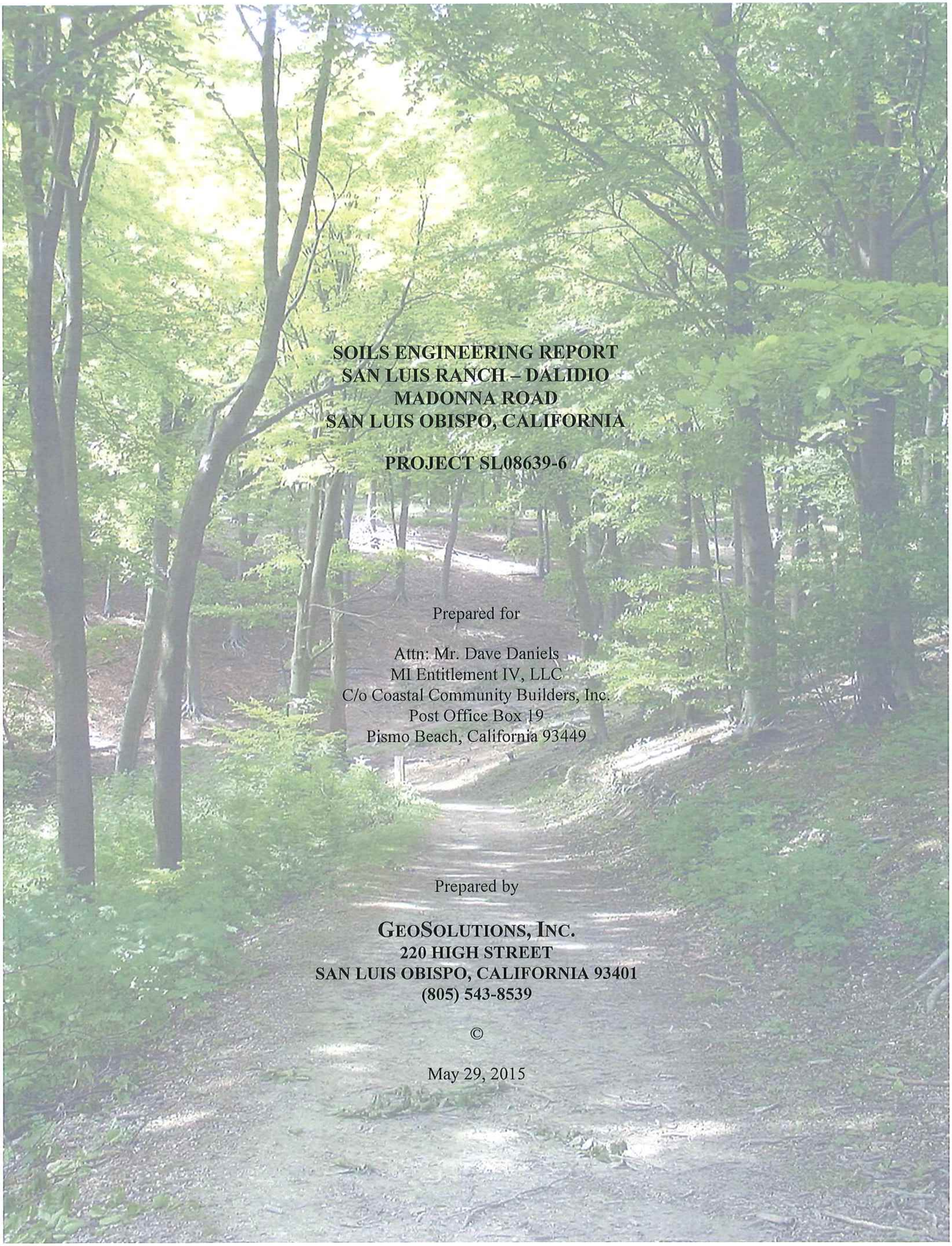
Appendix E

Soil Engineering Report



SOILS ENGINEERING REPORT
SAN LUIS RANCH – DALIDIO
MADONNA ROAD
SAN LUIS OBISPO, CALIFORNIA

PROJECT SL08639-6

Prepared for

Attn: Mr. Dave Daniels
MI Entitlement IV, LLC
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May 29, 2015

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May 29, 2015
Project No. SL08639-6

Attn: Mr. Dave Daniels
MI Entitlement IV, LLC
C/o Coastal Community Builders, Inc.
Post Office Box 19
Pismo Beach, California 93449

Subject: **Soils Engineering Report**
San Luis Ranch – Dalidio, Madonna Road
San Luis Obispo, California

Dear Mr. Daniels:

This Soils Engineering Report has been prepared for the proposed San Luis Ranch development located east of Madonna Road, west of the 101 Freeway, and south of Dalidio Drive in San Luis Obispo, California. This report is intended for use in the development of plans and specifications for the project.

Preliminary geotechnical recommendations for site preparation, earthwork, conventional foundations, post-tension foundations, exterior concrete flatwork, retaining walls, and pavement structural section design are presented herein. Based on the results of our field investigation, it is anticipated that graded pads will be constructed for the proposed structures within the development with foundations supported by engineered fill. Expansive surface soils are present within the development areas and should be taken into consideration during plan development due to the potential for expansive soil movement to negatively affect the proposed improvements, including but not limited to; foundation systems, concrete slabs-on-grade, exterior concrete flatwork, and pavements. 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.

Kraig R. Crozier, PE
Principal, C61361

REGISTERED PROFESSIONAL ENGINEER
KRAIG ROBERT CROZIER
No. C61361
Exp. 6-30-15
CIVIL
STATE OF CALIFORNIA

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MADONNA ROAD
SAN LUIS OBISPO, CALIFORNIA

PROJECT SL08639-6

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation for the proposed San Luis Ranch located east of Madonna Road, west of the 101 Freeway, and south of Dalidio Drive in San Luis Obispo, California. See Figure 1: Site Location Map for the general location of the project area. Figure 1: Site Location Map was obtained from the computer program *Topo USA 8.0* (DeLorme, 2009).

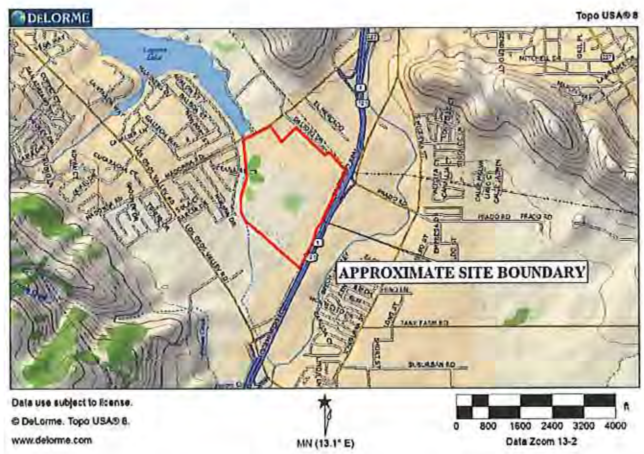


Figure 1: Site Location Map

1.1 Site Description

San Luis Ranch is located at 35.25612 degrees north latitude and 120.67920 degrees west longitude at a general elevation of 130 feet above mean sea level. The property is approximately rectangular in shape and 145 acres in size. The nearest intersection is where Madonna Road intersects Dalidio Drive at the northern corner of the property.

The topography of the Site is relatively flat with a gentle gradient that slopes down to the southwest at approximately 32:1 (horizontal to vertical). Surface drainage follows the topography to the southwest and flows to Prefumo Creek, which is located approximately 650 feet to the southwest. The Site is currently in use for agricultural purposes, with some existing structures and densely vegetated areas adjacent to Madonna Road.

1.2 Project Description

At the time of the preparation of this report, the proposed development is to include approximately 350 single-family residences, 150 multi-family residential units, 200,000 square feet of commercial space, 150,000 square feet of office space, and a 200 room hotel. It is anticipated that the proposed commercial structures will be located within the northeast portion of the Site, adjacent to Dalidio Drive. The proposed multi-family and single-family residences will be located to northwest and southwest of the commercial buildings, respectively. The project property will hereafter be referred to as the "Site." See Figure 2: Site Plan for the general layout of the Site.

It is anticipated that the proposed residential and commercial development areas will utilize slab-on-grade lower floor systems. Dead and sustained live loads are currently unknown, but they are anticipated to be relatively light for the residential development areas and moderate for the commercial development areas with maximum continuous footing and column loads estimated to be approximately 1.5 to 4 kips per linear foot and 15 kips to 40 kips, respectively.

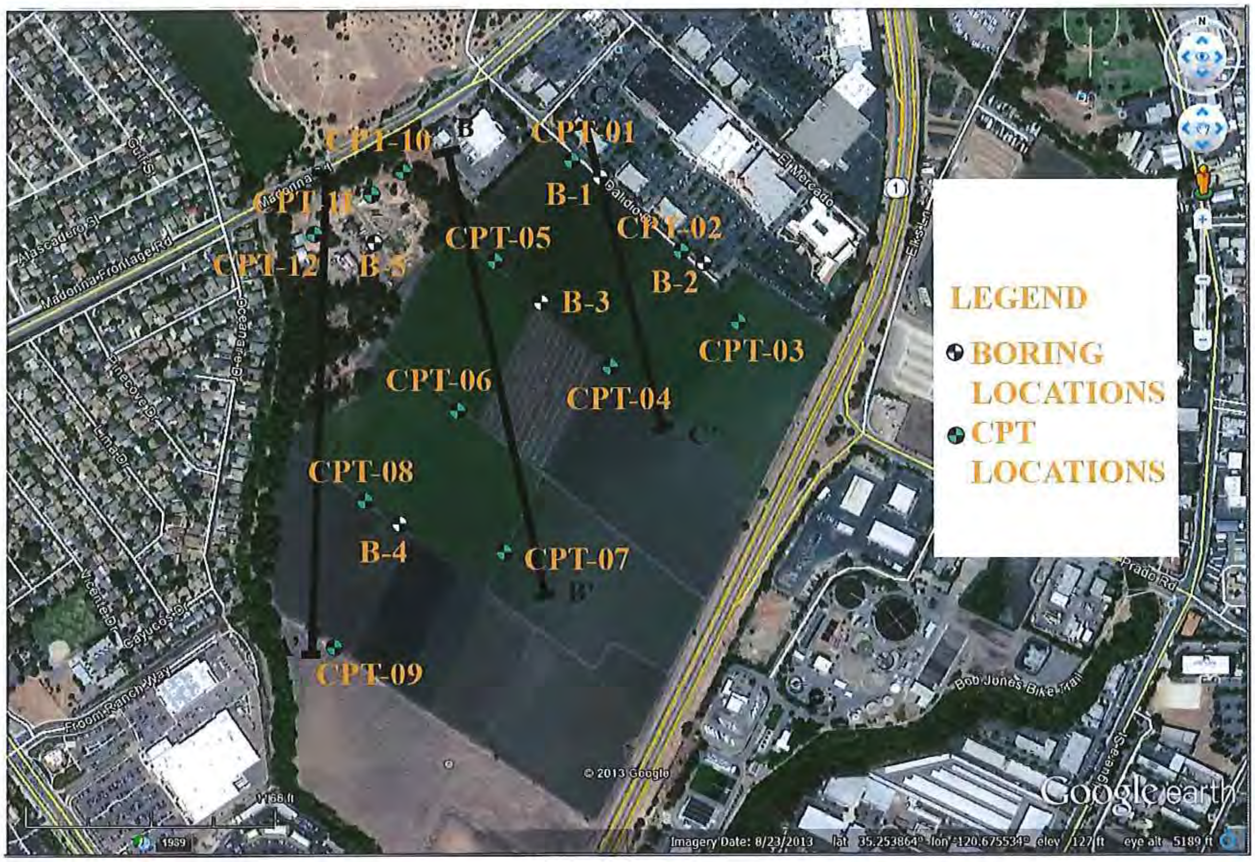


Figure 2: Site Plan

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

1. A literature review of available published and unpublished geotechnical data pertinent to the project site including geologic maps, and available on-line or in-house aerial photographs.

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

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

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

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

3.0 FIELD AND LABORATORY INVESTIGATION

The field investigation was conducted on March 11, 2015 using a CPT Truck provided by Middle Earth Geo Testing, Inc. and a Mobile B-24 drill rig. Twelve CPT soundings and five four-inch diameter exploratory borings were advanced to a maximum depth of 50 feet below ground surface (bgs) at the approximate locations indicated on Figure 3: Google Earth Image. Sampling methods included the Standard Penetration Test utilizing a standard split-spoon sampler (SPT) without liners and a Modified California sampler (CA) with liners. The Mobile B-24 drill rig was equipped with a safety hammer, which has an efficiency of approximately 60 percent and was used to obtain test blow counts in the form of N-values.

An electric cone was used during the CPT sounding. 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. The CPT soundings were advanced to provide a nearly-continuous soil behavior profile and to better characterize the Site. See **Appendix A** for CPT data and for a description and classification of the soil behavior types.

Figure 3: Google Earth Image

Data gathered during the field investigation suggest that the soil materials at the Site consist of interbedded layers of colluvial and alluvial soil. The surface materials at the Site generally consisted of dark grayish brown to black sandy fat CLAY (CH) encountered in a slightly moist to moist and stiff condition and dark brown to black sandy CLAY (CL) encountered in a slightly moist and stiff condition to approximately 3.0 to 4.5 feet bgs. The sub-surface materials consisted of brown to dark olive brown sandy CLAY (CL) with gravel encountered in a moist to wet and firm to very stiff condition underlain by light brown clayey SAND (SC) with gravel encountered in a moist and medium dense to very dense condition.

