SOILS ENGINEERING REPORT AND ENGINEERING GEOLOGY INVESTIGATION
6226 ONTARIO ROAD
APN: 076-114-052, SAN LUIS OBISPO AREA,
SAN LUIS OBISPO COUNTY, CALIFORNIA

PROJECT SL10844-1

Prepared for

Art Weldon
Post Office Box 369
Arroyo Grande, California 93421

Prepared by

GEOSOLUTIONS, INC.
220 HIGH STREET
SAN LUIS OBISPO, CALIFORNIA 93401
(805) 543-8539

September 21, 2018

GEO SOLUTIONS

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Client:
Art Weldon
P.O. Box 369
Arroyo Grande, CA
93421

Project name:
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area, San Luis
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SOILS ENGINEERING REPORT AND ENGINEERING GEOLOGY INVESTIGATION

Dear Mr. Weldon:

This Soils Engineering Report and Engineering Geology Investigation has been prepared for two proposed residences and associated roadway to be located at 6226 Ontario Road, in the San Luis Obispo area of San Luis Obispo County, California.

The two sites will be referred to as Parcel 2 building envelope (Lower Site) and Parcel 1 building envelope (Upper Site). Geotechnically, the sites are suitable for the proposed developments provided the recommendations in this report for site preparation, earthwork, foundations, slabs, retaining walls, and pavement sections are incorporated into the design.

Shallow rock was contacted across the majority of the two proposed sites (Upper and Lower) at shallow depths. Grading will be required to create a level and suitable building pads. However, to minimize grading and utilize the underlying bedrock for foundation support, we recommend that all footings be placed into rock and grading be limited with respect to over-excavations.

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) 614-6333.

Sincerely,
GeoSolutions, Inc.

Patrick B. McNeill

Patrick B. McNeill, PE
Principal

REGISTERED PROFESSIONAL ENGINEER
PATRICK B. MCNEILL
No. 59577
CIVIL
STATE OF CALIFORNIA

Jeffrey Pfost, CEG 2493
Principal Engineering Geologist

PROFESSIONAL GEOLOGIST
JEFFREY PFOST
NO. 2493
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

220 High Street
San Luis Obispo CA 93401
805.543.8539

1021 Tama Lane, Suite 105
Santa Maria, CA 93455
805.614.6333

PO Box 30159
Santa Barbara, CA 93105
805.966.2200

info@geosolutions.net

sbinfo@geosolutions.net

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1.0 INTRODUCTION

This report presents the results of the geotechnical investigation for two proposed residences and the associated roadway to be located at 6226 Ontario Road, in the San Luis Obispo area of San Luis Obispo County, California. See Figure 1: Site Location Map for the general location of the project area. Figure 1: Site Location Map was obtained from the computer program *Topo USA 8.0* (DeLorme, 2009).

1.1 Site Description

6226 Ontario Road is located at 35.204 degrees north latitude and -120.698 degrees west longitude at a general elevation of 90 feet (Lower Site) and 420 feet (Upper Site) above mean sea level. The property is "L" in shape, consists of 2 parcels and 161.71 acres in size. The project property will hereafter be referred to as the "Lower Site" and the "Upper Site". See Figure 2: Site Plan for the general layout of the Sites.

The Lower Site is situated on a generally level area adjacent to Ontario Road. The Lower Site resides above Ontario Road and a moderate slopes descends downward from the Lower Site to Ontario Road. The Lower Site is currently open grass lands.

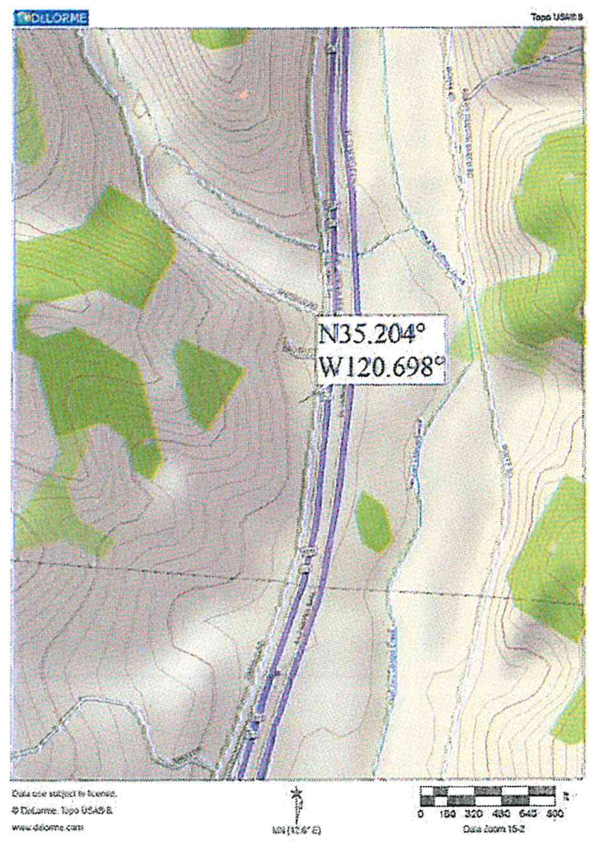

Figure 1: Site Location Map

The Upper Site is located several hundred feet above Ontario Road on a ridge that descends to the north and south. Oak trees, grasses and shrubs currently vegetate the Upper Site.

An existing jeep road currently provides access to both building envelopes.

1.2 Project Description

The proposed single-family residences are to be located in the eastern portion of Parcel 1 and 2. The Tentative Parcel Map depicts 3 building envelops and associated roadway, however at the time of this report the project scope is limited to Parcel 1 building envelope (Upper Site) and Parcel 2 building envelope (Lower Site) and the associated roadway. The structures are anticipated to be one or two stories in height. At the time of the preparation of this report, the proposed single-family residences are to be constructed using light wood framing. Retaining walls are expected to be constructed as part of this project.

It is anticipated that the proposed single-family residences will utilize a slab-on-grade lower floor systems. Dead and sustained live loads are currently unknown, but they are anticipated to be relatively light with maximum continuous footing and column loads estimated to be approximately 2.0 kips per linear foot and 30 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 and trenches 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.

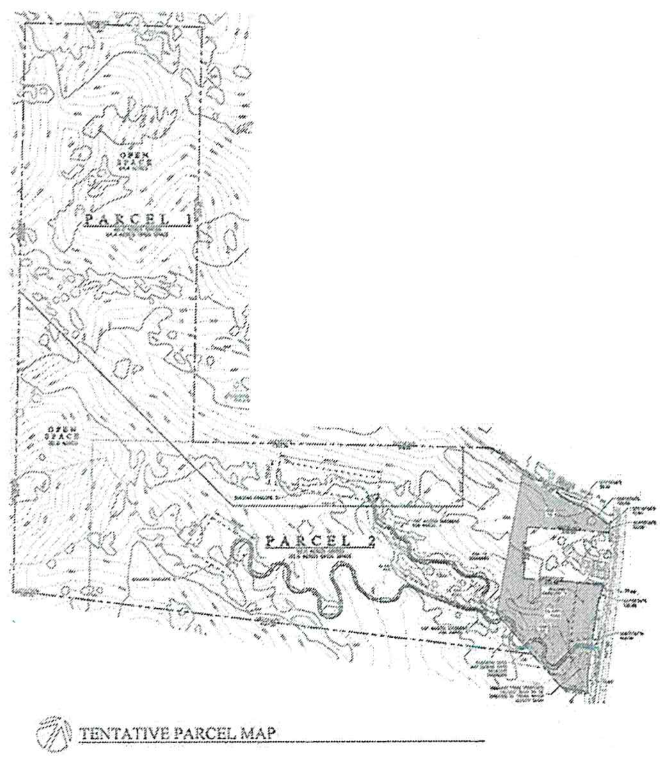

Figure 2: Site Plan

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

3.0 FIELD AND LABORATORY INVESTIGATION

The field investigation was conducted on July 23, 2018 using backhoe equipment. Eight twenty-four inch wide exploratory trenches were excavated to a maximum depth of 7.5 feet below ground surface (bgs) at the approximate locations indicated on Plate 1. Sampling methods included bulk bag samples.

Data gathered during the field investigation suggest that the soil materials at the Site consist of interbedded layers of colluvial overlying competent formational material. The surface material at the Site generally consisted of varying shades of silty SAND (SM) encountered in a dry to slightly moist condition to approximately 1.0 to 3.0 feet bgs. The sub-surface materials consisted of white SANDSTONE to light brown CLAYSTONE encountered in a dry and hard condition. Light gray silty SAND (SM) interpreted as fill was encountered in trench T-2 to a depth of 2.0 bgs. The Lower Site did have some undocumented fill on the eastern portion of the building envelop.

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	Maximum Dry Density, γ_d (pcf)	Optimum Moisture (%)	Angle of Internal Friction, ϕ (deg.)	Cohesion, c (psf)	Plasticity Index	Fines Content (%)	Compression Index, C_c	Recompression Index, C_r
B	Very Dark Grayish Brown Clayey SAND with Gravel	SC	2	Very Low	-	-	-	-	12	25.6	-	-
C	Very Dark Grayish Brown Clayey SAND	SC	10	Very Low	-	-	-	-	8	24.7	-	-

4.0 GEOLOGIC RECOMMENDATIONS

The proposed development is geologically suitable provided that the recommendations provided herein are implemented. The following are recommended for implementation at the Site.

1. It is recommended that numerical slope stability analyses be conducted on cut or fill slopes constructed steeper than 2-to-1 (horizontal to vertical). Locally steeper slopes may be allowed depending the results of a slope stability analysis.

