APPENDIX B
Updated Geotechnical Investigation

GEOCON
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GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

UPDATE GEOTECHNICAL INVESTIGATION

PALOMAR COLLEGE SOUTH EDUCATION CENTER IMPROVEMENT PROJECT SAN DIEGO, CALIFORNIA

PREPARED FOR

PALOMAR COMMUNITY COLLEGE DISTRICT
SAN MARCOS, CALIFORNIA

OCTOBER 24, 2012
PROJECT NO. 06647-42-03

GEOCON
INCORPORATED
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. 06647-42-03
October 24, 2012

Palomar Community College District
1140 West Mission Road
San Marcos, California 92069-1487

Attention: Ms. Kelley Hudson-Macisaac

Subject: UPDATE GEOTECHNICAL INVESTIGATION
PALOMAR COLLEGE SOUTH EDUCATION
CENTER IMPROVEMENT PROJECT
SAN DIEGO, CALIFORNIA

Dear Ms. Hudson-Macisaac:

In accordance with your authorization of our proposal No. LG-12153 dated June 20, 2012 and revised July 2, 2012, we have performed an update geotechnical investigation on the subject education center located in San Diego, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of constructing the improvements as presently proposed.

In our opinion the improvements can be constructed as proposed provided the recommendations of this report are followed. The presence of shallow rock at or near the surface in the area of the proposed secondary access road will require special consideration during site development.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

GEOCON INCORPORATED

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Attention Mr. Young Min

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UPDATE GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of an update geotechnical investigation for the proposed improvements to the existing Palomar College South Education Center campus in the northern portion of San Diego, California (see Topographic Vicinity Map, Figure 1). The purpose of this investigation was to evaluate soil and geologic conditions on the property, and based on conditions encountered, provide recommendations pertaining to the geotechnical aspects of constructing improvements as presently proposed. This report has been prepared in accordance with the requirements of CDMG Note 48 for submittal to the Division of the State Architect (DSA).

The scope of the investigation consisted of a site reconnaissance, review of aerial photographs and pertinent geologic literature (see list of References), and a field investigation. The field investigation was conducted between September 24 and October 3, 2012, and consisted of drilling 9 exploratory borings, performing two permeability tests and conducting four seismic refraction lines at the approximate locations shown on the Geologic Map/Site Plan, Figure 2. The seismic refraction lines were performed by Southwest Geophysics to assess the apparent rippability of the metavolcanic rock exposed within the area planned for the secondary access road. Details of the field exploration as well as boring logs and seismic line profiles are presented in Appendix A and C, respectively.

Laboratory tests were performed on selected soil samples obtained during the field investigation to evaluate pertinent physical and chemical properties of the soils encountered. Details of the laboratory tests and a summary of test results are presented in Appendix B.

The recommendations presented herein are based on analysis of the data and observations obtained during this investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

2. SITE AND PROJECT DESCRIPTION

The subject campus area encompasses approximately 24 acres of land situated on the south side of Rancho Bernardo Road between Matinal Road and Olmead Way in the Rancho Bernardo area of San Diego, California. The latitude and longitude coordinates of the site is approximately: 33° 1' 24" N, 117 ° 5' 22" W.

The existing site configuration consists of an irregular shaped graded pad, elongated in an east-west orientation elevated above Rancho Bernardo Road. The campus consists of an existing four-story building, a detached four-level parking structure, surface parking lot, several retaining walls and other associated improvements. Elevations on the property range from a high of approximately 730 feet above Mean Sea Level (MSL) along the southwestern portion of the property to approximately 585 feet MSL within the lower drainage basin located along Rancho Bernardo Road.

Natural hillside slopes lie on the west, south, and east sides of the property. Fill slopes constructed as part of the previous grading lie on the north side, with a small fill slope constructed on the east side of the property. The northern fill slope is approximately 50 feet high and constructed at a 2 to 1 (horizontal to vertical) inclination. The eastern fill slope is approximately 10 feet high and also constructed at a 2 to 1 inclination.

The graded pad was sheet-graded to drain into an upper desilting basin that is centrally located at the top of the north facing fill slope. The elevations for the buildable portion of the pad vary from approximately 658 to 640 feet MSL. Access to the site is off of Rancho Bernardo Road via an ascending access road situated on the west side of the property.

It is our understanding that the proposed improvements to the site include adding an approximately 1,200-foot long secondary access road on the eastern side of the property, three 4-story stair wells to the existing building, an 8-foot high grassy knoll, and other surface and hardscape improvements as shown on Figure 2. Grading for the improvements will require cuts and fills on the order of approximately 50 feet and 8 feet, respectively. The largest cuts are associated with the proposed access road that will extend from the graded pad down the eastern slope to Rancho Bernardo Road.

The locations and descriptions of the site and proposed development are based on a site reconnaissance, a review of the referenced plans and our understanding of project development. If project details vary significantly from those described above, Geocon Incorporated should be contacted to determine the necessity for review and possible revision of this report.

3. PREVIOUS GRADING OPERATIONS AND GEOTECHNICAL STUDIES

The site was previously graded in two phases between October 1999 and June 2009, which resulted in the current graded condition. The approximate extent of fill placed during previous grading is identified as $Qpf_{(1\text{-}2)}$ on Figure 1. The grading operations were conducted in conjunction with the observation and compaction testing services of URS Greiner Woodward Clyde (Qpf_1) and Christian Wheeler Engineering (Qpf_2). Based on our review of as-graded reports documenting previous grading, the fill soil was compacted to at least 90 percent relative compaction. Compaction and laboratory testing as well as professional opinions pertaining to previous grading are summarized in the following geotechnical reports:

1. *Report of Earthwork Observations and Compaction Test Results, Compacted Fill, Storm Drain Trench Backfill, and Wall Backfill, Bernardo Industrial Park North, Lot 11, San Diego, California,* prepared by URS Greiner Woodward Clyde, dated November 2, 2000.

2. *Final Report of Grading Observations and Relative Compaction Testing, Bernardo Terrace Corporate Center, 11111 Rancho Bernardo Road, San Diego, California,* prepared by Christian Wheeler Engineering, dated September 16, 2009.

Several geotechnical studies were performed on the property between December 2000 and August 2005 by Geocon Incorporated. These reports include the following:

1. *Update to Addendum Geotechnical Report, Bernardo Terrace Corporate Center, San Diego, California*, dated August 18, 2005 (Project No. 06647-42-02).

2. *Addendum Geotechnical Report [for] Rancho Bernardo Industrial Park North, Lot 11 Expansion, City of San Diego Project Tract No. 1096, San Diego, California,* dated November 18, 2003 (Project No. 06647-42-02).

3. *Update Geotechnical Investigation, Bernardo Industrial Park North, Lot 11 Expansion and Existing Sheet Graded Pad, San Diego, California,* dated January 8, 2002 (Project No.06647-42-01).

4. *Geotechnical Investigation, Bernardo Industrial Park North, Lot 11 Expansion, San Diego, California,* dated May 31, 2001 (Project No. 06647-42-01).

4. SOIL AND GEOLOGIC CONDITIONS

Based on our recent and previous field investigations and review of the as-graded geotechnical reports performed by others, the materials underlying the site consist of three surfical soil types and two formational units. A Geologic Map/Site Plan and Geologic Cross Sections A-A' and B-B' showing these units are presented as Figures 2 and 3, respectively. A Regional Geologic Map has also been included as Figure 4.

4.1 Previously Placed Fill (Qpf_{1-2})

Previously placed fill soil associated with prior grading operations for the site has been identified as Qpf_1 and Qpf_2 on Figure 2. Qpf_1 represents the compacted fill soils observed and tested by URS Greiner Woodward Clyde. Fills observed and tested by Christian Wheeler Engineering are identified as Qpf_2. Based on exploratory borings performed for this study, compacted fill depths range from approximately 12 to 14 feet at the stairwell additions. Fill depths are expected to be in excess of 40 feet near the top of the northern slope.

4.2 Topsoil (Unmapped)

Topsoil was observed overlying the geologic formations on the ungraded hillsides adjacent to the property. Based on our observations and experience in the area, it is estimated that the topsoil materials vary from approximately one to three feet thick and consist predominantly of silty to slightly clayey sand.

4.3 Landslide Debris (Qls)

A remnant landslide deposit has been mapped in the southeastern portion of the property. A buttress fill designed and observed by Christian Wheeler Engineering was constructed during previous grading to stabilize the upper portion of the slide mass that was left in place. The location of landside is outside the area of existing and proposed improvements and should not impact proposed improvements investigated for this update geotechnical investigation.

4.4 Friars Formation (Tf)

Tertiary-age Friars Formation underlies the previously compacted fill soil and overlies the Santiago Peak Volcanics. This formation typically consists of dense sandstones, hard claystones and siltstones. Highly expansive claystone layers and/or concretionary zones are commonly found within this unit. Excavations that extend through the fill cap and into this unit may encounter highly expansive claystones that could excavate as oversize (greater than 12 inches) cemented chunks that will require special handling and placement during grading.

4.5 Santiago Peak Volcanics (Jsp)

The Jurassic-age Santiago Peak Volcanics underlies the previously placed fill soils and is exposed at grade at the northeastern portion of the property and within a limited area above the access road in the western part of site. This formation consists of weakly metamorphosed volcanic and sedimentary rocks that appear relatively dark-colored where exposed. The metavolcanic rock constitution ranges from rhyolite to basalt and commonly includes tuff, tuff-breccias, and andesites. Very fine-grained, silicified sandstones, slate, and other types of metasedimentary rocks can also be present.

The rippability characteristics of the Santiago Peak Volcanics are discussed in the *Rippability and Rock Considerations* section of this report. The Santiago Peak Volcanics generally exhibits adequate bearing and slope stability characteristics. Cut slopes excavated at an inclination of 2:1 (horizontal:vertical) should be stable to the proposed heights if free of adversely oriented joints or fractures. It should be anticipated that excavations within this unit will generate boulders and oversize materials (rocks greater than 12 inches in length) that will require special handling and placement procedures.

5. RIPPABILITY AND ROCK CONSIDERATIONS

To aid in evaluating the rippability characteristics of the metavolcanic rock in proposed cut areas for the proposed eastern access road, four seismic traverses were performed by Southwest Geophysics. The approximate location of the seismic traverse lines is shown on Figure 2. The report prepared by Southwest Geophysics is included in Appendix C. The data suggests the area of the access road has a weathered mantel ranging in depth of approximately 1 to 7 feet which should be possible to excavate with heavy effort using conventional grading equipment. However, where excavations extend below the upper weathered mantel, very difficult ripping and blasting to excavate should be expected. Blasting was required to excavate on-site rock during original grading based on our review of the as-graded report prepared by URS Greiner Woodward Clyde

Seismic refraction data can be used to evaluate rock rippability and estimate the depth at which excavation difficulty will occur. It should be recognized that rock rippability is a function of natural weathering processes, which can be variable and change vertically and horizontally over short distances depending on jointing, fracturing and/or mineralogic discontinuities within the bedrock. Perspective contractors should use their own judgment to evaluate the boundary between productive and non-productive ripping, and rippable and non-rippable rock.

Blasting techniques can be expected to generate oversized rock (rocks greater than 12-inches in dimension), which will likely require exporting due to the lack of available fill areas on site. In addition, the close proximity of the access road to existing buildings and other improvements (i.e. utility lines, sidewalks, streets) will also need to be considered to avoid damage.

6. GROUNDWATER

Groundwater was not encountered during our recent or previous field investigations. A regional groundwater table was not observed and is not expected to adversely impact development of the proposed site. However, it is not uncommon for seepage conditions to develop where none previously existed. Seepage conditions are dependent on seasonal precipitation, irrigation, land use, among other factors, and vary as a result. Proper surface drainage will be important to future performance of the project.

7. GEOLOGIC HAZARDS

7.1 Regional Faulting and Seismicity

Regional faulting was evaluated with respect to the site by reviewing published geologic maps and by performing a deterministic analysis to establish fault locations and estimated peak site accelerations. Figure 5 presents a Regional Fault Map. A Regional Seismicity Map showing earthquake epicenters is shown on Figure 6. The site is not located within a State of California Earthquake Fault Zone.

7.2 Faulting

A review of geologic literature indicates that there are no known active or potentially active faults at the site. An unnamed fault was discussed in the Woodward-Clyde's 1997 geotechnical investigation (see References) and has been mapped approximately within the area of the previous grading at the north end of the property. This fault was exposed within the Santiago Peak Volcanics and has not been documented to have displaced Quaternary or Holocene-aged deposits and therefore is considered to be "inactive" according to current criteria of the California Geological Survey (CGS). Since the fault is "inactive" it should not pose a seismic risk to the proposed project development. In addition, the alignment of this fault does not cross the existing building pad.

7.3 Seismic Hazard Analysis

According to the computer program *EZ-FRISK (Version 7.62),* 7 known active faults are located within a search radius of 50 miles from the property. We used the 2008 USGS fault database that provides several models and combinations of fault data to evaluate the fault information. The nearest active faults are the Newport-Inglewood and Rose Canyon Fault Zones, located approximately 13 miles west of the site and are the dominant sources of potential ground motion. Earthquakes that might occur on the Newport-Inglewood and Rose Canyon Fault Zones or other faults within the southern California and northern Baja California area are potential generators of significant ground motion at the site. The estimated maximum earthquake magnitude and peak ground acceleration for the Newport-Inglewood Fault are 7.5 and 0.21g, respectively. Table 7.3.1 lists the estimated maximum earthquake magnitude and peak ground acceleration for the most dominant faults in relationship to the site location. We calculated peak ground acceleration (PGA) using Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA acceleration-attenuation relationships.

TABLE 7.3.1
DETERMINISTIC SEISMIC SITE PARAMETERS

Fault Name	Distance from Site (miles)	Maximum Earthquake Magnitude (Mw)	Peak Ground Acceleration		
			Boore-Atkinson 2008 (g)	Campbell-Bozorgnia 2008 (g)	Chiou-Youngs 2008 (g)
Newport-Inglewood	13	7.5	0.21	0.17	0.21
Rose Canyon	13	6.9	0.17	0.15	0.15
Elsinore	22	7.85	0.17	0.13	0.16
Coronado Bank	27	7.4	0.13	0.09	0.10
Palos Verdes Connected	27	7.7	0.14	0.10	0.12
Earthquake Valley	31	6.8	0.09	0.07	0.06
San Jacinto	44	7.88	0.10	0.08	0.09

We performed a site-specific probabilistic seismic hazard analysis using the computer program *EZ-FRISK*. Geologic parameters not addressed in the deterministic analysis are included in this analysis. The program operates under the assumption that the occurrence rate of earthquakes on each mappable Quaternary fault is proportional to the faults slip rate. The program accounts for earthquake magnitude as a function of fault rupture length, and site acceleration estimates are made using the earthquake magnitude and distance from the site to the rupture zone. The program also accounts for uncertainty in each of following: (1) earthquake magnitude, (2) rupture length for a given magnitude, (3) location of the rupture zone, (4) maximum possible magnitude of a given earthquake, and (5) acceleration at the site from a given earthquake along each fault. By calculating the expected accelerations from considered earthquake sources, the program calculates the total average annual expected number of occurrences of site acceleration greater than a specified value. We utilized acceleration-attenuation relationships suggested by Boore-Atkinson (2008) NGA USGS 2008, Campbell-Bozorgnia (2008) NGA USGS 2008, and Chiou-Youngs (2008) NGA in the analysis. Table 7.3.2 presents the site-specific probabilistic seismic hazard parameters including acceleration-attenuation relationships and the probability of exceedence.

TABLE 7.3.2
PROBABILISTIC SEISMIC HAZARD PARAMETERS

Probability of Exceedence	Peak Ground Acceleration		
	Boore-Atkinson, 2008 (g)	Campbell-Bozorgnia, 2008 (g)	Chiou-Youngs, 2008 (g)
2% in a 50 Year Period	0.37	0.35	0.39
5% in a 50 Year Period	0.28	0.26	0.28
10% in a 50 Year Period	0.22	0.20	0.21

The California Geologic Survey (CGS) has a program that calculates the ground motion for a 10 percent of probability of exceedence in a 50-year period based on an average of several attenuation relationships. Table 7.3.3 presents the calculated results from the Probabilistic Seismic Hazards Mapping Ground Motion Page from the CGS website.

TABLE 7.3.3
PROBABILISTIC SITE PARAMETERS FOR SELECTED FAULTS
CALIFORNIA GEOLOGIC SURVEY

Calculated Acceleration (g) Firm Rock	Calculated Acceleration (g) Soft Rock	Calculated Acceleration (g) Alluvium
0.24	0.26	0.30

7.4 Site Specific Seismic Hazard Analysis

Based on our review of as-graded geotechnical reports, evidence of potentially active or active faulting was not observed during mass grading. Additionally, faulting was not observed during previous geotechnical investigations performed by Geocon Incorporated. In accordance with ASCE 7-05, we performed ground motion hazard analyses utilizing the computer program *EZFRISK* (version 7.62) in conjunction with data from the US Geological Survey National Seismic Hazards Mapping Program (NSHMP ver. 5.10). The Maximum Considered Earthquake ground motion (MCE) having a 2 percent chance of exceedence in 50 years, with a statistical return period of 2,500 years was used in the probabilistic analysis. We calculated peak ground accelerations (PGA) with the maximum rotated components using Boore-Atkinson (2008), Campbell-Bozorgnia (2008), and Chiou-Youngs (2007) acceleration-attenuation relationships in conjunction with the 2008 USGS National Seismic Hazard Maps fault database.

We performed a deterministic analysis by evaluating the ground motions generated by maximum earthquakes on each of the active faults within a 50 mile radius of the site, modeling the soil underlying the site as a Site Class C as defined by Table 1613.5.2 of the 2010 CBC. The deterministic analysis used the 84th percentile of the maximum rotated component using the methodology described in the 2009 NEHRP Recommended Seismic Provisions. The effect of near source directivity (Somerville 1997 and Abrahamson 2000) was also considered in the analysis based on the site proximity to the Newport-Inglewood Fault. Using this methodology, the maximum earthquake resulting in the highest peak horizontal accelerations at the site would be a magnitude 7.5 event occurring on the Newport-Inglewood Fault.

The 84th percentile of the maximum ground motion acceleration was compared to the deterministic lower limit acceleration, the maximum of which was then compared to the results of the probabilistic analysis, which used the maximum rotated component of ground motion.

The lesser of the probabilistic and maximum deterministic ground motions is termed as the Site Specific MCE, of which ⅔ of this MCE is considered the Site Specific Design Spectral Response Acceleration (provided the results are not less than 80 percent of the General Response Spectrum generated by the NSHMP). Graphical representations of the analyses, including probabilistic and deterministic spectrum are presented on Figures 7, 8, and 9. The final site-specific design response spectral accelerations are presented graphically on Figure 8 and in tabular form on Figure 9.

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the site. Seismic design of the structures should be evaluated in accordance with the currently adopted California Building Code (CBC) guidelines.

The site could be subjected to moderate to severe ground shaking in the event of a major earthquake on the faults referenced herein or other faults in Southern California.

7.5 Liquefaction

Liquefaction typically occurs when a site is located in a zone with seismic activity, onsite soils are cohesionless or silt/clay with low plasticity, groundwater is encountered within 50 feet of the surface, and soil densities are less than about 70 percent of the maximum dry densities. If the four previous criteria are met, a seismic event could result in a rapid-pore water pressure increase from the earthquake-generated ground accelerations. Due to the lack of a near-surface groundwater table and the dense nature of the underlying compacted fill and formational rock materials, the potential for liquefaction at the site is considered very low.

7.6 Tsunamis and Seiches

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The first order driving force for locally generated tsunamis offshore in southern California is expected to be tectonic deformation from large earthquakes (Legg, *et al.*, 2002). Wave heights and run-up elevations from tsunamis along the San Diego Coast have historically fallen within the normal range of the tides. The site is located approximately 11½ miles from the Pacific Ocean at an elevation of approximately 655 feet above Mean Sea Level. Therefore, the risk of tsunamis affecting the site is considered very low.

Seiches are caused by the movement of an inland body of water due to seismic forces. The site is located approximately 2 miles south of Lake Hodges and is approximately 340 feet above the lake water level and is not downstream of the drainage path. Therefore, the potential for seiches to affect the site is considered very low.

7.7 Landslides

Based on our review of the referenced geologic materials and our previous investigations on the property, landslide deposits have been mapped on the property. However, the landslides have been mitigated using conventional grading practices (i.e. buttresses, stability fills, complete removal). Landslides left in-place on the property have been stabilized with a buttress fill and are located outside the area of the proposed new improvements. Landslide hazard to proposed improvements is considered low.

7.8 Subsidence and Seismic Settlement

Based on the subsurface conditions encountered during our field investigation, we do not expect the site would be subject to hazards from ground subsidence or seismic settlement.

7.9 Flooding

Based on existing topography, the site is not located within an alluvial drainage or floodplain; therefore, the potential for flooding is negligible as long as proper surface drainage is maintained.

