SOILS ENGINEERING REPORT
CANTINAS RANCH, LYNCH CANYON ROAD
APN: 080-062-038
LAKE NACIMIENTO AREA
SAN LUIS OBISPO COUNTY, CALIFORNIA

PROJECT SL07503-1

Prepared for

Camp Nacimiento Foundation
Attn: B. Wayne Hughes, Jr.
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October 13, 2011 (revised)

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(Revised) October 13, 2011
Project No. SL07503-1

Camp Nacimiento Foundation
Attn: B. Wayne Hughes, Jr.
22917 Pacific Coast Highway, Suite 300A
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Subject: **Soils Engineering Report**
Cantinas Ranch, Lynch Canyon Road, APN: 080-062-038
Lake Nacimiento Area, San Luis Obispo County, California

Dear Mr. Hughes,

This Soils Engineering Report has been prepared for proposed development to be located at Cantinas Ranch off Lynch Canyon Road, APN: 080-062-038, in the Lake Nacimiento area of San Luis Obispo County, California. Geotechnically, the site is suitable for the proposed development provided the recommendations in this report for site preparation, earthwork, foundations, slabs, retaining walls, and pavement sections are incorporated into the design.

It is anticipated that graded engineered fill pads will be constructed for the security main entry area with foundations excavated into engineered fill. It is anticipated that pier foundations will be constructed for the staff lodging area. Due to the shallow depth to competent formational material encountered during the field investigation it is anticipated that foundations for the structures proposed in the camper lodging, chapel, celebration arts campus, mill barn, lake shack and equestrian barns will be excavated into uniform competent formational material. As an alternative, graded engineered fill pads may be constructed in all proposed building areas. Recommendations for building pad preparation for both options are presented below.

All foundations are to be excavated into uniform material to limit the potential for distress of the foundation systems due to differential settlement. If cuts steeper than allowed by State of California Construction Safety Orders for "Excavations, Trenches, Earthwork" are proposed, a numerical slope stability analysis may be necessary for temporary construction slopes.

Natural seepage at the interface of two materials with different densities, such as native soil and formational material, is very common. This interface occurs at the Site and may require sub-surface drains. Sub-drains should be placed in established drainage courses, potential seepage areas, and during the development of all key and bench grading operations.

Thank you for the opportunity to have been of service in preparing this report. If you have any questions or require additional assistance, please feel free to contact the undersigned at (805) 543-8539.

Sincerely,
GeoSolutions, Inc.

REGISTERED PROFESSIONAL ENGINEER
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SOILS ENGINEERING REPORT
CANTINAS RANCH, LYNCH CANYON ROAD, APN: 080-062-038
LAKE NACIMIENTO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA

PROJECT SL07503-1

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation for the proposed Cantinas Ranch development project to be located off Lynch Canyon Road, APN: 080-062-038, in the Lake Nacimiento area of San Luis Obispo County, California. See Figure 1: Site Location Map for the general location of the project area. Figure 1: Site Location Map was obtained from the computer program *Topo USA 6.0* (DeLorme, 2006).

Cantinas Ranch is located at approximately 35.757613 degrees north latitude and approximately -121.011508 degrees west longitude at elevations ranging from 800 to 1140 feet above mean sea level. The property is irregular in shape and is approximately 560 acres in size. Lynch Canyon Road forms the northern property boundary of the development and Lake Nacimiento forms the southern boundary. The project property will hereafter be referred to as the "Site."

Figure 1: Site Location Map

Cantinas Ranch is a proposed organizational camp consisting of the following areas: security main entry, camper lodging, chapel, mission, celebration arts campus, staff lodging, mill barn, lake shack, equestrian barns, and associated roadways. The property is situated on a number of hillside slopes that vary from 3:1 to 12:1 (horizontal:vertical). Surface drainage flows approximately south toward the main body of Lake Nacimiento and north toward a finger extending off the main body of the lake. Native grasses, shrubs and oak trees currently vegetate the Site. An abandoned building is located at the proposed lake shack site.

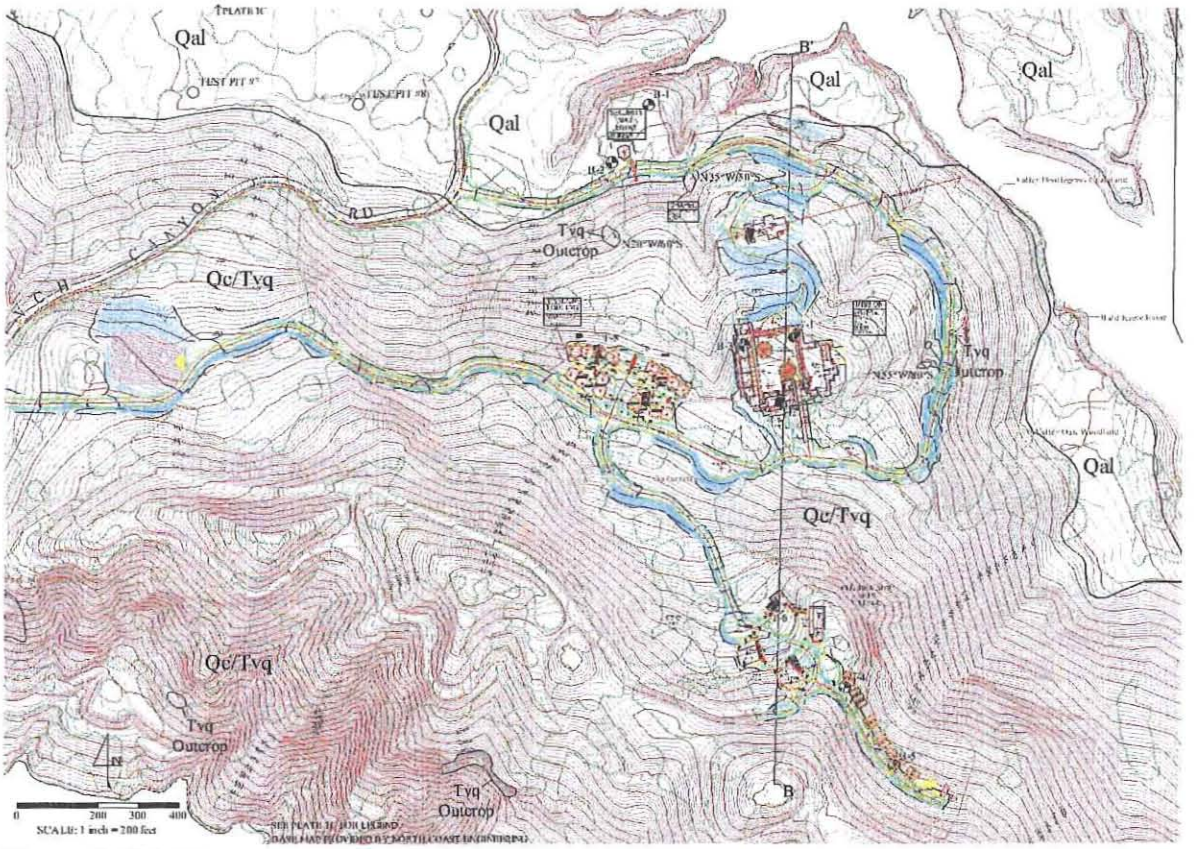

Figure 2: Site Plan (Eastern Portion)

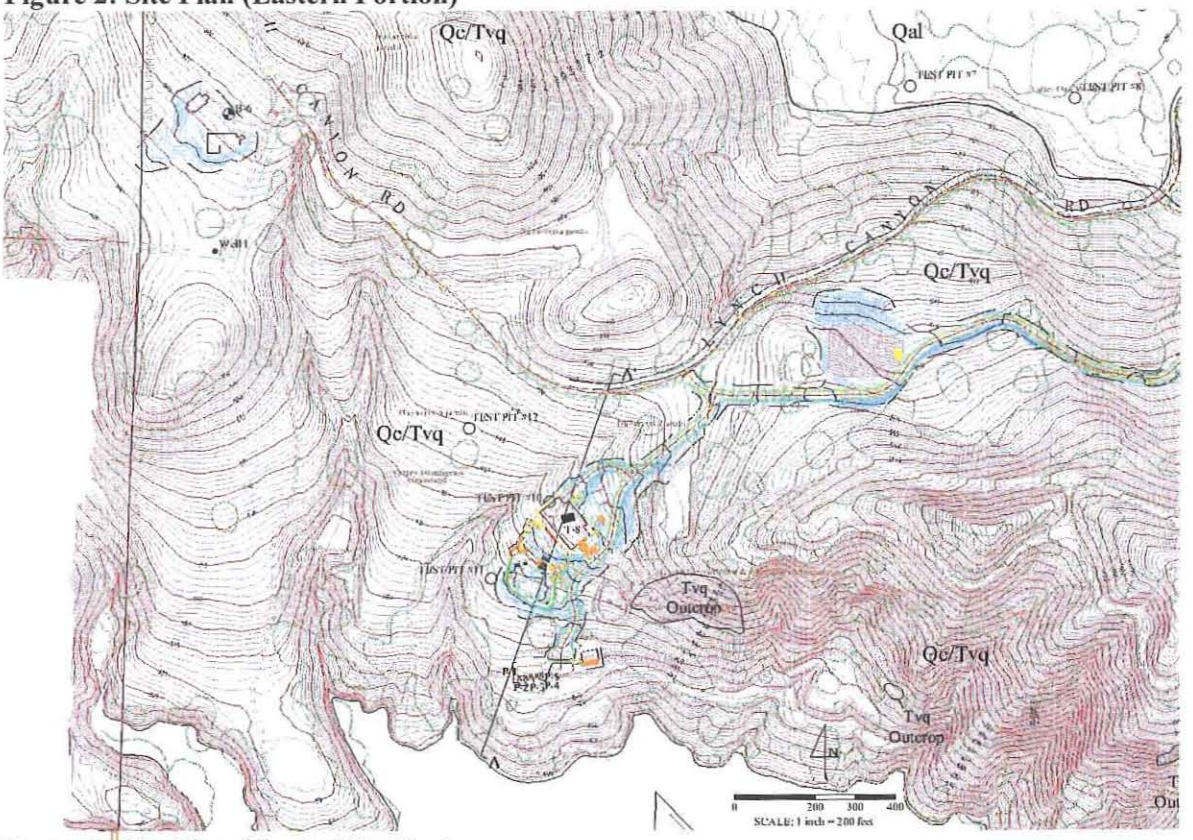

Figure 3: Site Plan (Central Portion)

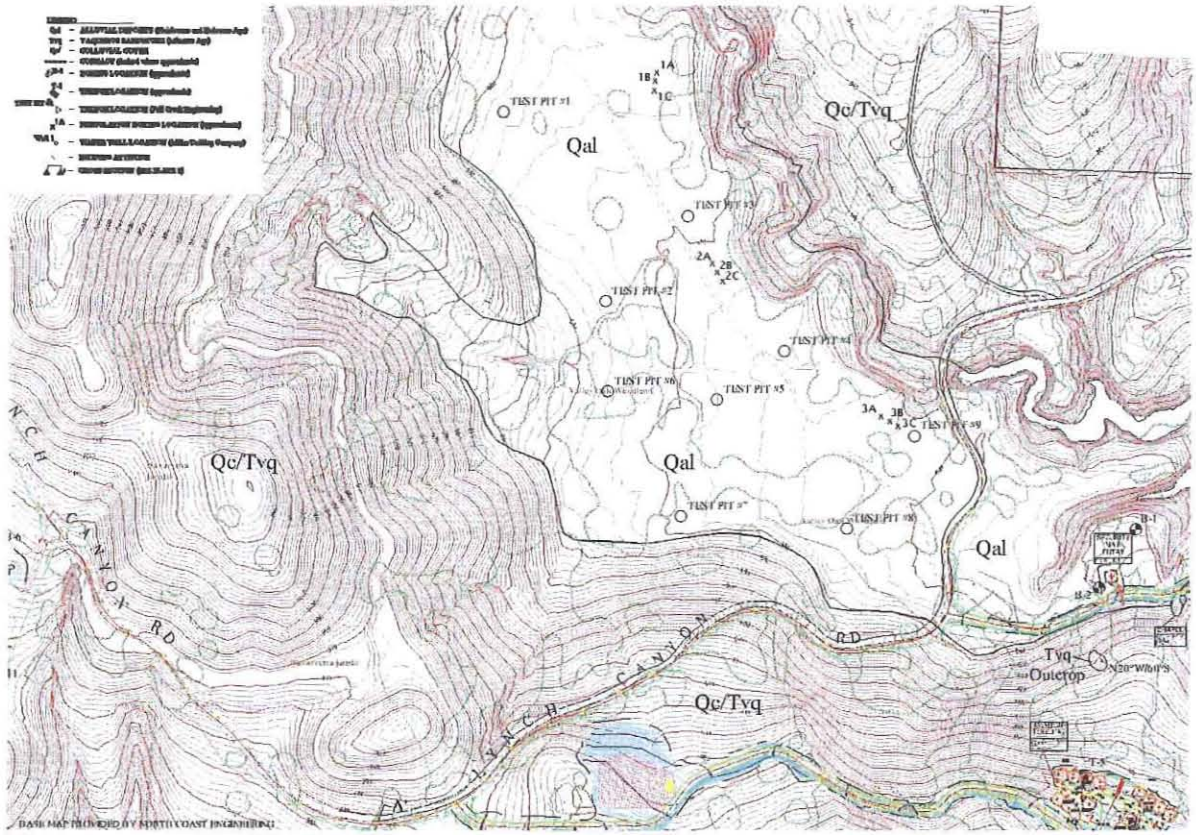

Figure 4: Site Plan (Northern Portion)

1.1 GENERAL DESCRIPTION OF PROJECT AREAS

Security Main Entry

The proposed security main entry building is to be located in the northeast portion of the property. The structure is anticipated to be two stories in height, approximately 1,280 square feet in size, and will be situated on a 9:1 (horizontal:vertical) slope gradient. It is anticipated that the proposed structure will utilize a slab-on-grade floor system.

Camper Lodging

The proposed camper lodging buildings are to be located in t he eastern portion of the property. The camper lodging site is anticipated to consist of 16 two story units situated on a 4:1 to 6:1 (horizontal:vertical) slope gradient. It is anticipated that the proposed structures will utilize a slab-on-grade system.

Chapel

The proposed chapel building is to be located in the eastern portion of the property. The structure is anticipated to be one story in height with a mezzanine and situated on an 8:1 (horizontal:vertical) slope gradient. It is anticipated that the proposed building will utilize a slab-on-grade floor system.

Mission

The proposed mission area is to be located in the eastern portion of the property. The area is anticipated to include development of a dining hall, kitchen, studios and nurse's office and a courtyard. The proposed construction area is situated on a 5:1 to 12:1 (horizontal: vertical) slope gradient. It is anticipated that the proposed buildings will utilize slab-on-grade floor systems.

Celebration Arts Campus

The proposed celebration arts campus area is to be located in the southeastern portion of the property. The area is anticipated to include development of four structures including a dance studio, theater, music building, and "black box" theater and is situated on a 7:1 (horizontal:vertical) slope gradient. It is anticipated that the proposed buildings will utilize slab-on-grade floor systems.

Staff Lodging

The proposed staff lodging area is to be located in the southeastern portion of the property. The staff lodging site is anticipated to consist of 20 one-story studio units situated on a 2:1 to 3:1 (horizontal: vertical) slope gradient. It is anticipated that the proposed building will utilize steel framed pier foundation systems.

Mill Barn

The proposed mill barn building is to be located in the central portion of the property. The structure is anticipated to be one story in height, approximately 8,000 square feet, and situated on a 7:1 (horizontal: vertical) slope gradient. It is anticipated that the proposed building will utilize a slab-on-grade floor system.

Lake Shack

The proposed lake shack building is to be located in the central portion of the property. The scope of work in this area is anticipated to consist of demolition and reconstruction of the existing one-story building currently situated on a 7:1 (horizontal: vertical) slope gradient. It is anticipated that the proposed building will utilize a slab-on-grade.

Equestrian Barns

The proposed equestrian barns are to be located in the western portion of the property. The equestrian barn site is anticipated to consist of tool and feed barn structures situated on a 10:1 (horizontal: vertical) slope gradient. It is anticipated that the proposed buildings will utilize slab-on-grade floor systems where necessary.

Dead and sustained live loads for the proposed structures are currently unknown, but they are anticipated to be relatively light with maximum continuous footing and column loads estimated to be approximately 1.5 kips per linear foot and 15 kips, respectively.

2.0 PURPOSE AND SCOPE

The purpose of this study was to explore and evaluate the surface and sub-surface soil conditions at the Site and to develop geotechnical information and design criteria. The scope of this study includes the following items:

1. A literature review of available published and unpublished geotechnical data pertinent to the project site.

2. A field study consisting of site reconnaissance and exploratory borings in order to formulate a description of the sub-surface conditions at the Site.

3. Laboratory testing performed on representative soil samples that were collected during our field study.

4. Engineering analysis of the data gathered during our literature review, field study, and laboratory testing.

5. Development of recommendations for site preparation and grading as well as geotechnical design criteria for building foundations, retaining walls, pavement sections, underground utilities, and drainage facilities.

3.0 FIELD AND LABORATORY INVESTIGATION

The field investigation was conducted on multiple dates between October 8 through October 20, 2010 using a track-mounted CME 55 drill rig and backhoe. Six four-inch diameter exploratory borings were advanced to a maximum depth of 25 feet below ground surface (bgs) and nine exploratory trenches were excavated to a maximum depth of 10.5 feet bgs at the approximate locations indicated on Figures 2, 3 and 4. For detailed maps, see Plates 1A, 1B, and 1C from the referenced Engineering Geology Investigation in the appendix. Sampling methods included the Standard Penetration Test utilizing a standard split-spoon sampler (SPT) without liners and a Modified California sampler (CA) with liners. The CME 55 drill rig was equipped with an automatic hammer, which has an efficiency of approximately 80 percent and was used to obtain test blow counts in the form of N-values.

Data gathered during the field investigation suggest that the soil materials at the Site consist of colluvial soil overlying competent formational material with the exception of the security main entry area which consists of alluvial soil. The surface materials at the Site generally consisted of SAND-SILT and SAND-CLAY materials encountered in dry to slightly moist and dense to very dense conditions varying in depth from approximately 1.0 to 13.0 feet bgs. The surface materials are underlain by white to tan SANDSTONE and SILTSTONE encountered in a dry and very hard condition. Using the *Geologic Map of the Bryson Quadrangle* (Dibblee, 2006), the SANDSTONE and SILTSTONE material was interpreted as Vaqueros Sandstone and will hereafter be referred to as competent formational material. Groundwater was not encountered in any of the borings or trenches.

During the boring operations the soils encountered were continuously examined, visually classified, and sampled for general laboratory testing. A project engineer has reviewed a continuous log of the soils encountered at the time of field investigation. See **Appendix A** for the Boring/Trench Logs from the field investigation.

Laboratory tests were performed on soil samples that were obtained from the Site during the field investigation. The results of these tests are listed below in Table 1: Engineering Properties. Laboratory data reports and detailed explanations of the laboratory tests performed during this investigation are provided in **Appendix B**.

In general, the soils encountered during the field investigation were categorized as having either very low, low, or medium expansion potential. This report provides foundation design recommendations for each of these three categories. Building pad areas proposed for construction on soil with **very low expansion potential** include **the chapel and security main entry.** Building pad areas proposed for construction on

soil with **low expansion potential** include **the mission, celebration arts campus, staff lodging, mill barn, lake shack, and equestrian barn.** Building pad areas proposed for construction on soil with **medium expansion potential** include **camper lodging** only.

