrock	31	431	140

Soil and Rock Properties

6.4. Groundwater Conditions

Because of the site's location on a bedrock ridge, the absence of stabilized groundwater in our test pits, and because of the depth of mapped groundwater in the site vicinity, we performed our analysis without the influence of groundwater.

6.5. Seismic Coefficient

A static (non-seismic) analysis was initially performed using no seismic coefficient. Based on information included in the Seismic Hazard Zone Report for the Santa Teresa Hills 7½-Minute Quadrangle, Special Publication 117A, and the site's probabilistic peak ground acceleration (10 percent exceedance in 50 years) obtained from the Ground Motion Interpolator tool (California Geological Survey, 2008), we derived a seismic coefficient of 0.23 for the site and utilized it in our pseudo-static analyses.

6.6. Slope Stability Analysis Results

Our analysis consisted of dozens of iterations to evaluate subsurface conditions and the influence of the proposed driveway fill. 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 global, deep-seated landsliding to occur under static and seismic conditions, respectively. The program searched potential failure planes initiating anywhere on the subject slope.

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 10 and 11, Slope Stability Analysis No. 1 and 2).

Analysis No.	Slope	Seismic	Factor of Safety
1	Cross-Section B-B'	Static	2.56
2	Cross-Section B-B'	0.23	1.41

Slope Stability Analyses and Results

7. 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 presence of an ancient landslide and colluvial filled swale east of the proposed homesite that will be crossed by the proposed driveway and the potential for future creep or landsliding;

- the potential for earthquake-induced landsliding along the steep spur ridge flanks; and

- the site's seismic setting.

7.1. Proposed Building Site and Access Road

Our subsurface study showed that the proposed home and accessory building site is underlain by competent sandstone bedrock. The supportive bedrock is blanketed by up to approximately 1-foot of non-supportive colluvium. In our opinion, the sandstone bedrock should provide adequate support for the foundations of a proposed residence and associated improvements.

Along the proposed driveway alignment our exploration pits encountered colluvium underlain by bedrock. In Pits 4 and 5, a thick sequence of colluvium is underlain by a thin layer of ancient landslide debris and bedrock. In Pits 6 through 8, the colluvium is underlain by shale and sandstone bedrock.

The eastern half of the driveway will be an unpaved and surfaced with gravel, and the western half of the driveway where the grades are steeper, will be paved with asphalt. Where the driveway is underlain by a differential thickness of fill, colluvium and/or ancient landslide deposits, it will be subject to differential movement related to consolidation and creep of these materials. Such differential movement will be less noticeable in the gravel section of the driveway and will likely result in minor cracking of the paved portion of the roadway. We understand, based upon our conversations with you, that you are willing to accept this risk. You should anticipate that periodic maintenance will be required to repair cosmetic distress to the driveway.

7.2. Proposed Leachfield

As currently proposed, the leachfield will be located in a gentle to moderately sloping area north-northwest of the proposed residence. Competent sandstone bedrock was encountered at shallow depths within the exploration pit and soil profile pits that we observed in the area of the proposed leachfield. In our opinion, the construction of the proposed leachfield in this area should not impact the stability of the slopes and should not degrade the quality of the local groundwater. In addition, we understand that percolation testing yielded favorable rates, and the fractured nature of the sandstone should promote the downward migration of septic effluent and, in our opinion, it is unlikely that effluent will surface and create a hazard or a nuisance on the slope below the proposed leachfield.

7.3. Slope Stability

Our study showed no evidence of recent landsliding on the property in the immediate vicinity of the proposed residence and associated improvements. However, as described above, an ancient landslide was identified during our 1998 study within the colluvium-filled swale east of the building site, and a second smaller ancient landslide is suspected to exist within the swale west of the building site. The landslide west of the proposed residence is beyond the limits of any proposed improvements; however, the landslide east of the residence will be crossed by the proposed driveway. The proposed homesite and leachfield will be located outside of either of the ancient landslides.

Based on the minimal surficial expression of the landslide east of the proposed residence, the thick sequence of accumulated colluvium, and the absence of a significant thickness of landslide debris, it is our opinion that this feature is inactive and probably has not experienced movement for thousands of years. Furthermore, our quantitative slope stability analysis yielded a Factor of Safety greater than 1.0 under seismic loading conditions. It is our opinion that there is a low risk landslide reactivation could affect the proposed improvements. The risk of landsliding to the site is no greater than the risk to the average hillside residential property within Santa Clara County.

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 by the acts of Man. 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 Man include improper grading activities, the introduction of excess water through excessive irrigation, improperly designed or constructed leachfields, and 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.

7.4. 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 assume that the site will be subjected to strong ground shaking from a major earthquake on at least one of the nearby active faults during the design life of future improvements. During such an earthquake, it is our opinion that the danger from fault offset through the site is negligible.

8. 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 basement daylighting toward the west will be constructed along the crest of the north to northwest plunging spur ridge within the central portion of the property. Additionally, a pole-barn accessory structure is planned upslope and southeast of the residence. We understand that the level pad for the structures will be created primarily by cutting into the hillside. A minor amount of fill may be placed to create a level yard area toward the north and east of the residence.

A new driveway will be constructed from Santa Teresa Boulevard to provide access to the homesite. We understand that the uphill side of the driveway will be cut into the hillside and fill will be placed along portions of the downhill side. The eastern portion of the road will be unpaved and covered with gravel, the western portion of the road will be paved with asphalt.

Landscape retaining walls, with a height of 3 feet or less, may be utilized to support cuts along the uphill side of the access road and/or to support fill for the yard north of the residence. Concrete slabs-on-grade may be utilized for the basement floor, patios, and walkways.

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

8.1. Location of Proposed Improvements

The proposed improvements must be confined to the approximate building and driveway areas shown on Figure 5. 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.

8.2. 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 2013 California Building Code (CBC). The mapped spectral accelerations and site coefficients were computed using the USGS Seismic Design Maps tool with the 2010 ASCE 7 design code reference (updated 2013).

Design Parameters

Latitude = 37.2163º
Longitude = -121.7622º
Site Class = C
S_s = 1.500
S_1 = 0.600
F_a = 1.0
F_v = 1.3

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.

8.3. Earthwork

At the time of this study, the full extent of any proposed earthwork had not been finalized. We anticipate that a moderate amount of grading will be required to construct the basement, the level building pad, and the driveway. Any proposed earthwork should be performed in accordance with the recommendations provided below.

8.3.1. Clearing and Site Preparation

- Clear all obstructions, including brush, trees not designated to remain, grass, 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. This material must not be used as engineered fill; however, it may be used for landscaping purposes.

8.3.2. Fill Material

- Based on our study, it is our opinion that the materials encountered in the pits should be suitable for use as fill. On-site or imported 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, and

3) no more than 15% of the fill may be greater than 2½ inches in maximum dimension.

- If on-site materials do not meet the requirements given above, they may be off-hauled or used for landscaping purposes only.
- In addition to the requirements above, any import fill must have a plasticity index (PI) of 15% or less.
- **Contact C2 with samples of proposed fill materials at least four days prior to fill placement for laboratory testing and evaluation.**

8.3.3. Keyways and Benches

- Fill placed on slopes in excess of 5:1 must be keyed and benched into the underlying colluvium and/or bedrock to provide a firm, stable surface for support of the fill.
- A keyway, located at the toe of proposed fill, must be excavated a minimum of 5 feet into the colluvium and/or bedrock below the zone of organic rich, root-laden topsoil, as measured on the downhill side of the keyway. We anticipate that the keyway excavation could be up to about 6 feet deep, as measured from the downhill side.
- Benches generally must be a minimum of 5 feet wide and must be excavated entirely into the colluvium and/or bedrock below the organic rich, root-laden topsoil.
- Temporary back slopes may be vertically excavated provided they are constructed in the dry season and meet Cal OSHA requirements.
- Both the keyway and any required 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 keyway and benches at the time of construction**.

8.3.4. 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 (see Figure 12, Conceptual Engineered Fill and Subdrain Diagram).
- Subdrain pipes must be provided with clean-out 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 clean-out.
- 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.

8.3.5. Compaction Procedures

- Prior to fill placement, scarify the surface to receive the fill to a depth of 6 inches.
- Moisture condition the imported 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.

8.3.6. Permanent Slopes

- Construct the gradients of cut and/or fill slopes to 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.

8.3.7. 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.**

8.3.8. Basement Excavation

- Excavate the basement using shoring or an OSHA approved benching or sloping cut configuration selected by an OSHA "Competent Person".
- The contractor is solely responsible for means and methods of construction and should designate appropriate personnel to act as the Competent Person.
- To aid the Competent Person in their selection of construction means and methods, consider the on-site alluvium to be an OSHA Soil Type A. This soil classification must be evaluated and validated by the Competent Person during construction.

8.4. Foundations

Because of the presence of shallow bedrock in the area of the proposed residence and accessory structure, we recommend that the structures be supported on a conventional spread footing foundation system, gaining support in the underlying bedrock. As an alternative, the residence and basement may be supported on a mat-slab foundation, gaining support in the underlying bedrock. The basement retaining walls should be designed and constructed in accordance with the recommendations presented below in the section named "Retaining Walls".

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

8.4.1. Spread Footing

- Embed spread footings a minimum of 12 inches into the underlying supportive bedrock below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the footing and the surface of the bedrock.
- Design the spread footings supported in the bedrock for an allowable bearing pressure of 3,500 psf for dead plus live loads, with a 1/3 increase for transient loads, including wind and seismic.

- All footings adjacent to utility trenches must have their bearing surface below an imaginary plane projected upward from the bottom edge of the trench at a 1:1 (horizontal to vertical) slope.
- Lateral loads may be resisted by friction between the foundation bottoms and the supporting subgrade using a friction coefficient of 0.35.
- As an alternative, a passive pressure equal to an equivalent fluid weight of 450 pcf may be used for footings poured neat in excavations into the bedrock below the plane at which there is a minimum of 5 feet horizontal separation between the downhill face of the footing and the surface of the bedrock.
- Use either passive pressure or the friction coefficient to design for lateral loading. Lateral loads resistance must not combine the use of the friction coefficient and passive pressure.
- The structural design engineer must determine concrete reinforcing; but, as a minimum, all continuous footings must be provided with at least two No. 4 steel reinforcing bars, one placed at the top and one placed at the bottom of the footing, to provide structural continuity and to permit the spanning of any local irregularities.
- Design for differential and total settlement for footings founded in supportive material of less than 1 inch.
- Clear the bottoms of the footing excavations of loose cuttings and soil fall-in prior to the placement of concrete.
- **Contact C2 to observe the footing excavations prior to placing reinforcing steel** to evaluate depth into supportive material.

8.4.2. Mat-Slab

- Support the proposed basement on a mat-slab bearing on the bedrock.
- Design support for the mat-slab in the bedrock for an allowable bearing pressure of 3,500 psf for dead plus live loads, with a 1/3 increase for transient loads, including wind and seismic.
- Lateral loads may be resisted by friction between the concrete mat bottom and the supporting subgrade using a friction coefficient of 0.35. If a waterproofing membrane will be placed between the bottom of the mat and the supportive subgrade, the friction coefficient will be compromised and lateral loads must be resisted by passive pressure or other means.
- As an alternative, a passive pressure equal to an equivalent fluid weight of 450 pcf may be used for the mat if it is poured neat in excavations into the supportive material, below the plane at which there is a minimum of 5 feet horizontal

separation between the downhill face of the excavation and the surface of the bedrock.

- Use either passive pressure or the friction coefficient to design for lateral loading. Lateral loads resistance must not combine the use of the friction coefficient and passive pressure.
- We anticipate differential and total settlement of the mat slab founded in supportive material to be less than 1 inch.
- Concrete reinforcing must be provided in accordance with the recommendations of the structural design engineer.
- Provide the mat-slab with the appropriate damp proofing. Damp proofing may affect the lateral load resistance (see above).
- **Contact C2 to observe the excavations prior to placing reinforcing steel** to evaluate depth into supportive material.

8.4.3. Retaining Walls

We anticipate that non-engineered landscape retaining walls, 3 feet tall or less, may be used on the site to support cuts and fills for the access road and yard. If you plan to construct engineered site retaining walls, please contact us to provide additional recommendations. The following recommendations are for cantilever type walls for the planned basement. Contact us to provide appropriate recommendations if you consider other types of walls.

- Support residential basement retaining walls on foundations designed in accordance with the recommendations given above for the support of the proposed residence.
- 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 percent of the wall height or less.
- Design unrestrained (active condition) walls with level backfill to resist an equivalent fluid pressure of 45 pcf. Design walls that are restrained from movement at the top or sides (at-rest condition) with level backfill to resist an equivalent fluid pressure of 68 pcf (see Figure 13, Conceptual Retaining Wall Pressure Diagram).
- 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 22 pcf.

