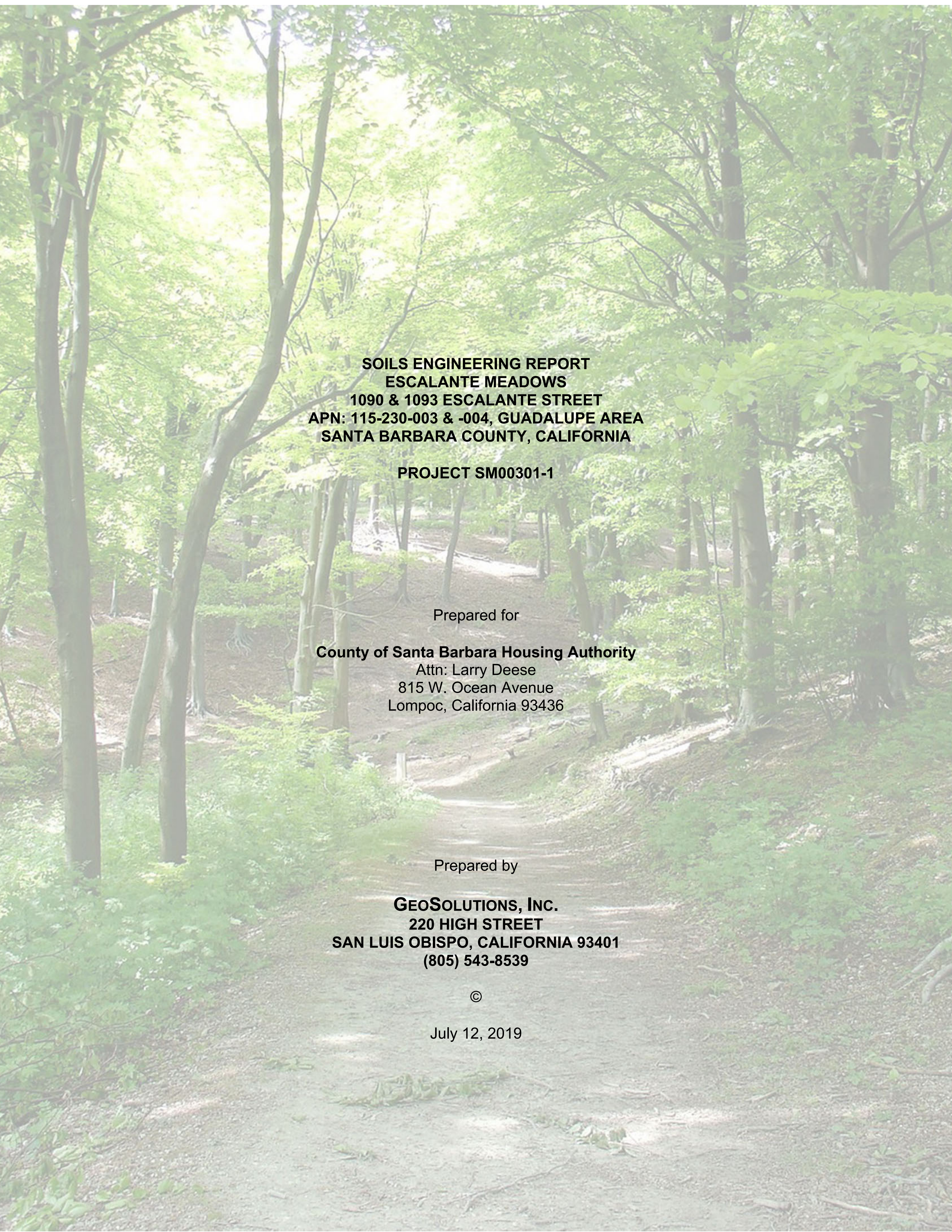

SOILS ENGINEERING REPORT
ESCALANTE MEADOWS
1090 & 1093 ESCALANTE STREET
APN: 115-230-003 & -004, GUADALUPE AREA
SANTA BARBARA COUNTY, CALIFORNIA

PROJECT SM00301-1

Prepared for

County of Santa Barbara Housing Authority
Attn: Larry Deese
815 W. Ocean Avenue
Lompoc, California 93436

Prepared by

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

July 12, 2019

GEO
SOLUTIONS

SOILS ENGINEERING REPORT

July 12, 2019
SM00301-1

Client:
County of Santa Barbara Housing Authority
Attn: Larry Deese
815 W. Ocean Avenue
Lompoc, California 93436

Project name:
Escalante Meadows
1091 & 1093 Escalante Street, APN: 115-230-003 & -004,
Guadalupe, California

Dear Mr. Deese:

This Soils Engineering Report has been prepared for the proposed housing development known as Escalante Meadows to be located at 1091 & 1093 Escalante Street, APN: 115-230-003 & -004, in the Guadalupe area of Santa Barbara, 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.

Due to the potential for liquefaction-induced seismic settlements and presence of highly expansive surface soils at the Site, we recommend that the proposed structures be supported using a structural waffle slab type foundation system founded over geogrid-reinforced engineered fill pad. 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.

Kelly M. Robinson, PhD, GE
Principal Engineer, GE 3118

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1090 & 1093 ESCALANTE STREET
APN: 115-230-003 & -004, GUADALUPE AREA
SANTA BARBARA COUNTY, CALIFORNIA

PROJECT SM00301-1

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation for the proposed housing development known as Escalante Meadows to be located at 1091 & 1093 Escalante Street, APN: 115-230-003 & -004, in the Guadalupe area of Santa Barbara, California. See Figure 1: Vicinity Map (DeLorme, 2009) for the general location of the project area, hereafter referred to as the Site.

1.1 Site Description

The proposed project area is located at approximately 35.9697 degrees north latitude and 120.5656 degrees west longitude. The combined parcel area is approximately rectangular in shape and about 7.44 acres in size. Escalante Street loops through the property, intersecting with 11th Street at the northeast property boundary. See Figure 2: Google Earth Image (Google Earth ©).

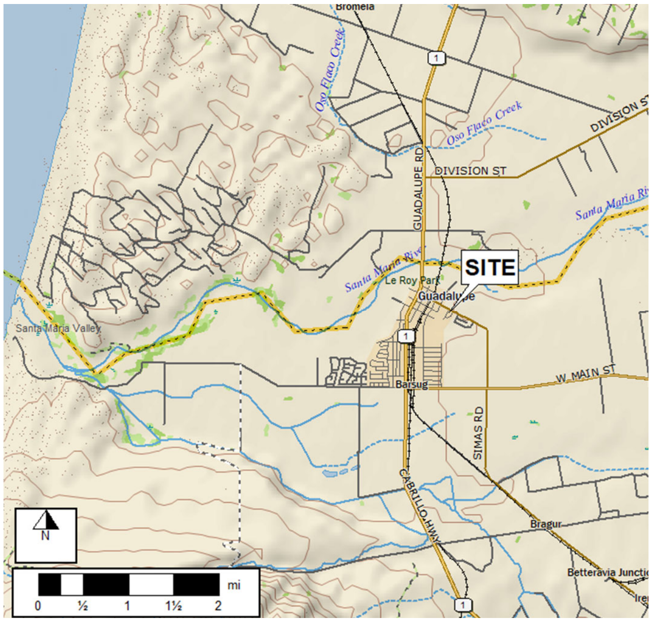

Figure 1: Vicinity Map

The Site is currently developed and consists of about 27 single-story housing units, office center, parking lot and park areas. In general, topography across the property is relatively flat with a slight down-slope in the southeast portion of the site, descending towards the existing basketball court. Site elevations range from about +90 feet (MSL) in the northwest to about +83 feet in the southeast. A drainage channel runs along the southern property boundary. The base of the channel is estimated to be about 20 feet below the southern property boundary which descends to the creek at about 2-to-1 horizontal-to-vertical. Surface drainage generally follows the topography to Escalante Road and existing drainage systems or to the south, towards the creek.

Figure 2: Google Earth Image

1.2 Project Description

The project will consist of demolishing the existing structures at the site and constructing a new housing development. The proposed project will consist of ten single-story residential structures, community center, parking areas, and playground. See Figure 3: Site Plan for the general layout of the proposed project.

At the time of the preparation of this report, the proposed single-family residences for the housing development are to be constructed using light wood framing. It is anticipated that the proposed single-family residences will utilize slab-on-grade lower floor systems.

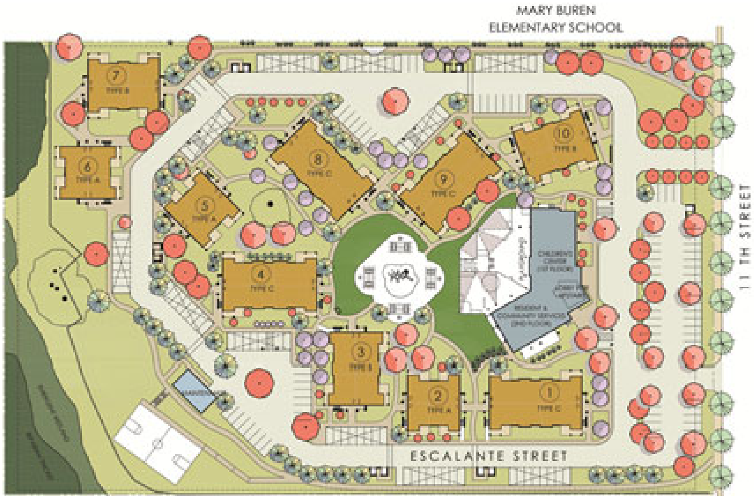

Figure 3: Site Plan

Dead and sustained live loads are currently unknown, but they are anticipated to be relatively light with maximum continuous footing and column loads estimated to be approximately 1.5 kips per linear foot and 15 kips, respectively

2.0 WORK PERFORMED

2.1 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 cone penetration testing (CPT) 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.

2.2 Field Investigation

6. The field investigation for the project was performed in two phases under the direction of the Project Engineer. The initial phase of the field exploration was conducted on May 7, 2019, and consisted of advancing seven (7) cone penetrometer tests (CPTs) at the approximate locations indicated in Figure 4: Field Exploration Plan. Middle Earth Testing of Orange, CA, used a 25-ton CPT rig to push an electric cone to depths of approximately 50 feet below ground surface (bgs).

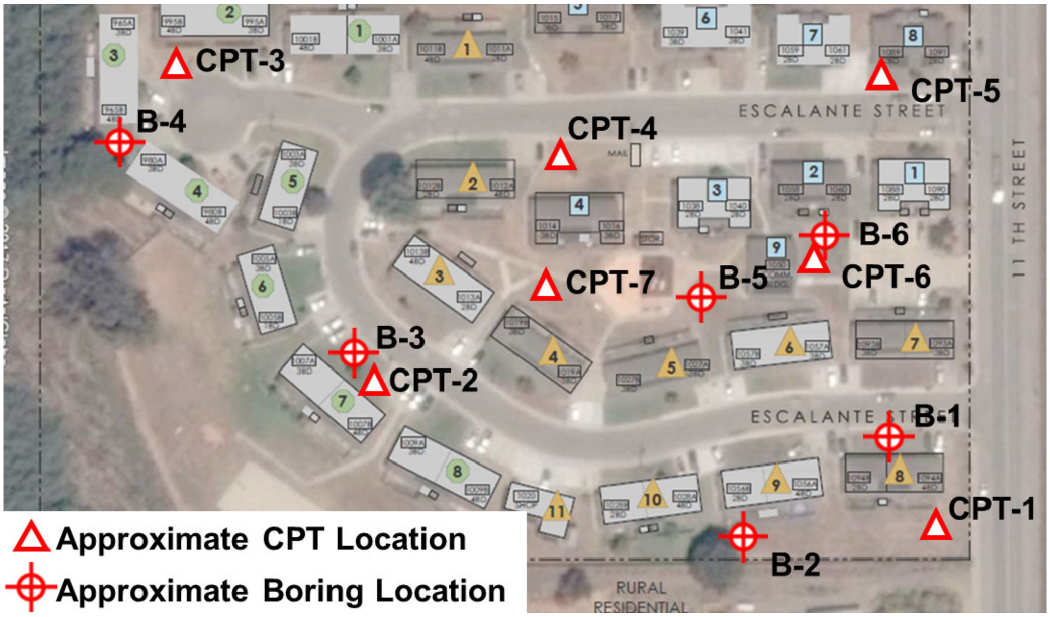

Figure 4: Field Exploration Plan

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 near-continuous soil behavior profile and to better characterize the Site for liquefaction hazard assessment and understanding of the subsurface conditions. Refer to **Appendix A** for logs of the CPT soundings which include interpretation of the soil behavior.

The second phase of the field investigation was conducted on May 29, 2019 using a Mobile B-24 drill rig to advance six (6) six-inch diameter exploratory borings to a maximum depth of 25 feet below ground surface (bgs). See Figure 4: Field Exploration Plan for the approximate boring locations. Sampling methods within the borings included the Standard Penetration Test utilizing a standard split-spoon sampler (SPT) and a Modified California sampler (CA). 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 SPT N-values. During the boring operations the soils encountered were continuously examined, visually classified, and sampled for general laboratory testing. Logs of the borings are provided in **Appendix A.**

2.3 Laboratory Testing

Laboratory tests were performed on soil samples that were obtained from the Site during the field investigation. Results and explanations of the tests are provided in **Appendix B**. Laboratory testing for this project was subcontracted to certified laboratories including NV5 of Ventura, California, and PEI of Redding, California. The testing performed for the project consisted of the following:

- Soil Classification (ASTM D2487, ASTM D2488)
- Particle Size Analysis (ASTM C136, ASTM C117)
- Liquid Limit, Plastic Limit, and Plasticity Index (ASTM 4318)
- Expansion Index (ASTM D4829)
- Direct Shear (ASTM D3080)
- Laboratory Maximum Density (ASTM D1557)
- R-Value (CT 301)

3.0 SUBSURFACE CONDITIONS

3.1 Geologic Setting

Regional site geology was obtained through review of the *Geologic Map of the Point Sal and Guadalupe Quadrangle* (Dibblee, 2006), available from the USGS MapView internet application (USGS, 2013). Figure 5: Regional Geologic Map provides the surficial geologic units in the project vicinity, as mapped by Dibblee (2006).

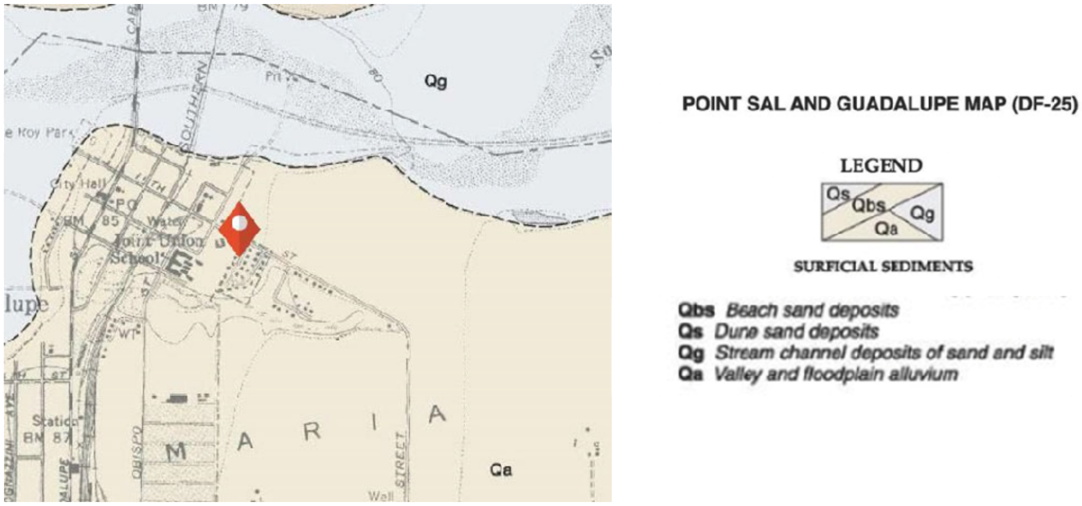

Figure 5: Regional Geologic Map

The Site is underlain by surficial sediments of alluvium (Qa) comprised of valley and floodplain deposits. The alluvial deposits are likely associated with flows from the Santa Maria River located north of the Site, as shown in Figure 5.

3.2 Soil Conditions

Data gathered during the field investigation suggest that the soil materials at the Site generally consist of fill and alluvial soil. Fill was interpreted to depths of about 1 to 4 feet in borings B-4 and B-3, located in the western portion of the development, and generally consisted of dark brown silty SAND (SM). The alluvial soils were encountered at the ground surface and underlying the fill material and extended to the maximum depth explored of 25 feet in the borings and 50 feet in the CPTs.

