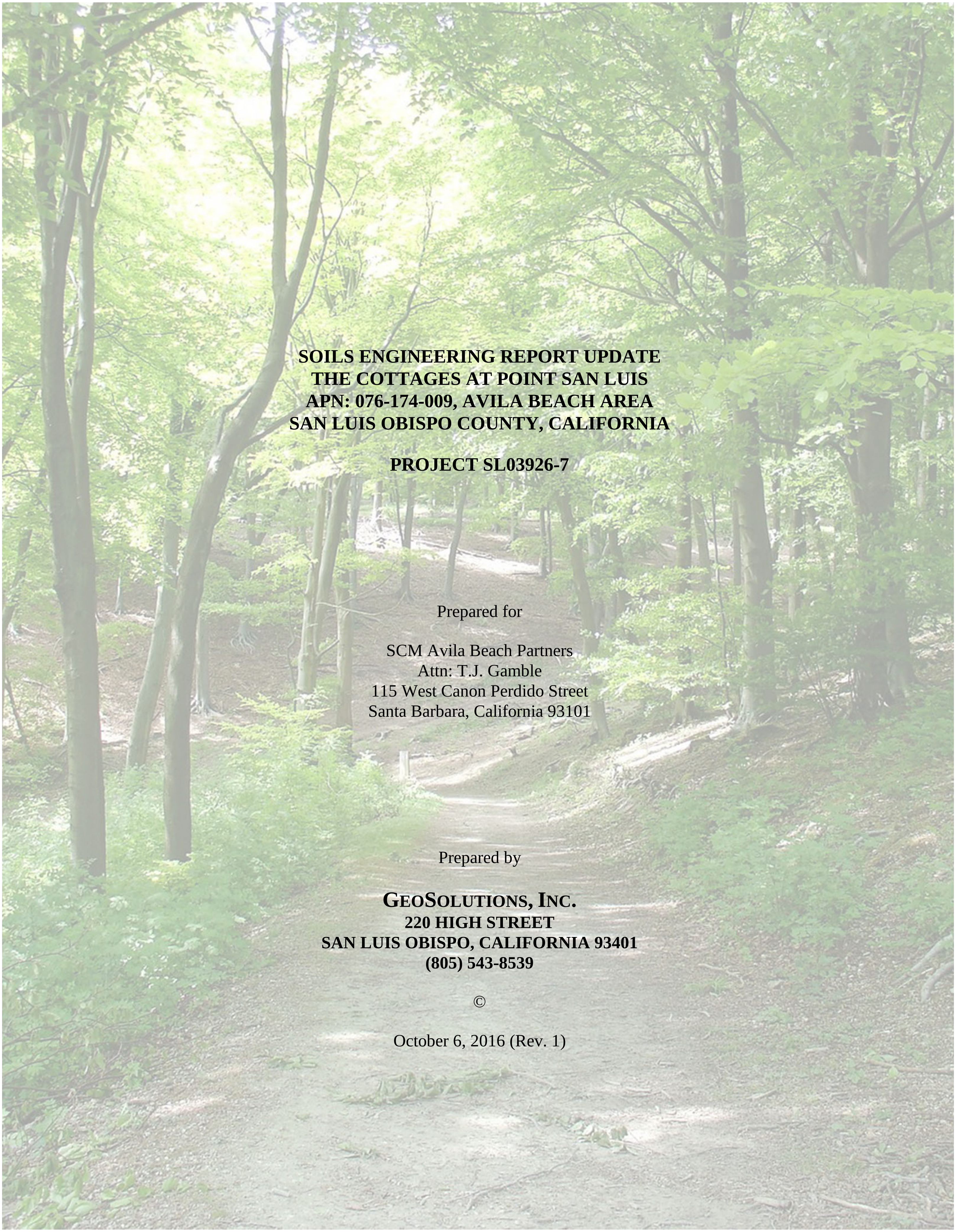

# SOILS ENGINEERING REPORT UPDATE
# THE COTTAGES AT POINT SAN LUIS
# APN: 076-174-009, AVILA BEACH AREA
# SAN LUIS OBISPO COUNTY, CALIFORNIA

## PROJECT SL03926-7

Prepared for

SCM Avila Beach Partners
Attn: T.J. Gamble
115 West Canon Perdido Street
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Prepared by

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October 6, 2016 (Rev. 1)

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October 6, 2016 (Rev. 1)
Project No. SL03926-7

**SCM Avila Beach Partners**
**Attn: T.J. Gamble**
115 West Canon Perdido Street
Santa Barbara, California 93101

Subject: **Soils Engineering Report Update**
The Cottages at Point San Luis, APN: 076-174-009
Avila Beach area, San Luis Obispo County, California

Dear Mr. Gamble:

This Soils Engineering Report Update has been prepared for the proposed Cottages Development to be located at APN: 076-174-009, Avila Beach Area, San Luis Obispo County, California. Geotechnically, the site is suitable for the proposed development provided the recommendations in this report for site preparation, earthwork, foundations, slabs, retaining walls, and pavement sections are incorporated into the design.

The project is comprised of a main lodge, a series of cottages, and a parking lot. In general, the main lodge and cottages are located in areas where grading activities will be acceptable, while the main parking lot is proposed in an area that mass grading will be limited to adding fill.

It is anticipated that the cottages will be constructed with conventional foundations placed on underlying sandstone.

It is anticipated that the main lodge will be placed on a system of drilled cast-in-place caissons utilizing a pile supported slab.

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, PE
Principal

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

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**SAN LUIS OBISPO COUNTY, CALIFORNI**

**PROJECT SL03926-7**

# 1.0 INTRODUCTION

This report presents the results of the geotechnical investigation for the proposed development known as *The Cottages at Point San Luis* to be located off of Ana Bay Drive, APN: 076-174-009, in the Avila Beach 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). This report is serves as an update to the referenced Preliminary *Soils Engineering Report*, Seaside Garden Cottages, APN: 076-174-009, Avila Beach area, San Luis Obispo County, California dated May 29, 2008, Project SL03926-4, by GeoSolutions, Inc. (GeoSolutions, Inc., 2008). It is intended to address the applicable changes to the referenced report (GeoSolutions, Inc., 2008) required by the adoption of the 2013 California Building Code (CBSC, 2013) and incorporate the current scope of work.

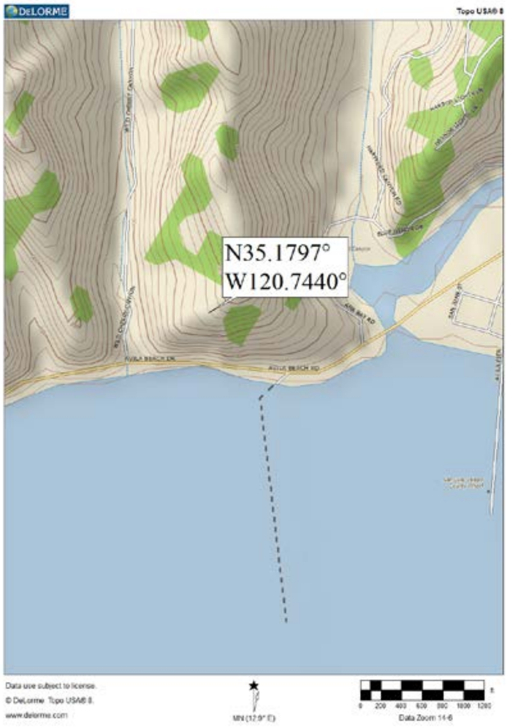

**Figure 1: Site Location Map**

## 1.1 Site Description

The property is located at 35.1797 degrees north latitude and 120.7440 degrees west longitude at a general elevation of 223 feet above mean sea level. The property is roughly rectangular in shape and 23 acres in size. Access to the property is provided by a private unpaved access road off of Ana Bay Drive, which is located along the east side of the property. The private unpaved access road leads uphill before branching off toward an existing single-family residence to the east and also north toward Wild Cherry Canyon.

The topography at the Site slopes downward to the south and west. At the existing unpaved access road the slope decreases to near level for approximately 500 feet before sloping steeply downward toward Avila Beach Drive. Topography in the area of the main lodge falls steeply to Avila Beach Drive. Surface drainage follows the topography southwest toward Wild Cherry Canyon, which drains into San Luis Bay. There is an existing single family residence currently located on the property, but not in the proposed development area. The proposed building area is currently vacant and covered with annual grasses and trees.

## 1.2 Project Description

A slope failure occurred in January of 2011. The slope was repaired by the County of San Luis Obispo. The failure occurred onto San Luis Bay Drive. See the referenced Engineering Geology Report (GeoSolutions, Inc., 2016) for specific details.

It is our understanding that the project will consist of fifty (50) bungalow style cottages and a supporting hospitality building (main lodge) including; a restaurant, spa, banquet rooms, yoga studio, laundry facilities, pool with bar and gift shop. At the time of the preparation of this report, the proposed cottages and hospitality building are to be constructed using light wood framing. Retaining walls are expected to be constructed as part of this project. The project property will hereafter be referred to as the "Site." See Figure 2: Site Plan for the general layout of the Site.

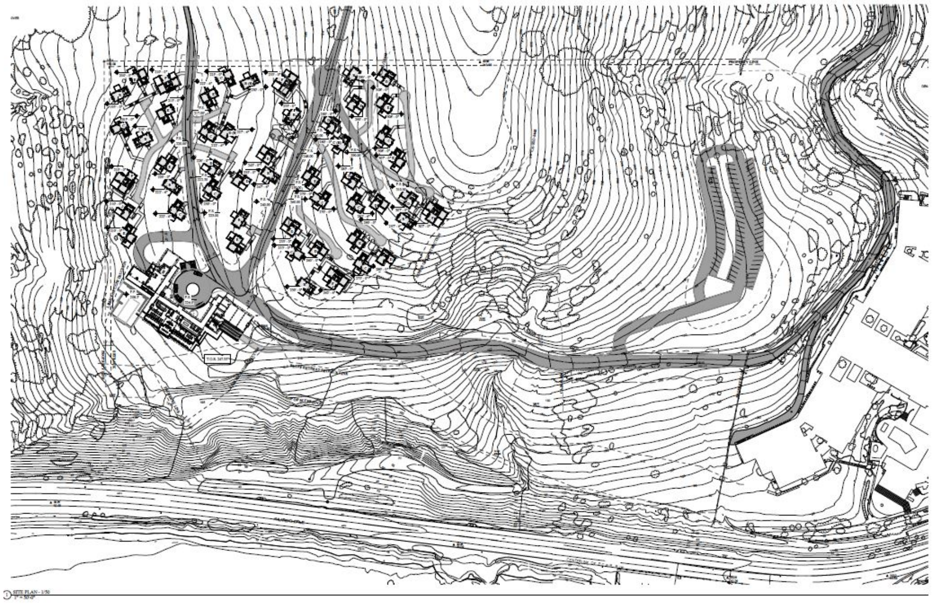

**Figure 2: Site Plan**

It is anticipated that the proposed cottages will utilize slab-on-grade and lower floor systems while the main lodge will utilize a pile supported slab. 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 3.0 kips per linear foot and 100 kips, respectively for the main lodge approximately 2.0 kips per linear foot and 15 kips, respectively for the cottages.

## 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, available on-line or in-house aerial photographs, and review of previous *Soils Engineering Report Update*, Seaside Garden Cottages, APN: 076-174-009, Avila Beach area, San Luis Obispo County, California dated May 29, 2008, Project SL03926-4, by GeoSolutions, Inc. (GeoSolutions, Inc., 2008) and Geologic Coastal Bluff Evaluation Update, Avila Beach Cottages, APN: 076-174-009, Avila Beach area, San Luis Obispo County, California dated January 8, 2016, Project No. SL03926-7 by GeoSolutions, Inc. (GeoSolutions, Inc., 2016).

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

3. A review of the laboratory testing performed on representative soil samples that were collected during our field study.

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

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

## 3.0 FIELD AND LABORATORY INVESTIGATION

Two field investigations were conducted. The first investigation, which included the advancement of three exploratory borings using a Mobile B-61 drill rig to a maximum depth of 15 feet below ground surface (bgs), was conducted during the preparation of the referenced soils engineering report (GeoSolutions, Inc., 2004). The referenced report (GeoSolutions, Inc., 2004) contains a description of the sub-surface soils and of the boring operations conducted during the associated field investigation. The second field investigation, which included the advancement of 6 additional exploratory borings to a maximum depth of 100 feet below ground surface (bgs), was conducted on February 20, 2008 using a track-mounted CME 55 drill rig. These additional borings, along with the previous borings, may be found at the approximate locations indicated on Figure 3: Google Earth Image and the Boring Logs are attached in **Appendix A.**

Figure 3: Google Earth Image

Sampling methods included the Modified California sampler (CA) with liners and rock cores were obtained using conventional core sampling equipment. The Mobile B-61 and CME 55 drill rigs were equipped with automatic and safety hammers, with efficiencies of approximately 60 and 80-percent, respectively, and were used to obtain test blow counts in the form of N-values.

Data gathered during the field investigation suggest that the soil materials at the Site consist of colluvial soil overlying competent formational material (rock). The surface material at the Site generally consisted of silty SANDs (SM) and sandy CLAYs (CL) encountered in a slightly moist and dense or firm condition to approximately 1.0 to 7.0 feet bgs. The sub-surface material consisted of pale green SANDSTONE of the Pismo Formation (Tpps) encountered in a dry and very dense condition with varying degrees of weathering to approximately 25.0 to 85.0 feet bgs. The SANDSTONE was underlain by highly fractured Franciscan Complex BASALT (Jfv), dark gray CLAYSTONE of the Pismo Formation (Tpps), and gray SILTSTONE of the Pismo Formation (Tpps) to depths of 50.0 to 100.0 feet bgs. Please refer to the referenced engineering geology report for a complete description.

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 SANDSTONE and the majority of all underlying material at the Site was interpreted as Squire Sandstone (Tsw) and will hereafter be referred to as competent formational material. See Figure 4: Regional Geologic Map. Groundwater was not encountered in any of the borings.

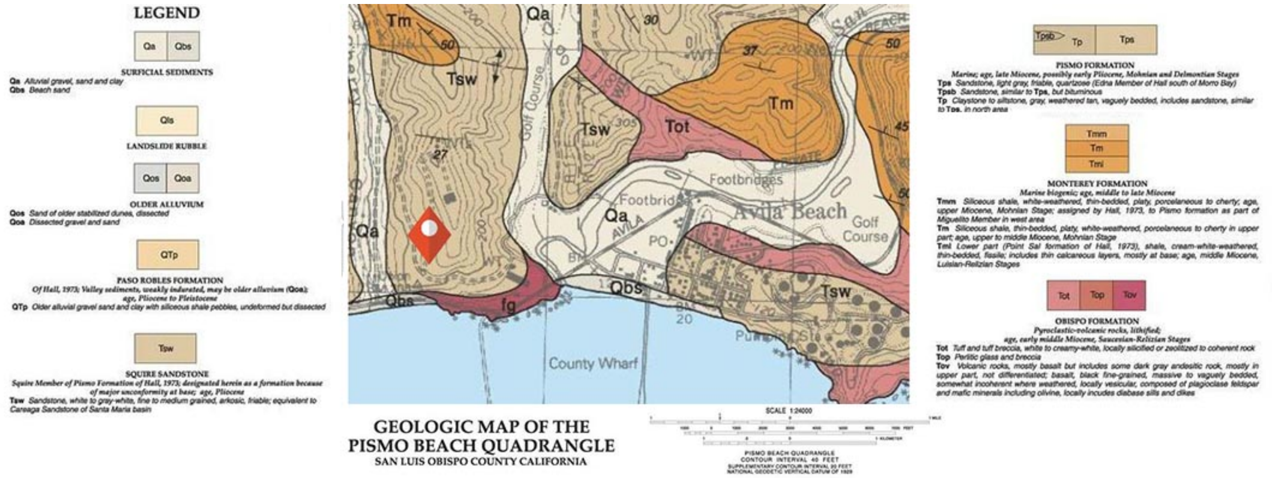

Figure 4: Regional Geologic Map

During the boring operations of the referenced report (GeoSolutions, Inc., 2004), 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. The Boring Logs from the referenced report (GeoSolutions, Inc., 2004) are attached in **Appendix A**.

As part of the preparation of the referenced report (GeoSolutions, Inc., 2004), laboratory tests were performed on soil samples obtained from the Site during boring operations. The results of these tests are listed below in Table 1: Engineering Properties - (GeoSolutions, Inc., 2008). Laboratory data reports and detailed explanations of the laboratory tests performed during this investigation are provided in **Appendix B**.