Regional site geology was obtained by using the *Geologic Map of the San Luis Obispo Quadrangle* (Dibblee, 2004) and the MapView internet application (USGS, 2013); the later application is available from the United States Geological Survey website (USGS, 2013) and compiles existing geologic maps. The majority of all underlying material at the Site was interpreted as Surficial Sediments. Groundwater was encountered at a depth of approximately 13 feet bgs. See Figure 4: Regional Geologic Map

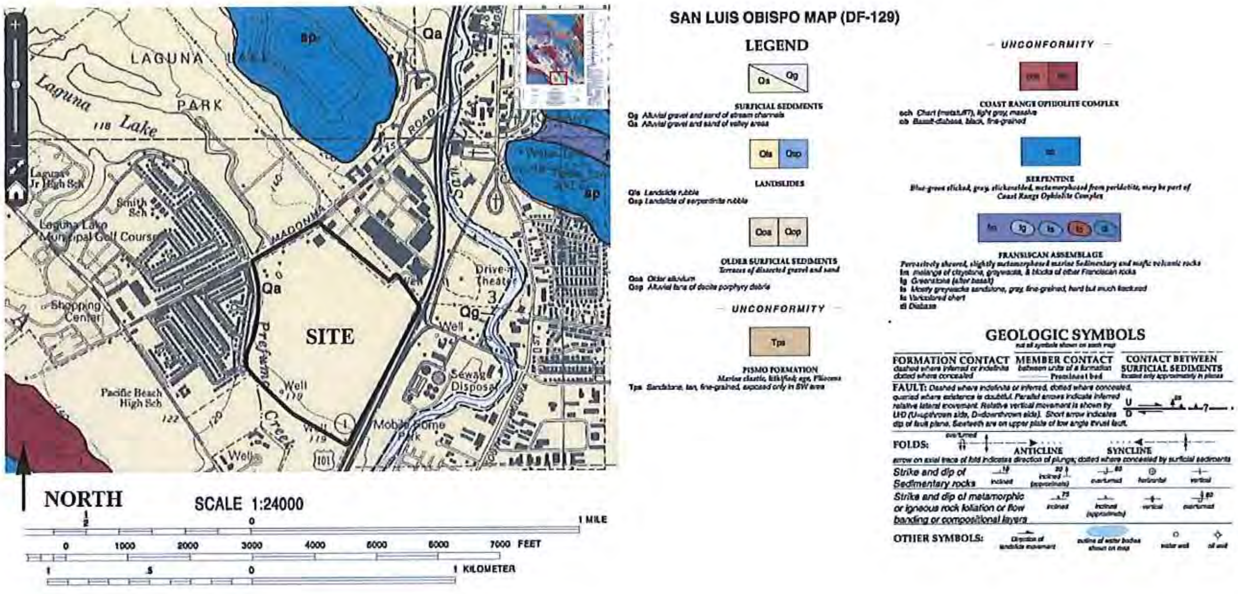


Figure 4: Regional Geologic Map

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

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

Table 1: Engineering Properties

Sample Name	Sample Description	USCS Specification	Expansion Index	Expansion Potential	Maximum Dry Density, γ_d (pcf)	Optimum Moisture (%)	Plasticity Index	Fines Content (%)	Compression Index, C_c	Recompression Index, C_r
A	Black Sandy Fat CLAY	CH	69	Medium	107.1	15.2	36	86.0	-	-
B	Dark Olive Brown Sandy CLAY	CL	51	Medium	-	-	23	63.0	-	-
C	Dark Grayish Brown Sandy CLAY	CL	38	Low	-	-	17	64.0	-	-
D	Olive Brown Sandy CLAY	CL	62	Medium	-	-	22	76.0	-	-
E	Grayish Brown Sandy CLAY	CL	57	Medium	-	-	23	62.0	-	-

Sample Name	Sample Description	USCS Specification	Expansion Index	Expansion Potential	Maximum Dry Density, γ_d (pcf)	Optimum Moisture (%)	Plasticity Index	Fines Content (%)	Compression Index, C_c	Recompression Index, C_r
F	Dark Gray Sandy CLAY	CL	52	Medium	-	-	19	53.0	-	-
G	Very Dark Gray Sandy Fat CLAY	CH	73	Medium	-	-	38	89.0	-	-
H	Dark Grayish Brown Sandy Fat CLAY	CH	79	Medium	-	-	30	64.0	-	-
I	Olive Brown Sandy CLAY	CL	77	Medium	-	-	27	68.0	-	-
B-1 @ 5.0 ft	Dark Olive Brown Sandy CLAY	CL	-	-	-	-	-	-	0.083	0.008
B-2 @ 5.0 ft	Very Dark Grayish Brown Sandy CLAY with Gravel	CL	-	-	-	-	-	-	0.070	0.007
B-3 @ 5.0 ft	Very Dark Brown Sandy CLAY with Gravel	CL	-	-	-	-	-	-	0.077	0.008
B-4 @ 5.0 ft	Dark Olive Brown Sandy CLAY with Gravel	CL	-	-	-	-	-	-	0.086	0.009
B-5 @ 5.0 ft	Dark Yellowish Brown Sandy CLAY	CL	-	-	-	-	-	-	0.101	0.010

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 D, Stiff Soil 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}=0.481 and S_{DS}=0.832,** 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 35.25612 degrees north latitude and 120.67920 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 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, 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 = 0.519 g** which is present on Sheet 5 of 6 of the USGS Design Maps Detailed Report (ASCE 7-10 Standard). See **Appendix D:** USGS Design Maps Summary and Detailed Report. This **PGA_M** was utilized in our liquefaction analysis.

4.3 Liquification Potential

1. Liquefaction occurs when saturated cohesionless soils lose shear strength due to earthquake shaking. Ground motion from an earthquake may induce cyclic reversals of shear stresses of large amplitude. Lateral and vertical movement of the soil mass combined with the loss of bearing strength usually results from this phenomenon. The potential for liquefaction estimated from the twelve CPT soundings is shown below in Figure 5: Liquefaction Potential.

2. Liquefaction potential of soil deposits during earthquake activity depends on soil type, void ratio, groundwater conditions, the duration of shaking, and confining pressures on the potentially liquefiable soil unit. Fine, poorly graded loose sand, shallow groundwater, high intensity earthquakes, and long duration of ground shaking are the principal factors leading to liquefaction

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

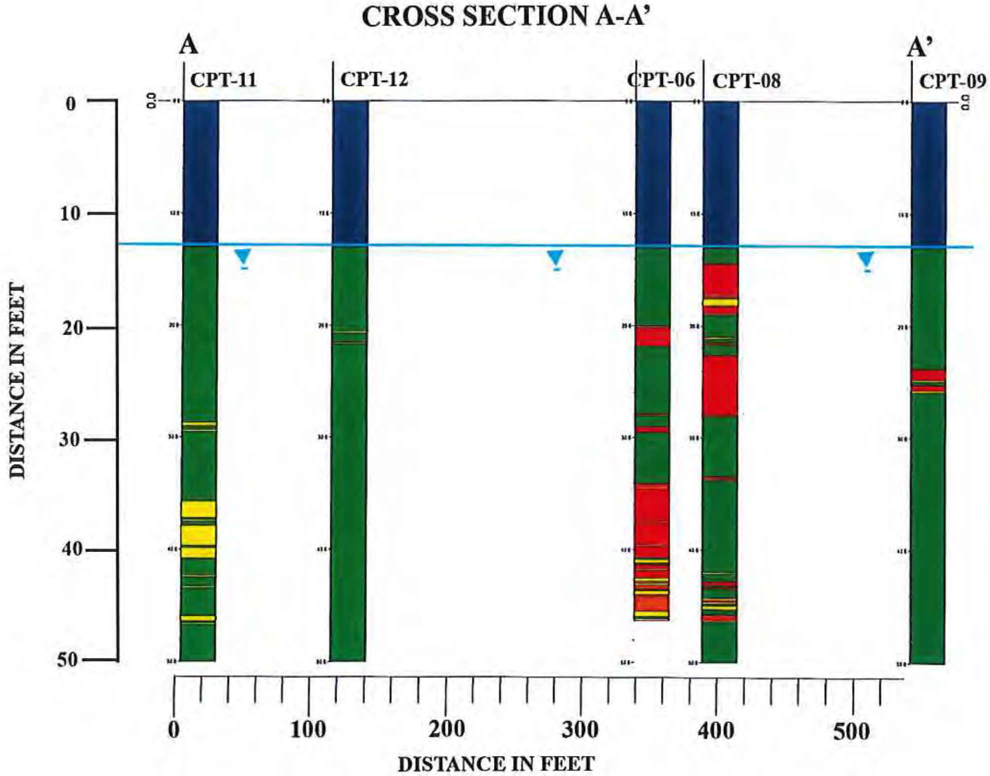
4.4 Liquefaction Analysis

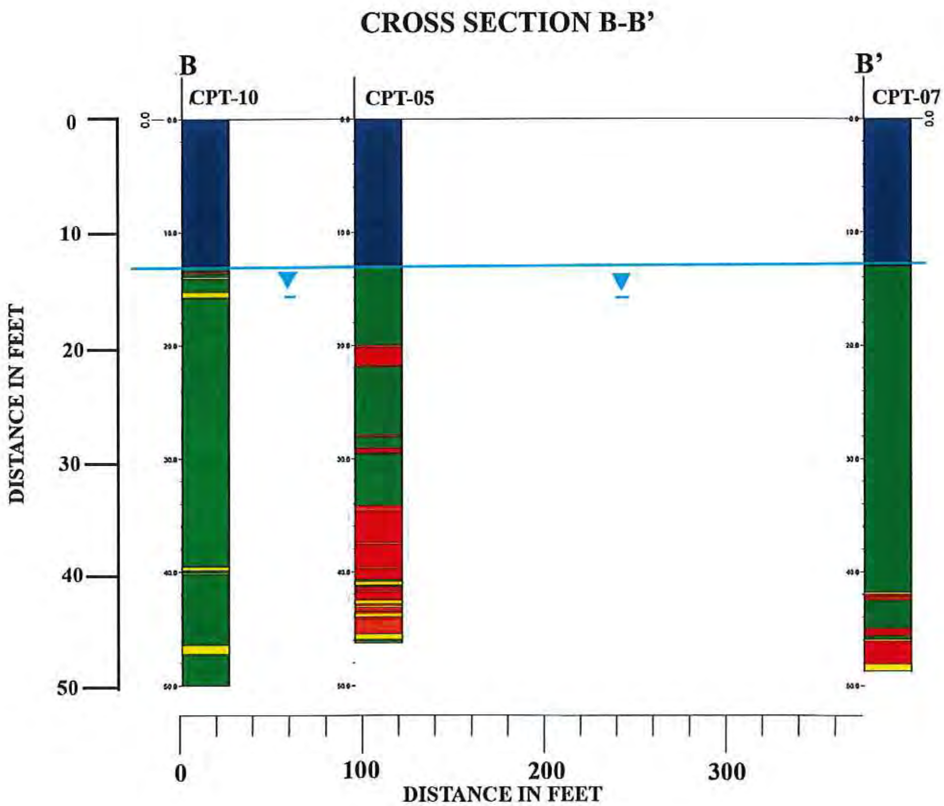
1. GeoSolutions, Inc. utilized computer software program NovoCPT Version 3.32.2014.1209 by Novo Tech Software, which was developed using methods recommended in most recent publication, NCEER Workshop and SP117 Implementation was used to determine the liquefaction and settlement potential of the Site. Seismic load is estimated with Seed's simplified method (Seed, 1971), which uses a Cyclic Stress Ratio (CSR) that is compared to the Cyclic Resistance Ration (CRR) of the soil.

2. CPT soundings of the Site indicated the presence of saturated SAND type soils encountered in a loose to very dense condition at various depths from ground surface (bgs) to termination of CPT soundings at 50 feet bgs. These values are used to determine Factors of Safety (FOS) for isolated layers below ground surface. Overall seismic settlement on the order of 1.0 to 3.5 inches where obtained from the program with (FOS) less than 1.0 for the sand soils encountered at various depths below ground surface. The results from this analysis are summarized in **Appendix B** of this report.

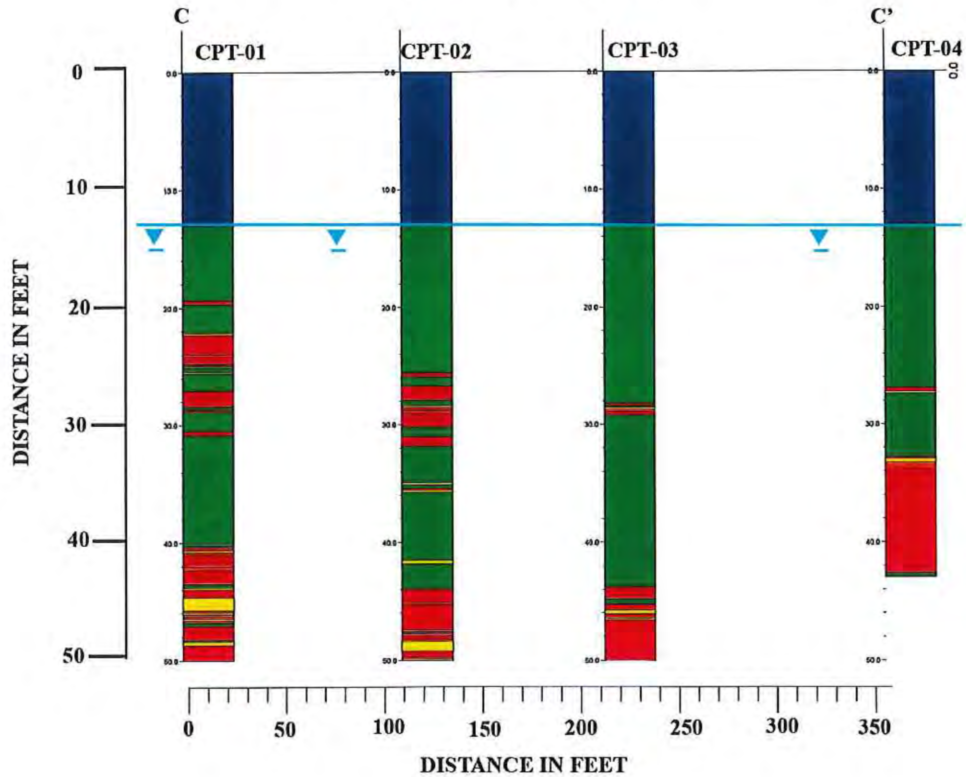
3. Based on the presence of sandy soils, the relative density of the in-situ soils, the depth to groundwater, and the expected ground acceleration caused by the Design Base Earthquake, the potential for seismic liquefaction of Site soils is high. Liquefaction was determined to likely occur in the sandy soil layers between the depths of 13 to 50 feet bgs and may manifest at the surface as seismically induced settlements. **Seismically induced settlements were estimated to be on the order of 1.0 to 3.5 inches.**

Figure 5: Liquefaction Potential





CROSS SECTION C-C'



5.0 GENERAL SOIL-FOUNDATION DISCUSSION

The results of our on-site investigation show poor sub-surface soil conditions and a shallow groundwater table located at approximately 13 feet below ground surface. Under seismic loadings, the soils below the groundwater interface may liquefy. The result of liquefaction would be settlements on the order of 1.0 to 3.5 inches across the Site. Due to the existing subsurface conditions, a post tension type foundation system may be considered for the proposed commercial and residential structures. As an alternative, a graded engineered fill pad may be constructed for the proposed commercial and residential structures with all foundations excavated into engineered fill. 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.