Table 1: Engineering Properties

Sample Name	Sample Description	USCS Specification	Expansion Index	Expansion Potential	Maximum Dry Density, γ_d (pcf)	Optimum Moisture (%)	Angle of Internal Friction, ϕ (deg.)	Cohesion, c (psf)
A	Dark Olive Brown Clayey SAND	SC	19	Very Low	-	-	-	-
C	Dark Olive Brown Clayey SAND	SC	21	Low	121.8	9.7	-	-
D	Light Olive Brown Clayey SAND	SC	21	Low	-	-	-	-
E	Dark Olive Brown Sandy CLAY	CL	-	-	105.1	13.8	-	-
G	Dark Brown Clayey SAND	SC	31	Low	116.2	10.9	-	-
I	Dark Olive Brown Clayey SAND	SC	1	Very Low	118.6	11.1	-	-
K	Dark Yellowish Brown Clayey SAND	SC	17	Very Low	115.0	11.2	-	-
L	Yellowish Brown Sandy CLAY	CL	58	Medium	115.7	13.5	-	-
M	Dark Yellowish Brown Silty SAND	SM	0	Very Low	123.7	9.1	-	-
B-3 @ 4.0 Ft.	Light Olive Brown Clayey SAND with Gravel	SC	-	-	-	-	26.6	0
B-3 @ 14.0 Ft.	Dark Yellowish Brown Clayey SAND	SC	-	-	-	-	55.0	0
T-6 @ 6.0 Ft.	Brown Silty SAND	SM	-	-	-	-	39.9	64

Sample Name	Sample Description	USCS Specification	Expansion Index	Expansion Potential	Maximum Dry Density, γ_d (pcf)	Optimum Moisture (%)	Angle of Internal Friction, ϕ (deg.)	Cohesion, c (psf)
T-8 @ 1.0 Ft.	Very Dark Grayish Brown Clayey SAND	SC	-	-	-	-	22.0	561
T-10 @ 1.0 Ft.	Brown Silty SAND	SM	-	-	-	-	33.2	90
T-12 @ 1.0 Ft.	Dark Yellowish Brown Clayey SAND	SC	-	-	-	-	26.2	283
T-8 @ 1.0 Ft.	Very Dark Grayish Brown Clayey SAND	SC	-	-	-	-	22.0	561

4.0 SEISMIC DESIGN CONSIDERATIONS

4.1 Seismic Hazard Analysis

1. According to section 1613 of the 2010 CBC (CBSC, 2010), all structures and portions of structures should be designed to resist the effects of seismic loadings caused by earthquake ground motions in accordance with the *Minimum Design Loads for Buildings and Other Structures* (ASCE7) (ASCE, 2006). ASCE7 considers the most severe earthquake ground motion to be the ground motion caused by the Maximum Considered Earthquake (MCE) (ASCE, 2006), which is defined in Section 1613 of the 2010 CBC to be short period S_{MS} and 1-second period S_{M1}, spectral response accelerations.

2. The a_{max} of the Site depends on several factors, which include the distance of the Site from known active faults, the expected magnitude of the MCE, and the Site soil profile characteristics.

3. As per section 1613.5.5 of the 2010 CBC (CBSC, 2010), the Site soil profile classification is determined by the average soil properties in the upper 100 feet of the Site profile. Based on the $(N_1)_{60}$ values calculated for the in-situ tests performed during the field investigation, the Site was defined as Site Class C, Very Dense Soil and Soft Rock profile per Table 1613.5.2 of the 2010 CBC (CBSC, 2010).

4. According to section 11.2 of ASCE7 (ASCE, 2006) and section 1613 of the 2010 CBC (CBSC, 2010), buildings and structures should be specifically proportioned to resist Design Earthquake Ground Motions (Design a_{max}). ASCE7 defines the Design a_{max} as "the earthquake ground motions that are two-thirds of the corresponding MCE ground motions" (ASCE, 2006, p. 109). Therefore, the **Design a_{max} for the Site is equal to S_{D1}=0.389 and S_{DS}=0.724,** which are 1-second period and short period design spectral response accelerations that are equal to two-thirds of the a_{max} or MCE for the Site.

5. Site coordinates of 35.757613 degrees north latitude and approximately -121.011508 degrees west longitude and a search radius of 100 miles were used in the probabilistic seismic hazard analysis.

4.2 Structural Building Design Parameters

1. Structural building design parameters within chapter 16 of the 2010 CBC (CBSC, 2010) and sections 11.4.3 and 11.4.4 of ASCE7 (ASCE, 2006) are dependent upon several factors, which include site soil profile characteristics and the locations and characteristics of faults near the Site. As described in section 4.1 of this report, the Site soil profile classification was determined to be Site Class C. This Site soil profile classification and the latitude and longitude coordinates for the Site were used to determine the structural building design parameters.

2. Spectral Response Accelerations and Site Coefficients were obtained from the Seismic Hazard Curves and Uniform Hazard Response Spectra, Earthquake Ground Motion Tool computer application (USGS, 2007); this program is available from the United States Geological Survey website (USGS, 2008). This computer program utilizes the methods developed in the 1997, 2000, and 2003 editions of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures and user-inputted Site latitude and longitude coordinates to calculate seismic design parameters and response spectra (both for period and displacement), for Site Classifications A through E. This data

is presented in tabular form in Table 2: 2010 California Building Code, Chapter 16, Structural Design Parameters. Analysis of the Design Spectral Response Acceleration Parameters for the Site and of the Occupancy Category for the proposed structure assign to this project a **Seismic Design Category of D** per Tables 1613.5.6(1) and 1613.5.6(2) of the 2010 CBC (CBSC, 2010).

Table 2: 2010 California Building Code, Chapter 16, Structural Design Parameters

Site Class - Soil Profile Type	C – Very Dense Soil & Soft Rock
Mapped Spectral Response Accelerations and Site Coefficients	$S_S = 1.087$, $S_1 = 0.424$ $F_a = 1.000$, $F_v = 1.376$
Adjusted Maximum Considered Earthquake Spectral Response Accelerations	$S_{MS} = 1.087$ $S_{M1} = 0.583$
Design Spectral Response Acceleration Parameters	$S_{DS} = 0.724$ $S_{D1} = 0.389$
Occupancy Category (from Table 1604.5, 2010 CBC)	II
Seismic Design Category – Short Period Accel. (from Table 1613.5.6(1), 2010 CBC)	D
Seismic Design Category – Long Period Accel. (from Table 1613.5.6(2), 2010 CBC)	D

4.3 Design Response Spectra – 2010 CBC

According to section 11.4.5 of ASCE7 (ASCE, 2006), a design response spectrum for a site may be required in order to design structures to resist lateral forces caused by ground motions at the Site. The design spectral response acceleration parameters, listed in Table 2: 2010 California Building Code, Chapter 16, Structural Design Parameters, are used to produce the design response spectrum. The Seismic Hazard Curves and Uniform Hazard Response Spectra computer program (USGS, 2007) was used to construct constructed a design response spectrum for the Site, which is shown in Figure 5: Design Response Spectra – 2010 CBC.

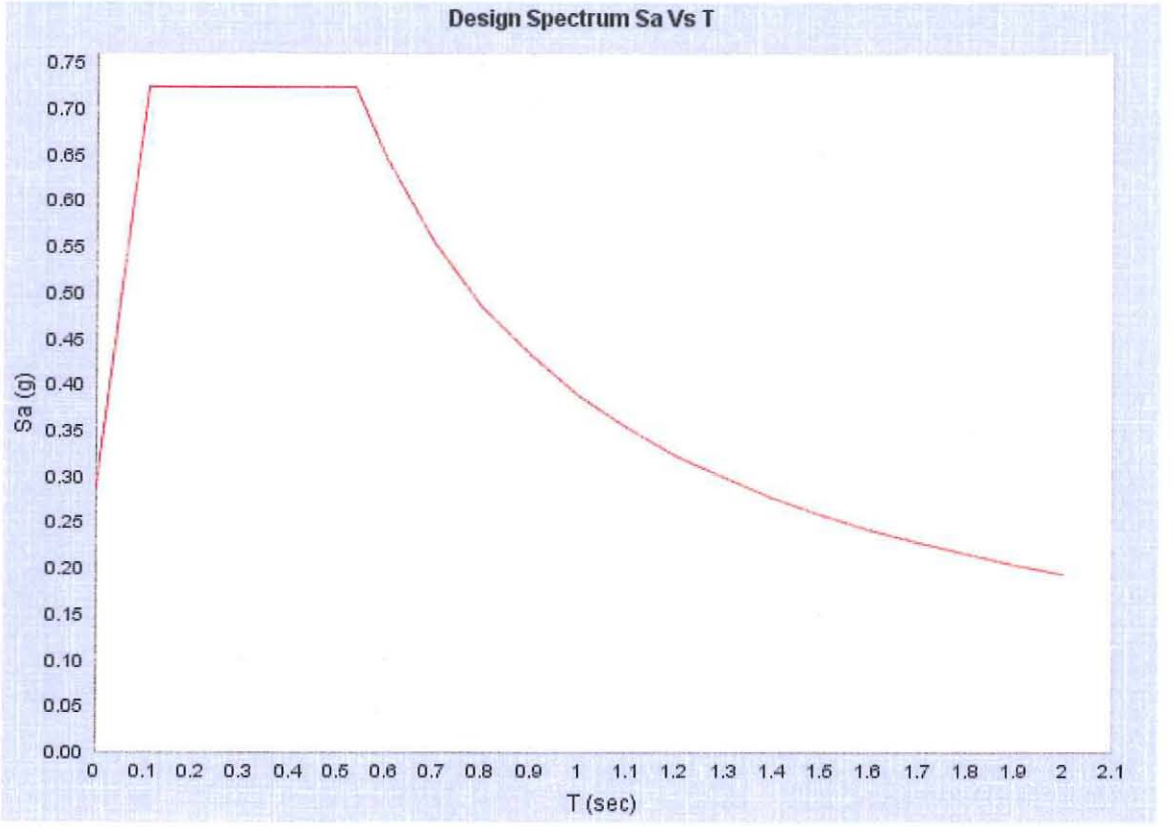

Figure 5: Design Response Spectra – 2010 CBC

4.4 Liquefaction Potential

1. In the context of soil mechanics, liquefaction is the process that occurs when the dynamic loading of a soil mass causes the shear strength of the soil mass to rapidly decrease. Liquefaction can occur in saturated cohesionless soils.

2. The most typical liquefaction-induced failures include consolidation of liquefied soils, surface sand boils, lateral spreading of the ground surface, bearing capacity failures of structural foundations, flotation of buried structures, and differential settlement of above-ground structures.

3. Liquefiable soils must undergo dynamic loading before liquefaction occurs. Ground motion from an earthquake may induce large-amplitude cyclic reversals of shear stresses within a soil mass. Repetitive lateral and vertical loading and unloading usually results from this process. This process is considered to be dynamic loading. In a liquefiable soil mass, liquefaction may occur as a result of the dynamic loading caused by ground motion produced by an earthquake.

4. The presence of loose, poorly graded, fine sand material that is saturated by groundwater within an area that is known to be subjected to high intensity earthquakes and long-duration ground motion are the key factors that indicate potentially liquefiable areas and conditions that lead to liquefaction.

5. Based on the consistency and relative density of the in-situ soils the potential for seismic liquefaction of soils at the Site is low. Assuming that the recommendations of the Soils

Engineering Report are implemented, the potential for seismically induced settlement and differential settlement at the Site is considered to be low.

5.0 GENERAL SOIL-FOUNDATION DISCUSSION

Based on the information obtained during the field investigation and following laboratory testing of the materials sampled from the proposed building areas, it is anticipated that graded engineered fill pads will be constructed for the security main entry area with foundations excavated into engineered fill. It is anticipated that pier foundations will be constructed for the staff lodging area. Due to the shallow depth to competent formational material encountered during the field investigation it is anticipated that foundations for the structures proposed in the camper lodging, chapel, celebration arts campus, mill barn, lake shack and equestrian barns will be excavated into uniform competent formational material. As an alternative, graded engineered fill pads may be constructed in all proposed building areas. Recommendations for building pad preparation for both options are presented below.

All foundations are to be excavated into uniform material to limit the potential for distress of the foundation systems due to differential settlement. If cuts steeper than allowed by State of California Construction Safety Orders for "Excavations, Trenches, Earthwork" are proposed, a numerical slope stability analysis may be necessary for temporary construction slopes.

Natural seepage at the interface of two materials with different densities, such as native soil and formational material, is very common. This interface occurs at the Site and may require sub-surface drains. Sub-drains should be placed in established drainage courses, potential seepage areas, and during the development of all key and bench grading operations.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The Site is suitable for the proposed development provided the recommendations presented in this report are incorporated into the project plans and specifications.

The primary geotechnical concerns at the Site are:

1. The potential for groundwater seepage.

2. The presence of loose surface soils.

3. The presence of potentially expansive surface materials. Influx of water from irrigation, leakage from the structures, or natural seepage could cause expansive soil problems. Expansive soils were encountered in the mission, celebration arts campus, staff lodging, mill barn, lake shack, equestrian barn and camper lodging areas

4. The potential for differential settlement occurring between foundations supported on two soil materials having different settlement characteristics, such as native soil and engineered fill or competent formational material. Therefore, it is important that all of the foundations are founded in equally competent uniform material in accordance with this report.

6.1 Preparation of Building Pad Areas

6.1.1 Building Pad Preparation – Engineered Fill

1. It is anticipated that graded engineered fill pads will be constructed for the proposed security main entry with foundations excavated into engineered fill. This method of

building pad preparation is available as an option for all proposed building areas on the Site.

2. For the development of engineered fill pads, the native material should be over-excavated a minimum depth of 24 inches below existing grade, 12 inches below the bottom of the footings, or one-half the depth of deepest fill, whichever is greatest. The horizontal limits of over-excavation should extend a minimum of five feet beyond the perimeter foundations. The exposed surfaces should then be scarified, moisture conditioned or allowed to dry back to 1 to 3 percent above optimum moisture content, and compacted to a minimum relative density of 90 percent, based on the ASTM D1557-07 test method. The over-excavated soil may then be processed as engineered fill. Refer to Figure 6: Sub-Slab Detail for under slab drainage material and Appendix C for more details on fill placement.

3. If fill areas are constructed on slopes greater than 10-to-1 (horizontal-to-vertical), we recommend that benches be cut every four feet as fill is placed. Each bench shall be a minimum of 10 feet wide with a minimum of two percent gradient into the slope. If fill areas are constructed on slopes greater than 5-to-1, we recommend that the toe of all areas to receive fill be keyed a minimum of 24 inches into underlying dense material. Sub-drains shall be placed in the keyway and benches as necessary during grading operations. See **Appendix C**, Detail A, Key and Bench with Backdrain for details on key and bench construction.

6.1.2 Building Pad Preparation – Foundations in Competent Formational Material

1. It is anticipated that foundations for the proposed camper lodging, chapel, celebration arts campus, mill barn, lake shack and equestrian barns will be excavated into competent formational material, as observed and approved by a representative of this firm. Deepened footings may be required in certain areas to achieve the required embedment depth in uniform competent formational material. In areas where deepened footings are required, placement of 2-sack sand cement slurry within the excavations up to footing design depth may be performed.

2. For slab-on-grade construction with footings founded a minimum of 12 inches into uniform competent formational material, pad areas should be graded so that all slabs are supported on uniform competent material. The native material should be excavated a minimum depth of 12 inches below finish floor elevation, to competent material, or one-half the depth of deepest fill, whichever is greater. The exposed surface should be scarified to a depth of 12 inches, moisture conditioned or allowed to dry back to 1 to 3 percent above optimum moisture content and compacted to a minimum relative density of 90 percent, based on the ASTM D1557-07 test method. Refer to Figure 6: Sub-Slab Detail for under slab drainage material and Appendix C for more details on fill placement.

3. For raised wood floor construction with footings founded a minimum of 12 inches into uniform competent formational material, the sub-floor areas should be excavated to the required depth below lowest adjacent grade, scarified an additional depth of 12 inches and compacted to a minimum relative density of 90 percent.

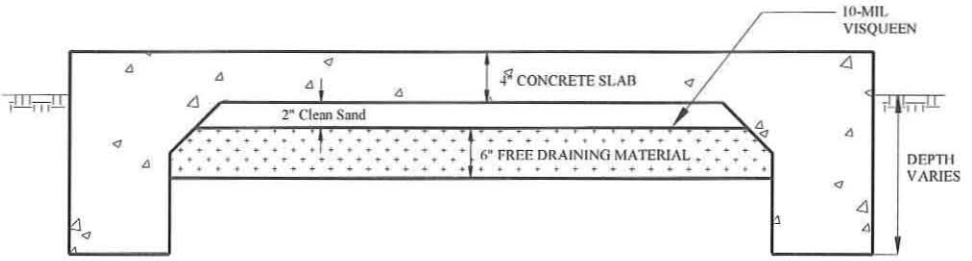

Figure 6: Sub-Slab Detail

6.2 Preparation of Roadway Areas

1. Prior to the placement of fill in roadway areas, preparation of original ground requires the removal of loose, unconsolidated soils and debris. Roadway areas should be over-excavated a minimum depth of 12 inches below existing grade. The exposed surface should be scarified, moisture conditioned or allowed to dry back to 1 to 3 percent above optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-07 test method). The over-excavated soil should then be moisture conditioned to produce a water-content of at least one to two percent above optimum value and then compacted to a minimum relative density of 90 percent. The top 12 inches of sub-grade soil in roadway areas should be compacted to a minimum relative density of 95 percent based on the ASTM D1557-07 test method at slightly above optimum.

2. If roadway fill areas are constructed on slopes greater than 10-to-1 (horizontal-to-vertical), we recommend that benches be cut every four feet as fill is placed. Each bench shall be a minimum of 10 feet wide with a minimum of two percent gradient into the slope. If fill areas are constructed on slopes greater than 5-to-1, we recommend that the toe of all areas to receive fill be keyed a minimum of 24 inches into underlying dense material. Sub-drains shall be placed in the keyway and benches as necessary during grading operations. See **Appendix C**, Detail A, Key and Bench with Backdrain for details on key and bench construction

3. Sub-grade soils should not be allowed to dry out or have excessive construction traffic between moisture conditioning and compaction, and placement of the pavement structural section.

4. Due to the expansive potential of the soils at the Site, the base courses beneath un-reinforced pavement sections may fail, causing cracking of the pavement surfaces, as the sub-grade materials move laterally during expansive shrink-swell cycles.

5. Therefore, in order to minimize the potential for the failure of paved roadway sections at the Site, GeoSolutions, Inc. recommends that a laterally-reinforcing geotextile grid, such as Tensar BX1100, or equivalent, be installed to reinforce the base courses under paved areas at the Site.

6. GeoSolutions, Inc. should be contacted prior to the design and construction of pavement sections at the Site in order to assist in the selection of an appropriate laterally-reinforcing geotextile grid product and to provide recommendations regarding the procedures for the installation of geotextile grid products at the Site.

6.3 Pavement Design

1. All pavement construction and materials used should conform to Sections 25, 26 and 39 of the latest edition of the State of California Department of Transportation Standard Specifications (State of California, 1999).

2. As indicated previously in Section 6.2, the top 12 inches of sub-grade soil under pavement sections should be compacted to a minimum relative density of 95 percent based on the ASTM D1557-07 test method at slightly above optimum moisture content. Aggregate bases and sub-bases should also be compacted to a minimum relative density of 95 percent based on the aforementioned test method.