2. Isolated seepage within formational units should be anticipated. Surface drainage facilities (graded swales, gutters, positive grades, etc.) are recommended at the base of cut slopes that allow surfacing water to be transferred away from the base of the slope. The project designer is recommended to offer specific design criteria for mitigation of water drainage behind walls and other areas of the site. This is especially imperative upslope of retaining walls for residences. Subsurface drainage systems should not be connected into conduit from surface drains and should not connect to downspout drainage pipes.

3. Surface drainage should be controlled to prevent concentrated water-flow discharge onto either natural or constructed slopes. Surface drainage gradients should be planned to prevent ponding and promote drainage of surface water away from building foundations, edges of pavements and sidewalks or natural or man-made slopes. For soil areas we recommend that a minimum of two (2) percent gradient be maintained.

4. Excavation, fill, and construction activities should be in accordance with appropriate codes and ordinances of the County of San Luis Obispo. In addition, unusual subsurface

conditions encountered during grading such as springs or fill material should be brought to the attention of the Engineering Geologist and Soils Engineer.

5. Rock rip-rap is recommended for concentrated drainage outfall locations that do not discharge onto paved or exposed rock surfaces. It is recommended that geotextile fabric (Enkamat 7010 or similar) be placed underneath the rip-rap and installed per the manufacturer's recommendations.

6. Gutters are recommended to be installed along all sloped rooflines. Gutter downspouts should not allow concentrated drainage to discharge near the residence foundations but rather should convey the water in solid piping away from the residence and toward drainage facilities.

5.0 GEOLOGIC FINDINGS

5.1 Regional Geologic Conditions

The Site is located in the vicinity of the San Luis Range of the Coast Range Geomorphic Province of California. The Coast Ranges lie between the Pacific Ocean and the Sacramento-San Joaquin Valley and trend northwesterly along the California Coast for approximately 600 miles between Santa Maria and the Oregon border.

The Site lies within geologic terrain known as the Irish Hills Sub-block of the San Luis/Pismo Structural Block (Lettis and Hall, 1994). The block is bordered on the north by the Los Osos Fault Zone and to the south by the Hosgri Fault Zone. Past tectonic activity along these and other faults in the vicinity have created complex structural and stratigraphic relationships between the various rock units. The principal structural features that account for bedrock and related topography in the area are the Pismo syncline, the Edna fault, the Los Osos fault, San Luis Bay fault and the Hosgri fault.

5.2 Local Geology

Locally, the site is located within the Squire Member, Belleview Member, Gregg Member, and Miguelito Member of the Pismo Formation as depicted on Plate 1, Site Engineering Geology Map. Hall, 1973, Dibblee, 2006 and Wiegers, 2011 mapped the development as underlain by lower Pliocene to upper Miocene age Pismo Formation units. Information derived from subsurface exploration was used to classify subsurface soil and formational units and to supplement geologic mapping.

As described in Section 3.0, eight trenches were excavated to determine the depth to formational units and determine the quality of the formational material. Plate 1 depicts the Squire Member (Tps), Belleview Member (Tpb and Tpbc), Gregg Member (Tpg), and Miguelito Member (Tpms) of the Pismo Formation throughout the property. Trench logs are presented in Appendix A.

5.2.1 Squire Member

Wiegers, 2011 maps the western portion of the proposed roadway as within Squire Member of the Pismo Formation (Tps). Wiegers, 2011 describes the Squire Member as "Massive, white, calcareous, fine- to medium-grained, quartzose to arkosic, silty sandstone. Sand grains subrounded to subangular; 75-80% quartz, 15-20% feldspar, less than 15% mafic minerals (Hall, 1973). Contains lenses of white, well-rounded pebbles and cobbles of Monterey and Obispo Formation clasts north of Edna Fault.Basal conglomerate of rounded chert and basalt cobbles near mouth of San Luis Obispo Creek." The Squire Member of the Pismo Formation (Tps) at the site consisted of white sandstone, coarse grained, slightly to moderately weathered (W3-W5), and soft to very soft (H6-H7).

5.2.2 Belleview Member

Wiegers, 2011 maps the lower portion of the proposed roadway (eastern portion) as within Belleview Members of the Pismo Formation (Tpb and Tpbc). Wiegers, 2011 describes the Belleview Member as "Light-gray, bedded, resistant sandstone and interbedded siltstone. Sandstone medium-grained; 60% quartz, 30% feldspar, locally 15% rock fragments (Hall, 1973). Tpbc – Interbedded, buff claystone, siltstone and fine-grained sandstone. Claystone speroidally fractured. Sandstone beds locally fossiliferous." The Belleview Member of the Pismo Formation (Tpb and Tpbc) at the site consisted of white sandstone, medium grained, slightly to moderately weathered (W3-W5), and soft to very soft (H6-H7) and brown claystone, thinly bedded, highly fractured to friable.

5.2.3 Gregg Member

Wiegers, 2011 maps the lower portion of the proposed roadway and southern portion of building envelop 1 as within the Gregg Member of the Pismo Formation (Tpg). Wiegers, 2011 describes the Gregg Member as "Massive, white, buff-weathering sandstone, soft to resistant, medium-grained; 65% quartz, 30% feldspar, clay 4%, mafic minerals 1% (Hall, 1973)." The Gregg Member of the Pismo Formation (Tpg) at the site consisted of white to tan sandstone, medium grained, slightly to moderately weathered (W3-W5), and soft to very soft (H6-H7).

5.2.4 Miguelito Member

Wiegers, 2011 maps the northern portion of building envelop 1 as within the Miguelito Member of the Pismo Formation (Tpms). Wiegers, 2011 describes the Miguelito Member as "Poorly bedded siltstone, diatomaceous siltstone and sandy siltstone." The Miguelito Member of the Pismo Formation (Tpms) at the site consisted of light gray siltstone and sandstone, highly fractured, slightly to moderately weathered (W3-W5), and soft to very soft (H6-H7).

5.3 Surface and Ground Water Conditions

Surface drainage follows the topography south and east toward existing drainage gullies. A drainage gully is located west of building envelope 1. Surface drainage should be directed away from proposed structures and slopes. No springs or seeps were observed at the project. Groundwater was not observed within any trenches or borings.

6.0 LANDSLIDES

Wiegers, 2011 did not map landslides in the vicinity of the property. During site mapping and review of aerial photography, landslides were not observed at the Site. Plate 4 presents an aerial photograph. There is a low rockfall potential to affect the proposed building envelopes based on the lack of boulders upslope of the proposed development.

7.0 REGIONAL FAULTING AND SEISMICITY

Many faults are mapped in the foothills of the Santa Ynez Mountains and coastal plains of Santa Barbara of varying types, lengths, and age. An active fault is one that shows evidence of displacement within the last 11,000 years (Recent epoch). A fault which displaces deposits of late Pleistocene age (500,000 to 11,000 years) but with no evidence of Recent movement is termed potentially active. Inactive fault is one that displace rocks of early Pleistocene or older (500,000 years or older).

Similar to the surrounding areas, the Site may be affected by moderate to major earthquakes centered on one of the known large, active faults listed in Table 2 below. Moment magnitudes are expressed, although any significant event on these faults could result in moderate to severe ground shaking at the subject site. The potential for ground failure of any portion of the Site during ground shaking is considered low.

Table 2: Active Faults near the Subject Property

Closest Active Faults to Site	Approximate Distance (miles)	Moment Magnitude (Mw)
Los Alamos	5.0	6.8
Hosgri	7.0	7.3
San Andreas	42.0	6.9

The closest known active portion of a Holocene age fault is an active portion of the Los Alamos fault that is located approximately 5.0 miles north of the Site (Jennings, 2010). Plate 3 is a Regional Fault Map for the area. The San Andreas fault is the most likely active fault to produce ground shaking at the Site although it is not expected to generate the highest ground accelerations because of its distance from the Site.

The Alquist-Priolo Earthquake Fault Zoning Act of 1972 requires that the California State Geologist establish Earthquake Fault Zones around the surface traces of active faults and to issue appropriate maps. The subject site is not located within an Earthquake Fault Zone.

8.0 TSUNAMIS AND SEICHES

Tsunamis and seiches are two types of water waves that are generated by earthquake events. Tsunamis are broad-wavelength ocean waves and seiches are standing waves within confined bodies of water, typically reservoirs. As the building envelopes are at elevations over 80 and 400 feet, the potential for a tsunami to affect the Site is low.

Flooding associated with a seismic event (seiche) is considered low due to the absence of a body of water upslope of the property.

9.0 FLOODING AND SEVERE EROSION

The site is not located within or near the 100-year or 500-year flood zone based on Federal Emergency Management Agency flood zone maps (FEMA, 2012).

The surficial and formational deposits are subject to erosion where not covered with vegetation or hardscape. The potential for severe erosion is considered low provided that vegetation and erosion control measures are implemented immediately after the completion of grading.

10.0 ON-SITE SEPTIC SYSTEMS

A concurrent percolation testing report is being performed by this firm. The location of the percolation test borings are presented on Plate 1A. It is recommended that the proposed septic systems not be located near adjacent slopes to minimize the potential for effluent to affect the natural slopes.

11.0 SEISMIC DESIGN CONSIDERATIONS

Estimating the design ground motions at the Site depends on many factors including the distance from the Site to known active faults; the expected magnitude and rate of recurrence of seismic events produced on such faults; the source-to-site ground motion attenuation characteristics; and the Site soil profile characteristics. According to section 1613 of the 2016 CBC (CBSC, 2016), 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 *ASCE 7 2010 Minimum Design Loads for Buildings and Other Structures*, hereafter referred to as ASCE7-10 (ASCE, 2013). The Site soil profile classification (Site Class) can be determined by the average soil properties in the upper 100 feet of the Site profile and the criteria provided in Table 20.3-1 of ASCE7-10.