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 and potential for debris resulting from demolition and removal of the existing structures and trees.

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

3. The potential for settlement due to seismic liquefaction. Several layers of soil were identified as potentially liquefiable between depths of 13 and 50 feet bgs. The low densities encountered, along with the low fines content of the soil and saturated conditions indicate that these layers may potentially be liquefiable, manifesting at the surface as dynamic settlements. Seismically induced settlements were estimated to be on the order of 1.0 to 3.5 inches.

6.1 Preparation of Building Pad

1. It is anticipated that graded engineered fill pads will be developed for residential and commercial structures within the proposed development with footings founded in engineered fill.

2. The ground surface within the development areas should be prepared for grading by removing the existing site improvements, foundations, vegetation, debris, disturbed soils, and other deleterious materials. Due to the presence of expansive soils within the surface and near-surface soils, removal of fractured, loose soil is recommended to expose competent native materials prior to fill placement.

3. For the development of an engineered fill pad, the native material should be over-excavated at least 48 inches below existing grade, 24 inches below the bottom of the footings, to competent material, or to one-half the depth of the deepest fill(measured from the bottom of the deepest footing); whichever is greatest. The limits of over-excavation should extend a minimum of 5 feet beyond the perimeter foundation, to property lines or existing improvements, whichever is least. The exposed surfaces should be scarified to a depth of 6 inches, moisture conditioned to 3 percent over optimum moisture content, and compacted to a minimum relative density of 90 percent (ASTM D1557-07). The over-excavated material may then be processed as engineered fill. Refer to Figure 6: Sub-Slab Detail for under-slab drainage material and **Appendix E** for more details on fill placement.

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

5. As an alternative, and to reduce the potential for moisture loss from within the engineered fill pad areas, replacement of the upper 24 inches of building pad soils with an approved non-expansive import material, such as a Class II/III aggregate sub-base placed as engineered fill, is recommended. The non-expansive material reduces the potential for movement of concrete slabs and exterior flatwork as well as the potential for desiccation cracks from soil shrinkage, to form within the pad areas.

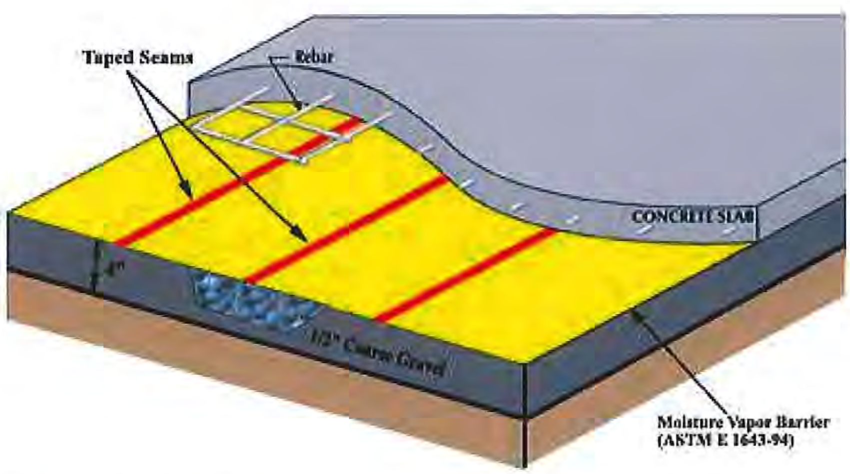


Figure 6: Sub-Slab Detail

GeoSolutions, INC.

6.2 Preparation of Paved Areas

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

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.

6.3 Pavement Design

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

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

3. The following table provides the recommended Hot Mix Asphalt (HMA) pavement sections based on an estimated **R-Value of less than 5** and Specification 7110 – Flexible Pavement Elements, from the City of San Luis Obispo

Table 2: Recommended Pavement Structural Sections

Traffic Index	Structural Section Thickness in Inches	
	HMA/AC	AB
4.5	2.5	10.0
5.5	3.5	11.0
6.5	4.0	14.0
7.0	4.25	15.5
8.5	5.5	19.0
9.5	6.5	21.5
HMA = Hot Mix Asphalt meeting Caltrans Specification HMA Type A ½ inch mix AB = Aggregate Base meeting Caltrans Specification for Class 2 aggregate base (R-Value = 78 Min)		

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

5. In order to minimize the potential for cracking of the pavement surfaces at the Site due to lateral movement of the base courses during expansive shrink-swell cycles of the sub-grade materials, GeoSolutions, Inc. recommends that a laterally-reinforcing biaxial geogrid, such as Tensar BX1100, Tenax MS220, Syntec SBX11, or equivalent, be installed between the prepared sub-grade and base materials at the Site.

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

6.4 Interlocking Concrete Pavers

1. Due to the expansion potential of the site soils and the potential adverse effects that expansive soils have on reinforced concrete flatwork and concrete pavement (heaving, cracking, settlement, etc.) interlocking concrete pavers may be utilized as the finish surface in these areas. Minimum recommended structural sections for driveways, sidewalks and alley approaches are provided based on; on-site soil properties, anticipated loading/use, and ICPI (Interlocking Concrete Pavement Institute) Technical Specifications.

2. In general we recommend a minimum section of Class II aggregate base over geotextile fabric in areas where vehicle loading is anticipated, and a minimum section of aggregate base material over prepared sub-grade in sidewalk areas. We recommend the use of geotextile fabric between the prepared sub-grade and Class II aggregate base materials in areas to receive vehicle loading to prevent the sub-grade soil from being pressed into the aggregate base under loads, especially during periods of wet weather, reducing the potential for ruts to form within the driveways. When geotextile fabrics are used they preserve the load bearing capacity of the base over a greater period of time than placement without them.

3. For sidewalk areas, we recommend a minimum of 6 inches of aggregate base material. The aggregate base material may consist of; Class II aggregate base, Class III aggregate sub-base, or and approved decomposed granite (D.G.) material. For construction of the paver section in sidewalk areas, the sub-grade soil should be moisture conditioned to a minimum of 3 percent over optimum moisture value and compacted to a minimum relative density of 90 percent (ASTM D1557-07 test method). Care should be exercised to maintain the moisture in the sub-grade soils prior to the placement of aggregate base material; sub-grade soils in a dry condition or with desiccation cracks will require re-processing prior to the placement of aggregate base material. The aggregate base material should be moisture conditioned to near optimum moisture content and compacted to a minimum relative density of 90 percent.

4. For driveways and alley approach areas, we recommend a minimum of 12 inches of Class II aggregate base material over an approved woven geotextile fabric such as Mirafi 500x or equivalent, placed on the prepared sub-grade soil. For construction of the paver section in driveway areas, the sub-grade soil should be moisture conditioned to a minimum of 3 percent over optimum moisture value and compacted to a minimum relative density of 95 percent (ASTM D1557-07 test method). Care should be exercised to maintain the moisture in the sub-grade soils prior to the placement of aggregate base material; sub-grade soils in a dry condition or with desiccation cracks will require re-processing prior to the placement of geotextile fabric. The fabric should be installed on the surface of the

prepared sub-grade, laid flat with no wrinkles on the bottom surface. Within the excavated area for the structural section, the fabric should be turned up the sides of the opening, covering the sides of the aggregate base layer. Where fabric requires overlap, a minimum overlap of 2.0 feet should be used. When the aggregate base material is placed on the fabric, care should be taken to avoid vehicle tires from contacting the fabric, to avoid wrinkling or tearing of the fabric. The aggregate base material should then be moisture conditioned to near optimum moisture content and compacted to a minimum relative density of 95 percent.

Table 3: Driveway and Sidewalk Base Sections

Location	Minimum Aggregate Base Section Thickness	Minimum Sub-grade Compaction	Minimum Aggregate Base Compaction	Geotextile Fabric
Sidewalks	6.0 inches	90 percent	90 percent	None
Driveways and Alley Approaches	12.0 inches	95 percent	95 percent	Mirafi 500x or equal

5. Installation of the bedding sand, pavers and joint sand should be performed in accordance with manufacturers' specifications and ICPI specifications.

6.5 Conventional Foundations

1. Conventional continuous and spread footings with grade beams may be used for support of the proposed structure. Isolated pad footings are not allowed. Foundations must be designed in accordance to section 1808.6.2, 2013 CBC, Foundations on Expansive Soils.

2. Minimum footing and grade beam sizes and depths in engineered fill should conform to the following table, as observed and approved by a representative of GeoSolutions, Inc. In addition all foundations should be designed to accommodate the movements in Table 4: Minimum Footing and Grade Beam Dimensions

Table 4: Minimum Footing and Grade Beam Dimensions

Excavated in Engineered Fill		
Building Type	**Minimum Depth Below Lowest Adjacent Grade**	**Minimum Width**
One-Story	24 inches	12 inches
Two-Story	24 inches	12 inches
Three- Story	24 inches	15 inches
Interior Grade Beams	18 inches	12 inches

3. Minimum reinforcing steel for footings and grade beams excavated in engineered fill should be designed by the Structural Engineer and in accordance with Section 1808.6 of the 2013 California Building Code for expansive soils.

4. A representative of this firm should observe and approve all foundation excavations for required embedment depth prior to the placement of reinforcing steel and/or concrete. Concrete should be placed only in excavations that are free of loose, soft soil and debris and that have been maintained in a moist condition with no desiccation cracks present.

5. An allowable dead plus live load bearing pressure of **1,500 psf** may be used for the design of footings founded in engineered fill.

6. A total settlement of less than 1 inch and a differential settlement of less than 1 inch in 30 feet are anticipated. Foundation design should also take into account the overall seismic settlement of 3.5 inches, and seismic differential settlement of 1.75 inches over 30 feet.

7. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the engineered fill and the bottom of the footings. For resistance to lateral loads, a friction factor of **0.30** may be utilized for sliding resistance at the base of footings extending a minimum of 24 inches into engineered fill. A passive pressure of **275-pcf** equivalent fluid weight may be used against the side of shallow footings in engineered fill. If friction and passive pressures are combined to resist lateral forces acting on shallow footings, the lesser value should be reduced by 50 percent.

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

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

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

6.6 Post-Tensioned Slabs

1. As an alternative to a conventional reinforced concrete foundation system with perimeter footings and grade beams, a post-tension foundation system may be utilized to support the proposed structures.

2. Post-tensioned slabs should be designed for total static and seismic settlements of 3.5 inches and a differential settlement of 1.0 inch over 30 feet.

3. Post-tensioned slabs should be designed according to the method recommended in the Design and Construction of Post-Tensioned Slabs-on-Ground (PTI, 2012 PTI DC 10.5-12). As a guideline, the following soil design criteria for the post-tensioned slab foundations may be used:

Table 5: Post-Tension Foundation Criteria

POST-TENSION FOUDATION DESIGN CRITERIA					
		Center Lift All Perimeter Beam Conditions		**Edge Lift** 24 Inch Deep Edge Beam	
Expansion Potential	Structure Type	Em (ft.)	Ym (in.)	Em (ft.)	Ym (in.)
Medium	**Commercial**	**6.0**	**1.60**	**2.9**	**2.52**
	Multi-Family	**6.0**	**1.74**	**2.9**	**2.75**
	Single-Family	**6.0**	**1.59**	**2.9**	**2.55**
Footing/Slab Dimensions					
The footing width, depth and structural slab-on-grade thickness should be specified by the architect/engineer based upon the soil parameters provided in this report and the 2013 CBC					
Slab Subgrade Moisture Recommendations					
Medium Expansive Potential		Minimum of 130 percent of optimum moisture content to a depth of 18 inches prior to concrete placement.			

4. The following values were assumed when developing the above design values (Table 2) using the computer program Volflo v1.5: Soil fabric factor $F_F = 1.1$, $K_0 = 0.33$ (drying) 0.67 (wetting); Thornthwaite Moisture Index = -20; constant suction value pF = 3.8; depth to constant suction = 13 feet (2); post equilibrium case assumed with wet (swelling) cycle going from 4.5 pF to 2.9 pF and drying (shrinking) cycle going from 2.9 pF to 4.5 pF. See **Appendix F**, Volflo 1.5 for summary results.

