SOILS ENGINEERING REPORT
GROVER BEACH LODGE
PISMO BEACH STATE PARK
WEST OF HIGHWAY 1 AND NORTH OF GRAND AVENUE
GROVER BEACH, CALIFORNIA

PROJECT SL07154-1

Prepared for

Pacifica Companies for the City of Grover Beach
1785 Hancock Street, Suite 100
San Diego, California 92110

Prepared by

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

September 14, 2010

GeoSolutions, INC.

2370 Skyway Drive, Suite 104, Santa Maria, CA 93455
(805)614-6333, (805)614-6322 fax
SBinfo@geosolutions.net

220 High Street, San Luis Obispo, CA 93401
(805)543-8539, (805)543-2171 fax
info@geosolutions.net

September 14, 2010
Project No. SL07154-1

Attn: Allison Rolfe
Pacifica Companies
1785 Hancock Street, Suite 100
San Diego, California 92110

Subject: **Soils Engineering Report**
Grover Beach Lodge, Pismo Beach State Park
West of Highway 1 and North of Grand Avenue
Grover Beach, California

Dear Ms. Rolfe:

This Soils Engineering Report has been prepared for the proposed commercial development referred to as the Grover Beach Lodge, to be located west of Highway 1 and north of Grand Avenue in the city of Grover Beach, 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 site is underlain by sandy soils that under static loading conditions may provide adequate bearing for foundations provided loads are kept relatively light. However, under seismic loadings the soils below the groundwater interface may liquefy. The result of liquefaction would be settlements on the order of 1 to 2 inches across the Site and the possibility of sand boils manifesting at the surface. The occurrence of sand boils would cause a sudden and complete loss of support under building foundations. The maximum size of the sand boils is difficult to quantify. For design purposes, sand boils could be expected to be 12 feet in diameter. This would be the most critical at building corners.

It is anticipated that a mat foundation may be used for support of the proposed structures. Where loads are relatively light, 1500 psf dead and live load, a matt foundation may be the best alternative. Foundations such as mats or post-tensioned slabs may enable a building to remain intact even with substantial movements. Based on our experience, a reinforced slab 20-30 inches thick with two reinforcement layers could be considered. All foundations are to be excavated into uniform material to limit the potential for distress of the foundation systems due to differential settlement. If cuts steeper than allowed by State of California Construction Safety Orders for "Excavations, Trenches, Earthwork" are proposed, a numerical slope stability analysis may be necessary for temporary construction slopes.

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

Sincerely,
GeoSolutions, Inc.

Patrick B. McNeill
Patrick B. McNeill, PE
Principal

REGISTERED PROFESSIONAL ENGINEER
PATRICK B McNEILL
59577
Expires 12/31/11
CIVIL
STATE OF CALIFORNIA

S:\jobs\SL07000-SL07499\SL07154-1 - Grover Beach Lodge\Engineering\SL07154-1 Transmittal Letter.doc

<u>**TABLE OF CONTENTS**</u>

LIST OF FIGURES

LIST OF TABLES

SOILS ENGINEERING REPORT
GROVER BEACH LODGE, PISMO STATE PARK
WEST OF HIGHWAY 1, NORTH OF GRAND AVENUE
GROVER BEACH, CALIFORNIA

PROJECT SL07154-1

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation for the proposed commercial development referred to as the Grover Beach Lodge, to be located west of Highway 1 and north of Grand Avenue in the city of Grover Beach, California. See Figure 1: Area Location Map.

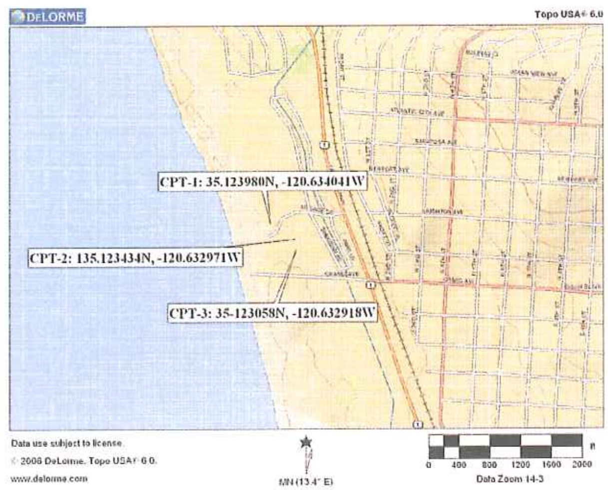


Figure 1: Area Location Map

The Pismo State Park property is approximately 1,343 acres in size with the Grover Beach Lodge portions to encompass approximately 7.5 acres. The area of property that is being considered for development is located north of Grand Avenue and west of Highway 1. The nearest intersection is located at Highway 1 and Grand Avenue. The property will hereafter be referred to as the "Site." See Figure 2: Site Location Map and Figure 3: Site Plan.

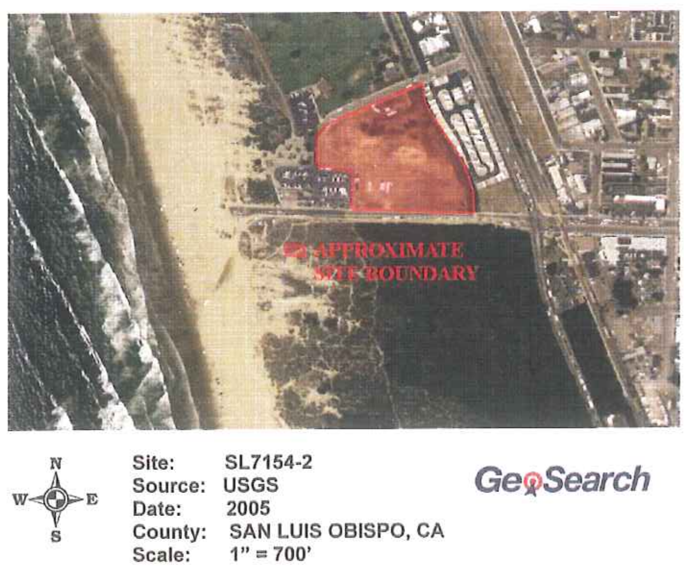


Figure 2: Site Location Map

The Site is situated on fairly level ground, with a modular home park to the east, undeveloped State Park property to the south and west, and a golf course to the north. Development on the Site includes a lodge, conference facility and associated parking. The proposed development as seen in Figure 3, Site Plan, is to include three buildings. The site is currently covered with annual grasses. Surface drainage follows the topography to the southeast to existing drainage ways.

During our investigation three exploratory CPT soundings were placed throughout the site. The loose and soft condition of the underlying soil necessitated the placement of the CPT soundings. Due to the potentially sensitive archeological nature of the Site, standard auger drilling operations were not performed.

It is anticipated the proposed commercial development will utilize slab-on-grade lower floor systems. Dead and sustained live loads are currently unknown but anticipated to be significant with maximum continuous footing and column loads estimated to be on the order of 3.5 kips per lineal foot and 200 kips, respectively.

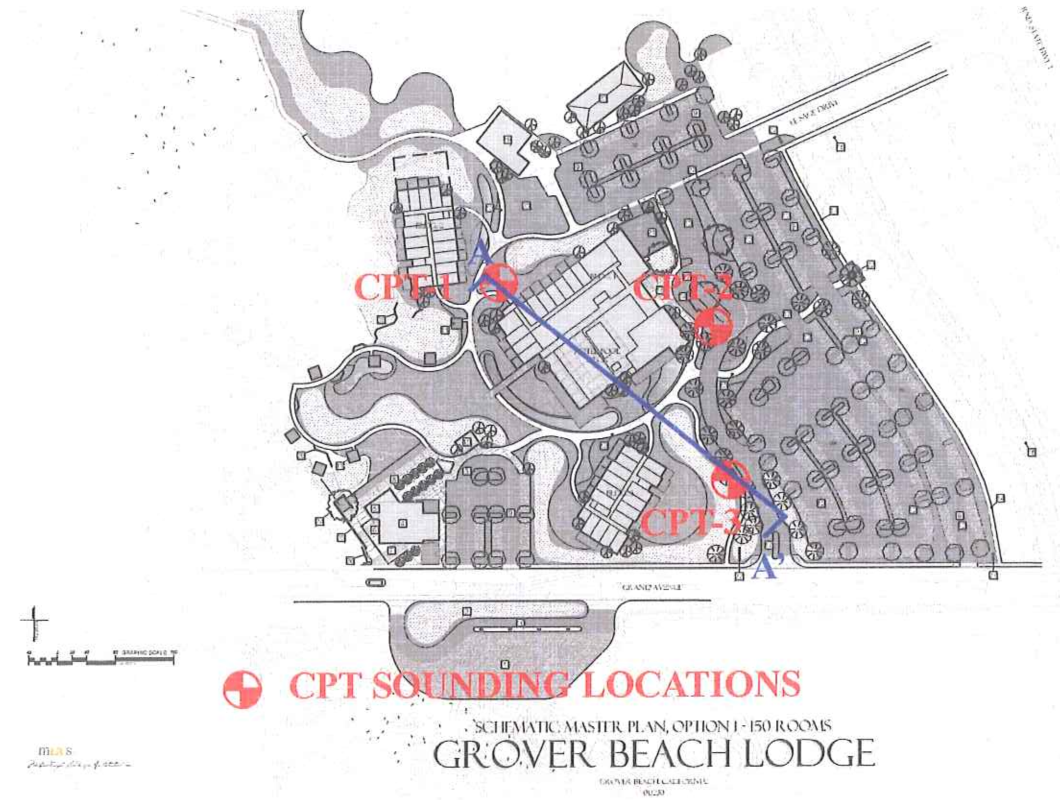


Figure 3: Site Plan

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 develop geotechnical information and design criteria. The scope of this study includes the following items:

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

2. A field study consisting of a site reconnaissance and an exploratory boring program to formulate a description of the sub-surface conditions.

3. A laboratory-testing program performed on representative soil samples collected from our field study.

4. Analysis of the data gathered during our field study.

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

3.0 FIELD AND LABORATORY INVESTIGATION

The field investigation was conducted on August 3, 2010 using a truck mounted 25 ton CPT rig. Cone Penetration Test (CPT) soundings (CPT-1, 2, & 3) were advanced to a maximum depth of 50.0 feet bgs. An electric cone was used during the CPT test. The electric cone has a 35.7-mm diameter cone-shaped tip with a 60° apex angle, a 35.7-mm diameter by 133.7-mm long cylindrical sleeve, and a pore pressure transducer in the sounding location. The CPT soundings were terminated at the recorded depths due to tip resistance in excesses of 700 tsf, which is approximately the upper limitation of the equipment.

The materials at the Site generally consisted of interbedded layers of sands, silty sands, clayey silt to sandy silt. The CPT soundings were advanced to provide a near continuous soil behavior profile and to better characterize the Site. The CPT soundings were advanced in three locations based on the proposed building plans and site underground utility constraints in the approximate locations indicated on the Site Plan. Site lithology is estimated from the three CPT soundings and is shown below on Figure 4: Soils Profile A-A'.

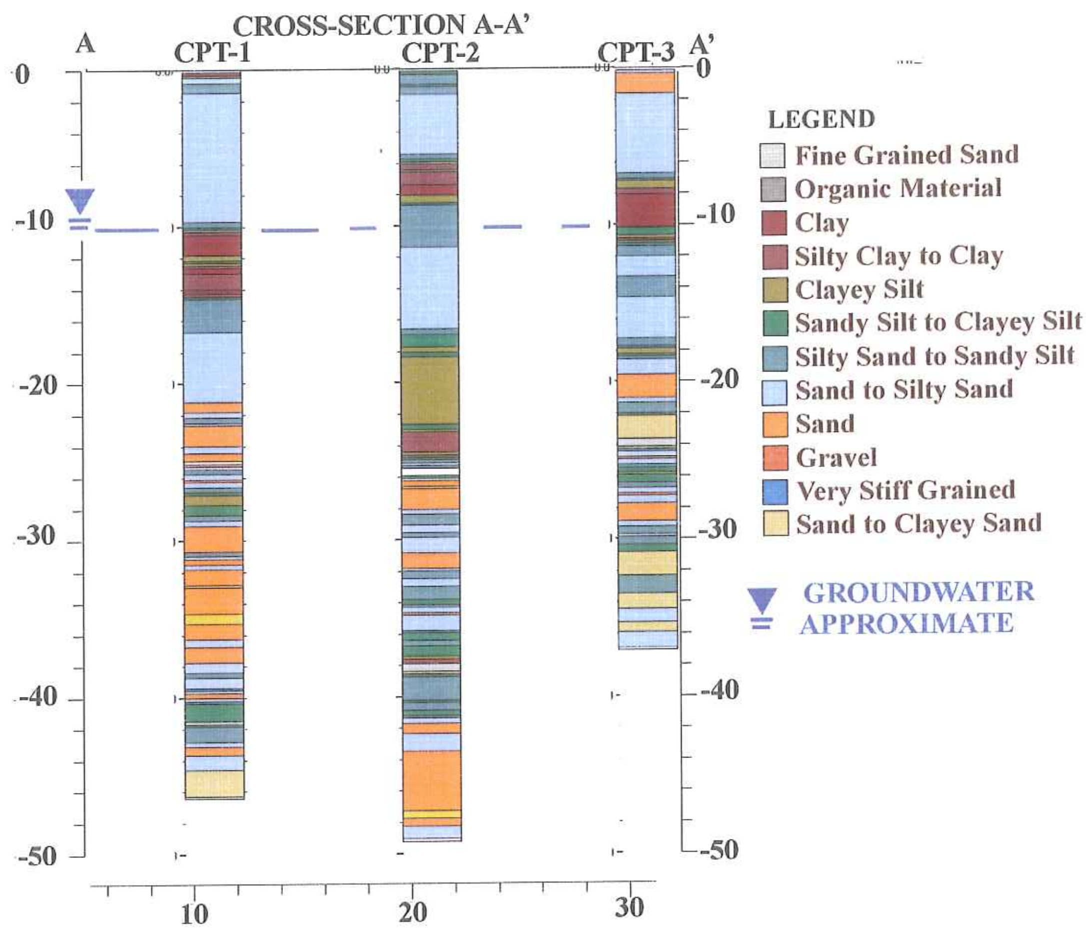


Figure 4: Soils Profile A-A'

Groundwater was encountered in the three soundings and was first encountered at a depth of 5.0 to 10.0 feet bgs. Water was identified in the CPT soundings from pore pressure readings. However, GeoSolutions, Inc. recommends that a groundwater monitoring well be installed prior to construction activities and be surveyed to better establish groundwater levels. This will be particularly helpful during underground utility construction. No soils samples were obtain during this portion of the investigation.