3. A minimum of six inches of Class II Aggregate Base is recommended for all pavement sections. All pavement sections should be crowned for good drainage.

4. In order to minimize the potential for cracking of the pavement surfaces at the Site due to lateral movement of the base courses during expansive shrink-swell cycles of the sub-grade materials, GeoSolutions, Inc. recommends that a laterally-reinforcing geotextile grid, such as Tensar BX100, or equivalent, be installed between the prepared sub-grade and base materials at the Site.

5. GeoSolutions, Inc. should be contacted prior to the design and construction of the pavement sections to provide recommendations regarding the selection of and installation of an appropriate laterally-reinforcing geotextile grid product.

6.4 Conventional Foundations

1. Conventional continuous and spread footings with grade beams may be used for support of proposed structures. Isolated pad footings should be a minimum of two feet square in size, are only allowed in soils with very low expansion potential and are permitted for single floor loads only.

2. Minimum footing and grade beam sizes and depths in engineered fill or uniform competent formational material should conform to the following tables, as observed and approved by a representative of GeoSolutions, Inc. Minimum reinforcing for footings should be as indicated on the following tables or as directed by the project Structural Engineer.

Tables 3A – 3C: Minimum Footing and Grade Beam Dimensions

Table 3A: Very Low Expansion Potential

Building areas included in the very low expansion potential category include; the chapel and security main entry.

Excavated in Engineered Fill – Very Low Expansion Potential		
Building Type	**Minimum Depth Below Lowest Adjacent Grade**	**Minimum Width**
One-Story	12 inches	12 inches
Two-Story	18 inches	15 inches

Excavated in Uniform Competent Formational Material – Very Low Expansion Potential			
Building Type	**Minimum Depth Below Lowest Adjacent Grade**	**Minimum Embedment into Uniform Competent Formational Material**	**Minimum Width**
One-Story	12 inches	12 inches	12 inches
Two-Story	18 inches	12 inches	15 inches

Isolated pad footings should be a minimum of two feet square in size and are permitted for single floor loads only.
Minimum reinforcing for footings should be four No. 4 bars, placed two at the top and two at the bottom, or as directed by the project Structural Engineer.

Table 3B: Low Expansion Potential

Building areas included in the low expansion potential category include; the mission, celebration arts campus, staff lodging, mill barn, lake shack, and equestrian barn.

Excavated in Engineered Fill – Low Expansion Potential		
Building Type	**Minimum Depth Below Lowest Adjacent Grade**	**Minimum Width**
One-Story	15 inches	12 inches
Two-Story	18 inches	15 inches

Excavated in Uniform Competent Formational Material – Low Expansion Potential			
Building Type	**Minimum Depth Below Lowest Adjacent Grade**	**Minimum Embedment into Uniform Competent Formational Material**	**Minimum Width**
One-Story	15 inches	12 inches	12 inches
Two-Story	18 inches	12 inches	15 inches

Isolated pad footings are not permitted.
Minimum reinforcing for footings should be four No. 4 bars, placed two at the top and two at the bottom, or as directed by the project Structural Engineer.

Table 3C: Medium Expansion Potential

Building areas included in the medium expansion potential category include camper lodging only.

Excavated in Engineered Fill – Medium Expansion Potential		
Building Type	**Minimum Depth Below Lowest Adjacent Grade**	**Minimum Width**
One-Story	24 inches	12 inches
Two-Story	24 inches	15 inches

Excavated in Uniform Competent Formational Material – Medium Expansion Potential			
Building Type	**Minimum Depth Below Lowest Adjacent Grade**	**Minimum Embedment into Uniform Competent Formational Material**	**Minimum Width**
One-Story	24 inches	12 inches	12 inches
Two-Story	24 inches	12 inches	12 inches
Isolated pad footings are not permitted.			
Minimum reinforcing for footings should be four No. 5 bars, placed two at the top and two at the bottom, or as directed by the project Structural Engineer.			

3. A representative of this firm should observe and approve all foundation excavations for required embedment depth prior to the placement of reinforcing steel and/or concrete. Concrete should be placed only in excavations that are free of loose, soft soil and debris and that have been lightly pre-moistened, with no associated testing required.

4. An allowable dead plus live load bearing pressure value of **2,000 psf** may be used for the design of footings founded in **engineered fill** and a value of **3,000 psf** may be used for the design of footings founded in **competent formational material**.

5. A total settlement of less than ¾ inch and a differential settlement of less than ½ inch are anticipated.

6. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the engineered fill or competent formational material and the bottom of the footings. For resistance to lateral loads, the friction factor of **0.3** may be utilized for sliding resistance at the base of footings extending a minimum of 12 inches into engineered fill and a friction factor of **0.4** may be utilized for sliding resistance at the base of footings extending a minimum of 12 inches into uniform competent native material. A passive pressure of **300-pcf** equivalent fluid weight may be used against the side of shallow footings in engineered fill and a passive pressure of **450-pcf** equivalent fluid weight may be used against the side of shallow footings in competent formational material. If friction and passive pressures are combined to resist lateral forces acting on shallow footings, the lesser value should be reduced by 50 percent.

7. Foundation excavations should be observed and approved by a representative of this firm prior to the placement of reinforcing steel and/or concrete.

8. Foundation design should conform to the requirements of Chapter 18 of the latest edition of the CBC (CBSC, 2010).

9. The base of all grade beams and footings should be level and stepped as required to accommodate any change in grade while still maintaining the minimum required footing embedment and slope setback distance.

10. Foundation excavations should be observed and approved by a representative of this firm prior to the placement of reinforcing steel and/or concrete.

11. Foundation design should conform to the requirements of Chapter 18 of the latest edition of the CBC (CBSC, 2007).

12. The base of all grade beams and footings should be level and stepped as required to accommodate any change in grade while still maintaining the minimum required footing embedment and slope setback distance.

13. Where appropriate the minimum footing setback distance from ascending or descending steeper than 3-to-1 (horizontal-to-vertical) but less than 1-to-1 must be maintained. See Figure 7: Setback Dimensions – Slope Gradients Between 3-to-1 and 1-to-1 for the minimum horizontal setback distances from ascending and descending slopes steeper than 3-to-1 but not steeper than 1-to-1.

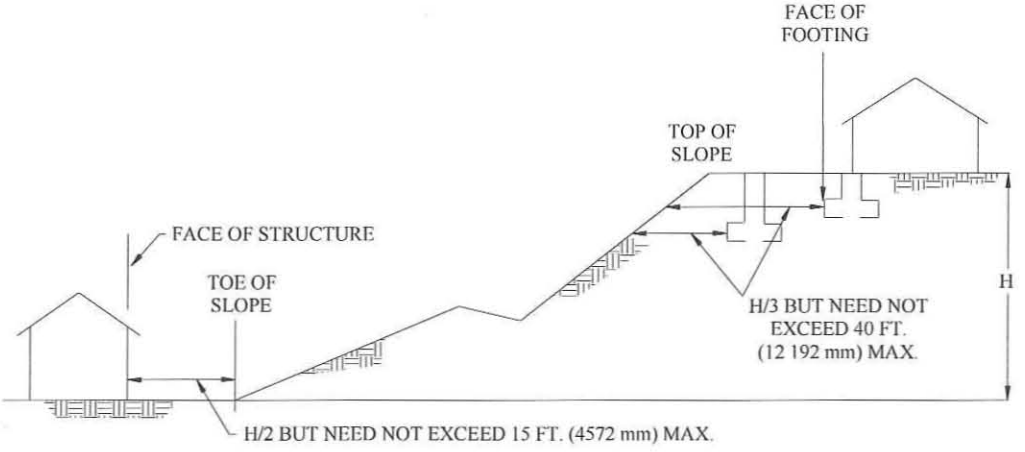

Figure 7: Setback Dimensions – Slope Gradients Between 3-to-1 and 1-to-1

14. If alternate footing setback distances from ascending or descending slopes are desired, GeoSolutions, Inc. may be contracted to perform an additional Numerical Slope Stability Study. Depending on the results of this study, alternate minimum footing setback distances from ascending or descending slopes may be provided.

6.5 Slab-On-Grade Construction

1. Concrete slabs-on-grade and flatwork should not be placed directly on unprepared native materials. Preparation of sub-grade to receive concrete slabs-on-grade and flatwork should be processed as discussed in the preceding sections of this report. Concrete slabs should be placed only over sub-grade that is free of loose, soft soil and debris and that has been pre-moistened to 120 percent of optimum moisture to a depth of 12 inches for soils with low expansion potential, to 130 percent of optimum moisture to a depth of 18 inches for soils with medium expansion potential, with associated testing required. Sub-slab soils in the very low expansion category do not require pre-moistening verification.

2. Concrete slabs-on-grade should be a minimum of 4 inches thick and should be reinforced with No. 3 reinforcing bars placed at 18 inches on-center both ways at or slightly above the center of the structural section with the exception of the slabs-on-grade for the proposed camper lodging structures, which should be reinforced with No. 3 reinforcing bars placed at 12 inches on-center both ways. Reinforcing bars should have a minimum clear cover of 1.5 inches. The aforementioned reinforcement may be used for anticipated uniform floor loads not exceeding 200 psf. If floor loads greater than 200 psf are anticipated, a Structural Engineer should evaluate the slab design.

3. Concrete for all slabs should be placed at a maximum slump of less than 5 inches. Excessive water content is the major cause of concrete cracking. If fibers are used to aid in the control of cracking, a water-reducing admixture may be added to the concrete to increase slump while maintaining a water/cement ratio, which will limit excessive shrinkage. Control joints should be constructed as required to control cracking.

4. Where concrete slabs-on-grade are to be constructed, the slabs should be underlain by a minimum of six inches of clean free-draining material, such as a coarse aggregate mix, to serve as a cushion and a capillary break. Where moisture susceptible storage or floor coverings are anticipated, a 10-mil Visqueen-type membrane should be placed between the free-draining material and the slab to minimize moisture condensation under the floor covering. See Figure 6: Sub-Slab Detail for the placement of under-slab drainage material. It is suggested that a two-inch thick sand layer be placed on top of the membrane to assist in the curing of the concrete, increasing the depth of the under-slab material to a total of eight inches. The sand should be lightly moistened prior to placing concrete.

5. Moisture condensation under floor coverings has become critical due to the use of water-soluble adhesives. Therefore, it is suggested that moisture sensitive slabs not be constructed during inclement weather conditions.

6.6 Retaining Walls

1. Retaining walls should be designed to resist lateral pressures from adjacent soils and surcharge loads applied behind the walls. We recommend using the lateral pressures presented in Table 4: Retaining Wall Design Parameters and Figure 8: Retaining Wall Detail for the design of retaining walls at the Site. The Active Case may be used for the design of unrestrained retaining walls, and the At-Rest Case may be used for the design of restrained retaining walls.

Table 4: Retaining Wall Design Parameters

Lateral Pressure and Condition	Equivalent Fluid Pressure, pcf
Static, Active Case, Engineered Fill ($\gamma' K_A$)	45
Static, Active Case, Competent Formational Material ($\gamma' K_A$)	35
Static, At-Rest Case, Engineered Fill ($\gamma' K_O$)	65
Static, At-Rest Case, Competent Formational Material($\gamma' K_O$)	55
Static, Passive Case, Engineered Fill ($\gamma' K_P$)	300
Static, Passive Case, Competent Formational Material ($\gamma' K_P$)	450

2. The above values for equivalent fluid pressure are based on retaining walls having level retained surfaces, having an approximately vertical surface against the retained material, and retaining granular backfill material or engineered fill composed of native soil within the active wedge. See Figure 9: Retaining Wall Active and Passive Wedges for a description of the location of the active wedge behind a retaining wall.

3. Proposed retaining walls having a retained surface that slopes upward from the top of the wall should be designed for an additional equivalent fluid pressure of **1 pcf** for the active case and **1.5 pcf** for the at-rest case, for every **two degrees** of slope inclination. This applies for slope angles up to 20 degrees; a 20 degree-slope is approximately equivalent to a slope with a 2.75-to-1 gradient. For slope angles greater than 20 degrees, the Soils Engineer should be consulted to obtain design equivalent fluid pressure values for retaining walls located at the Site.

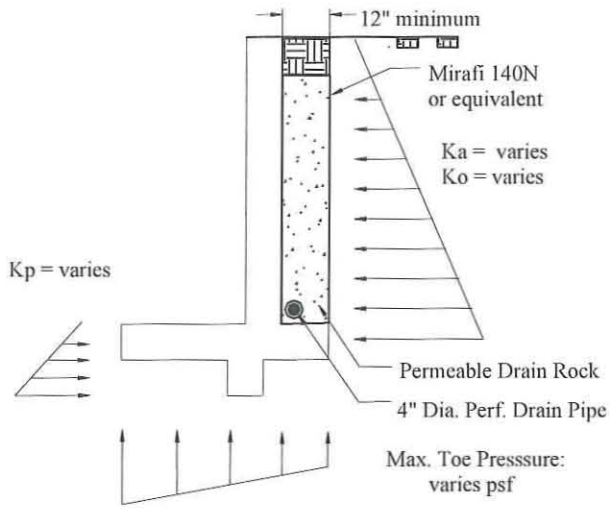

Figure 8: Retaining Wall Detail

4. We recommend that the proposed retaining walls at the Site have an approximately vertical surface against the retained material. If the proposed retaining walls are to have sloped surfaces against the retained material, the project designers should contact the Soils Engineer to determine the appropriate lateral earth pressure values for retaining walls located at the Site.

5. Retaining wall foundations should be founded a minimum of 12 inches below lowest adjacent grade in engineered fill with very low expansion potential or a minimum of 12 inches into competent formational material as observed and approved by a representative of GeoSolutions, Inc. Retaining wall footing depths should be increased to a minimum of 15 inches below lowest adjacent grade for low expansion potential and 24 inches below lowest adjacent grade for medium expansion category soil, refer to **Section 6.4 Conventional Foundations** for additional information regarding area specific soil expansion potential. A coefficient of friction value of **0.3** may be used between engineered fill and concrete footings and a coefficient of friction value of **0.4** may be used between competent formational material and concrete footings. Project designers may use a maximum toe pressure of **2,400 psf** for the design of retaining wall footings founded in engineered fill and a maximum toe pressure of **3,600 psf** for the design of retaining walls footings in competent formational material.

6. Seismic active lateral earth pressure values were determined using the Pseudostatic Method and the Design a_{max}. See section 4.1 for a description of the analysis used to determine the Design a_{max}. The seismic at-rest lateral earth pressure value was determined

by multiplying the seismic active lateral earth pressure value by approximately 1.5. The dynamic increment in lateral earth pressure due to earthquakes should be considered during the design of retaining walls at the Site. Retaining walls should be designed to resist an additional lateral soil pressure of **25 pcf** equivalent fluid pressure for unrestrained walls and **40 pcf** equivalent fluid pressure for restrained walls. For earthquake conditions, the pressure resultant force should be assumed to act a distance of $^{2}/_{3}$H above the base of the retaining wall, where H is the height of the retaining wall.

7. These seismic lateral earth pressure values are appropriate for retaining walls that have level retained surfaces, that have an approximately vertical surface against the retained material, and that retain granular backfill material or engineered fill composed of native soil within the active wedge. For other retaining wall designs, seismic lateral earth pressure values may be obtained using methods such as the Mononobe and Okabe Method developed by Mononobe and Matsuo (1929) and Okabe (1926), which are included in retaining wall computer design software such as Retain Pro.

8. Seismically induced forces on retaining walls are considered to be short-term loadings. Therefore, when performing seismic analyses for the design of retaining wall footings, we recommend that the allowable bearing pressure and the passive pressure acting against the sides of retaining wall footings be increased by a factor of one-third.

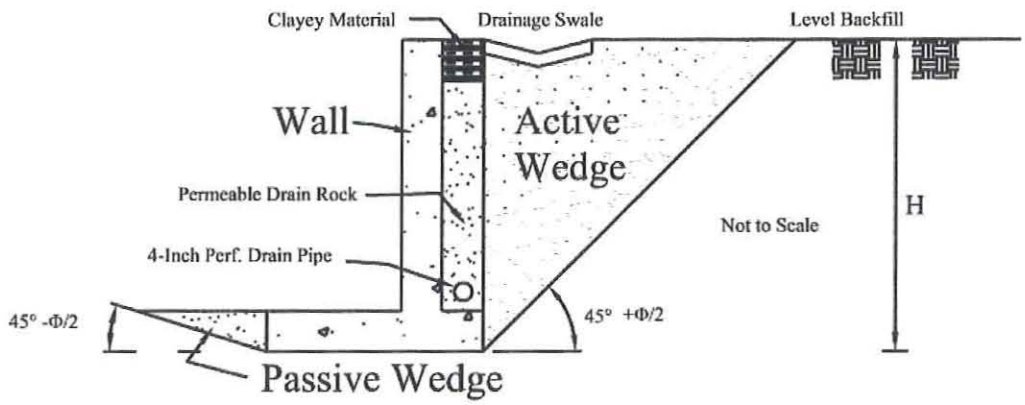

Figure 9: Retaining Wall Active and Passive Wedges

9. In addition to the static lateral soil pressure values reported in Table 4: Retaining Wall Design Parameters, the retaining walls at the Site should be designed to support any design live load, such as from vehicle and construction surcharges, etc., to be supported by the wall backfill. If construction vehicles are required to operate within 10 feet of a retaining wall, supplemental pressures will be induced and should be taken into account in the design of the retaining wall.

10. The recommended lateral earth pressure values are based on the assumption that sufficient sub-surface drainage will be provided behind the walls to prevent the build-up of hydrostatic pressure. To achieve this we recommend that a granular filter material be placed behind all proposed walls. The blanket of granular filter material should be a minimum of 12 inches thick and should extend from the bottom of the wall to 12 inches from the ground surface. The top 12 inches should consist of moisture conditioned,

compacted, clayey soil. Neither spread nor wall footings should be founded in the granular filter material used as backfill.

11. A 4-inch diameter perforated or slotted drainpipe (ASTM D1785 PVC) should be installed near the bottom of the filter blanket with perforations facing down. The drainpipe should be underlain by at least 4 inches of filter type material and should daylight to discharge in suitably projected outlets with adequate gradients. The filter material should consist of a clean free-draining aggregate, such as a coarse aggregate mix. If the retaining wall is part of a structural foundation, the drainpipe must be placed below finished slab sub-grade elevation.

12. The filter material should be encapsulated in a permeable geotextile fabric. A suitable permeable geotextile fabric, such as non-woven needle-punched Mirafi 140N or equal, may be utilized to encapsulate the retaining wall drain material and should conform to Caltrans Standard Specification 88-1.03 for underdrains.

13. For hydrostatic loading conditions (i.e. no free drainage behind retaining wall), an additional loading of 45-pcf equivalent fluid weight should be added to the active and at-rest lateral earth pressures. If it is necessary to design retaining structures for submerged conditions, the allowed bearing and passive pressures should be reduced by 50 percent. In addition, soil friction beneath the base of the foundations should be neglected.

14. Precautions should be taken to ensure that heavy compaction equipment is not used adjacent to walls, so as to prevent undue pressure against, and movement of the walls.

15. The use of water-stops/impermeable barriers should be used for any basement construction, and for building walls that retain earth.

7.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations contained in this report are based on a limited number of borings/trenches and on the continuity of the sub-surface conditions encountered. GeoSolutions, Inc. assumes that it will be retained to provide additional services during future phases of the proposed project. These services would be provided by GeoSolutions, Inc. as required by the County of San Luis Obispo, the 2010 CBC, and/or industry standard practices. These services would be in addition to those included in this report and would include, but are not limited to, the following services:

1. Consultation during plan development.

2. Plan review of grading and foundation documents prior to construction and a report certifying that the reviewed plans are in conformance with our geotechnical recommendations.