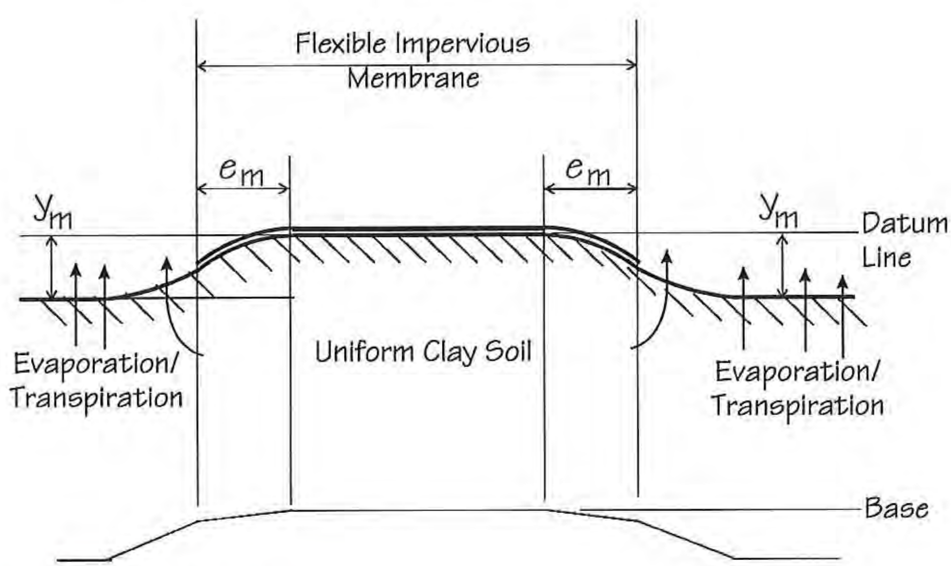


Figure 7: Center Lift Diagram

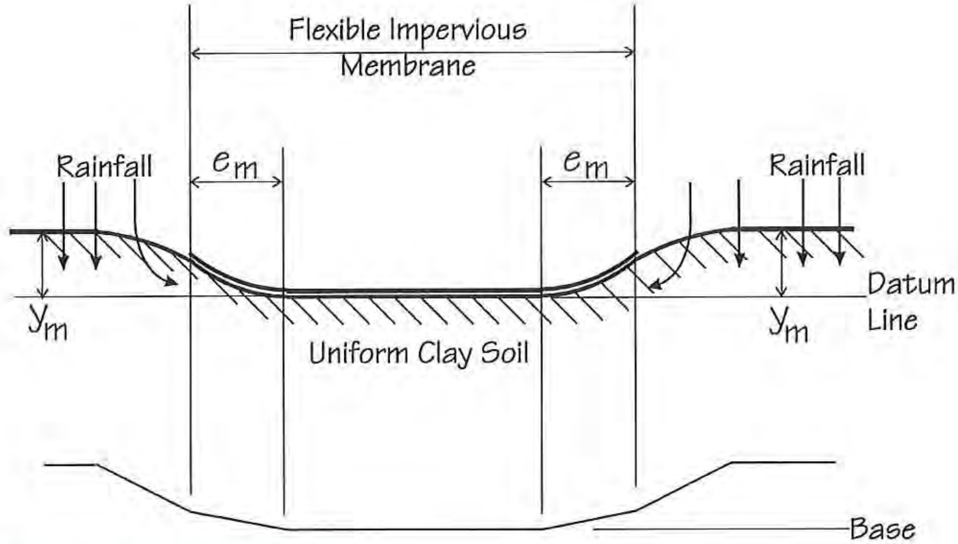


Figure 8: Edge Lift Diagram

5. These values should be confirmed after grading based upon soil conditions at subgrade level on the building pads. The post-tensioned slabs should be designed to impose a maximum allowable bearing pressure of 1,000 pounds per square foot (psf) for dead-plus-live loads. This value may be increased by one-third when considering total loads including wind or seismic loads.

6. A minimum slab thickness of 9 inches is recommended. The perimeter should be thickened to at least 12 inches, and the minimum backfill height of soil against the slab at the perimeter should be 6 inches. The final foundation plans should be reviewed by the Soils Engineer when they become available to verify conformance with these recommendations.

7. Provided the above recommendations are implemented into the design of the proposed structures, a total settlement of less than 1 inch and a differential settlement of less than 1 inch in 30 feet are anticipated.

6.7 Slab-On-Grade Construction

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

2. Concrete slabs-on-grade should be a minimum of 4 inches thick and should be reinforced with No. 3 reinforcing bars placed at 12 inches 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. The aforementioned reinforcement may be used for anticipated uniform floor loads not exceeding 200 psf. If floor loads greater than 200 psf are anticipated, a Structural Engineer should evaluate the slab design.

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

4. Where concrete slabs-on-grade are to be constructed, the slabs should be underlain by a minimum of four inches of clean free-draining material, such as a ½ inch coarse aggregate mix, to serve as a cushion and a capillary break. Where moisture susceptible storage or floor coverings are anticipated, a 15-mil Stego Wrap membrane (or equivalent installed per manufacturer's specifications) should be placed between the free-draining material and the slab to minimize moisture condensation under the floor covering. See Figure 6: Sub-Slab Detail for the placement of under-slab drainage material. It is suggested, but not required, that a two-inch thick sand layer be placed on top of the membrane to assist in the curing of the concrete, increasing the depth of the under-slab material to a total of six inches. The sand should be lightly moistened prior to placing concrete.

5. It should be noted that for a vapor barrier installation to conform to manufacturer's specifications, sealing of penetrations, joints and edges of the vapor barrier membrane may be required. 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.

6.8 Retaining Walls

1. Retaining walls should be designed to resist lateral pressures from adjacent soils and surcharge loads applied behind the walls. We recommend using the lateral pressures presented in Table 6: Retaining Wall Design Parameters and Figure 9: Retaining Wall Detail for the design of retaining walls at the Site. The Active Case may be used for the design of unrestrained retaining walls, and the At-Rest Case may be used for the design of restrained retaining walls.

Table 6: Retaining Wall Design Parameters

Lateral Pressure and Condition	Equivalent Fluid Pressure, pcf
Static, Active Case, Native ($\gamma' K_A$) Static, Active Case, Import Sand or Gravel ($\gamma' K_A$)	70 35
Static, At-Rest Case, Native ($\gamma' K_O$) Static, At-Rest Case, Import Sand or Gravel ($\gamma' K_O$)	85 50
Static, Passive Case, Engineered Fill ($\gamma' K_P$)	275

2. The above values for equivalent fluid pressure are based on retaining walls having level retained surfaces, having an approximately vertical surface against the retained material, and retaining granular backfill material or engineered fill composed of native soil within the active wedge. See Figure 9: Retaining Wall Detail and Figure 10: Retaining Wall Active and Passive Wedges for a description of the location of the active wedge behind a retaining wall.

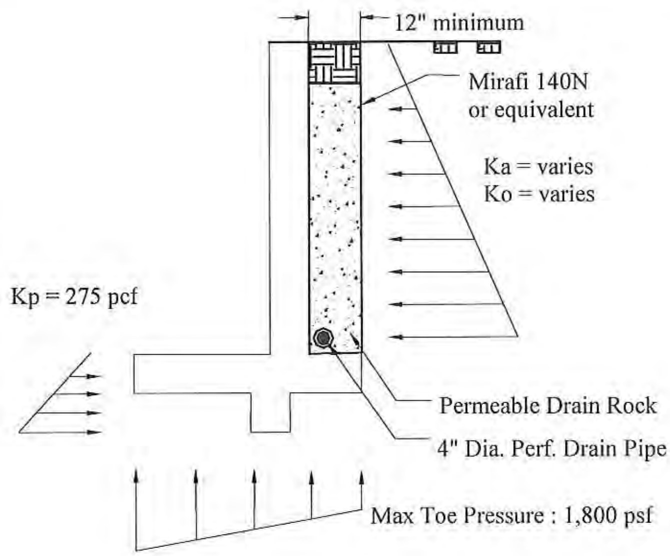


Figure 9: Retaining Wall Detail

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.

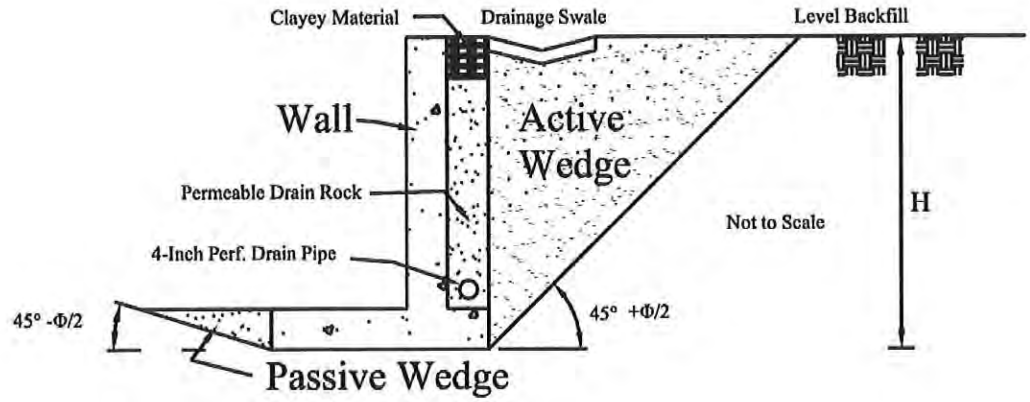


Figure 10: Retaining Wall Active and Passive Wedges

5. Retaining wall foundations should be founded a minimum of 24 inches below lowest adjacent grade in engineered fill as observed and approved by a representative of GeoSolutions, Inc. A coefficient of friction of **0.30** may be used between engineered fill and concrete footings. Project designers may use a maximum toe pressure of **1,800 psf** for the design of retaining wall footings founded in engineered fill.

6. Seismic active lateral earth pressure values were determined using the Pseudostatic Method and the Design a_{max}. See section 4.1 for a description of the analysis used to determine the Design a_{max}. The seismic at-rest lateral earth pressure value was determined by multiplying the seismic active lateral earth pressure value by approximately 1.5. The dynamic increment in lateral earth pressure due to earthquakes should be considered during the design of retaining walls at the Site. Retaining walls greater than 6 feet in height 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 $^{1}/_{3}$H above the base of the retaining wall, where H is the height of the retaining wall.

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

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

9. In addition to the static lateral soil pressure values reported in Table 6: Retaining Wall Design Parameters, the retaining walls at the Site should be designed to support any design live load, such as from vehicle and construction surcharges, etc., to be supported by the wall backfill. If construction vehicles are required to operate within 10 feet of a retaining wall, supplemental pressures will be induced and should be taken into account in the design of the retaining wall.

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.

6.9 Exterior Concrete Flatwork

1. Due the presence of highly expansive surface soils within the proposed development areas, there is a high potential for considerable soil movement and flatwork if conventional measures are used, such as the placement of 4 to 6 inches of imported sand materials placed beneath concrete flatwork. Heaving and cracking are anticipated to occur. To reduce the potential for movement associated with expansive soils, we recommend the placement of a minimum of 24 inches of approved non-expansive import material placed as engineered fill beneath the flatwork.

2. Minimum flatwork reinforcement should consist of No. 3 reinforcing steel bars placed at 24 inches on-center each-way at or slightly above the center of the structural section.

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

7.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations contained in this report are based on a limited number of borings/soundings 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 City 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. 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.

4. 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.

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

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

7. In addition to the construction inspections listed above, section 1705.6 of the 2013 CBC (CBSC, 2013) requires the following inspections by the Soils Engineer for controlled fill thicknesses greater than 12 inches as shown in Table 7: Required Verification and Inspections of Soils:

Table 7: Required Verification and Inspections of Soils

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

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

CPT Logs

FIELD INVESTIGATION

The field investigation was conducted March 11, 2015 using the Middle-Earth Cone Penetration Test (CPT) sounding equipment and a Mobile B-24 drill rig. The surface and sub-surface conditions were studied by advancing twelve CPT soundings and five exploratory borings. 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 was advanced to a maximum depth of 50 feet below ground surface (bgs) 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 were performed in accordance with ASTM D5778-95 (re-approved 2002) standards.

The Mobile B-24 drill rig with a four-inch diameter solid-stem continuous flight auger bored five exploratory borings near the approximate locations indicated on Figure 3: Google Earth Image. The drilling and field observation was performed under the direction of the project engineer. A representative of GeoSolutions, Inc. maintained a log of the soil conditions and obtained soil samples suitable for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System. See the Soil Classification Chart in this appendix.

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

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

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

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

SOIL CLASSIFICATION CHART

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

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

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

CONSISTENCY

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

RELATIVE DENSITY

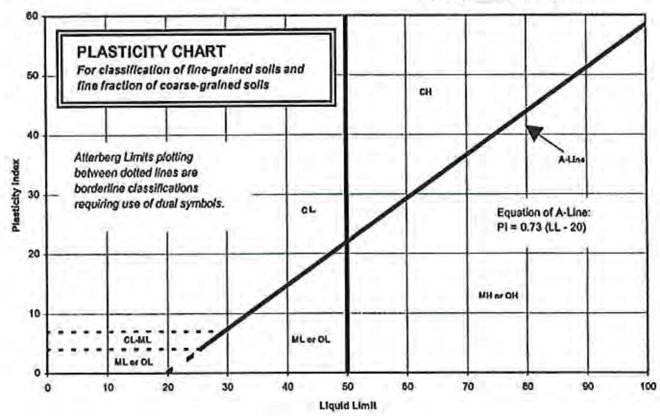
SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/ FOOT +
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	Over 50

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

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

CLASSIFICATIONS BASED ON PERCENTAGE OF FINES

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



Drilling Notes:

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

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

GeoSolutions, Inc.

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1021 West Tama Lane, Suite 105
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BORING LOG

BORING NO. **B-1**

JOB NO. **SL08639-6**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**San Luis Ranch Dalidio**	DRILL RIG:	**Mobile B-24**
DRILLING LOCATION:	**See Figure 2, Site Plan**	HOLE DIAMETER:	**4 Inches**
DATE DRILLED:	**March 11, 2015**	SAMPLING METHOD:	**SPT and CA**
LOGGED BY:	**JK**	HOLE ELEVATION:	**Not Recorded**

▼ Depth of Groundwater: **Not Encountered** Boring Terminated At: **15 feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SANDY FAT CLAY: black, slightly moist, colluvium, stiff	CH											
-2				A					15.2	107.1	69	86.0	36
-4.5	SANDY CLAY: dark olive brown with gravel and some cobbles, alluvium, very stiff	CL		CA	34	34							
-8				B							57	63.0	23
-9.5	stiff			SPT	16	22							
-12	SILTY CLAY: brown with gravel, alluvium	CL											
-14.5	very stiff			SPT	19	23							
-15													

0 -1 -2 -3 -4 -5 -6 -7 -8 -9 -10 -11 -12 -13 -14 -15 -16 -17 -18 -19 -20

GeoSolutions, Inc.

