GEOTECHNICAL AND GEOLOGIC INVESTIGATION
FOR
1440 GROWTH CENTER
SCOTTS VALLEY, CALIFORNIA

FOR
1440 FOUNDATION
SARATOGA, CALIFORNIA

BY
PACIFIC CREST ENGINEERING INC.
CONSULTING GEOTECHNICAL ENGINEERS
1404-SZ34-E54
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April 23, 2014 Project No. 1404-SZ34-E54

Mr. Scott Kriens
1440 Foundation
P.O. Box 3141
Saratoga, CA 95070-1141

Subject: **Geotechnical and Geologic Investigation – Design Phase**
1440 Growth Center
Former Bethany College Campus
Scotts Valley, California

Dear Mr. Kriens,

In accordance with your authorization, we have performed a geotechnical and geologic investigation for the proposed retreat center located on the former Bethany College campus in Scotts Valley, California. Our study was completed in conjunction with Zinn Geology, our engineering geology subconsultant. Their report is included herein as Appendix B.

The accompanying report presents our preliminary conclusions and recommendations as well as the results of the geotechnical and geologic investigation on which they are based.

We appreciate the opportunity to be of service. If you have any questions concerning the data, conclusions or recommendations presented in this report, please call our office.

Very truly yours,

PACIFIC CREST ENGINEERING INC.

Cara Al-Lami
Staff Geologist

Elizabeth M. Mitchell, PE
Principal Geotechnical Engineer
G.E. 2718
Exp. 12/31/14

Copies: 1 to Client
4 to Mr. Jerry Yates, AIA

GEOTECHNICAL INVESTIGATION

PURPOSE AND SCOPE

This report describes our geotechnical and geologic investigation and presents results, including recommendations, for the proposed retreat center located on the former Bethany College campus in Scotts Valley, California. Our scope of services for this project has consisted of:

1. Discussions with the 1440 Foundation and the members of the design team, including project architect Gerald Yates, AIA.

2. Site reconnaissance by the project geotechnical engineer and project geologist, and reconnaissance-level geological mapping of the proposed development area(s).

3. Review of existing available geological and geotechnical engineering reports pertinent to the project site.

4. Examination and interpretation of stereo-pair vertical aerial photographs, to assess the past history of landsliding on the property.

5. Field exploration, consisting of 22 exploratory borings drilled to depths of 9½ to 51 feet below the ground surface.

6. Laboratory testing of selected samples obtained during drilling operations. Tests included density, moisture content, grain size distribution, expansion potential, and shear strength.

7. Construction of a geological map and geologic cross sections.

8. Engineering analysis and geologic review of data collected from our literature review, site mapping and field exploration program. This information was used to develop qualitative and quantitative geotechnical and geologic recommendations pertinent to the design and construction of the proposed development. Our analysis included quantitative slope stability analyses at selected building locations, development of lateral earth pressures and foundation design criteria, development of general earthwork, materials and utility trench recommendations, deterministic evaluation of seismic design parameters in accordance with the California Building Code and discussion of seismic and geologic hazards.

9. Preparation of this Geotechnical and Geologic Investigation Report. The report includes summaries of our compiled data with related references, factual data, descriptions of the geologic and seismic settings, and descriptions of prospective geologic hazards and attendant risks with respect to landsliding, surface fault ground rupture and seismic shaking. The report also provides geotechnical conclusions and recommendations pertinent to the design phase of the project. As discussed, the geologic portion of this report was prepared by our sub-consultant Zinn Geology and is presented in Appendix B.

LOCATION AND DESCRIPTION

The 1440 Foundation plans to renovate the former Bethany College campus to create a retreat facility and encourage academic growth. The campus is located at 800 Bethany Drive in the City of Scotts Valley and is situated at the northern edge of the city limits. Please refer to Figure No. 1, Regional Site Map, for the general vicinity of the project site. Furthermore, the site is approximately located at the following coordinates:

Latitude = 37.075335 degrees
Longitude = -121.994515 degrees

At present the property is occupied by older buildings and residences scattered across the project area, as well as a partially completed cafeteria building. The proposed redevelopment plan includes the construction of new buildings and lodges, roadway and parking improvements, and renovations to several existing buildings.

The Master Plan for the proposed retreat center separates the project into two phases. Phase I includes the construction of three new buildings and the renovation of seven existing buildings. The construction of new lodging facilities and a parking structure are planned for Phase II. Both phases include the demolition of fifteen existing buildings and associated infrastructure.

The three proposed building sites for Phase I will comprise a spa facility, dining hall and a four-story lodging structure located along the southern portion of the property. The proposed development areas are situated adjacent to the top of a steep drainage that descends sharply to the east toward Carbonera Creek. The Phase II scope is still evolving at the writing of this report, however preliminary plans indicate a new multi-story parking structure at the former soccer field, a village of four-plexes at Gaston Circle, and new program space and lodging at the site of the former Swanson Hall.

We presume the new structures will be steel and/or wood frame and masonry construction, combined with some concrete slab-on-grade. Exact wall and column loads are not known but are expected to be typical of multi-story construction. The extent of renovations planned for the buildings to be remodeled is unknown at this time. The project will also require new roadway and parking improvements, retaining walls, utilities, and landscape improvements. Based on the existing site topography in conjunction with the proposed Master Plan, minor to moderate grading is expected.

FIELD INVESTIGATION

Soil Borings

Twenty-two (22) 4 to 6-inch diameter test borings were drilled on the site between February 6th, 2014 and March 20th, 2014. The borings were advanced by means of a hydraulically operated drill using continuous flight, solid-stem augers. Due to the variable site terrain three different drill rigs were used; a truck-mounted rig using 6-inch diameter augers, a John Deere tractor-mounted rig using 6-inch diameter augers, and a limited access “Little Beaver” rig using 4-inch diameter augers. These borings ranged in depth from 9½ to 51 feet, and were advanced within

areas accessible to our drilling equipment. The presence of trees, steep terrain, and existing buildings limited the placement of borings within the proposed Phase I buildings.

The location of the test borings are shown on Figure No. 2 of Appendix A, Site Map Showing Location of Test Borings. A geologist from Pacific Crest Engineering Inc. was present during the drilling operations to log the soil encountered and to choose soil sampling type and locations.

Relatively undisturbed soil samples were obtained at various depths by driving a split spoon sampler 18 inches into the ground. This was achieved by dropping a 140 pound down hole safety hammer through a vertical height of 30 inches. The number of blows needed to drive the sampler for each 6 inch portion is recorded and the total number of blows needed to drive the last 12 inches is reported as the Standard Penetration Test (SPT) value. The outside diameter of the samplers used in this investigation was either 3 inches or 2 inches, and is noted respectively as “L” or “T” on the boring logs.

All standard penetration test data has been normalized to a 2 inch O.D. sampler so as to reflect a SPT "N" value. The normalization method used was derived from the second edition of the Foundation Engineering Handbook (H.Y. Fang, 1991). The method utilizes a Sampler Hammer Ratio which is noted as either R_s for non-cohesive soils, or R_c for cohesive soils. This ratio is dependent on the weight of the hammer, height of hammer drop, outside diameter of sampler, and inside diameter of sample. Using the Sampler Hammer Ratio, the correlation can be made from the samplers used in the field to the standard SPT “N” Value. Based on this method, the average correction factor is 0.52 for non-cohesive soils, and 0.65 for cohesive soils.

The soils encountered in the borings were continuously logged in the field and visually described in accordance with the Unified Soil Classification System (ASTM D2488 (Modified), Figure No. 3). The soil classification was verified and or modified upon completion of laboratory testing in accordance with ASTM D2487.

Appendix A contains the site plan showing the locations of the test borings and the Log of Test Borings presenting the soil profile explored in each boring, the sample locations, and the SPT "N" values for each sample. Stratification lines on the boring logs are approximate as the actual transition between soil types may be gradual.

LABORATORY INVESTIGATION

The laboratory testing program was developed to help in evaluating the engineering properties of the materials encountered on the site. Laboratory tests performed include:

a. Moisture Density relationships in accordance with ASTM test D2937.

b. Direct Shear tests in accordance with ASTM test D3080.

c. Unconfined Compression tests in accordance with ASTM test D2166.

d. Atterberg Limits tests in accordance with ASTM test D4318.

e. Gradation tests in accordance with ASTM test D1140.

The results of the laboratory tests are presented on the boring logs opposite the sample tested and/or presented graphically within Appendix A.

SOIL CONDITIONS

Soil Borings

Our field exploration program consisted of a total of 22 test borings advanced to depths ranging between 9½ feet and 51 feet below ground surface. The boring locations were based upon preliminary conceptual site plans and were selected in order to characterize the underlying soil conditions across the proposed building envelopes. The following describes, in general terms, the subsurface conditions within the areas explored. The Log of Test Borings in Appendix A provides, in more descriptive terms, the soil and bedrock profile encountered within each test boring.

Building Area P1-1 Spa Building

Borings B-19, B-20, B-21, and B-22 were advanced within the proposed spa buildings. This site is blanketed by approximately 4½ to 9½ feet of soft to firm colluvium soils that ranged from highly plastic clays and silts to very fine grained, poorly graded silty sand. Rounded and weathered siltstone pebbles up to ¼ inch in diameter were typically observed within the relatively loose to firm near surface soils. Fill was encountered within the upper 2 to 2½ feet of B-20 and B-21. Bedrock was encountered between 4½ feet and 9½ feet at this site and was described as siltstone and sandstone with a varying amount of fines. In general, the very hard to very dense bedrock was thinly bedded, fissile, and friable.

Building Area P1-2 Dining Hall

Borings B-1, B-5, B-6, B-7, B-8, and B-12 were drilled in the general vicinity of the proposed dining hall. This site is underlain by a blanket of very loose to firm colluvium soils that ranged from 1 to 25 feet thick. The stratigraphy of the near-surface colluviums soils is varied and mostly described as intermediate to highly plastic silts and clays with occasional beds of very fine grained sands. Granitic, siltstone, sandstone and chert pebbles up to 1-inch in diameter were noted within the soil blanket near the top of the slope (B-8 and B-12). The surficial soils were underlain by siltstone to sandstone bedrock described as massive, friable, and fractured. The consistency/density of the bedrock was noted as very hard/very dense.

Building Area P1-3 Lodging

Borings B-2, B-3, and B-9 were advanced within the proposed lodging building. This site is underlain by artificial fill and/or colluvium overlying mudstone or sandstone bedrock. The near surface soils overlying the bedrock ranged from 3½ to 25 feet in thickness and were generally described as low to intermediate plastic clays or fine to medium grained silty sand with consistencies and densities noted as stiff to medium dense. The fill extended to depths of 4½ feet and 25 feet within B-2 and B-3, respectively. Mudstone clasts and chert pebbles up to 1 inch

in diameter were typically encountered within the soil deposits. In general, the very hard mudstone and very dense sandstone bedrock in this area was described as massive, friable, and quartz rich (sandstone) with a trace amount of oxidation.

Building Area P2-1 Program Space and Lodging

Borings B-4, B-10, and B-11 were drilled in the general vicinity of the proposed program space and lodging building. This site is underlain by a 1 to 6½ foot thick blanket of artificial fill and/or colluvium soils. The thin veneer of very fine grained silty sand encountered within B-4 and B-10 are likely a result of weathered bedrock. Approximately 6 feet of very fine grained, silty sand fill was encountered within B-11. The near surface soils were generally medium dense to dense and was underlain by mudstone, siltstone, and sandstone bedrock that were generally described as massive, fractured (siltstone and mudstone), friable and quartz rich (sandstone).

Building Area P2-2 Parking Structure

Borings B-17 and B-18 were advanced within the existing soccer field where a new parking structure is proposed. This site is comprised of an infilled drainage underlain by mudstone and sandstone bedrock. The soil blanket thins near the northern end and thickens to the south where the combined thickness of artificial fill and colluvium could reach upwards of 50 feet. Fill was encountered in the upper 25 and 14½ feet within B-17 and B-18, respectively. The fill within B-18 was underlain by approximately 14½ feet of colluvium before encountering bedrock at a depth of 25 feet. The fill and colluvium soils were generally described as low to intermediate plastic clays and very fine to fine grained clayey sands with near surface consistencies and densities noted as firm to stiff and loose to medium dense. Granitic, quartz, sandstone, and mudstone gravels up to ½ inch in diameter were commonly observed within collected samples. Sandstone or mudstone bedrock was encountered in our borings at a depth of approximately 25 feet below ground surface. In general, the very hard mudstone and very dense sandstone was described as massive, fractured (mudstone), friable and quartz rich (sandstone).

Building Area P2-3 Open Parking/Four-Plex Lodging

Borings B-13, B-14, B-15, and B-16 were advanced within the proposed Phase II Gaston Village lodging area. This site also consists of an infilled drainage underlain by mudstone or siltstone bedrock at depth. A blanket of artificial fill and/or colluvium ranging in depth from approximately 10 to 25 feet overlies the site. We encountered approximately 5 feet of fill over about 25 feet of colluvium in B-13, and 5 feet of fill over about 5 feet of colluvium in B-15. B-14 encountered a thin soil veneer and B-16 encountered approximately 26 feet of colluvium prior to encountering bedrock. The fill and colluvium was described as low to intermediate plastic clays and very fine to fine grained clayey sands that were stiff to loose near the surface and became very stiff to medium dense with depth. Sandstone and claystone gravels up to ¼ inch in diameter were commonly observed within collected soil samples. The depth to bedrock varied, ranging between 1 and 31 feet below ground surface. In general, the very hard mudstone, very hard siltstone, and very dense sandstone was described as massive, weathered, fractured (mudstone and siltstone), friable and quartz rich (sandstone).

Groundwater

Groundwater was encountered within five of the test borings at depths ranging between 5 and 47 feet below ground surface. The table below displays the completed depth of the borings that encountered groundwater as well as the depth in which groundwater was encountered:

Boring Number	Completed Boring Depth	Depth to Groundwater
B-2	35½ feet	35½ feet
B-13	37 feet	36 feet
B-16	36 feet	34½ feet
B-17	51 feet	47 feet
B-21	9½ feet	5 feet

It should be noted that the groundwater level was not allowed to stabilize for more than a few hours; therefore, the actual groundwater level may be higher or lower than initially encountered. Groundwater was typically observed perched upon contacts between colluvial deposits and bedrock or the contact between varying bedrock formations. A contrast in permeability between the overlying deposits and bedrock or between bedrock types allows for perched groundwater conditions to develop. Groundwater levels may also fluctuate seasonally as a result of precipitation which varies year to year.

It should also be noted that the groundwater encountered in B-21 may be a result of leaking on-site domestic lines. Considering the current drought conditions coupled with the relatively flat topography within our boring locations, if a local groundwater table was present we would expect to also encounter groundwater in B-19, B-20 or B-22. As indicated on our boring logs, these other borings were dry and groundwater was not observed seeping from the slope face during geologic mapping activities performed by the project geologist.

REGIONAL SEISMIC SETTING

Please refer to the Geology Report prepared by Zinn Geology in Appendix B for detailed discussion on local faulting in the general vicinity of the project site. The seismic setting of the site is one in which it is reasonable to assume that the site will experience significant seismic shaking during the lifetime of the project.

GEOLOGIC HAZARDS

Please refer to the Zinn Geology report in Appendix B for a detailed discussion on geologic hazards affecting this site, which include strong seismic shaking, landsliding, soil creep, and erosion. Geotechnical aspects of these issues are discussed below:

Ground Shaking

Ground shaking will be felt on the site during strong seismic events. Structures founded on thick soft soil deposits are more likely to experience more destructive shaking, with higher amplitude and lower frequency, than structures founded on bedrock. Generally, shaking will be more intense closer to earthquake epicenters. Thick soft soil deposits large distances from earthquake epicenters, however, may result in seismic accelerations significantly greater than expected in

bedrock. Structures built in accordance with the latest edition of the California Building Code have an increased potential for experiencing relatively minor damage which should be repairable. The seismic design of the project should be based on the 2013 California Building Code (CBC) as it has incorporated the most recent seismic design parameters.

TABLE No. 1, The 2013 CBC Seismic Design Parameters

Design Parameter	**Specific to Site ASCE 7-10**	**Reference (See Note 1)**
Site Class	D, Stiff Soil	Chapter 20 ASCE 7-10
Mapped Spectral Acceleration for Short Periods (**See Note 2**)	Ss = 2.088 g	Fig. 22-1 ASCE 7-10
Mapped Spectral Acceleration for 1-second Period	S_1 = 0.813 g	Fig. 22-2 ASCE 7-10
Short Period Site Coefficient	Fa = 1.0	Table 1613A.3.3(1)
1-Second Period Site Coefficient	Fv = 1.5	Table 1613A.3.3(2)
MCE Spectral Response Acceleration for Short Period	S_{MS} = 2.088 g	Equation 16A-37 CBC
MCE Spectral Response Acceleration for 1-Second Period	S_{M1} = 1.220 g	Equation 16A-38 CBC
5% Damped Spectral Response Acceleration for Short Period	S_{DS} = 1.392 g	Equation 16A-39 CBC
5% Damped Spectral Response Acceleration for 1-Second Period	S_{D1} = 0.813 g	Equation 16A-40 CBC
Seismic Design Category (**Notes 3 and 4)**	E	Section 1613A.3.5

Note 1: Design values may also have been obtained by using the Ground Motion Parameter Calculator available on the USGS website at http://geohazards.usgs.gov/designmaps/us/application.php

Note 2: Per Section 12.8.1.3 of ASCE 7-10 the S_s value can be reduced to 1.5 for the purposes of calculating C_S for regular structures five stories or less in height and having a period T of 0.5 seconds or less.

Note 3: Seismic Design Category assumes the structure is a Category II occupancy as defined by Table 1604A.5 of the 2013 CBC. Pacific Crest Engineering Inc. should be contacted for revised Table 1 seismic design parameters if the building has a different occupancy rating from the one assumed.

Note 4: Based on Section 1613A.3.5 of the 2013 CBC, the S_1 value exceeds 0.75g. Therefore, the appropriate Seismic Design Category is E rather than D assuming the buildings are Category II structures.

The recommendations of this report are intended to reduce the potential for structural damage to an acceptable risk level, however strong seismic shaking could result in architectural damage and the need for post-earthquake repairs. It should be assumed that exterior improvements such as pavements, slabs, sidewalks or patios will need to be repaired or replaced following strong seismic shaking. An increased depth of subgrade compaction below exterior improvements will assist in minimizing the damage to these elements.

Ground Surface Fault Rupture

Ground surface fault rupture occurs along the surficial trace(s) of active faults during significant seismic events. Pacific Crest Engineering Inc. has not performed a specific investigation for the presence of active faults on the project site. Since the nearest known active or potentially active fault is mapped approximately 1.4 miles (approximately 2.3km) from the site (Brabb, 1989 and CDMG, 1998), the potential for ground surface fault rupture at this site is low.

Liquefaction

Liquefaction tends to occur in loose, saturated fine grained sands, coarse silts or clays with a low plasticity. Based upon our review of the regional liquefaction maps (Dupre', 1975) the proposed development area is mapped as having a low potential for liquefaction. The soils encountered

within our test borings were generally colluvium, fill, or weathered bedrock deposits underlain by more competent bedrock at depth. Any loose sands encountered within our test borings were restricted to the upper 5 feet and typically contained varying amounts of fine grained soils. The consistency/density of the soils was described as firm/loose to very stiff/medium dense near the surface and became very hard to very dense with depth. When encountered, the groundwater table was generally 35 feet below ground surface and was perched upon bedrock or at the contact between varying bedrock formations. The exception was B-21, in which groundwater was encountered at a depth of approximately 5 feet below ground surface. This boring was located at the base of a slope and the groundwater table was perched upon the bedrock contact. Groundwater was not encountered within other test borings advanced in the vicinity of B-21.

Given the lack of a shallow groundwater table in conjunction with the lack of loose sandy material and the presence of weathered and competent bedrock at relatively shallow depths, it is our opinion that liquefaction has a low potential of occurring at this site and causing damage to the structures thereon.

Liquefaction Induced Lateral Spreading

Liquefaction induced lateral spreading occurs when a liquefied soil mass fails toward an open slope face, or fails on an inclined topographic slope. Our analysis of the project site indicates that the potential for liquefaction to occur is low, and consequently the potential for lateral spreading is also low.

Landsliding

The Zinn Geology report (Plate 1) shows several identified landslide or debris flow failures on the ridges above the campus and on the slopes above Carbonera Creek. Additional failures may be present in these areas but obscured by dense vegetation. The Canham Road debris flow in 1982 originated at the neighboring property to the northeast, but flowed across a portion of the campus in the area of Bethany Loop, eventually reaching Carbonera Creek.

As discussed in the Zinn Geology report, the steep slopes above Carbonera Creek are subject to localized soil creep, erosion or debris-flow type landslides that can mobilize several feet of soil per event. These processes will continue over the long term until a more stable angle of repose is achieved. We estimate this angle to be on the order of 25 to 35 degrees from horizontal, which has led to the development of the suggested failure zones as shown on the Zinn Geology cross sections. These failure modes have been calculated on the basis of the project geologist's understanding and experience with slope failures along this area of Santa Cruz County. Such failures inherently include all the geological processes (erosion, landsliding, co-seismic failures, etc.) which could conceivably contribute to slope retreat over the long term.

In our opinion the slopes above Gaston Village and the existing soccer field are also subject to similar instability hazards, which could affect the location and engineering design of structures for Phase II. We anticipate that additional work as part of Phase II studies will be required in order to adequately characterize this hazard and develop supplemental mitigation recommendations for proposed Phase II development areas.

A quantitative evaluation of global (deep-seated) stability of slopes below proposed development areas for Phase I was performed for each of the four geologic cross sections developed by Zinn Geology. The cross sections were modeled using field and laboratory data obtained from our test borings. The analysis was conducted in general accordance with the procedures outlined in the ASCE document "*Recommended Procedures for Implementation of DMG Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California*". The cross section locations are depicted on Figure No. 2 in Appendix A as well as Plate 2 within Appendix B.

The depth and thickness of the subsurface strata delineated on the cross sections was generalized and interpolated from field observations, test boring locations, and laboratory test results. The transition between materials may be more or less gradual than indicated. Both static and pseudo-static (seismic) conditions were evaluated, utilizing the computer software GSTABL7 by Gregory Geotechnical.

The model included a potential for perched groundwater conditions by assuming saturated soil conditions (R_u = 0.5) within the surficial colluvium or fill materials overlying the bedrock, and partially saturated conditions (R_u = .10) within the bedrock strata.

In an effort to identify potential failure mechanisms, the program was allowed to search for critical failure surfaces with the lowest factors of safety, assuming circular failures and the Modified Bishop Method of Slices. The analysis included a seismic coefficient to account for a design earthquake, in order to model the slope under seismic conditions. This coefficient was determined as explained below.

Screening Analysis and Seismic Coefficient

Horizontal forces generated by a design seismic event are typically modeled by applying a seismic coefficient value to the analysis, in order to develop a "pseudo-static" condition intended to represent earthquake effects on the slope model.

As outlined in the *Recommended Procedures*, the seismic coefficient is used to perform a screening analysis of seismic slope stability using a threshold displacement value (usually 5 cm). The screening analysis is used to evaluate the potential for significant seismic slope deformation during a design seismic event.

In the screening analysis, the seismic coefficient is applied to a conventional slope stability calculation to determine a minimum factor of safety. If the safety factor exceeds 1, the site passes the screen and no further analysis is required. If the safety factor is less than 1, the site does not pass the screen and a quantitative slope deformation analysis with evaluation of expected displacements should be performed.

A site-specific seismic coefficient was developed for this project using procedures outlined in *DMG Special Publication 117*. The basis for development of the seismic coefficient is the maximum horizontal bedrock acceleration (MHA_r) expected to occur at the site during the design lifetime of the project, multiplied by a factor (f_{eq}) related to the seismicity of the site. The MHA_r was deterministically evaluated for this site as a peak ground acceleration value (PGA) of

0.525g. The PGA value was determined using the California Geological Survey PSHA Ground Motion Interpolator (2008) with a 10% chance of exceedance in 50 years.

The seismic factor (f_{eq}) is a magnitude and distance-dependent, median value based on a Newmark-type deformation analysis. It is calculated using a model developed by Bray and Rathje (1998) and assumed threshold displacements designated by the user. As outlined in *DMG Special Publication 117,* threshold displacements of 0 to 15 cm correspond with a low likelihood of "serious landslide movement and damage".

For this project, the following seismic coefficients were calculated using the Bray and Rathje model and an estimated peak ground acceleration of 0.525g:

Threshold Displacement	f_{eq}	Seismic Coefficient
5 cm	0.607	0.319

Soil Strength Properties

Soil stratigraphy in the Scotts Valley area can be highly variable within similar soil deposits, with resulting variations in laboratory-derived soil strength parameters. Engineering judgment therefore often becomes necessary when assigning a singular, homogeneous soil strength to a variable heterogeneous soil deposit.

The inherent strength of soil in shear can depend on drainage conditions, total and/or effective stress on the soil, and strain rate. The soil strength can be selected at peak, ultimate, or residual states in the shearing process, depending on the conditions being modeled.

A drained condition exists when excess pore pressures are not present and volume change has been allowed to occur during loading, allowing pore pressure to dissipate. The shear strength under drained conditions can be described with effective stress parameters generated from consolidated, drained (CD) tests (i.e., direct shear or triaxial and ring shear testing with pore pressure measurements).

Undrained conditions assume no change in volume during loading, with a corresponding increase or decrease in pore pressure depending upon the contractive or dilative nature of the soil. Shear strengths are typically determined from total stress parameters obtained from consolidation undrained (CU) tests (i.e., triaxial, ring shear or unconfined compression testing.

Peak soil strength is the maximum shear strength achieved during testing, and is typically expressed in terms of cohesion and friction angle (i.e., c and phi). Residual soil strength is the lowest strength obtainable and represents a nearly constant value of shear strength following a large amount of deformation. Ultimate shear strength is a "post-peak" state of additional deformation as the shear strength begins its approach toward a residual condition.

SP 117 and the ASCE publication suggest that peak drained strengths can be used for granular, non-plastic soils, or unweathered crystalline bedrock materials. Residual strengths should be used for materials that have experienced previous deformation (such as landslide slip surfaces).

Values between peak and ultimate should be used for stiff clayey soils and clayey bedrock with no prior shear deformation.

The following table summarizes the strength values obtained from our laboratory testing program:

TABLE No. 2 – Summary of Shear Strength Test Results

Artificial Fill/Colluvium Deposits							
				Peak		Residual	
Boring	Depth, ft	UCSC	SPT Blows/ft	Cohesion, psf	Friction Angle, Degrees	Cohesion, psf	Friction Angle, Degrees
B-3	2	SM	27	20	39	10	38
B-4	2	ML	31	290	37	260	36
B-12	6	ML	17	0	44	0	43
B-16	6	SC	11	670	34	660	31
B-19	2	SM	12	660	34	680	31
		Average	20	328	38	322	36

Mudstone, Siltstone, and Sandstone Bedrock							
				Peak		Residual	
Boring	Depth, ft	UCSC	SPT Blows/ft	Cohesion, psf	Friction Angle, Degrees	Cohesion, psf	Friction Angle, Degrees
B-3	6	ML	100	380	53	110	51
B-3	26	ML	29	920	46	740	43
B-6	16	ML	51	160	56	0	54
B-6	26	SM	21	990	47	950	45
		Average	50	613	51	450	48

With consideration of the issues discussed above and based upon our laboratory testing results, strength values were assigned to the dominant soil types as follows:

Soil Type	Cohesion, psf	Friction Angle, Degrees
Artificial Fill/Colluvium Deposits	0	25
Mudstone, Siltstone, Sandstone Bedrock	550	45

Based on laboratory testing, field penetration tests, field observations and our experience with similar soil conditions, we believe our model represents a reasonably conservative estimate of in-situ soil properties the comprise the slopes above Carbonera Creek.

Appendix C, Slope Stability Calculations, presents the cross sections analyzed, the critical failure surfaces with their respective factors of safety, and the computer printouts.

Slope Stability Analysis Results

The following table summarizes the safety factors obtained from our analysis:

	Global Stability	
	Static Safety Factor	Pseudo-Static Safety Factor
Cross Section V-V'	2.0	1.2
Cross Section X-X'	1.9	1.2
Cross Section Y-Y'	2.0	1.2
Cross Section Z-Z'	1.6	1.2

The results of our slope stability analysis indicate computed safety factors against deep-seated landsliding on the order of 1.6 or greater under *static* conditions. Under *pseudo-static (seismic)* conditions, the computed minimum factor of safety using the screening analysis procedure outlined in *Stewart et al. 2003* is 1.2. This value exceeds the minimum recommended value of 1.0 and indicates that, on a global level, the slopes are unlikely to experience more than small ground movements (on the order of 5 cm or less) during the design seismic event. The calculated safety factors also meet or exceed generally accepted minimum safety factors of 1.5 and 1.2 for static and pseudo-static conditions, respectively.

Although considered stable against global, deep-seated landsliding, the slopes are considered subject to shallow, localized failures resulting from debris flows, erosion and/or soil creep. The effects of soil movement must be incorporated into the design and siting of structures located with the influence of these events.

Slope failures can also occur where surface drainage is allowed to concentrate onto unprotected slopes. Appropriate landscaping and good control of surface drainage around the project area becomes very important to minimize the potential for slumping within the colluvium deposits. Under no circumstances should surface runoff from the project be allowed to discharge upon, or at the top of, any topographic slopes.

It must be cautioned that slope stability analysis is an inexact science and the mathematical models of the slopes and soils contain many simplifying assumptions, not the least of which is isotropy and homogeneity. Engineering judgment is often necessary when assigning a singular homogeneous soil strength to a variable heterogeneous soil deposit. Density, moisture content and shear strength may vary within a soil type. There may be localized areas of loose, cohesionless sands or perched ground water within a soil. An analysis of ground water hydrology for this site was outside the scope of our study. Therefore actual ground water conditions which differ from those assumed in our analysis could result in a lower factor of safety. Slope stability analyses and the generated factors of safety should be used as indicating trend lines. A slope with a safety factor less than one will not necessarily fail, but the probability of slope movement will be greater than a slope with a higher safety factor. Conversely, a slope with a safety factor greater than one may fail, but the probability of stability is higher than a slope with a lower safety factor.

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

GENERAL

1. In general we find the proposed development to be feasible from a geotechnical engineering standpoint, provided our recommendations as well as those of the project geologist are included in the design and construction of the project.

2. Our field observations and laboratory testing indicates that the near surface soils possess low to high expansive properties. This analysis was based on #200 sieve analyses and Atterberg Limits tests.

3. Grading and foundation plans should be reviewed by Pacific Crest Engineering Inc. during their preparation and prior to contract bidding.

4. Pacific Crest Engineering Inc. should be notified at least four (4) working days prior to any site clearing and grading operations on the property in order to observe the stripping and disposal of unsuitable materials, and to coordinate this work with the grading contractor. During this period, a pre-construction conference should be held on the site, with at least you or your representative, the grading contractor, a City representative and one of our engineers present. At this meeting, the project specifications and the testing and inspection responsibilities will be outlined and discussed.

5. Field observation and testing must be provided by a representative of Pacific Crest Engineering Inc., to enable them to form an opinion as to the degree of conformance of the exposed site conditions to those foreseen in this report, the adequacy of the site preparation, the acceptability of fill materials, and the extent to which the earthwork construction and the degree of compaction comply with the specification requirements. **Any work related to grading or foundation excavation that is performed without the full knowledge and direct observation of Pacific Crest Engineering Inc., the Geotechnical Engineer of Record, will render the recommendations of this report invalid, unless the Client hires a new Geotechnical Engineer who agrees to take over complete responsibility for this report's findings, conclusions and recommendations.** The new Geotechnical Engineer must agree to prepare a Transfer of Responsibility letter. This may require additional test borings and laboratory analysis if the new Geotechnical Engineer does not completely agree with our prior findings, conclusions and recommendations.

PRIMARY GEOTECHNICAL CONSIDERATIONS

6. Based upon the results of our investigation, primary geotechnical considerations for the project include uniform and appropriate foundation support for new structures, unknown soil conditions beneath existing buildings to be replaced or remodeled, slope instability, strong seismic shaking, and surface drainage.

7. Variable and/or disturbed soil conditions across the proposed building sites are expected to occur and should be anticipated, especially following demolition operations. Soil conditions

beneath existing buildings could not be explored and are therefore unknown. Our office should observe the exposed soil conditions following demolition operations in order to provide additional recommendations, if necessary.

8. At this time, little is known about the existing foundations of the buildings planned to be remodeled or the extent to which the buildings will be remodeled. Additional recommendations may be necessary pending the completion of the final project design.

9. In order to develop uniform foundation support amid a wide disparity of soil properties and topographic challenges, we are recommending drilled pier foundations for the proposed new Phase I structures. We anticipate that Phase II structures may also be supported by drilled piers, although structural mat foundations bearing upon a zone of redensified soil may be considered in flat-lying areas underlain by colluvium and/or artificial fill. Where competent bedrock is exposed or relatively shallow, conventional foundations may be considered provided all footings bear uniformly and consistently upon the bedrock.

10. The slopes below the proposed Phase I buildings are generally comprised of a blanket of colluvium and/or fill, overlying sandstone and/or siltstone bedrock. The results of our investigation indicate that the near surface soils above the bedrock contact are subject to long term retreat due to erosion and/or landsliding. Structural elements within the influence of the projected failure zones should be designed to accommodate continued slope retreat or be set back behind the proposed failure surfaces as depicted by the project geologist in Plate 2 of Appendix B.

11. Additional geologic and geotechnical concerns for the Phase II Gaston Village area and the proposed parking structure at the existing soccer field include stability of the slopes flanking the building footprint(s) as well as the potential for excessive settlements due to loose colluvium and/or deep fill soils.

12. Supplemental geologic and geotechnical study should be performed for Phase II once a development plan has been formalized. We anticipate the supplemental work to likely include the excavation of exploratory test pits in conjunction with field mapping of the slopes above the proposed building sites. The landslides identified on these slopes should be investigated and stabilized where necessary to protect habitable structures down slope. Both Phase I and Phase II development plans are expected to require debris basins, impact walls or other slope stabilization measures to protect habitable areas below unstable slopes.

13. The project site is located within a seismically active area and strong seismic shaking is expected to occur within the design lifetime of the project. Improvements should be designed and constructed in accordance with the most current CBC and the recommendations of this report to minimize reaction to seismic shaking. Structures built in accordance with the latest edition of the California Building Code have an increased potential for experiencing relatively minor damage, which should be repairable, however strong seismic shaking could result in architectural damage and the need for post-earthquake repairs.

14. Scattered construction debris, abandoned drainage pipes, and man-made fill soils were observed by the project geologist along some of the steep slopes that descend into Carbonera Creek. These disturbed surfaces are susceptible to erosion, slumping, and debris flows, especially if storm water is not properly controlled and disposed of away from the slopes.

15. Surface runoff should be strictly controlled around the entire development and discharged in a controlled and reasonable manner to suitable outlets. Under no circumstances should runoff be discharged upon, or at the top of, any topographic slopes. An engineered drainage plan is recommended for this project.

16. It is our understanding that the design of proposed buildings will continue to change after the issuance of our report. As the Master Plan for Phase I and Phase II design elements continue to evolve, supplemental recommendations and/or field work may be required.

SITE PREPARATION

17. The initial preparation of the site for the Phase I work scope will consist of the removal of the existing buildings, landscaping and vegetation, and remnant flatwork. ***All foundation elements from the existing structures must be completely removed from the building areas***. Tree removal should include the entire stump and root ball. Septic tanks and leaching lines, if found, must be completely removed. The extent of this soil removal will be designated by a representative of Pacific Crest Engineering Inc. in the field. This material must be removed from the site.

18. Any voids created by removal of tree and root balls, septic tanks, and leach lines must be backfilled with properly compacted native soils that are free of organic and other deleterious materials or with approved imported fill.

19. Any wells encountered shall be capped in accordance with the requirements and approval of the local Health Department. The strength of the cap shall be equal to the adjacent soil and shall not be located within 5 feet of a structural footing.

20. Surface vegetation, tree roots and organically contaminated topsoil should then be removed ("stripped") from the area to be graded. In addition, any remaining debris or large rocks must also be removed (this includes asphalt or rocks greater than 2 inches in greatest dimension). This material may be stockpiled for future landscaping.

21. It is anticipated that the depth of stripping may be 2 to 4 inches, however the required depth of stripping must be based upon visual observations of a representative of Pacific Crest Engineering Inc., in the field. The depth of stripping will vary upon the type and density of vegetation across the project site and with the time of year. Areas with dense vegetation or groves of trees may require an increased depth of stripping.

22. It is possible that there are areas of man-made fill on the project site that our field investigation did not detect. Areas of man-made fill, if encountered, will need to be completely excavated to undisturbed native material. The excavation process should be observed and the

extent designated by a representative of Pacific Crest Engineering Inc., in the field. Any voids created by fill removal must be backfilled with properly compacted approved native soils that are free of organic and other deleterious materials, or with approved imported fill.

23. Following the stripping and backfilling of voids, the exposed soils in the building and paving areas should be removed to the recommended depths as outlined below, or as designated by a representative from Pacific Crest Engineering. The base of the excavation should be scarified to a minimum depth of 12 inches and the soil moisture conditioned and compacted as an engineered fill except for any contaminated material noted by a representative of Pacific Crest Engineering Inc. in the field. The moisture conditioning procedure will depend on the time of year that the work is done, but should result in the soils being 2 to 4 percent over their optimum moisture content at the time of compaction.

24. Following subexcavation and bottom processing, the area may be raised to design subgrade with approved native soil or imported fill as recommended in this report.

25. The following subexcavation and recompaction depths are recommended:

Roadway and parking areas: 18 inches below subgrade
Exterior concrete flatwork/slabs: 24 inches below bottom of slab
Interior slab-on-grade: 24 inches below capillary break
Shallow foundations or structural mats*: 48 inches below bottom of footing

*Excludes foundations to be founded upon competent bedrock

26. Recompacted sections should extend 5 feet horizontally beyond all slabs, footings and pavement areas, or to an adjacent property boundary, whichever is less.

Note: If this work is done during or soon after the rainy season, the on-site soils and other materials may be too wet in their existing condition to be used as engineered fill. These materials may require a diligent and active drying and/or mixing operation to reduce the moisture content to the levels required to obtain adequate compaction as an engineered fill. If the on-site soils or other materials are too dry, water may need to be added. In some cases the time and effort to dry the on-site soil may be considered excessive, and the import of aggregate base may be required.

27. The soil on the project site should be compacted as follows:

a. In pavement areas the upper 8 inches of subgrade, and all aggregate subbase and aggregate base, should be compacted to a minimum of 95% of its maximum dry density,
b. In pavement areas all utility trench backfill should be compacted to 95% of its maximum dry density,
c. All remaining soil on the project site should be compacted to a minimum of 90% of its maximum dry density.

28. The maximum dry density will be obtained from a laboratory compaction curve run in accordance with ASTM Procedure #D1557. This test will also establish the optimum moisture content of the material. Field density testing will be performed in accordance with ASTM Test #D6938 (nuclear method).

29. Expansive clay or silt soils are not suitable for engineered fill. Native sandy soils or imported soil to be used as engineered fill on this project should meet the following requirements:

a. free of organics, debris, and other deleterious materials,
b. free of "recycled" materials such as asphaltic concrete, concrete, brick, etc.,
c. granular in nature, well graded, and contain sufficient binder to allow utility trenches to stand open,
d. free of rocks in excess of 2 inches in size.

In addition to the above requirements, import fill should have a Plasticity Index between 4 and 12, and a minimum Resistance "R" Value of 30, and be non-expansive.

30. All native and import fill should be placed in maximum 8 inch lifts, before compaction, at a water content which is within 1 to 3 percent of the laboratory optimum value.

31. We recommend field density testing be performed in maximum 2 foot elevation differences. In general terms, we would recommend at least one compaction test per 200 linear feet of utility trench or retaining wall backfill, and at least one compaction test per 2,000 square feet of building or structure area. This is a subjective value and may be changed by the Geotechnical Engineer based on a review of the final project layout and exposed field conditions.

32. Samples of any proposed imported fill planned for use on this project should be submitted to Pacific Crest Engineering Inc. for appropriate testing and approval not less than ten (10) working days before the anticipated jobsite delivery. This includes proposed import trench sand, drain rock and aggregate base materials. Imported fill material delivered to the project site without prior submittal of samples for appropriate testing and approval must be removed from the project site.

CUT AND FILL SLOPES

33. All fill slopes should be constructed with engineered fill meeting the minimum density requirements of this report and have a gradient no steeper than 2:1 (horizontal to vertical). Fill slopes should not exceed 15 feet in vertical height unless specifically reviewed by Pacific Crest Engineering Inc. Where the vertical height exceeds 15 feet, intermediate benches must be provided. These benches should be at least 6 feet wide and sloped to control surface drainage. A lined ditch should be used on the bench.

34. Fill slopes should be keyed into the native slopes by providing a 10 foot wide base keyway sloped negatively at least 2% into the bank. The depth of the keyways will vary, depending on

the materials encountered. It is anticipated that the depth of the keyways may be 3 to 6 feet, but at all locations shall be at least 2 feet into firm material.

35. Subsequent keys may be required as the fill section progress upslope. Keys will be designated in the field by a representative of Pacific Crest Engineering Inc. See Figure No. 51 in Appendix A for general details.