3. Consultation during selection and placement of a laterally-reinforcing geotextile grid product.

4. Construction inspections and testing, as required, during all grading and excavating operations beginning with the stripping of vegetation at the Site, at which time a site meeting or pre-job meeting would be appropriate.

5. Special inspection services during construction of reinforced concrete, structural masonry, high strength bolting, epoxy embedment of threaded rods and reinforcing steel, and welding of structural steel.

6. Preparation of construction reports certifying that building pad preparation and foundation excavations are in conformance with our geotechnical recommendations.

7. Preparation of special inspection reports as required during construction.

8. In addition to the construction inspections listed above, section 1704.7 of the 2010 CBC (CBSC, 2010) requires the following inspections by the Soils Engineer for controlled fill thicknesses greater than 12 inches as shown in Table 5: Required Verification and Inspections of Soils:

Table 5: Required Verification and Inspections of Soils

Verification and Inspection Task	Continuous During Task Listed	Periodically During Task Listed
1. Verify materials below footings are adequate to achieve the design bearing capacity.	-	X
2. Verify excavations are extended to proper depth and have reached proper material.	-	X
3. Perform classification and testing of controlled fill materials.	-	X
4. Verify use of proper materials, densities and lift thicknesses during placement and compaction of controlled fill.	X	-
5. Prior to placement of controlled fill, observe sub-grade and verify that site has been prepared properly.	-	X

8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed during our study. Should any variations or undesirable conditions be encountered during the development of the Site, GeoSolutions, Inc. should be notified immediately and GeoSolutions, Inc. will provide supplemental recommendations as dictated by the field conditions.

2. This report is issued with the understanding that it is the responsibility of the owner or his/her 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 project plans and specifications. The owner or his/her representative is responsible to ensure that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. As of the present date, the findings of this report are valid for the property studied. With the passage of time, changes in the conditions of a property can occur whether they are due to natural processes or to the works of man on this or adjacent properties. Therefore, this report should not be relied upon after a period of 3 years without our review nor should it be used or is it applicable for any properties other than those studied. However many events such as floods, earthquakes, grading of the adjacent properties and building and municipal code changes could render sections of this report invalid in less than 3 years.

REFERENCES

REFERENCES

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PLATES

1A – Site Engineering Geology Map (GeoSolutions, Inc. 2011)

1B – Site Engineering Geology Map (GeoSolutions, Inc. 2011)

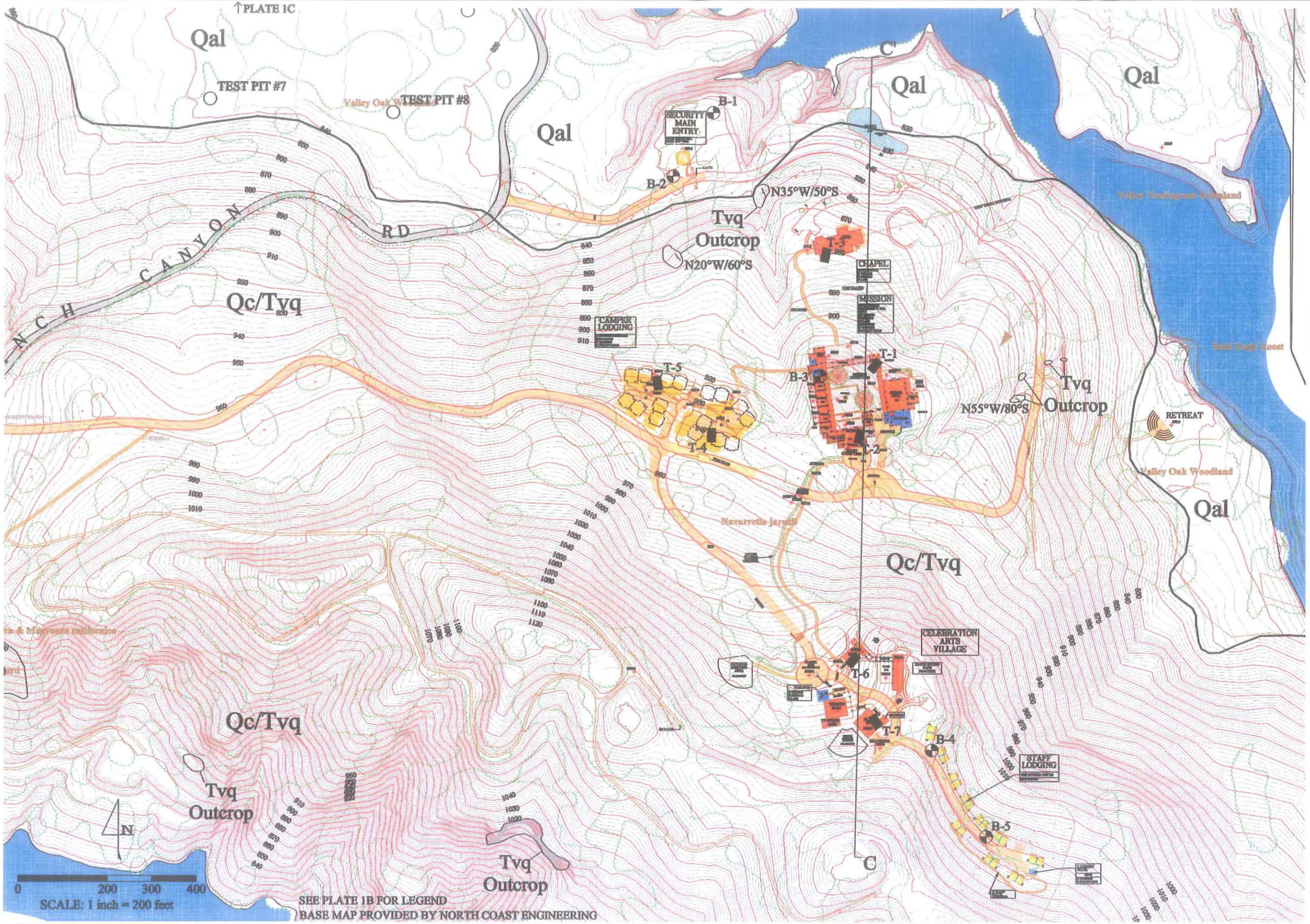


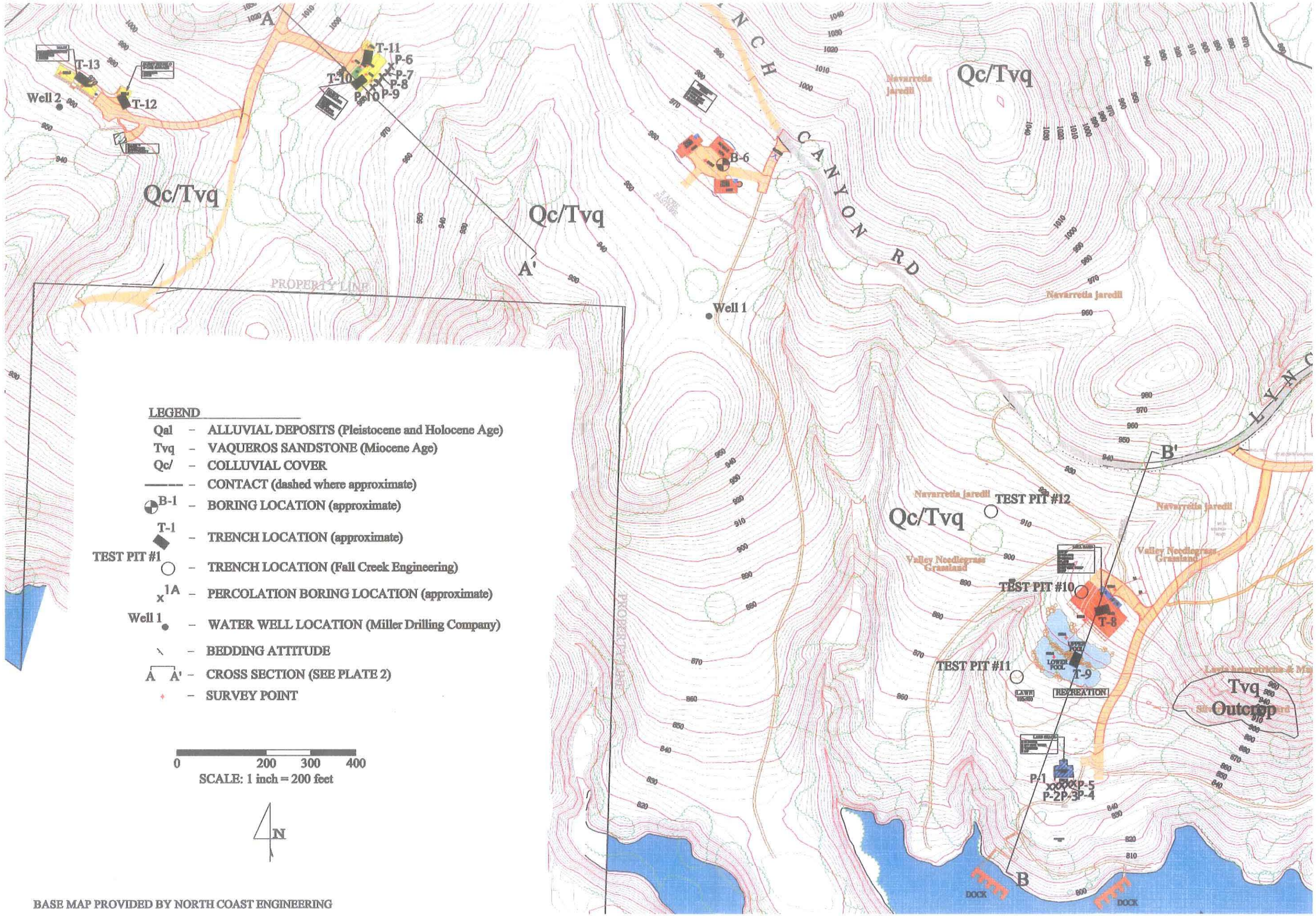
PLATE 1A

PROJECT SL07503-1

SITE ENGINEERING GEOLOGY MAP

CANTINAS RANCH, LYNCH CANYON ROAD,
LAKE NACIMIENTO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA

GeoSolutions, Inc.
220 High Street
San Luis Obispo, CA 93401
(805) 543-8539 Fax: (805) 543-2171



BASE MAP PROVIDED BY NORTH COAST ENGINEERING

PLATE 1B

PROJECT SL07503-1

SITE ENGINEERING GEOLOGY MAP

CANTINAS RANCH, LYNCH CANYON ROAD,
LAKE NACIMIENTO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA

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APPENDIX A

Field Investigation

Soil Classification Chart

Boring Logs

FIELD INVESTIGATION

The field investigation was conducted October 8 through 20, 2010 using a track-mounted CME 55 drill rig/backhoe. The surface and sub-surface conditions were studied by advancing six exploratory borings and thirteen exploratory trenches. This exploration was conducted in accordance with presently accepted geotechnical engineering procedures consistent with the scope of the services authorized to GeoSolutions, Inc.

The CME 55 drill rig with a four-inch diameter solid-stem continuous flight auger bored six exploratory borings and thirteen exploratory trenches near the approximate locations indicated on **Figure 3: Site Plan (Eastern Portion)**and **Figure 4: Site Plan (Western Portion)** and the enlarged plates 1A and 1B in the proceeding section. The drilling, trenching and field observation was performed under the direction of the project engineer. A representative of GeoSolutions, Inc. maintained a log of the soil conditions and obtained soil samples suitable for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System. See the Soil Classification Chart in this appendix.

Standard Penetration Tests with a two-inch outside diameter standard split tube sampler (SPT) without liners (ASTM D1586-99) and a three-inch outside diameter Modified California (CA) split tube sampler with liners (ASTM D3550-01) were performed to obtain field indication of the in-situ density of the soil and to allow visual observation of at least a portion of the soil column. Soil samples obtained with the split spoon sampler are retained for further observation and testing. The split spoon samples are driven by a 140-pound hammer free falling 30 inches. The sampler is initially seated six inches to penetrate any loose cuttings and is then driven an additional 12 inches with the results recorded in the boring logs as N-values, which area the number of blows per foot required to advance the sample the final 12 inches.

The CA sampler is a larger diameter sampler than the standard (SPT) sampler with a two-inch outside diameter and provides additional material for normal geotechnical testing such as in-situ shear and consolidation testing. Either sampler may be used in the field investigation, but the N-values obtained from using the CA sampler will be greater than that of the SPT. The N-values for samples collected using the CA can be roughly correlated to SPT N-values using a conversion factor that may vary from about 0.5 to 0.7. A commonly used conversion factor is 0.67 ($^2/_3$). More information about standardized samplers can be found in ASTM D1586-99 and ASTM D3550-01.

Disturbed bulk samples are obtained from cuttings developed during boring operations. The bulk samples are selected for classification and testing purposes and may represent a mixture of soils within the noted depths. Recovered samples are placed in transport containers and returned to the laboratory for further classification and testing.

Logs of the borings showing the approximate depths and descriptions of the encountered soils, applicable geologic structures, recorded N-values, and the results of laboratory tests are presented in this appendix. The logs represent the interpretation of field logs and field tests as well as the interpolation of soil conditions between samples. The results of laboratory observations and tests are also included in the boring logs. The stratification lines recorded in the boring logs represent the approximate boundaries between the surface soil types. However, the actual transition between soil types may be gradual or varied.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS		LABORATORY CLASSIFICATION CRITERIA		GROUP SYMBOLS	PRIMARY DIVISIONS
COARSE GRAINED SOILS More than 50% retained on No. 200 sieve	GRAVELS More than 50% of coarse fraction retainined on No. 4 (4.75mm) sieve	Clean gravels (less than 5% fines*)	C_u greater than 4 and C_z between 1 and 3	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			Not meeting both criteria for GW	GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with fines (more than 12% fines*)	Atterberg limits plot below "A" line or plasticity index less than 4	GM	Silty gravels, gravel-sand-silt mixtures
			Atterberg limits plot below "A" line and plasticity index greater than 7	GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS More than 50% of coarse fraction passes No. 4 (4.75mm) sieve	Clean sand (less than 5% fines*)	C_u greater than 6 and C_z between 1 and 3	SW	Well graded sands, gravely sands, little or no fines
			Not meeting both criteria for SW	SP	Poorly graded sands and gravelly and sands, little or no fines
		Sand with fines (more than 12% fines*)	Atterberg limits plot below "A" line or plasticity index less than 4	SM	Silty sands, sand-silt mixtures
			Atterberg limits plot above "A" line and plasticity index greater than 7	SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS 50% or more passes No. 200 sieve	SILTS AND CLAYS (liquid limit less than 50)	Inorganic soil	PI < 4 or plots below "A"-line	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
		Inorganic soil	PI > 7 and plots on or above "A" line**	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic Soil	LL (oven dried)/LL (not dried) < 0.75	OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (liquid limit 50 or more)	Inorganic soil	Plots below "A" line	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		Inorganic soil	Plots on or above "A" line	CH	Inorganic clays of high plasticity, fat clays
		Organic Soil	LL (oven dried)/LL (not dried) < 0.75	OH	Organic silts and organic clays of high plasticity
Peat	Highly Organic	Primarily organic matter, dark in color, and organic odor		PT	Peat, muck and other highly organic soils

*Fines are those soil particles that pass the No. 200 sieve. For gravels and sands with between 5 and 12% fines, use of dual symbols is required (i.e. GW-GM, GW-GC, GP-GM, or GP-GC).

**If the plasticity index is between 4 and 7 and it plots above the "A" line, then dual symbols (i.e. CL-ML) are required.

CLASSIFICATIONS BASED ON PERCENTAGE OF FINES

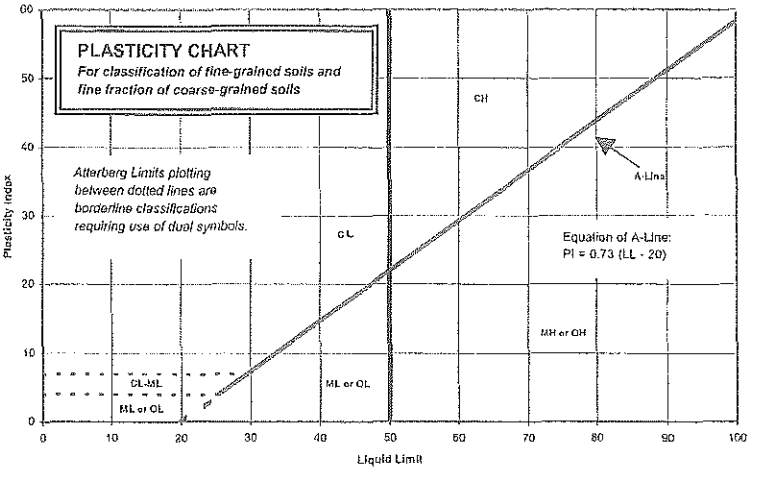
Less than 5%, Pass No. 200 (75mm)sieve	GW, GP, SW, SP
More than 12% Pass N. 200 (75 mm) sieve	GM, GC, SM, SC
5%-12% Pass No. 200 (75 mm) sieve	Borderline Classification requiring use of dual symbols

CONSISTENCY

CLAYS AND PLASTIC SILTS	STRENGTH TON/SQ. FT ++	BLOWS/ FOOT +
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	Over 4	Over 32

RELATIVE DENSITY

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/ FOOT +
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	Over 50



+ Number of blows of a 140-pound hammer falling 30-inches to drive a 2-inch O.D. (1-3/8-inch I.D.) split spoon (ASTM D1586).

++ Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D1586), pocket penetrometer, torvane, or visual observation.

Drilling Notes:

1. Sampling and blow counts
   a. California Modified – number of blows per foot of a 140 pound hammer falling 30 inches
   b. Standard Penetration Test – number of blows per 12 inches of a 140 pound hammer falling 30 inches

Types of Samples:
X – In-Situ
SPT - Standard Penetration
CA - California Modified
N - Nuclear Gauge
PO – Pocket Penetrometer (tons/sq.ft.)