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

BORING LOG

BORING NO. **B-2**

JOB NO. **SL08639-6**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**San Luis Ranch Dalidio**	DRILL RIG:	**Mobile B-24**
DRILLING LOCATION:	**See Figure 2, Site Plan**	HOLE DIAMETER:	**4 Inches**
DATE DRILLED:	**March 11, 2015**	SAMPLING METHOD:	**SPT and CA**
LOGGED BY:	**JK**	HOLE ELEVATION:	**Not Recorded**

▼ Depth of Groundwater: **13 feet** Boring Terminated At: **15 feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SANDY CLAY: dark brown, slightly moist, colluvium, stiff	CL											
-4.5	SANDY CLAY: brown with gravel, alluvium, very stiff	CL		CA	35	35							
-7	SANDY CLAY: dark grayish brown with gravel, very moist, alluvium	CL											
-10	firm			SPT	7	10							
-13	wet			C							38	64.0	17
-14.5				SPT	7	8							

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

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BORING LOG

BORING NO. **B-3**

JOB NO. **SL08639-6**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**San Luis Ranch Dalidio**	DRILL RIG:	**Mobile B-24**
DRILLING LOCATION:	**See Figure 2, Site Plan**	HOLE DIAMETER:	**4 Inches**
DATE DRILLED:	**March 11, 2015**	SAMPLING METHOD:	**SPT and CA**
LOGGED BY:	**JK**	HOLE ELEVATION:	**Not Recorded**

▼ Depth of Groundwater: **12.5 feet** Boring Terminated At: **15 feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SANDY CLAY: dark brown, slightly moist, colluvium, stiff	CL											
-4.5	SANDY CLAY: olive brown with gravel, moist, alluvium, stiff	CL		CA	27	27							
-8				D							62	76.0	22
-9.5				SPT	16	22							
-10	CLAYEY SAND: light brown with gravel, alluvium	SC											
-11.5	SANDY CLAY: light brown, wet, alluvium	CL											
-14	stiff			SPT	12	14							
-15													

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

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BORING LOG

BORING NO. **B-4**

JOB NO. **SL08639-6**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**San Luis Ranch Dalidio**	DRILL RIG:	**Mobile B-24**
DRILLING LOCATION:	**See Figure 2, Site Plan**	HOLE DIAMETER:	**4 Inches**
DATE DRILLED:	**March 11, 2015**	SAMPLING METHOD:	**SPT and CA**
LOGGED BY:	**JK**	HOLE ELEVATION:	**Not Recorded**

Depth of Groundwater: **10 feet** Boring Terminated At: **15 feet**

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SANDY FAT CLAY: very dark gray, slightly moist, colluvium, stiff	CH											
-2				G							73	89.0	38
-3	SANDY CLAY: grayish brown with gravel, moist, alluvium	CL											
-4				E							57	72.0	23
-4.5	stiff			CA	26	26							
-7	SANDY CLAY: dark gray, moist, alluvium	CL											
-9.5	firm			SPT	7	10							
-10	with gravel			F							52	53.0	19
-14.5	stiff			SPT	9	11							
-15													

GeoSolutions, Inc.

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BORING LOG

BORING NO. **B-5**

JOB NO. **SL08639-6**

PROJECT INFORMATION		DRILLING INFORMATION	
PROJECT:	**San Luis Ranch Dalidio**	DRILL RIG:	**Mobile B-24**
DRILLING LOCATION:	**See Figure 2, Site Plan**	HOLE DIAMETER:	**4 Inches**
DATE DRILLED:	**March 11, 2015**	SAMPLING METHOD:	**SPT and CA**
LOGGED BY:	**JK**	HOLE ELEVATION:	**Not Recorded**

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

DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SANDY FAT CLAY: dark grayish brown with gravel, colluvium	CH											
-2				H							79	64.0	30
-3.5	SANDY CLAY: olive brown, dry, alluvium	CL											
-4				I							77	68.0	27
-4.5	very stiff			CA	50/5"								
-7.5	CLAYEY SAND: light brown with gravel, moist, alluvium	SC											
-10	very dense			SPT	25	35							
-14	with some cobbles			SPT	39	48							
-15													

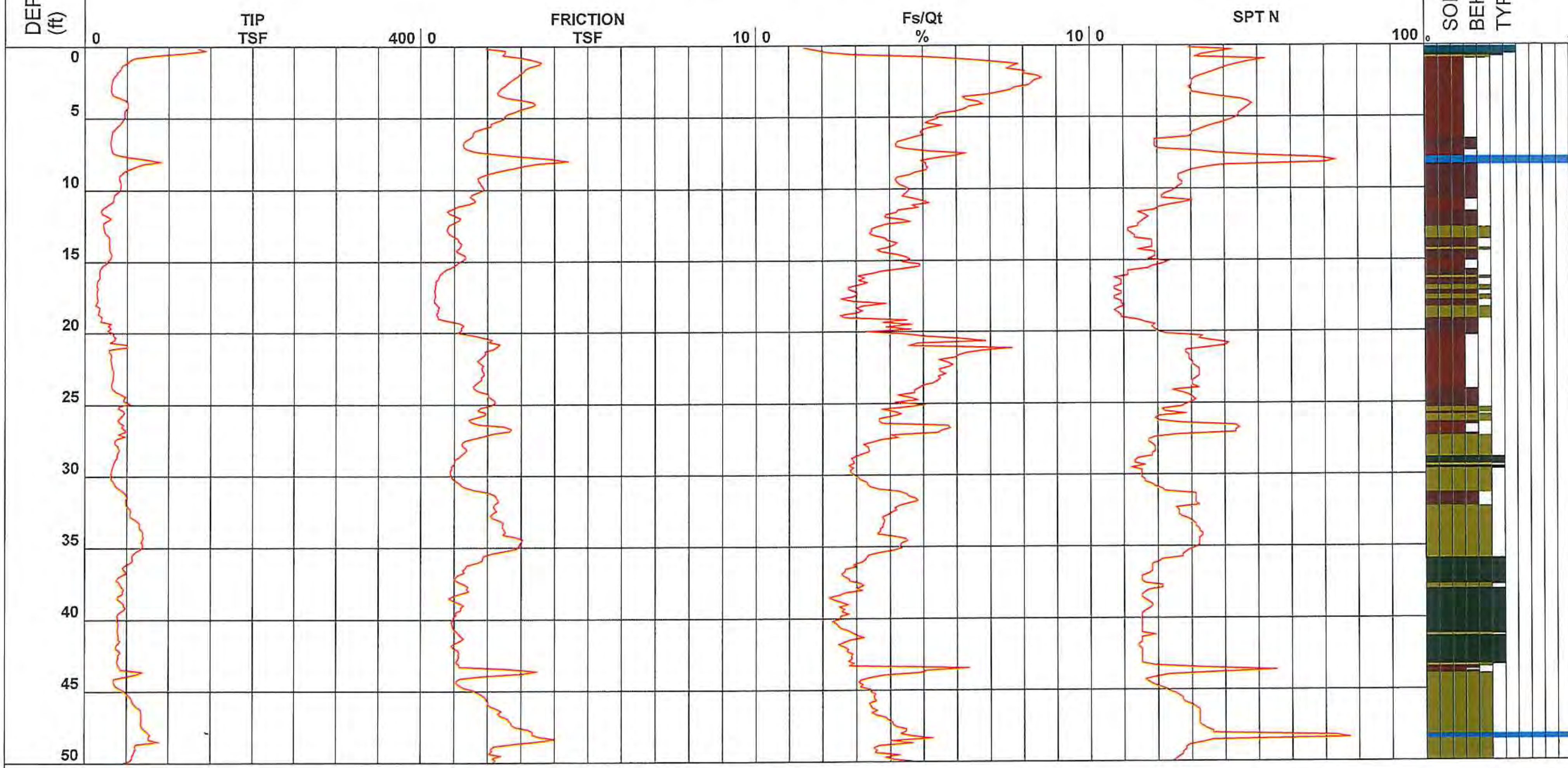
Depth scale: 0, -1, -2, -3, -4, -5, -6, -7, -8, -9, -10, -11, -12, -13, -14, -15, -16, -17, -18, -19, -20

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(301).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-11	Date and Time	3/11/2015 4:25:14 PM	Maximum Depth	50.52 ft
EST GW Depth During Test	13.00 ft				

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

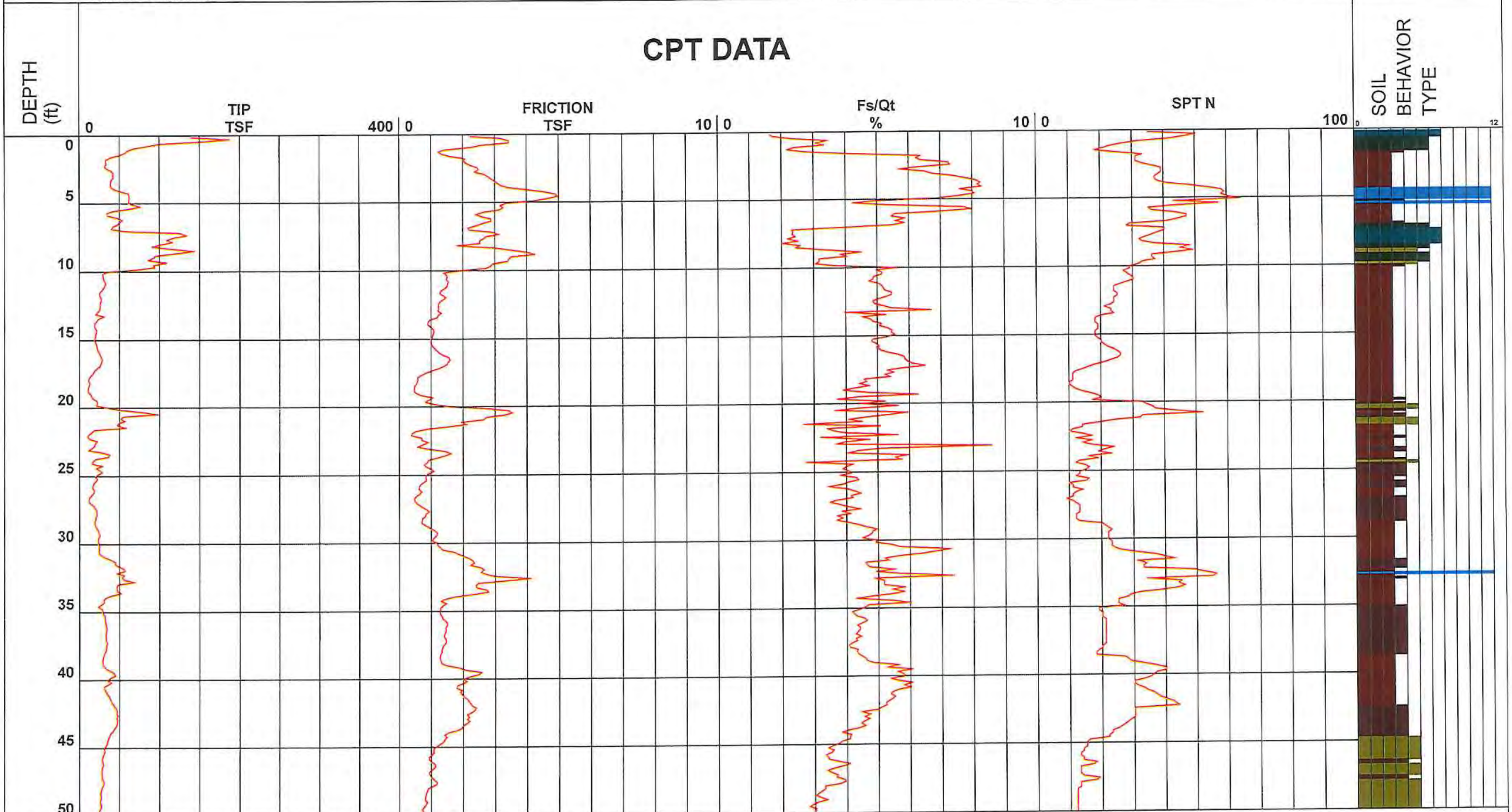
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(302).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-12	Date and Time	3/11/2015 5:09:31 PM	Maximum Depth	50.52 ft
EST GW Depth During Test	13.00 ft				

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

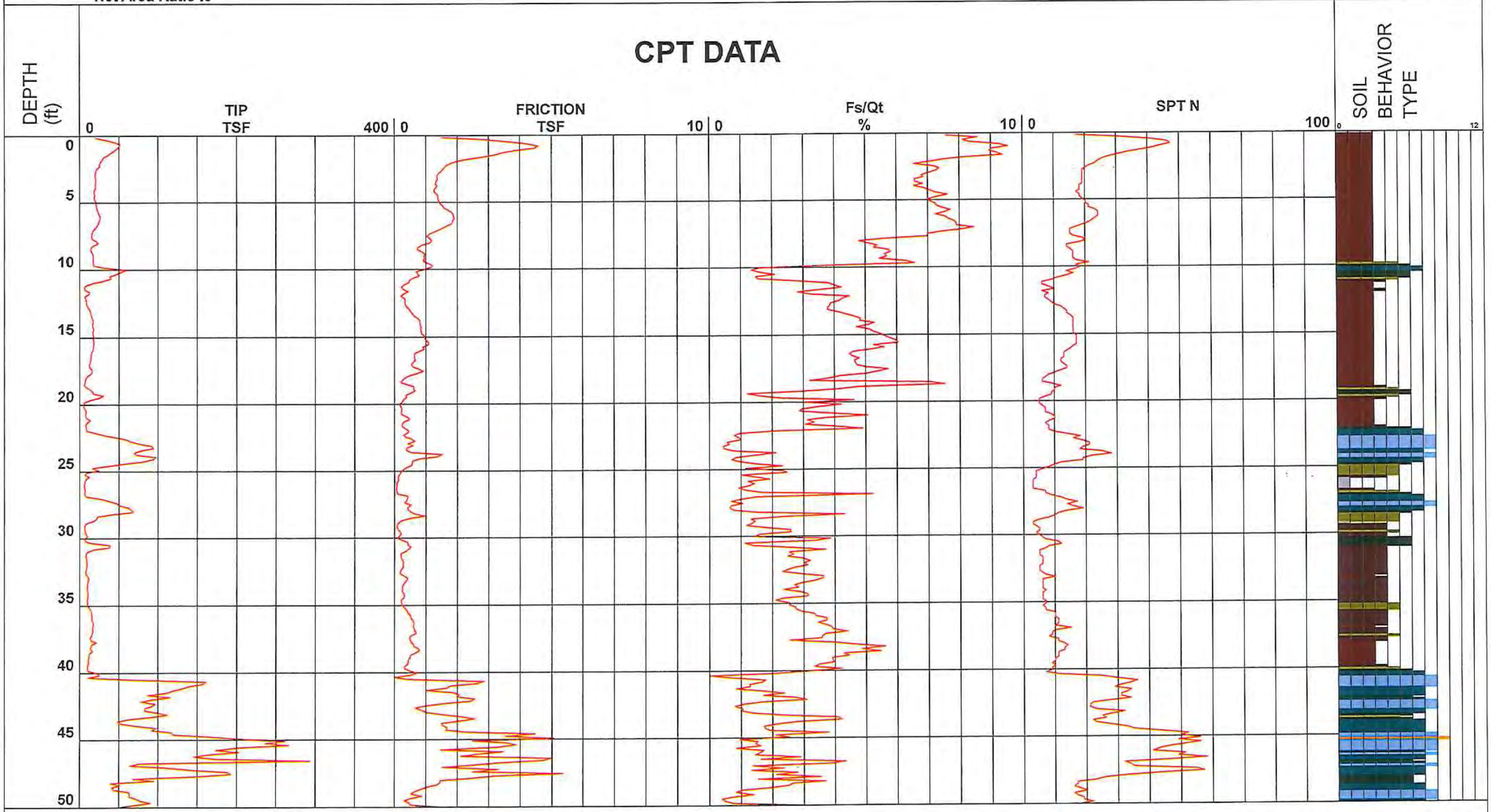
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(291).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-01	Date and Time	3/11/2015 8:41:48 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		13.00 ft			

