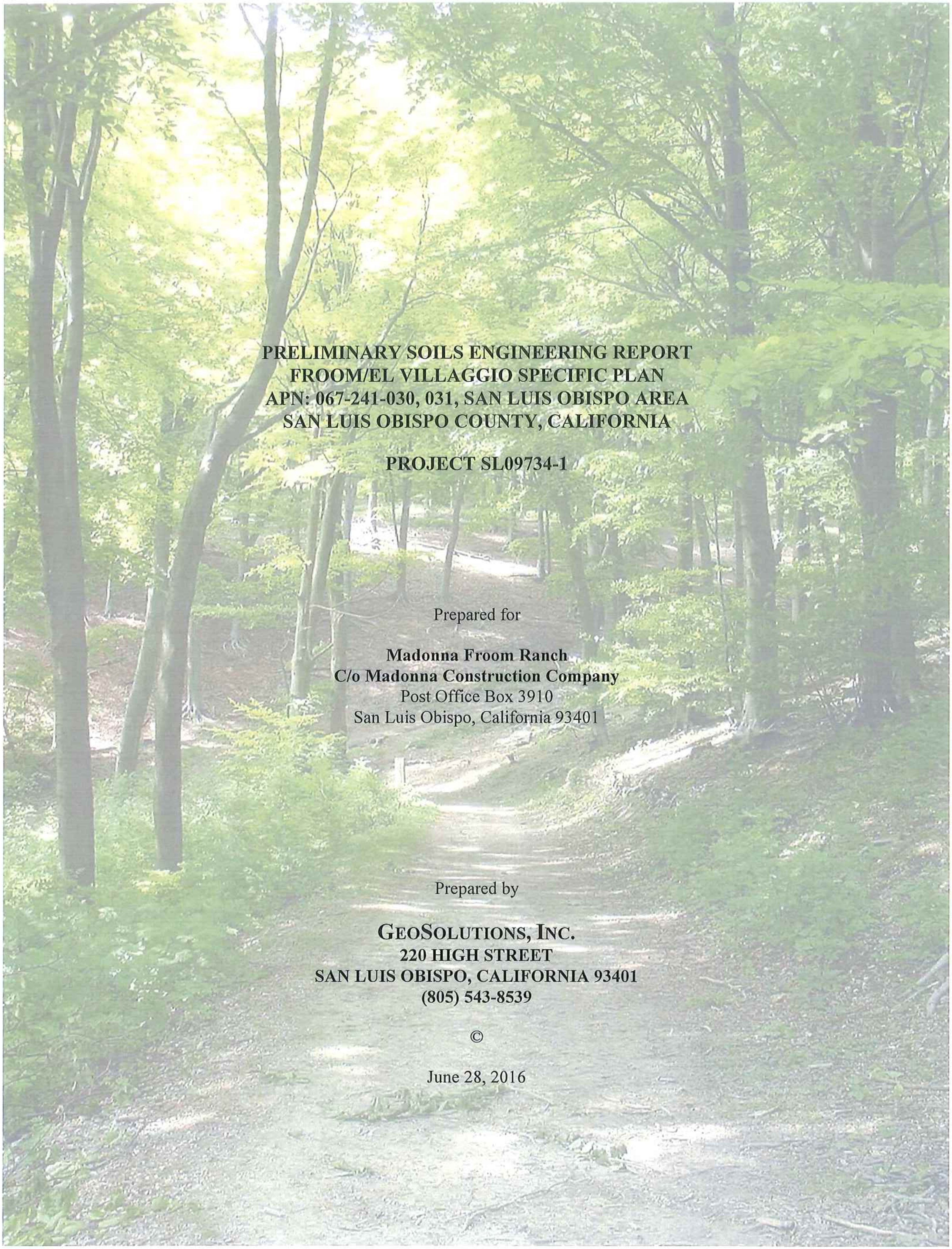

PRELIMINARY SOILS ENGINEERING REPORT
FROOM/EL VILLAGGIO SPECIFIC PLAN
APN: 067-241-030, 031, SAN LUIS OBISPO AREA
SAN LUIS OBISPO COUNTY, CALIFORNIA

PROJECT SL09734-1

Prepared for

Madonna Froom Ranch
C/o Madonna Construction Company
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June 28, 2016

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June 28, 2016
Project No. SL09734-1

Madonna Froom Ranch
c/o Madonna Construction Company
Post Office Box 3910
San Luis Obispo, California 93401

Subject: **Preliminary Soils Engineering Report**
Froom/ El Villaggio Specific Plan, APNs: 067-241-030 and -031
San Luis Obispo area, San Luis Obispo County, California

Dear Mr. Madonna:

This preliminary Soils Engineering Report has been prepared for the proposed development to be referred to as the Froom/ El Villaggio Specific Plan, located on APN Parcels 067-241-030 and -031, in the San Luis Obispo area, San Luis Obispo County, California. Geotechnically, the site is suitable for the proposed development provided the recommendations in this report are incorporated into the design.

It is anticipated that graded pads will be constructed for a majority of the proposed development and that foundations will be supported by engineered fill. Based on the sub-surface investigation in the upper sloping portions of the Site it is anticipated that the foundations for structures in this area will be excavated into competent formational material. Deepened footings may be required in some areas to maintain the minimum embedment into competent formational material.

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.

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
KRAIG ROBERT CROZIER
No. C61361
CIVIL
STATE OF CALIFORNIA

Kraig R. Crozier, PE
Principal, C61361

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

PRELIMINARY SOILS ENGINEERING REPORT
FROOM/EL VILLAGGIO SPECIFIC PLAN
APN: 067-241-030, 031, SAN LUIS OBISPO AREA
SAN LUIS OBISPO COUNTY, CALIFORNIA

PROJECT SL09734-1

1.0 INTRODUCTION

This preliminary report presents the results of the geotechnical investigation for the proposed development to be located on the property referred to as Froom/El Villaggio Specific Plan located on APN Parcels 067-241-030 and -031, in the San Luis Obispo area, 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 8.0* (DeLorme, 2009).

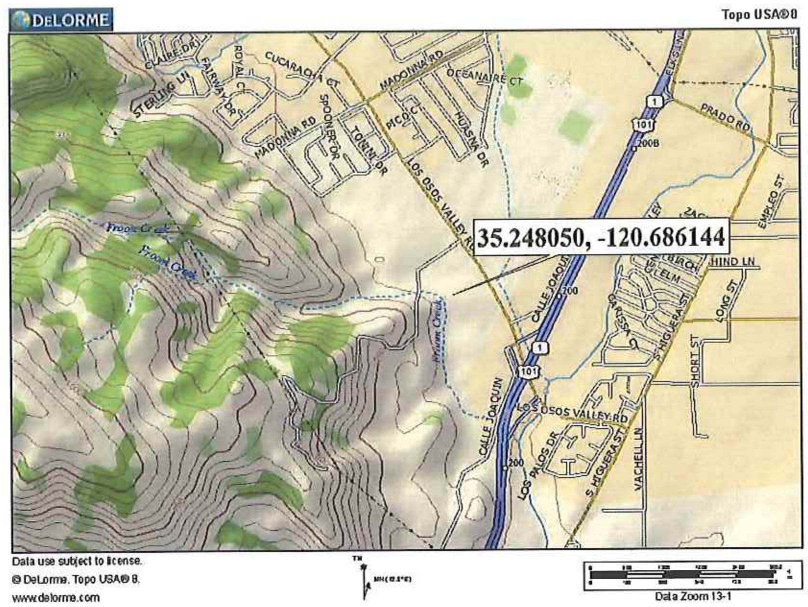


Figure 1: Site Location Map

1.1 Site Description

Froom/El Villaggio Specific Plan is located in the general vicinity of 35.248050 degrees north latitude and 120.686144 degrees west longitude at an approximate general elevation between 110 to 200 feet above mean sea level. The property is irregular in shape and 111.39 acres in size. The site is bounded by Los Osos Valley Road to the north-east, the Home Depot / T.J Maxx development to the north-west, a combination of development and open space to the south, and the Irish Hills Natural Reserve open space to the west. The nearest major intersection is where Los Osos Valley Road intersects Highway 101 approximately 500 feet south from the most easterly portion of the property.

The Site is characterized by flat to rolling grassland with an existing irrigation basin which rises to a steep, rocky hill at the westerly edge of the property, known as the Irish Hills Natural Reserve. The Site is currently developed with few farm/ranch buildings and undeveloped amongst most of the southern portion of the site utilized for animal grazing. Annual grasses, shrubs and a few trees currently vegetate the Site. Surface drainage generally follows the topography toward the east.

1.2 Project Description

The proposed development is to include the construction of mixed residential and commercial development. The proposed structures are anticipated to be constructed using light wood framing and/or light gauge steel/structural steel. The project property will hereafter be referred to as the "Site." See Figure 2: Site Plan for the general layout of the Site.

It is anticipated that the proposed structures will utilize slab-on-grade lower floor systems. Dead and sustained live loads are currently unknown, but they are anticipated to be moderate with maximum continuous footing and column loads estimated to be approximately 2.5 kips per linear foot and 25 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 including geologic maps, and available on-line or in-house aerial photographs.

2. A field study consisting of site reconnaissance and subsurface exploration including 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 preliminary 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 May 18 & 19, 2016 using a Mobile B-24 drill rig. Three six-inch diameter exploratory borings were advanced to a maximum depth of 10 feet below ground surface (bgs) and one six-inch diameter exploratory boring was advanced to a maximum depth of 46 feet below ground surface (bgs) at the approximate locations indicated on Figure 2: Site Plan. 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 Mobile B-24 drill rig was equipped with a safety hammer, which has an efficiency of approximately 60 percent and was used to obtain test blow counts in the form of N-values.

An additional investigation was conducted on June 7, 2016 using a backhoe and excavator. Seven exploratory trenches were advanced to a maximum depth of 25 feet below ground surface (bgs) at the approximate locations indicated on Figure 2: Site Plan. Sampling methods included bulk bags.