In general, the near surface alluvial soils consisted of dark brown lean CLAY with sand (CL) and yellowish brown lean CLAY (CL) encountered in a dry and very stiff condition to depths of about 7 to 9 feet bgs. Interbedded layers of silty SAND (SM), sandy SILT (ML), and SILT (ML) with occasionally CLAY lenses were encountered underlying the clay material to the maximum depth explored.

Results of laboratory testing performed on representative samples obtained in the field exploration are provided in Table 1: Engineering Properties. Laboratory data reports and detailed explanations of the laboratory tests performed during this investigation are provided in **Appendix B**.

3.1 Groundwater

Groundwater was measured during the field investigation in borings B-1, B-4, and B-6 at depths of 24 feet (bgs). Groundwater was interpreted from the CPT pore pressure data at depths of 23 to 25 feet below ground surface (bgs). It should be anticipated that groundwater levels may change seasonally and with irrigation practices.

Table 1: Engineering Properties

Sample Name	Sample Description	USCS Specification	Expansion Index	Expansion Potential	Plasticity Index	Fines Content (%)	Angle of Internal Friction, ϕ (deg.)	Cohesion, c (psf)
A: B-1 @ 1-3’	Dark Brown Lean CLAY with Sand	CL	108	High	18	80.0	27*	220*
B: B-1 @ 4-7’	Yellowish Brown Lean CLAY	CL	116	High	30	85.7	-	-

* Sample was remolded to 90% relative compaction

4.0 SEISMIC DESIGN CONSIDERATIONS

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

Spectral response accelerations, peak ground accelerations, and site coefficients provided in this report were obtained using the computer-based U.S. Seismic Design Map tool available from the United States Geological Survey website (USGS, 2013). This program utilizes the methods developed in the 1997, 2000, 2003, 2008 and 2013 errata editions of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures in conjunction with user-inputted Site latitude and longitude coordinates to calculate seismic design parameters and response spectra (both for period and displacement) for soil profile Site Classes A through E.

Site coordinates of 34.9686 degrees north latitude and -120.5643 degrees west longitude were used in the web-based probabilistic seismic hazard analysis (USGS, 2013). Based on the results from the in-situ tests performed during the field investigation, the Site was defined as **Site Class D**, “Stiff Soil” profile per ASCE7-10, Chapter 20. Relevant seismic design parameters obtained from the program area summarized in Table 2: Seismic Design Parameters. Refer to **Appendix C** for more information regarding the seismic hazard analysis performed for the project and detailed results.

Table 2: Seismic Design Parameters

Site Class	**D**, “Stiff Soil”
Seismic Design Category	**D**
1-Second Period Design Spectral Response Acceleration, S_{D1}	**0.422 g**
Short-Period Design Spectral Response Acceleration, S_{DS}	**0.750 g**
Site Specific MCE Peak Ground Acceleration, PGA_M	**0.449 g**

5.0 LIQUEFACTION HAZARD ASSESSMENT

5.1 Liquefaction Potential

Liquefaction occurs when saturated, cohesionless soils lose shear strength during strong ground shaking as a result of elevated pore water pressures. Consequences of liquefaction can include vertical settlement of the ground surface, sand boils, reduced bearing capacity, and lateral movement downslope or towards a free-face (lateral spreading). Liquefaction potential depends on various factors including soil type, relative density, groundwater conditions, earthquake ground motion, and overburden stress.

GeoSolutions, Inc. utilized computer software program CLiq (GeoLogismiki, 2006), to determine the liquefaction, settlement, and lateral spreading potential at the Site using CPT data from the field investigation. The program incorporates the methodology recommended in the most recent publications of the NCEER Workshop (Youd et al., 2001) and SP117 Implementation (CDMG, 2008). The program requires a user-defined peak ground acceleration, earthquake magnitude and depth to groundwater to assess the potential for liquefaction and consequential vertical settlement estimate. The lateral spreading calculation incorporates the site geometry including the ground slope (%) or, alternatively the height of the free-face (H) and distance to the free-face (L).

5.2 Analysis Parameters

The upper 50 feet of the soil profile was considered in the analysis, based on our understanding of the project and current standard of practice. The peak ground acceleration for the site (PGAM) was obtained from the Seismic Design Maps tool available the Structural Engineers Association of California (SEAOC, 2018) and was determined to be 0.45 g. An earthquake magnitude (Mw) of 7.0 was selected for the analysis based on our understanding of the seismicity in the region. As described ins Section 3.3, the depth to groundwater interpreted from the CPT soundings ranged from about 23 to 25 feet bgs (consisted with groundwater readings from the boring logs) and was input on a CPT-specific basis. For the lateral spreading estimate, geometric properties were determined based on the proximity of the site location to an unnamed creek channel running along the southern property boundary. The height, H, of the creek channel (free-face) was estimated to be about 20 feet based on our site visit performed on May 10, 2019. The distance to the free-face or creek channel was estimated from Google Earth © and generally ranged from about 300 to 875 feet at the CPT locations.

5.3 Results of Liquefaction Analysis

Based on the presence of loose to medium dense, saturated sandy soils encountered in the field exploration, the depth to groundwater, and the expected seismic demand assumed for the analysis, the potential for seismic liquefaction at the Site is high. Liquefaction was determined to predominantly occur in the sandy layers located beneath the Site between depths of about 23 to 50 feet bgs. The potential for liquefaction, estimated vertical settlements, and lateral displacements determined from the CPT soundings is shown in the CLiq Liquefaction Analysis Results, provided in **Appendix D**.

Overall, liquefaction-induced seismic settlements on the order of about **2 to 6 inches** were obtained from our analysis. We anticipate surface manifestation of liquefaction to consist of vertical settlements and sand boils.

Based on the empirical model used in the analysis, lateral displacements were estimated to be on the order of about 10 to 25 inches. However, it should be noted that lateral spreading is a highly complex phenomenon that is often difficult to capture with the current standard-of-practice empirical models (Robinson, 2016). The mechanism of lateral spreading failures require the presence of a continuous, liquefiable layer extending both laterally along the free-face as well as perpendicular to the free-face, away from the channel. Upon scrutiny of the subsurface data, the majority of the liquefiable strata susceptible to lateral spreading that were encountered in the CPTs do not appear to be continuous across the Site. In addition, many of the case histories of laterals spreading occur along low-lying river banks, where the liquefiable layers are within ten to 15 feet of the ground surface, e.g. Robinson (2016) which documents the liquefaction-induced lateral spreading from the Canterbury Earthquakes in New Zealand. Thus, given the depth to the liquefiable material, about 24 feet, and the absence of significant continuous, liquefiable strata along or away from the free-face, the potential for large-scale lateral displacements to occur at the Site is considered low, despite model predictions.

6.0 GENERAL SOIL-FOUNDATION DISCUSSION

There is a potential for liquefaction to occur at the Site and to likely manifest at the surface as vertical settlements and sand boils. Due to the potential for liquefaction induced settlements, it is anticipated that a structural waffle slab type foundation system founded over geogrid-reinforced engineered fill pad will be constructed for the proposed structures.

According to Section 8.0, Mitigation of Liquefaction Hazards of the DMG Special Publication 117 (SCEC, 1999) and the County of Los Angeles Department of Public Works Manual for Preparation of Geotechnical Reports (County of Los Angeles, 2013), structural mitigation, without the use of ground improvement, for liquefiable sites is acceptable provided the following criteria are satisfied:

(1) Seismically induced differential vertical displacements are less than 1 inch over a horizontal distance of 30 feet;

(2) Total seismically induced vertical settlements are less than 4 inches; and

(3) Seismically induced lateral displacements are less than 12 inches.

As indicated by our analysis of the subsurface soils at the Site (See Section 5.3), the estimated liquefaction-induced ground deformations exceed the above criteria with respect to vertical displacement limits. As mentioned above, lateral displacements are anticipated to be minimal given the depth and lack of continuity of the liquefiable layers. The use of the structural waffle slab foundation system over a geogrid reinforced fill pad will address life safety issues associated with the liquefaction hazard while maintaining the economic feasibility of the project. This type of foundation system has been found to perform well in areas subject to liquefaction-induced settlements and is similar to that recommended by New Zealand MBIE (2012) for mitigation.

As an alternative, ground improvement such as stone columns, may be used to minimize the potential for liquefaction-induced settlements. Given the highly expansive nature of the near-surface material, it is recommended the structures be founded on a structural waffle slab type foundation system overlying the area of ground improvement.

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.

7.0 CONCLUSIONS AND RECOMMENDATIONS

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

The primary geotechnical concerns at the Site are:

1. The potential for liquefaction-induced ground settlements and surface manifestation of sand boils.
2. The presence of loose surface materials and potential for debris resulting from demolition and removal of the existing structures.

3. The presence of potentially expansive material. Influx of water from irrigation, leakage from the residence, or natural seepage could cause expansive soil problems. Foundations supported by expansive soils should be designed by a Structural Engineer in accordance with the 2016 California Building Code.

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

7.1 Preparation of Geogrid-Reinforced Building Pads

1. It is anticipated that geogrid-reinforced graded engineered fill pads will be developed for the proposed structures with footings founded in engineered fill.

2. For the development of an engineered fill pad, the on-site material should be over-excavated to a minimum of **60 inches** below existing grade, 24 inches below the bottom of 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, where possible.

3. The exposed surface should be scarified to a depth of 8 inches; moisture conditioned to 3 percent over optimum moisture content; and compacted to a minimum relative density of 90 percent (ASTM D1557-12$_{e1}$).

4. A geotextile fabric (Mirafi HP370 or equivalent) should be placed at the bottom of the excavation with a 2-foot overlap and per manufacturer's specifications. A Tensar TX7geogrid (or equivalent) should then be placed immediately above the geotextile fabric, with a minimum 2-foot overlap and per manufacturer's specifications. The geotextile fabric and geogrid should continue along the sidewall of the excavation, extending up to existing grade.

5. 12 inches of aggregate base (Caltrans Class II or equivalent) should be placed over the geogrid, moisture conditioned to near optimum moisture and compacted to 90 percent relative compaction (ASTM 1557-12$_{e1}$), followed by an additional layer of Tensar TX7 geogrid (or equivalent) placed with a minimum 2-foot overlap and per manufacturer's specifications.

6. The over-excavated, on-site material can then be placed as engineered fill up to finish grade. Onsite soil and rock material are suitable as engineered fill material provided it is processed to remove concentrations of organic material, debris, and other particles. Imported fill should meet the requirements of the grading plan. GeoSolutions, Inc. should be notified at least 72 hours prior to delivery to the site to sample and test proposed imported fill materials. Refer to Figure 7: Sub-Slab Detail for under-slab drainage material and **Appendix E** - Preliminary Grading Specifications, for more details on fill placement.

7.2 Vibro Replacement (Stone Columns)

1. As an alternative to the geogrid-reinforced engineered fill pads, vibro replacement (stone columns) may be used to consolidate the subsurface soils at the Site and minimize vertical settlements associated with liquefaction. Figure 6: Vibro Replacement Illustration provides a schematic diagram of typical vibro replacement construction.

2. A design professional specializing in the design of vibro replacement (stone columns) should be provide this report to evaluate the site of for the design of a vibro replacement soil densification.

3. For preliminary purposes, the stone columns will likely be required to a depth of about 35 feet below existing ground surface, extending a minimum of 15 feet beyond the perimeter of the structure.

4. Building preparation and foundation design should be recommended by the design professional.

5. It is recommended that CPT soundings be placed between each stone column area after installation to verify that the estimated seismically-induced settlements have been reduced to less than one inch.

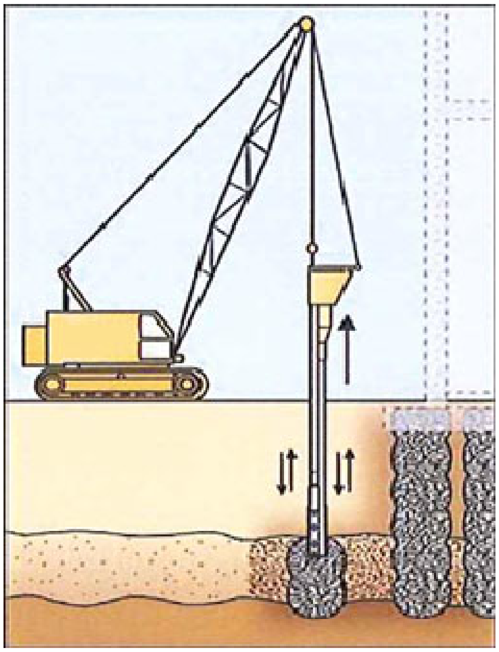

Figure 6: Vibro Replacement Illustration

7.3 Structual Waffle Mat Foundations

1. To reduce the potential for movement associated with highly expansive soils at the Site, as well as movement associated with liquefaction-induced settlements, a stiffened waffle mat slab foundation system should be used to support the proposed structures. The following recommendations are subject to change provided ground improvement techniques are utilized. GeoSolutions, Inc. should review all final foundation recommendations prior to construction.

2. Waffle mat slabs should be designed to resist differential settlements of approximately 2 inches in 30 feet. All foundations systems should be designed to withstand a 6-foot diameter loss of support in all directions due to the potential for sand boils to occur during a seismic event.

3. Static loading settlement on the order of less than 1 inch across the Site should be anticipated. Minimum reinforcing should be as directed by the project Structural Engineer.

4. The waffle mat slabs should be designed to impose a maximum allowable bearing pressure of **2,000** 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.

5. Exterior and interior footings should be founded a minimum of **30 inches** below finish pad grade and should be a minimum of **15 inches** wide. Grade beams should be placed a minimum of **15 feet** on-center and should be a minimum of **24 inches**. Isolated pad footings are not allowed.

6. 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 12 inches into engineered. A passive pressure of **250-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. A modulus of subgrade reaction (k_s) of **100 pci** pounds per cubic inch may be used.

7. Provided the above recommendations are implemented into the design of the proposed apartment structures a static total settlement of less than one inch and a differential settlement of less than ½-inch in 40 feet are anticipated.

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

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

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

11. A minimum **3,000 psi** concrete should be used for all foundations. The final foundation plans should be reviewed by the Soils Engineer when they become available to verify conformance with these recommendations.

7.4 Slab-On-Grade Construction

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

2. Concrete slabs-on-grade should be in conformance with the recommendations provided in Table 3: Minimum Slab Recommendations. Reinforcing should be placed on-center both ways at or slightly above the center of the structural section. Reinforcing bars should have a minimum clear cover of 1.5 inches. Where lapping of the slab steel is required, laps in adjacent bars should be staggered a minimum of every five feet (see WRI Design of Slab-on-Ground Foundations, Steel Placement). The recommended reinforcement may be used for anticipated uniform floor loads not exceeding 200 psf. If floor loads greater than 200 psf are anticipated, a Structural Engineer should evaluate the slab design.

Table 3: Minimum Slab Recommendations

Minimum Thickness	Per Structural Engineer
Reinforcing*	Per Structural Engineer
* Where lapping of the slab steel is required, laps in adjacent bars should be staggered a minimum of every five feet (see WRI/CSRI-81 recommendations for Steel Placement, Section 2).	

3. Concrete for all slabs should be a minimum design strength of **3,000 psi** and 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.

2. Where concrete slabs-on-grade are to be constructed for interior conditioned spaces, the slabs should be underlain by a minimum of four inches of clean free-draining material, such as a ½ inch coarse aggregate mix, to serve as a cushion and a capillary break. Where moisture susceptible storage or floor coverings are anticipated, a 15-mil Stego Wrap membrane (or equivalent installed per manufacturer's specifications) should be placed between the free-draining material and the slab to minimize moisture condensation under the floor covering. See Figure 7: 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.

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

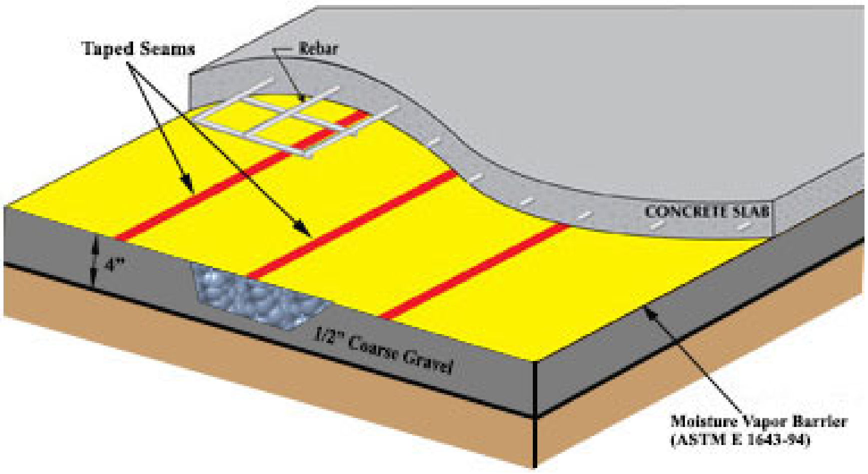

Figure 7: Sub-Slab Detail

4. 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.