- If seismic loading is considered, design basement retaining walls to resist the most critical loading: either the at-rest condition if the walls are restrained, or the active condition plus the seismic increment if the walls are unrestrained.
- Wherever the walls will be subjected to surcharge loads, they must be designed for an additional uniform lateral pressure equal to 1/2 or 1/3 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) placed behind the wall.
- Separate the drainrock from the backfill by a geotextile filter fabric, such as Mirafi 140 or an alternative, approved by C2. A 4-inch diameter heavy-duty rigid perforated subdrain pipe (Schedule 40, SDR 21 or equivalent), approved by C2, must be placed with the perforations down on a 2- to 3-inch layer of drainrock at the base of the drain. Where subdrain pipes will be buried deeper than 10 feet, Schedule 80 or equivalent pipe should be used. **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 mat's filter fabric must be placed around the drainpipe and between the pipe and the soil.
- 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.
- Details of backdrain options are presented on Figure 14, 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 clean-out 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 clean-out. 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. This maintenance must include inspection and flushing to make sure that subdrain pipes are free of debris and are in good working order; and 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.
- Provide retaining walls adjacent to living spaces and site walls with decorative facing with appropriate damp proofing. 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.
- If you select an alternative retaining wall type, you should contact C2 to provide additional recommendations.

8.4.4. Gravel Driveway and Flexible Pavement (Asphalt)

We anticipate that the eastern portion of the driveway will be unpaved and surfaced with gravel, and the western portion will be paved with flexible pavement (asphalt). The following recommendations are based upon an anticipated Traffic Index (TI) of 3. If a greater TI is required for the project, contact C2 for appropriate recommendations.

For the unpaved, gravel surfaced portion of the driveway:

- Scarify and re-compact the upper of 6-inches of the sub-base to the requirements for engineered fill given above.
- Use a minimum gravel thickness of 10 inches of CalTrans Class II baserock compacted to at least 90% relative compaction in accordance with the requirements for engineered fill given above.

For flexible pavement we recommend the following minimum requirements:

- Scarify and re-compact the upper of 6-inches of the sub-base to the requirements for engineered fill given above.

- We recommend that the driveway pavement be supported on a uniform thickness of engineered fill placed in accordance with the requirements for engineered fill provided above.
- Construct the driveway so that the entire width of the driveway at any given section is underlain by a uniform thickness of fill.
- Use a minimum pavement section of 2 inches of asphalt over 6 inches of CalTrans Class II baserock compacted to at least 95% relative compaction in accordance with the requirements for engineered fill given above.
- **Contact C2 to observe and test compaction of the sub-base recompaction and baserock compaction.**

8.4.5. Flatwork

We anticipate that concrete slabs-on-grade may be used for the basement, patios and walkways. Where located on 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 with engineered fill, keyed and benched into the bedrock, or support the slabs directly on the bedrock in accordance with the recommendation provided above.

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 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.

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.
- Traditionally, designers have specified the following: place 4 inches of free-draining gravel beneath the floor slab to serve as a capillary barrier between the subgrade soil and the slab. Following gravel placement, place a heavy-duty membrane over the gravel in order to minimize vapor transmission and then place 2 inches of sand over the membrane to protect it during construction. Just prior to placing concrete, lightly moisten the sand.
- More recent standards suggest 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.
- 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.
- Use the gravel, heavy-duty membrane, and/or sand (if specified) in lieu of the upper 6 inches of recommended non-expansive fill.

8.5. 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 risk of 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.
- 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 via buried, closed conduits or lined surfaces.

- Discharge collected water in an appropriate manner and at an appropriate location approved by C2. Do not locate the discharge on, or adjacent to, steep, potentially unstable terrain.
- Use buried conduits consisting of rigid, smooth-walled pipes (PVC). **Do not use flex-pipes.**
- Provide downspouts with slip-joint connectors or clean-outs, where they are connected to buried pipes, to facilitate maintenance (see Figure 15, Conceptual Downspout Clean-Out Diagram).
- 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 trench and covered with a mound of cobbles (see Figure 16, 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.

9. PLAN REVIEW AND CONSTRUCTION OBSERVATION

We must be retained to review the final grading, foundation, and drainage control plans in order to verify that our recommendations have been properly incorporated into the proposed project. **WE MUST BE GIVEN AT LEAST ONE WEEK 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:

- verify that 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 insure 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 keyway and bench excavations, evaluate the need for subdrainage, and to test compaction of engineered fill
2. **MAT-SLAB / FOOTING EXCAVATION:** Prior to placement of reinforcing steel to evaluate depth to supportive material
3. **BASEMENT RETAINING WALL BACKDRAIN:** During installation
4. **BASEMENT RETAINING WALL BACKFILL:** During backfill to observe and test compaction
5. **SLABS-ON-GRADE AND FLEXIBLE PAVEMENT:** 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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LIST OF AERIAL PHOTOGRAPHS

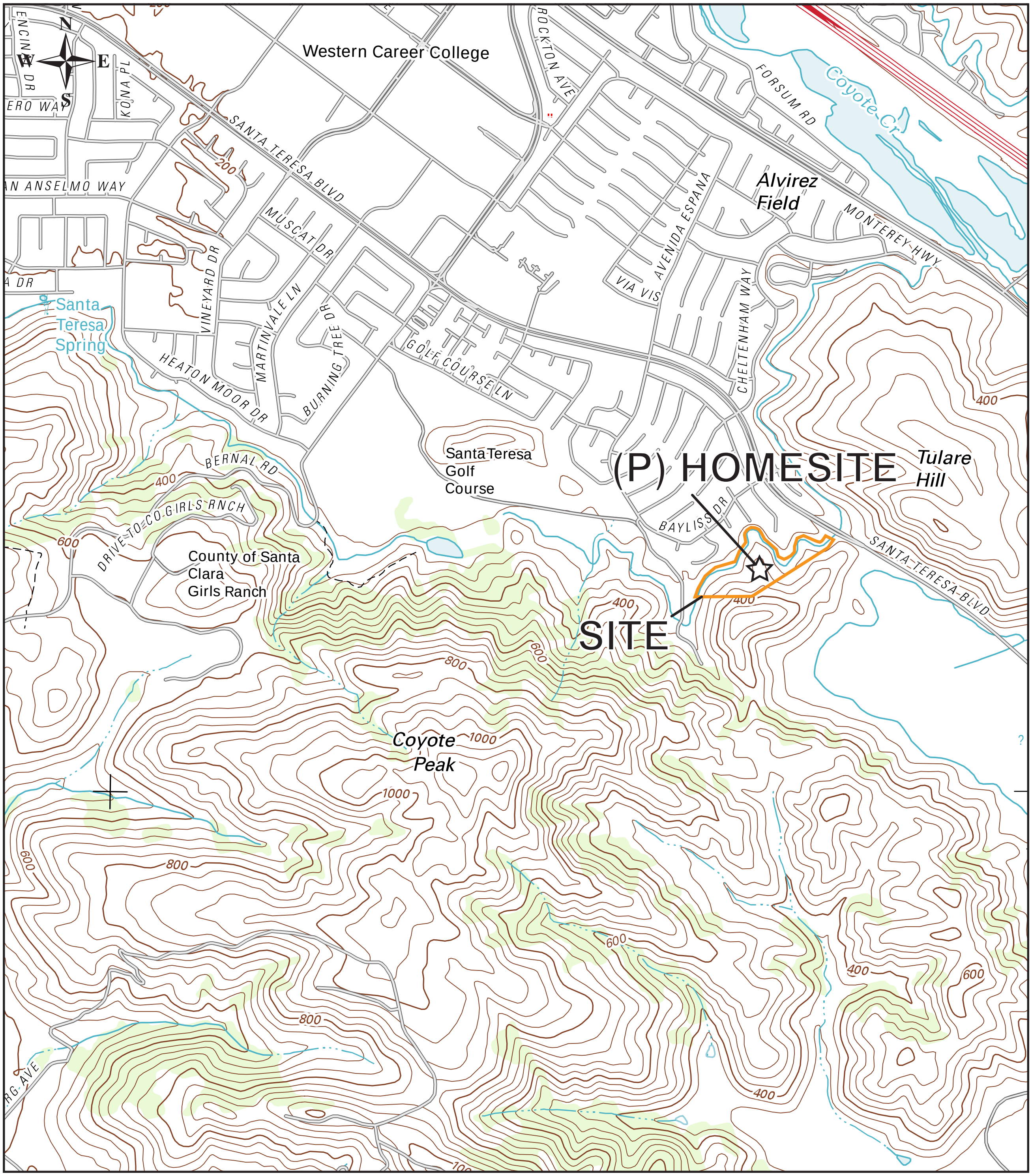
"BAY AREA TRANSPORTATION STUDY", black and white, dated May 14, 1965, at a scale of 1" = 1,000', Aerial Survey Contract No. 67615, Serial Nos. SCL 22-211 and SCL 22-212, State of California Highway Transportation Agency, Division of Highways.

FIGURES, TABLE, AND APPENDIX

FIGURE NO.

TABLE NO.

APPENDIX



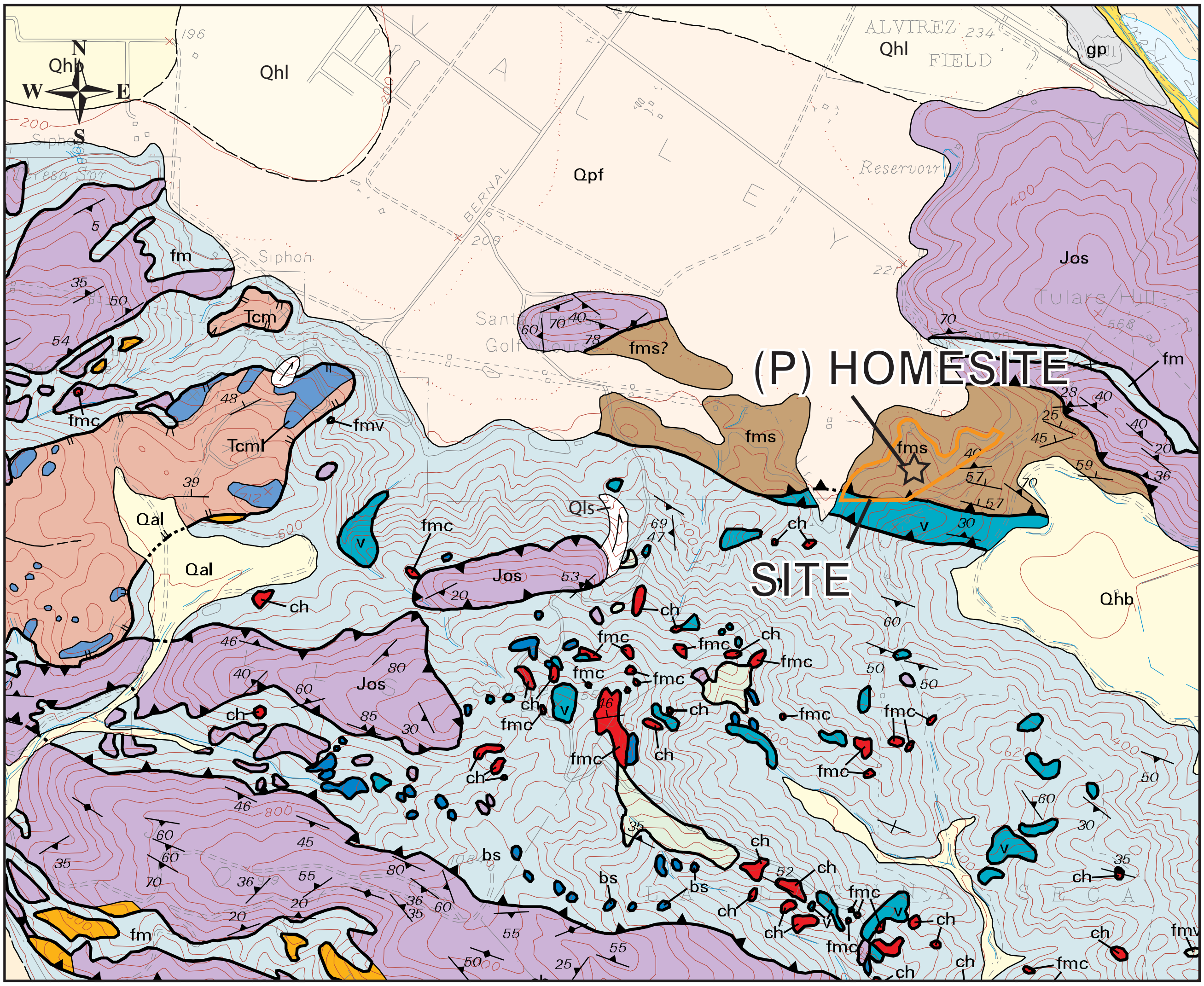
BASE: The National Map US Topo; UNITED STATES GEOLOGICAL SURVEY; 2012

SITE LOCATION MAP

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

GSCHWEND PROPERTY
APNs 708-21-004 and -005
Santa Teresa Boulevard
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	1" = 2,000'	15077C-01R1	January 2016	Figure 1



EXPLANATION

Qal	- Alluvium
Qls	- Landslide deposits
Qhb	- Basin deposits
Qhl	- Levee deposits
Qpf	- Alluvial fan deposits
Tcm	- Mottled mudstone/sandstone
fm	- Franciscan melange
fms	- Sandstone
Jos	- Serpentinized ultramafics

15 Strike and dip

Vertical dip

15 Inclined shear foliation

15 Vertical shear foliation

Landslide deposit
arrows indicate general direction of movement

Subject Property

Geologic contact
dashed where approximate and dotted where concealed

Fault
dashed where approximate and dotted where concealed

Thrust fault
barbs on upper plate

Attenuation fault
low angle normal fault, double hachures on down dropped (upper plate)