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BORING LOG

BORING NO. B-1

JOB NO. SL07503-1

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	Cantinas	DRILL RIG:	CME 55
DRILLING LOCATION:	Support Lodging	HOLE DIAMETER:	4 Inches
DATE DRILLED:	October 8, 2010	SAMPLING METHOD:	SPT and CA
LOGGED BY:	JAP	HOLE ELEVATION:	Not Recorded

Depth of Groundwater: **Not Encountered** Boring Terminated At: **25.0 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	CLAYEY SAND: olive brown, slightly moist	SC											
-1													
-2	dense			A SPT	12	28							
-3	very dense												
-4				CA	41	54							
-5													
-6													
-7													
-8	SILTY SAND: light brown, slightly moist	SM											
-9	very dense			SPT	21	40							
-10				B									
-11													
-12													
-13													
-14				SPT	24	38							
-15													
-16													
-17	very dense												
-18													
-19				SPT	33	46							
-20													
-21													
-22													
-23													
-24				SPT	41	56							
-25													
-26													
-27													
-28													
-29													
-30													

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BORING LOG

BORING NO. B-2

JOB NO. SL07503-1

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	Cantinas	DRILL RIG:	CME 55
DRILLING LOCATION:	Support Lodging	HOLE DIAMETER:	4 Inches
DATE DRILLED:	October 8, 2010	SAMPLING METHOD:	SPT and CA
LOGGED BY:	JAP	HOLE ELEVATION:	Not Recorded

Depth of Groundwater: **Not Encountered** Boring Terminated At: **25.0 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SANDY SILT: dark brown, slightly moist	ML											
-1													
-2	very stiff			SPT	14	33							
-3													
-4	very stiff			SPT	18	43							
-5													
-6													
-7													
-8	very stiff												
-9				CA	40	42							
-10													
-11													
-12													
-13	hard												
-14				SPT	31	51							
-15	SILTY SAND: light brown, with gravel, dry to slightly moist	SM											
-16													
-17													
-18													
-19													
-20													
-21													
-22													
-23													
-24													
-25													
-26													
-27													
-28													
-29													
-30													

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BORING LOG

BORING NO. B-3

JOB NO. SL07503-1

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	Cantinas	DRILL RIG:	CME 55
DRILLING LOCATION:	Mission	HOLE DIAMETER:	4 Inches
DATE DRILLED:	October 8, 2010	SAMPLING METHOD:	SPT and CA
LOGGED BY:	JAP	HOLE ELEVATION:	Not Recorded

▼ Depth of Groundwater: **Not Encountered** Boring Terminated At: **15.0 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SILTY SAND: light brown, with gravel	SM											
-2	very dense			C SPT	22	52							
-4	very dense			CA	55	73							
-6	SILTY SAND: tan to white, silty sand to fine grained sandstone, dry, massive, slightly fractured, severly weathered in upper 3 feet to fresh	SM											
-9	very dense			SPT	24	45							
-14	very dense			CA	50/5"	76							

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BORING LOG

BORING NO. B-4

JOB NO. SL07503-1

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	Cantinas	DRILL RIG:	CME 55
DRILLING LOCATION:	Support Lodge	HOLE DIAMETER:	4 Inches
DATE DRILLED:	October 11, 2010	SAMPLING METHOD:	SPT
LOGGED BY:	JAP	HOLE ELEVATION:	Not Recorded

Depth of Groundwater: **Not Encountered** Boring Terminated At: **10.0 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SILTY SAND: light brown, with gravel	SM											
-1													
-2	very dense			SPT	12	28							
-3													
-4	very dense			SPT	18	43							
-5	SILTY SAND: tan to white, silty sand to fine grained sandstone, dry, massive, slightly fractured, severly weathered in upper 3 feet to fresh, moderately soft to moderately hard	SM											
-6													
-7													
-8													
-9	very dense			SPT	50/6"	166							
-10													
-11													
-12													
-13													
-14													
-15													
-16													
-17													
-18													
-19													
-20													
-21													
-22													
-23													
-24													
-25													
-26													
-27													
-28													
-29													
-30													

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BORING LOG

BORING NO. B-5

JOB NO. SL07503-1

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	Cantinas	DRILL RIG:	CME 55
DRILLING LOCATION:	Support Lodging	HOLE DIAMETER:	4 Inches
DATE DRILLED:	October 11, 2010	SAMPLING METHOD:	SPT
LOGGED BY:	JAP	HOLE ELEVATION:	Not Recorded

Depth of Groundwater: **Not Encountered** Boring Terminated At: **20.0 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	CLAYEY SAND: dark brown, slightly moist, some gravel	SC											
-2				SPT	18	43							
-3	very dense												
-4				SPT	21	50							
-5	very dense												
-9	very dense			SPT	31	59							
-13	SILTY SAND: tan to white, fine grained sandstone, massive, fresh, slightly fractured very dense	SM											
-14				SPT	38	63							
-19	very dense			SPT	29	40							
-20													

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BORING LOG

BORING NO. B-6

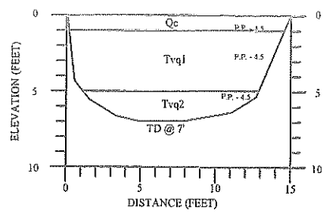
JOB NO. SL07503-1

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	Cantinas	DRILL RIG:	CME 55
DRILLING LOCATION:	Stables	HOLE DIAMETER:	4 Inches
DATE DRILLED:	October 11, 2010	SAMPLING METHOD:	SPT
LOGGED BY:	JAP	HOLE ELEVATION:	Not Recorded

Depth of Groundwater: **Not Encountered** Boring Terminated At: **5.0 Feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	CLAYEY SAND: light gray, dry	SC											
-1				F									
-2	SILTY SAND: white, fine grained sandstone, massive, fresh, slightly fractured	SM		SPT	34	81							
-3													
-4	very dense			SPT	50/5"								
-5													
-6													
-7													
-8													
-9													
-10													
-11													
-12													
-13													
-14													
-15													
-16													
-17													
-18													
-19													
-20													
-21													
-22													
-23													
-24													
-25													
-26													
-27													
-28													
-29													
-30													

T-1

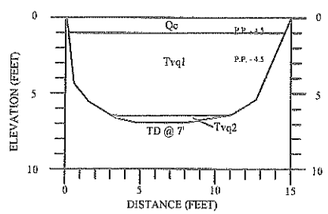


COLLUVIUM (Qc)
Sandy SILT, light gray, dry

VAQUEROS SANDSTONE (Tvq1)
Medium-Grained Sandstone to Claystone, light reddish brown to light gray, slightly moist, severely weathered, moderately soft, massive

VAQUEROS SANDSTONE (Tvq2)
Coarse-Grained Sandstone to Claystone, tan to white, S.S.- massive, moderate hard, fresh, Claystone - slight to moderately weathered, moderately soft, moderately fractured

T-2

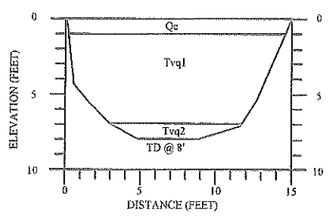


COLLUVIUM (Qc)
Sandy SILT, light gray, dry

VAQUEROS SANDSTONE (Tvq1)
Medium-Grained Sandstone to Claystone, yellowish brown, slightly moist, severely weathered, moderately soft, massive

VAQUEROS SANDSTONE (Tvq2)
Coarse-Grained Sandstone to Claystone, tan to white, S.S.- massive, moderate hard, fresh, Claystone - slight to moderately weathered, moderately soft, moderately fractured

T-3



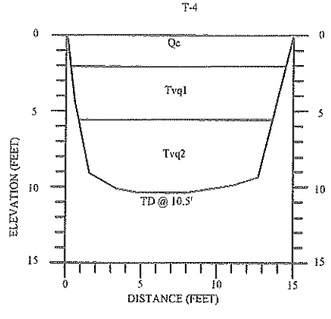
COLLUVIUM (Qc)
Sandy SILT, light gray, dry

VAQUEROS SANDSTONE (Tvq1)
Medium-Grained Sandstone to Claystone, yellowish brown, slightly moist, severely weathered, moderately soft, massive

VAQUEROS SANDSTONE (Tvq2)
Medium-Grained Sandstone, yellowish brown, massive, moderate hard, moderately weathered

P.P. = Pocket Pen

T-4

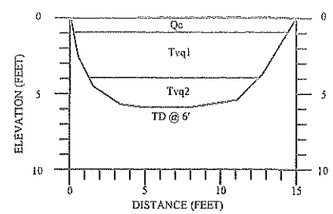


COLLUVIUM (Qc)
Sandy SILT, light gray, dry

VAQUEROS SANDSTONE (Tvq1)
Sandy CLAY, dark brown, dense, slightly moist, mottled

VAQUEROS SANDSTONE (Tvq2)
Medium-Grained Sandstone, tan to light gray, massive, moderately to severely weathered, moderately soft, roots throughout, some caliche

T-5

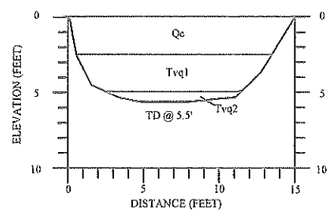


COLLUVIUM (Qc)
Sandy SILT, light gray, dry

VAQUEROS SANDSTONE (Tvq1)
Sandy CLAY, dark brown, dense, slightly moist, mottled

VAQUEROS SANDSTONE (Tvq2)
Siltstone, gray, massive, friable, fresh to slightly weathered, moderately hard

T-6

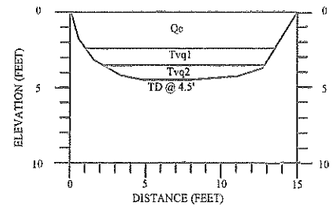


COLLUVIUM (Qc)
Sandy SILT, light brown, dry

VAQUEROS SANDSTONE (Tvq1)
Sandy CLAY, dark brown, dense, slightly moist, mottled

VAQUEROS SANDSTONE (Tvq2)
Sandstone, white to light gray, massive, slightly to moderately weathered, moderately soft

T-7

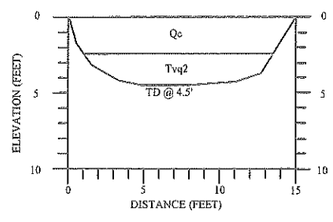


COLLUVIUM (Qc)
Sandy SILT, light gray, dry

VAQUEROS SANDSTONE (Tvq1)
Sandy CLAY, dark brown, dense, slightly moist, mottled

VAQUEROS SANDSTONE (Tvq2)
Siltstone, gray, massive, friable, fresh to slightly weathered, moderately hard

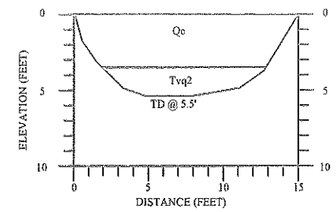
T-8



COLLUVIUM (Qc)
Sandy CLAY, light brown, dry to slightly moist

VAQUEROS SANDSTONE (Tvq2)
Medium- to Coarse-Grained Sandstone, white, massive, fresh to slightly weathered, moderately soft

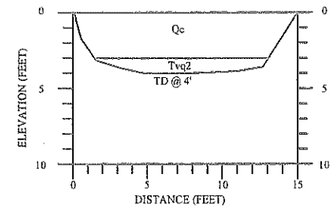
T-9



COLLUVIUM (Qc)
Sandy CLAY, light grayish brown, dry to slightly moist, large expansion cracks

VAQUEROS SANDSTONE (Tvq2)
Claystone, dark grayish brown to reddish brown, friable, fresh to slightly weathered, moderately soft, caliche

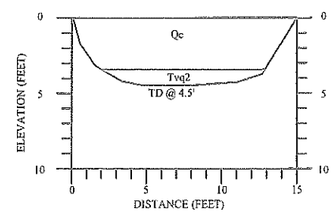
T-10



COLLUVIUM (Qc)
Sandy SILT, light gray, dry

VAQUEROS SANDSTONE (Tvq2)
Siltstone and Fine-grained sandstone, tan to light gray, fresh to slightly weathered, moderately soft

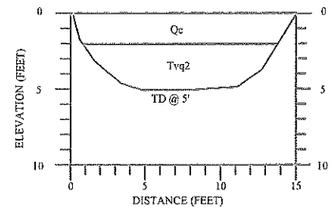
T-11



COLLUVIUM (Qc)
Sandy SILT, light gray, dry

VAQUEROS SANDSTONE (Tvq2)
Siltstone and Fine-grained sandstone, tan to light gray, fresh to slightly weathered, moderately soft, friable

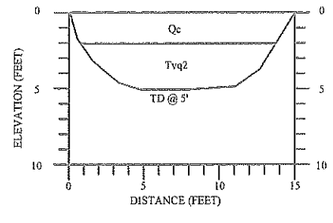
T-12



COLLUVIUM (Qc)
Sandy SILT, light grayish brown, dry

VAQUEROS SANDSTONE (Tvq2)
Medium-grained Sandstone, light brown to white, fresh moderately soft, massive

T-13



COLLUVIUM (Qc)
Sandy SILT, light grayish brown, dry

VAQUEROS SANDSTONE (Tvq2)
Medium-grained Sandstone, light brown to white, fresh moderately soft, massive

LOGS
1

PROJECT
SL07503-1

TRENCH LOGS

CANTINAS RANCH, LYNCH CANYON ROAD,
LAKE NACIMIENTO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA

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JEFFREY FROST
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APPENDIX B

Laboratory Testing

Soil Test Reports

LABORATORY TESTING

This appendix includes a discussion of the test procedures and the laboratory test results performed as part of this investigation. The purpose of the laboratory testing is to assess the engineering properties of the soil materials at the Site. The laboratory tests are performed using the currently accepted test methods, when applicable, of the American Society for Testing and Materials (ASTM).

Undisturbed and disturbed bulk samples used in the laboratory tests are obtained from various locations during the course of the field exploration, as discussed in **Appendix A** of this report. Each sample is identified by sample letter and depth. The Unified Soils Classification System is used to classify soils according to their engineering properties. The various laboratory tests performed are described below:

Expansion Index of Soils (ASTM D4829-03) is conducted in accordance with the ASTM test method and the California Building Code Standard, and are performed on representative bulk and undisturbed soil samples. The purpose of this test is to evaluate expansion potential of the site soils due to fluctuations in moisture content. The sample specimens are placed in a consolidometer, surcharged under a 144-psf vertical confining pressure, and then inundated with water. The amount of expansion is recorded over a 24-hour period with a dial indicator. The expansion index is calculated by determining the difference between final and initial height of the specimen divided by the initial height.

Laboratory Compaction Characteristics of Soil Using Modified Effort (ASTM D1557-07) is performed to determine the relationship between the moisture content and density of soils and soil-aggregate mixtures when compacted in a standard size mold with a 10-lbf hammer from a height of 18 inches. The test is performed on a representative bulk sample of bearing soil near the estimated footing depth. The procedure is repeated on the same soil sample at various moisture contents sufficient to establish a relationship between the maximum dry unit weight and the optimum water content for the soil. The data, when plotted, represents a curvilinear relationship known as the moisture density relations curve. The values of optimum water content and modified maximum dry unit weight can be determined from the plotted curve.

Liquid Limit, Plastic Limit, and Plasticity Index of Soils (ASTM D4318-00) are the water contents at certain limiting or critical stages in cohesive soil behavior. The liquid limit (LL or W_L) is the lower limit of viscous flow, the plastic limit (PL or W_P) is the lower limit of the plastic stage of clay and plastic index (PI or I_P) is a range of water content where the soil is plastic. The Atterberg Limits are performed on samples that have been screened to remove any material retained on a No. 40 sieve. The liquid limit is determined by performing trials in which a portion of the sample is spread in a brass cup, divided in two by a grooving tool, and then allowed to flow together from the shocks caused by repeatedly dropping the cup in a standard mechanical device. To determine the Plastic Limit a small portion of plastic soil is alternately pressed together and rolled into a 1/8-inch diameter thread. This process is continued until the water content of the sample is reduced to a point at which the thread crumbles and can no longer be pressed together and re-rolled. The water content of the soil at this point is reported as the plastic limit. The plasticity index is calculated as the difference between the liquid limit and the plastic limit.

Direct Shear Tests of Soils Under Consolidated Drained Conditions (ASTM D3080-04) is performed on undisturbed and remolded samples representative of the foundation material. The samples are loaded with a predetermined normal stress and submerged in water until saturation is achieved. The samples are then sheared horizontally at a controlled strain rate allowing partial drainage. The shear stress on the sample is recorded at regular strain intervals. This test determines the resistance to deformation, which is shear strength, inter-particle attraction or cohesion c, and resistance to interparticle slip called the angle of internal friction ϕ.

Density of Soil in Place by the Drive-Cylinder Method (ASTM D2937-04) and **Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass** (ASTM D2216-05) are used to obtain values of in-place water content and in-place density. Undisturbed samples, brought from the field to the laboratory, are weighed, the volume is calculated, and they are placed in the oven to dry. Once the samples have been dried, they are weighed again to determine the water content, and the in-place density is then calculated. The moisture density tests allow the water content and in-place densities to be obtained at required depths.

GeoSolutions, Inc.	SOILS REPORT	(805) 543-8539

Project:	Lynch Canyon Road			Date Tested:	October 14, 2010
Client:				Project #:	SL07503-1
Sample:	C	Depth:	2.0 Feet	Lab #:	14584
Location:	B-3			Sample Date:	October 11, 2010
				Sampled By:	JP

Soil Classification
ASTM D2487-06, D2488-06

Result: Dark Olive Brown Clayey SAND

Specification: SC

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4		
No. 8		
No. 16		
No. 30		
No. 50		
No. 100		
No. 200		

Sand Equivalent Cal 217 (11/1999)

1		SE
2		
3		
4		

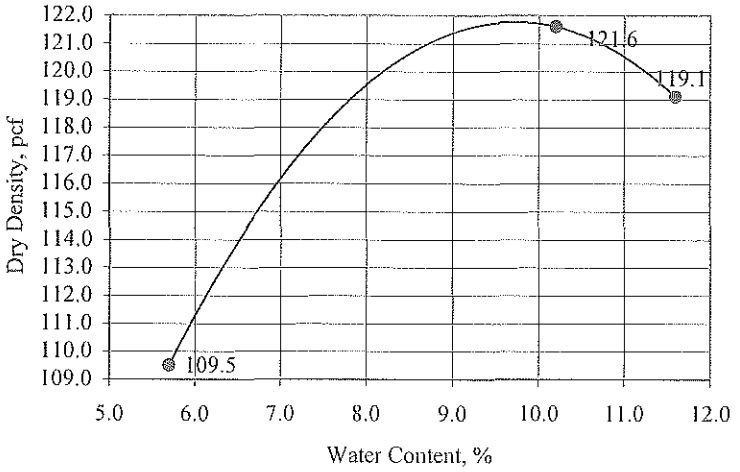
Plasticity Index
ASTM D4318-05

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	

Expansion Index
ASTM D4829-08

Expansion Index:	**21**
Expansion Potential:	Low
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve =

Trial #	1	2	3	4
Water Content:	5.7	10.2	11.6	
Dry Density:	109.5	121.6	119.1	

Maximum Dry Density, pcf:	**121.8**
Optimum Water Content, %:	**9.7**

Moisture-Density ASTM D2937-04, ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

Report By: Aaron Eichman

GeoSolutions, Inc. **SOILS REPORT** (805) 543-8539

Project:	Lynch Canyon Road			Date Tested:	November 8, 2010
Client:				Project #:	SL07503-1
Sample:	A	Depth:	2.0 Feet	Lab #:	14584
Location:	B-1			Sample Date:	October 11, 2010
				Sampled By:	JP

Soil Classification
ASTM D2487-06, D2488-06

Result: Dark Olive Brown Clayey SAND

Specification: SC

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	99	
No. 8	98	
No. 16	95	
No. 30	90	
No. 50	78	
No. 100	58	
No. 200	46.6	

Sand Equivalent Cal 217 (11/1999)

1		SE
2		
3		
4		

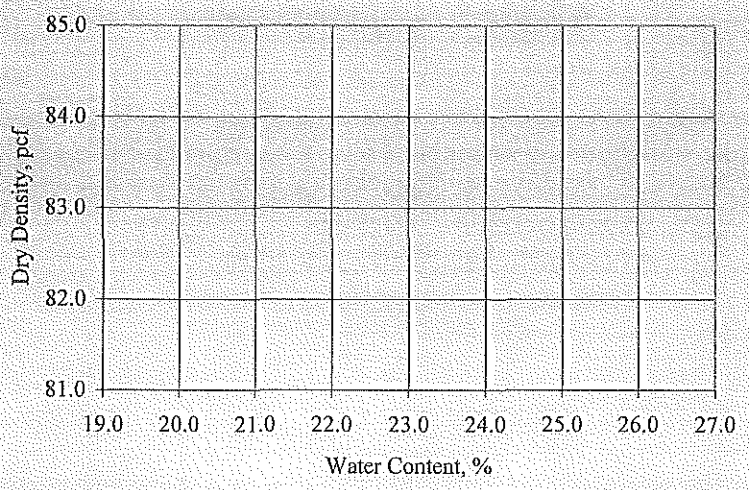
Plasticity Index
ASTM D4318-05

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	

Expansion Index
ASTM D4829-08

Expansion Index:	19
Expansion Potential:	Very Low
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve =

Trial #	1	2	3	4
Water Content:				
Dry Density:				

Maximum Dry Density, pcf:	
Optimum Water Content, %:	

Moisture-Density ASTM D2937-04, ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

Report By: Aaron Eichman

GeoSolutions, Inc. **SOILS REPORT** (805) 543-8539

Project:	Lynch Canyon Road			Date Tested:	November 9, 2010
Client:				Project #:	SL07503-1
Sample:	D	Depth:	2.0 Feet	Lab #:	14584
Location:	B-4			Sample Date:	October 11, 2010
				Sampled By:	JP

Soil Classification
ASTM D2487-06, D2488-06

Result: Light Olive Brown Clayey SAND

Specification: SC

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	98	
No. 8	96	
No. 16	94	
No. 30	90	
No. 50	72	
No. 100	46	
No. 200	36.4	

Sand Equivalent Cal 217 (11/1999)

1		SE
2		
3		
4		

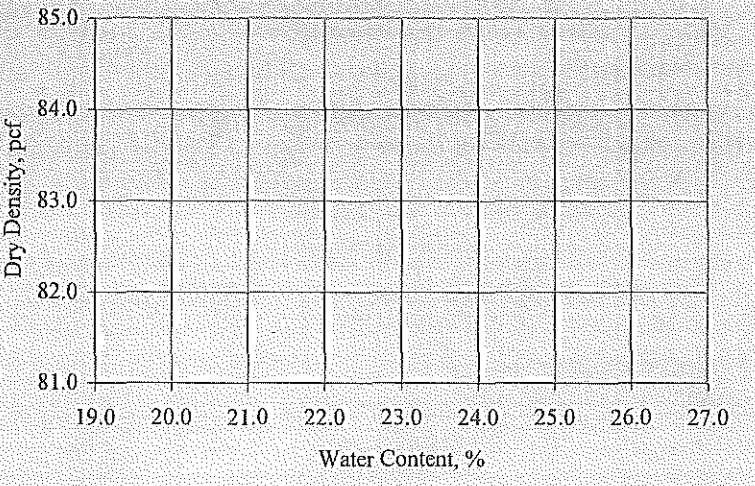
Plasticity Index
ASTM D4318-05

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	

Expansion Index
ASTM D4829-08

Expansion Index:	21
Expansion Potential:	Low
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve =

Trial #	1	2	3	4
Water Content:				
Dry Density:				

Maximum Dry Density, pcf:	
Optimum Water Content, %:	

Moisture-Density ASTM D2937-04, ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

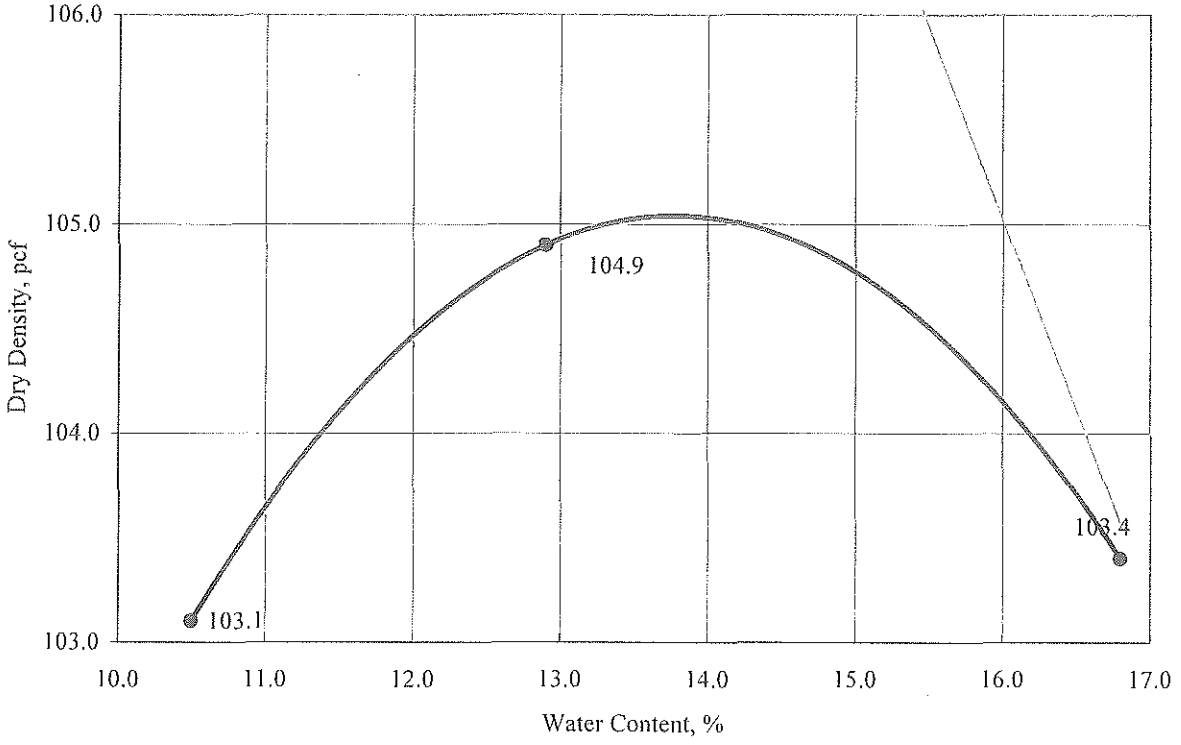
Report By: Aaron Eichman

GeoSolutions, Inc.

LABORATORY COMPACTION REPORT
ASTM D1557-07

(805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	November 2, 2010
Client:		Project #:	SL07503-1
Sample #:	E Depth: 9.0 Feet	Lab #:	14584
Source:	B-4	Sample Date:	October 11, 2010
Material:	Dark Olive Brown Sandy CLAY	Sampled By:	JP



ASTM Test Designation: [ ] D 698 [x] D 1557 Method: [ ] A [x] B [ ] C

100 % Saturation Curve-Estimated Specific Gravity: 2.3

Laboratory Test Results

Trial #	1	2	3	4
Water Content,%	10.5	12.9	16.8	
Dry Density, pcf	103.1	104.9	103.4	

MAXIMUM DRY DENSITY, pcf:	105.1	OPTIMUM MOISTURE, %:	13.8

Report By: Aaron Eichman

GeoSolutions, Inc. **SOILS REPORT** (805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	November 2, 2010
Client:		Project #:	SL07503-1
Sample: G	Depth: 1.0 Foot	Lab #:	14584
Location:	T-8	Sample Date:	October 11, 2010
		Sampled By:	JP

Soil Classification
ASTM D2487-06, D2488-06

Result: Dark Brown Clayey SAND

Specification: SC

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	100	
No. 8	99	
No. 16	97	
No. 30	90	
No. 50	77	
No. 100	55	
No. 200	45.2	

Sand Equivalent Cal 217 (11/1999)

1		SE
2		
3		
4		

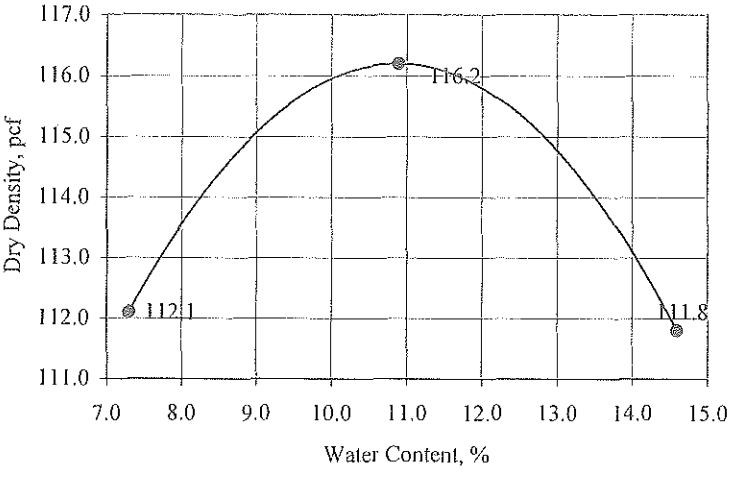
Plasticity Index
ASTM D4318-05

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	

Expansion Index
ASTM D4829-08

Expansion Index:	31
Expansion Potential:	Low
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve = 2.45

Trial #	1	2	3	4
Water Content:	7.3	10.9	14.6	
Dry Density:	112.1	116.2	111.8	

Maximum Dry Density, pcf:	**116.2**
Optimum Water Content, %:	**10.9**

Moisture-Density ASTM D2937-04, ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

Report By: Aaron Eichman

GeoSolutions, Inc. **SOILS REPORT** (805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	January 4, 2011
Client:		Project #:	SL07503-1
Sample: I	Depth: 1.0 Foot	Lab #:	14584
Location:	T-10	Sample Date:	October 11, 2010
		Sampled By:	JP

Soil Classification
ASTM D2487-06, D2488-06

Result: Dark Olive Brown Clayey SAND

Specification: SC

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	100	
No. 8	99	
No. 16	98	
No. 30	94	
No. 50	87	
No. 100	60	
No. 200	37.5	

Sand Equivalent Cal 217 (11/1999)

1		SE
2		
3		
4		

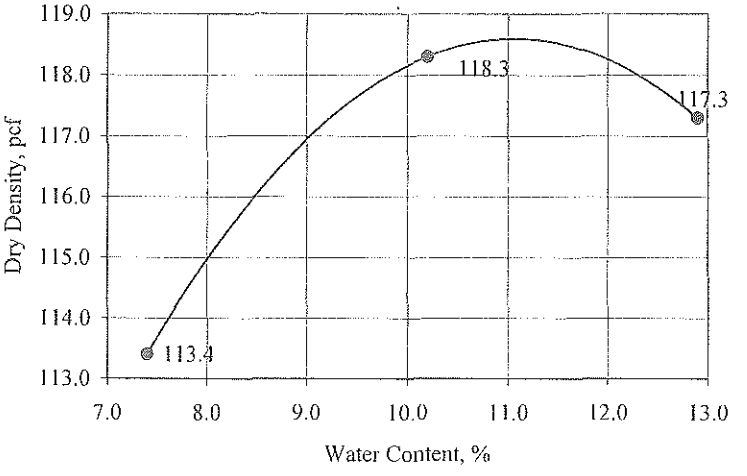
Plasticity Index
ASTM D4318-05

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	

Expansion Index
ASTM D4829-08

Expansion Index:	1
Expansion Potential:	Very Low
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve = 2.55

Trial #	1	2	3	4
Water Content:	7.4	10.2	12.9	
Dry Density:	113.4	118.3	117.3	

Maximum Dry Density, pcf:	**118.6**
Optimum Water Content, %:	**11.1**

Moisture-Density ASTM D2937-04, ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

Report By: Aaron Eichman

GeoSolutions, Inc. **SOILS REPORT** (805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	January 4, 2011
Client:		Project #:	SL07503-1
Sample: K	Depth: 1.0 Foot	Lab #:	14584
Location:	T-12	Sample Date:	October 11, 2010
		Sampled By:	JP

Soil Classification
ASTM D2487-06, D2488-06

Result: Dark Yellowish Brown Clayey SAND

Specification: SC

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	100	
No. 8	99	
No. 16	97	
No. 30	93	
No. 50	87	
No. 100	71	
No. 200	47.0	

Sand Equivalent Cal 217 (11/1999)

1		SE
2		
3		
4		

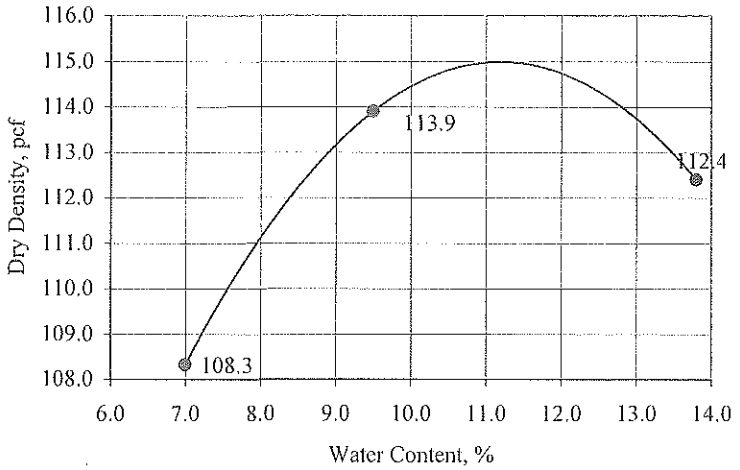
Plasticity Index
ASTM D4318-05

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	

Expansion Index
ASTM D4829-08

Expansion Index:	17
Expansion Potential:	Very Low
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve = 2.45

Trial #	1	2	3	4
Water Content:	7.0	9.5	13.8	
Dry Density:	108.3	113.9	112.4	

Maximum Dry Density, pcf:	**115.0**
Optimum Water Content, %:	**11.2**

Moisture-Density ASTM D2937-04, ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

Report By: Aaron Eichman

GeoSolutions, Inc. **SOILS REPORT** (805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	December 7, 2010
Client:		Project #:	SL07503-1
Sample: L	Depth: 4.0 Feet	Lab #:	14584
Location:	T-4	Sample Date:	October 11, 2010
		Sampled By:	JP

Soil Classification
ASTM D2487-06, D2488-06

Result: Yellowish Brown Sandy CLAY

Specification: CL

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	100	
No. 8	100	
No. 16	98	
No. 30	91	
No. 50	80	
No. 100	64	
No. 200	55.0	

Sand Equivalent Cal 217 (11/1999)

1		SE
2		
3		
4		

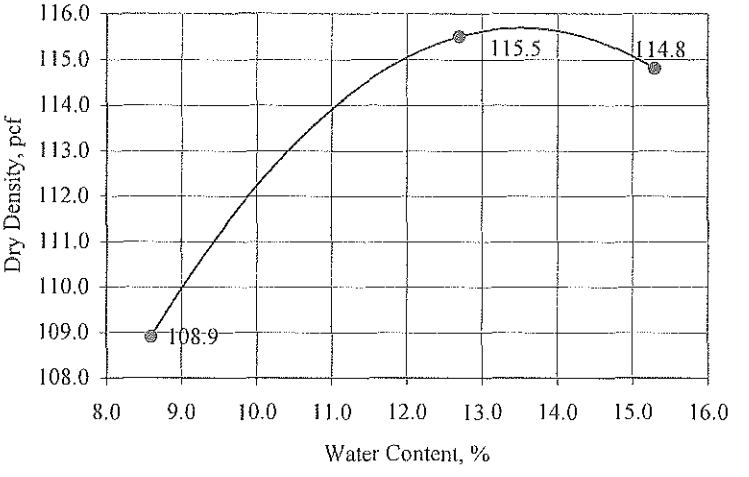
Plasticity Index
ASTM D4318-05

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	

Expansion Index
ASTM D4829-08

Expansion Index:	58
Expansion Potential:	Medium
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve = 2.6

Trial #	1	2	3	4
Water Content:	8.6	12.7	15.3	
Dry Density:	108.9	115.5	114.8	

Maximum Dry Density, pcf:	115.7
Optimum Water Content, %:	13.5

Moisture-Density ASTM D2937-04, ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

Report By: Aaron Eichman

GeoSolutions, Inc. **SOILS REPORT** (805) 543-8539

Project:	Lynch Canyon Road			Date Tested:	January 4, 2011
Client:				Project #:	SL07503-1
Sample:	M	Depth:	1.0 Foot	Lab #:	14584
Location:	T-6			Sample Date:	October 11, 2010
				Sampled By:	JP

Soil Classification
ASTM D2487-06, D2488-06

Result: Dark Yellowish Brown Silty SAND

Specification: SM

Sieve Analysis
ASTM D422-63R02

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	100	
No. 8	99	
No. 16	96	
No. 30	89	
No. 50	68	
No. 100	45	
No. 200	32.3	

Sand Equivalent Cal 217 (11/1999)

1		SE
2		
3		
4		

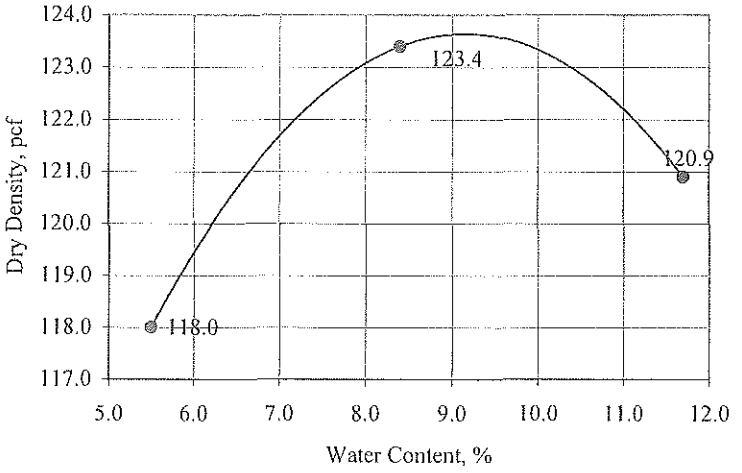
Plasticity Index
ASTM D4318-05

Liquid Limit:	
Plastic Limit:	
Plasticity Index:	

Expansion Index
ASTM D4829-08

Expansion Index:	0
Expansion Potential:	Very Low
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557-07



Mold ID	n/a	Mold Diameter, ins.	4.00
No. of Layers	5	Weight of Rammer, lbs.	10.00
No. of Blows	25		

Estimated Specific Gravity for 100% Saturation Curve = 2.55

Trial #	1	2	3	4
Water Content:	5.5	8.4	11.7	
Dry Density:	118.0	123.4	120.9	

Maximum Dry Density, pcf:	123.7
Optimum Water Content, %:	9.1

Moisture-Density ASTM D2937-04, ASTM D2216-05

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description

Report By: Aaron Eichman

GeoSolutions, Inc.