Spectral response accelerations, peak ground accelerations, and site coefficients provided in this report were obtained using the computer-based U.S. Seismic Design Map tool available from the United States Geological Survey website (USGS, 2013). This program utilizes the methods developed in the 2010 ASCE 7 with user-inputted Site latitude and longitude coordinates to calculate seismic design parameters and response spectra (both for period and displacement) for soil profile Site Classes A through E.

Site coordinates of 35.204 degrees north latitude and -120.698 degrees west longitude were used in the web-based probabilistic seismic hazard analysis (USGS, 2013). Based on the results from the in-situ tests performed during the field investigation, the Site was defined as **Site Class D** "Stiff Soil" profile per ASCE7-10, Chapter 20. Relevant seismic design parameters obtained from the program area summarized in Table 3: Seismic Design Parameters. Refer to **Appendix C** for more information regarding the seismic hazard analysis performed for the project and detailed results.

Table 3: Seismic Design Parameters

Site Class	D "Stiff Soil"
Seismic Design Category	D
1-Second Period Design Spectral Response Acceleration, S_{D1}	**0.501g**
Short-Period Design Spectral Response Acceleration, S_{DS}	**0.913g**
Site Specific MCE Peak Ground Acceleration, PGA_M	**0.590g**

12.0 LIQUEFACTION HAZARD ASSESSMENT

12.1 Liquefaction Potential

Liquefaction occurs when saturated cohesionless soils lose shear strength due to earthquake shaking. Ground motion from an earthquake may induce cyclic reversals of shear stresses of large amplitude. Lateral and vertical movement of the soil mass combined with the loss of bearing strength can result from this phenomenon. Liquefaction potential of soil deposits during earthquake activity depends on soil type, void ratio, groundwater conditions, the duration of shaking, and confining pressures on the potentially liquefiable soil unit. Fine, poorly graded loose sand, shallow groundwater, high intensity earthquakes, and long duration of ground shaking are the principal factors leading to liquefaction.

The determination that Site soils are liquefiable was made following guidelines set forth in the "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, 1997" as summarized by Youd and Idriss (2001). The procedure is termed the "simplified procedure" and is the current standard of care for liquefaction analysis.

Because material found at the Site is rock rather than soil, there is no potential for liquefaction, seismically induced settlement or differential settlement. Rock material differs from soil in that it cannot be saturated, cohesion is considered infinite and relative density is not applicable. Assuming the rock material encountered at the Site accurately represents these conditions, liquefaction potential does not apply.

13.0 GENERAL SOIL-FOUNDATION DISCUSSION

It is anticipated that a graded engineered fill pads will be constructed for the proposed residences with all foundations excavated into competent formational material (rock). The intent of this recommendation is to create level building pads but not over-excavate the cut portions of the pads and extend all foundations to rock.

All foundations are to be excavated into uniform material (rock) 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.

14.0 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.

2. The presence of loose surface soils.

3. The presence of shallow, hard bedrock materials. Difficult digging/excavation conditions are anticipated during construction.

14.1 Preparation of Building Pad

1. It is anticipated that grading will be limited to creating level pads and all footings will extend to rock.

2. For slab-on-grade construction with footings founded a minimum of 12 inches into uniform competent formational material (rock), 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 over-excavated beneath the slab at least 12 inches below finished pad elevation or to competent material; whichever is greatest. The exposed surface should be moisture conditioned to slightly above optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-12).

3. If fill areas are constructed on slopes greater than 10-to-1 (horizontal-to-vertical), we recommend that benches be cut every four (vertical) 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.

14.2 Preparation of Paved Areas

1. Pavement areas should be excavated to 1 foot below approximate sub-grade elevation (Base Section). 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 90 percent (ASTM D1557-12_{e1} test method).

2. 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 D1557-12_{e1} test method at slightly above optimum.

3. Roadway fill must comply with all key and bench requirements in section 14.1.3.

4. 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.

14.3 Pavement Design Standard

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

2. As indicated previously in Section 6.2, the top 12 inches of sub-grade soil under pavement sections should be compacted to a minimum relative density of 95 percent based on the ASTM D1557-12$_{e1}$test method at slightly above optimum moisture content. Aggregate bases and sub-bases should also be compacted to a minimum relative density of 95 percent based on the aforementioned test method.

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

14.4 Conventional Foundations

1. Conventional continuous and spread footings with grade beams may be used for support of the proposed structure(s).

2. Minimum footing and grade beam sizes and depths in rock should conform to the following table, as observed and approved by a representative of GeoSolutions, Inc.

Table 4: Minimum Footing and Grade Beam Recommendations

	Perimeter Footings	Grade Beams
Minimum Width	12 inches (one story) 15 inches (two story)	12 inches
Embedment Depth	12 inches (one story) 18 inches (two story)	12 inches (12 inches into rock)
Minimum Reinforcing*	2 #4 bars (1 top / 1 bottom)	2 #4 bars (1 top / 1 bottom)
Spacing	-	25 feet on-center each way

* Steel should be held in place by stirrups at appropriate spacing to ensure proper positioning of the steel.

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

4. An allowable dead plus live load bearing pressure of 3,000 psf may be used for the design of footings founded in rock.

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

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

7. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the rock and the bottom of the footings. For resistance to lateral loads, a friction factor of 0.60 may be utilized for sliding resistance at the base of footings extending a minimum of 12 inches into rock. A passive pressure of 450-pcf equivalent fluid weight may be used against the side of shallow footings in engineered fill. If friction and passive pressures are combined to resist lateral forces acting on shallow footings, the lesser value should be reduced by 50 percent.

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

9. Foundation design should conform to the requirements of Chapter 18 of the latest edition of the CBC (CBSC, 2016).

10. 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.

11. The minimum footing setback distance from ascending or descending slope steeper than 3-to-1 (horizontal-to-vertical) but less than 1-to-1 must be maintained. See Figure 3: 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.

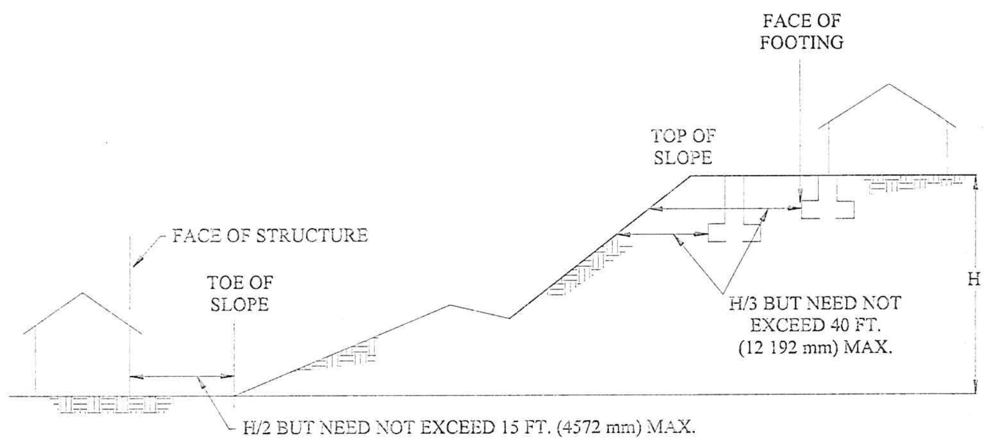

Figure 3: Setback Dimensions – Slope Gradients Between 3-to-1 and 1-to-1

14.5 Slab-On-Grade Construction

1. Concrete slabs-on-grade and flatwork should not be placed directly on unprepared native materials. Preparation of sub-grade to receive concrete slabs-on-grade and flatwork should be processed as discussed in the preceding sections of this report. Concrete slabs should be placed only over sub-grade that is free of loose, soft soil and debris and that has been lightly pre-moistened, with no associated testing required.

2. Concrete slabs-on-grade should be in conformance with the recommendations provided in Table 5: 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. 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 5: Minimum Slab Recommendations

Minimum Thickness	4 inches
Reinforcing*	#3 bars at 18 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	

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 4: 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.

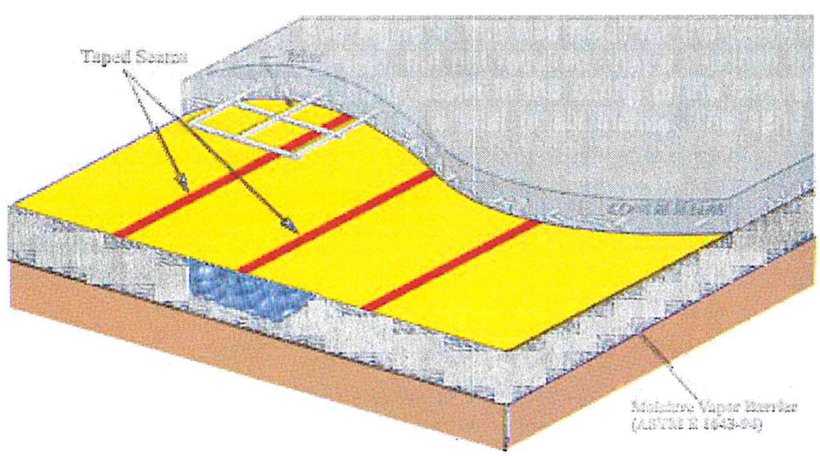

Figure 4: Sub-Slab Detail

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.

14.6 Exterior Concrete Flatwork

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

2. 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.