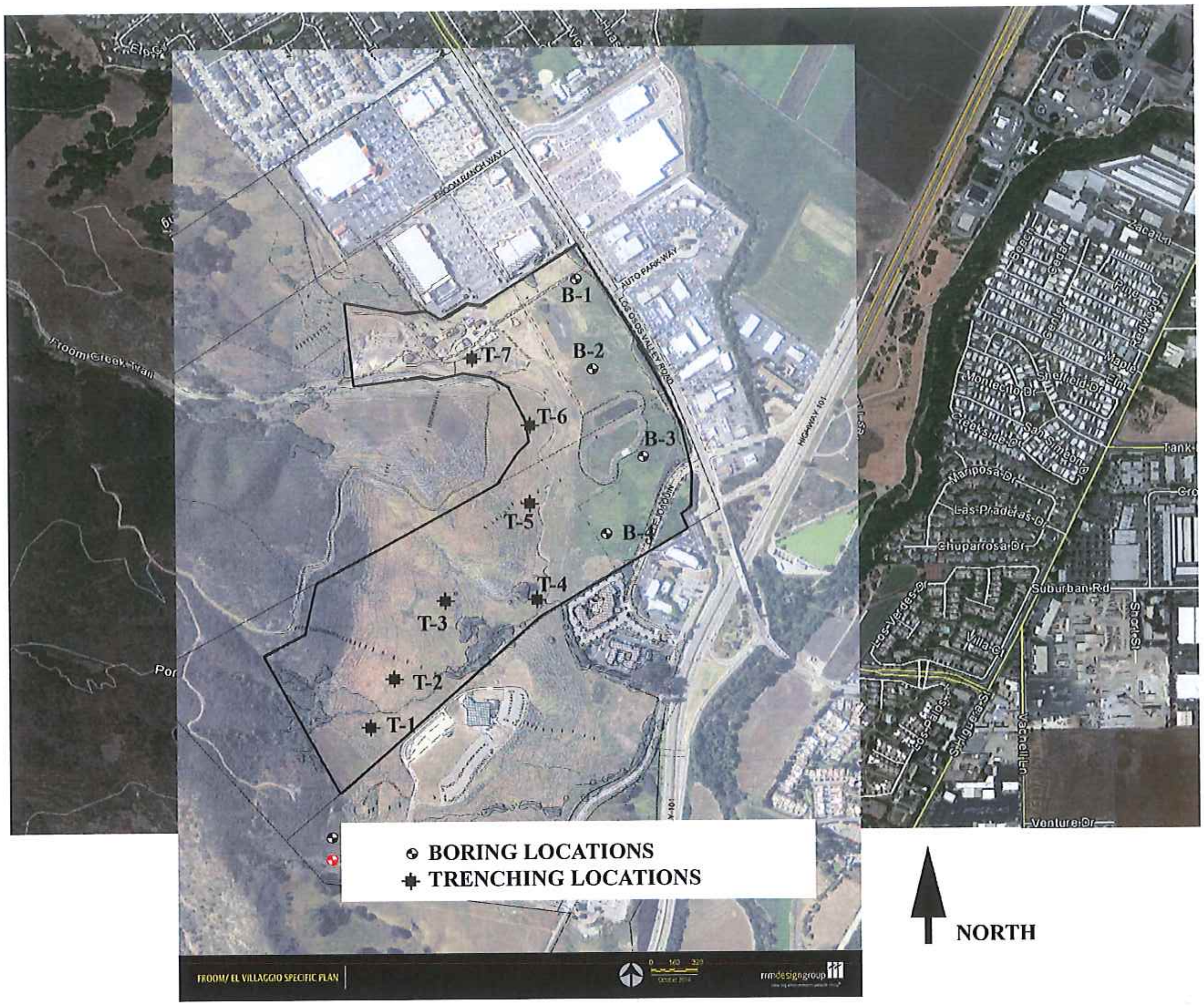


Figure 2: Site Plan

Data gathered during the field investigation suggest that the soil materials at the Site consist of alluvial soil overlying competent formational material. The depth to competent formational material varies across the site, increasing in depth towards Los Osos Valley Road to a maximum depth of approximately 50 feet bgs. The surface material at the Site generally consisted of dark gray to dark grayish brown sandy FAT CLAY (CH) encountered in a soft to stiff and moist to wet condition to termination depth of borings B1-B4, and dark reddish brown sandy CLAY (CL) encountered in a dry to moist condition to an approximate depth of 2.0 to 8.0 feet bgs, underlain by competent formational material in trenches T1-T7.

Regional site geology was obtained by using the *Geologic Map of the Pismo Beach Quadrangle* (Dibblee, 2006) and the MapView internet application (USGS, 2013); the later application is available from the United States Geological Survey website (USGS, 2013) and compiles existing geologic maps. The majority of all underlying material at the Site was interpreted as Surficial Alluvial Sediments alongside/overlying Franciscan Formation and Serpentinite, which will hereafter be referred to as competent formational material. **Shallow ground water was encountered between the depths of 1.5 to 4.0 feet**, although it should be expected that groundwater elevations may vary seasonally and with irrigation practices. See Figure 3: Regional Geologic Map.

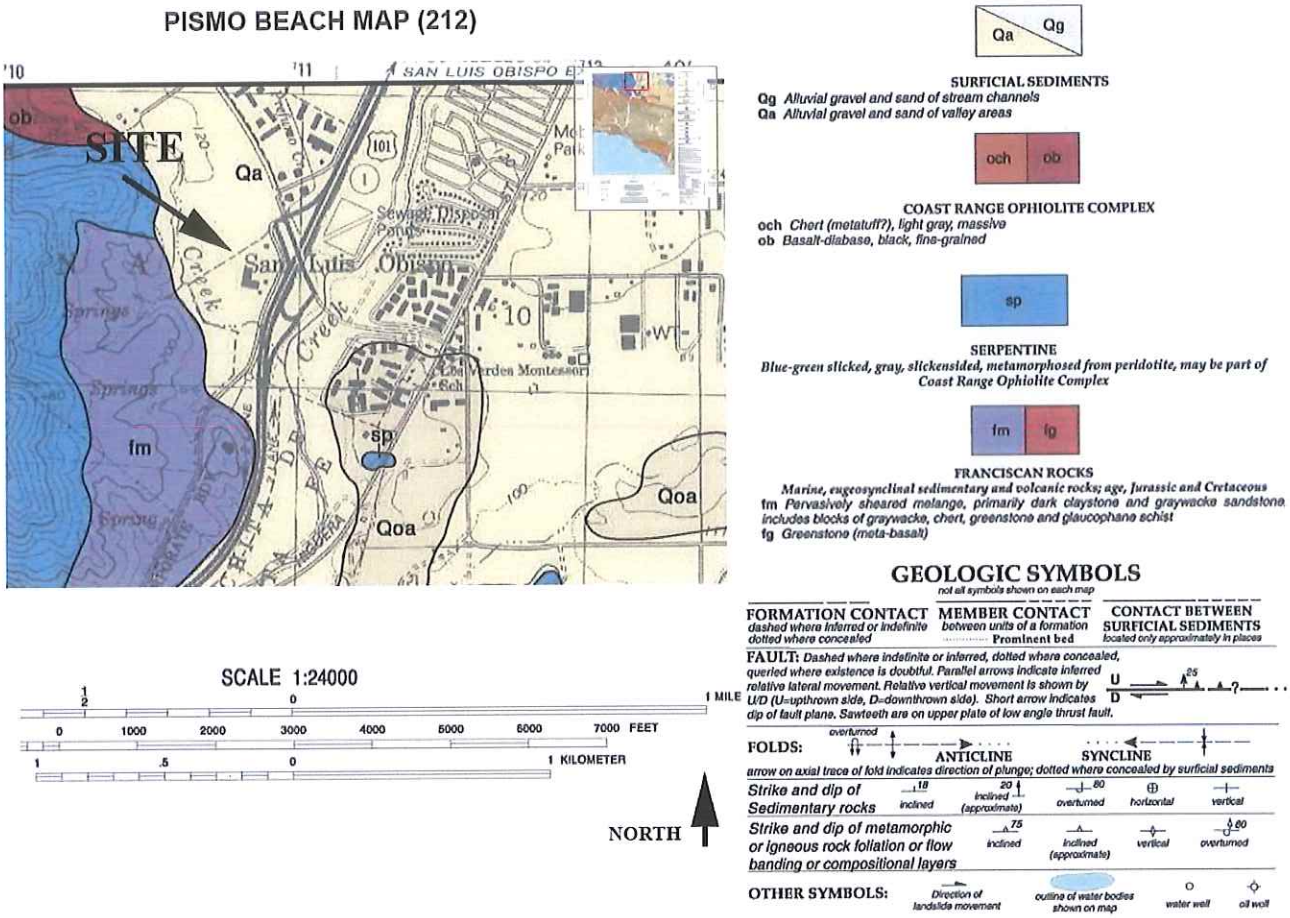


Figure 3: Regional Geologic Map

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 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**.

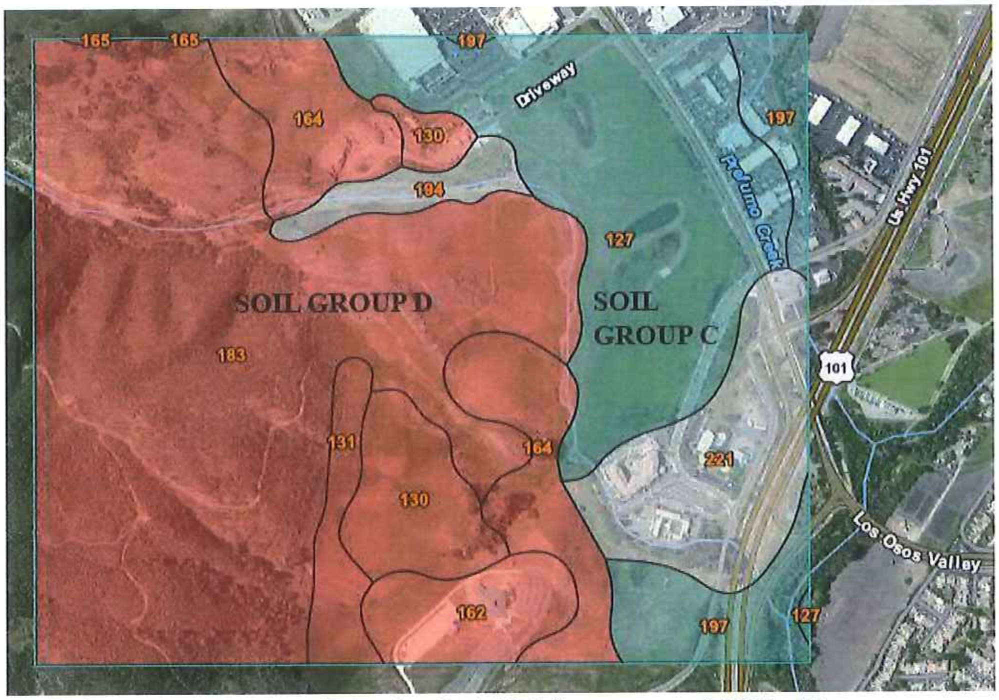
Table 1: Engineering Properties

Sample Name	Sample Description	USCS Specification	Expansion Index	Expansion Potential	Angle of Internal Friction, ϕ (deg.)	Cohesion, c (psf)	Plasticity Index	Fines Content (%)	Compression Index, C_C	Recompression Index, C_r
A B-1 @ 1.0'	Very Dark Grayish Brown Sandy FAT CLAY	CH	107	High	-	-	31	79.6	-	-
B B-3 @ 1.0'	Dark Gray FAT CLAY	CH	186	Very High	-	-	48	94.7	-	-
C T-2 @ 2.0'	Dark Reddish Brown Sandy CLAY	CL	79	Medium	-	-	-	-	-	-
B-1 @ 5.0'	Very Dark Grayish Brown Sandy CLAY	CL	-	-	28.7	110	-	-	0.165	0.017

4.0 HYDROLOGIC SOIL GROUP

Based on the Web Soil Survey provided by the Natural Resources Conservation Service, the Site was initially designated as containing Hydrologic Soil Groups C and D. Groups C and D are similar in that they are both comprised of fine-grained and/or nearly impervious material with slow to very slow infiltration rates. The main distinction between the two groups is that Group D soil conditions are less favorable for infiltration of storm water and runoff due to; very slow infiltration rate (high runoff potential), clays with high shrink-swell potential, soils with high water table, and soils that are shallow over nearly impervious material. Based on the sub-surface data obtained during the field investigation and the results of the laboratory testing, it is our opinion that the entire Site is best defined as **Hydrologic Soil Group D**. See Figure 4: Hydrologic Soil Group.

Due to shallow groundwater encountered at an approximate depth of 1.5 to 4.0 bgs during the field investigation, field infiltration testing was not performed. Observed shallow ground water and soil characteristics, including competent formational material encountered at shallow depths, are indicative of slow to very slow infiltration rate and are in support of the prescribed hydrologic soil group.



Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

Soil Rating Polygons

- A
- A/D
- B
- B/D
- C
- C/D
- D
- Not rated or not available

Figure 4: Hydrologic Soil Group

5.0 SEISMIC DESIGN CONSIDERATIONS

4.1 Seismic Hazard Analysis

1. According to section 1613 of the 2013 CBC (CBSC, 2013), 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, 2010). ASCE7 considers the most severe earthquake ground motion to be the ground motion caused by the Maximum Considered Earthquake (MCE) (ASCE, 2010), which is defined in Section 1613 of the 2013 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 1613.3.2 of the 2013 CBC (CBSC, 2013), the Site soil profile classification is determined by the average soil properties in the upper 100 feet of the Site profile

(ASCE 7). Based on the $(N_1)_{60}$ values calculated for the in-situ tests performed during the field investigation, and the results of the laboratory analysis of the in-situ soils the Site was defined as **Site Class E, Soft Soil** profile for the lower portion of the Site and **Site Class C, Very Dense Soil and Soft Rock profile** for the upper portion of the Site where shallow formational material was encountered, per ASCE 7 Chapter 20.

4. According to section 11.2 of ASCE7 and section 1613 of the 2013 CBC (CBSC, 2013), 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}=0.425 g and S_{DS}=0.854 g, for the upper portion of the Site (Site Class C) and S_{D1}=0.774g and S_{DS}=0.769 g for the lower portion of the Site (Site Class E)**, 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 35.24805 degrees north latitude and -120.686144 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 2013 CBC (CBSC, 2013) and sections 11.4.3 and 11.4.4 of ASCE7 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 and Site Class E**. 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, U.S. Seismic Design Map computer application (USGS, 2013); this program is available from the United States Geological Survey website (USGS, 2013). This computer program utilizes the methods developed in the 1997, 2000, 2003, 2008 and 2013 errata 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. 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 1613.3.5(1) and 1613.3.5(2) of the 2013 CBC (CBSC, 2013).