Net Area Ratio .8

CPT DATA



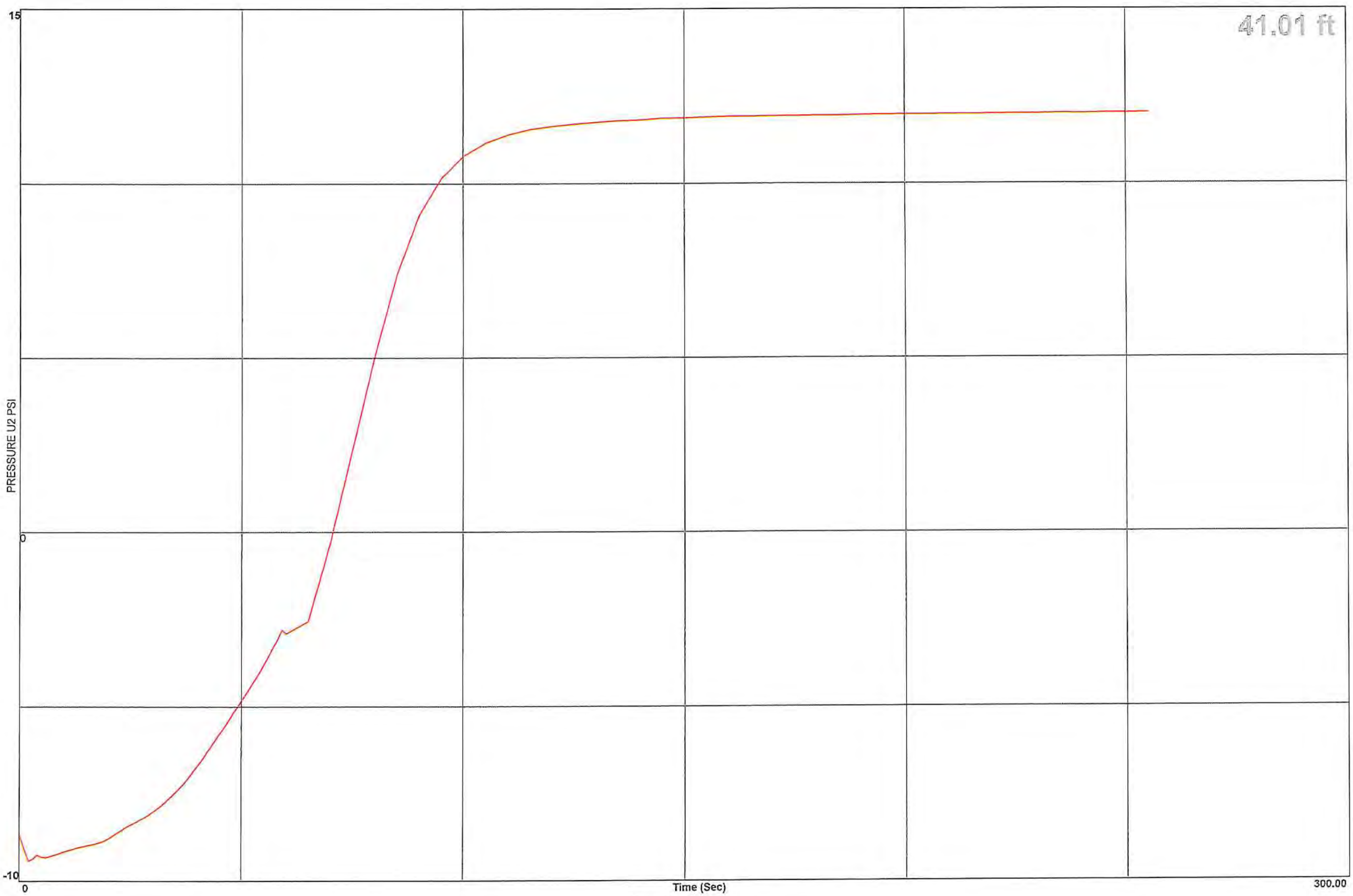
- 1 - sensitive fine grained
- 2 - organic material
- 3 - clay
- 4 - silty clay to clay
- 5 - clayey silt to silty clay
- 6 - sandy silt to clayey silt
- 7 - silty sand to sandy silt
- 8 - sand to silty sand
- 9 - sand
- 10 - gravelly sand to sand
- 11 - very stiff fine grained (*)
- 12 - sand to clayey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

Cone Size 10cm squared

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

Geosolutions Inc

Location	San Luis Ranch/Dalidio	Operator	RC-BH	GPS	
Job Number	SL08639-6	Cone Number	DSG0906		
Hole Number	CPT-01	Date and Time	3/11/2015 8:41:48 AM		
Equilized Pressure	12.0	EST GW Depth During Test	13.2		

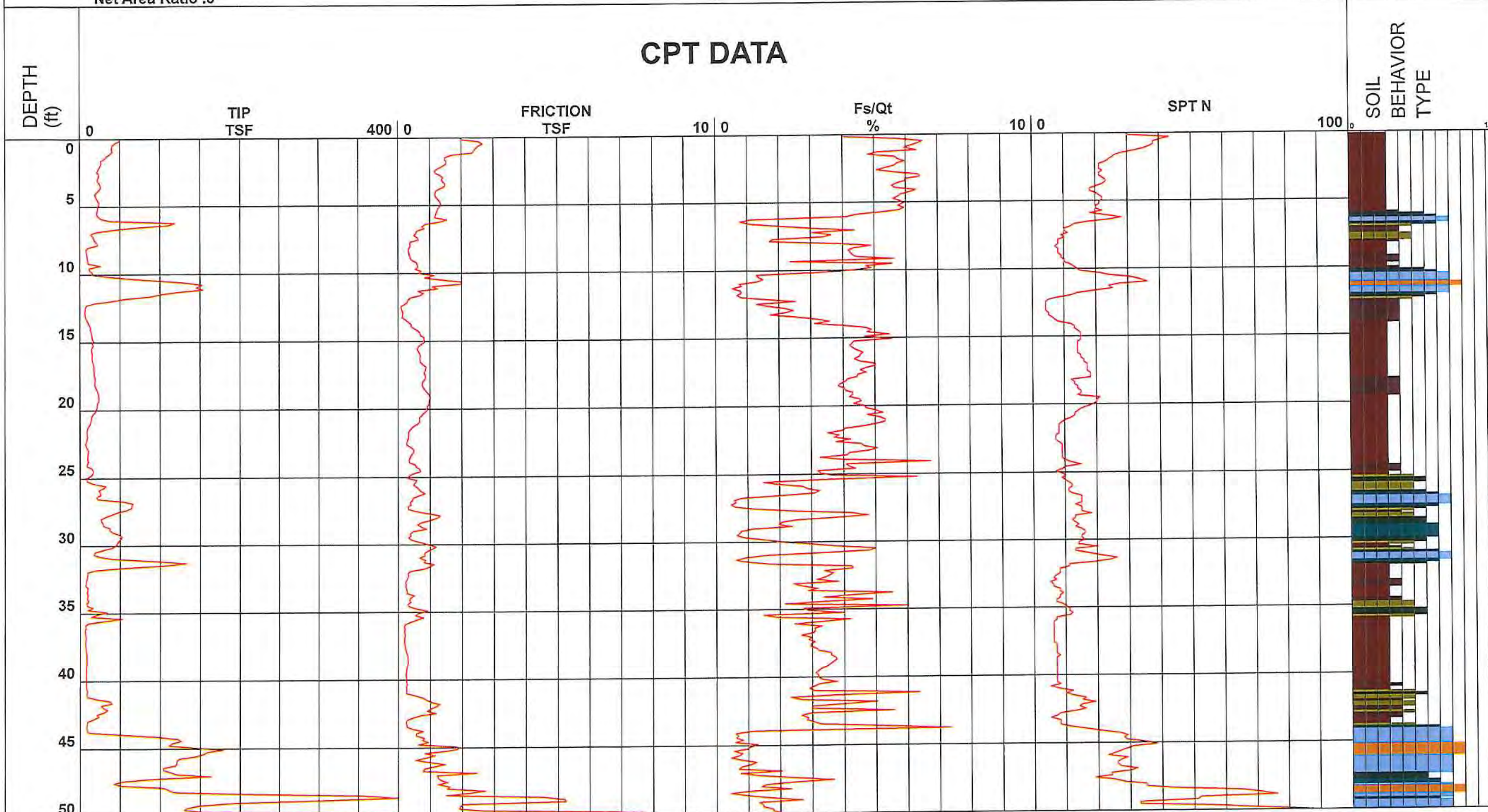


Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(292).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-02	Date and Time	3/11/2015 9:28:35 AM	Maximum Depth	50.52 ft
EST GW Depth During Test		13.00 ft			

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

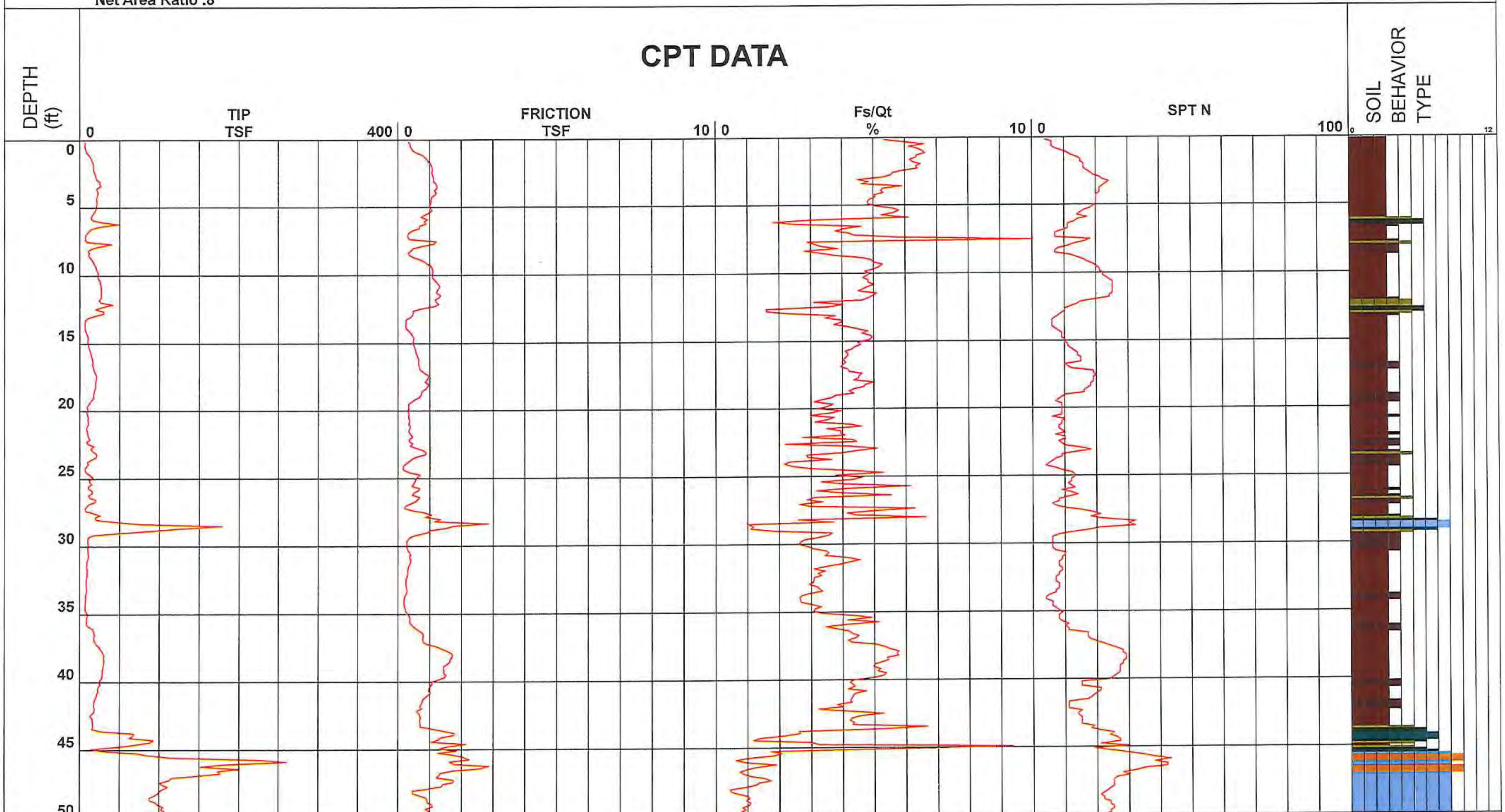
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(293).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-03	Date and Time	3/11/2015 10:13:46 AM	Maximum Depth	50.52 ft
EST GW Depth During Test	13.00 ft				