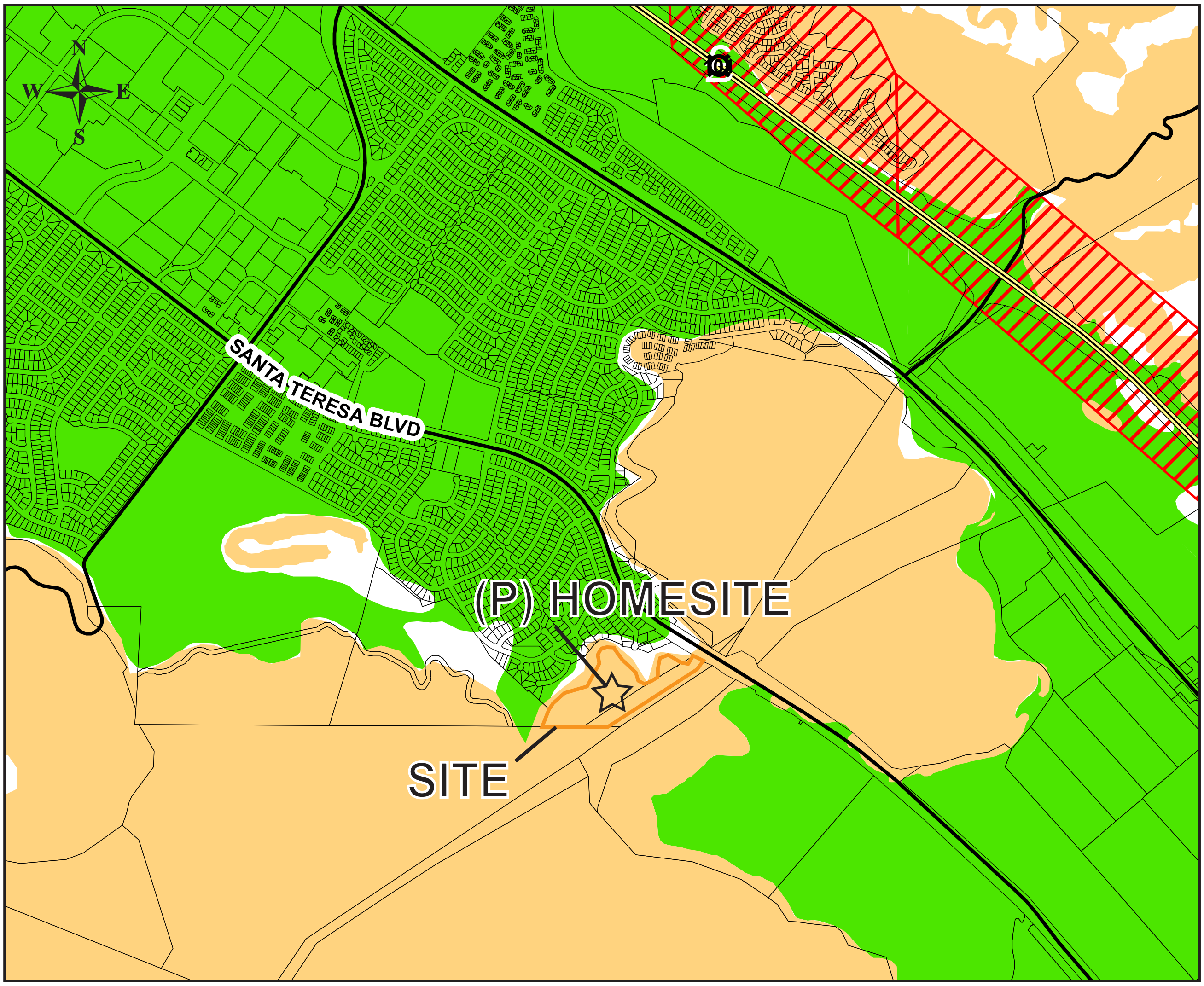
BASE: Sheet 4: Santa Teresa Hills and Southwestern Part of the Morgan Hill Quadrangles, Geologic Maps and Structure Section of the Southwestern Santa Clara Valley and Southern Santa Cruz Mountains, Santa Clara and Santa Cruz Counties, California; MCLAUGHLIN, ET AL.; 2001

REGIONAL GEOLOGIC MAP

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

GSCHWEND PROPERTY
APNs 728-21-004 and -005
Santa Teresa Boulevard
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	1" = 2,000'	15077C-01R1	January 2016	Figure 2



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 37; 26 October 2012

COUNTY HAZARD ZONES MAP

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

GSCHWEND PROPERTY
APNs 728-21-004 and -005
Santa Teresa Boulevard
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	1" = 2,000'	15077C-01R1	January 2016	Figure 3

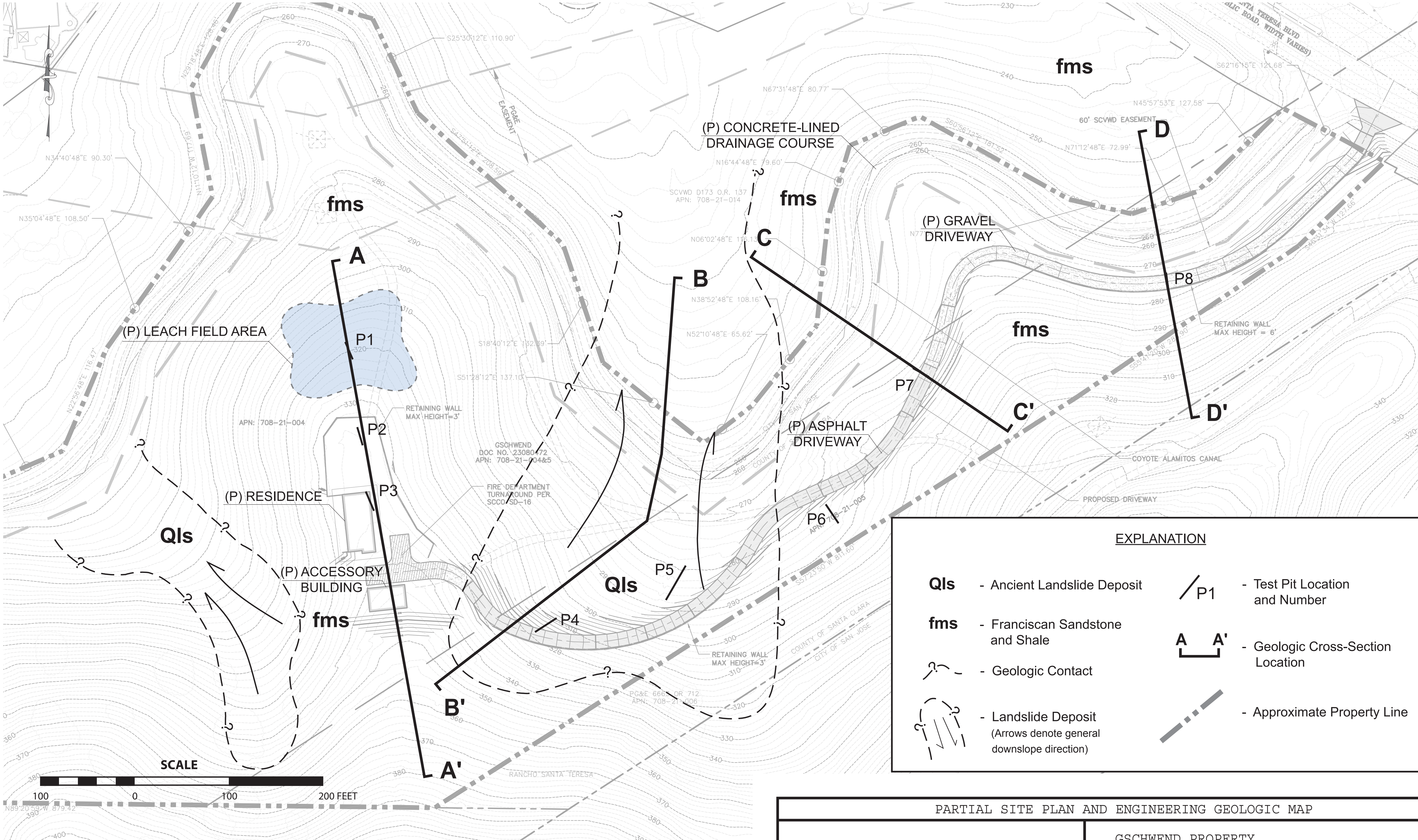
EXPLANATION

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

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

BASE: Seismic Hazard Zones, Santa Teresa Hills Quadrangle; California Geological Survey; 14 August 2003

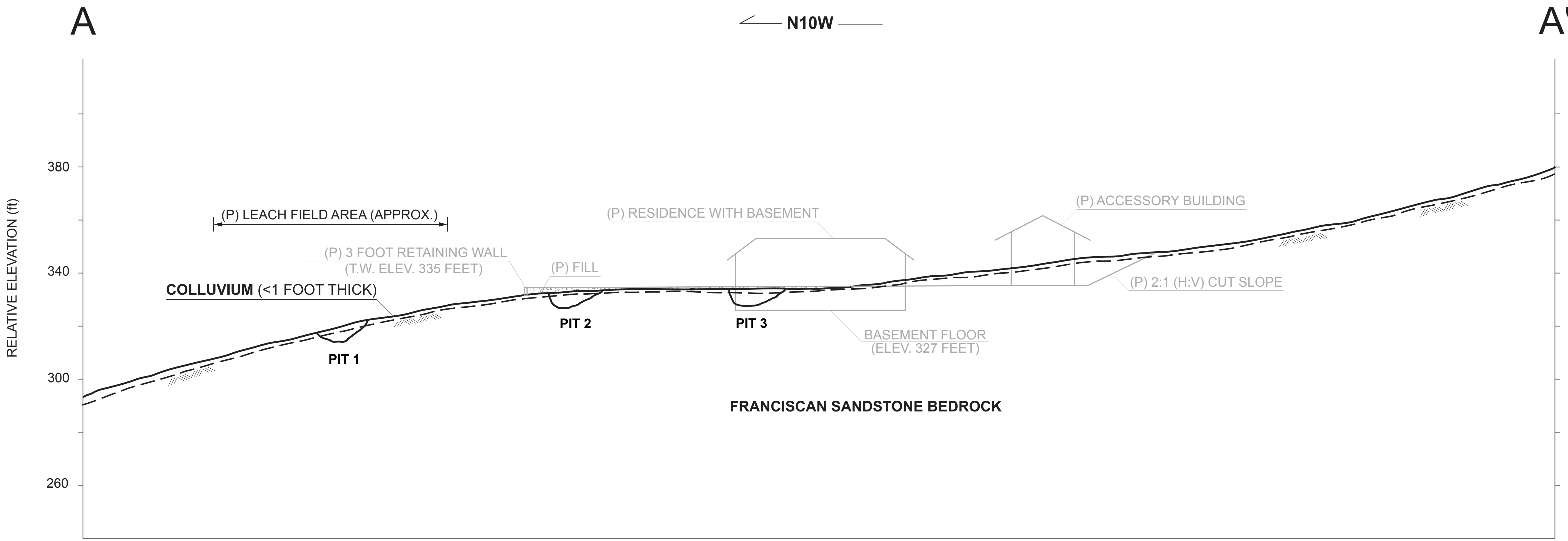
REGIONAL SEISMIC HAZARD ZONES MAP				
UPP GEOTECHNOLOGY a division of C2EARTH, INC.		GSCHWEND PROPERTY APNs 728-21-004 and -005 Santa Teresa Boulevard San Jose, California		
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	1" = 2,000'	15077C-01R1	January 2016	Figure 4



NOTE: This plan is a conceptual illustration of observed geologic and geotechnical features and should not be used for any other purpose.