**Table 1: Engineering Properties - (GeoSolutions, Inc., 2008)**

| Sample Name | Sample Description | USGS | Expansion Index | Expansion Potential | Plasticity Index | Maximum Dry Density, $\gamma_d$ (pcf) | Optimum Moisture (%) | Angle of Internal Friction, $\phi$ (deg.) | Cohesion, c (psf) |
|---|---|---|---|---|---|---|---|---|---|
| **A (B-1 @ 1')** | Very Dark Grayish Brown Silty SAND with Clay | SM | 6 | Very Low | - | 117.4 | 12.3 | - | - |
| **B (B-1 @ 4')** | Very Dark Grayish Brown Sandy CLAY with Silt | CL | 23 | Low | 23 | 109.1 | 14.5 | - | - |
| **C (B-1 @ 10')** | Olive Brown Silty CLAY | CL | 41 | Low | 25 | 106.5 | 17.3 | - | - |
| **B-1 @ 0'** | Very Dark Gray Silty SAND | SM | - | - | - | - | - | 32.2 | 111 |
| **B-1 @ 3.5'** | Pale Olive Silty SAND with Clay | SM | - | - | - | - | - | 53.7 | 0 |
| **B-1 @ 8.5'** | Pale Olive Silty SAND with Clay | SM | - | - | - | - | - | 33.9 | 1201 |
| **B-1 @ 13.5'** | Pale Olive Silty SAND | SM | - | - | - | - | - | 56.1 | 0 |
| **B-2 @ 0'** | Very Dark Gray Silty SAND | SM | - | - | - | - | - | 32.7 | 303 |
| **B-2 @ 3.5'** | Black CLAY | CL | - | - | - | - | - | 5.9 | 1410 |
| **B-2 @ 8.5'** | Olive Brown Clayey SILT | ML | - | - | - | - | - | 3.9 | 1199 |
| **B-2 @ 13.5'** | Pale Olive Silty CLAY with Sand | CL | - | - | - | - | - | 32.7 | 1796 |
| **B-3 @ 0'** | Black Silty SAND with Clay | SM | - | - | - | - | - | 34.1 | 17 |
| **B-3 @ 3.5'** | Grayish Brown Sandy SILT | ML | - | - | - | - | - | 44.9 | 2 |

# 4.0 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, the Site was defined as Site Class C, Very Dense Soil & Soft Rock profile 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 $S_{D1}$=0.423 g and $S_{DS}$=0.886 g,** 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 35.1797 degrees north latitude and 120.7440 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. 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) **$PGA_M$ = 0.564 g** which is present on Sheet 5 of 6 of the USGS Design Maps Detailed Report (ASCE 7-10 Standard). See **Appendix C:** USGS Design Maps Summary and Detailed Report.

## 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. 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.

## 5.0 GENERAL SOIL-FOUNDATION DISCUSSION

The project is comprised of a hospitality building (main lodge), a series of cottages, and a parking lot. In general, the main lodge and cottages are located in areas where grading activities will be acceptable, while the main parking lot is proposed in an area that mass grading will be limited to adding fill.

It is anticipated that the cottages will be constructed with conventional foundations placed on underlying sandstone.

It is anticipated that the main lodge will be placed on a system of drilled cast-in-place caissons utilizing a pile supported slab.

Project planners have indicated that the use of helical piers may be desired for support of cottages that may be placed outside areas acceptable for grading activities. Drilled helical piers anchored into competent bedrock may be used as support for the cottages. Helical pier design requires the soil strength parameters be identified for design. The strength is typically given as the blow count (N value) of any given soil layer. Blow counts with depth are provided in the Boring Logs, Appendix B. Hard rock conditions are anticipated with N-value greater than 50 for 5 inches. It may be preferred to use 12 inch diameter drilled pier and grade beams to support the cottages. Helicals may not be practical in Sandstone conditions.

The main parking lot is located in an area that is archeological sensitive. As a result, grading will be limited to placing fill. Placing fill in slopes without creating level benches is problematic. To “anchor” any fill on the slope a crushed aggregate material such as ½ inch granite should be used.

## 6.0 CONCLUSIONS AND RECOMMENDATIONS

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

The primary geotechnical concerns at the Site are:

1. The presence of loose surface soils.

2. The proximity to a steep and tall bluff face.

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

## 6.1 Preparation of Building Pad

1. It is anticipated that a grading will be minimal with footings founded in uniform competent native material (rock).

2. For slab-on-grade construction with footings founded a minimum of 12 inches into uniform competent formational material or piers placed into bedrock, 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 existing grade and finished slab elevation, to competent material, or to two-thirds the depth of the deepest fill; whichever is greatest. The exposed surface should be scarified to a depth of 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum relative density of 95 percent (ASTM D1557-07). The over-excavated material may then be processed as engineered fill. (The over-excavated material is not suitable for use as engineered fill. Imported non-expansive material may be used as engineered fill. All material to be used as non-expansive engineered fill must be observed and approved by a representative of GeoSolutions, Inc. prior to its delivery to the Site). Figure 5: Sub-Slab Detail for under-slab drainage material and **Appendix D** for more details on fill placement.

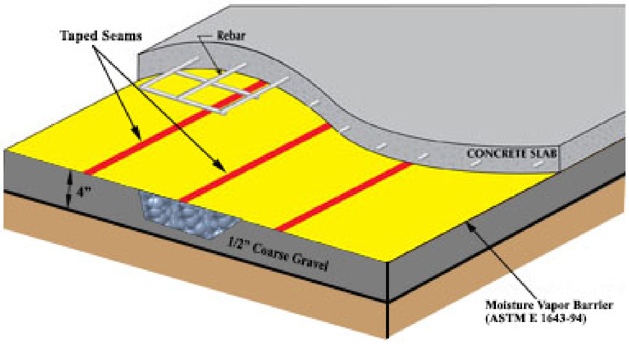

Figure 5: Sub-Slab Detail

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

## 6.2 Preparation of Pool Area

1. It is anticipated that the proposed pool will be incorporated into the pile supported slab system for the main lodge.

2. We recommend the pool rest on 4.0 inches of gravel placed over an exterior gunite or shotcrete shell. The exterior shell would act as a containment/drainage system if the pool were to leak or become damaged due to an unanticipated destructive event.

3. An outer gunite or shotcrete shell with pump system placed in a sump at the lowest elevation of the shell is suggested to protect the adjacent steep slope. Once the shell and pump system has been constructed, the interior walls of the shell should be water proofed with a flexible coating such as Aquafin 2K or equivalent, and backfilled with a thin 4 inch layer of gravel. The walls could be lined with either a slip sheet or thin foam to create a void between the walls of the shell and pool to allow any leakage to run to the bottom of the shells pump system and discharged to a containment basin. A 5 to 6 inch non-structural gunite liner would be used to form the interior pool and its appointments such as the spa.

## 6.3 Preparation of Paved Areas

1. Pavement areas should be over-excavated 12 inches below existing grade or finished sub-grade; whichever is deeper. The exposed surface should be scarified an additional depth of eight inches, moisture conditioned to near optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-07 test method). The over-excavated soil should then be moisture conditioned to produce a water-content of at least one to two percent above optimum value and then compacted to a minimum relative density of 90 percent. The top 12 inches of sub-grade soil 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. 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 twelve inches, moisture conditioned to near 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.

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

## 6.4 Pavement Design

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

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

3. A minimum of six inches of Class II Aggregate Base is recommended for all pavement sections. All pavement sections should be crowned for good drainage. During construction an R-value should be obtained and final pavement sections specified.

## 6.5 Conventional Foundations

1. Conventional continuous and spread footings with grade beams may be used for support of the proposed cottages. Isolated pad footings should be a minimum of two feet square in size and are permitted for single floor loads only.

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

**Table 2: Minimum Footing Recommendations**

| | **Perimeter Footings** | |
|---|---|---|
| **Minimum Width** | 12 inches (one story)<br>15 inches (two story) | 12 inches (into rock) |
| **Embedment Depth** | 12 inches (one story)<br>18 inches (two story) | 12 inches (into rock) |
| **Minimum Reinforcing*** | 2 #4 bars<br>(1 top / 1 bottom) | 12 inches (into rock) |
| ***** 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. (and that have been maintained in a moist condition with no desiccation cracks present.) (for expansive soils)

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

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

6. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the uniform competent formational material and the bottom of the footings. For resistance to lateral loads, a friction factor of **0.45** may be utilized for sliding resistance at the base of footings extending a minimum of 12 inches into uniform competent formational material. A passive pressure of **350-pcf** equivalent fluid weight may be used against the side of shallow footings in uniform competent formational material. If friction and passive pressures are combined to resist lateral forces acting on shallow footings, the lesser value should be reduced by 50 percent.

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

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

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

10. 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

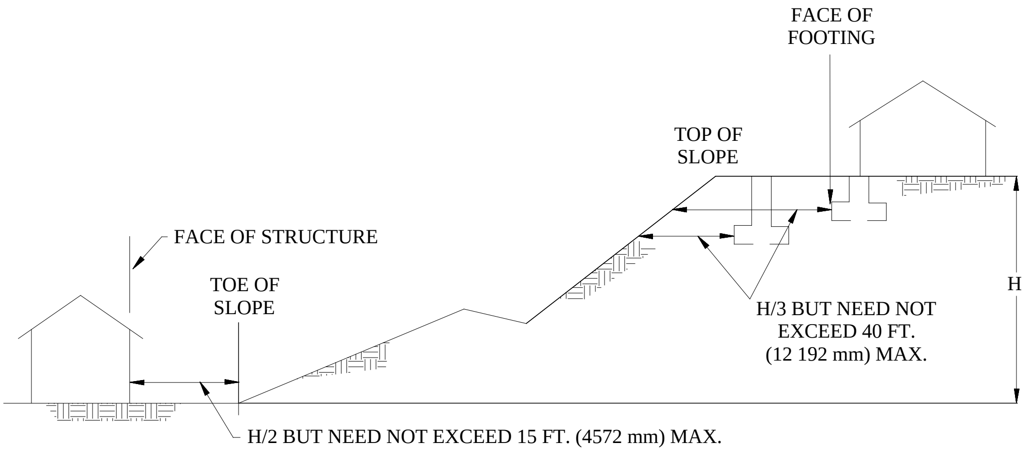

**Figure 6: Setback Dimensions – Slope Gradients Between 3-to-1 and 1-to-1**

## 6.6 Drilled Cast-in-Place Caissons- Main Lodge

1. The main lodge may be supported on a drilled pier foundation system. The main lodge should be designed with a pile supported slab, the following pier design criteria should be incorporated:

2. Pier diameter: Minimum 24 inches.

3. Pier depth: Minimum 20 feet.

4. Maximum allowable skin friction: 800 pounds per square foot (psf). This value may be increased by 1/3 when considering seismic or wind loads. Exclude the upper 10 feet of the pier shaft from pier load capacity computations. Refer to Figure 7: Caisson Detail.

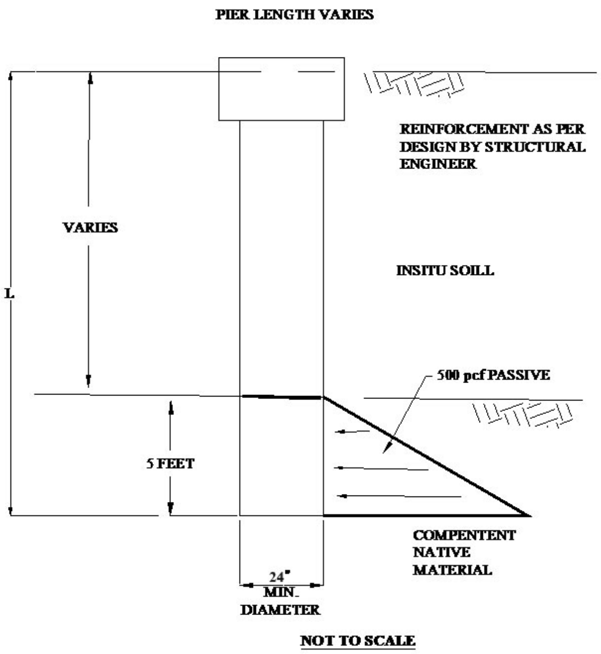

Figure 7: Caisson Detail

5. Minimum pier spacing: 3 pier diameters, center-to-center.

6. An equivalent fluid weight of 350 pounds per cubic foot acting on two times the pier diameter may be used to evaluate passive resistance. The passive pressure may be increased by 1/3 for transient loads such as wind or seismic.

7. Actual pier depths, spacing, and reinforcement should be determined by the structural engineer based on structural design considerations.

8. Caving and water intrusion is not anticipated to be a concern. If either occurs, the use of temporary casing may be required to facilitate construction. Casing and shaft diameters should be the same diameter. The casing should be progressively placed as drilling advances to design depth. If water intrusion is a problem, the concrete should be placed in the drilled holes prior to retrieving the temporary casing. The bottom of the casing should be maintained not less than 5 feet below the top of the concrete.

9. The Soils Engineer should be present at the Site during the caisson drilling and concrete placement operations to establish conformance with the design concepts, specification requirements, and to provide re-evaluation of these recommendations if site conditions vary from what is anticipated.

10. For the cottages, a 12 inch diameter pier, with a five foot embedment into underlying sandstone may be considered outside the setback distance.

## 6.7 Helical Piers

1. It is anticipated that the cottages may use helical piers to support the structures. An 8 inch diameter helical pier foundation system has been used successfully on similar projects. However, the structural engineer may consider other helical type systems.

2. Based on our experience, the helical piers will extend to approximately 10 feet below land surface on the eastern portion of the Site. The helical piers should be connected by footings and grade beams that extend a minimum of 12 inches below finished grade. Minimum reinforcing should be as directed by the project Structural Engineer.

3. 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.

4. 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 **0.45** may be utilized for sliding resistance at the base of footings.

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

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

## 6.8 Slab-On-Grade Construction - Cottages

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. 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/CSRI-81 recommendations for Steel Placement, Section 2). 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 (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, the slabs should be underlain by a minimum of six inches of clean free-draining material, such as a coarse aggregate mix, to serve as a cushion and a capillary break. Where moisture susceptible storage or floor coverings are anticipated, a 15-mil Stego Wrap membrane (or equivalent) 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 eight inches. The sand should be lightly moistened prior to placing concrete. However, the concrete contractor may select omitting the sand layer during construction.

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 may be 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.

## 6.9 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 3: Retaining Wall Design Parameters and Figure 8: Retaining Wall Detail for the design of retaining walls at the Site. The Active Case may be used for the design of unrestrained retaining walls, and the At-Rest Case may be used for the design of restrained retaining walls.

**Table 3: Retaining Wall Design Parameters**

| Lateral Pressure and Condition | Equivalent Fluid Pressure, pcf |
|---|---|
| Static, Active Case, Uniform Competent Formational Material ($\gamma' K_A$) | 45 |
| Static, At-Rest Case, Uniform Competent Formational Material ($\gamma' K_O$) | 65 |
| Static, Passive Case, Uniform Competent Formational Material ($\gamma' K_P$) | 350 |

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 8: Retaining Wall Detail and Figure 9: Retaining Wall Active and Passive Wedges for a description of the location of the active wedge behind a retaining wall.

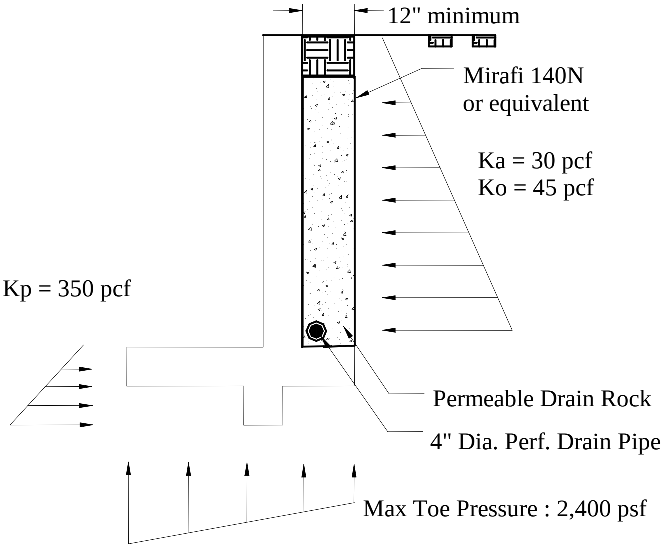

**Figure 8: Retaining Wall Detail**

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.

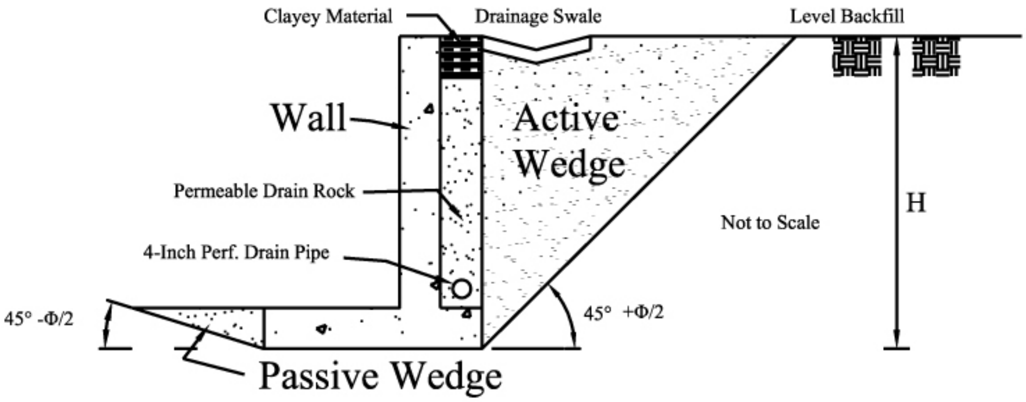

Figure 9: Retaining Wall Active and Passive Wedges

5. Retaining wall foundations should be founded a minimum of 12 inches below lowest adjacent grade in 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 **0.45** may be used between engineered fill or 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 engineered fill or 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 **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 (**$PGA_M$ = 0.564g**). 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. These seismic lateral earth pressure values are appropriate for retaining walls that have level retained surfaces, that have an approximately vertical surface against the retained material, and that retain granular backfill material or engineered fill composed of native soil within the active wedge. For other retaining wall designs, seismic lateral earth pressure values may be obtained using methods such as the Mononobe and Okabe Method developed by Mononobe and Matsuo (1929) and Okabe (1926), which are included in retaining wall computer design software such as Retain Pro.