5. Moisture condensation under floor coverings has become critical due to the use of water-soluble adhesives. Therefore, it is suggested that moisture sensitive slabs not be constructed during inclement weather conditions.

7.5 Exterior Concrete Flatwork

1. Due to the presence of expansive surface soils within the proposed development areas, there is a potential for considerable soil movement and distress to reinforced concrete 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.** As an alternative 12 inches of aggregate base placed over a BX1200 geogrid can be used.

2. Minimum flatwork for conventional pedestrian areas should be a minimum of 4 inches thick and consist of No. 3 (#3) rebar spaced at 18 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.

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

7.6 Infiltration Systems

1. All infiltration systems such as retention basins, permeable swales, pavers, etc. should be set back a minimum of **5 feet** from all structures. Stormwater infiltration systems adjacent to flatwork, a cutoff wall or curb shall be installed, extending a minimum of 12 inches below the bottom of the retention system.

2. An 18-inch wide, 3-sack slurry cut-off wall may be used to separate the retention system from proposed structures, flatwork, roadways, and parking areas.

7.7 Retaining Walls

1. Retaining walls should be designed to resist lateral pressures from adjacent soils and surcharge loads applied behind the walls. We recommend using the lateral pressures presented in Table 4: Retaining Wall Design Parameters and Figure 8: Retaining Wall Detail for the design of retaining walls at the Site. The Active Case may be used for the design of unrestrained retaining walls, and the At-Rest Case may be used for the design of restrained retaining walls.

Table 4: Retaining Wall Design Parameters

Lateral Pressure and Condition	Equivalent Fluid Pressure, pcf
Static, Active Case, Engineered Fill ($\gamma' K_A$)	60
Static, At-Rest Case, Engineered Fill ($\gamma' K_O$)	80
Static, Passive Case, Engineered Fill ($\gamma' K_P$)	250

2. The above values for equivalent fluid pressure are based on retaining walls having level retained surfaces, having an approximately vertical surface against the retained material, and retaining granular backfill material or engineered fill composed of native soil within the active wedge. See Figure 8: Retaining Wall Detail and Figure 9: Retaining Wall Active and Passive Wedges for a description of the location of the active wedge behind a retaining wall.

3. Proposed retaining walls having a retained surface that slopes upward from the top of the wall should be designed for an additional equivalent fluid pressure of 1 pcf for the active case and 1.5 pcf for the at-rest case, for every degree of slope inclination.

4. We recommend that the proposed retaining walls at the Site have an approximately vertical surface against the retained material. If the proposed retaining walls are to have sloped surfaces against the retained material, the project designers should contact the Soils Engineer to determine the appropriate lateral earth pressure values for retaining walls located at the Site.

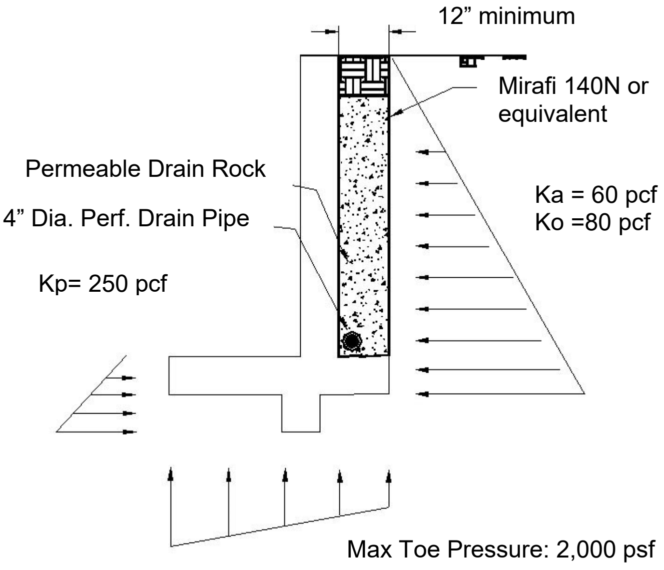

Figure 8: Retaining Wall Detail

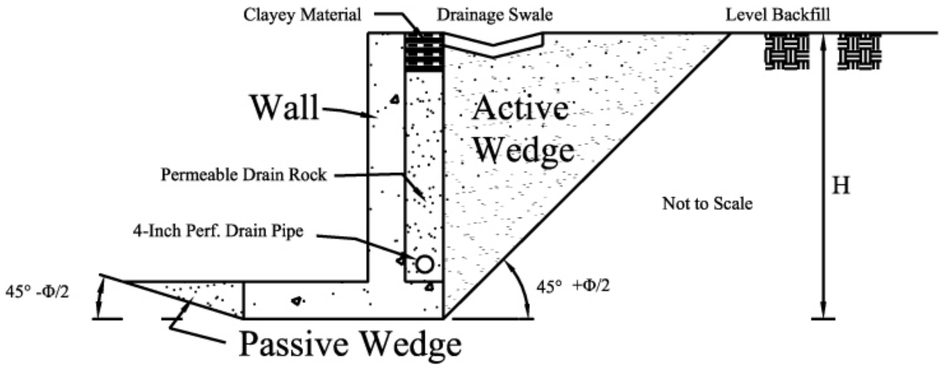

Figure 9: Retaining Wall Active and Passive Wedges

5. Retaining wall foundations should be founded a minimum of 12 inches below lowest adjacent grade in 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 **2,000 psf** for the design of retaining wall footings founded in engineered fill.

6. For earthquake conditions, retaining walls greater than 6 feet in height should be designed to resist an additional seismic lateral soil pressure of **27 pcf** equivalent fluid pressure for unrestrained walls (active condition). The pressure resultant force from earthquake loading should be assumed to act a distance of $^{1}/_{3}H$ above the base of the retaining wall, where H is the height of the retaining wall. Seismic active lateral earth pressure values were determined using the simplified dynamic lateral force component (SEAOC 2010) utilizing the design peak ground acceleration, PGA_M, discussed in Section

4.0 (**PGA_M = 0.45g**). The dynamic increment in lateral earth pressure due to earthquakes should be considered during the design of retaining walls at the Site. Based on research presented by Dr. Marshall Lew (Lew et al., 2010), lateral pressures associated with seismic forces should not be applied to restrained walls (at-rest condition).

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

8. In addition to the static lateral soil pressure values reported in Table 4: Retaining Wall Design Parameters, the retaining walls at the Site should be designed to support any design live load, such as from vehicle and construction surcharges, etc., to be supported by the wall backfill. If construction vehicles are required to operate within 10 feet of a retaining wall, supplemental pressures will be induced and should be taken into account in the design of the retaining wall.

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

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

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

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

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

14. The use of water-stops/impermeable barriers should be used for any basement construction, and for building walls that retain earth. Dam-proofing and waterproofing shall meet the minimum standards of Section 1805 of the 2016 California Building Code.

7.8 Preparation of Paved Areas

1. Pavement areas should be excavated to approximate sub-grade elevation or to competent material; whichever is deeper. The exposed surface should be scarified an additional depth of 12 inches, moisture conditioned to 3 percent over optimum moisture

content, and compacted to a minimum relative density of 95 percent (ASTM D1557-12 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 D1557-12 test method at slightly above optimum.

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

3. Due to the expansive potential of the soils at the Site, the base courses beneath un-reinforced pavement sections may fail, causing cracking of the pavement surfaces, as the sub-grade materials move laterally during expansive shrink-swell cycles.

4. Therefore, in order to minimize the potential for the failure of pavement sections at the Site, GeoSolutions, Inc. recommends that a laterally-reinforcing geotextile grid, such as Tensar BX1100, Syntec SBX11, ADS BX114GG, or equivalent, be installed to reinforce the base courses under paved areas at the Site.

5. GeoSolutions, Inc. should be contacted prior to the design and construction of pavement sections at the Site in order to assist in the selection of an appropriate laterally-reinforcing biaxial geogrid product and to provide recommendations regarding the procedures for the installation of geogrid products at the Site.

7.9 Pavement Design Standard

1. All pavement construction and materials used should conform to Sections 25, 26 and 39 of the latest edition of the State of California Department of Transportation Standard Specifications (State of California, 2015).

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

3. Table 5: Recommended Pavement Structural Sections provides the recommended Hot Mix Asphalt (HMA) pavement sections based on an R-Value of 24 (See Appendix B).

4. All pavement sections should be crowned for good drainage. 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.

Table 5: Recommended Pavement Structural Sections*

Traffic Index	**Minimum Street Section Thickness**	
	HMA	**AB**
4.0	2.0 inches	6.5 inches
	3.0 inches	6.0 inches
5.0	2.0 inches	9.0 inches
	3.0 inches	6.5 inches
6.0	2.5 inches	11 inches
	3.0 inches	10 inches

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)

** If asphalt will be placed in two lifts at different construction phases, we recommend a minimum of two inches of asphalt be placed for each lift. Prior to placing the final lift, GeoSolutions, Inc. should observe the condition of the HMA to ensure the initial lift is in a suitable condition for HMA placement.*

8.0 ADDITIONAL GEOTECHNICAL SERVICES

The recommendations contained in this report are based on a limited number of borings and on the continuity of the sub-surface conditions encountered. GeoSolutions, Inc. assumes that it will be retained to provide additional services during future phases of the proposed project. These services would be provided by GeoSolutions, Inc. as required by County of Santa Barbara, the 2016 CBC, and/or industry standard practices. These services would be in addition to those included in this report and would include, but are not limited to, the following services:

1. Consultation during plan development.

2. Plan review of grading and foundation documents prior to construction and a report certifying that the reviewed plans are in conformance with our geotechnical recommendations.

3. Consultation during selection and placement of a laterally-reinforcing biaxial geogrid product.

4. Construction inspections and testing, as required, during all grading and excavating operations beginning with the stripping of vegetation at the Site, at which time a site meeting or pre-job meeting would be appropriate.

5. Special inspection services during construction of reinforced concrete, structural masonry, high strength bolting, epoxy embedment of threaded rods and reinforcing steel, and welding of structural steel.

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

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

8. In addition to the construction inspections listed above, section 1705.6 of the 2016 CBC (CBSC, 2016) requires the following inspections by the Soils Engineer for controlled fill thicknesses greater than 12 inches as shown in Table 6: Required Verification and Inspections of Soils:

Table 6: Required Verification and Inspections of Soils

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

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

2. This report is issued with the understanding that it is the responsibility of the owner or his/her representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project, and incorporated into the project plans and specifications. The owner or his/her representative is responsible to ensure that the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

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

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REFERENCES

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APPENDIX A

Field Investigation

Soil Classification Chart

Boring Logs

CPT Logs

FIELD INVESTIGATION

The field investigation was conducted on May 7, 2019 using Cone Penetration Test (CPT) sounding equipment provided by Middle Earth Testing Equipment, Inc. and on May 29, 2019, using a Mobile B-24 drill rig. The surface and sub-surface conditions were studied by advancing six exploratory borings and seven CPT soundings at the approximate locations indicated in Figure 4: Field Exploration Plan. The exploration was conducted in accordance with presently accepted geotechnical engineering procedures consistent with the scope of the services authorized to GeoSolutions, Inc. The drilling and CPT operations were performed under the direction of the project engineer.

The CPT soundings were advanced using a 20-ton electronic CPT cone to a maximum depth of 50 feet below ground surface. The CPT cone recorded measurements for cone bearing (q_C), sleeve friction (f_S), and pore water pressure (u2) at approximately 5-cm intervals. This provides a near-continuous interpretation of the soil profile. All CPT soundings were performed in accordance with ASTM D5778 standards. Logs of the soundings showing the depths and interpreted soil behavior type and equivalent SPT N-values are provided in this appendix.

The Mobile B-24 drill rig used a six-inch diameter solid-stem continuous flight auger to advance the exploratory borings to a maximum depth of 25 feet below ground surface. 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 and ASTM D3550.

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

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

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS		LABORATORY CLASSIFICATION CRITERIA		GROUP SYMBOLS	PRIMARY DIVISIONS
COARSE GRAINED SOILS More than 50% retained on No. 200 sieve	GRAVELS More than 50% of coarse fraction retained 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

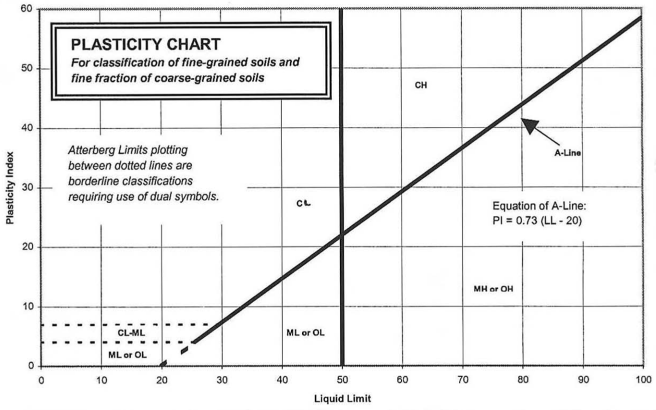
SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/ FOOT +
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	Over 50

\+ 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.)

220 High Street, San Luis Obispo, CA 93401
Phone: 805-543-8539
1021 Tama Lane, Ste 105, Santa Maria, CA 93455
Phone: 805-614-6333
201 S. Milpas St, Ste 103, Santa Barbara, CA 93103
Phone: 805-966-2200

BORING LOG

BORING NO. B-1
JOB NO. SM00301-1

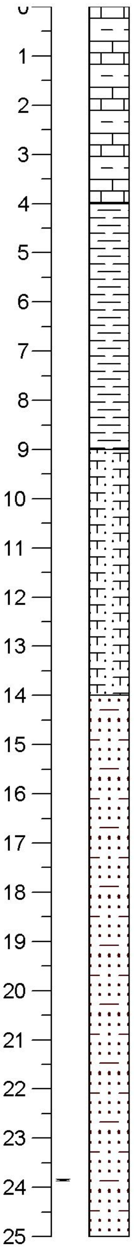
PROJECT INFORMATION

PROJECT: **Escalante Meadows**
DRILLING LOCATION: **See Figure 4: Field Exploration Plan**
DATE DRILLED: **5/29/19**
LOGGED BY: **KR**

DRILLING INFORMATION

DRILL RIG: **B-24**
HOLE DIAMETER: **6 Inches**
SAMPLING METHOD: **SPT and CA**
APPROX. ELEVATION: **Not Recorded**

Depth of Groundwater: **24 Feet**
Boring Terminated: **25 Feet**
Page 1 of 1



DEPTH	LITHOLOGY	USCS	SOIL DESCRIPTION	SAMPLE ID	SAMPLERS TYPE	BLOWS/ 12 IN	N 1/60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PLASTICITY INDEX (PI)	EXPANSION INDEX (EI)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	COHESION, C (psf)	FRICTION ANGLE, (degrees)
		CL	LEAN CLAY WITH SAND: dark brown, dry, trace rootlets, trace pin voids, mottled yellowish brown	A @ 1-3'					80	18	108	13.6	116.5	220	27
		CL	LEAN CLAY: yellowish brown, very stiff, dry, with rust mottling, with some fine sand	CA		29	26								
				B @ 4-7'					85.7	30	116				
		SM	SILTY SAND: yellowish brown, dry, medium dense, fine sand	SPT		19	21		20.2						
		SM / ML	SAND AND SILT: interbedded SILTY SAND (SM), SANDY SILT (ML), and SILT (ML), yellowish brown, moist, medium dense / firm to stiff, trace clay lenses	SPT		10	10		86.4						
			- wet at 24'												
				SPT		9	7		93.9						

GEO SOLUTIONS

220 High Street, San Luis Obispo, CA 93401
Phone: 805-543-8539
1021 Tama Lane, Ste 105, Santa Maria, CA 93455
Phone: 805-614-6333
201 S. Milpas St, Ste 103, Santa Barbara, CA 93103
Phone: 805-966-2200

BORING LOG

BORING NO. **B-2**
JOB NO. **SM00301-1**

PROJECT INFORMATION

PROJECT: **Escalante Meadows**
DRILLING LOCATION: **See Figure 4: Field Exploration Plan**
DATE DRILLED: **5/29/19**
LOGGED BY: **KR**

DRILLING INFORMATION

DRILL RIG: **B-24**
HOLE DIAMETER: **6 Inches**
SAMPLING METHOD: **SPT**
APPROX. ELEVATION: **Not Recorded**

Depth of Groundwater: **Not Encountered** Boring Terminated: **15 Feet** Page 1 of 1