4.0 SEISMIC DESIGN CONSIDERATIONS

4.1 Seismic Hazard Analysis

1. According to section 1613 of the 2007 CBC (CBSC, 2007), all structures and portions of structures should be designed to resist the effects of seismic loadings caused by earthquake ground motions in accordance with the *Minimum Design Loads for Buildings and Other Structures* (ASCE7) (ASCE, 2006). ASCE7 considers the most severe earthquake ground motion to be the ground motion caused by the Maximum Considered Earthquake (MCE) (ASCE, 2006), which is defined in Section 1613 of the 2007 CBC to be short period S_{MS} and 1-second period S_{M1}, spectral response accelerations.

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

3. As per section 1613.5.5 of the 2007 CBC (CBSC, 2007), the Site soil profile classification is determined by the average soil properties in the upper 100 feet of the Site profile. Based on the $(N_1)_{60}$ values calculated for the in-situ tests performed during the field investigation, the Site was defined as Site Class D, per Table 1613.5.2 of the 2007 CBC (CBSC, 2007).

4. According to section 11.2 of ASCE7 (ASCE, 2006) and section 1613 of the 2007 CBC (CBSC, 2007), 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}=0.531 and S_{DS}=0.981,** 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 35.123434 degrees north latitude and approximately 120.632971 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 2007 CBC (CBSC, 2007) and sections 11.4.3 and 11.4.4 of ASCE7 (ASCE, 2006) are dependent upon several factors, which include site soil profile characteristics and the locations and characteristics of faults near the Site. As described in section 4.1 of this report, the Site soil profile classification was determined to be Site Class D. This Site soil profile classification and the latitude and longitude coordinates for the Site were used to determine the structural building design parameters.

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

is presented in tabular form in Table 1: 2007 California Building Code, Chapter 16, Structural Design Parameters. Analysis of the Design Spectral Response Acceleration Parameters for the Site and of the Occupancy Category for the proposed structure assign to this project a **Seismic Design Category of D** per Tables 1613.3.5.6(1) and 1613.3.5.6(2) of the 2007 CBC (CBSC, 2007).

Table 1: 2007 California Building Code, Chapter 16, Structural Design Parameters

Site Class - Soil Profile Type	D – Stiff Soil
Mapped Spectral Response Accelerations and Site Coefficients	$S_S = 1.472$, $S_1 = 0.531$ $F_a = 1.000$, $F_v = 1.50$
Adjusted Maximum Considered Earthquake Spectral Response Accelerations	$S_{MS} = 1.472$ $S_{M1} = 0.797$
Design Spectral Response Acceleration Parameters	$S_{DS} = 0.981$ $S_{D1} = 0.531$
Occupancy Category (from Table 1604.5, 2007 CBC)	II
Seismic Design Category – Short Period Accel. (from Table 1613.5.6(1), 2007 CBC)	D
Seismic Design Category – Long Period Accel. (from Table 1613.5.6(2), 2007 CBC)	D

4.3 Design Response Spectra – 2007 CBC

According to section 11.4.5 of ASCE7 (ASCE, 2006), a design response spectrum for a site may be required in order to design structures to resist lateral forces caused by ground motions at the Site. The design spectral response acceleration parameters, listed in Table 1: 2007 California Building Code, Chapter 16, Structural Design Parameters, are used to produce the design response spectrum. The Seismic Hazard Curves and Uniform Hazard Response Spectra computer program (USGS, 2007) was used to construct constructed a design response spectrum for the Site, which is shown in Figure 5: Design Response Spectra – 2007 CBC.

4.4 Liquefaction Potential

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

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

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

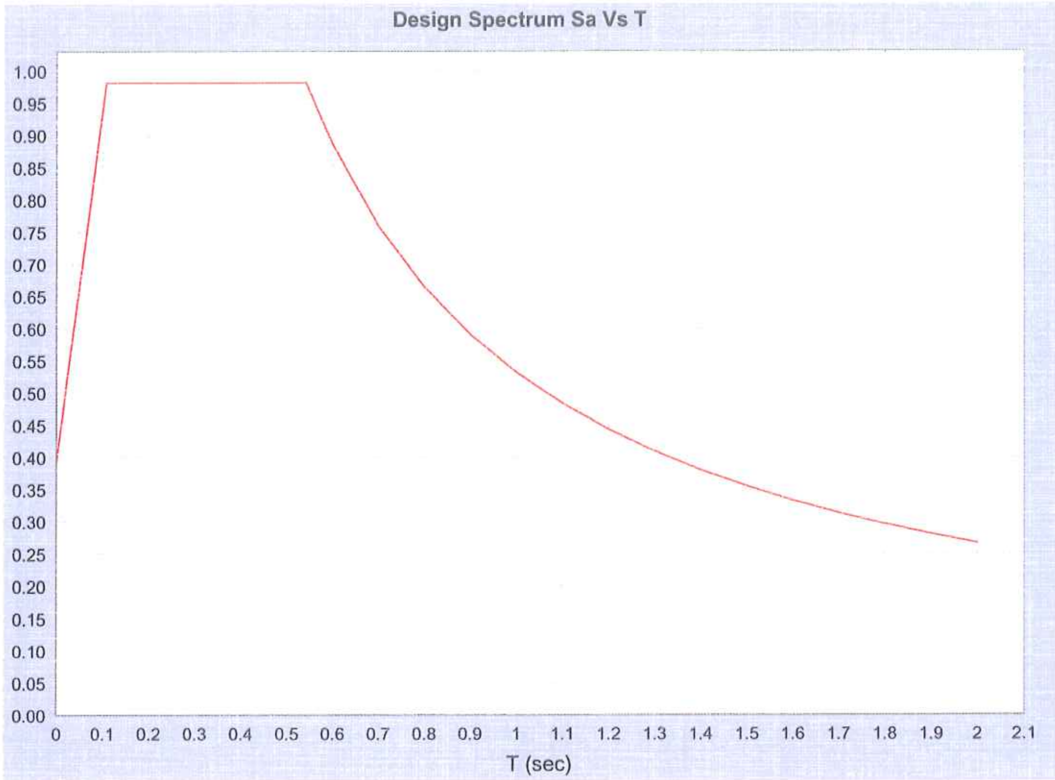


Figure 5: Design Response Spectra – 2007 CBC

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

5 Based on the layered nature of the site soils, the presence of loose sands, the relative density of the in-situ soils, the depth to groundwater, and the expected ground acceleration, the potential for seismic liquefaction of Site soils appears to be high. Liquefaction was determined to likely occur in discrete layers separated by non-liquefiable layers between the depths of approximately 5.0 feet bgs and 40.0 feet bgs.

5.0 GENERAL SOIL-FOUNDATION DISCUSSION

The site is underlain by sandy soils that under static loading conditions may provide adequate bearing for foundations provided loads are kept relatively light. However, under seismic loadings the soils below the groundwater interface may liquefy. The result of liquefaction would be settlements on the order of 1 to 2 inches across the Site and the possibility of sand boils manifesting at the surface. The occurrence of sand boils would cause a sudden and complete loss of support under building foundations. The maximum size of the sand boils is difficult to quantify. For design purposes, sand boils could be expected to be 12 feet in diameter. This would be the most critical at building corners.

A mat foundation may be used for support of the proposed structures. Where loads are relatively light, 1500 psf dead load and live load, a matt foundation may be the best alternative. Foundations such as mats or post-tensioned slabs may enable a building to remain intact even with substantial movements. Based on our experience, a reinforced slab 20-30 inches thick with two reinforcement layers could be considered.

To mitigate the liquefaction potential, the soil can undergo vibro-replacement (stone column) in-situ soil densification; or groups of driven pile deep foundations known as compaction piles, embedded into dense material. Stone columns may be required to a depth of 40 feet below land surface, extending 15 feet beyond the perimeter of the buildings. However, the data obtained during our field investigation indicates that the incremental improvement to the site may not justify this option.

Conventional driven piles may be required where loads exceed what a matt foundation can distribute over the foundation system. For the driven piles, the depth to dense material was identified from the cone penetration tests and is given as 40 feet bgs. Driven piles would help mitigate the effects of liquefaction and or excessive static and dynamic settlement.

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 and soft soils in the upper 40 feet of the soil profile.

2. The potential for differential settlement when foundations are supported on poorly consolidated soils. Therefore, it is important that all of the foundations are designed to consider the effects of static and dynamic settlement.

3. The potential for liquefaction. Several layers of soil were identified as potentially liquefiable. The low densities encountered, along with the low fines content of the soil and saturated conditions indicate these layers may potentially be liquefiable, manifesting at the surface as dynamic settlements and sand boils.

6.1 Preparation of Building Pads

1. It is anticipated that site grading will be limited to the development of a stable engineered graded pad to construct concrete foundations.

2. The native material should be over-excavated at least 12 inches below existing grade or the bottom of slabs, whichever is greater. The resulting surface should then be moisture conditioned to produce a water-content of at least 1 to 2 percent above optimum value and compacted to a minimum of 90 percent of maximum dry density. The removed material should then be replaced as engineered fill. Refer to **Appendix C** for more details on fill placement.

6.2 Mat Foundation

1. A mat foundation may be used for support of the proposed structures. Foundations such as mats or post-tensioned slabs may enable a building to remain intact even with substantial movements. Based on our experience, a reinforced slab 20-30 inches thick could be considered.

2. A mat foundation should be designed to resist a loss of support 12 feet in diameter. This will be the most critical at building corners. Minimum reinforcing should be as directed by the project Structural Engineer. Foundation excavations should be observed and approved by a representative of this firm prior to the placement of reinforcing steel and/or concrete.

3. A modulus of sub-grade reaction (k_s) of 50 pci may be used.

4. Static loading settlement on the order of 1 inch and a differential settlement from liquefaction of 1 to 2 inches across the Site are anticipated.

5. Allowable dead plus live load bearing pressure of 1500 psf (1250 DL, 250 LL) may be used for design of mat foundations.

6. 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 **0.40** may be utilized for sliding resistance at the base of footings.

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

6.3 Driven Piles

1. Groups of driven piles may be used to densify the subsurface soil and support the proposed structures. It is recommended that following the layout of the proposed structures, borings be drilled at various locations within the foundation footprint to estimate the required length of the piles. For preliminary purposes, a 5-foot minimum embedment depth is assumed below a depth of 40 feet bgs.

2. Precast prestressed concrete piles 1-foot (305mm) in diameter, or structural steel pipe piles, either Caltrans Class 400 or 900 should be considered. Steel piles are more easily driven through hard layers and are more easily spliced for varying penetration depths than either concrete or timber piles. Caltrans Class 400 piles are 14-inch diameter piles with an allowable capacity of 45 tons. Caltrans 900 piles are 16-inch diameter with an allowable capacity of 100 tons.

3. Steel piles should be driven to refusal. A specification for refusal can be developed for a selected section and driving hammer energy.

4. Continuous grade beams may be used to transfer loads to the driven piles.

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

6.4 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 8 inches, moisture conditioned to near optimum moisture content and compacted to a minimum relative density of 90 percent (ASTM D1557-07). The over-excavated soil should then be moisture conditioned to produce a water-content of at least 1 to 2 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. 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.

6.5 Foundation Settlement

1. Static loading settlements **1 to 1.5 inches** are anticipated in addition to any seismically induced dynamic settlement. Settlement was calculated using Schmertmann method and utilizing the q_c data obtained during our field investigation.