3. The site specific MCE peak ground acceleration (**PGA_M**) as determined by the USGS computer program (web based) **PGA_M = 0.541 g** for the upper portion of the Site (Site Class C) and **PGA_M = 0.487g** for the lower portion of the Site (Site Class E) which is present on Sheet 5 of 6 of the USGS Design Maps Detailed Reports (ASCE 7-10 Standard). See **Appendix C:** USGS Design Maps Summary and Detailed Reports.

4.3 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.

6.0 GENERAL SOIL-FOUNDATION DISCUSSION

It is anticipated that graded pads will be constructed for a majority of the proposed development and that foundations will be supported by engineered fill. Based on the sub-surface investigation in the upper sloping portions of the Site it is anticipated that the foundations for structures in this area will be excavated into competent formational material. Deepened footings may be required in some areas to maintain the minimum embedment into competent formational material.

Soil conditions encountered during the field investigation varied, including soft, highly compressible and expansive soils, and shallow competent formational material. Due to the presence of highly expansive surface soils and shallow groundwater within the lower development areas, it is recommended that as a minimum, the upper 36 inches (3 feet) of the development area should consist of a select import material on top of existing grade or in replacement of the existing surficial soils. This is intended to act as a ballast or cap, providing isolation and increased stability above areas of underlying soft, highly compressible and expansive soils. An increase in depth of select import material to a minimum of 5.0 feet will allow for a reduction in foundation requirements. This report discusses both options.

Due the presence of shallow weathered bedrock materials encountered during the field investigation, hard rock excavation conditions are expected during development of building pad areas and underground utility construction in the upper portion of the Site.

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 engineered fill/competent formational material, is very common. This interface occurs at the Site and is likely to 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.

7.0 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 of groundwater seepage, encountered between 1.5 to 4.0 feet bgs in the shallow subsurface.

2. The presence of highly expansive surface soils. Expansive soils tend to swell when exposed to excess moisture and shrink when allowed to dry. The soil zone within the upper 2 to 3 feet of the Site is most affected by these seasonal changes in moisture content. The volume change associated with this soil movement can stress and damage foundations, concrete flatwork, interior slabs-on-grade, and roadway pavements. Foundations supported by expansive soils should be designed by a Structural Engineer in accordance with the 2013 California Building Code.

3. The potential for loose soil materials generated from removal of existing trees and root systems within the proposed building pad areas.

4. The presence of shallow, hard bedrock materials within the upper portion of the development. Hard digging/excavation conditions are anticipated in some areas during building pad preparation and underground utility construction.

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

7.1 Site Preparation of Building Pad Areas

1. It is anticipated that graded pads will be constructed for a majority of the proposed development and that foundations will be supported by engineered fill. Based on the sub-surface investigation in the upper sloping portions of the Site it is anticipated that the foundations for structures in this area will be excavated into competent formational material. Deepened footings may be required in some areas to maintain the minimum embedment into competent formational material.

2. Due to the presence of highly expansive surface soils and shallow groundwater within the lower development areas, it is recommended that as a minimum, the upper 36 inches (3 feet) of the development area should consist of a select import material on top of existing grade or in replacement of the existing surficial soils. This will allow for support of mat foundations for the proposed structures. An increase in thickness of the select import

material to a minimum of 5.0 feet will allow for the use of conventional foundation systems.

3. For the development of an engineered fill pad with a **36 inch (3 feet) select import pad cap to receive a mat foundation**, the native material should be over-excavated at least 36 inches below slab sub-grade elevation, to competent material, or to one-half the depth of the deepest fill (measured from the bottom of the deepest footing); whichever is greatest. The exposed surface should be scarified to a depth of 6 inches; moisture conditioned to 3% over optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-07). **The upper 36 inches of building pad areas should consist of an approved select import material processed as engineered fill.** All material to be used as select import fill should be granular soil with a very low to low expansion potential (i.e., expansion index of 50 or less) and must be observed and approved by a representative of GeoSolutions, Inc. prior to its delivery to the Site. Refer to Figure 5: Sub-Slab Detail for under-slab drainage material and **Appendix D** for more details on fill placement.

4. For the development of an engineered fill pad with a **60 inch (5 feet) select import pad cap to receive conventional foundations**, the native material should be over-excavated at least 60 inches (5 feet) below existing grade, 36 inches (3 feet) below the bottom of the footings, to competent material, or to one-half the depth of the deepest fill (measured from the bottom of the deepest footing); whichever is greatest. The limits of over-excavation should extend a minimum of 5 feet beyond the perimeter foundation, to property lines, or existing improvements, whichever is least. The exposed surface should be scarified to a depth of 6 inches; moisture conditioned to 3% over optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-07). The over-excavated material, cleared of oversized aggregates and debris, should then be processed as engineered fill up to within 60 inches of the surface of the building pad. **The upper 60 inches (5 feet) of building pad areas should consist of an approved select import material processed as engineered fill.** All material to be used as select import fill should be granular soil with a very low to low expansion potential (i.e., expansion index of 50 or less) and must be observed and approved by a representative of GeoSolutions, Inc. prior to its delivery to the Site. Refer to Figure 5: Sub-Slab Detail for under-slab drainage material and **Appendix D** for more details on fill placement.

5. For slab-on-grade construction with footings founded a **minimum of 12 inches into uniform competent formational material**, the pad area to receive slab-on-grade construction should be graded such that all slabs are supported on uniform competent material. The native material should be excavated beneath the slab at least 12 inches below existing grade and finished slab elevation, to competent material, or to one-half the depth of the deepest fill; whichever is greatest. The exposed surface should be scarified to a depth of 6 inches, moisture conditioned to 3% over optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-07). The over-excavated material may then be processed as engineered fill. Figure 5: Sub-Slab Detail for under-slab drainage material and **Appendix D** for more details on fill placement.

6. There is potential that soils encountered at the required foundation excavation depth may exhibit soft, compressible conditions. If pumping soils are encountered at the bottom of the excavation, stabilization will be necessary and may require the installation of a woven geotextile fabric, such as Mirafi 600x or equivalent, on the prepared bottom of the excavation. If the soil within the excavation is not stable enough for proper installation of the geotextile fabric, rock stabilization of the exposed sub-grade may be required, with the placement and compaction of 3-inch to 8-inch diameter (gabion) crushed stone into the soft sub-grade, until stability is achieved, as observed and approved by a representative of this firm. Alternative recommendations may be prepared based on the conditions encountered.

7. 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 required. See **Appendix D**, Detail A, Key and Bench with Backdrain for details on key and bench construction.

8. The recommended soil moisture content should be maintained during construction and following construction of the proposed development. Where soil moisture content is not maintained, desiccation cracks may develop which indicate a loss of soil compaction, leading to the potential for damage to foundations, flatwork, pavements, and other improvements. Soils that have become cracked due to moisture loss should be removed sufficient depth to repair the cracked soil as observed by the soils engineer, and the removed materials should then be moisture conditioned to approximately 3 percent over optimum value, and compacted.

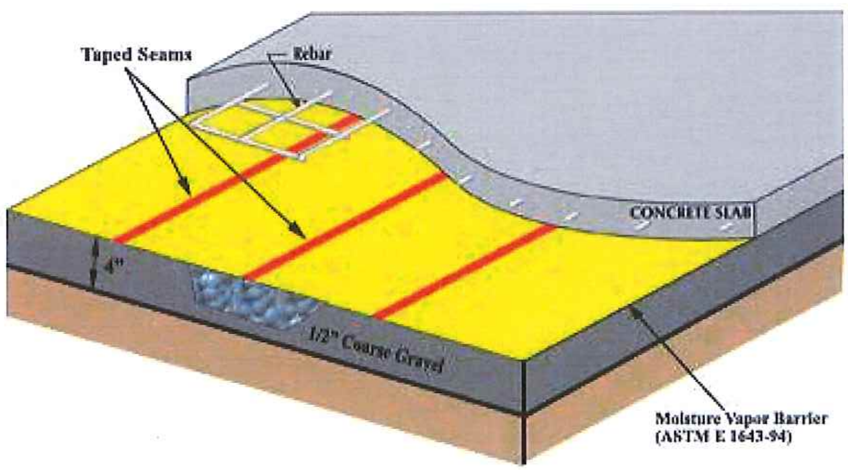


Figure 5: Sub-Slab Detail

7.2 Preparation of Paved Areas

1. Pavement areas should be excavated to approximate sub-grade elevation or to competent material; whichever is deeper. The exposed surface should be scarified an additional depth

of 12 inches, moisture conditioned to 3% over optimum moisture content, and compacted to a minimum relative density of 95 percent (ASTM D1557-07 test method). The top 12 inches of sub-grade soil under all 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.

2. 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.

3. There is potential that soils encountered at the required foundation excavation depth may exhibit soft, compressible conditions. If pumping soils are encountered at the bottom of the excavation, stabilization will be necessary and may require the installation of a woven geotextile fabric, such as Mirafi 600x or equivalent, on the prepared bottom of the excavation. If the soil within the excavation is not stable enough for proper installation of the geotextile fabric, rock stabilization of the exposed sub-grade may be required, with the placement and compaction of 3-inch to 8-inch diameter (gabion) crushed stone into the soft sub-grade, until stability is achieved, as observed and approved by a representative of this firm. Alternative recommendations may be prepared based on the conditions encountered.

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 pavement sections at the Site, GeoSolutions, Inc. recommends that a laterally-reinforcing geotextile grid, such as Tensar BX1100, Syntec SBX11, ADS BX114GG, 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 biaxial geogrid product and to provide recommendations regarding the procedures for the installation of geogrid products at the Site.

7.3 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 7.2, 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. Based on the soil conditions observed and the results of the laboratory testing performed on surface soils within the Site, an **R-Value of 5** is estimated for preliminary pavement design purposes. A minimum of **10 inches** of Class II Aggregate Base is recommended for all pavement sections. Following Site improvement with select import in proposed

development areas, additional R-Value testing may be performed to determine the appropriate pavement structural section. 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 BX1100, Syntec SBX11, ADS BX114GG, 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 biaxial geogrid product.

7.4 Conventional Foundations

1. Conventional continuous and spread footings with grade beams may be used for support of the proposed structures. Isolated pad footings are not allowed. Foundations must be designed in accordance to section 1808.6, 2013 CBC, Foundations on Expansive Soils.

2. Minimum footing and grade beam sizes and depths in engineered fill or uniform competent formational material should conform to the following table, as observed and approved by a representative of GeoSolutions, Inc.

Table 2: Minimum Footing and Grade Beam Dimensions

	Perimeter Footings	Grade Beams
Minimum Width	12 inches (one story) 15 inches (two story)	12 inches
Minimum Depth	24 inches	18 inches
Minimum Embedment in Competent Formational Material	12 inches	-
Minimum Reinforcing*	4 #5 bars (2 top / 2 bottom)	4 #4 bars (2 top / 2 bottom)
Spacing	-	19 feet on-center each way
* Steel should be held in place by stirrups at appropriate spacing to ensure proper positioning of the steel (see WRI Design of Slab-on-Ground Foundations and ACI 318, Section 7.5 – Placing Reinforcement).		

3. Minimum reinforcing for footings should conform to the recommendations provided in Table 2: Minimum Footing and Grade Beam Dimensions which meets the specifications of Section 1808.6 of the 2013 California Building Code for the soil conditions at the Site. Reinforcing steel should be held in place by stirrups at appropriate spacing to ensure proper positioning of the steel in accordance with WRI Design of Slab-on-Ground Foundations, and ACI 318, Section 7.5 – Placing Reinforcement.

4. 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 maintained in a moist condition with no desiccation cracks present.

5. An allowable dead plus live load bearing pressure of **2,000 psf** may be used for the design of footings founded in **60 inches of select import fill** in accordance with the recommendations presented in **Section 7.1 Site Preparation of Building Pad Areas, paragraph 4 (7.1.4)**, and an allowable dead plus live load bearing pressure of **2,500 psf** may be used for the design of footings founded in **uniform competent formational material** in accordance with the recommendations presented in **Section 7.1 Site Preparation of Building Pad Areas, paragraph 5 (7.1.5)**.