36. Cut slopes within the fractured mudstone bedrock shall not exceed a 2:1 (horizontal to vertical) gradient, and a 3:1 gradient for cuts in soil. Cut slopes should not exceed a 15 foot vertical height unless specifically reviewed by a representative of Pacific Crest Engineering Inc. Where the vertical height exceeds 15 feet, intermediate benches must be provided. These benches should be at least 6 feet wide and sloped to control surface drainage. A lined ditch should be used on the bench.

37. The above slope gradients are based on the strength characteristics of the materials under conditions of normal moisture content that would result from rainfall falling directly on the slope, and do not take into account the additional activating forces applied by seepage from spring areas or subsurface groundwater. Therefore, in order to maintain stable slopes at the recommended gradients, it is important that any seepage forces and accompanying hydrostatic pressure encountered be relieved by adequate drainage. Drainage facilities may include subdrains, gravel blankets, rock fill surface trenches or horizontally drilled drains. Configurations and type of drainage will be determined by a representative of Pacific Crest Engineering Inc. during the grading operations.

38. The surfaces of all cut and fill slopes should be prepared and maintained to reduce erosion. This work, at a minimum, should include track rolling of the slope and effective planting. The protection of the slopes should be installed as soon as practicable so that a sufficient growth will be established prior to inclement weather conditions. It is vital that no slope be left standing through a winter season without the erosion control measures having been provided.

39. The above recommended gradients do not preclude periodic maintenance of the slopes, as minor sloughing and erosion may take place.

40. If a fill slope is to be placed above a cut slope, the toe of the fill slope should be set back at least 8 feet horizontally from the top of the cut slope. A lateral surface drain should be placed in the area between the cut and fill slopes.

EROSION CONTROL

41. The surface soils are classified as having a moderate to high potential for erosion, especially within sloped areas on the property. Therefore, the finished ground surface should be planted with ground cover and continually maintained to minimize surface erosion. Specific and detailed recommendations regarding erosion control on and surrounding the project site should be developed by the project civil engineer or an erosion control specialist.

TEMPORARY SHORING

42. There is a possibility that temporary construction shoring may become necessary on this project. The design, construction and installation of the shoring system is the sole responsibility of the Contractor.

43. Basement, trench or retaining wall excavations should have temporary sidewall slopes in accordance with CAL-OSHA guidelines or be mechanically shored. Excavation safety and shoring is the sole responsibility of the contractor. Excavation design and shoring systems should be submitted to the geotechnical engineer and the structural engineer a minimum of 3 weeks prior to construction for a review to determine the conformance of the design with standard engineering practices and specific site conditions.

44. The "top" of any temporary cut slope should be set-back at least ten feet (measured horizontally) from any nearby structure or property line. Any planned excavation which cannot meet these side slope gradients will need to have a shoring system designed to support steeper sidewall gradients.

45. It should be understood that on-site safety is the *sole responsibility* of the Contractor, and that the Contractor shall designate a *competent person* (as defined by CAL-OSHA) to monitor the slope excavation prior to the start of each work day, and throughout the work day as conditions change. The competent person designated by the Contractor shall determine if flatter slope gradients are more appropriate, or if shoring should be installed to protect workers in the vicinity of the slope excavation. Refer to Title 8, California Code of Regulations, Sections 1539-1543.

46. The temporary shoring may consist of either a soldier pier wall with wood lagging or a soil nail wall with a shotcrete facing. Irrespective of the type of shoring, the chosen wall should be fully drained and should not obstruct nor significantly change the normal flow of moisture or groundwater through the project soils. Wall drainage should discharge to an approved location.

47. If a soil nail wall with shotcrete facing is utilized, it should include weep holes through the facing on a minimum 5 foot grid. The designers should note that the soil nails in clay may tend to creep when loaded in tension.

48. If a soldier pier wall with wood lagging is utilized, the wood lagging, and any gravel backfill (or other drainage material) behind the wall, must be completely removed as the excavation is backfilled, and prior to the completion of the project. Soldier piles should be cut off a minimum of 5 feet below finished grade.

49. All shoring backfill to be placed in maximum 8 inch lifts, at a water content which is 1 to 3 percent above the laboratory optimum value. The material should be compacted to at least 90 percent relative compaction. If a clean gravel backfill is utilized as shoring backfill, it should be compacted in maximum 1 to 2 foot lifts using a vibra-plate or similar equipment. **It is recommended that all voids behind the shoring system be completely filled with soil or gravel backfill while the shoring work is in progress.**

50. The temporary shoring wall system chosen by the designer should be designed using the geotechnical design criteria presented in the "Lateral Pressures" section of this report.

FOUNDATION RECOMMENDATIONS – PHASE I

P1-1, P1-2, and P1-3 Building Areas – Drilled Piers

51. At the time we prepared this report, the grading plans had not been completed and the structure location and foundation details had not been finalized. We request an opportunity to review these items during the design stages to determine if supplemental recommendations will be required.

52. As discussed previously, the slopes surrounding the Phase I building sites are subject to long term retreat due to erosion and/or landsliding. Structural elements within the influence of the projected failure zones as depicted on Plates 1 and 2 in Appendix B should be designed for the eventual removal of soil support within this zone. It is recommended that the buildings therefore be supported by drilled pier foundations. Any portion of the foundation located within the zone of the projected failure plane(s) will need to be designed for a lateral earth pressure on the piers due to the unstable wedge of colluvium that blankets the slope.

53. Based upon the results of our investigation, it is our opinion that an appropriate foundation system to support the proposed structure(s) will consist of skin friction bearing, cast-in-place reinforced concrete piers in conjunction with reinforced concrete grade beams. Piers should be designed and constructed according to Section 1810 of the 2013 CBC as well as the following recommendations:

a. Minimum pier embedment should be 10 feet into competent bedrock. This will necessitate pier depths of approximately 20 feet. Actual depths could depend upon a lateral force analysis performed by your structural engineer.

b. Piers constructed to the criteria outlined above may be designed for skin friction resistance due to the underlying bedrock of 450 psf of surface area, with a 1/3 increase for wind or seismic loading. Neglect skin friction resistance over the upper ten feet of pier depth within the projected failure zone.

c. Piers spacing should be based on floor, wall or roof loads determined by the Project Structural Engineer. We would recommend a minimum center-to-center spacing of four pier diameters.

d. Minimum pier size should be 18 inches in diameter and all pier holes must be free of loose material on the bottom.

e. A reduction for group action is not considered necessary for drilled piers unless the piers are spaced less than 3 pier diameters apart.

f. Drilled piers constructed on the slope face and/or within 15 feet inboard of the top of slope (35 feet for the P1-1 spa buildings) should be designed for an active earth

pressure from the upper 10 feet of soil against the piers. Use an active earth pressure of 50 psf/ft of depth acting on a plane which is 1½ times the pier diameter. This depth may be reduced by one foot for every two feet of horizontal distance inboard of the top of slope.

g. Passive pressures of 450 psf/ft of bedrock depth can be developed, acting over a plane 1½ times the pier diameter. Neglect passive pressure in the upper ten feet of pier depth within the projected failure zone.

h. All grade beams should be embedded at least 12 inches below lowest adjacent grade.

i. If expansive clay soils are noted at the base of grade beams, we recommend placing a SureVoid^{R} System or equivalent material approved in advance by the Geotechnical Engineer at the base of the grade beams.

j. All pier excavation spoils must be removed from slope areas which are steeper than 5:1 (horizontal to vertical).

k. All piers must be constructed within ½ percent of a vertically plumb condition.

l. It is possible that the piers will need to be cased during drilling and that the water will have to either be pumped before steel and concrete placement or the concrete placed through a tremie. If the casing is pulled during the concrete pour, it must be pulled slowly with a minimum of 4 feet of casing remaining embedded within the concrete at all times.

m. The Contractor should expect very hard rock drilling conditions at an approximate depth of 10 feet, based on the findings outlined in our test borings. Therefore, appropriately sized drilling equipment should be selected for these hard rock drilling conditions so that the piers may extend to the full depth outlined in the geotechnical report and the project plans and specifications.

54. **Drilled Pier Field Observation and Reporting**

a. All pier construction must be observed by a Pacific Crest Engineering Inc. Any piers constructed without the full knowledge and continuous observation of a representative from Pacific Crest Engineering Inc. will render the recommendations of this report invalid.

b. **Continuous** observation of pier drilling operations is required by 2013 CBC Chapter 17, Section 1704.9. You should notify your Contractor and drilling Subcontractor regarding this requirement. A representative from our firm should be on-site **at all times** while pier drilling operations are in progress.

c. Reporting will include a Daily Field Report (DFR) maintained by an on-site representative from Pacific Crest Engineering Inc. The DFR will maintain a record of

each pier drilled, and note pier diameters, depths, plumbness, and embedment into suitable soil or bedrock bearing strata, as required by the Geotechnical Report.

55. The piers and grade beams should contain steel reinforcement as determined by the Project Civil or Structural Engineer.

Remodeled Buildings R-1 through R-7

56. Our scope of work did not include subsurface investigations in the vicinity of these buildings. If foundation improvements or upgrades are required we can provide geotechnical recommendations once renovation plans have been provided to us. PCEI requests to review interim and final design plans in order to determine the need for supplemental recommendations.

57. The Zinn Geology report notes that with the exception of building R7, none of the buildings to be remodeled appear to be threatened by geologic hazards that present unacceptably high risks.

58. As described in the Zinn Geology report and depicted within Plate 1, R7 is located at the toe of a steep slope blanketed with loose colluvium and is adjacent to an existing landslide deposit. Consequently, there is a potential for the building to be impacted by debris flows initiating on the slope behind the building. The slope should be examined by the project geologist and geotechnical engineer in order to develop recommendations for stabilization or protection structures as needed.

FOUNDATION RECOMMENDATIONS – PHASE II
P2-1 Building Area – Spread Footings

59. At the time we prepared this report, the grading plans had not been completed and the structure location and foundation details had not been finalized. We request an opportunity to review these items during the design stages to determine if supplemental recommendations will be required.

60. Considering the current proposed building area, soil characteristics and site preparation recommendations, it is our opinion that an appropriate foundation system to support proposed P2-1 structure will consist of reinforced concrete spread footings bedded into competent bedrock. This system could consist of continuous exterior footings, in conjunction with interior isolated spread footings or additional continuous footings or concrete slabs.

61. The current proposed building area requires excavation into a bedrock slope and removal of the fill at the southeastern wing of the new building. All existing fill must be completely removed from improvement areas.

62. <u>All footings must be trenched the required depth into competent bedrock</u>. Footing widths and depths should be based upon the allowable bearing value but not less than the minimum widths and depths as shown in the table below.

TABLE No. 3, Minimum Footing Widths and Depths

Number of Stories	Footing Width	Footing Depth
1	12 inches	18 inches
2	15 inches	18 inches
3	18 inches	24 inches
Multi-story	24 inches	24 inches

Please note: The minimum footing embedment is measured from the <u>lowest existing and adjacent bedrock grade</u> and should not include any concrete slab-on-grade, capillary break and sand cushion in the total depth of embedment.

63. Footings constructed to the given criteria may be designed for the following allowable bearing capacities:

a. 3,500 psf for Dead plus Live Load
b. a 1/3rd increase for Seismic or Wind Load

Please note: In computing the pressures transmitted to the soil by the footings, the embedded weight of the footing may be neglected.

64. Provided our recommendations are followed, total settlement due to applied dead and live loads is not expected to exceed 1 inch across the length of the structure, with differential settlement of less than ½ inch.

65. No footing should be placed closer than 10 feet to the top of a fill slope nor 8 feet from the base of a cut slope.

66. No footing shall be placed on slopes steeper than 4:1 (h:v). **If the intent is to place the foundation on sloping ground which exceeds 4:1 (h:v), Pacific Crest Engineering Inc. should be contacted for an alternative pier and grade beam foundation design**.

67. All footings should be excavated into competent bedrock. No footings shall be constructed with the intent of placing engineered fill against the footing after the footing is poured, and counting that engineered fill as part of the embedment depth of the footing.

68. Footings may be assumed to have a resistance to lateral sliding coefficient of 0.30.

69. Footings may be assumed to have a lateral bearing pressure resistance value of 400 psf/foot.

70. All grade beams, thickened slab edges and other foundation elements which impart structure loads to the soil (from dead, live, wind or seismic loads) should be considered “footings” and constructed according to the recommendations of this section, including required depths below lowest adjacent soil grade.

71. The footing excavations must be free of loose material prior to placing concrete. The footing excavations should be thoroughly saturated prior to placing concrete.

72. Footing excavations must be observed by a representative of Pacific Crest Engineering Inc. before placement of formwork, steel and concrete to ensure bedding into proper material.

73. The footings should contain steel reinforcement as determined by the Project Civil or Structural Engineer in accordance with applicable CBC or ACI Standards.

Slab-On-Grade Construction

74. Interior concrete slabs should bear upon non-expansive engineered fill or competent bedrock as described above.

75. All exterior slabs, patios, walkways, etc., should be structurally independent of structural foundation system(s).

76. Interior concrete slabs-on-grade may be structurally integrated with the footings. If the slabs are constructed as "free floating" slabs, they should be provided with a minimum ¼ inch felt separation between the slab and footing. The slabs should be separated into approximately 15' x 15' square sections with dummy joints or similar type crack control devices.

77. All interior concrete slabs-on-grade should be underlain by a minimum 6 inch thick capillary break consisting of ¾ inch clean crushed rock (no fines). It is recommended that neither Class II baserock nor sand be employed as the capillary break material.

78. Where floor coverings are anticipated or vapor transmission may be a problem, a vapor retarder/membrane should be placed between the capillary break layer and the floor slab in order to reduce the potential for moisture condensation under floor coverings. We recommend a high quality vapor retarder at least 15 mil thick and puncture resistant (Stego Wrap or equivalent). The vapor retarder must meet the minimum specifications for ASTM E-1745, Standard Specification For Water Vapor Retarder. Please note that low density polyethylene film (such as Visqueen) may meet minimum current standards for permeability but not puncture resistance. Laps and seams should be overlapped at least six inches and properly sealed to provide a continuous layer beneath the entire slab that is free of holes, tears or gaps. Joints and penetrations should also be properly sealed.

79. Floor coverings should be installed on concrete slabs that have been constructed according to the guidelines outlined in ACI 302.2R and the recommendations of the flooring material manufacturer.

80. Currently, ACI 302-1R recommends that concrete slabs to receive moisture sensitive floor coverings be placed directly upon the vapor retarder, with **no sand cushion**. ACI states that vapor retarders are not effective in preventing residual moisture within the concrete slab from migrating to the surface. Including a low water-to-cement ratio (less than 0.50) and/or admixtures into the mix design are generally necessary to minimize water content, reduce soluble

alkali content, and provide workability to the concrete. As noted in CIP 29 (*Concrete in Practice by the National Ready Mixed Concrete Association*), placing concrete directly on the vapor retarder can also create potential problems. If environmental conditions do not permit rapid drying of bleed water from the slab surface then the excess bleeding can delay finishing operations (refer to CIP 13, 19 and 20). Most of these problems can be alleviated by using a concrete with a low water content, moderate cement factor, and well-graded aggregate with the largest possible size. **With the increased occurrence of moisture related floor covering failures, minor cracking of floors placed on a vapor retarder and other problems discussed here are considered a more acceptable risk than failure of floor coverings, and these potential risks should be clearly understood by the Client and Project Owner**.

81. If a sand layer is chosen as a cushion for slabs without floor coverings, it should consist of clean sand. Clean sand is defined as 100 percent passing the #4 sieve, and less than 5 percent passing the #200 sieve.

82. Requirements for pre-wetting of the subgrade soils prior to the pouring of the slabs will depend on the specific soils and seasonal moisture conditions and will be determined by a representative of Pacific Crest Engineering Inc. at the time of construction. It is important that the subgrade soils be properly moisture conditioned at the time the concrete is poured. Subgrade moisture contents should not be allowed to exceed our moisture recommendations for effective compaction, and should be maintained until the slab is poured.

Please Note: Recommendations given above for the reduction of moisture transmission through the slab are general in nature and present good construction practice. Moisture protection measures for concrete slabs-on-grade should meet applicable ACI and ASTM standards. Pacific Crest Engineering Inc. are not waterproofing experts. For a more complete and specific discussion of moisture protection within the structure, a qualified waterproofing expert should be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The waterproofing consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure as deemed appropriate.

83. Slab thickness, reinforcement, and doweling should be determined by the Project Civil or Structural Engineer. The use of welded wire mesh is not recommended for slab reinforcement.

Building Area P2-2 Parking Structure – Drilled Piers or Structural Mat Foundation

84. The proposed P2-2 building site is underlain by an infilled drainage swale and artificial fill. The depth of artificial fill and colluvium is relatively shallow at the north end of the site, but thickens to the south towards the fill crest where the combined soil thickness is as much as 50 feet. These materials are considered compressible and subject to excessive settlements under foundation loads.

85. Proposed structures in this area of the campus will need to be designed to withstand total and differential settlement. This issue can be addressed by a drilled pier foundation that extends

uniformly into a suitable bearing stratum, or a structural mat foundation bearing upon a zone of redensified soil. Geotechnical design criteria for either alternative will depend largely upon the selected location for the structure. Siting the structure at the north end of the field where the fill/colluvium thickness is relatively shallow will result in reduced pier depths and/or calculated settlements. As the building envelope moves toward the south potential settlements and depth to suitable bearing strata will increase with increasing fill thickness.

86. Should the interim design only include paving over the existing soccer field surface for open parking, the parking area should be paved in accordance with the Pavement Design recommendations presented within this report. However, please note that due to the thick, loosely compacted fill present at the site, a paved parking lot may be subject to surface cracking.

87. We anticipate that the development of additional design criteria and/or modifications to our recommendations will be necessary once grading plans have been completed and the structure location and foundation details have been developed. However, for initial planning purposes we offer the following recommendations:

88. Drilled piers may be designed in accordance with the criteria presented for the Phase I structures, however active loads against the piers will depend upon the depth of fill which could range from 10 feet at the north end of the site to 25 feet or more at the south end of the site. Pier depths on the order of 20 to 60 feet are possible, depending upon where the structure is located.

89. Alternatively, the structure could be supported by a reinforced concrete structural mat foundation system designed to withstand differential settlement and allow the structure to move as a single unit. The loading should be kept as even as possible in all areas of the structure. The mat should be supported upon a minimum of 4 feet of engineered fill that has been placed and compacted in accordance with the recommendations of this report.

90. The structural mat may be designed for an allowable bearing capacity of 1,700 psf (dead plus live load). This value may be increased by one-third for wind or seismic loads.

91. Estimates for post-construction total and differential settlements of the foundation will depend upon the final structure location, however we <u>conservatively</u> estimate total settlements ranging from about 6 inches at the north end, up to 12 inches at the south end. Differential settlements on the order of 4 to 8 inches over the length of the structure can be expected.

92. The coefficient of vertical subgrade reaction used in design should be 80 tons per cubic foot.

93. The mat may be assumed to have an allowable resistance to lateral sliding of 0.30. This value may be increased by one-third for short term wind/seismic loads.

94. The embedded portion of the mat may be assumed to have an allowable lateral bearing pressure resistance value of 350 psf/ft.

95. Utility connections crossing the slab perimeter should be designed with flexible connections to accommodate anticipated total settlements.

96. The structural mats should have thickened edge beams which extend at least 12 inches below lowest adjacent grade, not including sand or gravel sections.

97. Subgrade preparation and thickened mat excavations should be observed and approved by the Geotechnical Engineer prior to steel placement.

98. All interior structural slabs should be underlain by a minimum 6 inch thick capillary break, consisting of ¾-inch clean crushed rock. It is recommended that neither Class II baserock nor sand be employed as the capillary break material.

99. Where floor coverings are anticipated or vapor transmission through the slab may be a problem, a vapor/waterproof membrane should be placed between the capillary break layer and the floor slab in order to reduce the potential for moisture condensation under floor coverings. We recommend a high quality vapor retarder at least 10 mil thick and puncture resistant (Stego Wrap or equivalent). The vapor barrier must meet the specifications for ASTM E 1745, Standard Specification For Water Vapor Retarder. A 2-inch layer of moist sand on top of the membrane can help protect the membrane and will assist in equalizing the curing rate of the concrete. Please note that low density polyethylene film (such as Visqueen) may meet minimum current standards for permeability but not puncture resistance. Laps and seams should be overlapped at least six inches and properly sealed to provide a continuous layer beneath the entire slab that is free of holes, tears or gaps. Joints and penetrations should also be properly sealed.

100. Floor coverings should be installed on concrete slabs that have been constructed according to the guidelines outlined in ACI 302.2R and the recommendations of the flooring material manufacturer.

101. Currently, ACI 302-1R recommends that concrete slabs to receive moisture sensitive floor coverings be placed directly upon the vapor retarder, with no sand cushion. Vapor retarders are not effective in preventing residual moisture within the concrete slab from migrating to the surface. Including a low water-to-cement ratio (less than 0.50) and/or admixtures into the mix design are generally necessary to minimize water content, reduce soluble alkali content, and provide workability to the concrete. As noted in CIP 29 of the National Ready Mixed Concrete Association (NRMCA), placing concrete directly on the vapor retarder can also create potential problems. If environmental conditions do not permit rapid drying of bleed water from the slab surface then the excess bleeding can delay finishing operations. Bleed water trapped below a finished surface can cause delaminations (CIP 20) or blisters (CIP 13) if finishing operations are not performed at the correct time after bleed water has disappeared from the surface. Concrete may stiffen slower, which means that trowel finishing operations must be delayed; thus increasing the susceptibility of plastic shrinkage cracking. Curling (CIP 19) can occur due to differential drying and related shrinkage at different levels in the slab. Most of these problems can be alleviated by using a concrete with a low water content, moderate cement factor, and well-graded aggregate with the largest possible size. **With the increased occurrence of**

moisture related floor covering failures, minor cracking of floors placed on a vapor retarder and other problems discussed here are considered a more acceptable risk than failure of floor coverings, and these potential risks should be clearly understood by the Client and Project Owner.

Please Note: Recommendations given above for the reduction of moisture transmission through the slab are general in nature and present good construction practice. Moisture protection measures for concrete slabs-on-grade should meet applicable ACI and ASTM standards. Pacific Crest Engineering Inc. are not waterproofing experts. For a more complete and specific discussion of moisture protection within the structure, a qualified waterproofing expert should be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The waterproofing consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure as deemed appropriate.

102. Requirements for pre-wetting of the subgrade soils prior to the pouring of the slabs will depend on the specific soils and seasonal moisture conditions and will be determined by a representative of Pacific Crest Engineering Inc. at the time of construction. It is important that the subgrade soils be thoroughly and properly moisture conditioned at the time the concrete is poured. Subgrade moisture contents should not be allowed to exceed our moisture recommendations for effective compaction, and should be maintained until the slab is poured.

103. Mat thickness, reinforcement, and doweling should be determined by the Project Structural Engineer.

Building Area P2-3 - 4-Plex Lodging - Drilled Piers or Structural Mat Foundations

104. The proposed P2-3 site is also underlain by an infilled drainage swale, and we noted artificial fill in B13 at the south end of the parking lot. The upper 10 to 15 feet of these materials are considered compressible and subject to settlement under foundation loads.

105. Proposed structures in this area of the campus will also need to be designed to withstand total and differential settlement. This issue can be addressed by a drilled pier foundation that extends uniformly into a suitable bearing stratum, or a structural mat foundation bearing upon a zone of redensified soil. Geotechnical design criteria for either alternative will depend largely upon the selected location for the structure. Siting structures closer to the north end where the colluvium is relatively shallow will result in reduced pier depths and/or calculated settlements. As the building envelopes move toward the south potential settlements due to the fill and/or colluvium, as well as depth to suitable bearing strata, will increase.

106. We anticipate that the development of additional design criteria and/or modifications to our recommendations will be necessary once grading plans have been completed and the structure location and foundation details have been developed. However, for initial planning purposes we offer the following recommendations:

107. Drilled piers may be designed in general accordance with the criteria presented for the Phase I structures, however active loads against the piers will depend upon the depth of fill which could range from 0 feet at the north end of the site to approximately 5 feet at the south end of the site. Drilled piers should extend a minimum of 10 feet into competent native soil or bedrock. Pier depths on the order of 15 to 20 feet are possible, depending upon where the structure is located.

108. Alternatively, the structures could be supported by a reinforced concrete structural mat foundation system designed to withstand differential settlement and allow the structure to move as a single unit. The loading should be kept as even as possible in all areas of the structure. The mat should be supported upon a minimum of 4 feet of engineered fill that has been placed and compacted in accordance with the recommendations of this report.

109. The structural mat may be designed for an allowable bearing capacity of 1,700 psf (dead plus live load). This value may be increased by one-third for wind or seismic loads.

110. Estimates for post-construction total and differential settlements of the foundation will depend upon the final structure location; however we conservatively estimate total settlements ranging from about 3 inches at the north end, up to 6 inches at the south end. Differential settlements on the order of 2 to 4 inches over the length of the structure can be expected.

111. The coefficient of vertical subgrade reaction used in design should be 80 tons per cubic foot.

112. The mat may be assumed to have an allowable resistance to lateral sliding of 0.30. This value may be increased by one-third for short term wind/seismic loads.

113. The embedded portion of the mat may be assumed to have an allowable lateral bearing pressure resistance value of 350 psf/ft.

114. Utility connections crossing the slab perimeter should be designed with flexible connections to accommodate anticipated total settlements.

115. The structural mats should have thickened edge beams which extend at least 12 inches below lowest adjacent grade, not including sand or gravel sections.

116. Subgrade preparation and thickened mat excavations should be observed and approved by the Geotechnical Engineer prior to steel placement.

117. All interior structural slabs should be underlain by a minimum 6 inch thick capillary break, consisting of ¾-inch clean crushed rock. It is recommended that neither Class II baserock nor sand be employed as the capillary break material.

118. Where floor coverings are anticipated or vapor transmission through the slab may be a problem, a vapor/waterproof membrane should be placed between the capillary break layer and the floor slab in order to reduce the potential for moisture condensation under floor coverings.

We recommend a high quality vapor retarder at least 10 mil thick and puncture resistant (Stego Wrap or equivalent). The vapor barrier must meet the specifications for ASTM E 1745, Standard Specification For Water Vapor Retarder. A 2-inch layer of moist sand on top of the membrane can help protect the membrane and will assist in equalizing the curing rate of the concrete. Please note that low density polyethylene film (such as Visqueen) may meet minimum current standards for permeability but not puncture resistance. Laps and seams should be overlapped at least six inches and properly sealed to provide a continuous layer beneath the entire slab that is free of holes, tears or gaps. Joints and penetrations should also be properly sealed.

119. Floor coverings should be installed on concrete slabs that have been constructed according to the guidelines outlined in ACI 302.2R and the recommendations of the flooring material manufacturer.

120. Currently, ACI 302-1R recommends that concrete slabs to receive moisture sensitive floor coverings be placed directly upon the vapor retarder, with no sand cushion. Vapor retarders are not effective in preventing residual moisture within the concrete slab from migrating to the surface. Including a low water-to-cement ratio (less than 0.50) and/or admixtures into the mix design are generally necessary to minimize water content, reduce soluble alkali content, and provide workability to the concrete. As noted in CIP 29 of the National Ready Mixed Concrete Association (NRMCA), placing concrete directly on the vapor retarder can also create potential problems. If environmental conditions do not permit rapid drying of bleed water from the slab surface then the excess bleeding can delay finishing operations. Bleed water trapped below a finished surface can cause delaminations (CIP 20) or blisters (CIP 13) if finishing operations are not performed at the correct time after bleed water has disappeared from the surface. Concrete may stiffen slower, which means that trowel finishing operations must be delayed; thus increasing the susceptibility of plastic shrinkage cracking. Curling (CIP 19) can occur due to differential drying and related shrinkage at different levels in the slab. Most of these problems can be alleviated by using a concrete with a low water content, moderate cement factor, and well-graded aggregate with the largest possible size. **With the increased occurrence of moisture related floor covering failures, minor cracking of floors placed on a vapor retarder and other problems discussed here are considered a more acceptable risk than failure of floor coverings, and these potential risks should be clearly understood by the Client and Project Owner**.

Please Note: Recommendations given above for the reduction of moisture transmission through the slab are general in nature and present good construction practice. Moisture protection measures for concrete slabs-on-grade should meet applicable ACI and ASTM standards. Pacific Crest Engineering Inc. are not waterproofing experts. For a more complete and specific discussion of moisture protection within the structure, a qualified waterproofing expert should be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The waterproofing consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure as deemed appropriate.

121. Requirements for pre-wetting of the subgrade soils prior to the pouring of the slabs will depend on the specific soils and seasonal moisture conditions and will be determined by a representative of Pacific Crest Engineering Inc. at the time of construction. It is important that the subgrade soils be thoroughly and properly moisture conditioned at the time the concrete is poured. Subgrade moisture contents should not be allowed to exceed our moisture recommendations for effective compaction, and should be maintained until the slab is poured.

122. Mat thickness, reinforcement, and doweling should be determined by the Project Structural Engineer.

UTILITY TRENCHES

123. Utility trenches that are parallel to the sides of the building should be placed so that they do not extend below a line sloping down and away at a 2:1 (horizontal to vertical) slope from the bottom outside edge of all footings.

124. Utility pipes should be designed and constructed so that the top of pipe is a minimum of 24 inches below the finish subgrade elevation of any road or pavement areas. Any pipes within the top 24 inches of finish subgrade should be concrete encased, per design by the Project Civil Engineer.

125. For the purpose of this section of the report, backfill is defined as material placed in a trench starting one foot above the pipe, and bedding is all material placed in a trench below the backfill.

126. Unless concrete bedding is required around utility pipes, free-draining clean sand should be used as bedding. Sand bedding should be compacted to at least 95 percent relative compaction.

127. Approved imported clean sand should be used as utility trench backfill. The use of native soil as backfill is not recommended with the exception of the top 12 inches of the trench. Backfill in trenches located under and adjacent to structural fill, foundations, concrete slabs and pavements should be placed in horizontal layers no more than 8 inches thick. This includes areas such as sidewalks, patios, and other hardscape areas. Each layer of trench backfill should be water conditioned and compacted to at least 95 percent relative compaction. Clean sand is defined as 100 percent passing the #4 sieve, and less than 5 percent passing the #200 sieve.

128. Utility trenches which carry “nested” conduits (stacked vertically) should be backfilled with a control density fill (such as 2-sack sand\cement slurry) to an elevation one foot above the nested conduit stack. The use of pea gravel or clean sand as backfill within a zone of nested conduits is not recommended.

129. **All utility trenches beneath perimeter footing or grade beams should be backfilled with controlled density fill (such as 2-sack sand\cement slurry) to help minimize potential moisture intrusion below interior floors. The length of the plug should be at least <u>three times</u> the width of the footing or grade beam at the building perimeter, but not more than 36 inches. A representative from Pacific Crest Engineering Inc. should be contacted to**

observe the placement of slurry plugs. In addition, all utility pipes which penetrate through the footings, stemwalls or grade beams (below the exterior soil grade) should also be sealed water-tight, as determined by the Project Engineer or Architect.

130. A representative from our firm should be present to observe the bottom of all trench excavations, prior to placement of utility pipes and conduits. In addition, we should observe the condition of the trench prior to placement of sand bedding, and to observe compaction of the sand bedding, in addition to any backfill planned above the bedding zone.

131. Jetting of the trench backfill is not recommended as it may result in an unsatisfactory degree of compaction.

132. Trenches must be shored as required by the local agency and the State of California Division of Industrial Safety construction safety orders.

LATERAL PRESSURES

Debris Impact Walls – Phase I

133. Potential debris flow landsliding originating on the slope above the Phase I spa buildings could impact the structures with both dynamic impact forces and static surcharge forces. To mitigate the potential debris flow hazard, we recommend construction of a debris impact wall between the buildings and the base of the slope to the east.

134. Zinn Geology estimates a debris flow volume of approximately 10 cubic yards, moving at a velocity of 10 feet per second. Based on the estimated failure wedge thickness as depicted on Cross Section X-X' in Appendix B, we recommend designing for a 7 foot thick soil mass.

135. For design purposes a horizontal impact force of 2.5 kips per foot of wall length should be considered to act on the wall.

136. Soil may stack up against the wall after the initial impact. The entire wall height should be designed as a retaining structure. An active soil pressure of 80 psf/foot should be used under this condition.

Retaining Walls

137. Retaining walls with full drainage should be designed using the following criteria:

a. The following lateral earth pressure values should be used for design:

TABLE No. 4, Active and At-Rest Earth Pressure Values

Maximum Backfill Slope (H:V)	Active Earth Pressure (psf/ft of depth)	At-Rest Earth Pressure (psf/ft of depth)
Level	45	55
2:1	60	70

Active earth pressure values may be used when walls are free to yield an amount sufficient to develop the active earth pressure condition (about ½% of height). The effect of wall rotation should be considered for areas behind the planned retaining wall (pavements, foundations, slabs, etc.). **When walls are restrained at the top or to design for minimal wall rotation, use at-rest earth pressure values.**

b. For resisting passive earth pressure use 350 psf/ft of depth.

c. A "coefficient of friction" between base of foundation and soil of 0.30

d. Retaining walls to be structurally integrated with drilled pier foundation systems should be supported by drilled pier foundations designed in accordance with the drilled pier foundation recommendations contained in this report. Site walls not associated with a building structure may be designed for an allowable bearing capacity of 1,800 psf for dead plus live load, with a 1/3rd increase for short term loads.

e. To develop the resisting passive earth pressure, the retaining wall footings should be embedded a minimum of 18 inches below the lowest adjacent grade. There should be a minimum of 5 feet of horizontal cover as measured from the outside edge of the footing.

f. Any live or dead loads which will transmit a force to the wall, refer to Figure No. 52 in Appendix A.

g. For flexible (yielding) retaining walls, the resultant seismic force on the wall is $10H^2$. This force has been estimated using the Mononobe-Okabe method of analysis as modified by Whitman (1990), and assumes a yielding wall condition. **Note that the resultant seismic force should be assumed to act at a point 0.33H up from the base of the wall.**

h. For rigid (non-yielding) retaining walls, the resultant seismic force on the wall is $14H^2$. **Note that the resultant seismic force should be assumed to act at a point 0.33H up from the base of the wall.**

Please note: Should the slope behind the retaining walls be other than shown in Table No.4, supplemental design criteria will be provided for the active earth or at rest pressures for the particular slope angle.

138. The above criteria are based on **fully drained conditions**. Therefore, we recommend that permeable material meeting the State of California Standard Specification Section 68-1.025, Class 1, Type A, be placed behind the wall, with a minimum width of 12 inches and extending for the full height of the wall to within 1 foot of the ground surface. The permeable material should be covered with Mirafi 140N filter fabric or equivalent and then compacted native soil placed to the ground surface. A 4 inch diameter perforated rigid plastic drain pipe should be installed within 3 inches of the bottom of the permeable material and be discharged to a suitable,

approved location such as the project storm drain system. The perforations should be located and oriented on the lower half of the pipe. Neither the pipe nor the permeable material should be wrapped in filter fabric. Please refer to Figure No. 53 of Appendix A, Typical Retaining Wall Drain Detail.

139. The area behind the wall and beyond the permeable material should be compacted with approved material to a minimum relative dry density of 90%.

SURFACE DRAINAGE

140. Following completion of the project we recommend that storm drainage provisions and performance of permanent erosion control measures be closely observed through the first season of significant rainfall, to determine if these systems are performing adequately and, if necessary, resolve any unforeseen issues.

141. The current, dated drainage system uses disposal points that enter Carbonera Creek near the middle of the steep slopes above the creek. As noted by the project geologist, these mid-slope disposal points appear to coincide with shallow landslide scars present within the slope face. In our opinion, mid-slope disposal of storm water is unacceptable as concentrated discharge at these locations has triggered debris flows in the past. Disposal points into Carbonera Creek should be closer to the toe of the slope and orientated in a manner to minimize concentrated discharge from coming into contact with any portion of the slope.

142. Surface water must not be allowed to pond or be trapped adjacent to the building foundations nor on the building pad nor in the parking areas.

143. All roof eaves should be guttered, with the outlets from the downspouts provided with adequate capacity to carry the storm water from the structures to reduce the possibility of soil saturation and erosion. The connection should be in a closed conduit which discharges at an approved location away from the structures and the graded area. Under no circumstances should discharge locations be located at the top of, or on the face of any topographic slopes.

144. Buildings located on moderate to steep slopes should have a V-ditch system located in the area upgradient of the building pad to quickly intercept and remove surface runoff flowing towards the structure. The V-ditch may be located above planned retaining walls, if present, or on the natural hillside area. We would recommend a concrete or asphalt lined V-ditch system. The V-ditch should discharge to an appropriate drain inlet which channels the water via an underground pipe to areas down slope of the graded area and the structure.

145. Final grades should be provided with a positive gradient away from all foundations in order to provide for rapid removal of the surface water from the foundations to an adequate discharge point. Soil grades should slope away from foundation areas at least 5 percent for the first 10 feet. Impervious surface areas should slope away from foundations at least 2 percent for the first 10 feet. Concentrations of surface water runoff should be handled by providing necessary structures, such as paved ditches, catch basins, etc.

146. Cut and fill slopes shall be constructed so that surface water will not be allowed to drain over the top of the slope face. This may require berms along the top of fill slopes and surface drainage ditches above cut slopes. All cut, fill and disturbed native slope areas should be hydro-seeded or other means of erosion control provided, as determined by the Project Civil Engineer.

147. Irrigation activities at the site should not be done in an uncontrolled or unreasonable manner.

148. Drainage resulting from impervious surfaces such as patios, roofs, walkways, and roads should be collected into solid pipes and discharged into approved discharge points. As discussed previously, concentrated runoff should not be allowed to discharge near structures or flow over and down steep slopes.

149. It is recommended that the services of a project civil engineer be retained in order to develop a comprehensive drainage system that terminates in a disposal system along Carbonera Creek, or in an area where recharge is deemed geologically feasible by the project geologist.

150. The building and surface drainage facilities must not be altered nor any filling or excavation work performed in the area without first consulting Pacific Crest Engineering Inc. Surface drainage improvements developed by the project civil engineer must be maintained by the property owner at all times, as improper drainage provisions can produce undesirable affects.

ROADWAY/PAVEMENT DESIGN

Road Between P2-2 Parking Structure and Gaston Village

151. As noted in the Zinn Geology report and depicted on Plate 1, the proposed new road traverses two landslide deposits. Furthermore, the terrain in this location is steep and covered with loose soils that are likely prone to soil creep and shallow landsliding. Deep cuts into the soil and/or underlying bedrock, along with retaining walls, will be necessary for this proposed alignment.

152. PCEI recommends investigation of this area prior to determining the final placement of the road. The investigation should include field mapping using suitable topographic maps with a scale of 1”=20’ or greater as well as the advancement of small diameter test borings or exploratory test pits. The field investigation will allow for the development of landslide mitigation measures that will include potential stabilization of existing landslide deposits and the construction of retaining structures.

Pavement Recommendations

153. The soils that will comprise the pavement subgrade will in all likelihood be the silts and intermediate clays predominating on the site. We assumed an “R” Value of 44 for design of the pavement sections noted below, based on prior studies at the campus. This must be verified in the field and, if necessary, modifications made to these tentative sections.

154. For design purposes, the following traffic indices are suggested:

a. Parking stalls T.I. = 4½
b. Traffic aisles T.I. = 5
c. Truck usage areas T.I. = 6½

*Pacific Crest Engineering Inc. has not performed a site specific traffic study to determine the actual traffic indices associated with this project. These values are for general design purposes only and the values may need modification. Traffic volume and equivalent axle loads that exceed the assumed TI could be destructive to the pavement, resulting in an accelerated rate of deterioration and the need for increased maintenance.

155. The following table provides a flexible pavement design which is based on the Caltrans Highway Design Manual – Chapter 600.

The following pavement sections are suggested:

TABLE No.5, Recommended Pavement Sections

Material	Traffic Index		
	4½	5	6½
Asphalt Concrete	2.5 inches	3.0 inches	3.5 inches
Class 2 Aggregate Base, R=78 min.	5.0 inches	5.0 inches	6.0 inches

156. To have the selected pavement sections perform to their greatest efficiency, it is very important that the following items be considered:

a. Properly scarify and moisture condition the upper 8 inches of the subgrade soil and compact it to a minimum of 95% of its maximum dry density, at a moisture content 1 to 3% over the optimum moisture content for the soil.

b. Provide sufficient gradient to prevent ponding of water.

c. Use only quality materials of the type and thickness (minimum) specified. All aggregate base and subbase must meet Caltrans Standard Specifications for Class 2 materials, and be angular in shape. All Class 2 aggregate base should be ¾ inch maximum in aggregate size.

d. Compact the base and subbase uniformly to a minimum of 95% of its maximum dry density.

e. Use ½ inch maximum, Type “A” medium graded asphaltic concrete. Place the asphaltic concrete only during periods of fair weather when the free air temperature is within prescribed limits by Cal Trans Specifications.

f. Place ¼ gallon per square yard of SG-70 prime coat over the aggregate base section, prior to placement of the asphaltic concrete.

g. **Porous pavement systems which consist of porous paving blocks, asphaltic concrete or concrete are generally not recommended due to the potential for**

saturation of the subgrade soils and resulting increased potential for a shorter pavement life. At a minimum, porous pavement systems should include a layer of Mirafi HP370 geotextile fabric placed on the subgrade soil beneath the porous paving section. These pavement systems should only be used with the understanding by the Owner of the increased potential for pavement cracking, rutting, potholes, etc.

h. Maintenance should be undertaken on a routine basis.

