APPENDIX E

GEOTECHNICAL REPORT

Geotechnical & Environmental Solutions

SUPPLEMENTAL GEOTECHNICAL EVALUATION AND UPDATE

Proposed Mixed Use Development
The Koll Center Residences
City of Newport Beach, Orange County, California

October 31, 2016

EEI Project No. SHO-72189.4a

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SUPPLEMENTAL GEOTECHNICAL EVALUATION AND UPDATE

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1.0 INTRODUCTION

1.1 Purpose

The purpose of this Supplemental Geotechnical Evaluation is to provide preliminary geotechnical information to Shopoff Land Fund II, LLC ("Client") regarding the subject property in the City of Newport Beach, Orange County, California. The information gathered in this evaluation is intended to provide the Client with an understanding of the physical conditions of site-specific subsurface soils, groundwater, and the regional geologic setting which could affect the cost or design of the proposed development at the property (Site Location Map-**Figure 1**, Aerial Site Map-**Figure 2**).

EEI previously completed a Geotechnical Evaluation for the subject property in 2015. However, the proposed site design has been revised in scope, and as a result, this Supplemental Geotechnical Evaluation and Update for the current proposed design has been prepared.

EEI conducted supplemental onsite field explorations on October 10 and 18, 2016, which included drilling, logging and sampling of three (3) hollow stem auger borings and three (3) additional CPT soundings at the subject property. Previously, EEI conducted onsite field exploration on July 16, 17 and 21, 2015, which included drilling and sampling of seven (7) hollow stem auger geotechnical borings and four (4) Cone Penetrometer Test (CPT) soundings for the proposed development at the property.

This Supplemental Geotechnical Evaluation has been conducted in general accordance with the accepted geotechnical engineering principles and in general conformance with the approved proposal and cost estimate for the project by EEI, dated September 29, 2016. This Supplemental Geotechnical Evaluation has been prepared for the sole use of Shopoff Land Fund II, LLC. Other parties, without the express written consent of EEI and Shopoff Land Fund II, LLC should not rely upon this Supplemental Geotechnical Evaluation.

1.2 Project Description

Based on our conversation with you and our review of a conceptual plan provided by David Evans and Associates, dated September 20, 2016, it is our understanding that the proposed mixed-use development will include the addition of a parking structure including five levels above grade with three levels subterranean. Based on the information provided, the overall project development includes 260 residential units, ground floor retail, residential and commercial parking along with related improvements. Related improvements include paved parking and drive areas, underground utilities and other related improvements.

No grading plans were available at the time of our preparation of this evaluation; however, grading at the property is anticipated to include cut up to approximately 40 feet across portions of the subject property to achieve grades for three levels of subterranean parking (exclusive of any remedial earthwork). No foundation plans were available. Foundations are assumed to be typical for the type of building construction proposed. The existing improvements will be razed prior to new construction. No further information is known at this time.

1.3 Scope of Services

The scope of our services included:

- A review of readily available data pertinent to the subject property, including published and unpublished geologic reports/maps, and soils data for the area (**References**).

- Conducting a geotechnical reconnaissance of the subject property and nearby vicinity.
- Coordination with Underground Service Alert, a private utility locating service and property personnel to identify the presence of underground utilities for clearance of proposed boring locations.
- Subsurface exploration consisting of ten (10) small-diameter hollow stem auger borings (B-1 through B-10) and seven (7) Cone Penetration Test (CPT) Soundings. Borings were drilled to depths ranging from approximately 10 to 51½ feet below the existing ground surface. CPT soundings were advanced to depths ranging from approximately 55 to 75 feet below the existing ground surface within the area of the proposed developments. The approximate location of each of CPT soundings and borings are presented on **Figure 3 -** Field Exploration Map.
- The performance of four (4) field percolation tests at approximate depths ranging from 12 to 20 feet below the ground surface to provide preliminary information for stormwater design purposes. Testing was performed in accordance with County of Orange DEH guidelines for percolation test methods.
- An evaluation of seismicity and geologic hazards to include an evaluation of faulting and liquefaction potential.
- Completion of laboratory testing of representative earth materials encountered onsite to ascertain their pertinent soils engineering properties, including corrosion potential (**Appendix B**).
- The preparation of this report which presents our preliminary findings, conclusions, and recommendations.

2.0 BACKGROUND

2.1 Subject Property Description

The proposed development is located within the Koll Center in the City of Newport Beach, Orange County, California. Based on a review of online resources and information provided by you, the subject property is located on one contiguous parcel, identified by Assessor's Parcel Number (APN) 445-131-028. The overall property encompasses approximately 28-acres and appears to be developed with five large commercial/retail buildings and paved parking and drive areas. The proposed development is approximately 6.26 acres and is located within existing paved parking and drive areas within the north-central and southern portions of the Koll Center. In general, the property is situated within the northerly portion of the Koll Center, and is bordered by Birch Street to the east, Von Karman to the west and existing commercial/retail buildings and paved parking and drive areas to the north and south.

The center of the subject property is approximately situated at 33.6659° north latitude and 117.8603° west longitude (GoogleEarth™, 2015).

2.2 Topography

The subject property is located within the 7.5 minute Tustin, California Quadrangle at an elevation of approximately 50 feet above sea level (USGS, 2015).

In general, the overall subject property ground surface is relatively level with a gentle slope to the west. Surface drainage also appears to be generally directed to the west.

2.3 Geologic Setting

Regionally, the subject property lies within the Peninsular Ranges Geomorphic Province of southern California. This province consists of a series of ranges separated by northwest trending valleys; sub parallel to branches of the San Andreas Fault (CGS, 2002).The Peninsular Ranges geomorphic province, one of the largest geomorphic units in western North America, extends from the Transverse Ranges geomorphic province and the Los Angeles Basin, south to Baja California. It is bound on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province. The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks (CGS, 2002). Major fault zones and subordinate fault zones found in the Peninsular Ranges Province typically trend in a northwest-southeast direction.

Regional geologic maps of the subject property and vicinity (published by the United States Geological Survey - USGS) indicate the property is underlain by late to middle Pleistocene-aged Old Paralic deposits overlain by alluvial fan deposits (map symbol $Qopf_a$). The alluvial fan deposits generally consist of cobble, gravel, sand and silt deposits issued from confined valleys, while the old paralic deposits generally consist of fine-grained sand, silt, and clay from lake, playa and estuarine deposits.

The subject property is located within an area of California known to contain a number of active and potentially active faults. The property is not located within a State of California Earthquake Fault Zone (Hart and Bryant, 1997). The active San Joaquin Hills fault is located approximately 2.6 miles from the property.

Regional seismic hazard maps (CDMG, 2001) for the subject property area indicate that the property is located within an area that is not considered susceptible to landsliding, liquefaction and/or seismic induced settlement. Additionally, no historic landslides were mapped within or adjacent to the property, nor were there any indication of landslides encountered during our site reconnaissance.

2.4 Groundwater

At the time of our subsurface exploration, a zone of heavy seepage was encountered at depths ranging from 20 to 25 feet below the ground surface. Additionally, pore pressure dissipation testing performed in CPT sounding CPT-1 indicates that groundwater seepage was present at a depth of approximately 23 feet below the ground surface at the time of testing.

According to nearby groundwater data obtained from the Orange County Water District, the principal groundwater aquifer has ranged from approximately 50 to 110 feet below existing ground surface at the subject property in the past 10 years (Orange County Water District, 2015). In general, groundwater is expected to follow the direction of surface topography; therefore, local groundwater flow is expected to be in a general westerly direction. It should be noted that variations in groundwater may result from fluctuations in the ground surface topography, subsurface stratification, rainfall, irrigation and other factors that may not have been evident at the time of our subsurface exploration.

3.0 REGIONAL FAULTING AND SEISMICITY

The portion of Southern California that includes the subject property is considered to be seismically active. Due to the proximity of the property area to several nearby active faults, strong ground shaking could occur at the property as a result of an earthquake on any one of the faults. Our review indicates that there are no known active faults crossing the property and the property is not located within an Alquist-Priolo Earthquake Fault Zone as defined by the State of California (Hart and Bryant, 1997, CDMG, 2000).

While the potential risk of ground rupture cannot be completely ruled out, it is our opinion that the likelihood of surface fault rupture at the property is relatively low and the risk is considered similar to other sites in the vicinity.

Table 1 provides a summary of active fault zones within an approximately 40 mile radius of the subject property that may have a considerable effect on the property in the event significant activity is experienced. Fault names and approximate distances are based upon information provided in applicable references (Blake, 2000; Jennings, 1994).

TABLE 1
Summary of Major Active Faults

Fault Name	Approximate Distance From Site miles (kilometers)	Maximum Moment Magnitude
San Joaquin Hills	2.6 (4.1)	6.6
Newport-Inglewood (L.A. Basin)	5.5 (8.8)	7.1
Newport-Inglewood (Offshore)	6.0 (9.7)	7.1
Chino-Central Avenue (Elsinore)	15.8 (25.4)	6.7
Whittier	17.0 (27.3)	6.8
Palos Verdes	17.1 (276)	7.3
Puente Hills Blind Thrust	18.0 (29.0)	7.1
Elsinore (Glen Ivy)	18.3 (29.4)	6.8
San Jose	25.8 (41.6)	6.4
Coronado Bank	27.7 (44.6)	7.6
Elsinore (Temecula)	29.5 (47.5)	6.8
Upper Elysian Park Blind Thrust	31.0 (49.9)	6.4
Sierra Madre	32.2 (51.9)	7.2
Cucamonga	32.6 (52.5)	6.9
Raymond	34.4 (55.4)	6.5
Clamshell-Sawpit	36.2 (58.3)	6.5
Verdugo	36.3 (58.4)	6.9
Hollywood	37.8 (60.9)	6.4

3.1 Seismic Parameters and Peak-Ground Acceleration

Maximum considered ground motion maps provided in the California Building Code (CBC, 2013) were utilized with coordinates of 33.6659 ° north latitude and 117.8603° west longitude, to determine the subject property seismic parameters. EEI utilized seismic design criteria provided in the CBC (2013).

In accordance with the guidelines of the CBC (2013), the spectral parameters for the subject property (based on a Site Class B soil) are estimated to be S_s = 1.577g and S_1 = 0.578g.

Review of the geotechnical data obtained during our subsurface exploration, however, indicates that the property should be classified as Class D per ASCE 7-10 (Table 20-3.1). Consequently, Site Coefficients F_a= 1.000 and F_v = 1.500 appear to be appropriate for the subject property. Based on this information, the adjusted maximum considered earthquake spectral response parameters S_{MS} = 1.577g and S_{M1} = 0.867g and the spectral acceleration parameters of S_{DS} value of 1.051g and an S_{D1} value of 0.578g are recommended for seismic design of the project.

Final selection of the appropriate seismic design coefficients should be made by the structural consultant based on the local laws and ordinances, expected building response, and desired level of conservatism.

Seismic Hazard Response Parameters are listed in **Table 2**.

TABLE 2
Seismic Hazard Response Parameters and Design Parameters CBC (2013)

Seismic Parameter	Period (Sec)		Value
Mapped Spectral Acceleration Value, Site Class B	0.2	S_s	1.577g
Mapped Spectral Acceleration Value, Site Class B	1.0	S_1	0.578g
Site Coefficient, Subject Site Class D per 2013 CBC Table 1613.3.3	--	F_a	1.000
Site Coefficient, Subject Site Class D per 2013 CBC Table 1613.3.3	--	F_v	1.500
Adjusted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Site Class D	0.2	S_{MS}	1.577g
Adjusted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Site Class D	1.0	S_{M1}	0.867g
Design Spectral Response Acceleration Occupancy Category II per 2013 CBC Table 1604.5	0.2	S_{DS}	1.051g
Design Spectral Response Acceleration Occupancy Category II per 2013 CBC Table 1604.5	1.0	S_{D1}	0.578g
Peak Ground Acceleration Adjusted For Site Class Effects		PGA_M	0.615g

3.2 Ground Lurching or Shallow Ground Rupture

Based on the geography, topography and site-specific geotechnical conditions encountered during our geotechnical evaluation at the subject property, we consider the potential for ground lurching or shallow ground rupture at the property to be low. However, due to the active seismicity of California and the close proximity of the property to mapped portions of the San Joaquin Hills Fault, this possibility cannot be completely ruled out. In light of this, the unlikely hazard of lurching or ground-rupture should not preclude consideration of “flexible” design for onsite utility lines and connections.

3.3 Liquefaction, Seismic Settlement and Lateral Spreading

Liquefaction occurs when loose, saturated, sands and silts are subjected to strong ground shaking. The strong ground shaking causes pore-water pressure to increase and soil shear strength to decrease, potentially resulting in large total and differential ground surface settlements as well as possible lateral spreading during an earthquake.

EEI reviewed readily available and relevant maps and publications regarding liquefaction potential at the subject property.

It should be noted that the subject property is indicated to be within an area that is not considered susceptible to liquefaction based on a review of the Seismic Hazard Zones Map for the property vicinity (CDMG, 2001). According to the Seismic Hazard Map prepared by the City of Newport Beach (2008), the property is not located within a liquefaction zone.

Based on the seismic hazard report for the subject property and vicinity, historic high groundwater in the vicinity is indicated to be approximately 10 feet bgs (CDMG, 1998). However, groundwater data obtained from the Orange County Water District indicates the principal groundwater aquifer has ranged from approximately 50 to 110 feet below existing ground surface at the property in the past 10 years (Orange County Water District, 2015). As previously discussed, a zone of heavy seepage was encountered at depths ranging from 20 to 25 feet below the ground surface during our subsurface exploration. Additionally, pore pressure dissipation testing performed in CPT sounding CPT-1 indicated that groundwater was present at a depth of approximately 23 feet below the ground surface. Based on this information and the bottom elevations of the proposed development (two-to three-levels of subterranean parking), we assessed the liquefaction potential for the property utilizing a groundwater depth of 20 feet bgs.

The liquefaction potential was evaluated using the CLiq computer program (Geologismiki, 2015) using the CPT data from the 2015 and 2016 subsurface explorations. Our evaluation was based on the site class adjusted peak ground acceleration of 0.615g, obtained from the USGS Seismic Design Maps and Equation 11.8-1 of ASCE 7-10. Based on this reference a peak-ground acceleration of 0.615g is obtained, which is the value used in our evaluation as is presented in **Table 3**. Results of our seismic hazard deaggregation yielded a probabilistic 2,475 year modal magnitude (2 percent probability of exceedance in 50 years) of 6.96. Based on our evaluation, we consider the subject property to be susceptible to limited amounts of liquefaction. Generally, our evaluation indicates that potentially liquefiable soils consist of isolated and discontinuous thin lenses of saturated sands, silts and clays. The results of our liquefaction evaluation are included as **Appendix C**.

We estimate total seismic-induced settlement will vary between approximately less than ½- and 1-inch across the subject property. Differential seismic settlements are estimated at approximately ½-inch over a distance of 40 feet. The results of the liquefaction analysis are provided in **Appendix C** – Liquefaction Evaluation. Estimates of seismic settlement at each of the CPT sounding locations are provided in the following table.

TABLE 3
Estimated Seismic Settlement

CPT Sounding	Estimated Seismic Settlement (inches)	Maximum Depth of Liquefiable Material (feet)
CPT-1	1.0	55
CPT-2	0.60	42
CPT-3	0.70	52
CPT-4	0.70	54
CPT-5	0.02	51
CPT-6	0.20	53
CPT-7	0.20	55

Cyclic mobility is a liquefaction phenomenon triggered by cyclic loading, occurring in soil deposits with static shear stresses lower than the soil strength. Deformations due to cyclic mobility develop incrementally because of static and dynamic stresses that exist during an earthquake. Lateral spreading, a common result of cyclic mobility, can occur on gently sloping and on flat ground close to rivers and lakes. These conditions do not exist within the subject property, given the relatively level topography of the property and the lack of channel free faces in the general vicinity of the property.

4.0 FIELD EXPLORATION AND LABORATORY TESTING

4.1 Field Exploration

Field work for our Preliminary Geotechnical Evaluation was conducted on July 16, 17 and 21, 2015, and for our Supplemental Geotechnical Evaluation on October 10 and 18, 2016. Combined, a total of ten (10) hollow stem auger borings and seven (7) CPT soundings were advanced at the subject property. Borings were logged and sampled under the supervision of a Registered Professional Engineer and Certified Engineering Geologist at EEI. The approximate locations of the borings and CPT soundings are shown on **Figure 3**.

Exploratory Borings (2015 and 2016) - The ten (10) exploratory borings (B-1 through B-10) extended to depths of approximately 10 to 51½ feet below the existing ground surface (bgs) were logged and sampled under the supervision of a geologist with EEI.

Blow count (N) values were determined utilizing a 140 pound hammer, falling 30-inches onto a Standard Penetration Test (SPT) split-spoon sampler and a Modified California split-tube sampler. A truck mounted HSA drill rig was used to advance the borings. The blows per 6-inch increment required to advance the 18-inch long SPT and 18-inch long Modified California split-tube samplers was measured at various depth intervals (varying between 2 to 5 feet), or at changes in lithology, recorded on the boring logs, and are presented in **Appendix A**-Soil Classification Chart and Boring Logs.

Relatively “undisturbed” samples were collected in a 2.42-inch (inside diameter) California Modified split-tube sampler for visual examination and laboratory testing. Representative bulk samples were also collected for laboratory testing. The soils were classified in accordance with the Unified Soil Classification System (ASTM, 2008).

Additionally, field percolation testing was performed in four of the exploratory borings (B-6, B-7, B-9 and B-10) at depths ranging from 10 and 20 feet below the ground surface. Testing was performed in accordance with County of Orange DEH guidelines for percolation test methods.

CPT Soundings (2015 and 2016) - The CPT soundings were performed by Middle Earth Geo Testing Inc., under the supervision of a representative of EEI. CPT testing was conducted in general accordance with ASTM Test Method D3441. The CPT procedure includes pushing an electronic cone penetrometer, which records data including tip resistance, sleeve friction and dynamic pore pressure as it is advanced. A 25-Ton CPT rig equipped with a 10 square centimeter cone was used to conduct the CPT soundings. The CPT soundings were each advanced to approximate depths of 75 feet or to refusal, which was encountered at depths ranging from 55 to 65 feet bgs. CPT data are presented in **Appendix A**.

4.2 Subsurface Conditions

Subsurface conditions encountered in our exploratory borings and CPT soundings consisted of asphalt pavement and base, artificial fill and Pleistocene-aged old paralic deposits. Fill materials were encountered in each of the exploratory borings, and extended to depths ranging from approximately 2 to 5 feet below the ground surface across the subject property. In general, the fill was composed of reddish brown to dark reddish brown mixed sands, silts and clays. The Pleistocene-age old paralic deposits were encountered underlying the fill. In general, the old paralic deposits consisted primarily of loose to very dense sands with interbedded layers of loose to medium dense silty sand and clayey sand, and soft to hard sandy-clay and silty-clay. A layer of stiff to hard clay was encountered in each of the CPT soundings underlying the sands at depths ranging from approximately 35 to 60 feet below the ground surface. Practical refusal due to heaving and/or dense sands was encountered in exploratory borings B-1 through B-5 at depths ranging from 33 to 49 feet below the ground surface, and CPT soundings CPT-5 through CPT-7. Data obtained from the CPT soundings are generally consistent with materials logged and sampled during the subsurface exploration. Detailed descriptions of the encountered soils are provided on the boring logs and on the CPT logs included as **Appendix A**. Detailed geologic cross-sections shown on plan view in **Figure 1** are also presented on **Figures 4a**, **4b** and **4c**.

At the time of our subsurface explorations, a zone of heavy seepage was encountered at depths ranging from 20 to 25 feet below the ground surface. Additionally, pore pressure dissipation testing performed in CPT sounding CPT-1 indicates that groundwater was present at a depth of 23 feet below the ground surface at the time of testing. The groundwater encountered represented intermittent seepage and perched zones throughout the subject property. It should be noted that variations in groundwater may result from fluctuations in the ground surface topography, subsurface stratification, rainfall, irrigation, and other factors that may not have been evident at the time of our subsurface exploration.

Detailed geologic cross-sections, shown in plan view on **Figure 1,** are provided to show visual subsurface materials and groundwater depths beneath the proposed developments and are presented in **Figure 4a**, **4b** and **4c**.

4.3 Laboratory Testing and Classification

Representative samples were selected for laboratory testing to check their field classification(s) and to evaluate their pertinent engineering characteristics. Field descriptions and soil classifications were visually classified according to the American Society for Testing and Materials (ASTM D2488) which classifies soils under the USCS. Representative soil samples were tested in the lab for grain size distribution to determine actual classifications by ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes in accordance with the USCS. Final classifications of soils can be found on the boring logs in **Appendix A** and the laboratory test data in **Appendix B**.

4.3.1 Moisture Content and Dry Density

The in-situ moisture content and dry density of soils was determined for soil samples obtained from the borings. In-place moisture content and dry density of soils help in the evaluation of engineering design parameters for foundations, retaining walls, and other engineering structures. The moisture content determination of soil samples was conducted in general accordance with ASTM D2216, and was recorded as a percentage. The determination of dry density of soil samples was conducted in accordance with ASTM 2937, and recorded in pounds per cubic foot.

Moisture content and dry density for soil samples retrieved from the field can be found on the boring logs located in **Appendix A**.

4.3.2 Expansion Index

A bulk sample of soils obtained from within 5 feet of the existing grade from Boring B-2 was tested for its expansion potential. Our expansion index testing was conducted in general accordance to ASTM D4829. The results of our expansion index testing are presented in **Appendix B.**

4.3.3 Grain Size Distribution

To help check field classifications of soils, the grain size distribution of representative soil samples was determined. In order to find the percentages of fine grained particles in a particular soil stratum, soils were tested in general accordance with ASTM D422-Standard Test Method for Particle-Size Analysis of Soils. Gradation results are presented in **Appendix B**.

4.3.4 Direct Shear

Direct shear testing was conducted on ten representative samples at varying depths. The samples were tested in their natural state, to measure its shear strength characteristics for engineering purposes. The samples were inundated for at least 18 hours. The samples were placed in a shear box and a normal load was applied (10, 20, and 40 kilogram weights were used). The samples were then sheared at a controlled strain rate in a direct shear apparatus that measures horizontal displacement and shear resistance. Shear testing was run in general accordance with ASTM D3080. The results of our testing are presented in **Appendix B**.

4.3.5 Sulfate/Corrosion

A representative sample of the encountered onsite earth material was collected for analysis at Clarkson Laboratory and Supply, Inc. located in Chula Vista, California for corrosion/soluble sulfate potential. This corrosion testing included soil minimum resistivity and pH by California Test 643 sulfate by California Test 417, and chloride by California Test 422. Results of these tests are presented in **Appendix B**.

5.0 CONCLUSIONS

Based on our field exploration, laboratory testing and engineering and geologic analysis, it is our opinion that the subject property is suitable for the proposed mixed-use development from a geotechnical engineering and geologic viewpoint.

However, there are existing geotechnical conditions associated with the subject property that will warrant mitigation and/or consideration during planning stages. The following conclusions take in consideration the assumption that the property is proposed for the construction of mixed-use residential development and related improvements.

The building structures are proposed to include three towers with up to twelve (12) levels above two (2) levels of above-ground parking and two (2) levels of subterranean parking and an eight (8) level parking structure which includes five (5) levels above grade with three (3) levels subterranean. If site plans and/or the proposed building locations are revised, additional field studies may be warranted to address proposed site-specific conditions. As a result, EEI is providing the following conclusions:

- A total of ten (10) exploratory hollow-stem auger borings were advanced within the subject property boundaries during this evaluation. Exploratory boring depths ranged from approximately 10 feet to approximately 51½ -feet bgs. Additionally, field percolation testing was performed in four (4) of the exploratory borings (B-6, -7, -9 and -10) in accordance with County of Orange DEH guidelines at depths ranging from approximately 10 to 20 feet bgs. Overall, the property is underlain by an approximately 2- to 5-foot layer of artificial fill materials underlain by Pleistocene-aged old paralic deposits. The Pleistocene-age old paralic deposits were encountered underlying the fill. In general, the old paralic deposits consisted of interbedded layers of loose to very dense sand, silty-sand and clayey-sand, and soft to stiff to hard sandy-clay and silty-clay. Practical refusal was encountered in exploratory borings B-1 through B-5 at depths ranging from 33 to 49 feet below the ground surface, and refusal was encountered in CPT soundings CPT-5 though CPT-7.

- A total of seven (7) exploratory Cone Penetrometer Test soundings (CPT), were advanced to an approximate depth ranging from 55 to 75 feet below existing grade elevations. Data obtained from the CPT soundings are consistent with materials logged and sampled during the subsurface exploration.

- At the time of our subsurface explorations (2015 and 2016), a perched zone of heavy seepage was encountered at depths ranging from 20 to 25 feet below the ground surface. According to nearby groundwater data obtained from the Orange County Water District, the principal groundwater aquifer has ranged from approximately 50 to 110 feet below existing ground surface at the subject property in the past 10 years (Orange County Water District, 2015). Additionally, pore pressure dissipation testing performed in CPT sounding CPT-1 indicates that groundwater seepage was present at a depth of 23 feet below the ground surface at the time of testing.

- Laboratory test results performed on two samples from the site (B-1 from 1 to 8 feet and B-2 from 15 to 20 feet bgs) indicate that the tested soils are slightly to strongly alkaline (tested pH value of 7.4 and 8.5, respectively) and are considered highly corrosive to ferrous metals with a tested minimum resistivity value of 2,200 ohm-cm. Laboratory testing also yielded a maximum soluble sulfate concentration of 0.007 percent within the tested samples, indicating a negligible potential for sulfate attack on concrete. A maximum chloride concentration of 0.010 percent was detected within the sample of the upper soils, indicating that the upper soils possess a negligible potential for corrosion of steel reinforcement in concrete.

- The subject property is located within an area of southern California recognized as having a number of active and potentially-active faults located nearby. Our review indicates that there are no known active faults mapped as crossing the property and the property is not located within an Earthquake Fault Zone. The nearest active faults that could affect the property include the San Joaquin Hills fault located approximately 2.6 miles from the property.

Other nearby seismic sources includes the L.A. Basin and Offshore segments of the Newport Inglewood fault, the Elsinore segment of the Chino-Central Avenue fault and the Whittier fault; each of these active faults is capable of generating severe ground shaking at the subject property.

- Based on EEI's evaluation, earth materials underlying the subject property are considered susceptible to limited amounts of seismic induced liquefaction. Based on EEI's evaluation, the earth materials consisting of isolated and discontinuous lenses of saturated sands, silts and clays underlying the property of the proposed development appear to be susceptible to some seismically induced settlement on the order of 1-inch with differential settlements of less than a ½-inch over a 50-foot span. Liquefaction-induced lateral spreading does not appear to be a concern at the property, given the relatively level topography of the property and distance to channel free faces.

- The results of our laboratory Expansion Index (EI) testing of a localized pocket of fine grained materials sampled at a depth of 1 to 8 feet below the ground surface indicate an EI of 107, which represents a high expansion potential for those soils. However, the onsite soils are variable and are anticipated to range from very low to medium to highly expansive.

- Existing fill and native materials appear to be suitable for use as structural fill, provided they are free of deleterious materials and are properly moisture conditioned (as needed) and re-compacted to at least 90 percent of the maximum dry density (based on ASTM D1557). The upper 12-inches of pavement subgrade should be compacted to at least 95 percent of the maximum dry density (based on ASTM D1557).

- EEI evaluated static settlement utilizing results of laboratory testing and subsurface data to estimate settlement as a result of grading the pad(s) to a proposed finish slab grade. Based upon our evaluation and our recommendations for remedial earthwork, and a conventional or mat slab foundation system, EEI estimates total static settlement of less than 1-inch within the building envelope. Differential settlement is estimated to be approximately a ½-inch or less over a distance of 50 feet.

6.0 RECOMMENDATIONS

The recommendations presented herein should be incorporated into the planning and design phases of development. Guidelines for site preparation, earthwork and onsite improvements are provided in the following sections, based on a limited number of widely spaced exploratory borings, and the assumption that the planned development will consist of the construction of mixed-use residential development and related improvements.

6.1 General

Grading should conform to the guidelines presented in the 2013 California Building Code (CBC, 2013) and the requirements of the current edition of the County of Orange Building Code and City of Newport Beach Grading Code. Additionally, general Earthwork and Grading Guidelines are provided herein as **Appendix D**.

During earthwork construction, removals and reprocessing of fill materials, as well as general grading procedures of the contractor should be observed and the fill placed selectively, tested by representatives of the Geotechnical Engineer, EEI. If any unusual or unexpected conditions are exposed in the field, they should be reviewed by the Geotechnical Engineer and if warranted, modified and/or additional remedial recommendations will be offered. Specific guidelines and comments pertinent to the planned development are provided herein.

The recommendations presented herein have been completed using the information provided to us regarding site development. If information concerning the proposed development is revised, or any changes in the design and location of the proposed subject property improvements are made, the conclusions and recommendations contained in this report should not be considered applicable unless the changes are reviewed and conclusions of this report modified or approved in writing by this office.

6.2 Site Preparation and Grading

Debris and other deleterious material, such as organic soils and/or environmentally impacted earth materials should be removed from the subject property prior to the start of grading. Areas to receive fill should be properly benched in accordance with current industry standards of practice and guidelines specified in the CBC (2013).

Existing utilities should be removed within the proposed building envelope. Abandoned trenches should be properly backfilled and tested. If unanticipated subsurface improvements (utility lines, septic systems, wells, utilities, etc.) are encountered during earthwork construction, the Geotechnical Engineer should be informed and appropriate remedial recommendations would then be provided.

6.3 Remedial Earthwork

The extent of remedial grading required for the proposed property improvements are provided in the following sections. Unless noted otherwise, fill should be moisture conditioned to at least the optimum moisture content and compacted to at least 90 percent of the maximum dry density (based on ASTM D1557).

Mixed-Use and other Settlement Sensitive Structures: The encountered portions of the near surface existing fill and upper portions of the old paralic deposits were observed to be somewhat loose/soft and variable in moisture content and relative density. As such, they are considered potentially compressible and unsuitable for the support of settlement-sensitive structures or additional fill in their current condition. Therefore, where not already removed by the proposed demolition and excavations for site grading, the existing fill and upper weathered portions of the old Paralic deposit materials should be completely removed and recompacted in the areas to receive the proposed building improvement and other settlement-sensitive improvements.

Subterranean Parking Structures: To reduce the risk of the potential differential settlement in areas of the proposed subterranean levels, the exposed Paralic deposits should be removed and recompacted to provide a uniform building pad for the proposed structures. We recommend that these removals extend down to approximately 5 feet below the bottom of mat slabs or conventional foundations.

Furthermore, we recommend that the recompacted materials consist of select Paralic deposits and/or other non-expansive excavated materials to provide a uniform pad with very low to low expansion (expansion index of less than 50 as determined by ASTM D4829). Remedial grading should extend at least 5 feet beyond the perimeter of the building, where feasible.

Shallow Foundations: Remedial grading for non-structural improvements with shallow foundations situated within the upper materials at the property, if planned, should consist of removal and recompactions of the existing undocumented fill. We recommend that these removals extend down approximately 5 feet below the existing ground surface. Remedial grading should extend at least 2 feet beyond the foundation elements, where feasible. It should be understood that this remedial grading will not mitigate the potential seismic-induced settlements provided in **Section 3.3**.

Pavement: Remedial grading for pavement areas should consist of removal and recompaction to a depth of at least 24-inches below pavement subgrade. Remedial grading should extend at least 2 feet beyond the perimeter of the pavement, and should be performed in accordance with the recommendations provided in **Section 9.0**.

Hardscape: Remedial grading for hardscape areas should consist of removal and re-compaction to a depth of at least 24-inches below subgrade elevation.

It should be understood that based on the observations of our field representative, localized deeper removals may be recommended. The base of the removal area should be level to avoid differential fill thicknesses under proposed improvements. This remedial earthwork should extend at least 5 feet outside the proposed building limits and/or 5 feet beyond the area to receive fill. Note that vertical sides exceeding 5 feet in depth may be prone to sloughing and may require laying back to an inclination of 1:1 (horizontal to vertical).

6.4 Fill Material Placement

Fill material should possess a low expansion potential (expansion index of less than 51 as determined by ASTM D4829), be free of organic matter (less than 3 percent organics by weight) and other deleterious material. Much of the onsite materials appear to be suitable for re-use as fill, provided they do not contain rocks greater than 6-inches in maximum dimension, organic debris and other deleterious materials. Rock fragments exceeding 6-inches in one dimension should be segregated and exported from the subject property, or utilized for landscaping.

If import soils are needed, the earthwork contractor should ensure that all proposed fill materials are approved by the geotechnical engineer prior to use. Representative soil samples should be made available for testing at least ten (10) working days prior to hauling to the subject property to allow for laboratory tests.

Fill materials should be placed in 6- to 8-inch loose lifts, moisture conditioned as necessary to at least optimum moisture and compacted to a minimum of 90 percent maximum dry density according to ASTM D1557. The upper 12-inches of pavement subgrade should be moisture conditioned to at least optimum moisture and compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Suitable heavy grading equipment should be utilized to properly mix, spread, moisture condition or dry, and compact each fill lift.

Earthwork may be affected by the existing soil moisture content exceeding optimum. Moist to very moist earth materials may be difficult to mix and compact in their native condition, and drying or mixing with drier soils may be warranted to achieve the recommended relative compaction.

Those areas to receive fill (including over-excavated areas) or surface improvements should be scarified at least 12-inches, moisture conditioned to at least optimum moisture content and recompacted to at least 90 percent of the maximum dry density (based on ASTM D1557).

6.5 Shrinkage and Bulking

Several factors will impact earthwork balancing on the subject property, including shrinkage, bulking, subsidence, trench spoils from utilities and footing excavations, and final pavement section thickness as well as the accuracy of topography.

Shrinkage, bulking and subsidence are primarily dependent upon the degree of compactive effort achieved during construction. For planning purposes, the shrinkage factor is estimated to be on the order of 10 to 15 percent for the existing fill soils to be re-utilized as engineered fill. A shrinkage factor of approximately 5 to 10 percent is estimated for the upper portions of the old paralic deposits that will be re-utilized as engineered fill. These shrinkage factors may vary with methods employed by the contractor. Subsidence is estimated to be on the order of 0.1 feet. Losses from site clearing and removal of existing property improvements may affect earthwork quantity calculation and should be considered.

The previous estimates are intended as an aid for the project engineers in estimating earthwork quantities. It is recommended that the subject property development be planned to include an area that could be raised or lowered to accommodate final site balancing.

6.6 Temporary Site Excavations

Based on available information, it is anticipated that the majority of the property will be excavated to approximate depths of 20 to 40 feet below existing grade to allow for the construction of the subterranean levels. Excavations in the encountered materials should generally be accomplished with heavy-duty earthmoving equipment in good operating condition.

We were unable to directly evaluate the caving potential of the onsite soils at depth during our field evaluation. However, our experience with similar soil materials indicates the caving potential within the encountered old paralic deposit materials is generally moderate to severe.

Temporary excavations within the old paralic deposits (considered to be a Type C soil per OSHA guidelines) should be stable at 1.5H:1V inclinations for short durations during construction, and where cuts do not exceed 20 feet in height. Some sloughing of surface soils should be anticipated. Where space for the temporary layback slopes is not available, shoring will be required, which is most likely the case at this site. Soil parameters for the design of a shoring system are discussed in **Section 6.8**.

Relatively clean sand and silt layers that exhibit low cohesion will be encountered during excavation activities. These layers may ravel and might create potentially unstable slope conditions during installation of shoring. In addition, our experience with similar soil materials indicates that granular layers above the fine grained layers could have a higher potential for raveling due to difficulties in removing any possible perched water layers. This may result in lost ground and voids during installation of wooden lagging.

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with OSHA guidelines. All temporary excavations should be constructed in accordance with OSHA guidelines and local safety codes.

6.7 Geotechnical Parameters for Shoring Design

It is understood that a temporary or permanent shoring system may be used for the proposed excavation where space is not available for properly sloped backcuts. For shoring design, we recommend the use of the distribution of earth pressure shown on **Figure 4**. The shoring contractor should coordinate with the earthmoving contractor regarding sequence and requirements of installing the shoring system. The shoring contractor should also consider the presence of localized perched groundwater in the design and installation procedures of the shoring system.

A temporary and/or permanent shoring system will be necessary for the construction of the proposed subterranean levels and garage structures. We anticipate that the shoring system may consist of closely spaced steel H-Pile soldier piles and wooden lagging. Other suitable systems are also available, including a tied-back soldier pile wall with lagging. Drilled and grouted-in-place tieback anchors will potentially be required with the soldier piles and lagging in order to provide lateral restraint due to the depth of the excavations. In areas adjacent to property boundaries or adjacent structures, tiebacks may not be allowed and other options, such as rackers, may be required. Preliminary design considerations are presented in the following sections for this anticipated shoring method. Please note that the method of temporary support can impact the design earth pressures. As such, EEI should perform a review of the shoring design and provide additional recommendations, as warranted.

Horizontal and vertical movements of the shoring system should be monitored by a licensed surveyor. The construction monitoring and performance of the shoring system are ultimately the contractor's responsibility. At a minimum, we recommend that the tops of the soldier beams should be surveyed prior to excavation and that the top and bottom of the soldier beams be surveyed on a weekly basis until the foundation is completed. The surveyed soldier beam data points should be located at approximately 50 feet on-center. Surveying should consist of measuring movements in vertical and two perpendicular horizontal directions.

6.7.1 Lateral Pressures

The shoring system should be designed to resist the pressure exerted by the retained soils plus any additional lateral forces due to loads applied near the top of the excavations. Shoring walls supporting a level ground surface should be designed for a uniform horizontal pressure of 35H psf or 25H if tie-backs are used, where H is the height of retained earth in feet. Due to the presence of expansive clays encountered in portions of the site, an additional 20H psf should be incorporated into the shoring design for the buildings subjected to the presence of clayey soils. For surcharge loads due to traffic, the shoring should be designed for an additional uniform horizontal pressure of 75 psf for passenger car traffic and 150 psf for heavy truck traffic. For other surcharge loads, the wall should be designed for a uniform horizontal pressure equal to one-third the anticipated surcharge pressure.

These parameters all assume a level ground surface and that temporary shoring will not be subject to hydrostatic pressures. The shoring system should be properly embedded beneath the toe of the excavation to provide adequate structural stability.

6.7.2 Passive Resistance

It is recommended that the design of the shoring system incorporate a passive equivalent fluid weight of 250 pcf for the old paralic deposits materials. The soldier piles should be spaced no closer than 3 diameters on center. The soldier piles should be drilled and backfilled with concrete to the full depth of the passive resistance zone. The area providing the passive resistance can be assumed to have a width equal to twice the concrete pile diameter. The recommended passive pressure for the shoring assumes a horizontal surface for the soil mass extending at least 10 feet in front of the face of the shoring, or three times the height of the surface generating passive pressure, whichever is greater.

The shoring system should be embedded a sufficient depth beneath the toe of the excavation so as to provide structural stability. We recommend that a factor of safety of at least 1.2 be applied to the calculated embedment depth and that the passive pressure be limited to 2,500 psf. The assumed geotechnical conditions should be verified as necessary during shoring construction by a representative of EEI.

6.7.3 Tie-back Anchors (optional)

Due to the required depth of the proposed excavation, the use of tieback anchors may be necessary. Tiebacks may be installed by using hollow-stem auger drilling equipment to prevent caving of the drill hole. The tendon (high strength steel bar or cable) would be inserted into the hollow stem, the anchor drilled its full length, and grout pumped through the stem while retracting the auger. Generally, tieback anchors are installed at a slight, downward angle (typically no more than 20 degrees from the horizontal). In areas adjacent to property boundaries or adjacent structures, tiebacks may not be allowed and other options, such as rackers, may be required.

For the design of the grouted tieback anchors, the bond between the anchor and the soil should be considered effective only beyond an inclined "slip plane" behind the wall and below the upper contact of the native soil (**Figure 5**). All augered anchor boreholes should be completely cleaned of loose materials within the bonded length. If caving of the drilled holes occurs, drilling slurry or casing may be required. Caving of drilled holes below the water table should be anticipated. We recommend that the maximum allowable unit skin frictional resistance for soil anchor design above the static water level would be 250 psf, provided the anchor is installed at least a distance of 10 feet below the ground surface. Due to the presence of expansive clays encountered in portions of the site, tieback anchors should be designed using a maximum allowable unit skin frictional resistance of 150 psf for tieback design in the building areas subjected to the presence of clayey soils. The bonded length between the "slip plane" and the wall should not be assumed to provide anchorage for the wall. For testing purposes, the anchors should be grouted from the bottom of the hole to the "slip plane."

It is recommended that the tieback boreholes be at least 6-inches in diameter and be drilled at an angle between 15 and 20 degrees from horizontal such that the frictional resistance is provided by the competent native materials. It should be noted that as the anchor inclination increases from the horizontal, the lateral efficiency decreases, while the vertical load component increases. The anchor downward load components should be considered in the structural design.

6.7.4 Load Testing and Lock-off of Tie-Back Anchors

Load testing of all anchors is recommended in order to verify that the anchor systems can carry their design capacity. Prior to their being accepted, each anchor should be tested to a suitable overload, as established by the structural engineer. As a minimum we recommend that at least 10 percent of the anchors should be tested to 200 percent of the design load for 60 minutes, while the remaining anchors should be tested to 140 percent of the design load. At least 10 percent of the anchors tested to 140 percent of the design load should be held for 60 minutes. All failing anchors should be replaced or their design capacity reduced. Anchor lock-offs should be set at the design loads following the completion of load testing. Based upon the results of the load-testing program, modifications to the anchor design or additional load testing may be warranted. Other suitable load testing procedures, such as those recommended by the Post-Tensioning Institute, may also be appropriate for the subject property, depending upon the type and spacing of the proposed anchors. We recommend that we review alternative load testing methods, if proposed, for the planned construction.

EEI should observe and document the installation of the anchors in order to check the anticipated Geotechnical conditions. It is recommended that EEI observe the testing and lock-off of the anchors.

6.7.5 Lagging

Timber lagging may be used between the soldier piles to help support the exposed soils. If lagging is to remain after construction, treated lumber should be used. Lagging should be designed for the full lateral pressure recommended in **Section 6.7.1**. If possible, structural walls should be cast directly against the shoring, thus eliminating the need for placing backfill within a narrow space. Voids between the soil and lagging should be grouted or slurried to reduce the potential for the voids to propagate to the surface. Special provisions for wall drainage (such as the use of prefabricated composite drain) may be necessary above any groundwater table where this type of construction is used.