DEPTH	LITHOLOGY	USCS	SOIL DESCRIPTION	SAMPLE ID	SAMPLERS TYPE	BLOWS/ 12 IN	N 1/60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PLASTICITY INDEX (PI)	EXPANSION INDEX (EI)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	COHESION, C (psf)	FRICTION ANGLE, (degrees)
0		CL	LEAN CLAY WITH SAND: dark brown, dry												
1															
2															
3															
4		CL	LEAN CLAY: yellowish brown, dry, very stiff	SPT		13	17								
5															
6															
7															
8															
9				SPT		11	12								
10		SM	SILTY SAND: yellowish brown, dry, medium dense, fine sand												
11															
12															
13			- interbedded with SANDY SILT (ML) lenses and occasional CLAY (CL), slightly moist,with rust mottling												
14				SPT		10	10								
15															
16															
17															
18															
19															
20															
21															
22															
23															
24															
25															

GEO SOLUTIONS

220 High Street, San Luis Obispo, CA 93401
Phone: 805-543-8539
1021 Tama Lane, Ste 105, Santa Maria, CA 93455
Phone: 805-614-6333
201 S. Milpas St, Ste 103, Santa Barbara, CA 93103
Phone: 805-966-2200

BORING LOG

BORING NO. **B-3**
JOB NO. **SM00301-1**

PROJECT INFORMATION

PROJECT: **Escalante Meadows**
DRILLING LOCATION: **See Figure 4: Field Exploration Plan**
DATE DRILLED: **5/29/19**
LOGGED BY: **KR**

DRILLING INFORMATION

DRILL RIG: **B-24**
HOLE DIAMETER: **6 Inches**
SAMPLING METHOD: **SPT**
APPROX. ELEVATION: **Not Recorded**

Depth of Groundwater: **Not Encountered**
Boring Terminated: **15 Feet**
Page 1 of 1

DEPTH	LITHOLOGY	USCS	SOIL DESCRIPTION	SAMPLE ID	SAMPLERS TYPE	BLOWS/ 12 IN	N 1/60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PLASTICITY INDEX (PI)	EXPANSION INDEX (EI)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	COHESION, C (psf)	FRICTION ANGLE, (degrees)
0															
1		SM	SILTY SAND: dark brown, dry, trace glass fragments in cuttings - interpreted as fill												
2															
3															
4		CL	LEAN CLAY WITH SAND: dark brown, slightly moist, stiff, with some sand	SPT		11	15								
5															
6															
7															
8		SM	SILTY SAND: yellowish brown, slightly moist, medium dense, fine sand, trace fine gravel												
9				SPT		10	11								
10															
11															
12															
13			- turns to light yellowish brown, with SILT (ML) lenses												
14				SPT		18	18								
15															
16															
17															
18															
19															
20															
21															
22															
23															
24															
25															

GEO SOLUTIONS

220 High Street, San Luis Obispo, CA 93401
Phone: 805-543-8539
1021 Tama Lane, Ste 105, Santa Maria, CA 93455
Phone: 805-614-6333
201 S. Milpas St, Ste 103, Santa Barbara, CA 93103
Phone: 805-966-2200

BORING LOG

BORING NO. **B-4**
JOB NO. **SM00301-1**

PROJECT INFORMATION

PROJECT: **Escalante Meadows**
DRILLING LOCATION: **See Figure 4: Field Exploration Plan**
DATE DRILLED: **5/29/19**
LOGGED BY: **KR**

DRILLING INFORMATION

DRILL RIG: **B-24**
HOLE DIAMETER: **6 Inches**
SAMPLING METHOD: **SPT and CA**
APPROX. ELEVATION: **Not Recorded**

Depth of Groundwater: **24 Feet** Boring Terminated: **25 Feet** Page 1 of 1

DEPTH	LITHOLOGY	USCS	SOIL DESCRIPTION	SAMPLE ID	SAMPLERS TYPE	BLOWS/ 12 IN	N 1/60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PLASTICITY INDEX (PI)	EXPANSION INDEX (EI)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	COHESION, C (psf)	FRICTION ANGLE, (degrees)
0		SM	SILTY SAND: dark brown, dry - interpreted as fill												
2		CL	LEAN CLAY WITH SAND: dark brown, slightly moist, very stiff, PP = 2.5 tsf												
4				CA		21	19								
7		SM	SILTY SAND: yellowish brown, slightly moist, medium dense, fine sand, interbedded with SILT and CLAY lenses												
9				SPT		23	25								
13			- light yellow brown, dense												
14				CA		31	21								
22			- wet, loose												
24				SPT		10	9								
25															

GEO SOLUTIONS

220 High Street, San Luis Obispo, CA 93401
Phone: 805-543-8539
1021 Tama Lane, Ste 105, Santa Maria, CA 93455
Phone: 805-614-6333
201 S. Milpas St, Ste 103, Santa Barbara, CA 93103
Phone: 805-966-2200

BORING LOG

BORING NO. **B-5**
JOB NO. **SM00301-1**

PROJECT INFORMATION

PROJECT: **Escalante Meadows**
DRILLING LOCATION: **See Figure 4: Field Exploration Plan**
DATE DRILLED: **5/29/19**
LOGGED BY: **KR**

DRILLING INFORMATION

DRILL RIG: **B-24**
HOLE DIAMETER: **6 Inches**
SAMPLING METHOD: **SPT**
APPROX. ELEVATION: **Not Recorded**

Depth of Groundwater: **Not Encountered**
Boring Terminated: **15 Feet**
Page 1 of 1

DEPTH	LITHOLOGY	USCS	SOIL DESCRIPTION	SAMPLE ID	SAMPLERS TYPE	BLOWS/ 12 IN	N 1/60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PLASTICITY INDEX (PI)	EXPANSION INDEX (EI)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	COHESION, C (psf)	FRICTION ANGLE, (degrees)
0		CL	LEAN CLAY WITH SAND: dark brown, dry, hard, with gray mottling												
4				SPT		23	31								
8		SM	SILTY SAND: yellowish brown, medium dense, dry, fine sand												
9				SPT		16	18								
13			with sandy SILT lenses; slightly moist												
14				SPT		13	13								
15															

GEO SOLUTIONS

220 High Street, San Luis Obispo, CA 93401
Phone: 805-543-8539
1021 Tama Lane, Ste 105, Santa Maria, CA 93455
Phone: 805-614-6333
201 S. Milpas St, Ste 103, Santa Barbara, CA 93103
Phone: 805-966-2200

BORING LOG

BORING NO. **B-6**
JOB NO. **SM00301-1**

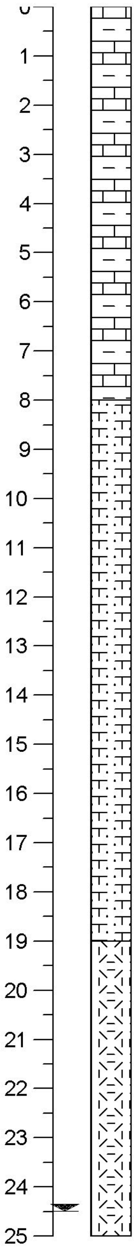
PROJECT INFORMATION

PROJECT: **Escalante Meadows**
DRILLING LOCATION: **See Figure 4: Field Exploration Plan**
DATE DRILLED: **5/29/19**
LOGGED BY: **KR/**

DRILLING INFORMATION

DRILL RIG: **B-24**
HOLE DIAMETER: **6 Inches**
SAMPLING METHOD: **SPT and CA**
APPROX. ELEVATION: **Not Recorded**

Depth of Groundwater: **24 Feet** Boring Terminated: **25 Feet** Page 1 of 1



DEPTH	USCS	SOIL DESCRIPTION	SAMPLE ID	SAMPLERS TYPE	BLOWS/ 12 IN	N 1/60	MOISTURE CONTENT (%)	FINES CONTENT (%)	PLASTICITY INDEX (PI)	EXPANSION INDEX (EI)	OPTIMUM WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	COHESION, C (psf)	FRICTION ANGLE, (degrees)
0	CL	LEAN CLAY WITH SAND: dark brown, slightly moist, very stiff, trace rootlets												
4			CA	△	26	23								
8	SM	SILTY SAND: yellowish brown, silty, slightly moist, medium dense, fine sand, interbedded with SANDY SILT (ML) lenses												
9			CA	△	25	18								
14			SPT	∧	12	12		41.3						
19			SPT	∧	24	21		8.2						
19	SP-SM	POORLY-GRADED SAND WITH SILT: yellowish brown, slightly moist, medium dense, fine to medium sand												
23		- wet, loose												
24			SPT	∧	11	9		9.5						

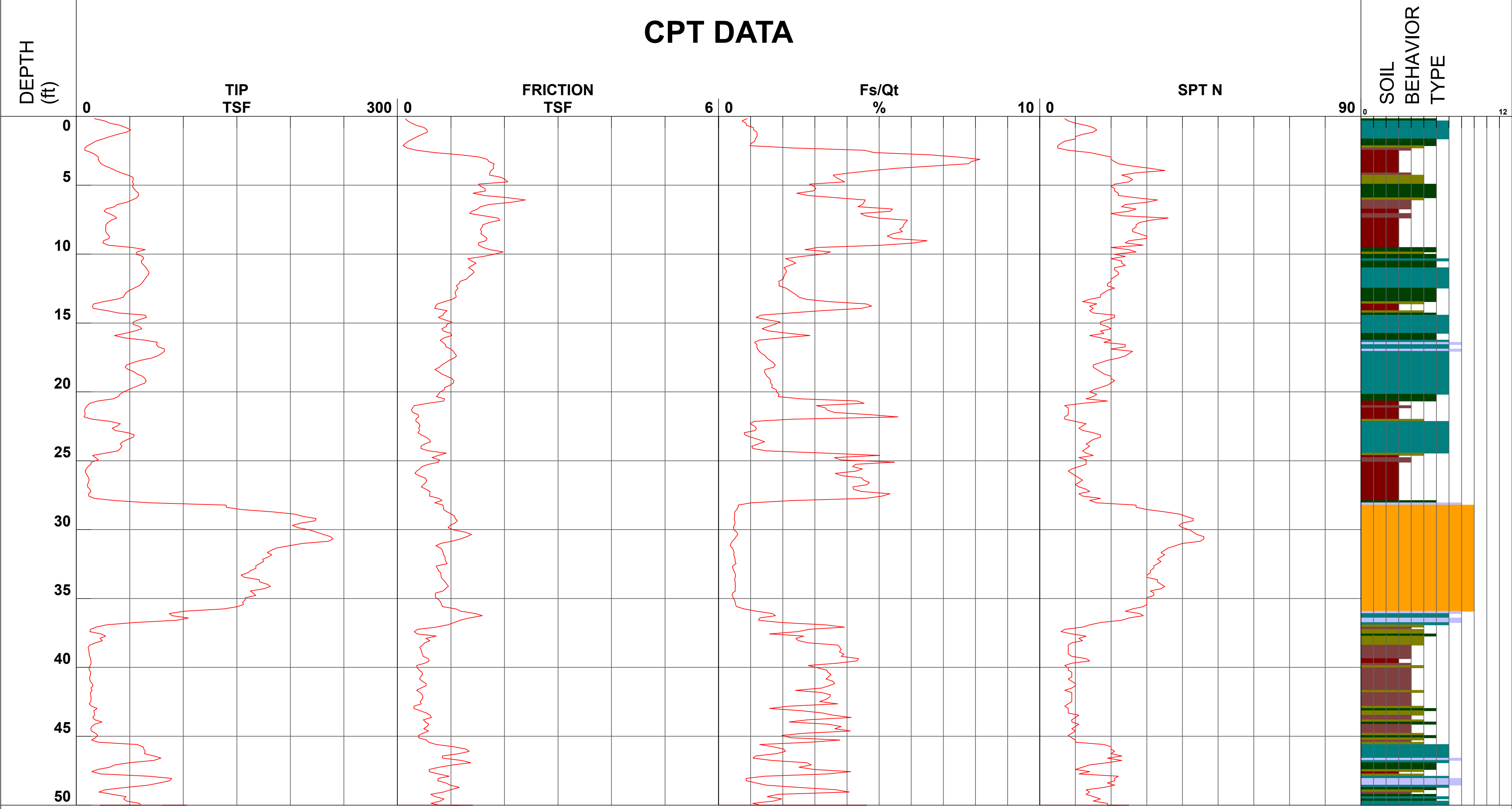
GeoSolutions, Inc.

Middle Earth
Geo Testing Inc.

Project: Escalante Meadows
Job Number: SM00301-1
Hole Number: CPT-01
EST GW Depth During Test: 26.20 ft
Operator: RC AS
Cone Number: DDG1379
Date and Time: 5/7/2019 8:42:45 AM
Filename: SDF(641).cpt
GPS:
Maximum Depth: 51.18 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

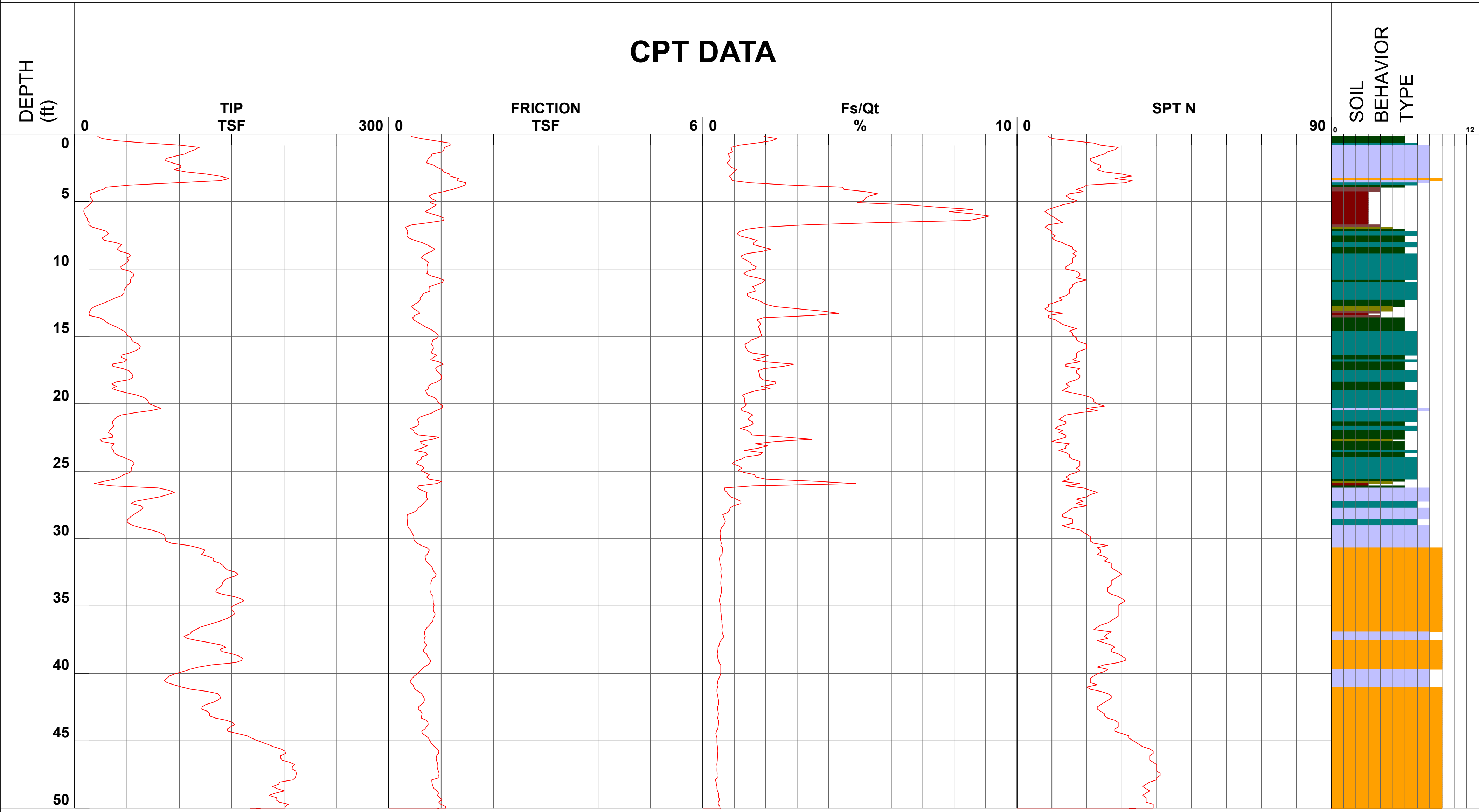
S*Soil behavior type and SPT based on data from UBC-1983

Middle Earth
GEO TESTING INC.

GeoSolutions, Inc.