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

9. In addition to the static lateral soil pressure values reported in Table 3: Retaining Wall Design Parameters, the retaining walls at the Site should be designed to support any design live load, such as from vehicle and construction surcharges, etc., to be supported by the wall backfill. If construction vehicles are required to operate within 10 feet of a retaining wall, supplemental pressures will be induced and should be taken into account in the design of the retaining wall.

10. The recommended lateral earth pressure values are based on the assumption that sufficient sub-surface drainage will be provided behind the walls to prevent the build-up of hydrostatic pressure. To achieve this we recommend that a granular filter material be placed behind all proposed walls. The blanket of granular filter material should be a minimum of 12 inches thick and should extend from the bottom of the wall to 12 inches from the ground surface. The top 12 inches should consist of moisture conditioned, compacted, clayey soil. Neither spread nor wall footings should be founded in the granular filter material used as backfill.

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

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

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

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

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

## 7.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations contained in this report are based on a limited number of borings 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, 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 4: Required Verification and Inspections of Soils Required Verification and Inspection of Soils:

**Table 4: Required Verification and Inspections of Soils**

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

## 8.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

2. This report is issued with the understanding that it is the responsibility of the owner or his/her representative to ensure that the information and recommendations contained herein are brought to

the attention of the architect and engineer for the project, and incorporated into the project plans and specifications. The owner or his/her representative is responsible to ensure that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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

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# REFERENCES

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# APPENDIX A

Field Investigation

Soil Classification Chart

Boring Logs (GeoSolutions, Inc., 2004 & 2008)

# FIELD INVESTIGATION

Two field investigations were conducted for this report. The first field investigation included a site investigation and literature review of the referenced report. The descriptions of the field investigation and associated Boring Logs by GeoSolutions, Inc. dated November 19, 2004. The second field investigation was conducted between February 4, 2008 and March 6, 2008 (see dated on boring logs) utilizing a track-mounted CME 55 drill rig.

The following Boring Logs were prepared for the GeoSolutions, Inc. report dated November 19, 2004 and February 4-March 6, 2008. A representative of GeoSolutions, Inc. maintained these Boring Logs 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 Soil Classification Chart, **Appendix A**.

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<br>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<br>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

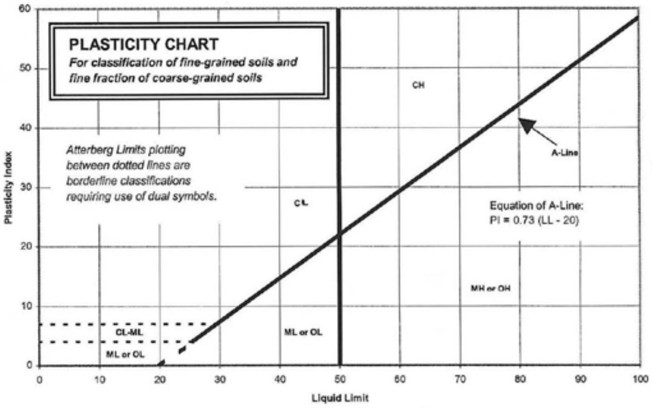
| SANDS, GRAVELS AND NON-PLASTIC SILTS | BLOWS/ FOOT + |
|---|---|
| VERY LOOSE | 0 - 4 |
| LOOSE | 4 - 10 |
| MEDIUM DENSE | 10 - 30 |
| DENSE | 30 - 50 |
| VERY DENSE | Over 50 |

\+ 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.

220 High Street
San Luis Obispo, CA 93401

# BORING LOG

BORING NO. **B-1**

JOB NO. **SL03926-1**

## PROJECT INFORMATION

PROJECT: **Seaside Garden Cottages**
DRILLING LOCATION: **See Figure 2, Site Plan**
DATE DRILLED: **2/5/04**
LOGGED BY: **ND**

## DRILLING INFORMATION

DRILL RIG: **Mobile B61**
HOLE DIAMETER: **6 Inches**
SAMPLING METHOD: **CA**
HOLE ELEVATION: **~235 Feet**

▼ Depth of Groundwater: **Not Encountered**

Boring Terminated At: **15 feet**

Page 1 of 3

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN, tsf | IN-SITU WATER CONTENT% | IN-SITU DRY DENSITY, pcf | OPTIMUM WATER CONTENT, % | MAXIMUM DRY DENSITY, pcf | EXPANSION INDEX (EI) | ATTERBERG LIMIT (PI) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | SILTY SAND: very dark greyish brown silty SAND with clay, slightly moist, medium dense | SM | | | | | | | | | | |
| | | | | X | 13 | | | | | | | |
| -1 | | | | A | | | | | | 117.4 | 6 | |
| -2 | | | | | | | | | 12.3 | | | |
| -3 | SANDY CLAY: very dark greysih brown with silt, slightly moist, hard, Pismo Formation | CL | | | | | | | | | | |
| | | | | X | 44 | | 12.0 | 103.8 | | | | |
| -4 | | | | B | | | | | 14.5 | 109.1 | 23 | 23 |
| -5 | | | | | | | | | | | | |
| -6 | | | | | | | | | | | | |
| -7 | | | | | | | | | | | | |
| -8 | | | | | | | | | | | | |
| | | | | X | 52 | | 25.4 | 87.7 | | | | |
| -9 | | | | | | | | | | | | |
| -10 | SILTY CLAY: olive brown, slightly moist, hard, Pismo Formation | CL | | C | | | | | 17.3 | 106.5 | 41 | 25 |
| -11 | | | | | | | | | | | | |
| -12 | | | | | | | | | | | | |
| -13 | | | | | | | | | | | | |
| | | | | X | 50/6 | | 27.6 | 88.9 | | | | |
| -14 | | | | | | | | | | | | |
| -15 | | | | | | | | | | | | |
| -16 | | | | | | | | | | | | |
| -17 | | | | | | | | | | | | |
| -18 | | | | | | | | | | | | |
| -19 | | | | | | | | | | | | |
| -20 | | | | | | | | | | | | |

# GeoSolutions, Inc.

220 High Street
San Luis Obispo, CA 93401

# BORING LOG

BORING NO. **B-2**

JOB NO. **SL03926-1**

| PROJECT INFORMATION | DRILLING INFORMATION |
|---|---|
| PROJECT: **Seaside Garden Cottages** | DRILL RIG: **Mobile B61** |
| DRILLING LOCATION: **See Figure 2, Site Plan** | HOLE DIAMETER: **6 Inches** |
| DATE DRILLED: **2/5/04** | SAMPLING METHOD: **CA** |
| LOGGED BY: **ND** | HOLE ELEVATION: **~235 Feet** |

Depth of Groundwater: **Not Encountered**  Boring Terminated At: **15 feet**  Page 2 of 3

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN, tsf | IN-SITU WATER CONTENT% | IN-SITU DRY DENSITY, pcf | OPTIMUM WATER CONTENT, % | MAXIMUM DRY DENSITY, pcf | EXPANSION INDEX (EI) | ATTERBERG LIMIT (PI) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | SILTY SAND: brown, with clay, slightly moist, medium dense | SM | | | | | | | | | | |
| -1 | | | | X | 12 | | | | | | | |
| -2 | | | | | | | | | | | | |
| -3 | | | | | | | | | | | | |
| -4 | | | | X | 17 | | 11.7 | 98.1 | | | | |
| -5 | | | | | | | | | | | | |
| -6 | | | | | | | | | | | | |
| -7 | SANDY CLAY: light yellowish brown, with silt, slightly moist, hard, Pismo Formation | CL | | | | | | | | | | |
| -8 | | | | | | | | | | | | |
| -9 | | | | X | 35 | | 25.7 | 111.7 | | | | |
| -10 | | | | | | | | | | | | |
| -11 | | | | | | | | | | | | |
| -12 | | | | | | | | | | | | |
| -13 | | | | | | | | | | | | |
| -14 | | | | X | 41 | | 27.2 | 88.9 | | | | |
| -15 | | | | | | | | | | | | |
| -16 | | | | | | | | | | | | |
| -17 | | | | | | | | | | | | |
| -18 | | | | | | | | | | | | |
| -19 | | | | | | | | | | | | |
| -20 | | | | | | | | | | | | |

# GeoSolutions, Inc.

220 High Street
San Luis Obispo, CA 93401

# BORING LOG

BORING NO. **B-3**
JOB NO. **SL03926-1**

## PROJECT INFORMATION

PROJECT: **Seaside Garden Cottages**
DRILLING LOCATION: **See Figure 2, Site Plan**
DATE DRILLED: **2/5/04**
LOGGED BY: **ND**

## DRILLING INFORMATION

DRILL RIG: **Mobile B61**
HOLE DIAMETER: **6 Inches**
SAMPLING METHOD: **CA**
HOLE ELEVATION: **~235 Feet**

▼ Depth of Groundwater: **Not Encountered**

Boring Terminated At: **15 feet**

Page 3 of 3

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN, tsf | IN-SITU WATER CONTENT% | IN-SITU DRY DENSITY, pcf | OPTIMUM WATER CONTENT, % | MAXIMUM DRY DENSITY, pcf | EXPANSION INDEX (EI) | ATTERBERG LIMIT (PI) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | SILTY SAND: brown, with clay, slightly moist, loose | SM | ▨ | X | 8 | | | | | | | |
| -1 | | | ▨ | | | | | | | | | |
| -2 | | | ▨ | | | | | | | | | |
| -3 | SILTY SAND: light brown, with clay, slightly moist, very dense, Pismo Formation | CL | ▨ | X | 50/3 | | 14.9 | 95.2 | | | | |
| -4 | | | | | | | | | | | | |
| -5 | | | | | | | | | | | | |
| -6 | | | | | | | | | | | | |
| -7 | | | | | | | | | | | | |
| -8 | | | | | | | | | | | | |
| -9 | | | | | | | | | | | | |
| -10 | | | | | | | | | | | | |
| -11 | | | | | | | | | | | | |
| -12 | | | | | | | | | | | | |
| -13 | | | | | | | | | | | | |
| -14 | | | | | | | | | | | | |
| -15 | | | | | | | | | | | | |
| -16 | | | | | | | | | | | | |
| -17 | | | | | | | | | | | | |
| -18 | | | | | | | | | | | | |
| -19 | | | | | | | | | | | | |
| -20 | | | | | | | | | | | | |

# GeoSolutions, Inc.

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# BORING LOG

BORING NO. **B-4**

JOB NO. **SL03926-3**

## PROJECT INFORMATION

PROJECT: **Seaside Garden Cottages**
DRILLING LOCATION: **See Plate 1, Eng. Geo. Map**
DATE DRILLED: **2-20-08**
LOGGED BY: **LZ**

## DRILLING INFORMATION

DRILL RIG: **CME 55**
HOLE DIAMETER: **4 Inches**
SAMPLING METHOD: **CA**
HOLE ELEVATION: **197 Feet**

▼ Depth of Groundwater: **Not Encountered**

Boring Terminated At: **100 Feet bgs**

Page 1 of 10

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE, (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | SM | | | | | | | | |
| -1 | SILTY SAND: dark brown to black, slightly moist to moist, Colluvium (Qc) | | | | | | | | | |
| -2 | | | | | | | | | | |
| -3 | SANDSTONE: light pale green, dry to slightly moist, severely weathered, soft to very soft, fine to coarse grained, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -4 | | | | CA | 56 | | | | | |
| -5 | | | | | | | | | | |
| -6 | | | | | | | | | | |
| -7 | | | | | | | | | | |
| -8 | | | | | | | | | | |
| -9 | | | | CA | 37 | | | | | |
| -10 | moderately weathered (at 11 feet) | | | | | | | | | |
| -11 | | | | | | | | | | |
| -12 | | | | | | | | | | |
| -13 | | | | | | | | | | |
| -14 | | | | CA | 57 | | | | | |
| -15 | | | | | | | | | | |
| -16 | | | | | | | | | | |
| -17 | | | | | | | | | | |
| -18 | | | | | | | | | | |
| -19 | | | | CA | 29 | | | | | |
| -20 | | | | | | | | | | |
| -21 | | | | | | | | | | |
| -22 | | | | | | | | | | |
| -23 | end of auger (25 feet)/Start of Coring | | | | | | | | | |
| -24 | | | | CA | 46 | | | | | |
| -25 | | | | | | | | | | |
| -26 | SANDSTONE: pale green, soft to very soft, moderately to slightly weathered, slight oxidation, very fine to medium grained, Squire Member of the Pismo Formation (Tpps) | | | | | 4.75 | 0 | 8 | | |
| -27 | | | | | | | | | | |
| -28 | | | | | | | | | | |
| -29 | | | | | | | | | | |
| -30 | | | | | | | | | | |
| -31 | | | | | | 4.75 | 0 | 12 | | |
| -32 | | | | | | | | | | |
| -33 | | | | | | | | | | |
| -34 | | | | | | | | | | |
| -35 | | | | | | | | | | |
| -36 | | | | | | 4.75 | 35 | 40 | | |
| -37 | | | | | | | | | | |
| -38 | | | | | | | | | | |
| -39 | | | | | | | | | | |
| -40 | | | | | | | | | | |
| -41 | | | | | | | 0 | N/R | | |
| -42 | | | | | | | | | | |
| -43 | | | | | | | | | | |
| -44 | | | | | | | | | | |
| -45 | | | | | | | | | | |
| -46 | | | | | | 4.75 | 8.3 | 40 | | |

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# BORING LOG

BORING NO (cont). **B-4**
JOB NO. **SL03926-3**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **100 Feet bgs**

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| -47 | | | | | | | | | | |
| -50 | | | | | | | 51.6 | 79 | | |
| -52 | SANDSTONE AND CLAYSTONE: pale green to light gray, very soft to soft, medium to very fine grained, appears to be intermingling of green and gray, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -55 | | | | | | 4.75 | 51.6 | 88 | | |
| -58 | SANDSTONE: pale green, two fractures with evidence of water, moderate to severe oxidation, slightly weathered to fresh, Squire Member of the Pismo Formation (Tpps)<br>(-61.5 feet) a single zone of highly fractured rock with severe oxidation approximately 2 inches in length, dense, but still easily scratches with fingernail. | | | | | | | | | |
| -60 | | | | | | 4.75 | 86.6 | 97 | | |
| -65 | CLAYSTONE: light gray, soft to very soft, medium to very fine grained, Squire Member of the Pismo Formation (Tpps) | | | | | 4.75 | 61.6 | 100 | | |
| -70 | | | | | | 4.75 | 58.3 | 43 | | |
| -71 | CLAYSTONE: dark gray, fresh, moderately soft to soft, fine grained to very fine grained, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -75 | | | | | | 4.75 | 83.3 | 59 | | |
| -80 | | | | | | 4.75 | 75.0 | 100 | | |
| -85 | | | | | | 4.75 | 96.7 | 82 | | |
| -90 | | | | | | 4.75 | 98.3 | 82 | | |
| -95 | | | | | | 4.75 | 63.3 | 54 | | |
| -100 | | | | | | | | | | |

# GeoSolutions, Inc.

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# BORING LOG

BORING NO. **B-5**

JOB NO. **SL03926-3**

## PROJECT INFORMATION

PROJECT: **Seaside Garden Cottages**
DRILLING LOCATION: **See Plate 1, Eng. Geo. Map**
DATE DRILLED: **2-4-08**
LOGGED BY: **LZ**