3. As an alternative, interlocking concrete pavers may be utilized for exterior improvements in lieu of reinforced concrete flatwork. Concrete pavers, when installed in accordance with manufacturers' recommendations and industry standards (ICPI), allow for a greater degree of soil movement as they are part of a flexible system. If interlocking concrete pavers are selected for use in the driveway area, the structural section should be underlain by a woven geotextile fabric, such as Mirafi 500x or equivalent, to function as a separation layer and to provide additional support for vehicle tire loads.

14.7 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 6: Retaining Wall Design Parameters and Figure 5: 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 6: Retaining Wall Design Parameters

Lateral Pressure and Condition	Equivalent Fluid Pressure, pcf
Static, Active Case, ($\gamma' K_A$)	35
Static, At-Rest Case, ($\gamma' K_O$)	50
Static, Passive Case ($\gamma' K_P$)	450

2. The above values for equivalent fluid pressure are based on retaining walls having level retained surfaces, having an approximately vertical surface against the retained material, and retaining granular backfill material or engineered fill composed of native soil within the active wedge. See Figure 5: Retaining Wall Detail and Figure 6: Retaining Wall Active and Passive Wedges for a description of the location of the active wedge behind a retaining wall.

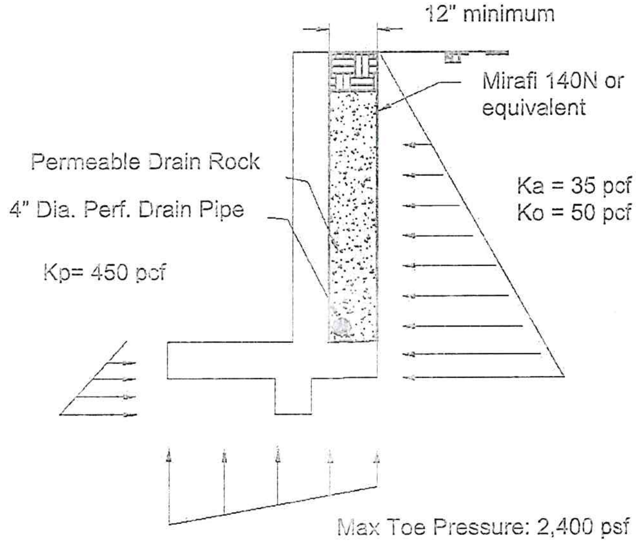

Figure 5: Retaining Wall Detail

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.

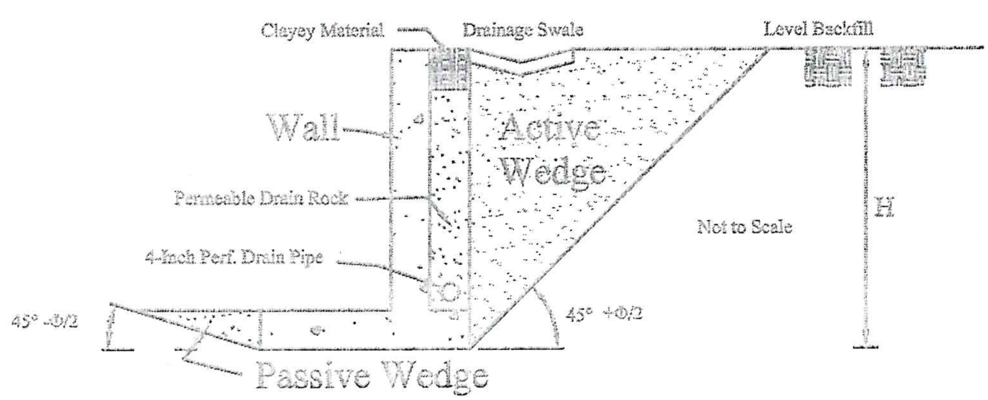

Figure 6: Retaining Wall Active and Passive Wedges

5. Retaining wall foundations should be founded a minimum of 12 inches below lowest adjacent grade in rock as observed and approved by a representative of GeoSolutions, Inc. A coefficient of friction of 0.60 may be used between engineered fill and concrete

footings. Project designers may use a maximum toe pressure of **3,400 psf** for the design of retaining wall footings founded in engineered fill.

6. For earthquake conditions, retaining walls greater than 6 feet in height should be designed to resist an additional seismic lateral soil pressure of **35 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 4.0 (**PGA_M = 0.590g**). 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 6: 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.

15.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations contained in this report are based on a limited number of borings 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 2016 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 2016 CBC (CBSC, 2016) requires the following inspections by the Soils Engineer for controlled fill thicknesses greater than 12 inches as shown in Table 7: Required Verification and Inspections of Soils:

Table 7: 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

16.0 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

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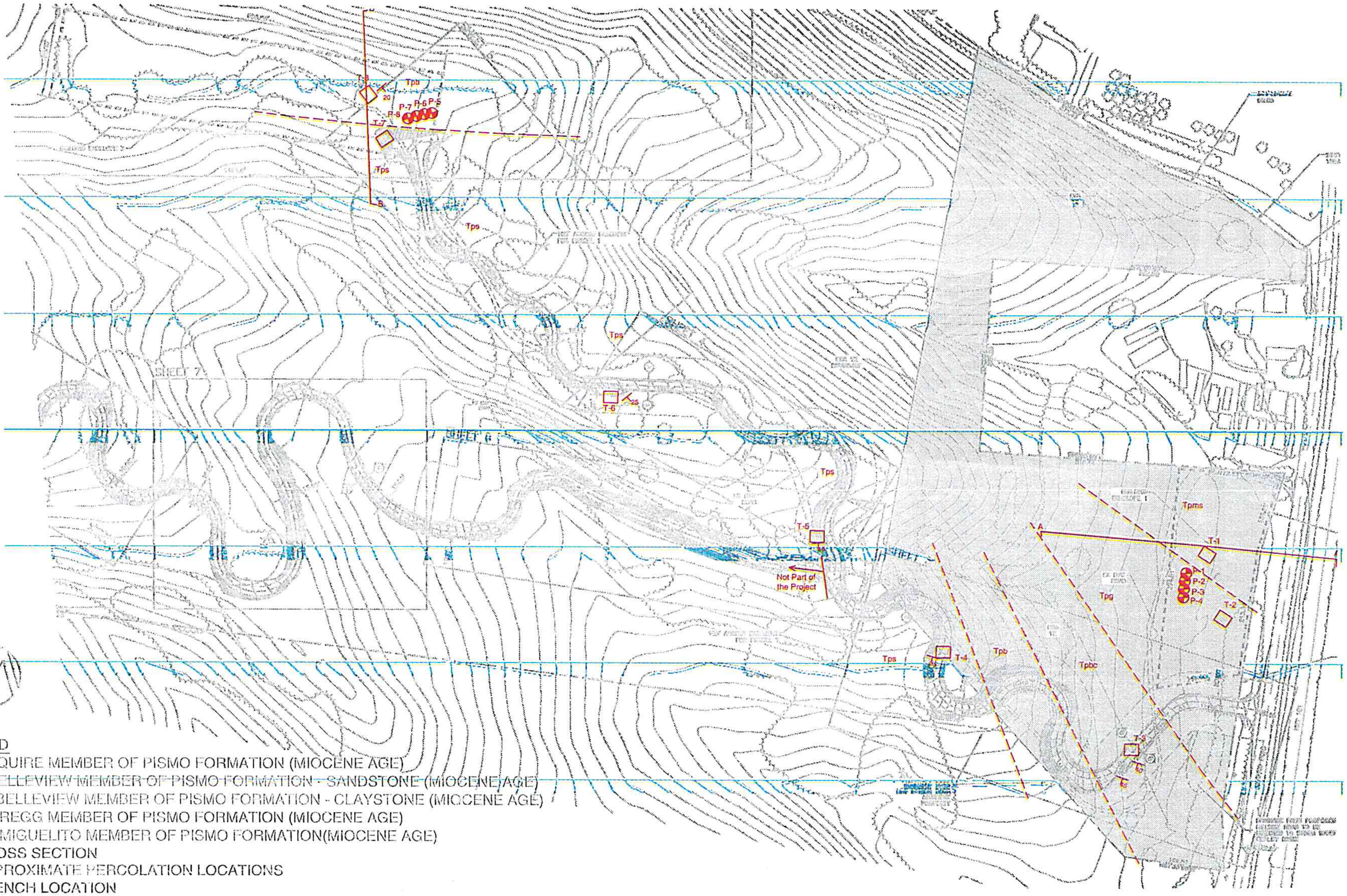
PLATES

Plate 1A, 1B - Site Engineering Geologic Map and Site Cross Sections

Plate 2A, 2B – Regional Geologic Map, Wiegers 2011 and Geologic Explanations

Plate 3 – Regional Fault Map, Jennings, 2010

Plate 4 – Aerial Photograph



1"=160'

LEGEND

Tps - SQUIRE MEMBER OF PISMO FORMATION (MIOCENE AGE)

Tpb - BELLEVIEW MEMBER OF PISMO FORMATION - SANDSTONE (MIOCENE AGE)

Tpbc - BELLEVIEW MEMBER OF PISMO FORMATION - CLAYSTONE (MIOCENE AGE)

Tpg - GREGG MEMBER OF PISMO FORMATION (MIOCENE AGE)

Tpms - MIGUELITO MEMBER OF PISMO FORMATION(MIOCENE AGE)

CROSS SECTION

APPROXIMATE PERCOLATION LOCATIONS

TRENCH LOCATION

BEDDING ATTITUDE

CONTACT(DASHED WHERE APPROXIMATE)