BASE: Site Plan; Sheet C1; RI ENGINEERING, INC.; October 2015; and,

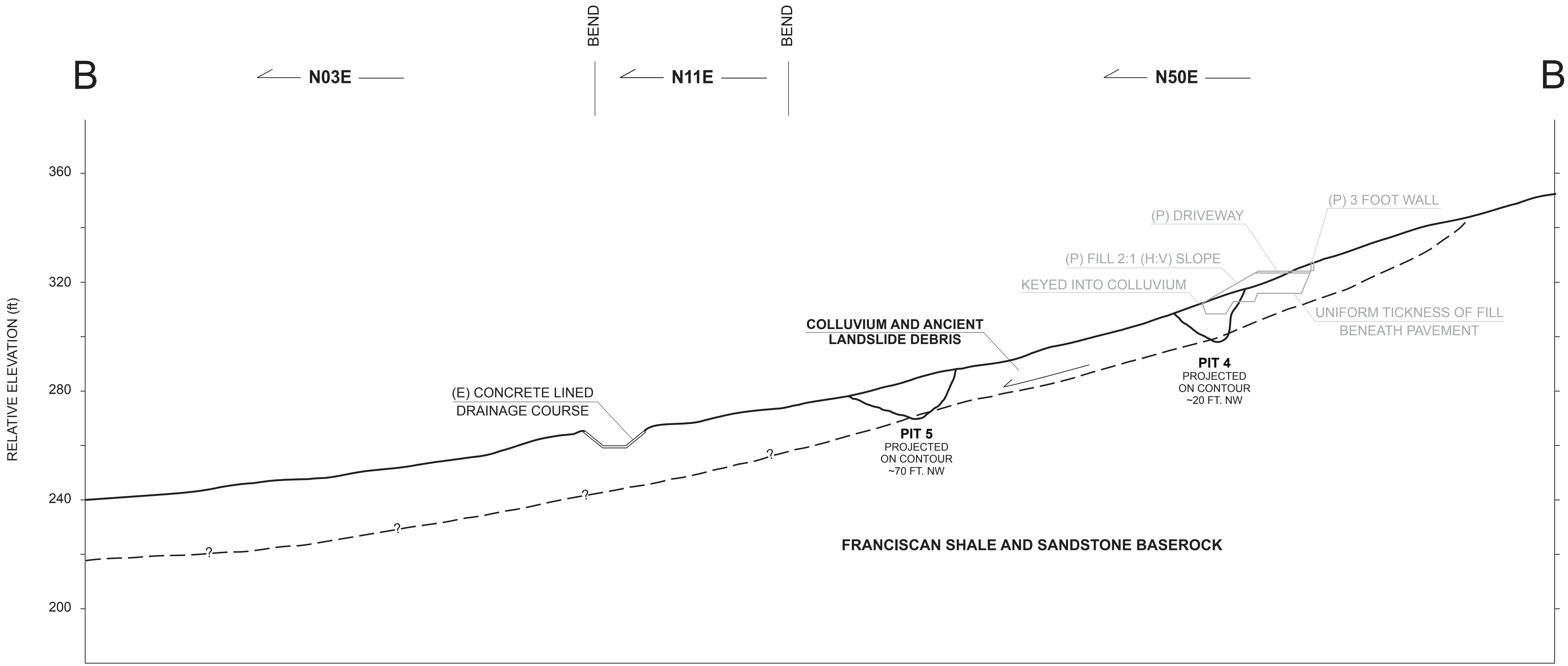
PARTIAL SITE PLAN AND ENGINEERING GEOLOGIC MAP				
UPP GEOTECHNOLOGY a division of C2EARTH, INC.		GSCHWEND PROPERTY APNs 708-21-004 and -005 Santa Teresa Boulevard San Jose, California		
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	As Shown	15077C-01R1	January 2016	Figure 5



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

BASE: Site Plan; Sheet C1; RI ENGINEERING, INC.; October 2015

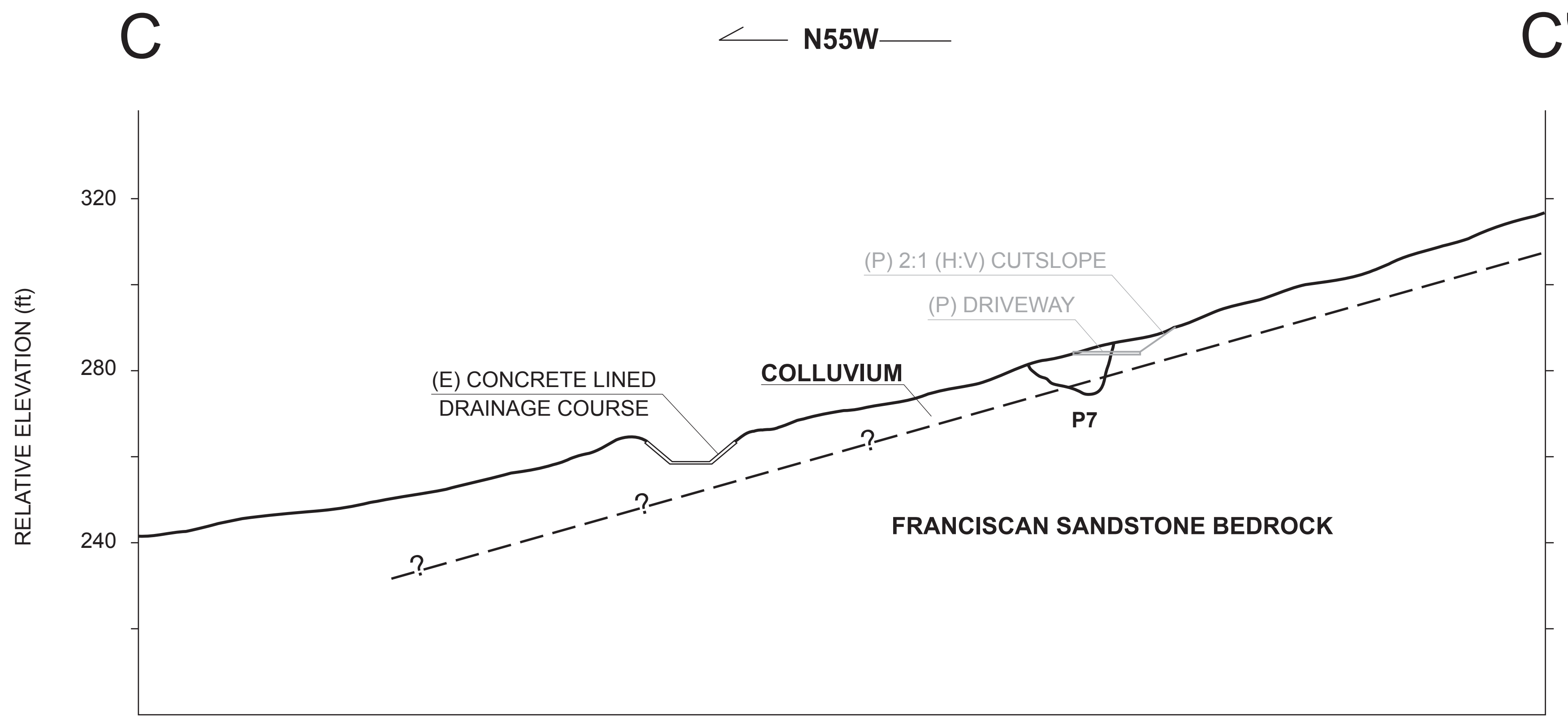
GEOLOGIC CROSS-SECTION A-A'				
UPP GEOTECHNOLOGY a division of C2EARTH, INC.		GSCHWEND PROPERTY APNs 728-21-004 and -005 Santa Teresa Boulevard San Jose, California		
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	1" = 40'	15077C-01R1	January 2016	Figure 6



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

BASE: Site Plan; Sheet C1; RI ENGINEERING, INC.; October 2015

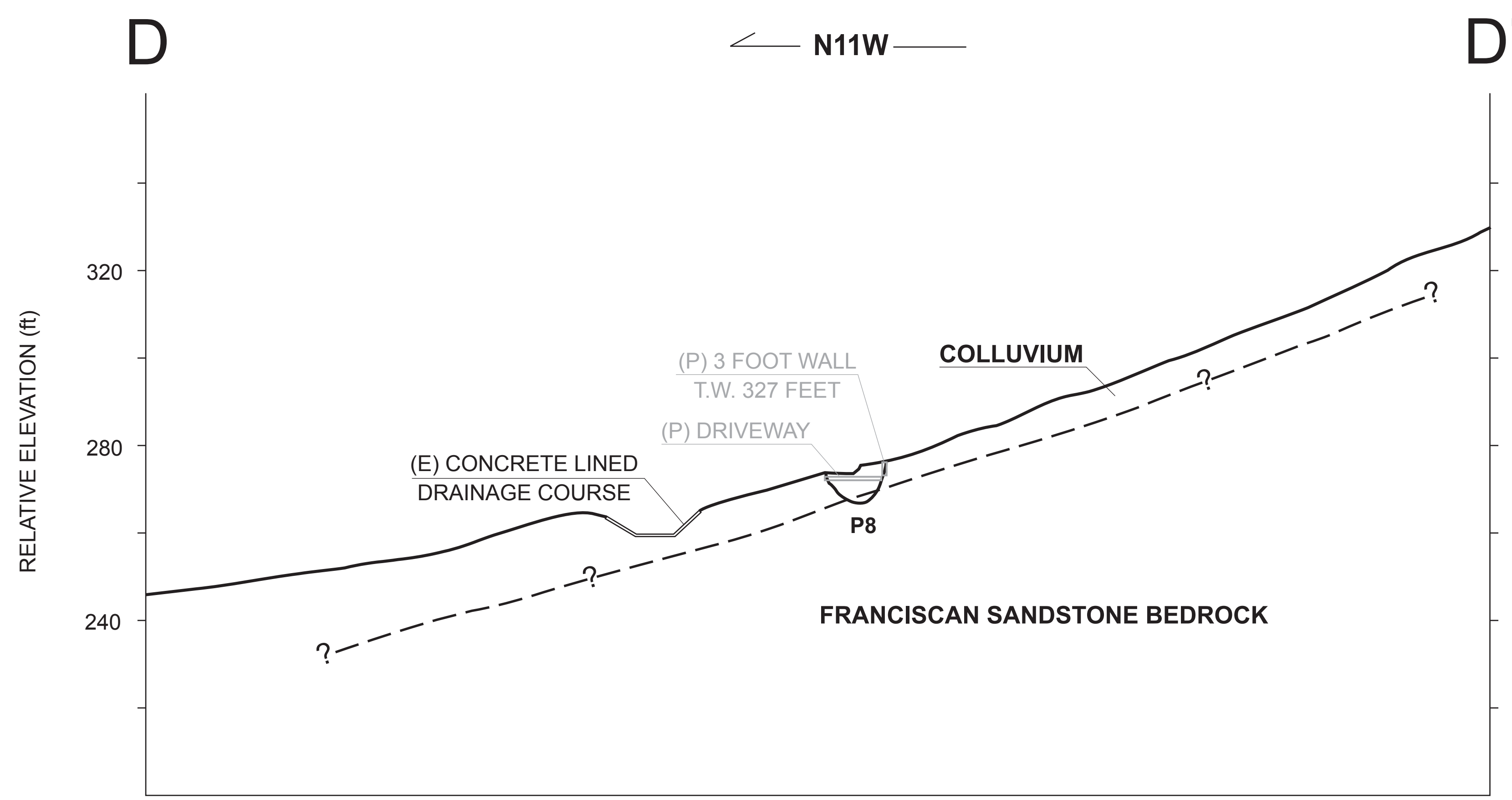
GEOLOGIC CROSS-SECTION B-B'				
UPP GEOTECHNOLOGY a division of C2EARTH, INC.		GSCHWEND PROPERTY APNs 728-21-004 and -005 Santa Teresa Boulevard San Jose, California		
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	Figure 7
EB/CH	1" = 40'	15077C-01R1	January 2016	



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

BASE: Site Plan; Sheet C1; RI ENGINEERING, INC.; October 2015

GEOLOGIC CROSS-SECTION C-C'				
UPP GEOTECHNOLOGY a division of C2EARTH, INC.		GSCHWEND PROPERTY APNs 728-21-004 and -005 Santa Teresa Boulevard San Jose, California		
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	1" = 40'	15077C-01R1	January 2016	Figure 8



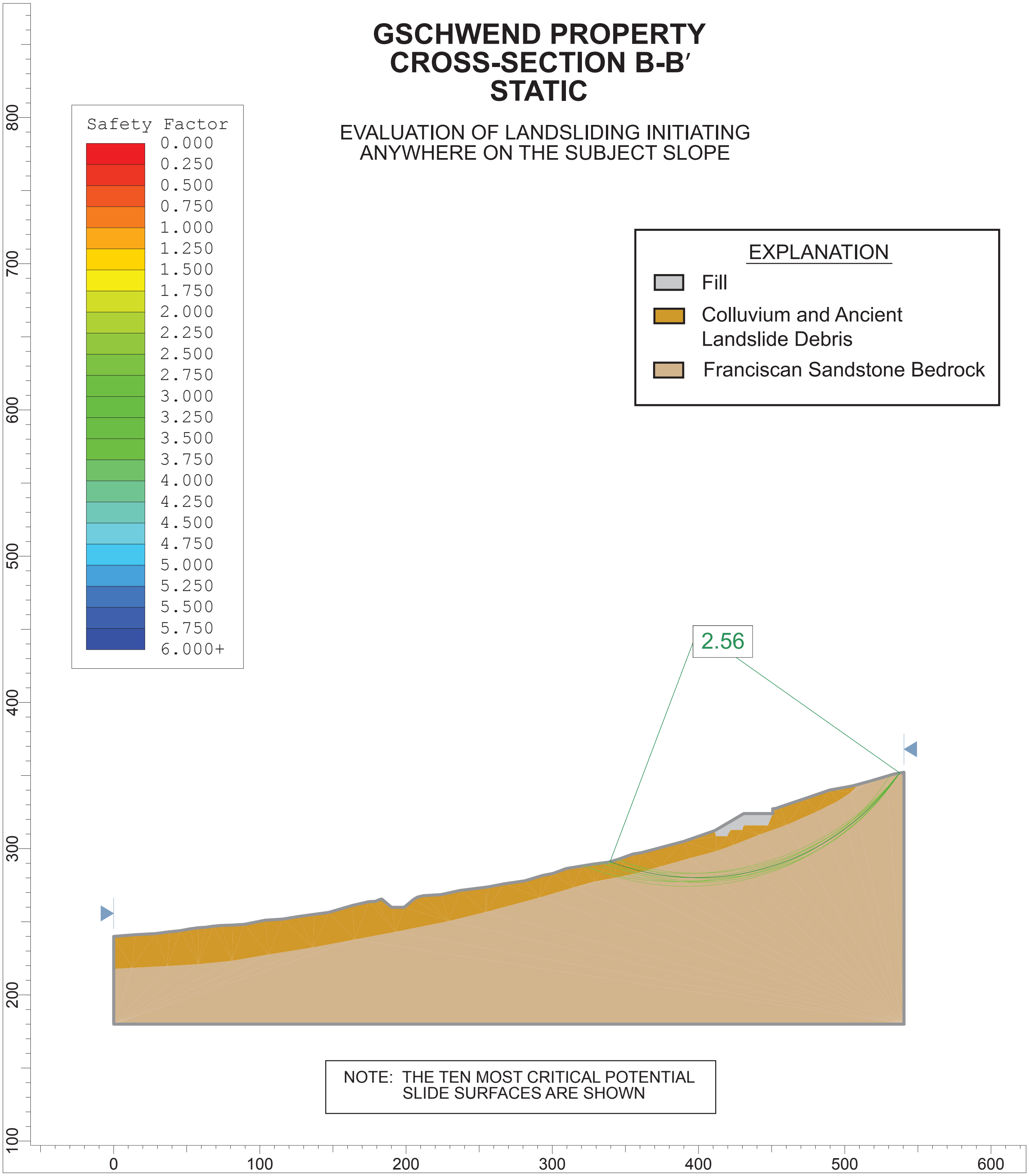
NOTE: This cross-section is a conceptual illustration of general subsurface relationships and should not be used for any other purpose.