2. Seismic settlements due to liquefaction were calculated in association with the liquefaction analysis using the computer program Liquefy Pro and the CPT data obtained during our field investigation; see **Appendix D**, Liquefaction Analysis Sheets.

6.6 Slab-On-Grade Construction

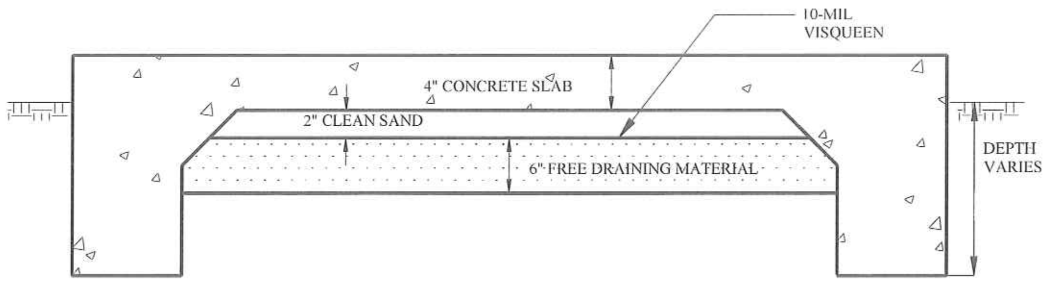
1. Concrete slabs-on-grade and flatwork should not be placed directly on unprepared native materials. Preparation of sub-grade to receive concrete slabs-on-grade and flatwork should be processed as discussed in the preceding sections of this report. Concrete slabs should be placed only over sub-grade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

2. Where concrete slabs-on-grade are to be constructed, the slabs should be underlain by a minimum of 6 inches of clean free-draining material, such as a coarse aggregate mix to serve as a cushion and a capillary break. Where moisture susceptible storage or floor coverings are anticipated, a 10-mil Visqueen-type membrane should be placed between the free-draining material and the slab to minimize moisture condensation under the floor covering. See Figure 6: Sub-Slab Detail. It is suggested that a 2-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 8 inches. The sand should be lightly moistened prior to placing concrete.

3. Concrete slabs-on-grade should be designed by the Structural Engineer.

4. 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.

5. 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.



6.7 Retaining Walls

1. The Site is flat and any walls are anticipated to retain a minimal depth of native material. Any retaining walls proposed for this project are anticipated to be site walls, and not part of any structural foundation.

2. Due to the presence of loose surface material and the potential for seismic liquefaction, it is anticipated that graded engineered fill pads will be constructed for Site retaining walls with footings excavated into engineered fill.

3. For construction of an engineered fill pad for site retaining walls that are not part of the structures, the in-situ material should be over-excavated at least 24 inches below existing grade. The limits of over-excavation should extend a minimum of 2 feet beyond the retaining wall footing, or up to the property line. The exposed surface should then be scarified an additional 12 inches, moisture conditioned to near optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-02$^{\varepsilon 1}$). The removed native material should then be replaced as engineered fill and compacted to a minimum relative density of 90 percent (ASTM D1557-07). Refer to **Appendix C** for more details on fill placement.

4. Retaining walls should be designed to resist lateral pressures from adjacent soils and surcharge loads applied behind the walls. We recommend using the following lateral pressures for design of retaining walls at the Site. See Table 2: Retaining Wall Design Parameters and Figure 7: Retaining Wall Detail.

Table 2: Retaining Wall Design Parameters

Lateral Pressure and Condition	Equivalent Fluid Pressure, pcf
Active Case, Engineered Fill or Native, drained (K_a)	35
At-Rest Case, Engineered Fill or Native, drained (K_o)	55
Passive Case, Engineered Fill (K_p)	380

5. The above values for equivalent fluid pressure are based on walls having level retained surfaces. Walls having a retained surface that slopes upward from the top of the wall should be designed for an additional equivalent fluid pressure of **1 pcf** for the active case and **1.5 pcf** for the at-rest case, for every **two degrees** of slope inclination.

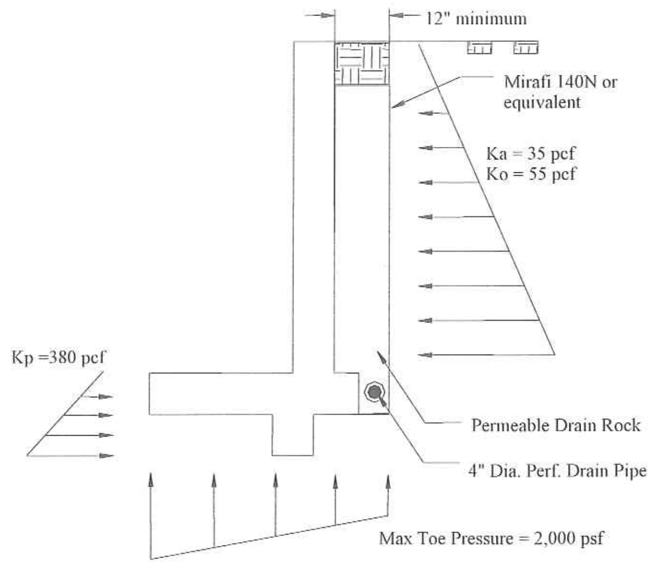


Figure 7: Retaining Wall Detail

6. Retaining wall foundations should be founded a minimum of 18 inches below lowest adjacent grade in engineered fill. A coefficient of friction of **0.40** may be used between engineered fill and concrete footings. Project designers may use a maximum toe pressure of **2,000 psf** for engineered fill.

7. In addition to the lateral soil pressure given above, the retaining walls 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 wall, supplemental pressures will be induced and should be taken into account through design.

8. The recommended pressures 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 filter material be placed behind all proposed walls. The blanket of 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. If the retaining wall is part of a structural foundation, the drainpipe must be placed below finished slab grade elevation.

9. 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. 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 grade elevation.

10. 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.

11. 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 above soil 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.

12. 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.

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

14. Seismic active lateral earth pressure values were determined using the Pseudostatic Method and the Design a_{max}. See section 4.1 for a description of the analysis used to determine the Design a_{max}. The seismic at-rest lateral earth pressure value was determined by multiplying the seismic active lateral earth pressure value by approximately 1.5. The dynamic increment in lateral earth pressure due to earthquakes should be considered during the design of retaining walls at the Site. Retaining walls should be designed to resist an additional lateral soil pressure of 25 pcf equivalent fluid pressure for unrestrained walls and 40 pcf equivalent fluid pressure for restrained walls. For earthquake conditions, the pressure resultant force should be assumed to act a distance of $^2/_3$H above the base of the retaining wall, where H is the height of the retaining wall.

15. 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.

16. 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.

6.8 Pavement Design

1. As indicated previously, 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 D1557-02$^{\varepsilon 1}$ 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.

2. All paving construction and materials used should conform to applicable sections of the latest edition of the State of California Department of Transportation Standard Specifications.

3. Aggregate bases and sub-bases should be compacted to a minimum relative density of 95 percent based on the ASTM D1557-02$^{\varepsilon 1}$ test method at slightly above optimum moisture content.

4. Preliminary recommended pavement sections were determined based on various traffic indexes (assumed), an R-Value of 50 (assumed based on soils encountered during field investigation), and specifications of the County of San Luis Obispo. Our analysis was based on the Caltrans Highway Design Manual and our recommended structural sections are presented in Table 3. It is recommended that if pavement is placed prior to construction of buildings that the placement of pavement be phased. Final design section will be determined after preliminary grading is finished and the California Test Method No. 301-F test is performed as a representative sample encountered at the Site.

Table 3: Recommended Pavement Design Thickness

Traffic Index	Street Section Thickness in Inches	
	AC*	AB**
5.0	2.00	4.0
5.5	2.00	6.0
6.0	3.00	6.0
6.5	3.25	6.0
7.0	3.5	6.0

*AC = Asphaltic Concrete meeting Caltrans Specification for Class 2 Asphalt Concrete
**AB = Aggregate Base meeting Caltrans Specification for Class 2 aggregate base (R-Value = 78 Min)

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. It is assumed that GeoSolutions, Inc. will be retained to perform the following services:

1. Consultation during plan development.

2. Plan review of grading and foundation documents prior to construction.

3. Construction inspections and testing as required including, but not limited to, stripping, grading, over-excavating, backfill placement, imported materials, Site densification, foundation excavation observations and compaction.

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.

S:\jobs\SL07000-SL07499\SL07154-1 - Grover Beach Lodge\Engineering\SL07154-1 Grover Beach Lodge SER.doc

REFERENCES

REFERENCES

2001 California Building Code. California Code of Regulations Title 24, Part 2, Volume 2. Whittier, California: International Conference of Building Officials, 2002.

Blake, Thomas F., "EQFAULT – A computer program for the deterministic estimation of peak acceleration from digitized faults," Windows Version 3.0, 1999.

Blake, Thomas F., "FRISKSP – A computer program for the probabilistic estimation of peak acceleration and uniform hazard spectra using 3-D faults as earthquake sources," Windows Version 4.0, 2000.

Bowles, Joseph E. *Foundation Analysis and Design.* 5th ed. New York: McGraw-Hill, 1996.

California Code of Regulations. 2001 Edition. Title 14. Division 2: Department of Conservation. Chapter 8: Mining and Geology. Article 10: Seismic Hazards Mapping. http://www.consrv.ca.gov/SMGB/Regulations/regulations.htm

California Department of Conservation, Division of Mines and Geology. *Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada.* Whittier, California: International Conference of Building Officials, 1998.

California Department of Conservation, Division of Mines and Geology. *Special Publication 117: Guidelines for Evaluating and Mitigating Seismic Hazards in California.* Adopted 1997. http://gmw.consrv.ca.gov/shmp/webdocs/sp117.pdf

Day, Robert W. *Geotechnical and Foundation Engineering: Design and Construction.* New York: McGraw-Hill, 1999.

Holzer, T.L., Noce, T.E., Bennett, M.J., Di Alessandro, C., Boatwright, J, Tinsley, J.C., Sell, R.W., and Rosenberg, L.I., 2004, "Liquefaction-Inducted Lateral Spreading in Oceano, California, During the 2003 San Simeon Earthquake," U. S. Geological Survey Open File Report 2004-1269.

Jennings, Charles W., Department of Conservation, Division of Mines and Geology, "Fault Activity Map of California and Adjacent Areas, With Locations and Ages of Recent Volcanic Eruptions," Sacramento, CA, 1994.

Macnab, Alan. *Earth Retention Systems Handbook.* New York: McGraw-Hill, 2002.

National Center for Earthquake Engineering Research. *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance Soils.* Technical Report NCEER-97-0022. Buffalo, New York: 1997.

Schmertmann, J.H., "Static Cone to Compute Settlement over Sand," ASCE *Journal of the Soil Mechanics and Foundations Division*, Vol. 96, No. SM3, p. 1011-1043, 1970.

Schmertmann, John.H., Hartman, John Paul and Brown, Phillip R., "Improved Strain Influence Factor Diagrams," ASCE *Journal of the Geotechnical Engineering Division*, Vol. 104, No. GT8, p.1131-1135, 1978.

Schmertmann, John H., *Guidelines for Cone Penetration Test: Performance and Design*, Report FHWA-TS-78-209, Federal Highway Administration, Washington, D.C., 1978.

Southern California Earthquake Center. *Recommended Procedures For Implementation of DMG Special Publication 117, Guidelines For Analyzing And Mitigating Liquefaction In California.* http://www.scec.org/resources/catalog/LiquefactionproceduresJun99.pdf March 1999

Tokimatsu, K. and Seed, H. B., "Evaluation of settlement in sands due to earthquake shaking," *ASCE Journal of Geotechnical Engineering*, 113 (8), pp. 861-878, 1987.

U.S. Geologic Survey, "CISN Peak Acceleration Map (in %g) Epicenter: 11 km NE of San Simeon, CA, Monday, December 22, 2004, 11:15:56 AM PST M6.5 N35.71 W121.10 Depth 7.6 km," www.earthquake.usgs.gov/shakemap/nc/skae/40148755/pga.jpg, 2004.

Youd, T.L., Corbett, M.H. and Bartlett, S.F. (2002), "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", Journal of Geotechnical and Geoenvironmental Engineering, ASCE Vol. 128, No. 12, pp. 1007-1017.

APPENDIX A

Field Investigation

CPT Logs

Classification Data/Soil Behavior Types

FIELD INVESTIGATION

The field investigation was conducted on August 3, 2010, using a truck mounted 25 ton CPT rig, The CPT rig advanced three CPT soundings to a maximum depth of 50.0 feet bgs. This exploration was conducted in accordance with presently accepted geotechnical engineering procedures consistent with the scope of the services authorized to GeoSolutions, Inc.