6. Allowable bearing capacities may be increased by one-third when transient loads such as wind and/or seismicity are included.

7. A total settlement of less than 1 inch and a differential settlement of less than 1 inch in 30 feet are anticipated.

8. 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 uniform competent formational material and the bottom of the footings. Values from Table 3: Foundation Lateral Resistance Parameters can be used to design for resistance to lateral loads. If friction and passive pressures are combined to resist lateral forces acting on shallow footings, the lesser value should be reduced by 50 percent.

Table 3: Foundation Lateral Resistance Parameters

Foundation Embedment Material	Lateral Passive Pressure, pcf	Friction Factor
Uniform Competent Formational Material	400	0.45
Select Import Material	300	0.35

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

10. Foundation design should conform to the requirements of Chapter 18 of the latest edition of the CBC (CBSC, 2013).

11. 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.

12. 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 6: Setback Dimensions – Slope Gradients Between 3-to-1 and 1-to-1 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.

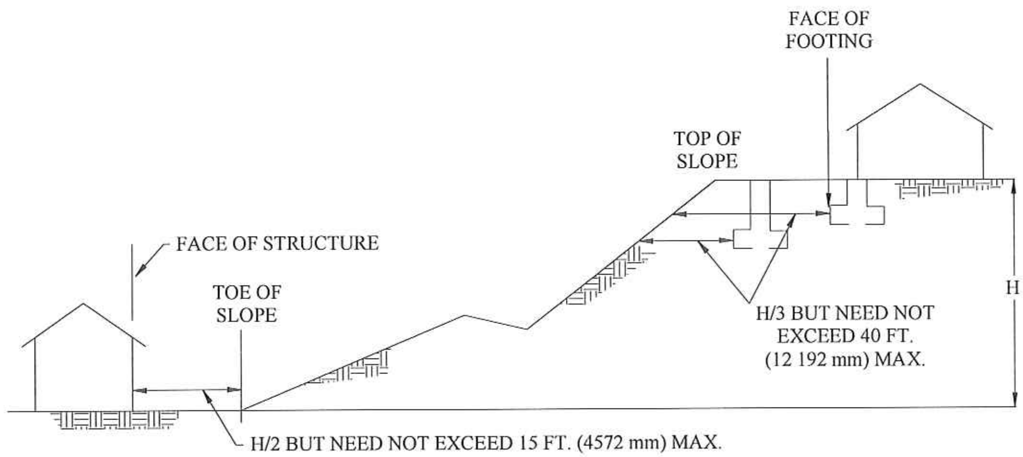


Figure 6: Setback Dimensions – Slope Gradients Between 3-to-1 and 1-to-1

7.5 Mat Foundations

1. A conventionally reinforced structural mat slab foundation system could be utilized to support the proposed structures as part of the development. The mat may be of a uniform thickness design, or may consist of shear and moment-resisting grade beams with structural slab elements connecting the grade beams. The mat may be constructed directly on the 36 inch layer of select import material, per section **7.1.3**.

2. Based on our experience, a mat slab approximately **10 to 15 inches thick** could be anticipated. A modulus of sub-grade reaction (**k_s**) of **50 pci** may be used in design.

3. An allowable dead plus live load bearing pressure of **1,000 psf** may be used for design of mat foundations in accordance with the recommendations presented in **Section 7.1 Site Preparation of Building Pad Areas, paragraph 3 (7.1.3).** Minimum reinforcing should be as directed by the project Structural Engineer.

4. Foundation excavations should be observed and approved by a representative of this firm prior to the placement of reinforcing steel and/or concrete. Concrete should be placed only in excavations that have been kept moist and are free of loose, soft soil or debris.

5. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the native material and the bottom of the footings. For resistance to lateral loads, a friction factor of **0.35** may be utilized for sliding resistance at the base of footings.

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

7. Foundation design should conform to the requirements of Chapter 18 of the latest edition of the California Building Code.

7.6 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 maintained in a moist condition with no desiccation cracks present.

2. Concrete slabs-on-grade should be in conformance with the recommendations provided in Table 4: Minimum Slab Recommendations. Reinforcing should be placed on-center both ways at or slightly above the center of the structural section. Reinforcing bars should have a minimum clear cover of 1.5 inches. Where lapping of the slab steel is required, laps in adjacent bars should be staggered a minimum of every five feet (see WRI Design of Slab-on-Ground Foundations, Steel Placement). The recommended 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.

Table 4: Minimum Slab Recommendations

Minimum Thickness	5 inches
Reinforcing*	#4 bars at 16 inches on-center each way
* Where lapping of the slab steel is required, laps in adjacent bars should be staggered a minimum of every five feet (see WRI/CSRI-81 recommendations for Steel Placement, Section 2).	

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 for interior conditioned spaces, the slabs should be underlain by a minimum of four inches of clean free-draining material, such as a ½ inch coarse aggregate mix, to serve as a cushion and a capillary break. Where moisture susceptible storage or floor coverings are anticipated, a 15-mil Stego Wrap membrane (or equivalent installed per manufacturer's specifications) should be placed between the free-draining material and the slab to minimize moisture condensation under the floor covering. See Figure 5: Sub-Slab Detail for the placement of under-slab drainage material. It is suggested, but not required, 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 six inches. The sand should be lightly moistened prior to placing concrete.

5. It should be noted that for a vapor barrier installation to conform to manufacturer's specifications, sealing of penetrations, joints and edges of the vapor barrier membrane are typically required. As required by the California Building Code, joints in the vapor barrier should be lapped a minimum of 6 inches. If the installation is not performed in accordance with the manufacturer's specifications, there is an increased potential for water vapor to affect the concrete slabs and floor coverings.

6. The most effective method of reducing the potential for moisture vapor transmission through concrete slabs-on-grade would be to place the concrete directly on the surface of the vapor barrier membrane. However, this method requires a concrete mix design specific to this application with low water-cement ratio in addition to special concrete finishing and curing practices, to minimize the potential for concrete cracks and surface defects.

The contractor should be familiar with current techniques to finish slabs poured directly onto the vapor barrier membrane.

7. 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.

7.7 Exterior Concrete Flatwork

1. Due the presence of highly expansive surface soils within the proposed development areas, there is a high potential for considerable soil movement and flatwork if conventional measures are used, such as the placement of 4 to 6 inches of imported sand materials placed beneath concrete flatwork. Heaving and cracking are anticipated to occur. To reduce the potential for movement associated with expansive soils, we recommend the placement of a minimum of **36 inches of approved select import material placed as engineered fill beneath the flatwork.**

2. Minimum flatwork reinforcement for conventional pedestrian areas should consist of No. 3 (#3) rebar spaced at 24 inches on-center each-way at or slightly above the center of the structural section. The flatwork should be a minimum of 4 inches thick.

3. Flatwork should be constructed with frequent joints to allow for movement due to fluctuations in temperature and moisture content in the adjacent soils. Flatwork at doorways, driveways, curbs and other areas where restraining the elevation of the flatwork is desired, should be doweled to the perimeter foundation by a minimum of No. 3 reinforcing steel dowels, spaced at a maximum distance of 24 inches on-center.

7.8 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 5: Retaining Wall Design Parameters and Figure 7: 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 5: Retaining Wall Design Parameters

Lateral Pressure and Condition	Equivalent Fluid Pressure, pcf
Static, Active Case, ($\gamma'K_A$) Select Import Material	40
Static, At-Rest Case, ($\gamma'K_O$) Select Import Material	60
Static, Passive Case, ($\gamma'K_P$) Select Import Material Competent Formational Material	300 400

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 7: Retaining Wall Detail and Figure 8: Retaining Wall Active and Passive Wedges for a description of the location of the active wedge behind a retaining wall.

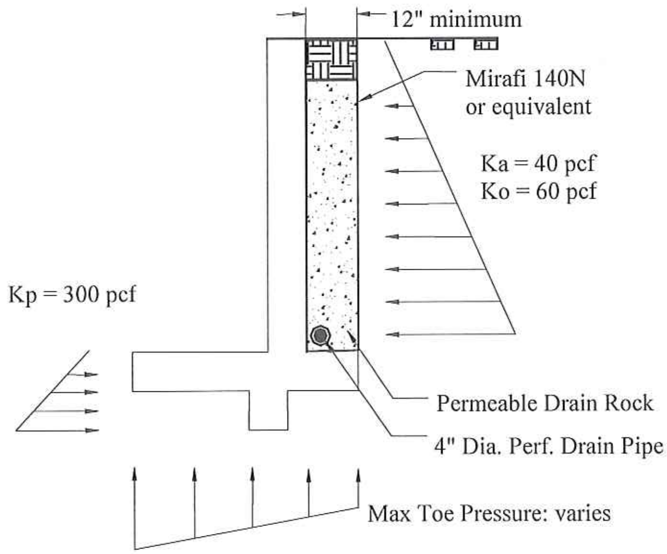


Figure 7: Retaining Wall Detail

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 degree of slope inclination.

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.

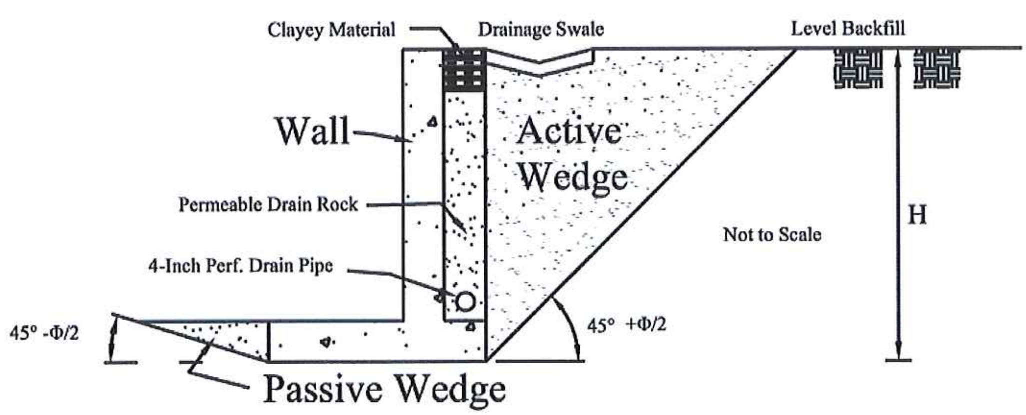


Figure 8: Retaining Wall Active and Passive Wedges

5. Retaining wall foundations should be founded a minimum of 24 inches below lowest adjacent grade in **select import fill** or founded a minimum of 24 inches below lowest adjacent grade with a minimum embedment of 12 inches in uniform **competent formational material** as observed and approved by a representative of GeoSolutions, Inc.

A coefficient of friction of **0.35** may be used between **select import fill** and concrete footings or **0.45** may be used between uniform **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 **select import fill** and **3,000 psf** for footings founded in uniform **competent formational material.**

6. For earthquake conditions, retaining walls greater than 6 feet in height should be designed to resist an additional seismic lateral soil pressure of **25 pcf** equivalent fluid pressure for unrestrained walls (active condition). The pressure resultant force from earthquake loading should be assumed to act a distance of $^1/_3H$ above the base of the retaining wall, where H is the height of the retaining wall. Seismic active lateral earth pressure values were determined using the simplified dynamic lateral force component (SEAOC 2010) utilizing the design peak ground acceleration, PGA_M, discussed in Section 5.0 (**PGA_M = 0.541g (Site Class C) or PGA_M = 0.487g (Site Class E)**). The dynamic increment in lateral earth pressure due to earthquakes should be considered during the design of retaining walls at the Site. Based on research presented by Dr. Marshall Lew (Lew et al., 2010), lateral pressures associated with seismic forces should not be applied to restrained walls (at-rest condition).

7. 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.

8. In addition to the static lateral soil pressure values reported in Table 5: 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.

9. 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.

10. 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.

11. 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.

12. 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.

13. 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.

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

8.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations contained in this report are based on a limited number of borings and 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 County of San Luis Obispo, the 2013 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 biaxial geogrid 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 1705.6 of the 2013 CBC (CBSC, 2013) requires the following inspections by the Soils Engineer for controlled fill thicknesses greater than 12 inches as shown in Table 6: Required Verification and Inspections of Soils:

Table 6: 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

9.0 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.