## DRILLING INFORMATION

DRILL RIG: **CME 55**
HOLE DIAMETER: **8 Inches**
SAMPLING METHOD: **CA**
HOLE ELEVATION: **221 Feet**

▼ Depth of Groundwater: **Not Encountered**

Boring Terminated At: **85 Feet bgs**

Page 3 of 10

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE, (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | SM | | | | | | | | |
| -1 | SILTY CLAY: black, slightly moist, Colluvium (Qc) | | | | | | | | | |
| -2 | | | | | | | | | | |
| -3 | | | | | | | | | | |
| -4 | | | | CA | 50/5" | | | | | |
| -5 | | | | | | | | | | |
| -6 | | | | | | | | | | |
| -7 | | | | | | | | | | |
| -8 | | | | | | | | | | |
| -9 | | | | CA | 49 | | | | | |
| -10 | SANDSTONE: pale green, slightly moist, very soft to soft, intensely, weathered, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -11 | | | | | | | | | | |
| -12 | | | | | | | | | | |
| -13 | | | | | | | | | | |
| -14 | | | | CA | 39 | | | | | |
| -15 | | | | | | | | | | |
| -16 | | | | | | | | | | |
| -17 | | | | | | | | | | |
| -18 | | | | | | | | | | |
| -19 | | | | CA | 33 | | | | | |
| -20 | moderately weathered | | | | | | | | | |
| -21 | | | | | | | | | | |
| -22 | | | | | | | | | | |
| -23 | slightly weathered | | | | | | | | | |
| -24 | | | | CA | 26 | | | | | |
| -25 | | | | | | | | | | |
| -26 | | | | | | | | | | |
| -27 | | | | | | | | | | |
| -28 | | | | | | | | | | |
| -29 | | | | CA | 22 | | | | | |
| -30 | | | | | | | | | | |
| -31 | | | | | | | | | | |
| -32 | | | | | | | | | | |
| -33 | | | | | | | | | | |
| -34 | | | | CA | 28 | | | | | |
| -35 | | | | | | | | | | |
| -36 | | | | | | | | | | |
| -37 | | | | | | | | | | |
| -38 | | | | | | | | | | |
| -39 | | | | CA | 52 | | | | | |
| -40 | end of auger (40 feet)/Start of Coring | | | | | | | | | |
| -41 | | | | | | | 85 | 43 | | |
| -42 | SANDSTONE: pale green, slightly moist, very soft to soft, intensely, weathered, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -43 | | | | | | | | | | |
| -44 | | | | | | | | | | |
| -45 | | | | | | | | | | |
| -46 | | | | | | | 65 | 100 | | |

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# BORING LOG

BORING NO (cont). **B-5**

JOB NO. **SL03926-3**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **85 Feet bgs**

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| -47 | | | | | | | | | | |
| -48 | SILTSTONE AND CLAYSTONE: gray, moderately soft to soft, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -49 | | | | | | | | | | |
| -50 | | | | | | 4.75 | 100 | 100 | | |
| -51 | | | | | | | | | | |
| -52 | | | | | | | | | | |
| -53 | | | | | | | | | | |
| -54 | | | | | | | | | | |
| -55 | | | | | | 4.75 | 68.3 | 53 | | |
| -56 | | | | | | | | | | |
| -57 | | | | | | | | | | |
| -58 | | | | | | | | | | |
| -59 | | | | | | | | | | |
| -60 | | | | | | 4.75 | 95 | 100 | | |
| -61 | | | | | | | | | | |
| -62 | | | | | | | | | | |
| -63 | | | | | | | | | | |
| -64 | | | | | | | | | | |
| -65 | | | | | | 4.75 | 43.3 | 100 | | |
| -66 | | | | | | | | | | |
| -67 | | | | | | | | | | |
| -68 | | | | | | | | | | |
| -69 | | | | | | | | | | |
| -70 | | | | | | 4.75 | 56.6 | 78 | | |
| -71 | | | | | | | | | | |
| -72 | | | | | | | | | | |
| -73 | | | | | | | | | | |
| -74 | | | | | | | | | | |
| -75 | | | | | | 4.75 | 70 | 80 | | |
| -76 | | | | | | | | | | |
| -77 | | | | | | | | | | |
| -78 | | | | | | | | | | |
| -79 | | | | | | | | | | |
| -80 | | | | | | 4.75 | 0 | N/R | | |
| -81 | | | | | | | | | | |
| -82 | | | | | | | | | | |
| -83 | | | | | | | | | | |
| -84 | | | | | | | | | | |
| -85 | | | | | | | | | | |
| -86 | | | | | | | | | | |
| -87 | | | | | | | | | | |
| -88 | | | | | | | | | | |
| -89 | | | | | | | | | | |
| -90 | | | | | | | | | | |
| -91 | | | | | | | | | | |
| -92 | | | | | | | | | | |
| -93 | | | | | | | | | | |
| -94 | | | | | | | | | | |
| -95 | | | | | | | | | | |
| -96 | | | | | | | | | | |
| -97 | | | | | | | | | | |
| -98 | | | | | | | | | | |
| -99 | | | | | | | | | | |
| -100 | | | | | | | | | | |

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# BORING LOG

BORING NO. **B-6**

JOB NO. **SL03926-3**

## PROJECT INFORMATION

PROJECT: **Seaside Garden Cottages**
DRILLING LOCATION: **See Plate 1, Eng. Geo. Map**
DATE DRILLED: **3/6/08**
LOGGED BY: **LZ**

## DRILLING INFORMATION

DRILL RIG: **CME 55**
HOLE DIAMETER: **8 Inches**
SAMPLING METHOD: **CA**
HOLE ELEVATION: **35 Feet**

▼ Depth of Groundwater: **Not Encountered**

Boring Terminated At: **50 Feet bgs**

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE, (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 to -2 | SILTY SAND: pale green, dry, Colluvium (Qc) | SM | | | | | | | | |
| -2 to -19 | SILTY SAND: pale brown, slightly moist, Colluvium (Qc) | SM | | | | | | | | |
| -4 | | | | CA | 52 | | | | | |
| -9 | | | | CA | 26 | | | | | |
| -14 | | | | CA | 9 | | | | | |
| -19 to -25 | SANDSTONE: pale brown, intensely to moderately weathered, coarse to fine grained, soft to very soft, Squire Member of the Pismo Formation (Tpps)<br>End of Auger/Start of Coring | | | CA | 9 | | | | | |
| -24 | | | | CA | 50/5" | | | | | |
| -25 to -48 | SANDSTONE: pale brown, Squire Member of the Pismo Formation (Tpps) | | | | | 4.75 | 45.8 | 72 | | |
| -30 | | | | | | 4.75 | 46.6 | 68 | | |
| -35 | | | | | | 4.75 | 53.3 | 59 | | |
| -40 | | | | | | 4.75 | 71.6 | 53 | | |
| -45 | | | | | | 4.75 | 30 | 30 | | |
| -48 to -50 | BASALT: Franciscan Complex (Jfv), pillow, highly fractured, slightly weathered to fresh, very hard to moderately hard | | | | | | | | | |

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# BORING LOG

BORING NO. **B-7**

JOB NO. **SL03926-3**

| PROJECT INFORMATION | | DRILLING INFORMATION | |
|---|---|---|---|
| PROJECT: | **Seaside Garden Cottages** | DRILL RIG: | **CME 55** |
| DRILLING LOCATION: | **See Plate 1, Eng. Geo. Map** | HOLE DIAMETER: | **8 Inches** |
| DATE DRILLED: | **3-3-08** | SAMPLING METHOD: | **CA** |
| LOGGED BY: | **LZ** | HOLE ELEVATION: | **~31 Feet** |

▼ Depth of Groundwater: **Not Encountered** Boring Terminated At: **50 Feet bgs** Page 6 of 10

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE, (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| -1 | SILTY SAND: greenish, moist, Colluvium (Qc) | SM | | | | | | | | |
| -3 | SANDY SILT: greenish brown, slightly moist, Colluvium (Qc) | SM | | | | | | | | |
| -4 | | SM | | CA | 41 | | | | | |
| -5 | SILTY SAND: greenish brown, moist, Colluvium (Qc) | | | | | | | | | |
| -9 | | | | CA | 9 | | | | | |
| -14 | | | | CA | 9 | | | | | |
| -18 | | SM | | | | | | | | |
| -19 | SAND: gray, medium to coarse grained, Colluvium (Qc) | | | CA | 15 | | | | | |
| -21 | SANDSTONE: pale green to greenish brown, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -24 | CLAYSTONE: gray, moderately soft to soft, slightly weathered to fresh, Squire Member of the Pismo Formation (Tpps)<br>End of Auger/Start of Coring | | | CA | 50/5" | | | | | |
| -26 | | | | | | 4.75 | 46.6 | 80 | | |
| -28 | CLAYSTONE: gray, moderately soft to soft, slightly weathered to fresh, with 1/4 inch pieces of gravel at 31 feet, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -31 | | | | | | 4.75 | 58.3 | 100 | | |
| -36 | BASALT: highly fractured, brittle, slightly weathered to fresh, very hard to moderately hard, Franciscan Complex (Jfv) | | | | | 4.75 | 6.6 | 40 | | |
| -41 | | | | | | 4.75 | 35 | 77 | | |
| -46 | | | | | | 4.75 | 31.6 | 80 | | |
| -50 | | | | | | | | | | |

## GeoSolutions, Inc.

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# BORING LOG

BORING NO. **B-8**

JOB NO. **SL03926-3**

| PROJECT INFORMATION | | DRILLING INFORMATION | |
|---|---|---|---|
| PROJECT: | **Seaside Garden Cottages** | DRILL RIG: | **CME 55** |
| DRILLING LOCATION: | **See Plate 1, Eng. Geo. Map** | HOLE DIAMETER: | **8 Inches** |
| DATE DRILLED: | **2-7-08** | SAMPLING METHOD: | **CA** |
| LOGGED BY: | **LZ** | HOLE ELEVATION: | **178.6 Feet** |

▼ Depth of Groundwater: **Not Encountered** Boring Terminated At: **100 Feet bgs** Page 7 of 10

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE, (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | SM | | | | | | | | |
| -1 | CLAY: black, slightly moist, Colluvium (Qc) | | | | | | | | | |
| -3 | SANDSTONE: intensely weathered, soft to very soft, massive, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -4 | | | | CA | 43 | | | | | |
| -9 | | | | CA | 43 | | | | | |
| -12 | moderately weathered (14 ft) | | | | | | | | | |
| -14 | | | | CA | 61 | | | | | |
| -19 | End of Auger/Start of Coring | | | CA | 50/5" | | | | | |
| -21 | SANDSTONE: pale green, very soft to soft, slightly weathered, some oxidation, fine to coarse grained, Squire Member of the Pismo Formation (Tpps) | | | | | 3.5-4.75 | 48.3 | 67 | | |
| -26 | | | | | | | 48.3 | 88 | | |
| -31 | SANDSTONE: pale green, much firmer section than other, still scratches with fingernail, Squire Member of the Pismo Formation (Tpps) | | | | | 2.25-4.75 | 70 | 100 | | |
| -36 | | | | | | | 40 | 50 | | |
| -41 | SANDSTONE: pale green, coarse to fine grained, slightly weathered, some oxidation in fractures, very soft to moderately soft, Squire Member of the Pismo Formation (Tpps) | | | | | | 48.3 | 77 | | |
| -46 | | | | | | 0.75-1.75 | 13.3 | 22 | | |

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# BORING LOG

BORING NO (cont). **B-8**

JOB NO. **SL03926-3**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **100 Feet bgs**

Page 8 of 10

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| -47 | SANDSTONE: pale green, coarse to fine grained, very soft to soft, slightly weathered, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -48 | | | | | | | | | | |
| -49 | | | | | | | | | | |
| -50 | | | | | | 0 | 0 | 6 | | |
| -51 | | | | | | | | | | |
| -52 | | | | | | | | | | |
| -53 | | | | | | | | | | |
| -54 | | | | | | | | | | |
| -55 | localized hard rock, slight oxidation, some fractures | | | | | 1.9 | 15 | 75 | | |
| -56 | | | | | | | | | | |
| -57 | | | | | | | | | | |
| -58 | | | | | | | | | | |
| -59 | | | | | | | | | | |
| -60 | | | | | | 2.5 | 0 | 8 | | |
| -61 | | | | | | | | | | |
| -62 | | | | | | | | | | |
| -63 | | | | | | | | | | |
| -64 | | | | | | | | | | |
| -65 | | | | | | 2.25-4.75 | 25 | 9 | | |
| -66 | | | | | | | | | | |
| -67 | | | | | | | | | | |
| -68 | | | | | | | | | | |
| -69 | | | | | | | | | | |
| -70 | | | | | | 1.2-4.75 | 48.3 | 75 | | |
| -71 | | | | | | | | | | |
| -72 | SANDSTONE: pale green, coarse to fine grained, soft clay, layers up to 1/4 inch thick, slightly weathered, medium to very fine grained, some oxidation in fractures, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -73 | | | | | | | | | | |
| -74 | | | | | | | | | | |
| -75 | | | | | | 0.5-2.5 | 6.6 | 70 | | |
| -76 | | | | | | | | | | |
| -77 | | | | | | | | | | |
| -78 | | | | | | | | | | |
| -79 | | | | | | | | | | |
| -80 | | | | | | 4.75 | 0 | 6 | | |
| -81 | | | | | | | | | | |
| -82 | | | | | | | | | | |
| -83 | | | | | | | | | | |
| -84 | | | | | | | | | | |
| -85 | CLAYSTONE: dark bluish gray, moderately hard to soft, fine to very fine grained, fresh to slightly weathered, Squire Member of the Pismo Formation (Tpps) | | | | | 4.25-4.75 | 71.6 | 100 | | |
| -86 | | | | | | | | | | |
| -87 | | | | | | | | | | |
| -88 | | | | | | | | | | |
| -89 | | | | | | | | | | |
| -90 | | | | | | 4.75 | 87.5 | 100 | | |
| -91 | | | | | | | | | | |
| -92 | | | | | | | | | | |
| -93 | | | | | | | | | | |
| -94 | | | | | | | | | | |
| -95 | | | | | | 4.75 | 81.6 | 100 | | |
| -96 | | | | | | | | | | |
| -97 | | | | | | | | | | |
| -98 | | | | | | | | | | |
| -99 | | | | | | | | | | |
| -100 | | | | | | | | | | |

**GeoSolutions, Inc.**

220 High Street
San Luis Obispo, CA 93401

# BORING LOG

BORING NO. **B-9**

JOB NO. **SL03926-3**

| PROJECT INFORMATION | | DRILLING INFORMATION | |
|---|---|---|---|
| PROJECT: | **Seaside Garden Cottages** | DRILL RIG: | **CME 55** |
| DRILLING LOCATION: | **See Plate 1, Eng. Geo. Map** | HOLE DIAMETER: | **8 Inches** |
| DATE DRILLED: | **2-14-08** | SAMPLING METHOD: | **CA** |
| LOGGED BY: | **LZ** | HOLE ELEVATION: | **211 Feet** |

▼ Depth of Groundwater: **Not Encountered**

Boring Terminated At: **85 Feet bgs**

Page 9 of 10

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE, (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | SM | | | | | | | | |
| -1 | CLAYEY SAND: black, with organics, slightly moist, Colluvium (Qc) | | | | | | | | | |
| -2 | | | | | | | | | | |
| -3 | SANDSTONE: pale green to white, very soft, highly weathered, coarse to fine grained, slightly moist, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -4 | | | | CA | 43 | | | | | |
| -5 | | | | | | | | | | |
| -6 | | | | | | | | | | |
| -7 | | | | | | | | | | |
| -8 | | | | | | | | | | |
| -9 | | | | CA | 51 | | | | | |
| -10 | | | | | | | | | | |
| -11 | | | | | | | | | | |
| -12 | | | | | | | | | | |
| -13 | end of auger/Start of Coring | | | | | | | | | |
| -14 | | | | CA | 50/5" | | | | | |
| -15 | | | | | | 4.0-4.25 | 26.6 | 32 | | |
| -16 | SANDSTONE: pale green, coarse to fine grained, very soft to soft, slightly weathered, Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -17 | | | | | | | | | | |
| -18 | | | | | | | | | | |
| -19 | | | | CA | | | | | | |
| -20 | | | | | | | 0 | 3 | | |
| -21 | SANDSTONE: pale green, medium to coarse grained, soft to very soft, slightly weathered, Squire Member of the Pismo Formation (Tpps) | | | | | 0.25-3.5 | | | | |
| -22 | | | | | | | | | | |
| -23 | | | | | | | | | | |
| -24 | | | | | | | | | | |
| -25 | | | | | | | 0 | 5 | | |
| -26 | | | | | | | | | | |
| -27 | | | | | | | | | | |
| -28 | | | | | | | | | | |
| -29 | | | | | | | | | | |
| -30 | | | | | | | 6.6 | 28 | | |
| -31 | | | | | | 2.25-4.75 | | | | |
| -32 | | | | | | | | | | |
| -33 | | | | | | | | | | |
| -34 | | | | | | | | | | |
| -35 | | | | | | | 6.6 | 48 | | |
| -36 | | | | | | 1.5-4.75 | | | | |
| -37 | | | | | | | | | | |
| -38 | | | | | | | | | | |
| -39 | | | | | | | | | | |
| -40 | | | | | | | 23.3 | 25 | | |
| -41 | SANDSTONE: pale green, with some gray splotches, fine to coarse grained, soft to very soft, Squire Member of the Pismo Formation (Tpps) | | | | | .5-4.75 | | | | |
| -42 | | | | | | | | | | |
| -43 | | | | | | | | | | |
| -44 | | | | | | | | | | |
| -45 | | | | | | | 53.3 | 73 | | |
| -46 | | | | | | 4.75 | | | | |

**GeoSolutions, Inc.**
220 High Street
San Luis Obispo, CA 93401

# BORING LOG

BORING NO (cont). **B-9**
JOB NO. **SL03926-3**

Depth of Groundwater: **Not Encountered** Boring Terminated At: **85 Feet bgs** Page 10 of 10

| DEPTH | SOIL DESCRIPTION | USCS | LITHOLOGY | SAMPLE | BLOWS/ 12 IN | POCKET PEN | RQD | PERCENT RECOVERED | FRICTION ANGLE (degrees) | COHESION, C (psf) |
|---|---|---|---|---|---|---|---|---|---|---|
| -47 to -50 | | | | | | | | | | |
| -50 to -55 | | | | | | 1-4.75 | 5.5 | 60 | | |
| -55 to -60 | | | | | | - | 0 | 14 | | |
| -60 to -65 | SANDSTONE: pale green, medium to coarse grained, soft to very soft, friable, Squire Member of the Pismo Formation (Tpps) | | | | | 2.5-4.75 | 0 | 21 | | |
| -65 to -70 | | | | | | | N/R | N/R | | |
| -70 to -75 | | | | | | -<br>1.5-4.75 | 0 | 7 | | |
| -75 to -80 | | | | | | 2.5-4.75 | 18 | 0 | | |
| -80 to -84 | | | | | | 4.75 | 21 | 11.6 | | |
| -84 to -85 | CLAYSTONE: Squire Member of the Pismo Formation (Tpps) | | | | | | | | | |
| -85 to -100 | | | | | | | | | | |

# APPENDIX B

Laboratory Testing

Soil Test Reports (GeoSolutions, Inc., 2004 & 2008)

# LABORATORY TESTING

This appendix includes a discussion of the test procedures and of the laboratory test results performed during the preparation of the referenced report(s) (GeoSolutions, Inc., 2004 & 2008). The purpose of the laboratory testing was to assess the engineering properties of the soil materials at the Site. The program was carried out employing, wherever practical, currently accepted test methods of the American Society for Testing and Materials (ASTM).