PLAN REVIEW

157. We respectfully request an opportunity to review the project plans and specifications as they continue to change and evolve during final preparation and before bidding to ensure that the recommendations of this report have been included and to provide additional recommendations or conduct additional fieldwork, if needed. These plan review services are also typically required by the reviewing agency. Misinterpretation of our recommendations or omission of our requirements from the project plans and specifications may result in changes to the project design during the construction phase, with the potential for additional costs and delays in order to bring the project into conformance with the requirements outlined within this report. Services performed for review of the project plans and specifications are considered “post-report” services and billed on a “time and materials” fee basis in accordance with our latest Standard Fee Schedule.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. This study was prepared specifically for the 1440 Foundation and for the specific project and location described in the body of this report. This report and the recommendations included herein should be utilized for this specific project and location exclusively. This Geotechnical Investigation should not be applied to nor utilized on any other project or project site. Please refer to the ASFE "Important Information about Your Geotechnical Engineering Report" attached with this report.

2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be provided.

3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field.

4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of our control. This report should therefore be reviewed in light of future planned construction and then current applicable codes. This report should not be considered valid after a period of two (2) years without our review.

5. This report was prepared upon your request for our services in accordance with currently accepted standards of professional geotechnical engineering practice. No warranty as to the contents of this report is intended, and none shall be inferred from the statements or opinions expressed.

6. The scope of our services mutually agreed upon for this project did not include any environmental assessment or study for the presence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site.

Important Information about Your

Geotechnical Engineering Report

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

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

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

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e-mail: info@asfe.org www.asfe.org

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APPENDIX A

Regional Site Map
Site Map Showing Test Borings
Boring Log Explanation
Log of Test Borings
Atterberg Limits
Direct Shear Test Results
Keyway Detail
Surcharge Pressure Diagram
Typical Retaining Wall Drain Detail

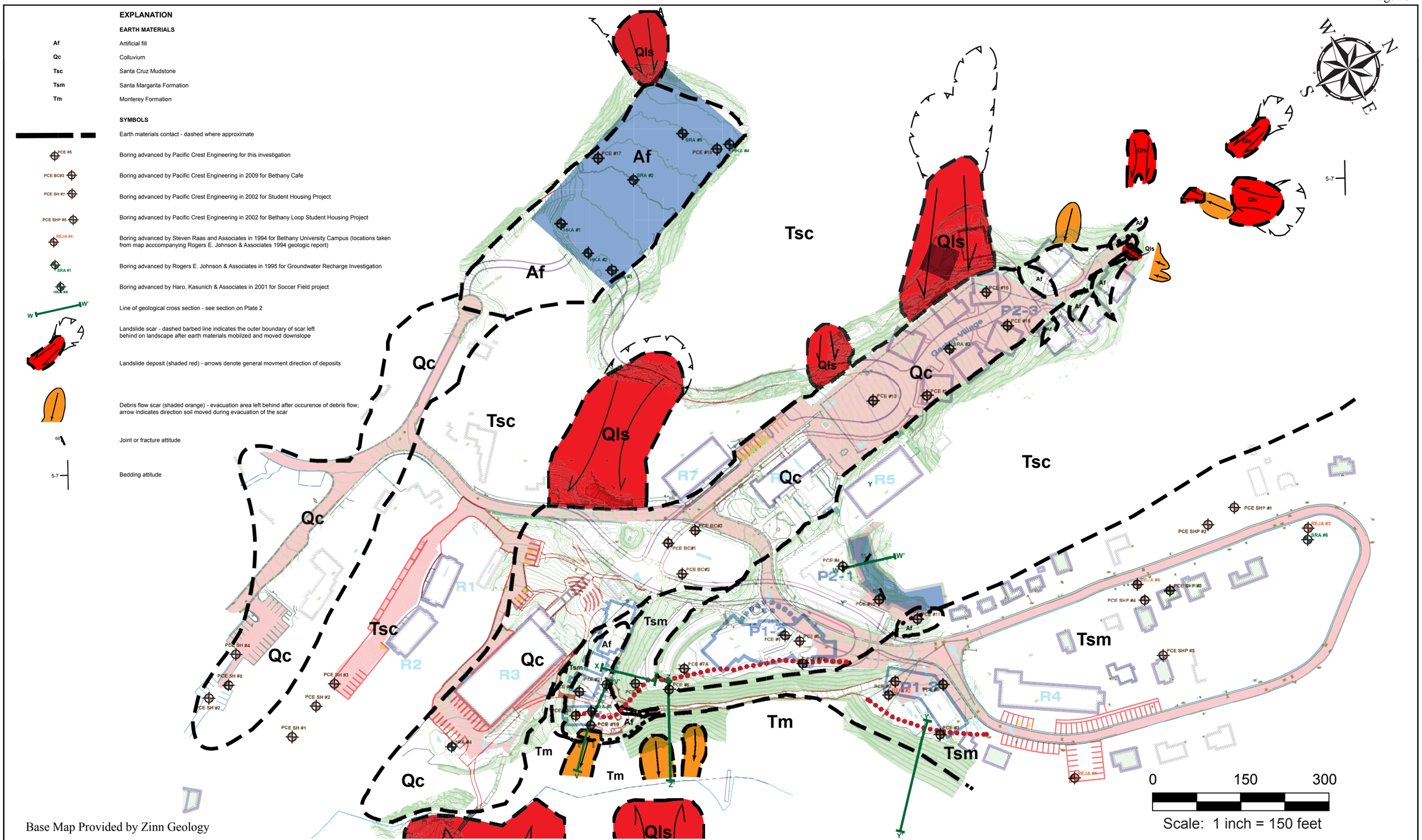
feet 3000
km 1

0 1067 ft.
Approximate Scale

N

Base Map from Google Maps

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Regional Site Map** 1440 Growth Center Scotts Valley, California	Figure No. 1 Project No. 1404 Date: 4/23/14



Base Map Provided by Zinn Geology

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Site Map Showing Test Boring Locations** 1440 Growth Center Scotts Valley, California	Figure No. 2 Project No. 1404 Date: 4/23/14

UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2488 (Modified)

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN #200 SIEVE SIZE	**GRAVELS** MORE THAN HALF OF COARSE FRACTION IS LARGER THAN #4 SIEVE	**CLEAN GRAVELS** (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravels-sand mixtures, little or no fines
		GRAVELS (MORE THAN 12% FINES)	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN #4 SIEVE	**CLEAN SANDS** (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS (MORE THAN 12% FINES)	SM	Silty sands, sand-silt mixtures, non-plastic fines
			SC	Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN #200 SIEVE SIZE	**SILTS AND CLAYS** LIQUID LIMIT IS LESS THAN 35%		ML	Inorganic silts and very fine clayey sand silty sands, with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly, sand, silty or lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS BETWEEN 35% AND 50%		MI	Inorganic silts, clayey silts and silty fine sands of intermediate plasticity
			CI	Inorganic clays, gravelly/sandy clays and silty clays of intermediate plasticity
			OI	Organic clays and silty clays of intermediate plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Organic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils

BORING LOG EXPLANATION

Depth, ft.	Sample No. and Type	Symbol	SOIL DESCRIPTION	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density, p.c.f.	Moisture % of Dry Wt.	MISC. LAB RESULTS
1			← Ground water elevation	NOTE: All blows/foot are normalized to 2" outside diameter sampler size					
2	1-1 L		← Soil Sample Number ← Soil Sampler Size/Type L = 3" Outside Diameter M = 2.5" Outside Diameter T = 2" Outside Diameter ST = Shelby Tube BAG = Bag Sample						
3									
4									
5									

RELATIVE DENSITY

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

CONSISTENCY

SILTS AND CLAYS	BLOWS/FOOT
VERY SOFT	0-2
SOFT	2-4
FIRM	4-8
STIFF	8-16
VERY STIFF	16-32
HARD	OVER 32

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Boring Log Explanation** 1440 Growth Center Scotts Valley, California	Figure No. 3 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 6" SS BORING NO. 1

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07503 Long: -121.99345	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1, 2	1-1 L		Yellowish brown (10YR 5/6) and very dark grayish brown (10YR 3/2) clayey SAND, very fine grained, poorly graded, slightly sticky, blocky competent, well indurated sandstone near 2 ½', sandstone is surrounded by a sandy clay matrix, moist, very loose.	SC	3		75.3	31.5	Qu = 0.6 ksi
3, 4	1-2 T		Very dark gray (10YR 3/1) & dark yellowish brown (10YR 3/6) grading to mottled very dark gray (10 YR 5/1), and yellowish brown (10YR 5/6) CLAY, trace very fine grained sand, intermediate plasticity, trace rootlets, moist, firm.	CI	8	16		29.6	
5, 6, 7, 8, 9	1-3 L		WEATHERED SANTA CRUZ MUDSTONE BEDROCK; Described as mottled very dark gray (10YR 3/1), gray (10YR 5/1), and yellowish brown (10YR 5/6) Sandy CLAY, very fine grained sand, low plasticity, massive, friable, damp, hard.	Tsc	42		86.9	34.5	Qu = 2.3 ksi
10, 11	1-4 T		Color change to mottled gray (10YR 5/1) and yellowish brown (10YR 5/6) , slightly more competent than the previous sample, damp, very hard.		50/6"			25.9	
12, 13, 14			Slow, very hard drilling conditions.						
15, 16, 17, 18, 19	1-5 L		Mudstone is significantly more competent than the previous sample, approximately 50% recovery, other 50% is slough, slight increase in sand content, slightly damp, very dense.		50/6"		98.3	23.1	Qu = 2.7 ksi
20, 21	1-6 T 1-7 T		Slough with trace mudstone chips, damp, very dense. Predominately gray (10YR 5/1), competent mudstone, "poker chipped", dry, very hard.		40/3" 50/4"			22.9 9.7	
22, 23, 24			Boring terminated at 21 feet. No groundwater encountered.						

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 4 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 6" SS BORING NO. 2

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07553 Long: -121.99292	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			AC: 4" AB:4"						
1 2 3 4	2-1 L 2-2 T		FILL: Mottled very dark brown (10 YR 3/2), dark brown (10YR 3/3), brownish yellow (10YR 6/6), and very pale brown (10YR 7/3) CLAY, trace very fine grained sand, trace angular shaped gravels up to ½" in diameter, very small mica flakes, intermediate plasticity, damp, very hard Color change to mottled gray (10YR 5/1 & 6/1) and brownish yellow (10YR 6/6), slight increase in sand content, slightly cemented, damp, hard	CI	50/4" 40	16	107.8	14.8 17.4	Qu = 2.7 ksi
5 6 7 8 9	2-3 L		NATIVE: Mottled very dark grayish brown (10YR 3/2), dark brown (10YR 3/3), brownish yellow (10 YR 6/6), and very pale brown (10YR 7/3) silty SAND, trace binder, very fine grained, poorly graded, very small mica flakes scattered throughout the sample, sample resembles very weathered conglomerate, slightly damp to dry, medium dense	SM	18		87.4	23.6	
10 11 12 13 14	2-4 T		Mottled greenish gray (GLEY1 10Y 5/1), very dark gray (GLEY1 3/N), and grayish green (GLEY1 5G-/2 5/2), CLAY, low to intermediate plasticity, silty, very small mica flakes scattered throughout the sample, discontinuous lenses of silty sand scattered throughout the sample, trace sub-angular shaped claystone clasts randomly distributed throughout the sample, damp stiff	CL-CI	12	15		26.2	
15 16 17 18 19	2-5 L		Very dark gray (10YR 3/1) and dark brown (10YR 3/3) CLAY, silty, intermediate plasticity, very small mica flakes scattered throughout the sample, trace sub-angular shaped very fine grained gravels, trace rootlets/organics randomly distributed throughout the sample, angular shaped mudstone gravels up to ½" in diameter, damp, hard	CI	35	20	93.8	25.8	Qu = 1.8 ksi
20 21 22 23 24	2-6 T		Mottled dark gray (10YR 4/1), very pale brown (10YR 7/3), and yellowish brown (10YR 5/6) Clayey SAND, very fine grained, slightly sticky, sub-angular shaped wx claystone clasts up to ½" in diameter, clasts resemble wx mudstone bedrock, damp, dense	SC	47			27.8	

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 5 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 6" SS BORING NO. 2

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			SANDSTONE BEDROCK; Described as yellow (2.5Y 7/6) SAND, fine grained, clean, massive, friable, poorly graded, very small mica flakes scattered throughout the sample, manganese oxide staining near 26', damp, very, dense.	SP					
25	2-7 L				50/4"		99.6	24.4	
26	2-8 T		Trace manganese oxide staining randomly distributed throughout the sample, damp, very loose.		50/6"			8.8	
27									
28									
29									
30	2-9 L		Sample slipped out of liner & sampler. Bagged color change to light yellowish brown (25Y 6/4), slight increase in moisture content, moist, very dense.		50/3"			22.2	
31									
32									
33									
34	2-10 T		Minimal recovery. Groundwater present at bottom of hole, wet, very dense.						
35			▽		50/2"			19.9	
36			Boring terminated at 35 1/2 feet. Groundwater initially encountered at 35 1/2 feet.						
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 6 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 6" SS BORING NO. 3

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07538 Long: -121.99273	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	3-1 L		Brown (7.5YR 4/3) Silty SAND, trace binder, fine grained, poorly graded, very small mica flakes scattered throughout the sample, angular to sub-angular shaped mudstone clasts up to 1" in diameter near 2 ½', damp, medium dense.	SM					Direct Shear: C_p = 20 psf ϕ_p = 39° C_r = 10 psf ϕ_r = 38°
2					27				
3	3-2 T		Sub-rounded chert pebbles up to ¼" in diameter randomly distributed throughout the sample, damp, medium dense.						
4			WEATHERED MUDSTONE BEDROCK; Described as mottled pale brown (10YR 6/3) and brownish yellow (10YR 6/6) Silty SAND, very fine grained, liesengang banding, poorly graded, friable, massive, very small mica flakes scattered throughout the sample, damp, medium dense.	SM	24			33.7	Direct Shear: C_p = 20 psf ϕ_p = 39° C_r = 10 psf ϕ_r = 38°
5	3-3 L								
6					50/5"				
7			Slightly damp, very dense.						
8									
9									
10	3-4 T		Color change to yellowish brown (10YR 5/4), sample is significantly more wx and friable than the previous sample, vertical clay filled lense spanned the entire sample, damp, medium dense.						
11					17			47.5	
12									
13									
14									
15	3-5 L		Mudstone is significantly more competent and less friable than the previous sample, moderately competent sub-angular shaped clasts up to 1½" in diameter present in the shoe, slight increase in moisture content, damp, very dense.						
16					50/2"		78.0	40.2	Qu = 2.2 ksi
17									
18			Slow, very hard drilling to 20'.						
19									
20	3-6 T		Color change to pale brown (10YR 6/3) and dark yellowish brown (10YR 4/4), mudstone is extremely wx, fairly to moderately competent clasts present within the sample, damp, firm, (highly fractured).						
21					8			40.8	
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 7 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 6" SS BORING NO. 3

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	3-7 L		Grayish brown (10YR 5/2) and yellowish brown (10YR 5/8) Silty SAND, resulting from highly fractured and wx mudstone bedrock, very fine grained, poorly graded, very small mica flakes scattered throughout the sample, damp, medium dense.	SM					Direct Shear: C_p = 920 psf ϕ_p = 46° C_r = 740 psf ϕ_r = 43°
26					29				
27									
28									
29									
30	3-8 T		More competent than the previous sample, highly fractured, damp, very loose.						
31					62			39.9	
32									
33									
34									
35	3-9 L		Color change to dark gray (10YR 6/1), damp, very dense.						
36					50/2"		90.9	26.0	
			Boring terminated at 36 feet. No groundwater encountered.						
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 8 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 4" SS BORING NO. 4

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07543 Long: -121.99354	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	4-1 L		Mottled dark gray (10YR 4/1), yellowish brown (10YR 5/6), and white (10YR 8/1) Silty SAND resulting from weathered and fractured mudstone bedrock, very fine grained, poorly graded, very small mica flakes scattered throughout the sample, sub-angular shaped, moderately competent clasts noted, slightly damp, dense.	SM					Direct Shear: C_p = 290 psf ϕ_p = 37° C_r = 260 psf ϕ_r = 36°
2									
3	4-2 T				31				
4			MUDSTONE BEDROCK; Described as pale brown (10YR 6/3) and brownish yellow (10YR 6/6) Silty SAND/ Sandy SILT, very fine grained, poorly graded, very small mica flakes scattered throughout the sample, massive, sample "poker chipped" during sampling, dry, dense.	SM/ ML	42			27.5	
5	4-3 L				50/6"		68.5	32.9	
6			More competent than the previous sample, slightly damp to dry, very dense, (sample is mostly slough).						
7									
8									
9	4-4 T		Liesengang banding, dry, very hard.		50/6"			31.4	
10			Boring terminated at 9 1/2 feet. No groundwater encountered.						
11									
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 9 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 6" SS BORING NO. 5

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07507 Long: -121.99355	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	5-1 L		WEATHERED MUDSTONE BEDROCK; Described as mottled dark grayish brown (10YR 4/2) and yellowish brown (10YR 5/8) Silty SAND, very fine grained, poorly graded, massive, highly fractured, liesengang banding, trace rootlets, very small mica flakes scattered throughout the sample, slightly damp, medium dense.	SM					
2					14		81.0	30.6	
3	5-2 T								
4			Mudstone becomes more competent with depth, damp, loose.		8			33.0	
5	5-3 L		Mottled very dark gray (10YR 3/1) and dark brown (7.5YR 3/3) CLAY, intermediate plasticity, very small mica flakes scattered throughout the sample, trace rootlets randomly scattered distributed throughout the sample, damp, very stiff, (trace pockets of reddish yellow very fine grained Silty SAND).	CI					
6					18	22	98.3	26.5	Qu = 1.6 ksi
7									
8									
9									
10	5-4 T		Mottled pale brown (10YR 6/3), yellowish brown (10YR 5/8) and greenish gray (10GY 6/1) Silty SAND resulting from highly wx & fx mudstone bedrock, very fine grained, poorly graded, massive, friable, planes of very dark gray (3/N) clay with rootlets scattered throughout the sample, damp, dense.	SM					
11					31			25.4	
12									
13									
14									
15	5-5 L		Color change to mottled strong brown (7.5YR 4/6) and yellowish brown (10YR 5/6 & 5/8), lack of clay vein, lack of rootlets and organics, damp, very dense.		50/6"		95.4	22.2	
16									
17									
18									
19									
20	5-6 T		Color change to mottled gray (10YR 5/1) and yellowish brown (10YR 5/6), slight increase in fines content, damp, very dense.		50/6"			22.9	
21									
22			Boring terminated at 21 feet. No groundwater encountered.						
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 10 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 4" SS BORING NO. 6

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07443 Long: -121.99365	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			WEATHERED MUDSTONE BEDROCK; Described as mottled pale brown (10YR 6/3) and yellowish brown (10YR 5/8) SILT, trace very fine grained sand, massive, friable, slightly damp to dry, stiff.	ML					
1	6-1 L								
2					9		80.5	21.7	Qu = 0.3 ksf
3	6-2 T		Liesengang banding, trace rootlets randomly distributed throughout the sample, slightly damp to dry, very stiff.						
4					19			24.5	
5	6-3 L		Highly fractured and weathered bedrock, slightly damp to dry, very stiff.						
6					26			25.1	
7									
8									
9									
10	6-4 T		Lack of liesengang banding, lack of rootlets, sample exhibited "poker chipping," slightly damp to dry, very stiff.						
11					22			27.3	
12									
13									
14									
15	6-5 L		Trace brown (7.5YR 4/4) oxidized patches randomly distributed throughout the sample, slightly damp to dry, hard.						Direct Shear: C_p = 160 psf ϕ_p = 56° C_r = 0 psf ϕ_r = 54°
16					51				
17									
18									
19									
20	6-6 T		Sample is more wx than sample collected at 15', damp, very stiff.						
21					29			26.5	
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 11 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 4" SS BORING NO. 6

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	6-7 L		WEATHERED SANDSTONE BEDROCK; Described as pale brown (10YR 6/3) and yellowish brown (10YR 5/8) Silty SAND, very fine grained, poorly graded, small mica flakes scattered throughout the sample, massive, friable, highly wx and fx, slightly damp to dry, medium dense.	SM					Direct Shear: C_p = 990 psf ϕ_p = 47° C_r = 950 psf ϕ_r = 45°
26					21				
27									
28									
29									
30	6-8 T		More weathered, fractured, and friable than the previous sample, slight decrease in sand content, damp, medium dense.						
31					14			23.9	
32			Boring terminated at 31 1/2 feet. No groundwater encountered.						
33									
34									
35									
36									
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 12 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 BORING DIAMETER 4" SS BORING NO. 7

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	7-1 L		Sampler did not stay plumb while sampling & was drove into the sidewall of the boring. Drilled out 6" to 1½' to straighten the bore hole. Re-sample WEATHERED SANDSTONE BEDROCK; Described as mottled pale brown (10YR 6/3) and yellowish brown (10YR 5/8) Silty SAND, very fine grained, poorly graded, trace rootlets randomly distributed throughout the sample, massive, friable, highly fx & wx. Sampler still driven crooked after cleaning/drilling out the bore hole. Moved boring 10"& started again.	SM	26/1"		66.1	29.5	
5			Boring terminated at 1 1/2 feet. No groundwater encountered.						

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 13 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/6/14 & 2/11/14 BORING DIAMETER 4" SS BORING NO. 7A

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07453 Long: -121.99373	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			No 7A-1 sample recovered. Upon attempting to sample, sampler was driven in the sidewall again due to near surface sandstone bedrock conditions (highly fx). Drilled hole to 2' to straighten it out and resume sampling. Hammer dropped approximately 15 times.	ML					
1	7A-1 L								
2	7A-2 L								
3			WEATHERED SILTSTONE/MUDSTONE BEDROCK; Described as mottled pale brown (10YR 6/3), yellowish brown (10YR 5/8) and gray (10YR 5/1) SILT, trace very fine grained sand, trace rootlets randomly distributed throughout the sample, massive, highly fx & wx, damp, hard.		31		77.3	31.4	
4	7A-3 T								
5	7A-4 L				36			28.5	
			Sample is slightly "poker chipped," slightly damp, hard.						
6			More wx and oxidized than the previous sample, damp, very stiff.		28		75.8	31.3	
7									
8									
9									
10	7A-5 T		Roots and rootlets randomly distributed throughout the sample, damp, very stiff.						
11					27			26.1	
12									
13									
14									
15	7A-6 L		Mottled pale brown (10YR 6/3) and yellowish brown (10YR 5/8) SILT, trace very fine grained sand, very small mica flakes scattered throughout the sample, massive, friable, highly wx and fx siltstone bedrock, slightly damp to dry, very hard.				87.6	24.6	
16					50/6"		90.8	24.3	
17									
18			More wx and friable than the previous sample, slight decrease in sand content, damp, very hard.						
19									
20	7A-7 T								
21					50/5.5"			25.8	
			Boring terminated at 21 feet. No groundwater encountered.						
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 14 Project No. 1404 Date: 2/00/14

LOGGED BY CLA DATE DRILLED 2/11/14 BORING DIAMETER 4" SS BORING NO. 8

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07431 Long: -121.99380	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1, 2			Started at 3' due to very soft near-surface soil conditions.	ML					
3, 4	8-1 L		Dark brown (7.5YR 3/2) and strong brown (7.5YR 5/6) SILT with Sand and Gravel, very fine grained sand, trace rootlets, gravels are angular shaped up to 1" in diameter and a result of highly wx and fx siltstone bedrock, damp, stiff.		11			22.4	
5, 6	8-2 T		WEATHERED SILTSTONE/MUDSTONE BEDROCK; Described as mottled gray (7.5YR 6/1) and reddish yellow (7.5YR 6/8) SILT, trace very fine grained sand, massive, friable, highly fx and wx, slightly damp to dry, stiff.	ML	14			26.5	
7, 8			Drilling resistance increased significantly at 7'. drilling remained slow and hrad to 10'.						
10, 11	8-3 L		Sample is significantly more competent and less wx and fractured than the previous sample, dry, hard.		52		93.5 91.3	24.8 25.1	Qu = 4.7 ksi
13, 14			Drilling resistance continues to increase, rig began lifting between 13' and 14' due to very hard drilling conditions, lifted rig ~2'.						
15, 16	8-4 T		Sample started to "poker chip" due to very hard drilling conditions, slightly damp, very hard.		50/6"			26.9	
17			Slow, very hard drilling to 18', Rig lifting ~2' while drilling, water/moisture on auger from adding water.						
18, 19	8-5 T		Sample is more competent and less friable than the previous sample, slightly damp to dry, very hard.		95			26.7	
20			Boring terminated at 19 1/2 feet. No groundwater encountered.						

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 15 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/18/14 BORING DIAMETER 6" SS BORING NO. 9

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07558 Long: -121.99298	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			AC: 2" AB: 2"						
1	9-1 L		FILL: Mottled brown (10YR 4/3), dark yellowish brown (10YR 4/6), and yellowish brown (10YR 5/4) Silty SAND that grades to SILT, fine to medium grained sub-rounded shaped, poorly graded sand, small mica flakes scattered throughout the sample, damp, medium dense/very stiff.	SM-ML				12.4	63.6% Passing No. 200 Sieve
2					11/16		84.8	28.1	
3	9-2 T		NATIVE: Relict wx fine to medium grained rounded sandstone pebbles randomly distributed throughout the sample, damp, stiff.	SM					
4					9			22.0	
5	9-3 L		NATIVE: Brown (7.5YR 4/3) CLAY with very fine grained sand, low plasticity, very small mica flakes scattered throughout the sample, trace very small oxidized nodes, relict siltstone bedrock present in the shoe, damp, very stiff.	CL					
6					23	12	112.4	17.7	Qu = 3.2 ksi
7			WEATHERED SILTSTONE BEDROCK; Described as....	ML					
8									
9									
10	9-4 T		Mottled light brownish gray (10YR 6/2) and yellowish brown (10YR 5/8) sandy SILT, very fine grained and poorly graded sand, very small mica flakes scattered throughout the sample, massive, friable, slightly damp, very hard.						
11					50/6"			23.2	
12									
13									
14			No sample recovered. Sampler broke, lost sample & shoe in bore hole. Had to abandon boring and drill new location 3' west.						
15	9-5 L				50/5.5"				
16			Boring terminated at 15 1/2 feet. No groundwater encountered.						
17									
18									
19									
20									
21									
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 16 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/18/14 BORING DIAMETER 6" SS BORING NO. 9A

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07553 Long: -121.99292	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			AC: 2" AB: 2"						
1			Straight drill to 15'.	ML					
2									
3									
4									
5									
6									
7									
8			Drilling resistance increased near 7 ½', noted wx siltstone in cuttings near 9'.						
9									
10			Hard drilling from 7' to 15'. Driller noted harder conditions than in B-9.						
11									
12									
13									
14									
15	9A-1 B		No sample recovered; No liners in sampler. Added water; followed with a "L". Moisture from drillers. Bagged sample		50/5"			23.3	
16	9A-2 L		WEATHERED SILTSTONE BEDROCK; Described as mottled light brownish gray (10YR 6/2), and yellowish brown (10YR 5/8) Sandy SILT, very fine grained poorly graded sand, very small mica flakes scattered throughout the sample, massive, friable, slightly damp, very hard.		50/5"		87.0	32.0	
17									
18									
19									
20	9A-3 T		Damp, very hard.						
21			WEATHERED SANDSTONE BEDROCK; Described as dark gray (GLEY1 4/N) very fine grained silty SAND, massive, friable, quartz rich, very small mica flakes scattered the sample, damp, very dense.	SM	50/6"			24.7	
22									
23									
24			Boring terminated at 21 1/2 feet. No groundwater encountered.						

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 17 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/18/14 BORING DIAMETER 6" SS BORING NO. 10

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07548 Long: -121.99354	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			AC: 2" AB: 2"						
1			WEATHERED SILTSTONE BEDROCK; Described as mottled brown (10YR 5/3), light brownish gray (10YR 6/2) and yellowish brown (10YR 5/8) Sandy SILT, trace binder, very fine grained poorly graded sand, small mica flakes scattered throughout the sample, massive, friable, damp, hard.	ML					
2	10-1 L								
3	10-2 T				35		87.4	27.9	Qu = 1.9 ksi
4			Color change to mottled light grayish brown (10YR 6/2) and yellowish brown (10YR 5/8), more competent and less friable than previous sample, started to "poker chip", fractured, slightly damp, hard, (lack of binder).		56			28.3	
5	10-3 L								
6			Sub-vertical oxidized lamina, slightly damp, very hard.		50/4"		91.4	29.9	
7									
8									
9									
10	10-4 T								
11			Color change to mottled brown (10YR 5/3), light brownish gray (10YR 6/2) and yellowish brown (10YR 5/8), trace binder, highly wx, damp, very stiff.		28			31.2	
12									
13									
14									
15	10-5 L		Color change to mottled light brownish gray (10YR 6/2) and yellowish brown (10YR 5/8), composition change to very fine grained silty sand, friable, damp, very dense.		50/5"		86.7	25.0	
16									
17									
18									
19									
20	10-6 T		WEATHERED SANDSTONE BEDROCK; Described as light gray (2.5YR 7/1) SAND, fine to medium grained, sub-rounded shaped, poorly graded, quartz rich, friable, massive, trace lithics, very small mica flakes scattered throughout the sample, slightly damp, very dense.	SP					
21					50/5"			9.1	
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Boring** 1440 Growth Center Scotts Valley, California	Figure No. 18 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/18/14 BORING DIAMETER 6" SS BORING NO. 10

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	10-7 L		WEATHERED SILTSTONE BEDROCK; Described as mottled light yellowish brown (2.5YR 6/3 & 6/4) SAND, fine to medium grained, sub-rounded shaped, poorly graded, quartz rich, trace lithics, massive, friable, slightly damp, very dense.	SP	50/6"		94.4	7.3	
26									
27			Boring terminated at 26 feet. No groundwater encountered.						
28									
29									
30									
31									
32									
33									
34									
35									
36									
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 19 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/18/14 BORING DIAMETER 6" SS BORING NO. 11

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07554 Long: -121.99332	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			AC: 2" AB: 2"						
1	11-1 L		FILL: Brown (7.5YR 4/3) Silty SAND, very fine grained, poorly graded, very small mica flakes scattered throughout the sample, trace rootlets randomly distributed throughout the sample, damp, medium dense.	SM					
2					13		97.3	17.2	
3	11-2 T		Trace fine grained sand, slighty damp, medium dense.						
4					22			10.6	
5	11-3 L		Patch of wx siltstone bedrock near 6', slightly damp, dense.						
6			WEATHERED SILSTONE BEDROCK; Described as mottled light brownish gray (10YR 5/8) SILT, trace very small mica flakes, friable, massive, slightly damp, hard.	ML	52		80.3	31.0	
7									
8									
9			Slow, hard drilling to 10'.						
10	11-4 T		Sample grades to Silty SAND, slightly cemented, slightly damp, very dense.						
11					77			21.1	
12									
13									
14									
15	11-5 L		Composition change to Sandy SILT, more wx than the previous sample, damp, hard.						
16					37		88.0	32.6	
17									
18									
19									
20	11-6 T		Trace sub-vertical lamination approximately 0.5 mm thick, damp, very hard.		50/6"			31.6	
21									
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 20 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/18/14 BORING DIAMETER 6" SS BORING NO. 11

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	11-7 L		WEATHERED SILSTONE BEDROCK; Described as mottled light brownish gray (10YR 6/2) and yellowish brown (10YR 5/8) SILT, trace very small mica flakes, massive, friable, damp, very hard.	ML	50/5"		79.5	26.4	
26			Boring terminated at 25 1/2 feet. No groundwater encountered.						
27									
28									
29									
30									
31									
32									
33									
34									
35									
36									
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 21 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/19/14 BORING DIAMETER 6" SS BORING NO. 12

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07505 Long: -121.99345	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1 2 3 4 5 6 7 8 9	12-1 L 12-2 T 12-3 L		Mottled dark grayish brown (10YR 4/2), yellow (10YR 7/8) and white (10YR 8/1) Sandy CLAY/SILT, high plasticity, very fine grained sand, very small mica flakes, trace rootlets and trace very fine grained rounded chert pebbles, damp, very stiff. Sample is slightly silty, relict rounded siltstone and sandstone gravels up to ¼" in diameter, damp, stiff. Color change to dark grayish brown (10YR 4/2) with brownish yellow (10YR 6/6) oxidation patches, slight increase in sand content, trace angular shaped granitic gravels (~1/8"), slight increase in rootlet content, damp, very stiff.	CH/ MH & OH	16 13 17	23	84.3	25.3 27.2	48.0% Passing No. 200 Sieve Qu = 1.0 ksi Direct Shear: C_p = 0 psf ϕ_p = 44° C_r = 0 psf ϕ_r = 43°
10 11 12 13 14	12-4 T		Mottled dark gray (7.5YR 4/1, strong brown (7.5YR 5/8) and light gray (10YR 7/1) Sandy CLAY, very fine grained, intermediate plasticity, trace rootlets randomly distributed throughout the sample, relict rounded siltstone and sandstone gravels, angular shaped very fine grained sandstone clasts resulting from highly wx and fx sandstone, damp, hard (wx conglomerate?).	CI	31	20		23.8	
15 16 17 18 19	12-5 L		Mottled dark gray (10YR 4/1) and dark yellowish brown (10YR 3/6) CLAY with trace sand, intermediate plasticity, silty, very small mica flakes scattered throughout the sample, sand is quartz rich, sub-angular to sub-rounded shaped and medium to coarse grained, damp, very stiff.	CL	24		84.2	38.2	
20 21 22 23 24	12-6 T		Mottled gray (10YR 5/1), light brownish gray (10YR 6/2), dark grayish brown (10YR 4/2) and brownish yellow (10YR 6/8) SILT, very small mica flakes scattered throughout the sample, friable, massive, highly wx and fx siltstone bedrock, damp, hard.	ML	33			34.6	

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 22 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/19/14 BORING DIAMETER 6" SS BORING NO. 12

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	12-7 L		WEATHERED SILTSTONE BEDROCK; Described as mottled gray (10YR 6/1) and yellowish brown (10YR 6/8) SILT, trace fine very grained sand, massive, friable, highly wx and fx, damp, very hard.	ML	50/3"				
26									
27			Slow, hard drilling to 30'.						
28									
29									
30	12-8 T		Slightly cemented but still friable, trace sub-angular shaped coarse grained quartz clasts randomly distributed throughout the sample, damp, very hard.		50/5"				
31									
32									
33									
34			Slow, hard drilling to 35 '.						
35	12-9 L		WEATHERED SILTSTONE BEDROCK; Described as light yellowish brown (2.5YR 6/4) SAND, clean, medium grained, sub-rounded to rounded shaped, poorly graded, quartz rich, massive, friable, slightly damp to dry, very dense.	ML	50/3"				
36									
37			Boring terminated at 36 feet. No groundwater encountered.						
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 23 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/19/14 BORING DIAMETER 6" SS BORING NO. 13

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07582 Long: -121.9947	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			AC: 3" AB: 2"						
1	13-1 L		FILL: Brown (10YR 4/3) and dark grayish brown (10YR 4/2) Silty SAND with gravel, fine grained with trace medium grains, sub-rounded shaped, poorly graded, very small mica flakes, gravels are sub-angular to sub-rounded shaped, quartz and siltstone and up to ¼ " in diameter, damp, loose, (organic rich).	SM					
2					9		89.9	25.8	
3	13-2 T								
4			NATIVE;Very dark grayish brown (10YR 3/2) CLAY, silty, low to intermediate plasticity, trace oxidation patches and trace coarse grained angular quartz gravels randomly distributed throughout the sample, damp, soft.	CI	4	14		24.3	
5	13-3 L								
6			Wx rounded siltstone gravel approximately ½" in diameter near 6', trace to medium grained sand, damp, stiff.		10		96.4	24.6	Qu = 0.8 ksi
7									
8									
9									
10	13-4 T		Color change to brown (10YR 4/3) and light yellowish brown (10YR 6/4), sample begins grading to a silt near 11', slight increase in gravel content, angular siltstone clasts up to ½" in diameter present within the sample, damp, stiff.						
11					10			27.5	
12									
13									
14									
15	13-5 L		Color change to dark brown (10YR 3/3), trace pockets of increased moisture randomly distributed throughout the sample, damp, very stiff.						
16					28		97.9	24.9	Qu = 0.7 ksi
17									
18									
19									
20	13-6 T		Mottled grayish brown (10YR 5/2) and dark yellowish brown SILT, trace very fine grained, sample is a result of highly wx siltstone bedrock, trace brown (7.5YR 4/3) clay patches randomly distributed throughout the sample, micaceous, fx, massive, damp, very stiff.	ML					
21					21			30.5	
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 24 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/19/14 BORING DIAMETER 6" SS BORING NO. 13

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	13-7 L		Mottled grayish brown (10YR 5/2) and dark yellowish brown (10YR 4/6) SILT resulting from highly wx and fx siltstone bedrock, massive, friable, damp, hard.	ML					
26			Brown (10YR 4/3) SAND with Silt, fine grained, poorly graded, sub-rounded shaped, quartz rich, very small mica flakes scattered throughout the sample, moist, medium dense.	SP-SM	27		98.3	25.4 21.3	
27									
28									
29									
30	13-8 T		Driller noted groundwater at 30'. Color change to dark grayish brown (10YR 4/2) with dark yellowish brown oxidation patches, rounded chert pebbles ~¼" in diameter near 31', wet, dense.						
31			WEATHERED MUDSTONE BEROCK; Described as very dark gray SILT, micaceous, massive, friable, highly fx and and wx, damp, very hard.	ML	69			27.9	
32									
33									
34									
35	13-9 L		Groundwater gone at 35'. Not full recovery. Damp, very hard.		50/4"		89.4	23.0	
36	13-10 T		Slow, very hard drilling to 36 ½'. Auger stopped advancing at 36 ½'. Minimal recovery, dry, very hard.		50/1"			18.0	
37			Boring terminated at 37 feet. Groundwater seep zone noted at 30 feet. Free standing ground water encountered at 36 feet.						
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 25 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/19/14 BORING DIAMETER 6" SS BORING NO. 14

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07608 Long: -121.99458	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			AC: 2" AB: 2"						
1	14-1 L		Light gray (10YR 7/2) and yellowish brown SILT resulting from highly wx and fx siltstone bedrock, trace very fine grained sand, micaceous, massive, friable, damp, very hard.	ML					
2					50/5"		88.1	26.8	Qu= 2.5 ksi
3	14-2 T		Trace brown (10YR 4/3) wx clay veins randomly distributed throughout the sample, damp, very hard.						
4					69			26.0	
5	14-3 L		Composition change to very fine grained Silty SAND, minimal recovery, mostly slough, massive, friable, damp, very dense.						
6					50/3"		99.2	22.1	
7									
8									
9									
10	14-4 T		Slight decrease in coarseness of sand, sample began to "poker chip", damp, very dense.						
11					50/6"			20.5	
12			Boring terminated at 11 feet. No groundwater encountered.						
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 26 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/19/14 BORING DIAMETER 6" SS BORING NO. 15

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07658 Long: -121.99496	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			AC: 3" AB: 3"						
1	15-1 L		FILL; Black (10YR 2/1) CLAY, organic rich, low plasticity, very small mica flakes scattered throughout the sample, trace rootlets, trace fine to medium grained sand, damp, stiff.	CL					
2					9		92.9	24.2	Qu= 1.0 ksi
3	15-2 T		Trace very fine grained angular shaped siltstone/mudstone gravels near 4', lack of rootlets, damp, stiff.						
4					7	11		27.3	
5	15-3 L		NATIVE; Very dark gray (10YR 3/1) and greenish gray (GLEY10Y 6/1) CLAY with Gravels, gravels concentrated near 6 ½' and consist of angular to sub-angular shaped siltstone/claystone clasts up to ¼" in diameter, trace fine to medium grained sand, damp, stiff.	CL					
6					11		93.7	27.3	Qu= 0.5 ksi
7									
8									
9									
10	15-4 T		WEATHERED SANDSTONE BEDROCK; Described as light brownish gray (10YR 6/2) and brownish yellow (10YR 6/8) Silty SAND, trace binder, very fine to fine grained, poorly graded, very small mica flakes scattered throughout the sample, quartz rich, massive, friable, trace very dark gray (10YR 3/1) clay veins and patches randomly distributed throughout the sample, damp, very dense.	SM					
11					53			14.1	
12									
13									
14									
15	15-5 L		WEATHERED SANDSTONE BEDROCK; Described as light gray (2.5Y 7/2) SAND, clean, quartz rich, fine grained with trace medium grains, sub-rounded to rounded shaped, dry, very dense.	SM	50/6"		101.2	4.9	
16									
17									
18									
19									
20	15-6 T		Brown (10YR 4/3 & 5/3) staining near 21 ½'. Stained portions contain a trace amount of binder, slightly damp to dry, very dense.						
21					50/3"			8.1	
22			Boring terminated at 21 1/2 feet. No groundwater encountered.						
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 27 Project No. 1404 Date: 2/00/14