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REFERENCES

REFERENCES

American Society of Civil Engineers (ASCE). *Minimum Design Loads for Buildings and Other Structures.* ASCE Standard ASCE/SEI 7-05 Including Supplement No. 1. 1801 Alexander Bell Drive, Reston, Virginia 20191. 2006.

Assessor's Office. County of San Luis Obispo. *San Luis Obispo Vicinity.* Book 067. Page 241.

California Building Standards Commission (CBSC). *2013 California Building Code, California Code of Regulations.* Title 24. Part 2. Vol. 2. California Building Standards Commission: January 2013.

DeLorme. *Topo USA 8.0.* Vers.8.0.0 Computer software. DeLorme, 2009. Microsoft Windows 7, DVD-ROM drive.

Dibblee, Thomas W., Jr.. *Geologic Map of the Pismo Beach Quadrangle.* Dibblee Geologic Center Map Number DF-212. Santa Barbara Museum of Natural History: April 2006.

State of California. Department of Industrial Relations. *California Code of Regulations.* 2001 Edition. Title 8. Chapter 4: Division of Industrial Safety. Subchapter 4, Construction Safety Orders. Article 6: Excavations. http://www.dir.ca.gov/title8/sub4.html.

State of California, Department of Transportation. *Standard Specifications.* State of California Department of Transportation Central Publication Distribution Unit: July 1999.

United States Geological Survey, Geologic Hazards Science Center, *U.S. Seismic Design Maps*, http://geohazards.usgs.gov/designmaps/us/application.php website. January 24, 2014.

United States Geological Survey. *MapView – Geologic Maps of The Nation.* Internet Application. USGS, 26 August, 2013. < http://ngmdb.usgs.gov/maps/MapView/>

APPENDIX A

Field Investigation

Soil Classification Chart

Boring Logs

Trench Logs

FIELD INVESTIGATION

The field investigation was conducted May 18-19 & June 7, 2016 using a Mobile B-24 drill rig and extendahoe equipment. The surface and sub-surface conditions were studied by advancing four exploratory borings and seven 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 Mobile B-24 drill rig with a six-inch diameter solid-stem continuous flight auger bored four exploratory borings and the extendahoe advanced seven exploratory trenches near the approximate locations indicated on Figure 2: Site Plan. The drilling 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.
the "A" line, then dual symbols (I.e. CL-ML) are required.

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

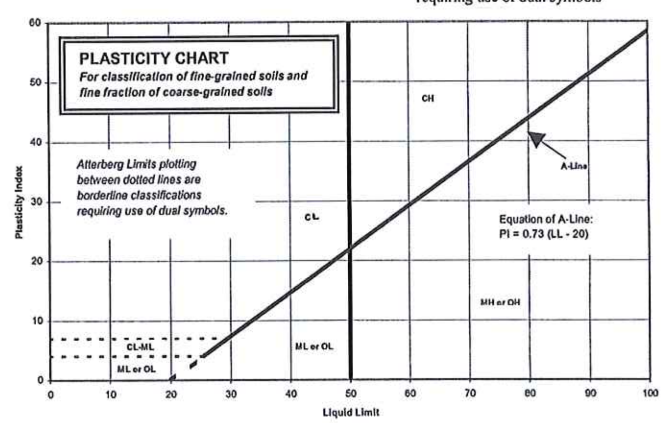
SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/ FOOT +
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	Over 50

+ 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.

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



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 – Sample
SPT - Standard Penetration
CA - California Modified
N - Nuclear Gauge
PO – Pocket Penetrometer (tons/sq.ft.)

GeoSolutions, INC.

GeoSolutions, Inc.

220 High Street, San Luis Obispo, CA 93401
1021 West Tama Lane, Suite 105
Santa Maria, CA 93454

BORING LOG

BORING NO. **B-1**

JOB NO. **SL09734-1**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**Madonna - Froom Ranch**	DRILL RIG:	**Mobile B-24**
DRILLING LOCATION:	**See Figure 2, Site Plan**	HOLE DIAMETER:	**6 Inches**
DATE DRILLED:	**May 18, 2016**	SAMPLING METHOD:	**SPT and CA**
LOGGED BY:	**GTV**	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	CLAYEY SAND: very dark grayish brown, dry	CH											
-1				A			28.7				107	79.6	31
-2	stiff			SPT	6	9							
-3	SANDY CLAY: dark brown, dry	CH											
-4													
-5	stiff			CA	8	8		110					
-6													
-7													
-8	stiff			SPT	9	9							
-9													
-10													
-11													
-12													
-13													
-14													
-15													
-16													
-17													
-18													
-19													
-20													

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BORING LOG

BORING NO. **B-2**

JOB NO. **SL09734-1**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**Madonna - Froom Ranch**	DRILL RIG:	**Mobile B-24**
DRILLING LOCATION:	**See Figure 2, Site Plan**	HOLE DIAMETER:	**6 Inches**
DATE DRILLED:	**May 19, 2016**	SAMPLING METHOD:	**SPT**
LOGGED BY:	**GTV**	HOLE ELEVATION:	**Not Recorded**

▼ Depth of Groundwater: **4.0 Feet** Boring Terminated At: **46.0 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: very dark grayish brown, dry	CH											
-2	SANDY CLAY: dark brown, dry	CH											
-4	▼ firm			SPT	5	7							
-6	SANDY CLAY: dark olive brown, very moist	CH											
-8.5	stiff			SPT	8	8							
-10	SANDY CLAY: black, very moist	CH											
-13.5	stiff			SPT	16	15							
-16	SANDY CLAY: dark olive brown, with gravel, coarse grained	CH											
-18	stiff												
-18.5				SPT	5	4							

Depth scale: 0, -1, -2, -3, -4, -5, -6, -7, -8, -9, -10, -11, -12, -13, -14, -15, -16, -17, -18, -19, -20, -21, -22, -23

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BORING LOG

BORING NO (cont). **B-2**

JOB NO. **SL09734-1**

Depth of Groundwater: **4.0 Feet** Boring Terminated At: **46.0 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)	FINE CONTENTS	PLASTICITY INDEX (PI)
-24													
-25	SANDY CLAY: dark yellowish brown, saturated	CH											
-26	very stiff												
-27													
-28													
-29				SPT	21	16							
-30													
-31													
-32													
-33													
-34													
-35													
-36													
-37													
-38													
-39													
-40													
-41													
-42													
-43													
-44													
-45	very sticky												
-46	FRANCISCAN COMPLEX: serpentinite & melange, meta volcanic, very weathered			SPT	50/1"								
-47													
-48													
-49													
-50													

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BORING LOG

BORING NO. **B-3**

JOB NO. **SL09734-1**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**Madonna - Froom Ranch**	DRILL RIG:	**Mobile B-24**
DRILLING LOCATION:	**See Figure 2, Site Plan**	HOLE DIAMETER:	**6 Inches**
DATE DRILLED:	**May 18, 2016**	SAMPLING METHOD:	**SPT and CA**
LOGGED BY:	**GTV**	HOLE ELEVATION:	**Not Recorded**

Depth of Groundwater: **1.5 Feet** 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	FAT CLAY: very dark grayish brown, dry	CH											
-1				B							186	94.7	48
-2	firm			SPT	4	6							
-3	SANDY CLAY: dark olive brown, very moist	CH											
-5	firm			CA	6	6							
-7	soft												
-9				SPT	2	2							
-10													

-11 -12 -13 -14 -15 -16 -17 -18 -19 -20

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BORING LOG

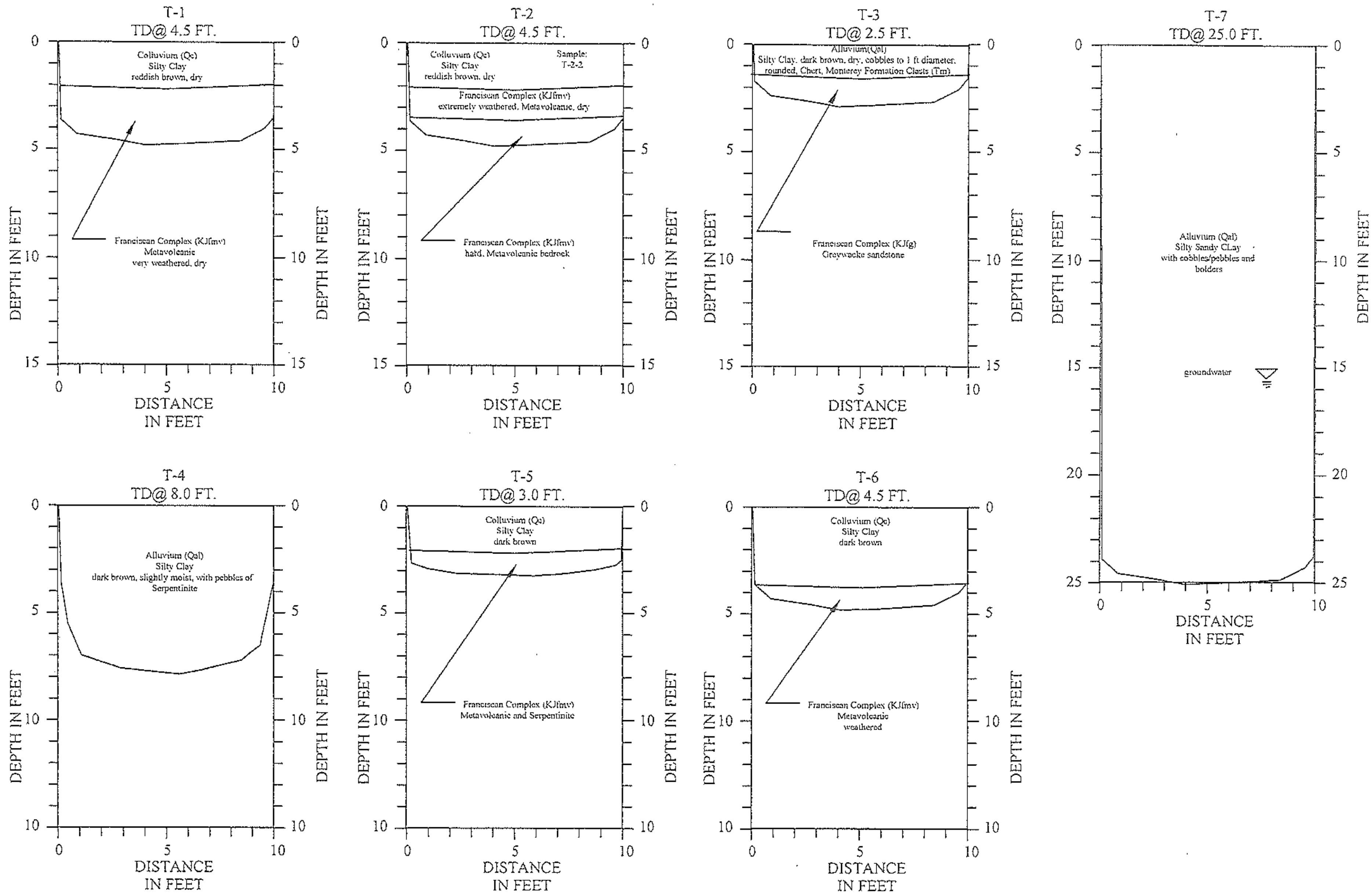
BORING NO. **B-4**

JOB NO. **SL09734-1**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**Madonna - Froom Ranch**	DRILL RIG:	**Mobile B-24**
DRILLING LOCATION:	**See Figure 2, Site Plan**	HOLE DIAMETER:	**6 Inches**
DATE DRILLED:	**May 18, 2016**	SAMPLING METHOD:	**CA and SPT**
LOGGED BY:	**GTV**	HOLE ELEVATION:	**Not Recorded**

Depth of Groundwater: **2.5 Feet** 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	CLAYEY SAND: very dark brown, very moist	CH											
-1.5	soft												
-5	SANDY CLAY: dark brown, very moist	CH		CA	3	3							
-8.5				SPT	6	6							
-9	firm												
-10													



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PROFESSIONAL GEOLOGIST
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GEOLOGIST
STATE OF CALIFORNIA

TRENCHING LOGS

MADONNA - FROOM RANCH
SAN LUIS OBISPO, CALIFORNIA

LOGS
1

PROJECT
SL09734-1

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 D4829-08) 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.

Liquid Limit, Plastic Limit, and Plasticity Index of Soils (ASTM D4318-05) 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.