Undisturbed and disturbed bulk samples used in the laboratory tests were obtained from various locations during the course of the field exploration described in the referenced report(s) (GeoSolutions, Inc., 2004 & 2008) and 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 during the referenced report(s) (GeoSolutions, Inc., 2004 & 2008) are described below; the associated Soil Test Reports are also included in this appendix: (make sure that the test methods match those listed in the referenced (original) report)

**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.

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

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

**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.

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

| | | | | |
|---|---|---|---|---|
| Project: | Seaside Garden Cottages | | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | | Project #: | SL03926-1 |
| Sample: | A | Depth: 1.0 feet | Lab #: | 3970 |
| Location: | B-1 | | Sample Date: | 2/5/2004 |
| | | | Sampled By: | ND |

**Soil Classification**

ASTM D2487-93, D2488-93

| | |
|---|---|
| Result: | Very Dark Grayish Brown Silty SAND w/ Clay |
| Specification: | SM |

**Sieve Analysis**

ASTM C136-96a

| Sieve Size | Percent Passing | Project Specifications |
|---|---|---|
| 3" | | - |
| 2" | | - |
| 1 1/2" | | - |
| 1" | | - |
| 3/4" | | - |
| No. 4 | 99 | - |
| No. 8 | 99 | - |
| No. 16 | 98 | - |
| No. 30 | 97 | - |
| No. 50 | 93 | - |
| No. 100 | 45 | - |
| No. 200 | 24.0 | - |

**Sand Equivalent Cal 217**

| | | SE |
|---|---|---|
| 1 | | |
| 2 | | |
| 3 | | |
| 4 | | |

**Plasticity Index**

ASTM D4318-95a

| | |
|---|---|
| Liquid Limit: | |
| Plastic Limit: | |
| Plasticity Index: | |

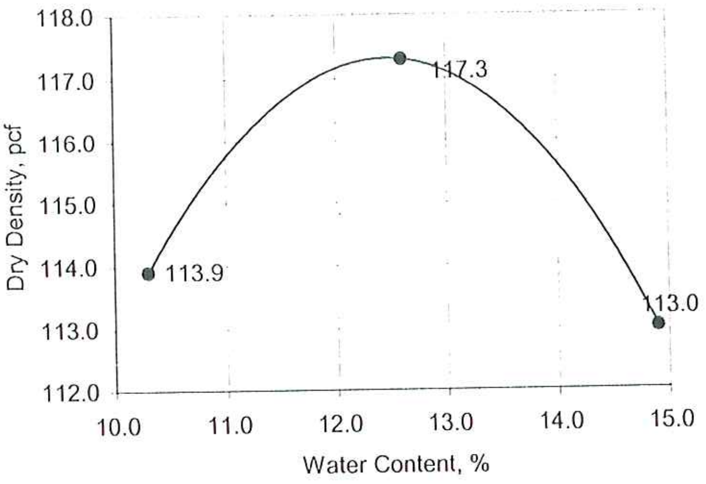
**Expansion Index**

ASTM D4829-95

| | |
|---|---|
| Expansion Index: | 6 |
| Expansion Potential: | Very Low |
| Initial Saturation, %: | 50 |

**Laboratory Maximum Density**

ASTM D1557-91



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

Estimated Specific Gravity for 100% Saturation Curve =2.55

| Trial # | 1 | 2 | 3 | 4 |
|---|---|---|---|---|
| Water Content: | 10.3 | 12.6 | 14.9 | |
| Dry Density: | 113.9 | 117.3 | 113.0 | |
| Maximum Dry Density, pcf: | | **117.4** | | |
| Optimum Water Content, %: | | **12.3** | | |

**Moisture-Density** ASTM D2937-94, ASTM D2216-92

| Sample | Depth (ft) | Water Content (%) | Dry Density (pcf) | Relative Density | Sample Description |
|---|---|---|---|---|---|
| B-1 | 3.5 | 12.0 | 103.8 | - | Very Dark Gray Silty SAND |
| B-1 | 8.5 | 25.4 | 87.7 | - | Pale Olive Silty SAND w/ Clay |
| B-1 | 13.6 | 27.6 | 88.9 | - | Pale Olive Silty SAND w/ Clay |
| B-2 | 3.5 | 11.7 | 98.1 | - | Very Dark Gray Silty SAND |
| B-2 | 8.5 | 25.7 | 111.7 | - | Black CLAY |
| B-2 | 13.5 | 27.2 | 88.9 | - | Olive Brown Clayey SILT |
| B-3 | 0.5 | 14.9 | 95.2 | - | Black Silty SAND w/ Clay |
| B-3 | 3.5 | 6.7 | 90.3 | - | Grayish Brown Sandy SIlt |
| | | | | | |
| | | | | | |

Report By: Darren Harrold

**GeoSolutions, Inc.**

# LABORATORY DATA REPORT

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample: | B    Depth: 3.5 feet | Lab #: | 3970 |
| Location: | B-1 | Sample Date: | 2/5/2004 |
| | | Sampled By: | ND |

### Soil Classification

ASTM D2487-93, D2488-93

| | |
|---|---|
| Result: | Very Dark Grayish Brown Sandy CLAY w/ Silt |
| Specification: | CL |

### Sieve Analysis

ASTM C136-96a

| 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

| | | SE |
|---|---|---|
| 1 | | |
| 2 | | |
| 3 | | |
| 4 | | |

### Plasticity Index

ASTM D4318-95a

| | |
|---|---|
| Liquid Limit: | 42 |
| Plastic Limit: | 19 |
| Plasticity Index: | 23 |

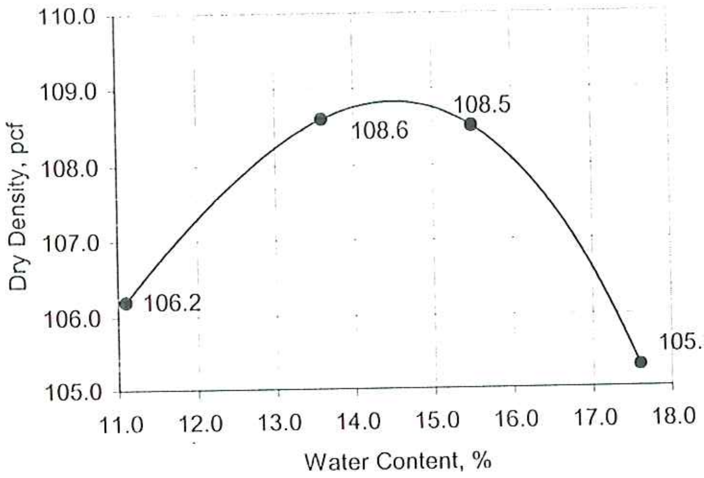
### Expansion Index

ASTM D4829-95

| | |
|---|---|
| Expansion Index: | 23 |
| Expansion Potential: | Low |
| Initial Saturation, %: | 50 |

### Laboratory Maximum Density

ASTM D1557-91



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

Estimated Specific Gravity for 100% Saturation Curve =2.55

| | | | | |
|---|---|---|---|---|
| Trial # | 1 | 2 | 3 | 4 |
| Water Content: | 11.1 | 13.6 | 15.5 | 17.6 |
| Dry Density: | 106.2 | 108.6 | 108.5 | 105.3 |
| Maximum Dry Density, pcf: | | **109.1** | | |
| Optimum Water Content, %: | | **14.5** | | |

### Moisture-Density ASTM D2937-94, ASTM D2216-92

| Sample | Depth (ft) | Water Content (%) | Dry Density (pcf) | Relative Density | Sample Description |
|---|---|---|---|---|---|
| | | | | | |
| | | | | | |
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| | | | | | |
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Report By: Darren Harrold

**GeoSolutions, Inc.** # LABORATORY DATA REPORT **(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample: | C Depth: 10.5 feet | Lab #: | 3970 |
| Location: | B-1 | Sample Date: | 2/5/2004 |
| | | Sampled By: | ND |

## Soil Classification
ASTM D2487-93, D2488-93

Result: Olive Brown Silty CLAY

Specification: CL

## Sieve Analysis
ASTM C136-96a

| 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 | | |

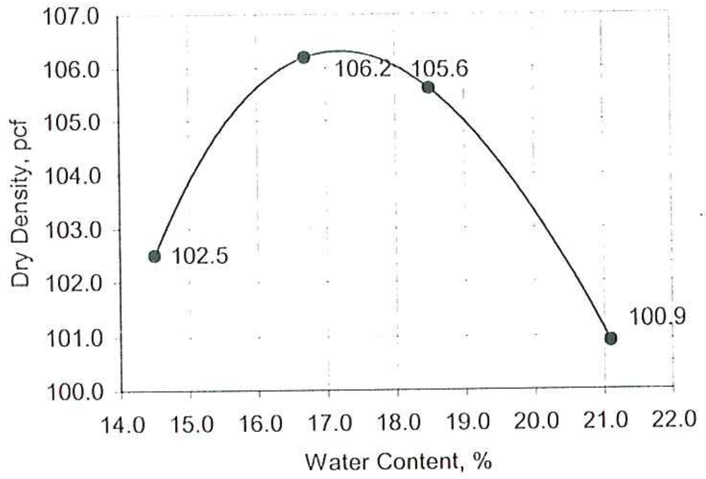
## Plasticity Index
ASTM D4318-95a

| | |
|---|---|
| Liquid Limit: | 46 |
| Plastic Limit: | 21 |
| Plasticity Index: | **25** |

## Expansion Index
ASTM D4829-95

| | |
|---|---|
| Expansion Index: | 41 |
| Expansion Potential: | Low |
| Initial Saturation, %: | 50 |

## Laboratory Maximum Density
ASTM D1557-91



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

Estimated Specific Gravity for 100% Saturation Curve =2.55

| | | | | |
|---|---|---|---|---|
| Trial # | 1 | 2 | 3 | 4 |
| Water Content: | 14.5 | 16.7 | 18.5 | 21.1 |
| Dry Density: | 102.5 | 106.2 | 105.6 | 100.9 |
| Maximum Dry Density, pcf: | | **106.5** | | |
| Optimum Water Content, %: | | **17.3** | | |

## Moisture-Density ASTM D2937-94, ASTM D2216-92

| Sample | Depth (ft) | Water Content (%) | Dry Density (pcf) | Relative Density | Sample Description |
|---|---|---|---|---|---|
| | | | | | |
| | | | | | |
| | | | | | |
| | | | | | |
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Report By: Darren Harrold

**GeoSolutions, Inc.**

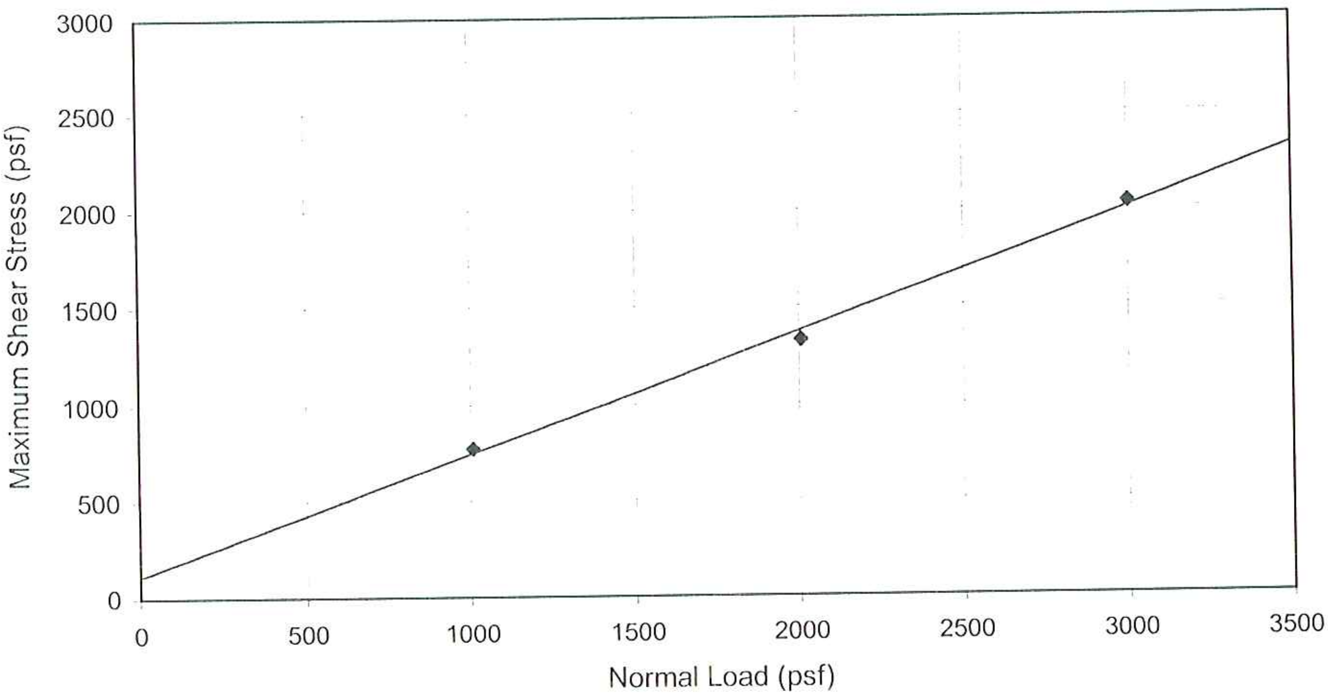
# DIRECT SHEAR TEST REPORT
## D-3080

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-1 Depth: 0.0 feet | Lab #: | 3970 |
| Location: | B-1 | Sample Date: | 2/5/2004 |
| Material: | Very Dark Gray Silty SAND | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.487 | 101.2 | 1008 | 770 | 18.3 | 113.3 | - |
| 2 | 0.388 | 126.0 | 2007 | 1324 | 18.1 | 121.4 | - |
| 3 | 0.411 | 110.1 | 3002 | 2026 | 16.8 | 119.5 | - |
| 4 | | | | | | | |
| 5 | | | | | | | |



The test specimens were in-situ samples.

| | |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 32.2 ° |
| Cohesion (In-Situ), C: | 111 psf |