BASE: Site Plan; Sheet C1; RI ENGINEERING, INC.; October 2015

GEOLOGIC CROSS-SECTION D-D'				
UPP GEOTECHNOLOGY a division of C2EARTH, INC.		GSCHWEND PROPERTY APNs 728-21-004 and -005 Santa Teresa Boulevard San Jose, California		
DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	1" = 40'	15077C-01R1	January 2016	Figure 9

GSCHWEND PROPERTY
CROSS-SECTION B-B′
STATIC

EVALUATION OF LANDSLIDING INITIATING
ANYWHERE ON THE SUBJECT SLOPE



NOTE: THE TEN MOST CRITICAL POTENTIAL
SLIDE SURFACES ARE SHOWN

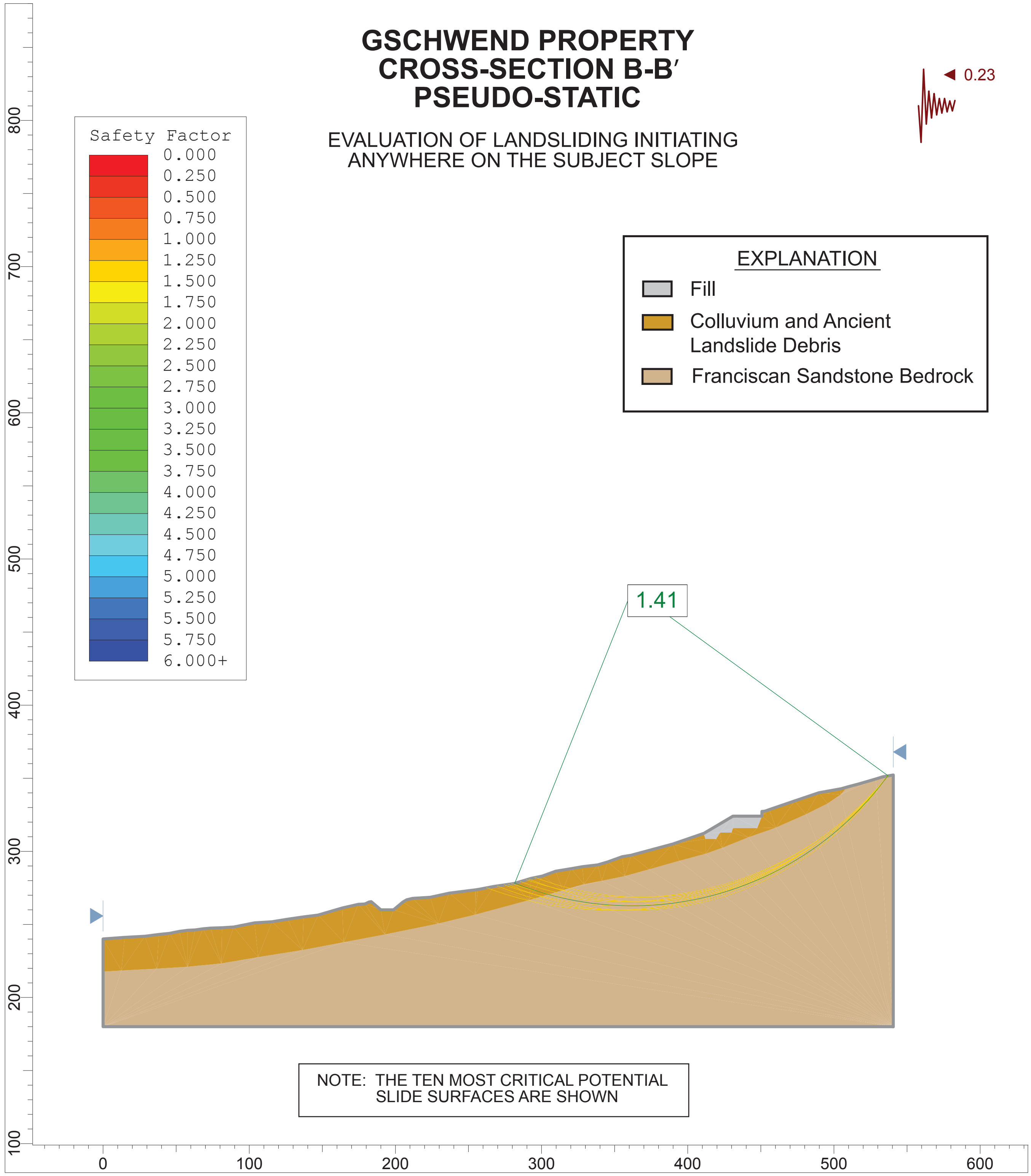
BASE: Slide 5.0; ROCSCIENCE INC.; Version 5.041; Build 7-8-2009

SLOPE STABILITY ANALYSIS NO. 1

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

GSCHWEND PROPERTY
APNs 728-21-004 and -005
Santa Teresa Boulevard
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	As Shown	15077C-01R1	January 2016	Figure 10



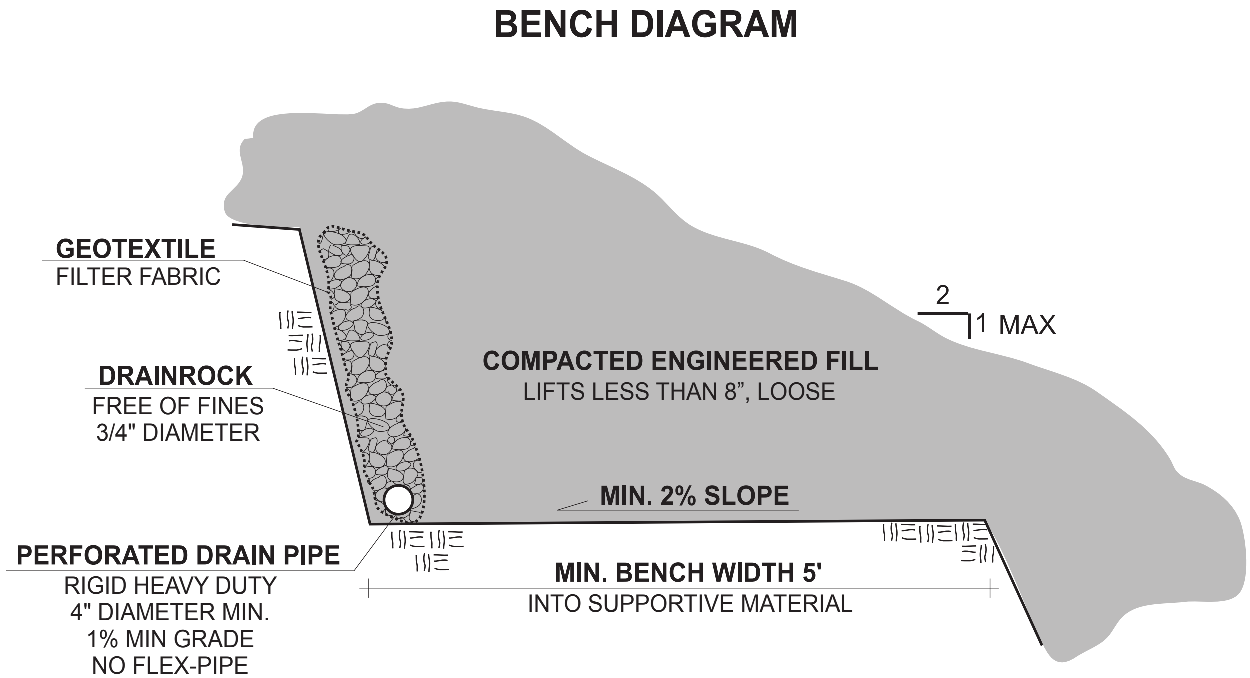
BASE: Slide 5.0; ROCSCIENCE INC.; Version 5.041; Build 7-8-2009

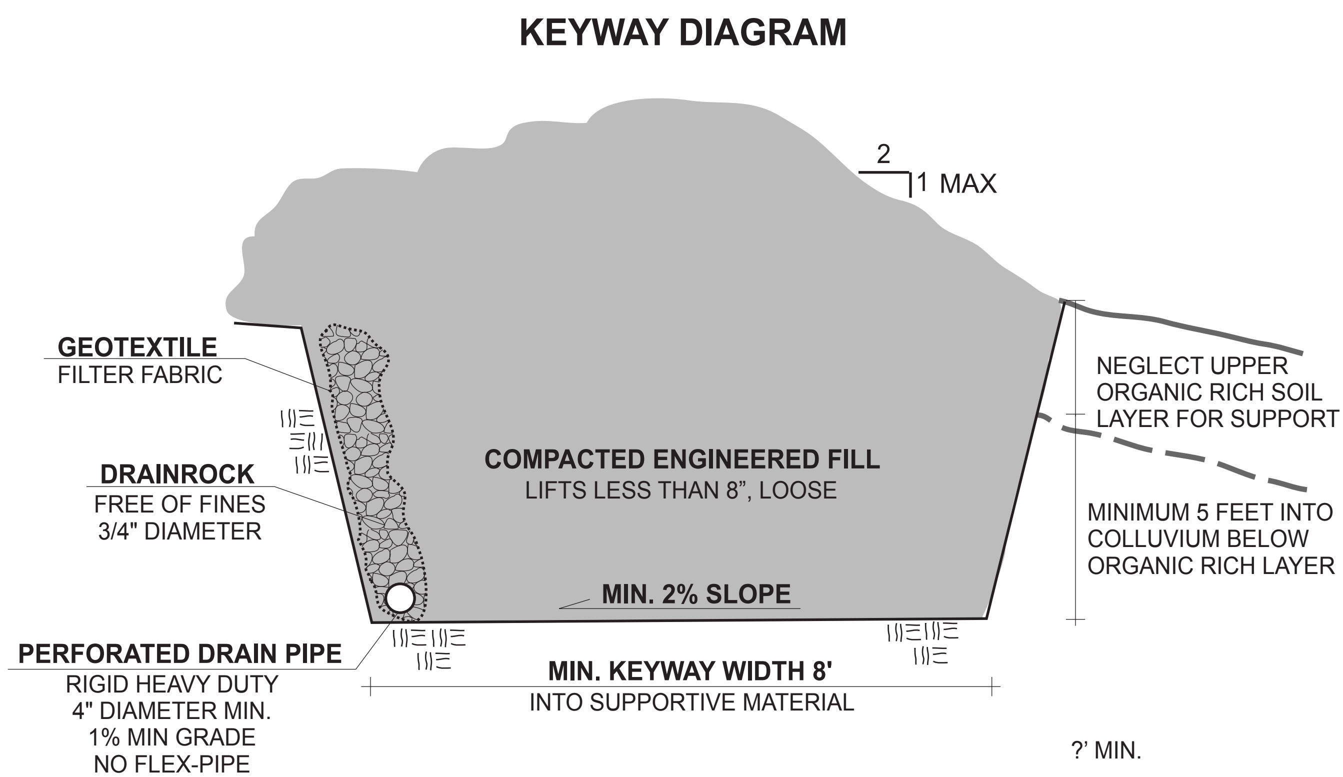
SLOPE STABILITY ANALYSIS NO. 2

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

GSCHWEND PROPERTY
APNs 728-21-004 and -005
Santa Teresa Boulevard
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	As Shown	15077C-01R1	January 2016	Figure 11