JEFFREY PFOST, CEG 2493

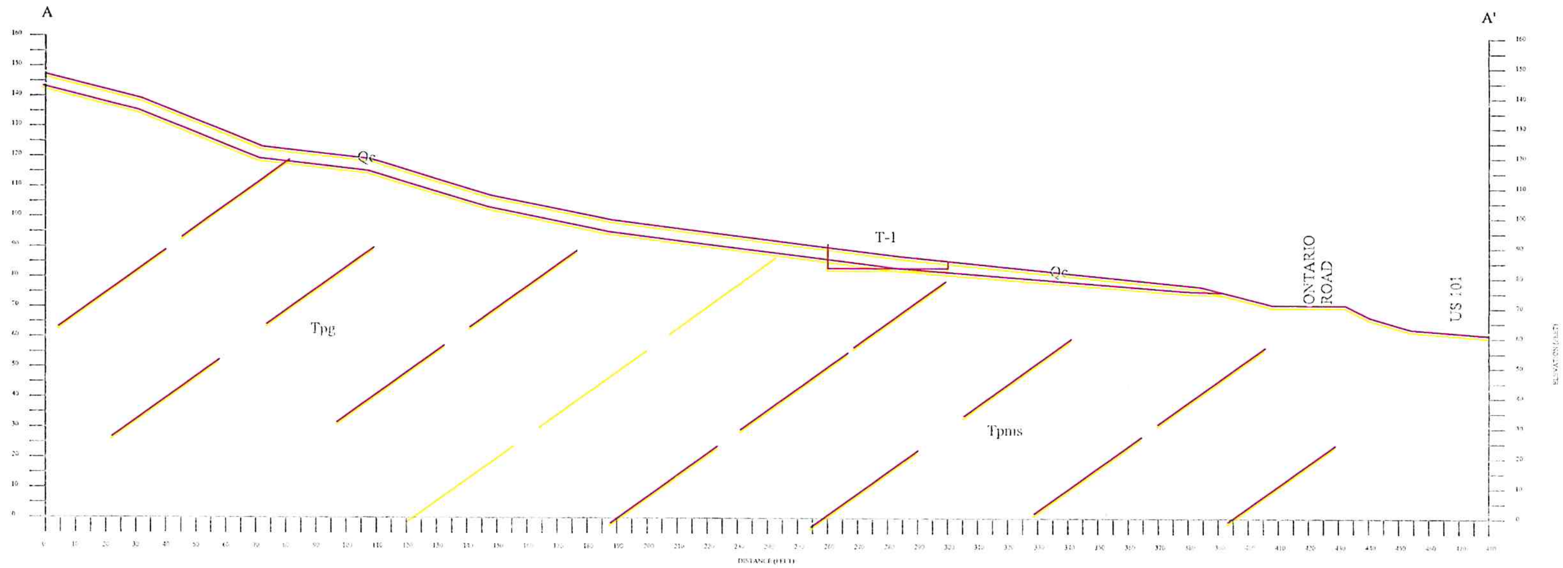
GeoSolutions, Inc.
1021 Tama Lane, Suite 105
Santa Maria, CA 93455
(805)614-5333

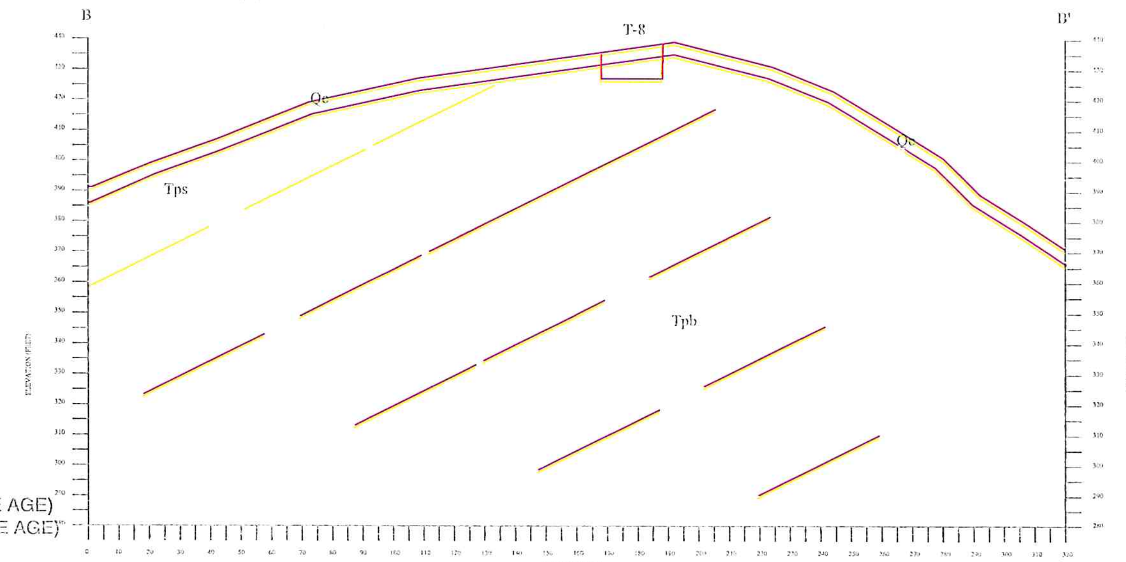
SITE ENGINEERING GEOLOGIC MAP

6226 ONTARIO ROAD, APN:076-114-052
SAN LUIS OBISPO AREA,SAN LUIS OBISPO COUNTY, CALIFORNIA

PLATE
1A

PROJECT
SL010844-1





1"= 40'

LEGEND

Tps - SQUIRE MEMBER OF PISMO FORMATION (MIOCENE AGE)

Tpb - BELLEVIEW MEMBER OF PISMO FORMATION - SANDSTONE (MIOCENE AGE)

Tpbc - BELLEVIEW MEMBER OF PISMO FORMATION - CLAYSTONE (MIOCENE AGE)

Tpg - GREGG MEMBER OF PISMO FORMATION (MIOCENE AGE)

Tpms - MIGUELITO MEMBER OF PISMO FORMATION(MIOCENE AGE)

CONTACT (DASHED WHERE APPROXIMATE)

BEDDING (APPARENT DIP)

JEFFREY PFOST, CEG 2493

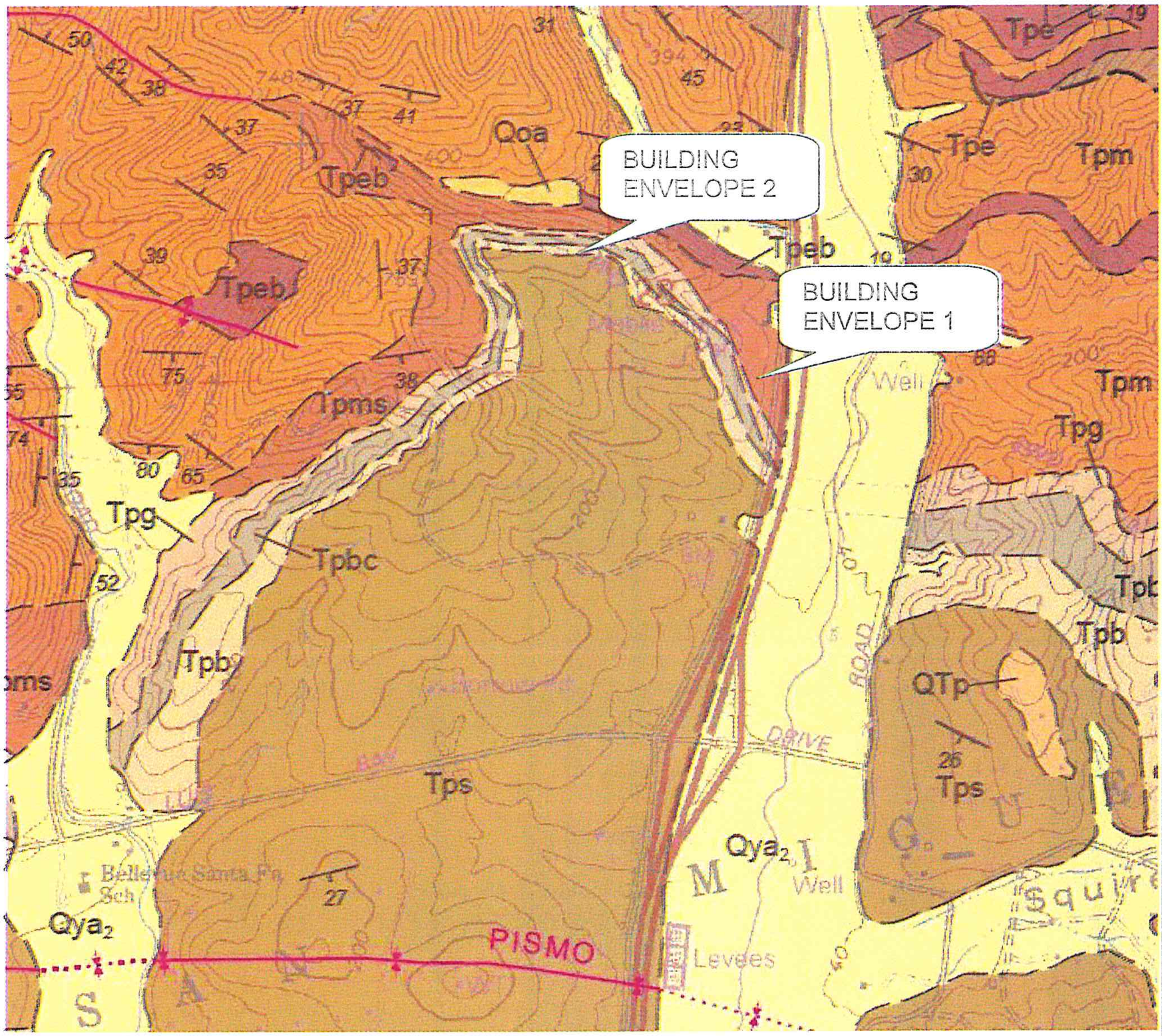
GeoSolutions, Inc.
1021 Tama Lane, Suite 105
Santa Maria, CA 93455
(805) 614-6333