7.0 PRELIMINARY FOUNDATION RECOMMENDATIONS

7.1 General

In the event that plans concerning the proposed building structures are revised in the project design and/or location or loading conditions of the planned structures are made, conclusions and recommendations contained in this report should not be considered valid unless they are reviewed, revised and/or approved in writing by EEI. The foundation recommendations provided herein are based on the soil materials near foundation level possessing a low expansion potential (Expansion Index < 51). Recommendations by the project's design-structural engineer or architect may exceed the following minimum recommendations. Final foundation and slab design should be provided based on the expansion potential of the near surface finish grade soils encountered during grading.

While remedial grading recommendations (as described in **Section 6.3** of this report) can be expected to mitigate the settlement of the loaded structures proposed for the subject property (such as the residential buildings proposed for the property), our geotechnical analyses and experience indicate that the more heavily loaded parking structures (which can often impose column loads on the order of 600 kips to 800 kips) could experience considerable settlements if they are supported on conventional shallow foundations bearing upon the onsite materials.

A number of different options are available to provide adequate support for the proposed parking structure, while mitigating the potential for settlement of the upper soils. We are providing herein recommendations for a conventional shallow foundation system to support the proposed residential buildings and a reinforced mat foundation system for the support of the proposed parking structure.

7.2 Foundation Design

Mixed-Use and other Settlement Sensitive Structures: The proposed mixed use building foundations constructed at grade can be supported on conventional continuous or isolated spread footings or mat slab foundations bearing entirely upon properly compacted fill materials, as detailed in **Section 6.3**. Foundations supporting the proposed building structures should be constructed with an embedment of at least 24-inches below adjacent grade. At these depths, footings may be designed for an allowable soil bearing value of 2,000 psf. This value may be increased by one-third for loads of short duration, such as wind and seismic forces. Continuous and isolated spread footings supporting the proposed structure should have a minimum width of 18- and 24-inches, respectively.

The proposed building foundations constructed at subterranean levels can be supported on conventional continuous or isolated spread footings bearing entirely upon firm natural materials comprising the old paralic deposit materials or properly compacted fill. A net allowable bearing capacity of 4,000 pounds per square foot (psf) can be used for foundations founded at a depth of at least 10 feet below the existing ground surface. A minimum base width of 18-inches for continuous footings and 24-inches for isolated pad foundations should be used. Footings should be embedded a minimum of 18-inches below adjacent finish grade. The bearing capacity value can be increased by 500 psf for each additional foot of depth to a maximum of 7,000 psf. additionally; this value may be increased by one-third for loads of short duration, such as wind and seismic forces.

Based on geotechnical considerations, footings should be provided with reinforcement consisting of at least four No. 4 rebars, two top and two bottom; however, the actual foundation reinforcement should be in accordance with the structural engineer's requirements.

Alternatively, a rigid mat foundation may be used for the support of the proposed buildings, provided the mat foundation is bearing within fill soils that are properly placed and compacted in accordance with the recommendations contained herein. When properly designed and constructed, a structural mat foundation system can be expected to support high structural loads and provide relatively uniform settlement across a structure, while being able to "bridge" over local areas of dynamic settlement. Mat foundations should be properly reinforced to form a relatively rigid structural unit in accordance with the structural engineers design. For designing a mat foundation, we recommend using an uncorrected modulus of subgrade reaction of 250 pounds per cubic inch (pci). For large foundations, the modulus is typically reduced by 75 percent. Mat foundations should be reinforced in accordance with structural considerations.

Parking Structures: As noted herein, a rigid mat foundation can be used for the support of the proposed parking structure at the subject property, provided the mat foundation is bearing upon at least 5 feet of fill soils that are properly placed and compacted in accordance with the recommendations contained herein. When properly designed and constructed, a structural mat foundation system can be expected to support high structural loads and provide relatively uniform settlement across a structure. Mat foundations should be properly reinforced to form a relatively rigid structural unit in accordance with the structural engineer's design. For designing a mat foundation, we recommend using an uncorrected modulus of subgrade reaction of 250 pounds per cubic inch (pci). For large foundations, the modulus is typically reduced by 75 percent. The mat foundation may also be designed for a maximum bearing pressure of 1,500 psf with a one third increase for transient loadings. Mat foundations should be reinforced in accordance with structural considerations.

7.3 Lateral Resistance of Foundations

Horizontal loads acting on foundations and stem walls cast in open excavations against undisturbed native soil or against properly placed and compacted fill will be resisted by friction acting along the base of the footing and by passive earth pressures against the side of the footing and stem wall. The frictional resistance acting along the base of footings founded on suitable foundation soils may be computed using a coefficient of friction equal to 0.30 with the normal dead load. Passive earth pressures acting against the side of footings and stem walls may be assumed to be equivalent to a fluid weighing 250 pounds per cubic foot. Passive pressure in the upper 1-foot should be neglected unless confined by concrete slabs-on-grade or asphaltic pavement. The values given above may be increased by one-third for transient wind or seismic loads.

7.4 Footing Setbacks

All footings should maintain a minimum 7-foot horizontal setback from the base of the footing to any descending slope (if existing onsite). This distance is measured from the outside footing face at the bearing elevation. Footings should maintain a minimum horizontal setback of H/3 (H=slope height) from the base of the footing to the descending slope face and no less than 7 feet, or greater than 40 feet.

Footings adjacent to unlined drainage swales or underground utilities (if any) should be deepened to a minimum of 6-inches below the invert of the adjacent unlined swale or utilities. This distance is measured from the footing face at the bearing elevation. Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 projection from the heel of the wall. Alternatively, walls may be designed to accommodate structural loads from buildings or appurtenances.

7.5 Concrete Slabs on Grade

Interior slabs can be grade supported by native soil or structural fill whose placement/compaction is documented by the project soils engineer/engineering geologist as recommended herein. The thickness of the slab should be in accordance with the structural engineer's design; however, based on geotechnical considerations, we recommend that concrete slabs be a minimum of 5-inches in thickness. The upper 4 feet of soil below interior concrete slabs-on-grade should have an expansion index of 50 or less. Subgrade materials should not be allowed to desiccate between the completion of grading and the construction of the concrete slabs. The floor slab subgrade should be thoroughly and uniformly moistened prior to placing concrete.

Mat foundations may be sufficiently thick such that a moisture vapor retarder/barrier may not be required. However, if the project team determines that a moisture vapor retarder/barrier is required, the following recommendations may be used as a guideline.

A moisture vapor retarder/barrier may be placed beneath slabs where moisture sensitive floor coverings will be installed. Typically, plastic is used as a vapor retardant. If plastic is used, a minimum 10-mil is recommended. The plastic should comply with ASTM E1745. Plastic installation should comply with ASTM E1643.

Current construction practice typically includes placement of a 2-inch thick sand cushion between the bottom of the concrete slab and the moisture vapor retarder/barrier. This cushion can provide some protection to the vapor retarder/barrier during construction, and may assist in reducing the potential for edge curling in the slab during curing. However, the sand layer also provides a source of moisture vapor to the underside of the slab that can increase the time required to reduce moisture vapor emissions to limits acceptable for the type of floor covering placed on top of the slab. The slab can be placed directly on the vapor retarder/barrier. The floor covering manufacturer should be contacted to determine the volume of moisture vapor allowable and any treatment needed to reduce moisture vapor emissions to acceptable limits for the particular type of floor covering installed.

The floor covering manufacturer should be contacted to determine the volume of moisture vapor allowable and any treatment needed to reduce moisture vapor emissions to acceptable limits for the particular type of floor covering installed. The project team should determine the appropriate treatment for the specific application.

In preparation for slab or flatwork construction, the earthwork contractor should ensure that the onsite soils have been prepared as recommended and that field density tests have been performed to adequately document the relative compaction of the structural fill. Preparation of the native soils should be documented prior to placement of aggregate, structural components and/or fill.

Some minor cracking of slabs can be expected due to shrinkage. The potential for this slab cracking can be reduced by careful control of water/cement ratios in the concrete. The contractor should take appropriate curing precautions during the pouring of concrete in hot or windy weather to reduce the potential for cracking of slabs. We recommend that a slipsheet (or equivalent) be utilized if grouted fill, tile, or other crack-sensitive floor covering is planned directly on concrete slabs. All slabs should be designed in accordance with structural considerations.

We recommend that all placement and curing be performed in accordance with procedures outlined by the American Concrete Institute and/or Portland Cement Association.

Special consideration should be given to concrete placed and cured during hot or cold weather conditions. Proper control joints should be provided to reduce the potential for damage resulting from shrinkage.

Laboratory test results indicate that the upper materials contains a maximum soluble sulfate concentration of 0.007 percent, which indicate a negligible sulfate corrosion potential of concrete that will be in contact with the onsite soils. Our analysis also indicates maximum chloride concentrations of 0.010, which indicates a negligible corrosion potential to concrete due to chloride in the soils. As such, Type II cement can be used in concrete elements that will be in contact with the upper materials.

7.6 Permanent Subterranean Walls

We anticipate that where temporary shoring is installed, the permanent restrained retaining walls for the subterranean levels will predominantly be placed directly against the temporary shoring. After permanent bracing (such as floor slabs) has been installed, the tieback anchors utilized for the shoring (if any) should be detentioned and documented by the Geotechnical Engineer, in accordance with the requirements of the County of Orange.

Permanent subterranean walls should be designed to resist the pressure exerted by retained soils plus any additional lateral forces due to loads placed adjacent to or near the wall. Retaining walls that are free-draining, are situated above groundwater and are to be restrained from movement at the top, should be designed for an equivalent fluid weight of 55 pcf (for a level surface of retained earth) or 70 pcf (for a surface of retained earth that is sloping at a 2H:1V inclination) plus a uniform lateral pressure of 10H psf, where H equals height of retained earth in feet.

Due to the presence of expansive clays encountered in portions of the site, an additional equivalent fluid weight of 10 psf should be incorporated into subterranean wall design for the buildings subjected to the presence clayey soils. If traffic loads are planned adjacent to the walls, the walls should be designed for an additional uniform horizontal pressure of 75 and 150 psf for passenger car and truck traffic, respectively. For other surcharge loads, we recommend the walls be designed to resist a uniform horizontal pressure equal to 30 percent of the uniform surcharge load.

Utilizing the estimated Peak Ground Acceleration seismic parameter PGA_M (where PGA_M is the design peak ground acceleration adjusted for site class effects presented in **Section 3.1** of this report), we estimate the seismic resultant for lateral pressure for a wall with level backfill to be $24H^2$ lbs, or $28H^2$ for sloping backfill, where H is the retained height in (feet).

The seismic resultant is expected to be exerted in addition to the lateral earth pressures presented above. The seismic resultant may be assumed to be applied at a height of 0.6H above the wall base. The magnitude and location of the seismic resultant are based on the assumption that the walls are constructed in accordance with the recommendations contained herein.

The permanent subterranean wall should be provided with an adequate backdrain system to reduce the potential for build-up of hydrostatic pressures. Backdrains should consist of a 4-inch diameter perforated PVC pipe (Schedule 40) surrounded by ½-inch to ¾-inch clean crushed rock and wrapped in Mirafi 140N filter fabric (or approved equivalent). Free-draining backwall material such as a continuous, clean gravel layer (also wrapped in filter fabric) or geocomposite (Miradrain 6000 or approved equivalent) should be placed along the height of the wall to 18-inches below finish grade and tied into the backdrain system. The drain system should be connected to a suitable outlet. Additionally, subterranean walls should be waterproofed or damp-proofed, depending on the degree of moisture protection desired.

For those portions of the wall not placed against shoring, the above values assume granular backfill and free-draining conditions to prevent buildup of hydrostatic pressure in the backfill. Backfill materials should meet the recommendations described in the following section of this report. Import fill materials should be approved by the soils engineer prior to placement. Wall backfill should be compacted by mechanical methods to at least 90 percent of the maximum dry density as determined by ASTM D 1557.

7.7 Other Site Retaining Walls

The design parameters provided herein assume that non-expansive select material (such as gravel wrapped in filter fabric) is used to backfill any retaining walls. If expansive soils are used to backfill the proposed walls, increased active and at-rest earth pressures will need to be utilized for retaining wall design, and can be provided upon request. Building walls below grade should be waterproofed or damp-proofed, depending on the degree of moisture protection desired. The foundation system for retaining walls should be designed in accordance with the recommendations presented in the preceding sections of this report, as appropriate. Footings should be embedded a minimum of 18-inches below adjacent finish grade. There should be no increase in bearing for footing width. Recommendations pertaining to “landscape” walls (i.e., Crib, Loffel, Earthstone, Geogrid, etc.) may vary from those provided herein, and should be provided upon request.

The design active earth pressure on a retaining wall may be considered equivalent to that produced by a fluid weighing 35 pounds per cubic foot (pcf). This design equivalent fluid pressure of 35 pcf is considered appropriate for cantilevered walls retaining non-expansive soils with a level ground surface, subject to lateral deflection at distances above grade due to lateral earth pressures.

A safety factor for sliding and overturning of 1.5 is typically prescribed for a cantilevered structure as described. All retaining structures should be fully free draining. Restrained walls (such as re-entrant corners), with a level backfill, should be designed for an equivalent fluid pressure of 55 pcf for at rest lateral earth pressure.

For resistance to lateral loads, an allowable coefficient of friction of 0.30 between the base of the foundation elements and underlying material is recommended. In addition, an allowable passive resistance equal to an equivalent fluid weighing 250 pcf acting against the foundation may be used to resist lateral forces. Passive pressure in the upper 1-foot should be neglected unless confined by concrete slabs-on-grade or asphaltic pavement. These values may be increased by one-third for transient wind or seismic loads.

Adequate subdrainage should be provided behind all retaining walls. The subdrainage system should consist of a minimum of a 4-inch diameter perforated PVC pipe (schedule 40 or approved equivalent) placed at the base of the retaining wall and surrounded by $^{3}/_{4}$-inch clean crushed rock wrapped in a Mirafi 140N filter fabric (or approved equivalent). The crushed rock wrapped in fabric should be at least 12-inches wide and extend from the base of the wall to within 2 feet of the ground surface. The upper 2 feet of backfill should consist of compacted native soil. The retaining wall drainage system should be sloped to an outlet into the storm drain system or other appropriate facility.

8.0 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable yielding materials encountered during grading should be removed. Once compacted fill and/or native soils are brought to the proposed pavement subgrade elevations, the subgrade should be proof-rolled in order to check for a uniform firm and unyielding surface. Representatives of the project geotechnical engineer should observe all grading and fill placement.

The upper 12-inches of pavement subgrade soils should be scarified; moisture conditioned to at least optimum moisture content and compacted to at least 95 percent of the laboratory standard (ASTM D1557). If loose or yielding materials are encountered during subgrade preparation, evaluation should be performed by EEI.

Aggregate base materials should be properly prepared (i.e., processed and moisture conditioned) and compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557. Aggregate base materials should conform to Caltrans specifications for Class 2 aggregate base.

All pavement section changes should be properly transitioned. Although not anticipated, if adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A representative of the project geotechnical engineer should be present for the preparation of subgrade and aggregate base.

For design purposes we have assumed a Traffic Index (TI) of 5.5 for the drive areas and 4.5 for the parking stalls at the subject property. This assumed TI should be verified as necessary by the Civil Engineer or Traffic Engineer. For preliminary design purposes, we have conservatively assumed a preliminary R-Value of 25 for the materials likely to be exposed at subgrade. The modulus of subgrade reaction (K-Value) was estimated at 85 pounds per square inch per inch (psi/in) for an R-Value of 25 (Caltrans, 1974). Pavement design was calculated for the parking lot structural section requirements for asphaltic concrete in accordance with the guidelines presented in the Caltrans Highway Design Manual. Rigid pavement sections were evaluated in general accordance with ACI 330R-08, based on an average daily truck traffic value of 10.

TABLE 4
Preliminary Pavement Design Recommendations

Traffic Index (TI)	Pavement Surface	Aggregate Base Material (1)
4.5 – Parking Stalls	3.0-inches Asphalt Concrete	5.0-inches
5.5 – Drive Areas	4.0-inches Asphalt Concrete	6.0-inches
Entrance/Exit Lane Areas	6.0-inches Portland Cement Concrete (2)	4.0-inches (optional)
(1) Reinforcement and control joints placed in accordance with the structural engineer's requirements		

The recommended rigid pavement section provided above is intended as a minimum guideline. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the assumed traffic index used for design, increased maintenance and repair could be required for the pavement section. Final pavement design should be verified by testing of soils exposed at subgrade after grading has been completed. Thicker pavement sections could result if R-Value testing indicates lower values.

9.0 DEVELOPMENT RECOMMENDATIONS

9.1 Landscape Maintenance and Planting

Water is known to decrease the physical strength of earth materials, significantly reducing stability by high moisture conditions. Surface drainage away from foundations and graded slopes should be maintained. Only the volume and frequency of irrigation necessary to sustain plant life should be applied.

Consideration should be given to selecting lightweight, deep-rooted types of landscape vegetation which require low irrigation that are capable of surviving the local climate. From a soils engineering viewpoint, "leaching" of the onsite soils is not recommended for establishing landscaping.

If landscape soils are processed for the addition of amendments, the processed soils should be re-compacted to at least 90 percent relative compaction (based on ASTM D1557).

9.2 Site Drainage

Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled over slopes or the subject property. Runoff should be channeled away from slopes and structures and should not be allowed to pond and/or seep uncontrolled into the ground. Pad drainage should be directed toward an acceptable outlet. Although not required, roof gutters and down spouts may be considered to control roof drainage, discharging a minimum of 10 feet from proposed structures, or into a subsurface drainage system. Consideration should be given to eliminating open-bottom planters directly adjacent to proposed structures for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized, with a properly designed drain outlet placed in the bottom of the planter.

9.3 Site Runoff Considerations - Stormwater Disposal Systems

It is EEI understanding that the Client is considering that runoff generated from the facility be disposed of in engineered subsurface features onsite.

9.3.1 Percolation Testing

Following the drilling of exploratory borings B-6, B-7, B-9 and B-10, a 3-inch diameter perforated polyvinyl chloride (PVC) pipe was placed in the cleaned- out holes and gravel was placed around the PVC pipe. The presoaking and percolation testing was performed in general accordance with Orange County Public Works Technical Guidance Document (OC Public Works TGD, 2011), Appendix VII, Infiltration Rate Evaluation Protocol and Factor of Safety Recommendations. Percolation testing was performed until consistent results were obtained. The results were used to calculate the pre-adjusted percolation rate for the test hole. Upon conclusion of testing, the PVC pipe was removed from the test hole and the test hole was backfilled.

During the presoaking process, it was observed that less than 30 minutes was required for a minimum 12-inch high column of water to seep away. Consequently, the borings were allowed to presoak and the test in the boring was run at approximate 10 minute intervals for a period of approximately two hours, when the highest and lowest readings from three consecutive readings were noted to be within 10 percent of each other. The reading obtained from the final 10 minute interval was then used to calculate the pre-adjusted percolation rate for each test hole. Upon conclusion of testing, the perforated pipe was removed from the test holes and the test excavations were backfilled.

We note that a soil profile's percolation rate is not the same as its infiltration rate. Therefore, the measured/calculated percolation rate was converted to an estimated infiltration rate. Therefore, the measured/calculated field percolation rate was converted to an estimated infiltration rate utilizing a reduction factor known as the Porchet method. **Table 4** presents the measured percolation rate and corresponding infiltration rate calculated for the test hole.

TABLE 5
Summary of Percolation Testing

Location	Depth (ft)	Pre-Adjusted Percolation Rate (in/hr)	Infiltration Rate (in/hr)
B-6	~13	288.0	55.38
B-7	~12	151.2	27.10
B-9	~10	133.9	15.32
B-10	~20	98.64	13.03

9.3.2 Summary of Findings

Based on the results of our field percolation testing, it appears that the percolation/infiltration rates presented herein are conducive to direct infiltration of surface stormwater for the preliminary design of subsurface storm water retention/disposal devices at the specific locations and approximate depths at the subject property as listed in **Table 4**.

9.3.3 Structural Setback from Retention Devices

It is recommended that retention/disposal devices be situated at least three times their depth, or a minimum of 15 feet (whichever is greater), from the outside bottom edge of structural foundations. Structural foundations include (but are not limited to) buildings, loading docks, retaining walls, and screen walls.

All stormwater disposal systems, including pervious pavement areas should be checked and maintained on regular intervals. Stormwater devices including bioswales that are located closer than 10 feet from any foundations/footings should be lined with an impermeable membrane to reduce the potential for saturation of foundation soils (also refer to **Section 7.6**).

9.4 Additional Site Improvements

Recommendations for additional grading, exterior concrete flatwork design and construction can be provided upon request. If in the future, additional property improvements were planned for the site, recommendations concerning the design and construction of improvements would be provided upon request.

9.5 Trenching

All temporary excavations for grading purposes and installation of underground utilities should be constructed in accordance with OSHA guidelines and local safety codes. Temporary excavations over 4 feet in height should be evaluated by the project engineer, and could require shoring, sloping, or a combination thereof. Temporary excavations within the onsite materials should be stable at 1.5:1 inclinations for cuts less than 20 feet in height.

Footing trench excavations for structures and walls should be observed and approved by a representative of the project soils engineer prior to placing reinforcement. Footing trench spoil and excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent (based on ASTM D1557) if not removed from the subject property. All excavations should conform to OSHA and local safety codes.

9.6 Utility Backfill

Fill around the pipe should be placed in accordance with details shown on the drawings, and should be placed in layers not to exceed 8-inches loose (unless otherwise approved by the Geotechnical Engineer) and compacted to at least 90 percent of the maximum dry density as determined in accordance with ASTM D1557 (Modified Proctor).

The Geotechnical Engineer should approve all backfill material. Select material should be used when called for on the drawings, or when recommended by the Geotechnical Engineer. Care should be taken during backfill and compaction operations to maintain alignment and prevent damage to the joints. The backfill should be kept free from stones, chunks of highly plastic clay, or other objectionable material. Backfill soils should be non-expansive, non-corrosive, and compatible with native earth materials. Backfill materials and testing should be in accordance with the CBC 2013 and City specifications.

All pipe backfill areas should be graded and maintained in such a condition that erosion or saturation will not damage the pipe bed or backfill. Flooding trench backfill is not recommended. Heavy equipment should not be operated over any pipe until it has been properly backfilled with a minimum 2 to 3 feet of cover. The utility trench should be systematically backfilled to allow maximum time for natural settlement. Backfill should not occur over porous, wet, or spongy subgrade surfaces. Should these conditions exist, the areas should be removed, replaced and recompacted.

10.0 PLAN REVIEW

Once the detailed and approved site and grading plans are available, they should be submitted to this office for review and comment, to reduce the potential for discrepancies between plans and recommendations presented herein. If conditions were found to differ substantially from those stated, appropriate recommendations would be provided. Additional field studies may be warranted once the final conceptual plans are produced.

11.0 LIMITATIONS

This Geotechnical Evaluation has been conducted in accordance with the generally accepted geotechnical engineering principles and practices. Findings provided herein have been derived in accordance with the current standards of practice, and no warranty is expressed or implied. Standards of practice are subject to change with time. This report has been prepared for the sole use of the Client, within a reasonable time from its authorization. Site conditions, land use (both onsite and offsite), or other factors may change as a result of manmade influences, and additional work may be required with the passage of time.

This Preliminary Geotechnical Evaluation should not be relied upon by other parties without the express written consent of EEI and the Client; therefore, any use or reliance upon this geotechnical evaluation by a party other than the Client should be solely at the risk of such third party and without legal recourse against EEI, its employees, officers, or directors, regardless of whether the action in which recovery of damages is brought or based upon contract, tort, statue, or otherwise.

The Client has the responsibility to see that all parties to the project, including the designer, contractor, subcontractor, and building official, etc. are aware of this report in its complete form. This report contains information that may be used in the preparation of contract specifications; however, the report is not designed as a specification document, and may not contain sufficient information for use without additional assessment. EEI assumes no responsibility or liability for work or testing performed by others. In addition, this report may be subject to review by the controlling authorities.

12.0 REFERENCES

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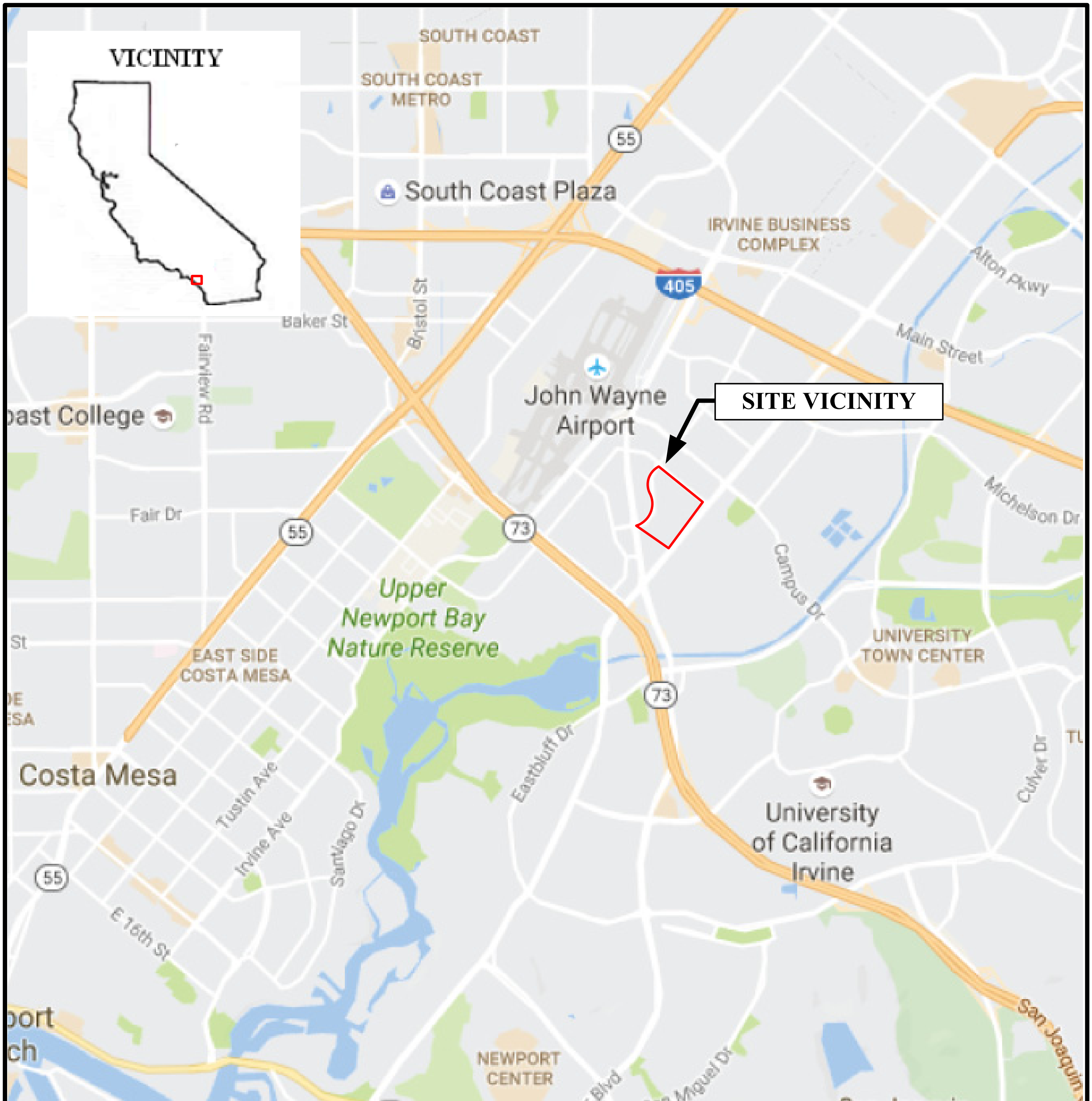
FIGURES



LEGEND

N

Scale: 1" = 1 mile

0 0.5 mi 1 mi 2 mi

Note: All Locations Are Approximate

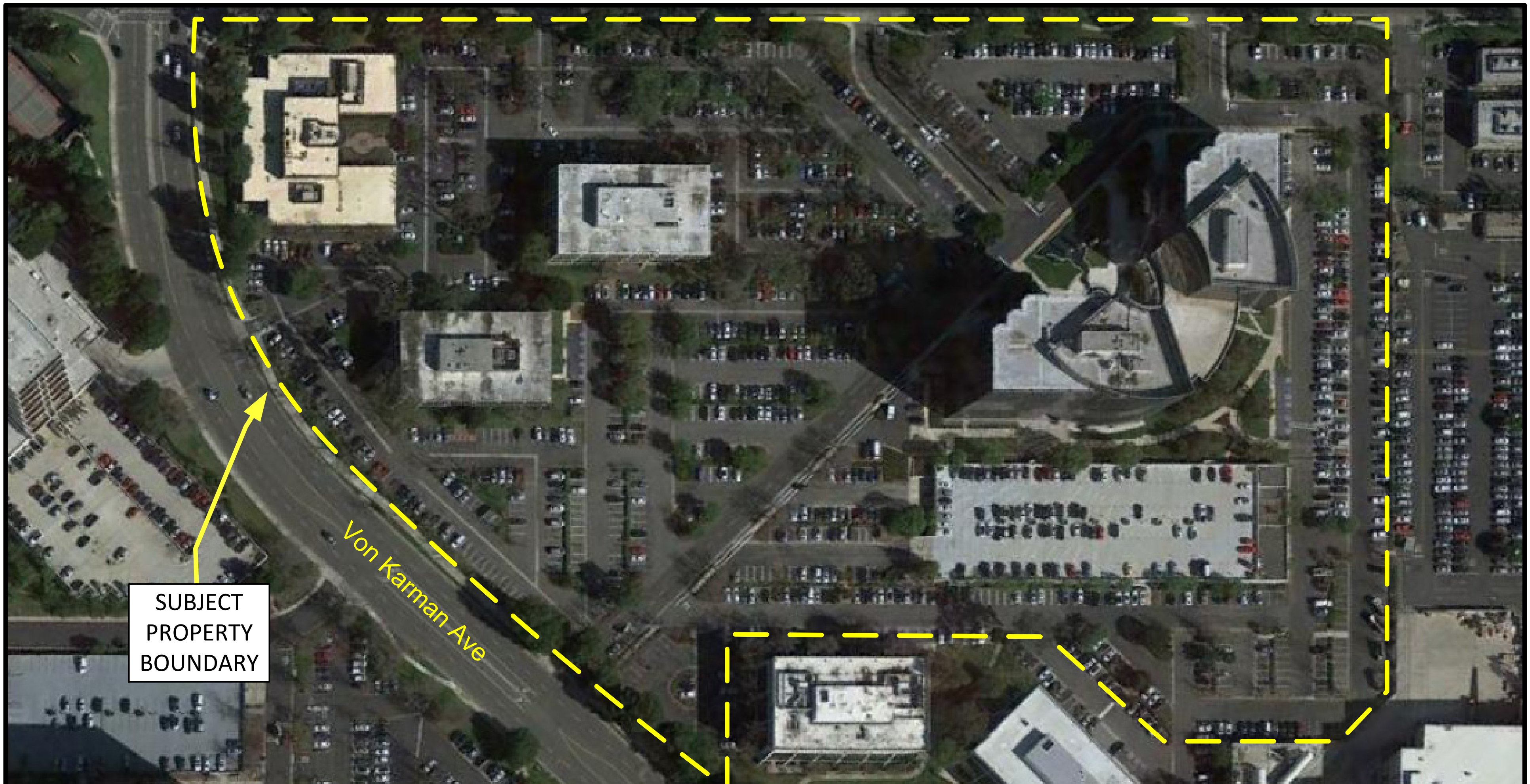
SITE VICINITY MAP

Shopoff Land Fund II, LP
The Koll Center Residences
Newport Beach, California
EEI Project No. SHO – 72189.4a
Created October 2016

EEI Geotechnical & Environmental Solutions

FIGURE 1

Source: Google Maps, 2016



LEGEND

N

Scale: 1" = 160'

0 FT 160FT 320FT

AERIAL SITE MAP

Shopoff Land Fund II, LP

The Koll Center Residences

Newport Beach, California

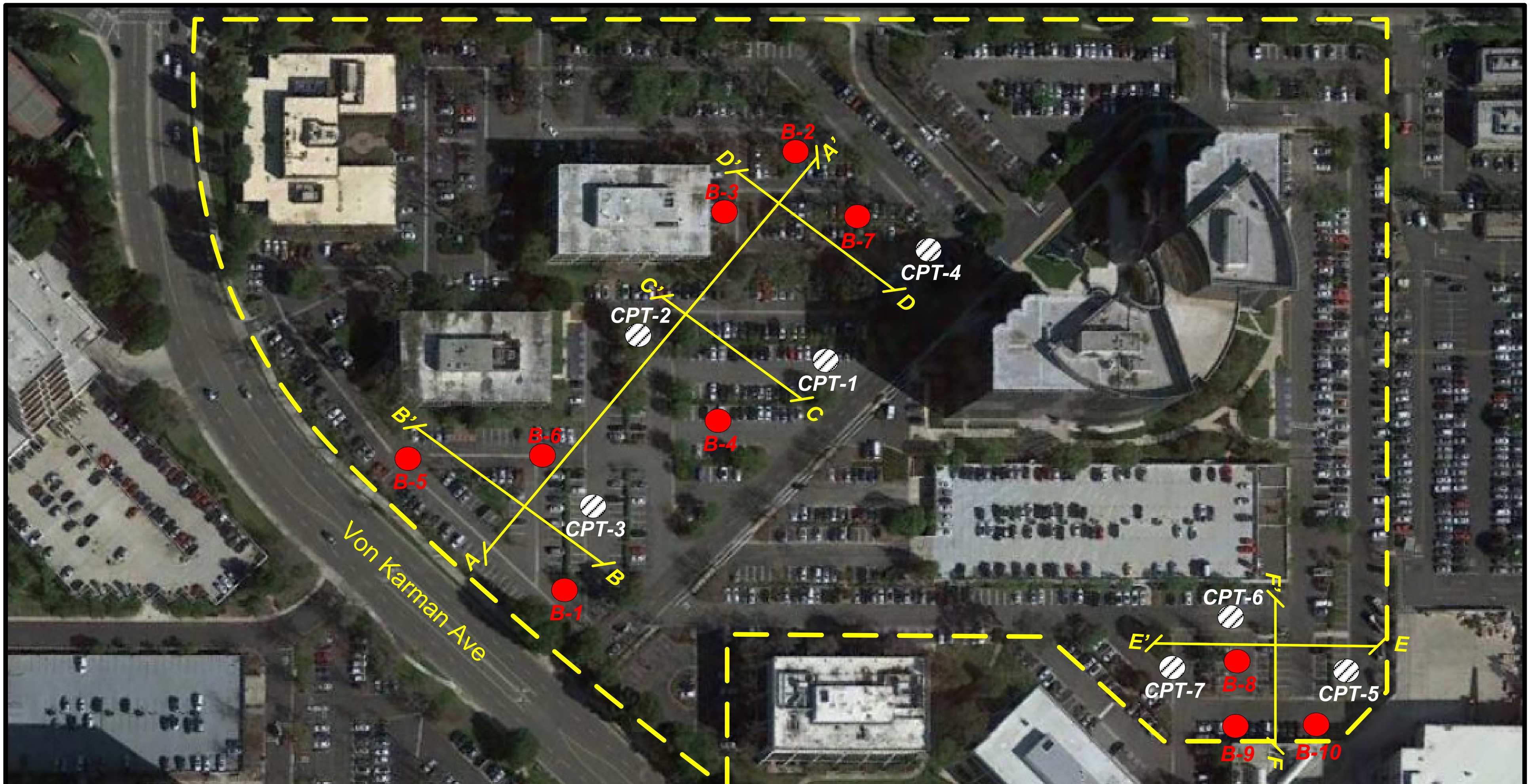
EEI Project No. SHO – 72189.4a

Created October 2016

EEI
Geotechnical & Environmental Solutions

FIGURE 2

Source: Google Earth, 2016; Image Date: October 27, 2016



LEGEND	
B-10	Approximate Boring Locations
CPT-7	Approximate CPT Locations
F — F'	Approximate Cross-Section Locations

N

Scale: 1" = 160'

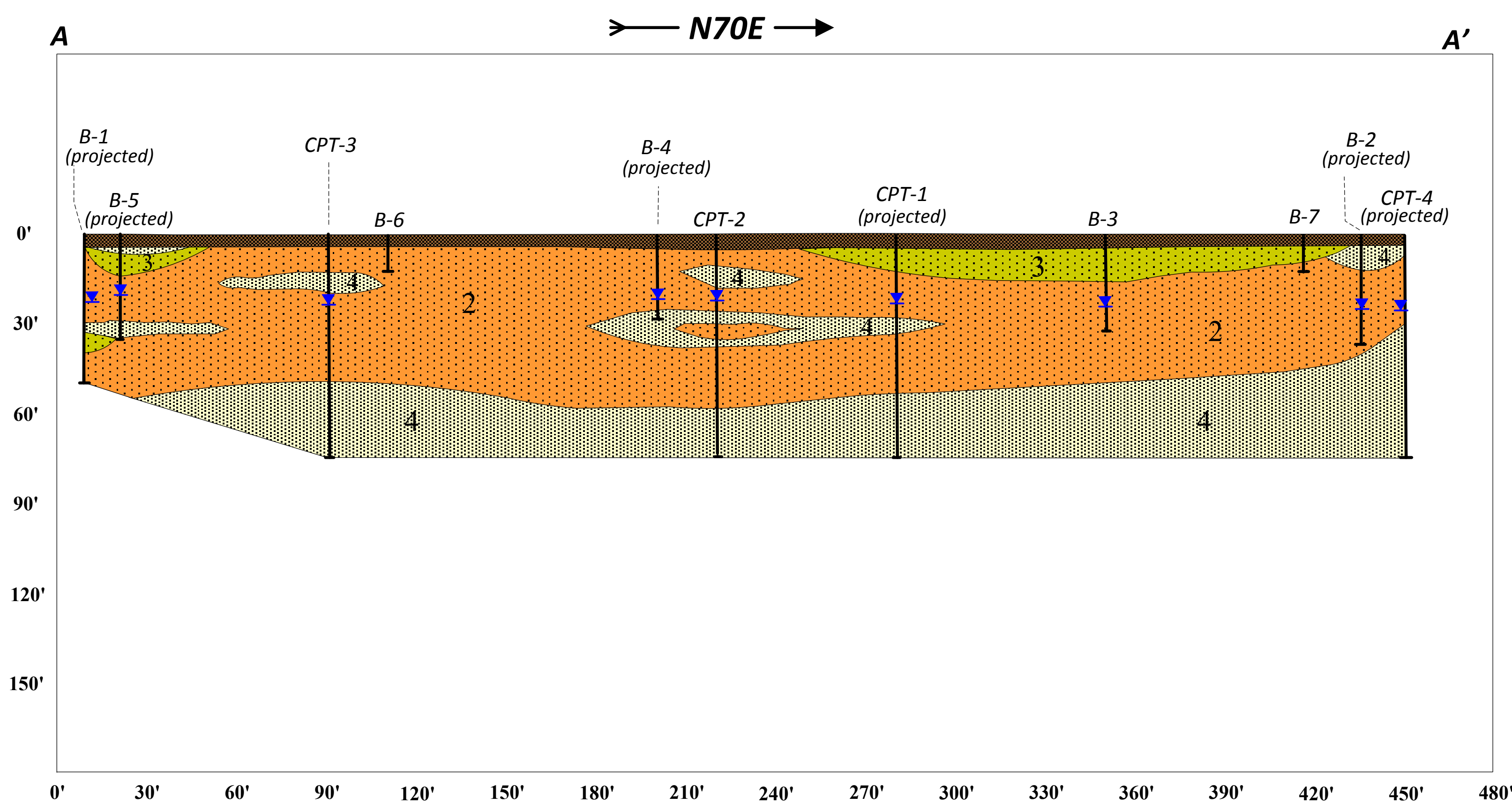
0 FT 160FT 320FT

GEOTECHNICAL MAP
Shopoff Land Fund II, LP
The Koll Center Residences
Newport Beach, California
EEI Project No. SHO – 72189.4a
Created October 2016

EEI Geotechnical & Environmental Solutions

FIGURE 3

Source: Google Earth, 2016; Image Date: October 27, 2016



LEGEND (Detailed description in Section 6.0 of this report)

1	Fill: Sands, Silts Clays and Gravels
2	Older Alluvial Fan Deposits: SAND, with trace Gravel, Silt or Clay
3	Older Alluvial Fan Deposits: SILTY-SAND and SANDY-CLAY
4	Older Alluvial Fan Deposits: CLAYS and SILTS
▼	Groundwater/seepage encountered

SCALE
1" = 60ft

GEOLOGIC CROSS-SECTION A-A'
Shopoff Land Fund II, LP
The Koll Center Residences
Newport Beach, California
EEI Project No. SHO – 72189.4a
Created October 2016

EEI
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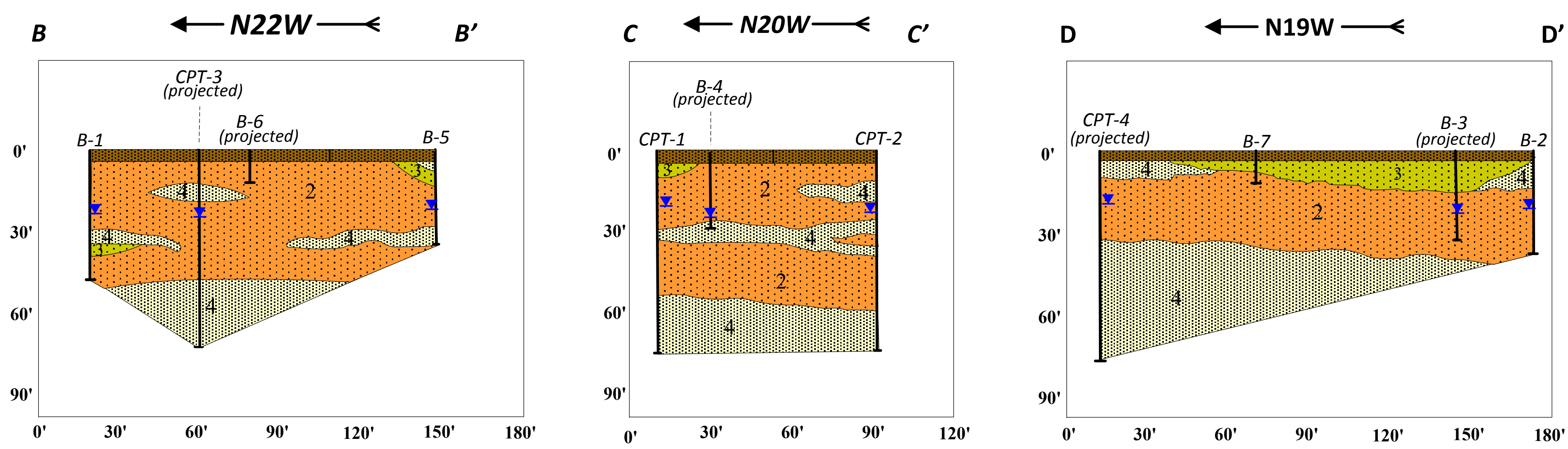
FIGURE 4a