7.10 Expansive Soil

Based on the recent and previous laboratory testing performed at the site, the upper portion of compacted fill placed within the existing building pads, flatwork and parking lot areas exhibits a "low" to "medium" expansion potential (Expansion Index of 90 or less). The formational materials and other compacted fill materials present on site have exhibited varying expansion potential ranging from "low" to "high" (Expansion Index between 20 and 130).

7.11 Geologic Hazard Category

The 2008 City of San Diego Seismic Safety Study Map Sheet 47 categorizes the site as Geologic Hazard Categories 12, 23, 24, 27, 52 and 53. Under *Fault Zones,* Category 12 is defined as *Potentially active, inactive, presumed inactive or activity unknown*; under *Slide-Prone Formations*, Category 23 is defined as *Friars: neutral or favorable geologic structure;* Category 24 is defined as *Friars: unfavorable geologic structure;* Category 27 is defined as *Otay, Sweetwater and others;* under *Other Terrain*, Category 52 is defined as *Other level areas, gently sloping to steep terrain, favorable geologic structure, low risk;* and Category 53 is defined as *Level or sloping terrain, unfavorable geologic structure, low to moderate.*

The inactive fault (Hazard Category 12), mapped in the northern portion of the site has presumably only displaced the Santiago Peak Volcanics and the trace and trend of the fault does not cross any structure or proposed structure. Categories 23 and 24 within the Slide Prone Formations (Friars Formation) has been addressed and mitigated during the mass grading operations. Category 27 represents the Stadium Conglomerate which is exposed topographically above any proposed site improvements. Category 52 is located under the previously placed fill soils and covers the area underlain by the Santiago Peak Volcanics exposed which is a "low risk" formation. Category 53 is located topographically above any proposed site improvements. In our opinion, with the implementation of the recommendations provided in this report, the site should have an overall low geologic risk.

8. SIOPE STABILITY EVALUATION

Cut slopes in rock materials (Santiago Peak Volcanics) do not lend themselves to conventional slope stability analyses. Based on experience with similar rock conditions, 2:1 cut slopes to the planned heights of up to 70 feet should possess a factor of safety of at least 1.5 with respect to slope instability, if free of adversely oriented joints or fractures. All cut slope excavations should be observed during grading by an engineering geologist to check that soil and geologic conditions do not differ significantly from those anticipated. In the event that adverse conditions are observed, stabilization recommendations can be provided. However, in order to satisfy CDMG Note 48 requirements, slope stability analyses were performed considering a 2:1 (horizontal:vertical), 70-high cut slope founded in metavolcanic rock. The analysis utilized the computer software program *GeoStudio 2007* to provide appropriate design recommendations to achieve a factor of safety of at least 1.5 against deep-seated failure. A summary of the static slope stability analysis performed is shown on Table 8.1.

TABLE 8.1
STATIC SLOPE STABILITY SUMMARY

Section	Figure Number	Condition Analyzed	Factor Of Safety
A-A'	D-1	Circular Failure	3.8

In accordance with CDMG Note 48 and using Special Publication 117 guidelines, seismic slope stability analyses were performed in accordance with *Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Landslide Hazards in California*, prepared by the Southern California Earthquake Center (SCEC), dated June 2002.

The seismic slope stability analysis was performed using the unweighted acceleration of 0.22g, corresponding to a 10 percent probability of exceedence in 50 years. In addition, a deaggregation analysis was performed on the 0.22g value for the site using the 2008 USGS interactive deaggregations website. A modal magnitude and modal distance of 7.59 and 36.1 kilometers, respectively, was used in the analysis and a plot of the hazard contribution is shown in Appendix D, Figure D-3.

Using the parameters discussed herein, an equivalent site acceleration, k_{EQ}, of 0.15g was calculated to perform the seismic slope stability analysis. The screening analysis was performed using an acceleration of 0.15g resulting in a factor of safety above 1.0. Table 8.2 presents a summary of the seismic slope stability screening evaluation. A slope is considered acceptable by the screening analysis if the calculated factor of safety is greater than 1.0 using k_{EQ}; therefore, the most critical failure surface depicted for the cut slope analyzed passed the screening analysis for the seismic slope stability.

TABLE 8.2
SEISMIC SLOPE STABILITY SCREENING EVALUATION (KEQ = 0.15G)

Section	Figure Number	Condition Analyzed	Factor Of Safety	Pass/Fail
A-A’	D-2	Circular Failure	2.3	Pass

The site geology, results of the subsurface investigation, observation and testing during site mass grading, laboratory testing performed during mass grading and for this study, and proposed topography were considered in the stability analyses.

Laboratory tests were performed on relatively undisturbed and bulk samples of the prevailing soil and geologic units and the results are presented in Appendix B. Table 8.3 presents the soil strength parameters that were utilized in the slope stability analyses.

TABLE 8.3
SOIL STRENGTH PARAMETERS

Soil Condition	Angle of Internal Friction ϕ (degrees)	Cohesion c (psf)
Compacted Fill	30	200
Santiago Peak Volcanics	40	1,000

The output files and calculated factor of safety for the 70-foot high, 2:1 cut slope is presented in Appendix D.

9. CONCLUSIONS AND RECOMMENDATIONS

9.1 General

9.1.1 It is our opinion that no soil or geologic conditions were encountered during the investigation that would preclude the construction of the proposed improvements as described herein provided the recommendations of this report are followed.

9.1.2 Based on our review of published geologic maps and geologic hazards studies, the subject site should not be affected by geologic hazards including fault rupture, landslides, liquefaction, tsunamis, seiches, and ground subsidence. The site is subject to moderate ground shaking from earthquakes within the Southern California or Northern Baja California region, however, the seismic risk is comparable to that of the surrounding area.

9.1.3 Excavations during grading for the secondary access road and any other improvements that extend into the Santiago Peak Volcanics should anticipate excavation difficulties and blasting for deeper excavations. The potential for these conditions should be taken into consideration when determining the type of equipment to utilize for future excavation operations.

9.1.4 Due to the absence of large areas of available fill volume, it is likely that the oversize material generated from excavations in the rock will need to be exported. Consideration for increasing the cut slope ratio from 2:1 (horizontal:vertical) to 1.5:1 to reduce the rock volume and oversize material can be contemplated.

9.1.5 Where rock may be present at or near the existing ground surface, consideration should be given to undercutting areas such as utility corridors and future foundations zones to facilitate trenching operations and to provide suitable backfill material.

9.1.6 The proposed improvements can be supported on conventional shallow foundations founded in properly compacted fill.

9.2 Excavation and Soil Characteristics

9.2.1 Soils encountered during our investigation and mass grading are considered to be "expansive" (expansion index [EI] of greater than 20) as defined by 2010 California Building Code (CBC) Section 1803.5.3. Table 9.2 presents soil classifications based on the expansion index. Based on our review of the referenced as-graded reports and laboratory testing, the soil placed during grading exhibits a "medium" expansion potential (Expansion

Index of 90 or less). The underlying Friars Formation has exhibited a "very low" to "high" expansion potential (expansion index less than 130).

TABLE 9.2
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Soil Classification
0 – 20	Very Low
21 – 50	Low
51 – 90	Medium
91 – 130	High
Greater Than 130	Very High

9.2.2 We performed laboratory tests on samples of the site materials to evaluate the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate content testing are presented in Appendix B and indicate that the on-site materials at the location tested possess "negligible" sulfate exposure to concrete structures as defined by 2010 CBC Section 1904.3 and ACI 318-08 Section 4.2 and 4.3. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

9.2.3 We performed water-soluble chloride ion content, potential of hydrogen (pH), and resistivity laboratory tests on soil samples to check the corrosion potential to metal structures and improvements in contact with soil. A soil is considered corrosive if the chloride concentration is 500 part per million (ppm) or greater, sulfate concentration is 2,000 ppm or greater, the pH is 5.5 or less, and the minimum resistivity is less than 2,000 ohm-cm according to Caltrans *Corrosion Guidelines,* dated September 2003 for buried metals. The chloride ion content, pH, and resistivity laboratory test results are presented in Appendix B.

9.2.4 Geocon Incorporated does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements that could be susceptible to corrosion are planned.

9.2.5 The compacted fill soils can be excavated with moderate effort using conventional heavy-duty grading equipment. Excavations during grading for the secondary access road and any other improvements that extend into the Santiago Peak Volcanics should anticipate excavation difficulties and blasting for excavations that extend beneath the weathered mantel. Oversize material (likely generated from the formational units on site) may have been placed during previous grading operations.

9.2.6 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations in order to maintain safety and maintain the stability of adjacent existing improvements.

9.3 Seismic Design Criteria

9.3.1 Table 9.3 summarizes site-specific design criteria obtained from the 2010 California Building Code (CBC; Based on the 2009 International Building Code [IBC]), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The values were derived using the computer program *Seismic Hazard Curves and Uniform Hazard Response Spectra*, provided by the USGS. The short spectral response uses a period of 0.2 second. The site can be designed using Site Class C according to 2010 CBC Section 1613A.5.2 and 1613A.5.5.

TABLE 9.3
CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2010 CBC Reference
Site Class	C	Table 1613.5.2
Spectral Response – Class B (short), S_S	1.023g	Figure 1613.5(3)
Spectral Response – Class B (1 sec), S_1	0.372g	Figure 1613.5(4)
Site Coefficient, F_a	1.000	Table 1613.5.3(1)
Site Coefficient, F_v	1.428	Table 1613.5.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.023g	Section 1613.5.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.531g	Section 1613.5.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.682g	Section 1613.5.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.354g	Section 1613.5.4 (Eqn 16-39)

9.3.2 Conformance to the criteria in Table 9.3 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

9.4 Site Modified Seismic Design Criteria

9.4.1 We performed ground motion hazard analyses utilizing the computer program *EZ-FRISK* (Version 7.62) in conjunction with data from the US Geological Survey National Seismic Hazards Mapping Program (NSHMP ver. 5.1.0). We used the Maximum Considered Earthquake ground motion (MCE) having a 2 percent chance of exceedence in 50 years, with a statistical return period of 2,500 years in the probabilistic analysis. We used attenuation relationships of Boore-Atkinson (2008) USGS 2008 MRC, Campbell-Bozorgnia (2008) USGS 2008 MRC, and Chiou-Youngs (2007) USGS 2008 MRC in the analyses.

9.4.2 We performed a deterministic analysis, assumed to attenuate to the site per the same NGA's as the probabilistic method, by evaluating the ground motions generated by maximum earthquakes on each of the active faults within a 50 mile radius of the site, modeling the soil underlying the site as a Site Class C as defined by Table 1613.5.2 of the 2010 CBC. The deterministic analysis used the 84^{th} percentile of the maximum rotated component using the methodology described in the 2009 NEHRP Recommended Seismic Provisions. Using this methodology, the maximum earthquake resulting in the highest peak horizontal accelerations at the site would be a magnitude 7.5 event occurring on the Newport-Inglewood/Rose Canyon Fault.

9.4.3 The 84th percentile of the maximum ground motion acceleration was compared to the deterministic lower limit acceleration, the maximum of which was then compared to the results of the probabilistic analysis, which used the maximum rotated component of ground motion.

9.4.4 The lesser of the probabilistic and maximum deterministic ground motions is termed as the Site Specific MCE, of which ⅔ of this MCE is considered the Site Specific Design Spectral Response Acceleration (provided the results are not less than 80 percent of the General Response Spectrum generated by the NSHMP). Graphical representations of the analyses, including probabilistic and deterministic spectrum are presented on Figures 7 and 8. The final site-specific design response spectral acceleration is presented graphically on Figure 8. The results of the analysis are presented in tabular form on Figure 9.

9.4.5 The Modified Seismic Design Parameters using the information from the site-specific seismic analyses is presented on Table 9.4.

TABLE 9.4
MODIFIED SEISMIC DESIGN PARAMETERS

Parameter	Modified Seismic Value
Maximum Considered Earthquake Spectral Response Acceleration (short), S_{MS}	1.055g
Maximum Considered Earthquake Spectral Response Acceleration – (1 sec), S_{M1}	0.576g
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.703g
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.384g

9.4.6 Conformance to the criteria in Table 9.4 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The intent of the code is "Life Safety," not to completely prevent damage to the structure, since such design may be economically prohibitive.

9.5 Grading

9.5.1 Grading should be performed in accordance with the Grading Ordinance of the City of San Diego and the *Recommended Grading Specifications* contained in Appendix E. Where the recommendations of Appendix E conflict with this section of the report, the recommendations of this section take precedence.

9.5.2 Prior to commencing grading, a pre-construction conference should be held at the site with the project architect, grading contractor, civil engineer, geotechnical engineer, and inspection officials in attendance. Special soil handling requirements can be discussed at that time.

9.5.3 Proposed structural improvement areas and areas to receive fill should be cleared of any deleterious material (i.e. vegetation, asphalt, concrete and debris), if any, prior to commencing grading. Any organic or unsuitable material generated should be exported from the site.

9.5.4 In general, soils generated during on-site excavations are suitable for reuse as fill if free of vegetation, debris, and deleterious matter. However, we expect soils within landscape areas to be wet and/or saturated and will require drying and/or mixing with drier soils prior to reuse as fill.

9.5.5 Within the proposed new stairwell areas, saturated and/or yielding soil should be removed and replaced with compacted fill. We expect the upper 2 to 3 feet of soil within existing landscaped areas will require removal and replacement. The actual depth of removal required will be determined during grading based on soil conditions exposed. The remedial excavation should extend to at least 5 feet beyond the edge of the new stairwell foundation where practical. Where competent previously placed fill is encountered, no remedial removals are required.

9.5.6 In areas outside of landscaping, prior to placing new fill, we recommend the upper 12 inches of compacted fill be scarified, moisture conditioned as necessary, and compacted to at least 90 percent relative compaction at or above optimum moisture content.

9.5.7 Care should be taken not to undermine or damage existing building foundations during remedial grading. Remedial excavations adjacent to existing building footings should be sloped at a 1:1 (horizontal:vertical) from the building footing.

9.5.8 Prior to placing fill, the base of excavations and surface of previously placed fill should be scarified at least 12 inches, moisture conditioned as necessary, and compacted. Fill soils may then be placed and compacted in layers to the design finish grade elevations. The layers should be no thicker than will allow for adequate bonding and compaction. All fill, including scarified ground surfaces and backfill, should be compacted to at least 90 percent of maximum dry density at or slightly above optimum moisture content, as determined by ASTM D 1557. Overly wet materials will require drying and/or mixing with drier soils to facilitate proper compaction. The upper 12 inches of subgrade in pavement areas should be compacted to 95 percent relative compaction.

9.5.9 The upper 3 feet of soil placed within the stairwell pads should have an Expansion Index (EI) less than 90.

9.5.10 Import fill, if any, should consist of granular material with a "very low" expansion potential (EI of 50 or less), generally free of deleterious material and rocks larger than 6 inches, and should be compacted as recommended herein. Geocon Incorporated should be notified of the import source and should perform laboratory testing on import soil samples prior to its arrival at the site to evaluate its suitability as fill material.

9.6 Foundation Recommendations

9.6.1 The proposed exterior stairwells can be supported on a shallow foundation system bearing on compacted fill. The foundation recommendations herein are based on the assumption that the prevailing soil within 3 feet of finish grade will possess a "medium" expansion potential (EI of 90 or less).

9.6.2 Foundations for the proposed stairwell should consist of continuous strip footings and/or isolated spread footings. Conventional continuous footings should have a minimum embedment depth of 24 inches below lowest adjacent pad grade and should be at least 12 inches wide. Reinforcement should consist of four, No. 5 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Isolated spread footings should be at least 2 feet square and founded at least 24 inches below lowest adjacent pad grade. The project structural engineer should design reinforcement for spread footings.

9.6.3 Footings proportioned as recommended above may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (dead plus live loads). The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

9.6.4 Footings should not be located within 7 feet of the tops of slopes. Footings that must be located within this zone should be extended in depth such that the outer bottom edge of the footing is at least 7 feet horizontally from the face of the finished slope.

9.6.5 The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, is not recommended. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams.

9.6.6 We estimate the total and differential settlement due to footing loads conforming to the recommended allowable soil bearing pressures is approximately ¾-inch and ½-inch, respectively.

9.6.7 Special subgrade presaturation (i.e., flooding to saturate soils to foundation depths to mitigate highly expansive soils) is not deemed necessary prior to placement of concrete. However, the slab and foundation subgrade should be moisturized as necessary to maintain a moist condition as would be expected in any concrete placement.

9.6.8 Foundation excavations should be observed by the geotechnical engineer (a representative of Geocon Incorporated) prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are consistent with those expected and have been extended to appropriate bearing strata. If unexpected soil conditions are encountered, foundation modifications may be required.

9.7 Concrete Slabs-on-Grade

9.7.1 Conventional concrete slabs-on-grade should be at least 5 inches thick and reinforced with No. 3 steel reinforcing bars spaced 18 inches on center in both horizontal directions at the slab midpoint. The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting planned loading. Thicker concrete slabs may be required for heavier loads.

9.7.2 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). In addition, the membrane should be installed in accordance with manufacturer's recommendations and ASTM requirements and installed in a manner that prevents puncture. The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

9.7.3 The project foundation engineer or architect should determine the bedding sand thickness Placement of 3 to 4 inches of sand or base material is common practice for this area. However, Geocon Incorporated should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. If aggregate base material is used, the vapor retarder material will need be able to resist puncture from the angular gravel. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

9.7.4 The foundation and slab-on-grade dimensions and minimum reinforcement recommendations are based upon soil conditions only and are not intended to be used in lieu of those required for structural purposes.

9.7.5 Exterior concrete slabs not subjected to vehicle loads should be at least 4 inches thick and reinforced with 6 x 6 - 6/6 welded wire mesh or No. 3 steel reinforcing bars spaced 18 inches on center in both directions. The steel should be placed in the middle of the slab. Proper steel positioning is critical to future performance of the slabs. The contractor should take extra care to provide proper steel placement. Prior to construction of concrete slabs, the subgrade should be moisture conditioned to optimum moisture content or above optimum moisture content and compacted to a dry density at least 90 percent of the laboratory maximum dry density.

9.7.6 Concrete slabs should be provided with adequate construction joints and/or expansion joints to control unsightly shrinkage cracking. The spacing should be determined by the project structural engineer based upon the intended slab usage, type and extent of floor-covering materials, thickness, and reinforcement. The structural engineer should take into consideration criteria of the American Concrete Institute (ACI) when establishing crack-control spacing patterns.

9.7.7 Where buildings or other improvements are planned near the top of a slope steeper than 3:1 (horizontal:vertical), special foundations and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- For fill slopes less than 20 feet high or cut slopes regardless of height, building footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.
- When located next to a descending 3:1 (horizontal:vertical) fill slope or steeper, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. An acceptable alternative to deepening the footings would be the use of a post-tensioned slab and foundation system or increased footing and slab reinforcement. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.
- If swimming pools are planned, Geocon Incorporated should be contacted for a review of specific site conditions.
- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height, and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon Incorporated should be contacted for a review of specific site conditions.
- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures that would permit some lateral soil

movement without causing extensive distress. Geocon Incorporated should be consulted for specific recommendations.

9.7.8 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soils (if present) and differential settlement of fill soil. However, even with the incorporation of the recommendations presented herein, foundations and slabs-on-grade placed on such conditions may still exhibit cracking. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and the placement of crack-control joints at proper locations, particularly where re-entrant slab corners occur.

9.7.9 Geocon Incorporated should be consulted to provide additional design parameters as required by the structural engineer.

9.8 Preliminary Pavement Recommendations

9.8.1 Preliminary pavement recommendations for the secondary access road are provided below. The final pavement design section should be determined subsequent to grading based on the R-Value of the subgrade soil encountered at final subgrade elevation. For preliminary design, we have assumed an R-Value of 17. Preliminary flexible pavement sections are presented in Table 9.8.1 for varying Traffic Indices (TIs). The project civil or traffic engineer should determine the appropriate traffic index based on planned traffic loads and volumes.

TABLE 9.8.1
PRELIMINARY FLEXIBLE PAVEMENT SECTION

Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
5	17	3	8
5.5	17	3	9.5
6	17	4	9
6.5	17	4	11
7	17	4	12.5
7.5	17	5	12.5
8	17	5	14

9.8.2 Class 2 aggregate base should conform to Section 26-1.02B of the *Standard Specifications for the State of California Department of Transportation (Caltrans)* and should be compacted to a dry density of at least 95 percent of the maximum dry density at near optimum moisture content as determined by ASTM D 1557. Asphalt concrete should conform to Section 203-6 of the *Standard Specifications for Public Works Construction (Green Book)*. Asphalt concrete should be compacted to at least 95 percent of the laboratory Hveem density as determined by ASTM D 2726.

9.8.3 Prior to placing base, pavement subgrade soils should be scarified, moisture conditioned as necessary, and recompacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content as determined by ASTM D1557. The depth of processing should be at least 12 inches.

9.8.4 The performance of pavement is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, the perimeter curb should extend at least 6 inches below the level of the base materials.

9.9 Conventional Retaining Walls

9.9.1 Retaining walls that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid having a density of 35 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal:vertical), an active soil pressure of 50 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall are sandy soils with suitable shear characteristics and an EI of 50 or less. Select import material may be required to conform to this criterion. Expansive soils should not be used for wall backfill. Laboratory tests should be performed on soils to be used as wall backfill to assess their suitability for use.