Project: Escalante Meadows
Job Number: SM00301-1
Hole Number: CPT-02
EST GW Depth During Test: 23.80 ft
Operator: RC AS
Cone Number: DDG1379
Date and Time: 5/7/2019 9:36:30 AM
Filename: SDF(642).cpt
GPS:
Maximum Depth: 51.34 ft

Net Area Ratio .8



■ 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

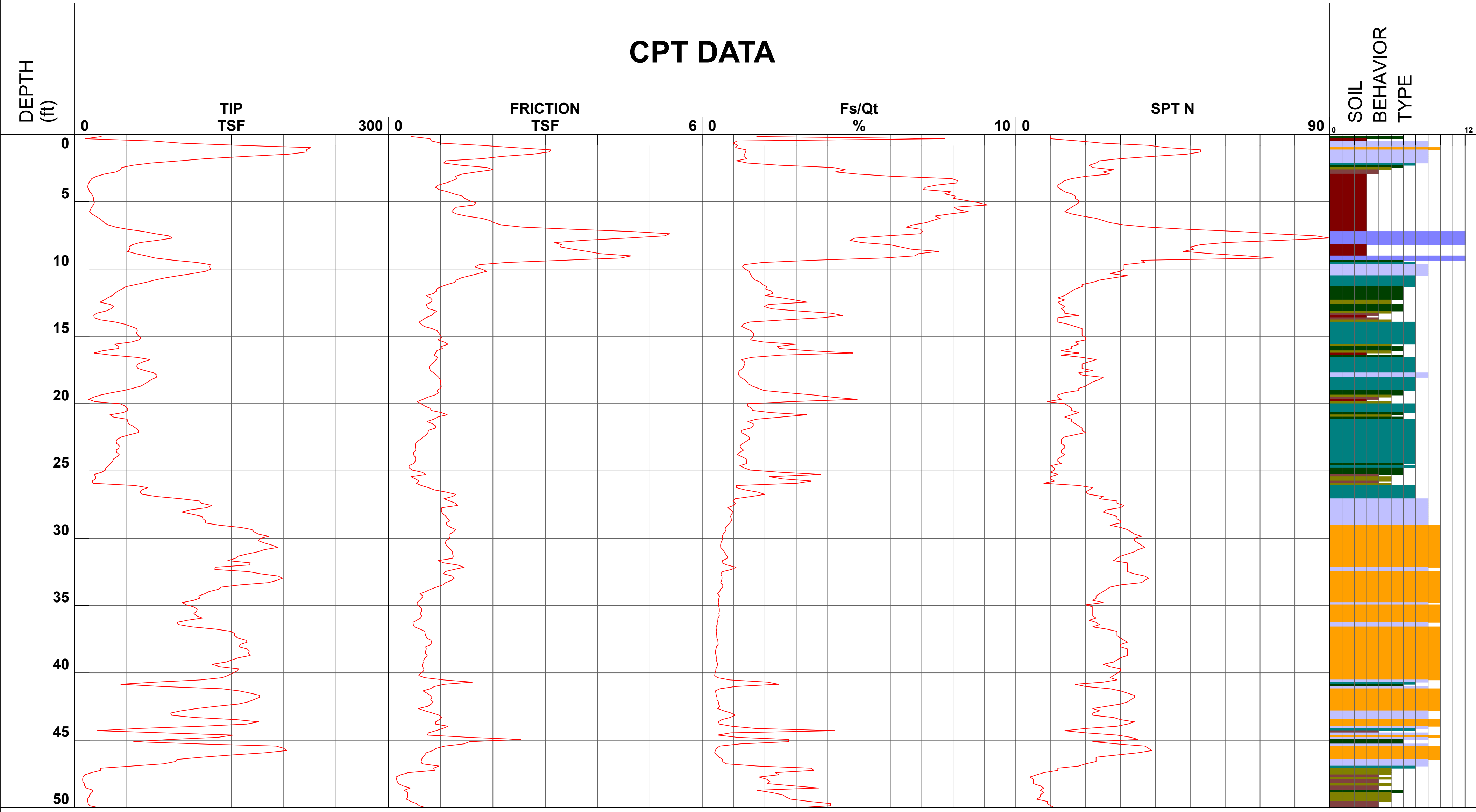
S*Soil behavior type and SPT based on data from UBC-1983

GeoSolutions, Inc.

Project	Escalante Meadows	Operator	RC AS	Filename	SDF(643).cpt
Job Number	SM00301-1	Cone Number	DDG1379	GPS	
Hole Number	CPT-03	Date and Time	5/7/2019 10:28:54 AM	Maximum Depth	51.34 ft
EST GW Depth During Test	26.60 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 (*)

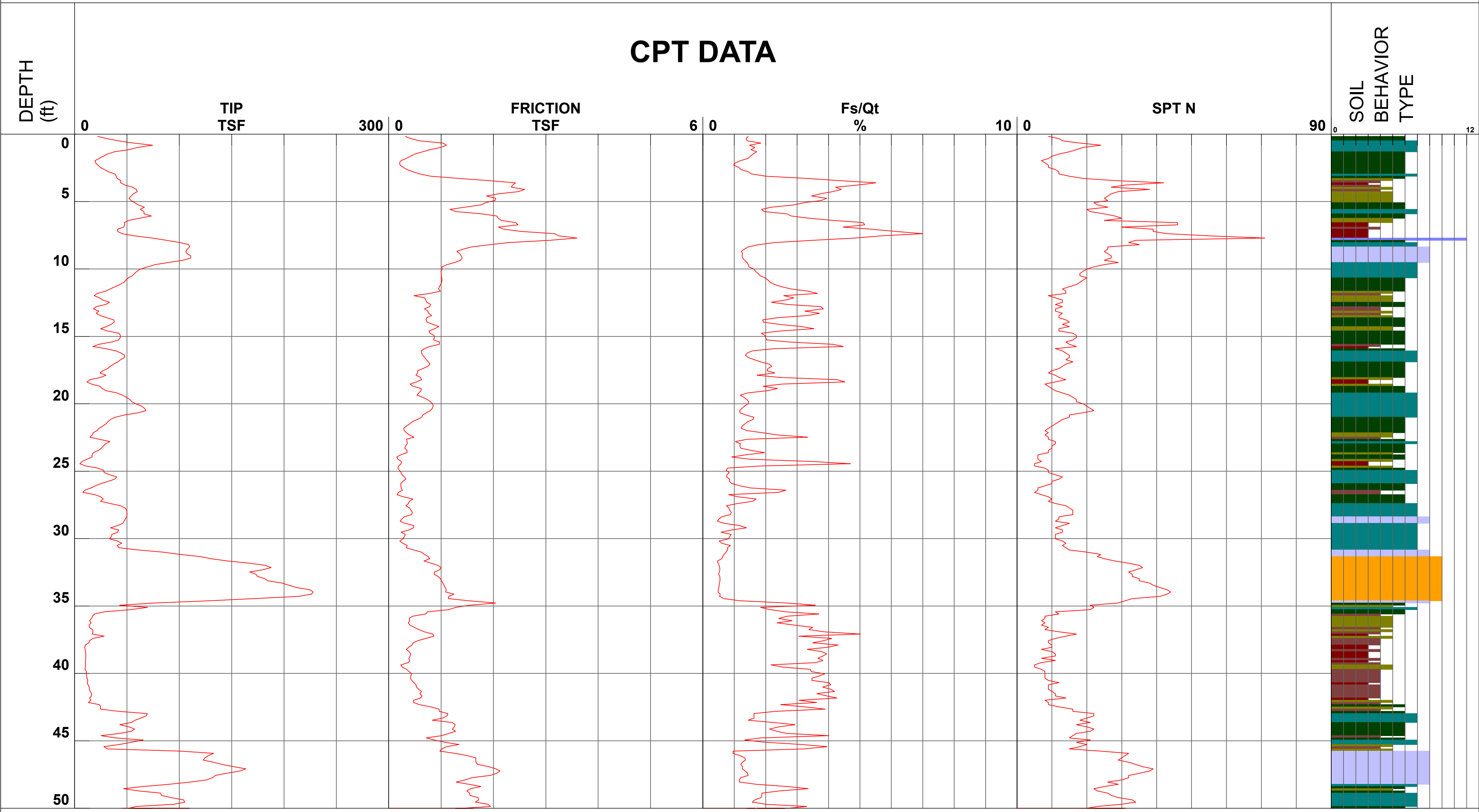
Cone Size 10cm squared

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

GeoSolutions, Inc.

Project	Escalante Meadows	**Operator**	RC AS	**Filename**	SDF(644).cpt
Job Number	SM00301-1	**Cone Number**	DDG1379	**GPS**	
Hole Number	CPT-04	**Date and Time**	5/7/2019 11:12:31 AM	**Maximum Depth**	51.51 ft
EST GW Depth During Test		25.00 ft			

Net Area Ratio .8



■ 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

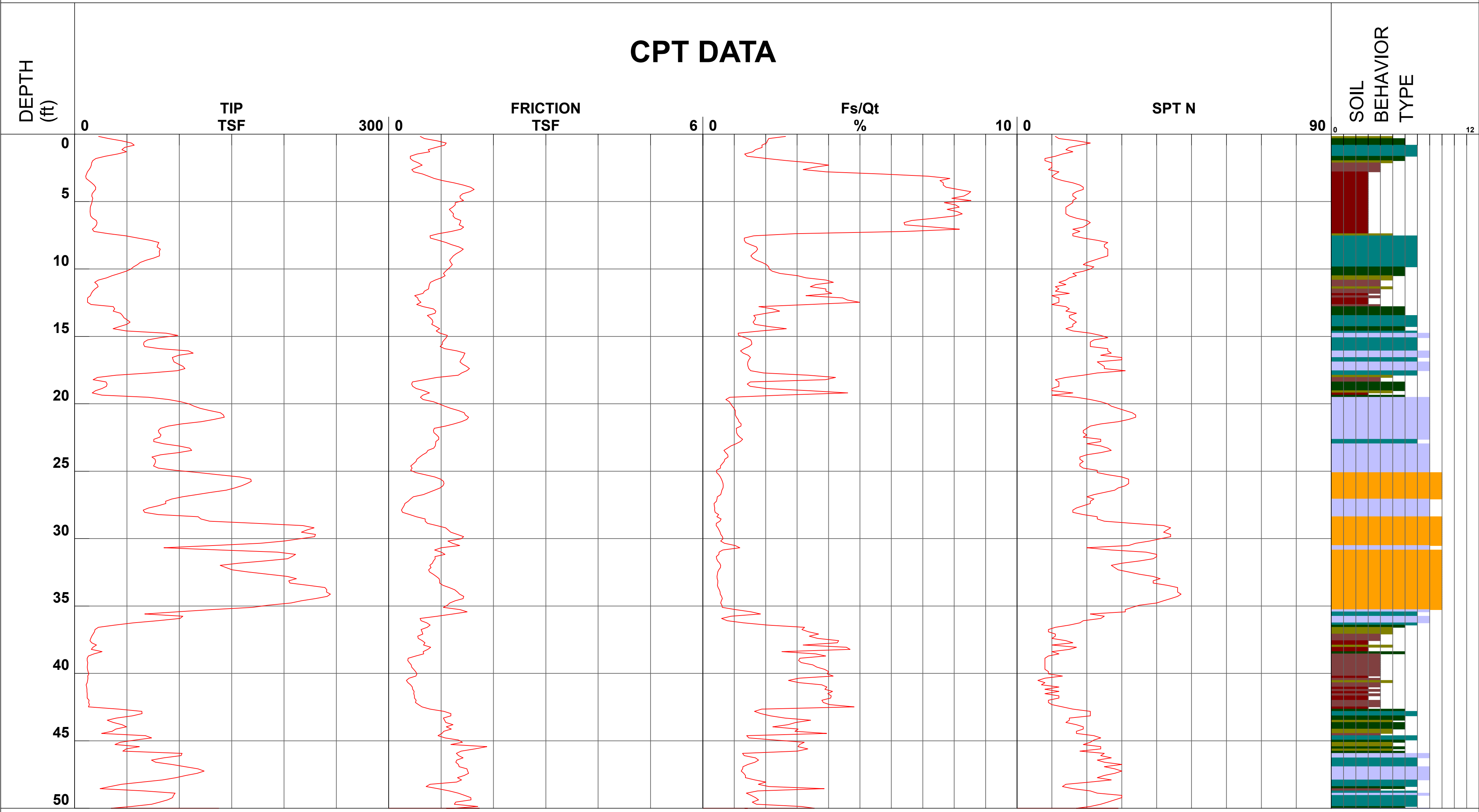
S*Soil behavior type and SPT based on data from UBC-1983

Middle Earth Geo Testing Inc.

GeoSolutions, Inc.

Project	Escalante Meadows	Operator	RC AS	Filename	SDF(645).cpt
Job Number	SM00301-1	Cone Number	DDG1379	GPS	
Hole Number	CPT-05	Date and Time	5/7/2019 11:57:03 AM	Maximum Depth	51.02 ft
EST GW Depth During Test		24.60 ft			

Net Area Ratio .8



■ 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

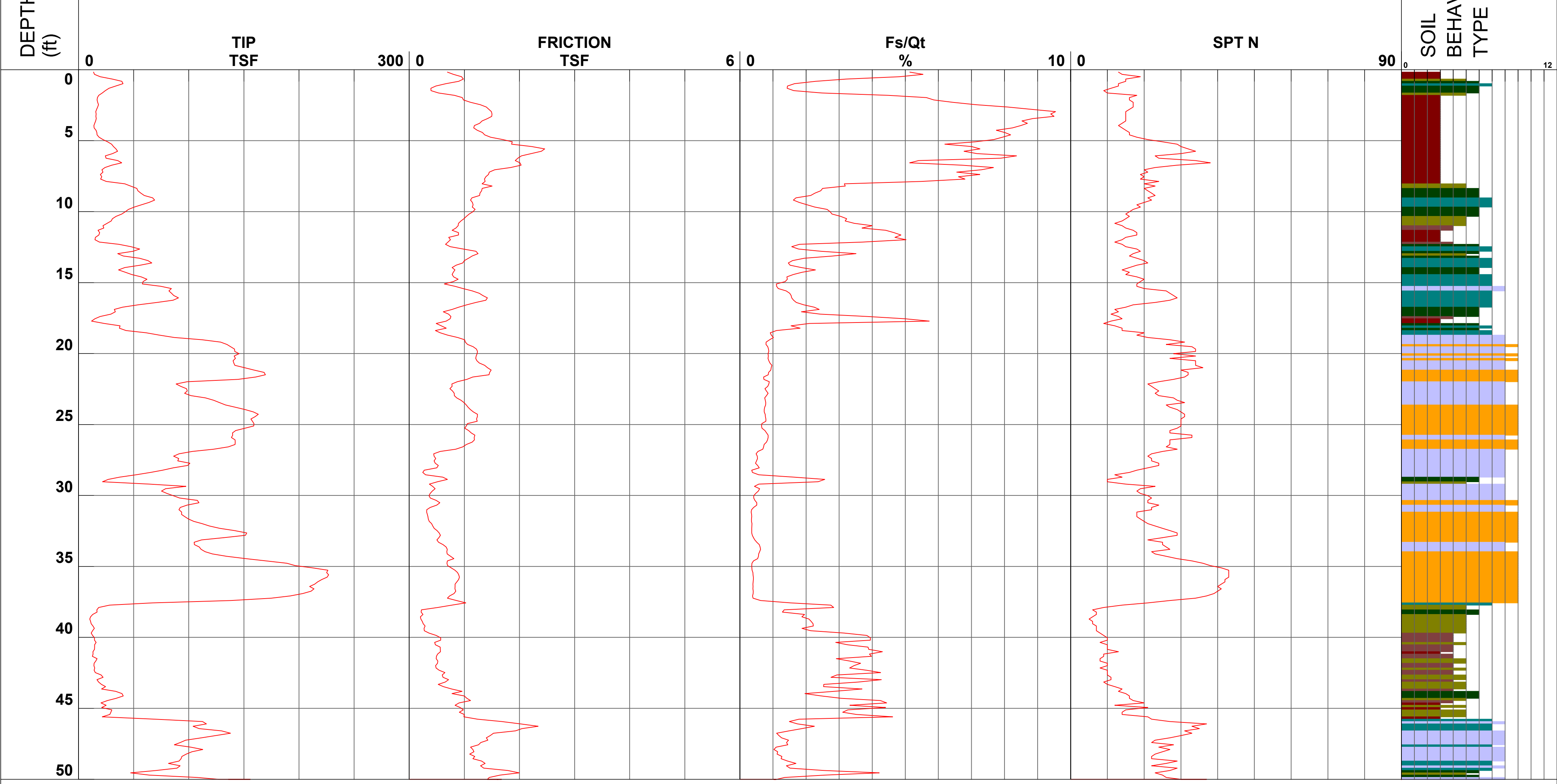
GeoSolutions, Inc.

Middle Earth
GEO TESTING INC.