LEGEND (Detailed description in Section 6.0 of this report)

1 Fill: Sands, Silts Clays and Gravels

2 Older Alluvial Fan Deposits: SAND, with trace Gravel, Silt or Clay

3 Older Alluvial Fan Deposits: SILTY-SAND and SANDY-CLAY

4 Older Alluvial Fan Deposits: CLAYS and SILTS

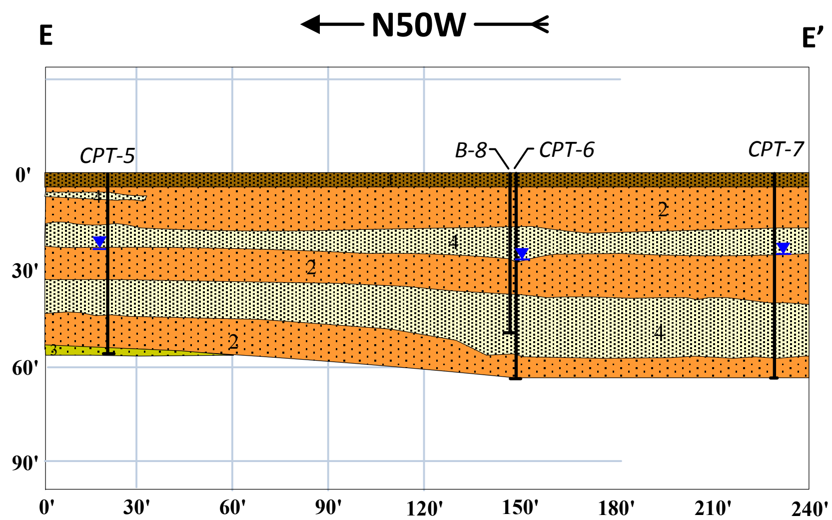
▼ Groundwater/seepage encountered

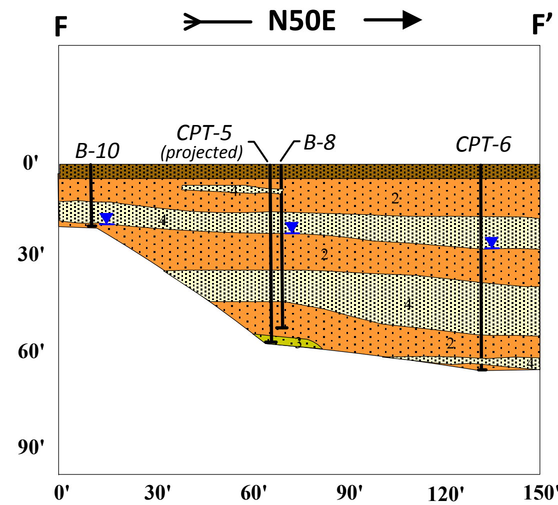
SCALE
1" = 60ft

GEOLOGIC CROSS-SECTIONS B-B’.C-C’.D-D’
Shopoff Land Fund II, LP
The Koll Center Residences
Newport Beach, California
EEI Project No. SHO – 72189.4a
Created October 2016

EEI Geotechnical & Environmental Solutions

FIGURE 4b





LEGEND (Detailed description in Section 6.0 of this report)

- 1 Fill: Sands, Silts Clays and Gravels
- 2 Older Alluvial Fan Deposits: SAND, with trace Gravel, Silt or Clay
- 3 Older Alluvial Fan Deposits: SILTY-SAND and SANDY-CLAY
- 4 Older Alluvial Fan Deposits: CLAYS and SILTS
- ▼ Groundwater/seepage encountered

SCALE
1" = 60ft

GEOLOGIC CROSS-SECTIONS E-E',F-F'
Shopoff Land Fund II, LP
The Koll Center Residences
Newport Beach, California
EEI Project No. SHO – 72189.4a
Created October 2016

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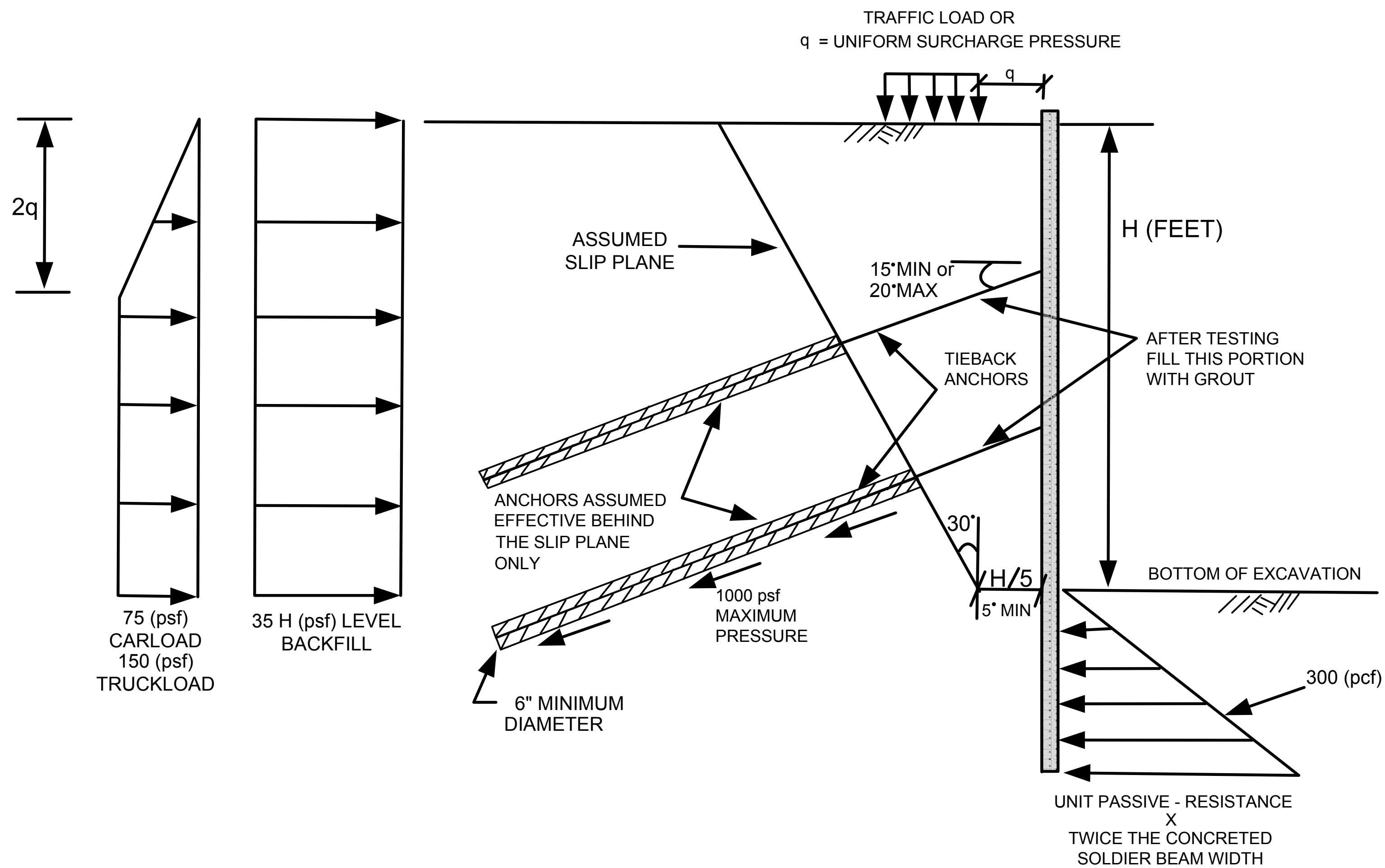
FIGURE 4c



RECOMMENDED LATERAL EARTH PRESSURES
FOR TEMPORARY SHORING

Shopoff Land Fund II, LP
The Koll Center Residences
Newport Beach, California
EEI Project No. SHO – 72189.4a
Created October 2016

NOT TO SCALE

EEI
Geotechnical & Environmental Solutions

FIGURE 5

APPENDIX A

SOIL CLASSIFICATION CHART AND BORING LOGS AND CPT SOUNDINGS

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/15/15 **COMPLETED** 7/15/15
GROUND ELEVATION
BORING DIAMETER 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12
CAL CORRECTION 0.61
LOGGED BY ML **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0	**ASPHALT (5") / BASE (6"),**										
1	**FILL** SAND, reddish brown, damp, medium dense, fine to medium grained sand										
3		SP	MC	15 16 20	22		2	101			
5	**OLD PARALIC DEPOSITS** @ 5' SAND WITH SILT, yellow to reddish brown, damp, medium dense, fine grained sand		MC	13 10 12	14		7	96			
7.5			BULK								
8			MC	8 13 17	18		2	99			
10		SP-SM	MC	10 16 24	25		3	102			
15	@ 15' SAND, light gray to yellow brown, wet, medium dense, fine grained sand		SPT	7 11 11	25		5				
20	@ 20' becomes white to yellow brown, fine to medium grained	SP	MC	5 16 25	25		3	115			
25	@ 25' becomes saturated; seepage encountered		SPT	4 8 11	21		17				

(Continued Next Page)

BORING NUMBER B-1

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-1 continued...*	SP									
30		@ 30' SANDY-CLAY, dark gray, moist, very stiff, trace fine grained sand, low plasticity in field	CL	MC	4 7 12	12		22	102			
35		@ 35' CLAYEY-SAND, gray, saturated, loose to medium dense, fine grained sand	SC	SPT	4 4 6	11		23				
40		@ 40' SAND WITH SILT, gray, saturated, medium dense, fine grained	SP-SM	MC	9 19 28	29		24	99			
45		@ 45' becomes dense		SPT	7 16 31	52		23				
49		@ 49' difficult drilling										

Total Depth: 49-feet (practical refusal)
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-10/P-4

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 10/10/16 **COMPLETED** 10/10/16
GROUND ELEVATION 50 feet **BORING DIAMETER** 8-inch
EQUIPMENT / RIG CalPac CME B61
HAMMER EFFICIENCY (%) 67
METHOD 8" Hollow Stem Auger 140 lbs Auto Hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY BM **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		5-INCH A/C OVER 4-INCH BASE										
1		**FILL** CLAY, dark orange-brown, moist, medium stiff	CL									
2		**OLD PARALIC DEPOSITS (Qopfa)** @ 2' SANDY-CLAY, orange-brown, moist, stiff	SC									
6		@ 6' SAND with SILT, light orange-brown, fine to medium-grained, moist, medium dense	SP-SM									
12		@ 12' CLAY, orange-brown and gray, very moist, stiff	CL									
19				SPT	3 6 11	19						
19.5		@ 19.5' SILTY-SAND, orange-brown, fine to medium-grained, moist, medium dense	SM									
20												

Total depth: 20-feet
No groundwater encountered
Percolation test performed
Boring backfilled on 10/10/2016

BORING NUMBER B-2

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP	**PROJECT NAME** The Koll Center Residences
PROJECT NUMBER SHO-72189.4a	**PROJECT LOCATION** Newport Beach, CA
DATE STARTED 7/15/15 **COMPLETED** 7/15/15	**GROUND ELEVATION** **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63	**HAMMER EFFICIENCY (%)** 67
METHOD 140 pound Auto-hammer	**SPT CORRECTION** 1.12 **CAL CORRECTION** 0.61
LOGGED BY ML **CHECKED BY**	**GROUNDWATER DEPTH (ft)** Not Encountered
NOTES	

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (5") / BASE (6")**										
1		**FILL** SILTY-CLAY, dark brown to reddish brown, damp, stiff										
3			CL-ML	MC	6 7 5	7		13	93			
5		**OLD PARALIC DEPOSITS** @ 5' SILTY-CLAY, yellow brown, damp, very stiff, non-plastic in field		BULK								
6				MC	7 10 18	17		14	93			
8		@ 7.5' becomes grey brown mottled		MC	6 10 19	18		22	95			
10			CL-ML	MC	4 9 24	20		18	105			
15		@ 15' SAND, light brown, damp, medium dense, fine grained		SPT	7 8 9	19		3				
17				BULK								
21			SP	MC	12 13 19	20		2	101			
25		@ 25' becomes saturated, medium dense, fine to medium grained; seepage encountered		SPT	3 6 14	22		22				
27												

(Continued Next Page)

BORING NUMBER B-2

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-2 continued...*										
30			SP	MC	11 16 21	23		21	97			
35		@ 35' SAND WITH SILT, reddish brown, saturated, dense, fine to medium grained sand	SP-SM	SPT	10 16 25	46		18				
38		@ 39' difficult drilling										
39												

Total Depth: 39-feet (practical refusal)
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-3

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/15/15 **COMPLETED** 7/15/15
GROUND ELEVATION ____ **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY ML **CHECKED BY** ____
GROUNDWATER DEPTH (ft) Not Encountered
NOTES ____

DEPTH (ft)	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0	**ASPHALT (5") / BASE (6")**										
1	**FILL** CLAY, dark reddish brown, damp, very stiff, medium plasticity in field	CL									
3			MC	6 8 13	13		15	113			
5	**OLD PARALIC DEPOSITS** @ 5' SILTY-SAND, yellow brown, damp, loose, fine grained sand	SM	MC	5 6 10	10		8	101			
7.5	@ 7.5' becomes light gray to yellow brown mottled, medium dense		MC	5 9 12	13		5	100			
10			MC	8 13 18	19		5	91			
15	@ 15' SAND, light gray to yellow brown, damp, medium dense, fine to medium grained	SP	SPT	6 9 12	23		2				
20			MC	11 19 23	26		2	92			
25	@ 25' becomes reddish brown, saturated, dense; seepage encountered		SPT	10 14 16	34		18				
27			BULK								

(Continued Next Page)

BORING NUMBER B-3

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-3 continued...*	SP									
29												
30		@ 30' SAND, light brown, saturated, very dense, fine to coarse grained; seepage encountered	SW	MC	26 40 50/5"			9	120			
31												
32		@ 33' difficult drilling										
33												

Total Depth: 33-feet (practical refusal)
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-4

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/15/15 **COMPLETED** 7/15/15
GROUND ELEVATION **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY ML **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (5") / BASE (6")**										
1		**FILL** CLAYEY-SAND, reddish brown, damp, medium dense, fine to medium grained										
3			SC	MC	7 9 11	12		8	114			
5		**OLD PARALIC DEPOSITS** @ 5' SAND, light reddish brown, moist, medium dense, fine to medium grained		MC	7 11 17	17		4	105			
8				MC	10 14 21	21		4	106			
10				MC	10 18 21	24		3	96			
15		@ 15' becomes light brown	SP	SPT	8 10 16	29		4				
20				MC	13 19 29	29		17	104			
25		▽ @ 25' becomes dense, saturated; seepage encountered		SPT	8 17 29	51		24				

(Continued Next Page)

BORING NUMBER B-4

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28 29		*Boring B-4 continued...*	SP									
30 31		@ 30' SILTY-CLAY, brown to reddish brown, moist, hard, medium plasticity in field	CL-ML	MC	16 17 30	29		33	83			

Total Depth: 31.5-feet
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-5

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/16/15 **COMPLETED** 7/16/15
GROUND ELEVATION **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY ML **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (8") / BASE (4")**										
1		**FILL** CLAYEY-SILT, gray and brown mixed, moist, medium stiff, trace sand										
3			CL-ML	MC	3 3 3	4		10	99			
5		**OLD PARALIC DEPOSITS** @ 5' SANDY-CLAY, reddish brown, moist, stiff	CL	MC	3 8 11	12		18	101			
7.5		@ 7.5' SILTY-SAND, light reddish brown, moist, medium dense		MC	7 14 26	25		5	113			
10			SM	MC	12 16 28	27		5	111			
15		@ 15' SAND, light gray, moist, dense, fine grained		MC	10 28 46	45		3	103			
20		@ 20' becomes saturated, medium dense, some thin clay layers; seepage encountered	SP	MC	10 14 16	18		17	98			
25				MC	11 22 30	32		16	108			

(Continued Next Page)

BORING NUMBER B-5

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-5 continued...*	SP									
30		@ 30' CLAY, gray-brown, moist, stiff	CL	MC	7 7 10	10		20	103			
35		@ 35' increase in silt content, becomes very stiff		MC	7 10 18	17		23	104			

Total Depth: 36.5-feet
Seepage encountered at 20'

Backfilled on 7/16/15 with benonite and drilled cuttings

GEOTECH LOG - COLUMNS SHO-72189.4 BORING LOGS.GPJ GINT STD US LAB.GDT 6/9/17

BORING NUMBER B-6 / P-1

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/16/15 **COMPLETED** 7/16/15
GROUND ELEVATION **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY ML **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (6") / BASE (5")**										
1		**FILL** SILTY-SAND, reddish brown, moist, medium dense	SM									
5		**OLD PARALIC DEPOSITS** @ 5' SAND, reddish brown, wet, medium dense, fine to medium grained	SP	MC	8 14 16	18		10	112			
10		@ 10' SAND, dark reddish brown, wet, medium dense, fine to coarse grained	SW	MC	9 15 20	21		3	136			
13												

Total Depth: 13-feet
No Groundwater Encountered
Percolation Test Performed

Backfilled on 7/16/15 with benonite and drilled cuttings

BORING NUMBER B-7 / P-2

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP		**PROJECT NAME** The Koll Center Residences	
PROJECT NUMBER SHO-72189.4a		**PROJECT LOCATION** Newport Beach, CA	
DATE STARTED 7/16/15	**COMPLETED** 7/16/15	**GROUND ELEVATION**	**BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63		**HAMMER EFFICIENCY (%)** 67	
METHOD 140 pound Auto-hammer		**SPT CORRECTION** 1.12	**CAL CORRECTION** 0.61
LOGGED BY ML	**CHECKED BY**	**GROUNDWATER DEPTH (ft)** Not Encountered	
NOTES			

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (6") / BASE (5")**										
1		**FILL** SILTY-SAND, reddish brown, moist, medium dense	SM									
5		**OLD PARALIC DEPOSITS** @ 5' CLAYEY-SAND, reddish brown, moist, medium dense, fine grained	SC	MC	5 9 13	14		4	112			
10		@ 10' SAND, yellow brown, damp, medium dense, fine grained	SP	MC	6 15 18	20		3	106			
12												

Total Depth: 12-feet
No Groundwater Encountered
Percolation Test Performed

Backfilled on 7/16/15 with benonite and drilled cuttings

BORING NUMBER B-8

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 10/10/16 **COMPLETED** 10/10/16
GROUND ELEVATION 51 feet **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B61
HAMMER EFFICIENCY (%) 67
METHOD 8" Hollow Stem Auger 140 lbs Auto Hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY BM **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		5-INCH A/C OVER 7-INCH BASE										
1		**FILL** CLAY, dark brown, moist, soft	CL									
2.5		**OLD PARALIC DEPOSITS (Qopfa)** @ 2.5' CLAYEY-SAND, orange-brown, fine to medium-grained, moist, loose	SC	BULK								
5				MC	5 9 12	13						
7.5		@ 7.5' SAND, light orange-brown, fine to medium-grained, damp, medium dense	SP	MC	8 14 21	21						
10		@ 10' SAND, light orange, well-sorted grains with some gravel, damp, medium dense	SW	MC	12 18 26	27						
15		@ 15' CLAY, orange-brown and gray, moist, stiff	CL	MC	4 7 14	13						
20				MC	5 8 12	12						
25		@ 25' SAND, light orange, fine to medium-grained, moist, medium dense @ 26' Perched groundwater encountered	SP	MC	16 28 40	42						

(Continued Next Page)

BORING NUMBER B-8

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28–30		@ 25' SAND, light orange, fine to medium-grained, moist, medium dense *(continued)*	SP									
30–35		@ 30' SAND, light gray, well-sorted grains, wet, medium dense	SW	MC	19 33 42	46						
35–40		@ 35' SAND, dark gray, fine-grained, wet, medium dense	SP	MC	11 37 50	53						
40–45		@ 40' CLAY, dark gray, high plasticity, saturated, medium stiff	CH	MC	2 4 6	6						
45–50		@ 45' Becomes soft	CH	SPT	1 2 2	4						
50–51.5			CH	SPT	1 2 2	4						

Total depth: 51.5-feet
Perched groundwater encountered at 26-feet
Boring backfilled on 10/10/2016

BORING NUMBER B-9/P-3

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 10/10/16 **COMPLETED** 10/10/16
GROUND ELEVATION 51 feet **BORING DIAMETER** 8-inch
EQUIPMENT / RIG CalPac CME B61
HAMMER EFFICIENCY (%) 67
METHOD 8" Hollow Stem Auger 140 lbs Auto Hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY BM **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		5-INCH A/C OVER 4-INCH BASE										
1		**FILL** CLAY, dark orange-brown, moist, medium stiff	CL									
2–5		**OLD PARALIC DEPOSITS (Qopfa)** @ 2' SANDY-CLAY, orange-brown, moist, stiff	SC									
5–10		@ 5' SAND with SILT, light orange-brown, fine to medium-grained, moist, medium dense	SP-SM	SPT (at 8.5–10)	7 11 14	28						
10												

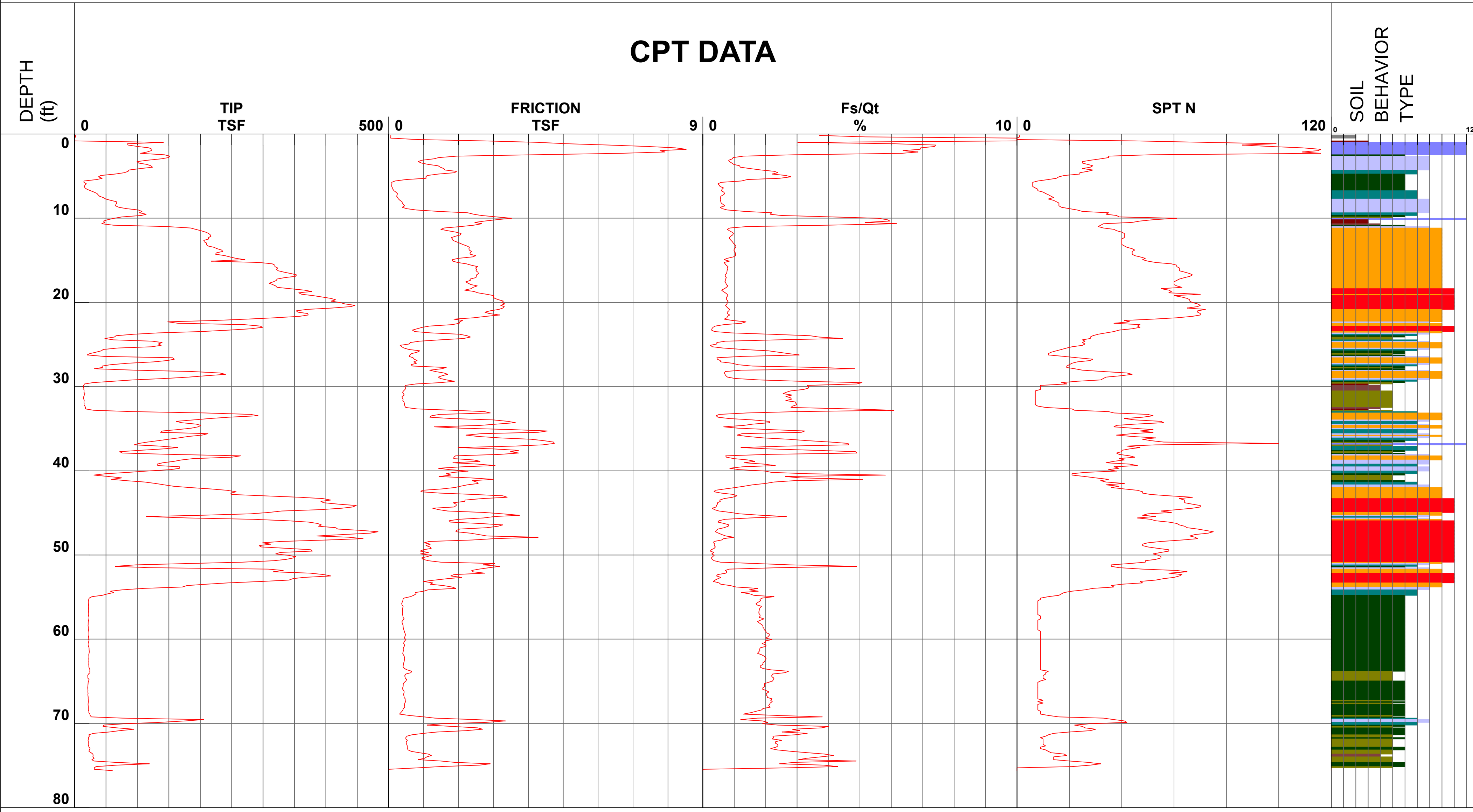
Total depth: 10-feet
No groundwater encountered
Percolation test performed
Boring backfilled on 10/10/2016

EEI

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(685).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	7/21/2015 8:28:08 AM	Maximum Depth	75.62 ft
EST GW Depth During Test		23.00 ft			

Net Area Ratio .8

CPT DATA



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

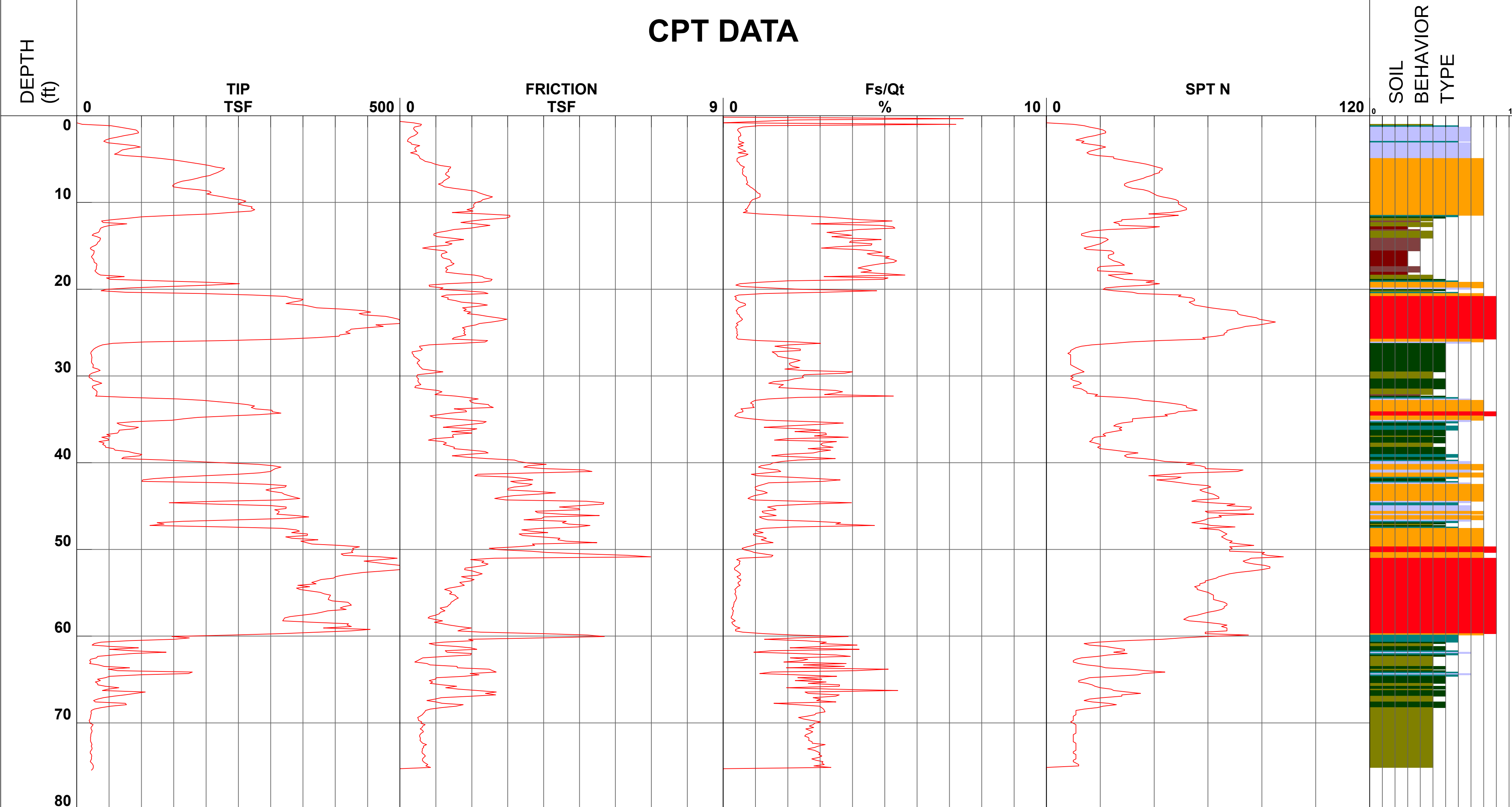
Cone Size 10cm squared

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

EEI

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(686).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	7/21/2015 9:44:18 AM	Maximum Depth	75.46 ft
EST GW Depth During Test		23.00 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 (*)

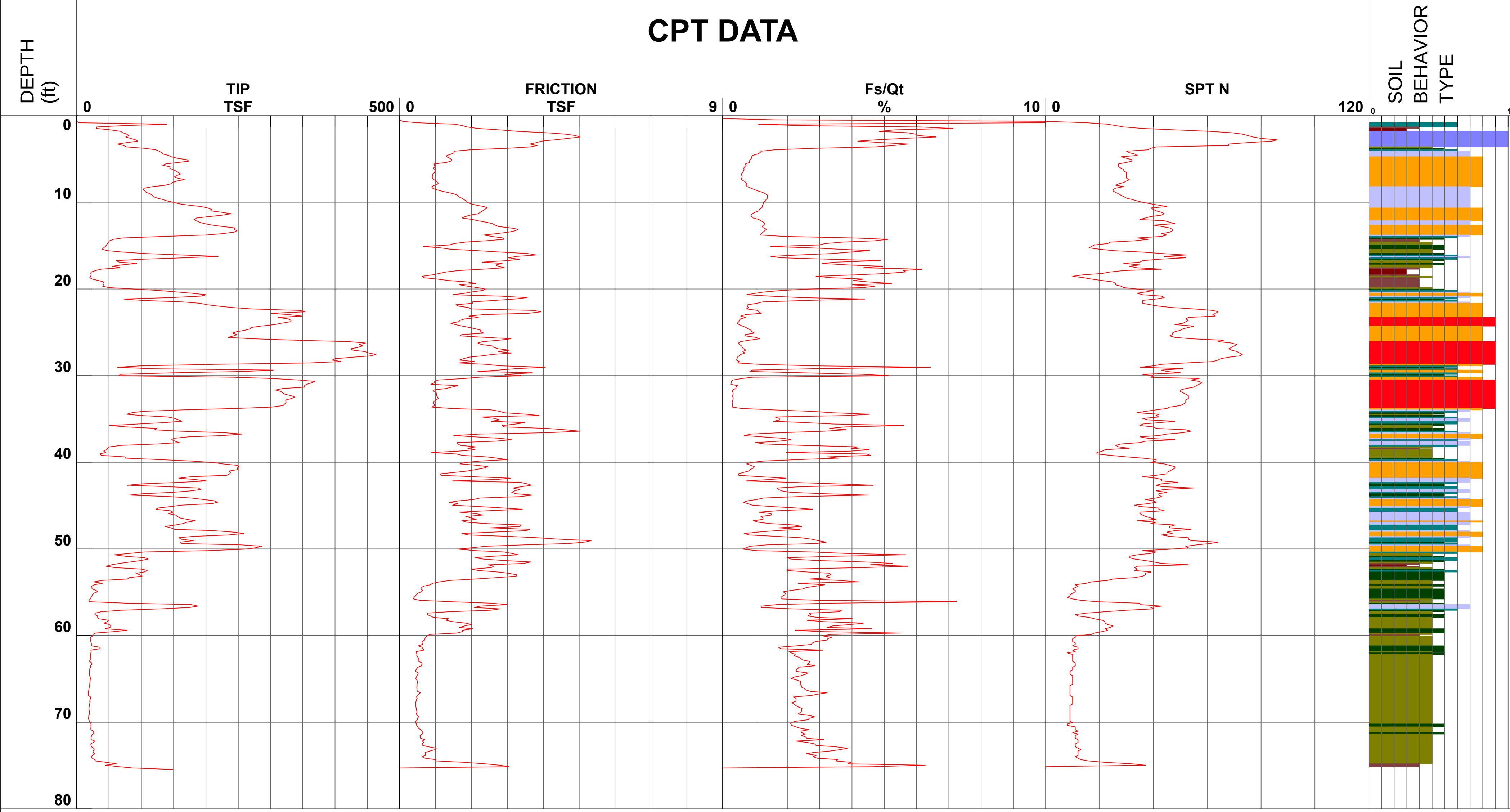
Cone Size 10cm squared

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

EEI

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(687).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-03	Date and Time	7/21/2015 10:50:47 AM	Maximum Depth	75.46 ft
EST GW Depth During Test		23.00 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

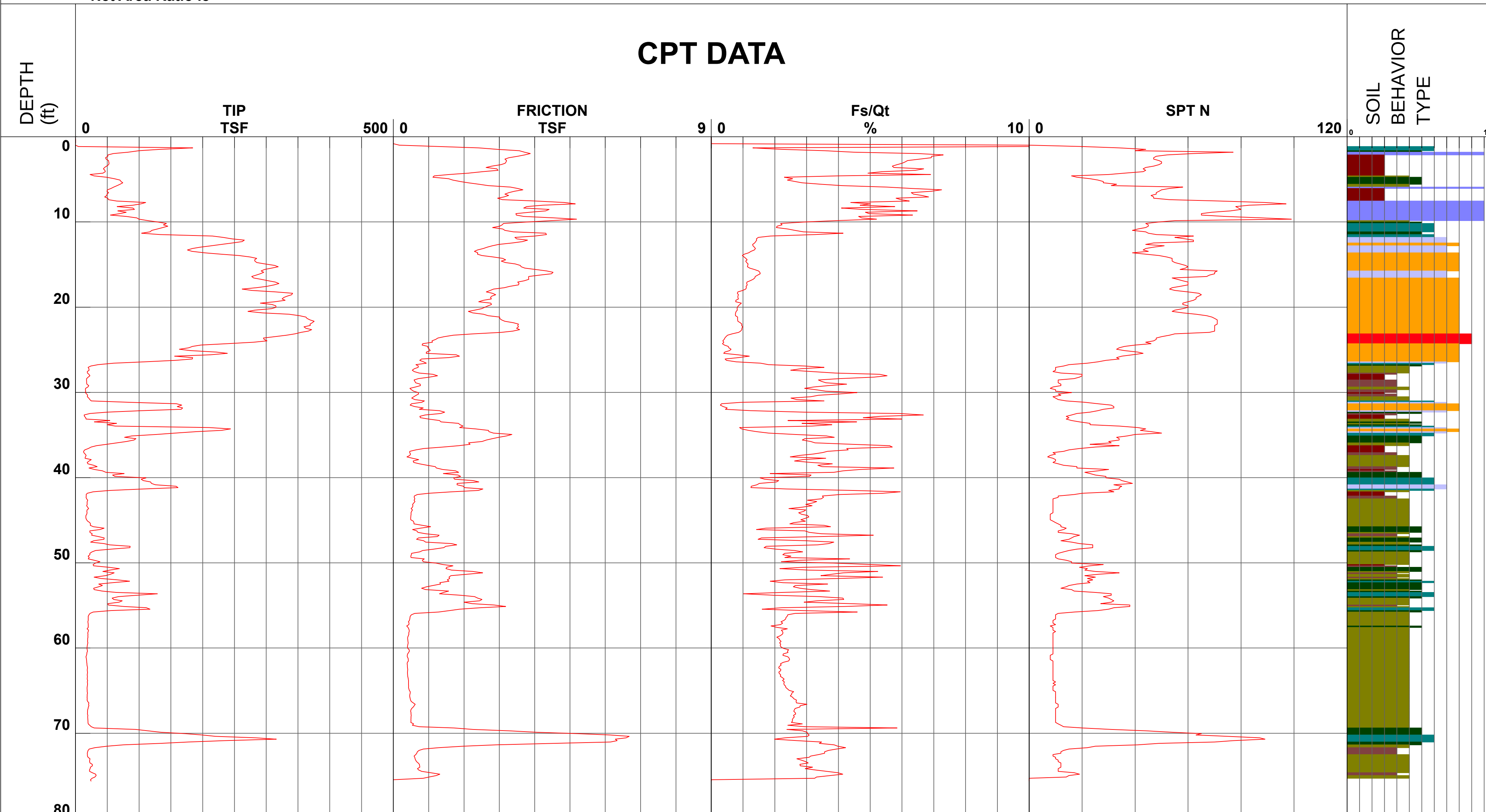
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Middle Earth Geo Testing Inc.