9.9.2 Soil contemplated for use as retaining wall backfill, including import materials, should be identified in the field prior to backfill. At that time Geocon Incorporated should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, on-site soil to be used as backfill may or may not meet the values for standard wall designs. Geocon Incorporated should be consulted to assess the suitability of the on-site soil for use as wall backfill if standard wall designs will be used.

9.9.3 Where walls are restrained from movement at the top and are 8 feet or less in height, an additional uniform pressure of 7H psf should be added to the above active soil pressure. Where the wall height exceeds 8 feet, the additional uniform pressure should be increased to 14H psf. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 foot of fill soil should be added (unit weight 130 pcf) for surcharge loading.

9.9.4 Retaining walls founded on compacted fill can be designed for an allowable bearing pressure of 2,000 psf for a 12-inch wide and 12-inch deep footing. The allowable soil bearing pressure can be increased by 300 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing of 4,000 psf. These values can be increased by 1/3 for seismic loading. Settlement of walls imposing the maximum allowable bearing pressure is not expected to exceed 1 inch.

9.9.5 The structural engineer should determine the seismic design category for the project and if retaining walls need to incorporate seismic lateral loads. A seismic load of 16H should be used for design. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at top of the wall. We used a horizontal peak ground acceleration of 0.28g calculated using S_{DS}/2.5 USGS and applying a pseudo-static coefficient of 0.33.

9.9.6 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140 (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. Figure 10 presents a typical retaining wall drain detail. If conditions different than those described are expected or if specific drainage details are desired, Geocon Incorporated should be contacted for additional recommendations.

9.9.7 The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

9.9.8 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. In the event that walls higher than 10 feet or other types of walls (such as crib-type walls) are planned, Geocon Incorporated should be consulted for additional recommendations.

9.9.9 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer

9.10 Lateral Passive Resistance

9.10.1 Resistance to lateral loads will be provided by friction along the base of the wall foundation or by passive earth pressure against the side of the footing. Allowable coefficients of friction of 0.3 are recommended for footings in compacted fill. Passive earth pressure may be taken as 150 pcf for walls founded on a 2:1 slope, and 300 pcf for horizontal ground in front of the wall. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

9.11 Bio-Retention Basin and Bio-Swale Recommendations

9.11.1 The site is underlain by compacted fill that is generally composed of silty and sandy clay. Based on our experience with the on-site soils and infiltration testing, the compacted fill has low permeability and generally low infiltration characteristics. It is our opinion the compacted fill is unsuitable for infiltration of storm water runoff. Infiltration tests performed for this study are provided in Appendix A and indicate field saturated hydraulic conductivity of 0.01 to 0.001 inches/hour.

9.11.2 Any bio-retention basins, bioswales, and bio-remediation areas should be designed by the project civil engineer and reviewed by Geocon Incorporated. Typically, bioswales consist of a surface layer of vegetation underlain by clean sand. A subdrain should be provided beneath the sand layer. A typical bioswale detail is presented as Figure 11. Prior to discharging into the storm drain pipe or other approved outlet structure, a seepage cutoff wall should be constructed at the interface between the subdrain and storm drainpipe. The concrete cut-off wall should extend at least 6 inches beyond the perimeter of the gravel-packed subdrain system. A typical cut-off wall detail is presented as Figure 12.

9.11.3 Distress may be caused to existing or planned improvements and properties located hydrologically down gradient or adjacent to these devices. The distress depends on the amount of water to be detained, its residence time, soil permeability, and other factors. We have performed a hydrogeology study at the site. Down-gradient and adjacent properties may be subjected to seeps, springs, slope instability, raised groundwater, movement of foundations and slabs, or other impacts as a result of water infiltration. Due to site soil and geologic conditions (i.e. compacted fills), permanent bio-retention basins should be lined with an impermeable barrier, such as 15-mil HDPE, to prevent water infiltration into the underlying compacted fill.

9.11.4 The landscape architect should be consulted to provide the appropriate plant recommendations if a vegetated swale is to be implemented. If drought resistant plants are not used, irrigation may be required.

9.12 Drainage and Maintenance

9.12.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2010 CBC 1803.3 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into storm drains and conduits that carry runoff away from the proposed structure.

9.12.2 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

9.12.3 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

9.13 Grading and Foundation Plan Review

9.13.1 Geocon Incorporated should review the final grading and foundation plans for the project prior to final design submittal to evaluate if additional analysis and/or recommendations are required.

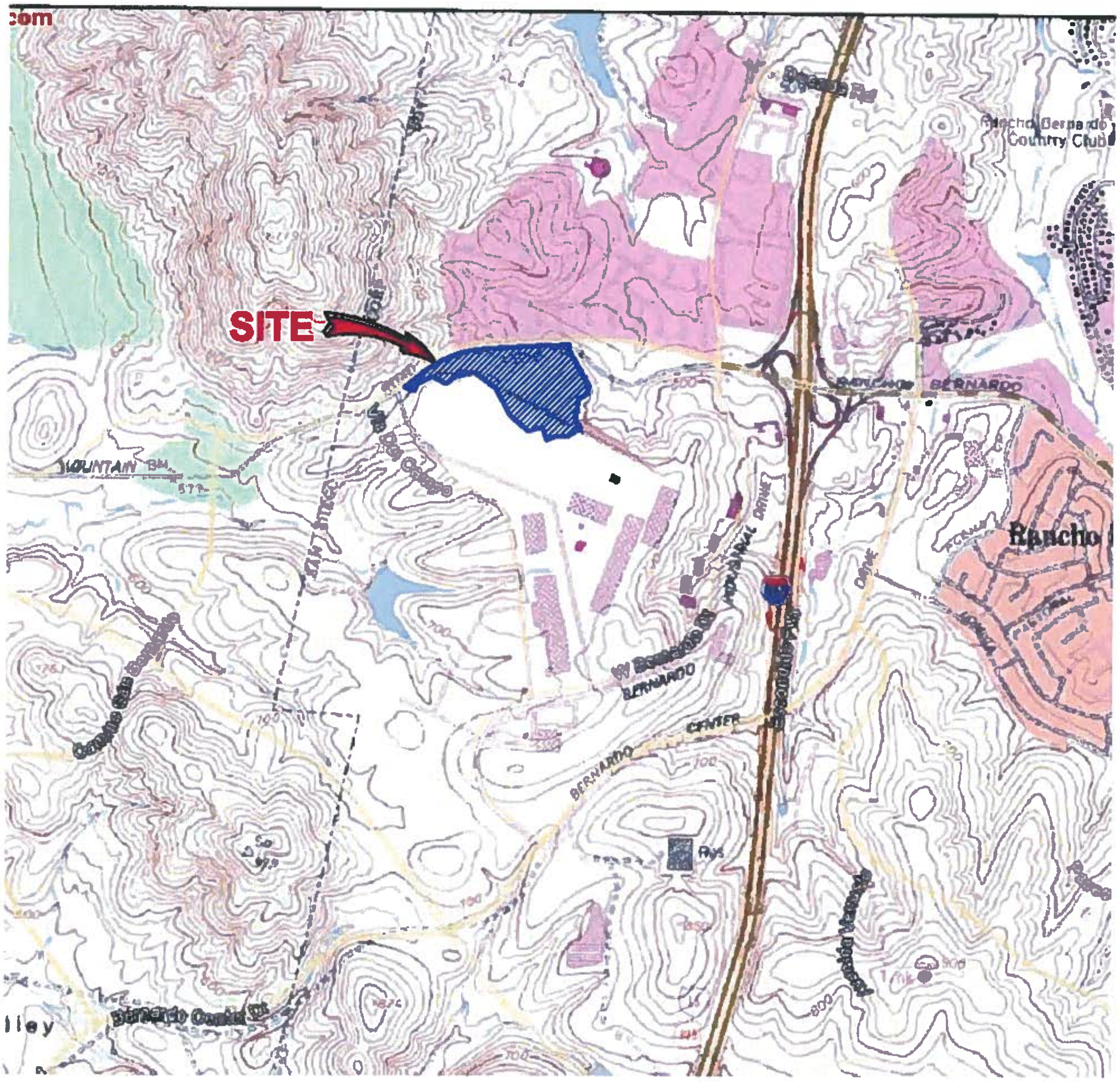
LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.

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 brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor 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 be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may 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 our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.



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VICINITY MAP

PALOMAR COLLEGE SOUTH EDUCATION CENTER
SAN DIEGO, CALIFORNIA

TR / AML		DSK/GTYPD	DATE 10 - 24 - 2012	PROJECT NO. 06847 - 42 - 03	FIG. 1

Vicinity Map

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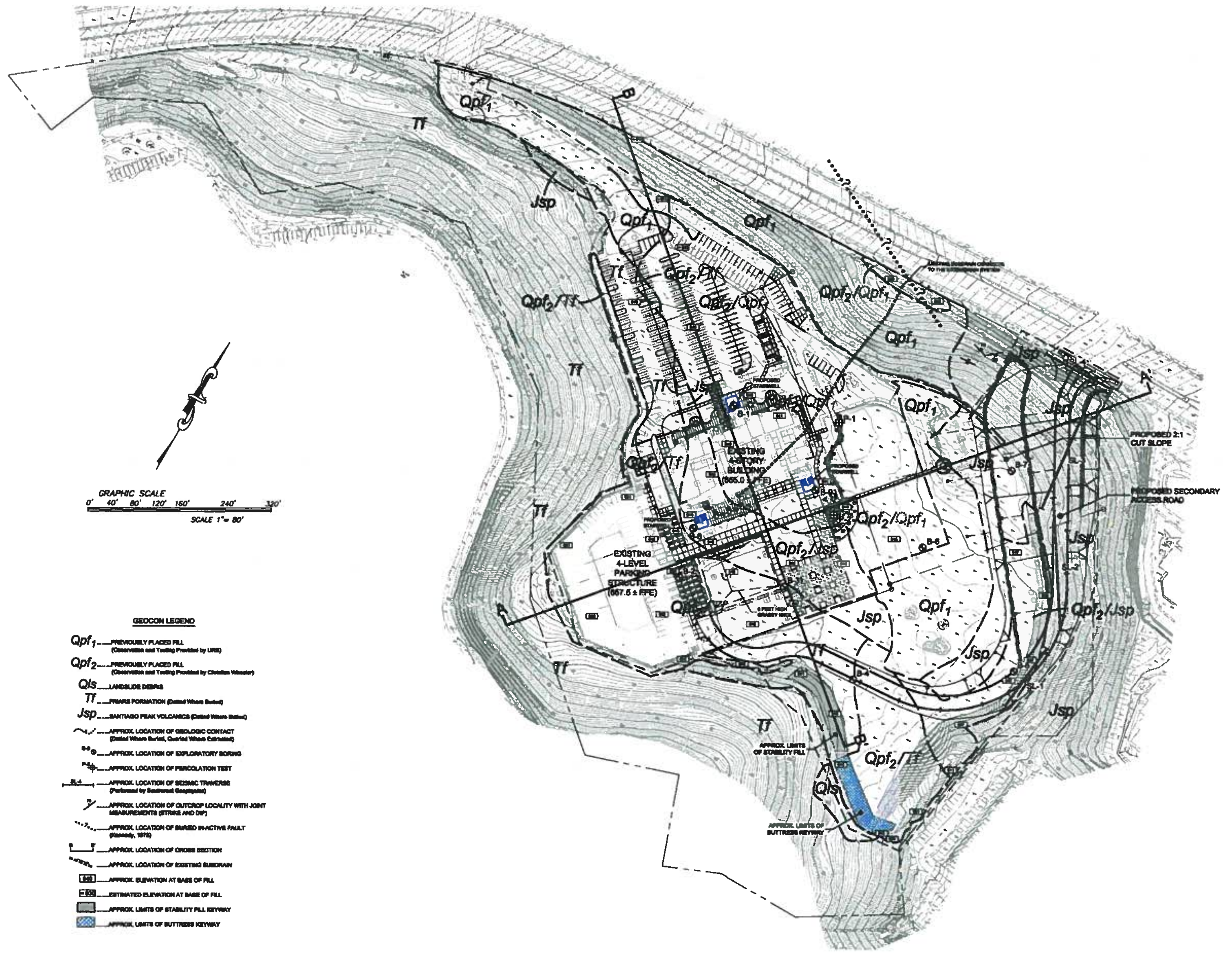
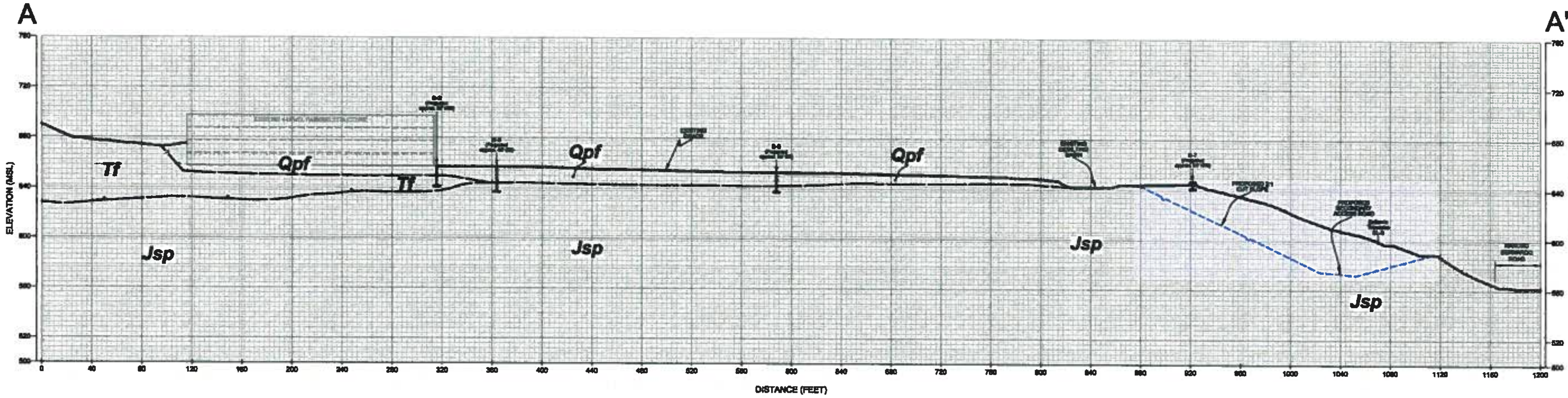


GEOLOGIC MAP

PALOMAR COLLEGE SOUTH EDUCATION CENTER
SAN DIEGO, CALIFORNIA

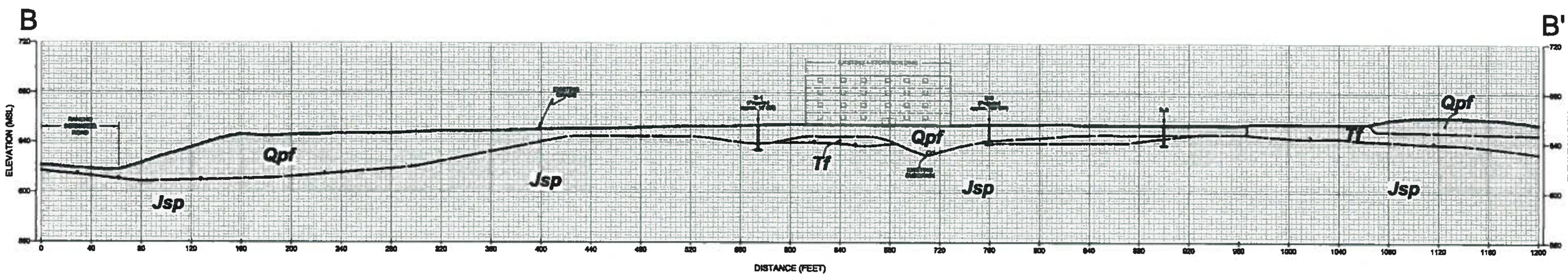
GEOCON INCORPORATED
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS

SCALE 1" = 80'		DATE 10 - 24 - 2012	
PROJECT NO. 06647 - 42 - 03			FIGURE 2
SHEET 1	OF 1		



GEOLOGIC CROSS-SECTION A-A'

SCALE: 1" = 40' (Vert. = Horiz.)



GEOLOGIC CROSS-SECTION B-B'

SCALE: 1" = 40' (Vert. = Horiz.)

GEOCON LEGEND

Qpf ....PREVIOUSLY PLACED FILL

Tf ....FRIARS FORMATION

Jsp ....SANTIAGO PEAK VOLCANICS

B-5 ....APPROX. LOCATION OF BORING

....APPROX. LOCATION OF GEOLOGIC CONTACT (Dotted Where Uncertain)

GEOLOGIC CROSS - SECTIONS

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SCALE 1" = 40'	DATE 10 - 24 - 2012	
PROJECT NO. 06847 - 42 - 03		FIGURE 3
SHEET 1 OF 1		

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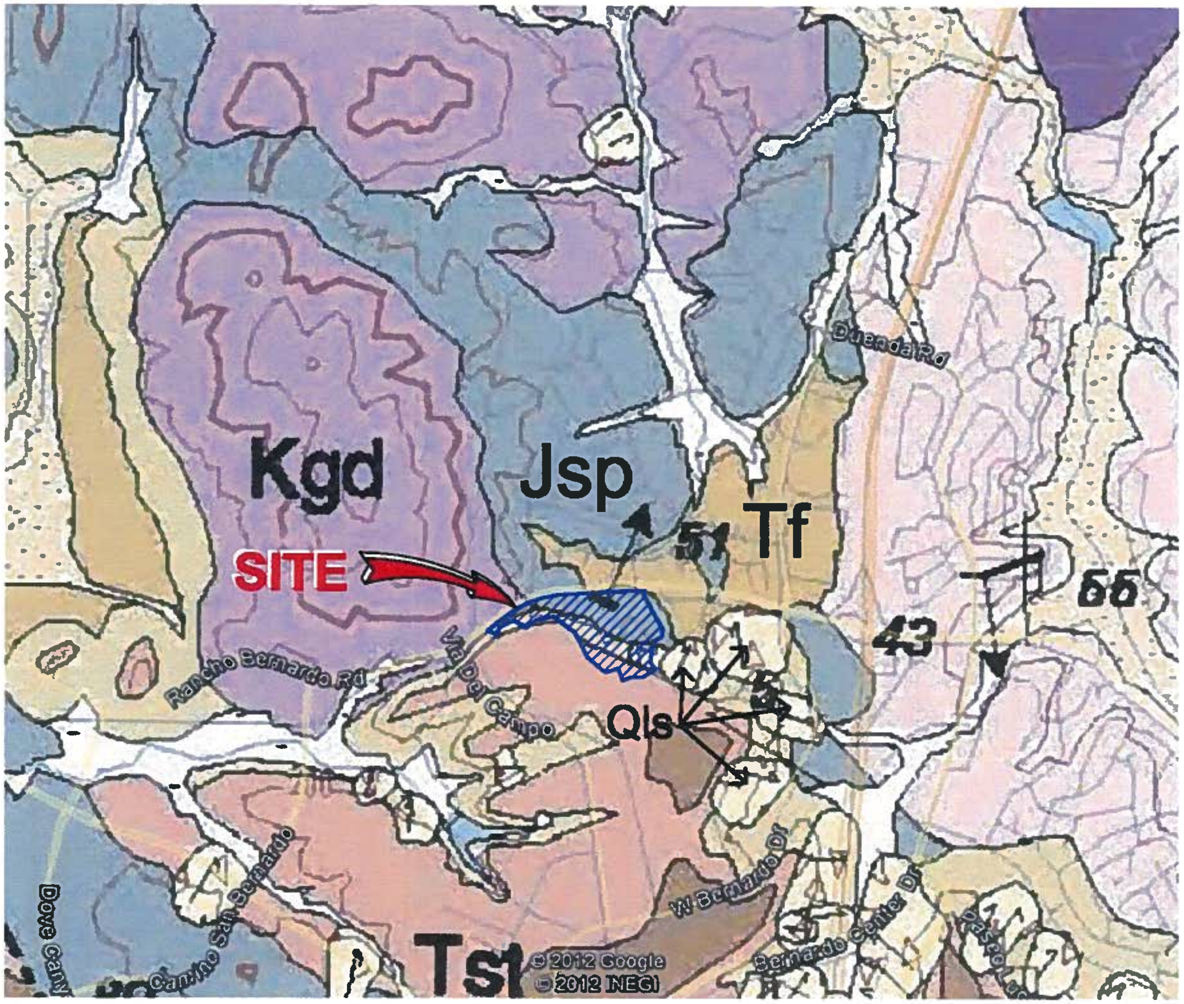

GEOCON LEGEND

- Qls .... LANDSLIDE DEBRIS
- Tst .... STADIUM CONGLOMERATE
- Tf .... FRIARS FORMATION
- Jsp .... SANTIAGO PEAK VOLCANICS
- Kgd .... GRANITIC ROCK

NO SCALE

SOURCE: TAN, S.S. and Kennedy, M.P., 2008, *Geologic Map of San Diego 30'x60' Quadrangle, California* *U.S. Geological Survey, Department of Earth Sciences, University of California, Riverside*