SITE CROSS SECTIONS

6226 ONTARIO ROAD, APN: 0076-114-032
SAN LUIS OBISPO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA

PLATE
1B

PROJECT
SL10844-1



GeoSolutions, Inc.
1021 Tama Lane, Suite 105
Santa Maria, California 93455
(805) 614-6333

REGIONAL GEOLOGY MAP
(WIEGERS, 2011)
6226 ONTARIO ROAD, APN: 076-114-052
SAN LUIS OBISPO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA

PLATE
2A

PROJECT NO:
SL10844-1

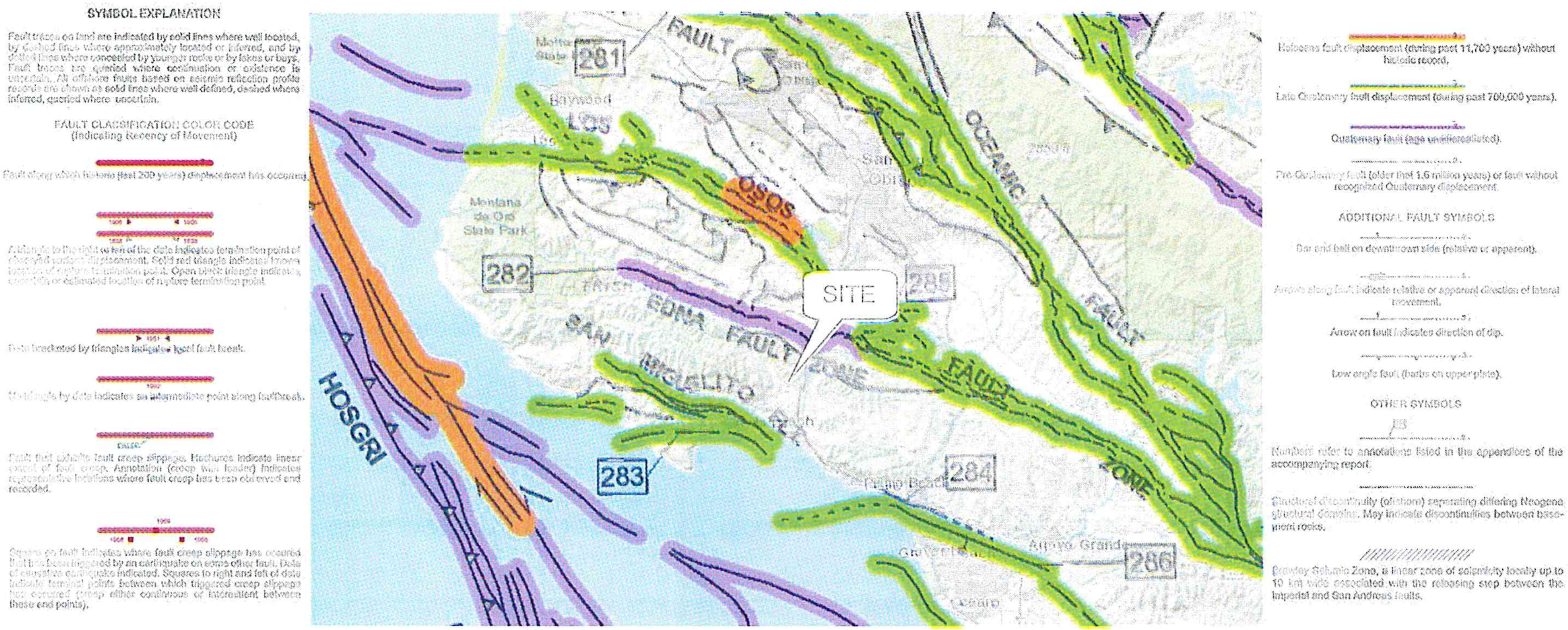
GeoSolutions, Inc.
1021 Tama Lane, Suite 105
Santa Maria, California 93455
(805) 614-6333

REGIONAL GEOLOGY MAP

(WIEGERS, 2011)
6226 ONTARIO ROAD, APN: 076-114-052
SAN LUIS OBISPO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA

PLATE
2B

PROJECT NO:
SL10844-1



GeoSolutions, Inc.
1021 Tama Lane
Santa Maria, California 93455
(805) 614-6333

REGIONAL FAULT MAP

(JENNINGS, 2010)
6226 ONTARIO ROAD, APN: 076-114-052
SAN LUIS OBISPO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA

PLATE
3

PROJECT NO:
SL10844-1

GeoSolutions, Inc. 1021 Tama Lane Santa Maria, California 93455 (805) 614-6333	**AERIAL PHOTOGRAPH** (GOOGLE EARTH) 6226 ONTARIO ROAD, APN: 076-114-052 SAN LUIS OBISPO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA	**PLATE** 4 **PROJECT NO:** SL10844-1

APPENDIX A

Field Investigation

Soil Classification Chart

Trench Logs

FIELD INVESTIGATION

The field investigation was conducted July 23, 2018 using a backhoe equipment. The surface and sub-surface conditions were studied by advancing eight 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 backhoe with a twenty four-inch diameter bucket excavated eight exploratory trenches near the approximate locations indicated on Plate 1. 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.

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 TONS/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

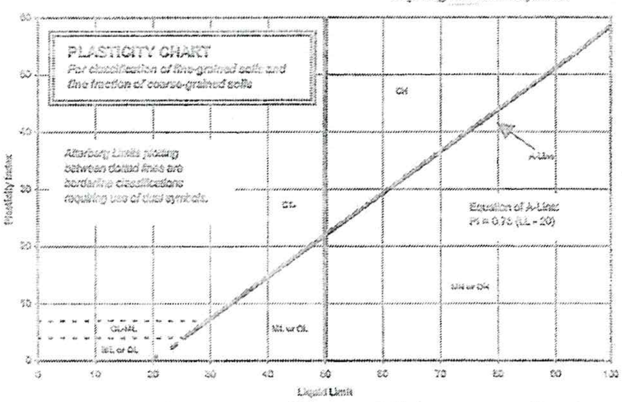
SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT +
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	Over 50

\+ 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



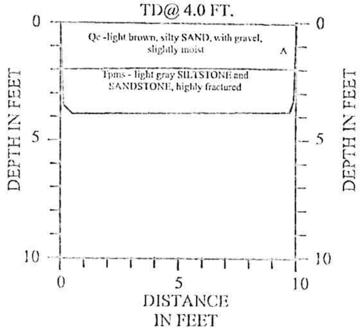
Drilling Notes:

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

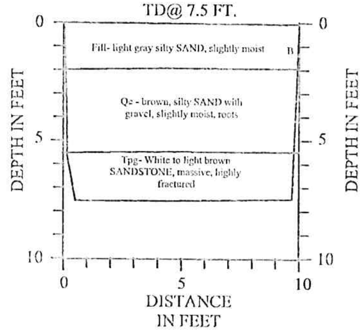
Types of Samples:
X – Sample
SPT - Standard Penetration
CA - California Modified
N - Nuclear Gauge
PO – Pocket Penetrometer (tons/sq.ft.)

GEO SOLUTIONS

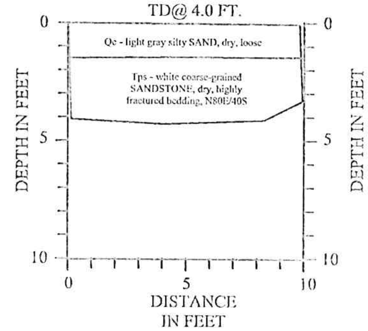
T-1
TD@ 4.0 FT.



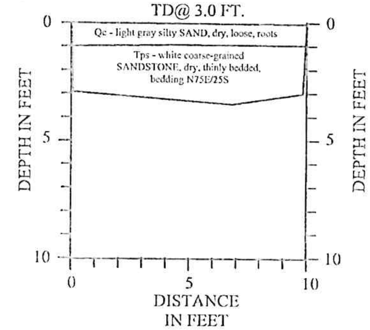
T-2
TD@ 7.5 FT.



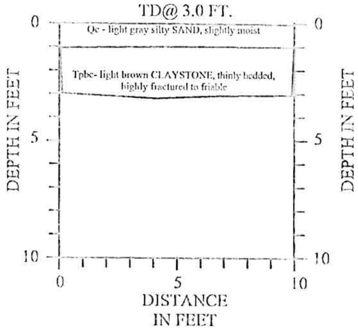
T-5
TD@ 4.0 FT.



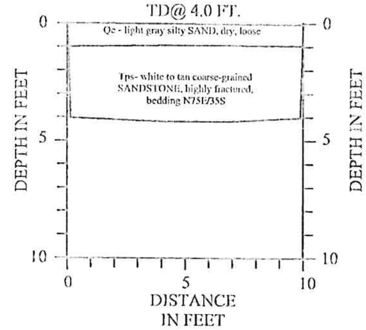
T-6
TD@ 3.0 FT.