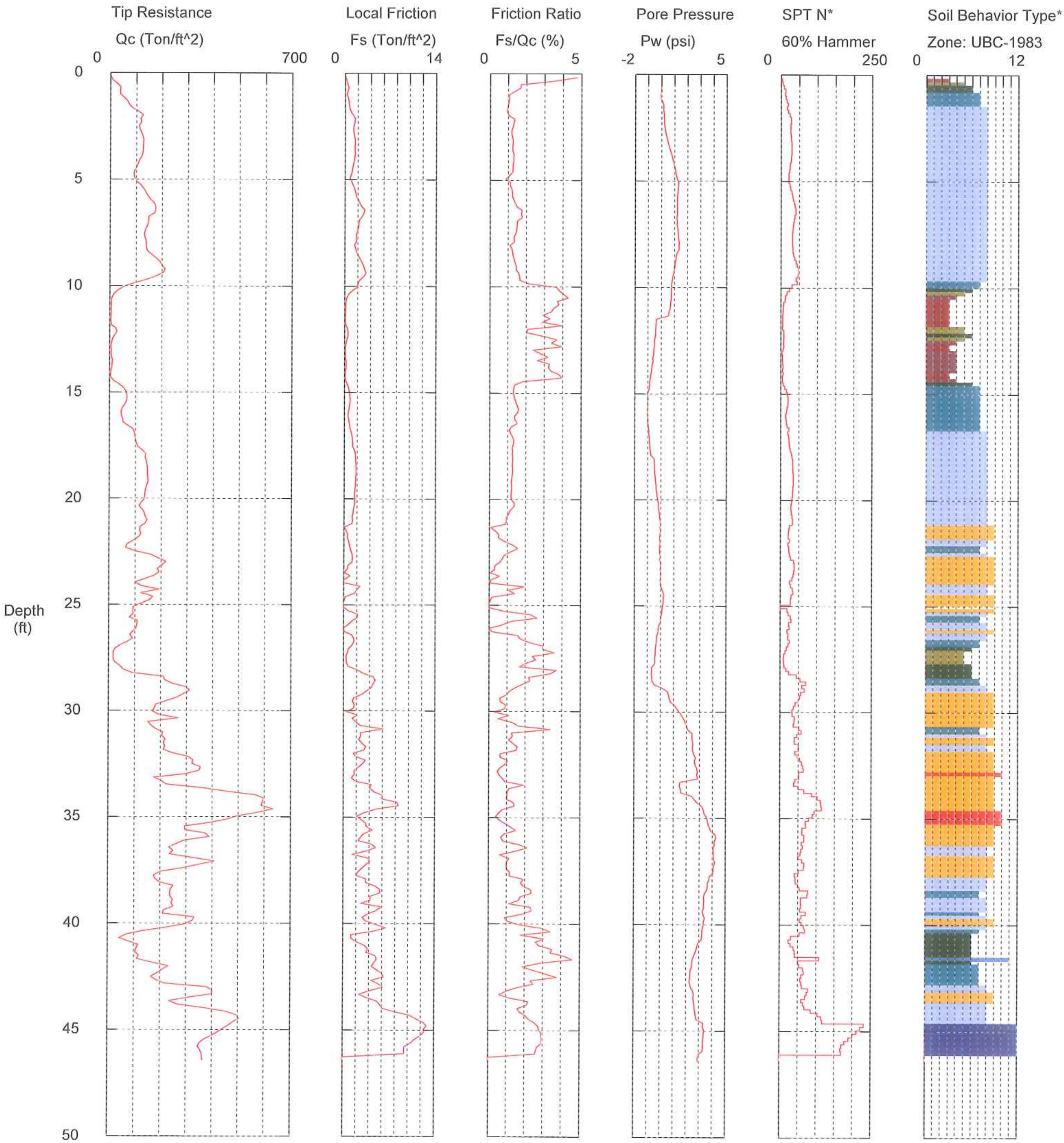
The CPT sounding with a 20-ton electronic CPT cone is advanced with measurements for cone bearing (q_C), sleeve friction (f_S), and pore water pressure (u2) measurements recorded at approximately 5-cm intervals. This provides a near continuous hydro geologic log. All CPT soundings are performed in accordance with ASTM D5778-95 (re-approved 2002) standards.

Logs of the soundings showing the depths and descriptions of the soils encountered, geologic structure where applicable, penetration resistance, and results of in-place density and moisture content tests are presented in this appendix. The logs represent the interpretation of field logs and tests, the interpolation of soil conditions between samples and the results of laboratory observations and tests. The noted stratification lines represent the approximate boundaries between the surface soil types. The actual transition between soil types may be gradual.

GeoSolutions, Inc.

Operator: ML-BH
Sounding: CPT-01
Cone Used: DSG0906

CPT Date/Time: 8/3/2010 10:08:
Location: Grover Beach Lod
Job Number: SL07154-1



Maximum Depth = 46.42 feet

Depth Increment = 0.164 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

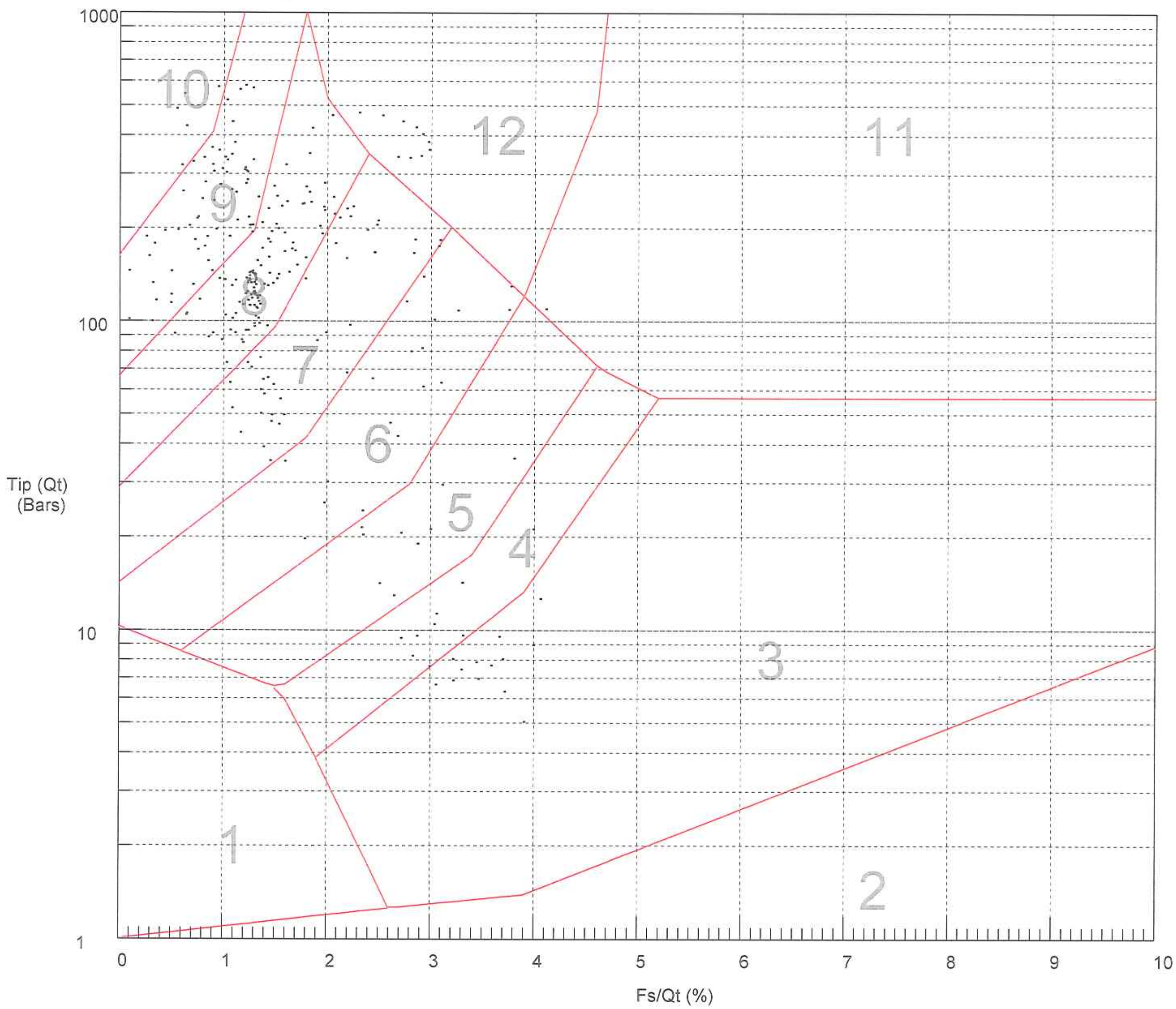
*Soil behavior type and SPT based on data from UBC-1983

GeoSolutions, Inc.

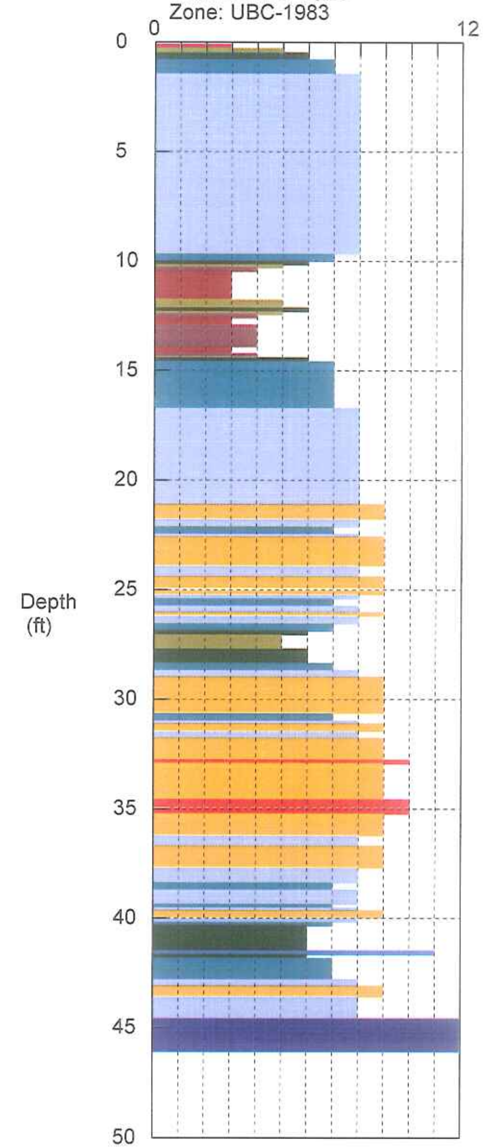
Operator: ML-BH
Sounding: CPT-01
Cone Used: DSG0906

CPT Date/Time: 8/3/2010 10:08:
Location: Grover Beach Lod
Job Number: SL07154-1

Classification Data:
Robertson and Campanella UBC-1983



Soil Behavior Type*
Zone: UBC-1983

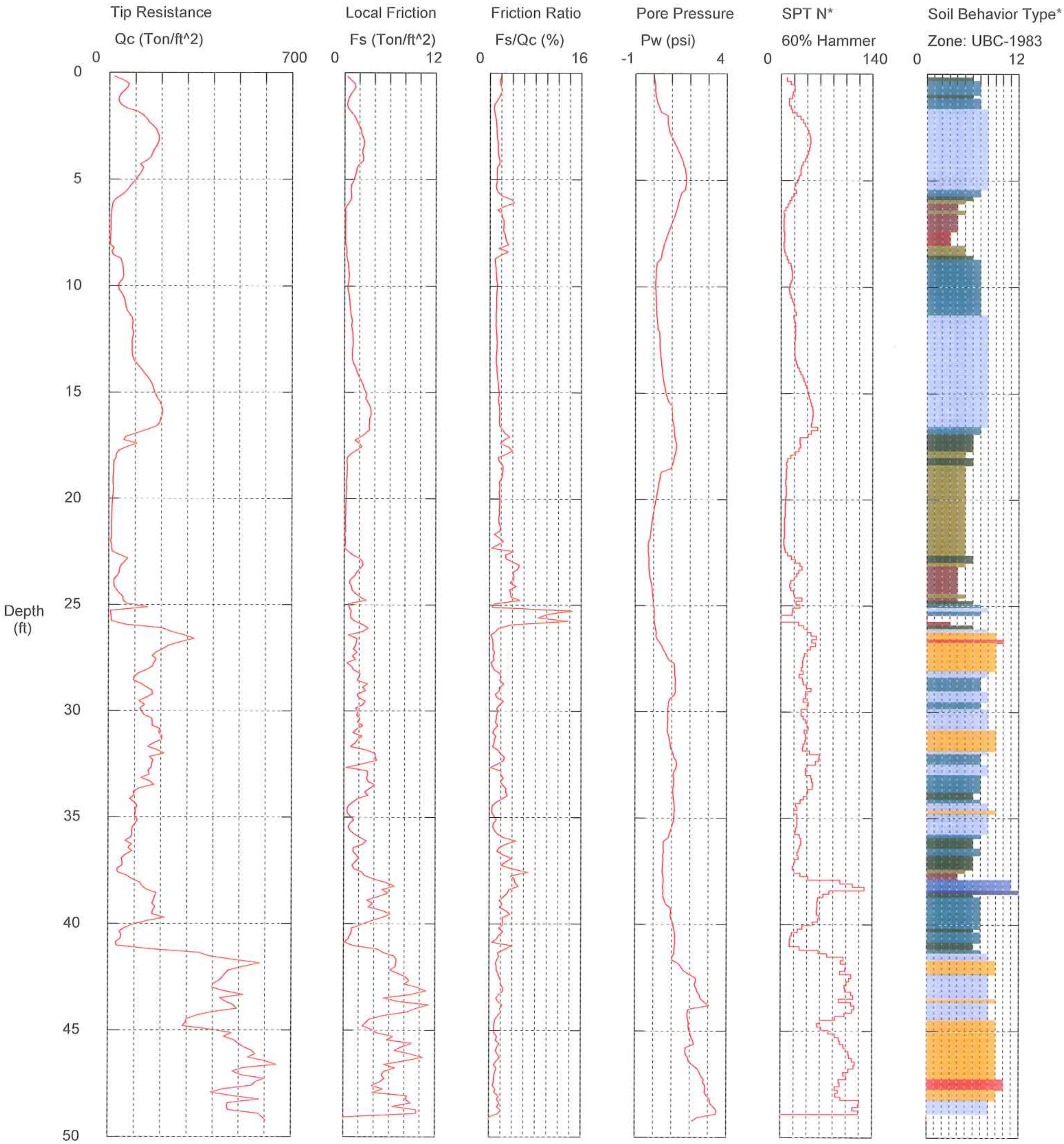


1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

GeoSolutions, Inc.