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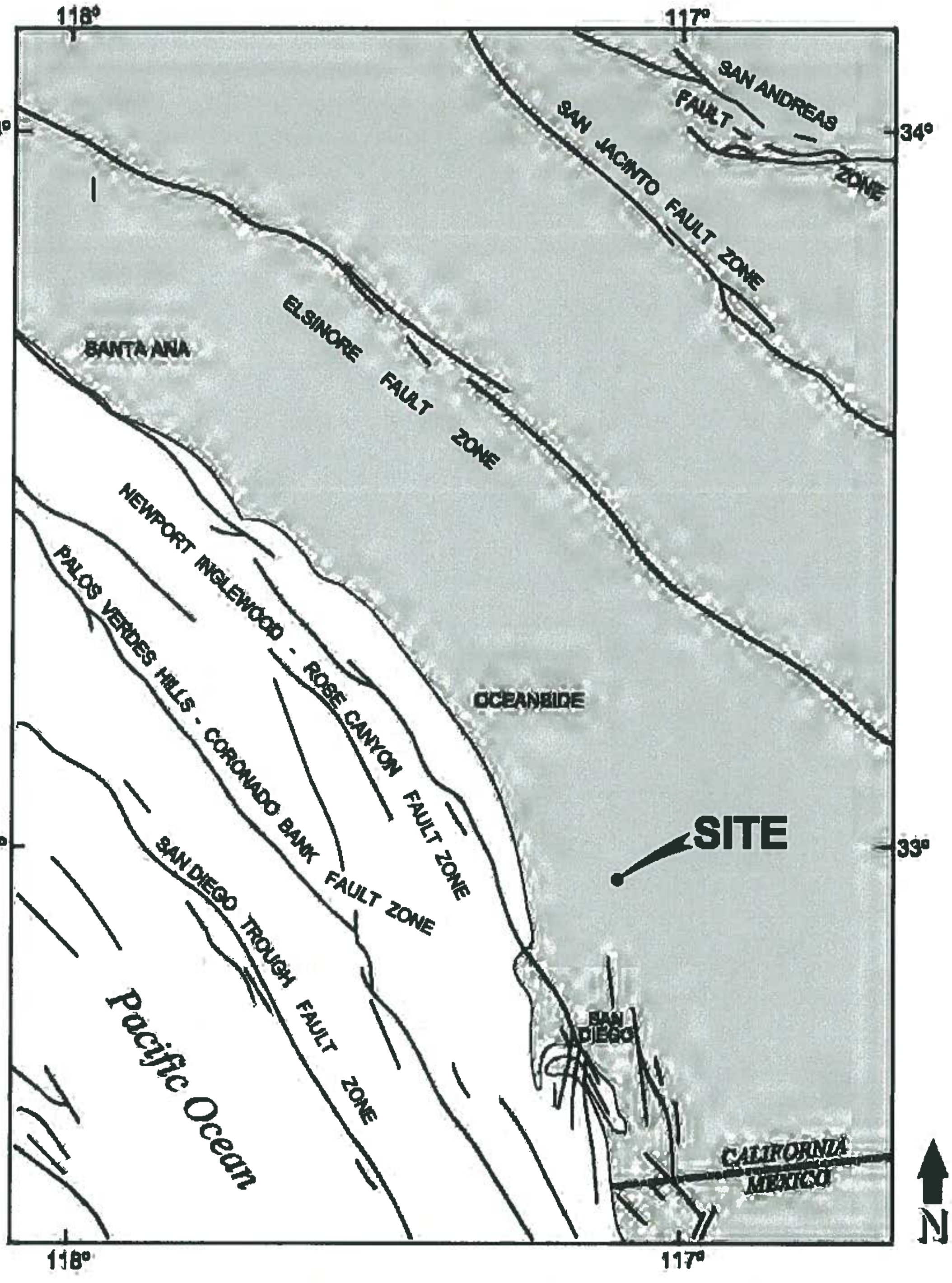
REGIONAL GEOLOGIC MAP

PROJECT NO. 06647 - 42 - 03
FIGURE 4
DATE 10 - 24 - 2012



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REGIONAL FAULT MAP

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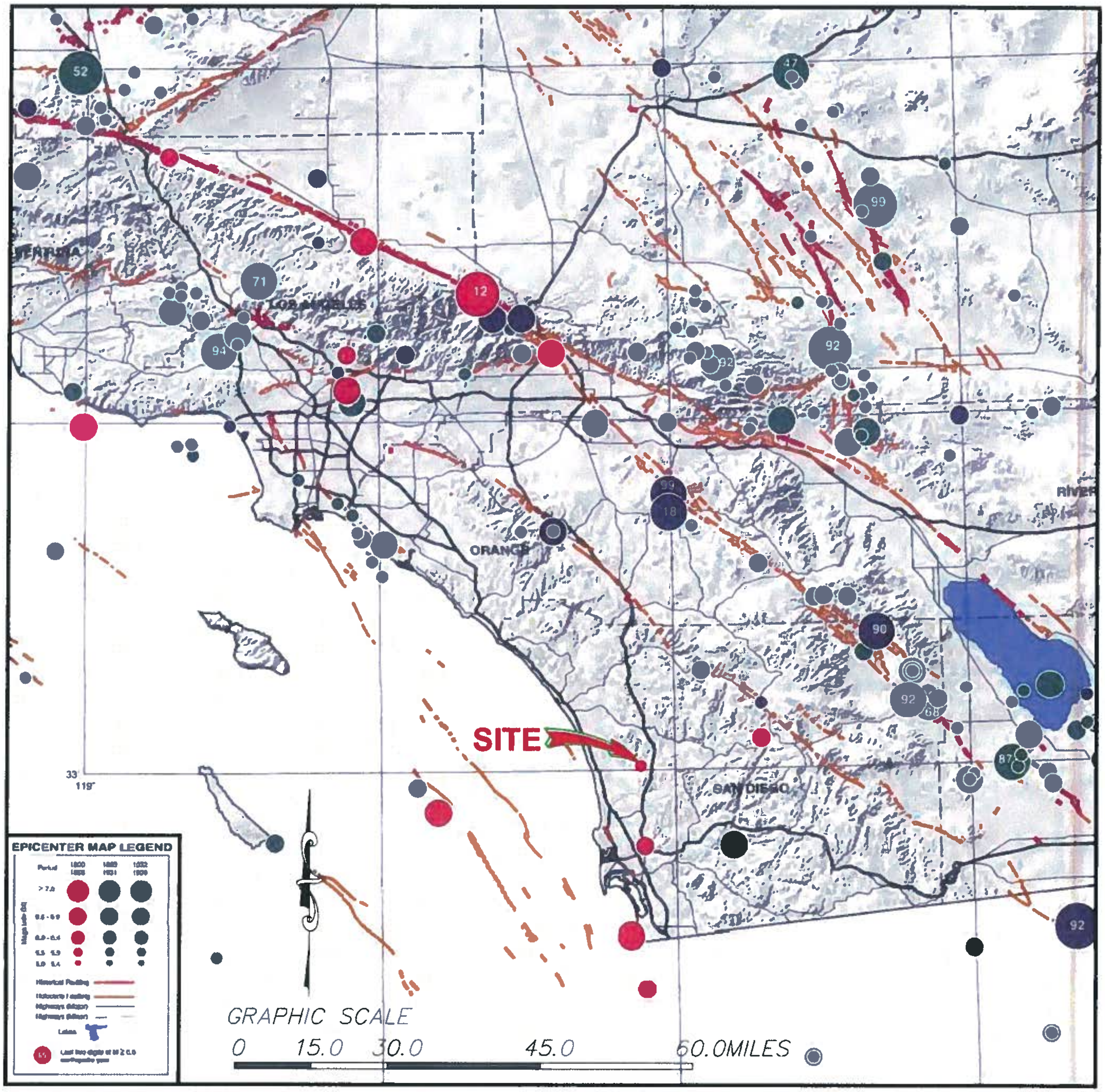
TR / AML		DSK/GTYPD	DATE 10 - 24 - 2012	PROJECT NO. 08647 - 42 - 03	FIG. 5

Regional Fault Map



EPICENTER OF ≥ 5 CALIFORNIA EARTHQUAKES, 1800-1999

REGIONAL SEISMICITY MAP

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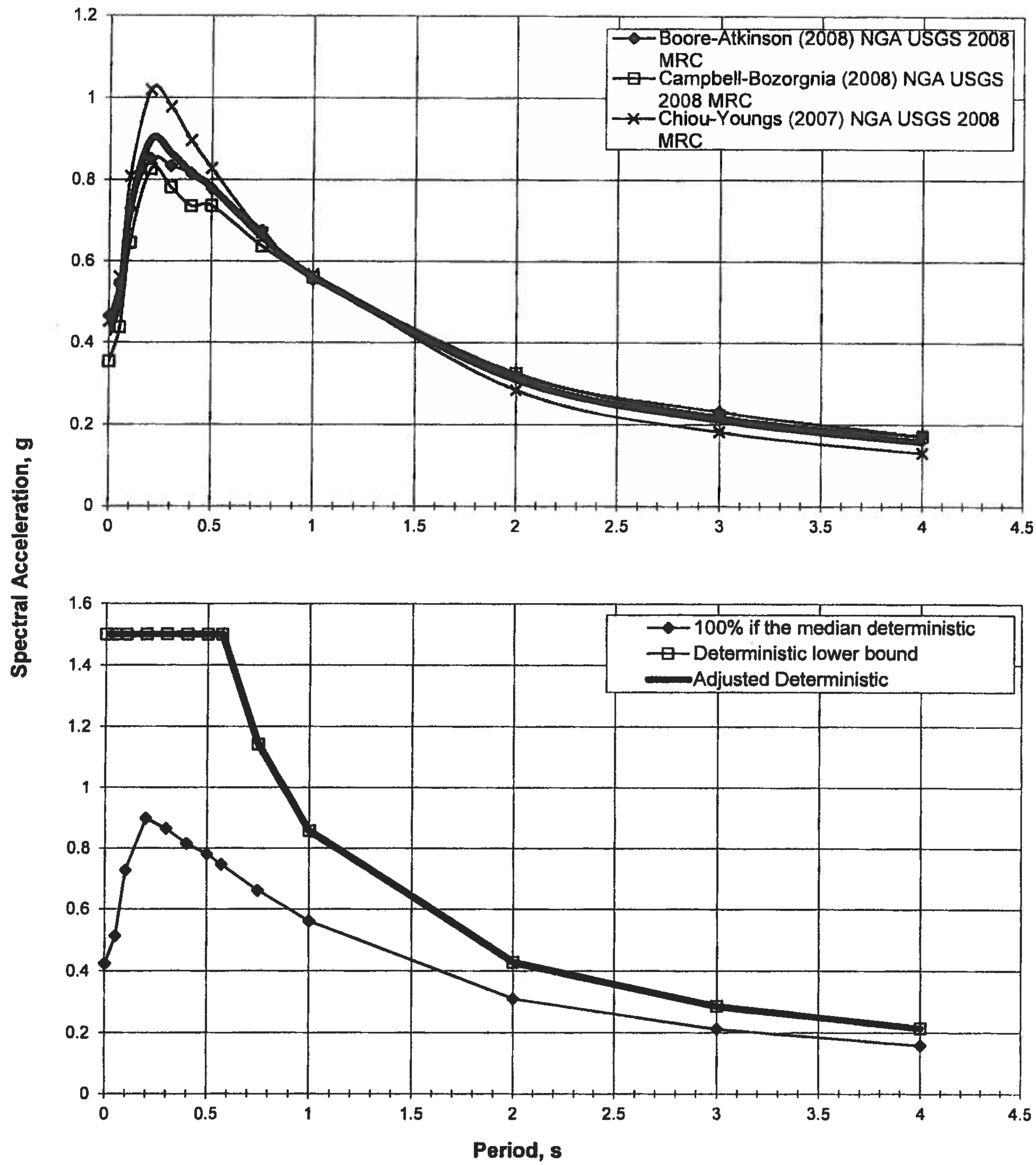
TR / AML		DSK/E0000	DATE 10 - 24 - 2012	PROJECT NO. 06647 - 42 - 03	FIG. 6

Y:/PROJECTS/06647-42-03 PALOMAR COLLEGE SOUTH EDUCATION CENTER/DETAILS/06647-42-03 RegionalSiesmicityMap.dwg

Y:\PROJECTS\06647-42-03 PALOMAR COLLEGE SOUTH EDUCATION CENTER\DETAILS\06647-42-03 RegionalSiesmicityMap.dwg, 10/22/2012 2:13:08 PM, Ladrillono A



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DESIGN RESPONSE SPECTRUM

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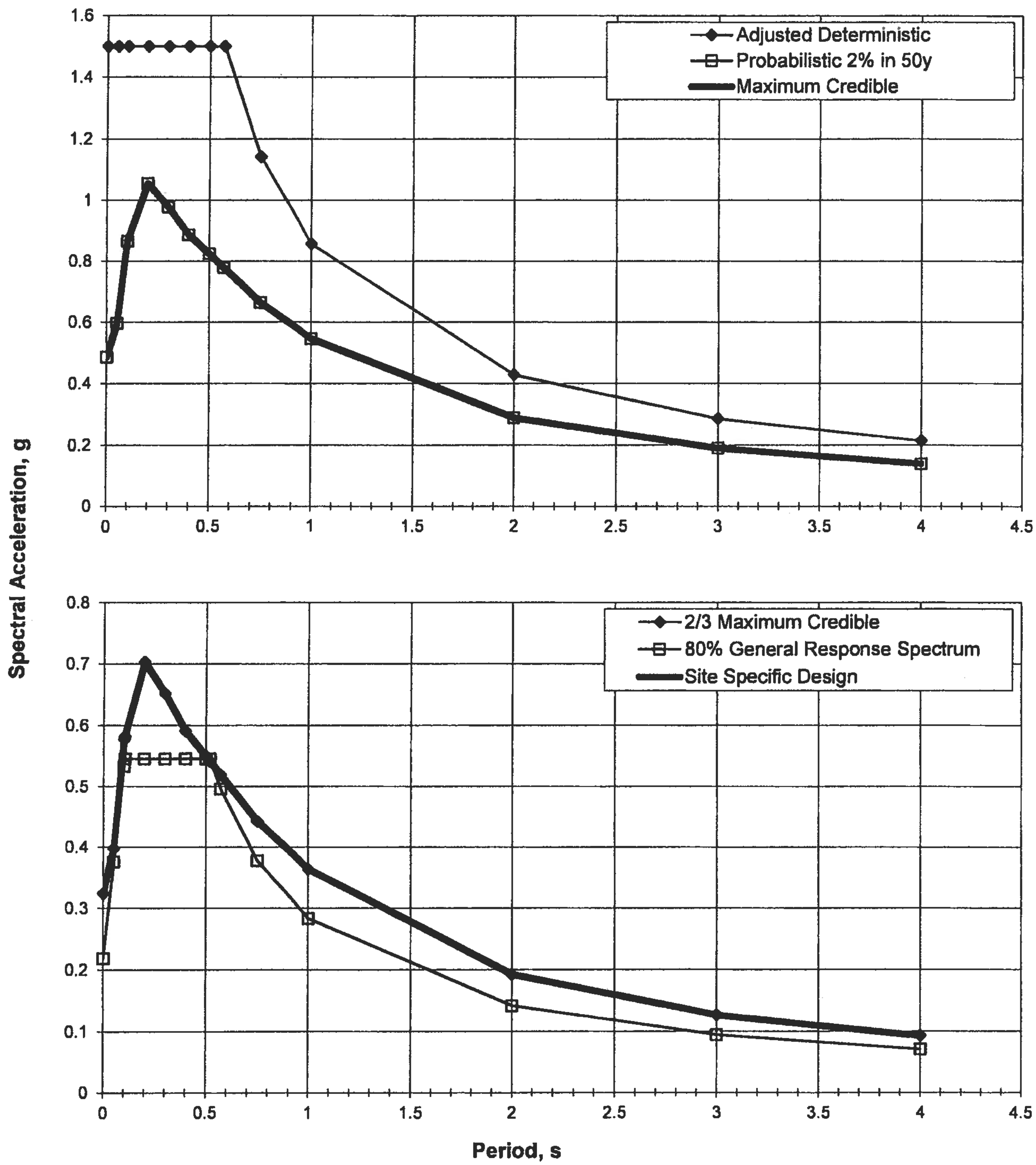
DATE: 10-24-12	PROJECT NO. 06647-42-03	FIG. 7



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DESIGN RESPONSE SPECTRUM

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DATE: 10-24-12 | PROJECT NO. 06647-42-03 | FIG. 8

Ground Motion Spectral Accelerations, (g)

Spectral Period (seconds)	2% in 50 Year Mean Prob.	Mean Det.	Det. Lower Bound	Site Specific MCE	2/3 SS MCE	80% of General Response	Final Site Specific Response
0.000	0.486	0.423	1.500	0.486	0.324	0.218	0.324
0.050	0.596	0.513	1.500	0.596	0.398	0.376	0.398
0.100	0.866	0.727	1.500	0.866	0.577	0.534	0.577
0.104	0.873	0.734	1.500	0.873	0.582	0.546	0.582
0.200	1.055	0.898	1.500	1.055	0.703	0.546	0.703
0.300	0.979	0.864	1.500	0.979	0.652	0.546	0.652
0.400	0.887	0.815	1.500	0.887	0.591	0.546	0.591
0.500	0.824	0.781	1.500	0.824	0.550	0.546	0.550
0.509	0.818	0.776	1.500	0.818	0.546	0.546	0.546
0.519	0.812	0.771	1.500	0.812	0.541	0.546	0.546
0.527	0.807	0.768	1.500	0.807	0.538	0.538	0.538
0.571	0.779	0.747	1.500	0.779	0.519	0.496	0.519
0.750	0.664	0.661	1.142	0.664	0.443	0.378	0.443
1.000	0.546	0.561	0.857	0.546	0.364	0.283	0.364
2.000	0.289	0.310	0.428	0.289	0.192	0.142	0.192
3.000	0.190	0.212	0.286	0.190	0.127	0.094	0.127
4.000	0.139	0.158	0.214	0.139	0.093	0.071	0.093

S_{DS} is Final Site Specific Response at 0.2 sec but not less than 90% of peak after 0.2 sec

S_{DS} = 0.703 g

S_{D1} is greater value of Final Site Specific Response at 1.0 sec or 2 times 2.0 sec

S_{D1} = 0.384 g

$S_{MS} = 3*S_{DS}/2$

S_{MS} = 1.055 g

$S_{M1} = 3*S_{D1}/2$

S_{M1} = 0.576 g

F_A = 1.000 From USGS

F_V = 1.428 From USGS

$S_S = S_{MS}/F_A$

S_S = 1.055 g

$S_1 = S_{M1}/F_V$

S_1 = 0.403 g

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DESIGN RESPONSE SPECTRUM

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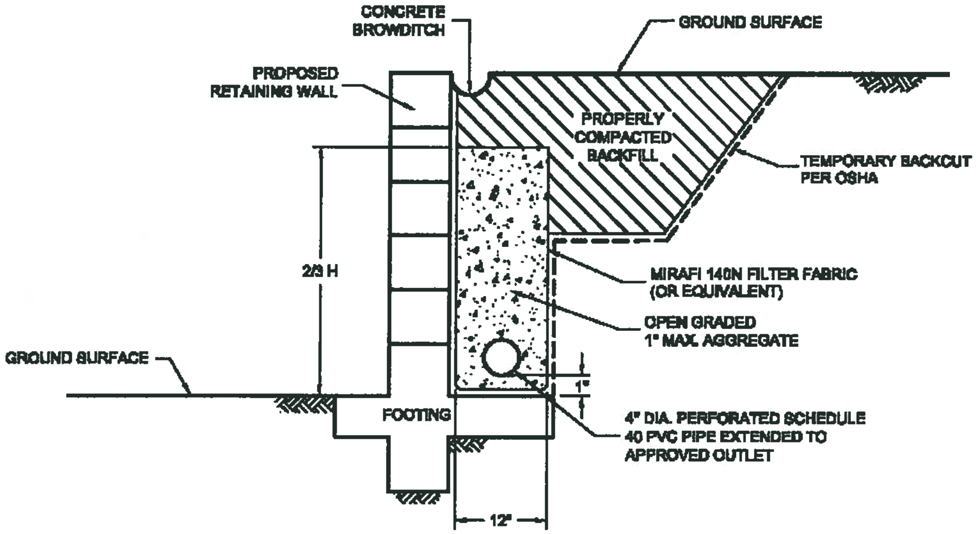
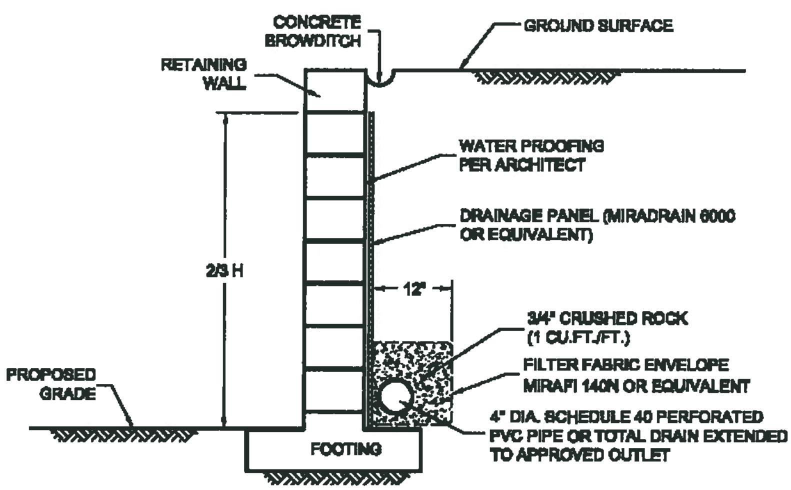
DATE: 10-24-12 | PROJECT NO. 06647-42-03 | FIG. 9