LOGGED BY CLA DATE DRILLED 2/19/14 BORING DIAMETER 6" SS BORING NO. 16

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07662 Long: -121.99465	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			AC: 3" AB: 3"						
1 2 3	16-1 L 16-2 T		Very dark grayish brown (10YR 3/2) with dark yellowish oxidation patches (10YR 4/6) CLAY, trace very fine grained sand, very small mica flakes scattered throughout the sample, trace sub-rounded siltstone clasts randomly distributed throughout the sample, damp, stiff.	CL	14		87.2	29.2	Qu= 1.2 ksi
4 5 6 7 8 9	16-3 L		Sample grades to Clayey SAND, very fine to fine grained, poorly graded, increase in gravel content, gravels typically angular to sub-rounded quartz, granitic and siltstone up to ¼" in diameter, damp, loose. Color change to brown (10YR 5/3), decrease in clay content, sample coarsens with depth, sand is quartz rich, slight increase in gravel content, damp, medium dense.	SC	6 11			26.9	Direct Shear: C_p = 670 psf ϕ_p = 34° C_r = 660 psf ϕ_r = 31°
10 11 12 13 14	16-4 T		Mottled light brownish gray (10YR 6/2) and yellowish brown (10YR 5/8) SILT with trace gravels resulting from highly wx and fx siltstone bedrock, high plasticity, trace brown (10YR 4/3) clay patches randomly distributed throughout the sample, siltstone clasts up to 1" in diameter, angular shaped, fairly competent, damp, stiff.	MH & OH	19	19		31.8	
15 16 17 18 19 20 21 22 23 24	16-5 L 16-6 T		Brown (7.5YR 4/3) and strong brown (7.5YR 4/6) CLAY, intermediate plasticity, very small mica flakes scattered throughout the sample, wx very fine grained sandstone gravels randomly distributed throughout the sample, gravels are angular to sub-rounded shaped and up to ¼" in diameter, moist to very moist, stiff. ▽ Color change to mottled dark gray (7.5YR 4/1) and dark brown (7.5YR 3/4), increase in moisture content, slight decrease in gravel content, very moist, firm.	CI	11 5	16	85.3	35.4 34.0	Qu= 0.2 ksi

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 28 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/19/14 BORING DIAMETER 6" SS BORING NO. 16

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	16-7 L		Mottled dark gray (7.5YR 4/1) & dark brown (7.5YR 3/4) CLAY, silty, intermediate plasticity, angular to sub-angular siltstone clasts up to ½" in diameter, mica flakes, damp to moist, hard.	CL					
26			WEATHERED SANDSTONE BEDROCK; Described as light gray (2.5YR 7/2) SAND with clay, slightly sticky, very fine to fine grained sand, poorly graded, quartz rich, massive, friable, damp, very dense.	SC	50/5"		98.5	23.0	
27									
28									
29									
30	16-8 T		Composition change to SAND, clean, trace oxidation patches/slightly oxidized near 3½', trace manganese oxide staining randomly distributed throughout the sample, damp to moist, very dense.		50/ 5.5"			18.6	
31									
32									
33									
34			Added rod prior to sampling. ▼						
35	16-9 L		Wet, very dense.				107.8	21.6	
36					50/4"		103.4	24.4	
37			Boring terminated at 36 feet. Groundwater encountered near 20 feet (seep zone?). Measured at 34 1/2 feet at the end of drilling.						
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 29 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/21/14 BORING DIAMETER 6" SS BORING NO. 17

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 36.65507 Long: -121.72890	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	17-1 L		FILL: Mottled very dark grayish brown (10YR 2/2), grayish brown (10YR 3/2), and grayish brown (10YR 5/2) sandy CLAY, very fine to fine grained sand, trace rootlets and trace sub-angular shaped granitic gravels, very small mica flakes, damp, stiff.	CL					
2					11		104.3	21.1	Qu= 1.0 ksi
3	17-2 T		Sample coarsens with depth and grades to Clayey SAND, slight increase in gravel content, gravels are granitic, quartz & sandstone, angular to sub-angular shaped & up to ¼" in diameter, low plasticity, damp, medium dense.	SC					39.8% Passing No. 200 Sieve
4					13	14		14.3	
5	17-3 L		Color change to mottled very dark grayish brown (10YR 2/2) and grayish brown (10YR 3/2), slight increase in gravel content, sample coarsens slightly with depth.						
6					19		113.1	12.0	
7									
8									
9									
10	17-4 T		FILL: Black (10YR 2/1) & gray (2.5Y 5/1) CLAY, intermediate plasticity, trace very fine grained sand. trace oxidation patches randomly distributed throughout the sample, trace angular to sub-angular shaped quartz and siltstone gravels up to ¼" in diameter randomly distributed throughout the sample, trace very wx clasts of mottled gray (2.5Y 6/1) and brownish yellow (10YR 6/8) siltstone bedrock, damp, very stiff, (piece of thread/twine noted in sample).	CI					
11					20	15		20.3	
12									
13									
14									
15	17-5 L		Color change to black (10YR 2/1), slight increase in sand and gravel content, relict very fine grained sandstone gravels up to ½" in diameter, large chunk of highly wx and fx siltstone bedrock present near 16½' and in the shoe, damp, hard.						
16					33		98.9	23.0	Qu= 1.5 ksi
17									
18									
19									
20	17-6 T		Dark red mottling (2.5YR 3/6) scattered throughout the sample, sample fines with depth as gravels and sand grade away by 21½', damp, very stiff.						
21					21			29.9	
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 30 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/21/14 BORING DIAMETER 6" SS BORING NO. 17

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	17-7 L		WEATHERED SANDSTONE BEDROCK; Described as mottled pale brown (10YR 6/3) and brownish yellow (10YR 6/6) Silty SAND, very fine grained, poorly graded, very small mica flakes scattered throughout the sample. highly wx and fx, massive, friable, damp, medium dense.	SC	23		81.3	38.8	Qu= 1.4 ksi
26									
27									
28									
29									
30	17-8 T		Slightly more competent than the previous sample, slightly damp, dense.						
31					37			29.5	
32			Increase in drilling resistance from 31½' to 35'.						
33									
34									
35	17-9 L		Composition change to Sandy SILT/SILT, overall sample is more wx than previous sample, more competent clasts present within the sample, damp very stiff.						
36					21		85.6	35.9	
37			Decrease in drilling resisntance near 37'. Driller noted it felt like he was drilling in fat clay. Drilling resistance increased near 39-40'. Added lots of water.						
38									
39									
40	17-10 T		Color change to mottled gray (2.5Y 5/1 & 6/1) and dark yellowish brown (10YR 3/4), more competent than the previous sample, but still wx and friable, damp, very hard.		50/ 5.5"			27.9	
41									
42									
43			Drilling resistance stayed moderately high from 40-45'.						
44									
45	17-11 L		WEATHERED MUDSTONE BEDROCK; Described as gray (2.5y 5/1) SILT with very fine grained sand, very small mica flakes scattered throughout the sample, massive, friable, damp, very hard.	ML					
46					50/4"		92.6	28.8	Qu= 2.9 ksi
47			▼						
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 31 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/21/14 BORING DIAMETER 6" SS BORING NO. 17

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
49			Added a 5' rod. ▽	ML					
50	17-12 T		WEATHERED MUDSTONE BEDROCK; Described as gray (2.5y 5/1) SILT, micaceous, massive, friable, slightly damp, to dry, very hard.		50/3"			24.8	
51			Boring terminated at 51 feet. Groundwater encountered near 49 feet. Measured at 47 feet at the end of drilling.						
52									
53									
54									
55									
56									
57									
58									
59									
60									
61									
62									
63									
64									
65									
66									
67									
68									
69									
70									
71									
72									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 32 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/21/14 BORING DIAMETER 6" SS BORING NO. 18

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07579 Long: -121.96643	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	18-1 L		FILL: Brown (10YR 4/3) Clayey SAND with gravel, fine grained, poorly graded, slightly sticky, very small mica flakes scattered throughout the sample, gravels are sub-angular to well rounded granitic, quartz, sandstone and chert & up to ¼" in diameter, very damp, medium dense. Decrease in clay content, increase in gravel content, fractured angualr shaped sandstone clasts up to ½" in diameter.	SC					
2					14		112.8	12.8	25.8% Passing No. 200 Sieve
3	18-2 T								
4					9			17.0	
5	18-3 L		SAND coarsens with depth, trace rounded shaped coarse grained sand near 6½', trace pockets of increased moisture randomly distributed throughout the sample, very damp to moist, very loose.						
6					4		97.0	27.0	
7									
8									
9									
10	18-4 T		Moist, very loose.						
11			NATIVE: Black (10YR 2/1) SILT, organic rich, sample is full of root and rootlets, trace very fine grained sand, intermediate plasticity, very small mica flakes scattered throughout the sample, moist, firm.	MI & OI	7	12		31.8	
12									
13									
14									
15	18-5 L		Mottled pale brown (10YR 6/3) and dark yellowish brown (10YR 4/6) Sandy SILT resulting from highly wx and fx siltstone bedrock, very fine grained sand, very small mica flakes scattered throughout the sample, trace pockets of brown (10YR 4/3) CLAY randomly distributed throughout the sample, damp, stiff.	ML					
16					15		87.3	27.8	Qu= 0.8 ksi
17									
18			Drilling resistance inscreased significantly at 17'. WEATHERED MUDSTONT BEDROCK; Described as ...	SM					
19									
20	18-6 T								
21			Gray (2.5Y 5/1) Silty SAND, very fine grained, poorly graded, very small mica flakes scattered throughout the sample, massive, friable, slightly damp to dry, very dense, (sample is "poker chipped").		50/3"			21.6	
22									
23			Very dense drilling conditions from 21 to 25'.						
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 33 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 2/21/14 BORING DIAMETER 6" SS BORING NO. 18

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	18-7 L		WEATHERED MUDSTONE BEDROCK; Described as gray (2.5Y 5/1) and dark gray (2.5Y 4/1) Silty SAND, very fine grained, poorly graded, micaceous, massive, friable, slightly damp, very dense.	SM					
26					50/4"		91.5	18.8	
27									
28									
29									
30	18-8 T		Wx mudstone from 30 to 30½', wx sandstone from 30½ to 31', damp, very dense.		50/4"			18.9	
31			Boring terminated at 31 feet. No groundwater encountered.						
32									
33									
34									
35									
36									
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 34 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 3/20/14 BORING DIAMETER 6" SS BORING NO. 19

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07503 Long: -121.99345	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	19-1 L		Mottled brown (10YR 4/3), brownish yellow (10YR 5/8), and light gray (10YR 7/2) Silty SAND, very fine grained, poorly graded, trace rootlets and trace clay pockets randomly distributed throughout the sample, sample resembles weathered bedrock, damp, medium dense.	SM					Direct Shear: C_p = 660 psf ϕ_p = 34° C_r = 680 psf ϕ_r = 31°
2					12				
3	19-2 T		Color change to mottled brownish yellow (10YR 5/8), gray (10YR 6/1), and brown (10YR 4/3), slight increase in coarseness of sand, damp, medium dense.						43.8% Passing No. 200 Sieve
4					23			25.7	
5	19-3 L		WEATHERED SANDSTONE BEDROCK; Described as mottled gray (10YR 6/1) and brownish yellow (10YR 5/8) Silty SAND, very fine grained, poorly graded, trace dark gray clay veins randomly distributed throughout the sample, fissile, thinly bedded, trace rip-up clasts randomly distributed throughout the sample, slightly damp, very dense	Tsc					
6					50/6"		95.8	23.1	Qu= 2.4 ksi
7									
8									
9									
10	19-4 T		Slightly finer grained than the previous sample, trace dark yellowish brown patches (10YR 4/6), slightly damp, very dense						
11					63			23.3	
12									
13									
14									
15	19-5 L		Slightly finer grained than the previous sample, on the verge of becoming a silt, slightly damp, very dense		50/5.5"		95.5	22.2	
16									
17									
18									
19			SILTSTONE BEDROCK; Described as greenish gray (GLEY1 5/5GY) and brownish yellow (10YR 5/8) Sandy SILT, very fine grained sand, fissile, small mica flakes scattered throughout the sample, damp, very hard						
20	19-6 T								
21					50/4"			20.4	
22			Boring terminated at 21 1/2 feet. No groundwater encountered.						
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 35 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 3/20/14 BORING DIAMETER 6" SS BORING NO. 20

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07503 Long: -121.99345	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	20-1 L		FILL; Mottled very dark gray and brown (10YR 3/1 & 10 YR 4/3) SILT, trace very fine grained sand, trace rounded and weathered siltstone pebbles up to 1/4 inch in diameter, damp, stiff	ML					
2	20-2 T		Mottled brown (10YR 5/3), yellowish brown (10YR 5/8), and pale brown (10YR 6/3) CLAY, high plasticity, trace binder, trace very fine grained sand, trace weathered siltstone clasts up to 1/2 inch in diameter, damp, stiff	CH	11		85.3	32.2	
3			Sample fines slightly with depth, portions of mottled very dark gray (10YR 3/1) and dark yellowish brown (10YR 3/6) near 4 feet, damp, soft		3	25		29.7	
5	20-3 L		Dark gray (10YR 4/1) and very pale brown (10YR 7/3) CLAY, silty, trace very fine grained sand, black (10YR 2/1) CLAY with roots and bark near 6 feet, trace rounded relict siltstone pebbles, damp, very stiff	CL	18			28.1	
7			NATIVE; Mottled grayish brown (10YR 5/2), brownish yellow (10YR 6/6), dark brown (7.5YR 3/4), and light gray (10YR 7/2) Sandy SILT, very fine grained sand, very small mica flakes, trace rootlets, damp, very stiff, (very weathered silstone bedrock)	ML					
10	20-4 T		SANDSTONE BEDROCK; Described as mottled light gray (10YR 7/2), brownish yellow (10YR 6/6), and yellowish brown (10YR 5/8) Silty SAND, very fine grained, poorly graded, friable, fissile, thinly bedded, slightly dipping, slightly damp to dry, very dense		50/6"			21.0	
15	20-5 L		Sample fines slightly with depth, decrease in oxidation patches with depth, very small mica flakes scattered throughout the sample, slightly damp to dry, very dense		50/3"		105.1	20.8	
20	20-6 T		SILTSTONE BEDROCK; Described as mottled light gray (10YR 7/2), brownish yellow (10YR 6/6), and yellowish brown (10YR 5/8) Sandy SILT, trace highly fractured relatively flat (sub-horizontal) quartz lamina scattered throughout the sample, damp, very hard		50/5"			21.7	
22			Boring terminated at 21 1/2 feet. No groundwater encountered.						

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 36 Project No. 1404 Date: 4/23/14

LOGGED BY CLA DATE DRILLED 3/20/14 BORING DIAMETER 6" SS BORING NO. 21

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07503 Long: -121.99345	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	21-1 L		FILL; Mottled brown (10YR 5/3), yellowish brown (10YR 5/8), and light gray (10YR 7/2) CLAY/SILT, high plasticity, trace sub-angular shaped siltstone pebbles up to 1/4 inch in diameter brick shards near 2 1/2 feet and in the shoe, trace black spots (10YR 2/1), mica flakes, damp, firm	CH-MH & OH	8	29	86.0	28.2	Qu=0.8 ksi
3	21-2 T		Mottled light gray (10YR 7/2) and dark yellowish brown (10YR 3/6 & 4/6) SILT, trace very fine grained sand, very small mica flakes scattered throughout the sample, relict siltstone bedrock structure, thinly bedded, slightly damp, soft	ML	4			35.6	
5	21-3 L		Trace fragments/clasts of more competent siltstone clasts as large as 1 inch in diameter, micaceous, damp to moist, firm		5		78.6	39.9	
7			Very hard drilling from 8 to 8 1/2 feet. Drilling resistancy eased slightly than increased again near 9 feet. Auger advanced approximately 1 inch in 3 minutes near 9 feet						
9	21-4 T		SANDSTONE BEDROCK; Described as dark greenish gray (GLEY1 4/5GY) Silty SAND, very fine grained, poorly graded, micaceous, friable, trace oxidation patches randomly distributed throughout the sample, damp, very dense		50/5"			28.7	
11			Boring terminated at 9 1/2 feet. Groundwater initially encountered near 9 feet and measured at 5 feet at the end of drilling activities.						
12									
13									
14									
15									
16									
17									
18									
19									
20									
21									
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 37 Project No. 1404 Date: 4/23/14

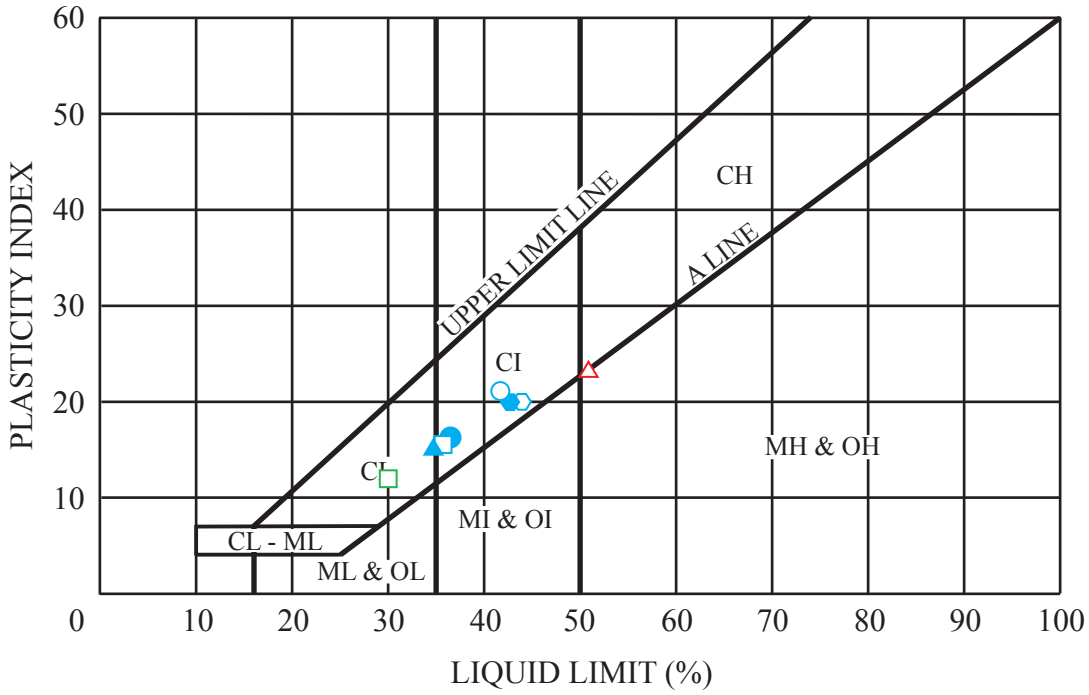
LOGGED BY CLA DATE DRILLED 3/20/14 BORING DIAMETER 6" SS BORING NO. 22

Depth (feet)	Sample No. and Type	Symbol	Soil Description Lat: 37.07503 Long: -121.99345	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
			Loose leaves, dander						
1			Pale brown (10YR 6/3) and yellowish brown (10YR 5/6) Sandy SILT, very fine grained sand, trace rootlets, sub-angular shaped siltstone clasts resulting from highly weathered siltstone bedrock, damp, stiff	ML					
2	22-1 L								
3	22-2 T		Brown (10YR 4/3) present within the sample, damp, firm		10		84.1	32.7	
4					6			32.1	
5	22-3 L		Mottled very dark gray (10YR 3/1), grayish brown (10YR 5/2), and yellowish brown (10YR 5/8) CLAY, silty, intermediate plasticity, trace rootlets, sub-rounded shaped weathered siltstone pebbles up to 1/2 inch in diameter, slightly moist to damp, firm	CL					
6					8	28	79.8	34.2	Qu= 0.8 ksi
7									
8									
9									
10	22-4 T		SILTSTONE BEDROCK; Described as mottled light brownish gray (2.5YR 6/2) and strong brown (7.5YR 5/8) Sandy SILT, very fine grained sand, poorly graded, micaceous, friable, fissile, damp, hard						
11					60			22.2	
12									
13									
14									
15	22-5 L		Slightly coarser grained than the previous sample, damp, very hard		50/4"		101.9	22.5	
16			Boring terminated at 16 feet. No groundwater encountered.						
17									
18									
19									
20									
21									
22									
23									
24									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Log of Test Borings** 1440 Growth Center Scotts Valley, California	Figure No. 38 Project No. 1404 Date: 4/23/14

ATTERBERG LIMITS - ASTM D4318

PLASTICITY CHART



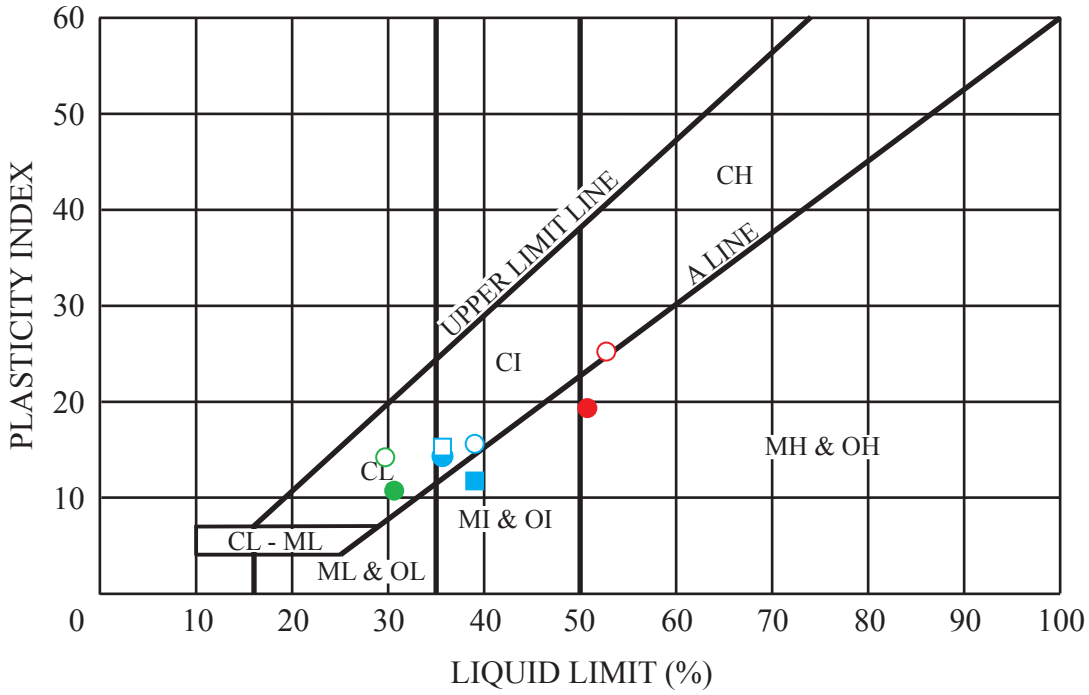
*This chart has been modified to include the intermediate classifications CI, MI and OI for clays and silts with liquid limits between 35 and 50.

SYMBOL	SAMPLE #	LL (%)	PL (%)	PI
●	**1-2**	**37**	**21**	**16**
□	**2-1-1**	**36**	**20**	**16**
▲	**2-4**	**34**	**19**	**15**
⬢	**2-5-1**	**43**	**23**	**20**
○	**5-3-1**	**42**	**20**	**22**
□	**9-3-1**	**30**	**18**	**12**
△	**12-1-1**	**51**	**28**	**23**
⬡	**12-4**	**44**	**24**	**20**

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Atterberg Limits** 1440 Growth Center Scotts Valley, California	Figure No. 39 Project No. 1404 Date: 4/23/14

ATTERBERG LIMITS - ASTM D4318

PLASTICITY CHART



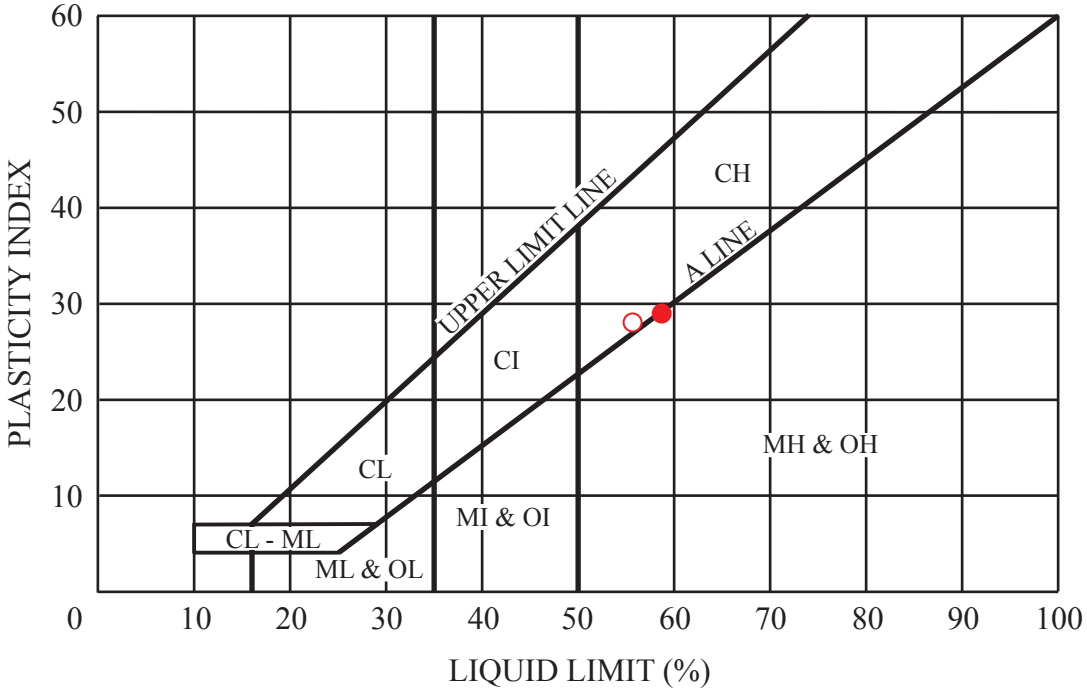
*This chart has been modified to include the intermediate classifications CI, MI and OI for clays and silts with liquid limits between 35 and 50.

SYMBOL	SAMPLE #	LL (%)	PL (%)	PI
●	**13-2**	**36**	**22**	**14**
●	**15-2**	**31**	**20**	**11**
●	**16-4**	**51**	**30**	**19**
○	**16-5-1**	**39**	**23**	**16**
○	**17-2**	**30**	**16**	**14**
□	**17-4**	**36**	**21**	**15**
■	**18-4**	**39**	**27**	**12**
○	**20-2**	**53**	**28**	**25**

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Atterberg Limits** 1440 Growth Center Scotts Valley, California	Figure No. 40 Project No. 1404 Date: 4/23/14

ATTERBERG LIMITS - ASTM D4318

PLASTICITY CHART



*This chart has been modified to include the intermediate classifications CI, MI and OI for clays and silts with liquid limits between 35 and 50.

SYMBOL	SAMPLE #	LL (%)	PL (%)	PI
●	**21-1-1**	**58**	**29**	**29**
○	**22-3-1**	**56**	**28**	**28**

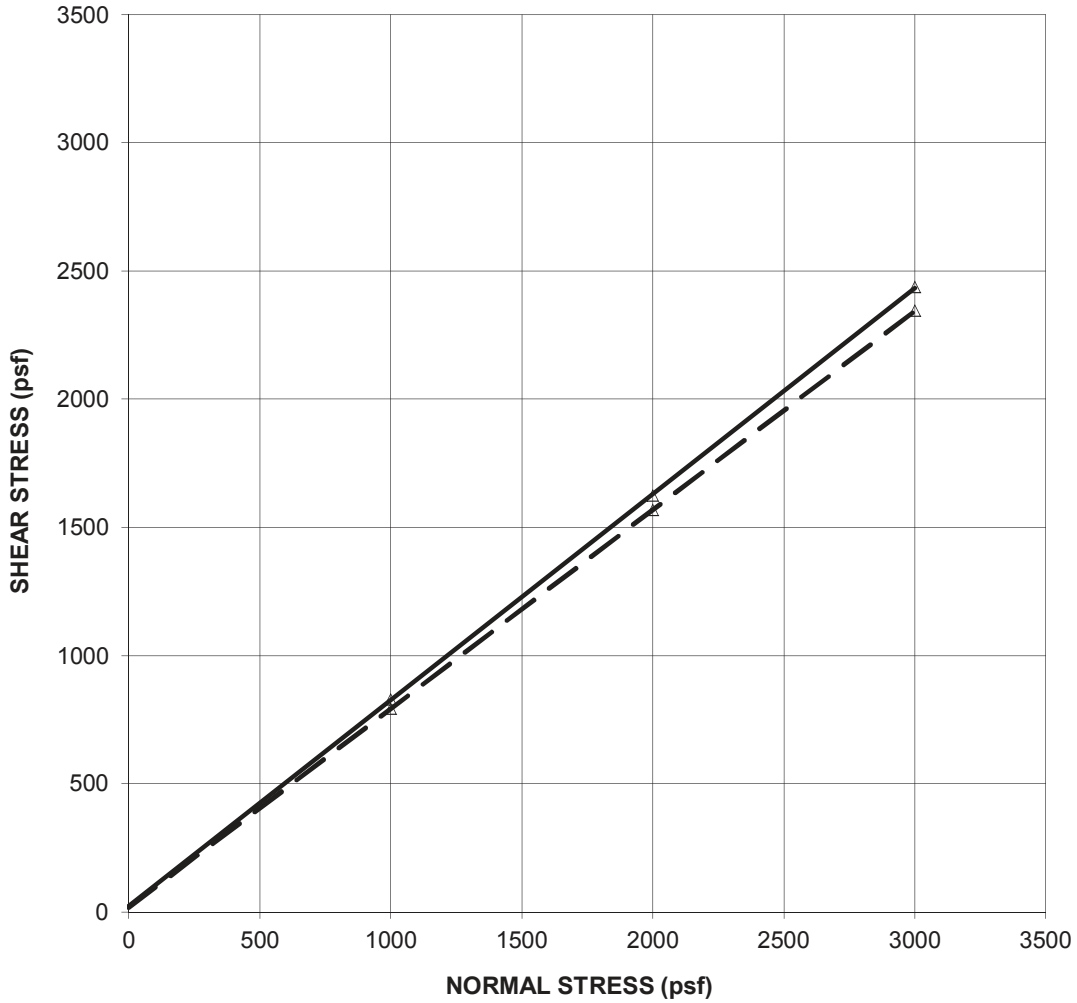
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Atterberg Limits** 1440 Growth Center Scotts Valley, California	Figure No. 41 Project No. 1404 Date: 4/23/14

DIRECT SHEAR TEST - ASTM D 3080

CLIENT:	PACIFIC CREST	DATE:	2/12/14
PROJECT:	RETREAT CENTER - 1404	TESTED:	SSC
SAMPLE DESCRIPTION:	Brown Silty SAND with trace Gravel		

			COHESION (psf)	FRICTION ANGLE
SAMPLE:	3-1-1			
SOIL TYPE:	SM			
		PEAK	20	39
		RESIDUAL	10	38

INITIAL WATER CONTENT:	9.9%
WATER CONTENT AT TEST:	17.0%



Bauldry Engineering, Inc.

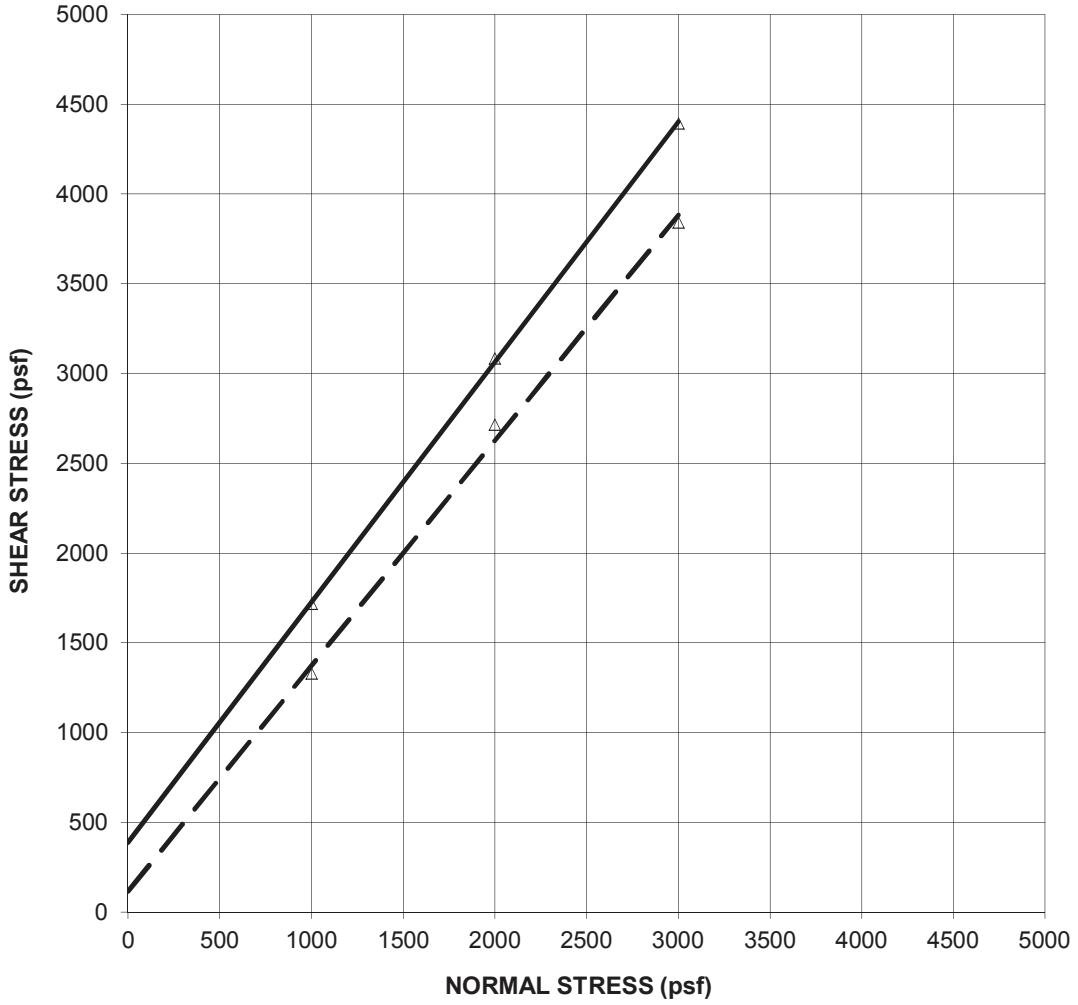
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Direct Shear Test Result** 1440 Growth Center Scotts Valley, California	Figure No. 42 Project No. 1404 Date: 4/23/14

DIRECT SHEAR TEST - ASTM D 3080

CLIENT:	PACIFIC CREST	DATE:	2/12/14
PROJECT:	RETREAT CENTER - 1404	TESTED:	SSC
SAMPLE DESCRIPTION:	Pale Yellow Siltstone, highly fractured		

SAMPLE: 3-3-1		COHESION	FRICTION
SOIL TYPE: (ML)		(psf)	ANGLE
	PEAK	380	53
	RESIDUAL	110	51

INITIAL WATER CONTENT:	35.3%
WATER CONTENT AT TEST:	47.2%



Bauldry Engineering, Inc.

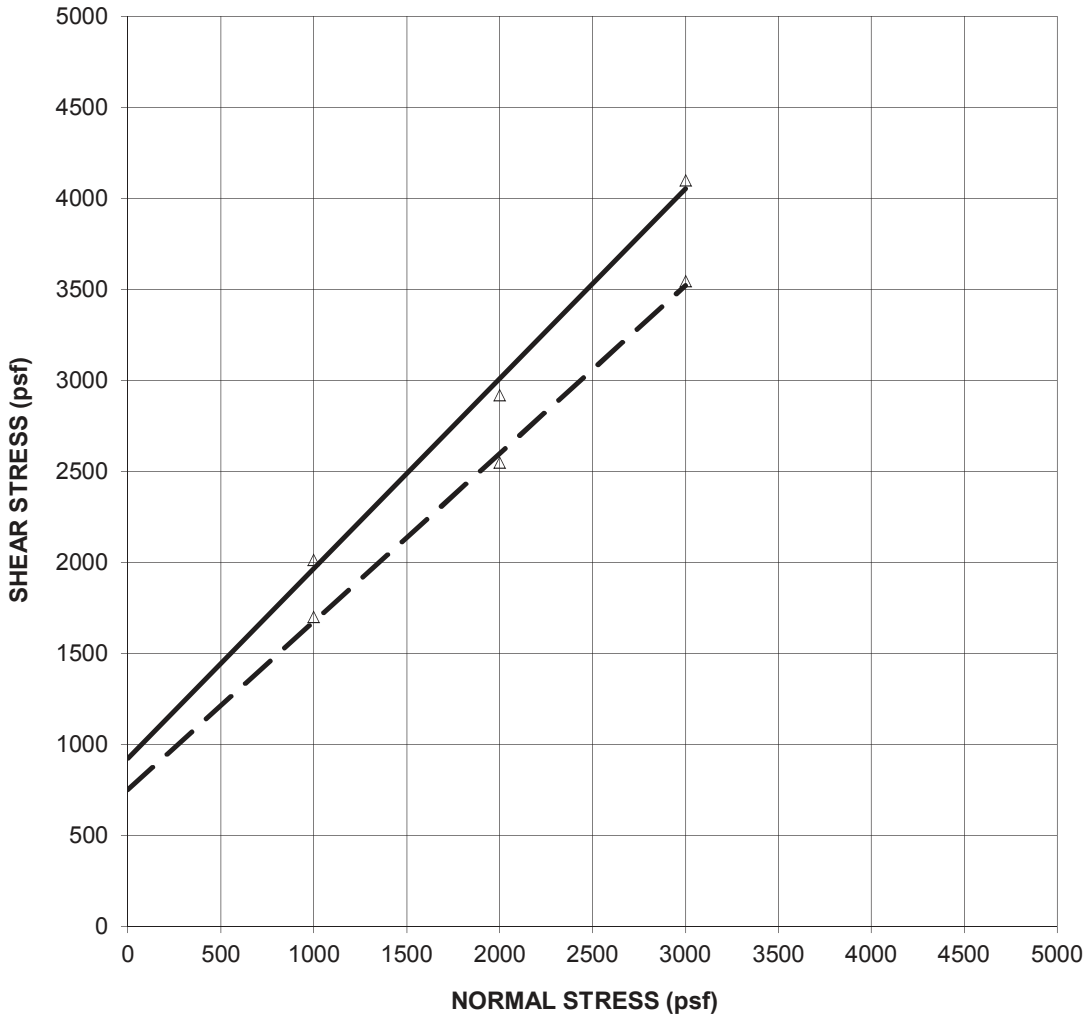
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Direct Shear Test Result** 1440 Growth Center Scotts Valley, California	Figure No. 43 Project No. 1404 Date: 4/23/14

DIRECT SHEAR TEST - ASTM D 3080

CLIENT:	PACIFIC CREST	DATE:	2/12/14
PROJECT:	RETREAT CENTER - 1404	TESTED:	SSC
SAMPLE DESCRIPTION:	Pale Yellow Siltstone, highly fractured		

SAMPLE: 3-7-1		COHESION (psf)	FRICTION ANGLE
SOIL TYPE: (ML)			
	PEAK	920	46
	RESIDUAL	740	43

INITIAL WATER CONTENT:	39.9%
WATER CONTENT AT TEST:	48.4%



Bauldry Engineering, Inc.

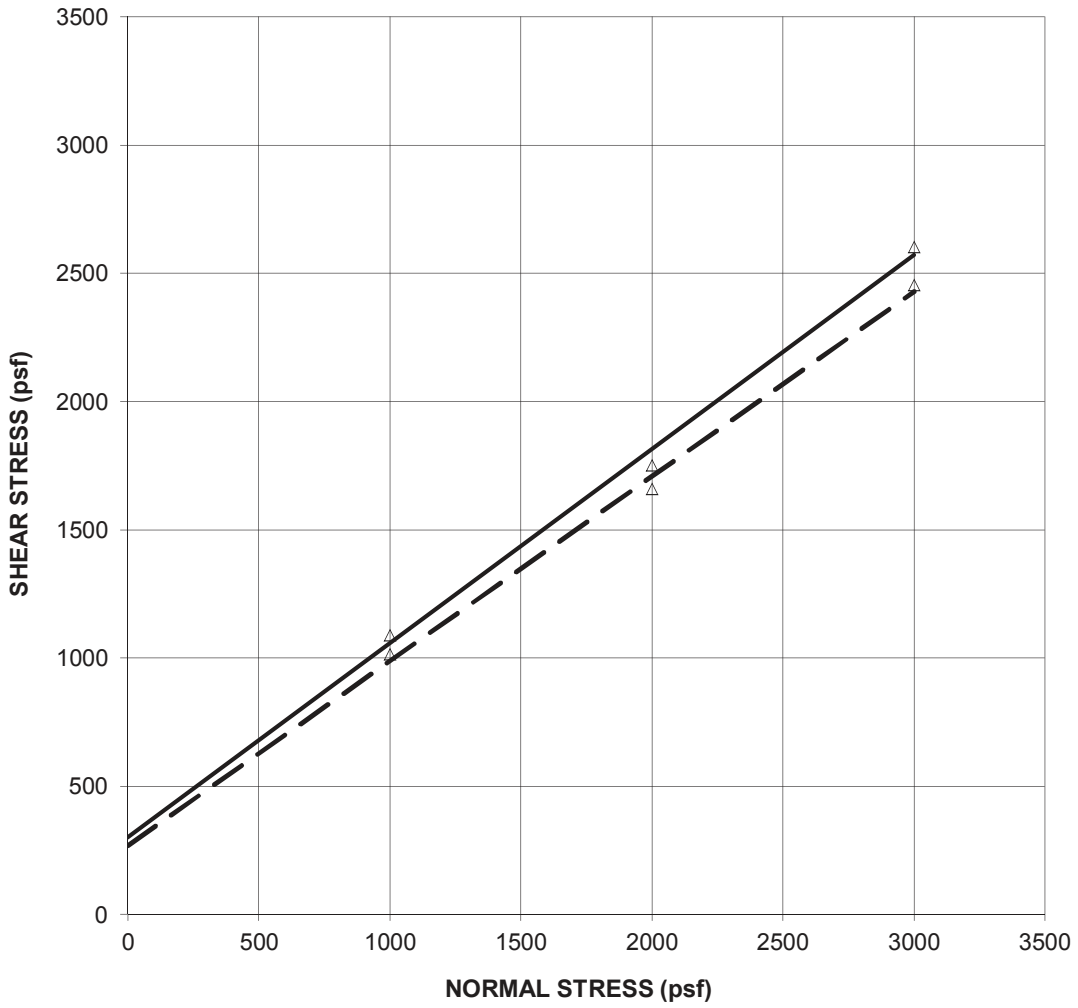
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Direct Shear Test Result** 1440 Growth Center Scotts Valley, California	Figure No. 44 Project No. 1404 Date: 4/23/14

DIRECT SHEAR TEST - ASTM D 3080

CLIENT:	PACIFIC CREST	DATE:	3/31/14
PROJECT:	RETREAT CENTER - 1404	TESTED:	SSC
SAMPLE DESCRIPTION:	Brown Clayey SILT w/Siltstone Gravels		

			COHESION (psf)	FRICTION ANGLE
SAMPLE:	4-1-1			
SOIL TYPE:	ML			
		PEAK	290	37
		RESIDUAL	260	36

INITIAL WATER CONTENT:	21.1%
WATER CONTENT AT TEST:	40.8%



Bauldry Engineering, Inc.