Operator: ML-BH
Sounding: CPT-02
Cone Used: DSG0906

CPT Date/Time: 8/3/2010 11:10:
Location: Grover Beach Lod
Job Number: SL07154-1



Maximum Depth = 49.21 feet

Depth Increment = 0.164 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

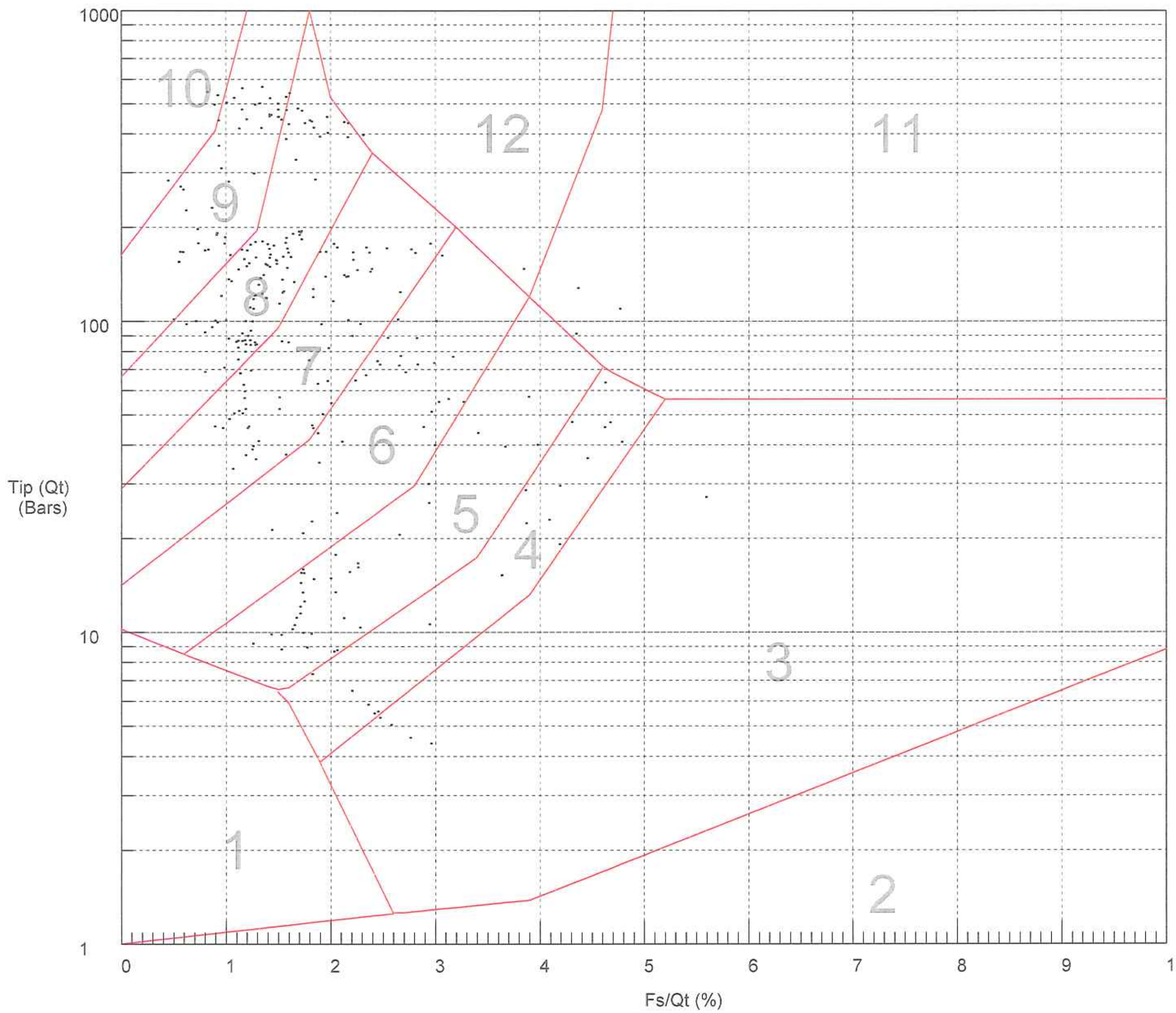
*Soil behavior type and SPT based on data from UBC-1983

GeoSolutions, Inc.

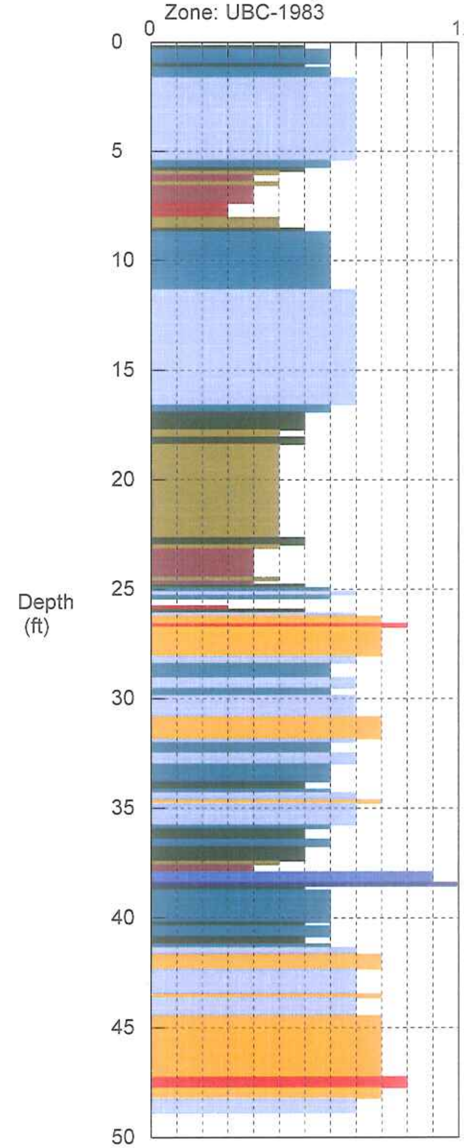
Operator: ML-BH
Sounding: CPT-02
Cone Used: DSG0906

CPT Date/Time: 8/3/2010 11:10:
Location: Grover Beach Lod
Job Number: SL07154-1

Classification Data:
Robertson and Campanella UBC-1983



Soil Behavior Type*
Zone: UBC-1983

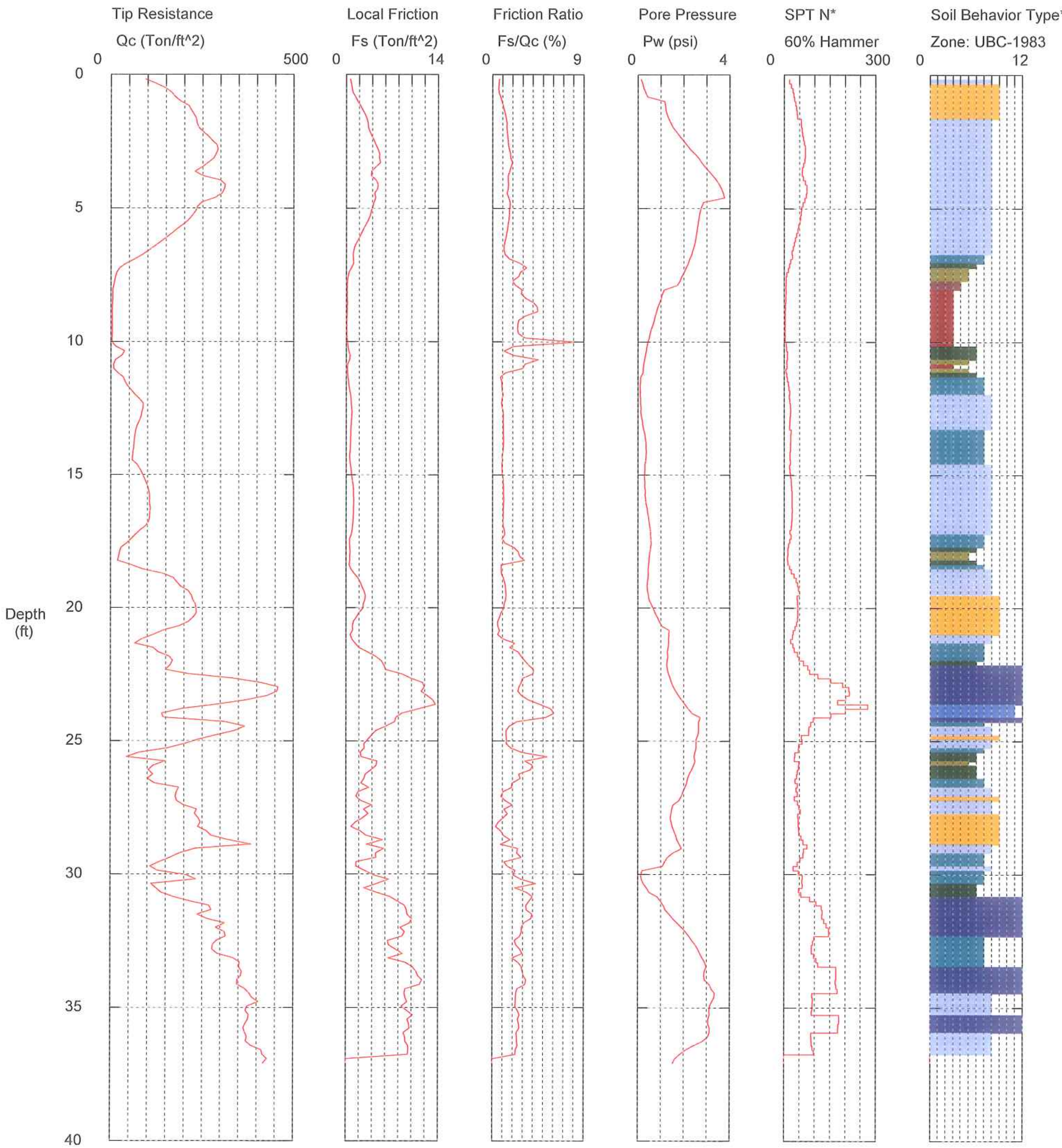


1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

GeoSolutions, Inc.