NOTE :

DRAIN SHOULD BE UNIFORMLY SLOPED TO GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPING

NO SCALE

TYPICAL RETAINING WALL DRAIN DETAIL

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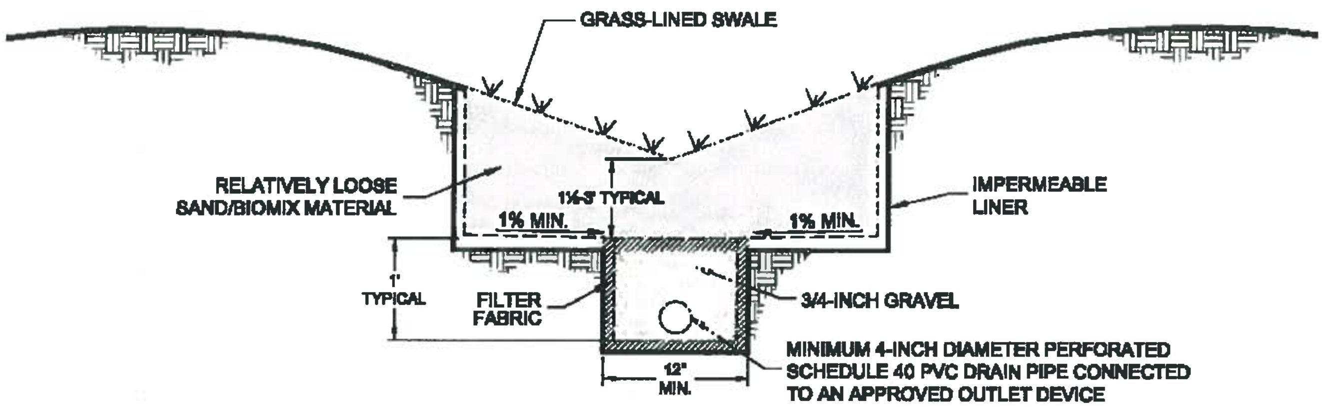
TM / AML		DSK/GTYPD	DATE 10 - 24 - 2012	PROJECT NO. G6647 - 42 - 03	FIG. 10

RWD07



NO SCALE

TYPICAL BIOSWALE DETAIL

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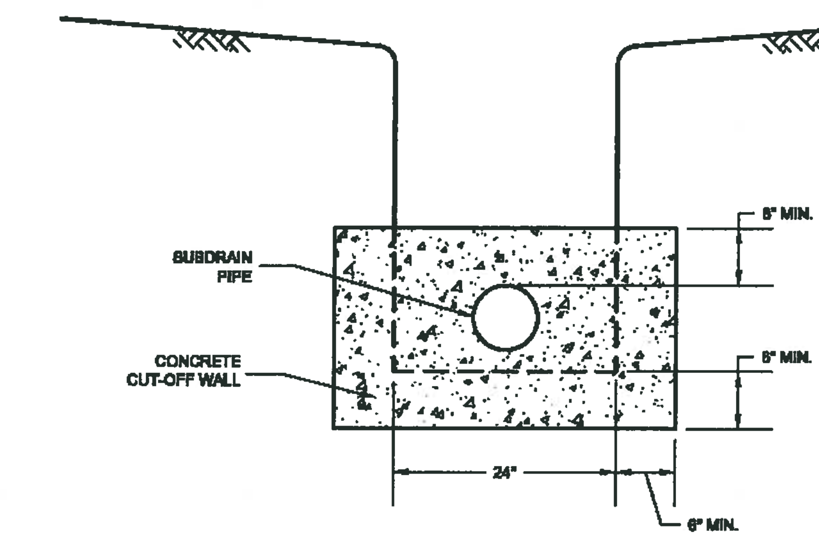
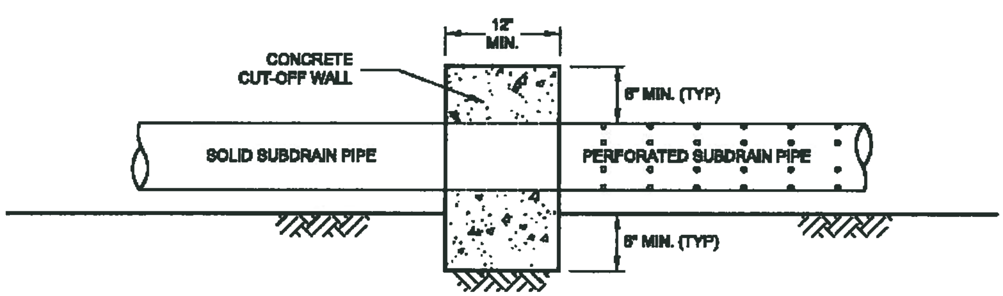
TR / AML		DSK/GTYPD	DATE 10 - 24 - 2012	PROJECT NO. 06647 - 42 - 03	FIG. 11

FRONT VIEW



NO SCALE

SIDE VIEW



NO SCALE

TYPICAL SUBDRAIN CUT-OFF WALL DETAIL

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TR / AML		DSK/GTYPD	DATE 10 - 24 - 2012	PROJECT NO. 06647 - 42 - 03	FIG. 12

RSCOW

Y:/PROJECTS/06647-42-03 PALOMAR COLLEGE SOUTH EDUCATION CENTER/DETAILS/RSCOW.dwg

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed between September 24 and October 3, 2012, and consisted of a visual site reconnaissance, advancing 9 hollow-stem auger borings, performing 2 permeability tests, and conducting 4 seismic refraction traverses. The approximate locations of the borings, permeability tests and traverses are shown on the *Geologic Map/Site Plan*, Figure 2. The soils encountered in the borings were visually classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual Manual Procedure D 2844).

The borings were performed by Baja Exploration and were advanced to a maximum depth of 18½ feet below existing grade using a CME-75 rig equipped with 6-inch, hollow-stem augers. Relatively undisturbed samples were obtained by driving a 3-inch, O.D., split-tube sampler into the "undisturbed" soil mass. The sampler was equipped with 1-inch by 2⅜-inch, brass sampler rings to facilitate removal and testing. Logs of the borings depicting the soil and geologic conditions encountered and the depth at which samples were obtained are presented on Figures A-1 through A-9.

Two permeability tests were performed at the approximate locations determined by LPA, Inc. The tests were conducted at depths ranging between 12 to 18 inches below the existing ground surface using a Guelph Permeameter. The 3 inch diameter test holes were hand augured to the testing depths. The test locations are shown on Figure 2 and the results of the permeability testing are presented below in Table A-I. The soil types encountered generally consisted stiff to very stiff, silty to sandy clays.

TABLE A-I
PERMEABILITY TEST RESULTS

Sample No.	Field Saturated Permeability [Guelph Permeameter] (in/hr)
P-1	0.01
P-2	0.001

The four seismic traverses performed by Southwest Geophysics are discussed in greater detail in their report presented in Appendix C.

BORING B 1

ELEV. (MSL.) **653'** DATE COMPLETED **09-24-2012**

EQUIPMENT **CME 75** BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**4" ASPHALT CONCRETE**			
	B1-1			CL	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, moist, mottled grayish brown and bluish gray, Sandy CLAY; few gravel			
2								
	B1-2					26	121.4	13.4
4					-Becomes mottled olive brown to brown, grayish brown and yellowish brown; little gravel			
	B1-3					36	114.2	15.4
6								
8				SM	Medium dense, moist, yellowish brown to olive brown, Silty, fine to coarse SAND; little gravel; little clay			
10	B1-4				-Blowcount at 10' not accurate due to gravel observed in shoe	67/11"	117.1	11.6
12	B1-5			CL	Stiff, moist, dark grayish brown to dark gray, Silty to Sandy CLAY; trace gravel			
14					**SANTIAGO PEAK VOLCANICS (Jsp)** Moderately weathered, grayish brown, moderately strong METAVOLCANIC ROCK			
	B1-6				-Disturbed sample due to rock	90		
16								
18	B1-7					50/4"		
					REFUSAL AT 18.5 FEET No groundwater encountered Backfilled on 09-24-2012			

Figure A-1,
Log of Boring B 1, Page 1 of 1

06647-42-03.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B 2

ELEV. (MSL.) **657'** DATE COMPLETED **09-24-2012**

EQUIPMENT **CME 75** BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			5.5" **ASPHALT CONCRETE** over 6.5" **AGGREGATE BASE**			
		CL	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, moist, mottled greenish gray, olive brown, and yellowish brown, Sandy CLAY, trace gravel; some chunks of sandy siltstone derived from Friars Formation			
2	B2-1			24		
4	B2-2					
	B2-3			67/10"		
6			-Becomes silty clay below 5 feet; blow counts not accurate due to gravel			
		SM	**FRIARS FORMATION (Tf)** Dense, damp to moist, mottled greenish gray, yellowish brown and olive brown, Silty, fine- to medium-grained SANDSTONE; weakly cemented			
8	B2-4			73/9"		
10	B2-5		-Becomes mottled light tan and yellowish brown; fine- to coarse-grained	77		
12						
14		SM/ML	Dense, damp, mottled light greenish gray, light tan and yellowish brown, Silty, fine-grained SANDSTONE/Sandy SILTSTONE; weakly cemented			
	B2-6			75/10"		
16			BORING TERMINATED AT 16 FEET No groundwater encountered Backfilled on 09-24-2012			

Figure A-2,
Log of Boring B 2, Page 1 of 1

06647-42-03.GPJ

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

BORING B 3

ELEV. (MSL.) **654.5'** DATE COMPLETED **09-24-2012**

EQUIPMENT **CME 75** BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B3-1			ML/CL	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, dry to damp, mottled light olive brown, light greenish gray, and light gray, Sandy SILT/Sandy CLAY; trace gravel			
2	B3-2				-Becomes moist	21	100.1	21.4
4								
	B3-3				-Becomes stiff, moist, mottled light olive brown, light greenish gray, and light gray	27	104.0	21.8
6								
	B3-4			ML/SM	**FRIARS FORMATION (Tf)** Hard/dense, damp, mottled light gray to white and dark yellowish brown, Sandy SILTSTONE/Silty, fine-grained SANDSTONE; strongly cemented			
8								
10	B3-5				**SANTIAGO PEAK VOLCANICS (Jsp)** Completely weathered, mottled pale green with red oxidation, weak METAVOLCANIC ROCK (saprolite)	50/4"		
12								
14					-Becomes less weathered and stronger with depth			
	B3-6				-Very poor recovery	50/2"		
16					-Very hard drilling; augers grinding with no advancement			
					REFUSAL AT 16.5 FEET No groundwater encountered Backfilled on 09-24-2012			

Figure A-3,
Log of Boring B 3, Page 1 of 1

06647-42-03.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 4 ELEV. (MSL.) 654' DATE COMPLETED 09-24-2012 EQUIPMENT CME 75 BY: N.G. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	B4-1			ML/SM	**FRIARS FORMATION (Tf)** Hard/dense, dry, mottled light gray to light greenish gray and yellowish brown, Sandy SILTSTONE/Silty, fine-grained SANDSTONE; weakly cemented			
2				SM	Very dense, damp, light gray and light greenish gray, Silty, fine- to medium-grained SANDSTONE; weakly cemented			
	B4-2					93/8"		
4								
	B4-3					50/6"		
6								
8								
10	B4-4					50/4"		
					BORING TERMINATED AT 10.5 FEET No groundwater encountered Backfilled on 09-24-2012			

Figure A-4,
Log of Boring B 4, Page 1 of 1

06647-42-03.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. 06647-42-03

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 5 ELEV. (MSL.) 651' DATE COMPLETED 09-24-2012 EQUIPMENT CME 75 BY: N.G. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	B5-1			ML	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, dry, light greenish gray to light olive brown, Sandy SILT; little angular gravel and cobble			
2	B5-2				-Blowcounts likely not accurate due to gravels	62		
4	B5-3				**SANTIAGO PEAK VOLCANICS (Jsp)** Moderately weathered, light grayish brown to orange brown, moderately strong METAVOLCANIC ROCK	50/4"		
					REFUSAL AT 5.5 FEET No groundwater encountered Backfilled on 09-24-2012			

Figure A-5,
Log of Boring B 5, Page 1 of 1

06647-42-03.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 6

ELEV. (MSL.) **651'** DATE COMPLETED **09-24-2012**

EQUIPMENT **CME 75** BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B6-1			ML/CL	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, dry to damp, grayish brown to light grayish brown, Sandy SILT/Sandy CLAY; few gravel and cobble			
2								
	B6-2					38		
4								
	B6-3					34		
				CL	Stiff to very stiff, moist, dark brown to yellowish brown, Sandy CLAY			
6	B6-4							
	B6-5					50/1"		
					-Very hard drilling at 7 feet; auger grinding on rock with no advancement; likely Santiago Peak Volcanics contact; no recovery			

REFUSAL AT 7 FEET
No groundwater encountered
Backfilled on 09-24-2012

Figure A-6,
Log of Boring B 6, Page 1 of 1

06647-42-03.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 7 ELEV. (MSL.) 646' DATE COMPLETED 09-25-2012 EQUIPMENT CME 75 BY: N.G. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	B7-1			SM	**SANTIAGO PEAK VOLCANICS (Jsp)** Moderately weathered, mottled yellowish brown, light tan and bluish gray, moderately strong METAVOLCANIC ROCK			
2								
4								
	B7-2					50/4"		
					BORING TERMINATED AT 5.5 FEET No groundwater encountered Backfilled on 09-25-2012			

Figure A-7,
Log of Boring B 7, Page 1 of 1

06647-42-03.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

BORING B 8

ELEV. (MSL.) **656'** DATE COMPLETED **09-25-2012**

EQUIPMENT **CME 75** BY: **N.G. BORJA**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		CL	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, damp to moist, dark grayish brown, Sandy CLAY; trace gravel			
2		SC & CL	Very stiff, moist, mottled dark brown and dark yellowish brown, Clayey, fine to coarse SAND and Silty CLAY; trace coarse; trace asphalt concrete			
	B8-1			39	125.2	11.2
4	B8-2					
	B8-3			45	121.9	9.5
6		CL	Stiff, moist, mottled yellowish brown, light tan and greenish gray, Sandy CLAY			
8	B8-4			28	117.3	14.0
		CL	Stiff, moist, mottled light tan brown, dark grayish brown and greenish gray, Silty CLAY			
10	B8-5	SC & CL	Stiff, moist, dark brown and brown, Clayey, fine to coarse SAND and Silty CLAY; trace gravel	25	107.0	19.3
12			**SANTIAGO PEAK VOLCANICS (Jsp)** Completely weathered, mottled pale green with red oxidation, weak METAVOLCANIC ROCK (saprolite)			
14						
	B8-6			67		
16						
18	B8-7			50/2"		
			BORING TERMINATED AT 18.5 FEET No groundwater encountered Backfilled on 09-25-2012			

Figure A-8,
Log of Boring B 8, Page 1 of 1

06647-42-03.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B 9 ELEV. (MSL.) 653' DATE COMPLETED 09-25-2012 EQUIPMENT CME 75 BY: N.G. BORJA	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	B9-1			ML/SM	**PREVIOUSLY PLACED FILL (Qpf)** Stiff, moist, yellowish brown to light tan, Sandy SILT/Silty, fine to medium SAND; trace gravel; few roots			
2								
4				CL	Stiff, moist, mottled dark grayish brown, brown and yellowish brown, Sandy/Silty CLAY; trace gravel			
	B9-2					28	109.0	20.2
6								
					-Becomes mottled yellowish brown and greenish gray			
8								
10	B9-3					35	112.6	14.7
					-Becomes dark brown to dark gray			
12					**SANTIAGO PEAK VOLCANICS (Jsp)** Moderately weathered, light bluish gray, moderately strong METAVOLCANIC ROCK; disturbed sample due to rock			
14								
	B9-4					50/6"		2.8
					REFUSAL AT 15.5 FEET No groundwater encountered Backfilled on 09-25-2012			

Figure A-9,
Log of Boring B 9, Page 1 of 1

06647-42-03.GPJ

SAMPLE SYMBOLS			
	☐ ... SAMPLING UNSUCCESSFUL	... STANDARD PENETRATION TEST	... DRIVE SAMPLE (UNDISTURBED)
	... DISTURBED OR BAG SAMPLE	... CHUNK SAMPLE	▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for in situ moisture density, maximum dry density and optimum moisture content, pH, resistivity, chloride ion content, water-soluble sulfate content, consolidation, grain size, R-value, and expansion characteristics. The results of the laboratory tests are summarized in Tables B-I through B-VII and Figures B-1 through B-3. The in-place dry density and moisture content of the samples tested are presented on the boring logs in Appendix A.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B8-2	Brownish gray, Sandy CLAY with trace gravel	124.9	11.8

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index	CBC Classification
	Before Test (%)	After Test (%)			
B1-1	10.1	24.0	108.5	69	Medium
B3-1	15.2	32.5	93.6	83	Medium

TABLE B-III
SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (PH) AND RESISTIVITY TEST RESULTS
CTM NO. 643

Sample No.	pH	Resistivity (ohm centimeters)
B1-1	7.24	540
B3-1	7.6	370

TABLE B-IV
SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
CTM NO. 417

Sample No.	Water-Soluble Sulfate (%)
B1-1	0.085
B3-1	0.018

TABLE B-V
SUMMARY OF LABORATORY CHLORIDE ION CONTENT TEST RESULTS
AASHTO TEST NO. T 291

Sample No.	Chloride Ion Content (%)	Chloride Ion Content (ppm)
B1-1	0.026	261
B3-1	0.059	593

TABLE B-VI
SUMMARY OF LABORATORY RESISTANCE (R-VALUE) TEST RESULTS
ASTM D 2844

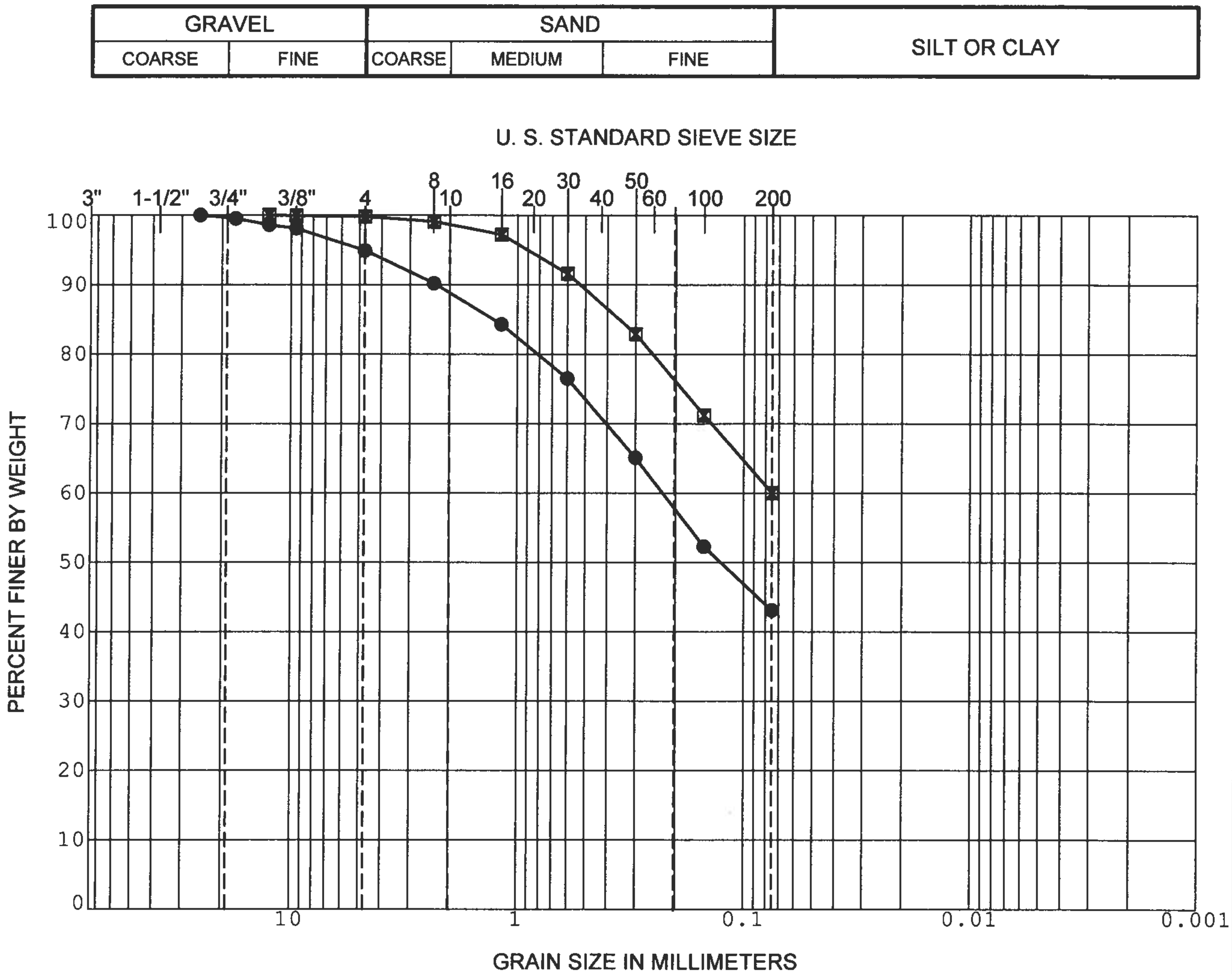
Sample No.	Description	R-Value
B5-1	Grayish brown, Clayey/Sandy SILT	17

TABLE B-VII
SUMMARY OF LABORATORY PLASTICITY TEST RESULTS
ASTM D 4318

Sample No.	Liquid Limit	Plastic Limit	Plasticity Index	USCS* Classification
B1-1	46	18	28	CL
B3-1	49	23	26	CL

*Test performed only on the material passing No. 40 Sieve.