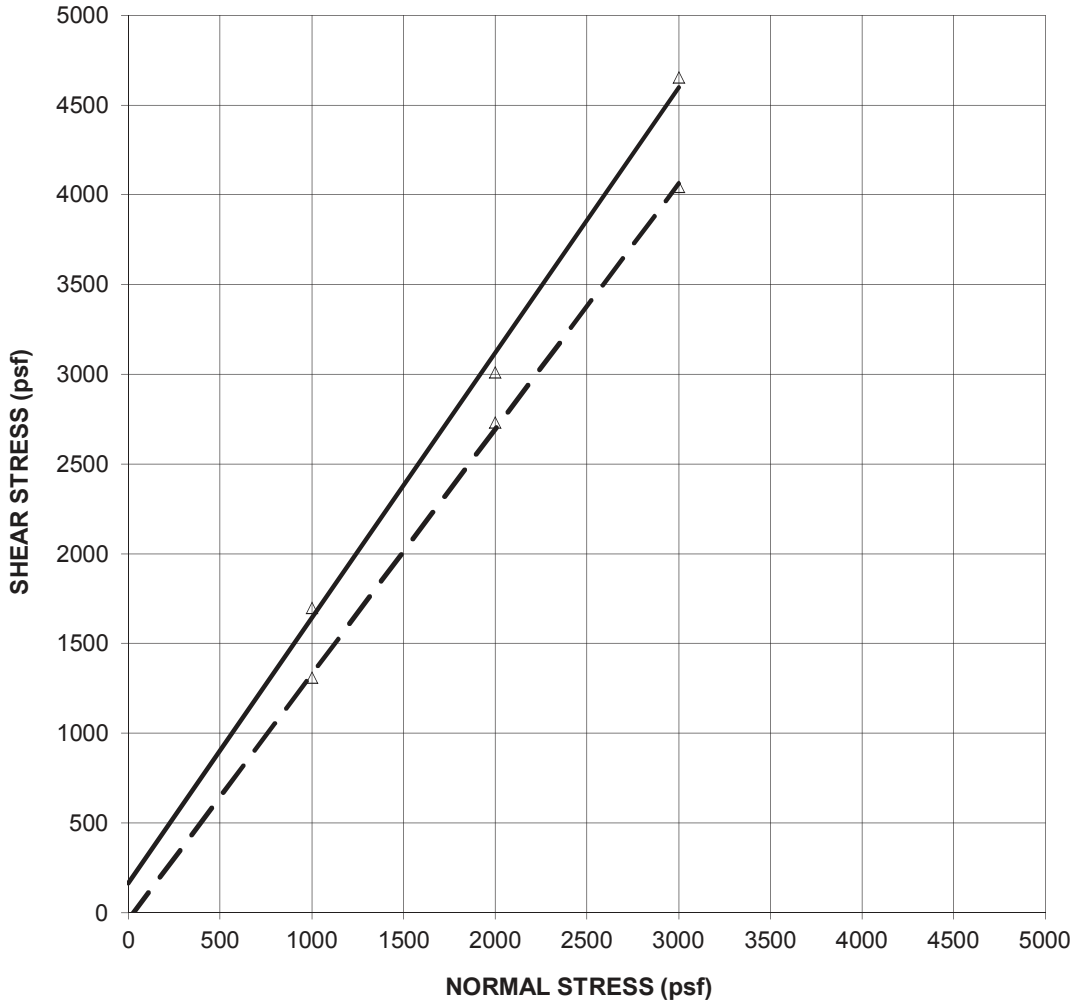
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Direct Shear Test Result** 1440 Growth Center Scotts Valley, California	Figure No. 45 Project No. 1404 Date: 4/23/14

DIRECT SHEAR TEST - ASTM D 3080

CLIENT:	PACIFIC CREST	DATE:	2/13/14
PROJECT:	RETREAT CENTER - 1404	TESTED:	SSC
SAMPLE DESCRIPTION:	Pale Yellow Siltstone, highly fractured		

SAMPLE: 6-5-1		COHESION	FRICTION
SOIL TYPE: (ML)		(psf)	ANGLE
	PEAK	160	56
	RESIDUAL	0	54

INITIAL WATER CONTENT:	27.7%
WATER CONTENT AT TEST:	36.4%



Bauldry Engineering, Inc.

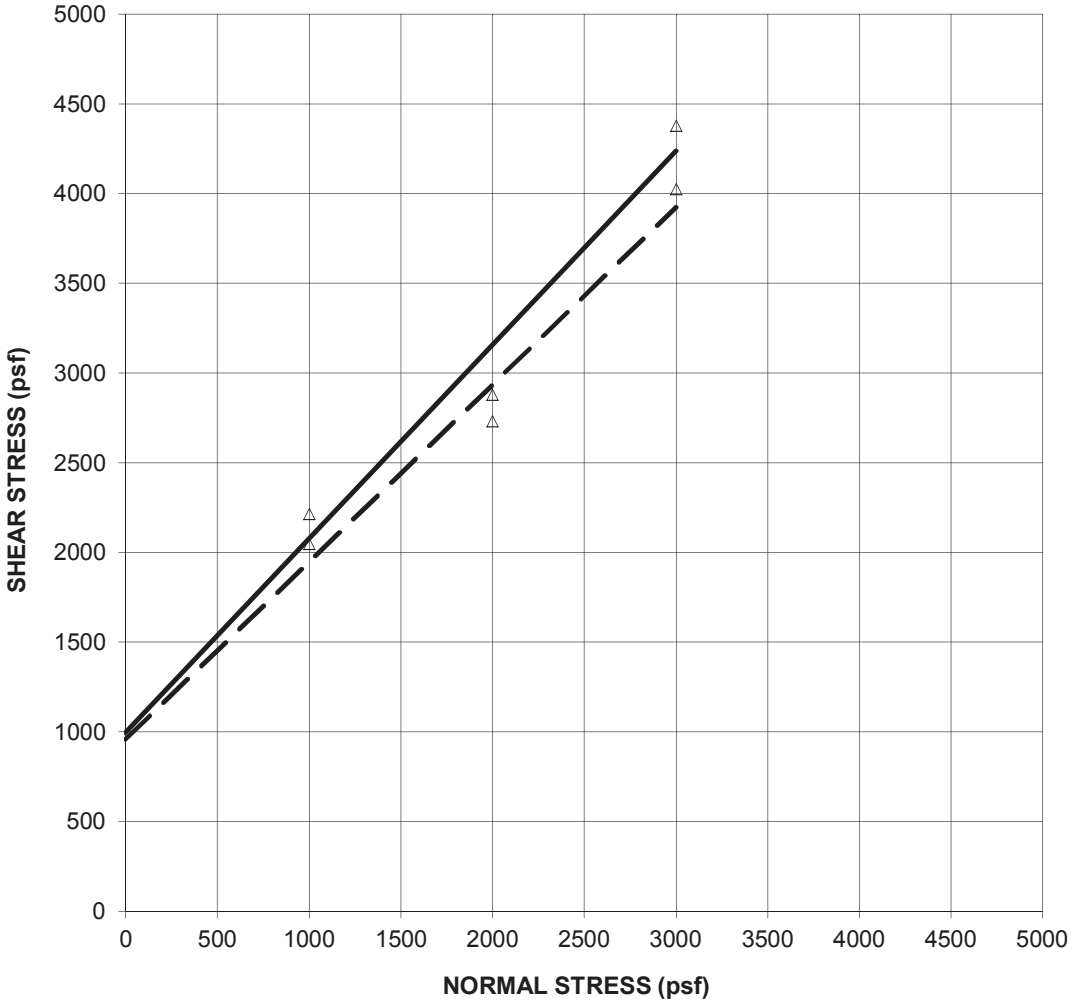
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Direct Shear Test Result** 1440 Growth Center Scotts Valley, California	Figure No. 46 Project No. 1404 Date: 4/23/14

DIRECT SHEAR TEST - ASTM D 3080

CLIENT:	PACIFIC CREST	DATE:	2/13/14
PROJECT:	RETREAT CENTER - 1404	TESTED:	SSC
SAMPLE DESCRIPTION:	Reddish Light Brown Fine-Grained Sandstone		

SAMPLE: 6-7-1		COHESION	FRICTION
SOIL TYPE: (SM)		(psf)	ANGLE
	PEAK	990	47
	RESIDUAL	950	45

INITIAL WATER CONTENT:	21.0%
WATER CONTENT AT TEST:	31.4%



Bauldry Engineering, Inc.

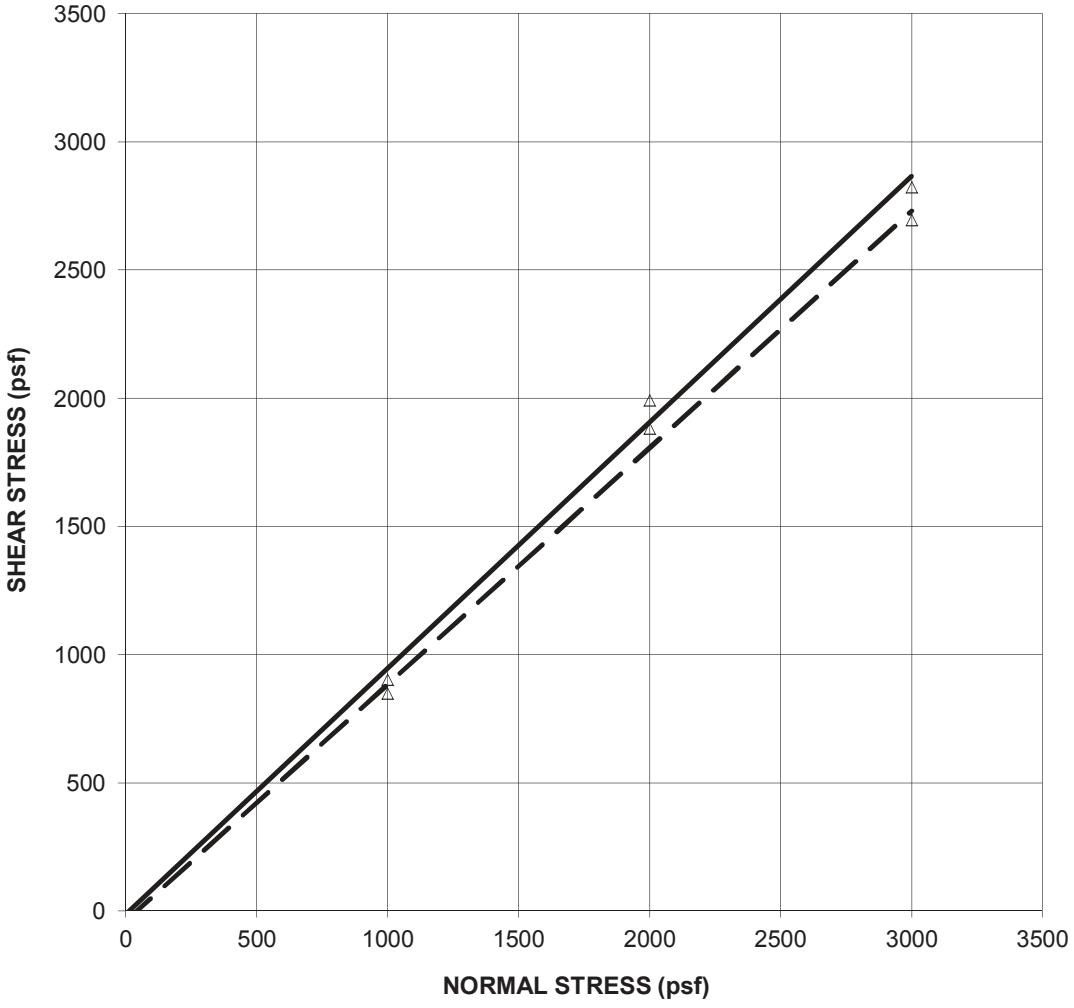
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Direct Shear Test Result** 1440 Growth Center Scotts Valley, California	Figure No. 47 Project No. 1404 Date: 4/23/14

DIRECT SHEAR TEST - ASTM D 3080

CLIENT:	PACIFIC CREST	DATE:	3/31/14
PROJECT:	RETREAT CENTER - 1404	TESTED:	SSC
SAMPLE DESCRIPTION:	Brown Clayey SILT some Siltstone Gravels		

SAMPLE: 12-3-1		COHESION (psf)	FRICTION ANGLE
SOIL TYPE: ML			
	PEAK	0	44
	RESIDUAL	0	43

INITIAL WATER CONTENT:	22.9%
WATER CONTENT AT TEST:	32.3%



Bauldry Engineering, Inc.

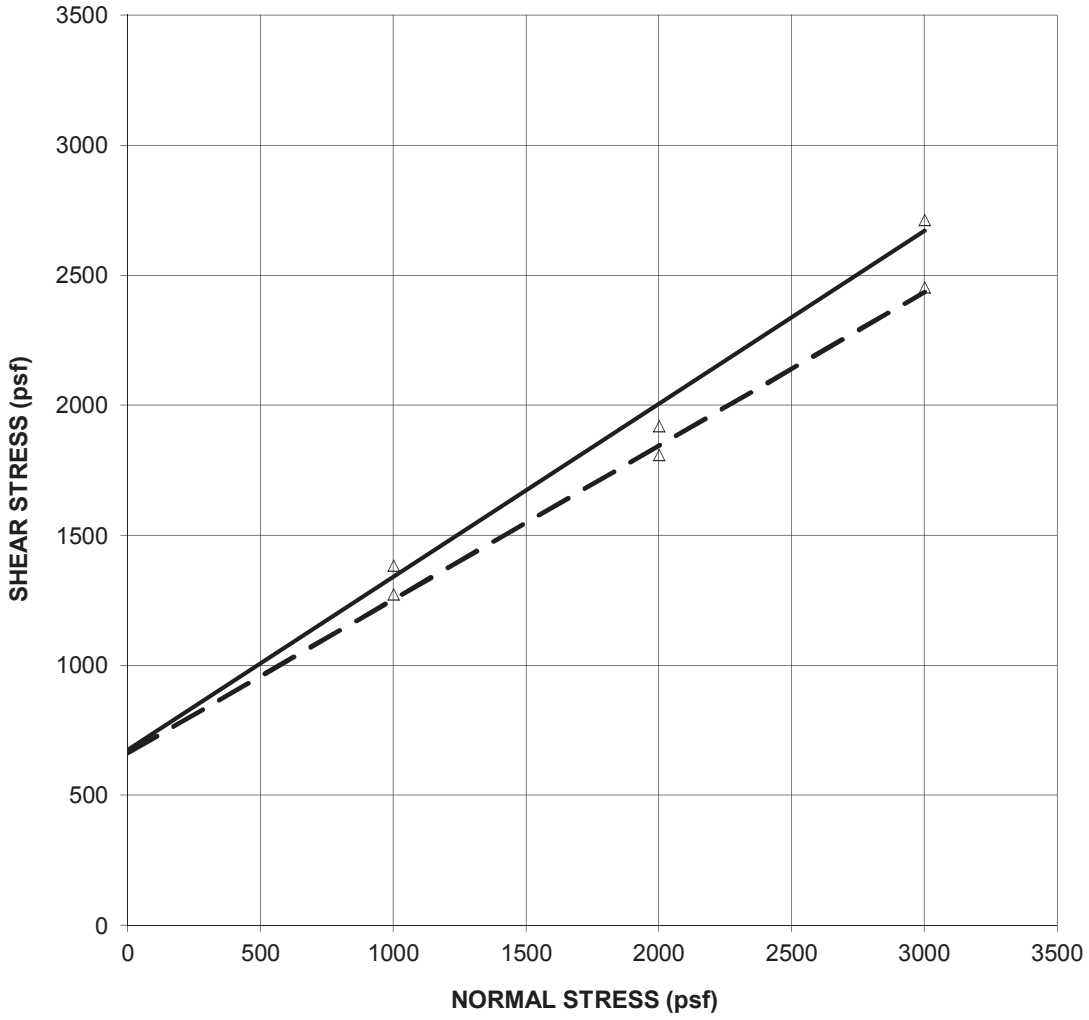
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Direct Shear Test Result** 1440 Growth Center Scotts Valley, California	Figure No. 48 Project No. 1404 Date: 4/23/14

DIRECT SHEAR TEST - ASTM D 3080

CLIENT:	PACIFIC CREST	DATE:	3/31/14
PROJECT:	RETREAT CENTER - 1404	TESTED:	SSC
SAMPLE DESCRIPTION:	Brown Clayey SAND some Siltstone Gravels		

			COHESION (psf)	FRICTION ANGLE
SAMPLE:	16-3-1			
SOIL TYPE:	SC			
		PEAK	670	34
		RESIDUAL	660	31

INITIAL WATER CONTENT:	28.7%
WATER CONTENT AT TEST:	29.7%



Bauldry Engineering, Inc.

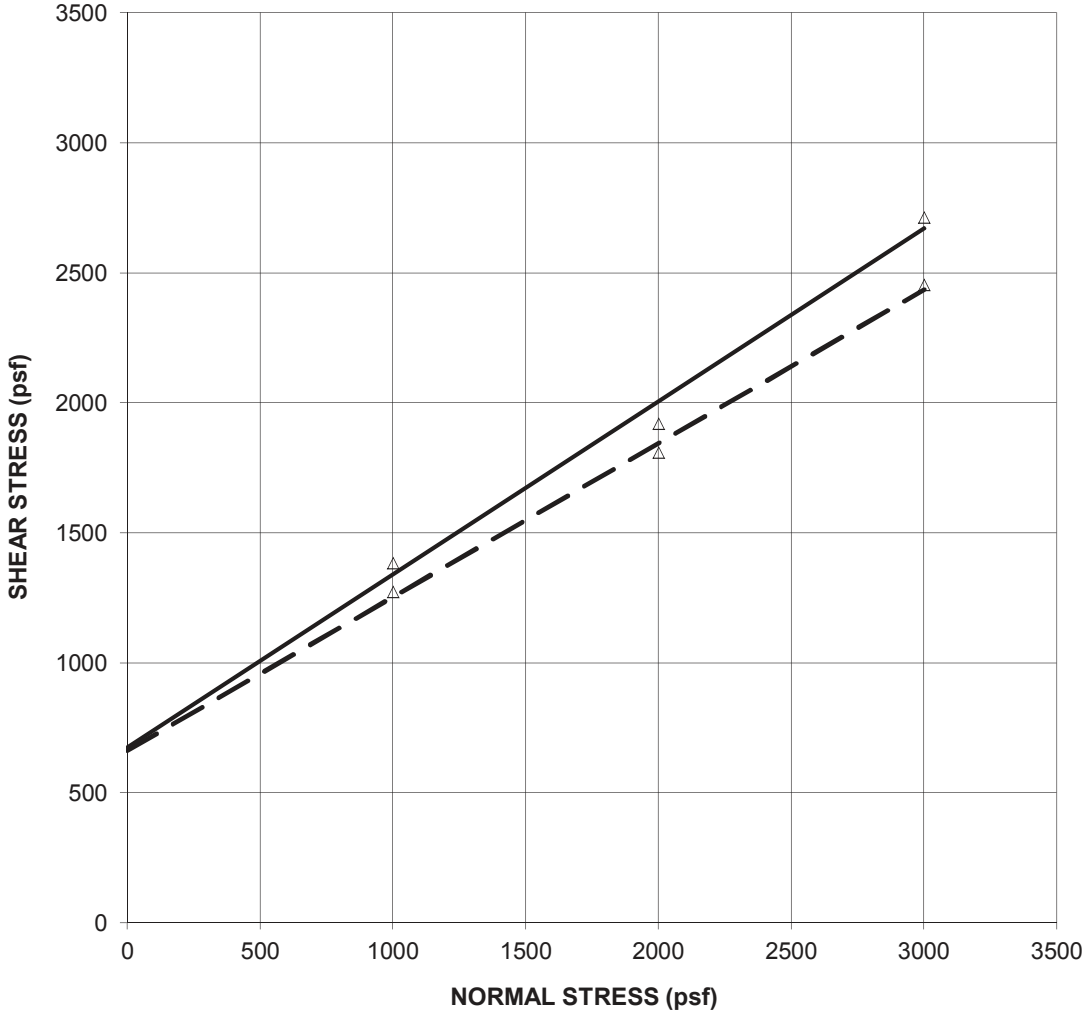
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Direct Shear Test Result** 1440 Growth Center Scotts Valley, California	Figure No. 49 Project No. 1404 Date: 4/23/14

DIRECT SHEAR TEST - ASTM D 3080

CLIENT:	PACIFIC CREST	DATE:	3/31/14
PROJECT:	RETREAT CENTER - 1404	TESTED:	SSC
SAMPLE DESCRIPTION:	Light brown Siltstone Gravels in Brown Clayey SILT matrix		

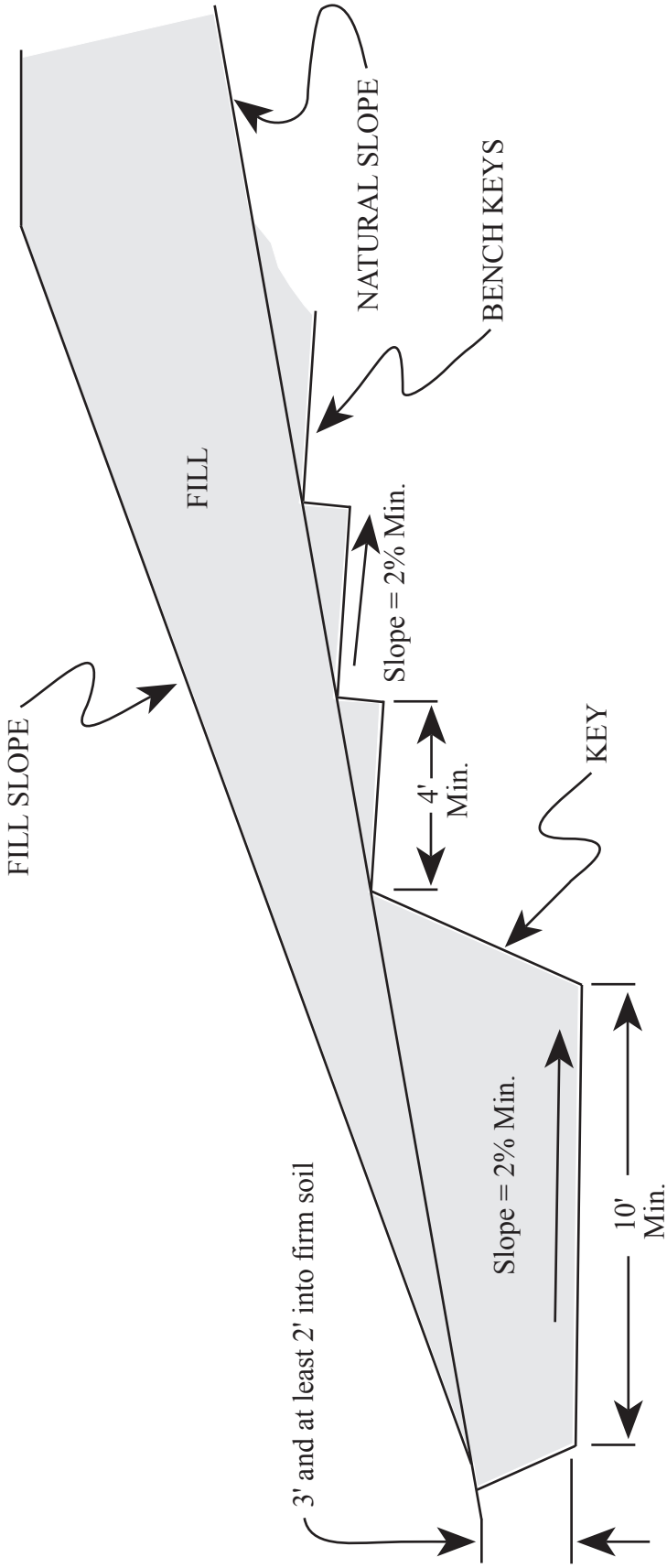
			COHESION (psf)	FRICTION ANGLE
SAMPLE:	19-1-1			
SOIL TYPE:	GM			
		PEAK	660	34
		RESIDUAL	680	31

INITIAL WATER CONTENT:	29.9%
WATER CONTENT AT TEST:	36.7%



Bauldry Engineering, Inc.

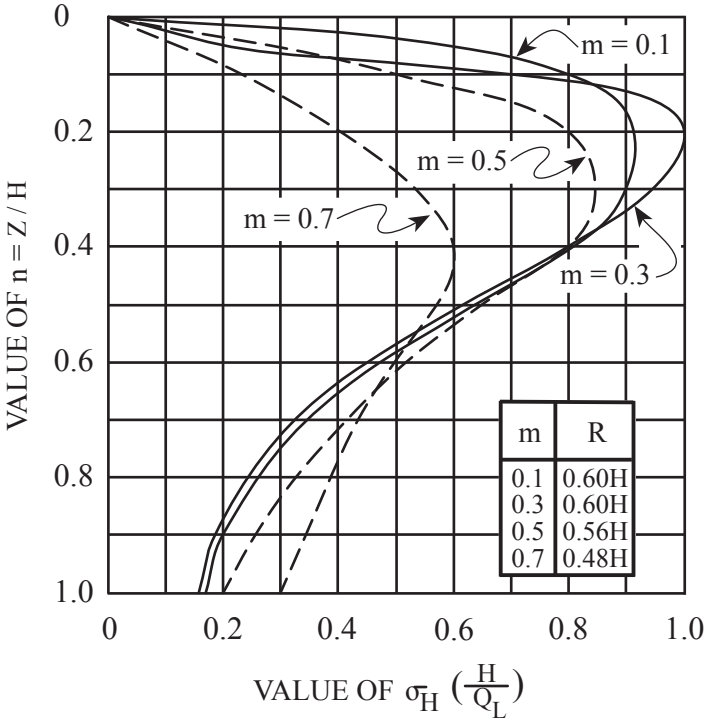
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Direct Shear Test Result** 1440 Growth Center Scotts Valley, California	Figure No. 50 Project No. 1404 Date: 4/23/14



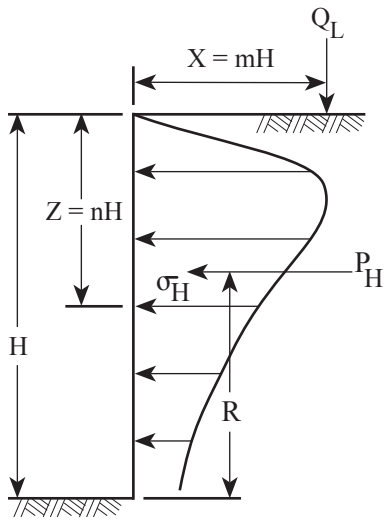
TYPICAL KEY AND BENCHES
not to scale

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Keyway Detail** 1440 Growth Center Scotts Valley, California	Figure No. 51 Project No. 1404 Date: 4/23/14

LINE LOAD



m	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H



FOR $m \leqq 0.4$:

$$\sigma_H \left(\frac{H}{Q_L}\right) = \frac{0.20\, n}{(0.16 + n^2)^2}$$

$$P_H = 0.55\, Q_L$$

FOR $m > 0.4$:

$$\sigma_H \left(\frac{H}{Q_L}\right) = \frac{1.28\, m^2 n}{(m^2 + n^2)^2}$$

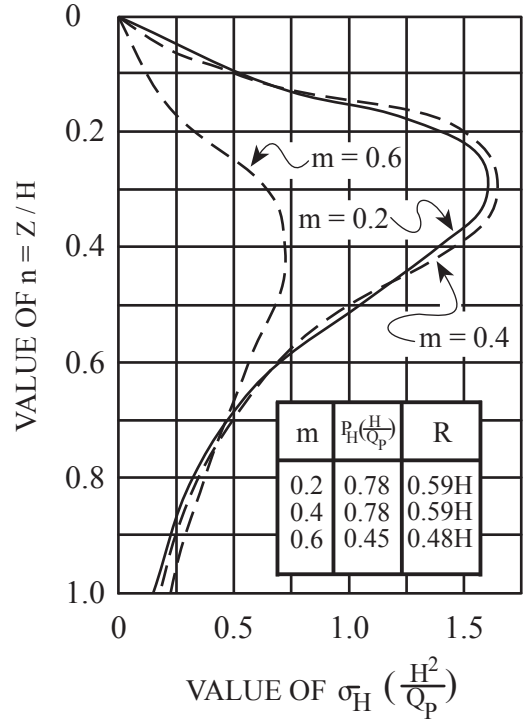
RESULTANT $P_H = \dfrac{0.64\, Q_L}{(m^2 + 1)}$

PRESSURES FROM LINE LOAD Q_L

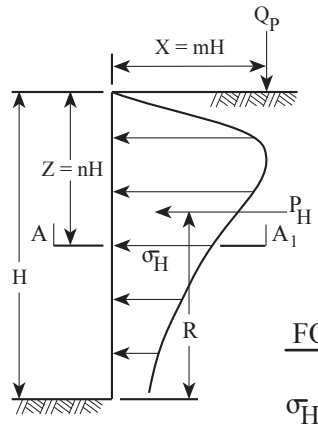
(BOISSINESQ EQUATION MODIFIED BY EXPERMENT)

REFERENCE: Design Manual
NAVFAC DM-7.02
Figure 11
Page 7.2-74

POINT LOAD



m	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



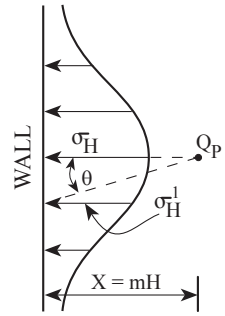
FOR $m \leqq 0.4$:

$$\sigma_H \left(\frac{H^2}{Q_P}\right) = \frac{0.28\, n^2}{(0.16 + n^2)^3}$$

FOR $m > 0.4$:

$$\sigma_H \left(\frac{H^2}{Q_P}\right) = \frac{1.77\, m^2 n^2}{(m^2 + n^2)^3}$$

$$\sigma_H^1 = \sigma_H \cos^2(1.1\,\theta)$$

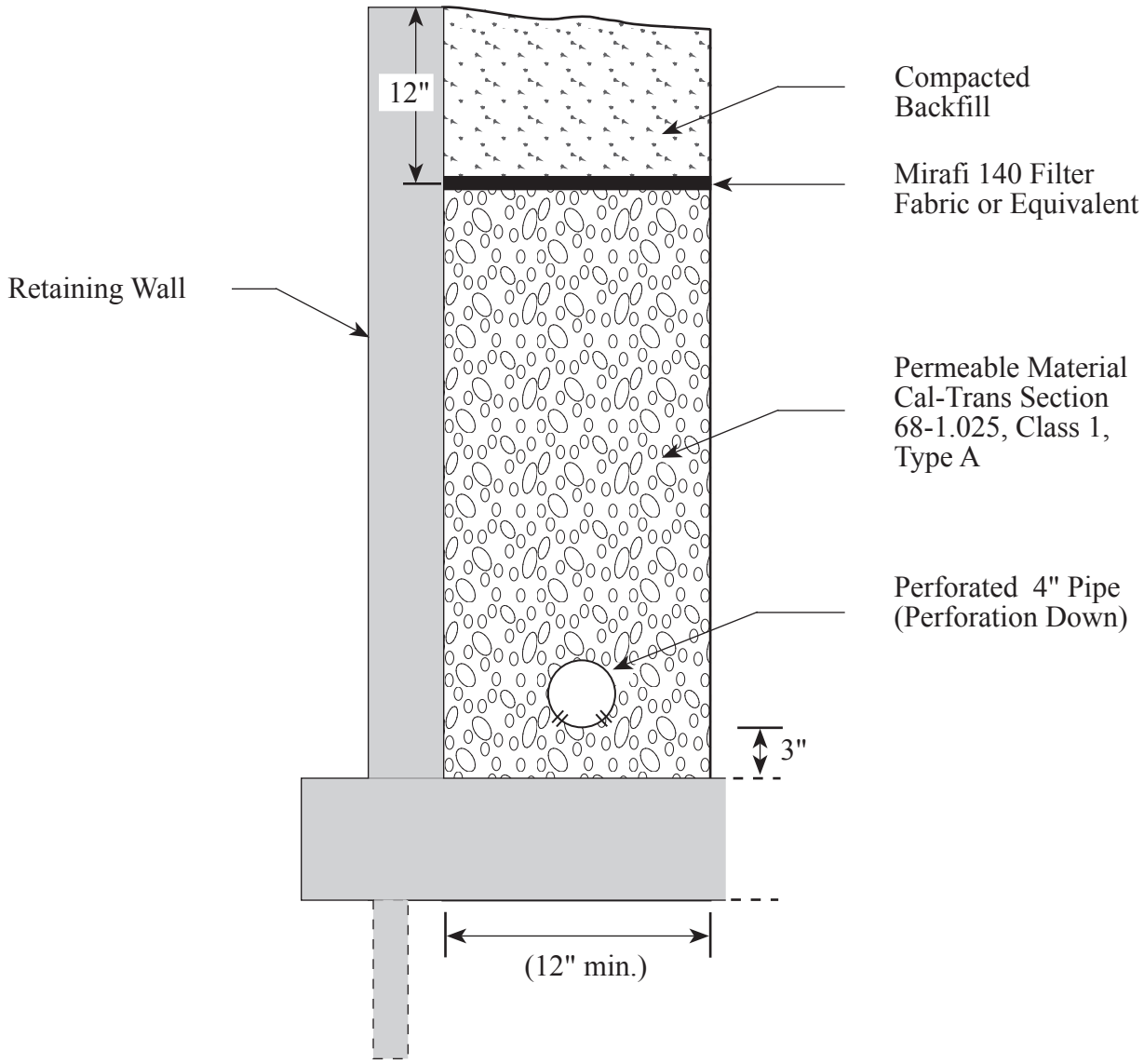


SECTION A-A_1

PRESSURES FROM POINT LOAD Q_P

(BOISSINESQ EQUATION MODIFIED BY EXPERMENT)

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Surcharge Pressure Diagram** 1440 Growth Center Scotts Valley, California	Figure No. 52 Project No. 1404 Date: 4/23/14



Not to Scale

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	**Typical Retaining Wall Drain Detail** 1440 Growth Center Scotts Valley, California	Figure No. 53 Project No. 1404 Date: 4/23/14

APPENDIX B

Geologic Investigation Report By Zinn Geology

GEOLOGIC INVESTIGATION
Proposed Redevelopment Of Bethany University Campus
The 1440 Foundation
800 Bethany Drive
Scotts Valley, California

Job #2014001-G-SC
21 April 2014

Engineering Geology ⚒ Coastal Geology ⚒ Fault & Landslide Investigations

21 April 2014 Job #2014001-G-SC

Pacific Crest Engineering
Attention: Elizabeth Mitchell
444 Airport Boulevard, Suite 106
Watsonville, California 95076

Re: Engineering geologic investigation
Proposed redevelopment of old Bethany University campus
Monterey Peninsula Water Supply Project
Marina, California

Dear Mrs. Mitchell:

Our geologic report for the proposed redevelopment of the old Bethany University campus by The 1440 Foundation is attached. This report documents geologic conditions on the campus property germane to the proposed redevelopment and addresses potential hazards such as landsliding, erosion, differential settlement and seismic shaking.

Based on the information gathered and analyzed, it is our opinion that the proposed redevelopment scheme depicted upon Plate 1 is geologically suitable and will be subject to "ordinary risks" as defined in Appendix B of this report, provided our recommendations are followed.

Landsliding is the predominant geological process that potentially threatens a number of the proposed structures and one structure related for remodeling (Building R7). In some cases future potential landsliding could undermine the structures if that process is not properly mitigated and in other cases, the proposed developments are at risk of potentially being struck by shallow fast moving landslides called debris flows.

Drainage on the campus appears to be complex and of several vintages, as evidenced by some of the damaged and abandoned drains we observed in the inner gorge of Carbonera Creek. Overall, the existing system appears to be cobbled together older systems that mostly dispose of storm water mid-slope above Carbonera Creek. The disposal points of these older systems appear to coincide with old shallow landslide scars that have been further etched by erosion after most of the surficial soils were swept down into Carbonera Creek via a debris flow. These past landslides that have been triggered by mid-slope disposal of collected storm water, along with

poorly controlled surface flow around the buildings serve as an object lesson for surface water drainage - mid-slope disposal of storm water is unacceptable. Our recommendations for each proposed development site is almost rote for this problem and primarily involves the careful control of surface water around structures and then disposal of the water into Carbonera Creek, or a place deemed suitable for recharge on the campus. In our opinion, a comprehensive drainage system should be developed by the Project Civil Engineer Of Record, terminating in a disposal system located along Carbonera Creek or in an area where recharge is geologically acceptable.

Seismic shaking created by large magnitude earthquakes on nearby active faults is an ubiquitous geological hazard in this region. This issue is typically handled via project engineers choosing the appropriate prescriptive code-based seismic shaking procedures and parameters for their analyses, unless there are special circumstances due to the geological conditions or building occupancy.

We understand that the location and design of the proposed remodel scheme may evolve and change after the issuance of our report. ***We therefore recommend that our firm be provided the opportunity to review the interim designs and specifications in order to perform supplemental work and issue supplemental recommendations where warranted.*** It is also important that we be given the opportunity to review interim designs and specifications so that our recommendations may be properly interpreted and implemented.

The body of the text should be read for a detailed discussion of the aforementioned hazards and the recommended mitigation schemes. We have broken up the report into focused abbreviated mini-reports that are organized by building, in an attempt to simplify the reading for the design team.

If you have any questions or comments regarding this report, please contact us at your earliest convenience.

Sincerely,
ZINN GEOLOGY

Erik N. Zinn
Principal Geologist
P.G. #6854, C.E.G. #2139

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PLATES 1 (Geologic Site Map) & 2 (Geologic Cross Sections) - In pocket at back of report

NOTE: Plates must accompany text of report in order for report to be considered complete.

INTRODUCTION

This report presents the results of our geologic investigation for the proposed redevelopment of the old Bethany University camps, located at 800 Bethany Drive, northeast of the City of Scotts Valley in the County of Santa Cruz (Figure 1).

The property is currently developed with older campus buildings and residences scattered across it, as well as a partially completed building. The campus is supported by an array older utilities, including at least several generations of storm drainage systems in various stages of decay. The proposed development will consist of an array of new campus buildings and residences, parking structures and attendant utilities and road construction (see our Plate 1). It is our understanding that completion of the proposed developments will be done in phases, at least as proposed at the time of this report. It is important to understand that the scope and layout of the proposed developments has been a dynamic work in progress, due to the compressed and integrated nature of the property purchase by the 1440 Foundation and preliminary design work by their team. Hence, this report is a snapshot in time of the proposed development scheme, which will likely be subject to change in the future. Therefore, the reader should refer to forthcoming documents and plates from the Project Architect Of Record and Project Civil Engineer Of Record to view the details of the proposed developments.

The purpose of this investigation was to evaluate the potential geologic hazards relevant to the proposed development scheme. The primary focus at the outset of the project was upon the geologic hazards and attendant risks posed by the landslides originating on the steep slopes that are adjacent to the proposed developments, both above and below. We also identified geological hazards that are ubiquitous to the mountainous Santa Cruz region such as erosion, soil creep and intense seismic shaking for this project.

SCOPE OF INVESTIGATION

Work performed during this study included:

1. A site meeting with Project Geotechnical Engineer Of Record, Elizabeth Mitchell of Pacific Crest Engineering [PCE], as well as telephone conversations with both Gerald Yates (Project Architect Of Record) and Elizabeth Mitchell.