Particle Size Analysis of Soils (ASTM D422-63R02) is used to determine the particle-size distribution of fine and coarse aggregates. In the test method the sample is separated through a series of sieves of progressively smaller openings for determination of particle size distribution. The total percentage passing each sieve is reported and used to determine the distribution of fine and coarse aggregates in the sample.

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.

One-Dimensional Consolidation Properties of Soils Using Incremental Loading (ASTM D2435-11) is used to determine the magnitude and rate of consolidation of a soil by applying a series of load increments to an undisturbed soil sample and recording sample deformation at selected time intervals. In this test method, a soil specimen is restrained laterally and drained axially while subjected to incrementally applied controlled-stress loading. Each stress increment is maintained until excess pore water pressures are completely dissipated. During the consolidation process, measurements are made of the change in the specimen height and this data is used to determine the relationship between the effective stress and void-ratio or strain, and the rate at which consolidation can occur by evaluating the coefficient of consolidation. The data from the consolidation test is used to estimate the magnitude and rate of both differential and total settlement of a structure or earth-fill.

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

Project:	Madonna Froom Ranch			Date Tested:	June 3, 2016
Client:				Project #:	SL09734-1
Sample:	A	Depth:	1.0 Foot	Lab #:	16516
Location:	B-1			Sample Date:	May 18, 2016
				Sampled By:	GV/TG

Soil Classification
ASTM D2487, D2488

Result:	Very Dark Grayish Brown Sandy Fat CLAY
Specification:	CH

Sieve Analysis
ASTM D422

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	93	
No. 8	90	
No. 16	89	
No. 30	87	
No. 50	84	
No. 100	82	
No. 200	79.6	

Sand Equivalent Cal 217

		SE
1		
2		
3		
4		

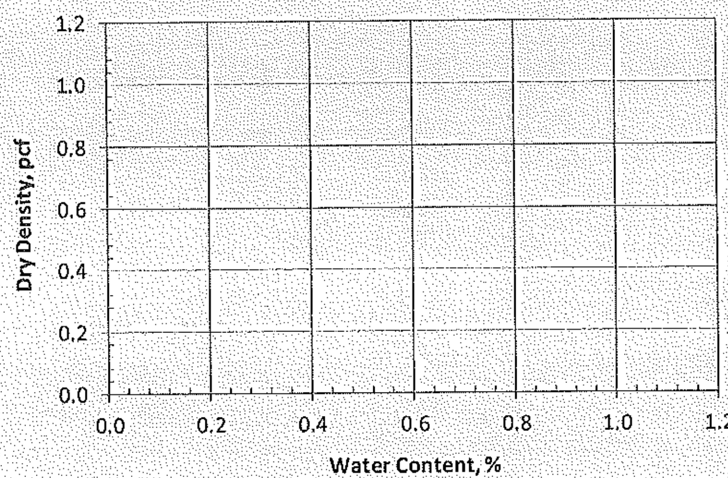
Plasticity Index
ASTM D4318

Liquid Limit:	54
Plastic Limit:	23
Plasticity Index:	**31**

Expansion Index
ASTM D4829

Expansion Index:	**107**
Expansion Potential:	High
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557



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, **Moisture Content** ASTM D2216

Sample	Depth (ft)	Water Content (%)	Dry Density (pcf)	Relative Density	Sample Description
B-1	2.5	38.5			Dark Olive Gray Sandy CLAY
B-2	2.5	27.4			Dark Olive Gray Sandy CLAY

Report By: Aaron Eichman

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

Project:	Madonna Froom Ranch			Date Tested:	June 8, 2016
Client:				Project #:	SL09734-1
Sample:	B	Depth:	1.0 Foot	Lab #:	16516
Location:	B-3			Sample Date:	May 18, 2016
				Sampled By:	GV/TG

Soil Classification
ASTM D2487, D2488

Result:	Dark Gray Fat CLAY
Specification:	CH

Sieve Analysis
ASTM D422

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"		
3/4"		
No. 4	100	
No. 8	100	
No. 16	100	
No. 30	99	
No. 50	98	
No. 100	96	
No. 200	94.7	

Sand Equivalent Cal 217

1		SE
2		
3		
4		

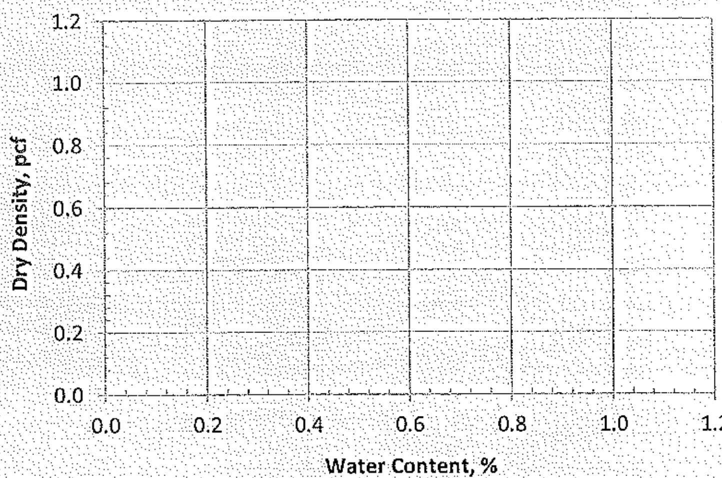
Plasticity Index
ASTM D4318

Liquid Limit:	84
Plastic Limit:	36
Plasticity Index:	**48**

Expansion Index
ASTM D4829

Expansion Index:	**186**
Expansion Potential:	Very High
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557



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, **Moisture Content** ASTM D2216

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

Report By: Aaron Eichman

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

Project:	Madonna Froom Ranch			Date Tested:	June 9, 2016
Client:				Project #:	SL09734-1
Sample:	C	Depth:	2.0 Feet	Lab #:	16516
Location:	T-2			Sample Date:	May 18, 2016
				Sampled By:	GV/TG

Soil Classification
ASTM D2487, D2488

Result:	Dark Reddish Brown Sandy Fat CLAY
Specification:	CH

Sieve Analysis
ASTM D422

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

1		SE
2		
3		
4		

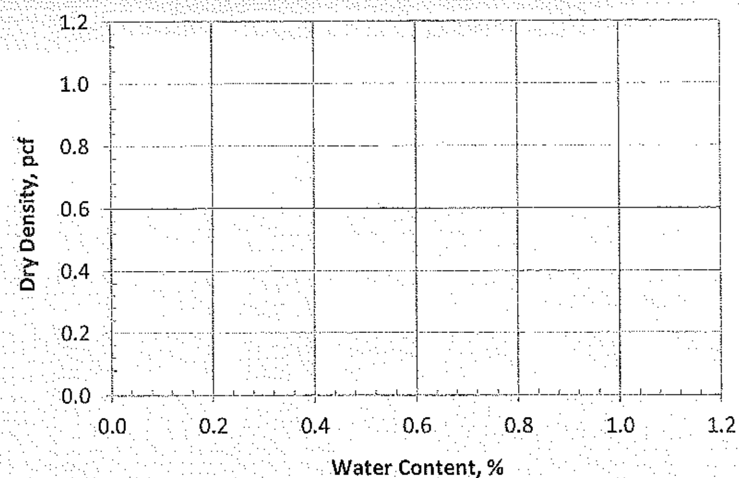
Plasticity Index
ASTM D4318

Liquid Limit:	65
Plastic Limit:	23
Plasticity Index:	**42**

Expansion Index
ASTM D4829

Expansion Index:	**79**
Expansion Potential:	Medium
Initial Saturation, %:	50

Laboratory Maximum Density
ASTM D1557



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, %:				

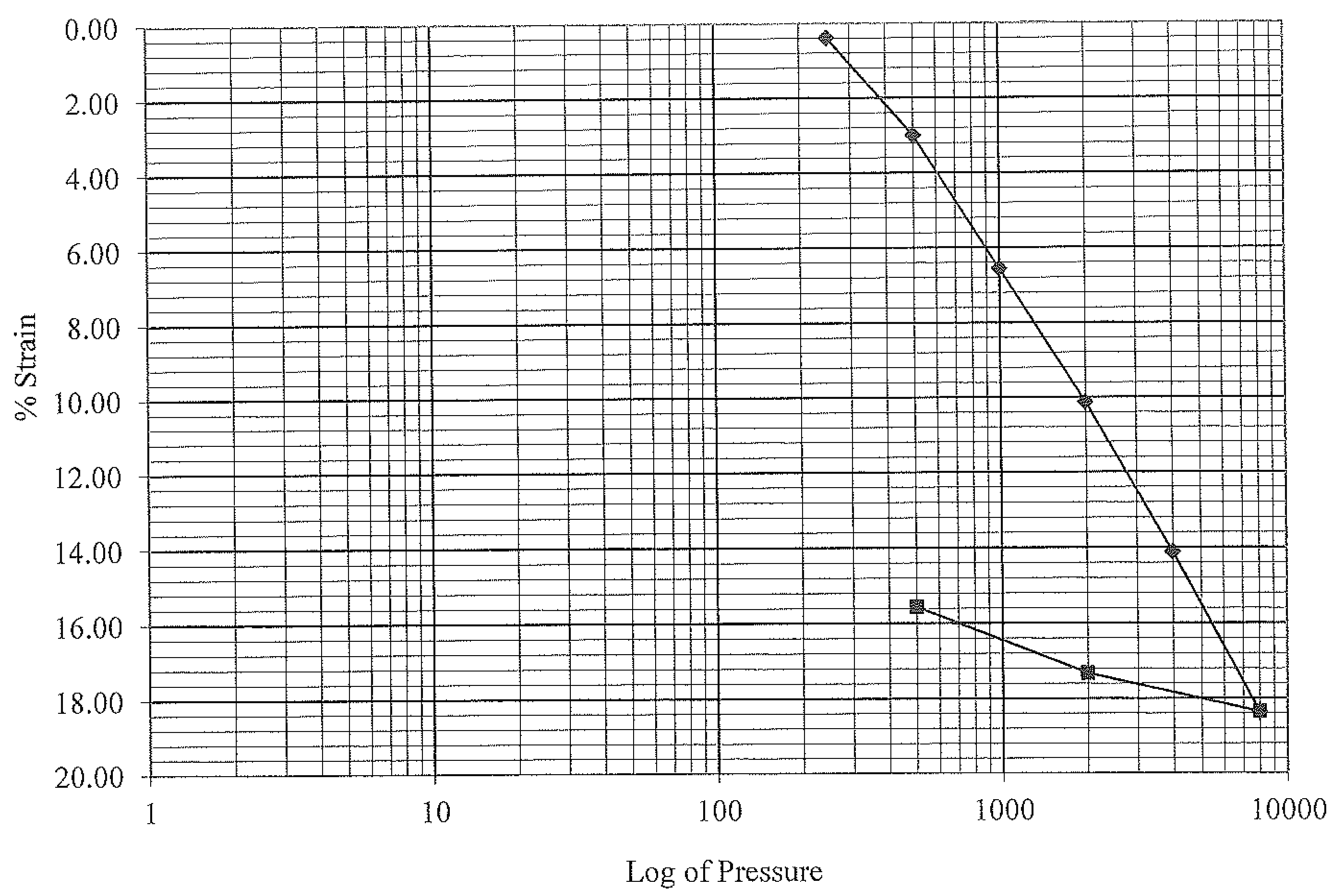
Moisture-Density ASTM D2937, **Moisture Content** ASTM D2216

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

Report By: Aaron Eichman

GeoSolutions, Inc.	CONSOLIDATION REPORT D2435-11	(805) 543-8539

Project:	Madonna Froom Ranch			Date Tested:	6/7/2016
Client:				Project #:	SL09734-1
Sample:	B-1 @ 5'	Depth:	5.0 Feet	Lab #:	16516
Location:	B-1			Sample Date:	5/18/2016
Material:	Very Dark Grayish Brown Sandy CLAY			Sampled By:	GV/TG



Applied Pressure (psf)	% Strain
250	0.38
500	2.99
1000	6.55
2000	10.12
4000	14.11
8000	18.35
2000	17.32
500	15.56

Compression Index, Cc
0.165
Recompression Index, Cr
0.017

Report By: Aaron Eichman

GeoSolutions, Inc.