PROJECT NO. 06647-42-03



	SAMPLE	DEPTH (ft)	CLASSIFICATION	NAT WC	LL	PL	PI
●	B1-1	0.5	CL - Sandy CLAY		46	18	28
⊠	B3-1	0.0	ML/CL - Sandy SILT / Sandy CLAY		49	23	26
▲							

GRADATION CURVE

PALOMAR COLLEGE SOUTH EDUCATION CENTER

SAN DIEGO, CALIFORNIA

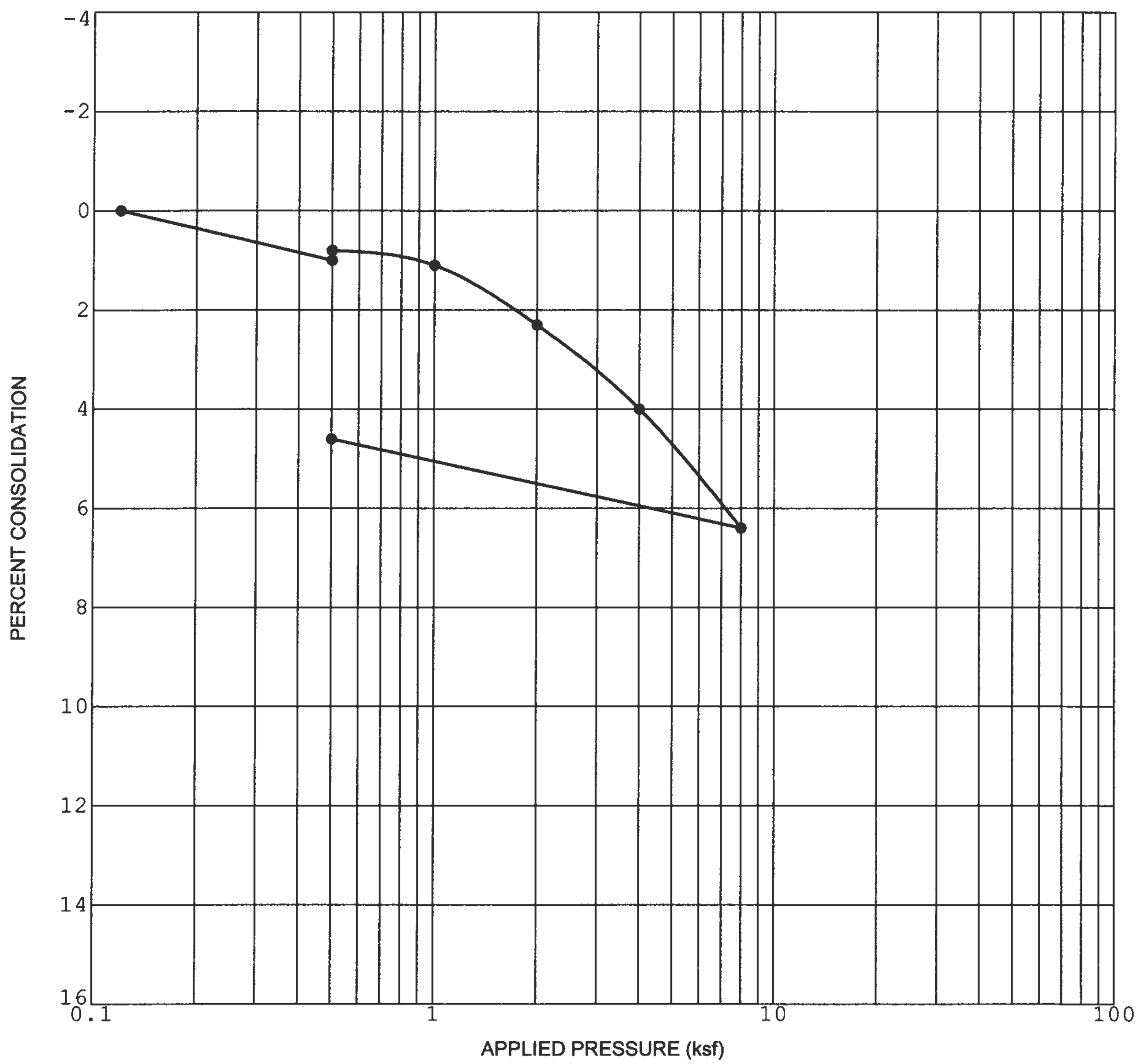
06647-42-03.GPJ

Figure B-1

GEOCON

PROJECT NO. 06647-42-03

SAMPLE NO. B1-3



Initial Dry Density (pcf)	114.2	**Initial Saturation (%)**	90.4
Initial Water Content (%)	15.4	**Sample Saturated at (ksf)**	.5

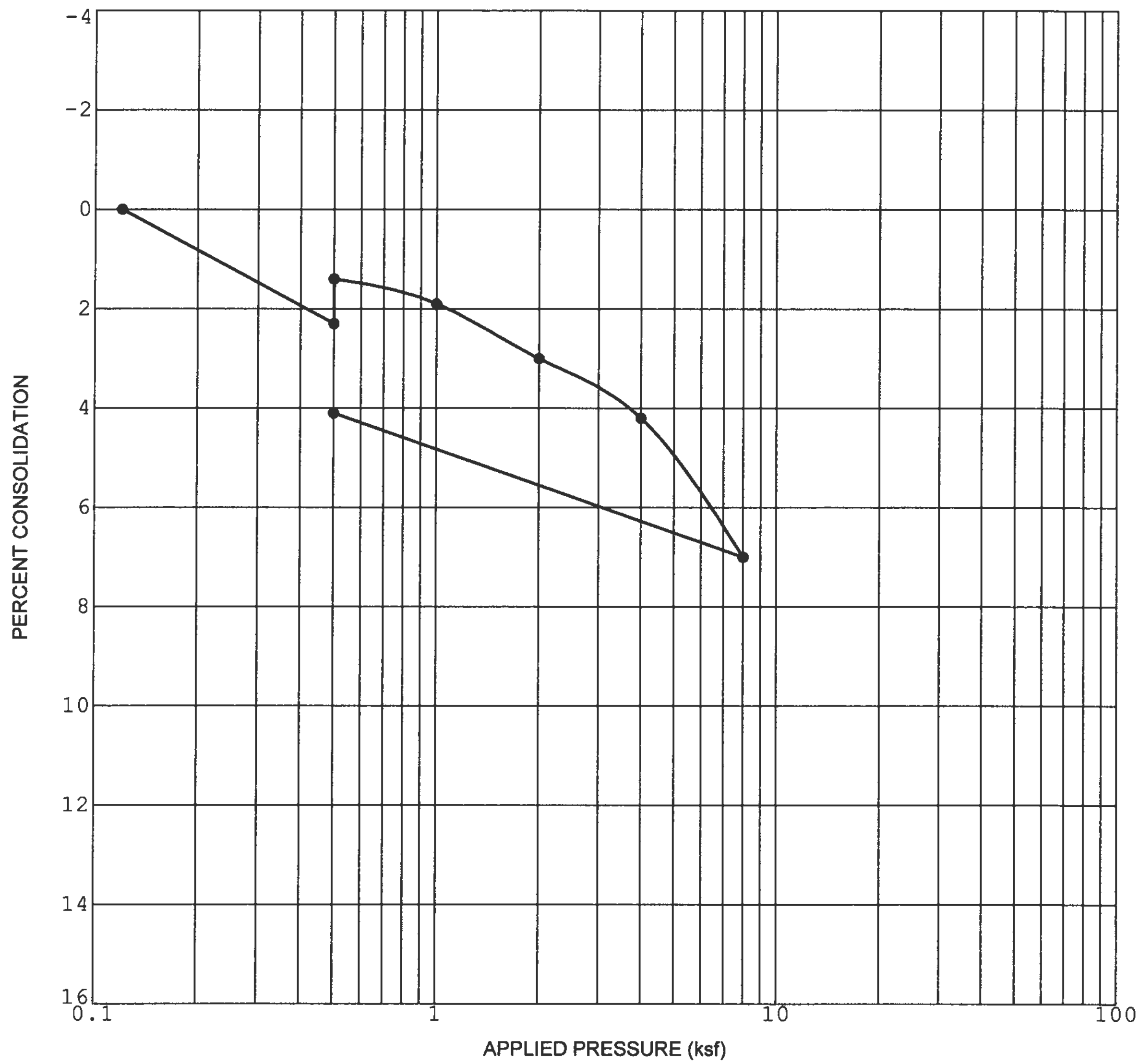
CONSOLIDATION CURVE

PALOMAR COLLEGE SOUTH EDUCATION CENTER

SAN DIEGO, CALIFORNIA

PROJECT NO. 06647-42-03

SAMPLE NO. B9-2



Initial Dry Density (pcf)	109.0
Initial Water Content (%)	20.2

Initial Saturation (%)	100
Sample Saturated at (ksf)	.5

CONSOLIDATION CURVE

PALOMAR COLLEGE SOUTH EDUCATION CENTER

SAN DIEGO, CALIFORNIA

06647-42-03.GPJ

Figure B-3

GEOCON

APPENDIX C

APPENDIX C

SEISMIC REFRACTION SURVEY REPORT
BY
SOUTHWEST GEOPHYSICS

FOR

PALOMAR COLLEGE
SOUTH EDUCATION CENTER
IMPROVEMENT PROJECT
SAN DIEGO, CALIFORNIA

PROJECT NO. 06647-42-03

SEISMIC REFRACTION SURVEY
11111 RANCHO BERNARDO ROAD
SAN DIEGO, CALIFORNIA

PREPARED FOR:
Geocon Incorporated
6960 Flanders Drive
San Diego, CA 92121

PREPARED BY:
Southwest Geophysics, Inc.
8057 Raytheon Road, Suite 9
San Diego, CA 92111

October 22, 2012
Project No. 112385

SOUTHWEST GEOPHYSICS, INC.

YOUR SUBSURFACE SOLUTION

October 22, 2012
Project No. 112385

Mr. Troy Reist
Geocon Incorporated
6960 Flanders Drive
San Diego, CA 92121

Subject: Seismic Refraction Survey
11111 Rancho Bernardo Road
San Diego, California

Dear Mr. Reist:

In accordance with your authorization, we have performed a seismic refraction survey pertaining to 11111 Rancho Bernardo Road located in San Diego, California. Specifically, our survey consisted of performing four seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. This data report presents our survey methodology, equipment used, analysis, and results.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Sincerely,
SOUTHWEST GEOPHYSICS, INC.

Aaron Puente
Staff Geologist/Geophysicist

Hans van de Vrugt, C.E.G., P.Gp.
Principal Geologist/Geophysicist

REGISTERED GEOPHYSICIST
Hans van de Vrugt
No 1012
Exp. 2/28/14
STATE OF CALIFORNIA

ATP/HV/hv

Distribution: (1) Electronic

8057 Raytheon Road, Suite 9 · San Diego · California 92111 · Telephone 858-527-0849 · Fax 858-225-0114

TABLE OF CONTENTS

1. INTRODUCTION

In accordance with your authorization, we have performed a seismic refraction survey pertaining to 11111 Rancho Bernardo Road located in San Diego, California (Figure 1). Specifically, our survey consisted of performing four seismic refraction traverses at the project site. The purpose of our study was to develop subsurface velocity profiles of the areas surveyed, and to assess the apparent rippability of the subsurface materials. This data report presents our survey methodology, equipment used, analysis, and results.

2. SCOPE OF SERVICES

Our scope of services included:

- Performance of four seismic refraction lines at the project site.
- Compilation and analysis of the data collected.
- Preparation of this data report presenting our results, conclusions and recommendations.

3. SITE DESCRIPTION

The project site is located along the south side of Rancho Bernardo Road, near its intersection with Olmeda Way (Figure 1). The study area included the moderate to steep slopes between a graded pad to the west and industrial/commercial buildings to the east (Figure 2). Vegetation in the area consists of annual grass, relatively dense brush, and scattered trees. Outcrops of metavolcanic rock were observed at and near the project site, and particularly in the road cut along Rancho Bernardo Road. Figures 2 and 3 depict the general site conditions in the study area.

Based on our discussions with you, it is our understanding that the proposed project includes the excavation of a new access road. Cuts up to roughly 50 feet deep are planned.

4. SURVEY METHODOLOGY

A seismic P-wave (compression wave) refraction survey was conducted at the site to evaluate the rippability characteristics of the subsurface materials and to develop subsurface velocity profiles of the areas surveyed. The seismic refraction method uses first-arrival times of refracted seismic waves to estimate the thicknesses and seismic velocities of subsurface layers. Seismic P-waves generated at the surface, using a hammer and plate, are refracted at boundaries separating materials of contrasting velocities. These refracted seismic waves are then detected by a series of surface vertical component geophones and recorded with a 24-channel Geometrics StrataView seismograph. The travel times of the seismic P-waves are used in conjunction with the shot-to-geophone distances to obtain thickness and velocity information on the subsurface materials.

Four seismic lines (SL-1 through SL-4) were conducted in the study area. The general locations and lengths of the lines were selected by your office. Shot points (signal generation locations) were generally conducted at five equally spaced locations along SL-1 and SL-4, and at nine equally spaced locations along SL-2 and SL-3.

The seismic refraction theory requires that subsurface velocities increase with depth. A layer having a velocity lower than that of the layer above will not generally be detectable by the seismic refraction method and, therefore, could lead to errors in the depth calculations of subsequent layers. In addition, lateral variations in velocity, such as those caused by core stones, intrusions or boulders can also result in the misinterpretation of the subsurface conditions.

In general, seismic wave velocities can be correlated to material density and/or rock hardness. The relationship between rippability and seismic velocity is empirical and assumes a homogenous mass. Localized areas of differing composition, texture, and/or structure may affect both the measured data and the actual rippability of the mass. The rippability of a mass is also dependent on the excavation equipment used and the skill and experience of the equipment operator.

The rippability values presented in Table 1 are based on our experience with similar materials and assumes that a Caterpillar D-9 dozer ripping with a single shank is used. We emphasize that the cutoffs in this classification scheme are approximate and that rock characteristics, such as fracture spacing and orientation, play a significant role in determining rock rippability. These characteristics may also vary with location and depth. For trenching operations, the rippability values should be scaled downward. For example, velocities as low as 3,500 feet/second may indicate difficult ripping during trenching operations. In addition, the presence of boulders, which can be troublesome in a narrow trench, should be anticipated.

Table 1 – Rippability Classification	
Seismic P-wave Velocity	**Rippability**
0 to 2,000 feet/second	Easy
2,000 to 4,000 feet/second	Moderate
4,000 to 5,500 feet/second	Difficult, Possible Blasting
5,500 to 7,000 feet/second	Very Difficult, Probable Blasting
Greater than 7,000 feet/second	Blasting Generally Required

It should be noted that the rippability cutoffs presented in Table 1 are slightly more conservative than those published in the Caterpillar Performance Handbook (Caterpillar, 2011). Accordingly, the above classification scheme should be used with discretion, and contractors should not be relieved of making their own independent evaluation of the rippability of the on-site materials prior to submitting their bids.

5. RESULTS

As previously indicated, four seismic traverses were conducted as part of our study. The collected data were processed using SIPwin (Rimrock Geophysics, 2003), a seismic interpretation program, and analyzed using both SIPwin and SeisOpt Pro (Optim, 2008). Both programs use first arrival picks and elevation data to produce subsurface velocity models. SIPwin uses layer-based modeling techniques to produce a layered velocity model, where changes in velocities are depicted as discrete contacts. SeisOpt Pro uses a nonlinear optimization technique called adaptive simulated annealing. The resulting velocity model provides a tomography image of the estimated geologic conditions. Both vertical and lateral velocity information is contained in the tomography model. Changes in layer velocity are revealed as gradients rather than discrete contacts, which typically are more representative of actual conditions.

Table 2 lists the approximate P-wave velocities and depths calculated from the seismic refraction traverse using the layered modeling method. The approximate locations of the seismic refraction traverses are shown on the Line Location Map (Figures 2). The velocity models are included in

Figures 4a through 4d. In general, the effective depth of evaluation for a seismic refraction traverse is approximately one-third to one-fifth the length of the traverse.

Table 2 – Seismic Traverse Results[1]

Traverse No. And Length	P-wave Velocity feet/second	Approximate Depth to Bottom of Layer in feet	Apparent Rippability[2]
SL-1 125 feet	V1 = 1,790 V2 = 3,530 V3 = 8,140	1 – 6 12 – 23 ---	Easy Moderate Blasting Generally Required
SL-2 230 feet	V1 = 1,730 V2 = 4,180 V3 = 9,780	1 – 7 17 – 23 ---	Easy Difficult, Possible Blasting Blasting Generally Required
SL-3 230 feet	V1 = 1,520 V2 = 4,710 V3 = 8,050	3 – 7 10 – 26 ---	Easy Difficult, Possible Blasting Blasting Generally Required
SL-4 125 feet	V1 = 1,620 V2 = 6,200	2 – 5 ---	Easy Very Difficult, Probable Blasting

1 Results based on the model generated using SIPwin, 2003
2 Rippability criteria based on the use of a Caterpillar D-9 dozer ripping with a single shank

6. CONCLUSIONS AND RECOMMENDATIONS

The results from our seismic survey revealed distinct layers/zones in the near surface that likely represent soil (fill and colluvium) overlying metavolcanic bedrock with varying degrees of weathering. Figures 4a through 4d provide the velocity models calculated from both SIPwin and SeisOpt Pro. Distinct vertical and lateral variations between the two models are evident. In general, the tomography results better characterize the onsite conditions than the layer models.

The cause of the velocity variations revealed in the data are likely related to the presence of remnant boulders, intrusions and differential weathering of the bedrock materials. In addition, the tomography models revealed pockets/zones of very high velocity material within a matrix of slower velocity material. Therefore, variability in the excavatability (including depth of rippability) of the subsurface materials should be expected across the project area.

Based on our results, very difficult conditions where blasting may be required will likely be encountered depending on the excavation depth, location, and desired rate of production. In addition, oversized materials should be expected. A contractor with excavation experience in similar difficult conditions should be consulted for expert advice on excavation methodology, equipment and production rate.

7. LIMITATIONS

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface surveying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics, Inc. should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

8. SELECTED REFERENCES

Caterpillar, Inc., 2011, Caterpillar Performance Handbook, Edition 41, Caterpillar, Inc., Peoria, Illinois.

Mooney, H.M., 1976, Handbook of Engineering Geophysics, dated February.

Optim, Inc., 2008, SeisOpt Pro, V-5.0.

Rimrock Geophysics, 2003, Seismic Refraction Interpretation Program (SIPwin), V-2.76.