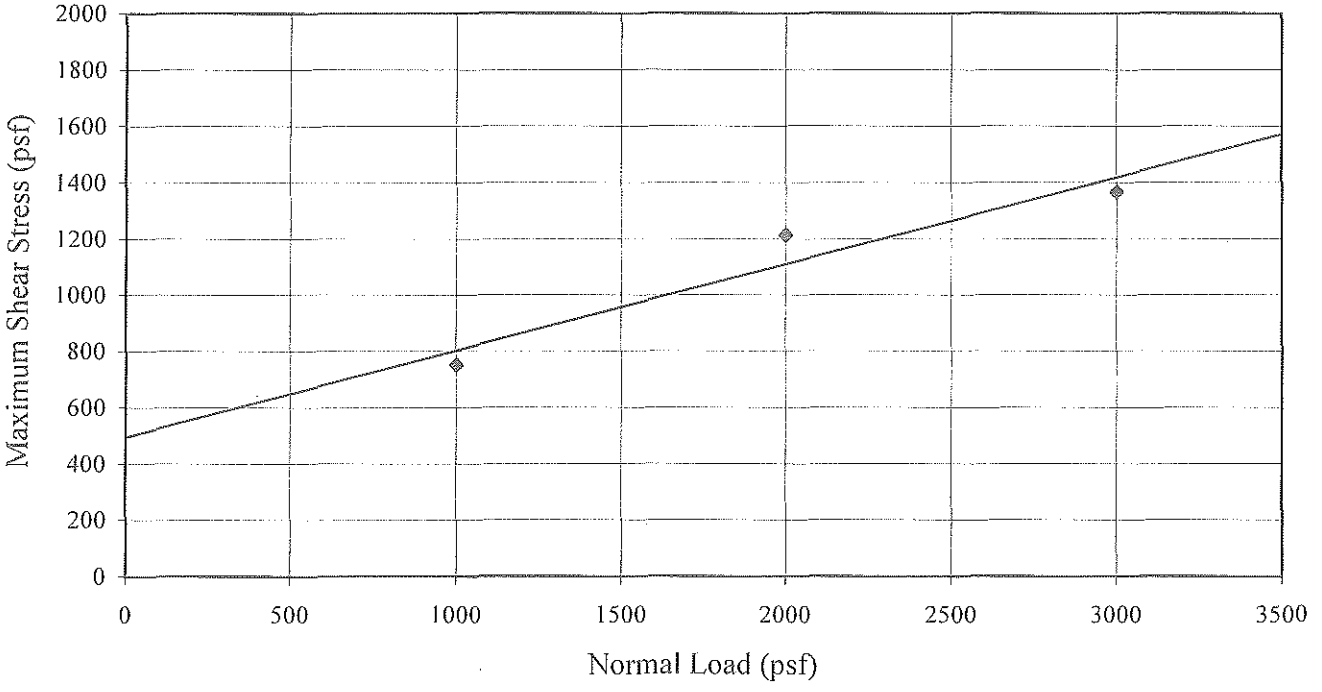
DIRECT SHEAR TEST REPORT
ASTM D3080-04

(805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	January 4, 2011
Client:		Project #:	SL07503-1
Sample #:	B-1 @ 4' Depth: 4.0 Feet	Lab #:	14584
Location:	B-1	Sample Date:	October 11, 2010
Material:	Dark Olive Brown Clayey SAND	Sampled By:	JP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	751	21.3	106.4	-
2	-	-	2000	1213	19.5	107.8	-
3	-	-	3000	1366	18.8	114.0	-
4							
5							



The test specimens were in-situ samples.

Angle of Internal Friction (In-Situ), Phi:	17.1 °
Cohesion (In-Situ), C:	495 psf

Report By: Aaron Eichman

GeoSolutions, Inc.

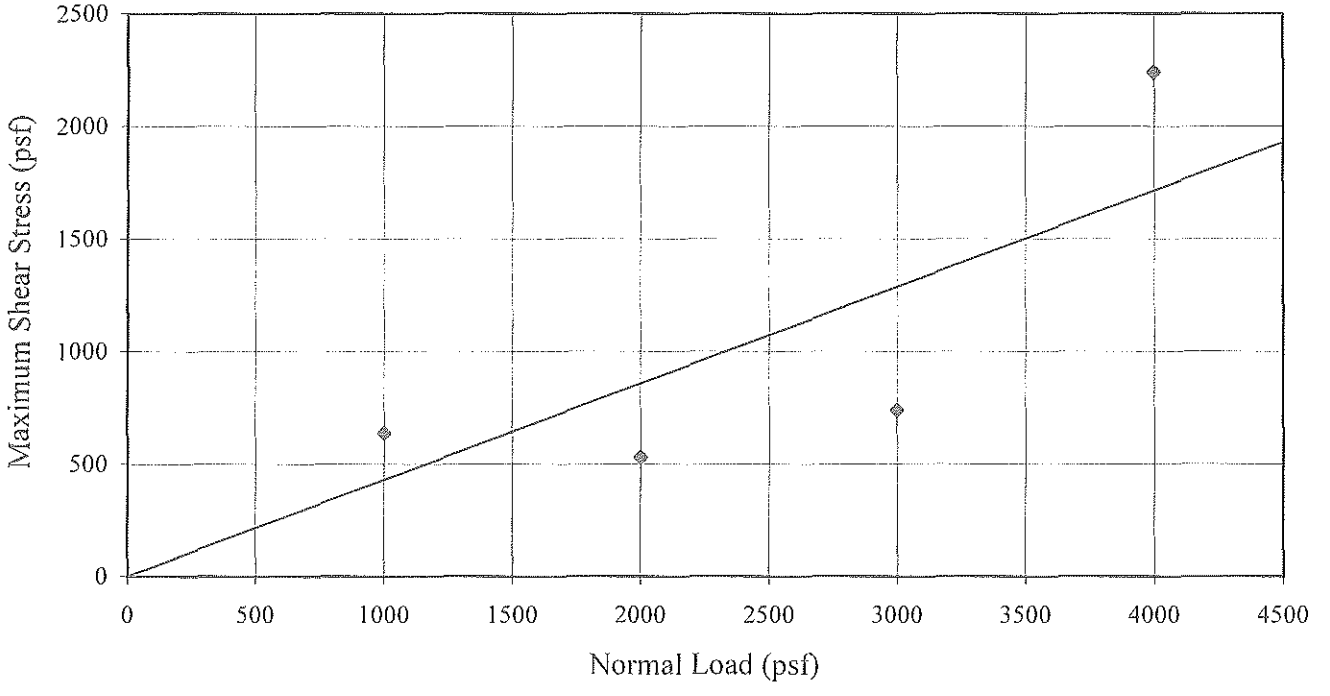
DIRECT SHEAR TEST REPORT
ASTM D3080-04

(805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	January 4, 2011
Client:		Project #:	SL07503-1
Sample #:	B-3 @ 4' Depth: 4.0 Feet	Lab #:	14584
Location:	B-3	Sample Date:	October 11, 2010
Material:	Light Olive Brown Clayey SAND with Gravel	Sampled By:	JP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	636	22.4	97.2	-
2	-	-	2000	529	20.7	105.4	-
3	-	-	3000	738	20.7	115.3	-
4	-	-	4000	2239	17.0	88.0	-
5							



The test specimens were in-situ samples.

Angle of Internal Friction (In-Situ), Phi:	26.6 °
Cohesion (In-Situ), C:	0 psf

Report By: Aaron Eichman

GeoSolutions, Inc.

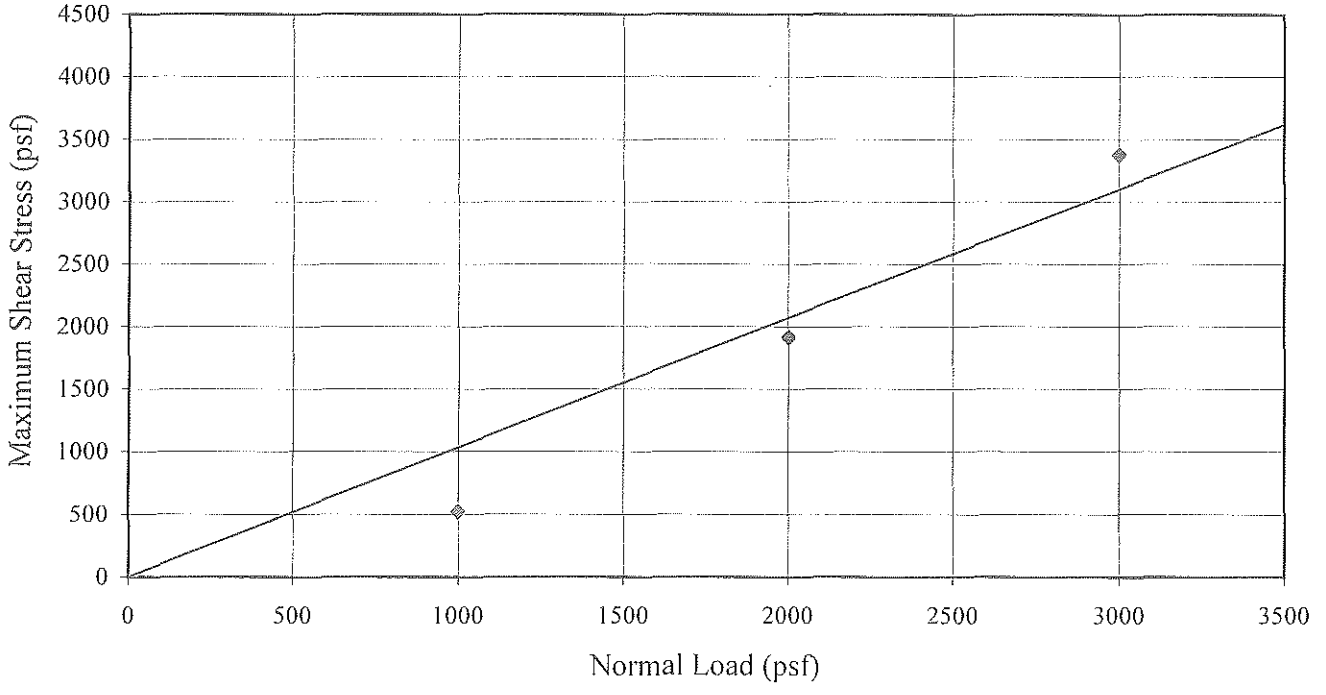
DIRECT SHEAR TEST REPORT
ASTM D3080-04

(805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	January 4, 2011
Client:		Project #:	SL07503-1
Sample #:	B-3 @ 14' Depth: 14.0 Feet	Lab #:	14584
Location:	B-3	Sample Date:	October 11, 2010
Material:	Dark Yellowish Brown Clayey SAND	Sampled By:	JP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	523	32.6	59.9	-
2	-	-	2000	1912	33.9	92.0	-
3	-	-	3000	3377	29.9	95.2	-
4							
5							



The test specimens were in-situ samples.

Angle of Internal Friction (In-Situ), Phi:	55.0 °
Cohesion (In-Situ), C:	0 psf

Report By: Aaron Eichman

GeoSolutions, Inc.

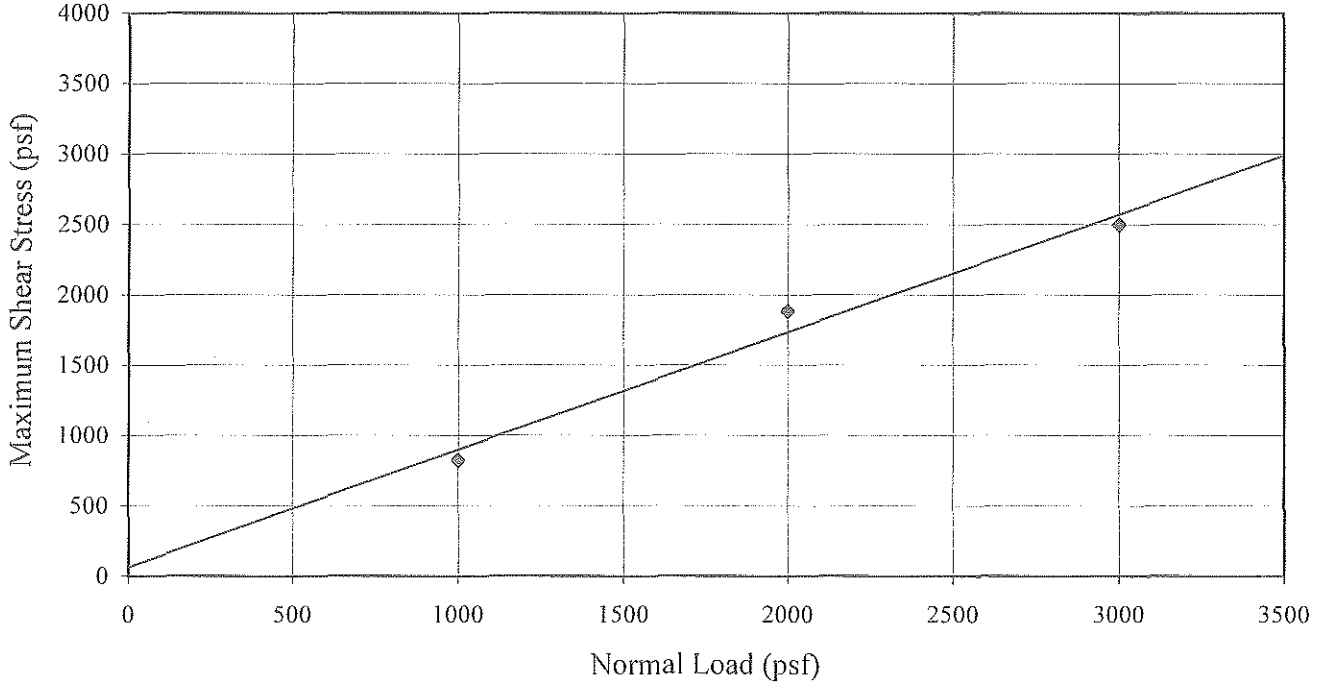
DIRECT SHEAR TEST REPORT
ASTM D3080-04

(805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	January 4, 2011
Client:		Project #:	SL07503-1
Sample #:	T-6 @ 6' Depth: 2.0 Feet	Lab #:	14584
Location:	T-6	Sample Date:	October 11, 2010
Material:	Brown Silty SAND	Sampled By:	JP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	825	20.0	91.1	-
2	-	-	2000	1883	19.0	91.8	-
3	-	-	3000	2496	17.9	92.8	-
4							
5							



The test specimens were in-situ samples.

Angle of Internal Friction (In-Situ), Phi:	39.9 °
Cohesion (In-Situ), C:	63.7 psf

Report By: Aaron Eichman

GeoSolutions, Inc.

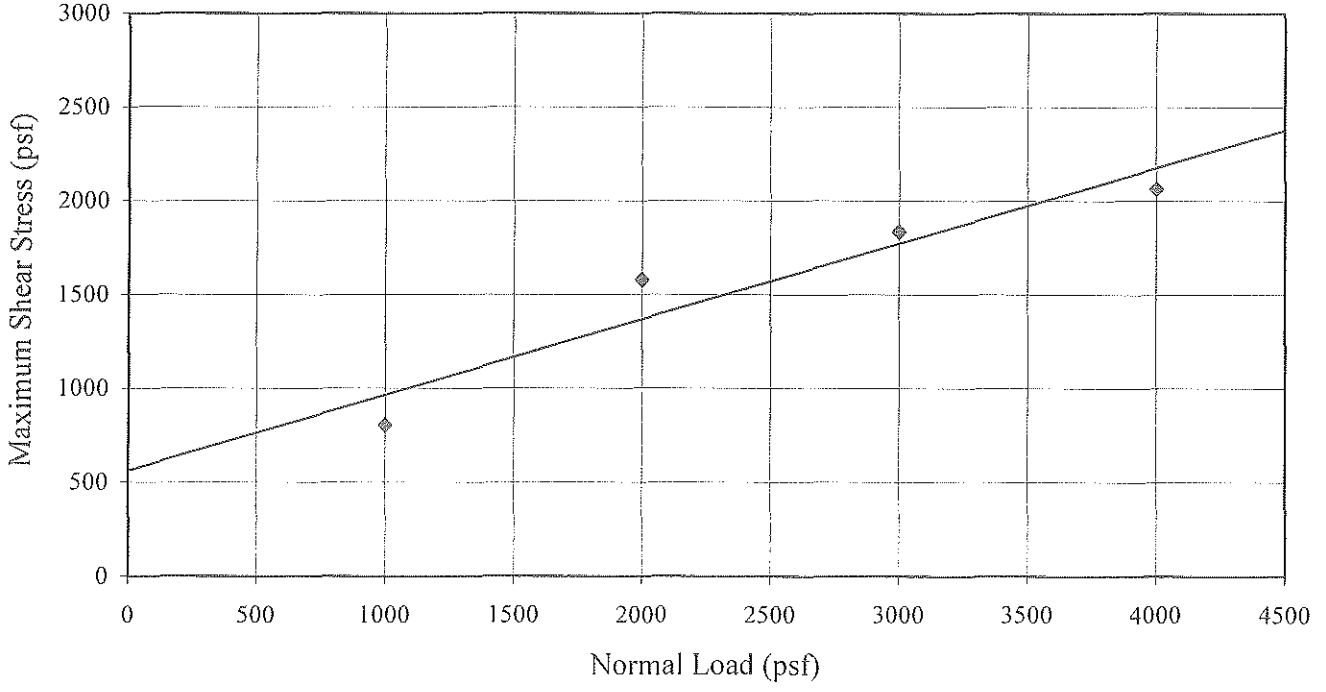
DIRECT SHEAR TEST REPORT
ASTM D3080-04

(805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	January 4, 2011
Client:		Project #:	SL07503-1
Sample #:	T-8 @ 1' Depth: 1.0 Foot	Lab #:	14584
Location:	T-8	Sample Date:	October 11, 2010
Material:	Very Dark Grayish Brown Clayey SAND	Sampled By:	JP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	803	25.7	83.2	-
2	-	-	2000	1579	22.7	87.7	-
3	-	-	3000	1836	20.7	89.5	-
4	-	-	4000	2063	21.8	81.1	-
5							



The test specimens were in-situ samples.

Angle of Internal Friction (In-Situ), Phi:	22.0 °
Cohesion (In-Situ), C:	561 psf

Report By: Aaron Eichman

GeoSolutions, Inc.

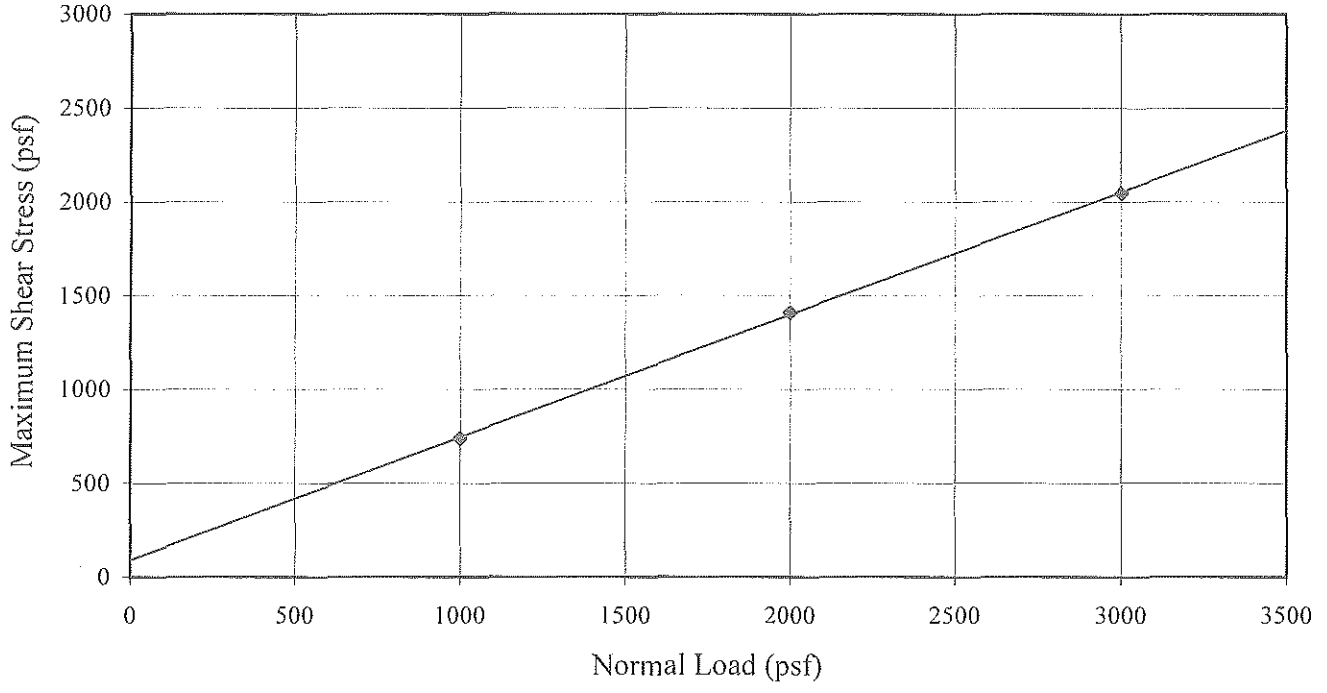
DIRECT SHEAR TEST REPORT
ASTM D3080-04

(805) 543-8539

Project:	Lynch Canyon Road			Date Tested:	January 4, 2011
Client:				Project #:	SL07503-1
Sample #:	T-10 @ 1'	Depth:	1.0 Foot	Lab #:	14584
Location:	T-10			Sample Date:	October 11, 2010
Material:	Brown Silty SAND			Sampled By:	JP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	738	22.0	79.0	-
2	-	-	2000	1410	19.0	89.1	-
3	-	-	3000	2046	17.7	91.4	-
4							
5							



The test specimens were in-situ samples.