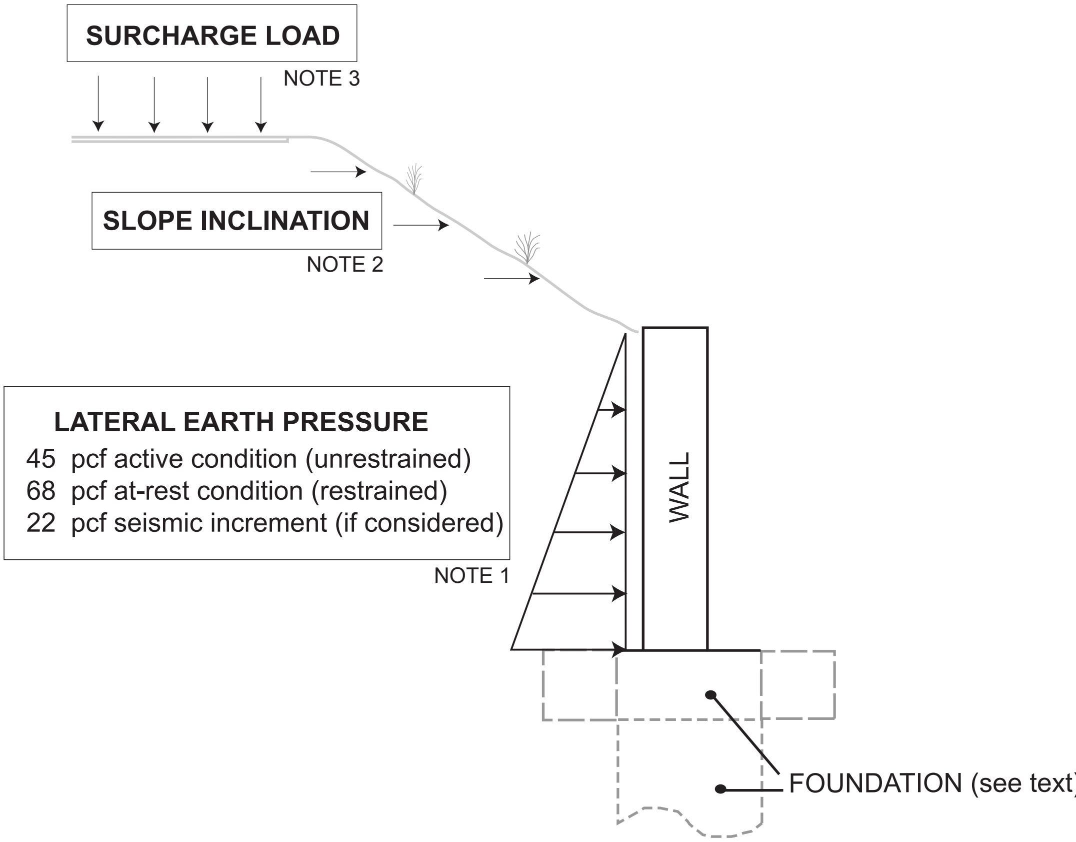




CONCEPTUAL ENGINEERED FILL AND SUBDRAIN DIAGRAM

UPP GEOTECHNOLOGY a division of C2EARTH, INC.	GSCHWEND PROPERTY APNs 728-21-004 and -005 Santa Teresa Boulevard San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	Not Applicable	15077C-01R1	January 2016	Figure 12



Note 1: Lateral earth pressures are shown for drained retaining walls. Contact us to provide additional recommendations if undrained walls are planned.

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

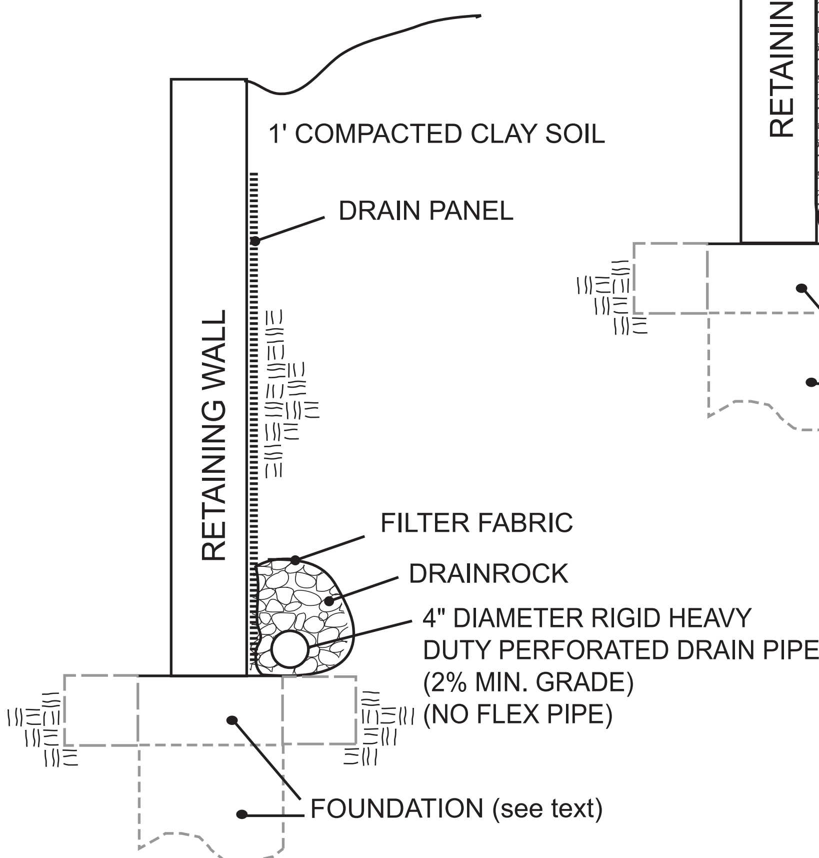
Note 3: Additional lateral load equal to 1/3 (unrestrained) or 1/2 (restrained) the anticipated surcharge load.

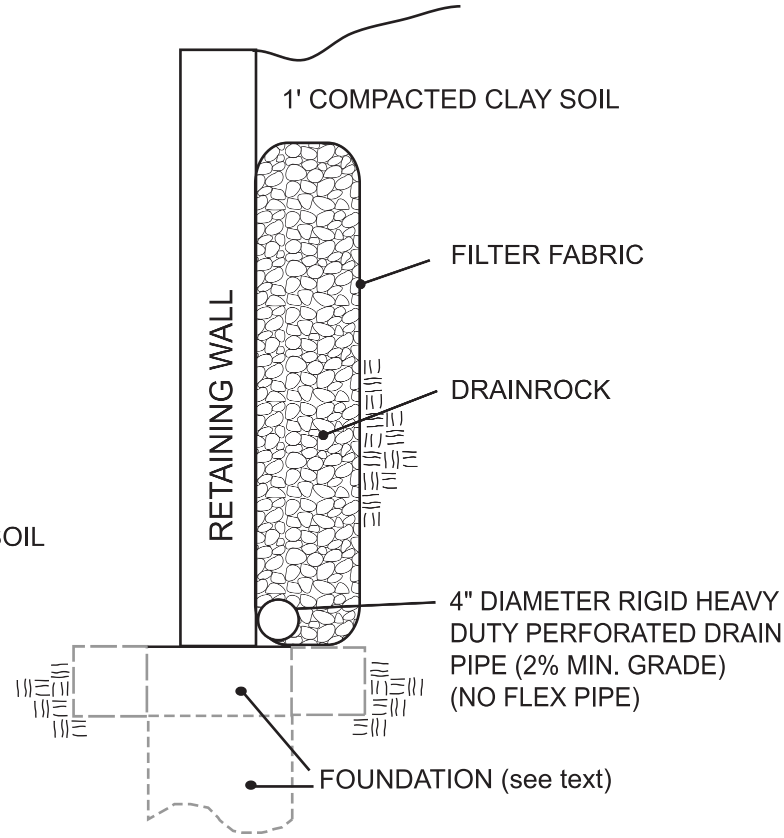
CONCEPTUAL RETAINING WALL PRESSURE DIAGRAM

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

GSCHWEND PROPERTY
APNs 728-21-004 and -005
Santa Teresa Boulevard
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	Not Applicable	15077C-01R1	January 2016	Figure 13



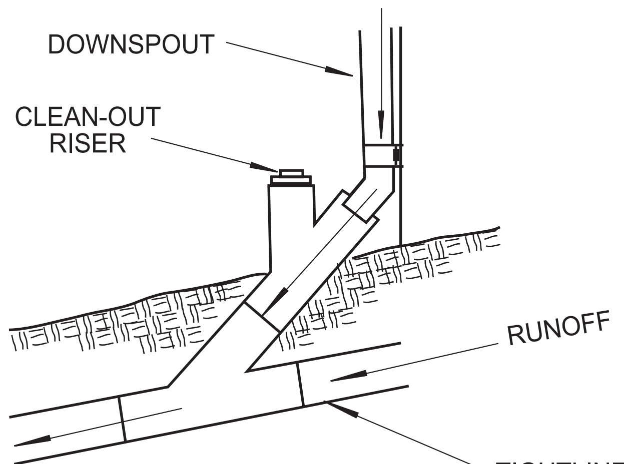


CONCEPTUAL RETAINING WALL BACKDRAIN DIAGRAM

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

GSCHWEND PROPERTY
APNs 728-21-004 and -005
Santa Teresa Boulevard
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	Not Applicable	15077C-01R1	January 2016	Figure 14

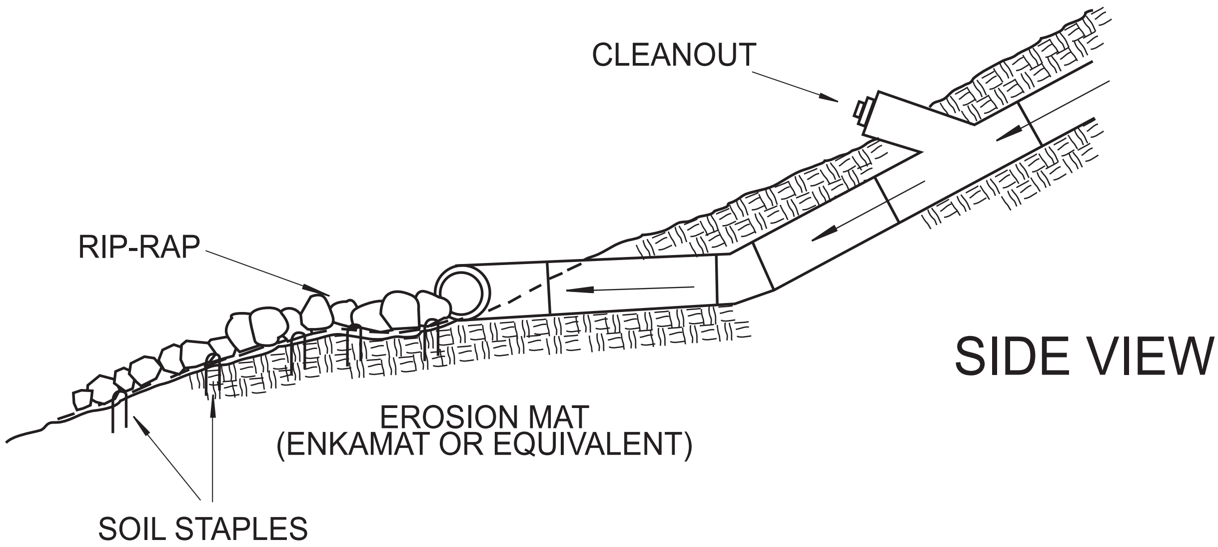


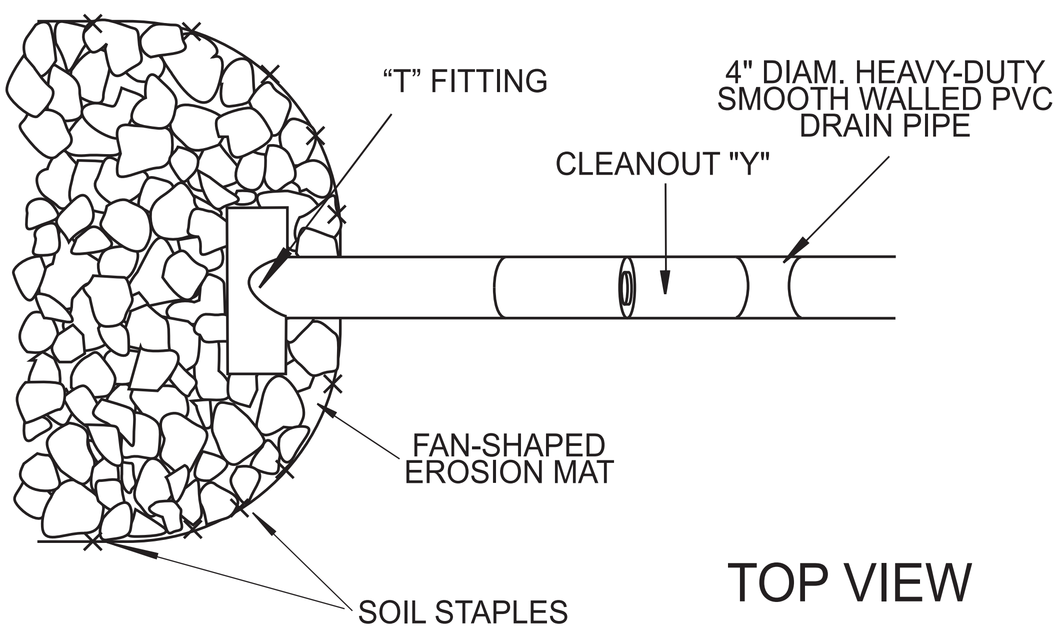
CONCEPTUAL DOWNSPOUT CLEAN-OUT DIAGRAM

UPP GEOTECHNOLOGY
a division of C2EARTH, INC.

GSCHWEND PROPERTY
APNs 728-21-004 and -005
Santa Teresa Boulevard
San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	Not Applicable	15077C-01R1	January 2016	Figure 15





CONCEPTUAL ENERGY DISSIPATER DIAGRAM

UPP GEOTECHNOLOGY a division of C2EARTH, INC.	GSCHWEND PROPERTY APNs 728-21-004 and -005 Santa Teresa Boulevard San Jose, California

DRAFTED/REVIEWED	SCALE	DOCUMENT ID.	DATE	
EB/CH	Not Applicable	15077C-01R1	January 2016	Figure 16

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.