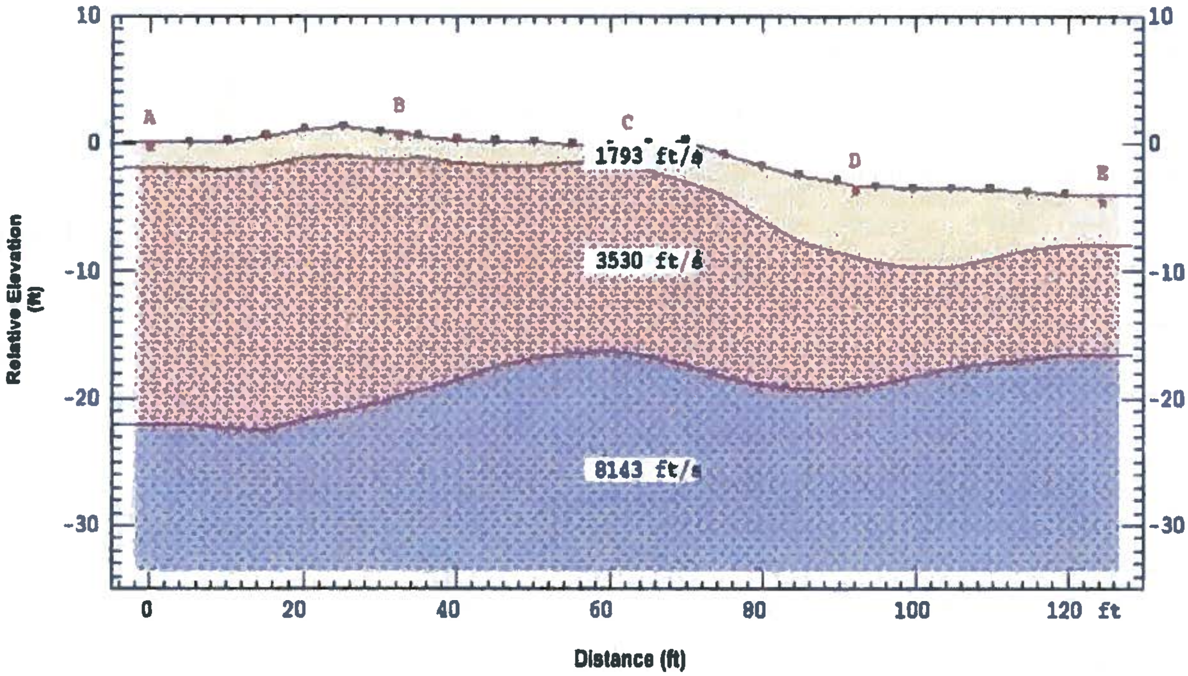
Telford, W.M., Geldart, L.P., Sheriff, R.E., and Keys, D.A., 1976, Applied Geophysics, Cambridge University Press.

SITE LOCATION MAP	N	11111 Rancho Bernardo Road San Diego, California	SOUTHWEST GEOPHYSICS INC
		Project No.: 112385 — Date: 10/12	Figure 1

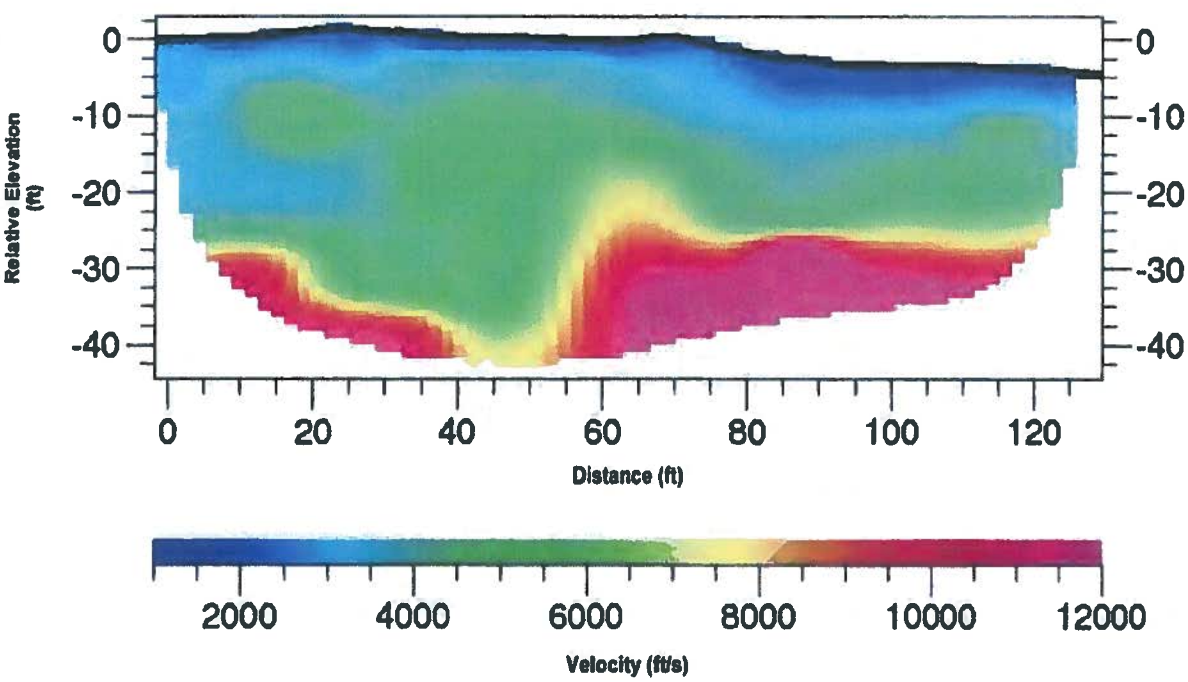
LINE LOCATION MAP	N	11111 Rancho Bernardo Road San Diego, California Project No: 112385 / Date: 10/12	SOUTHWEST GEOPHYSICS INC. Figure 2	0 100 200 approximate scale in feet

SITE PHOTOGRAPHS	11111 Rancho Bernardo Road San Diego, California Project No.: 112385 Date: 10/12	SOUTHWEST GEOPHYSICS INC Figure 3

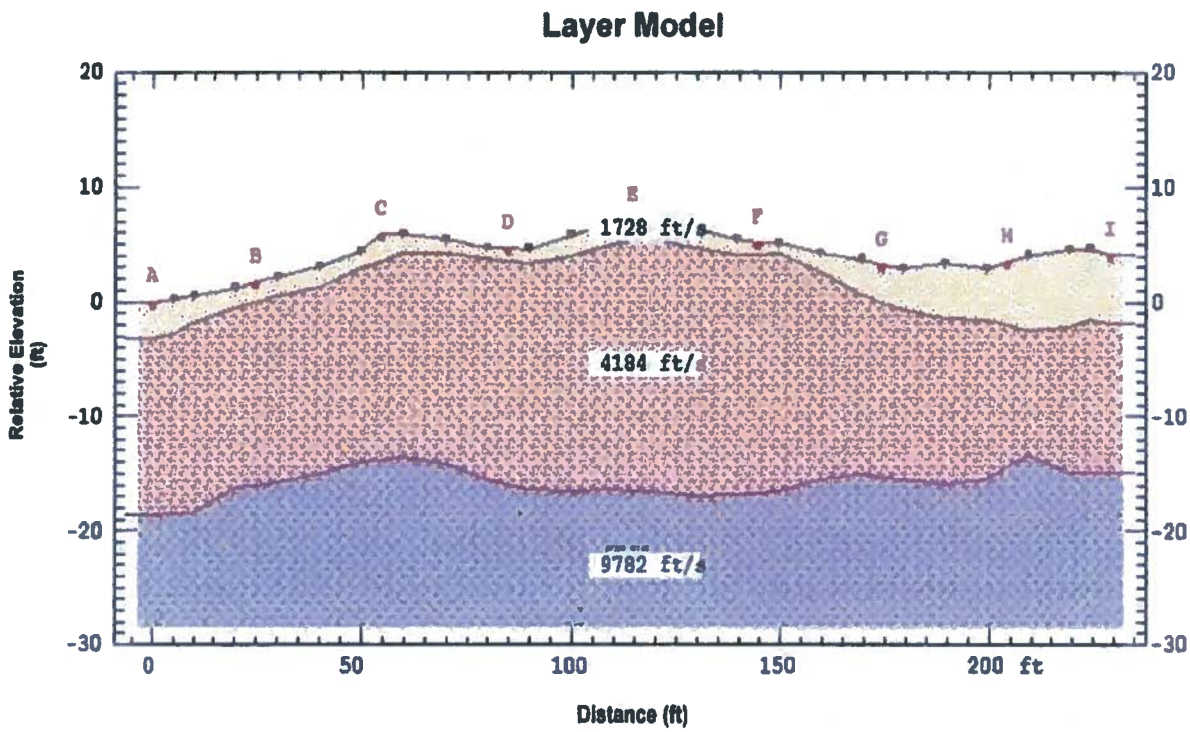
Layer Model

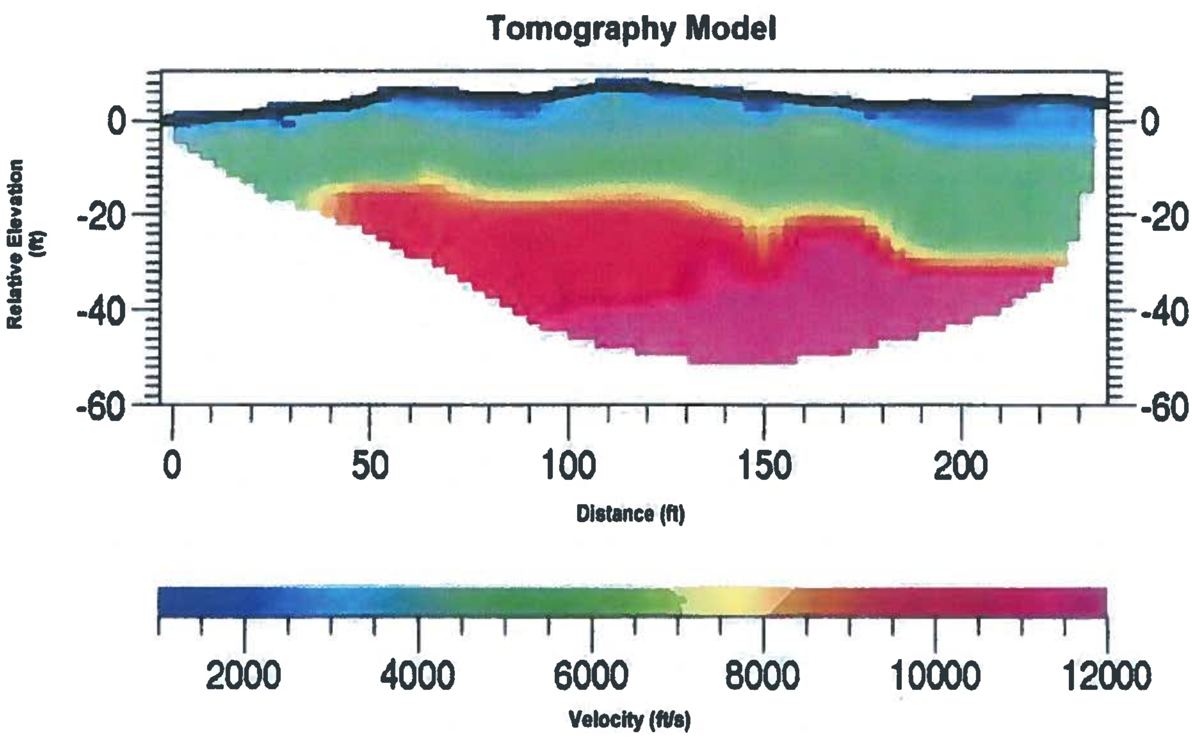


Tomography Model

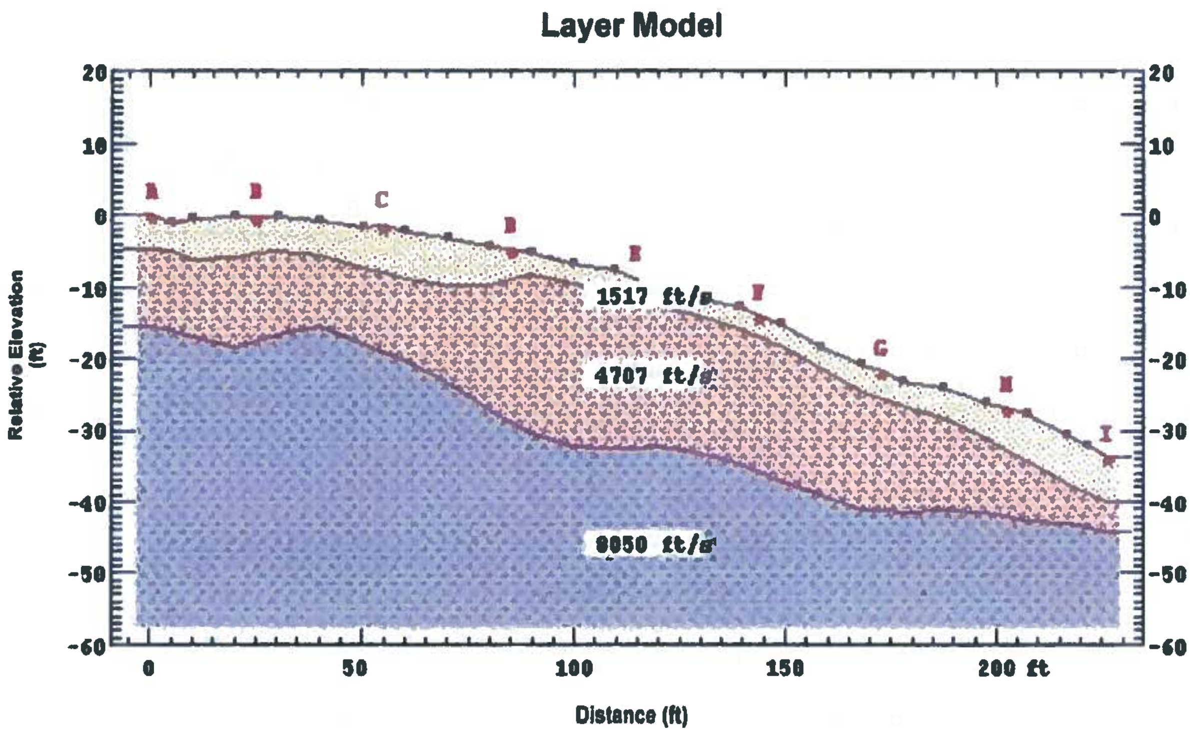


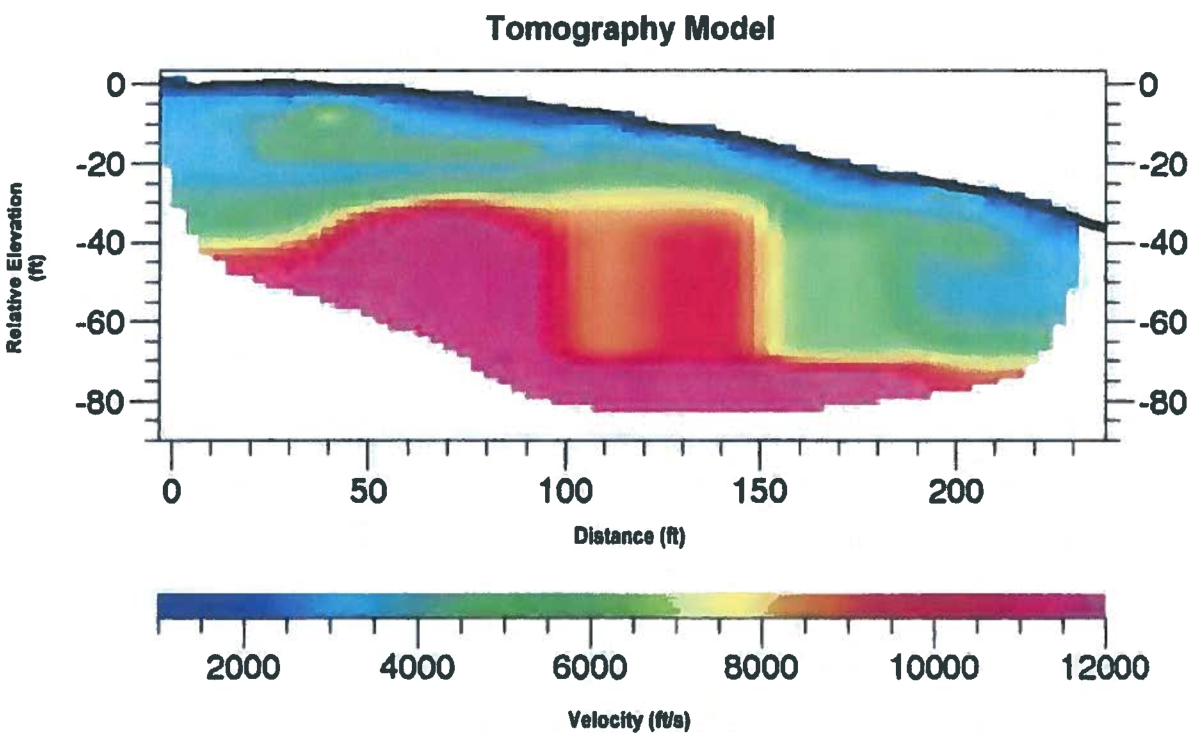
SEISMIC PROFILE SL-1	11111 Rancho Bernardo Road San Diego, California	SOUTHWEST GEOPHYSICS INC Figure 4a
	Project No.: 112385 / Date: 10/12	





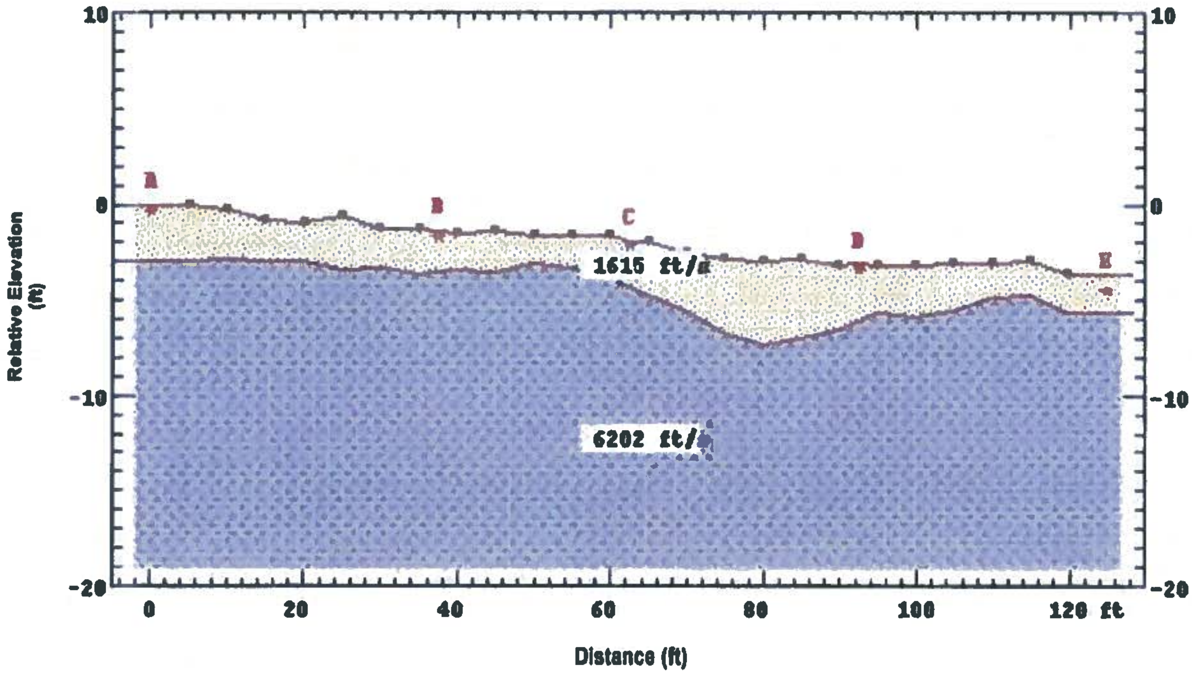
SEISMIC PROFILE SL-2	11111 Rancho Bernardo Road San Diego, California	SOUTHWEST GEOPHYSICS INC Figure 4b
	Project No.: 112385 Date: 10/12	



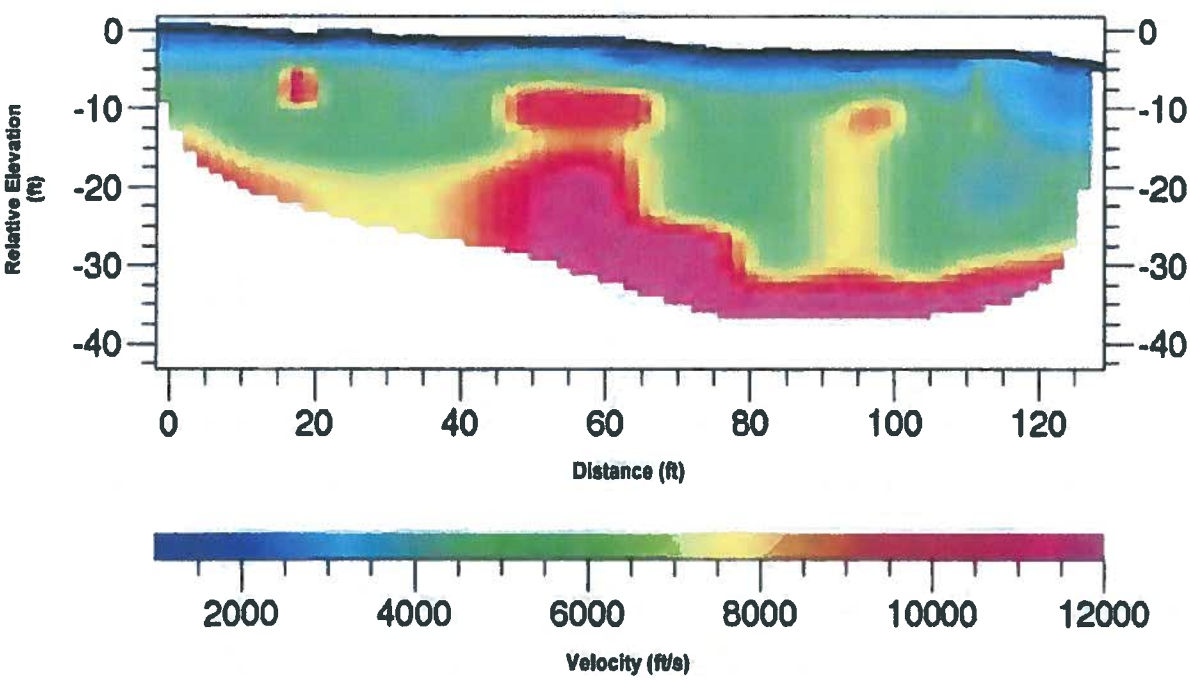


SEISMIC PROFILE SL-3	11111 Rancho Bernardo Road San Diego, California	SOUTHWEST GEOPHYSICS INC Figure 4c
	Project No.: 112385 — Date. 10/12	