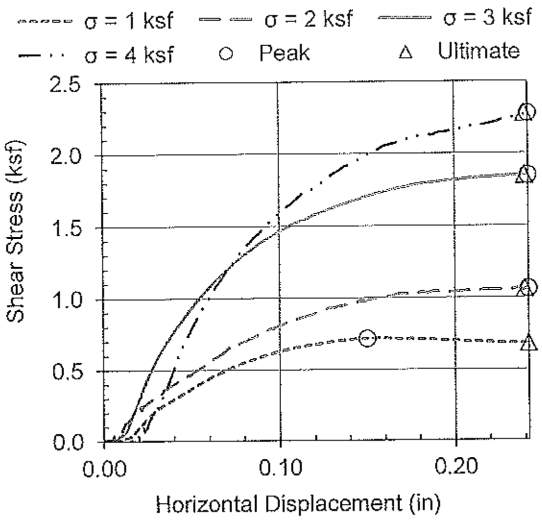
DIRECT SHEAR TEST
SUMMARY REPORT (ASTM D3080)

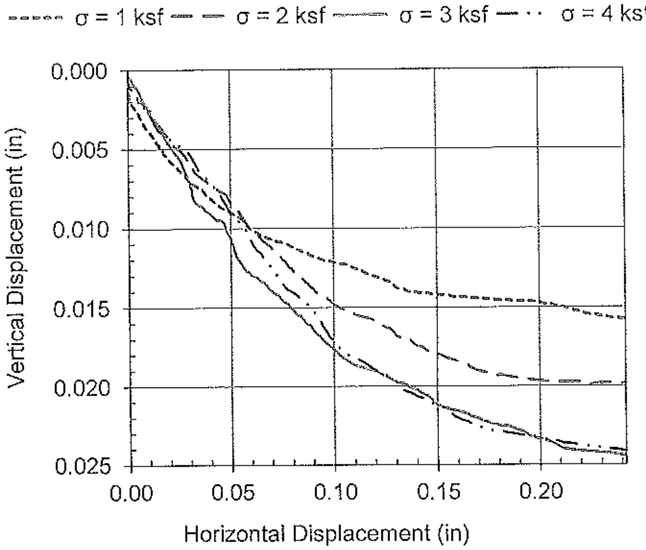
(805) 543-8539

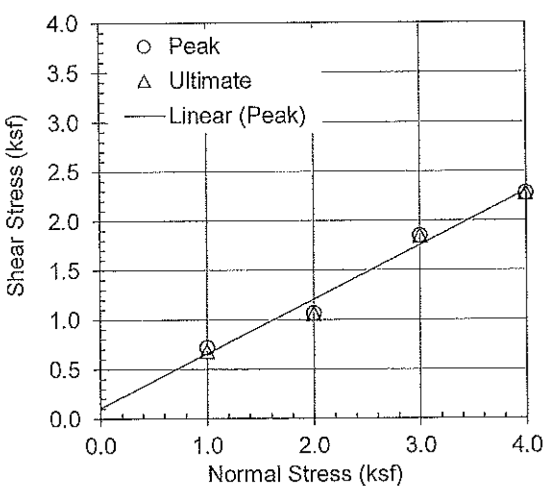
Project:	Madonna Froom Ranch		Project No.: SL09734-1
Client:			Date Tested: 6/9/2016
Sample No.:	B-1 @ 5'	Depth: 5.0 Feet	Lab No.: 16516
Location:	B-1		Checked By: AE

MATERIAL DESCRIPTION	LL	PL	PI	% passing No. 200	Gs *	Sample Type
Very Dark Grayish Brown Sandy CLAY	nm	nm	nm	nm	2.4	in-situ (rings)

* Gs = assumed; nm = not measured







Angle of Internal Friction, ϕ_{peak} (degrees):	28.7
Cohesion, C_{peak} (psf)	110

Initial Conditions	Specimen No. 1	2	3	4
Dry Density	92.3	90.3	87.4	95.7
Water Content (%)	25.8	25.8	25.8	25.8
Diameter (in)	2.42	2.42	2.42	2.42
Sample Height (in)	1.00	1.00	1.00	1.00

Test Data	Specimen No. 1	2	3	4
Normal Stress (ksf)	1.00	2.00	3.00	4.00
Peak Shear Stress (ksf)	0.72	1.06	1.85	2.28
Horiz. Displacenent at Peak Shear (in)	0.15	0.24	0.24	0.24
Ultimate Shear Stress (ksf)	0.68	1.06	1.85	2.28
Horiz. Displ. at Ult. Shear (in)	0.24	0.24	0.24	0.24
Rate of Deformation (in/min)	0.002	0.002	0.002	0.002

Remarks:

Samples were not saturated prior to shearing

APPENDIX C

USGS Design Map Summary Report

USGS Design Map Detailed Report

USGS Design Maps Summary Report

User–Specified Input

Report Title Froom Ranch - Upper Site
Tue June 21, 2016 23:46:53 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 35.24805°N, 120.68614°W

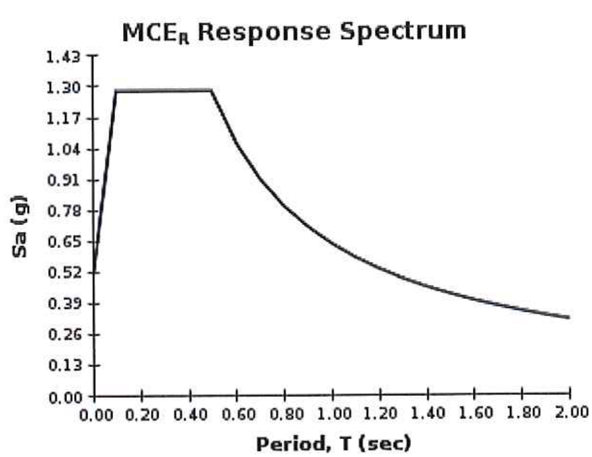
Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

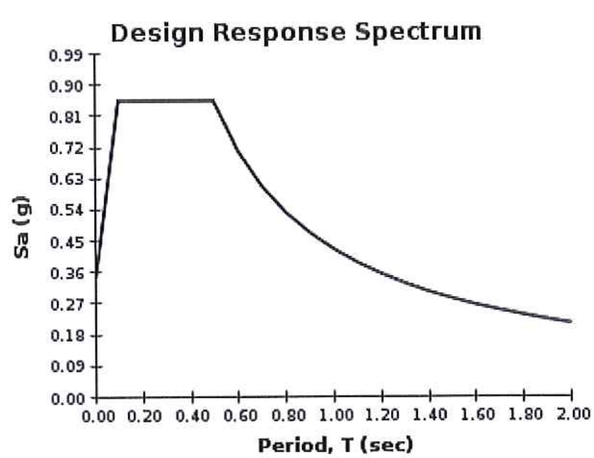
Risk Category I/II/III

USGS–Provided Output

$S_S =$	1.281 g	$S_{MS} =$	1.281 g	$S_{DS} =$	0.854 g
$S_1 =$	0.484 g	$S_{M1} =$	0.637 g	$S_{D1} =$	0.425 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (35.24805°N, 120.68614°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.281$ g

From Figure 22-2 [2] $S_1 = 0.484$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_S$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = C and S_S = 1.281 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.484 g, F_v = 1.316

Equation (11.4-1): $S_{MS} = F_aS_S = 1.000 \times 1.281 = 1.281$ g

Equation (11.4-2): $S_{M1} = F_vS_1 = 1.316 \times 0.484 = 0.637$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

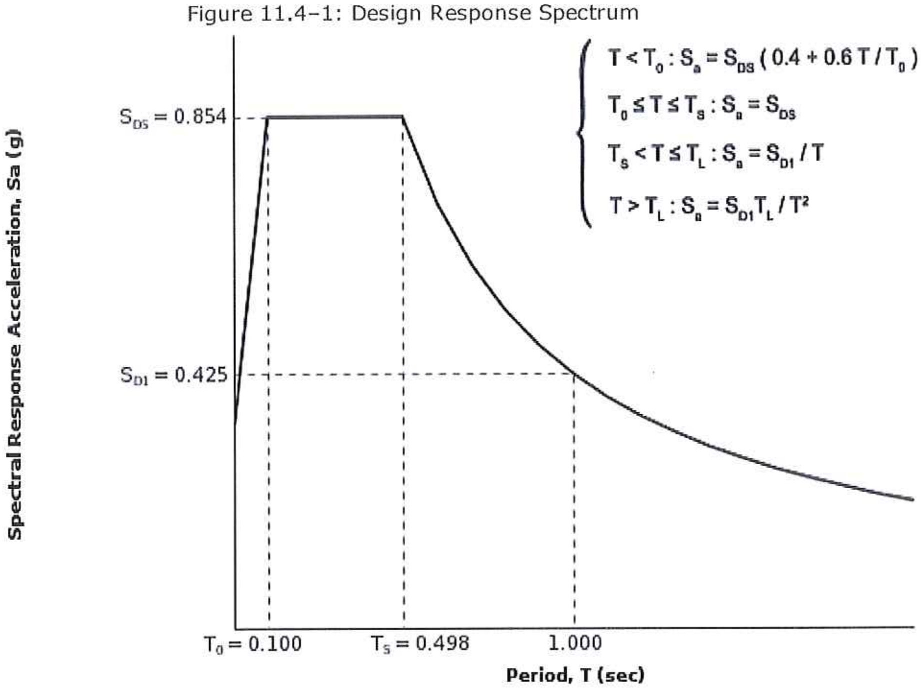
Equation (11.4-3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.281 = 0.854$ g

Equation (11.4-4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.637 = 0.425$ g

Section 11.4.5 — Design Response Spectrum

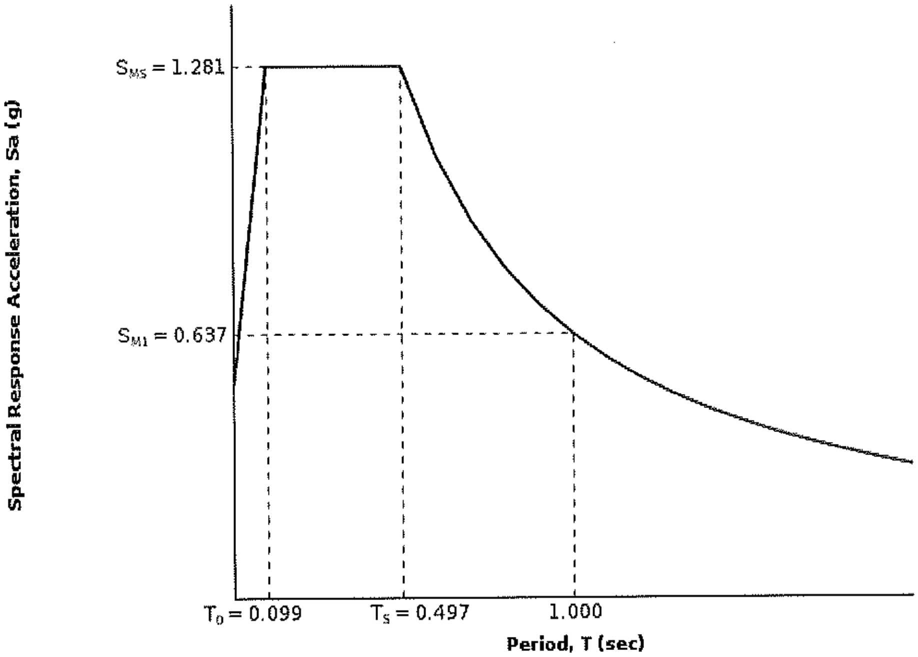
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.541

Equation (11.8-1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.541 = 0.541$ g

Table 11.8-1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	$\boxed{1.0}$
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = C and PGA = 0.541 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.902

From Figure 22-18 [6] C_{R1} = 0.939

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.854 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.425 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

USGS Design Maps Summary Report

User–Specified Input

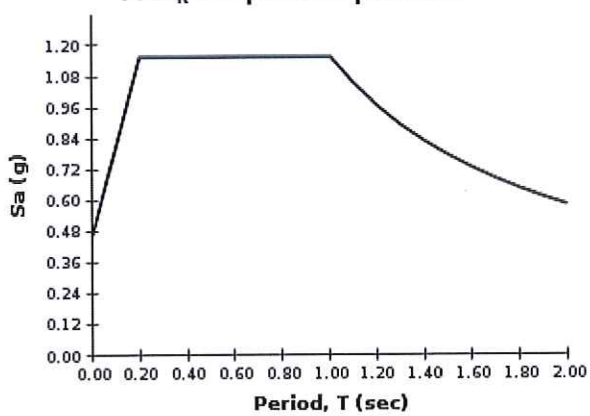
Report Title	Froom Ranch - Lower Site Tue June 21, 2016 23:48:46 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	35.24805°N, 120.68614°W
Site Soil Classification	Site Class E – "Soft Clay Soil"
Risk Category	I/II/III