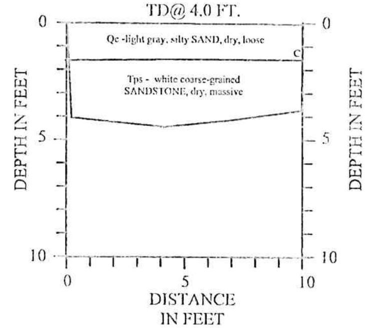
T-3
TD@ 3.0 FT.



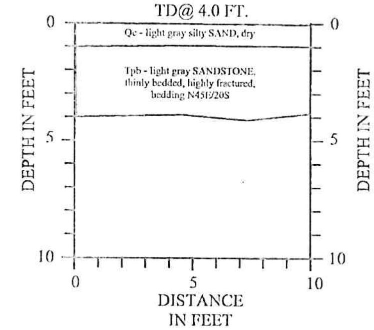
T-4
TD@ 4.0 FT.



T-7
TD@ 4.0 FT.



T-8
TD@ 4.0 FT.



Jeffrey Pfost CEG 2493

GeoSolutions, Inc.
1021 Tama Lane, Suite 105
Santa Maria, CA 93455
(805) 614-6333

TRENCH LOGS

6226 ONTARIO ROAD, APN: 076-114-052
SAN LUIS OBISPO AREA, SAN LUIS OBISPO COUNTY, CALIFORNIA

LOGS
1

PROJECT
SL10844-1

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.

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.

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

Project:	6226 Ontario Road			Date Tested:	August 3, 2018
Client:				Project #:	SL10844-1
Sample:	B	Depth:	1.0 Foot	Lab #:	17497
Location:	T-2			Sample Date:	July 23, 2018
				Sampled By:	JAP

Soil Classification
ASTM D2487, D2488

Result:	Very Dark Grayish Brown Clayey SAND with Gravel
Specification:	SC

Sieve Analysis
ASTM D422

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"	100	
3/4"	95	
No. 4	70	
No. 8	61	
No. 16	55	
No. 30	51	
No. 50	47	
No. 100	36	
No. 200	25.6	

Sand Equivalent Cal 217

1		SE
2		
3		
4		

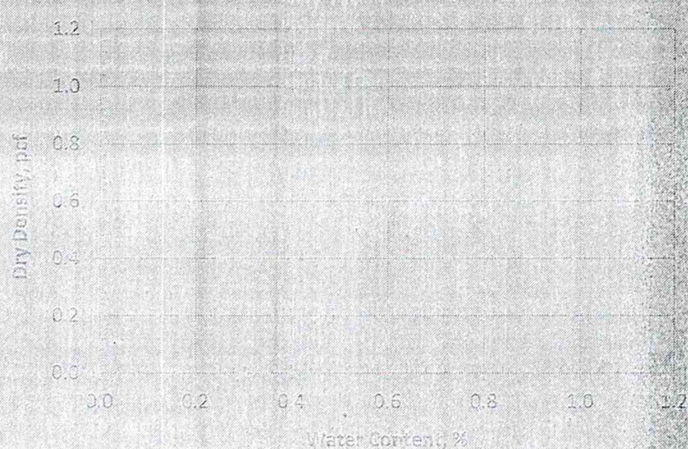
Plasticity Index
ASTM D4318

Liquid Limit:	30
Plastic Limit:	18
Plasticity Index:	12

Expansion Index
ASTM D4829

Expansion Index:	2
Expansion Potential:	Very Low
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:	6226 Ontario Road			Date Tested:	August 3, 2018
Client:				Project #:	SL10844-1
Sample:	C	Depth:	1.0 Foot	Lab #:	17497
Location:	T-7			Sample Date:	July 23, 2018
				Sampled By:	JAP

Soil Classification
ASTM D2487, D2488

Result:	Very Dark Grayish Brown Clayey SAND
Specification:	SC

Sieve Analysis
ASTM D422

Sieve Size	Percent Passing	Project Specifications
3"		
2"		
1 1/2"		
1"	100	
3/4"	95	
No. 4	74	
No. 8	68	
No. 16	65	
No. 30	62	
No. 50	59	
No. 100	36	
No. 200	24.7	

Sand Equivalent Cal 217

	SE
1	
2	
3	
4	

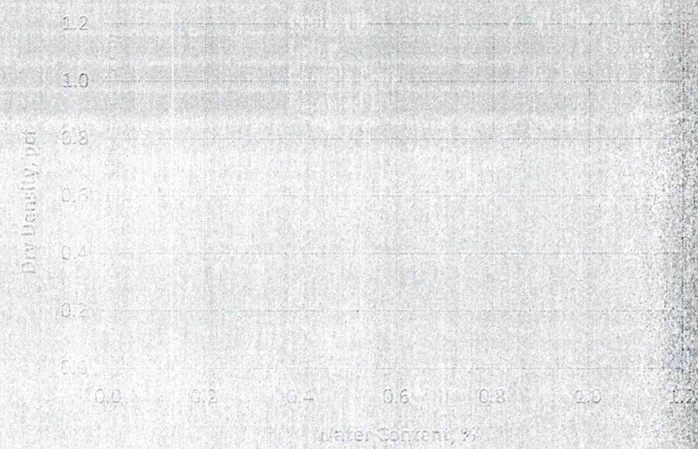
Plasticity Index
ASTM D4318

Liquid Limit:	31
Plastic Limit:	23
Plasticity Index:	8

Expansion Index
ASTM D4829

Expansion Index:	10
Expansion Potential:	Very Low
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

APPENDIX C

Seismic Hazard Analysis

USGS Design Map Summary Report

USGS Design Map Detailed Report

SEISMIC HAZARD ANALYSIS

According to section 1613 of the 2016 CBC (CBSC, 2016), 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 *ASCE 7 2010 Minimum Design Loads for Buildings and Other Structures*, hereafter referred to as ASCE7-10 (ASCE, 2013). Estimating the design ground motions at the Site depends on many factors including the distance from the Site to known active faults; the expected magnitude and rate of recurrence of seismic events produced on such faults; the source-to-site ground motion attenuation characteristics; and the Site soil profile characteristics. As per section 1613.3.2 of the 2016 CBC, the Site soil profile classification is determined by the average soil properties in the upper 100 feet of the Site profile and can be determined based on the criteria provided in Table 20.3-1 of ASCE7-10.

ASCE7-10 provides recommendations for estimating site-specific ground motion parameters for seismic design considering a Risk-targeted Maximum Considered Earthquake (MCE_R) in order to determine *design spectral response accelerations* and a Maximum Considered Earthquake Geometric Mean (MCE_G) in order to determine probabilistic geometric mean *peak ground accelerations*.

Spectral accelerations from the MCE_R are based on a 5% damped acceleration response spectrum and a 1% exceedance in 50 years (4975-year return period). *Maximum* short period (S_s) and 1-second period (S_1) spectral accelerations are interpolated from the MCE_R-based ground motion parameter maps for bedrock, provided in ASCE7-10. These spectral accelerations are then multiplied by site-specific coefficients (F_a, F_v), based on the Site soil profile classification and the maximum spectral accelerations determined for bedrock, to yield the *maximum* short period (S_{MS}) and 1-second period (S_{M1}) spectral response accelerations at the Site. According to section 11.2 of ASCE7-10 and section 1613 of the 2016 CBC, buildings and structures should be specifically proportioned to resist *design* earthquake ground motions. Section 1613.3.4 of the 2016 CBC indicates the site-specific *design* spectral response accelerations for short (S_{DS}) and 1-second (S_{D1}) periods can be taken as two-thirds of *maximum* ($S_{DS} = 2/3{*}S_{MS}$ and $S_{D1} = 2/3{*}S_{M1}$).

Per ASCE7-10, Section 21.5, the probabilistic maximum mean peak ground acceleration (PGA) corresponding to the MCE_G can be computed assuming a 2% probability of exceedance in 50 years (2475-year return period) and is initially determined from mapped ground accelerations for bedrock conditions. The site-specific peak ground acceleration (PGA_M) is then determined by multiplying the PGA by the site-specific coefficient F_h (where F_h is a function of Site Class and PGA).

Spectral response accelerations, peak ground accelerations, and site coefficients provided in this report were obtained using the web-based U.S. Seismic Design Map tool available from the United States Geological Survey website (USGS, 2013). This 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 in conjunction with user-inputted Site latitude and longitude coordinates to calculate seismic design parameters and response spectra (both for period and displacement) for soil profile Site Classifications A through E. Output from the web-based program are included in this Appendix.

USGS Design Maps Summary Report

User–Specified Input

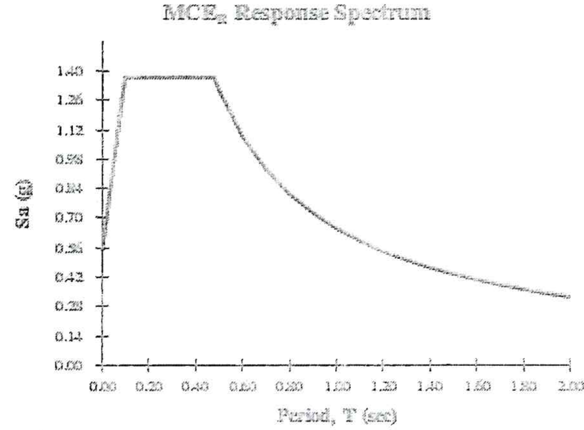
Report Title	6226 Ontario Road Wed August 29, 2018 16:42:59 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	35.204°N, 120.698°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

USGS–Provided Output

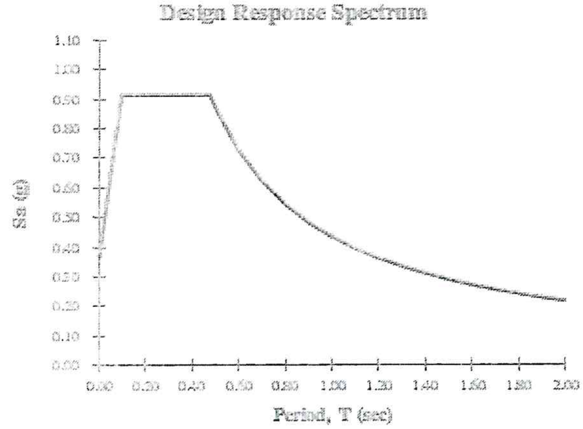
S_S = 1.370 g	S_{MS} = 1.370 g	S_{DS} = 0.913 g
S_1 = 0.501 g	S_{M1} = 0.651 g	S_{D1} = 0.434 g

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.