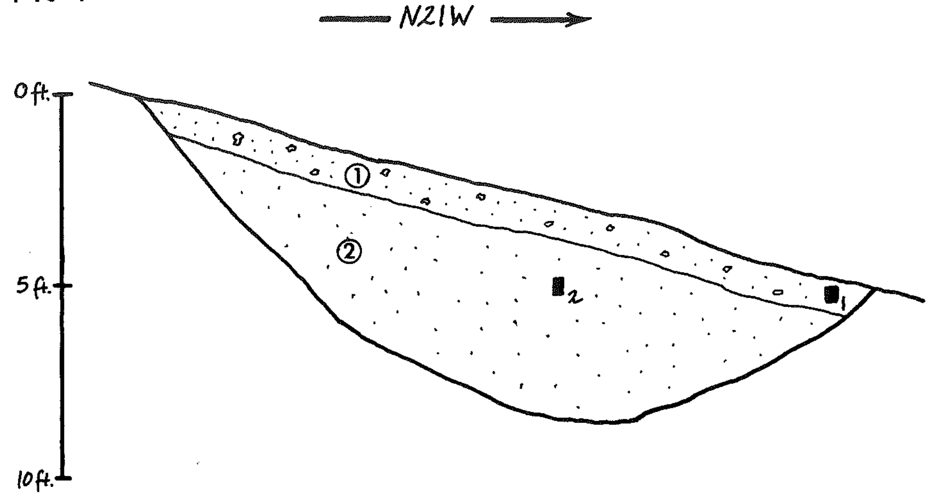
XII. Damage is total, with practically all works of construction severely damaged or destroyed. Waves are observed on ground surfaces, and all soft or wet soils are greatly disturbed. Heavy objects are thrown into the air, and large rock masses are displaced.

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

APPENDIX A

LOGS OF EXPLORATION PITS 1 THROUGH 8 (UPP GEOTECHNOLOGY, 1998)

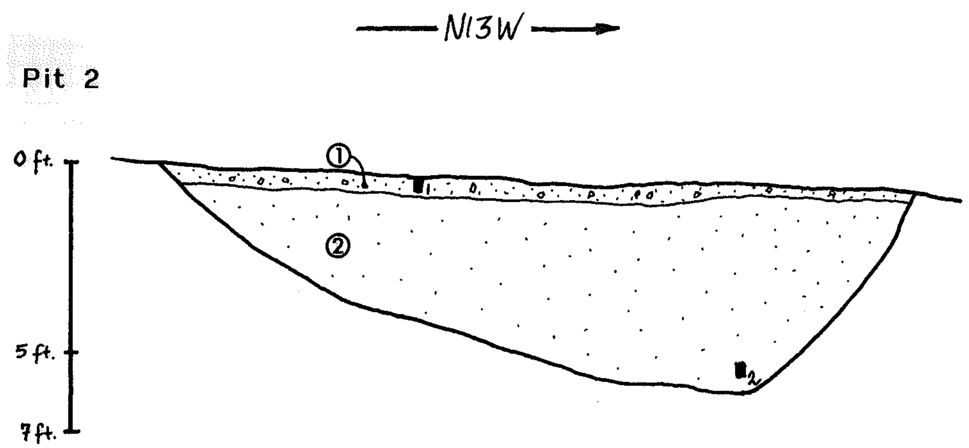
Pit 1



1. **SILT**; grayish brown; approximately 5% angular sandstone gravels; dry; scattered organics and rootlets (Colluvium)

2. **SANDSTONE**; brownish yellow; medium- to fine-grained sand; well indurated moderate blocky fractures; very dense; abundant oxidation and limonite staining; slightly moist; minor rootlets (Bedrock)

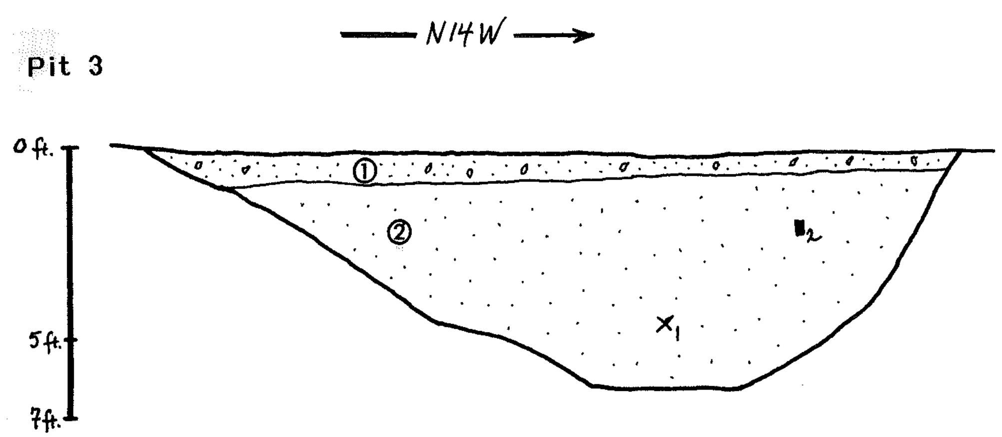
Pit 2



1. **SILT**; grayish brown; approximately 5% angular sandstone gravels; dry; scattered organics and rootlets (Colluvium)

2. **SANDSTONE**; brownish yellow; medium- to fine-grained sand; well indurated; moderate blocky fractures; very dense; abundant oxidation and limonite staining; slightly moist; minor rootlets (Bedrock)

Pit 3



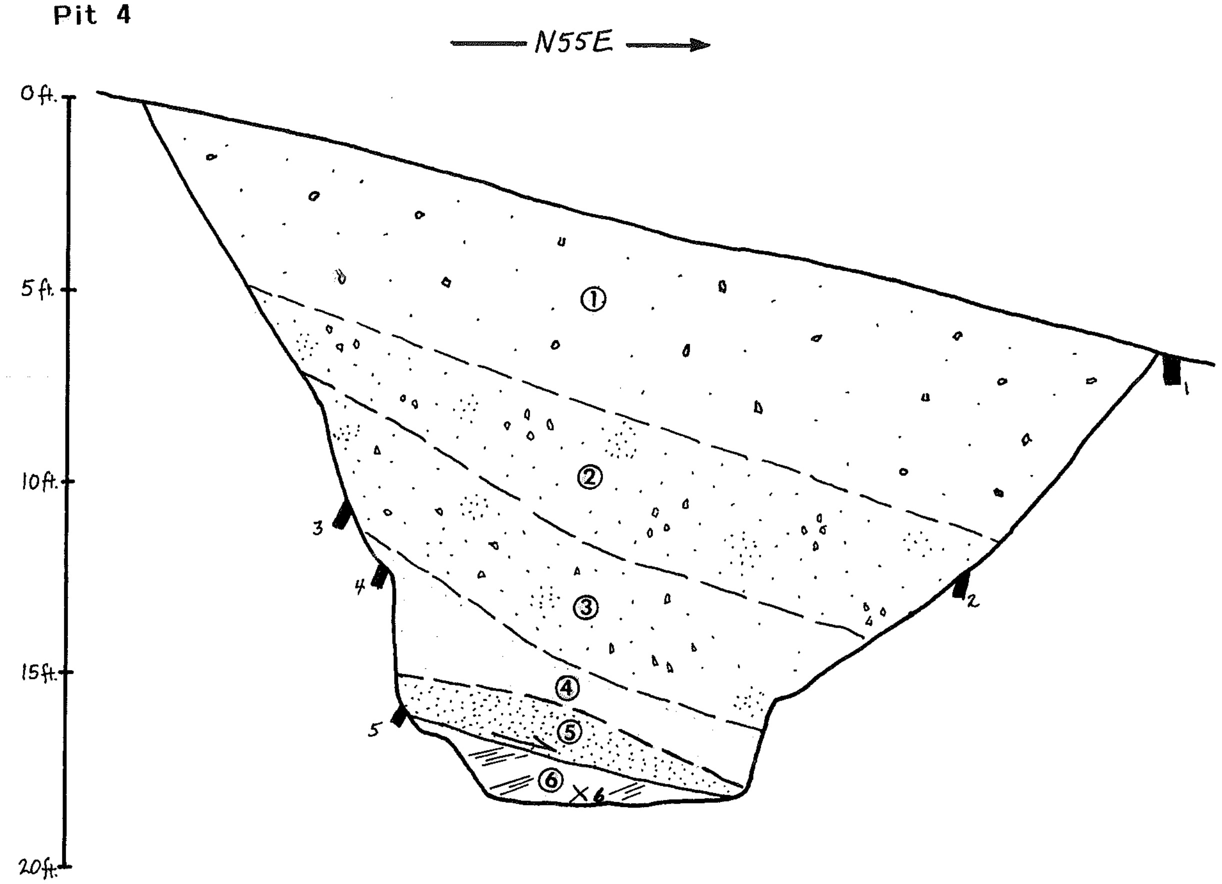
1. **SILT**; grayish brown; approximately 5% angular sandstone gravels; dry; scattered organics and rootlets (Colluvium)

2. **SANDSTONE**; brownish yellow; medium- to fine-grained sand; well indurated moderate blocky fractures; very dense; abundant oxidation and limonite staining; slightly moist; minor rootlets; highly weathered in upper 2 feet of bedrock (Bedrock)

LOGGED BY: M. Baumann and R. Urban; UPP GEOTECHNOLOGY, INC.; 10-29-98

LOG OF EXPLORATION PITS 1, 2, AND 3				
UPP GEOTECHNOLOGY Engineering Geology • Geotechnical Engineering		LANDS OF CAIN Santa Teresa Boulevard San Jose, California		
APPROVED BY	SCALE	PROJECT NO.	DATE	
MB	1" = 5'	1737.1R1	December 1998	Figure 6

Pit 4



1. **SILT**; grayish brown; approximately 5% angular sandstone gravels; dry; scattered organics and rootlets (Colluvium)

2. **SILTY CLAY**; dark brown; 5-10% angular chert and subangular sandstone fragments; slightly heterogeneous; stiff; moist (Older Colluvium)

3. **SILTY CLAY**; yellowish brown to gray; mottled; 10-20% angular chert and subangular sandstone fragments; heterogeneous; stiff; moist (Older Colluvium)

4. **SILT**; brown with faint gray mottling; 5-10% angular chert fragments; homogeneous; stiff; moist (Older Colluvium)

5. **SANDY GRAVEL**; yellowish brown, gray and brown (variegated); 50-70% sandstone and shale fragments in a clayey sand matrix; highly heterogeneous; medium dense; moist (Ancient Landslide Debris)

6. **SHALE**; dark olive gray; moderately fractured with brown to strong brown oxidation staining on fracture surfaces; very dense; dry (Bedrock)

NOTE: The contact between Units 5 and 6 is a 1-inch thick layer of light gray gravelly clay (Landslide Gouge)

SUMMARY OF LABORATORY TEST RESULTS

SAMPLE NUMBER	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	UNCONFINED SHEAR STRENGTH * (ksf)
1	½	9	---	---
2	7½	12	118	2.0
3	9½	18	111	6.1
4	11	15	113	---
5	15	16	111	1.9