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(688).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	7/21/2015 11:49:48 AM	Maximum Depth	75.62 ft
EST GW Depth During Test		23.00 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

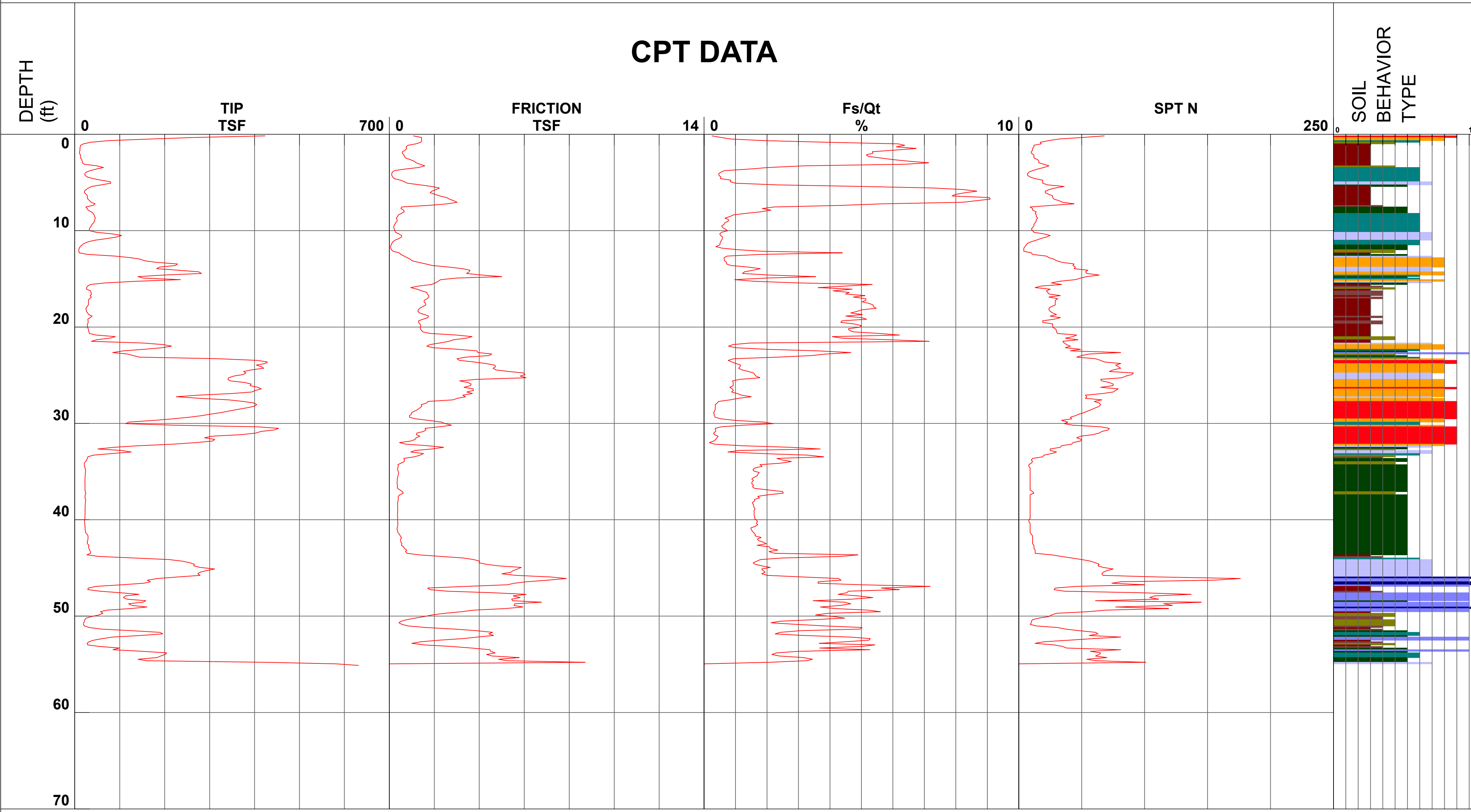
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project	The Koll Center Residences	Operator	DG-RC	Filename	SDF(195).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-05	Date and Time	10/18/2016 9:56:17 AM	Maximum Depth	55.12 ft
EST GW Depth During Test		26.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

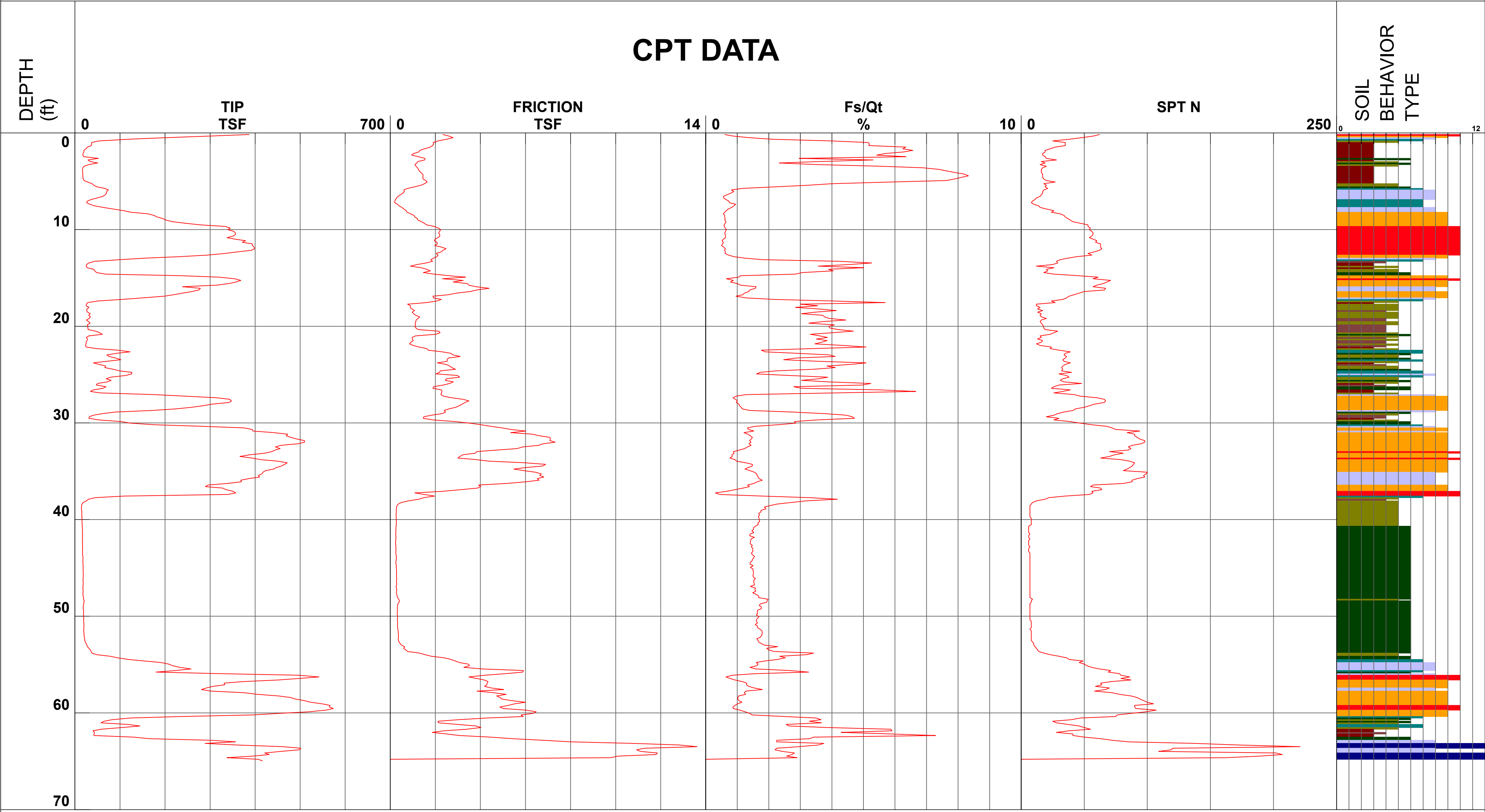
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project	The Koll Center Residences	Operator	DG-RC	Filename	SDF(196).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-06	Date and Time	10/18/2016 10:49:31 AM	Maximum Depth	64.96 ft
EST GW Depth During Test		26.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

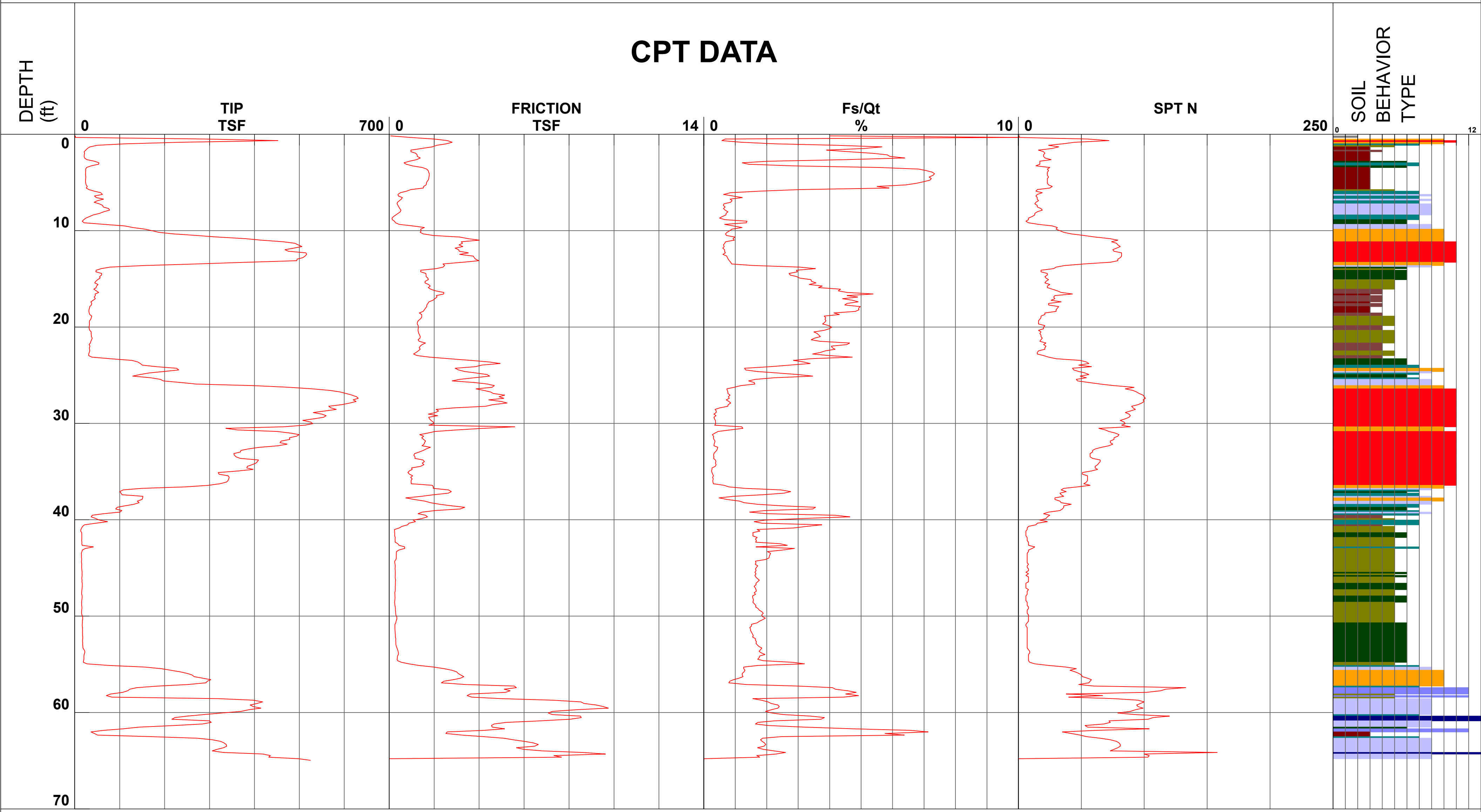
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project	The Koll Center Residences	Operator	DG-RC	Filename	SDF(197).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-07	Date and Time	10/18/2016 12:13:26 PM	Maximum Depth	64.96 ft
EST GW Depth During Test		26.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

BORING NUMBER B-1

CLIENT	Shopoff Land Fund II, LP	**PROJECT NAME**	The Koll Center Residences
PROJECT NUMBER	SHO-72189.4a	**PROJECT LOCATION**	Newport Beach, CA

DATE STARTED 7/15/15	**COMPLETED** 7/15/15	**GROUND ELEVATION**	**BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63		**HAMMER EFFICIENCY (%)** 67	
METHOD 140 pound Auto-hammer		**SPT CORRECTION** 1.12	**CAL CORRECTION** 0.61
LOGGED BY ML	**CHECKED BY**	**GROUNDWATER DEPTH (ft)** Not Encountered	
NOTES			

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (5") / BASE (6"),**										
1		**FILL** SAND, reddish brown, damp, medium dense, fine to medium grained sand										
3			SP	MC	15 16 20	22		2	101			
5		**OLD PARALIC DEPOSITS** @ 5' SAND WITH SILT, yellow to reddish brown, damp, medium dense, fine grained sand		MC	13 10 12	14		7	96			
7.5				BULK								
8				MC	8 13 17	18		2	99			
10			SP-SM	MC	10 16 24	25		3	102			
15		@ 15' SAND, light gray to yellow brown, wet, medium dense, fine grained sand		SPT	7 11 11	25		5				
20		@ 20' becomes white to yellow brown, fine to medium grained		MC	5 16 25	25		3	115			
21.5			SP									
25		@ 25' becomes saturated; seepage encountered		SPT	4 8 11	21		17				

(Continued Next Page)

GEOTECH LOG - COLUMNS SHO-72189.4 BORING LOGS.GPJ GINT STD US LAB.GDT 6/9/17

BORING NUMBER B-1

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP | **PROJECT NAME** The Koll Center Residences

PROJECT NUMBER SHO-72189.4a | **PROJECT LOCATION** Newport Beach, CA

DEPTH (ft)	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28–30	*Boring B-1 continued...*	SP									
30	@ 30' SANDY-CLAY, dark gray, moist, very stiff, trace fine grained sand, low plasticity in field	CL	MC	4 7 12	12		22	102			
35	@ 35' CLAYEY-SAND, gray, saturated, loose to medium dense, fine grained sand	SC	SPT	4 4 6	11		23				
40	@ 40' SAND WITH SILT, gray, saturated, medium dense, fine grained	SP-SM	MC	9 19 28	29		24	99			
45	@ 45' becomes dense	SP-SM	SPT	7 16 31	52		23				
49	@ 49' difficult drilling	SP-SM									

Total Depth: 49-feet (practical refusal)
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-10/P-4

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 10/10/16 **COMPLETED** 10/10/16
GROUND ELEVATION 50 feet **BORING DIAMETER** 8-inch
EQUIPMENT / RIG CalPac CME B61
HAMMER EFFICIENCY (%) 67
METHOD 8" Hollow Stem Auger 140 lbs Auto Hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY BM **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		5-INCH A/C OVER 4-INCH BASE										
1		**FILL** CLAY, dark orange-brown, moist, medium stiff	CL									
2		**OLD PARALIC DEPOSITS (Qopfa)** @ 2' SANDY-CLAY, orange-brown, moist, stiff	SC									
6		@ 6' SAND with SILT, light orange-brown, fine to medium-grained, moist, medium dense	SP-SM									
12		@ 12' CLAY, orange-brown and gray, very moist, stiff	CL									
19				SPT	3 6 11	19						
19.5		@ 19.5' SILTY-SAND, orange-brown, fine to medium-grained, moist, medium dense	SM									
20												

Total depth: 20-feet
No groundwater encountered
Percolation test performed
Boring backfilled on 10/10/2016

BORING NUMBER B-2

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/15/15 **COMPLETED** 7/15/15
GROUND ELEVATION ______ **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY ML **CHECKED BY** ______
GROUNDWATER DEPTH (ft) Not Encountered
NOTES ______

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (5") / BASE (6")**										
1		**FILL** SILTY-CLAY, dark brown to reddish brown, damp, stiff										
3			CL-ML	MC	6 7 5	7		13	93			
5		**OLD PARALIC DEPOSITS** @ 5' SILTY-CLAY, yellow brown, damp, very stiff, non-plastic in field		BULK								
5.5				MC	7 10 18	17		14	93			
7.5		@ 7.5' becomes grey brown mottled		MC	6 10 19	18		22	95			
10			CL-ML	MC	4 9 24	20		18	105			
15		@ 15' SAND, light brown, damp, medium dense, fine grained		SPT	7 8 9	19		3				
17				BULK								
20			SP	MC	12 13 19	20		2	101			
25		@ 25' becomes saturated, medium dense, fine to medium grained; seepage encountered		SPT	3 6 14	22		22				

(Continued Next Page)

BORING NUMBER B-2

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-2 continued...*	SP									
30				MC	11 16 21	23		21	97			
35		@ 35' SAND WITH SILT, reddish brown, saturated, dense, fine to medium grained sand	SP-SM	SPT	10 16 25	46		18				
39		@ 39' difficult drilling										

Total Depth: 39-feet (practical refusal)
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-3

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 7/15/15 **COMPLETED** 7/15/15
GROUND ELEVATION **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63
HAMMER EFFICIENCY (%) 67
METHOD 140 pound Auto-hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY ML **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0	**ASPHALT (5") / BASE (6")**										
1	**FILL** CLAY, dark reddish brown, damp, very stiff, medium plasticity in field	CL									
3			MC	6 8 13	13		15	113			
5	**OLD PARALIC DEPOSITS** @ 5' SILTY-SAND, yellow brown, damp, loose, fine grained sand	SM	MC	5 6 10	10		8	101			
7.5	@ 7.5' becomes light gray to yellow brown mottled, medium dense		MC	5 9 12	13		5	100			
10			MC	8 13 18	19		5	91			
15	@ 15' SAND, light gray to yellow brown, damp, medium dense, fine to medium grained	SP	SPT	6 9 12	23		2				
20			MC	11 19 23	26		2	92			
25	@ 25' becomes reddish brown, saturated, dense; seepage encountered		SPT	10 14 16	34		18				
27			BULK								

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EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP

PROJECT NAME The Koll Center Residences

PROJECT NUMBER SHO-72189.4a

PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28 29		*Boring B-3 continued...*	SP									
30 31 32 33		@ 30' SAND, light brown, saturated, very dense, fine to coarse grained; seepage encountered @ 33' difficult drilling	SW	MC	26 40 50/5"			9	120			

Total Depth: 33-feet (practical refusal)
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-4

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP		**PROJECT NAME** The Koll Center Residences	
PROJECT NUMBER SHO-72189.4a		**PROJECT LOCATION** Newport Beach, CA	
DATE STARTED 7/15/15	**COMPLETED** 7/15/15	**GROUND ELEVATION**	**BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63		**HAMMER EFFICIENCY (%)** 67	
METHOD 140 pound Auto-hammer		**SPT CORRECTION** 1.12	**CAL CORRECTION** 0.61
LOGGED BY ML	**CHECKED BY**	**GROUNDWATER DEPTH (ft)** Not Encountered	
NOTES			

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (5") / BASE (6")**										
1		**FILL** CLAYEY-SAND, reddish brown, damp, medium dense, fine to medium grained										
3			SC	MC	7 9 11	12		8	114			
5		**OLD PARALIC DEPOSITS** @ 5' SAND, light reddish brown, moist, medium dense, fine to medium grained		MC	7 11 17	17		4	105			
8				MC	10 14 21	21		4	106			
10				MC	10 18 21	24		3	96			
15		@ 15' becomes light brown	SP	SPT	8 10 16	29		4				
20				MC	13 19 29	29		17	104			
25		@ 25' becomes dense, saturated; seepage encountered		SPT	8 17 29	51		24				

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EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP **PROJECT NAME** The Koll Center Residences

PROJECT NUMBER SHO-72189.4a **PROJECT LOCATION** Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28 29		*Boring B-4 continued...*	SP									
30 31		@ 30' SILTY-CLAY, brown to reddish brown, moist, hard, medium plasticity in field	CL-ML	MC	16 17 30	29		33	83			

Total Depth: 31.5-feet
Seepage encountered at 25'

Backfilled on 7/15/15 with benonite and drilled cuttings

BORING NUMBER B-5

CLIENT Shopoff Land Fund II, LP		**PROJECT NAME** The Koll Center Residences	
PROJECT NUMBER SHO-72189.4a		**PROJECT LOCATION** Newport Beach, CA	
DATE STARTED 7/16/15	**COMPLETED** 7/16/15	**GROUND ELEVATION**	**BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63		**HAMMER EFFICIENCY (%)** 67	
METHOD 140 pound Auto-hammer		**SPT CORRECTION** 1.12	**CAL CORRECTION** 0.61
LOGGED BY ML	**CHECKED BY**	**GROUNDWATER DEPTH (ft)** Not Encountered	
NOTES			

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (8") / BASE (4")**										
1		**FILL** CLAYEY-SILT, gray and brown mixed, moist, medium stiff, trace sand										
3			CL-ML	MC	3 3 3	4		10	99			
5		**OLD PARALIC DEPOSITS** @ 5' SANDY-CLAY, reddish brown, moist, stiff	CL	MC	3 8 11	12		18	101			
7.5		@ 7.5' SILTY-SAND, light reddish brown, moist, medium dense		MC	7 14 26	25		5	113			
10			SM	MC	12 16 28	27		5	111			
15		@ 15' SAND, light gray, moist, dense, fine grained		MC	10 28 46	45		3	103			
20		▽ @ 20' becomes saturated, medium dense, some thin clay layers; seepage encountered	SP	MC	10 14 16	18		17	98			
25				MC	11 22 30	32		16	108			

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BORING NUMBER B-5

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28		*Boring B-5 continued...*										
29			SP									
30		@ 30' CLAY, gray-brown, moist, stiff		MC	7 7 10	10		20	103			
31												
32												
33			CL									
34												
35		@ 35' increase in silt content, becomes very stiff		MC	7 10 18	17		23	104			
36												

Total Depth: 36.5-feet
Seepage encountered at 20'

Backfilled on 7/16/15 with benonite and drilled cuttings

GEOTECH LOG - COLUMNS SHO-72189.4 BORING LOGS.GPJ GINT STD US LAB.GDT 6/9/17

BORING NUMBER B-6 / P-1

CLIENT Shopoff Land Fund II, LP		**PROJECT NAME** The Koll Center Residences	
PROJECT NUMBER SHO-72189.4a		**PROJECT LOCATION** Newport Beach, CA	
DATE STARTED 7/16/15	**COMPLETED** 7/16/15	**GROUND ELEVATION**	**BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63		**HAMMER EFFICIENCY (%)** 67	
METHOD 140 pound Auto-hammer		**SPT CORRECTION** 1.12	**CAL CORRECTION** 0.61
LOGGED BY ML	**CHECKED BY**	**GROUNDWATER DEPTH (ft)** Not Encountered	
NOTES			

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (6") / BASE (5")**										
1		**FILL** SILTY-SAND, reddish brown, moist, medium dense	SM									
5		**OLD PARALIC DEPOSITS** @ 5' SAND, reddish brown, wet, medium dense, fine to medium grained	SP	MC	8 14 16	18		10	112			
10		@ 10' SAND, dark reddish brown, wet, medium dense, fine to coarse grained	SW	MC	9 15 20	21		3	136			
13												

Total Depth: 13-feet
No Groundwater Encountered
Percolation Test Performed

Backfilled on 7/16/15 with benonite and drilled cuttings

BORING NUMBER B-7 / P-2

CLIENT Shopoff Land Fund II, LP		**PROJECT NAME** The Koll Center Residences	
PROJECT NUMBER SHO-72189.4a		**PROJECT LOCATION** Newport Beach, CA	
DATE STARTED 7/16/15	**COMPLETED** 7/16/15	**GROUND ELEVATION**	**BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B63		**HAMMER EFFICIENCY (%)** 67	
METHOD 140 pound Auto-hammer		**SPT CORRECTION** 1.12	**CAL CORRECTION** 0.61
LOGGED BY ML	**CHECKED BY**	**GROUNDWATER DEPTH (ft)** Not Encountered	
NOTES			

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		**ASPHALT (6") / BASE (5")**										
1		**FILL** SILTY-SAND, reddish brown, moist, medium dense	SM									
5		**OLD PARALIC DEPOSITS** @ 5' CLAYEY-SAND, reddish brown, moist, medium dense, fine grained	SC	MC	5 9 13	14		4	112			
10		@ 10' SAND, yellow brown, damp, medium dense, fine grained	SP	MC	6 15 18	20		3	106			
12												

Total Depth: 12-feet
No Groundwater Encountered
Percolation Test Performed

Backfilled on 7/16/15 with benonite and drilled cuttings

BORING NUMBER B-8

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 10/10/16 **COMPLETED** 10/10/16
GROUND ELEVATION 51 feet **BORING DIAMETER** 6-inch
EQUIPMENT / RIG CalPac CME B61
HAMMER EFFICIENCY (%) 67
METHOD 8" Hollow Stem Auger 140 lbs Auto Hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY BM **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		5-INCH A/C OVER 7-INCH BASE										
1		**FILL** CLAY, dark brown, moist, soft	CL	BULK								
2.5		**OLD PARALIC DEPOSITS (Qopfa)** @ 2.5' CLAYEY-SAND, orange-brown, fine to medium-grained, moist, loose	SC									
5				MC	5 9 12	13						
7.5		@ 7.5' SAND, light orange-brown, fine to medium-grained, damp, medium dense	SP	MC	8 14 21	21						
10		@ 10' SAND, light orange, well-sorted grains with some gravel, damp, medium dense	SW	MC	12 18 26	27						
15		@ 15' CLAY, orange-brown and gray, moist, stiff	CL	MC	4 7 14	13						
20				MC	5 8 12	12						
25		@ 25' SAND, light orange, fine to medium-grained, moist, medium dense @ 26' Perched groundwater encountered	SP	MC	16 28 40	42						

(Continued Next Page)

CLIENT Shopoff Land Fund II, LP **PROJECT NAME** The Koll Center Residences

PROJECT NUMBER SHO-72189.4a **PROJECT LOCATION** Newport Beach, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
28–30		@ 25' SAND, light orange, fine to medium-grained, moist, medium dense*(continued)*	SP									
30–35		@ 30' SAND, light gray, well-sorted grains, wet, medium dense	SW	MC	19 33 42	46						
35–40		@ 35' SAND, dark gray, fine-grained, wet, medium dense	SP	MC	11 37 50	53						
40		@ 40' CLAY, dark gray, high plasticity, saturated, medium stiff	CH	MC	2 4 6	6						
45		@ 45' Becomes soft	CH	SPT	1 2 2	4						
50–51.5			CH	SPT	1 2 2	4						

Total depth: 51.5-feet
Perched groundwater encountered at 26-feet
Boring backfilled on 10/10/2016

BORING NUMBER B-9/P-3

EEI Engineering Solutions

CLIENT Shopoff Land Fund II, LP
PROJECT NAME The Koll Center Residences
PROJECT NUMBER SHO-72189.4a
PROJECT LOCATION Newport Beach, CA
DATE STARTED 10/10/16 **COMPLETED** 10/10/16
GROUND ELEVATION 51 feet **BORING DIAMETER** 8-inch
EQUIPMENT / RIG CalPac CME B61
HAMMER EFFICIENCY (%) 67
METHOD 8" Hollow Stem Auger 140 lbs Auto Hammer
SPT CORRECTION 1.12 **CAL CORRECTION** 0.61
LOGGED BY BM **CHECKED BY**
GROUNDWATER DEPTH (ft) Not Encountered
NOTES

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	USCS SYMBOL	SAMPLE TYPE	PENETRATION RESISTANCE (blows/6-inches)	SPT N60	POCKET PEN (tsf)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	ATTERBERG LIMITS (PI:LL)	FINES CONTENT (%)	OTHER TESTS
0		5-INCH A/C OVER 4-INCH BASE										
1		**FILL** CLAY, dark orange-brown, moist, medium stiff	CL									
2		**OLD PARALIC DEPOSITS (Qopfa)** @ 2' SANDY-CLAY, orange-brown, moist, stiff	SC									
5		@ 5' SAND with SILT, light orange-brown, fine to medium-grained, moist, medium dense	SP-SM									
9				SPT	7 11 14	28						
10												

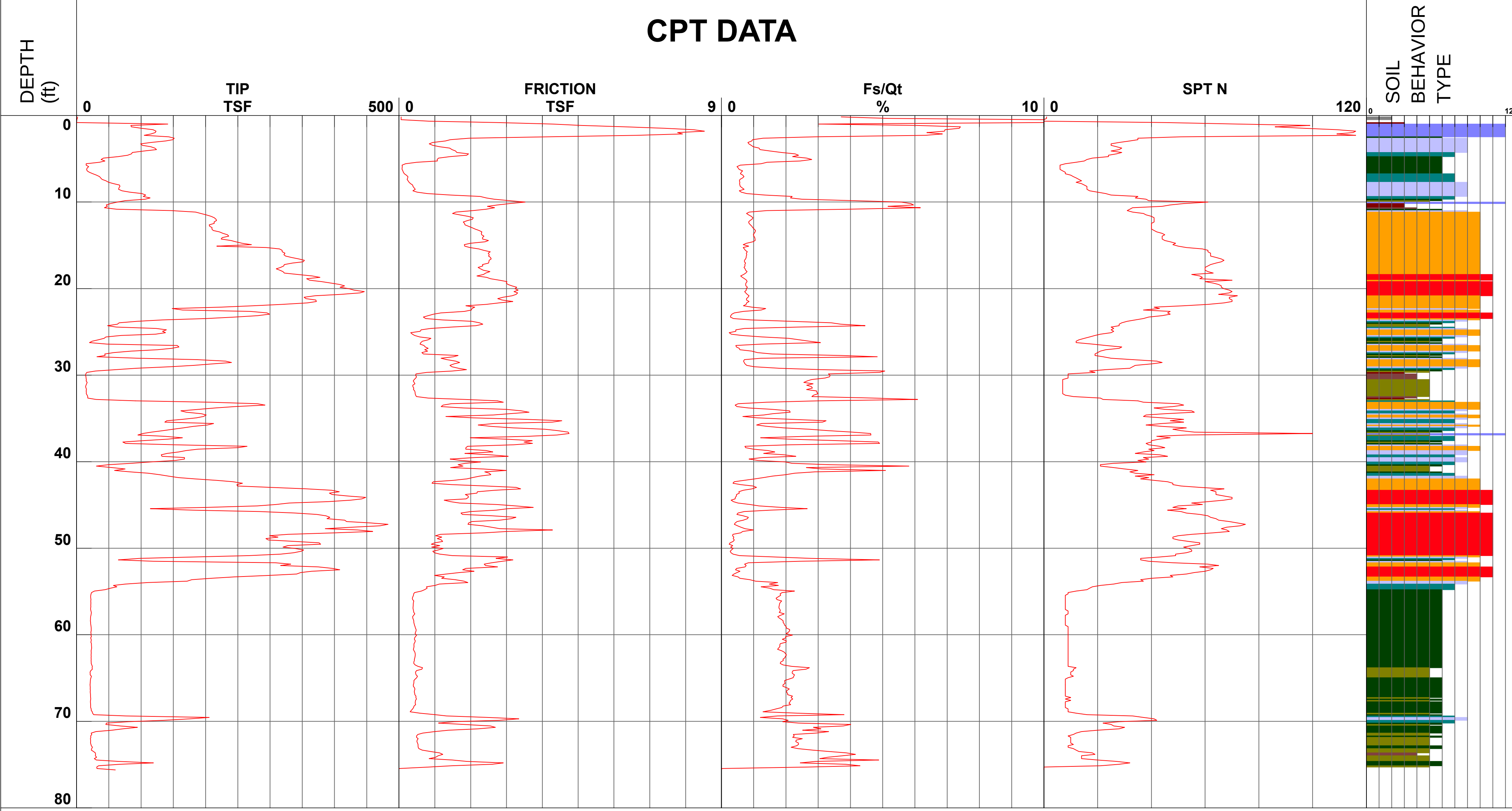
Total depth: 10-feet
No groundwater encountered
Percolation test performed
Boring backfilled on 10/10/2016

EEI

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(685).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-01	Date and Time	7/21/2015 8:28:08 AM	Maximum Depth	75.62 ft
EST GW Depth During Test		23.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

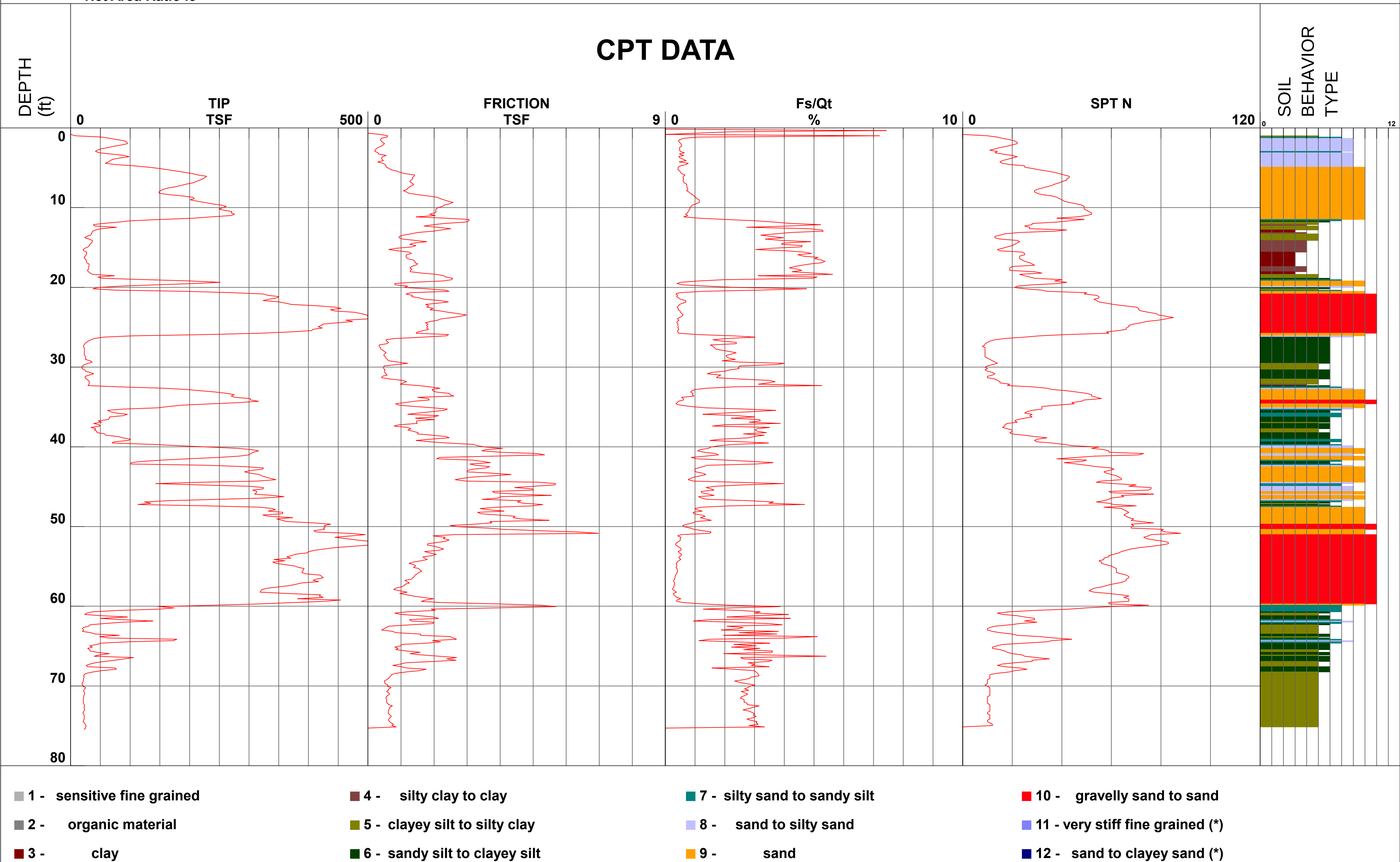
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Middle Earth Geo Testing Inc.

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(686).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-02	Date and Time	7/21/2015 9:44:18 AM	Maximum Depth	75.46 ft
EST GW Depth During Test		23.00 ft			

Net Area Ratio .8



Cone Size 10cm squared

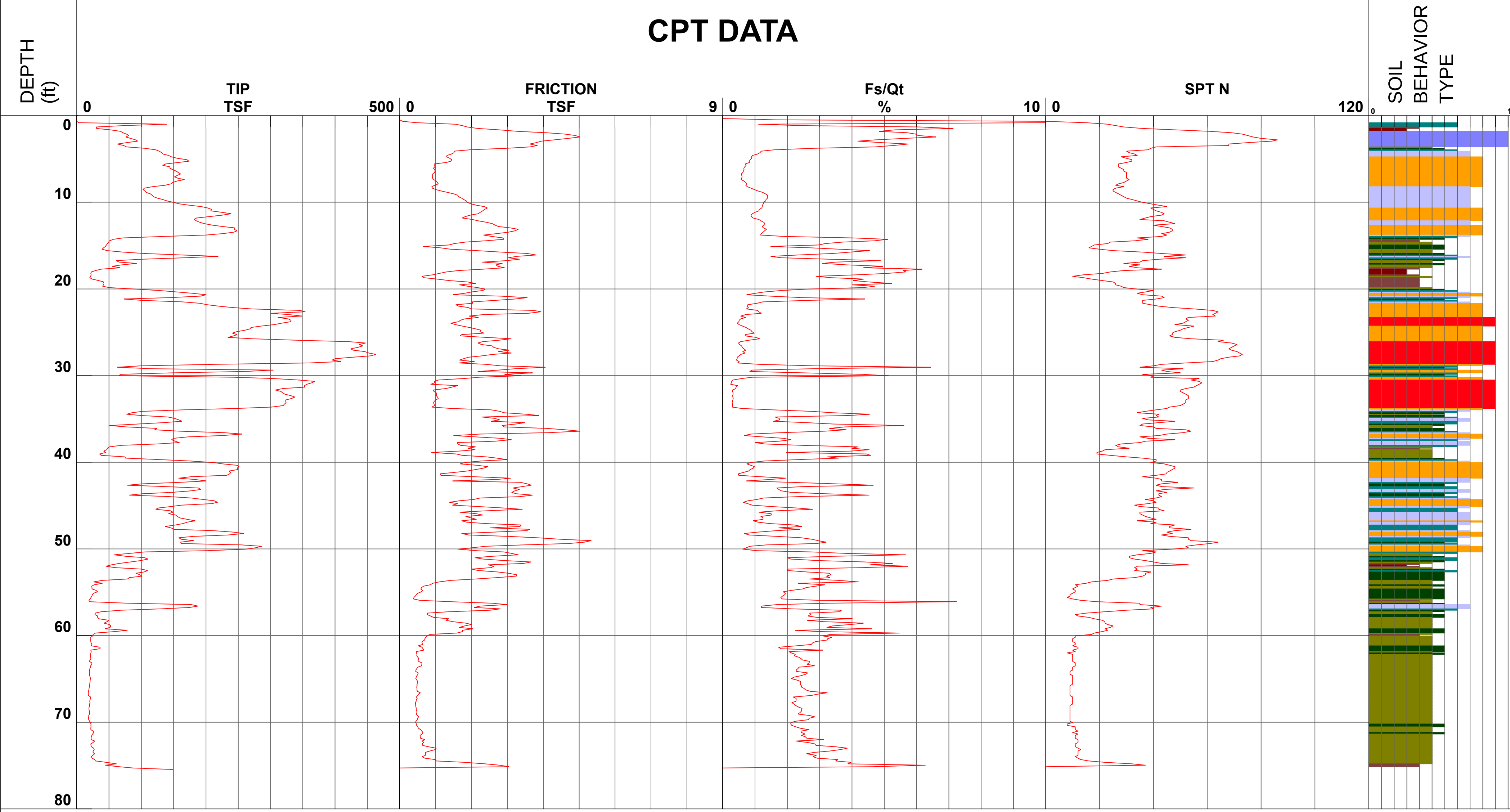
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(687).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-03	Date and Time	7/21/2015 10:50:47 AM	Maximum Depth	75.46 ft
EST GW Depth During Test		23.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

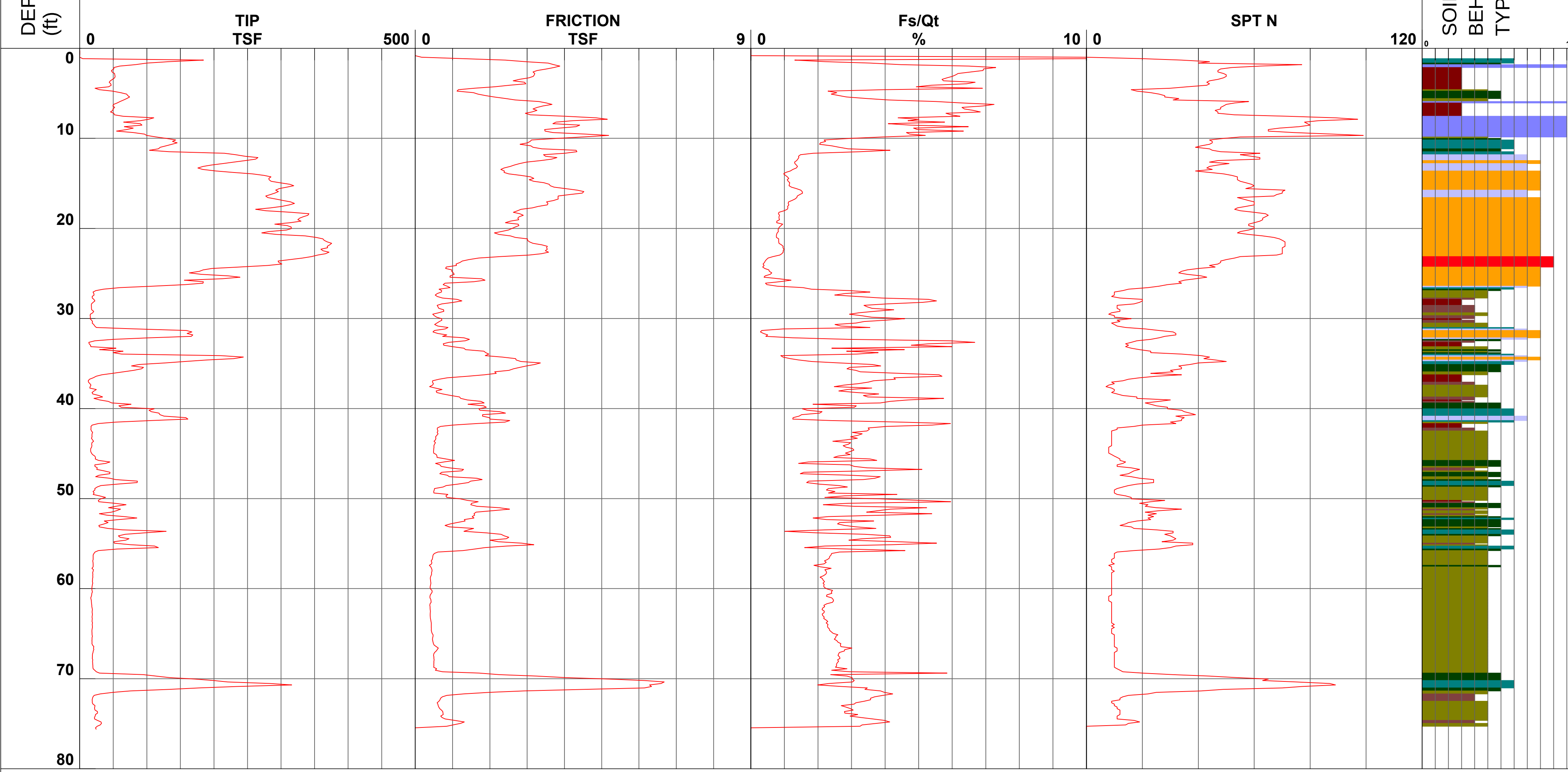
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project	The Koll Center Residences	Operator	RC-BH	Filename	SDF(688).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-04	Date and Time	7/21/2015 11:49:48 AM	Maximum Depth	75.62 ft
EST GW Depth During Test			23.00 ft		

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

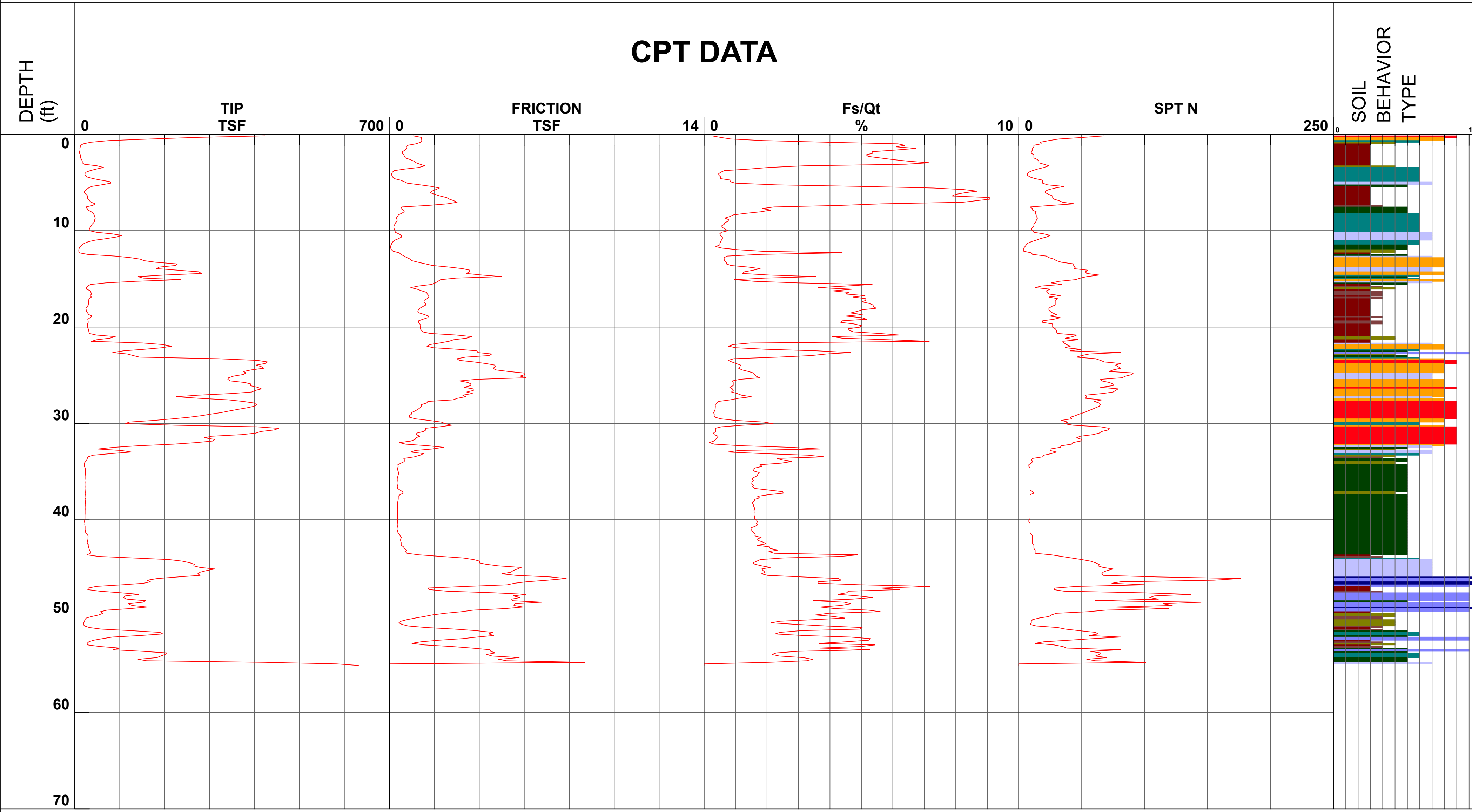
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project	The Koll Center Residences	Operator	DG-RC	Filename	SDF(195).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-05	Date and Time	10/18/2016 9:56:17 AM	Maximum Depth	55.12 ft
EST GW Depth During Test		26.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