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (35.204°N, 120.698°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 = 1.370 g
From **Figure 22-2** [2]	S_1 = 0.501 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 $w \ge 40\%$, 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				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
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 = 1.370 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				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
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 = 0.501 g, F_v = 1.300

Equation (11.4-1): $S_{MS} = F_aS_S = 1.000 \times 1.370 = 1.370$ g

Equation (11.4-2): $S_{M1} = F_vS_1 = 1.300 \times 0.501 = 0.651$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

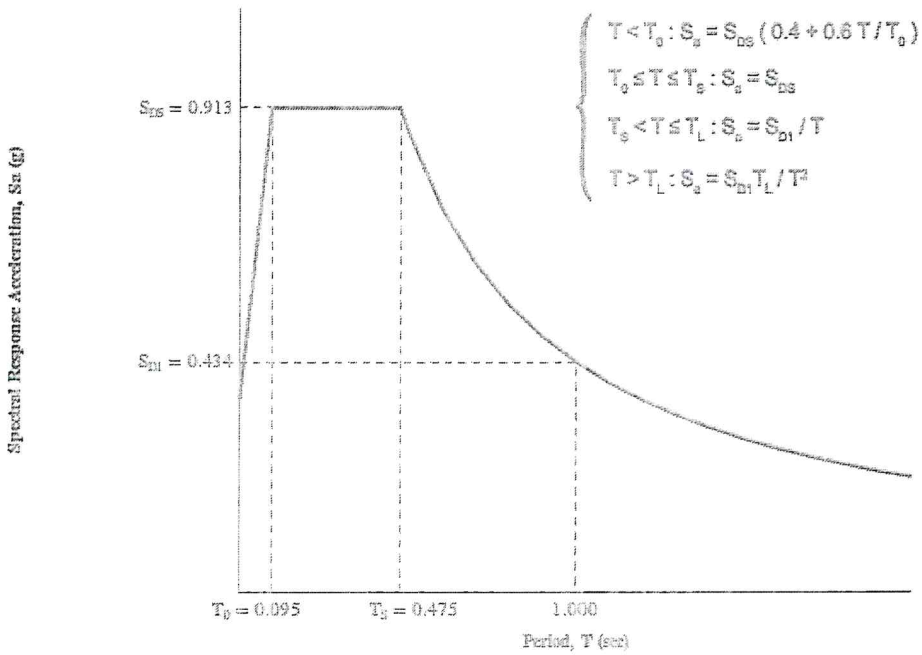
Equation (11.4-3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.370 = 0.913$ g

Equation (11.4-4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.651 = 0.434$ g

Section 11.4.5 — Design Response Spectrum

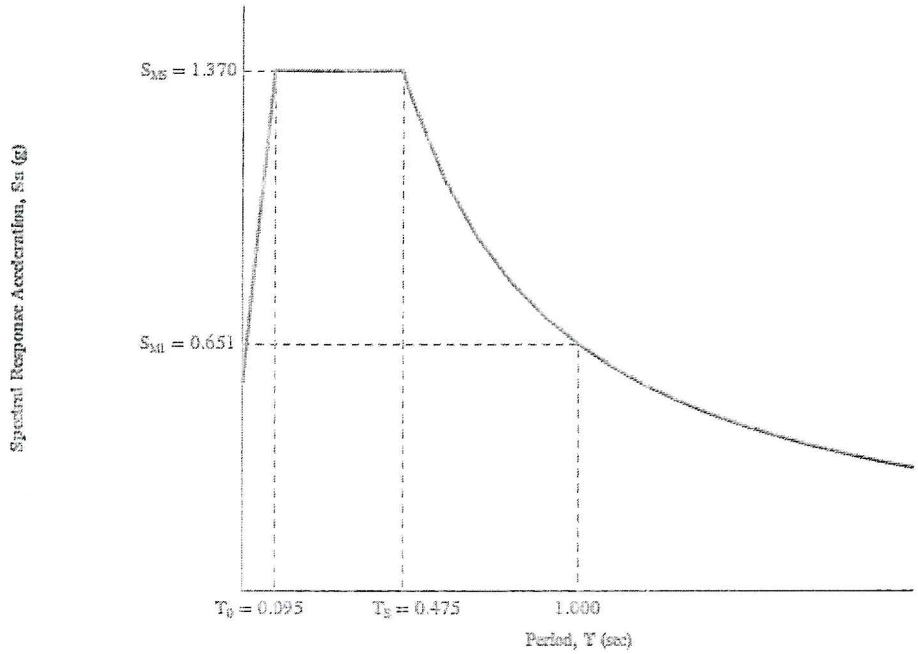
From Figure 22-12[3] $T_L = 8$ seconds

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 = 0.590
Equation (11.8–1):	$PGA_M = F_{PGA}PGA = 1.000 \times 0.590 = 0.59$ 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 = 0.590 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} = 0.882
From Figure 22-18[6]	C_{R1} = 0.922

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} = 0.913 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} = 0.434 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*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://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 2016 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 D1557-12$_{e1}$.

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 D1557-12_{e1}.

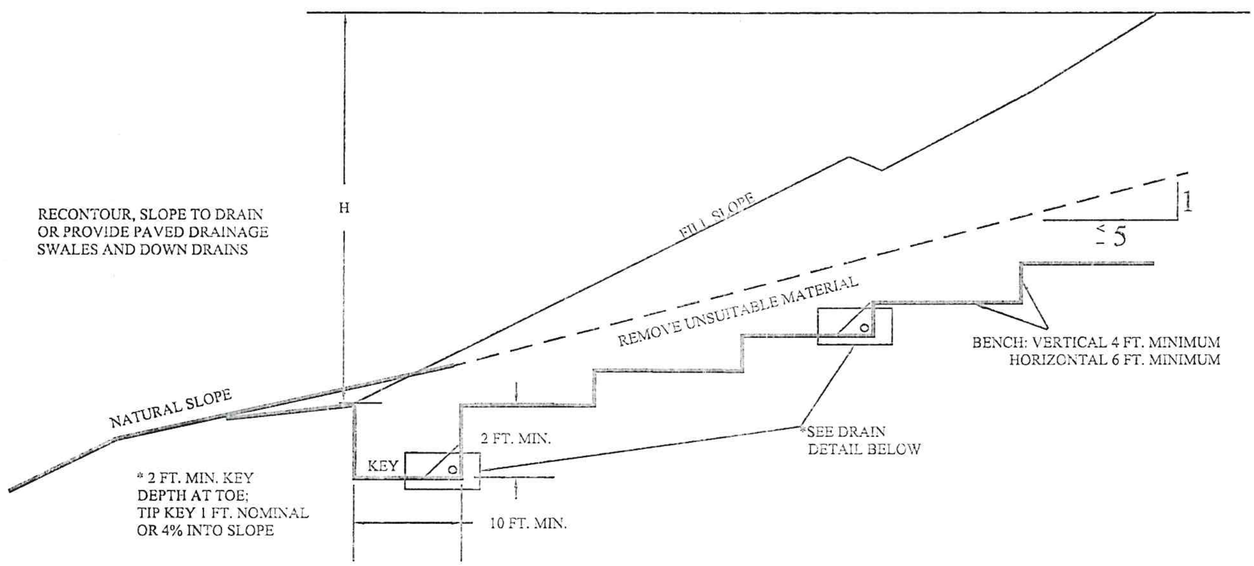
3. On-site inorganic soils, or approved import, may be used as utility trench backfill. Proper compaction of trench backfill will be necessary under and adjacent to structural fill, building foundations, concrete slabs, and vehicle pavements. In these areas, backfill should be conditioned with water (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 D1557-12_{e1}. 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 2016 CBC.

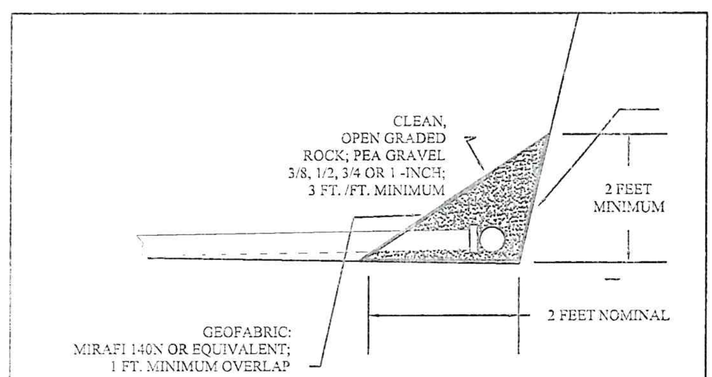
FILL OVER SLOPE



NOTES:

*BACKDRAIN AS RECOMMENDED BY GEOTECHNICAL
PER DETAIL.

DRAIN DETAIL



NTS

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

KEY AND BENCH WITH BACKDRAIN

DETAIL
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