Net Area Ratio .8

CPT DATA



- 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 (*)

Cone Size 10cm squared

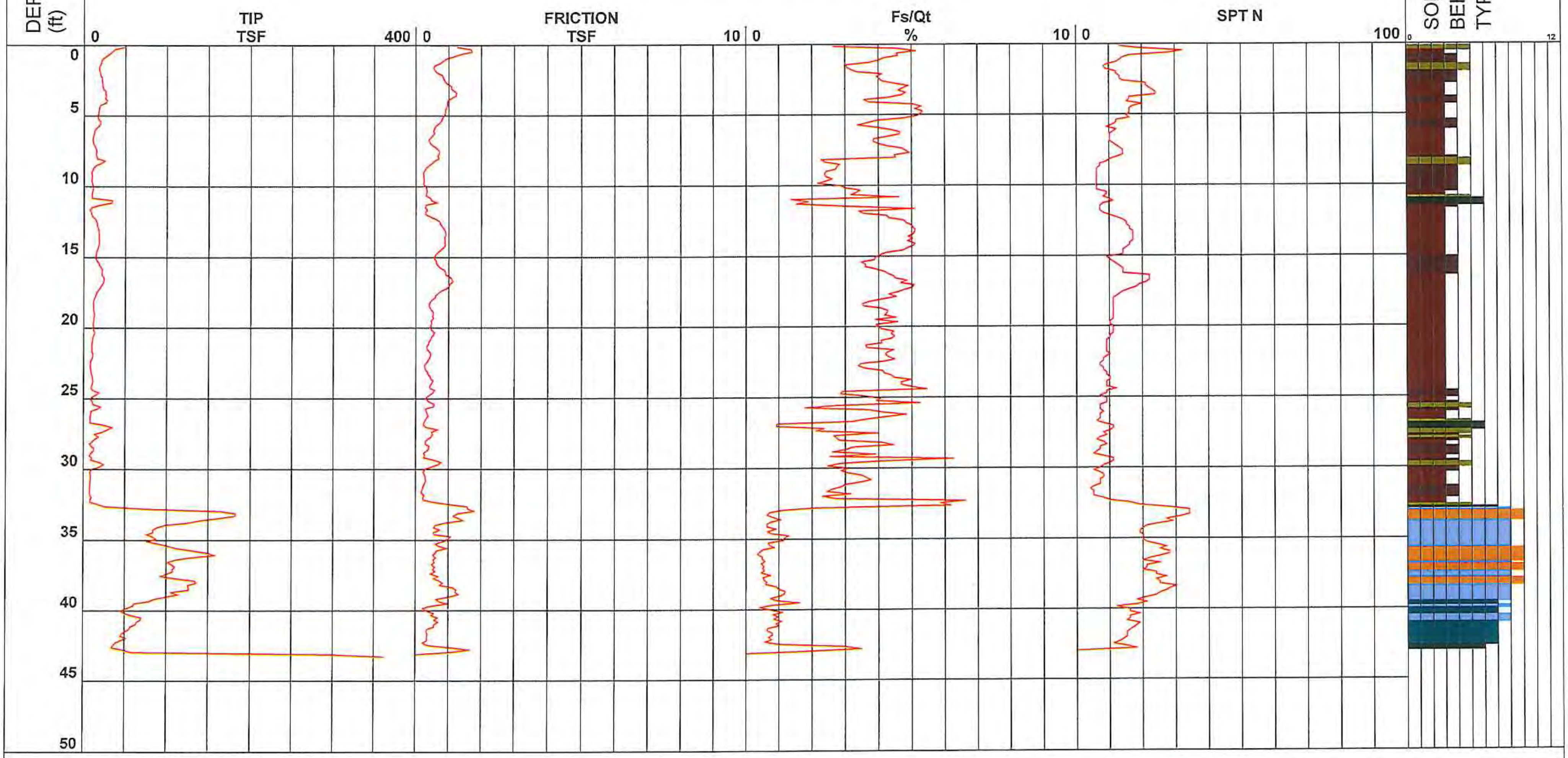
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(294).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-04	Date and Time	3/11/2015 11:07:11 AM	Maximum Depth	43.31 ft
EST GW Depth During Test	13.00 ft				

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

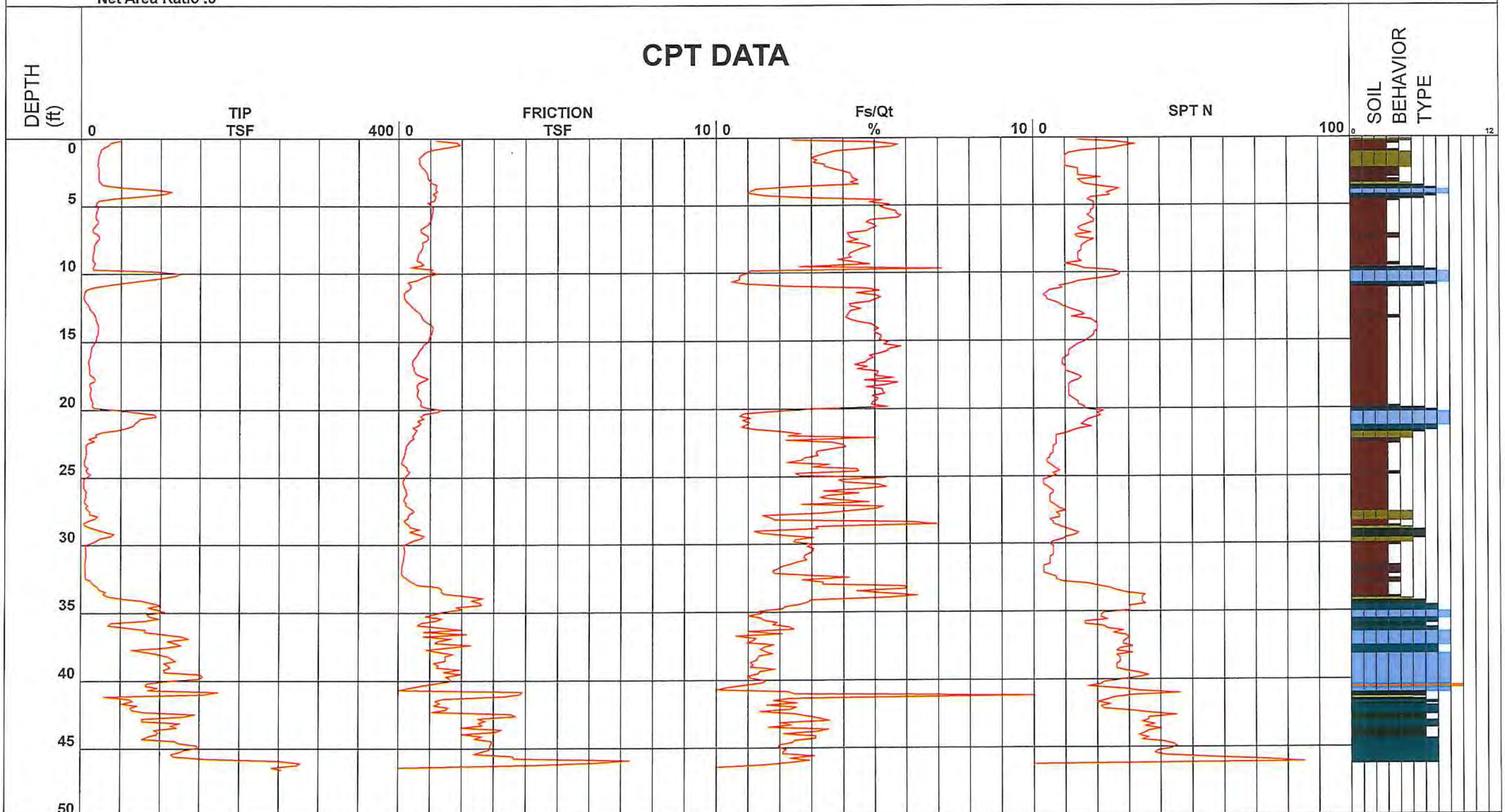
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(295).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-05	Date and Time	3/11/2015 11:56:34 AM	Maximum Depth	46.59 ft
EST GW Depth During Test		13.00 ft			

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

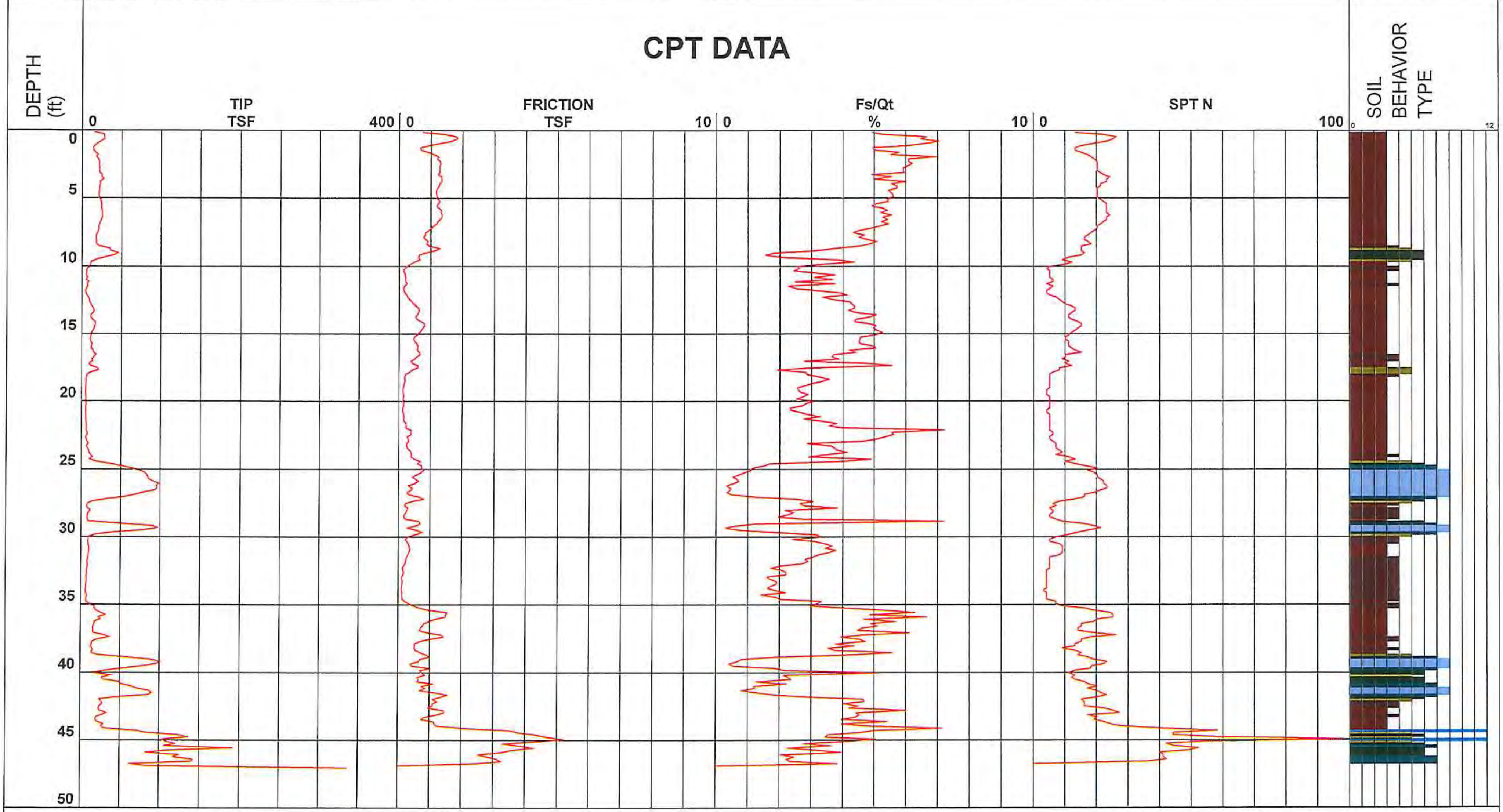
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(296).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-06	Date and Time	3/11/2015 12:31:41 PM	Maximum Depth	47.08 ft
EST GW Depth During Test	13.00 ft				

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

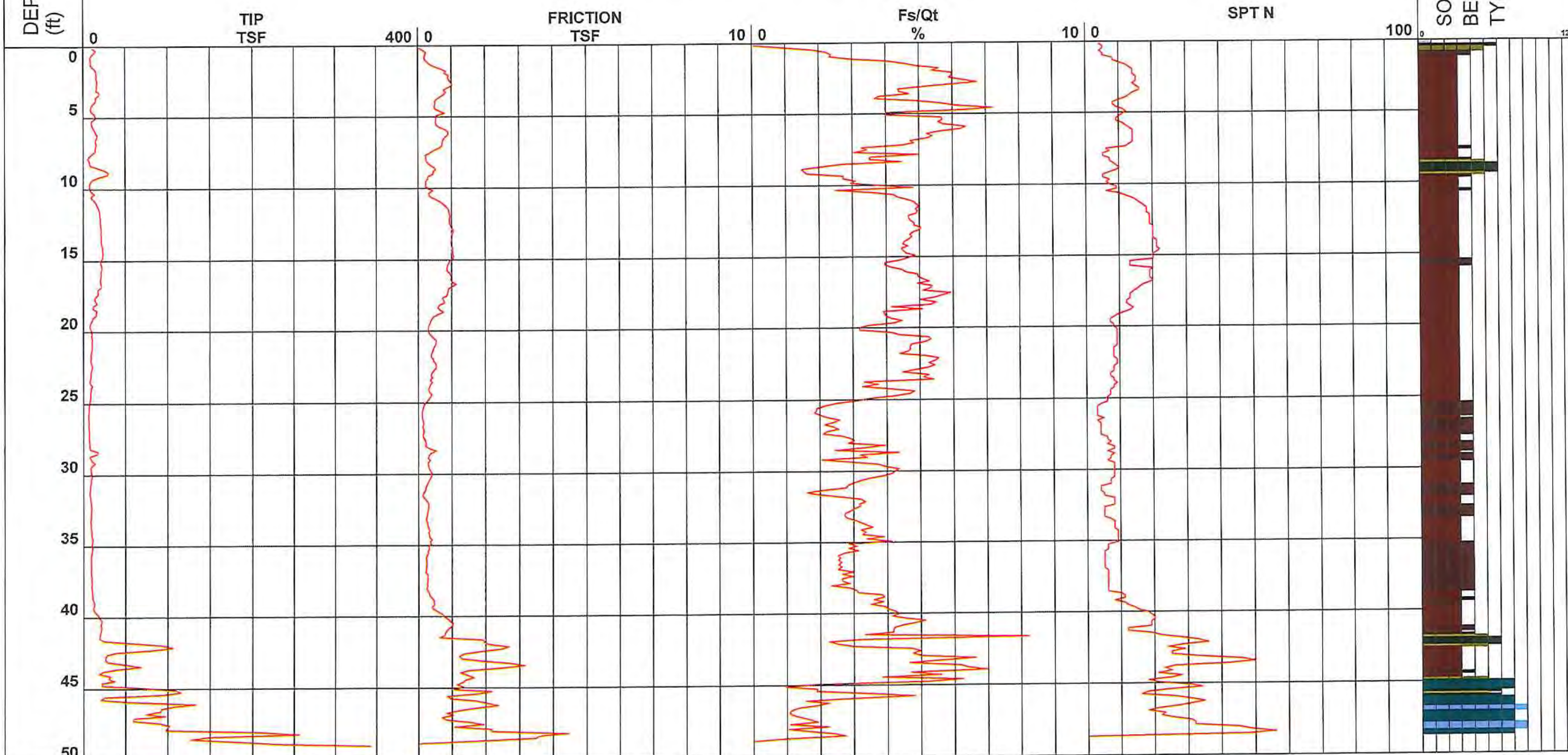
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(297).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-07	Date and Time	3/11/2015 1:11:07 PM	Maximum Depth	49.05 ft
EST GW Depth During Test	13.00 ft				