Report By: Darren Harrold

**GeoSolutions, Inc.**

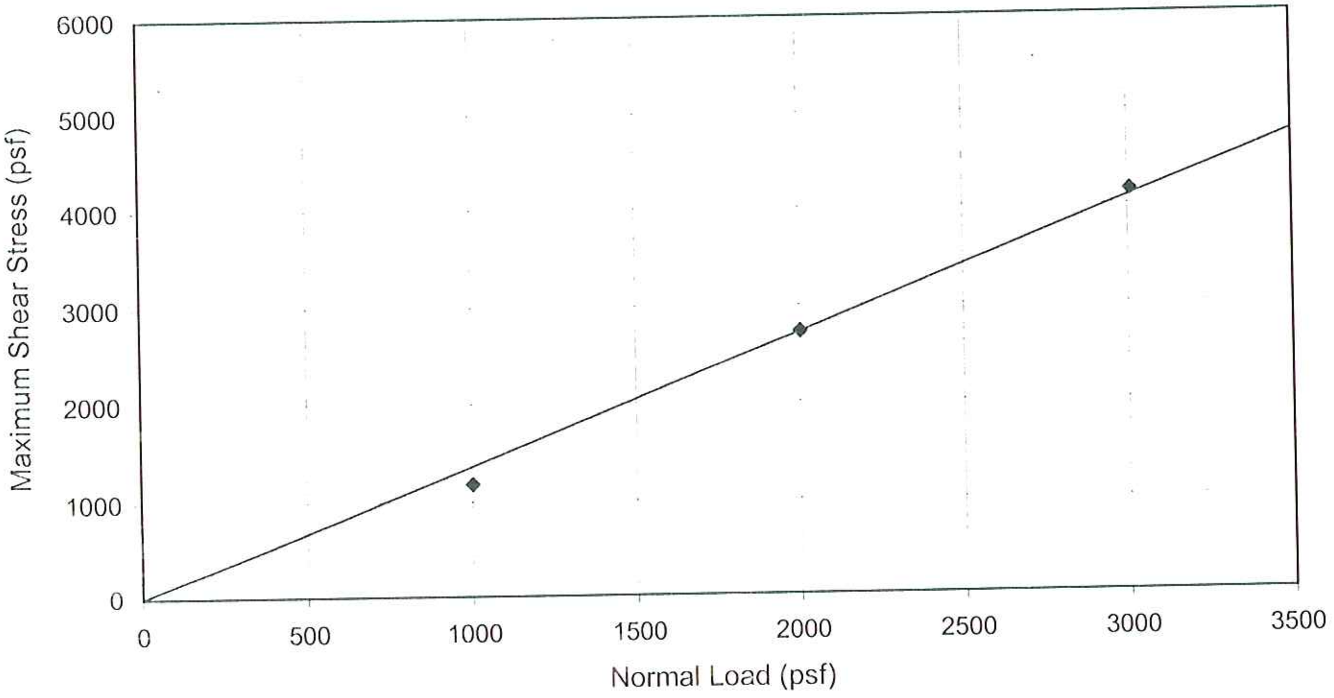
# DIRECT SHEAR TEST REPORT
## D-3080

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-1 Depth: 3.5 feet | Lab #: | 3970 |
| Location: | B-1 | Sample Date: | 2/5/2004 |
| Material: | Pale Olive Silty SAND w/ Clay | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.732 | 118.0 | 1002 | 1183 | 32.0 | 97.3 | - |
| 2 | 0.700 | 119.4 | 2004 | 2728 | 30.9 | 99.2 | - |
| 3 | 0.903 | 97.8 | 3009 | 4150 | 32.7 | 88.6 | - |
| 4 | | | | | | | |
| 5 | | | | | | | |



The test specimens were in-situ samples.

| | |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 53.7 ° |
| Cohesion (In-Situ), C: | 0.0 psf |

Report By: Darren Harrold

**GeoSolutions, Inc.**

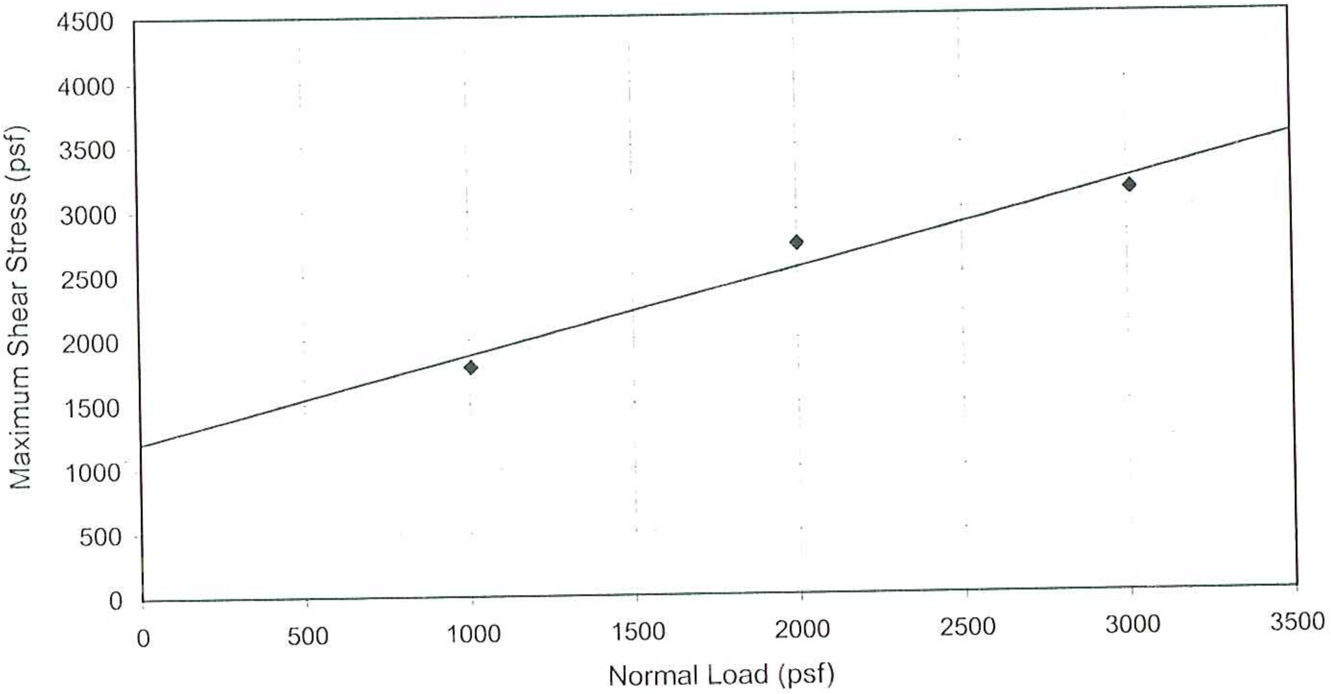
# DIRECT SHEAR TEST REPORT
## D-3080

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-1 Depth: 8.5 feet | Lab #: | 3970 |
| Location: | B-1 | Sample Date: | 2/5/2004 |
| Material: | Pale Olive Silty SAND w/ Clay | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.803 | 115.2 | 1005 | 1783 | 34.2 | 93.5 | - |
| 2 | 0.809 | 104.3 | 2001 | 2729 | 31.3 | 93.2 | - |
| 3 | 0.733 | 130.8 | 3012 | 3132 | 35.5 | 97.3 | - |
| 4 | | | | | | | |
| 5 | | | | | | | |



The test specimens were in-situ samples.

| | |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 33.9 ° |
| Cohesion (In-Situ), C: | 1201 psf |

Report By: Darren Harrold

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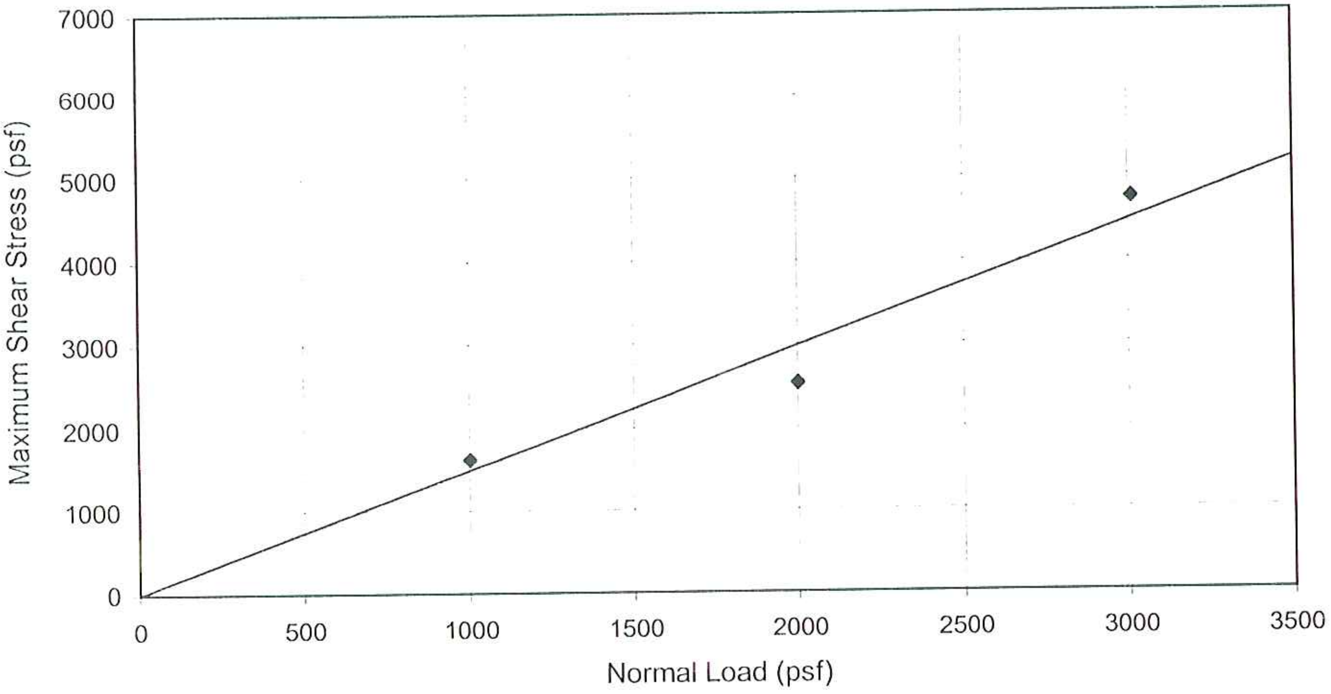
# DIRECT SHEAR TEST REPORT
## D-3080

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-1 Depth: 13.5 feet | Lab #: | 3970 |
| Location: | B-1 | Sample Date: | 2/5/2004 |
| Material: | Pale Olive Silty SAND | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.794 | 121.6 | 1002 | 1618 | 35.8 | 94.0 | - |
| 2 | 0.901 | 116.7 | 2001 | 2526 | 39.0 | 88.7 | - |
| 3 | 0.694 | 129.7 | 3012 | 4733 | 33.3 | 99.5 | - |
| 4 | | | | | | | |
| 5 | | | | | | | |



The test specimens were in-situ samples.

| | |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 56.1 ° |
| Cohesion (In-Situ), C: | 0 psf |

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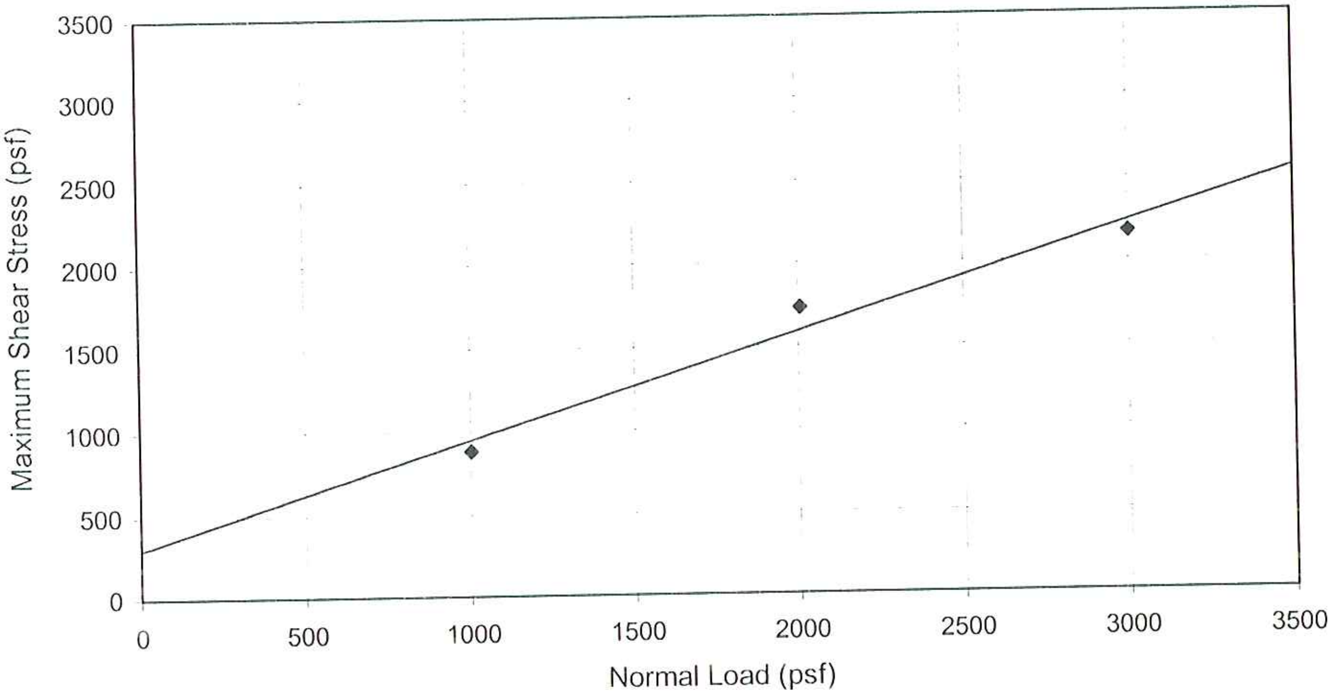
# DIRECT SHEAR TEST REPORT
## D-3080

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-2 Depth: 0.0 feet | Lab #: | 3970 |
| Location: | B-2 | Sample Date: | 2/5/2004 |
| Material: | Very Dark Gray Silty SAND | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.602 | 86.1 | 1002 | 880 | 19.2 | 105.2 | - |
| 2 | 0.556 | 87.0 | 2007 | 1727 | 17.9 | 108.3 | - |
| 3 | 0.448 | 104.8 | 3002 | 2165 | 17.4 | 116.4 | - |
| 4 | | | | | | | |
| 5 | | | | | | | |



The test specimens were in-situ samples.

| | |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 32.7 ° |
| Cohesion (In-Situ), C: | 303 psf |

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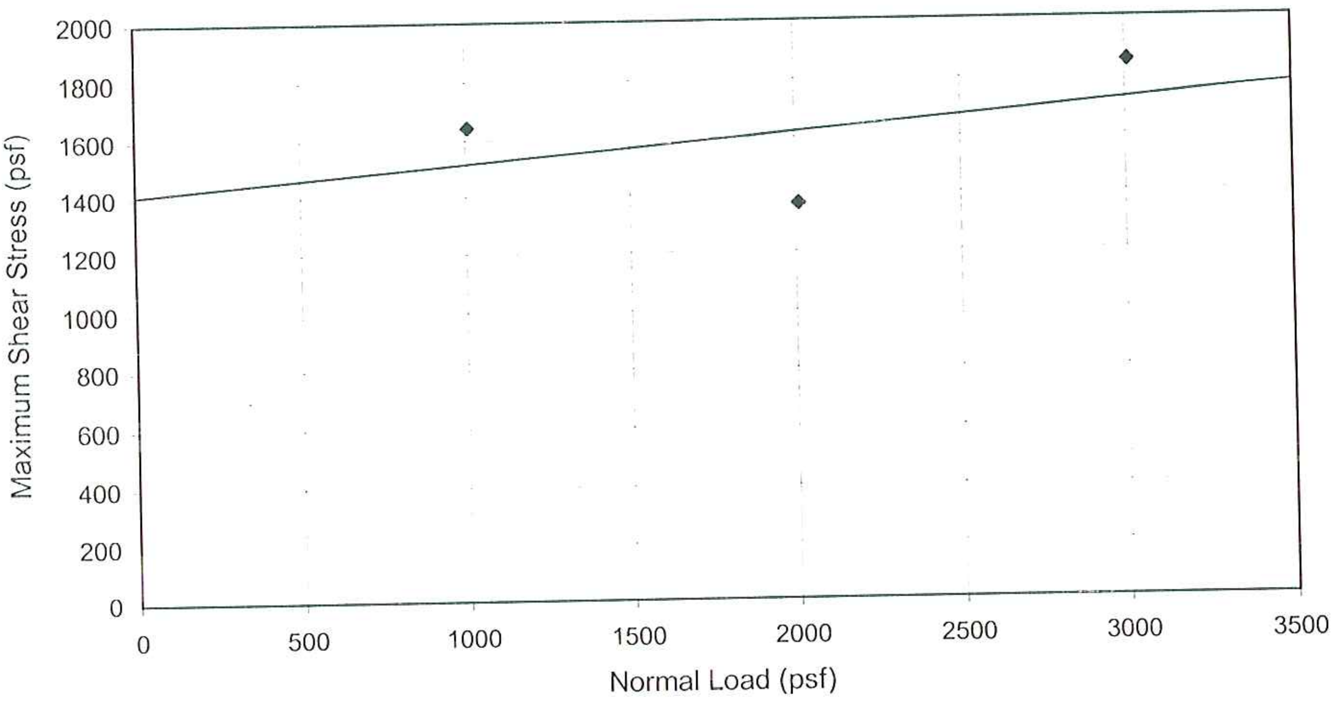
# DIRECT SHEAR TEST REPORT
## D-3080

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-2 Depth: 3.5 feet | Lab #: | 3970 |
| Location: | B-2 | Sample Date: | 2/5/2004 |
| Material: | Black CLAY | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.749 | 106.8 | 1008 | 1642 | 29.6 | 96.4 | - |
| 2 | 0.746 | 102.7 | 2010 | 1365 | 28.4 | 96.5 | - |
| 3 | 0.659 | 120.8 | 3009 | 1851 | 29.5 | 101.6 | - |
| 4 | | | | | | | |
| 5 | | | | | | | |