Project	Escalante Meadows	Operator	RC AS	Filename	SDF(646).cpt
Job Number	SM00301-1	Cone Number	DDG1379	GPS	
Hole Number	CPT-06	Date and Time	5/7/2019 1:08:00 PM	Maximum Depth	51.18 ft
EST GW Depth During Test		24.70 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.

Middle Earth
Geo Testing Inc.

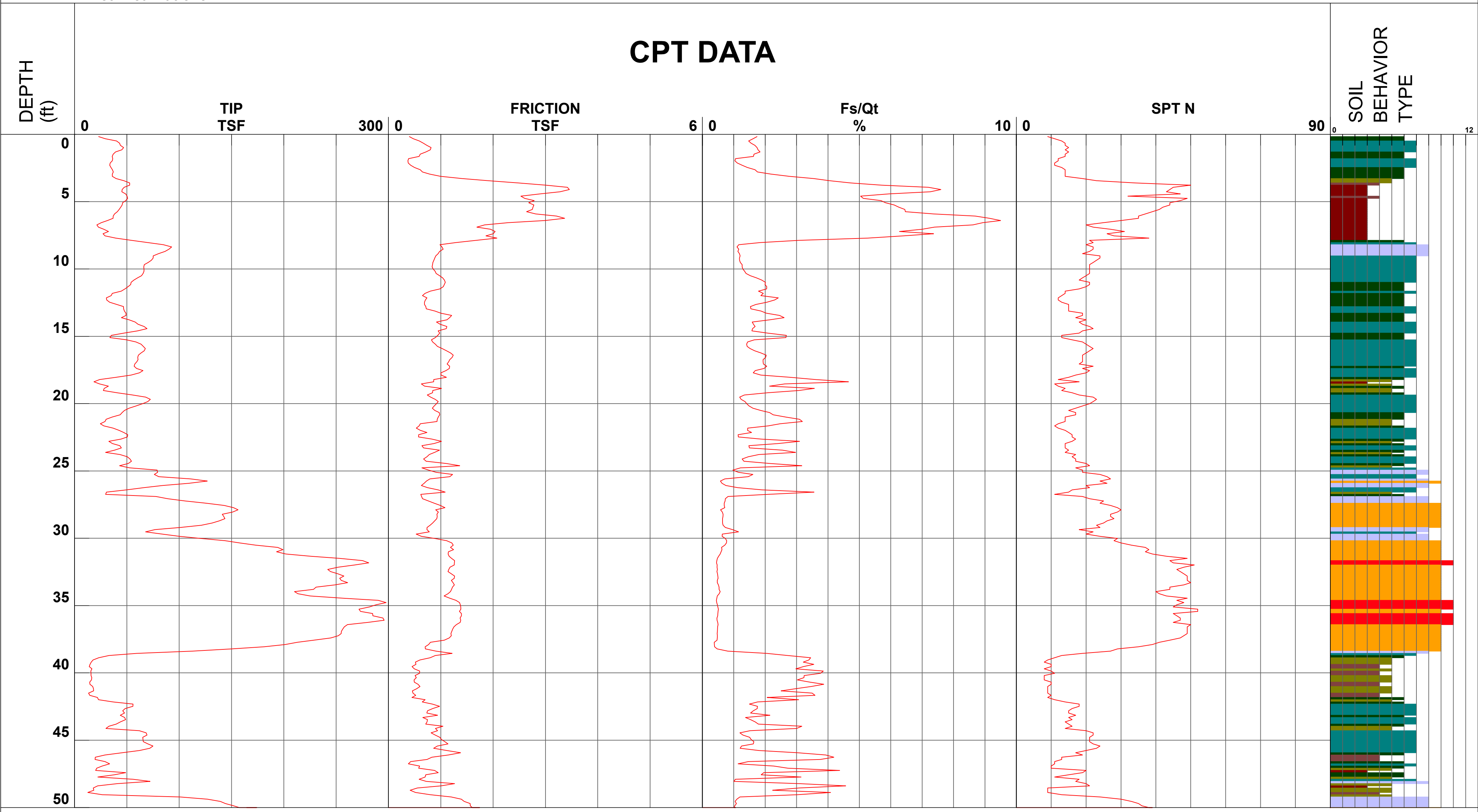
Project Escalante Meadows
Job Number SM00301-1
Hole Number CPT-07
EST GW Depth During Test 23.50 ft

Operator RC AS
Cone Number DDG1379
Date and Time 5/7/2019 1:49:33 PM

Filename SDF(647).cpt
GPS
Maximum Depth 51.02 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

APPENDIX B

Laboratory Testing

Soil Test Reports

LABORATORY TESTING

This appendix includes a discussion of the test procedures and the laboratory test results performed as part of this investigation. The purpose of the laboratory testing is to assess the engineering properties of the soil materials at the Site. The laboratory tests are performed using the currently accepted test methods, when applicable, of the American Society for Testing and Materials (ASTM).

Undisturbed and disturbed bulk samples used in the laboratory tests are obtained from various locations during the course of the field exploration, as discussed in **Appendix A** of this report. Each sample is identified by sample letter and depth. The Unified Soils Classification System is used to classify soils according to their engineering properties. The various laboratory tests performed are described below:

Expansion Index of Soils (ASTM D4829-08) is conducted in accordance with the ASTM test method and the California Building Code Standard, and are performed on representative bulk and undisturbed soil samples. The purpose of this test is to evaluate expansion potential of the site soils due to fluctuations in moisture content. The sample specimens are placed in a consolidometer, surcharged under a 144-psf vertical confining pressure, and then inundated with water. The amount of expansion is recorded over a 24-hour period with a dial indicator. The expansion index is calculated by determining the difference between final and initial height of the specimen divided by the initial height.

Liquid Limit, Plastic Limit, and Plasticity Index of Soils (ASTM D4318-05) are the water contents at certain limiting or critical stages in cohesive soil behavior. The liquid limit (LL or W_L) is the lower limit of viscous flow, the plastic limit (PL or W_P) is the lower limit of the plastic stage of clay and plastic index (PI or I_P) is a range of water content where the soil is plastic. The Atterberg Limits are performed on samples that have been screened to remove any material retained on a No. 40 sieve. The liquid limit is determined by performing trials in which a portion of the sample is spread in a brass cup, divided in two by a grooving tool, and then allowed to flow together from the shocks caused by repeatedly dropping the cup in a standard mechanical device. To determine the Plastic Limit a small portion of plastic soil is alternately pressed together and rolled into a 1/8-inch diameter thread. This process is continued until the water content of the sample is reduced to a point at which the thread crumbles and can no longer be pressed together and re-rolled. The water content of the soil at this point is reported as the plastic limit. The plasticity index is calculated as the difference between the liquid limit and the plastic limit.

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

Direct Shear Tests of Soils Under Consolidated Drained Conditions (ASTM D3080) is performed on undisturbed and remolded samples representative of the foundation material. The samples are loaded with a predetermined normal stress and submerged in water until saturation is achieved. The samples are then sheared horizontally at a controlled strain rate allowing partial drainage. The shear stress on the sample is recorded at regular strain intervals. This test determines the resistance to deformation, which is shear strength, inter-particle attraction or cohesion c, and resistance to interparticle slip called the angle of internal friction ϕ.

R-Value (CT 301) testing is used to determine the response of compacted material subject to vertical loading. The resultant value is used in pavement design in accordance with Caltrans specifications.

NV5 WEST, INC.
1868 Palma Drive, Suite A, Ventura, California 93003
Telephone: (805) 656-6074; Fax: (805) 650-6264

June 26, 2019

NV5 JOB No: **18-002374.06**
LAB No: 87260

Geosolutions, Inc.
220 High Street
San Luis Obispo, CA 93401

Attention: Kelly Robinson

Project: Geosolutions, Inc. - #SM00301-1 (Escalante Meadows)

The results of the requested laboratory tests are attached for your use.

This report includes the following test reports:

Test Description	Test Method	# of Tests
Max Density and Optimum Moisture Determination	ASTM D1557	1
Direct Shear (undisturbed)	ASTM D3080	1
Sieve Analysis	ASTM C136	2
Sieve Analysis (#200 Wash only)	ASTM C117	6
Expansion Index	ASTM D4829	2
Liquid and Plastic Limits Testing	ASTM D4318	2

NV5 WEST appreciates the opportunity to be of service. Please contact our office if you have any questions regarding this report.

Respectfully submitted,
NV5 WEST

Shaun Simon
Engineering Manager

Copies: 1-GeoSolutions/Kelly Robinson
1-File

NV5 WEST, INC.
1868 Palma Drive, Suite A, Ventura, CA
An NV5 Company - Offices Nationwide

REPORT OF MAXIMUM DENSITY / OPTIMUM MOISTURE TEST

(ASTM D1557)

June 26, 2019

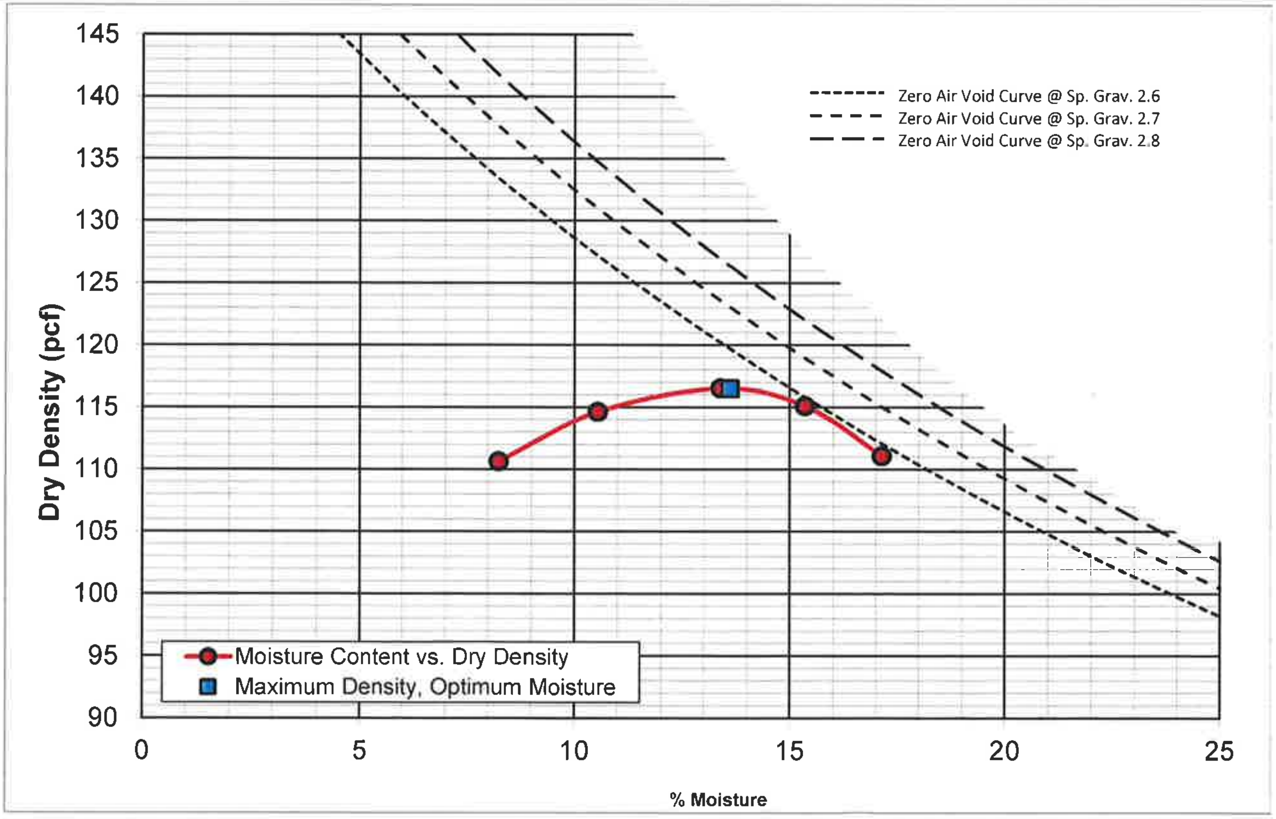
NV5 JOB No: **18-002374.06**
LAB No: **87260**

Geosolutions, Inc.
220 High Street
San Luis Obispo, CA 93401

Attention: Kelly Robinson

Project: Geosolutions, Inc. - #SM00301-1 (Escalante Meadows)
Material: Dark Brown Clayey SAND (SC)
Location: B-1 / 1-3'
Sampled By: KR
Date Sampled: 5/31/19
Tested By: PFH
Date Tested: 6/7/19

SOIL DATA				***TEST DATA***			
% Passing: 3/8in.	100.0%	Absorption (oversize)		Test Spec.	ASTM 1557B	Method:	B
		Sp.Gr. (oversize)		Mold Size (in)	4.0	# of Layers	5
		PL		Hammer Wt.(lb)	10.0	Blows / Layer	25
USCS		PI		Drop (in)	18.0		



MAXIMUM DENSITY (pcf)	**116.5**
OPTIMUM MOISTURE (%)	**13.6**

Reviewed By:

NV5 WEST, INC.
1868 Palma Drive, Suite A, Ventura, CA 93003; 805.656.6074

Geosolutions, Inc.
220 High Street
San Luis Obispo, CA 93401

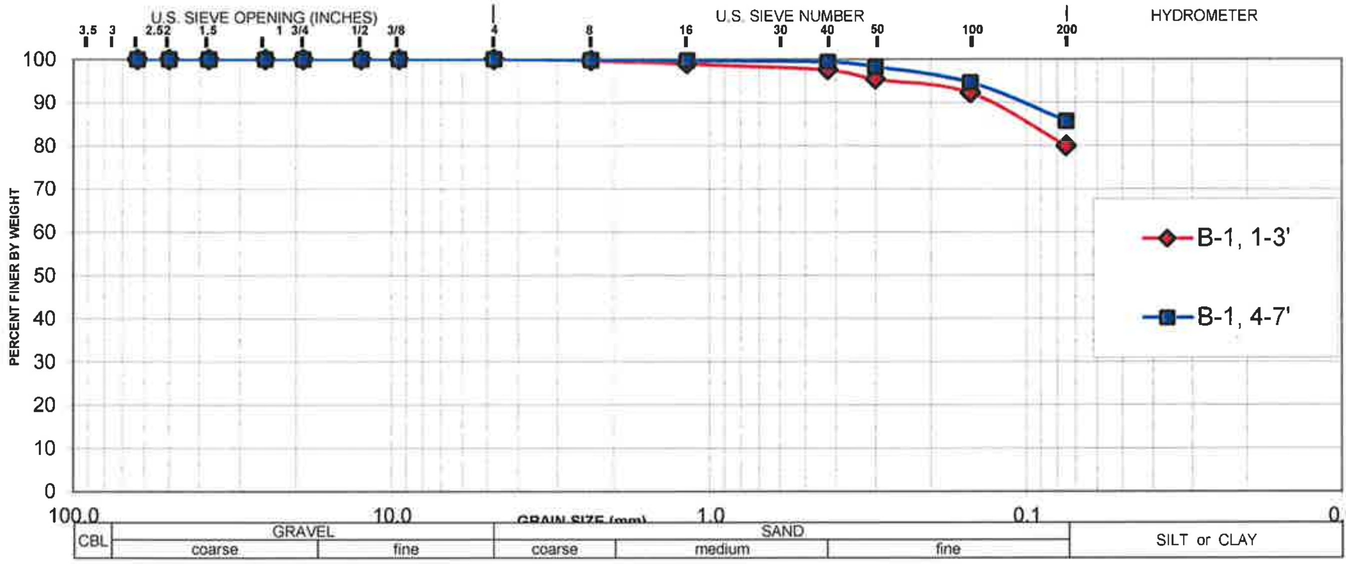
June 26, 2019
NV5 Project No: **18-002374.06**
NV5 Lab #: 87260

Project: Geosolutions, Inc. - #SM00301-1 (Escalante Meadows)

REPORT OF SIEVE ANALYSIS TEST

ASTM C136

SAMPLE ID:	**B-1, 1-3'**	**B-1, 4-7'**	
Description			
Material	Dark Brown CLAY (CL)	Light Brown CLAY (CL)	
Sample Location	B-1 @ 1-3'	B-1 @ 4-7'	
Date Sampled	5/31/19	5/31/19	
Sampled By	KR	KR	
Date Tested	6/13/19	6/13/19	
Tested By	PFH	PFH	



Sample ID:		B-1, 1-3'	B-1, 4-7'	
U.S. Sieve	mm	% Passing		
2 1/2"	63.0	100	100	
2"	50.0	100	100	
1 1/2"	37.5	100	100	
1"	25.0	100	100	
3/4"	19.0	100	100	
1/2"	12.5	100	100	
3/8"	9.50	100	100	
#4	4.75	100	100	
#8	2.36	100	100	
#16	1.18	99	100	
#40	0.425	98	99	
#50	0.300	95	98	
#100	0.150	92	95	
#200	0.075	80.0	85.7	
Fineness Modulus		2.07	2.05	
Coef. of Curvature (C_C)		0.4	0.5	
Coef. of Uniformity (C_U)		62.7	47.6	
% Gravel		0	0	
% Sand		20	14	
% Fines		80.0	85.7	
USCS Class:		Silt/Clay	Silt/Clay	

Comments:

PROJECT SPECIFICATIONS

Operating Range		Contract Compliance	
Min.	Max.	Min.	Max.