Operator: ML-BH
Sounding: CPT-03
Cone Used: DSG0906

CPT Date/Time: 8/3/2010 12:02:
Location: Grover Beach Lod
Job Number: SL07154-1



Maximum Depth = 37.07 feet

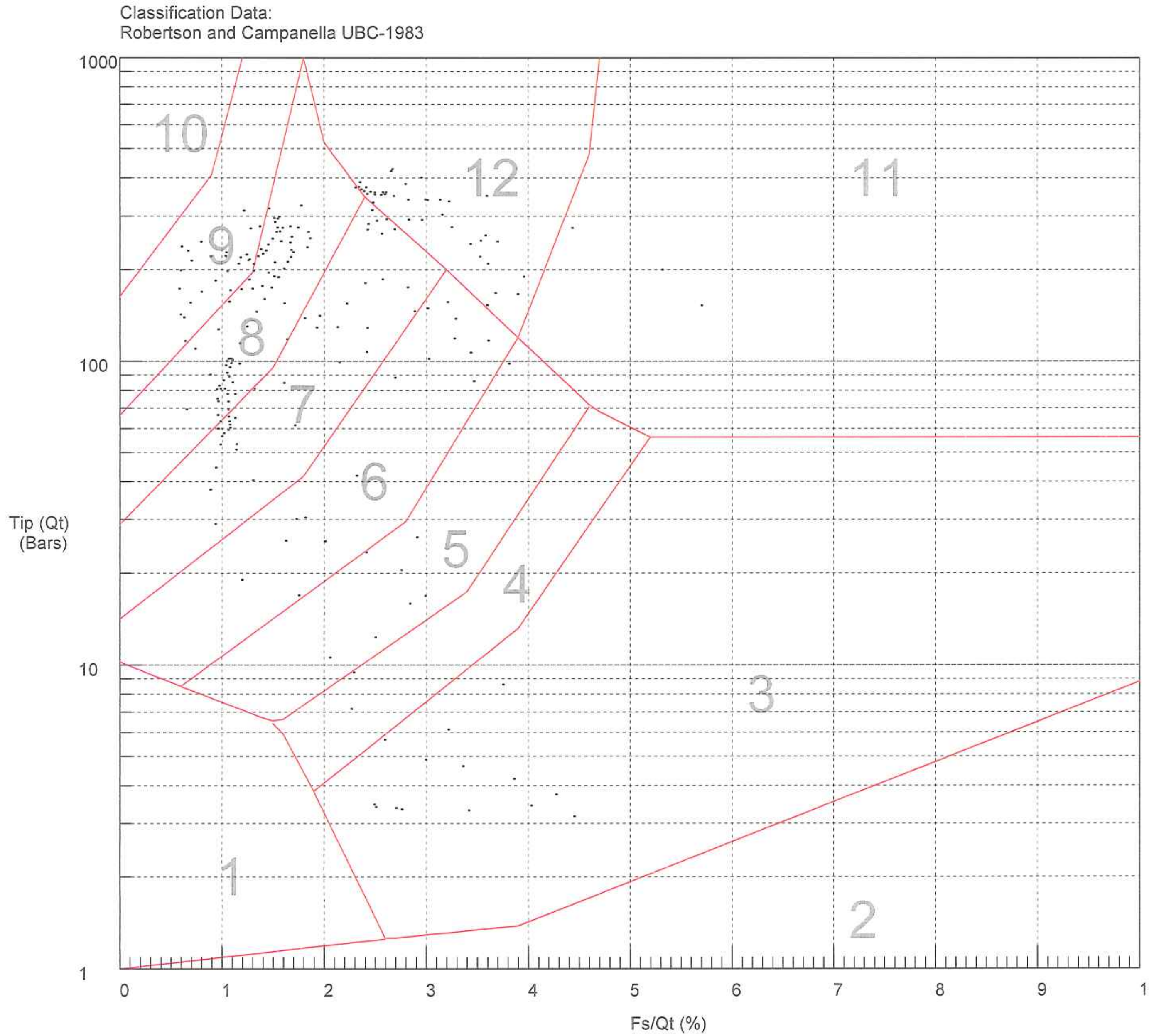
Depth Increment = 0.164 feet

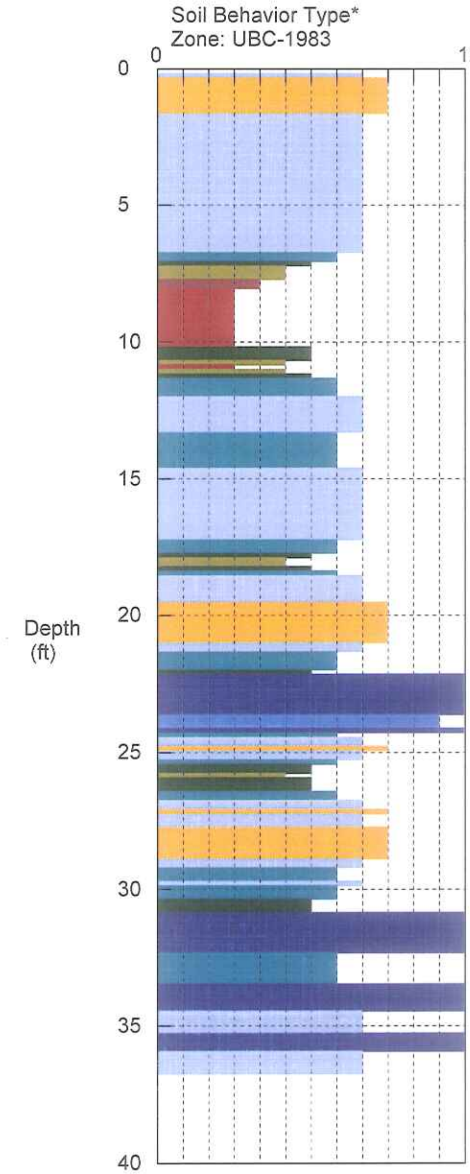
*Soil behavior type and SPT based on data from UBC-1983

GeoSolutions, Inc.

Operator: ML-BH
Sounding: CPT-03
Cone Used: DSG0906

CPT Date/Time: 8/3/2010 12:02:
Location: Grover Beach Lod
Job Number: SL07154-1





1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

APPENDIX B

Preliminary Grading Specifications

PRELIMINARY GRADING SPECIFICATIONS

A. General

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

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

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

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

B. Obligation of Parties

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

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

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

C. Site Preparation

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

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

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

D. Site Protection

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

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

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

E. Excavations

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

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

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

F. Structural Fill

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

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

G. Compacted Fill

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

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

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

iv. If fill areas are constructed on slopes greater than 5-to-1, we recommend that the toe of all areas to receive fill be keyed a minimum of 24 inches into underlying dense material. Key depths are to be observed and approved by a representative of GeoSolutions, Inc. Sub-drains shall be placed in the keyway and benches as required.

H. Drainage

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

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

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

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

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

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

I. Maintenance

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

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

J. Underground Facilities Construction

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

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

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

K. Completion of Work

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

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

APPENDIX C

Seismic Data – United States Geologic Society (USGS)

CPT Based Liquefaction Analysis – Liquefy Pro

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 35.123434
Longitude = -120.632971
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B - Fa = 1.0 ,Fv = 1.0
Data are based on a 0.009999999776482582 deg grid spacing

Period (sec)	Sa (g)
0.2	1.472 (Ss, Site Class B)
1.0	0.531 (S1, Site Class B)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 35.123434
Longitude = -120.632971
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.472 (SMs, Site Class D)
1.0	0.797 (SM1, Site Class D)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 35.123434
Longitude = -120.632971
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	0.981 (SDs, Site Class D)
1.0	0.531 (SD1, Site Class D)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 35.123434
Longitude = -120.632971
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	1.472 (SMs, Site Class D)
1.0	0.797 (SM1, Site Class D)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Latitude = 35.123434
Longitude = -120.632971
Design Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D - Fa = 1.0 ,Fv = 1.5

Period (sec)	Sa (g)
0.2	0.981 (SDs, Site Class D)
1.0	0.531 (SD1, Site Class D)

```
***********************
*                     *
*   E Q F A U L T     *
*                     *
*   Version 3.00      *
*                     *
***********************
```

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: SL07154-1
DATE: 09-10-2010

JOB NAME: grover beach lodge

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 35.1234
SITE LONGITUDE: 120.6330

SEARCH RADIUS: 100 mi

ATTENUATION RELATION: 3) Boore et al. (1997) Horiz. - NEHRP D (250)
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: cd_2drp
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
LOS OSOS	0.0(0.0)	6.8	0.675	XI
SAN LUIS RANGE (S. Margin)	0.8(1.3)	7.0	0.734	XI
HOSGRI	11.4(18.3)	7.3	0.276	IX
CASMALIA (Orcutt Frontal Fault)	13.2(21.2)	6.5	0.198	VIII
RINCONADA	14.5(23.3)	7.3	0.232	IX
LIONS HEAD	16.4(26.4)	6.6	0.178	VIII
LOS ALAMOS-W. BASELINE	30.8(49.6)	6.8	0.123	VII

SAN JUAN	31.3(50.4)	7.0	0.111	VII
NORTH CHANNEL SLOPE	36.1(58.1)	7.1	0.127	VIII
SAN ANDREAS - Cholame	42.0(67.6)	6.9	0.084	VII
SAN ANDREAS - 1857 Rupture	42.0(67.6)	7.8	0.134	VIII
SAN ANDREAS - Carrizo	45.5(73.2)	7.2	0.092	VII
SANTA YNEZ (West)	46.0(74.1)	6.9	0.078	VII
SAN ANDREAS - Parkfield Segment	47.3(76.1)	6.7	0.069	VI
M.RIDGE-ARROYO PARIDA-SANTA ANA	60.7(97.7)	6.7	0.069	VI
SAN ANDREAS (Creeping)	60.8(97.9)	6.5	0.051	VI
GREAT VALLEY 14	65.4(105.3)	6.4	0.055	VI
GREAT VALLEY 13	69.8(112.4)	6.5	0.056	VI
CHANNEL IS. THRUST (Eastern)	70.5(113.5)	7.4	0.089	VII
SANTA YNEZ (East)	71.5(115.0)	7.0	0.058	VI
RED MOUNTAIN	73.0(117.5)	6.8	0.063	VI
SANTA ROSA ISLAND	75.7(121.8)	6.9	0.064	VI
BIG PINE	76.4(123.0)	6.7	0.047	VI
PLEITO THRUST	76.7(123.4)	7.2	0.075	VII
MONTALVO-OAK RIDGE TREND	77.0(123.9)	6.6	0.054	VI
SANTA CRUZ ISLAND	78.5(126.3)	6.8	0.059	VI
VENTURA - PITAS POINT	81.5(131.2)	6.8	0.058	VI
GREAT VALLEY 12	82.0(131.9)	6.3	0.044	VI
OAK RIDGE(Blind Thrust Offshore)	85.0(136.8)	6.9	0.059	VI
WHITE WOLF	86.8(139.7)	7.2	0.068	VI
GREAT VALLEY 11	90.5(145.6)	6.4	0.043	VI
ANACAPA-DUME	92.8(149.4)	7.3	0.068	VI
SAN CAYETANO	94.1(151.5)	6.8	0.052	VI
PALO COLORADO - SUR	95.9(154.4)	7.0	0.046	VI
GARLOCK (West)	99.1(159.5)	7.1	0.048	VI
MONTEREY BAY - TULARCITOS	99.7 (160.5)	7.1	0.058	VI

****

-END OF SEARCH- 36 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE LOS OSOS FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.0 MILES (0.0 km) AWAY.

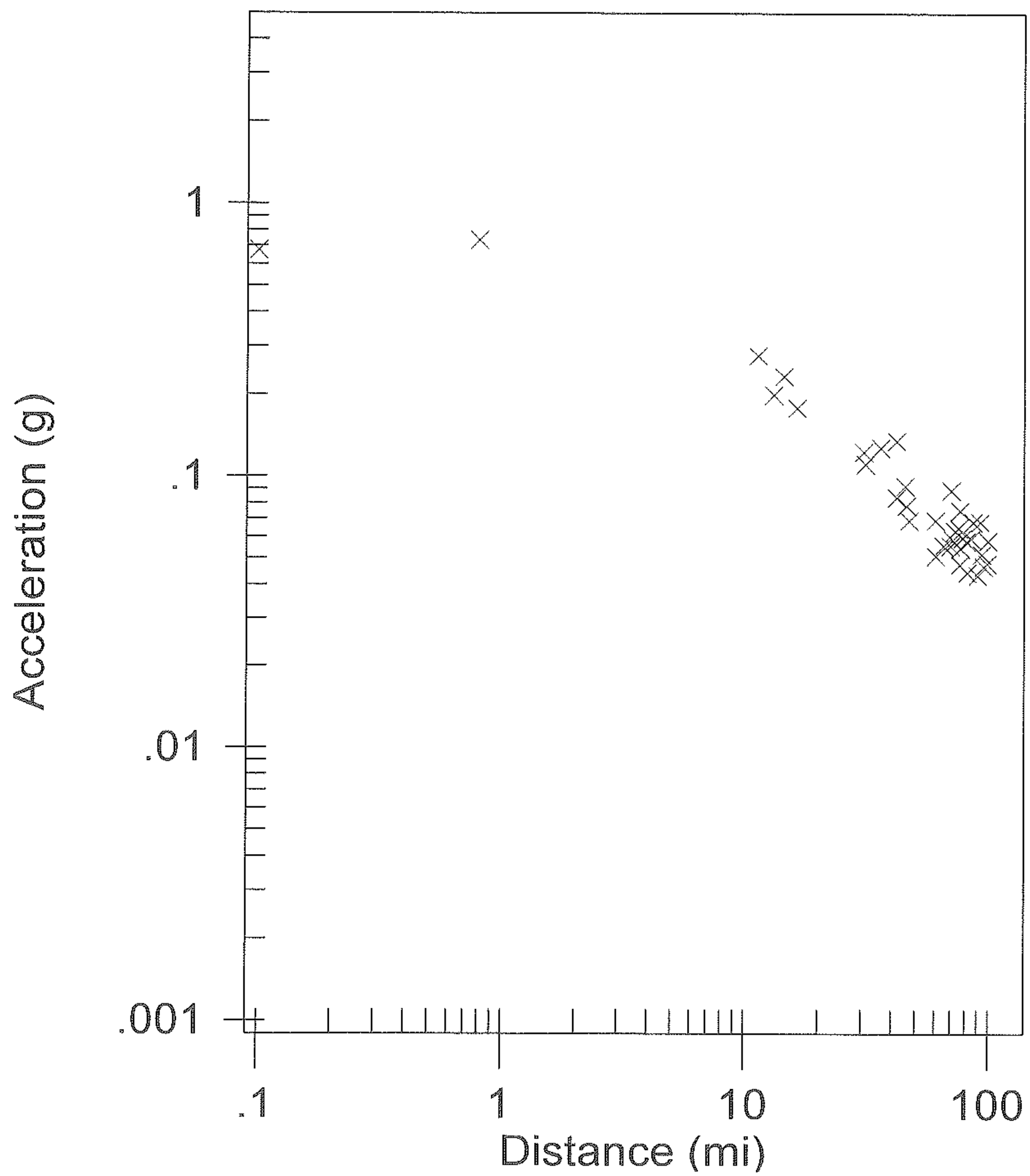
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.7344 g