2. A review of published and unpublished maps and reports germane to the project.

3. Performance of regional reconnaissance level mapping consisting of mapping natural and artificial exposures (mostly road cuts) of the bedrock in the vicinity of the property.

4. Examination and interpretation of stereo-pair vertical aerial photographs, to assess the past history of landsliding on the property.

5. Location of the small-diameter borings advanced by PCE and observation of the advancement of select borings. We have also plotted borings drilled by other consultants in the past for the campus upon the base map for the project (see Plate 1)

6. Preparation of a geologic site map and geologic cross sections for the project, including issuance of preliminary drafts to assist PCE with their slope stability analyses.

7. Analysis and interpretation of the geologic data and preparation of this report.

REGIONAL GEOLOGIC SETTING

The subject property is located within the central Santa Cruz Mountains. The Santa Cruz Mountains are formed by a series of rugged, linear ridges and valleys following the pronounced northwest to southeast structural grain of central California geology. Underlying most of the Santa Cruz Mountains is a large, elongate prism of granitic and metamorphic basement rocks, known collectively as the Salinian Block. These rocks are separated from contrasting basement rock types to the northeast and southwest by the San Andreas and San Gregorio-Sur Nacimiento strike-slip fault systems, respectively. Overlying the granitic basement rocks is a sequence of dominantly marine sedimentary rocks of Paleocene to Pliocene age and non-marine sediments of Pliocene to Pleistocene age (Figure 2).

Throughout the Cenozoic Era, this portion of California has been dominated by tectonic forces associated with lateral or "transform" motion between the North American and Pacific litho-spheric plates, producing long, northwest-trending faults such as the San Andreas and San Gregorio, with horizontal displacements measured in tens to hundreds of miles. Accompanying the northwest direction of the horizontal (strike-slip) movement of the plates have been episodes of compressive stress, reflected by repeated episodes of uplift, deformation, erosion and subsequent redeposition of sedimentary rocks. Near the crest of the Santa Cruz Mountains, this tectonic deformation is most evident in the sedimentary rocks older than the middle Miocene, and consists of steeply dipping folds, overturned bedding, faulting, jointing, and fracturing. Along the coast, the ongoing tectonic activity is most evident in the formation of a series of uplifted marine terraces. The Loma Prieta earthquake of 1989 and its continuing aftershocks are the most recent reminders of the geologic unrest in the region.

REGIONAL SEISMIC SETTING

California's broad system of strike-slip faulting has had a long and complex history. Some of these faults present a seismic hazard to the subject property. The most important of these are the San Andreas and the Zayante (-Vergeles) faults (Figure 2). These faults are either active or considered potentially active (Hall et al., 1974; Cao et al., 2003). Each fault is discussed below. Locations of epicenters associated with the faults are shown in Figure 3. The intensity of seismic

shaking that could occur at the subject property in the event of a future earthquake on one of these faults will be discussed in a later section.

San Andreas Fault

The San Andreas fault is active and represents the major seismic hazard in northern California. The main trace of the San Andreas fault trends northwest-southeast and extends over 700 miles from the Gulf of California through the Coast Ranges to Point Arena, where the fault extends offshore.

Geologic evidence suggests that the San Andreas fault has experienced right-lateral, strike-slip movement throughout the latter portion of Cenozoic time, with cumulative offset of hundreds of miles. Surface rupture during historical earthquakes, fault creep, and historical seismicity confirm that the San Andreas fault and its branches, the Hayward, Calaveras, and San Gregorio faults, are all active today.

Historical earthquakes along the San Andreas fault and its branches have caused significant seismic shaking in the Santa Cruz County area. The two largest historical earthquakes on the San Andreas to affect the area were the moment magnitude (M_w) 7.9 San Francisco earthquake of 18 April 1906 (actually centered near Olema) and the M_w 7.0 Loma Prieta earthquake of 17 October 1989. The San Francisco earthquake caused severe seismic shaking and structural damage to many buildings in the Santa Cruz Mountains. The Loma Prieta earthquake appears to have caused more intense seismic shaking than the 1906 event in localized areas of the Santa Cruz Mountains, even though its regional effects were not as extensive. There were also significant earthquakes in northern California along or near the San Andreas fault in 1838, 1865 and possibly 1890 (Sykes and Nishenko, 1984; Working Group On Northern California Earthquake Probabilities [WGONCEP], 1996).

Geologists have recognized that the San Andreas fault system can be divided into segments with “characteristic” earthquakes of different magnitudes and recurrence intervals (Working Group On California Earthquake Probabilities, 1988 and 1990). A more recent study by the WGONCEP in 1996 has redefined the segments and the characteristic earthquakes for the San Andreas fault system in northern and central California. Two overlapping segments of the San Andreas fault system represent the greatest potential hazard to the subject property. The first segment is defined by the rupture that occurred from the Mendocino triple junction to San Juan Bautista along the San Andreas fault during the great M_w 7.9 earthquake of 1906. The WGONCEP (1996) has hypothesized that this "1906 rupture" segment experiences earthquakes with comparable magnitudes in independent cycles about two centuries long.

The second segment is defined by the rupture zone of the M_w 7.0 Loma Prieta earthquake, despite the fact that the oblique slip and focal depth of this event do not fit the ideals of a typical, right-lateral strike-slip event on the San Andreas fault. Although it is uncertain whether this "Santa

Cruz Mountains" segment has a characteristic earthquake independent of great San Andreas fault earthquakes, the WGONCEP (1996) has assumed an "idealized" earthquake of M_w 7.0 with the same right-lateral slip as the 1989 Loma Prieta earthquake, but having an independent segment recurrence interval of 138 years and a multi-segment recurrence interval of 400 years.

The 2002 Working Group On California Earthquake Probabilities [WGOCEP] (2003) segmentation model is largely similar to that adopted by WGONCEP, although they have added far more complexity to the model, and have reduced the forecasted magnitudes for the different segments. Cao et al. (2003) appears to have largely adopted the earthquake magnitudes issued by the 2002 WGOCEP. The magnitudes for the sundry segments are as follows: Parkfield segment - Mw 6.5, Creeping Segment - Mw 6.2, Santa Cruz Mountains - Mw 7.0, Peninsula segment - Mw 7.1, North Coast North Segment - Mw 7.3, North Coast South Segment - Mw 7.4. The most significant change in modeling the San Andreas Fault Zone by 2002 WGOCEP and Cao et al. (2003) is the elimination of a the penultimate event, the 1906 Mw 7.9 earthquake.

Zayante(-Vergeles) Fault

The Zayante fault lies west of the San Andreas fault and trends about 50 miles northwest from the Watsonville lowlands into the Santa Cruz Mountains. The southern extension of the Zayante fault, known as the Vergeles fault, merges with the San Andreas fault south of San Juan Bautista.

The Zayante fault has a long, well-documented history of vertical movement (Clark and Reitman, 1973), probably accompanied by right-lateral, strike-slip movement (Hall et al., 1974; Ross and Brabb, 1973). Stratigraphic and geomorphic evidence indicates the Zayante fault has undergone late Pleistocene and Holocene movement and is potentially active (Buchanan-Banks et al., 1978; Coppersmith, 1979).

Some historical seismicity may be related to the Zayante fault (Griggs, 1973). For instance, the Zayante fault may have undergone sympathetic fault movement during the 1906 earthquake centered on the San Andreas fault, although this evidence is equivocal (Coppersmith, 1979). Seismic records strongly suggest that a section of the Zayante fault approximately 3 miles long underwent sympathetic movement in the 1989 earthquake. The earthquake hypocenters tentatively correlated to the Zayante fault occurred at a depth of 5 miles; no instances of surface rupture on the fault have been reported.

In summary, the Zayante fault should be considered potentially active. The WGONCEP (1996) considers it capable of generating a magnitude 6.8 earthquake with an effective recurrence interval of 10,000 years. Alternatively, Cao et al. (2003) considers this fault capable of generating a maximum earthquake of Mw 7.0, with no stated recurrence interval.

SITE GEOLOGIC SETTING, HAZARDS, CONCLUSIONS & RECOMMENDATIONS BY BUILDING

The Geologic Site Map (Plate 1) and Geologic Cross Sections (Plate 2) depict relevant site-specific topographic and geologic information for the project. See also the Local Geologic Map (Figure 4) for information of a more general nature.

Building P1-1 - Spa

Topography

The area traversed by this proposed complex of four buildings is a side-hill bench that wraps around the nose of a bedrock ridge. Most of the slopes above and below the bench are moderately steep, except for the slope at the southern end of the proposed development area, which descends from the flat pad very steeply to Carbonera Creek below the site.

The topography depicted on the regional topographic figure that accompanies this report (Figure 1) isn't an appropriate scale to discern the distinctions outlined above, nor is the topography used for the base map for this project (see our Plate 1) extensive enough to cover the areas of interest. The reader should refer to our cross sections depicted on Plate 2 in order to see how steeply the slope descends from the southern end of the development area to Carbonera Creek.

Drainage

Natural surface drainage across the area slated for development occurs primarily by overland sheet flow toward the southwest and to the south. There appears to be some existing storm drainage structures below and southwest of the site that could collect some of the sheet flow. Ultimately all the water for the site enters Carbonera Creek, whether it flows naturally down the steep slope flanks or is collected and disposed of in the creek.

Some of the rainfall on the property probably infiltrates the ground and enters the groundwater regime. Groundwater was encountered by PCE in Boring #21, but not in any of the other small diameter borings advanced on the pad in this area. The water encountered in Boring #21 was perched atop the gleyed or gray mottled bedrock, within the blanket of soil that caps the bedrock in this area. Considering our long standing drought conditions in this region, and the geometry and distribution of the earth materials on this site, it is likely that the groundwater encountered in Boring #21 was perched groundwater and derived from a domestic line on the site. The transmissivity contrast between the blanket of soil (higher relative transmissivity) and the underlying bedrock (lower relative transmissivity) is conducive to perched water conditions within the soil. So although the groundwater in Boring #21 was anomalous compared to the dry holes for the other borings, it is our opinion that water will perch atop the bedrock within the surficial soils during wet winters at this site.

Earth Materials

The P1-1 Spa site is underlain by a blanket of relatively loose surficial soil that is between 5' and 9 ½' thick, overlying "dirty" Santa Margarita Formation siltstone and silty sandstone bedrock. The contact between the surficial soils and bedrock appears to be very gently sloping to flat, which may indicate that the surface is either graded or was created by erosion from ancestral Carbonera Creek.

The blanket of soil thins considerably once it intersects the top of the steep slope that descends to Carbonera Creek. We observed abundant construction debris and garbage on this slope, which indicates that this has been a historical dumping area and that the soil mantling the steep slope is composed of loosely dumped non-engineered fill.

Geologic Hazards For Building P1-1 Spa

The primary potential geologic hazards that could affect the proposed P1-1 Spa site are landsliding, soil creep, erosion, and intense seismic shaking.

The site is flanked by a large debris flow scar that stretches from the top of the building area to Carbonera Creek, which has been partially filled with materials dumped over the top of the slope. In our opinion, the surficial soils on the steep slope will continue to fail down toward Carbonera Creek in response to large storms and possibly due to large earthquakes. We have constructed a hypothesized future slope configuration that takes downcutting of Carbonera Creek into account, as well as retreat of the top of the steep slope due to failure of the surficial materials. This line can be seen on our cross section for the site (see Plate 2), as well the projection of the line onto the ground surface (Plate 1).

The proposed structures will also be placed close to the toe of a steep slope that is mantled by relatively loose surficial soil, which is capable of creeping and landsliding in our opinion. In our opinion, it is likely that small debris flows or slumps will form on the slope and could potentially strike or push against the proposed structures.

The flat area of the site is ringed by artificial fill of unknown origin, replete with construction debris on the surface of the fill. It is likely that this fill is non-engineered. In our opinion, this fill is likely creeping slowly down slope within the fill slopes that partially surround the site. This process could undermine the proposed improvements and possibly cause them to move downhill slightly.

The surficial soils at the site are relatively loose and will be susceptible to erosion, particularly if storm water is not properly controlled and disposed of away from the site.

Seismic shaking at the site will be intense during the next major earthquake along local fault systems. It is important that recommendations regarding seismic shaking be applied where deemed appropriate by the project engineers for the project design, as well as where dictated by minimum prescriptive requirements in the different codes and standards utilized for the project.

Recommendations For Building P1-1 Spa

1. We recommend that any development located within the portion of surficial soil forecasted to fail in the future as depicted up Plates 1 and 2 take into account that the earth materials within this zone will slide and slowly creep downslope over the long term. Mitigation recommendations taking the aforementioned geologic process into account should be issued by the project geotechnical and structural engineers. Please note that this recommendation does not preclude more conservative recommendations by the project geotechnical engineer if the outcome of their analyses, including quantitative slope stability analyses, indicates that the wedge of potentially unstable material is deeper than depicted upon our cross section.

2. At a minimum, the project engineers should choose the appropriate prescriptive code-based seismic shaking analyses and parameters for their particular analyses.

3. We recommend that all drainage from improved surfaces such as walkways, patios, roofs and roads be collected in impermeable gutters or pipes and carried to Carbonera Creek, or a place deemed suitable for recharge on the campus. At no time should any concentrated discharge (point sources such as pipes dumping into rock dissipaters) be allowed to spill directly onto the ground adjacent to the proposed development or below the development on the steep slope. Any water landing on paved areas should not be allowed to flow toward the proposed developments. The control of runoff is essential for erosion control and prevention of ponding water against the foundation, as well as to prevent landslides from occurring on the steep slopes below the site.

A comprehensive drainage system should be developed by the Project Civil Engineer Of Record, terminating in a disposal system located along Carbonera Creek or in an area where recharge is geologically acceptable.

On a final note regarding drainage, we would like to head off any future drainage recommendations that might be issued by the lead jurisdiction reviewers that will require runoff from all added impervious areas to be retained on site. This type of recommendation is in direct conflict with the general standard of care in engineering geology for hill side drainage mitigation. In light of this observation, we feel that we should be emphatic with our drainage recommendations. Our recommendation is as follows: We do **not** recommend that any groundwater recharge structures be constructed near steep slopes on the site, as injecting all the drain water from the development into a point source at depth will create an **unnatural** condition that may trigger future landsliding at the site.

4. We understand that the location and design of the proposed buildings may evolve and change after the issuance of our report. ***We therefore recommend that our firm be provided the opportunity to review the interim designs and specifications in order to perform supplemental work and issue supplemental recommendations where warranted.*** It is also important that we be given the opportunity to review interim designs and specifications so that our recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of performing the recommended reviews we can assume no responsibility for misinterpretation of our recommendations.

5. We recommend that a representative from our firm be retained to inspect all cuts and pier holes made during grading for the foundation, prior to construction of the footings. This includes key ways excavated for any engineered fills.

Building P1-2 Dining Hall

Topography

This building site sits atop a large bench that appears to have been notched into a bedrock ridge above Carbonera Creek. The bench slopes very gently the southwest and drops off precipitously steep at its southeastern edge where it descends to Carbonera Creek.

Drainage

Natural surface drainage across the area slated for development occurs primarily by overland sheet flow toward the southwest and to the southeast. There appears to be some existing storm drainage structures scattered around the periphery of the existing building, as well as some remnants of old storm drainage systems. Ultimately all the water for the site enters Carbonera Creek, whether it flows naturally down the steep slope flanks or is collected and disposed of in the creek.

None of the systems present at the site are acceptable from the geological perspective, since they appear to direct water toward and upon the steep slope below the site.

Although none of the borings drilled in the vicinity of this building site encountered groundwater, there is the possibility that a small fraction of the rainfall in this area probably infiltrates the ground and enters the groundwater regime. There is a blanket of soil atop the bedrock and the transmissivity contrast between the blanket of soil (higher relative transmissivity) and the underlying bedrock (lower relative transmissivity) is conducive to perched water conditions within the soil.

Earth Materials

The P1-2 Dining Hall site is underlain by a blanket of surficial soil that is between 4½' and 10' thick, overlying either "dirty" Santa Margarita Formation siltstone and silty sandstone bedrock or Santa Cruz Mudstone (which is composed of siltstone, silty sandstone and mudstone in this area). The geometry of the contact between the surficial soils and bedrock is variable and possibly tied to either grading or weathering processes, or a hybrid of the two.

The blanket of soil thins considerably once it intersects the top of the steep slope that descends to Carbonera Creek. We observed scattered construction debris and abandoned drainage pipes on this slope, which indicates that the ground surface has been disturbed by grading and construction operations in the past.

Geologic Hazards For Building P1-2 Dining Hall

The primary potential geologic hazards that could affect the proposed P1-2 Dining Hall site are landsliding, soil creep, erosion, and intense seismic shaking.

The southeastern edge of the site is flanked by a slope that descends steeply to Carbonera Creek. The slope is mantled by relatively loose surficial soil sitting atop the relatively denser underlying bedrock. In our opinion, the surficial soils on the steep slope are prone to failure by both natural and man-made triggers, making future failure of soils likely, resulting in a long term retreat of the top of the steep slope. We have plotted a hypothesized position of the top of the steep slope upon our map of the site (see Plate 1), that takes downcutting of Carbonera Creek into account, as well retreat of the top of the steep slope due failure of the surficial materials.

The site is underlain by a variably thick blanket of soil atop relatively denser bedrock, which may create variable bearing capacity conditions across the foundation. This may result in differential settlement and damage to the foundation and building if the process of differential settlement is not taken into account for the design.

The surficial soils at the site are relatively loose and will be susceptible to erosion, particularly if storm water is not properly controlled and disposed of away from the site.

Seismic shaking at the site will be intense during the next major earthquake along local fault systems. It is important that recommendations regarding seismic shaking be applied where deemed appropriate by the project engineers for the project design, as well as where dictated by minimum prescriptive requirements in the different codes and standards utilized for the project.

Recommendations For Building P1-2 Dining Hall

1. We recommend that any development located southeast of our forecasted future top of slope line depicted upon Plate 1 take into account that up to 8' of earth materials within this zone will slide and slowly creep downslope over the long term. Mitigation recommendations taking the aforementioned geologic process into account should be issued by the project geotechnical and structural engineers. Please note that this recommendation does not preclude more conservative recommendations by the project geotechnical engineer if the outcome of their analyses, including quantitative slope stability analyses, indicates that the wedge of potentially unstable material is deeper.

2. At a minimum, the project engineers should choose the appropriate prescriptive code-based seismic shaking analyses and parameters for their particular analyses.

3. We recommend that all drainage from improved surfaces such as walkways, patios, roofs and roads be collected in impermeable gutters or pipes and carried to Carbonera Creek, or a place deemed suitable for recharge on the campus. At no time should any concentrated discharge (point sources such as pipes dumping into rock dissipaters) be allowed to spill directly onto the ground adjacent to the proposed development or below the development on the steep slope. Any water landing on paved areas should not be allowed to flow toward the proposed developments. The control of runoff is essential for erosion control and prevention of ponding water against the foundation, as well as to prevent landslides from occurring on the steep slopes below the site.

A comprehensive drainage system should be developed by the Project Civil Engineer Of Record, terminating in a disposal system located along Carbonera Creek or in an area where recharge is geologically acceptable.

On a final note regarding drainage, we would like to head off any future drainage recommendations that might be issued by the lead jurisdiction reviewers that will require runoff from all added impervious areas to be retained on site. This type of recommendation is in direct conflict with the general standard of care in engineering geology for hill side drainage mitigation. In light of this observation, we feel that we should be emphatic with our drainage recommendations. Our recommendation is as follows: We do **not** recommend that any groundwater recharge structures be constructed near steep slopes on the site, as injecting all the drain water from the development into a point source at depth will create an **unnatural** condition that may trigger future landsliding at the site.

4. We understand that the location and design of the proposed buildings may evolve and change after the issuance of our report. ***We therefore recommend that our firm be provided the opportunity to review the interim designs and specifications in order to perform supplemental work and issue supplemental recommendations where warranted.*** It is also important that we be given the opportunity to review interim designs and specifications so that our

recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of performing the recommended reviews we can assume no responsibility for misinterpretation of our recommendations.

5. We recommend that a representative from our firm be retained to inspect all cuts and pier holes made during grading for the foundation, prior to construction of the footings. This includes key ways excavated for any engineered fills.

Building P1-3 -Lodging

Topography

This building site sits atop a large bench that overlooks a steep slope that descends to Carbonera Creek. The bench slopes very gently to the south and drops off precipitously steep at its southern edge where it descends to Carbonera Creek.

Drainage

Natural surface drainage across the area slated for development occurs primarily by overland sheet flow toward the south. There appears to be some existing storm drainage structures and paved areas scattered around the periphery of the existing building that direct water toward the street, as well as the steep slope above Carbonera Creek. Ultimately all the water for the site enters Carbonera Creek, whether it flows naturally down the steep slope flanks or is collected and disposed of in the creek.

Some of the systems present at the site are unacceptable from the geological perspective, since they appear to direct water toward and upon the steep slope below the site.

Although only one of the borings drilled in the vicinity of this building site encountered groundwater (PCE boring B-2 at 35' below the ground surface), there is the possibility that a small fraction of the rainfall in this area probably infiltrates the ground and enters the groundwater regime. There is a blanket of soil atop the bedrock and the transmissivity contrast between some of the sandy layers within the blanket of soil (higher relative transmissivity) and the underlying bedrock (lower relative transmissivity) is conducive to perched water conditions within the soil.

Earth Materials

The P1-3 Lodging site is underlain by a blanket of surficial soil, comprised of artificial fill and possibly fluvial terrace deposits, that is between 3½' and 6' thick, overlying either “dirty” Santa Margarita Formation siltstone and Monterey Formation (which is composed of siltstone, silty sandstone and mudstone in this area). The geometry of the contact between the surficial soils

and bedrock is variable and possibly tied to either grading or weathering processes, or a hybrid of the two.

The blanket of soil thins considerably past its intersection with the top of the steep slope that descends to Carbonera Creek. We observed scattered construction debris and abandoned drainage pipes on this slope, which indicates that the ground surface has been disturbed by grading and construction operations in the past.

Geologic Hazards For Building P1-3 Lodging

The primary potential geologic hazards that could affect the proposed P1-1 Spa site are landsliding, soil creep, erosion, and intense seismic shaking.

The southeastern edge of the site is flanked by a slope that descends steeply to Carbonera Creek. The slope is mantled by relatively loose surficial soil sitting atop the relatively denser underlying bedrock. In our opinion, the surficial soils on the steep slope are prone to failure by both natural and man-made triggers, making it likely that the soil will fail in the future, resulting in long term retreat of the top of the steep slope. We have plotted a hypothesized position of the top of the steep slope upon our map of the site (see Plates 1 and 2), that takes downcutting of Carbonera Creek into account, as well retreat of the top of the steep slope due failure of the surficial materials.

The site is underlain by a variably thick blanket of soil atop relatively denser bedrock, which may create variable bearing capacity conditions across the foundation. This may result in differential settlement and damage to the foundation and building if the process of differential settlement is not taken into account for the design.

The surficial soils at the site are relatively loose and soft and will be susceptible to erosion, particularly if storm water is not properly controlled and disposed of away from the site.

Seismic shaking at the site will be intense during the next major earthquake along local fault systems. It is important that recommendations regarding seismic shaking be applied where deemed appropriate by the project engineers for the project design, as well as where dictated by minimum prescriptive requirements in the different codes and standards utilized for the project.

Recommendations For Building P1-3 Lodging

1. We recommend that any development located southeast of our forecasted future top of slope line depicted upon Plate 1 take into account that up to 8' of earth materials within this zone will slide and slowly creep downslope over the long term. Mitigation recommendations taking the aforementioned geologic process into account should be issued by the project geotechnical and structural engineers. Please note that this recommendation does not preclude more conservative

recommendations by the project geotechnical engineer if the outcome of their analyses, including quantitative slope stability analyses, indicates that the wedge of potentially unstable material is deeper.

2. At a minimum, the project engineers should choose the appropriate prescriptive code-based seismic shaking analyses and parameters for their particular analyses.

3. We recommend that all drainage from improved surfaces such as walkways, patios, roofs and roads be collected in impermeable gutters or pipes and carried to Carbonera Creek, or a place deemed suitable for recharge on the campus. At no time should any concentrated discharge (point sources such as pipes dumping into rock dissipaters) be allowed to spill directly onto the ground adjacent to the proposed development or below the development on the steep slope. Any water landing on paved areas should not be allowed to flow toward the proposed developments. The control of runoff is essential for erosion control and prevention of ponding water against the foundation, as well as to prevent landslides from occurring on the steep slopes below the site.

A comprehensive drainage system should be developed by the Project Civil Engineer Of Record, terminating in a disposal system located along Carbonera Creek or in an area where recharge is geologically acceptable.

On a final note regarding drainage, we would like to head off any future drainage recommendations that might be issued by the lead jurisdiction reviewers that will require runoff from all added impervious areas to be retained on site. This type of recommendation is in direct conflict with the general standard of care in engineering geology for hill side drainage mitigation. In light of this observation, we feel that we should be emphatic with our drainage recommendations. Our recommendation is as follows: We do **not** recommend that any groundwater recharge structures be constructed near steep slopes on the site, as injecting all the drain water from the development into a point source at depth will create an **unnatural** condition that may trigger future landsliding at the site.

4. We understand that the location and design of the proposed buildings may evolve and change after the issuance of our report. ***We therefore recommend that our firm be provided the opportunity to review the interim designs and specifications in order to perform supplemental work and issue supplemental recommendations where warranted.*** It is also important that we be given the opportunity to review interim designs and specifications so that our recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of performing the recommended reviews we can assume no responsibility for misinterpretation of our recommendations.

5. We recommend that a representative from our firm be retained to inspect all cuts and pier holes made during grading for the foundation, prior to construction of the footings. This includes key ways excavated for any engineered fills.

Building P2-1 -Program Space And Lodging

Topography

This building site sits astride the nose of a bedrock ridge, near its terminus along Bethany Drive. The nose of the ridge is moderately steep and the flanks are steep.

Drainage

Natural surface drainage across the area slated for development occurs primarily by overland sheet radially off of the ridge from southwest to southeast. The existing flow is captured by various paved areas and drains. The path that the collected water takes once it is captured by the drains is unknown at this point. Presumably all the water collected in this area ultimately enters Carbonera Creek, whether it flows naturally down the steep slope flanks or is collected and disposed of in the creek.

None of the borings drilled in the vicinity of this building site encountered groundwater. The bedrock exposed in the borings and the cut slope in the vicinity of the proposed building site is mudstone, which is not conducive to conducting or storing groundwater. There is a blanket of soil atop the bedrock and the transmissivity contrast between some of the sandy layers within the blanket of soil (higher relative transmissivity) and the underlying bedrock (lower relative transmissivity) is conducive to perched water conditions within the soil.

Earth Materials

The P2-1 Program Space And Lodging site is underlain by mudstone bedrock with a thin veneer of surficial soil up to several feet thick lying on top. The geometry of the contact between the surficial soils and bedrock mostly mimics the geometry of the ground surface, which is consistent with the formation of colluvium atop bedrock.

The bedrock exposed in the cut slope is mudstone that is crushed, ranging in predominant size from 1/2" to 2" in diameter. There are two distinct fracture sets on 1" to 2" spacing that are well developed enough to trigger potential thin slab failures if exposed. One set is roughly perpindicular to the trend of the ridge (i.e. parallel to a potential proposed cut slope for the building footprint) and dips 65 degrees toward the proposed building. The other set is roughly parallel to the trend of the ridge (i.e. perpindcular to the potential proposed cut) and dips 55 degrees to the west. See Plates 1 and 2 for the precise orientation of the fracture sets.

There is an approximately 35-degree cut slope in mudstone bedrock nearby the site and the exposed fractured bedrock appears to be ravelling off of the face in gravel-sized angular fragments. In our evidence this is evidence that the face of that cut slope is too steep and materials are failing under static conditions, albeit in a minor way, and moving down slope.

Geologic Hazards For P2-1 -Program Space And Lodging

The primary potential geologic hazards that could affect the proposed P2-1 -Program Space And Lodging site are landsliding, soil creep, erosion, and intense seismic shaking.

It appears that the proposed building will result in some form of cutting into the nose of the ridge to accommodate the building footprint (personal communication with Gerald Yates and Elizabeth Mitchell). In our opinion, both the bedrock and the overlying soil could potentially fail out of the cut slope, depending upon the configuration of the cut slope. In our opinion, any exposure of fractured mudstone bedrock on that ridge should not exceed 2:1 h:v, and any soil exposure should not exceed 20 degrees (36%). We therefore recommend that the slope be armored or retained in some fashion. Furthermore, care should be taken when excavating because of the orientation of the fracture sets that may parallel the cut slopes - the cut slopes may need to be shored temporarily to be safe, depending upon the height and orientation of the proposed cut.

The surficial soils at the site are relatively loose and soft and will be susceptible to erosion, particularly if storm water is not properly controlled and disposed of away from the site.

Seismic shaking at the site will be intense during the next major earthquake along local fault systems. It is important that recommendations regarding seismic shaking be applied where deemed appropriate by the project engineers for the project design, as well as where dictated by minimum prescriptive requirements in the different codes and standards utilized for the project.

Recommendations For P2-1 -Program Space And Lodging

1. We recommend that the cut slopes for this site be armored or retained in some fashion, if the cut slopes are to exceed 2:1 h:v. We further recommend that the Project Geotechnical Engineer Of Record and Project Civil Engineer Of Record consult with us prior to developing grading plans for this site, given that the cut slopes may need to be shored temporarily to be safe, depending upon the height and orientation of the proposed cut. Please note that this recommendation does not preclude more conservative recommendations by the project geotechnical engineer if the outcome of their analyses indicate that our recommendation is inadequate.

2. At a minimum, the project engineers should choose the appropriate prescriptive code-based seismic shaking analyses and parameters for their particular analyses.

3. We recommend that all drainage from improved surfaces such as walkways, patios, roofs and roads be collected in impermeable gutters or pipes and carried to Carbonera Creek, or a place deemed suitable for recharge on the campus. At no time should any concentrated discharge (point sources such as pipes dumping into rock dissipaters) be allowed to spill directly onto the

ground adjacent to the proposed development or below the development on the steep slope. Any water landing on paved areas should not be allowed to flow toward the proposed developments. The control of runoff is essential for erosion control and prevention of ponding water against the foundation, as well as to prevent landslides from occurring on the steep slopes below the site.

A comprehensive drainage system should be developed by the Project Civil Engineer Of Record, terminating in a disposal system located along Carbonera Creek or in an area where recharge is geologically acceptable.

On a final note regarding drainage, we would like to head off any future drainage recommendations that might be issued by the lead jurisdiction reviewers that will require runoff from all added impervious areas to be retained on site. This type of recommendation is in direct conflict with the general standard of care in engineering geology for hill side drainage mitigation. In light of this observation, we feel that we should be emphatic with our drainage recommendations. Our recommendation is as follows: We do **not** recommend that any groundwater recharge structures be constructed near steep slopes on the site, as injecting all the drain water from the development into a point source at depth will create an **unnatural** condition that may trigger future landsliding at the site.

4. We understand that the location and design of the proposed buildings may evolve and change after the issuance of our report. ***We therefore recommend that our firm be provided the opportunity to review the interim designs and specifications in order to perform supplemental work and issue supplemental recommendations where warranted.*** It is also important that we be given the opportunity to review interim designs and specifications so that our recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of performing the recommended reviews we can assume no responsibility for misinterpretation of our recommendations.

5. We recommend that a representative from our firm be retained to inspect all cuts and pier holes made during grading for the foundation, prior to construction of the footings. This includes key ways excavated for any engineered fills.

Building P2-2 -Parking Structure

Topography

This building site will span almost the entire width and length of an existing infilled drainage swale, remotely located in the northwestern corner of the campus (Plate 1). It appears, at this stage of the preliminary design, that some upper story elements of the structure will span the gap between the building wall and the nose of the bedrock ridge that forms the eastern edge of the swale. The reason for connecting the building to the ridge is to give access to the structure from

a proposed road that will be constructed across the ridge, connecting Gaston Circle with the proposed parking structure.

The site is essentially flat and is accessed by an unpaved road at the southeastern corner of the field, connecting the flat area with the terminus of Bethany Way.

Drainage

Natural surface drainage across the area slated for development occurs primarily by overland sheet flow to the south, toward the outboard edge of the very gently sloped field and the access road that connects Bethany Way with the field. The path that the collected water takes once it is captured by the drains is unknown at this point. Presumably all the water collected in this area ultimately enters Carbonera Creek, whether it flows naturally overland via sheet flow or is collected and disposed of in the creek.

Groundwater was encountered in boring B-17 approximately 47 feet below the ground surface, within the Monterey Formation, after several severe drought years in this region. The site is underlain by a very thick blanket of artificial fill and colluvium, so it is conceivable that some of the rainfall in this area probably infiltrates the ground and enters the groundwater regime. The transmissivity contrast between blanket of soil(higher relative transmissivity) and the underlying bedrock (lower relative transmissivity) is conducive to perched water conditions within the soil. The bedrock exposed in the borings and the cut slope in the vicinity of the proposed building site is mudstone, which is not as conducive to conducting or storing groundwater as the overlying soils. Therefore, it is probably likely that groundwater will perch atop the bedrock within the soils during wetter years.

Earth Materials

The P2-2 Parking Structure site is underlain by a blanket of soil comprised of artificial fill and colluvium, which overlies mudstone bedrock. The blanket of soil thins to nil at the northern end of the existing field and thickens to south toward the fill crest, where the combined thickness of artificial fill and colluvium is approximately 50 feet. The geometry of the contact between the surficial soils and bedrock follows the trajectory of the surface of the old original swale in the longitudinal direction and is mostly “u-shaped” in the transverse direction (orthogonal to axis of the old drainage swale).

It is unknown to us as to whether the artificial fill was adequately engineered when it was placed, so the safest assumption, in our opinion, is that it was not.

The P2-2 Parking Structure site is also ringed by several large and shallow landslide deposits, as well as steep mudstone bedrock slopes blanketed by relatively loose surficial soils that are prone to generating shallow debris flows.

Geologic Hazards For P2-2 -Parking Structure

The primary potential geologic hazards that could affect the proposed P2-2 -Parking Structure are landsliding, soil creep, erosion, differential settlement and intense seismic shaking

As noted in the prior section, the site is ringed by existing landslide deposits and slopes prone to generating shallow debris flows. In our opinion, the current proposed use of open space parking prior to the final design and construction of the proposed phase two work is geologically feasible. However, the proposed parking structure for the second phase will be subject to an unacceptable risk due to landsliding and steps should be taken by the design team to further characterize that risk and appropriately mitigate it.

The blanket of soil that forms the field surface is thick enough to warrant concern for any structures or loads placed upon it. The current use of open parking may simply result in cracking of any asphalt or pavement laid atop the soil, but the construction of a structure as large as a parking structure warrants some concern regarding this hazard.

Seismic shaking at the site will be intense during the next major earthquake along local fault systems. It is important that recommendations regarding seismic shaking be applied where deemed appropriate by the project engineers for the project design, as well as where dictated by minimum prescriptive requirements in the different codes and standards utilized for the project.

Recommendations For P2-2 Parking Structure

1. We recommend that a more extensive investigation of the parking structure site be performed in the future before the design of the structure is finalized. The investigatory work that needs to be completed includes, but is not limited to:
    a. More comprehensive coverage topographic mapping of the slopes that ring the site;
    b. Drilling of small diameter borings or excavation of exploratory test pits by the Project Geotechnical Engineer Of Record and Project Geologist Of Record at select locations on the slopes that ring the site.
    c. Development of comprehensive landslide mitigation schemes, including potential stabilization of existing landslide deposits, construction of debris flow impact walls and debris flow impact berms.

We urge the Project Surveyor Of Record to contact our firm before commencing with any supplemental work that is being driven by geological needs, so we can discuss the degree of coverage necessary and optimal scale for the maps.

2. We recommend that the project engineers consider the potential of risk differential settlement when contemplating the foundation design for the parking structure. The owners should also be made aware that even just paving over the existing field surface for the interim purpose of

parking may result in cracking of the surfacing material due to the ongoing process of differential settlement.

3. At a minimum, the project engineers should choose the appropriate prescriptive code-based seismic shaking analyses and parameters for their particular analyses.

4. We recommend that all drainage from improved surfaces such as walkways, patios, roofs and roads be collected in impermeable gutters or pipes and carried to Carbonera Creek, or a place deemed suitable for recharge on the campus. At no time should any concentrated discharge (point sources such as pipes dumping into rock dissipaters) be allowed to spill directly onto the ground adjacent to the proposed development or below the development on the fill slope. Any water landing on paved areas should not be allowed to flow toward the proposed developments. The control of runoff is essential for erosion control and prevention of ponding water against the foundation, as well as to prevent erosion and shallow failures from occurring on the fill slope below the field surface..

A comprehensive drainage system should be developed by the Project Civil Engineer Of Record, terminating in a disposal system located along Carbonera Creek or in an area where recharge is geologically acceptable.

On a final note regarding drainage, we would like to head off any future drainage recommendations that might be issued by the lead jurisdiction reviewers that will require runoff from all added impervious areas to be retained on site. This type of recommendation is in direct conflict with the general standard of care in engineering geology for hill side drainage mitigation. In light of this observation, we feel that we should be emphatic with our drainage recommendations. Our recommendation is as follows: We do **not** recommend that any groundwater recharge structures be constructed near steep slopes on the site, as injecting all the drain water from the development into a point source at depth will create an **unnatural** condition that may trigger future landsliding at the site.

5. We understand that the location and design of the proposed buildings may evolve and change after the issuance of our report. ***We therefore recommend that our firm be provided the opportunity to review the interim designs and specifications in order to perform supplemental work and issue supplemental recommendations where warranted.*** It is also important that we be given the opportunity to review interim designs and specifications so that our recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of performing the recommended reviews we can assume no responsibility for misinterpretation of our recommendations.

6. We recommend that a representative from our firm be retained to inspect all cuts and pier holes made during grading for the foundation, prior to construction of the footings. This includes key ways excavated for any engineered fills.

Building P2-3 Open Space Parking/Lodging 4-Plexes

We had almost completed our scope of services for this project with the understanding that this second phase site was going to continue to be used as open space parking. We found out too late in the process of completing our investigation and preparing this report that the design premise for this site has changed to a semi-circular layout of residences situated at the head of the infilled drainage swale. We incorporated this layout into our site map (Plate 1), but most assuredly will need to perform future supplemental work and analyses for the residential development scheme, due to the landsliding hazard and risk.

In the meantime, we have proceeded with our geological description of the site and have assessed the hazards and risks with respect to the original premise of continued open space parking in this area and will provide an amended or separate report for the residential design if the client elects to authorize us to provide the recommended supplemental work.

Topography

This building site will span almost the entire width and length of an existing infilled drainage swale, located in Gaston Circle (Plate 1). The surface of the infilled drainage swale is planar and slopes gently to the south. The swale is ringed by very steep hummocky slopes that represent the flanks of the prominent bedrock ridges that form the fabric of the terrain on the campus.

Drainage

Natural surface drainage across the area slated for development occurs primarily by overland sheet flow to the south, toward the mouth of the infilled drainage swale and the center of the existing campus. The path that the collected water takes once it is captured by the drains is unknown at this point. Presumably all the water collected in this area ultimately enters Carbonera Creek, whether it flows naturally overland via sheet flow or is collected and disposed of in the creek.

Groundwater was encountered between 34' and 36' below the ground surface in borings B-13 and B-16 advanced by PCE at this site. The site is underlain by a very thick blanket of artificial fill and colluvium, so it is conceivable that some of the rainfall in this area probably infiltrates the ground and enters the groundwater regime. The transmissivity contrast between blanket of soil(higher relative transmissivity) and the underlying bedrock (lower relative transmissivity) is conducive to perched water conditions within the soil during wetter years. The bedrock exposed in the borings and the cut slope in the vicinity of the proposed building site is mudstone, which is relatively less conducive to conducting or storing groundwater than the overlying soil.

Earth Materials

The P2-3 Open Space Parking site is underlain by a blanket of soil comprised of colluvium, which overlies the dirty sandstone and siltstone bedrock belonging to the Santa Margarita Formation. The blanket of soil thins to nil at the northern end of the existing swale and thickens to south toward mouth of the swale, where the total thickness is approximately 20 feet.

The colluvium that lies within the infilled drainage swale was created through a mixture of gentle deposition of colluvium and the more violent and rapid deposition of debris flows shed off the slopes that surround the drainage. The degree to which the colluvium filling the swale is "reworked" debris flow deposits is commensurate with the distance of from the head of the drainage swale, very near where the newly proposed residential units will be placed. Rogers E. Johnson and Associates (1995) clearly demonstrated in their investigation of the Gaston Circle area that the slopes have been issuing debris flows into the drainage for milliennia, and recommended in their report that the potential risk to the proposed residential development due to the debris flow hazard be mitigated through the construction of a detention basin. Their recommended design, which does not appear to have been implemented, consisted of a large earthern dam intended to stop their design debris flow event of 1,200 cubic yards of saturated soil moving 20 feet per second and an equivalent fluid pressure of 125 pounds per cubic foot (taken directly from Hollingsworth and Kovacs, 1981). We have reviewed their data and analyses and agree with their findings and recommendations for that particular application of residential development.

The P2-3 Open Space Parking site is also ringed by several large and shallow landslide deposits, as well as steep mudstone bedrock slopes blanketed by relatively loose surficial soils that are prone to generating shallow debris flows.