LOGGED BY: M. Bauman and R. Urban; UPP GEOTECHNOLOGY, INC.; 10-29-98

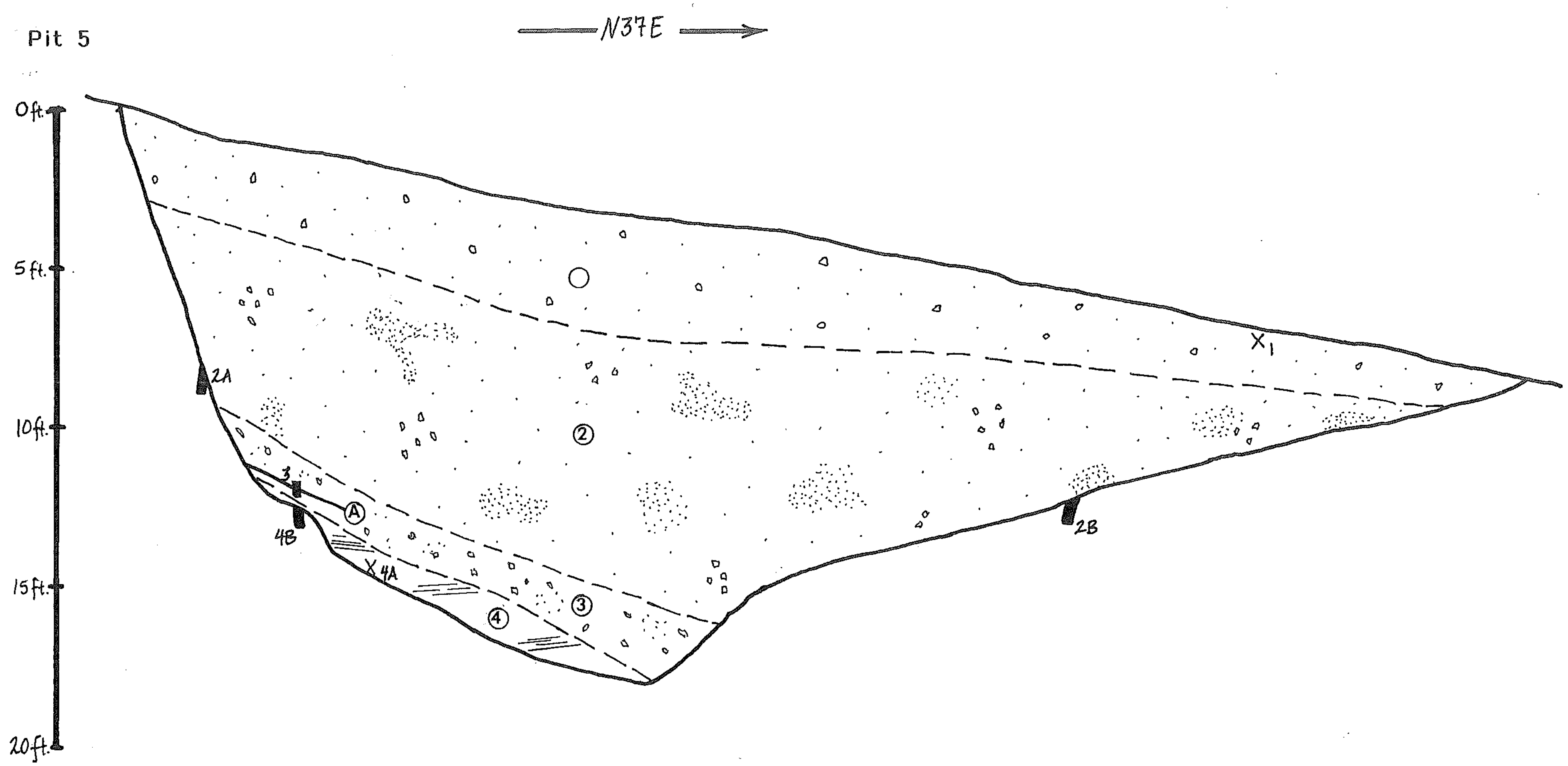
LOG OF EXPLORATION PIT 4

UPP GEOTECHNOLOGY
Engineering Geology • Geotechnical Engineering

LANDS OF CAIN
Santa Teresa Boulevard
San Jose, California

APPROVED BY	SCALE	PROJECT NO.	DATE	
MB	1" = 5'	1737.1R1	December 1998	Figure 7

Pit 5



1. **SILT**; grayish brown; approximately 5% angular sandstone gravels; dry; scattered organics and rootlets (Colluvium)

2. **SILT to SILTY CLAY**; dark yellowish brown; 10-20% angular chert and subrounded sandstone fragments; heterogeneous; stiff; moist (Older Colluvium)

3. **GRAVELLY CLAY**; dark brownish yellow; 20-40% angular shale and subangular sandstone fragments in a sandy clay matrix; heterogeneous; stiff; moist (Ancient Landslide Debris)

4. **SANDSTONE**; dark brownish yellow; fine- to medium-grained; moderately blocky fractures with oxidation staining on fracture surfaces; medium dense and moderately weathered in upper 6-12 inches, very dense and slightly weathered below; moist where moderately weathered, slightly moist below (Bedrock)

A. 1" thick discontinuous layer of gray CLAY (Landslide Gouge)

SUMMARY OF LABORATORY TEST RESULTS

SAMPLE NUMBER	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	UNCONFINED SHEAR STRENGTH * (ksf)
2A	7	15	115	2.5
2B	6½	14	115	---
3	10½	24	---	---
4B	11	11	126	---

LOG OF EXPLORATION PIT 5

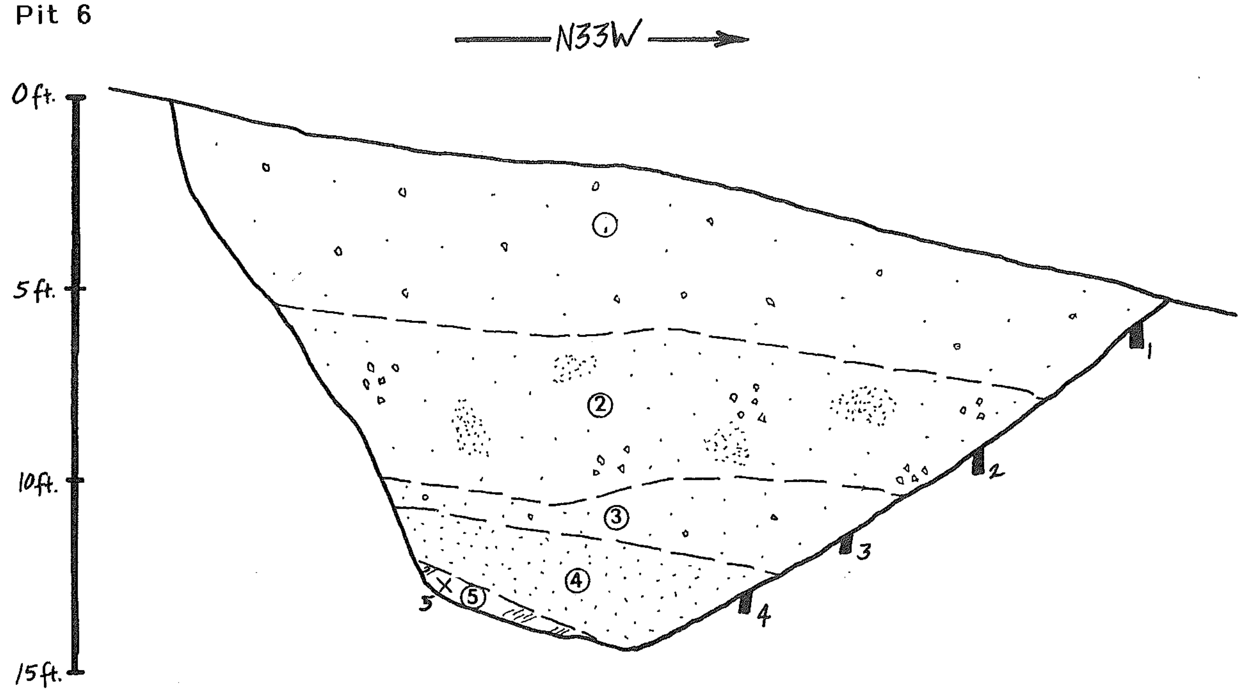
UPP GEOTECHNOLOGY
Engineering Geology • Geotechnical Engineering

LANDS OF CAIN
Santa Teresa Boulevard
San Jose, California

APPROVED BY	SCALE	PROJECT NO.	DATE	
MB	1" = 5'	1737.1R1	December 1998	Figure 8

LOGGED BY: M. Bauman and R. Urban; UPP GEOTECHNOLOGY, INC.; 10-29-98

Pit 6



1. **SILT**; grayish brown; approximately 5% angular sandstone gravels; dry; scattered organics and rootlets (Colluvium)

2. **SILT**; dark yellowish brown; 10-20% angular chert and subrounded sandstone fragments; heterogeneous; stiff; moist (Older Colluvium)

3. **CLAYEY SILT**; gray to dark yellowish brown; mottled; 20-30% fine-grained sand; 5-10% subrounded sandstone fragments; heterogeneous; stiff; moist (Older Colluvium)

4. **CLAYEY SAND**; gray to dark yellowish brown; mottled; 10-30% angular shale and subangular sandstone fragments; heterogeneous; stiff; moist (Old Colluvium)

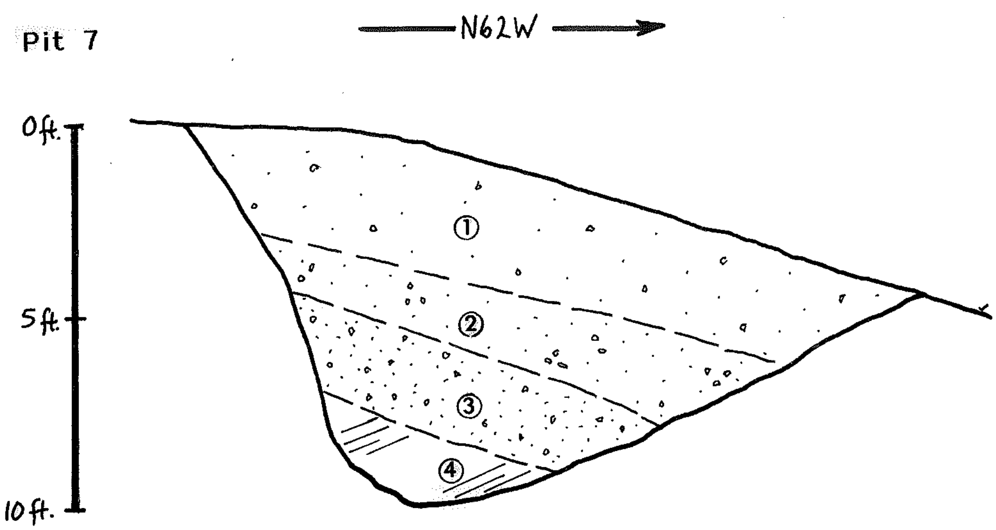
5. **SHALE**; dark olive gray; moderately fractured with brown to strong brown oxidation staining on fracture surfaces; very dense; dry (Bedrock)

SUMMARY OF LABORATORY TEST RESULTS

SAMPLE NUMBER	DEPTH (ft)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	UNCONFINED SHEAR STRENGTH * (ksf)
1	1	10	---	---
2	5	13	114	---
3	8	13	121	2.9
4	10½	12	126	2.7

LOGGED BY: M. Bauman and R. Urban; UPP GEOTECHNOLOGY, INC.; 10-29-98

LOG OF EXPLORATION PIT 6				
UPP GEOTECHNOLOGY Engineering Geology • Geotechnical Engineering		LANDS OF CAIN Santa Teresa Boulevard San Jose, California		
APPROVED BY	SCALE	PROJECT NO.	DATE	
MB	1" = 5'	1737.1R1	December 1998	Figure 9



1. **SILT**; grayish brown; <5% subangular sandstone fragments; homogeneous; stiff; dry; abundant soil pores and rootlets (Colluvium)

2. **SILT**; strong brown; 10-20% subrounded sandstone fragments; slightly heterogeneous; stiff; slightly moist (Older Colluvium)

3. **CLAYEY SAND**; dark brownish yellow; 20-30% angular sandstone fragments up to 12 inches; heterogeneous; medium dense; moist (Older Colluvium)

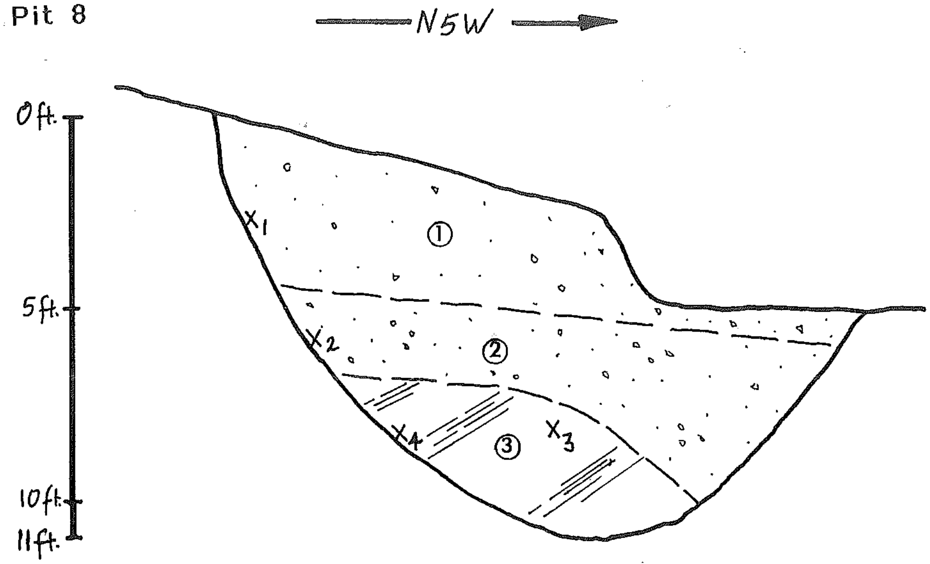
4. **SANDSTONE**; brownish yellow; fine- to medium-grained; well indurated with moderate blocky fractures; slightly moist (Bedrock)

LOGGED BY: M. Bauman and R. Urban; UPP GEOTECHNOLOGY, INC.; 10-29-98

LOG OF EXPLORATION PIT 7

UPP GEOTECHNOLOGY, INC. Engineering Geology • Geotechnical Engineering		LANDS OF CAIN Santa Teresa Boulevard San Jose, California		
APPROVED BY	SCALE	PROJECT NO.	DATE	
MB	1" = 5'	1737.1R1	December 1998	Figure 10

Pit 8



1. **SILT**; grayish brown; <5% subangular sandstone fragments; homogeneous; stiff; dry; abundant soil pores and rootlets (Colluvium)

2. **SILT**; strong brown; 10-20% subrounded sandstone fragments; slightly heterogeneous; stiff; slightly moist (Older Colluvium)

3. **SANDSTONE**; brownish yellow; fine- to medium-grained; highly weathered in upper 1½ feet; well indurated with moderate blocky fractures below; slightly moist (Bedrock)

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LOG OF EXPLORATION PIT 8				
UPP GEOTECHNOLOGY, INC. Engineering Geology • Geotechnical Engineering		LANDS OF CAIN Santa Teresa Boulevard San Jose, California		
APPROVED BY	SCALE	PROJECT NO.	DATE	
MB	1" = 5'	1737.1R1	December 1998	Figure 11

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