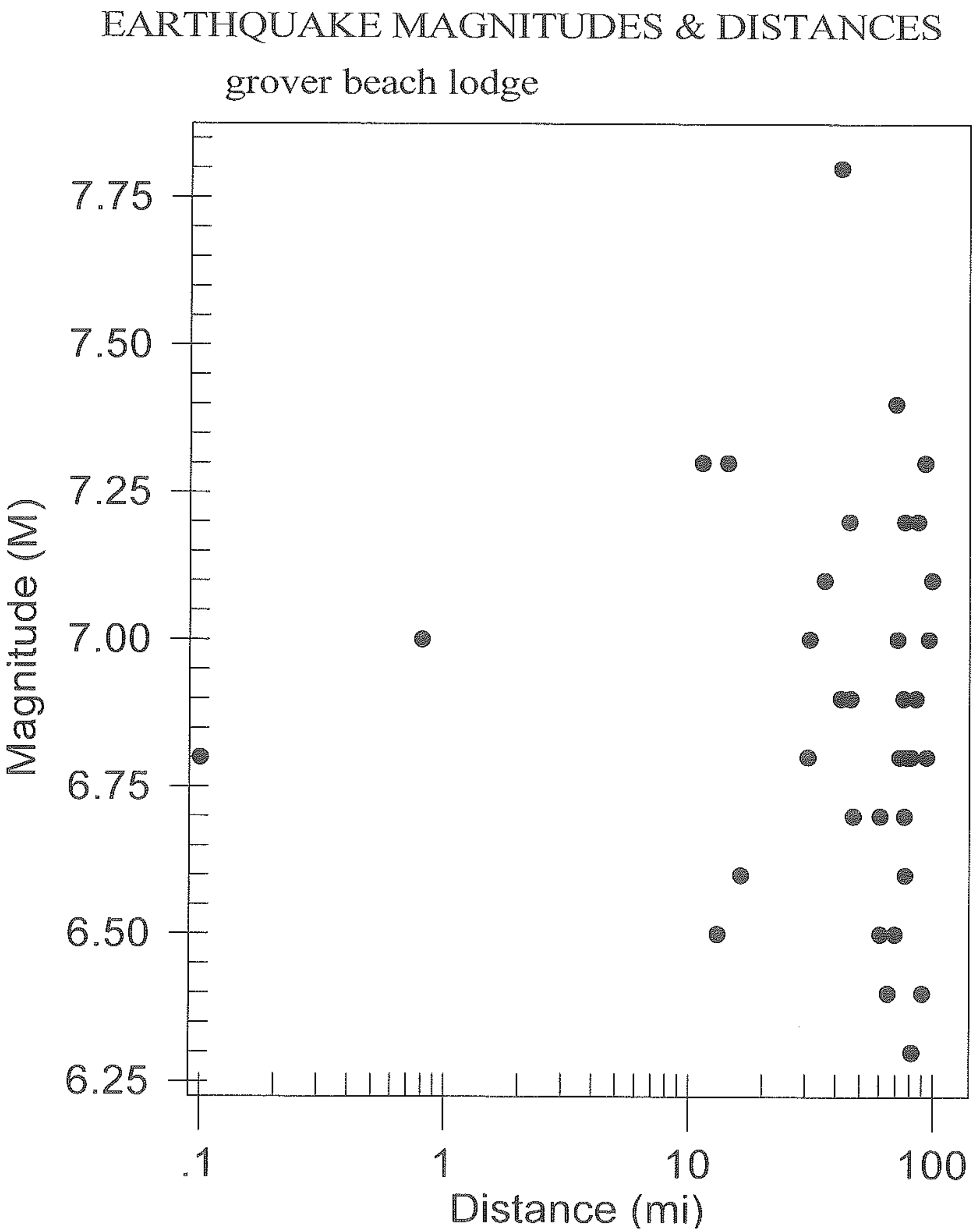
Footing B/2	B'	80
Poisson's Ratio	u	0.35
Mat Corner/Center	m	4
C1		0.9
C2		1.2
Delta P (psf)		1250
Total Settlement (in)		1.042

Layer No	Layer Thickness (in)	Qc TSF	Es Multiplier	Es	Iz	Iz/Es*delta z
1	12	50	3	150	0.04	0.0032
2	36	150	3.5	525	0.08	0.005485714
3	12	120	3.5	420	0.08	0.002285714
4	12	50	3.5	175	0.1	0.006857143
5	24	7	2	14	0.1	0.171428571
6	24	40	2	80	0.125	0.0375
7	48	80	3.5	280	0.15	0.025714286
8	30	175	3.5	612.5	0.2	0.009795918
9	12	50	2	100	0.25	0.03
10	48	12	2	24	0.3	0.6
11	30	40	2	80	0.325	0.121875
12	12	10	3.5	35	0.35	0.12
13	48	175	3.5	612.5	0.4	0.007836735
14	60	175	3.5	612.5	0.4	0.031346939
15	24	90	2	180	0.5	0.166666667
16	24	150	2	300	0.6	0.048
17	12	50	2	100	0.6	0.144
18	48	400	7	2800	0.5	0.002142857
19	60	500	7	3500	0.4	0.005485714
20	1000	850	7	5950	0.4	0.004033613
						1.543654872

MAXIMUM EARTHQUAKES

grover beach lodge

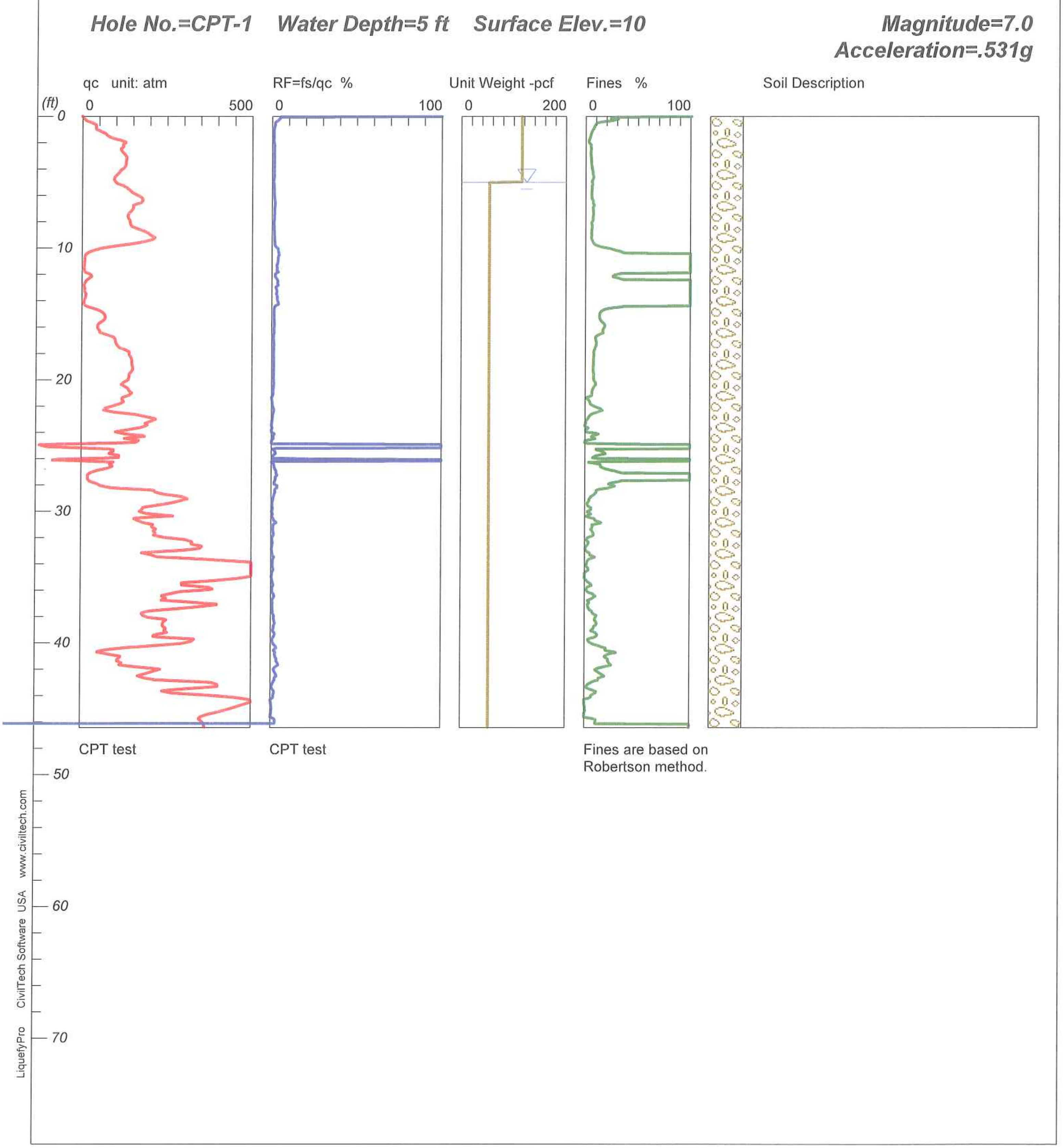




EARTHQUAKE MAGNITUDES & DISTANCES
grover beach lodge
Magnitude (M)
7.75
7.50
7.25
7.00
6.75
6.50
6.25
.1
1
10
100
Distance (mi)