Notes

N.R.: Not Recorded; N/A: Not Available.
*Result is **SHADED** if outside Contract Compliance*
*Result is **BOLD** if outside Operating Range.*

Reviewed By:

NV5 WEST, INC.
1868 Palma Drive, Suite A, Ventura, CA
An NV5 Company - Offices Nationwide

June 26, 2019

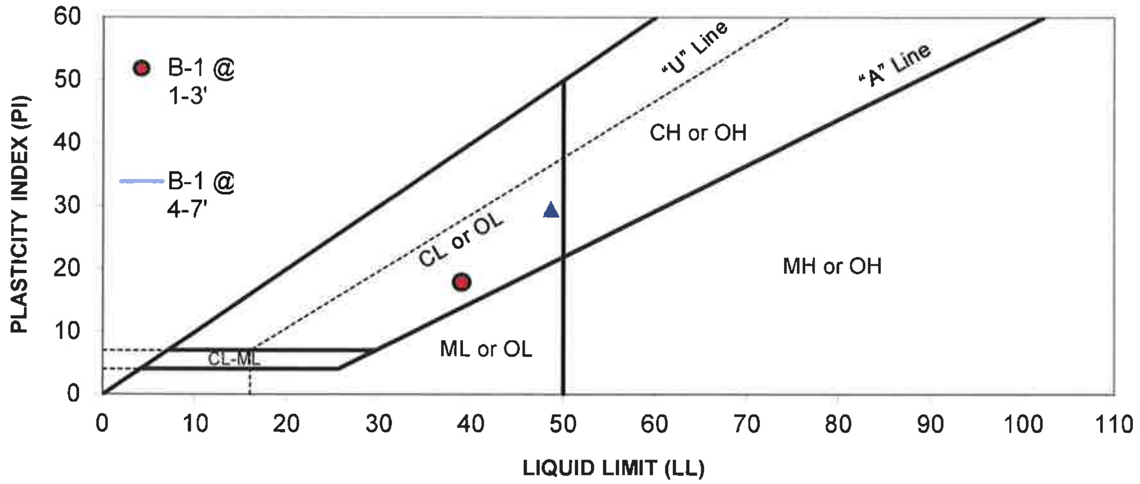
NV5 JOB No: 18-002374.06
LAB No: 87260

Geosolutions, Inc.
220 High Street
San Luis Obispo, CA 93401

REPORT OF LIQUID LIMIT, PLASTIC LIMIT & PLASTICITY INDEX TESTS

(ASTM 4318)

Project: Geosolutions, Inc. - #SM00301-1 (Escalante Meadows)
Site Address:
Sampled By: KR
Date Received: 6/11/2019



SUMMARY OF TEST RESULTS

SAMPLE ID	SOURCE / LOCATION	DEPTH/ ELEV.	%>#40	TEST RESULT			USCS	
				LL	PL	PI	Class	Group Name
B-1	B-1 @ 1-3'	1-3'		**39**	**21**	**18**	CL	lean CLAY
B-1	B-1 @ 4-7'	4-7'		**49**	**19**	**30**	CL	lean CLAY

Reviewed By:

N|V|5

NV5 WEST, INC.
1868 Palma Drive, Suite A, Ventura, CA
An NV5 Company - Offices Nationwide

June 26, 2019

Geosolutions, Inc.
220 High Street
San Luis Obispo, CA 93401

NV5 JOB No: **18-002374.06**
LAB No: 87260

Project: Geosolutions, Inc. - #SM00301-1 (Escalante Meadows)

Sampled By: KR

Amount of Material Finer than No. 200 (75µm) Sieve

(ASTM C117)

Sample	Size of Initial Dry Mass (g)	Percentage Finer than 75µm (No.200) by Washing (%)
B-1 @ 9'	543.1	20.2
B-1 @ 14'	283.6	86.4
B-1 @ 24'	386.8	93.9
B-6 @ 14'	252.6	41.3
B-6 @ 19'	350.5	8.2
B-6 @ 24'	463.3	9.5

N/AVA: Not Available.

Reviewed By:

NV5 WEST, INC.
1868 Palma Drive, Suite A, Ventura, CA
An NV5 Company - Offices Nationwide

June 26, 2019

Geosolutions, Inc.
220 High Street
San Luis Obispo, CA 93401

NV5 JOB No: **18-002374.06**
LAB No: **87260**

Attention: Kelly Robinson

Project: **Geosolutions, Inc. - #SM00301-1 (Escalante Meadows)**
Material: **Dark Brown CLAY (CL)**
Source: **B-1 @ 1-3'**
Date Sampled: **05/31/19**
Sampled By: **KR**

EXPANSION INDEX OF SOILS

(ASTM D4829)

Expansion Index		Potential	Measured Expansion Index
From	To		
	20	Very Low	
21	50	Low	
51	90	Medium	
91	130	High	**108**
130	130 +	Very High	

Saturation at test: **50.2**
Moisture at Test Time : **10.5**

Reviewed By: ____________________

N|V|5

NV5 WEST, INC.
1868 Palma Drive, Suite A, Ventura, CA
An NV5 Company - Offices Nationwide

June 26, 2019

Geosolutions, Inc.
220 High Street
San Luis Obispo, CA 93401

NV5 JOB No: **18-002374.06**
LAB No: **87260**

Attention: Kelly Robinson

Project: **Geosolutions, Inc. - #SM00301-1 (Escalante Meadows)**
Material: **Light Brown CLAY (CL)**
Source: **B-1 @ 4-7'**
Date Sampled: **05/31/19**
Sampled By: **KR**

EXPANSION INDEX OF SOILS

(ASTM D4829)

Expansion Index		Potential	Measured Expansion Index
From	To		
	20	Very Low	
21	50	Low	
51	90	Medium	
91	130	High	**116**
130	130 +	Very High	

Saturation at test: **51.4**
Moisture at Test Time : **9.3**

Reviewed By: ____________________

Project No. **18-2374.06**
Client: **GeoSolutions, Inc.**
Proj. Name: **SM00301.1**
Location: **Escalante Meadows**
Sample date **5/31/2019** Sample Location: **B-1** Boring No: B-1@3'

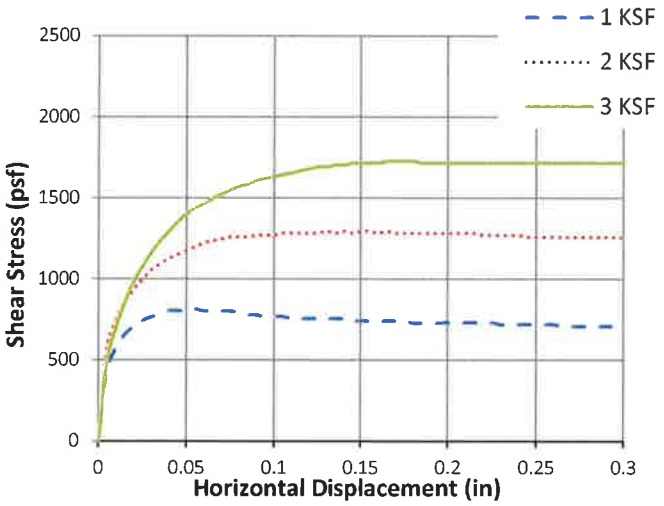
Lab No.: **87260**
Test Date: **6/13/2019**

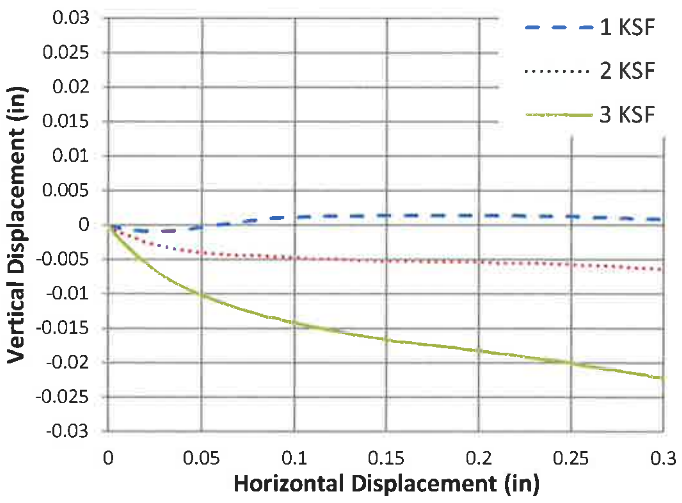
TEST DATA:

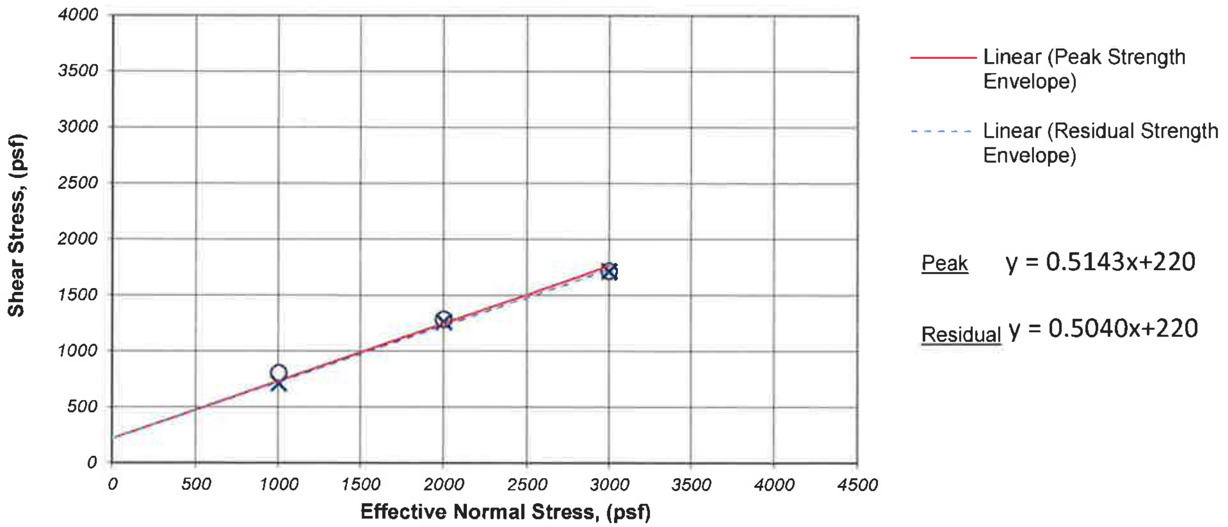
	Sample ID:	1 KSF	2 KSF	3 KSF
Initial	Water Content (%)	21.8	21.8	21.8
	Dry Density (%)	98.9	98.9	98.9
	Saturation (%)	88.5	88.5	88.5
Normal Stress (psf)		1000	2000	3000
Peak Shear Stress (psf)		804	1284	1716
Residual Shear Stress (psf)		708	1260	1716
Strain Rate: 0.0015 in/min				

Sample Type: Undisturbed

Description: Silty CLAY (CL), dark brown







Peak y = 0.5143x+220

Residual y = 0.5040x+220

Peak Cohesion, C'(psf): 220
Peak Friction,Φ' (deg): 27.2

Residual Cohesion, C'(psf): 220
Residual Friction,Φ' (deg): 26.7

DIRECT SHEAR TEST (ASTM D3080)

Reviewed By:

Figure No.: DS.B-1@1-3ft

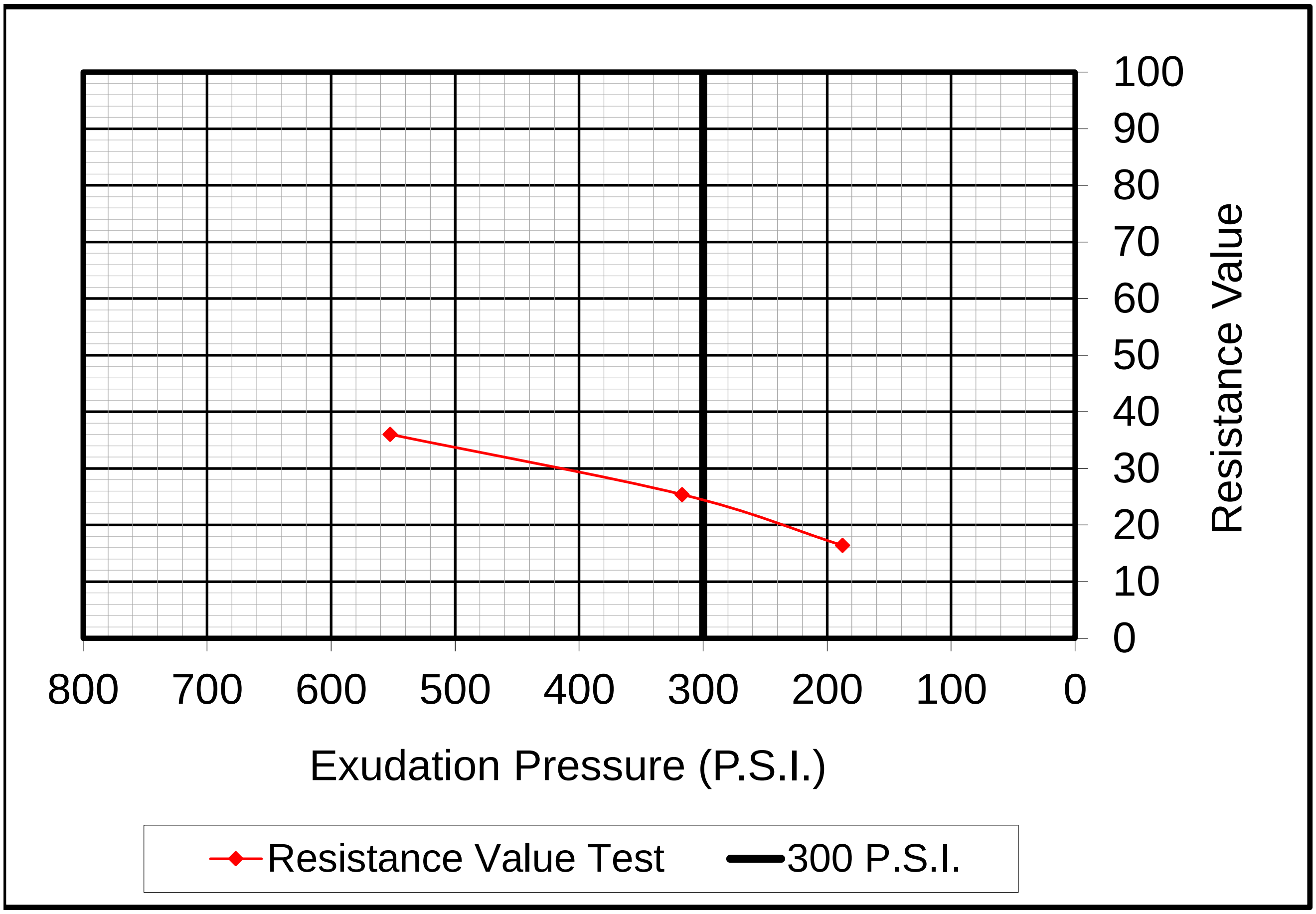
NV5

NV5 WEST, INC.
1868 Palma Drive Suite A
Ventura CA 93003
P. (805)656-6074 F. (805)650-6264

RESISTANCE (R) VALUE TEST

California Test 301

Laboratory No.:	L191099
Project No.:	190035 (GeoSolutions Project No.: SM00301-1)
Sample Date:	June 4, 2019
Report Date:	June 27, 2019
Client:	GeoSolutions, Inc.
Project Name:	2019 Laboratory Testing
Sample Description:	Brown Clayey Sand
Sample Location:	B-1 @ 0-1.5'



Specimen No.	7	8	9
Moisture Content (%)	18.1	19.5	17.5
Dry Density (PCF)	103.6	102.3	105.3
Resistance Value (R)	25	16	36
Exudation Pressure (PSI)	317	188	552
Expansion Pressure	139	100	217
As Received Moisture Content (%)	10.3		

RESISTANCE VALUE AT 300 P.S.I. **24**

Reviewed By:

Brandon Rodebaugh
Materials Engineer

AASHTO R18

APPENDIX C

Seismic Hazard Analysis

Design Map Summary (SEAOC, 2019)

SEISMIC HAZARD ANALYSIS

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

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

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

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

Spectral response accelerations, peak ground accelerations, and site coefficients provided in this report were obtained using the web-based Seismic Design Maps tool available from the Structural Engineers Association of California (SEAOC, 2019). This program utilizes the methods developed in ASCE 7-10 in conjunction with user-inputted Site location to calculate seismic design parameters and response spectra (both for period and displacement) for soil profile Site Classifications A through E. Output from the web-based program are included in this Appendix.