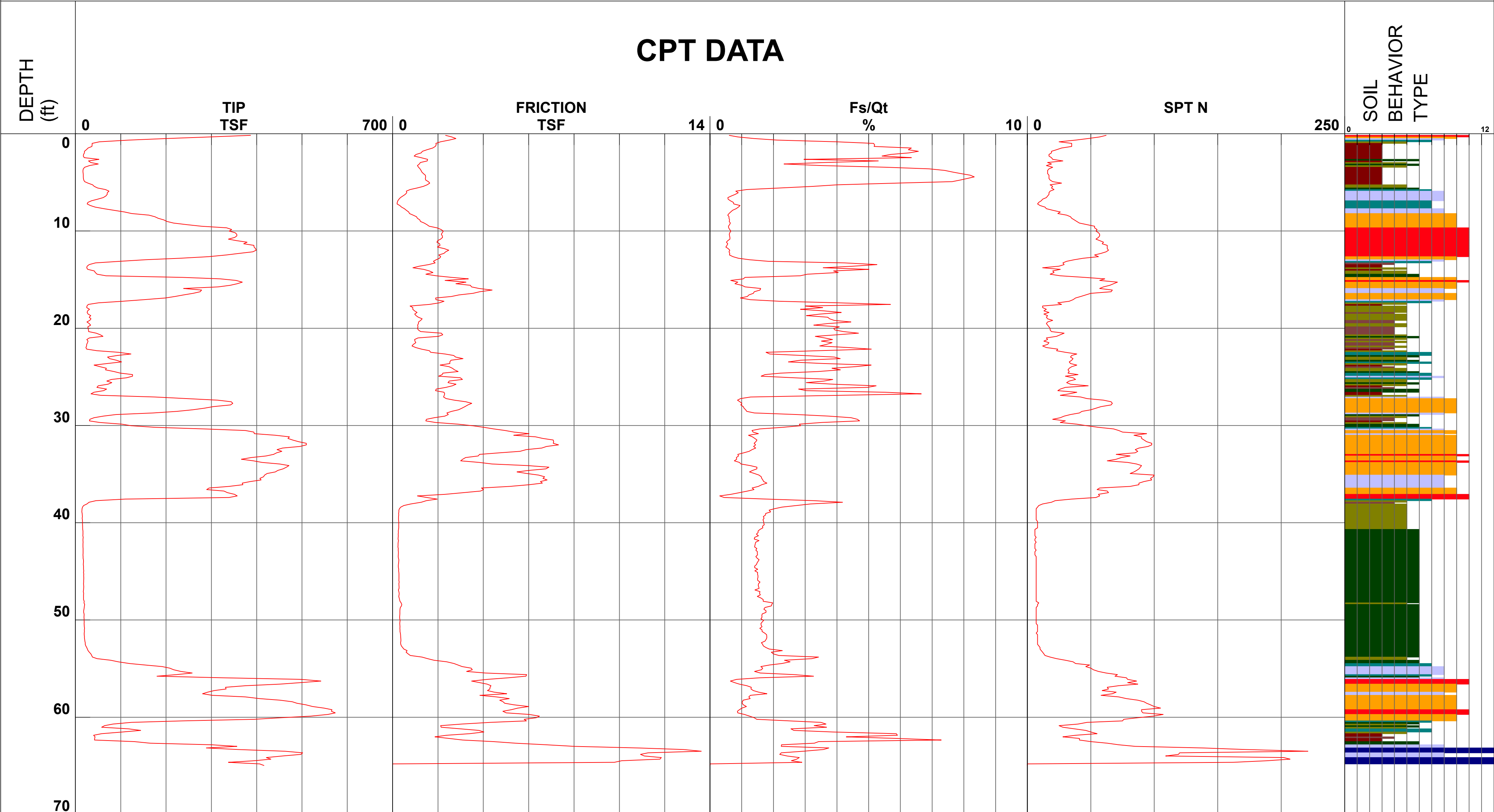
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project	The Koll Center Residences	Operator	DG-RC	Filename	SDF(196).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-06	Date and Time	10/18/2016 10:49:31 AM	Maximum Depth	64.96 ft
EST GW Depth During Test		26.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

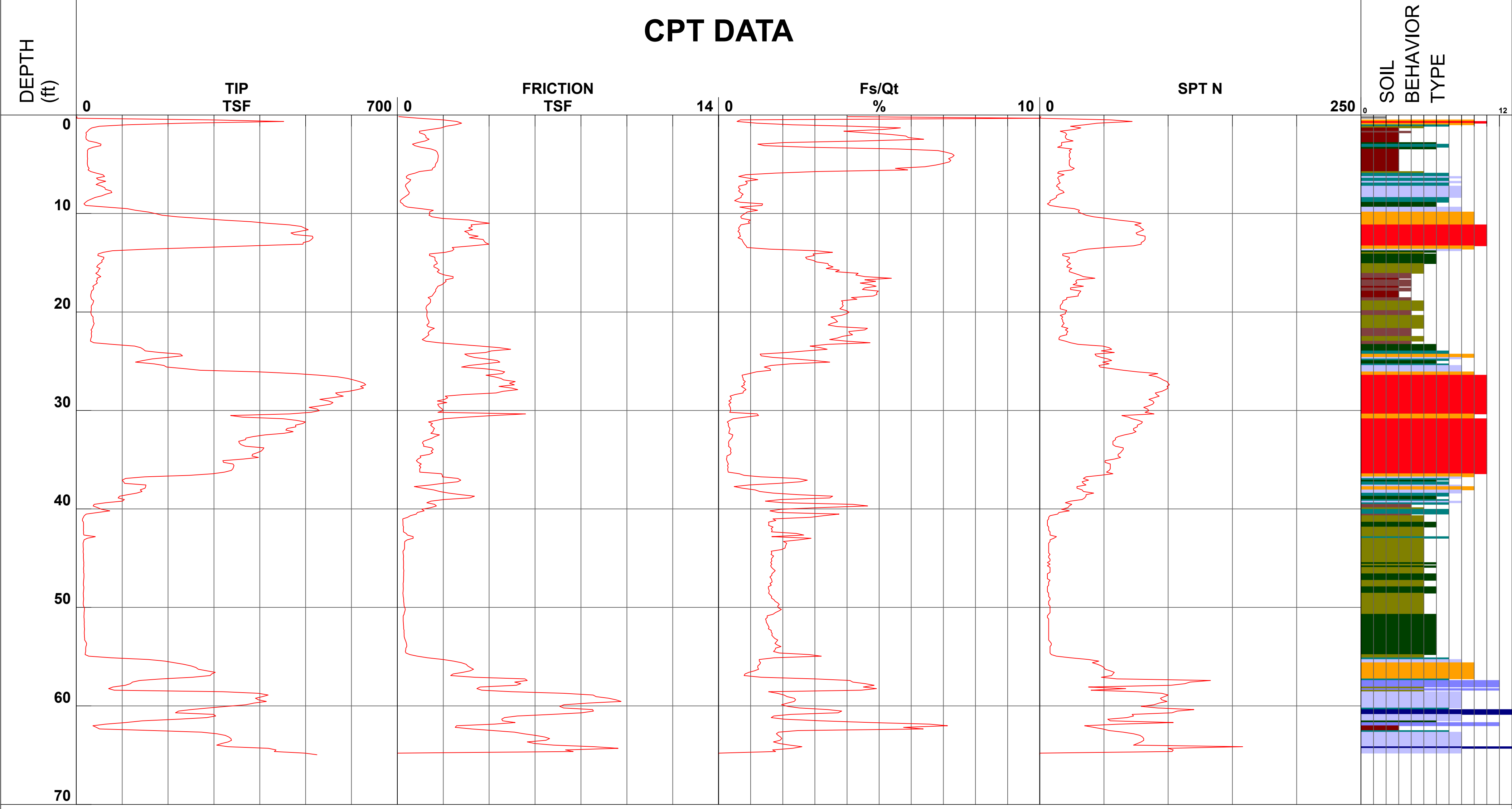
S*Soil behavior type and SPT based on data from UBC-1983

EEI

Project	The Koll Center Residences	Operator	DG-RC	Filename	SDF(197).cpt
Job Number	SHO-72189.4a	Cone Number	DDG1281	GPS	
Hole Number	CPT-07	Date and Time	10/18/2016 12:13:26 PM	Maximum Depth	64.96 ft
EST GW Depth During Test		26.00 ft			

Net Area Ratio .8

CPT DATA



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

Cone Size 10cm squared

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

APPENDIX B
LABORATORY TEST DATA

EXPANSION INDEX TEST

ASTM METHOD D 4829

Sample B-2 @ 1-8 ft.

Moisture Content of Initial Sample		% Saturation of Re-molded Sample		Moisture Content of Final Sample	
Tare No. -	55	Wt. of Soil and Ring (g) -	592.7	Wt. of Soil and Ring (g) -	635.9
Wet Weight and Tare (g) -	149.4	Ring Weight (g) -	198.9	Ring Weight (g) -	198.9
Dry Weight and Tare (g) -	138.2	Wet Weight of Soil (g) -	393.8	Wet Weight of Soil (g) -	437.0
Tare Weight (g) -	50.2	Dry Weight of Soil (g) -	349.3	Dry Weight of Soil (g) -	349.3
Water Loss (g) -	11.2	Volume of Ring (ft^3) -	0.0073	Weight of Water (g) -	87.7
Dry Weight (g) -	88.0	Dry Density (pcf) -	105.5	*Final Moisture (%)*	*25.1*
Initial Moisture (%) -	*12.7*	*Initital Saturation (%) -*	*57.6*	*Final Saturation (%) -*	*113.5*

Expansion Test - UBC (144 PSF)				
	Date	Time	Reading	
Add Weight	7/20/2015	9:20	0.000	
10 Minutes		9:30	0.000	Initial Reading
Add Water		10:30	0.065	
		1:00	0.095	
	7/21/2015	5:23	0.099	Final Reading

EImeasured	=	99
EI_{50}	**=**	**107**

Expansion Index, EI_{50}	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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Client: Shopoff Land Fund II, LP

Job Name; The Koll Center Residences

Job Number: SHO-72189.4a

Date: 7/23/15

Sample Number: B-2

Location: 1-8 ft.

Soil Description: Brown Silty Clay

Tested by: BD

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	SHO-72189.4a
Client:	Shopoff Land Fund II, LP
Date:	7/21/15
Sample Data	
Sample:	B-1 @ 20 ft.
Remolded:	Natural
Remarks:	Soaked Before Placing in Shear Box
Soil Description:	Yellow Brown Sand

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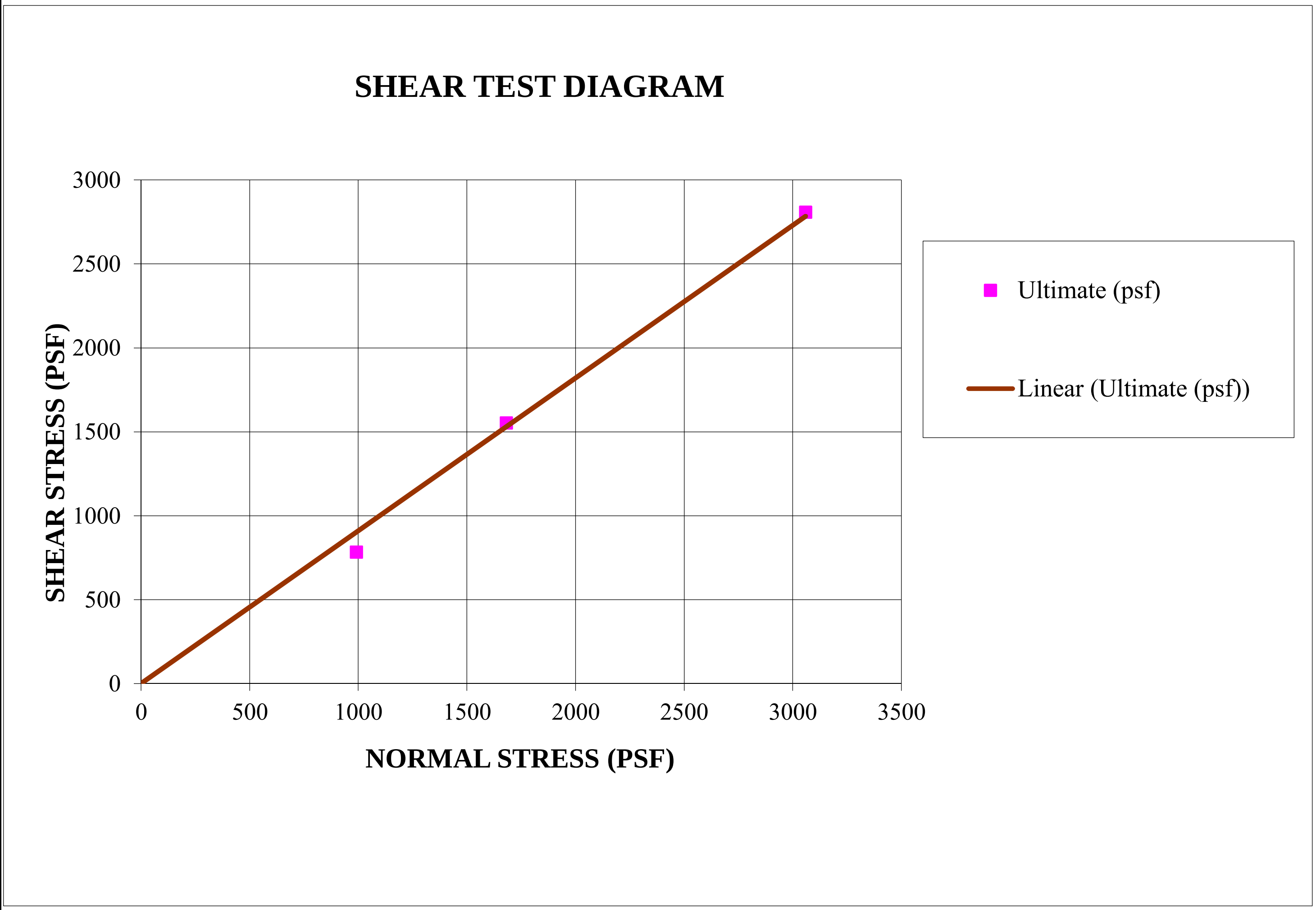

Test Results

	Phi		Cohesion	
Ultimate (psf)	44	degrees	0	psf

Average Initial Moisture	2.8%
Average Dry Density	123.3 pcf
Average Final Moisture	3.7%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	SHO-72189.4a
Client:	Shopoff Land Fund II, LP
Date:	7/21/15
Sample Data	
Sample:	B-1 @ 7.5 ft.
Remolded:	Natural
Remarks:	Soaked Before Placing in Shear Box
Soil Description:	Reddish Brown Sand with Silt

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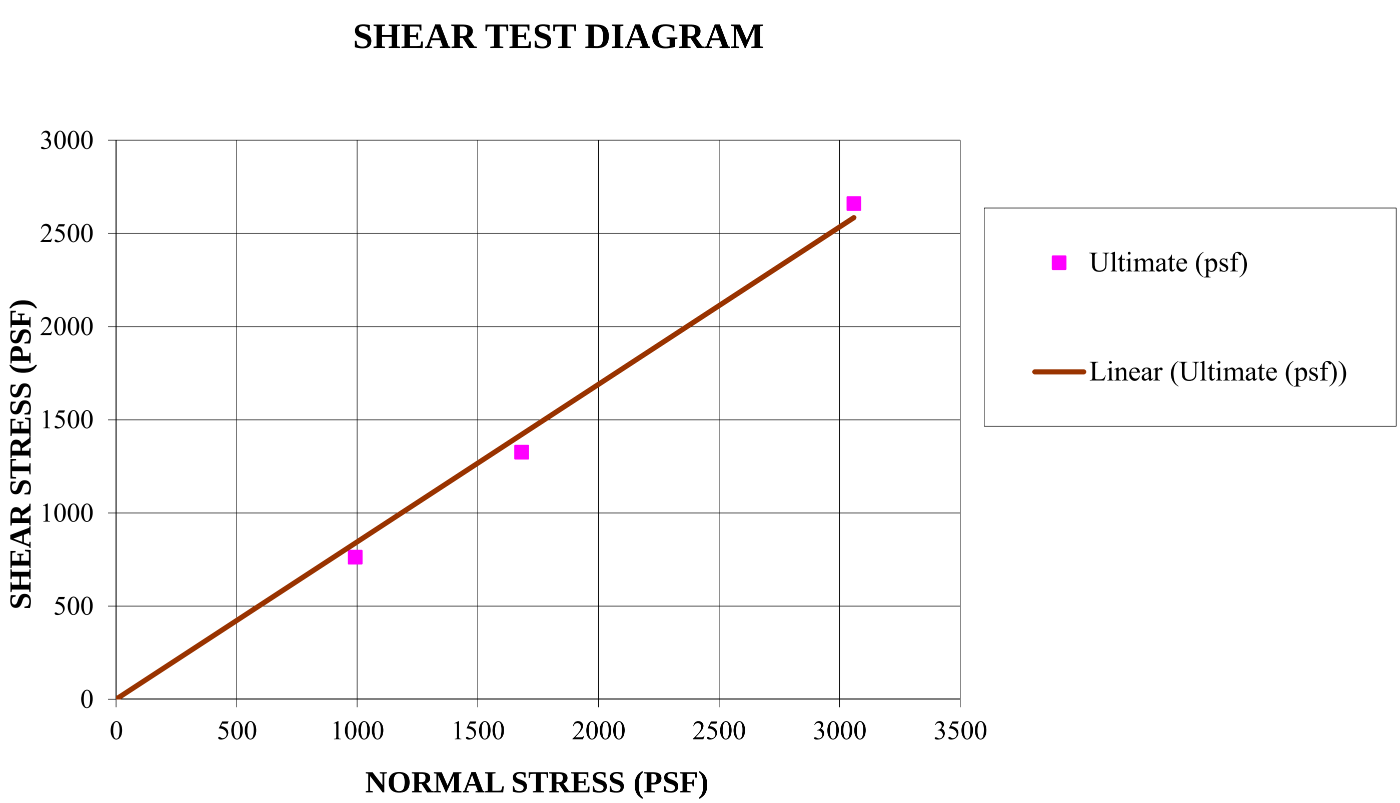

Test Results

	Phi		Cohesion	
Ultimate (psf)	43	degrees	0	psf

Average Initial Moisture	2.2%
Average Dry Density	102.7 pcf
Average Final Moisture	18.0%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	SHO-72189.4a
Client:	Shopoff Land Fund II, LP
Date:	7/21/15
Sample Data	
Sample:	B-2 @ 30 ft.
Remolded:	Natural
Remarks:	Soaked Before Placing in Shear Box
Soil Description:	Light Brown Sand

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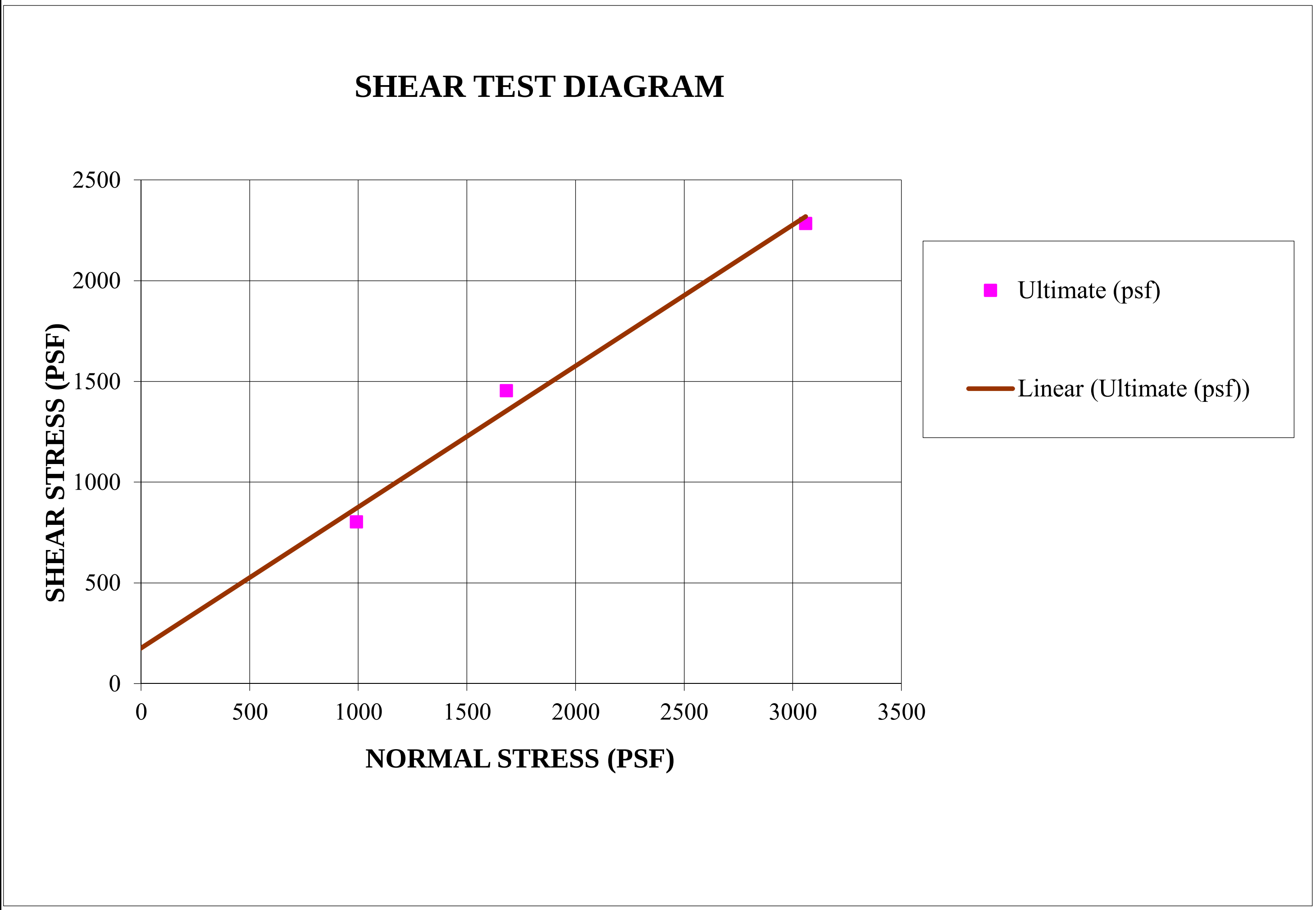

Test Results

	Phi		Cohesion	
Ultimate (psf)	35	degrees	176	psf

Average Initial Moisture	20.5%
Average Dry Density	103.0 pcf
Average Final Moisture	21.2%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	SHO-72189.4a
Client:	Shopoff Land Fund II, LP
Date:	7/21/15
Sample Data	
Sample:	B-2 @ 7.5 ft.
Remolded:	Natural
Remarks:	Soaked Before Placing in Shear Box
Soil Description:	Lt. Brn. Silty Clay

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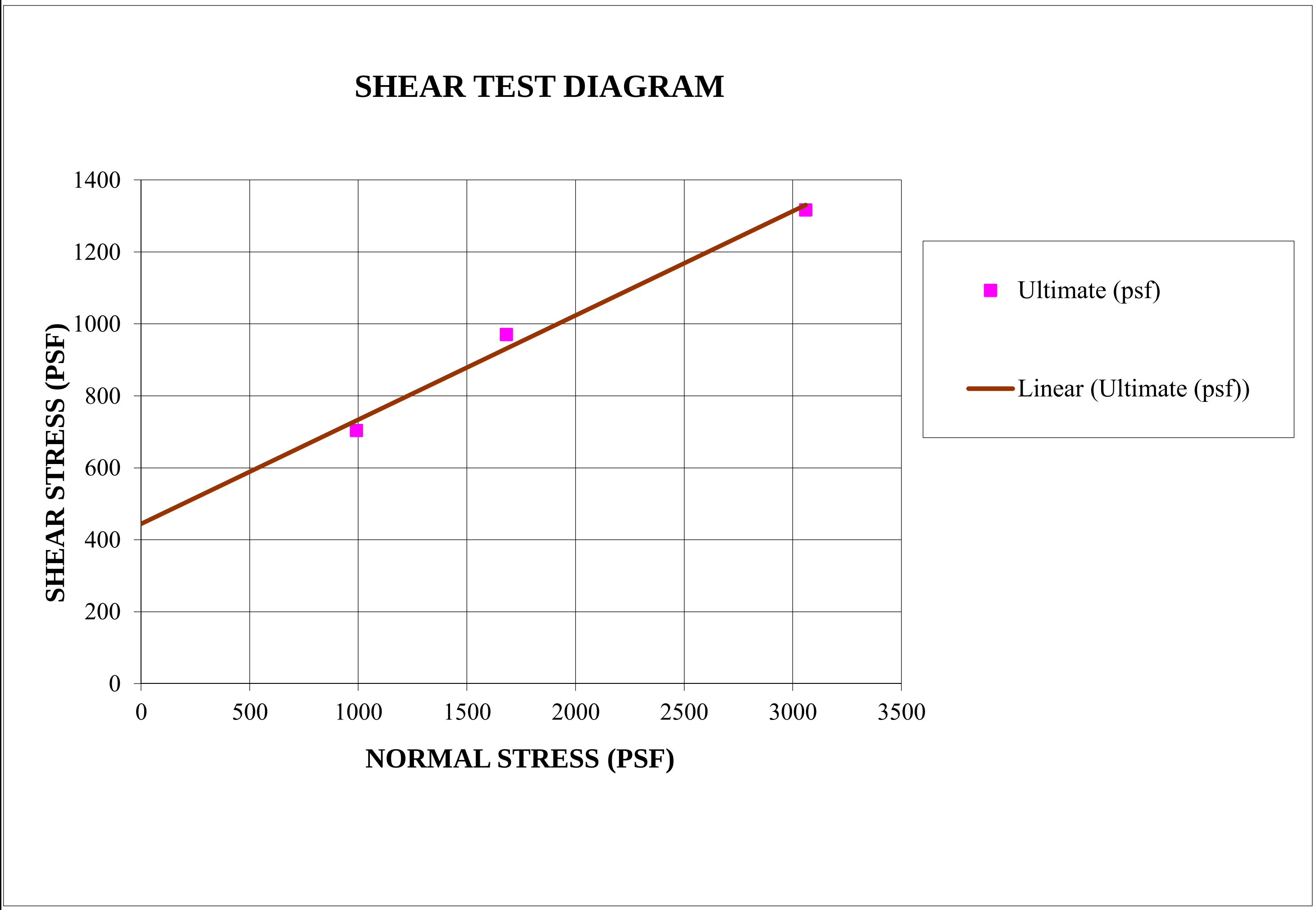

Test Results

	Phi		Cohesion	
Ultimate (psf)	16	degrees	444	psf

Average Initial Moisture	22.3%
Average Dry Density	99.6 pcf
Average Final Moisture	28.8%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	SHO-72189.4a
Client:	Shopoff Land Fund II, LP
Date:	7/21/15
Sample Data	
Sample:	B-4 @ 2.5 ft.
Remolded:	Natural
Remarks:	Soaked Before Placing in Shear Box
Soil Description:	Reddish Brown Clayey Sand

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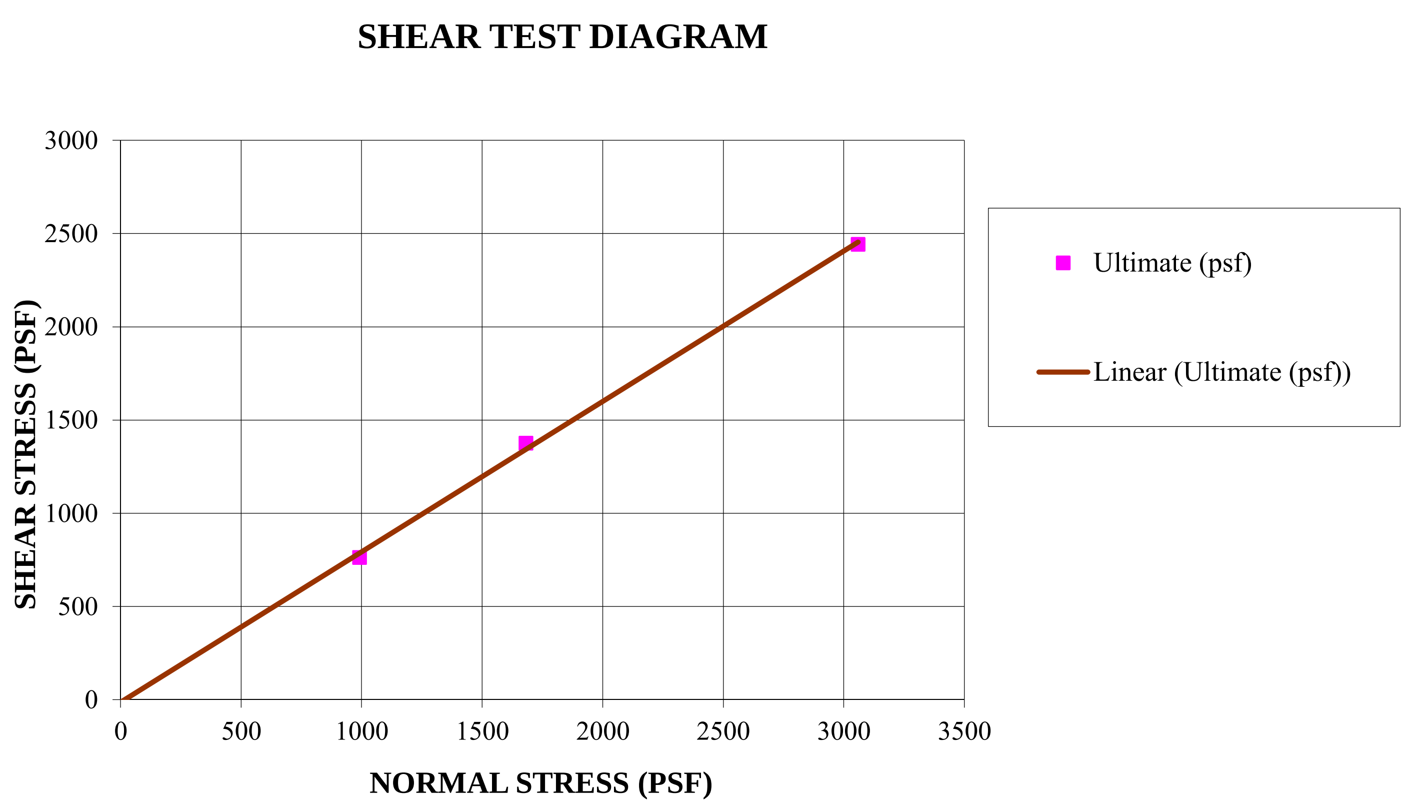

Test Results

	Phi		Cohesion	
Ultimate (psf)	39	degrees	0	psf

Average Initial Moisture	8.2%
Average Dry Density	114.0 pcf
Average Final Moisture	14.1%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	SHO-72189.4a
Client:	Shopoff Land Fund II, LP
Date:	7/21/15
Sample Data	
Sample:	B-4 @ 20 ft.
Remolded:	Natural
Remarks:	Soaked Before Placing in Shear Box
Soil Description:	Light Reddish Brown Sand

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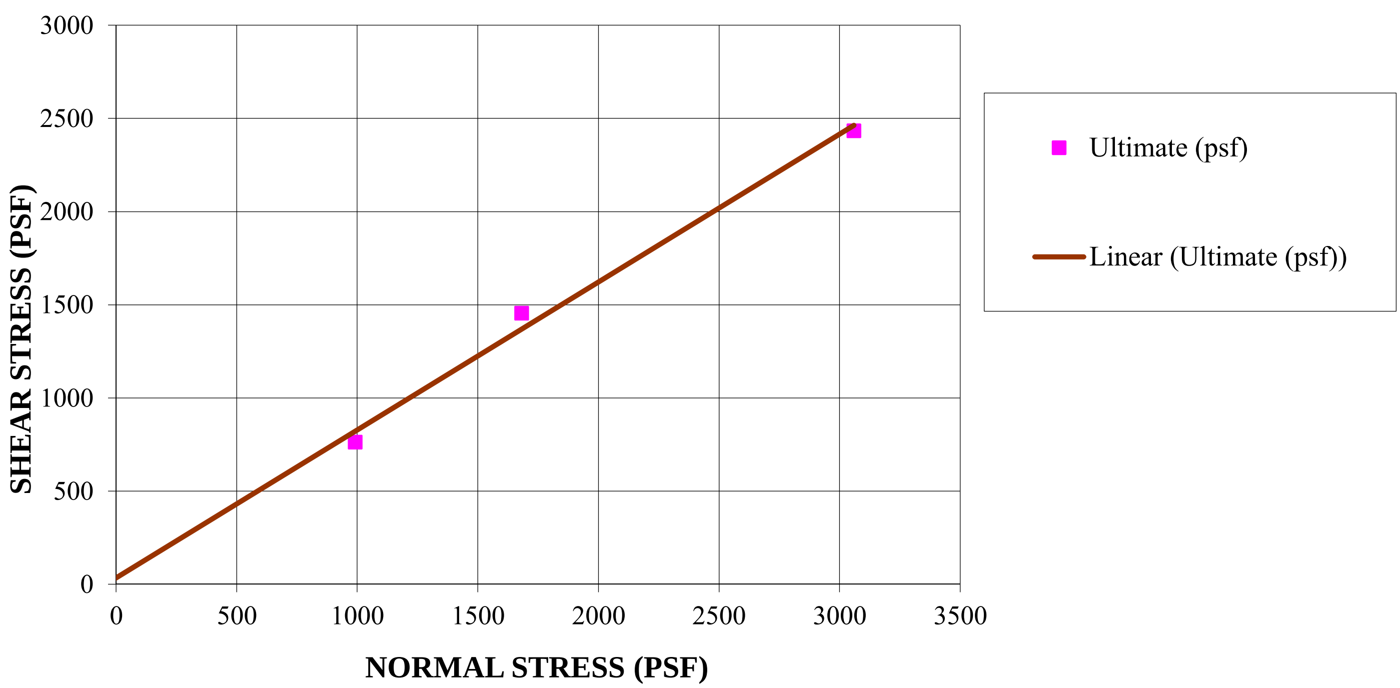
SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Ultimate (psf)	38	degrees	34	psf

Average Initial Moisture	16.9%
Average Dry Density	106.0 pcf
Average Final Moisture	17.5%

DIRECT SHEAR TEST (ASTM D3080)

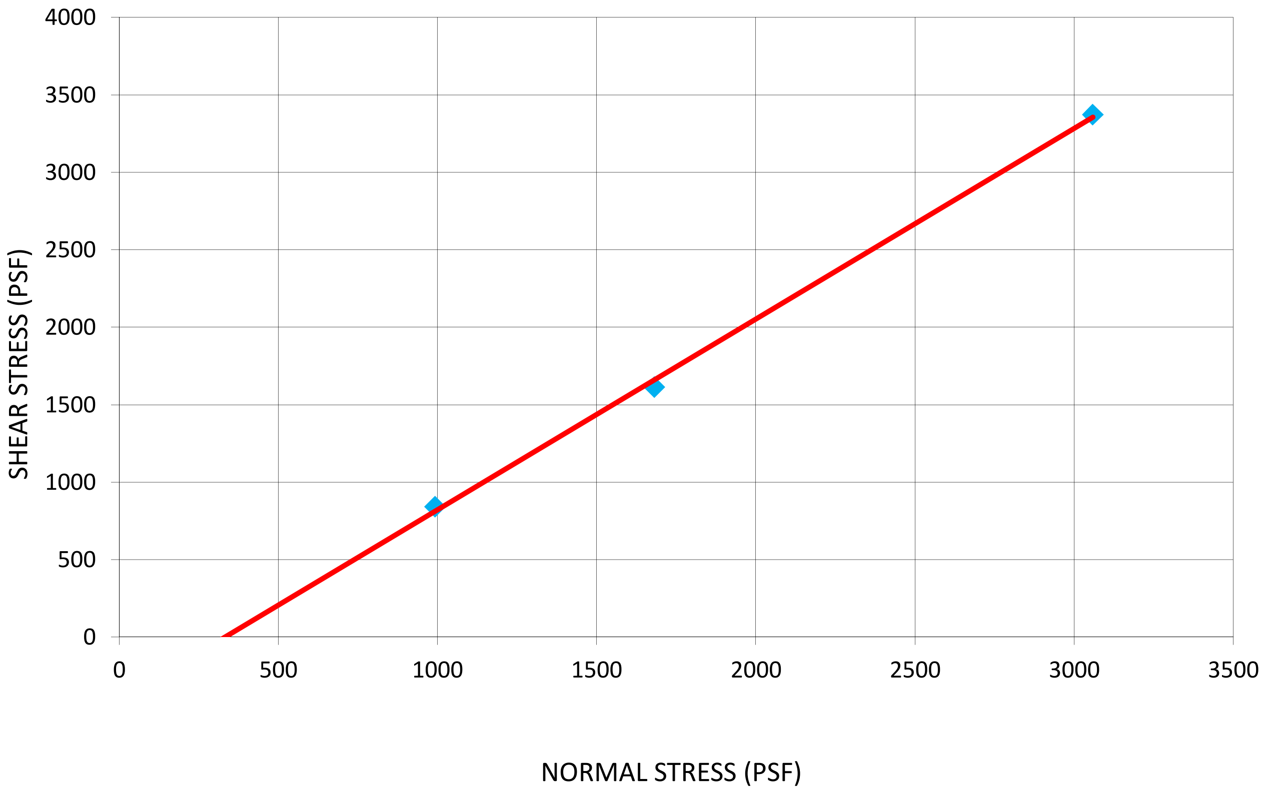
B-8 @ 10 ft.

Sample Data	
Remolded:	Natural
Remarks:	Sample inundated prior to testing
Soil Description:	Brown Sand (SP)

Test Results		
Average Initial Moisture =	2.1	%
Average Dry Density =	104.5	pcf
Average Final Moisture =	15.1	%

Peak Strength	ϕ = 51 deg.	c = -410 psf

SHEAR TEST DIAGRAM



Client:	Shopoff Land Fund II, LP
Project Name:	The Koll Center Residences
Project No.:	SHO-72189.4a
Date:	10/22/16
Boring/Sample No:	B-8
Depth/Location:	10 ft.
Soil Description:	Brown Sand (SP)
Tested by:	B D

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DIRECT SHEAR TEST (ASTM D3080)

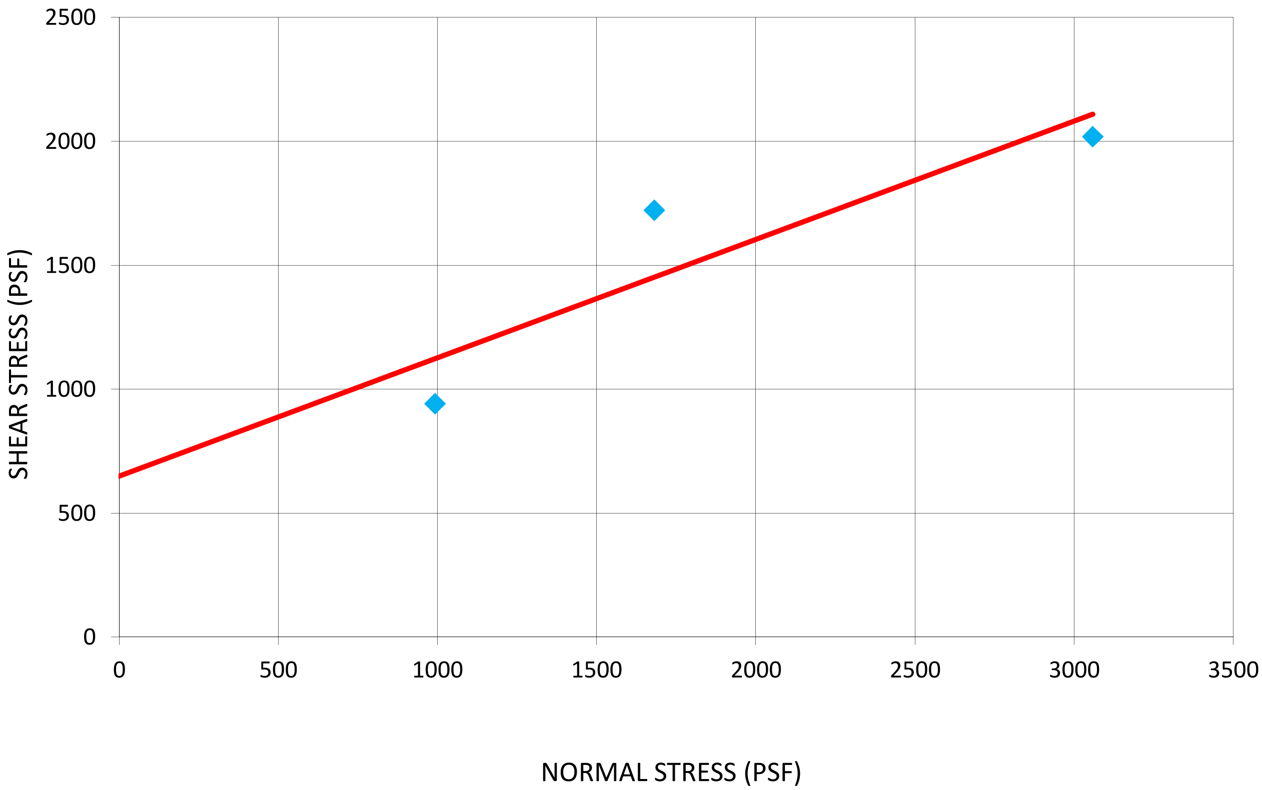
B-8	@	20 ft.