The test specimens were in-situ samples.

| | |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 5.9 ° |
| Cohesion (In-Situ), C: | 1410 psf |

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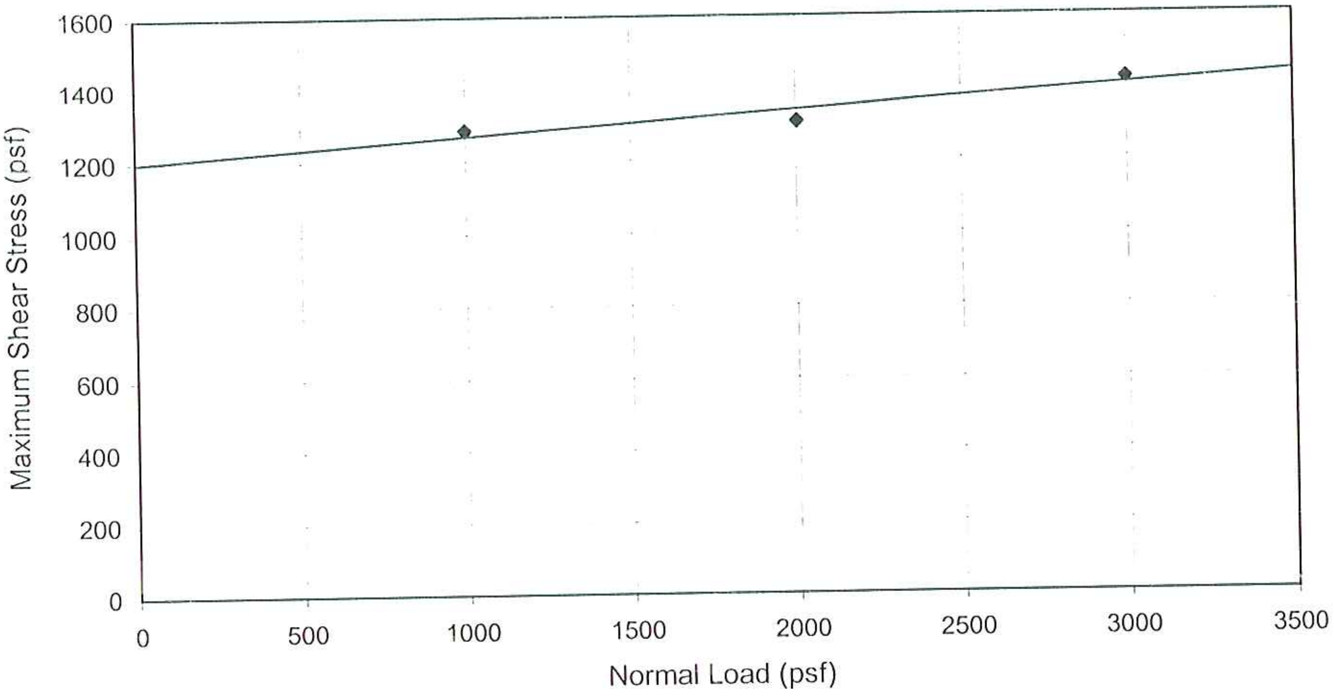
# DIRECT SHEAR TEST REPORT
## D-3080

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-2 Depth: 8.5 feet | Lab #: | 3970 |
| Location: | B-2 | Sample Date: | 2/5/2004 |
| Material: | Olive Brown Clayey SILT | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.861 | 111.5 | 999 | 1284 | 35.5 | 90.6 | - |
| 2 | 0.659 | 139.9 | 2004 | 1303 | 34.2 | 101.6 | - |
| 3 | 0.743 | 126.5 | 2999 | 1420 | 34.8 | 126.5 | - |
| 4 | | | | | | | |
| 5 | | | | | | | |



The test specimens were in-situ samples.

| | |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 3.9 ° |
| Cohesion (In-Situ), C: | 1199 psf |

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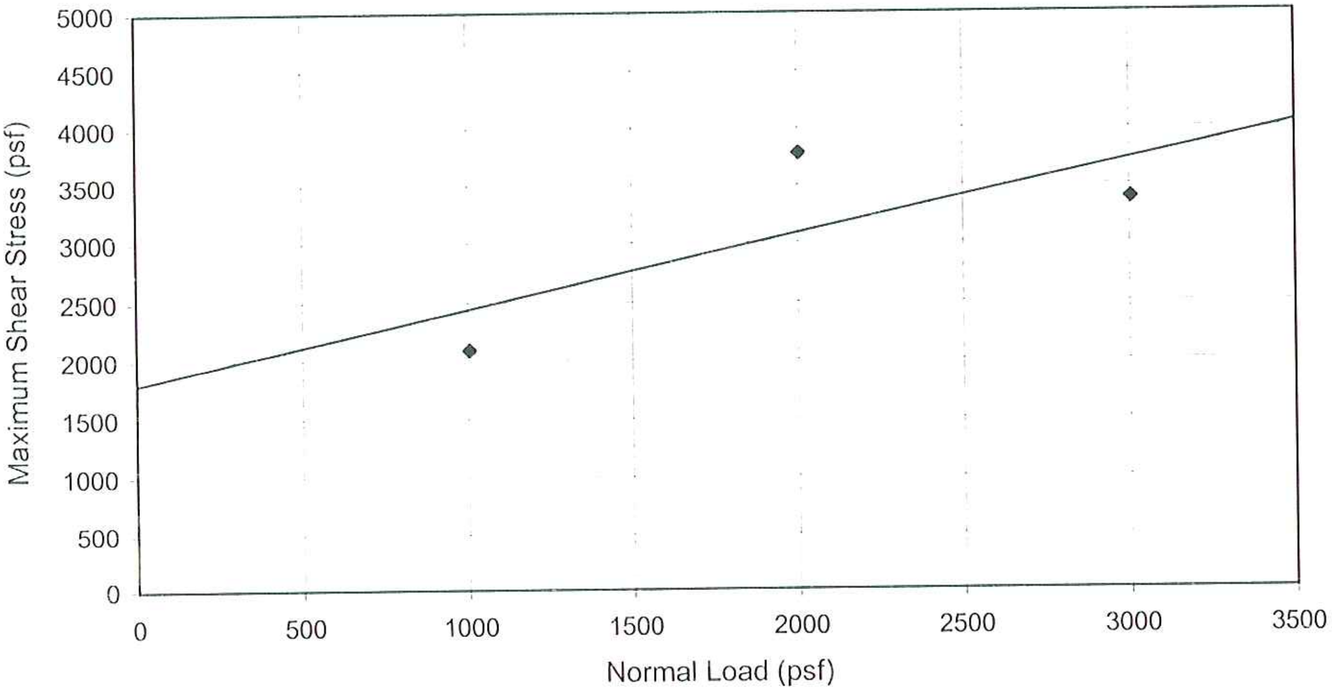
# DIRECT SHEAR TEST REPORT
## D-3080

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-2 Depth: 13.5 feet | Lab #: | 3970 |
| Location: | B-2 | Sample Date: | 2/5/2004 |
| Material: | Pale Olive Sandy SILT | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.961 | 129.8 | 1005 | 2092 | 46.2 | 85.9 | - |
| 2 | 0.869 | 123.3 | 2004 | 3779 | 39.7 | 90.2 | - |
| 3 | 0.871 | 151.9 | 3005 | 3377 | 49.0 | 90.1 | - |
| 4 | | | | | | | |
| 5 | | | | | | | |



The test specimens were in-situ samples.

| | |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 32.7 ° |
| Cohesion (In-Situ), C: | 1796 psf |

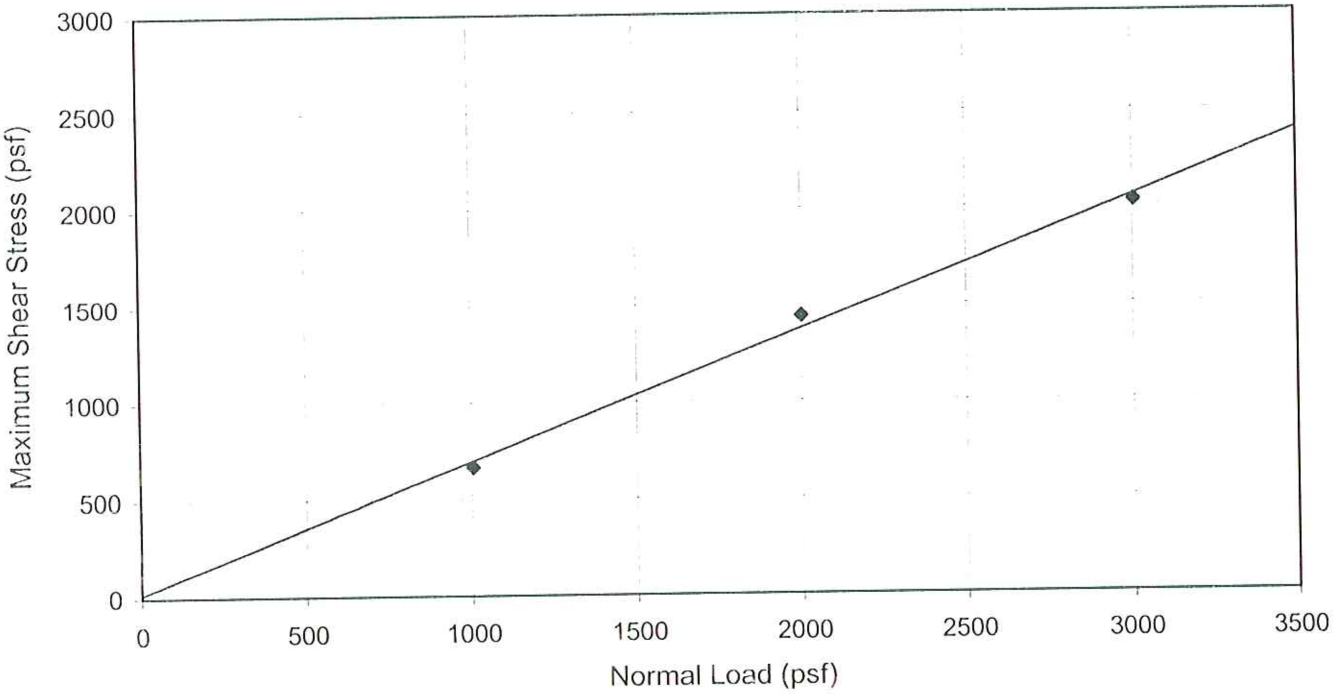
Report By: Darren Harrold

|  |  |  |
|---|---|---|
| **GeoSolutions, Inc.** | **DIRECT SHEAR TEST REPORT**<br>**D-3080** | **(805) 543-8539** |

|  |  |  |  |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-3 Depth: 0.0 feet | Lab #: | 3970 |
| Location: | B-3 | Sample Date: | 2/5/2004 |
| Material: | Black Silty SAND w/ Clay | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.661 | 110.6 | 1005 | 666 | 27.1 | 101.5 | - |
| 2 | 0.635 | 99.4 | 2001 | 1437 | 23.4 | 103.1 | - |
| 3 | 0.673 | 90.6 | 3005 | 2023 | 22.6 | 100.8 | - |
| 4 |  |  |  |  |  |  |  |
| 5 |  |  |  |  |  |  |  |



The test specimens were in-situ samples.

|  |  |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 34.1 ° |
| Cohesion (In-Situ), C: | 17.2 psf |

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**GeoSolutions, Inc.**

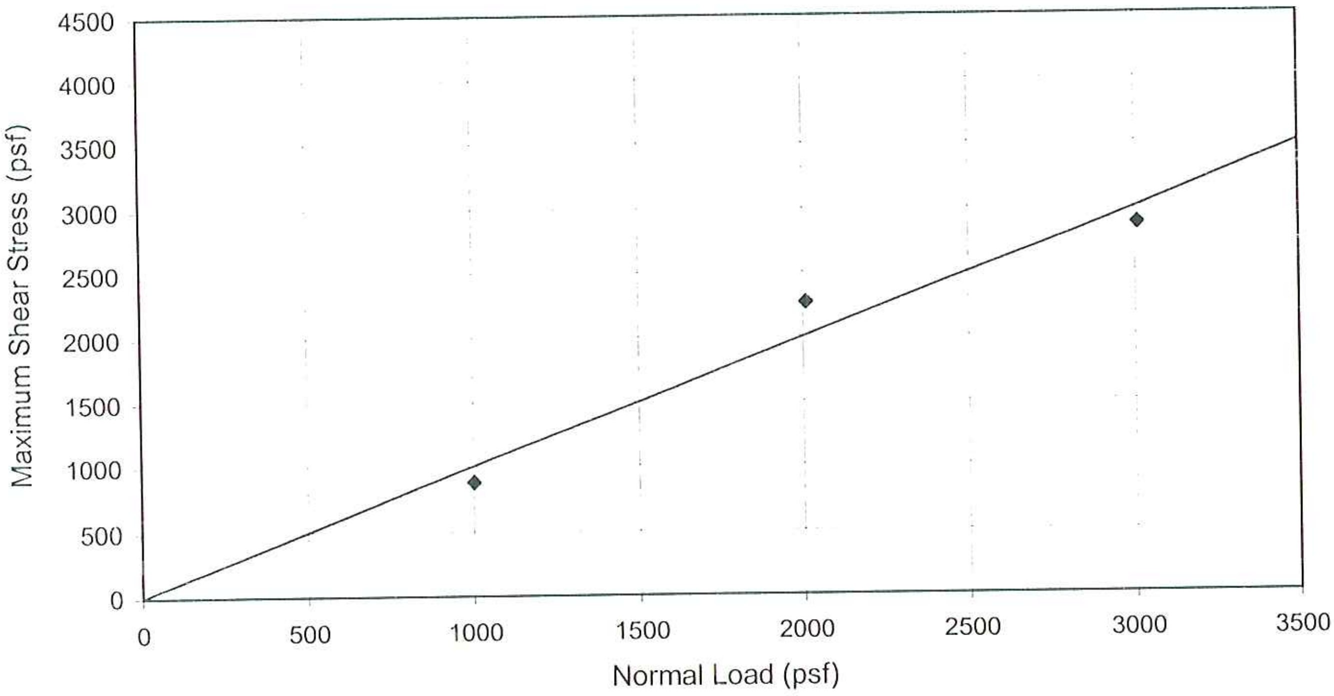
# DIRECT SHEAR TEST REPORT
## D-3080

**(805) 543-8539**

| | | | |
|---|---|---|---|
| Project: | Seaside Garden Cottages | Date Tested: | 2/17/2004 |
| Client: | Robin L. Rossi Living Trust | Project #: | SL03926-1 |
| Sample #: | B-3 Depth: 3.5 feet | Lab #: | 3970 |
| Location: | B-3 | Sample Date: | 2/5/2004 |
| Material: | Grayish Brown Sandy SILT | Sampled By: | ND |

Test Data

| Specimen Number | Void Ratio | Saturation, % | Normal Load, psf | Max Shear Stress, psf | Water Content, % | Dry Density, pcf | Relative Density, % |
|---|---|---|---|---|---|---|---|
| 1 | 0.622 | 101.5 | 1002 | 873 | 23.4 | 103.9 | - |
| 2 | 0.546 | 112.0 | 2010 | 2260 | 22.6 | 109.0 | - |
| 3 | 0.729 | 83.3 | 3009 | 2871 | 22.5 | 97.5 | - |
| 4 | | | | | | | |
| 5 | | | | | | | |



The test specimens were in-situ samples.

| | |
|---|---|
| Angle of Internal Friction (In-Situ), Phi: | 44.9 ° |
| Cohesion (In-Situ), C: | 2.0 psf |