Layer Model



Tomography Model



SEISMIC PROFILE SL-4	11111 Rancho Bernardo Road San Diego, California Project No. 112385 — Date. 10/12	SOUTHWEST GEOPHYSICS INC Figure 4d

APPENDIX D

APPENDIX D

SLOPE STABILITY ANALYSES

FOR

PALOMAR COLLEGE
SOUTH EDUCATION CENTER
IMPROVEMENT PROJECT
SAN DIEGO, CALIFORNIA

PROJECT NO. 06647-42-03

Palomar College South Ed Center
Project No. 06647-42-03
Cross-Section A-A'
Name: AA-Case1.gsz
Date: 10/16/2012 Time: 11:58:03 AM

Name: Qpf Unit Weight: 130 pcf Cohesion: 200 psf Phi: 30 °
Name: Jsp Unit Weight: 135 pcf Cohesion: 1000 psf Phi: 40 °

Proposed 70 ft. high, 2:1 (H:V) Cut Slope

Static Condition

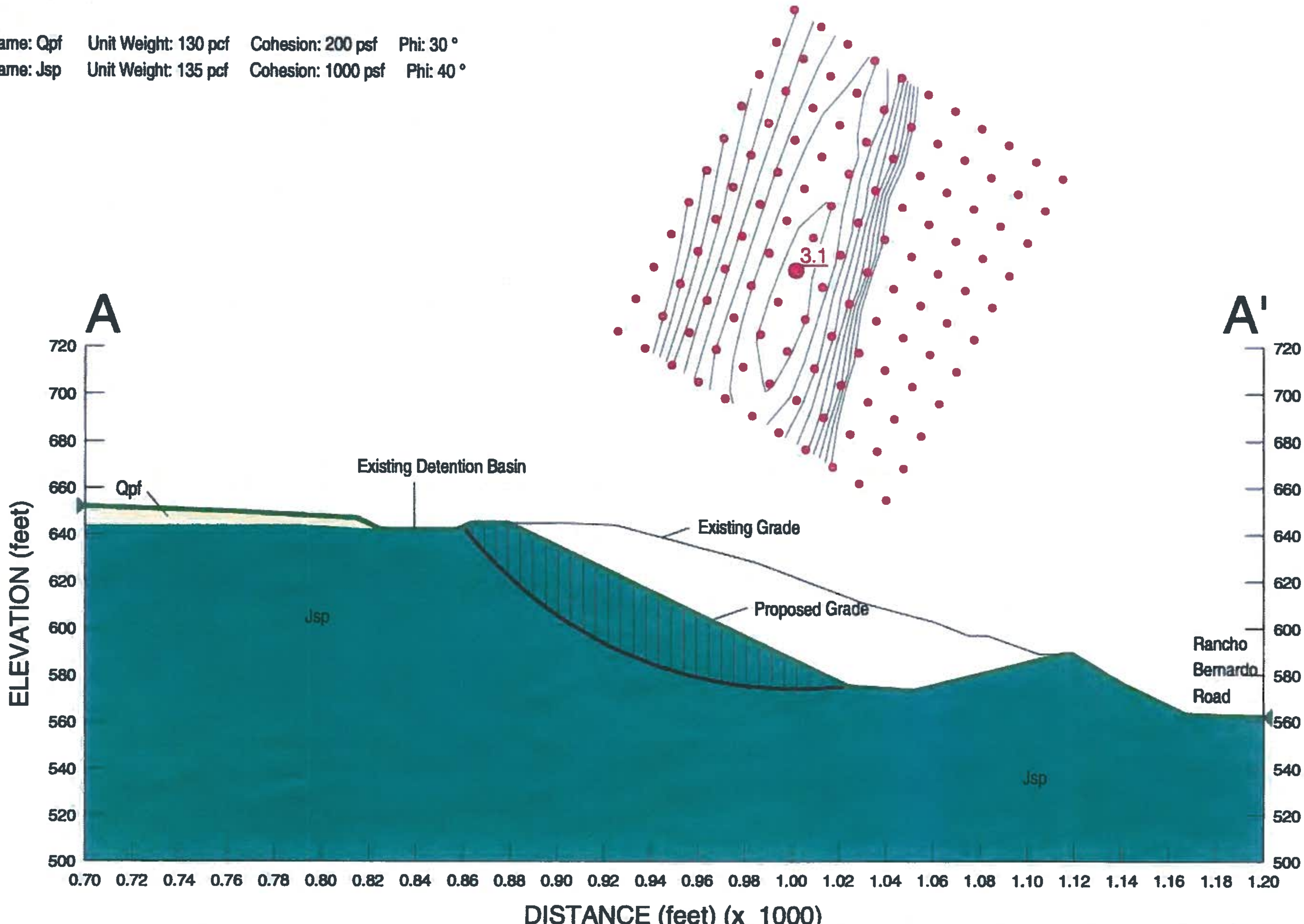
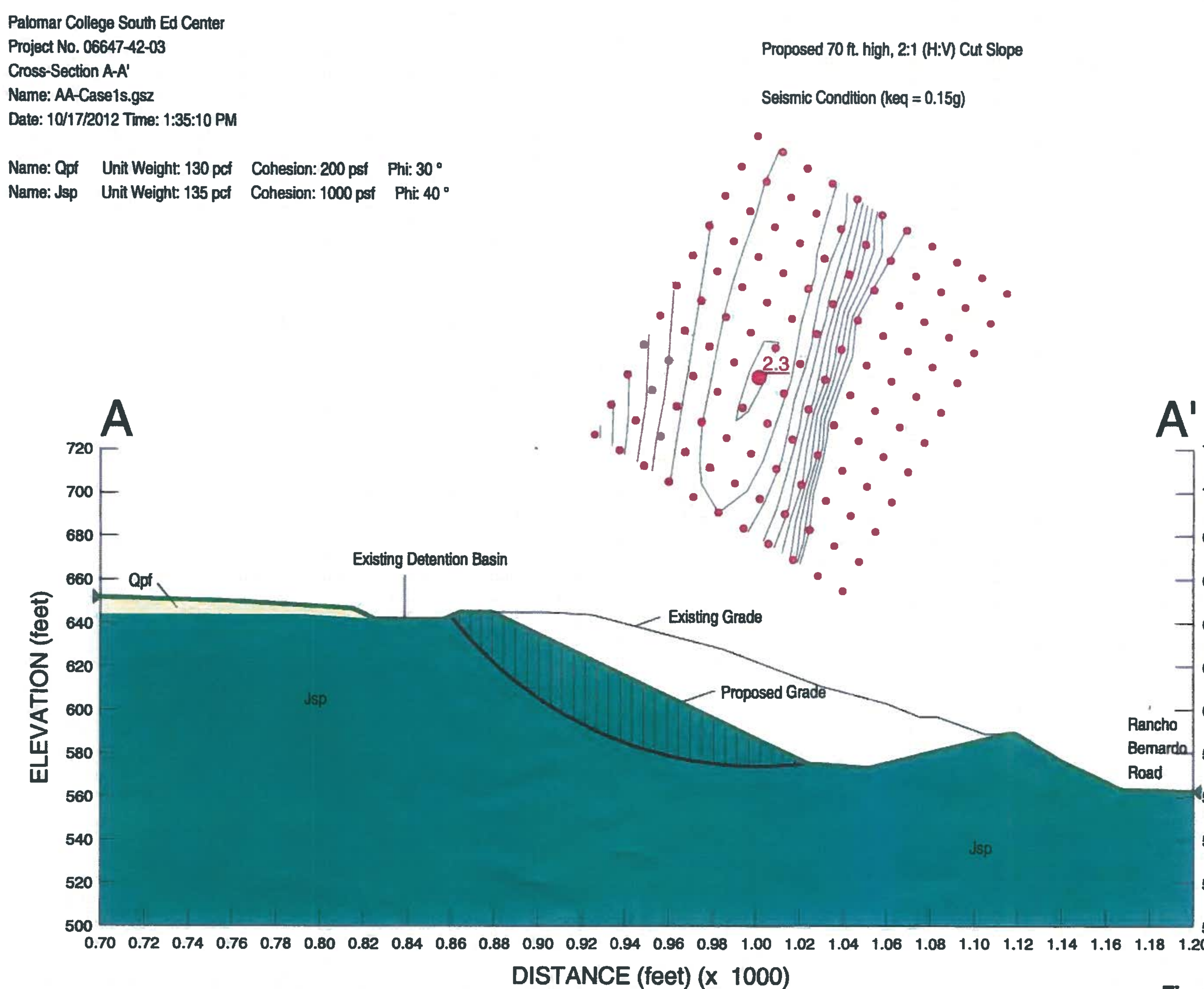
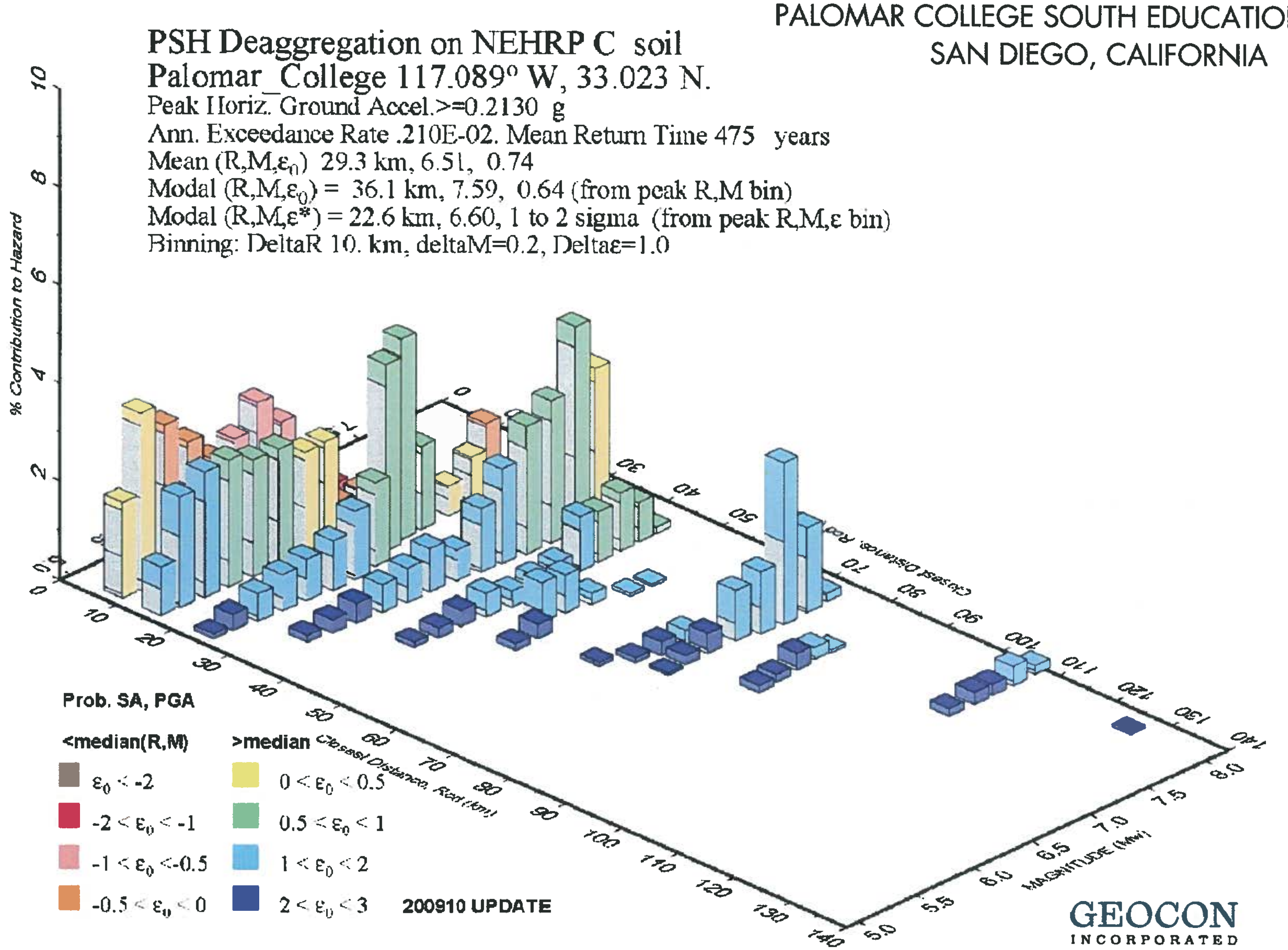


Figure D-1


Palomar College South Ed Center
Project No. 06647-42-03
Cross-Section A-A'
Name: AA-Case1s.gsz
Date: 10/17/2012 Time: 1:35:10 PM
Name: Qpf Unit Weight: 130 pcf Cohesion: 200 psf Phi: 30 °
Name: Jsp Unit Weight: 135 pcf Cohesion: 1000 psf Phi: 40 °
Proposed 70 ft. high, 2:1 (H:V) Cut Slope
Seismic Condition (keq = 0.15g)
2.3
A
A'
Qpf
Existing Detention Basin
Existing Grade
Proposed Grade
Jsp
Jsp
Rancho
Bernardo
Road
ELEVATION (feet)
720
700
680
660
640
620
600
580
560
540
520
500
DISTANCE (feet) (x 1000)
0.70 0.72 0.74 0.76 0.78 0.80 0.82 0.84 0.86 0.88 0.90 0.92 0.94 0.96 0.98 1.00 1.02 1.04 1.06 1.08 1.10 1.12 1.14 1.16 1.18 1.20
Figure D-2

PALOMAR COLLEGE SOUTH EDUCATION CENTER
SAN DIEGO, CALIFORNIA



GEOCON
INCORPORATED

GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS
6960 FLANDERS DRIVE - SAN DIEGO, CALIFORNIA 92121 - 2974
PHONE 858 558-6900 - FAX 858 558-6159

PROJECT NO. 06647 - 42 - 03
FIGURE D-3
DATE 10 - 24 - 2012

Y:\PROJECTS\06647-42-03 PALOMAR COLLEGE SOUTH EDUCATION CENTER\DETAILS\06647-42-03 DSH.dwg, 10/22/2012 2:15:45 PM, Ladrillono A

APPENDIX E

APPENDIX E

RECOMMENDED GRADING SPECIFICATIONS

FOR

PALOMAR COLLEGE
SOUTH EDUCATION CENTER
IMPROVEMENT PROJECT
SAN DIEGO, CALIFORNIA

PROJECT NO. 06647-42-03

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon Incorporated. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, adverse weather, result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

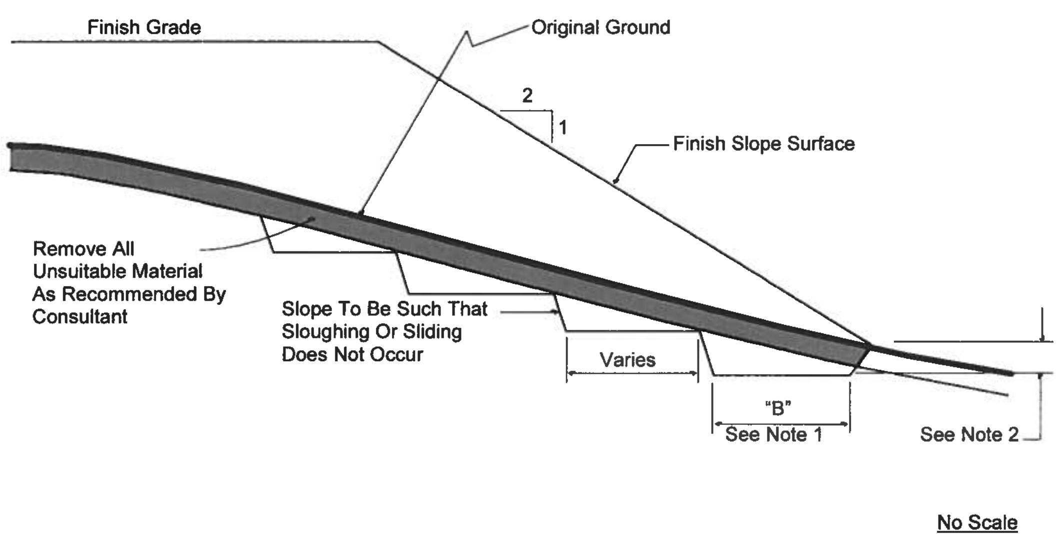
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Any asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557-02.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557-02. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196-93, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of "passes" have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. OBSERVATION AND TESTING

7.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

7.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

7.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

7.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

7.5 The Consultant should observe the placement of subdrains, to verify that the drainage devices have been placed and constructed in substantial conformance with project specifications.

7.6 Testing procedures shall conform to the following Standards as appropriate:

7.6.1 Soil and Soil-Rock Fills:

7.6.1.1 Field Density Test, ASTM D 1556-02, *Density of Soil In-Place By the Sand-Cone Method.*

7.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938-08A, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth).*

7.6.1.3 Laboratory Compaction Test, ASTM D 1557-02, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop.*

7.6.1.4. Expansion Index Test, ASTM D 4829-03, *Expansion Index Test.*

7.6.2 Rock Fills

7.6.2.1 Field Plate Bearing Test, ASTM D 1196-93 (Reapproved 1997) *Standard Method for Nonreparative Static Plate Load Tests of Soils and Flexible Pavement Components, For Use in Evaluation and Design of Airport and Highway Pavements.*

8. PROTECTION OF WORK

8.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

8.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

9. CERTIFICATIONS AND FINAL REPORTS

9.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

9.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.

LIST OF REFERENCES

1. Anderson, J. G., *Synthesis of Seismicity and Geologic Data in California,* U. S. Geologic Survey Open-File Report 84-424, 1984, pp. 1-186.

2. Boore, D. M., and G. M Atkinson (2008), *Ground-Motion Prediction for the Average Horizontal Component of PGA, PGV, and 5%-Damped PSA at Spectral Periods Between 0.01 and 10.0 S*, Earthquake Spectra, Volume 24, Issue 1, pages 99-138, February 2008.

3. California Department of Conservation, Division of Mines and Geology, *Probabilistic Seismic Hazard Assessment for the State of California,* Open File Report 96-08, 1996.

4. California Department of Water Resources, Water Data Library. http://www.water.ca.gov/waterdatalibrary.

5. California Geological Survey, *Seismic Shaking Hazards in California,* Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA) Model, 2002 (revised April 2003). 10% probability of being exceeded in 50 years. http://redirect.conservation.ca.gov/cgs/rghm/pshamap/pshamain.html

6. Campbell, K. W. and Y. Bozorgnia, *NGA Ground Motion Model for the Geometric Mean Horizontal Component of PGA, PGV, PGD and 5% Damped Linear Elastic Response Spectra for Periods Ranging from 0.01 to 10 s*, Preprint of version submitted for publication in the NGA Special Volume of Earthquake Spectra, Volume 24, Issue 1, pages 139-171, February 2008.

7. Chiou, Brian S. J. and Robert R. Youngs, *A NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra*, preprint for article to be published in NGA Special Edition for Earthquake Spectra, Spring 2008.

8. Jennings, C. W., 1994, California Division of Mines and Geology, *Fault Activity Map of California and Adjacent Areas,* California Geologic Data Map Series Map No. 6.

9. Kennedy, M. P. and S. S. Tan, *Geologic Map of the Oceanside 30'x60' Quadrangle, California*, USGS Regional Map Series, Scale 1:100,000, 2005.

10. Risk Engineering, *EZ-FRISK*, 2011.

11. United States Department of Agriculture, *1953 Stereoscopic Aerial Photographs, Flight AXN-14M,* Photos Nos. 82 and 83 (scale 1:20,000).

12. Unpublished reports and maps on file with Geocon Incorporated.

13. USGS computer program, *Seismic Hazard Curves and Uniform Hazard Response Spectra.*

14. Woodward-Clyde, *Update Geotechnical Investigation Bernardo Industrial Park North, Lot 11, San Diego, California,* dated February 27, 1997.

15. Woodward-Clyde, *Update Geotechnical Investigation, Bernardo Industrial Park North, Lot 11, San Diego, California,* dated June 6, 1997.