Sample Data	
Remolded:	Natural
Remarks:	Sample inundated prior to testing
Soil Description:	Grey/Brown Clay CL

Test Results		
Average Initial Moisture =	32.8	%
Average Dry Density =	91.1	pcf
Average Final Moisture =	36.2	%

Peak Strength	ϕ = 26 deg.	c = 649 psf

SHEAR TEST DIAGRAM



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Client:	Shopoff Land Fund II, LP
Project Name:	The Koll Center Residences
Project No.:	SHO-72189.4a
Date:	10/15/16
Boring/Sample No:	B-8
Depth/Location:	20 ft.
Soil Description:	Grey/Brown Clay CL
Tested by:	B D

DIRECT SHEAR TEST (ASTM D3080)

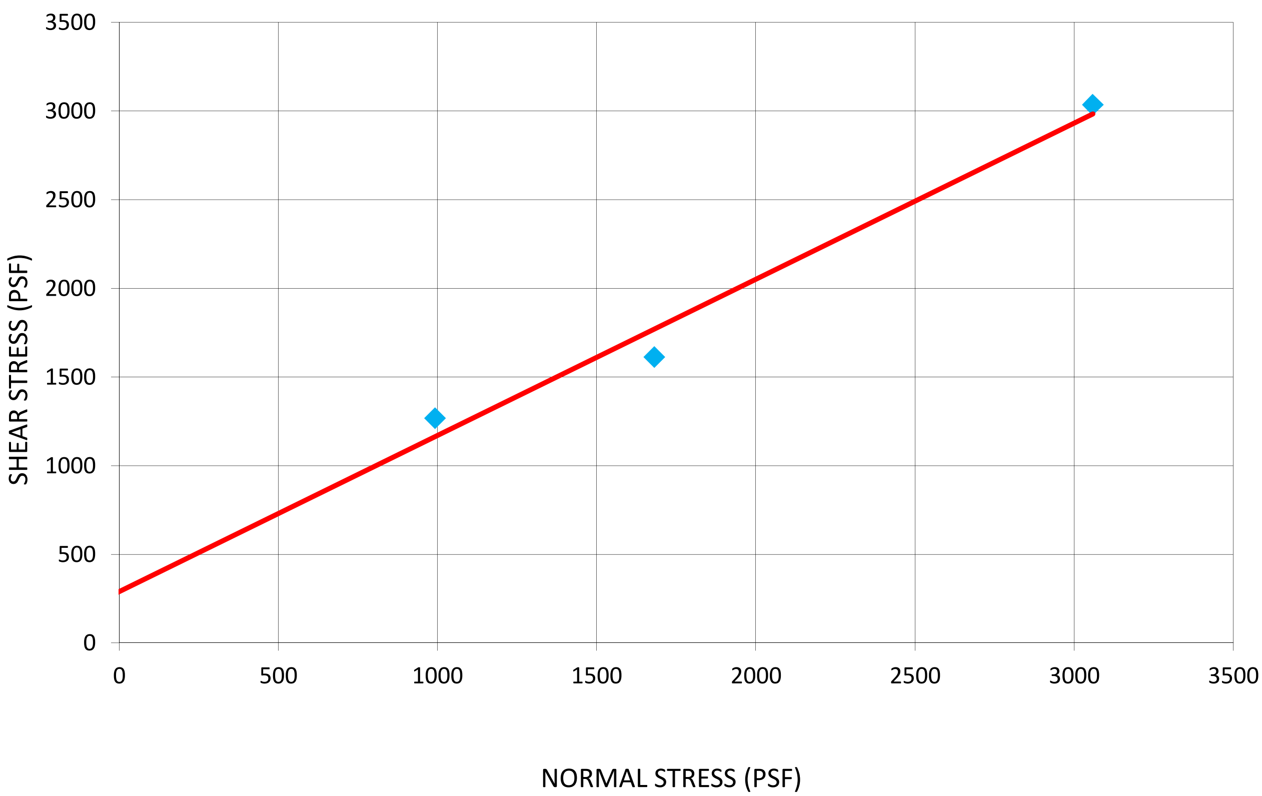
B-8 @ 30 ft.

Sample Data	
Remolded:	Natural
Remarks:	Sample inundated prior to testing
Soil Description:	Brown Sand

Test Results		
Average Initial Moisture =	7.2	%
Average Dry Density =	120.2	pcf
Average Final Moisture =	8.4	%

Peak Strength	ϕ = 41 deg.	c = 289 psf

SHEAR TEST DIAGRAM



Client:	Shopoff Land Fund II, LP
Project Name:	The Koll Center Residences
Project No.:	SHO-72189.4a
Date:	10/26/16
Boring/Sample No:	B-8
Depth/Location:	30 ft.
Soil Description:	Brown Sand
Tested by:	B D

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DIRECT SHEAR TEST (ASTM D3080)

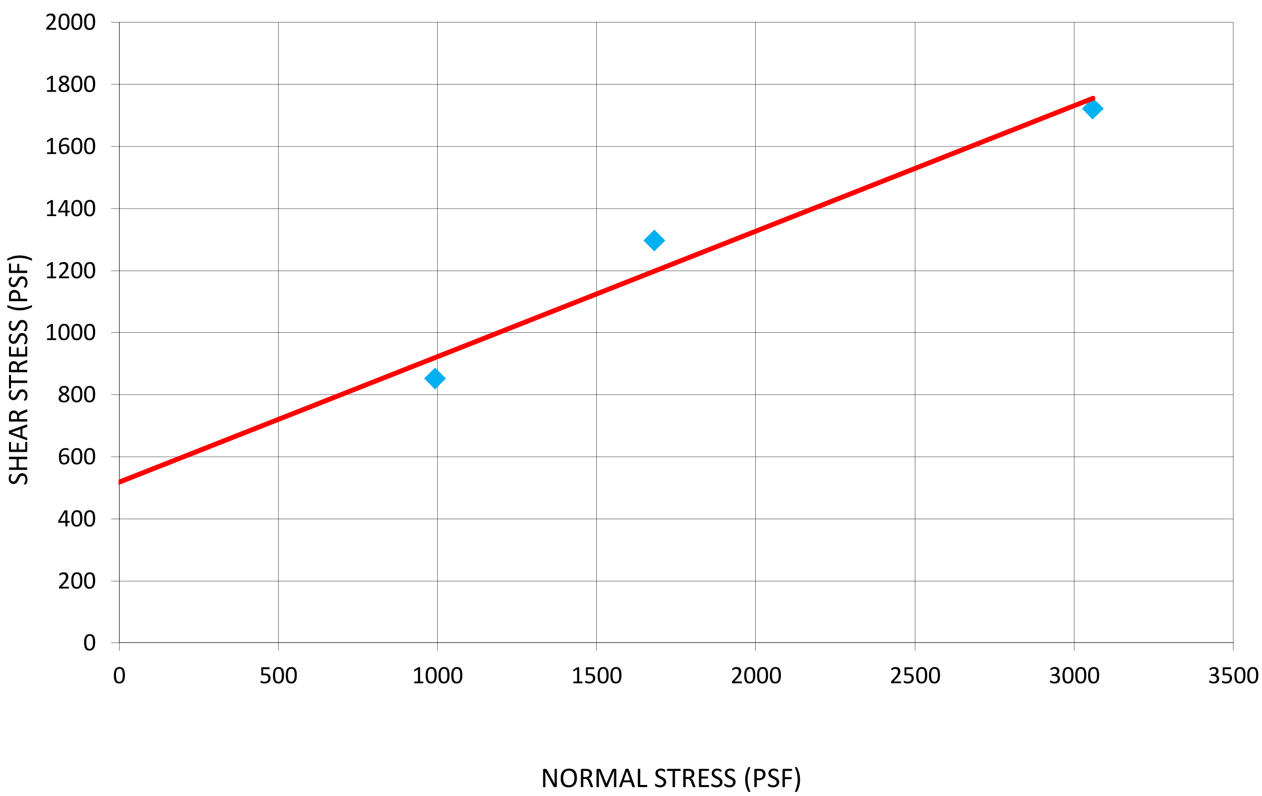
B-8 @ 40 ft.

Sample Data	
Remolded:	Natural
Remarks:	Sample inundated prior to testing
Soil Description:	Dk. Grey Clay (CL)

Test Results		
Average Initial Moisture =	31.6	%
Average Dry Density =	84.2	pcf
Average Final Moisture =	34.4	%

Peak Strength	ϕ = 22 deg.	c = 518 psf

SHEAR TEST DIAGRAM



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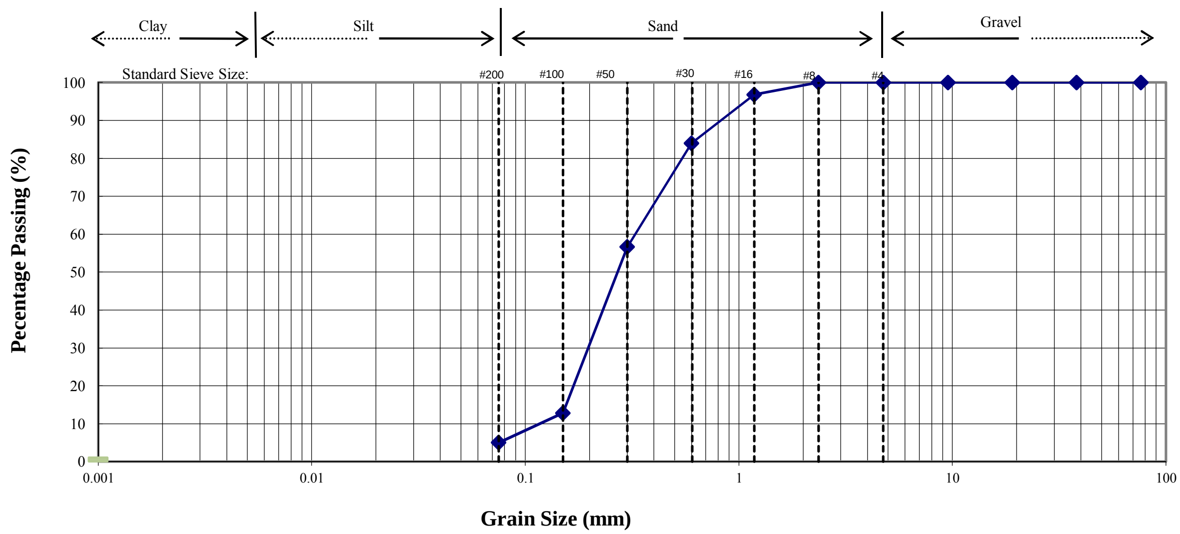
Client:	Shopoff Land Fund II, LP
Project Name:	The Koll Center Residences
Project No.:	SHO-72189.4a
Date:	10/29/16
Boring/Sample No:	B-8
Depth/Location:	40 ft.
Soil Description:	Dk. Grey Clay (CL)
Tested by:	B D

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 15 ft.	D10 (mm)	0.123
Total Weight (g)	115.1	D30 (mm)	0.21
Dry Weight (g)	110.0	D60 (mm)	0.34
Wet Sieve Weight (g)	104.5	Cu	2.74
Initial Moisture (%)	4.6	Cc	1.05

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 15 feet is classified as Poorly Graded Sand (SP)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	0.0	0.0	100.0
#8	2.36	0.0	0.0	100.0
#16	1.18	3.5	3.2	96.8
#30	0.6	17.6	16.0	84.0
#50	0.3	47.7	43.4	56.6
#100	0.15	95.9	87.2	12.8
#200	0.075	104.5	95.0	5.0

Client: Shopoff Land Fund II, LP

Project Name: The Koll Center Residences

Job Number: SHO-72189.4a

Date: 7/21/15

Boring Number: B-1

Depth: 15 ft.

Soil Description: Yellow Brown Poorly Graded Sand SP

Tested by: B D

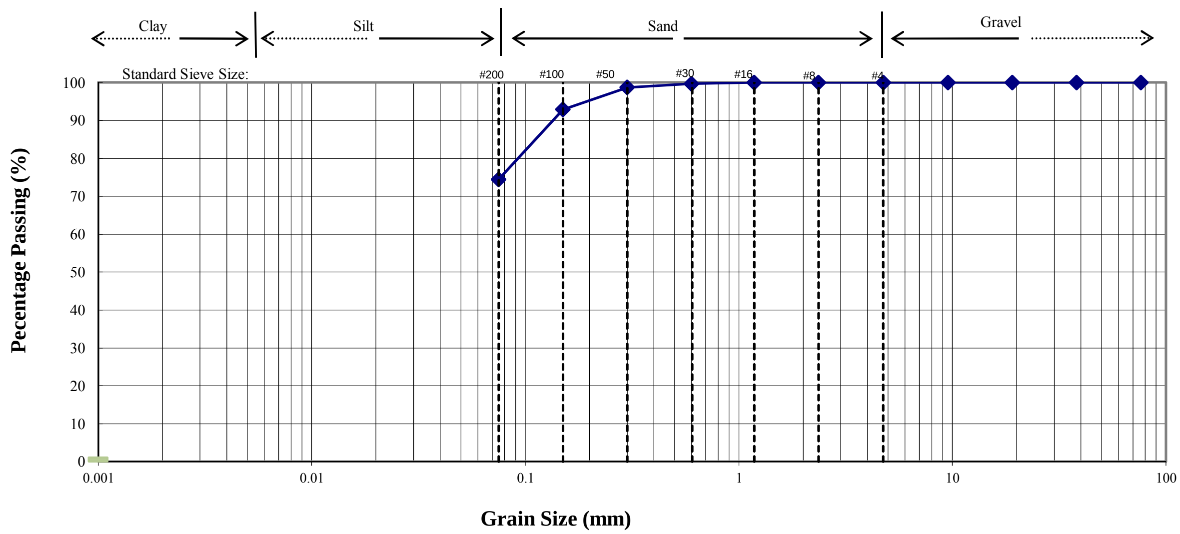
EEI
Geotechnical & Environmental Solutions

2195 Faraday Avenue, Suite K, Carlsbad CA 92008

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 30 ft.		D_{10} (mm)	N/A
Total Weight (g)	115.1		D_{30} (mm)	N/A
Dry Weight (g)	94.4		D_{60} (mm)	N/A
Wet Sieve Weight (g)	24.1		C_u	N/A
Initial Moisture (%)	21.9		C_c	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 30 feet is classified as Sandy Clay (CL)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	0.0	0.0	100.0
#8	2.36	0.0	0.0	100.0
#16	1.18	0.0	0.0	100.0
#30	0.6	0.3	0.3	99.7
#50	0.3	1.2	1.3	98.7
#100	0.15	6.7	7.1	92.9
#200	0.075	24.1	25.5	74.5

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Client: Shopoff Land Fund II, LP

Project Name: The Koll Center Residences

Job Number: SHO-72189.4a

Date: 7/21/15

Boring Number: B-1

Depth: 30 ft.

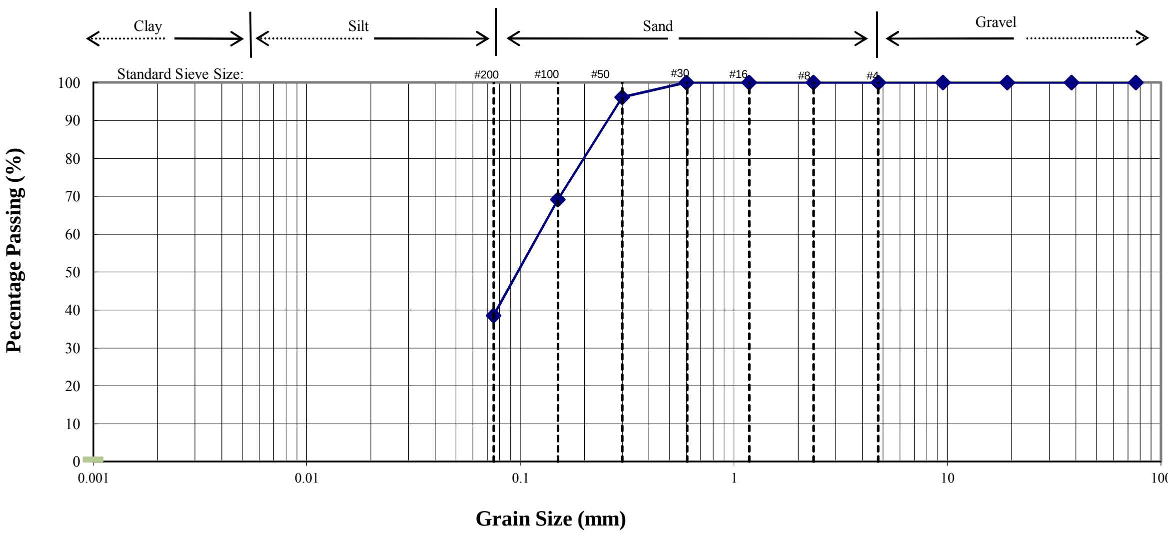
Soil Description: Grey Sandy Clay CL

Tested by: B D

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 35 ft.	D10 (mm)	N/A
Total Weight (g)	114.9	D30 (mm)	N/A
Dry Weight (g)	93.7	D60 (mm)	0.13
Wet Sieve Weight (g)	57.6	Cu	N/A
Initial Moisture (%)	22.6	Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 35 feet is classified as Clayey Sand (SC)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	0.0	0.0	100.0
#8	2.36	0.0	0.0	100.0
#16	1.18	0.0	0.0	100.0
#30	0.6	0.0	0.0	100.0
#50	0.3	3.6	3.8	96.2
#100	0.15	28.9	30.8	69.2
#200	0.075	57.6	61.5	38.5

Client: Shopoff Land Fund II, LP

Project Name: The Koll Center Residences

Job Number: SHO-72189.4a

Date: 7/21/15

Boring Number: B-1

Depth: 35 ft.

Soil Description: Grey Clayey Sand SC

Tested by: B D

EEI Geotechnical & Environmental Solutions

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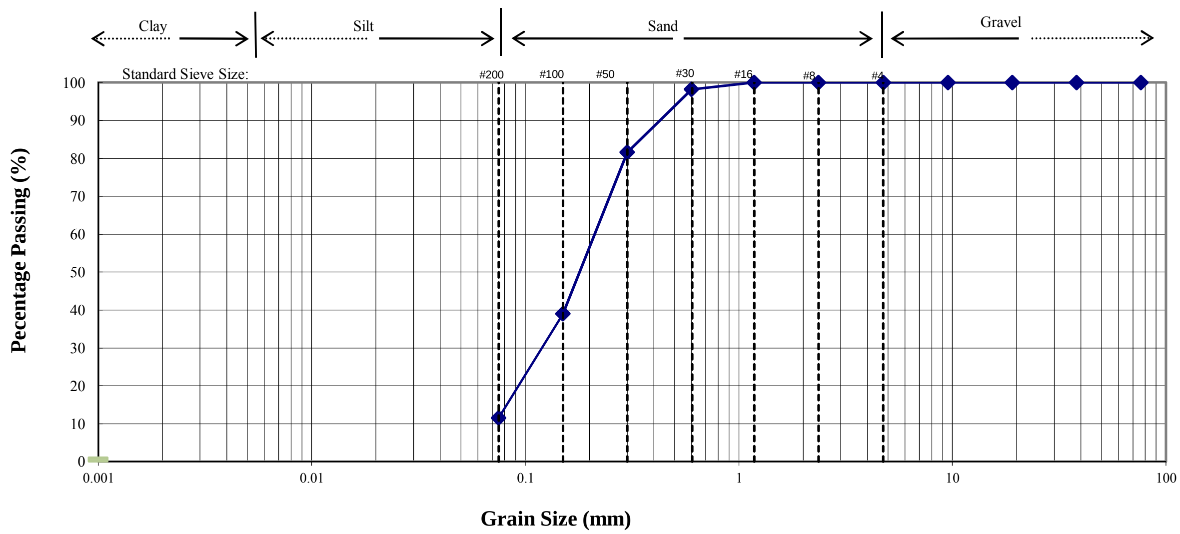
PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 45 ft.	D10 (mm)	N/A
Total Weight (g)	116.5	D30 (mm)	0.13
Dry Weight (g)	95.1	D60 (mm)	0.22
Wet Sieve Weight (g)	84.1	Cu	N/A
Initial Moisture (%)	22.5	Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 45 feet is classified as Poorly Graded Sand with Silt (SP-SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	0.0	0.0	100.0
#8	2.36	0.0	0.0	100.0
#16	1.18	0.0	0.0	100.0
#30	0.6	1.7	1.8	98.2
#50	0.3	17.5	18.4	81.6
#100	0.15	58.0	61.0	39.0
#200	0.075	84.1	88.4	11.6

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Client: Shopoff Land Fund II, LP

Project Name: The Koll Center Residences

Job Number: SHO-72189.4a

Date: 7/21/15

Boring Number: B-1

Depth: 45 ft.

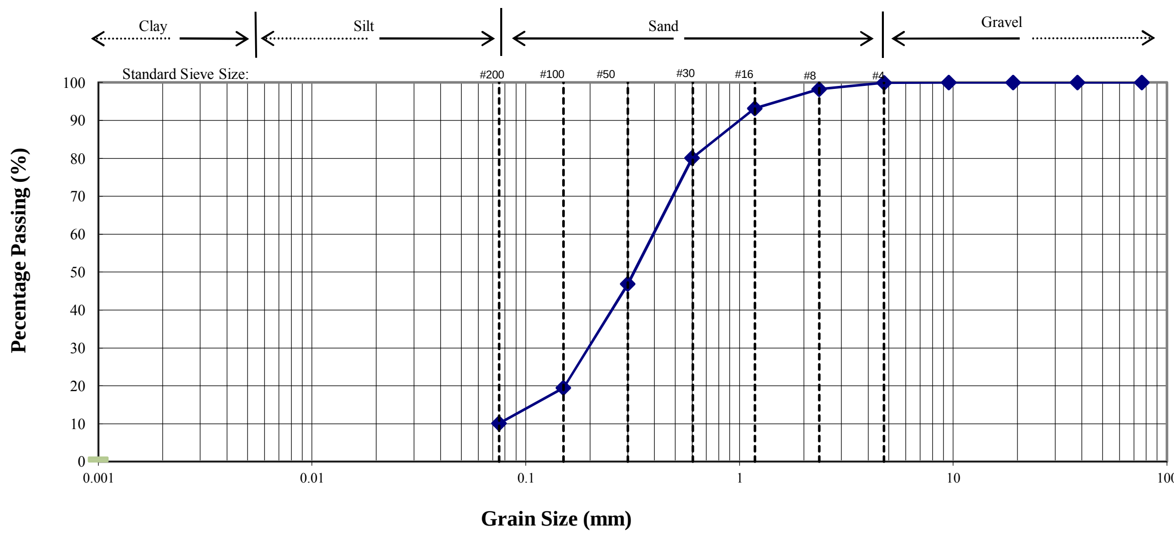
Soil Description: Grey Poorly Graded Sand with Silt SP-SM

Tested by: B D

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B-1 @ 5-10 ft.	D10 (mm)	N/A
Total Weight (g)	140.1	D30 (mm)	0.21
Dry Weight (g)	137.9	D60 (mm)	0.42
Wet Sieve Weight (g)	124.0	Cu	N/A
Initial Moisture (%)	1.6	Cc	N/A

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B-1 at 5-10 feet is classified as Sand with Silt (SP-SM)



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	0.1	0.1	99.9
#8	2.36	2.4	1.7	98.3
#16	1.18	9.4	6.8	93.2
#30	0.6	27.5	19.9	80.1
#50	0.3	73.3	53.2	46.8
#100	0.15	111.1	80.6	19.4
#200	0.075	124.0	89.9	10.1

EEI
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2195 Faraday Avenue, Suite K, Carlsbad CA 92008

Client: Shopoff Land Fund II, LP

Project Name: The Koll Center Residences

Job Number: SHO-72189.4a

Date: 7/21/15

Boring Number: B-1

Depth: 5-10 ft.

Soil Description: Reddish Brown Sand with Silt SP-SM

Tested by: B D

L A B O R A T O R Y   R E P O R T

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C L A R K S O N L A B O R A T O R Y A N D S U P P L Y I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L A N D C O N S U L T I N G C H E M I S T S

Date: July 24, 2015
Purchase Order Number: SHO-72189.4a
Sales Order Number: 27828
Account Number: EEI
To:
--
EEI Environmental Equalizers Inc
2195 Faraday Avenue Suite K
Carlsbad, CA 92008
Attention: Jeff Blake

Laboratory Number: SO5747-1   Customers Phone: 760-431-3747
Fax:

Sample Designation:
--
One soil sample received on 07/22/15 at 1:55pm, taken on 07/21/15 from Job# SHO-72189-4 marked as B-1 5'-10' SM.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 7.4

Water Added (ml)	Resistivity (ohm-cm)
10	13000
5	11000
5	9700
5	9900
5	11000
5	12000

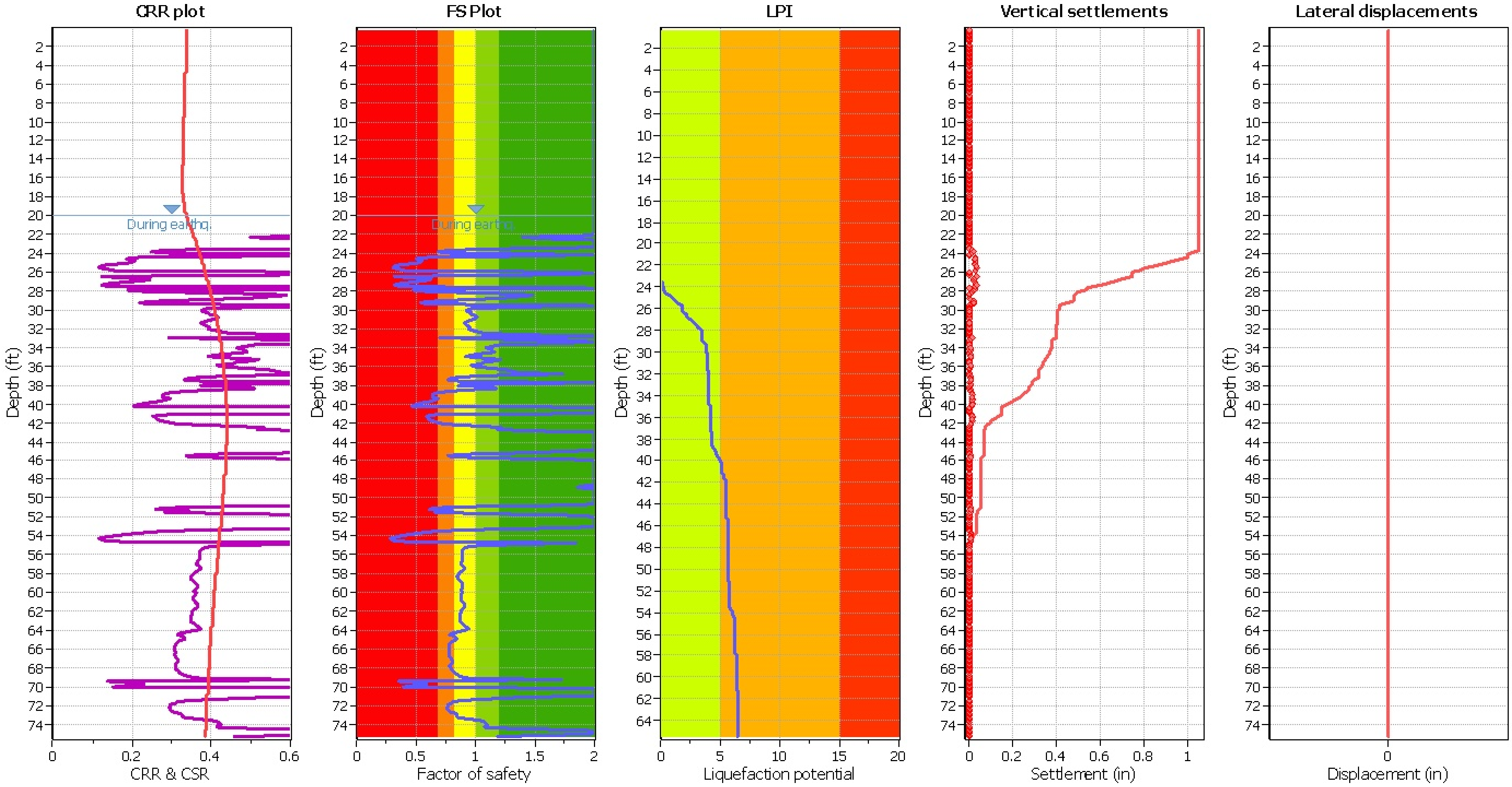
78 years to perforation for a 16 gauge metal culvert.
101 years to perforation for a 14 gauge metal culvert.
140 years to perforation for a 12 gauge metal culvert.
178 years to perforation for a 10 gauge metal culvert.
217 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.003%
Water Soluble Chloride	Calif. Test 422	0.002%

Rosa M. Bernal
RMB/ram

APPENDIX C
LIQUEFACTION ANALYSIS

Liquefaction analysis overall plot



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	20.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.00	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.62	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	20.00 ft	Fill height:	N/A	Limit depth:	N/A

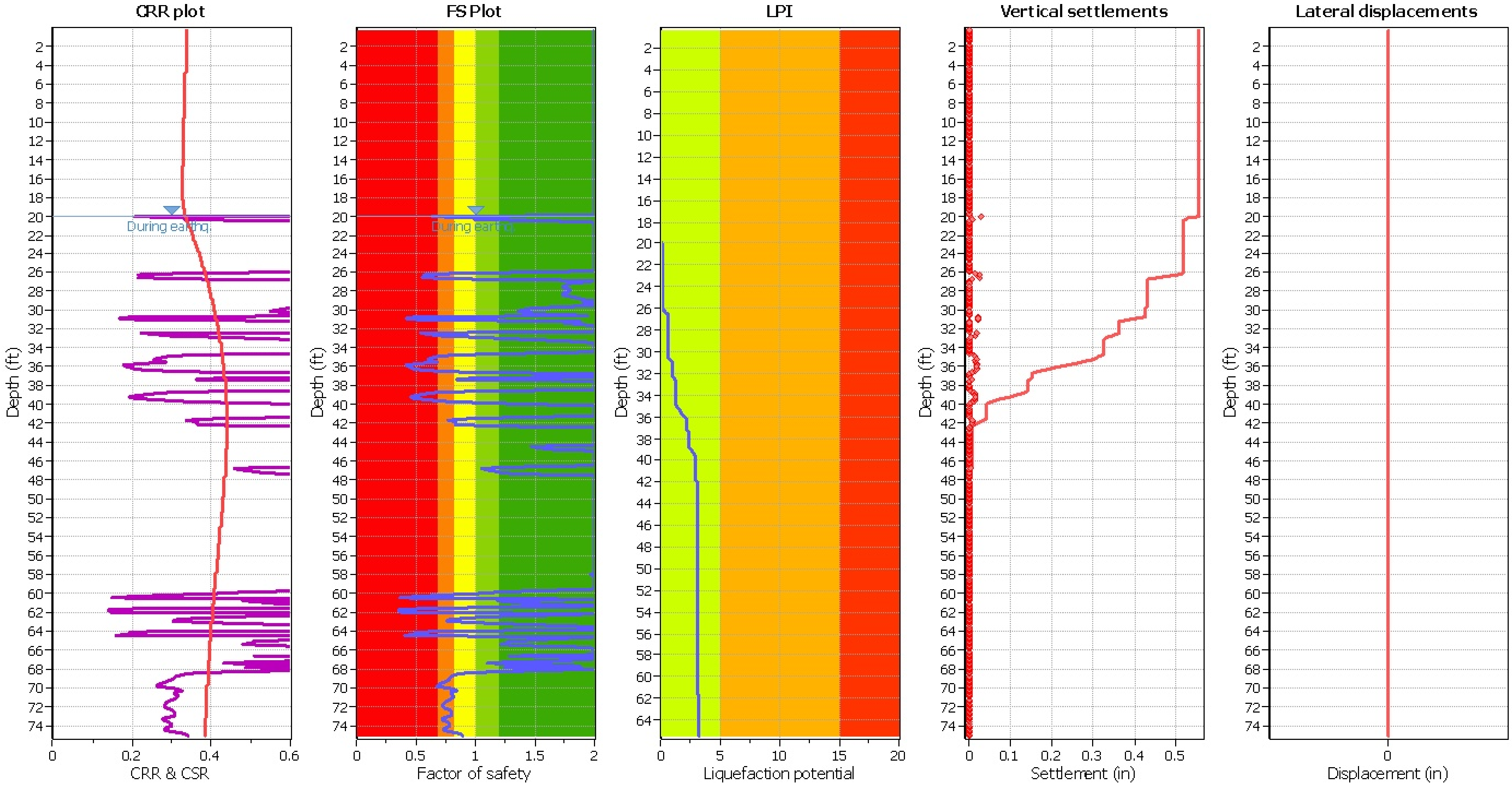
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis overall plot



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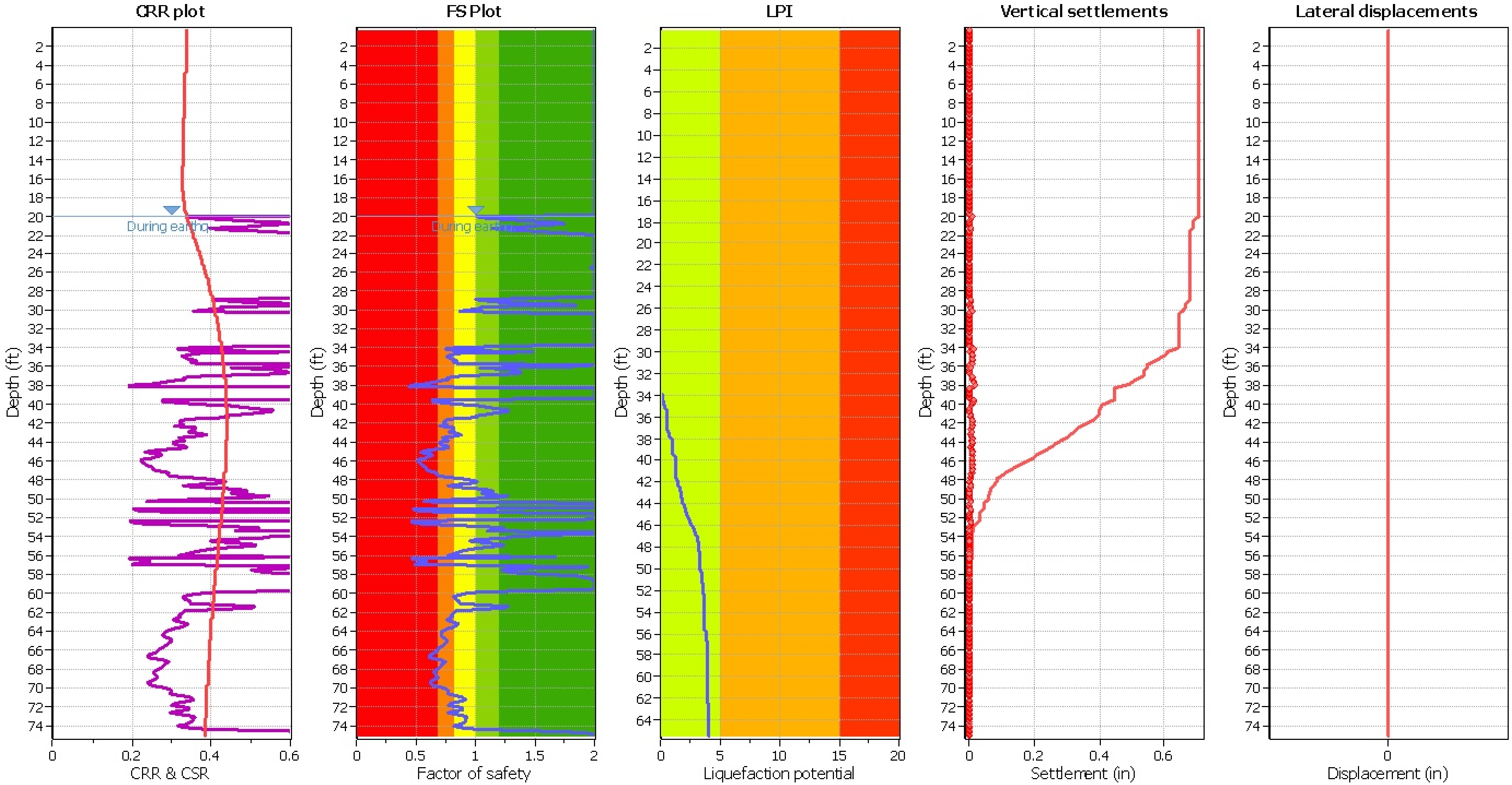

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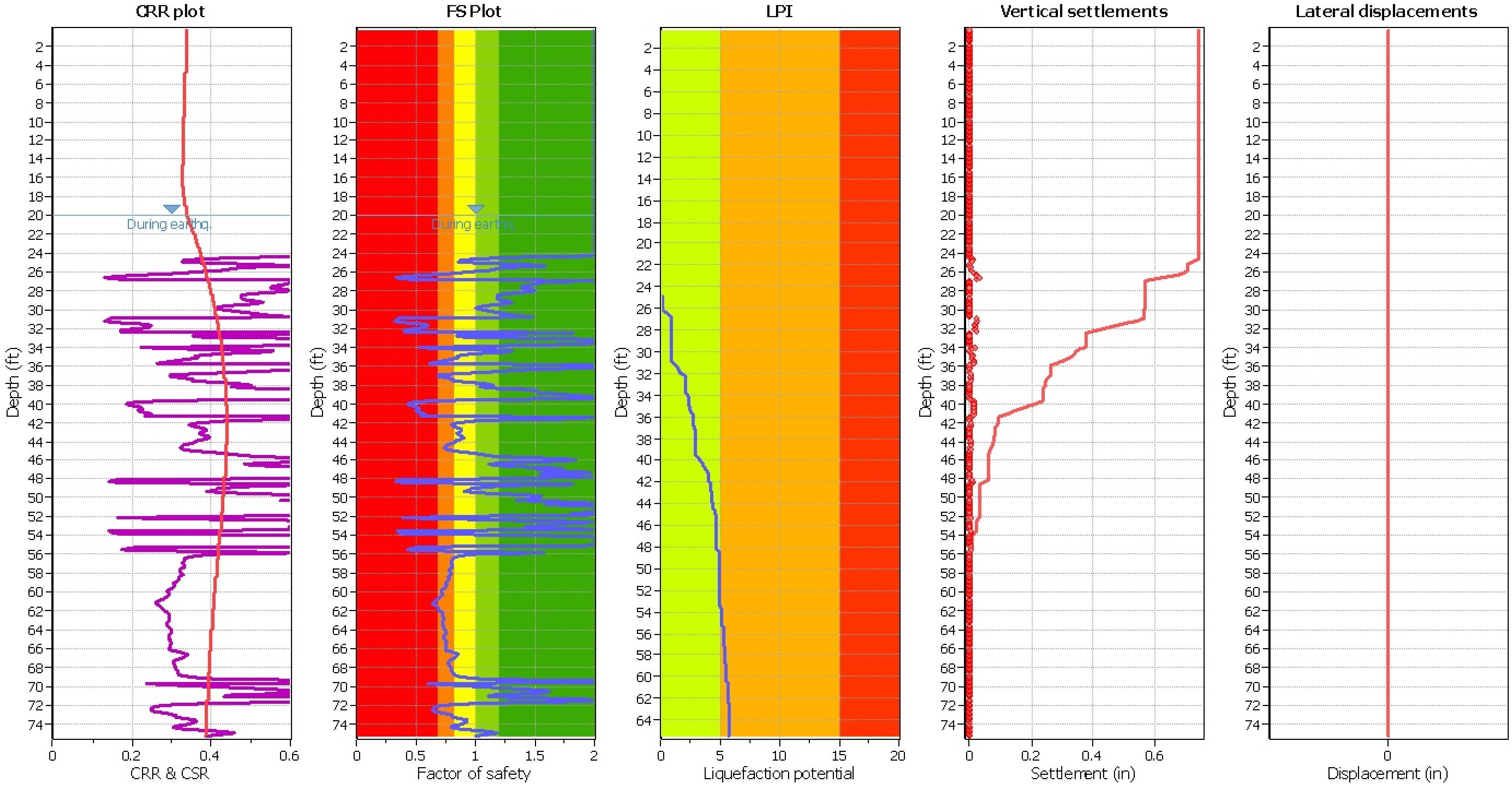

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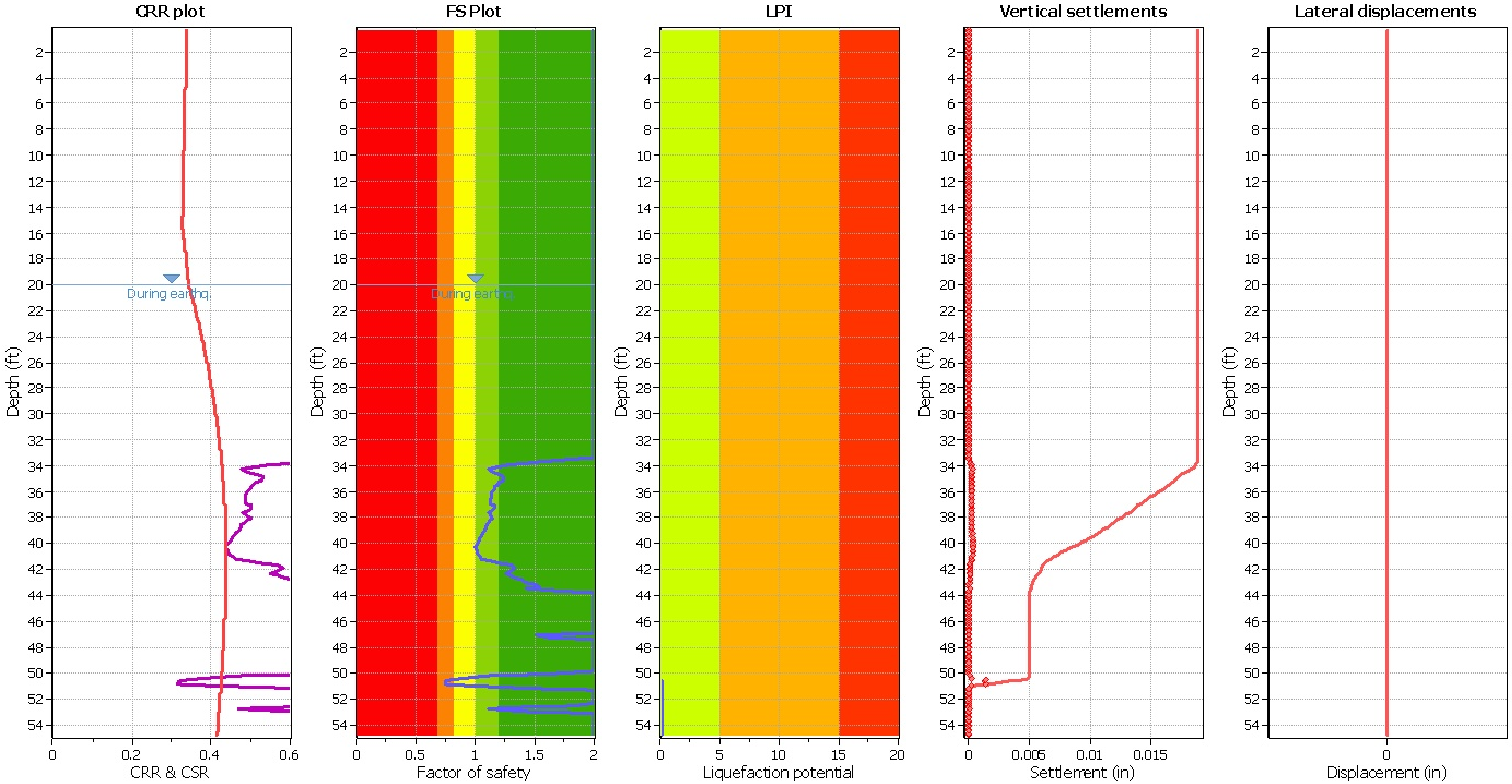