Escalante Meadows

1091 Escalante St, Guadalupe, CA 93434, USA

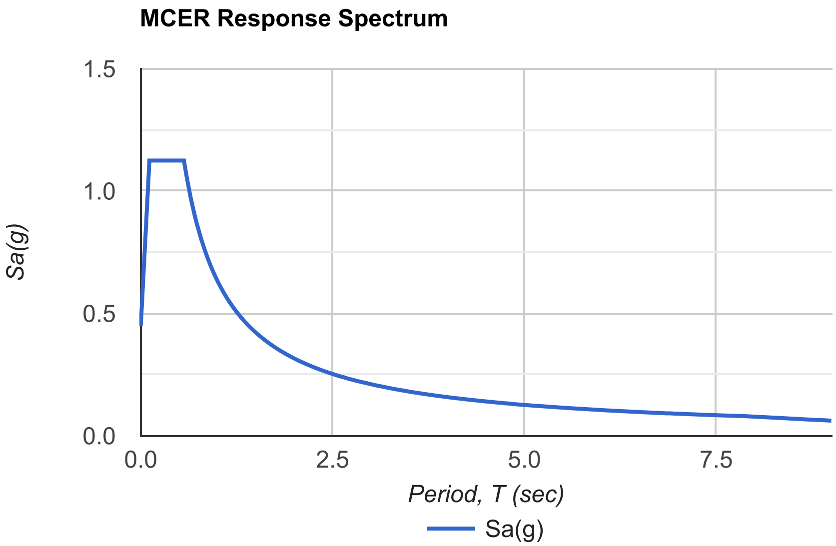
Latitude, Longitude: 34.9700871, -120.5659321

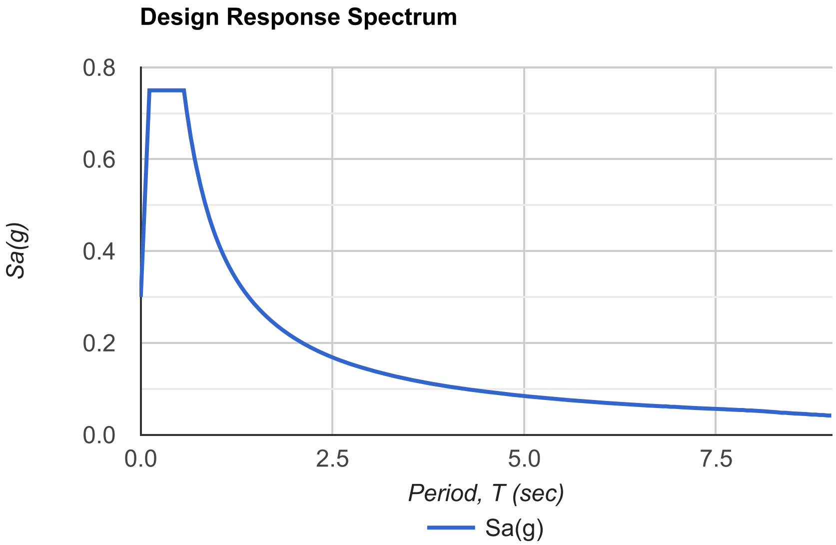
Date	5/10/2019, 10:06:58 AM
Design Code Reference Document	ASCE7-10
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.037	MCE_R ground motion. (for 0.2 second period)
S_1	0.391	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.125	Site-modified spectral acceleration value
S_{M1}	0.633	Site-modified spectral acceleration value
S_{DS}	0.75	Numeric seismic design value at 0.2 second SA
S_{D1}	0.422	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.085	Site amplification factor at 0.2 second
F_v	1.618	Site amplification factor at 1.0 second
PGA	0.414	MCE_G peak ground acceleration
F_{PGA}	1.086	Site amplification factor at PGA
PGA_M	0.449	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.037	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.103	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.52	Factored deterministic acceleration value. (0.2 second)
S1RT	0.391	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.396	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.571	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.94	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.989	Mapped value of the risk coefficient at a period of 1 s


MCER Response Spectrum
1.5
1.0
0.5
0.0
Sa(g)
0.0
2.5
5.0
7.5
Period, T (sec)
Sa(g)


Design Response Spectrum
0.8
0.6
0.4
0.2
0.0
Sa(g)
0.0
2.5
5.0
7.5
Period, T (sec)
Sa(g)

APPENDIX D

Liquefaction Hazard Analysis

Results of Liquefaction Analysis

LIQUEFACTION HAZARD ANALYSIS

GeoSolutions, Inc. utilized computer software program CLiq (GeoLogismiki, 2006), to determine the liquefaction, settlement, and lateral spreading potential at the Site using CPT data from the field investigation. The program incorporates the methodology recommended in the most recent publications of the NCEER Workshop (Youd et al., 2001) and SP117 Implementation (CDMG, 2008). The program requires a user-defined peak ground acceleration, earthquake magnitude and depth to groundwater to assess the potential for liquefaction and consequential vertical settlement estimate. The lateral spreading calculation incorporates the site geometry including the ground slope (%) or, alternatively the height of the free-face (H) and distance to the free-face (L).

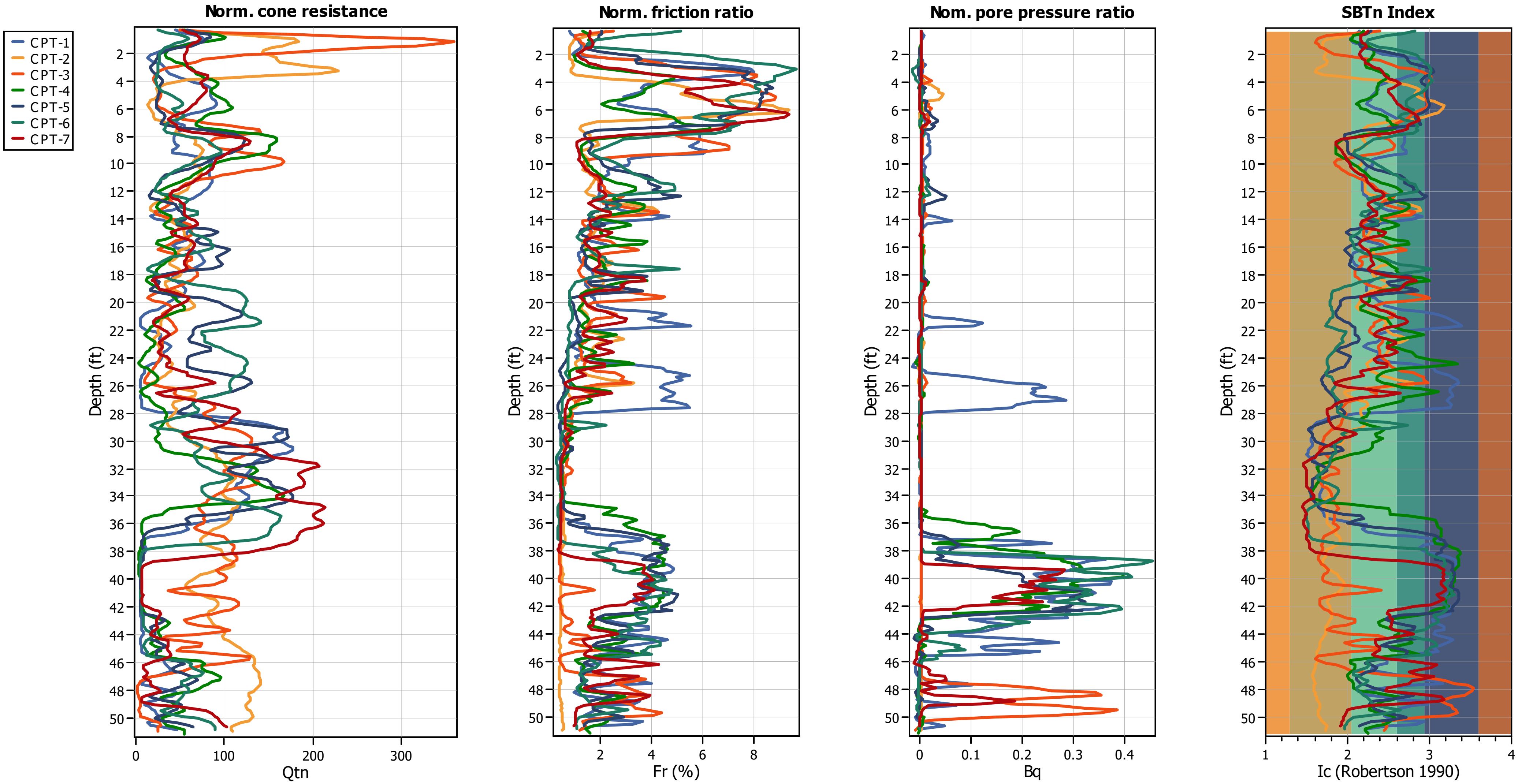
Within the software program, the Cyclic Stress Ratio (CSR) was estimated from the seismic load using Seed's simplified method (Seed and Idriss, 1971) and the user-defined peak ground acceleration. The CSR was then multiplied by a magnitude scaling factor (MSF) which is determined using the methods described by Youd et al. (2001) and the user-defined earthquake magnitude to produce a magnitude-weighted CSR ($CSR_{7.5}$).

For this analysis, the peak ground acceleration was taken as the site-specific mean peak ground acceleration (**PGA_M**) determined in Section 5.0 (**PGA_M = 0.449 g**). Based on our understanding of the seismicity in the region, an earthquake magnitude **M_w = 7.0** was used to determine the MSF for the Site. The depth to groundwater was determined from field measurements at the time of the CPT soundings.

The software then compared the $CSR_{7.5}$ with the computed Cyclic Resistance Ratio (CRR) of the soil, determined from the raw CPT data in accordance with the 1997 NCEER recommendations summarized in Youd et al. (2001). The result of the analysis yielded Factors of Safety (FS) for isolated SAND type layers located below the groundwater table. The analysis considered liquefiable layers with a factor of safety against liquefaction less than 1.3 within the upper 50 feet of the soil profile. Results of the analysis are included in this Appendix.

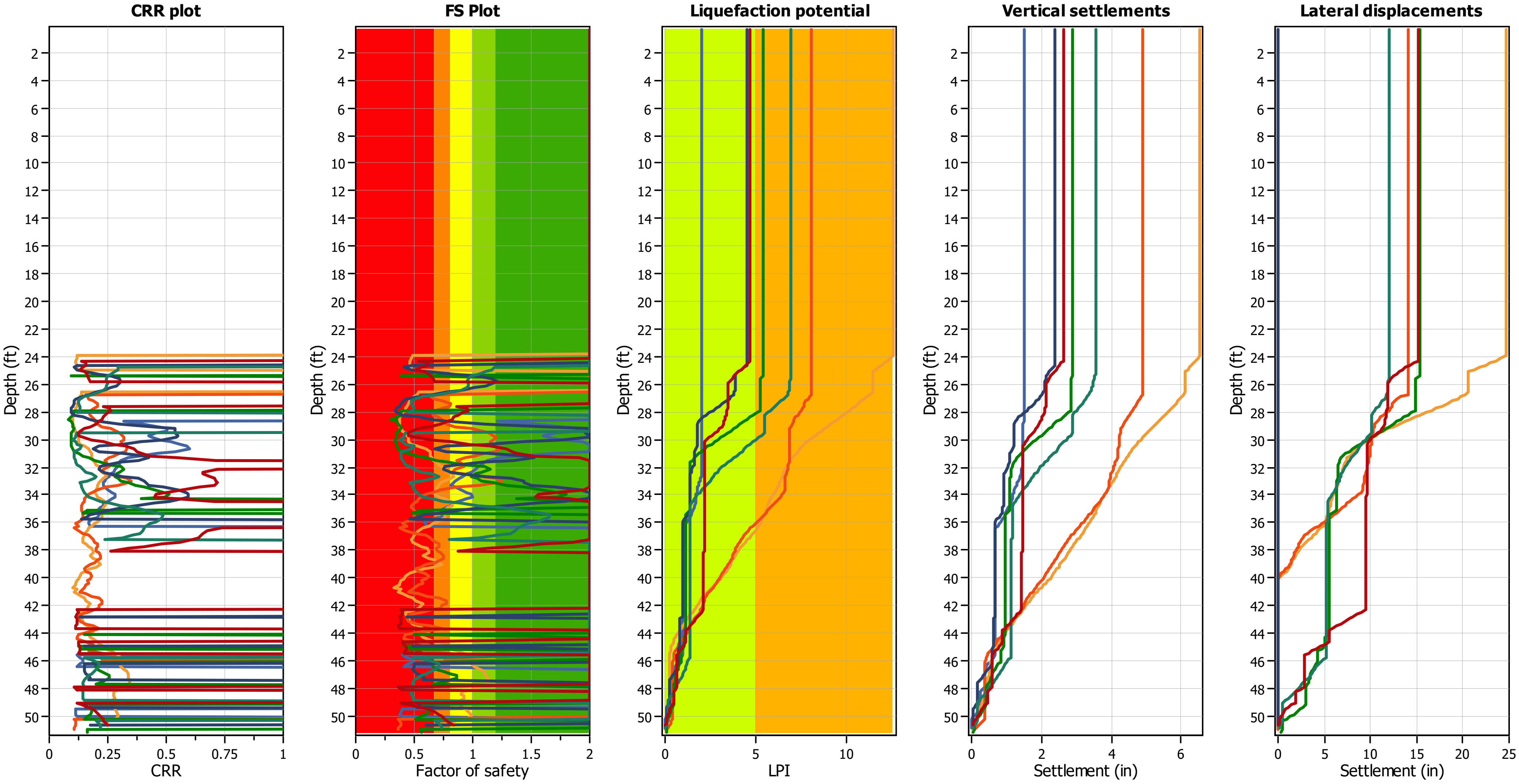
Project: Escalante Meadows

Overlay Normalized Plots



Project: Escalante Meadows

Overlay Cyclic Liquefaction Plots



APPENDIX D

Preliminary Grading Specifications

PRELIMINARY GRADING SPECIFICATIONS

A. General

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

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

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

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

B. Obligation of Parties

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

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

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

C. Site Preparation

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

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

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

D. Site Protection

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

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

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

E. Excavations

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

2. Unless otherwise recommended by the Soils Engineer and approved by the local building official, permanent cut slopes should not be steeper than 2:1 (horizontal to vertical). Final slope configurations should conform to section 1804 of the 2016 California Building Code unless specifically modified by the Soil Engineer/Engineering Geologist.

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

F. Structural Fill

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

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

G. Compacted Fill

1. Structural fill using approved import or native should be placed in horizontal layers, each approximately 8 inches in thickness before compaction. On-site inorganic soil or approved imported fill should be conditioned with water to produce a soil water content near optimum moisture and compacted to a minimum relative density of 90 percent based on ASTM D1557-12$_{e1}$.

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

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

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

H. Drainage

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

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

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

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

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

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

I. Maintenance

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

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

J. Underground Facilities Construction

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

2. Bedding is defined as material placed in a trench up to 1 foot above a utility pipe and backfill is all material placed in the trench above the bedding. Unless concrete bedding is required around

utility pipes, free-draining sand should be used as bedding. Sand to be used as bedding should be tested in our laboratory to verify its suitability and to measure its compaction characteristics. Sand bedding should be compacted by mechanical means to achieve at least 90 percent relative density based on ASTM D1557-12$_{e1}$.

3. On-site inorganic soils, or approved import, may be used as utility trench backfill. Proper compaction of trench backfill will be necessary under and adjacent to structural fill, building foundations, concrete slabs, and vehicle pavements. In these areas, backfill should be conditioned with water (or allowed to dry), to produce a soil water content of about 2 to 3 percent above the optimum value and placed in horizontal layers, each not exceeding 8 inches in thickness before compaction. Each layer should be compacted to at least 90 percent relative density based on ASTM D1557-12$_{e1}$. The top lift of trench backfill under vehicle pavements should be compacted to the requirements given in report under Preparation of Paved Areas for vehicle pavement sub-grades. Trench walls must be kept moist prior to and during backfill placement.

K. Completion of Work

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

2. Soils Engineers shall submit a statement that, to the best of their knowledge, the work within their area of responsibilities is in accordance with the approved soils engineering report and applicable provisions within Chapter 18 of the 2016 CBC.