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

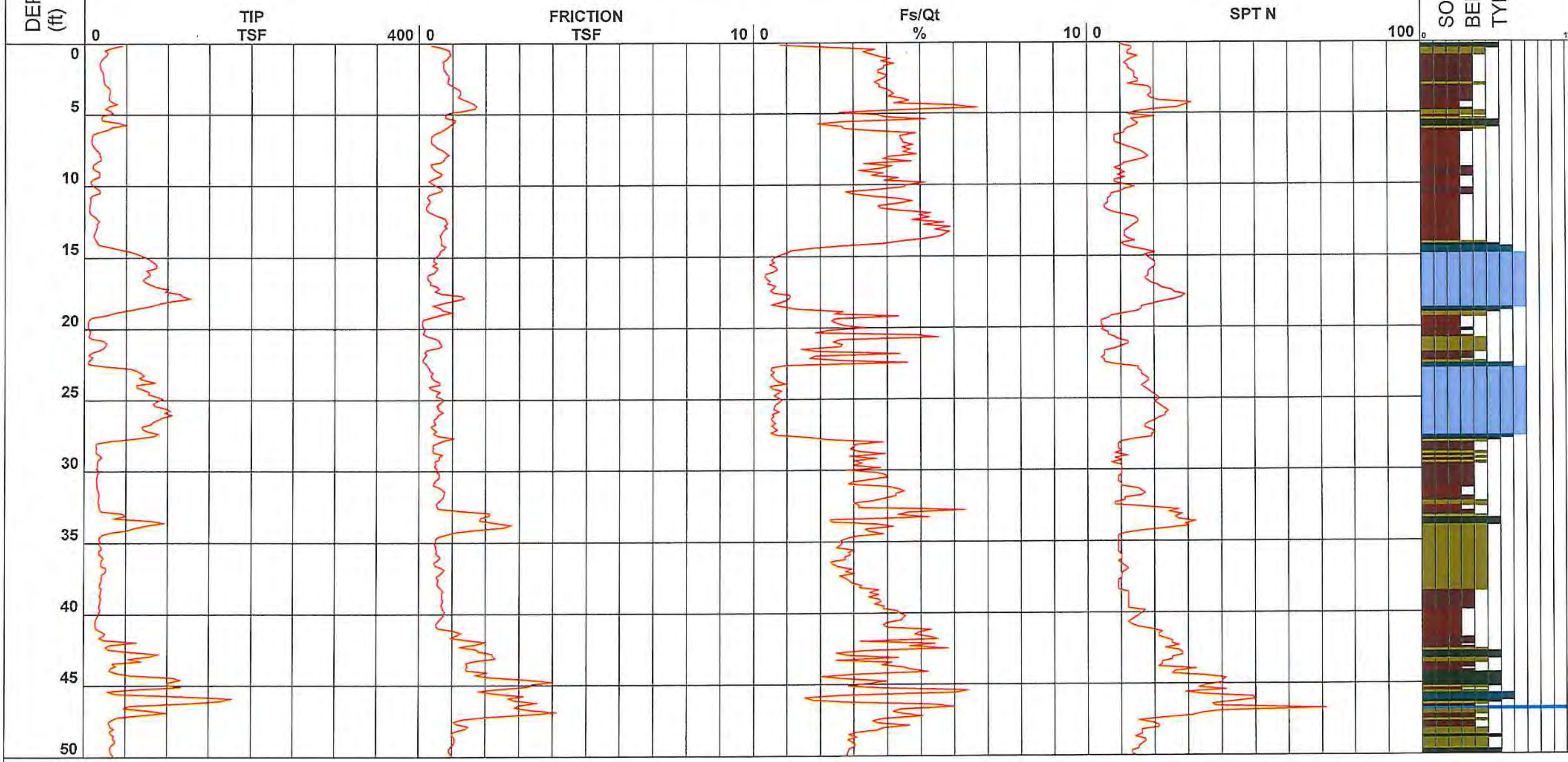
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(298).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-08	Date and Time	3/11/2015 1:54:44 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		13.00 ft			

Net Area Ratio .8

CPT DATA



- 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 (*)

Cone Size 10cm squared

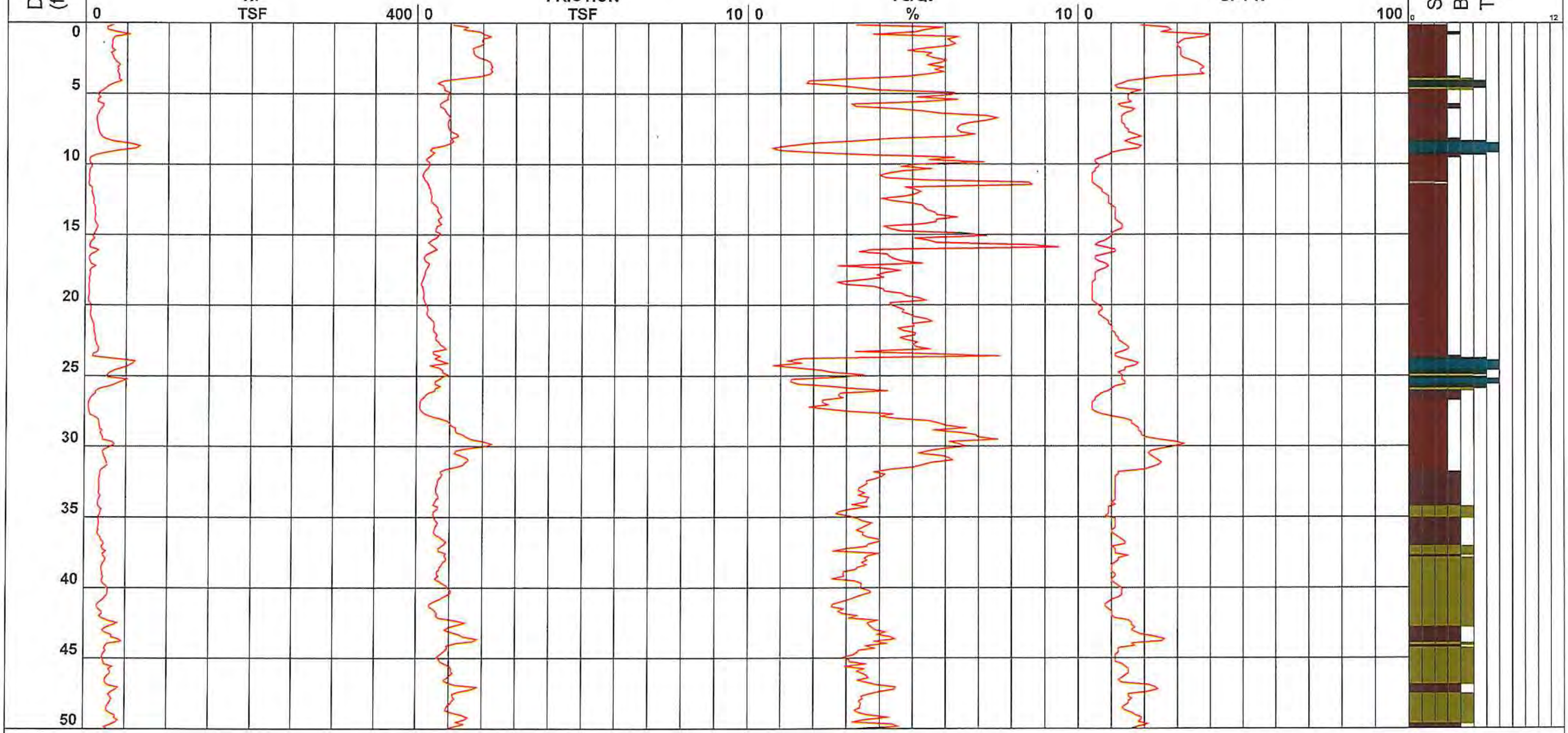
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(299).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-09	Date and Time	3/11/2015 2:38:49 PM	Maximum Depth	50.52 ft
EST GW Depth During Test	13.00 ft				

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

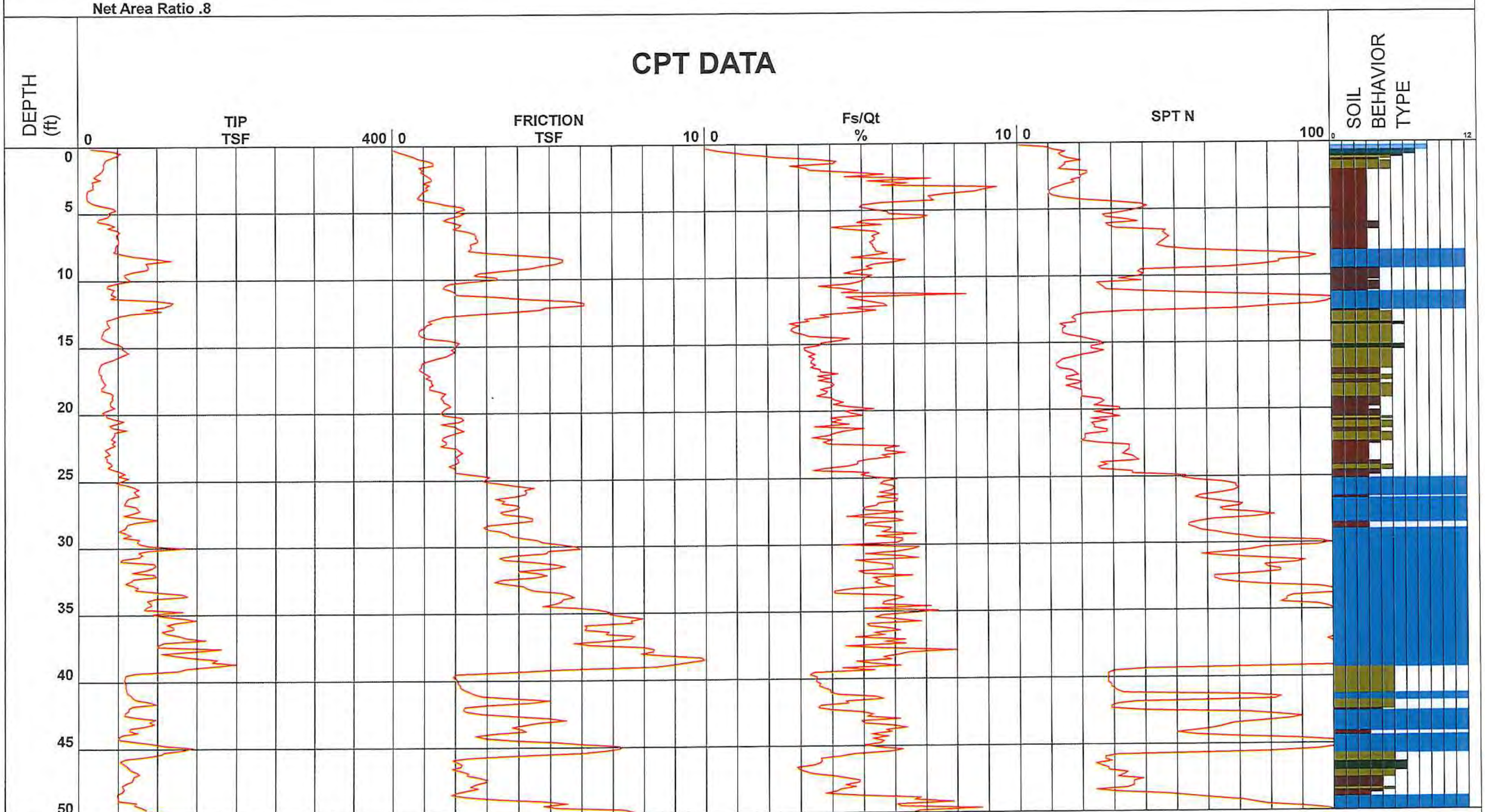
S*Soil behavior type and SPT based on data from UBC-1983

Geosolutions Inc

Project	San Luis Ranch/Dalidio	Operator	RC-BH	Filename	SDF(300).cpt
Job Number	SL08639-6	Cone Number	DSG0906	GPS	
Hole Number	CPT-10	Date and Time	3/11/2015 3:38:31 PM	Maximum Depth	50.52 ft
EST GW Depth During Test		13.00 ft			

Net Area Ratio .8

CPT DATA



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 (*)

Cone Size 10cm squared

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