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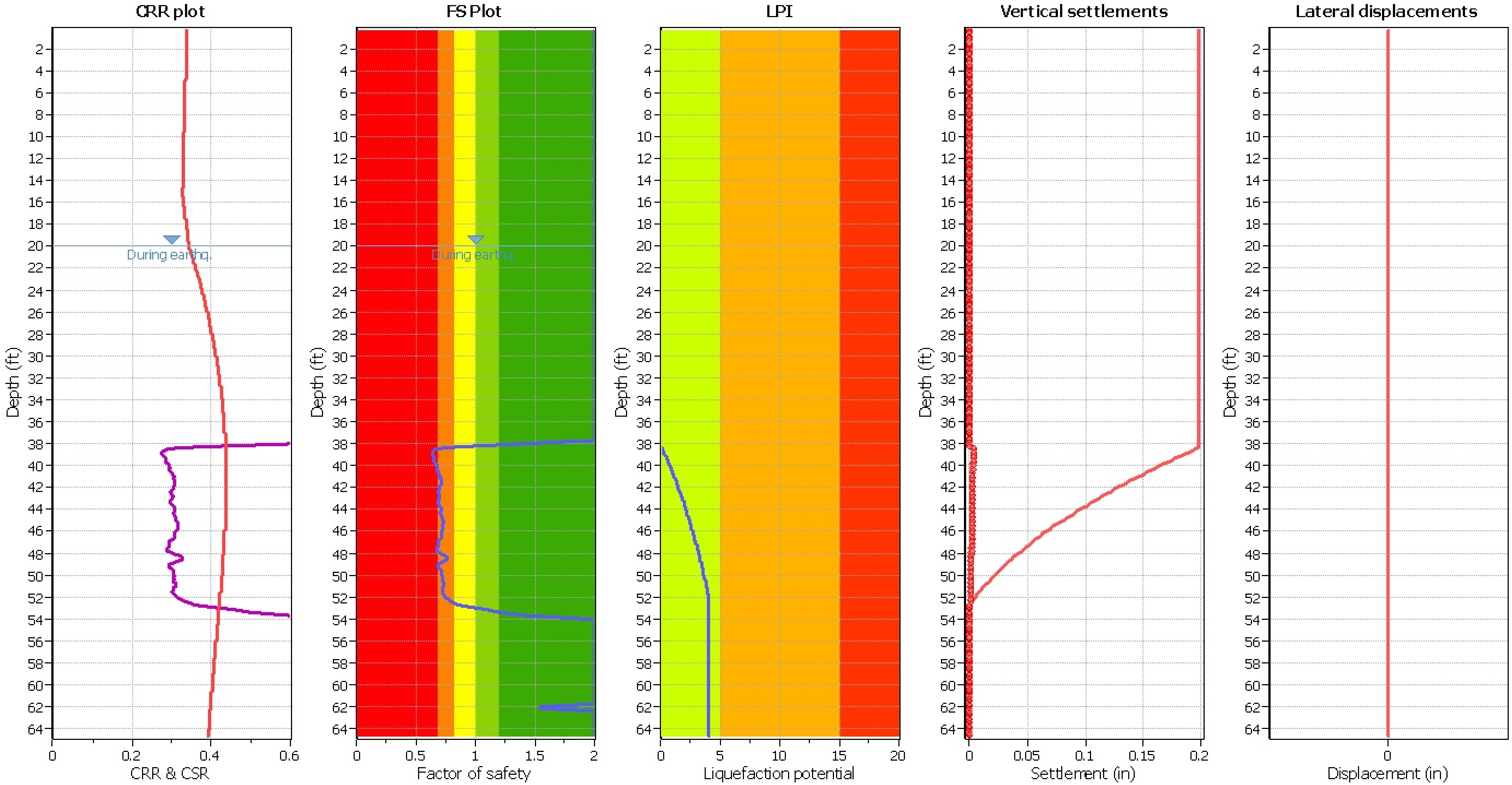

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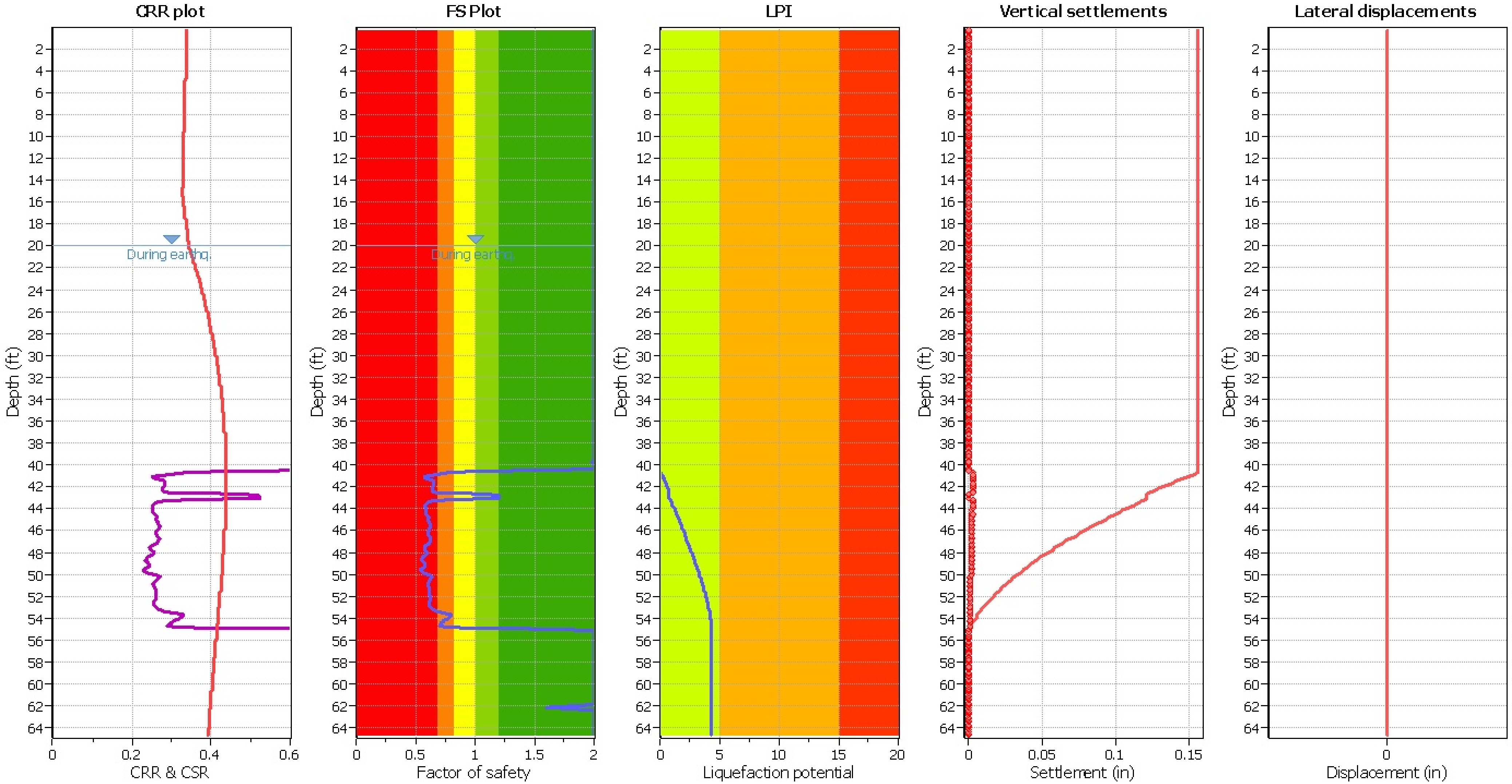

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APPENDIX D
EARTHWORK AND GRADING GUIDELINES

Geotechnical & Environmental Solutions

EARTHWORK AND GRADING GUIDELINES

GENERAL

These guidelines present general procedures and recommendations for earthwork and grading as required on the approved grading plans, including preparation of areas to be filled, placement of fill and installation of subdrains and excavations. The recommendations contained in the geotechnical report are applicable to each specific project, are part of the earthwork and grading guidelines and would supersede the provisions contained hereafter in the case of conflict. Observations and/or testing performed by the consultant during the course of grading may result in revised recommendations which could supersede these guidelines or the recommendations contained in the geotechnical report. Figures A through O are provided at the back of this appendix, exhibiting generalized cross sections relating to these guidelines.

The contractor is responsible for the satisfactory completion of all earthworks in accordance with provisions of the project plans and specifications. The project soil engineer and engineering geologist (geotechnical consultant) or their representatives should provide observation and testing services, and geotechnical consultation throughout the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

Prior to the commencement of grading, a qualified geotechnical consultant (a soil engineer and engineering geologist) should be employed for the purpose of observing earthwork procedures and testing the fills for conformance with the recommendations of the geotechnical report, the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that determination may be made that the work is being completed as specified. It is the responsibility of the contractor to assist the consultant and keep them aware of work schedules and predicted changes, so that the consultant may schedule their personnel accordingly.

All removals, prepared ground to receive fill, key excavations, and subdrains should be observed and documented by the project engineering geologist and/or soil engineer prior to placing any fill. It is the contractor’s responsibility to notify the engineering geologist and soil engineer when such areas are ready for observation.

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation D-1557-78. Random field compaction tests should be performed in accordance with test method ASTM designations D-1556-82, D-2937 or D-2922 & D-3017, at intervals of approximately 2-feet of fill height per 10,000 sq. ft. or every 1,000 cubic yards of fill placed. These criteria would vary depending on the soil conditions and the size of the project. The location and frequency of testing would be at the discretion of the geotechnical consultant

Contractor's Responsibility

All clearing, site preparation, and earthwork performed on the project should be conducted by the contractor, with observation by geotechnical consultants and staged approval by the appropriate governing agencies. It is the contractor's responsibility to prepare the ground surface to receive the fill to the satisfaction of the soil engineer, and to place, spread, moisture condition, mix and compact the fill in accordance with the recommendations of the soil engineer. The contractor should also remove all major deleterious material considered unsatisfactory by the soil engineer.

It is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in accordance with applicable grading guidelines, codes or agency ordinances, and approved grading plans. Sufficient watering apparatus and compaction equipment should be provided by the contractor with due consideration for the fill material, rate of placement, and climatic conditions. If, in the opinion of the geotechnical consultant, unsatisfactory conditions such as questionable weather, excessive oversized rock, deleterious material or insufficient support equipment are resulting in a quality of work that is not acceptable, the consultant will inform the contractor, and the contractor is expected to rectify the conditions, and if necessary, stop work until conditions are satisfactory.

The contractor will properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor will take action to control surface water and to prevent erosion control measures that have been installed.

SITE PREPARATION

All vegetation including brush, trees, thick grasses, organic debris, and other deleterious material should be removed and disposed of offsite, and must be concluded prior to placing fill. Existing fill, soil, alluvium, colluvium, or rock materials determined by the soil engineer or engineering geologist as unsuitable for structural in-place support should be removed prior to fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the soil engineer.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading are to be removed or treated in a manner recommended by the soil engineer.

Soft, dry, spongy, highly fractured, or otherwise unsuitable ground extending to such a depth that surface processing cannot adequately improve the condition should be over excavated down to firm ground and approved by the soil engineer before compaction and filling operations continue.

Over excavated and processed soils which have been properly mixed and moisture-conditioned should be recompacted to the minimum relative compaction as specified in these guidelines.

Existing ground which is determined to be satisfactory for support of the fills should be scarified to a minimum depth of 6-inches, or as directed by the soil engineer. After the scarified ground is brought to optimum moisture (or greater) and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6-inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to 6-inches in compacted thickness.

Existing grind which is not satisfactory to support compacted fill should be over excavated as required in the geotechnical report or by the onsite soils engineer and/or engineering geologists. Scarification, discing, or other acceptable form of mixing should continue until the soils are broken down and free of large fragments or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, or other uneven features which would inhibit compaction as described herein.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical) gradient, the ground should be benched. The lowest bench, which will act as a key, should be a minimum of 12 feet wide and should be at least 2 feet deep into competent material, approved by the soil engineer and/or engineering geologist. In fill over cut slope conditions, the recommended minimum width of the lowest bench or key is at least 15 feet with the key excavated on competent material, as designated by the Geotechnical Consultant. As a general rule, unless superseded by the Soil Engineer, the minimum width of fill keys should be approximately equal to one-half (½) the height of the slope.

Standard benching is typically 4 feet (minimum) vertically, exposing competent material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed four feet. Pre stripping may be considered for removal of unsuitable materials in excess of 4 feet in thickness.

All areas to receive fill, including processed areas, removal areas, and toe of fill benches should be observed and approved by the soil engineer and/or engineering geologist prior to placement of fill. Fills may then be properly placed and compacted until design grades are attained.

COMPACTED FILLS

Earth materials imported or excavated on the property may be utilized as fill provided that each soil type has been accepted by the soil engineer. These materials should be free of roots, tree branches, other organic matter or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the soil engineer. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated unsuitable by the consultant and may require mixing with other earth materials to serve as a satisfactory fill material.

Fill materials generated from benching operations should be dispersed throughout the fill area. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials, defined as rock or other irreducible materials with a maximum size exceeding 12-inches in one dimension, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the soil engineer. Oversized material should be taken offsite or placed in accordance with recommendations of the soil engineer in areas designated as suitable for rock disposal. Oversized material should not be placed vertically within 10 feet of finish grade or horizontally within 20 feet of slope faces.

To facilitate trenching, rock should not be placed within the range of foundation excavations or future utilities unless specifically approved by the soil engineer and/or the representative developers.

If import fill material is required for grading, representative samples of the material should be analyzed in the laboratory by the soil engineer to determine its physical properties. If any material other than that previously analyzed is imported to the fill or encountered during grading, analysis of this material should be conducted by the soil engineer as soon as practical.

Fill material should be placed in areas prepared to receive fill in near-horizontal layers that should not exceed 6-inches compacted in thickness. The soil engineer may approve thicker lifts if testing indicates the grading procedures are such that adequate compaction is being achieved. Each layer should be spread evenly and mixed to attain uniformity of material and moisture suitable for compaction.

Fill materials at moisture content less than optimum should be watered and mixed, and "wet" fill materials should be aerated by scarification, or should be mixed with drier material. Moisture conditioning and mixing of fill materials should continue until the fill materials have uniform moisture content at or above optimum moisture.

After each layer has been evenly spread, moisture-conditioned and mixed, it should be uniformly compacted to a minimum of 90 percent of maximum density as determined by ASTM test designation, D 1557-78, or as otherwise recommended by the soil engineer. Compaction equipment should be adequately sized and should be reliable to efficiently achieve the required degree of compaction.

Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction or improper moisture content, the particular layer or portion will be reworked until the required density and/or moisture content has been attained. No additional fill will be placed in an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements, and is approved by the soil engineer.

Compaction of slopes should be accomplished by over-building the outside edge a minimum of 3 feet horizontally, and subsequently trimming back to the finish design slope configuration. Testing will be performed as the fill is horizontally placed to evaluate compaction as the fill core is being developed. Special efforts may be necessary to attain the specified compaction in the fill slope zone.
Final slope shaping should be performed by trimming and removing loose materials with appropriate equipment. A final determination of fill slope compaction should be based on observation and/or testing of the finished slope face.

If an alternative to over-building and cutting back the compacted fill slope is selected, then additional efforts should be made to achieve the required compaction in the outer 10 feet of each lift of fill by undertaking the following:

- Equipment consisting of a heavy short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the slopes, and extend out over the slope to provide adequate compaction to the face slope.

- Loose fill should not be spilled out over the face of the slope as each lift is compacted. Any loose fill spilled over a previously completed slope face should be trimmed off or be subject to re-rolling.

- Field compaction tests will be made in the outer 2 to 5 feet of the slope at 2- to three 3-foot vertical intervals, subsequent to compaction operations.

- After completion of the slope, the slope face should be shaped with a small dozer and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to verify compaction, the slopes should be grid-rolled to achieve adequate compaction to the slope face. Final testing should be used to confirm compaction after grid rolling.

- Where testing indicates less than adequate compaction, the contractor will be responsible to process, moisture condition, mix and recompact the slope materials as necessary to achieve compaction. Additional testing should be performed to verify compaction.

- Erosion control and drainage devices should be designed by the project civil engineer in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the soil engineer or engineering geologist.

EXCAVATIONS

Excavations and cut slopes should be observed and mapped during grading by the engineering geologist. If directed by the engineering geologist, further excavations or over-excavation and refilling of cut areas should be performed. When fills over cut slopes are to be graded, the cut portion of the slope should be observed by the engineering geologist prior to placement of the overlying fill portion of the slope. The engineering geologist should observe all cut slopes and should be notified by the contractor when cut slopes are started.

If, during the course of grading, unanticipated adverse or potentially adverse geologic conditions are encountered, the engineering geologist and soil engineer should investigate, evaluate and make recommendations to mitigate (or limit) these conditions.
The need for cut slope buttressing or stabilizing should be based on as-grading evaluations by the engineering geologist, whether anticipated previously or not.

Unless otherwise specified in soil and geological reports, no cut slopes should be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies. Additionally, short-term stability of temporary cut slopes is the contractor’s responsibility.

Erosion control and drainage devices should be designed by the project civil engineer and should be constructed in compliance with the ordinances of the controlling governmental agencies, and/or in accordance with the recommendations of the soil engineer or engineering geologist.

SUBDRAIN INSTALLATION

Subdrains should be installed in accordance with the approved embedment material, alignment and details indicated by the geotechnical consultant. Subdrain locations or construction materials should not be changed or modified without approval of the geotechnical consultant. The soil engineer and/or engineering geologist may recommend and direct changes in subdrain line, grade and drain material in the field, pending exposed conditions. The location of constructed subdrains should be recorded by the project civil engineer.

COMPLETION

Consultation, observation and testing by the geotechnical consultant should be completed during grading operations in order to state an opinion that all cut and filled areas are graded in accordance with the approved project specifications.

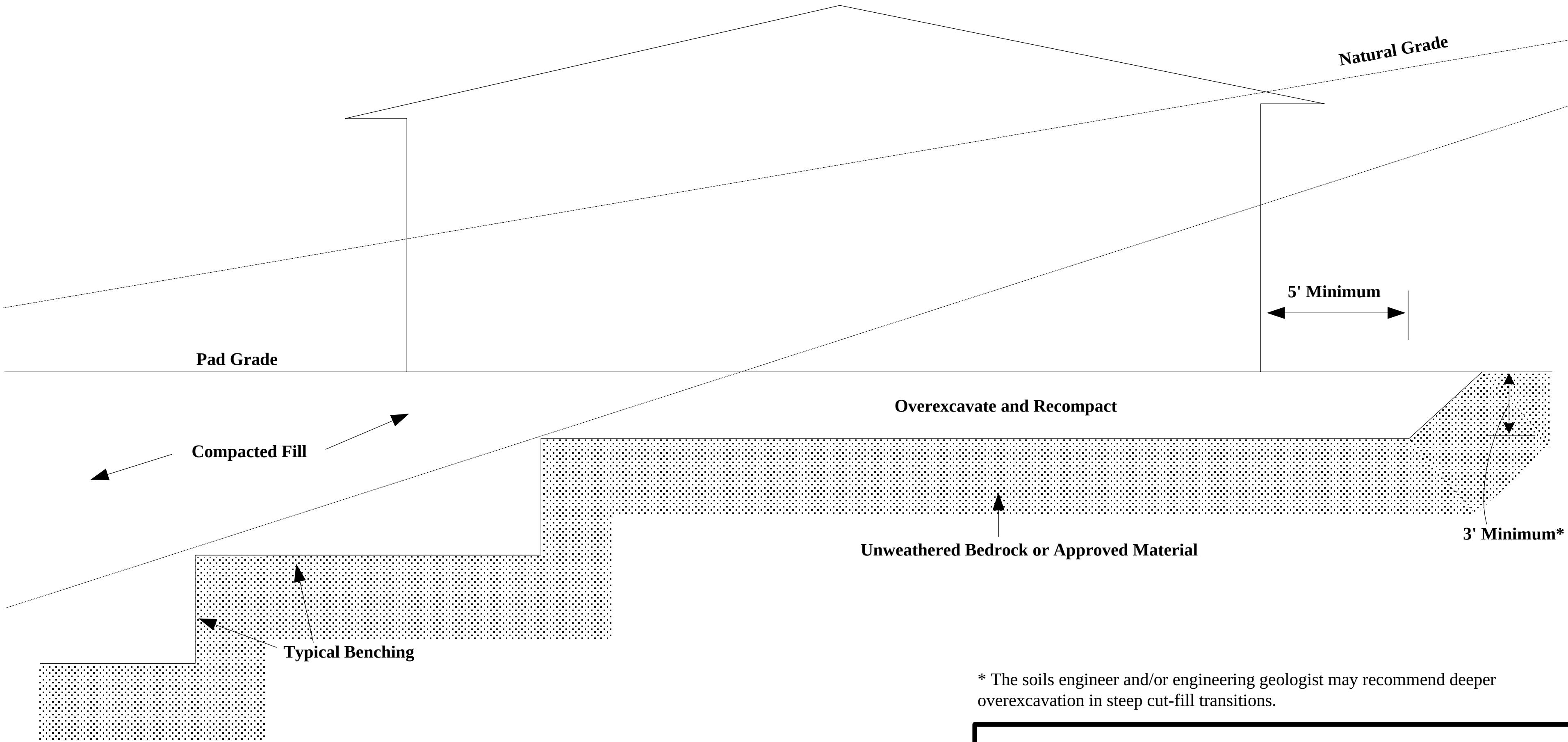
After completion of grading and after the soil engineer and engineering geologist have finished their observations, final reports should be submitted subject to review by the controlling governmental agencies. No additional grading should be undertaken without prior notification of the soil engineer and/or engineering geologist.

All finished cut and fill slopes should be protected from erosion, including but not limited to planting in accordance with the plan design specifications and/or as recommended by a landscape architect. Such protection and/or planning should be undertaken as soon as possible after completion of grading.

ATTACHMENTS

Figure A – Transition Lot Detail Cut Lot
Figure B – Transition Lot Detail Cut - Fill
Figure C – Rock Disposal Pits
Figure D – Detail for Fill Slope Toeing out on a Flat Alluviated Canyon
Figure E – Removal Adjacent to Existing Fill
Figure F – Daylight Cut Lot Detail
Figure G – Skin Fill of Natural Ground
Figure H – Typical Stabilization Buttress Fill Design
Figure I – Stabilization Fill for Unstable Material Exposed in Portion of Cut Slope
Figure J – Fill Over Cut Detail
Figure K – Fill Over Natural Detail
Figure L – Oversize Rock Disposal
Figure M – Canyon Subdrain Detail
Figure N – Canyon Subdrain Alternate Details
Figure O – Typical Stabilization Buttress Subdrain Detail
Figure P – Retaining Wall Backfill

TRANSITION LOT DETAIL
CUT LOT – MATERIAL TYPE
TRANSITION



* The soils engineer and/or engineering geologist may recommend deeper overexcavation in steep cut-fill transitions.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
TRANSITION LOT DETAIL
CUT LOT – MATERIAL TYPE TRANSITION

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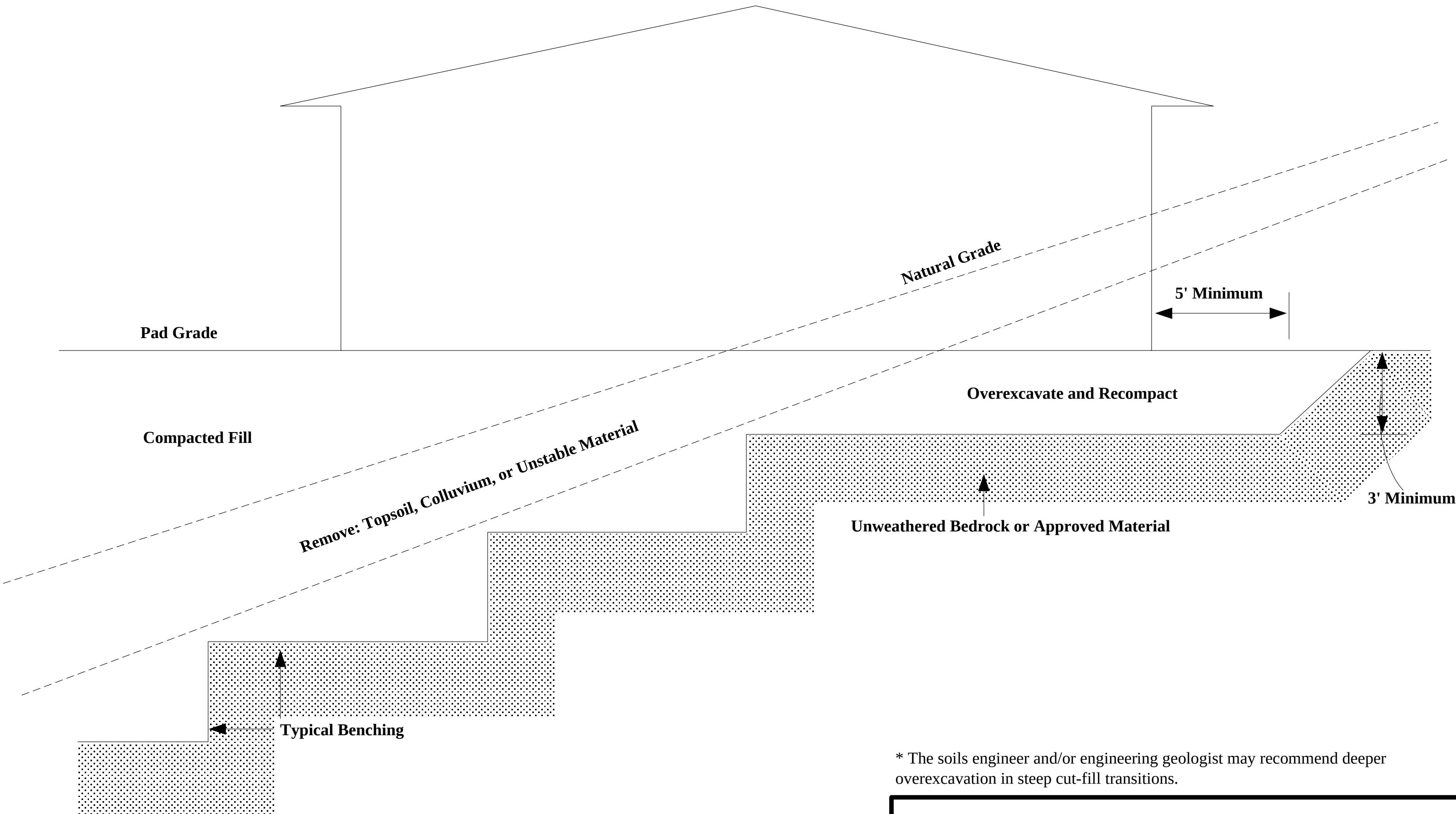
FIGURE A

TRANSITION LOT DETAIL
CUT – FILL – DAYLIGHT TRANSITION



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EARTHWORK AND GRADING GUIDELINES
TRANSITION LOT DETAIL
CUT – FILL – DAYLIGHT TRANSITION

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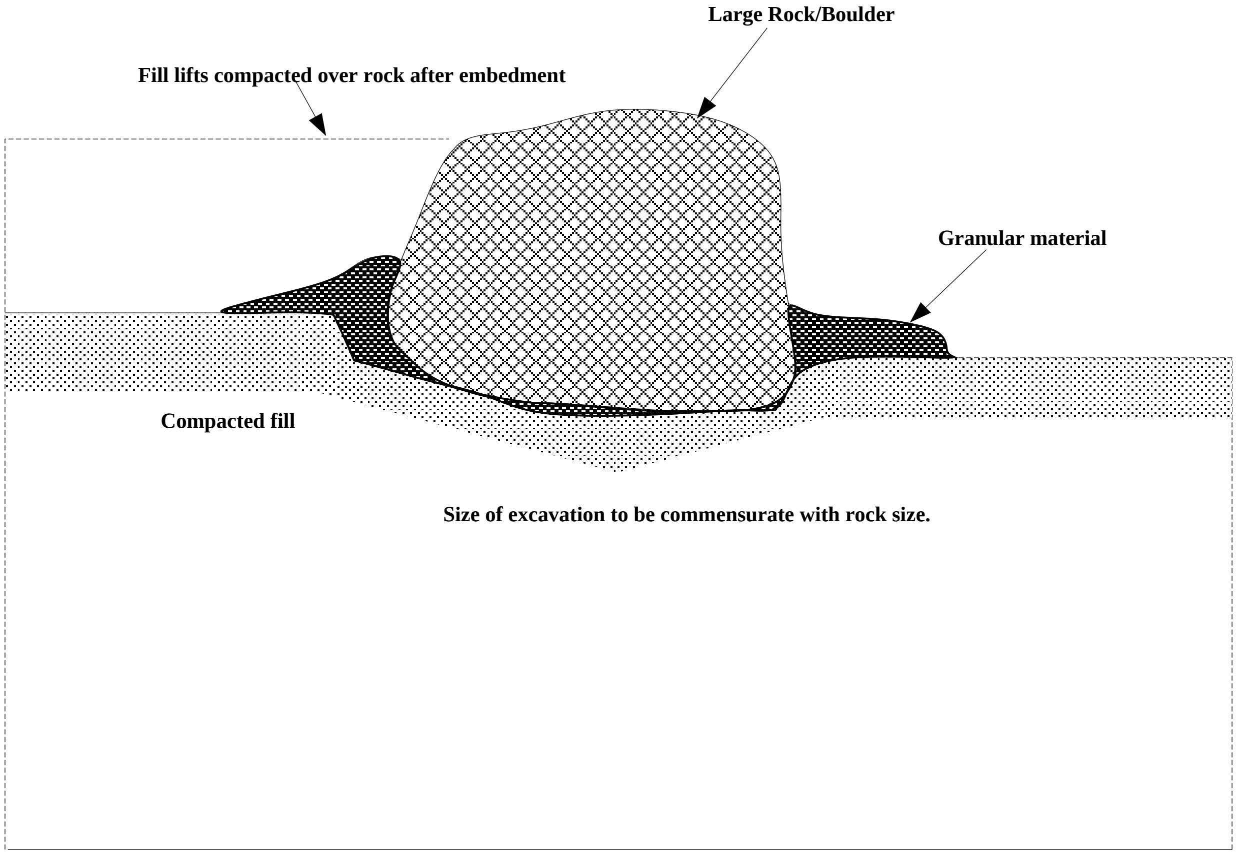
FIGURE B

ROCK DISPOSAL PITS



Note:
(1) Large rock is defined as having a diameter larger than 3 feet in maximum size.
(2) Pit shall be excavated into compacted fill to a depth equal to half of the rock size.
(3) Granular soil shall be pushed into the pit and then flooded around the rock using a sheepsfoot to help with compaction.
(4) A minimum of 3 feet of compacted fill should be laid over each pit.
(5) Pits shall have at least 15 feet of separation between one another, horizontally.
(6) Pits shall be placed at least 20 feet from any fill slope.
(7) Pits shall be used only in deep fill areas.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
ROCK DISPOSAL PITS

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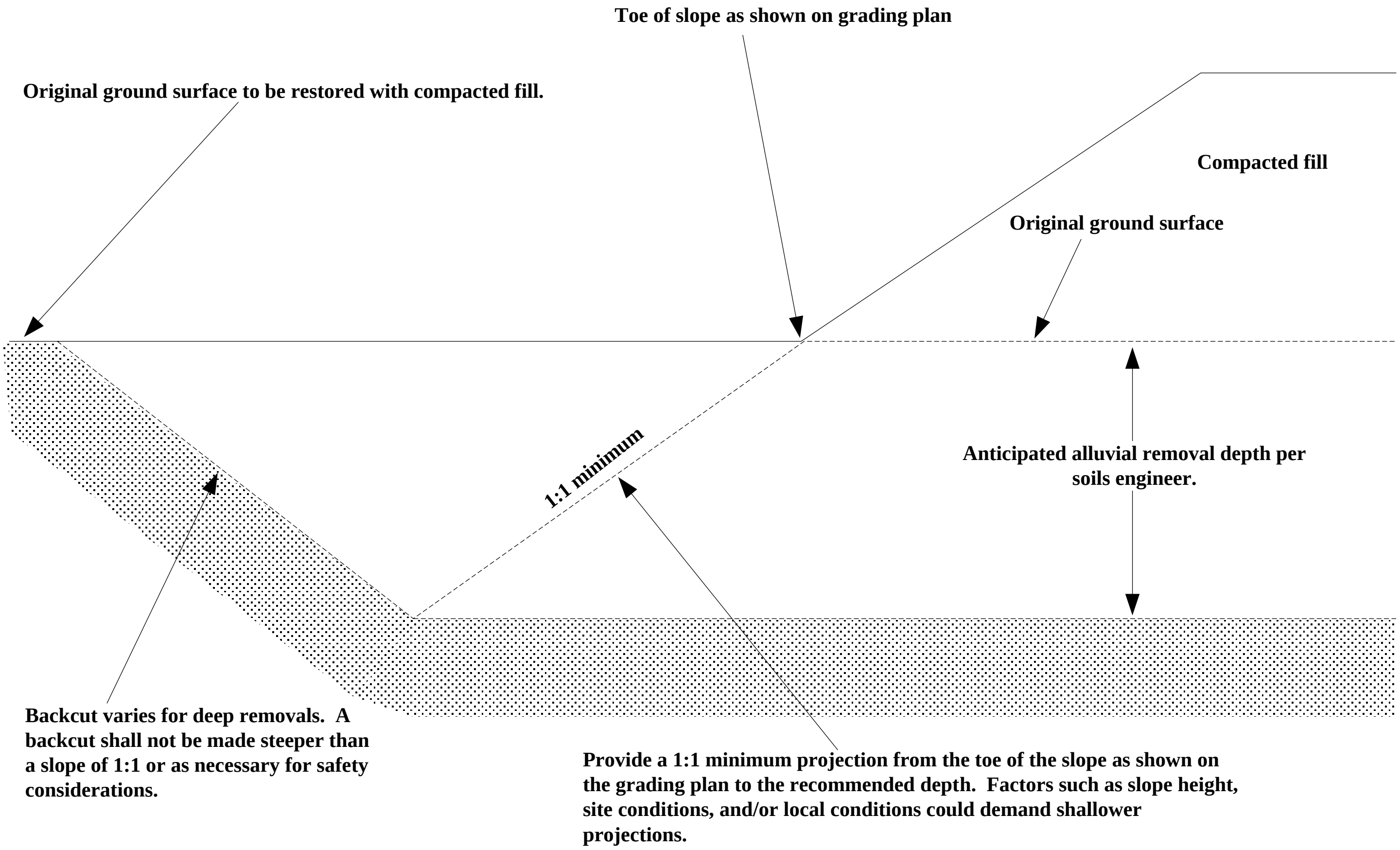
FIGURE C

DETAIL FOR FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON



Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
DETAIL FOR FILL SLOPE TOEING OUT ON A FLAT ALLUVIATED CANYON

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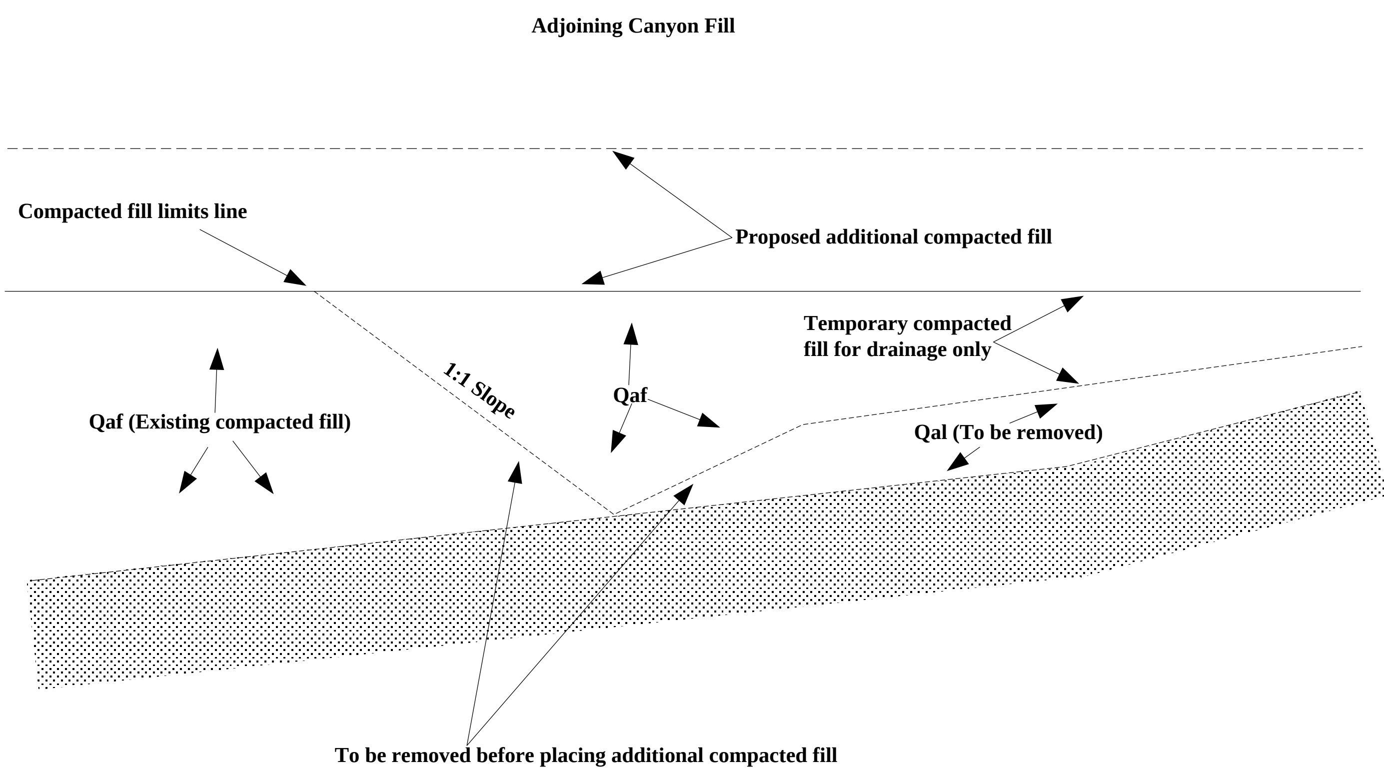
FIGURE D

REMOVAL ADJACENT TO EXISTING FILL



Legend

Qaf - **Artificial Fill**

Qal - **Alluvium**

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
REMOVAL ADJACENT TO EXISTING FILL

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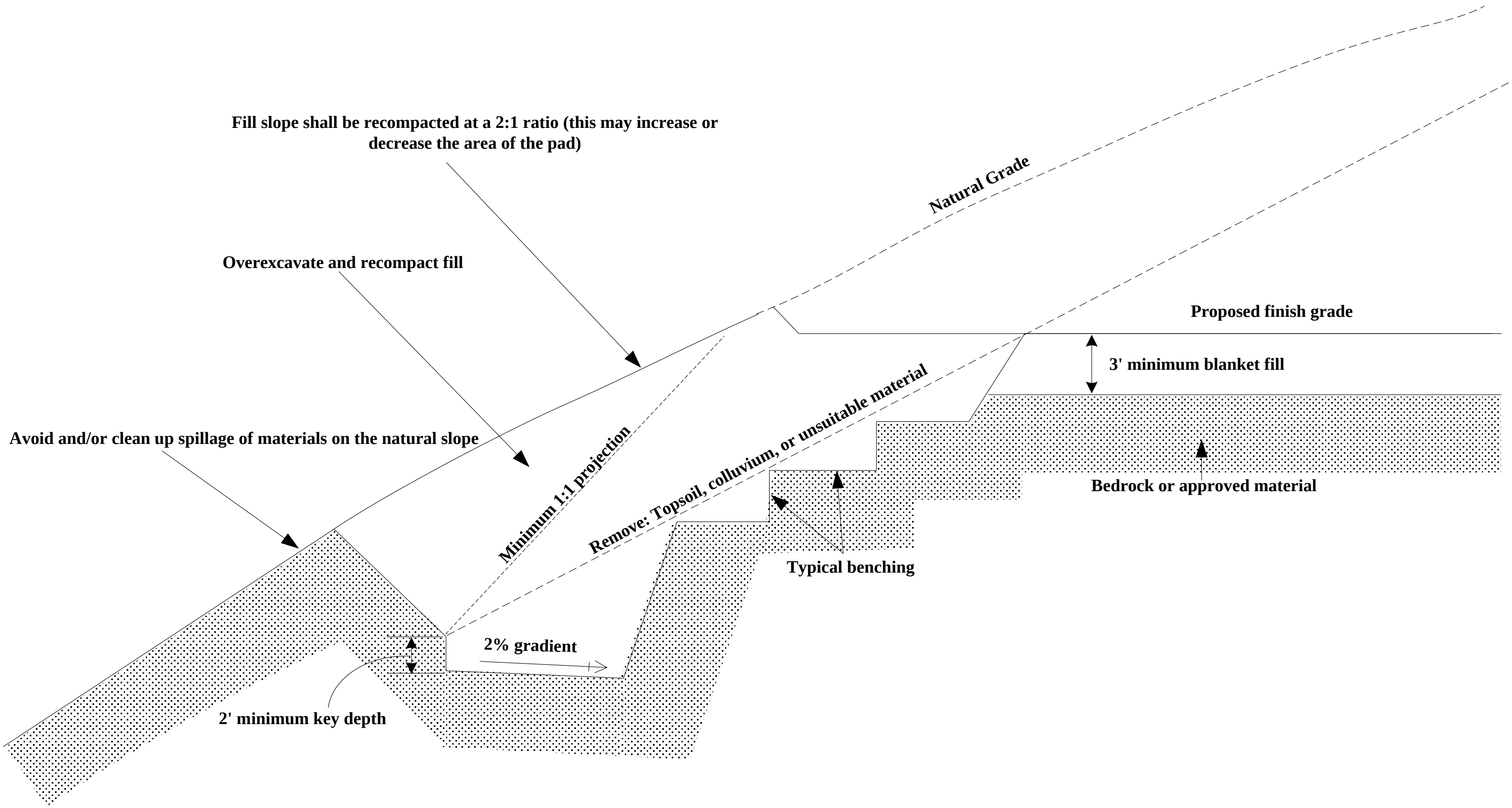
FIGURE E

DAYLIGHT CUT LOT DETAIL



Note: (1) Subdrain and key width requirements shall be determined based on exposed subsurface conditions and the thickness of overburden.