LIQUEFACTION ANALYSIS

Grover Beach Lodge

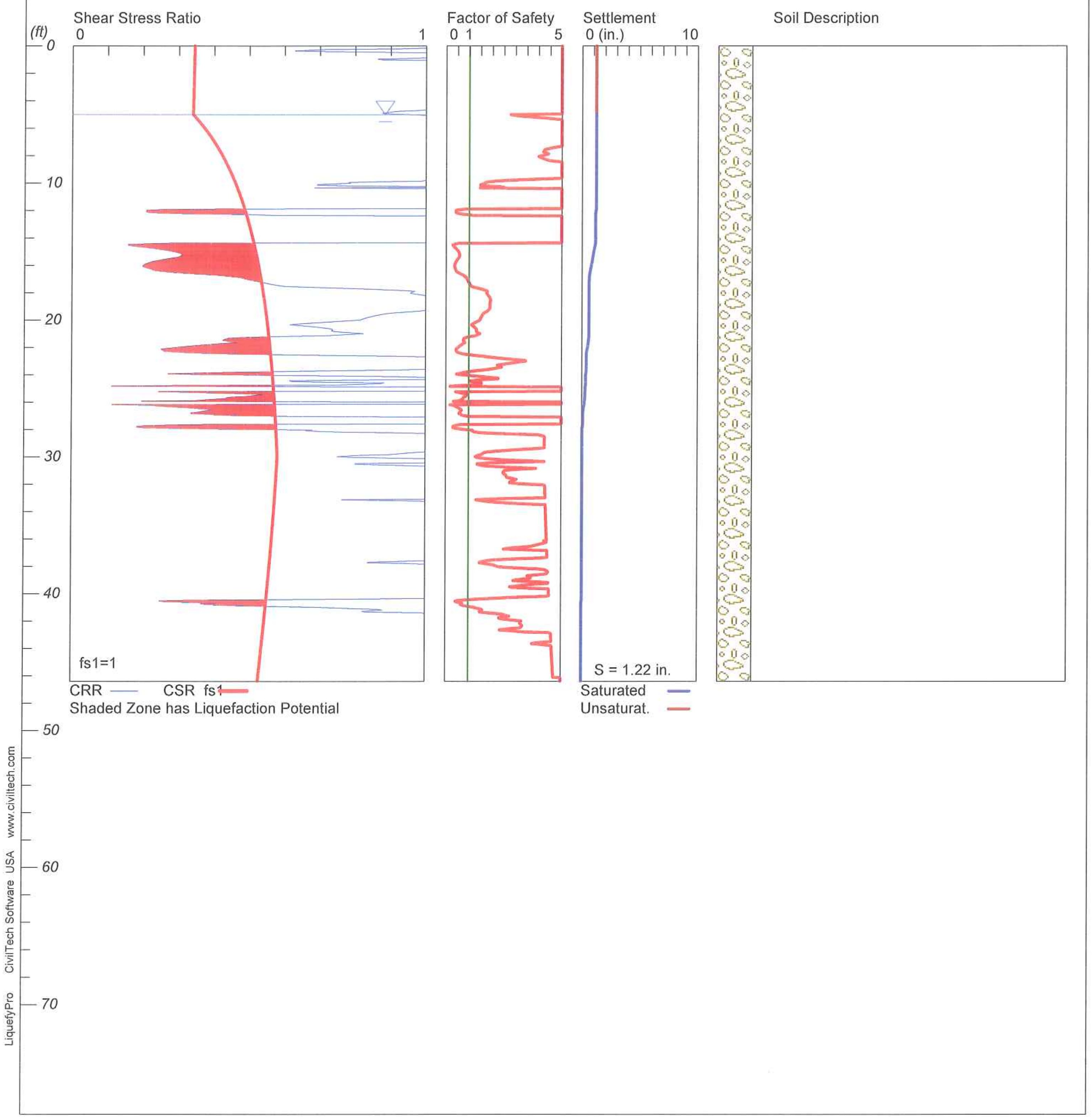


LIQUEFACTION ANALYSIS

Grover Beach Lodge

Hole No.=CPT-1 Water Depth=5 ft Surface Elev.=10

Magnitude=7.0
Acceleration=.531g

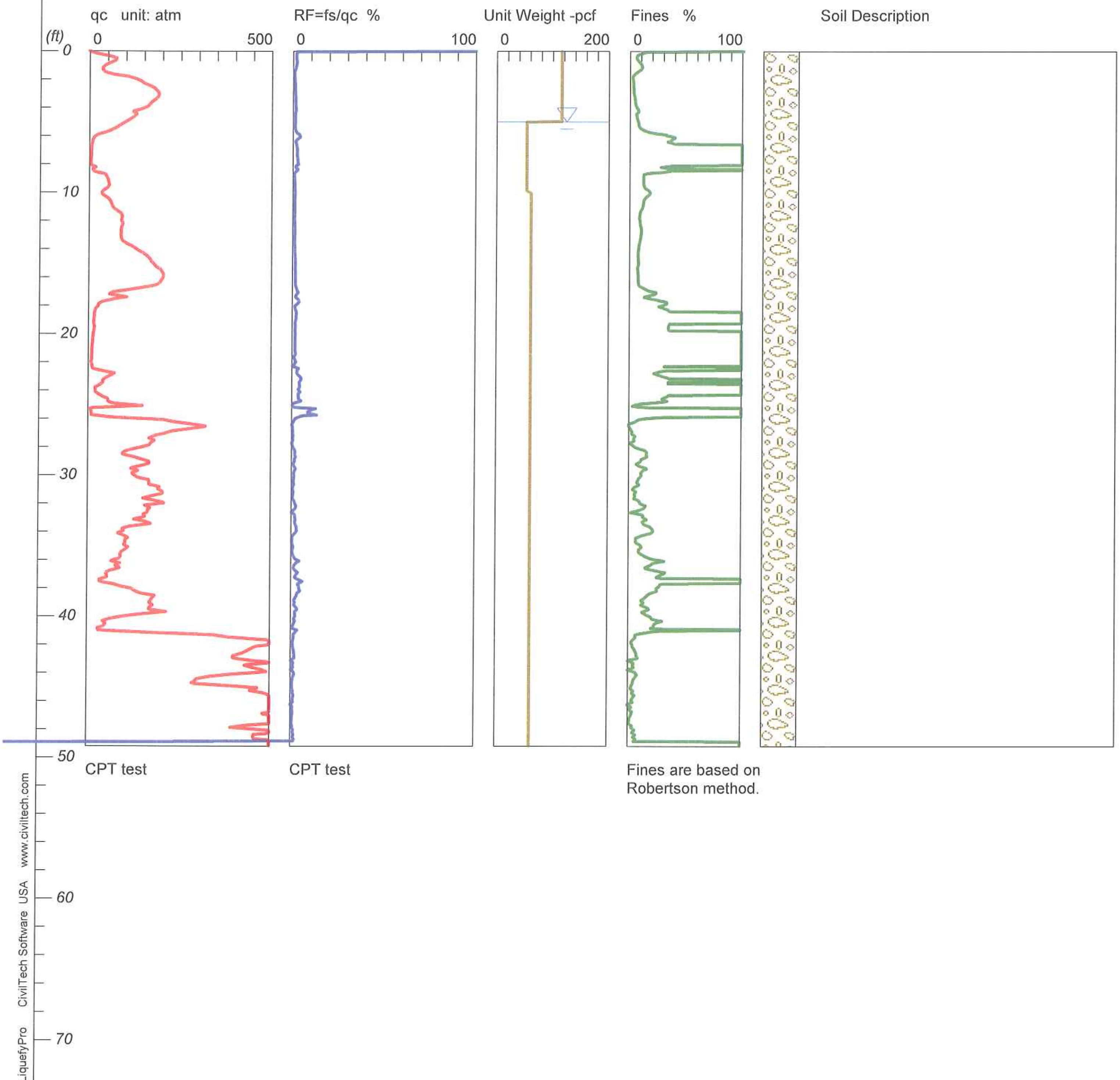


LIQUEFACTION ANALYSIS

Grover Beach Lodge

Hole No.=CPT-2 Water Depth=5 ft Surface Elev.=10

Magnitude=7
Acceleration=0.531g

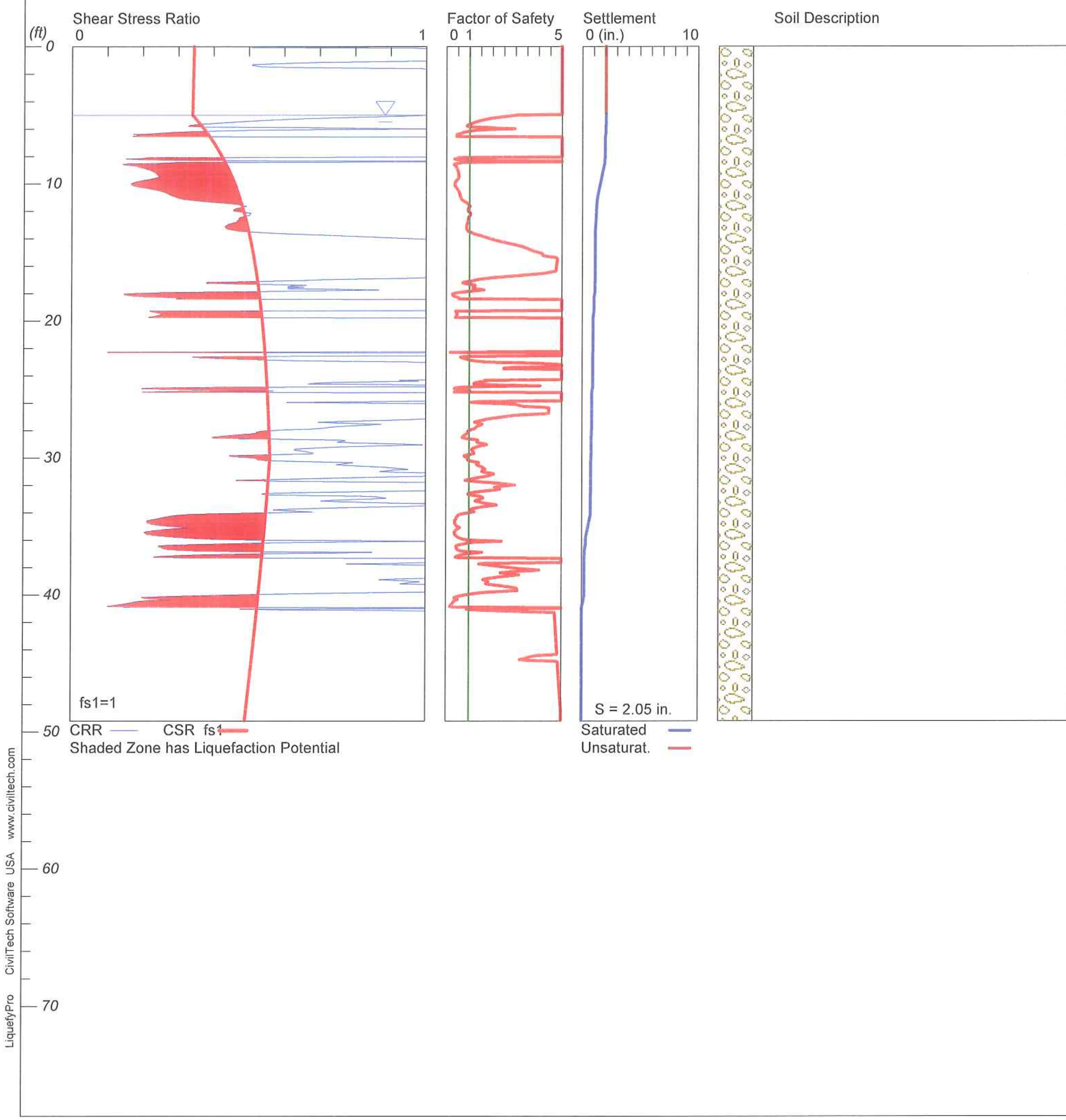


LIQUEFACTION ANALYSIS

Grover Beach Lodge

Hole No.=CPT-2 Water Depth=5 ft Surface Elev.=10

Magnitude=7
Acceleration=0.531g

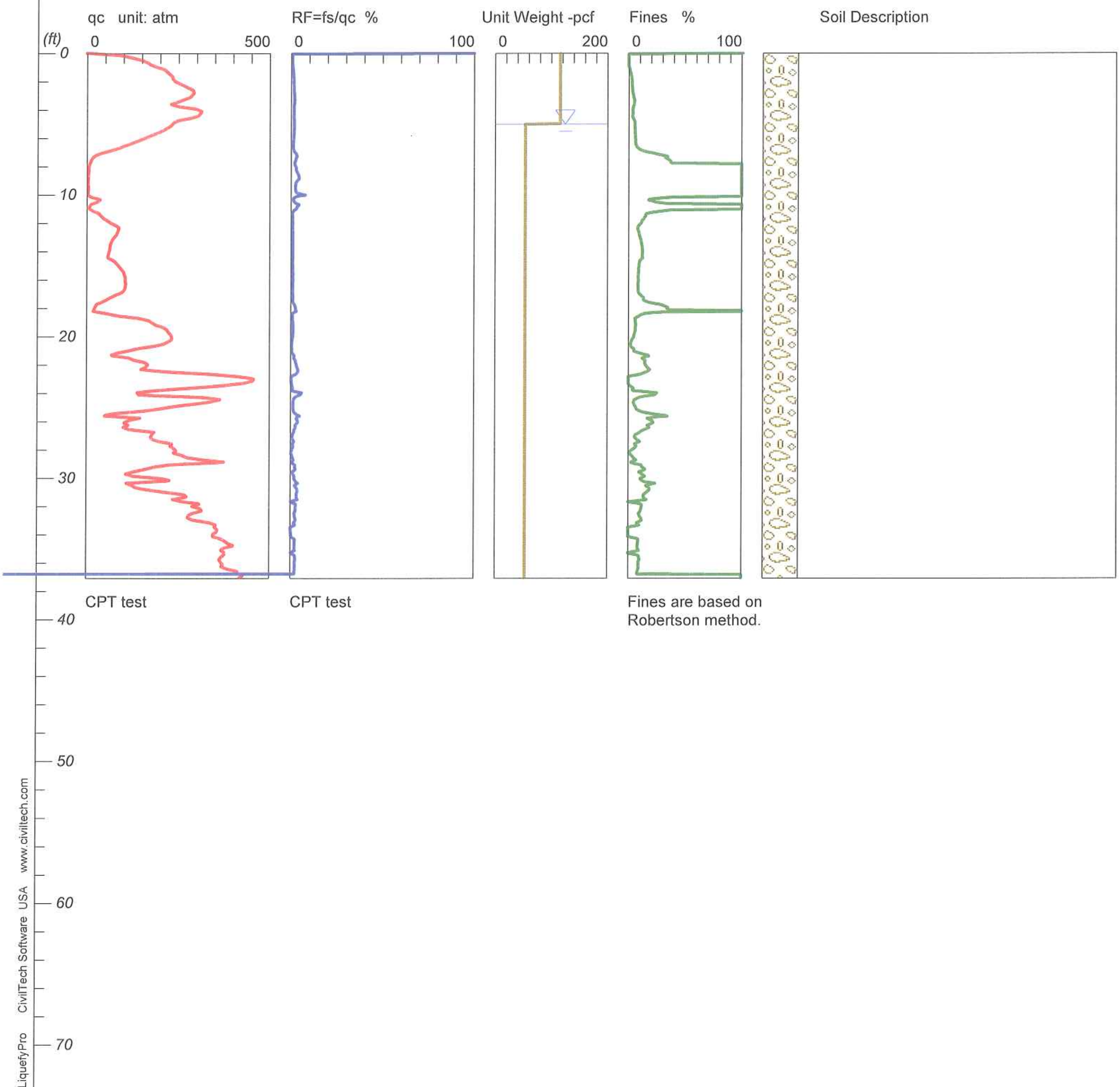


LIQUEFACTION ANALYSIS

Grover Beach Lodge

Hole No.=CPT-3 Water Depth=5 ft Surface Elev.=10

Magnitude=7.0
Acceleration=0.531g



LIQUEFACTION ANALYSIS

Grover Beach Lodge

Hole No.=CPT-3 Water Depth=5 ft Surface Elev.=10

Magnitude=7.0
Acceleration=0.531g