USGS–Provided Output

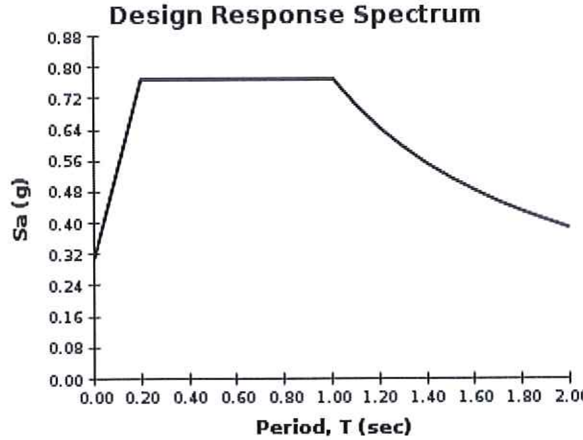
S_S = 1.281 g	S_{MS} = 1.153 g	S_{DS} = 0.769 g
S_1 = 0.484 g	S_{M1} = 1.162 g	S_{D1} = 0.774 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (35.24805°N, 120.68614°W)

Site Class E – "Soft Clay Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.281$ g

From Figure 22-2 [2] $S_1 = 0.484$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class E, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = E and S_S = 1.281 g, F_a = 0.900

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = E and S_1 = 0.484 g, F_v = 2.400

Equation (11.4-1):	$S_{MS} = F_aS_S = 0.900 \times 1.281 = 1.153$ g
Equation (11.4-2):	$S_{M1} = F_vS_1 = 2.400 \times 0.484 = 1.162$ g

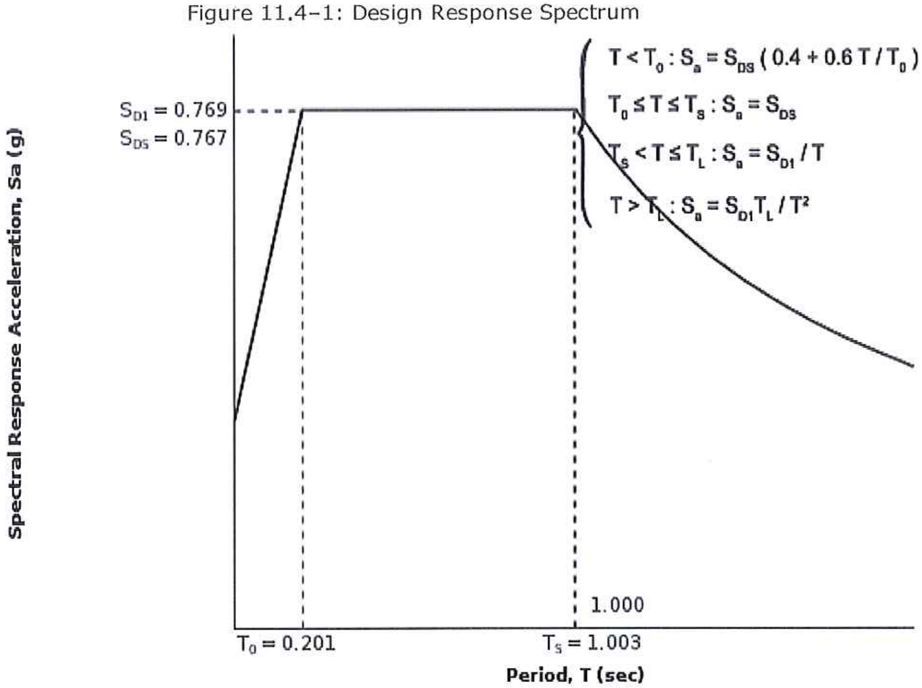
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4-3):	$S_{DS} = ⅔ S_{MS} = ⅔ \times 1.153 = 0.769$ g
Equation (11.4-4):	$S_{D1} = ⅔ S_{M1} = ⅔ \times 1.162 = 0.774$ g

Section 11.4.5 — Design Response Spectrum

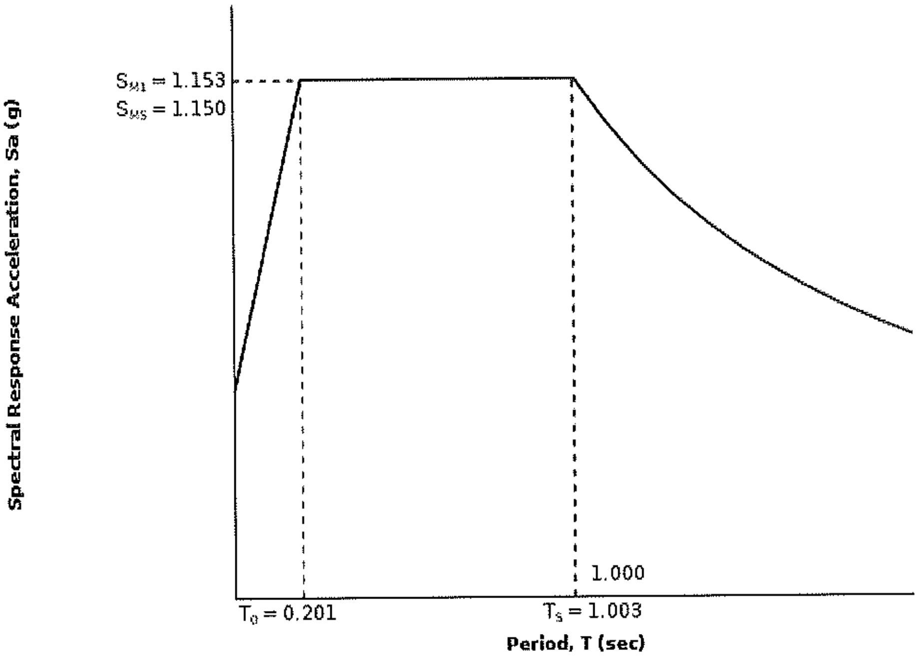
From Figure 22-12 [3]	$T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.541

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 0.900 \times 0.541 = 0.487$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = E and PGA = 0.541 g, F_{PGA} = 0.900

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.902

From Figure 22-18 [6] C_{R1} = 0.939

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.769 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.774 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX D

Preliminary Grading Specifications

Key and Bench with Backdrain

PRELIMINARY GRADING SPECIFICATIONS

A. General

1. 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.

2. 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.

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

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

B. Obligation of Parties

1. 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.

2. 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.

3. 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

1. 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.

2. 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.

3. 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

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

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

3. 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

1. 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.

2. 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 1804 of the 2013 California Building Code unless specifically modified by the Soil Engineer/Engineering Geologist.

3. 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

1. 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.

2. 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

1. 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.

2. 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.

3. 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.

4. 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.

H. Drainage

1. 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.

2. 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.

3. 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.

4. 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.

5. 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.

6. 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

1. 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.

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

J. Underground Facilities Construction

1. 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.

2. 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.

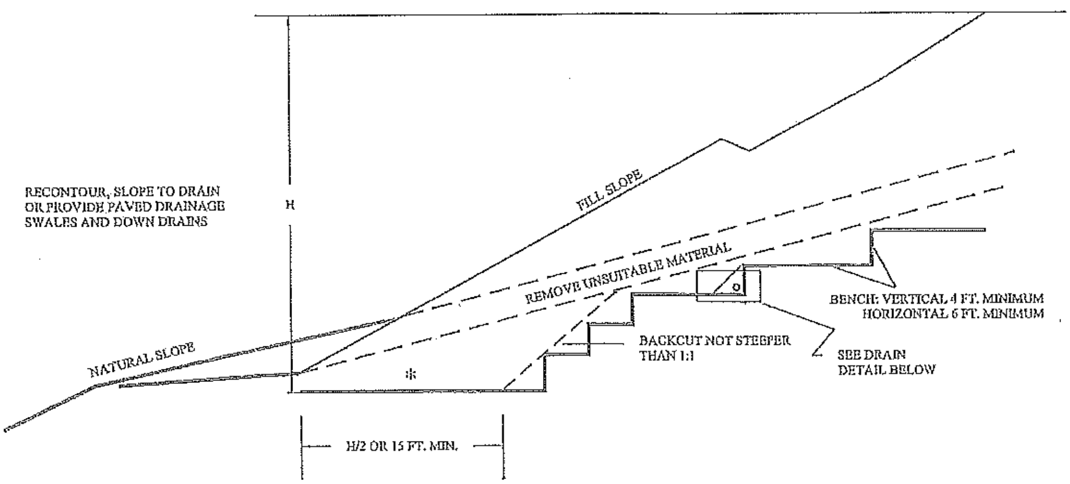
3. On-site inorganic soils, or approved import, may be used as utility trench backfill. Proper compaction of trench backfill will be necessary under and adjacent to structural fill, building foundations, concrete slabs, and vehicle pavements. In these areas, backfill should be conditioned with water (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

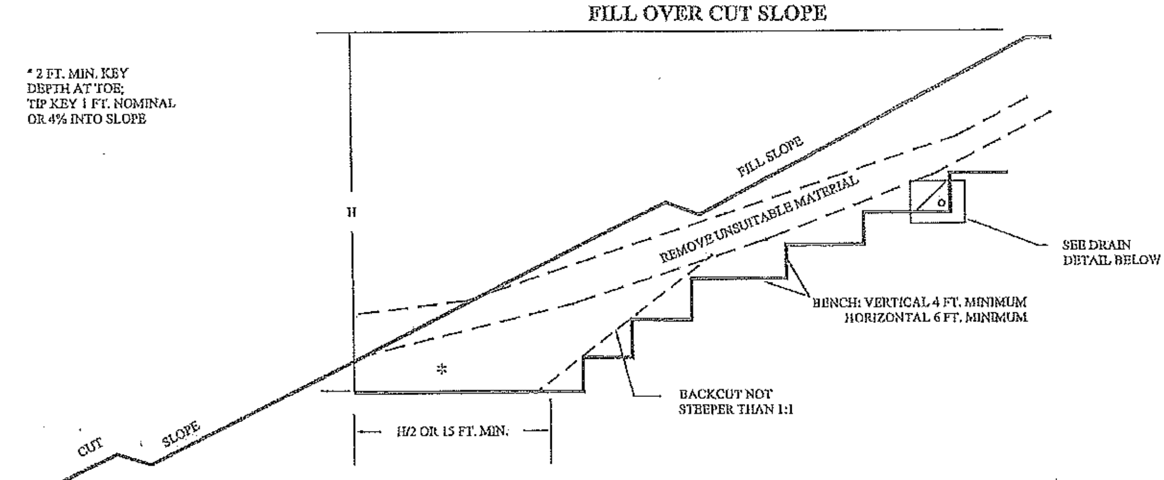
1. After the completion of work, a report should be prepared by the Soils Engineer retained to provide such services. The report should including locations and elevations of field density tests, summaries of field and laboratory tests, other substantiating data, and comments on any changes made during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

2. 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 Chapter 18 of the 2013 CBC.

FILL OVER NATURAL SLOPE



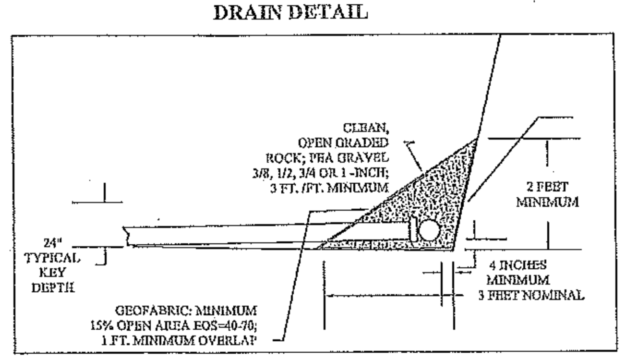
FILL OVER CUT SLOPE



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.
220 High Street
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(805) 543-8539 Fax: (805) 543-2171

KEY AND BENCH WITH BACKDRAIN

DETAIL
A