Geologic Hazards For P2-3 Open Space Parking

The primary potential geologic hazards that could affect the proposed P2-3 - Open Space Parking are landsliding, soil creep, erosion and intense seismic shaking

As noted in the prior section, the site is ringed by existing landslide deposits and slopes prone to generating shallow debris flows, some of which have been investigated in the past by other consultants. In our opinion, the current proposed use of open space parking prior to the final design and construction of the proposed phase two work is geologically feasible. As noted at the beginning of the text for this particular site, though, the newly proposed residential development for the second phase will be subject to an unacceptable risk due to landsliding and steps should be taken by the design team to further characterize that risk and appropriately mitigate it where warranted.

Seismic shaking at the site will be intense during the next major earthquake along local fault systems. It is important that recommendations regarding seismic shaking be applied where deemed appropriate by the project engineers for the project design, as well as where dictated by minimum prescriptive requirements in the different codes and standards utilized for the project.

Recommendations For P2-3 Open Space Parking

1. We recommend that a more extensive investigation of the P2-3 Open Space Parking site be performed in the future before the placement of the newly proposed structures is finalized. The investigatory work that needs to be completed includes, but is not limited to:
    a. More comprehensive coverage topographic mapping of the slopes that ring the site;
    b. Drilling of small diameter borings or excavation of exploratory test pits by the Project Geotechnical Engineer Of Record and Project Geologist Of Record at select locations on the slopes that ring the site.
    c. Development of comprehensive landslide mitigation schemes, including potential stabilization of existing landslide deposits, construction of debris flow impact walls and debris flow impact berms.

We urge the Project Surveyor Of Record to contact our firm before commencing with any supplemental work that is being driven by geological needs, so we can discuss the degree of coverage necessary and optimal scale for the maps.

2. At a minimum, the project engineers should choose the appropriate prescriptive code-based seismic shaking analyses and parameters for their particular analyses.

3. We recommend that all drainage from improved surfaces such as walkways, patios, roofs and roads be collected in impermeable gutters or pipes and carried to Carbonera Creek, or a place deemed suitable for recharge on the campus. At no time should any concentrated discharge (point sources such as pipes dumping into rock dissipaters) be allowed to spill directly onto the ground adjacent to the proposed development or below the development on the steep slope. Any water landing on paved areas should not be allowed to flow toward the proposed developments. The control of runoff is essential for erosion control and prevention of ponding water against the foundation, as well as to prevent landslides from occurring on the steep slopes below the site.

A comprehensive drainage system should be developed by the Project Civil Engineer Of Record, terminating in a disposal system located along Carbonera Creek or in an area where recharge is geologically acceptable.

The Gaston Circle area may be a suitable site for attempting to infiltrate water into the ground, because of the nature and thickness of the colluvium that overlies the "dirty" Santa Margarita Formation. Most of the infiltrated water will perch atop the underlying Santa Margarita Formation bedrock and flow to the south, where it will fan out into the blanket of colluvium in the southern part of the campus.

4. We understand that the location and design of the proposed buildings may evolve and change after the issuance of our report. ***We therefore recommend that our firm be provided the opportunity to review the interim designs and specifications in order to perform supplemental work and issue supplemental recommendations where warranted.*** It is also important that we be given the opportunity to review interim designs and specifications so that our recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of performing the recommended reviews we can assume no responsibility for misinterpretation of our recommendations.

5. We recommend that a representative from our firm be retained to inspect all cuts and pier holes made during grading for the foundation, prior to construction of the footings. This includes key ways excavated for any engineered fills.

Phase 2 - Road between Gaston Circle and P2-2 Parking Structure

Topography

It appears that this proposed road will wrap around the nose of the bedrock ridge that divides the Gaston Circle area from the P2-2 Parking Structure site, in order to connect the two sites (Plate 1). The terrain navigated by the road is very steep and hummocky.

Drainage

Natural surface drainage across the road traverse area is via overland flow radially from southwest to southeast, into the respective infilled-drainage swales occupied by the P2-2 Parking Structure site and the Gaston Circle area. All of the surface water draining off is eventually captured by storm drain systems and presumably carried Carbonera Creek.

No subsurface work was done along the alignment of the road, so we cannot comment definitively on groundwater for this area. However, it appears that the road is underlain by thin loose soils sitting stop Santa Cruz Mudstone, so we find it unlikely that groundwater is stored in this area, since mudstone is not conducive to conducting or storing groundwater.

Earth Materials

The Phase 2 Road Between Gaston Circle and P2-2 Parking Structure is underlain by a blanket of colluvium (relatively loose soil derived from the underlying mudstone bedrock in this area) that is several feet thick.

The bedrock exposed in scattered exposures across the ridge is crushed, ranging in predominant size from 1/2" to 2" in diameter. We did not observe reliable exposures of intact bedrock in this area, so we were unable to procure reliable bedrock discontinuity data, such as bedding planes,

joints and fractures. Given our observations at other sites on the former campus though, it seems reasonably prudent to assume that the bedrock may issue potential thin slab failures if exposed in an unretained cut slope.

The colluvium, which is soil derived from the underlying mudstone bedrock in this area, appears to be creeping and sliding across most of the steeper slopes that the road traverses, and in some cases this movement has turned into areally-large shallow earth flows and shallow debris flows (see Plate 1).

Geologic Hazards For Phase 2 Road Between Gaston Circle and P2-2 Parking Structure

The primary potential geologic hazards that could affect the proposed road are landsliding, soil creep, erosion and intense seismic shaking.

The proposed road alignment crosses several shallow landslide deposits, as well as steep terrain covered with loose soils prone to soil creep and shallow landsliding. Based upon the proposed alignment across steep terrain, it is likely that the road will require deep cuts into the underlying soil and bedrock. Potential instability to the cuts (and possibly related fills) may result unless the cut slopes are adequately retained. Additionally, the proposed road may be damaged and vehicular access eliminated if either of the two landslides it crosses remobilize in the future during large storms or earthquakes.

The surficial soils that lie atop the mudstone bedrock along the proposed alignment are relatively loose and soft and will be susceptible to erosion, particularly if storm water is not properly controlled and disposed of away from the road.

Seismic shaking at the site will be intense during the next major earthquake along local fault systems. It is important that recommendations regarding seismic shaking be applied where deemed appropriate by the project engineers for the project design, as well as where dictated by minimum prescriptive requirements in the different codes and standards utilized for the project.

Recommendations For Phase 2 Road Between Gaston Circle and P2-2 Parking Structure

1. We recommend that a more extensive investigation of the proposed road alignment be performed in the future before the placement of the road is finalized. The investigatory work that needs to be completed includes, but is not limited to:
    a. More comprehensive coverage of topographic mapping of the slopes the road cuts across.;
    b. Drilling of small diameter borings or excavation of exploratory test pits by the Project Geotechnical Engineer Of Record and Project Geologist Of Record at select locations along the proposed road alignment;

c. Development of comprehensive landslide mitigation schemes, including potential stabilization of existing landslide deposits and construction of retaining structures for inboard cut slopes.

We urge the Project Surveyor Of Record to contact our firm before commencing with any supplemental work that is being driven by geological needs, so we can discuss the degree of coverage necessary and optimal scale for the maps.

2. At a minimum, the project engineers should choose the appropriate prescriptive code-based seismic shaking analyses and parameters for their particular analyses.

3. We recommend that all drainage from improved surfaces such as walkways, patios, roofs and roads be collected in impermeable gutters or pipes and carried away from the proposed road alignment, or a place deemed suitable for recharge on the campus. At no time should any concentrated discharge (point sources such as pipes dumping into rock dissipaters) be allowed to spill directly onto the ground adjacent to the proposed road or below the road on steep slopes. Any water landing on paved areas should not be allowed to flow toward the steep slopes below. The control of runoff is essential for erosion control and prevention of landslides from occurring on the steep slopes below the road.

A comprehensive drainage system should be developed by the Project Civil Engineer Of Record, terminating in a disposal system located along Carbonera Creek or in an area where recharge is geologically acceptable.

The Gaston Circle area may be a suitable site for attempting to infiltrate water into the ground, because of the nature and thickness of the colluvium that overlies the "dirty" Santa Margarita Formation. Most of the infiltrated water will perch atop the underlying Santa Margarita Formation bedrock and flow to the south, where it will fan out into the blanket of colluvium in the southern part of the campus.

4. We understand that the location and design of the proposed road alignment may evolve and change after the issuance of our report. ***We therefore recommend that our firm be provided the opportunity to review the interim designs and specifications in order to perform supplemental work and issue supplemental recommendations where warranted.*** It is also important that we be given the opportunity to review interim designs and specifications so that our recommendations may be properly interpreted and implemented. If our firm is not accorded the privilege of performing the recommended reviews we can assume no responsibility for misinterpretation of our recommendations.

5. We recommend that a representative from our firm be retained to inspect all cuts and pier holes made during grading for the foundation, prior to construction of the footings. This includes key ways excavated for any engineered fills.

Remodeled Buildings

We did not investigate the buildings that will be remodeled or repurposed in the future. We do not know if the buildings will be reconstructed to the extent that their foundations need replacing. Additionally, there isn't sufficient coverage of subsurface work in order for us to analyze the conditions under the buildings.

None of the buildings slated for remodeling, except building R7, appear to be threatened by geological hazards that present unacceptably high risks to the structures and the occupants. If the design team requires specific input for a known geological condition or needs further explanation as to how a geological condition could have caused the genesis of any existing damage to the footings or buildings, we can address those issues with supplemental investigations. In the meantime, we do not have any further recommendations for the proposed remodel buildings, except for Building R7.

R7 is notched into the toe of a steep slope, adjacent to an existing landslide deposit. The slope behind the existing building rises steeply for an approximate elevation gain of fifty feet, and is mantled by a thin, relatively loose and soft blanket of colluvium that sits atop mudstone bedrock. Due to the aforementioned geological conditions, there is a potential for the structure to be struck by a damaging fast moving debris flow that initiates on the slope behind the building.

We therefore recommend that a supplemental debris flow investigation be pursued prior to the remodel, to ensure that the building doesn't need some form of protection from debris flows on the upslope side, or that the slope doesn't need to be stabilized above the building. The investigatory work that needs to be completed includes, but is not limited to:

a. More comprehensive coverage of topographic mapping of the slope above Building R7;
b. Drilling of small diameter borings or excavation of exploratory test pits by the Project Geotechnical Engineer Of Record and Project Geologist Of Record at select locations on the slope above Building R7;
c. Development of comprehensive landslide mitigation schemes where warranted, including potential stabilization of loose soil on the slope above the building or construction of debris flow impact structures or debris flow deflection structures.

We urge the Project Surveyor Of Record to contact our firm before commencing with any supplemental work that is being driven by geological needs, so we can discuss the degree of coverage necessary and optimal scale for the maps.

INVESTIGATION LIMITATIONS

1. The conclusions and recommendations noted in this report are based on probability and in no way imply the site will not possibly be subjected to ground failure or seismic shaking so intense that structures will be severely damaged or destroyed. The report does suggest that pursuing mitigation measures structures at the subject site, in compliance with the recommendations noted in this report, will result in an "ordinary risks"to the proposed developments as defined in Appendix B.

2. This report is issued with the understanding that it is the duty and responsibility of the owner or his representative or agent to ensure that the recommendations contained in this report are brought to the attention of the architect and engineer for the project, incorporated into the plans and specifications, and that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. If any unexpected variations in soil conditions or if any undesirable conditions are encountered during construction or if the proposed construction will differ from that planned at the present time, Zinn Geology should be notified so that supplemental recommendations can be given.

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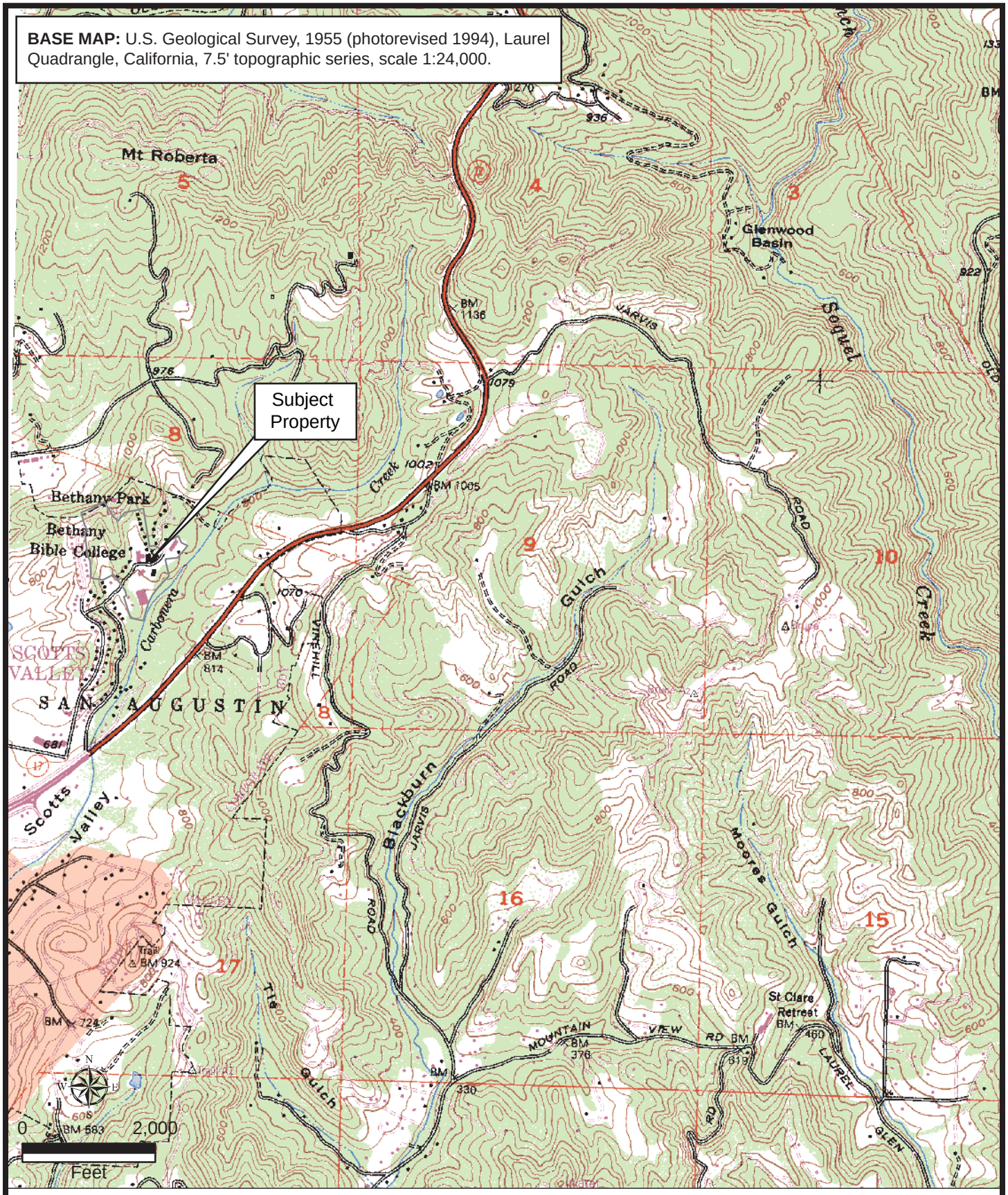
Working Group on California Earthquake Probabilities), 2003, Earthquake probabilities in the San Francisco Bay region: 2002-2031: U.S. Geological Survey Open-File Report 03-214.

Yanev, P., 1991, Peace of Mind in Earthquake Country, Chronicle Books, San Francisco, 218 p.

APPENDIX A

FIGURES



ZINN GEOLOGY

Topographic Index Map

Remodel Of Bethany University By The 1440 Foundation

Bethany Drive

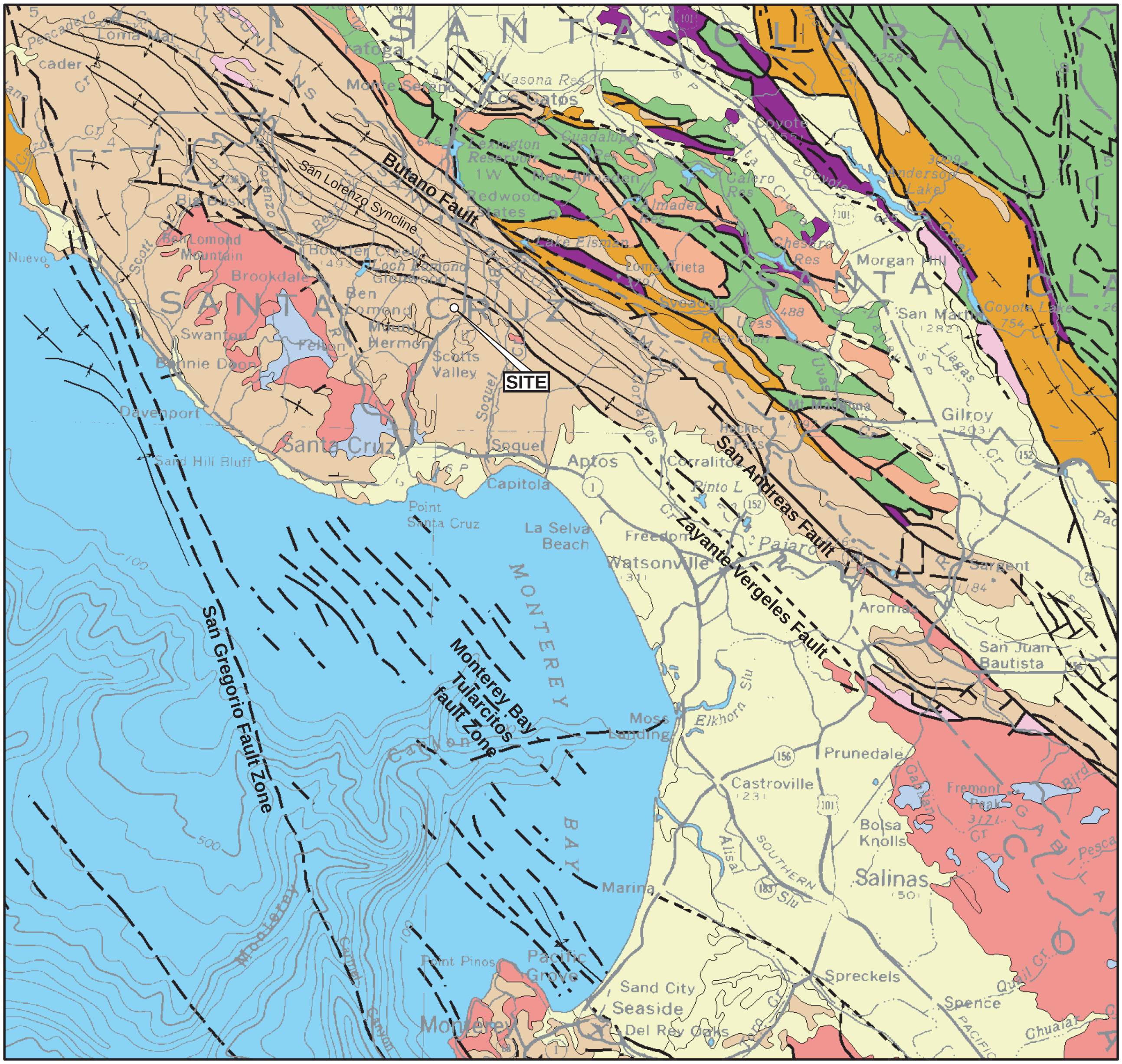
Scotts Valley, California

FIGURE #

1

JOB #

2014001-G-SC



Reference: Jennings, C.W., 1977, Geologic Map of California: California Department of Conservation, Division of Mines and Geology, scale 1:750,000.
Digital Data: Saucedo, G.J., Bedford, D.R., Raines, G.L., Miller, R.J., and Wentworth, C.M., 2000, GIS Data for the Geologic Map of California: California Department of Conservation, Division of Mines and Geology, CD-ROM 2000-007, ver. 2.0.

EXPLANATION

Geologic Units

- Quaternary Deposits
- Quaternary Volcanics
- Tertiary Sedimentary Rocks
- Tertiary Volcanic Rocks
- Pre-Tertiary Sedimentary Rocks
- Pre-Tertiary Volcanic Rocks
- Grantic Intrusive Rocks
- Franciscan Complex
- Ultramafic Rocks
- Pre-Tertiary Metamorphic Rock
- Pre-Cambrian Metamorphic and Igneous Rocks

Symbols

- contact
- fault, certain
- fault,approx. located
- fault, concealed or inferred
- anticline
- monocline
- syncline

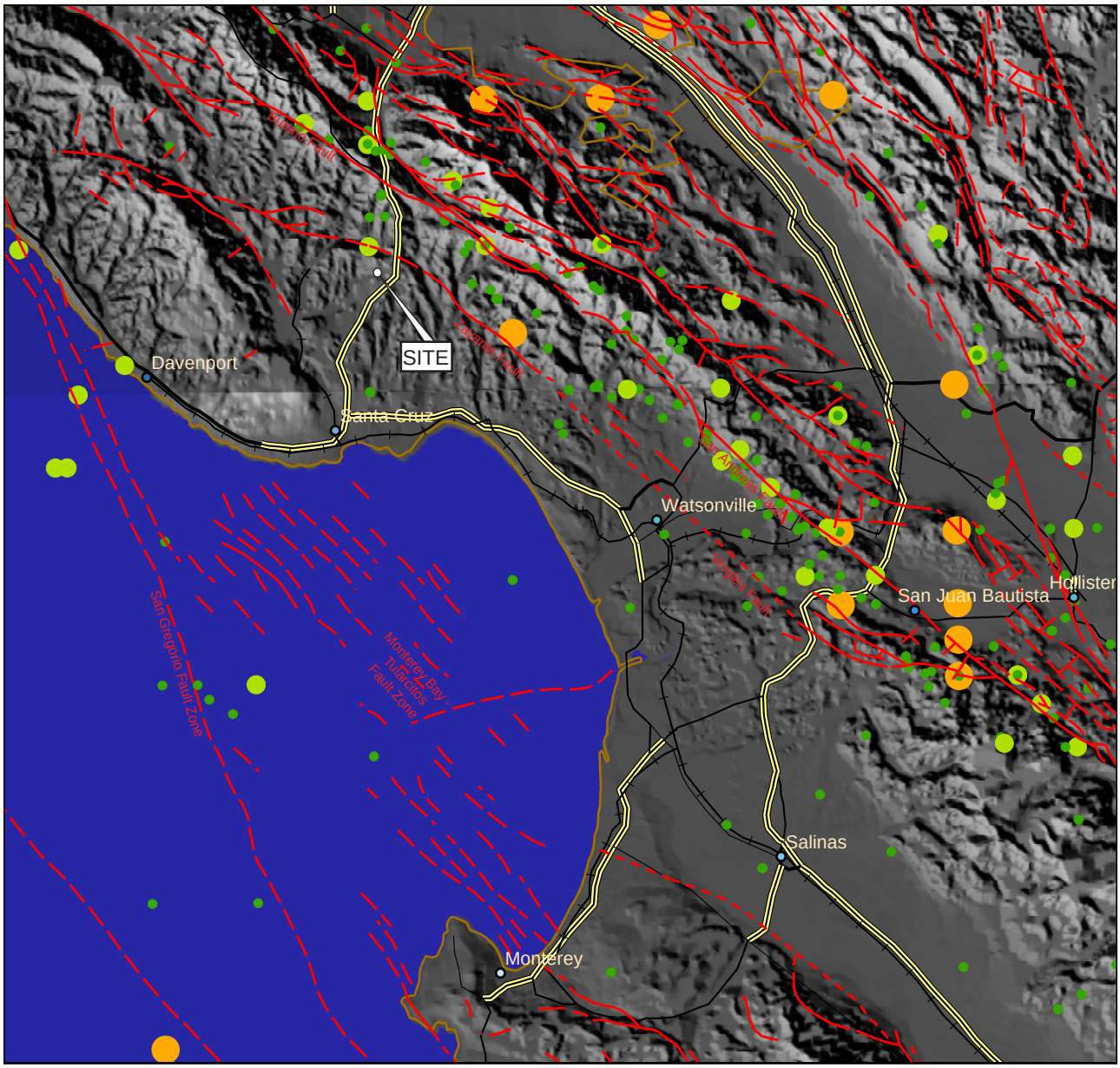
N
W E
S

SCALE 1:500,000
10 Miles 0

ZINN GEOLOGY

Regional Geologic Map
Remodel Of Bethany University By The 1440 Foundation
Bethany Drive
Scotts Valley, California

FIGURE #
2
JOB #
2014001-G-SC



Seismicity Information: Magnitude 4 and greater earthquakes, compiled from various sources, 1769 to 2000; available at www.consrv.cagov/CGS/rghm/quakes/cgs2000_fnl.txt
Fault Information: Jennings, C.W., 1977, Geologic map of California: California Department of Conservation, Division of Mines and Geology, scale 1:750,000

EXPLANATION

Symbols

- fault, certain
- fault,approx. located
- fault, concealed or inferred

Earthquake Magnitude

- 4.0 to 4.99
- 5.0 to 5.99
- 6.0 to 6.99
- 7.0 +

N
W E
S

SCALE 1:500,000

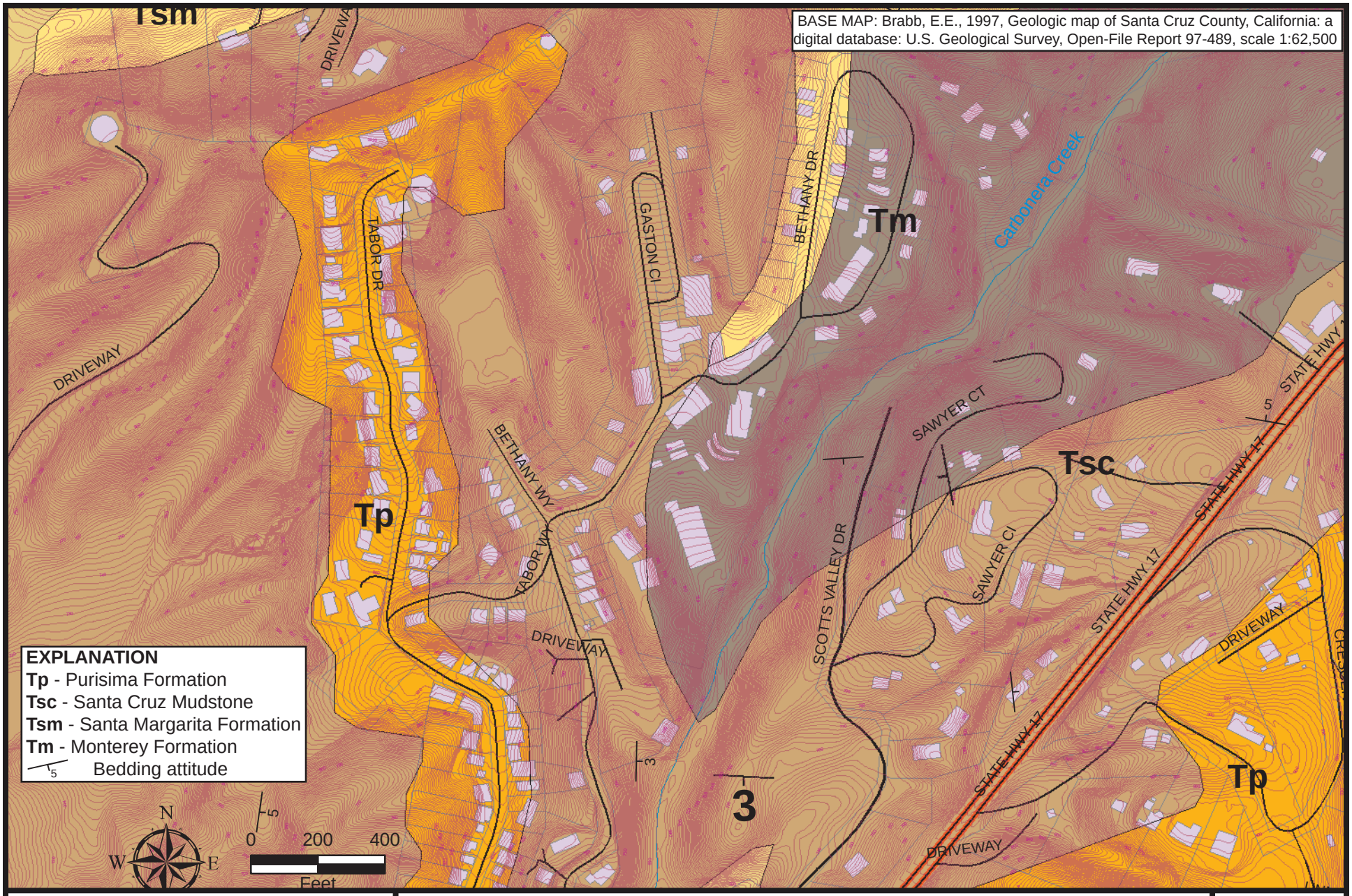
10 5 0 10

Miles

ZINN GEOLOGY

Regional Seismicity Map
Remodel Of Bethany University By The 1440 Foundation
Bethany Drive
Scotts Valley, California

FIGURE #
3
JOB #
2014001-G-SC



ZINN GEOLOGY

Local Geologic Map

Remodel Of Bethany University By The 1440 Foundation

Bethany Drive

Scotts Valley, California

FIGURE #

4

JOB #

2014001-G-SC

APPENDIX B

SCALE OF ACCEPTABLE RISKS FROM GEOLOGIC HAZARDS

SCALE OF ACCEPTABLE RISKS FROM SEISMIC GEOLOGIC HAZARDS

Risk Level	Structure Types	Extra Project Cost Probably Required to Reduce Risk to an Acceptable Level
Extremely low[1]	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	No set percentage (whatever is required for maximum attainable safety).
Slightly higher than under "Extremely low" level.[1]	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	5 to 25 percent of project cost.[2]
Lowest possible risk to occupants of the structure.[3]	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	5 to 15 percent of project cost.[4]
An "ordinary" level of risk to occupants of the structure.[3,5]	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1 to 2 percent of project cost, in most cases (2 to 10 percent of project cost in a minority of cases).[4]

1 Failure of a single structure may affect substantial populations.

2 These additional percentages are based on the assumptions that the base cost is the total cost of the building or other facility when ready for occupancy. In addition, it is assumed that the structure would have been designed and built in accordance with current California practice. Moreover, the estimated additional cost presumes that structures in this acceptable risk category are to embody sufficient safety to remain functional following an earthquake.

3 Failure of a single structure would affect primarily only the occupants.

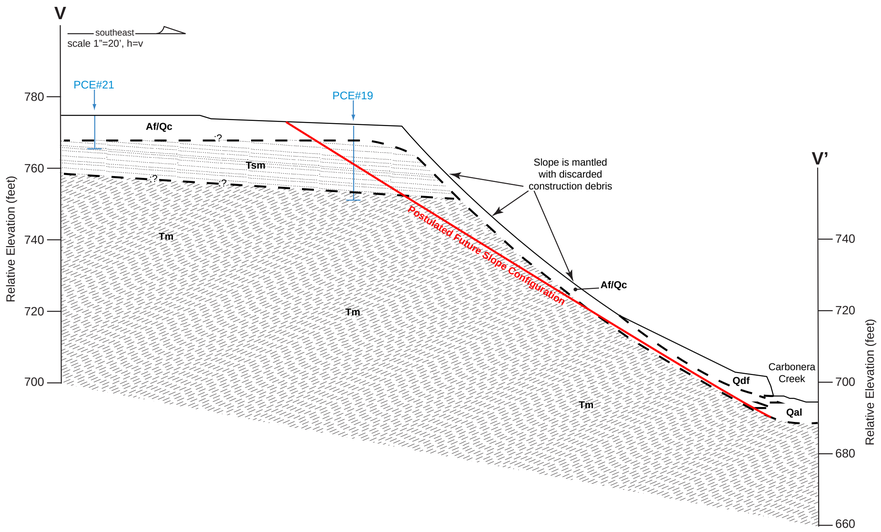
4 These additional percentages are based on the assumption that the base cost is the total cost of the building or facility when ready for occupancy. In addition, it is assumed that the structures would have been designed and built in accordance with current California practice. Moreover the estimated additional cost presumes that structures in this acceptable-risk category are to be sufficiently safe to give reasonable assurance of preventing injury or loss of life during and following an earthquake, but otherwise not necessarily to remain functional.

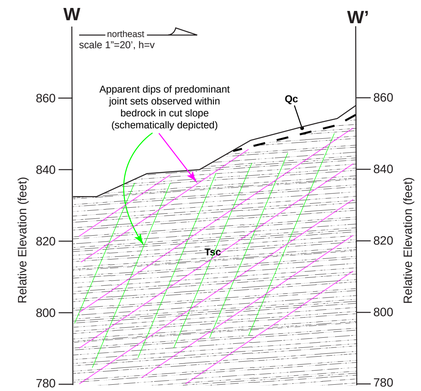
5 "Ordinary risk": Resist minor earthquakes without damage: resist moderate earthquakes without structural damage, but with some non-structural damage; resist major earthquakes of the intensity or severity of the strongest experienced in California, without collapse, but with some structural damage as well as non-structural damage. In most structures it is expected that structural damage, even in a major earthquake, could be limited to repairable damage. (Structural Engineers Association of California)

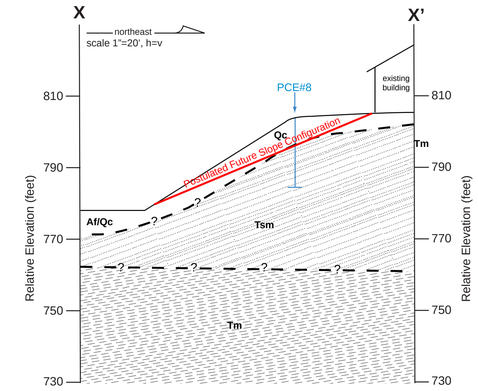
Source: *Meeting the Earthquake,* Joint Committee on Seismic Safety of the California Legislature, Jan. 1974, p.9.

SCALE OF ACCEPTABLE RISKS FROM NON-SEISMIC GEOLOGIC HAZARDS[6]		
Risk Level	Structure Type	Risk Characteristics
Extremely low risk	Structures whose continued functioning is critical, or whose failure might be catastrophic: nuclear reactors, large dams, power intake systems, plants manufacturing or storing explosives or toxic materials.	1. Failure affects substantial populations, risk nearly equals nearly zero.
Very low risk	Structures whose use is critically needed after a disaster: important utility centers; hospitals; fire, police and emergency communication facilities; fire station; and critical transportation elements such as bridges and overpasses; also dams.	1. Failure affects substantial populations. Risk slightly higher than 1 above.
Low risk	Structures of high occupancy, or whose use after a disaster would be particularly convenient: schools, churches, theaters, large hotels, and other high rise buildings housing large numbers of people, other places normally attracting large concentrations of people, civic buildings such as fire stations, secondary utility structures, extremely large commercial enterprises, most roads, alternative or non-critical bridges and overpasses.	1. Failure of a single structure would affect primarily only the occupants.
"Ordinary" risk	The vast majority of structures: most commercial and industrial buildings, small hotels and apartment buildings, and single family residences.	1. Failure only affects owners /occupants of a structure rather than a substantial population. 2. No significant potential for loss of life or serious physical injury. 3. Risk level is similar or comparable to other ordinary risks (including seismic risks) to citizens of coastal California. 4. No collapse of structures; structural damage limited to repairable damage in most cases. This degree of damage is unlikely as a result of storms with a repeat time of 50 years or less.
Moderate risk	Fences, driveways, non-habitable structures, detached retaining walls, sanitary landfills, recreation areas and open space.	1. Structure is not occupied or occupied infrequently. 2. Low probability of physical injury. 3. Moderate probability of collapse.

[6] Non-seismic geologic hazards include flooding, landslides, erosion, wave runup and sinkhole collapse







NOTES

1. Scale of cross section is different than the scale of the map on Plate 1.
2. Apparent dip of bedrock discontinuities is depicted schematically with patterning and lines.

EXPLANATION

Earth Materials

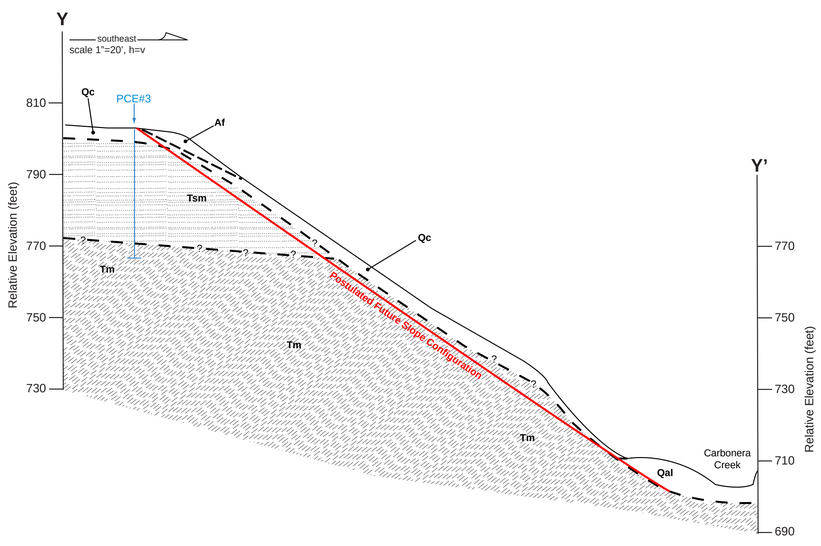
Af/Qc	Combined artificial fill and colluvium - designated where too difficult to tell from boring data or geomorphology
Af	Artificial fill
Qc	Colluvium
Qal	Alluvium
Tsc	Santa Cruz Mudstone
Tsm	Santa Margarita Formation
Tm	Monterey Formation

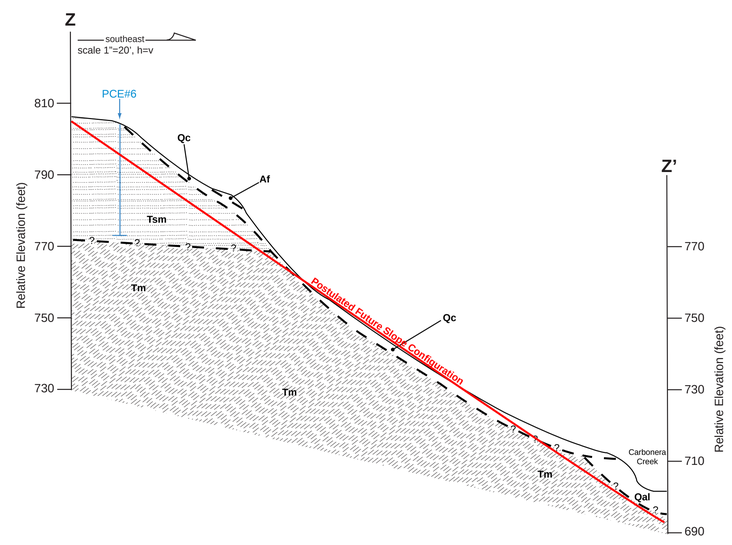
Symbols

Earth materials contact - dashed where approximate, queried where uncertain

PCE#6

Boring advanced by Pacific Crest Engineering for this investigation during March 2014





PROFESSIONAL GEOLOGIST
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Geologic Cross Sections
Proposed Redevelopment Of
Bethany University Campus By The 1440 Foundation
800 Bethany Drive
Scotts Valley, California

Date: 21 April 2014	Revised:
Job #2014001-G-SC	
Scale: 1"=20', h=v	**Plate 2**
Drawn by: ENZ/enz	

EXPLANATION

EARTH MATERIALS

Af	Artificial fill
Qc	Colluvium
Tsc	Santa Cruz Mudstone
Tsm	Santa Margarita Formation
Tm	Monterey Formation

SYMBOLS

Earth materials contact - dashed where approximate

PCE #5 — Boring advanced by Pacific Crest Engineering for this investigation

PCE BC#3 — Boring advanced by Pacific Crest Engineering in 2009 for Bethany Cafe

PCE SH #7 — Boring advanced by Pacific Crest Engineering in 2002 for Student Housing Project

PCE SHP #5 — Boring advanced by Pacific Crest Engineering in 2002 for Bethany Loop Student Housing Project

REJA #4 — Boring advanced by Steven Raas and Associates in 1994 for Bethany University Campus (locations taken from map acccompanying Rogers E. Johnson & Associates 1994 geologic report)

SRA #1 — Boring advanced by Rogers E. Johnson & Associates in 1995 for Groundwater Recharge Investigation

HKA #4 — Boring advanced by Haro, Kasunich & Associates in 2001 for Soccer Field project

W — W' — Line of geological cross section - see section on Plate 2

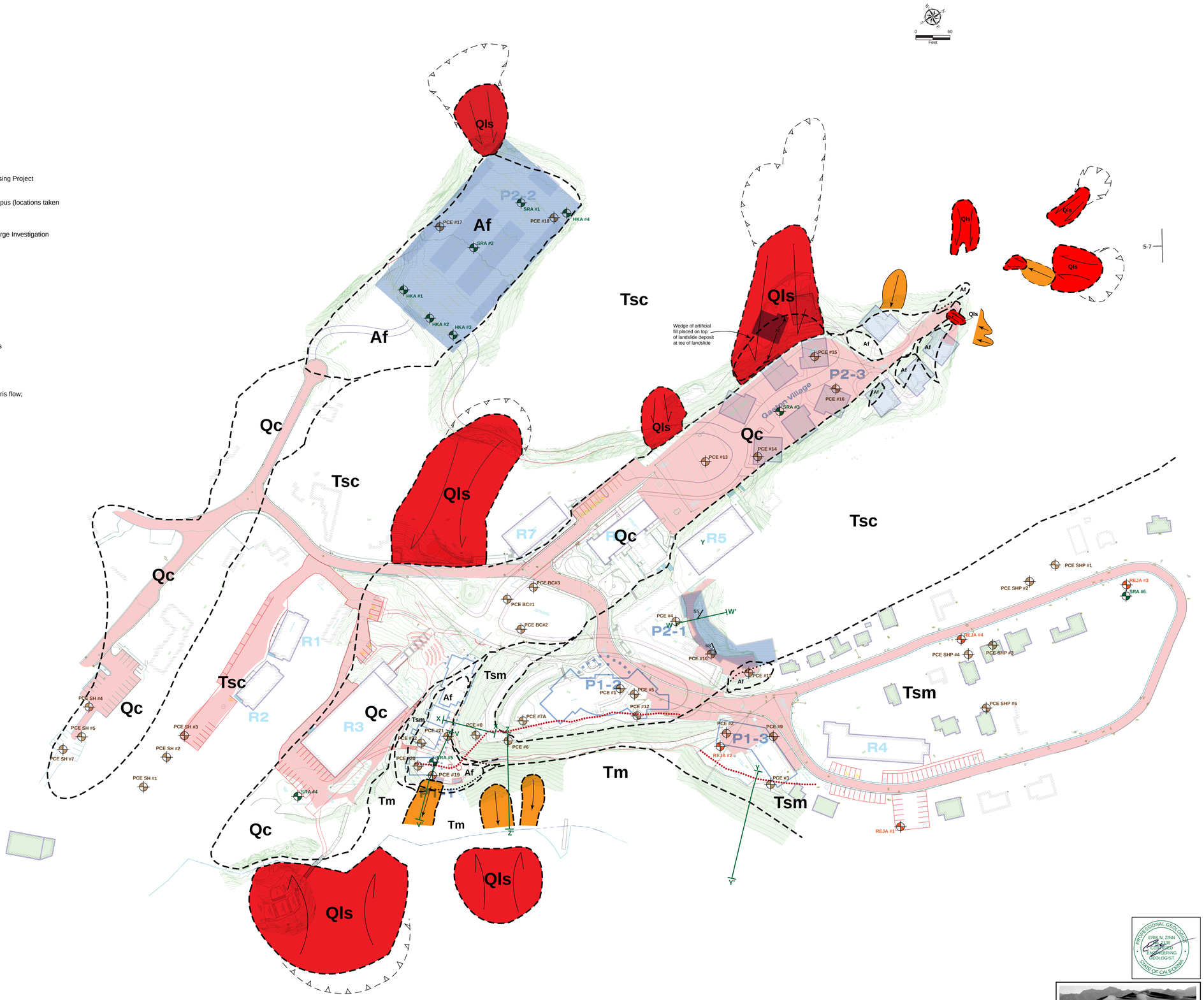
Landslide scar - dashed barbed line indicates the outer boundary of scar left behind on landscape after earth materials mobilzed and moved downslope

Landslide deposit (shaded red) - arrows denote general movment direction of deposits

Debris flow scar (shaded orange) - evacuation area left behind after occurence of debris flow; arrow indicates direction soil moved during evacuation of the scar

68 — Joint or fracture attitude

5-7 — Bedding attitude



Phase II New Buildings					
Building	Use	Area (sq. ft.)	Rooms/ Beds	Stories	Comments
P2 -1	Program Space & Lodging	18,000	22 / 44	2	
P2-2	Parking Structure	60,000		2	
P2-3	Lodging 4 Plexes	40,000	48/96	2	
Phase II Totals		158,000	70 / 140		

Phase I New Buildings					
Building #	Use	Area (sq. ft.)	Rooms / Beds	Stories	Comments
P1-1	Spa	7000	0		
P1-2	Dining Hall	13,500	0	1	
P1-3	Lodging	35,000	59 / 118	4	New Spot
Total		55,500			

Phase I Remodeled Buildings					
Building #	Use	Area (sq. ft.)	Rooms / Beds	Stories	Name
R1	Lodging	7050	22 / 44	2	Gerhart Hall
R2	Lodging	7050	22 / 44	2	Harp Hall
R3	Assembly & Lodging	22,074	6 Bunk Dorms 62 Beds	2	Redwood Auditorium
R4	Lodging	21,340	46 /92	3	Burnett Hall
R5	Classrooms & Assembly	15,820		2	Williams Hall
R6	Assembly	5135		1	Craig Chapel
R7	Administration	16,170		3	Stowell Center
Total		94,339	360 Total Beds		

BASE MAP: Topographic base map and proposed development map were extracted from a digital file given to us by Yates Architecture on 11 April 2014. Intended scale of publication displayed on digital map file is 1"=60'. The reader should check to ensure that this base map is current with respect to whatever future elements of the project are being referenced, since this base map is not the final design drawing.