(2) Pad overexcavation and recompaction shall be completed if determined as necessary by the soils engineer and/or engineering geologist.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
DAYLIGHT CUT LOT DETAIL

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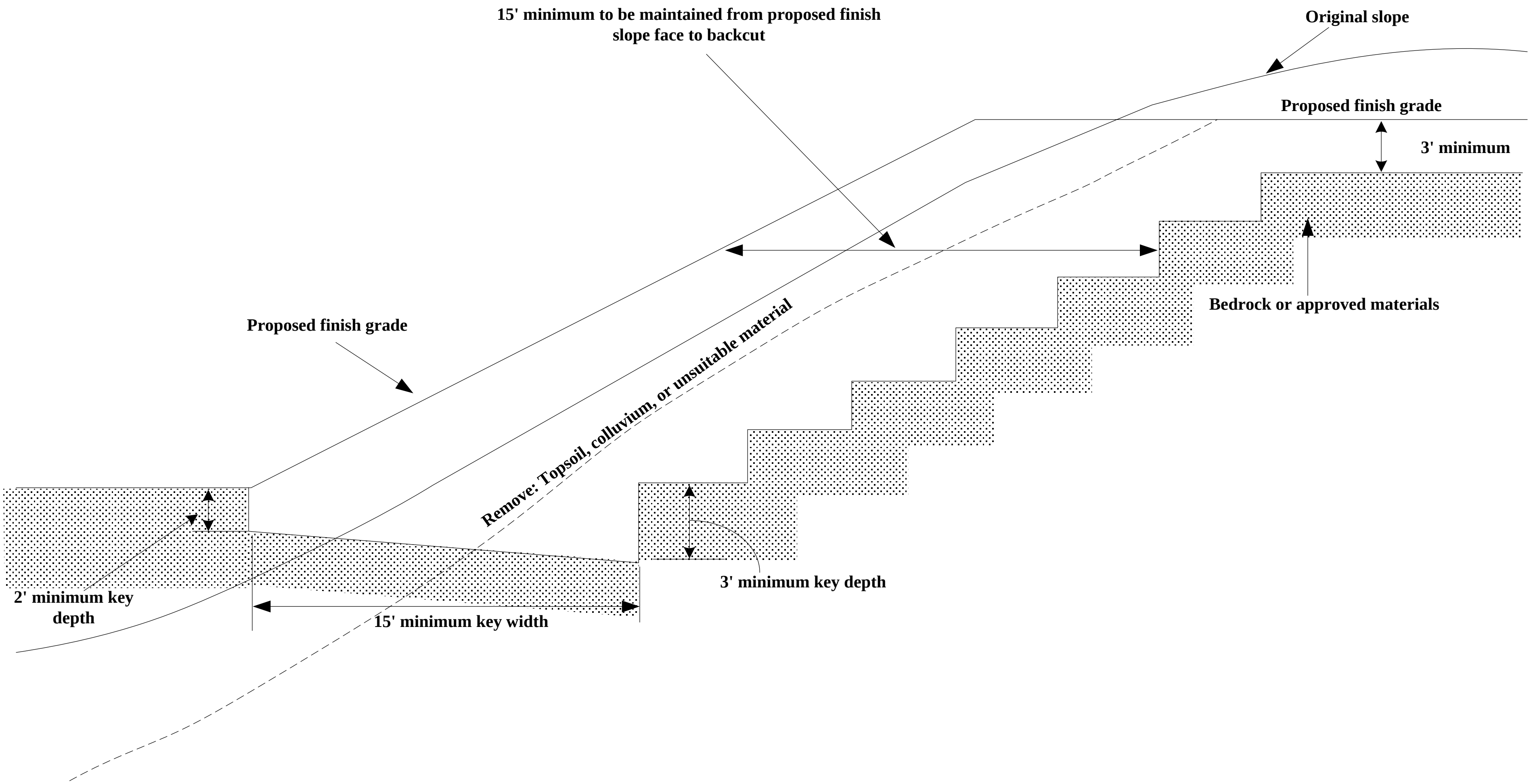
FIGURE F

SKIN FILL OF NATURAL GROUND



Note: (1) The need and disposition of drains will be determined by the soils engineer and/or engineering geologist based on site conditions.

(2) Pad overexcavation and recompaction shall be completed if determined as necessary by the soils engineer and/or engineering geologist.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
SKIN FILL OF NATURAL GROUND

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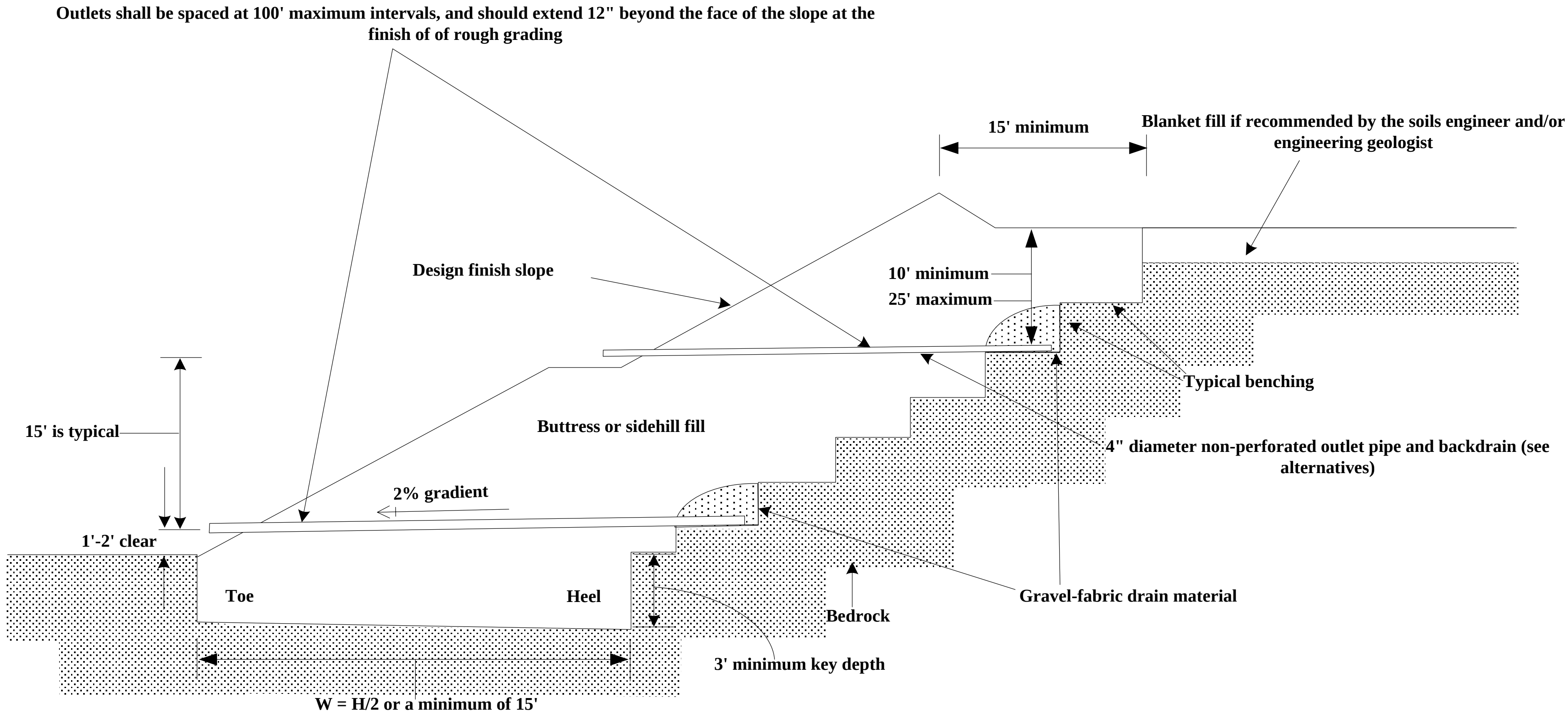
FIGURE G

TYPICAL STABILIZATION BUTTRESS FILL DESIGN



Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
TYPICAL STABILIZATION BUTTRESS FILL DESIGN

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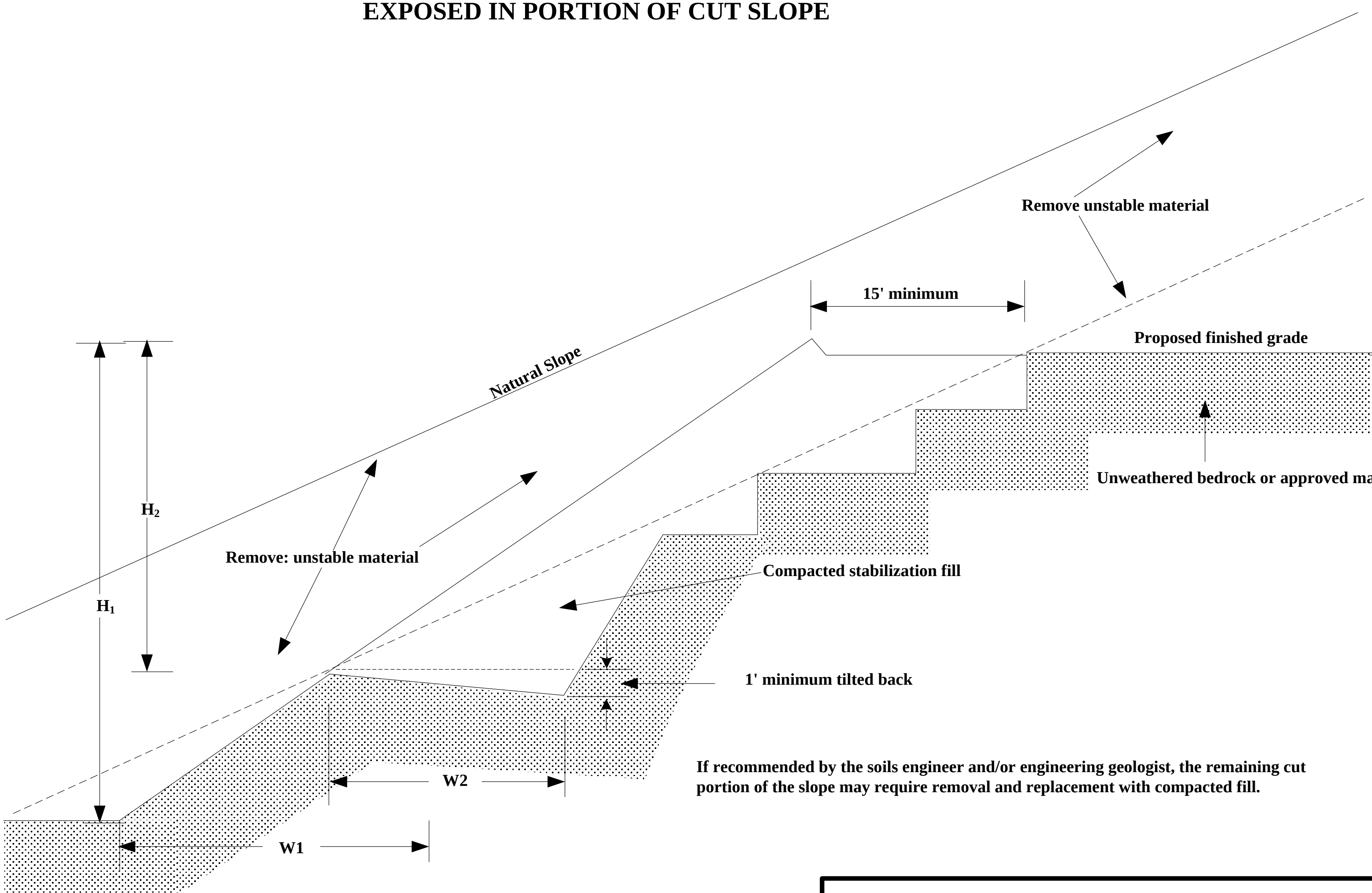
FIGURE H

STABILIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN PORTION OF CUT SLOPE



If recommended by the soils engineer and/or engineering geologist, the remaining cut portion of the slope may require removal and replacement with compacted fill.

Note: (1) Subdrains are required only if specified by the soils engineer and/or engineering geologist.

(2) "W" shall be the equipment width (15') for slope heights less than 25 feet. For slopes greater than 25 feet "W" shall be determined by the project soils engineer and/or the engineering geologist. "W" shall never be less than H/2.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES

STABILIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN PORTION OF CUT SLOPE

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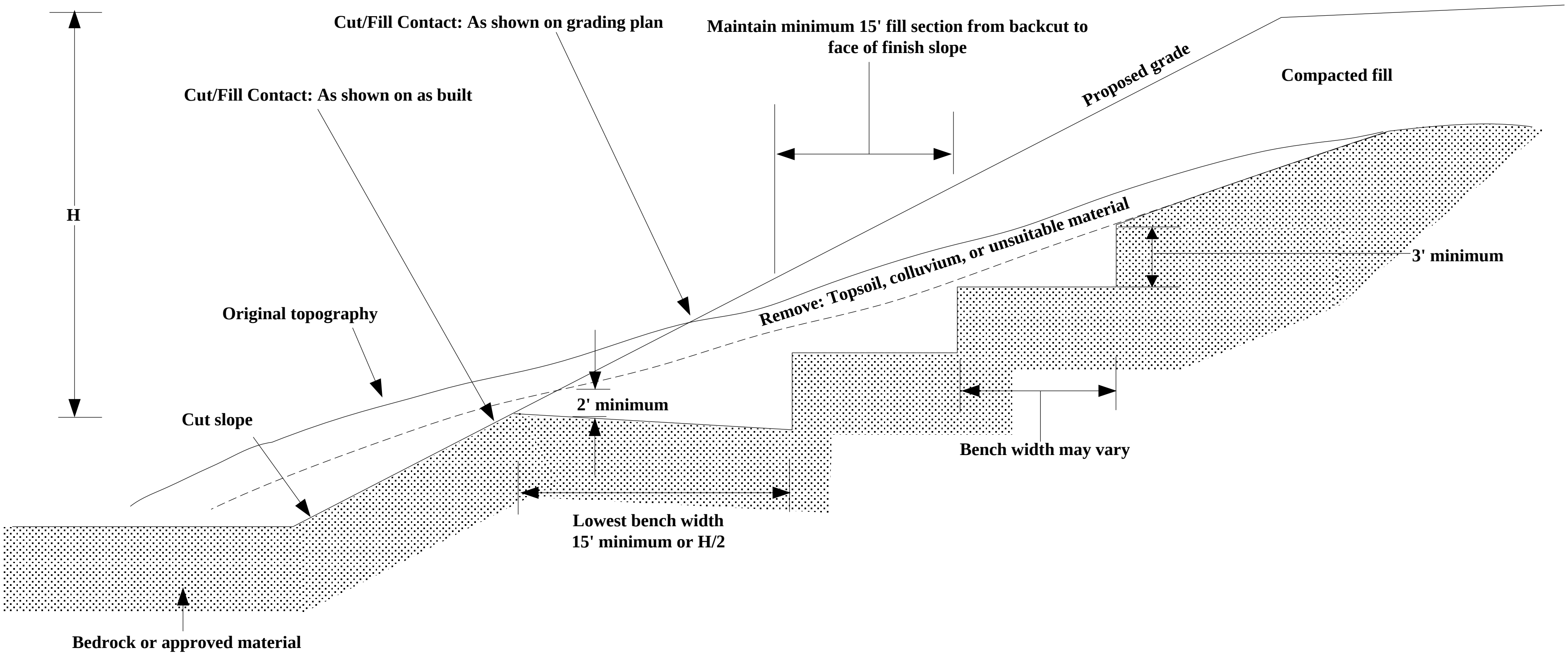
FIGURE I

FILL OVER CUT DETAIL



Note: The cut sectioin shall be excavated and evaluated by the soils engineer/engineering geologist prior to constructing the fill portion.

Note: Figure not to scale

EARTHWORK AND GRADING GUIDELINES
FILL OVER CUT DETAIL

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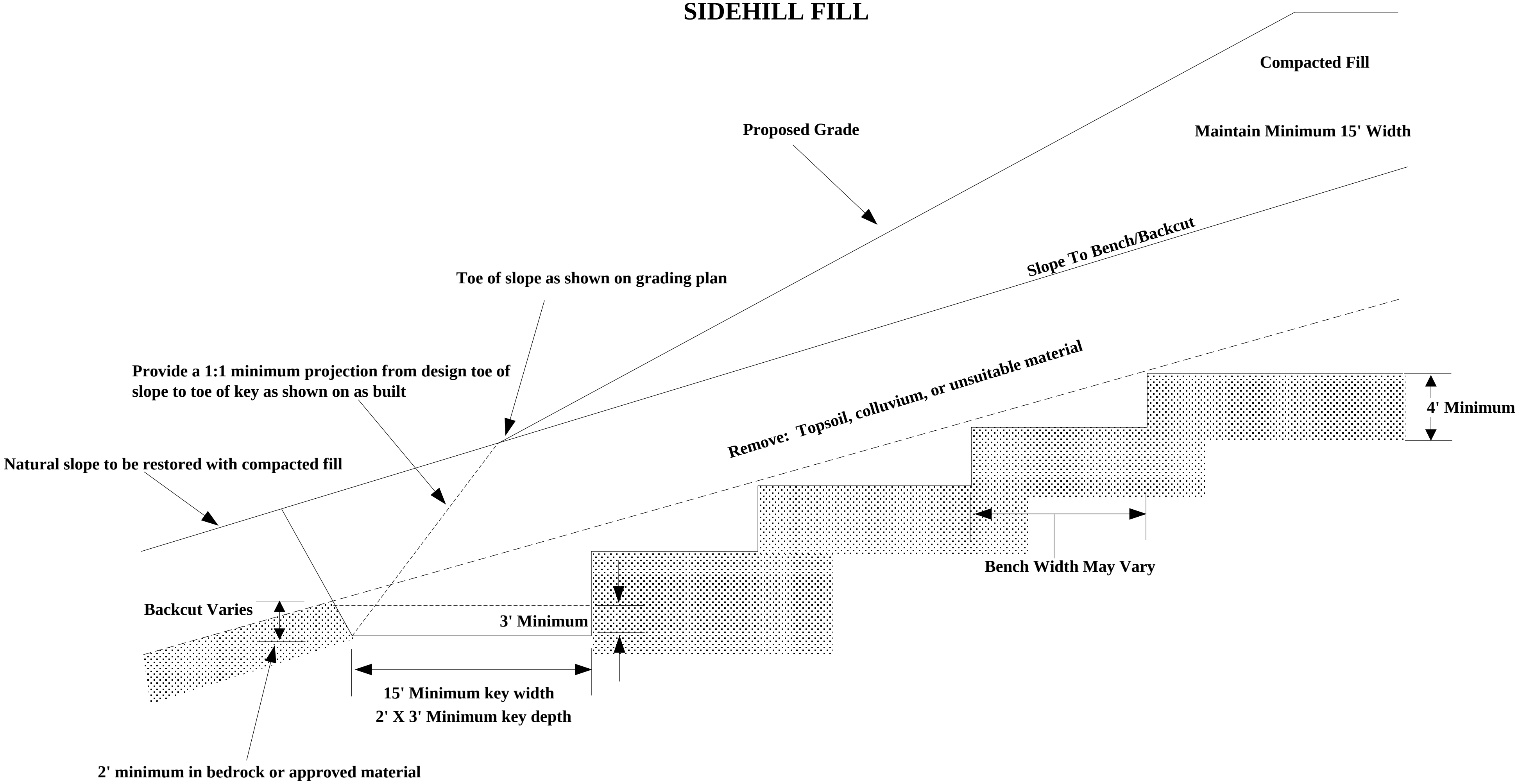
FIGURE J

FILL OVER NATURAL DETAIL
SIDEHILL FILL



Note: (1) Special recommendations shall be provided by the soils engineer/engineering geologist where the natural slope approaches or exceeds the design slope ratio.
(2) The need for and disposition of drains would be determined by the soils engineer/engineering geologist based upon exposed conditions.

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
FILL OVER NATURAL DETAIL
SIDEHILL FILL

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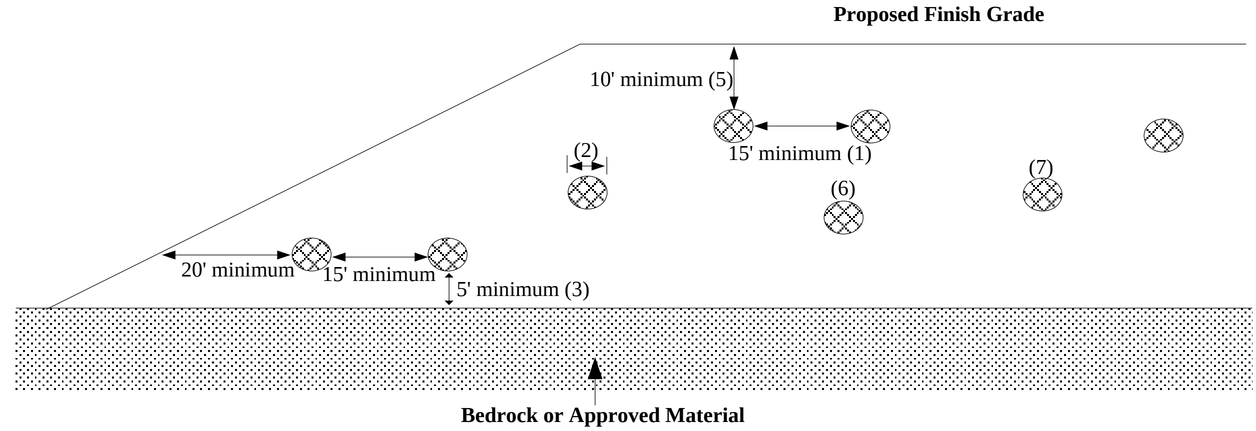
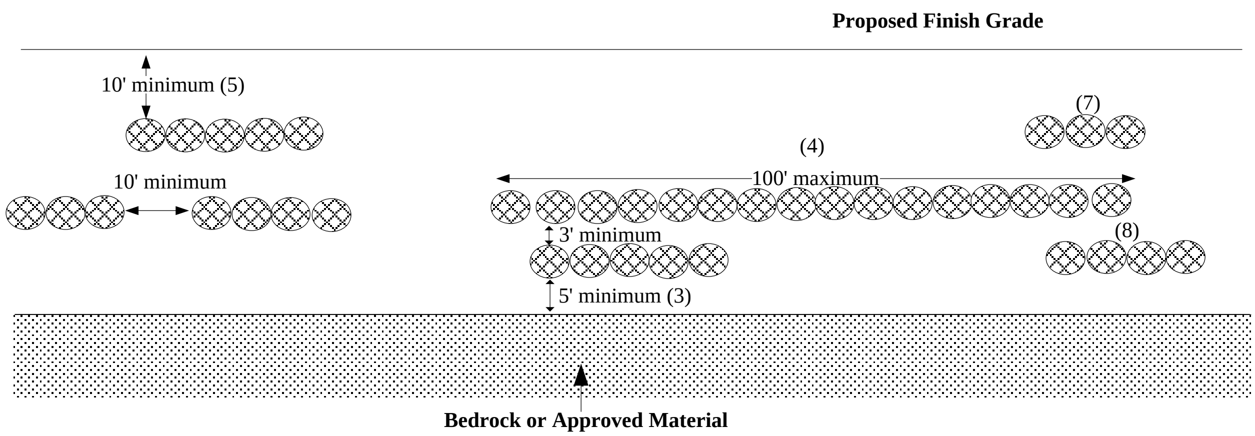
FIGURE K

OVERSIZE ROCK DISPOSAL

View Normal to Slope Face

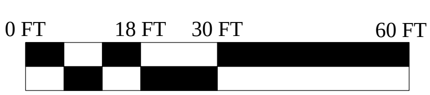


View Parallel to Slope Face



Note:
(1) One Equipment width or a minimum of 15 feet.
(2) Height and width may vary depending on rock size and type of equipment used. Length of windrow shall be no greater than 100 feet maximum.
(3) If approved by the soils engineer and/or engineering geologist.
(4) Orientation of windrows may vary but shall be as recommended by the soils engineer and/or engineering geologist. Unless recommended staggering of windrows is not necessary.
(5) Areas shall be cleared for utility trenches, foundations, and swimming pools.
(6) Voids in windrows shall be filled by flooding granular soil into place. Granular soil shall be any soil which has a unified soil classification system (Universal Building Code (UBC) 29-1). Designation of SM, SP, SW, GP, or GW.
(7) After fill between windrows is placed and compacted with the lift of fill covering windrow, windrow shall be proof rolled with a D-9 dozer or equivalent.
(8) Oversized rock is defined as larger than 12", and less than 4 feet in size.

Approximate Scale: 1" = 30'



Note: All distances are approximate

EARTHWORK AND GRADING GUIDELINES
OVERSIZE ROCK DISPOSAL

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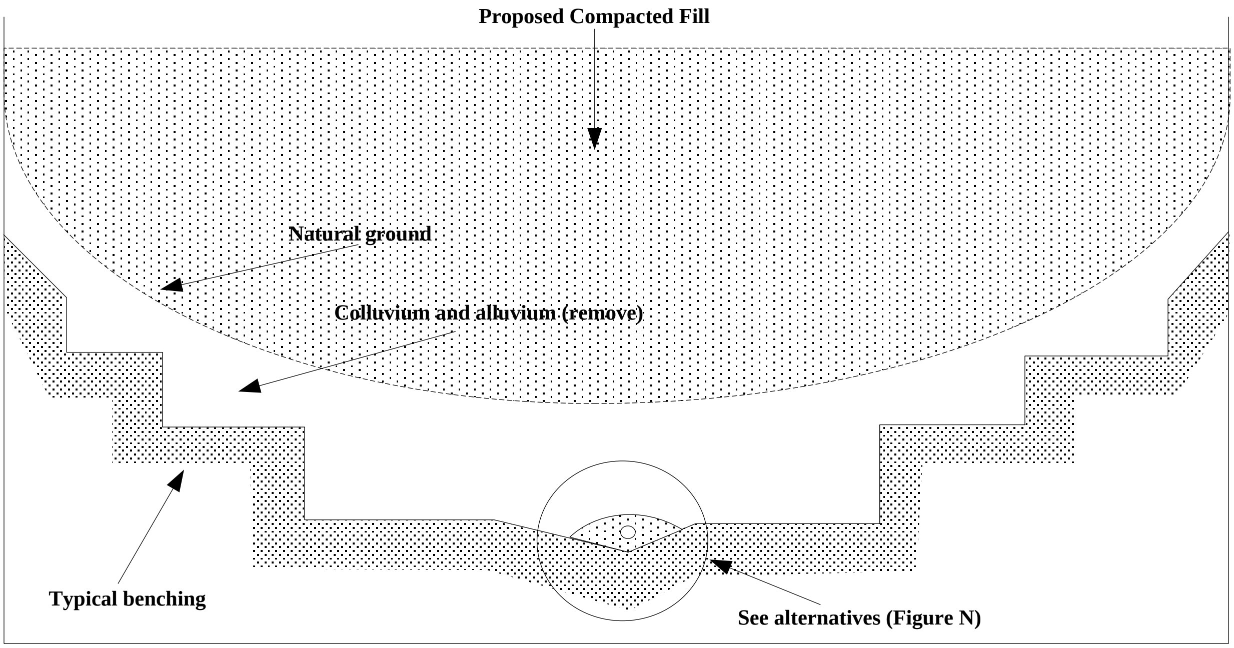
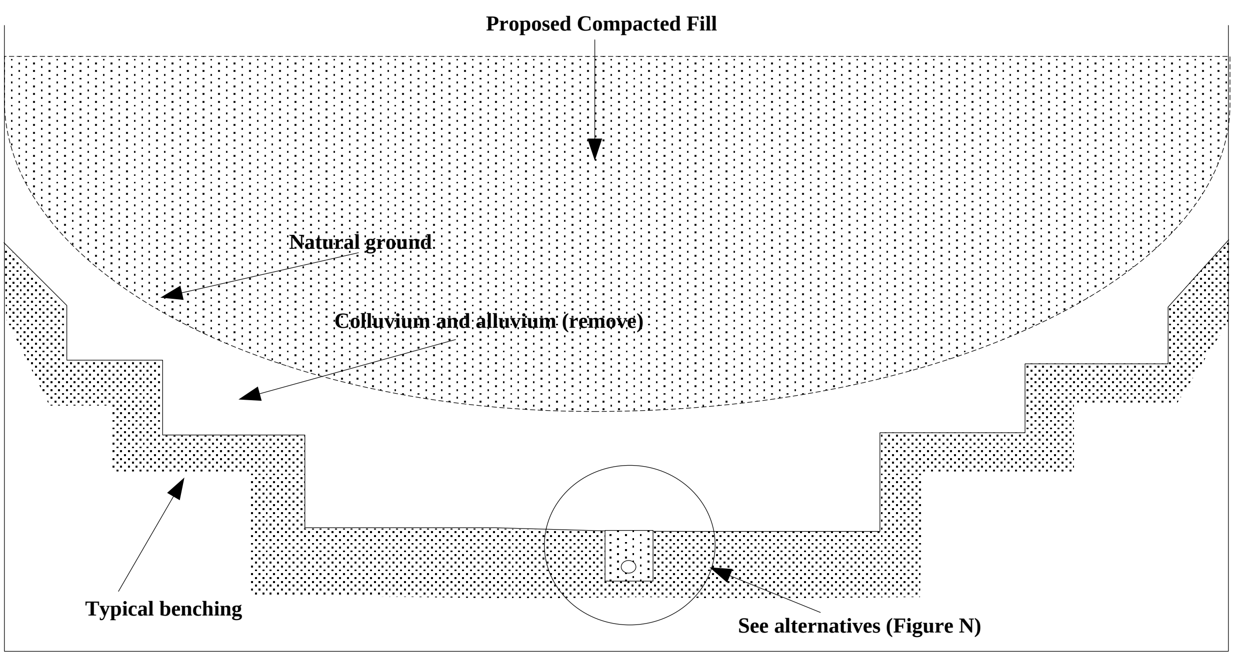
FIGURE L

CANYON SUBDRAIN DETAIL

Type A



Type B



Note: Alternatives, locations, and extent of subdrains should be determined by the soils engineer and/or engineering geologist during actual grading.

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
CANYON SUBDRAIN DETAIL

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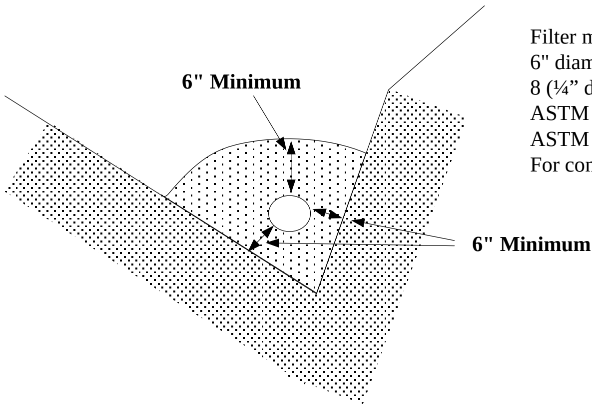
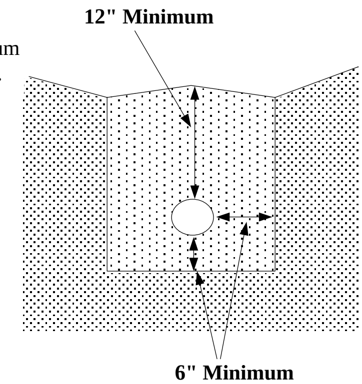
FIGURE M

CANYON SUBDRAIN ALTERNATE DETAILS

Alternate 1: Perforated Pipe and Filter Material



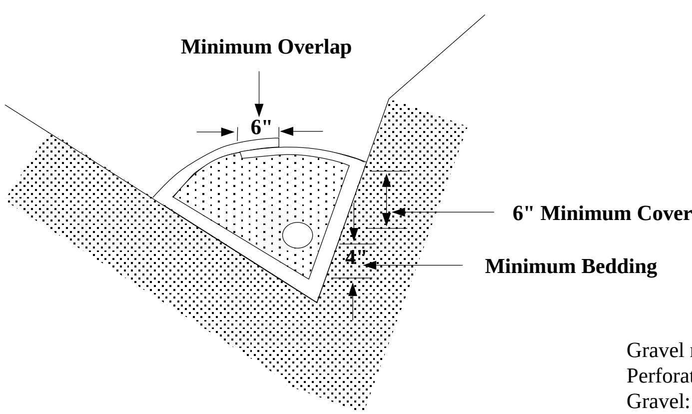
Filter material: Minimum volume of 9 feet^3/linear foot.
6" diameter ABS or PVC pipe or approved substitute with minimum 8 (¼" diameter) perforations per linear foot in bottom half of pipe.
ASTM D 2751, SDR 35 or ASTM D 1527, Schedule 40.
ASTM D 3034, SDR 35 or ASTM D 1785, Schedule 40.
For continuous run in excess of 500 feet use 8" diameter pipe.

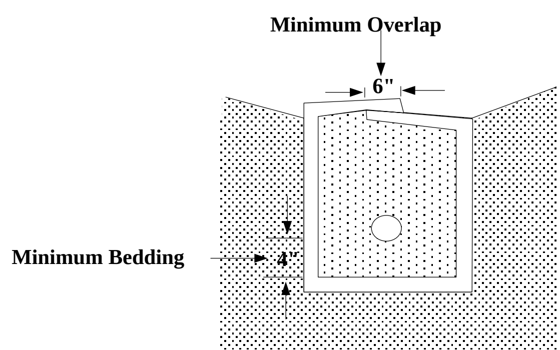


Filter Material

Sieve Size	Percent Passing
1"	100
¾"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Alternate 2: Perforated Pipe, Gravel and Filter Fabric





Gravel material 9 feet^3/linear foot.
Perforated pipe: see alternate 1.
Gravel: Clean ¾" rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
CANYON SUBDRAIN ALTERNATE DETAILS

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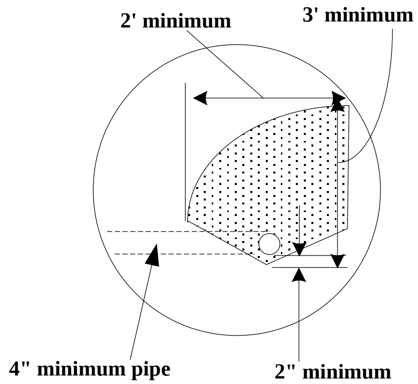
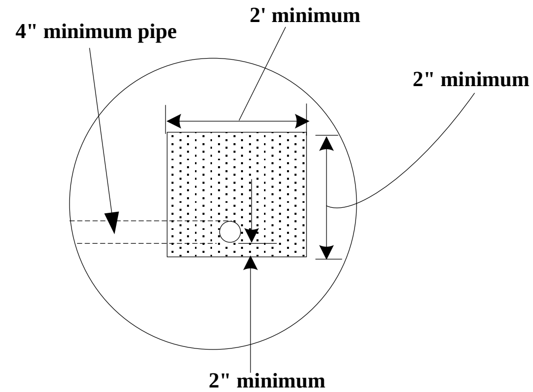
FIGURE N

TYPICAL STABILIZATION BUTTRESS SUBDRAIN DETAIL





Filter Material: Minimum of 5 ft^3/linear foot of pipe or 4 ft^3/linear foot of pipe when placed in square cut trench.

Alternative In Lieu Of Filter Material: Gravel may be encased in approved filter fabric. Filter fabric shall be mirafi 140 or equivalent. Filter fabric shall be lapped a minimum of 12" on all joints.

Minimum 4" Diameter Pipe: ABS-ASTM D-2751, SDR 35 or ASTM D-1527 schedule 40 PVC-ASTM D-3034, SDR 35 or ASTM D-1785 schedule 40 with a crushing strength of 1,000 pounds minimum, and a minimum of 8 uniformly spaced perforations per foot of pipe installed with perforations at bottom of pipe. Provide cap at upstream end of pipe. Slope at 2% to outlet pipe. Outlet pipe shall be connected to the subdrain pipe with tee or elbow.

Note: (1) Trench for outlet pipes shall be backfilled with onsite soil.
(2) Backdrains and lateral drains shall be located at the elevation of every bench drain. First drain shall be located at the elevation just above the lower lot grade. Additional drains may be required at the discretion of the soils engineer and/or engineering geologist.

Filter Material – Shall be of the following specification or an approved equivalent:

Filter Material

Sieve Size	Percent Passing
1"	100
¾”	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Gravel - Shall be of the following specification or an approved equivalent:

Filter Material

Sieve Size	Percent Passing
1½"	100
No. 4	50
No. 200	8

Sand equivalent: Minimum of 50

Note: Figures not to scale

EARTHWORK AND GRADING GUIDELINES
TYPICAL STABILIZATION BUTTRESS SUBDRAIN DETAIL

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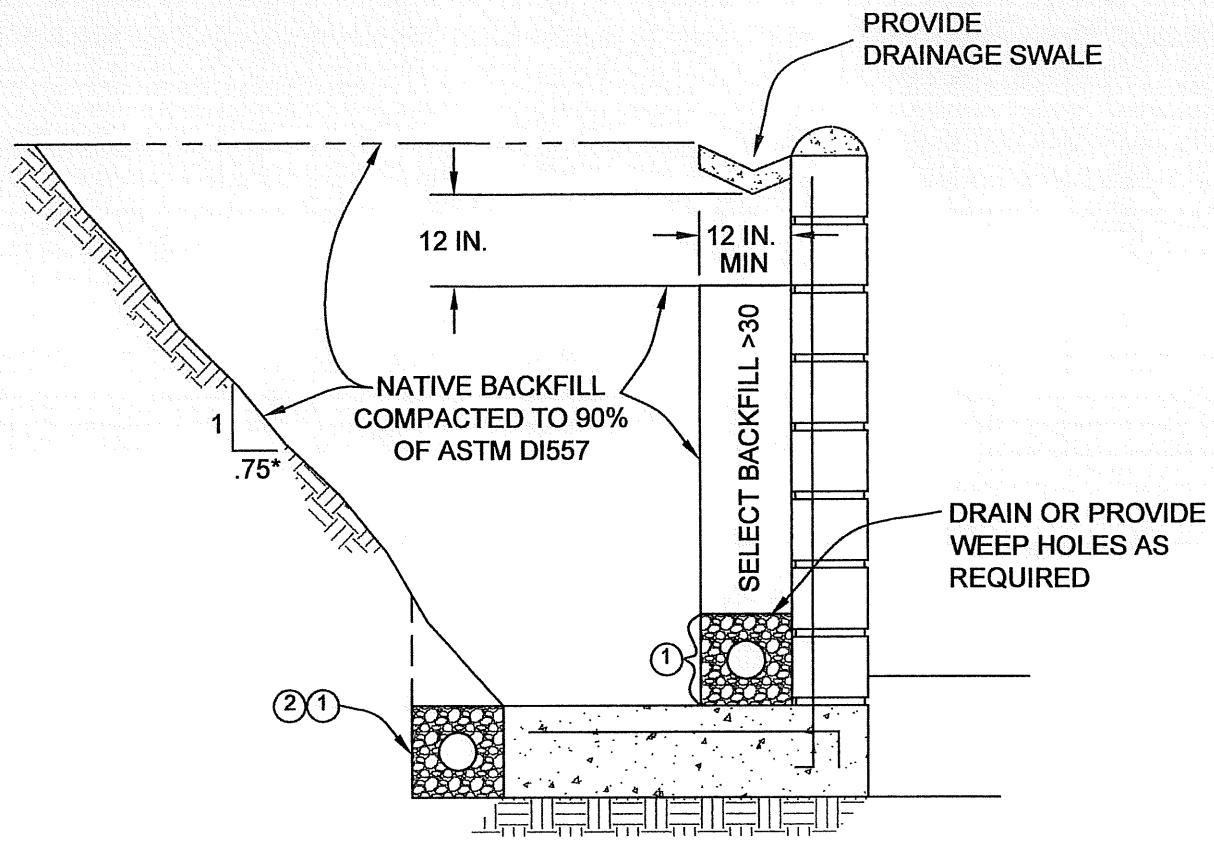
FIGURE O



* OR AS REQUIRED FOR SAFETY

NOTES

(1) 4-INCH PERFORATED PVC SCHEDULE 40 OR APPROVED ALTERNATE. PLACE PERFORATION DOWN AND SURROUND WITH A MINIMUM OF 1 CUBIC FOOT PER LINEAL FOOT (1 FT. /FT.) OF 3/4 INCH ROCK OR APPROVED ALTERNATE AND WRAPPED IN FILTER FABRIC.

(2) PLACE DRAIN AS SHOWN WHERE MOISTURE MIGRATION THROUGH THE WALL IS UNDESIRABLE.

NOTE: FIGURE NOT TO SCALE

EARTHWORK & GRADING GUIDELINES

TYPICAL RETAINING WALL BACKFILL

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FIGURE P