Report By: Darren Harrold

# APPENDIX C

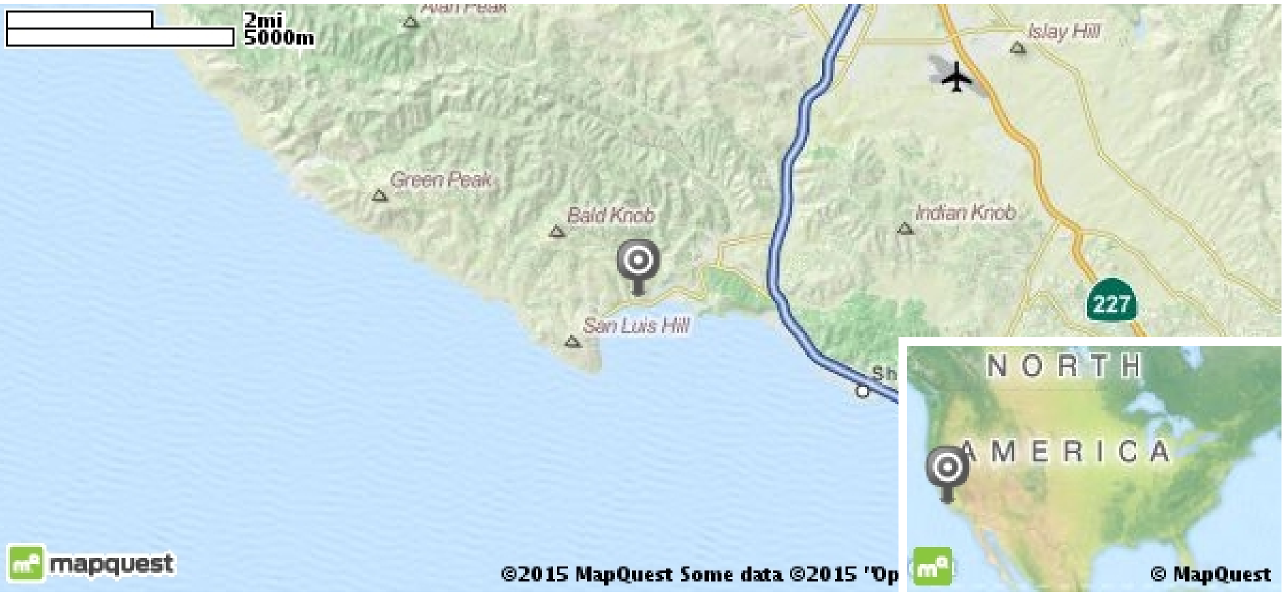
USGS Design Map Summary Report

USGS Design Map Detailed Report

# USGS Design Maps Summary Report

## User–Specified Input

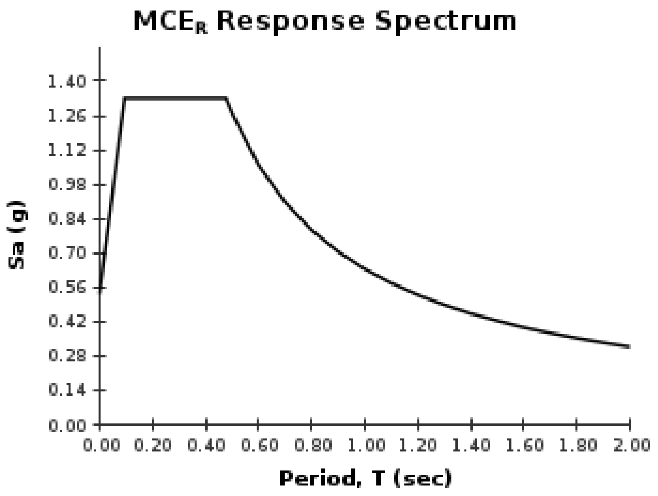
| | |
|---|---|
| **Report Title** | Seaside Garden Cottages<br>Mon November 2, 2015 17:56:36 UTC |
| **Building Code Reference Document** | ASCE 7-10 Standard<br>(which utilizes USGS hazard data available in 2008) |
| **Site Coordinates** | 35.1797°N, 120.744°W |
| **Site Soil Classification** | Site Class C – “Very Dense Soil and Soft Rock” |
| **Risk Category** | I/II/III |

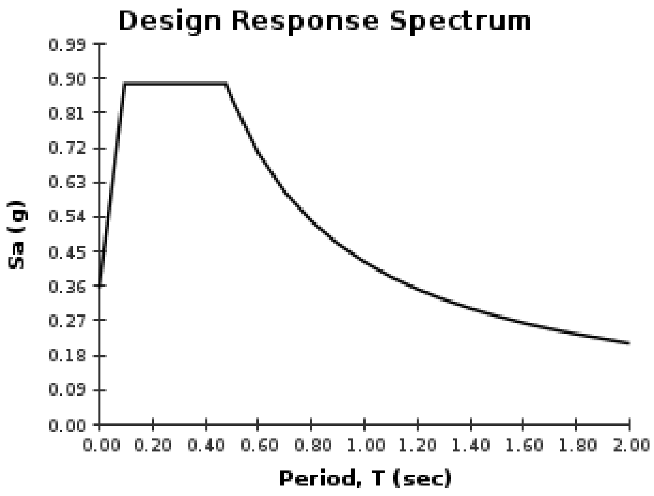


## USGS–Provided Output

$S_S =$ 1.330 g
$S_{MS} =$ 1.330 g
$S_{DS} =$ 0.886 g
$S_1 =$ 0.481 g
$S_{M1} =$ 0.634 g
$S_{D1} =$ 0.423 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.

# Design Maps Detailed Report

## ASCE 7-10 Standard (35.1797°N, 120.744°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] | $S_S$ = 1.330 g |
| **From Figure 22-2** [2] | $S_1$ = 0.481 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:<br>• Plasticity index *PI* > 20,<br>• Moisture content *w* ≥ 40%, and<br>• 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 $S_S$ = 1.330 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 $S_1$ = 0.481 g, $F_v$ = 1.319**

**Equation (11.4–1):** $S_{MS} = F_aS_S = 1.000 \text{ x } 1.330 = 1.330 \text{ g}$

---

**Equation (11.4–2):** $S_{M1} = F_vS_1 = 1.319 \text{ x } 0.481 = 0.634 \text{ g}$

---

## Section 11.4.4 — Design Spectral Acceleration Parameters

**Equation (11.4–3):** $S_{DS} = ⅔ S_{MS} = ⅔ \text{ x } 1.330 = 0.886 \text{ g}$

---

**Equation (11.4–4):** $S_{D1} = ⅔ S_{M1} = ⅔ \text{ x } 0.634 = 0.423 \text{ g}$

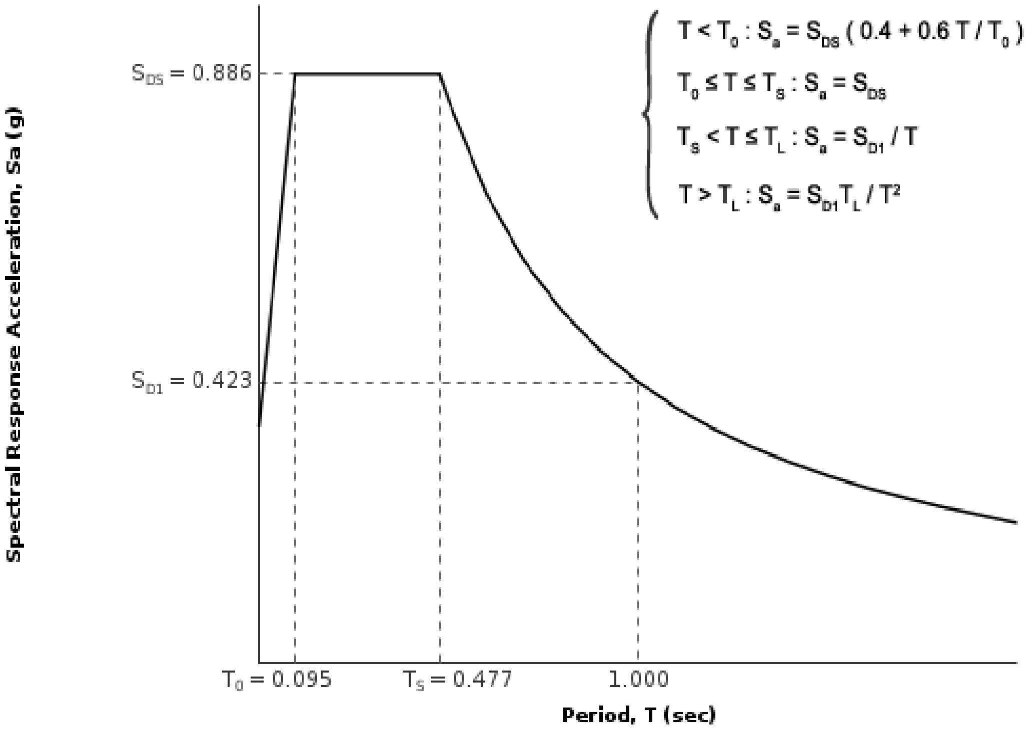
---

## Section 11.4.5 — Design Response Spectrum

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

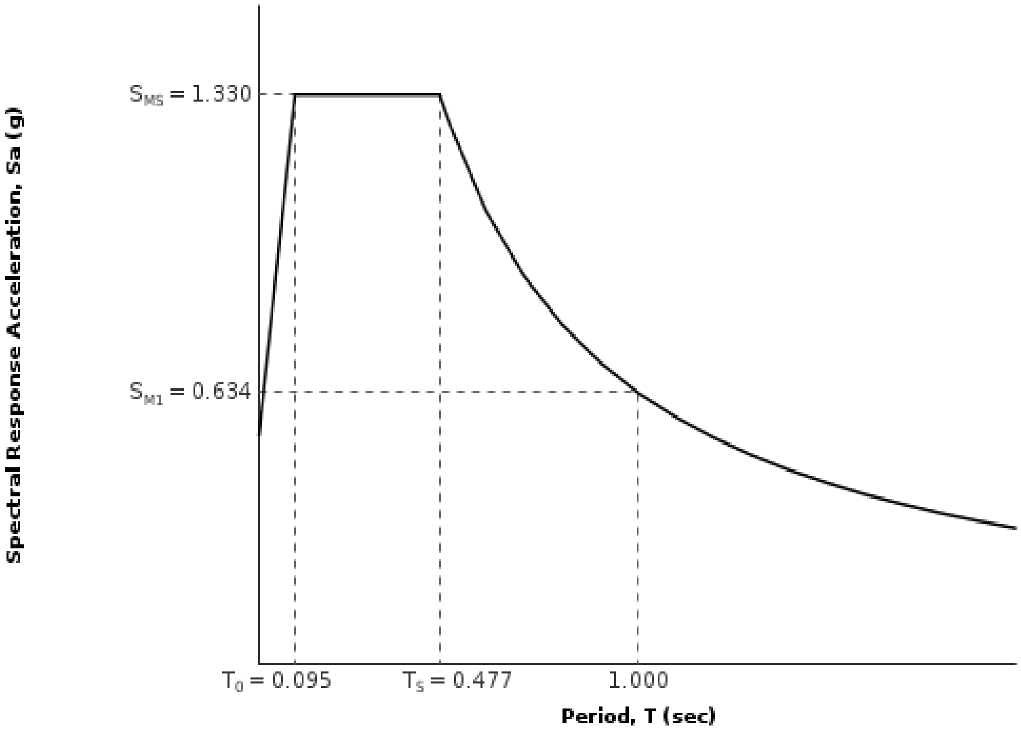
---

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 = 0.564

**Equation (11.8–1):** $PGA_M = F_{PGA}PGA = 1.000 \times 0.564 = 0.564$ 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 = C and PGA = 0.564 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] $C_{RS}$ = 0.874

**From Figure 22-18** [6] $C_{R1}$ = 0.917

## 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 $S_{DS}$ = 0.886 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 $S_{D1}$ = 0.423 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.

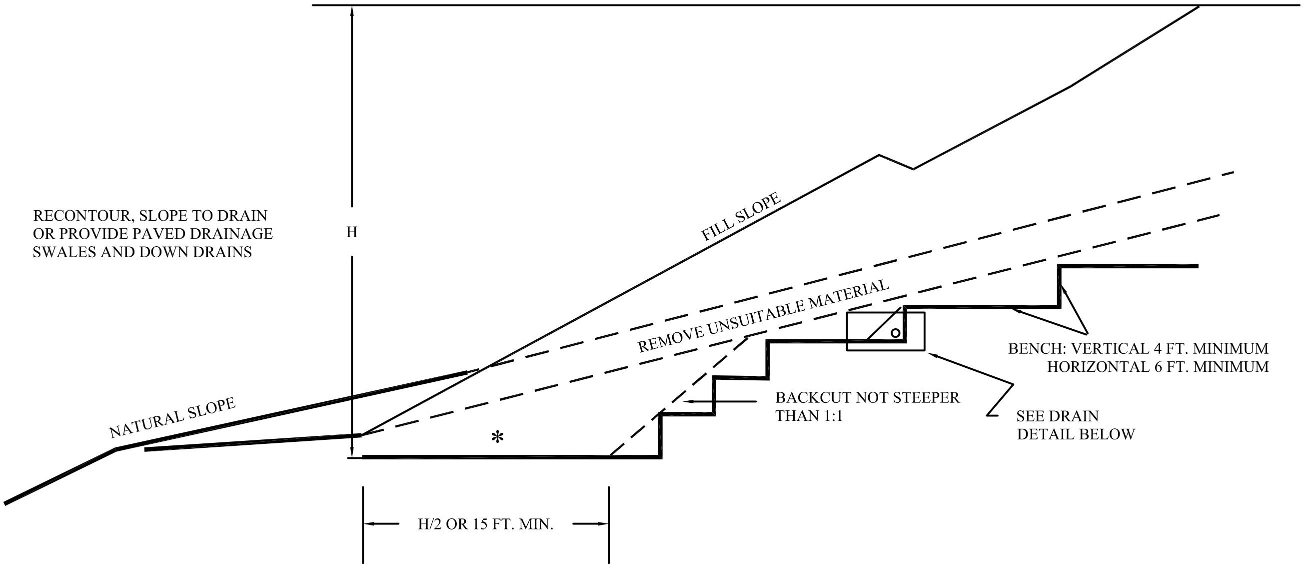
3. On-site inorganic soils, or approved import, may be used as utility trench backfill. Proper compaction of trench backfill will be necessary under and adjacent to structural fill, building foundations, concrete slabs, and vehicle pavements. In these areas, backfill should be conditioned with water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

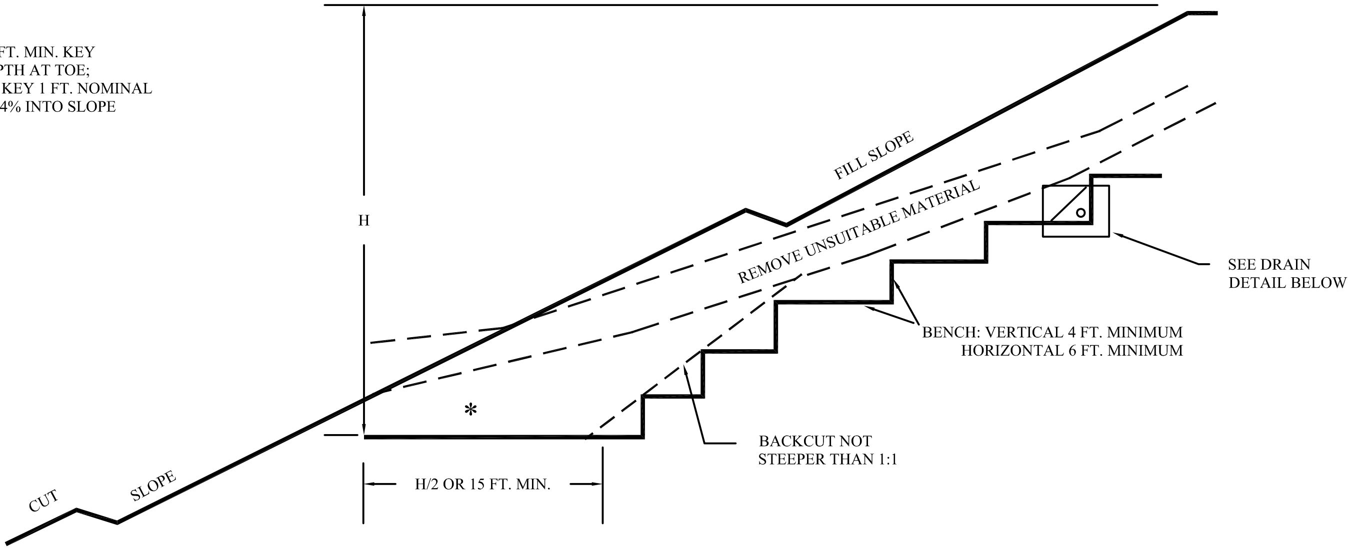
2. Soils Engineers shall submit a statement that, to the best of their knowledge, the work within their area of responsibilities is in accordance with the approved soils engineering report and applicable provisions within Chapter 18 of the 2013 CBC. .

## FILL OVER NATURAL SLOPE



## FILL OVER CUT SLOPE

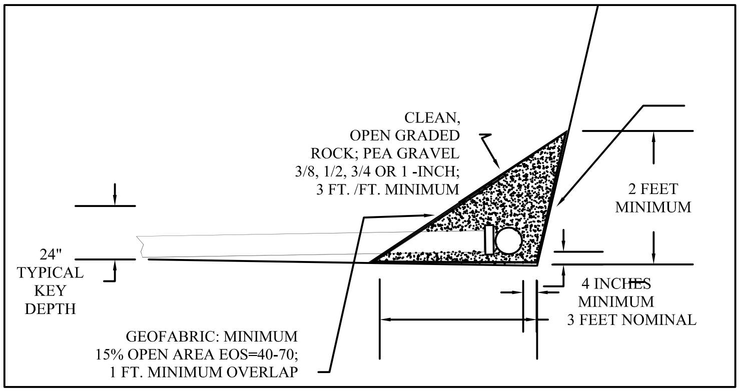
* 2 FT. MIN. KEY
DEPTH AT TOE;
TIP KEY 1 FT. NOMINAL
OR 4% INTO SLOPE



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



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# KEY AND BENCH WITH BACKDRAIN

**DETAIL**
A