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Geologic Site Map
Proposed Redevelopment Of
Bethany University Campus By The 1440 Foundation
800 Bethany Drive
Scotts Valley, California

Date: 21 April 2014 Revised:

Job #2014001-G-SC

Scale: 1"=60'

Drawn by: ENZ/enz

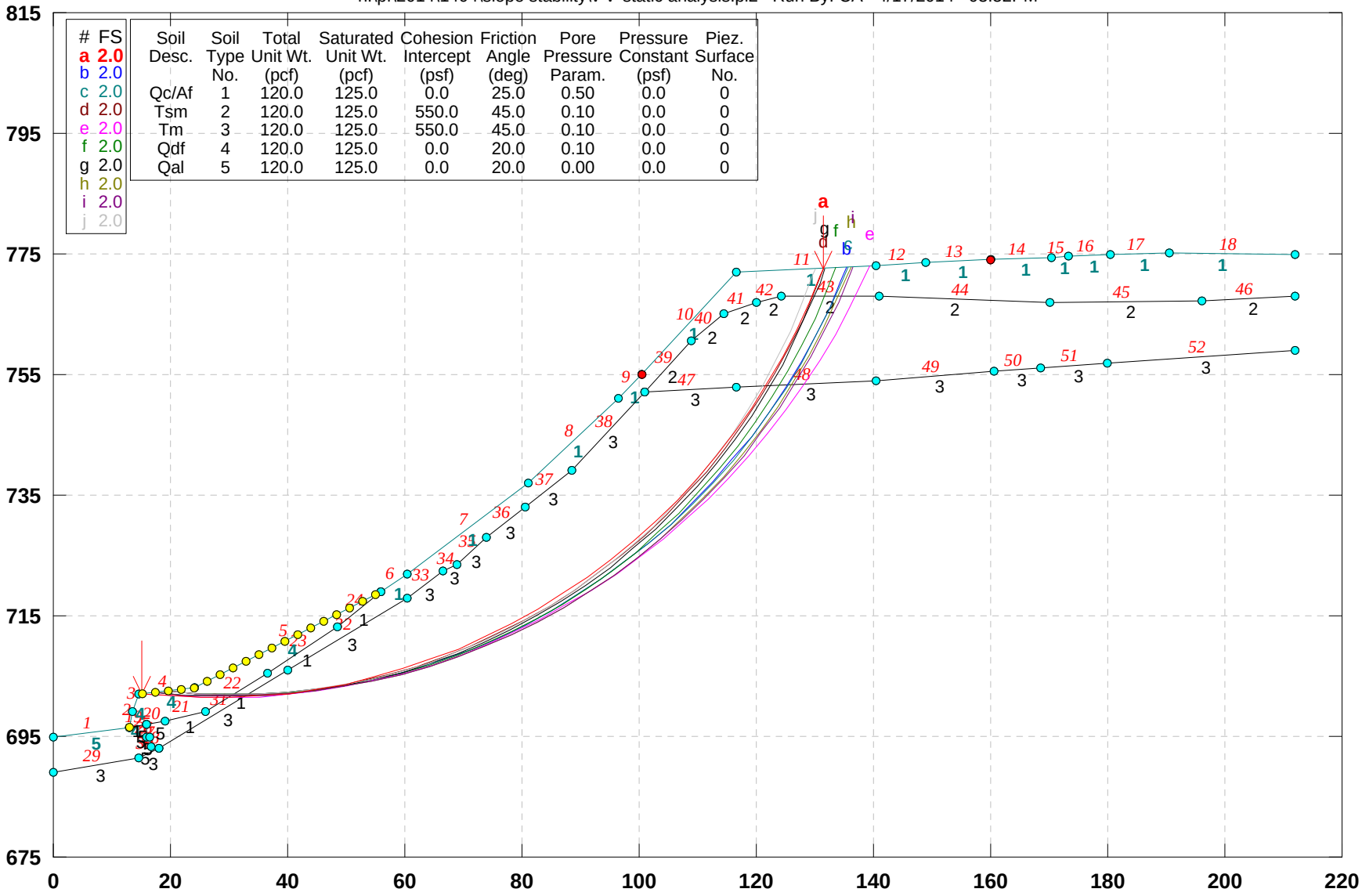
Plate 1

APPENDIXC

Slope Stability Calculations

1440 Growth Center V-V' Static Analysis

h:\pf\2014\1404\slope stability\v-v' static analysis.pl2 Run By: CA 4/17/2014 03:52PM



#	FS
a	2.0
b	2.0
c	2.0
d	2.0
e	2.0
f	2.0
g	2.0
h	2.0
i	2.0
j	2.0

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qc/Af	1	120.0	125.0	0.0	25.0	0.50	0.0	0
Tsm	2	120.0	125.0	550.0	45.0	0.10	0.0	0
Tm	3	120.0	125.0	550.0	45.0	0.10	0.0	0
Qdf	4	120.0	125.0	0.0	20.0	0.10	0.0	0
Qal	5	120.0	125.0	0.0	20.0	0.00	0.0	0

GSTABL7 v.2 FSmin=2.0

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

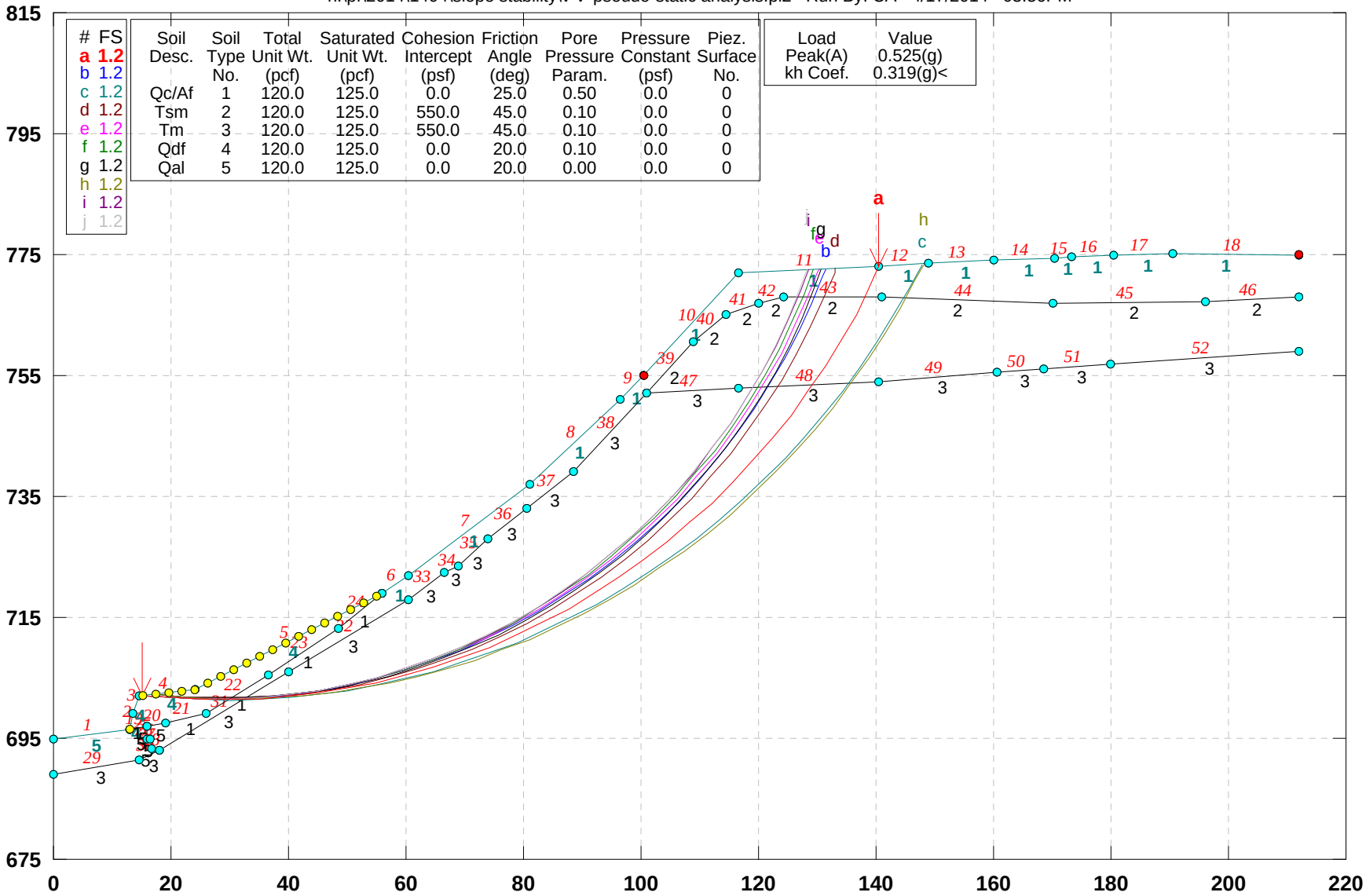
1440 Growth Center V-V' Pseudo-Static Analysis

h:\pf\2014\1404\slope stability\v-v' pseudo-static analysis.pl2 Run By: CA 4/17/2014 03:56PM

#	FS
a	1.2
b	1.2
c	1.2
d	1.2
e	1.2
f	1.2
g	1.2
h	1.2
i	1.2
j	1.2

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qc/Af	1	120.0	125.0	0.0	25.0	0.50	0.0	0
Tsm	2	120.0	125.0	550.0	45.0	0.10	0.0	0
Tm	3	120.0	125.0	550.0	45.0	0.10	0.0	0
Qdf	4	120.0	125.0	0.0	20.0	0.10	0.0	0
Qal	5	120.0	125.0	0.0	20.0	0.00	0.0	0

Load	Value
Peak(A)	0.525(g)
kh Coef.	0.319(g)<



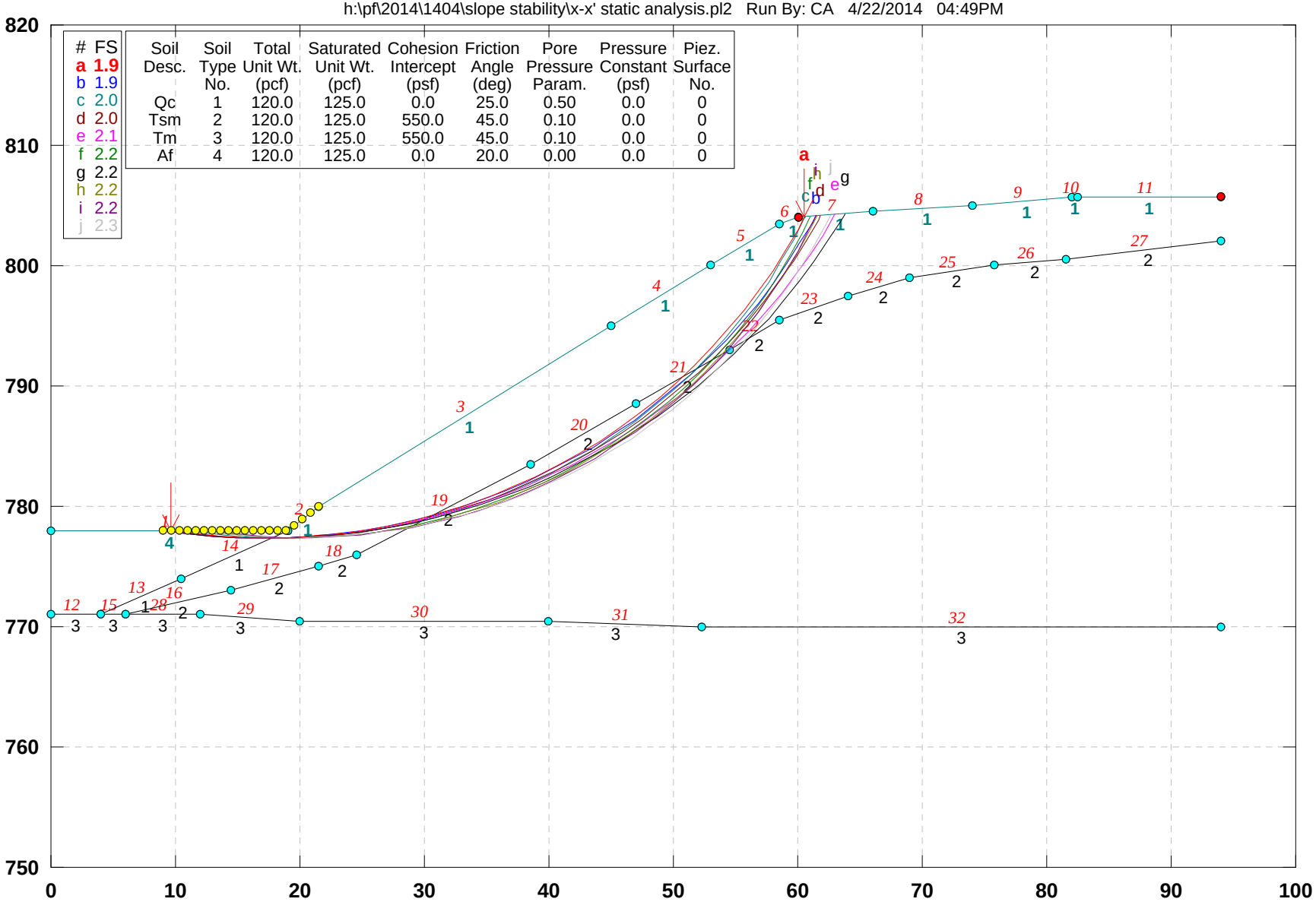
GSTABL7 v.2 FSmin=1.2

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

1440 Retreat Center X-X' Static Analysis

h:\pf\2014\1404\slope stability\x-x' static analysis.pl2 Run By: CA 4/22/2014 04:49PM



#	FS
a	1.9
b	1.9
c	2.0
d	2.0
e	2.1
f	2.2
g	2.2
h	2.2
i	2.2
j	2.3

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qc	1	120.0	125.0	0.0	25.0	0.50	0.0	0
Tsm	2	120.0	125.0	550.0	45.0	0.10	0.0	0
Tm	3	120.0	125.0	550.0	45.0	0.10	0.0	0
Af	4	120.0	125.0	0.0	20.0	0.00	0.0	0

GSTABL7 v.2 FSmin=1.9

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

1440 Retreat Center X-X' Pseudo-Static Analysis

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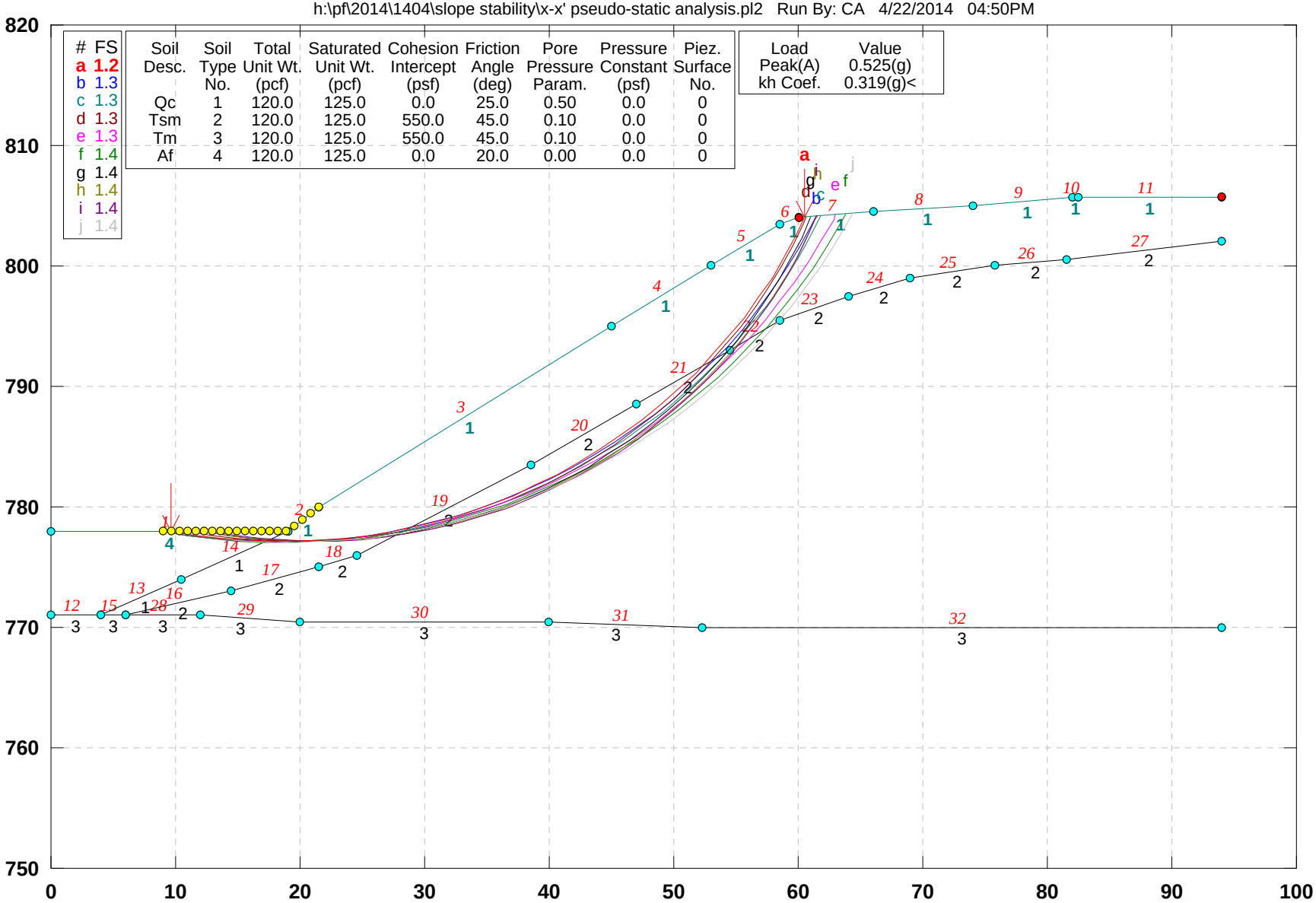

#	FS
a	1.2
b	1.3
c	1.3
d	1.3
e	1.3
f	1.4
g	1.4
h	1.4
i	1.4
j	1.4

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qc	1	120.0	125.0	0.0	25.0	0.50	0.0	0
Tsm	2	120.0	125.0	550.0	45.0	0.10	0.0	0
Tm	3	120.0	125.0	550.0	45.0	0.10	0.0	0
Af	4	120.0	125.0	0.0	20.0	0.00	0.0	0

Load	Value
Peak(A)	0.525(g)
kh Coef.	0.319(g)<

GSTABL7 v.2 FSmin=1.2

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

1440 Retreat Center Y-Y' Static Analysis

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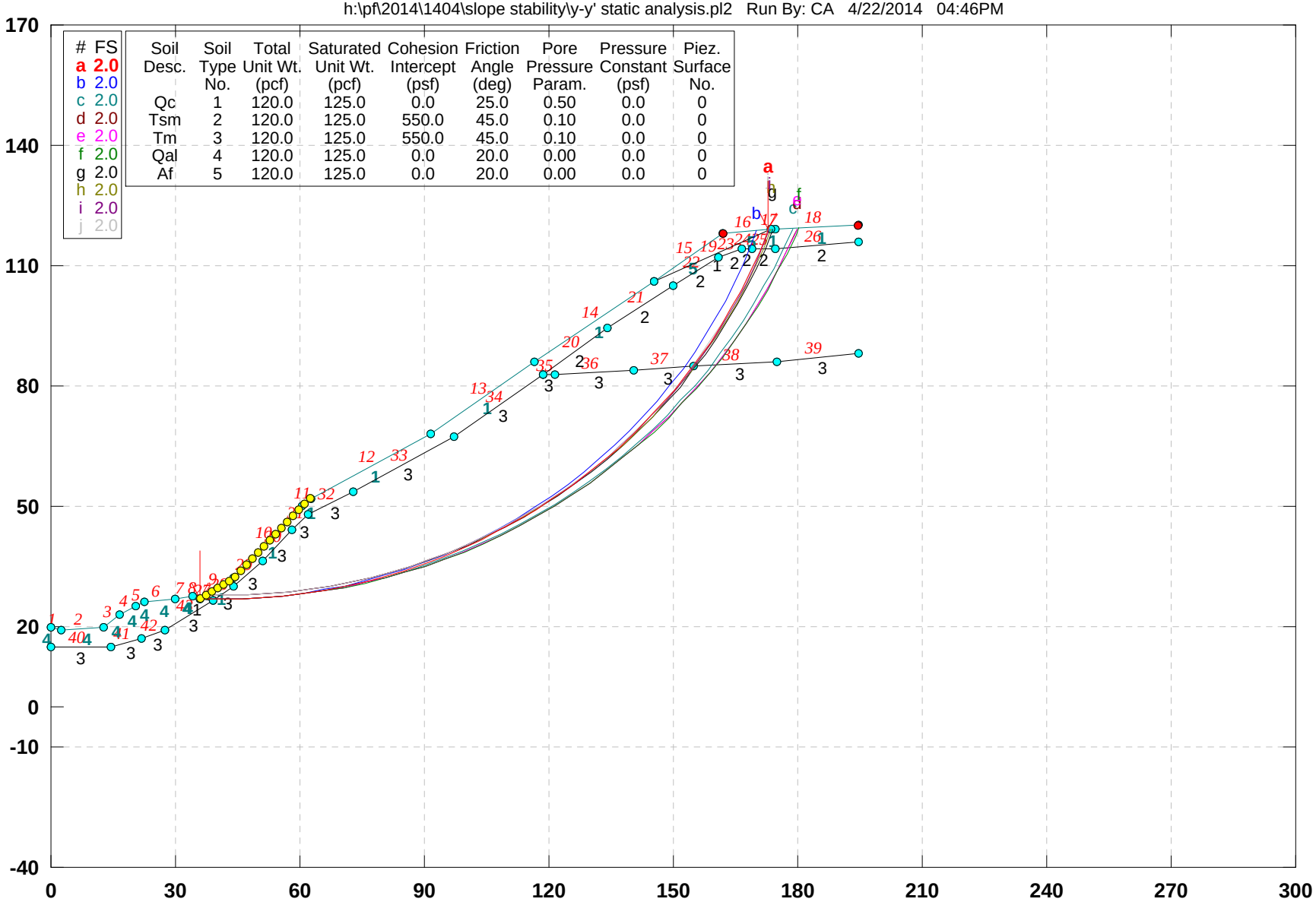

#	FS
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b	2.0
c	2.0
d	2.0
e	2.0
f	2.0
g	2.0
h	2.0
i	2.0
j	2.0

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qc	1	120.0	125.0	0.0	25.0	0.50	0.0	0
Tsm	2	120.0	125.0	550.0	45.0	0.10	0.0	0
Tm	3	120.0	125.0	550.0	45.0	0.10	0.0	0
Qal	4	120.0	125.0	0.0	20.0	0.00	0.0	0
Af	5	120.0	125.0	0.0	20.0	0.00	0.0	0

GSTABL7 v.2 FSmin=2.0

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

1440 Retreat Center Y-Y' Pseudo-Static Analysis

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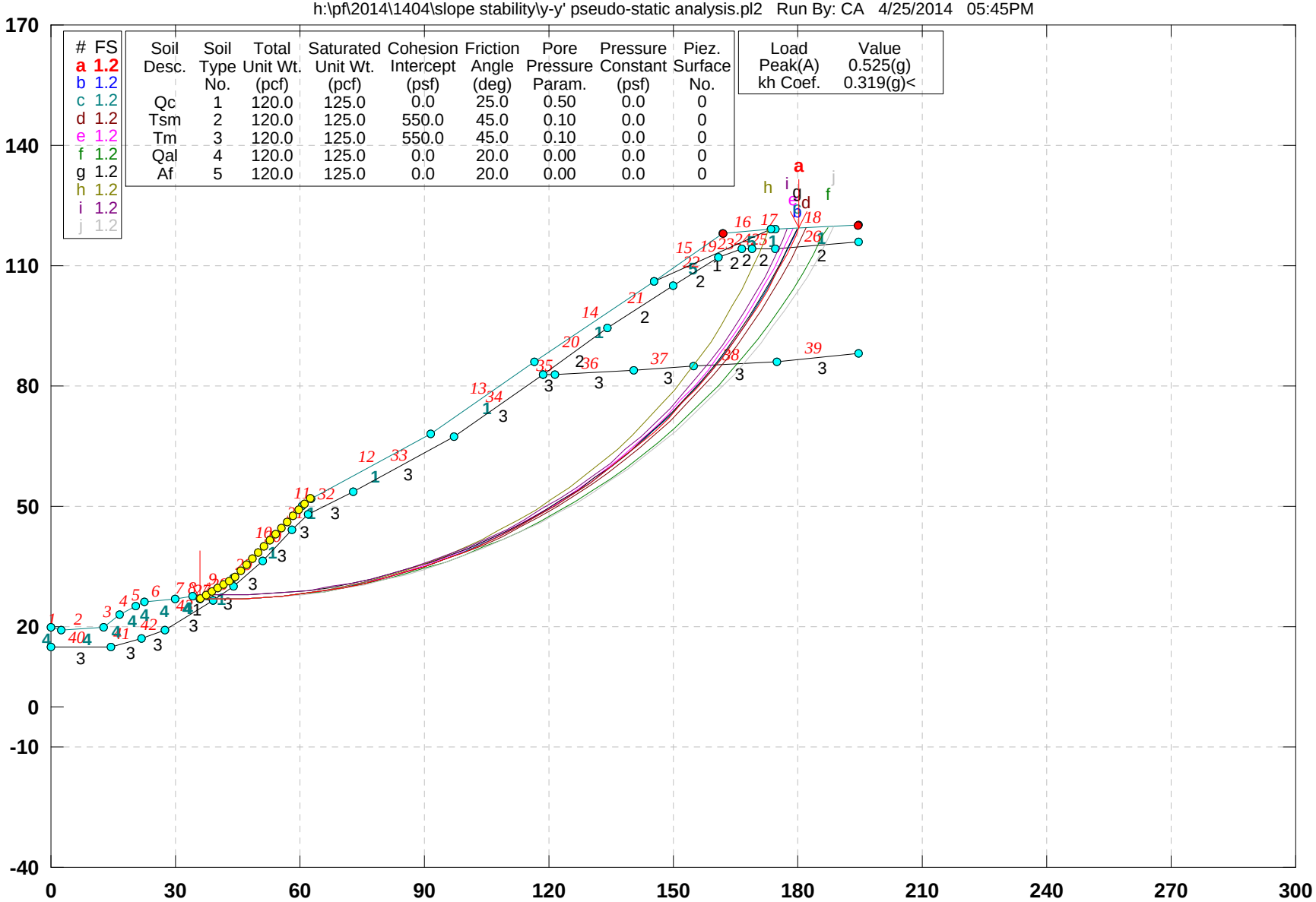
b	1.2
c	1.2
d	1.2
e	1.2
f	1.2
g	1.2
h	1.2
i	1.2
j	1.2

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qc	1	120.0	125.0	0.0	25.0	0.50	0.0	0
Tsm	2	120.0	125.0	550.0	45.0	0.10	0.0	0
Tm	3	120.0	125.0	550.0	45.0	0.10	0.0	0
Qal	4	120.0	125.0	0.0	20.0	0.00	0.0	0
Af	5	120.0	125.0	0.0	20.0	0.00	0.0	0

Load	Value
Peak(A)	0.525(g)
kh Coef.	0.319(g)<



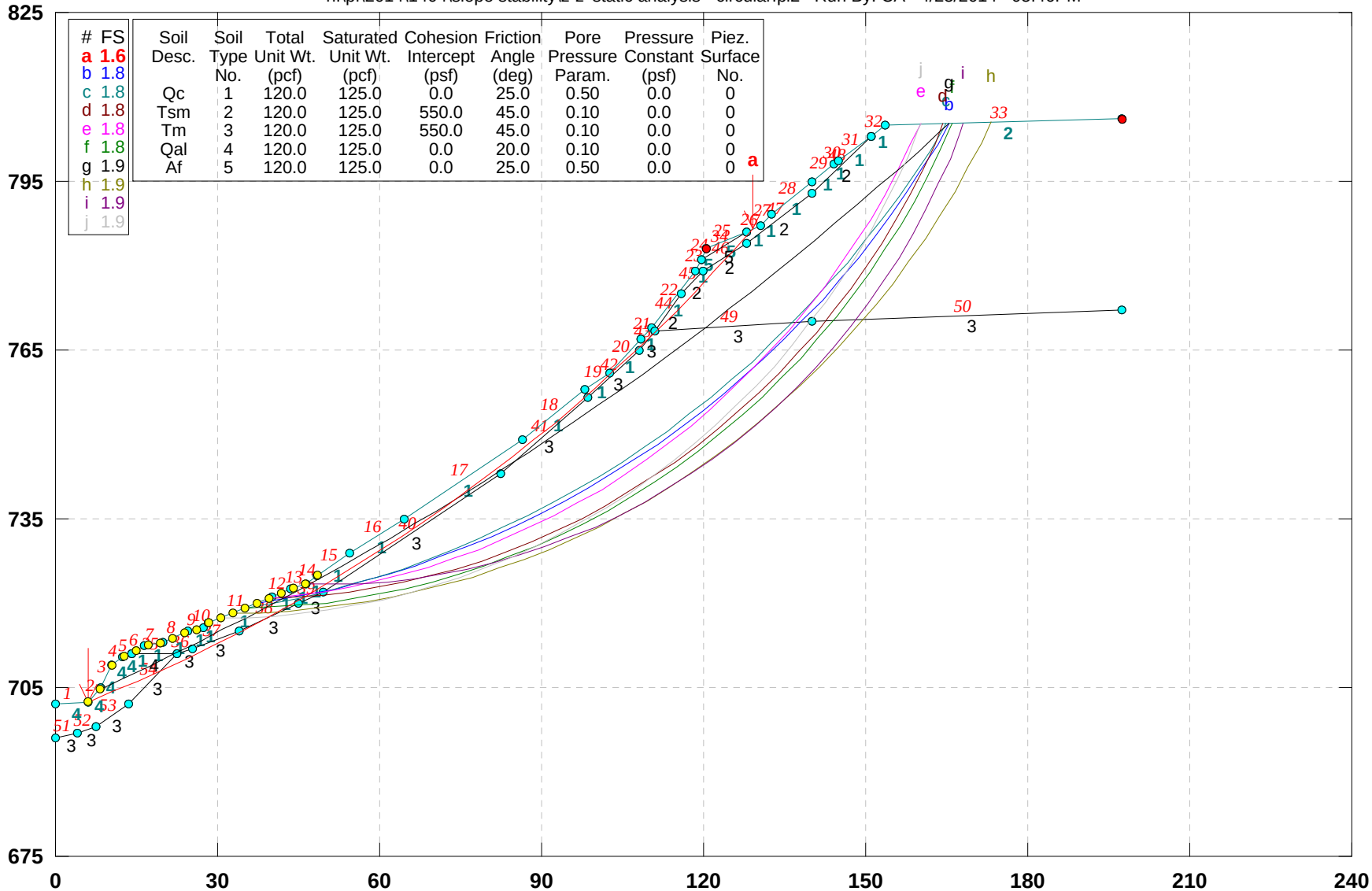
GSTABL7 v.2 FSmin=1.2

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

1440 Retreat Center Z-Z' Static Analysis

h:\pf\2014\1404\slope stability\z-z' static analysis - circular.pl2 Run By: CA 4/25/2014 05:46PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qc	1	120.0	125.0	0.0	25.0	0.50	0.0	0
Tsm	2	120.0	125.0	550.0	45.0	0.10	0.0	0
Tm	3	120.0	125.0	550.0	45.0	0.10	0.0	0
Qal	4	120.0	125.0	0.0	20.0	0.10	0.0	0
Af	5	120.0	125.0	0.0	25.0	0.50	0.0	0

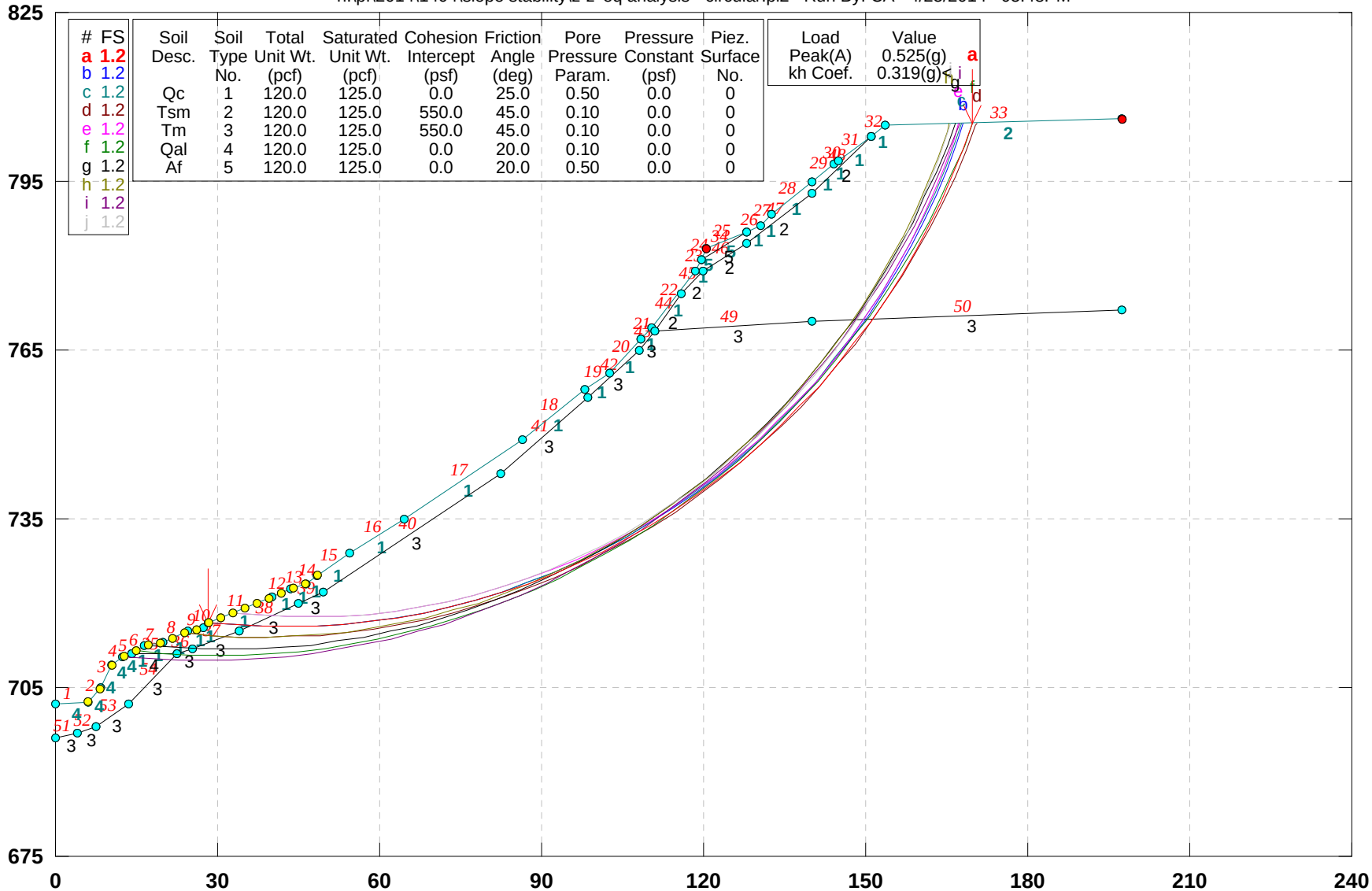
GSTABL7 v.2 FSmin=1.6

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

1440 Retreat Center Z-Z' Pseudo-Static Analysis

h:\pf\2014\1404\slope stability\z-z' eq analysis - circular.pl2 Run By: CA 4/25/2014 05:48PM



#	FS
a	1.2
b	1.2
c	1.2
d	1.2
e	1.2
f	1.2
g	1.2
h	1.2
i	1.2
j	1.2

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qc	1	120.0	125.0	0.0	25.0	0.50	0.0	0
Tsm	2	120.0	125.0	550.0	45.0	0.10	0.0	0
Tm	3	120.0	125.0	550.0	45.0	0.10	0.0	0
Qal	4	120.0	125.0	0.0	20.0	0.10	0.0	0
Af	5	120.0	125.0	0.0	20.0	0.50	0.0	0

Load	Value
Peak(A)	0.525(g)
kh Coef.	0.319(g)

GSTABL7 v.2 FSmin=1.2

Safety Factors Are Calculated By The Modified Bishop Method