Angle of Internal Friction (In-Situ), Phi:	33.2 °
Cohesion (In-Situ), C:	90 psf

Report By: Aaron Eichman

GeoSolutions, Inc.

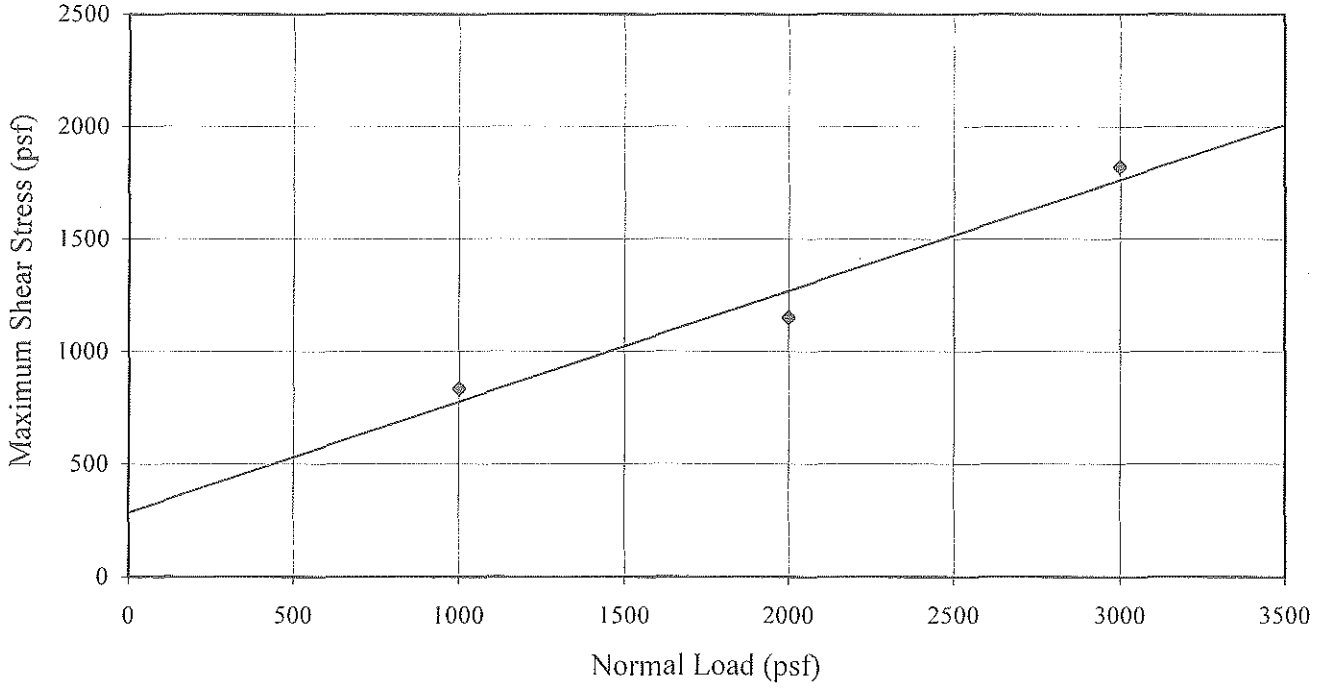
DIRECT SHEAR TEST REPORT
ASTM D3080-04

(805) 543-8539

Project:	Lynch Canyon Road	Date Tested:	January 4, 2011
Client:		Project #:	SL07503-1
Sample #:	T-12 @ 1' Depth: 1.0 Foot	Lab #:	14584
Location:	T-12	Sample Date:	October 11, 2010
Material:	Dark Yellowish Brown Clayey SAND	Sampled By:	JP

Test Data

Specimen Number	Void Ratio	Saturation, %	Normal Load, psf	Max Shear Stress, psf	Water Content, %	Dry Density, pcf	Relative Density*,%
1	-	-	1000	835	22.7	92.1	-
2	-	-	2000	1150	21.4	92.4	-
3	-	-	3000	1821	20.0	95.1	-
4							
5							



The test specimens were in-situ samples.

Angle of Internal Friction (In-Situ), Phi:	26.2 °
Cohesion (In-Situ), C:	282.7 psf

Report By: Aaron Eichman

APPENDIX C

Preliminary Grading Specifications

Key and Bench with Backdrain

PRELIMINARY GRADING SPECIFICATIONS

A. General

i. These preliminary specifications have been prepared for the subject site; GeoSolutions, Inc. should be consulted prior to the commencement of site work associated with site development to ensure compliance with these specifications.

ii. GeoSolutions, Inc. should be notified at least 72 hours prior to site clearing or grading operations on the property in order to observe the stripping of surface materials and to coordinate the work with the grading contractor in the field.

iii. These grading specifications may be modified and/or superseded by recommendations contained in the text of this report and/or subsequent reports.

iv. If disputes arise out of the interpretation of these grading specifications, the Soils Engineer shall provide the governing interpretation.

B. Obligation of Parties

i. The Soils Engineer should provide observation and testing services and should make evaluations to advise the client on geotechnical matters. The Soils Engineer should report the findings and recommendations to the client or the authorized representative.

ii. The client should be chiefly responsible for all aspects of the project. The client or authorized representative has the responsibility of reviewing the findings and recommendations of the Soils Engineer. During grading the client or the authorized representative should remain on-site or should remain reasonably accessible to all concerned parties in order to make decisions necessary to maintain the flow of the project.

iii. The contractor is responsible for the safety of the project and satisfactory completion of all grading and other operations on construction projects, including, but not limited to, earthwork in accordance with project plans, specifications, and controlling agency requirements.

C. Site Preparation

i. The client, prior to any site preparation or grading, should arrange and attend a meeting which includes the grading contractor, the design Structural Engineer, the Soils Engineer, representatives of the local building department, as well as any other concerned parties. All parties should be given at least 72 hours notice.

ii. All surface and sub-surface deleterious materials should be removed from the proposed building and pavement areas and disposed of off-site or as approved by the Soils Engineer. This includes, but is not limited to, any debris, organic materials, construction spoils, buried utility line, septic systems, building materials, and any other surface and subsurface structures within the proposed building areas. Trees designated for removal on the construction plans should be removed and their primary root systems grubbed under the observations of a representative of GeoSolutions, Inc. Voids left from site clearing should be cleaned and backfilled as recommended for structural fill.

iii. Once the Site has been cleared, the exposed ground surface should be stripped to remove surface vegetation and organic soil. A representative of GeoSolutions, Inc. should determine the required depth of stripping at the time of work being completed. Strippings may either be disposed of off-site or stockpiled for future use in landscape areas, if approved by the landscape architect.

D. Site Protection

i. Protection of the Site during the period of grading and construction should be the responsibility of the contractor.

ii. The contractor should be responsible for the stability of all temporary excavations.

iii. During periods of rainfall, plastic sheeting should be kept reasonably accessible to prevent unprotected slopes from becoming saturated. Where necessary during periods of rainfall, the contractor should install check-dams, de-silting basins, sand bags, or other devices or methods necessary to control erosion and provide safe conditions.

E. Excavations

i. Materials that are unsuitable should be excavated under the observation and recommendations of the Soils Engineer. Unsuitable materials include, but may not be limited to: 1) dry, loose, soft, wet, organic, or compressible natural soils; 2) fractured, weathered, or soft bedrock; 3) non-engineered fill; 4) other deleterious materials; and 5) materials identified by the Soils Engineer or Engineering Geologist.

ii. Unless otherwise recommended by the Soils Engineer and approved by the local building official, permanent cut slopes should not be steeper than 2:1 (horizontal to vertical). Final slope configurations should conform to section 1803 of the 2007 California Building Code unless specifically modified by the Soil Engineer/Engineering Geologist.

iii. The Soil Engineer/Engineer Geologist should review cut slopes during excavations. The contractor should notify the Soils Engineer/Engineer Geologist prior to beginning slope excavations.

F. Structural Fill

i. Structural fill should not contain rocks larger than 3 inches in greatest dimension, and should have no more than 15 percent larger than 2.5 inches in greatest dimension.

ii. Imported fill should be free of organic and other deleterious material and should have very low expansion potential, with a plasticity index of 12 or less. Before delivery to the Site, a sample of the proposed import should be tested in our laboratory to determine its suitability for use as structural fill.

G. Compacted Fill

i. Structural fill using approved import or native should be placed in horizontal layers, each approximately 8 inches in thickness before compaction. On-site inorganic soil or approved imported fill should be conditioned with water to produce a soil water content near optimum moisture and compacted to a minimum relative density of 90 percent based on ASTM D1557-07.

ii. Fill slopes should not be constructed at gradients greater than 2-to-1 (horizontal to vertical). The contractor should notify the Soils Engineer/Engineer Geologist prior to beginning slope excavations.

iii. If fill areas are constructed on slopes greater than 10-to-1 (horizontal to vertical), we recommend that benches be cut every 4 feet as fill is placed. Each bench shall be a minimum of 10 feet wide with a minimum of 2 percent gradient into the slope.

iv. If fill areas are constructed on slopes greater than 5-to-1, we recommend that the toe of all areas to receive fill be keyed a minimum of 24 inches into underlying dense material. Key depths are to be

observed and approved by a representative of GeoSolutions, Inc. Sub-drains shall be placed in the keyway and benches as required. See Detail A: Key and Bench with Backdrain.

H. Drainage

i. During grading, a representative of GeoSolutions, Inc. should evaluate the need for a sub-drain or back-drain system. Areas of observed seepage should be provided with sub-surface drains to release the hydrostatic pressures. Sub-surface drainage facilities may include gravel blankets, rock filled trenches or Multi-Flow systems or equal. The drain system should discharge in a non-erosive manner into an approved drainage area.

ii. All final grades should be provided with a positive drainage gradient away from foundations. Final grades should provide for rapid removal of surface water runoff. Ponding of water should not be allowed on building pads or adjacent to foundations. Final grading should be the responsibility of the contractor, general Civil Engineer, or architect.

iii. Concentrated surface water runoff within or immediately adjacent to the Site should be conveyed in pipes or in lined channels to discharge areas that are relatively level or that are adequately protected against erosion.

iv. Water from roof downspouts should be conveyed in solid pipes that discharge in controlled drainage localities. Surface drainage gradients should be planned to prevent ponding and promote drainage of surface water away from building foundations, edges of pavements and sidewalks. For soil areas we recommend that a minimum of 2 percent gradient be maintained.

v. Attention should be paid by the contractor to erosion protection of soil surfaces adjacent to the edges of roads, curbs and sidewalks, and in other areas where hard edges of structures may cause concentrated flow of surface water runoff. Erosion resistant matting such as Miramat, or other similar products, may be considered for lining drainage channels.

vi. Sub-drains should be placed in established drainage courses and potential seepage areas. The location of sub-drains should be determined after a review of the grading plan. The sub-drain outlets should extend into suitable facilities or connect to the proposed storm drain system or existing drainage control facilities. The outlet pipe should consist of a non-perforated pipe the same diameter as the perforated pipe.

I. Maintenance

i. Maintenance of slopes is important to their long-term performance. Precautions that can be taken include planting with appropriate drought-resistant vegetation as recommended by a landscape architect, and not over-irrigating, a primary source of surficial failures.

ii. Property owners should be made aware that over-watering of slopes is detrimental to long term stability of slopes.

J. Underground Facilities Construction

i. The attention of contractors, particularly the underground contractors, should be drawn to the State of California Construction Safety Orders for "Excavations, Trenches, Earthwork." Trenches or excavations greater than 5 feet in depth should be shored or sloped back in accordance with OSHA Regulations prior to entry.

ii. Bedding is defined as material placed in a trench up to 1 foot above a utility pipe and backfill is all material placed in the trench above the bedding. Unless concrete bedding is required around utility pipes, free-draining sand should be used as bedding. Sand to be used as bedding should be tested in our laboratory to verify its suitability and to measure its compaction characteristics. Sand bedding should be compacted by mechanical means to achieve at least 90 percent relative density based on ASTM D1557-07.

iii. On-site inorganic soils, or approved import, may be used as utility trench backfill. Proper compaction of trench backfill will be necessary under and adjacent to structural fill, building foundations, concrete slabs, and vehicle pavements. In these areas, backfill should be conditioned with water (or allowed to dry), to produce a soil water content of about 2 to 3 percent above the optimum value and placed in horizontal layers, each not exceeding 8 inches in thickness before compaction. Each layer should be compacted to at least 90 percent relative density based on ASTM D1557-07. The top lift of trench backfill under vehicle pavements should be compacted to the requirements given in report under Preparation of Paved Areas for vehicle pavement sub-grades. Trench walls must be kept moist prior to and during backfill placement.

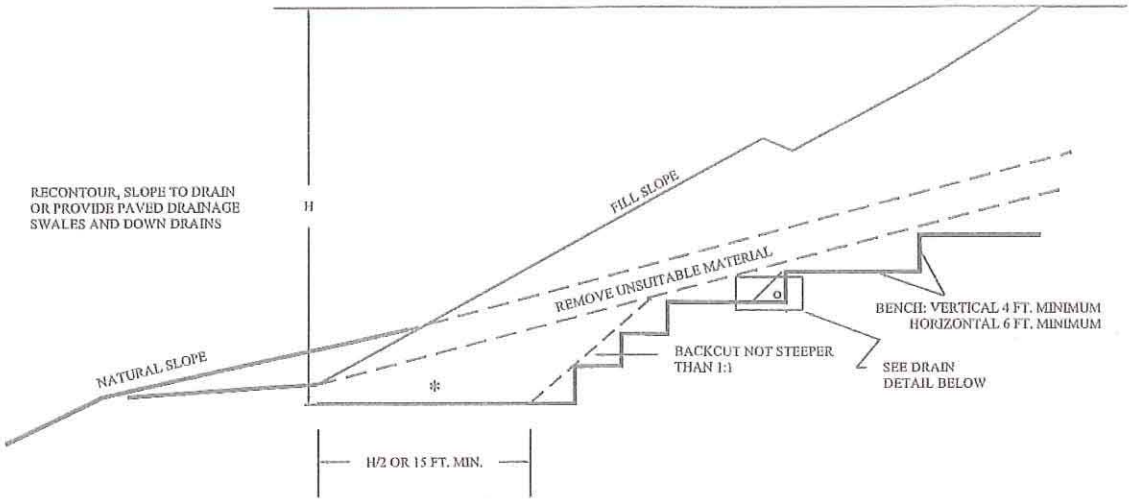
K. Completion of Work

i. After the completion of work, a report should be prepared by the Soils Engineer retained to provide such services in accordance with section 1803.5 of the 2007 CBC. The report should including locations and elevations of field density tests, summaries of field and laboratory tests, other substantiating data, and comments on any changes made during grading and their effect on the recommendations made in the approved Soils Engineering Report.

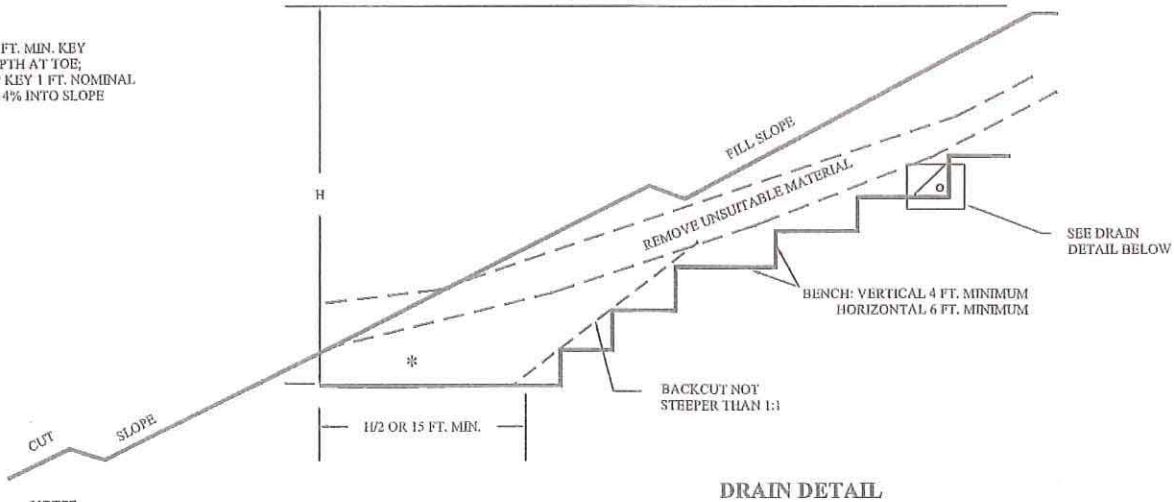
ii. Soils Engineers shall submit a statement that, to the best of their knowledge, the work within their area of responsibilities is in accordance with the approved soils engineering report and applicable provisions within section 1803 of the 2007 CBC.

END OF TEXT

FILL OVER NATURAL SLOPE



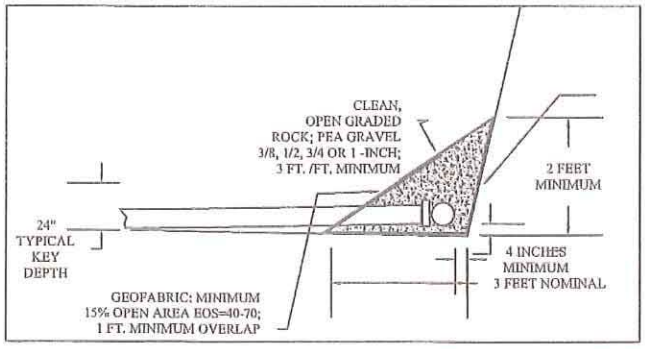
FILL OVER CUT SLOPE



NOTES:
1 - IF OVERFILLING AND CUTTING BACK TO GRADE IS ADOPTED, 15 FT. MIN. FILL WIDTH MAY BE REDUCED TO 12 FT. MIN. IN NO CASE SHOULD THE FILL WIDTH BE LESS THAN 1/2 THE HEIGHT OF FILL REMAINING.

1 - BACKDRAIN AS RECOMMEDNED BY GEOTECHNICAL CONSULTANT PER BUTTRESS BACKDRAIN DETAIL.

DRAIN DETAIL



GeoSolutions, Inc.
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San Luis Obispo, CA 93401
(805) 543-8539 Fax: (805) 543-2171

KEY AND BENCH WITH BACKDRAIN

DETAIL
A